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STABILIZATION OF COASTAL SUBAERIAL
DREDGED MATERIAL SITES IN
NORTH CAROLINA

By

M. Siipola[1], J. Machemehl[2] and V. Cavaroc[3]

Abstract

The factors governing the evolution of subaerially deposited, dredging material sites on the Cape Fear River below Wilmington, N. C. were studied. These sites were selected because they offered a variety in site age, shape and materials.

Profiles were run from the crest (i.e., point of discharge of material) to the toe of the site. Along these profiles, paired samples were taken at 20-foot intervals. A sample pair consisted of a carefully collected sample of the pile surface and a sample collected at a 5.0 cm. depth. Samples were also collected between 0.3 m and 0.9 m at one or two selected locations on the site.

Each sample was sieved at half-phi (ϕ) intervals and the mean grain size determined. The presence of an inherent dredge effect and distinctly coarser surface veneers on the sites were established. From the characteristics of the surface veneers and alterations of the dredge effect in the samples collected, it was possible to determine the effects of erosion processes (wind and rain)

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on the evolution of the subaerial dredged material sites. The dominant factors controlling these erosive processes were also established.

INTRODUCTION

The effects of dredged material disposal upon the environment have been studied by numerous investigators[5,16]. Studies in recent years have been conducted to determine plant and animal succession on disposal sites[16]. Investigators have found that disposal sites are eventually stabilized by a definite sequence of plants.

This study involves the physical factors affecting dredged material disposal sites prior to vegetation. The objectives of the study were (1) to evaluate the factors controlling evolution of a disposal site and (2) to evaluate the extent of environmental modification of several typical disposal sites in North Carolina.

Dredged Material Disposal Sites

Dredged material disposal sites can be classified as subaqueous, intratidal and/or subaerial. Each is modified by the environment and each has a unique evolutionary pattern.

Subaqueous Dredged Material Sites. Subaqueous sites are covered by water and are modified by waves and currents. Bassi and Basco[2] studied a subaqueous dredged material disposal site in Galveston Bay, Texas. They found that after five months the waves and currents had moved sixty-three percent of the original dredged material out of the area.

Intratidal Dredged Material Sites. The intratidal dredged material site is a transition between a subaqueous and a subaerial site. The dredged material is deposited so that its surface lies within, or only slightly above the normal tidal range. At low tide the majority of the surface is exposed while at high tide the majority of the surface is submerged. Although partially exposed to wind and rain, the dominant processes affecting the intratidal dredged material site are wave action and tidal currents.

Two small intratidal dredged material sites (designated LWF-A and LWF-B) on the lower Lockwoods Folly River were selected for study. Locations of the sites designated LWF-A and LWF-B are shown in Figure 1. These 40-year-old dredged material sites showed the effects of wave action and currents. The reworking of the dredged material with subsequent winnowing of the sand-sized material had left a resistant intratidal zone of carbonate cobbles and shell. Ridges of shells 15 to 30 cm. thick had formed in the intratidal zone.

Subaerial Dredged Material Sites. The subaerial dredged material site is above higher high water. The site can be either on land or open water and can be either diked or undiked. Wind and rain affect the subaerial site by eroding the surface until either a coarse protective surface veneer is formed or the site is eventually stabilized by vegetation.

Four subaerial dredged material sites (designated CFR-A, CFR-B, CFR-C, and CFR-D) on the Cape Fear River and one site (designated CBI) North of Carolina Beach inlet were selected for study. Locations of the sites are shown in Figure 1. These recent dredged material sites showed the effects of environmental modification. The surfaces of two subaerial dredged material sites on the Cape Fear River are shown in Figures 2 through 5. The sites were covered with sparse vegetation as shown in Figures 2 and 3.

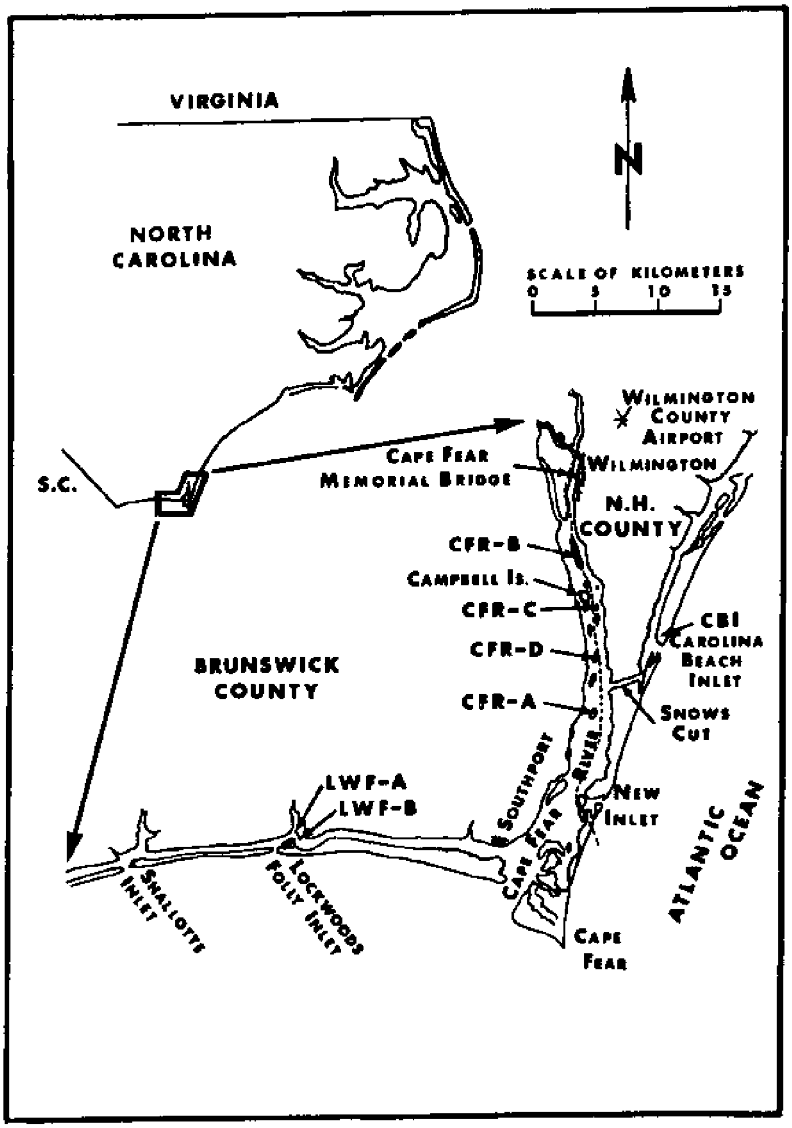


Figure 1.

Location Map of the Study Areas.

Figure 2.

Surface of Dredged Material Disposal Area Designated CFR-B. (Note the sparse covering of herbaceous vegetation (foreground) passing into dense rushes at the periphery of the disposal area.)

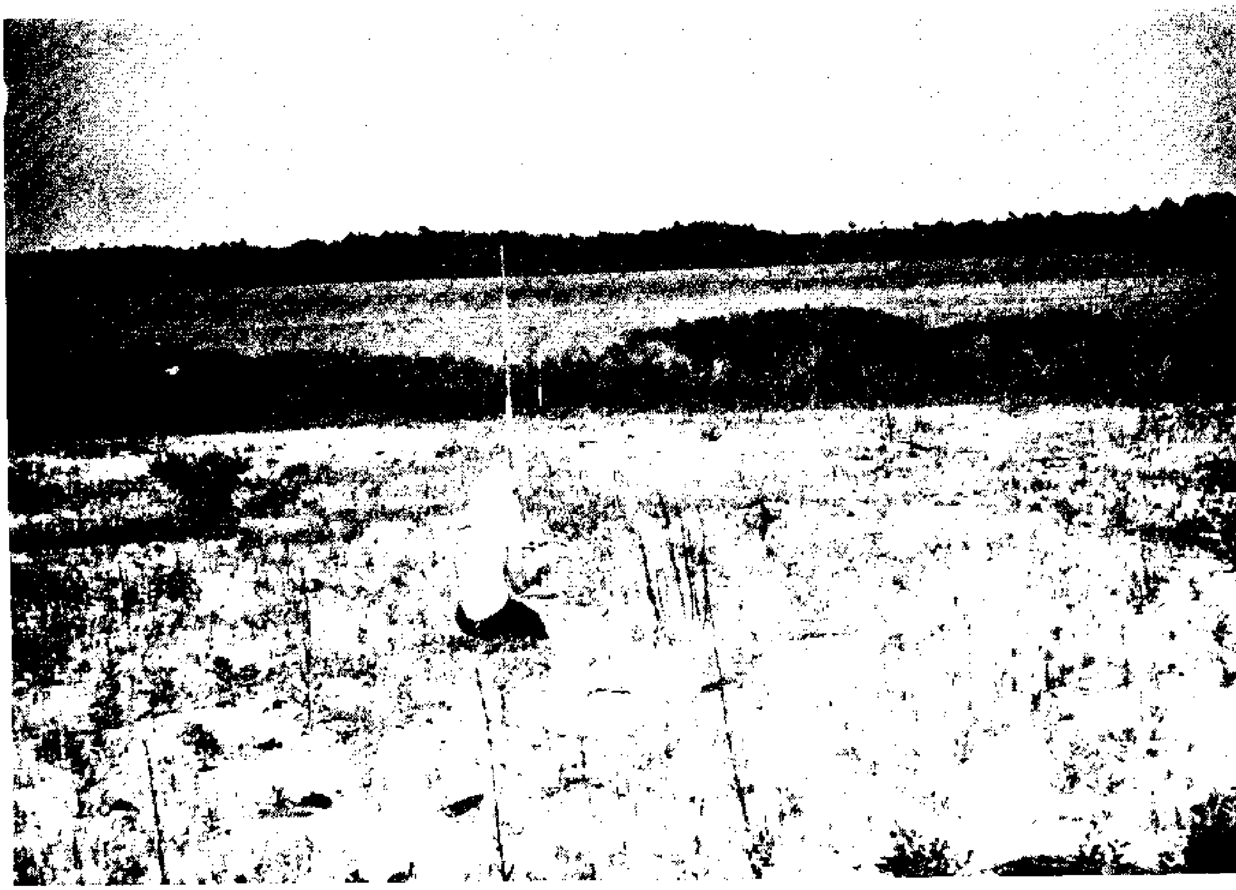


Figure 3.

Surface of Dredged Material Disposal Area Designated CFR-C. (Note the sparse covering of herbaceous vegetation (foreground) passing into dense rushes at the periphery of the disposal area.)

Figure 4.

Surface Veneer for Dredged Material Disposal Site Designated CFR-B Showing Gravel Enrichment.

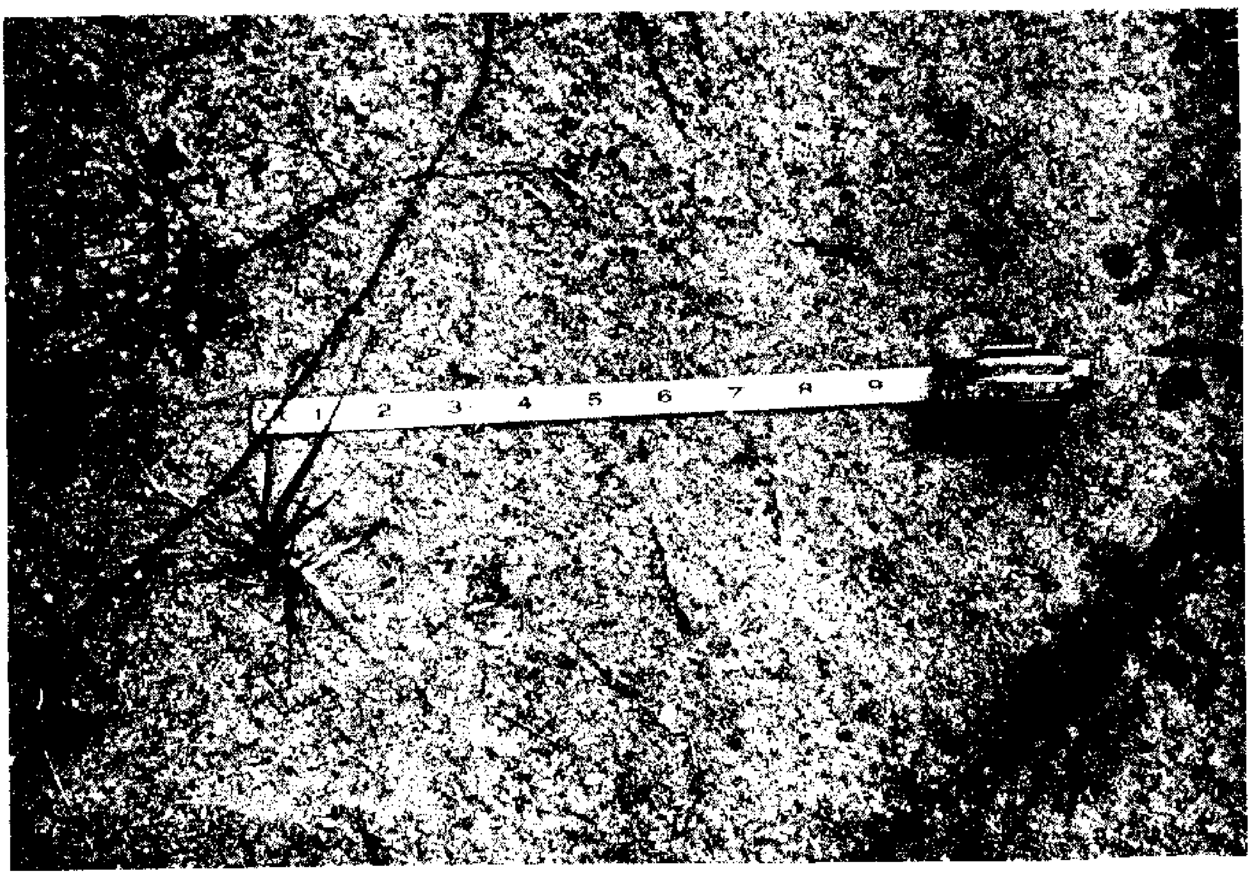


Figure 5.

Surface Veneer for Dredged Material Disposal Site Designated CFR-B Showing Coarse Sand Enrichment.

FACTORS MODIFYING DREDGED MATERIAL SITES

Factors affecting the evolution of exposed dredged material sites can be placed into three categories: environmental processes, controls (morphological and physical) and site age (site history).

Environmental Processes

Environmental processes include meteorological and oceanographical effects on dredged material sites. Wind begins to affect the dredged material as the surface is dried. Direct movement by wind can either be by suspension or saltation of particles. Large grains, immovable directly by wind, will move as a result of wind removal (deflation) of finer grains. As the finer grains are eroded away, the larger grains become undercut and roll into the depression left by the deflated grains. In time the large grains will become scattered across the surface. This type of movement is called wind creep.

Rain can also cause movement of particles either directly (raindrop impingement) or indirectly (runoff). Raindrops striking a sandy surface will transfer some of their kinetic energy to the sand grains which are, thereby, disturbed due to impact. Runoff in the form of sheet flow or channel flow will occur if the rate of precipitation is sufficient. Both raindrop impingement and runoff will move material downslope. The material will be deposited on the lower sloping flanks of a site. Wind, however, tends to move material completely off the site.

Subaqueous and intratidal sites, as well as areas of subaerial sites within the tidal zone, can be greatly altered by wave action and currents. These two factors combined can erode and redistribute dredged material over a wide area and possibly back into the area from which it was originally dredged. All sites studied were subaerial except for LWF-A and LWF-B, which have been classified here as intratidal. Bassi and Basco[2] discuss the effects of wave and currents on a subaqueous dredged material disposal area along the Intracoastal Waterway in Galveston Bay, Texas.

Controls

The evolution of dredged material sites is also controlled by the morphological characteristics of the site and physical characteristics of the dredged material. Of the morphological characteristics, the slope is an important control since it governs the rate of downslope movement of the dredged material caused by wind and rain. The slope of a newly placed site is not constant, but tends to flatten both towards the crest and towards the toe. The physical characteristics of the dredged material include both the grain size distribution of the material and the change in grain size distribution with respect to position on the site. The size distribution of the dredged material governs whether a site will become stable or be eroded away by the environmental processes. If the erosive forces acting upon the surface of a dredged material site are greater than or equal to that required to move the maximum grain size present, erosion will continue until all the dredged material has been removed. If the erosive force is less than that required to move the maximum grain size present, the removal of material will continue only until the surface becomes stabilized by the formation of a protective layer (surface veneer) of grains sufficiently large to be immovable. Nonuniform erosion may occur when the size and distribution of the dredged material is not uniform throughout the site and when the slope is not constant.

History of a Specific Site

An inherent factor governing the evolutionary stage of a dredged material site is its age. The length of time for stabilization depends upon the interactions between the processes and controls of a specific site. Given a set of controls, the magnitude of the processes and the length of time these processes have had to affect the site govern which process will have the dominant effect and the extent of that effect.

MOVEMENT OF MATERIAL BY WIND AND RAIN

Wind: Direct movement

The wind velocity profile over a sandy surface can be described by the general equation:

$$V = C \log Z/Z_o \qquad (1)$$

where V is equal to the wind velocity at a height Z and Z_o is a reference parameter. The coefficient C is equal to (2.31K) V_*, where V_* is the shear velocity and K is the universal constant for turbulent flow, sometimes called the von Karman constant. Bagnold[1] found the value of K to be 0.40, thereby giving the equation:

$$V = 5.73\, V_* \log Z/Z_o \qquad (2)$$

A thorough study by Zingg[19] resulted in a small value of 0.375 for the von Karman constant, yielding the equation:

$$V = 6.13\, V_* \log Z/Z_o \qquad (3)$$

for the wind velocity profile.

Zingg[19] further proposed the equation:

$$Z_o = 0.081 \log d/0.018 \qquad (4)$$

for the reference factor Z_o where Z_o and d, the sand grain diameter, are expressed in centimeters. This equation contains the results of Bagnold[1] (Z_o = d/30) for small diameter grains and that of (Z_o = d/9) for large grain sizes. Using Equations (3) and (4) Pierre-Yves Belly[3] found good correlation between theoretical and experimental data for the relations between wind velocities and heights above a sandy surface with mean grain sizes of 0.44 mm and 0.30 mm.

When the wind velocity is great enough over a sandy surface the particles will start to move due to the shear stress (τ). This shear stress is related to the shear velocity (V_*) by the equation:

$$V_* = \sqrt{\tau/\rho_f} \quad \ldots \quad (5)$$

where ρ_f is the density of the fluid causing the shear stress (ρ air = 1.22 X 10^{-3} gm/cm^3). Bagnold obtained the threshold value of this shear velocity (V_{*t}) by the equation:

$$V_{*t} = A\sqrt{\frac{\rho-\rho_s}{\rho_s}\, gd} \quad \ldots \quad (6)$$

where ρ_s equals the density of the particle being moved (ρ=2.65 gm/cm^3 for quartz), g is the acceleration due to gravity, and d is the grain diameter in centimeters. A is a constant which for air approximates 0.1 for grains larger than 0.25 mm. For smaller grains the surface becomes effectively "smooth" (the Reynolds number $\frac{V_* d}{\nu}<3.5$, ν is the kinematic viscosity of the fluid which equals 0.14 for air in c.g.s. units) and A is no longer constant. Zingg's[19] experiments corroborate this result.

Using Zingg's [19] modified wind velocity profile equation the threshold velocity (V_t) needed to move a certain grain size can be calculated by the equation:

$$V_t = 6.13\, V_{*t} \log Z/Z_o \quad \ldots \quad (7)$$

Table I has been developed relating the Wentworth grade scale of grain size to the threshold wind velocity at a height (Z) of 100 cm which is needed to move a particular grain size along flat sandy surface.

The fluid threshold velocities (V_t) presented in Table I are for dry sand surfaces of grains of uniform diameter. However, natural sand surfaces are made up of a mixture of grain sizes in which one grain size usually predominates. The size distribution is a bell-shaped curve in which the

grain size		V_{*t}	Z_o	log Z/Z_o	threshold wind velocity; V_t		
Ø size	mm(d)	cm/sec	cm		cm/sec	km/hr	mi/hr
-2.0	4.00	92.25	0.0109	4.96	2804.84	100.97	62.74
-1.5	2.38	77.60	0.0097	5.01	2383.20	85.79	53.31
-1.0	2.00	65.23	0.0085	5.07	2027.29	72.98	45.35
-0.5	1.41	54.77	0.0072	5.14	1725.70	62.12	38.60
0.0	1.00	46.13	0.0060	5.22	1476.10	53.14	33.02
0.5	0.71	38.87	0.0048	5.32	1267.61	45.64	28.36
1.0	0.50	32.62	0.0036	5.44	1087.79	39.16	24.33
1.5	0.35	27.29	0.0023	5.64	943.50	33.97	21.11
2.0	0.25	23.06	0.0012	5.92	839.67	30.22	18.78
2.5	0.177	*	*	*	*	*	*
3.0	0.125	*	*	*	*	*	*
3.4	0.088	*	*	*	*	*	*

* For surfaces composed of grains finer than 2.0 Ø the surface becomes effectively smooth and A is not constant, hence, Equation (7) is no longer valid.

Table I. Relationship Between Grain Size and Threshold Wind Velocity.

percentages of coarser and finer grain sizes decrease away from their predominant grain size. The initial threshold velocity (V_t) for natural sand, because of sheltering of the finer grains in crevices between the larger grains, is, therefore, the threshold velocity corresponding to the predominant grain diameter (Bagnold[1]).

If the wind does not increase above the initial fluid threshold strength, sand movement goes on until a majority of the predominant and smaller grain sizes have been carried away from the exposed surface, leaving predominantly only those of larger diameter. The sand movement then ceases. By raising the wind velocity, a further temporary movement is produced. Finally, a lowered sandbed covered by a surface veneer containing all the immovable grains which were present in the removed layers is left. The surface veneer, therefore, leaves a record of the maximum wind velocity. By examining the size distribution of the surface veneer samples collected and the maximum wind velocities recorded in the study area for the time period in which these surfaces have been exposed it was possible to make some inferences as to the role direct movement by wind played in the evolution of the subaerial dredged material piles studied.

Wind. - Indirect Movement

Large grains immovable directly by wind will move as a result of wind removal of finer grains around them, wind creep. Bagnold[1] noted that as the finer grains surrounding a group of larger grains are eroded away, the larger grains become undercut and roll away from the group. In time, the larger grains will become scattered across the surface. Although no quantitative work has been done on this type of movement, it is a very slow process and probably plays only a minor, if any, role in the early evolution of dredged material sites.

Rainfall

Laws and Parsons[12] were the first to quantitatively investigate rates of erosion with respect to raindrop impingement. Laws[11] found the maximum terminal velocity for falling drops of water to be approximately 32 kmph (20 mph), which exceeds the velocity of shallow runoff (0.32 to 3.2 kmph) by 10 to 100 times. This value (32 kmph) is the accepted maximum terminal velocity for natural rain and has been quoted by Ekern[9], Chow[6] and Baver, Gardner, and Gardner[4].

The size distribution for rain is a bell-shaped curve. The maximum size of natural raindrops is around 7 mm, corresponding closely to the 7.2-7.3 mm maximum values quoted by Ekern[8,9]. For North Carolina maximum rainfall rates are associated with convectional thunderstorms and average around 3.8 cm/hr (1.5 in/hr). These storms produce maximum raindrops of approximately 5.0 mm.

The erosion caused by raindrop impingement is governed by the amount of kinetic energy transferred from the raindrop to the surface. Ekern and Muckenhirn's[1] experiments indicated that the maximum kinetic energy imparted to the sand by the force of impact represented only about 2% of the total kinetic energy possessed by falling drops. Ellison[10] found that using a drop size of 5.1 mm and a drop velocity of 20 kmph (12.3 mph) fine soil fragments were moved 1524 centimeters (5 feet), 2 mm particles were carried 41 centimeters (16 inches) and 4 mm particles were splashed 20 centimeters (8 inches). Considering the maximum velocities and size of natural raindrops, normal raindrops impingement is capable of moving both small and large particles, and even gentle rains are potentially able to move fine sand. The unconsolidated dredged material on the sites studied was therefore vulnerable to this type of movement. During the field study pock marks formed by raindrops were found on several of the sites after local thundershowers passed over the study area.

On sloped surfaces such as dredged material sites, the direction of movement of material by impingement is not equally distributed, but has a net movement downslope. The percentage of the total transport that moves downslope varies directly with the percentage of slope by the equation (Ekern and Muckenhirn[7]):

$$\text{Percentage transport downslope} = 50 + (.94 \times \%\ \text{slope}) \quad . \; . \; . \; . \; . \quad (8)$$

The slopes of the CFR study sites varied between 5.5% and 4.4% giving downslope movement of material of 55.2% and 54.1% of that displaced by the raindrop impingement.

Raindrop Impingement vs. Runoff

Indirect movement of particles by raindrops can occur by the formation of surface runoff. The rate of runoff on a dredged material site is dependent on a number of factors, such as the rate of precipitation versus the rate of infiltration, the composition of the dredged material, and the type of vegetative cover. When the rate of precipitation is greater than the rate of infiltration, surface runoff will occur and sediment can be transported.

The rate of infiltration of a particular dredged material site is predominantly dependent on the type of material composing the site and the vegetative cover present (Sokoloskii[15]). On the sites in this study the vegetative cover, when present, consisted of a very sparse herbaceous cover which probably contributed little to the rate of infiltration. The rate of infiltration, therefore, is almost solely governed by the dredged material composition.

For dredged material sites composed of predominantly sand size material, one would expect the rate of infiltration to be quite high (Leopold et al.[13]). This, together with the fact that few positive runoff structures, such as channels or rilled structures, were seen on the sites studied, leads to the conclusion that runoff was not sufficient in the evolution of these sites. Raindrop impingement appears to be a much more important evolutionary factor.

Raindrop Impingement vs. Wind

Raindrop impingement can move coarser grains than can be moved directly by wind. These coarser grains, however, as shown by Ellison[10] will move more slowly than the finer grains being moved by impingement. Thus raindrop impingement, like wind, will cause the formation of a coarse surface veneer. Such a veneer, however, will be coarser than one formed solely by the wind since impingement will displace coarser grains.

In terms of relative rates of formation, a raindrop impingement surface veneer would take longer to develop than a surface veneer formed solely by wind. On very young sites, therefore, the surface veneer should be wind-formed, enriched in grains too coarse for wind removal but capable of movement by rain impingement. Conversely, the surface veneer of older sites should exhibit a relative paucity of grains in the range of too coarse to move by wind but capable of raindrop displacement.

CONCEPTUAL MODEL OF SITES

Figure 6 (A) shows a conceptual model of wind deflation acting alone on a dredged material site of uniform grain size distribution. The removal of finer sized material would be relatively uniform across the entire site until an immovable surface veneer formed. In cases where the maximum grain size is within the size range of that movable by wind, deflation should continue to occur until the site has been leveled, or until vegetation is established.

In none of the sites studied was the initial sand size distribution found to be uniform across the site, nor were the maximum grain sizes less than that movable by the maximum winds. Given the presence of an initial dredge effect, combined with a size fraction not movable by wind, deflation can be modeled similarly, except that the dredge effect of decreasing grain size downslope is preserved in both the surface veneer samples and depth samples. While this model is compatible with samples of the young site CFR-D, it does not fit the data for the older sites, CFR-B and CFR-C.

CFR-B and CFR-C sites have surface veneers too coarse to have formed directly by wind alone and are old enough to have been affected by some combination of surface runoff, raindrop impingement, and wind creep. In addition, the anticipated dredge effect, well developed in both surface veneer and depth samples of a younger site (CFR-D), was found only poorly developed in CFR-C and actually reversed in CFR-D depth samples. The anomalous behavior of these profile depth samples may be explained by Figure 6 (B). Concurrent with direct movement by wind but acting much more slowly, downslope movement of grains by raindrops impingement and wind creep (and possibly runoff) erodes the crest of the site including material too coarse to be deflated. This material was transported to and deposited on the lower gradient flanks of the site. Finer grains moved directly by the wind, as in Model A, tend to be carried beyond and entrapped by peripheral vegetation.

The net effect on dredged material piles caused by raindrop impingement, wind creep, and runoff on dredged material sites would be, first, to form a coarser surface veneer. Secondly, larger grains, if movable at all would move downslope more slowly than smaller grains, thus leaving more coarse grains towards the crest. The resultant surface veneer will show a decrease in mean grain size downslope. Thirdly, the surface veneer towards the crest would be sitting on the original undisturbed dredged

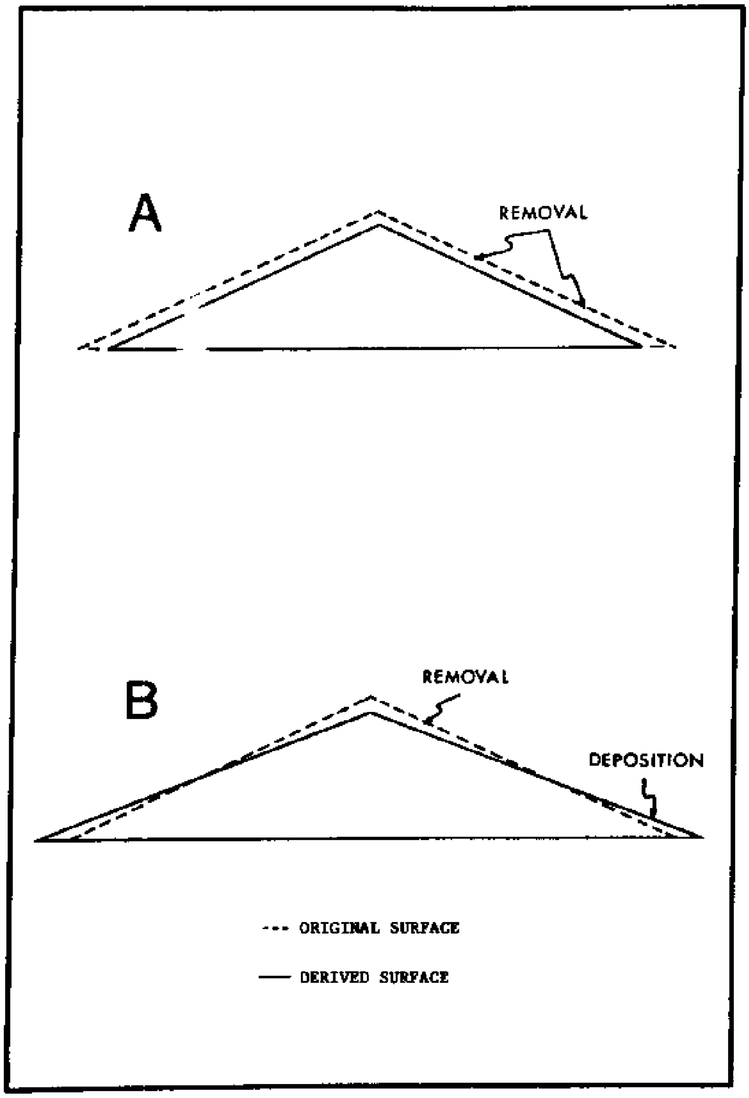


Figure 6.

Conceptual Models of Dredged Material Pile Evolution.

Model A: Evolution Due to Wind Deflation.

Model B: Evolution Due to the Combination of Wind Deflation Plus Surface Runoff, Raindrop Impingement, and/or Wind Creep.

material which should still show a dredge effect. The surface veneer towards the base would be derived from material that has been transported from above. Additionally, it would sit on transported material rather than undisturbed initial dredged material. Samples collected here would have been partially winnowed by wind and rain and would tend to be coarser than the depth samples collected towards the crest.

SUBAERIAL DREDGED MATERIAL SITES

The factors which apparently govern early (pre-vegetation) evolution of subaerial dredged material sites were investigated by studying sites of differing ages and initial materials. Tests were conducted (1) to demonstrate the initial downslope size gradient due to the dredging process, (2) to detect the presence of recognizable coarser surface veneers and (3) to indicate the relative influences of wind and/or rain in the early evolution of the dredged material sites and formation of the surface veneer.

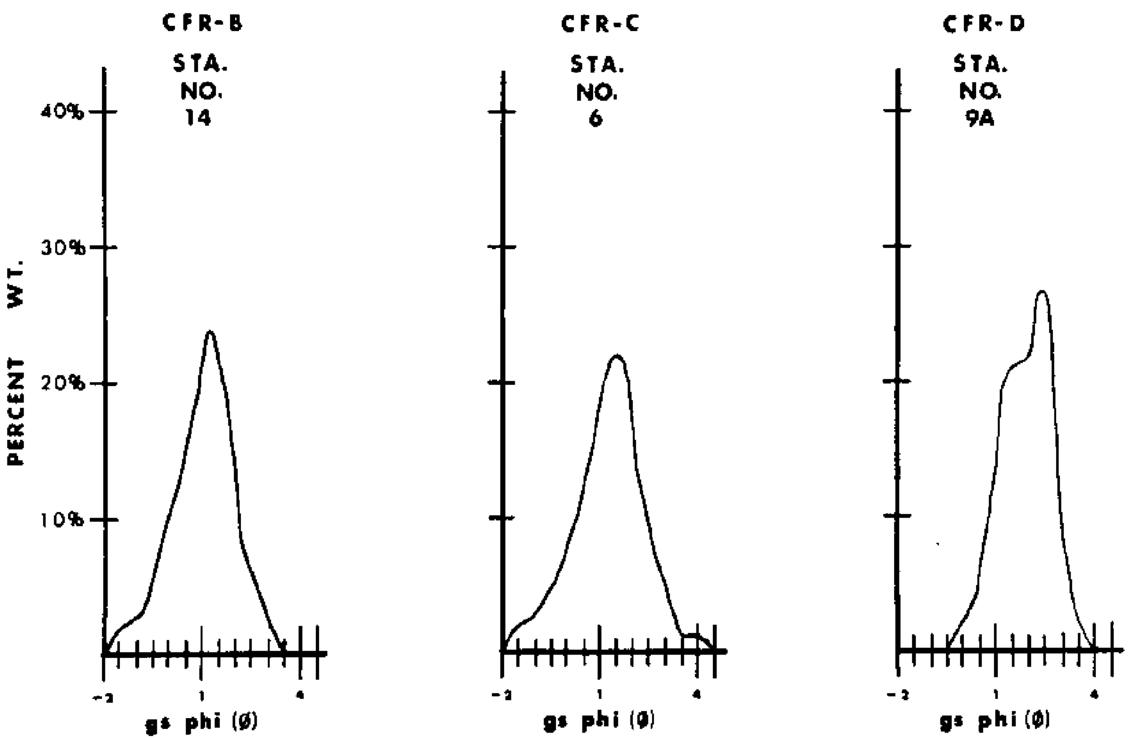
Three sites (CFR-B, CFR-C, and CFR-D) along the Cape Fear River were chosen for detailed analysis. These offered a variety in age and dredged material, and their close proximity to each other guaranteed (within reason) uniform meteorological influences. In addition, their positions offered easy access by boat while insuring they were isolated so as to minimize alteration by human or vehicular traffic.

Sites Selected

The site labeled CFR-B is one of a continuous series of sites that stretches for over 1.6 kilometers (1 mile) along the New Hanover-Brunswick County line. It is situated on the east side of the main navigation channel 8 kilometers (5 miles) downriver from the Cape Fear Memorial Bridge at Wilmington, North Carolina as shown in Figure 1. It was deposited in 1969 by a 41-centimeter (16-inch) pipe dredge during a navigation channel widening project. The material composing the site consisted of a clean, slightly gravelly to gravelly sand as indicated in Figure 7 with a few consolidated "clay balls" scattered throughout. A study profile running 64 meters (210 feet) from the crest (the point of discharge of the dredged material) to the toe of the pile along a bearing of N - 15° - E had an average slope of 5.5%.

CFR-C lies 13 kilometers (8 miles) downriver from the Cape Fear Memorial Bridge as shown in Figure 1 across the main shipping channel from Campbell Island. It was deposited under the same dredging project as CFR-B in 1969, and consisted of similar material as indicated in Figure 7. This site is undiked and differs from CFR-B in both size and average slope (4.4%) along a profile N - 23° - W from the crest to the point where dense vegetation was encountered.

CFR-D is situated 17 kilometers (10.5 miles) downriver from the Cape Fear Memorial Bridge and a little less than 1.6 kilometers (1 mile) northwest of the western entrance to Snows Cut as shown in Figure 1. It lies between the Cape Fear River main navigation channel and the north connecting channel to Snows Cut. This subaerial dredged material pile is diked and was most recently used in June 1975 (approximately 10 weeks prior to field study) as a disposal site for routine maintenance dredging of shoal areas in the main CFR navigation channel. The dredged material consisted of a clean, moderately sorted sand as indicated in Figure 7. A 61-meter (200-foot) study profile was run N - 65° - E from the point of discharge of the dredged material to the dike surrounding the site. The site had an average slope of 4.4%.



Mean Gs.
0.692ϕ

Mean Gs.
0.938ϕ

Mean Gs.
1.695ϕ

Figure 7.

Size Distribution Curve

Detection of Initial Dredge Effect

When dredged material is discharged down a slope, where the proportion of water to sediment is high, a site will exhibit a decrease in mean grain size from the crest to its toe. This is caused by the decrease in carrying capacity of the water as it spreads out across the site from its point of discharge at the crest. On older sites this trend can be expected to be preserved in undisturbed portions of the sites.

Test Procedure. Spearman's Rank/Order Correlation (Siegle[14]) was used to test the validity of this relationship between the relative profile positions of the depth samples and their mean grain sizes. Results summarized in Table II show CFR-C and CFR-D exhibited a significant (.05 level) correlation between profile positions and mean grain sizes of depth samples. Both CFR-C and CFR-D profiles showed the decreased grain size downslope anticipated by consideration of the dredge discharge processes. CFR-B depth samples, however, proved insignificant and in fact exhibit a reverse relationship.

Since sample points were uniformly spaced along the study profiles, a linear correlation-regression technique was used to test if grain size change detected by rank-order correlation was a linear (in phi (ϕ) scale) change across the study piles. The coefficient of determination (r^2) in Figure 8 indicates that 90% of the variations in sample grain size along the young CFR-D study profile correlates linearly to its position on the pile. Conversely, depth samples from the two older piles, CFR-B and CFR-C, did not show any detectable linear relationship between distance along the slope and mean grain size since the 95% confidence interval on the slope (b) encompasses the value $b = 0$.

On the youngest and, therefore, probably least altered site, CFR-D, rank-order correlation implied that the initial dredge effect both existed and could be detected. In addition, within this material of sand size,

CFR-B-VENEER

N=10

r_s =-.576

($r_{.05}$ = .564 w/9df)

Significant

CFR-B-DEPTH

N=10

r_s = -.442

($r_{.05}$ =-.564 w/9df)

Not Significant

CFR-C-VENEER

N=5

r_s = .30

($r_{.05}$ = .900 w/4df)

Not Significant

CFR-C-DEPTH

N=5

r_s = .900

($r_{.05}$ = .900 w/9df)

Significant

CFR-D-VENEER

N=7

r_s = .964

($r_{.05}$ = .714 w/6df)

Significant

CFR-D-DEPTH

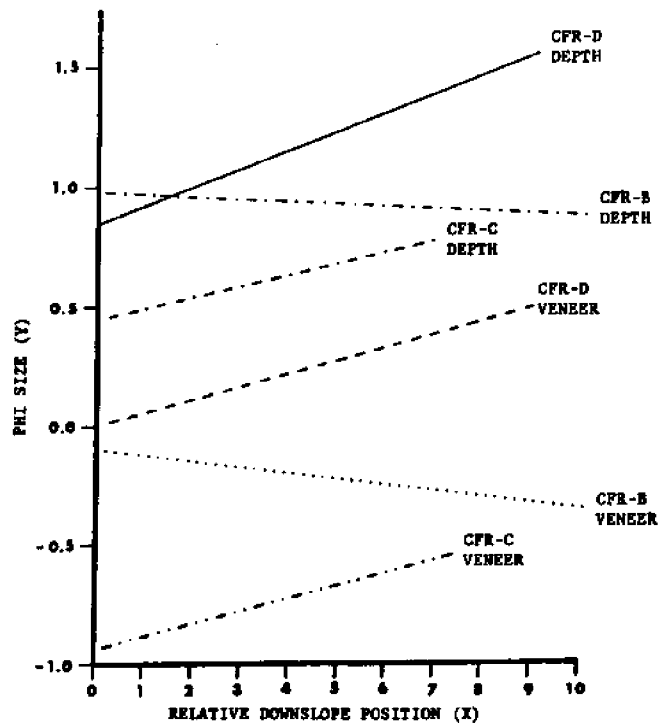
N=7

r_s = .964

($r_{.05}$ = .714 w/6df)

Significant

Table II. Spearman's Rank/Order Correlation of Mean Grain Size of Samples Collected on Sites Studied.



	CFR-B VENEER STA.1-19	CFR-B DEPTH STA.1-19	CFR-C VENEER STA.3-11	CFR-C DEPTH STA.3-11	CFR-D VENEER STA.3-15	CFR-D DEPTH STA.3-15
N	10	10	5	5	7	7
$\bar{X}$	10.00	10.00	7.00	7.00	9.00	9.00
$\bar{Y}$	-0.37	0.858	-0.858	0.762	0.486	1.522
a	-0.105	0.975	-0.933	0.450	0.001	0.847
b	-0.027	-0.012	0.052	0.045	0.054	0.075
r	-0.587	-0.344	0.590	0.758	0.929	0.949
r^2	0.345	0.118	0.348	0.575	0.863	0.900
CI_u	0.002	0.013	0.165	0.106	0.077	0.102
CI_l	-0.056	-0.036	-0.062	-0.017	0.031	0.048

Figure 8.

Correlation Regression Analysis for Veneer and Depth Samples.

mean grain size distributions (phi (ϕ) scale) display a marked linear relation with distance from the point of dredge discharge, the crest.

The older site, CFR-C, likewise exhibited a detectable dredge effect in the depth samples based upon the rank-order correlations. The pronounced linear relation of size with distance, however, is absent in the analyzed sand fraction.

Profile CFR-B in no way followed the pattern expected to be exhibited by a dredge effect. Not only was the linear correlation regression nonsignificant, but the rank order test even suggests a weak inverted relationship. Three reasons could be postulated for this result: (1) there is no dredge effect present, (2) restricting analysis to only the sand-sized material masked the dredge effect, or (3) the dredged material has been altered, destroying the dredge effect within the upper 5.0 cm (2 in.) sampled.

Both the younger CFR-D and the older ages CFR-C sites exhibited a dredge effect, so there was no reason to assume such did not occur for CFR-B.

Histograms for CFR-B indicated the non-sand fraction at a greater depth was small and similar to that of CFR-C which exhibited a dredge effect. Therefore, the inability to detect this effect on CFR-B was probably not due to analyzing truncated sand distributions. The remaining possibility for the origin of this anomalous relationship is alteration of the dredge effect in the upper several centimeters by some process or processes associated with the formation of the surface veneer. This dredge effect alteration is discussed further in the section dealing with surface veneer formation.

Detection of the Occurrence of a Surface Veneer

Processes (wind and rain) acting upon an exposed sandy surface will create a surface veneer by the selective removal of the fine grains. The presence of a surface veneer on dredged material sites with a gravel fraction (CFR-B and CFR-C) is readily observable by comparing simple histograms contrasting the surface veneer and depth samples. By the analysis of the sand-size fraction itself, the presence of a surface veneer on all sites likewise should be detectable with site-by-site comparison of depth and surface veneer samples. Paired t-tests were run on simple mean grain sizes of all three Cape Fear River study sites, to test the presence of surface veneer forming processes. The results are summarized in Table III. Samples were paired based on profile locations in order to eliminate location-to-location variation. Comparing mean grain size of paired surface veneer and depth samples shows that even in a relatively short time (CFR-D was approximately 10 weeks old when sampled) a markedly coarser surface veneer formed. The same effect for the sand size fraction can be expressed graphically by plotting the undifferentiated points of the cumulative probability distributions. Figure 9 shows a plot of all samples from profile CFR-C. The cumulative curve values of surface veneer samples occupy a distinctly coarser field than those of the depth samples. In addition, the cumulative curves for pit samples lie totally within the field of depth samples for CFR-C and CFR-D, which suggests that the 5-centimeter (2 in.) depth samples from these sites are undisturbed representatives of original dredge material. CFR-B is again anomalous. Here, the pit samples are not contained within the field of depth samples, suggesting that depth samples from this site had been modified.

Descriptive Character of the Surface Veneers. The coarse-grained residual accumulations for which the finer material has been winnowed, called surface veneer, have been labelled desert crust, desert pavement, pebble armor, or lag gravel. Whereas these latter terms either designate

CFR-B

$N = 11$

$\bar{\Delta} = -1.253$

$s_{\bar{\Delta}}$ 0.085

$t = \dfrac{\bar{\Delta} - 0}{s_{\bar{\Delta}}} = -14.74$

($t_{.01} = -2.764$ w/ 10 df)

Significant at $\alpha = .01$

CFR-C

$N = 4$

$\bar{\Delta} = -1.333$

$s_{\bar{\Delta}} = 0.072$

$t = \dfrac{\bar{\Delta} - 0}{s_{\bar{\Delta}}} = -18.51$

($t_{.01} = -3.747$ w/ 4 df)

Significant at $\alpha = .01$

CFR-D

$N = 9$

$\bar{\Delta} = -1.166$

$s_{\bar{\Delta}} = 0.112$

$t = \dfrac{\bar{\Delta} - 0}{s_{\bar{\Delta}}} = -10.41$

($t_{.01} = -3.143$ w/ 8 df)

Significant at $\alpha = .01$

Table III - Paired T-Test Between Mean Grain Sizes of Paired Surface Veneer and Depth Samples for CFR-B, CFR-C and CFR-D.

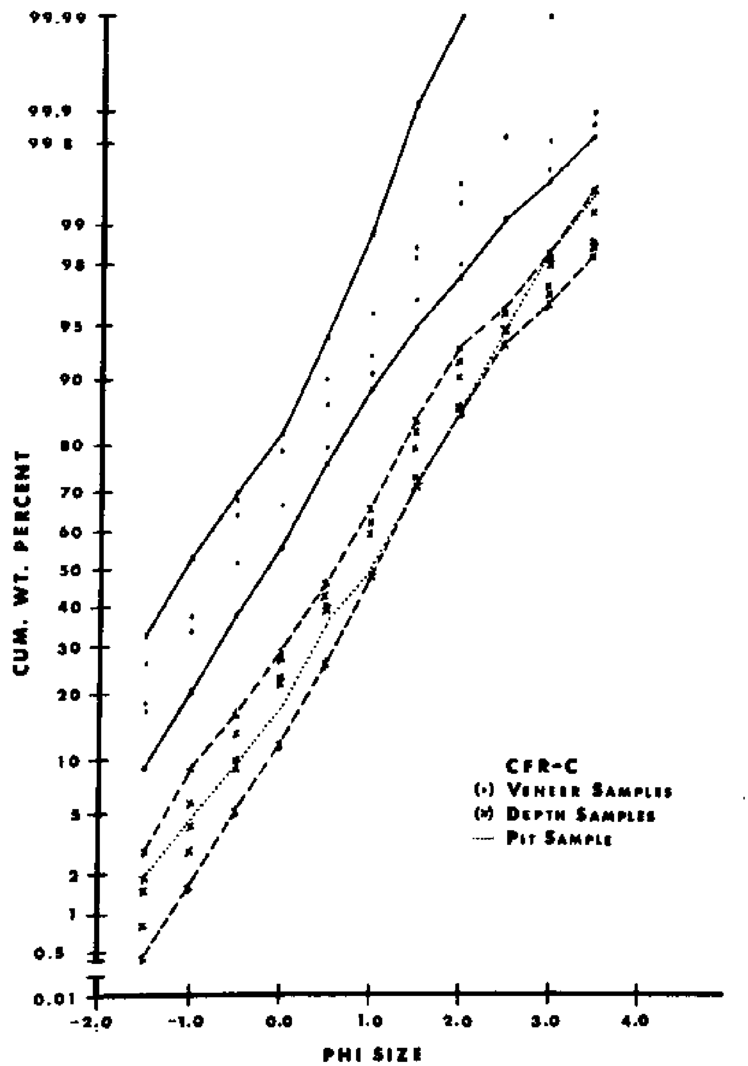


Figure 9

Composite Plot of Cumulative GrainSize Distribution Points for Samples From Dredged Material Site.

Solid Lines (-); Envelope of All Veneer Sample Size Distribution Points.

Dashed Lines (---); Envelope of All Depth Sample Size Distribution Points.

Dotted Lines (.....); Size Distribution Curve for the Pit Sample.

the geographic area of its occurrence or restrict the material make-up to pebble- or gravel-sized material, surface veneer, as used here, is a more general term. It denotes a thin protective surface layer of one to two grains in thickness and made up of material coarser than and derived from that which is directly below it. The grain size of the surface veneer will vary depending on the environmental factors acting upon it and the size distribution of the material from which it was derived.

The surface of CFR-B is predominantly bare with only sparse grasses and other herbaceous plant growth (Figure 2). The surface veneer composed of coarse-grained quartz is distinctively coarser at the crest. As one moves down the slope the veneer remains distinct and coarser than the underlying material, but the number of grains in the pebble and granule size range markedly decrease in number. When the dense vegetation around the periphery of the site was reached a surface veneer was no longer detectable.

CFR-C being deposited during the same project as CFR-B appears to be at the same developmental stage. The vegetation (Figure 3), though possibly a little more advanced than CFR-B because of its smaller size and lower elevation, consists of scattered grasses and herbaceous growth. The surface veneer again is coarser at the crest and finer towards the toe. No surface veneer was detectable in the dense vegetative growth at the toe of the study site.

No vegetation was present along the study profile on CFR-D due to its young age (approximately 10 weeks). The surface appeared smooth with no erosional or depositional structures such as wind ripples or runoff channels evident. Close examination of the surface was needed to detect a surface veneer which consisted of a thin layer of slightly coarser and better sorted quartz sand. A distinct decrease in grain size downslope could not be detected in the field and the preceding laboratory analysis was needed before this relationship became apparent.

Factors Influencing Dredged Material Pile Evolution

The expected factors affecting the evolution of dredged material sites, as mentioned earlier, can be broken down into processes, controls, and the history of a particular site. By varying the controls and keeping the histories and processes relatively constant, inferences as to dredged material site evolution can be drawn.

Factors Affecting CFR-D. To attempt to demonstrate wind as an effective agent in the early evolution of a dredged material site, CFR-D was chosen. This site, approximately 2 months old when sampled, would not have had sufficient time to be appreciably influenced by runoff, raindrop impingement, or sediment movement caused by wind creep. Any surface veneer, therefore, would have been primarily derived directly by wind. The presence of a surface veneer, as demonstrated, states that wind is an effective early agent in dredged material site evolution.

The highly significant "best fit" regression line for the surface veneer of this site indicates a linear grain size (in log scale) decrease with distance from the crest in a totally sand-size dredged material site. This relation appears to be an inherited dredge effect since the slopes of the surface veneer and depth samples regression lines are not appreciably different. Therefore, it can be inferred that this profile represents one of deflation, with deposition of winnowed sand occurring beyond the sampling boundaries.

If these conclusions are valid, the grain size distributions of the piles surface veneer should be compatible with wind conditions existent during its exposure. Maximum wind velocities, measured at the nearby Wilmington Airport Weather Station were used to estimate the movable sand fraction that should have been lost from the truncated size distribution of the surface veneer samples. From the time CFR-D was deposited to the time it was sampled, the maximum wind velocity for the months of June and July, 1975, was 51 kph (32 mph). This value lies between the calculated minimum wind velocities needed for removal of sand grains of up to 0.5 and 0.0 phi (ϕ) size respectively (Table II).

Figure 10 shows a plot of the sand size distribution for depth and surface veneer samples from each location on the three sites, together with the mean grain size of each of the samples. The mean grain size for the upper station #3 of the CFR-D surface veneer corresponds to the minimum grain size immovable by the maximum winds that occurred between the time it was deposited and sampled. The accompanying depth sample is finer than this threshold size. Grain sizes less than the mean grain size, therefore, must have been partially winnowed out from the original surface by the wind to form a coarser surface veneer. Sample sites lower on the profile exhibited mean grain sizes finer than expected for the given wind conditions. This probably is a reflection of the finer initial downslope grain size resulting from the dredge effect, combined with reduced maximum wind velocities lower on the slope resulting from the effects of topography.

Factors Affecting CFR-B and CFR-C

From 1969, when dredged material sites CFR-B and CFR-C were deposited, to 1975, when they were sampled, maximum wind velocities in excess of 64 kph (40 mph) occurred 5 times. From Table II, the maximum grain size movable by this magnitude of wind lies between -0.5 to -1.0.

It is recognized that vegetation not only protects the ground surface from wind erosion but causes entrained sand to be accumulated. This fact has been demonstrated by the stabilization of dredged material sites in the past (Woodhouse[18]) by planting them with a variety of vegetative covers including smooth cordgrass (Spartina alterniflora salt meadow cordgrass (Spartina patens), and Panicum amarulum.

Along the study profiles of CFR-B and CFR-C slight elevation rises of 3.8 to 5.1 centimeters (1.5 to 2.0 inches) were observed at the edge of the vegetation surrounding each site. At these sites no detectable surface veneers were present. Samples taken here should be the coarser of the windblown sand fraction winnowed from the adjacent sites and entrapped by the vegetative growth around the periphery. Maximum sample sizes of both stations Figure 10 correspond well with the phi (ϕ) size range computed for the maximum wind conditions which affected these sites.

It was demonstrated that the surface veneer mean grain size for station #3 of CFR-D corresponded well with the minimum grain size immovable by the maximum winds for the period of time from deposition to sampling. This is shown graphically for CFR-D veneer sample #3 in Figure 10. For sites CFR-B and CFR-C, samples from the surrounding vegetation at the foot of the slope likewise are indicative of wind deflation of the pile surfaces. However, Figure 10 also demonstrates that existent wind conditions alone could not have directly formed the surface veneers at station #1 on CFR-B or station #3 on CFR-C. The wind velocities needed to form a surface veneer of such a large mean grain size ($< -2.0\ \phi$) would exceed by far the maximum wind velocity, 71 kmph (44 mph) (April 2, 1970), measured at the Wilmington Airport Weather Station in the period since these two sites were deposited. Therefore, direct movement by wind, though important in the early stages, appears not to be the sole agent involved in the continued evolution of these older sites.

