Report for 2004MD58B: Beneficial reuse of Baltimore Harbor dredgings and coal fly ash in engineering applications and effects on the Chesapeake Bay

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BENEFICIAL REUSE OF BALTIMORE HARBOR DREDGINGS AND COAL FLY ASH IN ENGINEERING APPLICATIONS AND EFFECTS ON THE CHESAPEAKE BAY

Final report submitted to the USGS/Maryland Water Resources Research Center

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by

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ABSTRACT

Dredged sediments are obtained from the process of dredging coastal areas and harbors in order to maintain navigable waterways. This study focuses on the potential of using dredged sediments as vertical cut-off wall backfill material. Materials used as vertical cut-off wall material are expected to have low hydraulic permeability and good workable characteristics. The Baltimore Harbor dredged sediments used for this study were plastic in nature and finer than the commonly encountered wall materials, and a research study was needed to evaluate their beneficial reuse in such an application.

The objective of this study was to find an appropriate mix of sediment and bentonite that will be able to function as a vertical cut-off wall backfill material. The preliminary tests on the bentonite were carried out for screening purposes and to find an appropriate water content that will satisfy the desired viscosity range. Bentonite was then added to the dredged sediment in ratios of 1%, 2% and 3% of the total dredged sediment weight. The preliminary tests were repeated for each of these mixes to determine applicable trends and at what percentages of bentonite, the viscosity of the mixture was still in the workable range. The 1% bentonite mix was additionally modified with the addition of 5% and 8% fly ash by weight. These mixtures were then subjected to API filter press tests to determine the effect these mixes would have on the hydraulic conductivity. Adsorption testing was also carried out on the dredged sediment and all the mixes to determine their adsorption capacities to see if they can potentially be employed in reactive cut-off wall applications.

The results show that a suitable moisture content and viscosity of the dredged sediments can be obtained that makes it usable in the mix design. Increased bentonite content, to the percent tested (3%), lead to a decrease in the hydraulic conductivity and increased fly ash content, to the percent tested (8%), lead to an increase in the hydraulic conductivity. For the metals tested, an increased bentonite content enhanced the adsorption capacity of the mix and an increased fly ash content diminished the adsorption capacity of the mix. With the appropriate mix design, dredged sediments can serve as an effective inhibitor to the flow of ground water and hence serve as an in-situ containment and remediation system.

TABLE OF CONTENTS

Contents	Page
List of Figures & Tables	iii
Section 1 – Introduction	1
Section 2 – Literature Review	4
Section 3 – Materials and Methods	32
Section 4 – Trench Stability	51
Section 5 – Hydraulic Conductivity Test Results	72
Section 6 – Adsorption Test Results	88
Section 7 – Conclusion	115
References	119

LIST OF FIGURES & TABLES

CHAPTER 2

Figure 2.1 – (Boston Harbor) Fort Point Channel Subsurface Profile.

Table 2.1 – Sediment Mineralogy of Newton Creek

Table 2.2 – Summary of Organic Contaminants in Newton Creek Sediments.

Table 2.3 – Summary of Direct (Indirect) Test Methods for Determination of Soil Parameters (dredged sediments)

Table 2.4 – Physical Properties of Dredged Harbor Bottom Sediments/ Organic Deposits

Table 2.5 – Summary of Tests on Dredged Harbor Bottom Sediments/ Organic Deposits

Figure 2.2 – Index Properties, Moisture Content, and Undrained Shear Strength Profiles for Clay Fill Soil Material.

Figure 2.3 – Index Properties, Moisture Content, and Undrained Shear Strength Profiles for Dredged Sediment Soil Material.

Figure 2.4 – Total/Wet Unit Weight Profile for Clay Fill and Dredged Sediment Layers

Figure 2.5 – Plasticity Chart for Dredged Sediments and Clay Soil Materials.

Figure 2.6 – Configuration of South Blakeley Island disposal site

Figure 2.7 – Compressibility data for Mobile Harbor sediment

Figure 2.8 – Permeability data for Mobile Harbor sediment

Figure 2.9 – Predicted layer thickness over time for various drainage efficiency factors (Mobile Harbor sediment at South Blakeley Island)

Figure 2.10 – Summary of Values for Coefficient of Permeability (Toledo, Ohio sediment)

Figure 2.11 – Characteristic Water Retention Curves (Toledo, Ohio sediment)

Figure 2.12 – Excavation of trench and Placement of Soil-Bentonite Backfill

Table 2.6 – Comparison of Local Topsoil Material with Dredged Material and Ameliorant Mixes

CHAPTER 3

Figure 3.1 – Extraction points for dredged material

Figure 3.2 – Grain size distribution of the dredged material used in the current study

Table 3.1 - Engineering parameters of the dredged material used in the study

Table 3.2 Typical Physical Properties of Bentonite

Table 3.3 – Chemical Properties

Figure 3.3 – Particle size distribution of the fly ash used in the current study

Table 3.4 Grain parameters and water and solids content of fly ash used in the tests

Table 3.5 Chemical composition of the fly ash

Table 3.6 Corrosion test results of the fly ash

Figure 3.4 - Picture of Beaker of Sediment on Scale

Figure 3.5 - Laboratory Oven at 110 degrees with soil samples

Figure 3.6 – Picture of Marsh Funnel

Figure 3.7 - Filter Press Test Apparatus Setup

Table 3.7 - Legend and the composition for the mix designs

Table 3.8 - Summary of Tests Performed

CHAPTER 4

Figure 4.1 – Slurry Walls for Projects in Chicago

Table 4.1 – Comparison of rigid wall support and slurry support for plane strain and axisymmetric excavations

Fig 4.2 – Theoretical build-up with time of filter cake on walls of trench

Fig 4.3 – Change in strength of slurry with time

Fig 4.4 – Horizontal movement of sides of trench caused by excavation and reduction in density of slurry.

Fig 4.5 – (a) Stability analysis of slurry trench for c-ø soil (b) Polygon for forces when øu=0 (c) Triangle of forces when Cd=0

Fig 4.6 – Layout of container and slurry trenches

Fig 4.7 – Representative soil element for lateral extrusion analysis

Fig 4.8 – Arching effect upon sandwiched weak sublayer in slurry trench

Figure 4.9 – Failure scenario based on sandwiched weak soil layer

Table 4.2 – Simplified soil profile of site

Table 4.3 – Factors of stability of weak sublayer

CHAPTER 5

Table 5.1 – Mud Weight Densities
Fig 5.1 – Effect of Bentonite on Mud Weight Density

Table 5.2 – Moisture Content Results
Fig 5.2 – Effect of Bentonite Content on Moisture Content

Table 5.3- Marsh Funnel Test Results
Fig. 5.3 – Marsh Funnel Viscosity versus Bentonite Content for Different Bentonite Slurries

Table 5.4 – Effect of Bentonite Content on Hydraulic Conductivity, σ'= 48.3kPa
Figure 5.4 – Effect of Bentonite Content on Hydraulic Conductivity, σ'=48.3kPa

Table 5.5 - Effect of Bentonite Content on Hydraulic Conductivity, σ' = 13.79kPa
Figure 5.5 – Effect of Bentonite Content on Hydraulic Conductivity, σ'= 13.8kPa

Table 5.6 – Effect of Bentonite Content on Hydraulic Conductivity, σ'= 6.9kPa
Figure 5.6 – Effect of Bentonite Content on Hydraulic Conductivity, σ'= 6.9kPa

Table 5.7 – Effect of Fly Ash on Hydraulic Conductivity, σ'= 48.3kPa
Figure 5.7 – Effect of Fly Ash on Hydraulic Conductivity, σ'= 48.3kPa

Table 5.8 – Effect of Fly Ash on Hydraulic Conductivity, σ'= 13.8kPa
Figure 5.8 – Effect of Fly Ash on Hydraulic Conductivity, σ'= 13.8kPa

Table 5.9 – Effect of Fly Ash on Hydraulic Conductivity, σ'= 6.9kPa
Figure 5.9 – Effect of Fly Ash on Hydraulic Conductivity, σ'= 6.9kPa

Figure 5.10 – Filtrate loss results

CHAPTER 6

(For Bentonite Mix)
Table 6.1 – Adsorption of Cadmium

Fig 6.1 – Effect of Bentonite on Cadmium Adsorption

Table 6.2 – Adsorption of Chromium

Fig 6.2 – Effect of Bentonite on Chromium Adsorption

Table 6.3 – Adsorption of Lead

Fig 6.3 – Effect of Bentonite on Lead Adsorption

Table 6.3 – Adsorption of Zinc

Fig 6.4 – Effect of Bentonite on Zinc Adsorption

(For Fly Ash Mix)
Table 6.5 – Adsorption of Cadmium

Fig 6.5 – Effect of Bentonite on Cadmium Adsorption

Table 6.6 – Adsorption of Chromium

Fig 6.6 – Effect of Bentonite on Chromium Adsorption

Table 6.7 – Adsorption of Lead

Fig 6.7 – Effect of Bentonite on Lead Adsorption

Table 6.8 – Adsorption of Zinc

Fig 6.8 – Effect of Bentonite on Zinc Adsorption

Fig. 6.9 – Effect of Barrier Thickness on Adsorption

Fig 6.10 – Effect of Hydraulic Gradient on Adsorption

Fig 6.11 – Effect of Porosity on Adsorption

Table 6.9 – Summary of Partitioning Coefficient (K_d) Values

Table 6.10 – Summary of Estimated Breakthrough Times

SECTION 1

INTRODUCTION

Nowadays, due to increasing environmental regulations, there is a focus on using suitable recycled materials for beneficial purposes. This brings us to exploring the potential of dredged sediments as a vertical cut-off wall material. Materials, mainly soil-bentonite mixtures, used in vertical cut-off wall construction are expected to have low hydraulic permeability and good workable characteristics. Vertical barriers are used to limit the flow of groundwater and contain and remove contaminants from sites where necessary. Vertical barriers are often implemented in construction operations where trenching is necessary. Dredged sediments are obtained from the process of dredging coastal areas and harbors in order to maintain navigable waterways. After the sediment has been excavated, it is transported from the dredging site to the placement site or disposal area. Dredged sediments are generally defined as elastic silt with moderate organic content (typically 8%) with a low % of fine sand and clay. The Baltimore Harbor dredged sediments were used for this study and are classified as CH (according to the unified soil classification system (USCS)).

In practice, approximately 400 million cubic yards of sediment must be dredged annually from waterways and ports to improve and maintain the United States (U.S.) navigation system (Palmero & Wilson, 1997). Of this amount, approximately 60 million cubic yards are placed at 108 U.S. Environmental Protection Agency (EPA) designated ocean disposal sites. The remaining 340 million cubic yards are placed in inland, coastal,

or estuarine open water sites, confined disposal sites, or beneficial reuse sites (Palmero & Wilson, 1997). Navigable waterways are needed for trade through U.S. ports as this contributes greatly to the economy. Therefore, alternatives for disposal of dredged material should be looked at from a technical, economic, and environmental point of view.

Until the 1970s, dredging state of the practice focused on efficiency of the dredging operation and production capacity, with an emphasis on economics (Palmero & Wilson, 1997). However, nowadays, with new environmental legislation (since the early 1970s), the state of the practice has evolved to include a wide range of environmental considerations, and so the emphasis has shifted to a balance of economics and the environment. Additionally, sites often need to be remediated before further construction can proceed in current construction practices. Research and development of technologies that involve in-situ containment and treatment are therefore being promoted by industry and the U.S. EPA. Through re-use of dredged material, the number of placement sites and disposal areas and associated environmental concerns should be reduced. In addition, because dredged sediment is a by-product of the dredging industry, the cost of the vertical barrier system can be reduced substantially than if natural resources were used.

The objective of this study is to find an appropriate mix of dredged sediments and bentonite that will be able to function as a vertical cut-off wall backfill material. The preliminary tests on the bentonite were carried out for screening purposes and to find an appropriate water content that will satisfy the desired viscosity range. Bentonite was then added to the dredged sediments in ratios of 1%, 2% and 3% of the total sediment weight. The preliminary tests were repeated for each of these mixes to determine applicable

trends and at what percentages of bentonite, the viscosity of the mixture was still in the workable range. The 1% bentonite mix was additionally modified with the addition of 5% and 8% fly ash by weight. These mixtures were then subjected to API filter press tests (used as a rigid wall permeameter) to determine the effect of these mixes on the hydraulic conductivity. Adsorption testing was also carried out on the dredged sediment and all the mixes to determine their adsorption capacities in the case that the cut-off wall is intended to act as a reactive barrier.

A literature review about the origin and properties of dredged sediments is given in Section 2. Section 3 includes the materials used in the testing program and the test methods. Practical application of the research work is discussed in Section 4. Results of physical property tests and hydraulic conductivity are presented in Section 5. The adsorption test results are discussed in Section 6. Section 7 is a summary and conclusion of all the results.

SECTION 2 – LITERATURE REVIEW

2.1 – DREDGING PRACTICE TODAY

Dredging means to dig or gather with a dredge (Palmero & Wilson, 1997) – to deepen (a waterway) with a dredging machine. After the sediment has been excavated, it is transported from the dredging site to the placement site or disposal area. Dredging is often carried out using trailing suction hopper dredges, which has three cycles: loading, sailing and unloading or using more modern machinery. The transport operation, most often, is accomplished by the dredge plant or by using additional equipment such as barges, scows, pipeline, and booster pumps. The actual depth to which a channel may be dredged is referenced to an appropriate low water elevation. It may be greater than the authorized depth to accommodate needed vessel clearances, dredging " over depth" also allows for the accuracy of excavation, and provides room for accumulation of material before the next dredging cycle (Palmero & Wilson, 1997). The tendency of the shipping industry is to design and construct larger vessels for increased efficiency. This in turn, requires harbor channels to be periodically deepened, which increases the dredging requirement.

There are three general alternatives that may be considered for placement or disposal of dredged material: open water disposal, confined (diked/ dredged fill containment areas located in an upland environment) disposal, and beneficial use applications. Beneficial reuses involve the placement or use of dredged material for some productive purpose. Generally, beneficial reuse involves either open water or confined placement in some form. Some beneficial reuses involve unconfined disposal, e.g.

wetland creation or beach nourishment. Other disposal methods, such as mine reclamation and aquaculture are occasionally used or considered, but there are usually limitations imposed (Palmero & Wilson, 1997). Dredged material has also been used for landfill capping and lining. Brick manufacture using dredged sediments is another innovation being explored (Hamer & Karius, 2001). Selection of a disposal alternative is made based on considering the technical, economic, and environmental issues. Of the approximately 400 million cubic yards dredged annually in the United States (U.S.), approximately 60 million cubic yards are placed at about 108 Environmental Protection Agency (EPA) designated ocean disposal sites. The remaining 340 million cubic yards are placed in inland, coastal, or estuarine open water sites, confined disposal sites, or beneficial reuse sites (Palmero & Wilson, 1997).

2.2 – TYPES, MINERALOGY AND DEPOSITION OF DREDGED FILL

Dredged fill can be comprised of five soil types: sand; mixed type of soil between sand and clay, sandy silty clay or clayey silty sand; clay; mixed type of soil between clay and rock and rock.

Dredged fill originates from coastal areas, where there is a need to maintain and improve navigable waterways and harbors and therefore dredging operations occur. A few such locations in the United States include the New York/New Jersey Harbor, the Baltimore Harbor in Maryland, the Mobile Harbor in Alabama, the Oakland Harbor in California; and internationally, The River Clyde in Scotland and coastal areas of Grand Cayman. The mineralogy varies somewhat depending on the coastal area from where the sediments are dredged. At the Fort point Channel in Boston, a typical subsurface profile

includes: Miscellaneous Fill (20 Feet), Harbor Bottom Sediments/Organic Deposits (15 feet), Marine Clay (Boston Blue Clay, 80 feet), and Glacial Till (10 feet), overlying argillite bedrock (Fig. 2.1) (Vaghar et al, 1997). In this case there are two separate aquifers, one above and one below the Boston Blue Clay. The water in Fort Point Channel is tidal. At the Houston-Galveston Ship Channel, east of Houston, the soil which forms the upper limit of the Pleistocene Age Formation, is a stiff to very stiff overconsolidated clay material locally referred to as the 'Beaumont Clay'. For the NY/NJ Harbor, the mineralogy of Newton Creek sediment was measured using x-ray diffraction (McLauglin & Ulerich, 1996), (Jones et al, 1997). These results are shown in Table 2.1. The amounts of the organic contaminants in the sediments at Newton creek are shown in Table 2.2.

Regarding deposition, the main aspects to be concerned with on a reclamation area are the bulking, the bearing capacity and the trafficability. In the case of sand, on hydraulic (dredged) fills above the water level, the relative density is 60-70%, (approximately 98% of the normal maximum Proctor density) (Verhoeven et al, 1998). This information is important in calculating the difference in volume between the pit (when there is a good indication of density) and the fill. The trafficability of the fill is dictated by the permeability. Therefore the bearing capacity of wet sands can be misleading (too low) if the amount of fines is increasing or a section of the fill can have a lower bearing capacity than the other because of local separation of fines (Verhoeven et al, 1998).

With clay fill, its behavior is mainly judged from the percentage of lumps in relation to the percentage of slurry. The main aspects, that is, bulking, bearing capacity

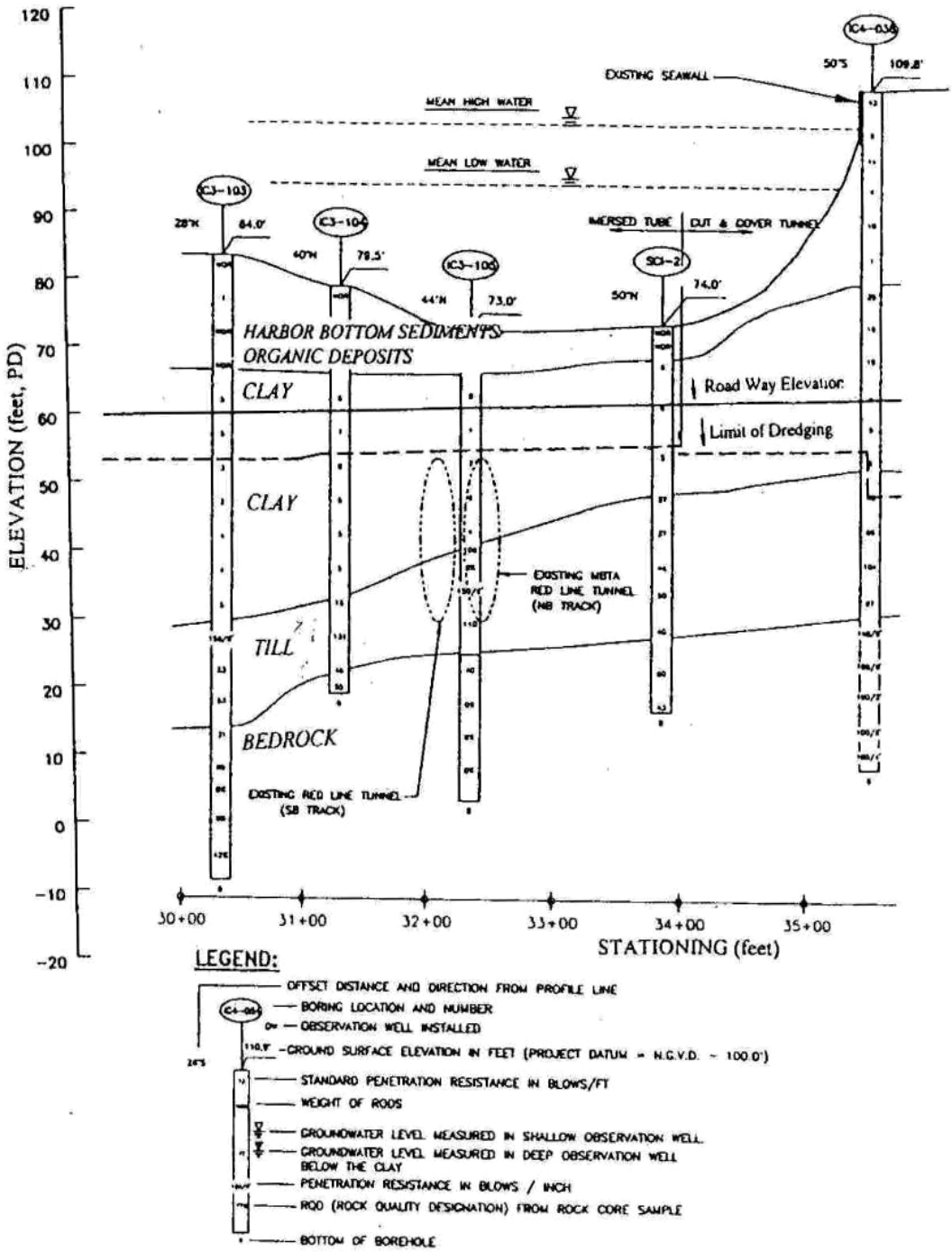

Fig. 2.1 – Fort Point Channel Subsurface Profile
(after Vaghar et al, 1997)

Table 2.1 – Sediment Mineralogy of Newton Creek

Mineral Species	**Chemical Formula**	**Weight Percent**
Quartz	SiO_2	66 to 75
Muscovite (Mica)	$K_2O.2MgO.Al_2O_3.8SiO_2.2H_2O$	11 to 15
Amorphous Phase	Organics	3 to 13
Kyanite	$Al_2O_3.SiO_2$	6 to 7
Hydrated Aluminum Silicate	$19Al_2O_3.173SiO_2.9H_20$	5 to 6
Cronstedtite	$4FeO.2Fe_2O_3.3SiO_2.2H_2O$	4 to 6

Table 2.2 – Summary of Organic Contaminants in Newton Creek Sediments

Contaminant	**Concentration (µg/g dry basis)**
Total Sulfides	7830
Total Organic Carbon (TOC)	73,200
Total Polychlorinated Biphenyls (PCB)	5.26
Total Chlorinated Pesticides	0.462
Total Polyaromatic Hydrocarbon (PAH)	117
Bis-2-ethylhexylphtalate	48.6
Fluoranthene	10.3
Phenanthrene	6.5
Others (24)	51.6
Total Dioxins	0.00645
Total Furans	0.0165

(after McLauglin & Ulerich, 1996)

and trafficability are influenced by this ratio. The bulking of the slurry can be determined from the consistency of the excavated soil and the Atterberg limits. It is assumed that a few days after the reclamation (at the start of consolidation under its own weight), the water content of the slurry is very high (2 to 3 times the liquid limit). The bulking can therefore be calculated from a clay with natural water content to the slurry. The bearing capacity and trafficability of clay fills are usually negligible when considering deposition.

In the case of the rock material, the main aspect concerned with its deposition is the amount of fines. If measures are not taken, a large amount of fines can be transported by the water outside the reclamation area leading to a pollution problem. The bulking can be estimated; the bearing capacity and the trafficability are generally good. Table 2.3, (Verhoeven et al 1998), shows a summary of soil parameters related to the dredging process.

For the mixed soil types, more information needs to be known. This information is needed to determine the soil parameters as similarly done for the clean soils.

Because of the deposition of dredged fill, that is, it is often hydraulically pumped along the banks of rivers or lakes or used to form small islands near dredging sites. In these areas the coarser solids settle from suspension and the excess water, with some suspended solids, then returns to the river/ lake through an overflow weir (Krizek et al, 1997). Problems with this method of disposal are high water content, low shear strength, and high compressibility of the dredged sediments. Therefore a landfill made using these materials would perform ineffectively for long periods of time. Also, the drainage condition at the bottom of the sediment layer has an effect on the consolidation rate during the period of deposition.

Table 2.3 – Summary of (In)direct Test Methods for Determination of Soil Parameters

Properties / Direct Tests	Mechanical/ Physical - A	In Situ Condition - B	Classification/ Identification - C
In situ	perm. probe, vane	dens. probe, pressiometer/dilatometer	field classification
Laboratory	(perm.) (triax.) (adhes.) (rheol.) tests	density/moisture c.	grain size, max/min density, spec. density, mineral comp., Atterberg limits
CORRELATIONS	(A)	(B)	(C)
	(D_r) –φ б (CPT/SPT)--dil. D_r/fines--k	CPT/SPT -- D_r	CPT/FR --comp.
	(SPT/CPT) (p. pen.) (LI -- C_u) (PI/σ_v) (CPT/FR)		
	(Brass. tens.) (point load) --σ_c		

(after Verhoeven et al 1998)

2.3 – CONVENTIONAL CONSOLIDATION AND STRENGTH PARAMETERS ASSOCIATED WITH DREDGED FILL

The soil properties can be sub-divided into three categories. Mechanical/physical properties, conditional properties and classification properties. The mechanical/ physical properties are associated with resistance (friction), strength and rigidity. Conditional properties include density and moisture content, which depend on stress history in the stages of loosening and deposition. They are important in mixing and transporting the individual particles or lumps (because of their resistance to disintegration and weight). The classification properties are those that are needed for the identification of the soil, that is from the grain-size distribution, the Atterberg limits and the density (maximum/minimum) of the soil.

The initial properties of sediments dredged from the bottom of a harbor are shown in Table 2.4 (Vaghar et al, 1997). The sediments mainly consist of a silt fraction and have an average bulk density of 76.2pcf. The initial water content exceeds 200%. These properties are usual for silty sediments in rivers and seas. (Need to show grain-size distribution chart). These soils are usually of very low strength. For sediments in this state, it is a common practice to add agents for solidification; some include quick lime, flyash, quicklime with ferric chloride and quicklime with calcium chloride. In order to decrease the water content of the mixtures, the pore water is extracted in a press chamber. The experimental results, for the different mixtures and the 'stand-alone' sample are shown in Table 2.5 (Vaghar et al, 1997).

Table 2.4 – Physical Properties of Dredged Harbor Bottom Sediment/Organic Deposits

Property	**Test Method**	**Result**
Bulk Wet Unit Weight	AASHTO T-19	74.2 – 78.1 pcf
Water Content	Modified AASHTO T255	260 – 270%
Specific Gravity of Solids	AASHTO T100	2.26
pH	ASTM D 4972	8 - 9
Organic Content	AASHTO T 267	< 1%
Classification	AASHTO M 145	Low Plasticity Inorganic to Organic SILT

(after Vaghar et al, 1997)

Table 2.5 – Summary of Tests on Dredged Harbor Bottom Sediments/Organic Deposits

T-180 (A)			Compaction Conditions			
Additives (% by weight)	**MDD (pcf)**	**OMC (%)**	**Moisture Content (%)**	**Dry Density (pcf)**	**Percent Compaction**	**UU Shear Strength (psf)**
None	85.2	27.7	21.0	78.8	93	3500
7% CaO	80.3	32.2	43.0	70.9	88	5195
7% (CaO/FA 50/50 mix)	82.6	31.6	37.4	76.4	92	4112
5%CaO/ 2% $FeCl_3$	80.6	33.8	49.3	66.5	83	4535
5% CaO/ 2% $CaCl_2$	79.1	34.7	47.2	67.8	86	3688
7% (CaO/HCFA, 50/50 mix)	79.5	32.9	44.7	69.4	87	4663

(after Vaghar et al, 1997)

For the unconfined disposal site at Spilman's Island, located along the Houston-Galveston Ship Channel, east of Houston, the following soil parameters were found. Index properties, moisture content and undrained shear strength profiles are shown for clay fill and dredged sediments are given in Figure 2.2 and Figure 2.3 respectively (Kayyal & Hassen, 1998). The total/wet unit weight profile for clay fill and dredged sediment layers is shown in Figure 2.4 (Kayyal & Hassen, 1998). Figure 2.5 (Kayyal & Hassen, 1998) shows the plasticity chart for dredged sediments compared with clay. From the plasticity chart, the average PI = 75.

