Subgrade Stabilization ME Properties Evaluation and Implementation

Minnesota Department of Transportation

RESEARCH SERVICES

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The state and many counties throughout Minnesota are using a variety of subgrade stabilization techniques for various materials used in road construction. Such methods appear to improve constructability and lead to increased performance and reduced maintenance. While a number of studies have investigated such stabilization efforts (including materials and techniques, relative increases in strength and/or stiffness, etc.) no overall quantification and summary of the effects of material stabilization have been brought forward with recommendations of parameters to be used for design purposes. Although these techniques and materials are commonly used, minimal information has been obtained relating to the Mechanistic-Empirical (ME) properties of these improved materials such that the more cost-effective designs can be implemented. Not having recommendations for the ME properties of the improved materials, the designer is forced to use values for the non-stabilized material. While this does likely lead to extended road life, costs could be greatly reduced by taking advantage of the improved properties of the stabilized roadway materials.

This project has involved determining which types of subgrade stabilization are being used, identifying which of these stabilization techniques/materials are of interest to the Minnesota Department of Transportation (MnDOT), compiling the results of past research relating to these stabilization techniques, summarizing the results of past research and proposing a mix design procedure that obtains material properties for use in design. This proposed mix design procedure will allow the designer to account for improved stiffness due to stabilization, reducing costs and improving the efficiency of the design.

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Final Report

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The state and many counties throughout Minnesota are using a variety of subgrade stabilization techniques for various materials used in road construction. Such methods appear to improve constructability and lead to increased performance and reduced maintenance. While a number of studies have investigated such stabilization efforts (including materials and techniques, relative increases in strength and/or stiffness, etc.) no overall quantification and summary of the effects of material stabilization have been brought forward with recommendations of parameters to be used for design purposes. Although these techniques and materials are commonly used, minimal information has been obtained relating to the Mechanistic-Empirical (ME) properties of these improved materials such that the more cost-effective designs can be implemented. Not having recommendations for the ME properties of the improved materials, the designer is forced to use values for the non-stabilized material. While this does likely lead to extended road life, construction costs could be greatly reduced by taking advantage of the improved properties of the stabilized roadway materials.

This project has involved determining which types of subgrade stabilization are being used, identifying which of these stabilization techniques/materials are of interest to the Minnesota Department of Transportation (MnDOT), compiling the results of past research relating to these stabilization techniques, summarizing the results of past research and proposing a mix design procedure that obtains material properties for use in design. This proposed mix design procedure will allow the designer to account for improved stiffness due to stabilization, reducing costs and improving the efficiency of the design.

The initial effort of this project involved developing a fairly comprehensive list of potential stabilization materials and/or techniques that might be of additional consideration. Such materials included fly ash, cement, lime, emulsion, foamed asphalt, and a variety of recycled byproducts. A list was developed to show the options available along with a brief description of that material/technique. Based on the comprehensive list, the researchers developed a revised list with proposed materials for further investigation and consideration. The Technical Advisory Panel (TAP) for the project reviewed the comprehensive and abbreviated lists and focused the study on materials which they felt were most appropriate for MnDOT consideration.

Based on this list from the TAP group, a significant literature review was performed to obtain as much information as possible relating to these materials. The original hope was to collect project data, from both field and laboratory testing, which provided material properties for native (unstabilized) materials along with modified properties of stabilized materials. This would provide guidance as to the degree of improvement that would be anticipated for various materials, in hopes of proposing a factor that would apply to all soils for which that stabilization material was utilized.

Several facts became evident as the literature review progressed. First, while the number of research and construction projects relating to subgrade stabilization was significant, the number of projects for which both unstabilized AND stabilized soil properties were determined and reported was quite small. Not having both sets of data did not allow quantification of the effects of stabilization from enough projects to clearly establish a “factor” of improvement. Second, for the cases where soil property information was available for both the stabilized and unstabilized soils, it was found that the degree of improvement was highly variable as a function of soil type, stabilization material content, water content, and other parameters. It became clear that establishing one factor of improvement for a given stabilization method that applied to all conditions was not realistic.

With this finding, the project scope was altered somewhat, developing a procedure to be used to establish appropriate factors as part of the mix design process for the stabilized soil. This procedure was developed for the two most probable subgrade stabilization materials, namely fly ash and cement. The procedure involves performing compaction and stiffness tests on the native (unstabilized) material, followed by similar tests on stabilized soils at specified stabilization material contents (6% fly ash and 2% cement). Obtaining the combination of unstabilized and stabilized soil parameters allows the degree of improvement to be quantified, which allows the designer to account for this improvement in the design of the roadway. This will allow a more efficient and less expensive roadway design that takes advantage of the improved subgrade properties due to stabilization.

CHAPTER 1.
PROJECT OVERVIEW

1.1 Project Background

The state of Minnesota and many counties throughout Minnesota, along with other entities throughout the Midwest, are using a variety of stabilization techniques for various materials used in road construction. Such methods appear to improve constructability and lead to increased performance and reduced maintenance.

While a number of studies in the past have investigated such stabilization efforts (including materials and techniques, relative increases in strength and/or stiffness, etc.) no overall quantification and summary of the effects of material stabilization have been brought forward with recommendations of parameters to be used for design purposes. Although these techniques and materials are becoming more commonly used, minimal information has been obtained relating to the Mechanistic-Empirical (ME) properties of these improved materials such that the more cost-effective designs can be implemented. Not having recommendations for the ME properties of the improved materials, the designer is forced to use values for the non-stabilized material. While this does likely lead to extended road life, costs could be greatly reduced by taking advantage of the improved properties of the stabilized roadway materials. In cases where resilient modulus data have been obtained for such materials, review should be done to ensure that these data are appropriate for use in pavement design.

This project has involved working with state personnel to determine which types of subgrade stabilization have been attempted (building on a related study by Gene Skok a few years ago), identifying which stabilization techniques are of interest to MnDOT, gathering research reports and papers relating to these stabilization techniques, summarizing the results of that research and proposing material property test recommendations for future use in design.

A significant portion of the project involved compiling and reviewing past research reports to compare data collected from various stabilization techniques as applied to a range of materials. This allowed some degree of quantification and/or estimation of the ME properties of the materials for implementation by state and county agencies in future road design. However, it was quickly noted that the degree of improvement was a function of many soil and stabilization material properties and that using a single factor for all conditions was not practical. In fact, there was a significant range of improvement factors (for either stiffness or strength) for each of the cases considered. Thus, a methodology for obtaining appropriate factors for a specified project was proposed instead of proposing a factor applicable to all cases. Being able to obtain appropriate values for the ME parameters for various improvement methods used on future projects and to summarize and make this data readily available would be a great benefit to MnDOT and many of the counties throughout the state that would like to incorporate such techniques and apply more appropriate design parameters but need confirmation that such a project would be cost-effective.

1.2 Project Tasks, Objectives and Scope

As mentioned in the previous section, MnDOT, counties and other entities throughout Minnesota have used various materials and techniques in an attempt to improve material and subgrade properties for improved pavements and unimproved gravel roads. Materials of note include foamed asphalt, shredded tires, fly ash, other byproducts, emulsion, lime, and additional items. An earlier study for MnDOT by Gene Skok (Skok et. al., 2003) presented various methods of subgrade stabilization in practice at the time.

The main objectives of this research were to: 1) determine the various materials/techniques that have been used (or are currently being used) by MnDOT and local agencies, using the Skok study as a basis, 2) evaluate the usefulness of such materials in achieving the desired results and identify those most useful to MnDOT, 3) obtain and assess data obtained from earlier research efforts relating to stabilization materials and techniques, 4) if possible, propose appropriate Mechanistic-Empirical (ME) material properties based on past research projects such that these properties can be implemented for more cost-effective road design, otherwise, 5) develop a procedure to be used to obtain ME parameters for stabilized subgrade materials, and 6) provide this information to the state and counties to aid in making more efficient designs with respect to future projects. Failed stabilization efforts should be addressed to determine (if possible) the reason for the failure and to comment on whether adjustments could be made such that the method could be used successfully in other circumstances. Successful utilizations can certainly be shared such that other groups can benefit from the implementable technologies and ME properties.

This project was intended to be a combination of a review of literature with respect to material stabilization techniques as well as a study to consolidate the design properties of such improved materials (from both field and laboratory testing in previous research) in order to come up with useable design parameters for various stabilization materials and techniques, if possible.

The literature review related to various applications of stabilization, specifically, literature sources that provide information regarding various materials and/or methods that are currently being used to facilitate stabilization for highway applications. This required numerous hours of literature review to provide and compile the appropriate references and research reports. Several facts became evident as the literature review progressed. First, while the number of research and construction projects relating to subgrade stabilization was significant, the number of projects for which both unstabilized AND stabilized soil properties were determined and reported was quite small. Not having both sets of data did not allow quantification of the effects of stabilization from enough projects to clearly establish a “factor” of improvement. Second, for the cases where soil property information was available for both the stabilized and unstabilized soils, it was found that the degree of improvement was highly variable as a function of soil type, stabilization material content, water content, and other parameters. It became clear that establishing one factor of improvement for a given stabilization method that applied to all conditions was not realistic.

With this finding, the project scope was altered somewhat, developing instead a procedure to be used to establish appropriate factors as part of the mix design process for the stabilized soil. This procedure was developed for probable subgrade stabilization materials. The procedure involves performing compaction and stiffness tests on the native (unstabilized) material, followed by similar tests on stabilized soils at specified stabilization material contents. Obtaining the combination of unstabilized and stabilized soil parameters allows the degree of improvement to be quantified, which allows the designer to account for this improvement in the design of the roadway. This will allow a more efficient and less expensive roadway design that takes advantage of the improved subgrade properties due to stabilization.

CHAPTER 2.
TASK 1 – IDENTIFY STABILIZATION MATERIALS AND TECHNIQUES OF INTEREST TO MNDOT

This task has built upon the Skok study (Skok et al., 2003) which identified a number of subgrade stabilization techniques being used at the time of that study. In addition to methods of subgrade stabilization, techniques applicable to base, sub-base and other roadway components have been identified. This section of the report will be reviewed by TAP members to identify/confirm which stabilization materials and techniques are of interest to MnDOT and will be further investigated in subsequent tasks.

2.1 Initial Recommendations from MnDOT

During the workplan development and scoping discussions in the early stages of this project, John Siekmeier (MnDOT Research) compiled a list of projects that had some degree of association with the requirements of this effort. He identified a number of projects with which he was aware that could be a starting point for the compilations and recommendations associated with this project. Table 2.1 shows the initial list provided to the researchers by Siekmeier as a starting point for this effort.

In reviewing the projects in Table 2.1, there were two main categories for these projects. One group includes many projects relating to obtaining/estimating material properties (Dynamic Cone Penetrometer, Light Weight Deflectometer, Intelligent Compaction, Resilient Modulus testing, etc.) These projects are a valuable reference in establishing baseline values for "unstabilized" materials, along with obtaining a clear picture as to how such tests are used to measure and/or approximate material properties for use in pavement design.

The second group includes a more focused effort to evaluate stabilization techniques and/or methods of stabilization, along with (in some cases) data collected from stabilized materials. The materials specifically included in the list from Table 2.1 include fly ash, waste/ recycled materials, geosynthetics, enzyme solutions, and others. Additional stabilization materials and techniques have been identified in several of these references and will be discussed in more detail during the course of this report.

Given this initial information as provided by MnDOT through John Siekmeier, the projects and techniques in Table 2.1 have provided a basis from which to build this effort.

2.2 Additional Information Collected

The next step was to determine additional materials and/or methods that might also be of interest to MnDOT in further examination. A number of additional projects and methods were identified in a literature review of available stabilization techniques. The additional information collected to date is included in Table 2.2, which includes only projects/materials/methods not provided in Table 2.1.

Table 2.1. List of relevant projects as provided by John Siekmeier (MnDOT).

Project Name	Start Date	End Date	Funding Source	Activity	Product
Validation of DCP and LWD Moisture Specifications for Granular Materials	2004	2006	LRRB 829	Complete	Mn/DOT 2006-20
Using the DCP and LWD for Construction Quality Assurance	2006	2009	LRRB 860	Complete	Mn/DOT 2009-12
Field Validation of IC Monitoring Technology for Unbound Materials	2005	2007	FHWA Mn/DOT	Complete	Mn/DOT 2007-10
Intelligent Compaction and In-Situ Testing at Mn/DOT TH 53	2005	2006	Mn/DOT	Complete	Mn/DOT 2006-13
Implementation of Fly Ash Screening Tool Training Phase 2	2005	2006	Mn/DOT	Complete	training
Implementation of Fly Ash Screening Tool Training Phase 1	2005	2006	LRRB	Complete	training
Demonstration of Ash Utilization in Low Volume Roads	2003	2007	LRRB 810	Complete	Mn/DOT 2007-12
Pavement Design Using Unsaturated Soil Technology	2003	2007	Mn/DOT	Complete	Mn/DOT 2007-11
Screening Tool for Using Waste Materials in Paving Projects	2003	2005	LRRB 795	Complete	Mn/DOT 2005-03
Moisture Effects on PVD and DCP Measurements	2002	2006	Mn/DOT	Complete	Mn/DOT 2006-26
Small Strain and Resilient Modulus Testing of Granular Soils	2001	2004	Mn/DOT	Complete	Mn/DOT 2004-39
The Use of Geosynthetics to Reinforce Low Volume Roads	1999	2001	LRRB 713	Complete	Mn/DOT 2001-15
Fly Ash Stabilization	1999	2010	LRRB 736	Active	
NCHRP Synthesis 38-09 Estimation of Resilient Modulus	2006	2009	NCHRP	Active	report
NCHRP 21-09 Examining the Benefits and Adoptability of IC	2005	2009	NCHRP	Active	report
FHWA National Pavement Subgrade Performance Study SPR-2 (208...) (Pavement Subgrade Performance Study)	2001	2010	Pool Funded	Active	
NCHRP Synthesis 278 Measuring In-Situ Mechanical Properties of P...	1997	1999	NCHRP	Complete	Syn 278
Intelligent Compaction Implementation Research Iowa State	2006	2009	FHWA Mn/DOT	Active	
Hydraulic and Mechanical Properties of Recycled Material	2006	2009	LRRB 846	Active	
Use of Fly Ash for Reconstruction of Bituminous Roads	2006	2009	LRRB 847	Active	Mn/DOT 2009-27
Mn/PAVE for Local Roads	2004	2007	LRRB 828	Active	
Chemical Inventory and Database Development for Recycled Material Substitutions	2003	2006	Mn/DOT	Complete	Mn/DOT 2006-28
Investigation of Stripping in Minnesota Class 7 (RAP) and Full-Depth Reclamation Base Materials	2005	?	LRRB 831	?	Mn/DOT 2009-05
Resilient Modulus and Strength of Base Course with Recycled Bituminous Material	2003	2006	LRRB 812	Complete	Mn/DOT 2007-05
Improvement and Validation of Mn/DOT DCP Specifications for Aggregate Base Materials and Select Granular Test	2003	2005	LRRB 794	Complete	Mn/DOT 2005-32
Moisture Retention Characteristics of Base and Sub-base Materials	2002	2005	Mn/DOT	Complete	Mn/DOT 2005-06
Monitoring Geosynthetics in Local Roadways	2001	?	LRRB 768	?	
Special Practices for Design and Construction of Subgrades in Poor, Wet, and/or Saturated Soil Conditions	2001	2003	LRRB 772	Complete	Mn/DOT 2003-36
Best Practices for the Design and Construction of Low Volume Roads (Revised)	2001	2002	LRRB 747	Complete	Mn/DOT 2002-17REV
Preliminary Laboratory Investigation of Enzyme Solutions as a Soil Stabilizer				Complete	Mn/DOT 2005-25
Geotechnical Solutions for Soil Improvement, Rapid Embankment Construction, and Stabilization of the Pavement Working Platform	2007	2009	SHRP2	Active	
Design Procedures for Bit Stabilized			INV 836	Complete	
Pavement Design 301			INV 645	Active	

Table 2.2 includes several of the materials and techniques provided in Table 2.1, as well as methods of determining and/or estimating material properties from various field and laboratory tests. In addition, materials such as shredded tires, recycled pavement, soil stabilization agents, emulsion, recycled/reclaimed glass, and various industrial byproducts were identified. Several of these were also mentioned in reports obtained and reviewed to date, with extensive research having been conducted on several of the materials/techniques listed.

Table 2.2. List of additional relevant projects obtained.

Additional Reports		
Project Name	Funding Source	Product
Continuous Compaction Control Mn/ROAD Demonstration		Mn/DOT 2005-07
Deformability of Shredded Tires		Mn/DOT 1999-13
Using Shredded Waste Tires as a Lightweight Fill Material for Road Subgrades		Mn/DOT 94-10
Intelligent Compaction Implementation: Research Assessment (Similar to 2007-10)		Mn/DOT 2008-22
Resilient Modulus Development of Aggeragate Base and Subbase Containing Recycled Bituminous and Concrete for 2002 Design Guide and Mn/PAVE Pavement Design		Mn/DOT 2007-25
Soil Stabilization Field Trial	FHWA	MS-DOT-RD-05-133
Performance of Soil Stabilization Agents		K-TRAN: KU-01-8
Stabilization Techniques for Unpaved Roads		VTRC 04-R18
Field Trial of Solvent-Free Emulsion in Oregon	FHWA	OR-RD-03-12
An Investigation of Emulsion Stabilized Limestone Screenings		HR-309
Laboratory Performance Evaluation of CIR-emulsion and Its Comparison against CIR-foam Test Results from Phase II		TR-578
Pavement Subgrade Stabilization and Construction Using Bed and Fly Ash		FDOT 44755
Fabric for Reinforcement and Separation in Unpaved Roads		Mn/DOT 1999-04
Construction and Instrumentation of Full-Scale Geogrid Reinforced Pavement Test Sections	Pool Funded	TPF-5(010)
Reclaimed Glass		2001MRRDOC011
Recycled Glass		3138-2A2e4 2001-07-10
Official Mn/DOT Standard of Engineering Practice for the use of Shredded Tires in Roadways		Fact Sheet
Utilization of Recycled Materials in Illinois Highway Construction		IL-PRR-142
Use of Factory-Waste Shingles and Cement Kiln Dust to Enhance the Performance of Soil Used in RoadWorks		143750
Use of Cement Kiln Dust for Subgrade Stabilization		KS-04-3
Wood Chips as Lightweight Fill		MN/RC 1998-05U
Historical Use of Taconite Byproducts as Construction Aggregate Materials in Minnesota		NRRI-RI-2006-02
Use of Taconite Aggeragates in Pavement Applications		MN/RC-2010-24
Use of Taconite Aggeragates in Pavement Applications		MPR-6(023)
Preliminary Investigation of RAP and RAS in HMAC		OR-RD-10-12
Incorporation of Recycled Asphalt Shingles in Hot-Mixed Asphalt Pavement Mixtures		MN/RC-2010-08
Waste Procucts in Hichway Construction		MN/RC-1993-16
Recycled Pavements Using Foamed Asphalt in Minnesota		MN/RC 2009-09
Investication of Recycled Asphalt Pavement (RAP) Mixtures		MN/RC – 2002-15

2.3 Recommendations for Further Consideration

The final step for Task 1 was to identify materials and methods appropriate to MnDOT and other local and regional entities for further consideration. Such materials and methods would then be researched to identify projects from which material property data could be collected and analyzed (Task 2). Initially, a summary of each of the techniques identified to this point was prepared, with a short description of the material/technique included. Similar materials were grouped together in cases where sufficient similarity in outcome/behavior was expected. In some cases, materials and/or methods seeming to be similar were left in distinct, separate groups until additional information is obtained to warrant combining them. This full list of materials is included in Table 2.3.

Table 2.3. Comprehensive list of stabilization materials and techniques.

Material	Description
Portland Cement	Generally well-graded granular materials that possess sufficient fines to produce a floating aggregate matrix are best suited for Portland cement stabilization. Used mainly for strength and stiffness stabilization.
Lime	Used to stabilize high plasticity soils to decrease plasticity, increase workability, reduce swell, and increase strength.
Fly ash	A pozzolanic material that reacts with lime and is usually used in combination with lime in soils that have little or no plastic fines.
Lime-cement-flyash (LCF)	Used successfully for base coarse stabilization
Asphalt Emulsion	Chipseal involves spraying the road surface with asphalt emulsion followed by a layer of crushed rock, gravel or crushed slag. Slurry Seal involves the creation of a mixture of asphalt emulsion and fine crushed aggregate that is spread on the surface of a road. Cold mixed asphalt can also be made from asphalt emulsion to create pavements similar to hot-mixed asphalt, several inches in depth and asphalt emulsions are also blended into recycled hot-mix asphalt to create low cost pavements.
Bed & Fly Ash Mixture	Used for sub-grade, sub-base, and base applications at different mixtures. Reffered to as EX-Base. Chipseal involves spraying the road surface with asphalt emulsion followed by a layer of crushed rock, gravel or crushed slag. Slurry Seal involves the creation of a mixture of asphalt emulsion and fine crushed aggregate that is spread on the surface of a road. Cold mixed asphalt can also be made from asphalt emulsion to create pavements similar to hot-mixed asphalt, several inches in depth and asphalt emulsions are also blended into recycled hot-mix asphalt to create low cost pavements.
Geotextiles	The geotextile is used to permanently separate two distinct layers of soil in a roadway. The classic example is where a road is to be built across a poorly drained, fine-grained soil (clay or silt) and a geotextile is laid down prior to placing gravel. This keeps the soft, underlying soil from working its way up into the expensive gravel and it keeps the gravel form punching down into the soft soil. The full gravel thickness remains intact and provides full support for many years.
Geogrids	Use Geogrids for subgrade stabilization for improved site access, reduced aggregate requirements, reduced undercut, reduced maintenance, and simple installation. The interlocking action of aggregate and Tensar Geogrids results in a stiffened granular platform. This platform enhances load distribution, much like a snowshoe distributes a person's weight evenly over soft snow. The "snowshoe effect" generated, reduces the stress applied to the subgrade.
Geofoam	"EPS Geofoam" describes low-density cellular plastic foam solids used in geotechnical applications such as lightweight fill for construction on soft ground, for slope stabilization, and retaining wall or abutment backfill; as well as for roadway and runway subgrade insulation and foundation insulation.
Foamed or Cellular Concrete	This product is generally job-produced as a cement/water slurry with preformed foam blended for accurate control and immediate placement.
Shredded Tires	Shredded tires have been used for lightweight fill in the United States and in other countries since the mid 1980s. More than 85 fills using shredded tires as a lightweight fill have been constructed in the United States. In 1995, three tire shred fills with a thickness greater than 8 m (26 ft) experienced an unexpected internal heating reaction. As a result, FHWA issued an Interim Guideline to minimize internal heating of tire shred fills in 1997, limiting tire shred layers to 3 m (9.8 ft).

Table 2.3. Comprehensive list of stabilization materials and techniques (cont.)

Material	Description
Boiler Slag	Boiler slag has been used for backfill since the early 1970s. Many state highway department specifications allow the use of boiler slag as an acceptable fine or coarse aggregate.
Air Cooled Slag	
Iron Blast Furnace Slag (BFS)	This is the by-product from the reduction of iron ores to produce molten iron and molten slag. 1. When allowed to cool slowly to a crystalline rod< form It becomes a light gray vesicular rock known as Air-Cooled Blast Furnace Slag. Principle uses include: **Uncrushed** - fill and embankments (particularly areas subject to severe loading such as mainline rail systems), working platforms on difficult sites pavements, where binding fines are produced by rolling to break the slag down to fill the voids. **Graded road base** -- on its own or blended with other slags and/or with other natural rocks and sands. **Crushed and graded** - for concrete aggregates, concrete sand, glass insulation wool, filter medium, and use under concrete slabs as a platform 2. By passing the molten slag through high volume high pressure water sprays, a glassy, sand-Type (granulated) material Is formed, known as Granulated Blast Furnace SIag.The color of this product is very similar to normal beach sand.
Basic Oxygen Steel making Slag (BOS)	This slag Is formed when molten Iron, scrap metals and various fluxes, such as lime, are oxidized by injecting large amounts of pure oxygen into the molten iron mix to create molten steel and molten slag. Slow cooling of the molten slag produces a dense rock material. Principal uses include: Blending with many other products such as granulated slag, fly ash and lime to form pavement material Other uses Include, skid resistant asphalt aggregate, rail ballast asphaltic concrete aggregate, soil conditioner, hard stand areas and unconfined construction fill.
Electric Arc Furnace Slag (EAF)	Produced when scrap metal and fluxes are oxidized by the use of an electric current, woken slag is generally placed into ground bays for cooling. Both BOS and EAF slags are somewhat heavier than Blast Furnace Slag and most quarried rock material. Uses include Blending with many other products such as granulated slag, fly ash and lime to form pavement material, skid resistant asphalt aggregate and unconfined construction fill.
Cement Kiln Dust	
Wood Products	Chipped Bark, Sawdust, Wood Fiber
Dried Peat	
Cinders	
Seashells	
Enzyme Solution	
Soy	
Sugar Beets	
Geosynthetics	
Recycled Bituminous & Concrete	
Taconite materials	Expansion and maintenance of roadway infrastructure creates a demand for high quality paving aggregates. Taconite industry rock and tailings are a potential source of virgin paving aggregates. Currently there is limited information available for implementing these products in construction design specifications. The main goal of this coordinated research effort is to determine if taconite aggregates can be practically used in pavement construction projects. The issues involve both engineering/ material properties and economics/logistics of transportation
Shingles	
Recycled Materials	Reclaimed Asphalt Pavement (RAP), Fly Ash (FA), Reclaimed Concrete Material (RCM), Foundry Sand (FS), Bituminous, Concrete, Glass, Air-Cooled Blast Furnace Slag, By-Product Lime, Fly Ash, Glass Beads, Ground Granulated Blast Furnace Slag, Microsilica, Steel Slag, Steel Reinforcement, Waste Plastics, Wet-Bottom Boiler Slag, etc.

Given this substantial list of stabilization methods and materials, the next step was to propose those methods/materials most relevant to the state of practice in Minnesota. Besides relevance to the state of practice, the applicability of the material and/or method with respect to ME design was also considered. For example, while geosynthetic or geotextile materials may be very useful in allowing construction on weak foundation materials and other applications, such function will not influence the stiffness parameters used in pavement design. Similarly, products with a relatively short lifespan were also removed from consideration (i.e., wood, dried peat products). Thus, for the purposes of this project, geosynthetics, geotextiles, wood products and other products were not considered as appropriate for additional research. Such topics may be investigated further, if deemed appropriate by the Technical Advisory Panel. Once such deletions were accomplished, the list of methods and materials provided in Table 2.4 remained. Descriptions (included in Table 2.3) have not been repeated in Table 2.4.

Table 2.4. Select list of stabilization materials and techniques for further consideration.

Material
Portland Cement
Lime
Fly ash
Lime-cement-flyash (LCF)
Asphalt Emulsion
Bed & Fly Ash Mixture
Foamed or Cellular Concrete
Shredded Tires
Boiler Slag
Air Cooled Slag
Iron Blast Furnace Slag (BFS)
Basic Oxygen Steel making Slag (BOS)
Electric Arc Furnace Slag (EAF)
Cement Kiln Dust
Cinders
Enzyme Solution
Soy
Sugar Beets
Taconite materials
Shingles
Recycled Materials

While the materials listed in Table 2.4 appear to have been used on a variety of projects, in many cases they are used in more of a “filler” capacity, where there may not be significant structural benefit from using the material as an additive or from a stabilization perspective. Additional information was made available as Task 3 moved forward and additional data were collected for such materials. Technical Advisory Panel (TAP) members reduced the list above by suggesting stabilization materials to be evaluated further.

CHAPTER 3.
TASK 2 – IDENTIFY PAST RESEARCH PROJECTS RELATING TO IDENTIFIED MATERIALS

Much of Task 2 occurred simultaneously to Task 1, since many of the methods/materials were discovered based on projects encountered in the literature review. Where possible, project report and related technical papers were collected to allow review during Task 3. A summary of the proposed stabilization methods for which additional research and assessment occurred is provided in Chapter 4, along with project/report references for that particular method. Many of the projects identified in Task 1 (and considered worth pursuing by TAP members) were compiled and reviewed. Additional reports and papers were also compiled through an extensive literature review performed for Task 3.

CHAPTER 4.
TASK 3 – OBTAIN AND ASSESS RESEARCH REPORTS RELATING TO STABILIZATION MATERIALS AND TECHNIQUES

The initial effort for this project involved working with the Technical Advisory Panel (TAP) to identify which stabilization techniques are of interest to MnDOT. This was accomplished during the Task 1 and Task 2 work. Task 3 was an effort to compile the results of past research relating to these stabilization techniques and to summarize the results of past research. A significant portion of the project involved compiling and reviewing past research reports to compare tests performed and data collected from various stabilization techniques as applied to a range of materials.

The literature review has investigated various applications of stabilization, specifically, literature sources that provide information regarding various materials and/or methods that are currently being used to facilitate stabilization for highway applications. This required numerous hours of effort to provide, compile and review the appropriate references and research reports. Electronic copies of these research reports and references have been collected. Hard copies, complete or in some cases partial, of these reports have also been compiled. Both hard copies and electronic copies of these reports have been provided to MnDOT. Assessment of past research efforts has included an effort to determine or estimate ME properties of the construction materials, along with procedures used to obtain such parameters. During the course of Task 3, an effort has been made to compile the list of materials and methods used and provide insights as to the usefulness of this information to MnDOT. Additional efforts during the remainder of the project included recommending modified methods to be used to obtain ME parameters when a general relationship/recommendation is not available.

This task built upon the previous tasks, which identified a number of subgrade stabilization techniques of interest to MnDOT. In addition to methods of true “subgrade” stabilization, techniques applicable to base, sub-base and other roadway components have been identified. This was done to maintain material property relationships which MAY be applicable to subgrade materials similar in nature to the base and sub-base materials. This section of the report provides a summary of the research reports evaluated. These include many of the reports identified in Tasks 1 and 2, along with a number of additional reports discovered during the course of the Task 3 effort. Report and paper citations are included, along with a summary of the project and an identification of the information from the project deemed most appropriate for the purposes of this study. Reports relating to the various techniques and materials are compiled by section, with some slight overlap in topics. Journal or report titles and full citations are included, along with the summary of the project/report and other important information.

4.1 Fly Ash and Variations

4.1.1 Class F Fly-Ash-Amended Soils as Highway Base Material

Arora, S, and Aydilek, A. H. (2005). "Class F Fly-Ash-Amended Soils as Highway Base Material." *J. Materials in Civil Eng.*, 17(6) 640-649.

Class F fly ash cannot be used alone in soil stabilization applications as it is not self-cementing. An activator such as Portland cement or lime must be added to produce cementitious products often called pozzolan stabilized mixtures. The developed mixture must possess adequate strength and durability, should be easily compacted, and should be environmentally friendly. Roadways have a high potential for large volume use of the fly ash stabilized soils. The main objective of this study is to investigate the use of Class F fly ash amended soil–cement or soil–lime as base layers in highways. A battery of tests was performed on soil–fly ash mixtures prepared with cement and lime as activators. Unconfined compression, California bearing ratio, and resilient modulus tests were conducted. Finally, required base thicknesses were calculated using the laboratory-based strength parameters. Results of the study show that the strength of a mixture is highly dependent on the curing period, compactive energy, cement content, and water content at compaction. Lime treatment does not provide sufficient strength for designing the mixtures as highway bases. Freeze–thaw cycles do not have any detrimental effect on cement-treated mixtures. A power function in terms of bulk stress used for granular soils can accurately model the resilient moduli. Most of the factors considered have an impact on the thickness of the base layer.

Fly ash has historically been used in concrete and soil stabilization. Due to the lack of self-cementitious characteristics, Class F fly ash needs an activator (e.g., cement), and currently only 32% of this ash has been beneficially reused. Roadways are the largest application area and reuse of fly ash could provide significant cost savings as most of the ash is landfilled today. This study was conducted to investigate the effect of fines content, curing period, molding water content, compactive effort, cohesion, and cement or lime addition on geomechanical properties of Class F fly ash amended highway bases. Mixtures with 40% Class F fly ash and sandy soil with 6–30% cohesionless fines were prepared. Three different molding water contents (optimum, 4% wet of optimum, and 4% dry of optimum) and two different compactive energies (standard and modified Proctor) were studied. Unconfined compression, CBR, and resilient modulus (M_R) tests were conducted to investigate the suitability of the mixtures as highway base materials. For the various mixtures tested, resilient modulus values varied by a factor of four due to variation in the water content and the percentage of fine material (the amount of fly ash was constant for all tests.)

The base thicknesses were calculated for different mixture designs using their corresponding q_u, CBR, and M_R values. It should be noted that the obtained results are valid for a particular fly ash used in this study. The observations are summarized as follows:

1. The variation of q_u with varying amounts of cohesionless fines was not consistent. On the other hand, addition of 10% kaolinite generally increased the strength of a mixture compacted at optimum moisture content. The cohesionless fines were varied from 6 to 30% of sand by weight in this study, and hence the observed results should be interpreted only for this range.
2. An increase in strength can be obtained in the field by compacting the soil using higher compactive efforts (i.e., modified versus standard effort). The strength increased with increased curing time for the specimens, irrespective of their molding water contents. The highest strength was observed at 90 days. The test results showed that the water content at compaction could affect the q_u of the mixture design considerably. The performance of the fly ash, soil, and cement mixture can be significantly increased by preventing the

intrusion of excess water in the field. It is recommended to compact the base layer at dry of optimum for higher strength. Alternatively, compaction may be performed at optimum moisture content; however, engineers should be careful concerning rain or any other addition of unwanted water at the time of compaction.

3. CBR, q_u, and M_R increased with increasing cement content; however, the rate decreased beyond 5% cement. That is, the strength of the mixture did not significantly increase when the added cement was higher than 5%. This was true for all three test methods.
4. Lime treatment had a detrimental effect on the mixture designs. An increase in lime content decreased the q_u of the specimens for both 7 and 28 day old specimens. The presence of cohesion lowered the q_u as well as CBR values during lime treatment.
5. For cement-treated specimens, the q_u of 7 day cured specimens increased with increasing number of freeze–thaw cycles. The increase in strength with the number of cycles was more prominent for mixtures that contained 7% cement than for mixtures with 4 and 5% cement. During freezing, the damage caused by ice lenses in the pores of the cement-treated cohesionless specimens was negligible. On the other hand, the presence of kaolinite had a detrimental effect on the durability of specimens, as lower strengths were observed for FA3K-C7 than its cohesionless companion (FA3-C7).
6. Lime-treated specimens survived during freezing and thawing, but their strengths decreased with increasing number of freeze–thaw cycles. The detrimental effect was more prominent on the specimens that contained cohesive fines.
7. Lower base thicknesses were required when higher amount of cement was used, whereas the presence of lime generally increased the required base thicknesses. Mixtures including cohesive fines did not always decrease the required base thickness indicating that use of cohesionless fines, such as sandy soils, should be preferred in designing highway bases.

