DREDGED MATERIAL RESEARCH PROGRAM
Technical Report D-78-59

Dredged Material Dewatering Field Demonstrations at Upper Polecat Bay Disposal Area, Mobile, Alabama

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Contents

List of Figures

List of Tables

Preface

This report presents the results of Dredged Material Research Program (DMRP) Disposal Operations Project (DOP), Dredged Material Densification, field demonstrations of potential dredged material dewatering methods conducted at the Upper Polecat Bay disposal area of the U.S. Army Engineer District, Mobile (MDO). The DMRP was sponsored by the Office, Chief of Engineers (DAEN-CWO-M) and was assigned to the U.S. Army Engineer Waterways Experiment Station (WES), Vicksburg, MS, under the former Environmental Effects Laboratory (EEL).

Various phases of the work were conducted, and this report was written, by Dr. T. Allan Haliburton, DMRP Geotechnical Engineering Consultant; Dr. Michael R. Palermo, Research Civil Engineer, EL, WES; Mr. Robert W. Chamlee, Civil Engineer, Foundations and Materials Branch (FMB), MDO; Mr. Alfred W. Ford, Research Electrical Engineer, EL, WES; Dr. James W. Spotts, Research Civil Engineer, Soils and Pavements Laboratory (SPL), WES; Dr. Robert L. Lytton, Professor of Civil Engineering, Texas A&M University; Mr. Joseph L. Gatz, Chief, Exploration Branch, SPL, WES; Dr. William E. Willoughby, Research Civil Engineer, Mobility and Environmental Systems Laboratory, WES; Mr. David P. Hammer, Research Civil Engineer, SPL, WES; Mr. Patrick A. Douglas, Civil Engineer, FMB, MDO; Dr. Charles E. O'Bannon, Professor of Civil Engineering, Arizona State University; and Ms. Judy P. Stout, Research Biologist, Dauphin Island Sea Lab. The report was prepared under the general supervision of Mr. Charles C. Calhoun, Jr., DOP Manager; Dr. R. T. Saucier, Special Assistant for Dredged Material Research; and Dr. John Harrison, Chief, EL.

Directors of the WES during this period were COL G. H. Hilt, CE, and COL John E. Cannon, CE. The Technical Director was Mr. F. R. Brown. Director of WES during publication of this report was Dr. Robert W. Whalin. Commander and Deputy Director was COL Leonard G. Hassell, EN.

The final draft of this report was lost in 1979 following the death of the principal author, Dr. Haliburton. A copy of the final draft was located at WES in 1986, and was later edited for publication. Dr. Lyndell Z. Hales, Research Hydraulic Engineer, and Ms. Holley Messing, Civil Engineering Technician, U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, Vicksburg, MS, finalized the report for publication as the last technical report of the DMRP series.

Summary

This report presents results of eight field demonstrations of various fine-grained dredged material dewatering techniques, evaluated under Dredged Material Research Program (DMRP) Disposal Operations Project (DOP), Dredged Material Densification, at the 34.4-hectare (85-acre) Upper Polecat Bay (UPB) disposal area of the USAE District, Mobile (MDO). Initial site characterization was begun in July 1975, and final dewatering demonstrations were concluded in September 1977.

The UPB disposal area was chosen for field evaluation of promising dewatering concepts because:

a. Fine-grained dredged material existing in the disposal area was a highly plastic clay with appreciable montmorillonite fraction, one of the most difficult types of dredged material to dewater.

b. Interest and cooperative assistance were available from the MDO.

c. The disposal area had easy access, was located relatively close to the U.S. Army Engineer Waterways Experiment Station, and had proper climatic conditions for year-round work.

The DMRP mission in this instance was to evaluate, within available time and funding constraints, as many potential dredged material dewatering methods as possible in such detail that opinions could be formulated relative to their technical feasibility, operational practicality, and cost-effectiveness in full-scale field application. Results of the various field demonstrations may be summarized as follows:

a. Use of surface trenching concepts to promote improved surface drainage, evaporative drying, and consolidation of fine-grained dredged material was found to be technically feasible, operationally practical, and cost-effective.

b. Technical feasibility of using wind generation systems to provide electrical power at remote disposal area locations was neither positively proved nor disproved. However, problems encountered during demonstration suggest that the concept would be operationally impractical until marked improvements were made in state-of-the-art equipment reliability and maintainability.

c. Dewatering fine-grained dredged material with conventionally installed vacuum wellpoints was found to be technically feasible and operationally practical, but is not cost-effective when compared to other alternatives.

d. Despite promising labortory results, capillary wicks were not found to be technically feasible, as the amount of dewatering produced by the devices was minimal.

e. Use of sand slurry to hydraulically fracture fine-grained dredged material and produce internal drainage layers of large horizontal areal extent was found to be technically feasible and operationally practical. Flow rates one order of magnitude greater than obtained from conventionally installed vacuum wellpoints were realized during drainage layer evaluation. Only an extremely small scale demonstration was carried out, and future detailed and long-term research is recommended for this concept, as its use in conjunction with vaccum wellpoint systems may hold promise for rapid and cost-effective dewatering.

f. Periodic mechanical agitation and mixing of upper surface crust with underlying subcrust above the liquid limit was found to accelerate the rate of dredged material surface subsidance and thus to be technically feasible, as well as cost-effective. However, such periodic mixing prevents establishment of surface vegetation, degrades disposal area aesthetics, and destroys surface support capacity of the dredged material. For these reasons and considering the amount of volume gained when compared to other alternatives, the technique was found to be operationally impractical.

g. Use of underdrainage installed prior to disposal, including gravity and vacuum-assisted underdrainage and gravity and vacuum-assisted seepage consolidation, was found to be technically feasible, operationally practical, and cost-effective for dewatering single lifts of material. Effectiveness of such systems in dewatering subsequent lifts of material was not evaluated.

h. The technical feasibility of using electro-osmosis to dewater fine-grained dredged material was neither positively established nor refuted by field demonstration, but results suggest that, unless the system is installed prior to disposal it is limited to fresh water dredged material, electro-osmosis dewatering will be technically ineffective, operationally impractical, and not cost-effective.

i. Attempts to artificially establish vegetation for dewatering purposes were unsuccessful and, had they been successful, would not have been cost-effective. Naturally established vegetation of similar species produced dense surface and subsurface growth, but with minimal reduction in dredged material water content from an engineering stand-point. Primary purposes of vegetation thus appear to be improved surface support capacity from root mat development and improved disposal site aesthetics and habitat. Better results appear to be obtained by creating conditions conducive to natural vegetation establishment rather than attempting to artificially establish desired vegetation.

Based on results of the demonstrations, it is recommended that Corps of Engineers (CE) Districts and other interest agencies use improved surface drainage techniques to promote dredged material dewatering and densification. These concepts should prove satisfactory in the great majority of instances and have the advantages of being fairly simple in concepts and low in cost.
In situations or during climatic periods when successful application of surface trenching concepts are not possible, improved underdrainage, preferably vacuum-

assisted, will provide effective dewatering and densification. When dewatering rates produced by surface drainage improvement and evaporative drying enhancement are inadequate, improved surface drainage may be combined with improved underdrainage, supplemented with vacuum consolidation, if possible, to achieve the maximum possible dewatering rate. Seepage consolidation concepts may have some use in areas where confined disposal is conducted offshore and the dredged material surface remains submerged during initial life of the site, as well as in applications where disposal area surface ponding is required for mosquito control or for other reasons.

Conversion Factors, Non-SI to SI Units of Measurement

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

Multiply	By	To Obtain
acres	4,046.873	square meters
cubic yards	0.7645549	cubic meters
degrees (angle)	0.01745329	radians
feet	0.3048	meters
gallons (U.S. liquid)	3,785412	cubic decimeters
inches	25.4	millimeters
pounds	0.4535924	kilograms

1 Introduction

Background

Statement of the problem

Dredging is the removal of sediment and other materials from the bottom of rivers, bays and other bodies of water, and is traditionally conducted for the purpose of deepening and/or widening these bodies of water to accommodate navigation. In many instances, waterways used for navigation are associated with major drainage courses which carry heavy silt loads, and thus, maintenance dredging for continued navigation is often a repetitive requirement.

Most dredging in the United States is conducted or contracted by the U.S. Army Corps of Engineers. In recent years (the 1970s), the Corps has averaged dredging 230 million cu m of material annually at a cost of approximately $170 million per year (DMRP 1976a).

The byproduct of hydraulic dredging is usually large quantities of waste material, produced in the form of a thick soil-water slurry and commonly called dredge spoil or, more recently, dredged material. This byproduct often consists of fine-grained (silt and clay) soils. Bishop and Vaughn (1972) have indicated that "...organic silty clay of high plasticity is a general and consistent product of maintenance dredging...." This waste material has been traditionally dumped into open water or placed on land in an unconfined manner. Increasing environmental concern has greatly reduced or negated open water and unconfined land disposal, especially of contaminated sediments. Therefore, the future of dredged material disposal would appear to rest with on-land disposal into areas surrounded with dikes or embankments which contain/confine the material (confined disposal areas). Several problems exist with land disposal. The most basic problem is difficulty in acquiring new disposal sites. It is estimated that approximately 7,000 acres[1] of new land will be required annually to contain material generated from maintenance dredging. Other aspects of land disposal are outlined by Boyd et al. (1972):

> *a*. . . . confined land disposal sites receive the poorest quality spoil (from the engineering point of view) and . . . the quality will likely get worse before it gets better. Consequently, problems associated with spoil

[1] A table of factors for converting Non-SI units of measurements to SI units is presented on page xvii.

drainage . . . containment area management, and subsequent utilization will become more acute.

b. Behavioral characteristics of dredge spoil in containment areas have not been thoroughly investigated. It is apparent that most dredge spoils improve with time if drainage is provided, and at some point in time these materials can be used as foundation or building materials. There is a need for research on the characteristics of dredge spoil which will enable it to play a more positive role in urban and regional development projects.

c. Dewatering techniques must be developed to allow full utilization of the capacity of diked containment areas and/or the reuse of such areas. Research is needed on . . . techniques to speed consolidation of material in the confined areas . . .

d. Efforts to make useful products such as building materials from spoil have been few and have met with mixed success.

Montgomery and Palermo (1976) discussed another dimension of the problem, of concern to the Corps of Engineers:

> Confined land disposal of dredged material fulfills one short-term Corps need (i.e., disposal of dredged material); however, it often creates rather than alleviates problems in land utilization and management. This, in turn, is of direct significance to the Corps since the problems created quickly influence public opinion and public acceptance of land disposal. The manifestation of this is in the increasing difficulty, by the Corps and its project sponsors, to acquire easements for additional land disposal sites.

There appears to be an obvious need for improvement in on-land confined disposal area management. Existing disposal areas and those developed in the future must be used to the maximum extent possible. This view (DMRP 1977) was reiterated by Congress in Section 148 of Public Law 94-587, the Water Resource Development Act of 1976:

The Secretary of the Army, acting through the Chief of Engineers, shall utilize and encourage the utilization of such management practices as he determines appropriate to extend the capacity and useful life of dredged material disposal areas such that the need for new dredged material disposal areas is kept to a minimum. Management practices authorized by this section shall include, but not be limited to, the construction of dikes, consolidation and dewatering of dredged material, and construction of drainage and outflow facilities.

Dredged Material Research Program (DMRP)

The need for addressing dredged material disposal problems resulted in a Congressionally-authorized research and development program, which was assigned to the U.S. Army Engineer Waterways Experiment Station (WES). Preliminary study was initiated in May 1971. Funding for a full-scale 5-year research program was authorized in February 1973. WES initiated the Dredged Material Research Program (DMRP) in March 1973, with the stated objective:

> To provide, through research, definitive information on the environmental impact of dredging and dredged material disposal operations and to develop technically satisfactory, environmentally compatible, and economically feasible dredging and disposal alternatives, including consideration of dredged material as a manageable resource.

Dredged material densification

Much DMRP effort has been devoted to developing techniques for dewatering and consolidating fine-grained dredged material placed in confined disposal areas. Dredged material is placed hydraulically, usually by pumping through pipes from a dredge, in a slurry state. Although a significant amount of water is removed from disposal areas through overflow weirs, the resulting settled dredged material, at equilibrium, contains large amounts of water and has the consistency of warm axle grease. The extremely high water content makes the dredged material unsuitable or undesirable for any commercial or productive use. Also, the volume of space occupied by the liquid portion of the dredged material greatly reduces the remaining volume available for future disposal. The Dredged Material Densification research of the DMRP Disposal Operations Project (DOP) was created to study and evaluate various methods of dewatering and consolidating fine-grained dredged material. The objective of this research was:

> . . . to develop and test promising techniques for dewatering or densifying dredged material using mechanical, biological, and/or chemical techniques prior to, during, and after placement in containment areas . . .

Three major reasons exist for dewatering fine-grained dredged material placed in confined disposal areas:

a. To promote material shrinkage and consolidation, leading to creation of more disposal volume which can then contain additional dredged material.

b. For reclamation of the dredged material into soil form for removal and use in dike-raising, other engineered construction, or other productive use, again creating more available disposal volume.

c. To create stable flat land at a known final elevation and with predictable geotechnical properties.

Rationale for Test Site Selection

The overall research plan included, after initial literature and laboratory feasibility studies, demonstration and evaluation of promising dewatering techniques under field conditions (DMRP 1976b). Thus, during the spring of 1975, the DMRP DOP staff began to discuss the type of field demonstration site needed.

Because of time and funding constraints, it was decided that comprehensive study of all dewatering alternatives at one test site would be preferable to evaluating individual techniques at test sites around the country. It was also believed that better relative comparisons among the individual dewatering techniques could be obtained if all were evaluated on similar dredged material. The cost of administration, management, and obtaining background data would be minimized. Extrapolation of field demonstration results to other potential sites could be made on the basis of soil properties and the known laws of soils engineering behavior. If the dredged material selected was of a type relatively difficult to dewater, successful results would have a high probability of useful application at other sites.

During the spring of 1975, the U.S. Army Engineer District, Mobile (MDO), contacted the DMRP for information on dewatering fine-grained dredged material. As a result of this mutual interest, discussions were held in the summer of 1975 between the DMRP DOP staff and the MDO. It was subsequently agreed that the MDO would make an existing disposal area available to the DMRP, provide some financial support, and provide cooperative assistance in planning and field operations. In return, the DMRP would conduct all dewatering demonstrations and support activities in an MDO disposal area. The MDO suggested the Upper Polecat Bay (UPB) disposal area in Mobile, AL, be used as the test site. After initial study, sampling, and characterization to determine the nature of the contained dredged material, which was found to be a highly plastic montmorillonite containing silty clay (and thus satisfactory for technology evaluation), the DMRP DOP staff agreed that the UPB disposal area would be satisfactory for conduct of field demonstrations.

Disposal Area Description

The UPB disposal area is located in Mobile, AL, on the Mobile River, just north of the Cochran Bridge. The 85-acre site was created in 1970 by end-dumping sand from previous new-work dredging to create a perimeter dike to an elevation of about 4.9 m surrounding and existing marsh at elevations ranging from 0 to approximately 0.3 m. In 1971 and 1973, the area was used for disposal of dredged material from maintenance dredging projects on the upper Mobile River and Chickasaw Creek channels. The material placed in the area was predominately an organic clay sediment of high plasticity. The disposal area is shown in Figure 1, prior to initiation of DMRP dewatering studies. Subsequent chapters describe the disposal area history and its geotechnical characterization in more detail.

Field Demonstrations Conducted at Site

DMRP field research operations at UPB were initiated in the summer of 1975 with immediate goals of removing surface water and initiating drying over the majority of the site while maintaining both the north and south ends of the disposal area in relatively undisturbed and undried condition. These virgin areas were to be used for evaluation of selected dewatering techniques during the spring and summer of 1976.

Figure 1. Aerial view of Upper Polecat Bay disposal area, Mobile, AL, prior to initiation of Dredged Material Research Program (DMRP) dewatering studies

As shown in Table 1, a total of ten field demonstrations for DMRP DOP were conducted at the UPB site. General locations of the various field demonstrations are show in Figure 2. The Remote Weather Station did not operate satisfactorily and was terminated. Weather data from the Mobile Office of the National Weather Service was used instead. Preliminary studies resulted in a decision to undertake full-scale dike raising activities using dewatered dredged material created by the other dewatering research.

Table 1
Field Demonstrations Conducted at Upper Polecat Bay Field Site

Tasks	Title	Location in Disposal Area
5A08	Progressive trenching	Center
5A09	Windmill powered vacuum wellpoints	N End
5A10	Capillary wicks	NW End
5A11	Sand slurry injection	N End
5A12	Remote weather station	N End
5A14	Crust mechanical stabilization	SW End
5A15	Gravity underdrainage dewatering	SE End
5A16	Electro-Osmotic dewatering	N End
5A18	Vegetation dewatering	NE End
5A20	Interior borrow development and mining	Center

Purposes of Field Demonstrations

The purposes of the DMRP DOP field demonstrations were to evaluate, within available time and funding constraints, as many potential dredged material dewatering methods as possible in such detail that opinions could be formulated relative to their technical feasibility, operational practicality, and cost-effectiveness in full-scale application. It was not intended that the various demonstrations result in a clear understanding of all facets of dredged material behavior for each method tried, as research on any one method at such an intensive level would have exhausted all of the available funding for the entire program, taken more calendar time than was available, and could not be justified for any site-specific application, given the existing state-of-the-art and the variability of soil (and thus dredged material) in general.

Criteria used in assessment of field demonstration results were as follows:

a. *Technical Feasibility*. A method or technique was judged technically feasible if it accomplished the desired result, i.e., dewatered and/or densified the fine-grained dredged material. In some instances described herein, this rather simple condition could not be conclusively established, as other factors (poor experiment design or unreliable equipment) affected the demonstration. However, in all instances, such cases did not satisfy other acceptance criteria.

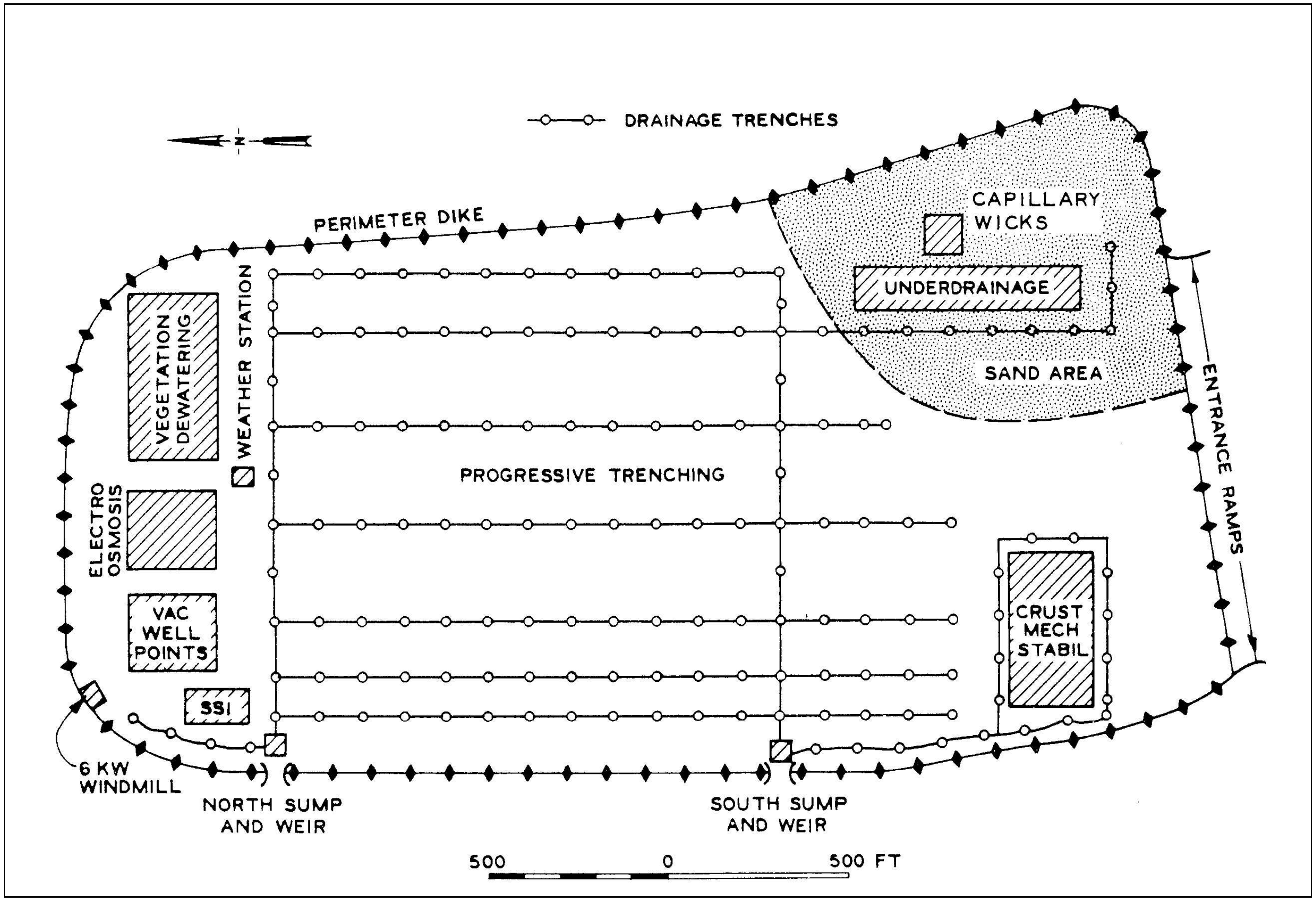

Figure 2. Location of various dewatering demonstrations at Upper Polecat Bay disposal area, Mobile, AL

b. *Operational Practicality*. A method was judged (subjectively) to be operationally practical if the materials, techniques, equipment, operation procedures, and related items required could be scaled-up without loss of efficiency, if conduct of such dewatering methodology could be immediately undertaken by normal CE field element personnel, either by contract or with in-house capability, if appreciable dewatering could be accomplished within the normal time intervals between disposal area filling, and if use of the methodology did not create other obvious problems in disposal area operation.

c. *Cost-Effectiveness*. A method was judged to be cost-effective when the unit cost of creating new disposal area storage volume by dewatering and consolidation was less than $4.00/cu m. This decision was based on previous DMRP research (Johnson et al. 1977) substantiated by data obtained in the Mobile area, that, except in localized high-cost areas, the unit cost of creating disposal area storage volume by perimeter dike construction was about $0.33/cu m - $0.40/cu m over most of the United States. It was believed that if unit volume creation costs of dewatering exceeded unit volume creation costs associated with dike-raising by more than an order-of-magnitude, dewatering per se should not be conducted without exploring alternate disposal or containment schemes.

Purposes of Report

The purposes of this report are to: (a) describe and characterize the field test site selected for evaluation of promising DMRP DOP dewatering methods, and (b) describe each field demonstration in sufficient detail that opinions concerning its technical feasibility, operational practicality, and cost-effectiveness may be developed.

2 Test Site Disposal History and Characteristics

Source of In Situ Dredged Material

Dredged material in the UPB disposal area is from maintenance dredging activities in the upper Mobile River and Chickasaw Creek, a main tributary of the Mobile River. Navigation channel dimensions in this area have been enlarged several times since initial improvement of Mobile Harbor in 1826. These channels are now maintained as a part of the Mobile Harbor Project to a mean low water (mlw) depth of 12.2 m and width of 152.4 m for the Mobile River and 7.6-m mlw depth and 76.2-m width for Chickasaw Creek, as shown in Figure 3.

The upper segments of the project are maintained by hydraulic pipeline cutterhead dredges. In past years, land disposal of material from maintenance dredging in the upper Mobile River was unconfined. Blakeley Island, shown in Figure 3, has a long history of unconfined disposal, and the area has undergone considerable physical change from accretion and shoaling. Disposal in confined land areas is now required because of environmental constraints.

Average shoaling rates require maintenance dredging volumes of approximately 900,000 cu m annually from the Mobile River Channel and 150,000 cu m annually from Chickasaw Creek. The sediment is primarily silt and clay, based on sampling programs conducted by the MDO.

Disposal Area Design and Construction

Requirements for confinement of dredged material from the upper part of Mobile Harbor led to selection of diked disposal areas at locations historically used for unconfined disposal on Blakeley Island. The total area originally approved for diked disposal included a large portion of upper Blakeley Island surrounding Polecat Bay. Prior to dike construction, an investigation of foundation conditions, including both field and laboratory testing programs, was conducted by the MDO. Ten classification borings and one boring to obtain undisturbed samples were made along the proposed dike centerline

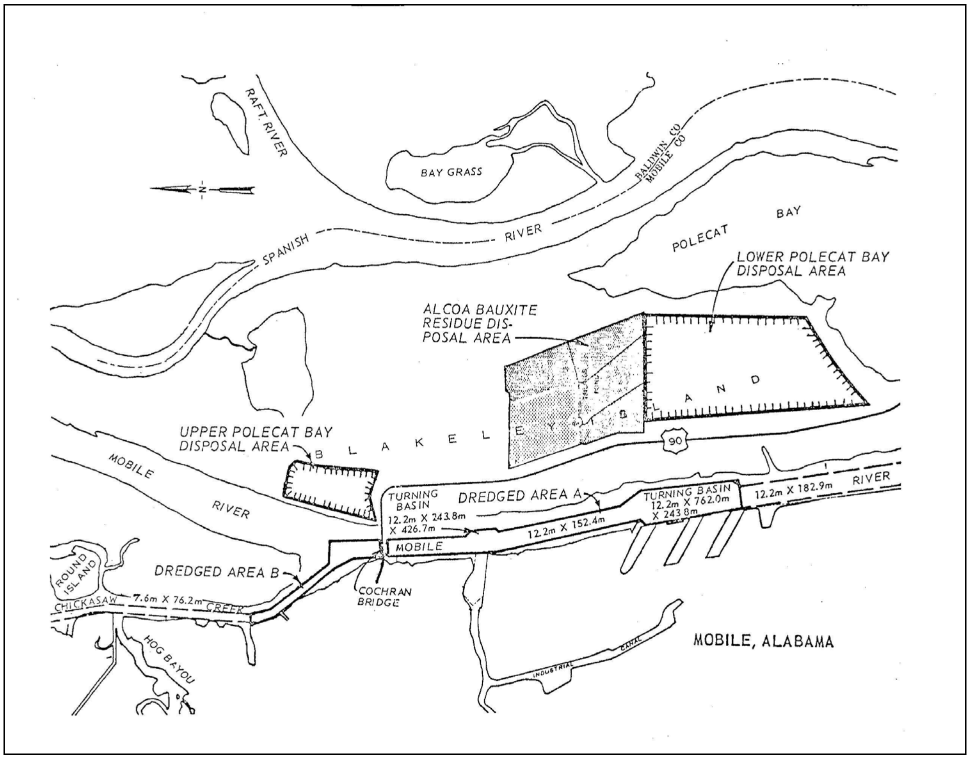

Figure 3. Sources of dredged material from upper Mobile River and Chickasaw Creek deposited in Upper Polecat Bay disposal area, Mobile, AL

(Palermo 1977b). These borings revealed that the proposed disposal area foundation consisted primarily of marsh deposits composed of soft organic highly plastic clays and silts (OH)[1] underlain by alternating strata of highly plastic clay (CH), silty sands (SM), and clayey sand (SC). Silty sands and sands overlay the organic material in the southern portion of the site, probably deposited by previous unconfined dredging operations. Some of this coarse-grained material was subsequently utilized for dike construction. Laboratory tests, including triaxial compression and consolidation tests, were performed on samples from the upper strata of highly plastic organic and inorganic clays. Assessment of test data indicated that the in situ foundation material had very low shear strengths and was highly compressible.

Construction of stable retaining dikes on the soft organic foundation was accomplished by end-dumping sand and displacing the soft materials. Approximately 191,139 cu m of fine sand (SP) available in the southern portion of the area was borrowed for dike construction. A bulldozer was used to push the dumped sand onto the soft foundation and shape a base for the advancing fill. During construction, soft foundation material was displaced, creating a mud wave at the head and sides of the base section. Following final placement of the base section, the embankment was formed by end-dumping sand. The embankment was semi-compacted by truck traffic, but no other compactive effort was used. Slopes of the completed dike were seeded, and overlapping sheets of

[1] Symbols in parentheses refer to USCS classification of the material.

6-mil polyethylene sheeting was placed on the interior slopes (USAE District, Mobile 1975).

Conventional dragline construction techniques were used to place dike material in the southeastern part of the site where better foundation conditions existed.

The dike was constructed with a total length of 2,895.6 m. Crown elevations varied between el 14 and 16 ft mlw [1], with side slopes of approximately 1V:1H. Natural ground in the site interior was el 2 to 3 ft mlw, except for higher elevations of el 5 to 10 ft mlw in the southern portion of the site. Two outlet weirs (box-type, fabricated from sheet steel with a weir crest length of 6.10 m) were located at points along the east dike.

Post-Construction Investigations

Three additional foundation borings were made by the MDO at the UPB site following completion of the dikes to determine displacement of foundation material resulting from dike construction. Examination of boring logs indicated that significant displacement of the soft marsh materials had occurred during dike construction.

A separate investigation was later conducted by the MDO to determine the nature of the sand dike foundation (Winter 1972). The sand base formed a bulb-shaped mass below the original ground line and displaced of the soft clays.

Two foundation borings were also made in 1976, after dredged material placement, in connection with DMRP studies at the UPB site. These borings were made to further define foundation stratification and to obtain undisturbed samples of the compressible foundation material as consolidated from the overburden of dredged material placed in the site. Piezometers were installed in both bore holes to determine groundwater conditions within the foundation. All foundation investigations at the UPB site indicated similar generalized foundation conditions, as shown in Figure 4.

Dredged Material Disposal Operations

Dredged material was first placed in the UPB area during the period December 1971 to March 1972. The material was dredged from the upper Mobile River navigation channel immediately below the site, from Cochran Bridge to a point approximately 842.8 m south of the bridge. The 0.686-m pipeline dredge DAVE BLACKBURN was used under contract with the MDO. A summary of dredging data is presented in Table 2. The limits of this work were indicated in Figure 3, and will hereafter be referred to as Dredged Area A.

The UPB site was designated as the primary disposal site for material from Dredged Area A, with a diked area opposite Chickasaw Creek as the secondary disposal site. Outlet pipes were placed at both disposal sites and were connected with a "Y" valve, allowing disposal into either site. As material was placed in

[1] Survey elevation data presented here and elsewhere in the report are reported in units of feet, as these were of official units of the U.S. Coast and Geodetic at the time of report preparation.

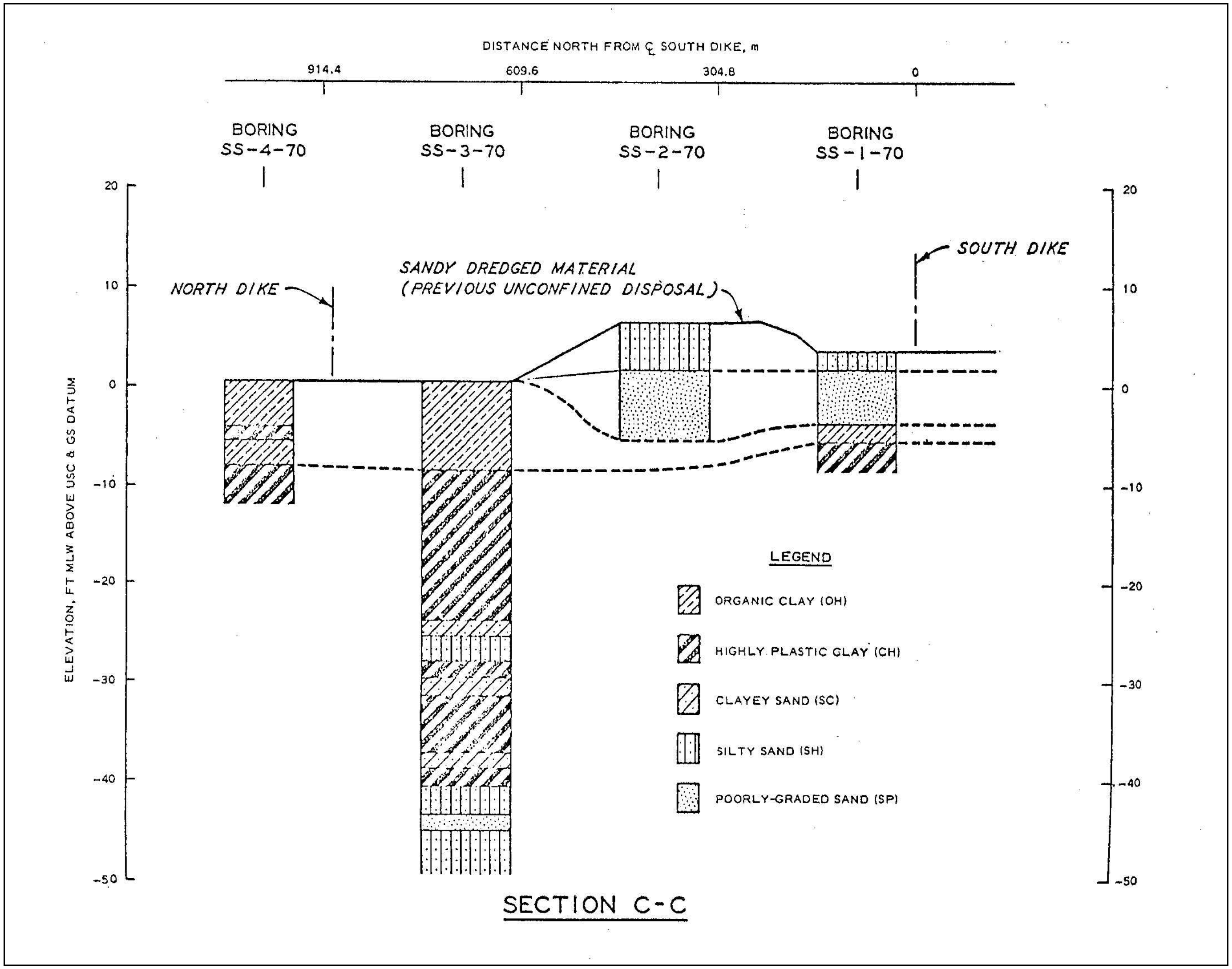

Figure 4. Generalized foundation conditions prior to dike construction and placement of dredged material into Upper Polecat Bay disposal area, Mobile, AL

Table 2
Pertinent Data for Dredging of Dredged Area A

Dredge DAVE BLACKBURN	
Dredge size	27-in. (0.686-m)
Dredging period	December 1971 to March 1972
Location	See Figure 3
Total days on job	83 days
Days lost	20 days
Total pumping time	894 hr
Gross capacity per hour	1,390.7 cu m
Average daily advance	117.4 m
Net yardage	1,069,746 cu m
Gross yardage	1,242,697 cu m

the primary site and suspended solids concentrations in the effluent reached limiting values, dredged material would then be temporarily route to the secondary site for disposal.

Approximately 1,223,288 cu m of sediment were removed from the channel. The great majority of the material was placed in the UPB site; however, records concerning exact volumes placed in the respective sites were not kept. The inlet pipe location at the UPB site was near the southeast corner of the disposal area. Sediment removed from Dredged Area A consisted primarily of fine-grained clays and silts, with a small fraction of sand. During disposal, the coarser materials were deposited near the inlet pipe, creating an area of high ground in the southeast corner of the disposal area. Fine-grained material was carried northwestward toward the discharge weir and was eventually deposited over most of the disposal area.

The UPB site remained inactive until January through March 1973, when dredging operations were again performed in the upper Mobile River. Material was placed in the UPB site from an area immediately north of Cochran Bridge, extending approximately 1,655 m into the Chickasaw Creek channel. The 0.457-m pipeline dredge STUART was used under contract with MDO. A summary of dredging data is presented in Table 3. The limits of this work were indicated in Figure 3, and will hereafter be referred to as Dredged Area B.

The inlet pipe for material from Dredged Area B was initially located in the southeast corner of the disposal area. Sandy material was encountered immediately above Cochran Bridge during the dredging operation, and this material further added to the high mounded area in the southeast corner of the disposal area. The inlet pipe was later moved adjacent to the south discharge weir during the dredging operation in the Chickasaw Creek channel. This weir was closed during that portion of the dredging operation. The dredged material from Chickasaw Creek primarily consisted of fine-grained clays with small fractions of sand and contained significant amounts of wood chips and bark present in the channel as residue from wood-processing industries located along the creek. The coarser-grained materials and wood chips were deposited in front of the inlet pipe. Fine-grained material was carried toward the north weir and into a low energy area created by poor circulation in the southwest corner of the disposal area. A total of 194,645 cu m of sediment was removed from Dredged Area B.

Table 3
Pertinent Data for Dredging of Dredged Area B

Dredge STUART	
Dredge size	18 in. (0.457-m)
Dredging period	October 1972 to June 1973
Location	See Figure 3
Total days worked	68
Total pumping time	887 hr
Gross capacity per hour	1,205.7 cu m
Average daily advance	69.2 m
Net yardage	733,738 cu m
Gross yardage	1,071,272 cu m

3 Field and Laboratory Geotechnical Characterization

Field Investigation Program

Field investigations at the UPB disposal area were conducted to characterize the site and to obtain information on dredged material properties. The investigations consisted of site surveys and borings taken in the dredged material to obtain samples for laboratory testing. Exploration services were provided by the Core Drill Section, MDO.

Initial site condition surveys at the time of field investigations indicated that a surface crust of dried material 51-mm to 152-mm thick existed over a majority of the 34.4-ha site. Desiccation cracks in the surface crust exhibited a typical polygonal pattern. In many areas, the thin crust would not support a man's weight. A layer of fine-grained dredged material approximately 2.44-m thick, having the consistency of warm axle grease, existed beneath the crust. This material was generally at water contents above the liquid limit. Approximately 80 percent of the site-surface area was under ponded water, and little vegetation existed. Topography was generally flat with a higher area of sandy material located in the southeast corner of the site and a gentle grade from this area down to the location of the north outlet weir.

An initial boring was made at the UPB disposal area in May 1975, located 152.4-m east of the north outlet weir, to obtain samples for initial definition of dredged material physical properties. Soil conditions would not support the weight of drilling equipment, so samples were taken by hand-pushing 127-mm ID Shelby tubes. The boring was carried to a depth of 3.05 m.

Twenty-six additional borings were made during July and August 1975, designated BI-1 through BI-26. Boring locations were chosen to characterize generally conditions within the entire disposal area and to obtain detailed information along the central east-west axis of the site. Borings were made by hand using a 76.2-mm ID piston-type Hvorslev sampler, with continuous samples taken to a maximum depth of 3.81 m (through the dredged material layer into foundation soils) below the dredged material surface. Borings BI-5 and BI-25 were attempted in the southeast corner of the disposal area, but the sandy material exhibited a high resistance to hand push sampling, so only surface samples were taken. A total of 102 undisturbed dredged material samples were taken during the investigation.

Groundwater level observation wells were installed in 24 boreholes. The wells were fabricated from 1.52-m sections of Schedule 80 slotted plastic pipe, connected to 2.44-m plastic pipe risers. The slots were wrapped in Filter-X filter cloth and seated to a depth of approximately 3 m. Details of the installation are shown in Figure 5.

Laboratory Testing Program

Samples from the May 1975 127-mm bores were tested by the Division Soils Laboratory of the USAE Division, South Atlantic. Testing included USCS classification, natural water content, in situ density, Atterberg limits, and particle size determination. The procedures followed conformed to EM 1110-2-1906.

Tests performed on the July-August 1975 76.2-mm undisturbed samples from borings BI-1 through BI-26 were carried out by the WES Soils and Pavements Laboratory and included USCS classification and natural water content determination on all samples with in situ density, particle size determination, specific gravity of solids, Atterberg limits, vane shear, and consolidation tests carried out on selected samples. The testing program sought to determine physical and engineering properties of the dredged material for use in estimating potential shrinkage and consolidation. The testing procedures conformed to EM 1110-2-1906. Selected samples were also tested to determine shrinkage behavior, using methods described in Appendix X of B (1977b), and testings was performed by the WES Environmental Effects Laboratory.

Results of Field and Laboratory Investigations

Test results for the 127-mm boring are summarized in Table 4. Densities increase with depth while water contents decreases with depth, reflecting gravity consolidation of the dredged material. Individual boring logs and test data are presented in Appendix B of Palermo (1977a).

Physical and Index Properties

Eighty-two of 102 dredged material samples from the 76.2-mm undisturbed borings were fine-grained and USCS classified CH (highly plastic inorganic clay). Despite its CH classification, the material typically contained approximately 5 percent organics. Grain-size analyses and Atterberg limit determinations were performed on 34 CH samples. A composite grain-size distribution curve is presented in Figure 6. Twenty-one samples contained minimal sand with an average of 93 percent (by weight) passing the U.S. No. 200 sieve and 41 percent finer then 1 μm (0.001 mm). The remaining fine-grained samples had an average of 78 percent passing the U.S. No. 200 sieve and 31 percent finer than 1 μ. Grain-size data are summarized in Table 5 and individual grain-size distribution curves are presented in Appendix C of (1977a).

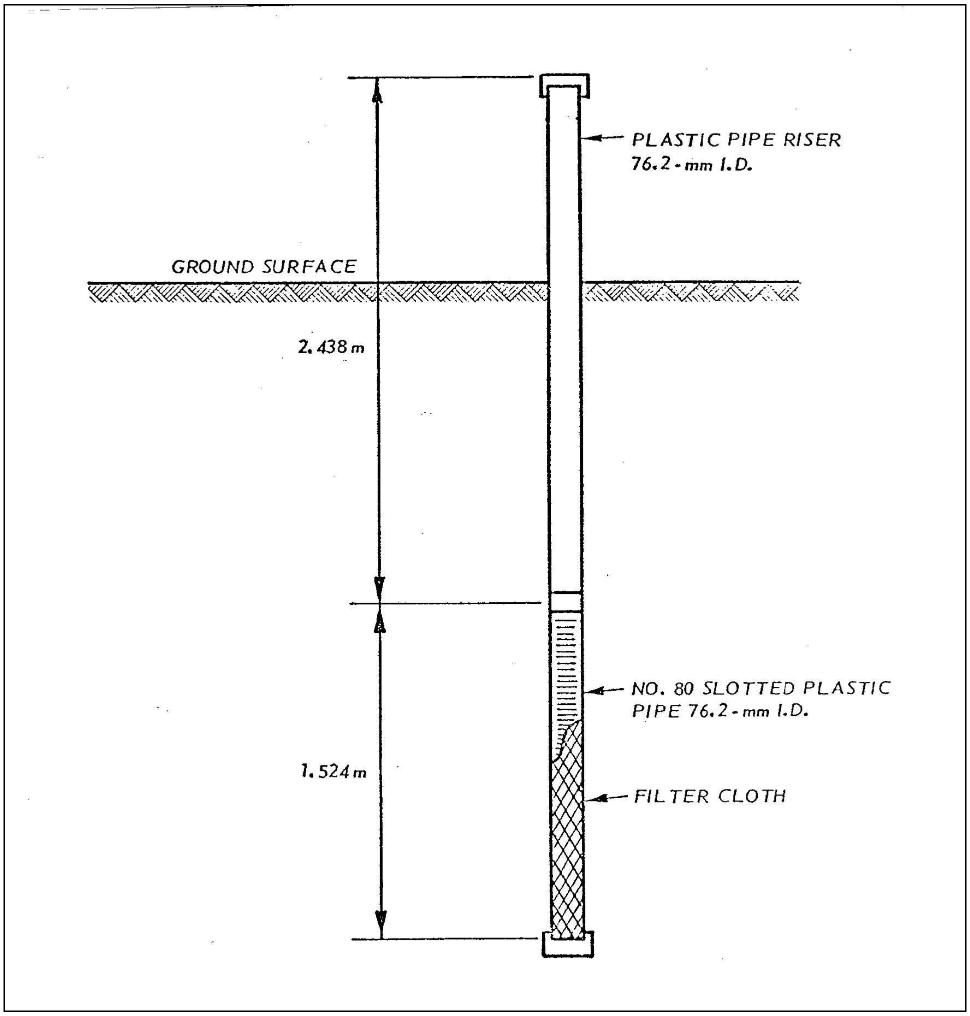

Figure 5. Observation well detail

Table 4
Test Results from 127-mm Undisturbed Samples

Depth, m	USCS Classification	Natural Water Content, %	Liquid Limit	Plasticity Index	Percent Passing U.S. No. 200 Sieve	Dry Density Kg/cu m, lb/cu ft	Wet Density kg/cu m, lb/cu ft
0.00-0.61	CH	167	111	70	89	43.4, 27.1	126.8, 79.2
0.61-1.22	CH	128	89	59	94	54.6, 34.1	132.9, 83.0
1.22-1.83	CH	98	71	45	88	68.8, 43.0	142.5, 89.0
1.83-2.44	CH	86	83	56	95	77.7, 48.5	148.9, 93.0
2.44-3.05	CH	70	66	41	85	90.6, 56.6	155.3, 97.0

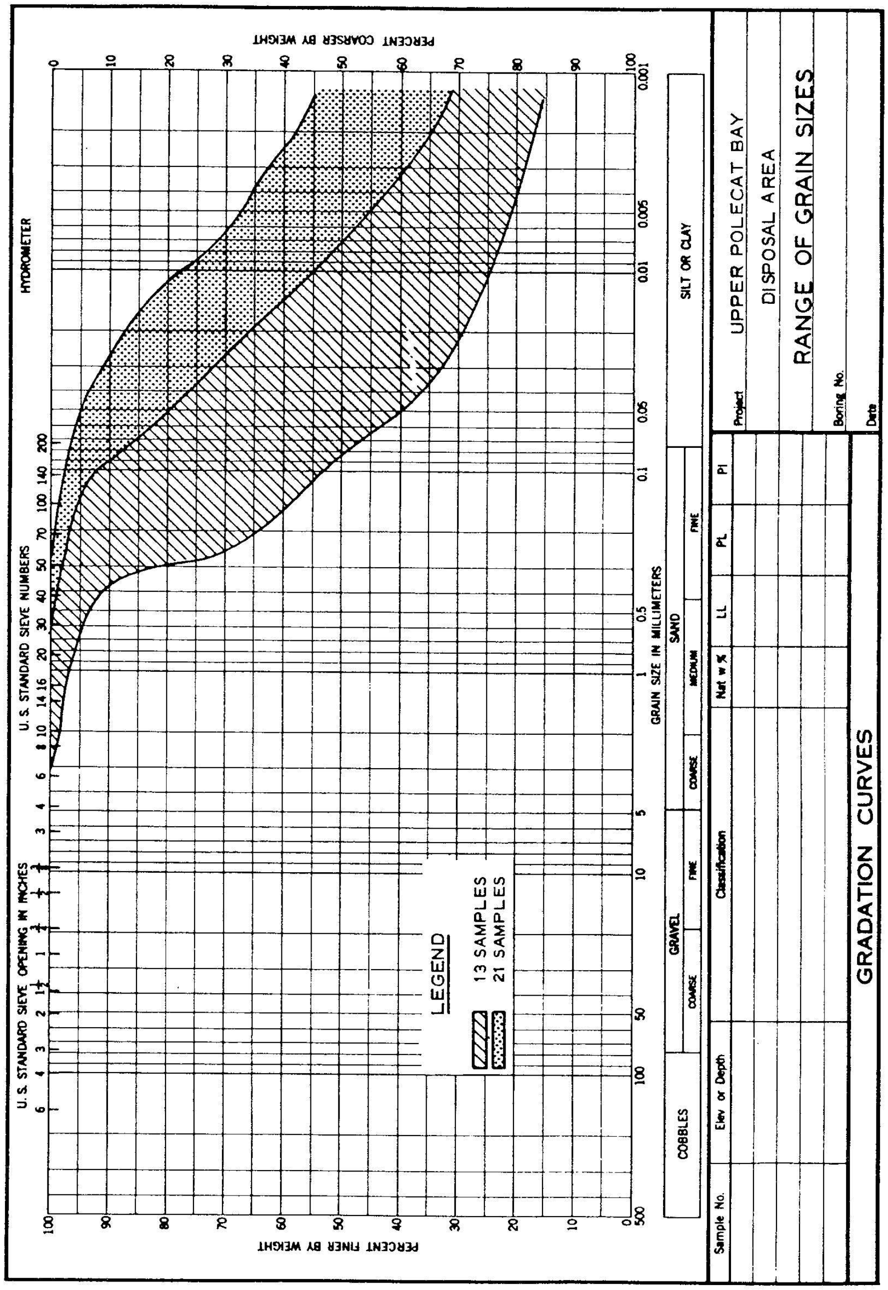

Figure 6. Composite grain-size curve for fine-grained dredged material

Table 5
Summary of Grain-Size Data for Fine-Grained Dredged Material

USCS Classification	No. of Samples	D_{90}, mm	D_{60}, mm	D_{50}, mm	Percent Passing, U.S. No. 200 Sieve	Percent Finer than .001 mm
Highly plastic clay (CH)	21	.058	.0087	.004	93	41
Highly plastic sandy clay (CH)	13	.123	.0360	.014	78	31

A few of the samples were USCS classified SM (silty sand), SC (clayey sand), or SP (poorly-graded sand). Coarse-grained material generally settled near the inlet location while finer-grained material was carried toward the discharge weirs. Some samples taken from the 2.28-m to 3.05-m and 3.05-m to 3.81-m depths had a USCS classification of OH (highly plastic organic clay and silt); these samples are of the disposal site foundation.

Atterberg limits were determined for 33 dredged material samples. The data are also summarized in Table 6. Testing included determination of the sticky limit, the water content above which a mixture of soil and water will adhere to a steel spatula in addition to the usual liquid and plastic limits. Results indicate that the material behaves as highly plastic inorganic clay (CH), even though it contained approximately 5 percent of organic material.

A total of 42 specific gravity of solids determinations were made. Individual values are presented in Appendix B of Palermo (1977a, b). The average specific gravity of solids for all samples tested was 2.66.

The in situ dry density of selected dredged material samples was determined during consolidation testing. Values are plotted separately and are also presented with consolidation data in Appendix D of Palermo (1977a). Dry density values generally increased with depth reflecting the self-weight gravity consolidation of the dredged material.

Engineering Properties

Consolidation tests were run on 20 selected samples of fine-grained dredged material. Specimens were prepared in 63.5-mm ID rings and loaded in fixed ring consolidometers. Load increments of 0.392, 3.992, 7.845, 15.690, 31.381, 62.762, and 125.525 kPa were used in the tests. Rebound increments were also run on a majority of the tests. Consolidation data are summarized in Table 7, including values of the compression index C_c and preconsolidation pressure P_c. Individual test data, including e-log P and e-log t plots, are presented in Appendix D of Palermo (1977a).

Table 6
Atterberg Limit Summary for Dredged Material

Boring	Sample	USCS Classification	Liquid Limit, LL	Plastic Limit, PL	Stickey Limit, SL	Plasticity Index, PI	Water Content, W_n, %	Liquidity Index, I_L
BI-3	1	CH	150	33	95	117	135	0.87
	2	CH	99	35	49	64	130	1.48
	3	CH	86	36	48	50	77	0.82
	4	CH	89	33	50	56	91	1.03
BI-4	1	CH	105	35	51	70	103	0.97
	2	CH	77	31	46	46	86	1.20
	3	CH	79	30	36	49	75	0.92
	4	CH	105	30	49	75	149	1.59
BI-8	1	CH	165	52	115	113	183	1.16
	2	CH	98	32	42	66	113	1.23
	3	CH	78	25	40	53	98	1.38
	4	CH	69	25	52	44	73	1.09
BI-9	1	CH	106	36	53	70	180	2.06
	2	CH	76	26	33	50	134	2.16
	3	CH	85	29	44	56	83	0.96
	4	CH	57	24	33	33	194	5.15
BI-12	1	CH	130	47	74	83	131	1.01
	2	CH	81	26	40	55	89	1.15
	3	CH	79	29	40	50	78	0.98
	4	CH	52	21	30	31	40	0.61
BI-16	1	CH	171	56	97	115	109	0.46
	2	CH	93	33	50	60	88	0.92
	3	CH	83	31	46	52	87	1.08
	4	CH	107	36	50	71	180	2.03
BI-19	1	CH	90	31	47	59	96	1.10
	2	CH	78	27	41	51	79	1.02
	3	CH	90	30	48	60	81	0.85
BI-22	1	CH	81	31	48	50	77	0.92
	2	CH	64	25	35	39	44	0.49
BI-24	1	CH	107	36	71	71	126	1.26
	2	CH	78	29	44	49	99	1.43
	3	CH	73	25	36	48	71	0.96
	4	CH	83	28	42	55	65	0.67

Laboratory consolidation tests on settled material usually indicate an over-consolidated condition until the effective overburden stress is exceeded (Bishop and Vaughan 1972). This behavior was noted for the material tested with the higher load increments for all tests indicating a virgin compression relationship with clearly defined values for C_c. Values for preconsolidation pressure P_c were determined by accepted EM 1110-2-1906 Casagrande graphical construction. P_c values were generally higher than existing overburden pressures for samples from shallow depths (0 to 0.76 m), probably from desiccation of the upper layers. Also, previous laboratory consolidation testing on dredged material indicates that P_c values greater than overburden may be attributable to sample disturbance during trimming and ring friction during the test (Salem and Krizek 1973).

