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The Dry DREdge™ (photo courtesy of DRE Technologies)

IN THIS ISSUE

AIMS & SCOPE OF THE JOURNAL

The *Journal of Dredging* is published by the Western Dredging Association (WEDA) to provide dissemination of technical and project information on dredging engineering topics. The peer-reviewed papers in this practice-oriented journal will present engineering solutions to dredging and placement problems, which are not normally available from traditional journals. Topics of interest include, but are not limited to, dredging techniques, hydrographic surveys, dredge automation, dredge safety, instrumentation, design aspects of dredging projects, dredged material placement, environmental and beneficial uses, contaminated sediments, litigation, economic aspects and case studies.

MARINE GEOTECHNICAL ASPECTS OF POPLAR ISLAND RESTORATION

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ABSTRACT

Dredged material placement is a constant challenge in major ports around the country. At most ports this dredged material must be placed in a containment facility. The Poplar Island project involves restoration of an eroded island using dredged material removed from the southern Chesapeake Bay approach channels of the Port of Baltimore. A marine geotechnical investigation was conducted to develop the site layout for Poplar Island. Results of the investigation revealed many geotechnical aspects including soft foundation and slope stability. The geotechnical aspects associated with the design and construction of the Poplar Island dredged material placement site are presented in this paper.

INTRODUCTION

The Chesapeake Bay is made up of navigable channels that are important for the economy of Maryland and surrounding areas. These channels shoal in continuously, resulting in a need for maintenance dredging projects. The United States Army Corps of Engineers (USACE) is responsible for the maintenance and operation of these channels. The Maryland Port Administration (MPA) is responsible for providing the placement sites for the dredged material. Typically, new work projects are funded through the Water Resources Development Act with the cost shared 75% by the federal government (USACE) and 25% by the local agencies (MPA). When these channels are dredged, the dredged material is currently being placed in Hart Miller Island (HMI) diked placement site. HMI will be reaching its maximum capacity soon and will be converted to a wildlife habitat, thus presenting the need for the design and construction for the new diked placement site. After many preliminary site investigations of the upper Chesapeake Bay, Poplar Island became the current option for the future placement site.

Poplar Island is located in the Upper Chesapeake Bay, south of Kent Island and northwest of Tilghman Island in Talbot County as shown in Figure 1. Poplar Island is currently composed of four small islands covering approximately 5 acres. In 1847, Poplar Island was over 1000 acres, but due to severe erosion, less than 1 percent of the original island exists today. The objectives in the design of the diked placement site include: the restoration of Poplar Island to its 1847 footprint, creation/restoration of habitat area for the Chesapeake Bay, optimization of capacity of Poplar Island for dredged material, and cost efficiency and environmental acceptability of the project (GBA and M&N, 1995). The site layout of Poplar Island will approximately follow the

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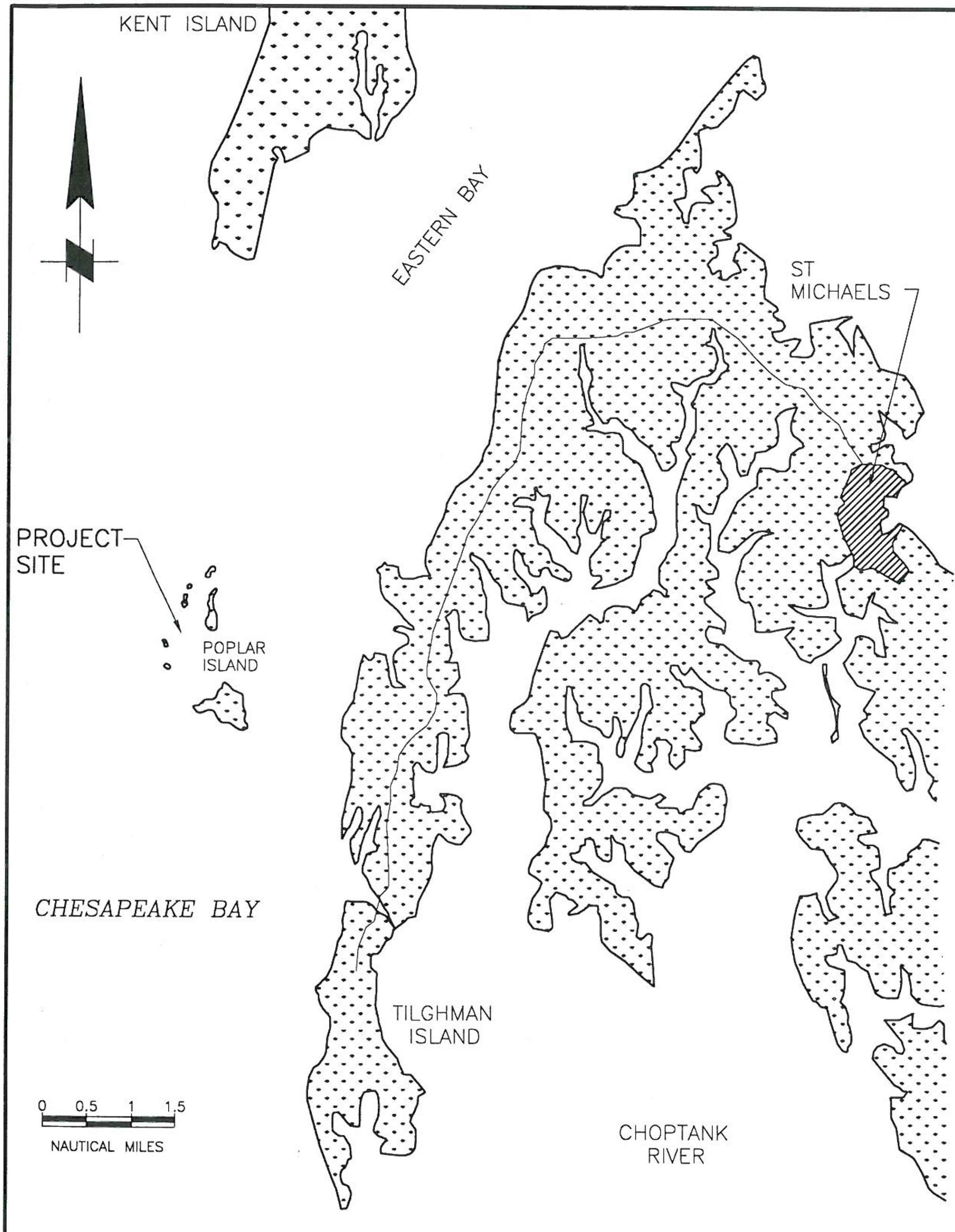

Figure 1. Poplar Island Restoration – Site Vicinity Map

1847 island footprint (MES, 1994). The containment structure will include an upland wildlife habitat and a tidal wetland, both of which will be filled with dredged material.