The coarse surface veneers that characterize these older sites therefore must have been formed by some undefinable combination of sediment movement by wind creep and raindrop impingement, as well as possible

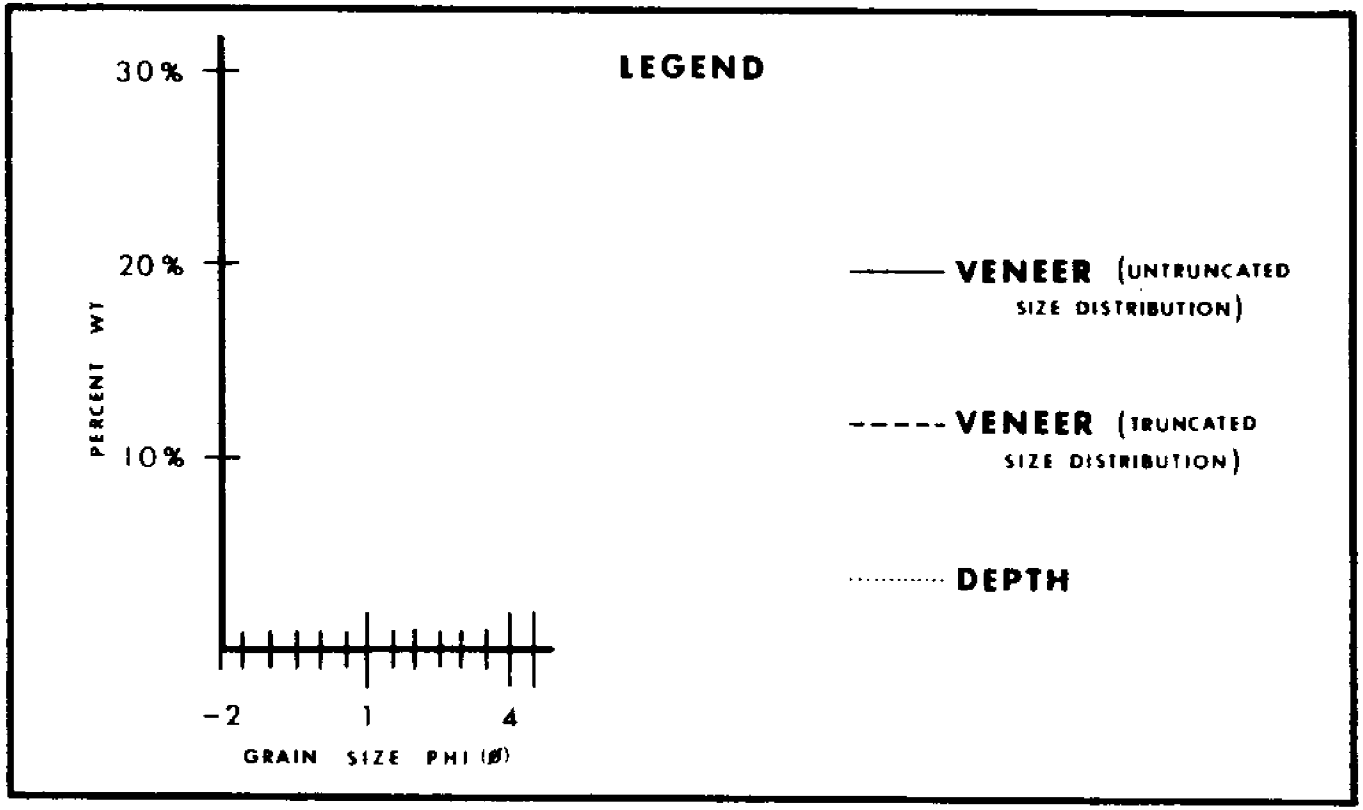
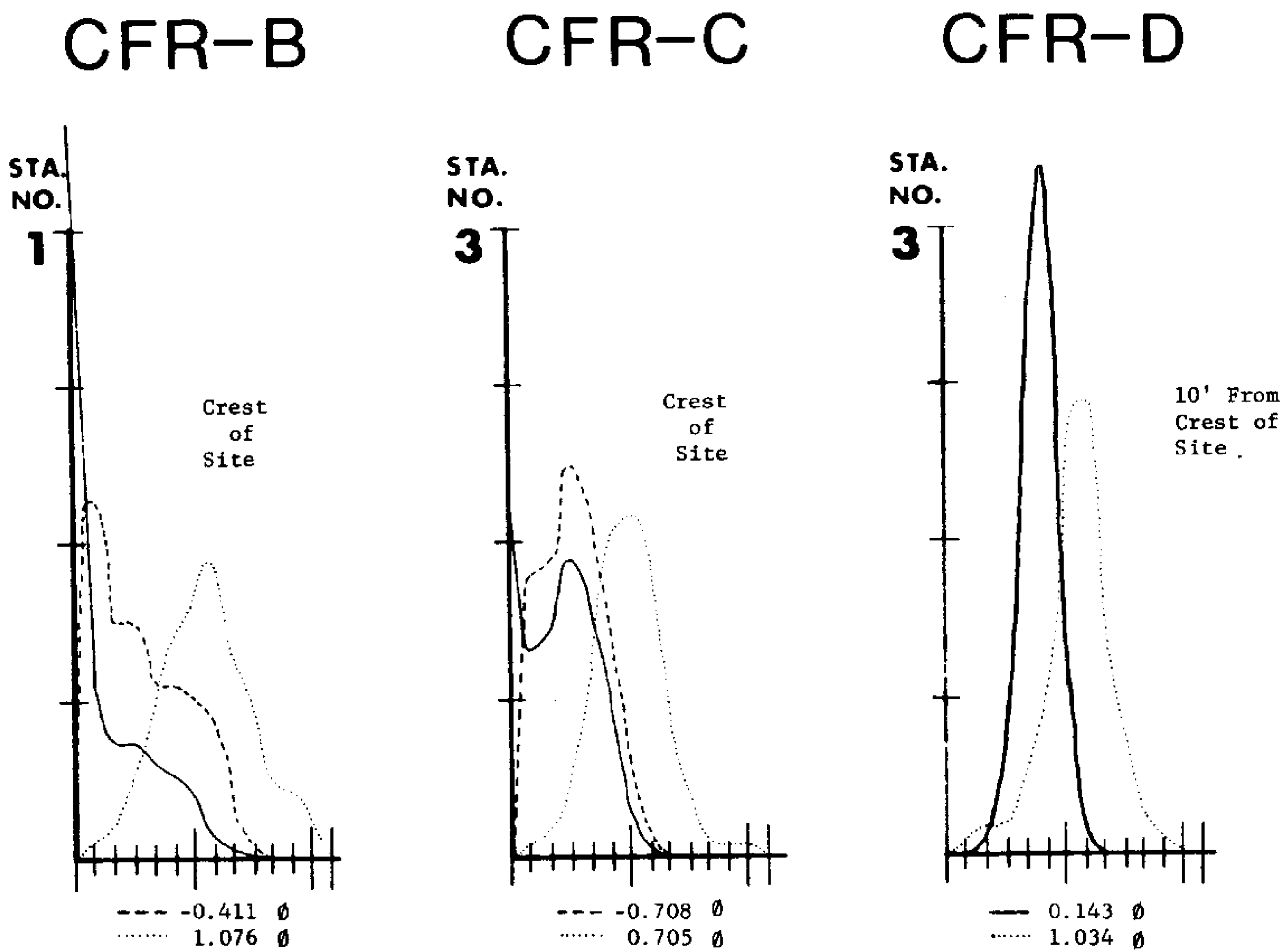
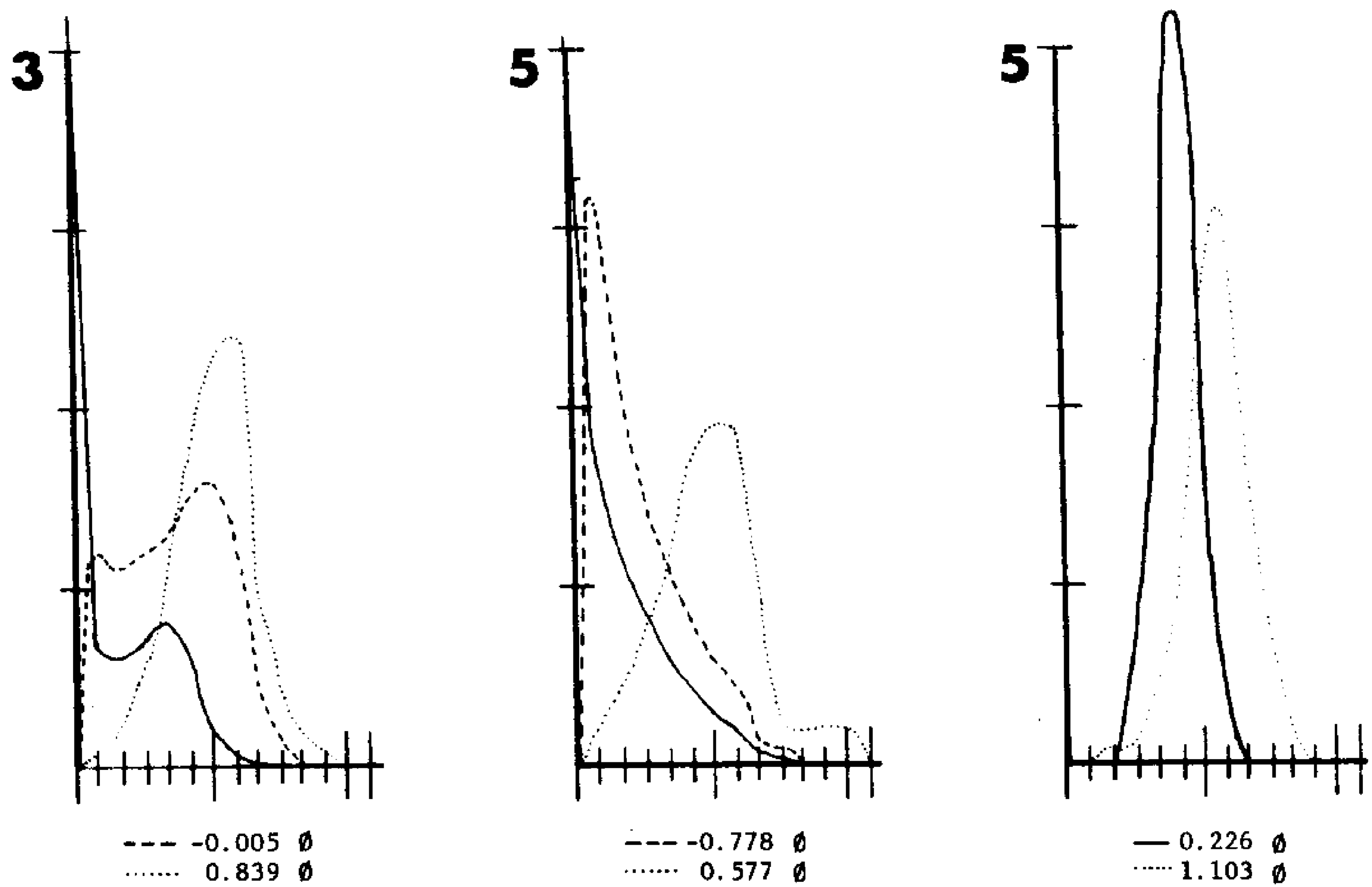
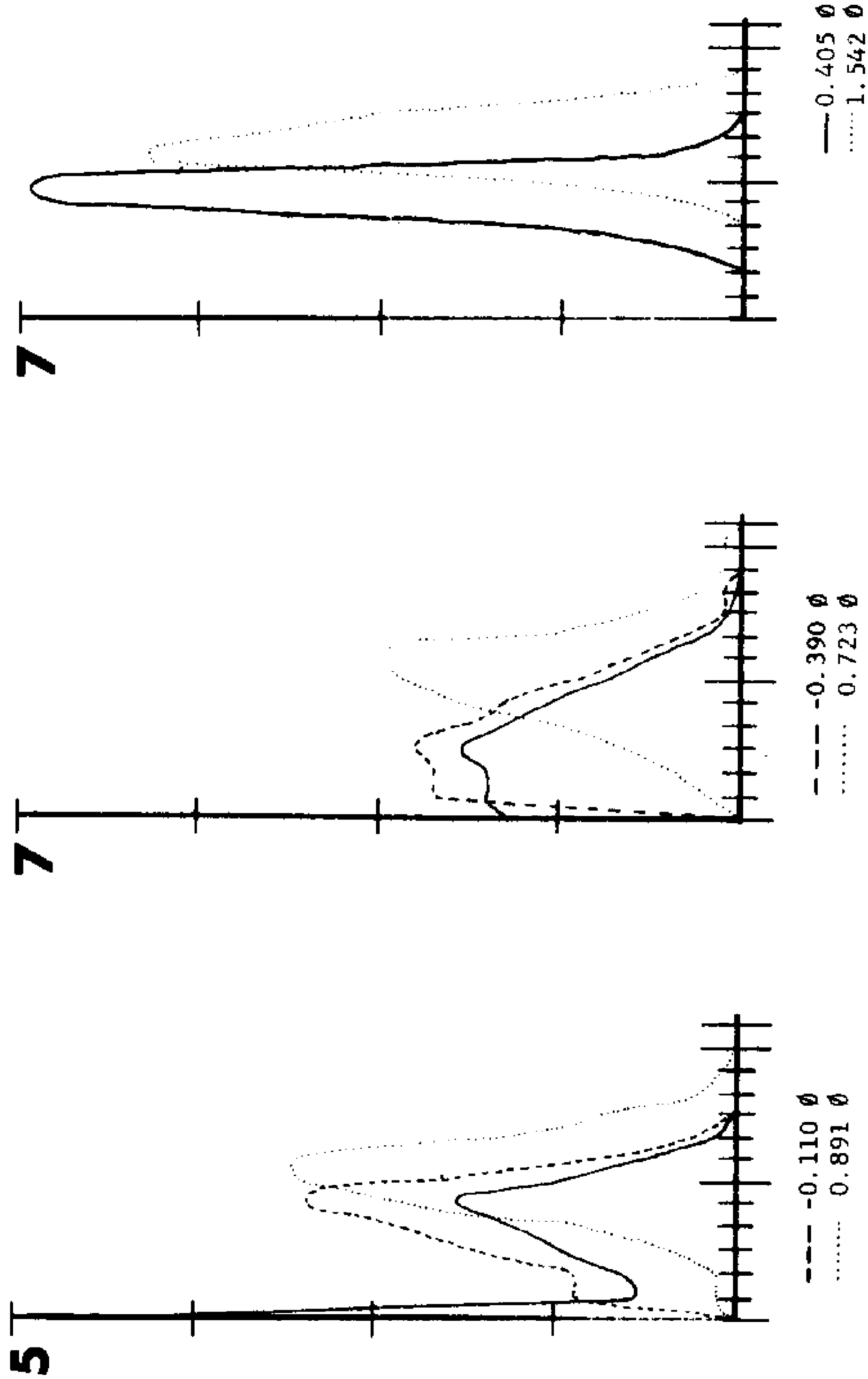
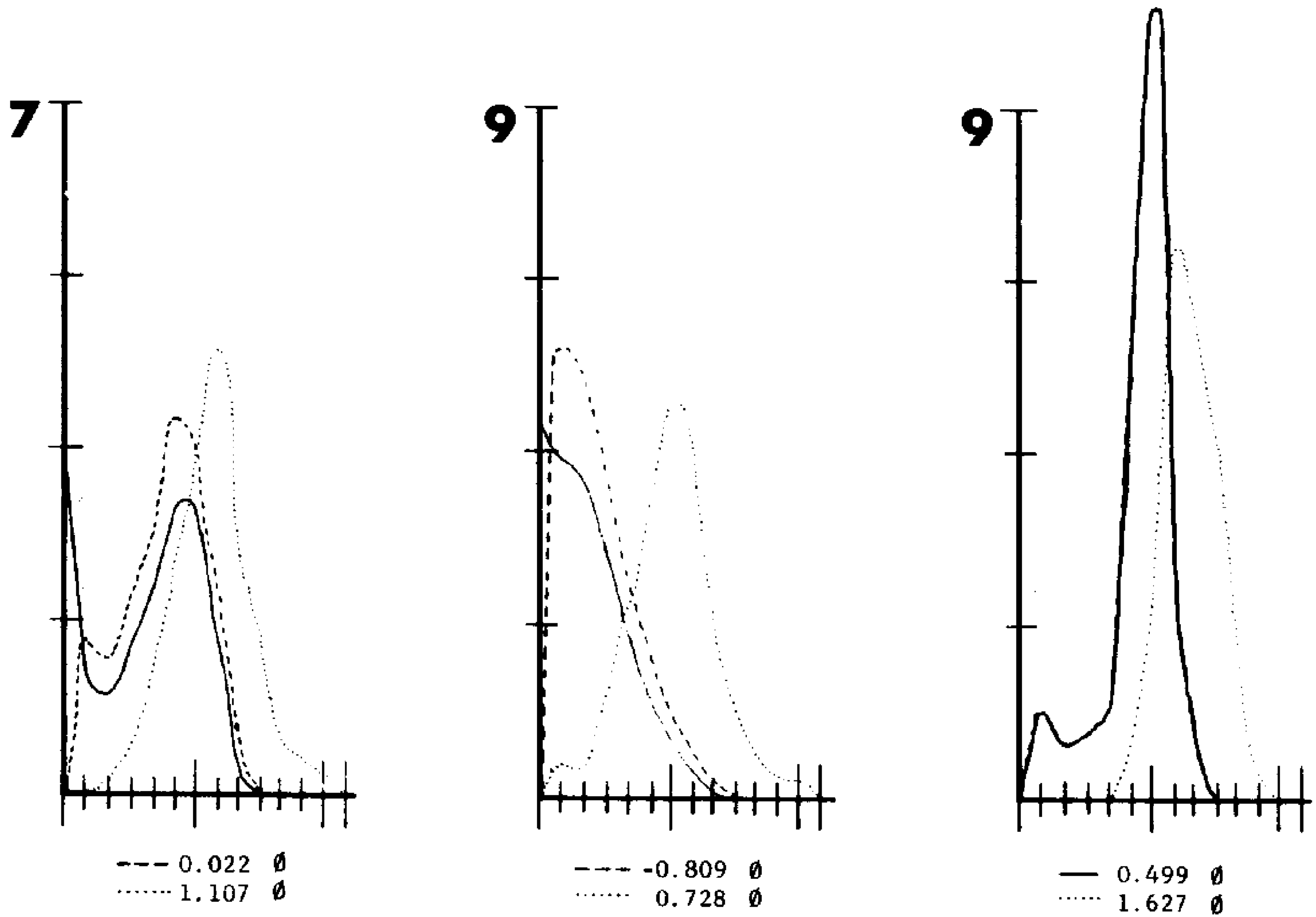


Figure 10. Size Distribution for All Samples Collected on CFR-B, CFR-C, and CFR-D. Size distributions below cut line shown the relationship between grain sizes and maximum wind velocities for selected locations.

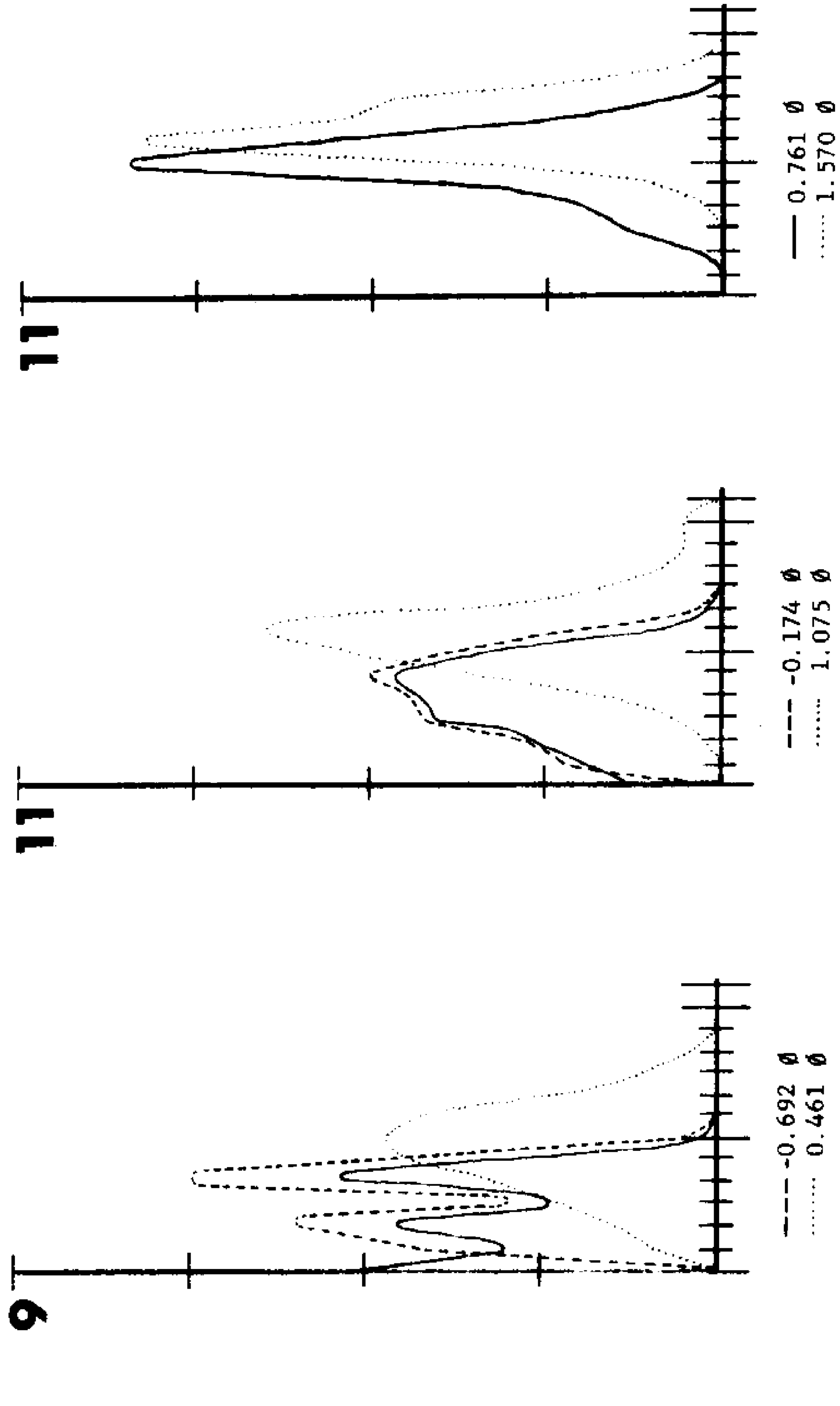

CFR–B
CFR–C
CFR–D
STA. NO. 1
STA. NO. 3
STA. NO. 3
Crest of Site
Crest of Site
10' From Crest of Site.
--- -0.411 Ø
...... 1.076 Ø
--- -0.708 Ø
...... 0.705 Ø
— 0.143 Ø
...... 1.034 Ø

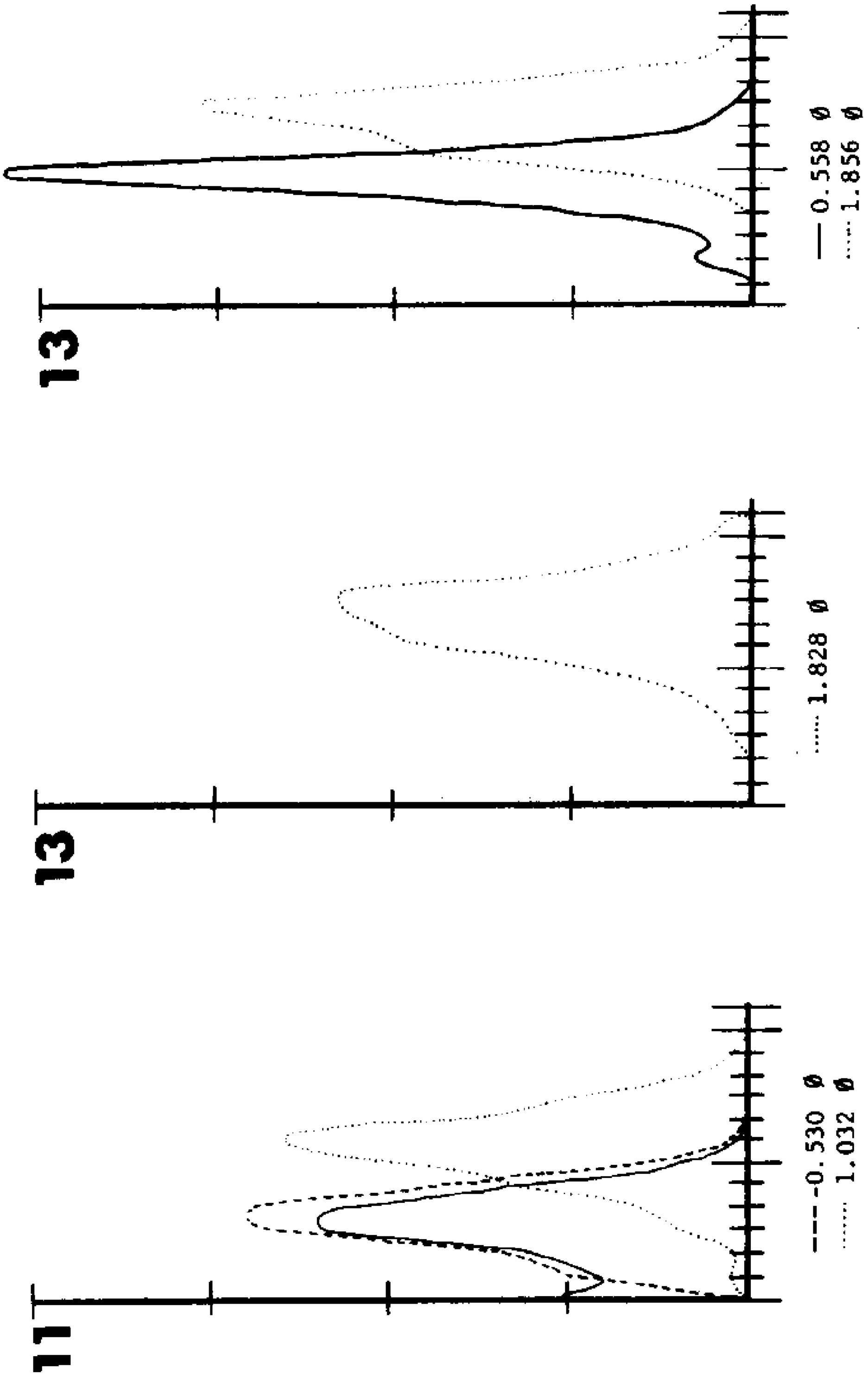

3
5
5
-0.005 Ø
0.839 Ø
-0.778 Ø
0.577 Ø
0.226 Ø
1.103 Ø


5
-0.110 Ø
0.891 Ø
7
-0.390 Ø
0.723 Ø
7
0.405 Ø
1.542 Ø

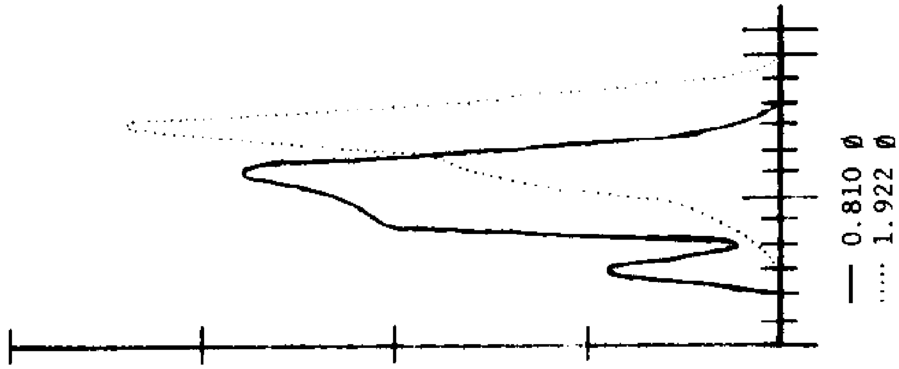


7
0.022 Ø
1.107 Ø
9
-0.809 Ø
0.728 Ø
9
0.499 Ø
1.627 Ø


9
--- -0.692 Ø
..... 0.461 Ø
11
--- -0.174 Ø
..... 1.075 Ø
11
— 0.761 Ø
..... 1.570 Ø


11
-0.530 Ø
1.032 Ø
13
1.828 Ø
13
0.558 Ø
1.856 Ø

15



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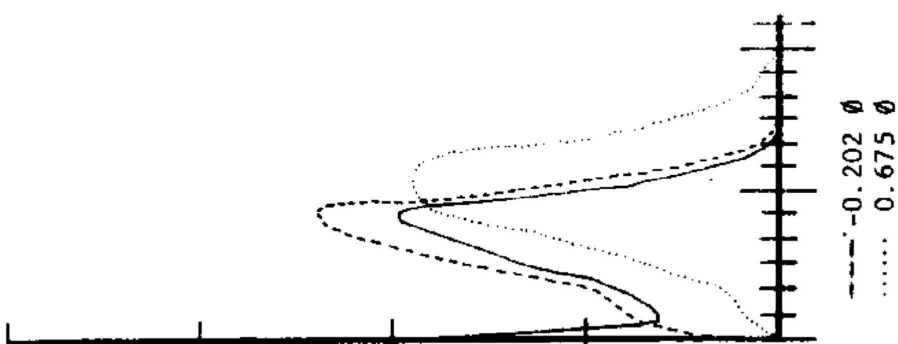

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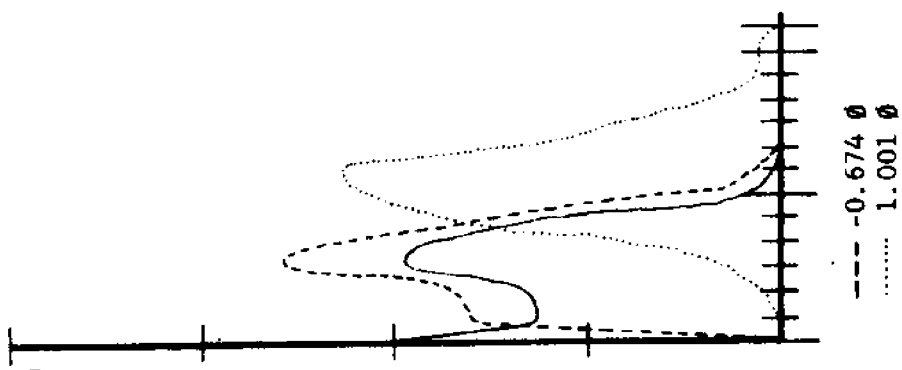
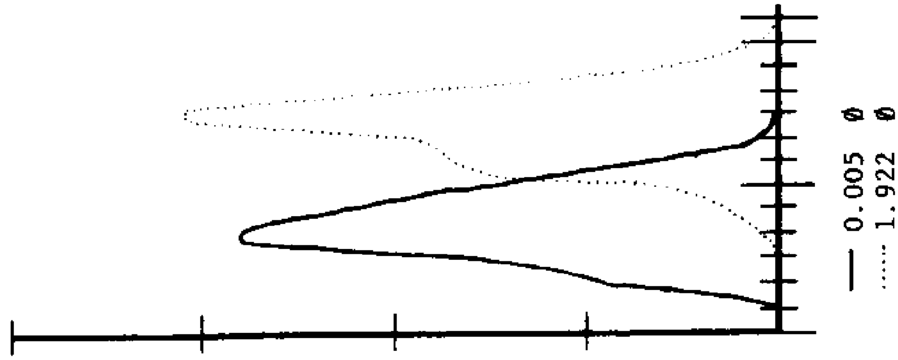
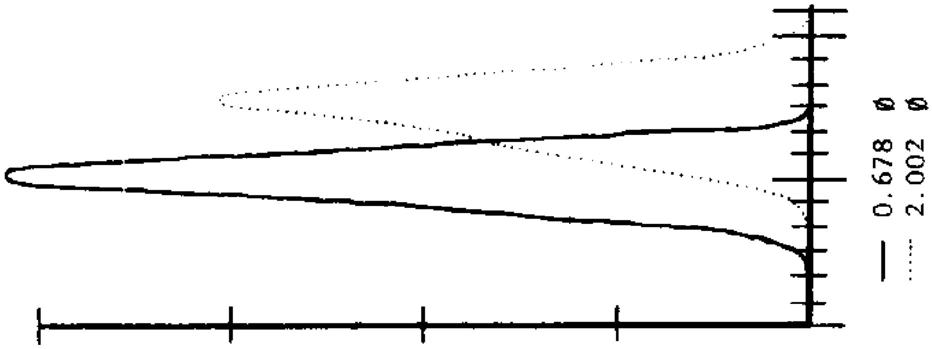
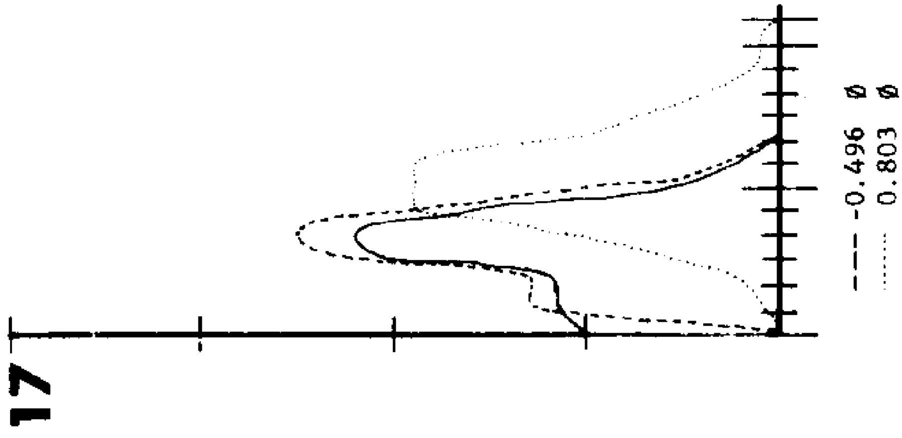


17

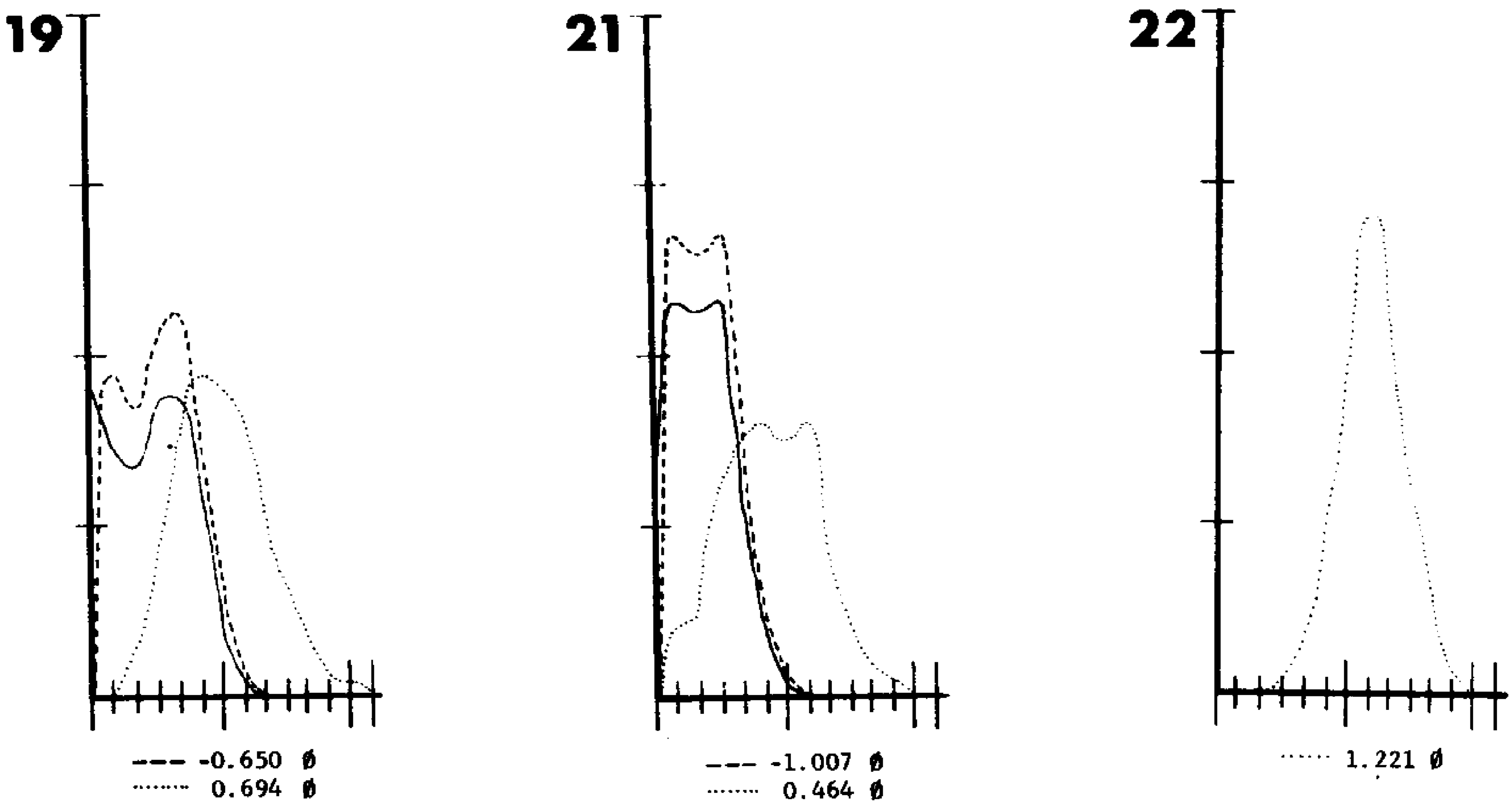




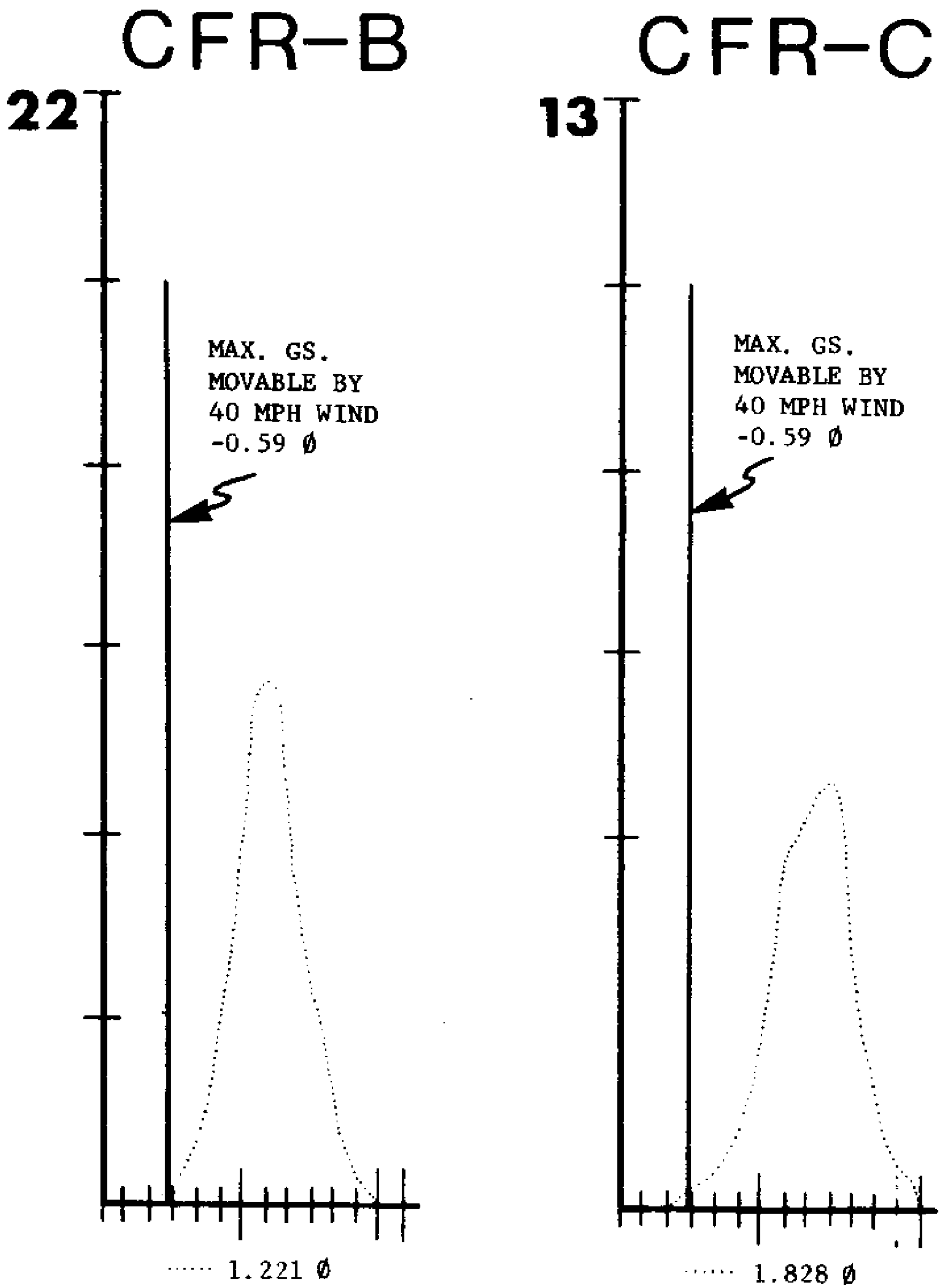
19

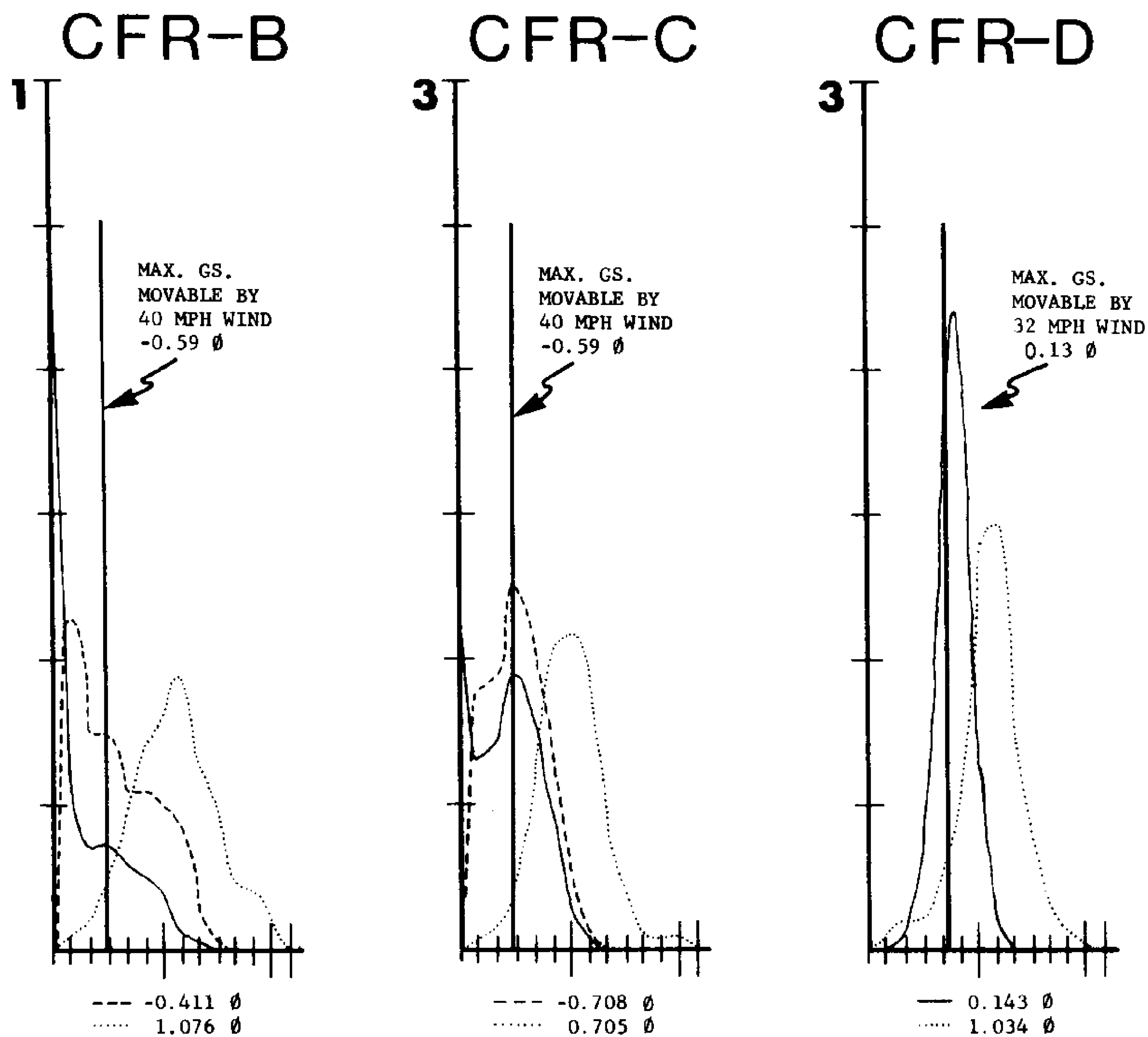


17



Note: Station Spacing is 20 ft.
Interval Downslope


CFR-B
22
MAX. GS.
MOVABLE BY
40 MPH WIND
-0.59 Ø
1.221 Ø
CFR-C
13
MAX. GS.
MOVABLE BY
40 MPH WIND
-0.59 Ø
1.828 Ø


CFR-B
1
MAX. GS.
MOVABLE BY
40 MPH WIND
-0.59 Ø
-0.411 Ø
1.076 Ø
CFR-C
3
MAX. GS.
MOVABLE BY
40 MPH WIND
-0.59 Ø
-0.708 Ø
0.705 Ø
CFR-D
3
MAX. GS.
MOVABLE BY
32 MPH WIND
0.13 Ø
0.143 Ø
1.034 Ø

erosion by surface runoff. The process of movement of coarse grains by wind creep has been discussed earlier. Briefly, Bagnold[1] observed that pebbles placed in a group became scattered across the surface by deflation of the finer grains surrounding the pebbles. This process, given sufficient time, would disperse coarse grains across the surface of a site.

The movement of material by raindrop impingement, also discussed in an earlier section, potentially is a major contributor to the total sediment movement, especially under the condition of a sloping site surface. Based upon the earlier discussions, the effect of raindrop impingement would tend to move material downslope. A finer material will be moved more readily than coarser material. The combined effect of both sediment movement by wind creep and raindrop impingement through time, therefore, would be to slowly move exposed coarse-grained material dominant on this site. The only reason that can be suggested for this, given its similarity to CFR-B, is its lower slope (5.5% for CFR-B and 4.4% for CFR-C). From Equation (8) the downslope movement of material was calculated to be 55.2% and 54.% of the total amount of material moved for CFR-B and CFR-C, respectively. This yields a 1.1% increase in downslope movement of material on CFR-B over CFR-C.

The presence or absence of a dredge effect in the depth samples taken systematically under the surface veneer of a dredged material site seems therefore to have the potential to yield important information as to the evolutionary factors which affected that site.

CONCLUSIONS

The initial sand size distribution was not uniform across the dredged material sites. The maximum grain size distributions were moveable by the maximum winds recorded for the area. The dredge effect of decreasing grain size downslope was preserved in both the surface veneer samples and depth samples of CFR-D (the youngest site). The older sites on the Cape Fear River (CFR-B and CFR-C) had surface veneers too coarse to have formed directly by wind alone and were old enough to have been affected by a combination of surface runoff, raindrop impingement and wind creep. The dredge effect was not well developed in CFR-C and reversed in CFR-D depth samples. Material from the crest was moved downslope to the flanks of the site by raindrop impingement, wind creep and runoff. A surface veneer was present on all the sites observed.

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AVAILABILITY OF SEDIMENT-ADSORBED HEAVY METALS TO BENTHIC DEPOSIT-FEEDING ORGANISMS

by

J. Frank Slowey[1]

and

Jerry M. Neff[2]

INTRODUCTION

Heavy metals are natural constituents of the aquatic environment, and are present in fresh and marine waters at concentrations usually in the range of 0.01-100 parts per billion (ppb). Very little of the heavy metals entering the aquatic environment are present in true solution. In most cases, they are associated with clays, organic matter, carbonates, insoluble oxides and sulfides, and other particulate matter. As a result, a large percent of the metals end up in the underlying sediments where metal concentrations may be several orders of magnitude higher than those in the waters. Natural background levels of most heavy metals in sediments are usually in the low parts per million (ppm) range. However, when man's activities impinge upon the aquatic environment, localized concentrations many orders of magnitude higher than these can result. This is especially true within the industrial waterways over which much of our waterborne transportation occurs.

To maintain these waterways, the U. S. Army Corps of Engineers dispose of approximately 250,000,000 cu. yd. of dredged material in the nation's fresh and

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marine waters annually. Since this dredged material often contains a wide variety of heavy metals, considerable concern exists over the potential immediate and long-range effects of these metals on aquatic and benthic organisms. At present, it is not known whether the metals present in these sediments are harmful to the organisms or even if they are available. However, these uncertainties have created sufficient concern, whether justified or not, so as to present problems in the disposal in open waters of material dredged from areas in which sediments have become contaminated with metals and other chemical substances. Understanding the effects of heavy metals upon aquatic or benthic organisms is somewhat complicated by the fact that a number of these metals at low levels are known to be essential micronutrients for the organisms. Among these are iron, manganese, copper, zinc, cobalt, vanadium, chromium and selenium. However, at slightly higher levels, many of these metals become toxic to the organisms. These toxic effects may be either immediate (acute) in nature or long term (chronic). The acute effects are usually lethal whereas the chronic may be lethal or sublethal. Sublethal effects may include poor growth, reduced longevity and impaired reproduction.

ACCUMULATION OF HEAVY METALS BY BENTHIC INVERTEBRATES

Accumulation of heavy metals by aquatic organisms is affected by a number of factors. These include availability, salinity, temperature, season, growth stage, species and stress. At present, little information exists concerning the availability of sediment - adsorbed metals in benthic organisms. Several investigators have suggested that the level of metals in these organisms reflects the metal level of sediments to which they are exposed. Mathis and Cummings (1971) studied the levels of metals in clams, tubifex worms and sediments in the Illinois River and found that the levels of copper, nickel, lead, chromium, zinc, cobalt and cadmium in both the clams and worms were closely related to the levels within the associated sediments. Drifmeyer and Odum (1975) reported significantly higher lead and manganese concentrations in the grass shrimp Palaemonetes pugio collected from ponds inside diked dredged material disposal areas compared to those collected from natural marsh areas. On the other hand, Cross and coworkers (1970) studied the relationship between trace metals concentrations of sediments with different heavy metals in a coastal plain estuary and the concentrations of metals in six species of polychaetes occupying these sediments. The concentrations of zinc, manganese and iron in the worms from different sediments varied little indicating either that these metals were regulated by the worms or that they were in a chemical form in the sediments not available to the worms. Similar results for the above metals plus zinc have been reported for the polychaete Nereis diversicolor (Bryan, 1971; Bryan and Hammerstone, 1971). However, copper levels in the worms did relate to the levels within the sediments. Tubifex worms were found to be unable to remove radioactive zinc, manganese, cobalt, iron and chromium from Columbia River sediments (Dean, 1974). Additional studies supporting both uptake and non-uptake of metals by benthic organisms may be found in the literature. Based upon these studies, there appears to be some question as to whether or not sediment-associated metals are available to benthic organisms and, if so, under what conditions.

Because of these uncertainties, the U. S. Army Corps of Engineers commissioned a study to determine availability of heavy metals to benthic organisms, particularly deposit-feeding infauna. This study had three goals:

(1) To determine availability of heavy metals to selected biota from dredged material sources,

(2) if uptake occurred, to determine if these organisms could regulate their metals levels, and

(3) to develop, if possible, a chemical extractant that would allow evaluation of future dredged material for metal availability and therefore a prediction of impact prior to dredging.

METAL LOCATION IN SEDIMENTS

In developing an extractant or extraction scheme, we needed to find one or more extractants that could identify the different physical and chemical forms of the metals present in the sediments to be dredged. As previously mentioned, heavy metals can be associated with a number of components in the sediments. These are:

1. Interstitial water - both ionic and complexed in solution
2. On normal exchange sites
3. On specific sorption sites
4. Occluded with hydrous oxides (Mn and Fe oxides)
5. In organic matter
6. As insoluble sulfides
7. In lattice structure of minerals

To determine the actual location of individual heavy metals, a number of extractants or sequential extraction schemes have been proposed (Brannon, et al., 1976; Chen, et al., 1976; Jenne, et al., 1974). Although no one method will establish with certainty the form or location of a metal within the sediments, the extraction scheme used by Brannon and coworkers appears best suited to this purpose. Briefly, this method involves the sequential extraction scheme shown in Figure 1. In addition, the present study evaluated a number of chemical extractants for non-sequential use in estimating the physical or chemical forms in which the various metals were found in the sediments. These are shown in Table 1. Of these, the first 8 provided the most useful information and are continuing to be evaluated along with the sequential extraction scheme given in Figure 1.

FIGURE 1

SEQUENTIAL EXTRACTION SCHEME

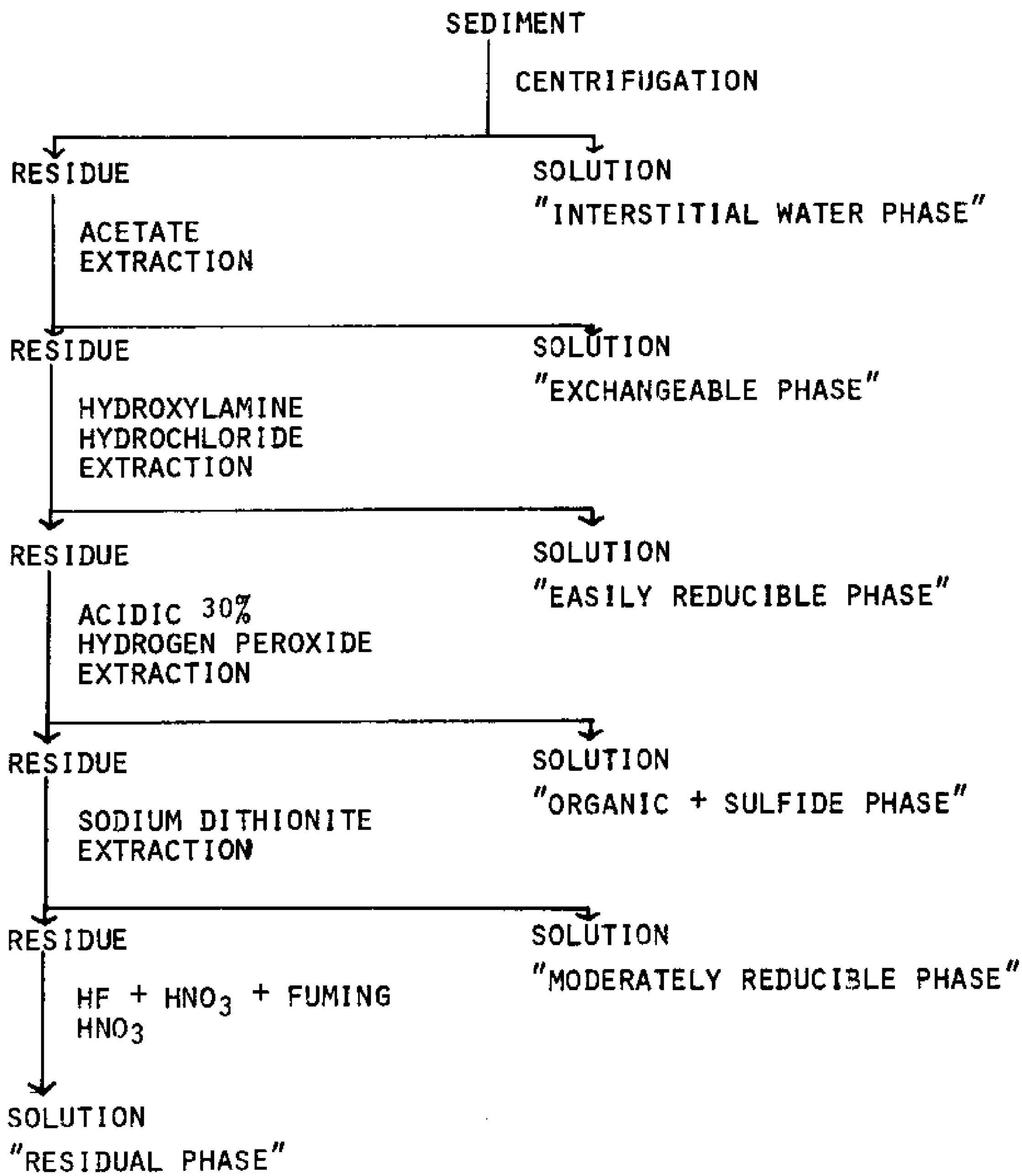

TABLE 1

Individual Sediment Extractants or Fractions Studied

1. Interstitial Water
2. Ammonium Acetate (1 N)
3. Hydroxylamine Hydrochloride (0.1 M)
4. Acidic Hydrogen Peroxide, 30% at pH 2.5
5. Sodium Dithionite - Citrate
6. Hydrochloric Acid (0.1 N or pH 2)
7. Standard Elutriate Test
8. Total by HNO_3 - HF - Fuming HNO_3
9. Distilled Deionized Water
10. 1-1/2% Sodium Chloride
11. 3% Sodium Chloride
12. Acetic Acid (1 N)
13. Calcium Chloride (0.1 M)
14. Diethylenetriamine Pentaacetic Acid (0.005 M) in Triethanolamine (0.1 M)

SAMPLE SITES

For this study, four marine sites along the Texas Coast were selected. These, shown in Figure 2, were chosen for their elevated metals levels over natural background levels (e.g. San Antonio Bay) as indicated in Table 2. One freshwater site in the Ashtabula River, Ohio, was also used (Figure 3). Of these sites, Texas City, Corpus Christi and Ashtabula proved best suited to this study. Houston and Freeport Ship Channel sediments were eliminated as exhibiting toxic effects upon the test organisms.

METHODS

Sediment samples were collected with an Ekman dredge, placed in 5-gallon plastic containers, sealed and stored in ice at 4°C for transport to the laboratory. Care was taken to minimize contact with air and to prevent possible contamination during collection and storage. Upon arrival at the laboratory, the samples were stored in refrigerators maintained at 2° to 4°C until used. Bottom water samples for the elutriate test were also taken at the sample sites using a Kemmerer water sampler. These samples were placed in precleaned one-gallon Cubitainers and returned to the laboratory for use. Before withdrawing samples for chemical analysis and biological tests, the sediments were thoroughly mixed in a large refrigerated mixer, under nitrogen, to insure homogeneity.