4.1.2 Fiber-Reinforced Fly Ash Sub-Bases in Rural Roads

Kumar, P, and Singh, S. P. (2008). "Fiber-Reinforced Fly Ash Sub-bases in Rural Roads." *J. Transportation Eng*. 134(4) 171-180.

The influence of various reinforced fly ash parameters was planned to be investigated through static and dynamic load tests and semi-field tests. It was planned to observe the effect of polypropylene fiber reinforcement on conventional parameters of fly ash such as unconfined compressive strength, modulus of elasticity, shear strength, and California bearing ratio. The effect of reinforcements and confinements on permanent strain, resilient strain, and resilient modulus of fly ash were also studied. Tests were carried out to study the effect of reinforcement on rut depth formation on a model section with simulation of field conditions. Based on the results, it has been concluded that fly ash is suitable in subbase, if it is reinforced with polypropylene fiber.

This study was unique in that it addressed the use of fly ash as fill material with only minimal amounts of soil (up to 25%), along with reinforcing fibers added to the fly ash. Thus, the results are not necessarily useful in addressing the effects of a fly ash-stabilized soil, but rather a fiber-stabilized fly ash material with some minimal soil. However, the results of the study did show obvious effects of the change in strength with varying amounts of soil and fly ash. The stiffness (Modulus of Elasticity) of the fly ash was also seen to vary by as much as 50% as a function of the confining pressure, consistent with smaller and larger amounts of soil mixed with the fly ash. The resilient modulus values obtained were shown to vary significantly as a function of the number of loading cycles (up to 10,000 cycles), with modulus values dropping by approximately 20% when comparing 1,000 cycles to 10,000 cycles.

From this study, the following conclusions can be made:

1. The stress–strain behavior of fly ash under static load condition improved considerably due to increase in fiber content and increase in soil content. For example, at a confining pressure 70 kPa and aspect ratio 80, deviator stress increased from 184 kPa in the case of unreinforced sample of fly ash to 422 kPa when the sample was reinforced with 0.5% fiber content.
2. The modulus of elasticity of fly ash increases linearly with confining pressure (σ_3) and nonlinearly with fiber content and aspect ratio of fiber. For example, the percent of increase in modulus of elasticity (E) of fly ash at confining pressure 70 kPa and aspect ratio 100 due to inclusion of 0.2% fiber =70%, due to 0.3% fiber content =107%, and due to 0.4% fiber content =120%.
3. The resilient strain and permanent strain increase with increase in number of load cycles and decreases with increase in confining pressure. For example, the resilient strain at deviator stress of 68 kPa and confining pressure 40 kPa is 0.096% and at 125 kPa deviator stresses and confining pressure 70 kPa; it is 0.151% for unreinforced fly ash after 100 cycles. The permanent strain during repeated loading was shown to obey simple exponential law.
4. Resilient modulus (Mr), modulus of subgrade reaction (k), and field CBR value of fly ash increase due to reinforcement and mixing of soil. For example, resilient modulus (Mr) at confining pressure 40 kPa and deviator stress 68 kPa, modulus of subgrade reaction (k), and field CBR value of unreinforced fly ash are 70.6 MPa, 84 MPa/m, and 9.7%, respectively, and they become 152.3 MPa, 114 MPa/m, and 23.2%, respectively, when reinforced with 0.3% fiber content.
5. Mr increases with confining pressure, but decreases with the increase in both number of load cycles and deviator stress. For example, Mr of fly ash with 0.2% fiber at confining pressure 40 kPa is 73.5 MPa, which becomes 98.2 MPa at confining pressure 100 kPa at 1,000 cycles. The Mr of unreinforced fly ash with 25% soil at deviator stress 95 kPa and 100 load cycles is 193.5 MPa, which becomes 97.3 MPa at deviator stress 125 kPa, and 1,000 load cycles.

4.1.3 Geotechnical Characterization of Some Indian Fly Ashes

Das, S. K., and Yudhbir, (2005). "Geotechnical Characterization of Some Indian Fly Ashes." *J. Materials in Civil Eng.*, 17(5) 544-552.

This paper reports the findings of experimental studies with regard to some common engineering properties (e.g., grain size, specific gravity, compaction characteristics, and unconfined compression strength) of both low and high calcium fly ashes, to evaluate their suitability as embankment materials and reclamation fills. In addition, morphology, chemistry, and mineralogy of fly ashes are studied using scanning electron microscope, electron dispersive x-ray analyzer, x-ray diffractometer, and infrared absorption spectroscopy. In high calcium fly ash, mineralogical and chemical differences are observed for particles, >75 μm and the particles of <45 μm size. The mode and duration of curing significantly affect the strength and stress–strain behavior of fly ashes. The geotechnical properties of fly ash are governed by factors like lime content (CaO), iron content (Fe_2O_3) and loss on ignition. The distinct difference between self-hardening and pozzolanic reactivity has been emphasized.

Indian fly ashes of both low calcium and high calcium were tested for utilization of these as geotechnical material. Scanning electron microscope (SEM), electron dispersive x-ray (EDX), and x-ray diffractometer (XRD) techniques are used to study the morphology, chemistry and mineralogy of fly ash. Geotechnical properties are correlated with chemical composition of fly ash. Based on the findings of investigation reported here the following are the main conclusions.

1. The specific gravity of high calcium fly ash is more than low calcium fly ash and it is comparable to that of soil. This can be due to absence of cenospheres and presence of small amount of plerospheres.
2. The residual carbon content seems to be the controlling factor for compaction characteristics of fly ash though other factors like gradation, iron content, morphology, etc. are also important.
3. For low calcium fly ashes most of the compressive strength of as-compacted samples at OMC is due to capillary forces, which are destroyed, as the sample is air dried. The loss is rapid in coarse-grained fly ash.
4. The gain in strength with time (self-hardening) for high calcium fly ash results from chemical reactivity of minerals like C3A, CS and formation of calcite, calcium silicate hydrate and ettringite. However, higher ettingite content may be detrimental to soil cement due to its expansive nature. For low calcium fly ash it is due to free lime content.
5. The fly ashes can be classified into three categories based on self-hardening and pozzolanic properties.
6. Factors like lime content (CaO), iron content (Fe_2O_3), loss on ignition, morphology, and mineralogy govern the geotechnical properties of fly ashes.

4.1.4 Pavement Subgrade Stabilization and Construction Using Bed and Fly Ash

Jackson, N.M., Mack, R., Schultz, S., and Malek, M. (2007) "Pavement Subgrade Stabilization and Construction Using Bed and Fly Ash." *World of Coal Ash* (WOCA) pp 1-12.

The disposal of ash produced from combustion of solid fuels has been a mayor subject of research and product development since the early 1900's. However, with the onset of modern environmental controls, the technical difficulties of finding suitable markets for such ash are growing ever more difficult. An innovative application has recently been employed by the Jacksonville Electric Authority (JEA) to recycle both the bottom ash and fly ash from two new Circulating Fluidized Bed (CFB) boilers as a stabilizer for non-cohesive sands, which are typical of north Florida. This by-product is currently being marketed under the brand name "EZ-BASE™." The results of laboratory testing and numerous field applications in the immediate market area illustrate that this by-product is effective in stabilizing such sandy soils in pavement and roadway construction applications.

Acceptance Testing is conducted with a nuclear density device and a Speedy moisture gage.

4.1.5 Performance of Stabilized Aggregate Base Subject to Different Durability Procedures

Khoury, N. N. and Brooks R. (2010). "Performance of Stabilized Aggregate Base Subject to Different Durability Procedures." *J. Materials in Civil Eng.*, 22(5) 506-514.

This study examined the effects of two freeze-thaw (FT) laboratory procedures, FT-1 and FT-2, on stabilized aggregate specimens. Cylindrical specimens were stabilized with 10% Class C fly ash (CFA) cured for selected periods and then subjected to either FT-1 or FT-2 cycles. FT-1 and FT-2 procedures consisted of freezing specimens at −25°C for 24 h and thawing them at 21.7°C for another 24 h with a high relative humidity; the only difference was that FT-1 required a membrane around each specimen, while FT-2 required no membranes during freezing and thawing. After being subject to freezing and thawing actions, specimens were then tested for resilient modulus (M_R) and unconfined compressive strength (UCS) values. Results showed that the M_R values of 28-day cured specimens increased as FT-1 cycles increased up to 12, beyond which a reduction in M_R values was observed. For 3-day cured specimens the M_R increased with FT-1 cycles up to 30. The UCS values of 28- and 3-day stabilized specimens also exhibited the same trend as the M_R with FT-1 cycles. In addition, the specimens subject to 30 FT-2 cycles exhibited a higher reduction in M_R values than the specimens subject to FT-1. This behavior is explained by the moisture increase in the specimens subject to FT-2 cycles which caused an ice lens formation in the fine matrix and destruction in the particle matrix. The study also showed that the M_R-stress model recommended by the new mechanistic-empirical pavement design guide for unbound pavement materials could be a statistically good and a reliable predictor of the M_R values of stabilized aggregate bases.

The study evaluated the effect of freezing and thawing actions on stabilized aggregate specimens cured for 28 and 3 days. The results showed that the resilient modulus of 28-day cured specimens increased with increased freezing and thawing cycles (FT-1) up to 12 cycles. After 12 cycles the resilient modulus began to decrease. On the other hand, the resilient modulus of 3-day cured specimens increased with up to 30 FT-1 cycles. It was also found that UCS values exhibited the same trends as the M_R of 28- and 3-day stabilized specimens subject to FT-1 cycles. The FT-1 cycles produced a negative effect on the resilient modulus of 28-day cured specimens and a positive effect on the M_R values of 3-day cured specimens. This effect is explained in terms of the retardation or acceleration of cementitious reactions in a stabilized aggregate specimen. The damage caused by the formation of ice lenses in the pores of stabilized specimens was found to have a negligible effect. The results also showed that the FT-2 cycle procedure has a more detrimental effect than the FT-1 procedure on M_R values. Finally, the ME-PDG M_R-stress model for unbound materials was shown to be a statistically good indicator for predicting the resilient modulus of stabilized materials.

4.1.6 Reconstituted Coal Ash Stabilization of Reclaimed Asphalt Pavement

Osinubi, K. H., and Edeh, J. E. (2011) "Reconstituted Coal Ash Stabilization of Reclaimed Asphalt Pavement." *Proceedings of Geo-Frontiers 2011.*, pp 1172-1181.

A laboratory evaluation of the characteristics of reconstituted coal ash stabilized reclaimed asphalt pavements (RAP) subjected to British Standard light, BSL (standard Proctor) compactive effort to determine the compaction characteristics and California bearing ratio (CBR) values was carried out. Test results show that the properties of RAP improved when treated with reconstituted coal ash, RA (coal fly ash, FA + coal bottom ash, BA). The particle size grading improved from 99.9% coarse aggregates and 0.1% fines for 100% RAP to 93 – 99% coarse aggregate with 1 – 7% fines for the various reconstituted coal ash-RAP mix proportions. The CBR values also improved from 13 and 9% for the unsoaked and soaked conditions respectively, for 100% RAP to 60% and 66% (soaked condition) for 10%RA(40%FA + 60%BA) + 90%RAP and 40%RA(20%FA + 80%BA) + 60%RAP mixes with corresponding unsoaked CBR values of 27 and 34%. Generally, soaked samples recorded higher CBR values than unsoaked samples. The reconstituted coal ash stabilized RAP proportion of 40%RA(20%FA + 80%BA) + 60%RAP and 10%RA(40%FA + 60%BA) + 90%RAP with CBR values of 66 and 60% (soaked for 24 hours) can be used as subbase or subgrade materials in road construction.

The compaction characteristics were affected by the proportions of fly ash and bottom ash in the mixes. The results recorded show that as the coal fly ash proportion of the reconstituted coal ash increased and the coal bottom ash proportion decreased, the MDD also decreased with corresponding increase in the OMC. Usually, a minimum CBR value of over 80% is required for base materials, 30-80% for subbases and 10-30% for subgrade (Nigerian General Specifications, 1997). The reconstituted coal ash stabilized RAP proportion of 40%RA (20%FA + 80%BA) + 60%RAP and 10%(20%FA + 80%BA) + 90%RAP with CBR values of 66 and 60% (soaked for 24 hours) achieved at their respective OMC and MDD can be used as subbase or subgrade materials in road construction according to the recommendation of Nigerian General Specifications (1997).

4.1.7 Stabilization of Organic Soils with Fly Ash

Tastan, E. O., Edil, T. B., Benson, C. H., and Aydilek, A. H. (2010) "Stabilization of Organic Soils with Fly Ash." *J. Geotechnical and Geoenvironmental Eng.*, pp 1-40.

The effectiveness of fly ash use in the stabilization of organic soils and the factors that are likely to affect the degree of stabilization were studied. Unconfined compression and resilient modulus tests were conducted on organic soil-fly ash mixtures as well as untreated soil specimens. Unconfined compressive strength of organic soils can be increased using fly ash, but the amount of increase depends on the type of soil and characteristics of the fly ash. Resilient modulus of the slightly organic and organic soils can also be significantly improved. The increases in strength and stiffness are attributed primarily to cementing caused by pozzolanic reactions, although the reduction in water content resulting from the addition of dry fly ash solid also contribute to strength gain. The pozzolonic effect appears to diminish as the water content decreases. The significant characteristics of fly ash affecting the increase in unconfined compressive strength and resilient modulus include CaO content and CaO/SiO2 ratio (or CaO/(SiO2 + Al2O3) ratio). Soil organic content is a detrimental characteristic for stabilization. Increase in organic content of soil indicates that strength of soil-fly ash mixture decreases exponentially. For most of the soil-fly ash mixtures tested, unconfined compressive strength and resilient modulus increased when fly ash percentage was increased.

The objective of this study was to determine if unconfined compressive strength (q_u) and resilient modulus (M_r) of soft organic soils can be increased by blending fly ash into the soil. Tests were conducted with three organic soils and six fly ashes. Portland cement and an inorganic silt were also used as a stabilizer for reference purposes. Fly ashes were mixed with soils at three different percentages and two different water contents (OWC and 9-15% wet of the OWC). Following conclusions are advanced:

1) Unconfined compressive strength of organic soils can be increased using fly ash, but the amount of increase depends on the type of soil and characteristics of the fly ash. Large increases in q_u (from 30 kPa without fly ash to > 400 kPa with fly ash) were obtained for two clayey soils with an organic content (OC) less than 10% when blended with some of the fly ashes. More modest increases in q_u (from 15 kPa without fly ash to >100 kPa with fly ash) were obtained for a highly organic sandy silty peat having OC = 27%. Resilient modulus tests could not be performed on organic soils without fly ash stabilization at wet conditions since specimens were too soft. The addition of fly ash, at wet conditions, to the slightly organic soils, Lawson and Theresa (OC=5% and 6%, respectively) produced M_r varying between 10 to 100 MPa depending upon the type and percentage of the fly ash. At OWC, M_r for these soils could be improved up to 120 MPa with addition of fly ash. However, for Markey Peat (OC=27%), stabilization with fly ash never produced M_r>30 MPa no matter what fly ash type and percentage (up to 30%), was used.
2) The significant characteristics of fly ash affecting the increase in q_u and M_r include CaO content and CaO/SiO2 ratio (or CaO/(SiO2 + Al2O3) ratio). The highest q_u and M_r were obtained when the CaO content was greater than 10% and the CaO/SiO2 ratio was between 0.5-0.8. Comparable increases in qu and M_r were obtained with the Class C ashes, normally used in concrete applications, and the off-specification fly ashes meeting the aforementioned criteria for CaO content and CaO/SiO2 ratio. However, much lower q_u and M_r were obtained with one off-specification fly ash primarily due to its low CaO content and CaO/SiO2 ratio. Carbon content of the fly ash (i.e., loss on ignition) seemed to have no bearing on the qu and M_r of the soil-fly ash mixtures.
3) For most of the cases q_u and M_r increased when fly ash percentage was increased. Exceptions were mixtures having less reactive Presque Isle and Coal Creek fly ashes (CaO<10% and CaO/SiO2<0.5)
4) The reactivity effect appears to diminish as the water content decreases, i.e, improvement in the q_u of the soil due to addition of fly ash or inorganic silt to the soil was approximately the same for the mixtures prepared at OWC. When the fly ash percentage in the mixture is 10%, expected trend of having higher qu when water content decreases was observed. On the other hand, as the fly ash percentage is increased to 20% (more reduction in water content compared to 10% fly ash case), soil-fly ash mixtures prepared at wet of OWC usually had higher qu than the ones prepared at OWC. The trend of having stronger mixtures at wet conditions as opposed to the ones prepared at optimum water content (OWC) is attributed to requiring more water for hydration reactions of the higher amount of fly ash.
5) Soil organic content is a detrimental characteristic for stabilization. Increase in organic content of soil indicates that strength of soil-fly ash mixture will decrease exponentially. No effect of soil pH and plasticity could be discerned on resilient modulus of the soil stabilized with fly ash. However, more research on the effect of these characteristics is required since variability in pH and plasticity of the soils in this study was not sufficient.
6) Fly ash stabilization of soils at OWC always resulted in higher resilient modulus than at wetter conditions. Resilient modulus can be estimated from unconfined compressive strength using a multiplication factor between 70 and 570. Estimation of resilient

modulus based on static E50 obtained from the unconfined compression test can be made using a multiplication factor in the range of 1.6 to 20, which shows that lower-strain resilient modulus is always higher than high-strain E50.

An effort was made to quantify the percent improvement of the various soils with varying amounts of fly ash added. The initial resilient modulus at zero percent additive needed to be assumed, since no modulus value was provided for unstabilized material. It was decided to use the lowest value of resilient modulus obtained for each soil type tested (where various fly ash types were tested). This initial resilient modulus value is expected to produce conservative results, since the actual value for unstabilized material should be less than the value obtained for a material stabilized with at least 10% fly ash. The values for resilient modulus were estimated from graphs supplied by the cited report. Unified Soil Classification notations for the three soils are Peat, Organic Silt and Organic Silt/Elastic Silt for the Markey Peat, Theresa Soil and Lawson Soil, respectively. Additional soil information is found in the original report.

Table 4.1. Summary of stiffness improvement for Tastan et. al. (2010) study.

Dewey fly ash content	Markey Peat		Theresa Soil		Lawson Soil	
	Resilient modulus (Mpa)	Modulus Factor	Resilient modulus (Mpa)	Modulus Factor	Resilient modulus (Mpa)	Modulus Factor
0%	7	-	7	-	16	-
10%	-	-	30	4.3	17	1.1
20%	10	1.4	52	7.4	88	5.5
30%	20	2.9	108	15.4	100	6.3

King fly ash content	Markey Peat		Theresa Soil		Lawson Soil	
	Resilient modulus (Mpa)	Modulus Factor	Resilient modulus (Mpa)	Modulus Factor	Resilient modulus (Mpa)	Modulus Factor
0%	7	-	7	-	16	-
10%	-	-	16	2.3	16	1.0
20%	18	2.6	70	10.0	36	2.3
30%	8	1.1	97	13.9	107	6.7

Presque Isle fly ash content	Markey Peat		Theresa Soil		Lawson Soil	
	Resilient modulus (Mpa)	Modulus Factor	Resilient modulus (Mpa)	Modulus Factor	Resilient modulus (Mpa)	Modulus Factor
0%	7	-	7	-	16	-
10%	-	-	20	2.9	19	1.2
20%	9	1.3	28	4.0	57	3.6
30%	8	1.1	58	8.3	38	2.4

Coal Creek fly ash content	Markey Peat		Theresa Soil		Lawson Soil	
	Resilient modulus (Mpa)	Modulus Factor	Resilient modulus (Mpa)	Modulus Factor	Resilient modulus (Mpa)	Modulus Factor
0%	7	-	7	-	16	-
10%	-	-	7	1.0	66	4.1
20%	7	1.0	9	1.3	64	4.0
30%	30	4.3	20	2.9	65	4.1

Columbia fly ash content	Markey Peat		Theresa Soil		Lawson Soil	
	Resilient modulus (Mpa)	Modulus Factor	Resilient modulus (Mpa)	Modulus Factor	Resilient modulus (Mpa)	Modulus Factor
0%	7	-	7	-	16	-
10%	-	-	19	2.7	37	2.3
20%	-	-	58	8.3	87	5.4
30%	-	-	60	8.6	106	6.6

Stanton fly ash content	Markey Peat		Theresa Soil		Lawson Soil	
	Resilient modulus (Mpa)	Modulus Factor	Resilient modulus (Mpa)	Modulus Factor	Resilient modulus (Mpa)	Modulus Factor
0%	7	-	7	-	16	-
10%	-	-	18	2.6	48	3.0
20%	-	-	68	9.7	-	-
30%	17	2.4	-	-	109	6.8

Boardman Silt content	Markey Peat		Theresa Soil		Lawson Soil	
	Resilient modulus (Mpa)	Modulus Factor	Resilient modulus (Mpa)	Modulus Factor	Resilient modulus (Mpa)	Modulus Factor
0%	7	-	7	-	16	-
10%	-	-	-	-	22	1.4
20%	-	-	10	1.4	31	1.9
30%	-	-	11	1.6	52	3.3

As noted in the summary table, the modulus factor (taken to be the ratio of the resilient modulus of the stabilized soil to the assumed resilient modulus of the unstabilized soil) varies significantly, ranging from 1 to 10. In other words, the stiffness increases from a negligible amount to as much as ten times, depending on the amount of fly ash mixed into the soil. For the lower fly ash contents (10% by weight), the modulus factor for these soils is approximately 2.3, with values ranging from 1.0 to 4.3. The average factor of improvement for 10% fly ash for these soils is about 2.

4.1.8 Stabilizing Soft Fine-Grained Soils with Fly Ash

Edil, T. B., Acosta, H. A., and Benson, C. H. (2006) "Stabilizing Soft Fine-Grained Soils with Fly Ash." *J. Materials in Civil Eng*., 18(2) 283-294.

The objective of this study was to evaluate the effectiveness of self-cementing fly ashes derived from combustion of subbituminous coal at electric power plants for stabilization of soft fine-grained soils. California bearing ratio (CBR) and resilient modulus (M_r) tests were conducted on mixtures prepared with seven soft fine-grained soils (six inorganic soils and one organic soil) and four fly ashes. The soils were selected to represent a relatively broad range of plasticity, with plasticity indices ranging between 15 and 38. Two of the fly ashes are high quality Class C ashes (per ASTM C 618) that are normally used in Portland cement concrete. The other ashes are off-specification ashes, meaning they do not meet the Class C or Class F criteria in ASTM C 618. Tests were conducted on soils and soil–fly ash mixtures prepared at optimum water content (a standardized condition), 7% wet of optimum water content (representative of the typical in situ condition in Wisconsin), and 9–18% wet of optimum water content (representative of a very wet in situ condition). Addition of fly ash resulted in appreciable increases in the CBR and M_r of the inorganic soils. For water contents 7% wet of optimum, CBRs of the soils alone ranged between 1 and 5. Addition of 10% fly ash resulted in CBRs ranging between 8 and 17 and 18% fly ash resulted in CBRs between 15 and 31. Similarly, M_r of the soil alone ranged between 3 and 15 MPa at 7% wet of optimum, whereas addition of 10% fly ash resulted in M_r between 12 and 60 MPa and 18% fly ash resulted in M_r between 51 and 106 MPa. In contrast, except for one fly ash, addition of fly ash generally had little effect on CBR or M_r of the organic soil.

A laboratory study was conducted where soil–fly ash mixtures were prepared at different fly ash contents (10–30%) to evaluate how addition of fly ash can improve the CBR and resilient modulus (M_r) of wet and soft fine-grained subgrade soils. Specimens were prepared at optimum water content, 7% wet of optimum water content (simulating the in situ condition in Wisconsin), and 9–18% wet of optimum water content (simulating a very wet condition). Based on this investigation, the following observations and conclusions are made:

1. CBR of soil–fly ash mixtures generally increases with fly ash content and decreases with increasing compaction water content. Adding 10 and 18% fly ash to fine-grained soils compacted 7% wet of optimum (the typical in situ condition) resulted in increases in CBR by a factor of 4 and 8, respectively. The CBR increased by a greater factor when fly ash was added to a wetter or more plastic (i.e., poorer) fine-grained soil.
2. Soil–fly ash mixtures prepared with 10% fly ash and fine-grained soil compacted 7% wet of optimum (the typical in situ condition) typically will have lower resilient modulus than soil alone compacted at optimum water content. However, when the fly ash content is on the order of 18%, the resilient modulus typically will be higher (30% higher, in this study) than the resilient modulus of soil alone compacted at optimum water content. Larger increases in resilient modulus typically should be expected for wetter or more plastic fine-grained soils (i.e., poorer subgrades); however, stabilization with fly ash results in comparable final CBR and M_r regardless of soil type.
3. The effect of curing time on resilient modulus was evaluated using one soil and two fly ashes. Between 7 and 14 days, the resilient modulus increased modestly. However,

between 14 and 56 days, the resilient modulus increased by 20–50%. Thus, fly ash stabilized subgrades should stiffen over time, resulting in increased pavement support.

4. The presence of 10% organic matter in one of the soils inhibited stabilization by most of the ashes. Soil–fly ash mixtures prepared with this soil typically had much lower CBR and M_r than obtained for inorganic soils. In some cases, the M_r was not measurable. However, a modest degree of stabilization was achieved for this soil with one of the off-specification fly ashes (a fly ash with high carbon content and a high CaO/SiO2 ratio). The mechanism making the off-specification fly ash effective in stabilizing organic soils needs further study.

4.1.9 Structural Performance Monitoring of an Un-Stabilized Fly Ash-Based Road Sub-Base

Mohanty, S. and Chugh, Y. P. (2006) "Structural Performance Monitoring of an Un-stabilized Fly Ash-Based Road Sub-base." *J. Transportation Eng*., 132(12) 964-969.

Beneficial utilization of stabilized fly ash as a structural layer in pavement construction is a common practice in the United States. At the same time, application of untreated and unstabilized fly ash in such cases is relatively rare. One such application was demonstrated at an Industry Access Truck Route in the state of Illinois. Approximately 60,000 m^3 of F-fly ash, compacted in place, was used to construct the sub-base for the 7.3 m wide and 3.4 km long road. The entire road was built into three control sections, i.e., AC1, fly ash, and AC2. Control Section AC1 was 0.75 km long and was built on fine sand subgrade; the fly ash control section was 2.2 km long and was built on silty/clayey subgrade using fly ash as sub-base; Control Section AC2 was 0.45 km long and was built on clayey subgrade. This paper presents the long-term post-construction structural performance results of the fly ash control section, as compared to Control Sections AC1 and AC2. Falling weight deflectometer studies performed on the truck route have indicated all the sections of the truck route to be performing satisfactorily. The fly ash control section in particular has exhibited very good structural performance.

Post-construction FWD studies performed on the road have indicated that the truck route has performed satisfactorily over the three year post-construction monitoring period. Control Section AC1 has demonstrated the best performance, in terms of back-calculated subgrade moduli and effective structural numbers, followed very closely by the fly ash section. Structural capacity of the pavement is found to be very satisfactory for this type of county roadway. Specifically, the structural performance of the fly ash section has been very good. It is believed that well considered, properly designed and constructed large volume uses of CCBs are appropriate as a natural resource conservation and environmental stewardship mechanism. Large volume utilization of fly ash should be encouraged with the appropriate geotechnical and environmental characterizations of the natural soils and fly ash, and appropriate post-construction performance monitoring.

4.1.10 Time Effect on Shear Strength and Permeability of Fly Ash

Porbaha, A, Pradhan, T. B. S., and Yamane, N. (2000) "Time Effect on Shear Strength and Permeability of Fly Ash." *J. Energy Eng*., 126(1) 15-31.

This paper investigates the effect of time on the shear strength and the permeability of fly ash, a major solid by-product of thermoelectric power plants. Direct shear tests using Mikasa's apparatus, conventional permeability tests, and consolidation tests were conducted on two silt-size fly ashes, with low free lime contents, obtained from two different power plants. The results show that the immediate settling of both fly ashes takes place in a short period of time during consolidation and does not change with time. The rate of increase in shear strength with time is different depending on the pozzolanic reactions taking place for the two ashes. The permeability tests under constant stresses of 49 and 98 kPa for 12 days show that the coefficient of permeability for the tested ashes is between 1026 and 1027 m/s. During this period the coefficient of permeability either remains constant (for the case of the ash with a lower free lime content) or is slightly reduced (for the ash with a higher free lime content). The practical implications and the limitations of using low lime silt-size fly ash in vertical drains in the stabilization of soft ground are also discussed.

In Europe 45% of the 57,500,000 t of coal combustion by-products are utilized, whereas in Japan the statistics show a utilization of 59%. Accordingly, finding new applications for fly ash, a solid by-product of coal combustion in thermoelectric power plants, will bring savings in terms of eliminating the high cost of landfill disposal. This study investigates the change in the shear strength and the permeability of fly ash with time. The direct shear tests using Mikasa's apparatus, a standard consolidometer, and conventional permeability tests were conducted on two silt-size low calcium fly ashes. The following conclusions are drawn from this study:

- An increase in consolidation time from 10 min to 3 days increased the shear strength of both fly ashes studied here. However, the rate of increase was different depending on the pozzolanic reactions taking place for the two ashes, each having a different Ca content.
- The transitional period between softening to hardening in the stress path during shearing is 720–1,390 min (0.5 to about 1 day) for the Hekinan ash and from 1,440 to 4,320 min (1 to 3 days) for the Matsushima ash, which has a lower free lime content.
- The immediate settling of both fly ashes takes place in a short period of time (i.e., <1 min) during consolidation. Secondary settling is small. This is somewhat similar to the behavior of granular materials.
- The permeability tests under a constant stress of 49 and 98 kPa for 12 days show that the coefficient of permeability for the ashes tested here is between 1026 and 1027 m/s. During this period the coefficient of permeability remained constant for the Matsushima ash, whereas it fell slightly for the Hekinan ash, which has a higher lime content. The examination of the microstructure using the SEM reveals the formation of needlelike substances for the Hekinan ash, which may contribute to the reduction of permeability with time.

4.1.11 Using FBC and Stoker Ashes as Roadway Fill: Case Study

Deschamps, R. J., (1998) "Using FBC and Stoker Ashes as Roadway Fill: Case Study." *J. Geotechnical and Geoenvironmental Eng.*, 124(11) 1120-1127.

Approximately 100,000 m^3 of atmospheric fluidized bed combustion (FBC) ash and stoker ash were used as structural fill in the construction of a large roadway embankment. The embankment is ~200 m long and 10 m high, and it supports an extension of a street across a gravel quarry in West Lafayette, Ind. The use of a coal combustion by-product on this project was motivated by the need to find cost-effective alternatives to the disposal of these materials in solid-waste landfills. Similar projects have been completed using the more common bottom ash and fly ash. However, little information is available related to the performance of FBC and stoker ashes in this application. An overview of the project and construction operations is described, and the results of geotechnical laboratory tests and field monitoring are presented. Instruments used in the monitoring of fill behavior include settlement plates, vertical and horizontal inclinometers, seismic cross-hole tests, and post-construction standard penetration tests. The compacted FBC material experienced significant swell and is still expanding 2 years after construction.

The findings of the study are summarized as follows.

1. The FBC and stoker ash embankment was easily built with conventional construction equipment.
2. Large quantities of water were added during construction to satisfy water requirements for the formation of ettringite and gypsum, to increase the efficiency of the compaction process and to control dusting.
3. The properties of the FBC ash and mixtures with stoker ash vary with the degree of weathering before compaction. Material that had access to moisture for some time in the past is less reactive than fresh material. The fresh material produced greater swell pressures and strains and had lower shear wave velocities and lower SPT N values, on average, than the weathered CCBs. In either case, the compacted ash is strong and stiff.
4. The uncompacted CCBs are very erodible while the compacted material is not.
5. Expansion is still taking place two years after construction of the embankment. A gravel surface is being used on the roadway until the swelling decreases sufficiently to construct a paved section.

The primary limitation for use of compacted FBC ash as a roadway fill is the tendency for swelling. The magnitude of swell that occurs will depend on many factors in the coal combustion process including the characteristics of the coal being burned, the amount of limestone added before combustion, and the specifics of the FBC burner. The FBC material used in this project, especially the material compacted fresh, produced large swell stresses and strains as recorded in the laboratory tests and in field monitoring. However, the duration of swelling experienced in the field was not evident from the laboratory test results where swelling was complete in a few days.

Until suitable tests can be devised to adequately estimate the magnitude and duration of swelling for a specific FBC ash under field conditions, the following recommendations for the use of this material as a structural fill are provided.

1. To minimize swell strains, stockpile the material in an area with adequate exposure to moisture for several months prior to use.
2. Use the materials in applications or areas where swell stress and strains will pose no difficulties.
3. Place utilities and other conduits outside the zone of compacted FBC ash.

The use of the FBC ash in the construction of a roadway embankment provided many lessons that would not have been learned from laboratory tests alone. The project also raises questions that may not have been asked regarding the ultimate magnitude and duration of swelling. Overall, the project is viewed as a success because there was flexibility in terms of construction scheduling, and the financial benefits of large volume utilization of the CCBs in construction, when compared to the disposal costs, was substantial.

4.1.12 Use of Fly Ash for Reconstruction for Bituminous Roads

Benson, C., Edil, T., Ebrahimi, A., Kootstra, B., Li, L. and Bloom, P. "Use of Fly Ash for Reconstruction for Bituminous Roads." *MN/RC-2009-27*, Minnesota Department of Transportation, 2009.

Recycling part or all of the pavement materials in an existing road during reconstruction is an attractive construction alternative. When reconstructing roads surfaced with hot mix asphalt (HMA), the HMA, underlying base, and a portion of the existing subgrade often are pulverized to form a new base material referred to as recycled pavement material (RPM). Compacted RPM is overlain with a new HMA layer to create a reconstructed or rehabilitated pavement. This process is often referred to as full-depth reclamation. Similarly, when an unpaved road with a gravel surface is upgraded to a paved road, the existing road surface gravel (RSG) is blended and compacted to form a new base layer that is overlain with an HMA surface. Recycling pavement and road materials in this manner is both cost effective and environmentally friendly. However, recycled base materials may contain asphalt binder, fines, and/or other deleterious materials that can adversely affect strength and stiffness. To address this issue, chemical stabilizing agents can be blended with RPM or RSG. Use of industrial material resources for stabilization (e.g., cementitious coal fly ash) is particularly attractive in the context of sustainability. The purpose of this study was to develop a practical method to design local roadways using stabilized RPM or SRSG as the base layer and Class C fly ash as the stabilizing agent. The design method was developed in the context of the “gravel equivalency” (GE) design methodology employed for local roads in Minnesota.

