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CALCASIEU RIVER AND PASS DREDGED MATERIAL SEDIMENTATION STUDY PHASE 2 STUDY

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Final Report

by

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1 Introduction

Background

As part of the Army Corps of Engineers mission to maintain navigable waterways of the US, an issue that must be addressed is the proper handling and storage/disposal/reuse of dredged material. One option for the storage of dredged material is the use of a confined disposal facility (CDF). A CDF is a diked area where dredged material is placed, either by mechanical methods of dredging or by hydraulic dredging. A CDF can either be located in an upland environment or in a near shore placement area. The conceptual design of the CDF requires an evaluation of the properties and settling behavior of the dredged material to be placed therein. This evaluation will provide information necessary to estimate storage requirements needed for the placement of dredged material along the Calcasieu River and Pass located in Calcasieu and Cameron Parishes in Louisiana

Purpose

The purpose of this report is to document and present the results of laboratory testing and computer modeling performed to project the long term storage capacity requirements and management alternatives for dredged material from the Calcasieu River and Pass, LA.

Objectives

The overall objective was to support the U.S. Army Corps of Engineers, New Orleans District in their mission to dredge the Calcasieu River and Pass and to provide storage of the resulting dredged material. To fulfill this objective, this study was performed in two phases. The Phase I study was performed for the short term disposal of material, 1-3 years. Settling tests were run to determine the settling behavior of the Calcasieu River and Pass sediments when they are hydraulically dredged. This will aid the District in managing the CDFs to meet their short term requirements as well as to provide data for the long term study. Data was also collected on the turbidity and total suspended solids (TSS) concentrations in the water column during the settling column tests. This facilitated the development of a correlation curve for turbidity and TSS that a contractor and/or inspector can use to quickly *estimate* TSS by *measuring* turbidity. Also, capacities of current CDFs along the Calcasieu River and Pass from mile 5-36 were evaluated to determine if they are sufficient to contain the initial storage volume occupied by the dredged material at the time of placement in the CDFs. The volume calculations were based on the safe dike elevation calculations made for the disposal areas. The Phase I report is provided in Appendix A.

The Phase II portion of the study deals with the long-term storage requirements for the dredged material from the Calcasieu River and Pass, approximately 20 year period. The consolidation properties of the dredged material were tested in order to evaluate long term storage requirements. The computer model Primary Consolidation, Secondary Compression, and Dessication of Dredged Fill (PSDDF) was used to model the consolidation of the material once it is placed in the CDF. Data from the Phase I portion of the study was used to perform the model runs. This information will aid the District in selecting alternatives and managing the disposal areas for the placement of the dredged material.

Several alternatives have been identified as having potential application to the dredging and disposal sites along the Calcasieu River and Pass. These alternatives have been grouped according to the categories of Dredging Management, CDF Storage and Management (existing and new facilites), Beneficial Use Options (marsh creation, mining existing CDF material), and Other Disposal Alternatives (ocean disposal, landfill). Some of these alternatives may be used as stand-alone options or several options may be used in combination. Each of these alternatives will be described in this report.

2 Dredging Management and Minimization

One approach that could be applied to Calcasieu River and Pass is to modify dredging management of the channel. Dredging management can be used to reduce dredged material quantities either by minimizing dredging requirements or by modifying the dredging operation to take up a smaller volume. A variety of options can be considered under this category to include erosion control to reduce sedimentation, construction of sediment traps by expanding the amount of over-depth dredging, matching dredging operations to existing disposal sites, selection of innovative dredges to increase solids capture, and use of innovative hydrodynamic dredging by fluidization techniques such as water injection. Each of these concepts will be described next along with their advantages and disadvantages and evaluation needs.

Erosion Control

The ERDC-EL is presently doing a study pertaining to the sediment budget for the Calcasieu River and Pass. Preliminary findings of this study show that bank erosion from the existing CDFs accounts for some portion (approximately 5 %) of the sediment that requires dredging from the waterway. Installment of erosion control structures to limit the volume of solids entering the waterway would reduce dredging requirements in the Calcasieu River and Pass. One method of controlling erosion that has been tested in the area is the use of rock dikes. The cost of this alternative would be significant to install rock dikes along the entire length of CDFs along the waterway, but, based on the study, it may help to reduce dredging requirements.

Another potential alternative for both reducing bank erosion is to expand the existing CDFs along the river-side toward the original CDF footprint. It appears that the original footprint of the CDF areas from mile 8 to 23 might have extended up to 200 feet into the waterway. This area could potentially be recaptured and filled approximately to the water level and planted with vegetation to provide a wave break. This planted area would assist in bank stabilization, thereby reducing the amount of solids re-entering the waterway. Additionally, dredged material could be used to bring the area up to the appropriate elevation, thus providing storage capacity for some dredged material.

Survey data, supplied by the New Orleans District, from 1972 and 1998 were compared to determine erosion rates of banks along the Calcasieu River and Pass. This data shows that some areas along the river have eroded a great deal in the past years. From Mile 8 to Mile 9 (along disposal area H) the erosion along the west bank has averaged approximately 424 feet from 1972 until 1998. Along Miles 9 to 11 the west back has eroded 503 feet from mile 9 to 10, 298 feet from Mile 10 to 11, and 417 feet from Mile 11 up the placement on the rock diked disposal area. The east bank along disposal area F did not erode a severely as the west bank . From Mile 9 to 10 there was little difference in the two surveys. From Miles 10 to 11 and 11 to 12 the erosion was 137 feet and 118 feet, respectively. From Miles 12 to 17 there was little erosion on the west bank of the river, due mainly to the area being small disposal areas that have rock protection. The east bank from Mile 12 to 17 showed that disposal area E decreased by 218 feet while disposal area D only decreased by 100 feet. Disposal areas 23 and 22 receded by approximately 150 feet. Miles 18 and 19 showed that the east bank along disposal areas 19 and 17 receded by 230 feet and 171 feet, respectively. The west bank along this area receded by 167 feet for mile 18 and 257 feet for mile 19, along disposal area 16S. From mile 20 to 24 the east bank showed little recession, averaging approximately 50 feet. The west bank along disposal area 16N and disposal area 15 showed the bank receding 290 feet. The west bank from mile 22 to 23 receded 190 feet but little recession was measured above mile 23.

This alternative would be designed similar to a nearshore CDF in which a berm approximately 2 feet above Mean High Higher Water (MHHW) would be constructed along the river-side original perimeter of the existing CDFs. The berm would be constructed so as to contain hydraulically dredged material, but to allow water flow into and out of the area. Upon placement of dredged material behind the berm up to the appropriate elevation, vegetation would be planted in such a manner as to withstand the wash caused by passing ships. As the dredged material consolidates over time, additional dredged material may need to be added to maintain the proper elevation. Elevation of the area is critical, as the area must be high enough support plant growth and to provide a wave buffer, yet low enough to provide the water level necessary for wetland plants. This alternative would require further investigation to determine whether it is feasible to construct the necessary berms and whether the immature plants could endure the wave wash before they are stabilized. If vegetation cannot be stabilized, a hardened structure such as a rock dike could be used to protect and retain the newly-placed dredged material.

Sediment Traps

Sediment traps are used in several Districts to extend the time between dredging events. Mobile District uses sediment traps on the upper reaches of the Mobile Ship channel with great success. Because navigation channels act as sediment traps for materials that are carried in suspension and settle naturally, deepening of the Calcasieu River and Pass by increasing over-depth dredging to provide additional storage could possibly be one solution to reduce dredging requirements. The answer to the question is not simple or easily supported at this time. The rationale for such a proposal is based on the fact that while the channel fills with material that is highly fluid and predominately silts and clays that settle poorly, these particles settle and densify (consolidate) slowly in a concentration gradient from the surface of the fluid mud to the bottom of the channel. Thus, there may be an opportunity to dredge material from the deeper, lower layers of the deposited sediment to essentially “drop the bottom” of the navigation channel, thereby, removing the more dense material in a maintenance mode. Initially, more work has to be done, but afterwards, maintenance dredging can probably be carried out less frequently and more economically due to the greater density. This concept has been proposed by Pat Langan, Mobile District, and Clark McNair, formerly Program Manager of the Corps Dredging Operations and Environmental Research (DOER) program as a management tool for fluid mud.

Matching Dredging Operations to Existing Disposal Sites

The practicality of this option depends upon the selection of appropriate dredges to meet project production, schedule, and disposal capacity requirements. In the past, various size dredges have been used but the selection of the dredge to meet capacity constraints has not been a major issue with the dredging of the Calcasieu River and Pass. Selection of smaller hydraulic dredges or slower dredging may yield higher solids contents, thus minimizing disposal volumes. Mechanical dredging also could be used produce high solids content dredged material.

Taking these recommendations a step further will require a careful study of existing and modified dredging and disposal practices. The advantages of such a demonstration would be to better define existing limits as well as achievable practical limits for existing dredging and disposal site management practices without immediately constructing expensive dike disposal. This option has potential to be a short-term solution if intermittent dredging or smaller dredges are used to match production with site capacity. However, a careful assessment of production rates versus surface area available for solids retention will be needed.

Another option is to consider increasing the frequency of dredging cycles. Dredging annually would reduce the annual solids loading to the disposal site resulting in the placement of thinner layers (lifts); thereby, allowing more time for retention of solids onsite while promoting faster dewatering and consolidation. Obviously, impact of such a practice might be the difficulty in budgeting for the annual dredging. A disadvantage of this practice would be that it might not be feasible if most of the shoaling occurs within the first year after dredging. A thorough knowledge of the shoaling rates is needed based on a review of the hydrographic surveys which are run on a quarterly basis. Given the extreme limitations on existing capacity, modifying dredging operations may be necessary, at least temporarily, until additional disposal alternatives can be acquired. However, this approach will not eliminate the need for additional future capacity.

Selection of Innovative High Solids Dredges

The fact that there are dredges that can pump high solids mixtures is well established (USEPA 1994, NRC, Marine Board 1997, PIANC 1998). Unfortunately, there are some agencies that believe dredges can create a thick mixture of dredged material without regard to the in-situ moisture content of the sediments in the channel. If high solids production is needed from a cutterhead suction dredge, the overriding factor will be on its ability to capture the sediment at near in-situ moisture content. Factors such as size (diameter) of the suction head, depth of burial, and swing speed will be important as well as the skill of the dredge leverman. Special hydraulic dredge heads have been developed to minimize resuspension of sediments and to capture in situ solids concentrations. These include the Environmental Disc Cutter (Boskalis) and Horizontal Auger (HAM and Mud Cat) to mention a couple.

The Environmental Disc Cutter (Figure 1) was developed by Royal Westminster Boskalis nv for dredging of contaminated sediments.

Figure 1. Environmental Disc Cutter (Boskalis)

The main features of the Environmental Disc Cutter are such that the soil is cut by means of a cylindrical box-shaped cutter with a flat closed bottom and vertical axis of rotation; the dredging profile is laid down in a Digital Terrain Model (both bottom and slopes) and followed automatically by the cutter; a suction mouth for the removal of cut material is situated inside the cutter; and a screen prevents the exchange of cut material with the surroundings. The screen consists of a shielding covering the full cutter height at those places where no soil is encountered. An adjustable shield is fitted automatically to the height of the soil as measured just in front of the cutter by means of echo sounders. The suction flow rate is adjusted automatically to the amount of soil being cut. For this, both the soil input and the soil throughput are measured. The orientation of the cutter can be adjusted in every direction even for the accurate dredging of slopes. Accuracy in delivery level is within 10 cm with a high density of mixture (average concentrations 70% solids). The Disc Cutter is used where sediment has to be dredged with high accuracy, resuspension and turbidity limitations are set, and, there is a need for high solid concentrations (e.g., because of long transport distances). Compared to conventional dredging equipment, dredging of thin layers does not affect the production level. Bean LLC Environmental Dredging has established a working relationship with Royal Boskalis Westminster nv and can possibly access (lease or purchase) the Disc Cutter head that can be outfitted on a U.S. Dredge.

The operating principle of the auger dredge is that the rotating auger cuts the material while actively transporting it towards the center of the auger. At the center, an oval suction mouth is mounted to pump out the disintegrated material. In more cohesive material, jets can be used to improve the flow of the mixture. At the back, a cover plate is closely fitted to the auger. To create a smooth bottom profile, the cover plate is provided with a cutting edge at the downward end. Dredging is executed in parallel tracks. To prevent dispersion, a silt screen may be deployed at the front of the auger. The silt screen consists of a metal plate at the top and a geotextile curtain at the front and the sides of the auger. The auger is suitable to remove thin layers while retaining mixture density and capacity. Depending on the sediment, the maximum layer thickness to be dredged in one cut can be 50 to 70 percent of the outer diameter of the auger. In other words, the auger dredge is suitable for bulk dredging. Examples of two auger dredges are the Ham 360 (Figure 2) and the Mud Cat (Figure 3) developed by Ellicott International, a Division of Baltimore Dredging Company.

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Figure 2. Ham 360

Figure 3. Mud Cat Shielded Auger.

The selection of a specialty dredge such as the ones above will depend on a variety of factors such as demonstrated need, availability, characteristics of sediments to be dredged, production needed, and costs. The demonstrated need will depend upon the ability of the dredging contractors to hydraulic dredge and pump slurries to the disposal site at near in-situ solids concentrations. This performance requirement should be considered first as part of the incentives for a contractor to use his own equipment. Obviously, the disadvantages of using specialty equipment may be the limited availability in the U.S. and potentially increased cost. Further assessment will be required to determine the feasibility of deploying such equipment.

Hydrodynamic Dredging

The concept of creating fluid sediment conditions with the objective of having the current or gravity carry them off to be deposited elsewhere is not new (USACE 1983). The Corps and others have practiced "agitation dredging" for years. The drawback with this type of dredging has been the deliberate attempt to resuspend sediments in the water column, which has drawn the disfavor from environmental groups and regulatory agencies. Nevertheless, the concept of "water injection" was brought to the U. S. for evaluation under the Corps Dredging Research Program (Clausner, Sardinas, Krumholtz, and Beauvais, 1993.).

Also, in 1999, the Working Group on Sea-Based Activities (SEBA) met in Hamburg as part of the OSPAR Commission. During that working session, the Central Dredging Association, CEDA, submitted it's technical note entitled, "Hydrodynamic Dredging: Principles, Effects and Methods," The note defines hydrodynamic dredging as *"the deliberate (re) suspension the fine fraction of sediment from the sea/riverbed with the aim of removing this material from the dredging areas using natural processes for transportation.* " (SEBA 1999). The note goes on to describe three different basic processes of hydrodynamic dredging depicted Figure 4.

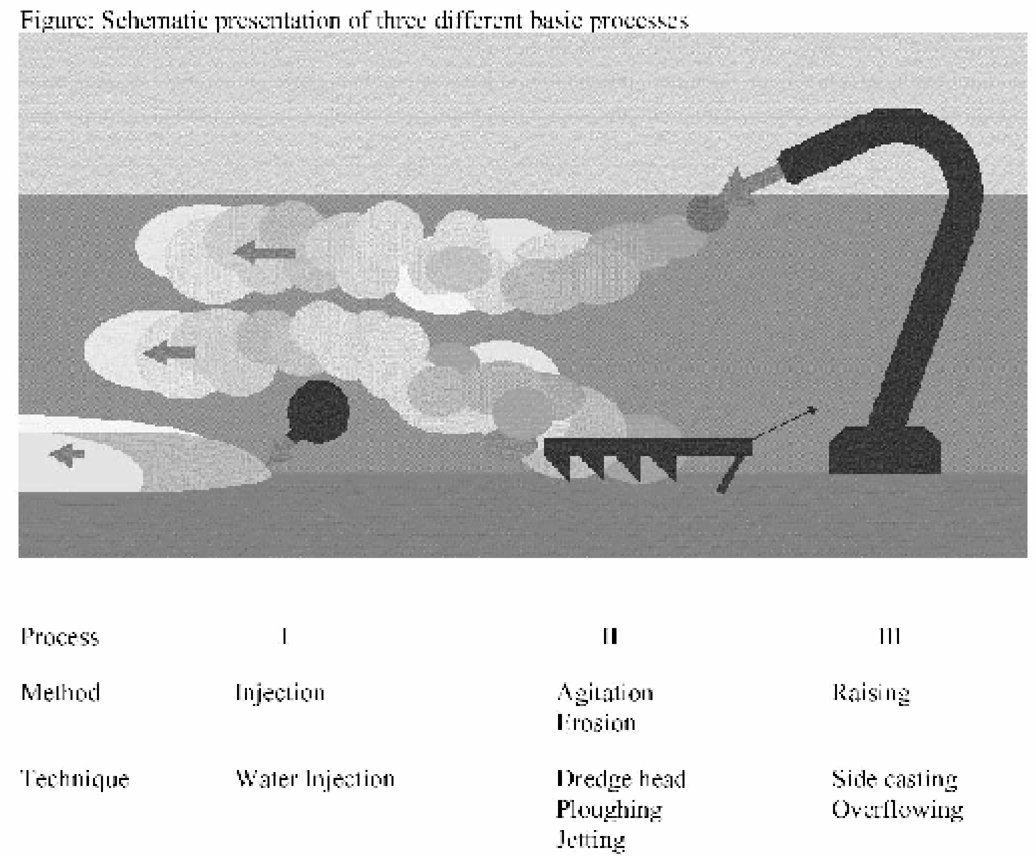

Figure 4. Schematic Diagram of Three Conditions of Hydrodynamic

The first process termed "water injection dredging" consists of mixing or diluting of the bed materials with water, causing a mud flow or density current that is transported to a down gradient location, normally a depression in the sea bed.

Agitation dredging is normally used to describe the second process that is based on a more vigorous, hydraulic or mechanical, stirring of the material, such that in suspended form it can be transported away by the current.

The third process occurs around rotating cutterheads, trailed dragheads and sliding dredged faces. This is a deliberate hydro-mechanical subsequent release of suspended material in the surface waters, for further movement by the natural flow.

Currently, there are two sources of hydrodynamic dredging techniques in the United States, Weeks Marine, Inc. and Silt (U.S.) Inc. Weeks has the patented "water injection dredge" better known as the WID. Figure 5 shows a photograph of the *WID* working in Michoud Canal, Louisiana. Silt (U.S.) has the patented "Wing Xcavator" that was brought to the United Stated in 2000.

Figure 5. Deployment of the WID in Michoud Canal, Louisiana

Figure 6 shows the deployment of the Wing Xcavator in Greens Bayou, Texas. Both of these technologies have been monitored by the DOER Innovative Technologies Demonstration Project of the DOER program (Francingues, Thompson, McNair and Saenz 2002, Clausner and Francingues 2002). Even though the Wing Xcavator and the *WID* use hydraulically generated currents to move bottom sediments, they do so in quite different ways. The Wing Xcavator uses high velocity and relatively large jets to scour material at the surface of bottom deposits and essentially washes them away. The *WID*, on the other hand, uses relatively small jets issuing low pressure water, injecting the water into surface deposits where pore pressures are increased, grain-to-grain contact is reduced, and the surface strata flows as a density current. The Wing Xcavator has the capability to scour and resuspend sediment deposits with its high velocity currents and carries the resuspended sediments away from the project site. The *WID* must have a favorable gravity gradient for liquefied sediments to flow along and a suitable area for sediments that are liquefied to be collected.

Figure 6. Deployment of the Wing Xcavator in Green Bayou, Texas

Conditions that favor use of hydrodynamic dredging are:

- erodable sediments susceptible to transport by fluidization
- potentially high current velocity, either natural or artificial
- areas down gradient of the dredging site where material can collect low levels of contamination, and
- environmental acceptance by regulatory agencies

The principal disadvantage of this technique is that the sediment is not removed from the waterway, only relocated within it, and may eventually need to be dredged to provide navigation depths down gradient. Consequently, more assessment will be required to determine the engineering feasibility and costs of hydrodynamic dredging for the Calcasieu River and Pass dredging project.

Dredging Management and Minimization – Summary

Several dredging management alternatives have been discussed for consideration for use in the Calcasieu River and Pass. Although these alternatives will not eliminate the need for additional dredged material alternatives, these option may be used to minimize both short- and long-term dredged material storage requirements. Further investigation of these dredging management options should be performed to determine their long-term economic impact.

3 Confined Disposal Facilities

Confined disposal facilities (CDFs) are engineered structures designed to retain dredged material solids and, in the case of hydraulic dredging, to provide acceptable suspended solids and/or contaminant concentrations for discharges to receiving waters. CDFs may be constructed as upland sites, nearshore sites with one or more sides in water, or island sites completely surrounded by water. CDFs constructed in water may become upland sites once the fill reaches elevations above the mean high water elevation.

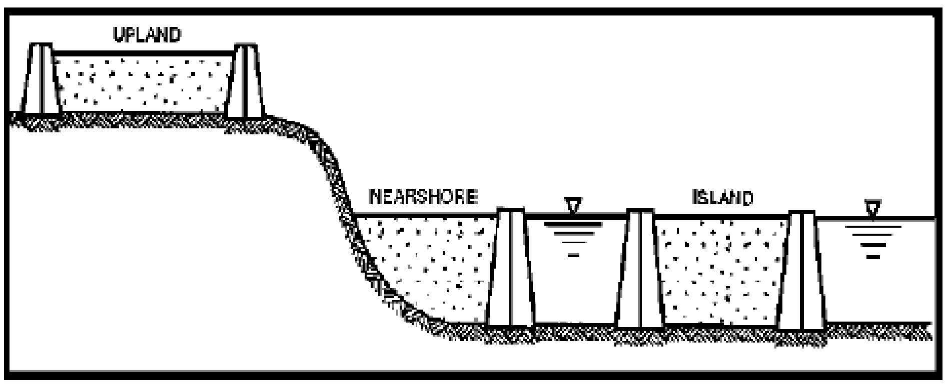

Figure 7. Schematic of upland, nearshore and island CDFs (after USACE/EPA 1992).

The two objectives inherent in design and operation of CDFs are to provide for adequate storage capacity for meeting dredging requirements and to maximize efficiency in retaining the solids. However, if contaminants are present, control of contaminant releases may also be an objective. Basic guidance for design, operation, and management of CDFs is found in EM 110-2-5027 (USACE 1987a).

Approximately 23 CDFs along the Calcasiu River and Pass are currently available for dredged material storage. Most of these facilities however are very near full capacity. Based on the Phase I study, there is insufficient capacity to contain even one more dredging cycle for all three reaches of the waterway. The Phase II portion of the study investigated the CDF area requirements to build new CDFs to provide for 20 years of dredging.

CDF CAPACITY REQUIREMENTS FOR THE 3 REACHES OF THE WATERWAY

Overview

The conceptual design for the upland CDF was developed for a storage capacity of 4,000,000 cu yd of material every other year for the lower reach , 4,500,000 every other year for the middle reach and approximately 6,500,000 cu yd of material every 5 years for the upper reach of the Calcasieu River. The conceptual design is for a 20 year time frame which equals to 44,000,000, 49,500,000 and 32,500,000 cu yd of dredged material for the lower, middle and upper reach areas, respectively. These volumes represent the in-situ volume of material before dredging. As discussed below, the site for the lower reach, 44,000,000 cu yd of dredged material, would occupy an area of approximately 2595 acres with a final thickness of 7.7 feet. The 49,500,000 cu yd in the middle reach would require 3175 acres with 8.0 final feet thickness. The site for the upper reach, 32,500,000 cu yd of dredged material, would occupy an area of approximately 1966 acres with a final thickness of 7.8 feet.

Capacity Requirements

An analysis was performed to determine the CDF storage areas required to safely hold the dredged material from the next 20 years of maintenance dredging of Calcasieu River and Pass. The river is divided into three reaches: the upper reach (mi. 24-36), the middle reach (mi. 14-24) and the lower reach (mi. 5-14). This analysis assumed that the in situ volumes dredged from each reach are 6.5 million cubic yards from the upper reach every five years, 4.5 million cubic yards from the middle reach every other year, and 4 million cubic yards every other year from the lower reach. Based on the column settling analyses performed in Phase I, dredging these volumes using a 30-inch dredge would result in the following volumes placed in the CDFs during each cycle: 7.67 million cubic yards in the upper reach; 6.89 million cubic yards in the middle reach; 6.81million cubic yards in the lower reach. (For the middle and lower reaches, this is based on the average in situ water contents of 165% and 150% respectively from the grab samples taken in the channel at miles 6, 8, 10, 12, 14, 16, 18, 19, 20, 21, 22, 23 and 24. The water contents ranged from 132% to 166% in the lower reach and 107% to 221% in the upper reach.)

Existing Disposal Areas

The existing CDFs for each reach are currently very near full capacity. The berm heights vary for the CDFs, but for the purposes of this analysis, it was assumed that a safe dike fill height of 11 ft is enforced, with actual dike heights of 13 feet to allow for 2 feet of freeboard. It is assumed that dikes that are currently higher will not be filled beyond the safe elevation and that dikes that are currently lower can be raised to 13 feet. The actual sediment fill height will vary depending on the size of the CDF. Normally, 2 feet of ponding must be accounted for, thus allowing a dredged material fill height of only 9 feet. For larger CDFs, ponding will only occur over a portion of the disposal area, thus decreasing the average ponded depth. Therefore the allowable sediment fill height will vary between 9 and 11 feet depending on the dimensions of the actual CDF, and thus the area requirements were computed over this range. Current average fill elevations for the existing disposal areas in each reach are 6.7 ft, 7.9 ft and 5.2 ft in the upper, middle and lower reaches. At the 9 foot elevation, the current capacities of the existing CDFs for each reach (assuming dikes built to 13 feet) are approximately: 5.57 million cubic yards in the upper reach (Disposal Areas 1-12B, excluding 6); 1.83 million cubic yards in the middle reach (Disposal Areas 13, 15, 16N, 17, 22, 23, D, E); 4.74 million cubic yards in the lower reach (Disposal Areas H, M, N). This capacity is insufficient to hold even one additional dredging cycle for each reach. Thus additional disposal areas will be required to continued placement of dredged material. At present, the District is also evaluation options to increase the safe dike heights by utilizing setbacks and/or geotextile reinforcement. Once details are available, the short- and long-term capacities gained by that option could be modeled. For now, it was assumed that the existing CDFs will be used for one more dredging cycle for each reach (in addition to other alternatives), then new CDFs will be required. Modeling was performed to determine the area requirements of the new CDFs.

Proposed New CDFs

To determine the area necessary to retain the dredged material over a 20 year span, one must consider the volume of material placed into the facility and the changes that occur as the in situ sediment is dredged and slurried, placed into the CDF where sedimentation occurs (as previously modeled using the SETTLE computer program), and the material consolidates under the weight of each layer and desiccates as ponded water is removed. Consolidation analysis was performed using the Primary Consolidation, Secondary Compression, and Desiccation of Dredged Fill (PSDDF) model (Stark, 1996). PSDDF is a one-dimensional program that uses finite strain consolidation theory, the C_α/C_c concept for secondary compression and, and an empirical desiccation model to estimate the changes in dredged material surface elevation with time.

Assumptions

Various assumptions were made regarding the proposed new disposal areas. The new CDFs will be filled to the safe dike height of 11 feet plus 2 feet freeboard, with average ponding between 0 and 2 feet. For the purpose of calculating the areas and volumes, square CDFs were assumed with the berms having 3V:1H slopes with 8.7 foot top widths. It was assumed that the facilities would be constructed at a bottom elevation of 0 ft msl.

Precipitation and evaporation data is used by the PSDDF model to estimate desiccation. Monthly average rainfall at Hackberry 8 SSW was used, and monthly evaporation data was estimated using the Hydrologic Evaluation of Landfill Performance (HELP) model as shown below.

Month	Rainfall (in.)	Evaporation (in.)
Jan	5.70	1.054
Feb	3.46	1.563
Mar	3.78	2.566
Apr	4.01	4.867
May	4.92	6.641
Jun	6.63	6.801
Jul	6.62	4.788
Aug	5.47	5.706
Sep	5.53	5.095
Oct	4.37	2.932
Nov	4.72	1.375
Dec	4.37	1.084

Laboratory self-weight (fixed ring) and oedometer consolidation tests were performed at ERDC on four samples; A and B from the middle reach, and C and D from the lower reach. The average consolidation curves from samples C and D were used in the PSDDF model for the lower reach. The average of curves A and B were used to estimate the consolidation properties for both the middle reach and the fines fraction of the upper reach.

There was little specific information available regarding the subsurface in the Calcasieu area. For the purpose of the consolidation model, it was assumed that a compressible layer exists between 0 and -50 ft msl. The incompressible layer is probably somewhat deeper, but the thinner the compressible layer, the more conservative. Consolidation testing was not performed on the subsurface material, and it was thus assumed to have the same consolidation properties as the dredged material for that reach.

PSDDF Results

PSDDF was used to model the consolidation of dredged material lifts placed at each dredging cycle and the resulting dredged material elevations over time. The lift thickness is dependent on the CDF area, and therefore the model was run iteratively, estimating the required area, determining the resulting final elevation at that area and then adjusting the area accordingly to reach the desired elevation. For the middle reach, disposal area requirements were modeled to be 3175 acres assuming a dredged material fill elevation of 9 feet (2 feet average ponded depth), and 2600 acres for a dredged material fill elevation of 11 feet (0 feet ponding). Similarly, for the lower reach, the disposal area requirements were estimated at 2595 acres and 2135 acres at the 9 and 11 foot dredged material fill elevations.

The analysis for the Upper Reach was performed differently due to the high sand content. The sand will likely fall out and pile up near the inflow pipe, rather than being incorporated into the fines layer. Consolidation will not occur in the sand and therefore the volume taken up by sand can be computed separately from the fines. Using SETTLE, the sand volume was computed as 1.33 million cubic yards per dredging cycle, or 6.65 million cubic yards over the five cycles in the 20 year span at a concentration of 1363 g/L. Allowing for two foot freeboard and no ponding, the sand will require an interior area of approximately 375 acres. The SETTLE model showed that the as-placed volume of fines will be 6.34 million cubic yards per cycle, at a concentration of 340 g/L (e=6.474). Several iterations of PSDDF revealed that approximately 1590 to 1298 acres would be required to handle the fines at dredged material fill elevations of 9 to 11 feet, resulting in a total footprint of 1966 to 1674 acres. Table 1 summarizes the results. Figure 8 shows the predicted elevation change as material is placed into the disposal areas and consolidates over time. Consolidation of the dredged material from the final placement in the existing disposal areas is shown in Figure 9 below.

Table 1. Summary Table of Estimated Future Disposal Area Requirements

	Lower	**Middle**	**Upper**
In situ volume dredged per cycle, yd^3	4,000,000	4,500,000	6,500,000
Dredging frequency, years	2	2	5
In situ void ratio	4.02	4.46	3.35
As placed dredged material volume, yd^3	6,813,365	6,889,127	7,666,138[1]
As placed void ratio	7.5508	7.3512	4.1261[1]
Assumed Properties:			
% Fines	93	89	54
Specific gravity	2.68	2.70	2.52
Consolidation curves[2]	C,D avg	A,B avg	A,B avg
Elevation of incompressible layer, ft msl	-50	-50	-50
Existing CDF Volume at 9 ft elevation, yd^3	4,738,204	1,828,948	5,573,323
Existing CDF Volume at 11 ft elevation, yd^3	7,139,674	4,747,237	10,361,464
Existing CDF average fill elevation, ft	5.24	7.86	6.86
Assuming dredged material fill to 9 ft:			
Aproximate area required (footprint), acres	2595	3175	1966
Average lift thickness, ft	1.64	1.35	2.48
Elevation 10 years after last placement, ft	7.71	7.96	7.77
Assuming dredged material fill to 11 ft:			
Aproximate area required (footprint), acres	2135	2600	1674
Average lift thickness, ft	1.99	1.65	3.05
Elevation 10 years after last placement, ft	9.47	9.75	8.5

[1] Values for the fines only, used in the PSDDF model are 6,335,975 yd^3 at a void ratio of 6.4737.
[2] Consolidation testing was performed on samples A, B, C and D. The resulting curves (void ratio vs. effective stress and permeability) for A and B were averaged, as were the curves for C and D. The specified curves were applied to both the dredged material and subsurface compressible layer.

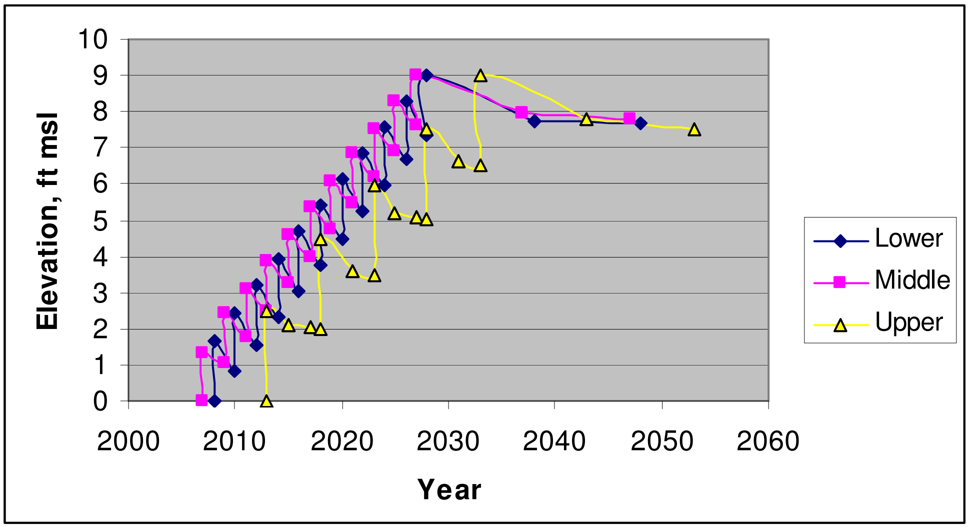

Figure 8. Elevation of Dredged Material in Proposed CDFs over Time (for sediment fill to 9')

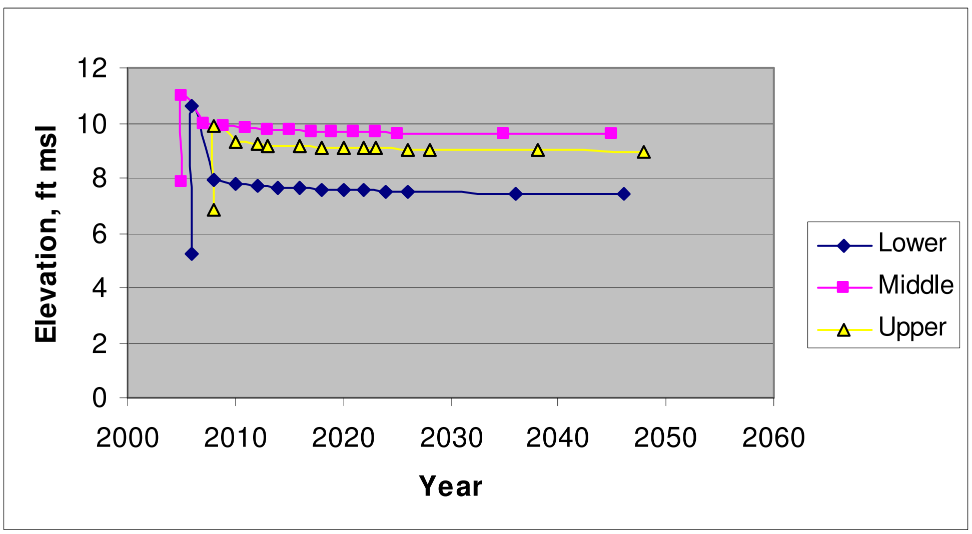

Figure 9. Average Predicted Elevation of Dredged Material in Existing CDFs over Time, Assuming One Additional Placement.

With hydraulic placement of the dredged material, outflow structures would be placed in the dikes to remove excess water from the dredged material. These outflow structures would consist of a weir or multiple weirs placed in the dikes. The weir would be used to remove excess water from the disposal area.

Potential locations for new CDFs have not yet been suggested. New CDFs could be placed in upland, nearshore or even island settings. One alternative might be to extend the existing CDFs out into the Calcasieu Lake. Also, extending the disposal areas towards the ship channel (as previously mentioned) would be another alternative.

To summarize, based on the given assumptions, three disposal areas totaling between 6410 and 7740 acres (10 to 12 mi^2) are needed to store the dredged material from 20 years of dredging

Calcasieu River and Pass. It should be noted that the values given here should be used for estimating purposes only due to the large degree of unknowns. Some of the unknowns include:

- proposed CDF dimensions, layout and elevations;
- subsurface properties (depth to incompressible layer, consolidation properties of compressible layer);
- consolidation properties of dredged material in the upper reach.

Once the planning phase begins, additional data should be collected regarding the subsurface properties and actual proposed CDF dimensions. Modeling could then be performed to better predict the lifespan of the new facilities. Furthermore, proper management of both existing and new CDFs is important to maximize the life and use of the facilities.

CONFINED DISPOSAL FACILITIES

Upland Confined Disposal Facilities

Upland confined disposal is placement of dredged material within upland (diked) CDFs. A true upland CDF would allow for all dredged material fill to be placed above the water table. CDFs constructed in water may become upland sites once the fill reaches elevations above the mean high water elevation. Upland CDFs are not solid waste landfills. They are designed and constructed specifically for disposal of dredged material and would normally have a return flow as effluent to waters of the United States. With such return flow, they would be regulated under Section 404 of the Clean Water Act. The issue of return waters and regulation under Section 404 is a major consideration. Placement of material in upland solid waste landfills is treated as an entirely separate alternative and is not covered in this report. Upland CDFs as described in this section are assumed to meet the requirements for regulation under Section 404.

Most sediment in the Calcaseiu River region is dredged hydraulically with dredges followed by disposal in upland CDFs. The three objectives inherent in design and operation of CDFs are to provide for adequate storage capacity for meeting dredging requirements, to maximize efficiency in retaining the solids, and to control contaminant releases to within acceptable limits. Basic guidance for design, operation, and management of CDFs is found in EM 1110-2-5027 (HQDOA 1987).

