Beneficial Use of Dredged Materials in Great Lakes Commercial Ports for Transportation Projects

by

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CHAPTER 1: INTRODUCTION

1.1. Scope

This chapter briefly introduces the problems and opportunities associated with dredged material (DM) management in the Great Lakes region and historical options for beneficial use of DM. The overall objective of the project and the structure and scope of this report are summarized.

1.2. Statement of Problem

Dredging is an indispensable part of maintaining marine transport and supporting the freight transport system by enlarging or deepening existing navigation channels and harbors. Hundreds of millions of cubic yards of sediment are dredged from U.S. ports, harbors, and waterways each year. Safe and economical disposal of this huge volume of DM is a significant and pressing issue.

Many existing confined disposal facilities (CDFs) that serve ports in the Great Lakes region are at or near capacity (Great Lakes Commission, 2001). High costs plus limited new site availability have made prospects for new or expanded disposal capacity increasingly unlikely. According to the US Army Corps of Engineers (USACE), at least six of the Great Lakes largest cargo-handling ports – Duluth/Superior, Calumet Harbor, Saginaw, Toledo, Lorain and Cleveland – are in “critical” status, meaning that DM management issues could “severely restrict channel availability within five years.” Another six ports – Green Bay, Sheboygan, Port Washington, Milwaukee, Rouge River and Ashtabula – have “pressing” needs that could restrict channel availability in ten years.

Implications of these restrictions to freight movement in the North American mid-continent are serious. Some 175 million to 200 million tons of primarily bulk commodities – including iron ore, coal, stone, petroleum products, chemicals and grain – are moved annually on the Great Lakes St. Lawrence Seaway system. The marine mode has been well documented as the most fuel efficient, least air toxic and safest mode for movement of this cargo, and Great Lakes marine transportation supports some of North America's most important core industries including steel manufacturing, automotive, construction and agriculture. For many Great Lakes bulk cargo movements, the sheer volume of material precludes shifts to other surface transportation modes.

Given the declining placement capacity, disposal of non-toxic DM in the historic sense, as solid waste, is no longer feasible as an ongoing management practice in the Great Lakes. Use or recycling of material suitable for beneficial use (BU) is emerging as a potentially practical approach to sustainable DM management in the region. One factor favoring increased BU is the improving physical quality of the material as toxic sediments in areas of concern (AOCs) and other waterways with industrial or otherwise toxic legacies have been remediated in recent decades. As toxic discharges have been eliminated, DM from natural sedimentation has become cleaner and more acceptable for beneficial use. Beneficial use of DM alone or in mixtures with other materials or managed byproducts could have a major impact solving the declining disposal capacity. Dredged material stabilized with other such materials (e.g., fly ash) is referred to herein as stabilized dredged material (SDM).

1.3. Objective

This project focuses on beneficial use of DM as an alternative material for earthwork construction applications in the transportation sector (e.g., embankments, pavement base, etc.). The long term objective of the effort is to contribute to sustainable construction by facilitating use of DM instead of natural mined materials. The immediate objective, as described here and summarized in Figure 1.1, is to produce a set of guidelines that explicitly links together: 1) applications for the use of DM as construction materials in transportation-related earthwork projects, 2) required geotechnical properties of materials for specific construction applications, 3) geotechnical laboratory and field test methods available to determine these properties, 4) specifications (values) of these properties required for specific transportation-related projects, and 5) locations within the Great Lakes from which dredged materials having properties meeting these specifications may be sourced. The project is intended to build upon existing and more general frameworks for beneficial use of DM from the Great Lakes region (Great Lakes Commission, 2004) but within the specific context of using DM in the transportation construction sector. Emphasis is placed entirely on suitability in terms of physical characteristics. Suitability in terms of toxicity or environmental characteristics of the material is assumed.

1.4. Structure

This thesis is organized into six interrelated chapters.

Chapter 1: Introduction. This chapter provides a brief introduction to the project and its long- and short-term goals. This includes description of historical and current options for management of DM in the Great Lakes regions, a summary of the framework for the project, and a summary of the organization and scope of this thesis.

Chapter 2: Background. This chapter provides basic information regarding DM management and discusses disposal as a general method of DM management. An introduction to beneficial use of DM is provided.

Chapter 3: Geotechnical Properties Required for Transportation Construction Applications. This chapter provides a summary of general geotechnical characteristics of materials required in different applications of roadway construction, along with the specific physical and engineering properties required.

Chapter 4: Geotechnical Properties and Test Methods. This chapter identifies the physical and engineering characteristics required for consideration of DM in various transportation applications. Tests and specifications are synthesized from information available from ASTM International (ASTM), the American Association of State Highway and Transportation Officials (AASHTO) and the Wisconsin Department of Transportation (WisDOT).

Chapter 5: Properties of Dredged Materials from Select Great Lakes Locations. This chapter contains a summary of geotechnical analysis and properties of DM obtained from select harbors and CDFs within the Great Lakes region. Geotechnical testing data are synthesized for select harbors using reports available in the literature (Calumet, Indiana, Waukegan and West-arms Burns) and from laboratory tests conducted at the University of Wisconsin-Madison (UW) for samples obtained directly from a confined disposal facility (CDF) in Milwaukee, WI.

Chapter 6: Implementation of a Beneficial Use Framework. This chapter describes the process and results of making the connection between DM sources and transportation sector applications based on the geotechnical properties of the materials identified in Chapter 5.

Chapter 7: Case Study: Stabilization of Raw Dredged Material with Fly Ash. This chapter mainly discusses the difference between the raw dredged material (RDM) and stabilized dredged material (SDM) in geotechnical properties and the effect of curing time and fly ash content on SDM materials. A comprehensive laboratory experimental program was conducted to determine engineering properties relevant to transportation construction applications for RDM and SDM obtained from the Milwaukee CDF.

CHAPTER 2: BACKGROUND

2.1. Scope

DM management options including open-water disposal, confined disposal, and beneficial use are summarized. Specific categories for beneficial use of DM and relative examples are described. Discussion in this chapter has been synthesized from the literature.

2.2. Dredged Material Management

Three general management alternatives may be considered for DM: open-water disposal, confined disposal, and beneficial use. Open-water disposal is the placement of DM in rivers, lakes, estuaries, or oceans via pipeline or release from hopper dredges or barges. Confined disposal is placement of DM within dikes located near shore or in upland disposal facilities via pipeline or other means. Beneficial use involves the placement or use of DM for some productive purpose.

2.2.1. Open Water Disposal

Open water disposal has historically been a major way of managing DM. To assess the suitability of open water disposal, the following aspects should be considered. Evaluation of site characteristics is a primary step to determine the suitability of the management approach. Site characteristics include environmental aspects (e.g., water depth and wave climate), physical, chemical and biological factors (e.g., sediment condition, habitat types), and site capacity affecting the operation and efficiency of disposal.

Site selection for open water disposal should be considered under the Marine Protection, Research and Sanctuaries Act (MPRSA). The intent of the criteria for site selection is to avoid unacceptable adverse impacts on biota and other amenities. Site specification should be considered under the Clean Water Act (CWA), which establishes sequential review of a proposed project, the first step of which is avoidance of adverse impacts to the aquatic environment through an evaluation of practicable alternatives that would have less impact on that environment. Table 2.1 summarizes several aspects of laws and regulations for open water disposal in the Great Lakes Region.

2.2.2. Confined Disposal

The appropriate disposal of DM in confined disposal facilities (CDF) is an important issue around the Great Lakes. Approximately two million cubic yards of contaminated sediments is dredged annually from the Great Lakes. Because polluted materials are not suitable for open water disposal, they may be placed in CDFs. The significant difference in site characteristics between open water disposal and confined disposal concentrates on two facets: one is real estate consideration, the other is safety. Generally speaking, CDFs represent a substantial economic investment, especially when considering long term capacity. Sites are normally visible to the public and are viewed as a competing interest for land use, especially in coastal areas where there is intense pressure for both development and preservation of lands. From the aspect of safety, unlike in the case of open water disposal, contaminant pathways are wider in confined disposal, and include volatilization of contaminants (e.g., from sediment to air) and odor.

2.2.3. Beneficial Use

The frequency of beneficial use in the Great Lakes Region is under 18 percent. However, around 2 million cubic yards of sediments dredged form Great Lakes annually can be considered as uncontaminated material, which means the beneficial use has great potential and could have significant advantages compared with other management options.

2.3. Types of Beneficial Use

Beneficial use of DM can take various forms depending on its geotechnical and chemical characteristics. For uncontaminated DM, fine-grained material can be used to form construction materials after stabilization with amendments such as fly ash and lime. Sands can be used as reinforced fill in Mechanically Stabilized Earth (MSE) retaining walls, or considered as raw material for building or improving fish and wildlife habitat. Gravel and rocks can be used as base or sub-base aggregate for pavement and roadway construction. Beneficial use is also acceptable for contaminated soils, such as using them in landfill capping applications. The USACE indicates more specific beneficial use category based on sediment types (Table 2.2), as summarized in the following.

2.3.1. Habitat Restoration and Development

DM can be used for creating, enhancing and restoring ecosystem habitats. A variety of material types including rock, gravel, sand, silt, clay and mixtures can be used as raw material for habitat restoration. However, contaminated DM is unsuitable for this alternative unless proper remediation methods to improve DM’s chemical and biological properties are followed.

The United States has a long history of using DM for habitat restoration. DM has been used in the construction of submerged gravel bar habitats since 1988. In 2010, The National Oceanic and Atmospheric Administration (NOAA) engaged in ecosystem restoration and sediment management in the Louisiana-Mississippi Gulf Coast. In the Great Lakes region, the Cat Island (located near the southern end of Green Bay) restoration project is designed to enhance wetland habitat.

2.3.2. Beach Nourishment

Beach Nourishment involves the use of DM (primarily sandy material) to restore beaches prone to erosion. Compared with other beneficial use alternatives, beach nourishment is a widely used option, especially in the Great Lakes region. According to the Great Lakes Commission (GLC), 17% of sediments dredged form Great Lakes annually is used as beach Nourishment. Thirty-one harbors located around the Great Lakes have included beach nourishment as a primary DM disposal method (Zande, et al, 1994). From 1987 to 1988, approximately 1.5 million cubic yards of gravelly sand was used for constructing the 72-acre North Point marina on the Illinois shore. As of 1999, 40,000 cubic yards of DM was placed around Ohio and Pennsylvania harbors.

2.3.3. Parks and Recreation

Recreational activities require corresponding facilities, such as trails for hiking and water access for fishing. All soil types can be considered for beneficial use in this context. In 2012, approximately 100,000 cubic yards of dredged material from the Havre de Grace Yacht Basin in Maryland, for example, was used for building a walking trail on top of the area’s dikes in a recreational area.

2.3.4. Agriculture, Forestry, Horticulture and Aquaculture

DM can be used to replace eroded topsoil, elevate the ground surface, or improve the physical and chemical characteristics of soils. Physical properties (e.g., gradation, texture and water content) significantly affect suitable use of DM in such applications. For instance, vegetables grow best on sandy loam soils of good texture, drainage, and aeration. Therefore, sandy or silty DM rather than clay is preferred for this beneficial use option. On the other hand, based on consideration of the chemical and biological aspects, organic matter is another important component in DM and can provide proper conditions to enhance soils. In contrast, high contaminant (e.g., heavy metal) levels are undoubtedly harmful for such applications. Planning considerations, site locations, weed infestation potential, and possible salinity problems must also be considered before deciding upon the suitability of a specific DM for agricultural application. In 1979, about 500 acres of the Old Daniel Island Disposal Site in South Carolina had been successfully truck-farmed, and other parts of the site are planted in soybeans.

2.3.5. Strip-Mine Reclamation and Solid Waste Management (Landfill Capping)

The most important characteristic of DM for this beneficial use option is low permeability. There are several examples of recent success in this application. In the Bark Camp Mine Restoration Project in Pennsylvania, DM blended with alkaline-activated coal ash was used as manufactured fill for abandoned mine reclamation with positive environmental benefits. In over five years of surface water and ground water monitoring, there was detection of semi-volatile or volatile organic compounds, pesticides, PCBs, dioxins. DM can also be used for daily cover, capping and closure of landfills.

2.3.6. Construction and Industrial Development

DM can be used as raw material for manufacture of concrete, asphalt, bricks and other construction materials. By adding fly ash or other stabilizers, the physical and chemical properties of raw DM can be improved to fulfill the requirements of these construction materials. Coarse-grained DM can be used as raw material for asphalt, as fill material, or to improve the physical properties of soils for construction of buildings, roads and bridge abutments. DM with a high percentage of clay can be mixed with cement and stabilizer to create cement-like bricks. DM can be dewatered, mixed with shale fines, extruded into pellets and fired in a kiln, which can be used as raw material for the manufacture of lightweight concrete, thus reducing the need for extractive mining operations.

2.4. Beneficial Use in Transportation Construction

Potential applications for beneficial use of DM in construction of transportation facilities include use in pavement structures (e.g., embankment, subgrade, base and sub-base), structural fills, and backfills behind retaining walls such as Mechanically Stabilized Earth (MSE) walls. In 1999, the New Jersey Department of Transportation (NJDOT) constructed two roadway embankments to study the feasibility of beneficially reusing Stabilized Dredged Material (SDM). Construction of a parking lot for the Jersey Garden’s Mall in New Jersey used approximately 600,000 cubic yards of SDM as structural fill.

Determining the efficacy of beneficial use in transportation construction requires understanding of geotechnical and structural elements of common transportation systems. Barriers to optimal use of DM for beneficial use include an inconsistency between screening metrics (e.g., gradation) and the way they can be applied (Brandon and Price, 2007). For example, fine-grained soil such as clay is generally not suitable for backfills in MSE walls due to its low permeability and strength. However, fine-grained material can potentially be used as geotube infill or regular fill in raising the elevation of depressed areas and in generating topsoil for landscaping purposes. Identifying relevant material characteristics is also important. Specific geotechnical properties need to be considered for essentially all earthwork applications in the transportation sector (e.g., grain size distribution, Atterberg limits, and compaction characteristics). Pavement design requires assessment of resilient modulus and durability characteristics (durability to freeze-thaw and wet-dry cycles). Design of structural fills or wall backfills requires consideration of shear strength affecting slope stability and hydraulic conductivity affecting drainage. The following chapter summarizes relevant geotechnical properties such specific applications.

CHAPTER 3: GEOTECHNICAL PROPERTIES REQUIRED FOR TRANSPORTATION CONSTRUCTION APPLICATIONS

3.1. Scope

This chapter provides a summary of geotechnical properties required for five representative transportation projects, including earth embankments, pavement base, sub-base, and subgrade, and backfill material for Mechanically Stabilized Earth (MSE) walls. Information in this chapter is synthesized from American Association of State Highway and Transportation Officials (AASHTO) and Wisconsin Department of Transportation (WisDOT) design guidelines.

3.2. Embankments

According to the American Association of State Highway and Transportation Officials (AASHTO), a roadway embankment is a raised structure of soil, soil-aggregate or rock. According to the Wisconsin Department of Transportation (WisDOT) Construction and Materials Manual (CMM), the success of a constructed embankment to support a pavement structure depends upon proper preparation of the foundation, use of suitable materials, and proper material placement and compaction. Particle size distribution (gradation) and Atterberg limit indices (plasticity) can be used to determine soil classification (suitable material) according to either Unified Soil Classification System (USCS) or AASHTO standards. The Proctor compaction test is recommended to determine the suitability of a specific material to be used as structural material in one of the different layers of road construction (Siham et al 2008). Therefore, for constructing roadway embankments, suitable materials should fulfill the relative requirements from the specification of AASHTO and Departments of Transportation (DOTs) in various states, especially with regard to physical properties (e.g. gradation) and engineering properties (e.g., compaction).

AASHTO provides specific requirements for soil used as embankment fill. Coarse-grained soils with low plasticity (plasticity index PI less than 10) or non-plastic soils are a primary preferred option, including materials classified in the A-1, A-2-4, A-2-5 or A-3 groups (Table 3.1). Coarse grained soils with relatively high plasticity (PI above 11) , such as A-2-6 and A-2-7 groups, and fine grained soils (silty soils and clayey soils), such as A-4, A-5, A-6 and A-7 groups can also be considered as an alternative when materials in former groups are not available. The WisDOT CMM also indicates that silty soils and clays are suitable for embankments when dried to optimum moisture. DM consisting of primarily fine-grained soils (as in most CDFs and harbors) is thus potentially applicable as embankment material if simple soil classification is considered the sole basis for suitability.

Compaction is necessary during the construction of an embankment and extremely important for ensuring slope stability and decreasing deformation and long-term settlement. Various DOT specifications provide detailed information about field compaction methods, required thickness and width of compaction layers (lifts), and appropriate compaction equipment for various material types. Proctor (compaction) tests are used to determine optimum water content and maximum dry density. Excessive or insufficient water content can both affect embankment performance negatively.

In 1998, the New Jersey DOT (NJDOT) established a project to assess the suitability of using DM in roadway construction. The project involved the construction of two roadway embankments and an access road using stabilized DM in Elizabeth, New Jersey. From this demonstration project, through using stabilized DM, embankment performance in terms of slope deformations and settlement characteristics was satisfactory according to NJDOT specifications.

3.3. Base and Subbase

Discussion of pavement sub-base and base course construction requires distinction between flexible pavements and rigid pavements. Flexible pavements usually consist of a prepared roadbed (subgrade), sub-base, base and surface course. In contrast, rigid pavements generally include subgrade, sub-base and a pavement slab. The sub-base is located between the subgrade soil and base course (in flexible pavements) or pavement slab (in rigid pavements). Sub-base is not necessary for the pavement if the subgrade soil is of relatively good quality, but can be an economical solution for construction of pavement over poor soils. According to AASHTO, the upper limit of grain size passing #200 sieve must be less than 25%. In other words, granular material is primary option for subbase material. Water content should be equal to or slightly below optimum to ensure the design density, and thus dewatering of DM is anticipated to be a crucial issue for this beneficial use option. In addition to a structural part of pavement, sub-base can be also used to prevent migration of fine-grained subgrade soils into the base course by using dense graded materials, minimize frost action effects by using materials that are not susceptible to frost action, and prevent free water accumulation in the pavement structure by using relative free draining materials.

Unlike the sub-base course, a pavement base course is only applicable in a flexible pavement structure. A base course usually consists of aggregate such as crushed stone or slag, crushed gravel and sand, or a combination of these materials. Since the major function of base is structural support, the requirements for strength, plasticity and gradation are more stringent than for sub-base materials. From the aspect of gradation, requirements for the base course are typically the same as for subbase course materials (i.e., coarse grained soils are suitable.)

DOTs have developed specifications for stabilization of base or subbase course materials. For example, Texas DOT has Guidelines for Modification and Stabilization of Soils and Base for Use in Pavement Structures. Beneficial use of DM can thus be potentially broadened by using stabilizing amendments if the raw DM cannot meet the requirements of base or sub-base course materials.

3.4. Subgrade

The pavement subgrade is that portion of the earth roadbed which, after having been constructed to reasonably close conformation with the lines, grades, and cross-sections indicated on plans, receives the base or surface material. According to AASHTO, the subgrade is regarded as a prepared and compacted soil immediately below the pavement system and extending to such depth that will affect the structural design. Subgrade as one of substructure components is located between embankment and sub-base or base.

In addition to soil classification requirements, the definitive material property used to characterize subgrade soils for pavement applications is the resilient modulus (MR). To improve the general reliability of the road structure, other soil properties, such as compression, permeability (drainage) and freeze and thaw, are also necessarily considered.

According to AASHTO soil classification (Table 3.1), granular materials are more proper than silt-clay material as subgrade. The Group Index (GI) can be used for evaluating the suitability from specific information obtained as part of the soil classification:

$$GI = (F-35)\ [0.2 + 0.005\ (LL-40)] + 0.01\ (F-15)\ (PI-10)$$

F = percentage passing No.200 sieve

LL = Liquid Limit, and

PI = Plasticity Index

Coarse soils with low F and PI have smaller GI than fine grained soils, which means these groups (A-1, A-2 and A-3) of soils are the primary choice as subgrade materials. Subgrade materials play an important role in their resistance to deformation under load. The resilient modulus indicates a basic material property which can be used in mechanistic analysis of multi-layered systems for predicting roughness, cracking, rutting and faulting (AASHTO Guide for Design of Pavement Structure, 1986). Its values are closely related to the various properties of the compacted layer of the subgrade soil.

Compressibility and expansion are other important properties in subgrade soil considerations. In general, fine-grained soils tend to be more susceptible to compressions or expansion. When fine-grained soils are subject to compression and rebound under cyclic load, adequate protection must be provided since small movements of this type may be detrimental to the pavement base and wearing course. Coarse-grained soils, on the other hand, exhibit much less tendency toward compressibility or expansion, which is one of reason why such soils are generally more suitable as subgrade materials. Compressibility and expansion is not only influenced by internal factors, such as soil structure and grain shape, but also by other external factors, such as weather conditions, which may change the water content in subgrade soils. To reduce the undesirable results caused by compression or expansion, one solution is to cover these soils with a greater thickness of selected materials. This method has limited effects when considering beneficial use of DM. Another is to stabilize unsuitable soils with cement, fly ash, or lime.