Laboratory uptake studies were conducted using small aquaria or chambers. Three experimental aquaria containing the test sediment and three control aquaria containing either no sediment or low background sediment, depending upon the organism, were prepared for each desired sampling interval (usually 2, 4, 8, 16 and 32 days). An appropriate amount of test sediment was placed in the experimental chambers and distilled, deionized water or artificial sea

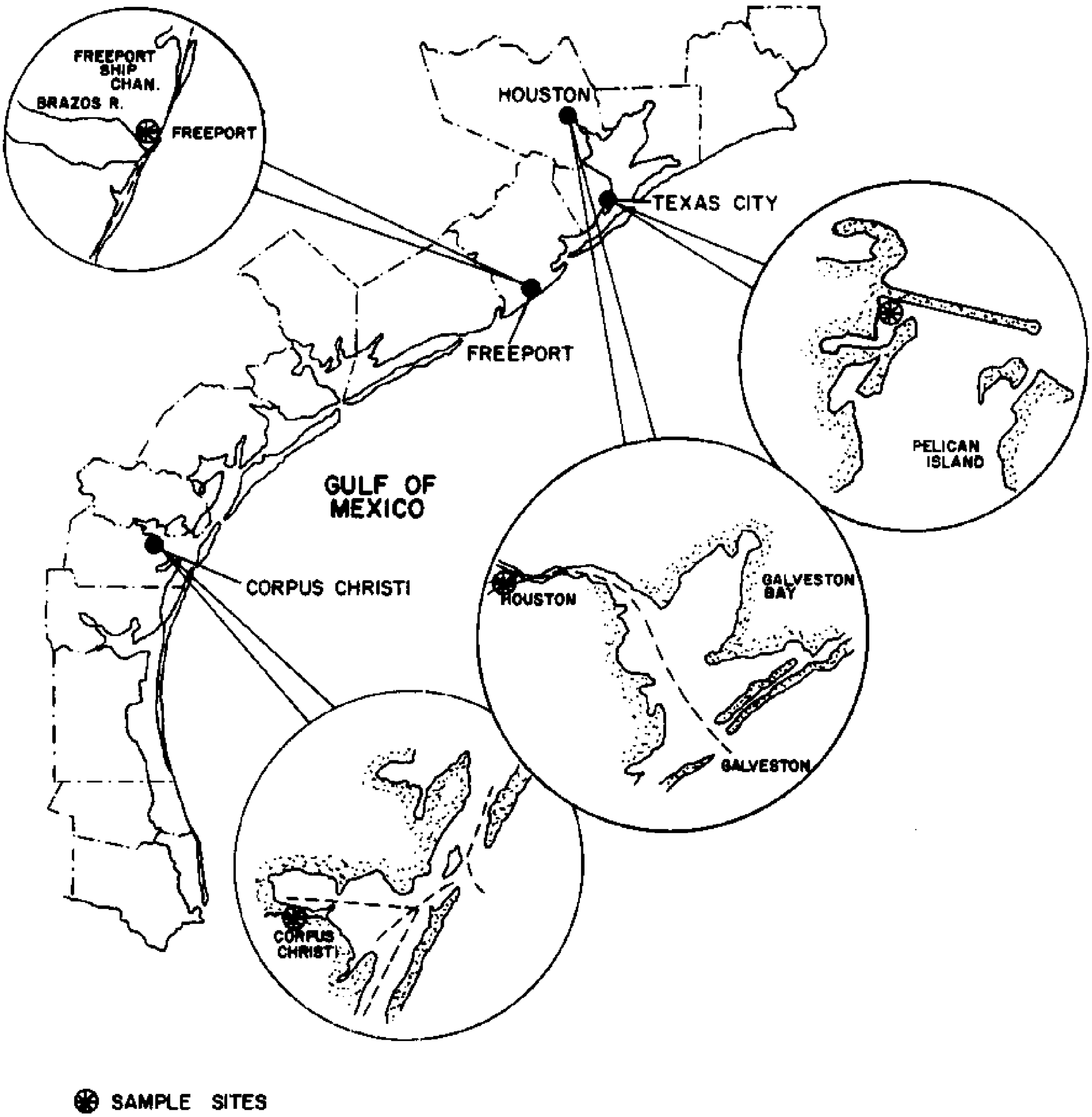

FIGURE 2

SAMPLING SITES ALONG TEXAS COAST

TABLE 2

Total Concentration of Metals in Sediments

(mg/kg-dry weight)

ELEMENT	HOUSTON SHIP CHANNEL	FREEPORT SHIP CHANNEL	TEXAS CITY CHANNEL	CORPUS CHRISTI CHANNEL	SAN ANTONIO BAY
Cu	73.0	87.0	48.0	120.0	7.9
Cr	137.0	44.0	188.0	82.0	23.0
Cd	6.1	2.1	2.4	21.0	1.1
Fe	33.5*	15.9*	14.5*	12.3*	11.6*
Ni	51.0	44.0	48.0	17.0	14.0
Mn	580.0	480.0	570.0	257.0	330.0
Pb	207.0	46.0	41.0	316.0	21.0
Zn	555.0	201.0	161.0	4055.0	39.0

*Fe in g/kg.

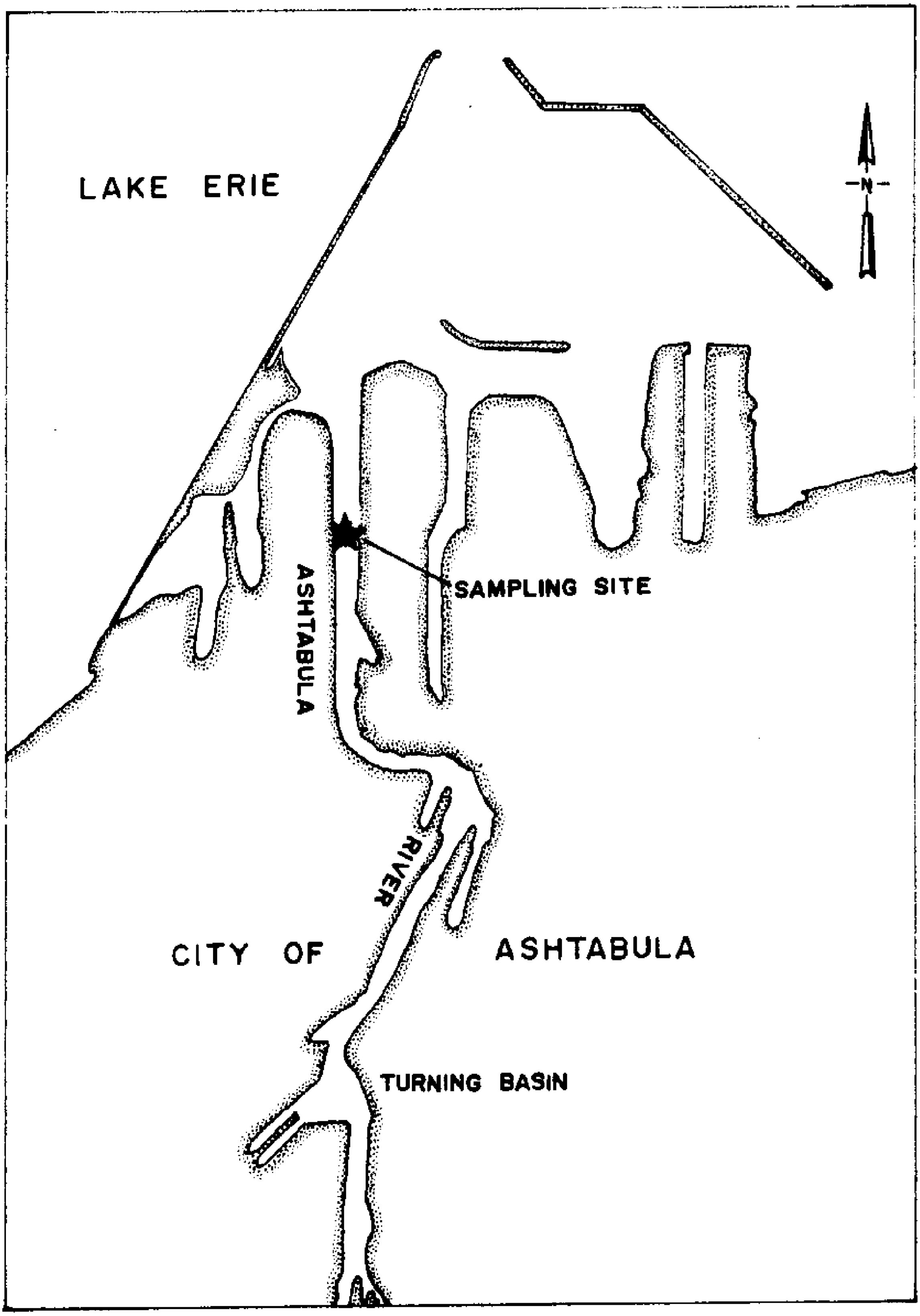

FIGURE 3

SAMPLING SITE IN ASHTABULA RIVER, OHIO

water was added along with the test organisms. The test sediments were not stirred but slight stirring through aeration was provided to the overlying water. Test organisms included the clam Rangia cuneate, the grass shrimp Palaemonetes pugio and the worms Neanthes arenacodentata and Tubifex sp. The clams and grass shrimp were placed in the same test chambers and the worms in separate chambers to prevent predation by the shrimp. At the desired sampling intervals, all test organisms were removed from the appropriate chambers and whole body metal content determined for each type of organism in each chamber using atomic absorption spectrophotometry. Where required, organism guts were purged prior to analysis and in some cases additional organisms allowed to depurate for periods up to 8 days in a clean environment to determine their ability to regulate metal content. Both the grass shrimp and clams were exposed to 0, 15 °/₀₀ and 30 °/₀₀ salinity waters to determine effect of salinity on uptake.

RESULTS

Some preliminary results of this study are now available. Results of the sequential extraction of the Texas City, Ashtabula and Corpus Christi sediments are given in Tables 3 through 5. It is evident that for all three sediments the bulk of the metals appears associated with the organic-sulfide phase, the residual mineral phase, and for some metals the easily and moderately reducible manganese and iron oxide phases. Only trace but possibly important levels of manganese and iron exist in the interstitial water and exchangeable form within the Texas City and Ashtabula sediments whereas only manganese appeared significant in these fractions in the Corpus Christi sediment. Application of the individual non-sequential extractions listed in Table 1 produced essentially the same results. Results obtained using the elutriate test which was developed for use in estab-

TABLE 3

Sequential Extracts of Texas City Channel Sediments

(mg/kg)

PHASE	Cu	Zn	Pb	Cr	Fe	Mn
Interstitial Water	0.02	0.12	<0.01	0.01	0.23	11.5
Exchangeable	0.02	0.26	<0.01	0.05	69.	182.
Easily Reducible	0.10	10.0	<0.1	4.3	2,550.	389.
Organic + Sulfide	31.	77.	29.	97.	5,056.	135.
Moderately Reducible	0.1	-	-	<5	4,250.	10.
Residual	13.	109.	7.9	69.	5,075.	82.
Sum of Above	44	196	37	170	17,000	810
Total (By Analysis)	48	161	41	188	14,500	570

TABLE 4

Sequential Extracts of Ashtabula River Sediments

(mg/kg)

PHASE	Cu	Zn	Pb	Cr	Fe	Mn
Interstitial Water	0.006	0.04	0.001	0.007	7.7	0.9
Exchangeable	0.01	0.2	0.007	0.04	397	30
Easily Reducible	0.08	60	1.8	21	5,123	335
Organic + Sulfide	33	190	37	114	19,410	218
Moderately Reducible	1.2	-	-	-	2,540	8
Residual	5.9	46	8.5	64	9,410	39
Sum of Above	40	296	47	199	36,890	551
Total (By Analysis)	37	315	42	175	27,350	450

TABLE 5

Sequential Extracts of Corpus Christi Sediments

(mg/kg)

PHASE	Cu	Zn	Pb	Cr	Fe	Mn
Interstitial Water	0.01	0.10	0.15	0.01	0.04	0.4
Exchangeable	<0.01	0.08	0.05	0.04	0.5	9.6
Easily Reducible	<0.1	117	0.6	0.14	235	108
Organic + Sulfide	88	4100	286	58	3,115	146
Moderately Reducible	29	185	6	9	1,685	15
Residual	10	145	18	21	7,495	15
Sum of Above	127	4547	311	88	12,530	294
Total (By Analysis)	120	4053	316	82	11,290	257

lishing criteria for water column effects during dredged material discharge (EPA, 1975) are shown in Table 6. Of the 6 metals studied, only iron and manganese were released to the water from sediments. This probably represents release of the iron and manganese in the soluble interstitial water phase into the water column. The kinetics of conversion of the soluble reduced manganese and iron into the insoluble oxidized form is sufficiently slow to prevent immediate removal of these 2 metals from the water phase. The four other metals studied were, for all purposes, insoluble in the interstitial water phase and therefore not released during the test to the overlying water. The exception to this was the Corpus Christi sediment in which traces of zinc and lead occurred in the interstitial water and considerable increase for both was observed in the elutriate test. Results of the biological uptake studies revealed that for the Texas City and Ashtabula sediments, the clam Rangia cuneate and grass shrimp Palaemonetes sp. took up only iron and manganese. From the results given in Tables 7 and 9 for the clams, it is apparent that even for these two metals the level taken up was not substantial in comparison to the levels existing in the control organisms. Comparison of this uptake with the metal levels in the various physical or chemical phases in the sediments suggests that since these two metals occur independently of the other metals in only two of the test categories, i.e. interstitial water and elutriate test, then these two tests best indicate the metals available to these organisms that feed at the sediment-water interface. However, metal levels observed for the clams exposed to Corpus Christi sediments, Table 9, do not suggest such correlation since no significant uptake was observed for any of these six metals. Similar results were obtained for the grass shrimp.

Results from studies involving the Tubifex worms exposed to Ashtabula River sediments also indicated no significant metal uptake. This is based upon

TABLE 6

Elutriate Test Results

(μg/1)

METAL	TEXAS CITY		ASHTABULA		CORPUS CHRISTI	
	Site water	Elutriate	Site water	Elutriate	Site water	Elutriate
Cu	20	9	8	6	9	3
Zn	44	28	85	50	325	1700
Mn	32	5800	3	550	22	890
Fe	44	52	15	650	10	20
Pb	1	1	1	1½	2	6
Cr	7	6	<5	<5	<5	<5

Table 7

Uptake of Metals by *Rangia cuneate* in the

Texas City Channel Sediments

(ppm-dry weight)

Time (Days)	Cu				Zn			
	Exposed		Control		Exposed		Control	
	$\bar{X}$	s	$\bar{X}$	s	$\bar{X}$	s	$\bar{X}$	s
2	20	±3	19	±2	85	±6	89	±5
4	24	±7	39	±2	80	±5	88	±9
8	22	±4	24	±10	97	±14	81	±13
16	30	±6	18	±1	86	±10	81	±1
32	24	±10	21	±2	75	±15	86	±20

Time (Days)	Mn				Fe			
	Exposed		Control		Exposed		Control	
	$\bar{X}$	s	$\bar{X}$	s	$\bar{X}$	s	$\bar{X}$	s
2	12	±4	14	±4	213	±74	188	±35
4	31	±5	14	±4	356	±44	193	±22
8	30	±33	16	±3	292	±39	194	±42
16	21	±7	6	±1	231	±4	159	±10
32	20	±2	6	±3	182	±97	177	±49

Time (Days)	Pb				Cr			
	Exposed		Control		Exposed		Control	
	$\bar{X}$	s	$\bar{X}$	s	$\bar{X}$	s	$\bar{X}$	s
2	0.7	±0.2	1.2	±0.3	7.9	±4.5	6.7	±2.1
4	0.8	±0.2	0.7	±0.1	5.6	±0.2	5.5	±1.4
8	0.3	±0.1	0.3	±0.2	6.5	±1.4	5.6	±0.2
16	0.6	±0.5	0.9	±0.3	8.2	±1.8	6.4	±0.4
32	0.5	±0.2	0.6	±0.3	5.6	±3.3	6.6	±0.7

Table 8

Uptake of Metals by *Rangia cuneate* in Ashtabula River Sediments

(ppm-dry weight)

Time (Days)	Cu				Zn			
	Exposed		Control		Exposed		Control	
	$\bar{X}$	s	$\bar{X}$	s	$\bar{X}$	s	$\bar{X}$	s
5	20	±4	18	±0	102	±1	90	±2
10	24	±0	18	±4	86	±1	98	±4
15	19	±2	25	±6	98	±2	87	±2
20	40	±12	16	±1	95	±19	97	±0

Time (Days)	Mn				Fe			
	Exposed		Control		Exposed		Control	
	$\bar{X}$	s	$\bar{X}$	s	$\bar{X}$	s	$\bar{X}$	s
5	14	±3	20	±12	780	±60	350	±20
10	12	±1	5	±1	635	±15	235	±30
15	15	±6	6	±2	630	±300	210	±20
20	34	±12	6	±2	2110	±750	290	±70

Time (Days)	Pb				Cr			
	Exposed		Control		Exposed		Control	
	$\bar{X}$	s	$\bar{X}$	s	$\bar{X}$	s	$\bar{X}$	s
5	1.5	±0.7	1.6	±0.0	7.3	±1.2	5.6	±0.4
10	1.2	±0.1	1.0	±0.0	5.6	±0.0	5.0	±0.1
15	1.2	±0.6	0.9	±0.2	9.2	±2.2	5.2	±0.8
20	3.0	±0.2	0.8	±0.2	11.0	±3.8	5.9	±0.6

Table 9

Uptake of Metals by *Rangia cuneate* in

Corpus Christi Sediments

(ppm-dry weight)

Time (Days)	Cu				Zn			
	Exposed		Control		Exposed		Control	
	$\bar{X}$	s	$\bar{X}$	s	$\bar{X}$	s	$\bar{X}$	s
2	16	±2	18	±1	50	±25	57	±5
4	16	±2	17	±1	83	±17	59	±3
8	16	±5	18	±0	69	±0	62	±2
16	27	±3	15	±1	62	±4	56	±3

Time (Days)	Mn				Fe			
	Exposed		Control		Exposed		Control	
	$\bar{X}$	s	$\bar{X}$	s	$\bar{X}$	s	$\bar{X}$	s
2	3.3	±1	15	±6	172	±21	152	±13
4	10	±2	13	±3	211	±52	155	±18
8	8.2	±1	14	±1	179	±3	166	±11
16	10	±4	15	±4	151	±4	158	±14

Time (Days)	Pb				Cr			
	Exposed		Control		Exposed		Control	
	$\bar{X}$	s	$\bar{X}$	s	$\bar{X}$	s	$\bar{X}$	s
2	0.2	±0.1	<0.1	-	3.4	±0.6	3.5	±0.1
4	0.2	±0.1	<0.1	-	4.0	±0.1	4.3	±0.1
8	0.2	±0.1	<0.1	-	4.6	±0.4	2.8	±0.2
16	0.3	±0.1	<0.1	-	3.3	±0.2	3.2	±0.1

analysis of test worms which had their guts purged for one day prior to analysis. Table 10 shows a comparison of the gut-purged and unpurged exposed worms at 4-day exposure versus control worms. This provides a good indication of the problem encountered in interpreting literature uptake data for which no mention is made as to whether or not the organisms have been purged. What appears as uptake may only be unpurged sediment within the gut and assumptions of metal availability may be misleading. For the marine sediments, the worm Neanthes arenacodentata was used. Worms exposed to the Texas City sediments did not appear to have any significant uptake of these six metals. However recent studies with Corpus Christi sediments showed uptake of both lead and zinc. Additional studies are underway to verify this.

In conclusion, it appears that, for the clam Rangia cuneate and shrimp Palaemonetes sp., our preliminary results suggest that only those metals that are soluble within the interstitial water and readily released to the water column appear to be available to these sediment-water interface feeders. In the sediments studied, these metals were iron and manganese, two metals with little if any adverse effect upon benthic organisms or the higher food chain. Studies with true deposit-feeding infauna such as the Tubifex and Neanthes worms indicate that for sediments from several marine and freshwater areas little if any uptake was observed if the organisms' guts were purged prior to analysis.

Based upon results to date, it is not possible to arrive at a simple extraction procedure that will predict the availability of heavy metals to benthic feeding infauna in general. Furthermore, sediments containing high total metal burdens do not appear to necessarily have an abundance of the metals available suggesting bulk metal analyses are not useful criteria for evaluating environmental impact of dredged material upon benthic organisms.

Table 10

Uptake of Metals by *Tubifex sp.* Exposed to

Ashtabula River Sediments for 4 Days

(mg/kg)

Metal	Control	Exposed	
		Unpurged	Purged
Cu	12 - 18	44	13
Cd	0.1	1.1	0.1
Cr	<2	38	<1
Fe	620 - 900	5565	945
Mn	12 - 22	188	19
Pb	2 - 4	25	4
Zn	220 - 240	220	207
V	<1	89	<1

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"FEASIBILITY OF DEVELOPING BIOLOGICAL HABITATS ON DREDGED MATERIAL"

by

Hanley K. Smith[1]

ABSTRACT

The feasibility of marsh, terrestrial, island, and aquatic habitat development on dredged material has been proven, and offers a wide range of disposal options. Marsh and terrestrial habitat development on dredged material are the best understood of these disposal alternatives and the state of the art is advanced to the point that the major engineering, economic, and biological constraints are recognized. Dredged material islands are know to provide essential habitats to many species of shorebirds, and the technology needed for the creation and management of these islands is rapidly being developed. The development of aquatic habitats, such as seagrass beds and oyster flats, on dredged material is the least understood of the disposal alternatives and needs substantial study before it is implemented on a large scale.

INTRODUCTION

Habitat development provides an alternative disposal technique that offers the potential for disposal of large amounts of dredged material and simultaneously establishes a desirable biological system. Four biological communities have been identified by the Corps of Engineers' Dredged Material Research Program (DMRP) as being particularly suited for establishment on dredged material. These are marsh, upland terrestrial, island, and aquatic habitats. Each of these habitats is discussed below in terms of its environmental, engineering, and economic feasibility.

MARSH HABITAT DEVELOPMENT

Dredged material placed intertidally or on wet non-tidal soils has frequently developed into desirable marsh habitat. In some cases this was planned habitat creation but more often it was simply fortuitous. The three

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basic requirements for marsh establishment are containment and stabilization of the substrate, attainment of a stable elevation within the tidal zone (generally the upper third of the tidal range), and reasonably low hydrologic energies. In general, fine-textured substrates are most productive; however, many marsh plants will thrive on sandy material and proper fertilizer application will enhance their growth response. The necessity for containment of the dredged material is a function of wave energies and the texture of the material. Sand placed in a low or intermediate energy situation may be quite stable. On the other hand, hydraulically placed silts and clays require some type of containing structure even in a low energy condition. The problem of wave energies can be minimized by placement of the dredged material in a naturally protected area or behind a breakwater structure.

The primary environmental concerns in marsh development are the potential for contaminant uptake and the loss of the habitat replaced by the marsh. It has been demonstrated that marsh plants can absorb heavy metals from polluted dredged material. Release of these contaminants into the environment is possible through detrital pathways or through direct ingestion. Preliminary surveys with saltmarsh species growing on dredged material indicate that heavy metal uptake is not a widespread problem, and should be of major concern only on highly polluted sediments. Similar data are not available for freshwater marsh plants, and at this point careful analysis of each site would be prudent.

The development of a marsh using dredged material as a substrate involves the replacement of one type of habitat with another. Often the habitat to be destroyed is a productive shallow water community, and the loss of this community must be considered in the assessment of feasibility. This concern is further compounded by the fact that meaningful comparisons between the relative values of marsh and shallow water habitats are highly subjective, and generally necessitate a best guess by local marine and estuarine biologists.

The major engineering concerns regarding marsh development revolve around the need for retention and protection of the dredged material. Requirements are highly site-specific and are generally dictated by the texture of the dredged material, wave and current energies at the site, and the soil foundation at the site. In a sheltered, low-energy situation using sandy dredged material, marsh development may involve no more than pumping the material up to the desired elevation. If the material is fine-textured, a retaining structure may be needed; if the energies are high, a protective structure may be required. If the retaining or protective device is to be a dike or similar item, the site foundations must be sufficient to support the structure. This is not to say that marsh development is infeasible using fine-textured material in high-energy conditions or on weak foundations, rather that it is more difficult under these circumstances or combinations of circumstances. In fact, productive freshwater and saltwater marshes have been established under all of these conditions.

The economic feasibility of marsh development is largely determined by the design and construction requirements of the site. Factors which are predominately influenced by engineering considerations have been discussed above. Some habitats, such as oyster beds, which might be replaced by marsh creation may have direct and quantifiable economic values. The social and political effects of establishing emergent vegetation on previously submerged land are highly site-specific but may involve esthetic perceptions, real estate values, and issues of land ownership. Such social concerns must also be part of the feasibility equation.

TERRESTRIAL HABITAT DEVELOPMENT

Upland terrestrial disposal sites include extratidal confined or unconfined dredged material deposits. Coarse material deposited on upland sites is frequently unconfined whereas containment is usually necessary for the upland disposal of fine-textured material. Upland coarse-textured material is typically infertile and droughty, and habitat development on these sites may require organic enrichment, fertilization, and careful selection of plant species and planting season. Fine-textured material often provides an excellent growing medium for plants, although improvement in drainage and adjustment of soil acidity may be necessary.

The concept of terrestrial habitat reclamation is well within the present state of the art and generally requires only the application of standard agronomic and wildlife management techniques. The most critical aspect of the implementation of terrestrial reclamation is the selection of a target plant community that will thrive at the site. Unless continuous management is acceptable, the target community should be capable of successfully competing with invading plant species. With sufficient effort virtually any habitat can be established; however, it is usually most efficient to select a low maintenance, somewhat self-perpetuating community. In most areas of the United States the necessary resource personnel and plant materials to implement this alternative are readily available. Likely information and material sources would include state agricultural extension services, fish and game agencies, private nurseries, and local universities.

The potential for environmental damage is a function of the value of the habitat displaced by the dredged material as compared to the value of the newly created habitat. Terrestrial habitat reclamation, when considered separately from the potential damage of initial disposal, presents relatively few environmental problems if reasonable attention is given to the capabilities of the site and the needs of the area. One exception appears worthy of attention. Highly polluted dredged material may result in the accummulation of contaminants in plant tissues and these contaminants may be subsequently released into the environment through animal consumption of these tissues. Sufficiently little is known regarding these pathways to warrant concern where high levels of contaminants are known to be present.

Assuming that the construction of the disposal site is complete, terrestrial reclamation will not generally present any extraordinary problems, as preparation of the site normally involves such standard items as tilling to incorporate lime or fertilizer, and ditching to improve drainage. Site-specific problems such as the use of heavy equipment on unconsolidated deposits or transporting of equipment to remote locations are commonly encountered, but these constraints are of an economic nature.

The economic feasibility of terrestrial habitat reclamation is very site-specific and will depend largely on the size and remoteness of the site and the selection of the vegetative community. A low-management habitat established on an easily accessible, well-consolidated site using readily available plant stock could cost as little as $300 per hectare.

ISLAND HABITAT DEVELOPMENT

As historic nesting grounds have been lost to development, some dredged material islands have become exceptionally important breeding areas for wading birds and shorebirds, including gulls, terns, egrets, herons, ibises, and pelicans. These islands are generally sandy mounds located in shallow water near a dredged waterway, and their value to wading and shorebirds is derived from the fact that many nest on the ground in large colonies and the island provides a relatively predator-free environment. In virtually all cases this form of habitat development has been unplanned. The total significance of this resource is not completely understood as the extent of usage has not been defined for most regions, many islands are not used by birds, and the relationship between the location of the islands and other essential habitats, such as feeding areas, is not known.

The environmental impacts of island creation are primarily defined as the loss of productive bottom or marsh communities. Again, the determination of which habitat is the most valuable from a biological standpoint becomes an issue that must be resolved by local authorities. Dredged material islands may change existing current patterns and thereby influence local salinities, deposition rates, oxygen levels and temperatures, and the potential impacts of such changes should be assessed.

Diked, or confined, dredged material islands are subject to the interrelationship of hydrologic energy, material texture, protective and retaining structures, and soil foundations which were discussed above under marsh development. The existence or longevity of unconfined disposal islands is determined largely by wave and wind energies and material texture. Such islands usually exist in relatively shallow, low-energy areas, and consist of sifted sand, shells, and gravel. Periodic disposal of dredged material is often necessary to replenish the island structure and maintain an early stage of vegetative succession. Erosion is a serious problem on many dredged material islands and in many situations stabilization of the high-energy surface of an island would greatly reduce erosion. The maintenance of dredged material islands as both disposal sites and nesting habitats requires a knowledge of the location of colonies and the breeding behavior of the species involved. In general, colony locations are predictable from year to year, the dates of breeding activity are known and only relatively few of a group of islands or a small portion of a specific island will be selected as colony sites. Conflicts between disposal area needs and bird use can be avoided by disposing on unoccupied areas or scheduling disposal after the breeding season.

In most cases, dredged material islands are created by open-water disposal just outside a navigation channel, and such placement of material is the least expensive disposal alternative. If the island is stable and emergent during spring high tides, it is immediately available as nesting habitat for some bird species. Essentially no economic restrictions should surface at this stage. If management, in the form of structural stability, scheduling disposal, or advancing or retarding vegetative succession, is introduced, then additional costs should be anticipated. Such expenses would obviously be highly site-specific, but, in general, island habitat management should be a low cost item.

AQUATIC HABITAT DEVELOPMENT

Aquatic habitats are those shallow water areas near or below mean low tide, and are usually within the zone of light penetration. This includes communities such as grassbeds, and oyster and clam flats. The concept of aquatic habitat development is that by disposing of dredged material on the bottom of a deep-water body that bottom could be elevated into a biologically more productive zone. This concept has been tested at several sites but is far behind the other disposal alternatives in terms of determination of environmental effects, engineering design, and economic evaluation. Although the concept appears highly promising, the state of the art is such that implementation of this disposal alternative should be confined to small projects or experimental sites.

SUMMARY AND CONCLUSIONS

Habitat development on dredged material is feasible in a variety of situations and offers a viable alternative to more conventional forms of disposal. Of those alternatives discussed, marsh and terrestrial habitat development are sufficiently understood to permit careful environmental, engineering, and economic analysis. The major environmental concerns in marsh and terrestrial habitat creation are the determination of the relative biological values of new and replaced communities, and the potential of heavy metal uptake from polluted material. Marsh habitat development requires the maintenance of a balance between site energy and foundation conditions and the texture of the dredged material. Terrestrial habitat development entails fewer engineering constraints than marsh development, and in most situations the techniques required will be well within the existing technology.

Island habitat development holds significant potential for the creation of shore and wading bird nesting areas. Scheduling and placement of material in a manner which is non-conflicting with bird use will usually require a minimal or no-cost output. The optimum condition of a well-managed wildlife habitat and an efficient disposal area is a feasible and practical disposal alternative.

Aquatic habitat development is a concept which appears to offer both the potential of high-volume disposal and increased biological productivity. However, this alternative remains in an experimental stage and is not now recommended as an alternative for disposal of large quantities of dredged material.

MINING OF PHOSPHATES BY DREDGE

By

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Several commercially viable phosphate deposits in North Carolina, Georgia and Florida offer potential for mining by cutter suction dredges. Where deposit conditions permit, dredges offer several potential advantages over present dragline techniques. Principal among these are that the water table need not be disturbed, that reclamation is an integrated part of the operation, and soil stability problems are circumvented.

Erickson Engineering has assisted a number of phosphate mining companies in assessing the practicality of mining by dredge. The owner of the first large-scale phosphate reserve to be dredged is presently in the process of obtaining its operating permit. Some observations may be of interest.

The phosphate values are contained in a matrix typically consisting of sand, pebble and clay. Mining requires the removal of overburden strata before the matrix can be removed and processed. The deposits which are economic in today's market will generally have between one and two parts overburden to one part matrix. Phosphate rock usually represents 6 to 9 percent of the total soils removed. It has a low value prior to processing and requires that both excavation and land restoration methods be highly efficient.

Mining of these deposits has traditionally been by use of large draglines working from the original grade level. The overburden is first removed and cast aside; the matrix is then mined with the same machine casting this into another pile, where it is slurried by hydraulicing. The slurry is then pumped to the processing plant. In some instances the operation will employ two draglines, one for removal of overburden and one for mining the matrix.

Phosphate rock or simply rock, as it is called in the trade, is a misnomer. Historically, when phosphate was first mined in Florida late in the last century it was pebble or gravel-like in size. It was recovered from the matrix by scrubbing and screening. These early mines also contained significant amounts of phosphate values in sand-sized particles, but these could not be recovered because there was no technology which could be economically employed. The sand-sized values were disposed with tailings.

In subsequent years, as the prime pebble deposits were being depleted and the market for rock increased, flotation methods were developed to recover the fine values. Today most operations recover both pebble and float product.

The matrix which reports to the plant is first screened, the pebble is recovered and the remaining soils degraded into sand and clay size particles. The sand size goes to the flotation cells, and the clays or slimes, are hydro-cycloned off and pumped to a settling reservoir. Eventually all the materials except the phosphate product are returned to the mine property where they must be incorporated into a land restoration program.

Reclamation would be a rather mundane task were it not for the clays. Because the overburden is presently handled dry there is no degradation of the clays contained therein, and consequently little growth of volume. However, the matrix will frequently contain over 30% clay, which is slimed and mixed with water in the processing. The nature of the clays will vary widely. In central Florida, Polk and Hillsborough counties have a type clay that makes a tight bond with the process water, swelling to 4 to 5 times original volume. This combined with the tailings and overburden will occupy perhaps 25% more volume than the unmined deposit. One of the major objections to removing overburden by dredge is the fact that the clays contained would be to some degree slimed and reclamation and disposal problems compounded.

The Lee Creek Mine in North Carolina has recently embarked upon a dredging program for the upper 40 feet of overburden. Through a complex pattern of dredging and locking from one dredge pond to another, and draining of the dredged areas, it is planned to locate the draglines at a lower bench level to improve efficiency. The upper stratum of the overburden in this instance is largely sand and it is believed that there will be no problem with the disposal of clay.

The owner of the property contiguous to the Lee Creek Mine investigated the use of dredges as a means of removing overburden; however, the higher clay content and the five miles to the disposal area made dredging impractical with present economics. This company's present plan is to use bucket wheel excavators.

There are rich phosphate reserves in the tidal flats south of Savannah which have been studied by several companies. These are presently inaccessible because of local opposition to mining, but will likely some day be mined. Here, there is no practical way to mine but by dredge. The clays are rapidly settling and present little problem of volumetric growth. The surface soils cannot support heavy mining equipment even if the area were diked and drained.

In Florida the Polk and Hillsborough phosphate reserves are becoming depleted and the industry is moving south into Manatee, Hardee and Sarasota counties. The geology of these new reserves generally differs from those to the North. There is less pebble, and most of the values would be recovered as flotation concentrate. The clays are different in that they are more rapid settling and trap less water. There is also generally less clay in both the matrix and the overburden. The matrix lies deeper. The soils are more permeable and the water tables higher. The soils are generally loose making it difficult and hazardous to support draglines close enough to the pit for convenient and economical operation.

Large-scale phosphate mining began in central Florida where the mines were initially shallow. Today's mines in this area average perhaps 60 feet deep. The Lee Creek Mine in North Carolina and the adjacent areas have deep overburden and a thicker deposit of matrix. These deposits bottom out at 150 feet or more. The deposits in coastal Georgia are about 90 feet deep. The future Florida mines south of the existing mining area will range to 95 feet or more. As the mines get deeper, dragline sizes and costs increase for equivalent production. There is another problem which exists at all mine sites to some degree, and that is soil stability to support these massive machinery loads. It is not unusual for the dragline lines to slip into the pit, and this is an ever present threat.

Another aspect of dry, open-pit mining is the fact that the mines must be dewatered by well points, ditches, etc. All other things equal, the deeper the mine the greater the dewatering problem. In some instances there is little effect on the adjacent water tables. In other instances, notably the Lee Creek Mine, dewatering can influence the water table for a radius of many miles.

All phosphate mines require vast amounts of water in processing, however, most of this is recirculated and reused. The only real need for make up water is for the flotation process and this is but a small percentage of the total in circulation. Whether or not public concern of water usage is justified in the individual situation, it is nevertheless there and it causes many permitting problems. The point is that a dredge pond need not disturb present water tables or draw down adjacent properties.

These factors have notivated mining companies to investigate dredging as an alternative to mining by dragline. The scale of mining phosphate by dredge will be large. A typical mine might require overburden removal at a cyclic rate of perhaps 4000 cubic yards per hour, with a time utility requirement in the order of 70%. It is independent of any requirement other than to remove overburden ahead of the matrix dredge. Its production rates can vary through a wide range without affecting mine operations, if it averages adequate production.

The requirements of the matrix dredge are more severe. It might be required to produce an average of 1500 cubic yards per hour. It delivers slurry directly to the washer, which is the first stage in the processing. Required pumping time is approximately 7500 hours per year. The size of the production precludes the use of surge facilities between the dredge and the plant; therefore, production rates can not exceed the overcapacity built into the washer, which may be in the order of 30 percent. It is seen that this dredge must operate continuously within a rather narrow range of production rates.

Booster pumps will be required in the matrix delivery line since it is not unusual to be working several miles from the processing facility. Any breakdown of the booster pumps, washer or dredge will detract from mining and processing time as all these elements are in series. Mechanical reliability and a first class planned maintenance program are essential. There are several hydraulic dredge mining operations which achieve this time availability, as do the present dragline operations.

Pipeline dredging consists of two major functions, first pumping and second, feeding solids to the suction. Either may limit production, both in quantity and in constancy. Pumping slurry is no longer a black art; it is reasonably predictable. If equipment design is flexible, adjustments for minor miscalculation of pump efficiency and pipeline friction losses are easily made in the field.

Feeding the pump suction is a different ball game. It involves almost every other piece of machinery and structure of the dredge. Unfortunately there is no broadly accepted quantative measure which the industry has developed which can be used as the basis of design in a given range of deposit conditions. The only concensus of those experienced in the industry is that large design safety factors are required if reliability is to be achieved.

The nature of the soils to be dredged is obviously going to affect the design of the dredging equipment. Nature's options are diverse and they are well exercised. The fact that a constant sustained production is not required of the overburden dredge allows more leeway in design and operation. Its only requirement is that it average enough production to stay ahead of the matrix dredge.

On the other hand, without adequate design, the requirement of a relatively constant output from the matrix dredge can be demanding of both the operators and the equipment. Where the matrix is loose, and sloughs freely to the suction, constant production is more easily obtained. However, many matrix bodies have frequent inclusions of clay-bound lenses as well as occasional limestone. These formations present conditions which must be adequately accommodated in the dredge design. Matrix dredges must be capable of digging effectively in both directions, evenly feeding material to the suction without reliance on natural sloughing.

The patterns of soils deposition are also of considerable interest when designing a dredge for use in mining. Navigational dredging normally involves the excavation of flat-bottomed prisms. However, phosphate overburden and base formations are seldom level or flat. Some of these have gradients in excess of 5 percent. Even though this may not be a deposit norm, it is a feature which should be considered in design. At this gradient the cutter swinging through a 250-foot horizontal cut would overdredge or underdredge 12 feet on the extreme edge of the cut. This would either waste matrix reserves, or introduce non-productive sands into the flotation cells which consume expensive reagents. The interfaces must be closely dredged to optimize economics.

Accurate dredging of the interfaces first requires that their location and gradients be established by sufficient borings. Bore hole information can be located on the horizontal plane by survey. The field information can then be recorded on computer tape organized along the path of the dredging. A dredge-mounted computer complete with programed tapes would continuously receive cutter coordinate data for an electronic triangulation system. The output from the computer would indicate the required cutter depth at any point of the swing and could automatically position the cutter elevation. Similar systems are in use on cutter dredges, lacking only the variable depth control which is not needed for navigation dredging.

Phosphate has a slightly characteristic color compared to the ambient soils. Present dragline practice in placing the bucket relies upon the sight of the operator, who must operate day and night, in the rain and other conditions of limited visibility, sitting perhaps 100 feet or more from the pit bottom. It seems reasonable that well-controlled dredging can be at least as accurate as the present dragline techniques.

The depth of both overburden and matrix in most deposits is sufficient to require suction assistance for the dredge pump, otherwise economical slurry concentrations can not be obtained. Inadequate concentrations require larger pipelines, more power consumption and larger dredges. In phosphate mining costs do not stop here since excess water must be recirculated and the plant construction and operating costs are substantially increased when the slurry density is too low. Average densities for both dredges should exceed 1300 grams per liter. This is roughly equivalent to 30 percent solids measured in situ.

Unless the solids can be introduced into the dredge suction at adequate rates of concentration, all else in the dredging process is for naught. Today the prevalent use of pumps on the ladder and jet booster systems have eliminated the historic production limitations set by atmospheric pressure. The present limiting factor is feeding the suction. Mechanically this involves at least cutter design, cutter attack angle, cutter power, swing power and speed, and adequate swing or cut width. Lost time setting ahead also enters the equation. This can be minimized by wide cuts and with more efficient spudding systems such as the traveling spud carriage which is incorporated on most of the better dredges of European design.

Draglines suffer little problem when the character of the matrix changes from sand to clay to light rock. However, a dredge produces by feeding the suction large quantities reduced to small sizes by the action of the cutter.

Every deposit is geologically somewhat unique and there can be wide variances within any one mining area. The overburdens can contain clay lenses and hardpan but are generally more sandy in nature. The matrix generally contains more clayey sands and sandy clays arranged in indiscriminately located lenses. In some instances the matrix is sufficiently loose to slough during the dredging process, but generally this is not the case. For the most part matrix must be fed to the suction mechanically.

Phosphate mines typically cover several thousand acres. The elevations of the interfaces can vary appreciably in a single deposit. For example, the matrix dredge may be required to dredge effectively from an elevation of perhaps 20 feet through 90 feet. It is the consensus of professional dredging contractors worldwide that ladders and cutters function most efficiently when the cutter axis is inclined between 25 through 45 degrees from the horizontal. Shallow angles cause persistent problems with the material spilling behind the cutter, interfering with the ladder swing and starving the suction among other things. Sectional ladders or some other means of altering the cutter inclination must be employed to prevent operating at extremely flat cutter angles.

Each mine will be to some degree unique. Dredging may not be practical or even possible, and in other instances it will offer significant advantages. Most deposits will require dredges of unique design. A contractor's dredge is a compromise machine, generally built to accommodate a variety of dredging situations. Performance and reliability are frequently sacrificed to obtain flexibility at low initial cost. On the other hand, mining dredges should be high efficiency, special purpose equipment designed to the specific conditions of each deposit and the field operations in general.

Finally, a mine operator faces the problem of selecting dredge equipment. To the newcomer there may seem to be an industry credibility gap. On one project three dredge manufacturers were asked to submit proposals for dredging equipment, given rough production requirements and deposit conditions. The initial proposals ranged from nine dredges operating on anchor wires to two dredges working off spuds, just to mention the more significant features.

None of these proposals conformed to the design concepts which were presented by several different dredging consultants. This curious situation was apparently caused by the fact that the dredge manufacturers were less interested in understanding the real needs of the project than in selling their standard designs and equipment components. Perhaps the poor responses were in part due to the mine owner's lack of explicitness when soliciting proposals. The vendors' self interest and the owners unfamiliarity with the nuances of dredging explain this phenomenon. Following subsequent revised proposals and lengthy negotiations, the owner made its purchase decision based upon designs more appropriate to the project.

Every mining situation is unique and presents unique equipment requirements. Dredging is no different.

TECHNIQUES FOR REDUCING TURBIDITY WITH PRESENT DREDGING PROCEDURES AND OPERATIONS

by

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Consulting Engineer

ABSTRACT

Techniques for reducing turbidity associated with present dredging procedures and operations fall principally in the categories of the cutter, ladder, suction, hull, pipeline, connections, barges, tenders, personnel, inspection, contracts, plans, and specifications. These techniques consist principally of good dredging procedures already known but not always followed by dredging contractors and their personnel. When these techniques are consistently applied, not only will dredge-induced turbidity be reduced, but economical operation will prevail in most instances. Other techniques for reducing dredge-induced turbidity tend to increase dredging costs and should be used only when necessary. Dredge-induced turbidity is normally apparent only in the immediate vicinity of the dredge plant and the levels of this turbidity are not usually as high as those created by open-water disposal of the dredged material. In addition to applying good dredging techniques to reduce turbidity, better inspection is needed on Corps of Engineers (CE) and CE-related projects. More CE supervision of dredging operations needs to be implemented. More training is required for inspectors, whether CE or private-company personnel. Consideration should be given to a nationwide school or short courses where dredge personnel could obtain basic technical knowledge of

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dredging. Contracts should clearly and uniformly specify requirements for turbidity-reduction measures and measurements. Contracts should be written to include smaller dredges. Where turbidity is a problem, dredging should be accomplished when natural background levels of turbidity are high.

INTRODUCTION

I'm John Huston from Corpus Christi, Texas. For the past 35 years or so I've been connected with dredging in one way or another.

Today I'm going to talk with you about a study we have just completed for the Corps of Engineers, and about which we presented a paper at this year's WODCON - - my talk to you today being a summary of that paper and the final report. It concerns some of the ways dredge-induced turbidity can be reduced - - ways which are actually just good dredging techniques, which, if practiced continually, will result not only in turbidity reduction, but efficient dredging.

First, though, a little background on the reason for the study.

Over the years, as the sediments in many of our waterways and harbors have become polluted, concern has grown that disposal of dredged material, and consequently dredging, adversely affects water quality and aquatic organisms.

My first involvement with turbidity was about 20 years ago. I had a job shut down because a few oystermen said we would kill all the oysters. I've been involved with it more and more ever since.

Today it's not just a few oystermen, but a veritable army of objectors. Among them are the environmentalists who not only view with alarm the water flowing into and out of a material disposal area, but the array of equipment producing it.

Most, never having been near a dredge, much less on one, are generally dismayed just by the array of equipment. The dredge plant, appearing as a gloomy, black mass on the horizon, presents to them an appalling picture. Not knowing of what it consists (what they don't understand, they distrust), they have developed an attitude that this monster is creating an ecological hazard, and is stirring up and depositing muck all over the countryside. Where they do have the opportunity to observe an operating dredge at close range and see the turbidity that is sometimes created, they become further appalled.

So, notwithstanding that nature can create more turbidity than all the dredges combined, turbidity has come to be a whipping boy.

Greatest among dredging's turbidity causes is the discharge from disposal areas, both confined and in open water. However, another turbidity source is often that of the dredge plant itself during the actual dredging operation.

Consequently, the Corps decided to investigate the causes of dredge-induced turbidity and attempt to determine ways of reducing them. This was the task we were assigned.

Since over 90 percent of CE and CE-related dredging is accomplished by pipeline and hopper dredges, our study focused only on hydraulic dredging operations. Furthermore, we did not look further than the discharge of the plant - - the end of the pipeline or the hold of the hopper. Neither did we include an assessment of new dredging equipment, nor the modification of that which exists.

One of our primary goals was to investigate operational techniques that could reduce turbidity. Some of them were cutter revolution rates, depth of cuts, different stepping and operational methods, operator training, and precision and timely dredging. In considering these, we gave attention to the cost, effect on production, ease of implementation, and maintenance of acceptable dredging operations.

Two other intents of our study were to present to the CE possible measures for reducing turbidity, and to point out to dredge personnel the problems turbidity creates in the public sector. We were also concerned with new contractual regulations and inspection programs, which might further the reduction of turbidity in CE and CE-related projects.

In all of this we attempted to indicate our findings in a report that was couched in the language of the dredgeman - - and a report that could be used, not filed away. For, after all, the dredgeman was to be the audience, and needs something practicable, not theoretical. For others, not fortunate in having a knowledge of dredging - - or the vernacular - - an abridged glossary of terms was prepared.

Today turbidity has become an important environmental parameter. However, how it should be defined or measured is confusing. Standards provided by most government agencies are ambiguous, to say the least. As a result everyone tends to establish his own definition for turbidity - - usually coming up with one which satisfies only his personal needs.

We believe current turbidity standards are practically and environmentally unrealistic - - particularly for dredging. Most of them are based upon a hypothesis that turbidity, like prostitution, is any departure from purity. In dredging it has to be considered as a departure from the existing, not the pure.

In assessing our study we concluded that those most cognizant of turbidity generation by dredge plants would be dredgemen themselves. Who else would be more aware of turbidity generation than men directly associated with dredging?

Consequently, our first efforts were devoted to contacting as many dredge-connected people as possible. We talked personally with approximately 50 dredgemen. Also, more than 600 members of the World Dredging Association were approached by an inquiry letter. We asked that they respond on any means they might know of to reduce dredge-induced turbidity.

It was interesting to note that the oral and letter information we received suggested that only United States' dredging personnel seemed to be much concerned with turbidity - - and in many cases not even them. Dredgemen from Europe, Asia, South America and other countries had not much to say concerning the reduction of dredge-induced turbidity. Many indicated turbidity had not yet become a social problem - - but in anticipation of it they were interested in the outcome of the study. A few had been doing some work on it.

Correspondence and oral communication was also exchanged with environmental groups, environmental protection agencies, fish and wildlife agencies, park divisions, water quality boards, and various individuals professing environmental expertise.

We were somewhat taken aback in our effort to establish rapport with environmental groups. It was, to say the least, discouraging. We made an attempt to explain the study to several of these groups - - with the hope of obtaining information or ideas they might have. Overall the response was negative. The final one - - which discouraged further queries - - warned that should we give a talk to their group, we would be required to defend ourselves. Apparently there was no common ground for anyone connected with dredging.

We reviewed all literature we could find that might be informative about dredge-induced turbidity, such as all issues of World Dredging and Marine Construction, all five transactions of the World Dredging Conferences of 1967 through 1975, the writer's own files, and all reports, papers and periodicals dealing with turbidity that were available. This survey indicated there is a complete dearth of published information on dredge-induced turbidity, and very little - - that can be used - - on any type of turbidity reduction.

As information came in, we analyzed and filed it according to previously established categories. Some of them were:

Cutters	Operations
Derrick barges	Operators
Discharge lines	Specifications
Dredged material types	Supervisors
Employees	Tenders
Field surveys	Tides and stream-flows
Hulls	Time of year
Ladders	Weather

Determinations were made as to what information was of most practicability, and what was of less value or useless, the decisions being based upon its potential for reducing turbidity, its cost of implementation, and its effect upon dredging production.

Minor field tests were undertaken to determine the relative magnitude of turbidity that might be generated by a cutter. We investigated the turbidity effects of different cutter speeds at different depths using an operating dredge. Cuts were made at 20, 30, and 40 feet, at cutter speeds of 10, 20 and 30 rpm.

Water samples taken during the tests were analyzed in terms of suspended solids and nephelometric turbidity units. In situ turbidity readings were taken in terms of percent transmission. The transmission data showed an increase in turbidity above background levels in the immediate vicinity of the cutter. Little of the turbidity created by the cutter, however, went into the upper water column, especially from depths of 40 feet; and no substantial surface turbidity was ever observed. Turbidity data collected in the immediate vicinity of the cutter were quite variable. This may have been due to the cutter-generated turbulence, variability of the material, and/or suction velocity.

Generally speaking, however, the cutter can be responsible for a great percentage of dredge-induced turbidity in particularly susceptible material.

Cutters are often not well designed - - either for good production or turbidity reduction. The use of an improperly designed cutter, or, the improper use of a properly designed one, often results in overcutting to provide material required by the suction. Overcutting causes more material to be loosened and consequently excess material is left to create turbidity.

Usual construction places the cutter in front of, and its center somewhat above the suction mouth. This creates a separation between the suction and the material. As dredging depth increases, so does the separation distance. As this distance increases, the suction's pickup capability is reduced. More material will then tend to remain in the water column and cause turbidity.

In many instances the removal of the cutter allows the suction to be placed closer to the material. This is especially helpful when doing maintenance work in material that does not have to be cut or dislodged.

There have been some attempts to contain turbidity by adding shields or hoods around or over the cutter. In the reported instances where these devices were tried, very little turbidity reduction resulted.

A rotating suction will decrease turbidity production. This assemblage allows for a smaller-diameter cutter, which can feed material into the suction from the top, sides, and bottom, rather than just the bottom, as with regular cutters. The pickup is then more universal and turbidity generation is lessened.

On an increasing number of dredges, jets and ladder-mounted pumps are being installed to increase the head available for lifting the material. The jet, which is usually mounted near the suction, provides more lifting energy by injecting a high-velocity stream of water into the suction. The stream's velocity head is changed into a pressure head, thereby adding energy to the suction. In this way higher suction velocity can be obtained without jeopardizing the lifting power of the system and turbidity reduction is obtained.

Ladder pumps overcome some of the effects of head loss. The higher velocities obtainable therefore enhance pickup and result in less turbidity generation.

Ladder drag contributes to turbidity generation. A too-long ladder will create more turbidity than one just long enough to do the work, particularly in shallow-depth jobs.

Most dredges are built with rectangular hulls whose dimensions are determined by internal and external equipment-supporting requirements. Hulls with drafts approaching the depth of cut are turbidity-producers. The proximity of the hull to the bottom of the cut creates water turbulence which can suspend loose bottom material.

Over-wide hulls in narrow channels can create turbidity problems. For instance, when a channel width is small, a dredge with a wide hull may bump against the sides of the cut when swinging. Turbidity is then created by the sloughing off of the sides of the bank.

Adequate freeboard should always be maintained. A dredge in calm water does not need much freeboard. However, when the weather is bad or when the dredge is swinging and, say, pulling heavily on anchors attached to anchor booms, water often inundates the decks.

Any flat surface on a dredge is usually a depository for oil barrels, old rope, and all forms of dirt and trash. When freeboard is not sufficient, this conglomerate is often washed overboard. Although turbidity, as one generally considers it, is not involved, the effect of trash in the water around the dredge is just as objectionable, if not more so.

One big objection of the public to dredges, and all shipping for that matter, is the overboard pumping of bilges. Bilges should only be cleaned by pumping into barrels or barges and the refuse hauled away.

Even the crumb boss contributes to dredge turbidity if he dumps the garbage overboard.

Greasing of sheaves, swivels, spud wells, and other dredge parts can create a problem. Although these parts need greasing, the oilers usually apply more than enough. The result is that most of the grease goes into the water and turbidity is created.

Big producers of turbidity, particularly in shallow-depth waters, are the tenders. Although most usually have shallow drafts, many do not. Most have powerful engines. With this combination, the prop wash creates much turbidity, particularly when docked at landings or when pushing pontoons around.

Barges used for storing or hauling equipment, such as pipe, water and fuel, can generate turbidity when tied up to banks, particularly those composed of soft materials. The constant movement of the hulls wears away the banks, the material falls into the water, and turbidity results.

Floating pipelines contribute to turbidity in several ways. It is always necessary to be adding or taking out sections of a floating line. If the pump is simply stopped and the line not washed out, material will settle to the bottom of the line with future plugging consequences. Then, if the line is broken it is thoroughly washed out, the material and water remaining in the line near the break will fall out into the surrounding water and create turbidity.

Floating lines do not always run from the dredge to the shore in a straight line. They often run towards shore and then parallel it. When these lines are close to shore the pontoons often come in contact with the bank and/or scrape the shallow bottom near the shore. This creates turbidity. Floating lines should be kept offshore enough that there is clearance for the pontoons to move about without hitting the bank or disturbing the bottom material.

Connections between sections of floating lines cause turbidity. If the joints are old or their gaskets worn, dredged material can leak out.

Shoreline leakage, although not likely to cause turbidity in the water around the dredge, can create an unpleasant effect. Actually the public objects to these leaks more often than they do those of floating lines, because they are more easily seen.

The requirement for tightness of pipe connections not only applies to floating- and shore-lines, but to the stern and shore connections as well. Poor seals in these will cause as much turbidity as poor connections in the lines.

The simplest form of setting the dredge ahead is the stabbing method. Here the dredge is swung from side to side, the spuds being interchanged at the ends of the swing. However, this does not allow all of the bottom to be covered by the cutter and suction. Each new arc is always advanced from the last and the suction skips part of the bottom. This leaves windrows.

These windrows usually contain material that can be suspended by water turbulence. The tops slough off and fall into the valley between. This fallout of material tends to create additional turbidity.

The stabbing method obviously is not conducive to turbidity reduction, or, generally, to efficient dredging. As a matter of fact, the best stabbing method to reduce turbidity generation is one where every bit of the bottom is covered by the suction's influence.

Large sets that bury the cutter are satisfactory only if the cutter and suction are capable of handling the material dislodged. However, if more material is loosened than can be handled by the suction, turbidity problems will arise.

Rate of swing has an effect on turbidity creation. Normally the dredge is swung across the cut at a rate to obtain maximum production. Depending upon the material and the effectiveness of the cutter-suction combination, a slow swing rate will cause less agitation of loose material and create less turbidity.

Swing wires can create turbidity. In narrow channels, anchors often have to be tied to deadmen on the banks. These anchorages can contribute to turbidity production, particularly when the banks are high. The sawlike motion of the wires, caused by the swinging and setting of the dredge, loosen the bank material and it subsequently falls into the water, creating turbidity.

In some dredging operations it is often economical to cut ahead on the job, leaving some of the bottom material behind, and then later setting back to clean up. The material left for later cleanup is subject to suspension by prop wash, dredge-hull drag, and tidal or river flow. In terms of turbidity generation it would be better to complete the required dredging as the dredge advances.

Agitation dredging is used occasionally to maintain the depth of dredged or natural channels. The bottom material is suspended by the prop wash of a boat, the current or tide being depended upon to carry away the material.

Although such operations are quite effective, the suspension of the disturbed material causes turbidity far in excess of that created by a dredge. This form of dredging should be discouraged whenever turbidity generation may cause problems.

Most contracts call for removing a specified amount of material from within a prism shown on the plans. Since the contractor does not want to dredge more material than he will be paid for, he usually establishes some means for controlling the dredge so that only a minimum amount of overdredging will occur.

The better the control, the less the dredging, and consequently, the less the turbidity.

If dredging is done in calm water, turbidity created by the dredge remains in the vicinity. However, when operating in areas where there are currents or turbulence, turbidity can be widely dispersed. If dredging proceeds downstream, the turbid water can spread ahead of the dredge uninterrupted. Therefore, where possible, it is often best to dredge into the current because the turbid water must pass by, around, and under the dredge and the floating line.

In this case there is a greater tendency for the suspended material to flocculate and settle or be dispersed. And the visibility is reduced.

Whenever possible, projects should be scheduled for the most propitious time. A calm, undisturbed surface will hold a turbidity cloud together longer than a disturbed one. Furthermore, a disturbed surface usually has its own turbidity, and any turbidity created by the dredge is less likely to be observable.

Capable dredge personnel can be an asset in the reduction of dredge-induced turbidity. A crew that is inexperienced, uninterested or incapable will thwart the efforts of others to use techniques for reducing turbidity. Little can be accomplished if these techniques are not practiced by the employees. Unfortunately the planned training of people manning this country's dredging fleet is practically nil.

This has been an extremely brief synopsis of our study. The full report has been published. If you are interested in the full discussion, with pictures, graphs, tables and do-and-don't recommendations, etc., you may obtain a copy by contacting Dorothy P. Booth, U.S. Army Engineer Waterways Experiment Station, CE, P.O. Box 631, Vicksburg, Mississippi 39180.