A principal design criterion of CDFs is to retain as high a percentage of the fine-grained sediment particles as practicable. This criterion was based on the findings of the USACE Dredged Material Research Program, which demonstrated that most chemical contaminants associated with sediments could be effectively contained through efficient solids containment. Since most contaminants in sediment remain attached to solid particles during dredging and placement in the CDF, this process is reasonably efficient for containment of contaminants.

A CDF is neither a conventional wastewater treatment facility nor a conventional solid waste disposal facility. What makes it different are the physical and chemical properties of the dredged materials placed in the CDFs. Wastewater treatment facilities are designed to receive water with low levels of solids. Solid waste facilities are designed to receive solids with very little water. Dredged sediments typically contain 10 to 70 percent solids, depending on the physical characteristics of the sediment and the dredging and handling techniques used. An effective CDF must borrow features from both the wastewater treatment facility and the solid waste disposal facility in a combination that is unlike either.

Hydraulic dredging generally adds several volumes of water for each volume of sediment removed, and this excess water is normally discharged as effluent from the CDF during the filling operation. The amount of water added depends on the design of the dredge, physical characteristics of the sediment, and operational factors such as pumping distance. When the dredged material is initially deposited in the CDF, it may occupy several times its original volume. The settling process is a function of time, but the sediment will eventually consolidate to its in situ volume or less, if desiccation occurs. Adequate storage volume must be provided during the dredging operation to contain the total volume of sediment to be dredged, accounting for any volume changes during placement.

In most cases, CDFs must be used over a period of many years, storing material dredged periodically over the design life. Long-term storage capacity of these CDFs is therefore a major factor in design and management. Once water is drained from the CDF following active disposal operations, natural drying forces begin to dewater the dredged material, adding additional storage capacity. The gains in storage capacity are therefore influenced by consolidation and drying processes and by the techniques used to manage the site both during and following active disposal operations.

Upland CDF applications

Upland CDFs are one of the most common disposal alternatives and such sites exist in most regions of the United States. The use of upland CDFs is extensive in the Atlantic and Gulf Coast regions. Many of these sites were constructed in areas adjacent to estuaries or tributary rivers near the navigation channels they were intended to serve. Some of these sites were constructed in wetland areas (prior to wetlands protection regulations) and have been filled to become upland areas. Large upland sites, some larger than 1,000 acres, are now in active use in the U.S. Army Engineer Districts, Wilmington, Charleston, Savannah, Jacksonville, Mobile, New Orleans, and Galveston. CDFs initially constructed in water and which are now upland sites are located in the Great Lakes area, California, and in the Puget Sound

Nearshore CDFs

Nearshore confined disposal is placement of dredged material within confined (diked) disposal facilities (CDFs) via barge, conveyor, bucket, pipeline, or other means at a site constructed partially or completely in water adjacent to the shore. A true nearshore site will take advantage of the shoreline as a part of the containment structure for the site, with in-water dikes or other containment structures required only for the remaining walls of the total enclosure. Nearshore CDFs discussed in this section have dikes with crest elevations above the mean high high water (MHHW).

Nearshore CDFs are most numerous in the Great Lakes region of the United States. Many of these sites were constructed adjacent to entrance channels or harbor channels. Large nearshore sites--CDFs initially constructed in water and which are now upland sites--are located in Puget Sound, the Great Lakes area, the Atlantic Coast, and California.

Processes and Design Considerations

There are several issues which must be carefully considered within the context of a CDF design:

a. Retaining dikes. The site conditions must allow for construction of structurally and geotechnically sound retaining dikes for long-term containment of dredged material solids and contaminants and excess water. If nearshore or island, the dike face will also be exposed to erosional forces due to currents and wave action, and some form of armor protection would normally be considered. Since the dikes must be constructed in water, marine construction techniques must be used, and these normally result in increased costs as compared to upland sites.

b. Transport and placement of material. Nearshore sites have waterfront access by definition. Material can be transported from dredging areas to a nearshore site by barge and directly offloaded to the site by mechanical rehandling or by hydraulic reslurry operations. Another technique used for Pier 91, the Port of Everett, and the Milwaukee Waterway site for material placed below MHHW is to leave a notch in the berm at el 1.5 to 3 m (5 to 10 ft) below MLLW. Dredged material is moved into the site contained in the barge and the barge hopper opened allowing the material to drop into the CDF. Once the elevation of the dredged material precludes movement of the barge into the facility, the dredged material must be unloaded by crane over the dike to a conveyor belt, or slurried in the barge and pumped into the facility. As the dredged material and cap rise above MHHW, low ground pressure earthmoving equipment may be used to spread the additional capping material by conventional earthmoving techniques. Placement by direct pipeline from hydraulic dredges is feasible if the site is located near dredging areas. Placement in upland sites can be accomplished by direct pipeline, crane or conveyor, or even hauled by trucks.

c. Site geometry and sizing. The site must be volumetrically large enough to meet both short-term storage capacity requirements during filling operations and long-term requirements for the anticipated life of the site. Sufficient surface area and dike height with freeboard must be available for retention of fine-grained material that may be resuspended during filling or storm events to maintain effluent water quality.

d. Contaminant pathway controls. Provisions for control of contaminant release through any of several pathways must be considered in the site design. These may include cutoff walls for groundwater moving from upgradient toward the site and, for nearshore sites, provisions to minimize biological uptake of contaminants if a notch in the berm is open.

e. Dewatering and long-term management. Upland sites should be managed to allow for passive or active dewatering of fine-grained material. Active dewatering normally involves creating drainage trenches for removal of surface precipitation water to allow for efficient drying. Removal of dewatered material to another disposal site such as an upland landfill or removal of separated sand fractions, if clean, off site for beneficial use may also be possible.

Both nearshore and upland sites can be managed for dewatering of material above MHHW. Dewatering of material in the saturated zone is limited by consolidation processes. If material is mechanically offloaded from the barge to the CDF, additional water is reduced compared to hydraulic offloading.

Each of these considerations must be appropriately addressed by the project design. More detailed discussion of these processes and design considerations is given in the following paragraphs.

Containment Dikes

General. Containment dikes are retaining structures used to form confined disposal facilities. They consist primarily of earth-fill embankments. The principal objective of a dike is to retain solid particles and pond water within the disposal area while at the same time allowing the release of clarified effluent or runoff to natural waters. The location or alignment of a containment dike will usually be established by site constraints. The heights and geometric configurations of containment dikes are generally dictated by containment capacity requirements, availability of construction materials, site restrictions, and prevailing foundation conditions.

The predominant retaining structure in a containment facility extends around the outer perimeter of the containment area and is referred to as the main dike. Except as otherwise noted, all discussion in this section applies to the main dike. Cross and spur dikes can also be constructed to divide the site or increase site effectiveness.

The engineering design of a dike includes selection of location, height, cross section, material, and construction method. The selection of a design and construction method are dependent on project constraints, foundation conditions, material availability, and availability of construction equipment. The final choice will be a selection among feasible alternatives.

The development of an investigation for the dike foundation and for proposed borrow areas, the selection of a foundation preparation method, and the design of the embankment cross section require specialized knowledge in soil mechanics. Therefore, all designs and specifications should be prepared under the direct supervision and guidance of a geotechnical engineer. Proposed cross section designs should be analyzed for stability, since the cross section is affected by foundation and/or embankment shear strength, settlement caused by compression of the foundation and/or the embankment, and external erosion. The extent to which the site investigation(s) and design studies are carried out is dependent, in part, on the desired margin of safety against failure. This decision will usually be made by the local design agency and is affected by a number of site-specific factors.

Containment dikes for nearshore sites must consider site-specific geotechnical conditions, wave effects, and maintenance requirements. Most Puget Sound in-water dikes have used sand and gravel as fill material. Soft foundation material along the center line of the berm may require excavation prior to placement of the fill to provide a suitable base for the berm. Rock fill dikes are more commonly found in the Great Lakes. Structures such as sheet-pile walls or cellular cofferdams have also been used for nearshore CDFs.

For CDFs situated in the water, the retaining dikes require protection from erosion due to waves. The erosion protection is generally an armor layer(s) made of rock; the size and extent (and cost) are a function of the severity of the wave climate. Depending on the size of the waves, the armor layer can have more than one layer of rock, progressing from small rock or gravel on an inner layer to the largest rock on the outer layer.

Engineering design of the CDF armor layer requires at a minimum defining the water depth where the CDF will be located, determining the wave climate and selecting a design wave, determining water levels, and deciding if wave run-up and overtopping need to be considered. From this information, the stable rock size, number of rock layers, and extent of the armor layer both above and below the waterline can be determined. The depth of water in which the CDF is located can also have a major impact on the CDF erosion protection design. As water depths increase, costs often increase due to the increased potential for larger waves.

In designing the armor layer for an in-water CDF, the most important information required is the wave climate. Based on the wave climate, a design wave is generally selected. The design wave is often the most severe wave expected in a return period ranging from 50 to 100 years. A risk-based approach balancing expected damages against initial costs is often used to determine the optimum design. Other factors relating to water levels and waves also need to be considered in design of the CDF erosion protection. Knowledge of the potential changes in water level, primarily resulting from tides and wind setup, is required. If the CDF is adjacent to shipping lanes, waves generated from a passing vessel may be a concern. The combination of waves and water levels determines runup which influences how high up the dike the erosion protection should extend. Depending on the height of the dike, waves can reach over the top of the dike.

CDF armor layer design should be conducted by an experienced coastal engineer, assisted by geotechnical engineers. The design of the armor layer should be integrated with the CDF dike design.

Seismic design of containment dikes. Special considerations for design of dikes in seismically active areas are warranted. The Calcasieu River and Pass, as outlined in ER-1110-2-1806, is located in seismic zone 0. Since this is the lowest risk zone a seismic analysis is not warranted for this project

Transport and placement

The method selected for transfer of dredged material from dredging areas to an upland CDF is dependent on the dredging technology used in the excavation of the sediments. Direct placement of material by pipeline dredge is economical only where the site is located near the dredging areas. For most Calcaseiu River projects, a pipeline hydraulic dredge is used with the disposal areas close by the waterway.

The method selected for transfer of dredged material from dredging areas to a nearshore CDF is dependent on the dredging technology used in the excavation of the sediments. Nearshore CDFs may be filled by mechanically rehandling dredged material from barges as described earlier. Material placed in the CDF in this manner is near its in situ water content. If such sites are constructed in water, the effluent volume may be limited to the water displaced by the dredged material, and the settling behavior of the material is not an important factor. Direct placement of material by pipeline dredge is economical if the site is located near the dredging areas.

If barges are used for transport, the sediment may be transferred from the barge to the CDF by several methods depending on the distance of the CDF from point of closest access by the barge. Unloading methods include the following:

a. Clamshell the dredged material from the barge directly into the CDF using a chute or other conveyance to transfer the dredged material beyond the interior toe of the dike.

b. Transfer material from the barge directly into the CDF by using a dragline bucket.

c. Unload the material from a flat-deck barge using a front-end loader and transport over the berm and into the CDF.

d. Clamshell to a conveyor belt transferring the dredged material over the dike to the CDF.

e. Provide a notch in the berm to allow a barge loaded with sediment to be moved into the CDF interior where the barge is emptied through a split-hull or bottom-dump barge. Opened or enclosed flat-deck barges may also be used where barge draft is critical due to shallow water. Material from the flat deck is unloaded with a front-end loader or bucket. (Hartman Associates 1996)

f. Slurry the material in the barge by adding water and mixing and pump the slurry through a pipeline to the CDF.

Initial storage capacity and solids retention

Design for initial storage capacity for material mechanically (not hydraulically) dredged and offloaded into the CDF is generally not critical. The dredged material will gain additional water and volume, estimated at less than 20 percent, during the dredging process. Once the material is placed in the CDF, it will consolidate over the course of the life of the facility to its original volume or less due to the added thickness of the fill compared to its in situ locale. Therefore, designing the CDF volume for the original (mechanically dredged and filled) sediment volume is conservative. However, if the CDF is filled hydraulically, the dredged material will initially increase to several times its original volume, depending on sediment characteristics and dredging technique. This volume increase becomes potentially important for design of a long-term storage site only as the CDF approaches design capacity and storage depth, hence volume, is limited.

A CDF must be designed and operated to provide adequate initial storage volume and surface area to hold the dredged material solids during an active filling operation. A hydraulically filled site must be designed and operated to retain suspended solids such that clarified water is discharged. The required initial storage capacity, ponded water depth, and surface area are governed by settling processes which occur in a CDF during placement of fine-grained dredged material. Tests of the sediments to be dredged are required to define their behavior in a dredged material containment area. The tests provide numerical values for design criteria that can be projected to the size and design of the containment area. Procedures for computer-assisted plotting and reduction of settling column data are available. Procedures to evaluate the required surface area and volume during active filling operations, to estimate effluent suspended solids concentrations, and to design other features for CDFs are described in EM 1110-2-5027 (HQDOA 1987).

Nearshore CDFs would be filled with water from the Calcasieu River during the initial stages of filling. The volume of water potentially discharged as effluent will be equal to or less than the volume of materials placed in the site. This water may be released through a permeable dike during ebb tides. For mechanically placed material, suspended solids will increase inside the CDF during filling. Solids suspended in salt water often flocculate and settle in a matter of hours. Those solids that do not settle will be filtered as excess water is released through the dikes. Surface area for solids retention during mechanical filling is not a critical criterion for a nearshore sites. Once the fill breaks the water surface, effluent will be formed by surface runoff. However, since the surface sediment is clean, the storm water should be relatively clean. If the CDF were filled hydraulically, surface area for solids retention would be more important, but not likely a controlling factor for a long-term site. Considerations for retention of suspended solids during hydraulic filling operations above MHHW are similar to that for an upland site (HQDOA 1987). A notched-dike option would increase the potential for release of suspended solids during mechanical or hydraulic filling. This release could be reduced by managing the offloading or barge dumping operations and/or by using turbidity barriers.

CDF Operation and Management

Placement of weirs and inflow points

Outflow weirs are usually placed on the site perimeter at the point of lowest elevation. The material offloading areas or the dredge pipe inlet is usually located as far away as practicable from these outflow weirs. However, these objectives may sometimes be conflicting. If the disposal area is large or if it has irregular foundation topography, considerable difficulty may be encountered in properly distributing the material throughout the area and obtaining the surface elevation gradients necessary for implementation of a surface trenching program. One alternative is to use interior or cross dikes to subdivide the area and thus change the large area into several smaller areas. Effective operation may require that the inlet/offloading location be moved periodically from one part of the site to another, to ensure a proper filling sequence and obtain proper surface elevation gradients. Also, shifting inflow from one point of the site to another and changing outflow weir location may facilitate obtaining a proper suspended solids concentration in disposal site effluent or rainfall runoff.

Installation and operation of multiple outflow weirs

In conjunction with provisions for moving the inflow point over the disposal site, it may also be worthwhile to contemplate installation of more outflow weirs than would be strictly required by design methods. Availability of more outflow points allows greater flexibility in site operation and subsequent drainage for dewatering, as well as greater freedom in movement of dredge inflow points while still maintaining the flow distances required to obtain satisfactory suspended solids concentrations in disposal site effluent. Also, a higher degree of flexibility in both disposal site inflow and outflow control will allow operation of the area in such a manner that desired surface topography can be produced, facilitating future surface trenching operations.

Interior dike construction

The basic rationale behind the construction of interior disposal area dikes is to subdivide the area into more manageable segments and/or to control the flow of dredged material through the disposal area. Control of material placement is normally to facilitate future disposal site operations, such as dewatering, or to provide proper control of disposal area effluent. Interior dikes may also be used as a haul road and access for movement of material for dike construction or other uses. Interior dikes may also serve to divide the site into cells with different levels of containment such as liners or slurry walls.

As a general rule, the use of interior cross dikes in any disposal area will increase the initial cost of construction and may result in increased operating costs. However, facilitation of disposal site operations, particularly future dewatering, may result in a general reduction in unit disposal cost over the life of the site. The benefit derived from dikes should be evaluated against the amount of disposal volume required for their construction. If the dikes can be constructed from dredged material or material available in the disposal site foundation and subsequently raised with dewatered dredged material, the net decrease in storage capacity will be approximately zero.

Surface water management

The management of surface water during the disposal operation can be accomplished by controlling the elevation of the outlet weir(s) throughout the disposal operation. A mechanically filled CDF will generate a minimum volume of excess water compared to a hydraulically filled site. This water can normally be contained within the site during filling. After active filling is completed, free water, not already removed by evaporation, may be drained from the site through the adjustable weirs.

At the beginning of a hydraulic disposal operation, the outlet weir is set at a predetermined elevation to ensure that the ponded water will be deep enough for settling as the containment area is being filled. As the disposal operation begins, slurry is pumped into the area; no effluent is released until the water level reaches the weir crest elevation. Effluent is then released from the area at about the same rate as slurry is pumped into the area. Thereafter, the ponding depth decreases as the thickness of the dredged material deposit increases. After completion of the disposal operation and the activities requiring ponded water, the water is removed as quickly as effluent water quality standards will allow.

Post dredging management activities

Periodic site inspections and continuous site management following the dredging operation are desirable. Once the dredging operation has been completed and the ponded water has been decanted, site management efforts should be concentrated on maximizing the containment storage capacity gained from continued drying and consolidation of dredged material and foundation soils. To ensure that precipitation does not pond water, the weir crest elevation must be kept at levels allowing efficient release of runoff water. This will require periodic lowering of the weir crest elevation as the dredged material surface settles.

Removal of ponded water will expose the dredged material surface to evaporation and promote the formation of a dried surface crust. Some erosion of the newly exposed dredged material may be inevitable during storm events; however, erosion will be minimized once the dried crust begins to form within the containment area.

Natural processes often need man-made assistance to effectively dewater dredged material, since dewatering is greatly influenced by climate and is relatively slow. When natural dewatering is not acceptable for one reason or another, then additional dewatering techniques should be considered. These techniques include trenching, vertical strip drains, and subsurface drainage to enhance drainage of water from saturated material beneath the crust.

Removal of coarse-grained material for productive off-site use by employing Disposal Area Reuse Management (DARM) techniques will further add to capacity. Dewatered fine-grained material may also be used for dike maintenance or raising. This concept has been successfully used by CE Districts and demonstrated in field studies. Guidelines for determining potential benefits through DARM are found in Technical Report DS-78-12 (USAEWES 1978). Additional information on productive uses of dredged material is found in EM 1110-2-5025 (HQDOA 1987).

Dewatering and Long-Term Storage

Factors affecting long-term storage capacity

Long-term storage capacity should be considered for an upland CDF intended for long-term use (Palermo 1992). Consolidation and desiccation are long-term processes which will affect the long-term storage capacity.

The coarse-grained fraction of dredged material (sands and coarser material) undergoes sedimentation quickly and will occupy essentially the same volume as occupied prior to dredging. However, the fine-grained fractions of the material (silts and clays) require longer settling times, initially occupy considerably more volume than prior to dredging, and will undergo a considerable degree of long-term volume change due to consolidation if hydraulically placed. Such materials are essentially under-consolidated soils, and the consolidation takes place due to self-weight loading.

Dredged material placement also imposes a loading on the containment area foundation, and additional settlement may result from consolidation of compressible foundation soils. Settlement resulting from consolidation is therefore a major factor in the estimation of long-term storage capacity. Since the consolidation process for fine-grained materials is slow, total settlement may not have taken place before the containment area is required for additional placement of dredged material. Settlement of the containing dikes may also significantly affect the available storage capacity and should be carefully considered.

Once a given active filling operation ends, any ponded surface water required for settling should be decanted, exposing the dredged material surface to desiccation (evaporative drying). This process can further add to long-term storage capacity and is a time-dependent and climate-dependent process. However, active dewatering operations such as surface trenching enhance the natural dewatering process.

Desiccation of dredged material is basically removal of water by evaporation and transpiration. Plant transpiration can also enhance dewatering but is not considered in this chapter. Evaporation potential is controlled by such variables as radiation heating from the sun, convective heating from the earth, air temperature, ground temperature, relative humidity, and wind speed. However, other factors affect actual evaporative drying rates. For instance, the evaporation efficiency is normally not a constant but some function of depth to which the layer has been desiccated and also is dependent on the amount of water available for evaporation.

Methods are readily available to predict the capacity gains possible through consolidation and desiccation. The data required to estimate long-term storage capacity include physical properties of the sediments and foundation soils such as specific gravity, grain size distributions, Atterberg liquid and plastic limits, and water contents; the consolidation properties of the fine-grained dredged material and foundation soils (relationships of void ratio and permeability versus effective stress); CDF site characteristics such as surface area, ultimate dike height, groundwater table elevations, average pan evaporation rates, average rainfall; and dredging data such as volumes to be dredged, rate of filling, and frequency of dredging (HQDOA 1987 and Stark 1991).

Dredged material dewatering operations

If the CDF is well-managed following active filling, the excess water will be drained from the surface and natural evaporation will act to dewater the material. However, active dewatering operations should be considered to speed up the dewatering process and achieve the maximum possible volume reduction, considering the site-specific conditions and operational constraints.

Dewatering results in several benefits. Shrinkage and additional consolidation of the material resulting from dewatering operations leads to creation of more volume in the CDF for additional dredged material. The drying process changes the dredged material into a more stable soil form amenable to removal. Dewatered material remaining in the CDF forms a more stable fast land with predictable geotechnical properties. Also, the drainage associated with dewatering helps control mosquito breeding.

A number of dewatering techniques for fine-grained dredged material have been studied (Haliburton 1978; Haliburton et al. 1991). However, surface trenching and use of underdrains were found to be the only technically feasible and economically justifiable dewatering techniques (Haliburton 1978). Techniques such as vacuum filtration or belt filter presses can be technically effective but are not economical for dewatering large volumes of fine-grained material.

The concept of surface trenching to dewater fine-grained dredged material was first applied by the Dutch (d'Angremond et al. 1978), and later field-verified under conditions typical of CDFs in the United States (Palermo 1977). Surface trenching has since become a commonly used management approach for dewatering in CDFs (Poindexter 1988, Poindexter-Rollings 1989).

Construction of trenches around the inside perimeter of confined disposal sites using draglines as shown in Figure 10 is a procedure that has been used for many years to dewater and/or reclaim fine-grained dredged material. In many instances, the purpose of dewatering has been to obtain convenient borrow material, if not contaminated, to raise perimeter dikes. Draglines and backhoes are adaptable to certain perimeter trenching activities because of their relatively long boom length and/or method of operation and control. The perimeter trenching scheme should be planned carefully so as not to interfere with operations necessary for later dewatering or other management activities. The low-ground-pressure chassis may be tracked or rubber-tired, as shown in Figures 11 and 12.

A suggested scheme for perimeter and interior trenching using a combination of draglines and a rotary trencher or other suitable equipment and incorporating both radial and parallel trenches is shown in Figure 13.

Figure 10. Small dragline operation for perimeter trenching

Figure 11. Rubber-tired rotary trencher

Figure 12. Track-mounted rotary trencher

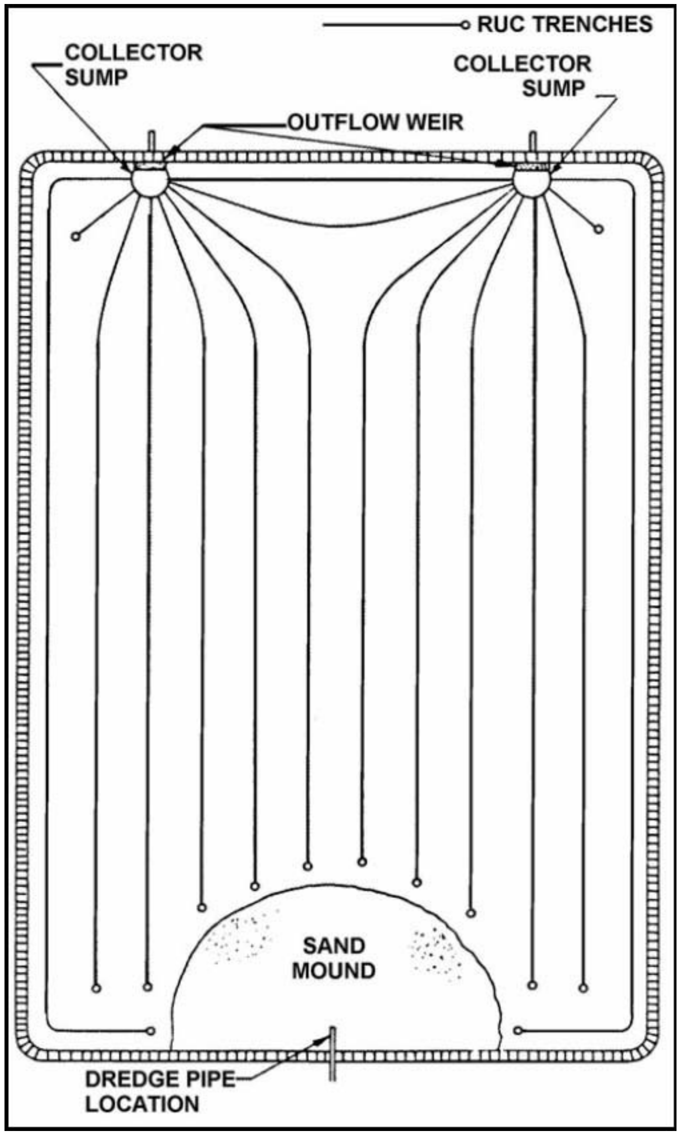

Figure 13. Combination radial-parallel trenching scheme (Riverine utility craft (RUC))

Design and Performance Standards for CDF Facilities

This section provides narrative and, where appropriate, numerical design and performance standards for the CDF option. The standards are based on available technical guidance in the literature, as well as available design information from projects nationwide and within Calcasieu River and Pass. Although the design would not strictly be considered as a functional design, the proposed standards in this section are technically compatible with design standards. The standards for various aspects of the design are stated in the following paragraphs, and all the proposed standards are summarized in Table 4.

Overall objectives for CDF option

An overall design objective for CDFs is to provide sufficient diked volumetric capacity to accommodate the required volume of dredged material and contain the dredged material such that water quality standards will be met for the effluent from the containment areas.

Engineering design procedures

The site design will be completed by competent professional engineers and standard USACE engineering design documents will be applied as appropriate in the design (a listing of USACE Engineer Manuals and other design documents is given in the "Index of Publications," EP-25-1-1 (HQDOA 1995b)).

Environmental evaluations and contaminant pathway controls

Environmental evaluations conducted as a part of the design will be in accordance with the Technical Framework (USACE/EPA 1992) and other applicable state and local guidelines.

Dike height

The constructed dike height for an upland CDF will be determined considering capacity requirements, surface area requirements, ponding requirements for any hydraulic filling, and the anticipated end use of the site, once filled.

Open dike configuration for bottom dump operation

For a nearshore CDF, during any phase of operation with a dike opening, the nearshore CDF must be operated to control direct exposure of offsite organisms to the contaminated sediments such that toxicity or unacceptable levels of bioaccumulation do not occur.

Table 4 Design and Performance Criteria for Upland and Nearshore CDFs

Item or Category	**Performance Criterion**
Overall design objective	Sufficient capacity provided.
Site characteristics	Equipment access to the disposal site must be available.
Engineering design	Design completed by competent professional engineers; standard USACE engineering design documents will be applied as appropriate.
Environmental evaluations	Sites evaluated in accordance with the USACE/EPA Technical Framework (1992).
Dike height	Height determined by considering capacity requirements, anticipated end use of the site once filled; and for nearshores sites, surface area requirements, initial water depth, tidal fluctuations, wave height and storm surge for a design storm event.
Dike stability	Dike of the embankment type must meet a safety factor of 1.0 to 1.5 against slope failure, depending on the loading condition; design must meet a safety factor of 1.5 against liquefaction of the embankment and/or foundation. Dike of the coffercell type must meet a safety factor of 1.1 to 1.5 against sliding and 1.0 to 3.0 against bearing failure, depending on the specific loading condition. Bed material and the depth of material layers must be investigated; confine sediment and avoid dike failure; accepted geotechnical and earthwork engineering methods used; structural strength and erosion protection will be incorporated in the design.
Erosion protection	Armor designed to resist changes in the armor stone profile or displacement of armor units under the design storm.
Excavated material	Dredged, transported, and placed in accordance with the proper disposal or use criteria of the materials excavated.
Dredging and Placement/offloading	Solids retention to be accommodated and spillage and leakage minimized
Outlet (weir) structures	Sized to pass 25-year rainfall event plus flow rate for offloading.
Solids retention	Site operated to ensure that applicable water quality standards for total suspended solids (TSS) or turbidity are met at the boundary of the designated mixing zone; adequate ponded surface area and water depth will be maintained during hydraulic filling operations and subsequent consolidation and dewatering of sediments; effluent quality release for hydraulic filling less than 100 mg/L using EM 1110-2-5027 (HQDOA 1987) procedures; discharge managed to provide maximum hydraulic efficiency for settling.
Effluent quality	Dissolved contaminant concentrations will not exceed applicable acute water quality standards, and effluent will not exceed applicable water column toxicity criteria at the boundary of the designated mixing zone.
Surface runoff	Site operated to contain runoff from 25-year storm event for controlled release with quality same as for effluent.
Dewatering	Volumetric disposal capacity maximized to the greatest extent feasible.
Site operations/ sequencing	As scheduling will allow, initially, materials placed with lower potential for adverse impact ; all contaminated sediments placed below the mean tide water elevation; contaminated sediments placed to remain anaerobic and wet for the long term.

Structural and geotechnical engineers

Where coffercell or concrete structures are to be used, the structural engineer selects a design configuration which is stable against sliding and overturning. The geotechnical engineer then assesses that concept for foundation bearing capacity, static and dynamic stability, including the foundation, and for settlement.

Retaining dike stability

Upland retaining dikes require all pertinent aspects of formal geotechnical design. Depending on the design alternatives, sliding, overturning, as well as slope instability must be thoroughly investigated. Soft foundation conditions can lead to shear failures in the foundation as well as erosion at the toe of the structure. Other loadings of concern may be earthquake loads, or even the active and passive soil pressures in the confined material itself pushing outward on the dike, especially if the structure is filled to near crest elevations.

The stability of the structure must be addressed from both static and dynamic points of view. Guidance for design practice and required factors of safety which are probably most pertinent to upland retention structures is found in the following USACE Engineer Manuals:

EM 1110-2-2502, “Retaining and Flood Walls” (HQDOA 1989).

EM 1110-2-1902, “Stability of Earth and Rock-Fill Dams” (Revision of this EM is nearing completion under the new and more general title, “Shear Strength and Slope Stability”) (HQDOA in preparation)

EM 1110-1-1904, “Settlement Analysis” (HQDOA 1990)

Engineering Regulation ER 1110-2-1806, “Earthquake Design and Evaluation for Civil Works Projects” (HQDOA 1995a)

Static stability

EM 1110-2-1902 (HQDOA 1970a) provides minimum factors of safety for all cases where the structure is an earthen embankment and other types of structures where stability checks must be made against failure through the foundation soils. In accordance with these criteria, the design of a dike of the embankment type will meet a safety factor of 1.0 to 1.5 against slope failure, depending on the loading conditio

Erosion protection

Dike features associated with resistance to wave erosion will be a major influence on the selection of the type of dike (cellular structure vs rubble mound, etc.) and the overall design. Design of the dike for erosion protection will follow available guidance in the Shore Protection Manual (1984) and Bruun (1989).

The selection of design wave height (discussed previously), wave period, and storm surge has a significant effect on the final structural design. A key factor in the structural design of armored retaining dikes (concrete or stone) is the hydraulic stability of the primary armor layer, a function of wave height.

Specific safety factors are not normally used in armor stability design. The question of allowable movement and safety factor with respect to armor design must be based on an acceptable degree of damage for the design storm event. For purposes of preliminary design, the Hudson equation is normally used to determine stone size. A model study is recommended for the final design. The armor will be designed to resist significant changes in the armor stone profile or displacement of armor units under the design storm.

Weir structures

Weir structures will be required to allow discharge of the excess carrier water as effluent during active filling through dredged material addition, storm water or storm surge over the dike. The flow rate of effluent discharge will be determined by the rate of filling. Weir structures will be sized to pass a design discharge consisting of rainfall runoff for a 25-year rainfall event plus flow rate for the largest offloading pump. If multiple weir locations are selected for purposes of site management, each of the structures should be sized to pass the design effluent discharge flow rate. The CDF should also be designed to accommodate an emergency draw down of ponded water if required.

Suspended solids retention

A CDF must be designed and operated to provide adequate initial storage volume and surface area to retain suspended solids such that clarified water is released or discharged. A mechanically filled site will have minimal flow over the weir because of the small rate of volume increase after the dredged material placement. Suspended solids that remain in the water column will be filtered by flow through the dikes.

For operations involving placement by barge using a notch in the dike, the retention of solids is dependent on the degree of water column dispersion during placement. Use of clamshell dredges and placement by bottom-dump barges should minimize dispersion. The dispersion can be controlled by the scheduling and frequency of placement, limits on the size of barges used, and the use of a silt curtain or other barrier across the dike notch. The barge placement operation will be controlled such that applicable water quality standards for TSS or turbidity are met at the boundary of the designated mixing zone.

For hydraulic filling, the required ponded surface area and ponded water depth is governed by the settling behavior of the fine-grained sediments placed in the site. Adequate ponded surface area and water depth will be maintained during hydraulic filling operations to ensure that applicable water quality standards for TSS or turbidity are met at the boundary of the designated mixing zone. Procedures for sizing the required ponded area and depth described in EM 1110-2-5027 (HQDOA 1987) should be followed.

Management for Dewatering and Long-Term Storage

The design and operation of the CDF should allow for efficient use and increases in the volumetric capacity available for disposal. Increase in storage capacity results from decreases in the height of dredged fill deposited over the long-term due to consolidation and drying or desiccation. The site will be operated and managed to maximize volumetric disposal capacity and dewater and density fine-grained material to the greatest extent feasible. Site operation and management for dewatering includes decanting the ponded water from the site whenever possible to expose the surface to drying. The most economical dewatering approach is to construct trenches around the site periphery and within the interior of the site to promote efficient surface drainage of precipitation and increase the effective rate of drying.

DISPOSAL SITE MANAGEMENT

The management of existing disposal sites encompasses a number of alternative concepts. The primary one of interest for the Calcaseiu River and Pass is the reconfiguration or reconditioning of existing disposal areas to provide for better use of BMPs with and without diked disposal. Furthermore, these techniques should also be considered for use in any new CDFs.

Confined Disposal with BMPs

Multiple Disposal Cells. One concept that may prove beneficial is to construct several disposal cells within the existing disposal area footprint using minimum height (< 5 feet) dikes of rock and geotextile fabric. New Orleans District uses this design for lake fills and reconstruction of marsh sites. The Lake Palourde Fill Site near Morgan City, LA is an example of this practice. Figure 14 is a schematic plan view of the Lake Palourde Fill Site showing the three cells (A, B, C).

Initially, a 24-inch pipeline dredge, was mobilized to create a lake fill at Lake Palourde, using material from Morgan City Harbor with the assistance of a booster pump and an additional 19,000 feet of pipeline. Two earthen-rock dikes were constructed at Lake Palourde Fill Site with approximately 15,000 square yards of geotextile fabric to cover the rock. Figure 15 is a photograph showing the geotextile fabric covered dike. Each subdivision (cell) was approximately 19 acres and was separated by an interior berm, nearly 4 feet high with a gap of about 4 feet located approximately 1/3 the length of the dike from the roadside. The purpose of the gap was to allow fish to escape and to let the fines flow to the next cell to settle. The site was filled over a two-year period. Approximately, one million cubic yards (550,000 +480,000) of a mixture of sand (20%-40%) and silt (80%-60%) were dredged and placed at the Lake Palourde Fill Site during 2001 and 2002.

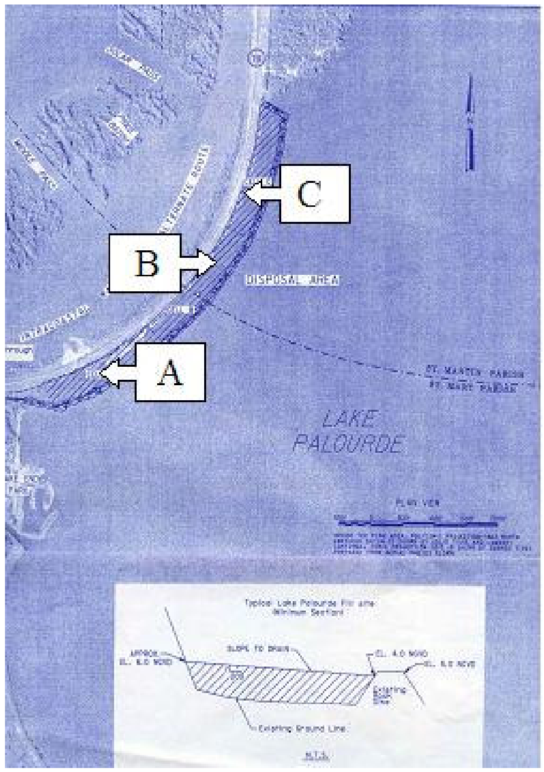

Figure 14. Schematic Plan View of Lake Palourde Fill Site Silt Curtains.

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Figure 15. Lake Palourde Fill Site Geotextile Fabric Covered Rock Dike

Figure 16 is a photograph showing this double curtain deployment. The two containment curtains are visible encircling the rock dike outfall and the third small curtain is shown along side the shoreline for bank erosion protection. Figure 17 show the quality of effluent being discharged from the site.