Table 7 Summary of Laboratory Consolidation Test Results							
Sample No. BI-	**USCS Classification**	**Compression Index, C_c**	**Preconsolidation Pressure, P_c kPa, lb/sq ft**	**Initial Void Ratio, e_o**	**Natural Water Content, w_o, %**	**Initial Dry Density kg/cu m, lb/cu ft**	**SP Gravity of solids, G_s**
2-2		1.084	10.5, 220	3.44	126.4	60.8, 38.0	2.70
3-1	CH	1.139	12.9, 270	3.27	119.8	57.6, 36.0	2.46
3-2	CH	1.148	8.0, 168	3.45	128.9	59.7, 37.3	2.66
3-3	CH	0.956	12.9, 270	2.82	99.6	71.7, 44.8	2.74
3-4	CH	0.785	11.9, 248	2.58	88.2	75.9, 47.8	2.74
3-5	CH	2.099	27.8, 580	4.04	158.5	47.2, 29.5	2.38
4-1	CH	1.259	15.4, 322	3.64	133.1	58.3, 36.4	2.70
4-2	CH	1.013	10.9, 228	3.14	117.4	64.0, 40.0	2.65
4-3	CH	0.600	9.1, 190	2.10	76.9	86.0, 53.7	2.67
4-4	CH	0.707	11.7, 244	2.20	77.7	85.2, 53.2	2.73
4-5	CH	1.460	24.9, 520	2.90	111.6	62.8, 39.2	2.45
8-2	CH	1.036	6.7, 140	3.20	115.3	64.7, 40.4	2.72
9-2	CH	1.103	5.1, 106	3.71	133.0	57.5, 35.9	2.71
10-2	CH	0.770	11.3, 236	2.59	92.2	75.4, 47.1	2.71
12-2	CH	0.860	10.6, 222	2.83	103.3	71.2, 44.5	2.73
16-2	CH	1.243	8.2, 172	3.87	138.6	56.0, 35.0	2.73
19-2	CH	0.945	13.2, 276	2.78	100.3	71.6, 44.7	2.71
21-2	CH	1.041	12.6, 264	3.12	108.9	66.0, 41.2	2.72
23-2	CH	0.966	13.6, 284	2.80	99.7	71.4, 44.6	2.72
24-2	CH	0.948	8.9, 186	2.57	90.8	75.6, 47.2	2.70

Values of the compression index C_c varied between 0.60 and 1.26 for the CE dredged material with an average value of 0.92. Foundation samples (OH) yielded higher values.

Values of the coefficient of consolidation c_v were computed using the e-log t data for 50 percent primary consolidation. Individual data are given in Appendix D of Palermo (1977a). Values of c_v ranged between 4.0×10^{-9} and 1.3×10^{-7} sq m/sec with an average minimum of 3.0×10^{-8} sq m/sec for effective stresses in the range 7.845 to 15.690 kPa.

Reduction in volume of fine-grained dredged material from desiccation shrinkage is an important part of any densification process. Previous research on dredged material drying and crust formation indicated that volume reduction is essentially equal to the volume of water removed (Brown and Thompson 1977). The change in volume from removal of water at low water contents is also dependent upon the type and relative amount of clay minerals present in the dredged material (Haines 1923). Clay mineralogy analyses indicated that the dredged material had a Montmorillonite content of about 25 percent and, thus, a high shrinkage potential. Linear shrinkage tests were carried out to estimate the dredged material densification expected from desiccation. Tests were run using linear shrinkage molds and a special test procedure (Appendix E of Palermo (1977a)) based on Test Method Tex 107-E used by the Texas Highway Department (THD) for determining the linear shrinkage. Dredged material was placed in linear shrinkage molds at initially high water contents and air-dried. In addition to drying the material to the shrinkage limit as called for the THD test, the water content and resulting volumetric change was determined periodically during the drying process. A correlation between water content and volumetric change was then determined.

A total of 49 linear shrinkage tests was performed on CH dredged material samples. The samples were initially placed in molds at water contents above the liquid limit and dried to the shrinkage limit. All tests indicated a linear relationship between water content (w) and percent of initial volume (%v). The slope of the linear relationship w/%v was defined as the coefficient of shrinkage C_s. The average C_s value for all tests was 2.34. Average volume reduction was equal to the volume of water removed. Differences in the value of C_s may thus be attributed to differences in initial water content of the samples.

At low water contents, particle repulsion and friction tends to limit volume reduction, slightly affecting the linear relationship. Previous research regarding soil drying and shrinking has also indicated a deviation from a linear relationship at low water contents (Brown and Thompson 1977). However, since it was doubtful that these extremely low water contents could be achieved by dredged material dewatering and densification techniques now available, a simple straight-line relationship (as shown on the shrinkage test results in Appendix D of Palermo 1977a) is representative of expected field behavior.

The hydraulic conductivity (coefficient of permeability) k of the dredged material was determined indirectly by laboratory consolidation testing data and directly by variable head field permeability tests. Consolidation test data were used to indirectly compute values of k using the EM 1110-2-1906 relationship. Based on the time for achieving 50 percent primary consolidation, computed values of k are shown plotted in Figure 7 for various consolidation pressures with an average indicated value of 1×10^{-9} m/sec.

Variable head permeability tests were run on 14 small diameter wellpoints installed at the UPB site. The wellpoints were of the Casagrande type, constructed using 152.4-mm lengths of 25.4-mm ID, 38.1-mm OD porous stone and 12.7-mm ID Saran tubing risers encased in 19.1-mm PVC pipe. Twelve of the points were encased in 101.6-mm diameter sand filters held in place by nylon mesh. Points were seated at various depths below the surface by hand pushing a 101.6-mm ID outer PVC pipe containing the tip and riser into the dredged material below the hardened crust lifting the outer pipe and leaving the riser, filter, and tip at the desired depth. Details of the installation are shown in Figure 8. Depths of installation and other pertinent data are summarized in Table 8.

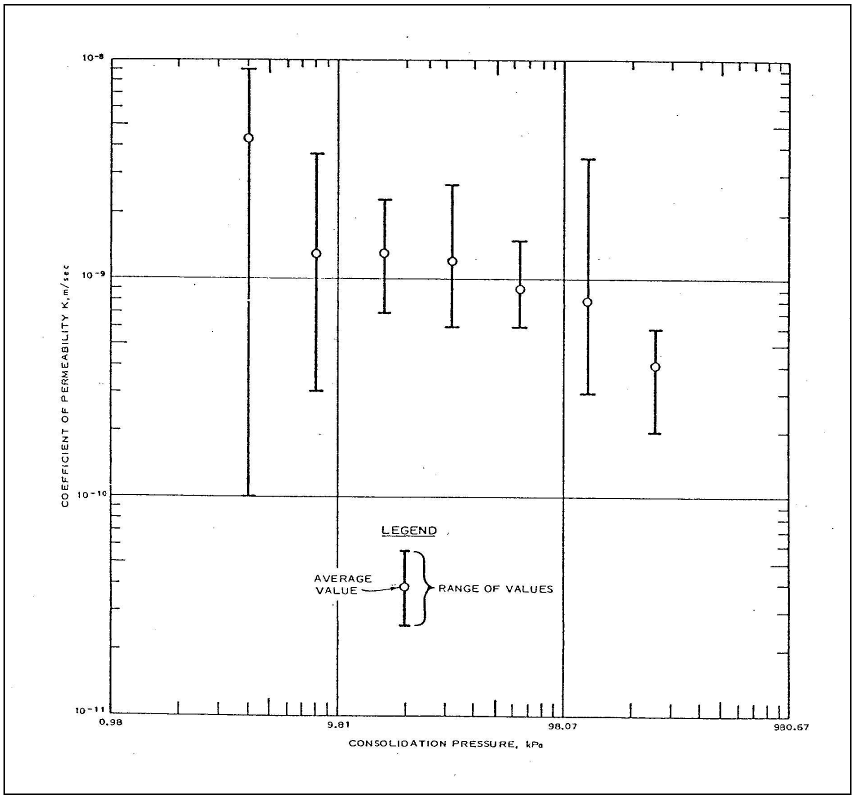

Figure 7. Relationship between consolidation pressure and coefficient of permeability for the Upper Polecat Bay disposal area, Mobile, AL

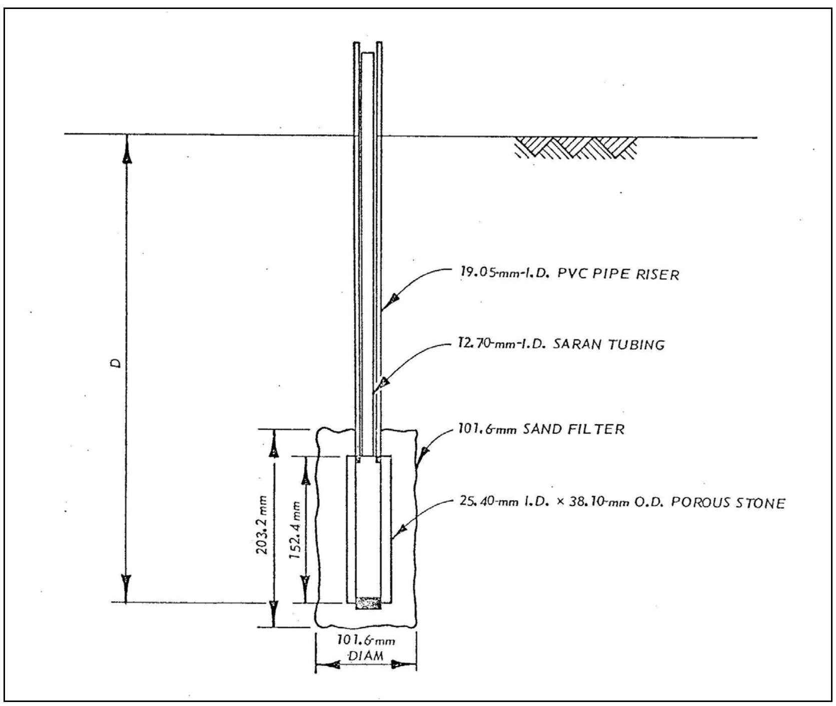

Figure 8. Detail of wellpoints used for variable head permeability tests

Table 8
Variable Head Field Permeability Test Results

Wellpoint	Tip Depth, m	Filter Surrounding Casagrande Tip	Type Test	Initial Head, m	Coefficient of Permeability, k, m/sec
C-1	2.32	sand	Falling Head Falling Head	0.26 0.23	7.7×10^{-7} 8.8×10^{-7}
C-2	1.52	sand	Rising Head Falling Head Falling Head	1.07 0.18 0.22	0.5×10^{-7} 10.3×10^{-7} 20.5×10^{-7}
C-3	0.91	sand	Falling Head Falling Head	0.15 0.39	1.5×10^{-7} 12.3×10^{-7}
C-4	2.44	sand	none	--	--
C-5	0.98	sand	Rising Head	0.54	1.0×10^{-7}
C-6	1.52	sand	Rising Head	1.33	0.4×10^{-7}
C-7	2.59	sand	Falling Head Falling Head	0.15 0.33	20.5×10^{-7} 61.5×10^{-7}
C-8	2.13	sand	Rising Head Falling Head	0.58 0.15	3.8×10^{-7} 61.5×10^{-7}
C-9	1.52	sand	Rising Head Falling Head	0.67 0.10	1.4×10^{-7} 41.0×10^{-7}
C-10	2.53	sand	none	--	--
C-11	2.10	sand	Rising Head	0.69	2.6×10^{-7}
C-12	1.52	sand	Rising Head	0.13	7.7×10^{-7}
T-1	2.44	none	none	--	--
T-2	2.41	none	none	--	--

Ten falling head tests and seven rising head tests were conducted on six of the wellpoints immediately after installation. Permeabilities were computed using the basic time lag procedure developed by the WES (Hvorslev 1951). Results of the rising head tests indicated an average k of 2.5×10^{-7} m/sec, significantly higher than values of about 1×10^{-9} m/sec indirectly determined from consolidation test data. The mass permeability of the dredged material is significantly higher than that indicated by laboratory size specimens from the presence of organic material, wood chips, silt and sand lenses, etc. The rising head tests also indicated higher permeability at the southernmost wellpoints C-8, C-9, and C-12.

Falling head test results indicated an average k of 2.5×10^{-6} m/sec, but the head could not be raised above 0.39 m. Relationship of head ratios with time also appeared non-linear, suggesting that hydraulic fracturing of the dredged material might have occurred. Other research confirmed that hydraulic fracturing does occur in the UPB dredged material with heads of approximately 0.3 m (see Chapter 7). More reliance should therefore be place on rising head test results.

Clay Mineralogy of Dredged Material

A petrographic analysis of six CH dredged material samples was performed by the WES Concrete Laboratory. X-ray diffraction methods were used to determine the mineralogical composition with special emphasis on clay mineralogy and clay content. Four qualitative and two quantitative analyses were performed with results presented in Tables 9 and 10. The samples were generally composed of montmorillonitic and chloritic clay, clay mica, quartz, and traces of other non-clay minerals. Organic content, as determined by ignition loss, was 5 percent.

Table 9
Qualitative Mineral Composition of Dredged Material[1]

	Samples					
Constituents[2]	**CL-7 SS-1 BI-1 No. 33**	**CL-7 SS-2 BI-6 No. 3**	**CL-7 SS-3 BI-2 No. 2**	**CL-7 SS-4 BI-13 No. 2**	**CL-7 SS-5 BI-16 No. 2**	**CL-7 SS-6 BI-19 No. 2**
Clays						
Montmorillonite	C	C	C	C	C	C
Chlorite	C	C	C	C	C	C
Clay-mica	M	M	R	M	M	M
Kaolinite	R	R	R	R	R	R
Nonclays						
Quartz	I	I	I	I	I	I
Potassium Feldspar	R	R	N.D.[3]	R	R	R
Plagioclase Feldspar	R	R	R	R	R	R
Halite	R	R	R	R	R	R
Hematite	N.D.	N.D.	R	M	N.D.	R
Calcite	N.D	N.D.	R	R	N.D.	N.D.

[1] Determined by X-ray diffraction.
[2] Relative amounts are indicated in the table. Intermediate (I) 25-50 percent; Common (C), 10-25 percent; Minor (M), 5-10 percednt; Rare (R), <5 percent.
[3] Not detected.

Table 10
Quantitative Mineral Composition of Dredged Material

Constituents	Samples[1]	
	CL-7 SS-1 (BI-1 No. 3)	**CL-7 SS-2 (BI-6 No. 3)**
Clays		
Montmorillonite	25	25
Chlorite	25	20
Clay-Mica	10	10
Kaolinite	Trace	Trace
Subtotal	60	55
Nonclays		
Quartz	30	35
Feldspars		5
Calcite and Hematite	5	Present
Halite		
Organic Material	5	5
Subtotal	40	45
TOTAL	100	100

[1] Amounts of constituents given as percentages.

4 Progressive Trenching Field Demonstration

As a result of DMRP planning seminars (DMRP 1974), a study of European practice (d'Angremond et al. 1978), a literature survey of existing U.S. dewatering methods (Johnson et al. 1977), and laboratory studies (Brown and Thompson 1977), it was determined that a potentially effective and inexpensive method of dewatering fine-grained dredged material would be to promote good surface drainage in confined disposal areas, rapidly removing precipitation and allowing evaporative forces to gradually lower the internal water table and shrink the dredged material into a cracked crust from desiccation. Shrinkage forces during drying would return the material to normal soil form, and lowering the water table would increase the effective stress on material remaining below the water table with resulting consolidation.

It was decided to conduct a full-scale demonstration of this concept in the UPB disposal area using approximately the center 26 hectares (60 acres) of the site, as shown in Figure 2. In addition to concept evaluation, research was conducted on equipment needed to improve site interior drainage to determine its work ability and capacity on varying conditions of soil support.

Conceptual Basis for Experiment

As a result of literature review and background study, several concepts were postulated as inherent in any evaporative dewatering project:

a. Establishment of good surface drainage would allow evaporative forces to dry the dredged material from the surface downward, even at locations where a net evaporative deficit (total evaporation minus total precipitation) existed.

b. The best mechanism for removal of precipitation would be runoff through crust desiccation cracks to drainage trenches which lead off the site through outlet weirs.

c. To maintain effective drainage, the flowline elevation of surface drainage trenches must always be lower than the base of crust desiccation cracks, or ponding will occur in the cracks. As drying occurs and the water table falls, the cracks will become progressively deeper.

d. Below the desiccation crust, fine-grained material may be expected to exist at water contents near the liquid limit, and thus, it will be difficult to construct trenches much deeper than elevations corresponding to the bottom of the adjacent desiccation crust.

e. To continue to promote surface drainage as drying occurs and the cracks deepen, it will be necessary to periodically deepen drainage trenches as the water table falls and the surface crust becomes thicker.

f. From concepts *b* through *d* above, the elevation difference between the internal water table and the flowline of any drainage trenches will be relatively small. When the relatively low permeability of fine-grained dredged material is combined with the small hydraulic gradient likely to be available, it appears doubtful that appreciable water could be drained from the dredged material by gravity seepage. Thus, criteria for trench location and spacing should be based on site topography rather than a desire to achieve seepage drawdown.

The study was designed to evaluate the validity of these concepts and to determine the dewatering rates and surface subsidance achievable by improving surface drainage. The field study was conducted for a period of 14 months, from October 1975 through December 1976. More information concerning the basis for the concepts is available elsewhere (Palermo 1977a).

Field-Trenching Operations

The test program was designed to evaluate the overall effectiveness of a surface drainage system, as well as the trenching performance of both conventional and specialized equipment. Working conditions within a confined disposal area present unusual problems of mobility and operation of vehicles or excavating equipment (Willoughby 1978). The method of field operation for various types of equipment was thus changed as conditions warranted, and the entire operation remained flexible in order to accommodate the changing character of the dredged material and other influencing factors. Localized topography, thickness of surface crust, location of the dredged material water table, and presence of vegetative cover all contributed to the overall performance of the trenching equipment and trench-drainage system. A general plan view of the test site is presented in Figure 9. Individual trenches are labeled A through I to aid in identification. Spacings of 15, 46, and 76 m were included to evaluate effect of varied spacing. The north and south ends of the UPB disposal area were not trenched to conduct other dewatering experiments and also to act as potential control sections. Dredged material in the test site, described in Chapter 3, was a highly plastic clay (CH). Topography within the study area generally sloped downward from south to north, and the initial crust thickness varied from approximately 200 mm opposite the south weir to approximately 50 mm opposite the north weir. Beneath the surface crust, the dredged material was generally at water contents above the liquid limit. These initial crust and topography conditions greatly influenced the mechanics of the trenching operations.

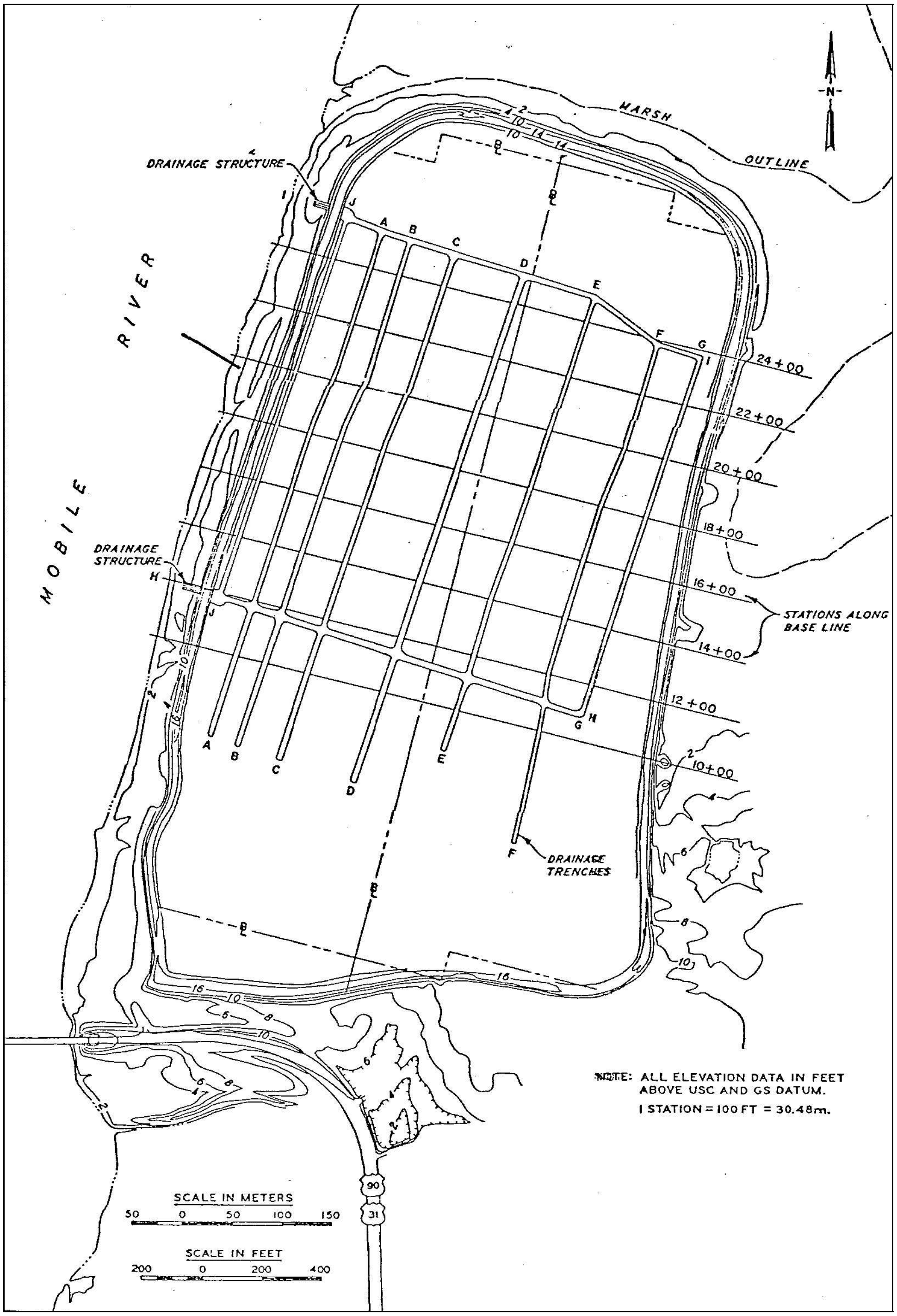

Figure 9. Plan of test site in Upper Polecat Bay disposal area showing trench network and reference coordinate system

Collector sumps were constructed at both the north and south weirs, as indicated in Figure 9, to gather water drained from the test site by the trenching system. Accumulated water was periodically pumped from the sumps. Pumping was required because weir inverts were at el 5.0 ft mlw[1] while sumps and flowlines of some trenches were eventually constructed to the original foundation level e1 2.0 ft mlw. Sump-pumping operations proved to be unsatisfactory, and, after 7 months of testing, a culvert was placed through the dike at the south weir, with the invert at e1 2.0 ft mlw to allow gravity flow to the Mobile River.

Initial trenching - north end of test site

The primary obstacle to trenching the north half of the test area was the inability of the existing 50-mm crust to support trenching equipment. Initial attempts to construct trenches in this area were made by the MDO Mobile Area Office using a contracted backhoe on mats. The contractor was confident he could trench the site, but he could not reduce the matted machine's approximately 14-kPa ground pressure, and it broke through the crust and sank about 10 m from the perimeter dike.

After the initial attempt, results of a DMRP preliminary study to determine suitable amphibious vehicles for use in thinly-crusted disposal area were scrutinized. Based on these data, the most suitable vehicle seemed to be the Riverine Utility Craft (RUC). The RUC is a prototype amphibious archamedian screw propulsion vehicle, originally built by Chrysler Corporation for the U.S. Navy as a high-speed 28- to 37-km/hr reconnisance craft for use in the riverine environment of the Mekong Delta of South Vietnam. General specifications for the craft are given in Figure 10. Twin styrofoam-filled rotors support the vehicle and provide flotation in water or on extremely soft ground. The rotors are fitted with double helical blades and propulsion is accomplished by rotation of the rotors in opposite directions. Lateral movement is possible by rotation of both rotors in the same direction. Forward or reverse movement of the vehicle of soft soil causes ruts to be formed by the twin archamedian screws. It was thought that these ruts might serve as effective drainage trenches.

After DMRP staff discussion, it was decided to acquire one of the RUCs from the U.S. Marine Corps (SUMC) at Parris Island, South Carolina, where they were currently used to traverse marsh around the USMC Parris Island Recruit Depot. The USMC subsequently loaned RUC Serial 0216-7 to the DMRP for evaluation as a trenching device in thinly-crusted disposal areas. Since the vehicle was designed for high-speed military reconnisance and troop/cargo transport, many vehicle features are not optimized for use in trenching. An aluminum alloy hull without frame was used, making it difficult to attach or tow trenching implements, plows, etc. Vehicle engress was limited by armor protection not needed in civilian use. Low gear was originally blocked out of RUC transmissions, making low-speed 4 to 7 km/hr trenching operations difficult

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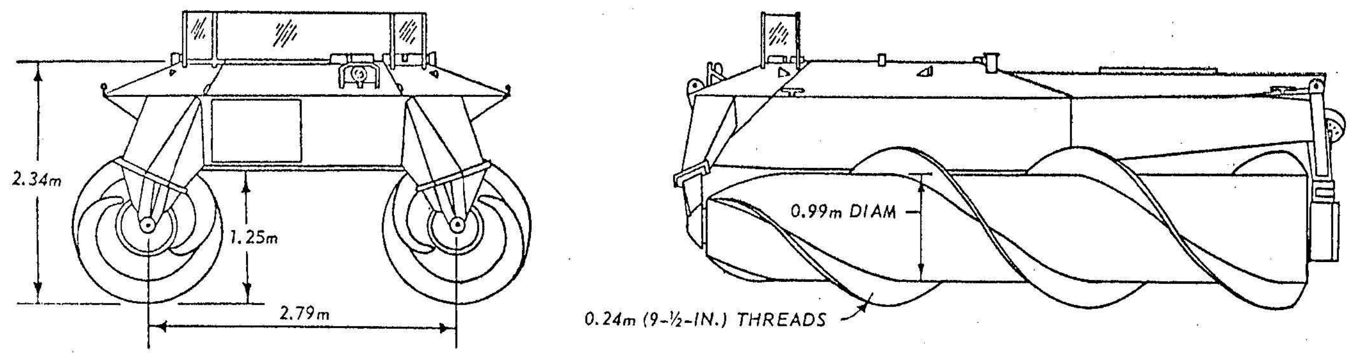
[1] All horizontal control reference and survey data presented in this Chapter are reported in units of stations, i.e., 1 station (sta) = 100.00 ft, and all vertical control reference and survey data are reported in units of feet, as these were the official horizontal and vertical control units of the U.S. Coast and Geodetic Survey during the test period. (1 sta = 30.48 m, 1 ft = 0.305 m).



RIVERINE UTILITY CRAFT

EMPTY WT (INCLUDING DRIVER & FUEL)	4990 kg	(11,000 LB)
GROSS WT (DRIVER, FUEL & PAYLOAD	5900 kg	(13,000 LB)
LENGTH (OVERALL)	6.25 m	(20.5 FT)
WIDTH (OVERALL)	4.27 m	(14 FT)
HEIGHT (OVERALL, LESS WINDSHIELD)	2.34 m	(7.67 FT)
ROTOR SPACING, CENTER TO CENTER	2.79 m	(110 IN.)
ROTOR DIAMETER (DRUM ONLY)	0.99 m	(39 IN.)
ROTOR DIAMETER (OVER HELIX)	1.47 m	(58 IN.)
ROTOR LENGTH (OVERALL)	5.64 m	(222 IN.)
ROTOR LENGTH (IN CONTACT WITH GROUND, NO RUT)	4.95 m	(195 IN.)
GROUND CLEARANCE	1.25 m	(49 IN.)
FLOATING DEPTH (EMPTY) (WATER)	0.55 m	(21.5 IN.)
FLOATING DEPTH (LOADED) (WATER)	0.61 m	(24 IN.)

Figure 10. Detail and general specifications for Riverine Utility Craft (RUC)

without overheating. Higher speed operation caused the RUC rotors to throw material back into their ruts. The 4.3-m width of the vehicle also caused complications in overland transport, because the legal width limit in most states is 3.7 m. The RUC rotors were dismounted and the vehicle tilted during transport to meet these requirements. Disadvantages not withstanding, with 567 kW available to power a 5,900 kg vehicle, adequate power was available to swim at high speed, turn rotors in stiff material, break through surface crust up to 0.3 m thick, and climb dikes for site access. The prototype RUC is thus valuable as trenching device and transportation vehicle in thinly-crusted disposal areas where no other vehicle can operate. Additional data concerning uses and performance of the RUC within disposal areas is available elsewhere (Willoughby 1978).

Initial RUC trenching

Initial trenching with the RUC at the UPB test site was performed during October 1975. Trenching performance was evaluated on various crust thicknesses between 50 and 200 mm, from approximate sta 10+00 to sta 22+00 (Figure 9). Trenches A through I, located as shown in Figure 9, were constructed in this phase. RUC trenching performance in thinly (50 mm) crusted areas, from approximately sta 16+00 to sta 22+00, was good, with trenches constructed 100 to 200 mm deep. As crust thickness exceeded 200 mm, trenches 0.3 to 0.5 m deep were formed after multiple passes, but the operation caused noticeable mechanical strain on the RUC drivetrain components. Based on the initial evaluation, RUC trenching operations were restricted to thinly crusted areas north of sta 16+00 until better determination of the RUC drivetrain reliability could be established.

The depth of RUC-constructed trenches varied with fine-grained material crust thickness and consistency. In crust of 50 mm or less, underlain by very wet material at or above the liquid limit, ruts were wallowed into the wet material, which tended to flow back into and fill the ruts after the vehicles had passed, giving two shallow depressions rather than distinct ruts. Optimum trenching occurred in crust 100 to 200 mm thick, with a cone penetration index (CI) of approximately 20. The CI is a WES-developed indicator of vehicle trafficability and soil support capacity. A CI of 20 is just sufficient to allow a man to walk on the surface. The most effective trenching method in this environment proved to be two passes, the first to break the initially-formed crust and throw it to the edge of the trench, and the second to smooth and clear the trench forming a semi-circular depression. More than two passes remold the underlying material, causing flow and a decrease in trench depth. In stiffer material (CI > 20) and with initial crust thickness greater than 200 mm, an incomplete trench is produced, and repeated passes of the vehicle are needed to form continuous trenches. Once the crust thickness exceeds 300 to 400 mm and the crust CI exceeds 50, the RUC rotors tend to ride on top of the crust without breaking through. Unless a hole is wallowed through the crust to allow the vehicle to first settle into the underlying very wet material, trenching is ineffective.

Because the depth of RUC trenches is controlled by crust thickness and material consistency, uniform depth will be produced in most areas where fine-grained material is deposited. Trench flowlines thus tend to follow the natural contours of the filled disposal area, which usually slopes gradually from the dredge pipe location to outlet weirs. Because the vehicle tends to float in the subcrust, it is difficult to establish a continuous grade when harder layers exist below the crust (sand layers, old interior dikes, etc.). Making repeated passes is not always effective, because the RUC simply rides up over the hard spot. The most effective procedure is to stop and reverse through the hard spot. The blunt rear end of the rotors will tend to gouge out the hard material, and going forward again smooths the trenches. This difficulty in grading RUC trenches presented some operational problems, because sufficient grade could not be established to initially drain a few isolated low areas at the north end of the test site. A particularly troublesome spot existed immediately adjacent to the north weir, at the intersection of trenches A and I (Figure 9). An underlying sand lens about 2.5 m wide, immediately in front of the weir, prevented proper grading and drainage from the RUC feeder trench leading to the weir sump. Repeated backward and forward RUC passes to deepen the trench were successful only for short periods, as the underlying sand material did not subside at the same rate as surrounding fine-grained material. Thus, after 1 or 2 weeks of effective drainage, the trench flowline in the hard area would be higher than the rest of the feeder trench flowline. When surrounding crust became thick enough to support a marsh dragline, the entire plug was removed. A backhoe attachment fitted to the RUC would have solved the problem much earlier.

Intersecting RUC trenches also caused minor problems as the vehicle tended to seal the earlier set of trenches when crossing a perpendicular trench. As a result, hand labor was used to reopen the trenches. The Dutch also reported similar problems in their use of the Amphirol, a small, underpowered vehicle similar to the RUC (Haliburton et al. 1978). Subsequent RUC trenching operations at other locations used radial trenches from a sump at each weir in order to eliminate the problem.

Design and use of implements

During December 1975 and January 1976, an extensive overhaul of the RUC was performed by the WES MESL, to increase general reliability and overall effectiveness of the RUC in trenching. The vehicle was also fitted a with a rear-mounted A-frame, winch, and fittings for towing various trenching implements.

The first implements designed by MESL were modeled after those successfully used in European practice. The basic configuration consisted of hollow wheels, 1.2 m in diameter and tapered in a vee pattern, as shown in Figure 11. The wheels were designed to grade and deepen the trenches by pushing crust and wet material into the sides of the semi-circular rotor rut, and they could be water-ballasted for deeper trenching capability in stiffer materials. RUC trenching operations with the wheel implements were performed at the UPB test site during February 1976, and as in previous RUC operations, the efficiency of trenched construction with the wheel implements was greatly dependent upon the crust thickness and the consistency of subcrust dredged material. The vee formed by the wheels remained open in material with water contents below the liquid limit,

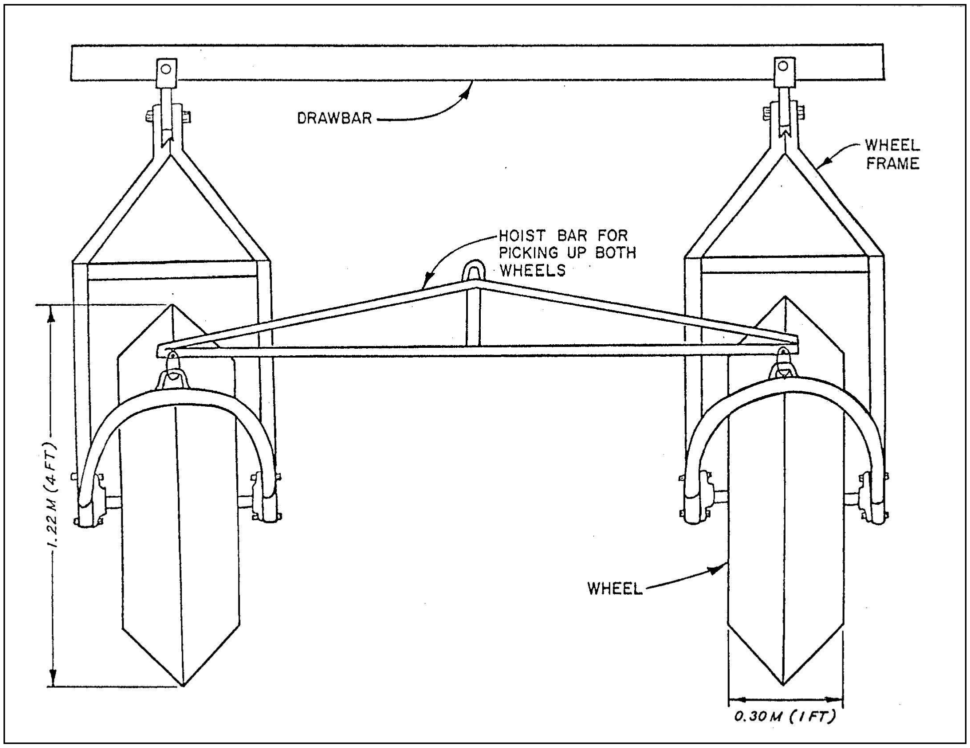


Figure 11. Detail of towed wheeled implements evaluated in Riverine Utility Craft (RUC) trenching tests

at times reaching a depth of 0.3 m below the rotor depth. In situations where the water content of the material below the crust was above the liquid limit, the wheels tended to float when empty, and did not deepen the trench. When ballasted with water, the wheels sank into the underlying material and pushed a large passive wedge of material before them, requiring full RUC power to pull the wheels. Full power application gave the RUC a nose-up rear-down attitude, causing the wheels to sink still deeper. In such instances, it was then necessary to stop and winch the wheels up out of the material against considerable suction. This trenching procedure was quickly abandoned.

Generally, the wheel implements did not significantly aid in deepening the RUC trenches. The winching system also proved to have limitations in controlling the elevation of the wheels relative to the rotors and did not allow effective trench grading. A better design would employ outrigger wheels to control depth and be pulled by cable or pinned towbar so that the implements would not be affected when excessive pull forced the RUC into a rear-down attitude. Other implements, including large discs and plows, were evaluated at the UPB test site in later trenching operations. These implements generally had the same limitations as the wheels, and no depth control was possible in very wet underlying material. In addition, towing force requirements in stiffer material exceeded the 1,800- to 2,300-kg drawbar of the RUC. The initial design purpose of the RUC proved to be the limiting factor in the use of implements for trenching. Absence of a heavy-duty frame for towing prevented the use of all available vehicle power. Also, fittings and bracing employed in the existing

RUC chassis did not provide the flexibility needed for trenching operations using directly-attached implements. Additional data regarding use and evaluation of trenching implements may be found elsewhere (Willoughby 1978).

Progressive RUC trenching

Experience gained during the initial phases of trenching indicated that limited periodic deepening of RUC trenches was possible, provided that a sufficient period was allowed for the dredged material in the bottom of the trench to dry between vehicle passes. This knowledge let to a tailoring of RUC trenching operations toward a progressive deepening approach during March 1976 through June 1976. Since the use of ballasted wheels did not significantly aid in trench deepening, these operations were performed with rotors only. A scheme was devised for RUC movement within the test site which utilized a looping pattern, resulting in only two vehicle passes through each trench. One month was allowed between successive RUC trenching operations. Desiccation and drying of the exposed dredged material within RUC trenches took place during this period. When retrenching was performed, the first pass down the trench tended to throw out blocks of dried material in a manner similar to that experienced when virgin crust was broken, exposing wetter underlying material. Re-trenching caused the trench to be deepened by an amount approximately 50 mm to 150 mm. The second pass cleaned out and smoothed the ditch bottoms in all cases. As the loop pattern was formed, trench intersections were cleared by hand.

Progressive RUC trenching allowed the material to dry extensively, and the trenches reached a depth of approximately 500 mm by April 1976. The area trenched by the RUC was reduced to Trenches A and B in May 1976 in order to allow deeper excavation of all other trenches north of sta 16+00, with a marsh dragline. Drainage and RUC deepening of Trenches A and B continued until the crust thickness began to inhibit RUC operation, and it was discontinued in June 1976.

Initial trenching - south end of test site

Trenching techniques have been successfully used around the perimeter of disposal areas by several CE Districts to promote localized water-table drawdown and drying of fine-grained dredged material. These operations are performed with draglines operating from the dikes or on mats immediately adjacent to the dikes. The dewatered material is used to raise the dike elevation. Similar trenching concepts were evaluated at the UPB test site with the trenching extended to the disposal area interior.

Amphibious or marsh draglines constructed by Quality Marsh Equipment Company have been used successfully for excavation in marshy areas and in water. These machines consist of a small-to-medium-size dragline placed on a chassis with twin flotation pontoons covered by a set of wide chain-driven tracks with open growsers. This system enables the machine to swim in open water, and its low ground pressure (17 kPa or less) and growser design allow it to track on soft material. These machines are better suited for very soft ground operations within dredged material disposal areas than conventional draglines of similar size. However, some limitations exist. Marshy areas have surface soils as soft as those in dredged material disposal areas, but the strength of natural marsh soils tends to increase with depth, and a well-developed root mat is usually present. After sinking some distance into the marsh, the track growsers of the marsh dragline can achieve sufficient traction for movement. Within underwatered fine-grained dredged material disposal areas, no root mat may exist, and wetter and softer dredged material lies beneath the surface crust. Further, marsh draglines are usually underpowered, capable of only very slow speed, and low ground-pressure considerations limit boom length and counter-weight size resulting in a relatively small bucket.

A marsh dragline was obtained by contract through the MDO in October 1975 for evaluation and use in trenching operations at the test site. The machine consisted of a Bantam Model 350 dragline mounted on pontoon chassis with 0.76-m wide tracks. The dragline was fitted with a 12-m boom and 0.3-cu m bucket. The dragline tracked (single pass) satisfactorily over the southern part of the test site. However, as the machine tracked into an area of 100-mm crust thickness in the north part of the test site, it broke through the crust and was immobilized. The consistency of dredged material beneath the crust was too viscous for effective swimming action and too liquid for effective tracking. Wet dredged material became packed in the track growsers, and the machine was unable to climb back up on the crust. The marsh dragline was subsequently restricted to areas in the test site south of sta 16+00, which had a surface crust thicker than 150 mm. A crust thickness of 150 mm or more was found to be needed to allow the machine to remain stationary and swing its boom when digging.

The marsh dragline constructed the initial trench system from approximately sta 10+00 to sta 16+00 (Figure 9) during October and November 1976. The dragline constructed the network by straddling the staked trench centerline trailing the boom, digging, and casting the excavated material to either side of the trench in broken windrows, which were then flattened with the bucket. Approximately 2,042 m (6,700 ft) of trench was constructed in 200 operating hours. The trenches were constructed with bottom widths 0.5 to 0.6 m, 1H:1V side slopes, and 0.9 to 1.8-m depths. Bank caving and sloughing occurred in localized areas but did not significantly affect drainage efficiency of the trenches. Lateral trenches A through F were dug to depths of approximately 0.9 m near sta 16+00, sloping downward to depths of 1.8 m near sta 12+00. Feeder trench H was dug 1.8 m deep and was sloped downward toward the south sump and weir. Attempts to dig deeper trenches were unsuccessful, because subcrust material flowed laterally overnight, filling the trench bottom. The trench flow lines were practically level at approximate e1 4.0 to 5.0 ft mlw with increasing trench depth resulting from increasing surface elevation toward the south.

During excavation, trenches quickly filled with water which drained from higher permeability lenses throughout the dredged material. After the network was completed and water was allowed to drain to the sumps, the trenches began to dry and desiccate. Similar behavior was observed in the excavated material, which exhibited significant shrinkage upon drying.

Second general site trenching

The northern portion of the test site had been initially trenched, and the trenches progressively deepened with the RUC during October 1975 to May 1976, as described previously. During this period, crust thickness had increased to 150 mm or more over this part of the test site. A second marsh dragline was procured by MDO contract to deepen this part of the trench network. This machine was similar in appearance to the first marsh dragline used at the site, but had 1.5-m wide pontoons and tracks allowing larger counterweights and a 0.6-cu m bucket with a 12-m boom. The machine had little difficulty tracking within the test site. When the machine occasionally broke through thinly crusted (150- to 200-mm) areas at the far northern part of the test area, empty 209-L oil drums were chained to the tracks, and the machine then walked back up onto the crust and continued work.

A majority of the existing trench system between sta 16+00 and sta 24+00 was deepened with this dragline during May 1976 to July 1976. Trenches A and B were not deepened in order to allow continued evaluation of progressive RUC trenching. Two passes of the machine were used to deepen the other trenches. On the first pass, the dragline trailed the boom sitting on one side of the existing RUC trenches and removed the dried material from between the two RUC trenches. The excavated material was spread in approximately 0.3-m-thick flattened broken windrows adjacent to the trench. On the second pass, the machine deepened the entire trench and spread this material operating from the opposite side of the trench. By placing the excavated material on both sides of the trench, thin layers resulted with drying and shrinkage cracks forming rapidly down to the original dredged material ground surface allowing unimpeded surface drainage into the trenches. Approximately 1,650 m of trench was deepened in 320 operating hours. A new trench was also constructed parallel to the west retaining dike, labeled J on Figure 9. This trench of approximately 425 m was constructed in one pass using 40 operating hours.

Some difficulty was experienced in keeping the deepened trenches open in the north part of the test site. Trenches C through F were dug an average of 0.9-m deep on the first pass, but lateral flow of subcrust dredged material back into the trench caused depths to be reduced to approximately 0.5 m. The second pass held these trenches open to an average depth of 0.8. Trench I at the north edge of the test site proved most difficult to keep open, and after two passes the stabilized depth of trench I was an average of only 0.5 m. During this deepening operation, trench banks cut into subcrust dredged material exhibited shallow sloughs or slides. These slides tended to occur over a period of several hours after the trench was deepened. Deepening was more effective toward the southern portion of the test site, and the deepened trenches were connected to the initial marsh dragline-made trenches near sta 16+00. Trench J, constructed parallel to the west dike, maintained a depth of 2.4 to 3.1 m, exposing sandy material from the dike foundation in several places. This trench connected the north and south sumps and allowed gravity flow from feeder Trench I to the culvert adjacent to the south sump and weir.

The trenching network from sta 10+00 to sta 16+00 in the south part of the test site had initially been constructed by marsh dragline during October and November 1975. Efficient surface drainage following this trenching promoted drying and formation of surface crust 200 to 600 mm thick. A conventional dragline operating on mats was used to clean out and deepen portions of this part of the trench network during April 1976. The machine used was a trackmounted Bucyrus-Erie Model 15B with 10.7-m boom and 0.5-cu m bucket. Single mats 0.9 by 6.1 m were used to support the dragline. Lateral Trenches A through G and feeder Trench H were deepened up to 0.9 m below existing depth from sta 12+00 to sta 16+00. Dried dredged material previously excavated and spread adjacent to the trenches by the marsh dragline was leveled by the dragline, and the mats were placed on this material to further reduce pressure on the subcrust material. When crossing trench network, trenches were filled temporarily with dry material. Approximately 460 m of trench was deepened in 122 operating hours. Trench depths of up to 3.7 m, near original foundation elevation, were reached adjacent to the south sump and weir enabling efficient grading of the entire trenching system toward the south sump and culvert. Subsequent improved drainage resulted in additional drying even during the relatively wet winter of 1976-77.

Summary of trenching methodology

Considering dredged material properties and the specific trenching abilities of the RUC, marsh dragline, and conventional dragline, a progressive deepening approach to trenching was utilized at the test site. Each piece of equipment had unique capabilities which, together, contributed to the construction of an adequate surface drainage system within the disposal area. The operational limitations of each item of equipment were adequately overcome by the others.

The RUC proved to be the only equipment item capable of initiating work in thinly-crusted (less than 150 mm) areas within the test site. RUC trenches were highly successful in draining surface water from the north part of the test area allowing evaporative drying to thicken the crust. Progressive deepening of the trenches by the RUC was achieved once a satisfactory operating procedure was developed, including a traffic pattern which limited passes of the vehicle in any one trench. Also, time periods which allowed drying and desiccation in the trench bottoms between successive operations were determined, and hand grading procedures were established to allow effective surface drainage at trench intersections. RUC trenching operations promoted growth of crust thickness from an initial value of 50 mm or less to 150 to 500 mm, thus allowing a marsh dragline to operate successfully in the wetter north part of the site.

The amphibious or marsh draglines proved effective in digging, grading, and deepening trenches when at least 150 mm of crust was available on which to work. As with the RUC, a progressive approach was successful in deepening the trenches. Trenches were excavated to depths of 0.5 to 0.8 m in the wetter northern portion of the test site and depths of 0.9 to 1.8 m in the dryer southern portion. The thicker surface crust which, developed in the southern part of the test site, eventually allowed the use of a small conventional dragline operating on single mats. The mats were placed on dried material previously excavated by the marsh dragline, and trench depths of 1.8 to 3.6 m were achieved. The progressive trenching approach, utilizing the RUC, the marsh dragline, and the conventional dragline allowed the surface drainage system to be successfully deepened and its efficiency to be improved, thereby promoting effective drainage within the disposal area. More data on RUC and dragline operations, including expected production capacity, alternate trenching schemes, and other equipment for trenching in confined disposal areas, are available elsewhere (Haliburton et al. 1978, Willoughby 1978).

Results of Field Trenching

Initial plans for conduct of the trenching study called for improvement of drainage in only the center portion of the UPB area. The untrenched north and south ends of the disposal area were to be used, among other things, as control to assess effect of the trenching program. However, the entire disposal area, including the north and south ends, was effected by the trenching study so much that direct comparison of results with the behavior of a control section is impossible. For comparative purposes then, it should be noted that minimal drying and crust formation occurred in the 2 years between use of the area for disposal and the initiation of trenching operations.

Water table drawdown

Open riser observation wells were installed in 24 borehole locations within the UPB disposal area in July and August 1975, as described in Chapter 3, were used to monitor changes in the dredged material water table. Thirteen of the observation wells survived all field trenching operations without damage. Trends of drawdown were practically identical for all well locations showing a drawdown of approximately 0.46 m over a period of 15 months. Piezometer locations between the trenches did not allow accurate evaluation of localized drawdown conditions near the trenches, so no well-defined drawdown curve between trenches was evident from available data. These data indicated gravity seepage had little influence on total dredged material water loss with drawdowns primarily the result of water loss through evaporation. The average drawdown-versus-time relationship for all observation wells within the progressive trenching study area is presented in Figure 12, as are precipitation data for the time period. Dredged material water table elevation versus time data for individual observation wells are presented elsewhere (Palermo 1977b).

Measured settlement

Dredged material surface settlement was determined through a series of cross-section surveys conducted by the Survey Branch, Operations Division, MDO. An initial survey was made in July 1975 prior to trenching. Subsequent surveys were made periodically during the test. Eight cross-section survey ranges were established on two-station centers, from sta 10+00 to sta 24+00. Because of some difficulty by survey personnel in finding the same elevation measurement points in succeeding surveys, location stakes were placed at 3.05-m intervals along the ranges.

Average elevations of the dredged material surface were determined for each survey at each station. These data are shown in Figure 13 and show almost constant settlement at all stations except sta 10+00 and sta 12+00, for which

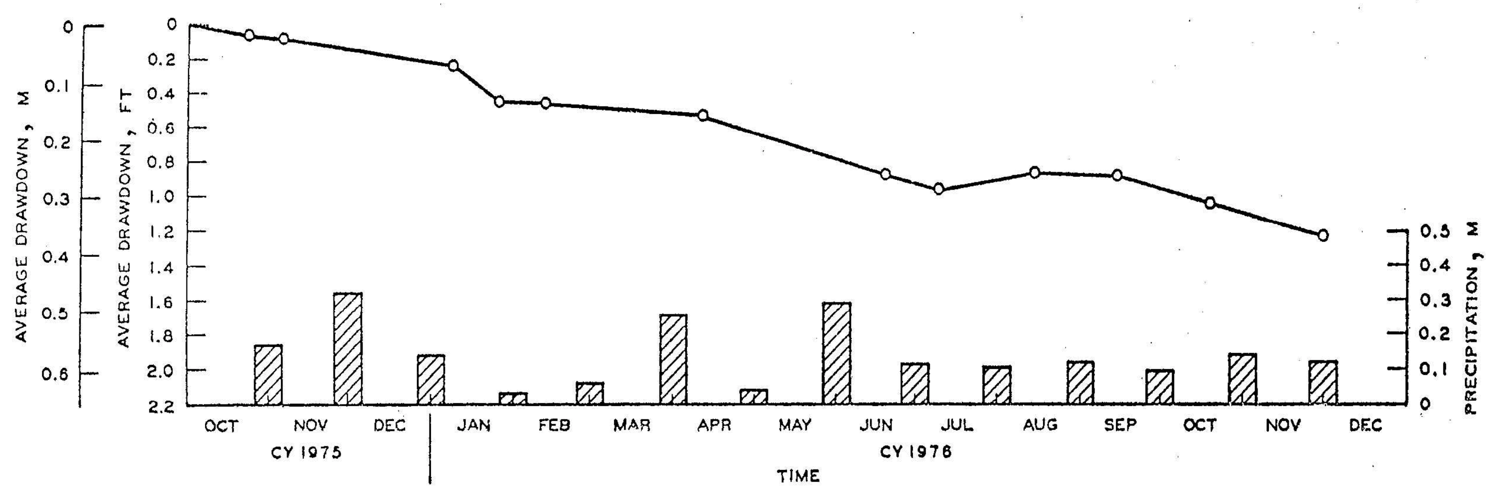


Figure 12. Average water table drawdown for trenching test site versus time

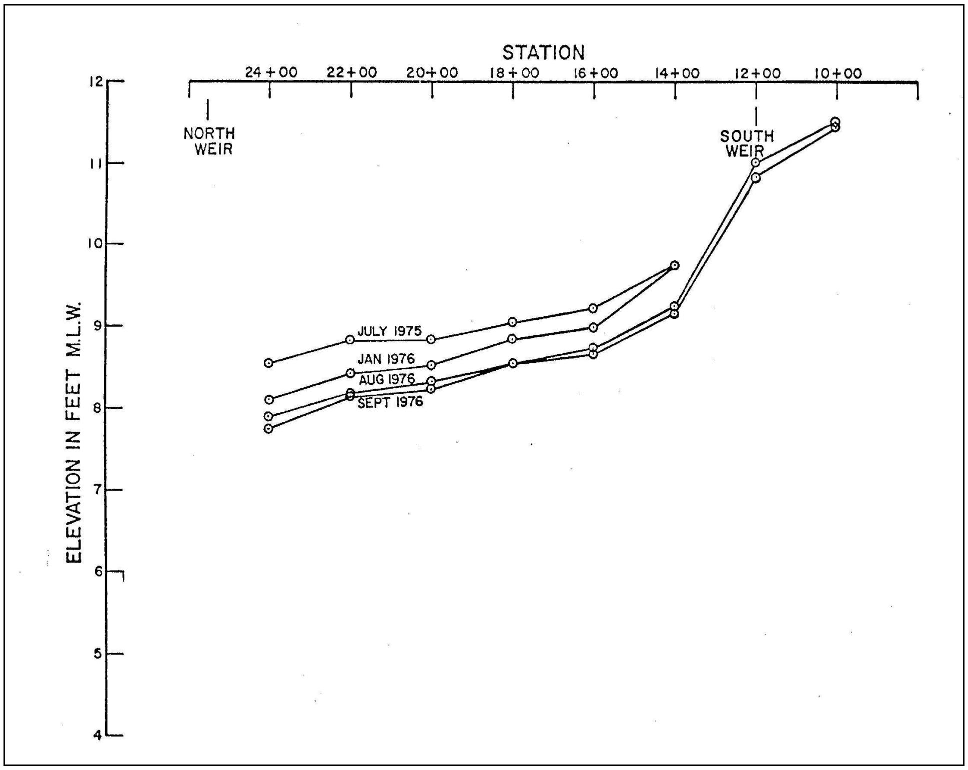


Figure 13. Average dredged material surface elevation at each cross-section survey range as a function of time

settlements are smaller. The surface crust was initially thicker in this vicinity because of disposal operations described in Chapter 2. Actual cross-section data are available elsewhere (Palermo 1977b). Average total settlement versus time was determined considering data from all Stations and is shown in Figure 14 indicating an approximately constant rate of settlement with time. Data from Figures 12 and 14 were used to determine a measured settlement versus drawdown relationship. The field data indicated measured settlements of approximately 0.6 times the magnitude of drawdown.