"THE FUTURE OF THE DREDGING MARKET"

by

Carl B. Hakenjos[1]
(Luncheon Speaker)

Ladies and gentlemen, and distinguished guests:

It is indeed a pleasure and an honor to be here, and I appreciate the opportunity to address this group. This is by far not my first visit to Texas A&M and it is gratifying to be able to return and see the advancement in the Center for Dredging Studies. I recall in 1968, when the Center was founded, all the apprehension and doubts that individuals had regarding the success of the project. We, who were close to the idea at the time, saw the merits and great need for the Center, but could others be convinced? The company I am with, along with four others, contributed eight thousand dollars each, which was $40,000.00, to assist with the Center's initial expenses. Those first years were very difficult. However, when I observe the facilities here today, see the activities and benefits of the Center, and look forward to excellent talks on dredging, I can truthfully say the time, effort and money were well spent.

I can also recall, since the conception of the Center, the papers I have presented here at A&M at the Second Seminar in November 1969 and at the Sixth Seminar in January 1974.

Before we get into the main topic of my presentation which is "The Future of the Dredging Market", I would like to discuss, if I may, and quote from my speech of 1969, which was entitled, "Pollution Control and Dredging". This has a direct bearing on my presentation and I want you to bear this

[1]Williams-McWilliams Co., New Orleans, Louisiana

following statement in mind, because the future of dredging is dependent upon its compatibility "with" the environment. I personally am very close to the present extensive research program for this, and I am firmly convinced that the compatibility can be accomplished.

Remember, this was in 1969, seven years ago. I quote, "It is time we impress our legislators that we are willing to cooperate to preserve the beauty of our environment, but not in line with the inflexible belief of our conservationists, that change always means destruction, and that to preserve the beauty of our environment means complete abandonment of economic progress.

"Water is a precious commodity. It is becoming more apparent each year that we cannot afford to waste, pollute, or in any way destroy this natural resource. Thus we must plan the use of our nation's water supplies to provide maximum benefits for all purposes--providing outdoor recreation opportunities, fish and wildlife conservation and enhancement, in conjunction with dredging to maintain existing navigation, provide for new navigation, and also assure the control of floods in our country." (End of Quote)

A very good example of the point I am trying to make appeared in Wednesday's Times-Picayune Newspaper. There was an interesting article about the Theodore Ship Channel, which is south of Mobile, Alabama. This channel would cut a deep-water link for ships between the bay and Theodore Industrial Park, a site south of Mobile where industries have announced plans for $800 million in investments.

A state conservation official said Tuesday he opposed plans to create a 1,300-acre dredged material island in Mobile Bay as part of the $56 million project. The chief biologist for the Marine Resources Division of the Conservation Department said dredged material should be hauled to open waters of the Gulf rather than used to create an island in the bay. I can't imagine taking good soil and wasting it in the Gulf, when it can be saved and used.

A spokesman for the Corps said the island would not disturb the salinity of the bay waters. He said extensive tests showed it was the best method from a cost and environmental standpoint. Some $42.8 million in federal funds recently were authorized for the seven-mile channel, with the State of Alabama contributing some $6 million or more to the project. This very beneficial $800 million industrial project, through lack of understanding and thorough investigation, could be cancelled.

I will now discuss the present status and future of the dredging market in this country and will attempt to delineate our present status, compare us with the rest of the world, and predict our future.

In the past ten years the dredging industry has been undergoing a slow but significant renaissance in technology. This was given added emphasis with the October 1973 energy crisis and oil embargo in the Middle East, giving a new focus to the diminishing supplies of energy sources along with the higher cost of fuel.

For the past 50 years there has been scarcely a change in the design features of cutter suction dredges, which are the dominant type in the world today, particularly in the United States. Unfortunately, there is no large manufacturer such as Terex, Caterpillar, American Hoist, Joy, etc., to do the research and development for dredging companies.

The over 25,000 miles of inland and coastal waterways of the United States dramatize the significant role that water transportation plays in the economy of this country. The major ports of the United States measure their annual financial contribution to the economy of surrounding regions in the billions of dollars. Nevertheless, this fact has been lost, for the most part, on the financial and industrial segments of industry and government, as to the potential for business expansion inherent in the development of ports and waterways. The United States port authorities and agencies responsible for local development of waterways and ports, in most cases, look to federal funding for their expansion with a delay of up to 15 years from initial planning until funding and execution.

Compare this with the dramatic development of ports and reclamation projects around the world over the past 10 years. Richards Bay, South Africa, building a brand new port at a virgin area to become what will probably be the largest port in the world, with a 100-foot draft; with the initial funding relating to dredging exceeding $150 million. Also, the ports of Fas near Marseille, Dunkirk; Le Havre, Rotterdam; Taichung in Taiwan, all massive undertakings.

More dramatic perhaps than any of these, however, has been development over the past 15 years of what may have been the largest single dredging reclamation program in the history of the world. Over eight billion cubic yards of material were dredged to create over 150,000 acres of new industrial land in Tokyo Bay. This has been the result of a boldly planned and efficiently executed program.

I would like to digress, if I may for a minute, and relate an interesting experience our company had with the Japanese about 18 years ago. We had received, at that time, a request from Washington to take some 25 or 30 Japanese engineers aboard our best dredge and assist them in any way. Other dredging firms across the country were also contacted. Once the trip was arranged and the engineers were aboard our dredge, they brought out camera after camera after camera. They took pictures of every part of that dredge. When they weren't taking pictures, they were making sketches.

The point I am trying to make is that prior to this visit to the United States, I doubt if Japan had one large efficient dredge, but in a matter of a few years they have dredged over 8 million cubic yards of material. Why? Necessity.
These people had the foresight to know by this time, they had to have more land, so they created 150,000 acres by dredging.

Necessity made the Dutch start creating land years ago, and I'm sure if you've ever visited Rotterdam and the surrounding areas, you were amazed at the astounding amount of land they have created and the deepwater ports they have developed.

We can take a lesson from the Japanese as well as other developing countries in planning our own economic development, assuming that there can be a rational and intelligent communication with our environmental groups and agencies. We, also out of necessity, have to maintain our waterways and deepen our ports.

The investment of capital from the private sector to accomplish the dredging of a port for the purpose of creating new land for industrial expansion has been applied in Japan, Korea and the Philippines. The developer is invited to participate where he can return the investment with a reasonable profit by providing the funds to perform the dredging without expense to the local port authority. By doing this the local authority is able to bypass the process of either appealing for federal funds or going through a bonding process.

A good example of this in our country was when Exxon and the Port of Houston began negotiations in the early 1960's to establish a new port in the Bayport area, which is located on the Houston Ship Channel just south of Houston. The facility was to be publicly owned and operated by the Port of Houston. Exxon provided the land and right of way for necessary transportation and utilities to the channel and turning basin areas.

Additionally, the two parties agreed to a method of financing that would not cost the Port of Houston a penny. Exxon agreed to buy Port of Houston revenue bonds to finance construction of the Bayport Division. These bonds would be retired with revenues derived from Bayport Division operations. It was specifically understood that no tax revenues nor income from other Port of Houston operations would be used to retire these bonds.

Rather than being suspicious and apprehensive about such offers on the part of private industry, our port authorities and government agencies should be eager to stimulate such initiative and thus save the taxpayers money.

The basic economic equation used by the oil industry for years to justify its development both in land and offshore oil reserves, should be applied to port and waterway development in the United States. The dramatic return on investment is comparable to the production of oil, when a waterway or port is opened to well-conceived and planned industrial development. The examples are many, such as the ports of New Orleans, Houston, Long Beach, and the Arkansas River project, to name only a few.

We need to convince our environmental agencies and citizens groups that the citizen and economy benefit equally from these types of developments and that they can be executed in a balanced way with nature. We have shown through private- and government-sponsored research in the past several years how dredging can be performed in harmony with nature through proper planning and execution of the projects. Dredged Material Research Program, which is being conducted at the Corps of Engineers Waterways Experiment Station at Vicksburg, Mississippi, is in its third year. When this program, which was approved by Congress, is completed in two more years, $30,000,000 will have been spent to study and prove the compatibility of dredging with the environment. Dredging can in fact out-do nature in forming new estuaries and birdlife refuges and all of the things so dear to the hearts of the environmentalists.

In attempting to make market projections for dredging in the United States, I am faced with a large unknown. Dredging projects today are dependent upon the initiative of companies and associations for joining together in a well planned and executed effort to convince citizens as well as government agencies of the benefits available through water-oriented, commercial and recreational development. Secondly, we need to develop an atmosphere of cooperation among industry, government and citizen groups for the best utilization of our resources. In this case, I mean to open the doors to private investment in the development of ports and waterways. If these things can be properly done, the market for dredging in the United States could be measured annually in the billions of dollars.

At present, the U.S. Army Corps of Engineers performs approximately $150,000,000.00 per year worth of dredging with their own dredges in competition with private industry. That market is available to a well planned and conceived development program of private dredges to replace the Corps of Engineers' dredges. The Corps of Engineers, likewise, contracts for approximately $175,000,000.00 per year of dredging by the private sector. It is estimated that there may be another one to two hundred million dollars worth of dredging funded by private sources over and above the $325 million per year funded or controlled through the Corps. Unfortunately, where government work is involved, we are tied to a fiscal funding program that is too political in execution.

There are also, in addition to conventional port; waterway and reclamation dredging, dredge mining activities, which include mining of sand and gravel, gold, platinum and diamonds. Within the United States we have a large sand and gravel dredging industry and a newly developing titanium dredging industry. The coal is dredged now in some locations and is pumped long distances through pipe as well as other ores in slurry form.

In conclusion, I will state some of the conspicuous problems inherent in dredging. These include, but are not limited to, high capital investment and sometimes long periods of inactivity where the amortization of the equipment must be carried. The cutter dredge has to be essentially an all-purpose tool which is limited to a fairly narrow range of activities. Also, it is difficult to recruit labor and train personnel due to

24-hour operations and, frequently, work in very remote areas. It is an admiralty risk which introduces an added cost of operation through marine insurance, Jones Act Litigation, liability claims and workmen's compensation.

Dredging is a challenging world of wet civil engineering and construction. A multi-disciplinary technology that has yet to experience the fruits of an accelerated technology, but that day is fast approaching.

Thank you!

Selected Environmental Aspects of Dredging in San Diego Bay, California

by

David D. Smith[1]
and
Katherine F. Graham[1]

INTRODUCTION

The intent of this paper is to review briefly: (a) selected environmental aspects of historic and present dredging in San Diego Bay, (b) other major man-induced environmental stresses affecting the bay, (c) the way in which environmental considerations are brought to bear on dredging via the federal and state regulatory process, and (d) the significant time and monetary effects that environmental considerations have had on a major San Diego Bay dredging project currently near completion.

The paper incorporates excerpts from papers presented by the writers at the Third International Estuarine Research Federation Conference in Galveston, Texas, October 1975; at the Seventh World Dredging Conference in San Francisco, July 1976; and at The Coastal Society Second Annual Conference in New Orleans, November 1976 (see Smith and Graham, 1976; Smith, 1977; and Smith and Graham, in press).

SAN DIEGO BAY

San Diego Bay (Figure 1), located in the extreme southwest corner of the United States, is one of the finest natural harbors

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Figure 1. San Diego Bay and part of the adjoining metropolitan area; view is to the east. (Photograph courtesy of the California Water Resources Control Board.)

in western North America. The bay is a landlocked, crescent-shaped body about 22.5 km long, ranging from 0.4 to 4 km in width. Except in the 7.5 to 20 m deep channel, depths range from generally deeper than 9 m in the north bay, from 3.0 to 4.5 m in the central bay, and from 0 to 2.4 or 3 m in the south bay. The extreme range of tides within the bay is 3.2 m.

The north and central parts of the bay have been modified extensively for use by naval and commercial vessels, and for recreational boating. The character and extent of the harbor's principal channels and the 12.8 m deep entrance channel are attributable primarily to naval requirements, inasmuch as San Diego Bay is the site of one of the largest Navy establishments in the country.

In addition, metropolitan San Diego, with an overall area population of about 1,150,000, now virtually surrounds the bay.

DREDGING AND FILLING IN SAN DIEGO BAY

As presented in Smith (1977), San Diego Bay is one of the most extensively dredged and filled estuaries in the United States. Approximately 55 percent of the bay floor has been dredged, and only about 17 to 18 percent of the 1918 bay area remains unmodified, if dredging and filling both are considered.

The history and extent of dredging and filling in San Diego Bay are unusually well documented (see Mudie, 1970; Gautier, 1972; Peeling, 1974; USACE, 1975b; and Smith, 1977). According to USACE (1975b),

> "Starting near the beginning of this century the channels in the north and central parts of the bay were straightened, widened, and deepened to facilitate navigation; and

> practically all of the marshlands in the north bay were filled to create lands for airfields, highways, docks, shipyards, parks, tourist and recreational facilities, and other uses.
>
> "...Extensive dredging since 1940 has... altered the character of (much of) the bay floor."

EXTENT OF FILL

Prior to major filling activities, San Diego Bay had an area of 54 to 57 km^2, as defined by the mean high tide line of 1918 (see Gautier, 1972). Filling activities, primarily using dredged material, began in 1888 (Peeling, 1974) and intensified markedly shortly before and during World War II. As illustrated in Figure 2, approximately 15.5 km^2 of the bay, amounting to about 27 percent, have been filled.

EXTENT OF DREDGING

The areal extent of dredging in the bay is illustrated in Figure 3. Approximately 31 km^2 of the 1918 bay floor, amounting to 55 percent, have been dredged. As evident in Figure 3, except for an area of about 10 km^2 in the extreme south bay area, most of the bay floor has been dredged. Further, this dredging has virtually doubled the depth of most of the shallower portions of the north and central bay.

If dredging and filling both are considered, only about 17 to 18 percent of the bay area represented by the 1918 mean high tide line remains undisturbed. More than half of this dredging and filling was associated with World War II naval activities.

VOLUMES MOVED

As to the volumes moved, it is estimated (Smith, 1977) that

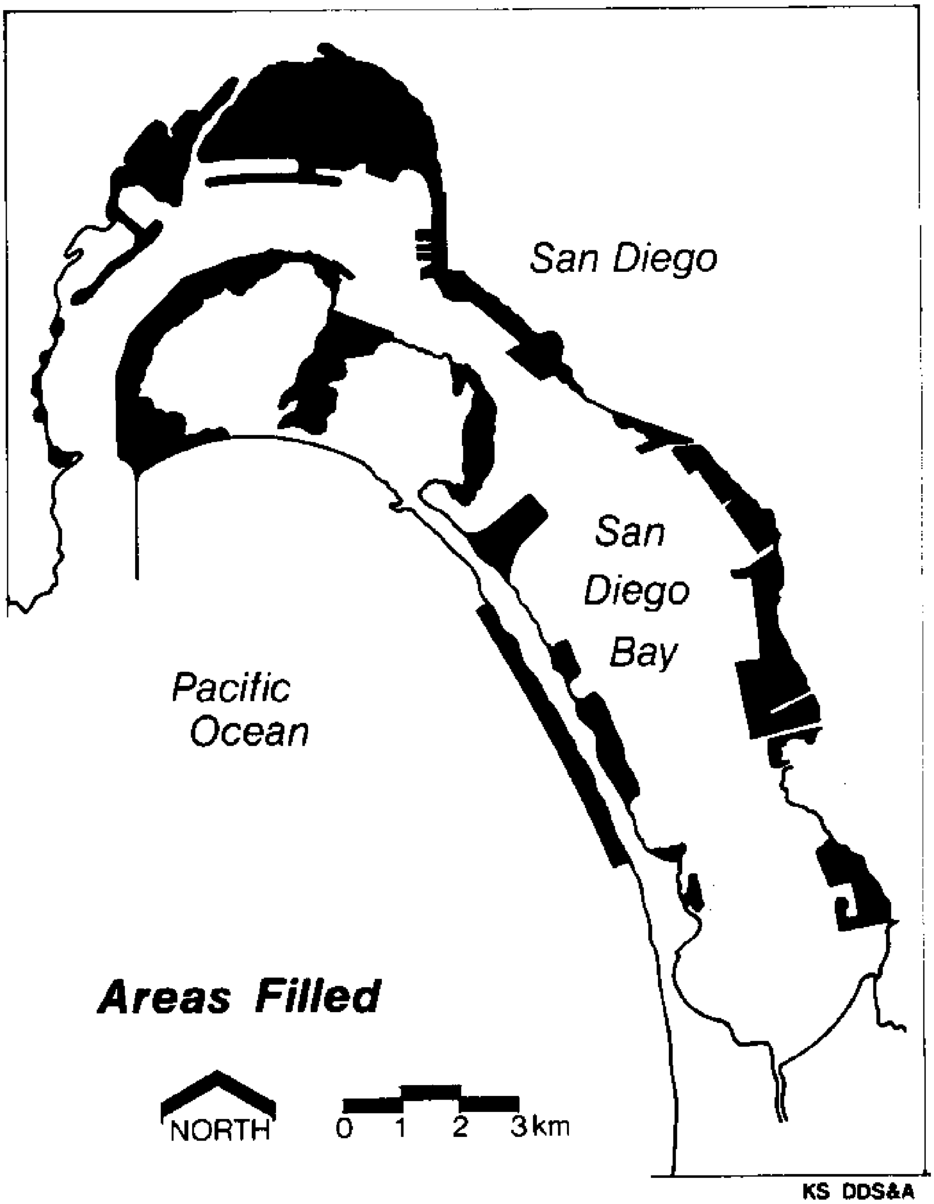

Figure 2

Areas filled in San Diego Bay during the period 1914-1971. Reproduced from Smith (1977) by permission of the Academic Press, Inc.

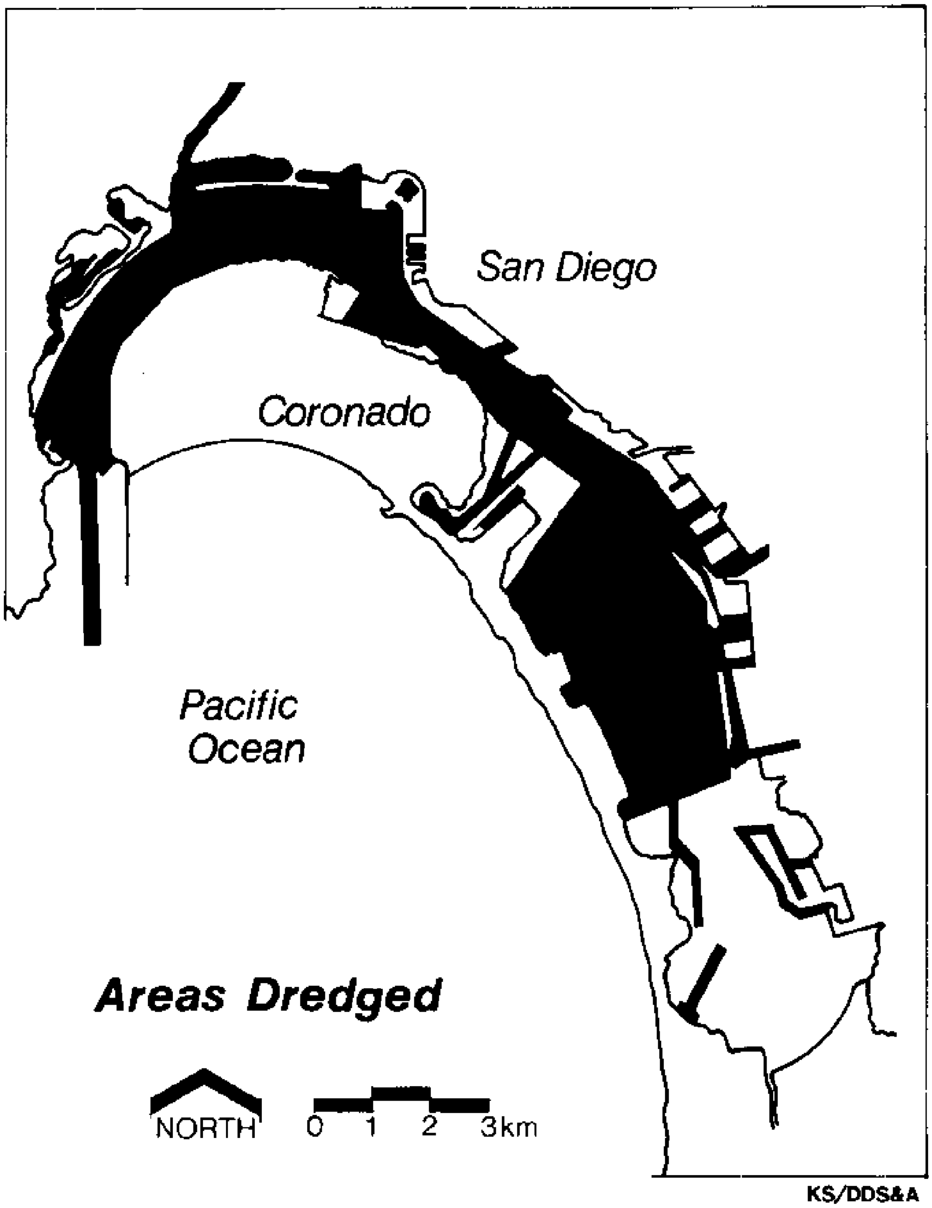

Figure 3

Areas dredged in San Diego Bay during the period 1936-1971. Reproduced from Smith (1977) by permission of the Academic Press, Inc.

dredging has shifted about 100 to 140 x 10^6 m^3 of bay floor sediment approximately as follows: (a) about 2 x 10^6 m^3 to deepwater ocean disposal sites by mechanical dredging and transport[1/], (b) about 22 x 10^6 m^3 to Silver Strand as beach replenishment by pumping (Peeling, 1974), and (c) roughly 75 to 115 x 10^6 m^3 to bay marginal areas as fill by pumping[1/], as illustrated in Figure 2. By comparison, (1) the present water volume of San Diego Bay is about 230 x 10^6 m^3, and (2) the San Diego River, the former chief tributary to the bay before diversion in 1875-77, is estimated to have delivered about 3.8 to 5.3 x 10^5 m^3 to the bay annually.

Clearly, past dredging and filling produced major changes in the geometry and sediment character of this estuary (see Smith, 1977), and probably in the biological communities living therein. Assessing the environmental significance of these changes, however, must take into account at least three other major man-induced environmental stresses which have affected the bay for most of the past century.

OTHER ENVIRONMENTAL STRESSES

Historically, the bay was heavily stressed for nearly a century prior to 1963, as a result of the following:

(1) diversion and damming of principal tributaries between 1875 and 1919 which all but eliminated freshwater input throughout most of the year and reduced natural sediment input by more than 80 percent (Smith, 1977);

[1/]Estimates based on review of available data with Mr. Donald R. Forrest, San Diego Unified Port District, who has extensive knowledge of San Diego Bay dredging projects.

(2) waste discharges to the bay, including: (a) metropolitan sewage and primary treatment plant effluent, (b) industrial wastes, and (c) power plant cooling water; and

(3) intensive urbanization of the land areas ringing the bay, with attendant introduction of urban contaminants from storm runoff and atmospheric fallout.

Although a detailed assessment of these stresses is outside the scope of this paper, a brief review of stresses resulting from diversion and damming is included as an example of an order-of-magnitude assessment which can be made despite incomplete historical data.

EFFECTS OF DIVERSION AND DAMMING

As presented in detail in Smith (1977), the principal tributaries to San Diego Bay (Figure 4) prior to man's intervention were the San Diego, Sweetwater, and Otay rivers, and a number of smaller streams; the drainage basin's tributary to the bay totalled about 2,330 km^2. As late as the 1850's, the prominent delta of the San Diego River was building in northern San Diego Bay (see Smith, 1977, Figure 1), as were the deltas of the Sweetwater and Otay rivers in the south bay. The presence of these deltas is strong evidence that natural sedimentation (primarily fluvial deposition) was gradually filling the bay. Beginning in the 1870's, however, freshwater input and fluvial sediment influx were greatly reduced as a result of: (1) the 1875-77 diversion of the flow of the San Diego River away from San Diego Bay and into nearby Mission Bay (Rambo and Speidel, 1969; and USACE, 1975b), and (2) construction of water storage

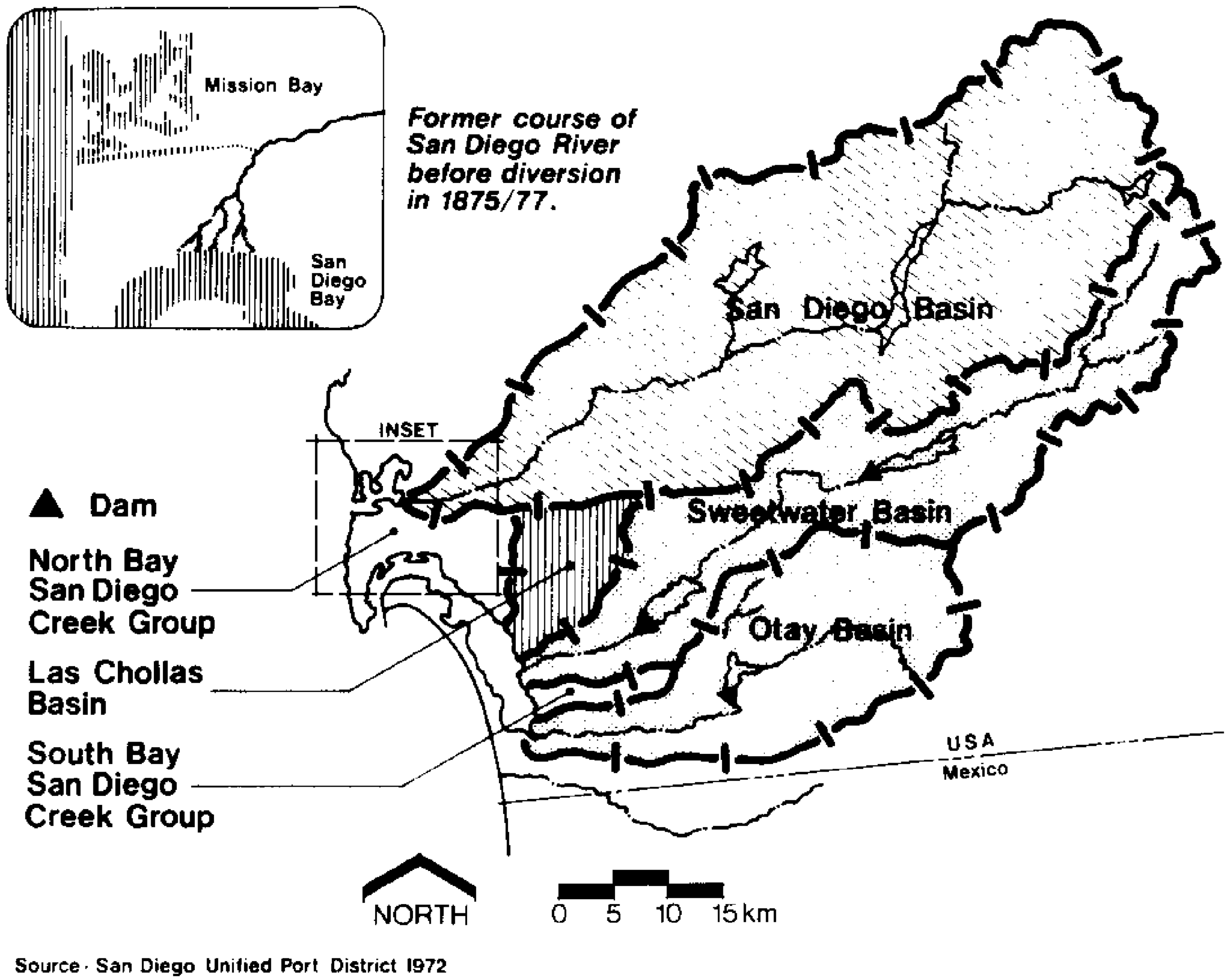

Figure 4. Tributaries and drainage basins contributing runoff and sediment to San Diego Bay. Reproduced from Smith (1977) by permission of the Academic Press, Inc.

reservoirs on the Sweetwater and Otay rivers in 1888 and 1919, respectively (see Figure 4). The diversion terminated the freshwater and sediment contribution of the San Diego River and, as shown in Smith (1977), the reservoirs reduced the freshwater and sediment input from the Sweetwater and Otay rivers by about 75 percent. The other tributaries to San Diego Bay (Las Chollas Creek and the creeks in the North and South Bay groups - see Figure 4) contribute insignificant amounts of freshwater and sediment to the bay (USACE, 1975b; Smith, 1977).

Based on Smith (1977) and the sources cited therein, the total fluvial sediment delivered to San Diego Bay annually prior to man's intervention was probably on the order of 0.8 to 1.1 x 10^6 m^3. Following diversion and dam construction, the annual fluvial sediment contribution dropped to 1.4 to 1.9 x 10^5 m^3; roughly this rate apparently has obtained for the last 55 years. In short, according to USACE (1975b), fluvial sediment contribution to the bay is minimal because: (1) few major drainages enter the bay, (2) many of these drainages are partially controlled by dams, and (3) stream velocities downstream from the dams are generally non-scouring.

EFFECTS OF WASTE DISCHARGES AND STORM RUNOFF

The effects of the various waste discharges to San Diego Bay are discussed in some detail in San Diego Regional Water Pollution Control Board (1952), Newman (1958), Ford (1968), Parrish and Mackenthun (1968), Ford et al. (1970, 1971, 1972), Dodson (1972), Gautier (1972), Browning et al. (1973), Ford and Chambers (1973, 1974), and Wagner (1976), and are summarized from an environmental point of view in Smith and Graham (in press), to which the interested reader is referred. To the writers' knowledge, no work has been published on the effects of urban storm runoff and atmospheric fallout on San Diego Bay.

SUMMARY OF PRINCIPAL MAN-INDUCED EFFECTS ON THE BAY

The principal adverse environmental effects of the various stresses discussed or listed above include: conversion of what probably was a brackish estuary to an entirely saline water body; destruction of 80 percent of the bay marginal salt marshes by filling; doubling of average depth as a result of dredging; buildup of sewage waste deposits and associated algal mats on the bay floor, with massive reduction in the benthic and pelagic biological communities; conversion of mud bottom to sandy substrate by dredging; and local contamination of bottom sediments with trace metals from shipyards and industrial discharges.

When contemporary dredging activities are viewed in this historic context of roughly a hundred years of environmental stresses (including the World War II episode of massive dredging and fill), environmental impacts of current dredging projects would appear to be of relatively minor significance. However, since the late 1960's, dredging projects in San Diego Bay (and throughout much of the U.S.) have been rigorously regulated and widely opposed by environmental groups.

REGULATION OF DREDGING

In recent years, institutional constraints brought to bear through Federal and state regulatory processes have become key determinants in the authorization of dredging projects in California. Dredging permit procedures require review by numerous federal, state, county, and municipal agencies as well as citizen interest groups. Opposition to a project by an agency or group may result in costly delays, possible project modification, or even cancellation.

Environmental and legal controls on dredging projects (Smith, 1975; Boerger and Cheney, 1976) stem from regulatory policies specified in various Federal and state laws[1/]. The specific requirements of the Federal laws (and the complicated regulatory procedures for implementing them) generally are set forth as detailed regulations and guidelines published in the Federal Register (USACE, 1975c; Smith, 1976). At the state level (in California, for example), the detailed regulations are issued by the appropriate regulatory boards and commissions. Generally, the pertinent regulatory procedures require issuance of permits for specific acts such as dredging, discharge of dredged material, placement of fill, etc.

PRIMARY CONTROL

As presented in Smith and Graham (1976), primary regulatory control of dredging projects is the responsibility of the Federal government. This responsibility is vested in the

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[1/]Pertinent federal laws include the River and Harbor Act; Federal Water Pollution Control Act; National Environmental Policy Act; Marine Protection, Research, and Sanctuaries Act; and the Coastal Zone Management Act. Examples of pertinent California state laws include the Porter-Cologne Act; the California Environmental Quality Act: and the Coastal Zone Act.

Secretary of the Army and is exercized by the Corps of Engineers. Federal regulatory control of dredging, construction, and related actions in U.S. navigable waters dates back more than 75 years. The River and Harbor Act of 1899 requires that a "Work in Navigable Waters" permit be obtained from the Department of the Army, Corps of Engineers, for virtually all structures or work in navigable waters of the United States.

More recently, Corps and EPA regulations (33CFR and 40CFR) require a "Dredged Material Permit" for discharge of dredged material into the territorial sea.

SECONDARY CONTROLS

Additional, secondary or quasi-regulatory controls on dredging projects are exercized by environmental and conservation agencies and organizations as they function in a review capacity during the permit application processing procedure (see Smith, 1976, Figure 2). For example, Corps of Engineers "Work in Navigable Waters" and "Dredged Material" permit application processing procedures for a dredging project provide for review of a permit application and supporting certifications by as many as two dozen Federal, state, county, and municipal agencies, and perhaps a dozen or more citizen interest groups (Smith and Graham, 1976).

HOW ENVIRONMENTAL CONSIDERATIONS FUNCTION AS CONSTRAINTS

As a result of the above cited laws and administrative procedures stemming therefrom, regulatory and conservation agencies and public interest environmental groups have substantially greater leverage on dredging and marine construction projects than on most terrestrial projects of equivalent size.

These agencies and/or citizen groups may oppose part or all of a given project at one or more of the several check points in the Corps' permit review procedure (USACE, 1974; Smith, 1975, 1976), with resulting costly delays and possible project modification or even cancellation. When issues cannot be resolved, opposing sides commonly invoke local, state, and Federal political pressures, and in some cases resort to litigation.

The following abbreviated case history of the San Diego Harbor Navigation Improvement Project is a near-classic, real world example illustrating: (a) how environmental considerations functioned as constraints on a major dredging project, and (b) the effects these constraints had on project schedule and cost.

SAN DIEGO HARBOR NAVIGATION IMPROVEMENT PROJECT

As described in detail in the Corps of Engineers' General Design Memorandum (USACE, 1975a), the San Diego Harbor Navigation Improvement Project will widen, deepen, and extend existing channels to allow larger and deeper draft vessels to use the present commercial facilities in the harbor. More specifically, these jointly funded improvements (Federal: 95.9 percent; local, 4.1 percent) will involve cutterhead hydraulic suction dredging of more than 5.5 million m^3, and clamshell dredging of about 765,000 m^3. The hydraulically dredged material will be delivered via pipeline to various disposal sites discussed below. The mechanically dredged material will be barged to an EPA approved ocean disposal site.

As detailed in Smith and Graham (1976), the project as originally proposed involved dredging about 9 million m^3 of material and, when authorized by Congress, was estimated to cost about $8 to 10 million, with work to commence in 1972. However, a series of institutional constraints (primarily of an environmental nature) caused substantial changes in the project design and a three-year delay in schedule.

When finally approved in 1975, the project had been reduced in size to about 6.5 million m^3 and the estimated cost had escalated to about $16.3 million. When the contract was awarded in August 1975, the actual price was $18.7 million.

CHANGES AND DELAYS OWING TO INSTITUTIONAL CONSTRAINTS

The principal changes in the project and the specific institutional constraints that were largely responsible for the changes and associated delays are discussed in detail in Smith and Graham (1976). A brief synopsis of that discussion follows.

During development of the General Design Memorandum (from July 1970 to December 1974), the Corps had more than 48 meetings (including a formal Public Hearing) with 17 Federal, state, and local agencies to ensure that the concerns of each were considered fairly in the final design. In addition, some 26 letters of comment on the General Design Memorandum were received from Federal, state, and local agencies and firms between January 1972 and January 1975; it is noteworthy that all but four of these addressed disposal of dredged material.

In the course of this protracted interaction between the

Corps and various agencies and public interest groups, institutional constraints (primarily quasi-regulatory conservation agencies acting per their charter to protect environmental resources they believe to be threatened by disposal of dredged material) were responsible for: (a) substantial modifications in the dredged material disposal plan, (b) elimination of most associated landfill sub-projects, and (c) delay in project approval of about three years. Specifically, the proposed southerly incremental extension of the channel was shortened by about 40 percent, the volume of the project was reduced by almost 30 percent, and major changes were made in the dredged material disposal plan, including: (1) deletion of a series of nearby, low-cost landfill disposal sites for dredged material, (2) addition of several distant sites requiring high pumping costs, and (3) costly ocean disposal of about three-quarters of a million m^3 originally scheduled for bay marginal disposal sites.

Because of escalation in dredging costs during this three-year period, and the added costs in meeting environmental restrictions, the eventual $18.7 million contract cost was almost $8 million higher than the estimated cost of $10.9 million at the time the project was initially funded in 1973. According to Ackerman (1975), about $5 million of the cost increase resulted from: (a) institutionally imposed environmental restrictions, and (b) the escalation in dredging costs during the time required to work through the institutional constraints just discussed.

In summary, the San Diego Harbor Navigation Improvement

Project illustrates clearly how environmental constraints can lengthen the authorization process, modify project design, and substantially increase design and construction costs.

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Corbicula manilensis Phillipi in the Arkansas River: Should the Corps of Engineers be Concerned?

by

Louise Russert Kraemer[1]

BACKGROUND

The Arkansas River arises in the mountains of Colorado, courses through a piece of Kansas and a sizable part of Oklahoma, then makes a vast northwest-to-southeast diagonal through Arkansas, and terminates finally in confluence with the White River. Since the 1930's the Arkansas River has sustained years of bank stabilization projects. In recent years the Arkansas River has been locked and dammed all the way up to the Port of Catoosa in Oklahoma, to become a major inland waterway of the United States. Today it has less the appearance of a river, than a long, costly reservoir.

Much maintenance dredging is now done to minimize shoaling of the navigation channel. In recent years of high water, the U.S. Army Corps of Engineers has moved as much as 4,240,000 cubic yards of dredged material in maintenance activities. Since 1970, the annual figure has not been less than about 1,500,000 cubic yards. In addition an equal or greater average amount of commercial dredging occurs annually on the Arkansas River--as a number of companies mine the river bottom sediments at many sites allocated by permit from the Corps of Engineers.

INTRODUCTION

In 1974, the Recreation and Resource Management Branch of the

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Little Rock District of the U.S. Army Corps of Engineers initiated a study concerned with the possible effects of maintenance dredging on the biota of the Arkansas River. Ironically this also constituted the first systematic effort ever made to examine the organisms living in the Arkansas River in Arkansas. Accordingly, the resulting contract called for a survey of the fish, phytoplankton, zooplankton and benthic animals in the river. Corps personnel took all plankton and benthic samples.

In the benthic part of the study, 3 series of collections were made: in October 1974, January 1975 and April 1975. These comprised more than 500 samples, all taken by ponar grab (with a $9.5cm^2$ "bite") from 13 stations at 56 collection sites along a 240-mile study reach (from river mile 283 near Fort Smith, Arkansas, down to river mile 43, near Mud Lake.) In the hope of coping both with the problem that we biologists lacked baseline data on the biota of the Arkansas River, <u>and</u> with the problem that the Corps wanted to know about the effect of their maintenance dredging activities, we asked for the following samples: from sites above, at and below areas of dredged material deposit; from sites above, at and below locks and dams, major tributaries, sources of sewage effluent inflow, paper mill discharge, etc. We also asked for many mid-channel samples and for sampling sites which transected the river. For a variety of logistic reasons, the Corps was only able to provide samples which were chiefly from sites at or behind revetments along the river. Nonetheless, these samples did provide essential data on the Arkansas River biota. It is hoped that future Corps activities on the river will incorporate findings from this study into a data base for biomonitoring the river.

What follows is (1) Results of some of the findings regarding the benthic animals of the Arkansas River, especially as the findings relate to a recently introduced animal, the Asian clam, *Corbicula*; (2) an analysis of some of the peculiar characteristics of *Corbicula* which may account for its apparent "takeover" of certain disturbed U.S. river bottoms; (3) a brief discussion of the implications of these findings, especially as they might relate to a continuing biomonitoring effort by the Corps; and (4) a suggestion as to how such a biomonitoring program, even done on a small scale, could be helpful to the ongoing Corps management of the River.

RESULTS

Major characteristics of the longitudinal distribution of the benthic fauna in the Arkansas River are summarized briefly in Table 1. At any point along the study reach, benthic samples which contained a variety of substrate particle sizes, and especially samples which contained detritus in the form of leaves, twigs, etc., contained a much richer fauna, both in numbers and in kinds of organisms--than did any of the other samples. Samples containing only medium to fine sand, however, were invariably quite barren of organisms.

Biologically the most significant finding of the benthic study was that the introduced Asian clam, *Corbicula*, was the most prevalent animal by far throughout the study reach. One expects that the majority of organisms in the substrate of a river will be arthropods (especially insect larvae) and mollusks (Hynes, 1971). This proved to be the case in the Arkansas River. What was unexpected, however, was the wide distribution and abundance of one genus (and perhaps one species*) of apparently recent introduction, the genus *Corbicula*.

Table 1. Apparent regional distribution of the benthic fauna of the Arkansas River.

Region of River	Characteristic Substrate	Characteristic benthic fauna
River Mile 283-249, above Lake Dardanelle	frequent boulders in substrate	May fly larvae (Ephemeroptera) and snails (Gastropoda)
River Mile 238-71	sandy substrate	Asian clam, *Corbicula*, and midge larvae (Chironomidae)
River Mile 45-43, near Mud Lake	silt, sand	Aquatic earthworms, (Oligochaeta), such as *Tubifex tubifex* and *Limnodrilus uedekiamus*

Figure 1 indicates the incidence of the more prevalent kinds of organisms in the benthic samples from the Arkansas River. From this figure, it is evident that the two most commonly encountered kinds of organisms throughout the study reach and in all three of the sampling series were midge larvae (Chironomidae, some 32 genera) and the single genus of molluscan bivalve, the Asian clam, *Corbicula*. Chironomid or midge larvae and *Corbicula* were also by far the most abundant organisms, in the Arkansas River benthic samples. Figure 2 shows

*The taxonomy of U. S. *Corbicula* is still a matter of dispute among malacologists. One view is that all U. S. *Corbicula* may belong to a single variable species, *C. manilensis*.

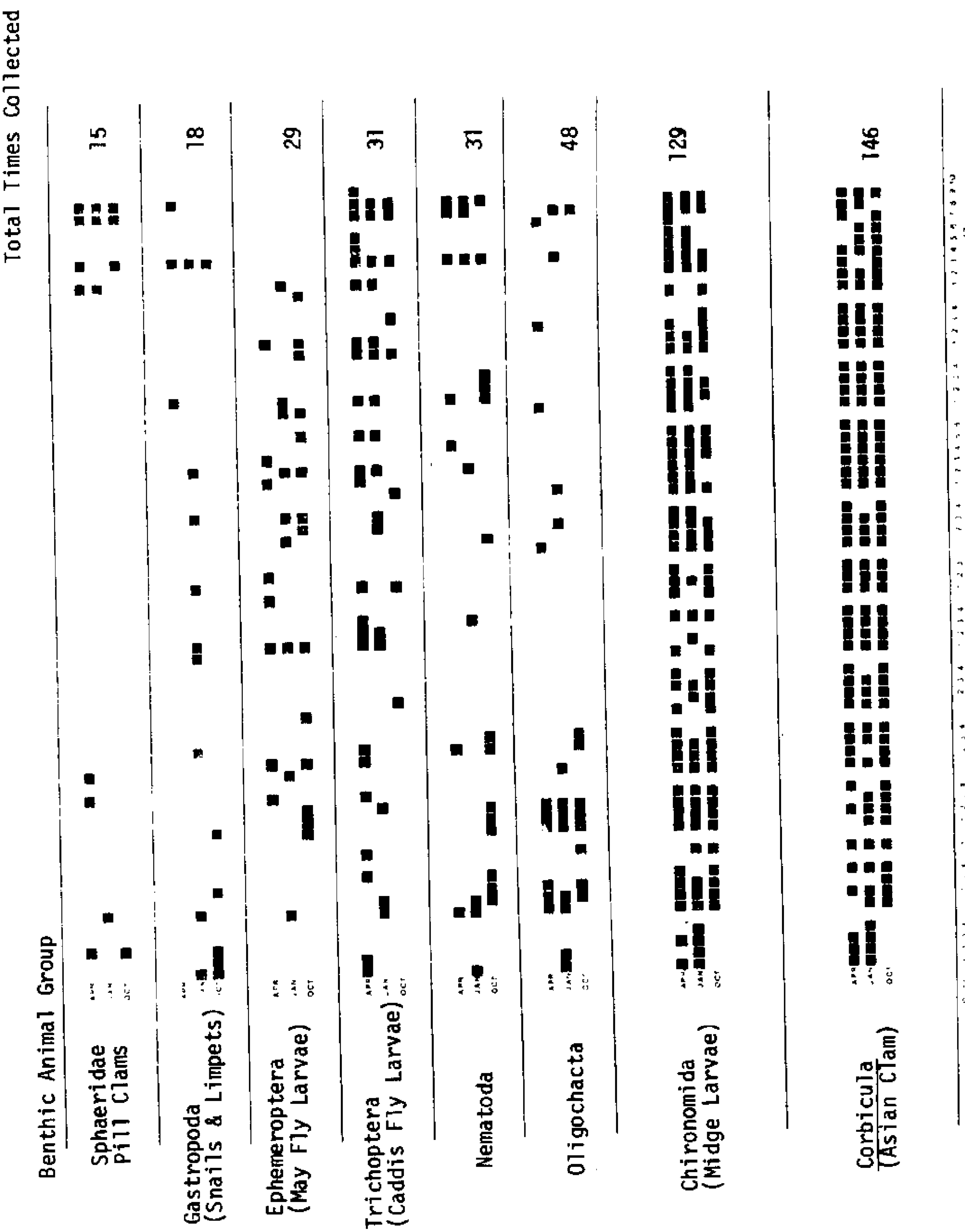

Figure 1. Incidence of prevalent kinds of organisms in benthic samples from the Arkansas River, in three sampling series (October 1974, January 1975, April 1975). (Kraemer, 1976)

the abundance of midge larvae. This may be compared with Figure 3, which indicates the substantially greater abundance of Corbicula throughout the three sampling series. While Corbicula shows some decline in number of organisms per square meter from October to April, note that average abundance of the midge larvae remains fairly constant. Seasonal decline in relative abundance of Corbicula may be related to life history patterns (Sinclair, 1971). At any time in the sampling series however, the numbers of Corbicula were much greater than those of the combined genera of midge larvae.

Size groups of Corbicula retrieved from the benthic samples included (a) those less than 1mm in length; (b) 2-4 mm; (c) 5-7 mm; and (d) greater than 8 mm. Most of the clams in the benthic samples were less than 4 mm (i.e. about the size of sand grains). Larger specimens of Corbicula (up to 50 mm long) were occasionally recovered from some of the fish samples taken in another part of this Arkansas River study.

SOME BIOLOGICAL CHARACTERISTICS OF CORBICULA

Figure 4 contrasts the external appearance of the clam, Corbicula with that of two indigenous kinds of river mussels, Lampsilis and Amblema. Full-grown Corbicula tend to be smaller than the mussels. Figure 5 shows two helpful distinguishing characteristics of the internal surface of the Corbicula shell. One of these is the cardinal teeth, pointed projections from the mid-dorsal part of the inner surface of the shell. (River mussels have no cardinal teeth, but do have pseudo-cardinal teeth, i.e. teeth displaced some distance anteriorly and posteriorly on the inner shell surface.) The second distinguishing characteristic of the Corbicula shell is that the lateral teeth (blade-like projections on either side of the

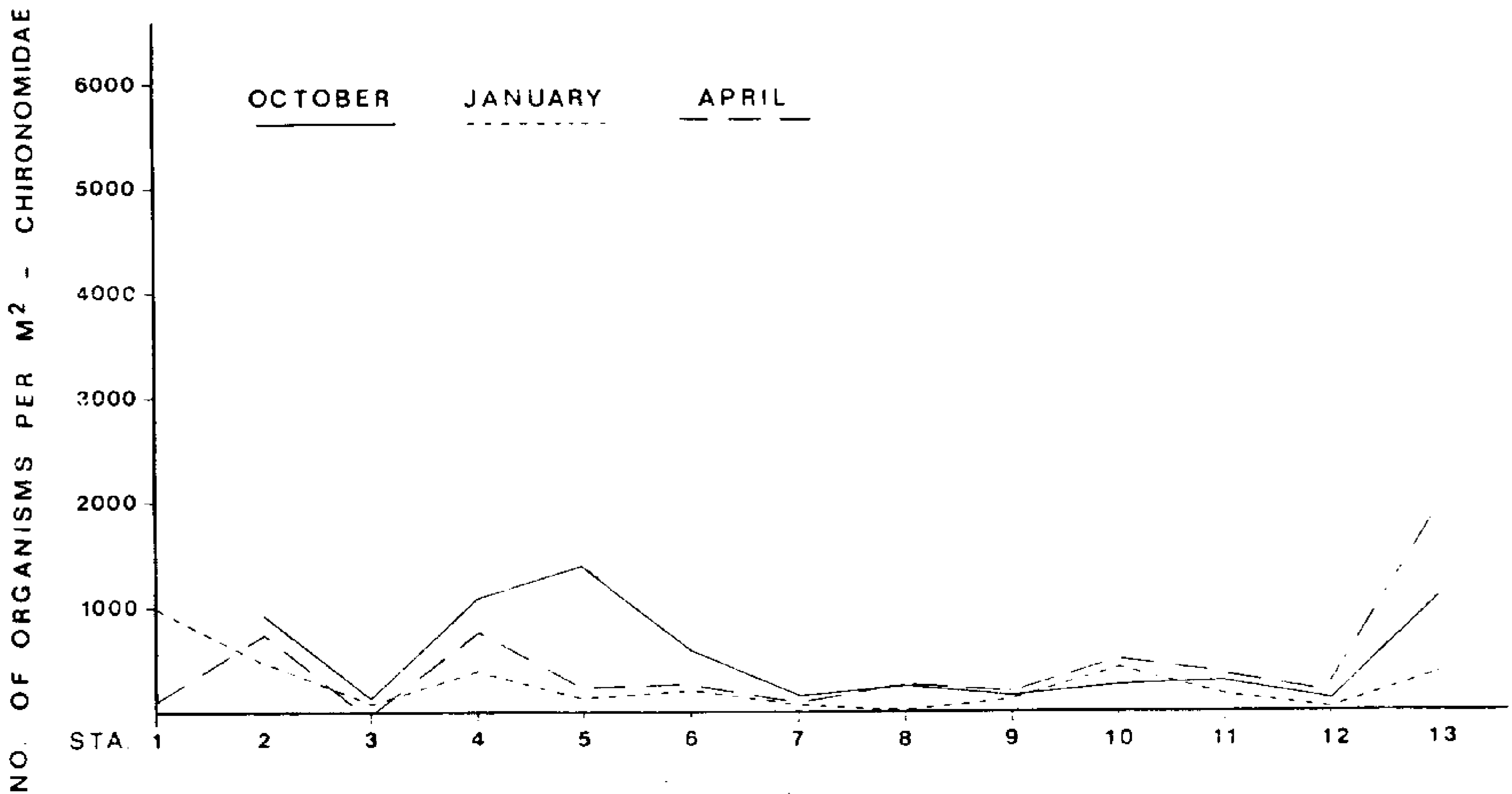

Figure 2

Abundance of midge larvae (Chironomidae) in benthic samples from the Arkansas River, for three sampling series (October 1974, January 1975, April 1975). (Kraemer, 1976)

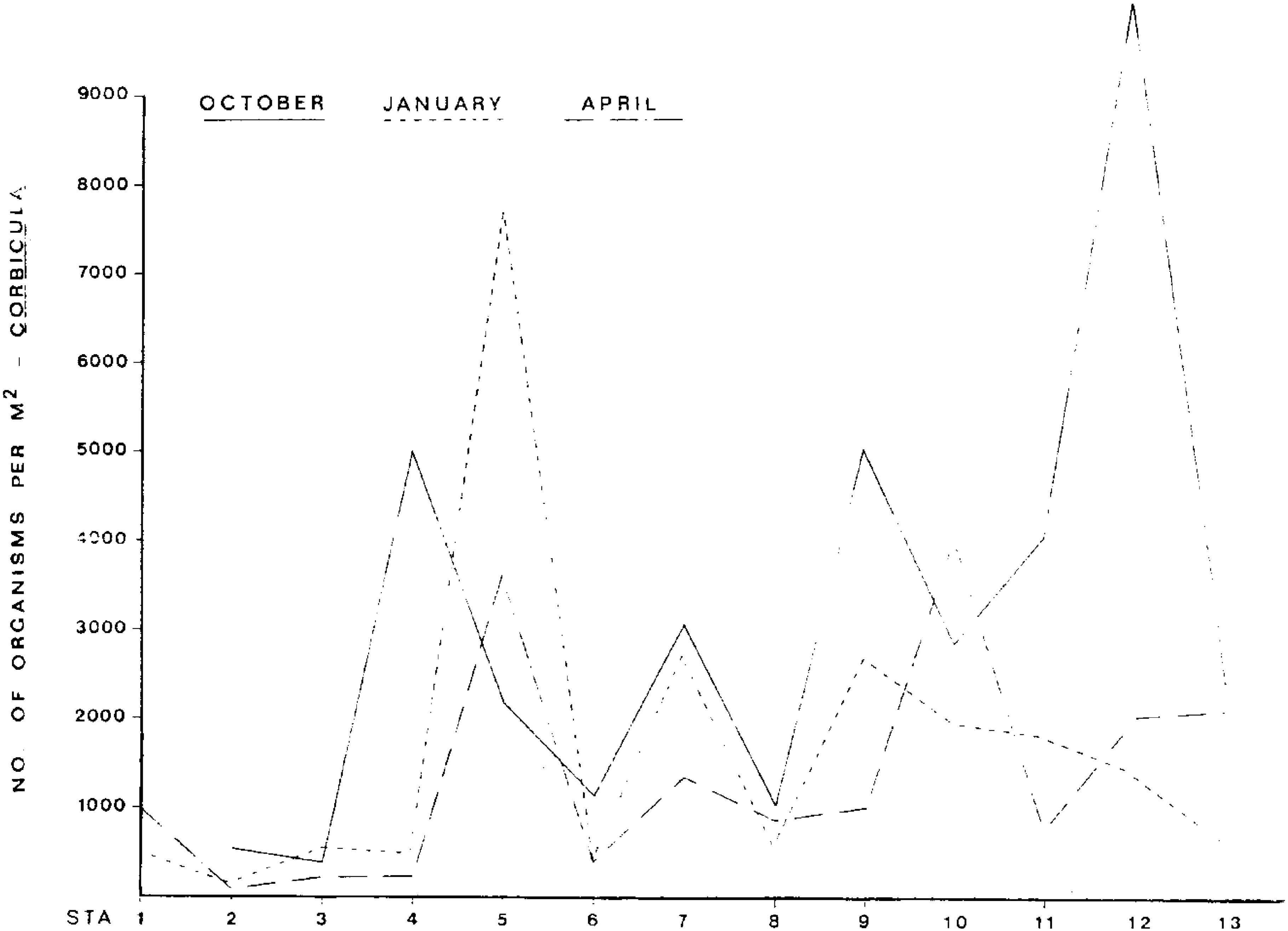

Figure 3
Abundance of <u>Corbicula</u> in benthic samples from the <u>Arkansas River</u> for three sampling series (October 1974, January 1975, April 1975). (Kraemer, 1976)

cardinal teeth) have a distinctly serrated or "saw-tooth" edge. The serrated lateral teeth are evident on the inner surface of even tiny, 1-mm-long Corbicula shells.

Like its short-lived smaller and indigenous relatives, the Sphaeriid pill clams, or finger-nail clams,(which Corbicula superficially resembles) Corbicula typically produces both eggs and sperm cells in the same animal (is hermaphroditic), and develops embryos which are housed in brood pouches in its gills. The juvenile pill clams (12-20 of them) are released directly into the water from the parent's tissues, and grow at once into adults. In contrast, Corbicula produces hundreds of tiny floating larvae or veligers, which soon settle down to grow into juvenile clams. Also unlike the pill clam, the juvenile Corbicula grows a slender, transparent, very tough byssal thread, shown in Figure 6. The young clam can twine its byssal thread about grains of sand, etc. in the substrate and thereby gain a tenacious "hold" there.

Unlike Corbicula and the sphaeriid pill clams, the typical river mussel has separate sexes (is dioecious). The female river mussel uses her gills as brood chambers for hundreds of larvae called glochidia, which are subsequently discharged into the water and attach to the fins or gills of particular fish host species. After a few weeks or months as parasitic "hitch-hikers", the metamorphosed glochidia drop off the fish host and sink onto shoals on the river bottom where they grow into adult mussels.

Some of the foregoing characteristics of the Asian clam, Corbicula, and the indigenous pill clams and river mussels are summarized in Table 2.

Figure 4

Mature specimen (about 3 cm. long) of Corbicula, the Asian clam (upper right), contrasted with indigenous river mussels, mature specimens of Amblema (upper left) and Lampsilis (below). All specimens from the Arkansas River, 1974. Note the characteristically triangular shape and distinct horizontal ridges on the external surface of the Corbicula shell. (line = 1 cm.)

Figure 5

Distinguishing characteristics of the inner surface of the Corbicula shell. C=cardinal tooth; S=serrate edge of lateral teeth. (line = 1 mm.)