Figure 16. Silt Curtains Deployed at Lake Palourde Fill Site

Figure 17. Effluent Discharge Water, Lake Palourde Fill Site

Site Expansion. Another concept that could be considered is to expand the site footprint (size of the disposal area) to incorporate wetlands as effluent polishing (natural treatment) systems. Training features could be constructed to direct the flow of effluent from the CDF through the marsh to further remove residual TSS. This practice would allow the effluent to actually nourish the marsh and to provide a natural polishing of the final discharge water back to the waterway.

The practice of effluent discharge through a salt marsh with return to the waterway is a standard practice in the Mobile District. Figure 18 shows the outfall pipes and salt marsh at the South Blakeley Disposal Area, AL.

Figure 18. South Blakeley Outfall Pipes and Salt Marsh

Site Controls. Several features could be incorporated into an expanded site to include the construction of temporary or permanent structures to control the direction flow and ponding of water, retention of solids and regulation of effluent discharges (water releases) from the expanded site.

a. Training Features. Runoff effluent training features could be constructed of a combination of hay bales and silt fences, earthen knee dikes, rock-fabric dikes, and even geotextile containers for the purpose of containing dredged material. The latter will be described in detail.

Geotubes have been used as temporary structures to contain dredged material before marsh or wetland can be established to stabilize the new site (Fowler, Sprague and Toups 1995, Olin, Fischenich, Palermo, and Hayes 2000). The geotextile fabric is usually sewn together longitudinally with the ends sewn shut to form a geotube that can be filled hydraulically. A schematic of how they are constructed is given in Figure 19. The filling operation is shown in Figure 20.

An example of the use of geotubes to construct a salt marsh at Bayport, Texas in the Galveston District is shown in Figures 21-22. The successful establishment of the salt marsh was dependent on the use of the geotube to retain the dredged material until the grasses could establish on the site. Grasses will eventually grow into the geotube and provide more stability for the salt marsh.

The geotube may be used to construct either permanent or temporary structures. If used as a permanent structure, as the slurry dewaters the size of the tube will shrink in size. Very soft, fine-grain material may be pumped into the fabric container in

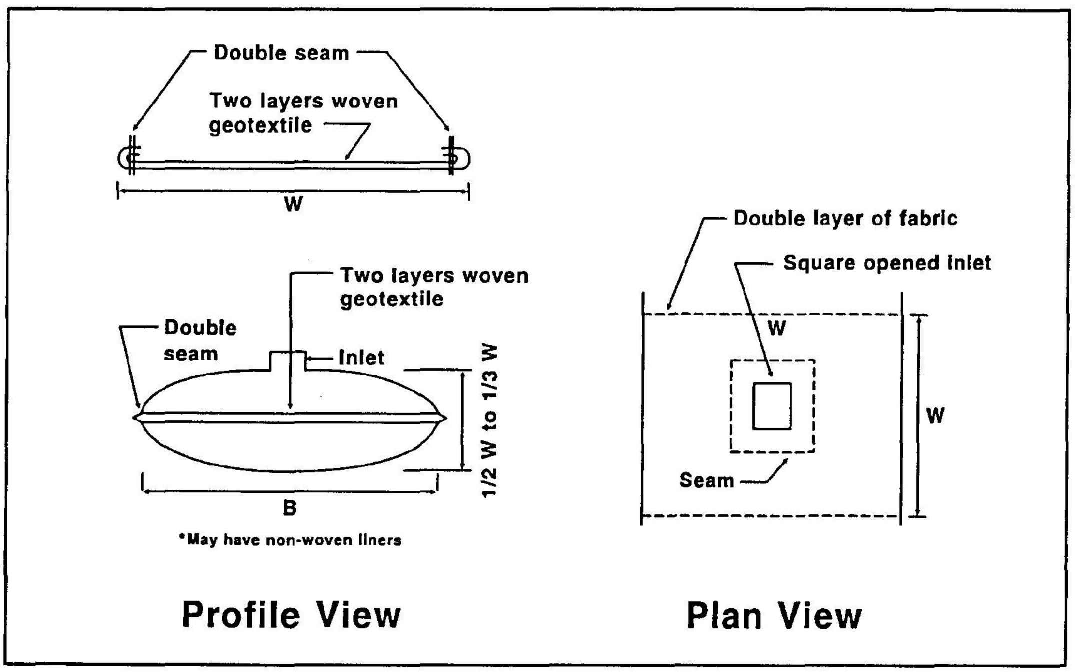

Figure 19. Schematic of Geotube Construction

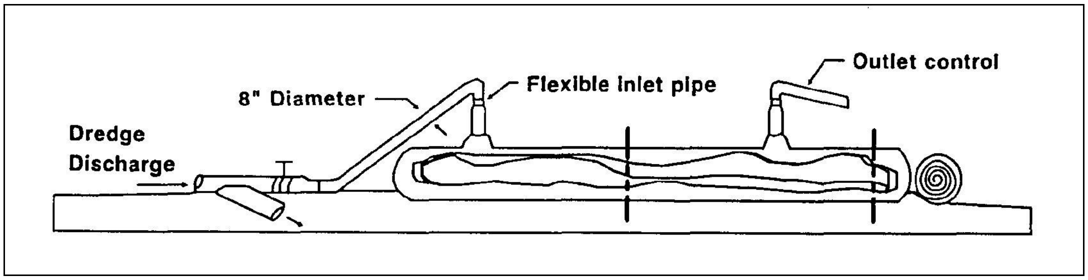

Figure 20. Schematic of Filling a Geotube

multiple lifts to create a geotube barrier or dike of reasonable width to height ratio. The shape of the geotube after filling with low-density material depends on the density of the slurry fill, the density of the surroundings, the circumference of the tube, and the stiffness of the bag material. Usually it will resemble a modified ellipse with a major portion of the bottom being flat, and the height of 30 to 40% of the resulting width. Since most geotextiles are constructed of material that is sensitive to UV-light, permanent barriers of geotextile fabric will need protection. Advances have been made recently with materials that will tolerate UV-light better. There are procedures that can be used to select geotextile materials for dewatering fine-grain slurries. A computer program, “Geosynthetics Applications Program” (GAP) was developed by Palmerton (1998) to design geotubes. The program assumes that the geotube is filled with a slurry material that does not have any shear strength. The ultimate strength of the geotube is directly dependent upon the available tensile strength of the seams of the geotube.

Figure 21. Geotube/Salt Marsh, Bayport, TX

Figure 22. Salt Marsh Behind Geotube

b. Effluent Weirs. Control of effluent discharges is normally done with outfall weir structures. Conventional weirs are engineered structures of various sizes, shapes, and lengths as dictated by the requirements of the disposal site. The two most critical weir design parameters are ponding depth and weir length. In order to maintain acceptable effluent quality, the upper layers containing low levels of suspended solids should be ponded at depths greater than or equal to the minimum depth of the withdrawal zone, which will prevent scouring settled material. The withdrawal zone is the area through which fluid is removed for discharge over the weir as shown in Figure 23. Efficient sedimentation is promoted by ponding water to a specified depth in the disposal site. This ponded depth is controlled by the elevation of the weir crest. In conventional operations, weir boards are raised to obtain the required ponding depth and lowered periodically as the dredging rate decreases to control the effluent quality and to discharge water during dewatering. Figure 24 contains a photograph of a conventional weir with boards discharging clear effluent.

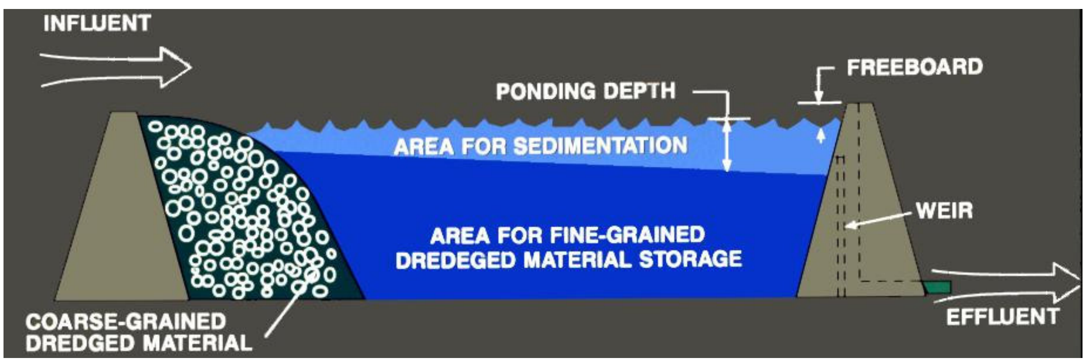

Figure 23. Schematic Cross-section of a Confined Disposal Area

Figure 24. Conventional Weir with Boards

During the past decade, a new outlet water control device has been invented by the Corps Norfolk District. This invention was named the "telescoping weir." The weir itself consists of a set of nested cylinders set on end with their axis vertical and one cylinder within the other. Figure 25 shows a schematic diagram of the telescoping weir. The bottom cylinder is fixed to a steel frame foundation that is anchored to a concrete pad the bottom of the placement site and connected to a discharge pipe. The telescoping weir is set within and attached to the base of a reaction frame that provides support for it and the machinery that controls the telescoping movements of the weir. The telescoping weir is raised and lowered by mechanical screw jacks that operate simultaneously either manually or by a solar/battery-powered motor.

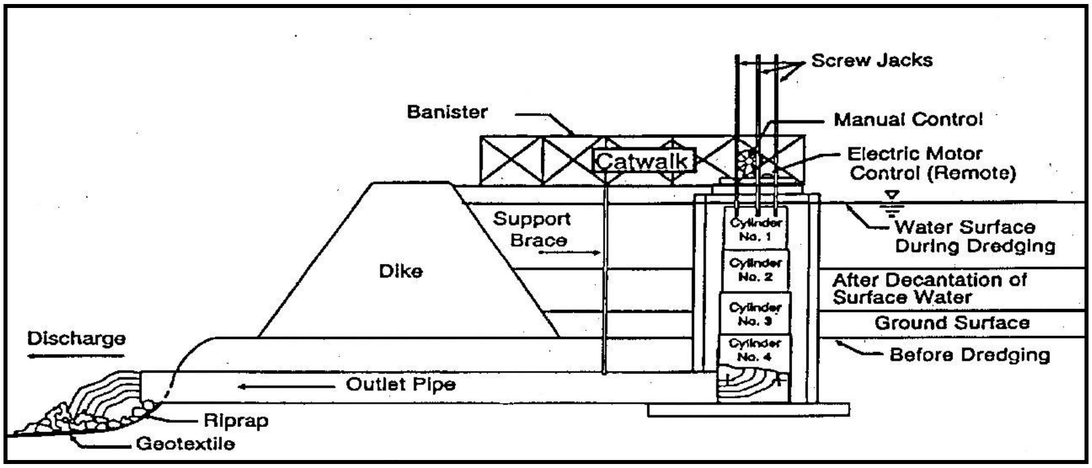

Figure 25. Schematic Diagram of the Telescoping Weir

The upper cylinders are extended in a telescoping manner to position the rim of the top cylinder to any desired elevation below or above the water surface. As the cylinders are lowered below the water surface the decant water flows over the weir crest into the interior sections and exits through the discharge pipe in the lower section and returns to the nearby waterway. Figure 26 is a photograph of the clear effluent water flowing over telescoping weir crest.

Figure 26. Effluent Flowing Over Telescoping Weir

In addition to Craney Island CDF, telescoping weirs have been installed in the North Blakeley confined disposal site in the Mobile District and at the

Baltimore District's Popular Island environmental restoration project in Maryland.

Design specifications for conventional weirs are available in Engineer Manual 1110-2-5027 (USACE 1987) and for telescoping weirs in Francingues, McNair, Vann, and Woodward (2001). An adjustable water level control device, such as the telescoping weir, should be considered as part of the expansion of an existing disposal site or construction of a new CDF.

c. Effluent Treatment. Control of solids in effluent discharges has been demonstrated with chemical addition, and guidelines on how to evaluate dredged material for chemical treatment have been developed by the Corps (USACE 1987). In-pipeline chemical addition has been tried to concentrate solids and reduce resuspension. Normally, chemical addition is applicable to fine-grain sediments that settle poorly at freshwater dredging sites. Chemical polymers have shown promise for success at freshwater sites. One example was the Yazoo River, Mississippi dredging project. Poor settling could be overcome with chemical addition and design of a two-cell site with the chemicals added through a mixing structure between the cells. If effluent TSS is a concern, chemical addition could be incorporated as a BMP if proven effective from both a technical and cost perspective.

3 Beneficial Use of Dredged Material

Beneficial uses of dredged material should always be a priority in developing a dredged material management strategy. Beneficial use includes a wide variety of options which utilize the material for some productive purpose. Some broad categories of beneficial uses have been identified: habitat restoration or enhancement (wetland, upland, island, and aquatic); beach nourishment; aquaculture; parks and recreation; agriculture, forestry, and horticulture; strip mine reclamation and landfill cover; shoreline stabilization and erosion control (fills, artificial reefs, and submerged berms); construction and industrial use (port development, airports, urban, and residential); material transfer (fill, dikes, levees, parking lots, and roads); and multiple purpose. A few potential beneficial use options for the Calcasieu dredged material are presented below. Detailed guidelines for beneficial use applications are given in EM 1110-2-5026 (USACE 1987b).

Marsh Creation

Marshes are considered to be any community of grasses or herbs that experience periodic or permanent inundation. Typically, these are intertidal freshwater or saltwater marshes and periodically inundated freshwater marshes. Marshes are recognized as extremely valuable natural systems and are accorded importance in food and detrital production, fish and wildlife cover,nutrient cycling, erosion control, floodwater retention, ground-water recharge, and aesthetics. Marsh values are highly site-specific and must be examined in terms of such variables as species composition, location, and extent, which in turn influence their impact upon a given ecosystem.

Marsh Development Considerations. Accurate techniques have been developed to estimate costs and to design, construct, and maintain man-made marsh systems. Methods are available to predict the impact of the alternatives on the environment and to describe the value of the proposed resource prior to its selection.

a. Advantages. Several advantages have been found in marsh development as a disposal alternative:

(1) Considerable public appeal.
(2) Creation of desirable biological communities.
(3) Considerable potential for enhancement or mitigation.
(4) Frequently a low-cost option.
(5) Useful for erosion control.

Marsh development is a disposal alternative that can generate strong public appeal and has the potential of gaining wide acceptance when some other techniques cannot. The created habitat has biological values that are readily identified and accepted by many in the academic, governmental, and private sectors. However, application requires an understanding of local needs and perceptions and the effective limits of the value of these ecosystems. The potential of this alternative to replace or improve marsh habitats lost through dredged material disposal or other activities is frequently overlooked.

Marsh development techniques are sufficiently advanced to design and construct productive systems with a high degree of confidence, even in moderate wave energy environments. For example, salt marshes have been established at Bolivar Peninsula, Texas, and Gaillard Island, Alabama, behind temporary breakwaters in moderate energy areas. These habitats can often be developed with very little increase in cost above normal project operation, a fact attested to by hundreds of marshes that have been inadvertently established on dredged material and by the more than 130 marshes that have been purposely created using dredged material substrates in U. S. waterways.

b. Disadvantages. Several problems are likely to be encountered in marsh development:

(1) Unavailability of appropriate sites.
(2) Loss of other habitats.
(3) Release of contaminants.
(4) Loss of site for subsequent disposal.

By far the most difficult aspect of the application of marsh development is the location of suitable sites. Low energy, shallow-water sites are most attractive; however, cost factors will become significant if long transport distances are necessary to reach low energy sites. Temporary protective structures may be required if low energy sites cannot be located and have been successful at several Gulf coast sites where moderate wave energy occurs. Marsh development frequently means the replacement of one desirable habitat with another, and this will likely be the source of most opposition to this alternative. There are few reliable methods for comparing the various losses and gains associated with this habitat conversion; consequently, determining the relative impact may best be made on the basis of the professional opinion of local authorities. Although studies have shown that contaminant uptake from soil in marsh environments is minimal, the planner should remain alert that the potential exists with highly contaminated sediment use. Development of a marsh at a given site can prevent the subsequent use of that area as a disposal site. In many instances, additional development on that site would be prevented by state and Federal resource agencies. Exceptions may occur in areas of severe erosion or subsidence, or where previous disposal created a low marsh and subsequent disposal would create a higher marsh.

c. Maintenance. Dredged material marshes should be designed to be relatively maintenance free. The degree of maintenance will largely depend on the energy conditions at the site, a factor that should be included in the cost analysis of the project. No maintenance may be required to protect the new marsh in low energy situations. In areas of somewhat higher energy conditions, protection may be required only until the marsh has a chance to mature. In those areas, protective structures may be designed for a relatively short life with no additional maintenance required. In high energy situations, perpetuation of the marsh may require planned periodic maintenance of protective structures and possibly periodic replanting. (EPA/USACE, 1987).

More information on the use of marshes and marsh creation for the disposal of dredged material can be obtained in EM 1110-2-5026, Beneficial Uses of Dredged Material. Also, the Engineer Manual for Beneficial Uses of Dredged Material and the Engineer Manual for Dredging and Dredged Material

Disposal (EM 1110-2-5025) deals with a wide array of uses for dredged material instead of placing the material into disposal facilities.

Mining Dredged Material from CDFs

Port Creation

The economic potential and social productivity of industrial/commercial activities provide a strong incentive for urban growth and development. These activities have flourished in natural harbors and along urban waterways where raw materials can be received and finished products shipped most economically. Industrial/commercial development near waterways has been aided by the availability of hydraulic fill material from nearby dredging activities. The use of dredged material to expand or enhance port-related facilities has generally received local support because of the readily apparent potential benefits to the local economy. Approval of the disposal operation is generally predicated on the advancement of the port development project and not on the incidental need for proper disposal of the dredged sediments. Traditionally, where disposal has been to advance the industrial development goal, attempts were made to use the dredged material beneficially; where it would not, the material was disposed of by the most economical means available. The key for the beneficial use planner is to identify how, when, and where dredged material from a navigation project can fulfill an economic need, while not overlooking biological beneficial uses and environmental considerations and limitations. Identification of economic or social benefits may help overcome some environmental opposition to disposal sites. Job-producing planned uses in cities with depressed employment are much more likely to gain approval than projects that appear to conflict with basic community needs.

There are numerous examples of dredged material sites that were used in harbor/port development. One such facility constructed on dredged material is the Presidents Island-Memphis Harbor Project located approximately 5 miles southwest of Memphis, Tennessee. It is a 960-acre site on the southeast side of the island (now a peninsula) filled with sandy dredged material. A slack-water area was created by diking, and an 800-foot-wide by 12-foot-deep channel was dredged and the sediments placed along 3.5 miles of the channel's north bank. Filling was completed in 1957, and within 20 years most industrial development was completed. By 1973 over 70 separate industrial concerns had bought or leased acreage on the site. A feasibility study of proposed harbor expansion alternatives prepared by the Memphis District recommended that a second harbor channel be dredged at Presidents Island and the material placed on the island along the new channel's south bank. This proposal would create an additional 1,000 acres above the floodplain for port and related industrial/commercial facilities. When the first facility was completed, there was little concern for the wetlands that were covered up. Expansion plans must take these wetlands into careful consideration.

In dozens of locations in U.S. rivers, dredged material is used for such benefits and for creating foundation above the floodplain for grain elevators, shipping terminals of all types, barge-fleeting areas, and storage facilities for U.S. products waiting to be moved to market (coal, timber, agricultural products). Two examples at Portland, Oregon, a container facility and a grain elevator located at convenient shipping points, were both built on dredged material. Another example is the harbor at Vicksburg, Mississippi, on the lower Mississippi. A large industrial site providing facilities to over 50 industries was built on dredged material from the Yazoo River. Other examples include port and shipping facilities at Texas City, Galveston, and Houston, Texas, in Galveston Bay; port facilities in the Duwamish River in Seattle, Washington; and facilities at Blakely and Brookley Island complexes in upper Mobile Bay, Alabama.

4 Other Disposal Alternatives

Ocean Disposal

Open-water disposal is the placement of dredged material in rivers, lakes, estuaries, or oceans via pipeline, direct mechnical placement, or release from hopper dredges or barges. The main consideration for conventional use of an open-water site is whether a given material to be dredged is acceptable for open-water disposal from the standpoint of contamination. Water-column contaminant impacts must be considered from the standpoint of water quality (chemical) and toxicity (biological). Benthic impacts must be considered from the standpoint of toxicity and bioaccumulation. A tiered approach to open-water contaminant testing and assessments is described in detail in the dredged material testing manuals for MPRSA and CWA (USEPA/USACE 1991; USEPA/USACE, 1998).

Three sites have been identified and used for the open water disposal of dredged material from the Calacsieu River and Pass. These sites are identified as Calcasieu River Bar Channel 1, Calcasieu River Bar Channel 2, and Calcasieu River Bar Channel 3. Bar Channel is located approximately 1.5 miles from the shoreline and encompasses an area of 1.76 square miles. The water depth ranges from 7 feet to 26 feet at the site. The Bar Channel 1 site was last used in 1997 for the disposal of dredged material from the Calcasieu River and Pass Bar Channel.

Bar Channel 2 is located close to the shoreline and is close to Bar Channel 1. Bar Channel 2 encompasses an area of 3.5 square miles and depths at the site range from 7 feet to 36 feet. This site has been used often since the late 90's and was lasted used in 2003 for dredging that took place at the bar channel

Bar Channel is located 12.8 miles from the shoreline and encompasses an area of 5.88 square miles. The water depth ranges from 36 feet to 46 feet at the site. This site has been used as often as the other two sites and was last used in 1981 for disposal of dredged material.

Landfill Disposal

Though not typically cost effective, if no other feasible alternatives exist, dredged material may be placed in a landfill. A factor to consider when placing dredged material in a landfill is that any material placed in a landfill must pass a paint filter test, (no liquids can be placed in landfills). Dredged material would have to be dewatered before the placement of the material. Since landfills require a daily cover to be placed over the material, dredged material can be used for this cover provided it has been tested and is approved for cover material. Many sites across the country have used dredged material as a daily cover.

5 Conclusions and Recommendations

A long term management plan is necessary to sustain dredging activities along the Calacasieu River and Pass. Based on the Phase I study by ERDC, existing CDFs are not sufficient to contain dredged material for even one dredging cycle. The Phase II portion of the study provided general information on potential alternatives to reduce or contain the dredged material predicted over a twenty year period. Minimizing dredged material volumes through erosion control and advanced dredging techniques is one logical alternative but would have to be implemented in conjunction with other alternatives. The alternative of constructing new or extended CDFs was analyzed in detail to provide estimates of the areas that would be required. The study showed that new CDFs would require areas of approximately 2595, 3175 and 1966 acres to account for long-term storage needs in the lower, middle and upper reaches, respectively. It has yet to be determined whether areas are available for new CDFs. Any new facilities as well as existing CDFs require proper management to optimize their use. An assessment of potential beneficial use options such as marsh creation or port building should be performed to determine how much material could be used, whether the dredged material meets the specifications (appropriate grain size, moisture content, and free of contaminants), the degree of processing/ handling required and associated cost. Other potential disposal alternatives that should be further investigated include use of open water disposal sites and even landfill disposal.

The data obtained from the sediment analysis and computer model evaluations indicate that the present dredging volumes being removed from the channel each year will require a large disposal area for the placement of the material. It will likely be difficult to obtain the tremendous area (12 square miles) required to accommodate the dredged material; therefore other alternatives for disposal of the material need to be evaluated. It is likely that a combination of alternatives will be required to handle the anticipated volumes. Raising the dikes on the present disposal areas is one alternative that is currently being evaluated by the New Orleans District. These raised dikes will need to be “stepped in” on the site in order to construct the correct slope of the dike to ensure the stability of the dike against failure. These disposal sites need to be properly managed in order to obtain the maximum dewatering of the area and the consolidation of the dredged material. Reworking the disposal areas to promote consolidation and dewatering may also help to alleviate some of the disposal area shortage. This report presented an array of options for the disposal of dredged material that need to be evaluated in conjunction with reworking the existing areas. While many of these options may not be feasible, each one should be evaluated in the DMMP phase in order to assess their practical use.

It is recommended that several alternatives for long term dredged material management be implemented concurrently to fully maximize existing capacity and minimize cost and dredging delays.

It appears the most effective long term solution would be to minimize the material dredged annually by reducing the amount of sediment entering the channel. At this point, the sediment budget analysis has been inconclusive regarding the entire source of material entering the channel. However, the study has identified that sediment washing from the banks of the CDFs does contribute to the total amount of material that is dredged. Therefore, bank stabilization using rock dikes could have a significant impact on reducing dredged material, even though this option would also have significant cost. Implementation would likely have to occur intermittently over time.

A detailed investigation of the sediment budget would better define the sediment source so that it could be controlled to prevent it from entering the channel. Hydrodynamic studies might be able to indicate where the transported sediment originates. Furthermore, it is suggested that one source of sediment in the navigation channel could be from the dredged sides of the channel sloughing off and encroaching upon the channel. Perhaps a geotechnical study of slope stability could determine the most efficient channel design to prevent this mode of failure. Also, more sampling of the in-situ material along the sides of the channel that is to be dredged needs to be collected to better narrow the void ratio of the sediments. This would allow for a more reliable number in calculating the amount of sediment derived from erosion. As it stands now, the samples collected are thought to have been collected along the centerline of the channel which is not indicative of the material that is dredged.

Maximize the capacity of existing CDFs. Although the existing CDFs are nearly full, their capacity could be increased somewhat by raising the berms which would require stepping in and reducing slope. As it appears the CDF banks have eroded significantly over time, some CDF capacity could be regained by extending existing CDF boundaries to the original footprints. Existing CDF could also be recovered to some extent by dewatering existing and newly placed material. An analysis of current practice should determine if CDFs could be more efficiently managed by reworking CDFs to prevent channeling/short-circuiting, relocating inlet points and weirs to maximize retention time using interior berms if necessary, and adding or removing weir boards to efficiently drain the ponded water while retaining solids.

In the meantime, further investigate local beneficial use options. Marsh nourishment is a likely candidate for the nearby wildlife refuge. The cost of transporting sediment for this and other options should be considered, along with the quantity of sediment that could be relocated in this manner. Consideration of dredged material as a resource rather than a waste product is a first step in locating a home for sediment outside the channel.

Despite all other efforts to minimize or relocate dredged material, it seems the existing CDFs will eventually reach their absolute maximum capacity and new disposal sites will be required. It is important to begin planning for new facilities before they are desperately needed. Potential site locations should be identified so that modeling can be performed in advance and easements can be procured.

Regarding the PSDDF model. The PSDDF consolidation modeling exercise was performed to provide an estimate of the area required to contain the dredged material resulting from 20 years of dredging. Numerous assumptions were made for the model because specific location and dimensions of a new CDF have not yet been proposed. Upon identifying a specific location upon which to construct a new CDF, the PSDDF model could be used to more accurately model consolidation and CDF capacity, given site specific information.

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Appendix A
Calcasieu River and Pass Dredged Material Sedimentation Study
May 2004
Phase 1 Final Report

US Army Corps of Engineers®
Engineer Research and Development Center

CALCASIEU RIVER AND PASS DREDGED MATERIAL SEDIMENTATION STUDY

May 2004
Final Report

by

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1 Introduction

Background

As part of the Army Corps of Engineers mission to maintain navigable waterways of the US, an issue that must be addressed is the proper handling and storage/disposal/reuse of dredged material. One option for the storage of dredged material is the use of a confined disposal facility (CDF). A CDF is a diked area where dredged material is placed, either by mechanical methods of dredging or by hydraulic dredging. The conceptual design of the CDF requires an evaluation of the properties and settling behavior of the dredged material to be placed therein. This evaluation will provide information necessary to estimate storage requirements needed for the placement of dredged material along the Calcasieu River and Pass located in Lake Charles. Louisiana

Purpose

The purpose of this report is to document and present the results of the laboratory tests performed to measure sedimentation properties of the dredged material from the Calcasieu River and Pass located at Lake Charles, LA. Also presented will be the correlation between turbidity and total suspended solids (TSS).

Objectives

The overall objective was to support the U.S. Army Corps of Engineers, New Orleans District in their mission to dredge the Calcasieu River and Pass and to provide storage of the resulting dredged material. To fulfill this objective, settling tests were run to determine the settling behavior of the Calcasieu River and Pass sediments when they are hydraulically dredged. This will aid the District in managing the CDFs to meet their requirements. Also in support of the overall objective, data was collected on the turbidity and TSS concentrations in the water column during the settling column tests. This facilitated the development of a correlation curve for turbidity and TSS that a contractor and/or inspector can use to quickly *estimate* TSS by *measuring* turbidity. Turbidity is a much more easily and quickly measured parameter than TSS because turbidity is measured with a commercially available meter, while TSS has to be measured in a laboratory using ovens, analytical balances, filtration apparatus, and etc. Also, capacities of current CDFs along the Calcasieu River and Pass from mile 5-36 were evaluated to determine the volume of the CDFs for the placement of the dredged material. The volume calculations were based on the safe dike elevation calculations made for the disposal areas.

2 Column Settling Test Procedures

Physical Characteristics

Historical MVN test data on soil samples previously retrieved from the bottom of the Calcasieu River channel were reviewed for the purpose of determining appropriate locations for additional sampling in support of the Scope of Work.

Previous sampling was conducted by several dredging contracts during the 1990's from the river mouth (approximate mile 0) north to the Lake Charles area (approximate river mile 36), including an ERDC study (Calcasieu River Sediment Removal Study TN-EL-94-9 by Roy Wade) and the 1961 New Orleans District Design Memorandum.

One major purpose for identifying additional sampling locations was to optimize the evaluation of the future post-dredged material. The material behavior will be determined by conducting column settling tests for each sampled material.

The general trend for material classification in the channel bottom (surficial deposits) from the mouth up to Lake Charles is observed as follows:

Bar channel: Silty to Highly Plastic Clay (generally fat clay, CH)
Mile 0 to Mile 6: Silty Clay to Low Plasticity Silt (generally silt, ML)
Mile 6 to Mile 9: Silty Clay (CL) to Low Plasticity Silt (generally silt, ML)
Mile 9 to Mile 11: Silty Clay (CL)
Mile 11 to Mile 13: Silty Clay (CL) to Highly Plastic Clay (generally fat clay,CH)
Mile 13 to Mile 22: generally silt, ML, with some sandy silt SM and silty clay CL
Mile 22 to Mile 30: generally fat clay, CH
Mile 30 to Mile 36: sands and clays

It was recommended that sediment with a high percentage of clay be sampled to represent worst-case settling behavior. As a very general observation, there are three areas along the river channel bottom which have the highest probability of containing fat clay (CH) sediments:

-Nearshore below the river mouth
-Mile 11 to 13
-Mile 22 to 30

Since the study area begins at river mile 4, obtaining nearshore sediments to model upland CDF sites was not considered necessary unless those sediments will be dredged and placed in future CDF sites above mile 4.

Surficial sediment sampling along the channel bottom was recommended to be conducted within river miles 11 to 13 and river miles 22 to 30. Sampling locations based on previous soil test results were suggested as follows:

Mile 11 to 13:

State Plane Coordinates (NAD 83)	GPS Coordinates
X= 2645340, Y= 525286	29 55 44.89312, 93 20 22.86931
X= 2646119, Y= 531242	29 56 43.97991, 93 20 15.20108
X= 2645722, Y= 531280	29 56 44.28727, 93 20 19.72067
X= 2646113, Y= 533248	29 57 03.83401, 93 20 15.66811
X= 2645908, Y= 533266	29 57 03.97667, 93 20 18.00171

Mile 22 to 30:

State Plane Coordinates (NAD 83)	GPS Coordinates
X= 2650740, Y= 583019	30 05 17.25525, 93 19 32.91513
X= 2650761, Y= 585003	30 05 36.89585, 93 19 33.06881
X= 2650972, Y= 585030	30 05 37.19940, 93 19 30.67250
X= 2651146, Y= 585048	30 05 37.40749, 93 19 28.69555
X= 2652590, Y= 585443	30 05 41.56518, 93 19 12.33755
X= 2650500, Y= 585775	30 05 44.49192, 93 19 36.19244
X= 2654095, Y= 586800	30 05 55.25432, 93 18 55.47425
X= 2653820, Y= 587067	30 05 57.84990, 93 18 58.65713
X= 2663271, Y= 622263	30 11 47.81335, 93 17 17.91267

It was recommended that four (4) of the above sites be selected as sampling locations either at or near the coordinates within 200 feet of the channel centerline. At each selected location enough sample material was collected to fill four (4) five-gallon buckets, plus four (4) five-gallon buckets of river water. The four buckets of sediment will be tested for material properties (water content, gradation and classification, organic ash content, specific gravity, and atterberg limits). The four buckets will then be homogeneously mixed and tested to determine the anticipated future post-dredging settling behavior.

The physical characteristics of the dredged material are important in the design of a CDF and starting the column settling tests. Four sediment samples were used to evaluate the physical characteristics of the lower (mi. 5-14) and middle (mi. 14-24) reaches of Calcasieu River and Pass sediment (Table 1). The remaining portion of this sample was used for the settling column tests. Eustis Engineering performed the settling column tests on the 4 samples with the ERDC Environmental Lab performing a duplicate settling column test on sample A. Prior column testing and physical analysis was performed on three sections of the upper reach (mi. 33-36, mi. 30-33 and mi. 23-30, respectively) as reported by Wade (1994). Descriptions of geotechnical and engineering testing are presented below. Based on the Unified Soil Classification System, the Calcasieu sediments were classified as a CH for all four samples tested.

Specific Gravity. Specific gravity (SG) of the particulates in the sediment was measured using the procedures given in the Laboratory Soils Testing Engineering Manual (USACE 1970). The specific gravities of the four Calcasieu River sediments were 2.76, 2.70, 2.675, and 2.69 for samples A, B, C, and D, respectively.

Table 1 Sediment Physical Characteristics				
Characteristic	**Sample A**	**Sample B**	**Sample C**	**Sample D**
Specific Gravity	2.76/2.73*	2.70/2.72*	2.74/2.675*	2.74/2.69*
In Situ Solids Concentration				
Water content	298**	169**	281**	244**
Void ratio	8.2	4.6		
Atterberg Limits				
Liquid limit	105	132	104	75
Plastic limit	30	29	29	24
Plasticity index	75	103	75	51
Grain-Size Distribution				
Percent gravel	0.0	0.0	0.0	0.0
Percent sand	0.6	6.3	8.3	13.7
Percent silt/clay	99.4	93.7	91.7	86.3
Classification	CH	CH	CH	CH

* Data from Eustis Engineering

** Water content was performed on samples from buckets

Water Content. The in situ water content (W) of fine-grained sediment samples is also an important parameter evaluating settling behavior and the volumetric changes occurring following dredging and disposal. It should be noted that the water content in this appendix is identical to the geotechnical engineering water content. Since the water content is defined as the ratio of weight of water to weight of solids expressed as percent, it can exceed 100 percent. The procedures are given in the Laboratory Soils Testing Engineering Manual (USACE 1970). Using the specific gravity and water content, the void ratio (e) and solids concentration (S) can be expressed as follows:

$$e = \frac{W * SG}{100}$$

$$S = \frac{1000 * SG}{1 + e}$$

Grain-size Distribution. Grain-size distributions were determined on the samples using standard sieve and hydrometer analyses as outlined in the Laboratory Soils Testing Engineer Manual (USACE 1970). The resulting gradation curves are shown in Figures 1-4. The samples ranged from 86.3 to 99.4% fines.