There are many geotechnical issues in the design, construction, and site layout of this dredged material placement site. An evaluation of the site soils, determination of borrow areas soils, settlement analysis and slope stability analysis for several types of dikes and dike alignments are the main issues that were encountered in the preliminary stages. Many marine geotechnical investigations and studies were conducted to determine the best design. Some marine geotechnical issues that will be reviewed in this paper are the testing and site analysis of the proposed area, borrow area analysis, dike design, dike fill material, and unsuitable backfill.

SITE LAYOUT & TESTING

Site conditions are very important to the initial site layout. Key conditions include bathymetry and topography, wind conditions, water levels, wave conditions, water currents, and current site soil characteristics. With the previously stated conditions noted, alternative dike footprints can be established. From these footprints, initial geotechnical investigations can be done to gather more information in order to determine several proposed alignments.

Poplar Island is currently made up of four separate islands that cover 5 acres (North Point Island, Middle Poplar Island, South Central Poplar Island and South Poplar Island) as shown in Figure 2. The dike alignment denotes the footprint for the placement sites, and is also the principal initial cost of the project. Key factors for alternative footprint analysis were the following:

- Bathymetry (deep water to the north and south)
- Soil conditions (soft material in the northern area)
- A desire to approximate the area of the 1847 footprint (1000 acres)
- Presence of oyster bars (east and west of project site)
- Poplar Harbor located on the eastern portion of the site
- The footprint of Poplar Island to be connected to Coach's Island
- Impact of the proposed island on adjacent islands
- Relationship between the site development costs and the capacity for placement of dredged material

Poplar Island is designed to have a capacity of approximately 44 million cubic yards (mcy) of dredged material. It will take twenty years to place 44 mcy in the site, but with proper site management and aggressive dewatering techniques, the island can possibly last longer. Poplar Island will be split into two different types of cells: upland cells and wetland cells (Figure 2). For construction funding Poplar Island was also split into two different areas, Phase I and Phase II. This split was to more effectively construct the island and to provide more opportunity for several construction companies to bid for the contract. Most of the capacity of the island is in

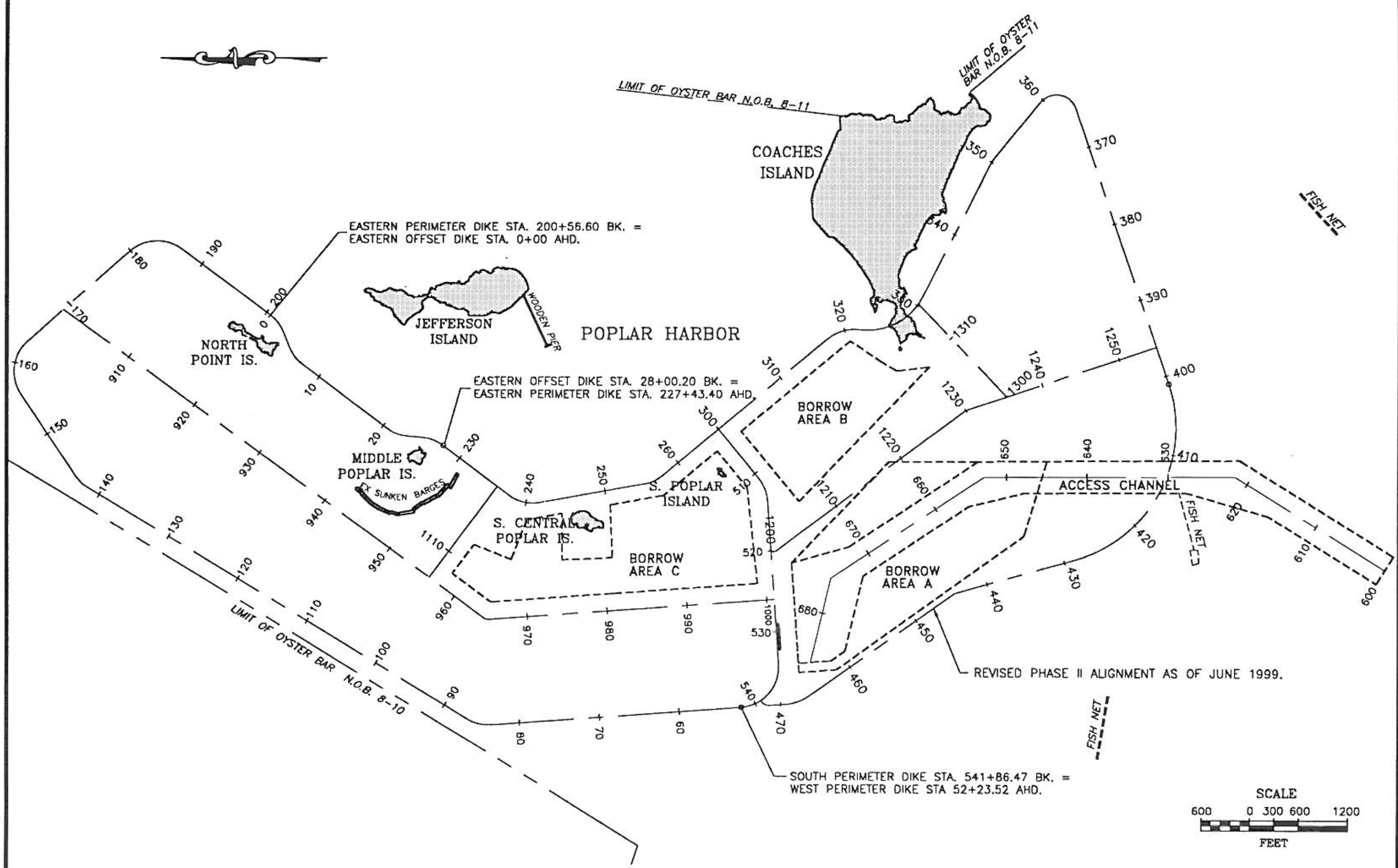

Figure 2. Poplar Island Restoration – Site Layout

the two upland cells on the west side. This is the one of the reasons that the west dike has to be designed stronger than the east dike.