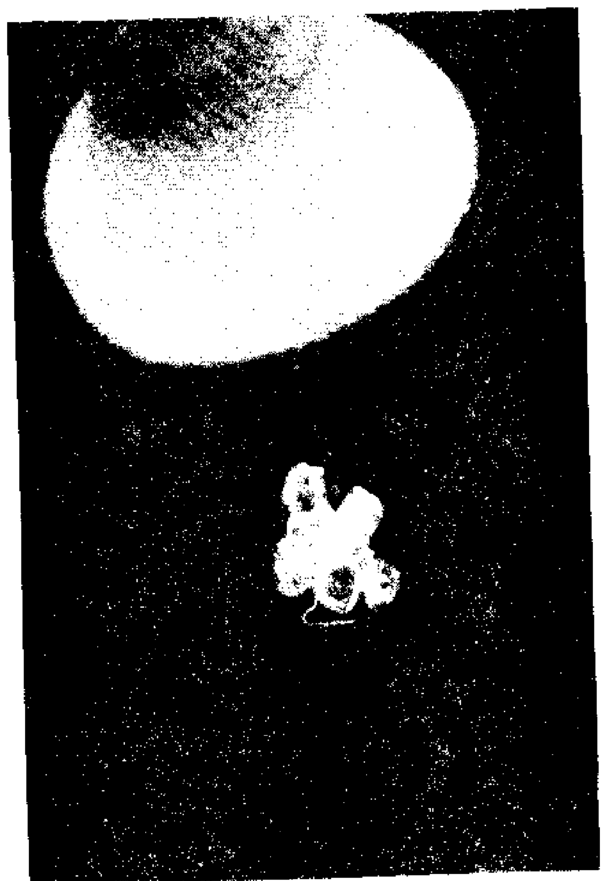

Figure 6

Photograph (x30) of juvenile Corbicula, showing byssal thread dangling from mid-ventral surface of shell, and twined about some sand grains (center of figure). (line = 1 mm.)

Table 2. Comparison of some characteristics of the pill clams (Sphaeriidae), of the Asian Clam (Corbicula), and of the river mussels (Unionidae). Asterisk (*) indicates characteristics peculiar to Corbicula.

Characteristic	Asian Clam (Corbicula)	Pill Clam (Sphaeriidae)	River Mussel (Unionidae) #
taxonomic group	Sphaeriacea	Sphaeriacea	Unionacea
sexes	hermaphroditic	hermaphroditic	dioecious (separate sexes)
early development of embryos	in gills of parent	in gills of parent	in gills of female
larva	*a floating veliger	none	a glochidium (parasitic on gills, fins of fish host)
byssal and/or attachment thread	*byssal thread on juvenile provides hold on substrate	none (direct development into adult)	some glochidia larvae have larval thread which may aid in attachment to fish host
adult size	*3 - 7 cm.	1 - 1.5 cm.	8 - 10 cm.
life span	*probably about 3 years	about 18 months	7 - 10+ years
cardinal teeth	present	present	usually absent
serrations on lateral teeth	*present	absent	absent
habitat	ubiquitous? increasingly in disturbed habitat; sand	backwaters silty substrate	on shoals, especially where substrate contains cobbles, boulders, etc.
method of distribution	*floating veligers	ubiquitous	hitch-hikers as parasitic larvae on specific fish hosts
indigenous	no. introduced into western U.S. from Asia, about 1938	yes	yes. center of origin probably on North American continent. many species confined to N.A.

Table II - continued

Characteristic	Asian Clam (_Corbicula_)	Pill Clam (Sphaeriidae)	River Mussel (Unionidae) #
present U. S. distribution	recorded from more than 30 states. spreading rapidly, especially in southern U. S.	ubiquitous	much diminished in recent years. many species on U. S. endangered species lists.

There is some controversy among malacologists at the present time as to whether river mussels should remain grouped into one family, Unionidae, or be split into 2 families, Unionidae and Amblemidae.

SOME PRACTICAL IMPLICATIONS OF THE FOREGOING FINDINGS

Among benthic biologists, power company engineers, etc. *Corbicula* is increasingly notorious. The animal has spread at an alarming rate through many drainages in the U.S. in the last few years. The rapid spread of *Corbicula*, especially in heavily managed rivers, coincides with rapid disappearance of indigenous river mussels (Unionidae) from those same rivers. Indeed, many species of river mussels are now on endangered species lists.

We have reason to believe *Corbicula* has been present in Arkansas rivers only since the mid-1960's (Kraemer, 1976). On the West Coast, in Tennessee, and increasingly in other U.S. areas, *Corbicula* has become a familiar scourge as it clogs power companies' intake screens. In the winter of 1975 at a meeting in Atlanta, Georgia, the shared concern of benthic biologists and power company engineers, etc., surfaced. The group realized that *Corbicula* must become the target of intensified data-gathering and research since far too little is known of the habits and basic biology of the animal. Subsequently a publication now known as the *Corbicula Newsletter* was inaugurated (Mattice, 1975).

Studies going on in our own laboratory seem to indicate that *Corbicula* is not necessarily a physiologically hardier animal than the river mussel. Rather, it seems the removal of habitat required by the indigenous river mussels (shoals, large and/or varied substrate particle size, access to necessary fish hosts) allows the successful establishment of the exotic Asian clam. Once established, this animal's large reproductive potential, its floating veliger larvae, its strong "holdfast" byssal thread, its relatively direct life cycle, its apparent longevity, its acceptance of sandy substrate--enable it to "take over" the river bottom.

In the present study, only one of the collection stations (station 6 at river mile 171) was dredged repeatedly at all of the sampling sites. Coincidentally the sampling sites at this station also invariably contained the fewest as well as the fewest kinds of benthic animals (primarily *Corbicula*). These findings may indicate a biological profile for disturbed (dredged) benthic habitat. Other workers (e.g. Crossman et al. 1973) have cited cogent reasons for emphasizing study of communities of benthic animals in streams subjected to environmental stress. Some (e.g. Cairns, 1971) are pointing out possible long-range benefits of benthic biomonitoring projects in streams.

Corbicula appears now to be the most characteristic benthic animal in the Arkansas River. The clam's establishment there coincides with the development of regular maintenance dredging in the river. Maintenance dredging will necessarily continue there. We can thus anticipate the need to know much more about what *Corbicula* is doing in the river, and what its long-term effect on the river may be. We can also note that even a single benthic sample containing just *Corbicula*, provides much more direct evidence of the livability of dredged river bottoms than do dozens of physico-chemical tests. It therefore seems advisable for any program assessing future effects of dredging, etc. on the Arkansas River to include year-to-year monitoring of the most prevalent animal on the river bottom, *Corbicula*.

A SUGGESTION FOR THE UTILIZATION OF EXISTING CORPS PERSONNEL, ETC. IN A PROGRAM FOR BIOMONITORING THE RIVER

A few changes in existing procedures could provide data for a biomonitoring program relative to effects of dredging activities, etc. These might include the following:

(1) Collection of regular data on location of actual dredge spoil sites. (This ought to include commercial as well as Corps dredging activities.)

(2) Collection of regular benthic samples above, at and below certain designated sites on the river. Examination of these samples, especially with regard to the composition of their *Corbicula* populations.

(3) Continuing collection of benthic fauna distribution data (especially of *Corbicula*) in the Arkansas River, for the purpose of anticipating environmental effects of any substantive change in benthic populations.

(4) Periodic comparison of benthic samples from heavily dredged areas with samples from undisturbed areas, for the purpose of monitoring real alterations of river benthos which seem related to dredging activities.

(5) Consideration of seasonal changes in benthic fauna (especially *Corbicula*) which might indicate preferred dredging times, etc.

Of course existing constraints on time and costs within the district would need to be considered in connection with any of the above undertakings. As an outsider it occurs to me that a review of water sampling and sediment sampling protocols now in use by the Corps might allow for innovative incorporation of the above procedures. In addition, more assistance should quite readily be available from the local and increasingly informed academic biological community.

A consequence of this study for me has been the realization that academic biologists and Corps engineers can and should work together on a continuing basis, to help each other anticipate problems. In the long run, such an interdisciplinary habit should minimize preventable environmental mishaps and reduce the possibility of costly confrontation with concerned citizens' groups.

Finally, should the Corps of Engineers be concerned about the presence of the Asian Clam, Corbicula in the Arkansas River? Yes, I think they should be concerned. I hope this article has indicated ways in which this could be a shared and constructive concern.

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Cost-effectiveness Analysis of Solids-Liquid Separation Alternatives in Dredged Material Disposal Operations

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ABSTRACT

An analysis has been carried out to find the most cost-effective system(s) to separate polluted dredging solids from their carriage water and meet effluent limitations in confined disposal operations. Candidate systems include sedimentation basin(s) and solids dewatering followed by polishing element(s). Polishing is achieved by flocculant addition, mechanized and semi-mechanized granular media filtration and mechanized straining. Annual costs of several example systems to meet a given effluent quality are compared. The stringency of standards imposed, the portability of equipment between disposal sites and ultimate solids disposal are the most crucial determinants of cost and choice of systems. Portability substantially reduces annualized cost. Very stringent standards imposed may preclude all but the use of chemical flocculants for effluent polishing. Extent of solids handling prior to ultimate disposal increases strongly as land costs at the disposal site increase. Further research is indicated to define applicability of several concepts to disposal operation, as well as dependence of performance on dredged material characteristics. Pilot testing is indicated for several of the polishing alternatives.

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Introduction

Maintenance of harbors and navigable waterways requires periodic removal and disposal of deposited sediments, often on a very large scale. In the United States, approximately 400 million cubic yards (300 million cubic meters) are dredged annually to fulfill this need (so-called,"maintenance dredging"), to develop new projects, and to expand existing system capacity (13). Sediments dredged in the performance of these functions have become increasingly more contaminated due to the polluted nature of the waterways. Industrial and domestic wastes receiving inadequate treatment, or none at all, contribute significantly to the contamination. Another major source of polluted sediments and water is unchecked and/or untreated urban and agricultural runoff.

Traditionally, dredged material disposal has been accomplished through open-water dumping sufficiently removed from the dredging site to prevent interference with beneficial water use, but near enough to minimize costs. Environmental concern has led to a trend towards disposal of polluted dredged material in diked contaminated areas. These disposal facilities provide a solution to the solids disposal problem, and allow decreased degradation of the receiving waters.

The quality of the effluent from the containment areas is a major environmental concern in this approach to dredged materials disposal. Prior to the introduction of the new interim guidelines (Public law 92-500, Section 404b) and the Jensen Criteria (13), the effluent-suspended solids concentration was the chief water quality parameter evaluated. Thus, solids-liquid separation by sedimentation in the disposal area provided the chief means to meet regional water quality standards. The various Army Corps of Engineers jurisdictional regions in the U.S. imposed highly varying effluent standards on waters overflowing diked disposal areas, ranging from 50 JTU turbidity and 1.5 times above the ambient-receiving water turbidity or suspended solids, to as high as 13 g/l. The wide variation in standards is partially explained by differences in dispersion and mixing patterns in the receiving waters, solids size, and the organic or inorganic nature of the solids. It is easy to see that suspended solids have been a major parameter of concern because they are considered a pollutant in themselves and they can transport other pollutants whether they be adsorbed, exchanged, or precipitated on the solids.

The new guidelines require a case by case evaluation of discharges from disposal areas. In order to provide a basis for evaluation of the effluents, the U.S. Army Corps of Engineers and the Environmental Protection Agency will prepare an interagency manual to define tests, procedures, calculations, etc.(20). At this time, it is unknown how the new guidelines will address levels of effluent-suspended solids from containment areas, except that local water quality criteria must be met. For this reason, a wide range of guidelines must be considered at present and probably in the future. Besides safeguarding water quality, the new guidelines provide protection for some of the land area near dredging sites, a restriction which will surely increase the difficulty of finding suitable disposal sites.

The nature of the dredging operation and the type of dredge used greatly influence the quality of the disposal area effluent. The operation and equipment play a significant role in determining the size of the required disposal facility and the fluids detention time to meet a particular water quality standard. For example, in hydraulic dredging used principally for maintenance dredging on the Great Lakes and in coastal regions, pumpout water comprises 4 to 5 times the total quantity of solids dredged. Various types of hydraulic dredges are in use in the U.S., including hopper, pipeline, and sidecaster dredges, whose chief differences involve methods of final solids disposal. The slurry may be hauled internally and pumped out, piped to confined or open water areas, or immediately discharged beside the area dredged. Mechanical dredging employs dippers, clamshells, or buckets to load sediments on barges or scows for transport to the disposal site. Hydraulic dredging accounts for better than 90 percent of the annual dredging in the United States (11), and this work is directed chiefly towards effluent quality control from confined disposal facilities for materials dredged in this manner.

The quantity of dredged material, the manner in which it is dredged, and water quality guidelines to be met illustrate the complex nature of the dredged material disposal problem. Attention here is focused on the effluent quality problem, and its cost-effective solution, but the nature of confined disposal operations dictates that solids handling certainly cannot be ignored, and moreover is an overriding economic factor.

The entire containment facility for sediment disposal must be analyzed as a solids-liquid separation system. Effluent quality control is not simply the selection of a single unit process to meet imposed water quality standards. Sedimentation, flocculation and filtration merit consideration as unit processes for adaptation to effluent quality control.

A design methodology for effluent quality control is illustrated in Figure 1 which documents a recent study by Krizek, FitzPatrick, and Atmatzidis (9,10). Following definition of dredged material disposal as both a solids disposal and water quality problem, a logical first step to a solution is a study of existing solids-liquid separation technology applied to similar problems. Laboratory and ultimately field testing of the promising traditional concepts needs to be done to determine the feasibility and application or adaptation of present technology to the problem at hand. Simultaneously, new concepts can be developed as possible solutions. Design alternatives can be determined from combinations of current technology in conventional and novel configuration, and application of new concepts developed in response to the specific nature of the dredging disposal problem. This process has already been considered in the aforementioned comprehensive study (9,10). The analysis performed herein considers preliminary designs and their cost-effectiveness, with pilot testing remaining to verify the theoretical systems.

Examination of previous work (9,10) indicates that several techniques can be used individually or collectively, to meet particular effluent quality standards, if large enough sizes, or a sufficient number of

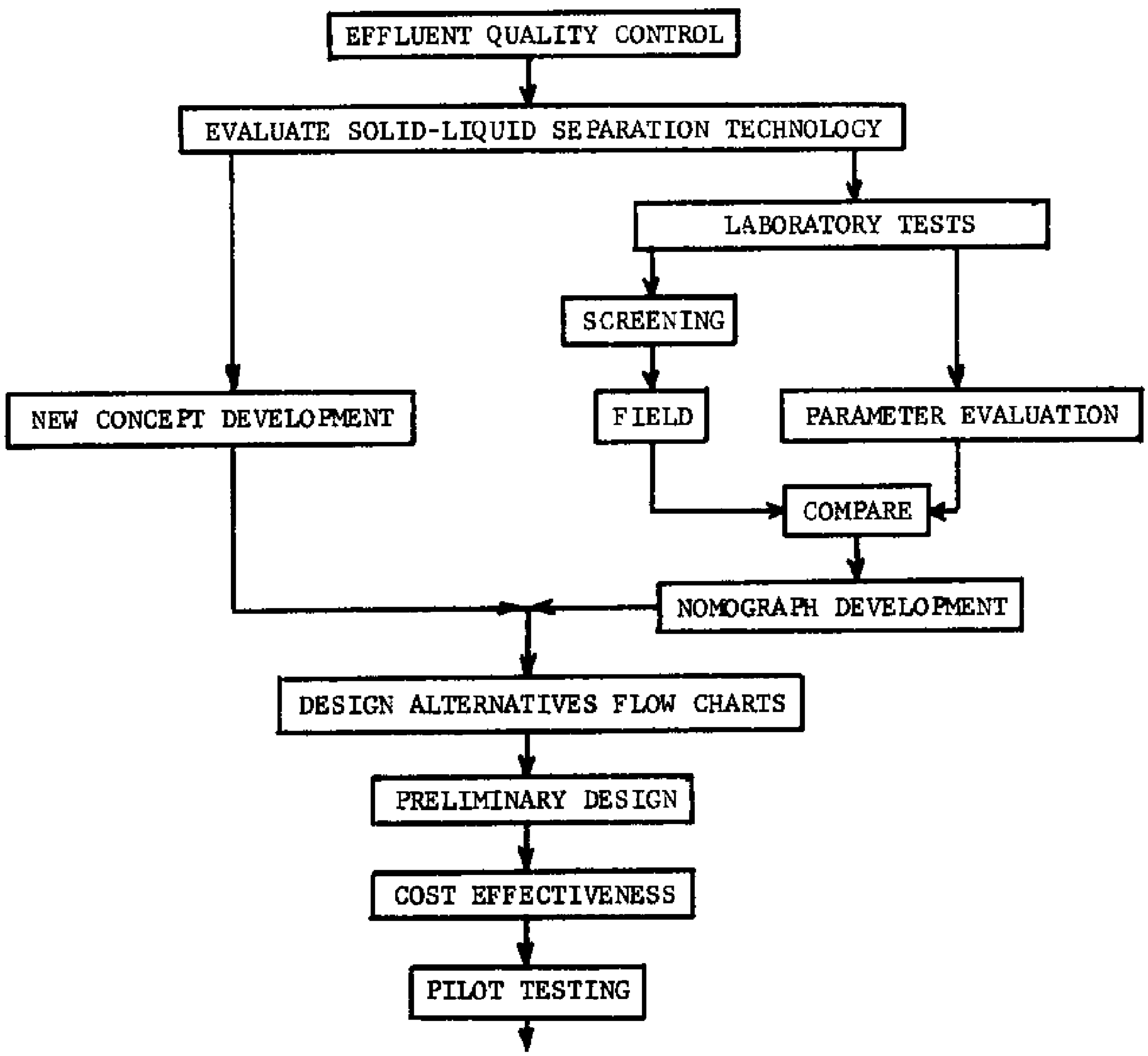

FIGURE 1 DESIGN METHODOLOGY FOR EFFLUENT QUALITY CONTROL

units is employed. By varying the sizes and number of units combined a large number of systems could be employed to meet effluent requirements. At this point, economics must be considered so that the most cost-effective system is designed.

The cost-effectiveness approach to decisionmaking has received significant attention in recent years, being applied to water supply (3), wastewater reuses (8), and solids disposal (18). To carry out this type of analysis, extensive cost and effectiveness information must be available, at least to the extent that reasonable estimates can be made.

The wide cost difference for several schemes designed to meet a particular water quality standard for an assumed dredging operation further indicates the need for a cost-effectiveness approach to dredged material disposal. A cost breakdown shows which elements contribute the greatest fraction of the total cost and dictates where efforts should be directed for cost reduction. For example, making some equipment portable can result in a significant decrease in the treatment system cost.

In most cost-effectiveness analyses, a non-linear function is expected, resulting in a different cost per unit effectiveness throughout. For this reason, the cost and effectiveness of a particular solution cannot be extrapolated for all cases. In any case it is clear that a common basis must be established, either fixed-cost or fixed-effectiveness, the latter being more applicable in dredged material disposal. For a given site, the known water quality standard can be combined with the influent water quality to yield the solids removal efficiency required for that site.

Methodology for Cost-Effectiveness Analysis

In order to carry out a useful and valid cost-effectiveness analysis, a great deal of information on both cost and effectiveness is required. One difficulty in undertaking this analysis is the fact that many of the proposed treatment techniques have never been applied to dredged material disposal. Thus much of the cost data and the operating characteristics of system elements must be estimated from similar use in industrial and wastewater treatment. Projection of data to larger scale is required in some cases, because several of the techniques considered have never been attempted at the magnitude required in dredging operations. Due to limited research on dredged material characteristics required for application of several unit processes, it must be assumed that the available data is representative. This was necessary in the use of flocculants with dredged material, and for determination of the specific resistance of the dredged material for vacuum dewatering. Moreover recent research appears to substantiate the approach taken and the data employed (2).

In this analysis, a methodology for system combination was developed after a compilation of the estimated cost and effectiveness data (5). Many sources were used in the acquisition of the cost and effectiveness data, resulting in a wide variety of unit bases (area, volume, weight), unit systems (English, metric), and cost-base years. To simplify use of the collected information, a common basis was used, where possible. Several graphs were developed to aid in quantity conversions from a weight to volume basis, depending on the solids concentration in the slurry.

This work has as its purpose the development of a methodology whereby systems designed to meet particular effluent water quality standards can be compared on a cost basis. As new information becomes available, e.g. pilot data from component field installations of processes as described herein, this analysis can be easily updated. The organization of this analysis is such that computer coding is possible, further simplifying updating and allowing more complex consideration of specific characteristics of the site (geography, topography) and dredged material (particle characteristics and resistance to dewatering). Application of the methodology also indicates which unit processes accomplish the best solids removal at a minimum cost, thus permitting better direction of future research and pilot testing on the most promising processes.

In order to illustrate the magnitude of the costs involved in confined disposal operations, three treatment systems are considered for a particular dredging operation and assumed water quality requirement.

Before such a system analysis can be undertaken the technology of solids-liquid separation and projected costs and effectiveness must be examined along with the methodology for combination of elements into systems.

Technology of Solids-Liquid Separation: Effectiveness and Costs

The correct philosophy in utilizing solids-liquid separation technology emphasizes the need for system design rather than the selection of a single process to meet effluent standards. The recent study by Krizek, FitzPatrick, and Atmatzidis (9, 10) indicates that several techniques can be used individually to meet a particular effluent quality, if large enough sizes or a sufficient number of units is employed. To capitalize on the characteristics of several unit processes, the following general separation technologies were considered in this analysis: a) sedimentation, discrete and flocculated, b) mechanized surface filtration, and c) granular media filtration.

Sedimentation

Traditionally, the most basic, widely used, and least expensive approach to solids-liquid separation has been sedimentation. A confined disposal area serves as a settling basin since in a typical facility, the dredged slurry is pumped into the area on one side and the effluent is discharged from a point roughly on the opposite side. In this manner, sedimentation of discrete particles occurs in what must be considered as primary treatment, where sufficient time is provided for solids removal to meet water quality guidelines, and/or equalization to allow uninterrupted use of a secondary treatment element for effluent polishing.

The suspended solids concentration in the effluent from a disposal area (considered a sedimentation basin) is a function of several parameters including the relative locations of the inlet and outlet, the nature of the dredged material, detention time of the fluid, the nature of the flow path through the settling area, the quantity and type of vegetation, and wind direction and velocity. Krizek, FitzPatrick and Atmatzidis (9, 10) describe a means to analyze the separation effectiveness of a settling area, based on classical sedimentation theories. Nomographs were developed which predict effluent quality based on particle sizes and the influent slurry solids concentration. Field data corroborates predictions based on these nomographs.

Reported costs of disposal facilities show a wide range of unit costs when compared on a common dollar basis (July, 1975). In Table 1 costs of diked disposal areas are compared on a unit area basis because the surface loading rate is the key parameter in a sedimentation efficiency assessment. It is necessary to distinguish between facilities built on land and those constructed by diking an underwater area because of differences in applicable construction techniques and required operation and maintenance. The values reported in the table are from several sources (17, 22) and consider dike construction and use in coastal regions and the Great Lakes. The costs reported in Table 1 are amortized at 6 percent for 10 years after converting to a unit area basis.

One overriding aspect which cannot be neglected in designing disposal facilities employing sedimentation is that of ultimate solids disposal. Sufficient volume for storage of solids for the life of the facility must be provided, or allowance should be made for excavation and ultimate disposal elsewhere. The costs presented in Table 1 are for diked areas with an average depth of 10 to 15 feet (3 to 4.6 m) with some depths as great as 30 feet (9.2 m). Selection of basin size must be based on both sedimentation and ultimate solids disposal. An economic tradeoff must be made between minimum sedimentation and temporary solids storage on site, and large-scale sedimentation and permanent solids disposal within the facility. Effluent polishing costs must be considered, but sedimentation and solids disposal appear to be the major factors.

Table 1

Unit Costs of Diked Disposal Areas (Per M^2)
(1 m^2 = 10.75 ft^2)

Component	Range		Average
Land	$0.014	$0.841	$0.218
Construction			
Land-Based	0.002	0.176	0.062
Water-Based	0.073	9.787	3.125
Operation and Maintenance			
Land-Based	0.017	2.938	0.990
Water-Based	0.014	2.473	0.741

The surface area required for sedimentation can be reduced by the addition of chemicals to the dredged slurry. Studies conducted by the Dow Chemical Company (7) and the Galveston District of the Corps of Engineers (14) indicate flocculants can be used to reduce the solids content of the disposal area effluent. An initial discrete sedimentation area may still be required, as complete mixing of chemicals and slurry is difficult at the solids concentrations (10 to 20 percent) encountered in the disposal area influent. Addition of chemicals to the effluent results in rapid sedimentation (7) requiring a small final basin.

This process can then be used effectively for polishing the effluent from initial discrete sedimentation basins. Existing disposal areas could be easily modified by placing a barrier dike across the basin, dividing it into two sections, one for discrete, the other for flocculant sedimentation promoted by chemical addition.

When the unit cost of containment facilities is projected to be very great, the use of flocculants is dictated to reduce the surface area required for sedimentation. Still, provision must be made for solids disposal for the life of the facility by use of a deep facility or inland disposal. It is conceivable that a smaller volume may be required for solids disposal as greater compaction may occur. Early tests with Purifloc C-31, manufactured by the Dow Chemical Company, indicate it is very effective in reducing the effluent solids when it is used at a concentration of 4 to 10 mg/l (7). The effectiveness of a given flocculant varies according to suspended solids concentration and the characteristics of the dredged material (12). At the solids concentrations where Purifloc is most effective (less than 10 g/l) the major cost over the lifetime of the disposal site will be chemicals; mixing equipment costs will be minor (27). The chemical can possibly be added as a liquid, with turbulent flow providing sufficient mixing. The unit cost of 5,000 lbs (2,265 kg) of Purifloc was $0.70/lb ($1.55/kg) (19) (July, 1975) with reduced unit costs for larger quantities.

Mechanized Surface Filtration

The transformation of slurried materials into a dewatered cake is the major objective of several forms of mechanized surface filtration applicable to dredged material disposal. Surface filtration techniques are common in both industrial solids handling and municipal sludge disposal. Microscreening, while not a cake filtration device, is a mechanized surface filter worthy of consideration for its clarification characteristics for filtrate or basin effluent polishing.

In recent years microscreening has been applied extensively as a clarification process in water and wastewater treatment. The Sonic Strainer Micro Screen developed by the FMC Corporation is an example of a novel microscreen design which has proven effective at high suspended solids concentrations. The unit employs a rapid rotating drum which repels most solids, concentrating them in an area surrounding the unit. The liquid flows through the drum to be discharged through a pipe at the bottom. Any solids which reach the screen are removed by an ultrasonic transducer.

This straining device has been tested principally with municipal and industrial wastes and it may not prove effective with the nature and sizes of particles encountered in dredging operations. The unit has been applied at solids concentrations of 10-20 g/l to as low as 100 mg/l, encompassing typical disposal area effluents. Reported removal efficiencies are 90 to 99 percent for particles larger than five microns (6, 16). Because the published results deal with screens of 5 microns or larger, further research is indicated for finer screens and colloidal particles prior to application in dredged material disposal operations.

The process flowrate through the Sonic Strainer is inversely proportional to the solids concentration in the fluid, indicating a large number of units or a two-stage filtration process may be required when the influent solids concentration is high. The accumulation of solids in the vicinity of the unit demonstrates the need for mobility of the unit within the disposal area, or a means to remove the solids surrounding the unit must be provided. The strainers are sufficiently small so that the units could possibly be used at several sites throughout a dredging season. An installed unit along with a conical tank for underflow solids removal is expected to have an annual cost of $5,550 (6) (July, 1975) for amortization at 6 percent for twenty years. This does not account for use at several sites. Operation and maintenance costs are projected at $7 per unit per day (6).

While the Sonic Strainer is a mechanized surface filter designed for clarification, rotary drum vacuum filters and the capillary suction dewatering device are surface filters designed for dewatering concentrated slurries. Both of these technologies require a preliminary sedimentation basin for flow equalization, a disposal area for the dried cake, and filtrate polishing facilities. Sedimentation was described previously, and filtrate processing can be accomplished by flocculated sedimentation, Sonic Straining, or by several granular media filtration techniques.

Disposal of the dewatered cake is a critical element from a cost standpoint. In dredging and subsequent slurry dewatering, the rate of cake solids production is relatively high for a short period of time i.e. when the dredge is in use at the particular site. This means an intensive scale landfilling operation must be provided for a relatively short time. The solids themselves may only be poor grade construction materials and the landfill site may be of little use when the disposal capacity is exhausted. The principal costs considered in the analysis (5) of solids disposal are the construction and operation costs of the landfill, (as reported by the EPA (30)) and land costs. Trucking costs, estimated at $0.15 per wet-ton mile ($219 per wet kg-km) based on values reported by the EPA (30) and Bennett, et al. (1) can be a major cost factor depending on cake water content and trucking distance.

Capillary suction and rotary vacuum filtration have solids yields per unit of filter area determined by the specific resistance of the slurry. Little experience has been reported on the use of dewatering techniques at the feed solids concentrations encountered in dredging operations, and data is limited with regards to the specific resistance of dredged material. The specific resistance reported for Toledo Harbor dredgings (10) is assumed representative for the purpose of this analysis. Because the scale of operation in dredged material disposal is atypical of most dewatering operations, cost projections are required from the reported values. Operation and cost data was adapted and updated from Gale (4), McCarthy (15), the EPA (29), and Lippert and Skriloa (14). It appears that capillary suction may be slightly less expensive than conventional rotary vacuum filtration.

The tremendous costs (5) associated with slurry dewatering can be reduced by use of the same equipment at several sites throughout a dredging season. The filter surface area required in typical dredging operations makes it impractical to consider installation on the dredge itself. Barge mounting and relocation with the dredge, however, is a conceivable alternative. This will allow a great reduction in the amortized capital cost, the major component of the cost. Other related costs, operation and maintenance, solids disposal, and filtrate processing, will not decrease, and could possibly increase due to continual use of the equipment, reducing its projected life. This cost reduction through use of portable equipment may make slurry dewatering competitive with other dredged material disposal concepts.

Granular Media Filters

Filtration through granular media, particularly sand and/or anthracite, is widely used in water clarification and wastewater polishing. This unit process can be employed in conventional form, or in new configurations as developed by the U.S. Army Corps of Engineers, or as proposed elsewhere (9, 10).

With specification of removal efficiency and filter depth the required effective media grain size may be computed with the aid of nomographs. Clogging time, the other major parameter, can be determined from a similar nomograph (9, 10). These graphs were based on laboratory and field testing with simulated and actual dredge slurries and overflow criteria. The nomographs are shown as Figures 2 and 3 and are discussed as an illustration of the cost-effectiveness for several filter system elements.

Conventional deep-bed granular filtration has been an important element in water purification for decades. In order to directly apply this process to dredged material disposal operations, provision must be made for frequent backwashing or large filter area, either of which may be undesirable. The required filter area should be divided into several beds to allow staggered backwashing. Cost for capital and operation and maintenance are based on EPA Technology Transfer Data (31). The reported values were converted to a metric basis and adjusted to the common base year (July, 1975). Cost savings could be achieved by using less sophisticated equipment than that normally used in water purification. The structural nature of the filter beds and appurtenances dictates that the amortization period for the capital cost should be the same as the design life of the disposal area.

Granular media cartridges are a novel application of deep-bed filtration technology. Each cartridge (diameter = 1 to 1.5 m) (3.3 to 4.9 ft) serves as a small filter which can be replaced and rejuvenated when clogging occurs. As noted elsewhere (10), cartridges allow great flexibility of operation to meet changes in sedimentation basin effluent quality. The major capital cost attributable to a given site when cartridges are used is the battery designed to hold the cartridges at the overflow structure. The cartridges, media, and media cleaning equipment need not be limited to use at one site only; their portable nature encourages continual use throughout a dredging season at all sites visited by the dredge. Annual costs are estimated from quoted media costs, unit cleaning costs and capacity, steel costs for cartridge use, and estimated concrete costs for a cartridge battery.

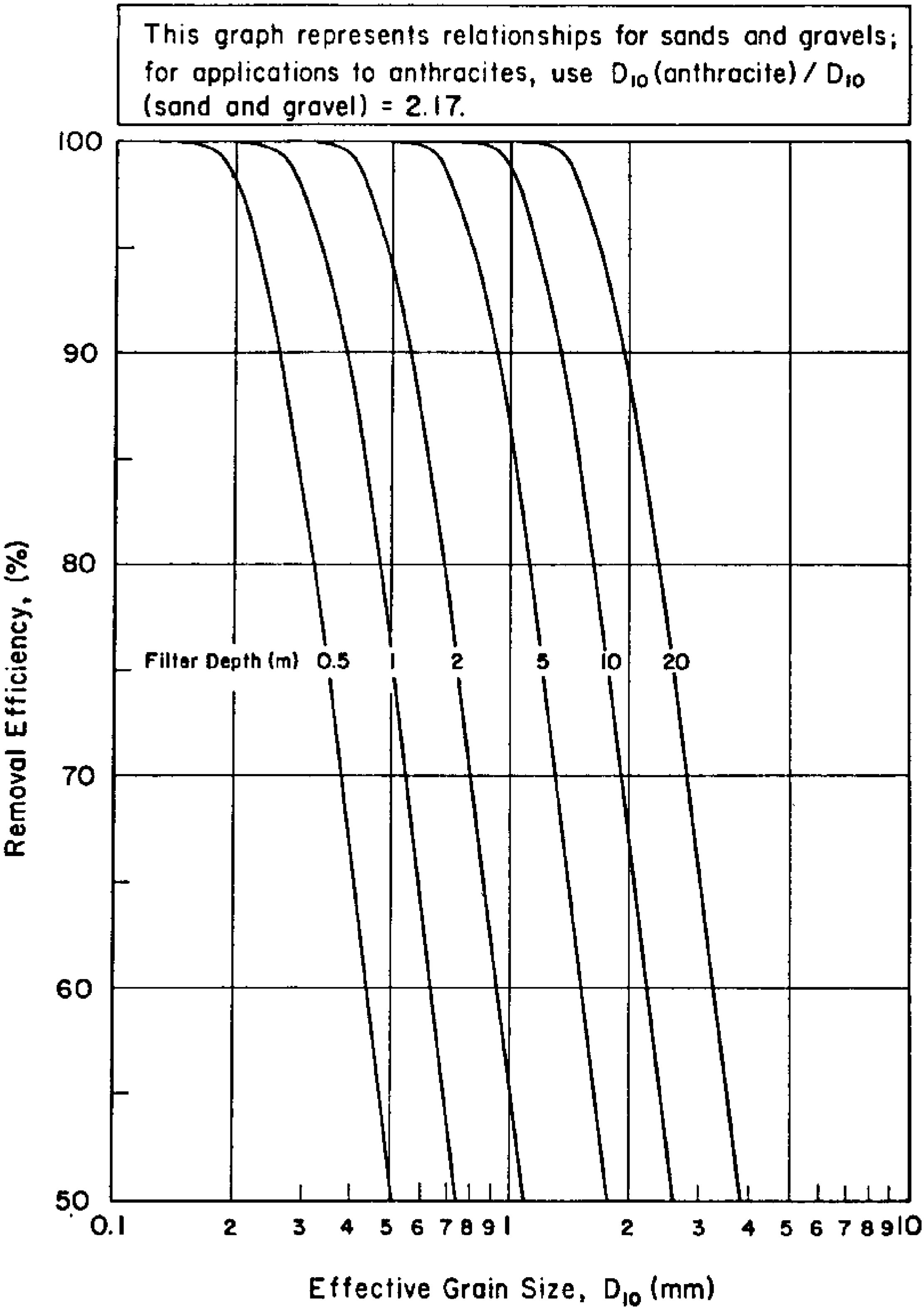

FIGURE 2 RELATIONSHIPS AMONG REMOVAL EFFICIENCY, DEPTH AND EFFECTIVE GRAIN SIZE OF FILTER (10)

Note: (1m = 3.28 ft; 1mm = 0.04 in)

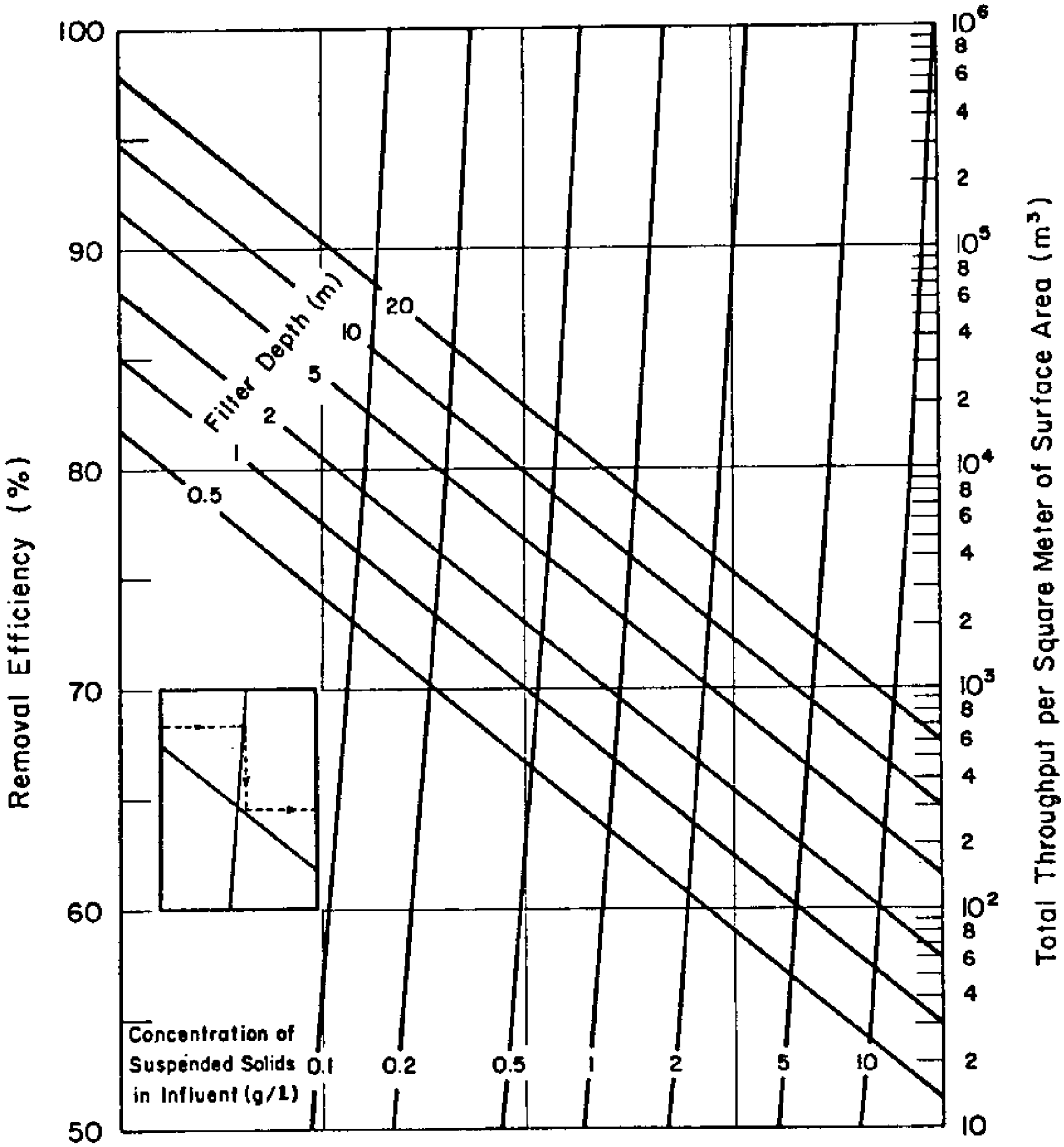

FIGURE 3 RELATIONSHIPS OF REMOVAL EFFICIENCY AND FILTER DEPTH, INFLUENT SUSPENDED SOLIDS CONCENTRATION, AND TOTAL THROUGHPUT TO CLOGGING (10)

Note: ($1m^2 = 10.75\ ft^2$; $1m^3 = 1.31\ yd^3$)

Sandfill weirs, or filter cells, like granular media cartridges, are a novel configuration of filtration technology. The Army Corps of Engineers has installed filter cells in the outfall structures of several confined disposal areas on the Great Lakes. Design of the cells, consisting of 3 ft. (0.9 m) or greater sand depth must provide for periodic media cleaning, or a sufficient filter surface area such that clogging is not foreseen for the life of the disposal area. Systems without even a simple backwash result in unrealized capacity because of early and/or inopportune filter clogging, as well as water degradation.

Capital costs for filter cells without backwash in Kewaunee (26) and Kenosha (25), Wisconsin were on the order of \$350 to \$430 (1975) per square meter (\$32.50 to \$40 per sq. ft.) of filter media, for a filter depth of approximately 1 meter (3.25 ft). Projected operation and maintenance costs (5) are a function of the time period between cleaning and the proposed method of filter rejuvenation (backwash, media replacement, on-site media cleaning).

Pervious filter dikes are a unique application of granular media filtration technology and offer an alternative to traditional disposal facility design. The dikes consist of filter media covered with rock or gravel for protection from waves and to prevent premature surface clogging. Appropriate provisions must be made for structural support and dike stability in pervious dike design. Use of filter dikes offers reduced operation and maintenance costs at the expense of considerable limitations in the flexibility of operation. If malfunctions occur, or the desired effluent quality is not achieved, corrections may be extremely expensive, if possible at all. The filter dike must be designed not to clog for the life of the disposal area, or another system element should be included to filter the effluent after the dikes clog, as is provided at Milwaukee (28). The costs of pervious diked confined disposal areas show the same wide range as that indicated for non-pervious facilities. The approximate costs for pervious dikes in Ashtabula, Ohio (21, 24), Buffalo, New York (23), and Menominee, Michigan (28) ranged from \$4 to greater than \$12 (1975) per cubic yard (\$ 5.23 to \$15.70 per cu. m.) of dredged material.

Methodology for System Element Analysis

The data collected and adapted for cost and effectiveness of unit processes to be applied in solids-liquid separation systems must be made compatible so that combination of elements into systems is possible. The available information has been gathered for determination of system cost-effectiveness by converting the data to the basis most appropriate to the process' application in solids-liquid separation (5). In the separation processes themselves, unit area is a logical basis, whereas, in solids disposal, a volume or weight base is required. Cost curves were developed on an area basis for the separation processes for use with effectiveness graphs which yield the surface area for a given solids removal efficiency and slurry flow rate.

The analysis procedure performed for each component is best illustrated through an example. To serve this purpose, conventional deep-bed granular media filtration is chosen, because it is easily demonstrated, and because the nomographs available for effectiveness can be applied to other granular media filter elements. Before filter design is approached, the solids concentration and flow rate to the filters must be projected, and the required effluent quality must be defined. A sedimentation basin is required in all systems, to provide flow equalization, as a minimum. Analysis of the confined disposal area as a sedimentation basin with known dredged material characteristics and slurry pumpout rate is performed to determine the quality of the water influent to the filters. The filter removal efficiency is easily determined from the influent and required effluent quality. After determination of the solids removal efficiency required, Figures 2 and 3 are used to specify the effective grain size and depth of filter, and ultimately the filter surface area, once the desired interval for filter rejuvenation is selected.

The result from Figure 3, the total throughput per square meter of surface area, is used to determine the filter surface area needed to achieve the required effluent quality. First, the desired time interval between backwashings must be selected, with the realization that a shorter time period permits a smaller filter area, at the expense of flexibility, downtime accounting for a greater percentage of the total time. Together the time interval, the expected disposal area overflow rate, and the throughput to clogging, determine the filter surface area. Pilot tests are required to verify proper design selection. Costs can be evaluated by use of Figures 4 and 5 for capital and operation and maintenance, respectively. The filter beds described by these figures are approximately 6 feet deep (1.83 m) and costs for other depths can be approximated by use of a scale factor. These costs are for highly mechanized, sophisticated systems, such as would be used in water purification. In application in dredged material disposal, reduced costs can be expected as less mechanized systems could be used.

The procedure outlined here for estimation of deep-bed filtration costs and effectiveness permits variation in filter design for different site conditions. Filter area, depth, and grain size can be combined in numerous ways to achieve an operational filter at a particular site to meet a given effluent quality standard. The backwashing capability of the filters allows great flexibility in the operation to handle unforeseen conditions, such as changes in the solids concentration at the filter inlet.

The cost and effectiveness of other system elements is determined by following a procedure similar to that described above. In some cases, the analysis is more complicated because some elements combine several components. For example, granular media cartridges require cost estimation of the cartridges, overflow control structure, media, media cleaning equipment, and a structure to hold the cartridges. For several other alternative elements, the effectiveness can only be approximated from the limited pilot and research data available, specifically for flocculated sedimentation and dynamic straining.

Systems Analysis

The analysis of individual solids-liquid separation elements can be summarized so that promising ones can be selected for combination into

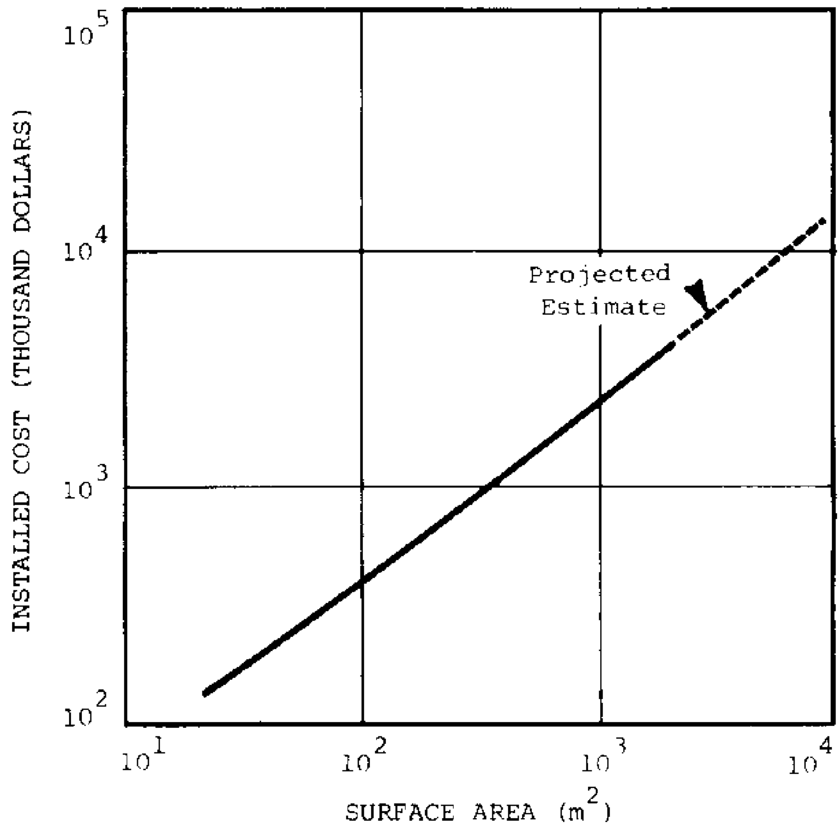

FIGURE 4 DEEP-BED GRANULAR MEDIA FILTERS, CAPITAL COST (31)

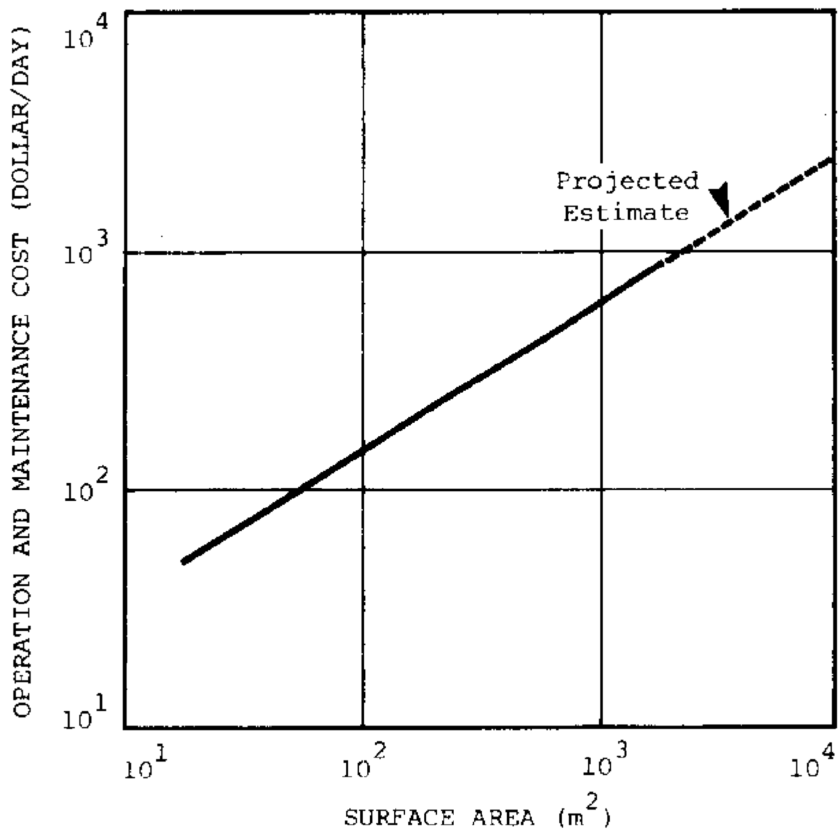

FIGURE 5 DEEP-BED GRANULAR MEDIA FILTERS, O & M COSTS (31)

NOTE: ($1 m^2 = 10.75\ ft^2$)

distinct systems. The elements shown in Table 2 have been considered in this analysis with relative costs,and limitations are indicated. The results summarized here provide guidance for the selection of the elements to be used in the systems. To illustrate the cost differences in the use of different systems for the same dredging operation and effluent quality, three systems are compared. One system employs mechanized surface filtration, while the others use different size sedimentation basins coupled with alternative effluent polishing systems. These are only three of a large number of possible system combinations which can be designed to meet a particular effluent quality.

A computer-based cost-effectiveness analysis may allow determination of the optimum combination and sizing of system elements. Use of a computer will permit greater consideration of given site conditions than was attempted here. This is most applicable to sedimentation basin costs because land or underwater topography,and land and material costs for the particular site can be considered.

A dredging operation of 10,000 cu. yd. (7,650 cu. m) per day is assumed for the purposes of this analysis. Treatment elements are considered operating 24 hours per day for the 20 days the dredge is in operation at the site. The desired effluent quality is specified as a suspended solids concentration less than or equal to 500 mg/l, a value within the range of water quality standards specified throughout the U.S. The feed solids concentration to the disposal area is taken to be 15 percent, with the in-place material being approximately 35 percent solid.

The cost analysis is based on a disposal area and structure life of ten years, at an interest rate of six percent. Equipment used at the site is assigned a life of twenty years, to account for relocation after the disposal facility is exhausted. Mechanized granular media filters are an exception because they are primarily structural in nature and are not easily relocated; they are assigned a life of ten years, the same as the disposal facility itself. The sedimentation areas are assumed to be land-based, and an average annual cost of $1.27 per square meter ($0.12 per sq. ft.) of surface area for capital, land, and operation and maintenance is used throughout.

Table 2

System Element Summary

System Element	Cost: Capital	Cost: O&M	Remarks
Discrete Sedimentation	Wide Range	Wide Range	Size- and location-dependent
Flocculated Sedimentation	Low	Low	Great promise as a supplementary treatment element
Mechanized Surface Filtration			
Solids Disposal			
On Site	Low	Mod.	Low capital cost if dikes are unnecessary; transport distance and solids content of dewatered material are crucial
Inland Landfill	Mod.	Mod.	
A. Rotary Vacuum Filtration			
Single Site	V. High	Mod.	High cost and possible requirement of filtrate polishing
Portable	High	Mod.	
B. Capillary Suction			
Single Site	V. High	Mod.	Estimate appears lower than rotary vacuum filtration
Portable	High	Mod.	
C. Dynamic Straining			Very promising as secondary treatment element; pilot testing is dictated
Single Site	Mod.	Low	
Portable	Low	Low	
Mechanized Deep-Bed Filtration	Mod.	Low	Reduced costs are possible with less sophisticated equipment
Granular Media Cartridges	Low	Low	Lower costs by reuse of cartridges at other sites; pilot testing is dictated
Sandfill Weirs			
With Backwash	Low	Low	Very site-dependent
Without Backwash	Mod.	Low	Site-dependent
Pervious Dikes			
Water-Based	Wide Range	Wide Range	Very site-dependent
Land-Based	Wide Range	Wide Range	Site-dependent

The first system considered is one employing rotary vacuum filters. Sizes and costs are approximated for a sedimentation basin providing one-day storage, solids disposal and inland landfill of the dried cake assuming a 15-mile (24.2 km) truck haul, and filtrate treatment facilities (Sonic Strainer) to accommodate the high expected effluent solids concentration. Four types of effluent polishing are considered for use with a small and a large sedimentation basin for the second and third systems respectively. The polishing elements considered are flocculated sedimentation, mechanized granular media filtration, granular media cartridges, and dynamic straining. The second system employs a sedimentation basin designed to deliver an effluent solids concentration of 10 g/l to the polishing units. Inland solids disposal is expected to be required following each dredging season. The third system yields a 1 g/l solids concentration from the sedimentation basin overflow. Less extensive polishing facilities are required in this case and ultimate solids disposal is in the confined disposal facility itself.

The costs for the treatment systems are presented in Table 3 and Figures 6 and 7. To provide greater detail the graphical analysis employs both an arithmetic and logarithmic scale. In schemes II and III, the system costs associated with each secondary treatment element represent the total cost of the polishing technique, sedimentation basin, and solids disposal, where indicated. No accounting is made for use of the equipment at several sites throughout the dredging season by barge mounting and relocation with the dredge.