4.1.13 Volume Change Behavior of Fly Ash-Stabilized Clays

Phanikumar, M. R., and Shamar, R. S. (2005) "Volume Change Behavior of Fly Ash-Stabilized Clays." *J. Materials in Civil Eng*., 19(1) 67-74.

This paper presents, by way of comparison, the effect of fly ash on the volume change of two different types of clay, one a highly plastic expansive clay and the other a nonexpansive clay, also of high plasticity. Expansive clays swell on absorbing water and shrink on drying. Nonexpansive clays undergo large compression at high water contents. The effect of fly ash content on free swell index, swell potential, and swelling pressure of expansive clays was studied. Compression index and secondary consolidation characteristics of both expansive and nonexpansive clays were also determined. Swell potential and swelling pressure, when determined at constant dry unit weight of the sample (mixture), decreased by nearly 50% and, when determined at constant weight of clay, increased by nearly 60% at 20% fly ash content. Compression index and coefficient of secondary consolidation of both the clays decreased by 40% at 20% fly ash content.

Laboratory test data on the effect of fly ash content on the volume change of two expansive clays and a nonexpansive clay are obtained. The variation of free swell index, swell potential, swelling pressure, compression index, and secondary consolidation of the clays with fly ash content is presented. The effect of fly ash on expansive clays and nonexpansive clays is compared.

1. Addition of fly ash to expansive soils reduced their swelling. Free swell index decreased on addition of fly ash, as evidenced by the tests done on two highly swelling clays. Swell potential and swelling pressure also decreased significantly with decreasing fly ash content. For the type of fly ash and expansive clays used, 20% fly ash content reduced FSI, swell potential, and swelling pressure (as determined by the free swell method) by about 50%.
2. The reduction in the swelling characteristics (at constant dry unit weight of blend) is, basically, by replacement of plastic fines of clay by nonplastic fines of fly ash. The reduction in swelling can be attributed to the flocculation and cementation effects developed on addition of fly ash. Based on theoretical considerations it can be demonstrated that suction would be reduced in fly ash-blended expansive clay samples and consequently swelling would be reduced. When testing was done keeping the weight of the clay constant, swell potential and swelling pressure increased with increasing % fly ash. As the weight is kept constant, the dry unit weight of the blend increased with increasing fly ash content, thereby increasing the dry unit weight of the expansive clay also, resulting in consequential increase in swelling. Hence, even though the amount of nonplastic fines of fly ash is increased, the swell potential and swelling pressure increase when the weight of the clay is kept constant.
3. Maximum dry unit weight increased and optimum moisture content decreased with increasing fly ash content. At 20%, fly ash content maximum dry unit weight of nonexpansive clay increased by about 7%, and optimum moisture content decreased by about 15%. The compression index of both the expansive and nonexpansive clays decreased by about 50% at 20% fly ash content, indicating that addition of fly ash reduced compressibility characteristics of both expansive and nonexpansive clays. However, the effect of fly ash is more pronounced on the compressibility behavior of expansive clays.
4. Secondary consolidation characteristics of fly ash-blended clays also showed improvement in comparison to those of untreated clays. The volume change due to creep and slippage of particles after the end of primary consolidation was better resisted by clay blended with fly ash. The time required for the end of primary consolidation and the beginning of secondary consolidation was shortened in clays blended with fly ash. This means that the amount of settlement of structures built on fly ash-amended expansive clays decreases and the rate of settlement increases reducing the time required for reaching the final settlement.

4.2 Cement

4.2.1 Durability of Cement Stabilized Low Plasticity Soils

Zang, Z., and Tao, M. (2007) "Durability of Cement Stabilized Low Plasticity Soils." *J. Geotechnical and Geoenvironmental Eng.*, 134(2) 203-213.

Three testing methods for predicting the durability of cement-stabilized soils—the tube suction (TS), 7-day unconfined compression strength (UCS), and wetting–drying durability tests—were tested and compared for their correlations and influence factors using a problematic low plastic silt clay from subgrade commonly encountered in Louisiana. A series of samples was molded at six different cement dosages (2.5, 4.5, 6.5, 8.5, 10.5, and 12.5% by dry weight of the soil) and four different molding moisture contents (15.5, 18.5, 21.5, and 24.5%). The test results indicate that the water–cement ratio of cement-stabilized soil had the dominant influence on the maximum dielectric value (DV), 7-day UCS, and durability of stabilized samples tested, although the dry unit weight of cement-stabilized soil could cause the variation of the results. This study confirms that TS, 7-day UCS, and wetting-drying durability tests are equivalent in predicting durability, and tentative charts to ensuring the durability of cement-stabilized low plasticity soils are developed using their 7-day UCS or the maximum DV values.

Many state highway agencies are replacing wetting–drying durability tests with 7-day UCS tests and trying to use the reduced 7-day UCS criterion (1.03 MPa or 150 psi) with increased layer thickness for mix design of cement-stabilized cohesive soils to reduce reflection cracking. On the other hand, the tube suction test shows promise as an alternative to the regular soil durability test, although the published works are mainly confined to coarse soils. This paper reports some interesting information about the durability prediction based on 7-day UCS, DV, and wetting drying tests of fine-cohesive soils stabilized with cement. Detailed laboratory tests included the tube suction, 7-day UCS, and wetting–drying durability tests on the soil samples stabilized with six different cement contents at four different molding moisture contents. The test results indicate that the water–cement ratio of cement-stabilized soil had the significant influence on the maximum DV, 7-day UCS, and durability of stabilized samples although the dry unit weight of cement-stabilized soil could cause the variation of the results. Good correlations exist among the durability prediction based on the maximum DV, 7-day UCS, and soil–cement mass loss. Therefore, there is the equivalency of TS, 7-day UCS, and wetting–drying durability tests regarding the water susceptibility for cement stabilized soils. Although the prediction charts developed in this paper for passing the durability requirement were only based on the results of one CL soil, they are the start point to develop statistically sound prediction charts using the 7-day UCS or maximum DV. Different sources of CL soils should be tested and plotted in the charts with their durability and the 7-day UCS or maximum DV to make these two charts usable for future design and construction.

For this study, the values for unconfined compressive strength (UCS) were estimated from the provided graph. In order to assure a conservative strength increasing factor, the initial soil UCS was assumed to be equal to the value given at 2.5% cement content. As expected, the strength of the soil is observed to increase with the addition of cement. It is stated that the optimum moisture content for the lean clay evaluated in this study is 18.5%.

Table 4.2. Summary of strength improvement for Zang and Tao (2007) study.

Cement Content	%w = 15.5 UCS (qu) (MPa)	%w = 15.5 Strength Increasin	%w = 18.5 UCS (qu) (MPa)	%w = 18.5 Strength Increasin	%w = 21.5 UCS (qu) (MPa)	%w = 21.5 Strength Increasin	%w = 24.5 UCS (qu) (MPa)	%w = 24.5 Strength Increasin
0.0%	0.5	-	0.35	-	0.25	-	0.22	-
2.5%	0.5	1.0	0.35	1.0	0.25	1.0	0.22	1.0
4.5%	1.25	2.5	1	2.9	0.65	2.6	0.65	3.0
6.5%	1.75	3.5	1.4	4.0	1.1	4.4	1.1	5.0
8.5%	2.5	5.0	1.9	5.4	1.4	5.6	1	4.5
10.5%	3.4	6.8	2.3	6.6	1.5	6.0	1.25	5.7
12.5%	4	8.0	2.8	8.0	1.7	6.8	1.5	6.8

Since the strength of the unstabilized soil was not provided, the strength factor was taken to be the ratio of the strength of the soil at the measured cement content to the strength of the soil at 2.5% cement content. The factor of improvement (for strength) for the soil was between 2.5 and 3.0 for the water contents tested and a cement content of 4.5%, such that the average factor of strength improvement was approximately 2.75 for a cement content of 4.5%.

4.2.2 Field Testing of Stabilized Soil

Janoo, V. C., Firicano, A. J., Barna, L. A., and Orchino, S. A. (1999) "Field Testing of Stabilized Soil." *J. of Cold Regions Eng*., 13(1) 37-53.

Remediation of a Superfund site in Stratford, Conn., involved stabilization of the subgrade with Portland cement. Part of the remediation site was to be used as a parking area. The stabilized soil was to be covered with natural base/subbase course materials and capped with an asphalt concrete cover. During the course of the remediation, a base-course layer could not be placed prior to the onset of winter. A field study was conducted to quantify any changes in the mechanical properties of the open stabilized subgrade subjected to freeze-thaw cycling during the winter of 1996 – 97. Field evaluation was conducted with pavement industry tools: the Clegg impact hammer and the dynamic cone penetrometer. Evaluation results show the viability of the Clegg hammer as an instrument for quality assurance and also show that there can be up to 50% loss in compressive strength of the subgrade within the uppermost layer of the material caused by freeze-thaw cycling.

Based on the Clegg hammer and DCP test results, the mean strength of the stabilized areas was reduced by approximately 50% during the freezing season of 1996–97. However, based on the NED minimum requirement of 207 kPa unconfined compressive strength, it was found that approximately one-half of the data from site 1 from March fell below the 207 kPa limit based on results from the Clegg hammer tests. The findings from the DCP data show that the mean strength was below 207 kPa in approximately the top 50 mm of the structure in the testing areas. Consideration was given to the findings from this field study, as well as minimum strength criteria, equipment limitations, and the presence of debris within the soil to determine the extent of restabilizing the material.

Based on the temperature data measured at the site, frost penetration for the 1996–97 freezing season was approximately 500 mm. The design thickness of 910 mm base cover is sufficient to prevent frost penetration into the stabilized waste fill.

4.2.3 Key Parameters for Strength Control of Artificially Cemented Soils

Consoli, N. C., Fappa, D., Festugato, L., and Deineck, K. S. (2007) "Key Parameters for Strength Control of Artificially Cemented Soils." *J. Geotechnical and Geoenvironmental Eng*., 133(2) 197-205.

Often, the use of traditional techniques in geotechnical engineering faces obstacles of economical and environmental nature. The addition of cement becomes an attractive technique when the project requires improvement of the local soil. The treatment of soils with cement finds application, for instance, in the construction of pavement base layers, in slope protection of earth dams, and as a support layer for shallow foundations. However, there are no dosage methodologies based on rational criteria as exist in the case of the concrete technology, where the water/cement ratio plays a fundamental role in the assessment of the target strength. This study therefore aims to quantify the influence of the amount of cement, the porosity and the moisture content on the strength of a sandy soil artificially cemented, as well as to evaluate the use of a water/cement ratio and a voids/cement ratio to assess its unconfined compression strength. A number of unconfined compression tests, triaxial compression tests, and measurements of matric suction were carried out. The results show that the unconfined compression strength increased linearly with the increase in the cement content and exponentially with the reduction in porosity of the compacted mixture. The change in moisture content also has a marked effect on the unconfined compression strength of mixtures compacted at the same dry density. It was shown that, for the soil-cement mixture in an unsaturated state (which is usual for compacted fills), the water/cement ratio is not a good parameter for the assessment of unconfined compression strength. In contrast, the voids/cement ratio, defined as the ratio between the porosity of the compacted mixture and the volumetric cement content, is demonstrated to be the most appropriate parameter to assess the unconfined compression strength of the soil-cement mixture studied.

From the data presented in this paper, and bearing in mind the limitations of this study, the following conclusions can be drawn:

- The addition of cement, even in small amounts, greatly improves the soil strength. For the cement contents studied here, the unconfined compression strength increased approximately linearly with an increase in the cement content. The rate of strength gain, represented by the gradient of the fitted curves, increased with an increase in the dry density of the compacted soil cement, indicating that the effectiveness of the cement is greater in more compacted mixtures;
- The reduction in the porosity of the compacted mixture greatly improves the strength. It was shown that the unconfined compression strength increased approximately exponentially with a reduction in the porosity of the compacted mixture;
- For a given dry density, the variation in moisture content affected the unconfined compression strength of the soil cement. Generally, an increase in strength is observed with increasing moisture content until a maximum value is reached, after which the

strength decreases. It appears that this effect of moisture content varies with the cement content;

- It was found that there is no relationship between the unconfined compression strength and the water/cement ratio for the material studied; and
- The voids/cement ratio, defined by the porosity of the compacted mixture divided by the volumetric cement content, adjusted by an exponent [0.28 for the soil and cement used in this research] has been shown to be a more appropriate parameter to evaluate the unconfined compression strength of the soil-cement mixture studied. It is probable that this exponent will be a function of the materials soil and cement used.

With a soil classification of SM (USCS) or A-4 (AASHTO), the unconfined compression strength was calculated through lab testing at cement concentrations of 1, 2, 3, 5, 6, and 7% at four different dry unit weights. A best fit line was plotted for each unit weight, and with the given line equation, the unconfined compression strength at 0% cement was assumed to be the y-intercept for cement content C=0%. The strength was observed to increase with the increase in cement content. Table 4.3 summarized the impact of the cement content on the soil strength.

Table 4.3. Summary of strength improvement for Consoli et. al. (2007) study.

Soil Classification		ϒd = 17.3 kN/m^3			ϒd = 18.0 kN/m^3			ϒd = 19.0 kN/m^3			ϒd = 19.7 kN/m^3		
USCS	AASHTO	% cement	UCS (qu) (kPa)	Strength Increasing Factor	% cement	UCS (qu) (kPa)	Strength Increasing Factor	% cement	UCS (qu) (kPa)	Strength Increasing Factor	% cement	UCS (qu) (kPa)	Strength Increasing Factor
SM	A-4	0	72	-	0	121	-	0	129	-	0	206	-
SILTY SAND, POORLY GRADED SAND-SILT MIXTURES	SILTY SOILS, FAIR TO POOR	1	242	3.36	1	355	2.93	1	474	3.67	1	628	3.05
		2	412	5.72	2	589	4.87	2	819	6.35	2	1050	5.10
		3	582	8.08	3	823	6.80	3	1164	9.02	3	1472	7.15
		4	752	10.44	4	1057	8.74	4	1509	11.70	4	1894	9.19
		5	922	12.81	5	1291	10.67	5	1854	14.37	5	2316	11.24
		6	1092	15.17	6	1525	12.60	6	2199	17.05	6	2738	13.29
		7	1262	17.53	7	1759	14.54	7	2544	19.72	7	3160	15.34

As seen in the table, the strength factors for the soil (taken to be the ratio of the stabilized strength to the unstabilized strength) varied significantly. For low cement contents (from 1% to 3%) the factor of improvement ranged from 3 to 8, with an average value of 5.5. For a cement content of 2%, the factor of improvement for this soil would be approximately 5.

4.2.4 Modeling of Moisture Loss in Cementitiously Stabilized Pavement Materials

Kodikara, J. and Chakrabarti, S. (2004) "Modeling of Moisture Loss in Cementitiously stabilized Pavement Materials." *J. of Geomechanics,* 5(4) 295-303.

The paper presents a theoretical and experimental approach for the modeling of moisture loss during the drying of cementitiously stabilized pavement materials containing varying contents of fine-grained soil. The process of moisture loss was characterized by the isotropic nonlinear diffusion theory. Laboratory tests were undertaken using general purpose Portland cement and two binders comprising industrial waste products. Measurement of material characteristics included the coefficient of moisture diffusivity and the humidity isotherm. Locally available basaltic crushed rocks and clay were respectively used as the host pavement material and fine-grained soil. Independent laboratory tests were undertaken to validate the adopted theoretical approach, which showed close agreement between the experimental and predicted results. The laboratory results indicated that moisture loss decreased with the inclusion of clay soil within the mix. As the drying progressed, the rate of moisture loss became slower, which can be explained by the reduction in the coefficient of moisture diffusivity with the decrease of moisture content.

Binders used in these experiments include general purpose Portland cement, general blended cement, and a blend including alkali activated slag.

This paper presented laboratory and theoretical studies on the moisture transfer within mixtures of crushed rock and clay stabilized with three cementitious binders. The work is relevant to ground stabilization, particularly to pavement stabilization. The laboratory drying studies indicated that the rate of moisture loss decreased as the drying time increased, finally approaching values that are in equilibrium with the drying environment. The decrease of rate of moisture loss can be explained by the reduction in moisture diffusivity with the water content. The moisture loss process may be characterized by isotropic nonlinear diffusion theory, as was validated in the light of laboratory experimental results. The work reported herein is useful for modeling of drying and associated drying shrinkage for stabilized materials, particularly for road pavement materials, subjected to field conditions. Further research, however, is necessary to extend the theory to incorporate environmental fluctuations that prevail under the field conditions.

4.2.5 Parameters Controlling Tensile and Compressive Strength of Artificially Cemented Sand

Consoli, N. C., Cruz, R. C., Floss, M. F., and Festugato, L. (2010) "Parameters Controlling Tensile and Compressive Strength of Artificially Cemented Sand ." *J. Geotechnical and Geoenvironmental Eng.*, 136(5) 759-763.

The enhancement of local soils with cement for the construction of stabilized pavement bases, canal lining, and support layer for shallow foundations shows great economical and environmental advantages, avoiding the use of borrow materials from elsewhere, as well as the need of a spoil area. The present research aims to quantify the influence of the amount of cement, the porosity, and the voids/cement ratio in the assessment of unconfined compressive strength (q_u) and splitting tensile strength (q_t) of an artificially cemented sand, as well as in the evaluation of q_t / q_u relationship. A program of splitting tensile tests and unconfined compression tests considering three distinct voids ratio and seven cement contents, varying from 1 to 12%, was carried out in this study. The results show that a power function adapts well q_t and q_u values with increasing cement content and with reducing porosity of the compacted mixture. The voids/cement ratio is demonstrated to be an appropriate parameter to assess both q_t and q_u of the sand-cement mixture studied. Finally, the q_t / q_u relationship is unique for the sand-cement studied, being independent of the voids/cement ratio.

From the data presented in this technical note, the following conclusions can be drawn:

- A power function adapts well to both q_t −C and q_u −C sand-cement mixture relations;
- The reduction in porosity of the compacted mixture increases both the tensile and compressive strengths;
- The voids/cement ratio (η/Cv) has been shown to be an appropriate index parameter to evaluate both splitting tensile (q_t) and unconfined compressive (q_u) strength of sand-cement mixtures. Both q_t and q_u reduce with increasing η/Cv values; and

- The q_t / q_u ratio is a scalar (0.15) for the sand-cement mixture evaluated in this study, being independent of voids/ cement ratio. As a consequence, dosage methodologies based on rational criteria can concentrate either on desired tensile or compressive strengths.
  - η -- porosity
  - C_v -- volumetric cement content
  - q_t -- splitting tensile
  - q_u -- unconfined compressive strength

The values for unconfined compressive strength (UCS) were estimated from the data in the reference. In order to assure a conservative strength increasing factor, the initial soil UCS was assumed to be equal to the value given at 1% cement content. The strength of the soil is observed to increase with the addition of cement.

No soil classification was supplied, although a soil description was provided in the cited report and is described as follows; "The Osorio sand used in the testing was obtained from the region of Porto Alegre, in Southern Brazil, being classified (ASTM 1993) as nonplastic uniform fine sand with rounded particle shape and specific gravity of the solids 2.65. Mineralogical analysis showed that sand particles are predominantly quartz. The grain size is purely fine sand with a mean effective diameter (D50) of 0.16 mm, being the uniformity and curvature coefficients of 1.9 and 1.2, respectively. The minimum and maximum void ratios are 0.6 and 0.9, respectively."

Table 4.4. Summary of stiffness improvement for Consoli et. al. (2010) study.

	Void Ratio = 0.64		Void Ratio = 0.70		Void Ratio = 0.78	
Cement Content	UCS (qu) (kPa)	Strength Increasing Factor	UCS (qu) (kPa)	Strength Increasing Factor	UCS (qu) (kPa)	Strength Increasing Factor
0%	93	-	122	-	130	-
1%	93	1.0	122	1.0	130	1.0
2%	187	2.0	250	2.0	335	2.6
3%	350	3.8	415	3.4	500	3.8
5%	560	6.0	665	5.5	836	6.4
7%	840	9.0	1000	8.2	1250	9.6
9%	1160	12.5	1330	10.9	1660	12.8
12%	1710	18.4	1920	15.7	1710	13.2

As seen in the table, for low cement contents (2%), the factor of improvement with respect to the strength of the soil at 1% cement content varied from 2.0 to 2.6 for the range of soil densities (void ratios) addressed. A factor of improvement for strength of 2 would be justified for 2% cement for this soil.

4.2.6 Physicochemical and Engineering Behavior of Cement Treated Clays

Chew, S. H., Kamruzzaman, A. H. M., and Lee, F. H. (2003) "Physicochemical and Engineering Behavior of Cement Treated Clays." *J. Geotechnical and Geoenvironmental Eng.*, 130(7) 696-706.

This paper examines the relationship between the microstructure and engineering properties of cement-treated marine clay. The microstructure was investigated using x-ray diffraction, scanning electron microscopy, pH measurement, mercury intrusion porosimetry, and laser diffractometric measurement of the particle size distribution. The engineering properties that were measured include the water content, void ratio, Atterberg limit, permeability, and unconfined compressive strength. The results indicate that the multitude of changes in the properties and behavior of cement-treated marine clay can be explained by interaction of four underlying microstructural mechanisms. These mechanisms are the production of hydrated lime by the hydration reaction which causes flocculation of the illite clay particles, preferential attack of the calcium ions on kaolinite rather than on illite in the pozzolanic reaction, surface deposition and shallow infilling by cementitious products on clay clusters, as well as the presence of water trapped within the clay clusters.

The foregoing discussion shows that the various facets of the behavior of cement-treated soft clay can be explained by the interplay of a few underlying mechanisms. The mixing of cement slurry into Singapore marine clay leads to an immediate increase in water content arising from the additional water in the slurry. Hydration reaction of the cement follows shortly, leading to a decrease in water content and the production of primary cementitious products as well as hydrated lime. The calcium ions released by the hydrated lime give rise to a flocculated clay structure comprising clay clusters separated by large intracluster voids. The flocculation of the clay particles also causes water to be trapped within the cluster. The increase in effective size of the particles and the presence of entrapped water lead to a rise in the plastic and liquid limits. The presence of large intracluster voids also leads to an increase in the permeability of the soil.

The rapid hydration reaction is accompanied by the much slower pozzolanic reaction over time. The findings of this study indicate that, in the pozzolanic reaction, kaolinite is preferentially attacked in comparison to illite. For cement content above 10%, this leads to virtually complete destruction of the kaolinite. The secondary cementitious products appear to be deposited on or near the surfaces of the clay clusters. This gives rise to a reduction in entrance pore diameter but an increase in particle size. This, in turn, leads to a reduction in permeability over time as indicated in the permeability of 7 days and 28 days of curing. The continued increase in particle size leads to an increase in the plastic limit over time with increase in cement content. The deposition of secondary cementitious products on the clay clusters, on the other hand, leads to a decrease in surface activity of the illite clusters. As a result, the liquid limit decreases over time and at higher cement content.

The results of this study also show that, at cement content above ~10%, the amount of pozzolanic reaction stabilizes to a limiting level. This is supported by the changes in cementitious product, water content as well as unconfined compressive strength. The pH measurements show that this stabilization is not brought about by exhaustion of the lime but rather by exhaustion of the kaolinite. This is rather surprising since illite is also pozzolanic. One possible explanation for this comes from Eades and Grim (1960) who found that a pozzolanic reaction that involves illite is much slower than that with kaolinite, and requires much higher cement content to become initiated. This explains why kaolinite is preferentially attacked. As secondary cementitious products from the kaolinite driven pozzolanic reaction are deposited onto the illite cluster surface, the clusters gradually become encapsulated by the cementitious products. This encapsulation protects the illite from further attack by the lime and no further pozzolanic reaction can occur. Whatever the cause of the limited pozzolanic reaction, it seems clear that, in such instances, cement may be a more effective stabilizing agent than lime since the effectiveness of the latter would be hindered by the lack of pozzolanic clay minerals.

The stiffness values were approximated from the visual slope quantities associated with the stress versus strain graph provided in the reference. The stiffness increasing factor was then calculated from the estimated stiffness values. The strength values were approximated from data points on the graph provided in the reference. The strength increasing factor was then calculated.

Table 4.5. Summary of stiffness improvement for Chew et. al. (2003) study.

Soil Classification		% cement	Stiffness (kPa)	Stiffness Increasing Factor
USCS	AASHTO			
CH	A-7-5	0	21	-
Inorganic clays of high plasticity, fat clays	Clayey soils, fair to poor	5	85	4.05
		10	299	14.24
		20	299	14.24
		30	619	29.48
		50	1091	51.95

Table 4.6. Summary of strength improvement for Chew et. al. (2003) study.

Soil Classification		wi = 120% & curing time = 7 days			wi = 90% & curing time = 7			wi = 120% & curing time = 28			wi = 90% & curing time = 28 days		
USCS	AASHTO	% cement	UCS (qu) (kPa)	Strength Factor	% cement	UCS (qu) (kPa)	Strength Factor	% cement	UCS (qu) (kPa)	Strength Factor	% cement	UCS (qu) (kPa)	Strength Factor
CH	A-7-5	0	21	-	0	21	-	0	21	-	0	21	-
Inorganic clays of high plasticity, fat clays	Clayey soils, fair to poor	5	33	1.57	5	48	2.29	5	64	3.05	5	80	3.81
		10	80	3.81	10	100	4.76	10	155	7.38	10	235	11.19
		20	160	7.62	20	200	9.52	20	310	14.76	20	400	19.05
		30	245	11.67	30	310	14.76	30	475	22.62	30	620	29.52
		40	390	18.57	40	455	21.67	40	725	34.52	40	840	40.00
		50	520	24.76	50	625	29.76	50	800	38.10	50	1040	49.52
wi = initial water content		60	-	-	60	-	-	60	825	39.29	60	1060	50.48

As seen in the first table, a stiffness factor of 4 would apply to this soil for a 5% cement content. As per the second table, a long-term strength factor of between 3 and 4 would apply to a 5% cement content. This was one of very few references where both stiffness and strength improvement could be evaluated.

4.2.7 Physical and Chemical Behavior of Four Cement-Treated Aggregates

Davis, K. A., Warr, L. S., Burns, S. E., and Hoppe, E. J. (2006) "Physical and Chemical Behavior of Four Cement-Treated Aggregates." *J. Materials in Civil Eng.*, 19(10) 891-897.

Cement-treated aggregate (CTA) is commonly used to provide a stable base for pavements that are placed over weak soil subgrades. Because CTA reduces the thickness of the aggregate required to provide a durable base by approximately one-half, using it as a bearing layer for pavement can limit the quantity of unsuitable soil that must be excavated and removed, and can reduce the erodability of the stabilized soils. However, the field performance of CTA is variable, even when prepared according to set standards. This laboratory based investigation explored the effects of fines content, cement content, mineralogy, pH, and freeze/thaw cycling on the unconfined compressive strength of cement-treated aggregate. The mineralogy of the base aggregate was found to make a significant difference in the strength of the CTA, with strength increasing in the following order: mica, limestone, and diabase. The granite aggregate yielded variable results, but the strengths were generally on the order of those determined for the diabase aggregate. The pH of the samples also correlated well, with the measured strengths increasing as the pH increased. As was anticipated, increasing the cement content increased the measured unconfined compressive strength of cylinders that were not subjected to freeze/thaw cycling. The same basic trend was observed in cylinders that were subjected to freeze/thaw cycling; however, the increase was less pronounced in the cylinders that were subjected to physical abrasion during thaw cycles. The fines content did not significantly influence the unconfined compressive strength of the cylinders that were not subjected to freeze/thaw cycling; however, the fines content appeared to confer a protective effect to the durability of the cylinders that were subjected to freeze/thaw. For the freeze/thaw test conditions (with and without physical abrasion), the unconfined compressive strength increased as the fines content was increased. The factor of improvement of the unconfined compressive strength was also observed to change by a factor of two or more with an increase in cement content of only 1 or 2 percent.

Aggregate was obtained from four quarries commonly used as source material for CTA by the Virginia Department of Transportation (VDOT). The quarries were located in Dale, Virginia; Lynchburg, Virginia; Manassas, Virginia; and Skippers, Virginia, and will be referred to as Dale, Lynchburg, Manassas, and Skippers, respectively. The primary mineral constituent of the aggregates from each quarry were: mica (Dale), limestone (Lynchburg), diabase (Manassas), and granite (Skippers). Aggregate was delivered to the Virginia Transportation Research Council (Charlottesville, Va.) in 22.7 kg (50 lb) bags, which were then sieved with a number 200 mesh sieve to remove the fines. Fines were added back into the samples at 4, 7, 10, and 14% by weight in order to control the proportion of fine material in the samples as a variable during the testing program. Test specimens were then prepared with Type I Portland cement at contents of 3, 4, 5, and 6% by weight, at each fines content, yielding 16 test conditions for the aggregate from each quarry (Table 1). Fines were added only to the same aggregate from which they were screened; that is, the coarse and fine aggregates from different quarries were not mixed. Three samples were prepared for each test condition, and the data were averaged for analysis. The ranges for the fines and cement contents in the testing program were chosen to encompass the quantities used in current VDOT standards.

Chemical characterization of the aggregates included x-ray diffraction (XRD) for the identification of the predominant mineral phases and measurement of aggregate pH. X-ray diffraction samples were disaggregated and powdered using a SPEX 8000 tungsten carbide ball mill, and backpacked sample mounts of the fine powder were used for XRD analysis. X-ray diffraction patterns were generated using Cu $K\alpha$ radiation on a PanAlytical, Theta-Theta X-ray diffractometer, using a spinner sample stage operating at 45 kV and 40 mA, between 5 and 75° (2θ) at a step size of 0.0330°. X'Pert High Score search/match software was used for sample phase identification. X-ray diffraction analysis was performed at James Madison University (Harrisonburg, Va.). The pH of the aggregate samples was determined using EPA method 9045C (SW-846) (Lancaster Laboratories, Lancaster, Pa.).

Physical characterization tests for the four soil samples included a particle-size analysis performed according to ASTM D 422 and determination of the liquid limit for the fine materials according to ASTM D 4318. Standard Proctor compaction tests were conducted according to ASTM D 698 and unconsolidated soil compression tests were conducted according to ASTM D 1633. Freeze–thaw testing was designed to test a specimen's endurance and strength under simulated temperature cycling and was performed according to ASTM D 560.

Optimum moisture content for each test condition was determined according to standard Proctor (ASTM D 698), and ranged between 5.2 and 12.5% for the 64 combinations tested (16 test variations times four quarries). Unconfined compression test specimens were then compacted at optimum moisture content, in three layers with 25 blows per layer using an automatic soil compactor. Specimens were extruded with a hydraulic jack, and cured in a moisture room at 100% relative humidity and 20°C for seven days. Specimens were soaked in tap water for 4 h prior to measurement of the unconfined compressive strength, according to ASTM D 1633. Compression testing was performed at a loading rate of approximately 64 kPa/ s (10 psi/ s), at the lower acceptable bound specified in the standard.

The Manassas aggregate was chosen to quantify the effect of freeze–thaw cycling on the compressive strength of the CTA. Freeze–thaw testing was designed to test a specimen's endurance and strength under simulated temperature cycling. Each cylinder was made identically to those of the compression cylinders and then stored in the moist cure room for seven days. For freeze–thaw testing, two separate cylinders were tested.

This laboratory-based investigation explored the effects of fines content, cement content, mineralogy, pH, and freeze/thaw cycling on the unconfined compressive strength of cement-treated aggregate. The mineralogy of the base aggregate was found to make a significant difference in the strength of the CTA, with strength increasing in the following order: mica (Dale), limestone (Lynchburg), and diabase (Manassas). The Skippers aggregate, composed primarily of granite, yielded variable results, but the strengths were generally on the order of those determined for the Manassas diabase aggregate. The pH of the samples also correlated well with the measured strengths increasing as the pH increased: mica (Dale pH=8.9), limestone (Lynchburg pH=9.0), and diabase (Manassas pH=9.3). Skippers granite, with strengths on the order of those measured for Manassas, had a pH=9.2. As was anticipated, increasing the cement content increased the measured unconfined compressive strength of cylinders that were not subjected to freeze/thaw cycling. The same basic trend was observed in cylinders that were subjected to freeze/thaw cycling; however, the increase was less pronounced in the cylinders that were subjected to physical abrasion. The fines content did not significantly influence the unconfined compressive strength of the cylinders that were not subjected to freeze/thaw cycling; however, the fines content appeared to confer a protective effect to the durability of the cylinders that were subjected to freeze/thaw. For both freeze/thaw test conditions (with and without physical abrasion), the unconfined compressive strength increased as the fines content was increased, with strengths between 50 and 100% greater as the fines content was increased.

The soils were classified through the use of the grain size distribution graph, which was supplied in the report. There was no initial strength supplied for any of the raw soils, so it was decided to use the lowest strength value of each soil at 3% cement content as the raw soil strength. This is assumed to be a conservative initial value for each soil. For low cement contents (up to 6%), a strength factor ranging from 1.0 to 2.5 was common to the various soils.

Table 4.7. Summary of strength improvement for Davis et. al. (2006) study.

		Dale		Lynchburg		Manassas		Skippers	
		USCS	AASHTO	USCS	AASHTO	USCS	AASHTO	USCS	AASHTO
		GW	A-1-a	GW	A-1-a	GW	A-1-a	SW	A-1-a
Fines (%)	Cement content (%)	Strength (psi)	Strength Factor	Strength (psi)	Strength Factor	Strength (psi)	Strength Factor	Strength (psi)	Strength Factor
4	0	141	-	141	-	290	-	300	-
	3	141	1.00	141	1.00	294	1.01	338	1.13
	4	205	1.45	205	1.45	453	1.56	182	0.61
	5	282	2.00	282	2.00	476	1.64	402	1.34
	6	273	1.94	273	1.94	644	2.22	623	2.08
7	0	141	-	141	-	290	-	300	-
	3	160	1.13	160	1.13	293	1.01	300	1.00
	4	131	0.93	131	0.93	492	1.70	403	1.34
	5	180	1.28	180	1.28	508	1.75	350	1.17
	6	238	1.69	238	1.69	706	2.43	479	1.60
10	0	141	-	141	-	290	-	300	-
	3	158	1.12	158	1.12	290	1.00	344	1.15
	4	181	1.28	181	1.28	411	1.42	443	1.48
	5	260	1.84	260	1.84	561	1.93	676	2.25
	6	345	2.45	345	2.45	630	2.17	815	2.72
14	0	141	-	141	-	290	-	300	-
	3	189	1.34	189	1.34	345	1.19	343	1.14
	4	208	1.48	208	1.48	516	1.78	764	2.55
	5	262	1.86	262	1.86	663	2.29	789	2.63
	6	335	2.38	335	2.38	695	2.40	1000	3.33

4.2.8 Results from a Forensic Investigation of a Failed Cement Treated Base

Chen, D-H., Schullion, T., Lee, T-C., and Bilyeu, J. (2007) "Results from a Forensic Investigation of a Failed Cement Treated Base." *J. of Performance of Constructed Facilities,* 22(3) 143-153.