Predictive Analyses

Predictive analyses were conducted to determine potential surface settlement of the test site from consolidation and desiccation shrinkage. Data accumulated from field and laboratory investigations described in Chapter 3 were used to perform the analyses which were then compared with field results.

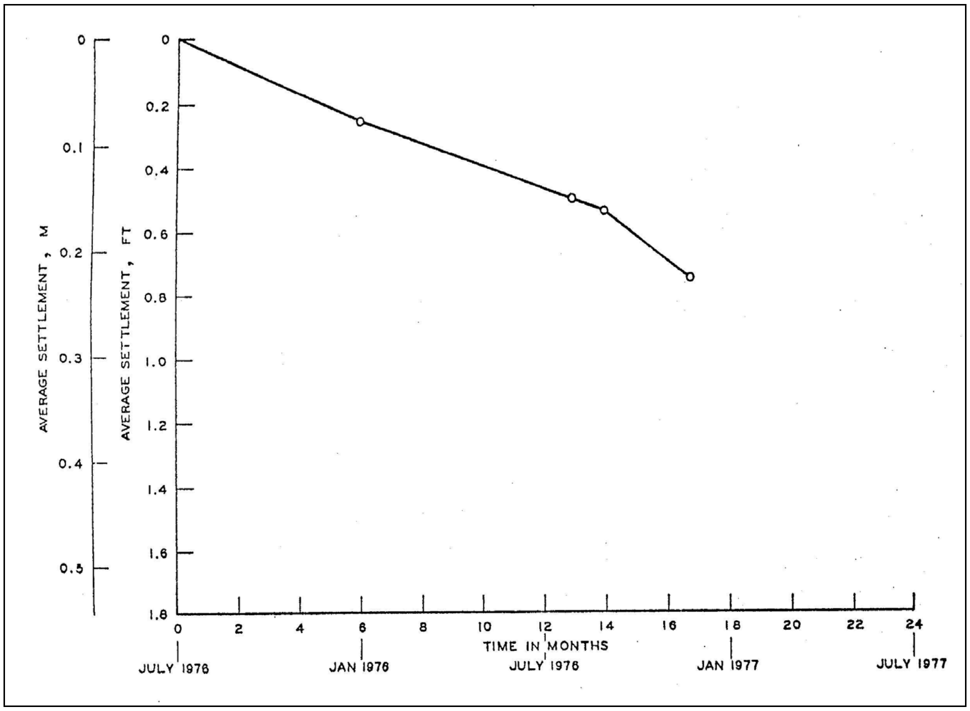


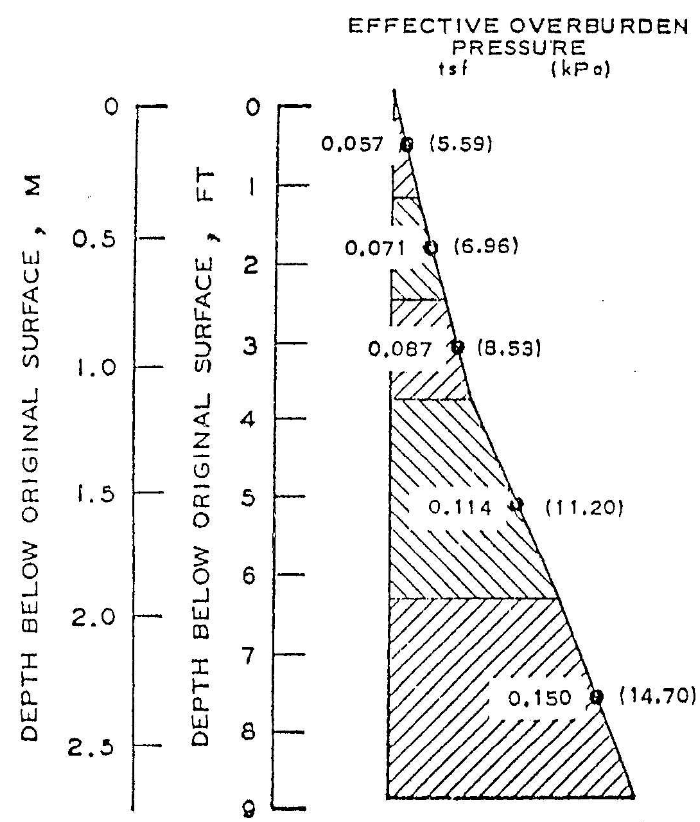
Figure 14. Average settlement of test site ground surface versus time

Potential dredged material consolidation

Results of consolidation tests performed on undisturbed samples were used to predict potential dredged material consolidation from water-table drawdown. The analysis assumed drawdown would increase the effective stress on underlying dredged material by removing the bouyant force provided by the water. One-dimensional consolidation theory was then used to compute expected settlements. Parameters used in the settlement analysis were determined by averaging numerous consolidation test results to obtain average values for individual strata as shown in Figure 15. The initial dredged material ground-water table was at an average depth of 0.378 m below the surface. The initial effective stress plot for this condition, shown in Figure 15, was determined using saturated unit weights above the initial water table and submerged unit weights below the water table. Values for unit weights were determined from consolidation test specimens.

Data presented in Chapter 3 indicated that the dredged material below about 0.3 m was normally consolidated, thus settlements for each stratum indicated in Figure 15 were computed using the well-known relationship:

$$\Delta H = H \frac{C_c}{1 + e_1} \log \frac{P_2}{P_1} \qquad (1)$$



H FT	H (m)	e_1	C_c	PI	C_s
1.25	(0.38)	3.45	1.20	83	3.35
1.25	(0.38)	2.92	1.01	53	2.06
1.25	(0.38)	2.83	1.01	53	2.06
2.50	(0.76)	2.30	0.78	52	2.12
2.50	(0.76)	2.17	0.75	52	2.26

Figure 15. Stress conditions and data used in consolidation calculations

where

ΔH = settlement of the stratum under consideration

H = thickness of stratum

C_c = compression Index, average of all tests for respective strata

e_1 = initial void ratio at pressure P_1

P_2 = final pressure at center of stratum

P_1 = initial pressure at center of stratum

Values for the initial field void ratio e_1 were determined from virgin consolidation curves, extending them to the P_1 pressure if necessary. The total settlement for a given drawdown was determined by adding the contribution of each stratum. Total settlements computed for drawdowns of 0.381 m (1.25 ft), 0.762 m (2.50 ft), and 1.143 m (3.75 ft) are shown in Figure 16.

Laboratory consolidation test data indicated a permeability of approximately 10^{-9} m/sec for the fine-grained dredged material while variable head field tests described in Chapter 3 indicated a value of approximately 10^{-7} m/sec. This two-orders-of-magnitude difference is typical of laboratory versus field results. Field values were believed to more nearly represent actual conditions and were used to compute an average Coefficient of Consolidation c_v for time-settlement analysis using the well-known relationship:

$$t = \frac{H^2 T}{c_v} \tag{2}$$

where

t = time required for consolidation

H = drainage thickness of stratum

T = time Factor, constant for various degrees of consolidation

c_v = coefficient of Consolidation

The time relationship using variable head field permeability data gave c_v values of approximately 3×10^{-5} sq m/sec indicating that dredged material would reach 90 percent of primary consolidation approximately 2 months after stress imposition. Despite the well-known low reliability of Equation 2 in predicting accurate field behavior, it may be concluded that very rapid field consolidation will occur.

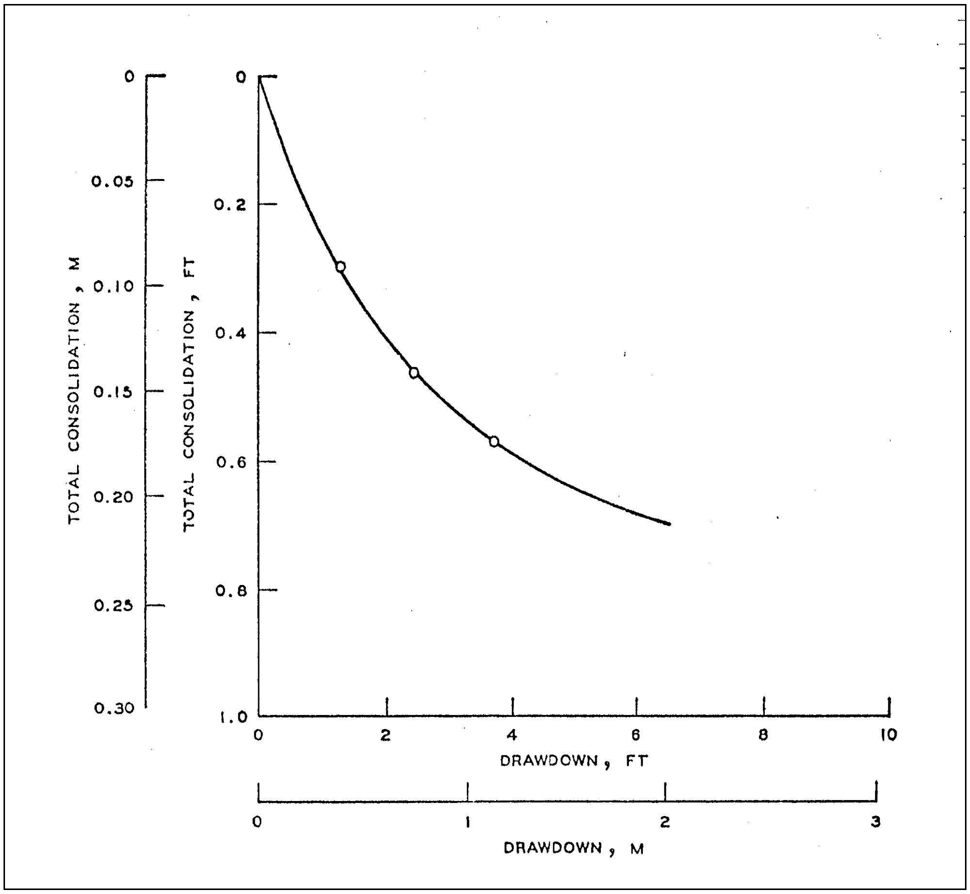


Figure 16. Computed below-water-table consolidation versus water-table drawdown

Potential foundation consolidation

Foundation soils at the test site consist of compressible layers of stratified highly plastic organic (OH) and inorganic silts and clays (CH - MH) overlying fine silty sand (SM) which is found at a depth of approximately 15 m below the original marsh surface. Detailed information concerning foundation conditions is presented in Chapter 3 and elsewhere (Palermo 1977a). The potential for additional consolidation of foundation soils from water-table drawdowns within the dredged material is dependent upon the interaction of the dredged material water table with that of the foundation. It is highly probable that, for the compressible foundation soils, disposal of fine-grained dredged material slurry would impose an immediate increase in foundation-effective stress equal to the total weight of the soil and water contained in it causing rapid foundation consolidation immediately after disposal. However, no data were available concerning whether or not piezometric stresses had stabilized during the period from the last disposal until the initiation of the trenching study. If the foundation water table was continuous up into the disposal area, further foundation consolidation might occur with drawdown from the increase in effective stress. However, if the water table in the disposal area was not continuous with that in the foundation (perched), drawdown would cause a decrease in effective foundation stress by decreasing the super imposed load.

Consolidation tests performed on undisturbed samples of the compressible foundation soils, taken both before and after placement of dredged material within the containment area, indicated that approximately 90 percent of foundation primary consolidation had taken place prior to the initiation of the trenching study (Palermo 1977a).

Piezometers were installed in both foundation soils and dredged material to determine possible interaction of the respective water tables in the test area. Data from the piezometers indicated that the water table in the test site was perched because the foundation water table corresponded with the mean water level in the adjacent Mobile River. Water-table drawdown within the dredged material would not increase the effective stress on foundation soils under these conditions. Further consolidation of foundation soils induced by progressive trenching was therefore assumed not to occur. Also, because of the initial high compressibility and low strength of cohesive foundation materials, any rebound from unloading by water-table drawdown in the dredged material was considered negligible.

Potential dredged material shrinkage

Laboratory test described in Chapter 3 indicated that large amounts of shrinkage and densification of dredged material would occur from evaporative drying following a drawdown of the water table. These data are further substantiated by other DMRP research on crust formation and behavior for fine-grained dredged material (Brown and Thompson 1977), which determined that crust formation occurs down to the base of the water table. In tests described in Chapter 3, the volume of desiccation shrinkage was equal to the volume of water lost.

After surface drainage is improved, water-table drawdown will occur slowly as was shown in Figure 12. Drying and shrinkage of the dredged material will keep pace with or lag slightly behind the drawdown, eventually forming a crust down to the lowered water table. As drying progresses, the water content of the material will be reduced from a pre-crust value of about the liquid limit to some lower value as the material is transformed into crust. The extent of drying and water-content reduction during crust formation is controlled by a great many factors, including climatic conditions, soil plasticity, clay mineralogy, initial water content, water-table location, and absence or presence of vegetation (Brown and Thompson 1977). Data from other research, including some with UPB dredged material (Brown and Thompson 1977, Haliburton et al. 1977), indicate that evaporative drying in fine-grained dredged material should proceed at environmental/climatic demand rates for extended periods as capillary resupply of water to the dredged material surface from very wet underlying material will equal or exceed the environmental demand. Under these conditions, it is probable that the upper surface of the dredged material dries to a "just saturated" water content. Such consistency is close to that denoted by the Atterberg plastic limit. In the very wet subcrust material, a water content near the Atterberg liquid limit appears to be a rational assumption.

Water-content behavior between these bounds, i.e., its variation between plastic and liquid limits with increasing crust depth, is not well-known but important as volumetric shrinkage is related directly to the volume of water lost. Results of field-crust water content determinations at the test site (Palermo 1977b, Lacasse et al. 1977) indicated that water contents were normally within the estimated upper and lower bounds, but no well-defined trend with depth was positively established. In consequence, the net effect on total shrinkage and settlement of several assumed crust water content distributions was determined. Figure 17 shows the various water content distribution assumptions. An initial crust, which thickens as the water table drawdown occurs, is assumed to be present. The shaded areas denote the relative amount of water loss and thus shrinkage. The coefficient of shrinkage (C_s), defined in Equation 3, was used to relate average change in crust water content to settlement for the various assumptions using the relationship:

$$\Delta H = H \frac{\Delta w}{C_s} \qquad (3)$$

where

ΔH = vertical settlement from shrinkage

H = thickness of crust

Δw = average change in water content

C_s = coefficient of shrinkage

Parameters used in this analysis were determined by averaging results of laboratory shrinkage tests and Atterberg limit data for the same strata of dredged material considered in the one-dimensional consolidation analysis.

Comparison between computed and measured total settlement

Total settlement from shrinkage and consolidation was determined for the various crust water content assumptions and plotted versus drawdown in Figure 18, as were the measured total settlement data. Based on the results shown, Assumptions No. 4 and 6 appear to give best agreement with measured data. It should be noted that both measured and predicted data are results of average measurements and soil properties determined over a 26-ha test site so exact agreement of measured and predicted results would be a matter of chance. However, agreement between measured and predicted values is good enough to substantiate assumptions about general types of dredged material behavior produced by the trenching program.

Economic Analysis

An economic analysis of trenching operations at the UPB test site was made to determine an estimated unit cost of creating dredged material storage volume by dewatering densification using trenching. The MDO benefit-cost ratio was also determined for progressive trenching operation.

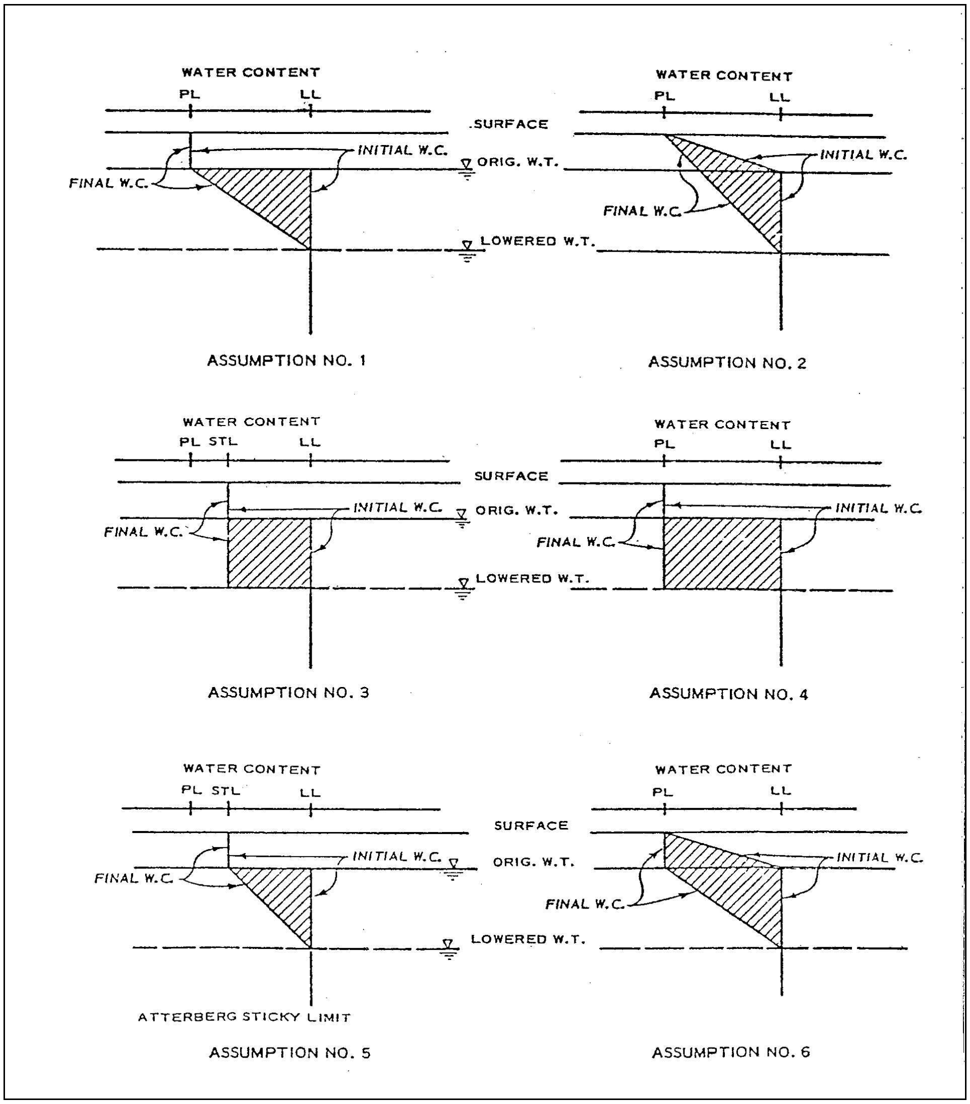


Figure 17. Various crust water content distribution assumptions used to estimate settlement from desiccation shrinkage

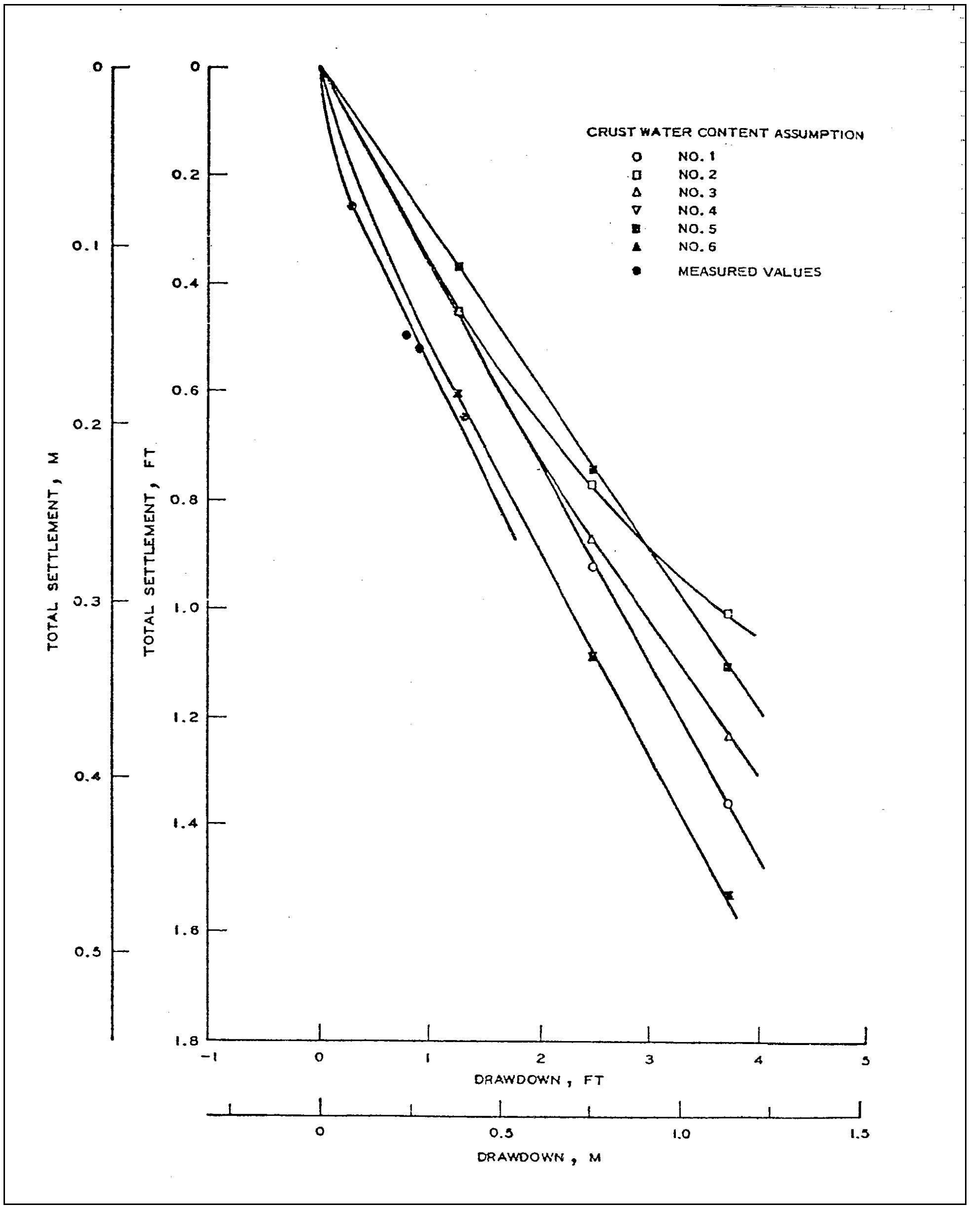


Figure 18. Calculated and measured total surface settlement versus drawdown

Probable cost

In computing costs of construction and maintenance of the trenching system at the UPB test site, an attempt was made to segregate operational costs which would be incurred by a CE element in implementing the work from the extra research costs incurred during the study. Itemized actual costs, based on 1977 dollars, are summarized as:

Item	Cost
Site characterization Includes surveys, sampling, and laboratory testing	$2,000
RUC operation 181 operating hours @ $75/hr	13,600
Amphibious dragline operation 520 operating hours @ $55/hr	28,600
Conventional dragline operation 122 operating hours @ $35/hr	4,270
Technical supervision	10,000
Total operational cost for trenching work	$58,470

Costs of site characterization are based on those necessary for obtaining adequate data to allow one to choose equipment and predict potential benefits. RUC operational costs of $600 per day include the use of a trained two-man Corps of Engineers (CE) crew, operation for 8 hr per day, normal maintenance costs, and per diem for the crew. The availability of RUC equipment for use in disposal area reconnaisance and trenching operations is currently being expanded by DMRP overhaul of additional RUC vehicles, which may be made available to interested CE Divisions or Districts.

Benefits

Computation of benefits from trench dewatering can be related directly to increased capacity within a disposal site. Accumulation of benefits continues as long as the water table in the dredged material is being lowered. An average settlement throughout the 26-ha trenching test site of 0.23 m (0.75 ft) from October 1975 through November 1976 gives an approximate increase in capacity of 55,200 cu m considering only the lowering of the surface. Net worth or replacement cost per cubic meter of disposal capacity was estimated at $2.63/cu m by MDO personnel in January 1977 for the Upper Mobile Harbor area. A net monetary benefit of $145,200 was therefore gained by progressive trenching operations at the test site, compared to costs of approximately $58,470. In reality, benefits may be even greater because, replacement of existing disposal areas along the upper Mobile River today would be nearly impossible because of environmental constraints (USAE District, Mobile 1975). Also, the computation of created volume does not consider either the volume gained in desiccation shrinkage cracks which amounted to 8 percent of the crust volume in another DMRP study (Haliburton et al. 1977), or the volume gained from general disposal area settlement outside the test site from improved drainage. Further, dewatering and densification from improved drainage continued past November

1976 and will continue to occur as long as the constructed surface drainage system functions properly. In addition, the dewatered material may be removed to raise the perimeter dikes or for other productive use.

Unit cost and benefit-cost ratio

In assessing the effectiveness of the RUC trenching program, it is estimated that approximately one-third of the operational costs were expended in a non-productive manner, i.e., the work did not result in the desired change in the trenching system. This inefficiency is primarily a result of the learning process as no prior experience in interior trench construction in disposal area was available. Further conduct of trench dewatering, at UPB or elsewhere, following guidelines developed herein and elsewhere (Haliburton et al. 1978, Willoughby 1978) should result in a unit operational cost about one-third less than was actually incurred in this test program. Actual operational cost incurred in the research was $1.07/cu m of additional storage volume created, and operational costs for future trenching programs should be on the order of $0.71/cu m of additional storage volume created. The benefit-cost ratio was computed as 2.48 using the cost of the actual operation and total benefits as of November 1976 and should be on the order of 3.7 for future trenching operations.

Conclusions and Recommendations

Based on the procedures, results, analyses, and discussion presented herein, it may be concluded that:

a. Densification and dewatering of fine-grained dredged material can be induced by improving the surface of the drainage disposal site and thereby increasing the evaporative drawdown of the internal water table. Densification and dewatering will result from a combination of subcrust dredged material consolidation under increased effective stress and shrinkage from evaporative drying above the lowered water table.

b. Construction of surface drainage systems by trenching within confined disposal areas is operationally feasible. Trenching operations must remain flexible to adjust to changing conditions within the disposal area resulting from progressive drainage and drying of dredged material.

c. As dredged material drying and crust formation progresses, the trenching system must be progressively deepened to allow continued drainage from the crust and to promote further crust formation.

d. Trenching with the RUC is the best available method to initiate surface drainage in disposal areas with a crust thickness less than 150 mm.

e. RUC-constructed trench flowlines are somewhat governed by material consistency and existing disposal area topography. Thus, ponding in low areas and flow blockage from harder ridges of material may reduce trenching efficiency. A backhoe mounted on the RUC would solve such problems.

f. The maximum trench depth which can be attained by RUC trenching is approximately 0.5 m. The use of available implements gave no apparent advantage in trench deepening or grading.

g. The most effective method of RUC utilization is a progressive trenching approach with sufficient time allowed between trenching operations for drying and reformation of crust in the RUC trenches. If intersecting trenches are used, hand cleaning of trench intersections is required to insure efficient drainage.

h. Amphibious or marsh chassis draglines are effective in trenching operations when crust thicknesses are in excess of 0.15 m. Lightweight draglines operating from mats can be effectively employed when existing crust thicknesses are in excess of 0.3 m. Draglines are effective in deepening trenches and grading trench bottoms to allow efficient flow. The most effective method of employing draglines is to side cast the excavated material to the same side of the trench trailing the boom. Broken windrows should be formed to allow paths for surface drainage to the trench, and the excavated material should be flattened with the bucket to improve drying.

i. Trench depths of 1 to 4 m or more may be attained using dragline equipment. The thickness of the surface crust and the water content of subcrust dredged material are limiting factors. The stability of dragline-constructed trenches presented no significant problems in maintaining drainage efficiency although flow and sloughing limited trench depths in areas where the subcrust dredged material was near the liquid limit.

j. The magnitude of drawdown was not greatly affected by trench spacing, based on available observation well data. No well-defined phreatic profile indicating higher drawdown near the trenches was observed. Trench location and spacing should therefore be controlled by surface topography, and only enough trenches to stop ponding of surface water need be employed.

k. The results of analyses to predict the expected amount of settlement from consolidation and shrinkage compared favorably with the measured field behavior.

It is recommended that progressive trenching techniques be used by CE field elements or other agencies interested in dewatering and densification of fine-grained dredged material placed in confined disposal areas from maintenance activities.

5 Windmill-Powered Vacuum Wellpoint Field Demonstration

At the DMRP Planning Seminar I, held at the WES in October 1974 (DMRP 1974), it was suggested by invited consultants and technical experts that vacuum consolidation be studied as a potential technique for dewatering fine-grained dredged material. While hydraulic permeability of fine-grained highly plastic dredged material is lower than normally recommended for application of vacuum consolidation techniques, the material is highly compressible and unable to support even small conventional surcharge loads without bearing failure. Thus, even a small amount of vacuum surcharging might cause appreciable consolidation. Further, if vacuum application could be maintained for an extended period of months, or even years, negative pore pressure might be propagated through the entire dredged material mass despite the low permeability of the material.

After DMRP staff discussion, it was decided to investigate vacuum-assisted consolidation for two cases: (a) when the vacuum consolidation system could be installed prior to any disposal of material, and (b) when the vacuum consolidation system must be installed through or into already existing underwatered dredged material. The former case was investigated as part of the research to evaluate underdrainage dewatering, and is described in Chapter 9. In the latter case, vacuum wellpoints were selected for evaluation; that field demonstration is described below.

In addition to the decision to evaluate vacuum wellpoint feasibility, it was decided to evaluate windmills as a possible source of providing electrical power at remote disposal area locations. In this instance, the power would be used to run the vacuum pump and other powered items required to operate a vacuum wellpoint system. The study was conducted in two parts. The windmill-powered generation feasibility phase of the study was conducted by the WES EEL Environmental Engineering Division, Design and Concept Development Branch. The vacuum wellpoint feasibility phase of the study was conducted by the MDO Engineering Division, Foundations, and Materials Branch.

Feasibility of Using Windmill Power

One consideration of any dredged material dewatering operation is the availability of adequate electric power for treating and reducing the volume of dredged material. Other DMRP research (Parker et al. 1977) identified and formulated detailed descriptions of presently available systems for converting resources located in the vicinity of the confined disposal sites to usable and reliable energy for operating equipment to separate, filter, rehandle, and otherwise treat dredged material.

The windmill power feasibility phase was thus designed to investigate the status and availability of wind-power equipment and technology, and, based on this evaluation, to design and fabricate a system that would supply adequate power to a 0.4-hectare (1-acre) vacuum wellpoint dewatering demonstration. Simply stated, there are three basic tasks to address in harnessing power from the wind: (a) selection of a suitable site because the amount of electricity generated is critically dependant upon wind speed, (b) selection of wind generator equipment, and (c) selection of devices for storing energy produced during periods of peak wind activity for use during periods of reduced wind activity. More detail is available elsewhere (Long and Grana 1978).

Field Demonstration

Site evaluation and selection

As previously stated, the predominant consideration for wind energy exploitation is the availability of adequate wind. Based on long-term wind data from the National Weather Service, Mobile presented conditions favorable for demonstrating a wind-powered system. Table 11 gives long-term data on wind velocity in Mobile, and statistics reveal that the average wind speed is greater than the minimum conditions for successful windmill operation (Long and Grana 1978). For 6 consecutive months the average wind speed for this area equals or exceeds 17.2 knots/hr. The UPB disposal area was thus believed suitable for evaluation of windmills as an energy source. The windmill and control trailer are located on an enlarged section of the dike while the wellpoints are installed in the disposal area.

System power requirements

Power requirements were established by the vacuum wellpoint system with the major power demand item being a two-stage high vacuum pump requiring a 1.5 kW motor. Additional power was needed to drive a small water pump and to operate instrumentation.

Table 11
Percentages Frequencies of Hourly Wind Speed in the Vicinity of Mobile, AL

(1)Percent of Time Hourly Wind Speed Observed to Have Given Velocity knots/hr, miles/hr

Month	0-5 0-3	6-11 4-7	12-19 8-12	20-29 13-18	30-39 19-24	40-50 25-31	51-61 32-38	62-75 39-46	76 47-over	Total	Avg. Wind Speed knots/hr, miles/hr
January	5	19	39	24	11	2	+	+	0	100	18.5/11.5
February	3	18	36	28	12	2	+	+	0	100	19.5/12.1
March	4	19	35	27	13	1	+	+	0	100	19.2/11.9
April	6	23	38	24	9	1	+	+	0	100	17.5/10.9
May	7	29	38	20	5	+	+	0	0	100	15.6/9.7
June	10	34	37	15	2	+	+	0	0	100	13.7/8.5
July	12	40	36	11	1	+	+	0	0	100	12.4/7.7
August	11	40	39	10	1	+	+	0	0	100	12.4/7.7
September	8	34	38	16	3	1	+	+	+	100	14.5/9.0
October	8	30	41	18	4	+	+	0	0	100	15.0/9.3
November	4	25	40	22	7	1	+	+	+	100	17.2/10.7
December	4	23	39	24	8	1	+	+	0	100	17.7/11.0
Annual	7	28	38	20	6	1	+	+	+	100	16.1/10.0

Note: + Less than 1percent.

System equipment and operation

Windmill-powered generator. Commercial windmill-powered generators are available in sizes from 0.05 to 12 kW with the larger units being of foreign manufacture. The machine chosen for this study was a 115 VDC ELEKTRO Model WVG50G, fabricated in Switzerland. Pertinent specifications for the unit are given in Table 12 while its power output as a function of wind speed is shown in Figure 19. Because of friction and inertia forces, there is a minimum wind velocity or cut-in speed required for a windmill to rotate. For the ELEKTRO machine, this velocity is 12.9 knots/hr. A more important consideration is the percentage of time a specific wind speed is available. Figure 20 gives power and wind duration curves for the Mobile area which show that windspeeds of 16.1 knots/hr should be experienced approximately 45 percent of the time, thus providing enough total power to run the wellpoint experiment.

In small systems, it is usually necessary to connect the windmill output to lead-acid or other types of storage batteries using the windmill to charge the batteries during calm or intermittent wind conditions. The modifications required to produce a periodic and predictable output are the main disadvantages of windmill-powered systems.

Table 12
Specifications for ELEKTRO Model WVG50G Windmill

Propeller	
Diameter	5.0 m
No. of blades	3
Generator output	
Phase	30 AC
Voltage	65 V
Power	5 kW
Rectifier output	
Voltage	110 VDC
Power	6 kW
Rated wind speed	37 km/hr
Cut-In Speed	13 km/hr
Furling Speed	72 km/hr
Power Coefficient Cp	0.37 @ 37 km/hr
Cost	
Generator and controls	$6,600
Tower	$1,200
Expected Life	30 year

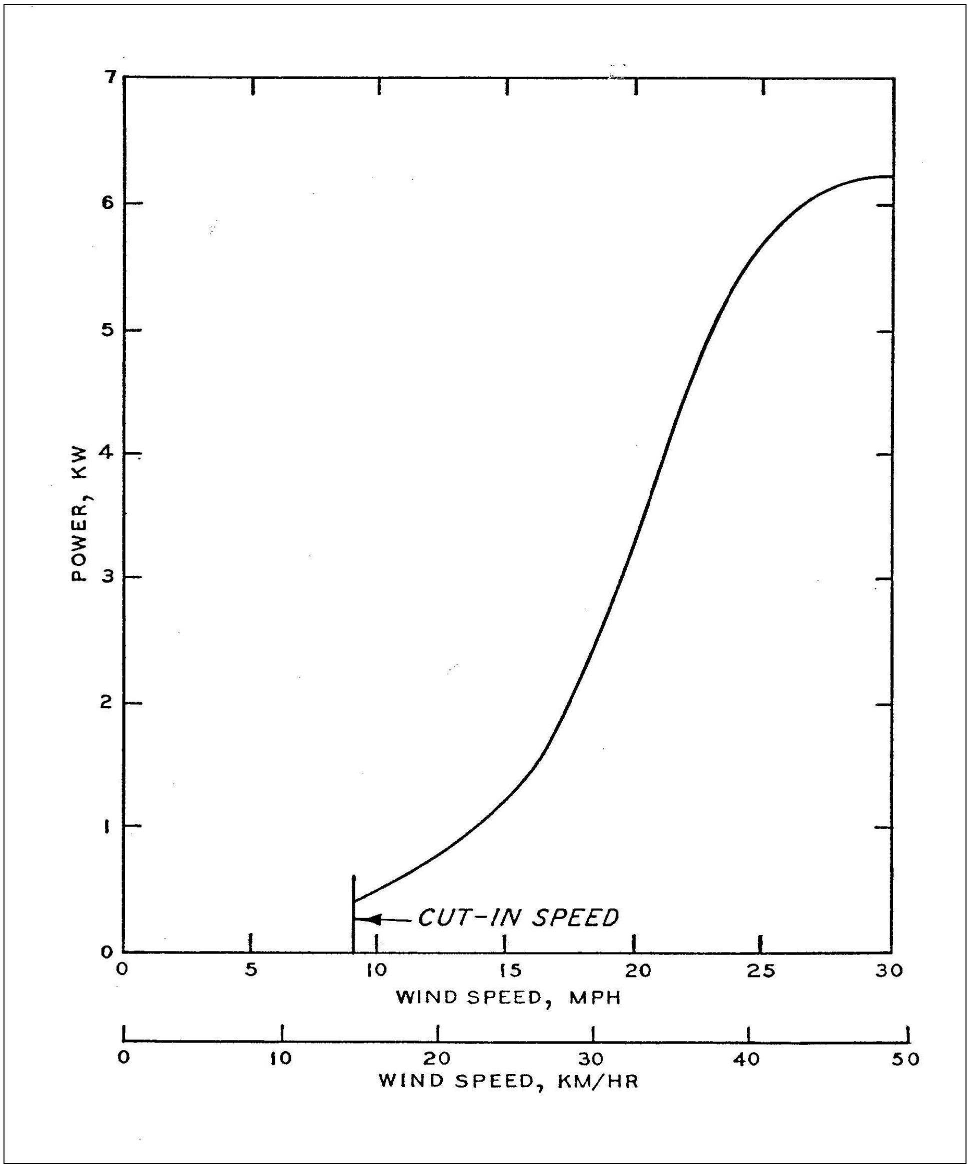


Figure 19. Power output versus windspeed for ELEKTRO Model WVG50G windmill

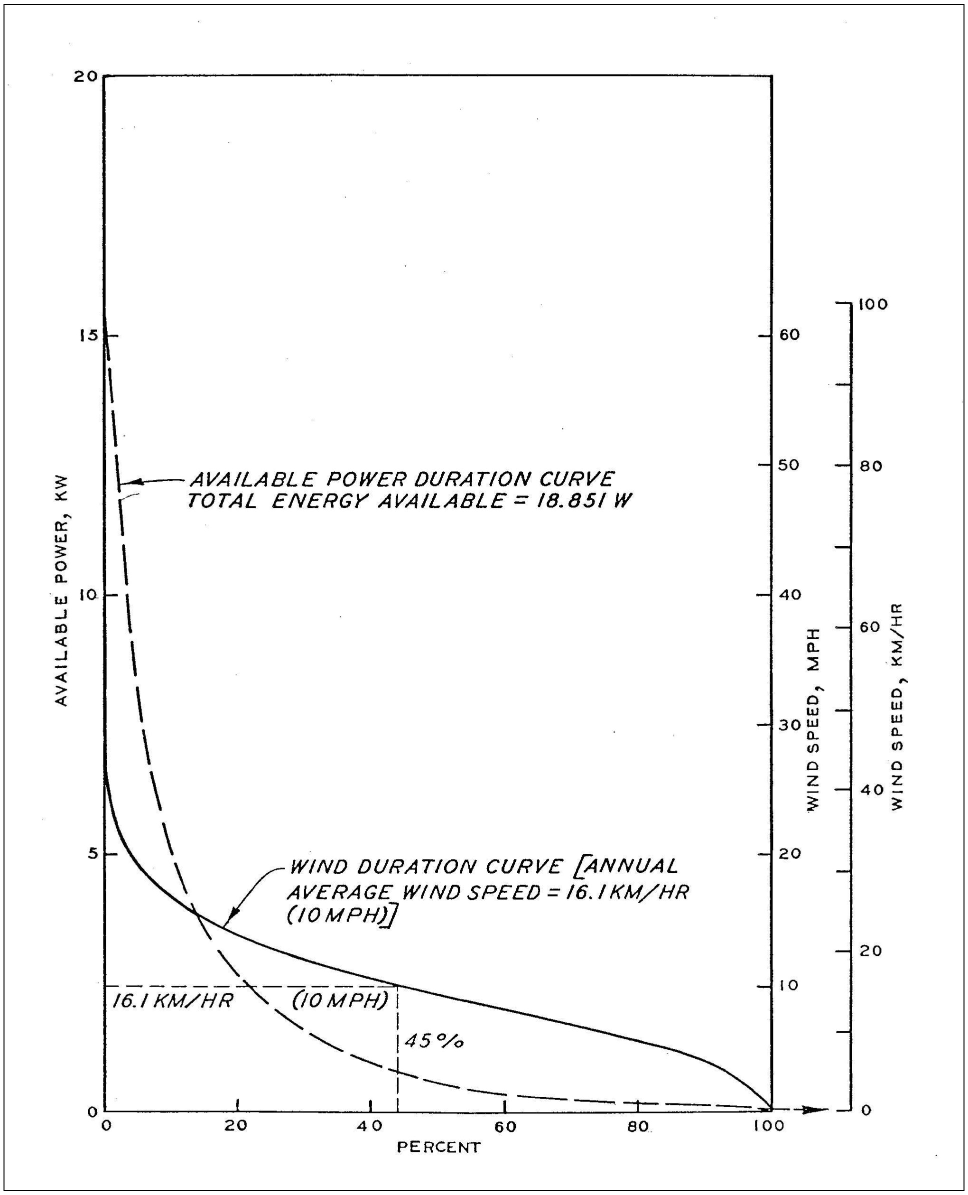


Figure 20. Average wind duration for Mobile, AL, and resulting power

Several precautions were taken to protect operating personnel and equipment. An adequate grounding arrangement for protection during electrical storms was available from ELEKTRO Model WVG50G windmill made by installing a 25-mm-diam, 3-m-long grounding rod into the dike, attached to the windmill tower by a 250-mil solid copper conductor. To protect the windmill during periods of high wind speed (72 knots/hr or greater), and overspeed (furling) device was installed to automatically rotate the windmill sail parallel to the wind. A low voltage detector was installed to automatically remove the power system load when the battery voltage decayed to 95 VDC to protect the batteries from being totally discharged. More detail is available elsewhere (Long and Grana 1978).

Windmill foundation and tower. A reinforced concrete footing was constructed on the top of the northwest corner of the dike designed to support 907 kg of equipment. The windmill tower was 12-m tall and was constructed from prefabricated sections Nos. 5N and 6N of a ROHN, Model SSV, standard tower, as supplied by ELEKTRO.

Control system. Power and controls were needed to vacuum pump water from the wellpoint test site into a sump and, when the sump was filled, automatically turn the vacuum pump off, vent the tank to relieve the vacuum, and then activate a water pump to empty the tank. After the tank was emptied, vacuum would be reapplied.

Power system. A detailed schematic diagram of the windmill power system is shown in Figure 21. The wind-pressure switch senses for high winds and then signals the tower control box containing the motor and related circuitry to furl the windmill sail. The low voltage detector contains the switching mechanism for removing electrical load from the batteries to keep them from being fully discharged or overcharged. The battery pack is composed of 38 6V batteries. Two battery circuits are connected in parallel with each circuit containing 19 series-connected batteries providing a nominal voltage of 115 VDC and 550 amp-hr capacity. The main control panel and rectifier contains circuits for rectifying the 3φ voltage from the windmill and distributing it to the load. To insure that adequate power was available for the wellpoint system during the entire test, a back-up auxiliary generator was used to augment the power supplied by the windmill. Wind power, even when stored in a battery pack, may be available only periodically whereas the power demand of the vacuum wellpoint system is relatively constant. To be compatible with the windmill, the electrical specification for the auxiliary generator, in concert with the variable transformer and rectifier, was 115 VDC. With the system operating at a fixed load, the windmill and generator connected in parallel could not put out more than the system required. For example, if the windmill and generator were both operating at rated output, each would be furnishing half the power required by the load. As the output of the windmill varied with windspeed, more or less power would be supplied by the generator.

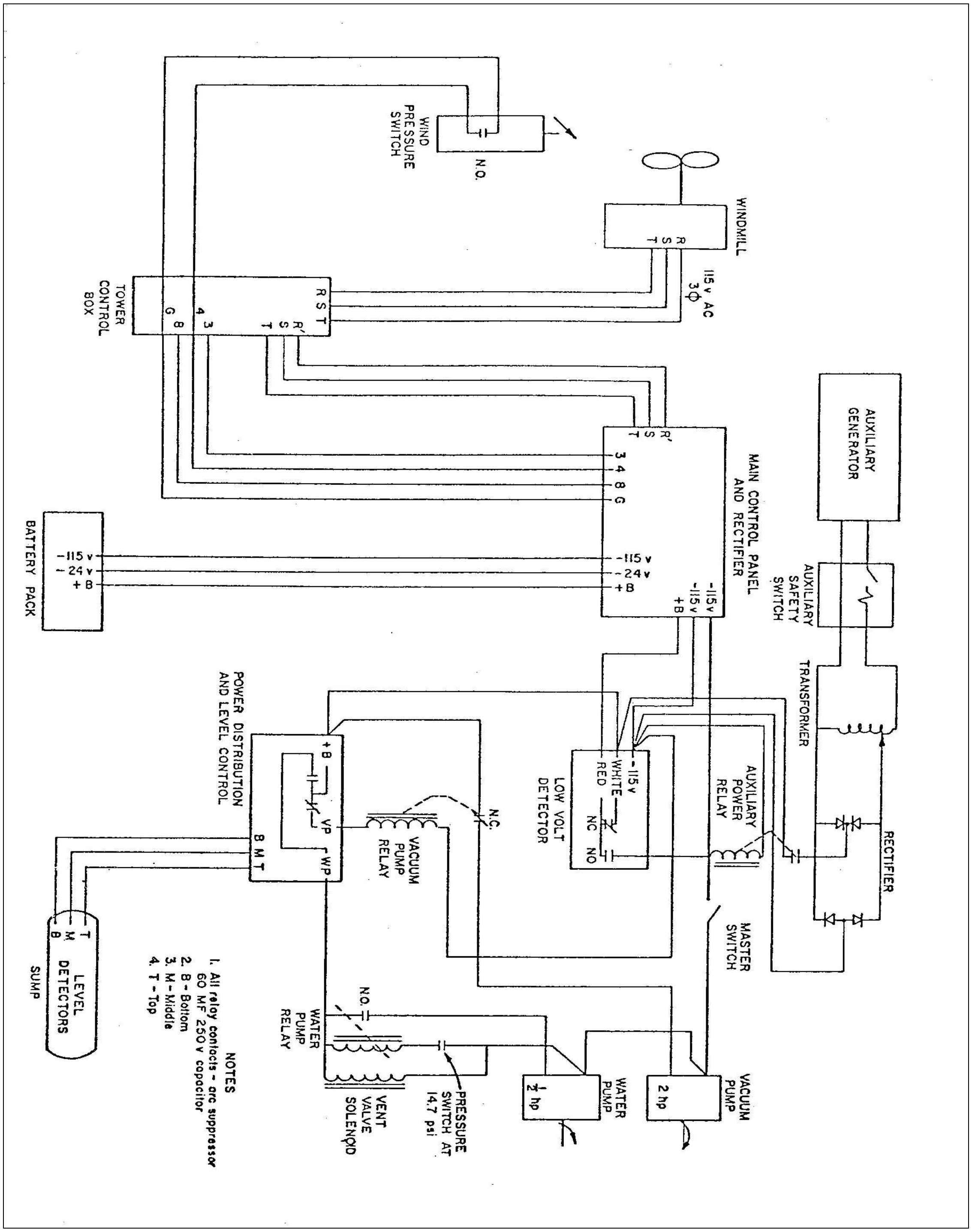


Figure 21. Windmill and pumping system power distribution schematic

The water pump is located behind the vacuum pump, sump tank, and the header lines from the wellpoints. Other peripheral hardware required for system operations were two 115-VDC shunt connected motors (1.5 kW and 0.4 kW), a pressure switch set at atmospheric pressure to control the "on" time of the water pump, and a solenoid vent valve to relieve the sump vacuum.

Windmill-Powered System Performance

The system was installed and debugged during the spring of 1976 and, at the outset, performed satisfactorily. During the summer of 1976, the vacuum pump developed a shaft-seal leak that shut down the experiment until it was repaired. This repair was accomplished, but because of a low incidence of wind in July and August 1976, insufficient power was available. The gasoline-powered auxiliary generator was then procured and installed.

Also, during July and August 1976, there were numerous reports that the windmill was not revolving and also that lightning had struck the windmill tower. At first, it was suspected that this caused windmill malfunction because the windmill had performed satisfactorily for the first several months after installation. However, detailed inspection revealed that wear between the windmill main and drive gears caused the shaft to bend after less than 300 hr of operation. Because of this premature curtailment in windmill operation, no quantitative analysis of output power availability was attempted. The overall wellpoint demonstration continued to operate on auxiliary power. The system was operated continuously, except for periodic maintenance, for six weeks. At this time, the generator failed and was replaced by a diesel-powered generator. The system again performed satisfactorily for another 6-week period. At this time, the vacuum pump failed and the wellpoint investigation was terminated.

In addition to the windmill equipment failure, numerous other problems affected system performance. The windmill manufacturer (ELEKTRO) grossly understated maintenance requirements. Also, the relays and pumps were required to operate under adverse environmental conditions. Extremely moist and saline air caused relays to corrode and fail, and highly caustic and corrosive water pumped from the dredged material (pH approximately 10) ultimately caused the vacuum pump to fail.

Summary

The field investigation revealed a number of uncertainties involving field deployment of the windmill-powered system and its associated components:

a. The reliability and maintainability of the system used were unsatisfactory for long-term deployment. These shortcomings could be somewhat alleviated with adequately trained personnel and a thorough inspection and testing program, but they are also dependent upon better product design and quality control.

b. The operational environment had an adverse effect on maintenance requirements, showing that an all-weatherproof design is needed for long-term operation.

c. In this test, Mobile area data from the National Weather Service overpredicted available windspeed at UPB and thus were unreliable in establishing prior feasibility of the specific site. To ensure that adequate wind energy is available, a comprehensive investigation should be initiated on-site prior to any final decisions concerning wind-power suitability.

d. Because of the numerous equipment problems and malfunctions, data establishing the long-term technical feasibility of utilizing wind energy to provide power at a remote installation are inconclusive. Results of the experiment were discouraging.

Vacuum Wellpoint Dewatering Feasibility

Review of wellpoint development

Initial uses of wellpoints date to the turn of the century. These early dewatering systems were moderately successful when used to lower the elevation of the groundwater table within a small area. Also, initial uses of wellpoints were confined to lowering the groundwater elevation in clean sands which are free-draining.

Tremendous strides have been made since then in the development of wellpoint dewatering equipment. Modern conventional wellpoint systems consist of one or more stages of wellpoints. The wellpoint or tip is a small screen constructed of brass or stainless steel mesh, slotted brass, or plastic pipe. Well screens are usually 50 to 100 mm in diam and 0.6 to 1.5 m long. Wellpoints may include a special tip for jetting the wellpoint into position. Riser pipes are generally 40 to 50 mm in diam. A series of riser pipes and wellpoints are interconnected by a header system. The header system is connected to a wellpoint pump. These pumps generally have both a vacuum and a centrifugal component to remove water that drains to the wellpoint.

Despite advances in equipment development, conventional wellpoint systems are primarily used to dewater free-draining granular materials. It is suggested in some literature that silts and sandy sills with permeability coefficients of approximately 10^{-6} m/sec cannot be drained by gravity methods (Department of the Army 1971), but that vacuum wellpoint systems may be successful in stabilizing the materials by establishing a partial vacuum at the wellpoint.

Dewatering and consolidation of fine-grained materials (permeability coefficients of 10^{-8} m/sec or less) is normally accomplished by means other than conventional wellpoints or vacuum wellpoints. These methods do not produce the groundwater drawdown required on modern construction projects. However, there are at least two known references in which dewatering and consolidation of fine-grained materials with vacuum wellpoints were accomplished.

Vacuum was used to consolidate fine-grained material by Kjellman (1952). This field test required installing a series of vertical sand drains in soft clay. The area to be dewatered was covered with a surface sand blanket. An impermeable membrane was placed over the horizontal sand blanket. A suction pipe was placed through the membrane and was connected to a vacuum pump. By applying vacuum to the horizontal sand blanket and vertical drains, the pore pressure in the soft clay was gradually decreased allowing atmospheric pressure at the surface to act as a surcharge. A maximum settlement of approximately 0.54 m was achieved after pumping for 110 days. An average vacuum of 71 kPa was developed equivalent to a surcharge weight of sand fill approximately 5 m thick.