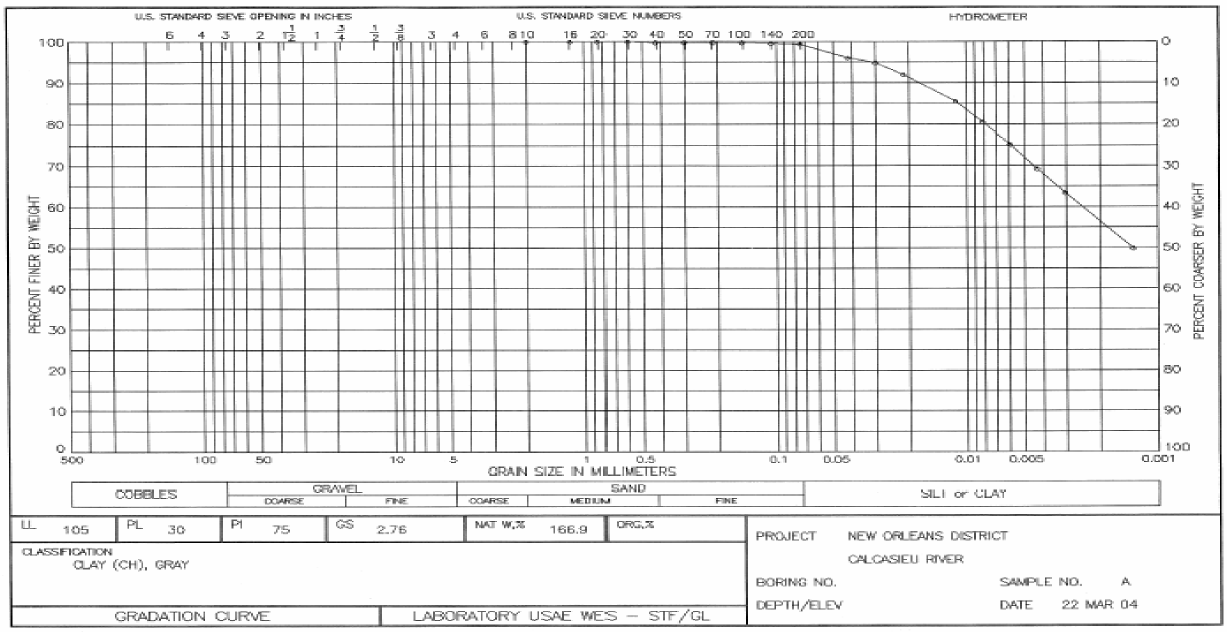

Figure 1. Gradation curve for Sample A of the Calcasieu River and Pass

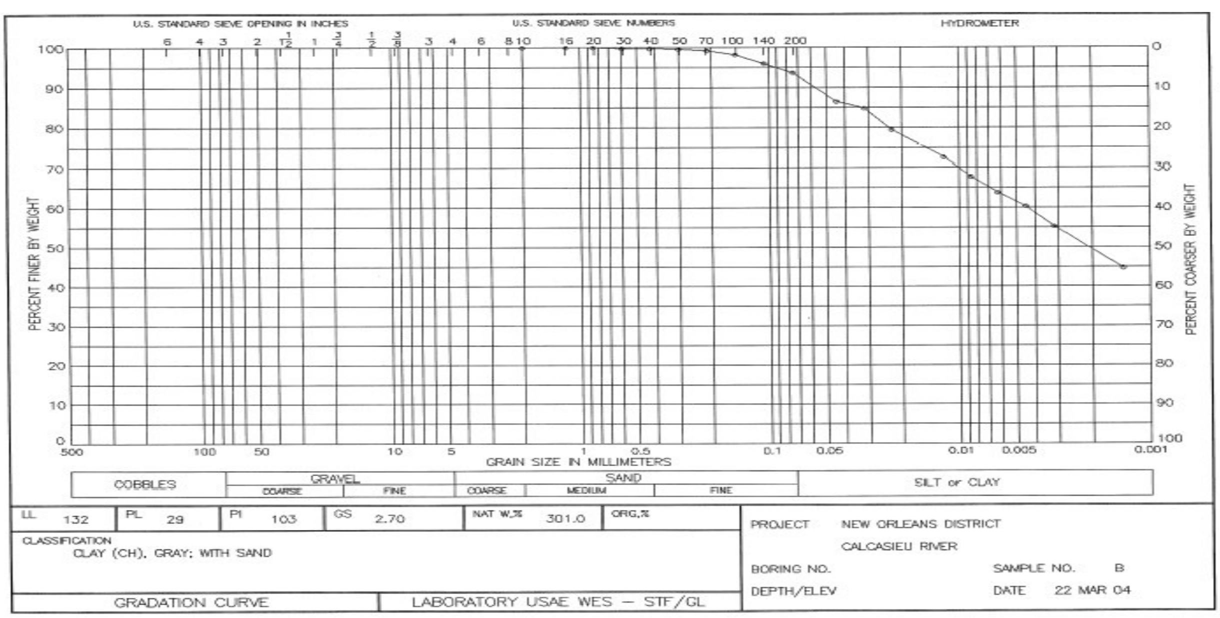

Figure 2. Gradation curve for Sample B of the Calcasieu River and Pass

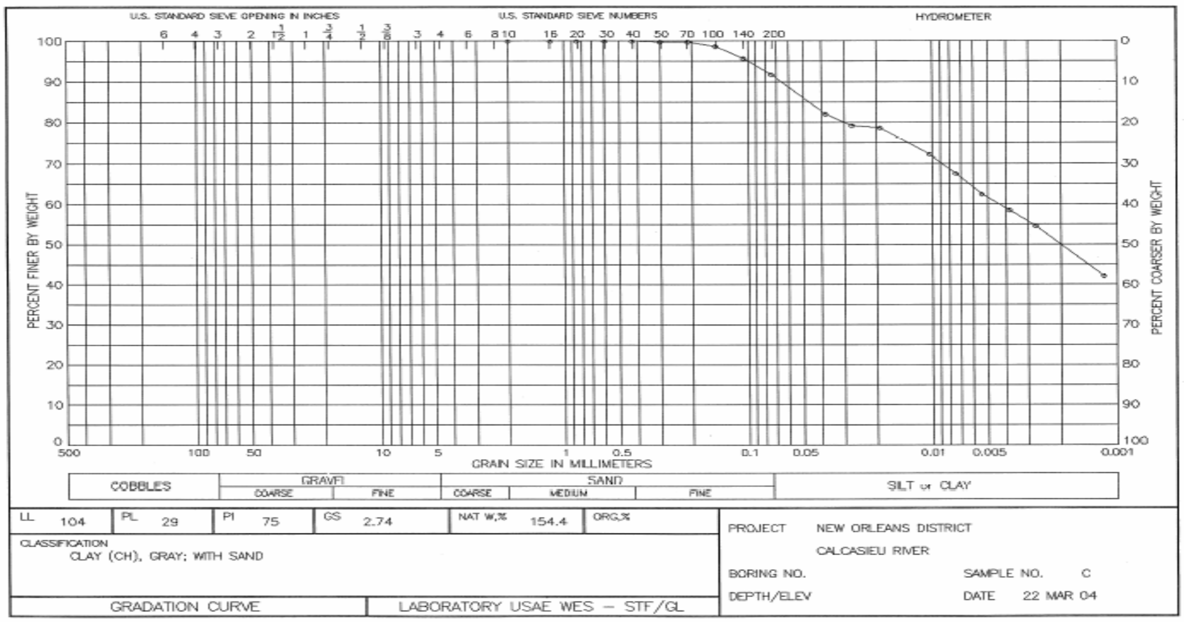

Figure 3. Gradation curve for Sample C of the Calcasieu River and Pass

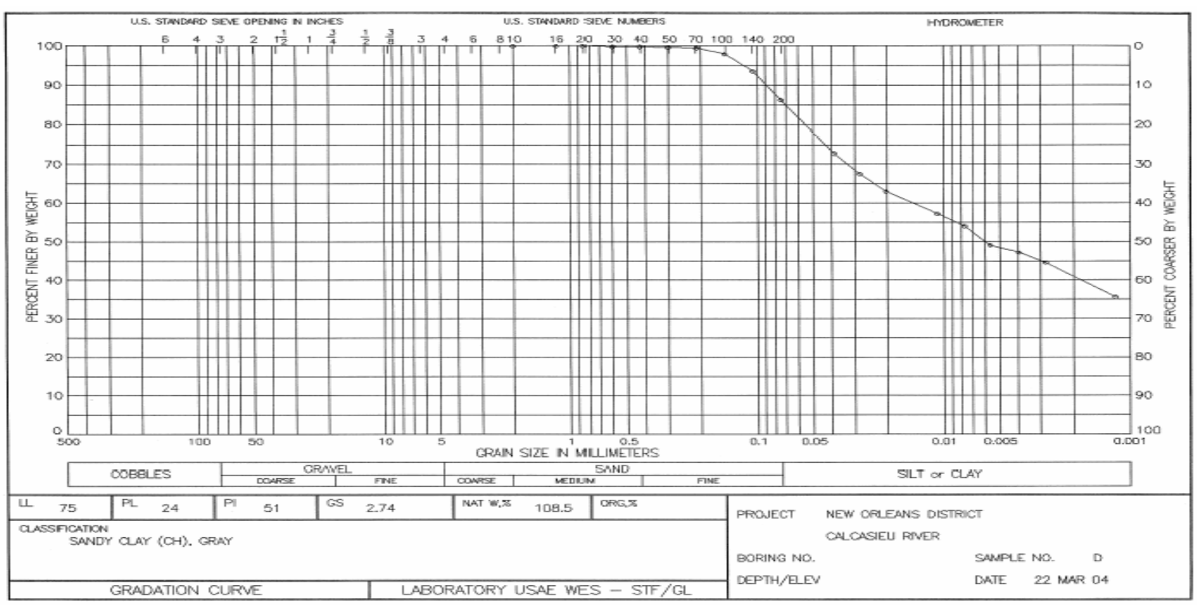

Figure 4. Gradation curve for Sample D of the Calcasieu River and Pass

Plasticity. The Liquid Limit (LL) and Plastic Limit (PL) were determined for composite sediment samples using standard soils testing procedures as outlined in the Laboratory Soils Testing Engineer Manual (USACE 1970). The plasticity index (PI) was then computed; PI = LL – PL.

Unified Soil Classification System (USCS) Classification. Visual classifications and classifications using results of the grain-size distribution and plasticity tests were determined using the USCS as outlined in the Laboratory Soils Testing Engineer Manual (USACE 1970).

Settling Column Test Experimental Procedures

The settling column test procedures described by Palermo, Montgomery, and Poindexter (1978), U.S. Army Corps of Engineers (USACE, 1987), and Palermo and Thackston (1988) provided the approach used to run the laboratory tests for determining the sedimentation properties of the Calcasieu River and Pass, samples A, B, C, and D, dredged material.

Settling tests

The column settling tests involved mixing sediment and site water to simulate the concentration of a dredged material slurry, placing the material in a settling column, and observing the different types of settling behavior. Conducting a single settling test for the composite samples collects all three types of settling data (zone, compression, and flocculent settling data). The general procedures are described below.

Laboratory Procedures

Slurry preparation

A target slurry concentration is used to simulate the solids concentration anticipated during production by a hydraulic dredge. Usually, target slurry concentrations selected for settling tests are dependent on the grain size distribution of the sample estimated by %fines plus 3 times the % coarse fraction. Solids concentrations were determined for the column settling tests by taking samples from the discharge pipe of a dredge performing work on the Calcasieu River just before the column settling tests were run. The average solids concentration measured from the dredge was 126 g/L.

After completely mixing the slurry, the mixing intensity was decreased to allow the majority of the coarse-grained material to settle in the mixing chamber while keeping the fine-grained material in suspension. While slowly mixing, the fine-grained slurry was transferred from the 130-liter mixing chamber to an 8-in. diameter, 6-ft tall column with ports at 0.5-ft intervals starting near the 6.0-ft height (Figure 5). Immediately after loading the column with the slurry, samples were extracted from the sampling ports at 1.0-ft intervals throughout the column. The average of the total solids samples collected from the column was used as the solid concentration for the column settling test. The total solids concentrations for the slurry (representing the fine-grained fraction of the original slurry) that was transferred into the columns are given in Table 2. The average total solids concentration was determined to be 127.65 g/L, 125.46 g/L, 127.98 g/l, and 113.12g/L for samples A, B, C, and D, respectively. The sample A tested by the Environmental Lab had an average suspended solids concentration of 135.4 g/L. A photo of the settling test of the Calcasieu sediments is shown in Figure 6.

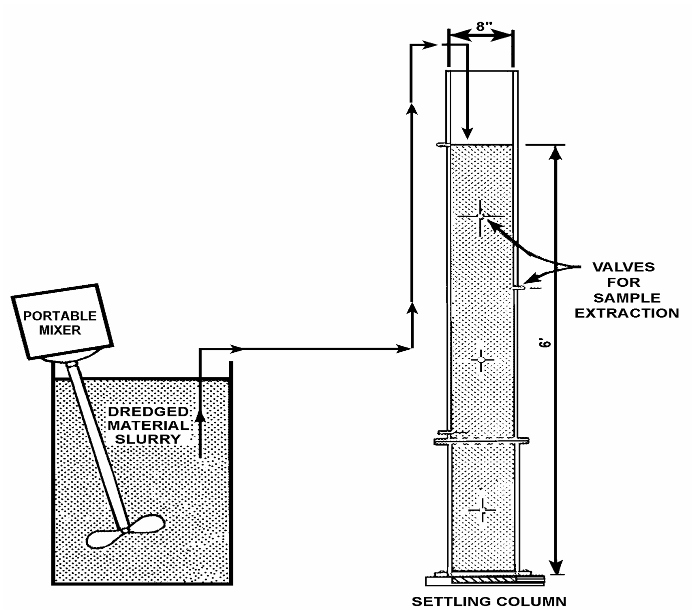

Figure 5. Schematic of settling column

Table 2. Total Solids Concentration of Column Slurry Sample

Port Height, (ft)	Sample A (g/L)	Sample B (g/L)	Sample C (g/L)	Sample D (g/L)
1.0	127.4/131.9*	125.9	126.2	122.2
2.0	126.4/136.5*	124.5	129.3	122.5
3.0	127.2/136.6*	128.4	129.5	113.3
4.0	128.1/136.8*	123.1	130.0	112.8
5.0	129.5/136.8*	124.0	127.0	109.2
6.0	127.3/133.6*	126.9	125.9	98.7
Average	127.65/135.4*	125.46	127.98	113.12

* Denotes samples collected by Environmental Laboratory

Figure 6. Calcasieu settling column test

Zone settling test

The zone settling test consists of recording the fall of the liquid-solids interface with time after placing the slurry in a sedimentation column. These data are plotted as height of the interface versus time. The slope of the curve in the constant velocity settling zone is the zone settling velocity, which is a function of the initial slurry concentration. The zone settling velocity is used in the design process to determine the minimum ponded area required for a given flow rate.

The zone settling test was performed concurrently with the compression settling test on the same slurry in the same column. Zone settling typically occurs during the first 12 hours of a dredged material settling test and compression settling occurs after the first 24 hours of the test. The height of the interface was read periodically during the first 12 hours with sufficient frequency to define the zone settling velocity. From the plot of the interface height (ft) versus time (hr), the zone settling velocity was determined.

Compression settling test

The compression settling test must be run to obtain data for estimating the volume required for initial storage of the dredged material. Following the zone-settling test (the first 12 hours immediately after the column was loaded with the slurry), the height of the interface was measured at approximately daily intervals for the next 15 days. The interface height, the initial height of the slurry, and the initial solids concentration of the slurry in the column are used to estimate the concentration of settled solids below the interface as a function of time as required in the compression settling analysis.

Flocculent settling test

The flocculent settling test consists of measuring the concentration of suspended solids above the liquid-solids interface at various depths and time intervals in a settling column. Normally, an interface forms near the top of the settling column during the first day of the test; therefore, sedimentation of the material below the interface is described as zone settling. The flocculent test procedure is performed only for that portion of the water column above the interface. Samples of the supernatant were extracted from each sampling port above the liquid-solid interface at different time intervals and the suspended solids concentrations were determined.

The flocculent settling test was performed concurrently with the zone and compression settling tests on the same slurry in the same column. Therefore, the initial slurry concentrations for the flocculent, zone, and compression settling tests were the same. Samples of the supernatant, if available, were extracted with a syringe at fixed ports located every 0,5 feet above the bottom of the column. Supernatant samples were collected at approximately 2, 4, 7, 12, 24, 48, 72, 96, 168, 264, and 360 hours after loading the slurry. Samples were taken at all ports above the supernatant-settled solids interface where supernatant was available. Suspended solids concentrations were then determined on the supernatant samples by Standard Method 2540D (APHA-AWWA-WPCF 1989). Turbidity of the supernatants were measured using a Hach Digital model 2100 turbidimeter and determined by Standard Method 2130B (APHA-AWWA-WPCF 1989). Substantial reductions of suspended solids are expected to occur during the early part of the test, but reductions should lessen at longer retention time (USACE 1987).

3 Data Analysis and Results for Column Settling Test

The behavior of the Calcasieu sediment at slurry concentrations equal to that expected for inflow to a CDF is governed by zone settling processes. The sediments exhibited a clear interface between settled material and clarified supernatant.

The settling test data were analyzed using the Automated Dredging and Disposal Alternative Management Systems (ADDAMS) (Schroeder and Palermo 1995) which is a family of computer programs developed at ERDC to assist in planning designing, and operating dredging and dredged material disposal projects. The SETTLE module of ADDAMS was used for the settling test data (Hayes and Schroeder 1992).

Data adjustment

Column settling tests were performed by Eustis on four sediment samples (A, B, C, D) from the Calcasieu River. A replicate of Sample A was also tested at ERDC. Upon examination of the Eustis data, it was discovered that the column settling tests were not performed exactly according to the column testing procedure guidance. At each sample interval, samples were taken from both above and below the sediment-water interface for Total Suspended Solids (TSS) and Total Solids (TS) analyses, respectively. The procedure guidance calls for only sampling below the interface for TS at the beginning of the test, and from that point on, sampling only above the interface for TSS.

Sampling below the interface throughout the test caused the measurements of the interface height over time to be lower than they should have been. Other effects could also have occurred, such as disturbance of the column, which may affect the settling rate, although there is no way to know these effects. The interface height measurements by Eustis include height reduction due to settling of the solids and sampling. To develop settling curves, the interface height as a function only of compression settling is needed. To account for the effect on the interface height, a series of calculations were performed to estimate what the interface height should have been in the absence of sampling. The calculations used to correct the interface height are based on the mass lost during each sampling event.

In theory, if sampling below the interface does not occur, the mass (M) of solids in the column remains constant. The mass (M) is equal to the solids concentration (C) times volume (V) below the solids interface, or since the column area is constant, we can simplify using the interface height (H) rather than volume; so M = CH. The following definitions will be used to develop the equations for estimating the theoretical interface height without sampling.

M_o -original mass
M_i -actual mass (after sampling) at time i
C_o -original solids concentration (average TS from initial TS sampling)
$\overline{C}_i$ -average solids concentration at time i, (average TS from TS sampling at time i)
C_i -solids concentration at time i (calculated based on mass at time i)
H_o -original height (slurry height at start of test, after initial TS sampling)
H_i -actual height at time i, (recorded interface height)
ΔH_i -height differential at time i due to sampling
H_{oi}' -theoretical original height if had started with mass M_i
H_i' -theoretical height at time I if had not sampled
M_o' -original mass if had started with the actual mass at time i, = M_i
M_i' -theoretical mass at time i, = M_o

The original mass of solids in the column can be calculated as

$$M_o = H_o C_o$$

Without sampling, mass is constant, $M_o = M_i$. However, since sampling occurred, a portion of the solids mass was removed at each sampling event, and $M_o \neq M_i$. M_i can be calculated as the original mass minus the cumulative mass lost:

$$M_i = H_o C_o - \sum_i^n \Delta H_i \overline{C}_i$$

Then, the theoretical original height, if had started with mass M_i, can be calculated as:

$$H_{oi}' = \frac{M_i}{C_o}$$

Then, mass at time i is equal to the theoretical original mass ($M_i = M_o'$), which is equal to the original solids concentration times theoretical original height:

$$M_i = H_i C_i = H_{oi}' C_o = M_o'$$

Or, solving for the concentration at time i,

$$C_i = \frac{H_{oi}' C_o}{H_i}$$

Then, the theoretical mass (had sampling not occurred) at time i, M_i' should equal the concentration at time i times the theoretical height at time i, and should equal the original mass ($M_o = M_i$'):

$$M_o = H_o C_o = H_i' C_i = M_i'$$

Rearranging, to solve for the theoretical height at time i:

$$H_i' = \frac{H_o C_o}{C_i}$$

This series of equations was used to adjust the data from Eustis to estimate the interface height had sampling below the interface not occurred. The computed values of H_i' from each column settling test were used to develop the compression settling curves.

Compression Settling Tests

For the compression tests, the initial slurry concentration and height, and height of the interface versus time were entered into SETTLE (Appendix A) for each of the 4 samples tested. The SETTLE program uses the initial slurry concentrations of 127.65 g/l, 135.4 (EL sample A) 125.46, 127.98, 113.12, and height of 6.85 ft, 6.24 ft (EL sample A), 6.46 ft, 6.61 ft, and 6.39 ft for samples A, B, C, and D, respectively, to determine the solids concentration at a given time. A plot was generated showing the relationship between solids concentration (g/L) and retention time (days) and is presented in Figure 7 for all the samples tested, including the results from prior testing of the three sections of the upper reach (Wade 1994). Appendix A shows the compression settling curves for each individual sample. SETTLE also generated a regression equation for the resulting power curve relating solids concentration to time. The composite sample regression equation may be used to determine the solid concentration at any given time. The regression coefficients are presented in Table 3. The regression equation used was:

$$C = aT^b$$

where:

C = settled solids concentration, g/L
T = time, days
a,b = regression coefficients

Zone Settling Tests

Zone settling velocity for the Calcasieu River sediment sample was determined to be 0.195 ft/hr, 0.175 ft/hr (EL sample A), 0.153 ft/hr, 0.172 ft/hr, and 0.131 ft/hr for samples A, B, C, and D, respectively, for the zone settling test. The height of the interface and their corresponding elapsed time from the start of the test when the height was measured were entered (Appendix B) and plotted in the SETTLE program to determine the zone settling velocity. Figure 8 presents the zone settling curves for all samples tested.

Appendix B presents the zone settling curves for each individual sample. When the zone settling curve departs from a linear relationship, compression settling begins. The transition from zone to compression settling occurred between 10 and 12 hours (Appendix B). The zone settling velocity is adjacent to the plot of the zone settling data.

Table 3. Compression Settling Regression Coefficients

Coefficient	Sample A	Sample A (EL)	Sample B	Sample C	Sample D
a	174	198	179	221	231
b	0.083	0.105	0.092	0.118	0.186

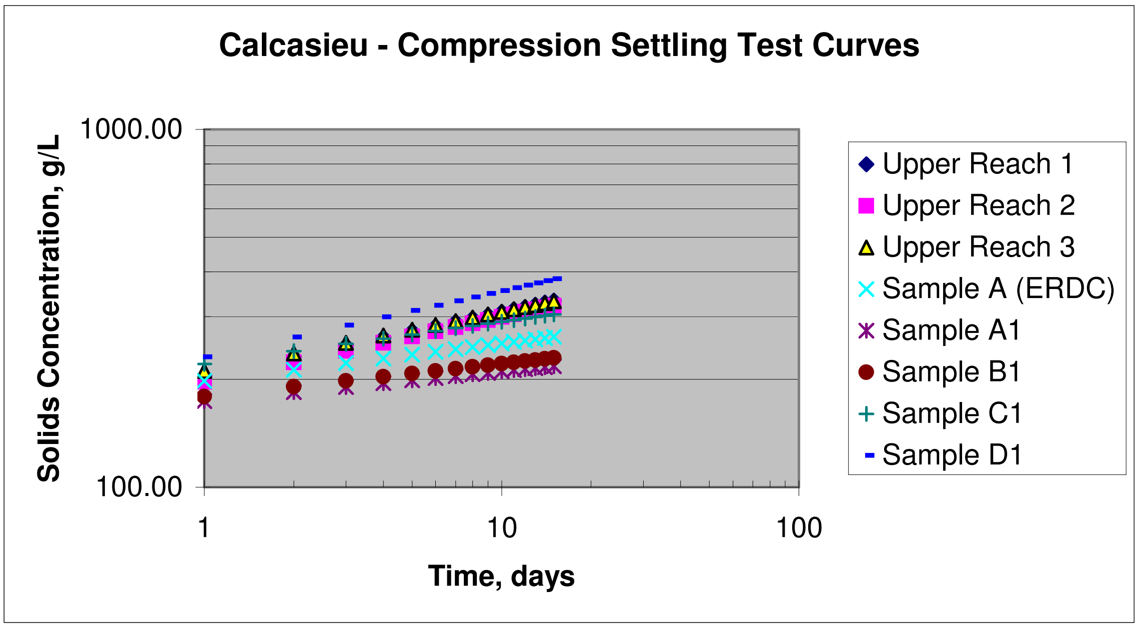

Figure 7. Compression settling curves for all samples.

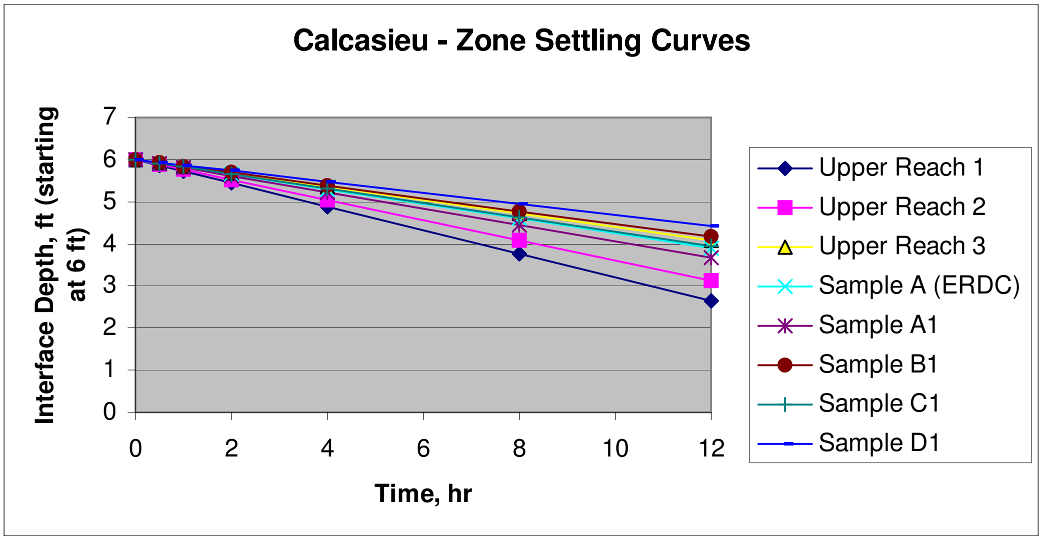

Figure 8. Zone settling curves for all samples, assuming initial slurry height of 6.0 feet

Flocculent Settling Tests

An extension of the flocculent settling test is presented in USACE (1987). Palermo (1985) analyzed the effects of several possible assumptions regarding the magnitude of the value to be used as the initial concentration in the laboratory test and showed that all gave essentially the same final result. Therefore, it was recommended that, for simplicity, the concentration in the first sample taken at the highest sampling port be used as the initial concentration. SETTLE generates two curves based on the settle data presented in Appendix C. The plot generated by SETTLE is the concentration profile curve (Appendix C). The concentration profile curve, which plots the depth below the surface (ft) versus percent of initial concentration, shows that the suspended solids concentrations decrease with time and increase at deeper ponding depths (1, 2, and 3 ft) at the weir. The actual depth of withdrawal is a function of the flow rate and the weir length; the depth is shallower for lower flow rates and longer weir lengths. The supernatant suspended solids curves derived from the concentration profile curves compare the effects of retention time on the supernatant suspended solids concentration at 1-, 2-, and 3-ft ponding depths. Figure 9 shows that increasing the retention time beyond 24 hr for 1, 2, or 3 ft of ponding depth provide little additional improvement in supernatant suspended solids concentration. Actual field suspended solids will be somewhat greater because of resuspension by wind and wave action. Based on field experience, a re-suspension factor is estimated to range from 1.5 to 2.5 depending on ponding depth and surface area (Shields, Schroeder, and Thackson 1987) (Table 4).

Table 4 Recommended Re-suspension Factors For Various Ponding Areas and Depths		
	Anticipated Average Ponded Depth	
Anticipated Ponded Area	Less than 2 ft	2 ft or Greater
Less than 100 acres	2.0	1.5
Greater than 100 acres	2.5	2.0

Turbidity

Samples of the supernatant from the flocculent settling test were split to measure turbidity of corresponding TSS concentration (Appendix D). TSS is commonly used as an indicator of the overall performance of CDFs, both for solids retention and for most other contaminants, which are strongly associated with the solid particles by adsorption or ion exchange. Turbidity, being much more easily measured than TSS, may be used instead of TSS during routine operational monitoring if approved by the regulatory agency.

The figures presented in Appendix D show the correlation curves between TSS and turbidity for the Calcasieu River sediment. The field inspector and others can measure the turbidity of the effluent with a turbidity meter and estimate a TSS concentration from the curve. Samples for TSS measurement can be collected less frequently for compliance monitoring and to field verify the correlation for laboratory samples.

Slope Stability and Stress Deformation Analysis.

A preliminary estimate of safe containment dike elevation (rotational stability analysis) was performed using GeoSlope's Sigma/W and Slope/W packages. This preliminary estimate was performed for comparison purposes and to provide an early idea (phase I) of the storage capacity. For this estimate, the supporting data (foundation borings and soil properties) were taken from the 1961 New Orleans District Design Memorandum, in particular from Plate 23's Retention Dike Shear Stability Analysis.

For a 10 ft-elevation (measured from the bottom to top of dike), the slope stability analysis results were similar to those for the 10 ft dike on Plate 23, with a safety factor around 1.4. Since the analyses fairly agree for a 10 ft dike, the same soil properties were used for analyzing the stability of higher dikes.

An 11 ft dike elevation filled to its top was found to have a minimum slope stability factor of safety of 1.2, and a 12 ft dike filled to its top has a minimum slope stability factor of safety of 1.0. If actual dike elevations reported to be 16 ft tall contain dredged material filled to the dike top, the factor of safety is less than one, and the dikes would be highly unstable.

Taking the fill elevation behind the dike into account, Figure 9 shows that a fill elevation between 8 ft and 11 ft yields safety factors approaching unity as the fill elevation is increased. With an 11 ft dike filled to capacity (no freeboard), the factor of safety against rotational failure is 1.2.

After construction of an 11 ft dike, finite element analysis indicated that initial deformation (immediate settlement) would be approximately 1 ft. An initial 11 ft-high dike would in effect become a 10 ft-high dike.

Based on the preliminary analyses for slope stability and initial stress deformation, it is recommended that the retention dikes be built no higher than 11 ft in elevation, with freeboard for 10 ft dredged material fill elevation. The factor of safety against slope failure should be between 1.2 and 1.3.

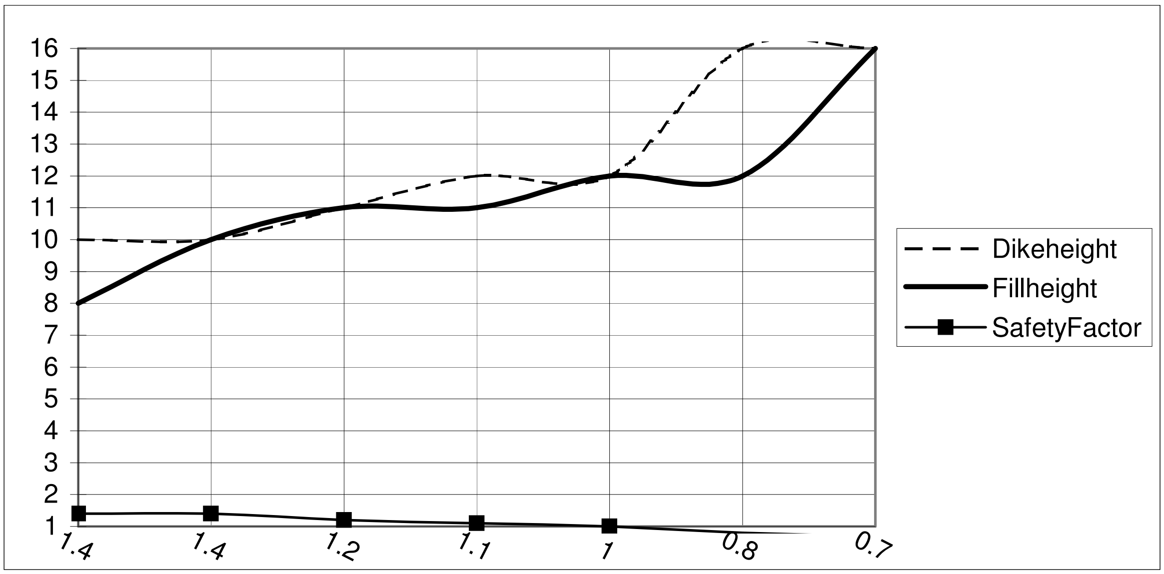

Figure 9. Fill elevation Versus safety factor for dike elevations

Alternatives for allowing higher dike elevations such as soil modification or reinforcement were not explored in this report. Higher dike elevations should be possible using such techniques, based on past projects in Mobile District, Norfolk District, and others.

4 CDF Volumes

Based on the data from the column settling tests, the CDF capacities were calculated for varying fill elevations and volumes. A lidar survey of the disposal areas was provided by the New Orleans District that provided data on the dike elevation of each CDF and the volume at varying fill elevations. The CDFs were grouped into three groups that represent the three reaches of the Calcasieu River that were studied for this phase of the DMMP. The upper reach incorporated CDFs 1 through 12B, the middle reach consisted of CDFs 13 through E, and the lower reach consisted of CDFs H, M, and N. Appendix E presents the lidar survey that was used for calculating the fill elevations.

The data for the column settling test for the upper reach, mile 24 to 36, was obtained from the study performed by Wade in 1994. The upper reach for the Wade report was divided in to three sub-reaches due to differences in the geotechnical characteristics of the sediments found in the upper reach. The three sub-reaches were identified as Reach 1, mile 33-36, Reach 2, mile 30-33, and Reach 3, Mile 24-30. The in-situ volume of material to be dredged from Reaches 1, 2, and 3 are 1.52 million yd^3, 1.73 million yd^3, and 3.25 million yd^3, respectively, for a total of 6.5 million cubic yards. Based on these volumes the SETTLE model computes the storage area needed for the material but does not include ponding within the CDF or freeboard. SETTLE models were run using two different dredge sizes, 27 inch and 30 inch. Using the settling column data, geotechnical data, and dredge size, Reach 1 requires a storage capacity of 2,180,950 yd^3 for the 27 inch dredge and 2,250,217 yd^3 for the 30 inch dredge. Reach 2 requires 1,519,669 yd^3 for the 27 inch dredge and 1,563,445 yd^3 dredge. Reach 3 requires 3,744,171 yd^3 for the 27 inch dredge and 3,852,476 yd^3 for the 30 inch dredge. The total volume requirement for the upper reach of the Calcasieu River is 7,444,790 yd^3 for a 27 inch dredge and 7,666,138 yd^3 for a 30 inch dredge.

Based on the Lidar surveys of disposal areas 1-12B, volumes were calculated at three different fill elevations. These elevations were 10 feet, 12 feet, and 14 feet. This was assuming a 2- foot freeboard within the disposal area so the dikes would be 2 feet higher than the fill elevations. Ponded area was not considered for this evaluation but should be added to assure adequate effluent quality and settling of the material within the CDF. At the 10- foot fill elevation, the volume of the present CDFs is 3,751,821 yd^3. The 12-foot fill elevation had a volume of 4,412,484 yd^3 and the 14-foot fill elevation had a volume of 4,537,518 yd^3. With the information provided on the safe dike elevation of 11 feet, with a safety factor of 1.2, it is not recommended going above this elevation for the dikes unless measures are taken to reinforce the dikes to prevent failure. The 10-foot fill elevation calculations were performed assuming a 12- foot elevation dike with a safety factor of 1.0.

The middle reach of the river, mile 14-24, has 4,500,000 yd^3 of in-situ material to be removed from the channel. The storage area needed for this material using a 27 inch and 30 inch dredge varies from between 4.5 and 9.6 million cubic yards. The large variance of volume needed for the material is due to the fact that the samples collected for the column settling tests had a wide range of moisture content. The moisture content of samples A, B, C, and D ranged from approximately 170% to almost 300%. Since these were grab samples and not cores taken from the channel these results could be misleading and not reflect what is actually present in the channel. Due to this, a range of moisture contents were input into the SETTLE model to predict the volume needed for storage of the dredged material. Using the 10- foot, 12- foot, and 14- foot fill elevations for the disposal areas used for the middle reach, the volumes that are currently available are 1,277,765 yd^3, 1,810,167 yd^3, and 2,520,037 yd^3, respectively.

The lower reach of the river, mile 5-14, has approximately 4 million cubic yards of in-situ material to be dredged from the river. Using calculations for a 27 and 30 inch dredge, the storage capacity needed for the lower reach CDFs ranges between 4 and 9 million cubic yards. Like the middle reach, the samples collected varied in moisture content so different moisture contents were entered into SETTLE in order to obtain a range of the storage volume needed to hold the 4 million cubic yards of in-situ material. The CDFs on the lower reach of the river do not presently have dike elevations over 10 feet. Site H has a dike elevation of 10 feet, site M has an elevation of 6 feet, and site N has a dike elevation of 8 feet in the front and 6 feet in the back. Due to these dike elevations there is only a capacity of 567,896 yd^3 for the placement of dredged material in the lower reach. Depending on the scheduling of the dredging to be performed on the Calcasieu River, disposal areas E and D could be used for some of the dredging done for the lower reach. Table 5 shows that depending on the dike elevation, this would increase the lower reach storage capacity to approximately 1 million cubic yards based on the 10 foot dike elevation. It should be noted that if these areas are used for the lower reach then the capacity for the middle reach will be decreased for the storage of the dredged material from that reach.

The total amount of sediment to be dredged for the Calcasieu River between miles 5 and 36 is 15 million cubic yards of in-situ material. Depending on the dredge size used, 27 or 30 inches, the total storage area needed to dispose of this material is between 16 and 26.5 million cubic yards. The range of the storage area needed is due to the fact that water contents used for the model runs were 100%, 150%, 200%, 250%, and 300% due to the fact that the samples collected and those previously done by Wade in 1994 varied from around 100% to close to 300%. Table 5 presents the current capacities for each CDF used for the placement of dredged material from miles 5-36 along the Calcasieu River. Table 8 presents the storage capacity for each CDF at the 10 foot, 12 foot, and 14 foot fill elevation along with the total volume of material to be dredged.