Sensitivity Analysis

The results illustrated in Table 3 and depicted in Figures 6 and 7 are sensitive to conditions specified for the example. Identification of the key factors in the cost of each element will indicate possible savings in system costs and may make the costs more competitive between systems. This analysis is directed towards identification of these key factors and to give direction to research for cost reduction. Dredging

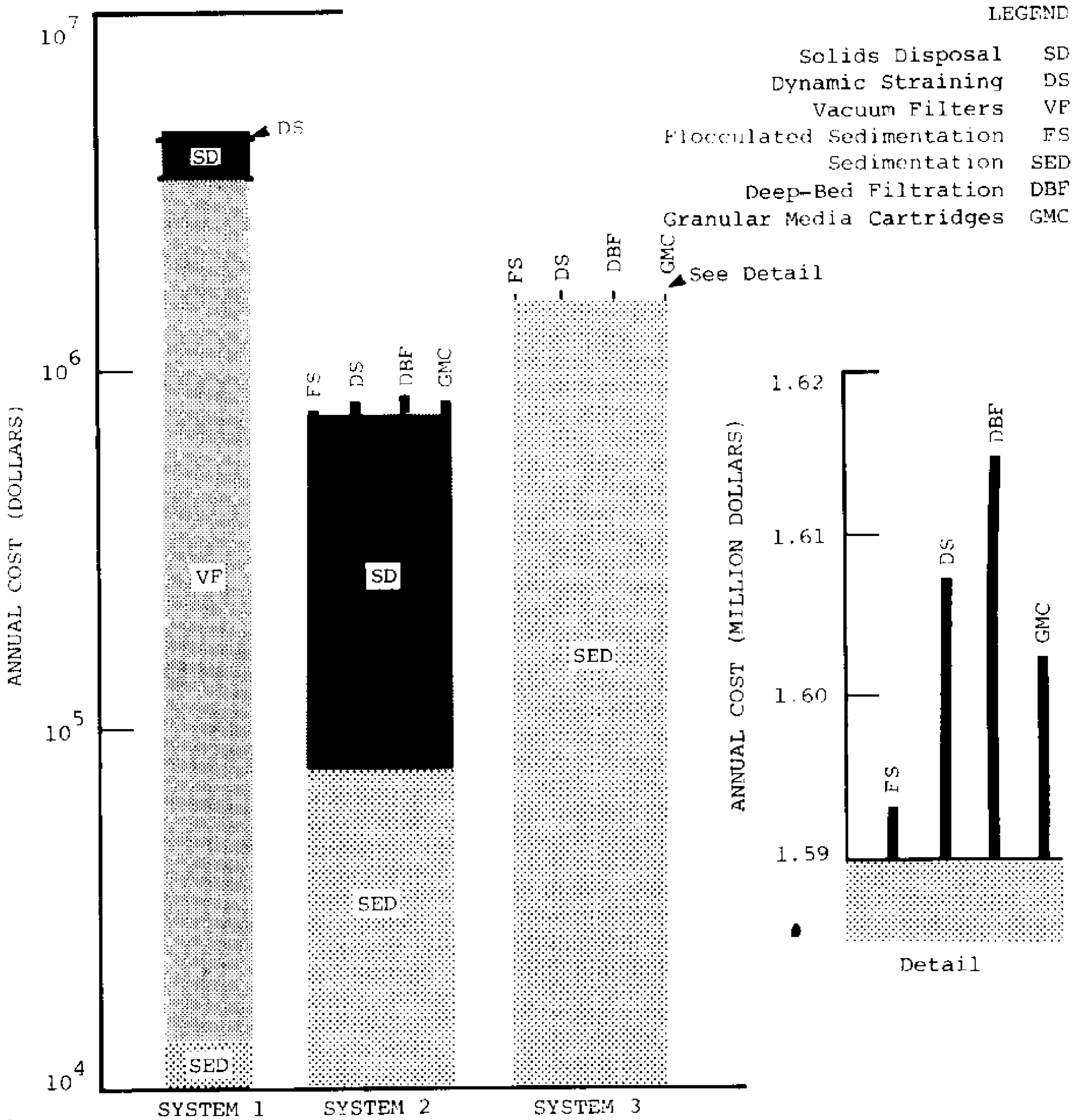

FIGURE 6 EXAMPLE SYSTEMS ANNUAL COSTS

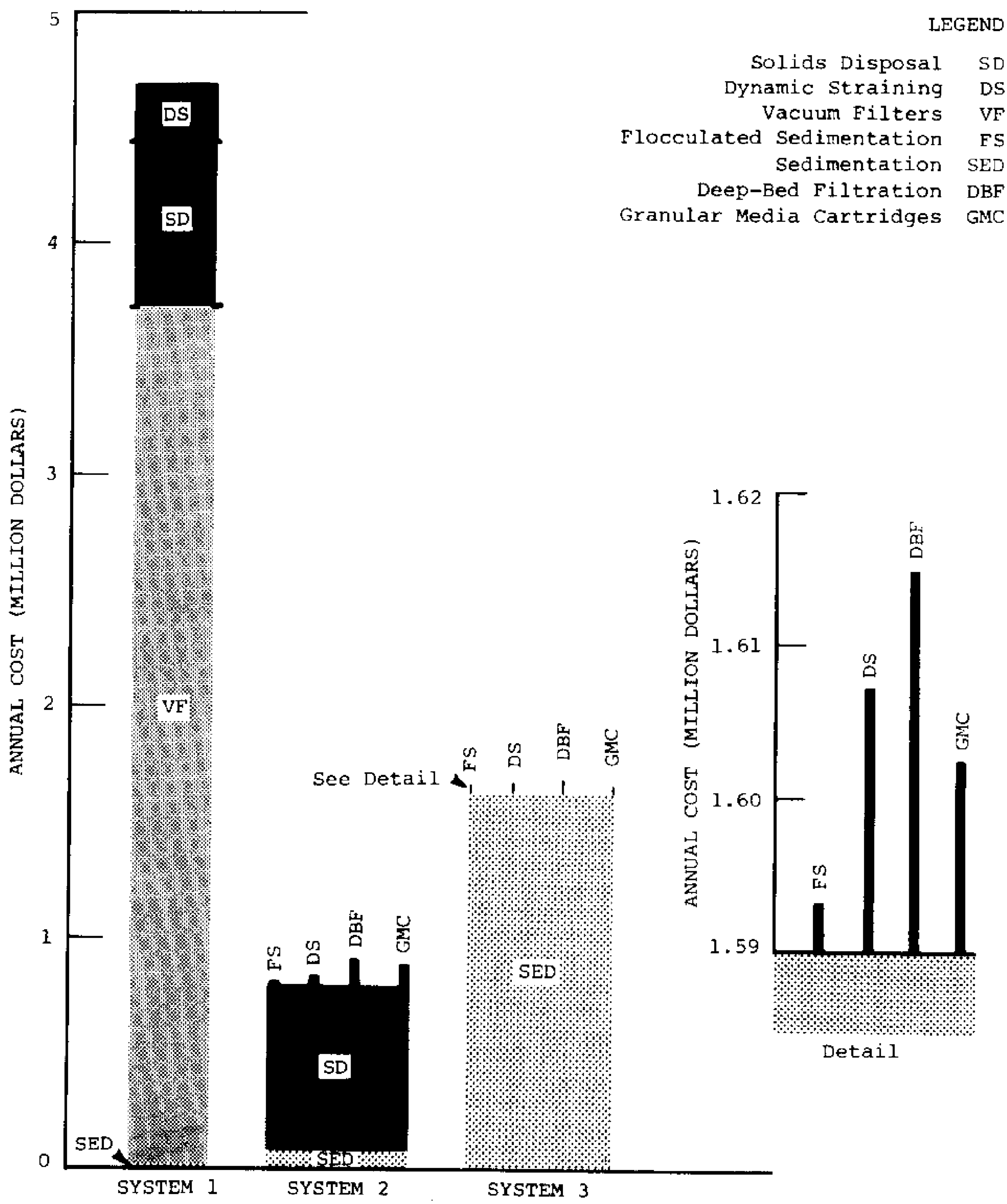

FIGURE 7 EXAMPLE SYSTEMS ANNUAL COSTS

Table 3

Example Systems Annual Costs

For 10,000 Cu. Yd. (7,650 Cu. M.) Dredged Per Day

System Number	System Element	Element Cost	System Cost
I	Vacuum Filter System		$ 4,610,500
	Presedimentation	$ 13,000	
	Vacuum Filters	3,640,000	
	Solids Disposal	707,500	
	Filtrate Processing	250,000	
II	Small Sedimentation Basin		
	A. Sedimentation	76,200	
	Solids Disposal	707,500	
	B. Secondary Treatment		
	Flocculated Sedimentation	4,950	788,700
	Dynamic Straining	34,890	818,600
	Deep-Bed Filtration	104,400	880,100
	Granular Media Cartridges	82,000	865,700
III	Large Sedimentation Basin		
	A. Sedimentation	1,590,000	
	B. Secondary Treatment		
	Flocculated Sedimentation	3,350	1,593,000
	Dynamic Straining	17,450	1,607,000
	Deep-Bed Filtration	24,840	1,615,000
	Granular Media Cartridges	11,070	1,602,000

operations a factor three greater and less than the example presented here were considered using the same element combinations. Costs vary nearly linearly with material quantities, indicating very little economy of scale. Dredging operations of short duration such as the one considered as the example may adversely affect the treatment operations because of non-steady state operation during start-up and close-down periods. In small operations, these disturbances represent a greater fraction of the total treatment time. During the non-steady state periods, the system is not expected to function at its design efficiency.

A change in water quality standards is likely to result in changes in the effluent polishing system chosen. Less stringent standards may eliminate the need for filtrate processing in the vacuum filter system. These standards will lead to the use of small sedimentation basins designed on a solids disposal basis at the expense of sedimentation efficiency. More stringent standards may dictate the use of effluent polishing system elements, possibly requiring the use of flocculants.

In this example, the amortized capital cost of vacuum filtration equipment accounts for 85 to 90 percent of the total cost of the units, with operation and maintenance accounting for the remainder. A means to reduce the cost was mentioned previously, barge mounting and relocation of the equipment throughout the dredging season. It is conceivable that the cost could be reduced by 50 to 80 percent after accounting for the additional capital for a barge and a protective structure for the equipment on the barge. The total system cost may be reduced by as much as 50 percent or greater in this manner.

The water content of the feed to the vacuum filters greatly influences the annual cost of this treatment scheme. In the example considered here, a feed to the filters at 15 percent solids, instead of 10 percent, will yield an overall cost saving of nearly 10 percent. The savings is not greater because increased filtrate processing costs outweigh part of the vacuum filtration cost saving. However, substantial overall savings can result from the removal of the coarse fraction of the influent solids by sedimentation. In fact, the cost saving is proportional to the quantity of material removed by sedimentation.

In attempts to devise portable, or relatively compact dewatering systems, alternate vacuum filtration schemes employing antecedent hardware such as trash screens, a classifier, and a thickener/clarifier have been considered. It is doubtful these could be adapted for use in treating dredged material primarily because of the nature of dredging operations. Hydraulic dredging with high pumpout rates for a short duration dictates the use of an equalization basin so that filtration equipment need not be designed for the peak pumpout rate. However, it is unavoidable that the equalization basin also functions as an inefficient sedimentation and thickening basin. Unfortunately, the alternate scheme then requires the resuspension of the solids, classification and rethickening prior to vacuum filtration. The equalization basin then requires the solids to be resuspended and thickened twice which negates the gains achieved by the smaller mechanized solids processing equipment.

Solids disposal costs are very sensitive to the distance the dredged material must be hauled for ultimate disposal. In this example, transport accounts for about half of the disposal cost, with landfill operation costs being about 25 percent. Land costs are expected to become an increasingly greater cost factor in the future as available landfill sites are exhausted.

Another important factor in solids disposal is the water content of the material and possibly a tradeoff can be made between the costs of drying the material, the transport distance and costs, and escalating land costs. It is possible that it may become more economical to pump the slurry inland to a landfill site where dewatering techniques can be applied.

The cost of sedimentation is a function of site conditions, being greatly influenced by land or underwater topography and materials costs. The costs of diked disposal areas in Table 1 show that for land-based systems, operation and maintenance account for about 80 percent of the costs, on the average, with land accounting for about 20 percent and amortized construction costs being a very small factor. Water-based systems appear to be much more expensive for construction due to provisions for protection of the dikes from wave action. Reported land costs are expected to increase in the future due to the impact of the interim guidelines and the exhaustion of suitable sites. It is axiomatic that where a small sedimentation basin is used to avoid expensive basin costs, ultimate solids disposal must still be taken into account.

Flocculated sedimentation costs are greatly influenced by the cost of the chemicals themselves especially in the case of Purifloc C-31. The final sedimentation basin can be a diked section of the confined disposal facility. Discrete sedimentation must precede use of flocculants so that complete mix and maximum flocculant efficiency is achieved.

The chief cost associated with dynamic straining is the amortized capital cost. It is likely that these units could be easily transported from site to site with the dredge, resulting in tremendous cost reductions for each site, possibly as high as 80 percent. A possible limitation on use of these units is the fact that the processing rate is inversely related to the solids concentration at the unit indicating a large number of units are required at high solids levels. The equipment works best at reduced solids loads (less than 10 to 20 g/l), and as yet little research and testing has been performed with these units and the clay type particles often encountered in dredging.

The costs of mechanized granular media filters were described previously. As is characteristic of most filter systems, the amortized capital cost represents the largest portion of the cost of using the filters, accounting for greater than 90 percent of the total in this example. Use of less sophisticated equipment could result in a substantial reduction of the total filter cost.

The use of granular media cartridges in dredged material disposal may be impractical in cases where the liquid delivered to the cartridges has a high solids concentration. The cleaning frequency and the solids concentration at the filter inlet greatly influences the number of cartridges required. While the estimated cost for cartridges may be low, it may become unrealistic to work with more than a given number of cartridges, possibly 100 being the dividing line. In the example, the cartridge battery and operation and maintenance costs each account for about one third of the total. The remainder is attributed to the amortized capital cost of cleaning equipment, media, and cartridges. Reduced costs are foreseen by use of the cleaning unit, media, and cartridges at several sites throughout a dredging season. Cartridge battery costs are not likely to be reduced, but operation and maintenance costs can be reduced by equipment reuse and development of an economical cartridge replacement system.

Summary and Conclusions

The results of this study, the example systems, and the sensitivity analysis indicate that certain processes show great promise as cost-effective effluent control candidate system elements. The magnitude of the costs and the differences for the different example systems designed for the same effluent quality illustrate the importance of cost in confined disposal facility design. This study illuminates several important concepts to be considered in future dredged material disposal operations.

A two-stage treatment scheme appears to be the minimum system to economically meet effluent quality standards. A sedimentation basin is necessary for flow equalization in dewatering systems and it is required in other systems to lower the overflow solids level to a point where a polishing element can be employed to complete the solids-liquid separation required. This analysis indicates that flocculated sedimentation, mechanized granular media filtration, granular media cartridges, and dynamic straining are viable alternatives as secondary treatment processes.

Several processes can be made more economical by use of portable equipment, for example, by installation on a barge for transport to other sites throughout a dredging season. Mechanized surface filtration costs can be substantially reduced in this manner and similar cost reductions are also foreseen for dynamic straining and granular media cartridges.

Further research is indicated for better definition of dredged material characteristics and to determine the applicability of several new concepts to dredged material disposal operations. Additional tests with dredged material should be performed to determine dewatering characteristics and to project the cake solids and solids capture likely to be encountered in slurry dewatering. Research with flocculants is needed to establish the applicability of this technique to dredged material. The Sonic Strainer and the capillary suction dewatering device are two units which may prove to be economical solids-liquid separation devices for dredged material.

Granular media filtration is an effective means of effluent polishing. Mechanized deep-bed filtration may be economical with use of less sophisticated equipment, and several, novel configurations (cartridges, filter cells, and pervious dikes) show promise. Pilot testing is indicated, however.

Although effluent quality is an overriding factor in disposal area design, solids disposal is the basic problem. In the selection of facilities to provide cost-effective water quality control, the ultimate disposal of the dredged material must be considered. This is extremely crucial in the design of the sedimentation basin, where it may be most economical based on water quality to use a small basin and extensive polishing facilities. When inland solids disposal is considered, another treatment scheme may prove to be more cost-effective.

The foregoing analysis already indicates that substantial solids reduction in dredged material disposal is possible. Furthermore it appears that more stringent effluent standards may be forthcoming and it can only be hoped that they will be of a reasonable nature. If more stringent effluent standards are promulgated they may shift the most cost-effective system to use of a flocculant for more effective sedimentation prior to effluent polishing by filters. The elements described here can be combined to meet strict guidelines, but the most stringent and zero discharge, if attainable, may be reached only at excessive cost. The unit of government responsible for such standards should consider cost-effectiveness, before proposing requirements achievable only at unreasonable costs.

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RESEARCH TO DEWATER DREDGED MATERIAL

by

T. Allan Haliburton[1]

INTRODUCTION

In March 1973, the US Army Engineer Waterways Experiment Station (WES), as directed by the Congress, initiated the Dredged Material Research Program (DMRP), a $30 million, five-year applied research effort to identify current problems associated with dredging and disposal of dredged material, and to develop environmentally sound, technically feasible, and cost-effective alternatives for disposal of dredged material, including use of such material as a resource. Task 5A of the DMRP, Disposal Operations Project Dredged Material Densification, is concerned with development and evaluation of promising techniques for dewatering and/or densifying dredged material after placement in confined (diked) disposal or containment areas. Of primary importance is fine-grained material produced from maintenance dredging activities.

With increasing environmental restrictions on open-water and unconfined land disposal and use of wetlands for construction of new confined disposal areas, increasing emphasis is being placed on extending the life of existing confined disposal areas and insuring that any new areas constructed will have maximum useful life, as recently indicated by the Congress in Section 148 of the Water Resource Development Act of 1976 (PL94-587):

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

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GOALS OF RESEARCH

The goals of Task 5A research are three-fold:

1. To increase available storage volume in confined containment areas by dewatering and densification of dredged material.

2. To reconvert dredged material into soil form, such that it may be borrowed from disposal areas (creating additional storage volume) and used for raising perimeter dikes or other off-site productive use.

3. To create stable fast land at a known final elevation with predictable geotechnical properties, for future use and development.

NEED FOR DEWATERING TECHNOLOGY

After deposition and sedimentation in a confined disposal area, fine-grained cohesive dredged material consolidates to an "equilibrium" moisture content somewhat above the liquid limit (Ref 1), with a relative consistency of "warm axle grease," as shown in Figure 1. After reaching this state, the stress-deformation relationship of the material is such that little additional settlement occurs from self-weight consolidation for thicknesses of dredged material less than about 3 m (10 ft), at least during the normal 1- to 5-year interval between disposal area fillings (Ref 2). Further, most diked containment areas, either initially or shortly after filling with cohesive dredged material slurry, resemble "bathtubs", with a perched water table in the containment area very close to or at the surface of the dredged material. Because most disposal areas are located in coastal environments where the net evaporation (total evaporation less total precipitation) is extremely small or often negative, the dredged material will not dry on its own, existing indefinitely in the "axle grease" state beneath a thin surface crust (see Figure 1) whose base denotes the lowest annual position of the perched water table in the containment area (Ref 3).

Thus, because of the particular stress-deformation properties of the dredged material at extremely low effective stress, the construction characteristics and soil properties of typical containment area dikes and

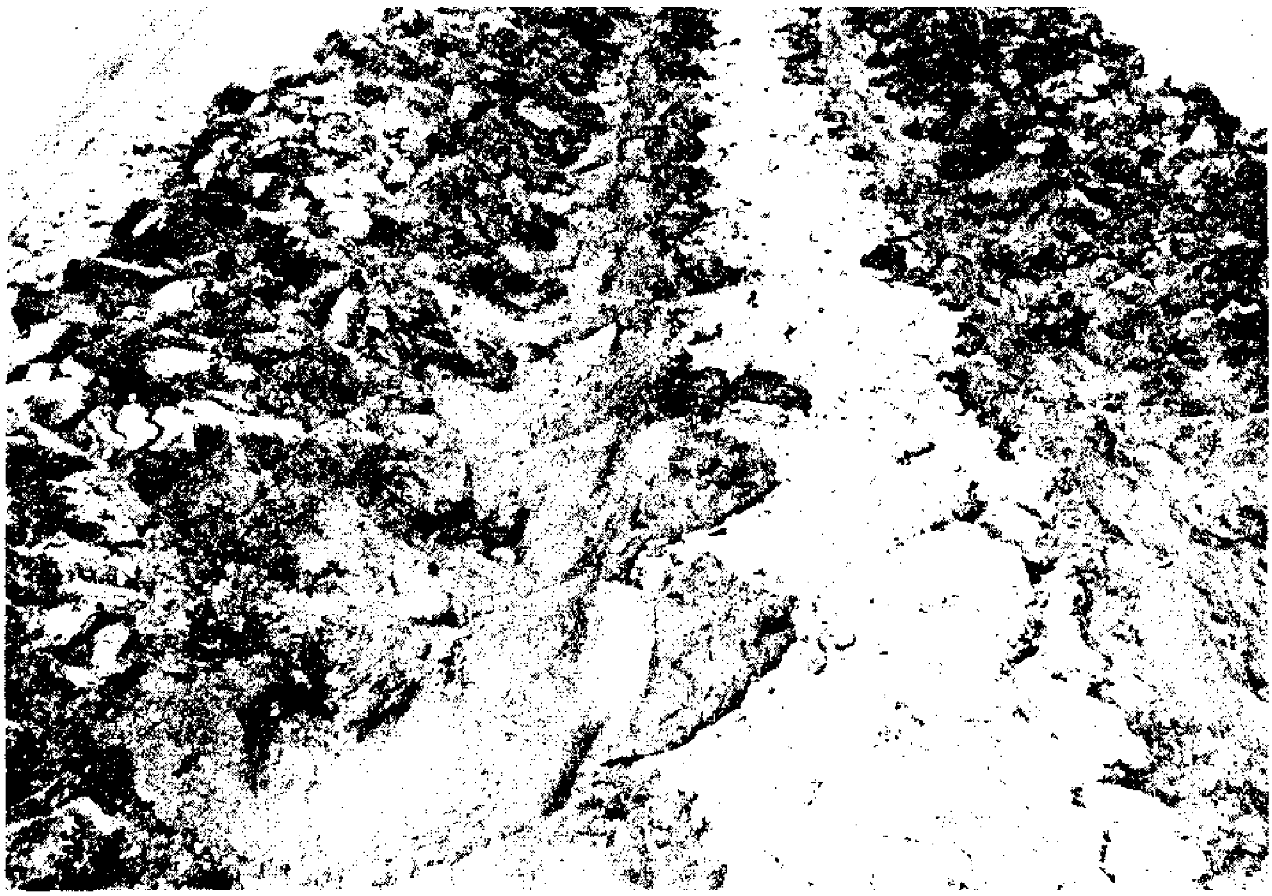

Figure 1.

Fine-Grained Dredged Material Exists in "Axel-Grease" State Above Liquid Limit Beneath Thin Surface Crust.

foundation, and the climatological conditions where most disposal sites are located, the material is, more often than not, prevented from significant natural consolidation or drying. Dewatering and densification of such material must then be accomplished with the aid of man.

RESEARCH PHILOSOPHY

In planning applied research to satisfy Task 5A goals, it was necessary to work within the primary DMRP constraint of time (i.e., formal publication of useful design guidelines 43 months after initiation of detailed research planning). A second major constraint was that of available funding, for Task 5A, while important, must compete for funding with numerous other DMRP tasks.

Initial research planning, survey of available literature, observation of existing Corps of Engineers disposal areas and dredging operations, and detailed discussions with numerous Corps of Engineers personnel concerned with day-to-day dredging and disposal activities indicated that (Refs 4, 5):

1. Application of most conventional civil engineering dewatering and densification techniques was limited by the expected low permeability of fine-grained dredged material and the inability of the dredged material surface to support surcharges, men, and equipment.

2. The large areal extent of most disposal areas, compared to that of conventional foundation dewatering projects, hindered economical use of existing dewatering procedures.

3. Engineering properties of the cohesive fine-grained soil, either in the channel as sediment or in containment areas as dredged material, were not well-defined, at least in useful geotechnical terminology; materials of interest were usually classified as either mud or muck.

4. Predictive methodology for determining disposal area volumes needed to contain dredged channel sediment consisted of relatively crude and extremely empirical "bulking factors", which often gave considerable error in under- or over-prediction.

5. A minimal number of management programs existed for operation and maintenance of disposal areas between dredging cycles, and those in existence were concerned primarily with drying dredged material around the disposal area perimeter, for future use as borrow in raising dikes.

With the limited body of available information, it appeared that a multi-subtask research effort was needed to explore several potential approaches for dredged material dewatering and densification, thus maximizing the chance of successful technology development in the available time. The multi-subtask approach adopted, and shown in Figure 2, required simultaneous research into extension of conventional dewatering techniques to consider lower permeability soils, on a larger scale and at lower cost than previously considered, investigation of innovative or new dewatering techniques, development of methods for more effectively managing containment areas, and development of analytical models to predict the time-dependent volumetric behavior of dredged material and the effects of dewatering treatment upon such volume. An additional sub-task, better definition of dredged material engineering properties, was considered but discarded on grounds that other DMRP studies were engaged in classification and characterization of dredged material and that specific properties required for comprehensive analysis would be identified during predictive model development.

Initial work was concerned with identification of specific research needs. Numerous literature and laboratory feasibility studies were funded to assess the potential of various dewatering methods for use on dredged material. Planning seminars were convened at the WES to obtain advice of knowledgeable professionals concerning which alternatives to pursue (Refs 6, 7). Topics selected for feasibility study are shown in Table 1, as are the initial studies into containment area operation and management, and predictive model development. Studies which appeared feasible were selected for field evaluation. These are shown in the Table, and will be considered as second-stage studies of containment operation, management and predictive model development.

Because of funding and time constraints, only one set of field evaluations could be carried out. Thus, best use of available funds could be made by evaluating all concepts on the same dredged material, to assess relative effectiveness. It was also decided to seek, insofar as possible, a site containing dredged material which would be difficult

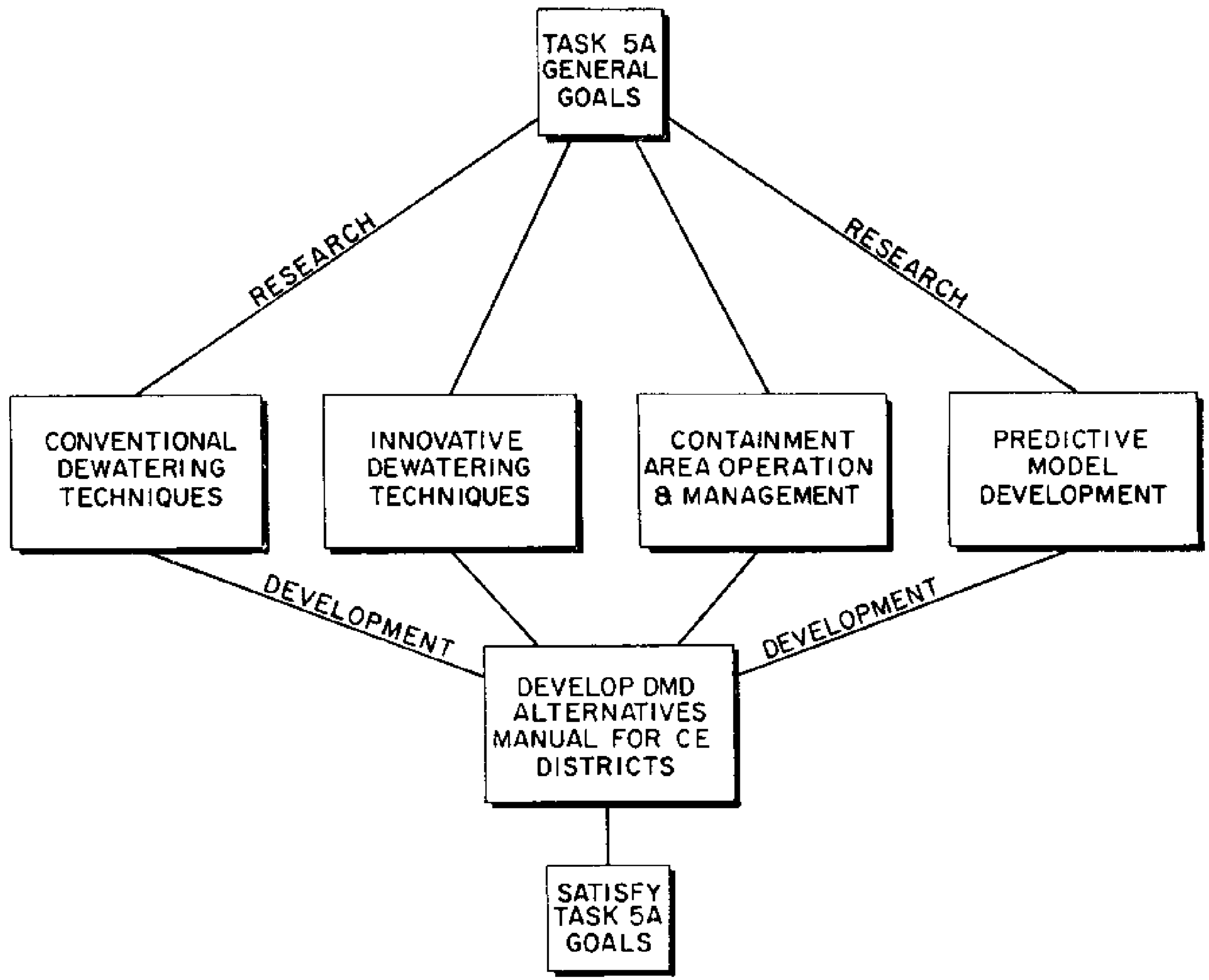

Figure 2. Multi-Subtask Research Approach Adopted for Task 5A.

TABLE 1 - SUMMARY OF TASK 5A DEWATERING RESEARCH STUDIES

RESEARCH PHASE	CONVENTIONAL DEWATERING TECHNIQUES	INNOVATIVE DEWATERING TECHNIQUES	CONTAINMENT AREA OPERATION & MANAGEMENT	PREDICTIVE MODEL DEVELOPMENT
FEASIBILITY STUDIES	State of Art Survey of Conventional Dewatering Methods	Mechanical Agitation Electro-Osmosis Aeration Freeze-Thaw Capillary Wicks Sand Slurry Injection	General Crust Management	Containment Area Sizing Model
FIELD EVALUATIONS	Gravity Underdrainage Seepage Consolidation Vacuum Underdrainage Vacuum Seepage Consol-idation Progressive Trenching	Vacuum Well Points Periodic Crust Mixing Electro-Osmosis Capillary Wicks Vegetation	Guidelines for Area Management Mobile District Design Guide-lines	Effects of Dewatering Model
IMPLEMENTATION GUIDELINES	Full-Scale Implementation of Task 5A Technology Task 5A Design Alternatives Manual			

to dewater, compared to other dredged material available nationwide, such that successful technology would be expected to work as well, or better, on other dredged material. Prediction of expected dewatering effects at other locations would be based on dredged material engineering properties, known laws of soil behavior, and results from the evaluation site.

FIELD TEST SITE DESCRIPTION

With the cooperation of the USAE District, Mobile, the Upper Polecat Bay (UPB) disposal area was selected for evaluation of promising dewatering technology. This 34.4 hectare (85 acre) disposal area, located in Mobile, Alabama, was created in 1970 by end-dumping sand from previous new work dredging to form a perimeter dike around an existing marsh at El. 1-2 MLW. The sand displaced existing, soft, cohesive foundation material down to approximately El. -16 MLW and the dike was constructed up to approximately El. 16 MLW. Disposal operations in 1970 and 1972 resulted in placement of approximately 2.4-3.0 m (8-10 ft) of sedimented fine-grained dredged material over the area. This site is shown in Figure 3, prior to initiation of DMRP dewatering studies. Much of the site was covered with up to 0.5 m (18 in.) of ponded surface water and a thin surface crust of .05-.15 m (2-6 in.) thickness existed over much of the site. Below surface crust, the dredged material existed at natural water contents of 0.9-1.5 times its liquid limit and the perched water table in the containment area was slightly above, at, or within 0.3 m (1 ft) of the dredged material surface (Ref 8).

Except for a deposit of fine to very fine, fairly clean sand around the location of the disposal pipe, characterization of the area (Ref 9) indicated that dredged material over the remainder of the site had approximately the same engineering properties, with (average values) 93% of the material passing the U. S. No. 200 sieve, 75% of the material finer than 5 micron and 41% finer than 1 micron size. Specific gravity of solids averaged about 2.72. The liquid limit of the dredged material ranged from 73-171 (avg. about 100), with the plastic limit varying from 27-56 (avg. about 35). Most of 102 samples tested had about 5% organic material (dry weight basis) but Atterberg limits plotted slightly above and parallel to the "A-line," thus classifying the material as CH by the USCS. Results of X-ray

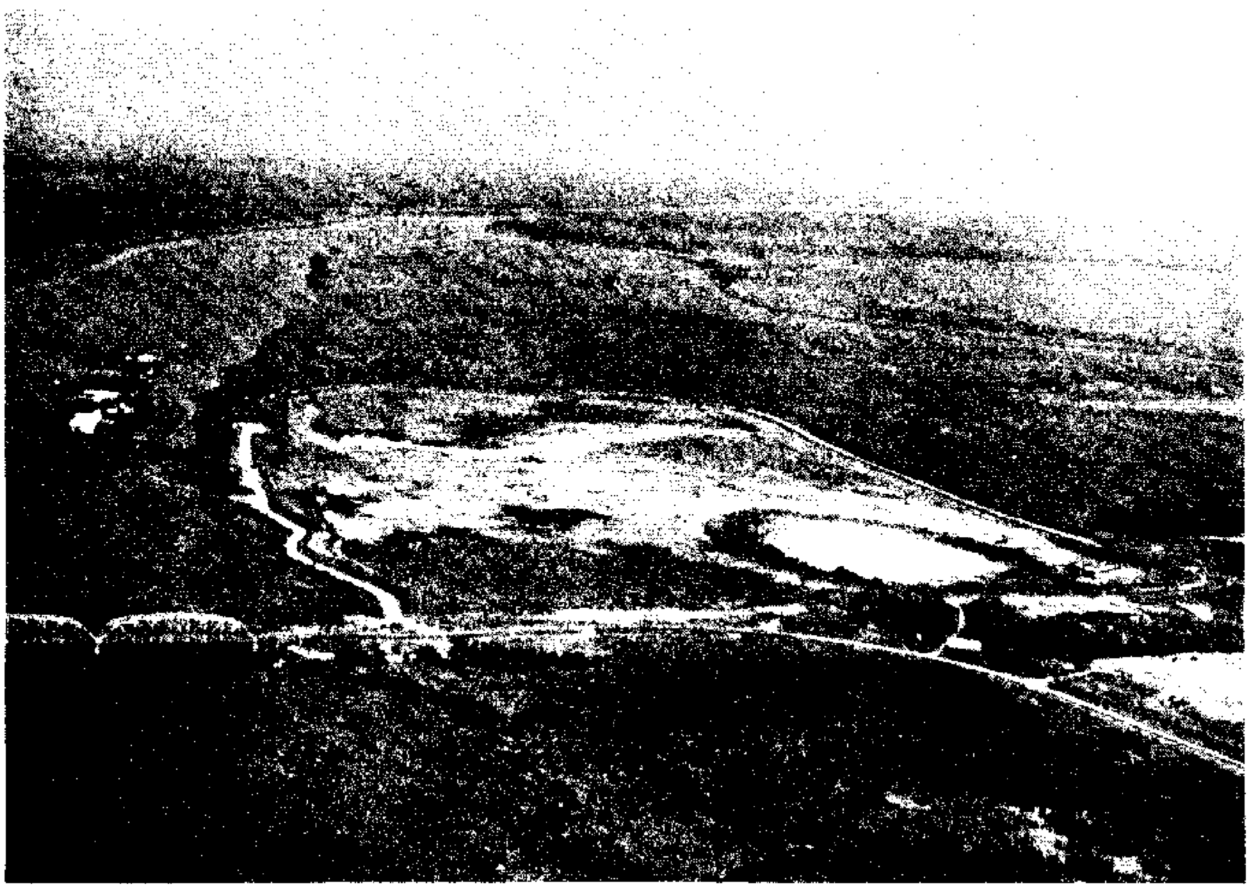

Figure 3.

Upper Polecat Bay Disposal Area, Mobile, AL, Prior to Initiation of DMRP Dewatering Research, Seen from the South.

diffraction analyses indicated the majority of the clay fraction present to be montmorillonite; the remainder consisting mostly of chlorite, with some clay-mica and a trace of kaolinite. Field tests indicated a coefficient of permeability on the order of 1-10 x 10^{-5} cm/sec, considerably higher than expected for a "clay" soil, but attributable perhaps to the high void ratio of the sedimented dredged material. Consolidation tests run on the material confirmed its high compressibility, with a C_c of approximately 0.6 - 1.3 (avg. about 1.1).

FIELD EVALUATION PROJECTS

As shown in Table 1, 10 field evaluation projects were undertaken at the UPB Disposal Area. Locations of the various projects are shown in Figure 4, and they may be broadly divided into four categories:

1. Assisting and promoting the effects of natural drying and desiccation.
2. Promoting internal gravity drainage and assisting self-weight consolidation.
3. Promoting consolidation with negative pore pressures.
4. Electro-osmotic dewatering.

Promoting Natural Drying and Desiccation

Three projects were designed to make better use of evaporative forces to dry fine-grained dredged material. The first, Progressive Trenching, was concerned with improving surface drainage, such that precipitation was rapidly removed from the disposal area, allowing evaporative forces to dry the dredged material into surface crust and gradually lower the perched water table. This study has been underway since August 1975, and is carried out over the majority of the UPB disposal area, as it is essentially a full-scale field demonstration. It is necessary to trench and retrench the site progressively as, at any given time, it is difficult to maintain a stable trench more than about 0.3 m (1 ft) below existing crust. The trenches must always be lower than the surface crust, as precipitation runs through crust desiccation cracks to the trenches, then drains off the site

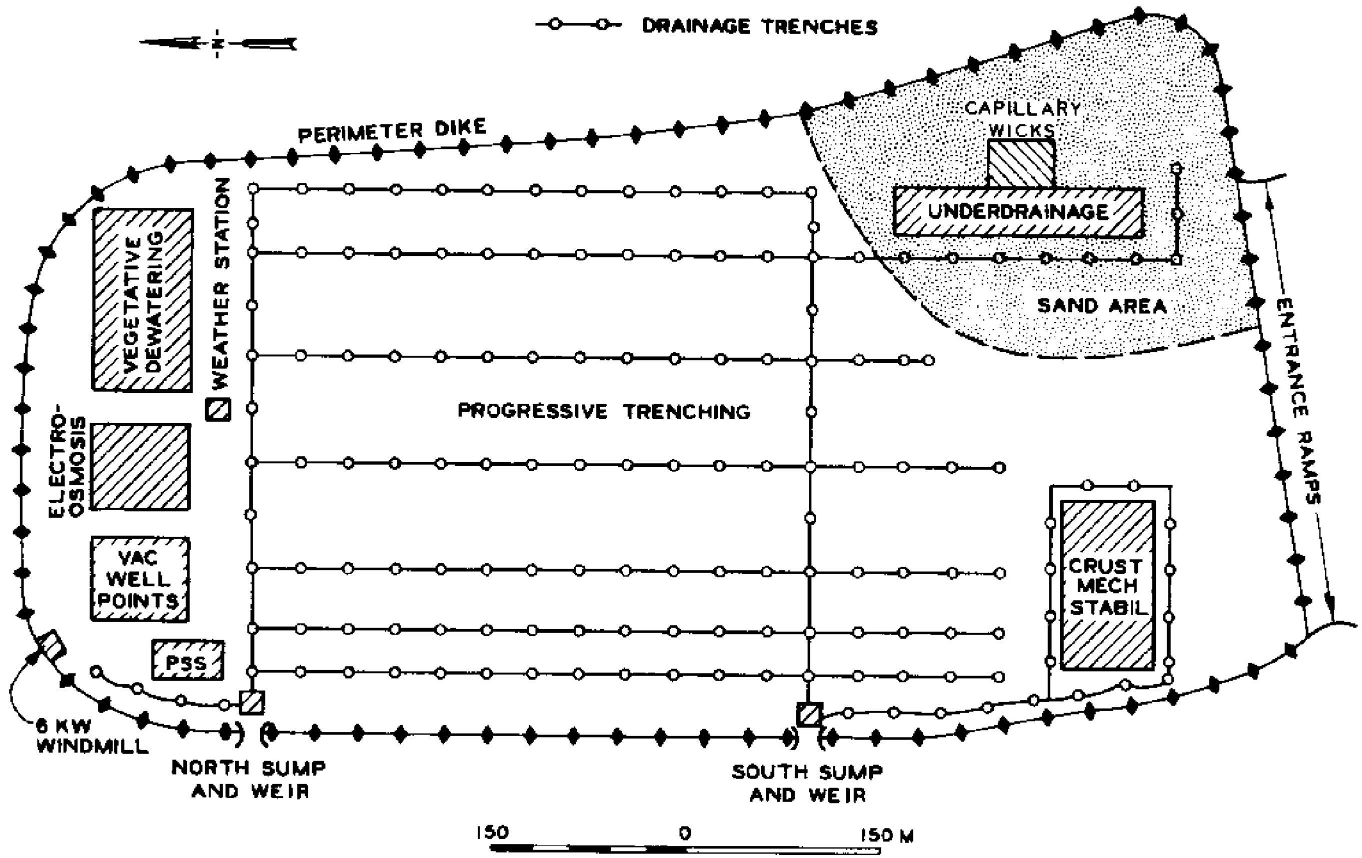

Figure 4

Dredged Material Dewatering Field Demonstrations In Upper Polecat Bay Disposal Area, Mobile, AL.

through weirs. Any benefit from subsurface seepage into the trenches is considered extra; the primary purpose of the trenches is to rapidly remove precipitation. The extent of trenching carried out should be based on existing surface topography. A more ideal pattern than the parallel trenches shown in Figure 4 would be to make radial or "finger" trenches out from each weir into the disposal area.

While initial DMRP studies considered surface desiccation crust as the factor responsible for preventing dredged material drying (Ref 5), subsequent research (Ref 3) identified the perched water table and low net evaporation as causal factors, and determined that, because of the large volumetric shrinkage (50%) from desiccation and resulting exposed shrinkage-cracked surface area, the evaporation rate remains high as the surface dries into a crust and causes the perched water table to fall. Estimates can be made of the annual water loss caused by surface drying and progressive trenching, using data on precipitation and evaporation at a particular disposal site location, plus evaporation and shrinkage properties of the available dredged material.

Primary operational problems are those associated with development of the first 0.3-0.5 m (12-18 in.) of crust, as this thickness and resulting support capacity is necessary to allow trenching by amphibious dragline or other low ground-pressure equipment. The Riverine Utility Craft (RUC), shown in Figure 5, which produces trenches by its method of locomotion, has been found to be the only equipment which can successfully work in disposal areas when less than 0.3 m (12 in.) of crust exist. After initial crust development by the RUC, amphibious draglines may be used to deepen the RUC trenches (Figure 6) and, with thickening crust, conventional draglines (Figure 7) may be used. After 1 yr of trenching by RUC, amphibious dragline, and conventional dragline, trenches in the UPB disposal area were excavated down to original foundation level, with resulting formation of approximately 1.2-1.5 m (4-5 ft) of surface crust, as shown in Figure 8.

An alternate method of progressive trenching is by use of a small dredge, as shown in Figure 9. Such a dredge will float in about 0.6 m (2 ft) of water and can pump the excavated dredged material away from the

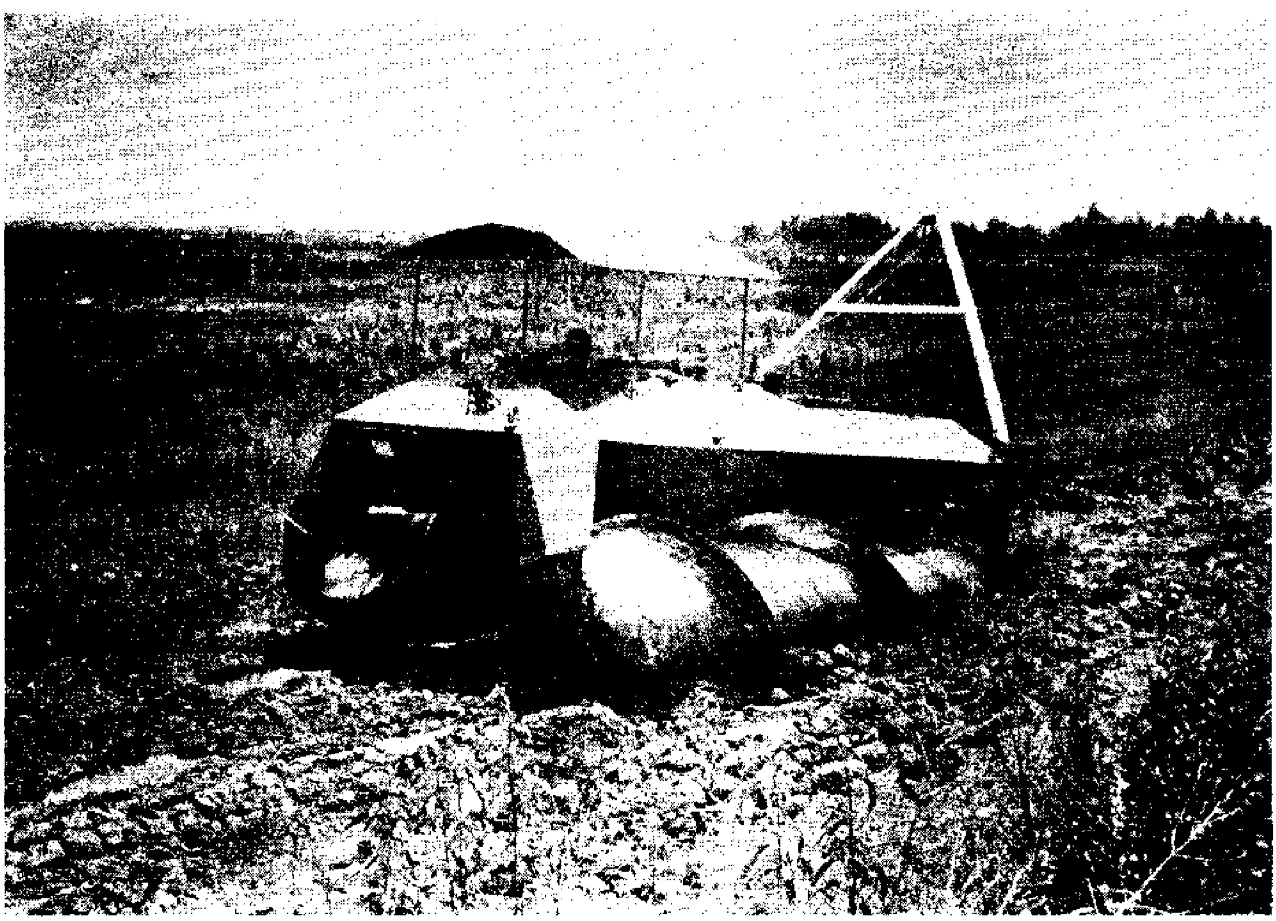

Figure 5.

Riverine Utility Craft (RUC)

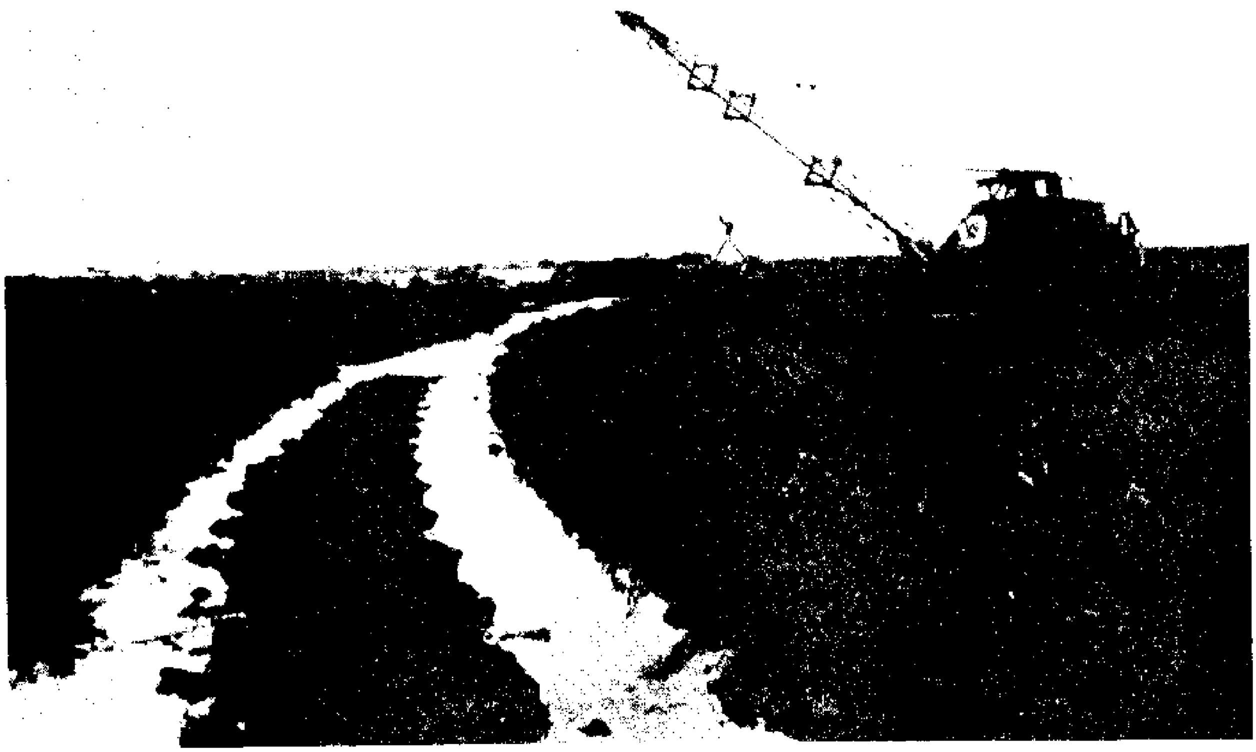

Figure 6.

Amphibious Dragline-Deepening Drainage Trenches Established by the [illegible]

Figure 7.

Conventional Dragline on Mats-Deepening Drainage Trenches Made by Amphibious Dragline

Figure 8.

Drainage Trenches Constructed Progressively Over 14 Months in Upper Polecat Bay Disposal Area.

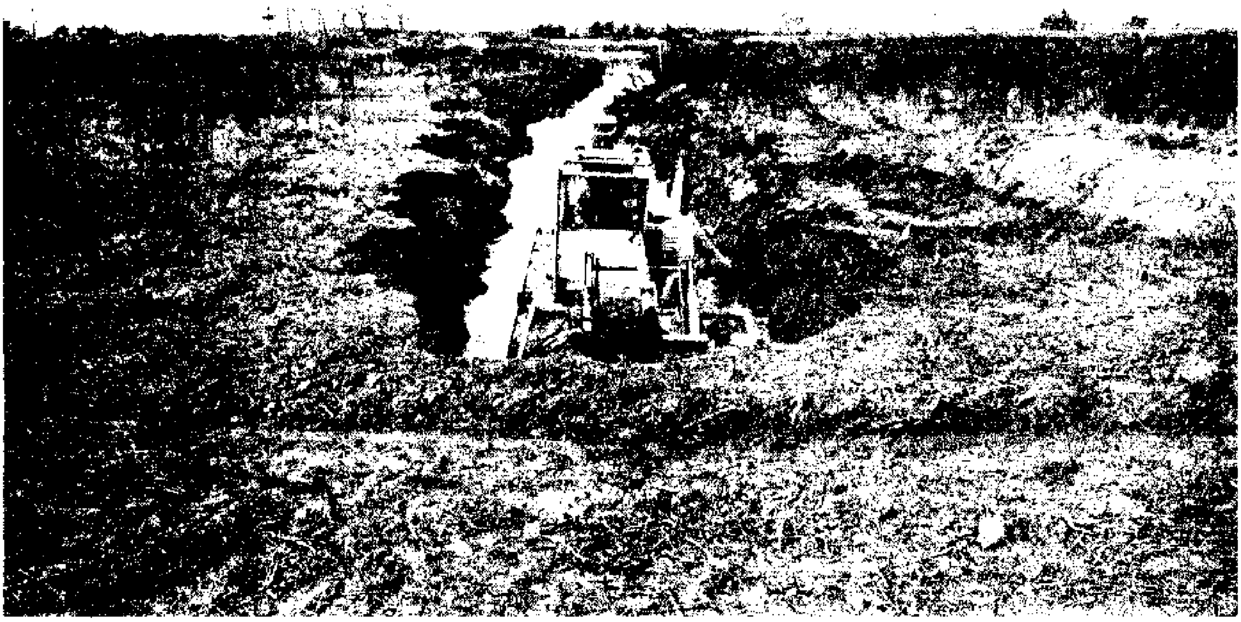

Figure 9.

Progressive Trenching by Small (Mudcat) Dredge in Upper Polecat Bay Disposal Area edges of the trench, spreading it thinly over the disposal area surface where it will dry rapidly. Keeping the trenches filled with water to allow dredge mobility also inhibits initial bank caving. Once trenches are cut or deepened, the dredge is removed and the water level gradually drawn down.

Another method of drying investigated was periodic mixing of dried surface crust with underlying slurry, similar to repeated agricultural tillage to dry wet fields. During the period February-September 1976, a one-acre test plot in the southwest corner of the UPB disposal area was plowed monthly, using a RUC, as shown in Figure 10, and the behavior of this plowed area compared with an adjacent control area of similar size. Periodic mixing caused a greater (15%) rate of volume gain from desiccation than in the control area, but at a loss of surface support capacity.

The third technique evaluated to enhance natural drying was use of capillary wicks. The wicks, of paper, wood fibers, and/or plastic, are placed in the dredged material and use capillary suction to draw water from the dredged material mass to the surface, where it is evaporated. Laboratory studies indicated that wicks also accelerate development of desiccation cracks. In addition, they are inexpensive and require no maintenance after placement. To conduct a controlled field demonstration, test pits were prepared and 1.5 m (5 ft) of dredged material slurry was pumped into test pits and sedimented prior to wick placement.

Other Field Evaluation Projects

Other field evaluation projects at the UPB site included gravity underdrainage and seepage consolidation, and vacuum-assisted underdrainage and seepage consolidation, all evaluated in 12.2 m (40 ft) square by 1.8 m (6 ft) deep test pits filled with dredged material after underdrain placement. After the fact vacuum consolidation and dewatering was attempted by use of well points, placed both in conventional sand pack and in thin horizontal sand drainage layers made by hydraulically fracturing the dredged material with sand grout. Well points were placed in 2.5 m (8 ft) of dredged material. All these techniques were intended to take advantage of the high compressibility and higher than expected permeability of the dredged material. The minimal bearing capacity of dredged material usually

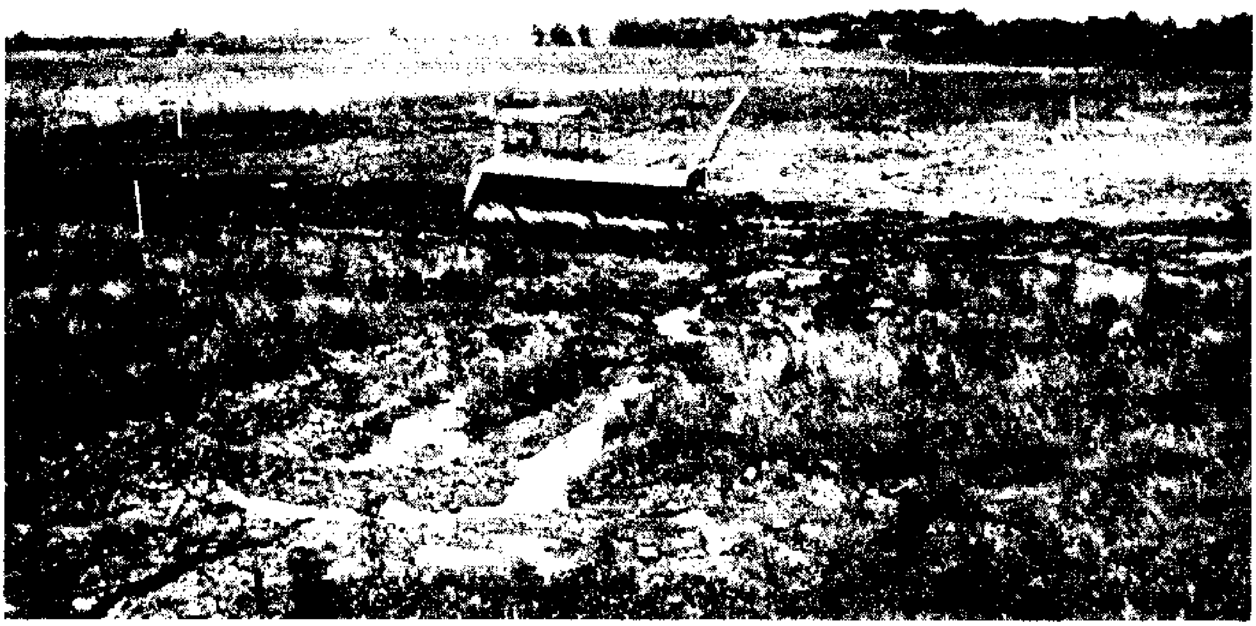

Figure 10.

RUC Plowing at Periodic Crust Mixing Experiment Test Site.

inhibits application of conventional earth surcharge to develop excess pore pressures, as does the relatively large size (>50 hectares) of most disposal areas and the unavailability of suitable surcharge borrow. Application of vacuum to produce negative pore pressure appeared a more logical way to induce consolidation (Ref 1).

Low voltage gradient (0.01-0.05 v/cm) electro-osmotic dewatering was also evaluated, as this method, using vertical anodes and cathodes, is not limited with respect to depth of effective treatment. Preliminary laboratory studies (Ref 10) indicated 30-45 kwh was required to create 0.76 m^3 (1 cu yd) of space, by removing 0.76 m^3 of water.

More detailed information on all of the field evaluation projects is available elsewhere (Ref 8).

ASSESSMENT OF RESEARCH RESULTS

Although data reduction and evaluation are still underway for many of the projects, enough data are available to allow generalized statements with respect to techniques and methodology for optimizing rates of dredged material dewatering. Relative success of the DMRP-sponsored field experiments at UPB may be seen in Figure 11, taken approximately 13 months after initiation of the first dewatering research projects. In locations where maximum crust development was attempted, 1.2-1.5 m (4-5 ft) of dried dredged material exists and the dredged material surface will now support conventional construction equipment without necessity for mats. Current plans call for removal of dried dredged material from the southwest portion of the UPB disposal area during 1977, for use as borrow in raising perimeter dikes to further increase UPB disposal area capacity. The cost of obtaining this borrow is estimated at \$1.30/m^3 (\$1.00 per cu yd) transported and dumped on the dike, approximately 1/2 to 1/3 the cost of off-site borrow, and without considering the cost benefit of additional disposal volume provided when it is removed.

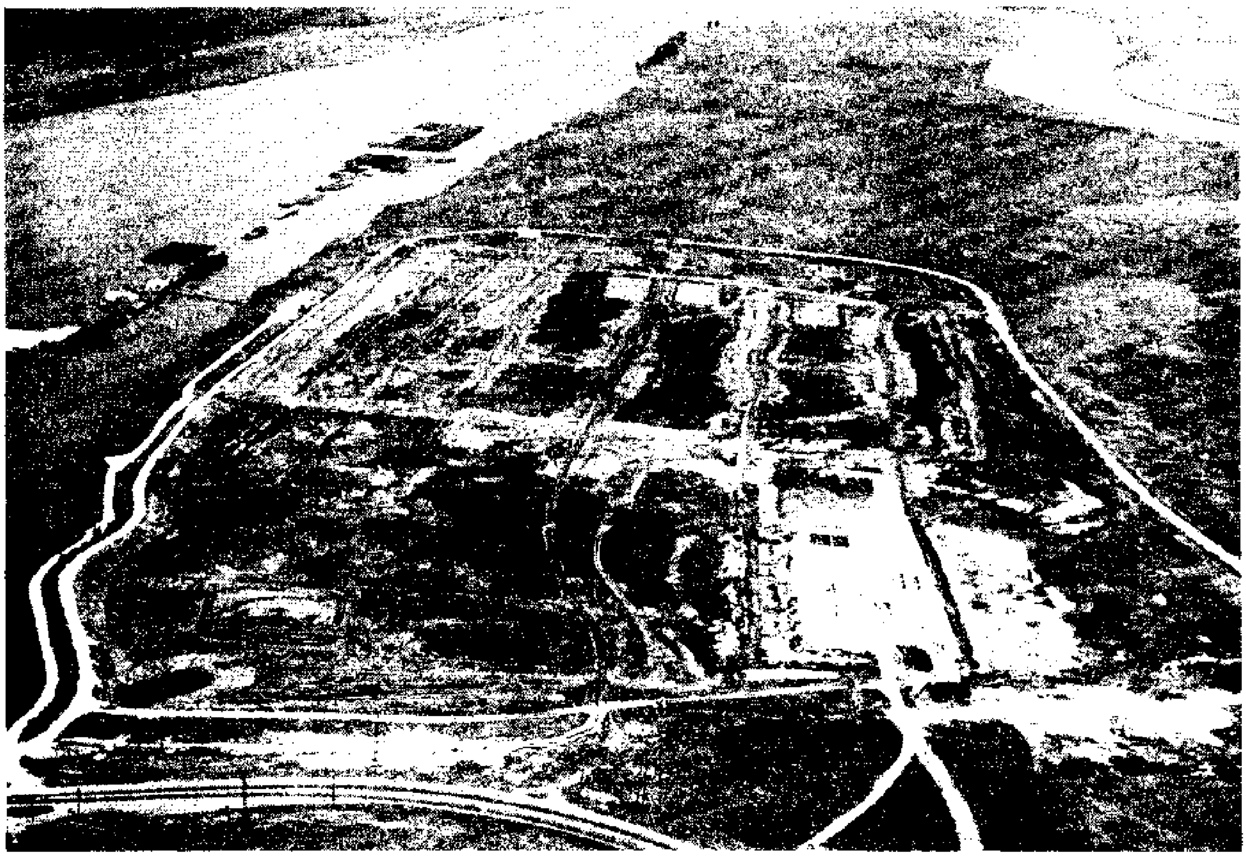

Figure 11

Upper Polecat Bay Disposal Area in September 1976, 13 Months After Start of DMRP Dewatering Research. Seen from the South.