The dredged fill for the disposal site at South Blakeley Island (Figure 2.6) (Poindexter & Walker, 1998), was taken from the Mobile Harbor in Alabama. Laboratory tests were done on this material by the Waterways Experiment Station. The soil was classified according to the Unified Soil Classification System (USCS) as a CL-ML, with a liquid limit of 96, a plastic limit of 28, and a specific gravity of 2.7 (Poindexter & Walker, 1998). The self-weight consolidation test and the modified fixed ring consolidometer were used to determine the compressibility and permeability characteristics of the material, which are shown in Figure 2.7 and 2.8 (Poindexter & Walker, 1998) respectively. Predicted layer thickness over time for various drainage efficiency factors for this site is shown in Figure 2.9 (Poindexter & Walker, 1998). They realized though that additional comprehensive field data was needed for thorough model verification and to better correlate empirical model coefficients with field operations.

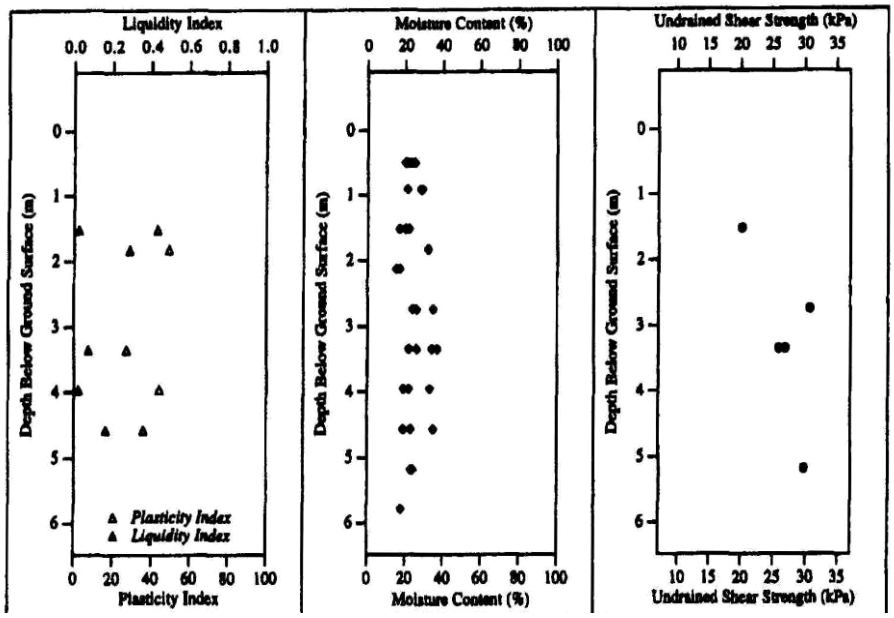

Fig. 2.2 – Index Properties, Moisture Content, and Undrained Shear Strength Profiles for Clay Fill Soil Material

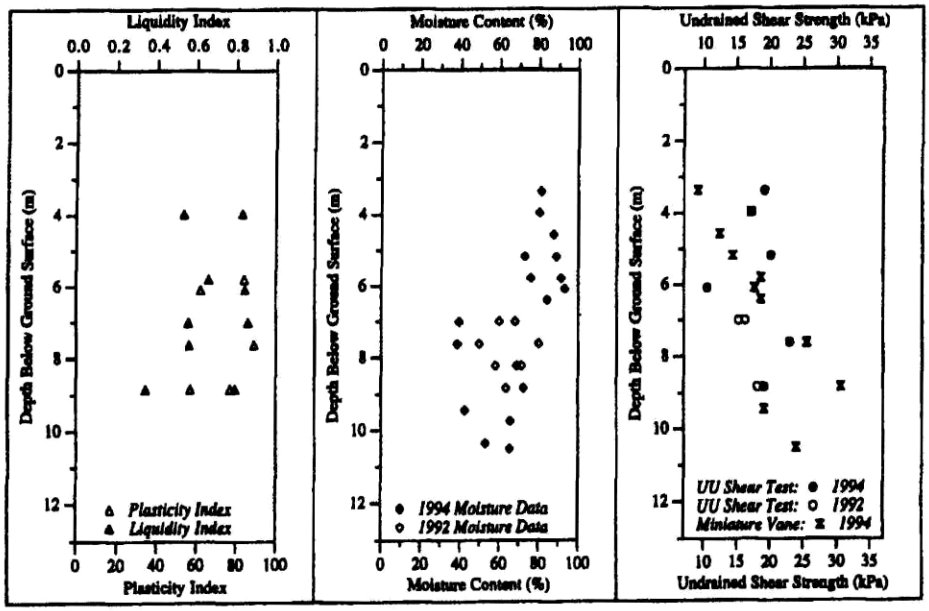

Fig 2.3 - Index Properties, Moisture Content, and Undrained Shear Strength Profiles for Dredged Sediment Soil Material

(after Kayyal & Hassen, 1998)

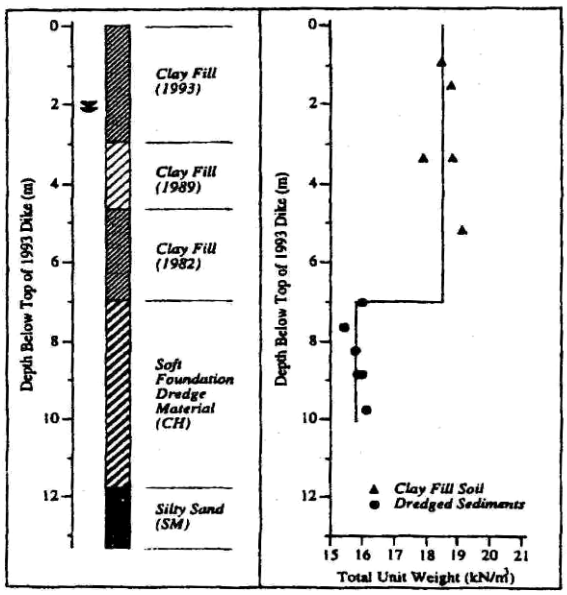

Fig. 2.4 – Total or Wet Unit Weight Profile for Clay Fill and Dredged Sediment Layers

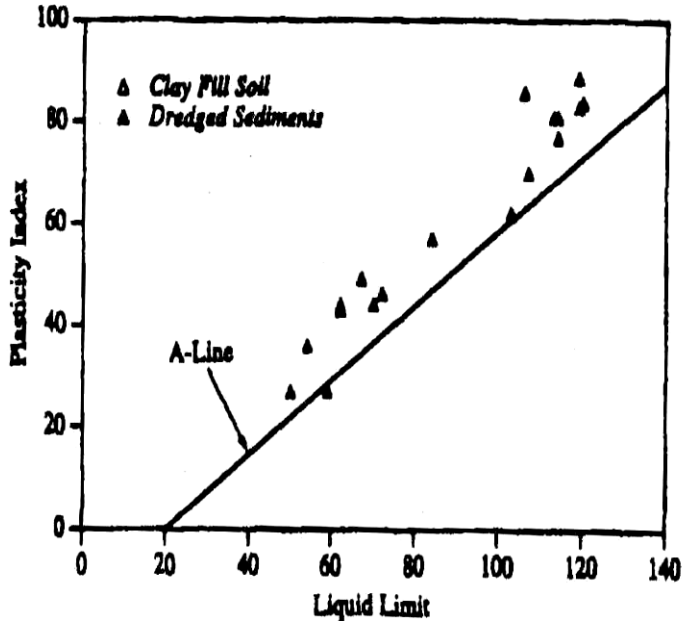

Fig 2.5 – Plasticity Chart for Dredged Sediments and Clay Soil Materials

(after Kayyal & Hassen, 1998)

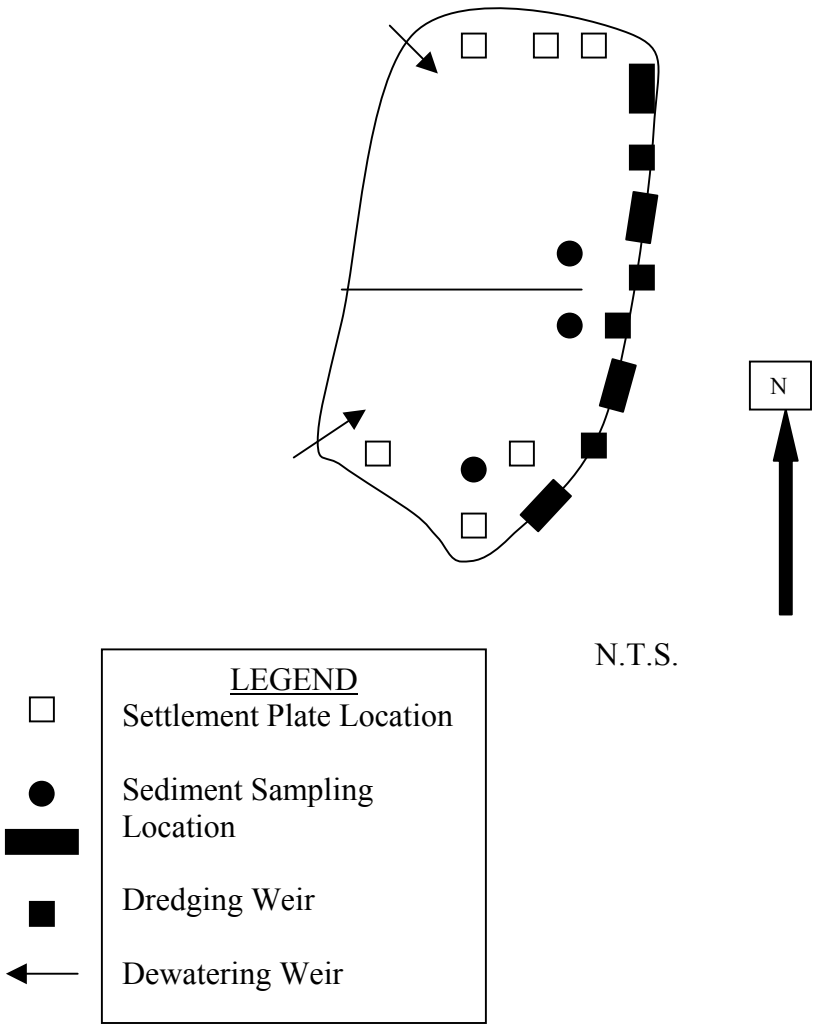

Fig. 2.6 – Configuration of South Blakeley Island Disposal Site
(after Poindexter & Walker, 1998)

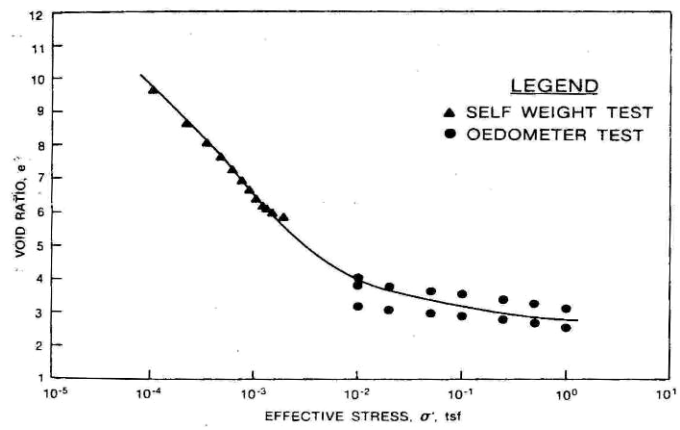

Fig. 2.7 – Compressibility data for Mobile Harbor Sediment

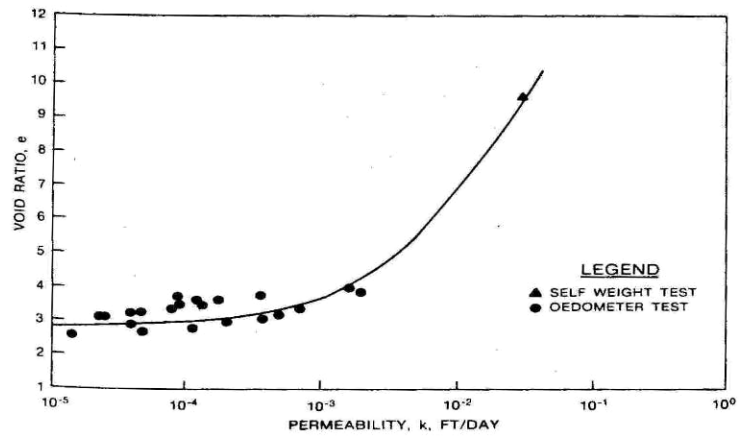

Fig. 2.8 – Permeability data for Mobile Harbor Sediment

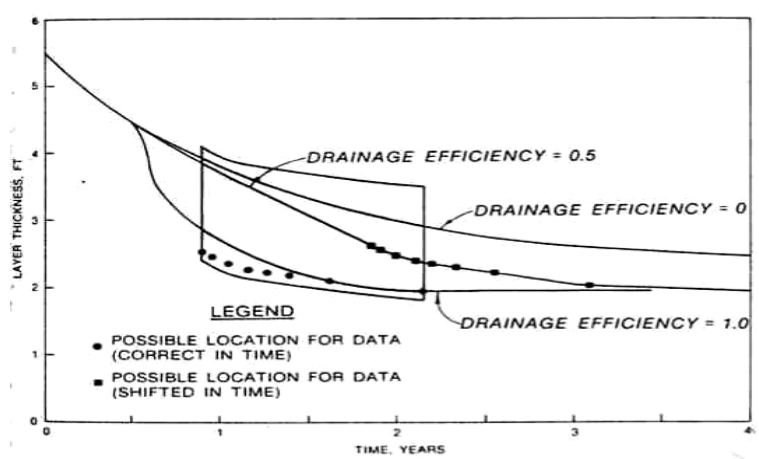

Fig. 2.9 – Predicted layer thickness over time for various drainage efficiency factors (Mobile Harbor Sediment @South Blakely Island)

(after Poindexter & Walker, 1998)

To more clearly understand the changes in water content that happen in sediments during and after the deposition process, a mathematical (empirical) model was developed to describe the flow of water through the consolidating media. The model gives the water content distribution in the fill at any time after the beginning of the deposition process; predicts the consolidation rate of the dredged sediments; and helps to evaluate the different techniques that can be used to accelerate the dewatering process (Krizek et al, 1997). Data was used from a laboratory and field test program done on dredged sediments from the harbor around Toledo, Ohio. These materials were found to have a liquid limit of 70%+_20%, a plastic limit of 35%+_10%, and a sand-silt-clay composition in the ratio of approximately 2:3:1 (Krizek et al 1997). Figure 2.10 (Krizek et al, 1997) shows the relationship for void ratio with coefficient of permeability for these materials. Figure 2.11 (Krizek et al, 1997) shows the characteristic water retention curves.

Concerning volume change characteristics, based on the results from a large number of conventional consolidation tests performed during this experimental program, the compression index Cc of dredged materials was confidently expressed as: $Cc = 0.01 (w - 7) = 0.01(37e - 7)$, where w = weight water content expressed as a percentage; and e = the void ratio (the specific gravity of the solids was assumed to be 2.70).

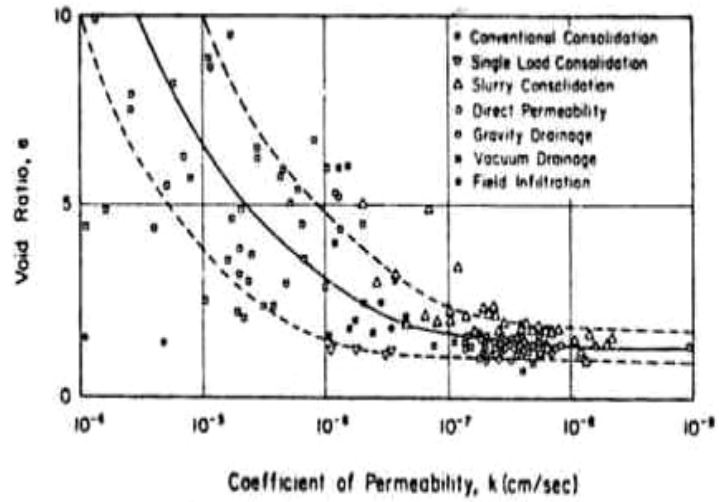

Fig. 2.10 – Summary of Values for Coefficient of Permeability (Toledo, Ohio Sediment)

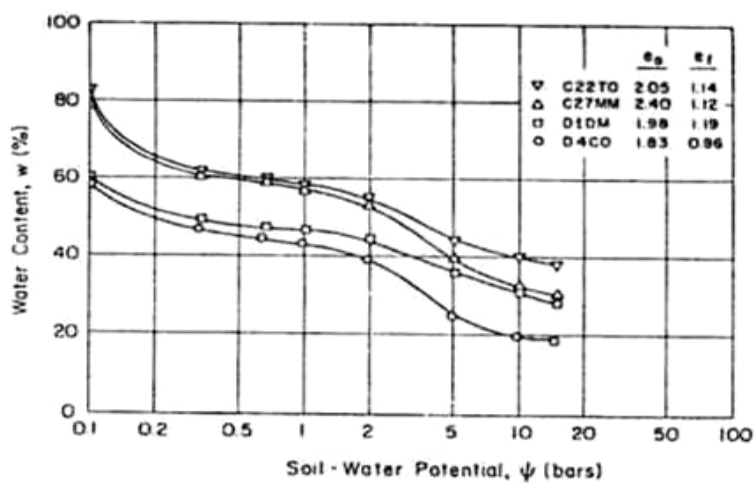

Fig. 2.11 – Characteristic Water Retention Curves (Toledo, Ohio Sediment)

(after Krizek et al, 1997)

2.4 - REVIEW OF RECENT 'BENEFICIAL REUSE' RESEARCH FOR DREDGED SEDIMENTS AND SIMILAR MATERIALS.

It has been established that there are several engineered beneficial reuses for dredged material. These include landfill/brownfield liners and covers, certain transportation applications, levee construction, use as structural and non-structural fill, and recently use as an ingredient in the brick manufacturing process.

The transportation applications that were investigated include reuse of the dredged sediments as roadway material and embankment material. The embankment study seems to be the most thorough, complete study on the reuse of dredged materials to date. This study involved use of the New Jersey dredged sediments. The New Jersey sediments were characterized as MH/OH, defined as an elastic silt with moderate organic content (8%) and they were found to have a low % of fine sand and clay. It was found that it performed satisfactorily as an embankment material. The dredged sediments were mixed with Portland Cement PC (4% and 8%) and fly ash - (in the case that already had the 8% PC). The dredge sediments were dewatered to near optimum moisture content before compaction - (specific % was not given). But in another beneficial reuse study, they mentioned generally dewatering to 85 to 45% water content.

The Baltimore dredged sediments compare with the NJ ones. There is a difference in that the Baltimore sediments are classified as a CH soil. The Baltimore sediments have more fines than the New Jersey ones. Therefore it seems appropriate that the Baltimore sediments would perform satisfactorily under similar circumstances.

In California, investigations are ongoing in terms of the beneficial reuse of dredged sediments for levee construction. Several demonstration projects have been initiated.

Flowable fills are self-leveling liquid-like materials that cure to the consistency of a stiff clay (Abichou et al 1998). Flowable fill is typically made up of sand or foundry sand (which contains bentonite), cement or flyash, and water. It is used for some applications similar to that which soil-bentonite is used for. These applications include use as backfill in utility trenches, building excavations and underground storage tanks. Design mixes are developed to satisfy local and state strength and flow requirements.

Foundry sands, a by product of casting and a mixture of fine sand and bentonite, (an ingredient in flowable fills), can potentially be used as a hydraulic barrier, e.g. in landfill liner and cover materials. A study done by Abichou et al (1998), indicated that foundry sands (green sands) offer a superior barrier than conventional clays and possibly at a lower cost. In order to determine the suitability of soil-bentonite mixes for vertical backfill material applications, it is hypothesized that similar testing can be conducted. The testing (as it relates to dredged sediments) includes determination of index properties and hydraulic conductivity testing.

Paper mill sludge is another material being investigated for its use as a vertical barrier material. A study by Moo-Young et al (2000) focused on the constructability, physical properties, and adsorption potential of paper mill sludge. A similar study was undertaken here with respect to dredged sediments.

This study will focus on the beneficial reuse of Baltimore dredged sediments as vertical cutoff wall backfill material.

2.5 – VERTICAL CUT-OFF WALLS

Vertical cutoff walls are installed in the subsurface to control horizontal movement of groundwater and contaminants (Daniel, 1993). The use of vertical cutoff walls in the subsurface initially began with groundwater control and structural applications in Europe before 1950 (Xanthakos, 1979). Slurry trench cutoff walls have been used in environmental applications since the 1970s and have come into widespread usage since the 1980s as a component in the overall remedial system to control flow of groundwater (Spooner et al., 1984), (Daniel, 1993).

In defining the vertical cutoff wall objectives, it is important to decide whether the barrier is to act as a ground water barrier with low hydraulic conductivity or as a contaminant transport barrier. Depending on the objective of the vertical cutoff wall, different criteria are considered in the design. For instance, if the objective is to minimize the rate of contaminant transport off-site, it is necessary to consider contaminant transport through the wall, potential degradation of the wall, and the consequences of inadequate cutoff wall performance. Also, the environmental control system could include a vertical cutoff wall along with a low permeability cover, groundwater withdrawal system and treatment systems for the pumpage. Vertical cutoff walls often key into a stratum of naturally low hydraulic conductivity. A key is not always necessary or cost effective when contaminated groundwater is being extracted or when the contaminants are concentrated near the ground surface or floating on the water table.

Here we will focus on vertical cutoff walls of low hydraulic conductivity. Typically, low permeability vertical cutoff walls are constructed by installing a vertical

barrier into the subsurface. The vertical wall has a lower hydraulic conductivity than the surrounding formation. The vertical cutoff walls can be divided into groups based on construction methods. Different types of vertical cutoff walls can be constructed using the slurry trench method of excavation and are therefore called 'slurry trench cutoff walls' (Daniel, 1993). In the slurry trench method of construction, a vertical trench is excavated into the subsurface using a slurry, typically of bentonite and water, for trench stability. The slurry is typically a bentonite-water mixture consisting of 5% bentonite and 95% water by weight. Bentonite, a montmorillonitic clay, swells in the presence of water, giving a viscous nature to the fluid and helps in the formation of a filter cake along the walls of the trench. For trench stability, the slurry level is kept at or near the top of the trench, typically within 1m. Trench widths vary between 0.6 and1.5m, with 0.9m trench widths being typical. The completed excavation is then used to form the geometry of the cutoff wall (Daniel, 1993). The completed cutoff wall can consist of soil-bentonite (Millet and Perez, 1981), cement-bentonite (Jefferis, 1981), plastic concrete (Evans et al., 1987), or structurally reinforced concrete (Boyes, 1975). Once the trench is excavated to the desired depth, the bentonite–water slurry is replaced with a (soil)-bentonite backfill with a low hydraulic conductivity. The backfilled trench forms the completed vertical cutoff wall. The backfill optimally consists of a mixture of sand, silt, and clay, and bentonite-water slurry. The soil-bentonite is placed in the trench at a consistency of high slump concrete (100-150mm) (Daniel, 1993). In order to attain this consistency, the material needs to be 'fluidized' by adding bentonite-water slurry to the soil. Figure 2.12 (Evans, 1993) shows a representation of the construction process.

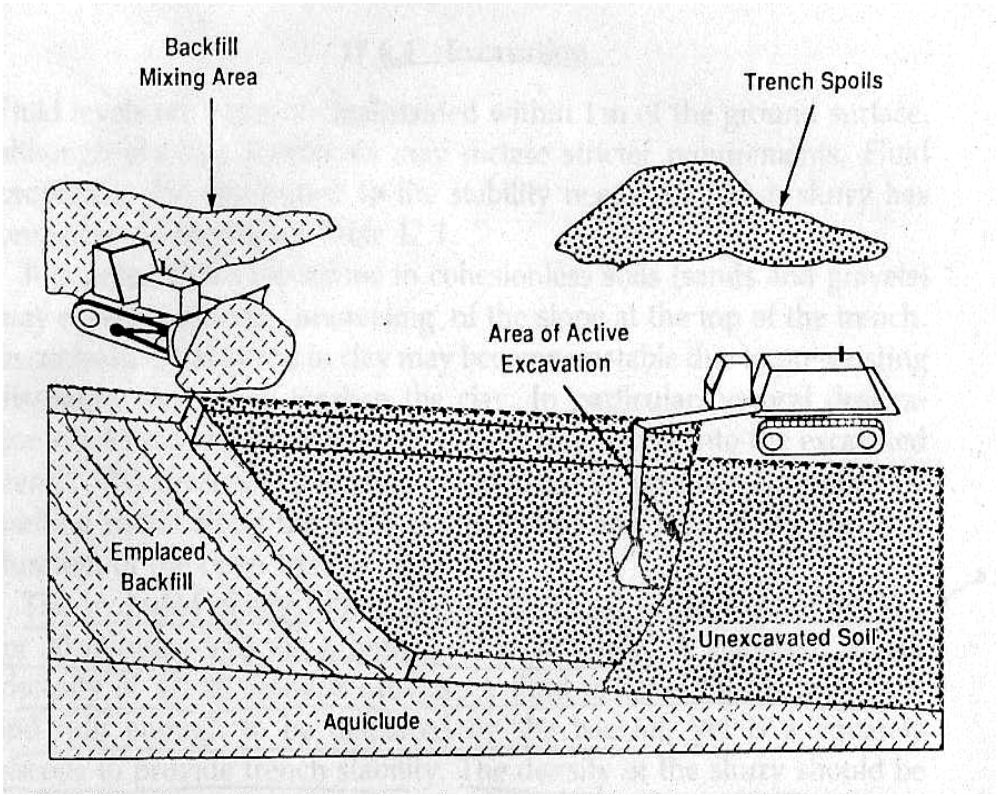

Fig. 2.12 – Excavation of trench and Placement of Soil-Bentonite Backfill
(after Evans, 1993)

Cutoff walls using the soil-bentonite backfill method of slurry trench construction were first used in the U.S. in the early 1940's (Daniel et al, 1994). In general terms, soil-bentonite slurry trench cutoff walls are among the least expensive techniques available in the U.S. for vertical barriers in the subsurface. Here we will be focusing on such soil-bentonite cutoff walls, with the possibility of substituting the soil with dredged sediments from the Baltimore Harbor. After evaluating the hydrogeologic and geotechnical aspects of the site, it is necessary to determine the appropriate properties for the soil-bentonite cutoff wall. During the feasibility studies, it is important to determine the short-term and long-term performance of the cutoff wall. That is, it is necessary to determine its properties in terms of hydraulic conductivity, strength, and compressibility. One needs to determine if these properties are satisfactory to meet the objectives of the project in the short-term as well as the long-term.

In this case, we will determine specific parameters of the Baltimore dredged sediment-bentonite slurry. These will be compared with typical parameter values of soil-bentonite slurry. The correct proportions of each ingredient in the sediment-bentonite slurry will be determined in order to achieve similar workability and purpose.

Sediment parameters that will be checked for include grain-size distribution, density, viscosity, and flowability/filtrate loss. The main mix parameters that will be tested for include: density and viscosity of the sediment-bentonite slurry (slump) and hydraulic conductivity of the sediment-bentonite slurry (using a rigid wall permeameter/ API filter press test). The adsorption potential of the mixes will also be tested.

2.6 – MOTIVATION FOR THE CURRENT RESEARCH

The prospect of substituting dredge sediments for soil in the bentonite slurry looks promising as the Baltimore sediments contain a high percentage of fines (classified as CH) and has a hydraulic conductivity of between $5x10^{-6}$ cm/s and $2x10^{-5}$ cm/s. Admixtures would be needed, which would further lower the mixture's hydraulic conductivity. This should not pose a problem though, as it has been proven that materials with lower hydraulic conductivities have performed effectively, namely soil-bentonite slurry trench cutoff walls ($1x10^{-7}$ to $1x10^{-8}$ cm/s).