After only 2 months in service, the frontage road of US 290 developed a series of depressions that caused a very poor ride. The main cause of the premature failure was attributed to disintegration of the cement treated base (CTB) layer. This was attributed to two primary factors: (1) a very coarse gradation of the aggregate used in the CTB layer which produced a mix that was prone to segregation during placement; and (2) the CTB layer was placed in two lifts, which were not well bonded together. Another contributing factor was the lack of bond between the CTB and the hot mix asphalt (HMA) surface layer. Secondary factors include high air voids in the HMA layer and low HMA layer thickness. The material, when prepared carefully in the lab at the design cement content, passed the strength requirement of 2.07 MPa. But this coarse mix appears to have been difficult to place correctly in the field. The coarsely graded aggregate used on this project appears to be prone to segregation, either during placement or compaction. The ground penetration radar results (with confirmation by core samples) indicated that most of the problems were at the bottom of the upper CTB lift. The CTB was placed in two lifts and very poor condition was found between the two CTB layers. This problem was coupled with a thin, porous, and poorly bonded HMA layer that permitted moisture to enter the CTB layer. Similar failures have also been reported recently on other CTB projects in Houston.

Nondestructive testing and coring was performed in August 2006. Nondestructive testing consisted of ground penetration radar (GPR), falling weight deflectometer (FWD), and dynamic cone penetrometer (DCP).

The premature failure of the frontage road on US 290 was a result of heavy traffic loads applied to defective pavement layers. Pavement layer defects consisted of: (1) voids in the CTB, segregation of coarse aggregate, and lack of bond between the two CTB lifts; and (2) high air voids and low density in the HMAC surface throughout the project, which allowed moisture to enter the base layer. The writers believe that when there is a significant portion of large aggregates and an absence of fine material, segregation during placement is likely. Also, CTB requires sufficient fines to establish a dense cemented matrix. GPR is a powerful tool for the detection of subsurface defects. The GPR was able to map out the subsurface defects for the entire length of the project.

In general, close attention should be paid to base gradation, uniformity of mixture, and compaction of base layers. These precautions, along with further checking and enforcement of HMAC density requirements, would have prevented the base problems documented in this paper.

Recommendations for Future Consideration

1. Consider placing the CTB in one lift rather than two, when the layer thickness is 300 mm or less;
2. Ensure that future projects use a CTB gradation that is not prone to segregation; and
3. The District should revise its prime coat requirements for future CTB projects.

4.2.9 Stabilization and Erosion Control of Slopes Using Cement Kiln Dust

Ghazivinian, B., and Razavi, M. (2010) "Stabilization and Erosion Control of Slopes using Cement Kiln Dust." *GeoFlorida 2010*, pp. 2454-2461.

In this study the effects of cement kiln dust (CKD), a by-product of cement, on the geotechnical properties and erosion control of a natural slope consisting of fine-grained soil in the state of New Mexico, USA are investigated. Laboratory tests to determine Atterberg limits, compaction characteristics, unconfined compressive strength, and pH were performed on the native soil as well as the treated soil with CKD. The slope in the field was divided into several different portions, and each portion was treated with different percentage of CKD. Laboratory results showed that CKD reduces the plasticity index, maximum dry unit weight, and ductility of the soil while it increases the optimum water content, unconfined compressive strength, and pH of the soil. The field results showed that CKD reduces soil erosion significantly.

From the results of this study, it can be concluded that CKD is an effective additive as a stabilizer to control and minimize soil erosion. CKD reduces the plasticity index of the soils significantly. An increase in CKD content, increases optimum water content but reduces the maximum dry unit weight. CKD increases unconfined compressive strength of the soil, while it decreases soil ductility. CKD has a major effect on the pH of the treated soils; it increases the pH and makes the soil more alkaline. Soil loss due to erosion decreases by increasing CKD content.

Soil stabilization by CKD is a cost effective method of erosion control of slopes. For erosion control treating a thin layer of the soil (about 30 cm thick) on the surface with CKD reduces the soil loss due to erosion, significantly. Additionally, CKD reduces swelling of the soil due to reduction of soil plasticity index. Determination of the proper CKD content depends on the soil type and application. For a similar soil type used in this research, a CKD content of 10% or less by dry weight of the soil is recommended for stabilization and erosion control purposes. However, when low ductility or high pH level is a major concern, CKD content must be limited to 5% and even less. Further studies are required to evaluate the long term behavior of the treated soil with CKD.

4.2.10 Studies on Resilient Moduli Response of Moderately Cement-Treated Reclaimed Asphalt Pavement Aggregates

Puppala, A. J., Hoyos, L. R., and Potturi, A. K. (2011) "Studies on Resilient Moduli Response of Moderately Cement-Treated Reclaimed Asphalt Pavement Aggregates." *J. Materials in Civil Eng*., pp. 1-40.

The use of reclaimed asphalt pavement (RAP) aggregate materials in road construction reduces natural resource depletion and promotes the recycling of RAP materials for other applications. However, product variability and low resilient moduli characteristics often limit RAP applications in road bases. Stabilization of RAP materials with cement was hence attempted in a research study to evaluate the effectiveness of cement treatments in enhancing resilient characteristics of RAP aggregates. The present paper describes the results from a series of resilient modulus tests that were conducted in laboratory environment using a repeated load triaxial test setup. The effects of three different cement dosages and various confining and deviatoric stress levels on the resilient modulus (MR) response of treated RAP materials were studied. MR values of untreated and cement treated RAP aggregates ranged between 180 to 340 MPa and 200 to 515 MPa, respectively, which reveal the enhancements with cement treatment. Regression modeling analyses of MR test results, using two- and three-parameter models, are also presented. The analyses show that both models are reasonably capable of capturing the effects of stress levels on treated RAP resilient properties. Test results were also analyzed to determine structural coefficients for pavement design purposes, which ranged from 0.16 to 0.22, suggesting a greater structural support of cement treated RAP layers when compared to untreated aggregates.

The following provides a summary and major conclusions obtained from the present experimental results performed on cement treated RAP mixtures:

1. Resilient moduli test results on RAP materials showed standard deviations ranging from 1.8 to 5.2 MPa for untreated aggregate materials, and from 4.7 to 30 MPa for cemented

aggregate materials. High standard deviations were only recorded at the highest resilient moduli properties measured in this study. Overall, the coefficients of variation (COV) values of the present tests are reasonably small in magnitudes (highest COV value of 7% was determined for cement treated RAP) and, hence, it can be concluded that both the resilient modulus test procedure and the triaxial equipment utilized in this work yielded repeatable results.

2. An addition of 2% cement treatment increased the MR value of the RAP material by 32% when compared to the same of untreated aggregate specimen and with the addition of 4% cement, the MR value was increased by about 50%. The percent moduli increase with respect to cement treatment of the present RAP material can be regarded as moderate, but are reaching statistically significantly high level at 4% cement dosage. All these results were measured at a 7-day curing period which is typically required by the Texas state department of transportation.
3. Resilient moduli of the „untreated aggregate‟ materials increased with an increase in the applied confining pressure. For cement treated aggregate specimens, this was not evident as the cemented aggregate material was stiff and was hence not affected by the applied confining pressures. This behavior is similar to weak concrete materials which are unaffected by the applied confining pressure.
4. The modeling analysis utilizing two and three parameter resilient modulus formulations and following linear regression tools provided excellent fit of the experimental data with high coefficients of determination values. For two parameter theta model, the model constants indicate that the resilient modulus property of untreated aggregates showed a non-linear dependency on the bulk stress attribute of the formulation and for treated aggregates, the dependency is close to linear as k2 value is close to 1.
5. For the 3-parameter formulation, the constant parameter of the correlation, log k3 varied from 3.47 to 3.69, with a low values obtained for untreated aggregates and high values being obtained for cement treated aggregates. The k4 parameter (exponent of confining stress) is close to 0.19, indicating the resilient moduli results show a nonlinear type dependency on confining pressures. It is interesting to note that the cement treatment resulted in no major variation of this constant parameter. The k5 parameter which is an exponent of deviatoric stress, on the other hand, varies between 0.09 to 0.15, with a low value of 0.09 was being obtained for untreated aggregate specimens and a value close to 0.15 was obtained for cement treated aggregates. Since the k5 value is a positive, it can be interpreted that stress hardening is taking place in the present cement and untreated aggregate materials. This is expected since granular aggregate materials in general tend to display or undergo stress hardening phenomenon, i.e. an increase in resilient moduli with an increase in deviatoric stress under the same confining pressure.
6. The base layer coefficients for untreated aggregates ranged from 0.13 to 0.19, those for 2% cement treated aggregates ranged from 0.15 to 0.21 and those for 4% cement treated aggregates ranged from 0.16 to 0.22. As expected, the structural coefficient values increase with percent cement treatment and the confining pressure. Also, these values are in agreement with those of similar type materials reported in the literature.
   These values are currently recommended to be used for pavement design utilizing cemented RAP aggregates.
   One of the important aspects of sustainability in civil constructions is the utilization of materials and processes that are environmentally friendly. Utilization of RAP materials,

which have been stockpiling, will enhance sustainability efforts. This research demonstrates that RAP material can be effectively used as a base material to support pavement infrastructure. Further applications are also anticipated in backfills in embankments and walls, as this material demonstrated stiffness enhancements which would make them ideal as fill materials. The use of RAP will contribute to enhanced sustainable efforts in civil infrastructure constructions by utilizing this material instead of land filling them.

The soil was classified with the use of the supplied grain-size distribution graph. The value of resilient modulus was formulated by taking the average resilient modulus value for each cement content (%). The resilient modulus increasing factor was then calculated for each cement content. The resilient modulus values increase with the addition of cement content, with a factor of improvement for stiffness ranging from 1.2 to 1.4 for cement contents of 2% and 4%, respectively.

Table 4.8. Summary of stiffness improvement for Puppala et. al. (2011) study.

Soil Classification		Cement	Resilient	Modulus
USCS	AASHTO	Content (%)	Modulus (Mpa)	Factor
SW	A-1-a	0	266.5	-
		2	318.2	1.19
		4	372.1	1.40

4.3 Cement/Fly Ash

4.3.1 Behavior of Cement-Stabilized Fiber-Reinforced Fly Ash-Soil Mixtures

Kaniraj, S. R., and Havanagi, V. G. (2001) "Behavior of Cement-Stabilized Fiber-Reinforced Fly Ash-Soil Mixtures." *J. Geotechnical and Geoenvironmental Eng.*, 127(7) 574-584.

An experimental program was undertaken to study the individual and combined effects of randomly oriented fiber inclusions and cement stabilization on the geotechnical characteristics of fly ash-soil mixtures. An Indian fly ash was mixed with silt and sand in different proportions. The geotechnical characteristics of the raw fly ash-soil specimens and fly ash-soil specimens containing 1% randomly oriented polyester fiber inclusions were investigated. Unconfined compression tests were carried out on fly ash-soil specimens prepared with 3% cement content alone and also with 3% cement and 1% fiber contents, after different periods of curing. The study shows that cement stabilization increases the strength of the raw fly ash-soil specimens. The fiber inclusions increase the strength of the raw fly ash-soil specimens as well as that of the cement-stabilized specimens and change their brittle behavior to ductile behavior. Depending on the type of fly ash-soil mixture and curing period, the increase in strength caused by the combined action of cement and fibers is either more than or nearly equal to the sum of the increase caused by them individually.

Multiple tables and graphs are available if there is interest in pursuing Fiber-Reinforced additives.

An experimental program was undertaken to investigate the individual and combined effects of randomly oriented fiber inclusions and cement stabilization on the geotechnical characteristics of fly ash-soil mixtures. Experiments were conducted on fly ash-soil specimens of different forms as (1) unstabilized- unreinforced specimens; (2) cement-stabilized specimens; (3) fiber-reinforced specimens; and (4) fiber-reinforced cement-stabilized specimens. The following conclusions are drawn from the study:

1. In direct shear tests, the randomly oriented fiber inclusions increase the failure displacement and the vertical displacement of the fly ash-soil specimens compacted at the MDD-OMC state. The trend in the change of c and f due to fiber inclusions is not very consistent. Still, generally the fiber inclusions increase the shear strength.
2. The fly ash-soil specimens compacted at the MDD-OMC state exhibit brittle behavior in unconfined compression tests. The brittle behavior is more marked in cementstabilized specimens than in unstabilized specimens. The fiber inclusions change the behavior in both instances to ductile behavior.
3. The increase in the UCS of unstabilized fly ash-soil specimens due to fiber inclusions depends on the UCS of the unreinforced specimens. The relative gain in UCS decreases as the UCS in the unreinforced state increases.
4. The fiber inclusions increase the failure deviator stress of the unstabilized fly ash-soil specimens. However, the gain in deviator stress due to fiber inclusions is not as high as that in the UCS.
5. The failure envelope in the unconsolidated undrained test plotted as the variation of the major principal stress at failure (15% axial strain) with confining stress is bilinear for the unstabilized fiber-reinforced fly ash-soil specimens.
6. The fiber inclusions increase the compression index. They have no effect on the coefficient of consolidation.
7. The UCS of a fly ash-soil mixture increases due to addition of cement and fibers. Depending on the type of the mixture and curing period, the increase in UCS caused by the combined action of cement and fibers is either more than or nearly equal to the sum of the increase caused by them individually.

4.3.2 Durability of Soil-Cements against Fatigue Fracture

Sobhan, K., and Das, B. M. (2007) "Durability of Soil-Cements against Fatigue Fracture." *J. Materials in Civil Eng.*, 19(1) 26-32.

Cementitious stabilization is a common method of ground improvement when weak foundation soils are encountered in practice, and marginal or nonstandard materials such as recycled aggregates are used in civil engineering construction. Long-term durability of stabilized materials becomes an issue, especially when it involves one or more recycled materials with unknown and often questionable properties. The study presented herein investigates the fatigue durability, endurance limit, and damage accumulation process in recycled crushed concrete aggregate stabilized with cement–fly ash mixtures. Results show that the 2 million cycles fatigue endurance limit for the stabilized recycled aggregate was nearly 53% of the static modulus of rupture, indicating that the fatigue strength of this material is quite similar to or better than other traditional cementitious composites. It was also found that the accumulated permanent deformation and the expended fatigue life can be related by a nonlinear power law, and the fatigue damage in this material approximately follows Miner's rule for cumulative damage. Finally, the importance of developing innovative testing methods for durability assessment are highlighted, which include coupled mechanical and environmental loadings to simulate the most damaging field conditions, and accelerated aging and life prediction processes using the Arrhenius time–temperature superposition methods.

The concept of chemical stabilization is most frequently associated with improving the strength and stability of weak soils, and marginal or nonstandard aggregate used as foundation materials. Engineers are therefore often concerned with the long-term durability and future performance of stabilized materials, especially when they involve recycled waste products. The focus of the current study was to investigate the durability characteristics and damage processes in cement–fly ash SRA when subjected to repeated flexural (tensile) stresses. It was found that the fatigue endurance limit of SRA is comparable to concrete and other traditional stabilized materials, and that the damage accumulation approximately obeys the Miner's rule of cumulative damage. Empirical damage models are developed for predicting permanent deformation at various stages of the fatigue life at different stress ratios. In the absence of standardized tests specifically developed for evaluating the durability of cement-bound alternative or recycled materials, several advanced testing techniques are suggested which can provide valuable insights into the long-term performance of these materials. The writers believe that with the advent of many new-generation products for chemical stabilization, proper assessment of durability of the stabilized material will play a crucial role when making decisions on the choice of stabilizing agents and appropriate construction materials.

4.3.3 Environmental Effects of Durability of Aggregates Stabilized with Cementitious Materials

Khoury, N. N., and Zaman, M. M. (2007) "Environmental Effects of Durability of Aggregates Stabilized with Cementitious Materials." *J. Materials in Civil Eng*., 19(1) 41-48.

The present study focuses on investigating the effect of freeze–thaw (FT) cycles, referred to as environmental effect in this paper, on aggregates stabilized with various stabilizing agents, namely, cement kiln dust (CKD), Class C fly ash (CFA), and fluidized bed ash (FBA). Cylindrical specimens were compacted and cured for 28 days in a moist room with a constant temperature and controlled humidity. After curing, specimens were subjected to 0, 8, 16, and 30 FT cycles, and then tested for resilient modulus (M_r). Results showed that M_r values of stabilized specimens decreased with increasing FT cycles up to 30. The reasons for such changes are explained by the increase in moisture content during thawing and the formation of ice lenses within the pores during freezing, causing distortion of the matrix of particles. It was also found that the decrease in M_r values varied with the type of stabilizing agents. The CKD-stabilized Meridian and Richard Spur aggregates exhibited a higher reduction in M_r values than the corresponding values of CFA- and FBA-stabilized specimens. The CFA-stabilized Sawyer specimens performed better than their CKD- and FBA-stabilized counterparts. Data can be observed in not only the reduction in stiffness due to the freeze-thaw behavior of the soils tested, but also in the surprisingly high variation in stiffness values observed for similar soils with the same stabilizing agents used. Numerous tables of information show the resilient modulus data obtained for various stabilizing agents at various contents.

This study was undertaken to evaluate the effect of FT action on the resilient modulus of aggregates stabilized with different stabilizing agents, namely, 15% CKD, 10% CFA, and 10% FBA. Results showed that the resilient modulus decreases as the FT cycles increase up to 30. Such a decrease can be explained by the amount of water absorbed by the specimens during the thawing process. The more the amount of water absorbed the more the distortion of specimens during the freezing phase due to the formation of ice lenses. It was also found that the degree of damage or distortion due to FT action is dependent on the type of stabilizing agent used. The FT action has a higher effect on the CKD stabilized specimens for Meridian and Richard Spur aggregates, than CFA and FBA stabilization. On the other hand, Sawyer specimens were found to perform better with CFA, compared to CKD, and FBA stabilization. This study was limited to evaluating the effect of FT cycles on the variation of resilient modulus. No laboratory tests such as X-ray diffraction, X-ray fluorescence, and scanning electron microscopy were conducted to examine the microstructural behavior under FT cycles. A study is currently in progress that utilizes some of these tests to address this issue.

4.3.4 Influence of Fly Ash on Unconfined Compressive Strength of Cement-Admixed Clay at High Water Content

Jongpradist, P., Jomlongrach, N., Youwai, S., and Chucheepsakul, S. (2010) "Influence of Fly Ash on Unconfined Compressive Strength of Cement-Admixed Clay at High Water Content." *J. Materials in Civil Eng*., 22(1) 49-58.

This research studies the potential of using disposed fly ash to add up or partially replace Portland cement Type I in ground improvement by cement column technique. The strength characteristic of cement-fly ash admixed Bangkok clay was investigated by means of a series of unconfined compression tests, paying special attention to the influence of ground fly ash in this mixture. From testing results, the unconfined compressive strength and elastic modulus improved with an increasing of fly ash content. With the cement portions of greater than or equal to 10%, ground disposed fly ash could be employed as a pozzolanic material for partial replacement of cement in cement column construction. Based on the equivalent cementitious material content concept, an empirical equation relating the efficiency factor, α with mixing proportions was proposed. Then, together with this proposed efficiency factor, strength prediction of cement-fly ash admixed clay by Feret's equation and Abram's law were carried out and discussed.

The potential and efficiency of adding disposed fly ash from Mae Moh Electric Power Plant, Thailand, into cement-admixed clay were studied by means of a series of UC and physical tests. From this limited investigation, it is confirmed that, with suitable cement content, this ground disposed fly ash could be successfully added into soil cement to enhance both strength and physical characteristics. The strength of cement-fly ash admixed clay at high water content increased with increasing amount of cementitious material content and duration of the curing time and decreased with increasing water content. The efficiency of fly ash depended on the portion of cement, disposed fly ash, and water content in mixtures. To predict strength of clay-cement-fly ash mixtures, equivalent cementitious content concept, Aw, in conjunction with efficiency factor α can be successfully employed. The predictions of strength by the proposed empirical equations produced satisfactory agreements with the testing results. However, the proposed empirical equations are based on limited data of specific soil and source of fly ash, broader set of studies are needed for a more generalized form of these equations. Moreover, the long-term strength of this mixture should be investigated.

A strength factor was developed through estimation of graphical data provided in the cited report. This report provides properties of a soil with variations in fly ash and cement at different remolding water contents. Overall, the strength of the soil is observed to increase with higher quantities of cement and fly ash, though when the water content increases, the unconfined compression strength has a lower value. The table below provides factors of improvement for strength based on the data from the report.

Table 4.9. Summary of strength improvement for Jongpradist et. al. (2010) study.

USCS Classification (CH) with remolding water content of 130% and 28 day curing time												
Cement content 5%			Cement content 10%		Cement content 15%		Cement content 20%		Cement content 25%		Cement content 35%	
Fly Ash content	UCS (kPa)	Strength Factor	UCS (kPa)	Strength Factor	UCS (kPa)	Strength Factor	UCS (kPa)	Strength Factor	UCS (kPa)	Strength Factor	UCS (kPa)	Strength Factor
0%	65	-	180	-	300	-	400	-	420	-	660	-
5%	90	1.4	350	1.9	590	2.0	725	1.8	875	2.1	925	1.4
10%	140	2.2	400	2.2	625	2.1	880	2.2	950	2.3	1100	1.7
15%	180	2.8	420	2.3	825	2.8	950	2.4	1100	2.6	1275	1.9
20%	225	3.5	430	2.4	870	2.9	1030	2.6	1215	2.9	1360	2.1
25%	160	2.5	440	2.4	900	3.0	1210	3.0	1300	3.1	1525	2.3
30%	150	2.3	465	2.6	920	3.1	1360	3.4	1425	3.4	1680	2.5

USCS Classification (CH) with remolding water content of 200% and 28 day curing time												
Cement content 5%			Cement content 10%		Cement content 15%		Cement content 20%		Cement content 25%		Cement content 35%	
Fly Ash content	UCS (kPa)	Strength Factor	UCS (kPa)	Strength Factor	UCS (kPa)	Strength Factor	UCS (kPa)	Strength Factor	UCS (kPa)	Strength Factor	UCS (kPa)	Strength Factor
0%	15	-	85	-	150	-	200	-	225	-	325	-
5%	25	1.7	100	1.2	165	1.1	220	1.1	255	1.1	355	1.1
10%	40	2.7	110	1.3	250	1.7	300	1.5	328	1.5	390	1.2
15%	42	2.8	115	1.4	260	1.7	350	1.8	470	2.1	485	1.5
20%	50	3.3	125	1.5	275	1.8	375	1.9	495	2.2	525	1.6
25%	45	3.0	145	1.7	320	2.1	465	2.3	530	2.4	625	1.9
30%	43	2.9	150	1.8	330	2.2	545	2.7	590	2.6	690	2.1

The project used both fly ash and cement to stabilize the clay soil considered. For low cement and fly ash contents (5% of each), the factor of improvement for strength with respect to the unstabilized material was 1.4 to 1.7, such that a factor of 1.5 would be reasonable for this fat clay soil.

4.3.5 Strength and Dilatancy of a Silt Stabilized by a Cement and Fly Ash Mixture

Lo, S. R., and Wardani, S. P. R. (2002) "Strength and Dilatancy of a Silt Stabilized by a Cement and Fly Ash Mixture." *Can. Geotech. J.*, Vol. 39, 77-89.

The mechanical behavior of a weakly cemented silt was studied experimentally. The cementing agent was a cement and fly ash slurry, and the samples so formed were slightly cemented. In triaxial testing, both drained and undrained tests on saturated samples were conducted. Special zero effective stress tests were conducted to measure, directly, the contribution of bonding between grains to the enhanced strength and stiffness. The cemented soils were initially less dilatant than their respective parent soils but eventually became more dilatant than the parent soils. The shear strength data followed a curved failure surface that merged back, at high stress, into that of the parent soil. This feature can be captured by a failure function that models the contribution of cementing agent to strength as two parts, true bonding and increase in dilatancy rate at failure. Both parts degrade with an increase in confining stress, but at different rates.

The stress–strain–strength test data imply that the cementing agent contributes to both stiffness and strength via two mechanisms, namely bonding between grains and additional dilation. The bonding or its breakage can be modeled by yielding. Due to the low level of cementation, with the exception of testing at $\sigma 3 \leq 30$ kPa, the samples had the bond essentially destroyed prior to failure and hence the failures are all post-yield.

The bonding between grains was measured directly by special zero effective confinement tests. The stress–strain curves so measured were brittle, with the peak resistance mobilized at an axial strain of about 1–1.5% and with strain softening developing rapidly with post-failure shearing. Since the axial strains at failure, $\varepsilon 1f$, for confined tests were generally higher than 4%, the contribution of this component to strength will at least be partly lost prior to attaining the failure state. It is also consistent with the inferences from the dilatancy data at failure.

The cementing agent always led to an increase in peak strength via an increase in dilatancy at failure. The higher dilatancy at a high stress ratio (and at failure) can be viewed as the remanence of bonding.

The presence of a more dilating fabric is consistent with the hypothesis by Barton (1993) that one of the effects of cementation is to produce interlocking that can be overcome by dilatancy. A careful examination of the dilatancy rate throughout all states of shearing (Fig. 11) reveals that at stress state remote from failure, the dilatancy rate of the cemented soil is in fact smaller than that of the parent soil, but can continue to increase to a higher *D*f value than that of the parent soil (which ceased as *D* approached unity). This implies that the higher dilatancy rate may, at least partly, be a consequence of bond breakage.

A new failure function was proposed for the cemented soil. This failure function embodies the influences of a cementing agent on dilatancy and strength and can fit the strength data of cemented soils better than a power function. The initial portion of this failure function resembles that of a power function, but the complete curve has a shape that merged back to that of the parent soil at adequately high confining stress relative to the cementation. Hence the failure function may be of general applicability.

4.4 Fly Ash/Lime/Gypsum

4.4.1 Hydraulic Conductivity and Leachate Characteristics of Stabilized Fly Ash

Ghosh, A., and Subbarao, C. (1998) "Hydraulic Conductivity and Leachate Characteristics of Stabilized Fly Ash." *J. Environmental Eng*., 124(9) 812-820.

Disposal of fly ash on land amounts to sacrificing precious land space. Recycling of fly ash is one of the methods of solving the disposal problem. Stabilization of a low lime fly ash with lime and gypsum was studied through large scale tests on the stabilized material designed to simulate field recycling conditions as closely as possible, and found to be a very effective means to control hydraulic conductivity and leachate characteristics. The effects of molding water content, lime content, gypsum content, curing period, and flow period on hydraulic conductivity, and on leachate of metals flowing out of the stabilized fly ash are reported herein. With proper proportioning of the mix, and adequate curing, the values of hydraulic conductivity on the order of 10^{-7} cm/s were achieved. The concentrations of As, Cd, Cr, Cu, Fe, Hg, Mg, Ni, Pb, and Zn in the effluent emanating from the hydraulic conductivity specimens of mixes with higher proportions of lime or lime and gypsum were below threshold limits acceptable for contaminants flowing into ground water.

Disposal of fly ash has become a critical problem in waste management as it requires large sites to dispose of enormous quantities of fly ash on land. Recycling fly ash with proper treatment for environmental safety may transform this liability into an asset. Stabilization of low lime fly ash is difficult because of its subdued reactivity and pozzolanic action. Coal fly ash with lime constituent as low as 1.4% was stabilized with lime and gypsum. The lime contents were 4, 6, and 10% of fly ash and the gypsum contents were 0.5 and 1.0% of fly ash in the stabilized mixes.

An experimental setup was designed to evaluate the effect of stabilization on the performance of the stabilized material through large scale tests to simulate field conditions of compaction, flow, and leachate generation as closely as possible. The stabilized mixes were compacted to the standard Proctor density of ASTM D 698 (Standard 1992). The compacted specimens were cured for 7 and 28 days. Continuous flow of water through the specimens for 7 days under a head of 1.5 m of water generated the leachate and permitted the determination of hydraulic conductivity (K), and leachate characteristics. The quantity of a metal in the leachate is predominantly influenced by the hydraulic conductivity of the stabilized material and the concentration of the metal in the leachate. The pH value of the pore water is one of the factors which governs the concentration of a metal in the leachate. Lime and gypsum stabilization reduced the hydraulic conductivity by -500 times with respect to that of the unstabilized fly ash. The ambience of high alkalinity in the pore fluid of the stabilized fly ash was conducive to the precipitation of some of the metals. Encapsulation of metals by the hydration products also possibly decreased the quantity of metals in the leachate.

The following conclusion may be drawn from the test results of the present investigation.

Addition of lime to fly ash reduces hydraulic conductivity. Gypsum, in the presence of lime, helps to decrease hydraulic conductivity even further. It makes the matrix more stable and enhances the pozzolanic reaction. Hydraulic conductivity of the stabilized material reduces with an increase in molding water content, and the reduction in hydraulic conductivity is less on the wet side of optimum. For all the mixes of fly ash and lime or fly ash lime and gypsum there is a reduction in hydraulic conductivity with an increase in the curing period. Stabilized compacted low lime fly ash mixed with 10% lime and 1% gypsum and cured for 28 days could produce an impermeable layer useful for base layers or waste containment liners with permeability on the order of 8 X 10-8 cm/s from fly ash with permeability 4.5 X 10-' cm/s.

Molding water content on the wet side of optimum is more effective in reducing leachates of metals in terms of total quantity per day compared to the dry side of optimum. The effectiveness of stabilization to reduce leaching varies from metal to metal. For all the mixes the concentrations of Cu, Fe, Mg, Ni, and Zn in the leachate flowing out of the stabilized specimens were below allowable limits of drinking water quality, whereas concentrations of As, Cd, Cr, and Pb were above allowable limits, but below threshold limits. The concentration of Hg was above threshold limits for some of the mixes. The concentration of Hg could be restricted below threshold limits with the addition of a higher percentage (10%) of lime or lime and gypsum (RF

+ l0L + IG). Leachate load ratio, R, is useful in indicating the effect of stabilization in reducing the quantity of metal in the leachate.

For fly ash-lime-gypsum mixes, a molding water content in the range of OMC and OMC + 5% can be specified for field control of fill moisture content. This molding water content on the wet side of optimum has the advantages of low hydraulic conductivity, reduced leaching, marginal variations of hydraulic conductivity and obviously better workability. Fly ash stabilized with lime and gypsum can be considered for structural fill in road bases and embankments and for use in impermeable barriers, such as covers and liners and cutoff trench walls, minimizing the potential for ground water contamination.

4.4.2 Leaching of Lime from Fly Ash Stabilized with Lime and Gypsum

Ghosh, A., and Subbarao, C. (2006a) "Leaching of Lime from Fly Ash Stabilized with Lime and Gypsum." *J. Materials in Civil Eng.,* 18(1) 106-115.

Stabilization of fly ash is one of the promising methods to transform the waste material into a safe construction material. The longevity of lime-stabilized fly ash is related to the amount of lime that remains in the matrix after leaching. This paper presents leaching test results of a class F low lime fly ash stabilized with varying percentages of lime (4, 6, and 10%) alone or in combination with gypsum (0.5 and 1.0%). Addition of gypsum has been found to be very effective in reducing the leaching of lime from fly ash stabilized with lime. The effects of factors like lime content, gypsum content, curing period, and flow period on leaching of lime from a compacted stabilized fly ash matrix are reported herein. Compacted specimens were cured for seven and 28 days. The concentrations of calcium in the effluent emanating from hydraulic conductivity test specimens were measured daily for seven days in succession. The concentration of calcium in the leachate was reduced to 80 from 540 ppm for addition of 1.0% gypsum to fly ash stabilized with 10% lime and cured for 28 days. Two nondimensional parameters αcoi and βcoi are presented herein to study the effect of gypsum on the total amount of calcium leached out from the compacted stabilized fly ash. A model is also presented to estimate the amount of calcium leached out from the stabilized fly ash.

The concentration of calcium in the leachate collected from compacted and stabilized fly ash mixes was measured. Fly ash was stabilized with 4, 6, and 10% lime alone or in combination with 0.5 and 1% gypsum. The compacted specimens were cured for seven and 28 days. The leaching test was conducted under a water head of 1.5 m for continuous seven days of flow. The amount of lime leached out from compacted stabilized fly ash due to flow of water is calculated considering the most vital parameters like hydraulic conductivity of the specimen and the concentration of calcium in the leachate. The amounts of calcium leached out from the compacted specimens were compared with the calcium added to fly ash considering the purity of the lime. The following conclusions may be drawn from the study presented herein.

- The leaching of calcium in ppm per percentage addition of lime to fly ash is more for a lower percentage of lime addition and decreases with addition of higher lime content up to 10% for class F fly ash.
- Gypsum is very effective in reducing the concentration of calcium in the leachate from compacted fly ash–lime–gypsum specimens. Addition of only 1% gypsum to fly ash

stabilized with 10% lime reduces the concentration of calcium in the leachate from 540 to 80 ppm for the 28-day cured specimen.

- The concentration of calcium in the leachate decreases with increase in curing period (up to 28 days) for all the specimens of this study irrespective of lime content and gypsum content.
- The total amount of calcium leached out from a compacted specimen is reduced significantly with addition of 1% gypsum to fly ash stabilized with lime. The total amount of calcium leached out in seven days, as a percentage of the total amount of calcium added in the specimen, i.e., βcoi, was reduced from 0.3352 to 0.0078% for addition of 1% gypsum to a fly ash mix containing 10% lime, cured for 28 days.
- The proposed nonlinear power model which takes into consideration the important parameters like lime content, gypsum content, flow period, and curing period may be used for estimation of the lime leached out from stabilized fly ash. However, the model may be further refined with more databases.

4.4.3 Role of Gypsum in the Strength Development of Fly Ashes with Lime

Sivapullaiah, P.V., and Moghal, A. A. B. (2011) "Role of Gypsum in the Strength Development of Fly Ashes with Lime." *J. Materials in Civil Eng.*, 23(2) 197-206.