Table 5. Storage capacity of current CDFs using varying fill elevations								
Disposal Area	**Current capacity fill up to 10'**	**Current capacity fill up to 10'**	**Current capacity fill up to 12'**	**Current capacity fill up to 12'**	**Current capacity fill up to 14'**	**Current capacity fill up to 14'**	**In Situ dredge volume**	**Capacity needed for 30" dredge**
1	150,041		292,014		292,014			
2	48,400		153,267		278,301			
3 (Clooney Island)	194,407		509,815		509,815			
4	171,014		171,014		171,014			
5	192,794		291,208		291,208			
6	Out		Out		Out			
7	772,790		772,790		772,790			
8	909,924		909,924		909,924			
9	0		0		0			
10	204,894		204,894		204,894			
11	197,634		197,634		197,634			
12A	258,134		258,134		258,134			
12B	651,789	3,751,821	651,789	4,412,484	651,789	4,537,518	6,500,000	7,666,138
13 (Choupique Island)	379,135		379,135		379,135			
15	422,695		422,695		422,695			
16N	0		0		0			
16S	Out		Out		Out			
17	0		0		0			
22	0		145,201		145,201			
23	0		24,200		161,334			
D	121,000		484,002		1,056,737			
E	354,935	1,277,765	354,935	1,810,167	354,935	2,520,037	4,500,000	4.5 to 9.8 MCY
H	164,561		164,561		164,561			
M	403,335		403,335		403,335			
N	0	567,896	0	567,896	0	567,896	4,000,000	4.0 to 9.0 MCY
Total	5,597,482	5,597,482	6,790,547	6,790,547	7,625,450	7,625,450	15,000,000	16.2 to 26.5 MCY

5 Conclusions

Conclusions

Based on the result of the settling tests, consolidation tests, and turbidity measurements, it is concluded that:

a. Dredged material from the Calcasieu River and Pass is predominantly fine grain material in the middle and lower reaches accounting for approximately 90 % of the material. The upper reach of the study area averages approximately 40 % sand and 60 % fines.

b. The Calcasieu River sediment exhibited zone settling. The zone settling velocity was 0.195 ft/hr, 0.175 ft/hr (EL sample A), 0.153 ft/hr, 0.172 ft/hr, and 0.131 ft/hr for samples A, B, C, and D, respectively.

c. The curves developed for the correlation between TSS and turbidity for the 4 samples had varying R^2 values ranging from0.4611 to 0.9636. It is suggested that the curve developed by ERDC be used for determining the correlation between TSS and turbidity. It should be noted that this is a rough approximation and should be used for no other reason than to estimate TSS.

d. A slope stability analysis was performed to approximate the safe dike elevation that could be used for the disposal areas. The analysis was performed using data supplied by the New Orleans District in a 1961 memorandum. The safe dike elevation was determined to be 11 feet with a safety factor of 1.2. A dike elevation of 12 feet gives a safety factor of 1.0. It is recommended that dikes not be built above the 11 foot elevation unless measures are taken to strengthen the foundation materials so as to reduce the chance of dike failure.

e. Water contents varied greatly for the samples collected from sites A, B, C, and D. Due to this, and the fact that the upper reach samples previously collected by Wade, 1994 were lower, a range of water contents were used in running the SETTLE model. This resulted in a range of estimated dredged material storage requirements in the middle and lower reaches. More accurate estimates could be achieved if representative water contents were available for the in-situ material to be dredged in each reach.

f. The total volume of material to be dredged from the Calcasieu River and Pass in the short term is 15,000,000 yd^3. The upper reach has a total of 6.5MCY, the middle reach has a total of 4.5 MCY, and the lower reach has a total of 4.0 MCY.

g. Depending on the size dredge used for the removal of the material, 27 or 30 inch, the upper reach requires a storage volume of 7.5 to 7.75 MCY. The middle reach requires a storage area between 4.5 and 9.8 MCY. The lower reach requires a storage volume of between 4.0 and 9.0 MCY. The total area needed for storage between miles 5 and 36 is between 16.0 and 26.5 MCY.

h. Three fill elevations were used to determine the present storage capacities of the CDFs along the Calcasieu River. The 10-foot fill elevation has a storage capacity of approximately 5.6 MCY, the 12-foot fill elevation has a storage capacity of approximately 6.8 MCY, and the 14-foot fill elevation has a storage capacity of approximately 7.6 MCY

i. From the results of the column settling tests and the SETTLE model for the samples collected and the data from Wade 1994, the results indicate that the storage volume present in the CDFs along the Calcasieu River and Pass is not adequate for the storage of all the dredged material that is proposed to be removed in the next 1-3 years.

j. A long-term DMMP needs to performed on the Calcasieu River and Pass to address the issue of the lack of storage capacity for the placement of dredged material over the next 20 years. This DMMP would look at management of the existing CDFs and the siting of new disposal areas along with other uses of the dredged material such as beneficial uses and erosion control.

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APPENDIX A
COMPRESSION SETTLING DATA AND CURVES

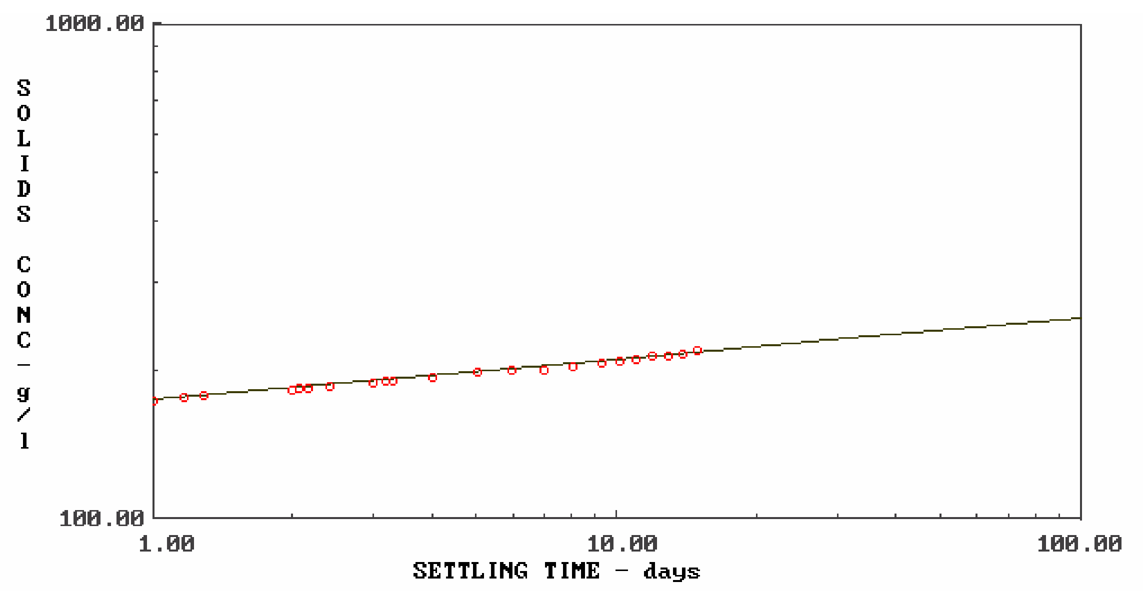
Table 3 Compression Settling Test Data For Sample A (Eustis)				
Date	**Time**	**Time Interval (Hours)**	**Time Interval (Days)**	**Interface Height (Ft)**
09 Dec 2003	0900	0	0	6.85
10 Dec 2003	0900	24	1	5.05
11 Dec 2003	0900	48	2	4.81
12 Dec 2003	0900	72	3	4.67
15 Dec 2003	0800	143	6	4.41
16 Dec 2003	0900	168	7	4.41
17 Dec 2003	1000	193	8	4.35
18 Dec 2003	1530	222.5	9	4.29
19 Dec 2003	1400	245	10	4.25
22 Dec 2003	0900	312	13	4.14
24 Dec 2003	0900	360	15	4.08

Table 3 Compression Settling Test Data For Sample A (EL Sample)				
Date	**Time**	**Time Interval (Hours)**	**Time Interval (Days)**	**Interface Height (Ft)**
09 Dec 2003	0900	0	0	6.24
10 Dec 2003	0900	24	1	4.23
11 Dec 2003	0915	48.25	2	3.98
12 Dec 2003	1000	73	3	3.83
15 Dec 2003	0930	144.5	6	3.58
16 Dec 2003	1000	169	7	3.51
17 Dec 2003	0900	192	8	3.45
18 Dec 2003	1100	218	9	3.40
19 Dec 2003	0915	240.25	10	3.36
22 Dec 2003	1000	313	13	3.24
24 Dec 2003	1000	361.5	15	3.18

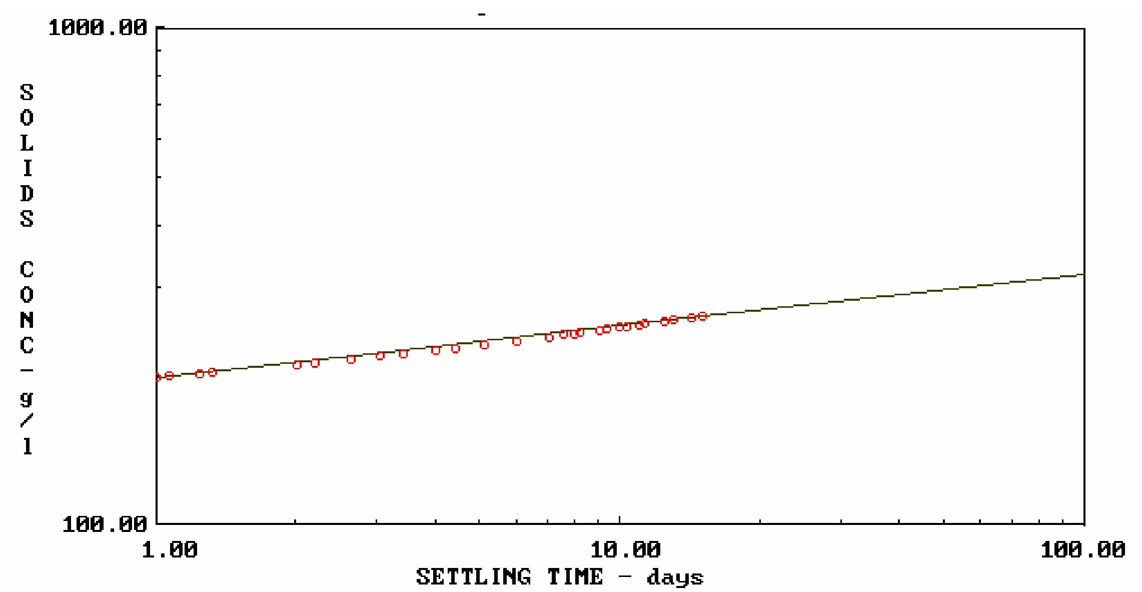
Table 3 Compression Settling Test Data For Sample B				
Date	Time	Time Interval (Hours)	Time Interval (Days)	Interface Height (Ft)
30 Dec 2003	0830	0	0	6.46
31 Dec 2003	0830	24	1	4.88
01 Jan 2004	0830	48	2	4.63
02 Jan 2004	0830	72	3	4.52
05 Jan 2004	0830	144	6	4.26
06 Jan 2004	0830	168	7	4.21
7 Jan 2004	1020	193.83	8	4.16
8 Jan 2004	0830	216	9	4.12
9 Jan 2004	1045	242.25	10	4.09
12 Jan 2004	1300	316.5	13	3.98
14 Jan 2004	0830	360	15	3.93

Table 3 Compression Settling Test Data For Sample C				
Date	Time	Time Interval (Hours)	Time Interval (Days)	Interface Height (Ft)
15 Jan 2004	0800	0	0	6.61
16 Jan 2004	0800	24	1	3.92
17 Jan 2004	0800	48	2	3.65
18 Jan 2004	0800	72	3	3.52
21 Jan 2004	1530	151	6	3.28
22 Jan 2004	0800	168	7	3.24
23 Jan 2004	1600	200	8	3.20
24 Jan 2004	1230	220.5	9	3.14
25 Jan 2004	1200	244	10	3.10
28 Jan 2004	0800	312	13	3.01
30 Jan 2004	0800	360	15	2.98

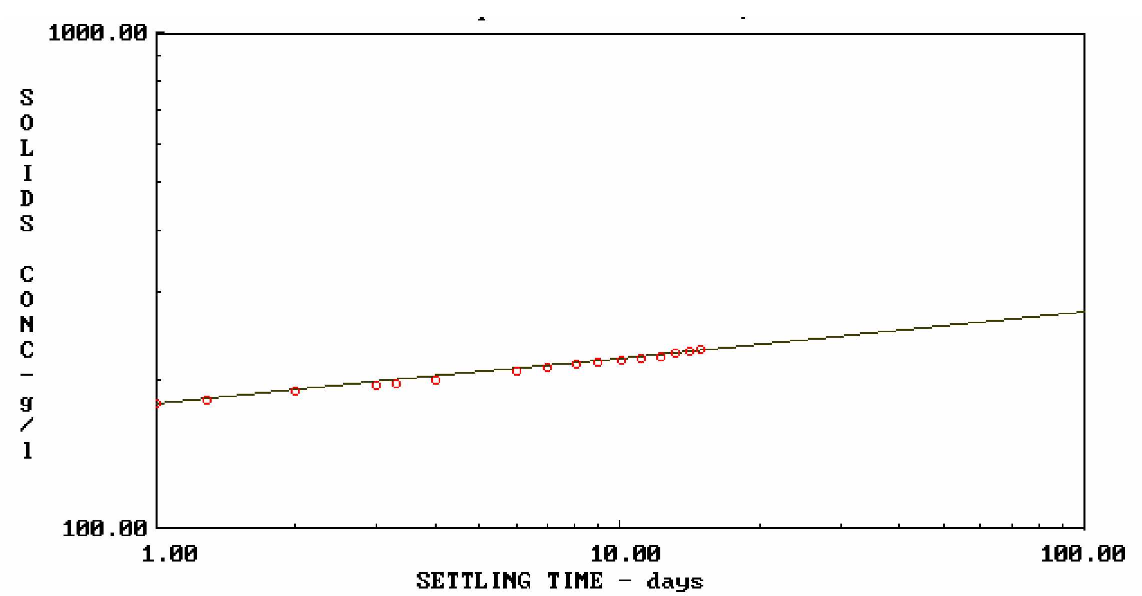
Table 3 Compression Settling Test Data For Sample D				
Date	**Time**	**Time Interval (Hours)**	**Time Interval (Days)**	**Interface Height (Ft)**
31 Jan 2004	0900	0	0	6.38
01 Feb 2004	0900	24	1	3.35
02 Feb 2004	0900	48	2	2.90
03 Feb 2004	0900	72	3	2.74
06 Feb 2004	0800	143	6	2.49
07 Feb 2004	0900	168	7	2.42
09 Feb 2004	1000	217	9	2.33
10 Feb 2004	1550	246.83	10	2.28
13 Feb 2004	1300	316	13	2.19
15 Feb 2004	0900	360	15	2.14



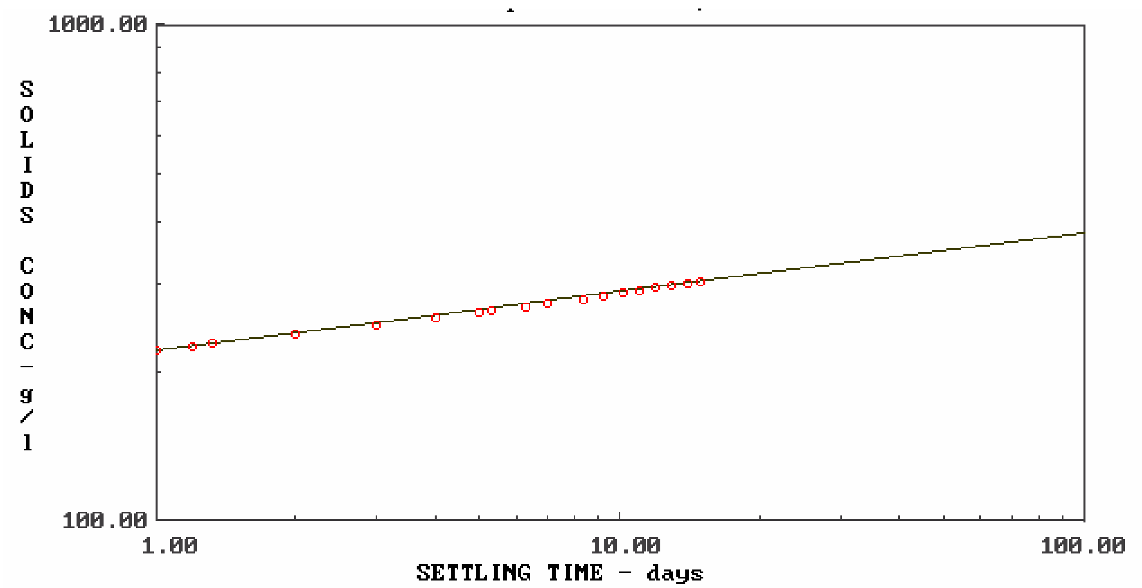
Calcasieu River Sample A (Eustis) compression settling curve



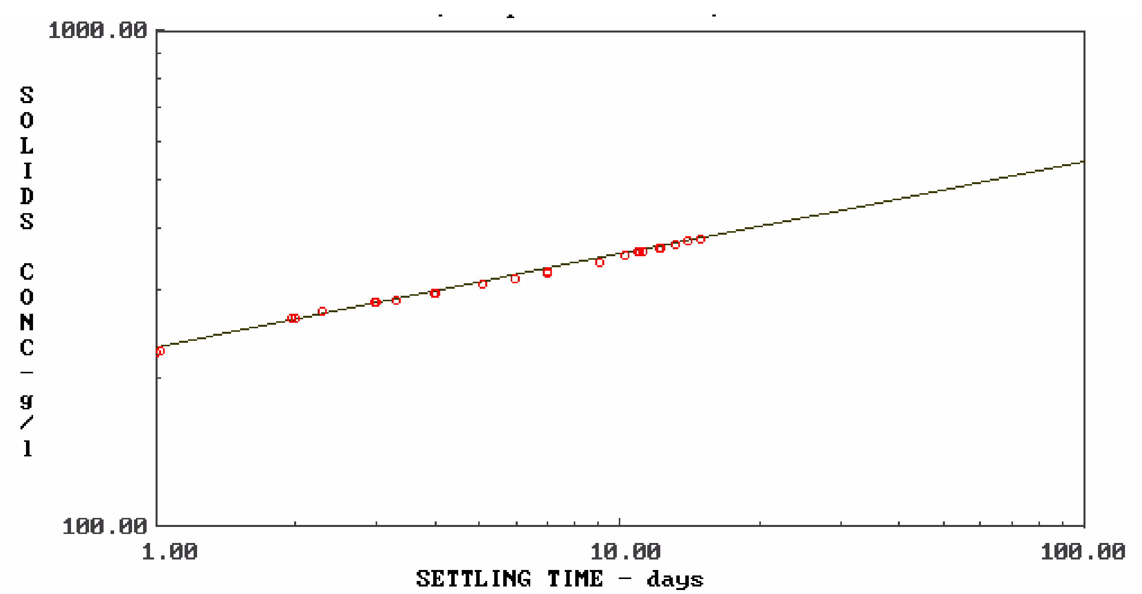
Calcasieu River Sample A (ERDC) compression settling curve



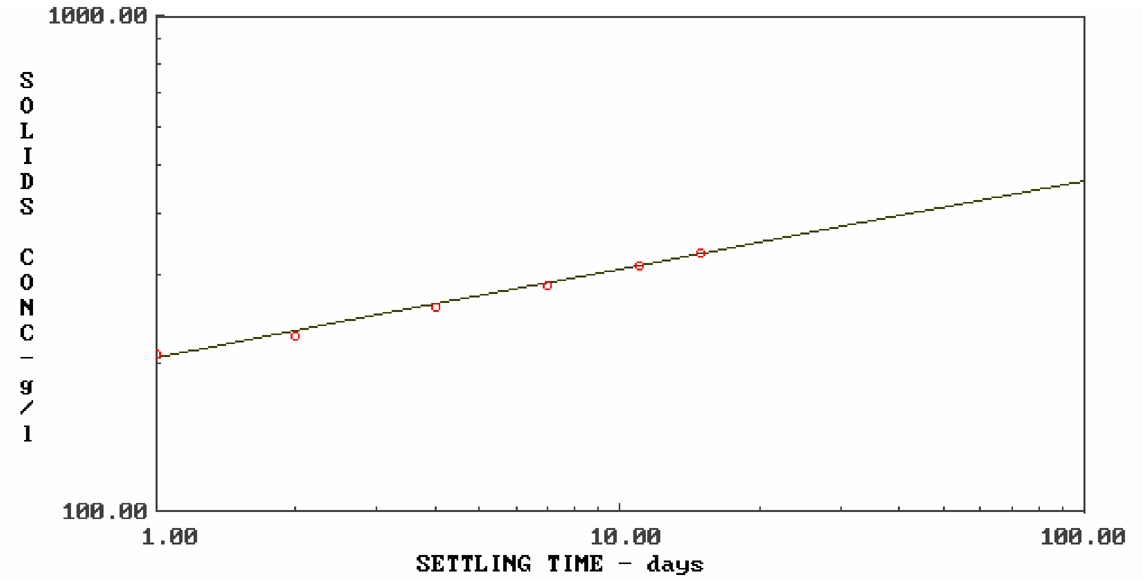
Calcasieu River Sample B compression settling curve



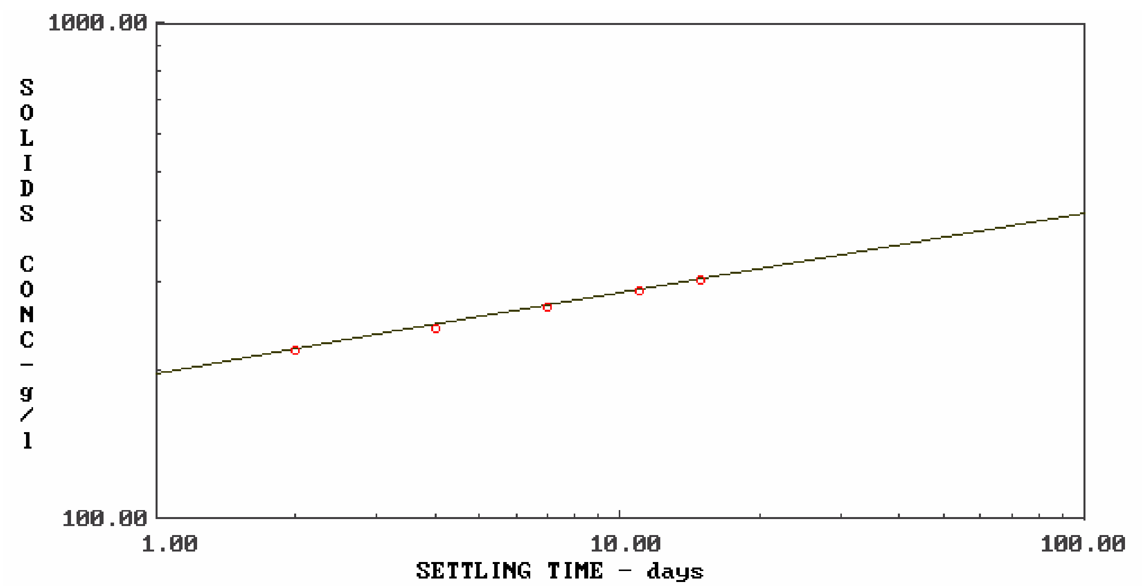
Calcasieu River Sample C compression settling curve



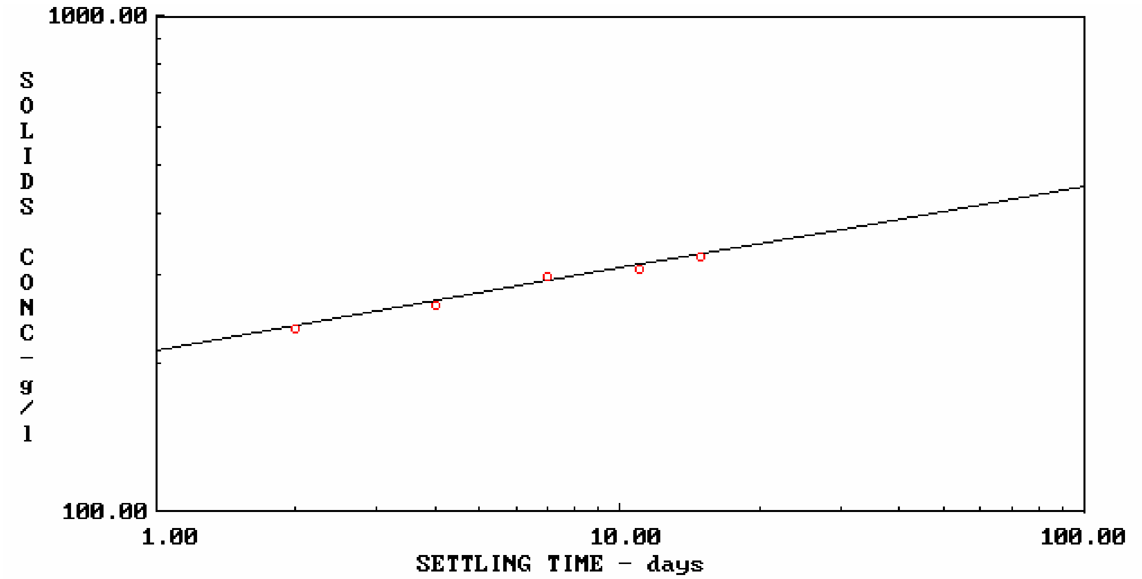
Calcasieu River Sample D compression settling curve



Calcasieu River Upper Reach 1 compression settling curve



Calcasieu River Upper Reach 2 compression settling curve



Calcasieu River Upper Reach 3 compression settling curve

APPENDIX B
ZONE SETTLING DATA AND CURVES

Table 4 Zone Settling Test Data Sample A

Time	Elapsed Time, hrs	Interface Height, ft
0900 09 Dec 2003	0.00	6.85
0930	0.50	6.82
1000	1.00	6.78
1030	1.5	6.76
1100	2.00	6.74
1130	2.5	6.72
1200	3.0	6.69
1300	4.0	6.58
1330	4.5	6.50
1400	5.0	6.41
1430	5.50	6.33
1500	6.0	6.21
1530	6.5	6.11
1600	7.0	6.01
1800	9.0	5.69
1900	10.0	5.51
2100	12.0	5.39
0900 10 Dec 2003	24.0	5.05

Notes: The slurry concentration was 127.65 g/L. The salinity was 26.5 parts per thousand.

Table 4 Zone Settling Test Data Sample A (Environmental Lab)

Time	Elapsed Time, hrs	Interface Height, ft
0900 09 Dec 2003	0.00	6.24
0930	0.50	6.24
0945	0.75	6.24
1000	1.0	6.22
1015	1.25	6.20
1030	1.5	6.17
1045	1.75	6.14
1100	2.0	6.115
1115	2.25	6.08
1130	2.5	6.04
1145	2.75	6.0
1215	3.25	5.91
1230	3.5	5.866
1240	3.66	5.86
1300	4.0	5.81
1330	4.5	5.715
1400	5.0	5.62
1430	5.5	5.54
1500	6.0	5.48
1530	6.5	5.38
1600	7.0	5.29
1700	8.0	5.14
1900	10.0	4.79
2130	12.5	4.53
2315	14.25	4.46
0200	17.0	4.367
0900 10 Dec 2003	24	4.23

Notes: The slurry concentration was 135.4 g/L. The salinity was 26.5 parts per thousand.

Table 4 Zone Settling Test Data Sample B		
Time	Elapsed Time, hrs	Interface Height, ft
0830 30 Dec 2003	0.00	6.46
0845	0.25	6.44
0900	0.50	6.42
0915	0.75	6.40
0930	1.0	6.40
0945	1.25	6.38
1000	1.50	6.37
1030	2.0	6.36
1045	2.25	6.36
1100	2.5	6.35
1115	2.75	6.31
1130	3.0	6.31
1145	3.25	6.30
1200	3.50	6.28
1215	3.75	6.25
1230	4.0	6.22
1245	4.25	6.20
1300	4.50	6.16
1315	4.75	6.12
1330	5.0	6.08
1345	5.25	6.05
1400	5.50	6.01
1415	5.75	5.97
1430	6.0	5.93
1445	6.25	5.89
1530	7.0	5.78
2030	12.0	5.28
0830 31 Dec 2004	24	4.88
Notes: The slurry concentration was 125.46 g/L. The salinity was 26.5 parts per thousand.		

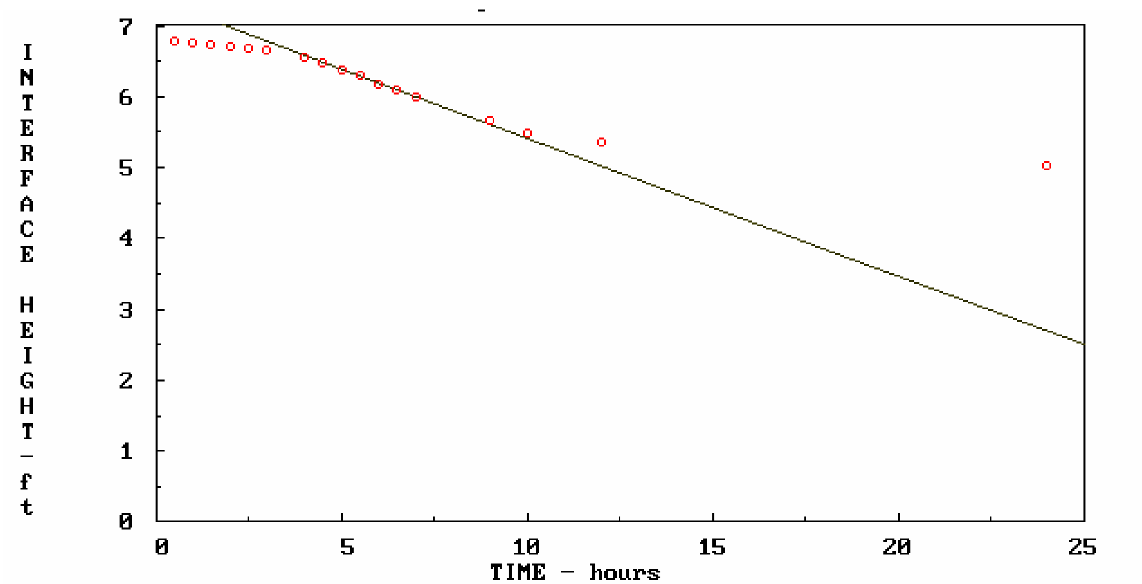
Table 4 Zone Settling Test Data Sample C

Time	Elapsed Time, hrs	Interface Height, ft
0800 15 Jan 2004	0.00	6.61
0815	0.25	6.60
0830	0.5	6.57
0845	0.75	6.54
0900	1.0	6.49
0915	1.25	6.47
0930	1.50	6.43
0945	1.75	6.37
1000	2.0	6.34
1030	2.5	6.26
1045	2.75	6.21
1115	3.25	6.10
1130	3.50	6.08
1200	4.0	6.00
1230	4.5	5.90
1245	4.75	5.83
1300	5.0	5.79
1315	5.25	5.74
1330	5.50	5.71
1345	5.75	5.66
1400	6.0	5.63
1415	6.25	5.58
1430	6.50	5.55
1445	6.75	5.50
1500	7.0	5.47
1530	7.50	5.42
1600	8.0	5.35
1730	9.5	5.13
2000	12.0	4.79
0800 16 Jan 2004	24	3.92

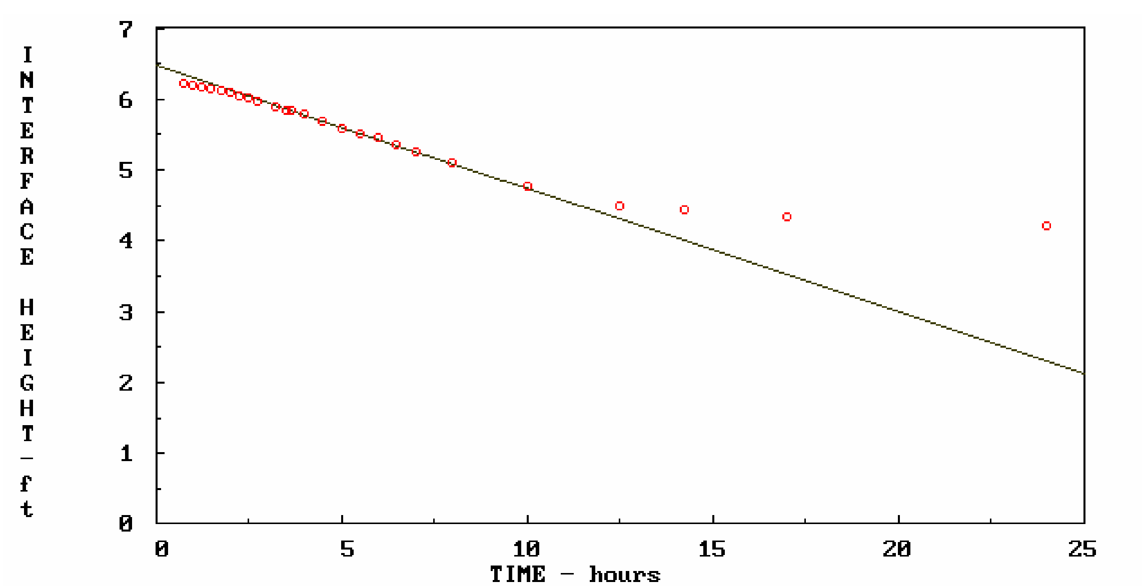
Notes: The slurry concentration was 127.65 g/L. The salinity was 26.5 parts per thousand.

Table 4 Zone Settling Test Data Sample D		
Time	**Elapsed Time, hrs**	**Interface Height, ft**
0900 31 Jan 2004	0.00	6.38
0915	0.25	6.36
0930	0.50	6.32
0945	0.75	6.28
1000	1.0	6.24
1015	1.25	6.21
1030	1.50	6.18
1045	1.75	6.15
1100	2.0	6.12
1115	2.25	6.09
1130	2.50	6.06
1145	2.75	6.03
1200	3.0	6.00
1215	3.25	5.96
1230	3.50	5.92
1245	3.75	5.88
1300	4.0	5.85
1315	4.25	5.82
1330	4.50	5.79
1345	4.75	5.76
1400	5.0	5.73
1415	5.25	5.69
1430	5.50	5.65
1445	5.75	5.62
1500	6.0	5.59
1515	6.25	5.56
1530	6.50	5.53
1545	6.75	5.49

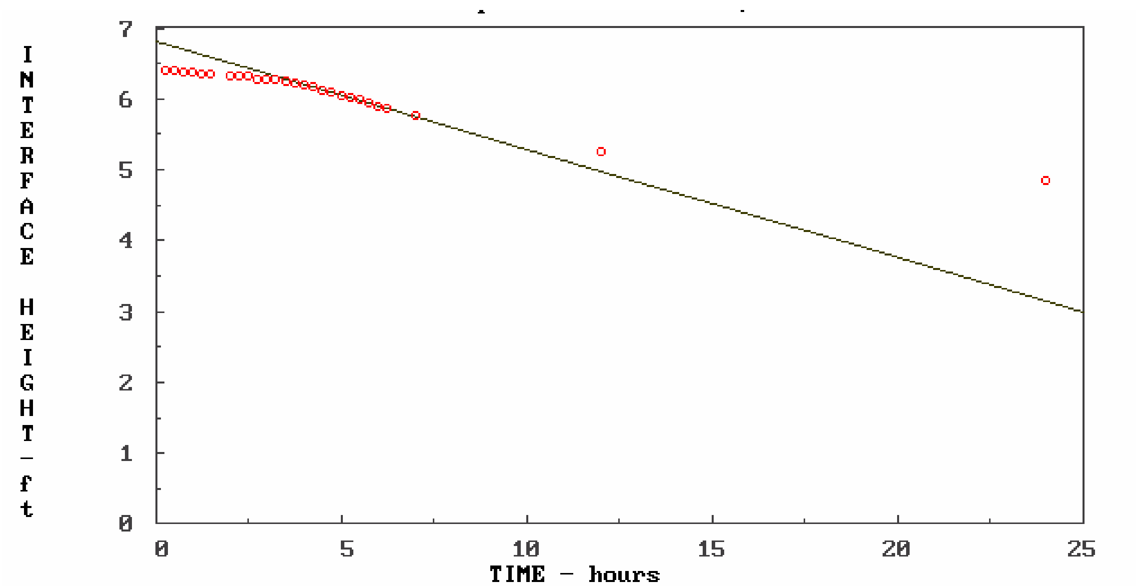
1600	7.0	5.46
2100	12.0	4.84
0845 01 Feb 2004	23.75	3.36
Notes: The slurry concentration was 113.12 g/L. The salinity was 26.5 parts per thousand.		