Dredging projects are usually expensive and time consuming; they require large survey operations, sometimes making use of interactive Geographical Information Systems (GIS). In the beginning phase there is hardly enough time and opportunity to carry out a complete survey with the help of enough soil investigation methods. In practice, approximately 400 million cubic yards of sediment must be dredged annually from waterways and ports to improve and maintain the United States' navigation system (Palmero & Wilson, 1997). Therefore alternatives for the disposal of dredged material from these projects have to be carefully looked at and developed from an economic, technically feasible, and environmentally acceptable point of view. Until the 1970s, the dredging state of the practice focused on efficiency of the dredging operation and production capacity, with an emphasis on economics (Palmero & Wilson, 1997). However, nowadays, with new environmental legislation (since the early 1970s), the state of the practice has evolved to encompass a wide range of environmental considerations, and so the emphasis has shifted to being able to balance economics and the environment. The U.S. water transportation system has been operating since the early 1800s and has

played a major role in the growth of the U.S. economy. The importance of navigation is evidenced by the fact that approximately 95 percent of all U.S. international trade moves through its ports (Wilson 1996) (Palmero & Wilson, 1997). Baltimore Inner Harbor is rich in maritime history and has been an active new world port and center of trading since the early 1600's. Since then shipping has combined with heavy industrialization in the 1800s (Snyder et al, 1997).

Unfortunately, the U.S., like other countries, does not have naturally deep ports or channels. Therefore there is a great need for dredging, to maintain and improve these waterways. Dredging can be a difficult issue from the standpoint of environmental concerns, and so any national dredging program must be managed to balance economics and the environment.

Potential environmental impacts from dredged disposal may be caused by physical or chemical processes. Physical impacts could come about from direct burial of organisms, loss of habitat, or generation of turbidity. Many of the waterways are located in industrial and urban areas therefore sediments are often contaminated from these sources. Table 2.6 (DeSilva et al, 1991) shows a comparison of elements in local topsoil material with dredged material and ameliorant mixes. The contaminated sediment, in turn, has led to concerns about water quality and aquatic organisms. Therefore good planning, design and management, (with appropriate environmental controls), of dredging operations are necessary for dredged material disposal to be done efficiently.

In addition, in current construction practices, sites often need to be remediated before further construction can proceed. Present remediation methods include pump and

Table 2.6 – Comparison of Local Topsoil Material with Dredged Material and Ameliorant Mixes

Dredged Material Treatments

		Night Soil	Sewage Sludge	Saw Dust	R. Cart	R. Cart/ Peat Night Soil	PFA	Bark	MC	DM	Peat only	Peat
TOPSOIL higher in	Soil Pb	***	***	***	***	***	***	***	***	***	***	***
	Soil Zn	***	***	***	***	***	***	***	***	***	***	***
	Soil Fe	***		***			***	***	***			
	Soil Cd		***	***		***	***	***			***	***
	Soil Cu		***	***	***	***	***	***	***		***	***
	Soil Ni		***	***	***		***	***	***	***	***	
	Soil Hg		***						***			
	Soil Cr								***			
	Soil Cu					**			**			
	% germ biomass	***	***	***	***	***	***	***	***	**	***	
TOPSOIL lower in	Soil Cu									***		
	Soil Cd	***			***				***	***		
	Soil Cr	***	***					***			***	***
	Soil Hg	***								***	***	***
	Soil Fe				***							***
	Soil Zn	**	***	***	***	***	***	***		***	***	***
	Soil Mn	**	***	***	***	***	**	***	***	***	***	***
	Soil Pb	**				***	**					
	Soil Cd	**						***				
	Soil Ni											***

Left hand column indicates status of the topsoil control in relation to each of the other treatments

MC – Mushroom Compost

DM – Dredged Material

* - Significant at $p < 0.05$

** - Significant at $p < 0.01$

*** - Significant at $p < 0.001$

(after DeSilva et al, 1991)

treat and electrokinetics. They generally require a large amount of energy and maintenance and have been found to produce limited long-term success (Suthersan 1997). Factors that affect the long term cost and success of a remediation project include composition of the contaminant, permeability and composition of the soil matrix, geologic setting and hydraulic characteristics of the area (Gallager 1998, Moo-Young et al 2000). Therefore research and development of technologies that involve in-situ containment and treatment are being promoted by industry as well as the EPA. Soil-bentonite barriers are commonly used for these purposes. These barriers basically form large containment systems and need another component to remove the contaminant from the ground. The dredged sediment barrier is one that is anticipated to contain and remove contaminants with relatively little energy input compared to pump and treat technologies. If this material proves to be suitable for limiting water flow and reacting with the representative contaminant, then the vertical cut-off wall can contain and attenuate the representative contaminant as the ground water passes slowly through the barrier system. In addition, because dredged sediment is a by-product of the dredging industry, the cost of the barrier system should be reduced substantially.

The slurry wall technique of stabilization is considered an outstanding innovation in underground construction. So, in developing another useful engineering application, specifically this one using dredged sediments, it is anticipated that this will offset some costs of having to use natural resources for such applications. It is also hoped that this will minimize the number of necessary disposal sites and associated environmental problems. It is also hoped that this investigation will give knowledge on the proposed

technology, thereby encouraging further research leading to possible widespread use of the technology.

SECTION 3

MATERIALS AND METHODS

3.1 MATERIALS

3.1.1 Dredged Sediments

The dredged material used for this study was obtained from Tolchester Channel located in Baltimore Harbor, Maryland. The dredged material was dredged from six different points in the channel. The extraction points are shown in the Figure 3.1. The material was black in color and had some odor. As it was received, the natural water content of the material was 400-600%.

Sieve analysis indicated that 95% of the material passed through the No. 200 sieve size (0.075 mm). The grain size distribution curve, obtained from hydrometer testing, is given in Figure 3.2. The Atterberg limits were measured in accordance with ASTM D 4318. The liquid limit and plastic limit of the material were found to be 85 and 35, respectively. The material was classified as a high plasticity clay (CH) according to the Unified Soil Classification System (USCS). Table 3.1 summarizes the index parameters of the dredged material.

3.1.2 Bentonite

Bentonite is a common clay mineral found in sedimentary and residual soils. Bentonite is commonly mixed with soils to form material for use in vertical cut-off walls.

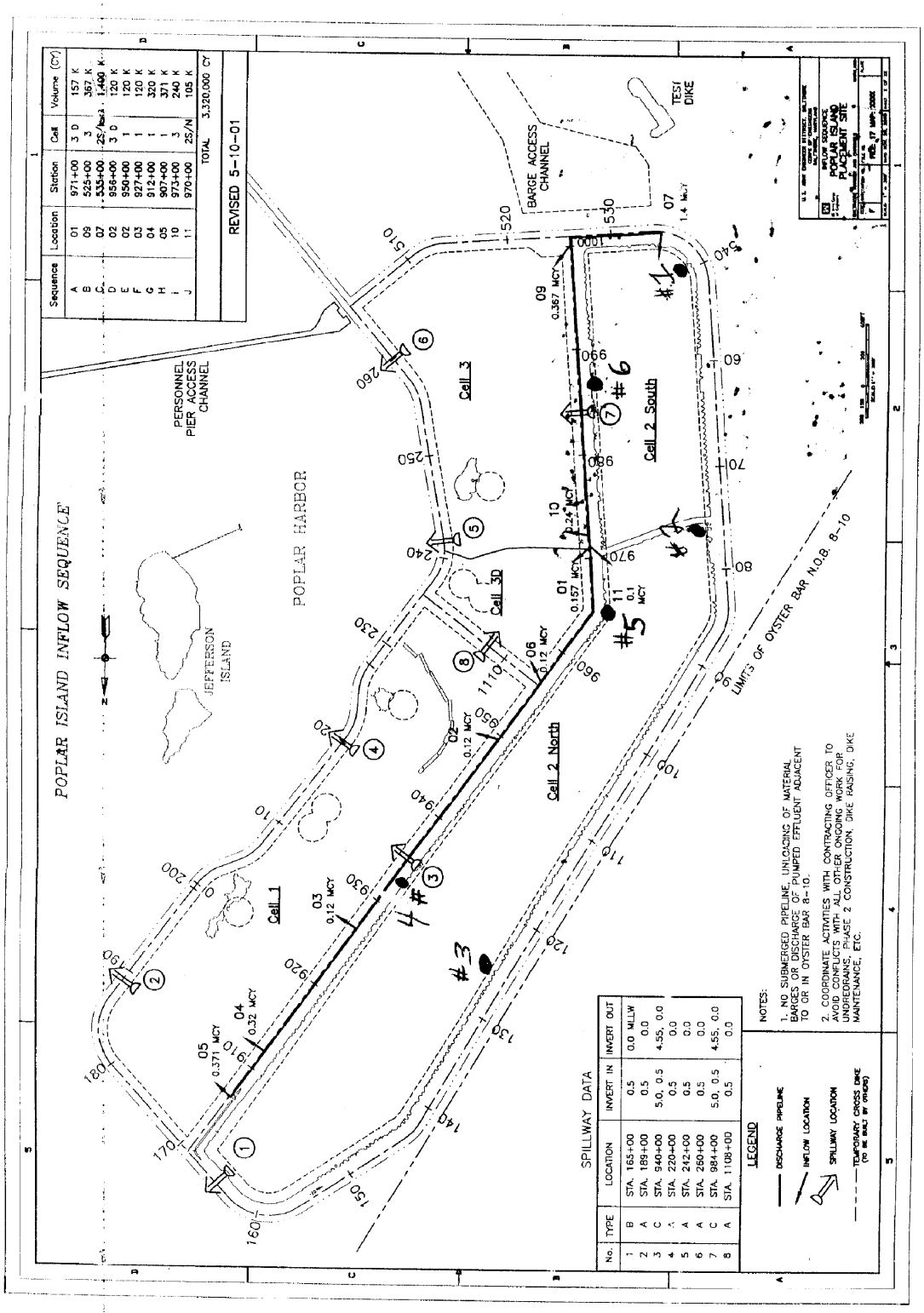

Figure 3.1 – Extraction points for dredged material

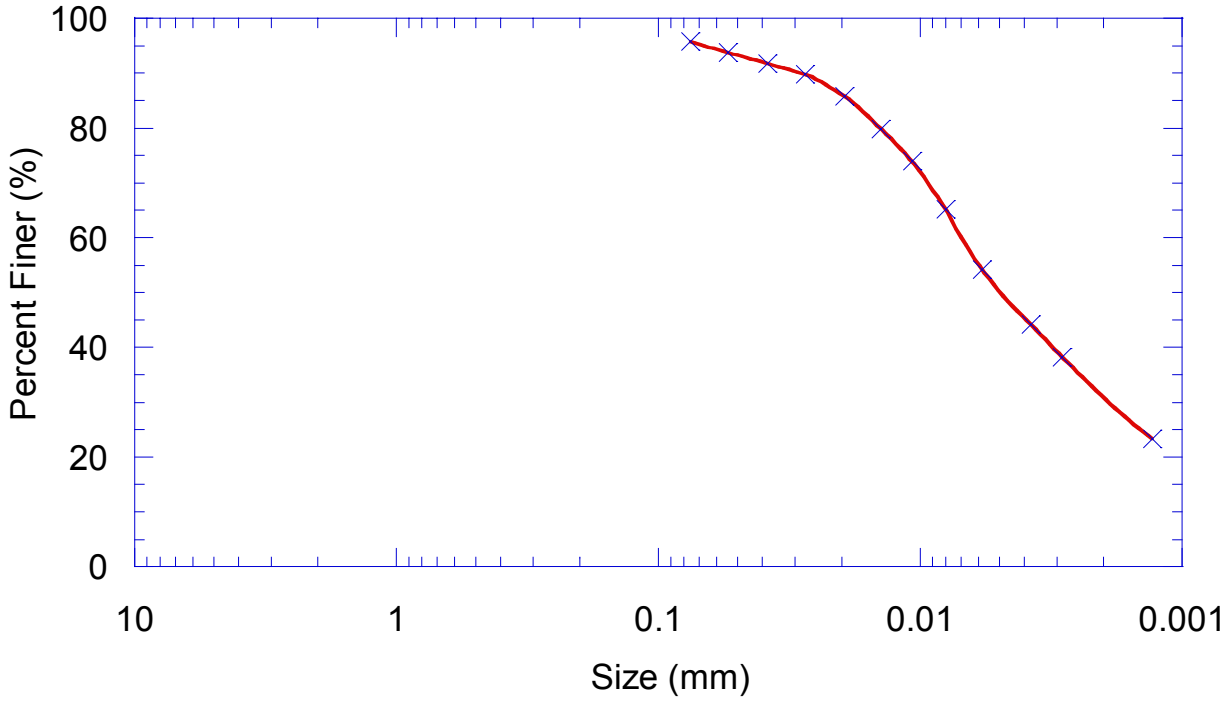

Figure 3.2 – Grain size distribution of the dredged material used in the current study

Table 3.1 - Engineering parameters of the dredged material used in the study.

D_{15}, (mm)	D_{50}, (mm)	D_{85}, (mm)	D_{90}, (mm)	C_u, (D_{60}/D_{10})	Specific Gravity G_s	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	USCS Class.
0.0009	0.005	0.019	0.038	11.7	2.6	85	35	50	CH

In this study, different mixes of sediment and bentonite were experimented with to find an optimum combination that could be used for vertical cut-off wall backfill material. The bentonite used in this study, BARA-KADE 90 was provided from Bentonite Performance Minerals, Inc. from the Colony, Wyoming Plant. The material obtained was a high quality, powdered sodium bentonite used in slurry wall construction, soil sealing and other hydraulic barriers. The product conforms to API specification 13A. Because of its fine particle size, it is mainly used with pugmill mixing operations in soil/bentonite liner construction. The typical physical properties are given in Table 3.2, and the typical chemical properties are given in Table 3.3. Wet sieve results according to ASTM WK217 resulted in 98% fines.

3.1.3 Fly ash

The fly ash used in the study was obtained from Brandon Shores Facility of Baltimore Gas and Electric Company, located in Baltimore, MD. The fly ash was produced from burning bituminous coal and had pozzolanic properties. The natural water content of the fly ash was 30 % when it was received. The appearance of the material was like fine-grained gray powder, and it had no odor. Specific gravity of the material was measured in accordance with the ASTM D 854 and found to be 2.22. Optimum moisture content and maximum dry density of the material were determined by using the test method ASTM D 1557 (modified proctor effort) and found to be 25 % and 12.8 kN/m^3, respectively.

Particle size analysis indicated that 85 % of the material passed the No. 200 U.S. standard sieve size. Figure 3.3 provides the complete particle size distribution curve of

Table 3.2 Typical Physical Properties of Bentonite

a) Slurry Properties (6% Suspension)

Property	Value	Specification
Viscosity, FANN 600 rpm	37	30 min
Marsh Funnel, s/quart	36	NA
Apparent Viscosity, cps	18.5	NA
Plastic Viscosity	12	
Yield Point, (N/m^2)	0.479	3x max plastic viscosity
Filtrate loss at 30 min under $\sigma' = 696.5$ KPa	14	15 cm^3 max
Filter cake thickness, mm	2.38	NA

b) Industrial Properties

Moisture Content (%)	9	NA
Swell Index	28	NA
Specific Gravity	2.7	NA
pH, 6% suspension	9.5	NA
Bulk Density (kN/m^3) (uncompacted)	7.7	NA
Bulk Density (kN/m^3) (compacted)	11.32	NA

Table 3.3 – Chemical Properties

a) X-ray Analysis

Mineral	Percentage (%)
Montmorillonite	88-90
Quartz	0-6
Feldspar	5-7
Cristobalite	0-1
Biotite & Mica	0-2

b) Chemical Analysis

Compound	Percentage (%)
SiO_2	63.59
Al_2O_3	21.43
Fe_2O_3	3.78
CaO	0.66
MgO	2.03
Na_2O	2.07
K_2O	0.31
Bound Water	5.50

* Metals listed in the chemical analysis are complexed in the mineral. They do not necessarily exist as free oxides.

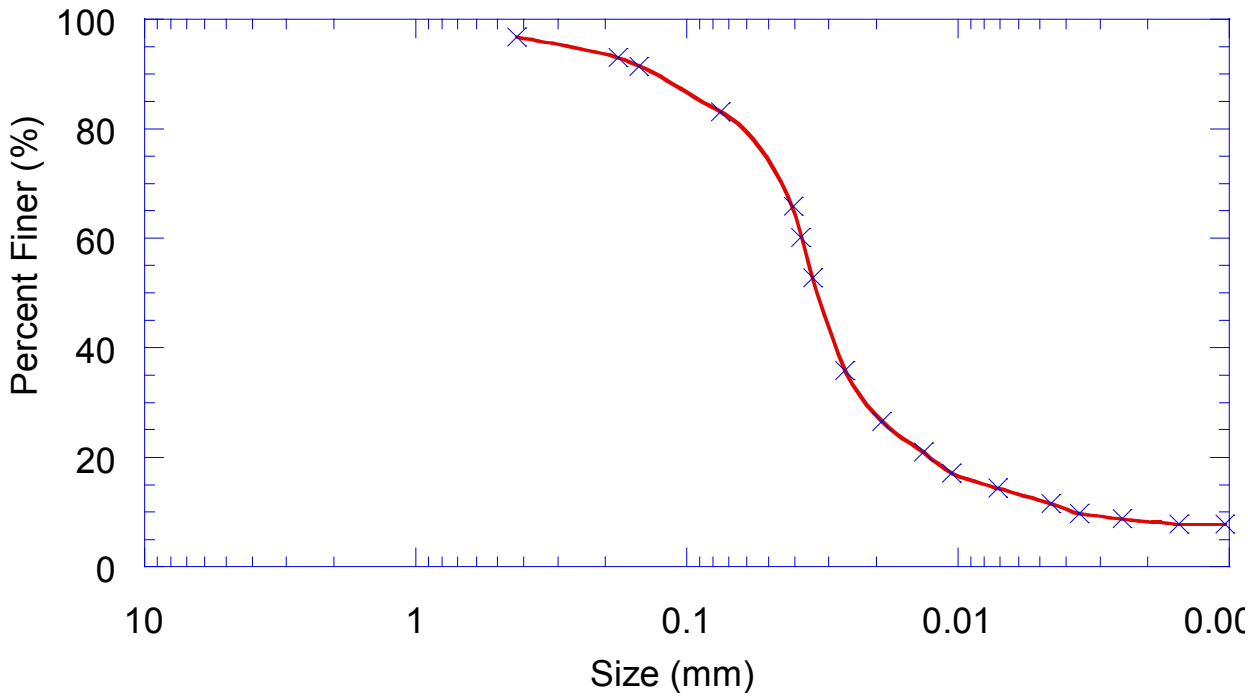

Figure 3.3 – Particle size distribution of the fly ash used in the current study

the fly ash, determined by sieve and hydrometer analyses. Characteristic particle sizes (D_{15}, D_{50}, D_{85} and D_{90}), coefficient of uniformity (C_u) and water and solids contents of the fly ash are given in Table 3.4.

Chemical analysis of the fly ash indicated that silicon dioxide, aluminum oxide and ferric oxide make up approximately 94% of the material. Details of the chemical analysis and the corrosion test results are shown in Table 3.5 and 3.6, respectively.

3.2 TEST PROCEDURES ON THE SEDIMENT

3.2.1 Density Testing and Determination of Water Content

The density of the sediment was first determined by measuring the physical dimensions of the material. That is the weight, and actual volume which the mass occupied was determined. The mass of a given volume (500 mL) of the sample of sediments was found on a grams scale. Density was found as the mass per unit volume the sediments occupied. Unit conversions were necessary to convert the measured value of grams per milliliter to kilonewtons per cubic meter. Samples were taken from each of the two buckets of sediments collected from the shore, Craighill Angle Project. One set of samples were called Sample A, and the other was called Sample B. Two series of tests were done on each sample. Figure 3.4 shows a picture of the beaker of sediment on the scale.

The water content was determined using the ASTM D2216 procedure. Figure 3.5 shows a picture of the laboratory oven at 110 degrees with the soil samples.

Table 3.4 Grain parameters and water and solids content of fly ash used in the tests.

D_{15}, (mm)	D_{50}, (mm)	D_{85}, (mm)	D_{90}, (mm)	C_u, (D_{60}/D_{10})	Specific Gravity G_s	Water Content (%)	Solids Content (%)
0.008	0.033	0.092	0.137	9.87	2.22	30	77

Table 3.5 Chemical composition of the fly ash.

Elements		%
SiO_2		50
Al_2O_3		30
Fe_2O_3		14
CaO		1.8
MgO		0.3
K_2O		1.5
TiO_2		1.1
Arsenic		0.03
Other		1.27

Table 3.6 Corrosion test results of the fly ash.

Test	Results
pH	7.7
Resistivity (ohm-cm)	1700
Sulfides (mg/kg)	<1
Soluble sulfates (mg/kg)	1550
Chlorides (mg/kg)	145
Redox potential (mV)	350

Fig 3.4 - Picture of Beaker of Sediment on Scale

Fig. 3.5 - Laboratory Oven at 110 degrees with soil samples

3.2.2 Marsh Funnel Viscosity Test

Viscosity is the measure a fluid's resistance to flow in an unenclosed apparatus. Therefore the test used to determine the viscosity of the dredged sediments was the Marsh Funnel viscosity test. The test measures the total-time it takes for a given volume of dredged sediments to pass through a calibrated orifice. Figure 3.6 shows a picture of the Marsh Funnel.

The orifice at the bottom of the Marsh Funnel was stopped by a finger. The funnel was then filled with the dredged sediments by pouring it through the screen on the top of the funnel until it reaches the screen level. A 1-quart test cup was placed under the funnel. The stop-watch was started simultaneously while removing the finger from the funnel orifice to allow the fluid to flow into the test cup. The stop-watch was stopped when the fluid level reached the 1-quart line in the test cup. The fluid viscosity is described in terms of the amount of time, in seconds, that was necessary for 1 quart of the fluid to flow through the calibrated orifice.

3.2.3 The Filter Press Test

The American Petroleum Industry (API) filter press test is commonly used to measure the hydraulic conductivity of soil - bentonite mixtures (Filz et al. 2001), slurries, oil well cements and cement additives (Aydilek 1996). The filter press test was used in this case to determine the filtrate loss and flow rate of the dredged sediments. The sediments were placed into the filtration device.

Fig. 3.6a – Side View of Marsh Funnel

Fig. 3.6b – Top view of Marsh Funnel

The filter press test equipment consisted of (1) a pressure cell made of a steel pipe which had a diameter of 79 mm and a height of 90 mm, (2) a cap having a pressure hole and (3) a bottom part which had a flow hole (refer to Figure 3.7). There were two O-ring rubber gaskets at the connections between the cap and cell and bottom part to prevent leakage. A T-screw was used to tighten the cap, the pressure cell and the bottom part. The test method determined the flow rate of the dredged sediments.

The time for a given volume of filtrate was recorded in order to determine the flow rate. When it was observed that the rate at which the water came out of the bottom decreased, the main part of the experiment had been completed. The volume of water collected under the filtration device was recorded in given increments, along with the corresponding time it took to reach that volume. Finally, the characteristics of the filter-cake were recorded to observe the compacted dredged material.

The clean, dry parts of the filter cell were assembled in the following order (refer to figure 3.4); base cap, rubber gasket, screen, a sheet of geotextile, rubber gasket, filter cell. The cell was secured to the base cap by rotating it clockwise. The cell was filled with the test sample to within approximately ¼" (6mm) of the top. The filter press cell assembly was set in place within the frame. The top cap was checked to make sure that the rubber gasket was in place. The top cap (already connected to the pressure source) was placed onto the filter cell and secured in place with the T-Screw. A dry graduated cylinder was placed under the filtrate tube. Varying pressures were applied to the cell. At the end of the test, the pressure source valve was closed, the regulator was backed off

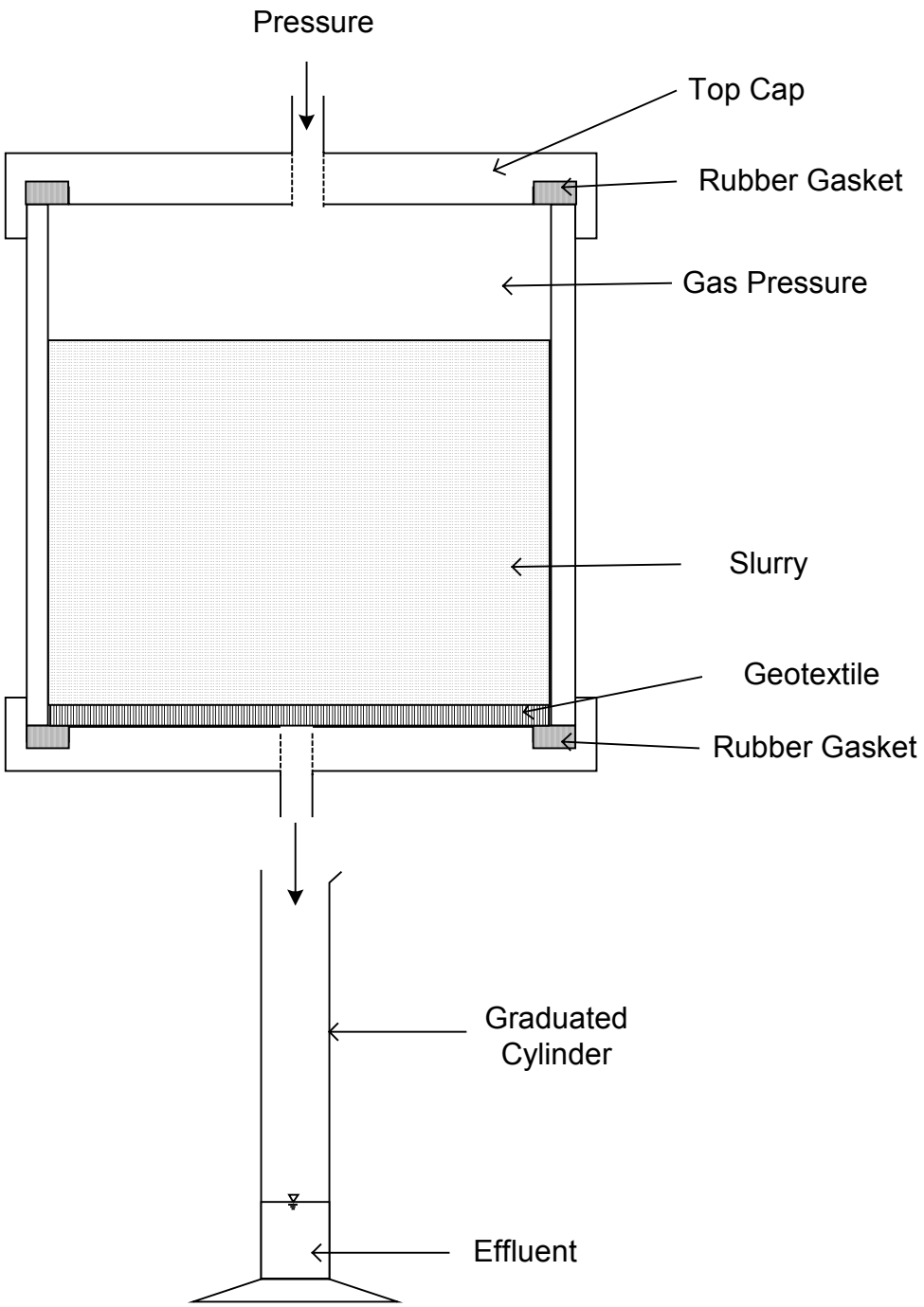

Figure 3.7 - Filter Press Test Apparatus Setup

and the safety-bleeder valve was opened. This releases the pressure on the entire system. The volume of filtrate collected was then measured in mL in the graduated cylinder. The T-screw was loosened, the top cell removed, and the cell removed from the frame. The filter cell was disassembled and the filter cake, geotextile and filter paper were carefully removed from the base cap. The thickness of the filter cake was measured and recorded to the nearest 1/32" (0.8mm). The properties of the filter cake such as texture, hardness and flexibility were recorded.