Organic and frost-susceptible soils are not suitable as subgrade materials. The problem with high organic material is its extremely compressible nature and is exacerbated when deposits are heterogeneous. Organic content can be an appreciable component of DM from some CDFs and harbors. Therefore, it is necessary to consider this characteristic when evaluating the applicability of DM in subgrade or other structural applications. Silt and sand tend to be more susceptible to frost action compared to clay and gravel. Environmental factors (e.g., weather and temperature) also significantly affect frost action, and thus climatic factors needed to be considered when evaluating DM as potential subgrade materials. For example, the climatic zone in the Great Lakes region is characterized as wet-freeze, based on the long-term pavement performance program. This means that a cold climate and supply of water are common during the winter, and thus frost heave tends to occur.

3.5. Backfills in MSE walls

Mechanically Stabilized Earth (MSE) is the term used to describe the practice of reinforcing a mass of soil with either metallic or geosynthetic soil reinforcement, which allows the mass of soil to function as a gravity retaining wall structure (WisDOT). An MSE wall system consists of the original ground, concrete leveling pad, wall facing panels, coping, soil reinforcement, select backfill and any loads and surcharge.

Grain size distribution, permeability, and soil strength are critical properties when evaluating if a material can be used as backfill in an MSE wall application. These characteristics are closely correlated. Gradation is used to differentiate two basic soil types: fine-grained soil and coarse-grained soil, which in turn affects permeability and shear strength. Compared to fine-grained soil, coarse-grained soil has higher hydraulic conductivity and strength (friction angle), which are critical properties to consider for backfill applications (Table 3.2).

Figure 3.1 indicates the upper limit of gradation for backfill soils based on synthesis of specifications from WisDOT, AASHTO, and the National Concrete Masonry Association (NCMA). Due to potential drainage and strength problems with fine-grained soils, 48 states limit the material passing the #200 (75 μm) sieve to no more than 15%, which conforms to the AASHTO requirement (Christopher and Stulgis, 2005). In general, fine-grained soil (at least 50% finer than #200 sieve), especially that with high plasticity, has limited use for backfill applications.

Permeability is another important soil property in backfill considerations. Drainage is crucial for MSE wall performance, since poor backfill drainage can lead to elevated pore pressure, a decrease in effective stress, low soil strength, and correspondingly large lateral forces on the wall. Permeability decreases with increasing percentage of fines. During wetting of reinforced soil, pore water pressure generation and loss of strength are inevitable if drainage is poor.

MSE wall design generally consists of three analyses: working stress, equilibrium, and deformation. All three analyses need to consider the soil strength. Internal friction angle and shear strength are extremely useful properties when evaluating the suitability of soil as backfill and measuring the safety factor of slopes. According to AASHTO, a 34 degree friction angle is a minimum value permitted, since that angle is approximately the shear strength that will mobilize in the structure for most granular soils meeting the gradation requirements (Anderson, et al, 2012).

There are many other properties affecting backfill soil performance, such as modulus (Christopher, 1993), compaction (compressibility), shrink and swell potential and frost susceptibility. All of these factors are important considerations in the performance of backfill soil when using relative high percentage fine grained soil that still fulfill the AASHTO or DOTs' specifications.

High quality granular is considered primary choice as backfill material in MSE wall applications. To evaluate the beneficial use of DM in such applications, it is necessary to consider the implications of using fine-grained soils (a major component of most DM) as an alternative. In 1998, for example, the Louisiana Transportation Research Center (LTRC) constructed a full-scale reinforced test wall for studying the feasibility of using available low quality silty-clay as an economical and practical solution for the construction of MSE walls where high quality backfill is not readily available. By monitoring the lateral and vertical deformations over four years, it was found that there was a relatively high amount of deformation as compared to conventionally designed walls. LTRC recommended a detailed drainage system behind the MSE walls if using fined grained soils in such applications.

CHAPTER 4: GEOTECHNICAL PROPERTIES AND TEST METHODS

4.1. Scope

This chapter summarizes specific values of geotechnical engineering properties of DM as potential source materials for specific transportation sector uses. Physical properties including particle size distribution, Atterberg limits, density, water content, and organic content all influence the applicability and potential use of DM in construction. Hydraulic conductivity, compaction characteristics, consolidation characteristics, stiffness and shear strength are also relevant engineering properties. Testing standards (Table 4.1) are also discussed in this chapter.

4.2. Physical Properties

4.2.1. Particle Characteristics

Particle Characteristics including grain size distribution and particle shape influence the geotechnical properties of DM and are a primary indicator for assessing the quality and expected performance of construction materials. Grain size distribution (GSD) influences the density and water content. Grain size distribution and particle shape also influence the stability, shear strength, permeability, compressibility, and compactability. ASTM D422 is the standard test method for particle-size analysis of soils (with corresponding AASHTO standard in Table 4.1). Grain shape is also important. Rounded particles tend to provide better workability and easier compaction. Angular particles, on the other hand, tend to interlock and can result in a stable, dense mass capable of significant bearing capacity. The strain required to reach failure is approximately twice as large for angular-shaped particles as that required to reach failure for spherical particles.

4.2.2. Atterberg Limits

The objective of Atterberg limits testing is to obtain basic index information about the fine-grained fraction of soils or to indirectly estimate strength and settlement characteristics. Atterberg limits most commonly measured in practice include the liquid limit (LL) and plastic limit (PL), and can be used to assess the amount of dewatering needed before DM can be handled and processed. The LL, PL, and corresponding plasticity index (PI = LL – PL) are commonly used when investigating DM in harbors and confined disposal facilities (CDFs) or for evaluating suitability of any raw construction material in roadway construction. Some engineering properties, such as shear strength, shrink-swell compressibility and hydraulic conductivity (permeability), can be correlated with Atterberg Limits. The plasticity index (PI), liquidity index (LI), and activity index (AI) are derived from the PL and LL. PI is predominantly related to clay content. Large PI materials generally have a higher percentage of clay than materials having low PI. The effects of water content on the strength of saturated remolded soils can be quantified using the liquidity index. Activity index can potentially be used to identify the type of clay minerals present in raw DM.

4.2.3. Water Content

Water content is one of the most important factors affecting geotechnical properties (compaction, compressibility and shear strength) of DM. High water content in sediments could preclude use of DM in road construction as fill, subgrade or base material. Dewatering of raw DM with high water content may be necessary in roadway construction projects. The relation between density and water content determined via compaction testing is also important in applications such as pavement bases or fills.

4.2.4. Organic Content

Organic matter from plants, microbes, and carbonaceous materials is often prevalent in DM. In some cases, high levels of organic matter has some benefits, such as in applications requiring improved water infiltration (permeability). More generally, however, high organic content material is not desirable for use in roadway construction. Soils with high levels of organics generally have lower shear strength, higher compressibility, and higher shrinkage potential than those composed mainly of inorganic minerals. High shear strength, low compressibility, and low shrinkage potential are all important characteristics when evaluating material suitability in construction. According to NYDOT specifications, raw materials for embankments should be inorganic. Soils containing greater than 3% by dry weight calcium, magnesium carbonate, or organic material are generally not allowed within the specified thickness of the subgrade.

4.3. Engineering Properties

4.3.1. Hydraulic Properties

Hydraulic properties include permeability and hydraulic conductivity. Permeability is dependent on the pore size, pore geometry, and pore size distribution, and is independent of the fluid properties, whereas hydraulic conductivity is dependent on fluid properties. Permeability is one of the factors that influences shear strength through its influence on pore pressure and corresponding effective stress. Permeability also is an important indicator of the degree of frost susceptibility. Silts or silty sands with relatively low permeability can be susceptible to severe frost action. ASTM D2434, D5084, and D5856 are the major test methods for determining of the coefficient of permeability in granular soils that are primary materials for building embankments and bases.

4.3.2. Compaction

Compaction of porous material increases the amount of solids per unit volume. Compaction generally improves engineering properties so that the required shear strength, structure, and void ratio are obtained, while decreasing the shrinkage, permeability, and compressibility. Compaction is often required when building sub-grades or bases for airport pavements, roads, embankments, earth fill dams, or similar structures.

Laboratory Proctor tests and California Bearing Ratio (CBR) tests are two commonly used compaction tests in transportation-related construction. Procter tests include the standard, modified, and the 15-blow compaction tests. The standard compaction test is generally used in routine foundation and embankment design to simulate field compaction; the modified compaction test is used when a higher level of compaction is desired; and the 15-blow compaction test is used when lower levels of compaction are required. The standard Proctor test (ASTM D698) is for coarse-grained soils and low-plasticity fine-grained soils. For most DM, with medium to high plasticity and fine grained soils, the modified Proctor test (ASTM D1557) may be more suitable.

The CBR test (ASTM D1883) is used to determine resistance to penetration of a material (sub grades or bases). Its primary use has been in the design of flexible pavements located in areas where frost action is not a controlling factor. Since moisture affects the results, tests must be conducted using a moisture content that approximates the moisture content anticipated at the site where the pavement is to be constructed. CBR values usually range from 3 to 80 depending on the type of material tested.

4.3.3. Consolidation

Consolidation tests are required to estimate long-term settlement and plastic deformation likely to occur when soil is subjected to increasing pressures or loads and to determine the compressibility of the material. It is a rate process based on the time required for pore fluid to flow out of soil pores (void-ratio reduction).The rate of consolidation is dependent on (a) the degree of saturation, (b) the coefficient of soil permeability, (c) the nature of pore fluid (air or water), and (d) the distance the pore fluid has to travel for equilibrium to occur. The amount of consolidation or settlement likely to occur must be determined before DM is used as a base or subgrade. ASTM D2435 is standard test method for one-dimensional consolidation properties of soils.

4.3.4. Stiffness

Relevant stiffness tests mainly include the Resistance Value (R-value) test and Resilient Modulus (MR) test. The Resistance Value (R-value) test procedure quantifies a material’s resistance to deformation as a function of the ratio of transmitted lateral pressure to applied vertical pressure. According to WisDOT specifications, the R-value test is necessary for evaluating soils as subgrade materials. ASTM D2844 is the standard method for testing R-value and expansion pressure of compacted soils.

Resilient Modulus is a dynamic soil property determined from the ratio of axial cyclic stress to the recoverable strain. A material's resilient modulus is an estimate of its modulus of elasticity (E). While the modulus of elasticity is stress divided by strain for a slowly applied load, resilient modulus is stress divided by strain for rapidly applied and repeated loads such as those experienced by pavements. The resilient modulus test provides a means of characterizing base, sub-base and subgrade materials for the design of pavement systems. It indicates basic material properties which can be used in mechanistic analysis of multilayered systems for predicting roughness, cracking, rutting, and faulting. AASHTO T307 is the standard method for testing Resilient Modulus of subgrade soils and untreated base/subbase materials. AASHTO T292 is followed to prepare and test untreated subgrade soils and base/subbase materials for determination of resilient modulus. AASHTO also allows using CBR and R-value to estimate MR if the equipment for performing the resilient modulus test is not available. For fine grained soils, the following equations can be used to evaluate the MR:

$$MR\ (psi) = 1500 * CBR$$

$$MR = 1000 + 555 * R\text{-value}$$

4.3.5. Shear Strength

Shear strength is an important engineering property when evaluating DM as pavement structural materials or backfills in retaining wall systems. When using materials as embankment or backfills, shear strength parameters (undrained shear strength, cohesion, and friction angle) are typically used determine the safety factor of slope. Shear strength parameters may be determined using a number of laboratory and field tests.

CHAPTER 5: PROPERTIES OF DREDGED MATERIALS FROM SELECT GREAT LAKES LOCATIONS

5.1. Scope

This chapter summarizes geotechnical properties of representative DM samples from select harbors in the Great Lakes region: West Arm-Burns harbor, Waukegan harbor, Indiana harbor, Calumet harbor, and Milwaukee harbor. Results from West Arm-Burns, Waukegan, Indiana, and Calumet were synthesized from reports available in the literature. Results for the Milwaukee harbor material were obtained in the UW-Madison laboratory using representative samples obtained on site.

5.2. West Arm-Burns Harbor

5.2.1. Introduction

West Arm-Burns Harbor is located in Porter County, Portage, Indiana (Figure 5.1). Results described here were synthesized from the Final Report for The Harbor Boring Project West Arm-Burns Harbor, Portage, Indiana (August 2003). Geotechnical characteristics were reviewed for material sampled from the east seawall of the harbor, including samples from two soil borings spaced approximately 1500 feet apart (BH-01-03 and BH-02-03). Analysis included physical index properties (particle size distribution, Atterberg limits, water content) and mechanical properties (unconfined compressive strength). Table 5.1 indicates the soil classification of raw DM samples from both boring locations. According to the borehole log, saturated silty fine sand (SM) and silty clay (CL) were encountered at boring location BH-01-03. At boring location BH-02-03, clay with various density, ranging from soft to very stiff, was found over a range of depths. Table 5.2 is summary of corresponding geotechnical properties.

5.2.2. Physical Properties

A total of four particle size distribution tests (ASTM D2217) and five Atterberg limits tests (ASTM D4318) were reported in the 2003 final report. As Figure 5.2 indicates, the particle size distribution and corresponding Atterberg limits of samples from the boring BH-01-03 (samples SS-1-1, SS-1-5, and SS-1-10) classify as silty sand (SM). Samples from boring BH-02-03 classify predominantly as low plasticity clay (CL). Liquid limit and plasticity index does not vary significantly (Figure 5.3). According to Figure 5.4, water contents from different depths at the two locations tend to remain relatively constant and have an average value of 20.9 %.

5.2.3. Engineering Properties

Unconfined compressive strengths of representative materials are 5200 psf and 7400 psf at strain levels of 14.9% and 16.2%, respectively. Corresponding undrained shear strength, calculated as one half of the unconfined compressive strength, ranges from 2600 psf to 3700 psf.

5.3. Waukegan Harbor

5.3.1. Introduction

Sediments in Waukegan Harbor (Figure 5.5) located in Illinois have been researched for several decades. Representative geotechnical properties for DM in the harbor, including grain size, plasticity, density, compaction characteristics, and shear strength properties were obtained by review of a report associated with those efforts. (Summary of Sediment Sampling Events and Analytical Results for Waukegan Inner Harbor and Entrance Channel, April 1998 and Data Evaluation Summary Report Waukegan Harbor Area of Concern, Waukegan, IL, April 2005).

5.3.2. Physical Properties

As summarized in Table 5.3 and Figure 5.6, major soil types are silt and sand (67% and 22% respectively). Five of the nine total samples considered can be classified as ML (low plasticity silt) (Figure 5.7). Water content tends to vary significantly and can be as high as 80% to 120% (Figure 5.8). Organic content measured for of 44 samples in the harbor indicates that ten samples have organic content higher than 5%, with an average value for all samples of 3%.

5.3.3. Engineering Properties

Results from standard Proctor compaction tests to determine optimum water content and maximum dry density are summarized in Table 5.4. Results from direct shear tests to determine cohesion intercept and friction angle are also synthesized in the table.

5.4. Indiana Harbor

5.4.1. Introduction

The Indiana Harbor and Canal (Figure 5.9) is an artificial waterway located on the southwest shore of Lake Michigan, in East Chicago, Indiana. The Main Canal connects the Grand Calumet River to Lake Michigan from two branch canals through Indiana Harbor. Representative geotechnical properties for DM in the harbor, including grain size, plasticity, density, consolidation characteristics, hydraulic conductivity, and shear strength properties were obtained by review of reports from sampling performed in the Harbor and Main Canal, near the harbor. (Sediment Sampling and Analysis Report Indiana Harbor and Canal Harbor, Indiana September 2010 and Geotechnical Engineering Services For the Indiana Harbor Confined Disposal Facility Chicago CDF Borrow Source Material Testing Project, September 2009).

5.4.2. Physical Properties

As summarized on Figure 5.10 and Figure 5.11, representative samples classify as CL (low plasticity clay). Water content changes variably and specific gravity tends to remain constant (Table 5.6).

5.4.3. Engineering Properties

Hydraulic conductivity, triaxial shear strength and standard compaction test results are summarized in Table 5.6.

5.5. Calumet Harbor (Chicago Area CDF)

5.5.1. Introduction

The Chicago Area confined disposal facility (CDF) is located on the southern corner of the intersection of Lake Michigan and the Calumet River (Figure 5.12). Representative geotechnical properties, including grain size, plasticity, density, consolidation characteristics, and shear strength properties were obtained by review of reports from the US Army Corps of Engineers (USACE). (Collection and Analysis of Environmental Samples for Calumet Harbor and River Dredged Material Management Plan (DMMP), July 2006).

5.5.2. Physical Properties

Based on grain size distribution (Figure 5.13), representative materials at the site fall into the general category of fine-grained soils. Other physical properties, such void ratio, density, water content, and specific gravity are summarized on Table 5.7.

5.5.3. Engineering Properties

Results from two triaxial compressions tests (CU and UU) are summarized on Table 5.9. Figure 5.14 indicates the relationship between applied load in a 1D consolidation test and coefficient of consolidation.

CHAPTER 6: IMPLEMENTATION OF A BENEFICIAL USE FRAMEWORK

6.1. Scope

As described in Chapter 1, the overall goal of this project includes several major objectives. Guidelines are being developed to link: 1) applications for use of DM in transportation-related projects, 2) required geotechnical properties, 3) available geotechnical test methods, 4) geotechnical specifications for specific uses, and 5) locations within the Great Lakes region where dredged materials meeting these specifications may be sourced. Previous chapters have addressed objectives 1, 2, 3, and 4. Chapter 5 summarized geotechnical properties from five select DM sources in the Great Lakes region. In this chapter, a framework for evaluating the potential use of DM in transportation projects is demonstrated for those select materials.

6.2. Framework Demonstration

The framework herein is derived primarily from Wisconsin DOT (WisDOT) specifications for earthwork construction. WisDOT standard specifications delineate geotechnical properties of soils in several transportation applications. Table 6.1 summarizes three earthwork applications (base, sub-base, and backfill), corresponding geotechnical properties of importance, and the corresponding American Association of State Highway and Transportation Officials (AASHTO) testing standards for determining these properties.

Table 6.2 is a more general summary of typical engineering characteristics for specific soil types and corresponding rating (applicability) in various transportation sector applications. Columns 1 and 2 show the USCS soil classification including major divisions and specific group symbols. Columns 3 and 4 give typical ranges of optimum water content and corresponding maximum dry unit weight based on standard proctor, AASHTO T99 (after Carter and Bentley, 1991). Columns 5 and 6 indicate typical ranges of cohesions and friction angles of different soil groups (www.geotechdata.info). Column 7 shows the typical ranges of permittivity of different soil groups (after Casagrande and Fadum, 1940). Column 8 evaluates drainage characteristics based on permittivity of soils (Sowers, et al. 1970). Column 9 shows the typical ranges of CBR value of soils (FM5-410, Military Soil Engineering). Column 10 evaluates the compressibility and expansion characteristics of soils (FM5-410, Military Soil Engineering). Column 11 evaluates the potential frost action of soils (FM5-410, Military Soil Engineering). Column 12 evaluates the compaction characteristics of soils (Sowers, et al. 1970). Column 13 evaluates soils value as embankment based on material suitability. Column 14 evaluates soils value as subgrade materials (FM5-410, Military Soil Engineering). Column 15 evaluates soils value as subbase courses (FM5-410, Military Soil Engineering). Column 16 evaluates soils value as base courses (FM5-410, Military Soil Engineering). Column 17 evaluates soils value as backfills in MSE wall.

Figure 6.1 is a flow chart developed in accordance with WisDOT specifications. The flow chart is intended to guide identification of suitable dredged materials for specific transportation applications. Vertical arrows with a “yes” in the flow chart indicate that the material fulfills the geotechnical requirements of the corresponding level. Horizontal arrows with a “no” indicate the material does not meet the specification.

6.3. Results

Based on the limited geotechnical information evaluated in available reports (Chapter 5), the representative materials in Indiana Harbor, West Arm-Burns Harbor and the Chicago area CDF may be considered clay with low plasticity (CL) (Table 6.3). Representative Waukegan Harbor material is considered low plasticity silt (ML). Average organic content in the Waukegan Harbor material is relatively low. However, the organic matter in DM from Chicago Area CDF is relatively high.

Considering the framework outlined in these figures and tables, un-amended or “raw” DM from Indiana Harbor, West Arm-Burns Harbor, the Chicago area CDF, and Waukegan Harbor could potentially be considered as embankment construction material. No material meets the gradation criteria for use as structural fill, backfill, or base material. Based this evaluation, the material potentially sourced from these locations has limited direct use for transportation-related construction in its raw or un-amended form. Ongoing efforts, therefore, are focusing on quantifying geotechnical characteristics of raw DM from Great Lakes sources stabilized with cementitious materials (e.g., coal combustion fly ash). DM stabilization has been successfully used to enhance strength, reduce compressibility, and modify drainage characteristics.

CHAPTER 7: CASE STUDY: STABILIZATION OF RAW DREDGED MATERIAL WITH FLY ASH

7.1. Scope

Previous research has indicated that fine-grained dredged sediments in their natural state, referred to herein as raw dredged material RDM), may not be suitable as road construction material. Engineering properties of RDM often do not meet construction material specification for various applications, as summarized in previous chapters. To enhance the engineering properties of fine-grained DM, therefore, pozzolanic materials (e.g., lime, cement, and fly ash) may be blended with RDM to produce stabilized dredged material (SDM) with improved engineering properties. The engineering characteristics and more general feasibility of beneficially using SDM stabilized with various materials have been demonstrated through several laboratory testing programs (Grubb et al 2010; Maher et al 2004; Zentar et al 2008) and at field scale (Bennert et al 2000; Maher et al 2003; Sadat Associates Inc. 2001).