Vacuum wellpoints were used to consolidate soft clay subgrade materials prior to extending a runway at Philadelphia International Airport (Halton et al. 1965). The airport had been constructed on marginal property adjacent to marshland reclaimed by use as disposal areas for maintenance dredging of the Delaware River. The arrival of the jet age necessitated extending a runway approximately 700 m. The extension was to be constructed over the old disposal area. Construction restraints included maintaining operation of the existing runway which prohibited the use of high fills to surcharge the soft subgrade. Therefore, it was decided to stabilize the subgrade with a system which combined vacuum dewatering and sand drains. Vertical sand drains, capped at the surface with bentonite clay, were placed through the soft material into an underlying granular layer. Deep wells connected to the vacuum system were placed around the site, extending into the underlying granular layer. A vacuum of 380 mm Hg was developed and maintained for 18 days. Approximately 0.2 m maximum pre-construction settlement was obtained by vacuum application.

Rationale for use of vacuum wellpoints

As indicated previously, dewatering and consolidation of fine-grained materials with vacuum systems is not a new concept. In fact, the potential use of vacuum systems in consolidating fine-grained dredged material was discussed by Bishop and Vaughn (1972). However, previous use of vacuum wellpoint in fine-grained soils has incorporated closely-spaced vertical sand drains and/or pumping from a granular subdrain layer.

In many instances, dredging is accomplished on a periodic cycle of several years. Therefore, confined dredged material is often deposited and left unattended for several years. If a vacuum could be established in the dredged material, the long-term potential for volume reduction could be significant, perhaps without the need for extensive sand drains and surface blankets. Preliminary calculations based on consolidation test data from typical UPB dredged material indicate that a surcharge of 100 kPa (equivalent to atmospheric pressure) could produce volume reductions on the order of 100 percent. The consolidated material would occupy approximately half the volume occupied by the underwatered dredged material.

The cost of installing wellpoint systems of conventional steel or brass materials over large disposal areas would be enormous. Also, the surface support capacity of the unconsolidated dredged material is very low, and both the disposal area environment and the dredged material pore water may be corrosive. Therefore, lightweight, low-cost, and non-corrodible materials would be required for construction of any vacuum wellpoint system.

Conducting the Field Demonstration

Installation of vacuum wellpoint system

An approximate 0.4-ha site, located in the northwest corner of the UPB disposal area, was selected for conducting the vacuum wellpoint demonstration (Haliburton 1976), as shown in Figure 2. The test site was divided into four sections. Two sections (Sections B and D) were established as undisturbed control sections. These sections were to be monitored to establish a basis for comparison between settlement of vacuum-pumped and non-pumped dredged material. Sections A and C were established to evaluate the dewatering/consolidation effect of applying vacuum at different wellpoint spacings. Wellpoints were to be placed on 6-m centers in Section A and 12-m centers in Section C. These wellpoints are relatively wide-spaced in comparison to conventional wellpoint applications. However, in order for vacuum dewatering/consolidation to be economically feasible, wellpoint spacings of at least these distances would be required.

Plastic (PVC) pipe (Schedule 40) was used to construct the wellpoints and header system. Several advantages exist for use of plastic pipe. The piping is light and easy to handle. This is important since installation of the wellpoint system had to be accomplished entirely with manual labor because the dredged material would not support heavy equipment. The pipe can be easily cut and glued to accommodate varying length requirements. However, one uncertainty was how well the glued pipe system would hold a vacuum. Also, plastic pipe should be relatively durable in a corrosive environment, so one of the objectives was to verify this. It is also a relatively inexpensive material when compared to steel piping.

The vacuum wellpoints were installed according to the following procedure:

a. The surface crust, to a depth of approximately 0.3 m, was excavated by shovel.

b. Metal casing, 130 mm OD, was manually pushed into the dredged material to a depth of approximately 2.7 m, the approximate thickness of dredged material at the test site.

c. Dredged material within the metal casing was flushed out by a water jet.

d. The vacuum wellpoint riser pipe, with slotted tip, was placed inside the metal casing.

e. Sand was placed between the riser pipe and the metal casing to a height of approximately 0.3 m above the slotted section. The slotted section had been previously wrapped with “Filter X” brand filter cloth. The bottom 0.76 m of the riser pipe was slotted with 0.2-mm openings to form a pervious tip section.

f. A 0.6-m-thick bentonite clay pellet section was placed above the sand fill to form a seal.

g. Dredged material was used to backfill to the surface.

h. The metal casing was manually extracted, leaving the wellpoint in place.

The installation detail is shown in Figure 22. A deviation in wellpoint installation procedure was used to install wellpoints A-1, A-2, A-3, and A-4. These wellpoints were installed according to procedures shown in Figure 23. The wellpoint tip was inserted in a sand-filled burlap bag. After breaking the surface crust, the sand bag and riser pipe were manually shoved through the soft dredged material to the required depth.

Wellpoint riser pipes were connected to a 100-mm ID header pipe by a wire-reinforced flexible hose. The 100-mm header pipes were connected to a 150-mm OD collector pipe which was connected to a water collection sump. The relatively large diameter header system was chosen to minimize vacuum loss by flow friction. Later evaluation indicated smaller piping could have been used successfully.

Vacuum was supplied by a Precision Scientific Company Model D1500 two-stage vacuum pump with free air capacity of 1,500 L/min and ultimate vacuum of 0.01 µm at the inlet. The vacuum pump was driven by a 1.5-kW DC electric motor. During various phases of the experiment, electric power was developed from a windmill generator, gasoline-powered generator and diesel-powered generator, as described previously.

Using a four-man crew, approximately 10 8-hr working days were required to install 38 wellpoints. Approximately 5 working days were required to install the vacuum header system.

Instrumentation

Open tube piezometers were installed to record variations in the groundwater table. Initially, 37 piezometers were installed. These piezometers were installed by the procedure indicated on Figure 23. During the operations phase of the test, additional piezometers (designated 0-38 through 0-57) were installed. These piezometers utilized a 13-mm riser pipe and a porous tip. The porous tip was placed in a sand-filled burlap bag. All piezometers were placed by manually pushing the riser pipe and tip through the soft dredged material to a depth of 2.7 or 1.2 m.

Vacuum gages were installed in the header system at each wellpoint location. Cutoff valves were placed at each wellpoint location to avoid vacuum loss over the entire system in case of isolated leaks.

At each soil-moisture tensiometer location, tensiometers were installed to depths of 0.6, 1.5, and 2.4 m (2, 5, and 8 ft) below the surface. These tensiometers are normally used in conjunction with agriculture studies and indicate hydraulic heads without regard to whether soil water pressures are positive or negative (Richards et al. 1973). Tensiometers used in this experiment were commercially available Model 2710 instruments manufactured by the Soilmoisture Equipment Corporation.

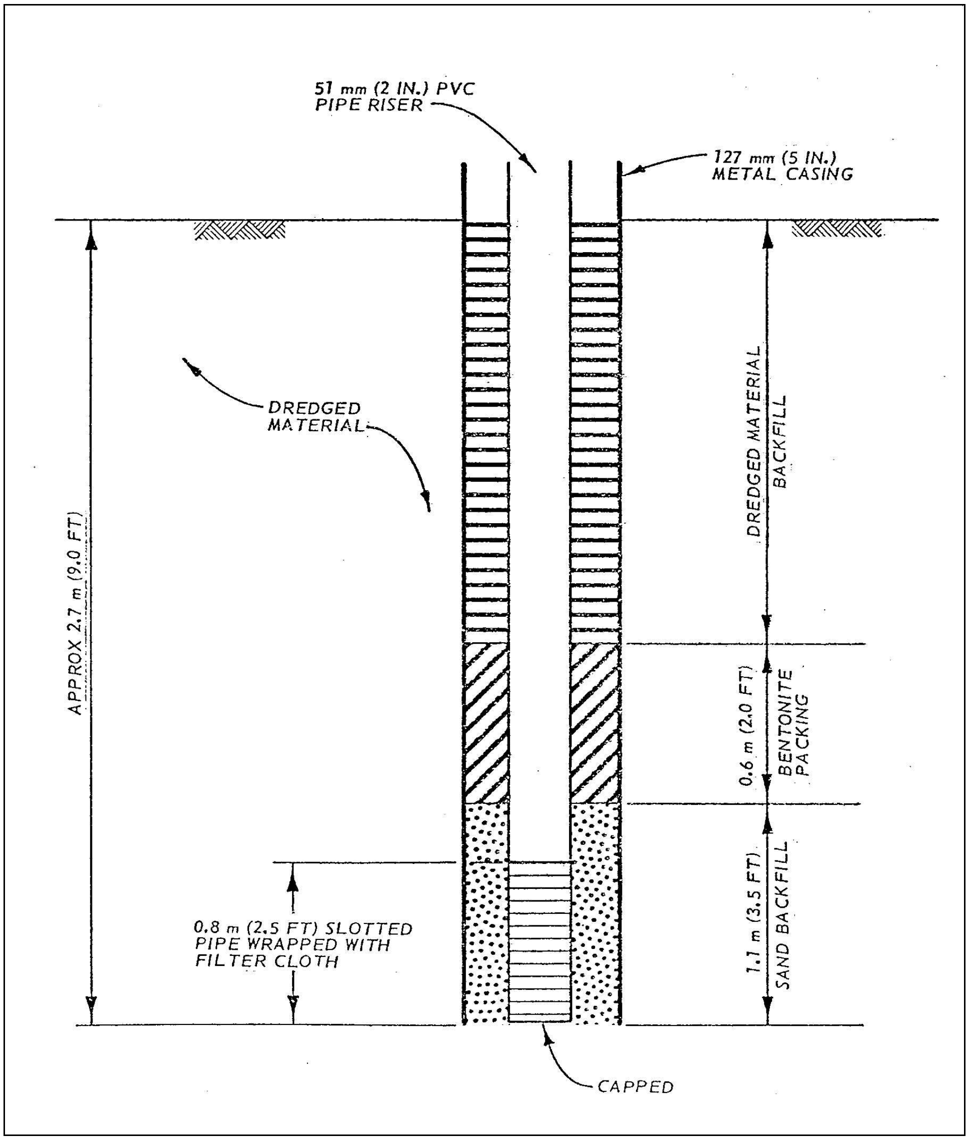


Figure 22. Typical section of vacuum wellpoint as installed by casing method

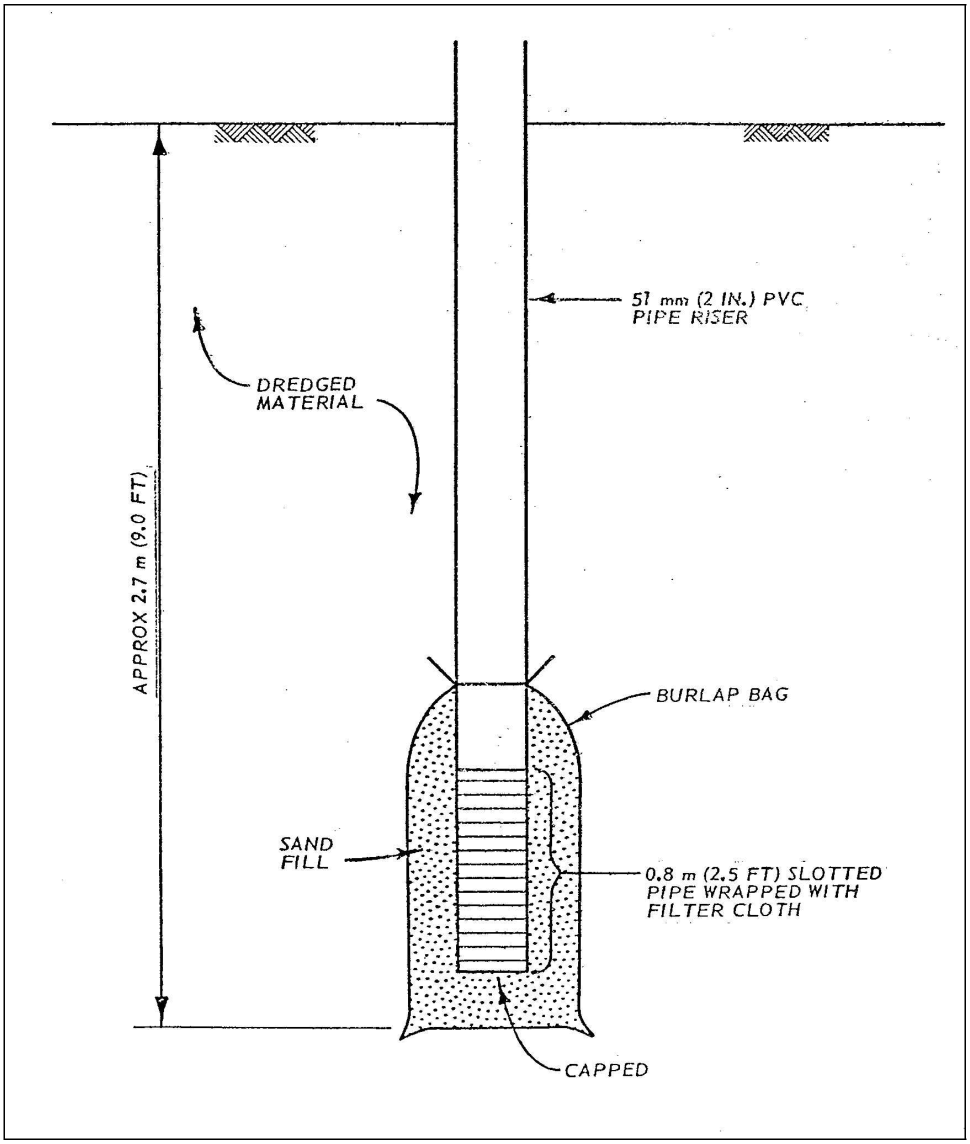


Figure 23. Typical section of installed piezometer

A standard water meter, similar to those used by utility companies to measure single residence water consumption, was placed at the discharge side of the water pump used to empty the collection sump.

Operation and data collection

The vacuum wellpoint system was designed to draw water to a main sump. Water from each wellpoint was carried through the 100-mm header pipe to the 150-mm collector pipe and on to the collection sump. One collection sump served for both Sections A and C. An automatic pumping and discharge cycle was used, as described previously.

During the test, vacuum gages, piezometers, tensiometers, and the water meter were periodically monitored. Weather data were obtained from the National Weather Service at the Mobile Airport. In addition, samples were obtained at distances of 0.3, 0.6, and 0.9 m (1, 2, and 3 ft) from wellpoints A-11, A-21, C-2, and C-9. A hand-operated piston sampler was used to retrieve the samples. Water-content test results are given in Appendix B of Palermo (1977a). Surface elevations were taken frequently on 3-m centers by survey crews of the MDO Mobile Area Office. A summary of surface elevations is given in Appendix B of Palermo (1977a). Two series of cone penetrometer measurements were taken at selected locations. Approximate testing locations are indicated on the test data summary sheets in Appendix B of Palermo (1977a).

A laboratory testing program was conducted on samples from the vacuum wellpoint site. As indicated above, the water content of the dredged material was periodically determined from samples taken at various locations and depths. The initial set of samples were visually classified according to the Unified Soil Classification System. Eleven water-content samples were also tested for Atterberg liquid limit, plastic limit, and shrinkage limit. Sieve and hydrometer analyses were also conducted. Five relatively undisturbed samples were obtained by hand-pushing a 130-mm Shelby tube. Dredged material densities and water contents were obtained from these samples. One sample was selected for consolidation testing and was incrementally loaded to 100 kPa. All testing was conducted in accordance with EM 1110-2-1906 by the USAE Division, South Atlantic, Laboratory. Test data sheets are given in Appendix B of Palermo (1977a).

Installation of the vacuum system, including wellpoints and header systems, vacuum pump and sump, and windmill power source, was completed in May 1976. Operation of the vacuum system was constantly plagued by mechanical and electrical failures. Pumping equipment often required repair, as did the electrical control system. In addition, nature was uncooperative. During the summer months of 1976, the average wind speed was not sufficient to drive the windmill. During January 1977, record low temperatures froze water in the discharge line and centrifugal pump. The most productive and consistent pumping was achieved during the periods 23 November to 31 December 1976 and 28 January to 20 march 1977. Operation of the vacuum system was terminated on 20 march 1977 because of mechanical failure of the vacuum pump. A log of operation, highlighting daily operational problems, is given in Appendix D of Palermo (1977a).

Summary of Field Study Results

Laboratory test results

A summary of laboratory test results on dredged material samples are given in Table 13. The dredged material at the test site is typically a dark gray fat clay (CH) with a trace of sand. From 90 percent to 99 percent of the sampled material was smaller than U.S. No. 200 sieve. Results of plasticity tests indicated a range in liquid limit from 69 to 165, and corresponding plasticity indices ranged from 44 to 96. These results compare favorably with results of similar tests over the entire UPB disposal area, reported in Chapter 3. In situ densities, as determined from 130-mm Shelby tube samples, averaged approximately 510 kg/cu m (32 lb/cu ft), slightly lower than the overall disposal area values summarized in Chapter 3.

Vacuum system

An average gage reading of 630 mm Hg vacuum was obtained at the riser for Section A wellpoints. Similarly, an average reading of 673 mm Hg vacuum was obtained for Section C. These average measurements correspond to pressures of 84 and 89 kPa, respectively. The actual gage readings are recorded in Appendix D of Palermo (1977a).

The two-stage pump used in this demonstration seemed to be adequate for developing a high vacuum over a much larger test area. However, constant attention and frequent maintenance was required to keep the pump operating. The corrosive nature of the water (pH = 10) necessitated weekly changes of vacuum pump oil, and it finally caused mechanical failure of the pump. A disadvantage of using this equipment for routine purposes would be its mechanical sophistication, which rendered the pump susceptible to corrosion damage and would probably necessitate returning the pump to the manufacturer for major repairs.

The Schedule 40 PVC pipe header and wellpoint systems were not affected by the corrosive environment. Glued connections were tight enough to maintain the vacuum. Vertical sagging between supports in the header pipes was very noticeable after the large diameter header system became filled with water. However, the pipes did not break. The original connections between the header pipe and the wellpoints were made with lightweight flexible hose. This hose collapsed under full vacuum and was replaced with a heavy-duty wire-reinforced hose which performed satisfactorily. However, after approximately 1 year of service, the hose appears to be developing cracks.

Table 13
Summary of Classification and Density Tests

Location	Depth m, ft	Classification	LL	PL	PI	SL	Percent Passing No. 200 Sieve	Dry Density kg/m^3, lb/cu ft	Water Content, %
VW-1[1]	0.2-0.5, 0.5-1.5							462.5, 28.9	158.1
VW-1	0.5-0.8, 1.5-2.5	Black and gray fat clay (CH)	130	34	96		97	511.0, 31.9	142.6
VW-1	1.2-1.7, 4.0-5.5							565.5, 35.5	123.6
VW-1	2.4-2.7, 8.0-9.0							517.4, 32.3	123.3
VW-2	0.3-0.8, 1.0-2.5							482.2, 30.1	134.7
VW-2	1.2-1.5, 4.0-5.0	Dark gray fat clay (CH) with a trace of sand sizes	119	33	86	21	98		
VW-4	2.1-2.4, 7.0-8.0	Dark gray fat clay (CH) with a trace of sand sizes	84	26	58	18	90		
VW-8	0.3-0.6, 1.0-2.0	Dark gray fat clay (CH) with a trace of sand sizes	114	35	79	24	91		
VW-10	1.2-1.5, 4.0-5.0	Dark gray fat clay (CH) with a trace of sand sizes	114	36	78	21	99		
VW-13	2.1-2.4, 7.0-8.0	Dark gray fat clay (CH) with a trace of sand sizes	100	31	69	18	98		
VW-17	0.3-0.6, 1.0-2.0	Dark gray fat clay (CH) with a trace of sand sizes	131	41	90	20	99		
VW-19	1.2-1.5, 4.0-5.0	Dark gray fat clay (CH) with a trace of sand sizes	111	33	78	21	99		
VW-22	1.2-1.5, 4.0-5.0	Dark gray fat clay (CH) with a trace of sand sizes	116	34	82	19	99		
	Sample								
BI-8[2]	1	Black plastic clay (CH) with sand	165	53	113		92		
BI-8	2	Plastic clay (CH)	98	32	66		95	647.2, 40.4	115.3
BI-8	3	Plastic clay (CH) with sand	78	25	53		92		
BI-8	4	Black sandy fat clay (CH)	69	25	44		85		

[1] Testing performed by South Atlantic Division Laboratory, Corps of Engineers.
[2] Testing performed by Waterways Experiment Station, Corps of Engineers.

Tensiometer readings are summarized in Appendix D of Palermo (1977a). Unfortunately, these instruments were installed outside the zone of initial wellpoint influence, and it is suspected that the instruments were damaged during severe cold weather in January 1977. Therefore, these measurements were not considered in the final data analysis.

Dewatering

A primary objective of the field demonstration was to determine if vacuum wellpoints could effectively dewater fine-grained dredged material. Hopefully, this would be accomplished by lowing the groundwater table and initially consolidating the dredged material adjacent to the wellpoint tip with the zone of influence of each wellpoint gradually expanding over an extended time period. Fifty-nine piezometers were installed to record variations in the groundwater table, and readings are summarized in Appendix D of Palermo (1977a). Water-table measurements were recorded periodically between April 1976 and April 1977 for piezometers 0-1 through 0-37 and between February 1977 and April 1977 for piezometers 0-38 through 0-57. In analyzing the piezometer data, readings on certain dates were considered more appropriate for determining the results of vacuum wellpoint groundwater table lowering. Primary production pumping was achieved between the periods of 23 November to 31 December 1976 and 28 January to 20 march 1977. Therefore, observation well readings made during these periods were compared with readings made before and after these periods.

Piezometers located in control areas indicated variations in piezometric levels ranging from reductions of 0 to 0.3 m to increases of 0 to 0.5 m. Piezometers located within the pumping areas indicated variations in piezometer level ranging from reductions of 0 to 0.9 m to increases of 0 to 0.09 m. Significant reductions in piezometric level were recorded at observation wells located close to wellpoints. Typical drawdown curves, developed from a composite of observation well readings, are shown in Figure 24.

A water meter was installed at the end of the collection sump discharge line to record flow rates developed from the vacuum wellpoints. Flowmeter readings are recorded in Appendix D of Palermo (1977a). Readings were obtained for the period of 22 February 1977 to 20 March 1977. During this period, the vacuum system operated continuously for 22 days. A total discharge of 15,100 L was recorded from the vacuum wellpoint system during this period for an average daily discharge of 688 L/day. If equal water flow is assumed from all 38 wellpoints installed (probably no the actual case), and a total daily discharge of 688 L/day is achieved, then a daily discharge of approximately 19 L per wellpoint can be assumed.

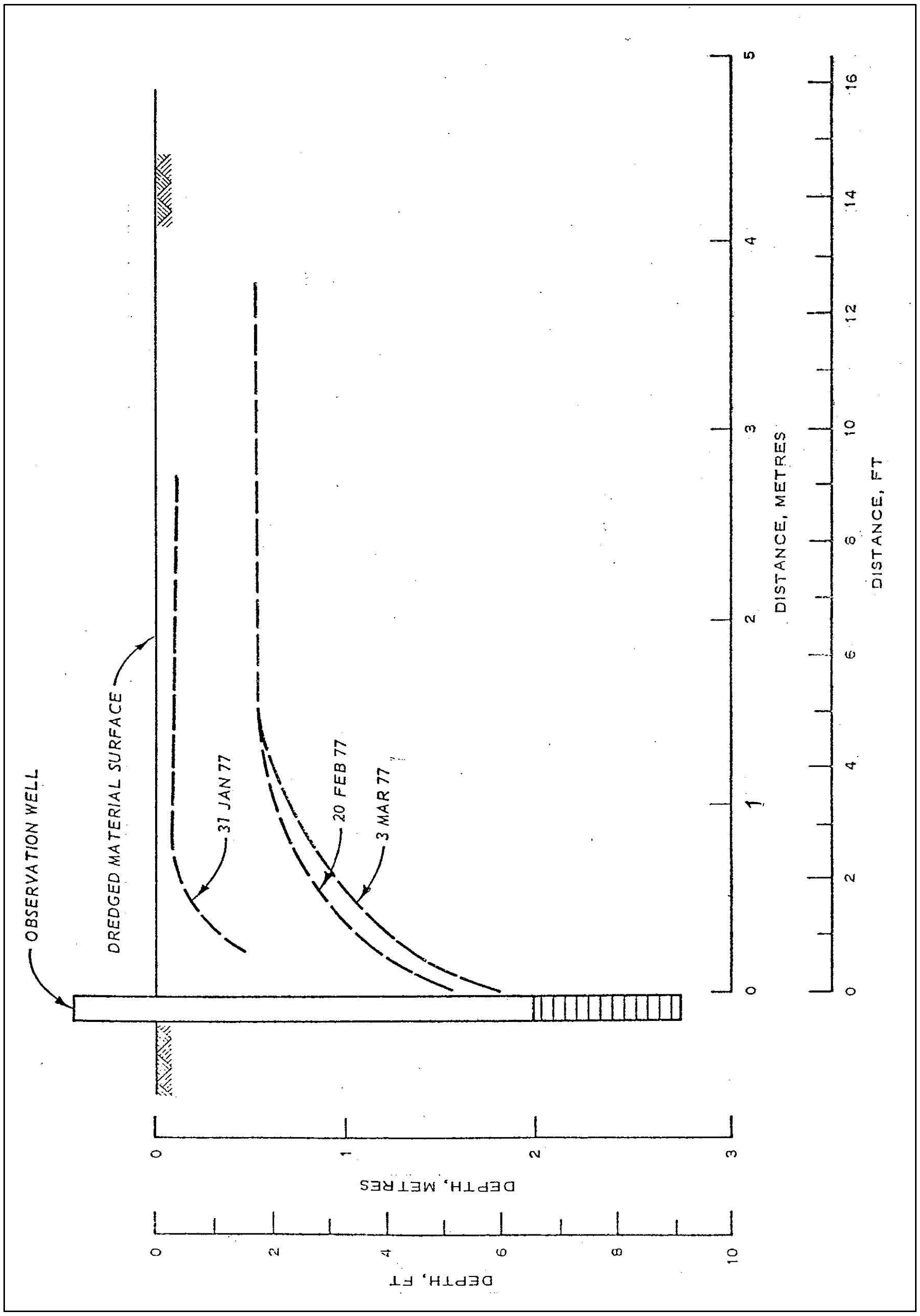


Figure 24. Typical drawdown curves

Water-content samples were obtained periodically to evaluate the effect of pumping on the water content of the dredged materials. Additional moisture samples were taken at distances of 0.3, 0.6, and 0.9 m from wellpoints A-11, A-21, C-2, and C-9. Moisture content results are summarized in Appendix D of Palermo (1977a).

Large periodic variations in water content were obtained for samples taken at depths of 0.3 to 0.6 m. Most of this variation may be attributed to monthly variations in rainfall and the amount of resultant surface ponding. Poor surface drainage allows infiltration of surface water through shrinkage cracks to recharge the upper subcrust material. The most noticeable reduction in water content was achieved close to wellpoints and at the 2.1- to 2.4-m depth, based on comparing the water contents of samples taken during the period 2 November 1976 to 29 March 1977 at locations VW-1 through VW-22. In Section A, the maximum water-content reduction approached 30 percent; the average reduction was approximately 9 percent. During the same period, comparative samples taken from Sections B, C, and D registered increases in water content. Comparison of test results from samples taken close to wellpoints A-11, A-21, C-2, and C-9 supports the above conclusions. Samples taken 0.3 m from wellpoints at the 2.1- to 2.4-m depth averaged reductions of 24 percent in water content during the period of 24 February to 29 march 1977. However, samples taken 0.6 and 0.9 m from wellpoints actually recorded increases in water content over the same period.

Cone-penetration measurements were obtained at many of the water-content sampling locations. These measurements are recorded in Appendix D of Palermo (1977a) and summarized in Table 14 and reflect a general increase in penetration resistance with depth. In addition, penetration measurements taken close to wellpoints A-11, A-21, C-2, and C-9 reflect significantly higher resistances within 0.9 m of the wellpoints. As indicated by Salem and Krizek (1973), these measurements should be considered ". . . as a rough approximation to the undrained shear strength . . ." and are submitted for relative comparison purposes.

Site consolidation

A primary objective of the field demonstration was to determine the effect of vacuum wellpoints on consolidation of dredged material. It was hoped that consolidation would be achieved from two conditions. First, the groundwater table would be lowered, increasing the effective stress from overlying material by removing the buoyancy force. Secondly, the vacuum system would create negative pore pressures in dredged material, thereby allowing the atmospheric pressure above the water table to act as a surcharge. If successful, the overall effective stress in the dredged material would be increased, resulting in consolidation and lowering at the surface.

A series of elevation readings were taken by surveyor's level and rod at the test site to determine the actual site consolidation. Level readings were taken on 3-m centers over the area. Actual elevations are recorded in Appendix D of Palermo (1977a).

Table 14
Summary of Cone Penetration Readings

Distance from Well - point	Average Penetration Reading at Depth of					
	0.9 m (3 ft)	1.2 m (4 ft)	1.5 m (5 ft)	1.8 m (6 ft)	2.1 m (7 ft)	2.4 m (8 ft)
Date: 2/10/7			**Cone Size: 51 mm (2 in.)**			
0.3 m (1 ft)	42	50	64	79	100	129
0.6 m (2 ft)	36	41	50	66	84	107
0.9 m (3 ft)	33	36	50	63	72	94
1.2 m (4 ft)	31	37	46	59	74	87
Date: 3/9/77			**Cone Size: 25 mm (1 in.)**			
0.3 m (1 ft)	35	46	52	64	74	89
0.6 m (2 ft)	32	35	42	51	60	68
0.9 m (3 ft)	28	34	39	45	53	63
1.2 m (4 ft)	27	32	39	45	53	57

Comparison of elevations taken in April 1976 and March 1977 indicates that the test-site surface subsided approximately 130 mm. This subsidence was rather uniform over the whole site, including control areas, and thus most of the subsidence can be attributed to natural consolidation of the dredged material induced by the groundwater table lowering described in Chapter 4. Comparison of elevations taken before and after the last pumping period should reflect any site consolidation that can be attributed to pumping with vacuum wellpoints. For this purpose, average changes in elevation (between these dates) for Sections A (6-m spacing), B (control), C (12-m spacing), and D (control) were computed. The average surface subsidence in Section A during this period was 37.1 mm. Similarly, the subsidence in Sections B, C, and D averaged 14.2 mm, 21.8 mm, and 20.1 mm, respectively. The average consolidation for control Sections B and D was 17.0 mm. Therefore, the net gain in consolidation for Section A was 20.1 mm. Similarly the net gain in consolidation for Section C was 4.8 mm.

Discussion of Test Results

Based on an average dewatering rate of 19 L/day (5 gal/day), it would take 53 days for each wellpoint to remove 1 cu m (1.3 cu yd) of water and thus create 1 cu m of reusable disposal-area volume. Each wellpoint could thus create 7 cu m of disposal area volume per year of continuous operation. For the UPB dredged material, approximately 21 days of continuous pumping would be required to reduce the water content of a cubic meter of dredged material initially at the liquid limit to about the plastic limit, assuming that the material was adjacent to the wellpoint and isolated from other effects. The actual volume of water removed from Section A during 22 February 1977 - 20 March 1977 was 11,290 L or 11.3 cu m. The volume created (based on average surface subsidence) in the 0.1-ha test section was approximately 18.5 cu m. For Section C, 4600 L of water was removed, or 4.6 cu m. Based on average surface subsidence, 5.9 cu m of volume was created.

The above comparisons are based on averages of elevation data collected at points spaced across the test sections and assumed average water flow for all wellpoints. Nevertheless, the order-of-magnitude agreement of volume created by the two separate sets of computations lends credence of reliability of both sets of data. Averaging the two results, 14.9 cu m storage was created in Section A, while 5.3 cu m of storage was created in Section C. The average ratio of storage created in Sections A and C is thus 2.8:1 while the ratio of wellpoints in the sections is 2.5:1. These results indicate that, during the period of measurement, each wellpoint acted independently, which is confirmed by the measured drawdown data shown in Figure 24.

Extrapolation of the test results for extended periods, which may or may not be warranted, would indicate that continuous vacuum wellpoint dewatering with 6-m spacing would create 2,500 cu m/ha-year of storage volume. For comparative purposes, the progressive trenching field demonstration (Chapter 4) produced an average storage volume gain of 2,250 cu m/ha over a 13-month test period. This comparison is not entirely valid because the long-term rates for vacuum wellpoint dewatering are unconfirmed and storage continued to accrue from the trenching study after measurements were terminated, but it serves to indicate that both may produce results of a similar magnitude. For wellpoint spacings closer than 6 m, more rapid rates should be expected while at a 12-m spacing, about one-third of the 6-m spacing dewatering rate should be expected.

Installation and Operational Costs

Wellpoint system installation costs have been summarized in Table 15. The cost analysis has been developed based on varying wellpoint spacings and method of installation. The estimates are based on the use of 50-mm PVC wellpoints and header pipes and 100-mm PVC collector pipes. Two methods of wellpoint installation were analyzed. The casing methods and the sandbag method were described in detail earlier. Cost estimates have been computed from data developed during installation of the test site.

In addition to the costs of buying and installing the vacuum system, the costs of power must be estimated. It is also assumed that operation of a vacuum wellpoint system on a routine basis would require constant attention. Therefore, the cost of maintaining a technician on-site should be included in any analysis.

Table 15 Cost Estimates for Installing Vacuum Wellpoints				
Wellpoint Spacing			**Cost of Materials and Installation**	
m	**ft**	**Method of Installing Wellpoints**	**$/acre**	**$/ha**
3.1	10	Casing	$22,800	$56,300
3.1	10	Sandbag	$14,000	$34,600
6.1	20	Casing	$ 8,500	$21,000
6.1	20	Sandbag	$ 6,000	$14,800
12.2	40	Casing	$ 3,900	$ 9,600
12.2	40	Sandbag	$ 3,250	$ 8,000

Assuming that the entire 34-ha UPB site was composed of fine-grained dredged material and was to be dewatered and densified with vacuum wellpoints, an estimated technician salary plus fringe benefits and overhead of $15,000 per year may be distributed over 34 ha to give a unit cost of $440/ha. The cost of material and installation for a 6-m spacing has been estimated at $14,800/ha. After review of observed vacuum-pump performance, it was estimated that the pump used in the study could effectively provide vacuum to dewater 1 ha. Capital costs for the pump and motor were $2,400, and with 1.4-kWhr power demand, 12,300 kWhr/ha would be required to power the system for 1 year. At $0.02/kWhr, the power cost would be $250/ha. The total cost for dewatering over 1 year would thus be approximately $18,000/ha ($7,300/acre), considering capital investment for the pump, motor, wellpoints, and piping, which are reusable. In a 1-year period, 2,500 cu m/ha of storage volume would be created. The estimated unit costs of creating storage volume are thus $7.20/cu m. If the system were operated for 2 years, the unit costs of creating storage volume would drop to $3.72/cu m, assuming that benefits would continue to accrue at the same note. If power costs exceeded $0.02/kWhr, the unit cost would be increased, but not appreciably, because the major cost item is for wellpoint materials and installation. Unit costs for closer spacing would be disproportionally higher because of material and labor costs, while greater spacing would result in disproportionally lower volume creation, also giving higher unit costs. Thus, a 6-m spacing appears closets optimum, compared to 3-m (10-ft) and 12-m spacings.

Summary

Technical feasibility

A review of published reports on the subject and data gathered from this field demonstration tend to support the conclusion that vacuum wellpoints are technically feasible for consolidating fine-grained materials. A major difference between this field study and previous applications was an attempt to employ widely spaced wellpoints and achieve long-term consolidation of the dredged material. Previous applications used horizontal and vertical sand drains to increase the consolidation rate. The net effect of eliminating the large area drains is to retard the dewatering rate; such behavior was observed to occur. Field observations and vacuum gage measurements indicate that PVC pipe can be reliably used to construct the vacuum system. This material has the advantages of being lightweight, relatively inexpensive, resistant to corrosion, and easy to assemble.

Results of periodic moisture tests and observation well readings indicate that dewatering was primarily confined to within 0.9 to 1.5 m of the wellpoints. However, as reflected in Figure 24, dewatering is time-dependent. This information, combined with results of the cone-penetration measurements, suggests that the radius of influence of the wellpoints was approximately 0.9 to 1.5 m after 22 days of continuous pumping. The radius of influence should expand further with continued pumping. Dewatering of the surface material was hampered by lack of surface drainage. During dry weather, shrinkage cracks developed in the surface of the dredged material. However, surface ponding during wet weather appeared to recharge the upper stratum of dredged material. Surface ponding was generally noticeable at wellpoint locations. Saucer-shaped depressions developed at several wellpoints and collected surface water.

Results of surface elevation measurements indicate that pumping with vacuum wellpoints did produce additional consolidation above that amount measured at the control sites. This consolidation should be attributed to atmospheric surcharge and lowering of the groundwater table.

Economic feasibility

The magnitude of reusable disposal volume created and the estimated costs of installation and operation indicate that vacuum wellpoints would not be economically feasible when compared to similar rates of volume gain produced by the progressive trenching concepts of Chapter 4, but at one-sixth the unit cost of vacuum wellpoint dewatering. To make vacuum dewatering and consolidation more nearly cost-effective, either the cost of system material and installation must be markedly reduced or the dewatering rate must be increased. Low-cost materials were used in the system, and the 6-m spacing appears to be close to optimum for conventional wellpoints. Increased efficiency and thus reduced unit dewatering costs may lie in installation of drainage and vacuum blankets prior to disposal (see Chapter 9) or in using other methods to install drainage layers of larger areal extent in previously-placed fine-grained dredged material (see Chapter 7).

Operational problems

Certain operational problems must be faced prior to routine use of a vacuum wellpoint or any vacuum dewatering system. First, a reliable power source must be available. The often remote locations of confined disposal areas may require use of an independent power source. For this field study, diesel generators proved to be the most reliable power source.

Maintenance of pumping equipment was another major problem in conducting this field demonstration. For the size of the experiment chosen, most laboratory-type vacuum pumps were too small while the vacuum pumps normally used in full-scale wellpoint dewatering systems were too large. A large size laboratory-type high-vacuum pump was used, but it did not have adequate resistance to the corrosive environment. These problems might be resolved in full-scale applications by using large conventional industrial-rated vacuum pumps. However, continuous maintenance of any pumping equipment will probably be required.

Operational problems were also experienced with the electrical system which controlled cycling of the vacuum and water discharge pumps, which were designed to allow continuous operation of the system. This control system was improved during the conduct of the experiment. However, routine pumping operations should not be dependent upon sophisticated and non-weatherproof control systems.

Any routine use of vacuum wellpoints should include provision for constant monitoring by competent technical personnel. Generators and pumps require frequent maintenance and are subject to breakdown when operated continuously. Provisions such as ditching will be necessary to minimize surface-water ponding. A vacuum wellpoint system should not be considered unless the user is prepared to provide equipment and personnel to maintain operation of the system.

Conclusions and Recommendations

Based on the results, analysis, and discussion described herein, it may be concluded that:

a. Technical feasibility or non-feasibility of using windmill-powered generation systems to provide electrical power at remote disposal-area locations was not positively established. Because of mechanical, operational, and maintenance problems encountered, the demonstration suggests that field use will be impractical until the reliability of current state-of-the-art equipment is markedly improved.

b. Vacuum wellpoints appear to be a technically feasible methods of dewatering and consolidating fine-grained dredged material after placement in confined disposal areas and a spacing of 6 m appears to be close to optimum for any full-scale applications.

c. PVC pipe is a practical material for use in construction of vacuum wellpoint systems. It has lighter weight, lower cost, and higher corrosion resistance than comparable steel piping. It is easy to cut and glue on-site, and it maintains vacuum without difficulty.

d. Based on estimated unit storage volume costs of $3.70 to $7.20 per cu m for a 6-m spacing, to produce 2,500 cu m/ha-year of storage volume, conventional vacuum wellpoints are not cost-effective when compared to the unit costs for progressive trenching of $0.82/cu m, which creates storage volume at approximately the same rate.

e. The major cost item in vacuum wellpoint operation is for piping and the labor required to fabricate and install the wellpoints and header system. These costs will scale up directly in full-scale application. Lower unit dewatering costs may be thus realized by increasing the dewatering rate of the vacuum wellpoints. Alternatives available may include use of the wellpoints in conjunction with free-draining layers of larger areal extent, such as underlying granular layers and sand blankets placed prior to disposal, surface membrane-covered sand blankets placed after disposal, or drainage lenses created in existing dredged material by hydraulic fracture.

It is recommended that conventional vacuum wellpoint dewatering of fine-grained dredged material placed in confined disposal areas be attempted only if progressive trenching dewatering techniques cannot be employed. Every attempt should be made to incorporate any existing drainage layers or lenses into the system. CE Districts and other interested users should be aware of the need for continuous monitoring and periodic maintenance of equipment used to provide power and pumping capacity.

6 Capillary Wick Dewatering Field Demonstration

In March 1975, WES conducted preliminary experiments which suggested that capillary wicks inserted into fine-grained dredged material might increase the rate of dredged material drying and densification, through capillary wick attraction and transfer of internal pore water to the dredged material surface. Based on these preliminary findings, this research was initiated in October 1975 with immediate goals of (a) developing an analytic mechanism to explain wick behavior, and (b) conducting a comprehensive literature survey on previous uses of capillary dewatering devices. Results of this effort were presented to invited technical experts and consultants at DMRP Planning Seminar II in January 1976 (DMRP 1974). As a result of this presentation and subsequent discussion, it was recommended that this research be extended to undertake laboratory evaluation of potentially applicable wick materials to determine if wicks suitable for field evaluation were commercially available or could be developed. As a result of this further research, acceptable wick materials were identified and tested, and a field demonstration was initiated in the fall of 1976 at the UPB disposal area.

Initial Wick Dewatering Experiment

To qualitatively evaluate the concept of capillary wick dewatering, a small experiment was carried out during March 1975 in which three plastic 23-L buckets were filled with a fine-grained dredged material slurry having an initial water content of 202 percent. Bucket 1 was designated as an undisturbed control while Bucket 2 contained a vertical wick made by stapling absorbent paper toweling to a lath. Evaporation could occur from both the exposed horizontal surface of the dredged material and the vertical faces of the wick. Bucket 3 also contained a vertical wick, but the bucket was covered by a plastic lid with only the vertical wick surface protruding above the lid exposed for evaporation. Daily water-loss measurements were made for a 2-week period. Buckets were exposed to normal Vicksburg, Mississippi, warm spring climatic conditions and were protected from rainfall. The water-loss rate for the three treatments is show in Figure 25. The uncovered bucket containing the wick had the fastest rate of water loss, and the covered bucket containing the wick also lost and appreciable quantity of water. During the 2-week test period, the initial 202 percent water content was reduced to an average water content of 100 percent for Bucket 1 (control), 60 percent for Bucket 2 (uncovered wick), and 160 percent for Bucket 3 (covered wick).

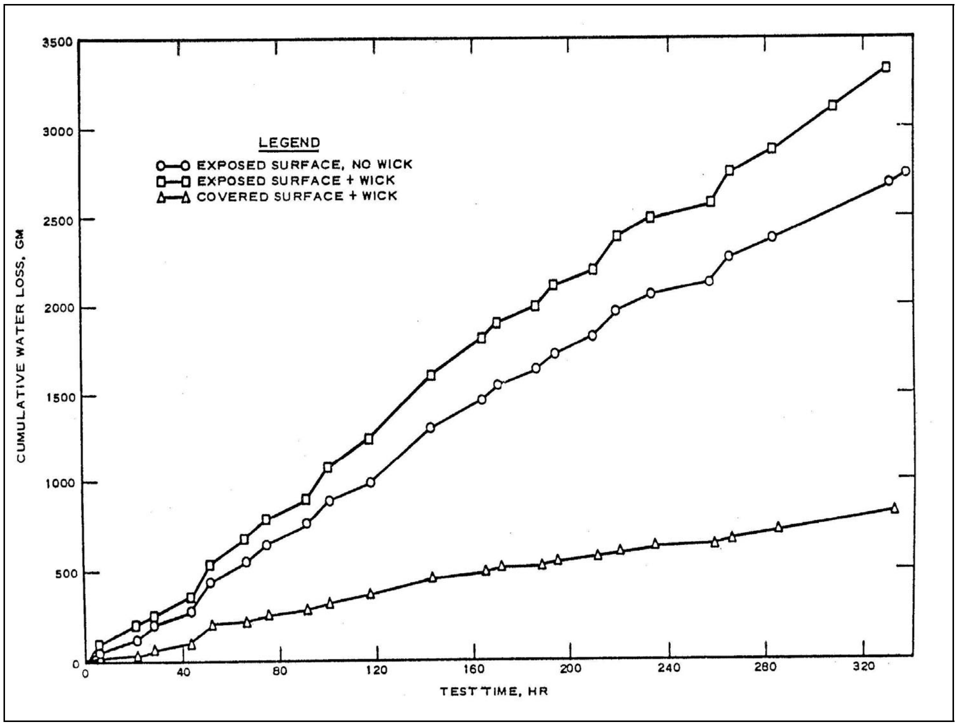


Figure 25. Water loss versus time for the three treatments in initial wick dewatering experiment

Conceptual Basis for Wick Behavior

As a result of the preliminary experiments plus discussion by the technical experts and consultants at the DMRP Planning Seminar II, conceptual relationships were developed for expected capillary wick dewatering behavior:

a. Capillary wicks inserted into fine-grained dredged material immediately after deposition and initial sedimentation should act as vertical drains to increase the rate of internal excess pore water pressure dissipation by shortening the effective drainage distance and should thus promote more rapid self-weight consolidation. Preliminary calculations (Spotts 1977) indicate that order-of-magnitude consolidation rate increases are theoretically possible with closely-spaced wick drains.

b. Once self-weight consolidation is completed, wick capillary attraction should collect free pore water from the dredged material and transport this water vertically upward a maximum distance equal to the height of wick capillary rise above the existing dredged material water table. If the height of capillary rise is such that the water is brought above the dredge-material surface, the additional wet exposed surface area of the wick would provide more evaporative surface and should increase the total evaporative water loss of the dredged material through the horizontal surface, the sides of dessication cracks, and the exposed wick surface area. This behavior should continue as long as the height of capillary rise is above the dredged material surface and may also continue after internal drying lowers the water table below the height of capillary rise if a desiccation crack forms around the wick.

c. Fine-grained materials, especially those with high plasticity, have relatively small pore sizes and thus considerable capillary suction potential in the unsaturated state. Capillary wicks should be effective in removing water from fine-grained dredged material only when the capillary suction of the wick is greater than the capillary suction of the soil. Under these conditions, it is expected that wicks will work most efficiently at high water contents with efficiency dropping as internal water content decreases. The wicks may become relatively ineffective once equivalent pore size in the dredged material is below equivalent pore size in the wicks. This behavior is similar to that of plant roots. The soil-water content at which roots can remove no further moisture from the soil is called the wilting point. Obviously, the higher the height of wick capillary rise, the higher the amount of equivalent capillary suction, and the lower the equivalent wilting point for the wick.

d. Capillary wicks are essentially a relatively inefficient form of deep-rooted vegetation. However, the wicks go where placed and may continue to function throughout the year whereas the plant roots have a tendency to go no deeper than necessary and are essentially ineffective during periods of vegetation dormancy.

More detail on wick dewatering concepts is available in Spotts (1977).

Preliminary Wick Identification and Selection

Results of a comprehensive literature search (Spotts 1977) revealed that previous use of capillarity as a fine-grained material dewatering mechanism was essentially non-existent and that no previous experiments on any scale had been carried out relative to capillary wick dewatering. Further, no criteria were available for appropriate selection and/or evaluation of potential wick materials for field use.

After considerable study, it was decided that an appropriate field-use capillary wick would have a capillary rise height greater than or equal to 200 mm and have resistance to biodegradability and climatic exposure. No standard test methods were available for the evaluation of prospective wick materials, so original test procedures were devised. The capillary-rise height was measured by immersing a section of the proposed wick material in distilled water and measuring the height of capillary rise after equilibrium had been achieved. A test for biodegradability was developed in which the prospective wick material was repeatedly saturated with extracted dredged material pore water and allowed to dry while climatic exposure resistance was tested by subjecting wick materials to alternating ultraviolet light, sunlight, and dark conditions. More detail on specific test procedures is available elsewhere (Spotts 1977). Surfactants were applied to many of the wick materials, but the effect of such application was usually to reduce the height of capillary rise.

Results of a large number of height-of-capillary-rise tests are shown in Figure 26, where height of capillary rise is plotted versus Rise Rate Coefficient K, an arbitrary parameter which increases with increasing speed of capillary rise (Spotts 1977). As noted from this figure, soil columns with relatively small pore openings, had the highest capillary-rise potential, but also the slowest equivalent flow rates. Various other paper, wood, and natural and artificial fiber materials were also tested, and their results fell into relatively well-defined groups. Their larger pore openings resulted in lower height of capillary rise but a faster rise rate. In general, natural fibers such as paper and wood suffered from biodegration, while artificial fibers were affected by exposure to sunlight (i.e., ultraviolet radiation).

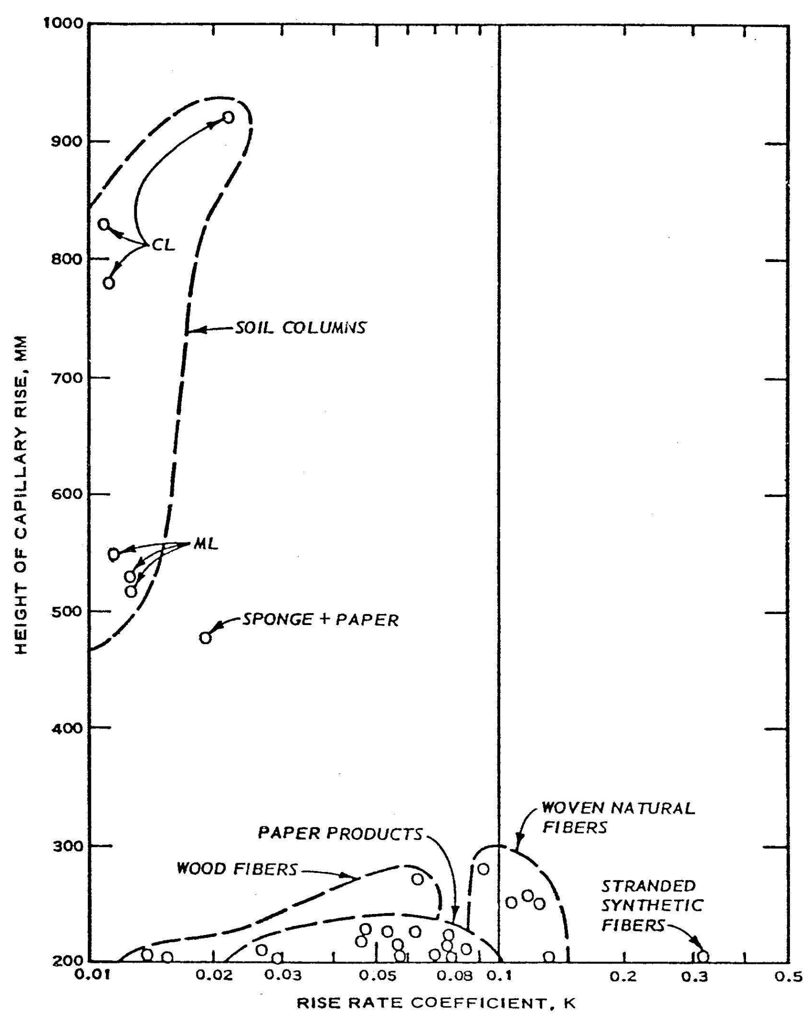


Figure 26. Results of height of capillary rise testing for various wick materials

As a result of laboratory evaluation, three different wicks were found to be appropriate for field evaluation:

a. The most promising material was a combination of natural and artificial fibers considering of a three-ply fabric approximately 2-mm thick and composed of 50 percent wood and 50 percent blended polyester and polypropylene. The fabric plys were heat-bound by rectangular spot fusions approximately 2 by 1.5 mm spaced on a grid pattern 3 mm. This combination material was manufactured especially for the wick test by International Paper Company in strips 150 mm wide and 1.8 m (6 ft) long. This material was furnished to the Government at no charge.

b. A natural softwood kraft paper produced by the James River Paper Company, manufacturer's designation n.k. R9/5634/F, gave the best overall laboratory test results from the various paper wicks tested. Material for use in a field test was furnished for use by the James River Paper Company, at no cost to the Government, in a continuous roll 150-mm wide and 1-mm thick.

c. To evaluate the wicking potential of low plasticity soils, a combination soil wick was fabricated. The soil-containment tube was made of one-ply fabric about 0.6 mm thick and composed of 30 percent wood fiber and 70 percent polypropylene. This fabric was formed into a closed tube approximately 50 mm in diam and 1.8 m long by heat sealing the length seam and closing one end. These operations were performed by the fabric manufacturer, International Paper Company, who furnished the material at no cost to the Government. A small wood lath was inserted into the tube to provide vertical stiffness, and the tube was then filled with Vicksburg löess having a USCS classification of CL. More data on soil properties are available elsewhere (Spotts 1977).

Field Evaluation of Promising Wick Types

Test site construction

The three promising wick types determined from laboratory evaluation were evaluated under field conditions at the UPB disposal area. A test site was chosen in the southeast corner of the disposal area, as shown in Figure 2. The sand mound placed in this portion of the disposal area by previous dredging (Chapter 2) was chosen as a test site location because it was desired to evaluate wick behavior starting from initial deposition of dredged material in slurry form. At the test site, an excavation was made in the sand and six test pits 6.1 m square and 1.8 m deep with 19-mm plywood sides were constructed. The sides and bottom of each pit were lined with nylon-reinforced black polyethylene sheeting (Giffolin T-55) to maintain imperviousness. The wick-evaluation test layout is shown in Figure 27. The six test pits were to be used in evaluation of three different wick materials plus a control pit and allowed evaluation of different spacings for the most promising material.