The strength of fly ash mixture often needs to be enhanced for its better utilization in geotechnical and environmental applications. Many fly ashes often improve their strength with lime but may not meet the requirements. Gypsum, which reduces the lime leachability, further improves the strength. An attempt is made in this paper to study the effect of gypsum on the strength development of two Class F fly ashes with different lime contents after curing them for different periods. The sustainability of improved strength has been examined after soaking the cured specimens in water and with different leachates containing heavy-metal ions. The strength of both the fly ashes investigated improved markedly up to a particular amount of the lime content, which can be taken as optimum lime content, and thereafter the improvement is gradual. The improvement in strength at higher lime contents continues for a longer period (even up to 180 days). Gypsum accelerates the gain in strength for lime-stabilized fly ashes, particularly in the initial curing periods at about optimum lime content. At high lime contents gypsum attributes very high strength after curing for long periods mainly due to the alteration of fly ash lime reaction compounds. Gypsum not only improves the reduction in the loss of strength due to soaking even at low curing periods but also improves the durability of stabilized fly ashes due to repeated cycles of wetting and drying.

Scanning electron microscope, X-ray diffraction, wet/dry cycles, Unconfined compression tests were carried out.

The present study examines the role of lime and gypsum additions and curing periods on the strength behavior of fly ashes under unsoaked and soaked conditions. The effect of alternate wet and dry cycles on the strength characteristics has also been studied. The following major conclusions are drawn:

1. The strength of low lime-fly ashes which increases with lime content is significant up to an optimum lime content of about 5% and proceeds gradually thereafter.

2. Addition of gypsum increases the strength of fly ashes at any lime content. At lower curing periods with lower lime contents the increase in strength with gypsum is quite significant. Increase in strength is observed at higher lime contents (above 5%) after a considerable period of 60 days. This increase in strength has been attributed due to the formation of calcium-sodium-aluminate-silicate hydrate along with calcium-silicate hydrate.
3. Fly ash which responds readily to lime stabilization shows accelerated gain in strength due to the addition of gypsum at early curing periods.
4. The increase in strength achieved with gypsum is not susceptible to the effect of alternate wet and dry cycles.
5. The effect of soaking on the strength behavior under different chemical environments is observed to be the least with the fluid containing chromium ions.

4.4.4 Tensile Strength Bearing Ratio and Slake Durability of Class F Fly Ash Stabilized with Lime and Gypsum

Ghosh, A., and Subbarao, C. (2006b) "Tensile Strength Bearing Ratio and Slake Durability of Class F Fly Ash Stabilized with Lime and Gypsum." *J. Materials in Civil Eng*., 18(1) 18-27.

This paper presents the results of a laboratory investigation on tensile strength, bearing ratio, and slake durability characteristics of a class F fly ash stabilized with lime alone or in combination with gypsum. The effects of lime content (4, 6, and 10%), gypsum content (0.5 and 1.0%), and curing period (up to 90 days) on the tensile strength, bearing ratio, and durability characteristics of the stabilized fly ash are highlighted. Unconfined compressive strength test results for the mixes cured up to 90 days are presented to develop relationships between different tensile strengths (Brazilian and flexural) and unconfined compressive strength. Both soaked and unsoaked bearing ratio tests were also carried out on this stabilized fly ash. The Brazilian tensile strength of the lime and gypsum stabilized fly ash mixes varied between 309 and 1,084 kPa for 45 days curing. The flexural strength of the lime and gypsum stabilized mixes cured for 45 days varied between 665 and 1,459 kPa. Fly ash stabilized with lime and gypsum showed medium durability at 28 days curing and there was enhancement of durability with increase in curing period. Empirical models to estimate tensile strength, bearing ratio, and slake durability indices of stabilized fly ash from unconfined compressive strength test results are also proposed herein. With enhanced tensile strength and durability characteristics, the stabilized fly ash may find potential use in civil engineering construction.

The suitability of class F fly ash stabilized with lime (up to 10%) alone or in combination with gypsum (0.5 and 1.0%) as road base material is studied through unconfined compressive strength, Brazilian tensile strength, flexural strength, bearing ratio, and slake durability indices. The effect of curing period on these parameters is also highlighted. An attempt has also been made to develop relationships to predict Brazilian tensile strength, flexural strength, bearing ratio, and slake durability indices in terms of unconfined compressive strength. The class F fly ash containing only 1.40% CaO after stabilizing with 10% lime and 1.0% gypsum has achieved a compressive strength of 6.308 MPa at 90 days curing and a flexural strength of 1.459 MPa at 45 days curing. With improved engineering properties the stabilized class F fly ash may find a potential application in providing a strong road base.

The following conclusions may be drawn from the test results presented herein:

- Stabilization of the class F fly ash with lime alone or in combination with gypsum is effective to enhance the unconfined compressive strength, *qu*. The value of *qu* increases with increase in curing period and can attain a value of 6,308 kPa for fly ash stabilized with 10.0% lime and 1.0% gypsum at 90 days curing;
- The Brazilian and flexural strength of the stabilized fly ash increased with increase in lime content and curing period;
- To achieve considerable tensile strength, fly ash stabilized with only lime needs to be cured up to 45 days;
- Addition of small percentages of gypsum (0.5 and 1.0%) to fly ash lime mixes increased the tensile strength even at 28 days curing;
- Brazilian and flexural strength of specimens containing 10% lime and 1% gypsum were 1,084 and 1,459 kPa, respectively, at 45 days curing;
- The ratio of Brazilian tensile strength to unconfined compressive strength for lime and gypsum stabilized fly ash mixes cured for 45 days were between 12.30 and 21.50% depending on mix proportions;
- The ratio of the flexural strength to unconfined compressive strength for mixes stabilized with lime and gypsum, cured for 45 days, was varying between 22.55 and 29.62% depending on mix proportions;
- The bearing ratio value of fly ash can be increased with lime stabilization;
- Addition of small percentages of gypsum (0.5 and 1.0%) along with lime increases the bearing ratio value even at 7 days curing;
- The bearing ratio values of the specimens containing gypsum along with lime soaked for four days were higher than the bearing ratio values of the unsoaked specimens;
- The bearing ratio value of the unsoaked specimens containing 10% lime and 1.0% gypsum was 172 at 28 days curing period compared to 34 for unstabilized fly ash for the same curing period;
- The class F fly ash containing only 1.4% CaO achieved considerable durability due to lime and gypsum stabilization. From the slake durability indices study it is revealed that gypsum is very much effective in enhancing the durability of the fly ash; and
- Simple power models proposed herein may be used to estimate Brazilian tensile strength, flexural strength, bearing ratio, and slake durability indices from unconfined compression test results.

4.4.5 Strength Characteristics of Class F Fly Ash Modified with Lime and Gypsum

Ghosh, A., and Subbarao, C. (2007) "Strength Characteristics of Class F Fly Ash Modified with Lime and Gypsum ." *J. Geotechnical and Geoenvironmental Eng*., 133(7) 757-766.

This paper presents the shear strength characteristics of a low lime class F fly ash modified with lime alone or in combination with gypsum. Unconfined compression tests were conducted for both unsoaked and soaked specimens cured up to 90 days. Addition of a small percentage of gypsum (0.5 and 1.0%) along with lime (4–10%) enhanced the shear strength of modified fly ash within short curing periods (7 and 28 days). The gain in unsoaked unconfined compressive strength (qu) of the fly ash was 2,853 and 3,567% at 28 and 90 days curing, respectively, for addition of 10% lime along with 1% gypsum to the fly ash. The effect of 24 h soaking showed reduction of qu varying from 30 to 2% depending on mix proportions and curing period. Unconsolidated undrained triaxial tests with pore-pressure measurements were conducted for 7 and 28 days cured specimens. The cohesion of the Class F fly ash increased up to 3,150% with addition of 10% lime along with 1% gypsum to the fly ash and cured for 28 days. The modified fly ash shows the values of Skempton's pore-pressure parameter, A_f similar to that of over consolidated soils. The effects of lime content, gypsum content, and curing period on the shear strength parameters of the fly ash are highlighted herein. Empirical relationships are proposed to estimate the design parameters like deviatoric stress at failure, and cohesion of the modified fly ash. Thus, this modified fly ash with considerable shear strength may find potential use in civil engineering construction fields.

The shear strength characteristics of a Class F fly ash were studied through unconfined compression tests and unconsolidated undrained triaxial tests with pore-pressure measurements. The fly ash was stabilized with 4–10% lime alone or in combination with gypsum (0.5 and 1.0%). The specimens were cured up to 90 days. Both soaked and unsoaked unconfined compression tests were conducted. Empirical relationships are developed to estimate deviatoric stress at failure and cohesion as functions of unsoaked unconfined compressive strength. The following conclusions may be drawn from the test results and the discussions presented herein.

- Stabilization of a low lime Class F fly ash with lime _up to 10%_ is effective to improve the shear strength characteristics;
- Addition of a small percentage of gypsum _0.5 and 1.0%_ along with lime to fly ash enhances the gain in shear strength at early curing periods _7 and 28 days_;
- Gypsum along with lime is effective to control the loss of shear strength due to soaking for specimens cured for 28 days or more. The loss of shear strength due to soaking of such specimens is limited to 25%. Specimen stabilized with only lime showed soaked qu about 72% of unsoaked qu at 90 days curing;
- Fly ash stabilized with only lime requires longer curing period, 45 days and more, to gain considerable shear strength;
- Fly ash stabilized with 10% lime and 1% gypsum has achieved unconfined compressive strength _qu_ of 6308 kPa at 90 days curing;
- The pore-pressure response of the stabilized fly ash is similar to that of stiff soils. The peak pore pressure develops before the deviatoric stress reaches its maximum value irrespective of the mix proportions. Skempton's pore-pressure parameters Af and B vary from −0.065 to +0.057 and 0.68 to 0.13, respectively, for the stabilized fly ash specimens tested in this investigation; and
- Simple empirical relationships are recommended to estimate deviatoric stress at failure and cohesion from unsoaked unconfined compressive strength.
- Thus, fly ash containing CaO as low as 1.4%, stabilized with lime and a small percentage of gypsum may find potential application in road and embankment constructions for its strength characteristics, durability, longevity, and environmental safety. The stabilized fly ash having low hydraulic conductivity and alkaline environment of pore fluid may find use in construction of waste containment liners, cut off walls, and vertical barriers.

4.5 Cement/Lime/Slag/Gypsum

4.5.1 On Yield Stresses and the Influence of Curing Stresses on Stress Paths and Strength Measured in Triaxial Testing of Stabilized Soils

Ahnberg, H. (2007) "On Yield Stresses and the Influence of Curing Stresses on Stress Paths and Strength Measured in Triaxial Testing of Stabilized Soils." *Can. Geotech. J.*, Vol. 44, 54-66.

Studies on the behavior of stabilized soils under different loading conditions are essential to identify which parameters are relevant in the design of deep mixing. An investigation has been performed on soils stabilized with different types of binders with the purpose of demonstrating the effects of quasi-preconsolidation pressures, i.e., yield stresses that are not primarily linked to previous consolidation pressures but to the cementation taking place, on the strength behavior of stabilized soil. The effect of stresses applied during curing has also been studied. Drained triaxial compression tests and undrained triaxial compression and extension tests were performed on two stabilized clays. The binders used were cement, lime, slag, and fly ash in different combinations. Comparisons have also been made with results from previous tests on two organic soils stabilized with much the same types of binder. The results show that both the cementation processes involved and the stresses applied during curing affect the quasi-preconsolidation pressure. This pressure is strongly linked to the strength of the stabilized soil and has a considerable influence on its deformation behavior. A model is proposed which describes the strength behavior in the same effective stress plane that is commonly used for natural clays.

The results of the investigation performed on the two clays stabilized with different types of binder showed that triaxial tests and oedometer tests are useful tools for studying the strength behavior of the materials.

1. Quasi-preconsolidation pressures, $\sigma qp'$, governed by cementation effects and by previous curing stresses were observed in the triaxial tests and in the oedometer tests. The influence of this quasi-preconsolidation pressure was observed in compression and extension triaxial tests. The quasi-preconsolidation pressure affects the stress paths and is closely associated with the strength of the stabilized soils. The results show that the stabilized soils behave in an overconsolidated manner when the consolidation stresses are significantly lower than the quasi-preconsolidation pressures and in a normally consolidated manner when the consolidation stresses are of the order of 0.8–1.0$\sigma qp'$. No tendencies were observed towards differences in the general strength behavior when using different types of binder.
2. A common yielding model for natural clays (Larsson 1977) was also found suitable for describing the behavior observed in the tests on stabilized soils. The yield surface is schematically described by the failure strength envelopes in compression and extension and by$\sigma v' = \sigma qp'$ and $\sigma h' = K0nc\sigma qp'$. An isotropic yield surface could be adopted for the samples cured in the normal way without being subjected to external stresses.
3. Normalization of the deviator stress with respect to the quasi-preconsolidation pressure was effective for studying the behavior of different types of stabilized soils. The undrained compressive strength varied from approximately 0.6$\sigma qp'$ to 1.0$\sigma qp'$ for highly overconsolidated and normally consolidated samples, respectively, and the variation in

drained strength of the different types of stabilized clays could be expressed by a mean cohesion intercept of $0.15\sigma qp'$ and a mean friction angle of 33°.

4. Stresses applied shortly after mixing will compress the stabilized soil and result in increased strength and increased quasi-preconsolidation pressure. A K0 value lower than unity applied during curing results in a ratio of the horizontal to vertical yield stresses, K0nc, lower than that of samples cured without stresses.
5. Although the results represent a large variation in strength depending on the type of soil, type of binder, binder quantity, and curing time, consistent patterns could be observed in the stress–strain relations. The behavior is linked to the degree of overconsolidation. In undrained tests, there is only limited further change in deviator stress at continued loading after failure. In drained tests on specimens consolidated at stresses well below the quasi-preconsolidation pressure, failure occurs at small strains, and a significant reduction in shear stress with strain after failure is observed. Stabilized samples that are close to normally consolidated exhibit failure at large strains.

4.6 Portland & Bituminous Cement/Fly Ash/Slag/Lime/Chemicals/Polymers

4.6.1 Material Properties of High Volume Fly Ash Cement Paste Structural Fill

Doven, A. G., and Pekrioglu, A. (2005) "Material Properties of High Volume Fly Ash Cement Paste Structural Fill." *J. Materials in Civil Eng.*, 17(6) 686-693.

Fly ash can be effectively used as compacted or flowable fill material in the construction of structural fills for building foundations, embankments, base and subbase courses for highways and railroads, dikes, levees, bridge abutments, and landfill cover in lieu of conventional earth materials. In this study, high volume fly ash cement paste composite formed of various combinations of fly ash, cement, lime, silica fume, and chemical admixtures has been examined in terms of its physical (dry unit weight, void ratio, apparent specific gravity, linear shrinkage), mechanical (unconfined compressive strength, flexural strength), and durability properties (hydraulic conductivity, soundness by use of sodium sulphate) at 0, 100, and 200 mm slump values. The results indicate that the cement paste, providing certain advantages over conventional fill materials with its lower unit weight, higher strength, and higher volume stability can be designed for any required engineering performance for use in the construction of compacted to self-compacting structural fills and promises high volume fly ash utilization with low technology requirements avoiding scarce use of raw resources.

The experimental results indicate that the composite material can be effectively designed for use in a wide variety of fill applications as flowable fill to structural fill such as in the construction of sewer, water, and other transmission pipelines, embankments for highways and railroads, dikes, vehicle parking areas, foundations below buildings and other structures, dams, liners, and landfill covers based on the following results.

- The unit weight of the composite material ranges between 11.9 and 16.5 kN/m3; being lower than those of natural soils and concrete, the composite is expected to provide benefits in terms of the design and construction of nonstructural/ structural fills due to the lower stress level to be exerted on the foundation.

- The linear shrinkage values range between −0.0763 and 0.0830%, not exceeding the 0.8% limit value as specified by ASTM C151-89, which is quite low for a cementitious matrix formed without the passive component, the aggregates; the composite promises higher volume stability in comparison to those of natural soils.
- The initial and final setting times range between 1 h 9 min and 11 h 57 min; providing ease for use in the construction of light to massive structural fills by adjusting the hardening time.
- The unconfined compressive and flexural strengths of the composite material range between 2.6 and 33.0 and 0.64 and 4.40 MPa, respectively, between 7 and 90 days curing period within a 0–200 mm slump range. The relatively high strength, quite close to that of ordinary Portland cement concrete, enables the composite to be used in a wide variety of nonstructural to structural fill applications with higher load bearing capacity.
- The coefficient of hydraulic conductivity of the composite material range between 1.73_10−4 and 2.97_10−8 cm/ s, mostly within the range of those of silty clay to clay, low enough to enable the use of material in the construction of landfill coversand impermeable liners.
- The soundness values obtained by use of sodium sulphate in relatively severe simulation conditions indicate that the composite material has a moderate to high durability, which must further be investigated for wetting—drying and freeze—thaw cycles.
- The flexibility in obtaining the desired engineering performance by arranging/adjusting the combination/amount of mineral admixtures in the mix as well as the compactive effort, even higher than that obtained for the mix combinations investigated in this study, should also be mentioned. On the other hand, the material has also to be investigated in terms of its creep and fatigue behavior in order to ensure the in situ engineering performance.

4.6.2 Cohesion, Adhesion, and the Durability of Stabilized Materials

Inyang, H. I., and Bae, S. (2006) "Cohesion, Adhesion, and the Durability of Stabilized Materials." *J. Materials in Civil Eng*., Editorial, pp. 133-134.

Stabilization projects have been implemented in various parts of the world to improve the strength and bearing capacity of foundations and to control the permeability, shrink–swell potential, and related characteristics of soil materials. A primary stabilization method is using amendments that have textural, mineralogical, and chemical characteristics that are capable of generating the required changes in physicochemical characteristics of soils when they are mixed. Stabilization agents that are currently available for soil improvement include portland cement, bituminous cement, fly ash, slags, lime, lignins, calcium salts, and polymers. The composite materials formed through soil stabilization vary in degree of cementation from free particles to monoliths in which soil and other introduced particles are bound. Whatever the degree of cementation attained, pores within particles _intragranular pores_ and pores among particles (intergranular pores) cannot be completely eliminated from stabilized materials. Invariably, solid–solid, solid–liquid, solid–air, liquid–air, and triple interfaces exist in stabilized materials. These interfaces are the primary locations of physicochemical and biological processes that have significant implications on the initial strength and durability of stabilized materials.

Across solid–solid interfaces, particles may be of the same or different materials, the statistics of which depend on the mix proportions of the host material (often, a multicomponent soil) and the stabilization agent, if it is initially in solid form. Usually, the agent is applied in the liquid form, but regardless, subsequent precipitation of new solid phases derived from substances in the agent as well as in the host material, can still create new solid– solid interfaces. Obviously, the prominence of interfaces involving liquids and air depends on the porosity and permeability of the stabilized material and pore fluid chemistry. Interfaces are weak links in materials, because they provide greater opportunities than intact portions of materials for stress concentrations through a variety of mechanochemical processes. These mechanisms and processes are briefly discussed herein, with respect to the durability of chemically stabilized geomaterials.

The surficial environment (typically, 0–10 m deep) in which field stabilization projects are usually performed is subjected to cyclical stresses and reversals of environmental conditions over hours, days, and months. Within the bounds of the microclimate of the region of concern, daily and seasonal reversals in ground temperature, as well as moisture conditions, occur. As a result of contaminant emissions from industrial and civil facilities and contact of precipitation runoff with soluble materials on the ground, moisture that comes in contact with in situ stabilized materials is never neutral in chemistry. The long-term durability and environmental performance of a stabilized material depends on the response of its interfaces to loading and physicochemical attack in the surficial environment. The stabilization of unstable clayey soils with such chemical agents as cement and lime usually generates new solid phases, although the intergranular and intragranular porosities mentioned in a preceding paragraph still exist. At the fundamental level, the parameters of interest are cohesion (in terms of the soundness of individual particles) and adhesion, in regard to bonding of adjacent articles across solid–solid interfaces. In this context, cohesion is not defined as in soil mechanics, where it is considered to be the binding strength of particles, which is most significant in clays.

With respect to the load-deformation response, interface flaws that are distributed in a stabilized material as a result of incomplete cementation during stabilization and damage by the previously discussed environmental stresses, can grow. For soils, flaw sizes are likely to be somewhat directly proportional to soil lump sizes in plastic clays and to particle sizes in cohesionless soils (silt, sand, and gravel). On this account, flaws in boundary areas among large particles or internally sound lumps are likely to degrade more because of greater opportunity for their extension. However, a Poisson distribution of flaw sizes should be expected in stabilized, poorly sorted, cohesionless soils, since the larger pores will be fewer than the smaller pores, as a reflection of the typical distribution of particle sizes in soils. During the mixing of stabilization agents with a lumpy clay soil, disintegrating the lumpy clay soil is necessary so that particles or aggregates of particles that are internally weaker than the major particle interfaces because of their nonpermeation by the stabilization agent are not significantly present. In clay, nonpermeation can allow the swelling and weakening of lumps on contact with liquids.

The allowance of pores by incomplete cementation has many material durability implications. First, new materials can precipitate within the pores and can close both pores and debonded particle contacts so that the strength of the material is increased. This rationale is often used for stabilizing soils with cement and lime. However, if the crystallizing minerals exert expansion pressures that are beyond the tensile strength of the stabilized material, fracturing may intensify even without external loads. This phenomenon is exemplified in aggregate soundness tests in which the extent of aggregate damage by pressures exerted by anhydrous sodium sulfate is assessed. Pore fluids that have aggressive chemistry can attack solid–solid interface bonds and weaken stabilized materials through a suite of processes that are recognized in mechanochemistry of materials as the "Rehbinder effect." As an example, dispersion of ineffectively stabilized clay of appropriate mineralogy can lower the magnitudes of shear modulus and other strength-related parameters of clay. This phenomenon occurs with Na-montmorillonitic soils that are exposed to a high pH environment. The hydroxyl ions (OH-) present in pore fluid can attach to positive charges on broken edges of clay particles (those that have not been neutralized by stabilization agents), causing the expansion of double layers around affected clay particles, and leading to dispersion and lowering of the shear strength of some portions of the clay soil.

Another significant aspect of pore fluid action in stabilized materials relates to contaminant leachability. Ashes produced by municipal waste incinerators and coal combustion in electric power plants are used in soil stabilization in many countries. Some papers in this special edition focus on ash use for this purpose. A concern with ash use in exposed structural systems is the excessive leachability of chemical substances that are known to be distributed internally and on the surfaces of ash particles. When ash is bound with soils in cemented monoliths, the diffusion coefficient of a potential contaminant through soil lumps and ash particles into the percolating leachant (rainwater or groundwater) in the intergranular pore fluid is likely to be rather small. Of course, the contaminant diffusion coefficient is directly proportional to material porosity when all other factors are held constant. This porosity is controlled operationally by the design of the stabilized material mix in terms of its component mix proportions. Consequently, for an ash-stabilized soil, the mix proportions of the ash, soil, water, and any additionally applied binder affect the porosity and hence the leachability of chemical substances (e.g., the contaminant) from the stabilized soil. The concentration gradient that drives the flow of the diffusing contaminant from the host particles (ash) to the intergranular pores where flushing occurs is partly dependent on the volumetric fraction of ash in the stabilized soil mix. The greater the ash content, the greater the gradient and the contaminant leaching time required to dissipate it. Furthermore, greater particle or lump sizes mean greater distances for diffusion, with a consequent high probability of transport constraint to the introduction of the contaminant into the flushing pore fluid at large flow rates.

Semi-plastic materials such as bitumen have also been used in soil stabilization to improve the bearing capacity of soils. Sandy or silty soils can be effectively stabilized through this method. Progressive aging of the bituminous binder can cause the stabilized soil to be brittle and fail in the long run. The effects of aging are most prevalent at solid–solid interfaces.

The significance of interfaces with respect to the durability of stabilized materials should be given greater recognition in stabilization projects. On the basis of scenarios of the environment in which the stabilized material is expected to perform, the most significant stress-inducing mechanisms and relevant parameter magnitudes should be identified for use in designing simulative tests. Subjecting the stabilized material (with varied mix composition) to field and laboratory tests is likely to enable the optimization of material mix composition for cost-effective project implementation. Mix design can then be combined with monitoring and maintenance planning to improve stabilization programs. This special edition of the ASCE *Journal of Materials in Civil Engineering* on "Stabilization of Geomedia Using Cementitious Materials" exhibits research findings and practical projects that represent advances in this direction.

4.6.3 Preliminary Laboratory Investigation of Enzyme Solutions as a Soil Stabilizer

Marasteanu, M. O., Hozalski, R., Clyne, T. R., and Velasquez, R. "Preliminary Laboratory Investigation of Enzyme Solutions as a Soil Stabilizer." *MN/RC-2005-25*, Minnesota Department of Transportation, 2009.

This research studied the effect of two enzymes as soil stabilizers on two soil types to determine how and under what conditions they function. Researchers evaluated the chemical composition, mode of action, resilient modulus, and shear strength to determine the effects of the enzymes A and B on the soils I and II. The enzymes produced a high concentration of protein and observations suggest the enzymes behave like a surfactant, which effects its stabilization performance. The specimens were subjected to testing of varying lengths of time to determine their performance. Researchers observed an increase in the resilient modulus as the curing time increased but that an increase in application rate, as suggested by manufacturers, did not improve the performance of the enzymes. The study also suggests noticeable differences between the two enzymes and their effects on the soils in terms of resilient modulus and the stiffness of the soil.

Based on the analysis performed on the experimental data obtained in this study the following conclusions can be drawn:

1. The specimen preparation process showed that both product A and B reduced the compaction effort and improved soil workability. Thus, less pressure was used to obtain the target density of the treated specimens compare to the untreated specimens.
2. Enzyme A contains a high concentration of protein, but does not appear to contain active enzymes based on standard enzymatic activity tests. No chemical analysis was performed on enzyme B in this project.
3. The results from surface-tension testing and qualitative observations suggest that enzyme A behaves like a surfactant (reduction of the surface tension of water with the increase of enzyme concentration), contrary to the behavior observed in enzyme B.
4. The resilient modulus for soil I and II (cohesive soils) follow the trend found in the literature: decreases with the increase of the deviatoric cyclic stress and increases with the increase of the confining pressure.
5. The addition of enzyme A did not improve the resilient modulus of soil I but increased in average by 54% the resilient modulus of soil II.
6. The addition of enzyme B to soils I and II had a pronounced effect on the resilient modulus. The stiffness of soil I increased in average by 69% and for soil II by 77%.
7. The type of soil affected significantly the effectiveness of the treatments. The percent of fines and the chemical composition are properties that affect the stabilization mechanism. Therefore, special attention should be paid to select the proper treatment to be used for different soils.
8. The resilient modulus increased as the curing time increased for all the combination of soils and enzymes.
9. Increasing the application rate suggested by the manufacturer did not improve the effectiveness of the stabilization process.
10. Limited number of tests showed that at least four months of curing time are needed to observe an improvement on the shear strength of both soils.

11. Enzyme A increased in average the shear strength of soil I by 9%, and the shear strength of soil II by 23%.
12. Enzyme B increased in average the shear strength of soil I by 31% and of soil II by 39%.

The conclusions presented above refer to a limited number of soil and enzyme stabilizers combinations tested in laboratory conditions and should not be extrapolated to other combinations of materials. These results should be validated with field experiments that involve the same combination of materials used in this study. At this time, it is not known if the results obtained in this study accurately predict field performance. Assuming that the laboratory work reasonably predicts the field behavior, the following steps are recommended for practical applications:

- Obtain representative soil samples from the construction site and prepare enzyme modified soil specimens for laboratory testing following the manufacturer guidelines and the method proposed in this study
- Perform the tests described in this study on the enzyme modified specimens
- If the laboratory results show a significant improvement in the soil properties, use the product for field operations as indicated by the manufacturer.

4.7 Pond Ash/Lime/Phosphogypsum

4.7.1 Compaction Characteristics and Bearing Ratio of Pond Ash Stabilized with Lime and Phosphogypsum

Ghosh, A. (2010) "Compaction Characteristics and Bearing Ratio of Pond Ash Stabilized with Lime and Phosphogypsum." *J. Materials in Civil Eng*., 22(4) 343-351.

Recycling of waste material is one of the effective solutions of its disposal problem. Fly ash produced by coal-based thermal power plants and phosphogypsum (PG) produced by fertilizer plants producing phosphoric acid as constituent of fertilizers, take huge disposal area and creates environmental problems. Stabilization/solidification of fly ash improves the engineering properties and reduces the environmental problem like leaching and dusting. This paper presents the laboratory test results of a Class F pond ash alone and stabilized with varying percentages of lime (4, 6, and 10%) and PG (0.5, and 1.0), to study the suitability of stabilized pond ash for road base and subbase construction. Standard and modified Proctor compaction tests have been conducted to reveal the compaction characteristics of the stabilized pond ash. Bearing ratio tests have been conducted on specimens, compacted at maximum dry density and optimum moisture content obtained from standard Proctor compaction tests, cured for 7, 28, and 45 days. Both unsoaked and soaked bearing ratio tests have been conducted. This paper highlights the influence of lime content, PG content, and curing period on the bearing ratio of stabilized pond ash. The empirical model has been developed to estimate the bearing ratio for the stabilized mixes through multiple regression analysis. Linear empirical relationship has been presented herein to estimate soaked bearing ratio from unsoaked bearing ratio of stabilized pond ash. The experimental results indicate that pond ash-lime-PG mixes have potential for applications as road base and subbase materials.

The compaction and bearing ratio characteristics of pond ash stabilized with lime (4–10%) alone or in combination with PG (0.5 and 1.0%) were studied through laboratory experiments. The compaction tests were conducted for both standard and modified Proctor compaction energy. The specimens for bearing ratio tests were cured for 7, 28, and 45 days. Both unsoaked and soaked bearing ratio tests were conducted. Empirical model for a nondimensional parameter has been developed to find out unsoaked bearing ratio of stabilized pond ash as function of bearing ratio of unstabilized pond ash at 7 days curing, lime content, PG content, and CP. Linear relationship between unsoaked bearing ratio and soaked bearing ratio of the stabilized pond ash has also been developed.

The following conclusions may be drawn from the analysis of experimental results presented in this paper:

- The variation of dry density with moisture content of pond ash stabilized with lime and PG is similar to that of unstabilized pond ash in this study;
- With increase in compaction energy from standard to modified Proctor compaction energy, the MDD of stabilized pond ash increases and OMC decreases;
- Bearing ratio of stabilized pond ash increases with increase in lime content up to 10%, however the contribution of lime is more at the lower percentage _4%_ of lime addition to pond ash compare to that of 10% of lime addition;
- The contribution of lime is more prominent to enhance soaked bearing ratio of lime stabilized pond ash compare to that of unstabilized pond ash;
- Addition of small percentages of PG _0.5 and 1.0%_ to lime stabilized pond ash increases both soaked and unsoaked bearing ratio;
- Linear relationship exists between soaked and unsoaked bearing ratio of pond ash stabilized with lime and PG; and
- Pond ash stabilized with lime and PG may find potential application in road construction.

4.8 Lime

4.8.1 Accelerated Design Process of Lime-stabilized Clays

Celaya, M., Veisi, M., Nazarian, S., and Puppala, A. (2011) "Accelerated Design Process of Lime-stabilized Clays." *Geo-Frontiers 2011*, pp. 4468-4478.

Selection of the proper concentration of lime for clay stabilization is primarily based on achieving a target pH value. However, interaction between mineralogy of materials, curing processes and construction methods significantly impact the performance of stabilized clays. If the selected concentration of additives is not adequate to ensure short- and long-term strength and durability, stabilization will be ineffective. Many highway agencies complement the design process with other tests to ensure that proper strength, stiffness and durability are achieved. The most common parameter considered is the unconfined compressive strength of specimens that are cured and moisture conditioned for a number days. In this paper, several accelerated testing methods that could minimize the time required for specimen preparation, curing, conditioning and testing time to complete the stabilizer design process are evaluated.

The standard TxDOT protocol (Tex-121-E) was used to replicate the current specifications and conduct the mix design. Standard Protocol, Dry Protocol, Tube Suction Test (TST) Protocol, Back Pressure Conditioning Method, Vacuum Method, and Submergence Method.

The focus of this study was on different ways to accelerate moisture conditioning of lime stabilized clays, and reduce the time needed to complete the mix design as compared to current TxDOT specifications. The TST protocol was used as reference to isolate the impact of moisture conditioning alone. The following conclusions can be drawn:

- The TST and current protocols did not moisture condition the specimens homogenously in almost all cases. A number of the specimens became supersaturated at the bottom of the specimen, but remained dry on top.
- The distribution of moisture within the specimen was much more uniform from the vacuum and back-pressure methods.
- Back-pressure method provided similar results to TST after moisture conditioning method if water could penetrate throughout the length of the specimens.
- All proposed moisture conditioning methods could moisture condition the specimens in less than 24 h to saturation.
- The vacuum conditioning was the fastest method. However, the control of the vacuum suction other than 1 atmosphere is expensive. The back-pressure method is marginally slower but the equipment needed is rather simple and it is easier to control the test parameters. In addition, other information, such as permeability, can be obtained.
- Alternative methods generally resulted in lower strengths and moduli as compared to the standard method. This was attributed to the longer curing time imposed on the current specifications, and the lack of uniform and full saturation of the specimens subjected to capillary saturation for the standard and TST methods.

4.8.2 Short-Term Electrical Conductivity and Strength Development of Lime Kiln Dust Modified Soils

Chen, R., Drnevich, V. P., and Daita, R. K. (2009) "Short-term Electrical Conductivity and Strength Development of Lime Kiln Dust Modified Soils." *J. Geotechnical and Geoenvironmental Eng.*, 135(4) 590-594.

Lime kiln dust (LKD) is used for modifying pavement subgrades to expedite construction on wet clayey soils. This paper describes the short-term development (typically, over the first 3 to 7 days) of electrical conductivity and penetration resistance of LKD modified soils. The normalized net change of electrical conductivity is solely related to the LKD dosage. The decrease of electrical conductivity with time coincides with the increase of penetration resistance with time. The correlations of electrical conductivity with strength gain in LKD and lime-modified soils suggest that electrical conductivity measurements can potentially be useful for quality control in field applications.

Measurements of penetration resistance and electrical conductivity monitored the physicochemical changes of two LKD-modified clay soils with time. The observations and conclusions from this investigation include:

1. The addition of LKD into clay soils causes the electrical conductivity to increase. Higher LKD contents and higher water contents of the mixture both increase the initial electrical conductivity.
2. The normalized net reduction of electrical conductivity with time is solely related to the LKD dosage.
3. Most of the penetration resistance gain is achieved within 1 day of mixing and compacting. 4. The increase in penetration resistance is strongly related to the decrease of the electrical conductivity.
4. Future research will attempt to develop a model that relates the electrical conductivity, amount of lime or lime kiln dust and water content of a modified soil to soil strength increase and apply it to both laboratory and field situations.