After the dike alignment was established, an intensive geotechnical investigation by Earth Engineering & Sciences, Inc. was initiated. (E2Si, 1995) This investigation began by obtaining 81 borings that were taken around the dike alignment in 500 ft spacing, and in potential borrow area sources. The boring spacing was reduced when soft material was encountered, and the depth of each boring varied between 20 ft to 50 ft. All borings were drilled using truck-mounted drill rigs and advanced using hollow-stem augers. Standard penetration tests were conducted and split spoon samples were obtained in every boring at depth intervals of 2.5 ft in the upper 10 ft and at a depth interval of 5 ft thereafter.

The prefeasibility study that was conducted prior to this geotechnical investigation suggested that the dike would be built of clay (due to the perceived shortage of quality sands at the site). But during this investigation, it became clear that there was enough sand to use in the design of the dikes. So bulk samples of each type of material were taken and sent to the laboratory. Vane shear tests and standard penetration resistance tests were correlated and conducted in cohesive soils. E2Si (1995) conducted these tests in accordance with ASTM D-2573. The shear strengths obtained from the vane shear tests are shown in Table 1.

Table 1. Summary of Vane Shear Data (E2Si, 1995)

Boring Number	Depth (ft)	Soil Type	Standard Penetration Number	Undrained Shear Strength (Undisturbed) (psf)	Undrained Shear Strength (Remolded) (psf)
B-14A	4			230	70
	4-5.5	Gray silty CLAY	1		
	8			750	220
	8-9.5	Gray silty CLAY	4		
	15			1100	700
	15-16.5	Brown silty fine SAND	12		
B-71A	4			400	180
	4-5.5	Brown gray silty CLAY	3		
	8			1030	230
	8-9.5	Brown gray silty CLAY	7		
	15			680	450
	15-16.5	Brown gray silty CLAY	5		

Laboratory tests were used to evaluate the geotechnical properties of the foundation soils and potential borrow area soils. These tests consisted of the following:

- Visual inspection of every sample
- Water content tests on every sample of silty clay

- Atterberg Limit tests on cohesive soils
- Sieve analysis on potential borrow area sites
- Consolidation tests on foundation clays to evaluate their stress history and their consolidation and settlement characteristics
- Unconfined compression tests on shelby tube samples to evaluate shear strength

In addition, special laboratory model tests were conducted for the following: (1) to evaluate the shear strength of mechanically dredged and placed clay; (2) to evaluate the angle of internal friction, (ϕ), of the sand excavated from the borrow area; and (3) to evaluate the slope obtained by hydraulically placing sand above and below water. Details of these test procedures are available from E2Si (1994), Das (1994), and Hunt (1984).

For the design of the dike, two parameters are important: the stress history and shear strength of the soil. At this site, the preconsolidation stress greatly exceeds the effective overburden stress. Therefore, the soils are preconsolidated with overconsolidation ratios (OCR) varying from 7 to 20. This preconsolidation is probably due to the stress from the old island along the design footprint. Shear strength of the clays in the foundation was evaluated by: (1) in-situ vane shear tests; (2) electric cone penetrometer; (3) unconfined compression strength based on laboratory tests on shelby tube samples; (4) published correlation between standard penetration resistance and shear strengths; (5) published correlation between liquid limit and the ratio of undrained strength to effective overburden pressure for over consolidated soils. The silty clay that underlies the entire project site has a cohesion of at least 400 psf. However, it was found that the recent, normally consolidated deposits had much lower shear strength.

CONTAINMENT STRUCTURE & BORROW AREA

The containment structure represents the principal of the cost of the project. Hence, containment structures are typically constructed with locally available material. Three types of dikes were considered for this project: clay dikes; sand-filled geotextile tubes and sand dikes.

Clay Dike

There was no literature on case histories regarding construction of an underwater mechanically or hydraulically dredged clay dike. Several engineers from the Corps of Engineers indicated that they had constructed clay dikes under water, by "end dumping" the clay, but not by hydraulic or mechanical dredging. The critical factor, relating to clay dikes, is the shear strength of the clay fill in the dike. The laboratory tests on the Shelby tubes that were collected during the geotechnical investigation indicated a very low shear strength (100 psf). Scaled laboratory model tests indicated that the cohesion was around 140 psf. Because of the lack of experience with building clay dikes under water using mechanical or hydraulic dredging, and the difficulty in establishing a reliable design shear strength for the clay dike; the clay dike concept was terminated.

Sand-Filled Geotextile Tubes

The key factors favoring for the geotextile tubes are the significant cost savings and ease of construction. A geotextile tube is one or two layers of geotextile, shaped like a "sock", that is laid on the ground and is pumped full with a sand-water mixture. The water will seep through the geotextile resulting in an elliptical sand dike. Geotextile tubes have been used in water depths of up to 2 ft. Some problems that were recorded were rolling of the geotextile tubes, splitting of the tubes at the seams, and consolidation and settlement of the fine grained material to fill the tube. A test section of the geotextile tubes were incorporated in the design of the southwest dike alignment. However, the project contractor could not meet the specification and the schedule for the geotextile tube, so the concept was scrubbed for the typical dike section.

Sand Dikes

The sands at the site are non-plastic silty fine sands, and are available in adequate quantities. The successful completion and continued use of Hart Miller Island, proves the effectiveness of sand dikes built in the Chesapeake Bay. Table 2 presents a comparison of sand between Hart Miller Island and Poplar Island. With the abundant quantity of sand available, and the low turbidity associated with hydraulically placed sand, sand filled dikes were considered the most practical method for the construction of the containment dike. However, the Phase I contractor for Poplar Island chose to construct the containment dikes mechanically with sand.

Table 2. Comparison of Hart Miller Island and Poplar Island Sand (E2Si, 1995)

Description	Poplar Island Sand	Hart Miller Island Sand
1 – *Type of Sand* • Coarse • Medium • Fine	 1% 4% 78%	Silty fine to coarse Sand With gravel and cobble, Pockets of Clay
2 – *Angle of Internal Friction* • Below Water • Above Water	 28 32	 27 32
3 – *Percentage of Fines*	17%	Highly Variable
4 - *Plasticity*	Non-Plastic	Non-Plastic
5 – *Shape*	Angular to Sub-Angular	Angular to Sub-Angular

Borings drilled and cone penetrometer tests conducted in the potential borrow areas indicated that there was an adequate amount of sand (approximately 10,000,000 cubic yards) in the south portion of the project site (see Figure 2). Unsuitable overburden will be stripped and deposited in the north portion of the island (unsuitable dump area). Boring analysis revealed that the sand beds might contain layers and pockets of clay. There is evidence of localized pockets of "ironite" (iron-cemented sands), but are located well outside of the proposed borrow areas. Four borrow areas (Borrow Area A, Borrow Area B, Borrow Area C, entrance channel) were chosen as potential sources for construction, as shown in Figure 2.