3.3 TEST PROCEDURES ON THE SEDIMENT-BENTONITE MIX

The sediments were mixed with bentonite in proportions including 1%, 2% and 3% of bentonite by weight. Similarly, density testing was performed on these different mixes of the sediment-bentonite. The moisture content of the mixes was also determined in a similar fashion to that of the sediments. The viscosity of the mixes was determined using the Marsh Funnel test. Some of the mixtures prepared with 1% bentonite were modified with the addition of fly ash. Flyash was added in the proportions of 5% and 8% by weight. Hydraulic conductivity testing was performed on the bentonite mixes as well as the fly ash mixes. Adsorption testing was also performed on the dredged sediment and mixes.

3.3.1 Hydraulic Conductivity Testing

The hydraulic conductivity of the mixes was determined using the rigid wall permeameter test as specified in ASTM D 5856. The API filter press test apparatus was used for the testing. The American Petroleum Institute (API) filter press is a rigid wall

cell used to determine the filtrate loss of bentonite slurries (American Petroleum Institute 1985). It is also commonly used to measure the hydraulic conductivity of soil-bentonite, both during mix design and as part of construction quality assurance and quality control (Filz et al, 2001). This study also compared hydraulic conductivity testing using a rigid-wall consolidometer permeameter test, flexible-wall permeameter test as well as the API filter press test. The analysis showed that the results compared well. This test is also a little less labor intensive than the other tests. A study by Zamojski et al ( ) showed similar results obtained for hydraulic conductivity through using both flexible-wall and fixed-wall permeameter tests. A study by Evans, (1994) reveals that a fixed wall API Filter Press has been used to conduct rapid evaluations of hydraulic conductivity in the field as the construction progresses. D'Appolonia, (1980) also mentions use of the API Filter Press Test for testing purposes representative of field conditions during slurry trench construction. This provided justification for the test method used in the present study.

3.3.2 Adsorption Testing

Equilibrium batch testing, according to ASTM D4646 specification was performed on the dredged sediments and mixes with aqueous solutions containing heavy metals to determine the adsorption capacities of the dredged material and mixes. The metal sorbates used were cadmium, chromium, lead and zinc.

3.4 TEST PLAN

The intention of this testing plan is to find an appropriate mix of sediment and bentonite that will be able to function as a vertical cut-off wall backfill material. The preliminary tests on the bentonite were carried out for screening purposes and to find an appropriate water content that will satisfy the desired viscosity range.

Bentonite was then added to the dredged sediment in ratios of 1%, 2% and 3% of the total dredged sediment weight. The preliminary tests were repeated for each of these mixes to determine applicable trends and at what percentages of bentonite, the viscosity of the mixture was still in the workable range. The 1% bentonite mix was additionally modified with the addition of 5% and 8% fly ash by weight. These mixtures were then subjected to API filter press tests (used as a rigid wall permeameter) to determine the effect these mixes would have on the hydraulic conductivity. Adsorption testing was also carried out on all the mixes to determine their adsorption capacities. Table 3.7 shows a legend for the composition of the mix designs and Table 3.8 shows a summary of the tests performed.

Table 3.7 - Legend and the composition for the mix designs

Sample ID	Description
1B	Sediment +1% Bentonite
1B-5FA	Sediment, 1% Bentonite, 5% Flyash
1B-8FA	Sediment, 1% Bentonite, 8% Flyash
2B	Sediment, 2% Bentonite
3B	Sediment, 3% Bentonite

Table 3.8 - Summary of Tests Performed

Sample	Density Testing	Moisture Content	Viscosity	Filter Press Test	Adsorption Testing
Dredged Sediment	X	X	X	X_1	X
1B	X	X	X	X_2	X
2B	X	X	X	X_2	X
3B	X	X	X	X_2	X
1B-5FA				X_2	X
1B-8FA				X_2	X

X_1 – Test carried out to determine filtrate loss results
X_2 – Test carried out to determine hydraulic conductivity of mix

SECTION 4

TRENCH STABILITY

4.1 TRENCHING

Trenching is used for many construction operations today including controlling groundwater flow, contaminant migration, construction shafts, foundation pits, bridge foundations, basement walls, tunneling for subways, slope stabilization and sewage projects. Trenches can be stabilized using primarily braced stabilization or slurry stabilization. A few projects, involving the use of slurry walls, are shown in Fig. 4-1a, and a detail in Fig. 4-1b.

4.1.1 Stabilization Methods

In practice, two types of stabilization methods are commonly used; braced stabilization (involving the use of timber or steel supports with struts) and slurry stabilization which has become more popular in recent years. Table 4.1 shows a comparison of the effect of rigid lateral support and slurry support stabilization methods for plane strain, where the upper part is rigidly supported, as well as, unsupported axisymmetric excavations – the investigation was carried out under undrained conditions. Both wall failure and base failure are considered. In summary, it shows that the slurry support method is more effective than the rigid lateral support method for all failure conditions examined except for plane strain wall failure, refer to Table 4.1 (Brito and Kusakabe, 1984). The slurry support method reduces the amount of surface settlement and also stabilizes the trench against base failure. The slurry support method is, therefore,

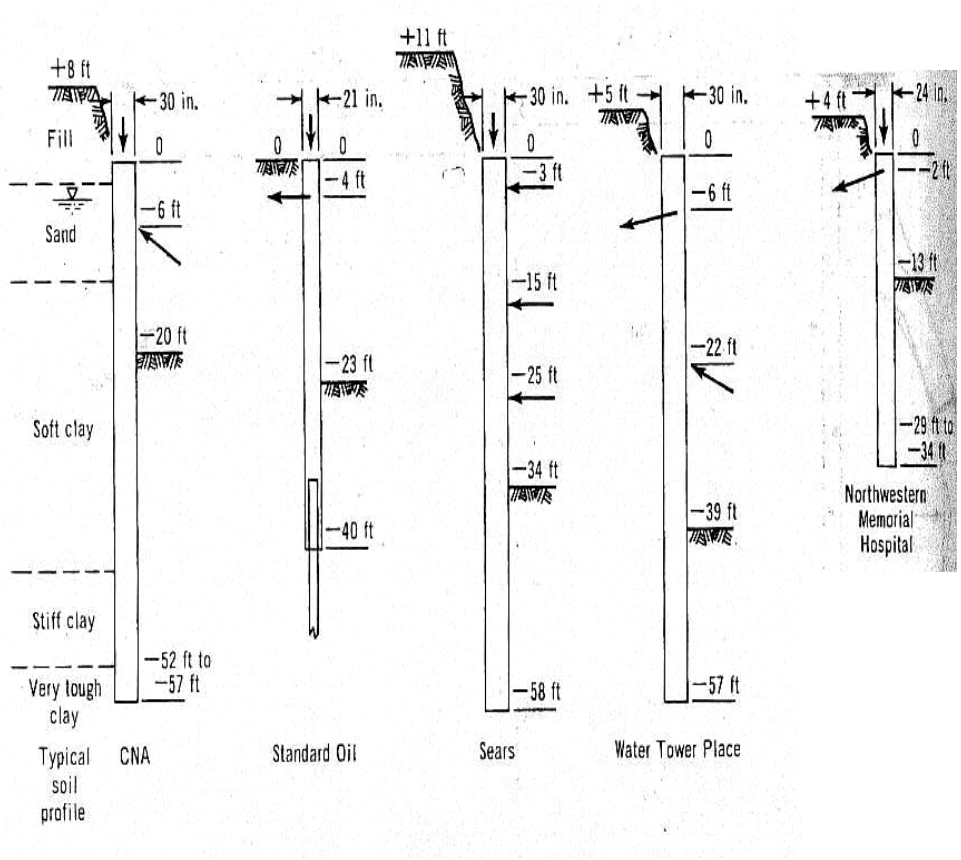

Fig. 4.1a - Slurry walls for projects in Chicago
(after Gill, 1980)

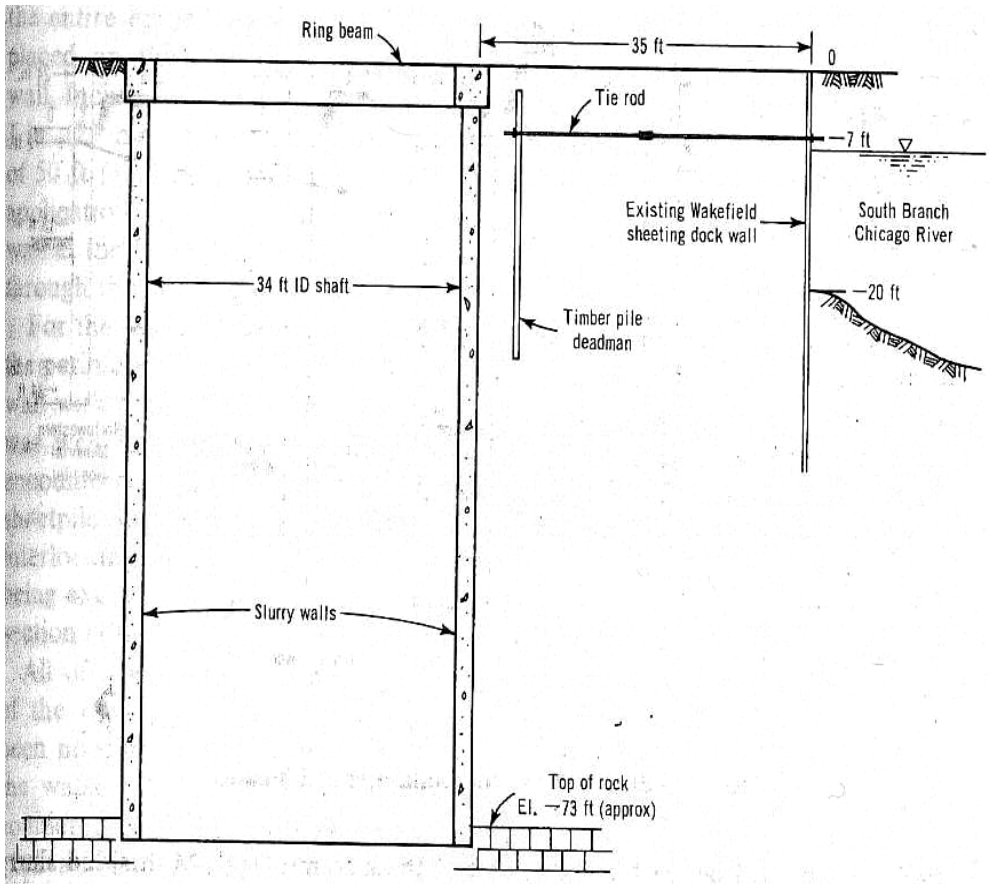

Fig 4.1b – Slurry Walls for Projects in Chicago
(after Gill, 1980)

Table 4.1 – Comparison of rigid wall support and slurry support for plane strain and axisymmetric excavations

Type		**Rigid Lateral Support**	**Slurry Support**
Plane Strain	Wall failure	More effective against wall failure	Less effective than lateral support*
	Base failure	No effect	Effective
Axisymmetry	Wall failure	Less effective than slurry	Effective
	Base failure	No effect	Effective

*Dependent on the density of slurry.

(after Brito and Kusakabe, 1984)

a preferred method of trench stabilization. The use of slurry wall projects has increased during the past two decades. Soil-bentonite slurry trench cutoff walls are most commonly used, because of their relatively low hydraulic conductivity and cost. In this study, the efficiency of a dredged sediment/ bentonite vertical backfill material (used to displace the slurry) are considered, therefore it is helpful to understand the design principles behind slurry stabilization.

4.2 SLURRY STABILIZATION

4.2.1 Design Principles

The main factors involved in slurry stabilization are change in strength of slurry with time and the theoretical build-up of filter cake with time on the walls of the trench. Slurry trench stabilization depends on the bentonite cake to prevent ground water flow and erosion of soil grains. A study by Nash (1974) revealed that the build-up depends on the square root of time (from original placement) and leads to a thickness of about ½ cm after 24 hr. for typical slurry and filter cake at a depth of 20m. Refer to Figure 4.2 and 4.3.

There is limited information on the effective stresses that exist within the soil-bentonite slurry trench cutoff wall in its as constructed condition. It is thought that a portion of the stability is given by incalculable forces such as gel strength, resistance of the filter cake, electric potential between the slurry and the soil, dynamic gradient of slurry flowing into the soil, rigidity imparted to the slurry by the bentonite as well as by the suspended cuttings and the effect of permeating slurry on the soil strength (Gill,

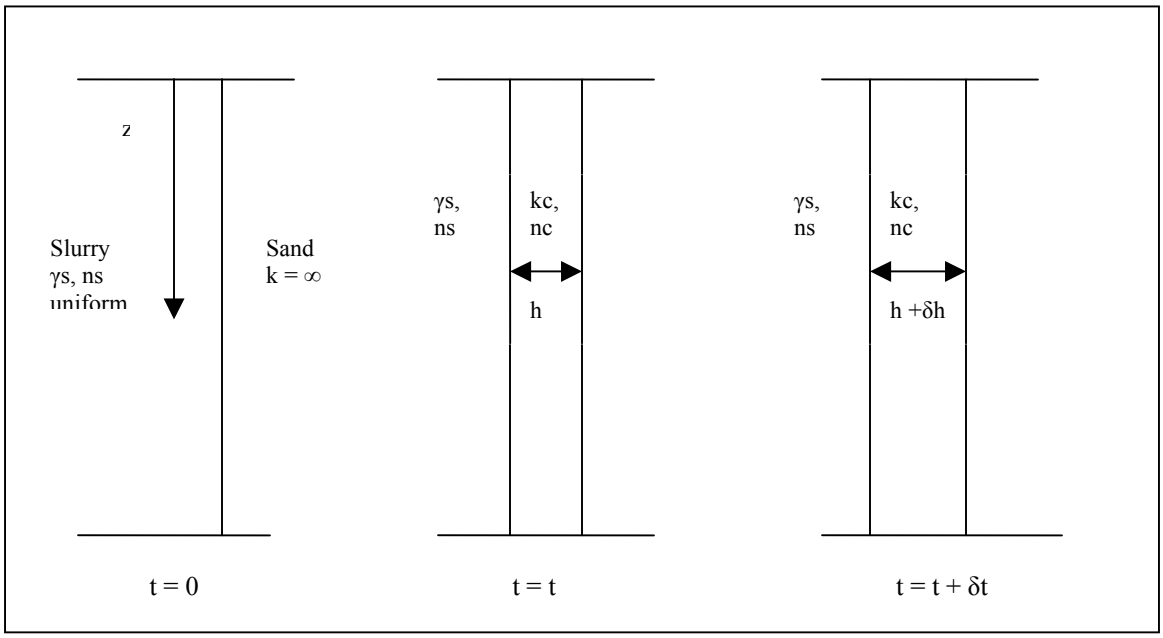

Fig 4.2 – Theoretical build-up with time of filter cake on walls of trench (after Nash, 1974)

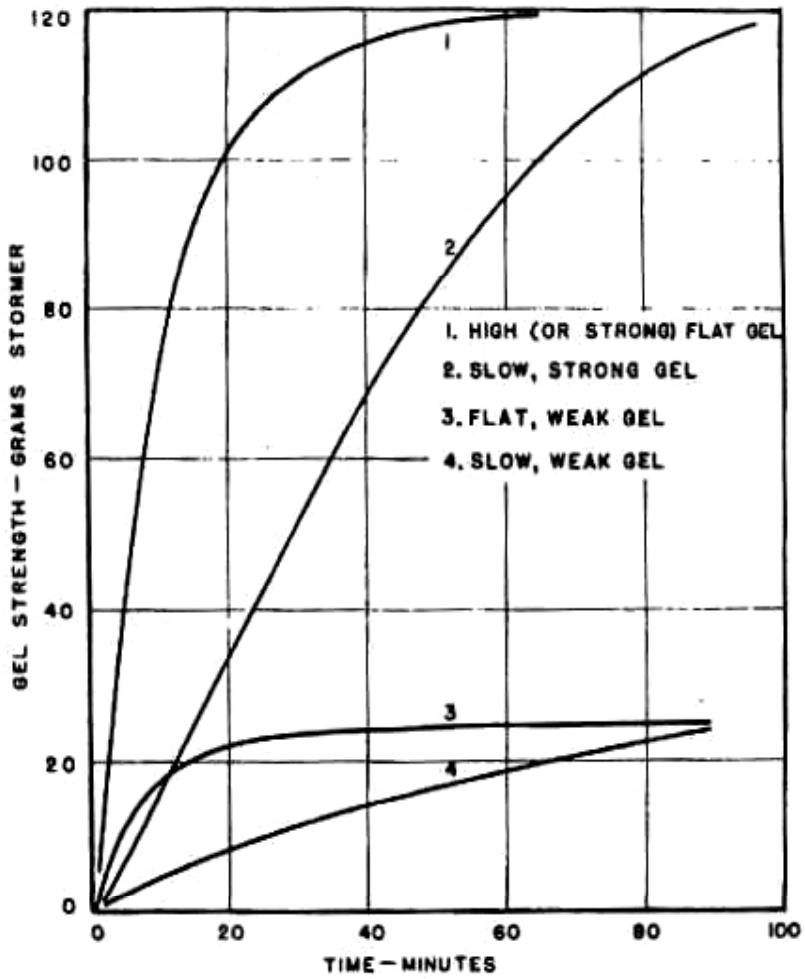

Fig 4.3 – Change in strength of slurry with time
(after Nash, 1974)

1980). However Evans et al (2000) has suggested that the state of effective stress does not increase geostatically. Sidewall friction forces "support" the backfill and the resulting

stress is usually less than the geostatic stresses. Data also show that after an initial non-linear increase, within a relatively short distance, the stresses at a given depth become essentially constant and predictable. By using these data and assuming a constant Cu/Po' ratio results in an estimate of the effective stress profile similar to that developed from consolidation testing. Using the predicted effective stress, the hydraulic conductivity (which is strongly stress dependent) can be more reliably determined.

The study by Nash (1974) shows the effect of slurry density. With reductions in slurry density (as well as excavation) there is horizontal movement of the sides of the trench as shown in Figure 4.4, Nash (1974) used the distribution of forces involved for the stability analysis, based on the Coulomb wedge theory with hydrostatic thrust against the vertical face and these diagrams are shown in Figure 4.5. The factor of safety, F, can be defined by the equations below:

For clays: $F = 4C_u / H(\gamma - \gamma_f)$ (4.1)

For sands and gravels: $F = 2(\gamma - \gamma_f)^{1/2} \tan ø_d / \gamma - \gamma_f$ (4.2)

where, C_u is the undrained cohesion, H is the depth of trench, γ is the total unit weight of the soil, γ_f is the total unit weight of the fluid mud and $ø_d$ is the drained friction angle of soil.

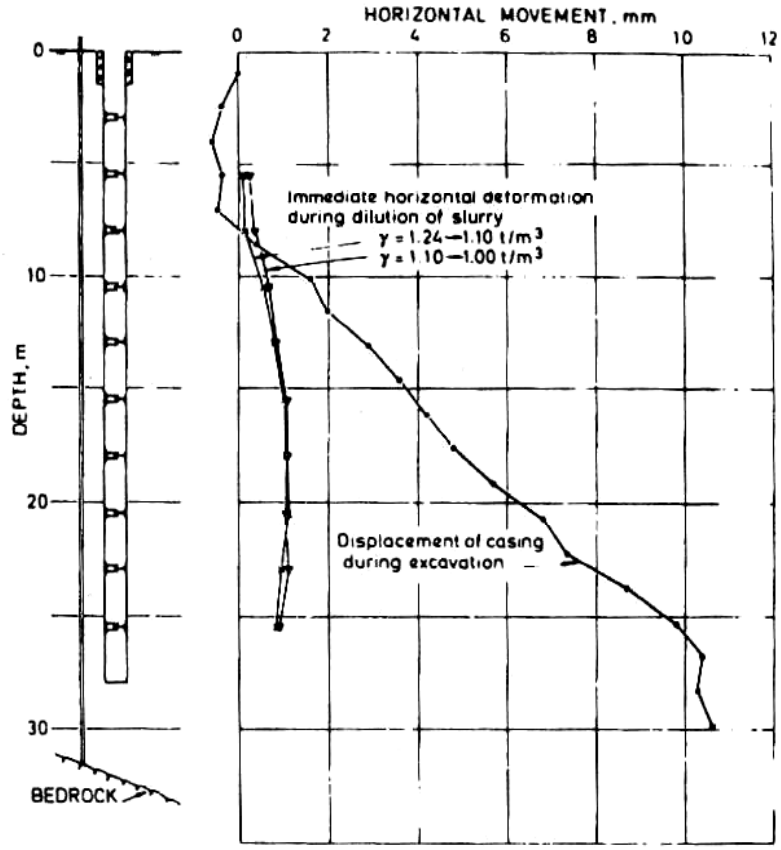

Fig 4.4 – Horizontal movement of sides of trench caused by excavation and reduction in density of slurry.

(after Nash, 1974)

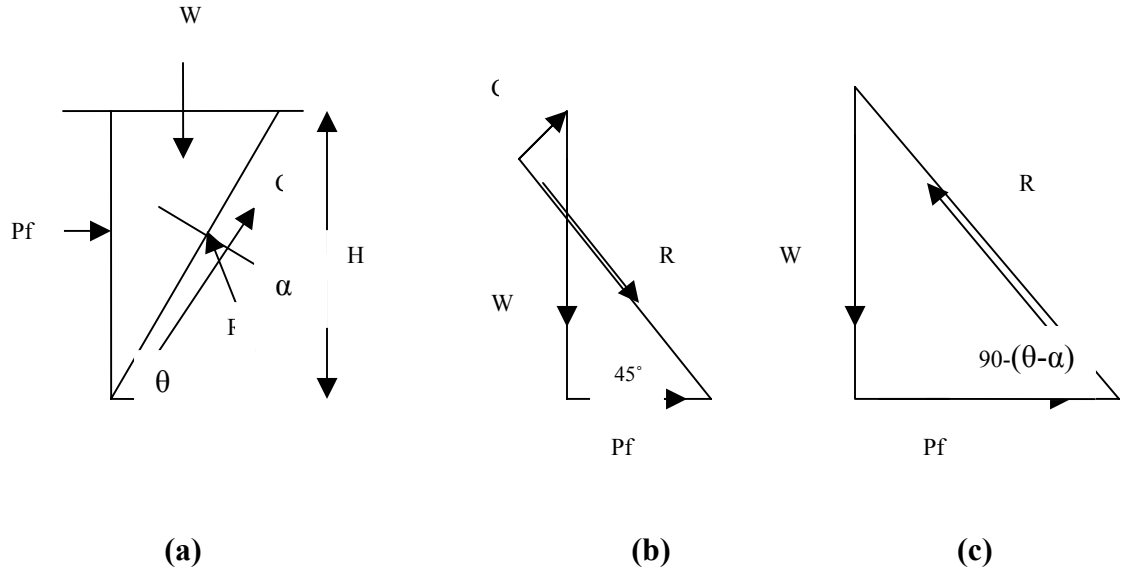

Fig 4.5 – (a) Stability analysis of slurry trench for c-ø soil (b) Polygon for forces when øu=0 (c) Triangle of forces when Cd=0

(after Nash, 1974)

Reliance is still mostly placed on experience in similar soil and ground water conditions. When there are unstable conditions, adjustments are made in the construction

procedures, level and density of the slurry, length of the panels, and in the shape and type of cutting tools (Gill, 1980). The presence of artesian water pressure, large gravel and boulders, very loose soils, soft clays, recently placed hydraulic fill with undissipated pore pressures, and sudden changes in the soil strata are potential conditions of instability and require careful consideration prior to deciding on the construction procedures.

Surcharge loads are more important when the slurry is located near to existing footings, the settlement of the structure that is supported should be considered.

4.2.2 Advantages of using slurry stabilization

There are several advantages that can be realized through using the slurry method of stabilization. These include minimal disruption of the surface, minimal construction noise and vibrations and positive cutoff for ground water. This method also eliminates underpinning of adjacent structures and these slurry walls can be constructed through soil and rock. Apart from having the slurry displaced by a soil-bentonite backfill material, the slurry method of stabilization can be used in combination with cast-in-place (C.I.P.) or precast concrete walls. This provides a rigid, smooth, watertight wall that can be used for earth retention purposes and serve as the permanent structure. The precast prestressed method also allows for large unsupported spans between bracing levels. It is ideal for use under dams, excavations in water bearing soils and containment of wastewater ponds and leachate from landfills. Specialist contractors are working on advances in the joint system for the future to eliminate the weak link and seepage path between panels. The use of slurry walls is projected to happen at an accelerated rate in the future.

4.2.3 Case histories for slurry stabilization – analyzed by the arching method

Trench stability is affected by factors such as electric potentials in the ground and adjacent building/foundation surcharges. Three case histories of slurry trench excavation including two failures were back-analyzed using the arching theory (Wong, 1984). The case histories analyzed were Gerstheim along the River Rhine, Charter Garden Test Panel in Hong Kong and Swire House in Hong Kong. The soil type was fill, marine deposit, and colluvium. From Prandtl's limiting plasticity theory, the distribution of the vertical pressure being exerted on soft clay, which mobilizes the driving lateral pressure, is calculated considering the arching effect induced by the finite length of the slurry trench.

In this process of designing a slurry trench for stability, a minimum value of the factor of safety (excess bentonite pressure/earth pressure) needs to be decided on, a value of 1.2 is commonly used. The geological parameters of cohesion (c) and friction angle (ø) and design ground water level need to be determined. Adjacent building foundations, foundation loads, and building superstructure should be studied for the magnitude and position of surcharge loads so that an allowable distortion and settlement can be decided on. A preliminary design of trench panel length, slurry density and slurry head then needs to be done. The reduction in surcharge load (if applicable) should then be determined. The (reduced) surcharge load should be applied to evaluate the earth pressure. The calculated earth pressure should then be compared with the effective slurry pressure to obtain the factor of safety. If the factor of safety is inadequate, the trench stability should be redesigned by using a higher slurry density, higher slurry head, shorter panel length, lowering the ground water table or strengthening adjacent buildings to increase stiffness.

The findings of the case histories revealed that failure usually occurred when the slurry head in the trench fell to approximately 1m below ground water level. The point at which the earth pressure became greater than the excess (over ground water pressure) slurry pressure was termed the critical depth. The local failure propogated, and a general failure occurred with the slip surface starting from the critical depth. Therefore the failure zone extended only to a depth of 5-12m, although at the time of failure, the trenches were excavated over 20m. The failure occurred progressively. It is thought that failure progresses upward from the critical depth to form a sliding soil mass. The analysis showed that for typical cases where the critical depth is 5-12m; because of the arching effect, the failure zone extended laterally on the ground surface to a distance of half the panel length behind the trench face. It was also found that slurry trenches subjected to surcharge load will develop very large earth pressures near the point of application. When the point of application is within half-panel length of the trench, the surcharge will act totally on the trench wall. Surcharge located more than half-panel length away from the trench will be distributed to soil adjacent to the panel.

4.2.4 Failure analysis based on lateral extrusion of weak soil

The failure based on the lateral extrusion of sandwiched weak soil in a slurry-supported trench was evaluated through use of a field case study in Southwest Taiwan, refer to Figure 4.6, (Tsai et al, 1998). The problem was theoretically modeled as the compression of weightless soft clay between rough plates. It was evaluated using a factor of stability. The factor of stability is defined as the ratio of the slurry pressure to the (horizontal) lateral extrusion pressure at the level of the weak soil.

The total stress concept was used. Important factors for the stability of the weak sublayer include the undrained shear strength of weak soil, the slurry pressure being exerted on the trench face and the ground water level. A theory on the bearing capacity of a thin clay layer of soil along with the theory of soil arching was used (using the modified Schneebeli's formula, (Schneebeli 1964, Wong 1984)) to calculate the vertical driving pressure on the top boundary of the weak soil. The analysis was performed by assuming a limiting equilibrium state where both the strength and the arching of soil are fully mobilized. Limiting equilibrium was considered for the lateral pressure being exerted on the open end of a weak sublayer. It was realized that an approximate, simplified, but realistic solution could therefore be arrived at. Figure 4.7 shows the forces acting on a representative soil element for lateral extrusion analysis. And Figure 4.8 shows the arching effect on a sandwiched weak sublayer in a slurry trench.