4.9 Slag/Lime

4.9.1 Characterization of Cementitiously Stabilized Granular Materials for Pavement Design Using Unconfined Compression and IDT Testings with Internal Displacement Measurements

Piratheepan, J., Gnanendran, C. T., and Lo, S.-C. R. (2010) "Characterization of Cementitiously Stabilized Granular Materials for Pavement Design Using Unconfined Compression and IDT Testings with Internal Displacement Measurements." *J. Materials in Civil Eng*., 5(1) 495-505.

This paper presents the findings of a laboratory investigation on the characterization of a freshly quarried granular base material lightly stabilized with slag-lime cementitious binder involving unconfined compression (UC) testing and indirect diametrical tensile (IDT) testing, both with internal displacement measurements. The UC test investigation involved the determination of the unconfined compressive strength (UCS) and four different types of stiffness moduli from both internal and external displacement measurements. The IDT testing included the determination of IDT strength as well as the static and dynamic stiffness moduli (i.e., SSM and DSM) of the lightly stabilized granular base material from monotonic and cyclic load IDT testing. This study indicates that the stiffness moduli of a lightly cementitiously stabilized granular base material can be determined consistently from UC testing by measuring the deformations internally, and the modulus is more reliably defined as either the tangent modulus at half the ultimate stress or secant modulus at 0.02% strain. The UC stiffness modulus could be estimated reliably from the corresponding UCS value using the regression relationships established from this study. The IDT strength was determined to be equal to 0.1143 times the UCS value and this conforms to the recommendation given in design guides such as AUSTROADS. Moreover, the IDT, DSM, and SSM of a lightly cementitiously stabilized granular base material could be estimated from the UCS of the same material using the correlations developed in this paper which could be used in pavement design. Modulus values were noted to vary fairly significantly with respect to binder content and moisture content.

A detailed laboratory investigation was carried out to study the stress-strain behavior and the strength and stiffness moduli characteristics of a lightly cementitiously stabilized granular material using slag-lime from UC testing and monotonic and cyclic load IDT testing with internal displacement measurements. One of the objectives of this study was to investigate whether the stiffness characteristics in addition to UCS could be determined reliably from UC testing. This study was also aimed at developing relationships between the UCS and stiffness moduli from UC testing as well as the IDT strength and stiffness characteristics that could be used for pavement design.

The UC testing was conducted based on the AS-1141.51 (Australian Standard 1996) on 28 days cured samples prepared with a typical freshly quarried granular material stabilized by the addition of 3, 4, and 5% slag-lime slow-setting binder under 8–10% MC variations (i.e., OMC_1%). Details of the monotonic and cyclic load IDT testing program can be found in Gnanendran and Piratheepan _2008_ but a brief outline of the IDT testing program and pertinent results required for developing the correlations with UC test characteristics are presented in this paper. The significant findings are summarized as follows:

- The DDs of the UCS test samples of the granular base material stabilized with the addition of slag-lime slow-setting binder prepared adopting Standard Proctor compaction test procedure are highly influenced by the target MC. For the narrow range of MC (i.e., OMC ± 1%) and BC (i.e., 3–5%) variations that were investigated, the DD increased slightly with the increase in BC.
- For 1% increase in the BC of slag-lime binder there was a corresponding increase in the UCS of the stabilized material of around 24% for the narrow range of MCs (i.e., OMC ± 1%) that were investigated. The target MC also influenced the UCS values and it was found to be highest at OMC for all BC cases. This study further suggests that it is preferable to select the working MC range for a field stabilization project on the dry side of OMC rather than on the wet side.
- In general practice the target MC for the cementitiously stabilized materials is selected by arbitrarily adding 1–2% for the OMC of the parent material alone. However, this investigation suggests that the target MC should be selected from the DD-MC relationship for the stabilized material mix (i.e., the OMC with the cementitious binder added to the parent material) in order to achieve maximum strength and stiffness values for the stabilized material.
- This study indicates that the stiffness moduli of lightly cementitiously stabilized granular base materials can be determined consistently from UC testing by measuring the deformations internally and the modulus is more reliably defined as either the tangent modulus at half the ultimate stress or secant modulus at 0.02% strain. This study further suggests that the stiffness moduli should be determined based on the on-sample (internal) displacement measurements rather than from the external displacement measurements which leads to underestimation of stiffness moduli.
- This study suggests that there are no reliable simple relationships between DDR (i.e., DD/MDD) and UCS as well as between DDR and the different types of stiffness moduli from UC testing. However, somewhat less reliable ($R \approx 0.6$) linear relationships were obtained between DDR and UCS as well as between DDR and secant modulus at ultimate stress.
- Quite reliable regression relationships were developed between the different types of elastic moduli from UC testing and the UCS. This suggests that the UC stiffness modulus of a lightly cementitiously stabilized granular base material with a particular BC and MC could be estimated from its corresponding UCS value.
- Correlations were developed between the UCS and the strength and stiffness characteristics from monotonic and cyclic load IDT testing (i.e., the SSM and DSM from IDT). In particular, the IDT strength was determined to be equal to 0.1143 times the UCS

value and this conforms to the recommendation in AUSTROADS (2004). Moreover, the IDT DSM and SSM of a lightly cementitiously stabilized granular base material could be estimated from the UCS of the same material using the correlations developed in this investigation which could be used in pavement design. Since UCS is a relatively simple test, these relationships may be very useful and time saving.

4.10 Lime/Fly Ash/Cement Kiln Dust

4.10.1 An Assessment of Soil Parameters Governing Soil Strength Increases with Chemical Additives

Hussey, N. L., Cerato, A. B., Grasmick, J. G., Holderby, E. S., Miller, G. A., and Taber, W. (2010) "An Assessment of Soil Parameters Governing Soil Strength Increases with Chemical Additives." *GeoFlorida 2010*, pp. 2702-2711.

This study examined the relationship between soil physico-chemical parameters and unconfined strength in various fine-grained soils when mixed with chemical additives. To be considered effective, the soil-additive mixture must exhibit a strength increase of at least 345 kPa (50 psi). The research focused on AASHTO Soil Group Classifications falling under the fine-grained soil category (A-4 to A-7). Additive amounts are given for Fly Ash (FA), Cement Kiln Dust (CKD), and Lime (hydrated and quick lime).

The results of this study suggest that the surface area is perhaps the most important factor in determining if a chemical additive will be effective for stabilization. Out of the eight tested soils, the seven soils with surface areas lower than 150 m2/g reached the 345 kPa strength gain with the different additives. However, the one soil with a surface area higher than 150 m2/g never reached the strength gain with any of the additives used at any of the tested percentages.

Analysis and testing of the soil samples consists of three methods: compaction, unconfined compression tests (UCT) and Atterberg limits.

The use of unconfined compression tests to measure strength gain of stabilized soils is the first step in completely characterizing the behavior of stabilized sub-bases. In order to make a more reliable method for determining the type and amount of an additive to add to a certain soil, we must also look at the stiffness of the soil so as to ensure the longevity of pavement placed over these sub-grades. We must also determine alternative soil parameters that better predict stabilized soil behavior because based on the aforementioned results, the commonly used Atterberg Limits do not always provide adequate information about soil behavior when stabilizing and strengthening problematic soils within the soil classifications A-4 to A-7-6. In several instances, such as the A-6 soils with hydrated lime and the A-4 soils with fly ash, the recommended additive amount was several percentage points higher than what was determined to be needed to achieve the requisite unconfined compression strength increase of 345 kPa, and in some cases, such as with the A-7-6 soil, no amount of stabilizer was adequate in strengthening the soil. Therefore, it is imperative for engineers to determine an adequate, easily measurable soil parameter or several parameters that will better indicate soil behavior after stabilization. Specific Surface Area (SSA) and Cation Exchange Capacity (CEC), are two of the parameters being investigated. The surface area will be tested according to Cerato and Lutenegger (2002).

After comparing the initial properties of each soil type with the resulting strength gains from the different additives, it seems that the SSA of the soil is one of the most important parameters in determining the amount of a chemical additive to use in stabilization. All the tested soils with surface areas less than approximately 150 m2/g reached the necessary strength gain with at least one stabilizer, but the one soil with a surface area higher than 150 m2/g (Hollywood at 220 m2/g) never reached the 345 kPa strength gain with any of the additives at any of the tested percentages and the Anadarko County soil did not reach the needed strength gain with fly ash. This is a promising correlation and SSA plus other fundamental soil properties should be further investigated in light of these findings to determine if they can be used in conjunction with the AASHTO classification system to better advise engineers on what type and amount of stabilizer to use on roadway projects. This improved design protocol will save not only time and money in the sub-grade and pavement design phase, but time and money during the lifecycle of the roadway from lowered maintenance costs because of a stronger sub-base and pavement.

The graphs in the reference provided the stiffness data that are represented in the summary tables. A visual approximation was taken in order to acquire the tabulated data. The stiffness factor could only be calculated for the soils with properties presented in the report. The stiffness is typically observed to increase to an optimum content, where it then levels or decreases, with increasing additives.

As seen in the summary table, the factor of improvement with respect to strength was approximately 2 for fly ash and cement kiln dust contents of 5% and 6%, respectively, for the Heiden soil. (The Heiden soil was the only one tested at relatively low fly ash and kiln dust contents.) Factors for lime stabilization are also provided in the third section of the table.

Table 4.10. Summary of strength improvement for Hussey et. al. (2010) study.

Fly Ash (%)	Devol UCS (kPa)	Devol Strength Factor	Anadarko UCS (kPa)	Anadarko Strength Factor	Payne UCS (kPa)	Payne Strength Factor	Hollywood UCS (kPa)	Hollywood Strength Factor	Heiden UCS (kPa)	Heiden Strength Factor
0	110	-	95	-	210	-	350	-	300	-
5	-	-	-	-	-	-	-	-	680	2.27
6	-	-	-	-	-	-	540	1.54	-	-
7	-	-	-	-	-	-	625	1.79	725	2.42
8	-	-	-	-	-	-	630	1.80	-	-
9	-	-	205	2.16	605	2.88	720	2.06	770	2.57
12	215	1.95	225	2.37	625	2.98	835	2.39	920	3.07
15	310	2.82	200	2.11	400	1.90	980	2.80	975	3.25

CKD (%)	Devol UCS (kPa)	Devol Strength Factor	Anadarko UCS (kPa)	Anadarko Strength Factor	Payne UCS (kPa)	Payne Strength Factor	Flower UCS (kPa)	Flower Strength Factor	Ashport UCS (kPa)	Ashport Strength Factor	Kirkland UCS (kPa)	Kirkland Strength Factor
0	110	-	95	-	210	-	285	-	210	-	250	-
6	-	-	-	-	-	-	350		-	-	-	-
7	-	-	-	-	-	-	410		595	2.83	520	2.08
8	470	4.27	1000	10.53	1210	5.76	700	2.46	-	-	-	-
9	-	-	-	-	-	-	870	3.05	765	3.64	800	3.20
10	670	6.09	1250	13.16	1320	6.29	-	-	-	-	-	-
11	-	-	-	-	-	-	-	-	870	4.14	930	3.72
12	795	7.23	970	10.21	1110	5.29	1200	4.21	-	-	-	-
15	-	-	-	-	-	-	1300	4.56	-	-	-	-

Lime (%)	Flower UCS (kPa)	Flower Strength Factor	Ashport UCS (kPa)	Ashport Strength Factor	Kirkland UCS (kPa)	Kirkland Strength Factor	Hollywood UCS (kPa)	Hollywood Strength Factor	Heiden UCS (kPa)	Heiden Strength Factor
0	285	-	210	-	250	-	350	-	300	-
1	320	1.12	-	-	-	-	360	1.03	600	2.00
2	480	1.50	690	3.29	1310	5.24	590	1.69	1030	3.43
3	630	1.31	-	-	-	-	805	2.30	1650	5.50
4	715	1.13	630	3.00	1500	6.00	815	2.33	1535	5.12
5	810	1.13	640	3.05	1330	5.32	795	2.27	1420	4.73
6	-	-	640	3.05	1255	5.02	-	-	-	-

4.10.2 Engineering Properties and Moisture Susceptibility of Silty Clay Stabilized with Lime, Class C Fly Ash, and Cement Kiln Dust

Solanki, P., Khoury, N., and Zaman, M. M. (2009) "Energy Properties and Moisture Susceptibility of Silty Clay Stabilized with Lime, Class C Fly Ash, and Cement Kiln Dust." *J. Materials in Civil Eng*., 21(12) 749-757.

A laboratory study was undertaken to evaluate the effectiveness of different percentages of hydrated lime, class C fly ash (CFA), and cement kiln dust (CKD) as soil stabilizers. Cylindrical specimens were compacted and cured for 28 days in a humidity room having a constant temperature and controlled humidity. At the end of the curing period, specimens were tested for resilient modulus (M_r), modulus of elasticity (ME), and moisture susceptibility using tube suction test. The study revealed that the values of M_r, ME, and unconfined compressive strength

(UCS) for the stabilized specimens increased with the increase in the amount of the stabilizing agent. It was also found that the increase in M_r, ME, and UCS values varies with the type of stabilizing agents. The CKD-stabilized specimens exhibited a higher increase in M_r, ME, and UCS values than the corresponding values of lime- and CFA-stabilized specimens. Additionally, CKD-stabilized specimens exhibited the least moisture susceptibility in terms of lowest dielectric values as compared to lime- and CFA-stabilized specimens. Table 4 in the paper provides a nice summary of the range of values obtained.

This study was undertaken to evaluate the effect of different percentages of lime, CFA and CKD on the engineering properties of soil. An increase in OMC and a decrease in MDD were observed with increasing amount of lime and CKD. All three stabilizers improved resilient modulus, UCS and modulus of elasticity values, with CKD treatment providing the maximum enhancements to soil properties. The TST results revealed that CKD-stabilized specimens are least susceptible to moisture as compared to limeand CFA-stabilized specimens. Regression equations were developed for the lime-, CFA- and CKD-stabilized soil to estimate M_r values. Predicted values were well correlated with measured values. To conclude, CKD as an additive provided highly effective soil stabilization as compared to lime and CFA, usually with the added benefit of highest strength-stiffness gain and least moisture susceptibility.

This study evaluated only three (strength, stiffness and durability) out of the required four categories that have been identified as key to performance (Mallela 2004). Further study is needed to evaluate and compare the fatigue fracture of subgrade soils stabilized with lime, CFA and CKD, for an overall pavement performance evaluation.\

The resilient modulus was calculated from the tabulated test results. The final modulus is taken to be the average value of the fifteen test values for each sample. The modulus factor was then calculated. The modulus factor is typically observed to increase with the addition of various additives. The soil under consideration was a CL-ML material (low plasticity clay/silt), as classified by USCS.

Table 4.11. Summary of stiffness improvement for Solanki et. al. (2009) study.

Soil Additive	Raw	3% lime	6% Lime	9% Lime	5% CFA	10% CFA	15% CFA	5% CKD	10% CKD	15% CKD
Resilient Modulus (Mpa)	127.87	773.93	759.14	857.00	202.67	586.60	1398.07	528.33	2136.92	2452.83
Modulus Factor	-	6.05	5.94	6.70	1.58	4.59	10.93	4.13	16.71	19.18

As seen in the table, the factor of improvement with respect to stiffness was about 1.6 for 5% fly ash content and about 4 for 5% cement kiln dust content.

4.10.3 Factors Influencing the Strength of Cement Fly Ash Base Courses

Kaniraj, S. R., and Gayathri, V. (2003) "Factors Influencing the Strength of Cement Fly Ash Base Courses." *J. Transportation Eng.*, 129(5) 538-548.

Fly ash is a waste produced in coal-fired thermal power stations. It has pozzolonic properties and can therefore be stabilized with either cement or lime to achieve the strength required for use as base courses in pavements. Agencies such as the Electric Power Research Institute (EPRI) have specified criteria and guidelines for the determination of the stabilizer content. This requires carrying out unconfined compression tests on stabilized fly ash specimens prepared and cured as per standard procedures. The stabilizer content is the minimum amount of the stabilizer for which the unconfined compressive strength of the specimens complies with the specified values. The actual curing conditions of the stabilized fly ash bases in the field, however, will differ from those of the laboratory specimens. This will affect the strength development of the bases, their durability, and their performance. The paper explains the details and results of a laboratory experimental program carried out to study the influence of curing conditions and other factors on the development of strength. The program comprised compaction tests and unconfined compression tests. Two Indian fly ashes and a commercial portland cement were used in the study. Six different curing conditions, including controlled and ambient conditions, were adopted. The influence of differences in the dry unit weight and water content was also investigated.

A laboratory experimental study was carried out to investigate the effect of cement content, curing period, controlled and uncontrolled ambient conditions of curing, unit weight, and water content on the development of the strength of cement stabilized class F fly ashes with reference to their use as pavement base courses. The following are the main conclusions from the study:

1. In the determination of fly ash-cement design mixes for pavement base courses, attention should be paid to the watercement ratio also, particularly for the high MDD/low OMC fly ashes. By appropriate selection of dry unit weight and degree of saturation of the mixes, it may be possible to achieve the required strength by providing adequate water for hydration of cement. More experimental studies are needed to understand the influence of water-cement ratio on the stabilization of fly ashes with cement.
2. For any cement content, the unconfined compressive strength ~UCS! increased till a certain curing period and then tended to decrease. However, for any cement content the rate of change in strength decreased as the curing period increased. The rate of increase in strength was high till about 14 days, decreased drastically during 28–90 days, and became very small beyond 90 days.
3. Immersion of the specimens in water before the unconfined compression test increased their water contents and decreased their strength. The increase in water content depended on the cement content. The average increase in water content varied from 9 to 11% in specimens with cement contents of 4–10%, while in the 18% cement content specimens it was 7%. The average loss of strength due to immersion was about 11%.
4. A decrease in unit weight reduced the UCS of the specimens. The decrease was more when there was a reduction in the water content also. The lower water content specimens also absorbed more water upon immersion. When the fly ashcement mixtures are compacted at less than MDD values, because of the qualities of higher strength and lesser water absorption, compacting these mixtures at wet-of-optimum moisture content appears to be a better option than compacting them at dry-of-optimum moisture content.
5. The UCS of the specimens depended on the method of curing. The differences between the UCS of the specimens cured by the STD method and the other methods were minimum in the case of the ASTM method. The ASTM method produced a slightly lower

UCS than the STD method. The HYD method of curing generally produced lower values of UCS than the ASTM method. The strength developed was the least in the ATM method of curing, where the specimens were exposed to the ambient atmospheric elements of light, temperature, rain, and humidity.

6. Studies made by curing the specimens in the uncontrolled ambient temperature conditions of the summer, monsoon, and winter seasons showed that the strength was generally more than that obtained in the STD method of curing when the ambient temperature was higher than the STD method and the loss of moisture was prevented during curing, as was the case in the NAT-SUM and NAT-MON methods. The increase in strength was more during summer than the monsoon season because of the higher temperature during these seasons. Conversely, the specimens cured by the NAT-WIN method developed significantly lower strengths than the STD method, as the ambient temperature was lower than that in the STD method.

4.10.4 Performance of Soil Stabilization Agents

Milburn, H. P., and Parson, R. L. "Performance of Soil Stabilization Agents." *K-TRAN: KU-01-8*, Kansas Department of Transportation, 2004.

Poor subgrade soil conditions can result in inadequate pavement support and reduce pavement life. Soils may be improved through the addition of chemical or cementitious additives. Such chemical additives range from waste products to manufactured materials and include lime, Class C fly ash, Portland cement and proprietary chemical stabilizers. These additives can be used with a variety of soils to help improve their native engineering properties.

This report contains a summary of the performance of lime, cement, Class C fly ash, and Permazyme 11-X used with a wide range of soils. Each of the chemical additives tested is designed to combine with the soil to improve the texture, increase strength and reduce swell characteristics. These products were combined with a total eight different soils with classifications of CH, CL, ML, SM, and SP. Durability testing procedures included freeze-thaw, wet-dry, and leach testing. Atterberg limits and strength tests were also conducted before and after selected durability tests. Changes in pH were monitored during leaching. Relative values of soil stiffness were also tracked over a 28-day curing period using the soil stiffness gauge. Lime and cement stabilized soils showed the most improvement in soil performance for multiple soils, with fly ash treated soils showing substantial improvement. The results showed that for many soils more than one stabilization option may be effective for the construction of durable subgrades. The enzymatic stabilizer did not perform as well as the other stabilization alternatives.

It is recommended, based on the results of this research, that some testing of the contribution of proposed stabilization agents be conducted prior to construction. For pavement designs that expect a relatively limited strength contribution from the soil, the primary anticipated benefit of stabilization is generally the control of volume change.

The following conclusions were reached based on the interpretation of the results of this study.

General Conclusions

1. Lime, fly ash and cement were effective in improving the Atterberg limits on the soils used in the study. Each soil showed improvements in plasticity with each additive tested, however fly-ash- and cement-treated samples generally retained some plasticity. Lime-treated soils had the greatest improvement with all soils becoming non-plastic with the addition of sufficient amounts of lime.
2. The native swell values for the CL soils were lowered dramatically with lime, fly ash and cement. The reaction of the calcium-based additives with the sulfate-bearing CH soils resulted in swelling similar to or higher than the native state. Additional mellowing prior to compaction to allow the sulfates to react with the calcium-based additives helped to lower the sulfate-related swelling.
3. Significant strength improvements were observed for soils treated with lime, fly ash, and cement while enzyme-treated soils showed modest strength gains. Most of these strength gains were retained after freezethaw and leaching. Cement- and lime-treated soils generally retained the most strength after leaching while fly-ash-treated soils retained some of their strength gains.
4. The cement treated soils had the least soil loss in freeze-thaw testing, while fly ash treated soils had lower soil losses in freeze-thaw testing than lime treated soils. Relative performance in the wet-dry cycles was mixed, with lime generally performing better on fine-grained materials and cement on coarse-grained soils, although cement performed relatively well with the CH clays. Fly ash performed well only on the SM soil, where it survived the full 12 cycles. The enzyme-treated soils had soil losses similar to those treated with fly ash in freeze-thaw testing but did not survive the first cycle of wet-dry testing.
5. Lime- and cement-treated soils maintained higher strengths and lower plasticity values than fly ash treated soils after leaching. Higher permeability, lower pH, and higher plasticity values after leaching and relatively constant soil modulus values for cured samples suggests that flyash- treated soils may not be experiencing the formation of additional interparticle bonds over time, and that the improvements in soil properties with fly ash may be reversed to some degree by leaching. The pH va lues also declined for cement treated soils after 21 days of leaching, which may indicate a reduction in pozzolanic reactions. However, permeability values also declined during the first 21 days, suggesting that some reactions may have been occurring during that period.
6. The enzymatic stabilizer did not substantially improve soil performance when evaluated using the test regimen described.
7. The improvements in soil properties reported were observed under laboratory conditions. Less thorough mixing resulting in larger soil lump sizes, as may occur in the field, could result in less effective stabilization as shown by Petry (29).

Conclusions Based on the Soil Stiffness Gauge

1. The soil stiffness gauge can be used to monitor changes in the stiffness of standard samples of stabilized soils.
2. A moisture-stiffness relationship exists for most stabilized soils and can be evaluated with the soil stiffness gauge.

3. Lime- and cement-stabilized samples resulted in samples with a higher stiffness than the fly-ash-treated samples. Permazyme-stabilized samples had the lowest recorded stiffness values of the soils that had been stabilized.
4. Lime- and cement-stabilized samples showed strength gains over the 28-day testing period, suggesting that stabilization reactions and strength gains were ongoing. Most of the fly ash stiffness gains were achieved very early in the curing process with little additional gains over time. This observation, in addition to the leaching results, suggests that for LaCygne fly ash the most significant portion of the stabilization reactions occur very quickly after mixing.

Basis for Selecting the Additive

The procedure adopted for the selection of the most appropriate soil additive should be a function of the expected contribution of the stabilized soil to the pavement system. For pavement designs that expect a relatively limited contribution from the soil, the primary benefit of stabilization is generally the control of volume change. For these conditions it is recommended that additive selection should be based on the ability of the additive to control shrink/swell behavior.

It is recommended that the evaluation of the potential for volume change be done using swell tests. Test methods for swell evaluation could include the existing KDOT volume change method, ASTM D 4829 Standard Test Method for Expansion Index or Soils, ASTM D 3877 Standard Test Methods for One-Dimensional Expansion, Shrinkage, and Uplift Pressure of Soil- Lime Mixtures, or ASTM D 3668 Standard Test Method for Bearing Ratio of Laboratory Compacted Soil- Lime Mixtures. Other test methods have also been developed that could be used.

Use of Atterberg limits for the evaluation of the effect of additives on swell potential should be used with caution, as Atterberg limits will often not fully reflect the contribution of fly ash and cement to swell control.

Lime, fly ash, and cement were all successfully used to limit swell for soils other than CH clays. If control of volume change is the only criteria, it is recommended that selection of additives for treatment of these soils be an economic decision. Selection of the additive could be left as an option to the contractor, after treatment percentages for each additive have been established.

It is recommended that selection of additives for CH soils be done with special care because of their higher potential for significant volume change. There are alternatives to lime, particularly cement, that can be effective for these soils. However, it is recommended that testing be conducted to confirm that a sufficient percentage of additive is specified. It is possible that effective treatment options will be eliminated due to the treatment expense. It was observed that cement became economically less competitive with decreasing grain size and higher plasticity, as more cement was required.

It is likely that stabilized subgrades are providing a significant contribution to the pavement system. If the design procedure is altered to account for this contribution, then strength and durability testing should be included as a part of developing the additive specifications. For this purpose, wet-dry testing may be more appropriate than freeze-thaw testing, as it is likely to be the more common environmental condition. Leaching may also provide guidance to long-term performance, however it must be done with care to realistically model field conditions.

Permazyme is not recommended as a stabilization additive, as it did not perform as well as the other additives for the test program described.

Determining the Amount of Additive to Use

The results from this research show that the amount of a given additive required to stabilize a soil varies with soil type, which is consistent with previously published work. It is therefore recommended that KDOT consider implementation of a procedure for establishing the percentage of additive to use based on the soil to be compacted.

The quantity of additive to use can be estimated by comparison with similar soils from this study, by published correlations, as shown for lime in Figure 7.1, or directly by various test methods, such as ASTM D 6276 (for lime). Guidelines from the Portland Cement Association were adequate to stabilize the silty soils used in this research. The cement modified CL and CH soils performed well, but additional amounts of cement may have resulted in additional durability performance improvements. Selection of the fly ash percentages was the least sophisticated and it is recommended that the fly ash percentages be further researched to better optimize the percentage specified for use.

Sulfate-bearing Soils

Sulfate-bearing soils are present in Kansas. Reactions between calcium hydroxide and sulfates can produce expansive minerals that can result in an *increase* in the swell potential as was shown with the Beto “Tan” soil. Tests for identifying the presence of sulfates both in lab samples and in the field are available and more are under development. It is recommended that KDOT investigate the feasibility of incorporating one or more methods for identifying the presence of sulfates in subgrade soils as a part of the soil characterization process.

For those soils where sulfates are present, it is recommended that KDOT consider modifying subgrade construction/stabilization procedures to account for the formation of expansive minerals. Expansive minerals may form with any calcium-based stabilizer. Construction amendments could include a double treatment with lime and an extended mellowing period between treatments to allow for the formation of expansive minerals prior to compaction and trimming. The National Lime Association has published more detailed recommendations on the treatment of these soils (27).

4.10.5 Reclaimed Hydrated Fly Ash as a Geomaterial

White, D. J. "Reclaimed Hydrated Fly Ash as a Geomaterial." *J. Materials in Civil Eng*., 18(2) 206-213.

To effectively use recycled geomaterials in earthwork and pavement base/subgrade construction, engineering properties, design values, proper construction practices, and long-term behavior must be known. Recent experience in Iowa reveals that reclaimed hydrated fly ash (HFA) can be designed and constructed to meet performance objectives as demonstrated by three field test projects where HFA materials were used in construction of structural pavement base layers. In addition to construction operations and field performance monitoring, this paper summarizes HFA engineering properties determined from laboratory tests including: compaction characteristics, shear strength parameter values, hydraulic conductivity, freeze–thaw durability, and microstructural features. Results are also presented for HFA materials activated with high calcium stabilizers including hydrated lime, cement kiln dust, Class C fly ash, and atmospheric fluidized bed combustion residue. Successful use of these materials follows 10 years of research and field performance monitoring in Iowa.

Reclaimed hydrated fly ash is being used in Iowa as an alternative geomaterial for pavement base construction. This paper summarizes engineering properties determined from laboratory measurements, typical field construction practices, and performance monitoring results for three test roads. Engineering property evaluations from laboratory measurements show that HFA materials have relatively high strength with low permeability and poor freeze–thaw durability. If stabilized with calcium activators such as hydrated lime, CKD, and Class C fly ash, the strength and freeze–thaw durability increase. Design values are provided for CBR, unconfined compressive strength and layer coefficients to determine pavement base thicknesses.

Field performance monitoring shows that test roads with HFA bases are performing well, however, because of the delamination and lower strength observed, it is recommended that AFBC not be used as a stabilizer in future HFA bases.

4.10.6 Soil Stabilization Field Trial

George, K. P. "Soil Stabilization Field Trial." *FHWA/MS-DOT-RD-05-133*, Mississippi Department of Transportation, 2006.

A five-year study was initiated seeking materials/additives and procedures that help to mitigate crack susceptibility in cement-treated material (CTM). A field test program of six 305-m (1000-ft) test sections was implemented in August 2000. The following additives/procedures were included for investigation:

- 5.5% cement additive (control section); design based on a reduced strength criteria.
- 5.5% cement precracked 24 to 48 hours after finishing.
- 5.5% cement precut (grooved) every 3 m (10 ft).
- 3.5% cement with 8% fly ash (CFA).
- 6% ground granulated blast furnace slag (GGBFS) with 2% lime admixture (LGBFS).
- 3% lime and 12% fly ash; stabilization technique used by MDOT (LFA).

First interim report covering the first phase of investigation/monitoring during the 28-day period was submitted on April 21, 2001. Two layers of hot mix asphalt (HMA) – 110 mm (4.5 inch) base, 60 mm (2.25 inch) polymer modified binder – were placed over the stabilized layer beginning September 21, 2000, followed by the second field monitoring on November 13, 2001. Field tests included deflection tests employing Falling Weight Deflectometer (FWD), retrieval of 100-mm (4-inch) cores for compression tests, and a manual crack survey. The results were presented in Interim Report II. On June 16, 2003, (nominally 3 years) the test sections were monitored; this time again deflection test employing FWD, and a manual crack survey. Prior to the June 2003 survey, a 50-mm (2- inch) polymer modified surface course was placed, with the road opening to traffic on July 8, 2002.

Nominally five years after construction, again deflection tests deploying FWD (December 1, 2004), compression tests on 102-mm (4-inch) cores and a manual crack survey (March 8, 2005) were conducted. Presented in this final report are, (i) the results of deflection analysis and moduli of layers (ii) the compressive strength results of 102-mm (4-inch) diameter cores, and (iii) the crack survey results.

Backcalculation of moduli from deflection data was accomplished by deploying MODULUS v.6, with pavement modeled as a four-layer system and in few cases, as a three-year system as well. The backcalculated results show that the moduli of all of the sections, except that of the CFA, increased steadily from 28 days to 1654 days. In CFA, however, the modulus was not only relatively low but it also leveled off after 440 days. In the LFA section, modulus remained significantly low in the beginning and continued at a low level over the five-year period. Unconfined compressive strength (UCS) determined from 102-mm (4 inch) diameter cores consistently increased with time in all of the six mixes. The strength gain of the 5.5% cement control mix leveled off after 440 days, thus not attaining the target strength of 2070 kPa (300 psi). Lime-fly ash mix strength was indeed low compared to those of the other mixes. With 220 mm (8.75 inches) of HMA overlay, no reflection cracks were observed throughout the five-year monitoring period.

For a comparative evaluation of the six sections, their short- and long-term performance had been examined; short-term performance in terms of 28-day shrinkage cracks in the base layer and long-term performance in terms of stiffness modulus and UCS. Though considered satisfactory in regard to shrinkage cracks, the long-term performance of LFA mix is suspect as evidenced by its low stiffness, and in turn, large deflection. Though structurally adequate, based on the questionable short term performance of both CFA and LGBFS mixtures, their use in flexible pavement beneath HMA, especially thin layers, (102 mm (4 inches) or less) is deferred. Mixing two additives in small proportions is another construction-related problem in the CFA and LGBFS mixtures. The control CTM with 5.5% cement not only suffered excessive shrinkage cracking, but also its long term strength fell short of expectation. The precut CTM though structurally sound, two problems dissuade its application: the excessive shrinkage cracking, and logistics of cutting grooves while the layer is being compacted. From the point of view of overall performance, precracked CTM indeed excelled all of the other treatments/admixtures and, therefore, is recommended for stabilization of base layers. Modulus values as backcalculated from FWD tests were shown over a period of five years, showing the change in modulus with time at various sections. These data are given in Table 3.3 through 3.5 of the reference.

Investigations during construction, and evaluation tests thereafter for a period of five years reveal that large variation in compaction and moisture is real, attributable to inherent difficulties of in-place mixing and compacting. Owing partly to inadequate mixing, field mixed material strength on average was 50% lower than that of the laboratory mixed material. Mix non-uniformity was pronounced when two additives were employed, for example, cement and fly ash combination. It was discovered while coring that lower reach of the stabilized section was deficient in stabilizer chemical which only caused partial disintegration of core samples. The low modulus of CFA over the five-year period (except the 28-day value), and the slightly large FWD deflection are cited here in support of this premise. The strength gain over the monitoring period of all of the sections are satisfactory, one exception being the cement control section, whose 1564-day UCS turns out to be 1730 kPa (250 psi), falling short of the design strength, 2070 kPa (300 psi).