DIKE DESIGN

As discussed before, the dike design was based on using sand. The critical factors in the design of sand dikes are: (1) angle of internal friction (ϕ) of the sand of the dike, especially below the water; (2) factor of safety; (3) and dike geometry

The angle of internal friction (ϕ) of hydraulically placed sand in the dike was evaluated based on laboratory model tests and field experience at Hart Miller Island. Direct shear tests on the fine sand from the borrow area reveal a ϕ of 35°. It is recognized during construction that the sand will have more fines, thus ϕ. will probably be lower. E2Si (1994) conducted a scaled model experiment, where the slope of the sand under water was determined to be 2H:1V. A ϕ of 27° corresponds to that slope. At Hart Miller Island, the values were $\phi = 30°$ below water and $\phi = 35°$ above water. Based on the above factors, ϕ for the sand dike fill was assumed to be 28° below water and 32° above water. For this design a factor of safety (FS) of 1.3 was selected as acceptable for long term stability considerations based on discussions with the USACE. A factor of safety of 1.2 was used as the lowest acceptable for short-term stability considerations.

The dike section has been separated into several types of dikes, as shown on Figure 3. Material used in the construction of the dikes are: geotextile; fill (sand); slope armor stone (3000 pound and 4000 pound); toe armor stone (1500 pound and 2000 pound); toe quarry run; slope underlayer (250 pound stone; and 3 & 6 inches). The proposed west perimeter dike will consist of a toe dike constructed of either 2000 lb. or 1500 lb. armor stone on top of quarry run on a slope of 2H:1V slope, to elevation 0 ft. Sand will then be mechanically placed against the toe dike, at a slope of 4H:1V to maximum elevation 10 ft. A single layer of geotextile is placed on top of the fill before the underlayer is applied. A 6-inch layer of 3's & 6's are placed on the sand fill slope with a 2 ft layer of 250 lb. stone on top of that. The final outer protection of the perimeter dike is two layers of either 3000 lb. or 4000lb. armor stone. East perimeter dike is essentially the same design, but not as strong as the west dike due to expected lesser wave forces. The slope armor stone that is placed onto the sand fill is a layer of 250 lb. stone, 2.5 ft thick. The coastal analysis (GBA and M&N, 1994) revealed that the northwest, west, and southwest sections of the island are going to experience the most amount of weather and wave actions toward it.

The dike was originally designed as a sand dike that will be constructed by hydraulic dredging and placement of the sand. The dike was designed to be stable against two types of failure. (1) shallow failure through the sand dike, and (2) deep-seated failure through the foundation. The above analyses were conducted using Purdue University's PC STABL 5 slope stability computer program (E2Si, 1994).