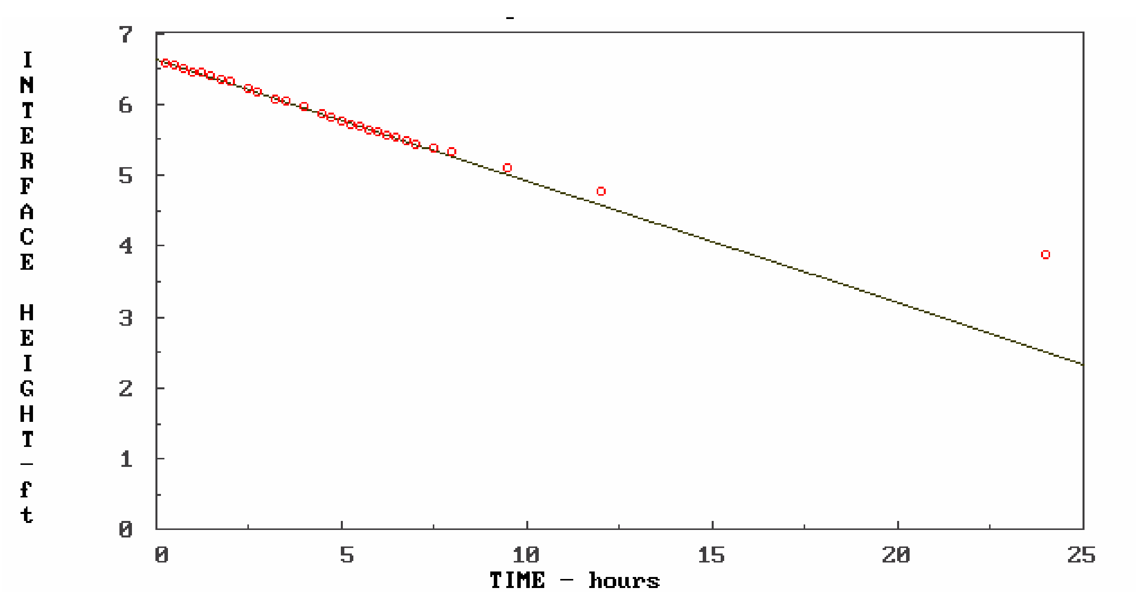
Calcasieu River Sample A (Eustis) zone settling curve



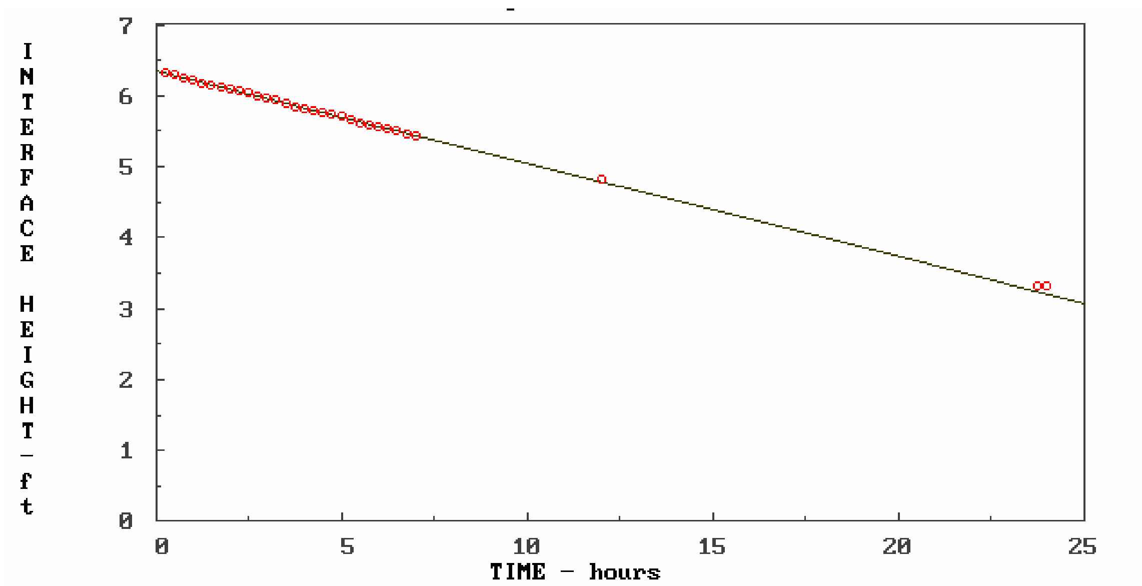
Calcasieu River Sample A (ERDC) zone settling curve



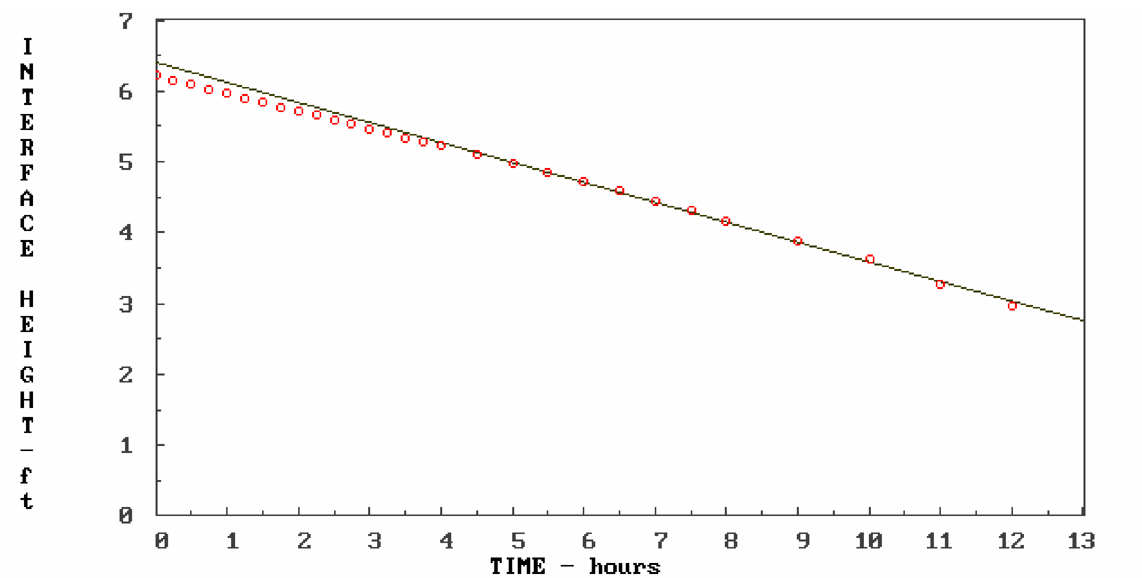
Calcasieu River Sample B zone settling curve



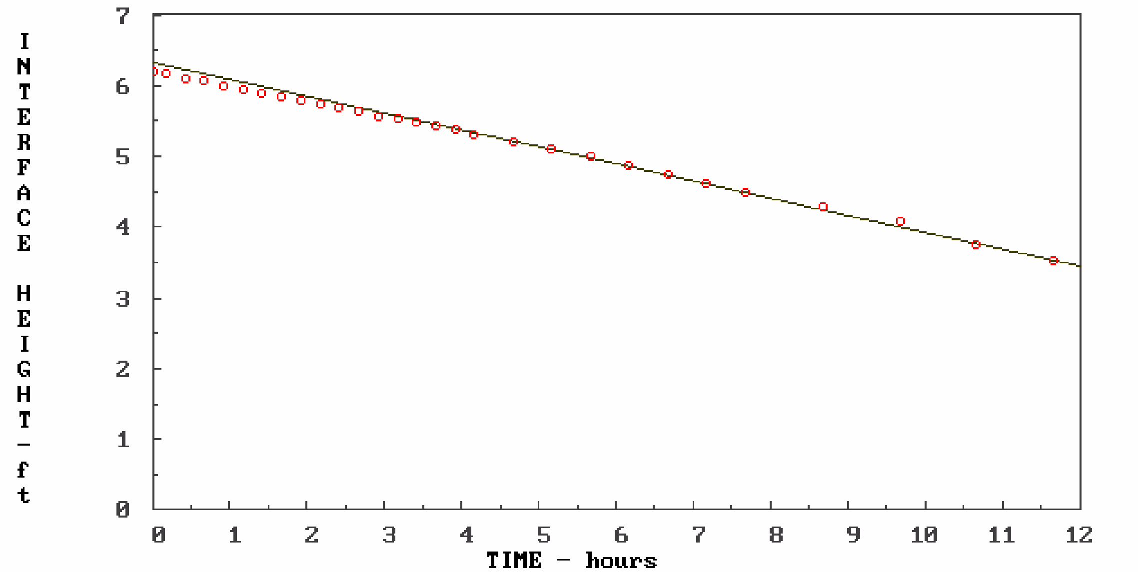
Calcasieu River Sample C zone settling curve



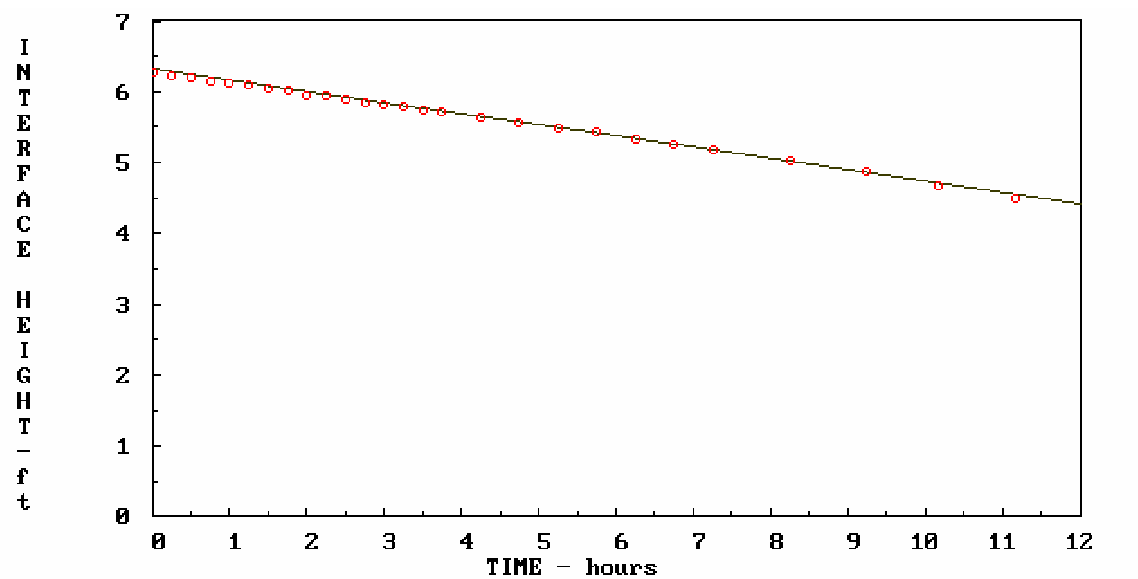
Calcasieu River Sample D zone settling curve



Calcasieu River Upper Reach 1 zone settling curve



Calcasieu River Upper Reach 2 zone settling curve



Calcasieu River Upper Reach 3 zone settling curve

APPENDIX C
FLOCCULENT SETTLING DATA AND CURVES

Flocculent Settling Test Data Sample A								
Time, hr	Port Height, ft[1]							
	6.0	5.5	5.0	4.5	4.0	3.5	3.0	2.5
7	153	BI	BI	BI	BI	BI	BI	BI
12	76	76	BI	BI	BI	BI	BI	BI
24	67	76	134	BI	BI	BI	BI	BI
48	64	62	73	BI	BI	BI	BI	BI
72	32	57	71	158	BI	BI	BI	BI
96	35	41	96	66	BI	BI	BI	BI
168	32.67	46	85.56	38.10	BI	BI	BI	BI
264	22.22	27	70.97	41.76	44	BI	BI	BI
360		15.38	47.78	40	44.32	BI	BI	BI

[1]The initial slurry concentration was 127.65 g/L.
[2]Concentration at highest port used as initial supernatant concentration (mg/l).
BI = Port is Below Interface, and no sample was collected at this time interval.

Flocculent Settling Test Data Sample A (Environmental Lab)

Time, hr	Port Height, ft[1]							
	6.0	5.5	5.0	4.5	4.0	3.5	3.0	2.5
3.5	113	BI	BI	BI	BI	BI	BI	BI
5	68	BI	BI	BI	BI	BI	BI	BI
7	58	63	BI	BI	BI	BI	BI	BI
12.5	23.4	39	24	BI	BI	BI	BI	BI
24	29	25	35	85	BI	BI	BI	BI
48		22.5	20	25	BI	BI	BI	BI
73		9.92	13.19	18.68	20	BI	BI	BI
96		7.75	9.73	9.69	7.54	BI	BI	BI
169		7	5.5	6.5	11.5	BI	BI	BI
240.25		8	5	9.5	5	14.5	BI	BI
361.5			2.65	4.4	4.04	5.86	BI	BI

[1]The initial slurry concentration was 135.4 g/L.
[2]Concentration at highest port used as initial supernatant concentration (mg/l).
BI = Port is Below Interface, and no sample was collected at this time interval.

Flocculent Settling Test Data Sample B

Time, hr	Port Height, ft[1]							
	6.0	5.5	5.0	4.5	4.0	3.5	3.0	2.5
7	226	BI	BI	BI	BI	BI	BI	BI
12	78.72	70.08	206	BI	BI	BI	BI	BI
24	74.31	77	83	100	BI	BI	BI	BI
48		44	64	69.23	BI	BI	BI	BI
72		45	54	51.11	148	BI	BI	BI
96		38	54	41.93	87.64	BI	BI	BI
168		30	40.66	70.3	140	BI	BI	BI
264		23.76	29.52	27.78	76.9	217	BI	BI
360		17.43	21.74	30.43	124.78	159	BI	BI
							BI	BI
							BI	BI

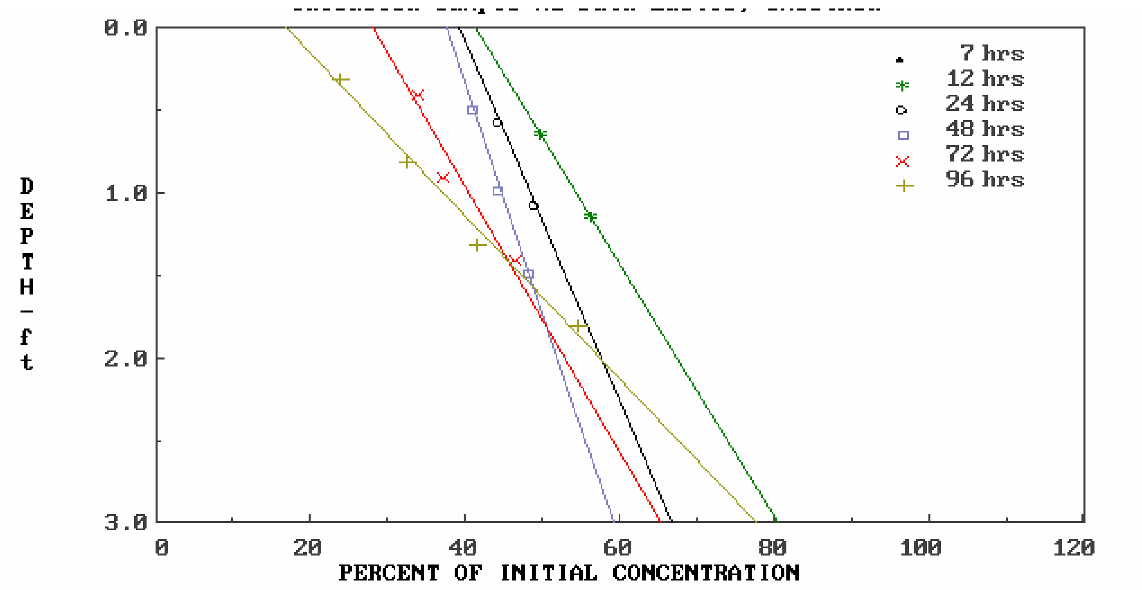
[1]The initial slurry concentration was 135.4 g/L.
[2]Concentration at highest port used as initial supernatant concentration (mg/l).
BI = Port is Below Interface, and no sample was collected at this time interval.

Flocculent Settling Test Data Sample C								
Time, hr	Port Height, ft[1]							
	6.0	5.5	5.0	4.5	4.0	3.5	3.0	2.5
4	129	BI	BI	BI	BI	BI	BI	BI
7	98.9	55.8	BI	BI	BI	BI	BI	BI
12	57.3	42.7	120	BI	BI	BI	BI	BI
24	12.6	30	45.6	101	53	BI	BI	BI
48	47.6	90	43.3	50	137	53.6	BI	BI
72	22.8	26.6	37.2	41.3	46.1	19.5	BI	BI
96		14.4	26.8	23.5	72.5	18.4	BI	BI
168		12.3	18.4	19.4	18.1	17.5	142	BI
264		10.2	18.1	17.1	14.6	11.5	26.7	BI
360		5.3	8.2	25.5	17.8	6.1	74	BI

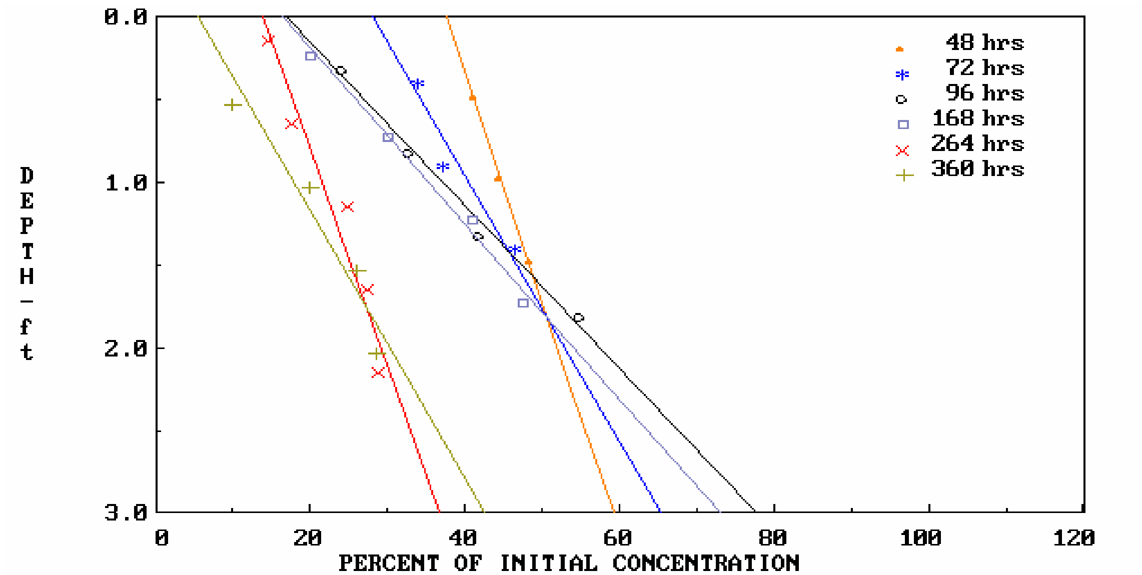
[1]The initial slurry concentration was 135.4 g/L.
[2]Concentration at highest port used as initial supernatant concentration (mg/l).
BI = Port is Below Interface, and no sample was collected at this time interval.

Flocculent Settling Test Data Sample D								
Time, hr	Port Height, ft[1]							
	6.0	5.5	5.0	4.5	4.0	3.5	3.0	2.5
4	198	BI	BI	BI	BI	BI	BI	BI
7	8.2	39.3	BI	BI	BI	BI	BI	BI
12	21.6	34.4	36.6	BI	BI	BI	BI	BI
24	12.1	15.6	26.9	55.3	82.7	88	BI	BI
48		18.6	3.5	20.9	13	36.9	192	BI
72		4.4	11.6	16.3	56	15.3	220	BI
96		3.6	11.6	2.7	22.6	9.9	61	BI
168			2.7	5.3	10.8	8.6	45	66
264			4.8	6.9	2.7	5.9	19.4	160
360			3.4	5.4	8.5	6.1	18.8	45

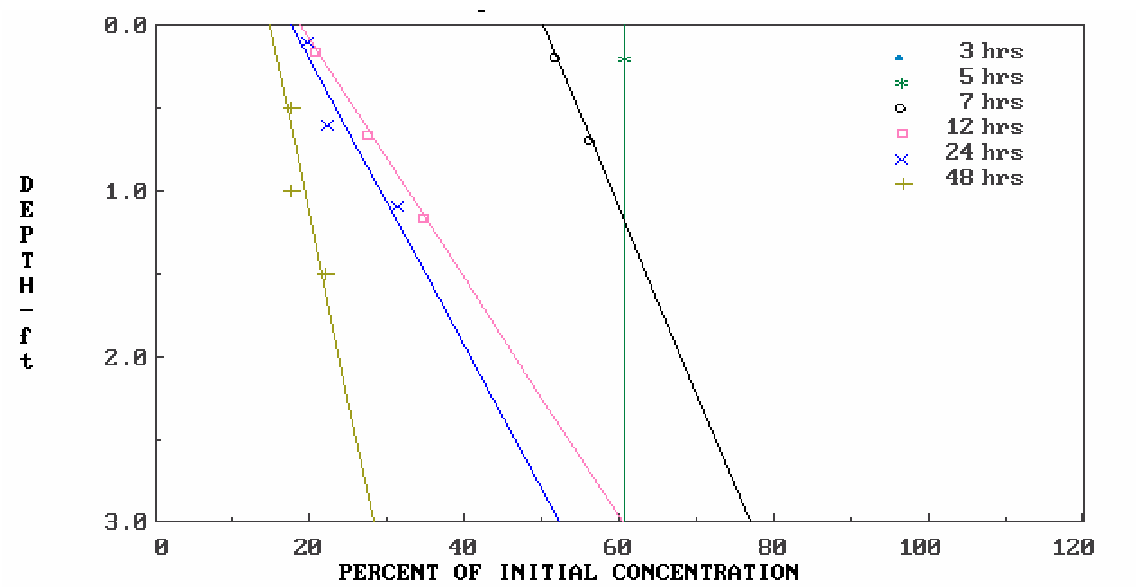
[1]The initial slurry concentration was 135.4 g/L.
[2]Concentration at highest port used as initial supernatant concentration (mg/l).
BI = Port is Below Interface, and no sample was collected at this time interval.



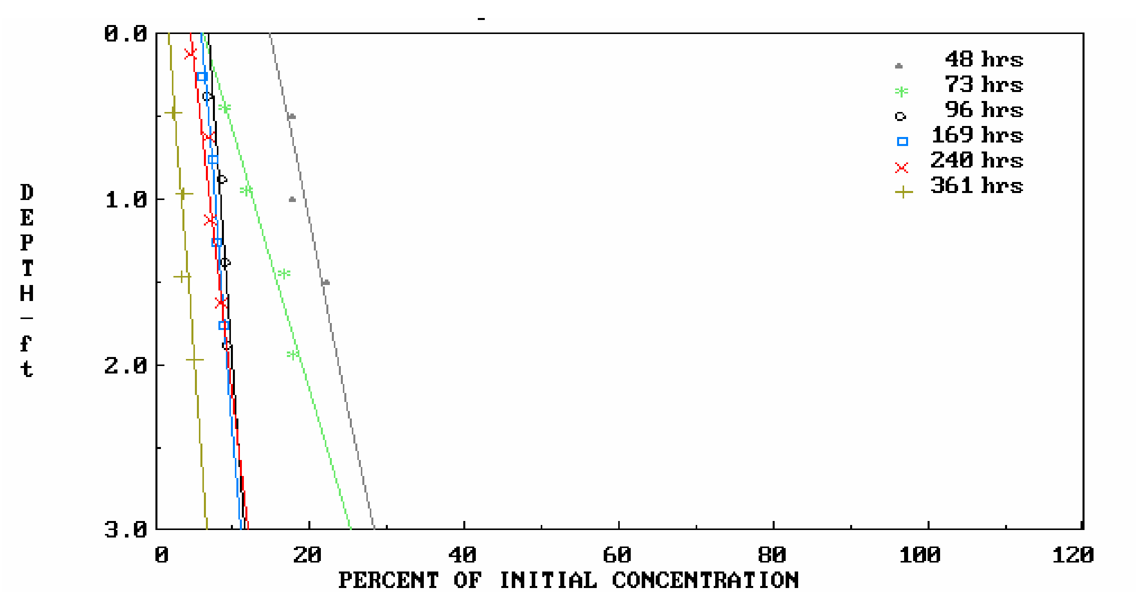
Calcasieu River Sample A (Eustis) flocculent settling curves, set 1



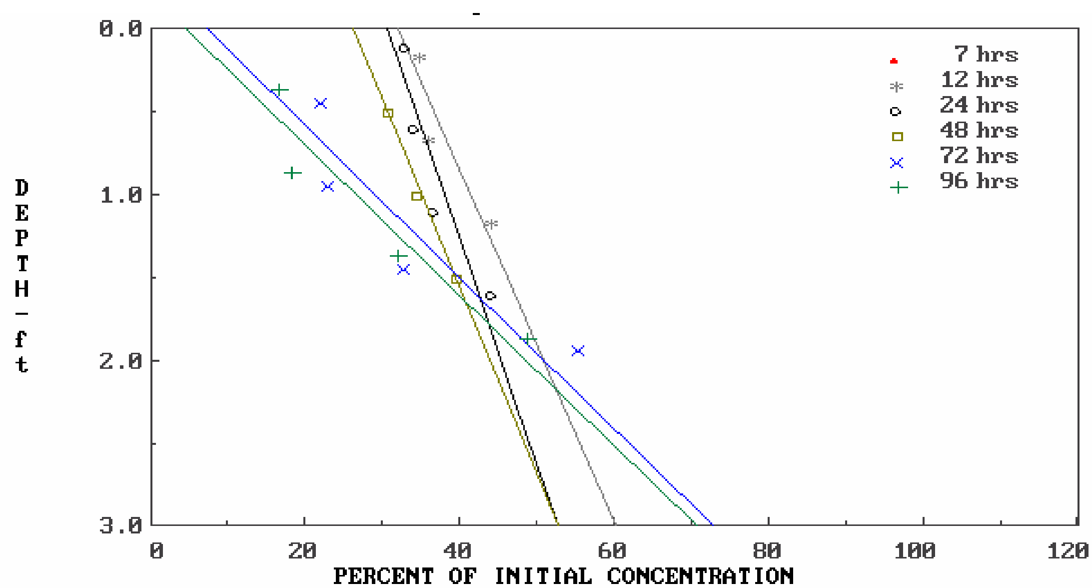
Calcasieu River Sample A (Eustis) flocculent settling curves, set 2



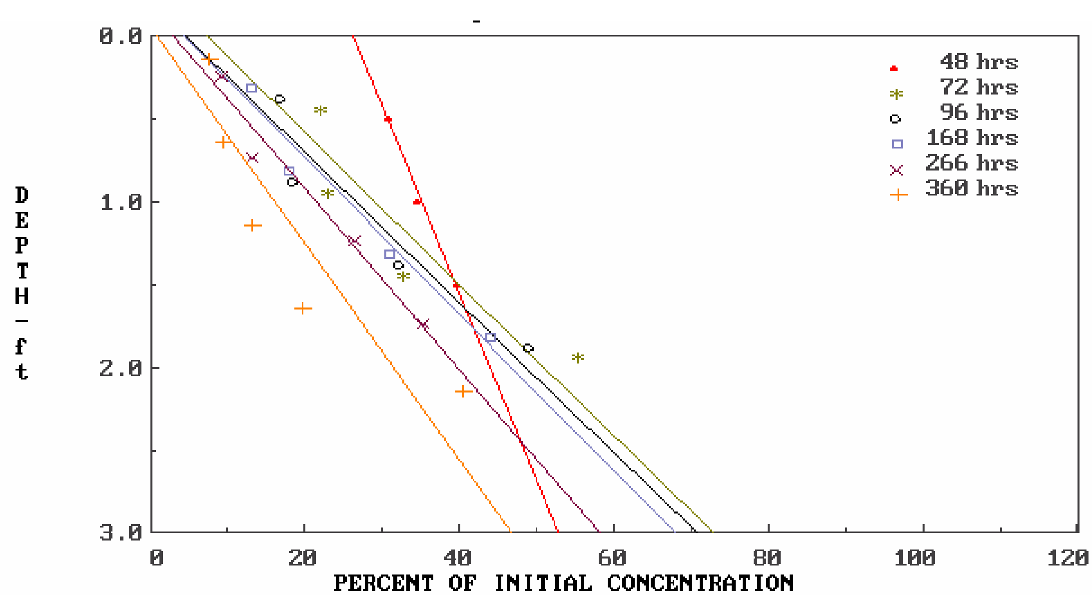
Calcasieu River Sample A (ERDC) flocculent settling curves, set 1



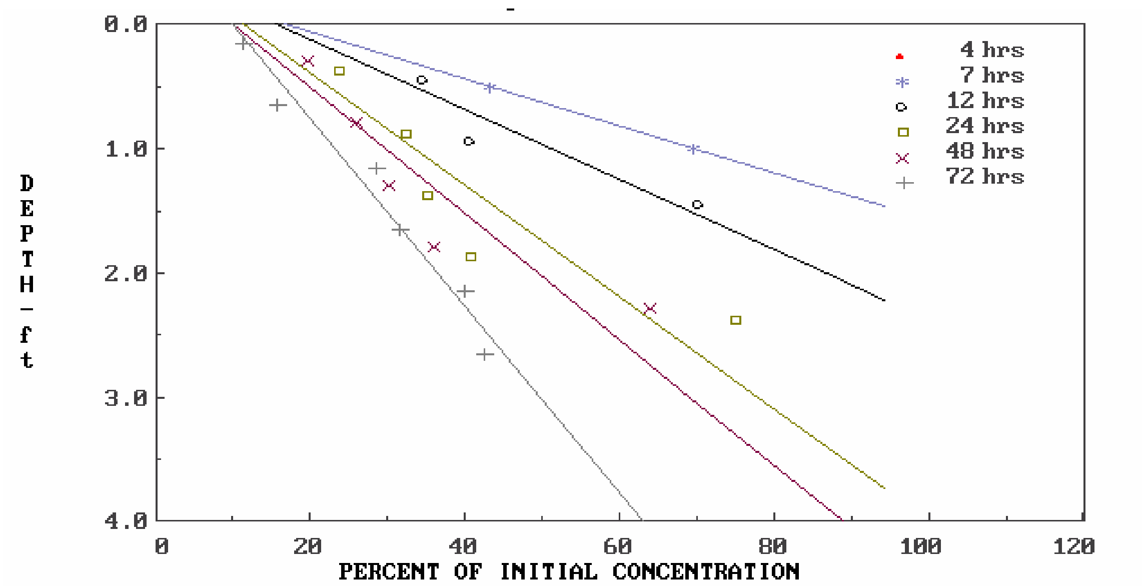
Calcasieu River Sample A (ERDC) flocculent settling curves, set 2



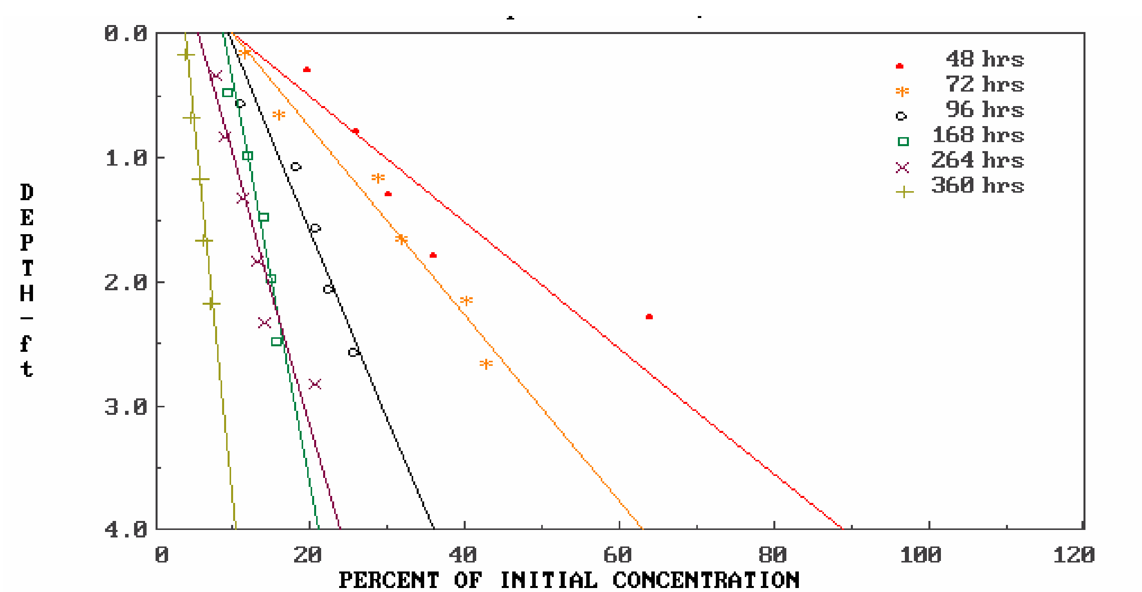
Calcasieu River Sample B flocculent settling curves, set 1



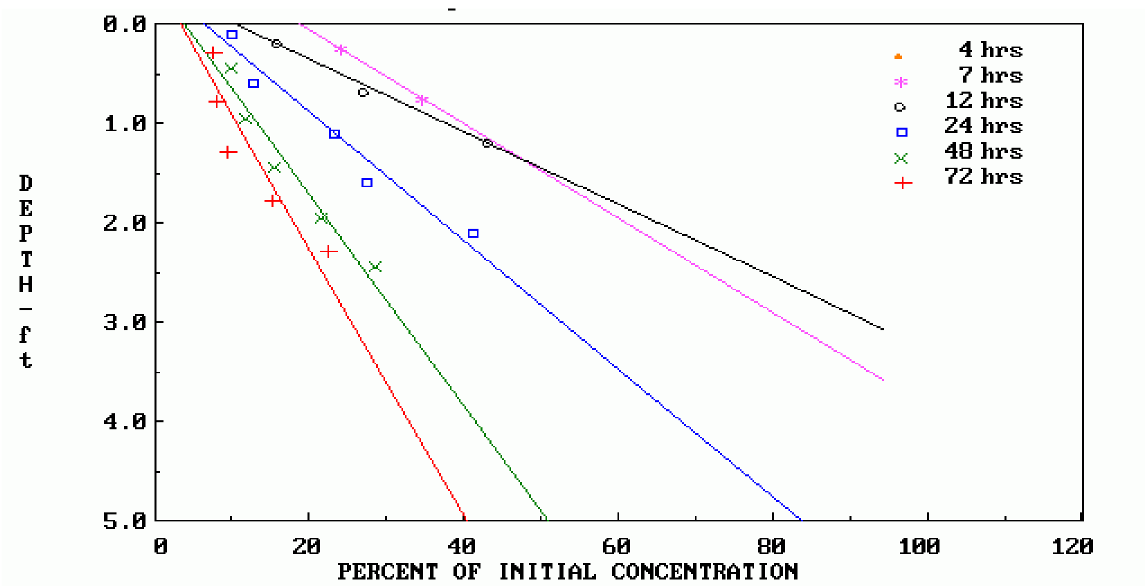
Calcasieu River Sample B flocculent settling curves, set 2



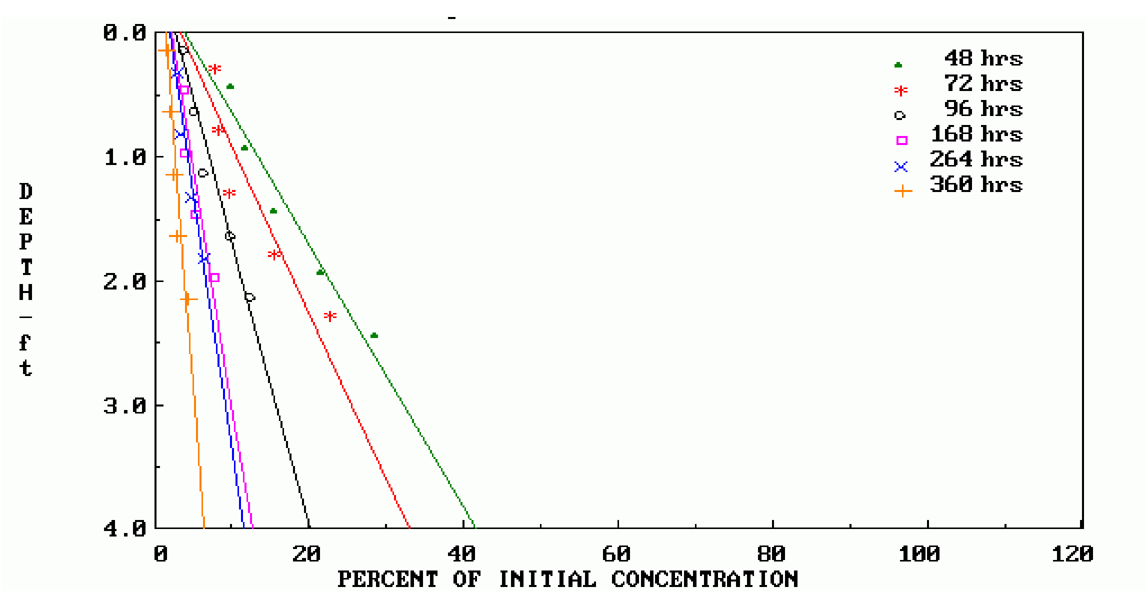
Calcasieu River Sample C flocculent settling curves, set 1



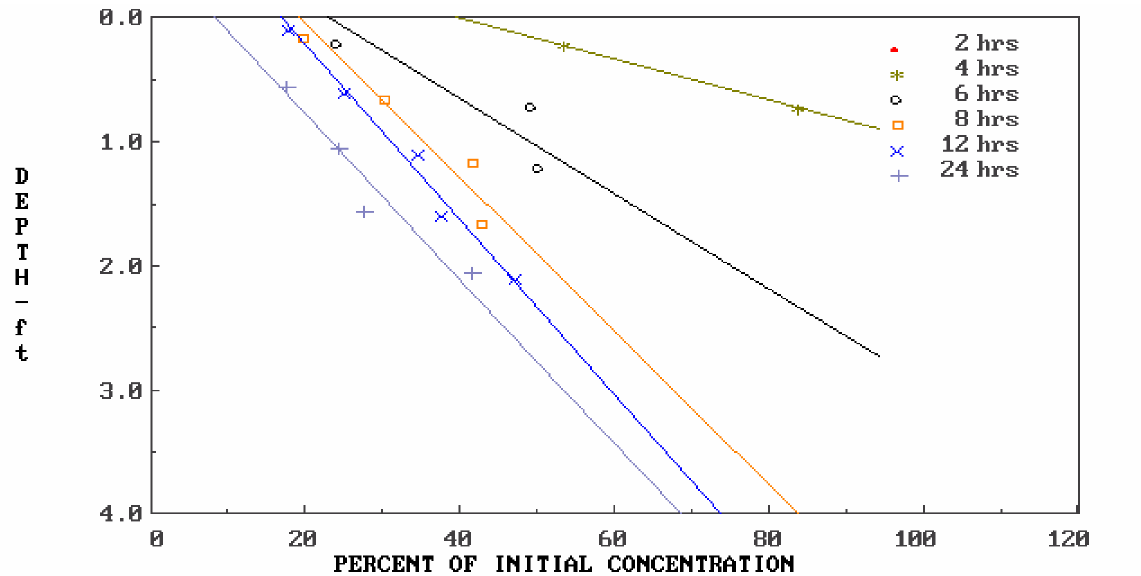
Calcasieu River Sample C flocculent settling curves, set 2