This chapter presents results form a laboratory testing series designed to quantify the engineering characteristics of raw, fine-grained DM obtained from the Milwaukee harbor CDF after stabilization with fly ash. The testing series had three primary objectives: (1) to investigate the improvement of SDM in geotechnical properties that are relevant to roadway construction, (2) to evaluate the effect of fly ash content and curing time on SDM, (3) to study the relationship among geotechnical properties of the RDM and SDM.

7.2. Materials and Methods

7.2.1. Dredged Material

DM samples were collected from the ground surface in a disturbed manner (using a shovel and bucket) at depths ranging from 0.2 m to 0.5 m from the Milwaukee Harbor (N 43^o 00’ 26.0”; W 87^o 53’ 22.9”) confined disposal facility. The Milwaukee CDF is an in-lake facility located at the south end of Milwaukee Harbor. Figure 7.1 shows the location of the Milwaukee Harbor CDF.

Major physical and engineering properties of the RDM are summarized in Table 7.1. RDM sampled from various locations throughout the CDF had in-situ water content as high as 67.3% and contained as much as 96.6% fines. The ratio of the liquid limit (LL) of a representative oven-dried sample to that of the sample in an air-dried state was 0.87, thus indicating that the sample is an inorganic material. The RDM is classified as a high plasticity silt (MH) according to the Unified Soil Classification System (USCS) and as A-7-5 according to the American Association of Highway and Transportation Officials (AASHTO). For the proctor compaction test, samples were evaluated and prepared by using a Harvard Miniature compactor following modified compaction method (ASTM D698).

7.2.2. Fly Ashes

Self-Cementing Class C Fly ash, which has relatively high relatively high CaO content (compared with Class F fly ash), has been shown to significantly improve the engineering properties of both inorganic (Ferguson 1993) and organic soils (Tastan et al 2011). Therefore, it is considered as an effective stabilizing material for a large quantity of construction applications (Mackiewicz and Ferguson 2005). Class C fly ash has been used alone to stabilized soils. The basis for stabilization is, when fly ash is blended with soil and water, the series of reactions lead to dissociation of lime (CaO) and the formation of cementitious and pozzolanic gels (Tastan et al 2011). During the hydration process, free lime reacts pozzolanically with the clay and this reaction reduces clay plasticity (Litter and Nair 2009).

Fly ash for this study was obtained from the Oak Creek power plant in Oak Creek, Wisconsin. The fly ash classifies as Class C ash according to ASTM C618. The general chemical properties of the fly ash is summarized in Table 7.2.

7.3. Methods

As described in previous chapters, index and geotechnical characteristics of DM are necessary to achieve specifications with considering potential use of DM in construction applications. Specifications for physical and engineering properties are typically evaluated through the following tests: grain size distribution, specific gravity, Atterberg limits, organic content, hydraulic conductivity, compaction, frost susceptibility, unfrozen moisture content, resilient modulus, and CBR (Mallick and El-Korchi 2009). According to ASTM D7762, testing procedures for mechanical properties of self-cementing fly ash stabilized materials include CBR, resilient modulus, unconfined compressive strength, and freeze-thaw testing. Strength tests, such as the UU test are also often required to demonstrate, successful beneficial use of DM in transportation projects (DM was used as roadway embankments at New Jersey in 2001). In the following, comparisons are made between RDM and SDM samples at various curing times in terms of in Atterberg limits, compaction properties, undrained shear strength, resilient modulus, CBR, unconfined compressive strength, and freeze-thaw cycling.

Class C fly ash and DM were used in all tests. Three different fly ash contents were evaluated, specifically 10%, 20%, and 30% (by the total dry weight of RDM and fly ash). These are respectively designed FA10D, FA20D, and FA30D, as summarized in Table 7.3. After mixing thoroughly, each mixture was then subdivided into 3 groups to evaluate the effects of curing time, including curing for 2 hours, 7 days and 28days. A complete summary of the testing program including the number of specimens for each test and corresponding ASTM or AASHTO testing standard followed is presented in Table 7.4.

7.3.1. Proctor Compaction Procedures

To prepare compacted specimens for subsequent use as specimens for the unconsolidated undrained (UU) test, unconfined compressive test, CBR test, and resilient modulus test, the RDM samples were first air-dried. Samples were then processed to pass through the No.4 sieve (4.75 mm). RDM samples were blended by using a spatula with Class C FA to 10%, 20%, and 30% by weight. Five subsamples of each were then mixed with various amounts of tap water (ranging from 10% to 40% by mass) and compacted into a steel mold with a diameter of 33 mm and height of 71mm using a Harvard Compactor, which produced using an effort equivalent to the modified Proctor effort according to ASTM D698.

Typical bell shaped compaction curves were obtained for all specimens with different FA content (Figure 7.2 a). In general, as the FA content increased, the maximum dry unit weights of all specimens increased and optimum water contents decreased (Figure 7.2 b). The FA10D and FA20D samples had approximately the same maximum dry unit weights (13.3 – 13.4 kN/m^3) and optimum water content (26.0 – 26.5%). Subsequent geomechanical tests were conducted using specimens compacted to optimum water content and maximum dry unit weight as obtained from the proctor tests.

7.3.2. Atterberg Limits Procedures

Following ASTM D4318, air-dried RDM samples passing through the No.40 sieve (475 μm) were used for the Atterberg limits tests. Different amounts of tap water were separately added to the RDM, FA10D, FA20D, and FA30D to approximately achieve optimum water content based on previous proctor tests. After thoroughly mixing the samples in sealed plastic bags, each was divided into three groups and allowed to cure for curing 2 hours, 7 days, and 28 days in a moisture room maintained at 100% relative humidity and 25 Celsius. Additional tests were conducted using samples tested immediately after mixing. The 2 hour curing time was selected to more accurately represent field construction conditions more accurately (Senol et al 2004). Specimens cured 7 days and 28 days were selected to represent early and relatively long term curing conditions in roadway construction applications

7.3.3. Unconsolidated-Undrained Strength Procedures

In the UU test, specimens are sheared in compression without drainage by applying constant rate of axial deformation. The undrained shear strength obtained from the UU test is important to evaluate performance of roadway construction materials performance in the short term (i.e., undrained loading). As summarized in Table 4, three specimens were prepared for UU testing at each specific FA content. All specimens were prepared using the Harvard Miniature Compaction method, wrapped and sealed immediately with plastic wrap to minimize possible moisture change, and then cured in the moisture room (100% relative humidity and 25 Celsius) for 2 hours, 7 days, and 28 days. Cured specimens were tested for undrained shear strength according to ASTM D2850 under 100 kPa isotropic confining pressure.

7.3.4. Freeze-Thaw Cycling Procedures

The freeze-thaw (F-T) cycling tests in this study followed procedures from ASTM D560 in terms of number of cycles, cyclic duration and temperature conditions. Specimens were prepared to optimum water content and maximum dry unit weight using a Harvard compactor. After sealing with plastic wrap, the specimens were placed in a freezing cabinet having a constant temperature lower than -23 Celsius for 24 hours. Following the freezing stage, all specimens were placed in the moisture room maintaining a temperature of 25 Celsius and a relative humidity of 100% for 23 hours. Freeze-thaw cycles including one cycle (48 hours) and 12 cycles (24 days) were considered in this study.

7.3.5. Unconfined Compressive Strength Procedures

Two groups of cured samples were tested for unconfined compressive strength: one group was not subjected to F-T cycling and one group was subjected to F-T cycling. For the first group, specimens compacted using a Harvard compactor to optimum water content and maximum dry unity weight were sealed and then placed in moisture room for 24 days. For the second group, samples that had undergone 12 F-T cycles (24 days) were tested. Strain rate in both cased was 1 %/min according to ASTM D 2166.

7.3.6. CBR Test Procedures

The California Bearing Ratio (CBR) test is a penetration test that can be used to evaluate the strength of materials for potential use as pavement subgrade, subbase, and base course material. Following ASTM D1883, air-dried RDM passing through the No. 4 sieve (4.75 mm) and blended with FA (0%, 10%, 20%, and 30% by weight) were compacted to optimum water content and maximum dry density into a steel mold with a height of 152.4 mm (6 inches) and diameter of 116.8 mm (4.6 inches). Materials were compacted in 5 layers and using 25 blows for each layer. Compacted specimens were then sealed with plastic wrap and placed in the moisture room for 2 hours, 7 days, and 28days. Cured specimens were then placed in a water bath for 96 hours of soaking to simulate the worst-case conditions under which pavements may perform (Mallick and El-Korchi 2009). After soaking, a standard CBR piston was used to penetrate the specimens at a constant rate of 1.27 mm (0.05 in.) /min.

7.3.7. Resilient Modulus Test Procedures

As described in the AASHTO Guide for Design of Pavement Structures (AASHTO 1986), resilient modulus is a measure of the elastic property of soils applicable to pavement design. The main advantage of resilient modulus tests is that dynamic loading, as opposed to static loading in the strength tests discussed above, is applied to the materials. This is intended to simulate stress conditions for pavement systems in actual field conditions under dynamic traffic loading.

Specimens for resilient modulus tests were prepared using the same compaction effort as specimens prepared using the Harvard miniature compaction procedures. The mold used to prepare the resilient modulus specimens had a diameter of 102 mm (6 in.)

and height of 203 mm (12 in.). Specimens were compacted in the mold in 5 layers with 25 blows per layer using a Modified Proctor hammer. As with the CBR tests, specimens were prepared and assumed to achieve optimum water content and maximum dry unit weight. All specimens were then extruded from the mold after compaction, sealed with plastic wrap, and cured at 25°C and 100% humidity for curing periods of 2 hours, 7 days, and 28 days. Procedures described in AASHTO T 307-99 were followed using the loading sequence for cohesive soils. A deviator stress of 21 kPa was used as that is typical subgrade conditions (Edil et al 2006).

7.4. Results and Analysis

7.4.1. Atterberg Limits

Figure 7.3 is a summary of Atterberg limits testing in the form of a Casagrande Plasticity Chart (PI vs. LL).

In general, as fly ash content increases, both LL and PI decrease for all the specimens. There is linear relationship between LL and PI for the entire suite of RDM and SDM materials having different fly ash content and curing time (R^2 = 0.93). The slope of trend line of DM – fly ash mixtures chart is 0.7, which indicates this trend line is approximately parallel to A – line (the slope of A - line is 0.73). The Linear relationship between the LL and PI may be not the specific characteristic for DM. Other fine-grained soils also show the similar property. Figure 7.4 illustrates the plasticity chart of fine-grained soils and DM. For high plastic clay (CH) and low plastic clay (CL), both of they indicate linear relationship (R^2_{CH} = 0.97 and R^2_{CL} = 0.99) between LL and PI for specimens having different fly ash content (Table 7.5). For DM used for constructing for embankments for New Jersey DOT (NJDOT), compare to testing sample in this study, its soil category is also classified as high plastic silt (MH) and it stabilized with different cementitious material-Portland cement. However, as Figure 7.4 shows, scatter points on the plot don’t show a strong linear relationship ($R^2_{MH(DM)} = 0.48$) between LL and PI for DM having various curing time and cement content.

Figure 7.5 illustrates the effect of curing time. Specimens cured for different times have similar decreasing trend as fly ash contents increases, which indicates that the effect of curing time on reducing the plasticity of SDM is limited. In contrast, for the effect of fly ash content, Figure 7.6 shows, fly ash content can significantly affect the plasticity of SDM materials. For relatively low fly ash content (Figure 7.6a), the range of decreasing plasticity is limited. When the fly ash content reaches to 30% (Figure 7.6c), a significant decrement of plasticity of the specimens with different curing time may be observed.

7.4.2. Undrained Shear Strength

Triplicate UU specimens were tested for undrained shear strength (c_u) as quality control. Figure 7.7 reports, average undrained shear strength as a function of FA content for three different curing times. The dashed line in the figure is shear strength of the un-amended RDM. In general, compared to the RDM, the c_u values increase for all SDM samples with increasing fly ash content and curing time. For specimens cured for 2 hours, however, the effect of fly ash content is not as significant. The percentage increase in c_u for the 2 hour curing time specimens ranges from 6.2% to 22% compared to the RDM. By increasing the curing time, the improvement in c_u at different fly ash increases significantly. The percentage increase over c_u for the RDM for specimens cured for 7 days ranges from 29.1% to 108%. Percentage increase for specimens cured for 28 days ranges from 55.4% to 197.5%. The effect of curing time on c_u for specimens with high fly ash content is also more significant than for low fly ash content. For the FA10D, for example, c_u for specimens cured for 2 hours and for 28 days increases 46.3%. For the FA30D, c_u for specimens cured for 2 hours and for 28 days increases 143.6%.

7.4.3. Freeze and Thaw Cycling and Unconfined Compressive Strength

One objective of this research was to investigate how F - T cycling and fly ash percentage affects unconfined compressive strength (UCS) of DM - fly ash mixtures. As noted previously, specimens were divided into two groups: one tested with 12 F - T cycles and one without 12 F - T cycles but cured for the same period of time (24 days). A closed system (no external source of water available) was used in this study to prevent effects due to possible changes in moisture content during the F-T cycling. Triplicate specimens were tested for quality control, and the averages of these tests are reported as results.

Unconfined compressive strengths (UCS) of the RDM and SDM samples with and without 12 F-T cycles are shown as a function of fly ash in Figure 7.8. As the fly ash content increases, UCS values for all specimens, regardless of whether or not it is subjected to F-T cycling, tends to increase. However, the range increase in UCS is different. By comparing RDM and FA10D, the UCS values for specimens with F – T cycles and without F – T cycles increases 31.8% and 30.3%. When fly ash percentage increases from 10% to 20%, the UCS values of specimens with F - T cycles and without

F - T cycles increase only a limited amount: 6.3 % and 2.6 %, respectively. From 20% to 30% fly ash, both of UCS values increase more significantly, reaching 43.2% for specimens with F – T cycles and 56.6% for specimens without F – T cycles. Similar trends were also observed in changes of maximum dry unit weight and optimum water content due to different fly ash percentage in previous proctor tests. All of the specimens show a reduction in UCS values after F - T cycling. The decrement of UCS is relatively low for RDM, FA10D, and FA20D (ranges from 0.8% to 4.7%). The decrement for the FA30D, on the other hand, is 9.2%.

Figure 7.8 also includes CBR values of other stabilized fine-grained soils. In Senol's research, soil specimens blended with various Class C fly ash content were compacted to maximum dry density and optimum water content and then cured for 7 days in the moisture room. The increase in UCS obtained as the fly ash content increased from 0% to 10% was larger than those obtained when the fly ash content was increased from 10% to 20%.

In general, both fly ash content and F –T cycling can affect the UCS of DM – fly ash mixtures. The effect of F – T cycling on UCS of DM – fly ash mixtures is, however, limited, especially for specimens with low fly ash contents including RDM, FA10D, and FA20D. Additionally, UCS increases slightly when the FA content increases from 10% to 20%, which demonstrates UCS does not increase linearly with fly ash content and DM and soil specimens show similar trend of UCS values change as fly ash increases from 0% to 20%. However, unlike the subsequent change of UCS in DM, for some fine-grained soils (Tastan 2011), the benefits accrued by adding more fly ash diminish when fly ash content increases from 20% to 30%.

7.4.4. CBR

Figure 7.9 shows a general trend of increasing CBR values with increasing fly ash content and curing time. CBR increases significantly from SDM cured for 2 hours to SDM cured for 7 days, which reaches 208% on average. As curing time increases to 28 days, the effect of stabilization on the CBR tends to be constant, which increases 16.4% on average compared to the SDM cured for 7 days.

To study the effect of fly ash content, CBR values of fine-grained soils stabilized with Columbia fly ash (Class C) are also showed in Figure 7.9. These specimens were compacted to optimum water content and cured for 7 days in moisture room before running the CBR tests. For most mixtures, as fly ash content ranges from 10% to 20%, CBR values increase slightly. In contrast, as fly ash content ranges from 0% to 10%, different trends were obtained. For STH 60, TSL, and all SDM specimens, they indicate the similar trend of increasing CBR. For LRC and STH 28, the CBR tend to remain constantly as fly ash content increases from 0% to 20%. All SDM specimens have relatively low CBR values compared to the other inorganic soil specimens, one possible reason causing this phenomenon is RDM has medium organic content (9.8%). Due to various factors including lower solids content, higher water content, lower pH, and chemical interferences that occur in the cementing reactions, soils with high organic content have been more difficult to stabilize than soils with low organic content (Janz and Johansson 2002). Thus, strength of the soil-fly ash mixture significantly decreases as the organic content increases (Tastan et al. 2011).

7.4.5. Resilient Modulus

Figure 7.10 shows that resilient modulus increases with increasing curing time and fly ash content. The resilient modulus tends to increase significantly with fly ash content. For example, M_r of the FA30D specimens ranges between 3.2 and 4.0 times the M_r of the RDM specimens for curing from 2 hours to 28 days. However, The M_r of the FA10D only increases from 1.2 to 1.7 times the M_r of the RDM specimens.

The effect of curing time on the resilient modulus is shown in Figure 7.11. The specimens (RSCT) were prepared with CL (7% optimum water content) mixed with 18% Columbia fly ash (Class C) and 18% Dewey fly ash (off specification). For SDM (FA10D, FA20D, and FA30D) and RSCT specimens, the resilient modulus at each curing time has been normalized by the resilient modulus measured at 2 hours and 14 days, respectively. A significant increase in resilient modulus was obtained in relative early stage in all specimens (between 2 hours to 7 days for SDM and 14 days to 28 days for RSCT). Subsequently, little additional increase in resilient moduli occurs. The reason resilient moduli of RSCT specimens didn’t significantly increase from 7 days to 14 days may be due, unlike the SDM, RSCT specimens were prepared to 7% optimum water content.

Other parameters, such as CBR may also be used to estimate the M_r based on the empirical correlations. Heukelom and Foster (1960), for example, have reported correlations between CBR value and the in situ modulus of soil, as:

$Mr = 10\ CBR$ -- (1)

Through studying the fined grained soils and mixtures of fine-grained soils and fly ash, Edil et al. (2012) suggested:

M_r = 3 CBR-- (2)

Figure 7.12 shows the relationship resilient moduli and CBR values in this study. The data of SDM specimens cured for 2 hours fit Eq. (1) line well. For the SDM cured for 7 days and 28 days, Eq. (2) line is more accurate.

A relationship between resilient moduli and unconfined compressive strengths is plotted in Figure 7.13. Specimens for these two tests were prepared at the same water contents (optimum water contents), same fly ash type and percentages, and approximately the same length of curing (28 days for CBR testing and 24 days for UC testing). Figure 7.13 indicates a linear relationship (R^2 = 0.92) between the resilient modulus and UCS for RDM and SDM specimens in this study. The slope value is 0.32, which is similar to the slope value (0.32) obtained by Tastan et al (2011) for mixtures of organic fine grained soils and fly ash. Results from that study were for small-size specimens (33 mm in diameter and 72 mm in height) for UC testing and standard size specimens (102 mm in diameter and 203 mm in height) for M_r testing.

7.5. Conclusions

The purpose of this study has been to identify the stabilization effect of Class C fly ash on fine grained dredged materials and to evaluate the effects of curing time and fly ash content. Emphasis has been placed on index and mechanical properties that are frequently considered for evaluating materials as roadway construction materials. A laboratory study was conducted where soil–fly ash mixtures were prepared at different fly ash contents (10%, 20%, and 30%) and curing time (2 hours, 7 days, and 28 days) to evaluate how addition of fly ash and increment of curing time can improve engineering properties of dredged materials.

In general, the engineering properties of SDM significantly improve by increasing the fly ash content. However, for the stabilization of construction materials in the field, fly ash maybe not as high as its optimum content for achieving the highest engineering properties in laboratory due to other factors, such as environmental impacts and costs. For instance, Indiana DOT suggested Class C fly ash ranges from 10% to 16% by weight for soil stabilization.

The resilient modulus, as an important property of roadway construction materials, provides a means of characterizing base, sub-base and subgrade materials for the design of pavement systems. Figure 7.14 compares all SDM specimens having different curing time and fly ash content with DMs stabilized with Portland cement (4%PC and 8% PC) for two embankments in New Jersey and subgrade soils that currently underlie roadways (Rt. 23, Rt. 295, and Rt. 206) in NJ. The modulus for FA20D and FA30D with all curing times is higher than that of the base materials taken from Route 206 and Route 295. The resilient modulus for FA30D specimens is higher than 4% PC (cured for 1 month), 4% PC (cured for 6 months), and 8% PC (cured for 6 cured months), which are SDM materials for embankments in NJ.

In summary, regardless of environmental impacts and DOT's specification on fly ash content, engineering properties of SDM can reach their highest values with maximum fly ash content (30%) and longest curing time (28 days). By comparing the resilient modulus of materials that have been successfully on transportation projects, all FA30D specimens may be considered as embankment materials. And FA20D and FA30D have high possibility of beneficial use as base materials on roadway construction.