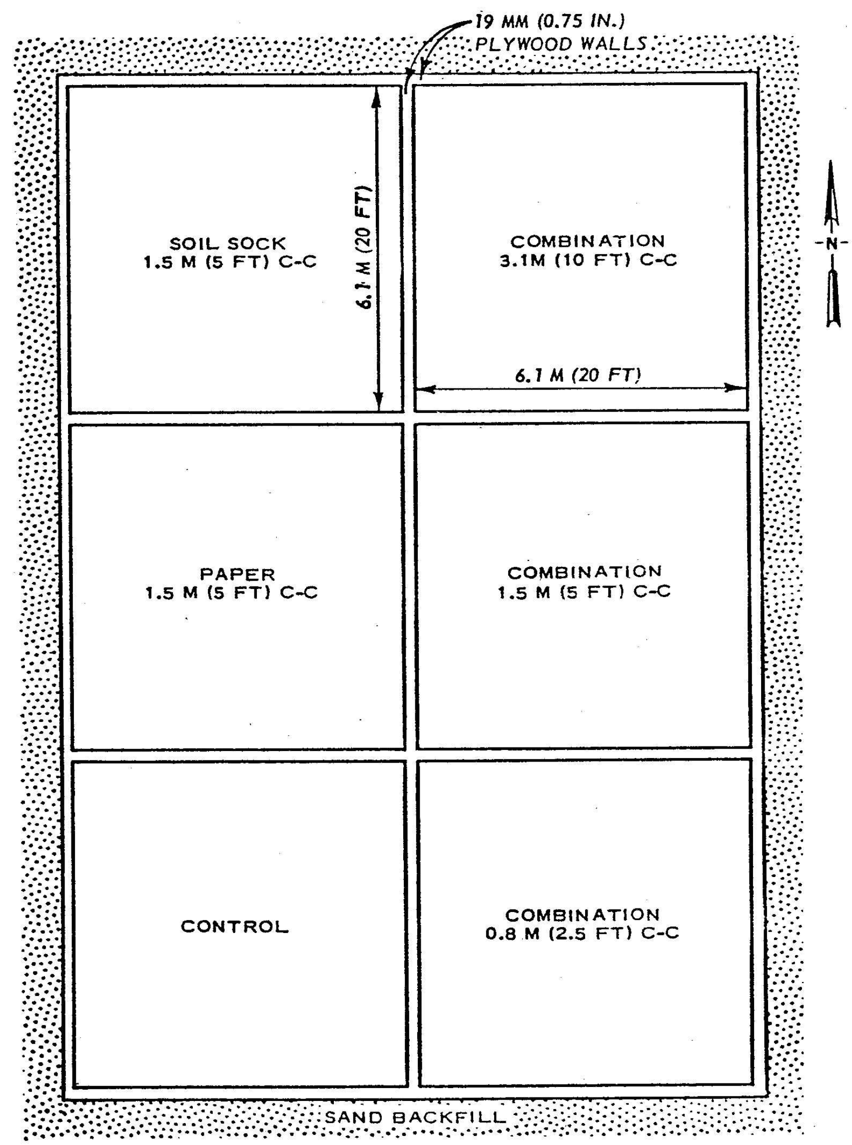


Figure 27. Layout detail for capillary wick field demonstration

The wick-test pits were filled with dredged material slurry provided by a 200-mm Mudcat dredge, which pumped dredged material hydraulically from a borrow site at the southwest corner of the disposal area. This dredge was obtained to fill the test pits for the Underdrain Dewatering Field Demonstration, and more detail concerning the dredging operation is given in Chapter 9. The material moved by the dredge was a black, highly plastic CH clay with general engineering properties corresponding to those obtained in site characterization described previously in Chapter 3. Additional test data for the material dredged is given in Chapter 9.

As the dredged material was provided in slurry form at solids contents between 5 and 30 percent (weight basis), it was necessary to fill the pits to the design elevation incrementally in order to fill with solids. A sluice arrangement was used to divert dredged material slurry to the different wick-test pits. Incremental filling was accomplished by filling the pits completely full of slurry, allowing sedimentation, which took approximately 24 hr, pumping off the clear supernatant, and then refilling the pits, allowing sedimentation, etc. This filling operation was scheduled in conjunction with test-pit filling and was accomplished during the period 30 September 1976 to 12 October 1977. The final depth of settled solids in all test pits was approximately 1.5 m.

Wick insertion

Reference string lines were laid across the top of each test pit to determine locations for wick placement. Paper and combination wicks were installed by double-folding one end at 45-deg angles and stapling the folds. A wooden pole was inserted into the pocket thus created and used to push the wick vertically through the slurry. The pole was then extracted, leaving the wick in place with approximately 0.3 m of wick material protruding above the dredged material surface. This exposed length was then stapled to the alignment strings stretched over the test pits. Soil sock wicks were pushed vertically to the bottom of the slurry using the wood stiffening strip contained in each wick. Each soil-sock wick was also secured to alignment strings. All wicks were installed during the period 14-15 October 1976.

Data collection

Piezometers made from 13-mm ID plastic tubing were placed in each test pit. In order to prevent surface discontinuities, these tubes were placed into the test pit through the plywood walls and monitored periodically to determine the internal water level and the presence of any excess pore pressures from self-weight consolidation. Data collected during wick evaluation included relative elevation of the dredged material surface, water content with depth, and cone-penetration index with depth. Water-content samples were taken with the Hayden Slurry Sampler (described in more detail in Chapter 9) at locations 0.3 m below the material surface, approximate mid-depth of the dredged material layer, and 80 mm above the bottom in each test pit. Cone penetration index, a WES-developed method of indicating relative soil strength, was determined periodically for depths of 0, 150, 300, 600, 900, and 1,200 mm below the dredged material surface. The relative height of solids was determined by periodic elevation measurements of the dredged material surface, referenced to a permanent bench mark installed nearby. A beam walkway was constructed across the test pits in order to allow data collection and sampling from each test pit interior without causing surface disturbance.

Operational problems

Considerable difficulty was encountered during the first three months of the experiment in removing surface water, caused by a combination of dredged material consolidation and precipitation, from the wick test pits. Initially, vertical slots were cut into the sides of the plywood test pits to allow surface drainage. However, dredged material below the surface simply flowed against the slot and effectively plugged it while negative skin friction against the sides of the pits resulted in more surface settlement toward the center of each test pit. Water continually ponded in the resulting saucer-shaped depression. This problem was resolved by laying strips of wick material horizontally on the surface and out through a slot cut in the side of each test pit. These wicks removed standing surface water and allowed desiccation-crack formation to begin. Once desiccation cracks and resulting surface drying had stabilized the upper material, water drained through the desiccation cracks to the edge of the test pits and exited through the slots.

Within a few days after kraft paper-wick installation, the majority of the wicks had torn at the surface of the dredged material and separated from the upper portion which was dangling from the alignment strings. Investigation revealed that upon being saturated, the paper wicks had extremely low strength. Wind action caused flexure of the wicks which were effectively supported at the alignment strings and at the soil surface. This flexure resulted in fatigue failure of the wicks at the soil surface. The paper wicks were reinstalled and the problem reoccurred. At this point, the wicks were simply left with their top surfaces flush with the dredged material.

Test results and discussion

At the time of report preparation, wicks had been in place for approximately 235 test days. Surface settlement in the wick test pits and the control pit is shown, plotted versus test time, in Figure 28. As noted in this figure, little difference in settlement has occurred among the various wick types and spacings and the control pit. The soil-sock wicks have apparently produced the greatest rate of settlement. It should be noted that the kraft paper wicks, as described previously, were torn shortly after installation and did not project above the dredged material surface.

Water-content data for the various wick treatments and the control pit are shown in Figure 29, for the top, middle, and bottom zones of the test pits. As may be seen from this figure, there is no marked difference in water-content behavior among any treatment, including the control section. The combination wicks at 0.8-m spacing appear to have slightly lower water content than other treatments, but the trend is not clear, and it may be the result of normal sampling variation.

Periodic cone penetration with depth measurements reveal the same trends (i.e., no marked difference between penetration resistance with depth in any test pit). These data are given elsewhere (Spotts 1977).

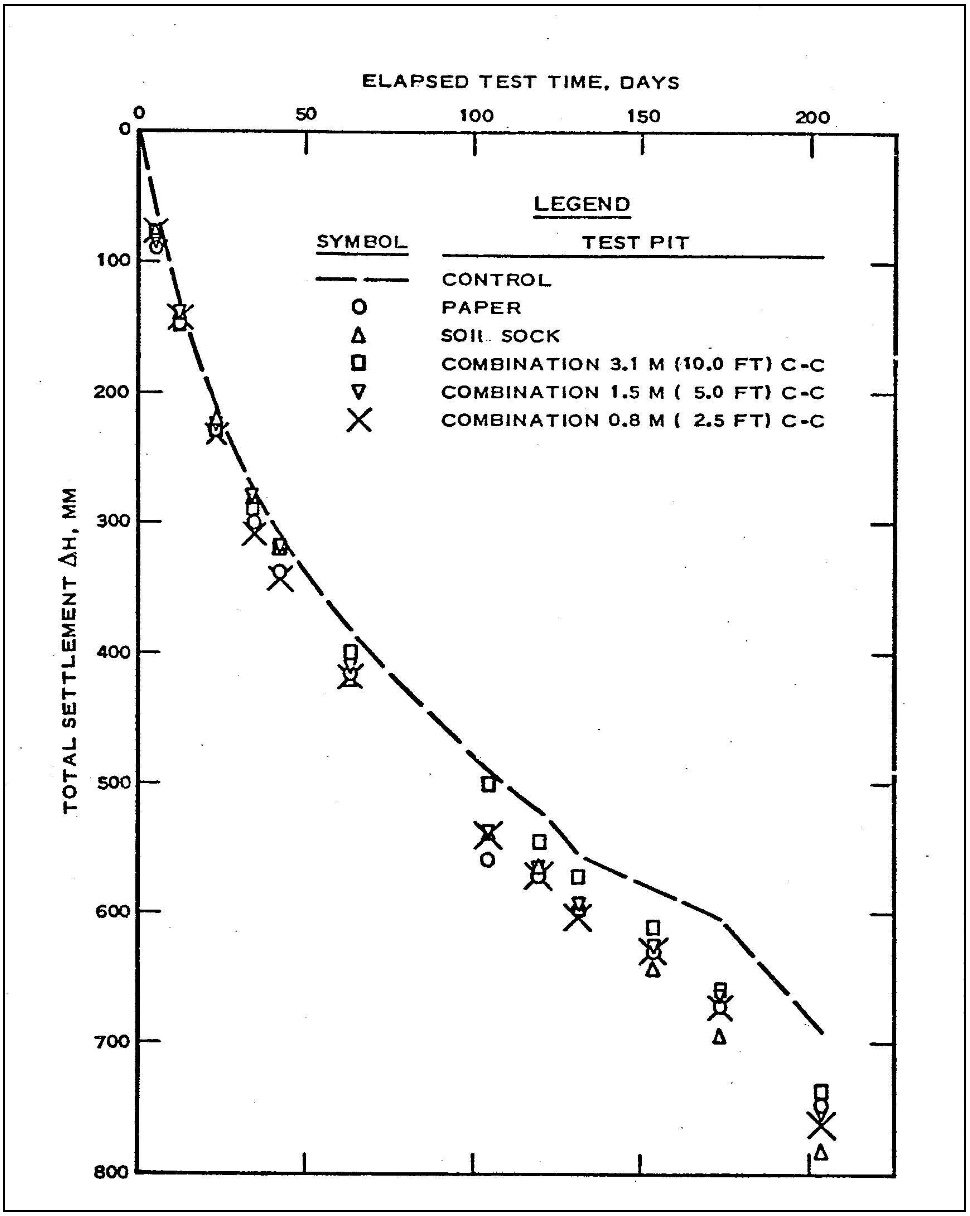


Figure 28. Dredged material surface settlement with time for various wick types and spacings

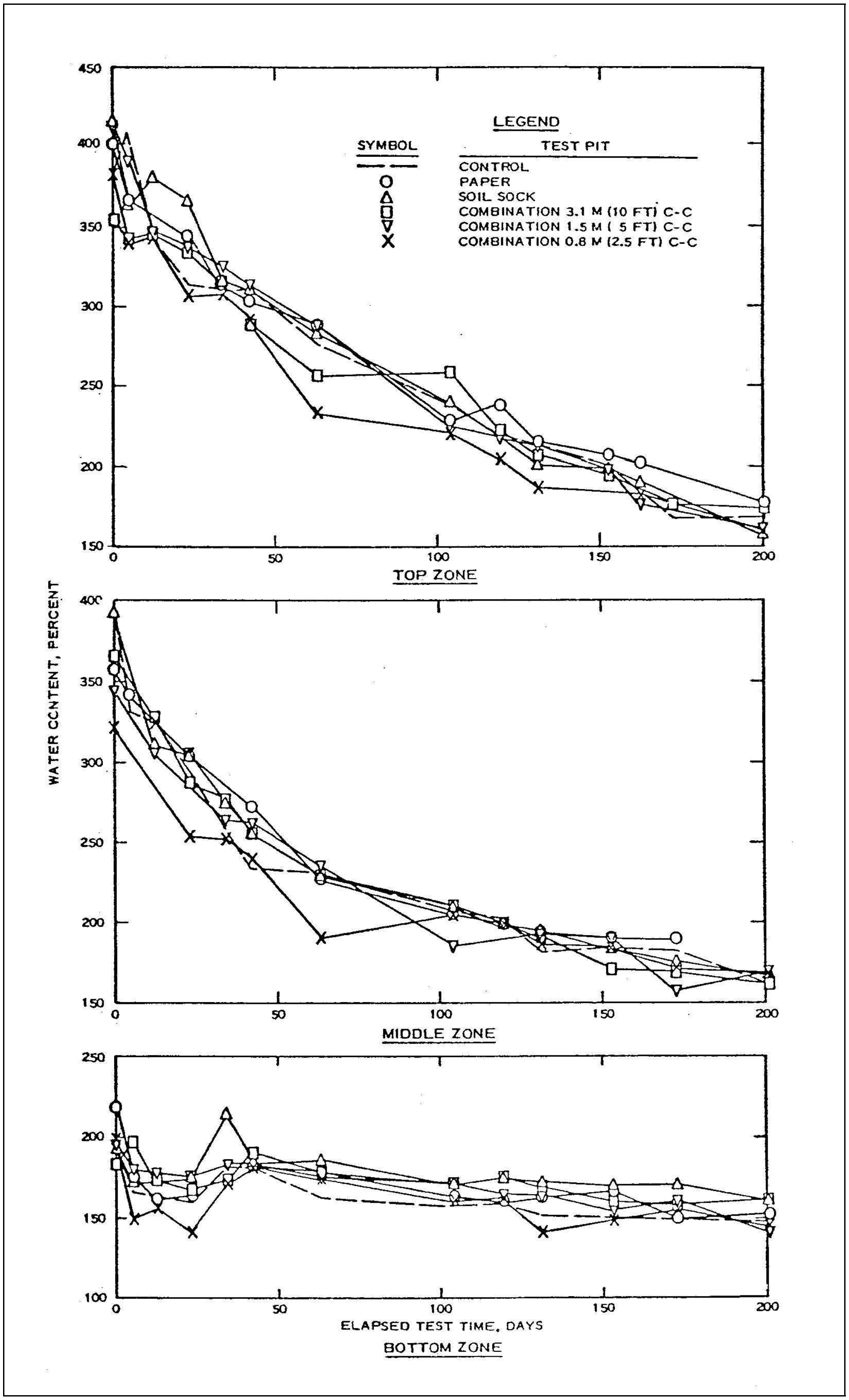


Figure 29. Water content versus time behavior in top, middle, and bottom zones of wick test pits

Initial concept development for wick behavior postulated that wicks should act initially as drains to accelerate the rate of dredged material self-weight consolidation and then, once self-weight consolidation occurred, should begin to transport free pore water to the surface by capillary action. As free pore water was transported, with resultant dredged material drying, wick effectiveness was expected to diminish as it became more and more difficult to extract water from soil pores by capillary suction. Further, as dredged material densification took place, capillary water-retention ability of the material would be increased.

Computations made elsewhere (Spotts 1977) indicated that a wick spacing of approximately 0.8 m would result in a order-of-magnitude increase in dredged material consolidation rate compared to a similar layer without wicks. Further, supposedly conservative calculations made elsewhere (Spotts 1977) indicated that each wick might be expected to remove up to 3.8 L of water per day from fine-grained dredged material by capillary suction. Verification of the increased consolidation rate by improved drainage hypothesis could be verified by an experimentally observed increase in rate of surface settlement in the various wick-treatment pits, as opposed to the control pit, with the 0.8-m wick spacing production the fastest rate of settlement. Also, if each wick caused evaporation of 3.8 L of water per day, then each wick could evaporate a maximum of approximately 890 L during the test period. Approximately 1,000 L of water must be removed to create 1 cu m of disposal area volume. Even assuming that the effective wick production rate would be considerably less than 3.8 L/day, any appreciable removal of water by capillary wick action should result in a measurable decrease in water content and should increase in the amount of dredged material surface settlement.

A review of the various data indicate that, for practical purposes, there was no difference in the rate of initial surface settlement among all treatments, including the control section. The wicks thus did not accelerate initial self-weight consolidation of the dredged material. Further, as self-weight consolidation excess pore pressures begin to dissipate, there was no significant increase in the rate of either water loss or dredged material surface settlement in the test pits where capillary wicks were placed. The greatest surface settlement appears to have occurred in the section containing CL soil wicks. This material had the smallest pore openings and the greatest height of capillary rise in laboratory tests. Thus, it would be expected to have the greatest capillary-suction potential and the most chance of attracting free water from the fine-grained dredged material mass. However, the increase in surface settlement and water-content reduction noted from this wick type was, for engineering purposes, inconsequential. The other wicks, with greater effective pore size and less capillary suction potential, do not appear to have been effective in removing water from the dredged material under optimum conditions and may be expected to perform even less efficiently as water contents decrease.

Conclusion and Recommendations

Based on the procedures, results, and discussion presented herein:

a. Capillary wicks were found not to be technically feasible as a method of increasing the rate of self-weight consolidation of fine-grained dredged material slurry placed hydraulically.

b. Capillary wicks were found not to be technically feasible as a method of promoting and increasing the rate of dredged material drying after placement, sedimentation, and self-weight consolidation. Apparently, the capillary suction potential of the wicks is not strong enough to overcome the capillary retention potential of the fine-grained dredged material, even at relatively high water contents.

The use of capillary wicks by CE field elements or other interested agencies to increase the rate of fine-grained dredged material self-weight consolidation or to increase the rate of dredged material drying is not recommended.

7 Pressure-Injected Sand-Slurry Field Demonstration

At the Dredged Material Research Program (DMRP) Planning Seminar I, held at the Waterways Experiment Station (WES) in October 1974 (DMRP, 1974) to discuss both conventional and innovative methods of dewatering fine-grained dredged material placed in confined disposal areas, it was suggested that sand grout or slurry be pressure-injected into dredged material, hydraulically fracturing the mass and creating internal drainage layers. These drainage layers could be subsequently pumped to provide both internal drainage and vacuum consolidation of the dredged material. This suggestion was discussed at some length by the DMRP staff, and when the Upper Polecat Bay (UPB) disposal area became available for evaluation of DMRP research, a small-scale field feasibility demonstration was planned to evaluate the technique. This demonstration was conducted during February 1976.

Test-Site Location

The test site was located in the northwest corner of the UPB area, as shown in Figure 2. At the test location, a surface crust approximately 0.15 m thick existed over approximately 2.1 m of fine-grained dredged material. Below the crust, material existed at natural water contents at or slightly above the liquid limit and with average geotechnical properties as summarized in Chapter 3. The initial water table location was the base of the crust, or 0.15 m below the surface. Ponded surface water had been removed from the test area in November-December 1976.

Test Program

Sand slurry was pressure injected into the material at six locations, as shown in Figure 30. Also shown in this figure are the locations of four control section tip wellpoints conventionally installed in the dredged material and subsequently pumped for comparative purposes. The sand-injection locations were numbered 1 through 6. After sand-slurry injection, section tip-wellpoints were placed into the injected sand masses and pumped. The wellpoints bear corresponding numbers. The control wellpoints were numbered 7 through 10.

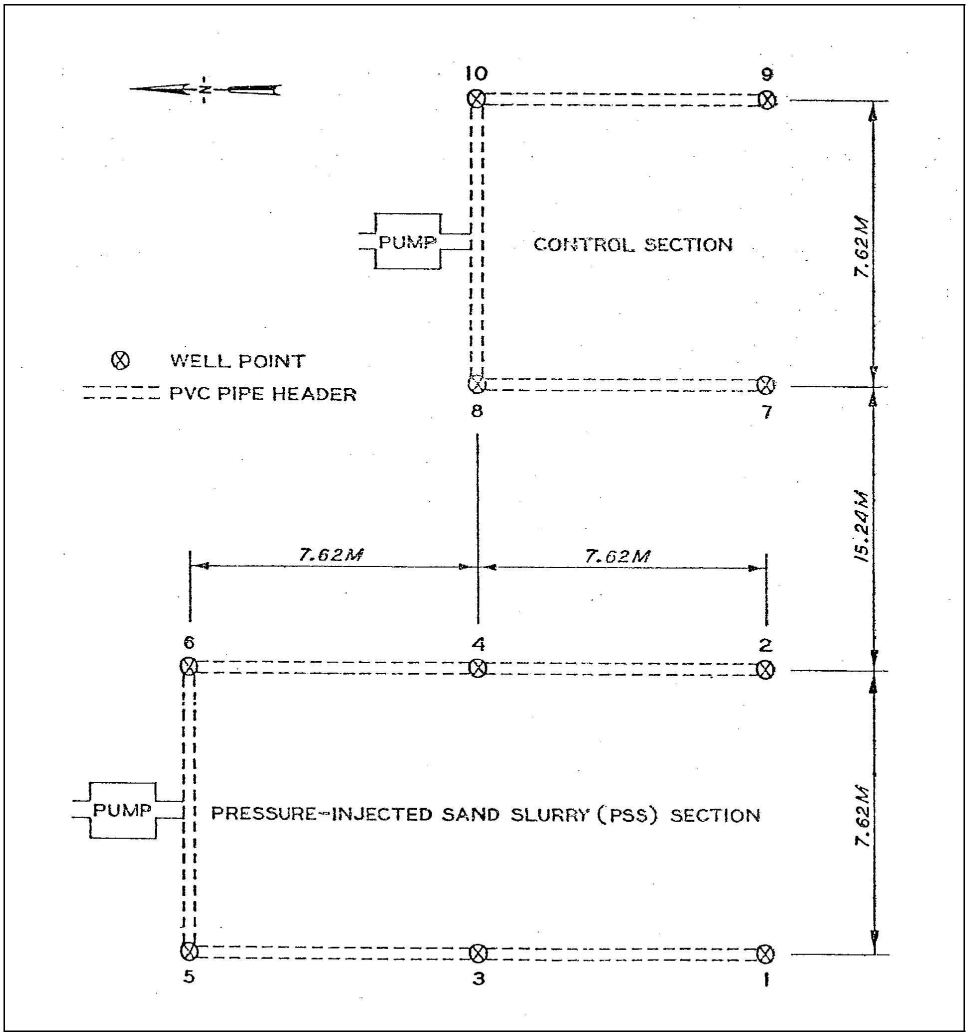


Figure 30. Sand injection test section and control section layout

Sand-injection procedure

Prior to sand injection, 76-mm-diam ABS plastic pipe was pushed by hand to the proper depth for use as casing to control the fracture location. Cement grout plugs about 0.5 m in diameter and 0.15 m thick were placed around the casing at Locations 1, 2, 3, and 4 to resist potential uplift forces generated by the pressurized slurry. Locations 5 and 6 had no grout plugs.

The sand slurry was prepared by mixing masonry sand, water, and Revert, a product of the Johnson Well Screen Company. Revert temporarily increases the viscosity of water to that of a bentonite-water gel. Approximately 72 hr after mixing, the viscosity returns to that of water. Revert is normally used in water-well drilling and completion when formation plugging might result from use of bentonite or other drilling mud. The mortar sand was purchased commercially in Mobile and had a gradation of 96 percent passing the U.S. No. 20 sieve, 46 percent passing the U.S. No. 40 sieve, 5 percent passing the U.S. No. 80 sieve, and 0.5 percent passing the U.S. No. 200 sieve. It may be described as a fine, poorly-graded clean sand with a USCS classification of SP. The sand met standard Corps of Engineer filter criteria for use with cohesive soil. Slurry was mixed 110-L batches. Initially a mixture of one part sand and two parts of Reverted water was used and injected at Locations 1 and 2. Midway through injection at Location 2, a thicker slurry, consisting of one part sand and one part Reverted water, was mixed and used at the remainder of the injection locations. The reason for use of a thicker mixture was to keep the sand in slurry suspension longer, allowing it to be pumped a greater horizontal distance at each injection point.

A Moyno positive displacement pump was used to inject the sand slurry into the dredged material. The pump was capable of providing pressures in excess of 690 kPa. However, pressures of such magnitude were not required during injection. Sand slurry was injected into the dredged material at foundation level (2.3-m depth) except at Location 5. Location 5 was injected at a depth of 1.2 m until slurry emerged around the casing at the surface. The casing was then pushed to the 2.3-m depth and injection continued.

A variety of experiments were conducted at the six injection locations. The various combinations of grout collar casing, injection tip geometry, sand-slurry composition, depth of injection, and volume of slurry injected are shown in Table 16. In each instance, a pipe collar was attached to the 76-mm ABS casing and then connected to the Moyno positive displacement pump. During injection, a continuous column of sand slurry was not maintained inside the casing because an excess head of approximately 0.3 - 0.6 m above the dredged material was sufficient to cause fracture and allow the sand slurry to enter. Instead, the positive displacement pump moved the sand slurry to the top of the casing where it fell freely down to a point 0.3 to 0.6 m above the surface and then gradually flowed into the dredged material mass. Injection was continued at each location unit noticeable slurry return was observed at the surface. Specific occurrences at each location are summarized as follows:

Table 16
Experiment Arrangement for Pressure-Injected Sand-Slurry Demonstration

Injection Location	Grout Collar	Injection Casing Tip Geometry	Sand Slurry Compositio n	Depth of Injection, m	Suction[1] Tip	Wellpoint Cased	Volume of Slurry Injected, L
1	Yes	Flat	Thin	2.3	a	Yes	397.5
2	Yes	Flat	Thin/Thick	2.3	a	Yes	1,211.3
3	Yes	Slotted	Thick	2.3	a	Yes	397.5
4	Yes	Slotted	Thick	2.3	b	Yes	1,438.5
5	No	Flat	Thick	1.2, 2.3	a	No	1,249.2
6	No	Flat	Thick	2.3	a	Yes	340.7

1 a = cemented sand piezometer tip.
b = No. 10 Clayton Mark suction strainer.

a. *Injection Location 1*. At Location 1, 398 L of thin (1:2) slurry was injected at foundation level with a flat tip. Injection was discontinued when the slurry flowed to the surface around the perimeter of the grout collar.

b. *Injection Location 2*. The casing at Location 2 is a replicate of Location 1. The only difference between the two locations was that the sand slurry was thickened to a 1:1 mixture midway through the injection process. This location took 1,200 L of slurry before a radial surface crack full of slurry was found to be opening at the surface. Slurry injection was discontinued, and, 0.5 hr later, the crack was noted to be closing.

c. *Injection Location 3*. The casing at Location 3 had a grout collar, and the end of the injection casing was slotted in a cross pattern, with four slots 100 mm long and 12 mm wide. This slotting was made in an attempt to form a set of vertical fractures which would then propagate outward and upward to form vertical fans of sand slurry. After 398 L (105 gal) of slurry had been injected, the slurry surfaced. The crack was excavated to below the surface crust to determine if a continuous vertical curtain of slurry had been formed. Observations indicated that this had occurred.

d. *Injection Location 4*. Location 4 also had a grout collar and a slotted tip, but in this instance three slots were cut 120 deg apart with a slot size approximately 150 mm long and 12 mm wide. Shortly after grouting was initiated, slurry return at the surface was noted as a leak around the grout collar. Soil was tamped over the leak and injection continued, but more slurry was forced to the surface around the grout collar. Plywood was then placed around the injection point to prevent premature cracking of the dredged-material crust from upward leakage around the grout collar. After 1,440 L of slurry had been injected, the injection was terminated when the sand slurry surfaced in three radial cracks around the injection location in the wye pattern. At the same time that the sand slurry created the wye-crack pattern at Location 4, it also surfaced and reopened the long crack previously noted at Location 2. The observed behavior indicated that horizontal flow of the sand slurry had occurred over the 7.6-m distance between the two injection locations.

e. *Injection Location 5*. Sand-slurry injection at Location 5 was carried out at two depths, 1.2 m and then 2.3 m below the surface. No grout collar was used, and the injection tip was flat. Sand slurry was injected at the 1.2-m depth until it surfaced around the casing, and the casing was then pushed by hand to the 2.3-m depth, where injection was resumed. Injection was discontinued when the slurry again surfaced around the casing after a total of 1,250 L had been injected. Upon completion of the injection, the casing was pulled. This experiment was an attempt to determine if multiple drainage layers could be produced by sand injection and also to determine if the casing needed to be left at the injection site for future pumping or if it could be removed immediately. The unit weight (approximately 184 kg/cu m) of the sand slurry was great enough to keep the hole open and to allow a cemented sand-suction tip with a 31.75-mm riser to be inserted into the slurry to the 2.3-m depth.

f. *Injection Location 6*. The casing at Location 6 had a float tip and no grout collar. It was hand-pushed down to the 2.3-m depth, where injection commenced. This location received 340 L of sand slurry before the slurry surfaced around the casing, flowing upward through existing desiccation cracks in the surface crust.

Immediately after sand-slurry injection had been terminated at each location, a suction tip was placed through the slurry to the 2.3-m depth. Two types of suction tips were used. A cemented sand-type piezometer tip with 31.75-mm riser was used at Locations 1, 2, 3, 5, and 6, and a No. 10 Clayton Mark suction strainer with 19-mm riser was used at Location 4. At Locations 1, 2, 3, 4, and 6, the suction tip and riser was placed to the 2.3-m depth inside the 76-mm diam ABS casing. At Location 5, the casing was removed prior to installation of the suction tip. The casings and risers were then cut off at ground level, and a header system was installed to allow vacuum pumping of the system. All suction-tip wellpoints were valved so that pumping experiments could be made with individual wellpoints if desired. Priming plugs were placed in the header system at Locations 1 and 2. After priming, vacuum gages were installed at the priming plug locations.

Control area

Suction tips and risers were placed by conventional means to the 2.3-m depth at the four locations shown in Figure 30, and denoted as Locations 7 through 10. These suction-tip wellpoints were installed for comparison with similar points placed at the sand-injection locations. At Locations 7 and 9, cemented sand piezometer tips with 31.75-mm risers were used and No. 10 Clayton Mark suction strainers with 19-mm risers were used at Locations 8 and 10. All suction tips were surrounded with the same mortar sand used in the sand slurry. Each tip was surrounded with sand, placed in a cloth sack, and lowered to the 2.3-m depth inside 76-mm-diameter ABS casings which were then removed. The dredged material closed around the risers of these uncased wellpoints and formed an effective seal. The risers at the four locations were then cut off near the surface and connected in a header system similar to that used for the suction-tip wellpoints placed at the sand injection locations, as shown in Figure 30.

Pumping tests

Both the sand injection and control suction-tip well systems were pumped by using two gasoline-powered 38-mm centrifugal pumps connected in series with an orifice-type eductor between the pumps. The eductor was connected to the header system. Circulation of water through the eductor applied a constant vacuum to the header system. As the same water was recirculated, an increase in its volume resulted from flow through the wellpoints into the header. Each system was pumped individually. A vacuum of 648 mm Hg was maintained in the sand-slurry injected header system.

Pumping tests were conducted on both systems during the period 10-13 February 1976. Prior to initiating the pumping test, 12 elevation check points were set up in a cross pattern around each wellpoint for vertical control. Check points consisted of 0.3-m square plywood plates affixed to the top of a stake pushed into the soil. The center of each plate was marked for the point of control. These check points provided only crude control since the dredged material was fluid to the extent that movement occurred about one meter away from a person walking on the surface. Relative elevation of the ground surface was established at each check point to determine any settlement of the dredged material during pumping. A point on a steel weir box located in the dike adjacent to the test area was used as a bench mark and assigned el. 100 ft mlw.[1]

A 12-hr pumping test on the sand injection system was conducted on 12 February 1977. Data were recorded during the entire 12-hr period. A steady stream of water was produced with initial flow rate of 138 L/hr-wellpoint, which gradually decreased to a steady 11 L/hr-wellpoint at the end of the test. A total of 1,128 L were produced. Results of the pumping test are displayed graphically in Figure 31. During the pumping test, bubbles were visible surfacing in standing surface water around the injection locations. As pumping continued, bubbles and foam appeared in the water produced from the sand-injection system. When a burning match was dropped into the foam, it burned, indicating that the wells were also producing methane gas.

The control wellpoint header system was pumped on 13 February 1977. After 2 hours of pumping with an average vacuum of 762 mm Hg, the four wells produced 1.55 L of water. The wellpoints were back-flushed with about 284 L of water and repumped. After 20 min of pumping at the same vacuum, the wells produced only 0.76 L of water. The wellpoints were again back-flushed with about 568 L of water. Pumping was again started with a vacuum equivalent to 762 mm Hg and continued for 2 hr. This last test produced a total volume of 9.1 L. The initial rate was 6.8 L/hr-wellpoint and the final rate was 0.57 L/hr-wellpoint.

Elevations were measured around each wellpoint as described previously, before and after the pumping test, to determine whether the ground surface had subsided during pumping. Measurements around the control wellpoints were inconclusive, indicating a net rise of the ground surface of approximately 15 mm. As this behavior is physically impossible, it must be assumed that the level instrument placed in the soft dredged-material crust must have either sunk by that amount during the test period or else the traffic around and through the control area on the very soft dredged material resulted in upward displacement of the soil around the header systems. Surface settlement around the sand-injection wellpoints was more noticeable and is shown in Figure 32.

Discussion of Field Results

In Table 17 the average surface settlement measured 0.6 m away from each injection well head is tabulated, arranged in order of decreasing amount by injection location. A number of conclusions may be made from these data and those shown in Figure 32:

a. Flat injection tips result in more surface settlement than slotted injection tips, probably because the slotted tips encourage formation of vertical cracks

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[1] All original elevation data in Chapter 7 are reported in units of feet, the official units of the USC&GS during the test period.

which may then propagate upward and outward and form a series of vertical fans around the injection casing. Conversely, flat injection tips may encourage the formation of flat horizontal fans of large areal extent at the interface between the foundation and the dredged material.

b. The Clayton Mark brass suction strainer does not appear to be as efficient as the cemented sand suction tip.

c. Grout collars do not appear to add to the effectiveness of the injection or suction process.

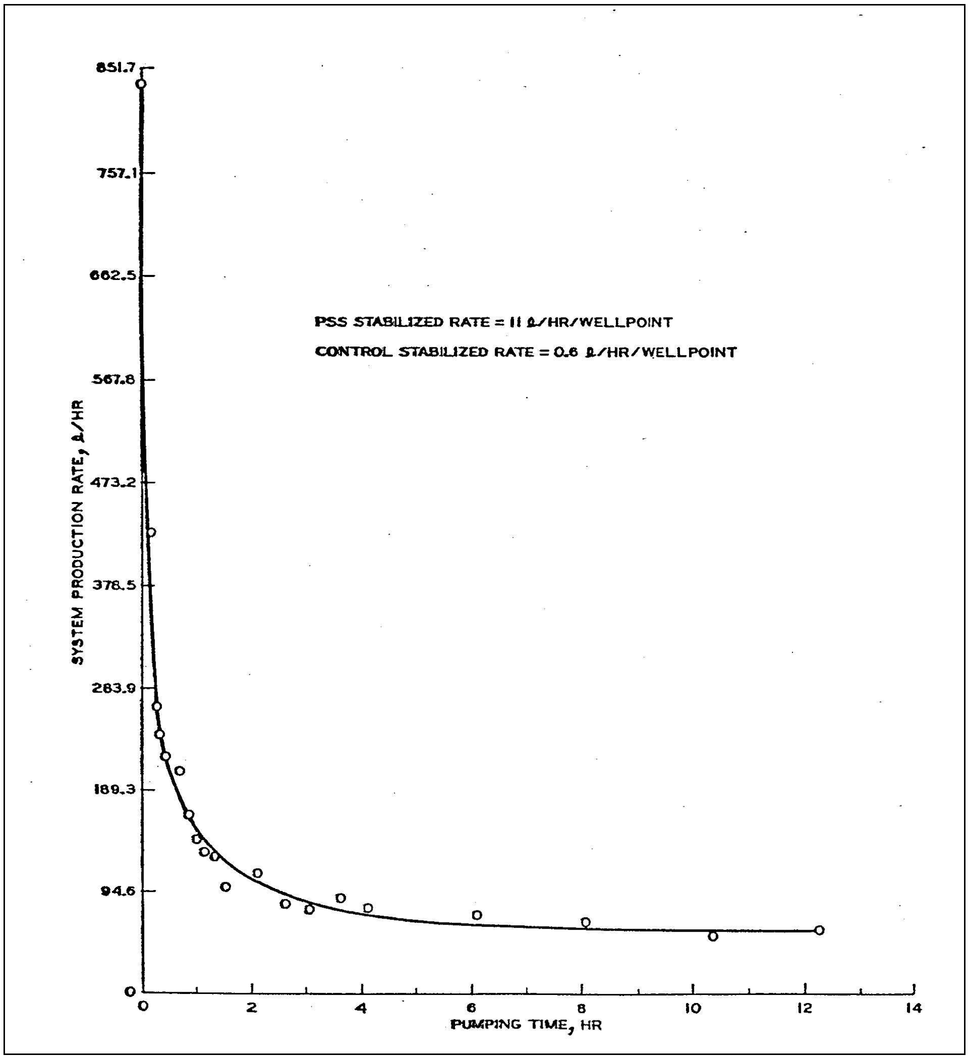

Figure 31. Results of 12-hr pumping test on six wellpoints placed in sand-injected drainage lenses

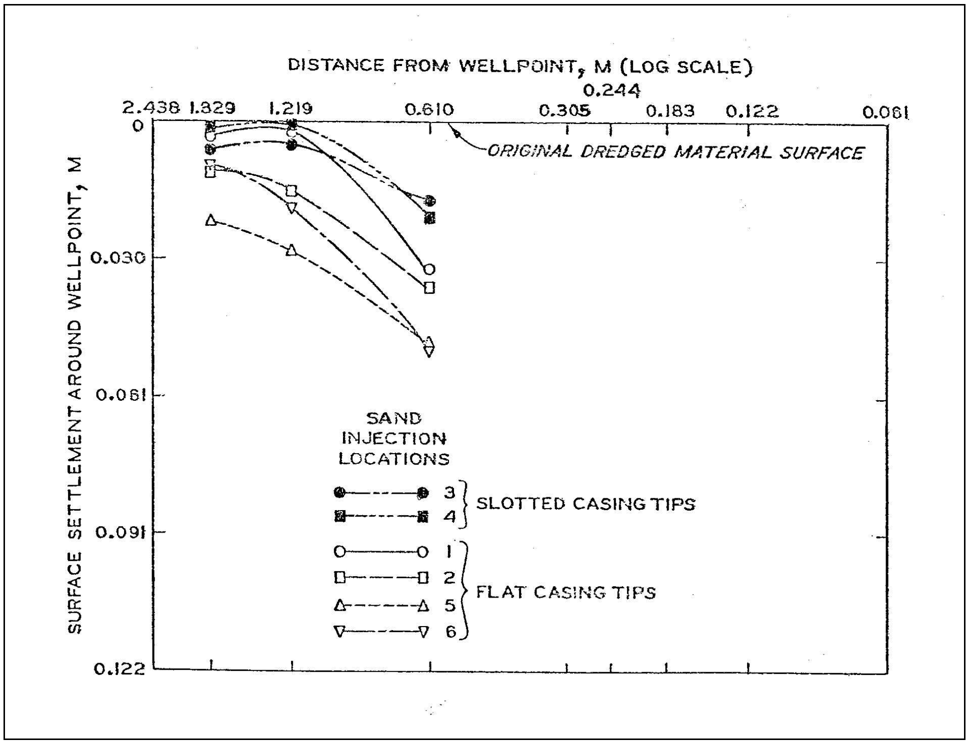

Figure 32. Measured surface settlement during 12-hr pumping test on sand-injected lenses

Table 17
Surface Settlement at Sand-Slurry Injection Locations

Injection Location	Surface Settlement 0.6 m away, m	Injection Casing Tip Geometry	Grout Collar	Suction[1] Tip	Casing[2]	Slurry Consistency	Volume of Slurry Injected, L
6	0.050	Flat	No	A	a	Thick	340
5	0.049	Flat	No	A	b	Thick	1,250
2	0.037	Flat	Yes	A	a	Thin/Thick	1,200
1	0.034	Flat	Yes	a	a	Thin	398
4	0.021	Slotted	Yes	b	a	Thick	1,440
3	0.018	Slotted	Yes	a	a	Thick	398

1 a = Cemented sand-suction tip, b = No. 10 Clayton Mark suction strainer.
2 a = Casing remained in place, b = Casing was removed.

d. Contrasting the effects achieved at Locations 5 and 6 with thick (1:1) and slurry to those attained at Locations 1 and 2 with thin (1:2) slurry, it appears that the thick slurry results in a more effective sand injected layer. The thick slurry may keep the sand in suspension longer and deposit it at a greater distance from the injection point, resulting in more surface area in the sand lenses. The lesser effects achieved at Locations 3 and 4 with thick slurry may perhaps be ascribed to the fact that they used slotted injection tips and thus concentrated more material immediately adjacent to the wellpoint with a smaller horizontal distance around the wellpoint covered by the sand slurry.

e. Casing removal prior to pumping appears to result in a greater amount of overall surface subsidence, as noted in Figure 32.

f. The effectiveness of the wellpoints placed in the sand-injected locations does not appear directly related to the amount of sand slurry injected, but rather to the geometrical distribution of the sand around the injection point and the effectiveness of the suction tip.

Despite numerous attempts, it was impossible to detect the exact location of sand lenses with a penetrometer probe. Thus, it is not possible to state conclusively that flat injection tips did actually cause more nearly horizontal fans than did the slotted injection tips, but the sand probably did not bulge out from the injection tip forming a sand bulb because such a bulb should have been detected by penetrometer probing. Also, the observation that sand slurry pumped into Location 4 surfaced at Location 2, 7.6 m away indicates that the sand slurry may move horizontally in thin layers.

Theoretical Aspects of Hydraulic Fracture in Dredged Material

Actual behavior of pressure-injected sand slurry and the paths it may take are probably best explained by fracture mechanics and technology which has been developed for hydraulic fracturing of oil-sand formations. Experiments by Hubbert and Willis (1957) showed that a fracture would most likely occur in a plane normal to direction of least principal stress unless layers were present. Layers of different stiffness encourage injected fluid to flow along the layer interface. Earlier experiments and analyses by Harrison, Keeschnik, and McGuire (1954) showed that near the earth's surface, expected fracture patterns would be horizontal. Then, depending upon the Poisson's Ratio of the material, at some greater depth the dominant fracture pattern would become vertical. In their paper, they calculated that vertical fracture would begin to occur at the 910-m (3,000-ft) depth for rock with a Poisson's Ratio of 0.25. Based on these data, a predominantly horizontal fracture pattern would be expected in dredged material placed in confined disposal areas.

Using the work of Harrison, Keeschnik, and McGuire (1954), it is possible to estimate the excess pressure required to induce vertical and horizontal fracture in dredged material. These researchers' formulae for fracture pressure are as follows:

a. *Vertical Fracture*: $P_f = 2\,(\mu/(1 - \mu))\,\sigma_z + S_t$ (7.1)

b. *Horizontal Fracture*: $P_f = \sigma_z + S_t$ (7.2) where

P_f = required fracture pressure

σ_z = effective vertical overburden pressure

μ = poisson's ratio

S_t = potential strength of the material normal to the direction of crack propagation

If one assumes typical geotechnical properties of the dredged material, underlying surface crust, to be a wet unit weight γ_t of 125 - 150 Kg/cu m, μ equal to 0.4 - 0.5, S_t of 14.4 kPa maximum or 0.29 σ_2 (assuming normally consolidated soil) and unit weight of sand slurry γ_{slurry} of 184 kg/cu m, then the data shown in Table 18 may be calculated, assuming the sand slurry injection tip is 2.3 m below the dredged-material surface. Potential strength of the dredged material has been assumed equal to the cohesive shear strength *c*. The well-known relationship between cohesion *c* and preconsolidation pressure P_c ($c/P_c = 0.29$) for a normally consolidated clay has been used.

The last three columns of Table 18 indicate the excess head or height above the ground surface required to cause hydraulic fracture of the dredged material. These data indicate that the dredged material would fracture horizontally by gravity-induced excess pressure if the sand slurry were at or slightly above the dredged-material surface. As an additional amount of pressure is required to cause sand slurry to flow outward from the injection tip, it is probable that more and more pressure is required as the leading edge of the sand slurry advances further away from the injection point. At some point in the slurry advance, required pressure for continued horizontal advancement will exceed the vertical fracture pressure which will then allow the slurry to move upward toward the ground surface. In field tests, slurry injection was discontinued when the slurry broke through the surface. As noted in Table 18, vertical fracture will occur with the sand slurry standing about 0.6 to 0.9 m above the ground surface, for a Poisson's Ratio of 0.4. During the field test, observations indicated that the sand slurry stood at about 0.3 to 0.6 m above the surface in all tests, confirming the reasonableness of the theoretically calculated values given in Table 18. The results observed in the field injection tests, together with the calculated results shown in Table 18, indicate that sand slurry can be injected into dredged material with no special pumping or pressure injection equipment.

Summary

The sand-slurry injection field demonstration was intended to be a small-scale determination of gross feasibility for a previously untried process. For this reason, no detailed instrumentation, soil sampling, or testing were employed, and as penetrometer probing was unable to identify the actual extent of any of the lenses formed by sand injection, any discussion, theoretical or otherwise,

Table 18
Calculated Fracture Pressures for Dredged Material with Injection Tip 2.29 m Below Surface

Tensile Strength, kPa	Effective Unit Weight of Dredged Material, kg/m^3	Vertical Fracture Pressure, kPa — Poisson's Ratio $\mu = 0.4$	Vertical Fracture Pressure, kPa — Poisson's Ratio $\mu = 0.5$	Horizontal Fracture Pressure, kPa	Height of Slurry Above Ground, m — Horizontal Minimum	Height of Slurry Above Ground, m — Vertical $\mu = 0.5$ Maximum	Height of Slurry Above Ground, m — Vertical $\mu = 0.4$ Maximum
14.36 (Probable Maximum)	124.9	51.6	70.38	42.37	0.061	1.615	0.579
	147.3	56.32	60.44	47.40	0.335	2.164	0.945
0.30 σ_z[1] (Assuming Normally Consolidated Dredged Material)	124.9	40.89	64.45	36.44	-0.274	1.280	0.00
	147.3	53.87	75.99	42.95	0.091	1.920	0.701

1 σ_z = Effective overburden pressure at depth z below surface.

concerning the actual extent of negative pore pressures applied to the soil, theoretical amounts of consolidation possible from the process, effective permeability of the sand injected layers, and other data needed to precisely explain the system behavior cannot be reliably determined. However, as a demonstration of feasibility, the experiment was a success in that it practically demonstrated that considerably more water may be pumped from sand-injected layers than from wellpoints installed by conventional techniques. Based on the results obtained from the demonstration, it may be concluded that:

a. Hydraulic fracturing and injection of a 1:1 mortar sand/reverted water slurry is a technically feasible method for creating internal drainage layers in fine-grained dredged material.

b. Best results appear to be obtained when flat injection tips are used and the fracturing occurs at foundation level, where the difference in relative stiffness between the two layers may cause fracturing and sand injection along the horizontal plane.

c. Observations during the field demonstration and theoretical calculations both tend to indicate that when fracturing occurs at foundation level, sand slurry will spread in horizontal layers until the pressure required for further horizontal movement exceeds the vertical fracture pressure of the dredged material mass. At such time, surface return of the slurry will be noted.

d. The exact horizontal extent obtainable by the sand slurry before vertical fracturing occurs is not well known. It was at least 7.6 m in one instance.

e. No special pressure grouting equipment is needed in practical application of the technique to fine-grained dredged material placed in confined disposal areas, and casings need not be left in place after injection has been completed. As an excess head of approximately 0.3 to 0.6 m was required to fracture the dredged material and force horizontal slurry entrance, it is suggested that future application of the technology consider a reusable casing which may be, perhaps, mechanically placed and removed rapidly with sand slurry poured into the casing using a large funnel rather than the expensive positive-displacement pumping system used in this field demonstration.

Because of the success of the initial field-feasibility demonstration and the potential applicability of this technique for after-the-fact rapid dewatering and vacuum consolidation of fine-grained dredged material placed in confined disposal areas, it is strongly recommended that subsequent more detailed investigations be undertaken to determine the exact behavior of sand-injected slurry and to refine slurry injection techniques and formulate methods for prediction of potential effects when and if the technique is used on a large-scale basis. The initiations of such studies is justified on the basis of the relatively high flows obtained from the sand-injected layer wellpoints. Stabilized flows of 11 L/hr-wellpoint were 18 times greater than from the study control wellpoints and 14 times greater than the average wellpoint flow (19 L/day-wellpoint or 0.8 L/hr-wellpoint) obtained from the vacuum wellpoint experiment of Chapter 5.

Improving the efficiency of the vacuum dewatering system by an order of magnitude in production rate might result in a unit volume dewatering cost approaching that of Progressive Trenching, but at a several times faster rate of volume storage creation. While such comments are partly speculative at this stage, they show the need for additional study.

In the opinion of the DMRP staff, Laboratory studies of dredged-material hydraulic fracturing with sand slurry under carefully controlled conditions may be the initial step in this process. Based on these laboratory results and further theoretical investigations, a properly designed large-scale field demonstration should be undertaken to provide more precise data on all aspects of the sand injection concept.

8 Periodic Mixing of Crust and Underlying Dredged Material

A field demonstration was designed to investigate the effect of periodic mixing of dry surface crust with underlying very wet fine-grained dredged material. The technique was somewhat similar to techniques of repeated tillage used by bottomland and delta farmers to dry wet fields and also may be compared to mechanical stabilization of plastic clay by addition of cohesionless material with the blocks of dried surface crust (representing coarse aggregate) added to the very wet and plastic underlying dredged material. The procedure was to consist of mixing the dried crust with underlying wet material, allowing a new drying crust to form, remixing, etc., until the resulting mass was either too stiff to mix or the desired soil properties (water content reduction and volume change) had been achieved. The mixing process was believed to cause a significant increase in the rate of evaporative drying of fine-grained dredged material, and thus gain of disposal area volume.

This procedure was first suggested at Planning Seminar I, held at the WES in October 1974 (USAEWES 1974), and suggested again at Planning Seminar II, held at the WES in January 1975 (USAEWES 1976c). No consensus of opinion was reached by seminar participants concerning the potential effect of such periodic mixing. The responses from more than 30 knowledgeable experts were divided among the following opinions:

a. The process would produce a significant increase in evaporative drying rate, compared to unmixed material.

b. The process would produce an increase in the evaporative drying rate, both the gain would not be great enough to justify the effort and cost of mixing.

c. The process would have no significant effect on the evaporative drying rate.

Further, in evaluating their field studies of mechanical agitation carried out for the U.S. Navy, Western Division, at the Naval Facilities Engineering Command at Mare Island, CA, Harding-Lawson and Associates (Final Report, December 1975, "Engineering Study for Dredged Material Processing, Mare Island Naval Shipyard, Ballejo, California") indicated that periodic agitation or crust mixing was the only agitation process that appeared worthy of consideration for full-scale implementation.

Because of the contradictory opinions existing about the concept and because of the potential for applicability in CE confined disposal areas, if valid, it was decided to design and carry out a small-scale field demonstration of the concept.

Test-Site Location, Material, and Program

Test site

The southwest corner of the UPB site was selected for conduct of the field demonstration. At this location, fine-grained dredged material had been placed to approximately e1 8.0 to 8.5 ft mlw, above a fine silty sand (SM) foundation at approximately e1 1.0 to 1.5 ft mlw. In July 1975, approximately 0.3 m of ponded water covered the site with approximately 2.1 to 2.3 m of dredged material of axle grease consistency existing beneath a 0.05-m crust. Surface water was removed in November 1975, and by February 1976, a 0.15-m crust existed. Two 0.4-ha test plots were laid out on the existing crust, one plot to be periodically plowed and one plot to act as an undisturbed control section. A perimeter ditch was placed around the test area so that precipitation would rapidly run off into an existing drainage ditch to the west of the test site. Casagrande- type piezometers were placed in the test and control areas.

Test material

Results of engineering tests on the fine-grained dredged material in test and control areas indicated the material had about the same average engineering properties as were obtained during the site characterization studies described in Chapter 3. Water-content measurements made at the start of the experiment indicated that below the crust, the material was up to 20 percent to 30 percent water content above its liquid limit.

Test program

During the first week of February 1976, piezometers were installed and allowed to stabilize, and initial cross-section survey data 3.1-m grid) were taken on both the test and control areas. The perimeter ditch was then placed around the test and control areas with the Riverine Utility Craft (RUC). After these preparations, the existing approximately 0.15-m-thick surface crust in the test plot was thoroughly mixed with underlying wet dredged material by action of the RUC rotors. Approximately 3 hr was required to achieve thorough mixing.