From the study, if the vertical driving pressure is greater than the threshold pressure ($1.57S_u$), the factor of stability is given by:

$$FS = P_{sl}\text{-}0.43S_u/\sigma_v\text{-}1.57S_u \qquad (4.3)$$

If the vertical driving pressure is equal to or less than the threshold pressure ($1.57S_u$), the factor of stability is given by:

$$FS = Psl/0.43S_u \qquad (4.4)$$

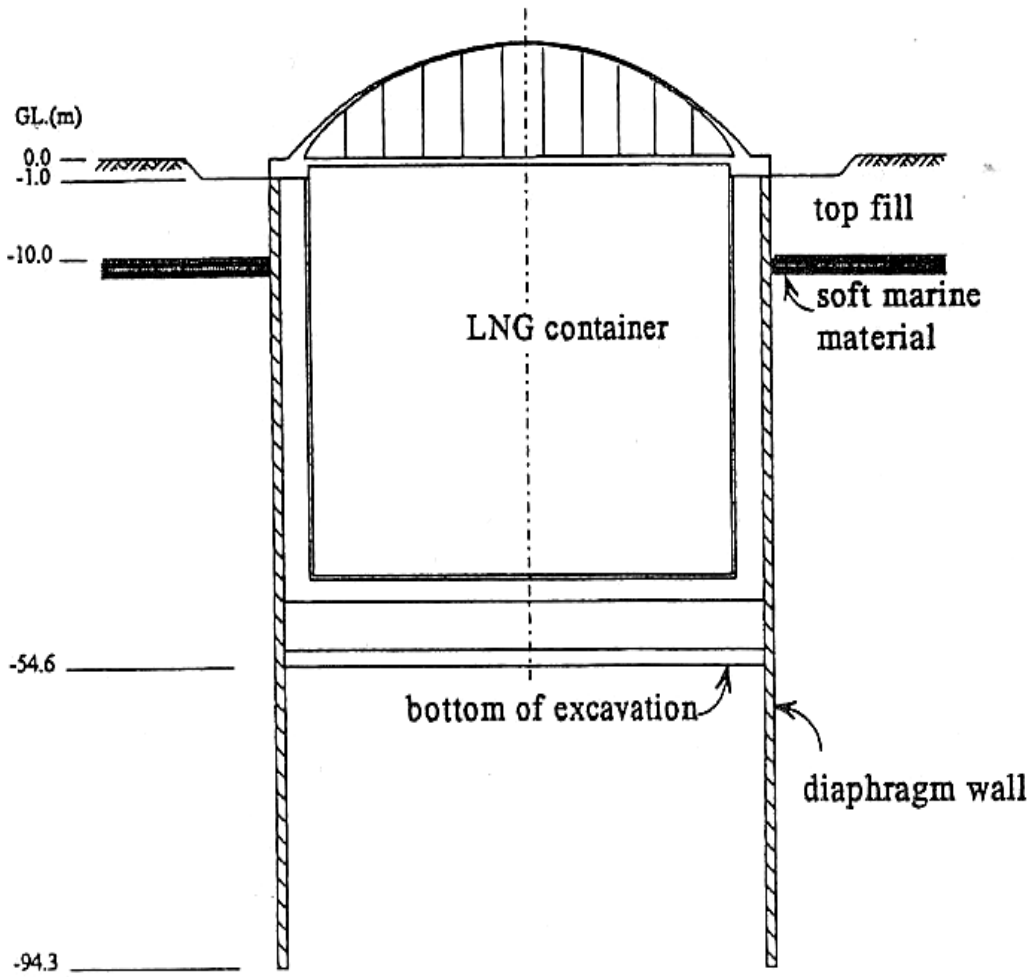

Fig 4.6 – Layout of container and slurry trenches

(after Tsai et al, 1998)

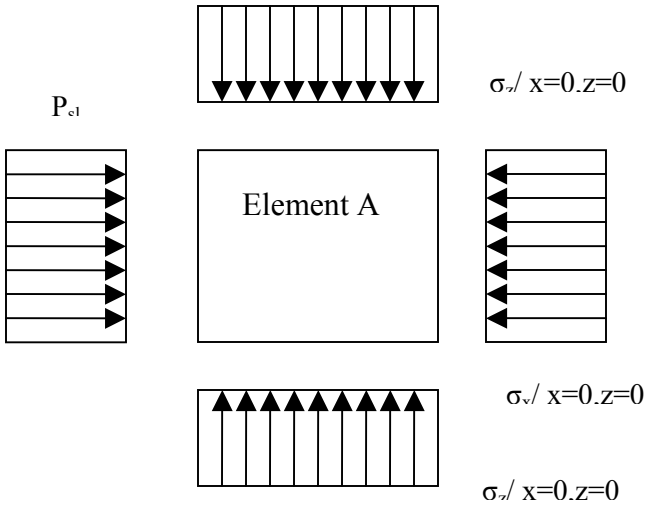

Fig 4.7 – Representative soil element for lateral extrusion analysis

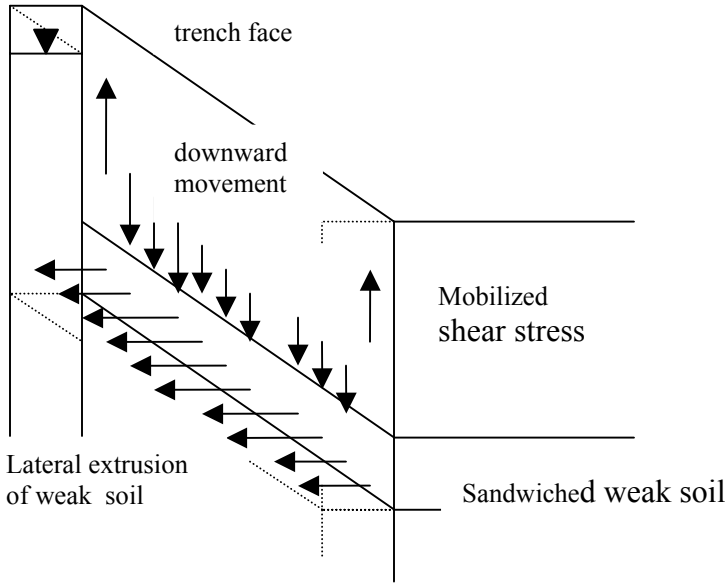

Fig 4.8 – Arching effect upon sandwiched weak sublayer in slurry trench

(after Tsai et al, 1998)

where, P_{sl} is the slurry pressure, S_u is the undrained shear strength and σ_v is the vertical driving pressure. The application of the above method for a given site gives favorable agreement with field observations. Figure 4.9 a-d shows a failure scenario involving the lateral extrusion of a weak layer of soil. Table 4.2 shows the simplified soil profile of the site and Table 4.3 shows the factors of stability of a weak sublayer.

4.2.5 Comparison of Analysis Methods and Limitations

In the current practice of slurry trench design, the earth pressure exerted on the trench wall at various depths is calculated by the Huder or Schneebeli formula. The estimated earth pressures are compared with the excess bentonite pressure at the corresponding depth. If the factor of safety (excess bentonite pressure/earth pressure) at each depth is more than 1.2, it is stable overall. From back-analysis of several case histories (Wong, 1984), the Schneebeli's method of analysis adequately predicts trench stability. It is thought that the Schneebeli's method can be better applied to practical problems than the Huder method. The Schneebeli's method, however, usually results in earth pressure lower than the Huder's method (using Huder's recommended earth pressure coefficient). The Huder's method is therefore conservative for the case histories analyzed. The wedge method of analysis is thought to be less conservative than the arching methods because it considers global equilibrium of the failure mass. The finite element method can be used to solve the problem as well.

Practically, a complete analysis of a soil mass loaded to failure is a very complicated problem. It deals with an elastic-plastic-rupture transition, which involves an initial linear elastic state, a post yielding state, a near-failure state, and the post-failure

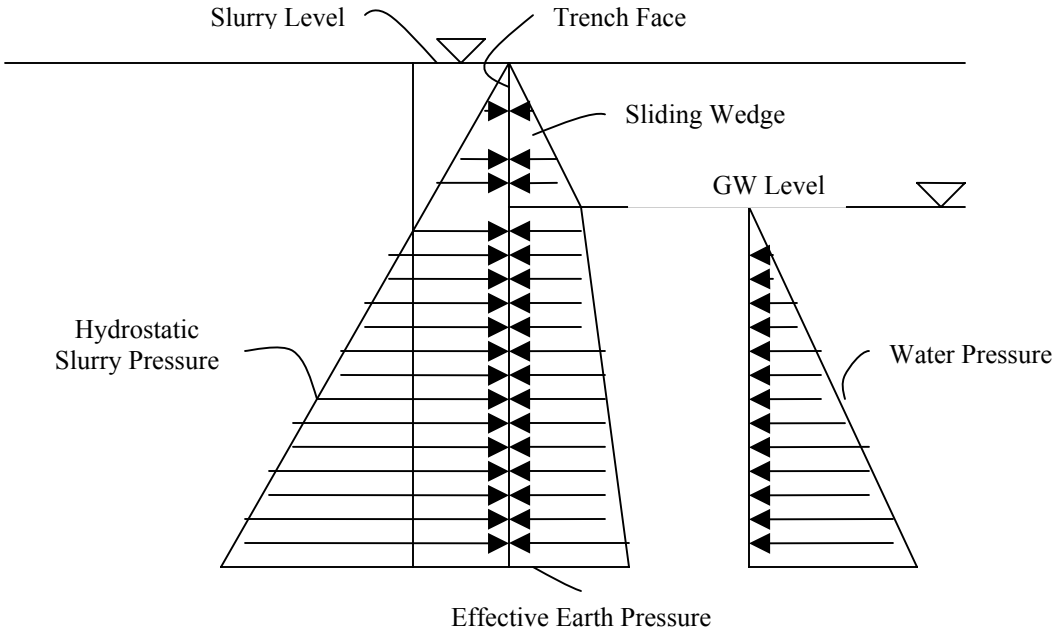

Fig 4.9a - Equilibrium of Pressure along Trench Face for Stability of Slurry Trench

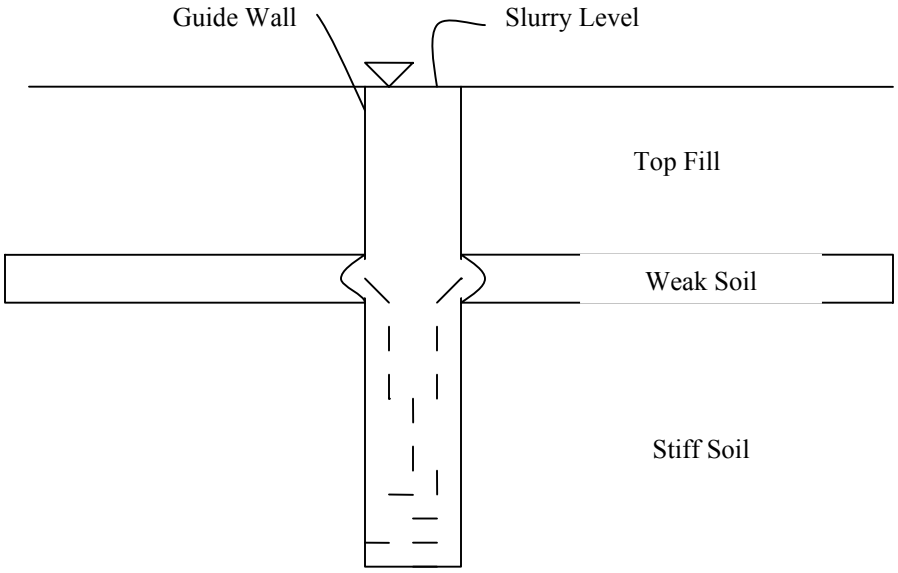

Fig 4.9b – Falloff of Sandwiched Weak Soil

(after Tsai et al, 1998)

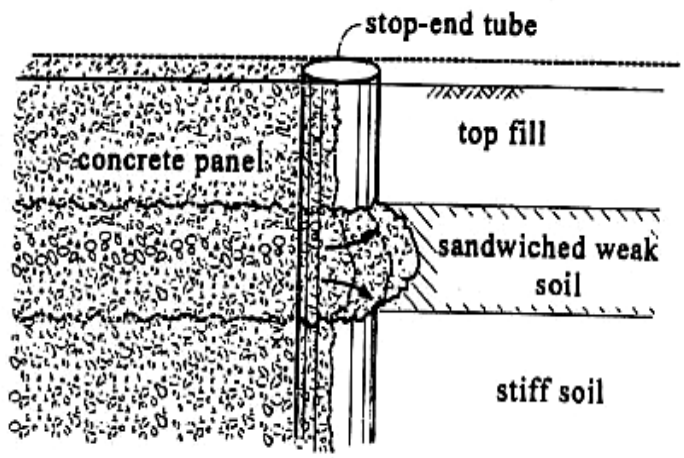

Fig 4.9c – Penetration of Concrete beyond Stop-End Tube due to Falloff at Sandwiched Weak Soil

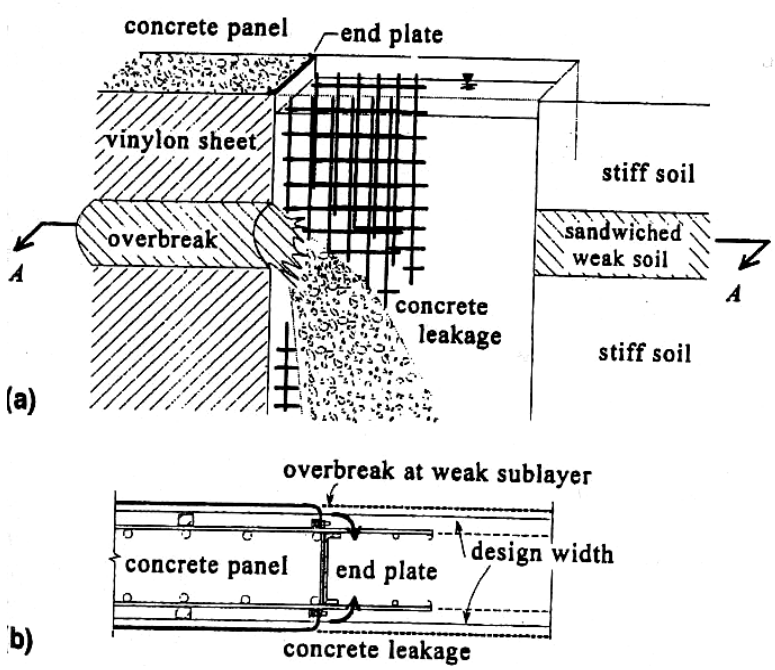

Fig 4.9d – Vinylon Sheet and Steel-Plate Joint: (a) Concrete Leaking into Overlapping Section through Broken Vinylon Sheet, (b) Plan View of Overlapping Joint

(after Tsai et al, 1998)

Table 4.2 – Simplified soil profile of site

Layer	Depth (m)	USCS	γ_t (kN/m³)	γ_{sat} (kN/m³)	C_u*	$Ø_u$*	PI	S_u (kPa)
1	0-10.0	SM/fill	17.0	19.6	-	33	-	-
2	10.0-11.5	CL	-	19.0	-	-	10	20.0
3	11.5-15.0	SM	-	19.2	-	38	-	-
4	15.0-22.0	CL	-	20.0	28.0	14	25	-
5	22.0-28.0	SM	-	20.0	-	35	-	-
6	28.0-30.0	CL	-	19.8	86.0	18	16	-
7	>30.0	SM	-	20.1	-	37	-	-

USCS is the Unified Soil Classification System
C_u and $ø_u$ are the strength parameters and results of consolidated undrained triaxial tests

Table 4.3 – Factors of stability of weak sublayer

Trench length (m)	Analytical method	K_z	P_{sl} (kPa)	P_a (kPa)	P_w (kPa)	S_u (kPa)	σ_v (kPa)	FS	Remark
L=10.2	Proposed	-	94.5	-	-	20.0	141.3 (142.0)	0.78	Falloff
	Rankine's	0.27	94.5	31.4	70.0	-	186.2 (193.0)	0.92	Falloff
	Hajnal's	0.27	94.5	21.2	70.0	-	148.6 (149.3)	1.04	Falloff
L=4.0	Proposed	-	94.5	-	-	20.0	110.6 (111.3)	1.08	Stable
	Rankine's	0.27	94.5	31.4	70.0	-	186.2 (193.0)	0.93	Stable
	Hajnal's	0.27	94.5	13.3	70.0	-	119.1 (119.8)	1.13	Stable

K_d value used in Rankine's and Hajnal's methods is estimated by effective shear strength parameters ø' = 35˚, which is correlated with PI of weak sandwiched soil according to data of some normally consolidated natural and remolded clays (Kenney 1959).
Values in parenthesis are vertical driving pressures including self-weight of weak soil. They show that influence of self-weight of weak soil is negligible in the proposed method.

(after Tsai et al, 1998)

state. Therefore for the latter case study, the proposed limit analysis is a great simplification of the true behavior of the soil.

4.2.6 Summary

In summary, the slurry stabilization of trenches is a widely used method, relatively cost effective, performs well and provides for versatile uses. Furthermore, the prospect of using dredged sediments as the soil in soil-bentonite backfill material should not pose any problems, provided that the adequate design and construction procedures are adhered to.

SECTION 5

HYDRAULIC CONDUCTIVITY TEST RESULTS

5.1 TESTING ON DREDGED SEDIMENT AND SEDIMENT MI X

Preliminary testing on the dredged sediments was done to determine suitable moisture content and corresponding viscosity of the sediment, that could then be used in the mix design. The sediment was tested for density, moisture content, viscosity and filtrate loss.

Similar tests were performed on the sediment mixes, made with 1%, 2% and 3% of bentonite by weight. These tests were carried out to determine the effect of bentonite on mud weight densities, moisture content, viscosity, filtrate loss and hydraulic conductivity. Some of the mixtures prepared with 1% bentonite were modified with the addition of fly ash, to determine the effect of fly ash on hydraulic conductivity.

5.1.1 Results of preliminary testing on dredged material.

Density testing of the dredged sediments revealed an average density of 10.8kN/m³ (68.49 pcf), which compares to results from a study conducted on dredged harbor bottom sediments by Vaghar et al (1997), which gave unit weight values in the range of 11.70-12.32 kN/m^3 (74.2-78.1 pcf). The mud weight density of the Baltimore Harbor dredged sediments also falls within the range of the density of bentonite slurries (64-80pcf).

The moisture content of the sediment ranged between 400-600 %. For the dredged material to be in the upper allowable viscosity range of 32-40 s, the moisture content was

on the lower end of its range, and when the viscosity was in the lower end of the allowable range, the moisture content was on the upper end of its range.

The filtrate loss test resulted in a volume filtrate of 23.8mL collected after 30 min, 50 ml after 1½ hours, and a total volume of filtrate of 375mL collected after an estimated time of 11hrs. 13min. The gradient of the filtrate loss curve was 8.6×10^{-3} mL/s. The filter cake thickness was 10 mm, it was smooth and black in color.

5.2 EFFECT OF BENTONITE ON SEDIMENT MIX

5.2.1 Effect of Bentonite on Mud Weight Densities & Moisture Content

The results of the mud weight densities revealed that with increasing bentonite content, the mud weight density increased. This is an expected trend. Refer to Table 5.1.

With increasing bentonite content there was decreasing moisture content. Bentonite , a montmorillonite clay, usually swells in the presence of water, therefore it is expected that the bentonite introduced would absorb some of the water in making the homogeneous mixture. Table 5.2 and Figure 5.2 show the resulting trend.

5.2.2 Effect of Bentonite on Viscosity

The results showed that the viscosity increased as the bentonite content increased. An initial water content which gave a viscosity measure on the lower end of the suitable range (32-40 s) was used. It was not difficult to adjust the water content, so that with the initial addition of bentonite a higher or lower viscosity could be attained. Also, the sediment-bentonite mix was workable, even though there was some difficulty in attaining a homogeneous mixture in the laboratory due to the nature of the materials. As bentonite is a type of clay and denser than the dredged sediments, it is expected that the mixture,

Table 5.1 – Mud Weight Densities

Mud Weight Density (kN/m³)	
	Sample A
Bentonite	Test Series
Content (%)	#1
1	10.66
2	10.82
3	10.87

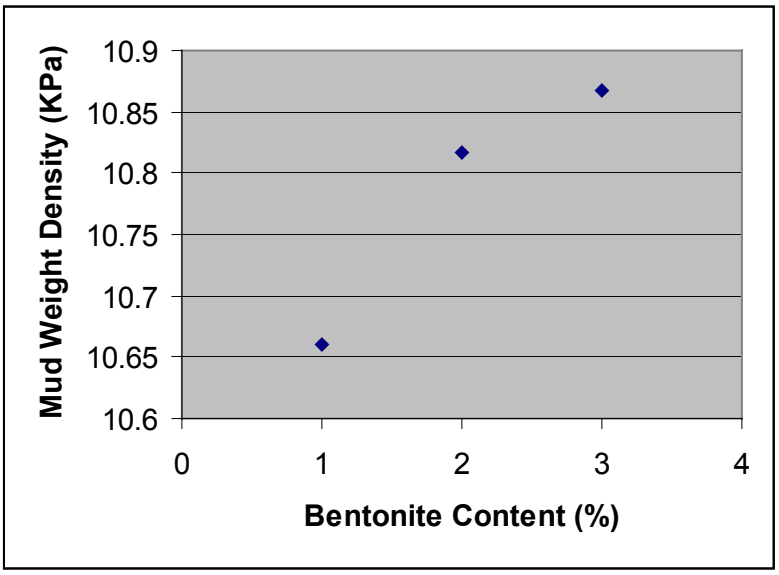

Fig 5.1 – Effect of Bentonite on Mud Weight Density

Table 5.2 – Moisture Content Results

	Moisture Content (%)	
	Sample A	**Sample B**
Bentonite	Test Series	Test Series
Content (%)	#1	#2
1	583.4	531.5
2	552.9	494.0
3	517.0	462.4

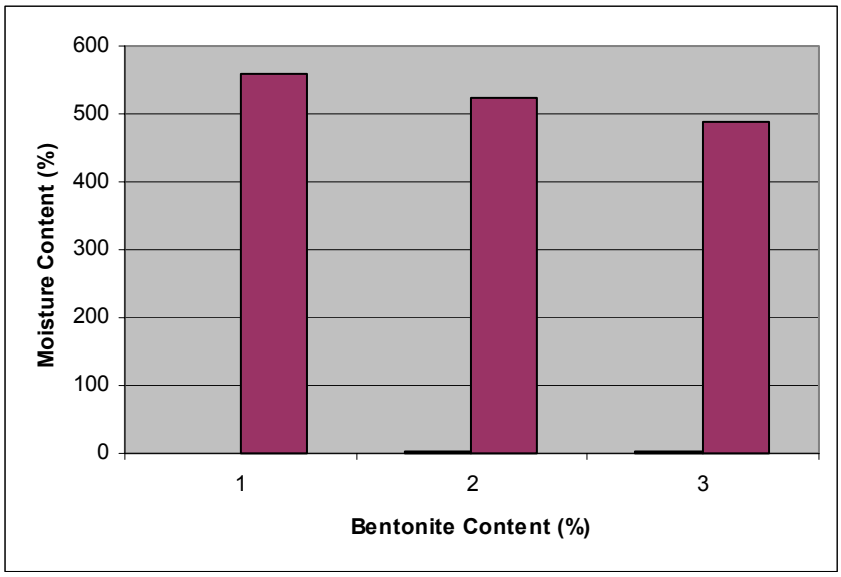

Fig 5.2 – Effect of Bentonite Content on Moisture Content

will become more viscous with increasing bentonite content. Table 5.3 and Figure 5.3 show the values measured with increasing bentonite content.

5.2.4 Effect of Bentonite Content on Hydraulic Conductivity

The principal factor in the performance of vertical barrier systems is the hydraulic conductivity (Evans, 1994). Increasing bentonite content resulted in decreasing hydraulic conductivity. Comparable results have been found in studies by Evans, (1994). It was found that increasing the bentonite content in a vertical barrier will decrease the hydraulic conductivity in soil-bentonite and in-situ mixed walls; there may, however, be an optimum. For a particular mix used, the minimum hydraulic conductivity was found at a bentonite content of about 3%. A similar trend of decreasing hydraulic conductivity with increasing bentonite content was found in a study done by D'Appolonia (1980). The hydraulic conductivity of soil-bentonite used in vertical barrier construction is typically between $1x10^{-7}$ cm/s and $1x10^{-8}$ cm/s (Evans, 1994).

The hydraulic conductivity, k in cm/s, was calculated by use of the following equation:

$$k = (q/h) \times t \qquad (5.1)$$

where q is the ratio of the flow in mL/s and the area of the apparatus (cm^2), h is the pressure head converted to cm, and t is the thickness of the filter cake formed. The permeability was then normalized through dividing by the relevant filter cake thickness.

Tables 5.4 through 5.6 and the corresponding Figures 5.4 through 5.6 show the effect of bentonite content on hydraulic conductivity at given applied pressures. The

Table 5.3- Marsh Funnel Test Results

Marsh Funnel Viscosity (s)	
Bentonite	Test
Content (%)	Series #1
1	36.53
2	39.39
3	42.05

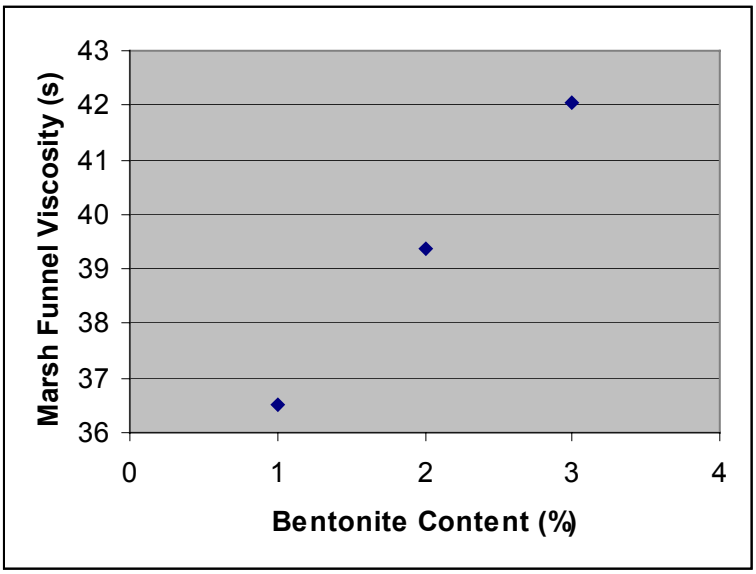

Fig. 5.3 – Marsh Funnel Viscosity versus Bentonite Content for Different Bentonite Slurries.