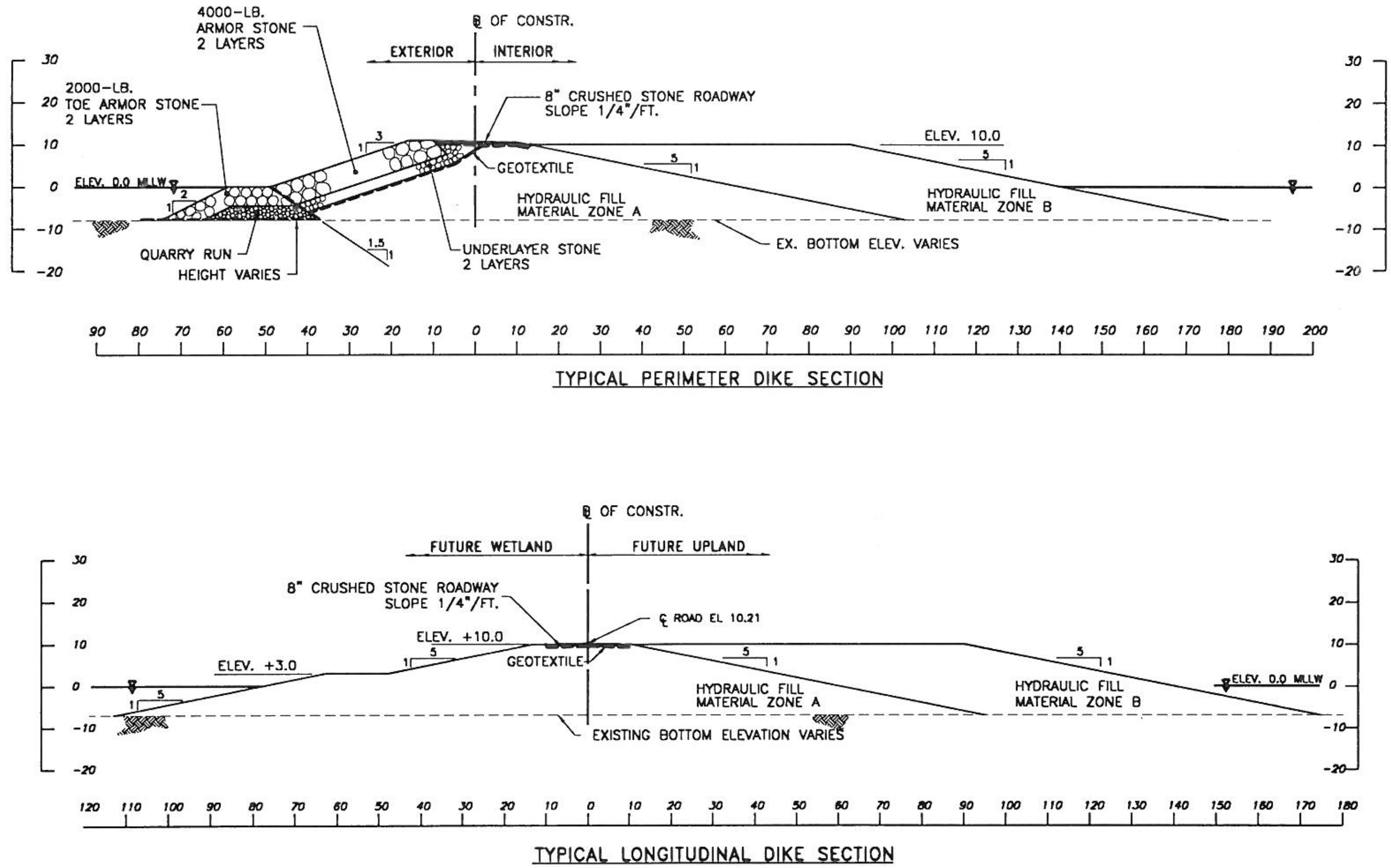

Figure 3. Poplar Island Restoration – Typical Dike Cross Sections

Two types of settlement were considered in the dike design: settlement within the dike itself, and settlement of the foundation soil under the weight of the dike. The foreseen settlement of the dike was estimated by using a Compression Index of 0.035 for *silty sands* and an Initial Void Ratio of 0.85. The settlement of a 20 ft dike is anticipated to be approximately 2 to 3 inches. Over most of the dike alignment, which is composed of non-plastic silty sands, the settlement will occur very rapidly and will probably already have occurred before the slope armor stone is placed. Therefore, the settlement in the sand foundation will have no impact on the long-term performance of the dike. The light and dark gray silty clays along certain portions of the dike are preconsolidated, with preconsolidation pressure being about 2 tsf to 3 tsf. The maximum stress from the dike is estimated to be 1800 psf. Also, the normally consolidated clays will have far greater settlements than the preconsolidated clays. These normally consolidated clays will be undercut and replaced with sands removed from the localized borrow sources. The resulting settlements are not considered to be significant, due to the lack of structures and due to the fact that the roadway on top will not have a rigid pavement.

UNSUITABLE BACKFILL

Subsurface conditions along the dike alignment varied considerably. Conditions include very soft silty clay, very loose silty sands, stiff silty clays, and medium dense silty sands at shallow depths. The design of dike stability is based on cohesion of 400 psf in the silty clays or an internal friction angle of 22° along the silty sands, in the dike foundation. It is critical that the shear strength in the foundation not be less than the strength assumed in the design. There are areas along the dike alignment where the strength is less than 400 psf, however, these areas will be backfilled with sand.

One way to attack soft material is the dike design that would be overbuilt by 3 feet in height, while maintaining the design slope. Construction would be carried out such that the soft soils would not be trapped underneath the dike. The top of the front face of the dike would always be kept at the design elevation of plus 3 ft, and the slope of the front face of the dike would be 4H:1V or flatter. The side slope would be built to design. If the foundation soils are soft, the soil will be displaced to the side and not to the front of the dike. The overbuilt dike is expected to slump causing soft material to build up on the sides, therefore improving stability. If the soft materials were absent, the dike would not slump. Four days after the initial construction, the dike can be reshaped to the design elevation. This approach eliminates the need for undercutting. One problem with this concept was the payment method and inability to precisely calculate quantity of material displaced and backfilled. Undercutting was therefore chosen along the dike alignment.

Undercut control criterion is needed to determine the extent of undercutting during construction. Two approaches were considered for establishing the undercut criterion: standard test borings and cone penetration tests (CPT). The CPT was recommended to determine the extent of undercutting. Some pros and cons of the CPT are: it is not operator dependant; the quasi-static test causes minimal remolding; it yields strength directly; it needs a grab sample to identify sands and clays at the surface; and it yields strength data every two inches with depth. When a grab sample is applied to determine if the sample is clay or sand, only the soft clay should be undercut. (E2Si, 1994)

Cone penetration tests was recommended to be conducted at 200 ft intervals, for a minimum distance of 2500 ft ahead of the working face of the dike. Stability analyses have also indicated that very loose sands can be left in place, and need not be undercut. Estimated volume of undercut is approximately 500,000 cubic yards.

SUMMARY AND CONCLUSIONS

The key geotechnical issues involved in the site layout of a placement site were discussed in this paper using Poplar Island as an example. A geophysical survey is needed to characterize the existing site conditions. Existing information should be reviewed to determine any prior problems related to the project site. After the environmental issues have been addressed, site specific surveys completed, and all marine geotechnical properties evaluated, the design of the project site layout can begin.

There are many issues that have to be researched in order to design an effective dike. Unsuitable cutting of soft material is very important when dealing with the dike stability. Other stability problems deal with settlement within the dike and settlement of the foundation soil under the weight of the dike. The amount of material available at the site to complete construction of the dike is also important in the design.

The containment dike design is a function of several important factors. The angle of internal friction, the factor of safety, and the dike geometry are critical when designing the dike. For Poplar Island, it has already been determined that the dike fill will be composed of silty sand that is available at the site. The angle of internal friction has been determined to be close to the angle of internal friction at Hart Miller Island, which happens to be a successful diked placement site in the Chesapeake Bay. A factor of safety was developed for the long term and short-term stability considerations. The dike geometry is very important to provide protection against weather, to contain the dredged material, and from an economical viewpoint. The dike geometry at Poplar Island was split into an east exterior dike and a west exterior dike, based on coastal factors. The west dike is designed stronger than the east dike, due to the weather and the waves that annually pound the west side of the island.

Poplar Island was split into two different types of cells: wetland cells, which will have an average elevation of plus 1 ft, and upland cells, which will have an average elevation of plus 20 ft. Poplar Island's capacity will be approximately 44 million cubic yards depending on site management. Upland cells will hold the bulk of the dredged material.

Currently, the contractor is nearing completion of Phase I of Poplar Island (GBA, 1999). Several cells are completely enclosed and are being dewatered to below bay-elevations using numerous pumps. The U. S. Army Corps of Engineers is expected to release the bid documents for Phase II in the near future.

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DREDGING AND DEWATERING OF HAZARDOUS IMPOUNDMENT SEDIMENT USING THE DRY DREDGE™ AND GEOTUBES

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ABSTRACT

The purpose of this paper is to describe the application of an innovative dredging technology coupled with geotubes in the dredging and dewatering of hazardous sediments. The paper describes the project objectives, description of the dredge and geotube technologies, and the results of applying this technique. The Dry DREdge™ was jointly developed and tested by DRE Technologies and the U.S Army Waterways Experiment Station (WES), under the Corps of Engineers Construction Productivity Research Program (CPAR).

BACKGROUND

Ashland Inc. has operated a hazardous waste landfill as part of its refinery operations in Catlettsburg, Kentucky since 1976. The landfill is located in Boyd County, Kentucky, approximately 3 miles south of Catlettsburg, Kentucky. In September 1998, the Kentucky Division of Waste Management was notified that the 20-acre, head of hollow, single cell landfill would be closed by December 1999. Approximately 1.1 million cubic yards of petroleum refinery waste had been landfilled at the site during the past 22 years.

As part of the landfill operation, a wastewater treatment unit was constructed to control surface water discharges. The purpose of the wastewater treatment unit was to collect and treat surface water runoff and leachate that was generated from the landfill during operations. The wastewater treatment unit consisted of a concrete sedimentation basin and water treatment process, which involved chemical precipitation, ozonation and granular activated carbon processes. The water discharge from the wastewater treatment unit was discharged to a nearby creek and was monitored under a Kentucky Department of Environmental Safety (KYDES) permit.