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TABLES

Table 2.1 Laws and Regulations for Open Water Disposal in Great Lakes Region

State	Permit Open Water Disposal	Law/Regulation
IL	Yes	Must comply with state water quality standards; negative impacts are to be mitigated.
IN	Yes	Must comply with state water quality standards; contaminated sediments are prohibited.
MI	Yes	Must comply with state water quality standards; contaminated sediments are prohibited.
MN	No	Only beneficial use projects that result in an improvement of natural conditions such as habitat enhancement and creation are permitted
NY	Yes	Must follow state management guidelines for sediments classified under specific material categories.
OH	Yes	Must comply with state water quality standards; state wants to gradually phase-out open water disposal.
PA	Yes	Must comply with state water quality standards
WI	No	Open water disposal is a last resort; direct legislative authority is needed.

Source: Great Lakes Commission

Table 2.2 Beneficial Use Options for Dredged Materials (Source: USACE)

Category	Examples of Beneficial Use Activities	Dredged Material Sediment Type				
		Rock	Gravel & Sand	Stiff Clay	Silt/Soft Clay	Mixture[1]
Agriculture/ Product Uses	Aquaculture			x	x	x
	Construction Materials	x	x	x	x	x
	Decorative Landscaping Products		x	x	x	x
	Topsoil				x	x
Engineering Uses	Beach Nourishment		x			
	Berm Creation	x	x	x		x
	Capping		x	x		x
	Land Creation	x	x	x	x	x
	Land Improvement	x	x	x	x	x
	Replacement Fill	x	x			x
	Shore Protection	x	x	x		
Environmental Enhancement	Fish& Wildlife Habitats	x	x	x	x	x
	Fisheries Improvement	x	x	x	x	x
	Wetland Restoration			x	x	x

Note: 1. a mixture of materials such as boulders lumps of clay, gravel, organic matter, and shells, with varying densities.

Table 3.1 Classification of Soils and Soil-Aggregate Mixture

General Classification	Granular Materials[1]							Silt-Clay Materials[2]				
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7	
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5	A-7-6
Sieve analysis:												
2.00 mm (No.10)	50 max	-	-	-	-	-	-	-	-	-	-	-
0.425 mm (No. 40)	30 max	50 max	51 min	-	-	-	-	-	-	-	-	-
75 µm (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min	36 min
Atterberg Limits												
Liquid Limit	-		-	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min	41 min
Plastic Index	6 max		NP	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min	11 min
Usual types of materials	Stone fragments, gravel and sand		Fine sand	Silty and Clayey gravel and sand				Silty soils		Clayey Soils		
General rating as subgrade	Excellent to Good							Fair to poor				

Note: 1, 35 Percent or Less Passing 75 um; 2, More Than 35 Percent Passing 75 um

Source: AASHTO Designation

Table 3.2 Soil Properties in Backfill of MSE Wall

Wall backfill Classification	Description	USCS Classification	Friction Angle (ϕ) Range	Hydraulic Conductivity Range (cm/s)
Good	Sand, Gravel, Stone	GW,GP,GM, GC,SW,SP	32° - 36°	$10^{2} - 10^{-2}$
Moderate	Silty Sands, Clayey Sands	SM,SC	28° - 32°	$10^{-2} - 10^{-6}$
Difficult	Silts, Low Plastic Clays	ML,CL,OL	25° - 30°	$10^{-6} - 10^{-10}$
Bad	High Plastic Silts and Clay, Organics	CH,MH,OH,Pt	0° - 25°	$10^{-6} - 10^{-10}$

Table 4.1 ASTM Designation versus AASHTO Designation

	Test Category	ASTM	AASHTO	Description
Sampling		D75	T2	Sampling Aggregates
Physical Properties	Particle Characteristics	D2488/D3398		Visual classification/Aggregate Particle Shape and Texture
	Sieve Analysis	D422	T88	Particle-Size Analysis (soil)
		C136	T27	Particle-Size Analysis (aggregates)
		D5444	T30	Gradation of Extracted Aggregate
		D2217	T146	Wet Preparation of Soil Samples for Particle-Size Analysis
		C117	T11	Percent Passing The 200 Sieve (aggregates)
		D1140		Percent Passing The 200 Sieve (soil)
	Atterberg Limits	D4318	T89 (LL) T90 (PI)	Liquid Limit, Plastic Limit, and Plasticity Index of Soils
	Organic Matter	D2974	T267	Organic Content (loss on ignition)
	Specific Gravity	D854	T100	Specific Gravity of Soil
	Density	D1556	T191	In-Place Density and Unit Weight (Sand-Cone Method)
		D2937	T204	In-Place Density (Drive Cylinder Method)
		D6938	T310	In-Place Density and Water Content (Nuclear Method)
	Moisture Content	D2216	T265	Moisture Content (soil)
		C566	T255	Moisture Content (aggregates)
Engineering Properties	Compaction	D698	T99	Standard Proctor Test
		D1557	T180	Modified Proctor Test
		D1883	T193	California Bearing Ratio
		D558	T134	Moisture-Density Relations of Soil-Cement Mixture
	Durability	D559	T135	Wetting and Drying Compacted Soil-Cement Mixtures
		D560	T136	Freezing and Thawing Compacted Soil-Cement Mixtures
	Consolidation	D2435	T216	One-Dimensional Consolidation
	Stiffness	D2844	T190	Resistance R-Value and Expansion Pressure of Compacted Soils
			T307	Resilient Modulus of Subgrade Soils and Untreated Base/Subbase Materials
	Shear Strength	D3080	T236	Direct Shear (under consolidated drained condition)
		D2166	T208	Unconfined Compressive Strength of Cohesive Soil
		D2850	T296	Unconsolidated Undrained Triaxial Compression (Q-Test)
		D7181		Consolidated Drained Triaxial Compression (S-Test)
		D4767	T297	Consolidated Undrained Triaxial Compression (R-Test)
	Wear	C131	T96	Resistance to Degradation of Small Size Coarse Aggregate
	Soundness	C88	T104	Sodium Sulfate Soundness (aggregates)
			T103	Freeze/Thaw Soundness (aggregates)
	Hydraulic Properties	D2434	T215	Permeability of Granular Soils (constant head)
		D5084		Hydraulic Conductivity (flexible wall)
		D5856		Hydraulic Conductivity (rigid wall)

Table 5.1 Classification of DM samples from West Arm-Burns Harbor

Soil Classification Type	Group	Number of Samples	Percent of Samples (%)
Total	-	39	100
Gravel	G	0	0
Silty Sand	SM	12	31
Low Plastic Silt	ML	1	2
Low Plastic Clay	CL	26	67

Table 5.2 Geotechnical Results of DM Samples in West Arm-Burns Harbor

Geotechnical Properties	Atterberg Limits		Natural Moisture Content (%)	Unconfined Compressive Strength	
	LL	PI		Strength (psf)	@ Strain (%)
Average (%)	29	14	21	6300	15.5
Maximum (%)	33	18	39	7400	16.1
Minimum (%)	26	11	15	5200	14.9
Number of Samples	5	5	27	2	

Table 5.3 Classification of DM samples from Waukegan Harbor

Soil Classification	Number of Samples	Percent of Samples (%)
Gravel	0	0
Sand	2	22
Silt	6	67
Clay	1	11
Total	9	100

Table 5.4 Geotechnical Results of DM Samples in Waukegan Harbor

Geotechnical Properties	Atterberg Limits		Moisture Content (%)	Specific Gravity	Organic Content (%)	Standard Compaction		Direct Shear	
	LL (%)	PI (%)				Opt. Water Content (%)	Max. Dry Density (pcf)	Cohesion (psf)	Friction Angle (deg.)
Average	33.6	9.3	68	2.5	3.0	15	103.2	143	34.6
Maximum	49.8	17.6	121	2.7	7.9	15.6	106.4	200	35
Minimum	24.5	3.8	28.7	2.3	0.4	14.1	99.6	100	34.1
Number of Samples	7		9		44	3			

Table 5.5 Classification of DM Samples from Indiana Harbor

Soil Classification	Number of Samples	Percent of Samples (%)
Gravel	0	0
Sand	8	38
Silt	0	0
Clay	9	43
Organic fines	4	19
Total	21	100

Table 5.6 Geotechnical Results of DM Samples in Indiana Harbor

Geotechnical Properties	Atterberg Limits		Moisture Content (%)	Specific Gravity	Compaction		Hydraulic Conductivity (cm/sec)	Consolidated-Undrained (CU)				Unconsolidated-Undrained (UU)	
	LL (%)	PI (%)			Opt. Water Content (%)	Max. Dry Density (pcf)		Total Cohesion (psf)	Total Friction Angle (deg.)	Effective Cohesion (psf)	Effective Angle (deg.)	Cohesion (psf)	Friction Angle (deg.)
Average	42	19.3	32.5	2.70	18.8	103.0	2.06E-07	104.9	25.7	63.7	36.5	1036.7	14.8
Maximum	48	24	42.6	2.71	19.3	108.7	4.82E-07	147.4	29.4	111	36.5	1124	23.7
Minimum	36	17	17.9	2.69	18	99	6.14E-08	24.2	20.9	15.2	36.4	968	0
Number of Samples	3												

Table 5.7 Classification of DM Samples from Calumet Harbor

Soil Classification	Number of Samples	Percent of Samples (%)
Gravel	0	0
Sand	30	26
Silt	56	49
Clay	29	25
Total	115	100

Table 5.8 Geotechnical Results of DM Samples in Calumet Harbor

Geotechnical Properties	Atterberg Limits		w (%)	G_s	Dry Density (psf)	n (%)	Consolidated-Undrained (CU)				Unconsolidated-Undrained (UU)	
	LL (%)	PI (%)					Total Cohesion (psf)	Total Friction Angle (deg.)	Effective Cohesion (psf)	Effective Angle (deg.)	Cohesion (psf)	Friction Angle (deg.)
Average	43.8	16	32.5	2.70	18.8	103	380	21.1	140	30.8	100	3.4
Maximum	47	17	42.6	2.71	19.3	108.7	720	36.5	250	33.9	130	6.7
Minimum	40	15	17.9	2.69	18	99	40	5.6	30	27.6	70	0
Number of Samples	4		12				2					

Table 5.9 Triaxial Compression Results for Soil Samples from Chicago Area CDF

Soil Samples	Consolidated-Undrained (CU)				Unconsolidated-Undrained (UU)	
	Total Cohesion (psf)	Total Friction Angle (deg.)	Effective Cohesion (psf)	Effective Friction Angle (deg.)	Cohesion (psf)	Friction Angle (deg.)
G1	720	5.6	250	27.6	70	6.7
G2	40	36.5	30	33.9	130	0

Table 6.1: Relevant Properties and Testing Standards for Three Transportation Earthwork Applications

Transportation Application		Geotechnical Properties	Testing Standards
Base	Open Graded Base & Dense Graded base	Gradation	AASHTO T27
		Wear	AASHTO T96
		Sodium sulfate soundness	AASHTO T104
		Freeze/thaw soundness	AASHTO T103
		Liquid limit	AASHTO T89
		Plasticity index	AASHTO T90
		Fracture	CMM 8-60
Subbase		Percent passing the 200 sieve	AASHTO T11
		Gradation	AASHTO T27
		Liquid limit	AASHTO T89
		Plasticity index	AASHTO T90
Backfill	Structural Backfill	Percent passing the 200 sieve	AASHTO T2
		Gradation	AASHTO T11
	Granular Backfill	percent passing the 200 sieve	AASHTO T11
		Gradation	AASHTO T27
		Liquid limit	AASHTO T89
		Plasticity index	AASHTO T90
Embankment/Borrow		No gradation requirements except highly frost, swelling, and compression susceptible or highly organic soils, such as CH, OH, and MH.	

Table 6.2 Required Geotechnical Properties and Suitability for Several Applications

Soil Classification		Rating and Magnitude of Soil Engineering Properties												
USCS Divisions (1)	Symbols (2)	Optimum Water Content (%) (3)		Max. Dry Unit weight (pcf) (4)		Cohesion (psf) (5)	Friction Angle (deg.) (6)	Hydraulic Conductivity (cm/s) (7)	Drainage Characteristics (8)	CBR (9)	Compressibility and Expansion (10)	Potential Frost Action (11)	Compaction Characteristics (12)
Gravel and Gravelly Soil	GW	8-11[a]	11.4[b]	125-135[a]	124.2[b]	0	33-41	$>10^{-2}$	good (pervious)[c]	40-80	almost none	none to very slight[d]	good
	GP	11-14	11.2	115-125	121.7	0	35-41	$>10^{-2}$	good (pervious)	30-60	almost none	none to very slight	good
	GM	8-12	15.8	120-135	113.3	0	32-38	10^{-3} - 10^{-6}	poor (semi pervious)	20-60	slight	slight to medium	good
	GC	9-14	13.9	115-130	116.6	0	29-33	10^{-6} - 10^{-8}	poor (impervious)	20-40	slight	slight to medium	good
Sand and Sandy Soil	SW	9-16	9.1	110-130	126.1	0	35-41	$> 10^{-3}$	good (pervious)	20-40	almost none	none to very slight	good
	SP	12-21	10.8	100-120	115.6	0	31-39	$> 10^{-4}$	good (pervious)	10-40	almost none	none to very slight	good
	SM	11-16	12.5	110-125	116.6	0	33-35	10^{-3} - 10^{-6}	poor (impervious)	10-40	slight	slight to high	good
	SC	11-19	12.4	105-125	118.9	0	30-36	10^{-6} - 10^{-8}	poor (impervious)	5-20	slight to medium	slight to high	fair to good
Silt and Clay (LL<50)	ML	12-24	19.7	95-120	103.3	0	29-37	10^{-3} - 10^{-6}	poor (impervious)	<= 15	slight to medium	medium to very high	poor to good
	CL	12-24	16.7	95-120	109.3	210-625	26-32	10^{-6} - 10^{-8}	no drainage (impervious)	<= 15	medium	medium to high	fair to good
	OL	21-33	NA	80-100	NA	105-315	22-32	10^{-4} - 10^{-6}	poor (impervious)	<= 5	medium to high	medium to high	poor to fair
Silt and Clay (LL>50)	MH	24-40	33.6	70-95	85.1	0-210	24-30	10^{-4} - 10^{-6}	poor (impervious)	<= 10	high	medium to very high	poor to fair
	CH	19-36	25	80-105	95.3	315-730	17-27	10^{-6} - 10^{-8}	no drainage (impervious)	<= 15	very high	medium	poor to fair
	OH	21-45	NA	65-100	NA	105-315	17-35	10^{-6} - 10^{-8}	no drainage (impervious)	<= 5	high	medium	poor to fair

Continued

Soil Classification		Soil Value as Transportation Sectors				
USCS (1)	Symbols (2)	Embankment (13)	Subgrade (14)	Subbase (15)	Base (16)	Backfill in MSE Wall (17)
Gravel and Gravelly Soil	GW	excellent	excellent	excellent	good	good to excellent
	GP	fair to good	excellent to good	good	good to fair	excellent
	GM	fair to good	excellent to good	good to fair	good to unsuitable[2]	good to fair
	GC	fair to good	good	fair	poor to unsuitable	fair
Sand and Sandy Soil	SW	excellent	good	good to fair	poor	good
	SP	fair to good	good to fair	fair	poor to unsuitable	good
	SM	fair to good	good to fair	good to poor[1]	poor to unsuitable	fair
	SC	fair to good	good to fair	poor	unsuitable	poor
Silt and Clay (LL<50)	ML	poor	fair to poor	unsuitable	unsuitable	very poor to unsuitable
	CL	good	fair to poor	unsuitable	unsuitable	unsuitable
	OL	unsuitable	poor	unsuitable	unsuitable	unsuitable
Silt and Clay (LL>50)	MH	unsuitable	poor	unsuitable	unsuitable	unsuitable
	CH	fair	poor	unsuitable	unsuitable	unsuitable
	OH	unsuitable	poor to very poor	unsuitable	unsuitable	unsuitable

Note:

1, If LL<25 and PI, SM' value as subbase ranged from fair to good. Otherwise, SM's value as subbase ranged from poor to fair.

2, If LL<25 and PI, GM's value as base ranged from fair to good. Otherwise, GM's value as subbase ranged from poor to unsuitable.

a, geotechdata.info

b, Average values of compacted soils from Western United States (USBR)

c, According USBR, k less than 1 ft/year as impervious (no drainage), k between 1 and 100 ft./year as semipervious (poor); k greater than 100 ft./year as pervious (good)

d, American Concrete Pavement Association (ACPA)

Table 7.1 Geotechnical properties of the RDM in Milwaukee Harbor CDF

Item	Properties
Specimen Name	RDM
USCS	MH
AASHTO	A-7-5
w_N	67.3
Organic Content (%)	9.8
G_s	2.59
Gravel (%)	0
Sand (%)	3.4
Fines (%)	96.6
LL	61.5
PI	19.3
γ_d (kN/m^3)	12.9
w_{OPT} (%)	30
CBR	1.5
c_u (kPa)	240
UCS (kPa)	27.7

Note: w_N = in situ water content; G_s = specific gravity; Fines = percentage passing No. 200 sieve; LL = liquid limit; PI = plasticity index; γ_d = maximum dry unit weight; w_{OPT} = optimum water content (*ASTM D698*); CBR = California bearing ratio (performed with optimum water content); c_u = undrained shear strength (performed with 100kPa confining pressure); UCS = unconfined compressive strength.

Table 7.2 Chemical ingredients of Class C fly ash tested (provided by the manufacturer)

Chemical	Content (%)
SiO_2 (amorphous silica)	20 - 60
SiO_2 (crystalline silica)	0 - 10
Fe_2O_3	4 -33
Al_2O_3	10 -33
CaO	1 - 30
MgO	0 - 4
TiO_2	0 - 3
Na_2O	0 - 10
K_2O	0 - 3
Carbon	0 - 50
Trace Metals	< 0.1

Table 7.3 Contents of RDM and fly ash in specimens

Specimen	RDM conten (%)	Fly ash content (%)
RDM	100	0
FA10D	90	10
FA20D	80	20
FA30D	70	30

Table 7.4 Summary of testing programs

Testing Program	Standards	Numbers of Samples									
		Curing Time: 2 hours				Curing Time: 7 days			Curing Time: 28 days		
		RDM	FA10D	FA20D	FA30D	FA10D	FA20D	FA30D	FA10D	FA20D	FA30D
Gradation	ASTM D1140 and D422	1									
LL	ASTM D4318	1	1	1	1	1	1	1	1	1	1
PL		1	1	1	1	1	1	1	1	1	1
Specific Gravity	ASTM D854	2									
Water Content	ASTM D2216	2									
Organic Content	ASTM D2974	2									
Proctor Test	ASTM D698	1	1	1	1						
Triaxial Test (UU)	ASTM D2850	3	3	3	3	3	3	3	3	3	3
Resilient Modulus	AASHTO T307	1	1	1	1	1	1	1	1	1	1
CBR	ASTM D1883	1	1	1	1	1	1	1	1	1	1
Durability Test (F-T)	ASTM D560	3	3	3	3						
UC Test	ASTM D2166	3	3	3	3						

Table 7.5 Atterberg limits for untreated and treated soil

Soil Type	Cementitious material content (by weight) (%)	Liquid Limit (LL)	Plastic Limit (PL)
CL	0	40.3	21.3
	5	38	20.8
	8	36	20
	10	34	19.4
	15	33	19.2
CH	0	64	39
	5	58	34
	8	52	25
	10	47	22
	15	41	18
MH (DM)	0	104	43
	4 (cured for 1 month)	83.6	40
	4 (cured for 6 months)	56.7	19
	8 (cured for 1 month)	89.4	17
	8 (cured for 6 months)	65.8	16