An overall comparison of performance of all of the sections based on shrinkage cracking (referred to as short-term performance) and stiffness and strength (signifying long-term performance) is presented in Table 5.1. Judging short term performance in terms of 28-day cracks, sections #2 and #6 have out-shined the other four sections. Indeed, sections #1, #3, #4 and #5 suffered excessive shrinkage. The long term performance of LFA section is suspect, as evidenced by its relatively large deflection, due in part to the LFA base not attaining the expected strength/stiffness. Though the shrinkage cracking of CFA section was excessive, and the FWD deflection of the section slightly larger than that of the control section, its structural performance so far is on target. Mixing problems in incorporating two admixtures and attendant weakness of the base layer could have been the primary reason for the enhanced deflection. This mixing problem existed in lime-GGBFS mixture as well; nonetheless, its adverse effect on strength and stiffness seems to be minimal. The use of these two mixtures – CFA and lime-GGBFS – is deferred until the shrinkage cracking problem is addressed adequately.

The three remaining admixtures/treatments include the low-strength cement-treated material (5.5% cement admixture), an identical CTM receiving precutting during construction, and again the same cement admixture subjected to precracking 24 hours after completion of construction. The 5.5% cement mixture, designated control mix, not only suffered excessive shrinkage cracking (17.2%) but also its long-term strength gain fell short of expectations. The precut cement mixture, though structurally sound with adequate long-term strength, it underwent shrinkage cracking to the tune of 13.9%, which is considered excessive. From the point of view of overall performance, precracked material indeed excelled all of the other treatments/admixtures.

The results of the study show that the stabilized base layers perform satisfactorily if the overlying HMA is sufficiently thick (say, more than 152 mm (6 inches)). Should the HMA layer be 102 mm (4 inches) or less, early shrinkage cracking becomes an issue, and should be addressed. Toward this end, early strength (7-day strength) should be limited to 2070 kPa (300 psi) in conjuction with some form of conditioning implemented in the constructed layer. Of the two conditioning techniques experimented in this study, precracking the stabilized cement layer was highly successful and, therefore, recommended for implementation. Tentative specifications for constructing precracked CTM layers have been developed by the Texas Transportation Institute, which may be referred for guidance (3). The performance of the other section with precut was not entirely satisfactory. Considering the complexity of implementing this procedure, precutting freshly laid cement layer cannot be recommended at this time.

Cement-fly ash not only failed to show improvements in shrinkage cracks, but gain in bending strength, as judged by elastic modulus, suffered as well, due in part to non-uniform mixing of additives. This combination of two admixtures, therefore, cannot be recommended. The shrinkage cracking performance of lime-GGBFS combination was less than satisfactory, however, the structural performance of the mix deemed to be above average. Its use, therefore, could be promoted should the design warrant a thick HMA layer. As precracking this material is likely to alleviate shrinkage cracking problems, a recommendation would be to conduct a study incorporating this feature in lime-GGBFS material and its adoption conditioned upon the success of the project. Very effective in mitigating shrinkage cracks and also being able to preserve long-term strength and stiffness, the precracked CTM (7-day strength (2070 kPa (300 psi)) emerges as a clear choice for pavement applications.

4.11 Lime/Fly Ash

4.11.1 Behavior of Compacted Soil-Fly Ash-Carbide Lime Mixtures

Consoli, N. C., Prietto, P. D. M., Carraro, J. A. H., and Heineck, K. S. (2001). "Behavior of Compacted Soil-Fly Ash-Carbide Lime Mixtures." *J. Geotechnical Geoenvironmental Eng.*, 127(9) 774-782.

Unconfined compression tests, Brazilian tensile tests, and saturated drained triaxial compression tests with local strain measurement were carried out to evaluate the stress-strain behavior of a sandy soil improved through the addition of carbide lime and fly ash. The effects of initial and pozzolanic reactions were investigated. The addition of carbide lime to the soil-fly ash mixture caused short-term changes due to initial reactions, inducing increases in the friction angle, in the cohesive intercept, and in the average modulus. Such improvement might be of fundamental importance to allow site workability and speeding construction purposes. In addition, under the effect of initial reactions, the maximum triaxial stiffness occurred for specimens molded on the dry side of the optimum moisture content, while the maximum strength occurred at the optimum moisture content. After 28 days, pozzolanic reactions magnified brittleness and further increased triaxial peak strength and stiffness; the maximum triaxial strength and stiffness occurred on the dry side of the optimum moisture content.

An extensive laboratory testing program was carried out to investigate the effectiveness of using industrial by-products such as carbide lime (a residue derived from the manufacturing of acetylene gas) and thermal power plant fly ash (a residue from coal burning) to improve the engineering behavior of a weathered sandstone soil, prepared using a variety of curing and compaction conditions. The observations and conclusions can be summarized as follows:

- The addition of carbide lime significantly improved strength and stiffness properties of the soil, even considering the nonplastic characteristic of the silty sand utilized. However, the presence of fly ash is fundamental to further improve the material behavior, due essentially to the occurrence of a larger amount of time-dependent pozzolanic reactions.
- Factors such as curing temperature, compressive and tensile strength mobilization rate, and compaction parameters, which definitely affect the stress-strain-strength behavior of the soil-fly ash-carbide lime mixture with time, have to be cautiously taken into account when designing or executing ground works with such material.
- Unconfined compression and Brazilian tensile test results showed that, for the temperature of curing selected (227C), a gain in strength largely occurs after 90 days, probably due to an induction period for the pozzolanic reactions between lime and fly ash. Undoubtedly, this might be a drawback to the practical utilization of the stabilized soil. However, at a greater temperature of curing, which is reasonable for most of the year in tropical and subtropical regions, a reduction in the induction period is expected to occur.

- Based on triaxial compression tests, it can be stated that, after a short delay following compaction, the maximum stiffness occurs for specimens compacted on the dry side of the optimum moisture content and the maximum strength at the optimum moisture content. After 28 days, maximum strength and stiffness occur on the dry side of the optimum moisture content. Such behavior is suggested to result from the coupled effects of the two main contributing factors to the stress-strain response of the fly ashcarbide lime stabilized soil, namely, the structure imparted by compaction, mainly density and packing, which predominates in the short-term, and the formation of a cementitious matrix, which predominates after pozzolanic reactions have been developed.
- The results indicate that, for the soil-fly ash-carbide lime investigated, the compaction should be performed about 2% to the dry side of the optimum moisture content obtained from a standard Proctor compaction test. On this particular issue, further studies considering different types of soil are necessary.

The present work has been envisaged as a contribution to the field of ground improvement and soil stabilization by discussing some fundamental aspects of a soil-fly ash-carbide lime mixture behavior, such as the short-term changes due to initial reactions, which is of fundamental importance to allow site workability, and the influence of compaction parameters on strength and stiffness properties. Experimental evidences, obtained from high quality triaxial tests, were useful in identifying patterns from which the stress-strain behavior of the stabilized soil can be characterized. Finally, it is necessary to emphasize that the present study was constrained to the range of low confining stresses, making it attractive to a great range of problems such as shallow foundations assented on improved layers and pavement structures.

4.11.2 Characterization of Lime and Fly Ash-Stabilized Soil by Indirect Tensile Testing

Solanki, P., and Zaman, M. "Characterization of Lime and Fly Ash-Stabilized Soil by Indirect Tensile Testing." *Geo-Frontiers 2011*, pp. 4438-4448.

A laboratory study was undertaken to evaluate the indirect tensile characteristics of soil specimens stabilized with lime and fly ash. Cylindrical specimens were compacted and cured for 28 days in a moist room maintained at constant temperature and controlled humidity. After curing specimens were tested for indirect tensile strength and resilient modulus (Mrt) under indirect tension mode. These properties were compared with those of the control soil specimens (raw clay) to determine the extent of enhancement. Results indicate increase in indirect tensile strength and Mrt values for stabilized specimens. However, the degree of improvement varies and depends on type of soil and additive.

This study was undertaken to evaluate two soils namely, P-soil (silty clay) and V-soil (lean clay) from Oklahoma for the effect of lime and CFA on the indirect tensile characteristics for critical performance prediction. Cylindrical specimens stabilized with 6% lime and 10% CFA were molded using a Superpave gyratory compactor, cured for 28 days and subjected to different stress sequences in indirect tension mode to study the Mrt. Also, stabilized cylindrical specimens were tested for tensile strength under indirect tension. It was found that all the three additives improved the Mrt and indirect tensile strength values of P- and V-soil specimens; however, degree of improvement varied with the type of additive and soil. The resilient modulus in tension ranged between approximately 443 – 908 MPa (64 – 132 ksi) and 315 – 656 MPa (46 – 95 ksi), respectively, for 6% lime- and 10% CFA stabilized silty clay specimens. On the other hand, stabilization of lean clay with 6% lime and 10% CFA provided Mrt values ranging between approximately 444 – 691 MPa (64 – 100 ksi) and 580 – 839 MPa (84 – 122 ksi), respectively. The test results suggest that the Mrt depends on the applied load. Based on the test results, the Mrt decreased with increase in stress ratio.

As mentioned, this report investigates indirect tensile strength and resilient modulus. The resilient modulus is observed to increase with the increasing additive content. There were 5 values given for each of the raw and improved soils, which were averaged in order to obtain the final resilient modulus value. A summary of the results of this research is given in the following table.

Table 4.12. Summary of stiffness improvement for Solanki and Zaman (2011) study.

Soil	Soil Classification			Atterberg Limits			Type and % of additive			Indirect Tension	Stiffness
Identificatio	USCS	USCS Name	AASHTO	LL	PL	PI	None	Lime	CFA	Resilient Modulus (MPa)	Factor
P-soil	CL-ML	Silty clay w/ sand	A-4	27	21	5	100	0	0	54.8	-
							94	6	0	654.6	11.9
							90	0	10	734	13.4
V-soil	CL-ML	Lean clay	A-6	37	26	11	100	0	0	26	-
							94	6	0	568.8	21.9
							90	0	10	685	26.3

Factors of improvement with respect to stiffness were quite high for the two soils, such that the stiffness was over ten times and over twenty times the unstabilized soil stiffness for 10% Class C fly ash for the two soils, respectively.

4.11.3 Geotechnical Properties of Philadelphia Problem Soil Stabilized with Fly Ash and Limestone Dust

Brooks, R., Udoeyo, F. F., and Takkalapelli, K. V. (2008) "Geotechnical Properties of Philadelphia Problem Soil Stabilized with Fly Ash and Limestone Dust." *J. Materials in Civil Eng.*, pp. 1-13.

This paper presents the results of a laboratory experimental program to evaluate the potential of limestone dust (LSD) and coal fly ash (CFA) to stabilize some problem soils in South Eastern Pennsylvania. Some of the geotechnical characteristics of the soils investigated include Atterberg limits, compaction, California bearing ratio (CBR), Swell and Unconfined compressive strength (UCS). A one-way analysis of variance (ANOVA) test performed on the generated data confirmed that LSD and CFA contents significantly influenced the compaction and the strength characteristics of Philadelphia soils stabilized with these additives. Results of the study showed that the plasticity and swell of the soils were reduced by 40% and between 40-70%, respectively. The results further showed a marked increase in strength of the soils, in terms of CBR and UCS, when stabilized with the additives.

In this study, two weak soils from south eastern Pennsylvania were stabilized using coal fly ash and limestone dust. The results of the study show that the geotechnical properties of soils are enhanced significantly by stabilization with the aforementioned additives with an exception of UCS for one soil. The improvement includes reduction in plasticity and increase in California bearing ratio. The results further revealed that compaction characteristics such as maximum dry density of soil-additive mixture decreased with increase in additive content while optimum moisture content of the mixture increased with increase in additive content. These trends are consistent with the findings of past workers on soil stabilization (Thompson, 1968; Osinubi, 1996, 1997; Okagbue, 1999, 2007). The reduction in the MDD can be attributed to cation exchange reaction in the mixture, which caused the flocculation and agglomeration of the clayey particles of the soil, resulting in larger particles with larger voids in between particles, and consequently leading to reduction in the weight-volume ratio of the mixture. It was also observed in the study that the swell potential of soil reduced significantly when treated with CFA and LSD. The presence of differential electric charge, sodium present in the clay coupled with cation exchange capacity accounted for the swelling potential of the soil. The replaceable monovalent ions of clay were substituted by the calcium ions of lime.

This report provides the unconfined compression strength for three curing times (in days) and varying limestone and fly ash contents. The stiffness factor typically begins below one soon after mixing and eventually provides an increase in results. The soil tested was a lean clay. The table below summarizes the results of the study.

Table 4.13. Summary of stiffness improvement for Brooks et. al. (2008) study.

Raw/Stabilized soil												
Curing	Raw soil	UCS (kPa) of % limestone content						UCS (kPa) of % fly ash content				
days	0	3	Stiffness Factor	6	Stiffness Factor	9	Stiffness Factor	15	Stiffness Factor	25	Stiffness Factor
Day 1	610	442	0.72	490	0.80	498	0.82	335	0.55	298	0.49
Day 7	610	548	0.90	680	1.11	1210	1.98	396	0.65	335	0.55
Day 28	610	955	1.57	1176	1.93	1955	3.20	485	0.80	390	0.64
Day 1	540	400	0.74	448	0.83	440	0.81	568	1.05	298	0.55
Day 7	540	440	0.81	536	0.99	870	1.61	600	1.11	486	0.90
Day 28	540	686	1.27	842	1.56	1230	2.28	780	1.44	640	1.19

As given in the table, the long-term factor of improvement for stiffness for the soil ranged from 1.3 to 2.3 for limestone contents between 3% and 9%. The factor of improvement for stiffness for the soil ranged from 1.2 to 1.4 for fly ash contents of 15% and 25%.

4.11.4 Laboratory Performance Evaluation of Stabilized Sulfate Containing Soil with Lime and Class C Fly Ash

Singh, D., Ghabchi, R., Laguros, J. G., and Zaman, M. (2010) "Laboratory Performance Evaluation of Stabilized Sulfate Containing Soil with Lime and Class C Fly Ash." *GeoFlorida 2010*, pp. 757-766.

A laboratory study was undertaken to evaluate the effectiveness of lime and Class C fly ash (CFA) as soil stabilizing agents for a sulfate containing, A-7-6 type CL Oklahoma soil. Using various percentages and employing as criteria the 28 day unconfined compressive strength test (UCS) and the pH value, the optimum amount of lime (5 %) and CFA (16 %) were determined. Using these percentages further performance evaluation was effected. Cylindrical samples were compacted at optimum moisture content (OMC) and cured for 28 days for resilient modulus (M_R) and UCS tests. Laboratory tests revealed that the M_R and UCS of soil increased substantially with the addition of both lime and CFA compared to those of raw soil at OMC. The M_R value increased from 50 Mpa of the raw soil to 250 Mpa of its stabilized counterpart, and the UCS improved from approximately 100 kPa to 1000 kPa. In addition, 3-D swelling and Tube suction tests (TST) were conducted on cured samples to determine swelling and moisture susceptibility behaviors; however, the volume change was minimal, from 1651 cm3 to 1664 cm3, and the dielectric constant decreased from 30 to 20. The scanning electron microscopic (SEM) studies showed a change in the soil fabric with the formation of some crystals and qualitative changes in the densification resulting from the mastic filling part of the original voids. Finally, the change in the plasticity index was observed to be from 22 % to 10 %, a manifestation of the ameliorating effects of stabilization.

The effect of different percentages of lime and CFA on the engineering properties of stabilized soil was examined. For selected amounts of OAC, the lime stabilized soil showed a higher improvement in MR and UCS, compared to those of CFA stabilized soil. TST results revealed that lime stabilized specimens were more susceptible to moisture as compared to CFA stabilized specimens, however both materials display moisture susceptibility in excess of 16. The 3-D swell tests indicated no significant difference in the volume of the specimens and the effect of sulfate was minimal due to its low content of 630 ppm. The void domain matrix analysis of stabilized soils showed that lime and CFA produces denser and compact fabric compared to raw soil. Finally, the change in the plasticity index was observed to be from 22 to10, a manifestation of the ameliorating effects of stabilization. Hence it becomes important to determine the lowest critical limit of sulfate content of soil prior to any stabilization efforts.

The soil was classified by both the USCS and AASHTO classification systems. The resilient modulus was given, in the report, for the fifteen sequences of the test, where those values were then averaged in order to find representative resilient modulus value. The resilient modulus factor was then calculated from the raw soil data and the soil/additive data. The resilient modulus factor was observed to increase with additional additives. The table below provides a summary of the data obtained.

Table 4.14. Summary of stiffness improvement for Singh et. al. (2010) study.

Soil Classification		Atterberg Limits			Resilient Modulus (Mpa)		
USCS	AASHTO	LL	PL	PI	Raw soil	5% Lime	16% Fly ash
CL	A-7-6	41	19	22	37.6	269.5	241.8
Resilient Modulus Factor					-	7.17	6.43

For the 16% fly ash content, the factor of improvement for stiffness was greater than 6 for this lean clay soil.

4.11.5 Parameters Controlling Strength and Coal Fly Ash - Lime Improved Soil

Consoli, N. C. and Rosa, A. D. (2010) "Parameters Controlling Strength and Coal Fly Ash - Lime Improved Soil." *GeoFlorida 2010*, pp. 89-98.

Unconfined compressive tests and suction measurements were carried out on sandy specimens with distinct Class F fly ash amounts (6.25%, 12.5% and 25%), lime contents (3% to 9%), porosities and 28 days as the curing period to assess key parameters controlling strength of fly ash-lime amended soil. A special effort has been allocated in order to develop a dosage methodology for fly ash-lime improved soils based in a rational criterion, as it exists in concrete technology where the water/cement ratio plays a fundamental role in the assessment of the target strength. The results show that the unconfined compressive strength (qu) increased linearly with the amount of lime for soil-fly ash-lime mixtures at all curing time periods studied. A power function fits better the relation qu – porosity for soil-fly ash-lime mixtures. The bigger the amount of fly ash and the curing time, the larger the qu for any given porosity and lime content. Finally, the porosity/volumetric lime content ratio shows to be a good parameter in the evaluation of the unconfined compressive strength of the soil studied.

From the data presented in this paper, the following conclusions can be drawn:

- For the fly ash and lime contents studied here, the unconfined compressive strength increased approximately linearly with an increase in the lime content;
- The larger the amount of fly ash, the bigger the unconfined compressive strength (qu) for a given density and lime content;
- The reduction in the porosity of the compacted mixture greatly improves the strength. It was shown that the unconfined compressive strength increased exponentially with a reduction in the porosity of the compacted soil-fly ashlime mixtures;
- The porosity/volumetric lime content ratio, the latter adjusted by an exponent (0.12 for the soil, fly ash and lime used in the present research) has been shown to be a more appropriate parameter to evaluate the unconfined compressive strength of the soil-fly ash-lime mixture studied. It is likely that this exponent will be a function of the materials (soil, fly ash and lime) used.

With varying quantities of lime and fly ash, the improved soil strength is observed to increase with the increasing amount of lime and fly ash. When lower amounts of fly ash is added to the soil, the strength factor increases more drastically with the introduction of higher lime contents. As the fly ash content increases, the addition of lime is observed to increase the soil's strength at a slower rate. Overall, as more fly ash is added to the soil in this report, the unconfined compressive strength is observed to increase at a higher rate than the addition of lime. Unfortunately, no raw soil properties exist in this evaluation.

Table 4.15. Summary of strength improvement for Consoli and Rosa (2010) study.

Soil improved with 6.25% fly ash			Soil improved with 12.5 % fly ash			Soil improved with 25% fly ash		
Lime content (%)	UCS (kPa)	Strength Factor	Lime content (%)	UCS (kPa)	Strength Factor	Lime content (%)	UCS (kPa)	Strength Factor
0	47	-	0	229	-	0	685	-
3	128	2.7	3	401	1.8	3	1010	1.5
5	182	3.9	5	516	2.3	5	1227	1.8
7	236	5.0	7	631	2.8	7	1444	2.1
9	290	6.2	9	746	3.3	9	1660	2.4

This research used a combination of fly ash and lime to stabilize the soil. The factor of improvement with respect to strength for the silty sand in this study was 2.7 for a lime content of 3% and a fly ash content of 6.25%.

4.11.6 Utilization of Recycled Materials in Illinois Highway Construction

Griffiths, C. T. and Krstulovich, J. M. Jr. "Utilization of Recycled Materials in Illinois Highway Construction." *IL-PRR-142*, Illinois Department of Transportation, 2002.

According to the Illinois Environmental Protection Agency's 2000 Annual Landfill Capacity Report "as of Jan. 1, 2001, 53 landfills reported having a combined remaining capacity of 743.4 million gate cubic yards, or 49.3 million gate cubic yards less than on Jan. 1, 2000, a decrease of 6.2 percent." Also, at current waste generation rates "landfill life expectancy in Illinois [is] 15 years barring capacity adjustments." As waste continues to accumulate and availability and capacity of landfill spaces diminish, agencies are increasing application and use of recycled materials in highway construction.

The Illinois Department of Transportation utilizes millions of tons of highway materials annually. The basic building materials in roadway and bridge construction are primarily aggregate, cement, and asphalt. The annual usage of recycled materials is over 1.5 million tons. The educated use of recycled materials can result in reduced cost potentials and may enhance performance; however, not all recycled materials are well suited for highway applications due to limited or compromised performance-based benefits and/or high cost. This report reviews current usage of various recycled materials, as well as discusses reclaimed materials not currently being utilized by the Department.

1. air-cooled blast furnace slag
2. byproduct lime
3. fly ash
4. glass beads
5. granulated blast furnace slag
6. microsilica
7. reclaimed asphalt pavement
8. recycled concrete pavement
9. steel reinforcement
10. steel slag
11. wet-bottom boiler slag

4.12 Lime & Lime/Cement

4.12.1 Key Parameters for the Strength Control of Lime Stabilized Soils

Consoli, N. C., Lopes Jr., L. S., and Heineck, K. S. (2008) "Key Parameters for the Strength Control of Lime Stabilized Soils." *J. Materials in Civil Eng*., 21(5) 210-216.

The addition of lime is an attractive technique when the project requires improvement of the local soil. The treatment of soils with lime finds an application, for instance, in the construction of pavement base layers, in slope protection of earth dams, and as a support layer for shallow foundations. However, there are no dosage methodologies based on rational criteria as exist in the case of the concrete, where the water/cement ratio plays a fundamental role in the assessment of the target strength, and in the case of soil–cement technology, where the voids/cement ratio is shown to be a good parameter for the estimation of unconfined compression strength. This study, therefore, aims to quantify the influence of the amount of lime, the porosity, and the moisture content on the strength of a lime-treated sandy lean clay soil, as well as to evaluate the use of a water/lime ratio and a voids/lime ratio to assess its unconfined compression strength. A number of unconfined compression tests and measurements of matric suction were carried out. The results show that the unconfined compression strength increased linearly with the increase in the lime content as well as with the reduction in porosity of the compacted mixture. The change in moisture content has not presented an obvious effect on the unconfined compression strength of mixtures compacted at the same dry density. It was shown that, for the soil–lime mixture in an unsaturated state (which is usual for compacted fills), the water/lime ratio is not a good parameter for the assessment of unconfined compression strength. In contrast, the voids/lime ratio, defined as the ratio between the porosity of the compacted mixture and the volumetric lime content, is demonstrated to be the most appropriate parameter to assess the unconfined compression strength of the soil-lime mixture studied.

From the data presented in this paper, and bearing in mind the limitations of this study, the following conclusions can be drawn:

1. For the lime contents studied here, the unconfined compression strength increased approximately linearly with an increase in the lime content.
2. The reduction in the porosity of the compacted mixture greatly improves the strength. It was shown that the unconfined compression strength increased approximately linearly with a reduction in the porosity of the compacted mixture.
3. It was found that there is no relationship between the unconfined compression strength and the water/lime ratio for the material studied.
4. The voids/lime ratio, defined by the voids volume of the compacted mixture divided by the lime volume, adjusted by an exponent (0.06 for the soil and lime used in the present research) has been shown to be a more appropriate parameter to evaluate the unconfined compression strength of the soil–lime mixture studied. It is probable that this exponent will be a function of the materials (soil and lime) used.

With a soil classification of SM (USCS) or A-4 (AASHTO), the unconfined compression strength was calculated through lab testing at lime percent concentrations at 3, 5, 7, 9 and 11% at four different dry unit weights and at curing times of 28 days and 90 days. A best fit line was plotted for each unit weight, and with the given line equation, the unconfined compression strength at 0% and all missing contents of lime were calculated. The strength is observed to increase with the increase of lime content. The following table summarizes the data.

Table 4.16. Summary of strength improvement for Consoli et. al. (2008) study.

	Soil Classification		Ɣd = 16.0 kN/m^3			Ɣd = 17.0 kN/m^3			Ɣd = 18.0 kN/m^3			Ɣd = 18.8 kN/m^3		
	USCS	AASHTO	% Lime	UCS (qu) (kPa)	Strength Increasing Factor	% Lime	UCS (qu) (kPa)	Strength Increasing Factor	% Lime	UCS (qu) (kPa)	Strength Increasing Factor	% Lime	UCS (qu) (kPa)	Strength Increasing Factor
28 Day Curing	SM	A-4	0	173	-	0	292.7	-	0	422.2	-	0	543.7	-
	SILTY SAND, POORLY GRADED SAND-SILT MIXTURES	SILTY SOILS, FAIR TO POOR	1	183.1	1.06	1	305.3	1.04	1	442.7	1.05	1	563.5	1.04
			2	193.2	1.12	2	317.9	1.09	2	463.2	1.10	2	583.3	1.07
			3	203.3	1.18	3	330.5	1.13	3	483.7	1.15	3	603.1	1.11
			4	213.4	1.23	4	343.1	1.17	4	504.2	1.19	4	622.9	1.15
			5	223.5	1.29	5	355.7	1.22	5	524.7	1.24	5	642.7	1.18
			6	233.6	1.35	6	368.3	1.26	6	545.2	1.29	6	662.5	1.22
			7	243.7	1.41	7	380.9	1.30	7	565.7	1.34	7	682.3	1.25
			8	253.8	1.47	8	393.5	1.34	8	586.2	1.39	8	702.1	1.29
			9	263.9	1.53	9	406.1	1.39	9	606.7	1.44	9	721.9	1.33
			10	274	1.58	10	418.7	1.43	10	627.2	1.49	10	741.7	1.36
			11	284.1	1.64	11	431.3	1.47	11	647.7	1.53	11	761.5	1.40
90 Day Curing	Same as above	Same as above	0	282.6	-	0	435.2	-	0	712.4	-	0	914.9	-
			1	301.4	1.74	1	468	1.60	1	751.1	1.78	1	977.2	1.80
			2	320.2	1.85	2	500.8	1.71	2	789.8	1.87	2	1039.5	1.91
			3	339	1.96	3	533.6	1.82	3	828.5	1.96	3	1101.8	2.03
			4	357.8	2.07	4	566.4	1.94	4	867.2	2.05	4	1164.1	2.14
			5	376.6	2.18	5	599.2	2.05	5	905.9	2.15	5	1226.4	2.26
			6	395.4	2.29	6	632	2.16	6	944.6	2.24	6	1288.7	2.37
			7	414.2	2.39	7	664.8	2.27	7	983.3	2.33	7	1351	2.48
			8	433	2.50	8	697.6	2.38	8	1022	2.42	8	1413.3	2.60
			9	451.8	2.61	9	730.4	2.50	9	1060.7	2.51	9	1475.6	2.71
			10	470.6	2.72	10	763.2	2.61	10	1099.4	2.60	10	1537.9	2.83
			11	489.4	2.83	11	796	2.72	11	1138.1	2.70	11	1600.2	2.94

As seen in the summary table, the factor of improvement with respect to strength varies from 1.0 to 2.9 for lime contents ranging from 1% to 11%, also varying somewhat as a function of dry density at a fixed lime content.

4.12.2 Small-Strain Shear Moduli of Chemically Stabilized Sulfate-Bearing Cohesive Soils

Puppala, A. J., Kadam, R., Madhyannapu, R. S., and Houos, L. R. (2006) "Small-Strain Shear Moduli of Chemically Stabilized Sulfate-Bearing Cohesive Soils." *J. Geotechnical Geoenvironmental Eng.*, 132(3) 322-336.

An experimental study was conducted to measure small-strain shear moduli of chemically treated sulfate-bearing expansive soils using the bender element test. The bender element test was chosen because it provides reliable and repeatable small strain shear modulus measurements and allows for the periodical monitoring of stiffness property responses of soil specimens under varying curing conditions. Bender element tests were conducted on cement and lime treated soils and the results were then analyzed to study the variations in stiffness properties of soil specimens at different sulfate levels and curing conditions. Both cement and lime treated natural and artificial clays with low sulfate level of 1,000 ppm showed considerable enhancements in small strain shear moduli, whereas the same treated soils at high sulfate level of 10,000 ppm showed less enhancements in shear moduli due to sulfate heaving. Also, enhancements in shear moduli were lower for soil specimens continuously soaked under water compared to those cured in the humidity room. Rates of stiffness enhancements due to stabilizer type, compaction moisture content, type of curing, and sulfate levels are quantified and summarized.

The present experimental studies were undertaken to study the variations in small strain shear moduli properties with an intent to use these moduli variations to describe sulfate induced heaving in both cement and lime treated soils and to establish threshold "rates of enhancements of shear moduli" that correspond to sulfate heaving. Based on these studies, the following conclusions were derived: The bender element testing procedure was successfully used to estimate small strain shear moduli of chemically treated sulfate rich soils. This procedure provided reasonable repeatability of stiffness measurements with low coefficient of variation (less than 5%)

The majority of small strain shear moduli of control, lime treated clays, and cement treated clays ranged between 0 and 100, 50 and 250, and 200–500 MPa, respectively. Cement treated clays showed considerable increase in small strain stiffness properties when compared to lime treated clays. The percent increase in soil stiffness properties of cement and lime treated clays ranged between 500–1000% and 150–250%, respectively. The percentage increase was measured with respect to the moduli properties of control soils. The percent increase in stiffness properties is in good agreement with the well-known observation that the cement binder provides considerable enhancements to the strength and related stiffness properties of soils than does the lime stabilizer.

At high sulfate contents, the soaking based curing (S1) provided lower stiffness property enhancements than humidity room based curing (S2). This shows that the method of curing with continuous moisture access resulted in continuous Ettringite formation at high sulfates, which in turn reduced stiffness properties when hydrated.

The logarithmic rates of enhancements in shear moduli per log cycle, determined for the present soils indicate that the majority of these slope values varied from −4 to 9 MPa for untreated soil conditions, −13 to 40 MPa for cement treated soil conditions, and −13 to 22 MPa for lime treated soil conditions. Negative rates of enhancements in lime treated soils were attributed to detrimental effects caused by the hydrated Ettringite expansion and small shear moduli enhancements that begin with the lime treatment.

The logarithmic rates of enhancements plots of shear moduli properties from the present research can be used as benchmarks to identify the potential heaving problems in field conditions. Future field studies on chemically treated sulfate-rich soils under full moisture hydration conditions utilizing geophysical investigations would help in better assessments of this approach. A potential benefit of this approach is that it could lead to quicker assessments of this heave in field conditions by utilizing nondestructive geophysical tests, which in turn could help in immediate rehabilitation actions on heave-distressed site soils.

4.12.3 Stress Path Testing of Realistically Cured Lime and Lime/Cement Stabilized Clay

Rogers, C. D. F., Boardman, D. I., and Papadimitriou, G. (2006) "Stress Path Testing of Realistically Cured Lime and Lime/Cement Stabilized Clay." *J. Materials in Civil Eng.*, 18(2) 259-266.

This paper aims to show that the addition of lime and lime/cement can significantly improve both the stiffness and the resistance to permanent deformation of clay soils even under adverse, though realistic, curing conditions. The AASHTO Guide for Design of Pavement Structures incorporates a method of calculating subgrade resilient modulus (MR), using repeated load triaxial testing, with respect to seasonal variations of the soil's moisture content and temperature, while the equivalent British specifications adopt a "soaked California bearing ratio" approach. In practice 8°C is a realistic worst case curing temperature in situ and a (necessarily) wetter underlying clay will allow water to be drawn into the chemically treated layer during curing as a result of suctions, while the material will experience a small degree of confinement after compaction. These conditions have been replicated in a series of repeated load triaxial tests on lime and lime/cement stabilized, predominantly kaolinitic clay and the results compared with those from sealed samples, which were also cured at 8°C. All samples were subjected to 100 cycles of stresses representing 20, 40, 60, and 80% of the relevant undrained shear strength. This paper concludes that allowing the samples access to water is necessary if field performance is to be replicated.

In order to achieve an accurate description of the load carrying capabilities of lime and lime/cement stabilized soils, its shearing capacity must be examined in conjunction with resilient modulus and permanent deformation measurements. The quick undrained shear test, apart from providing a strength measurement, gives an indication of the likely material deformations. Having established the maximum shearing capacity (qmax), the elastic and plastic behavior of the material under repeated loading must be examined for samples cured under realistic conditions, a curing temperature of 8°C, a confinement of 20 kPa to simulate the mean normal effective stress acting on the material, and controlled access to water being recommended as realistic worst-case conditions. Using this protocol, swelling was kept to acceptably low levels. By applying different proportions of qmax it is possible to simulate a range of traffic conditions, 20% increments of qmax apparently being sufficient to make appropriate engineering judgments. The resilient elastic modulus (MR) graphs will provide the engineer with information on the asymptotic tendencies of the curves after a certain number of cycles and therefore appropriate design limits. However, this must always be done after careful scrutiny of the permanent deformation graphs, which not only provide a detailed description of the deformations likely to occur under certain traffic loading conditions but also indicate where failure due to excessive plastic deformation is likely to occur.

The specific conclusions related to the performance of lime and lime/cement stabilized English China clay, a relatively pure kaolinite, were as follows:

1. The application of small confining pressures during curing is essential and closely linked to the amount of water absorbed, and hence swelling and strength development. This novel curing method produced relatively homogeneous samples, as evidenced by the water content distribution, and apparently realistic shearing capacities as a result of avoidance of excessive swelling.
2. Detailed study of the qmax graphs provides a link to deformation control, and hence potential rutting and subgrade deterioration, during early trafficking.
3. Informed design MR values can be obtained by combining the MR and permanent deformation data resulting from the recommended test protocols to ensure that failure or excessive deformation does not occur.
4. Water availability proved beneficial in the early days of stabilization (3 days), while curing for prolonged periods in continuous contact with water, while still producing

acceptable performance, proved detrimental. In general, lower moduli and increased permanent deformation values should be expected in such cases.

5. After 14 days of curing no significant degree of clay mineral dissolution, and hence crystallization of the pozzolanic calcium silicate hydrate and/or calcium aluminate hydrate reaction products, occurred when lime only was added.
6. The addition of cement not only improves the strength of the material but also alters the elastic properties (i.e., MR increases) and significantly reduces the plastic strains developed under repeated loading.