Table 5.4 – Effect of Bentonite Content on Hydraulic Conductivity, σ'= 48.3kPa

Specimen ID	Mix Type	Q (ml)	Flow rate (ml/s)	Kc	Kc/tc
N	B1	200	0.101	3.78E-07	4.20E-06
M	B1	195	0.100	4.16E-07	4.16E-06
E	B2	91	0.040	1.98E-07	1.65E-06
A	B3	324	0.011	4.48E-08	4.48E-07

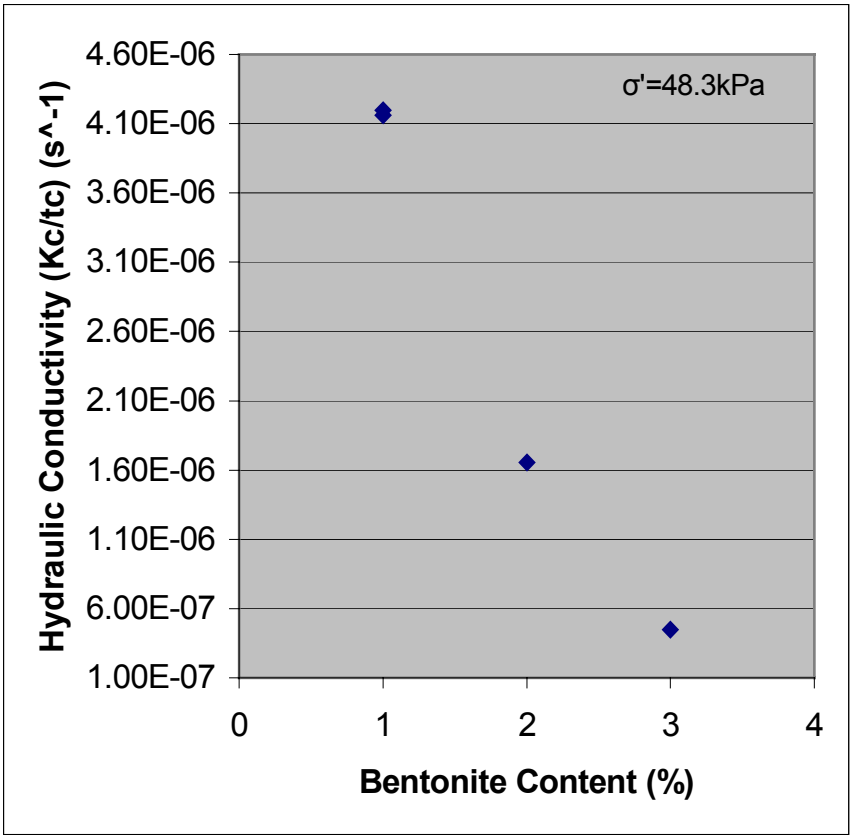

Figure 5.4 – Effect of Bentonite Content on Hydraulic Conductivity, σ'=48.3kPa

Table 5.5 - Effect of Bentonite Content on Hydraulic Conductivity, σ' = 13.79kPa

Specimen ID	Mix Type	Q (ml)	Flow rate (ml/s)	Kc	Kc/tc
P	B1	200	0.069	1.00E-06	1.00E-05
O	B1	37	0.100	1.01E-06	1.45E-05
F	B2	299.7	0.076	5.51E-07	1.10E-05
T	B3	44	0.032	4.61E-07	4.61E-06

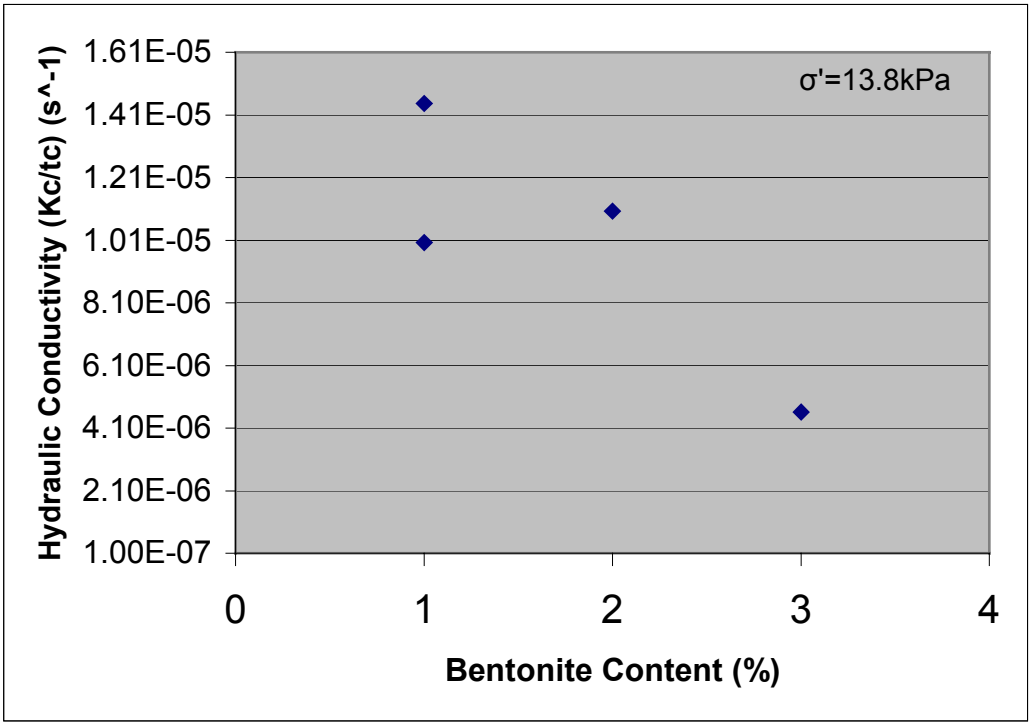

Figure 5.5 – Effect of Bentonite Content on Hydraulic Conductivity, σ'= 13.8kPa

Table 5.6 – Effect of Bentonite Content on Hydraulic Conductivity, σ'= 6.9kPa

Sample ID	Mix Type	Q (ml)	Flow rate (ml/s)	Kc	Kc/tc
H	1B	50	0.072	2.31E-06	2.1E-05
K	1B	195	0.071	2.87E-06	2.05E-05
Y	1B	45	0.099	3.17E-06	2.88E-05
R	1B	200	0.127	2.21E-06	3.69E-05
L	2B	200	0.069	9.98E-07	2.00E-05
S	2B	200	0.052	1.50E-06	1.50E-05
X	3B	287	0.010	3.11E-07	2.82E-06

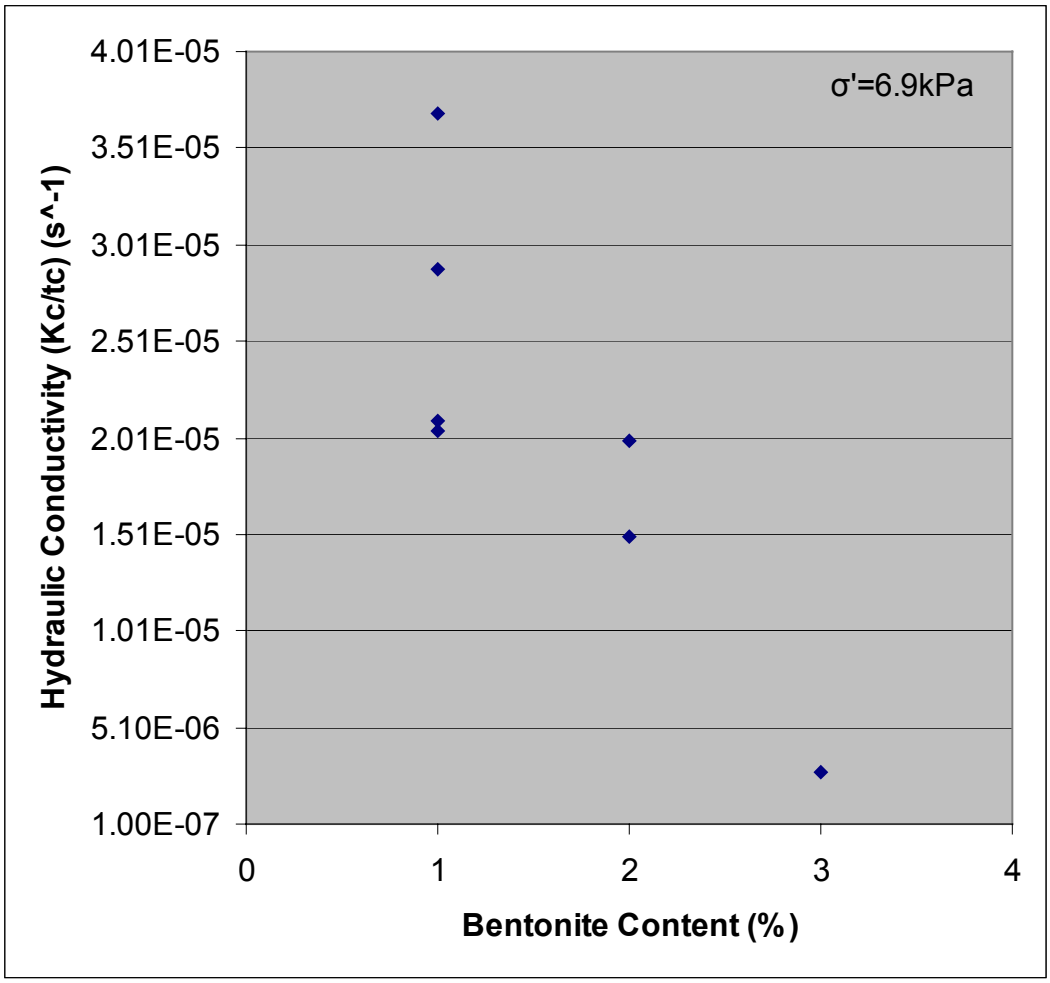

Figure 5.6 – Effect of Bentonite Content on Hydraulic Conductivity, σ'= 6.9kPa

hydraulic conductivity achieved was also generally lower for the same mixes, when greater pressure was exerted. This is expected as with increasing effective stress, the void ratio decreases and so would the hydraulic conductivity. A study by Evans (1994) showed similar results. For any given sample of vertical barrier material, the hydraulic conductivity decreases as the effective consolidation pressure increases.

5.3 – EFFECT OF FLY ASH ON SEDIMENT MIX

5.3.1 – Effect of Fly Ash on Hydraulic Conductivity

With increasing fly ash content, and the same base mixture, the hydraulic conductivity of the mix was increased. This can be theoretically explained in that as the fly ash attaches itself to the fines present, the mixtures resemble a more granular structure, hence increasing the void ratio and hydraulic conductivity. Tables 5.7 through 5.9 and the corresponding Figures 5.7 through 5.9 show the effect of fly ash on hydraulic conductivity at given pressures.

5.4 – FILTRATE LOSS RESULTS FROM SEDIMENT MIX

There does not seem to be a particular trend with the filtrate loss curves, apart from the fact that those developed at 1% bentonite and different pressures have a more irregular shape than those developed at 2% and 3% bentonite. The filtrate loss results for the bentonite specimens tested ranged from 2.27 mL/s to 0.04 mL/s. The filtrate loss results for the fly ash specimens tested ranged from 14.29 mL/s to 1.54 mL/s. Figure 5.10 is given as an example to show the filtrate loss with time. The tests shown were

Table 5.7 – Effect of Fly Ash on Hydraulic Conductivity, σ'= 48.3kPa

Sample ID	Mix	Q (ml)	Flow rate (ml/s)	Kc	Kc/tc
B	1B-5FA	336	0.383	7.93E-07	1.59E-05
W	1B-8FA	20	1.333	7.18E-06	5.52E-05

Note: Pressure is 48.3kPa and base mixture is 1% bentonite

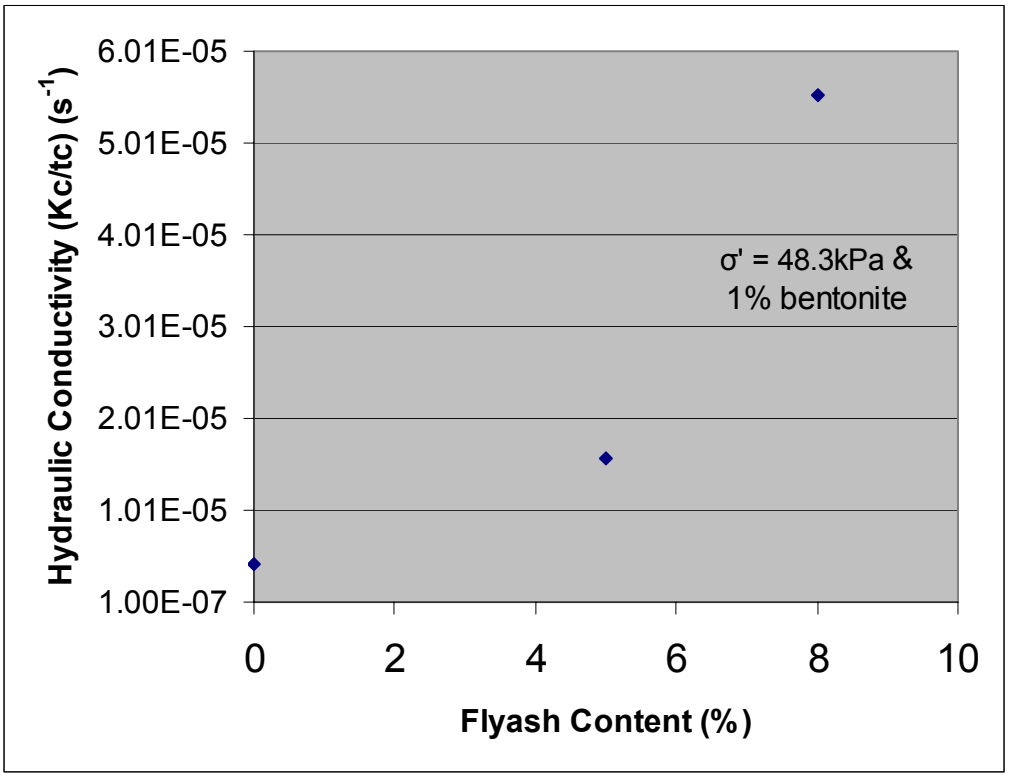

Figure 5.7 – Effect of Fly Ash on Hydraulic Conductivity, σ'= 48.3kPa

Table 5.8 – Effect of Fly Ash on Hydraulic Conductivity, σ'= 13.8kPa

Sample ID	Mix Type	Q (ml)	Flow rate (ml/s)	Kc	Kc/tc
I	1B-5FA	263.5	0.151	2.20E-06	2.2E-05
C	1B-8FA	259	0.778	1.13E-05	1.13E-04

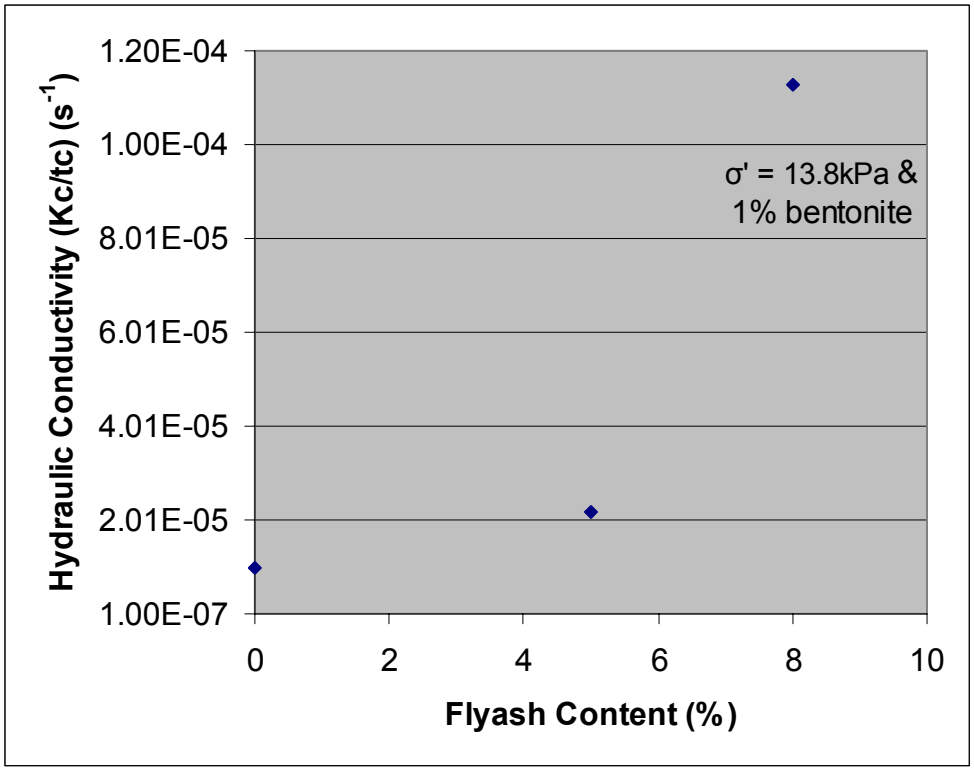

Figure 5.8 – Effect of Fly Ash on Hydraulic Conductivity, σ'= 13.8kPa

Table 5.9 – Effect of Fly Ash on Hydraulic Conductivity, σ'= 6.9kPa

Sample ID	Mix Type	Q (ml)	Flow rate (ml/s)	Kc	Kc/tc
D	1B-5FA*	318	0.530	1.08E-05	1.54E-04
J	1B-8FA	340	0.567	1.64E-05	1.64E-04
G	1B-8FA*	259	0.778	5.64E-06	1.13E-04

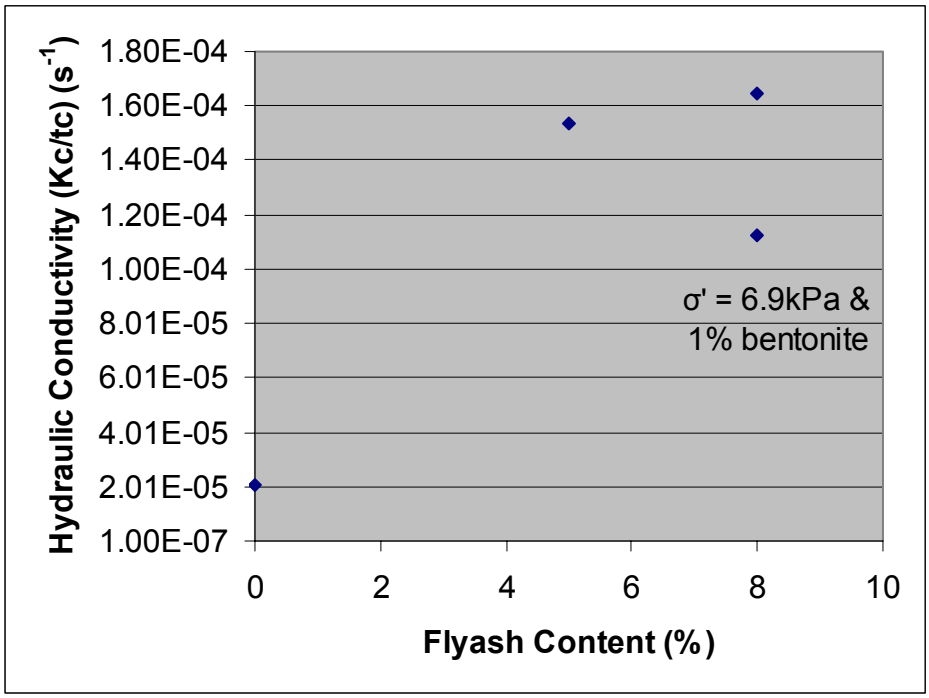

Figure 5.9 – Effect of Fly Ash on Hydraulic Conductivity, σ'= 6.9kPa

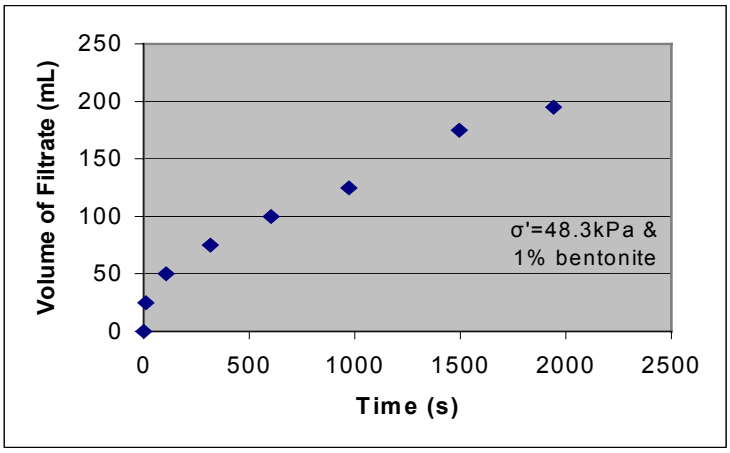

Figure 5.10a – Filtrate loss for specimen at σ'= 48.3kPa and 1% bentonite

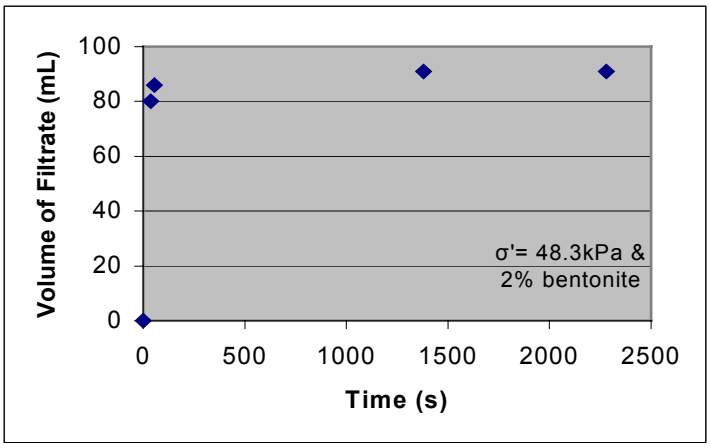

Figure 5.10b – Filtrate loss for specimen at σ'= 48.3kPa and 2% bentonite

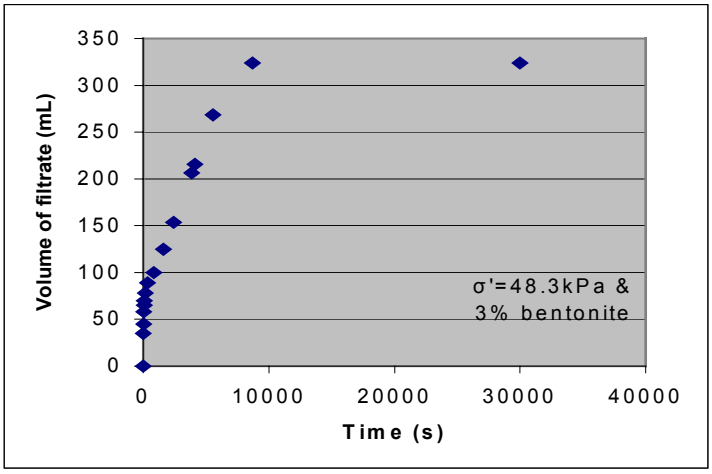

Figure 5.10c – Filtrate loss at σ'= 48.3kPa and 3% bentonite

conducted under a pressure of 48.3kPa. Figure 5.10a shows the filtrate loss (gradient) of 0.21 mL/s at 1% bentonite. Figure 5.10b gives a gradient of 1.54 ml/s at 2% bentonite and Figure 5.10c gives a gradient of 0.04 mL/s at 3% bentonite. In a study by D'Appolonia (1980), it was said that filtrate loss properties of the slurry have a minor influence on (Kc/tc). It was also said that slurries having a high filtrate loss develop a thicker but more pervious filter cake than slurries having a low filtrate loss. As a result, the ratio (Kc/tc) is relatively unchanged.

5.5 – SYNTHESIS

The dredged sediments were put through preliminary testing of mud weight density, moisture content, marsh funnel viscosity and filtrate loss to determine properties and a suitable moisture content and viscosity that could be used in the mix design.

Upon the addition of 1%, 2% and 3% bentonite by weight, several tests were performed on the mix to determine the effects of the bentonite added. The effects on density, moisture content, Marsh funnel viscosity were examined. Upon the addition of 5% and 8% fly ash by weight to the 1% bentonite mix, hydraulic conductivity and filtrate loss were examined for all the mixes.

The conclusions that can be drawn are as follows:

- The mud weight density of the dredged sediments 10.77KN/m³ (68.49 pcf) falls within the range of bentonite slurries (64-80pcf).
- A suitable moisture content and viscosity of the dredged sediments can be attained that will make it usable in the mix design.

- Increasing bentonite content leads to an increase in mud weight density. This is helpful as the backfill material must be denser than the slurry in order to displace it in the construction process.
- An increase in bentonite content allows for decreased moisture content and correspondingly increased viscosity.
- Increasing bentonite content, to the percent tested (3%), resulted in a decrease in hydraulic conductivity. The optimum range to serve the purpose of use as a vertical cutoff wall ($1x10^{-7}$ to $1x10^{-8}$) was near realized ($4.48x10^{-7}$) at 3% bentonite and an applied pressure of 48.3kPa.
- Increasing the fly ash content to the percent tested (8%), (with the base as dredged sediments and 1% bentonite by weight), increased the hydraulic conductivity.
- The filtrate loss results did not show any distinct trends. The filtrate loss on the bentonite specimens tested ranged from 2.27mL/s to 0.04mL/s. The filtrate loss on the fly ash specimens tested ranged from 14.29mL/s to 1.54mL/s.
- Dredged sediments can therefore serve the purpose of inhibiting the flow of ground water which is the most important function of a vertical cut-off wall.

SECTION 6

ADSORPTION RESULTS

6.1 ADSORPTION TESTING

Equilibrium batch testing, according to ASTM D4646 specification was performed on the sediment mixes and aqueous solutions containing heavy metals to determine the adsorption characteristics of the sediment mix. The mixes were made with 1%, 2% and 3% of bentonite by weight. These tests were carried out to determine the effect of bentonite on the adsorption capacity of the mix. Some of the mixtures prepared with 1% bentonite were modified with the addition of fly ash, to determine the effect of fly ash on its adsorption capacity. Fly ash was added in the proportions of 5% and 8% by weight. Typical field values of thickness of barrier (L), hydraulic gradient (i) and effective porosity (n_e) were used in the analysis. The metal sorbates used were cadmium, chromium, lead and zinc.

K_d was determined from the equilibrium batch testing. K_d (L/mg) is the linear partitioning coefficient (slope of the linear portion of mass sorbed per mass sorbent versus equilibrium contaminant concentration in solution at the end of test (mg/ L)). The results obtained from the batch reaction testing were used to model the transport of certain metals through the dredged sediment 'barrier'. This follows analysis done in determining the adsorption characteristics of paper clay, Moo-Young et al (2000), and clay liners. The advection-dispersion-adsorption equation was used for modeling contaminant flow. The effluent concentration passing through a barrier is predicted with the advection-dispersion-adsorption equation which is defined as follows:

$$C/C_o = 0.5\{erfc\ [(1-T_r)/(2(Tr/P_l)^{0.5})] + \exp(P_l)\ erfc[(1+T_r)/(2(Tr/P_l)^{0.5})]\} \quad (6.1)$$

where C/C_o = dimensionless relative concentration, C = effluent concentration (mg/L), C_o = influent concentration (mg/L), T_r = dimensionless time factor (Eqn. 6.2), v = seepage velocity, t = time, L = length of barrier (thickness) (m), P_l = peclet number (Eqn. 6.4), D_x = hydrodynamic dispersion (m^2/s) (Eqn. 6.5), D_m = diffusion coefficient, R_d = retardation factor (Eqn 6.6), erfc = complementary error function.

In defining equation 6.1, other equations should be defined:

$$T_r = vt/LR_d \quad (6.2)$$

$$v = ki/n_e, \quad (6.3)$$

$$P_l = v\ L/D_x \quad (6.4)$$

$$D_x = (0.1Lv + D_m) \quad (6.5)$$

$$R_d = 1 + (\gamma K_d/n_e) \quad (6.6)$$

where k is the experimentally determined hydraulic conductivity. In using Equation (6.1), certain values were assumed, and certain typical field values were varied. D_m was assumed to be $2x10^{-10}$ m^2/s, the hydraulic gradient was varied between (0.02 and 0.08), the wall thickness was varied between (0.8 and 1.1m) and the porosity was varied between (0.27 and 0.37).

Equation (6.1) was used to calculate the dimensionless effluent concentration (C/C_o) for a range of times. By plotting (C/C_o) versus the dimensionless time factor T_r, a breakthrough curve was obtained to estimate the time required to reach a point where the

effluent concentration exceeded established limits. After this point, the dredged material would have to be replaced, or an alternate remediation method would have to be used.

6.1.1 Effect of Bentonite Content on Adsorption

The adsorption capacity of the mix for the given metals was increased (i.e. the breakthrough time was extended) with increasing bentonite content. A range of values of typical field conditions related to hydraulic gradient in the field, effective porosity and the thickness of the barrier were varied in the analysis. The following reported values, however, are the results obtained using the average of these typical field values.

With respect to the sorption of cadmium, 1% , 2% and 3% bentonite mixes gave corresponding breakthrough times of approximately 1, 2.25 and 4.5 years respectively. Table 6.1 and Figure 6.1 illustrate this trend. Figure 6.1(a) shows C/C_o versus the dimensionless time factor, T_r. To more clearly show the time effects, Figure 6.1 (b) shows C/C_o versus time in years. With respect to the sorption of chromium, using 1% , 2% and 3% bentonite gave corresponding breakthrough times of approximately 0.75, 1.75 and 20 years respectively. Table 6.2 and Figure 6.2 illustrate this trend.