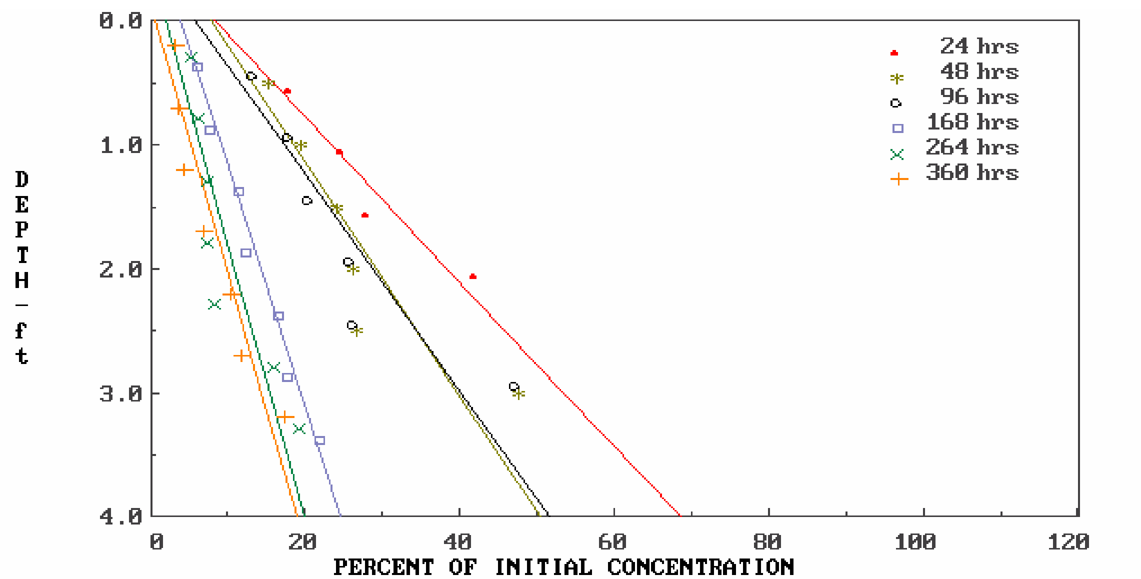
Calcasieu River Sample D flocculent settling curve, set 1



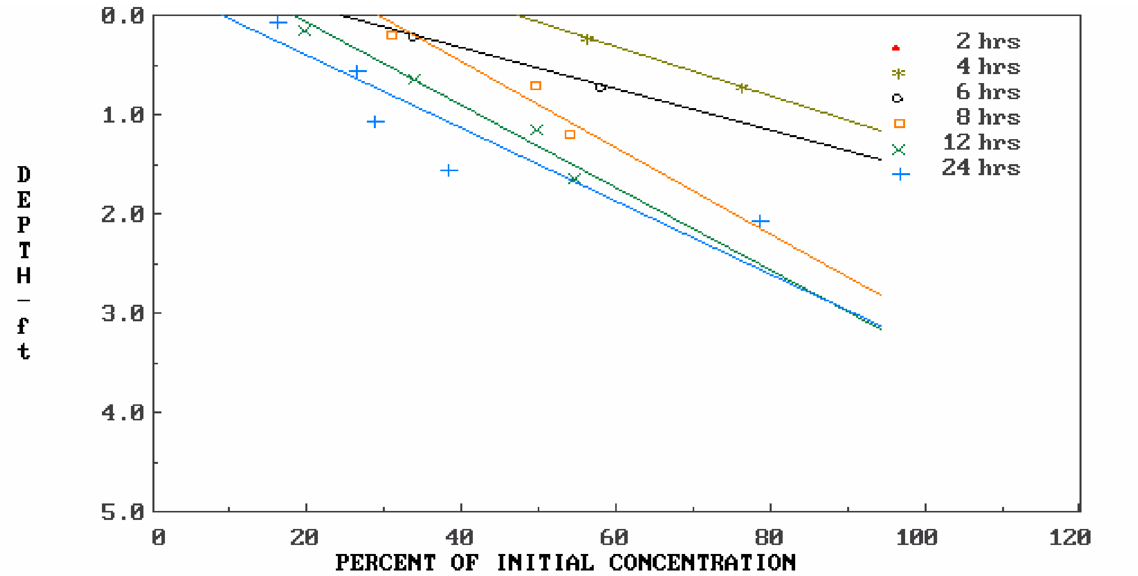
Calcasieu River Sample D flocculent settling curve, set 2



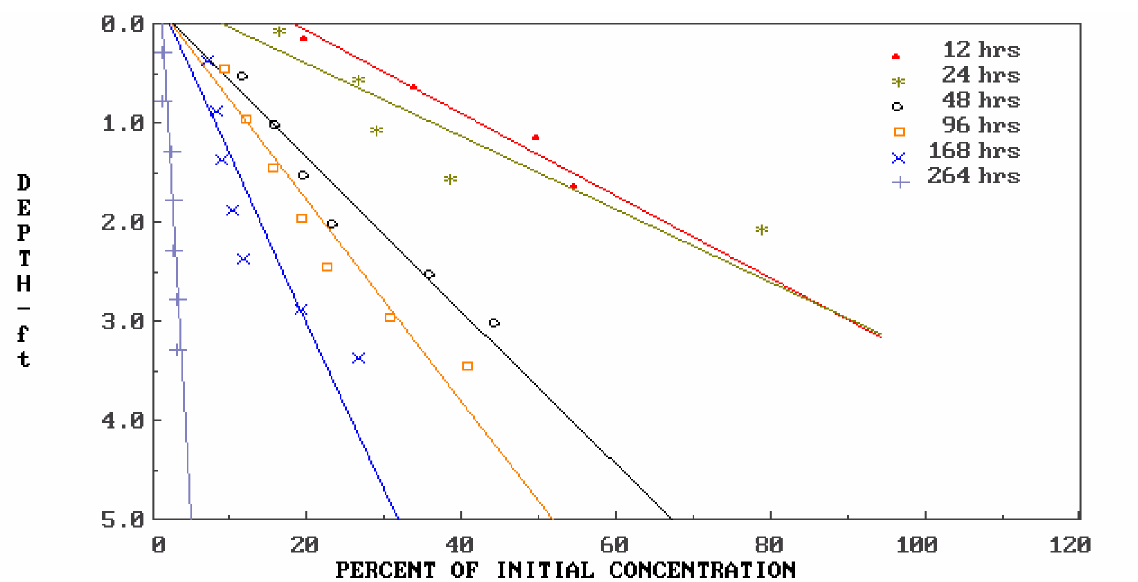
Calcasieu River Upper Reach 1 flocculent settling curves, set 1



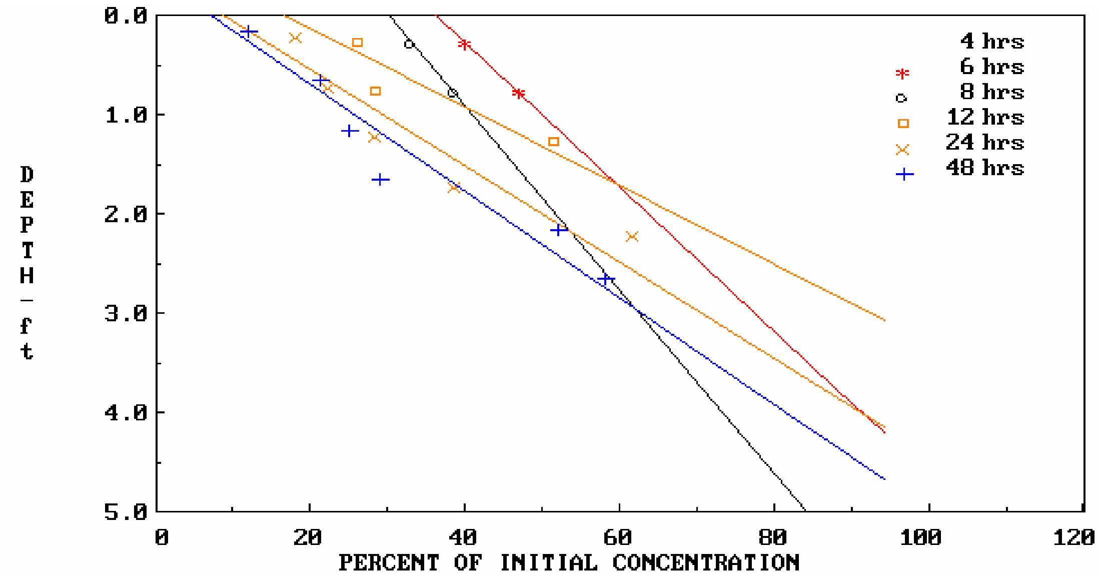
Calcasieu River Upper Reach 1 flocculent settling curves, set 2



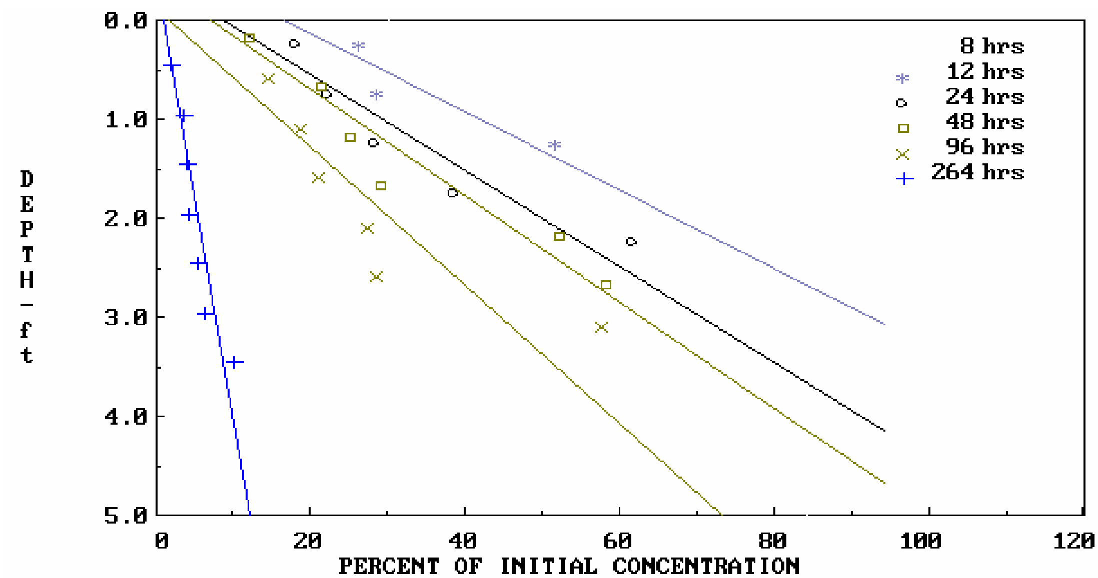
Calcasieu River Upper Reach 2 flocculent settling curves, set 1



Calcasieu River Upper Reach 2 flocculent settling curves, set 2



Calcasieu River Upper Reach 3 flocculent settling curves, set 1



Calcasieu River Upper Reach 3 flocculent settling curves, set 2

APPENDIX D
TSS vs TURBIDITY DATA AND CURVES

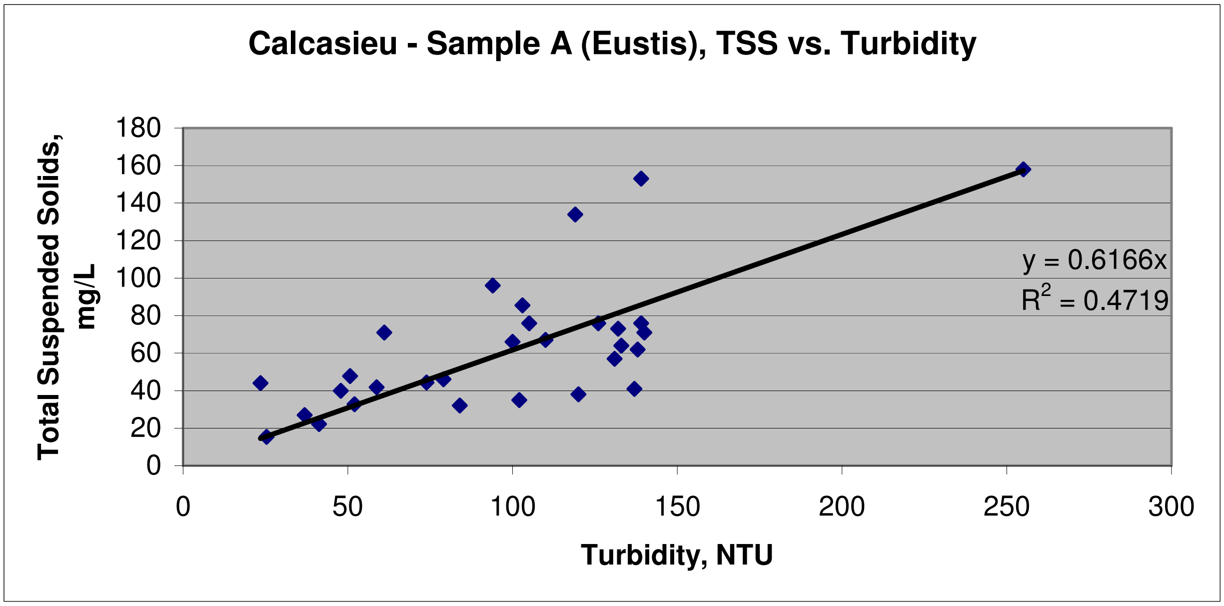
TSS Concentrations and Turbidity Measurements Sample A (Eustis)							
Time, hr	Port No.	TSS mg/L	Turbidity NTU	Time, hr	Port No.	TSS mg/L	Turbidity NTU
7	6	153	139	168	4.5	38.1	120
12	6	76	126	264	6	22.2	41.3
12	5.5	76	139	264	5.5	27	36.9
24	6	67	110	264	5	70.9	61.1
24	5.5	76	105	264	4.5	41.7	58.7
24	5	134	119	264	4	44	23.5
48	6	64	133	360	5.5	15.4	25.3
48	5.5	62	138	360	5	47.8	50.7
48	5	73	132	360	4.5	40	47.9
72	6	32	84	360	4	44.3	73.9
72	5.5	57	131				
72	5	71	140				
72	4.5	158	255				
96	6	35	102				
96	5.5	41	137				
96	5	96	94				
96	4.5	66	100				
168	6	32.67	52				
168	5.5	46	79				
168	5	85.5	103				

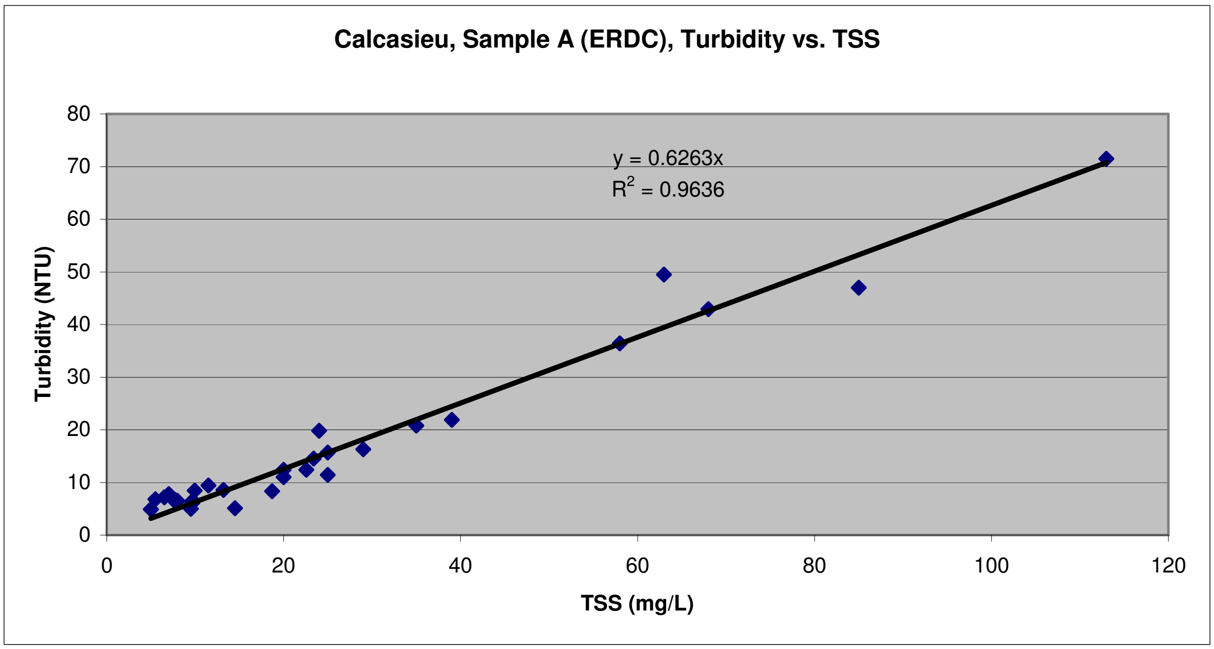
TSS Concentrations and Turbidity Measurements Sample A (Environmental Lab)							
Time, hr	Port No.	TSS mg/L	Turbidity NTU	Time, hr	Port No.	TSS mg/L	Turbidity NTU
3.5	6	118	71.5	96	5.5	7.75	6.71
3.5	6	108	71.5	96	5	9.7	6.55
3.5	6	113	71.5	96	4.5	9.7	6.17
5	6	68	42.9	96	4	7.5	6.69
7	6	58	36.4	169	5.5	7	7.79
7	5.5	63	49.5	169	5	5.5	6.8
12.5	6	23.4	14.5	169	4.5	6.5	7.17
12.5	5.5	39	21.9	169	4	11.5	9.45
12.5	5	24	19.8	240.25	5.5	8	6.49
24	6	29	16.3	240.25	5	5	4.93
24	5.5	25	15.7	240.25	4.5	9.5	4.98
24	5	35	20.8	240.25	4	5	4.89
24	4.5	85	47	240.25	3.5	14.5	5.12
48	5.5	22.5	12.4	361.5	5	2.6	2.6
48	5	20	12.4	361.5	4.5	4.4	2.6
48	4.5	25	11.4	361.5	4	4	2.6
73	5.5	9.9	4.46	361.5	3.5	5.8	3.7
73	5	13.2	8.56				
73	4.5	18.7	8.31				
73	4	20	11				

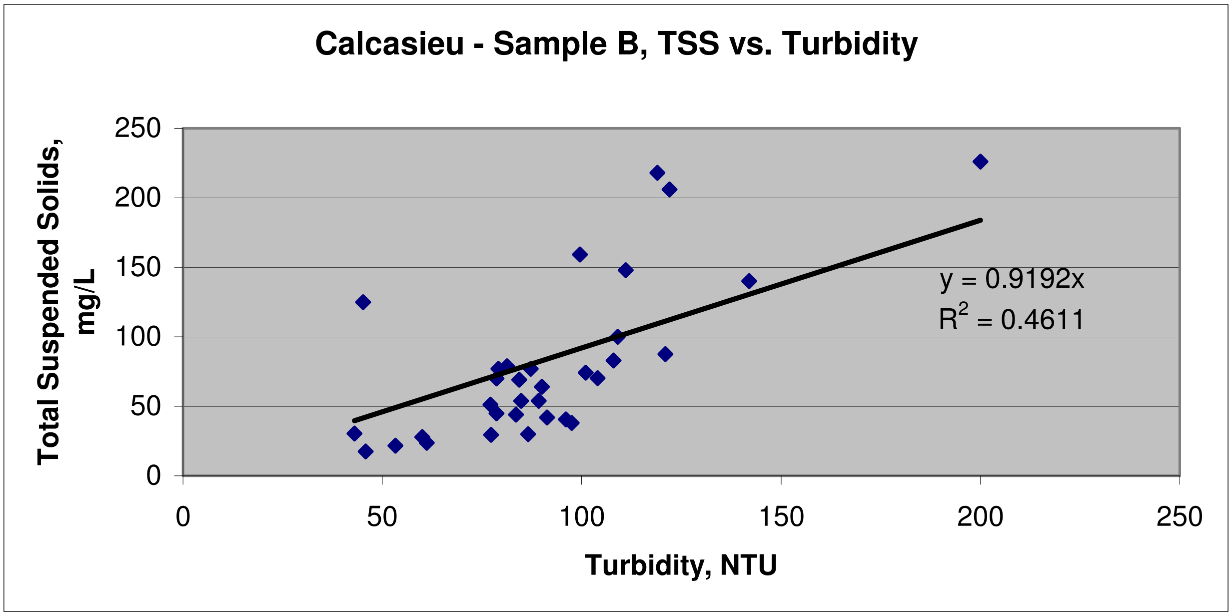
TSS Concentrations and Turbidity Measurements Sample B (Eustis)							
Time, hr	Port No.	TSS mg/L	Turbidity NTU	Time, hr	Port No.	TSS mg/L	Turbidity NTU
7	6	226	200	168	5	40.6	96
12	6	78.7	81.3	168	4.5	70.3	104
12	5.5	70	78.6	168	4	140	142
12	5	206	122	264	5.5	23.7	61.1
24	6	74.3	101	264	5	29.5	77.3
24	5.5	77	87.2	264	4.5	27.8	60
24	5	83	108	264	4	76.9	79.1
24	4.5	100	109	264	3.5	217	119
48	5.5	44	83.5	360	5.5	17.4	45.8
48	5	64	90	360	5	21.7	53.3
48	4.5	69.2	84.3	360	4.5	30.4	43
72	5.5	45	78.6	360	4	124	45.2
72	5	54	89.2	360	3.5	159	99.6
72	4.5	51.1	77.1				
72	4	148	111				
96	5.5	38	97.5				
96	5	54	84.8				
96	4.5	41.9	91.3				
96	4	87.6	121				
168	5.5	30	86.6				

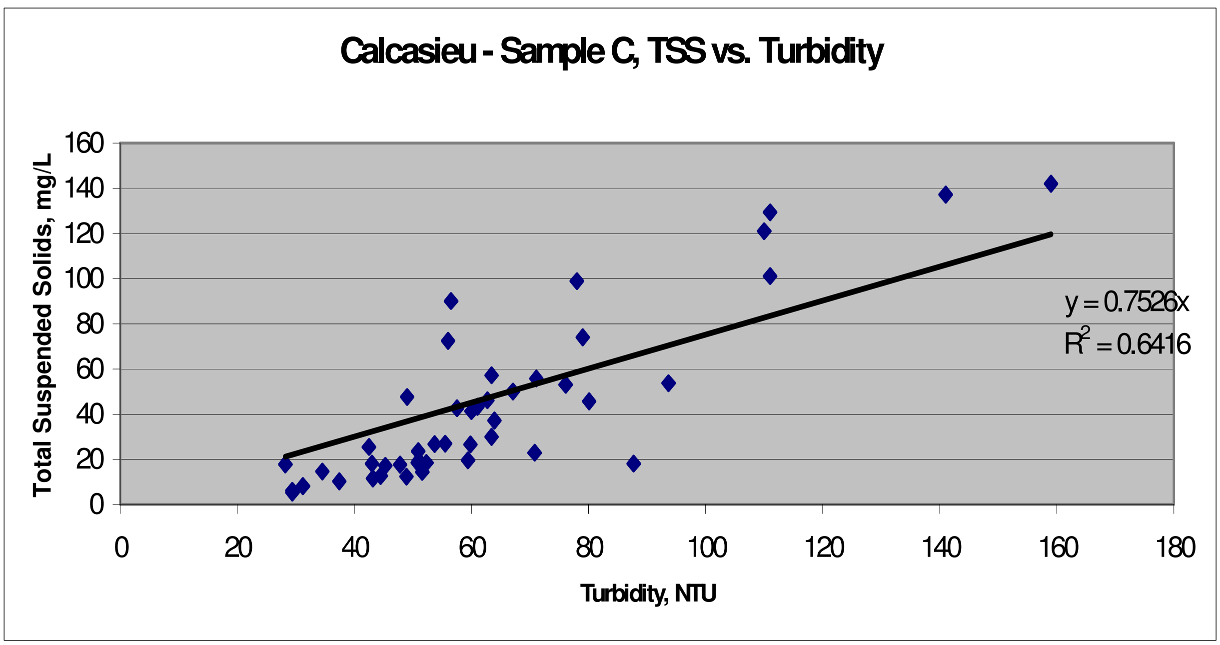
TSS Concentrations and Turbidity Measurements Sample C (Eustis)							
Time, hr	Port No.	TSS mg/L	Turbidity NTU	Time, hr	Port No.	TSS mg/L	Turbidity NTU
4	6	129	111	96	5.5	14.4	51.6
7	6	98.9	78	96	5	26.9	55.5
7	5.5	55.8	71.1	96	4.5	23.5	50.9
12	6	57.3	63.4	96	4	72.5	56
12	5.5	42.7	57.5	96	3.5	18.5	52.3
12	5	120	110	168	5.5	12.3	48.9
24	6	12.6	44.5	168	5	18.4	50.8
24	5.5	30	63.4	168	4.5	19.4	51
24	5	45.6	80.1	168	4	18.2	87.7
24	4.5	101	111	168	3.5	17.6	47.8
24	4	53	76.1	168	3	142	159
48	6	47.6	49	264	5.5	10.2	37.4
48	5.5	90.1	56.5	264	5	18	43
48	5	43.3	61	264	4.5	17.1	45.3
48	4.5	50	67.1	264	4	14.6	34.5
48	4	137	141	264	3.5	11.6	43.2
48	3.5	53.6	93.7	264	3	26.7	53.7
72	6	22.9	70.8	360	5.5	5.3	29.4
72	5.5	26.6	59.8	360	5	8.2	31.2
72	5	37.2	63.9	360	4.5	25.6	42.5
72	4.5	41.3	60	360	4	17.8	28.2
72	4	46.1	62.7	360	3.5	6.1	29.4
72	3.5	19.5	59.4	360	3	74	79

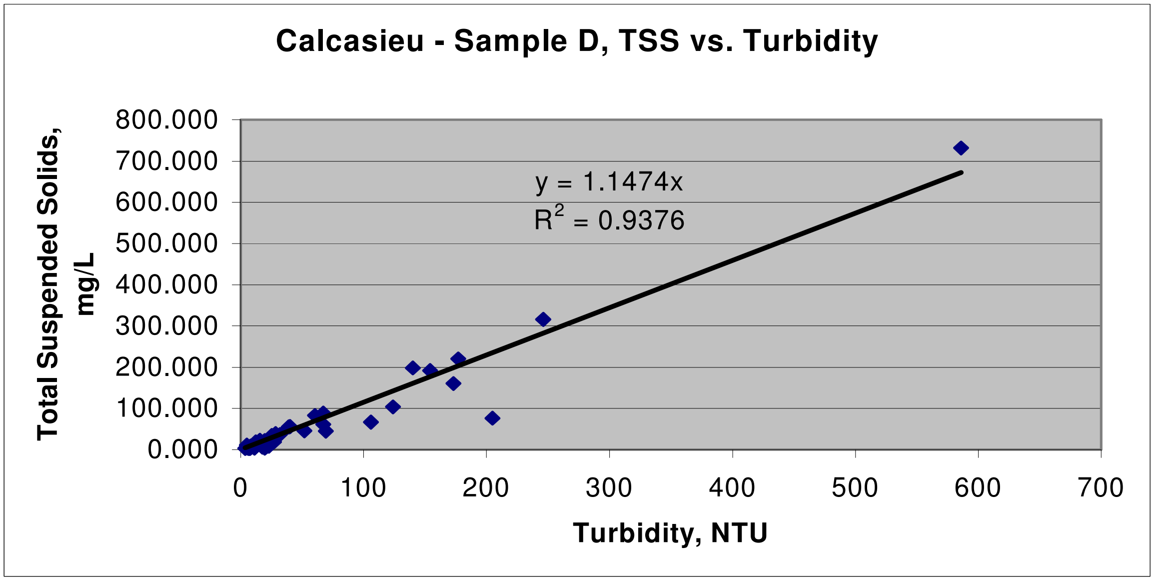
TSS Concentrations and Turbidity Measurements Sample D (Eustis)							
Time, hr	Port No.	TSS mg/L	Turbidity NTU	Time, hr	Port No.	TSS mg/L	Turbidity NTU
4	6	198	140	96	4.5	2.7	7.4
7	6	8.2	10.6	96	4	22.6	15.7
7	5.5	39.4	28.4	96	3.5	9.9	20
12	6	21.7	19.5	96	3	61	67.2
12	5.5	34.5	25.2	96	2.5	104	124
12	5	36.6	27.7	168	5	2.7	3.6
24	6	12.1	12.8	168	4.5	5.3	5.4
24	5.5	15.7	12.3	168	4	10.9	5
24	5	26.9	23.9	168	3.5	8.6	7.9
24	4.5	55.4	39	168	3	45.7	51.8
24	4	82.7	60.5	168	2.5	66.8	106
24	3.5	88.6	67.2	264	5	4.8	6.1
48	5.5	18.6	12.2	264	4.5	6.9	8.7
48	5	3.5	5.7	264	4	2.7	6.4
48	4.5	20.9	15	264	3.5	5.9	8.9
48	4	13	15.7	264	3	19.4	26.3
48	3.5	36.9	32.1	264	2.5	160	173
48	3	192	154	264	2	732	586
72	5.5	4.4	7	360	5	3.4	19.5
72	5	11.7	12.8	360	4.5	5.4	17.8
72	4.5	16.4	16.7	360	4	8.5	22.8
72	4	56	40.1	360	3.5	6.1	20.9
72	3.5	15.4	22.8	360	3	18.9	27
72	3	220	177	360	2.5	45.2	69.2
72	2.5	316	246	360	2	76.4	205
96	5.5	3.6	11.3				
96	5	11.6	11.9				


Calcasieu - Sample A (Eustis), TSS vs. Turbidity
Total Suspended Solids, mg/L
Turbidity, NTU
y = 0.6166x
R2 = 0.4719
0
20
40
60
80
100
120
140
160
180
0
50
100
150
200
250
300


Calcasieu, Sample A (ERDC), Turbidity vs. TSS
Turbidity (NTU)
TSS (mg/L)
y = 0.6263x
R2 = 0.9636
0
10
20
30
40
50
60
70
80
0
20
40
60
80
100
120


Calcasieu - Sample B, TSS vs. Turbidity
Total Suspended Solids, mg/L
250
200
150
100
50
0
0
50
100
150
200
250
Turbidity, NTU
y = 0.9192x
R^2 = 0.4611


Calcasieu - Sample C, TSS vs. Turbidity
Total Suspended Solids, mg/L
160
140
120
100
80
60
40
20
0
0
20
40
60
80
100
120
140
160
180
Turbidity, NTU
y = 0.7526x
R^2 = 0.6416


Calcasieu - Sample D, TSS vs. Turbidity
y = 1.1474x
R2 = 0.9376
Total Suspended Solids, mg/L
800.000
700.000
600.000
500.000
400.000
300.000
200.000
100.000
0.000
0
100
200
300
400
500
600
700
Turbidity, NTU

APPENDIX E
LIDAR SURVEY SEPTEMBER 2002

Calcasieu River CDF Capacities Based Upon Lidar Survey

Volume at Contour (CY)

Disp. Area	Total Acr.	App. Avg Dike Elev. (Ft)	Add. Dike Lift (Ft)	App. Disp Dike Elev. (Ft)	-2	2	4	6	8	10	12	14	16	18	20	Useable Vol. Cap. (CY)	Cap W/2:1 Bulk Fact.	Pay+Ovd Cap.
1	50	14	0	14	0	0	0	0	251,681	96,800	19,360	0				348,481		
2	45	16	0	16	0	0	0	0	0	290,401	32,267	24,200	0			322,668		
3 (Clooney Island)	112	14	2	16	0	0	0	0	238,774	629,202	183,921	0				1,051,897		
4	112	12	4	16	0	0	0	0	251,681	459,802	290,401					1,001,884		
5	30.5	14	2	16	0	0	58,080	242,001	70,987	67,760	0	0				438,828		
6	39	4	OUT	OUT												0		
7	255	16(front)-12(back)	2	14	0	0	0	0	1,422,966	245,228	80,667	0	-80,667			1,668,193		
8	188	12	2	14	0	0	0	0	1,819,847	0	0					1,819,847		
9	169	12(front)-8(back)	2	10	0	0	0	0	504,975	0	0					0		
10	127	10(front)-8(back)	2	10	0	0	0	819,577	0	0						819,577		
11	135	8	2	10	0	0	0	790,536	40,333							790,536		
12A	160	8	2	10	0	0	0	1,032,537	0							1,032,537		
12B	430	16(Flare)-12(North End)	2	14	0	0	0	0	759,883	1,087,391	345,255	0	-41,947			1,847,274	3,670,348	1,835,174
13 (Choupique Island)	700	16(front)-10(back)	2	12	0	0	0	0	1,516,539	645,336	0	-242,001	-225,868			1,516,539		
15	180	12	4	16	0	0	0	0	529,175	1,345,525	0					1,874,701		
16N	115	12	2	14	0	0	0	0	0	0	371,068					0		
16S	40	20	OUT	OUT												0		
17*	200	8	2	10	0	0	0	0	371,068	0						0		
22	135	14	2	16	0	0	0	0	0	0	580,802	145,201				580,802		
23	115	16	0	16	116,160	0	0	0	0	0	96,800	177,467	0	-48,400	-64,534	212,961		
D	250	16	0	16	0	0	0	0	0	726,003	484,002	177,467	0			1,210,005	5,395,008	2,697,504
E	150	12	4	16	0	0	0	0	451,735	1,113,204	0					1,564,939	6,959,947	3,479,974
H	140	10	2	12	0	0	0	0	658,243	93,574	0					658,243		
M	390	6	4	10	0	0	2,420,009	419,468	242,001							2,839,478		
N	215	8(front)-6(back)	4	10	0	0	0	1,290,672	48,400							1,290,672	4,788,392	2,394,196
																	18,101,671	9,050,835

* Large spoils mounds with max elevations at +22'. Spoil mounds are approx. 63 acres.

Calcasieu River CDF Capacities Based Upon Lidar Survey
Volume at Contour (CY)

Disposal Area	Add. Dike Lift (Ft)	Appr. Disp. Dike Elev. (Ft)	-2	2	4	6	8	10	12	14	16	18	20	Useable Vol. Cap. (CY)	Cap. W/2:1 Bulk Fact.	Pay+Ovd Cap.
1	0	14	0	0	0	0	41,947	108,094	141,974	156,494	161,334	161,334	161,334	150,041		
2	0	16	0	0	0	0	0	48,400	104,867	125,034	141,167	145,201	145,201	153,267		
3 (Clooney Island)	2	16	0	0	0	0	29,847	164,561	315,408	361,388	361,388	361,388	361,388	509,815		
4	4	16	0	0	0	0	31,460	139,554	288,788	361,388	361,388	361,388	361,388	459,802		
5	2	16	0	0	4,840	33,880	66,954	87,120	98,414	98,414	98,414	98,414	98,414	291,208		
6	OUT	OUT		124,227	125,034	125,840	125,840	125,840	125,840	125,840	125,840	125,840	125,840	out		
7	2	14	0	0	0	0	237,161	535,629	637,269	709,869	782,470	822,803	822,803	772,790		
8	2	14	0	0	0	0	303,308	606,616	606,616	606,616	606,616	606,616	606,616	909,924		
9	2	10	0	0	0	0	252,488	525,142	545,309	545,309	545,309	545,309	545,309	0		
10	2	10	0	0	0	204,894	409,788	409,788	409,788	409,788	409,788	409,788	409,788	204,894		
11	2	10	0	0	0	197,634	415,435	435,602	435,602	435,602	435,602	435,602	435,602	197,634		
12A	2	10	0	0	0	258,134	516,269	516,269	516,269	516,269	516,269	516,269	516,269	258,134		
12B	2	14	0	0	0	0	126,647	525,142	969,617	1,243,885	1,366,499	1,387,472	1,387,472	651,789	1,107,558	553,779
13 (Choupique Island)	2	12	0	0	0	0	379,135	1,080,938	1,653,673	2,024,741	2,202,209	2,258,675	2,258,675	379,135		
15	4	16	0	0	0	0	66,147	356,548	580,802	580,802	580,802	580,802	580,802	1,003,497		
16N	2	14	0	0	0	0	0	0	185,534	371,068	371,068	371,068	371,068	0		
16S	OUT	OUT		0	0	0	0	0	0	0	32,267	96,800	129,067	out		
17*	2	10	0	0	0	0	185,534	406,562	442,055	442,055	442,055	442,055	442,055	0		
22	2	16	0	0	0	0	0	0	145,201	363,001	435,602	435,602	435,602	145,201		
23	0	16	0	0	0	0	0	0	24,200	137,134	258,134	314,601	354,935	24,200		
D	0	16	0	0	0	0	0	121,000	363,001	572,736	734,070	806,670	806,670	484,002	2,036,035	1,018,017
E	4	16	0	0	0	0	56,467	298,468	484,002	484,002	484,002	484,002	484,002	838,937	2,874,971	1,437,486
H	2	12	0	0	0	0	164,561	375,908	437,215	451,735	451,735	451,735	451,735	164,561		
M	4	10	0	0	403,335	911,537	1,137,404	1,258,405	1,258,405	1,258,405	1,258,405	1,258,405	1,258,405	1,314,872		
N	4	10	0	0	0	322,668	669,536	693,736	693,736	693,736	693,736	693,736	693,736	322,668	1,802,100	901,050
														9,236,369	4,618,185	

* Large spoils mounds with max elevations at +22'. Spoil mounds are approx. 63 acres.