As part of landfill closure, the Kentucky Department of Environmental Protection (KYDEP) requested that all sediments from the concrete basin be removed. In April 1999, it was estimated that approximately 5,000 cubic yards of sediment was contained in the basin. Since the sediment was collected from a hazardous waste landfill, the material was considered to be a listed waste. Analytical testing indicated the principal chemical constituents were semi-volatile organic compounds (i.e. phenanthrene, chrysene, and naphthalene). In April 1999, the KYDEP indicated that it would be feasible to dispose of the sediment from the basin in a local landfill during closure and prior to final capping. This option provided a cost-effective alternative to off-site

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disposal. The only requirements that KYDEP required for disposal was the material needed to pass the paint filter test and no free liquids could remain.

Since the KYDEP approved disposal of the sediment from the basin into the local landfill, it was necessary to evaluate several sediment removal alternatives. Management of contaminated surface water and controlling discharge from the 40-acre watershed during sediment removal was one of the principal factors in evaluating sediment removal technologies. This was significant because runoff could not be diverted around the basin during removal and access to the basin was limited because it was considered a confined space.

As a result of the evaluation process, the project team selected The Dry DREdge™ technology combined with in-place geotube dewatering of the wet sediment as the preferred method.

MATERIAL PROPERTIES

Three composite samples were obtained from the basin for geotechnical testing. Particle size distribution and hydrometer tests were conducted to characterize the dredge materials. Plots of these data are shown in Figure 1. Other geotechnical tests conducted included Atterberg Limits (liquid limit and plastic limit), natural water content, specific gravity, and geotechnical description. Results of these tests are shown tabulated in Table 1. From these test results, the void ratio and the saturated wet unit weight were computed.

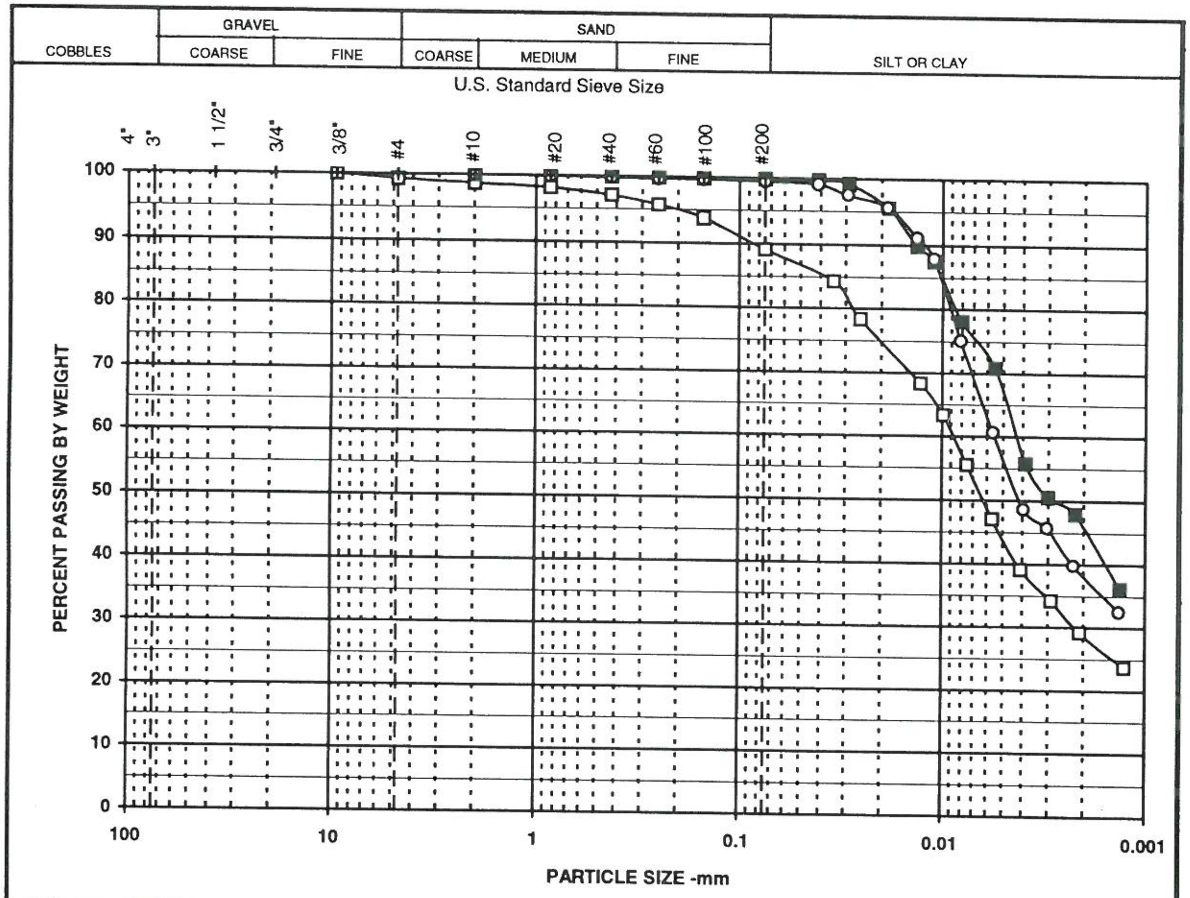
The dredged materials were classified as fine-grained dark gray plastic clay (CH to CL) with a trace of sand. Particle size distribution testing showed that composite sample 1 had 90 percent passing a 200 sieve and samples 2 and 3 showed that 99 to 100 percent passing the 200 sieve.

Atterberg limit tests indicated that the dredged material had liquid limits ranging from 45 to 60 and plastic limits ranging from 22 to 25 with the plasticity index varying from 23 to 35. The specific gravity of the soil material varied from 2.75 to 2.78. The natural water content ranged from 64 to 104 percent with the void ratio ranging from 1.76 to 2.89. The saturated wet unit weights for composite samples 1, 2 and 3 were 1.28, 1.5 and 1.46 gm/cc respectively.

The dredged material exhibited water content values greater than the liquid limit indicating that the material would act as a fluid mud. The dredged material was very soft in consistency and exhibited very low shear strength. When the fine-grained dredged material was clam-shelled from the sedimentation basin and placed into the positive displacement pump hopper, it flowed to the bottom of the hopper.