For the lead sorption, 1%, 2% and 3% bentonite mixes gave corresponding breakthrough times of approximately 2.75, 10 and 72 years respectively. Table 6.3 and Figure 6.3 show this trend. For the zinc sorption, 1%, 2% and 3% bentonite mixes gave corresponding breakthrough times of approximately 1.25, 5 and 13.5 years respectively. Table 6.4 and Figure 6.4 show this trend.

Table 6.1 (a) – Adsorption of Cadmium

1% BENTONITE

Time (yrs.)	Time Factor	C/Co
0.100	0.343	9.49E-03
0.250	0.859	4.47E-01
0.500	1.717	9.28E-01
0.750	2.576	9.93E-01
1.000	3.435	9.99E-01
1.250	4.293	1.00E+00
1.500	5.152	1.00E+00
1.750	6.011	1.00E+00
2.000	6.869	1.00E+00
2.250	7.728	1.00E+00
2.500	8.587	1.00E+00
2.750	9.445	1.00E+00
3.000	10.304	1.00E+00

(b)

2% BENTONITE

Time (yrs.)	Time Factor	C/Co
0.100	0.153	1.22E-06
0.250	0.382	1.95E-02
0.500	0.764	3.45E-01
0.750	1.146	7.01E-01
1.000	1.528	8.83E-01
1.250	1.910	9.57E-01
1.500	2.292	9.84E-01
1.750	2.674	9.94E-01
2.000	3.057	9.98E-01
2.250	3.439	9.99E-01
2.500	3.821	1.00E+00
2.750	4.203	1.00E+00
3.000	4.585	1.00E+00

(c)

3% BENTONITE

Time (yrs.)	Time Factor	C/Co
0.100	0.078	1.96E-13
0.500	0.391	2.39E-02
1.000	0.783	3.68E-01
1.500	1.174	7.19E-01
2.000	1.565	8.92E-01
2.500	1.957	9.60E-01
3.000	2.348	9.86E-01
3.500	2.739	9.95E-01
4.000	3.131	9.98E-01
4.500	3.522	9.99E-01
5.000	3.914	1.00E+00
5.500	4.305	1.00E+00
6.000	4.696	1.00E+00

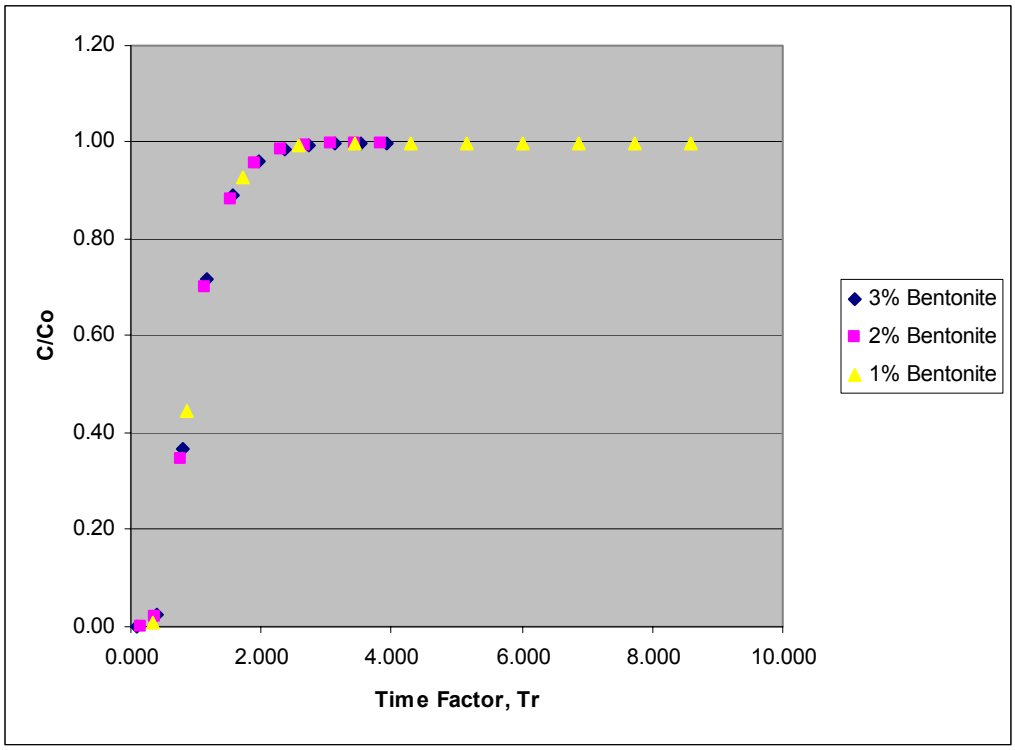

Fig 6.1(a) – Effect of Bentonite on Cadmium Adsorption

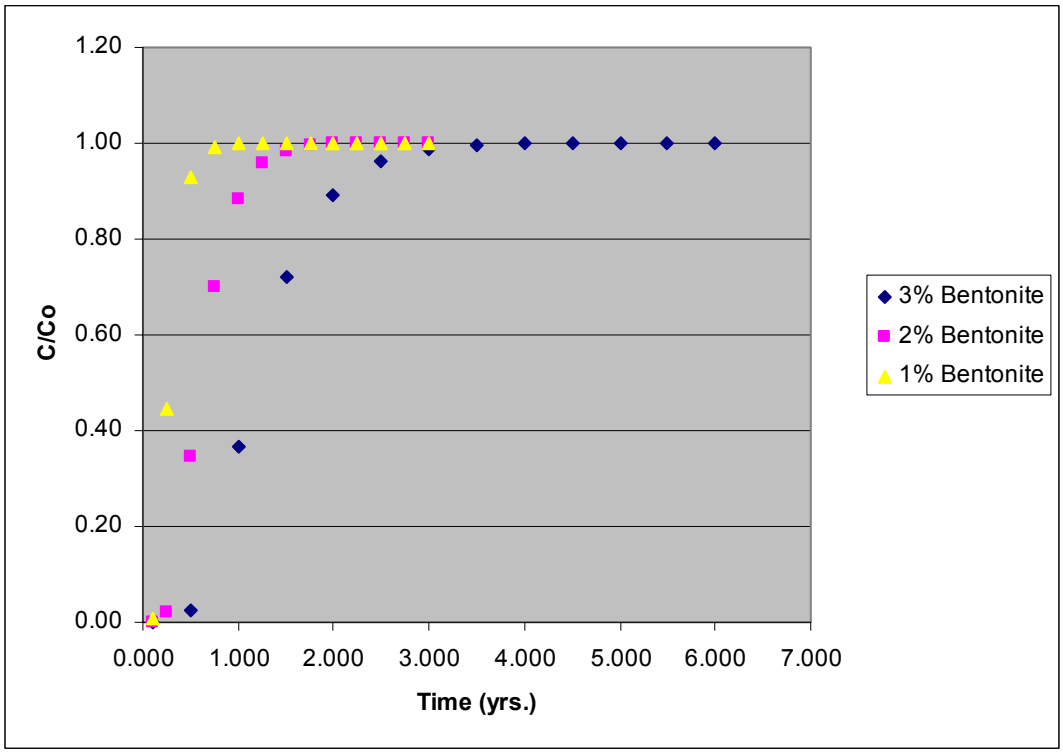

Fig 6.1(b) – Effect of Bentonite on Cadmium Adsorption

Table 6.2 (a) – Adsorption of Chromium

1% BENTONITE

Time (yrs.)	Time Factor	C/Co
0.100	0.593	1.60E-01
0.250	1.482	8.69E-01
0.500	2.965	9.97E-01
0.750	4.447	1.00E+00
1.000	5.930	1.00E+00
1.250	7.412	1.00E+00
1.500	8.895	1.00E+00
1.750	10.377	1.00E+00
2.000	11.860	1.00E+00
2.250	13.342	1.00E+00
2.500	14.825	1.00E+00
2.750	16.307	1.00E+00
3.000	17.790	1.00E+00

(b)

2% BENTONITE

Time (yrs.)	Time Factor	C/Co
0.100	0.192	3.48E-05
0.250	0.481	6.78E-02
0.500	0.962	5.51E-01
0.750	1.442	8.54E-01
1.000	1.923	9.58E-01
1.250	2.404	9.88E-01
1.500	2.885	9.97E-01
1.750	3.366	9.99E-01
2.000	3.846	1.00E+00
2.250	4.327	1.00E+00
2.500	4.808	1.00E+00
2.750	5.289	1.00E+00
3.000	5.770	1.00E+00

(c)

3% BENTONITE		
Time (yrs.)	Time Factor	C/Co
0.100	0.016	0.00E+00
2.500	0.401	2.74E-02
5.000	0.801	3.89E-01
7.500	1.202	7.37E-01
10.000	1.603	9.01E-01
12.500	2.003	9.65E-01
15.000	2.404	9.88E-01
17.500	2.805	9.96E-01
20.000	3.205	9.99E-01
22.500	3.606	9.99E-01
25.000	4.007	1.00E+00
27.500	4.407	1.00E+00
30.000	4.808	1.00E+00

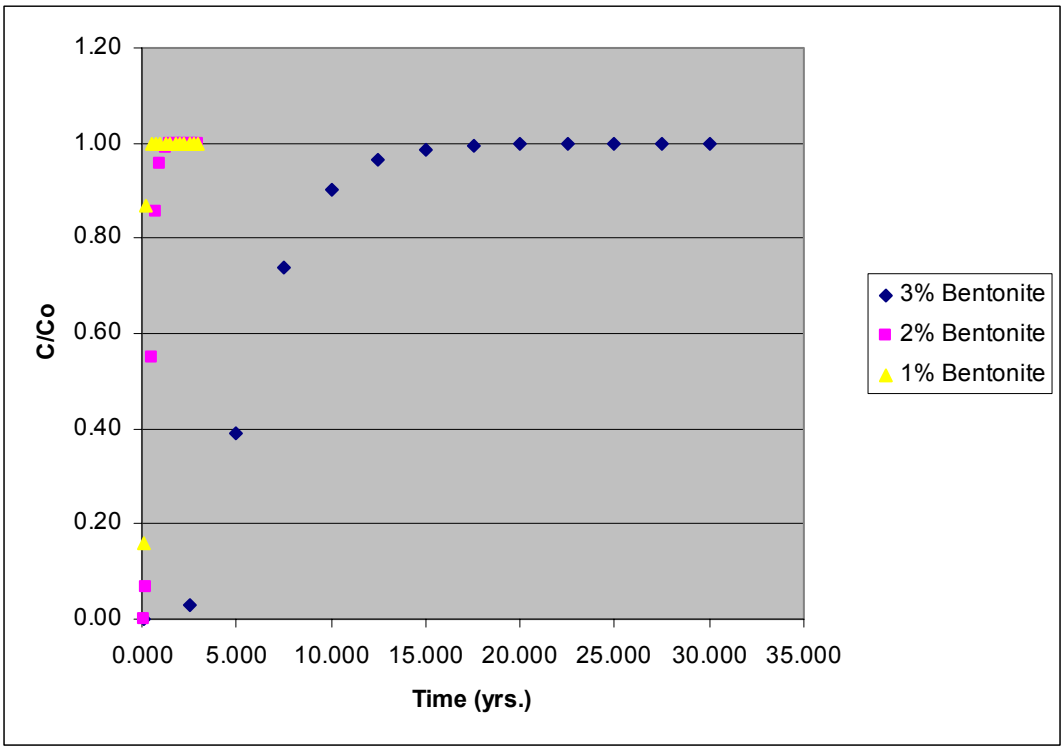

Fig 6.2 – Effect of Bentonite on Chromium Adsorption

Table 6.3 (a) – Adsorption of Lead

1% BENTONITE

Time (yrs.)	Time Factor	C/Co
0.100	0.120	1.38E-08
0.250	0.301	3.51E-03
0.500	0.602	1.69E-01
0.750	0.903	4.93E-01
1.000	1.205	7.39E-01
1.250	1.506	8.76E-01
1.500	1.807	9.43E-01
1.750	2.108	9.75E-01
2.000	2.409	9.89E-01
2.250	2.710	9.95E-01
2.500	3.011	9.98E-01
2.750	3.312	9.99E-01
3.000	3.614	1.00E+00

(b)

2% BENTONITE

Time (yrs.)	Time Factor	C/Co
0.100	0.035	0.00E+00
1.000	0.348	1.06E-02
2.000	0.697	2.70E-01
3.000	1.045	6.24E-01
4.000	1.393	8.35E-01
5.000	1.741	9.32E-01
6.000	2.090	9.73E-01
7.000	2.438	9.89E-01
8.000	2.786	9.96E-01
9.000	3.134	9.98E-01
10.000	3.483	9.99E-01
11.000	3.831	1.00E+00
12.000	4.179	1.00E+00

(c)

3% BENTONITE

Time (yrs.)	Time Factor	C/Co
0.100	0.005	0.00E+00
8.000	0.383	2.09E-02
16.000	0.765	3.49E-01
24.000	1.148	7.02E-01
32.000	1.531	8.82E-01
40.000	1.913	9.56E-01
48.000	2.296	9.84E-01
56.000	2.679	9.94E-01
64.000	3.061	9.98E-01
72.000	3.444	9.99E-01
80.000	3.827	1.00E+00
88.000	4.209	1.00E+00
96.000	4.592	1.00E+00

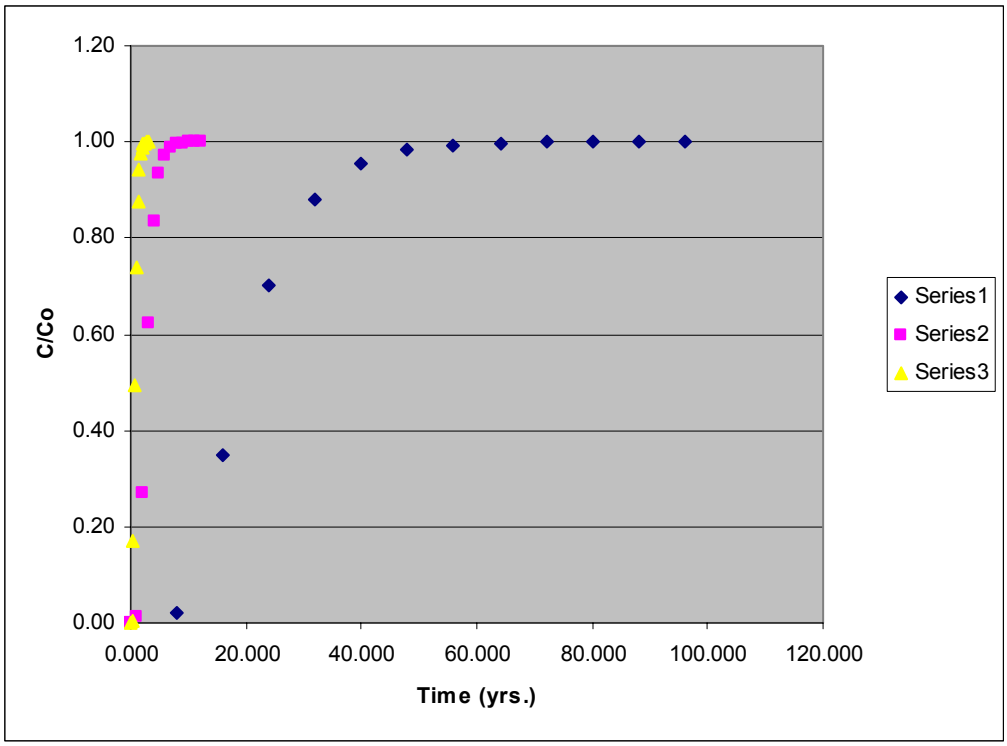

Fig 6.3 – Effect of Bentonite on Lead Adsorption

Table 6.3 (a) – Adsorption of Zinc

1% BENTONITE

Time (yrs.)	Time Factor	C/Co
0.100	0.120	1.38E-08
0.250	0.301	3.51E-03
0.500	0.602	1.69E-01
0.750	0.903	4.93E-01
1.000	1.205	7.39E-01
1.250	1.506	8.76E-01
1.500	1.807	9.43E-01
1.750	2.108	9.75E-01
2.000	2.409	9.89E-01
2.250	2.710	9.95E-01
2.500	3.011	9.98E-01
2.750	3.312	9.99E-01
3.000	3.614	1.00E+00

(b)

2% BENTONITE

Time (yrs.)	Time Factor	C/Co
0.100	0.035	0.00E+00
1.000	0.348	1.06E-02
2.000	0.697	2.70E-01
3.000	1.045	6.24E-01
4.000	1.393	8.35E-01
5.000	1.741	9.32E-01
6.000	2.090	9.73E-01
7.000	2.438	9.89E-01
8.000	2.786	9.96E-01
9.000	3.134	9.98E-01
10.000	3.483	9.99E-01
11.000	3.831	1.00E+00
12.000	4.179	1.00E+00

(c)

3% BENTONITE

Time (yrs.)	Time Factor	C/Co
0.100	0.005	0.00E+00
8.000	0.383	2.09E-02
16.000	0.765	3.49E-01
24.000	1.148	7.02E-01
32.000	1.531	8.82E-01
40.000	1.913	9.56E-01
48.000	2.296	9.84E-01
56.000	2.679	9.94E-01
64.000	3.061	9.98E-01
72.000	3.444	9.99E-01
80.000	3.827	1.00E+00
88.000	4.209	1.00E+00
96.000	4.592	1.00E+00

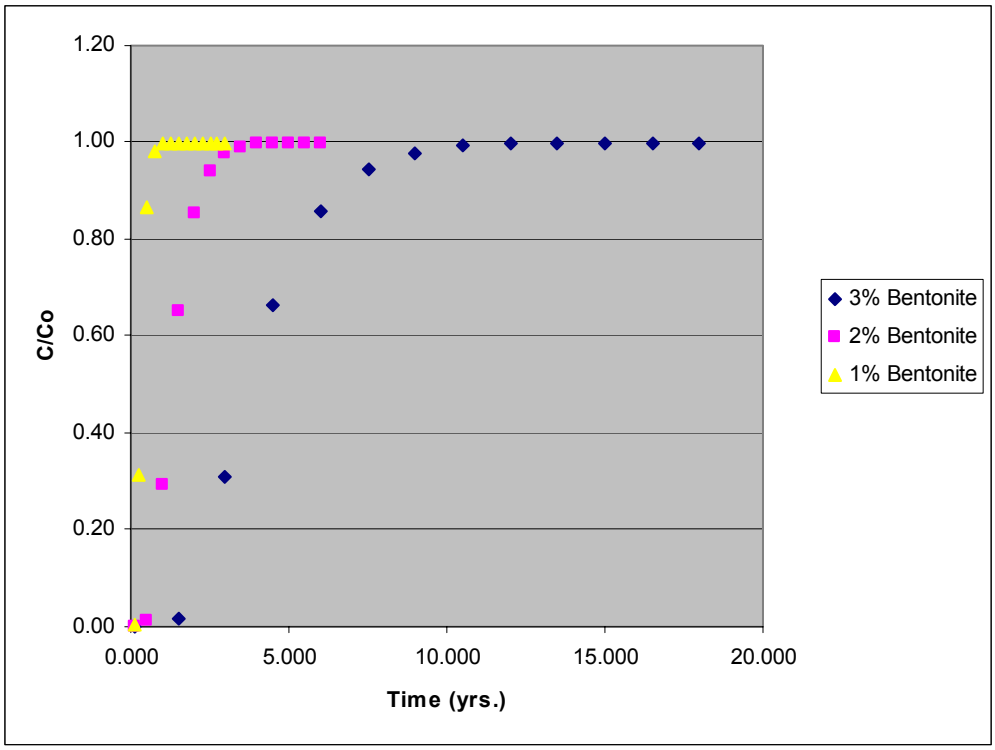

Fig 6.4 – Effect of Bentonite on Zinc Adsorption

6.1.2 – Effect of Fly ash Content on Adsorption

In using the metals tested, it was found that the corresponding breakthrough time decreased with increasing fly ash content. It was expected that the carbon content in the fly ash would enhance the adsorption capacity of the mix. However, the increased hydraulic conductivity of the mix (which leads to lower adsorption capacity) may have outweighed the beneficial adsorptive effects of the carbon in the fly ash. In the case of cadmium, 5% and 8% fly ash content gave corresponding breakthrough times of approximately 110 and 36 days respectively. Table 6.5 and Figure 6.5 show the behavior of cadmium. Figure 6.5(a) shows C/C_o versus the dimensionless time factor, T_r. To more clearly show the time effects, Figure 6.5(b) shows C/C_o versus time in years. In the case of chromium, 5% and 8% fly ash content gave corresponding breakthrough times of approximately 90 and 40 days respectively. Table 6.6 and Figure 6.6 show the behavior of chromium.

In the case of lead, 5% and 8% fly ash content gave corresponding breakthrough times of approximately 275 and 135 days respectively. Table 6.7 and Figure 6.7 show the behavior of lead. In the case of zinc, 5% and 8% fly ash content gave corresponding breakthrough times of approximately 220 and 100 days respectively. Table 6.8 and Figure 6.8 show this behavior trend.

6.1.3 – Effect of Varying Field Conditions on Adsorption

Under the circumstances tested, typical field condition values for hydraulic gradient in the field, effective porosity and thickness of barrier were varied to see the

effect they would have on the breakthrough time. This was analyzed using Equation 6.1 and the associated breakthrough curve plots.

In all cases, an increase in the thickness of barrier from 0.8 m to 1.1 m leads to a moderate increase in the breakthrough time. For the mix containing 3% bentonite that was being used to adsorb zinc, the breakthrough time increased from approximately 12 years to 16.5 years. Similar trends were obtained for the other metals tested with different design mixes, the trend is shown in Figure 6.9. This could be theoretically explained in that there is a larger specific surface area of the material that has the capacity to adsorb the respective metal. This theory has been seconded in a study by Moo-Young et al (2000).

An increase in the hydraulic from 0.02 to 0.08 lead to a substantial decrease in the breakthrough time. For the mix containing 3% bentonite that was being used to adsorb zinc, the breakthrough time decreased from approximately 36 years to 9 years. Similar trends were obtained for the other metals tested, the trend is shown in Figure 6.10. This can be explained in that an increased hydraulic gradient causes ground water or contaminant solution to flow faster and therefore drives the contaminants (metals) through the barrier mix at a faster rate.

An increase in the effective porosity from 0.27 to 0.37, lead to no significant change in the breakthrough time. For the mix containing 2% bentonite that was being used to adsorb zinc, the breakthrough time remained at approximately 5 years. Similar results were obtained for the other metals tested, the trend is shown in Figure 6.11. It would be theoretically expected that there would be a decrease in the breakthrough time. This is because an increased effective porosity means greater pore space for the

Table 6.5(a) – Adsorption of Cadmium

1%BENTONITE & 5%FLY ASH

Time (days)	Time Factor	C/Co
0.100	0.003	0.00E-00
10.000	0.306	4.00E-03
20.000	0.613	1.79E-01
30.000	0.919	5.09E-01
40.000	1.225	7.52E-01
50.000	1.532	8.84E-01
60.000	1.838	9.48E-01
70.000	2.145	9.77E-01
80.000	2.451	9.90E-01
90.000	2.757	9.96E-01
100.000	3.064	9.98E-01
110.000	3.370	9.99E-01
120.000	3.676	1.00E+00

(b)

1%BENTONITE & 5%FLY ASH

Time (days)	Time Factor	C/Co
0.100	0.010	0.00E+00
3.000	0.298	3.15E-03
6.000	0.595	1.62E-01
9.000	0.893	4.82E-01
12.000	1.191	7.31E-01
15.000	1.488	8.71E-01
18.000	1.786	9.40E-01
21.000	2.084	9.73E-01
24.000	2.381	9.88E-01
27.000	2.679	9.95E-01
30.000	2.976	9.98E-01
33.000	3.274	9.99E-01
36.000	3.572	1.00E+00

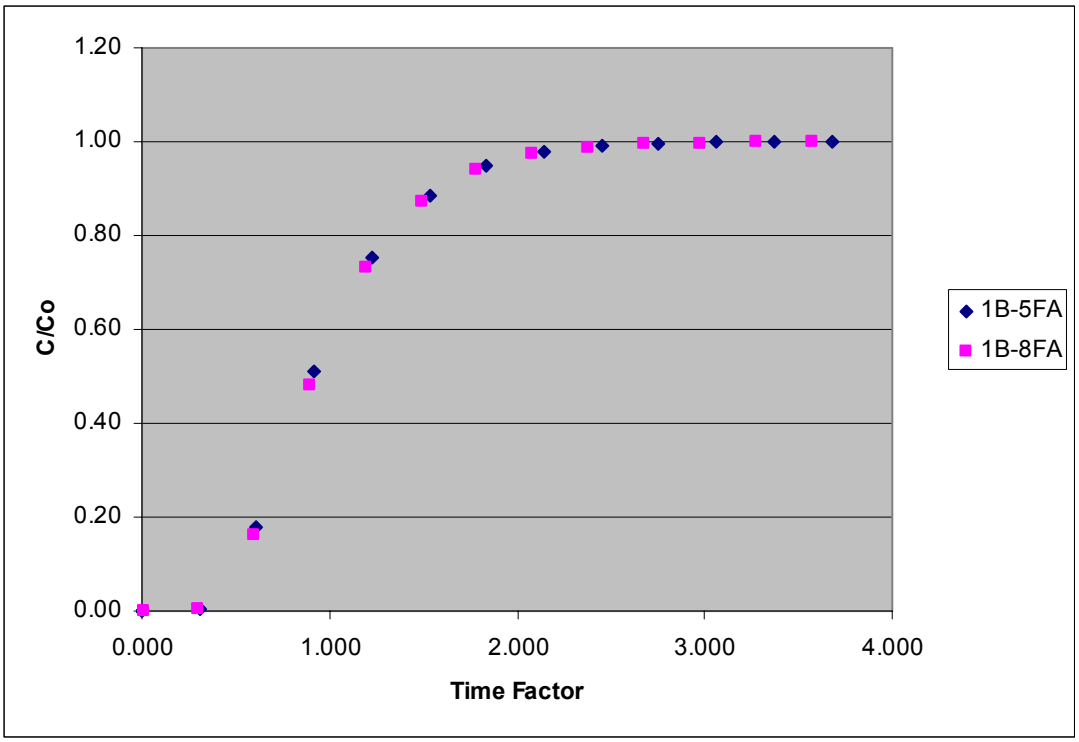

Fig 6.5(a) – Effect of Bentonite on Cadmium Adsorption

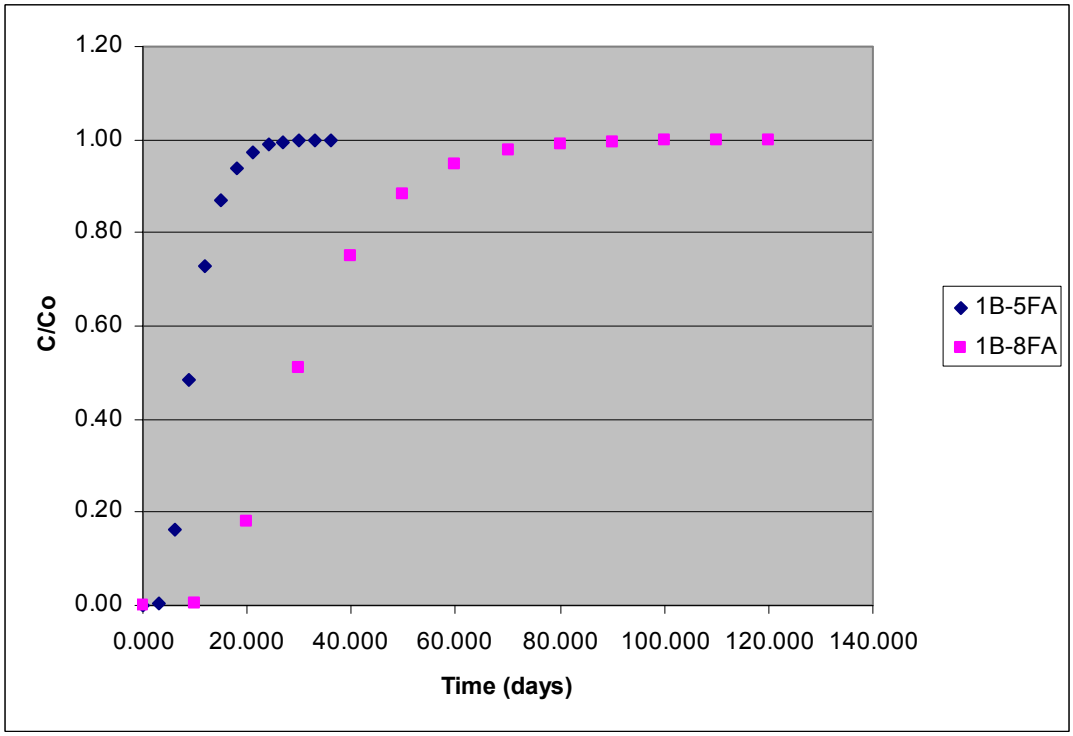

Fig 6.5(b) – Effect of Bentonite on Cadmium Adsorption

Table 6.6(a) – Adsorption of Chromium

1%BENTONITE & 5%FLY ASH

Time (days)	Time Factor	C/Co
0.100	0.004	0.00E+00
10.000	0.391	2.19E-02
20.000	0.782	3.63E-01
30.000	1.172	7.19E-01
40.000	1.563	8.93E-01
50.000	1.954	9.62E-01
60.000	2.345	9.87E-01
70.000	2.736	9.95E-01
80.000	3.127	9.98E-01
90.000	3.517	9.99E-01
100.000	3.908	1.00E+00
110.000	4.299	1.00E+00
120.000	4.690	1.00E+00