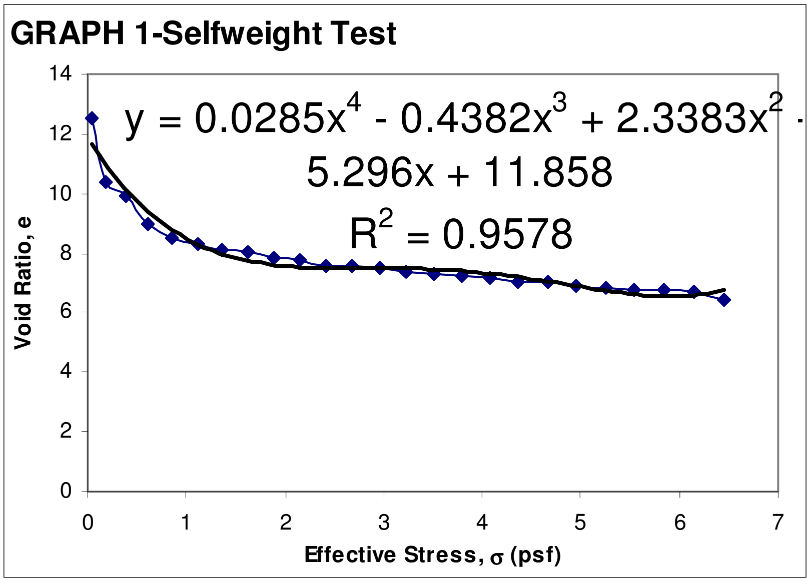
Disp. Area	Total Acr.	App. Avg. Dike Elev. (Ft)	Vol. at 10' (if dikes raised)	10' Dike Cap.(12' dk, 2' fb)	Vol. at 10' (current, w/ 2' fb)	Current Cap.	In Situ Dredge Vol	Cap. Needed (30" dredge)
1	50	14	150,041		150,041			
2	45	16	48,400		48,400			
3 (Clooney Island)	112	14	194,407		194,407			
4	112	12	171,014		171,014			
5	30.5	14	192,794		192,794			
6	39	4	out		out			
7	255	16(front)-12(back)	772,790		772,790			
8	188	12	909,924		909,924			
9	169	12(front)-8(back)	777,630		0	current dikes below 12'		
10	127	10(front)-8(back)	1,024,471		204,894			
11	135	8	1,048,671		197,634			
12A	160	8	1,290,672		258,134			
12B	430	16(Flare)-12(North End)	651,789	7,232,602	651,789	3,751,821	6,500,000	7,666,138
13 (Choupique Island)	700	16(front)-10(back)	1,460,072		379,135			
15	180	12	422,695		422,695			
16N	115	12	0		0			
16S	40	20	out		out			
17*	200	8	592,096		0			
22	135	14	0		0			
23	115	16	0		0			
D	250	16	121,000		121,000			
E	150	12	354,935	2,950,798	354,935	1,277,765	4,500,000	5,307,326
H	140	10	540,469		164,561			
M	390	6	3,710,681		403,335			
N	215	8(front)-6(back)	1,685,940	5,937,090	0	567,896	4,000,000	4,717,623
Total			16,120,490	16,120,490	5,597,482	5,597,482	15,000,000	17,691,088

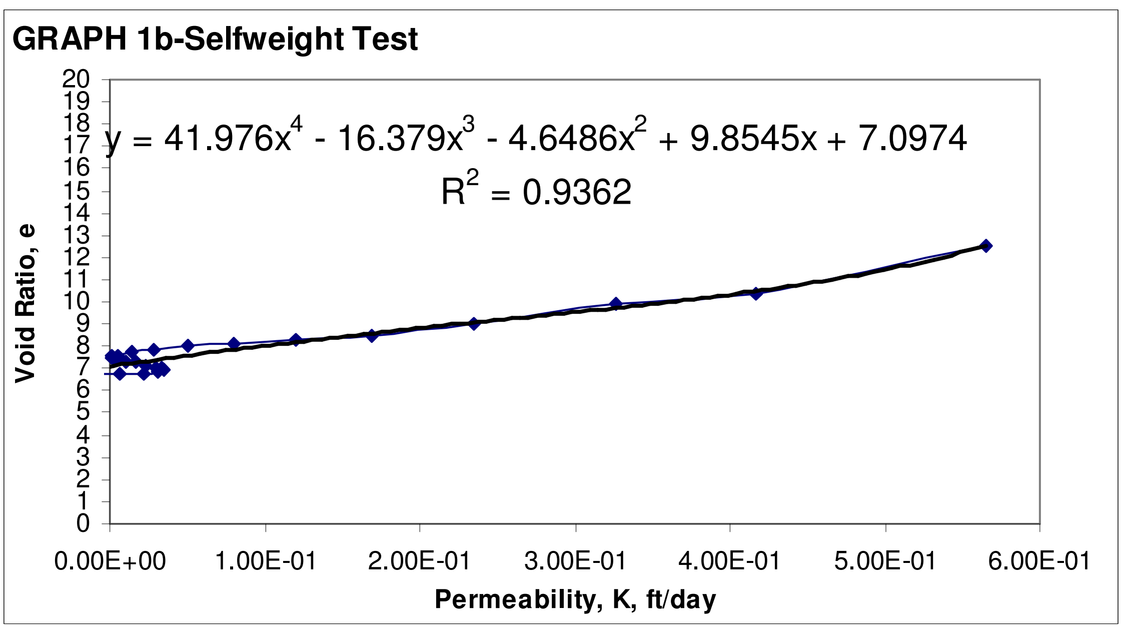
*Estimated based on ratio of upper reach
I am currently revising these numbers.

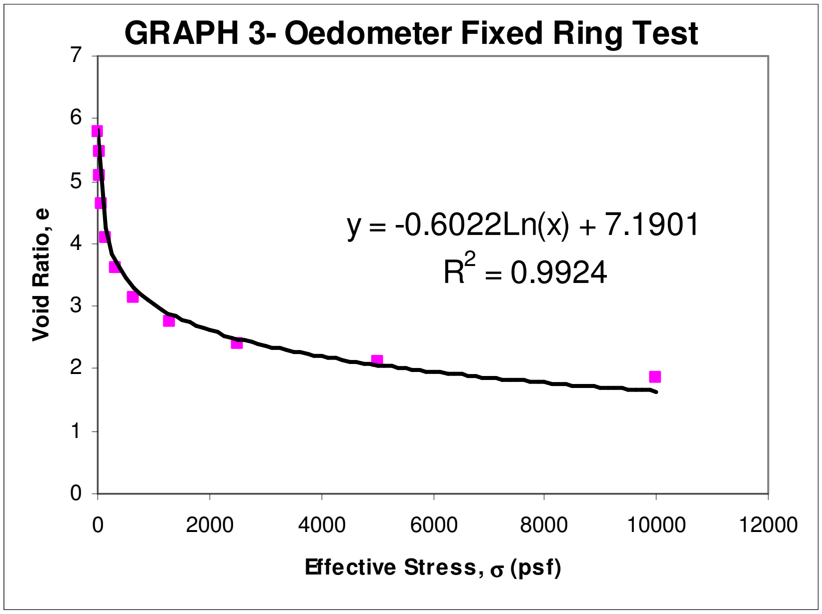
Appendix B
Consolidation Tests performed by ERDC-GSL

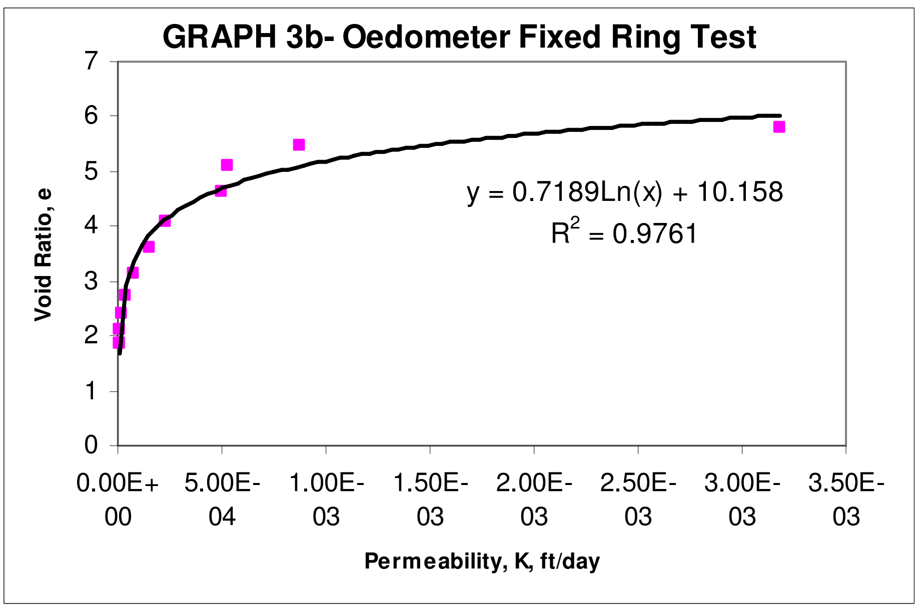
Self Weight and Oedometer Fixed Ring Consolidation Curves

SITE A

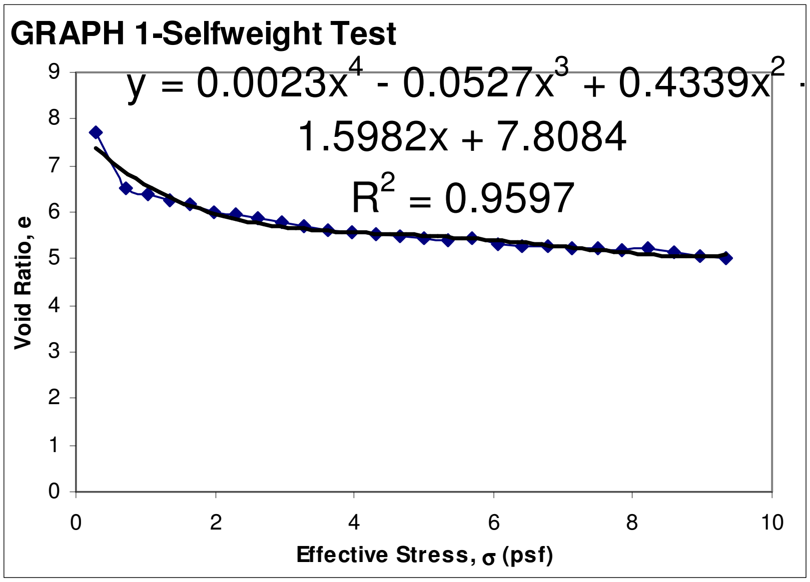




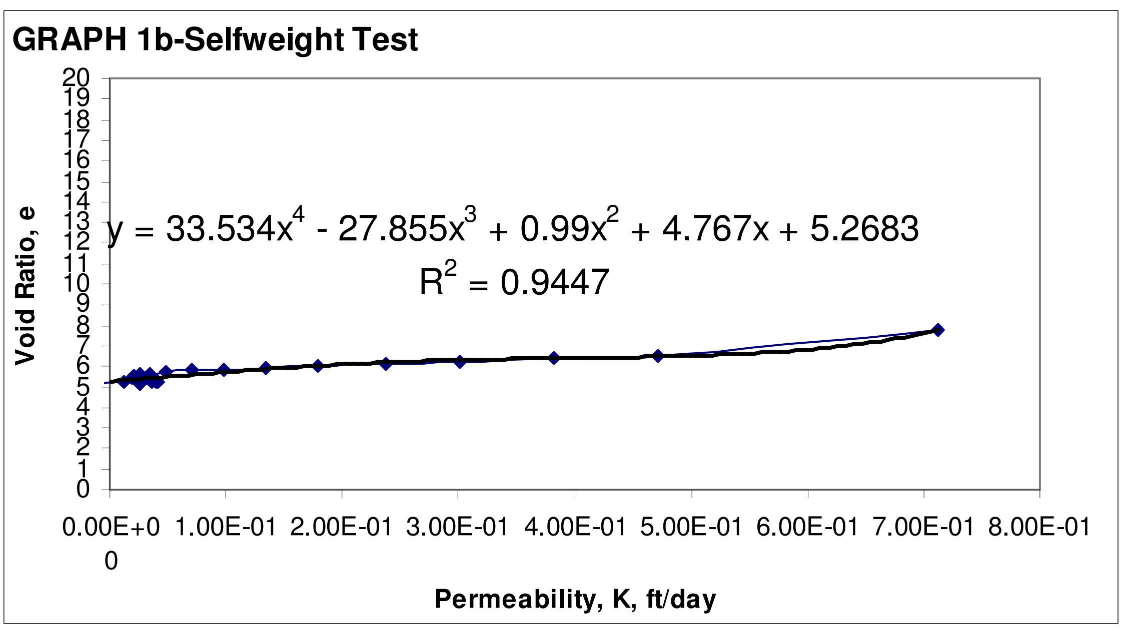

GRAPH 3- Oedometer Fixed Ring Test
y = -0.6022Ln(x) + 7.1901
R^2 = 0.9924
Void Ratio, e
Effective Stress, σ (psf)
0
1
2
3
4
5
6
7
0
2000
4000
6000
8000
10000
12000


GRAPH 3b- Oedometer Fixed Ring Test
y = 0.7189Ln(x) + 10.158
R^2 = 0.9761
Void Ratio, e
Permeability, K, ft/day
0
1
2
3
4
5
6
7
0.00E+00
5.00E-04
1.00E-03
1.50E-03
2.00E-03
2.50E-03
3.00E-03
3.50E-03

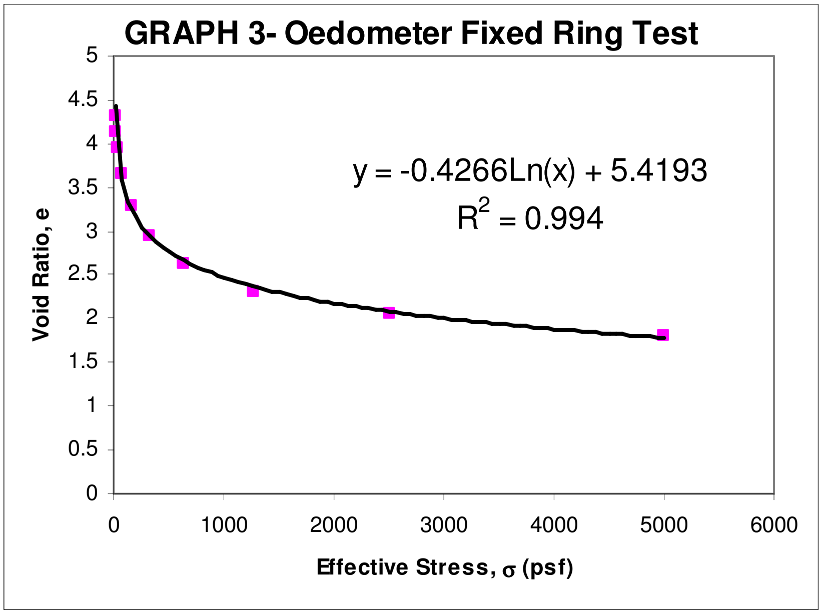
SITE B

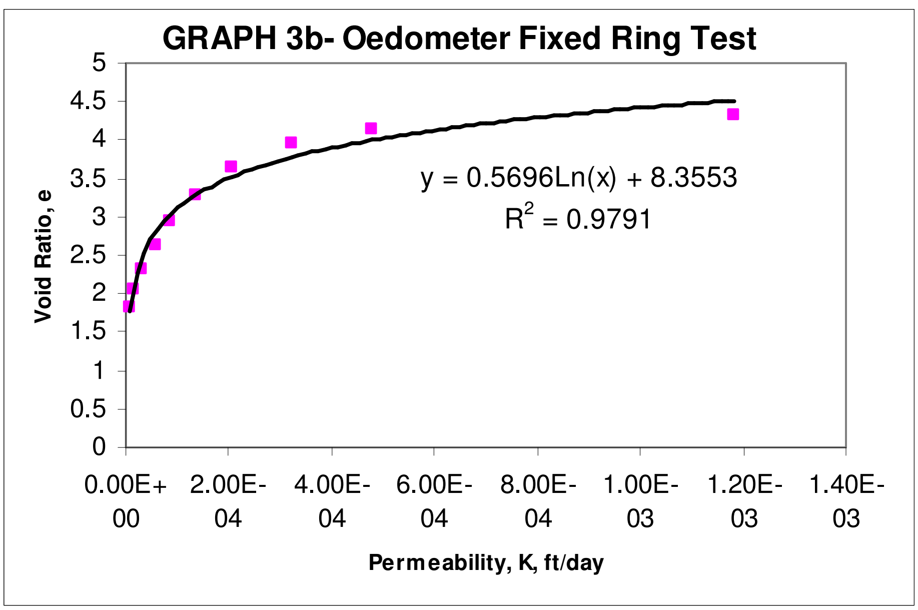


GRAPH 1-Selfweight Test
y = 0.0023x4 - 0.0527x3 + 0.4339x2 - 1.5982x + 7.8084
R2 = 0.9597
Void Ratio, e
Effective Stress, σ (psf)
0 1 2 3 4 5 6 7 8 9
0 2 4 6 8 10

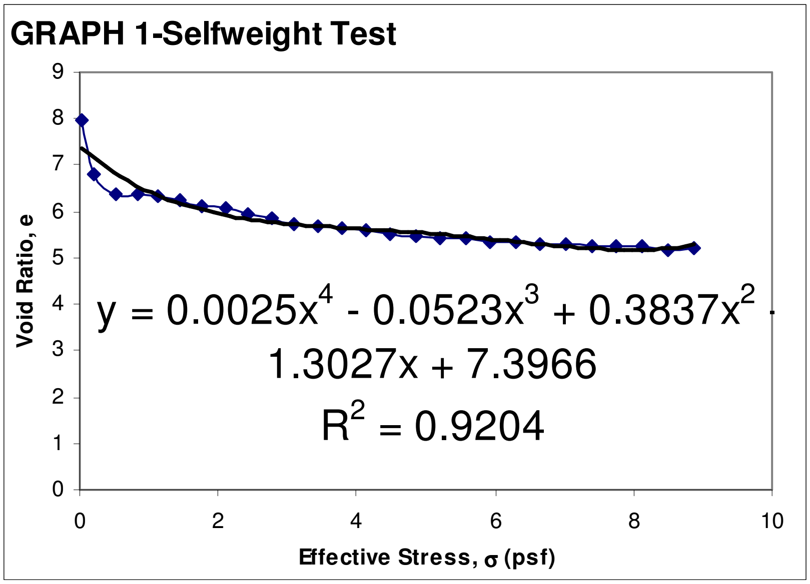


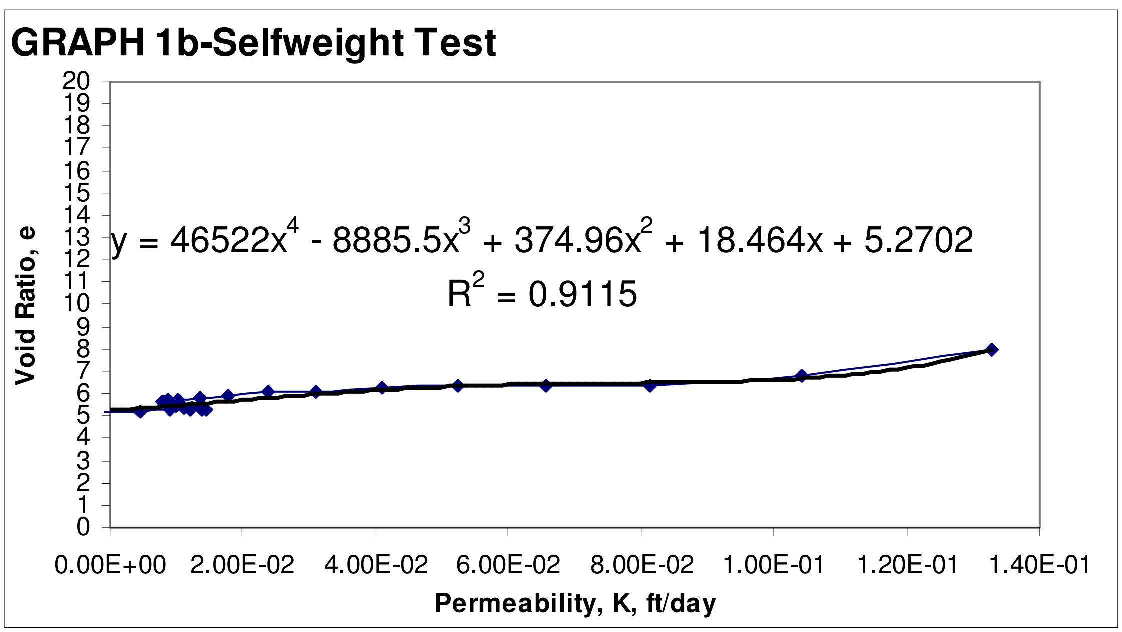
GRAPH 1b-Selfweight Test
y = 33.534x4 - 27.855x3 + 0.99x2 + 4.767x + 5.2683
R2 = 0.9447
Void Ratio, e
Permeability, K, ft/day
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20
0.00E+00 1.00E-01 2.00E-01 3.00E-01 4.00E-01 5.00E-01 6.00E-01 7.00E-01 8.00E-01

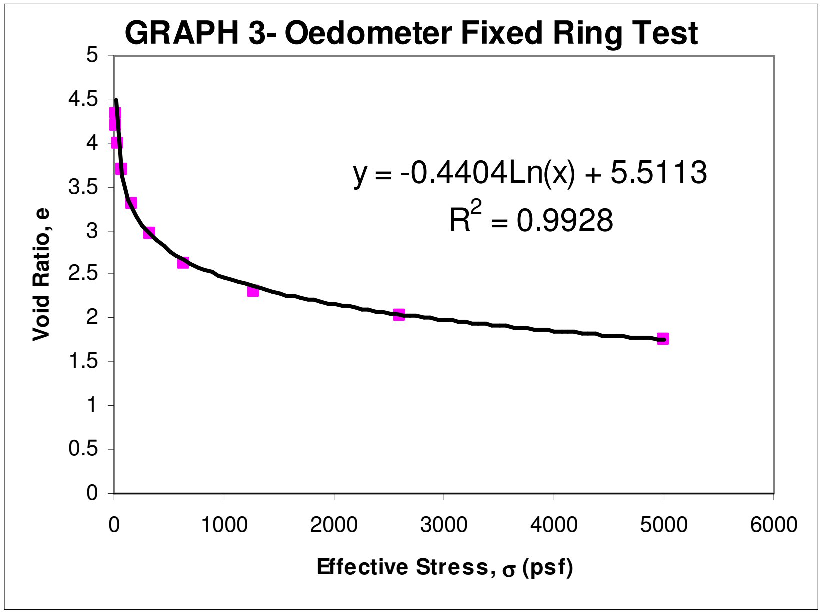

GRAPH 3- Oedometer Fixed Ring Test
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
Void Ratio, e
y = -0.4266Ln(x) + 5.4193
R^2 = 0.994
0
1000
2000
3000
4000
5000
6000
Effective Stress, σ (psf)

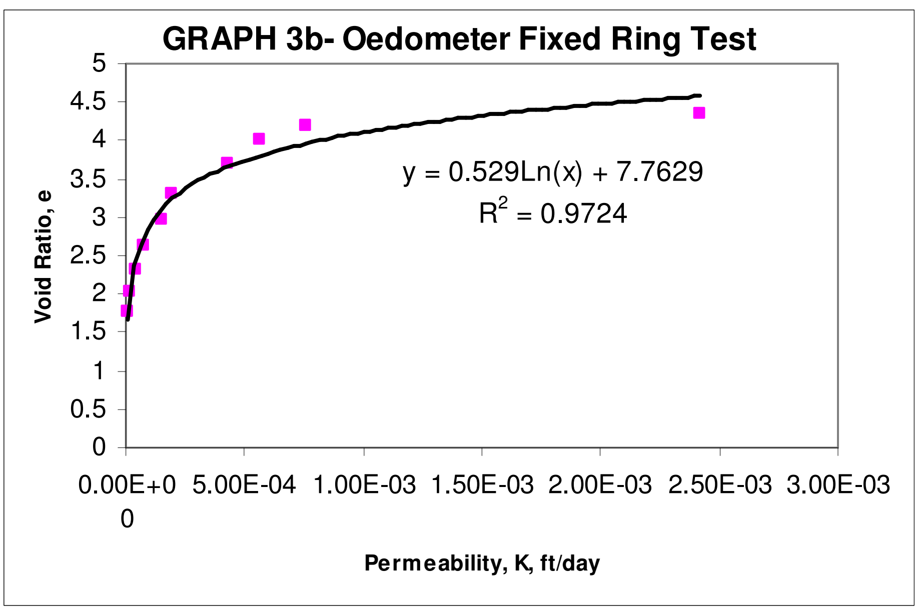

GRAPH 3b- Oedometer Fixed Ring Test
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
Void Ratio, e
y = 0.5696Ln(x) + 8.3553
R^2 = 0.9791
0.00E+00
2.00E-04
4.00E-04
6.00E-04
8.00E-04
1.00E-03
1.20E-03
1.40E-03
Permeability, K, ft/day

SITE C

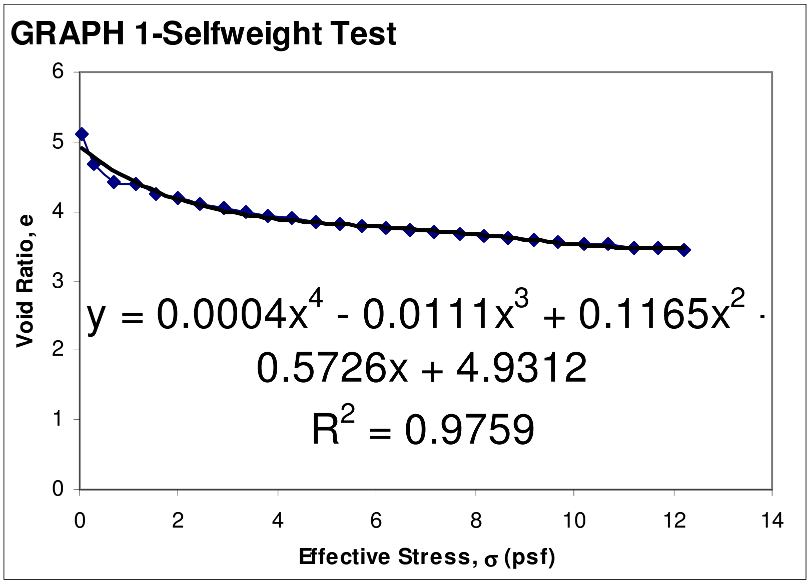

GRAPH 1-Selfweight Test
Void Ratio, e
Effective Stress, σ (psf)
y = 0.0025x4 - 0.0523x3 + 0.3837x2 -
1.3027x + 7.3966
R2 = 0.9204
0
1
2
3
4
5
6
7
8
9
0
2
4
6
8
10

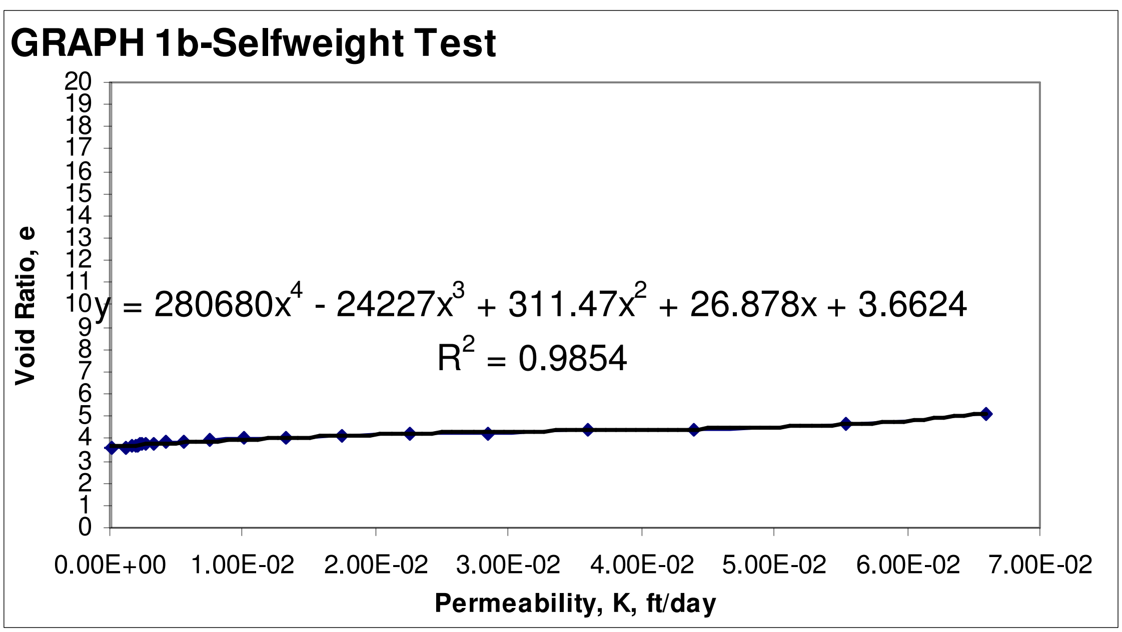

GRAPH 1b-Selfweight Test
Void Ratio, e
Permeability, K, ft/day
y = 46522x4 - 8885.5x3 + 374.96x2 + 18.464x + 5.2702
R2 = 0.9115
0.00E+00
2.00E-02
4.00E-02
6.00E-02
8.00E-02
1.00E-01
1.20E-01
1.40E-01

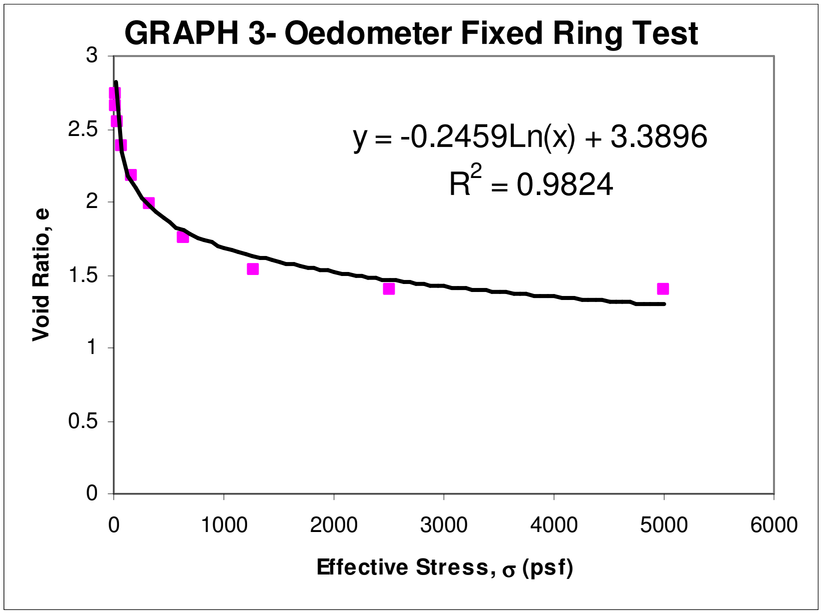

GRAPH 3- Oedometer Fixed Ring Test
y = -0.4404Ln(x) + 5.5113
R2 = 0.9928
Void Ratio, e
Effective Stress, σ (psf)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
0
1000
2000
3000
4000
5000
6000

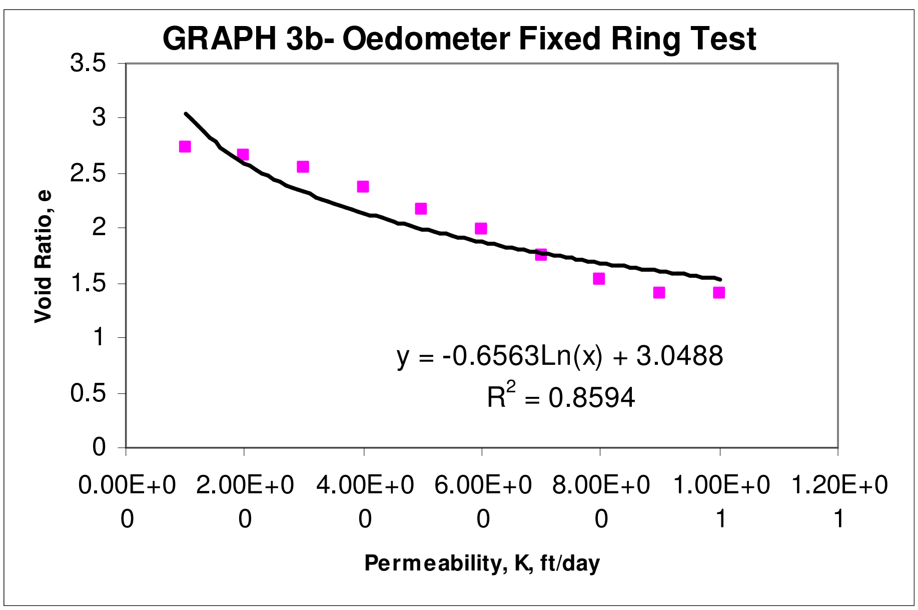

GRAPH 3b- Oedometer Fixed Ring Test
y = 0.529Ln(x) + 7.7629
R2 = 0.9724
Void Ratio, e
Permeability, K, ft/day
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
0.00E+00
5.00E-04
1.00E-03
1.50E-03
2.00E-03
2.50E-03
3.00E-03

SITE D

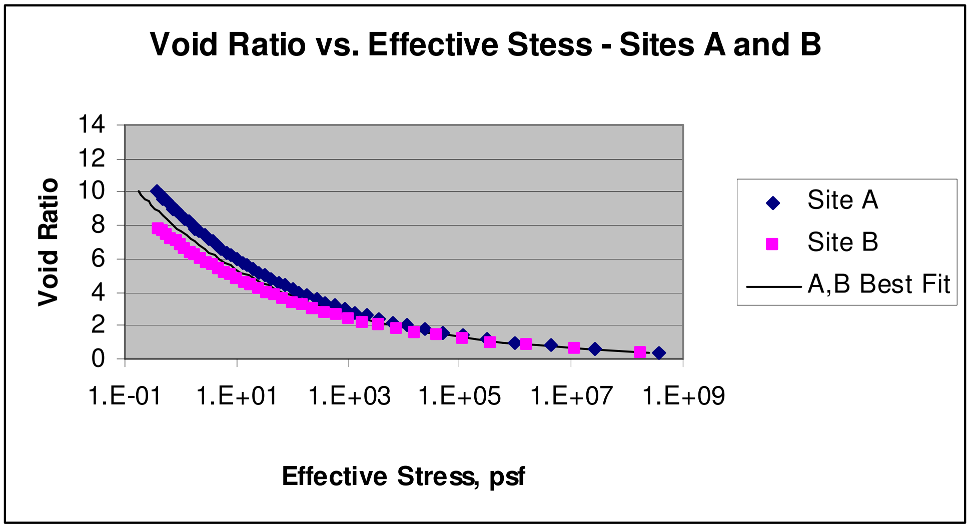

GRAPH 1-Selfweight Test
Void Ratio, e
0
1
2
3
4
5
6
0
2
4
6
8
10
12
14
Effective Stress, σ (psf)
y = 0.0004x4 - 0.0111x3 + 0.1165x2 -
0.5726x + 4.9312
R2 = 0.9759

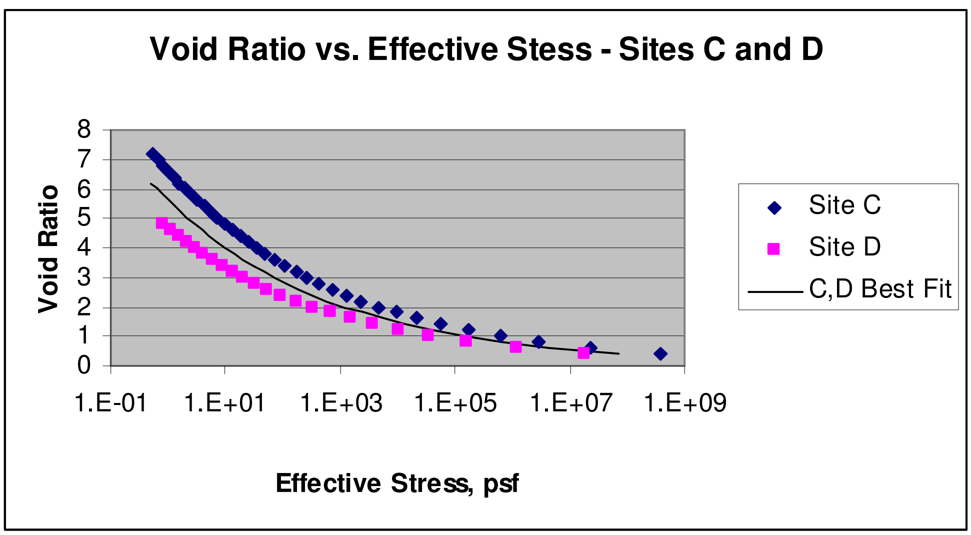

GRAPH 1b-Selfweight Test
Void Ratio, e
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
0.00E+00
1.00E-02
2.00E-02
3.00E-02
4.00E-02
5.00E-02
6.00E-02
7.00E-02
Permeability, K, ft/day
y = 280680x4 - 24227x3 + 311.47x2 + 26.878x + 3.6624
R2 = 0.9854


GRAPH 3- Oedometer Fixed Ring Test
y = -0.2459Ln(x) + 3.3896
R2 = 0.9824
Void Ratio, e
3
2.5
2
1.5
1
0.5
0
0
1000
2000
3000
4000
5000
6000
Effective Stress, σ (psf)


GRAPH 3b- Oedometer Fixed Ring Test
y = -0.6563Ln(x) + 3.0488
R2 = 0.8594
Void Ratio, e
3.5
3
2.5
2
1.5
1
0.5
0
0.00E+00
2.00E+00
4.00E+00
6.00E+00
8.00E+00
1.00E+01
1.20E+01
Permeability, K, ft/day

Resulting Void Ratio vs. Effective Stress Curves





Resulting Void Ratio vs. Permeability Curves