FIGURES

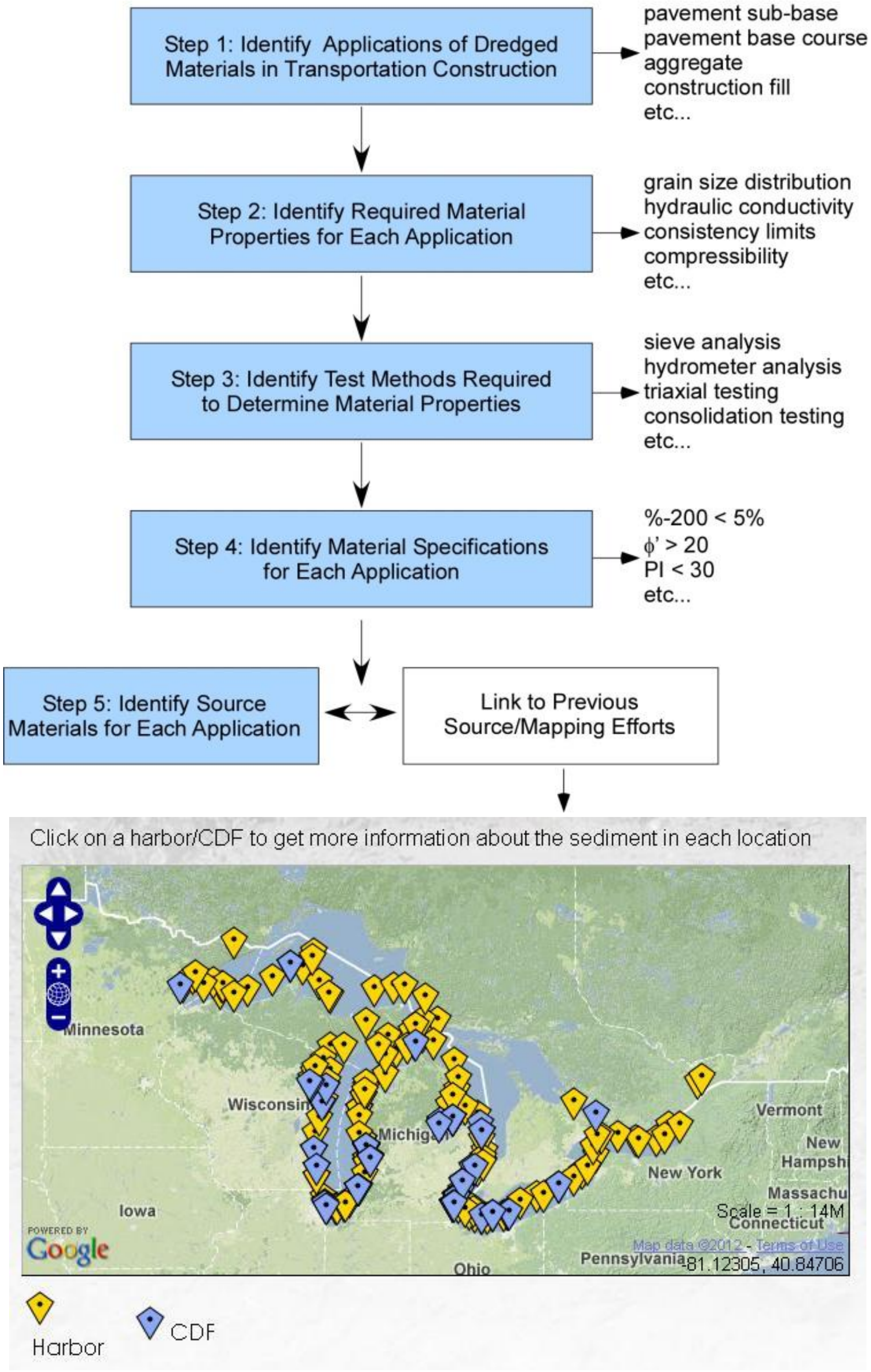


Figure 1.1 Summary of project scope for beneficial use of dredged materials in the Great Lakes region (map from http://www.glc.org/rsm/mapholder.html)

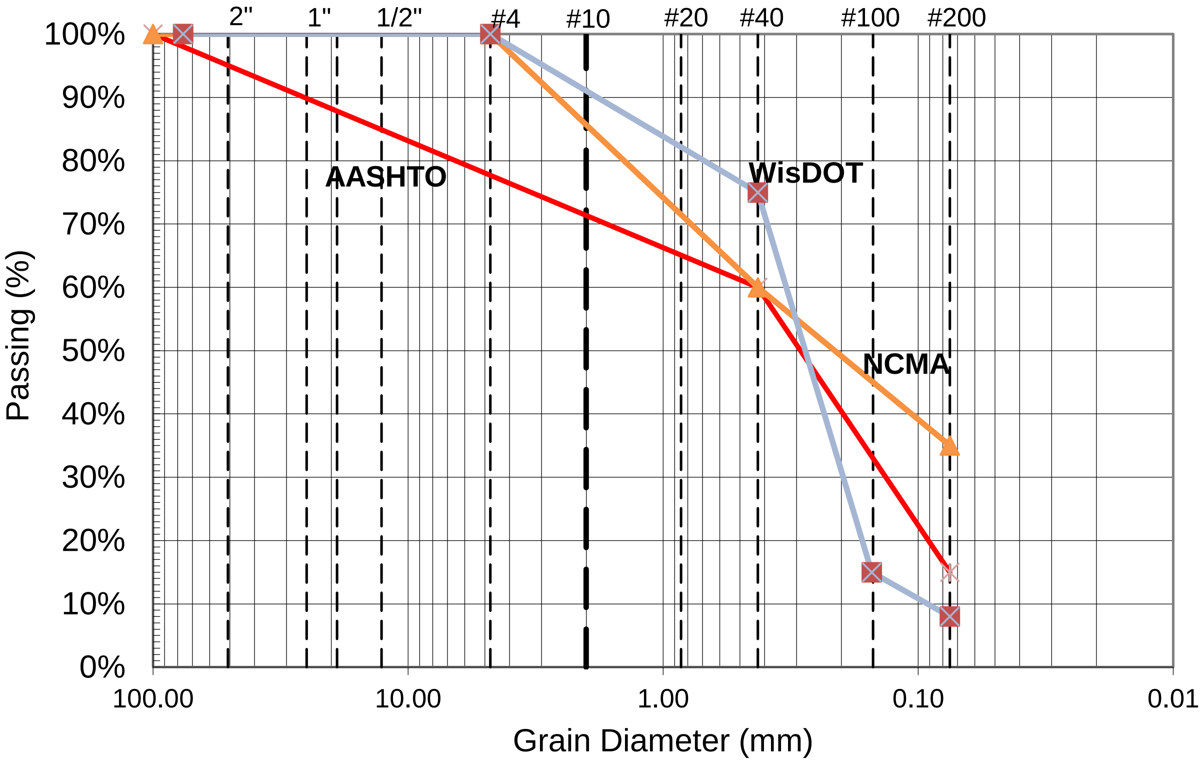


Figure 3.1 Upper Limit of Gradation for Backfill

📍: The location of DM samples collected

Figure 5.1 Project Site of West Arm-Burns Harbor (2003)

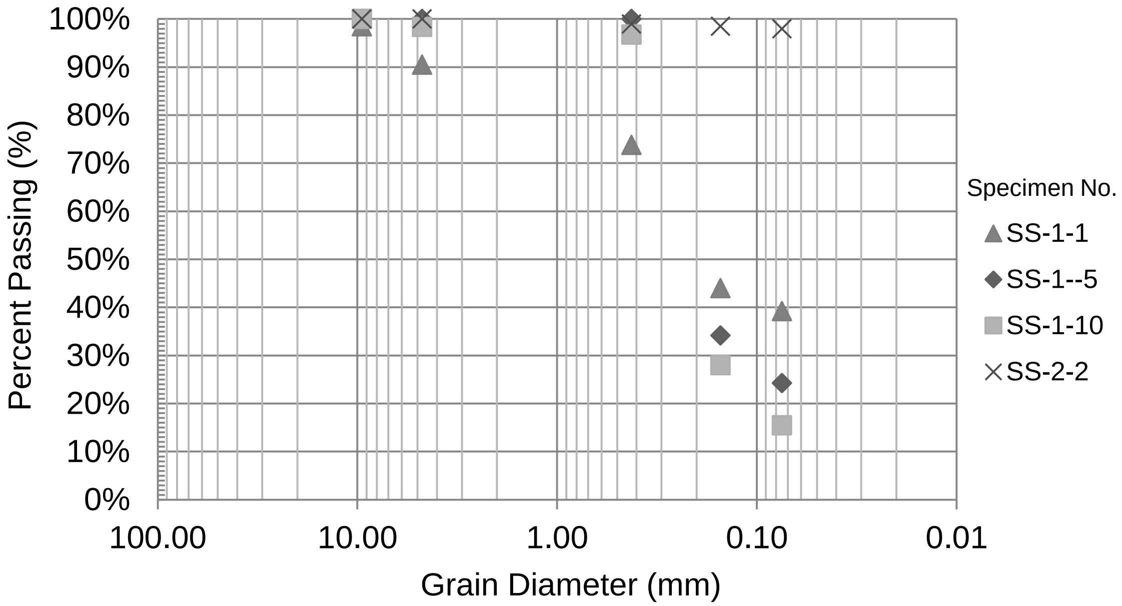


Figure 5.2 Grain Size Distribution of DM Samples in West Arm-Burns Harbor

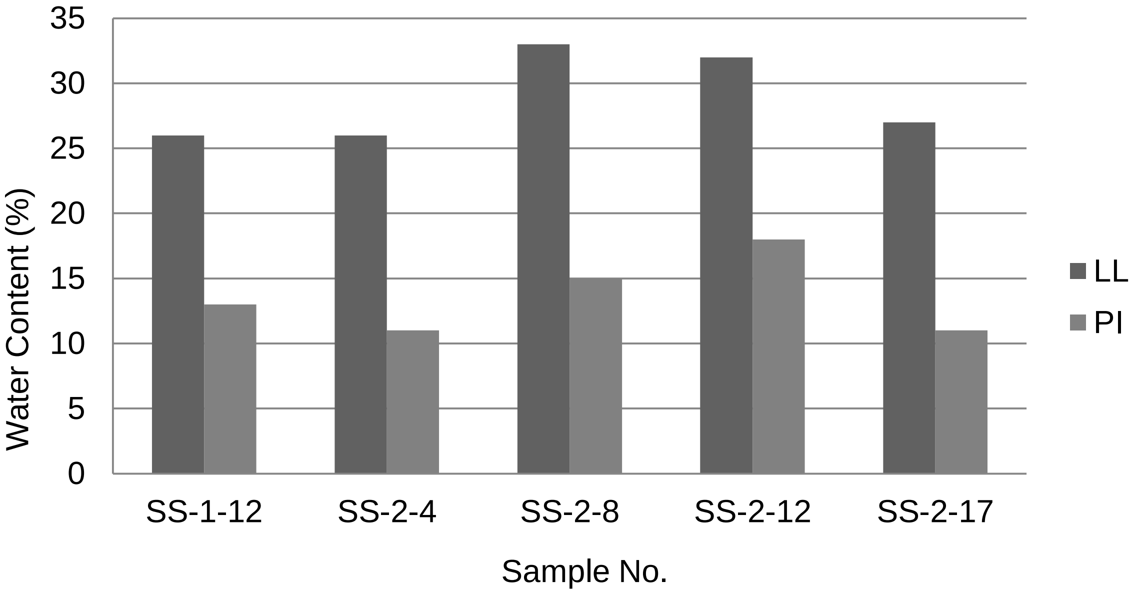


Figure 5.3 Atterberg Limits of DM samples in West Arm-Burns Harbor

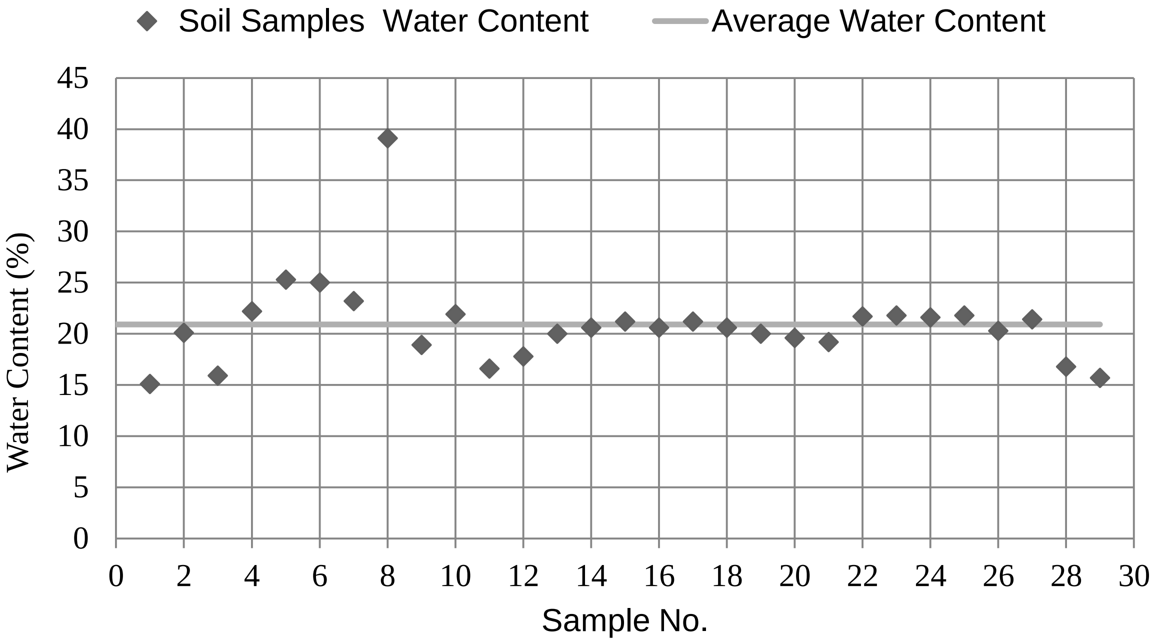
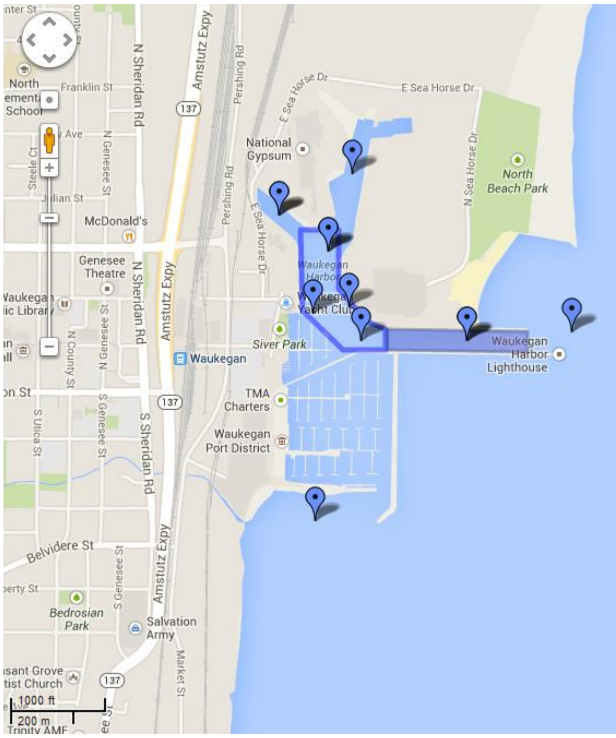


Figure 5.4 Water Content of DM Samples in West Arm-Burns Harbor



📍 : The location of DM samples collected

Figure 5.5 Project Site of Waukegan Harbor

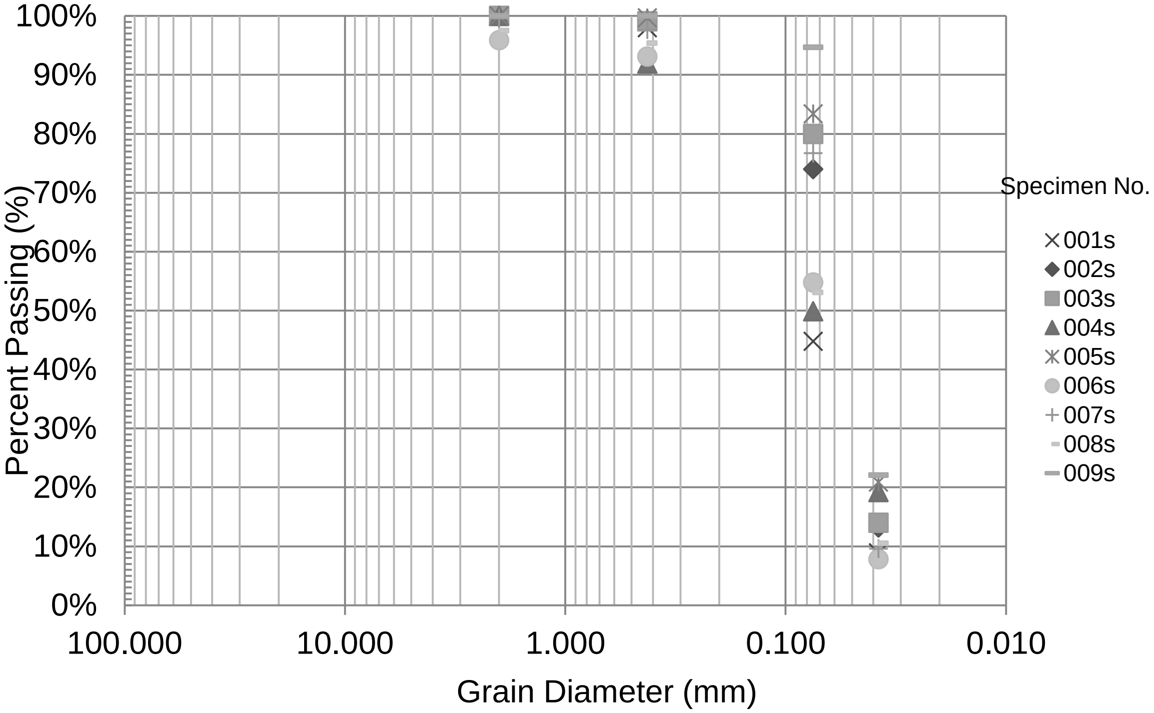


Figure 5.6 Grain Size Distribution of DM Samples in Waukegan Harbor

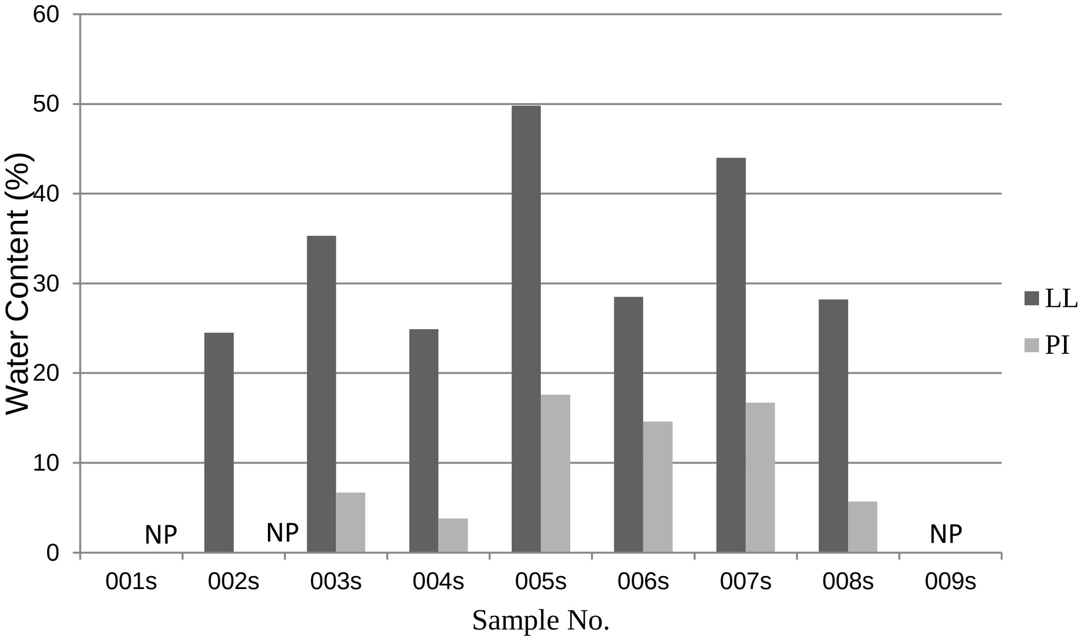


Figure 5.7 Atterberg Limits of DM Samples in Waukegan Inner Harbor

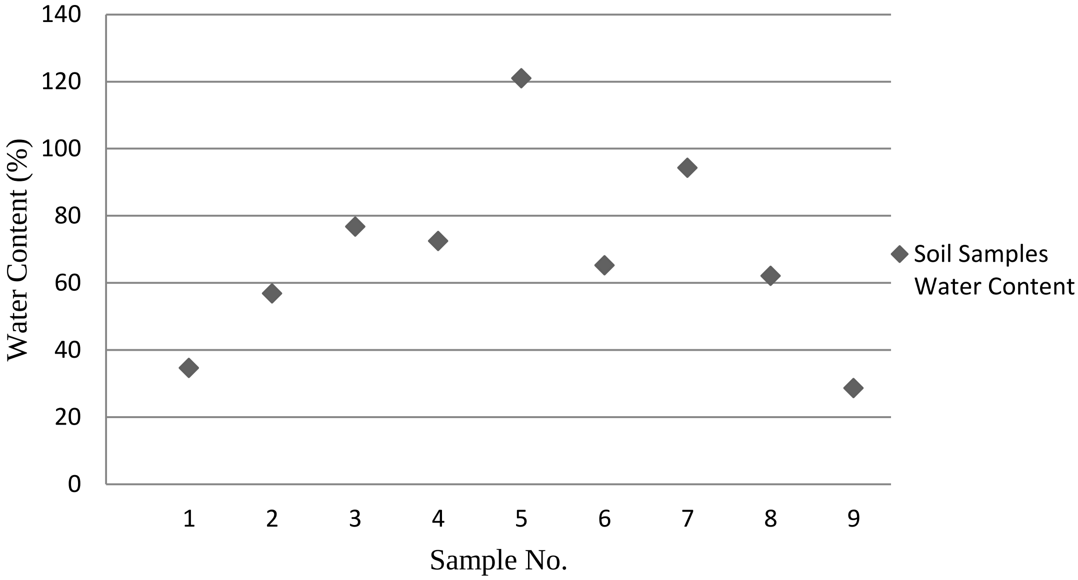


Figure 5.8 Water Content of DM Samples in Waukegan Harbor

📍 : The location of DM samples collected

Figure 5.9 Project Site of Indiana Harbor (2010)