SELECTION OF DEWATERING ALTERNATIVES

Based on preliminary field results, all four general categories of dredged material dewatering appear to be technically feasible. Thus, their use in specific situations must be justified on a cost-effective basis compared to other alternatives for dredged material disposal and containment. The current best estimates of the author concerning dewatering rate and cost obtainable by proper application of the four dewatering methodologies are shown in Table 2. As might be expected, both cost and rate of dewatering increase in proportion to the amount of man's involvement in controllingand directing the forces of nature, and in dewatering in spite of the forces of nature.

However, dewatering technology is not a panacea for magically obtaining soil from dredged material slurry. While effective technology has been developed, successful application of such technology, as in other areas of geotechnical engineering, must be carried out on a site-specific basis. The volume of sediment to be dredged over the selected design period must be carefully estimated and combined with appropriate analytical prediction methods to determine the containment area volume required for design period disposal, and the additional volume which may be gained during the design period by proper management techniques, including dewatering. A comprehensive well-engineered plan must then be formulated, considering available funding, local and national constraints and policy, available disposal areas, annual dredging volumes, available personnel and equipment, and other constraints. Once such a plan is developed, it must be implemented, results monitored, and, with appropriate feedback, continually updated so that adequate confined disposal volume will always be available.

SUMMARY

DMRP research in dredged material densification, encompassing some 21 projects and expenditure of approximately $1.9 million, has resulted in development of several alternatives to physically densify/dewater fine-grained dredged material and return it to soil form. Further, methods have been developed for operation and management of confined disposal areas to maxi-

COMPARISON OF FOUR GENERAL
DREDGED MATERIAL DEWATERING METHODS

GENERAL METHOD	MAXIMUM RATE (m-tkns/yr)	ESTIMATED COST ($\$/m^3$-space)	TECHNICAL PROBLEMS
DESICCATION	1.5	.65 - 1.30	OBTAINING FIRST 18 IN. OF CRUST
GRAVITY DRAINAGE	2.1 - 3.1	2.00 - 3.25	INITIAL CLOGGING
VACUUM CONSOLIDATION	2.4 - 3.7	2.50 - 4.00	PUMP RELIABILITY
ELECTRO-OSMOSIS	6+	3.25 - 4.50	CONTINUOUS MONITORING REQUIRED

Table 2. Comparison of Dredged Material Dewatering Methods Applied to Cohesive Fine-Grained Material

mize the rate of dredged material dewatering, and analytical techniques have been developed to predict the volumetric state of dredged material after disposal, and during and after dewatering. While choice and application of particular concepts and methods must be site-specific, nevertheless, a basis for geotechnical engineering practice in dredged material densification now exists.

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PRIMARY CONSOLIDATION AND COMPRESSIBILITY OF DREDGINGS

by

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Amr S. Azzouz[3]

INTRODUCTION

Since maintenance dredgings that are deposited in diked containment areas usually have a high water content and contain a substantial amount of fines, the usefulness and economy of the resulting landfill is strongly dependent on the time-consuming process of consolidation. In order to provide some guidance for engineers concerned with the planning and operation of such landfills, the consolidation and compressibility characteristics of various hydraulically placed dredgings are evaluated herein by conducting an extensive series of laboratory consolidation tests on "undisturbed" samples taken by piston sampler from three different landfill sites. Classical consolidation theory was used to analyze the results, and linear regression analyses were employed to establish useful, simple, and convenient relationships between the consolidation and compressibility characteristics, and the index properties of the dredgings.

MATERIALS TESTED

Sixty-four "undisturbed" samples of dredgings from three different landfills in the vicinity of Toledo, Ohio (Figure 1) were tested; sixteen of these samples were obtained from Riverside Site during the summer of 1972, and two sets of twenty-four samples each were taken during the summers of 1973 and 1974 from Riverside Site, the Island Site, and Penn 8. A three-inch diameter, light-weight, manually operated piston sampler (Hummel and Krizek, 1974) was used to obtain all samples

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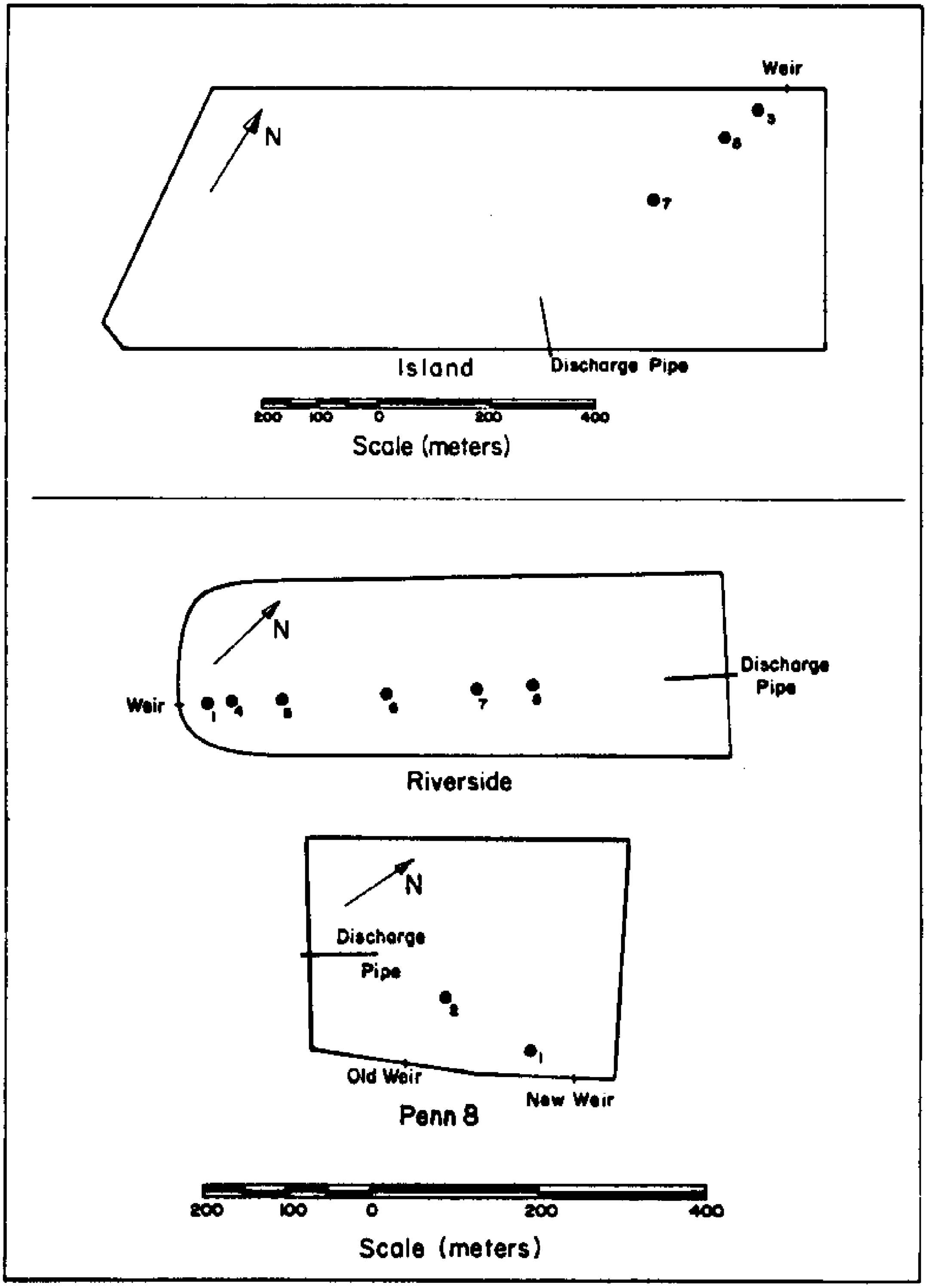

Figure 1. Test Locations at Toledo Disposal Areas

from boreholes located along the line between the discharge pipe and the overflow weir of each site; prior to testing, the samples were stored in a humid room while they were still inside their sampling tubes with both ends capped, wrapped with tape, and waxed. Each sample was identified by the letter E (which designates the consistency of the material), a chronological number, and the letters TO, which designate the first letters of the city and state (Toledo, Ohio) from which the sample was obtained.

EXPERIMENTAL PROCEDURE

Conventional consolidation tests (Lambe, 1951) were conducted on 3-inch (7.6-cm) diameter tube samples that were extruded and trimmed to fit 2.5-inch (6.65-cm) diameter, teflon-lined, brass consolidometer rings with a cross-sectional area of 31.6 cm^2 and a height of 1.0 inch (2.5 cm). The test specimens were trimmed about 0.15 cm shorter than the height of the ring to ascertain that the loading cap fit properly into the ring, and filter paper was placed on each side of the specimen to prevent the loss of fines near the drainage faces. All specimens were submerged and loaded with a load-increment ratio, $\Delta p/p$, of unity under conditions of double drainage. For the first sixteen specimens the initial load was 31 kN/m^2 and the maximum load was 248 kN/m^2; eight specimens were rebounded incrementally to the initial load, at which point the test was terminated, while the load of 248 kN/m^2 was maintained constant for over 7 months on the other eight specimens to investigate their secondary compression characteristics (Salem and Krizek, 1974). The load for the rest of the specimens started with 31 kN/m^2, increased to 496 kN/m^2, and was rebounded incrementally to 31 kN/m^2.

ANALYSIS OF RESULTS

In order to establish approximate, but practical, relationships between the consolidation characteristics of these dredged materials and certain index properties, regression analyses with associated confidence limits (Crow, Davis, and Maxfield, 1960; Draper and Smith, 1966) were employed. Among the various multiple linear regression methods of analysis, the step-wise procedure seems to offer the most practical compromise between completeness and tractability; therefore, it was used in the analyses presented herein.

Void Ratio versus Consolidation Pressure

Each load increment was sustained for about 24 hours, and the deformation-time response was monitored continuously to ascertain the existence of sufficient data to back-calculate the time for completion of primary consolidation, t_{100}. A typical series of deformation-time response curves is illustrated in Figure 2. The selection of one particular point (usually the 24-hour reading) from each deformation-time curve for a given applied consolidation pressure, p, allows a deformation-load curve to be plotted. The total settlement, S, in a stratum of thickness, H, due to a given load, p, is usually determined from the relationship

$$S = H \frac{C_c}{1 + e_o} \log \frac{p_c + p}{p_c} \quad , \qquad (1)$$

where C_c is the compression index, e_o is the initial void ratio, and p_c is the preconsolidation pressure (Casagrande, 1936). The preconsolidation pressures for all specimens tested in this experimental program varied between 20 and 90 kN/m^2; in most cases these values are higher than the estimated overburden pressures, and these deviations can be explained in whole or in part by (a) disturbances associated with sampling and testing and (b) desiccation that occurs during

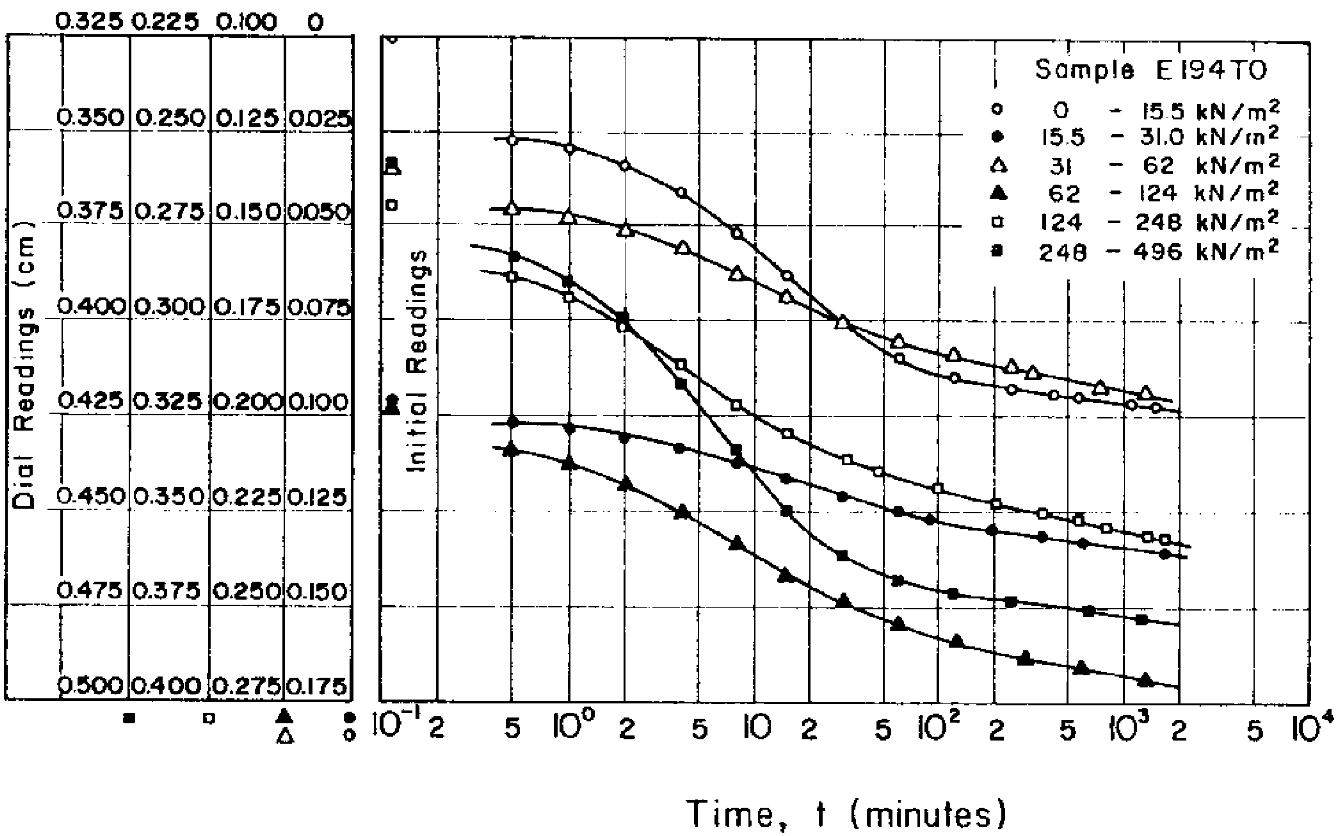

Figure 2. Typical Deformation - Time Response for Different Load Increments

seasonal dredging cycles. The so-called virgin compression curve began at pressures between 30 kN/m^2 and 130 kN/m^2 for all specimens, and it retained its character for all pressures within the scope of this program.

Compression Index

Various empirical correlations have been advanced to enable C_c for specific classes of soils to be estimated from a knowledge of one or more soil index properties. For example, Terzaghi and Peck (1948) suggested that

$$C_c = 0.009 \; (w_L - 10) \quad , \tag{2}$$

where w_L is the liquid limit expressed as a percentage. Van Zelst (1948) found a straight-line relationship between the initial void ratio and compression index of a lacustrine clay. Regression analyses by Cozzolino (1961) indicated that the compression indices of two Brazilian clays were related to their liquid limits and natural void ratios by means of the following empirical relations:

$$C_c = 0.0046 \; (w_L - 9) \pm 0.086 \tag{3a}$$

$$C_c = 0.256 + 0.43 \; (e - 0.84) \pm 0.063 \tag{3b}$$

for Motley clays from Sao Paulo City and

$$C_c = 0.0186 \; (w_L - 30) \pm 0.41 \tag{4a}$$

$$C_c = 1.21 + 1.055 \; (e - 1.87) \pm 0.34 \tag{4b}$$

for soft silty clays from the lowlands of Santos. The compression indices for these dredging samples varied between about 0.3 and 0.7, which lies in the lower portion of the 0.4 to 1.4 range reported by the Corps of Engineers (1969).

When C_c is plotted in Figures 3a and 3b versus the natural water content, w_n, and the liquid limit, w_L, respectively, the observed trend in both cases justifies the use of a first order linear model, and the associated equations are

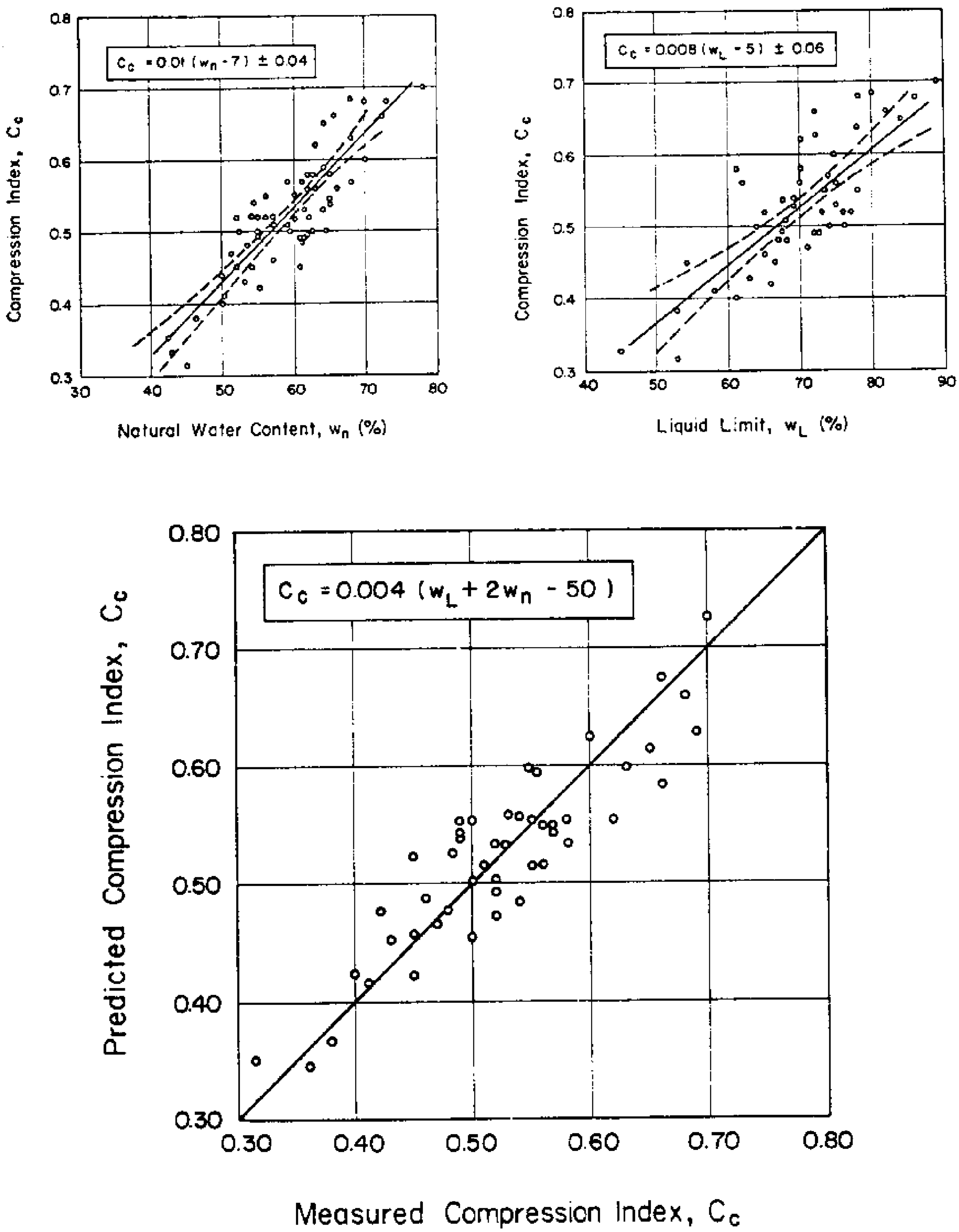

Figure 3. Compression Index as Functions of Natural Water Content and Liquid Limit

and

$$C_c = 0.01\ (w_n - 7) \pm 0.04 \tag{5}$$

$$C_c = 0.008\ (w_L - 5) \pm 0.06 \tag{6}$$

where w_n and w_L are expressed as percentages; for the work reported herein, the respective ranges of validity of Eqs. (5) and (6) are approximately $40 < wn < 80$ and $45 < w_L < 90$. As explained by Benjamin and Cornell (1970), the dimensionless correlation coefficient measures the goodness of fit and equals unity for perfect correlation; the plots shown in Figures 3a and 3b represent correlation coefficients of 0.89 and 0.80 for Eqs. (5) and (6), respectively. The dashed curves in each figure are the loci of the 90% confidence limits for the true mean value of C_c for any given value of w_n or w_L, (Crow, Davis, and Maxfield, 1960); as an example, for w_n equal to 60%, one can say with 90% confidence that the corresponding true mean value of C_c, lies between 0.52 and 0.54. It is clear that the best predictions can be made in the middle of the observed range of the independent variable. The simultaneous effect of both w_n and w_L on C_c may be expressed as

$$C_c = 0.004\ (w_L + 2w_n - 50) \quad , \tag{7}$$

for which the computed multiple correlation coefficient is 0.88. Figure 3c shows the comparison between measured values of C_c and those predicted by use of Eq. (7).

Constrained Modulus

The constrained modulus provides an alternate procedure for characterizing the compressibility of soils (Schultze and Kotzias, 1961; Janbu, 1963; Stamatopoulos and Kotzias, 1973; Salazar, 1973), and many reasons are given to suggest its preferred use; of these, the fact that the constrained modulus is more or less analogous to our conventional concept of modulus is perhaps the strongest argument in its favor. Based on a statistical analysis of many soils, Schultze and Kotzias (1961) found that the constrained modulus, M, could be related to the axial stress (vertical stress), p, by

$$M = B\ p^A \quad , \tag{8}$$

where A and B are empirical coefficients that depend on soil type; in general, A was found to equal unity for normally consolidated clays, and B was determined to be quite dependent on the porosity, the natural water content, and the grain size (particularly for the clay-size fraction). A few years later, Janbu (1963) proposed the relationship

$$M = mp_a \left[\frac{p}{p_a} \right]^{1-a} , \tag{9}$$

in which both the modulus number, m, and the exponent, a, are primarily functions of porosity, and p_a designates atmospheric pressure, which is introduced to normalize the axial stress, p. The coefficients m and a of Eq. (9) can be found by use of any two points on a given stress-strain curve. Janbu (1963) calculated the values of m and a for materials ranging from sound "elastic" rock to normally consolidated clays, and he found that a varied between one and zero, respectively, whereas m ranged from 10^5 to 10^6 for sound rock to as low as 2 to 30 for clays. Krizek, Parmelee, Kay, and Elnaggar (1971) studied the results presented by Osterberg (1952) and suggested that the constrained modulus of compacted soil may be a unique function of the dry density and the overburden pressure. Regression analyses by Salazar (1973) indicated that a modified Janbu expression yielded the highest multiple correlation coefficient for the constrained tangent modulus of compacted soils.

In order to develop compressibility relationships in terms of the constrained modulus, the e-log p data were replotted in terms of axial stress, p, versus axial strain, ε_a. The axial strain corresponding to a particular axial stress, p, was determined by taking the ratio of the change in specimen height, ΔH, associated with a stress increment, Δp, and the initial specimen height, H, at the time of application of this load increment. Most of these plots can be reasonably well described by a parabola with its axis parallel to that of the axial stress. The constrained tangent modulus, defined as $M = \frac{dp}{d\varepsilon_a}$, was evaluated

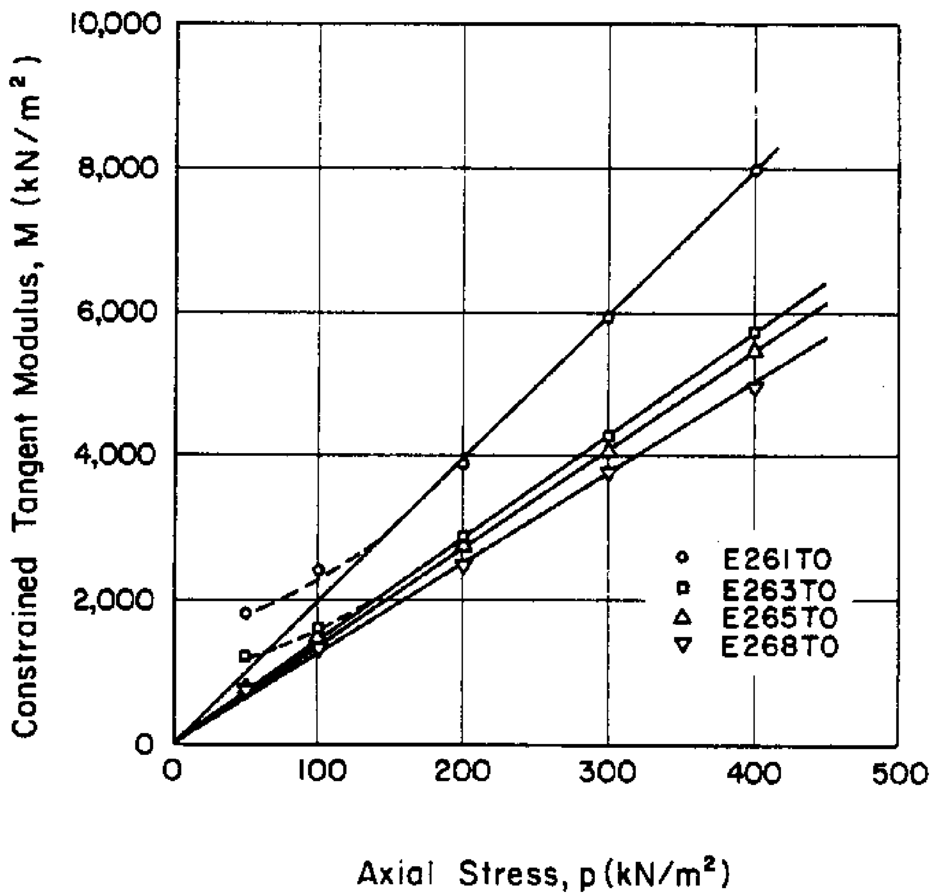

Figure 4. Tangent Constrained Modulus versus Axial Stress for Four Typical Tests

at various arbitrary stress levels, and some typical results are plotted in Figure 4, from which it is clear that M varies with p in a simple linear manner with a constant of proportionality, m, known as the modulus number; in mathematical form we may write

$$M = m\ p \tag{10}$$

which verifies that the parameter **a** of Eq. (9) equals zero, as suggested by Janbu (1963) for normally consolidated clays. The occasional small deviations from linearity generally occur at very low values of stress. In most cases these samples probably have been slightly preconsolidated by the desiccation associated with the intermittent seasonal drying between dredging seasons.

For all samples tested the modulus number, m, in Eq. (10) was found to lie between 8 and 20, which is within the range of 2 to 30 reported by Janbu (1963) for clays. In order to establish a quick method for estimating the expected settlement under a certain stress, m was plotted in Figures 5a, 5b, and 5c versus the dry density, γ_d, percent clay, and natural water content, w_n, respectively, and the following equations with correlation coefficients of 0.47, 0.42, and 0.67 were obtained:

$$m = 11\ \frac{\gamma d}{\gamma_w} + 1.27 \tag{11a}$$

$$m = 17 - 0.12\ (\%\ \text{Clay}) \tag{11b}$$

$$m = 24 - 0.2\ w_n \tag{11c}$$

A more comprehensive relationship involving all three independent parameters was found to be

$$m = 1.04\ \frac{\gamma d}{\gamma_w} - 0.17\ w_n - 0.07\ \ (\%\ \text{Clay}) + 24 \quad , \tag{12}$$

where w_n and (% Clay) are expressed as percentages, and γ_w was introduced to retain the dimensionless character of the regression equation; the computed

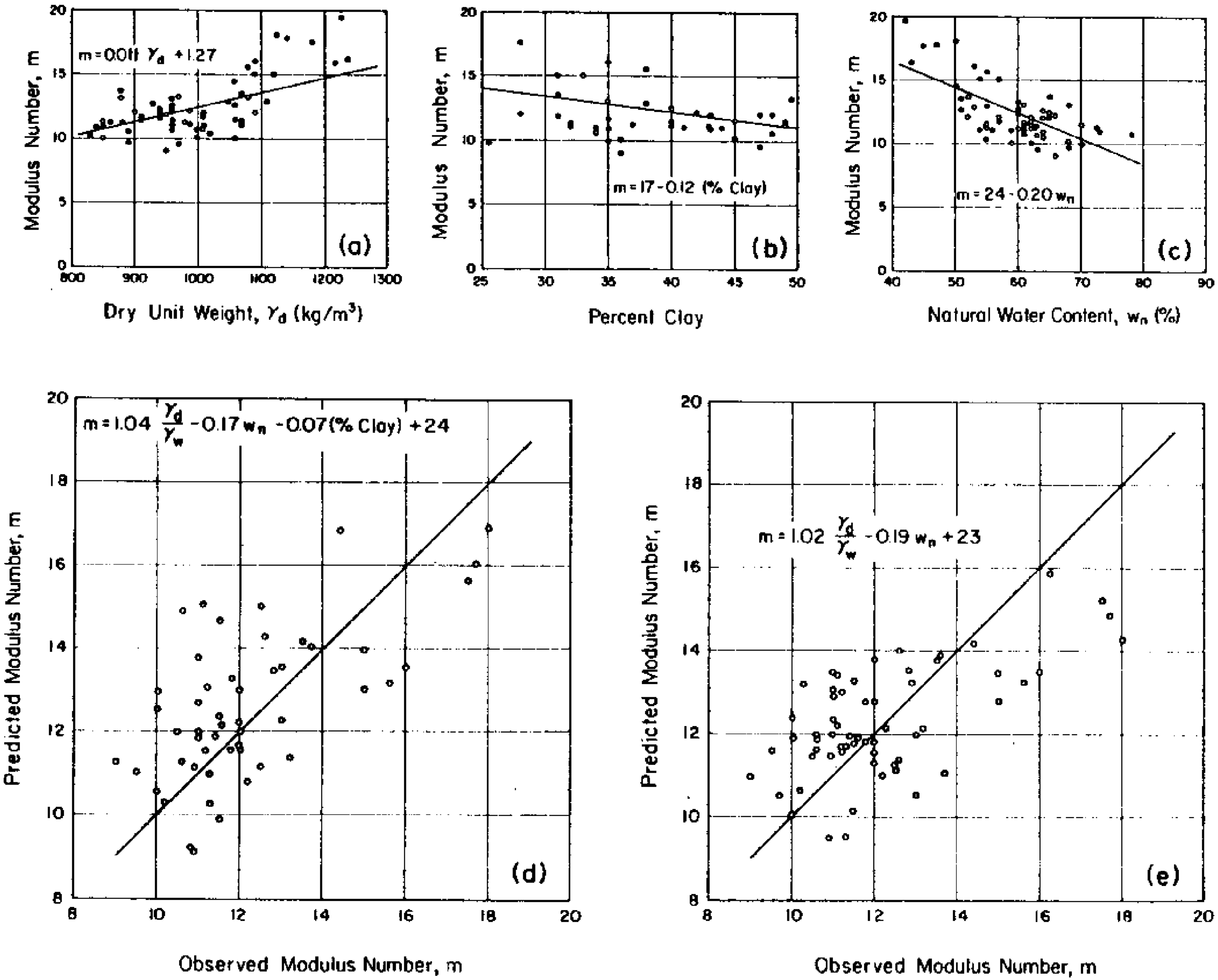

Figure 5. Modulus Number as Functions of Dry Unit Weight, Percent Clay, and Natural Water Content

correlation coefficient is 0.71, and a comparison of the observed and predicted values of m is given in Figure 5d. Based on a step-wise regression analysis (Draper and Smith, 1966) with the independent parameters in Eq. (12), it was found that m correlated most strongly with γ_d and negligibly with (% Clay); this fact, together with the time-consuming nature of the particle size analysis required to determine the clay fraction, suggests the use of the following simplified relationship involving only γ_d and w_n:

$$m = 1.02 \frac{\gamma_d}{\gamma_w} - 0.19\, w_n + 23 \tag{13}$$

As can be seen from Figure 5e and the calculated correlation coefficient of 0.67, the predictive ability of Eq. (13) is not quite as good as that of Eq. (12).

Coefficient of Consolidation

The coefficient of consolidation, c_v, corresponding to each load increment can be calculated from

$$c_v = \frac{Th^2}{4t} \tag{14}$$

in which t is time, h/2 is the length of the shortest drainage path at the time of application of each load increment, and T is the time factor, which is a function of the degree of consolidation, U. In the present study the logarithm of time method was used to determine the time, t_{50}, corresponding to a degree of consolidation equal to 50 percent for all consolidation pressures; since the corresponding time factor, T_{50}, equals 0.197, Eq. (14) can be written as

$$c_v = \frac{0.197\, h^2}{4t_{50}} \tag{15}$$

The ranges of c_v values corresponding to the various consolidation pressures are shown in Figure 6 together with the average values and ranges of the 70 percentile (70% of the values lie within this range). Although this relationship exhibits a minimum for c_v at a consolidation pressure of about 125 kN/m^2, a reasonably

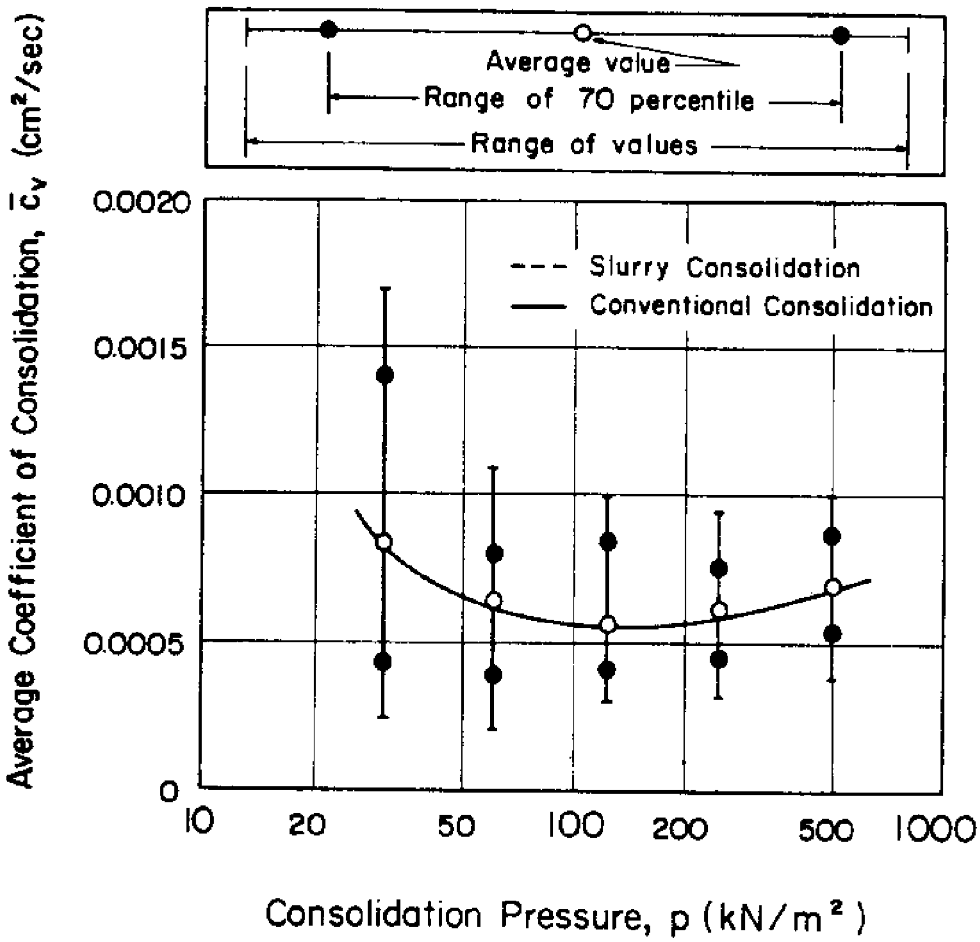

Figure 6. Average Coefficient of Consolidation versus Consolidation Pressure

constant value of about 0.0006 cm^2/sec can be used to characterize the time dependency of the consolidation response of these dredgings (except for the somewhat higher c_v values at the lowest consolidation pressure, 31 kN/m^2). As can be seen, the range of c_v values at a given consolidation pressure decreases somewhat as p increases, but this effect is not particularly large. The manifested variation of $\bar{c}_v$ with p is similar to those reported by Taylor (1948) for Boston blue clay, Chicago clay, Newfoundland silt, and Newfoundland peat. The increase in c_v with consolidation pressure for p values greater than 125 kN/m^2 may be attributed to the high gradients across the sample.

Coefficient of Volume Compressibility

The coefficient of volume compressibility, m_v, represents the increment of axial strain, $\Delta\varepsilon_a$, due to a unit increment of axial stress, Δ_p; in mathematical form m_v (which is essentially the reciprocal of the constrained tangent modulus) may be written as

$$m_v = \frac{\Delta\varepsilon_a}{\Delta_p} = \frac{a_v}{1 + e} \quad , \tag{16}$$

in which a_v is the coefficient of compressibility and e is the initial void ratio corresponding to any applied pressure. Figure 7 shows the total ranges, ranges of the 70 percentile, and the average values of m_v as a function of the consolidation pressure. It is significant in this case that the ranges for m_v decrease substantially as p increases, although the percent variations from the average values remain essentially the same. Furthermore, the average values of m_v can be represented by

$$\bar{m}_v = 0.0036 - 0.00125 \log_{10} p \tag{17}$$

where $\bar{m}_v$ and p are expressed in m^2/kN and kN/m^2, respectively.

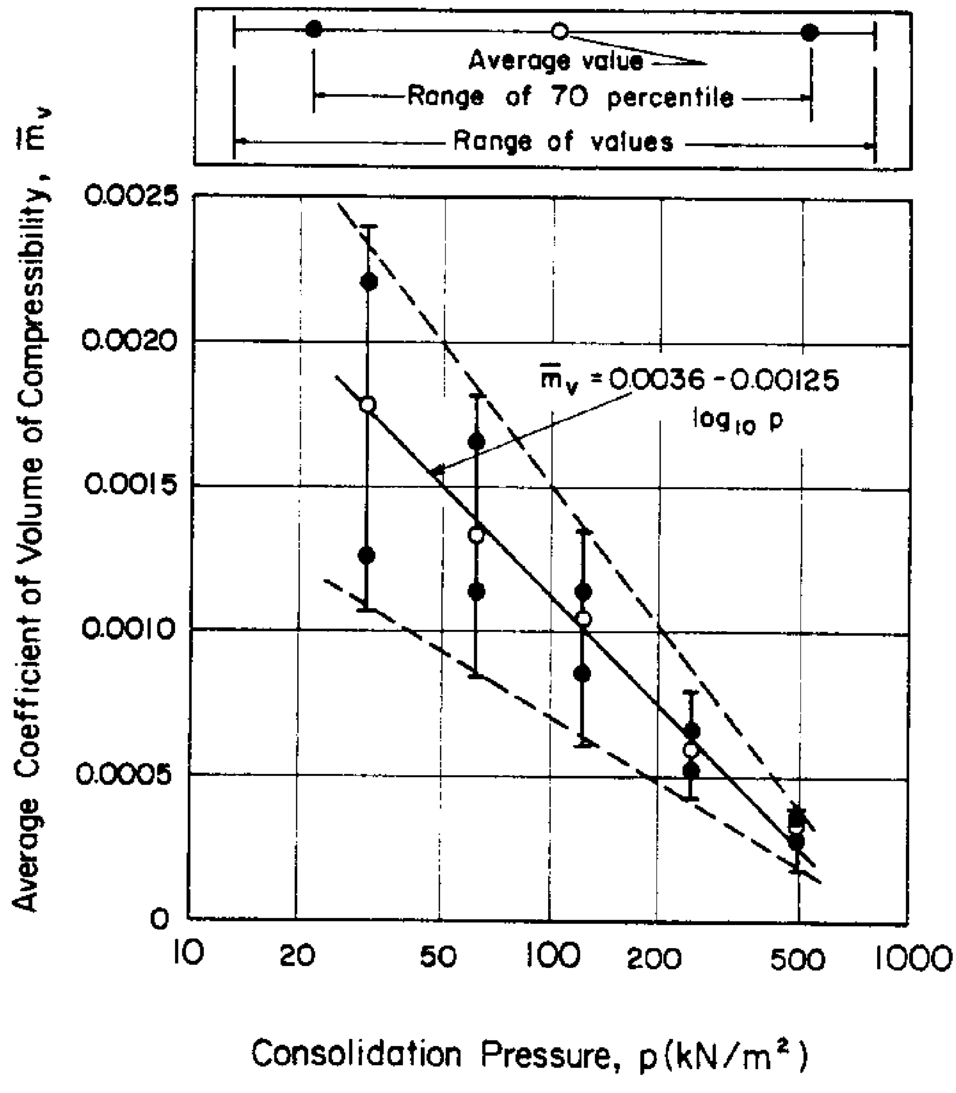

Figure 7. Average Coefficient of Volume Compressibility versus Consolidation Pressure

Coefficient of Permeability

Permeability values determined from consolidation data have been found by Newland and Allely (1960) and others to agree well with measured permeability values. In order to evaluate this correlation for dredged materials, the coefficient of permeability, k, has been evaluated from consolidation test data at each consolidation pressure by use of

$$k = \frac{a_v \gamma_w c_v}{1 + e} = c_v m_v \gamma_w \tag{18}$$

where e is the initial void ratio for each load increment; values for the initial void ratio have been calculated from a knowledge of natural water contents (measured when trimming the specimen in preparation for the test) and volume changes due to consolidation under each load increment. The change in k with e during the process of consolidation for a typical sample is shown in Figure 8a, and the variations of k with e for different values of p are shown in Figures 8b through 8f. The associated regression equations (based on test data from forty-eight speciments) describing each case all take the form

$$k = \log_{10}^{-1}[\alpha - \beta] \tag{19}$$

where α and β are empirical coefficients. Values for these coefficients, together with the standard deviations and correlation coefficients for the respective relationships are given in Table 1 for the case where k is expressed in cm/sec.

Table 1. Summary of Empirical Coefficients for Permeability Relationships

Range of Consolidation Pressure (kN/m^2)	α	β	Standard Deviation	Correlation Coefficient
0 - 31	1.10	8.66	0.28	0.71
31 - 62	0.92	8.54	0.14	0.73
62 - 124	0.66	8.27	0.12	0.61
124 - 248	0.92	8.66	0.11	0.68
248 - 496	0.97	8.77	0.10	0.72

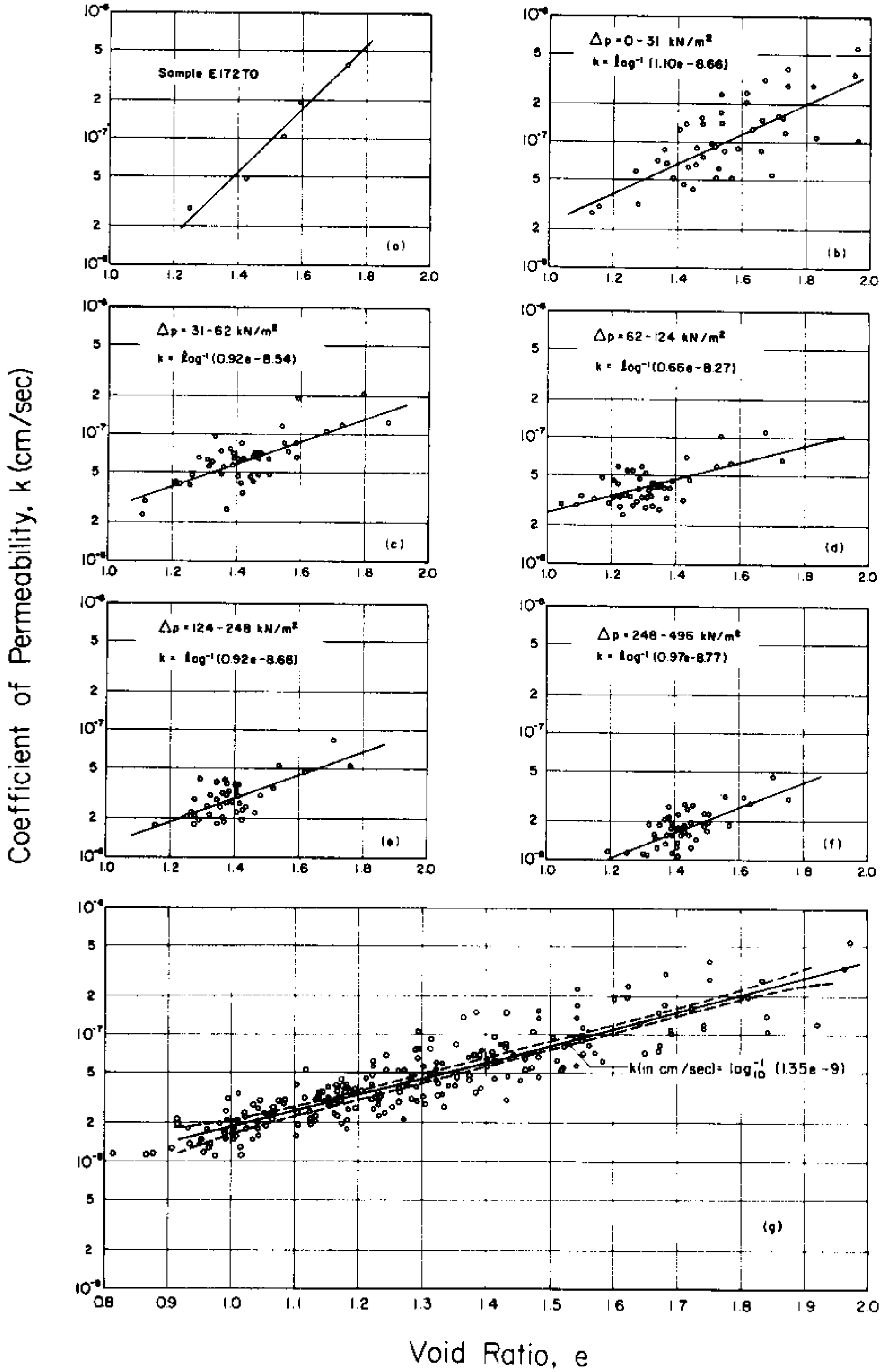

Figure 8. Coefficient of Permeability versus Void Ratio

Although the variations of γ and β in Table 1 suggest that k is a function of both e and p, the latter two parameters are not mutually independent; hence, it should be sufficient to relate k to either e or p. Accordingly, Figure 8g shows a correlation of k with e irrespective of p, and a regression analysis for k (in cm/sec) yields

$$k = \log_{10}^{-1}(1.35\, e - 9) \tag{20}$$

with a correlation coefficient of 0.85. The dashed curves in Figure 8g give the 90% confidence limits for the true mean value of k for any given value of e. The average coefficient of permeability, $\bar{k}$, together with its total and 70 percentile ranges, is given in Figure 9 as a function of p, and the relationship can be represented by

$$\bar{k} = \log_{10}^{-1}[-5.55 - \log_{10} p] \tag{21}$$

where p is in kN/m^2 and $\bar{k}$ in cm/sec.

As can be seen from Figure 8, most values for k, as determined from consolidation test data, lie between 10^{-6} cm/sec and 10^{-8} cm/sec with the lower values being associated with higher consolidation pressures and lower void ratios. This range of values may be compared with that found from direct permeability tests conducted in an Anteus device (Lowe, Jonas, and Obrician, 1969) on a series of six dredging samples which were similar (but not the same) to those tested in the experimental program reported herein. In the direct tests undisturbed samples were trimmed and placed in the Anteus apparatus, subjected to a back-pressure of 80 psi (550 kN/m^2), consolidated under a 10 psi (69 kN/m^2) stress for at least 12 hours, unloaded, and tested under falling head conditions while still subjected to back-pressure. The initial head difference was about 1.5 feet (0.5 meters), and the seepage length was about one inch (2.5 cm). The coefficients of permeability determined by this technique ranged from about 6×10^{-7} cm/sec to 9×10^{-8} cm/sec, which is in very good agreement with the range determined from consolidation test data.

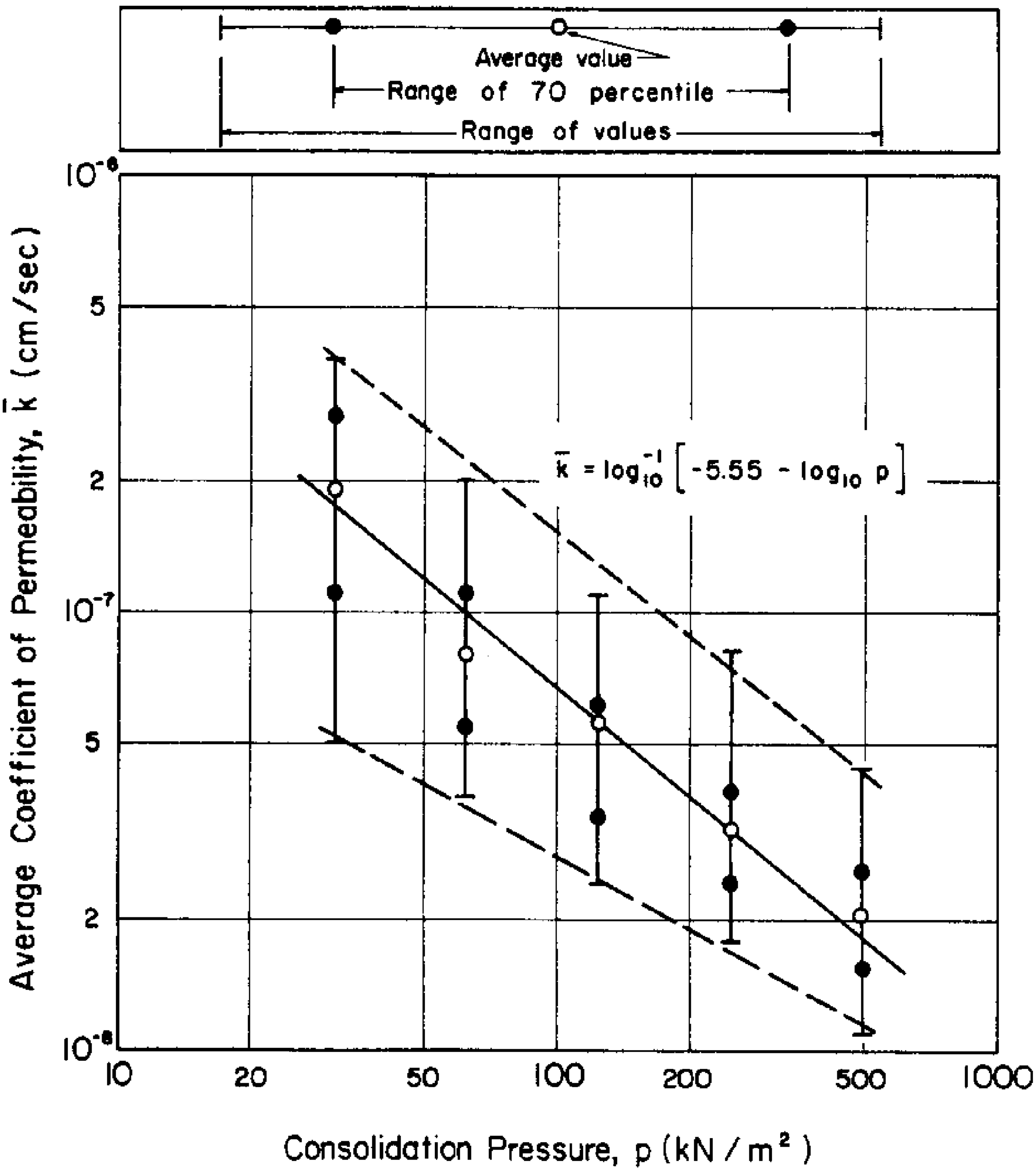

Figure 9. Average Coefficient of Permeability versus Consolidation Pressure

CONCLUSIONS

Based on various statistical analyses of data from sixty-four consolidation tests on samples of maintenance dredgings, the following conclusions can be advanced:

1. Consistent with the observed response for most types of clayey soils, the compression index increases linearly with both the natural water content and the liquid limit; virtually all values lie between 0.3 and 0.7, which comprises the lower end of the range reported by other investigators.

2. The average coefficient of consolidation varies with the logarithm of the consolidation pressure in a nonlinear manner, but the degree of nonlinearity is relatively insignificant; for all practical purposes a value of 0.0006 cm^2/sec can be assumed, except possibly for very low pressures

3. The average value of the coefficient of volume compressibility decreases linearly with an increase in the logarithm of the consolidation pressure.

4. The logarithm of the coefficient of permeability increases linearly with void ratio at all consolidation pressures, and its average value decreases with the logarithm of the consolidation pressure; values of the coefficient of permeability varied between 10^{-8} cm/sec and 10^{-6} cm/sec.

5. The constrained tangent modulus varies in a linear fashion with the stress level; the constant of proportionality, designated as the modulus number, was found to vary between 8 and 20, which lies in the range of 2 to 30 reported for a large variety of clays.

6. The modulus number varies linearly with the dry density, natural water content, and percent clay, increasing with the first and decreasing with the other two; however, its dependence on the percent clay was found to be relatively small.

ACKNOWLEDGMENTS

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NOTATION

A = coefficient introduced in the relationship for constrained tangent modulus;

a = coefficient introduced in the relationship for constrained tangent modulus;

a_v = coefficient of compressibility;

B = coefficient introduced in the relationship for constrained tangent modulus;

C_c = compression index;

c_v = coefficient of consolidation;

$\bar{c}_v$ = average value of coefficient of consolidation;

e = void ratio;

e_o = initial void ratio;

H = thickness of soil layer or soil sample;

h = length of shortest drainage path;

k = coefficient of permeability;

$\bar{k}$ = average value of coefficient of permeability;

M = constrained tangent modulus;

m = modulus number;

m_v = coefficient of volume compressibility;

$\bar{m}_v$ = average value of coefficient of compressibility;

p = consolidation pressure or uniaxial stress;

p_a = atmospheric pressure;

p_c = preconsolidation pressure;

S = settlement;

T = time factor;

t = time;

t_{50} = time corresponding to 50 percent of primary consolidation;

t_{100} = time corresponding to completion of primary consolidation;

U = degree of consolidation;

w_L = liquid limit;

w_n = natural water content;

α = coefficient introduced in the relationship for the coefficient of permeability

β = coefficient introduced in the relationship for the coefficient of permeability;

ϵ_a = axial strain;

ΔH = change in sample height;

Δe = change in void ratio;

Δp = pressure increment;

$\Delta\epsilon_a$ = increment of axial strain.

STABILIZATION OF POLLUTED DREDGINGS BY ELECTRO-OSMOSIS

by

Raymond J. Krizek[1]

Francis B. Cularte[2]

Paul L. Hummel[3]

INTRODUCTION

Until recently, most of the sediments dredged from the harbors and waterways adjacent to the Great Lakes were deposited in the open waters at selected disposal areas located sufficiently near the harbors to minimize dredging costs, but far enough away to avoid interference with water intakes, beaches, and other facilities. However, with the population growth and industrial development of a given region, the dredged sediments have become increasingly polluted, and current concern for protection of the environment has led to a ban against open lake disposal of dredged materials that are classified as polluted by the Environmental Protection Agency. Pursuant to a study by the Corps of Engineers (3), the currently used major alternative to open lake disposal is to deposit these materials behind dikes in areas near the harbor; such a procedure has been found to satisfactorily prevent the polluted sediments from reaching the open waters, and, although very costly, it is generally more economical than any other means of handling the dredgings, except lake disposal.

With the established desirability of using diked containment areas to dispose of polluted dredgings, there is a need to study the methods whereby

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the quality of the resulting landfill can be enhanced. The potential usefulness of a landfill composed of polluted dredgings is largely governed by the degree of stability which can be achieved by treatment during and/or after deposition, and this study is directed toward evaluating the possibility of stabilizing such a land mass by use of electro-osmosis (1,3,4,5). To achieve this end, four typical dredged materials from different locations in the Great Lakes region have been subjected to laboratory tests with two different electrical gradients, plus control tests with no electrical gradient, and the effects of this treatment on the strength, compressibility, time rate of dewatering, and release of contaminants in the effluents have been measured. The resulting data serve to indicate the practical and economical feasibility of applying electro-osmosis to improve the engineering properties of polluted dredged materials.

EXPERIMENTAL PROGRAM

The materials used in this study are designated as C22TO (slurry from the discharge pipe at the island disposal site at Toledo, Ohio), C27MM (slurry from the discharge pipe at the disposal site at Monroe, Michigan), D1DM (bottom sediment from the Rouge River at Detroit, Michigan), and D4CO (bottom sediment from the Cuyahoga River at Cleveland, Ohio). Some characteristics of these samples are summarized in Table 1, and the respective particle size distribution curves, as obtained with and without a dispersant (Calgon), are shown in Figure 1.