The process (obtaining cross-section survey data, water-content samples, cone-penetration data, and piezometer levels, followed by RUC rotor mixing of the surface crust with underlying wet dredged material) was repeated monthly until July 1976. In March 1976, approximately 1.5 hr was required to remix the approximate 0.08-m surface drying crust with underlying wet dredged material. The drying crust thickness which developed varied monthly from 0.025 to 0.1 m during the test period. Hereafter, required RUC rotor mixing time increased, up to 6 hr in July 1976. The RUC effort required to mix the material during this last period was such that transmission overheating problems developed frequently, so the mixing program was terminated.

Results and Discussion

During the test program, several qualitative observations could be made:

a. After initial mixing, the test area usually required more effort to accomplish mixing each succeeding month, except in June 1976, following a month of high precipitation.

b. The control area surface crust thickness and surface firmness increased with time, except for softer surface conditions in June 1976.

c. Precipitation quickly ran off the surface of the control area into the perimeter ditch but was often trapped in furrows caused by the RUC rotors in the test area.

d. The surface crust which formed in the test area between monthly mixing was just sufficient to support the weight of a man.

e. Final consistency of the test area after mixing always appeared to be slightly below the liquid limit, despite the increase in mixing time with which was required to obtain this condition.

Data indicate that the surface of the test ploy subsided up to 0.45 m during the test period with an average settlement of approximately 0.31 m. The surface of the adjacent control section settled approximately 0.11 m during the test period. Thus, an increase of 0.21 m of surface settlement (about 10 percent of the original thickness) was obtained by the periodic mixing, or 2,120 cu m of volume per hectare, or 858 cu m for the 0.4-ha test section. Actual volume gain was probably not quite this much because the desiccation cracks in the control area increased in depth, width and number during the test period. The volume of these cracks was not measured but was found to be about 8 percent of the crust volume in other DMRP research (DMRP 1976d).

Water-content depth and precipitation data with time are shown for the test area in Figure 33, and for the control area in Figure 34. Each data point is the average of five determinations taken throughout the area. Samples were taken at the same locations each month, using a Hvorslev-type piston sampler. Examination of the data indicate that initially, the test area dredged material was slightly wetter than the control area material, but that drying appeared to occur at approximately the same average rate for both sections. At the time mixing was terminated, the water content of the test area material was in the vicinity of the average liquid limit of the material. The amount of precipitation appeared to affect behavior in the upper 0.6 m of the material of both sections, but underlying material was not greatly affected.

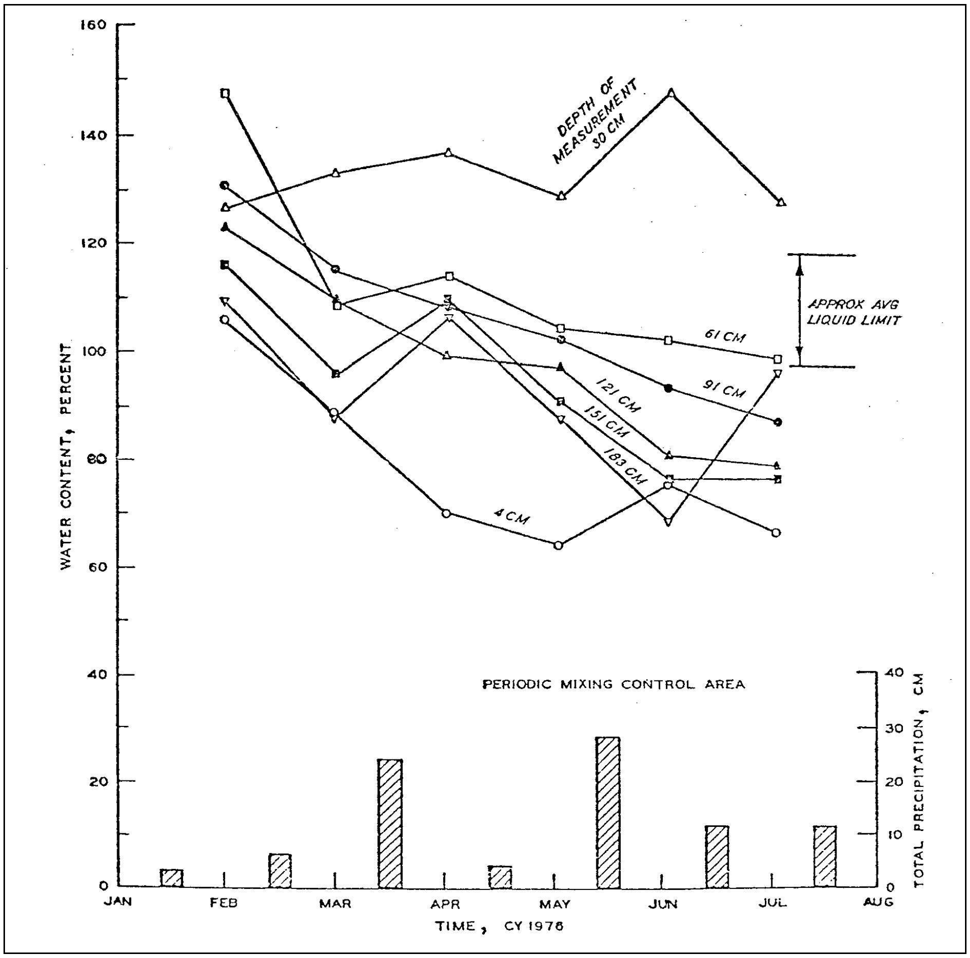

Figure 33. Water content and precipitation with time for test area

Results of average cone index (CI) data at various depths are plotted versus time for the test area in Figure 35 and for the control area in Figure 36. Monthly precipitation and the average piezometric level versus time are also plotted in the Figures. Data of Figure 35 indicate that below 0.15 m, the support capacity of the test section remained approximately constant with depth and increased only slightly with time, despite a continuously falling water table. In July 1976, when the average CI of the test area dredged material approached 20, the RUC had extreme difficulty in mixing the crust and underlying material, causing the test to be terminated. The average CI of the crust would support a man and after May 1976, support a low-ground pressure vehicle for a single pass. However, below the crust, the average CI was such that a man would have trouble traversing the area. In the control section, the average CI in the upper 0.3 m increased greatly with time, reflecting increased surface crust development as the water table dropped. Below 0.3 m, the average CI data was markedly similar to that of the test area below 0.15 m. The average CI of the control area crust in July 1976 would have supported numerous vehicles for a single pass and would have allowed low ground-pressure vehicles or conventional vehicles on mats to work in the area.

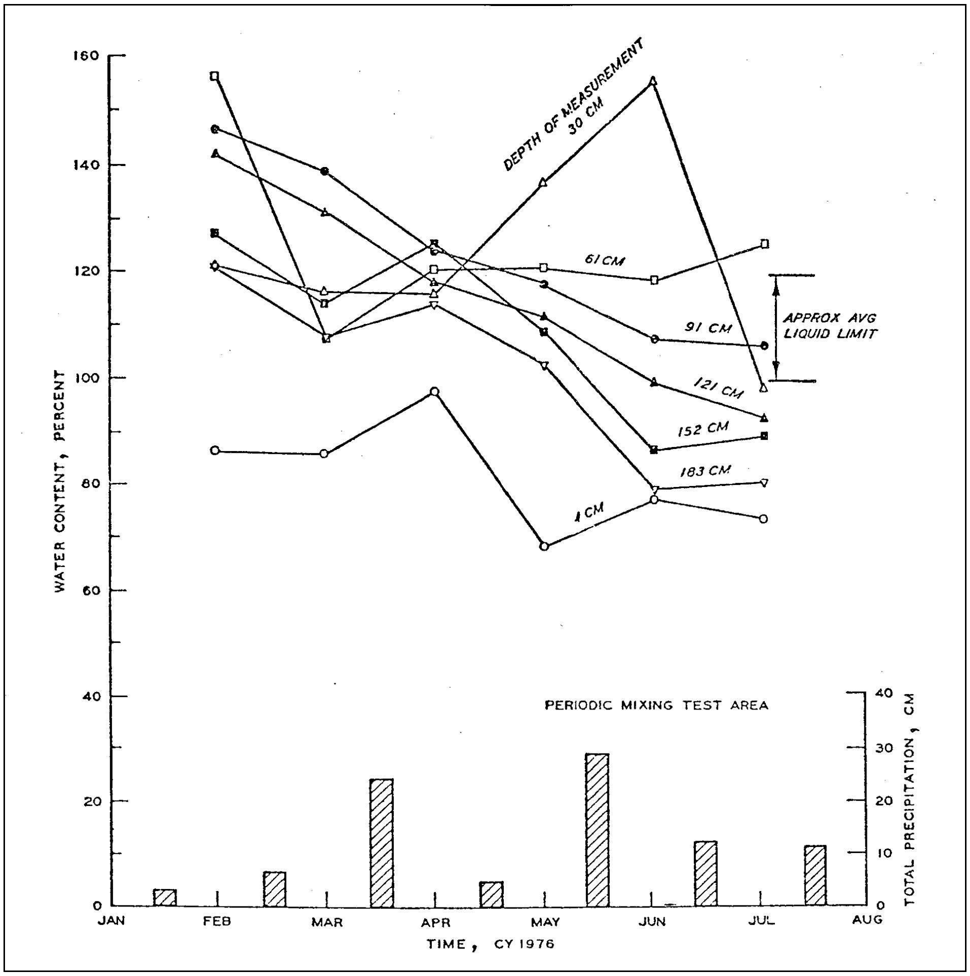

Figure 34. Water content and precipitation with time for control area

A rather extensive volunteer fresh-water vegetative cover became established over the control area while the surface of the test area remained entirely bare during the test. The pore water of the dredged material had an initial saline concentration approaching that of sea water in Mobile Bay. Precipitation apparently leached sodium chloride from the surface crust in the control area while the periodic mixing continually brought saline soil to the surface, inhibiting vegetation establishment between mixing cycles.

Approximately 17 hr of continuous RUC operating time was required for the six mixing cycles, plus approximately 4 hr of downtime from mechanical problems. The approximate cost of RUC operation (primarily labor and fuel) is $75/hr. Thus, the cost of providing an additional 858 cu m of disposal area volume was $1,575 or $1.84/cu m. Under normal conditions, a RUC could be expected to mix about 1.0 to 1.5-ha per working day, depending upon initial material consistency and equipment downtime. Lower unit operating costs could probably be obtained with a cable drag-plow system pulled between the perimeter dikes or from a central tower to the perimeter dikes, but at considerably

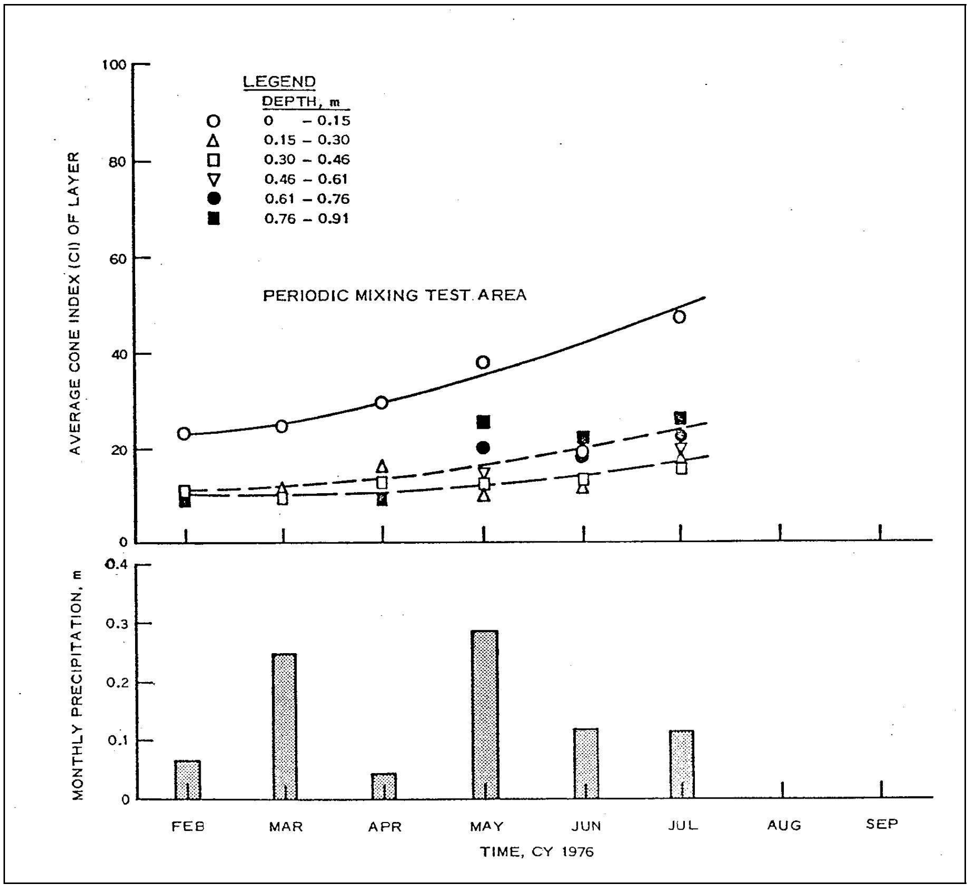

Figure 35. Average cone penetration index with time, depth, and precipitation for periodic mixing test area

higher capital investment. The estimated capital cost of a RUC is approximately $75,000 while a semi-permanent cable system might cost two or three times this amount.

Summary

Based on the data obtained, it may be stated, for the given test conditions and material, that:

a. Periodic RUC rotor mixing of dried surface crust and underlying fine-grained CH dredged material initially above the liquid limit resulted in an increase of 0.21 m of vertical subsidance over a 6-month period, compared to an adjacent unmixed area subjected to the same climatic conditions. Corrections were not made for any additional volume gained in the control area from increase in size, depth, and number of crust desiccation cracks. The cost of creating disposal area volume was estimated at $1.84 cu m.

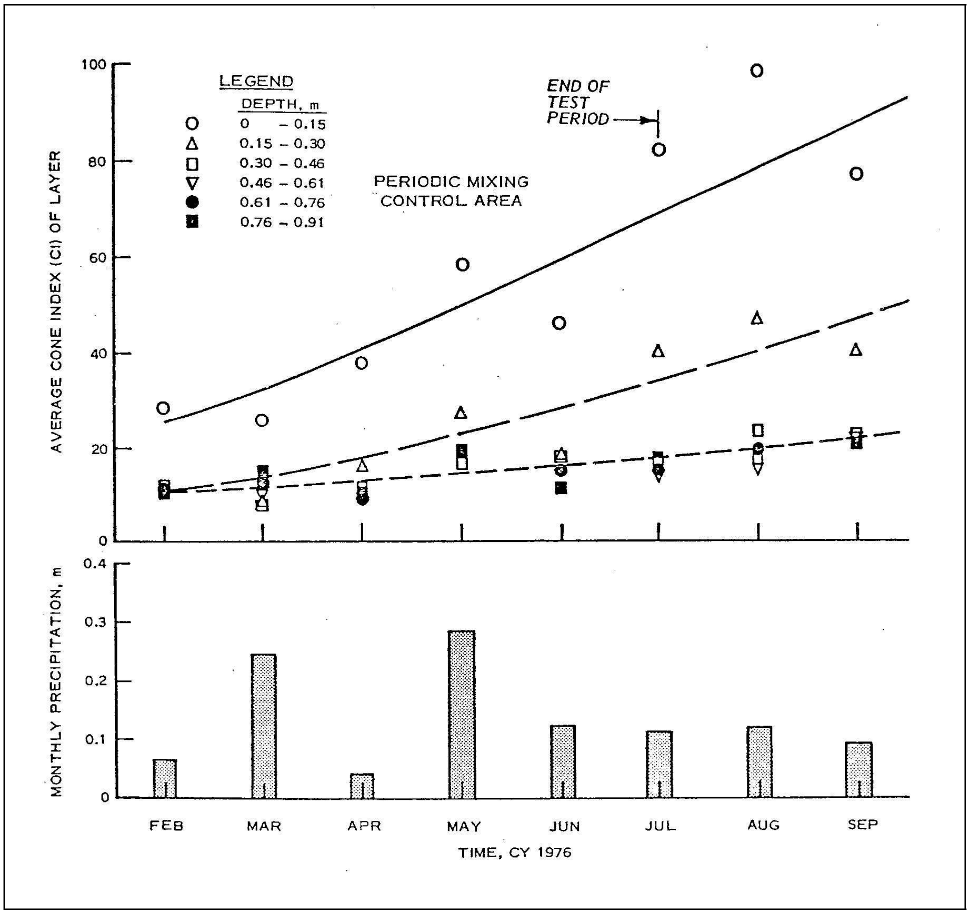

Figure 36. Average cone penetration index with time, depth, and precipitation for periodic mixing control area

b. Water-content data below the zone of RUC rotor mixing (approximately the upper 0.6 m) appeared unaffected by the mixing action, and below this zone, water content profiles and drying trends in the test and control areas remained approximately the same. This behavior tends to indicate that the observed surface subsidance is not related to the thickness of dredged material if the thickness is greater that the RUC rotor mixing depth.

c. Once the average CI below the crust in the RUC rotor mixing zone approached 20, corresponding to a water content approximating the liquid limit of the dredged material, the RUC was ineffective in mixing the surface and underlying material. These data imply that, for the test conditions and material, the 6-month value of 0.21 m of additional subsidance is an absolute value, not an indication of subsidance rate expected from the process. If the data are generalized, it might be expected that RUC rotor mixing would be ineffective once the water content of any fine-grained dredged material approached its liquid limit and/or its CI approached 20. While additional mixing at lower water contents could perhaps be obtained by RUC-towed or cable-drawn plows or discs, other DMRP studies (Willoughby 1978) have indicated that drawbar requirements for pulling implements in fine-grained dredged material increase markedly once the CI exceeds 20.

d. Reduction of surface-support capacity by periodic mixing is a detriment to conducting further dewatering work in a disposal area because, if the volume gain from mixing is inhibited once the water content in the upper 0.6 m approaches the liquid limit, other equipment may have to be brought into the area to work on the surface to continue the dewatering.

e. Prevention of vegetation establishment degrades the aesthetics of the area, reduces available habitat, and further reduces equipment support capacity that might be expected from any vegetative mat.

While some volume gain was achieved by the periodic mixing process, the net overall effect of mixing does not appear to justify the effort required. Behavior of both test and control areas appeared to be more nearly influenced by precipitation than any other factor. The most important operation may thus be rapidly remove precipitation before it can be absorbed by the upper layers of the dredged material. The periodic mixing process is not recommended for full-scale implementation by CE field elements.

9 Underdrainage Dewatering Field Demonstration

Research conducted by Johnson et al. (1977) surveyed existing state-of-the-art methods for promoting drying and densification of fine-grained dredged material placed in confined disposal areas. Results of the study were presented to and discussed by the DMRP Planning Seminar II consultants and technical experts in January 1976 (DMRP 1976b) who recommended field evaluation of promising underdrainage techniques identified by the study. The rationale for the recommendations was that, despite relatively low hydraulic permeability of fine-grained dredged material, at least a year is normally available between disposal cycles and appreciable gravity seepage might occur over such an interval, even at relatively low drainage rates. Also, settled fine-grained dredged material is extremely compressible, and even small changes in the existing effective stress regime, caused by removal of perched water table conditions, seepage forces, and/or vacuum-induced negative pore pressure, might produce significant consolidation and thus rapidly create additional disposal-area storage volume. If significant benefits were obtainable, CE field elements might allocate the additional funds to place underdrainage systems in confined disposal areas prior to initial disposal.

Based on this potential applicability, field evaluation of promising underdrainage dewatering concepts was undertaken. Initial planning was started in February 1976, and field testing was concluded in September 1977.

Rationale Concerning Selection of Techniques to be Evaluated

A comprehensive review of the results of Johnson et al. (1977) identified six viable underdrainage dewatering concepts. These concepts were then rated, as shown in Table 19 as to which exhibited the most promise for full-scale application. As a result of this evaluation, four concepts were selected for field evaluation: (a) underdrainage, (b) seepage consolidation, (c) partial vacuum in underdrainage layer, or vacuum-assisted underdrainage, and (d) a combination of (b) and (c) (i.e., seepage consolidation with a partial vacuum in the underdrainage layer, or vacuum-assisted seepage consolidation). Advantages and disadvantages of each method are summarized in Table 20.

Table 19
Rating of Densification Techniques Studied for Field Evaluation

Technique	Rating (1 = Best, 4 = Worst)				
	Increase in Effective Stress Over No Treatment1	Relative Cost1	Full-Scale Applicability	Construction Practicality	Overall Rating
Underdrainage (constructed sand blanket and collector system)	4	2	1	2	2.25
Underdrainage (collector system installed in pervious foundation)	4	1	1	1	1.75
Seepage consolidation [3.1 m deep-surface ponding in underdrainage]	3	2	1	2	2.00
Seepage consolidation [3.1 m surface ponding with underdrainage and vaccum]	1	3	2	2	2.00
Temporary earth surcharge [1.5 m high earth fill with underdrainage]	3	3	3	3	3.00
Temporary earth surcharge [3.1 m high earth fill with underdrainage]	2	4	4	4	3.50
Temporary water surcharge [2.4 m deep water with membrane on top of dredged material and underdrainage below]	3	4	2	2	2.75
Temporary water surcharge [4.9 m deep water with membrane on top of dredged material and underdrainage below]	2	4	3	4	3.25
Partial vacuum in underdrainage layer	1	3	1	2	1.75
Partial vacuum in surface drainage layer	2	4	1	2	2.25

[1] See Johnson et al. (1977) for actual values and assumptions made in obtaining these values.

Table 20
Advantages and Disadvantages of Densification Techniques Selected for Study

Technique	Advantages	Disadvantages
Underdrainage	Reatively low cost	Possible construction problems if placement of
	Sand from required dredging or naturally occuring pervious foundation	Occupies storage space in disposal areas.
	Can be used in conjunction with other densification techniques	Must have collector pipes in order to be effective
	Filters the effluent from dredged material	
	Can be used in conjunction with surface drying and vegetative growth	
Seepage Consolidation	Relatively low cost	Requires underdrainage layer
	Almost doubles effective stress in dredged material as compared to underdrainage alone	Requires higher dikes to contain water
	Elimates possible odor problems	Prohibits surface drying and vegetative growth
		Dike erosion from wave action could be a problem
Partial Vacuum in Underdrainage Layer	Cost of adding vacuum pumps to underdrain system is low	Maintenance required during operation
	Results in very high effective stress in dredged material	Energy required to operate
	Can be used in conjunction with other densifaction procedures	Possible problems from leakage which could lessen magnitude of desired vacuum
	Does not prohibit surface drying or vegetative growth	

Underdrainage

This technique consists of providing free drainage at the base of the dredged material. Downward flow of water from the dredged material into the underdrain takes place by gravity.

Seepage consolidation

In this technique, water is ponded on the surface of the dredged material, and underdrainage is provided at the base of the dredged material. Downward seepage gradients then act as a consolidating force, causing densification with typical effective stress conditions.

Vacuum-assisted underdrainage

As in the previous techniques, drainage is provided at the base of the dredged material, but a partial vacuum is also maintained by vacuum-pumping the underdrainage layer. This technique greatly increases typical effective stresses in the dredged material.

Vacuum-assisted seepage consolidation

This technique combines the effects of seepage consolidation with those of induced partial vacuum in the underdrainage layer.

No treatment

A confined disposal area with impervious foundation and perimeter dikes functions essentially as a "bathtub," as there is no drainage other than from the surface. A relative comparison of the different effective stresses shown is given in Table 21.

Table 21
Comparison of Typical Effective Stresses from Different Densification Techniques

Densification Technique	Effective Stress		Effective Stress Increase	
	kPa	lb/sq ft	kPa	lb/sq ft
None	6.7	140	0	0
Underdrainage	21.8	455	15.1	315
Seepage consolidation	36.9	770	30.2	630
Vacuum-assisted underdrainage	56.3	1,175	49.6	1,035
Vacuum-assisted seepage consolidation	71.4	1,490	64.7	1,350

Underdrainage provision

The previously described dewatering techniques all require underdrainage, which may be accomplished by use of a naturally occurring pervious foundation or a constructed free-draining sand layer. However, drainage layers must be provided with collector pipes for water removal. Otherwise, head losses in the drainage layers would be excessive and prohibit the drainage layer from functioning as intended. This requirement essentially limits use of the concepts to new disposal areas where installation of drainage layers can be accomplished prior to deposition of dredged material or to existing areas where material is stable enough to support placement of such drainage layers prior to disposal of additional material.

Experiment Design

Site selection

The underdrainage test site was located in the southeast corner of the UPB disposal area, as shown in Figure 2. A large mound of sand had been placed in this part of the disposal area by past disposal operations (Chapter 2) and may be seen in Figure 1. In contrast to other UPB demonstrations which sought to densify and dewater existing in-situ material, the underdrainage tests were to be conducted on freshly dredged material. Thus, the sand area was chosen to allow construction of test sections with fine-grained material from a nearby borrow area to be placed hydraulically over the installed drainage layers.

Material properties

The fine-grained material to be dewatered was to be dredged hydraulically from a borrow site located in the southwest corner of the UPB disposal area. The material would be excavated by hydraulic dredge and pumped to the test site as a slurry through a 200-mm-diam pipeline.

The borrow site material was a highly plastic black fine-grained clay (CH), containing approximately 6 to 8 percent organic matter. Borings taken in the borrow site indicated the dredged material to be about 2.4 to 3.1 m thick, uniform, and very soft, with water contents varying from about 60 to 140 percent. Test data indicated the material had engineering properties similar to those obtained in overall site characterization (Chapter 3).

Test selections

To properly evaluate the four methods selected for study, five test sections were designed, one for each technique to be evaluated and one control section to which no treatment would be applied. Test sections with a 1.8-m (6-ft) depth of settled dredged material deposited in an excavation with bottom size of 9.1 m by 9.1 m (30 ft by 30 ft) would provide sufficient volume to avoid scale effects. Test section side slopes were designated at 1V:2H, primarily for ease of construction. The overall depth of each test section was controlled by specific requirements of the technique being evaluated. The underdrainage layer used in all test sections (except for the control section, which had none) was 0.61-m (2-ft) thick and contained collector pipes. To insure no flow of water into or out of the test sections, each was fully lined with two layers of 8-mil-thick polypropylene plastic sheeting. Access to the Test Sections, primarily for instrumentation, sampling, and in-situ testing, was provided by bridges constructed to span each section.

Selection of drainage material and drainage systems

To aid in selection of suitable drainage material, laboratory filtration tests were performed in 230-mm-diameter lucite cylinders, using dredged material from the selected borrow site and six different drainage materials. Because of time constraints, tests were not extensive but involved only measurements of dredged-material drainage rate and visual observations of how much dredged material penetrated and passed through the drainage media. A summary of test results is given in Table 22. Drainage rates for all tests were essentially the same because the much lower permeability of the overlying dredged material controlled flow. Based on these data and the observations on dredged material penetration into the drainage media, the drainage materials in Test Nos. 1, 3, and 6 appeared adequate. All other things being equal, standard concrete sand was selected because of availability and usually lower first cost, as compared to pea gravel under filter cloth.

Table 22
Summary of Filtration Test Results

Test No.	Drainage Material Tested	Remarks
1	Filter cloth (openings equivalent to U.S. No. 70 sieve size) over pea gravel	Very little penetration of dredged material into pea gravel. No evidence dredged material in discharge water
2	Filter cloth (openings equivalent to U.S. No. 30 sieve size) over pea gravel	Complete penetration of dredged material into pea gravel. Considerable amount of dredged material in discharge
3	Standard well-graded concrete sand	Very little penetration of dredged material into sand. Discharge water perfectly clear
4	Coarse uniform sand	Considerable penetration of dredged material into sand. Some dredged material visible in discharge water
5	Fine uniform sand (material from test site sand mound at Upper Polecat Bay disposal area	Very little penetration of dredged material into sand. However, a small amount of dredged material visible in discharge water
6	Filter cloth (openings equivalent to U.S. No. 100 sieve size) over pea gravel	Some penetration of dredged material into pea gravel, but discharge water clear

Note: Drainage rates were essentially identical for all tests.

All collector pipes were Schedule 40 PVC plastic pipe. Table 23 contains details of the collector pipe system design. Collector pipes for the vacuum system were designed by Wellpoint Dewatering Corporation, New York, New York, as a part of a contract to design and install the entire vacuum system. All collector pipe systems were connected to solid PVC pipe at the inside toe of the test section slope. The solid pipe extended under the slope and discharged at the outside toe of the section slope.

Table 23
Summary of Data for Collector Pipe Systems

Test Section No.	Type Drainage	Type Pipe	Dia. of Slotted Pipe		Slot Width		No. Slotted Pipes	Slotted Pipe Spacing	
			mm	in.	mm	in.		m	ft
1	Gravity	Sch 40 PVC	152	6.0	1.3	0.05	1	9.1[1]	30
2	Vacuum	Sch 40 PVC	38	1.5	0.3	0.01	3	2.3	8.5
3	Vacuum	Sch 40 PVC	38	1.5	0.3	0.01	3	2.3	7.5
4	Gravity	Sch 40 PVC	152	6.0	1.3	0.05	1	9.1[1]	30

[1] As 1 pipe was placed in the center of a 9.1-m (30-ft) wide area, it is equivalent to a 9.1-m (30-ft) spacing.

Test Sections 2 and 3 each had separate vacuum systems with piping and valving to run both sections with either pump. The two vacuum pumps had a free air capacity of 473 L/m and were each driven by a 2.3-kW electric motor. Vacuum pumps were located at the outside slope toe of the Test Section 2 and 3.

Data Collection Equipment and Procedures

Instrumentation

Types of instrumentation selected for use and the data expected from each are listed in Table 24.

In situ measurements

In addition to permanently installed instrumentation, in situ measurements to determine soil properties, as given in Table 25, were also planned.

Sampling and laboratory testing

Water-content samples of the deposited dredged material in each test section were to be taken periodically at each 0.3-m depth. Samples would also be taken for Atterberg limits and specific gravity determination.

Table 24
Instrumentation Summary

Parameter to be Measured	Instrument
Positive pore-water pressure in dredged material	Porous stone (Casagrande type) piezometer WES transducer piezometer
Negative pore-water pressure in dredged material	Tensiometer WES transducer piezometer
Positive pore-water pressure in underdrainage layer	Porous stone (Casagrande type) piezometer
Vacuum in underdrainage layer	Vacuum piezometer
Settlement of underdrainage	Settlement plate
Discharge from underdrainage layer in Test Sections 1 and 4 (gravity)	Hourmeter on sump pump
Discharge from underdrainage layer in Test Sections 2 and 3 (vacuum)	Water meter

Table 25
Summary of In Situ Measurements

Dredged-Material Parameter to be Measured	Measurement
Water content and density	Nuclear moisture and density probes
Shear strength	Vane shear device
Penetration resistance	Trafficability penetrometer
Settlement	Surface level readings

Two different samplers were planned for use in obtaining disturbed samples, a Hayden Slurry Sampler and a Hvorslev piston sampler. The Hayden sampler would be used to sample very high water-content material still essentially in a slurry state while the Hvorslev sampler would be used for firmer material. The lower part of the Hayden Slurry Sampler, described in detail elsewhere (Lacasse et al. 1977) consists of 63.5-mm ID aluminum tubing containing a 6.4-mm plunger rod and end-attached tapered rubber stopper. To operate, the device is lowered to the elevation at which the sample is desired, with the stopper in place against the opening in the lower end of the tubing. The inside plunger rod is then used to push the stopper out. Slurry flows into the tubing and the plunger rod is then pulled up, forcing the tapered stopper into the tubing and trapping the sampled material. The whole device is then pulled and the sample allowed to flow out into a container. The Hvorslev sampler is a hand-operated fixed-piston vacuum sampler which obtains a 47.6-mm-diam sample. This sampler is described elsewhere (Department of the Army 1972).

Undisturbed sampling of the dredged material was planned after enough consolidation had occurred to enable such sampling to be accomplished, using the Hvorslev sampler, for the purpose of performing laboratory unconfined compression and triaxial Q tests. The same sampler, fitted with a larger diameter sampling tube, would be used to obtain samples for laboratory consolidation tests.

Test-Site Construction

Site preparation

The sand mound from which the test sections were to be built contained a perched water table, caused by interbedded thin layers of fine-grained material. It was thus necessary to drain the area prior to test section excavation with a series of dragline excavated ditches. A dragline was also used for rough excavation and filling operations immediately after ditching was complete.

Fine grading was accomplished with dragline and small bulldozers. Each test section was built to grade except for the back slope where the original drainage ditch was located. This slope was left open for installation of the solid portion of the collector pipe system which would carry water from the underdrainage layer to an exit point. After test section closure, final side slopes were established by hand raking.

Impervious liner and access bridge placement

Following hand raking of all slopes, two layers of 8-mil-thick polypropylene plastic sheeting were placed in each test section to provide and impervious liner. This liner was continuous except where the drainage pipe entered to connect with the slotted collector pipes. At this point, waterproofing compound was placed around the pipe and a concrete collar poured to ensure a waterproof seal. Sandbags were used to secure the liner at the crest of each test section. Each liner was proof-tested for imperviousness by filling each test section with water and by monitoring the water level for approximately 1 week.

After the impervious liner was in place, access bridges which had been fabricated on-site were moved into place across the test sections with a crane.

Underdrainage layers

Slotted collector pipes for gravity drainage Test Sections 1 and 4 were installed prior to placement of the sand-drainage material. Pipe joints were sealed and the entire pipe blocked up to proper elevation. Sand was then placed by clamshell and spread by hand to a uniform thickness.

Sand-drainage material was placed first in the two vacuum-assisted Test Sections 2 and 3. Trenches were then dug in the sand and slotted collector pipes placed, the trenches backfilled, and the sand smoothed to final elevation. Table 26 contains final surface elevations of the underdrainage layers for Test Sections 1 through 4.

Table 26
Final Surface Elevations of Underdrainage Layers

Test Section	Design msl Elevation		Final msl Elevation[1]	
	m	ft	m	ft
1	3.66	12.0	3.61	11.85
2	3.66	12.0	3.57	11.72
3	3.66	12.0	3.60	11.80
4	4.27	14.0	4.19	13.74
5	4.27	14.0[2]	4.12	13.53

[1] Prior to filling with dredged material.
[2] Bottom elevation (Test Section 5 has no underdrainage layer).

Following installation of the underdrainage layers and collector systems, sumps and sump pumps were installed in Test Sections 1 and 4 and vacuum pumps in Test Section 2 and 3. All pumps were located at the outside toe of their respective test section slopes.

Instrumentation described previously was installed after test sections were completed. Readout of piezometers plus vacuum sensing devices was simplified by routing all leads to a separate readout box for each test section.

Test section filling

The five completed test sections were filled with fine-grained dredged material from the designated borrow site in the UPB disposal area. Material was moved under MDO contract with Ed Nemer Construction Company, Florence, Alabama. A 200-mm Mudcat dredge powered by a 131-kW diesel engine, transported the dredged material hydraulically from the borrow site to the test site. As dredged material with a uniform slurry consistency was desired, it was necessary for the dredge to strip and waste the vegetation and desiccated surface crust in the borrow site. The softer subcrust material, at or above the liquid limit, more nearly simulated sediment removed by maintenance dredging operations.

To prevent scour of the underdrainage layers high exit velocity, from a floating energy dissipator was placed over the dredge pipe outlet. The energy dissipator consisted of a barrel with peripheral slots cut in its lower half and mounted on a 1.2-m by 2.4-m raft, made from a metal-frame plywood deck, with flotation provided by three styrofoam sticks. A special collar was welded on top of the barrel to receive the dredge discharge pipe, which was provided with a 90-deg elbow. This device worked extremely well, not only in dissipation of the force from the pump dredged material, but also in permitting movement of the discharge pipe within the test section to insure uniform filling.

To further limit scour of the drainage material, a large piece of polypropylene was placed on the sand surface in the area immediately under the dissipator, and filling was started by the initial pumping of clear water at low dredge operating rate. The low rate water pumping continued until the sand was saturated and the dissipator raft was barely afloat. At that time, dredged material was pumped under a slightly higher, but still relatively low, rate until about 0.6 m of slurry was in the test section. Dredging was then discontinued and the polypropylene pulled out by the dragline. Dredging was then continued with the dredge operating at approximately one-half its maximum rate. After another 0.6 m of slurry had been deposited, the dredge was allowed to operate at maximum rate.

As the slurry pumped by the dredge contained 15 to 25 percent solids (weight basis), it was necessary to fill each test section several times before the 1.8 m desired depth of settled material was attained. The general procedure used was to pump the test section full, to allow the solids to settle (generally 24 hr was sufficient), to pump the clear water off, and to refill it. This procedure was repeated until each test section contained sufficient solids. Generally, about 2 m of settled dredged material had been placed in each test section when filling was terminated.

Conduct of Experiment

There are two general phases of behavior governing natural densification of dredged material after confined disposal, hindered sedimentation, and self-weight consolidation. As the purpose of this experiment was to study and perhaps accelerate the consolidation phase, it was necessary to wait until sedimentation was complete and the material was entering the consolidation phase before beginning the experiment. More detail on fine-grained dredged material sedimentation-consolidation behavior is available elsewhere (Lacasse et al. 1977). The sedimentation phase was monitored by settlement readings of the dredged material surface and by visual observation of the amount of decant water on the dredged-material surface. When these observations indicated sedimentation was complete or nearing completion, data collection was begun on 9 November 1976, 19 days after filling was complete, and consisted of taking initial instrumentation readings and samples for water-content determinations, turning on the vacuum pumps, and allowing drainage from the underdrainage layers.

Data collection

The frequency of data collection is given in Table 27. In some instances the actual frequency varied somewhat from that given because of instrument malfunction, inclement weather, holidays, etc. However, data were usually collected according to the schedule given. The only type of data collection not given in Table 27 is for the nuclear moisture and density probes. This equipment did not provide useful data, so the measurements were terminated. All other instrumentation data were reduced, plotted, and checked prior to leaving the site.

Table 27
Frequency of Data Collection

Type	Frequency	Duration
Readings of instruments and dredged material surface-elevation determinations	Daily	First 2 weeks
	Weekly	Next 2 months
	Semimonthly	Next 3 months
	Monthly	Remainder of test
In situ measurements (vane shear and penetration resistance)	Semimonthly	First 2 months
	Monthly	Next 3 months
	Every 2 months	Remainder of test
Sampling for water content determinations	Weekly	First 2 weeks
	Semimonthly	Next 2 months
	Monthly	Next 3 months
	Every 2 months	Remainder of test

In situ vane shear and penetration resistance measurements were taken every 0.3 m of depth through the entire thickness of dredged material at two locations in each test section. Samples for water content determination were also taken ever 0.3 m of depth. Initially, the Hayden Slurry Sampler was used for all sampling except the bottom sample immediately overlying the underdrainage layer, which was taken with the Hvorslev sampler. As the material consolidated, it became possible to use the Hvorslec sampler at higher and higher elevations until, after 4 months of data collection, it was used exclusively except for surface samples in the seepage consolidation test sections.

A set of samples was taken at 0.3-m intervals through each test section in February 1977 for use in laboratory Atterberg limit and bulk specific gravity of solids determinations. Specific gravity of solids ranged from 2.62 to 2.67 and averaged 2.65. Three different sets of undisturbed samples of dredged material immediately overlying underdrainage layers were taken. The first set was tested in Q triaxial shear while the second and third sets were tested in unconfined compression. Samples were taken with the Hvorslev sampler and all were firm enough to be tested, except those from Test Section 5 (control), which were extremely soft and slumped badly upon extrusion from the sampler.

Control of surface drying

Initially, dredged-material surface drying was prevented in Test Sections 3, 4, and 5 (those without ponded water) to more accurately define effects of underdrainage without desiccation drying. This was accomplished by installing sumps in each of the three test sections. Each sump was equipped with an automatic on-off float pump so that no accumulation of rainwater would occur, but the dredged material surface would be kept moist by allowing a thin

25-50-mm layer of water to remain on the surface. Sumps were lowered manually as the dredged material surface settled. This procedure was followed until test data showed that most excess pore pressure had dissipated and surface settlements were occurring at a steady rate. At this time (15 March 1977), the dredged material surface was allowed to dry to more nearly model proper disposal area operation and to determine if the underdrainage techniques being evaluated would accelerate or increase the magnitude of surface drying and the resultant desiccation drying. As a result, sumps were lowered to coincide with the dredged material subcrust surface and were thereafter maintained at this level. The maximum depth of surface cracking in Test Section 3 was about 150 mm.

Maintenance of ponded water in test sections 1 and 2

It was originally planned to maintain a ponded water surface in Test Sections 1 and 2 (seepage consolidation) at a constant elevation, rather than to maintain a constant head with respect to the dredged material surface. This aim was accomplished within an accuracy of ±0.3 m.

Maintenance of vacuum in test sections 2 and 3

Initially, a vacuum of about 69 kPa was maintained in each test section. Shortly after initial vacuum application, vacuum pump No. 3 became inoperative because of a sheared coupling, and, for the next 3 months, vacuum pump No. 2 pulled an average vacuum of about 59 kPa in both sections. When pump No. 3 was back in operation, an average vacuum of about 68 kPa was maintained in both sections. The steady decrease in Test Section 3 (vacuum) probably results from surface drying of the dredged material with resultant air leakage. The vacuum decrease shown in Test Section 2 during May 1977 resulted from intermittent vacuum-pump operation as hot weather caused circuit breaker overload. This problem was remedied.

Additional instrumentation

Settlement of the dredged material surface caused piezometers at the 1.5-m level to be above the dredged material surface, after about 2 months of data collection. In order to properly define the pore water pressure profile with depth, additional porous stone piezometers were installed at a nominal 460-mm level below the existing material surface in January 1977.

Test Results and Discussion

At the time of report preparation, data had been collected and evaluated through May 1977, with data collection scheduled to occur through September 1977. For this reason, results and discussion presented herein are preliminary in nature and may be subject to some revision when all data are available for evaluation. A more detailed analysis of test results is also available elsewhere (DMRP 1976b).

Pore water pressure

Data indicate that excess pore water pressure initially existed in all five test sections but, by March 1977, had fully dissipated in all except the control test section. Lack of initial excess pore water pressure at the bottom of the underdrainage test sections (while such pressures did exist at the bottom of the control test section) would indicate that some drainage occurred into the sand during the 19 days between completion of filling and beginning of data collection. Also, there was some slight leakage through valves at the discharge pipe outlets during this time, which could have contributed to the condition.

Data show the rate of excess pore water pressure dissipation and indicate when dissipation was complete. Not only did Test Sections 1 and 2 with ponded water have considerably higher initial excess pore water pressures than the Test Sections 3 and 4 without ponded water, but excess pore pressures took about 4 months to dissipate, while for Test Sections 3 and 4 without ponded water, only about 1 week was needed. Exact reasons for this behavior are unknown, but such initially higher stresses may result from quasi-surcharging by the ponded surface water before piezometric stresses in the dredged material mass stabilized, and the slower rate of pore pressure dissipation may result from different effective drainage distances in Test Sections, as Test Sections 3 and 4 had double drainage faces while pore pressure dissipation in Test Sections 1 and 2 from upward drainage was inhibited by downward seepage from the surface.

Discharge from underdrainage layers

Flow from Test Sections 1 and 4 (gravity drainage) slowly decreased with time and appeared to have stabilized at about 26.6 L/day. For this flow rate and dredged-material thickness of 1.2 m, a dredged material permeability of about 3×10^{-8} m/sec may be computed, within the range of other field and laboratory tests on UPB dredged material.

Operational problems with flowmeters used in Test Sections 2 and 3 (vacuum) prevented reliable discharge measurements during the early stages of the test. Attempts to use the flowmeters were finally abandoned, and discharge measurements in Test Sections 2 and 3 were made by periodically measuring the time it took for the discharge to fill an 11.4-L container. However, flow from these test sections was erratic, and periodic measurement did not indicated development of steady-state conditions.

Settlement

The majority of settlements in the foundation and/or drainage layer occurred immediately after filling from saturation of the relatively uncompacted sand. Test Section 5 had no drainage layer. However, a sand pad about 150 mm thick was placed under the settlement plate base to keep it from tearing the impervious liner. Some indicated settlement in Test Section 5 may thus result from lateral spreading of the sand pad, as it had no lateral restraint.

All treatments caused additional settlement when compared to the control test section. Despite the differences in effective stress shown for Test Sections 1 and 2 (seepage consolidation without and with vacuum), seepage consolidation behavior was markedly similar for both test sections during the first 120 test days, perhaps reflecting the slowness of upward negative pore pressure propagation in the dredged material. After 190 days of vacuum application, negative pore pressures had been induced in only about the lower quarter of the dredged material layer. These data, as well as those summarized in Table 28, indicate that vacuum application had little effect on the rate of seepage consolidation settlement. The rate of settlement for both treatments appeared to be leveling off, perhaps because, as the material consolidates and decreases in thickness and as effective stresses increase from the bottom upward, the net force produced by seepage is reduced.

Table 28
Summary of Dredged Material Settlement Data

Test Section	Initial Dredged-Material Layer Thickness		Dredged Material Layer Thickness May 1977		Settlement of Dredged Material Layer as of May 1977		Percent Settlement[1] as of May 1977
	m	ft	m	ft	m	ft	
1	1.91	6.26	1.19	3.90	0.72	2.36	37.7
2	1.94	6.35	1.15	3.77	0.79	2.58	40.6
3	1.67	5.48	0.94	3.09	0.73	2.39	43.6
4	1.74	5.72	1.10	3.60	0.64	2.12	37.1
5	1.92	6.29	1.40	4.60	0.52	1.69	26.9

[1] Expressed in percent of initial layer thickness.

For Test Sections 3 and 4 (vacuum-assisted and gravity underdrainage), the vacuum-assisted Test Section 3 consistently settled at a more rapid rate. Data points at about 120 test days seemed to indicate settlement rates were decreasing rapidly, resulting in the decision to allow surface drying which apparently rejuvenated the total settlement rate. However, the rate of settlement of Test Section 4 (gravity) after surface drying was initiated was only slightly faster than for the control Test Section 5, indicating that the majority of surface settlement is being induced by surface drying rather than gravity drainage. Settlement of the vacuum-assisted Test Section 3 is proceeding at a faster rate, and settlement rates for gravity and vacuum-assisted underdrainage begin to diverge at about 120 test days. Reasons for diverse behavior may be easily understood when the relative water table and effective stress levels for the two treatments are compared.

As summarized in Table 28, all underdrainage treatments produced about the same magnitude of surface settlement. Additional data collected through September 1977 should give better information on long-term settlement rates.

Water content

The data show that all test sections have undergone appreciable water-content reduction with the average water content in the control test section above that in the four treatment test sections. The lower initial water contents for the material adjacent to the underdrainage layers (lowermost data points) in Test Sections 1, 3, and 4 may result from consolidation during the 19 days between completion of filling and experiment startup. Since this lowermost material was subjected to the greatest overburden loading and was the first lift pumped, the most consolidation would occur there. Also, some additional consolidation undoubtedly occurred because of leakage in discharge pipe valves during the initial 19 days. The effects of crust formation and surface drying on water content reduction in Test Section 3, 4, and 5 is noticeable when their upper level water content data are compared to those in Test Sections 1 and 2.

The data allow comparison between the untreated control test section and the untreated disposal area, where dredged material had been in place for approximately 6 years. Based on these data, the rate of water-content change in the control test section should decrease substantially in the near future. The behavior of all test sections as average water contents approach 100 percent (near the average liquid limit) will be better understood when data taken through September 1977 are available.

Shear strength

Shear strength data with depth from tests made in November 1976, January 1977, and May 1977 clearly show the greater gain in shear strength with time of the dredged material in the treated test sections as opposed to the control test section, and indicate that the material has minimal strength until its consistency approaches the liquid limit (approximately 100 percent water content). Until water contents approximate the liquid limit, shear strengths will reflect the behavior of a viscous liquid rather than a soil.

Also of importance is the fact that the strength data not only show a large strength increase for the material immediately adjacent to the underdrainage layers but also show a steady increase with time up through the entire thickness of dredged material. The increased strength of the dredged-material surface in Test Sections 3, 4, and 5 (from desiccations) is also clearly evident from these data.

Data of May 1977 substantiate previously presented settlement and water-content data. The material in test sections with vacuum (Test Sections 2 and 3), which showed the greatest settlement and lowest average water contents, also appears to have the highest shear strengths. The very low strengths of the control Test Section 5, as compared to the treated Test Sections, are also clearly indicated in this plot, again showing the marked changed in dredged-material shear strength once water contents approach the liquid limit.

Relative Cost of Creating Storage Volume

Though test results discussed in the previous section are subject to some revision, several definite trends appear to be established. Preliminary calculations of volume creation rate and unit cost will provide at least tentative data on potential treatment feasibility, practicality, and cost-effectiveness.

To estimate system installation costs, it was assumed that a 0.3-m-thick drainage layer could be used in full-scale installation, as the 0.6-m-thick layer in the test sections has not remained saturated. A 15-m collector pipe spacing was also assumed, based on observed system behavior. If free-draining sand was purchased and delivered to the disposal site at a cost of $4.00/cu m and spread at a cost of $0.70/cu m, an estimated cost of $13,940/ha is required for drainage material. Estimated costs of $3.30/cu m for collector pipe and $1.60/m for placement indicate a collector pipe cost of $2,960/ha. A total installation cost of $16,900/ha is thus estimated for the drainage system if material is purchased. Assuming sand of proper gradation was available at no cost from other dredging work, the drainage layer material could be obtained for the cost of loading, transport, and spreading, perhaps reducing the cost by about one-third, for a total cost of $9,300/ha plus $2,960/ha or $12,260/ha. If a pervious drainage layer already existed at the site, drainage-material costs would be zero, but the cost of installing collector pipe might double. Thus, an estimated cost of $3,950/ha may be reasonable.

To estimate annual operating costs, they were assumed to be negligible for the gravity underdrainage system. While for the seepage consolidation system, the cost of a one-fourth-time technician was assumed to periodically adjust site ponding depth. Based on data from Chapter 5, this cost is estimated at $110/ha. For the vacuum systems, it was assumed that adequate vacuum could be developed with pump power of 3.7 kWhr/ha. Power costs at $0.02/kWhr would be $650/ha-year.

A one-half technician was assumed to be available for monitoring vacuum-system operation at a cost of $220. Rental or capital amortization and maintenance costs for vacuum pumps were assumed at $1,200. Thus, annual operating costs for the treatments were estimated as $110/ha for seepage consolidation, $2,070/ha for vacuum-assisted seepage consolidation, zero for gravity underdrainage, and $2,070/ha for vacuum-assisted underdrainage.

To estimate a rate of volume creation over and above that from no treatment conditions, percentages of settlement increase were determined by subtracting Test Section 5 (control) data from percent settlement data for the four treatments, given in Table 28. These percentages were then applied to an assumed typical thickness of 3.1 m, and additional unit storage volume created for the 190-day period (test duration through May 1977) was tabulated in Table 29. These data were extrapolated (which may or may not be warranted) to estimate volume gain produced by 1-year treatment operations, which are also tabulated, along with estimated annual operating costs, in Table 29.

Based on the estimated volume created and the estimated installation and operation costs, unit disposal-area storage-creation costs were computed and are also tabulated in Table 29. It should be noted that these costs are only estimates and are based on projected rates of operation for 1 year. If actual dewatering rates decrease during the remainder of the test period, computed unit costs will be too low. Conversely, if the treatments produce effective dewatering for more than 1 year after installation and disposal, computed unit costs will be too

Table 29
Estimated Costs of Creating Additional Storage Volume for Initial 3.1-m (10-ft) Thickness of Fine-Grained Dredged Material if Treatment Operated for 1 Year

	Additional Storage Volume Created by Treatment				Cost of Treatment, Installation, and Operation					
	190-Day Interval		1-Year Interval		Buy and Place Sand		Place Free Sand		Existing Pervious Layer	
Treatment	cu m/ha	cu yd/acre	cu m/ha	cu yd/acre	$/ha	$/acre	$/ha	$/acre	$/ha	$/acre
Seepage consolidation	3,270	1,740	6,660	3,550	17,010	6,895	12,370	5,015	4,060	1,645
Seepage consolidation + vacuum	4,150	2,210	7,980	4,250	18,970	7,690	14,330	5,810	6,020	2,440
Underdrainage + vacuum	5,050	2,690	9,720	5,180	18,970	7,690	14,330	5,810	6,020	2,440
Underdrainage	3,100	1,650	5,930	3,160	16,900	6,850	12,260	4,970	3,950	1,600
					Unit Cost of Created Additional Storage Volume					
					Buy and Place Sand		Place Free Sand		Existing Pervious Layer	
					$/cu m	$/cu yd	$cu m	$/cu yd	$/cu m	$/cu yd
Seepage consolidation					2.55	1.97	1.86	1.41	0.61	0.46
seepage consolidation + vacuum					2.38	1.81	1.80	1.37	0.75	0.67
Underdrainage + vacuum					1.95	1.48	1.47	1.12	0.62	0.47
Underdrainage					2.85	2.17	2.07	1.57	0.67	0.51

high. Thus, a 1-year operation interval seems rational for preliminary relative comparisons.

A review of the resulting costs, even if the data are of only preliminary nature, is very illustrative. Costs for all treatments are approximately within the same order of magnitude, with unit volume-creation costs for systems where drainage material must be purchased ranging from \$1.95/cu m - \$2.58/cu m (\$1.48/cu yd - \$2.17/cu yd) down to \$0.75/cu m - \$0.61/cu m (\$0.57/cu yd - \$0.46/cu yd) when pervious layers already exists in the disposal area, a unit cost comparable to dewatering by the progressive trenching concepts described in Chapter 4.