DREDGE DESCRIPTION

Conventional excavation methods, such as, hydraulic dredging and mechanical dredging with clamshells or draglines typically suffer from several limitations. These include resuspension of



SYMBOL	DESCRIPTION AND REMARKS
□	dark gray plastic CLAY, trace sand
■	dark gray plastic CLAY
○	dark gray plastic CLAY

Symbol	□	■	○
Boring			
Sample	Composite 1	Composite 2	Composite 3
Spec			
Depth			
% +3"			
% Gravel	0.7		
% SAND	10.2	0.2	0.5
% FINES	89.1	99.8	99.5
% -2μ	29	45	38
Cc			
Cu			
LL	45	57	60
PL	22	24	25
PI	23	33	35
USCS	CL	CH	CH
w (%)	64.1	91.8	104.4

Particle Size (Sieve #)	PERCENT FINER □	PERCENT FINER ■	PERCENT FINER ○
4"			
3"			
1 1/2"			
3/4"			
3/8"	100.0		
4	99.3		
10	98.7	100.0	100.0
20	98.2	100.0	99.9
40	97.2	100.0	99.8
60	95.7	99.9	99.7
100	93.8	99.9	99.7
200	89.1	99.8	99.5

Figure 1. Particle Size Distribution Curve

Table 1. Laboratory Testing Assignment and Data Summary

BORING/ SAMPLE NO.	IDENTIFICATION TESTS								
	WATER CONTENT w (%)	LIQUID LIMIT LL (%)	PLASTIC LIMIT PL (%)	PLASTICITY INDEX PI (%)	USCS SYMBOL (1)	SIEVE MINUS NO. 200 (%)	HYDROMETER % MINUS 2 μm (%)	TOTAL UNIT WEIGHT (pcf)	SPECIFIC GRAVITY G
Composite 1	64.1	45	22	23	CL	89.1	29	101.9	2.750
Composite 2	91.8	57	24	33	CH	99.8	45	93.5	2.766
Composite 3	104.4	60	25	35	CH	99.5	38	90.8	2.776
	Void Ratio e=wxG	Void Ratio at LL	In Situ Density gm/cc		Volume Reduction to LL	Liquidity Index LI	Activity Ratio A		Ratio w/LL
Composite 1	1.76	1.237	1.63		0.81	1.8	0.25		1.42
Composite 2	2.54	1.577	1.50		0.73	2.1	0.24		1.61
Composite 3	2.90	1.666	1.46		0.68	2.3	0.25		1.74

Note:
(1) Plasticity of fines for USCS Symbol is based on visual observation unless Atterberg limits are reported.

sediments at the point of excavation, imprecise excavation of "hot spots", and free water entrainment in sediments requiring expensive dewatering and return water treatment.

The Dry DREdge™ incorporates a specially designed, sealed clamshell mounted on a rigid, extensible boom (see Figure 2). The open clamshell is hydraulically driven into the sediments at low speed, minimizing sediment disturbance and resuspension. The clamshell is then hydraulically closed and sealed, excavating a plug of sediment at its *in-situ* moisture content.

Figure 2. The Dry DREdge™

The sediment is deposited in the hopper of a positive displacement pump. Depending on the application, the hopper can be equipped for debris screening, size reduction, vapor emission control, sediment homogenization, and blending of additives to modify flow properties or stabilize contaminants. The sediment is pumped in a plastic flow regime (see Figure 3) through a pipeline to its appropriate disposition. The discharge has the consistency of toothpaste. Depending on the in-situ moisture content and degree of hazard posed by the sediment, the disposition may be direct feed to a dewatering process, thermal treatment or stabilization process, direct feed to on-site land disposal, or direct feed to a transport vehicle.

Figure 3. Consistency of Dredged Material

The most unique advantage of this dredge is its ability to deliver sediments at high solids concentration corresponding to the *in-situ* moisture content. High solids content sediment delivery can offer major economic advantages through the reduction or elimination of dewatering and return water treatment. Solids concentrations up to 70% by weight have been pumped by this dredge (Parchure et al., 1997). Other advantages include the following:

- Excavation is accurate and precise. The azimuth, declination, and extension of the clamshell is electronically displayed in the operator's cabin and available for electronic input to a programmable controller. Therefore, the extent of the excavation (length, width, and depth) is easily controlled by the operator. The programmable controller can be configured to completely excavate the area within range of the dredge by systematically making a grab, depositing the material in the pump hopper, and returning to make another grab immediately adjacent to, or overlapping, the last grab.

- The clamshell-boom configuration allows the dredge to work around rocks and pilings. It is not limited to rectangular excavation patterns as are horizontal auger dredges, or the inverted cone excavation patterns of rotating basket dredges. These excavation capabilities are ideal for "hot spot" remediation.

• Excavation is achieved with minimal resuspension of sediments. Hydraulic dredge cutter heads agitate the sediments in the vicinity of the pump suction. Conventional clamshells are allowed to free-fall in order to impact the bottom with enough force to penetrate. Draglines are pulled randomly through the sediments. All these operations disturb the surrounding sediments, resuspending particles and contaminants. Resuspension is a major concern when dredging is conducted in bodies of flowing water such as estuaries. The dredge is intrinsically sound for debris management. Unlike hydraulic dredges, the pump suction is above surface allowing visual inspection of debris by the operator. Debris can be removed or shredded and pumped. The decision-making capability is critical for certain types of debris.

GEOTUBE DESIGN

The design requirements were for a geotube that had a circumference of 90 ft, a height of 5 ft and a length of 160 ft. A maximum wet bulk density of 1.6 gm/cc was used in the design analysis. A factor of safety of 5.0 was used in the design, which included factors of safety of 2.0 for seams, 1.5 for creep and 1.5 for biological degradation. A cross-sectional view for the geotube design for this project is shown in Figure 4. This geotube design was determined using the computer program, "Geosynthetics Applications Program," (Palmerton,1998). This program assumes that