Apparatus

An apparatus was designed and constructed to enable (a) the uniform consolidation of dredging samples under a pressure of approximately 0.6 kg/cm^2 and (b) the simultaneous application of an electrical voltage across the ends

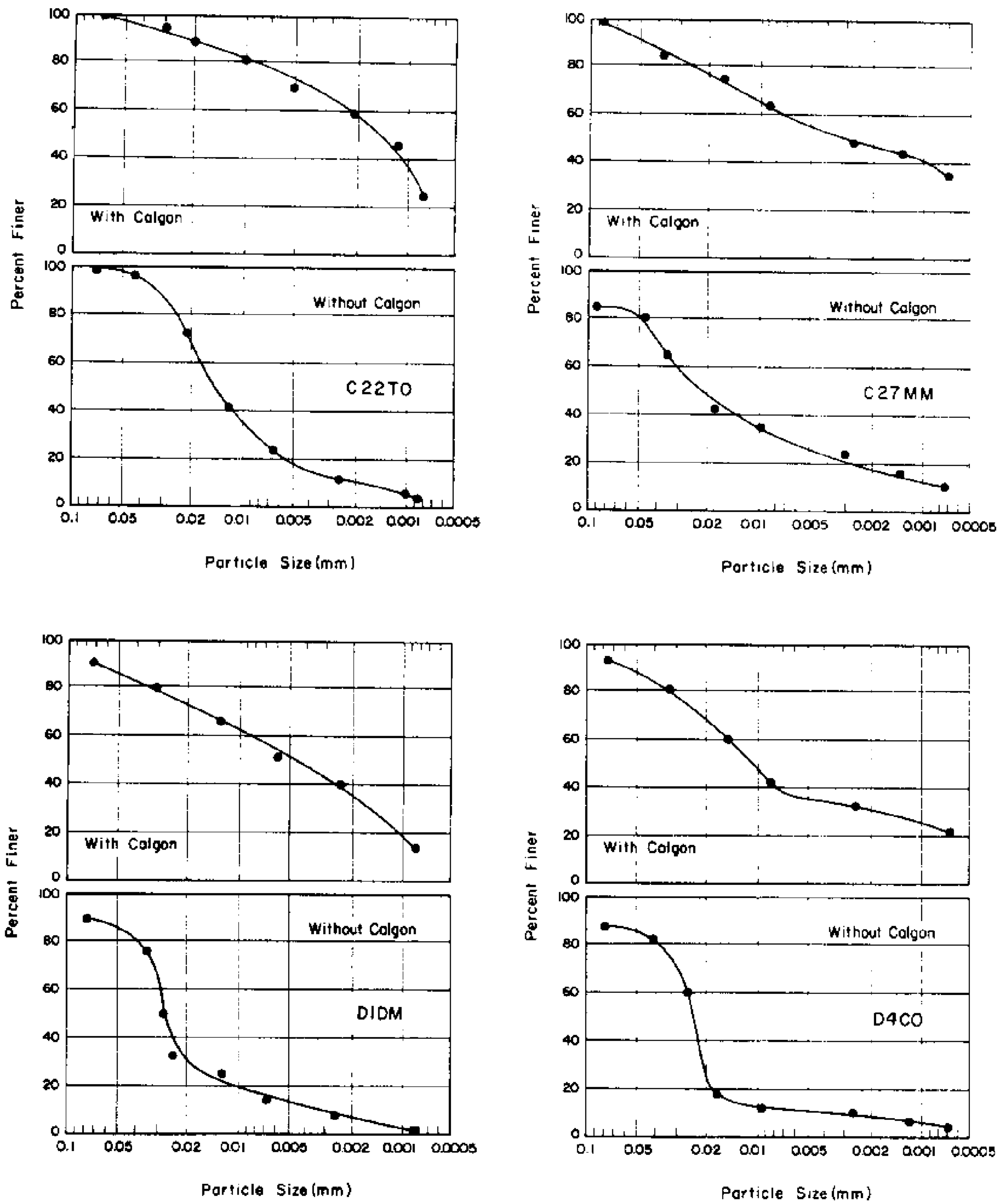

Figure 1. Particle Size Distribution Curves

Table 1

Engineering Characteristics of Samples Tested

Sample	Water Content %	Solids Content (gm/cc)	Organic Content %	Specific Gravity	Liquid Limit %	Plasticity Index %	Shrinkage Limit %
C22TO	138.0	0.510	3.7	2.56	69	29	24
C27MM	142.8	0.531	7.6	2.51	59	10	23
D1DM	99.3	0.717	12.5	2.53	56	17	40
D4CO	117.0	0.650	8.2	2.74	50	13	40

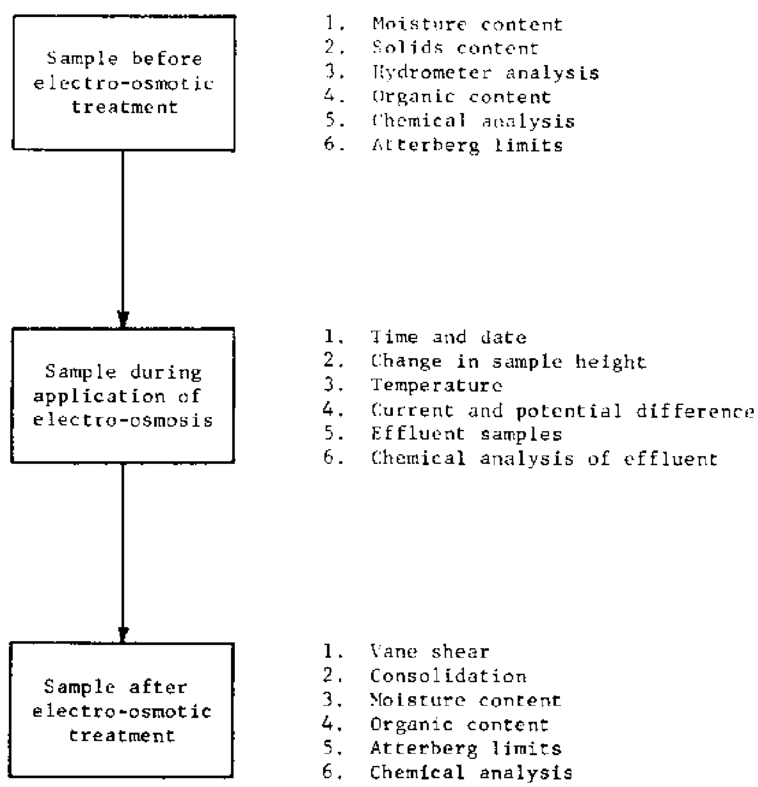

Figure 2. Outline of Experimental Program

of the sample. Pilot tests indicated that the use of electro-osmosis alone yields poor results because the dredged materials tended to contract and pull away from the anode, thereby weakening or breaking the electrical contact; hence, the simultaneous application of a 0.6 kg/cm^2 vertical stress together with the electrical gradient was used to eliminate this effect, and control tests were conducted to evaluate the effect of the load alone. The apparatus consisted of three plexiglass cells (height of 25 cm, inside diameter of 14 cm, and wall thickness of 0.6 cm), a loading frame capable of loading three cells simultaneously, and the necessary electrical equipment to apply a voltage difference across two of the cells; no electrical gradient was applied across the third cell. A thin teflon sheet was affixed to the inside surface of each cell to provide more uniform consolidation by minimizing the side-wall friction as much as possible. A movable piston with two O-rings was fitted to the inside of the lined cell with the lowest amount of drag consistent with maintaining an effective seal for the pore water pressures developed due to loading. Carborundum porous stones were attached to the piston and the base, and separate drainage outlets were provided for each. The cell was held in place by means of four assembly rods which connected a top plate to the base.

A fixture for holding a thermometer was attached to the exterior of each cell at approximately the mid-point of the initial sample height; this permitted the measurement of temperature changes in the samples. Several small holes were drilled into the side of one of the cells to allow the electrical voltage to be monitored at several points within the sample by means of a stainless steel probe. Scales were affixed to the outside of the cells to measure changes in sample height. The loading system consisted of a structural steel frame with hydraulic cylinders mounted directly over each cell. The hydraulic cylinders were connected to a common air-oil interface reservoir, wherein the air pressure was supplied by a constant air pressure source and regulated by a constant bleed air regulator, which controlled the air pressure to within 0.05 kg/cm^2 during testing. The electrical system consisted of two power sources and a Wheatstone bridge, which was arranged to measure the voltage difference across the ends of the sample. The direct current power sources were supplied by a constant voltage regulator to reduce line fluctuations.

Testing Procedure

Voltage differences corresponding to nominal electrical gradients of one-half and one volt per centimeter of initial sample height were applied and maintained constant throughout the tests (hence, the actual electrical gradient across each sample increased with time, because the sample height decreased); in addition, a control test with no electrical voltage difference was conducted on each material. All samples were drained at both top and bottom surfaces, but the electrical gradient always induced flow toward the bottom.

The testing sequence and parameters measured during each stage are outlined briefly in Figure 2; all tests were performed inside a humid room to minimize evaporation and temperature fluctuations. In addition to the usual measurements associated with such a test, effluent samples were collected and subjected to chemical analyses to assess the concentrations of contaminants that were being emitted with the pore water. After the electro-osmosis portion of the test was completed, the consolidated sample was subjected to a series of vane shear tests. A motor-driven vane, rotated at a rate of one revolution per hour until failure of the soil, was used to perform three vane shear tests at each of three different depths in each sample. The remolded strength was determined by revolving the vane rapidly through ten revolutions and then repeating the test at the same location. A specimen taken from the center of the sample was immediately subjected to a conventional consolidation test. The tests for organic content, Atterberg limits, and chemical constituents were then performed on the remainder of the sample.

RESULTS AND INTERPRETATION

Presented herein are the results of the experimental investigation and interpretations of the observed effects, which include the effects on chemical constituents and the resulting changes in the engineering properties of the dredgings.

Chemical Effects

Since the primary purpose for placing dredged materials in confined disposal areas is to prevent pollutants from entering the open water, it is important to assess the quality of both the effluent and the dewatered material in terms of the concentrations of contaminants present in the original dredged material. It is also of interest to assess the effect, if any, of the various constituents in the dredged materials on the effectiveness of dewatering by electro-osmosis.

The major chemical effects recorded in Tables 2 and 3 are probably due to two principal causes, namely, the loss of ions into the effluent and the loss of compounds in the generated gases. The desired effect of dewatering by electro-osmosis is reflected in Table 2 in the noticeable increase in the percentage of total solids with larger electrical gradients. A similar trend in the percentage of total volatile solids is not so well defined; this is

Table 2

Chemical Analyses of Samples Tested

Sample	Electrical Gradient (volt/cm)	Total Solids %	Total Volatile Solids %	In-Organic Carbon %	Organic Carbon %	pH	NH_3-N (mg/gm)	Organic Nitrogen (mg/gm)	Al (mg/gm)	Ca (mg/gm)
C22TO	Slurry	48.83	4.39	1.43	3.72	7.50	211.06	12.77	7.02	13.20
	0	60.97	5.26	1.36	3.38	8.55	225.72	877.35	8.94	18.30
	½	61.82	6.53	1.14	3.60	7.50	130.92	372.50	10.30	20.90
	1	63.11	5.83	1.37	1.84	8.85	49.46	426.70	7.98	23.20
C27MM	Slurry			1.93	7.55					
	0	62.27	7.82	1.97	5.23	8.06	56.05		8.58	18.20
	½			1.88	6.45					
	1	70.74	8.93	1.86	6.12	7.56	36.12		7.26	17.20
D1DM	Slurry			1.62	12.50		161.68	81.25		
	0	66.99	10.38	1.01	13.13	8.10	144.36		7.46	10.90
	½			1.48	12.74					
	1	70.02	12.06	1.57	13.49	6.95	0		7.57	11.60
D4CO	Slurry	46.72	4.60	0.96	8.21	8.25			6.22	4.80
	0	67.21	6.76	0.90	7.87	8.95	493.40		9.95	8.32
	½			1.01	7.98					
	1	74.98	7.07	1.22	7.76	8.40	44.48		10.30	6.77

Sample	Electrical Gradient (volt/cm)	Cd (mg/gm)	Cu (mg/gm)	Fe (mg/gm)	K (mg/gm)	Na (mg/gm)	Pb (mg/gm)	SiO_2 (%)	CEC (mg/gm) Na	CEC (mg/gm) K	CEC (mg/gm) Ca
C22TO	Slurry	0.003	0.017	14.10	0.86	0.122	0.033	31.60			
	0			16.00	2.08	0.190		44.90			
	½			18.80	2.55	162.0		45.80			
	1			18.70	2.57	0.250		48.50			
C27MM	Slurry										
	0	0.002	0.208	10.70	2.40	0.220	0.095	40.10	0.067	0.140	3.02
	½										
	1	0.001	0.220	12.70	2.21	0.220	0.100	46.05	0.040	0.104	4.82
D1DM	Slurry										
	0	0.006	0.096	90.40	2.22	0.310	0.239	37.20	0.061	0.126	1.63
	½										
	1	0.063	0.105	88.90	2.52	0.370	0.232	38.40	0.016	0.081	2.53
D4CO	Slurry	0.014	0.095	82.60	1.61	0.300	0.160	25.80			
	0	0.022	0.152	131.00	2.62	0.370	0.246	35.70	0.177	0.148	1.88
	½										
	1	0.024	0.158	144.10	2.31	0.270	0.245	41.30	0.016	0.065	1.77

Table 3

Chemical Analyses of Effluents from Samples Tested

Sample	Electrical Gradient (volt/cm)	Specimen Number	Total Solids (gm/l)	Total Volatile Solids (gm/l)	NH_3-N (mg/l)	pH	Al (mg/l)	Ca (mg/l)	Cu (mg/l)	Fe (mg/l)	K (mg/l)	Na (mg/l)
C22TO	0	1			132	8.9	0	102	0.35	0.19	12	290
		2	0.129		84	8.2	25	2000		14.00	87	260
	½	1	0.561		138	9.9	0	65	0.80	9.20	7	320
		2	1.849		246	12.4	0	550	0.70	8.30	20	530
		3	0.233		320	12.5	0	640		2.50	290	330
	1	1			172	12.1	12	340	0.85	9.30	15	360
		2	2.297		400	12.3	0	678	0.35	2.00	28	630
		3	0.257		456	12.6	26	5800		33.00	160	205
C27MM	0	1	0.607	0.129	46	8.1		13.8			13	39
		2	0.732	0.154	32	8.2		11.0			16	41
	1	1	0.539	0.105	115	11.4		0.5			15	90
		2	2.073	0.125	228	12.4		240			24	38
D1DM	0	1	0.988		132	7.9	0	340		0	120	900
		2			120	9.4						
	1	1	0.107		210	12.0	26	950		101.00	170	1170
		2			384	11.3						
D4CO	0	1 Bottom	0.703	0.252	255	8.6		12.5			18	97
		1 Top	0.760	0.289		8.4		5.8			18	100
		2	0.904	0.329	354	8.9		5.4			21	170
	1	1 Bottom	1.363	0.208	724	12.3		1.2			170	380
		1 Top	2.377	1.179		5.0		68			12	41
		2	1.574	0.109	1117	12.4		2.4			225	420

probably due to the increased generation of gases with the larger gradients and the difficulty in obtaining a truly representative sample of the specimen for chemical analysis. The inorganic and organic carbon showed minor random variations, which are very likely due to a combination of the minor effects of the electrical treatment and sampling difficulties.

The electrical treatment did, however, have a pronounced effect on the concentrations of ammonia nitrogen. Smaller amounts of ammonia nitrogen remained in the specimens subjected to the larger gradients; this is due to the fact that large amounts were removed with the effluent and probably to the generation and release of gases. The losses of soluble ions from the sediments subjected to electrical treatment are reflected in gains for the same ions in the respective effluents, wherein the concentrations of the soluble ions per unit volume, as well as the quantities of effluent discharged, increased as the gradient increased. In Table 3 effluent samples 1, 2, and 3 designate samples which were collected at the beginning, middle, and end, respectively, of the electro-osmotic dewatering process; for cases where a third effluent sample was not taken, sample 2 represents the effluent collected at the end of the test.

As expected, the heavy metals remained in the solids, regardless of the degree of electrical treatment; observed variations are probably due to testing and sampling variations. A comparison of the test data for potassium and calcium from effluent samples 1,2, and 3 for the C22TO material indicates a general increase in concentration with the progress of dewatering, but the concentration of sodium reached a maximum for the middle sample and then decreased. In the case of the D4CO material, the initial effluents obtained from the top and bottom of the specimens subjected to no electrical gradient and 1 volt/cm electrical gradient were analyzed separately; it is noted that the concentrations of both the potassium and the sodium ions are much higher in the fluids drained from the bottom compared to those drained from the top for the 1 volt/cm electrical gradient, but there is little change in the ion concentrations of the effluents from the top and bottom for the zero electrical gradient test. The decreases in the cation exchange capacity for sodium and potassium again reflect the loss of these soluble ions to the effluents, but the effects on calcium are not so pronounced. The pH generally increased slightly in the effluent with larger gradients, while it decreased in the remaining solid.

Effects on Engineering Properties

The effectiveness of electro-osmosis in dewatering and consolidating these samples is reported in Figures 3 through 6, which show the volume of fluid discharged from the sample, the change in sample height, and the current used as a function of time for various electrical gradients. A study of these plots indicates that an increase in the electrical gradient caused an increase in the amount of fluid discharged. The higher gradient produced its major effect in a shorter period of time than the lower gradient; for the thicker slurries (D materials), the major effect caused by the higher gradient occurred in about one day, whereas that for the lower gradient occurred in about two days.

In each case the dewatering process with an electrical gradient is faster than that associated with a pressure gradient only. However, in making a comparison of the time rates of fluid discharged with and without an electrical gradient, attention must be given to the fact that drainage was allowed from both the top and bottom stones. Under zero electrical gradient,

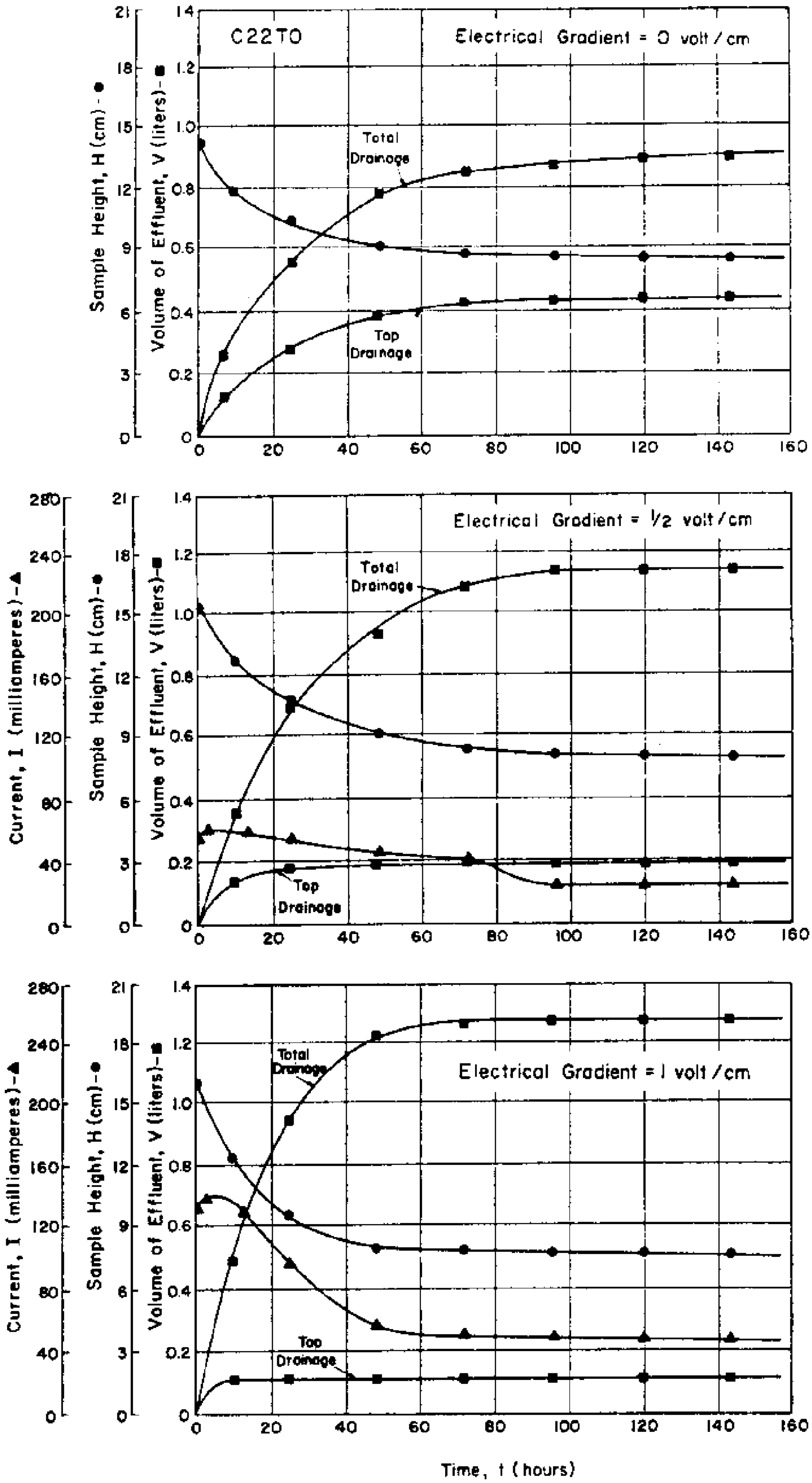

Figure 3. Response of Sample C22TO

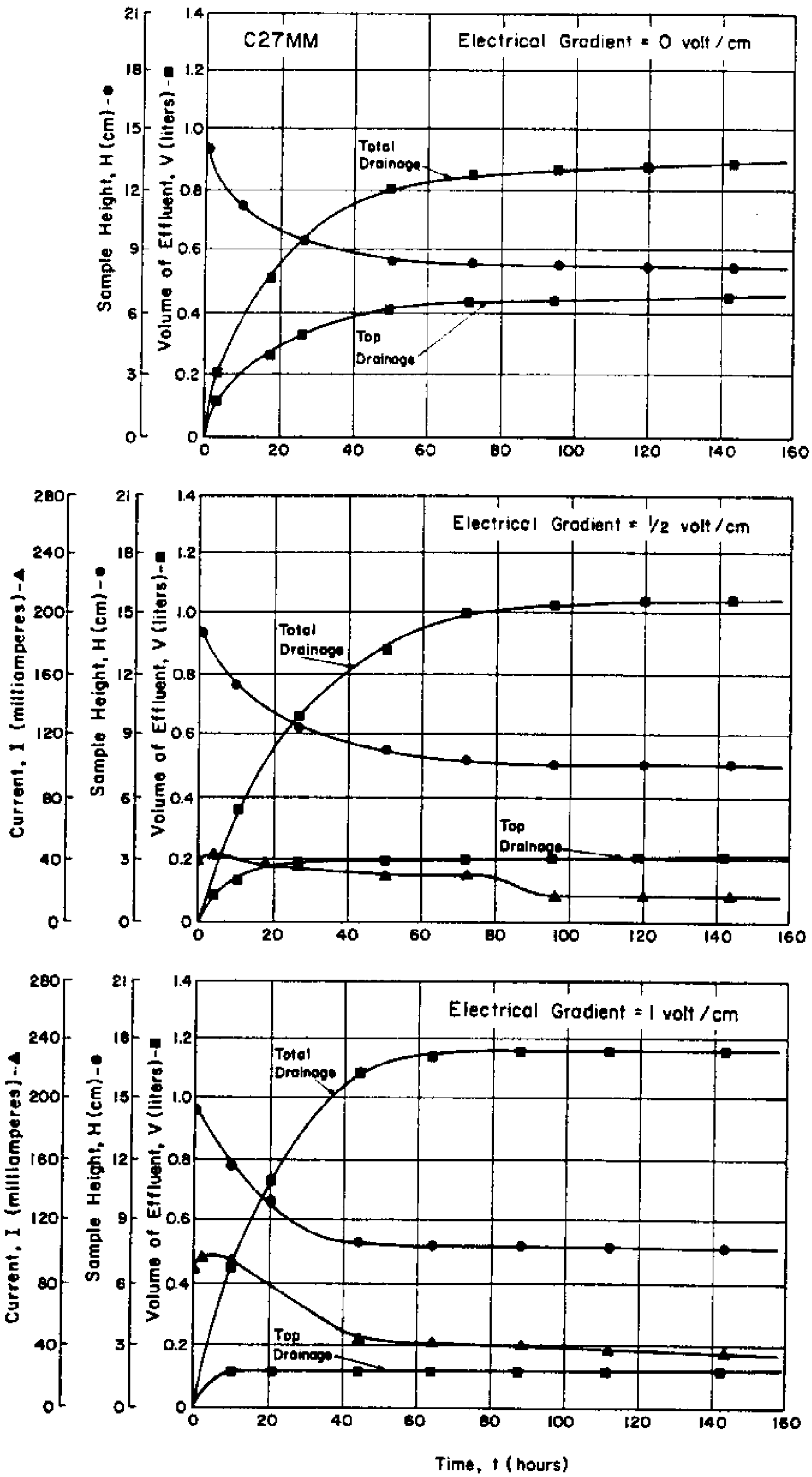

Figure 4. Response of Sample C27MM

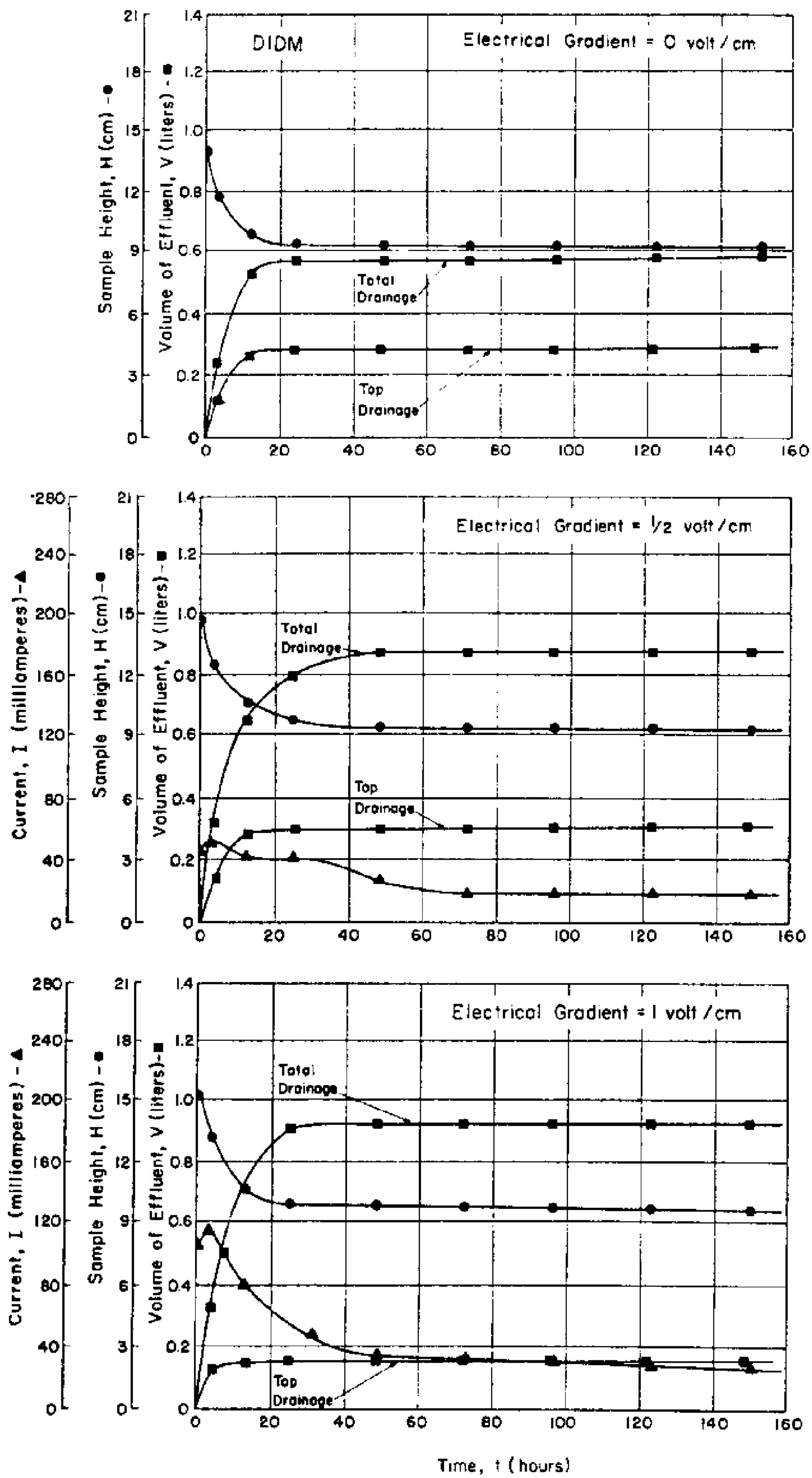

Figure 5. Response of Sample DIDM

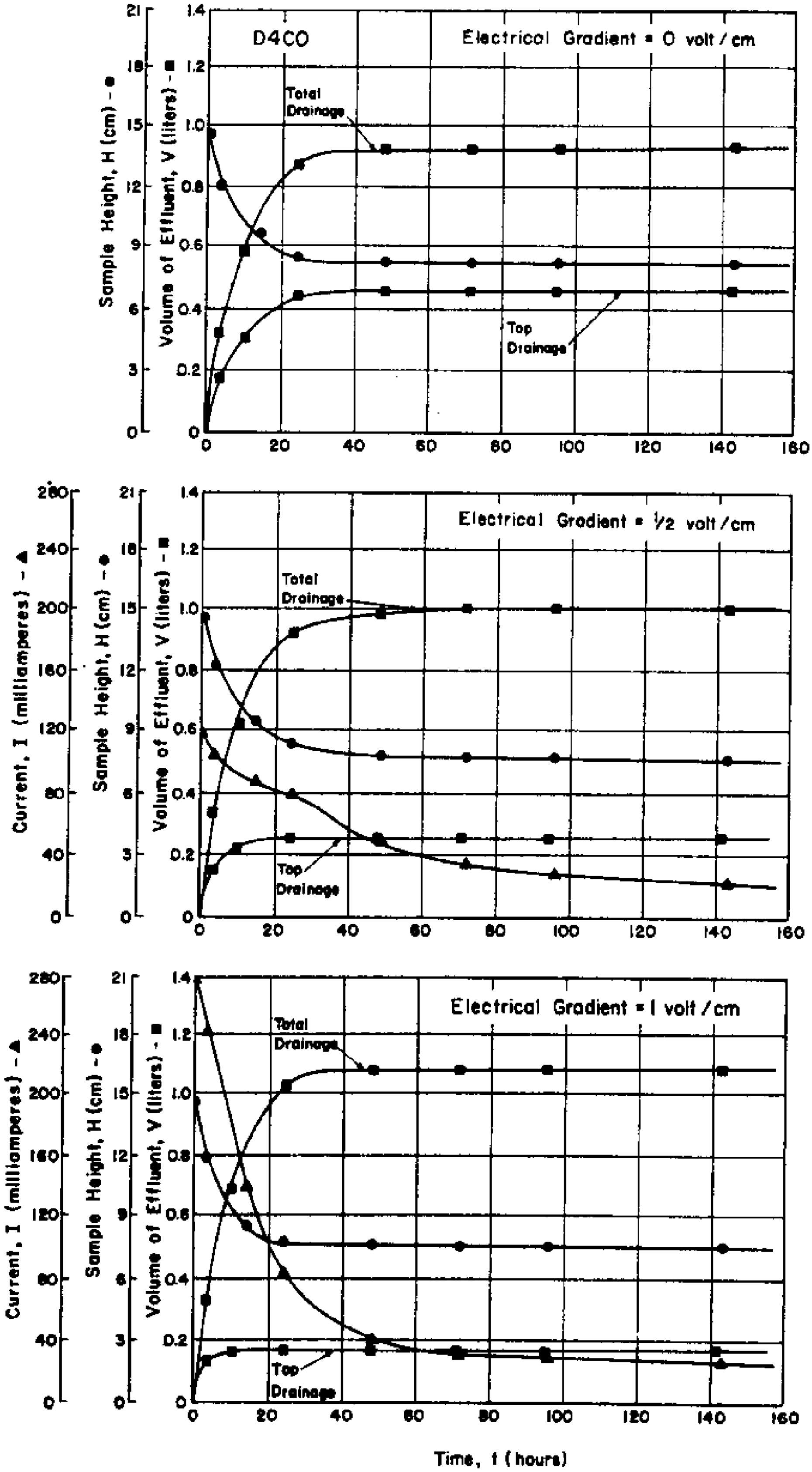

Figure 6. Response of Sample D4CO

drainage would take place from the center of the sample toward both the upper and lower porous stones. Since the applied electrical gradient tends to drive the water downward, the pressure and electrical gradients oppose each other in the upper half of the specimen; this effect can be noted in Figures 3 through 6 by observing the smaller proportion of effluent that is discharged through the upper stone. For all practical purposes, gravity effects are essentially negligible compared to the other gradients involved in this phenomenon. The current can also be seen to decrease as the rate of flow decreases. Although the complex nature of these tests (variable gradient with time, drainage conditions, time-dependent changes in material characteristics, etc.) preclude any straightforward simple interpretation of the data, a first-order approximation of the electro-osmotic coefficient of permeability, k_e, can be determined from the information presented in Figures 3 through 6 and use of the relationship

$$k_e = \frac{Q\,H}{t \Delta V\,A} \quad , \tag{1}$$

where Q is the total volume of fluid discharged in time, t, from a sample of cross-sectional area, A, and height, H, due to a voltage difference, ΔV. The k_e values given in Table 4 were found by using the average quantity of flow due to the electro-osmotic action over a particular period of time (either 10, 20, or 40 hours). Although substantial variations are observed, values for the electro-osmotic coefficient of permeability, k_e, are generally of the same order of magnitude as those which characterize a large variety of soils (that is, 5×10^{-5} cm/sec per volt/cm). Also given in Table 4 are values for the conventional coefficient of permeability, k, as determined from consolidation test data.

Table 4

Effectiveness of Dewatering by Electro-osmosis

Sample	Time, t (hours)	Flow Rate, q (cc/sec)	Average Gradient, $\bar{v}$ (volts/cm)	Electro-osmotic Coefficient of Permeability, k_e (cm^2/volt-sec)	Coefficient of Permeability, k (cm/sec)	Flow Efficiency: $\frac{\text{milliliters}}{\text{watt-hour x liter}}$	Flow Efficiency: $\frac{\text{gallons}}{\text{watt-hour x cu. yd.}}$
C22TO	20	0.00264	1.16	1.49×10^{-5}	3.2×10^{-7}	17.9	3.62
	40	0.00246	1.23	1.31×10^{-5}		17.9	3.62
C27MM	20	0.00493	1.19	2.70×10^{-5}	5.5×10^{-7}	24.6	4.97
	40	0.00396	1.28	2.02×10^{-5}		23.1	4.66
D1DM	10	0.00805	1.16	4.54×10^{-5}	2.9×10^{-7}	30.4	6.14
	20	0.00611	1.21	3.30×10^{-5}		27.4	5.54
D4CO	10	0.00695	1.23	3.69×10^{-5}	9.0×10^{-7}	13.0	2.62
	20	0.00556	1.31	2.78×10^{-5}		12.9	2.60

These same data may be used to calculate the effectiveness of the electro-osmotic treatment in terms of the anticipated quantity of flow per watt-hour of energy per cubic yard of dredgings. The calculations involve the use of flow quantities from only the bottom half of the sample (where the pressure gradient and electrical gradient are additive), with the flow quantity caused by the pressure gradient being subtracted from the total flow quantity caused by both the pressure gradient and the electrical gradient to give the flow quantity due to the electrical gradient only. Average values of data over the first 10- and 20-hour periods were used for samples D1DM and D4CO and over the first 20- and 40-hour periods for samples C22TO and C27MM; appropriate flow quantities were determined from the experimental curves at the specified times, and the energy consumed during the associated time interval was taken as the area under the current-time curve multiplied by the applied voltage. The effectiveness of the electro-osmotic treatment is expressed in Table 4 as the quantity of water per watt-hour of energy per unit volume of slurry. A study of these calculations indicates that the quantity of water removed from each of the four materials varies from about 3 to 6 gallons per watt-hour per cubic yard for treatment times of 10, 20, or 40 hours; also, the effectiveness is nearly the same for each material for both time periods selected. However, the energy consumption would increase sharply for time periods longer than those selected, since approximately 75 to 95 percent of the total flow occurs in the selected time periods, and additional treatment would produce little additional drainage.

Plots of the electro-osmotic flow rate versus the current are given in Figure 7 for each of the four dredgings. A general pattern of increasing flow rate with increasing current is observed, but the relationship is not

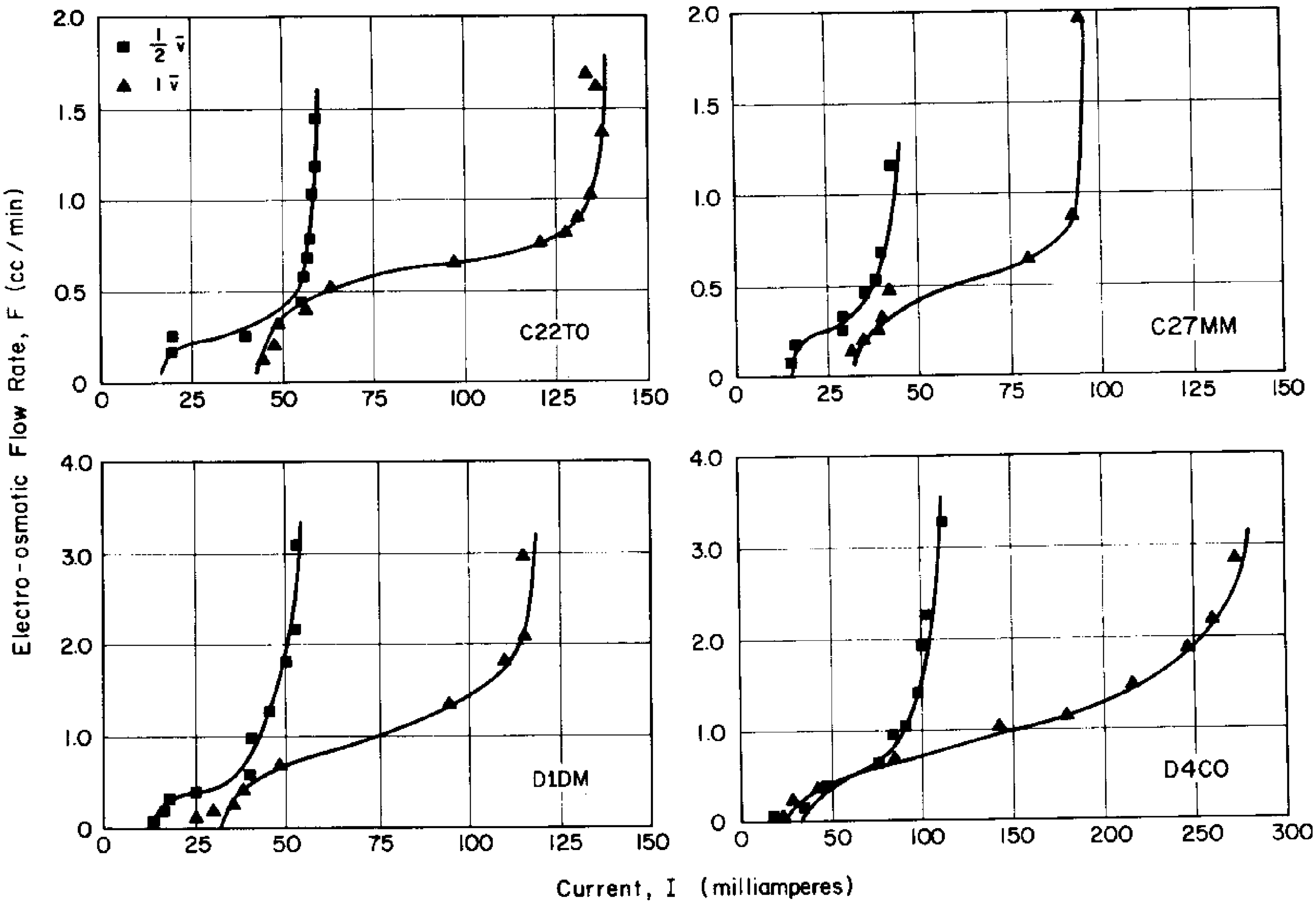

Figure 7. Electro-osmotic Flow Rate versus Current

linear, as found by Gray and Mitchell (4) for tests on clays under very well-controlled laboratory conditions. If Equation (1) is rewritten as

$$\rho k_e = \frac{Q}{I\ t} \quad , \tag{2}$$

where ρ and I are the specific electrical resistance of the material and the current, respectively, the basis for the Gray and Mitchell relationship can be readily appreciated, namely, the product of the material properties is a constant, ρk_e. However, the complex nature of the chemical constituents in the dredgings tested apparently does not justify the expectation that ρk_e be constant throughout these tests. Rather, the initial portions of the curves in Figure 7 generally increase at an increasing rate and exhibit a final very steep slope. The physical explanation for this phenomenon follows from the curves presented in Figures 3 through 6, where the flow rate is the slope of the volume of effluent versus time curve and the current is read directly. In most cases it takes a short period of time before the current reaches its maximum value, after which it decreases. At the instant the voltage is applied, the system is static; shortly afterward, it becomes dynamic, and the current reaches a maximum during the early stage of flow, which is represented in Figure 7 by the data in the upper right-hand portion of each curve. As time passes, the rate of fluid flow, as well as the current, decreases, and the curves in Figure 7 move downward and toward the left. Hence, the dewatering process continues at a decreasing rate for all samples and eventually reaches a condition where very little additional drainage occurs. This condition is indicated by the relatively rapid change in the slope of the curves in Figure 7. It is possible that a point is reached where the continuity breaks down between the dredgings sample and the anode, thereby causing a sudden drop in power consumption and flow of current.

Figure 8 shows the variation of moisture content with depth in each specimen tested. The specimens subjected to no electrical treatment show essentially no change in moisture content with depth, whereas the electrically treated specimens portray a marked decrease in moisture content at the top of the specimen with relatively larger moisture contents toward the bottom. The larger gradients produced lower water contents in all materials at all depths. The Atterberg limits and associated indexes presented in Table 5 indicate that the electrical treatment produced no noticeable trends. The minor differences observed may be attributed to normal variations in the test results, variations of material properties within the specimen, and the loss of ions from the original sample.

The conventional consolidation test data resulting from tests made on specimens trimmed from the samples consolidated by the electrical and pressure gradients are presented in Figure 9. The major effect seems to be due to the decreased initial void ratio in the specimens subjected to an electrical gradient. Although the preconsolidation stress generally appears to increase with increasing electrical gradient, this effect is not very well defined in these results. The compression index, C_c, for the different materials, varies from about 0.38 to 0.66, but it is relatively constant for a given material, regardless of the amount of electrical treatment imposed. These observations are consistent with expectations based on available theories for consolidation by electro-osmosis (3, 5).

In Table 6 are tabulated the average shear strengths of the top (anode end), center, and bottom (cathode end) of the sample. General trends of decreasing shear strength with increasing depth and decreasing electrical gradient can be noted. Although the overall shear strength was increased

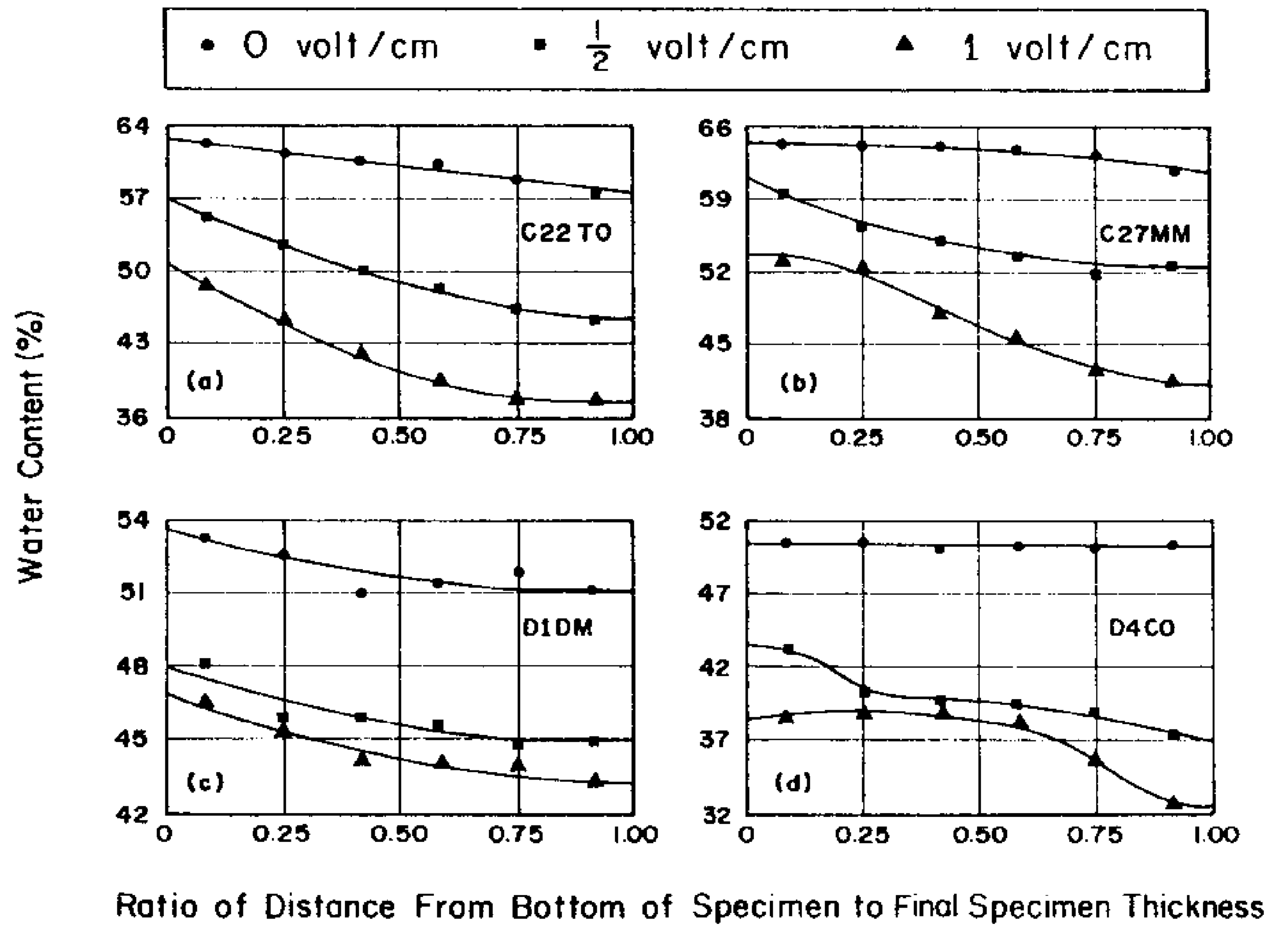

Figure 8. Variation of Moisture Content with Depth

Table 5

Effect of Electro-osmotic Treatment on Engineering Characteristics

Sample	Electrical Gradient	Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	Shrinkage Limit	Flow Index	Toughness Index
C22TO	INITIAL	138	69	40	29	24	16	181
	0 volt/cm	60	78	44	34	26	22	154
	½ volt/cm	50	75	45	30	30	20	150
	1 volt/cm	42	70	37	33	21	19	174
C27MM	INITIAL	143	59	49	10	23	14	72
	0 volt/cm	63	87	45	42	24	28	150
	½ volt/cm	55	81	55	26	25	16	163
	1 volt/cm	47	79	51	28	29	21	133
D1DM	INITIAL	99	56	39	17	40	13	128
	0 volt/cm	52	60	51	9	46	11	82
	½ volt/cm	46	58	45	13	53	15	87
	1 volt/cm	45	58	50	8	43	13	62
D4CO	INITIAL	117	50	36	13	40	9	149
	0 volt/cm	50	51	41	10	26	12	83
	½ volt/cm	40	55	44	11	37	14	79
	1 volt/cm	37	52	41	11	31	16	69

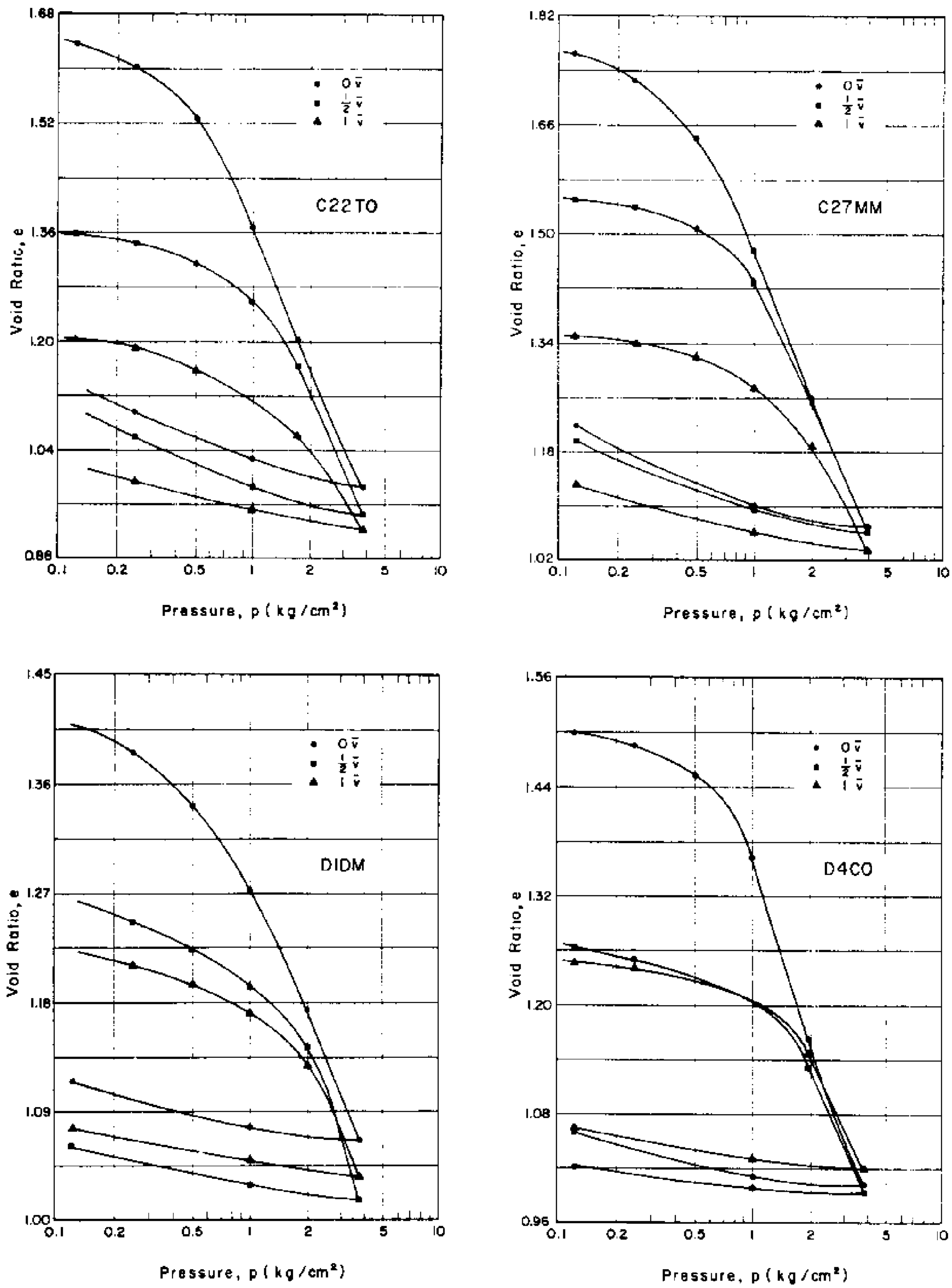

Figure 9. Void Ratio versus Pressure Curves

Table 6

Average Shear Strength Determinations

Sample	Electrical Gradient (volt/cm)	Location in Sample: Top — Average Shear Strength (psf)	Top — Sensitivity	Center — Average Shear Strength (psf)	Center — Sensitivity	Bottom — Average Shear Strength (psf)	Bottom — Sensitivity	Overall — Average Shear Strength (psf)	Overall — Sensitivity
C22TO	0	470	3.1	430	2.7	400	2.9	430	2.9
	½	1562	3.7	1006	2.8	811	2.9	1130	3.1
	1	2850	4.7	1720	3.0	1460	3.1	2010	3.6
C27MM	0	980	4.3	830	4.2	840	4.3	880	4.3
	½	2180	4.8	1620	4.2	1320	3.6	1710	4.2
	1	5040	7.5	3200	4.6	2180	3.4	3470	5.2
D1DM	0	550	3.1	550	3.3	480	3.2	530	3.2
	½	1580	4.5	1330	4.7	1270	3.7	1400	4.3
	1	1720	4.1	1500	3.6	1610	3.3	1610	3.7
D4CO	0	550	3.9	550	3.5	580	4.4	560	3.9
	½	2980	15.6	2650	8.4	2320	6.1	2650	6.1
	1	2570	4.4	2460	4.6	2370	4.5	2470	4.5

Table 7

Summary of Various Test Parameters

Sample	Electrical Gradient (volt/cm)	Initial Density (gm/cc) Wet	Initial Density (gm/cc) Dry	Final Density (gm/cc) Wet	Final Density (gm/cc) Dry	Final Height / Initial Height	Final Void Ratio	Change in Dry Density %	Total Energy Consumed (watt-hours)	Average Shear Strength (psf)
C22TO	0	1.38	0.58	1.56	0.98	0.61	1.66	69	0	430
	½	1.36	0.57	1.66	1.11	0.53	1.36	95	43	1130
	1	1.33	0.56	1.65	1.15	0.49	1.21	105	154	2010
C27MM	0	1.31	0.54	1.53	0.94	0.58	1.81	74	0	880
	½	1.33	0.55	1.54	1.00	0.54	1.60	82	26	1710
	1	1.36	0.56	1.53	1.04	0.54	1.35	86	97	3470
D1DM	0	1.43	0.72	1.74	1.15	0.66	1.43	60	0	530
	½	1.44	0.72	1.60	1.10	0.64	1.28	53	86	1400
	1	1.47	0.74	1.67	1.16	0.64	1.23	57	114	1610
D4CO	0	1.40	0.65	1.71	1.14	0.57	1.53	75	0	560
	½	1.42	0.65	1.73	1.24	0.53	1.28	91	47	2650
	1	1.41	0.65	1.69	1.23	0.51	1.26	89	115	2470

considerably by electrical treatment in all the dredging samples, the sensitivity varied little between the untreated and treated samples. Table 7 lists several of the more important properties of the dredged materials, and specific values are tabulated in order to enable the effects of the electrical treatment to be better evaluated. The greatest percentage change in dry density occurred in Sample C22TO, but somewhat lesser changes were observed for the other materials. A large portion of the percentage change in dry density resulted from the consolidating pressure (0.6 kg/cm^2) alone and not the electrical gradient. This table also lists the total energy consumed by the various samples, and a general comparison of its effect on the increased density and shear strength can be made.

From the flow efficiencies presented in Table 4, the economics of electro-osmosis can be evaluated in a very general way; for example, the volume of water per unit volume of dredged material per watt-hour of power is conveniently tabulated. However, it must be appreciated that the foregoing interpretations are based solely on a series of laboratory tests, and, before applying this information to an actual field problem, it would be desirable to evaluate the power requirements which occur in a comparable large-scale treatment process and the scaling factors associated with extrapolating these data to a field situation. Despite the technical feasibility of dewatering and stabilizing dredged materials by electro-osmosis, its use for the large volumes of dredged material that must be handled would likely be economically prohibitive (based on the flow efficiencies given in Table 4) in all except very unusual situations. An added difficulty arises from the fact that many disposal sites are located in rather remote areas where electrical power may be expensive to obtain. Nevertheless, it is possible that the use of smaller electrical gradients (for which no data are currently available) may offer some potential for useful application.

CONCLUSIONS

Based on the interpretation of experimental data accumulated from laboratory tests on four samples of typical polluted dredgings, the following conclusions can be advanced:

1. Electro-osmosis serves to significantly increase the volume of fluid discharged from the dredgings and to accelerate the rate of dewatering, with the associated decreases in water content and void ratio and increase in shear strength.

2. Values for the electro-osmotic coefficient of permeability for polluted dredgings are generally of the same order of magnitude (but less) as those which characterize a large variety of soils, that is, 5×10^{-5} cm/sec per volt/cm.

3. The Atterberg limits and the conventional consolidation characteristics are not significantly affected by the electrical treatment.

4. The electro-osmotic process carries substantial amounts of soluble ions out of the dredged material with the effluent.

5. The chemistry of the sediment and the dewatering process are influenced to some degree by the generation and release of gases.

6. The major effect of the electrical treatment occurs soon after the current is applied, but this response varies with the material.

7. Although technically feasible, the use of electro-osmosis to dewater and stabilize landfills of dredged materials does not seem to be economically justified except in very unusual cases; the use of electrical gradients smaller than those studied in this experimental program may offer some promise, but supporting data are needed before definitive evaluations can be made.

ACKNOWLEDGMENT

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