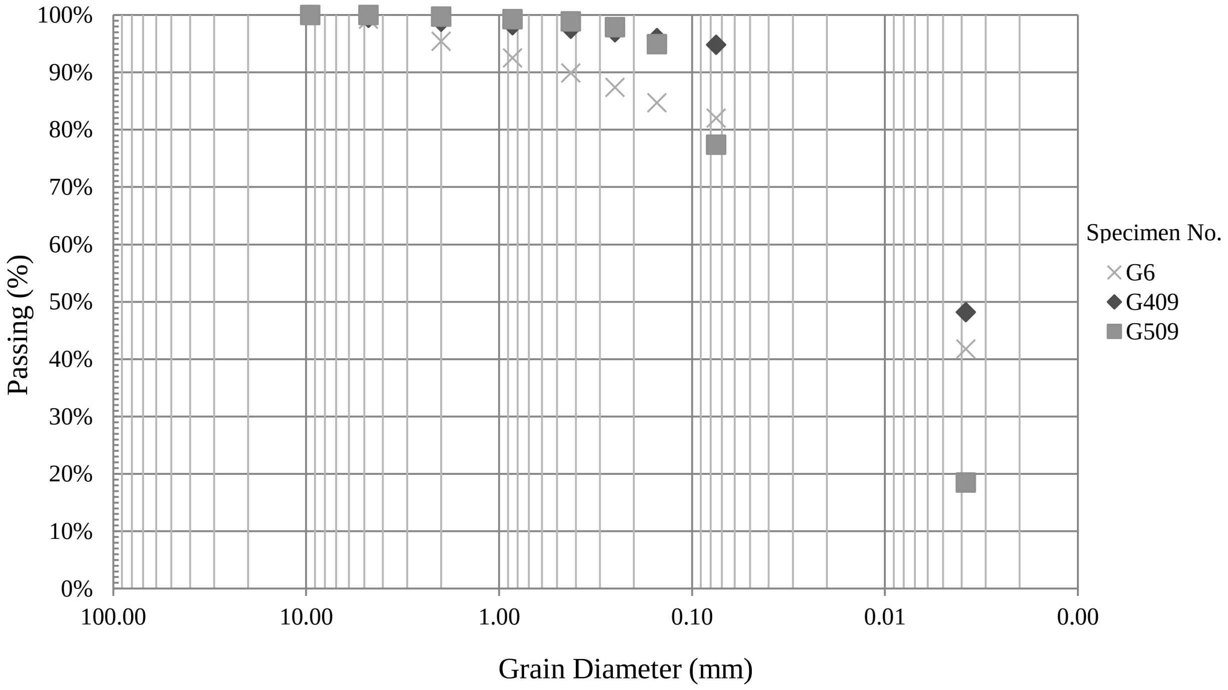


Figure 5.10 Grain Size Distribution of DM Samples in Indiana Harbor

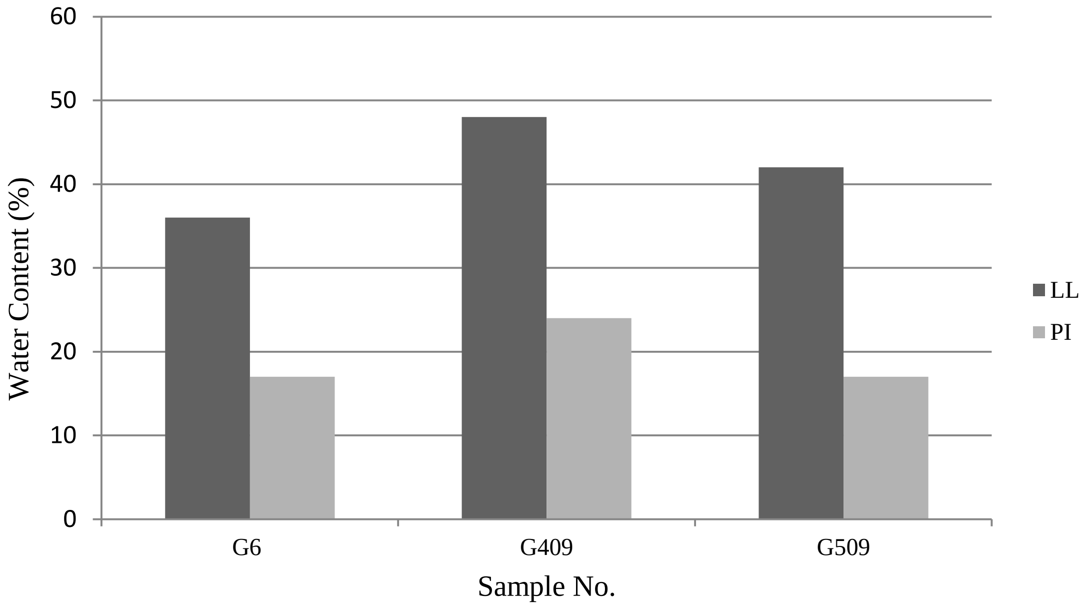
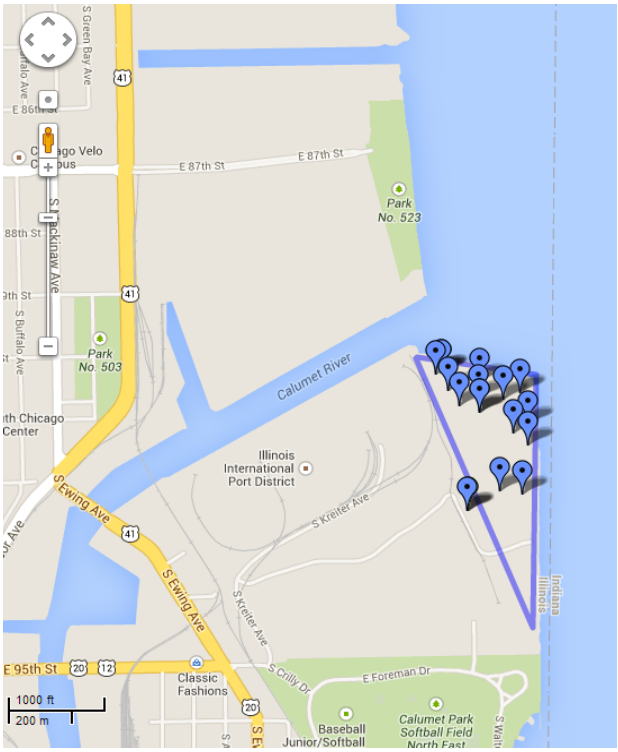


Figure 5.11 Atterberg Limits of DM Samples in Indiana Harbor



📍: The location of DM samples collected

Figure 5.12 Project Site of Calumet Harbor (2006)

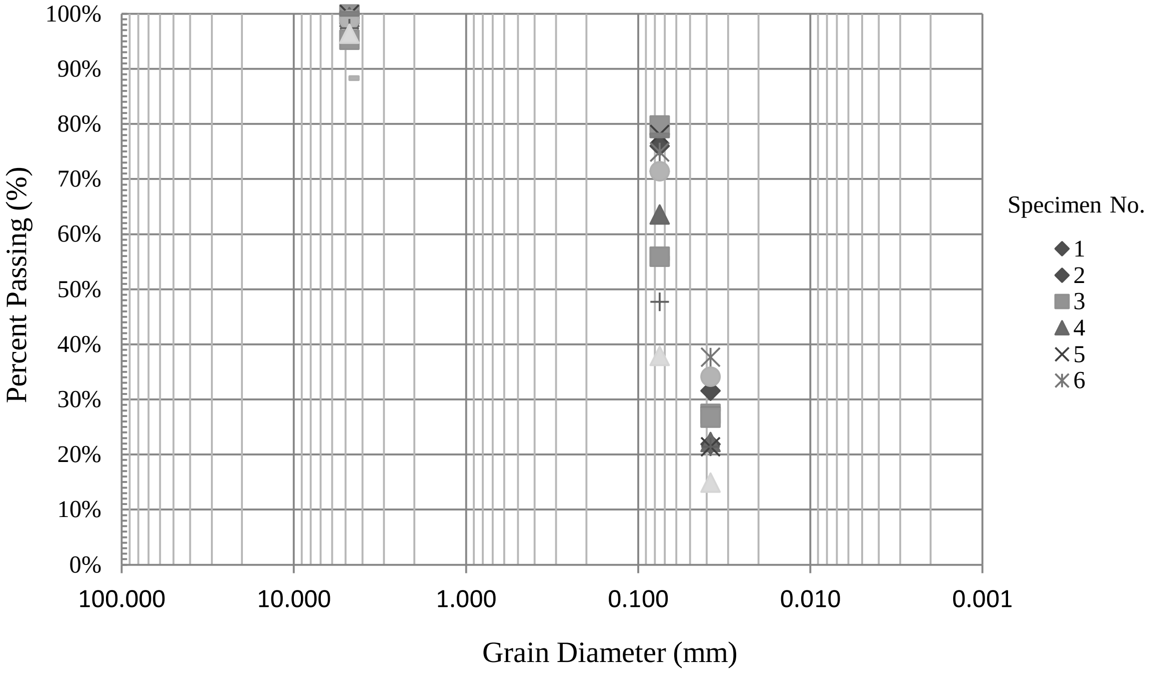


Figure 5.13 Grain Size Distribution of DM Samples in Calumet Harbor (Chicago Area CDF)

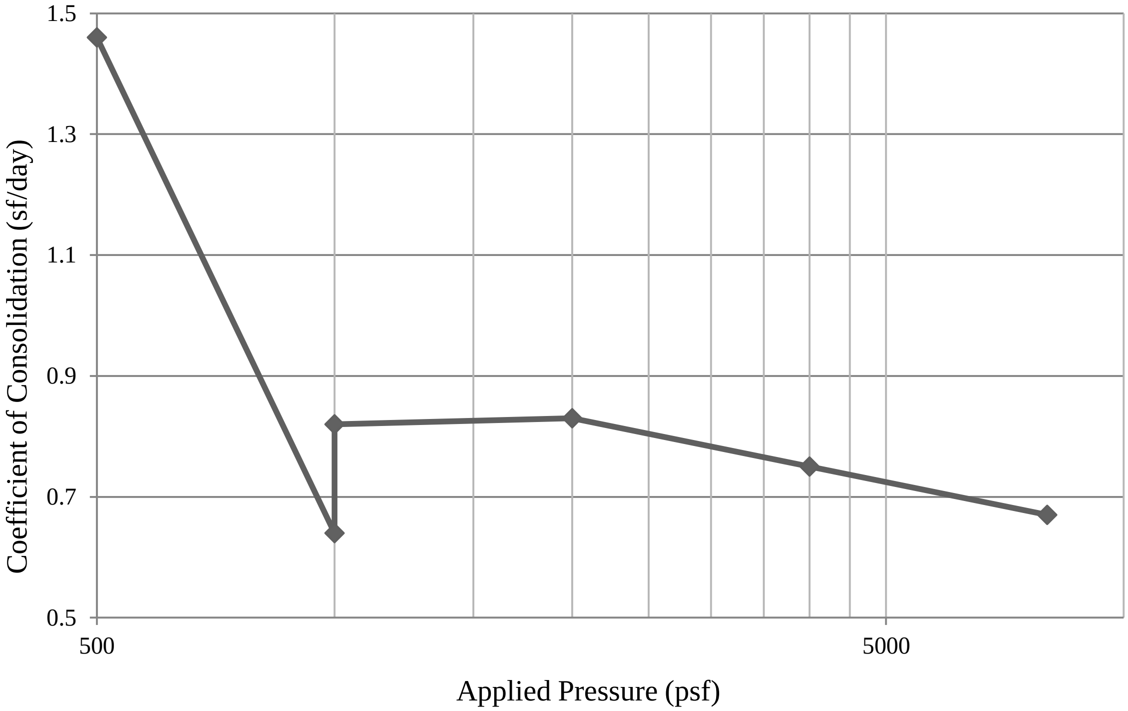
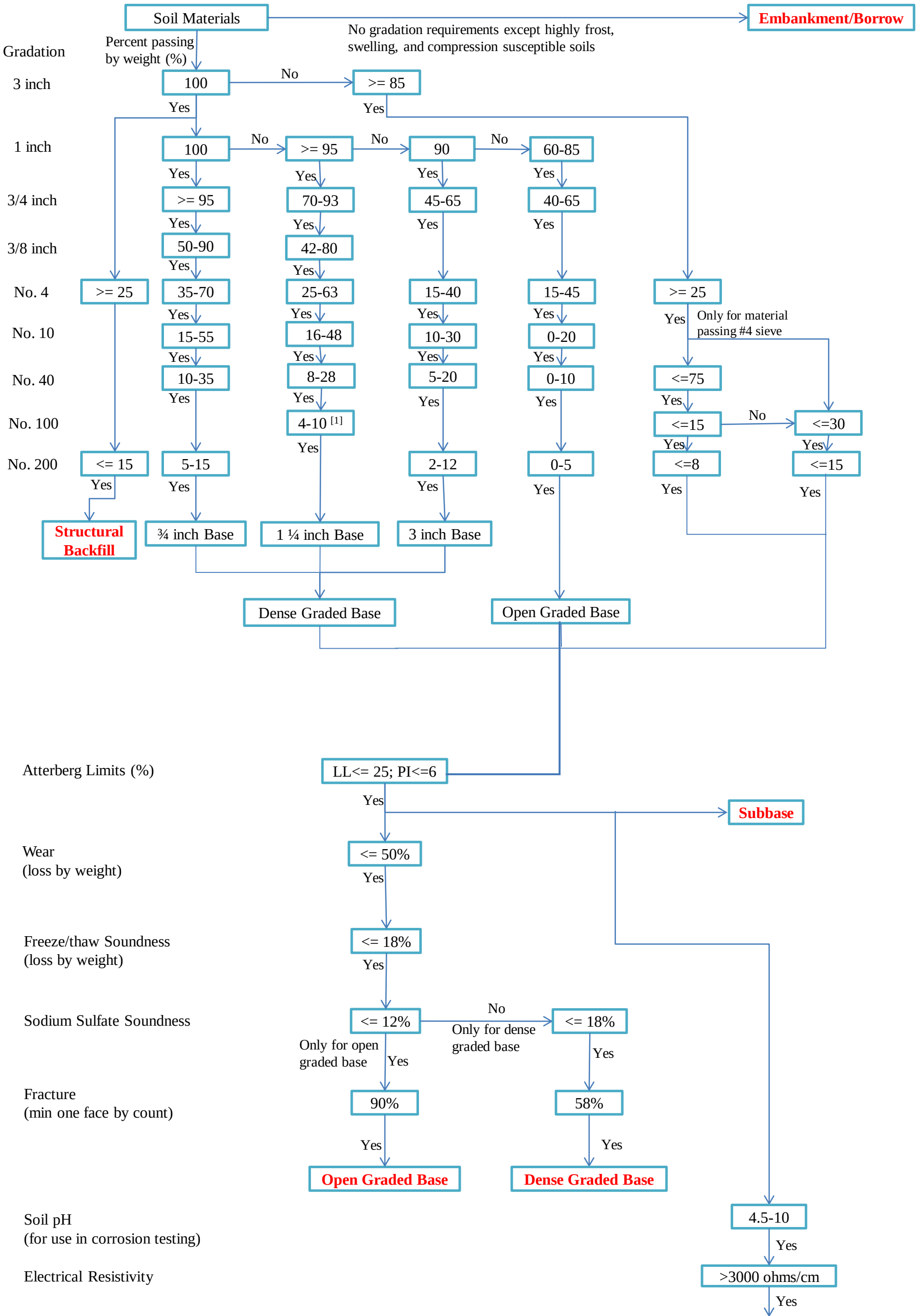
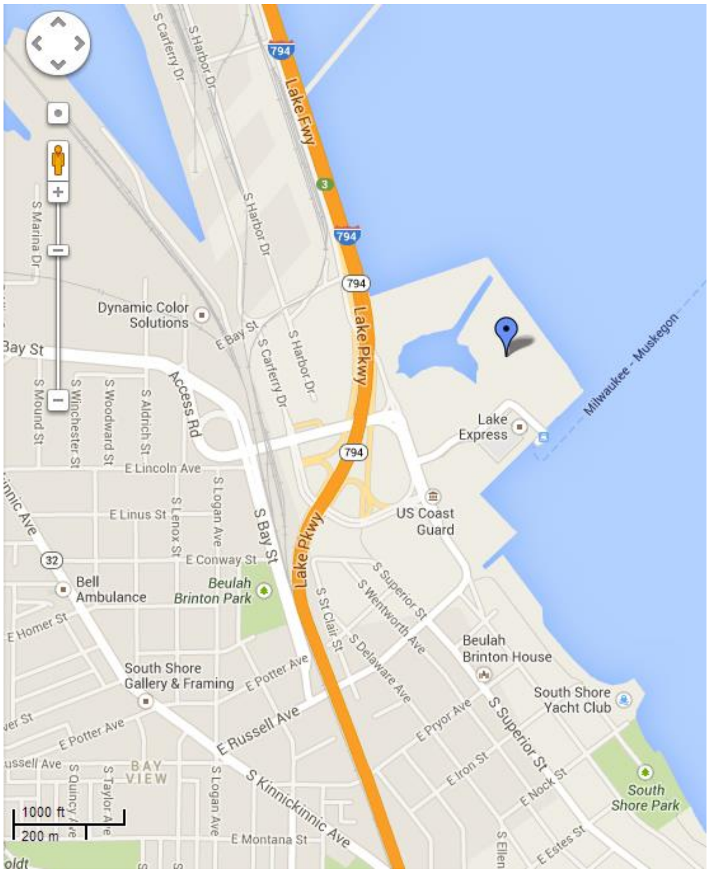


Figure 5.14 Consolidation Characteristics of DM Samples from Chicago CDF



Note: [1] 8-15.0 percent if base is >= 50 percent crushed gravel;
4-10.0 percent if base is >= 50 percent crushed gravel.

Figure 6.1 Framework for evaluation of soil suitability in the transportation sector



📍: The location of DM sample collected

Figure 7.1 Project Site of Milwaukee Port (2012)

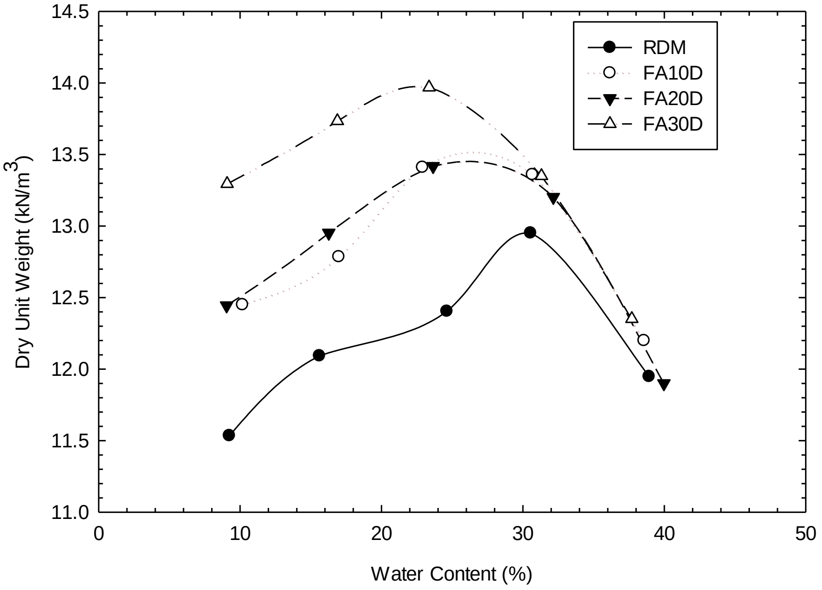


Figure 7.2 (a) Compaction curves of the RDM and SDM specimens without curing

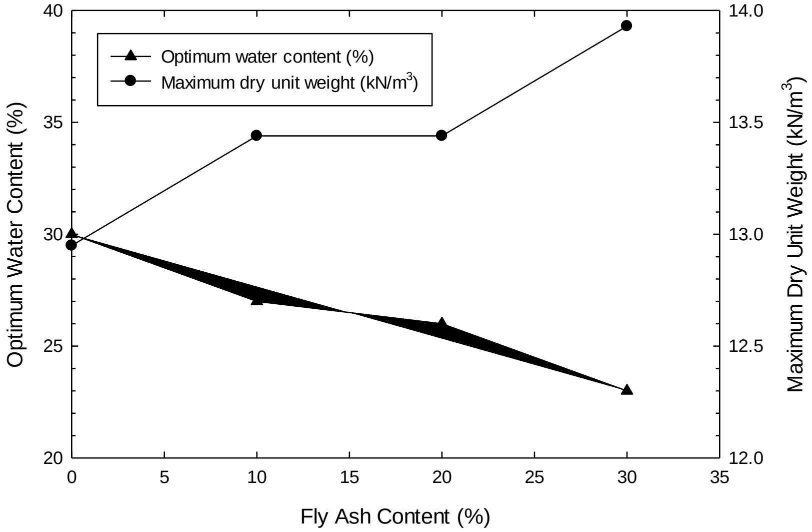


Figure 2 (b). Optimum water content and maximum dry unit weight as function of fly ash content