These data graphically show the advantages to be gained from using relatively clean sand available from other dredging operations to construct the drainage layer and the advantages to be gained from installing underdrain systems in pervious disposal area foundations prior to disposal. Further, the highest unit costs given in Table 29 are considerably lower than the lowest unit costs estimated from conventionally installed vacuum wellpoints, as described in Chapter 5. These data indicate that it is more cost-effective to consider vacuum-induced dewatering before disposal, rather than after disposal.

The data of Table 29 also illustrate behavior of the two basic systems evaluated. For seepage consolidation and vacuum-assisted seepage consolidation, unit-volume creation costs are approximately the same, implying that no particular advantage except a slightly faster settlement rate accrues from vacuum pumping. Under these conditions, the operationally simpler seepage consolidations may be more desirable in full-scale application. However, despite the extra operational cost, vacuum-assisted underdrainage has consistently lower unit-volume creation costs than gravity underdrainage, with the cost differential becoming more significant when drainage material must be purchased and/or placed. Under these conditions the expense of vacuum pumping is more than justified, as the end result is both faster and more cost-effective dewatering. Also, seepage consolidation appears to produce volume gain at a unit cost comparable with underdrainage, even when surface drying is allowed, and thus may be a viable alternative when material must be deposited underwater in construction of offshore confined disposal sites or when surface ponding is desirable for other reasons.

Conclusions and Recommendations

It should be noted that the trends and values for results presented in this Chapter may be subject to change as more test data become available. Further, it should be noted that the demonstration described herein was concerned with dewatering a single lift of settled dredged material, and determining the effect of subsequent lift placements was not incorporated in experiment design. However, based on the results, analysis, and assessment described herein, it may be concluded:

a. All four treatment methods evaluated (seepage consolidation, vacuum-assisted seepage consolidation, gravity underdrainage, and vacuum-assisted underdrainage) were found to be technically feasible, operationally practical, and cost-effective for dewatering fine-grained dredged material placed in confined disposal areas.

b. Unit-volume creation costs are considerable lower when drainage materials do not have to be purchased and/or placed on-site. If pervious drainage layers are already present in the disposal area, unit dewatering costs are comparable with those obtainable by progressive trenching.

c. Unit-volume creation costs for seepage consolidation and vacuum-assisted seepage consolidation are similar, indicating that the more operationally simple seepage consolidation may be the preferable alternative.

d. Unit-volume creation costs for vacuum-assisted underdrainage were significantly lower than for gravity underdrainage, indicating that the additional expense of vacuum pumping if justified.

e. Comparison with unit-volume creation cost data from Chapter 5 (vacuum wellpoints) indicates that it is considerably easier and more cost-effective to install drainage systems before disposal than after disposal.

f. Vacuum-assisted underdrainage combined with improved surface drainage should produce the fastest dewatering rate.

It is recommended that CE field elements and other interested users consider installation of underdrainage (preferably vacuum-assisted) when progressive surface trenching concepts cannot be applied or in conjunction with progressive surface trenching if such trenching cannot produce an acceptable dewatering rate. Conditions conducive to such combination dewatering are likely to occur when deposited dredged-material lift thicknesses exceed 1.0 to 1.5 m annually. Seepage consolidation is recommended in situations where dredged material densification is desired but surface ponding is necessary.

10 Electro-Osmotic Dewatering Study

When a direct electrical current is applied across a saturated soil mass, the electrical field aligns dipolar water molecules, and current flow across the electrical circuit results in movement of water molecules from anode to cathode. More detailed data on the theory of electro-osmotic (EO) dewatering are available elsewhere (Casagrande 1959). It may be simply stated that EO flow in saturated soils is similar to Darcy's Law hydraulic flow, except that the EO permeability of all normal soils is approximately a constant on the order of 5×10^{-7} m/sec and the flow rate is approximately proportional to the voltage gradient between anode and cathode, i.e., the electrical voltage drop across the circuit divided by the distance between anode and cathode.

Because of the relatively constant EO permeability, more or less independent of soil type, EO dewatering is normally used when the hydraulic permeability is significantly lower than the EO permeability, i.e., for fine-grained silts and clays. In many instances the amount of water produced from the material is secondary to soil mass stabilization resulting from internal seepage forces produced by the induced flow. In such conventional geotechnical engineering applications, rapid results (days or weeks) are desired, and a voltage gradient of 0.1 V/mm is often used.

During initial DMRP research planning, the possibility of EO dewatering as a means of dewatering fine-grained dredged material placed in confined disposal areas was discussed in detail. The Bureau of Mines, U.S. Department of the Interior, has successfully used EO as a means of dewatering and stabilizing fine-grained mine slimes and tailings (Sprute and Kelsh 1975). In the contemplated DMRP application, “rapid” dewatering was not as much a requirement as economical dewatering. Therefore, interest was developed for evaluating EO dewatering feasibility at very low voltage gradients, on the order of 0.001 to 0.01 V/mm, in hopes that the time could be traded for power cost. At least 1 year is usually available between successive fillings of confined disposal areas, and often longer periods are available. If the unit cost of dewatering decreased faster than flow rate with decreasing voltage gradient, long-term EO dewatering might prove economically feasible. Further, EO dewatering appeared to have potential for dewatering relatively thick (3 m or more) layers of existing dredged material.

These concepts were discussed at Planning Seminar I in October 1974 (DMRP 1974), and the invited consultants and technical experts recommended that investigations to determine feasibility be conducted. This research was initiated to investigate laboratory feasibility of EO fine-grained dredged-material dewatering.

Results of the laboratory feasibility study (O'Bannon, Segall, and Matthias 1976) were presented at DMRP Planning Seminar II in January 1976 (DMRP 1976c) and discussed by the invited consultants and technical experts. Based on the laboratory results, a definitive decision concerning feasibility could not be made. Results of the laboratory testing were promising, but several major questions were still unresolved. Casagrande Consultants (CC), Cambridge, Massachusetts, was then retained by the DMRP to study the laboratory test data, conduct limited testing of UPB dredged material, visit the proposed UPB test site, and render an independent opinion concerning low voltage gradient EO dewatering feasibility. CC advised that the process possessed enough technical feasibility to warrant field evaluation, though success was not guaranteed. Based on these recommendations and the potential importance of the technique, if successful, it was decided to conduct a field demonstration of EO dewatering at the UPB disposal area. The demonstration was conducted by the Foundation and Materials Branch, Engineering Division, MDO.

Laboratory Study of Electro-Osmotic Dewatering Feasibility

The laboratory study was conducted to determine the technical feasibility of low voltage gradient EO dewatering of fine-grained dredged material to determine appropriate voltage gradients and other design parameters needed for the field test and to estimate the power use and the anticipated dewatering costs for various voltage gradients.

The laboratory study was conducted on several types of typical fine-grained dredged material, including samples from Mobile. Results of 70-mm-diam by 127-m-long tube tests on UPB dredged material are shown in Figure 37. The average water contents are shown, which are not entirely indicative of behavior, because when the tube tests were completed, the tubes were disassembled, and it was observed that samples had well-defined wet and dry sides. Approximately half of each sample adjacent to the cathode was uniformly wet. The remaining sample fraction adjacent to the anode was uniformly dry. The dry side and wet side values are, respectively, lower and higher than the average values shown in Figure 37, but could not be measured except at the conclusion of the tests. It was anticipated that dry side values could be achieved in the field by periodically moving anodes closer to cathodes as the drying process progressed. The volume and moisture content of each side of the dewatering test cylinders were determined and the percent-volume reduction was calculated for each sample. These data are shown in Table 30.

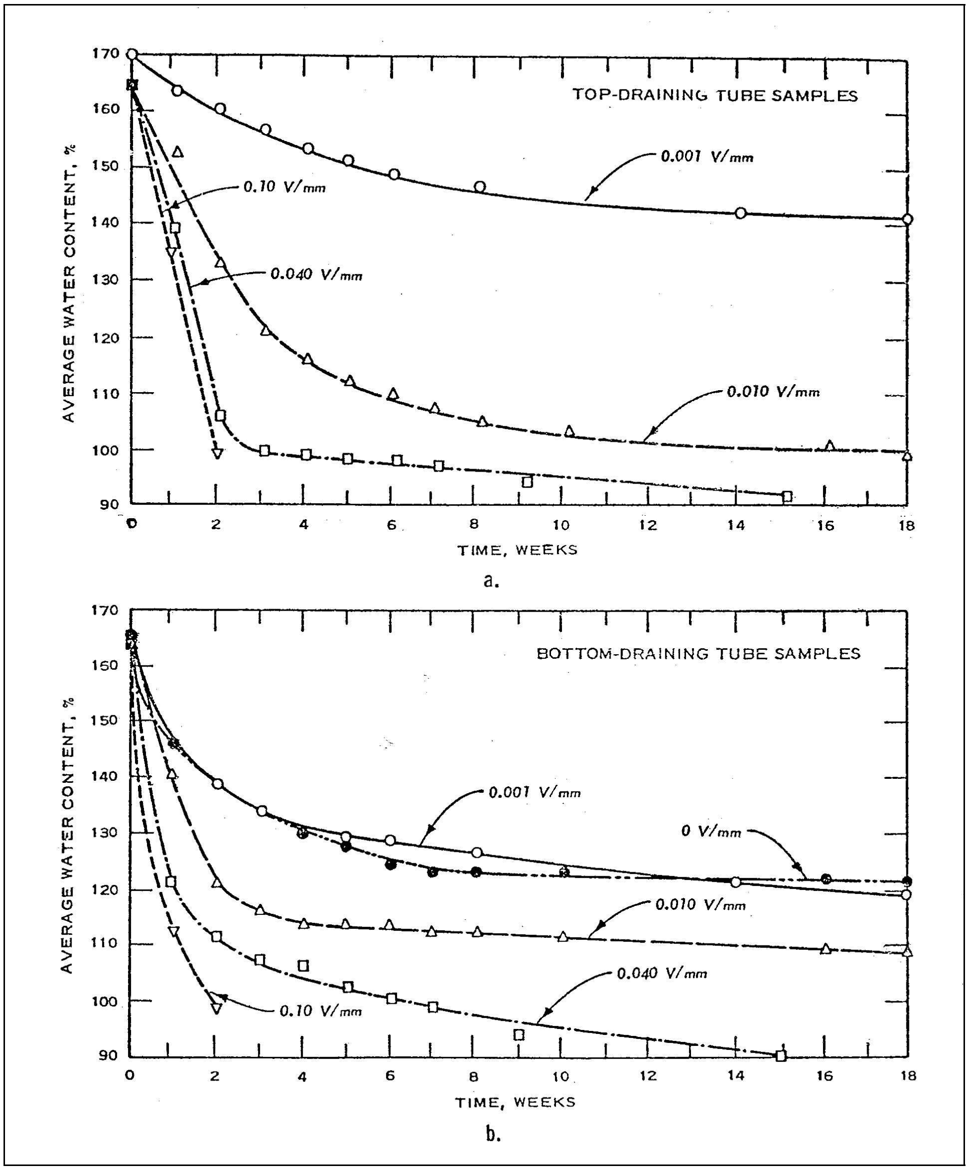

Figure 37. Results of electro-osmotic dewatering tests on laboratory samples of Upper Polecat Bay disposal area fine-grained dredged material

Table 30
Constant Voltage Laboratory Tube Samples Percent Volume Reduction

Sample No.	Voltage Gradient V/mm	Initial Water Content, %	Dry Side Water Content, %	Wet Side Water Content, %	Init. Vol. cu m	Final Vol. cu m	Percent Volume Reduction
8-10-22	0.001	166.2	113.9	117.0	.0004867	.0004548	6.5[1]
8-11-25	0.001	166.2	100.9	121.9	.0004867	.0004005	17.7
8-2-6	0.01	163.7	85.8	98.0	.0004867	.0003362	30.9
8-5-18	0.01	163.7	76.2	122.7	.0004867	.0003654	24.9
8-16-52	0.04	170.5	56.2	115.4	.0004867	.0003311	31.5
8-17-65	0.04	170.5	45.0	114.6	.0004867	.0003664	24.7
8-9-19	0.10	163.7	62.2	146.3	.0004867	.0003650	24.9
8-4-17	0.10	163.7	57.9	154.2	.0004867	.0003552	27.0
8-3-16	0.0	163.7	115.0	115.0	.0004867	.0004552	6.5

[1] Probable error in measurement.

To model field conditions of horizontal flow to a vertical well casing, a series of box model tests were performed. Soil samples of Mobile dredged material were placed into 229-mm cubical boxes at water contents above the liquid limit. The average initial void ratio was 3.93. The boxes were completely filled with dredged material, and one vertical slotted steel pipe cathode was placed in the center of the box, extending to the bottom. Steel electrodes placed in each corner of the box served an anodes. Results of the tests are given in Figure 38, which shows weekly average water content as a function of time and applied voltage gradient. The curves are very similar in shape to those obtained in the tube tests of Figure 37. Voltage gradient again had a marked effect upon dewatering rate. During the first 2 weeks of EO dewatering, the water content decreased from an initial value of 162 to 114 percent at a gradient of 0.04 V/mm, to 143 percent at 0.01 V/mm, and to 148 percent at 0.001 V/mm. After 13 weeks, the sample at 0.04 V/mm had decreased to 70 percent water content, with 50.2 percent volume reduction; the water content of the sample at 0.01 V/mm was reduced to 118 percent with volume reduction of 22.9 percent; and in the sample at 0.001 V/mm, the final water content was 125 percent with volume reduction of 22.2 percent. The large volume reduction in the 0.04 V/mm test (50.2 percent) was obtained by moving the anodes when the current demand and rate of water removal decreased. In this test there was no difference in final wet and dry side water contents.

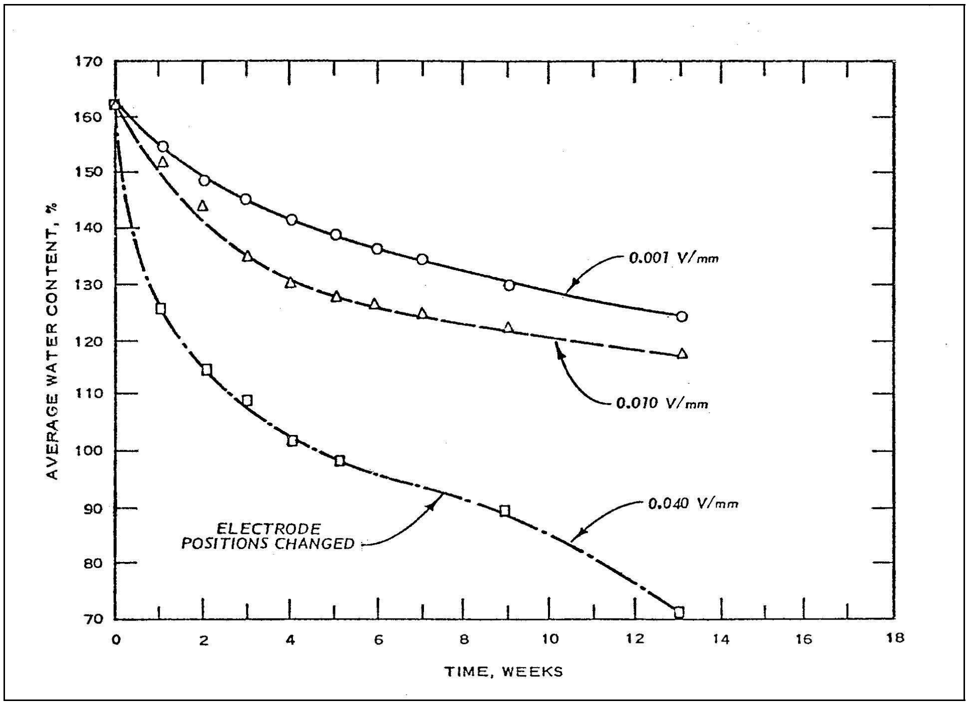

Figure 38. Results of electro-osmotic dewatering tests on laboratory box samples of Upper Polecat Bay disposal area fine-grained dredged material

Probable Energy Requirements

An estimate of energy consumption was made by averaging all UPB dredged material data by voltage gradient. These data are shown in Table 31 and indicate that an increase in voltage gradient results in a proportional increase in energy consumption/per unit of water removed at very low voltage gradients and a greater proportional increase at the highest gradient.

Summary

Results of the laboratory study indicated that low voltage gradient EO would dewater fine-grained dredged material and validated the DMRP staff hypothesis that dewatering could be conducted more economically at lower voltage gradients. If 1,000 L of water must be removed to create 1 cu m of additional dredged material storage volume, the power required would range from about 8.6 kWh at a voltage gradient of 0.001 V/mm to about 1,158 kWhr at a voltage gradient of 0.04 V/mm. The dewatering rate decreases in approximate linear proportion to decrease in voltage gradient. The contractor concluded that a voltage gradient of 0.001 V/mm was the lowest possible gradient technically feasible, and it was doubtful that this gradient would produce an acceptable field dewatering rate. A voltage gradient of about 0.005 V/mm as an “optimum” for

Table 31 Energy Consumption for Tube and Box Models				
			Average Energy Consumption	
Sample Type	Voltage Gradient, V/mm	Final-Initial Water Content, %	kWhr/gal	KWhr/L
Tube	0.001	60-80	0.046	0.012
Tube	0.001	100-170	0.018	0.005
Tube	0.01	50-80	0.45	0.12
Tube	0.01	90-170	0.52	0.14
Tube	0.04	30-60	4.0	1.1
Tube	0.04	80-170	4.6	1.2
Box	0.01	100-160	0.64	0.17
Box	0.01	70-150	2.4	0.63

economy and useful dewatering rate was recommended, and it was estimated that about 80 kWh would be required to create 1 cu m of disposal volume by EO dewatering. At a power cost of $0.02/kWhr, it would cost $1.60 for power to create 1 cu m of space.

Unfortunately, the laboratory study did not produce total desired drying of dredged material samples, except for one box model test where anodes were moved toward the cathode after power demand and dewatering rate decreased. In all other tests, dewatering rates decreased significantly after a few weeks because of drying at the anode and electrode polarization. As the "average" values were unacceptably high and no positive assurance was presented to show that periodic anode movement would rejuvenate the process and continue effective EO dewatering technical feasibility, the contractor suggested periodic current reversal as an alternative to anode movement in periodically rejuvenating the EO process. It was subsequently decided to conduct a field demonstration which would evaluate both techniques.

Design and Conduct of Field Demonstration

Criteria established for the design of the field demonstration included determination of the efficiency and cost of low voltage gradient EO dewatering, as well as the practicality of field installation and operation. The demonstration should provide data needed for development of a workable field dewatering system which optimized electrode spacing, water removal, and electrode material.

Equipment, Materials, and Test Layout

Design of the field installation was based on results of laboratory experimentation and preliminary recommendations from the contractor. The test site was located in the north center part of the UPB disposal area (Figure 2) and consisted of four experimental test sections, labeled Sections 1 through 4, laid out as shown in Figure 39. Test Sections 1 and 2 were designed to determine the effect of periodic anode movement toward the cathode while Sections 3 and 4 were designed to evaluate the effect of current reversal. Two cathodes (discharge wells) and four anodes were placed in each test section. Anodes in Sections 1 and 3 were placed 12 m from each cathode on opposite sides; in Sections 2 and 4, anodes were placed at 6.1-m distances. Figure 39 shows anode and cathode spacing as well as the location of piezometers and soil sampling locations.

Anode and cathode details for Sections 1 and 2 are shown in Figure 40; cathodes were 0.15-m-diam slotted steel pipe, approximately 2.7 m long. Anodes used in Sections 1 and 2 were initially 50.8-mm-diam steel pipes, but, during the experiment, these pipes were replaced by No. 18 steel reinforcing rods, later by 50.8-mm-diam graphite rods, and finally by massive 49.1-kg/m steel railroad rail. These successive anodes, with increasing mass, were used as it became apparent that corrosion rendered the anodes inoperative in relatively short periods of time. Figure 41 shows details of the 0.10-m-diam coke-breeze wells used as cathodes and anodes in Sections 3 and 4. Coke breeze is a form of granulated coke produced as a by-product of steelmaking and is commercially available in various gradations. The coke breeze used in this experiment had a gradation of 96 percent passing the U.S. No. 4 sieve, 10 percent passing the U.S. No. 10 Sieve, and 2 percent passing the U.S. No. 40 sieve, so that it was essentially coarse sand-sized. The contractor recommended that this material be used for both anodes and cathodes in the sections where current reversal was to be tried because its porous texture would allow upward water migration without the need for special cathodic wells. Contact between the coke breeze and the electrical system was made by implanting a hardened metal electrode in the coke breeze; this electrode was wired to the power system. Commercially available 2.3-kg Duriron electrodes were used initially.

The initial demonstration design proposed that float valve-actuated small pumps be used to periodically pump accumulated water from each cathode well. However, because of the difficulties encountered in the adjacent vacuum wellpoint experiment with corrosion of relays, electrical connections, motors, and pumps, caused by the corrosive environment and dredged material pore water, it was decided to eliminate the pumps and use EO flow principles to bring the water to the surface for flow by gravity to collector sumps where the quantity of flow could be measured.

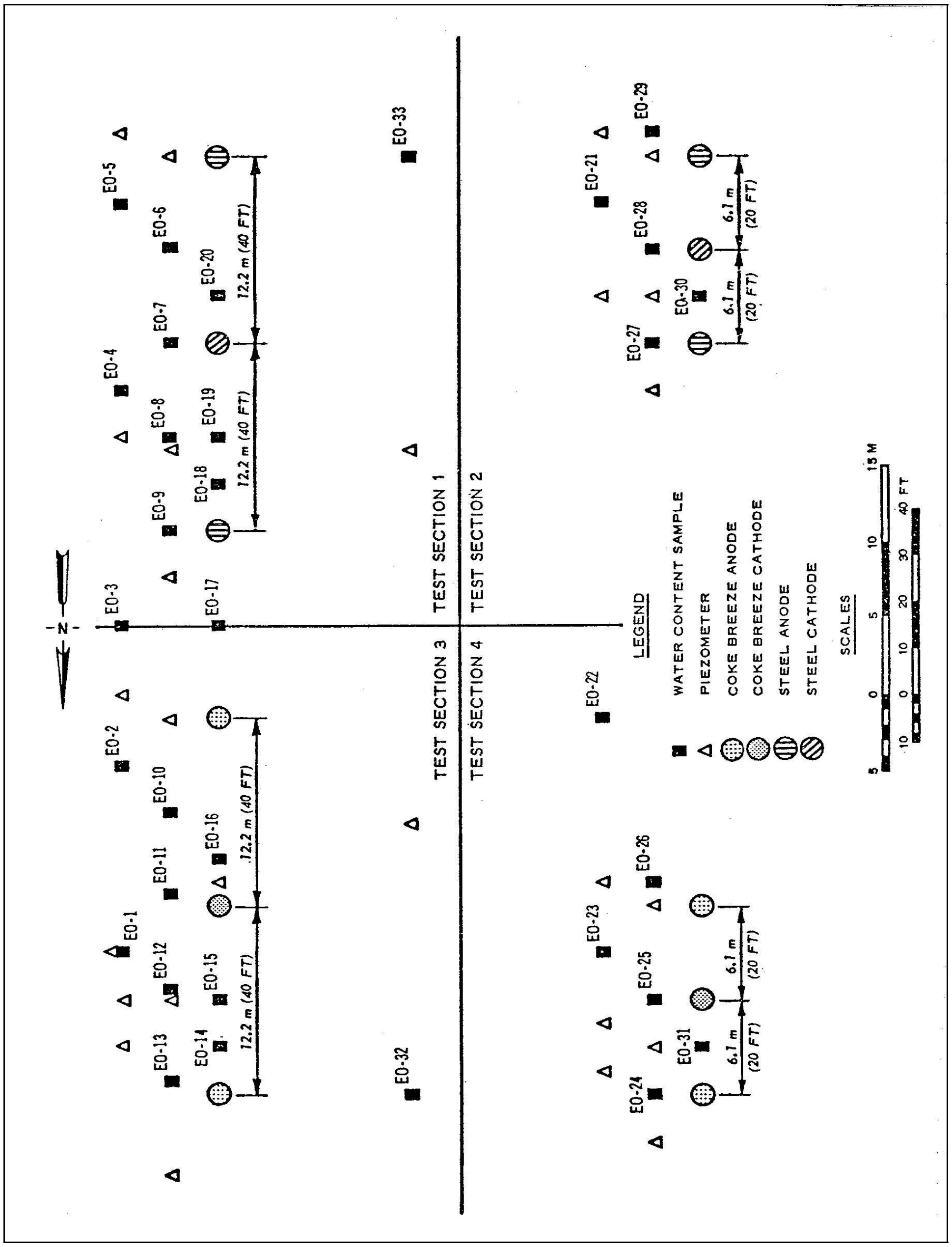

Figure 39. Layout detail and sampling locations for electro-osmosis field test site

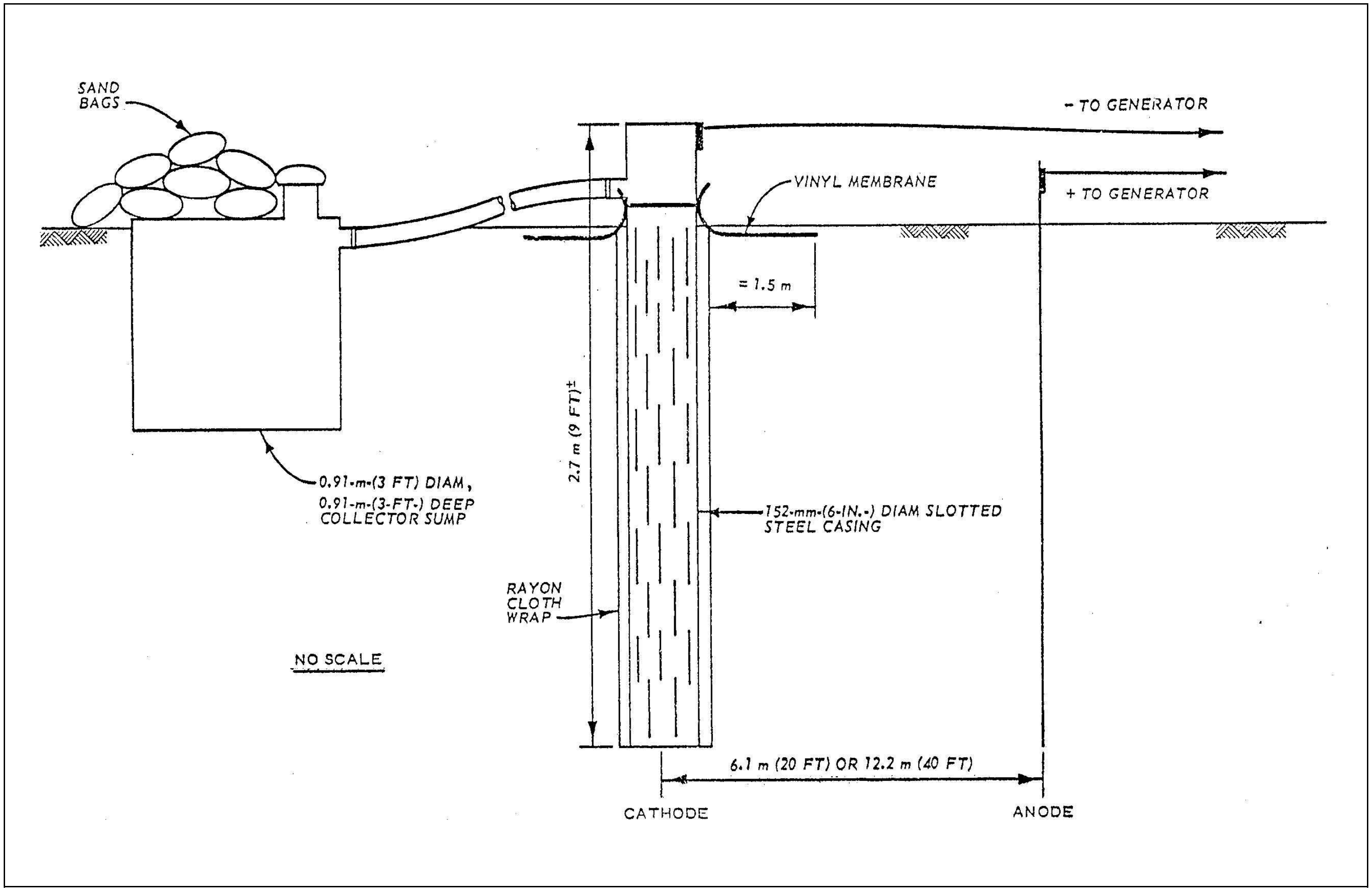

Figure 40. Anode and cathode details for Sections 1 and 2

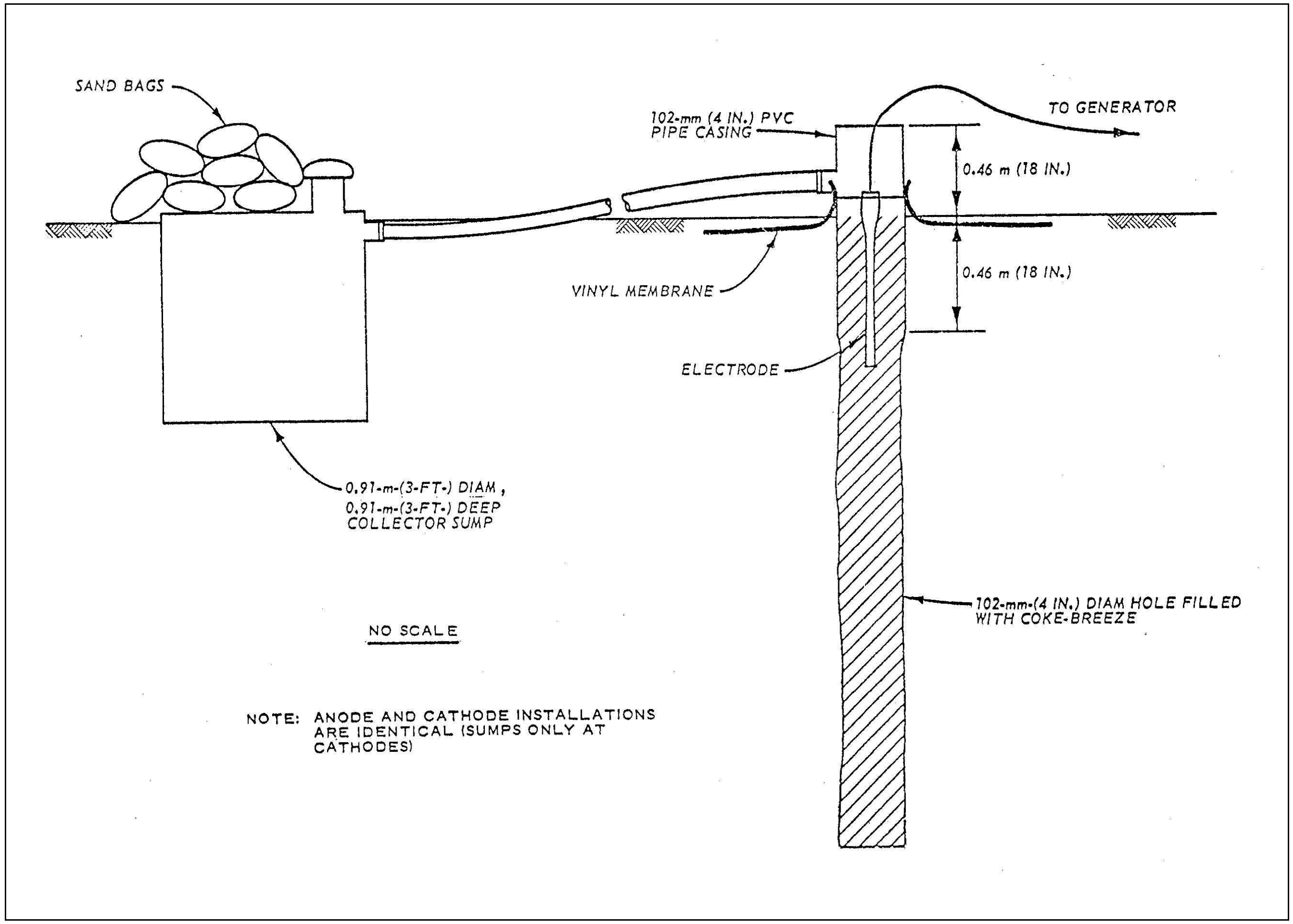

Figure 41. Anode and cathode detail for Sections 3 and 4

At each cathode, a 50.8-mm PVC drain pipe was initially provided 25.4 mm above the ground surface connecting the cathode casings with 209-L metal drums. The drums did not prove satisfactory as sumps; they corroded rapidly and were subsequently replaced by 0.91-m-diam, 0.91-m-deep, 12.7-mm-thick steel plate sumps. Water flowing from cathodes to the sumps was removed periodically by pumping. By not using pumps to remove the collected water from the cathode wells, pump reliability problems were avoided. However, once the cathode wells were filled, the EO flow came to the surface around the cathode where it filled surface desiccation cracks and drained away rather than rising inside the cathode wells and flowing out the PVC drains to collector sumps. As a result, considerable difficulty was encountered in measuring the actual flow from the test sections, and the ponded corrosive water around the cathodes became an inconvenience. A 3.05-m-sq vinyl membrane was placed around each cathode in an attempt to prevent water migration to the surface on the outside of cathode casings, and was partially successful.

Electrical power was provided by a diesel-powered 62-kW-rated direct current generator loaned to the Corps of Engineers by the Arizona Department of Transportation. However, the generator would produce only about 30 kW. Failure of the generator to supply rated power necessitated reducing the number of replicate test wells, so only one cathode discharge well and two anodes were used in each of the four test sections. Average test voltage was 51 V, resulting in a voltage gradient of 0.004 V/mm over the 12.2-m anode-cathode spacing and a gradient of 0.009 V/mm over the 6.1-m spacing. Amperage varied from about 195 A maximum to zero when electrical circuits were severed, usually as a result of severe anode corrosion. Apparatus and spacing for each test section is summarized in Table 32. Field installation, system debugging, and operation were accomplished during the period August 1976 - March 1977.

Table 32
Electrode Spacing and Voltage Gradients

	Section Number			
	1	**2**	**3**	**4**
Spacing, m	12.2	6.1	12.2	6.1
Cathode material	Slotted Steel Pipe		Coke-Breeze Column Duriron Electrode	
Voltage Gradient, v/mm	0.004	0.009	0.004	0.009

Test Procedure

The test was designed for application of a singe constant-voltage gradient within each test section. Current demand was monitored continuously by strip-chart recorder. Test disruption caused by anode corrosion, poor surface drainage, equipment malfunction, and procedural changes resulted in actual continuous application of current for between 70 and 78 days over the 8-month test period.

Five series of moisture samples, one series per month, were taken to determine soil moisture content changes with time. Samples were taken over depths of 0.6 to 0.9 m, 1.2 to 1.5 m, and 2.1 to 2.4 m. Based upon anticipated horizontal flow of water from anode and cathode, piezometers would not necessarily register a general drop in water level but would indicate drying front passage. Piezometer levels were measured daily for two weeks in November 1976 and weekly thereafter at locations shown in Figure 39. Water was pumped from the final (heavy steel plate) sumps in all four sections during a 45-day period from 29 January 1977 through 14 March 1977 and volumes were recorded daily.

Operational Problems

Two main problems were encountered during conduct of the demonstration which severely limited useful operations-surface water ponding and rapid anode corrosion.

Surface water ponding

The relatively flat surface of the test site and lack of available means for adequate means for adequate surface drainage caused precipitation to pond on the surface of the site so that the surface desiccation cracks were full of ponded rainwater at least 75 percent of the 8-month test period. It should be noted, however, that surface-drainage improvement from trenching was not conducted over the north part of the UPB area on purpose to retard dredged-material drying from desiccation and thus allow evaluation of other dewatering concepts. Also, during initial conceptual formulation of the EO field demonstration, it was believed that the higher EO permeability would result in effective subsurface dewatering and that hydraulic permeability-controlled gravity recharge of the EO-dewatered subcrust would be relatively small. Effect of this surface water on system behavior will be discussed later.

Initial operation of the EO system caused localized depressions to be formed around each anode, and shrinkage cracks occurred at least 1-m deep around the anodes. The depressions and surface cracks immediately filled with surface water, and attempts to remove the water were unsuccessful as more water simply flowed in from between adjacent desiccation cracks. Attempts to remove ponded water from the site with small hand-dug trenches were unsuccessful because the trenches could not be graded properly in subcrust above the liquid limit and 25 to 50 mm of ponded water still remained in surface desiccation cracks after the trenches were dug.

As mentioned previously, EO water came to the surface around the cathodes, and, in addition to causing problems in determining actual amounts of water removed, aggravated the surface water ponding problems. Excess water drained away through desiccation cracks, in some instances flowing toward to anode depressions. Attempts to intercept this “return flow” with drainage trenches were only partially successful. Placement of 3.05-m-sq vinyl membranes on the subcrust around each cathode helped to keep the EO water in the cathode wells until it could drain to collector sumps, but some upward EO water migration outside the membranes was still observed.

Anode corrosion

Chemical oxidation at anodes, when saline soil pore water is moved electro-osmotically, includes production of chlorine gas from chloride ions and dissolution of the steel anode. The two primary oxidation reactions are:

$$2\,Cl \rightarrow Cl_2\,(gas) + 2e$$

$$Fe^{O}(s) \rightarrow Fe^{+++} + 3e$$

In addition to the electrochemical dissolution of iron, generation of chlorine, which remains in intimate contact with an anode, creates a highly corrosive localized environment. Reaction of chlorine with the steel anode causes characteristic pitting, crevice corrosion, and general corrosion.

Reaction of chlorine gas with infiltrating surface water around the anode also produces hydrochloric acid (HC1), which is highly corrosive to steel. Steel anode corrosion occurred very rapidly during the test in Sections 1 and 2. The 50.8-mm OD steel pipe, used initially, failed from corrosion in about 4 days. The No. 18 reinforcing bar used next failed in a little over 4 days from fairly uniform corrosion over most of its 2.7-m length, with excessive corrosion at the tip of the bar and localized corrosion just below the subcrust surface. Graphite rods were tried next because of higher carbon corrosion resistance but were difficult to install without breaking and failed in 10 days. The steel railroad rail failed by corrosion in about 17 days. Duriron electrodes were used as connectors in the coke-breeze Sections 3 and 4. The Duriron electrodes lasted considerably longer than the steel anodes. However, it was impossible to determine the extent of corrosion and material loss of the coke-breeze that actually comprised the anodes in these sections.

Field Study Results

Table 33 shows the number of days that current was applied, continuous and total, for each month during the test period. These data include only days in which the applied current was greater than 25 A. Laboratory studies indicated that effective dewatering would not occur at voltage gradients possible when the test section current dropped below about 25 amps. During the first five months of the study (August - December 1976), the demonstration was operable for about 1 week per month. Operational difficulties were encountered, and experimental procedures and equipment were modified. During January 1977, 2 weeks of effective testing was accomplished, but the longest continuous test period was less than 1 week. Long-term continuous operation was finally achieved during February 1977 and the first 2 weeks of March 1977, for a period of about 45 days.

Table 33
Number of Days Current was Applied to Test Sections During the 8-Month Test Period (Maximum Number of Continuous Days/Total Number of Days)

Month	Section 1	Section 2	Section 3	Section 4
August	0	0	0	0
September	1/1	3/3	1/2	1/1
October	5/9	5/8	4/7	4/8
November	2/3	2/2	3/5	5/9
December	5/7	4/8	7/12	8/11
January	4/12	6/15	6/15	6/11
February	24/27	11/22	23/26	24/27
March	12/12	12/12	12/12	5/9
Totals for Test Period	39/72	34/70	38/78	39/76

The long debugging period resulted from a number of factors. Problems associated with anode corrosion and surface water ponding on the site were not fully anticipated in the field design. Heavy work loads on MDO personnel precluded continuous on-site supervision by professionals and resulted in frequent change of technical support personnel.

Soil-water content data collected during the months of August, November, and December 1976 are of minimal value as a measure of electro-osmotic dewatering as there were few days during the entire period in which there was an established continuous voltage gradient. In any case, the data showed no appreciable change in water content with time. Samples for water-content determination were not taken in January as MDO technicians were not available for sampling duties. Water-content data collected from August 1976 to March 1977 are summarized in Table 34, and selected water content data at various depths are plotted with time in Figure 42. It should be noted that these data show no appreciable change in water content. Piezometer levels and ground-surface elevation measurements over the entire 8-month period indicated negligible changes from EO demonstration operations, as compared with piezometer levels and surface elevations of the adjacent vacuum wellpoint control sections (Chapter 5).

In January 1977, daily measurements of water depth in the sumps were begun. A continuous record of water produced by electro-osmotic flow is available for period 29 January 1977 through 14 March 1977; these data are shown in Table 35. A considerable portion of the water removed from the soil at both the slotted steel pipe and coke-breeze cathodes remained on the surface

Table 34
Water Content with Depth and Time at Selected Sampling Locations

Location	Depth of Sample, m	Moisture Content, %				
		12 Aug 1976	3 Nov 1976	28 Dec 1976	28 Feb 1977	17 Mar 1977
E.O.-14	0.61-0.91	133.6	124.4	133.3	132.4	141.2
	1.22-1.52	108.3	97.1	103.1	98.4	113.4
	2.13-2.44	104.1	86.5	99.5	107.1	81.8
E.O.-18	0.61-0.91	120.8	122.8	136.9	123.8	122.7
	1.22-1.52	103.1	96.9	112.2	83.4	101.1
	2.13-2.44	79.8	88.1	86.4	73.9	87.1
E.0.-32	0.61-0.91	106.5	132.8	117.6	135.1	-
	1.22-1.52	112.3	107.1	130.7	123.3	-
	2.13-2.44	95.9	02.2	80.4	99.2	-
E.O.-33	0.61-0.91	127.2	116.0	136.9	129.2	-
	1.22-1.52	94.9	118.0	126.4	120.3	-
	2.13-2.44	82.4	80.4	86.6	102.8	-
E.O.-31	0.61-0.91	124.7	124.6	139.4	120.9	127.4
	1.22-1.52	128.1	116.4	114.9	123.6	122.8
	2.13-2.44	80.0	104.0	106.0	115.1	110.8
E.O.-30	0.61-0.91	98.0	118.0	124.3	117.3	150.8
	1.22-1.52	113.9	114.3	109.8	99.1	115.7
	2.13-2.44	103.3	92.9	77.8	93.3	82.9

around the well casings, and thus, water quantities derived from sump water level measurements and shown in Table 35 are probably conservative. In Section 1, 1,791 L of water were removed for the sumps and measured power consumption was 6.0 kWhr/L. Data for Section 1 show two distinct periods, an initial 20-day period of low water production and a final 25-day period in which 1,393 L were removed from the sump and power consumption was 4.4 kWhr/L. Data for Section 2 shows water-quantity measurements and power consumption relatively constant throughout the 45-day test period. Power used to remove water from Sections 3 and 4, 5.8 kWhr/L and 3.8 kWhr/L, respectively, are consistent with the results obtained from Section 1. The 4.0 kWhr/gal obtained from Section 2 was the lowest power demand measured.

Ponded water accumulation was enhanced during the period of continuous testing because during January 1977, it rained for 14 of 31 days, with accumulated rainfall of 141 mm. In February 1977, 47 mm fell in 6 days, and, during the first 2 weeks of March 1977, 74 mm fell in 7 days. Total rainfall for the month proceeding the test period and during the test period was 262 mm.

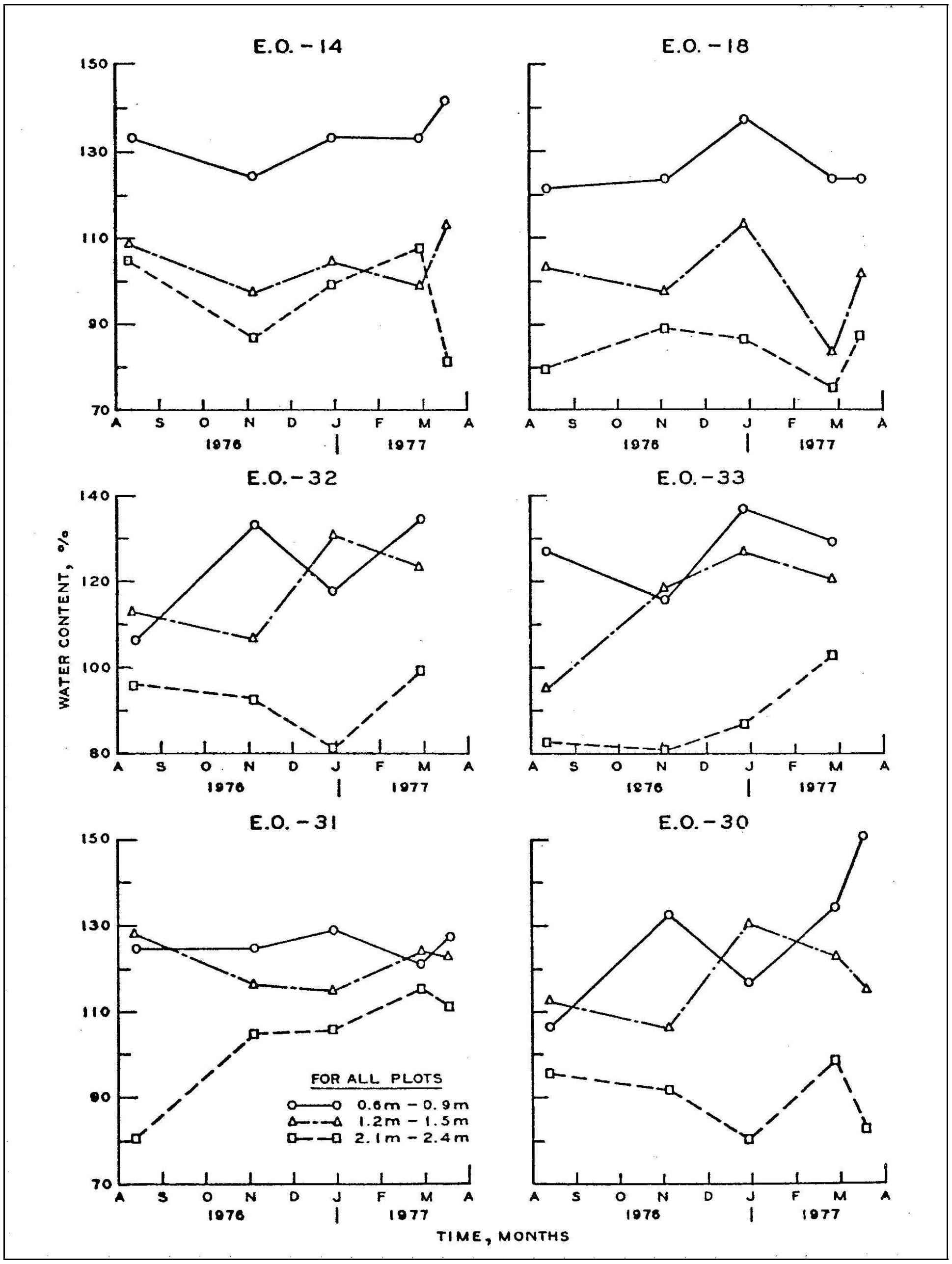

Figure 42. Dredged material water content versus time and depth at selected sampling locations

Table 35
Summary of Water Quantities Removed by Electro-Osmosis and Energy Consumption

Test section	**1**	**2**	**3**	**4**
Spacing: m ft	12.2 40	6.1 20	12.2 40	6.1 20
Type of cathode	Steel Casing	Steel Casing	Coke-Breeze	Coke-Breeze
Average voltage gradient: V/mm	0.004	0.009	0.004	0.009
Total water removed: L gal	2762 473	3986 1053	742 196	1204 318
Average amperage: A	195	78.8	78.7	83.8
Average voltage: V	51	51	51	51
Time period: days	45	44	45	45
Power consumption: kWhr	10,740	4,240	4,330	4,620
Power consumption per Unit volume of water removed: kWhr/L kWhr/gal	 5.0 22.7	 1.1 4.0	 5.8 22.1	 3.8 14.5
Power cost to remove 1 cu m (1 cu yd) of water with unit power cost of $0.02/kWhr: $/cu m $/cu yd	 120.84 91.71	 22.15 16.16	 116.81 89.28	 76.53 58.58

Discussion of Results

In evaluating the results of the EO demonstration, it is necessary to characterize the relative effect of the effort. Based on very approximate flow-net estimates, approximately 142 cu m of water was contained in the dredged material influenced by EO dewatering at the start of the demonstration. A total of 8 cu m was measured as removed from all four test sections, for an average water volume reduction of 5.6 percent. In Section 2, the greatest amount of water was produced, 4 cu m from 30 cu m approximately available, or a water volume reduction of 13 percent. For this best rate, dewatering should have resulted in an approximate average water-content reduction of 14 percent. Assuming the best (Section 2) EO water-removal rate would be maintained, approximately 5.2 months of EO operation would be required to reduce the water content from initial values near the liquid limit to near the plastic limit. As the amount of water actually measured was less than actually produced, the water-removal data are conservative and could be so by a factor of perhaps as much as two.

All the above calculations and estimates are essentially valueless, however, when compared to actual field measurements of water content, piezometer level, and surface elevation, which essentially showed no discernable EO-induced change during the test period. Based on conservative measurements of water quantity removed, some reduction in average water content should have been noted in Section 2, where maximum rates were obtained. For the other sections, the total amount of water removed was insignificant compared to the water available, and thus it is not surprising that measurements indicated little change.

Perhaps the lack of change in water content may be better explained by observations of system behavior. As previously noted, surface-water ponding caused water to continuously stand around anodes and on top of the subcrust, in desiccation cracks, between anodes and cathodes. It is highly probable that water infiltrating around the anode was then moved electro-osmotically through the dredged material toward the cathode. By continually recharging the dredged material with water from the anode wells the dredged material water content was maintained at initially high water contents despite the quantity to water collected at cathodes. Further, the standing water around anodes greatly contributed to anode-corrosion problems. While the exact extent of coke-breeze corrosion and anode loss could not be reliably estimated, the small amounts of water produced by Section 3 and 4 may have resulted from such behavior, as opposed to Section 1 and 2 where new anodes were placed as soon as major corrosion was evident.

The concepts of periodic anode movement in Section 1 and 2 and periodic current reversal in Sections 3 and 4 could not be evaluated because the dewatering rate per se did not decrease from drying around anodes.

Evaluation of Field Results

A general evaluation of the experiment would indicate it was unsuccessful. Numerous unanticipated operational problems were encountered and only partially solved, and the cost of removing water and thus creating disposal area volume varied from two orders of magnitude (Sections 1, 3, and 4) to one order of magnitude (Section 2) more than anticipated from preliminary laboratory testing. Based on observed behavior, an ex post facto evaluation of the design is as follows:

a. The decision to eliminate sump pumps in each cathode well was incorrect as it led to numerous operational problems and inability to correctly measure the amount of water removed. However, in a full-scale field application, pumps would be impractical because of the number, cost, and maintenance required.

b. In any realistic field application, water produced at cathodes must be allowed to run-off the test site through some sort of effective surface drainage system. Further, surface or other water infiltration around anodes must be prevented, or else this water will be introduced into the dredged material. To realistically protect the anodes from infiltration, it is suggested that a horizontal anode-cathode system by employed, with anodes at the bottom of the dredged material and cathodes at the surface, a configuration found workable by the U.S. Bureau of Mines (Sprute and Kelsh 1955).

c. The presence of sodium chloride in the dredged material pore water caused high current demand (and thus power cost) for the test sections, and disassociated chlorine caused severe anode corrosion problems. During initial design, it was anticipated that longer-term operation would remove the salts, and long-term power demands would be lower. The test was apparently not conducted long enough for such behavior to occur or to determine if it would occur under field conditions.

d. Only Section 2 produced an appreciable quantity of water. The 6.1-m anode spacing, the need for weekly or bi-weekly replacement of anodes, and the high power cost of $22/cu m to create volume make full-scale application impractical and non-cost effective even if surface water ponding problems could be overcome.

e. The need for continuous long-term monitoring of EO operation, because of changing current demand and required equipment maintenance requiring on-site availability of technical personnel, indicates that any full-scale use of EO dewatering should be conducted by contract with CE personnel as inspectors rather than operators of the EO process.

f. The concepts of periodic current reversal and periodic anode movement could not be evaluated as mechanisms for increasing EO dewatering efficiency in fine-grained dredged material.

g. The highly caustic (pH = 12) water produced at the cathodes is undesirable as an effluent from a water quality viewpoint.

Conclusions and Recommendations

Based on the results of laboratory and field testing described herein, the following conclusions were drawn:

a. The technical feasibility of electro-osmosis as a means for dewatering fine-grained dredged material placed in confined disposal areas was neither positively proved nor disproved.

b. The field experiment, as designed and conducted, indicated that EO dewatering was technically ineffective, operationally impractical, and non-cost effective.

Future field EO dewatering of fine-grained dredged material on any scale is not recommended unless adequate surface drainage can be provided. Any future testing should consider horizontal (rather than vertical) anode-cathode placement, and the best results are likely to be obtained with freshwater dredged material. Provisions should be made to dispose of potentially contaminated effluent.

11 Vegetative Dewatering Study

During Dredged Material Research Program (DMRP) Disposal Operations Project Planning Seminar II, held at the Waterways Experiment Station in January 1975 (DMRP 1976c), an invited technical expert suggested that while the DMRP has expended considerable effort in research toward mechanically dewatering fine-grained dredged material placed in confined disposal areas, little attention had been given to the use of vegetation as a dewatering medium. Invited DMRP consultants and Corps technical experts at the close of the seminar indicated a consensus that this concept should be studied by the DMRP staff and evaluated at the Upper Polecat Bay disposal area if feasible.