(b)

1% BENTONITE & 8% FLY ASH

Time (days)	Time Factor	C/Co
0.100	0.008	0.00E+00
4.000	0.330	7.09E-03
8.000	0.661	2.29E-01
12.000	0.991	5.77E-01
16.000	1.321	8.03E-01
20.000	1.651	9.15E-01
24.000	1.982	9.65E-01
28.000	2.312	9.85E-01
32.000	2.642	9.94E-01
36.000	2.973	9.98E-01
40.000	3.303	9.99E-01
44.000	3.633	1.00E+00
48.000	3.963	1.00E+00

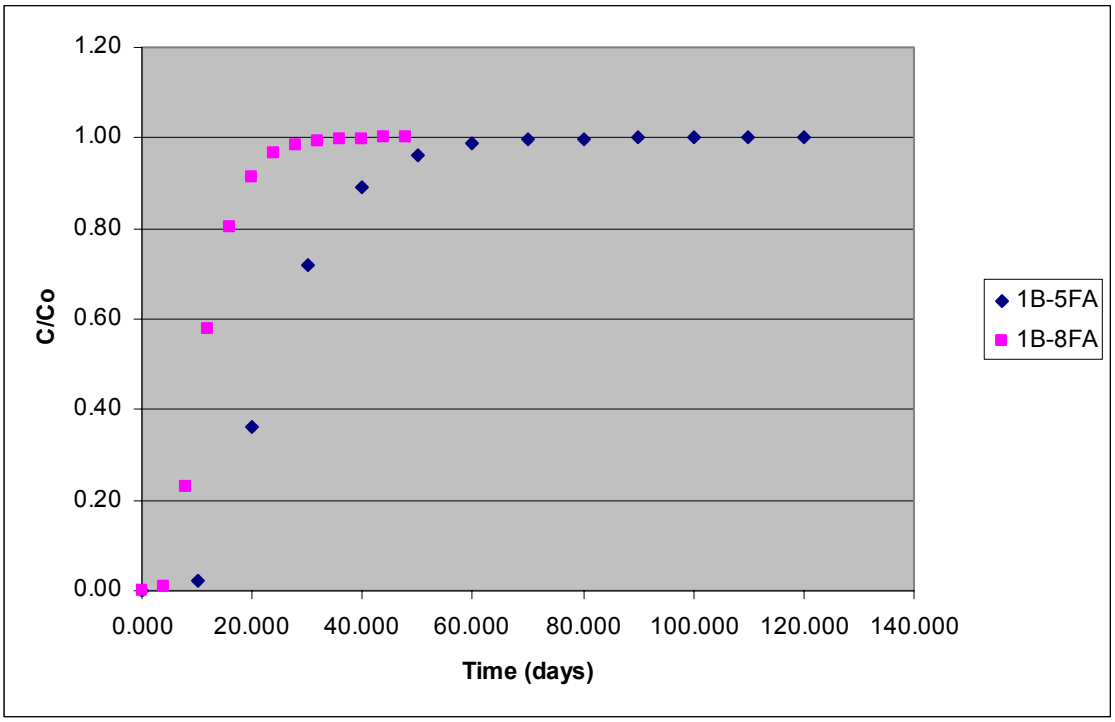

Fig 6.6 – Effect of Bentonite on Chromium Adsorption

Table 6.7(a) – Adsorption of Lead

1%BENTONITE & 5%FLY ASH

Time (days)	Time Factor	C/Co
0.100	0.001	0.00E+00
25.000	0.288	2.42E-03
50.000	0.577	1.44E-01
75.000	0.865	4.53E-01
100.000	1.153	7.06E-01
125.000	1.441	8.54E-01
150.000	1.730	9.31E-01
175.000	2.018	9.68E-01
200.000	2.306	9.85E-01
225.000	2.595	9.93E-01
250.000	2.883	9.97E-01
275.000	3.171	9.99E-01
300.000	3.459	9.99E-01

(b)

1% BENTONITE & 8% FLY ASH

Time (days)	Time Factor	C/Co
0.100	0.002	0.00E+00
15.000	0.370	1.55E-02
30.000	0.740	3.16E-01
45.000	1.110	6.75E-01
60.000	1.479	8.68E-01
75.000	1.849	9.50E-01
90.000	2.219	9.81E-01
105.000	2.589	9.93E-01
120.000	2.959	9.97E-01
135.000	3.329	9.99E-01
150.000	3.699	1.00E+00
165.000	4.069	1.00E+00
180.000	4.438	1.00E+00

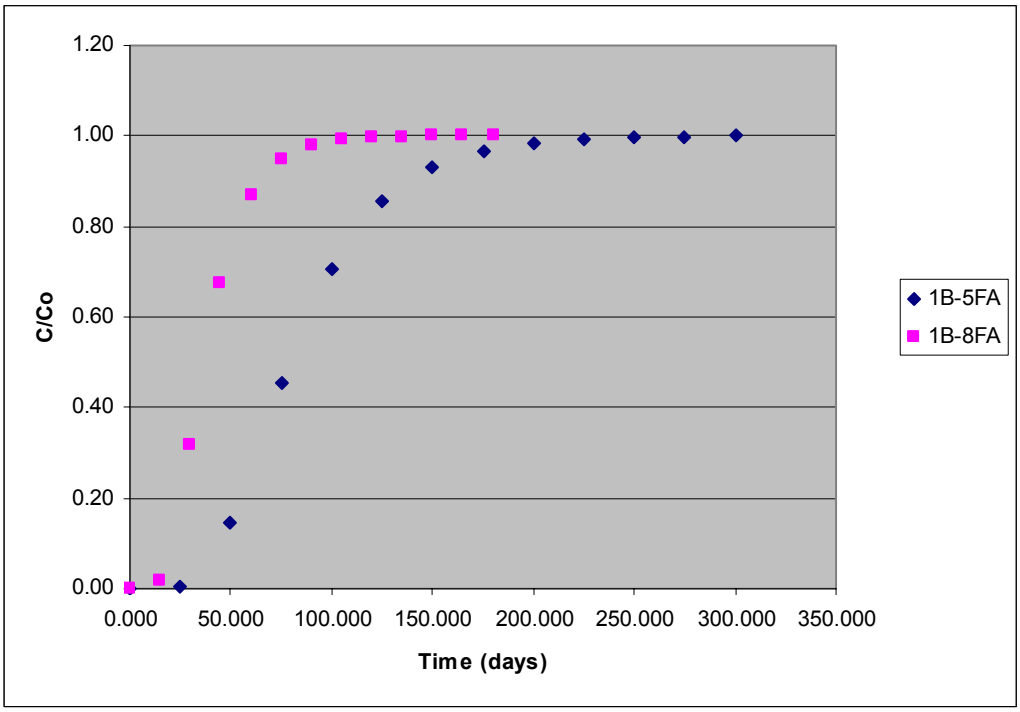

Fig 6.7 – Effect of Bentonite on Lead Adsorption

Table 6.8(a) – Adsorption of Zinc

1%BENTONITE & 5%FLY ASH

Time (days)	Time Factor	C/Co
0.100	0.002	0.00E+00
20.000	0.305	3.86E-03
40.000	0.610	1.77E-01
60.000	0.915	5.05E-01
80.000	1.220	7.49E-01
100.000	1.525	8.82E-01
120.000	1.830	9.47E-01
140.000	2.136	9.76E-01
160.000	2.441	9.90E-01
180.000	2.746	9.95E-01
200.000	3.051	9.98E-01
220.000	3.356	9.99E-01
240.000	3.661	1.00E+00

(b)

1% BENTONITE & 8% FLY ASH

Time (days)	Time Factor	C/Co
0.100	0.004	0.00E+00
10.000	0.361	1.32E-02
20.000	0.722	2.97E-01
30.000	1.084	6.55E-01
40.000	1.445	8.56E-01
50.000	1.806	9.43E-01
60.000	2.167	9.78E-01
70.000	2.528	9.92E-01
80.000	2.889	9.97E-01
90.000	3.251	9.99E-01
100.000	3.612	1.00E+00
110.000	3.973	1.00E+00
120.000	4.334	1.00E+00

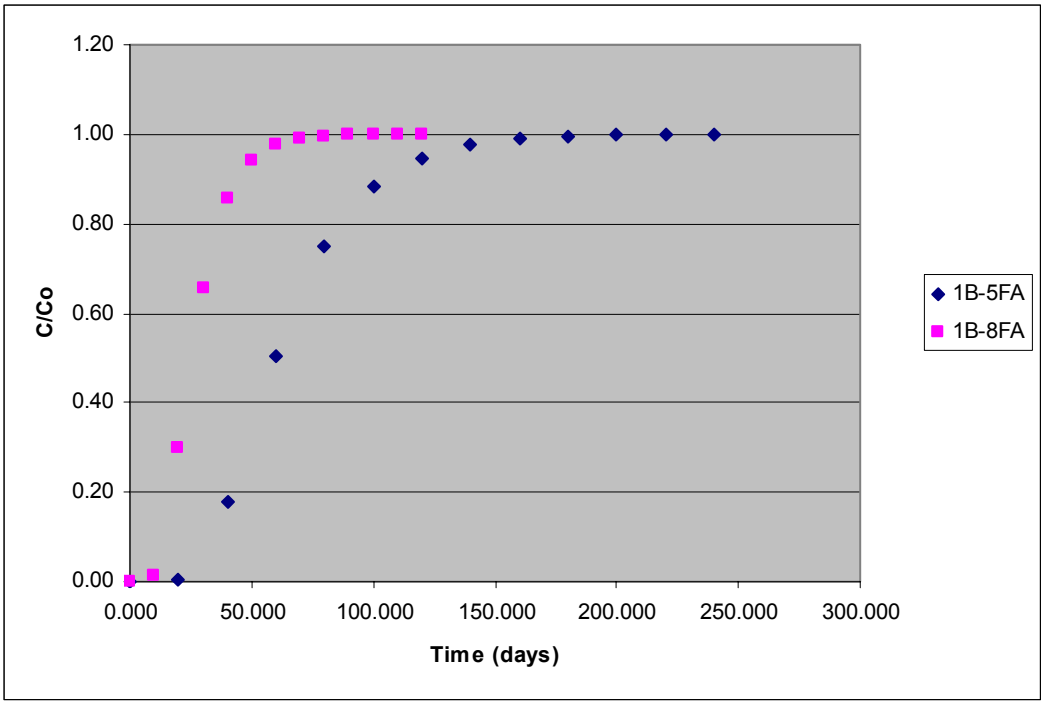

Fig 6.8 – Effect of Bentonite on Zinc Adsorption

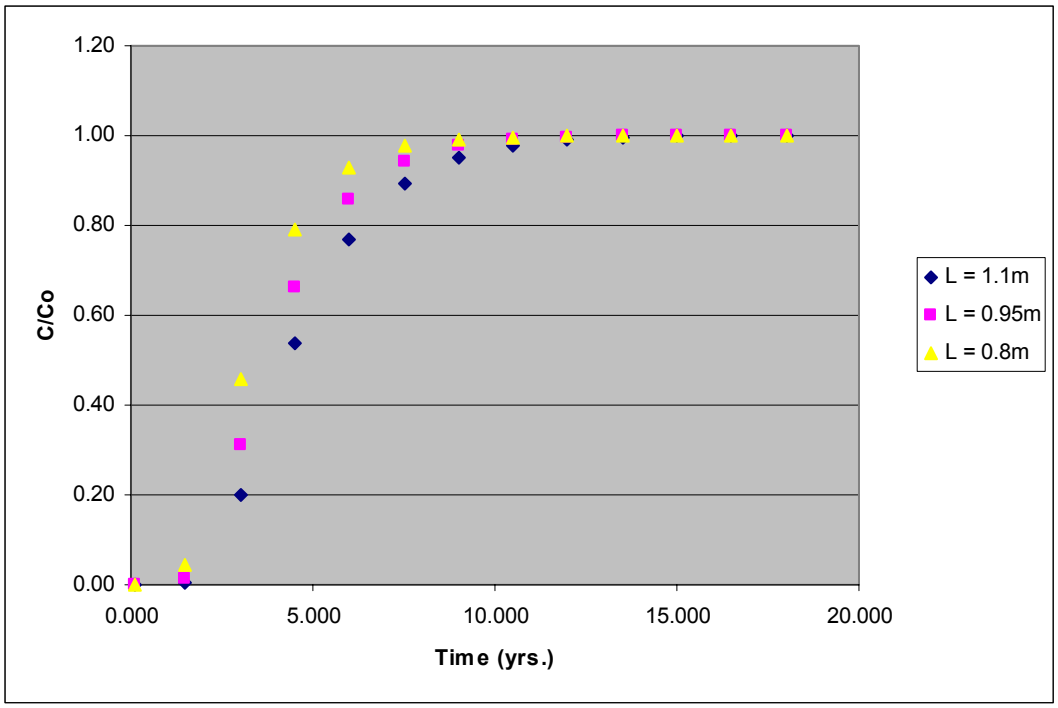

Fig. 6.9 – Effect of Barrier Thickness on Adsorption

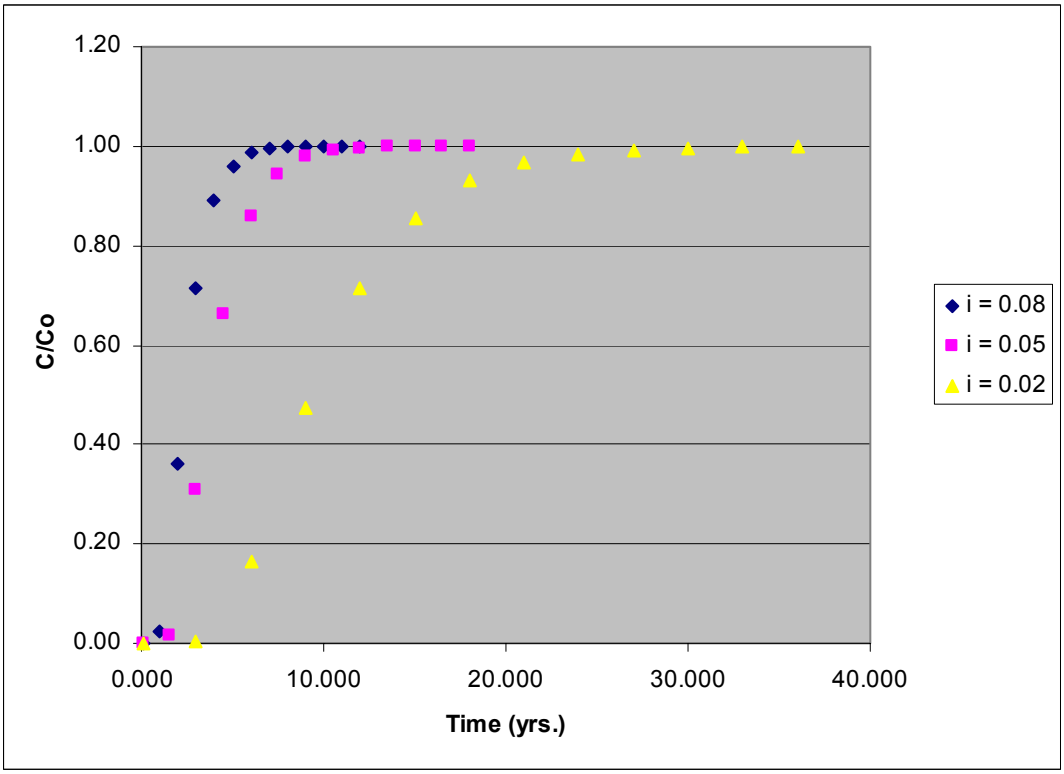

Fig 6.10 – Effect of Hydraulic Gradient on Adsorption

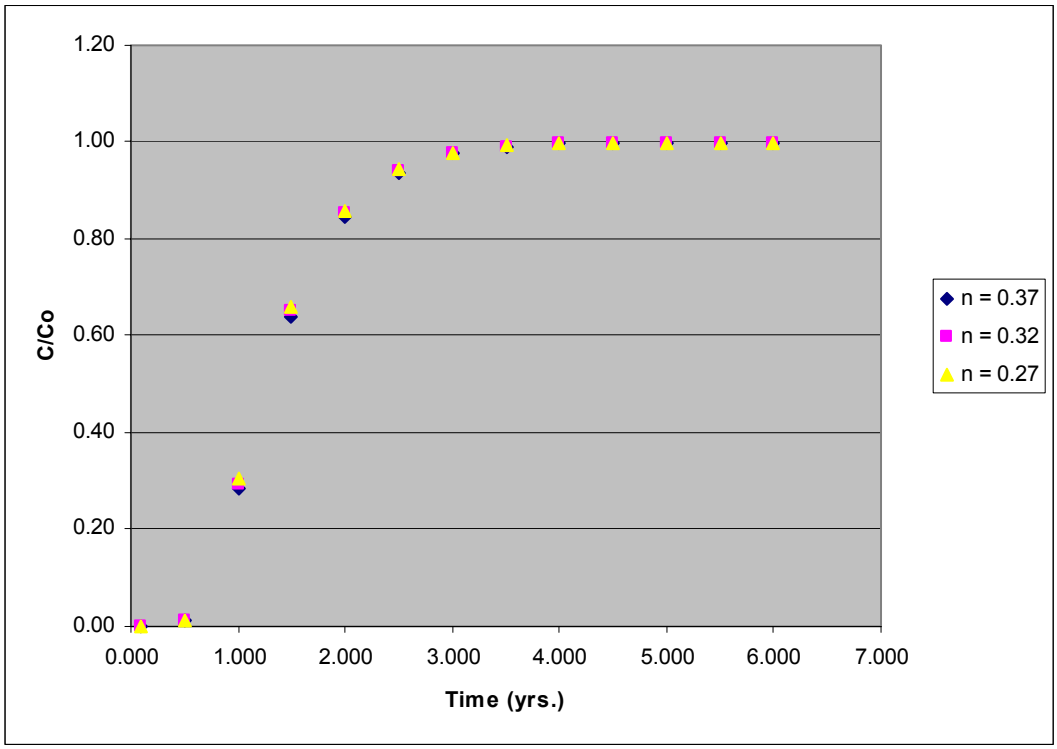

Fig 6.11 – Effect of Porosity on Adsorption

respective metal to pass through the barrier material, hence the breakthrough time would be reduced. The minimal variation in porosity values in the field that were used in the analysis might explain the lack of change in the breakthrough time.

6.1.4 Summary of Values

Table 6.9 shows a summary of the partitioning coefficient (K_d) values obtained for the metals tested using the different design mixes. Table 6.10 shows a summary of the estimated breakthrough time for the dredged sediment mixes. These values show that lead tends to be attenuated for the longest time before reaching its breakthrough time, hence the most threatening, followed by zinc, cadmium then chromium. These values compare well with the values obtained from the study by Moo-Young et al, (2000), in testing the adsorption characteristics of paper mill sludge and comparing it to the adsorption potential of materials in use today. As the values obtained in this study compare well with the adsorptive materials in use today, it can be said that dredged sediment barriers can serve as an effective containment as well as remediation system.

Table 6.9 – Summary of Partitioning Coefficient (K_d) Values

	Average Kd Values (L/kg)			
Mix Type	**Cadmium**	**Chromuim**	**Lead**	**Zinc**
1% BENTONITE+5% FLY ASH	12.00	9.00	35.00	26.00
1% BENTONITE+8% FLY ASH	13.00	16.00	58.00	39.00
1% BENTONITE	10.00	5.00	32.00	12.00
2% BENTONITE	8.00	6.00	41.00	19.00
3% BENTONITE	3.22	22.05	77.85	13.98

Table 6.10a – Summary of Estimated Breakthrough Times with Fly Ash Mixes

	Estimated Breakthrough Times (days)			
Mix Type	**Cadmium**	**Chromuim**	**Lead**	**Zinc**
1% BENTONITE+5% FLY ASH	110	90	275	220
1% BENTONITE+8% FLY ASH	36	40	135	100

Table 6.10b – Summary of Estimated Breakthrough Times with Bentonite Mixes

	Estimated Breakthrough Times (years)			
Mix Type	**Cadmium**	**Chromuim**	**Lead**	**Zinc**
1% BENTONITE	1	0.75	2.75	1.25
2% BENTONITE	2.25	1.75	10	5
3% BENTONITE	4.5	20	72	13.5

6.2 - SYNTHESIS

The mixes containing 1%, 2% and 3% bentonite were tested for their adsorption properties. The mix containing 1% bentonite was additionally modified by the addition of 5% and 8% fly ash by weight to test the effects of fly ash on adsorption. Typical field values for thickness of barrier, hydraulic gradient and effective porosity were varied to determine their effects on adsorption capacity. The metals tested include cadmium, chromium, lead and zinc.

The conclusions that can be drawn are as follows:

- For all metals tested, increased bentonite content lead to an increase in the adsorption capacity of the mix.
- For all the metals tested, increased fly ash content lead to a decrease in the adsorption capacity of the mix.
- A larger barrier thickness resulted in a longer breakthrough time, hence greater adsorption capacity.
- An increased hydraulic gradient resulted in a shorter breakthrough time, hence less adsorption capacity.
- An increased effective porosity lead to minimal to no change in the breakthrough time and adsorption capacity.
- It can be said that dredged sediment barriers can serve as an effective containment and remediation system under appropriate conditions.

SECTION 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 SUMMARY AND CONCLUSIONS

The intention of the testing plan is to find an appropriate mix of sediment and bentonite that will be able to function as a vertical cut-off wall backfill material. The preliminary tests on the bentonite were carried out for screening purposes and to find an appropriate water content that will satisfy the desired viscosity range.

Bentonite was then added to the dredged sediment in ratios of 1%, 2% and 3% of the total dredged sediment weight. The preliminary tests were repeated for each of these mixes to determine applicable trends and at what percentages of bentonite, the viscosity of the mixture was still in the workable range. The 1% bentonite mix was additionally modified with the addition of 5% and 8% fly ash by weight. These mixtures were then subjected to API filter press tests (used as a rigid wall permeameter) to determine the effect these mixes would have on the hydraulic conductivity. Adsorption testing was also carried out on all the sediment mixes and aqueous solutions containing heavy metals to determine the adsorption capacities of the sediment mix. The metals used include cadmium, chromium, lead and zinc.

Detailed conclusions of the study can be found at the end of Sections 5 and 6. They are summarized as follows:

- The mud weight density of the dredged sediments was 10.77 kN/m³ (68.49 pcf) and this value falls within the range of bentonite slurries 10.06 - 12.58 kN/m3 (64-80 pcf).

- A suitable moisture content and viscosity of the dredged sediments can be attained that will make it usable in the mix design. It is workable and pumpable.
- Increasing bentonite content leads to an increase in mud weight density. This is helpful as the backfill material must be denser than the slurry in order to displace it in the construction process.
- An increase in bentonite content allows for decreased moisture content and correspondingly increased viscosity.
- Increasing bentonite content, to the percent tested (3%), resulted in a decrease in hydraulic conductivity. The optimum range to serve the purpose of use as a vertical cutoff wall ($1x10^{-7}$ to $1x10^{-8}$ cm/s) was near realized ($4.48x10^{-7}$ cm/s) at 3% bentonite and an applied pressure of 48.3kPa.
- Increasing the flyash content to the percent tested (8%), (with the base as dredged sediments and 1% bentonite by weight), increased the hydraulic conductivity.
- The filtrate loss results did not show any distinct trends. The filtrate loss on the bentonite specimens tested ranged from 2.27 mL/s to 0.04 mL/s. The filtrate loss on the fly ash specimens tested ranged from 14.29 mL/s to 1.54 mL/s.
- For all metals tested, increased bentonite content lead to an increase in the adsorption capacity of the mix.
- For all the metals tested, increased fly ash content lead to a decrease in the adsorption capacity of the mix. It was expected that the carbon content in the flyash would enhance the adsorption capacity of the mix. However, the increased hydraulic conductivity of the mix (which leads to lower adsorption capacity) may have outweighed the beneficial adsorptive effects of the carbon in fly ash.

- A larger barrier thickness resulted in a longer breakthrough time, hence greater adsorption capacity.
- An increased hydraulic gradient resulted in a shorter breakthrough time, hence less adsorption capacity.
- An increased effective porosity lead to minimal to no change in the breakthrough time and adsorption capacity.
- It can be said that dredged sediment barriers can serve as an effective inhibitor to the flow of ground water and serve as an effective containment and remediation system under appropriate conditions.

7.2 PRACTICAL APPLICATIONS

The dredged sediments can be manipulated to a moisture content, viscosity (32 - 40s) and density that satisfy the appropriate range of corresponding slurry values used in field practice today. The optimum mix design of the mixes tested was the mix of dredged sediments and 3% bentonite by weight which gave desirable properties and a hydraulic conductivity of 4.48×10^{-7} cm/s at an applied pressure of 48.3 kPa. Increased bentonite content decreases the hydraulic conductivity and increases the breakthrough time for the relevant metal, which is the desired effect in the performance of a vertical cut-off wall. Increasing fly ash content had the opposite expected effect, that is, it increases the hydraulic conductivity and decreases the breakthrough time for the relevant metal.

7.3 RECOMMENDATIONS FOR FUTURE RESEARCH

The main requirements of a good vertical barrier system are its ability to limit the flow of groundwater and contain and remove or attenuate contaminants. In performing the literature review relative to this topic, very few detailed technical studies were found to specifically address the beneficial re-use of dredged sediments. More studies need to be undertaken on this topic –(specifically their use as a vertical cut-off wall material)– in order to have a larger comparative base. More design mixes can be explored to determine possibly a more optimum design mix. In addition, the behavior of these walls in the field (as discussed in Chapter 4), should be explored further to better understand the stresses they experience in the field and, hence, lead to better design work in terms of applied pressures.

A great deal of dredging operations is carried out in harbors in industrial areas where the dredged sediments are already contaminanted with certain metals. The remediation and use of these sediments can be explored to see if their use is still more economical than the alternative of substituting natural soil resources. As dredged sediments are the by-product of the dredging industry, cost savings will be realized through its use. The use of dredged sediments is also appealing from an environmental and social point of view, since the beneficial reuse applications reduce landfilling costs.

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