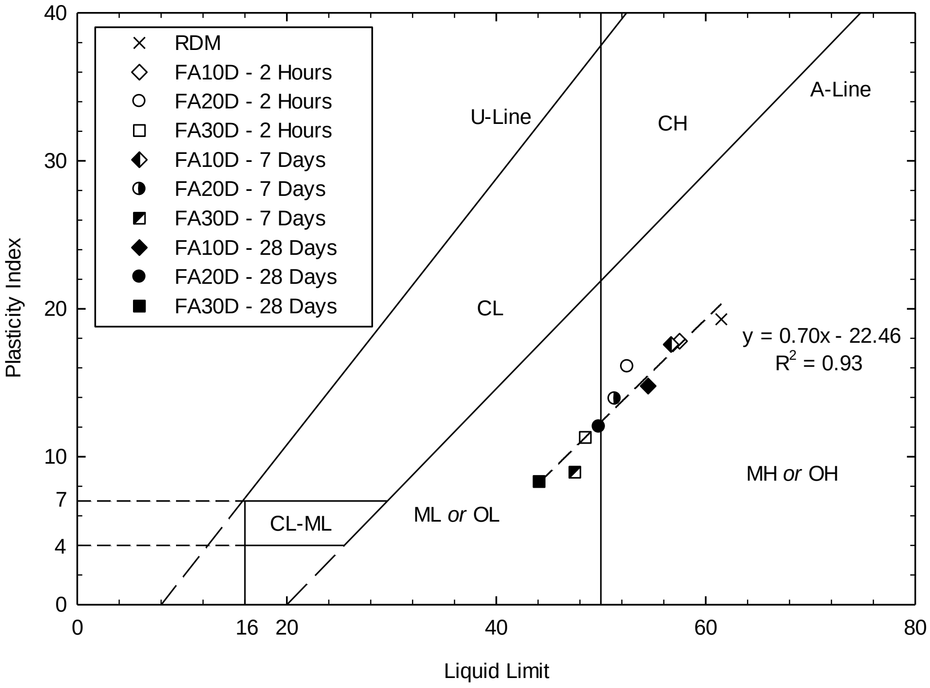


Figure 7.3 Summary of the plasticity chart of RDM and SDM specimens

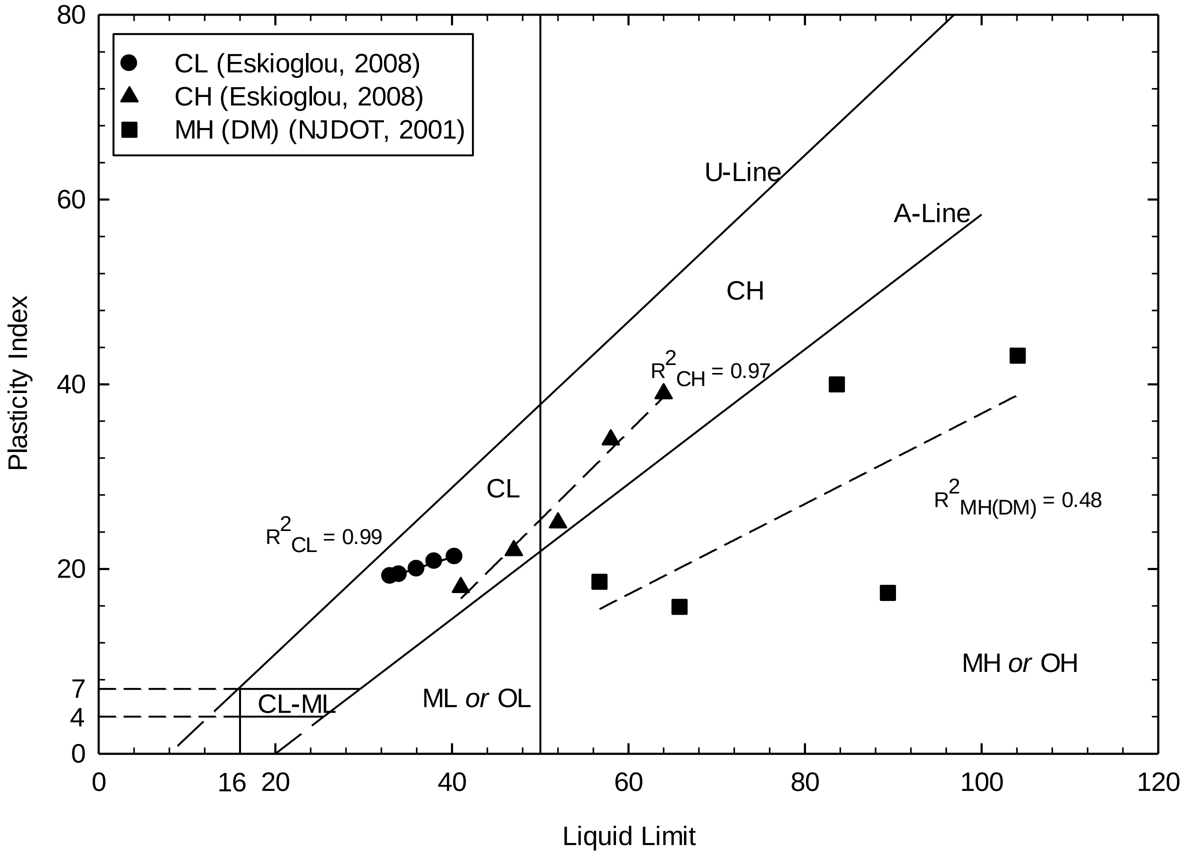
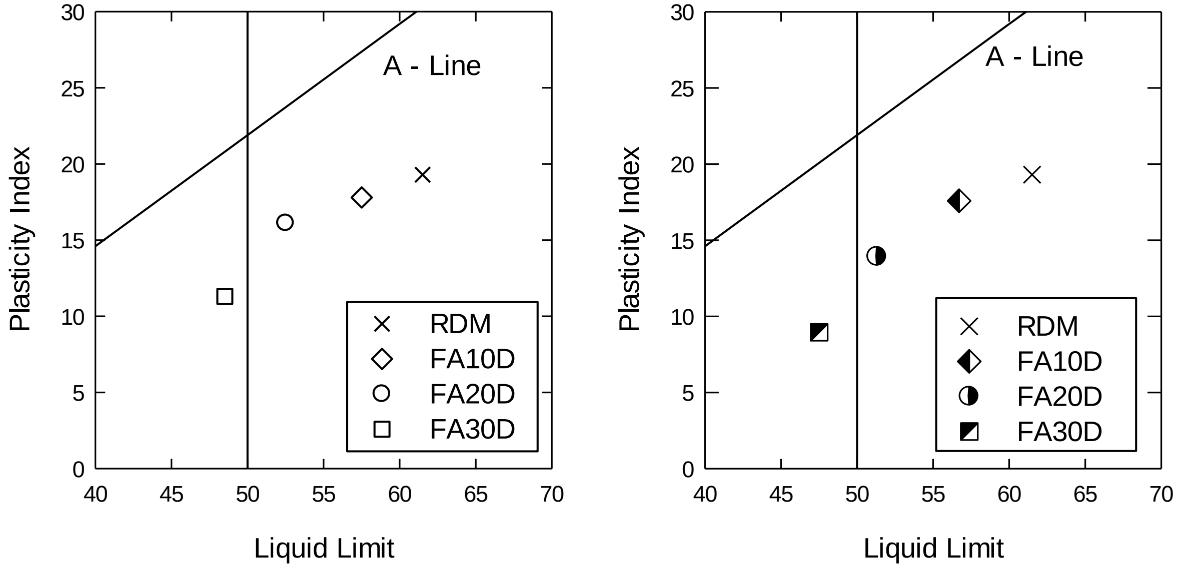


Figure 7.4 Summary of the plasticity chart of stabilized fine-grained soils

(a) Curing time: 2 Hours

(b) Curing time: 7 Days



(c) Curing Time: 28 Days

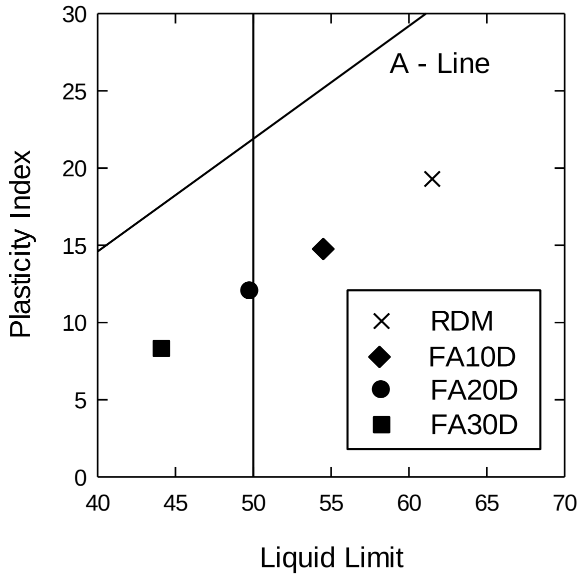


Figure 7.5 Plasticity chart of the RDM and SDM specimens as a function of the curing time

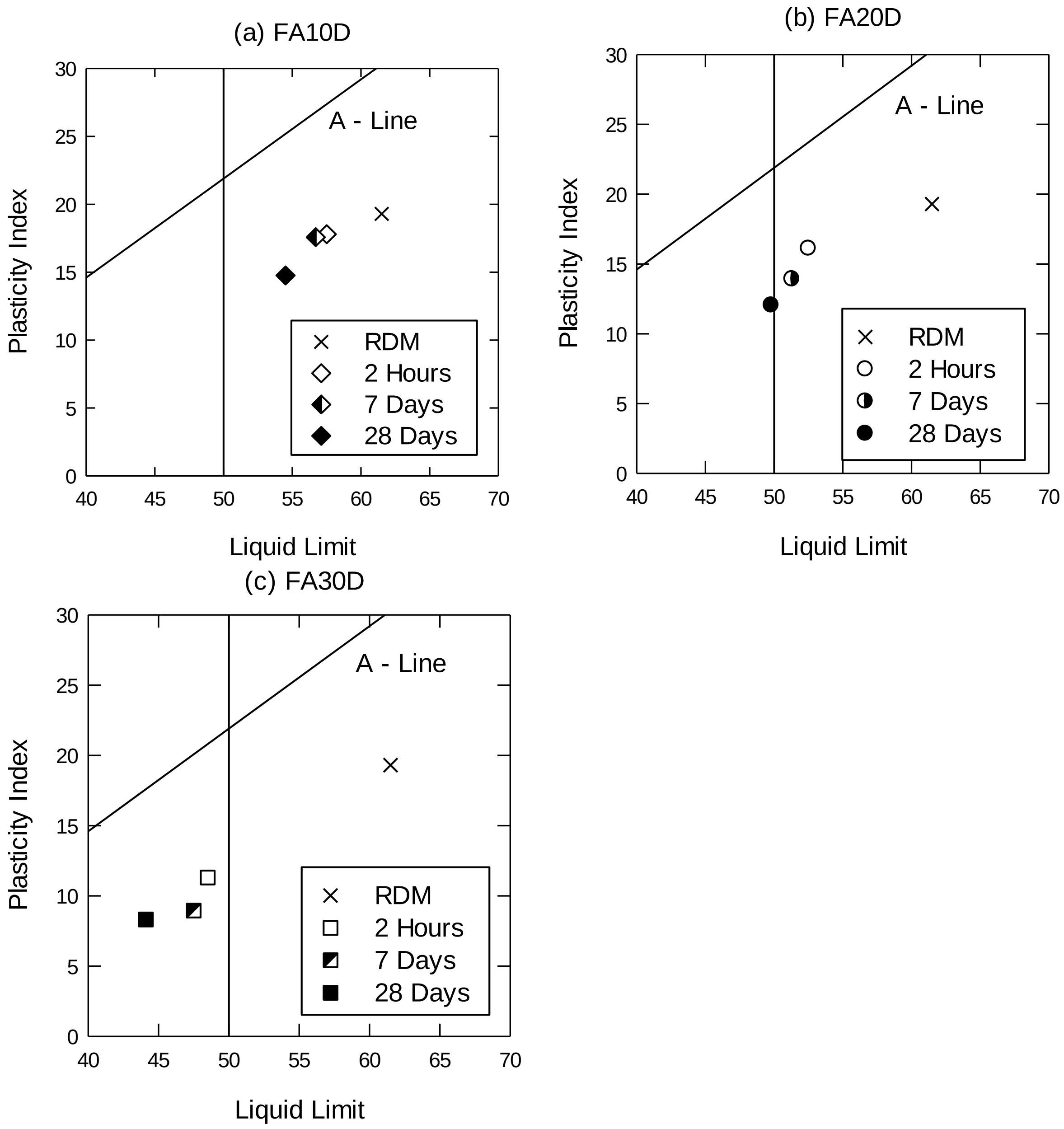


Figure 7.6 Plasticity chart of the RDM and SDM specimens as a function of the fly ash content

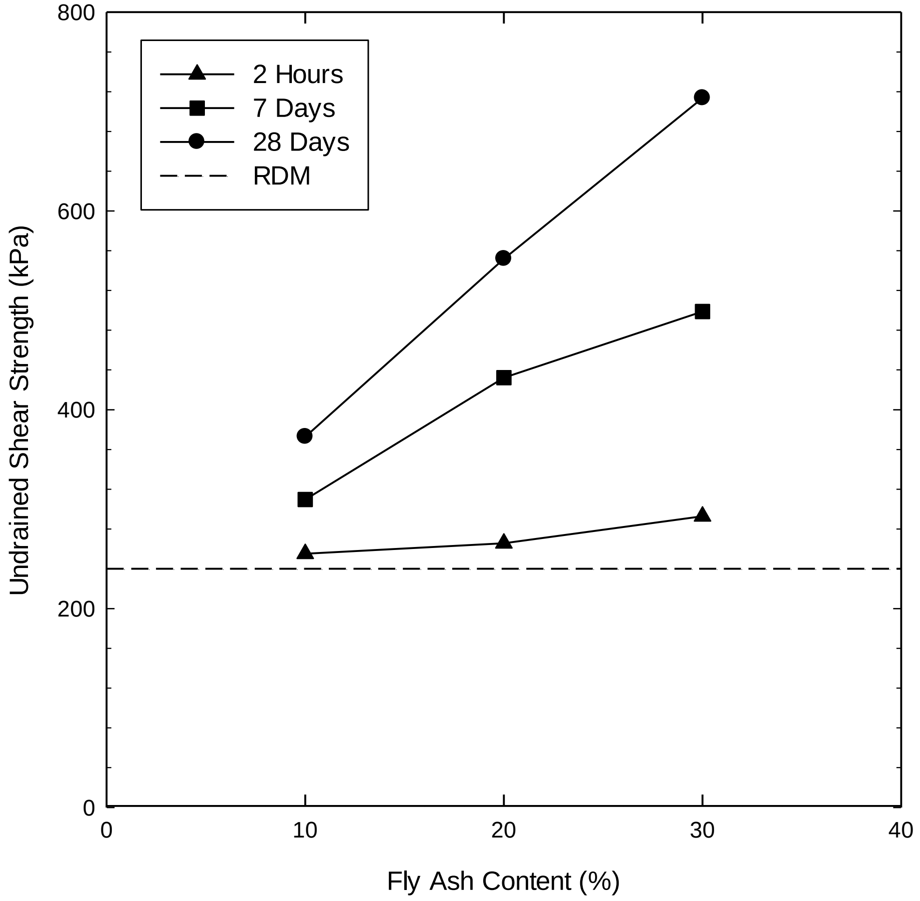


Figure 7.7 Undrained shear strength of the RDM and SDM specimens with different curing time

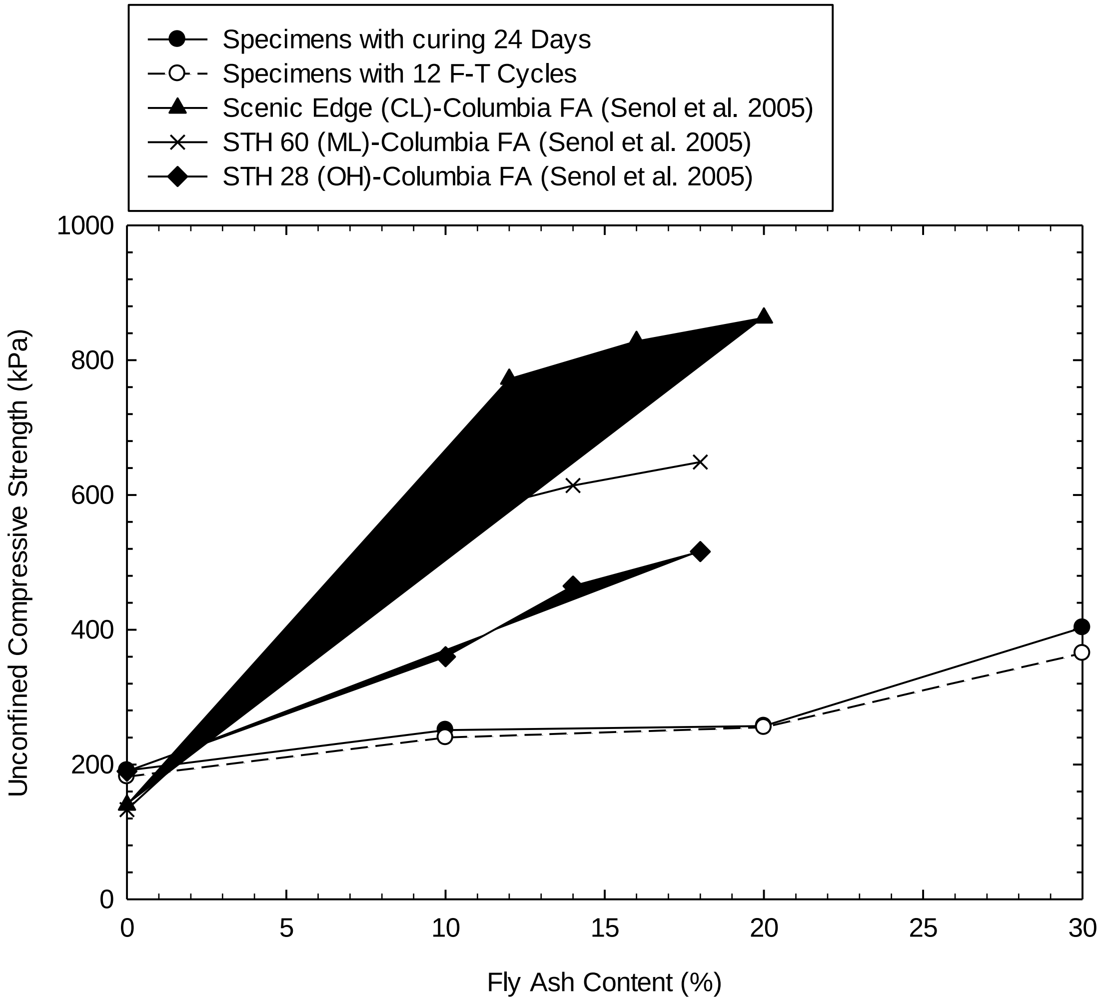


Figure 7.8 Unconfined compressive strength of fine-grained specimens as a function of fly ash percentage