Desired Vegetation Characteristics and Site Suitability

As a result of the Seminar II recommendations, the DMRP staff began preliminary study of the potential of vegetation as a dewatering medium. As a result of this study, several factors were established as follows:

a. Using the transpiration capacity of plants to dewater and dry fine-grained dredged material should be relatively inexpensive with the combination of evaporation from the dredged material surface and transpiration from plants keeping the total evapotransporative drying at or very near the maximum evaporative potential of the climatic environment.

b. Development of vegetation root mat support capacity, allowing men and equipment to work on the disposal area surface, may be more important than dewatering from vegetation per se. Availability of a root mat would allow low ground pressure vehicles to work on a substantially thinner crust, thus allowing more nearly conventional construction operations to begin earlier in the dewatering process.

c. A need exists for vegetation growth and root-mat development during the fall and winter months (normal construction off-season, disposal area wet-weather season, and volunteer vegetation dormancy period) so that equipment could move onto the site more quickly in the warmer and drier spring and summer months.

d. Grass-type vegetation offers the best potential engineering behavior because it does not grow tall enough to shade and reduce wind speed near the surface and thus inhibit evaporation from crust desiccation cracks.

Additional benefits potentially derived from vegetation establishment include production of wildlife habitat during the drying/dewatering phase and improvement of aesthetic appearance of the disposal site.

In order for vegetation to provide effective dewatering and stabilization of dredged material, several prerequisites must be met. The plants should have the following characteristics and be:

a. Perennial and hardy.

b. Easily established at low cost.

c. Capable of rapid growth with minimum maintenance and fertilization.

d. Able to maintain a high transpiration rate.

e. Fast spreading with a thick root mat and low profile.

f. Of such type that they cause minimal interference in future disposal use of site.

The two objectives of a proposed vegetation dewatering study are thus to find plants with the desired characteristics for

a. dewatering and densification of the dredged material at the Upper Polecat Bay disposal site.

b. production of a vegetative root mat to help support men and equipment, both during the native vegetation growing period and during winter months.

Suitability of Upper Polecat Bay as a study site

A survey of vegetation existing at the UPB site in March 1976, indicated that the UPB site was in an early stage of succession. Types of predominant vegetation in the lower elevation areas (north and south ends) consist of *Panicum dichotomiflorum* (fall panicum), *Cynodon dactylon* (bermuda grass), and *Pluchea purparascens* (marsh fleabane). The higher elevations of the site were vegetated with *Baccharis halimifolia* (groundsel tree), *Eupatoruim capillifoluim* (dog-fennel), and *Aster subulatus* (wild aster). The dikes were vegetated with typical weedy species which are usually classed as “early invaders.”

Almost all plants which invaded the site are freshwater species, despite the saline composition of dredged material pore water, indicating that the ultimate volunteer community will probably be one typical of surrounding freshwater communities growing at the same elevation. Vegetation will probably become established as the internal water table is lowered by evaporation and as precipitation leaches salts from the crust.

Since invasion of the disposal area appears to be proceeding quite rapidly, a study which will monitor the invasion of natural plants and compare growth with those in planted plots should be the most effective approach.

In summary, the UPB site appears to be an excellent area to conduct a vegetative dewatering study. Although annual rainfall is high, the high nutrient status of the dredged material and long growing season offset this problem. Further, the site is typical of many such Corps disposal areas so that results of the study may be extrapolated to other locations with some confidence.

Method of Conducting Study

After DMRP staff study had established feasibility of a vegetative dewatering study at the UPB site, the study was initiated in two parts. Botanical aspects of the work were contracted through the U.S. Army Engineer District, Mobile (MDO), to the Marine Environmental Sciences Consortium, Dauphin Island Sea Lab (DISL), Dauphin Island, AL. Engineering aspects of the work were conducted by the WES Mobility and Environmental Systems Laboratory (MESL).

As a result of the UPB existing vegetation characterization and a survey of literature (Eleuterius 1974), four species of marsh grass were selected for transplanting to the test site: *Panicum repens* (panic grass), *Spartina alterniflora* (smooth cordgrass), *S. cynosuroides* (big cordgrass), and *Phragmites communis* (common reed). All are common in Alabama coastal areas and are accessible near most disposal sites. Available information (Lee et al. 1976) indicates that all four species are well suited to the physical character of the site and are good dewatering agents. Table 36 summarizes various favorable features of the four species selected.

Test Program, Materials, and Methods

Properties of dredged material

The northeast corner of the UPB site was selected for conduct of the vegetative dewatering study, as shown in Figure 2. At this location, the dredged material had been under ponded surface water from at least July 1975 until January 1976. Beneath a surface crust of about 0.08-m thickness, about 2.23 m of dredged material existed at water contents above the liquid limit and with general geotechnical properties as described in Chapter 3. To estimate agricultural potential of the dredged material, soil samples were sent to the Soil Testing Laboratory at Auburn University, Auburn, Alabama. Results of pH tests ranged from 5.1 to 7.0, well within the range of tolerance of all species selected for transplant.

Table 36
Suitability of Selected Marsh Plant Species for Use as Potential Transplants in Dredged-Material Disposal Areas

	Environment				Root Depth		Growth Rate			Regeneration		Dewatering	
		Fresh	Brackish	Saline	Shallow	Deep	Slow	Moderate	Rapid	Poor	Good	Poor	Good
Genus Species	pH												
Phragmites communis	3.7-8.0	X	X		X	X			X		X		X
Spartina alterniflora	4.5-8.5		X	X					X		X		X
S. cynosuroides	4.3-6.9	X	X	X	X				X		X		X
Panicum virgatum	4.5-7.5	X	X		X				X		X		X

Rooting Depth: Shallow = < .6m; Deep = > .6m.

Description of test plots

Eight rectangular test plots 27.5 m × 64 m and designated I-VIII were placed in the northeast corner of UPB site in areas of minimum crust thickness. Plots were surveyed and staked by personnel of the WES MESL. The existing 0.15 to 0.23-m surface crust and vegetation were turned under during the period of 17-21 May 1976 by repeated passes of the Riverine Utility Craft (RUC). Each test plot was staked into subplots by MESL personnel, as shown in Figure 43.

Transplant materials

Transplant materials included 2,850 individuals each of *Spartina alterniflora, S. cynosuroides, Panicum repens,* and *Phragmites communis.* With exception of *S. alterniflora*, all transplanted specimens were obtained from natural populations adjacent to the study site. *S. alterniflora* plants were obtained from a small population adjacent to the north weir of the South Blakeley Island (Lower Polecat Bay) disposal area. Plants were then transported via the RUC to the planting site. Efforts were made to dig younger, smaller plants from the edges of populations, but this was not completely possible because of the numbers required.

Plants were dug in clumps with soil intact, bagged, and placed in shade under a shelter. Individual healthy culms with adequate root systems were immediately separated from the clumps, placed in plastic bags, and taken to the appropriate plot for planting.

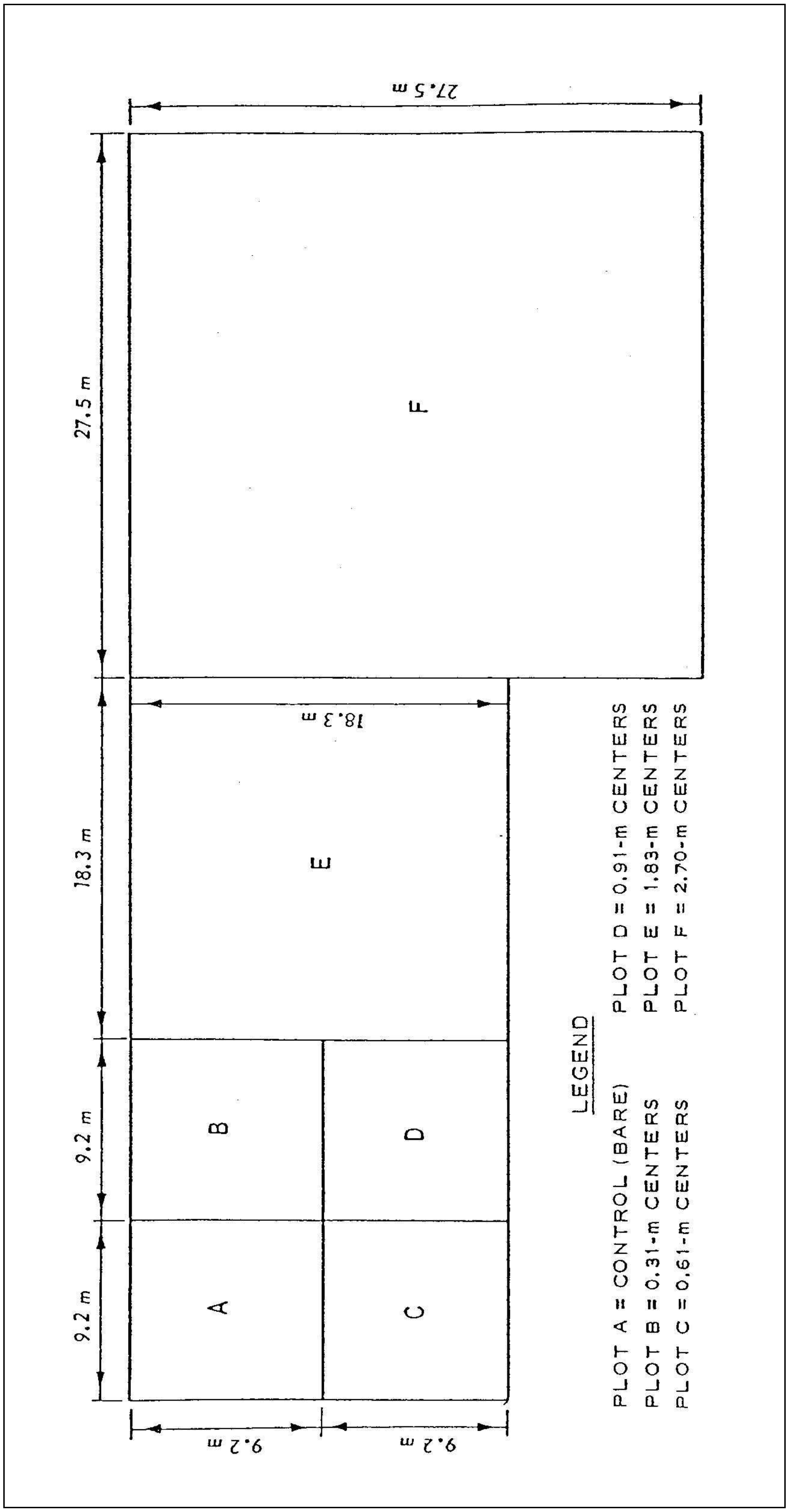

Figure 43. Subplot layout used in vegetative dewatering test plots

Transplant operations

After a period of 4 weeks, the dredged material in the prepared test plots had formed a desiccation crust approximately 0.05 to 0.10 m thick and could support the weight of a person if some care was taken when traversing the area. During the period 14-18 June 1977, MESL personnel staked the needed subplots in each test plot. Actual transplant operations were started on 17 June 1976.

Two replicates of each species were transplanted, located as shown in Figure 44. Within each plot, individual culms were planted in three subplots 9.2-m-sq, 0.3 m, 0.6 m, and 0.9 m apart, respectively (Figure 43). A fourth 9.2-m-sq plot was treated with Triox, a soil sterilant, to serve as a control without vegetation. Culms on 1.83-m centers were planted in a subplot 18.3-m-sq while a final subplot 27.5-m-sq was planted with culms 2.75 m apart.

The surface crust was broken with a mattock, and a hole of sufficient depth to reach below the crust to wet dredged material was made. Individual culms were inserted in each hole, and the soil pressed firmly around the plant. During planting, one test plot of each species was treated by placing Up Start, a rooting hormone, in each transplant hole.

Efforts were made to minimize transplant shock to the plants by completing the entire process, from digging to planting, as rapidly as possible. The maximum time from digging to planting was 15 hr (overnight), but such occurrence was rare. When unable to return plants to the ground immediately, they were placed in shade with their roots protected in plastic bags.

Planting was completed on 8 July 1976. All transplanting was performed manually, including digging, separation, and replanting in prepared plots. Man-hours required for each work element of the four species used are given in Table 37. Man-hours required for planting represents the effort over an entire test plot with subplots planted on varying centers as indicated. Records for each spacing were not maintained.

Natural vegetation plots

Five Natural Vegetation Control Plots, 9.2-m-sq, were established to monitor rapidly invading natural vegetation for comparative purposes. Locations are indicated in Figure 44.

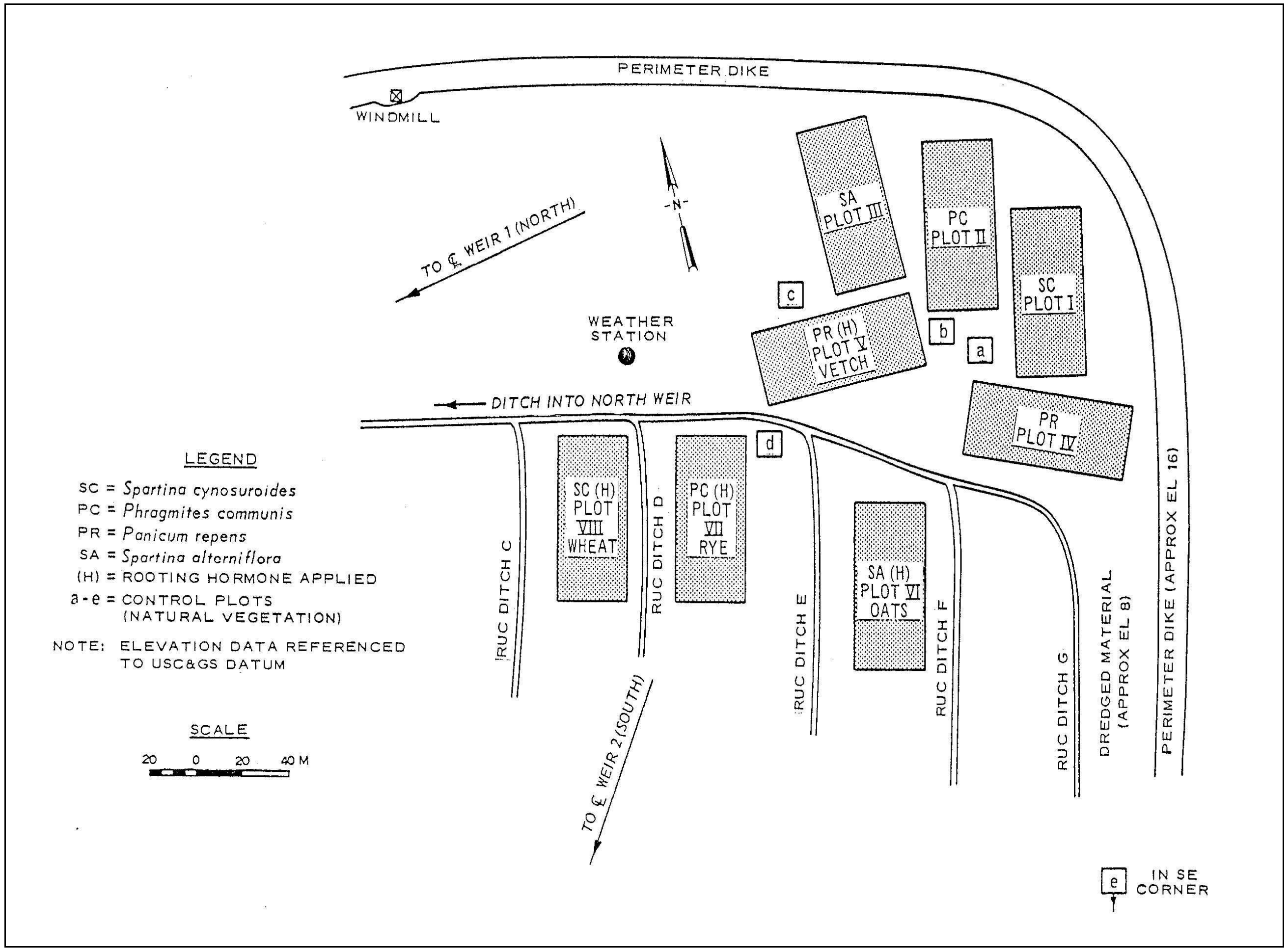

Figure 44. Location of, and species transplanted in, vegetative dewatering test plots

Table 37
Man-Hours Required to Transplant all Plants to the Eight Prepared Test Plots

Species	Digging Clumps	Sorting Clumps	Replanting Clumps	Total
S. alterniflora	28	6	88	122
S. cynosuroides	60	32	46	138
Panicum repens	6	18	42	66
Phragmites communis	216	(as dug)	40	256

Winter cover seeding

Following recommended planting schedules of the Alabama Cooperative Extension Service (ACES), four test plots (V-VIII) were seeded with Hairy Vetch (*Vicia villosa*), Oats (*Avena sativa*), Winter Rye (*Lolium perenne*) and Winter Wheat (*Triticum aestivum*) respectively, in mid-October, 1976. The existing 0.15- to 0.20-m surface crust was not disturbed. Instead, seeds were broadcast over the entire 27.5-m × 64-m test plot surface utilizing hand-operated chest-type rotary spreaders. An application rate of 23 kg per plot was used and was in excess of all ACES-recommended rates. It was hoped that, by excess application, the percentage of seeds falling into cracks and other favorable locations would be increased. At this rate of application, time required for seeding was 2.5 to 5.0 hr per ha.

To determine a comparative rate of seed germination when covered, additional planting were performed on 23 November 1976. Four subplots, 4.6-m-sq, were seeded in Plot VI. This plot was initially seeded with oats, most of which were immediately consumed by birds. In the first subplot, furrows were made through existing crust and seeded with oats, then covered. Winter rye seeds were planted in furrows covered in a second subplot. Surface crust in a third subplot was broken with hoes and forks, and winter wheat seeds were broadcast over the surface. Soil was then raked over the seeds.

In some parts of the various test plots, surface salt crusts were observed. Salinity measurements with an A0 hand-held refractometer were made of surface waters in ponded rainwater at one of these locations in November 1976. Salinity levels ranged from 15 to 26 parts per thousand (ppt). To attempt establishment of vegetation in such areas, *Salicornia bigelovii* (pickleweed) plants were collected from salt marshes at Point of Pins, west of Bayou La Batre, AL. The species was then currently disbursing seeds into the hypersaline salt pans. Only dehiscing plants were selected, broken into pieces, and broadcast over the surface of the fourth subplot, which had a noticeable concentration of salts on the surface crust. These plants were not covered.

Engineering measurements

After subplots were surveyed and staked in each of the eight test plots during the week of 14-18 June 1976, Casagrande-type porous stone piezometers with 12.7-mm risers were installed at each corner and in the center of each test plot, and an initial cross-section survey was conducted on a 3.05-m grid. Water-content samples and cone-penetration data were collected for the 0- to 0.305-m, 0.305- to 0.610-m, and 0.610- to 0.915-m depths in each subplot (Figure 43) of each test plot (Figure 44). Piezometers were placed in similar manner, and similar initial cross-section, water-content and cone-penetration data were obtained for the five natural vegetation test plots shown on Figure 44, beginning in August 1976.

Piezometers were read and 3.05-m grid cross-section, water-content, and cone-penetrometer data were obtained on monthly intervals through January 1977 for Test Plots I-IV and the five Natural Vegetation Control Plots, being terminated because the vegetation had become dormant. Data were collected for winter-cover Test Plots V-VIII through February 1977.

Results and Discussion

Transplanted materials

On 19 July 1977, approximately one month after transplanting was initiated in the test plots, the transplants were surveyed for survival rate. The survival rate in all plots was so small as to be insignificant (0-6 plants total per test plot). Even those classed as "alive" were considered so only because of the presence of some green tissues remaining and were in very poor condition.

The most probable explanations for the failure of the transplanted material are the time of year and climatic conditions under which the work was done. Daytime ambient air temperatures during transplanting were usually above 32°C, and precipitation was minimal during the two weeks following transplant. Both *S. cynosuroides* and *Panicum repens* were at anthesis and probably severely traumatized by the transplanting. *Spartina alterniflora* and *Phragmites* were approaching peak annual growth and were large, established plants. The RUC plowing, carried out to turn under existing vegetation, brought wet dredged material with saline interstitial water to the ground surface. Further, lack of significant precipitation during the last two weeks of June 1976 (less than 30 mm) resulted in evaporative deposit of salts on the crust surface without their being flushed and leached by precipitation. However, as noted in Table 36, all four transplants species had tolerance for brackish environment and two for saline environment. Thus, soil salinity is not thought to be predominant factor in transplant failure.

Eleuterius (1974) experimented with transplanting sods, sprigs, and rooted cuttings of the four species used in this study, as well as several others. Survival results and best transplant times determined by his work are summarized in Table 38. Its should be noted that the data of Eleuterius (1974) also show relatively low survival rates should be expected, even at best planting times, and one of the two planting efforts reported may be, for practical purposes, classed as a failure.

Table 38
Survival Percentage of Transplants by Sprigs to Marsh Sites at Gulf Park Estates Beach (A) and Simmons Bayou (B) Mississippi, and Best Time of Transplanting (from Lee et al. 1976)

Genus Species	Percent A	Survival B	Best Transplant Time
Panicum repens	95.3	2.3	January - March
S. cynosuroides	64.4	2.2	January
Phragmites communis	56.6	22.2	February
S. alterniflora	26.0	33.0	November - December

Numerous trips were made to the UPB site DISL personnel to observe results of the transplanting and the growth of natural vegetation in the disposal area. Periodic removal of transplants revealed steady rotting of below ground portions with total success observed as one resprouting *Phragmites* stem in Test Plot II (0.3-m centers). Natural vegetation immediately adjacent to the transplant test plots was dense, healthy, and included:

a. *Panicum dichotomiflorum* (fall panicum).

b. *Pluchea purpurascens* (marsh fleabane).

c. *Aster sublatus* (wild aster).

d. *Amaranthus cannabinus* (amaranth).

e. *Salicornia bigelovii* (pickleweed).

f. *Heliotropium curassavicum* (Heliotrope).

g. *Solidago ssp.* (goldenrod).

h. *Scirpus robustus* (bullrush).

i. *Cynodon dactylon* (bermuda grass).

Natural vegetation control plots

Five species of natural volunteer vegetation were found to dominate the invading cover of the UPB Site. Five natural vegetation-control plots were established to monitor natural invasion as indicated in Table 39. Root mat growth from natural volunteer vegetation occurred during the previous (1976) growing season.

Table 39
Species of Dominant Vegetation in Five Natural Vegetation Control Plots, Samples Obtained on 12 August 1976

	Occurrence in Study Plots (per 1/4sq m)				
Genus Species	**a**	**b**	**c**	**d**	**e**
Panicum dichotomiflorum	29.7	22.3	14.7	13.0	-
Pluchea purpurascens	-	3.0	10.0	17.0	-
Amaranthus cannabinus	-	3.3	1.3	3.5	-
Aster sp.	-	4.0	-	-	11.0
Phragmites communis	-	-	-	-	3.0

Natural vegetation-control plot list-count quadrant determinations were made in three square quadrants, along an east-west transect across the center 0.25 m of each control plot in August 1976. For encountered *Panicum dichotomiflorum* and *Phragmites communis*, where clumps or individual plants are often indistinguishable, numbers of stems were counted. Other data obtained were for individual plants. Based upon list-count data, dominant species were sampled in September 1976 for biomass determinations and growth characteristics, including stem. Results of list-count determinations in the five control plots are shown in Table 39 while dry weight biomass and shoot and root growth results are summarized in Tables 40 and 41. No attempt was made to separate roots and rhizomes, where both occurred, for biomass determinations. Distinct rhizomes in *Phragmites* allowed determination of maximum rhizome depth, but, in most cases, it was necessary to cut rhizomes at the edge of a quadrant. Thus, lateral rhizome growth was not determined.

The average depth of roots and rhizomes, excluding those of *Phragmites*, did not exceed 0.20 m, corresponding to the maximum depth to the disposal-area water table measured with piezometers at the control sites. These data confirm the accepted hypothesis that roots will go no deeper than necessary to find an adequate water supply.

Difficulties were encountered working in the dredged material. The sticky nature of the material below crust and the nearness of the water table to the surface resulted in loss of plant material in digging and washing. Therefore, biomass and growth determinations should be considered as conservative underestimates.

Table 40
Dry-Weight Biomass Determinations for Dominant Plants in Natural Vegetation Control Plots, Sampled 23 September 1976

Control Plot	Genus Species	Aboveground Biomass gm/.25 sq m	Belowground Biomass gm/.25 sq m	Total Biomass gm/.25 sq m
A	*Panicum dichtomiflorum*	42.6	45.3	87.9 87.9
B	*P. dichotomiflorum* Amaranthus cannabinus *Pluchea*	28.4 174.9 78.6 281.9	30.2 56.4 7.5 94.1	58.6 231.3 86.1 376.0
C	*P. dichotomiflorum* *Pluchea purpurascens* *A. cannabinus*	14.2 262.2 68.9 345.3	15.1 25.0 22.2 62.3	29.3 287.2 91.1 407.6
D	*Pluchea purpurascens* *P. dichotomiflorum* *A. cannabinus*	445.4 14.2 290.9 740.5	42.5 15.1 90.3 147.9	487.9 29.3 371.2 888.4
E	*Phragmites communis* *Aster sublatus*	168.3 1,509.2 1,677.5	124.5 221.1 345.6	292.8 1,730.3 2,223.1

Table 41
Growth Characteristics of Dominant Plants in Natural Vegetation Control Plots, North Blakeley Island, Measured 23 September 1976

Genus Species	Mean Height Aboveground mm (in.)	Mean Max. Vertical Root[1] Penetration mm	Mean Max. Horizontal Root Growth mm
Panicum dichotomiflorum	523	143	[5]94 ([4]72)
Aster sublatus	1,254	167	177
Amaranthus cannabinus	1,713	216	240
Pluchea purpurascens	885	174	121
Phragmites communis	223	208[2] 629[3]	210

[1] Includes roots and rhizomes.
[2] Depth of rhizome from surface.
[3] Maximum root penetration beyond rhizome.
[4] Mean clump width.
[5] Mean maximum horizontal distance from outer edge of clump on surface.

Winter cover

Two weeks after broadcast seeding, birds had consumed all oat seeds, and there was no apparent germination. Isolated plants of wheat and rye, approximately 50 mm tall, occurred in small patches. Four weeks after seeding, a few vetch seeds were beginning to germinate on high ground, but not in cracks and depressions. However, a few wheat and rye seedlings were noted and had reached 50- to 100-mm height in desiccation cracks and crevices. No germination occurred on the drier and more exposed crust surface.

Surface accumulations of salt crystals were common in all study plots during November. Salinity measurements of drainage ditch water and ponded water on the test plots yielded levels ranging from 15 to 25 ppt. Low rainfall and high salinity may have retarded further germination and growth during this period.

Results from the second (covered) seeding in November 1976 were slightly more favorable. A thin layer of soil over the seeds appeared to have enhanced germination and provided some protection from birds. Young leaves were obvious in wheat, oats and rye within 10 days of seeding. Based upon density of germination, rye was the most successful, and wheat the least successful. Oats again suffered from being consumed by birds though not as badly as when they were sown on the surface.

In February 1977, 3 months after planting, there was minimal germination and growth of winter wheat in Test Plot VI. Both vetch and *Salicornia* failed to germinate. By March 1977, all initially covered rye plantings had reached an average height of 20 cm.

Engineering measurements

Results of engineering measurements made monthly in Test Plots I-VIII during the period June - November 1976 and in Test Plots V-VIII during the period December 1976 - February 1977 are essentially useless. These data were supposed to indicate the relative changes in internal water table, soil- moisture content, soil-volume change, and soil-support capacity induced by different types of vegetation transplanted at different spacings. Because, for all practical purposes, the entire transplant and winter cover seeding programs were unsuccessful, the engineering data are valueless for their specified purpose and have been omitted from this report.

Engineering data were also collected at the five Natural Vegetation Control Plot locations. Water-content data for the upper 0.6 m (2 ft) and water-table level are plotted against time for Natural Vegetation Control Plots A through E in Figure 45. Site-rainfall data are also shown on this figure. As noted previously, most root growth occurred in the upper 200 mm at all locations except Plot E (*Phragmites communis*). Inspection of all water-content data for all Control Plots (each data point is an average of five determinations) does not reveal any noticeable trend of drying produced by plant growth. If anything, the data show that the vegetation had negligible effect on the water content-time relationship. The Control Plots were specially established at locations in the disposal area where surface drainage effects from the progressive trenching experiment

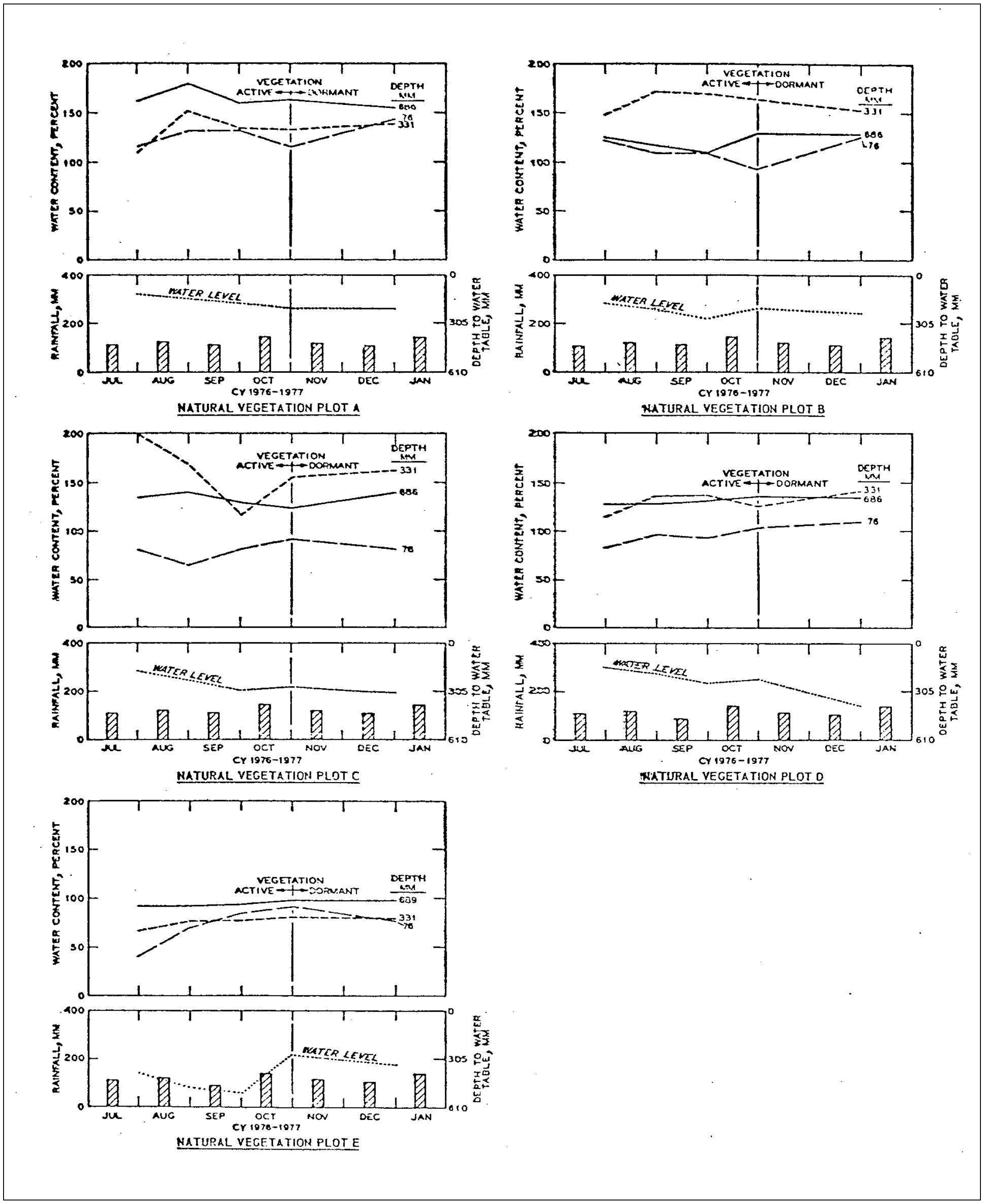

Figure 45. Water content, water level, and precipitation with time for natural vegetation control Plots A through E

(Chapter 4) would be minimized in an attempt to isolate the drying effect produced by vegetation. However, the data appear to show that, for these conditions, no appreciable drying occurs from vegetative growth, at least not of the magnitude useful in engineering operations. A general trend of water-table level increase is noted in all plots, corresponding with the onset of dormancy for the various types of vegetation. However, this behavior occurred over the entire site and is as likely to be related to reduced evaporation during cooler weather and trenching operations which restricted site drainage as it is to dormancy of vegetation.

Cone index and surface-settlement data taken in the Natural Vegetation Control Plots showed little change with time and were similar for both plots, as is to be expected based on observed water-content behavior.

Assessment of Results

As has been mentioned previously, for all practical purposes, the transplant and winter-cover seeding experiments may be classified as totally unsuccessful. Several reasons for the high fatality rate have been advanced by personnel involved in the research, including transplanting at the wrong time of the year, unfavorable climatic conditions, and high salinity concentrations in upper surface soil from RUC mixing in test plot preparation. However, it should be noted that a higher survival rate was observed among plants which had been mixed and torn by the RUC rotors than was obtained from plants of the same species placed with some care. Winter cover seeding was also ineffective, except when seeds were placed in furrows and covered, a procedure which, for all practical purposes, appears uneconomical.

It should be noted, however, that nature provided a rather thick and luxuriant vegetative cover of both grass and woody-stemmed weeds over the entire site in a single growing season with the great majority of vegetation apparently invading from surrounding areas. Even during winter months, when most disposal area vegetation was dormant, winter cover provided by nature was considerably more dense and productive than winter cover established by these experiments. Because of the negligible drying effects measured in the densely vegetated Natural Vegetation Control Plots, vegetation establishment should be viewed as a means for obtaining root mat support capacity, improving disposal area aesthetics, and providing wildlife habitat, and should not be expected to produce dredged material drying of engineering significance.

From a cost-effectiveness viewpoint, the expense of transplanting in large disposal areas may be substantial, assuming enough soil-support capacity exists for carrying out transplant operations. Based on data provided in Table 37, and average of 365 man-hr/ha is needed for transplanting, or an approximate cost of $1,100 per hectare, assuming low cost ($3.00/hr) labor is available. On this basis, a total cost of $37,800 would be required to conduct transplant operations over the entire UPB site. When the relatively high probable cost of conducting transplanting operations is coupled with the extremely low percentage of success obtained in the subject experiment (for whatever reason), it becomes fairly obvious that man may not compete effectively with nature in vegetation establishment. A more rational and cost-effective approach, with a higher probability for success, may well be for man to expend effort in removing ponded surface water and providing good surface drainage in confined disposal areas, so that the evaporative forces of nature may dry the material and draw down the internal water table, allowing further precipitation to leach objectionable salts. Once this procedure occurs in areas where vegetation surrounds the site, it appears that rapid natural invasion of the disposal area will occur. Future efforts toward optimizing disposal area operation and maintenance may then perhaps be oriented toward selective spraying, mowing, or some other techniques to remove or retard objectionable species of naturally invading vegetation and allow those with more desirable characteristics to gain ascendancy.

Conclusions and Recommendations

Based on the test conditions and results of this study, several conclusions may be formulated:

a. Vegetation transplanting operations conducted during the most opportune parts of the normal engineering construction season (i.e., late spring and summer) may have an extremely low probability of success, primarily because of mature plant size and resulting transplanting shock.

b. Establishment of winter cover by broadcasting does not appear to be a successful technique, even when an extremely high rate of seed application is used. Only a relatively small amount of seed finds its way into productive growing areas (crevices and desiccation cracks), and the saline content of the dredged material appears to inhibit germination.

c. Once ponded surface water had been removed from the disposal area and precipitation had leached salts from the upper surface crust, a thick luxuriant cover of varied native vegetation invaded the disposal area with surprising rapidity, causing an excellent root mat to be formed during the first growing season.

d. Even for dredged material with high salt content in the interstitial water, vegetation established on the site was primarily of freshwater species.

e. The amount of measured dredged-material dewatering produced by dense naturally established vegetation was, for engineering purposes, minimal. Primary benefits to be derived may thus accrue from root mat development, improved aesthetics, and wildlife habitat development.

It is recommended that CE field elements interested in obtaining vegetative cover in confined disposal areas expend maximum effort toward removing surface water and providing adequate surface drainage so that evaporative forces may dry the upper portion of the dredged material into surface crust and lower the water table, thus providing conditions more nearly conducive for the natural invasion of vegetation. Efforts to artificially establish vegetation probably have a low probability of technical success, and the cost of establishment will probably be substantial. It is suggested that, once natural vegetation has been established in the area, efforts be directed toward eliminating or controlling undesirable species and assisting the more desirable species to become dominant.

12 Conclusions and Recommendations

Conclusions

Based on the test data, procedures, analyses, evaluations, and assessments described herein, conclusions may be reached as to the technical feasibility, operational practicality, and cost-effectiveness of the various fine-grained dredged-material dewatering and densification methods evaluated during the Upper Polecat Bay disposal area field study. Criteria used for assessment were described in Chapter 1. Based on these criteria it may be concluded that:

a. Dewatering and densification of fine-grained dredged material by progressive surface trenching concepts is technically feasible, operationally practical, and cost-effective. Dewatering and densification result from evaporative water table lowering, with desiccation shrinkage occurring above the water table and consolidation under increased effective gravity stresses occurring below the lowered water table. Methods and procedures described herein may be used as a basis for designing effective surface-trenching dewatering programs.

b. The technical feasibility or nonfeasibility of using wind-powered electrical generation systems to provide electrical power at remote disposal area locations could not be positively established. However, demonstration results suggest this concept will be operationally impractical until marked advances are made in state-of-the-art equipment reliability and maintainability. Should technical feasibility and operational practicality be established at some future date, cost-effectiveness will depend upon the specific application and the availability of alternative power sources.

c. Conventionally installed vacuum wellpoints were found to be technically feasible and operationally practical for dewatering fine-grained dredged material placed in confined disposal areas. However, this methodology is not cost-effective when compared to other alternatives. Improved cost-effectiveness appears possible only by incorporation of existing or installed pervious drainage layers of greater areal extent into the system, thereby increasing the effective production rate per unit wellpoint.

d. Capillary wicks placed immediately after deposition and sedimentation of fine-grained dredged material were not found to be technically feasible as dewatering devices. Various types and spacings of wicks were ineffective in accelerating gravity consolidation through action as drainage channels and in dewatering dredged material by capillary action.

e. Hydraulically fracturing fine-grained dredged material in confined disposal areas by use of pressure-injected sand slurry was found to be a technically feasible and operationally practical method to construct internal drainage channels of large areal extent. Extremely limited and short-term data indicated that use of vacuum wellpoints in conjunction with sand-injected drainage layers might be cost effective and might produce rates of dredged-material dewatering and densification comparable with other successfully-developed methodology.

f. Periodic mixing of surface desiccation crust with underlying subcrust was found to be technically feasible and cost-effective in creating additional disposal area surface subsidence. However, the total subsidance obtainable by such methodology is limited, and the procedure reduces surface support capacity and degrades disposal area aesthetics. Thus, the technique was found to be operationally impractical.

g. All four underdrainage concepts evaluated (seepage consolidation, vacuum-assisted seepage consolidation, gravity underdrainage, and vacuum-assisted underdrainage) were found to be technically feasible, operationally practical, and cost-effective in dewatering initially placed lifts of fine-grained dredged material. Comparison of dewatering rates and unit-volume creation costs for underdrainage and vacuum wellpoint systems indicate that it is much more desirable to install drainage systems prior to disposal than after disposal. Maximum cost-effectiveness occurs when pervious drainage layers already exist or when suitable drainage material is available from prior dredging activities. Vacuum-assisted underdrainage produced more rapid dewatering rates and lower unit-volume creation costs than gravity underdrainage, indicating that vacuum pumping is economically justified. Seepage consolidation behavior with and without vacuum assistance produced similar unit-volume creation costs, indicating that the more operationally simple seepage consolidation may be a better alternative. Seepage consolidation produced densification rates comparable with those of underdrainage. Thus, its use should be considered in densification of confined dredged material placed underwater or when other considerations necessitate disposal-area surface ponding.

h. Technical feasibility or non-feasibility of electro-osmotic dewatering could not be positively established. However, demonstration results suggest that unless horizontal electrode configurations are used and freshwater dredged material is available, the process will be technically ineffective, operationally impractical, and not cost-effective.

i. Attempts to establish selected species of vegetation for dewatering purposes were unsuccessful, and had they been successful, they would have been non-cost effective. Naturally established vegetation of similar species produced dense root and surface growth in one growing season, providing increased root mat support capacity, improved disposal-area aesthetics, and produced considerable wildlife habitat. However, the increase in surface-crust thickness and the amount of dredged-material dewatering produced by such vegetation was insignificant for engineering purposes. While academically inconclusive, results of the demonstration suggest that best results can be obtained by providing disposal-area surface conditions conducive to natural vegetation establishment and that the benefits of vegetation establishment, while significant, probably do not include effective dewatering of fine-grained dredged material.

Recommendations for Implementation

Based on results obtained from this study, the following recommendations are made to CE field elements and other interested agencies concerning technically feasible, operationally practical, and cost-effective procedures for dewatering fine-grained dredged material placed in confined disposal areas:

a. Improvement of disposal area drainage by progressive surface trenching, to promote desiccation drying, should be considered initially by all agencies, because it is the most cost-effective and operationally simple methodology developed. In many instances, use of these concepts alone may provide adequate rates of dredged-material dewatering and densification.

b. In situations where progressive trenching concepts may not be used, underdrainage layers placed prior to disposal should be considered. When dewatering rates produced by progressive surface trenching are not adequate, the maximum possible rate may be obtained when surface-drainage improvement is combined with underdrainage. Vacuum pumping of underdrainage layers is recommended whenever possible. Whenever previously-deposited pervious strata are available in disposal areas, underdrainage should definitely be considered because the costs of unit-volume creation approach those of progressive surface trenching for this instance. It should be noted that successful application of progressive surface-trenching concepts will result in a system of stable trenches leading toward outlet weirs, which then may be filled with proper drainage material and collector pipes, providing an effective underdrainage system for dewatering subsequently placed lifts.

c. Seepage-consolidation concepts should be considered whenever dredged-material densification is desired, but the material must be submerged. This concept may be applicable when dredged material is placed underwater during filling of offshore confined disposal areas, or when surface ponding is necessary for other reasons.

Recommendations for Future Research

Based on the results of the research described and assessed herein, the following concepts are recommended for additional post-DMRP study:

a. The dewatering effect on multiple lifts of dredged material placement of previously placed underdrainage should be investigated. Research described herein has indicated that underdrainage concepts are useful in dewatering a single lift of material. However, data concerning the effectiveness of underdrainage in dewatering subsequent lifts of material, with and without further modification of the underdrainage system, would provide better information concerning the long-term effectiveness of this concept in confined disposal area operations.

b. The use of pressure-injected sand slurry to hydraulically fracture fine-grained dredged material and thus create internal drainage layers of large horizontal extent may hold the key to effective vacuum consolidation and dewatering of such material after placement in confined disposal areas. Dewatering rates produced by this experiment were an order-of-magnitude greater than those obtained from conventional vacuum wellpoints while techniques for installation were operationally simpler and less expensive. Increasing vacuum wellpoint unit production rates by an order-of-magnitude would result in cost comparable with those of progressive surface trenching. Further research, both in the laboratory and in the field, is therefore recommended for the concept of vacuum-consolidation dewatering with pressure-injected sand-drainage layers.

References

Bishop, A. W., and Vaughan, P. R. (1972). "Consolidation of fine-grained dredged material after hydraulic deposition," Report to National Ports Council of Great Britain by Department of Civil Engineering, Imperial College of Science and Technology, London, August, 1972.

Boyd, M. B., Saucier, T. R., Keely, J. W., Montgomery, R. L., Brown, R. D., Mathis, D. B., and Guice, C. J. (1972). "Disposal of dredged spoil-problem identification and assessment and research program development," Technical Report H-72-8, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Brown, K. W., and Thompson, L. J. (1977). "Feasibility study of general crust management as a technique for increasing capacity of dredged material containment areas," Technical Report D-77-17, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Casagrande, Leo. (1959). "Review of past and current work on Electro-Osmotic stabilization of soils," *Harvard Soil Mechanics Series No. 45*, Harvard University, Cambridge, MA.

d'Angremond, K., Brakel, J., Hoekstra, A. J. Kleinbloesem, W. C. H., Nederlof, L., and de Nekker, J. (1978). "Assessment of certain European dredging practices and dredged material containment and reclamation methods," Technical Report D-78-58, Dredged Material Research Program, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Department of the Army. (1971). "Dewatering and groundwater control for deep excavations," Technical Manual 5-818-5, Washington, DC.

__________. (1972). "Soil sampling," Engineer Manual EM 1110-2-1907, Washington, DC.

DMRP. (1974). "Dredged material research; notes, news, reviews, etc.," Miscellaneous Paper D-74-9, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

__________. (1976a). "Third annual report, dredged material research program," U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

__________. (1976b). "Dredged material research; notes, news, reviews, etc.," Miscellaneous Paper D-76-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

__________. (1976c). “Dredged material research; notes, news, reviews, etc.,” Miscellaneous Paper D-76-2, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

__________. (1977). “Dredged material research information exchange bulletin,” Vol D-77-1 (Jan), U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Eleuterius, L. N. (1974). “A study of plant establishment on spoil areas in Mississippi sound and adjacent waters,” Final Report Contract No. DACW01-72-C-0001, U.S. Army Engineer District, Mobile, Mobile, AL.

Haines, W. B. (1923). “The volume changes associated with variations of water content in soils,” *Journal of Agricultural Science* 13, pp 296-310.

Haliburton, T. A. (1976). “Mobile District field test site,” Information Exchange Bulletin, Vol D-76-6, Dredged Material Research Program, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Haliburton, T. A., Durham, G. N., Brown, K. W., Peters, R. E., and Delancy, T. B., Jr. (1977). “Effects of mechanical agitation on drying rate of fine-grained dredged material,” Technical Report D-77-10, Dredged Material Research Program, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

__________. (1978). “Perimeter dike raising with dewatered fine-grained dredged material at Upper Polecat Bay disposal area, Mobile, Alabama,” Miscellaneous Paper D-78-3, Dredged Material Research Program, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Halton, G. R., Loughney, R. W., and Winter, E. (1965). “Vacuum stabilization of subsoil beneath runway extension at Philadelphia International Airport,” *Proceedings of the Sixth International Conference on soil mechanics and foundation engineering* 2, Montreal, Canada, pp 61-65.

Hammer, D. P., and Blackburn, E. D. (1977). “Design and construction of retaining dikes for containment of dredged material,” Technical Report D-77-9, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Harrison, E., Keeschnik, W. F., Jr., and McGuire, W. J. (1954). “The mechanics of fracture induction and extension,” Transactions, AIME, Petroleum Branch (T.P. 3916), Vol 201, pp 262-363.

Hubbert, M. K., and Willis, D. G. (1957). “Mechanics of hydraulic fracturing,” Transactions, AIME, Petroleum Branch (T.P. 4597), Vol 210, pp 153-168.

Hvorslev, J. (1951). “Time lag and soil permeability in ground-water observations,” Bulletin No. 36, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Johnson, S. J., Cunny, R. W., Perry, E. B., and Devay, L. (1977). “State-of-the-art applicability of conventional densification techniques to increase disposal area storage capacity,” Technical Report D-77-4, Dredged Material Research Program, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Kjellman, W. (1952). "Consolidation of clay soil by means of atmospheric pressure," *Proceedings of the conference on soil stabilization*, Massachusetts Institute of Technology, pp 258-263.

Lee, C. R., Hoeppel, R. E., Hunt, P. G., and Carlson, C. A. (1976). "Feasibility of the functional use of vegetation to filter, dewater, and remove contaminants from dredged material," Final Report to the U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, MS.

Lacasse, S. E., Lambe, T. W., and Marr, W. A. (1977). "Sizing of containment areas for dredged material," Technical Report D-77-21, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Long, B. W., and Grana, D. J. (1978). "Feasibility study of vacuum filtration systems for dewatering dredged material," Technical Report D-78-5, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Montgomery, R. L., and Palermo, M. R. (1976). "First steps toward achieving disposal area reuse," Miscellaneous Paper D-76-16, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

O'Bannon, C. E., Segall, B. A., and Matthias, J. S. (1976). "A laboratory study to determine the feasibility of Electro-Osmotic dewatering of dredged material," IWD for Dredged Material Research Program for Work Unit 5A04, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Palermo, M. R. (1977a). "Characteristics of the Upper Polecat Bay disposal area as related to potential dredged material shrinkage and consolidation properties," MS Thesis, Department of Civil Engineering, Mississippi State University.

__________. (1977b). "An evaluation of progressive trenching as a technique for dewatering fine-grained dredged material," Miscellaneous Paper D-77-4, Dredged Material Research Program, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Parker, C. E., Pal, D., Vodraska, K. F., and Ciani, J. B. (1977). "Identification of alternative power sources for dredged material disposal operations," Naval Civil Engineering Laboratory, Technical Report D-77-32, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Richards, S. J., Willardson, L. S., Davis, S., and Spencer, J. R. (1973). "Tensiometer use in shallow ground-water studies," *Journal of the Irrigation and Drainage Division* IR4, ASCE, pp 457-464.

Salem, A. M., and Krizek, R. J. (1973). "Consolidation characteristics of dredging slurries," *Journal of the Waterways, Harbors, and Coastal Engineering Division* 99(WW4), ASCE.

Spotts, J. W. (1977). "The wick technique for dewatering contained dredged material," Internal Working Document, Dredged Material Research Program, Work Unit 5A10, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Sprute, R. H., and Kelsh, D. J. (1975). "Limited field test in Electro-Kinetic densification of mill tailings," *Bureau of Mines Report of Investigations No. 8034*, U.S. Department of Interior.

U.S. Army Engineer District, Mobile. (1975). "Final environmental statement, Mobile Harbor (maintenance dredging), Mobile County, Alabama," U.S. Army Engineer District, Mobile, Mobile, AL.

Willoughby, W. E. (1978). "Assessment of low-ground pressure equipment for use in containment area operation and maintenance," Technical Report D-78-9, Dredged Material Research Program, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Winter, C. (1972). "Three vane shear test in the vicinity of Polecat Bay, Mobile, Alabama," Unpublished Report, U.S. Army Engineer District, Mobile, Mobile, AL.

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14. ABSTRACT

Eight field demonstrations of various fine-grained dredged material dewatering techniques were conducted at the 34.4-hectare (85-acre) Upper Polecat Bay (UPB) disposal area of the USAE District, Mobile (MDO), between July 1975 and September 1977. The UPB disposal area was chosen for these field evaluations of promising dewatering concepts for several reasons: fine-grained dredged material existing in the disposal area was a highly plastic clay with appreciable montmorillonite fraction, one of the most difficult types of dredged material to dewater; interest and cooperative assistance were available from MDO; and the disposal area had proper climatic conditions for year-round work.

The Dredged Material Research Program mission in this instance was to evaluate as many potential dredged material dewatering methods as possible in such detail that conclusions could be formulated relative to their technical feasibility, operational practicality, and cost-effectiveness in full-scale field application. Results of the various field demonstrations include:

a. Use of surface trenching concepts to promote improved surface drainage, evaporative drying, and consolidation of fine-grained dredged material was found to be technically feasible, operationally practical, and cost-effective.

(continued)

15. SUBJECT TERMS
See reverse.

16. SECURITY CLASSIFICATION OF: a. REPORT	b. ABSTRACT	c. THIS PAGE	17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES	19a. NAME OF RESPONSIBLE PERSON / 19b. TELEPHONE NUMBER *(include area code)*
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14. ABSTRACT (Concluded).

b. Technical feasibility of using wind generation systems to provide electrical power at remote disposal area locations was neither positively proved nor disproved.
c. Dewatering fine-grained dredged material with conventionally installed vacuum wellpoints was found to be technically feasible and operationally practical, but is not cost-effective when compared to other alternatives.
d. Despite promising laboratory results, capillary wicks were not found to be technically feasible, as the amount of dewatering produced by the devices was minimal.
e. Use of sand slurry to hydraulically fracture fine-grained dredged material and produce internal drainage layers of large horizontal areal extent was found to be technically feasible and operationally practical. Long-term research is recommended for this concept, as its use in conjunction with wellpoint systems may hold promise for rapid and cost-effective dewatering.
f. Periodic mechanical agitation and mixing of upper surface crust with underlying subcrust above the liquid limit was found to accelerate the rate of dredged material surface subsidence and thus to be technically feasible as well as cost-effective, although operationally impractical.
g. Use of underdrainage installed prior to disposal, including gravity and vacuum-assisted underdrainage and gravity and vacuum-assisted seepage consolidation, was found to be technically feasible, operationally practical, and cost-effective for dewatering single lifts of material.
h. The technical feasibility of using electro-osmosis to dewater fine-grained dredged material was neither positively established nor refuted by field demonstrations, but results suggest that, unless the system is installed prior to disposal, it is limited to fresh water dredged material, and electro-osmosis dewatering will be technically ineffective, operationally impractical, and not cost-effective.
i. Attempts to artificially establish vegetation for dewatering purposes were unsuccessful and, had they been successful, would not have been cost-effective.

Based on results of these UPB field evaluations, it is concluded that when dewatering rates produced by surface drainage improvement and evaporative drying enhancement are inadequate, improved surface drainage may be combined with improved underdrainage, supplemented with vacuum consolidation, if possible, to achieve the maximum possible dewatering rate.

15. SUBJECT TERMS (Concluded).

Dredged material
Fine-grained
Dewatering techniques
Surface trenching
Wind power systems
Vacuum wellpoints
Capillary wicks
Sand slurry
Underdrainage
Electro-osmosis