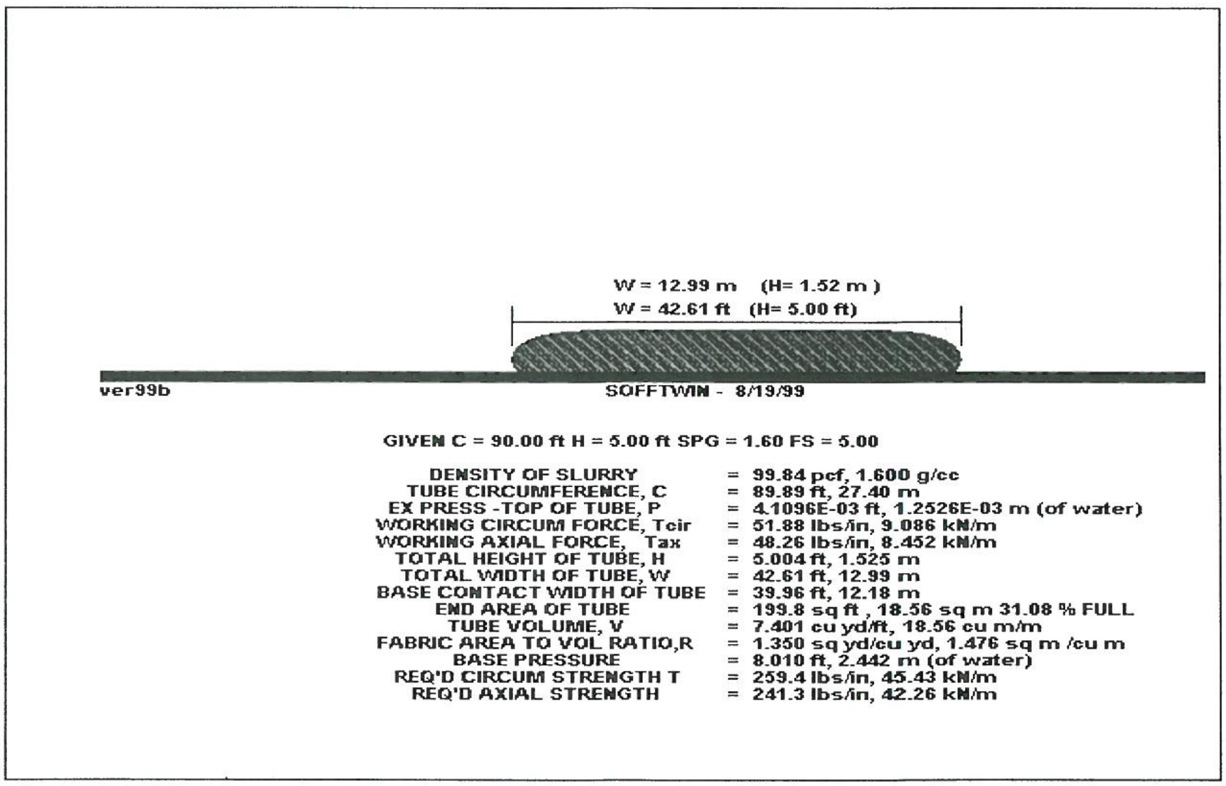


Figure 4. Cross Sectional View of Geotube

the geotube is filled with a fluid and does not have any shear strength. The ultimate strength of the geotube is directly dependent on the available wide tensile strength of the seams. Since the seam strength available is 300 pounds per inch width (pli) and the required seam strength is 259.4 pli from the design analysis then the geotextile fabric selected is satisfactory.

GEOTUBE CONSTRUCTION

This project consisted of three 90-ft circumference geotubes, 160 ft long constructed from 15 ft wide panels of a woven polypropylene fabric. The woven geotextile fabric had an ultimate breaking strength in the warp of 400 pounds per linear inch width (pli) and in the weft directions of 550 pli and the seam strength was 300 pli at 10 percent elongation respectively for both the warp and weft (ASTM, 1999). The Area Opening Size (AOS) for the geotube fabric, which is also equivalent to US Standard sieve size number, was about 50 (ASTM, 1999).

The geotubes were manufactured by the TC Mirafi Corporation and shipped to the project site in a protective covering. Two rows of inlet ports with 1.5-ft diameter, 5-ft long sleeves were provided every 25-ft along the top of the geotube. Nylon anchor straps sewn to the geotube perimeter every 10 ft that were used to secure the geotube prior to and during filling. A 16-oz per square yard non-woven polypropylene fabric was placed beneath the geotubes to facilitate vertical and lateral drainage during consolidation of the dredged material in the geotube.

A very small amount of fines, less than 5 to 10 mg/liter, were evident in the decant water passing through the geotube during the initial filling but this water became very clear as the geotube was filled to the design height of 5 ft. The decant water looked to have a very light tan to clear color and it was felt to be a insignificant loss of dredged material.

The 15-ft panels were sewn perpendicular to the longitudinal axis of the geotube. All factory seams were sewn with double stitched butterfly seams. All seams consisted of type 401 double lock stitch that was sewn with a double needle Union Special Model #80200 sewing machine. The machine is capable of sewing two parallel rows of stitching about one quarter inch apart. The thread was a 2 ply 1000 denier passing through the needles and 9 ply 1000 denier passing through the looper.

CONCLUSIONS

The project was started in April 1999 and completed in June 1999. Approximately 5,000 cubic yards of material was dredged from the sediment basin and sequentially pumped directly into five geotextile tubes located on the side of a mountain. Filtrate was routed from each dewatering pad to the existing runoff collection system and returned to the basin. Random sampling of collected sediment indicated the majority of the material would pass the paint filter test within 7 days. Limited measurements indicated a free water loss of approximately 20 percent. Observation would indicate the bulk of this water is interstitial. Thus, the use of the

Dry DREdge™, geotube technology, and onsite disposal resulted in cost savings of approximately $1.0 million dollars.

ACKNOWLEDGEMENTS

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GENERAL

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$$y = a + b + cx^2 \tag{1}$$

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Hunt, J.B. (1995). "*Environmental Dredging*". Smith & Son, Inc., New York, NY.

Donegan, T.M., and Dinicola, W.J. (1986). "*Turbidity Associated With Dredging Operations*". Technical Report, XYZ Consultants, Inc., Baltimore, MD., 60 p.

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White, F.K. and Jones, J.M. (1991). "*The Analysis of Flow Fields Around Dragheads.*" Journal of Waterway, Port, Coastal and Ocean Engineering, ASCE, Vol. 121, No. 5, pp. 1-16.

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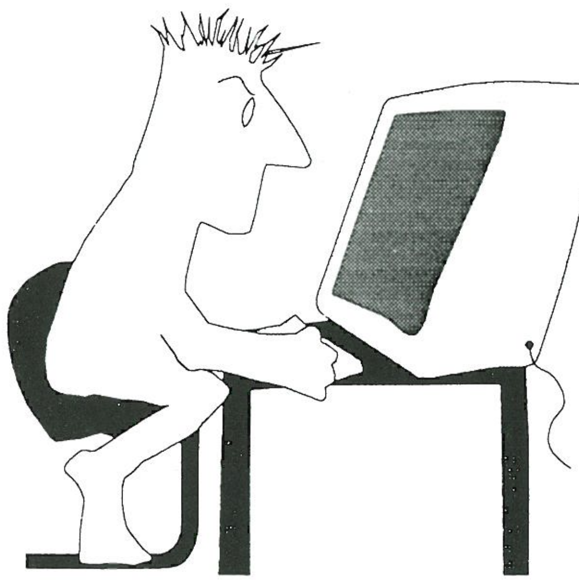

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