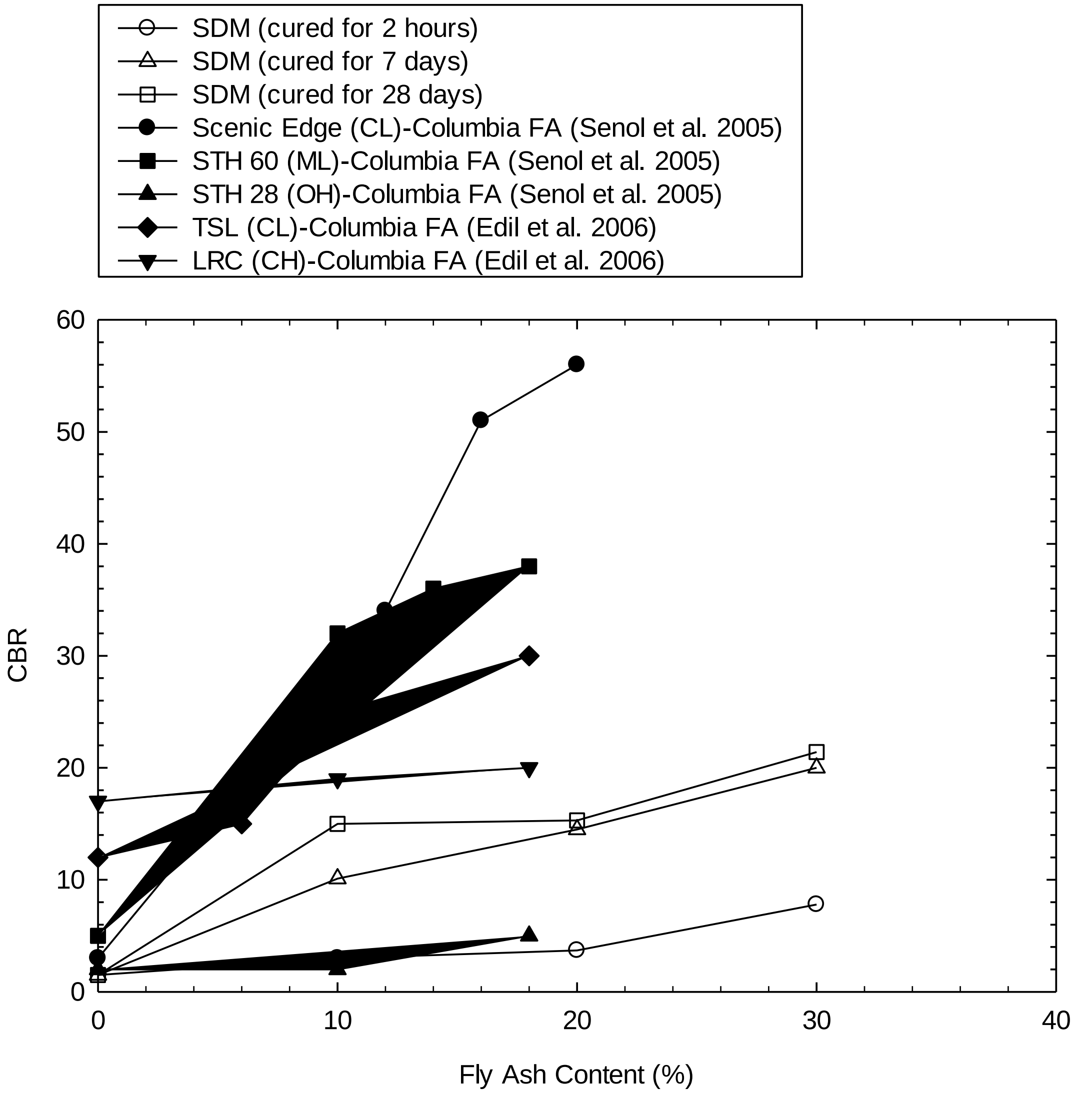


Figure 7.9 The comparison of fly ash with CBR values

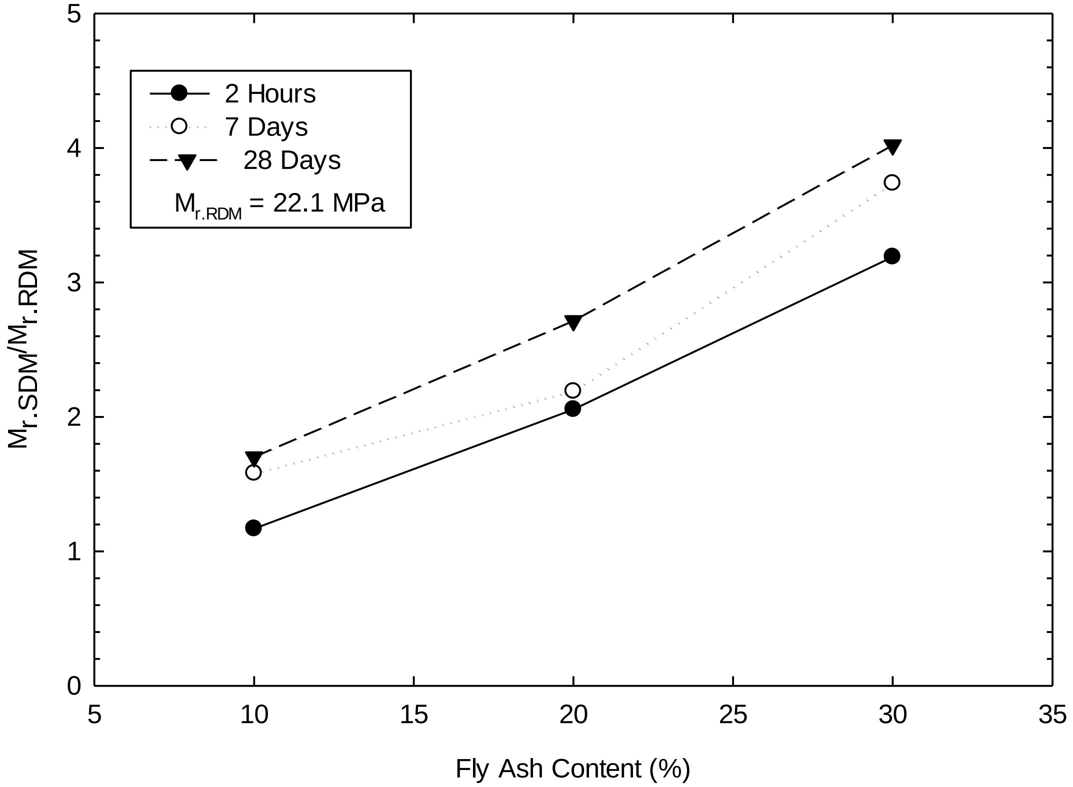


Figure 7.10 Ratio of M_r of SDM specimens cured with 2 hours, 7 days, and 28 days to M_r of RDM specimens. All resilient Moduli are at deviator stress of 21 kPa

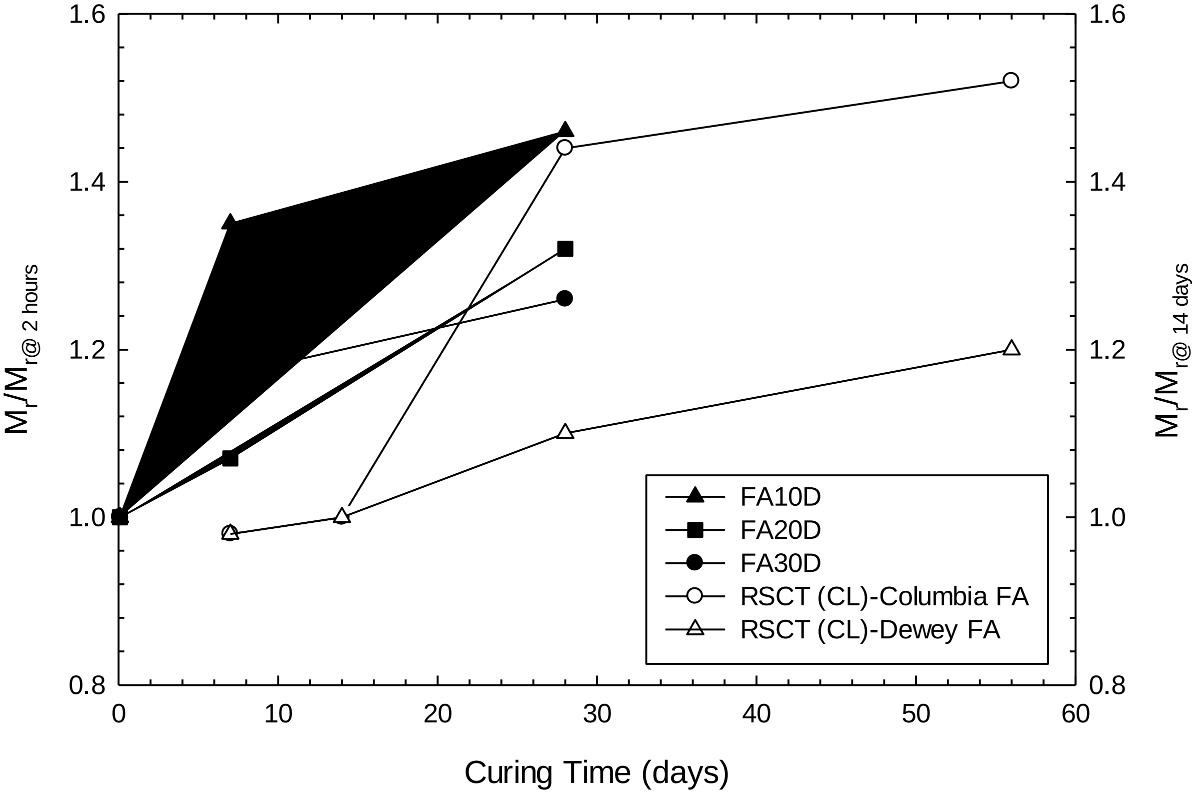


Figure 7.11 Effect of curing time of M_r of soil-fly ash mixtures

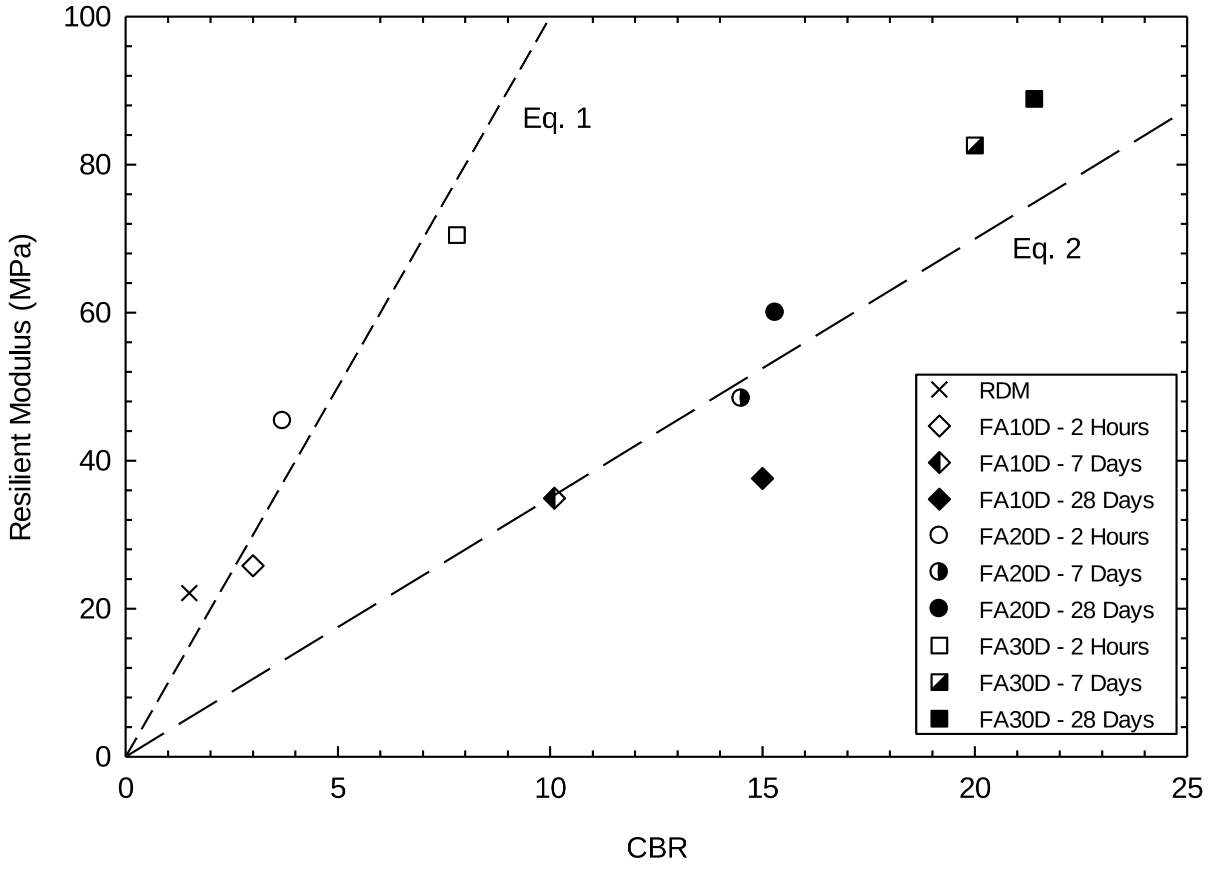


Figure 7.12 Resilient Modulus (at deviator stress = 21 kPa) versus CBR of SDM and RDM along with Eqs. (1) - (2). SDM specimens were cured for 2 hours, 7 days, and 28 days for resilient modulus testing and CBR testing. After curing, specimens soaked 4 days prior to CBR testing

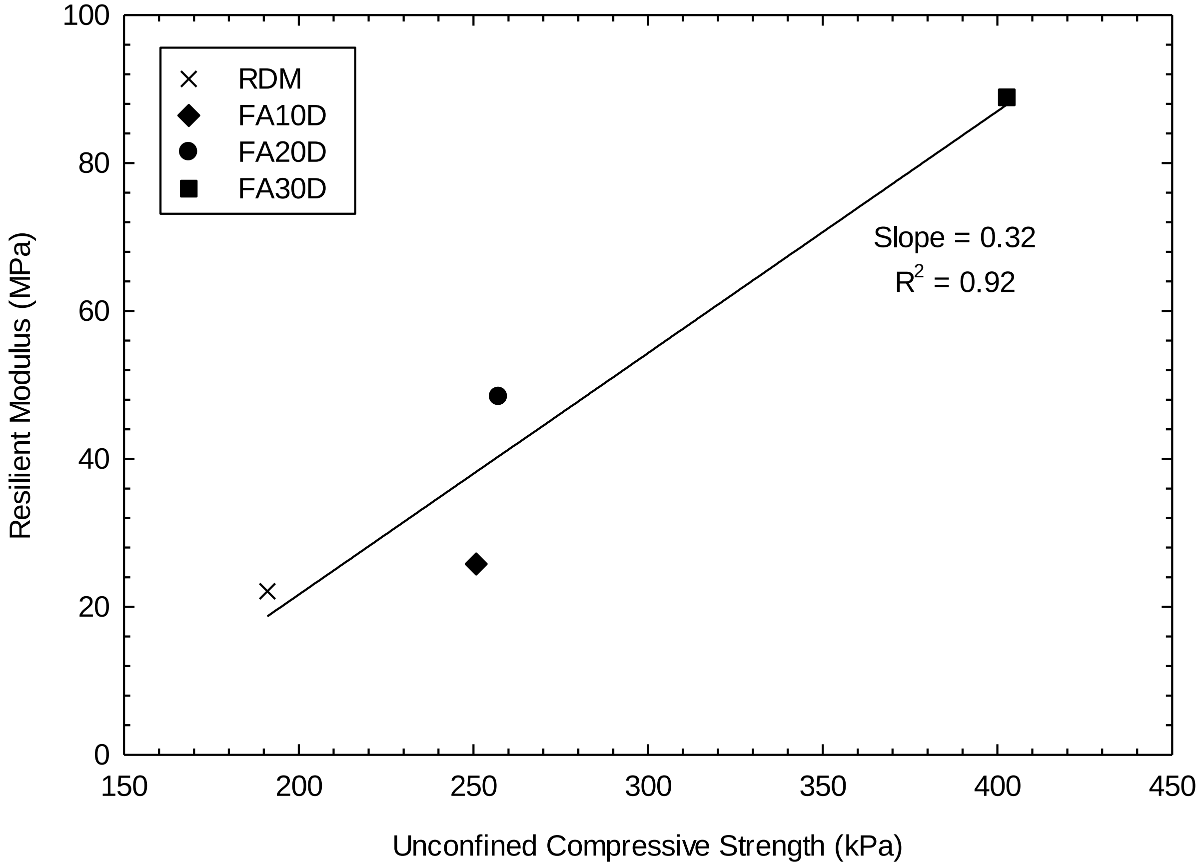


Figure 7.13 Resilient modulus (at deviator stress = 21 kPa) versus unconfined compressive strength of RDM and SDM specimens. SDM specimens were cured for 28 days prior to resilient modulus testing and 24 days prior to unconfined compressive testing

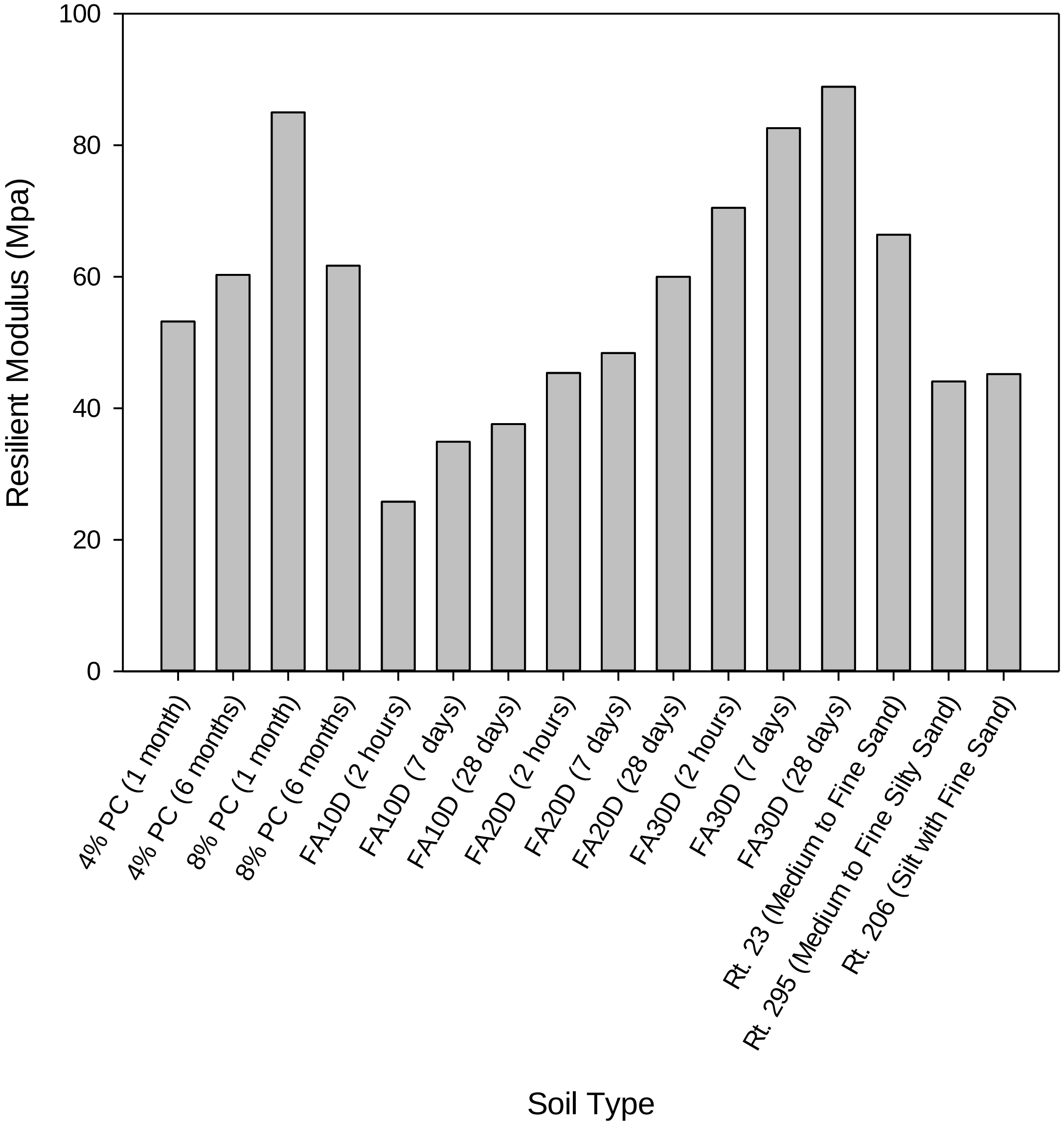


Figure 7.14 Comparison of resilient modulus values between SDM and typical NJ base materials

APPENDIX D

GEOTECHNICAL TESTING DATA IN CHAPTER 7

Table A.1. Optimum water contents and maximum dry unit weights of RDM and SDM specimens

Samples	W_{opt} (%)	γ_d(kN/m^3)
FA0D	30	12.9
FA10D	27	13.4
FA20D	26	13.4
FA30D	23	13.9

Figure A.2. Liquid limit (LL) and plasticity index (PI) of RDM and SDM specimens

Samples	LL			PL			PI		
Curing Time	2hrs	7days	28days	2hrs	7days	28days	2hrs	7days	28days
FA0D (RDM)	61.50			42.20			19.30		
FA10D	57.50	56.70	54.50	39.70	39.12	39.73	17.80	17.58	14.77
FA20D	52.50	51.30	49.80	36.40	37.38	37.77	16.10	13.92	12.03
FA30D	48.50	47.50	44.10	37.20	38.56	35.78	11.30	8.94	8.32

Figure A.3. Undrained shear strength (UU test) of RDM and SDM specimens

c_u (kN/m2)	2 hours	7days	28 days
RDM	240		
FA10D	255.25	309.75	372.75
FA20D	265.75	432.25	551.75
FA30D	292.75	498.75	713.75

Figure A.4. Unconfined compressive strength of RDM and SDM specimens with and without 12 F-T cycles

Specimen		RDM	FA10D	FA20D	FA30D
UCS (kPa)	with 12 F-T cycles	182	240	255	265
	cured for 24 days	191	251	257	403

Figure A.4. CBR Value of RDM and SDM specimens

FA Content (%)	CBR Value		
	2 Hours	7 Days	28 Days
0	1.5		
10	3.0	10.1	15.0
20	3.7	14.5	15.3
30	7.8	20.0	21.4

Figure A.5. Resilient Modulus of RDM and SDM specimens

Resilient Modulus (MPa)	2 Hours	7 Days	28 Days
RDM	22.1		
FA10D	25.8	34.9	37.6
FA20D	45.4	48.4	60
FA30D	70.5	82.6	88.9

Figure A.6. Comparison of resilient modulus values between SDM and typical NJ base materials

Sample Type	Curing Time	Compaction	Resilient Modulus
4% PC	1 Month	90%	53.2
4% PC	6 Month	90%	60.3
8% PC	1 Month	90%	85
8% PC	6 Month	90%	61.7
10% FA	2 Hours	Max. Dry Density	25.8
10% FA	7 Days	Max. Dry Density	34.9
10% FA	28 Days	Max. Dry Density	37.6
20% FA	2 Hours	Max. Dry Density	45.4
20% FA	7 Days	Max. Dry Density	48.4
20% FA	28 Days	Max. Dry Density	60
30% FA	2 Hours	Max. Dry Density	70.5
30% FA	7 Days	Max. Dry Density	82.6
30% FA	28 Days	Max. Dry Density	88.9
Rt. 23 (Medium to Fine Sand)		Max. Dry Density	66.4
Rt. 23 (Medium to Fine Silty Sand)		Max. Dry Density	44.1
Rt. 23 (Silt with Fine Sand)		Max. Dry Density	45.2