4.13 Foamed Asphalt/Foamed Bitumen

4.13.1 Evaluation of Foamed Asphalt Cold In-Place Pavement Recycling Using Nondestructive Techniques

Loizos, A. and Papavasiliou, V. (2006) "Evaluation of Foamed Asphalt Cold In-Place Pavement Recycling Using Nondestructive Techniques." *J. Transportation Eng*., 132(12) 970-978.

In light of the increasing cost of hot-mixed asphalt mixtures and the limited availability of good materials, cold in-place recycling using foamed asphalt stabilization offers an attractive alternative for rehabilitating pavements. The lack of experience—at least as far as the performance of the aforementioned technique for heavy duty pavements is concerned—led the Greek Ministry of Public Works to undertake a field experiment with the purpose of the rehabilitation of a severely damaged heavy trafficked semi-rigid pavement, part of the Trans European Network. In order to achieve this goal, a comprehensive monitoring and data analysis research study was performed, concentrating on the falling weight deflectometer nondestructive technique (NDT) as a major tool for the in situ evaluation (mainly in terms of deflection and layer moduli) of the recycled pavement's early life performance. Regarding layer thickness estimation, two different approaches are used, analyzed, and thoroughly discussed, reflecting cases often encountered in practical international

Based on systematically designed and constructed trial pavement sections, an effort was made to evaluate the early life performance of the CIPR using foamix treatment on the purpose of the rehabilitation of a heavy trafficked and severely damaged highway semirigid pavement part of the TEN. The evaluation was made using, mainly, NDT methods and related analysis tools. The major findings and discussion points are the following:

- The evaluation of the recycled pavement, taking into account the deflection plots, presented an improvement to the overall pavement structural condition over time. Deflections seem to be a useful tool (i.e., an overall performance indicator) for the in situ evaluation of such a recycling technique.
- In most cases there was an increase in the foamix material modulus over time, since the first 14 months the trial sections were given to traffic. An exception to this observation was a section where some particularly high values were obtained 8 months after construction (higher than the respective ones 14 months after construction).
- The ITSM tests on asphalt mix cores of the bituminous overlay, extracted 6 to 14 months (ML4) after the trial sections were given to traffic, generated higher values of moduli than the respective tests on the cores extracted from the bituminous overlay soon after given to traffic (ML3). This seems to confirm the effect of the post compaction.

- During early pavement life (i.e., the traffic period that the foamix material has not entirely cured), the bituminous overlay must have sufficient thickness and structural adequacy to sustain the traffic load. This observation was confirmed by the entire evaluation procedure performed in the study. The arguments, often mentioned by the international scientific community, that a rather thin bituminous overlay would be a sufficient structural cover for a CIPR foamix layer was not confirmed in the present study at least in the case of heavy duty pavements.
- The back-calculation of the composite modulus (combined bituminous and foamix layers) provided, in most cases, results with a lower RMS error, which renders the method a more reliable and effective tool for the overall evaluation of such a pavement rehabilitation technique. An increase in the composite modulus along with aging of the foamix material was detected since the first 14 months of traffic. This occurs, mainly, due to the curing of the foamix material and is partly associated with the increase in the modulus of the bituminous layers resulting from post compaction of the traffic.
- The use of GPR for the estimation of the different layer thicknesses reduces the uncertainties of the back-analysis procedure resulting in a lower variance coefficient of the back-calculated composite moduli, thus also facilitating the detailed estimation of the remaining CBM thickness. The last point is a crucial factor for the back-analysis especially in the case where the CBM cannot be removed for economical and technical reasons related to the rehabilitation concept of the cold recycling process. Consequently, the variance of the results is reduced and representative mainly of the material inhomogenity when compared to the results based on an average composite thickness due to limited cores availability.
- More specifically, the GPR NDT technique is useful when there is lack of foamix coring data, or even when such destructive core samples are to be avoided in highways due to roadtraffic closure delays. In addition, the use of GPR facilitates the structural evaluation by enabling the back-analysis to be undertaken for the AC layers separately from the foamix layers, and consequently the concept of a composite modulus may be avoided.
- Back-analysis showed that during ML5 (12–14 months), even the minimum values of the composite moduli were higher than the ones taken into account during the design, a fact that confirms, at least up until the time of the n investigation, the accuracy of the analytical pavement design estimation.
- The detailed back-analysis with systematic FWD data accomplished with ITSM asphalt mix cores testing in the laboratory proved to be, despite all the difficulties and uncertainties, an efficient and effective tool, at least as far as the overall assessment and structural characterization of the foamix treated CIPR heavy duty highway pavement is concerned.
- The choice of NDT as a means of acceptance, not rejection, of such a recycling technique depend upon the purpose of the analysis on the pavement composite stiffness reaching a value prestated in a contract, obtained using the analysis of the FWD survey data. Where the FWD stiffness values do not reach the required level, then the destructive coring and testing of foamix core samples option may be invoked as a last resort.

4.13.2 Investigation of the Curing Mechanism of Foamed Asphalt Mixes Based on Micromechanics Principles

Fu, P., Jones, D., Harvey, J. T., and Halles, F.A. (2010) "Investigation of the Curing Mechanism of Foamed Asphalt Mixes Based on Micromechanics Principles." *J. Materials in Civil Eng.*, 22(1) 29-38.

This study investigated the curing mechanism of foamed asphalt mixes. Various laboratory strength and stiffness tests were performed on mixes with various asphalt and portland cement contents, and the specimens were subjected to two relatively extreme curing conditions. It was found that portland cement enhances certain properties of foamed asphalt mixes by strengthening the mineral filler phase, with the curing mechanism similar to that of typical cement treated materials. The curing mechanism of foamed asphalt mastic is primarily related to water evaporation. The bonding between asphalt mastic and aggregate particles cannot fully develop until most of the water retained at the interface evaporates. This bonding, once formed, is only partially damaged by reintroduced water. This proposed mechanism was supported by observations of fracture faces on tested specimens. A long-term curing study confirmed the validity of this mechanism regardless of the curing duration. Standard curing procedures are proposed for use in project level mix design and evaluation based on the findings.

Observations from the results presented in this paper have important implications for full-depth reclamation of pavements. They indicate that the bonding provided by foamed asphalt develops as the mixing/compaction water evaporates, and only fully develops once most of this water is no longer present. If, under certain conditions, this water is retained after compaction (e.g., by early placement of the asphalt wearing course, or because of inadequate drainage) the bonds will not develop, even after a prolonged period of time (months or years). However, once the bonds have formed, occasional reintroduction of water into the treated layer will only partially damage the bonding, provided that extended soaking periods along with repetitive loading do not occur. It is therefore critical to allow the initial mixing/compaction water to evaporate from the recycled layer before the asphalt concrete surface layer is placed, to ensure that the road is adequately drained, and to ensure that roadside practices (e.g., irrigation) do not adversely affect the moisture condition of the pavement.

The strategy (Curing Conditions A and B) adopted in this study is recommended as a curing process for research and project level design laboratory testing. Precisely duplicating or simulating the field curing conditions in a laboratory is extremely difficult, if not impossible. It also creates problems for procedure standardization given the large variety of environments in which FDR-foamed asphalt can be implemented. The two curing conditions adopted in this study, simulating optimistic and conservative conditions for water evaporation, respectively, are recommended to characterize the fundamental properties of foamed asphalt mixes specific to curing. Curing Condition A is used to evaluate the long-term effectiveness of foamed asphalt treatment, whereas Curing Condition B is used to optimize the use of active fillers to ensure sufficient strength development for early open to traffic. Site specific criteria should be noted during project assessments to determine whether the tested materials will meet the conditions of the site.

Portland cement was shown to be very effective in improving strength, stiffness, and permanent deformation resistance of the foamed asphalt mixes tested in this task (weathered granite material), especially in the early stages when the foamed asphalt has not cured. Other active fillers may provide equal or better performance depending on the aggregate characteristics and chemistry. Many foamed asphalt recycling projects, including those in Calif., typically require that the rehabilitated section of road is opened to traffic before darkness each day. Early strength is therefore a key issue in the design, thereby supporting the use of appropriate active fillers in conjunction with foamed asphalt. The results of this and other tasks in the UCPRC study have shown that the addition of foamed asphalt and active fillers (in this case portland cement) both serve the same purpose of bonding aggregate particles together, but that their roles are complementary rather than interchangeable. The bonds formed by hydrated cement are strong but brittle compared to those of foamed asphalt which are weaker, but more ductile. Portland cement reduces water susceptibility and increases early as well as long-term strength, while foamed asphalt improves ductility or flexibility of the mixes. The effects of foamed asphalt and the selected active filler on a mix should be optimized separately in a project mix design procedure, since most conventional laboratory testing methods cannot differentiate these effects.

4.13.3 Failure Investigation of a Foamed-Asphalt Highway Project

Chen, D.-H., Bilyeu, J., Scullion, T., Nazarian, S., and Chiu, C.-T. (2006) "Failure Investigation of a Foamed-Asphalt Highway Project." *J. Infrastructure Systems*, 12(1) 33-40.

This paper documents an investigation into the cause of structural distress (alligator cracking and deep rutting) found in a foamed-asphalt warranty project in Texas. This is a unique forensic study involving a warranty specification, where the contractor may or may not be liable for the cost of repairs, depending on the outcome of the study. Extensive field tests, including FWD, seismic, GPR, and DCP, were conducted. Laboratory tests were also done to determine gradation, moisture content, capillary action, and indirect tensile strength. Four trenches were opened to test each layer directly and to obtain samples for laboratory testing. Two of the trenches were located in distressed/failed areas, while two others were in intact areas. Based on this study, the forensic team concluded that the rutting and alligator cracking are associated with failure of the foamed asphalt base. Since the subgrade strength in the failed areas was similar to the intact areas, the subgrade strength could not have been the sole cause of the failure. The foamed asphalt base was found to be susceptible to moisture and exhibited a severe loss of strength when subjected to moisture. Thus, it is imperative to verify the moisture susceptibility during the design phase. Based on the findings of this study, the contractor did repair the failed section at his own expense.

All results indicate that the rutting and alligator cracking are associated with the base layer’s lack of strength. The base stiffness in the distressed area is about three times lower than in the intact area, much lower than would be expected for any stabilized base. Neither the AC layer nor subgrade show significant differences along the distressed and intact areas, so the failure appears to be primarily base related. GPR and lab results did not show that the distressed areas have higher moisture content. It seems that a combination of generally high moisture in the subgrade, overly fine gradation, and inconsistent construction of the base layer caused the failure.

The results indicate that the foamed asphalt base samples allowed the moisture to wick upward, which had a large impact on the retained strength. The samples from the sections judged as intermediate performers failed the retained-strength criteria test with both the 24-h soak and the 10-day capillary rise conditioning. The samples from the poorly performing areas could not be tested, as they were all badly deteriorated. Based on the findings of this study, the contractor did repair the failed section at his own expense in October 2001.

This problem can be avoided in the future by carefully monitoring the construction variables, and by designing the mixture to suit local moisture conditions. Perhaps an increase in the asphalt or cement stabilizer content would have helped to reduce the intrusion of moisture and served as a factor of safety against inconsistent construction or materials (Scullion et al. 2003). For whatever reason, the foamed asphalt base quality was not consistent.

4.13.4 Strength and Deformational Characteristics of Foamed Bitumen Mixes under Suboptimal Conditions

Gonzalez, A., Cubrinovski, M., Pidwerbesky, B., and Alabaster, D. (2011) "Strength and Deformational Characteristics of Foamed Bitumen Mixes under Suboptimal Conditions." *J. Transportation Eng.*, 137(1) 1-10.

The effects of foamed bitumen contents on the strength and deformational behavior of foamed bitumen mixes used for road pavements is very complex and not fully understood yet. While some writers report an increase in strength using one type of laboratory test, other writers report either only a small increase or even a decrease in strength using other types of tests, thus detracting foamed bitumen from being implemented as a cold-recycling technique for road pavement rehabilitation. This paper presents a laboratory study carried out on a specific granular material from New Zealand containing 1% cement and different foamed bitumen contents using indirect tensile strength (ITS), monotonic load triaxial (MLT), and repeat load triaxial (RLT) tests. The curing procedure, loading regime, and moisture contents were selected to simulate construction practice and suboptimal conditions normally found in New Zealand pavements. The results from these tests showed that an increase in foamed bitumen content up to an "optimum" content, increases the ITS but, at the same time, decreases both the permanent deformation resistance measured in RLT tests and the peak strength in MLT tests. In order to systematically examine the results from these tests, a general stress analysis was conducted, in which the stress paths applied in laboratory tests were plotted in I1−J21/2 stress diagrams. The stress analysis showed that adding foamed bitumen results in a reduction of the compressive strength of the mixes and a simultaneous increase in the tensile strength, which explains the apparently "contradictory" effects of foamed bitumen reported in the literature depending on the type of test used.

The effects of FB content on the deformational behavior and strength of FB mixes with 1% cement have been studied using different tests in the laboratory. The laboratory procedures, materials, curing, moisture contents, and stress regime presented in this paper correspond to condition normally found in New Zealand pavements. Results from ITS, MLT, and RLT compression tests lead to the following findings:

- The addition of FB increases the ITS of FB mixes up to an optimum bitumen content of approximately 2.8%, at which ITS reaches the maximum value.
- The results obtained from MLT tests shows that the addition of FB reduces the peak axial stress attained in triaxial compression.
- The shear strength properties of the mixes (i.e., angle of internal friction and apparent cohesion) were estimated using MLT test data, indicating a reduction in the angle of internal friction (Φ) with FB content. The effects of FB content on the apparent cohesion were found to be relatively small for the investigated mixes.
- In the RLT, the addition of FB increased the cumulative permanent strain of the mixes, creating a relatively weaker material as compared to that without FB. In this regard, both MLT and repeat load triaxial tests were consistent as they showed an increase in the permanent deformation of the material with increasing FB content.
- Interpretation of test results using stress analysis revealed that adding FB reduces the compressive strength and simultaneously increases the tensile strength of the mixes, which explains the apparently opposite effects of FB content on the deformational behavior reported in the literature, depending on the particular type of test _stress condition_ used.
- The stress paths applied in RLT tests were compared with the failure stresses using the stress ratio between the applied stress and the respective stress at failure (which is considered to be a measure for the relative size of the load). The results showed that an increase in the FB content effectively increases the stress ratio for a given deviatoric stress, as a result of the reduction in the compressive strength of the mix.
- The work presented herein considers only stresses under specific laboratory test conditions. The stress conditions in road pavements in the field are more complex and involve both compressive and tensile stresses depending on the particular location in the base course layer. In addition, confining stresses in pavements are applied dynamically, which changes the material response of the materials. These stress conditions together with the strength and deformational characteristics presented herein need to be considered in order to fully understand the contribution of FB on the improved performance of pavements.
- Results from an accelerated field experiment, in which the same materials used in the laboratory work were tested under field stresses, showed that small amounts of FB improve pavement performance. In the accelerated field experiment, the section with the best performance corresponds to the section with the highest associated ITS value, indicating that ITS was a reasonably good predictor of pavement performance.

4.13.5 Recycled Pavements Using Foamed Asphalt in Minnesota

Eller, A. and Olson, R. "Recycled Pavements Using Foamed Asphalt in Minnesota." *MN/RC-2009-09*, Minnesota Department of Transportation, 2009.

Foamed asphalt was discovered in Iowa by Csanyi in 1956, and has become a useful road rehabilitation tool when used in conjunction with cold in-place recycle (CIR) and full-depth reclamation (FDR) processes. The advance of pavement recycling and foaming technology has made foamed asphalt a common rehabilitation technique in many parts of the world including Europe, Asia, Africa, Canada, and parts of the United States. Iowa has used the technique extensively and has developed specifications for the construction of foamed asphalt FDR and

CIR stabilized roadways. The intention of this research project, Investigation 873, is to develop FDR and CIR foamed asphalt specifications and report data and information that will assist engineers in Minnesota with successfully implementing foamed asphalt recycling techniques.

There are already several foamed asphalt CIR projects in Minnesota that have been completed on low volume roads. The roadways were rehabilitated in Fillmore and Olmsted Counties from 2004 to 2008, and are performing quite well to date. The Minnesota Department of Transportation (Mn/DOT) has taken Falling Weight Deflectometer (FWD) and core data from these projects in order to examine the in-situ properties of the stabilized pavement layer, as well as the material properties of the foamed asphalt itself. The FWD data analysis reveals that the recycled pavement layer develops a relatively uniform strength despite the high variability inherent in most low-volume roads. Core data indicates that the foamed asphalt forms a cohesive matrix when mixed with the fines from the reclaimed material, which does not disintegrate when cored. Overall PG grade of the recycled layer changed significantly from the original mix in some cases, but not in others. The cause of this is unknown, however, differences in the procedures used and materials present at the different projects may help explain this. It is recommended that FWD, ground penetrating radar (GPR), and core analysis be performed before and after foamed asphalt projects to more accurately define these differences.

The concept of using foamed asphalt as a technique to stabilize FDR and CIR materials has been around for several decades and in fact is used in many parts of the world already. Iowa, Nevada, California, Illinois, Wisconsin, Colorado, and Louisiana have tried foamed asphalt road reclamation techniques, to name a few. Two Counties in Minnesota have tried foamed asphalt CIR methods to rehabilitate some of their low volume roads. Overall the projects have been a success, by both providing a smooth, durable platform for placing a surface treatment, and by effectively increasing the time it takes for reflective cracking to appear. It is recommended that monitoring and data collection continue throughout the life of these pavements in order to properly examine lifetime performance.

Initial data collection included post-construction FWD testing and cores taken from the roads. The cores showed that the foamed asphalt portion remained intact after coring and appeared very similar to the HMA layers upon visual inspection. The pavement layer thicknesses measured from the cores were used to perform forward- and back-calculations on the FWD data. Since back-calculation is highly sensitive to layer thicknesses less than 4 inches, it was necessary to combine several of the layers in order to determine a meaningful modulus. Furthermore, the LTPP forward-calculation methods are based on a three layer pavement system. Thus, a three layer pavement system was chosen in order to utilize the LTPP method for checking the backcalculated moduli. The three layer model that minimized the difference between the forward and back-calculated moduli consisted of:

1) The top lifts of HMA combined with the foamed asphalt layer,
2) The underlying old HMA combined with the base layer,
3) The subgrade.

The fact that the best correlation between the ELMOD back-calculations and the LTPP forward calculations was achieved by combining the HMA and foamed asphalt layers, does not necessarily indicate that the properties of the foamed asphalt layer are similar to that of the

HMA. Structural evaluation of the foamed asphalt layer is best left to a more comprehensive study with a larger set of roads, in order to ensure statistical meaning and identify outliers. Future research projects should also consider taking FWD data prior to the construction of the projects in order to compare with the after-construction data.

In addition to providing pavement layer thickness data, the cores taken from Fillmore and Olmsted Counties were sent to the Mn/DOT Materials Lab for extraction and gradation testing. Lab analysis shows that overall binder grade may or may not change significantly after the addition of foamed asphalt (see Table 4.3). The binder grade of the original mix did not change significantly for the Fillmore County projects, whereas the binder grade did change significantly for the Olmsted County projects. This effect occurred despite all roadways in the study starting at similar asphalt grades in the old bituminous (the reclaimed layer), and undergoing injection of nearly the same proportion of foamed PG 52-34. The major aspect that differed significantly between the roadways, besides the change in PG grade of the foamed layer, is the percent rap contained in the foamed layer. Since the various layers of the pavement vary significantly in thickness from the base, to the initial HMA layer(s), to the subsequent overlay(s), differences in the PG grade of the individual layers may be contributing to the differences in PG grade of the foamed layer. Continued study of the foamed asphalt projects in Fillmore and Olmsted Counties will reveal whether or not this difference in binder grade affects overall roadway performance.

Mix design is another factor that may play a strong role in roadway performance and life. Mix design may be most critical for FDR pavement operations with foamed asphalt because of inherent variations in base thickness. Overestimation of base thickness based on data from coring may cause the reclamation machine to unintentionally mix additional subgrade material in with the pavement materials. This increase in minus 200 material might require adjustment of the foamed asphalt injection rate. If an adjustment is not made, the mixture of AC and recycled pavement may be too lean and may leave the stabilized layer more susceptible to water saturation and/or lower strengths. Mn/DOT requires a mix design (see Appendix B) for all FDR foamed asphalt stabilized projects for this reason. However, Mn/DOT does not require a mix design for foamed asphalt CIR operations because of the greater level of homogeneity of the pavement section above the base. Since a CIR project leaves 1-2 inches of pavement, there is less likelihood that base materials would become part of the mixture. The CIR foamed asphalt projects in Fillmore and Olmsted counties did not use a mix design, and the projects have performed quite well to date. It remains unknown whether a mix design would have improved the performance witnessed thus far.

Certainly, the success of foamed asphalt elsewhere helped determine the overall direction of this research, with the goal of providing local engineers with the information necessary to design and construct a foamed asphalt reclamation project. It is hoped that engineers will gain confidence in foamed asphalt reclamation through continued monitoring of such newly constructed projects; as well as monitoring of the projects referenced in this report.

4.14 Emulsion

4.14.1 An Investigation of Emulsion Stabilized Limestone Screenings

Nelson, J. D., Tymkowicz, S., and Callahan, M. "An Investigation of Emulsion Stabilized Limestone Screenings." *HR-309*, Iowa Department of Transportation, 1994.

A significant amount of fine sized waste limestone screenings is produced during aggregate production. This waste material, which is to fine to be used in either asphalt or portland cement concrete paving, is becoming an ever increasing burden of disposal for aggregate producers. Large stockpiles of the material are at most Iowa quarries. Any road construction process which could successfully use this material would be assured of a continuous supply of inexpensive aggregate.

Linn County was interested in developing such a construction process. An Iowa State University laboratory study (see Appendix B, page 42, reference 1) sponsored by Linn County showed that waste limestone screenings could be used as the sole aggregate in an emulsified asphalt mix. Such a mix could be used to replace selected granular surfaced roads and/or provide the base for stage construction of a future asphalt or Portland cement concrete pavement.

The objective of this research project was to construct and evaluate an experimental roadway base using a waste limestone screenings/emulsion mix. Specific topics to be investigated included:

1. The development of an efficient roadway construction technique using the waste limestone screenings/emulsion mix.
2. The mix strength, stability and durability properties obtainable in the field.
3. The optimum residual asphalt content and base thickness required to adequately support local traffic.
4. The validity of the anionic/cationic relationship existing between waste limestone aggregate and an. asphalt emulsion.

This research on emulsion stabilized limestone screenings support the following conclusions:

1. A low maintenance roadway can be produced using a seal coat surface on 6 inches (150 mm) of stabilized limestone screenings with 4.5% asphalt cement.
2. A 6 inch (150 mm) emulsion stabilized base with less than 3.5% asphalt cement does not produce a satisfactory low cost maintenance roadway.
3. A 4 inch (100 mm) emulsion stabilized base does not produce a satisfactory low cost maintenance roadway.
4. A 2 inch (50 mm) asphalt concrete surface would be necessary on many roads to provide a low maintenance roadway using emulsion stabilized limestone screenings.

4.14.2 Field Trial of Solvent-Free Emulsion in Oregon

Lundy, J. R. and Remily, M. D. "Field Trial of Solvent-Free Emulsion in Oregon." *FHWA-OR-RD-03-12*, Oregon Department of Transportation, 2003.

This final report summarizes construction, laboratory and performance information gathered by ODOT personnel from a single field trial of solvent-free emulsion mix constructed in June 2001. The solvent-free emulsion mix presented several placement problems as it built up on the laydown screed and gouged the mat. A second project trial section, scheduled for construction during 2001, was not completed due to construction scheduling problems.

Following standard ODOT design policy, both the solvent-free and conventional emulsion mixes were overlaid with a chip seal shortly after placement. After fourteen months, the performance of both mixes appeared to be equal. The indirect tensile strengths of the two mixes were statistically similar at all ages up to one year.

Plant production and fine tuning of the solvent-free mix was essentially equivalent to the process used for the solvent-loaded mix. The principal differences included the lower pump rates for the higher viscosity solvent-free emulsion and the need to adjust the emulsion input into the pugmill to improve aggregate coating. The buildup of fine material on the screed during placement of the solvent-free mix caused the paving to be halted. Although the exact cause is not known, one plausible explanation follows:

The solvent-free mix developed a thin crust of broken mix on the surface of the windrow. This broken material tended to stick to the front of the screed, particularly in locations where the augers do not do a good job of continuously moving the mixture, such as at the center auger gearbox. The mixture at these locations is static for a longer period of time, so the broken asphalt would have more of an opportunity to buildup on the screed surfaces. Once the buildup starts, the areas grow as more and more fines are caught by the initial buildup. This does not occur with solvent-loaded mixes because the solvent acts as a lubricant making the mixture less sticky and less apt to stick to the metal surfaces, even in a partially broken state.

Several possible solutions to the fine material buildup are listed below:

- Use of a more uniform, lower temperature, less intensely heated screed, such as those heated with hot transfer oil, would be less likely to affect the mix, relative to the propane (open flame) heated screed used on this project;
- Use of an alternate release agent;
- Use of end-dump trucks might be more effective in minimizing the amount of the thin crust of broken material;
- Placement of tarps on the belly dump loads to minimize exposure to the wind during delivery might minimize premature breaking;
- Minimization of the windrow amount deposited in front of the paver;
- Adjustment of the emulsion break mechanism to produce adequate mixes;
- Experimentation with different pre-strike off elevations on the screed to improve the flow of material under the screed;
- Experimentation with different auger elevations and auger positions relative to the screed.

The tenderness of the solvent-free mix evident during construction could be reduced by holding the choking and intermediate compaction operations back about one-half hour to allow a greater depth of the mat to break. This would extend the length of the paving train and the duration of paving.

This field trial demonstrated that a solvent-free emulsion mix could be successfully mixed using conventional equipment and that the strengths would be comparable to solvent-loaded or conventional EAC mixes. However, the solvent-free emulsion mix could not be successfully placed using the available equipment. If additional projects are considered, these paving problems must be addressed.

4.14.3 Laboratory Comparison of Solvent-Loaded and Solvent-Free Emulsions

Leahy, R. B., Root, S., and James, D. D. "Laboratory Comparison of Solvent-Loaded and Solvent-Free Emulsions." *FHWA-OR-RD-01-05,* Oregon Department of Transportation, 2000.

Asphalt emulsions have been widely used in highway construction and maintenance since the 1920s, initially as dust palliatives and spray applications. More recently, they have been used in more diverse paving applications such as base and surface course mixes, surface treatments and maintenance activities. The Oregon Department of Transportation (ODOT) uses nearly 450,000 Mg (500,000 t) of cold mix, i.e., emulsified asphalt concrete (EAC), for construction and maintenance at a cost of approximately $10 million per year. For safety, environmental and economic reasons, the use of emulsions is likely to increase dramatically in the next ten years. The decrease in highway funding and the public’s heightened environmental awareness demand innovative technology for roads of the 21st century. Recognizing the opportunities inherent in this challenge, some commercial enterprises have already developed solvent-free alternatives. Preliminary laboratory testing of solvent-free emulsions in standard dense-and open-graded EAC mixes indicated that mechanical properties are comparable to or exceed those of conventional solvent-loaded emulsions. Accordingly, the objective of this research was to quantify the difference between conventional solvent-loaded and solvent-free EAC as measured by indirect tensile strength.

Two aggregates typically used in ODOT Regions 4 and 5 were combined with three asphalt emulsions: a conventional CMS-2S and two commercially produced solvent-free emulsions. The results from this laboratory study are extremely promising. Specimens made with solvent-free emulsions had consistently greater indirect tensile strengths than did those made with conventional solvent-loaded emulsions. Furthermore, specimens made with the solvent-free emulsions achieved that strength gain more rapidly. Minor problems with the solvent-free emulsion consistency, i.e., uniformity, were encountered, but are considered an artifact of the production process rather than a problem with the material. Given the obvious effects on mixing, coating, adhesion and strength properties, this product consistency problem should be addressed prior to field trials, the logical extension of this very promising laboratory study. To that end, experiment designs for additional laboratory testing and field trials have been proposed.

The results of this and subsequent research could reduce, if not entirely eliminate the use of volatile solvents in EAC, yielding both economic and environmental benefits. Elimination of volatile solvent minimizes the fire hazard enhancing worker safety during manufacture of the emulsion and construction of the pavement section. Two-fold environmental benefits are expected with the use of solvent-free emulsions: improved air quality because of the elimination of volatile fumes; and reduction in the possibility of ground water contamination.

The complete data and a summary are included in Tables 3.1 through 3.6, and are shown graphically in Figures 3.1 and 3.2. Typical data reflecting replicate variability are shown in Figures 3.3a - 3.3f. An example of large variability and an example of small variability are shown for each aggregate-emulsion combination. Summaries of the variability as a function of curing time are shown in Figures 3.4 and 3.5. Data from all the specimens were used in the formulation of these figures.

4.14.4 Laboratory Performance Evaluation of CIR-Emulsion and its Comparison against CIR-Foam Test Results from Phase II

Hoisin, D. L., Thomas, K., and Byunghee, T. H. "Laboratory Performance Evaluation of CIR-Emulsion and its Comparison against CIR-Foam Test Results from Phase II." *IHRB Report TR-578 Phase III*, Iowa Department of Transportation, 2009.

Currently, no standard mix design procedure is available for CIR-emulsion in Iowa. The CIR-foam mix design process developed during the previous phase is applied for CIR-emulsion mixtures with varying emulsified asphalt contents. Dynamic modulus test, dynamic creep test, static creep test and raveling test were conducted to evaluate the short- and long-term performance of CIR-emulsion mixtures at various testing temperatures and loading conditions. A potential benefit of this research is a better understanding of CIR-emulsion material properties in comparison with those of CIR-foam material that would allow for the selection of the most appropriate CIR technology and the type and amount of the optimum stabilization material. Dynamic modulus, flow number and flow time of CIR-emulsion mixtures using CSS- 1h were generally higher than those of HFMS-2p. Flow number and flow time of CIR-emulsion using RAP materials from Story County was higher than those from Clayton County. Flow number and flow time of CIR-emulsion with 0.5% emulsified asphalt was higher than CIR-emulsion with 1.0% or 1.5%. Raveling loss of CIR-emulsion with 1.5% emulsified was significantly less than those with 0.5% and 1.0%. Test results in terms of dynamic modulus, flow number, flow time and raveling loss of CIR-foam mixtures are generally better than those of CIR-emulsion mixtures. Given the limited RAP sources used for this study, it is recommended that the CIR-emulsion mix design procedure should be validated against several RAP sources and emulsion types.

The previous research developed and validated the mix design procedure for cold in-place recycling using foamed asphalt (CIR-foam). The current CIR using engineered emulsion (CIR-EE) mix design procedure is complex and requires special equipment that is not commonly available. Currently, no standard mix design is available for CIR using emulsified asphalt (CIR-emulsion) in Iowa. The main objective of the study is to determine if the CIR-foam mix design process can be applied to CIR-emulsion with some minor adjustments.

The CIR-foam mix design process was applied to CIR-emulsion mixtures with varying emulsified asphalt contents. The simple performance testing (SPT) equipment was used to predict the field performance of various CIR-emulsion mixtures. Dynamic modulus test, dynamic creep test, static creep test and raveling test were conducted to evaluate the short- and long-term performance of CIR-emulsion mixtures at various testing temperatures and loading conditions. A potential benefit of this research is a better understanding of CIR-emulsion material properties in comparison with CIR-foam materials that would allow for the selection of the most appropriate CIR technology and the type and amount of the stabilization material.

Conclusions

Based on the limited laboratory experiment, the following conclusions are derived:

1. The mix design procedure developed for CIR-foam is applicable to CIRemulsion.
2. Indirect tensile strength of gyratory compacted specimens is higher than that of Marshall hammer compacted specimens.
3. Based on the wet indirect tensile strength of the gyratory compacted CIRemulsion specimens, the residual asphalt content of emulsion was found at around 1.0% with a clear peak.
4. Dynamic modulus of the CIR-emulsion is not as sensitive to temperature and loading frequency as HMA.
5. Dynamic modulus, flow number and flow time of CIR-emulsion mixtures using CSS-1h were generally higher than that of HFMS-2p.
6. Dynamic modulus of CIR-emulsion using RAP materials from Clayton County was higher than that of Story County.
7. Flow number and flow time of CIR-emulsion using RAP materials from Story County was higher than those of Clayton County.
8. Flow number and flow time of CIR-emulsion with 0.5% emulsified asphalt was higher than CIR-emulsion with 1.0% or 1.5%.
9. Raveling loss of CIR-emulsion with 1.5% emulsified was significantly less than those with 0.5% and 1.0%.
10. Test results of CIR-foam mixtures are generally better than those of CIR emulsion mixtures.

Based on the limited laboratory experiment, the following recommendations are made:

1. The mix design procedure for CIR-foam should be adopted for CIR-emulsion.
2. RAP materials should be characterized in terms of penetration index and amount of extracted asphalt binder and extract aggregate gradation.
3. It is recommended that flow number and raveling tests should be performed for predicting the field performance of CIR-emulsion.

4.15 Gypsum

4.15.1 Enhanced Performance of Stabilized By-Product Gypsum

Tao, M. and Zhang, Z. (2005) "Enhanced Performance of Stabilized By-Product Gypsum." *J. Materials in Civil Eng*., 17(6) 617-623.

Blended calcium sulfate (BCS), a by-product gypsum blended with lime or limestone, has been used as a base course material on some of Louisiana’s highways. Although BCS has high strength in dry conditions, its poor water resistance has largely limited its application in highway construction. A series of laboratory tests identified the factors that dictate the strength development of nonstabilized BCS. BCS was subsequently stabilized with various cementing agents to improve its water resistance. The mechanical properties of stabilized BCS, including its unconfined compressive strength, water resistance, volumetric expansion, and durability, were also evaluated. Finally, recommendations were made for the application of stabilized BCS as a base course material.

Unconfined compressive strength and various procedures for testing were implemented.

The strength development of nonstabilized BCS is a function of multiple variables, including dry unit weight, compaction effort, curing conditions, and final moisture content. Of these factors, the final moisture content proved to be the most significant individual factor. The presence of free water in nonstabilized BCS dramatically compromised its strength. The strength of nonstabilized BCS is heavily dependent on the magnitude of its final moisture content, but relatively independent of its moisture history. The variation of UCS with the final moisture content is roughly reversible. The water susceptibility of nonstabilized BCS implied that it is inappropriate for use as a pavement base course.

BCS can be satisfactorily stabilized with several cementitious materials, such as GGBFS, portland cement, lime, fly ash, or their mixtures. Of the aforementioned stabilization schemes, GGBFS stabilization is the most cost effective and resulted in the least volumetric deformation. Its long-term performance was also confirmed by the laboratory durability test. However, its early strength was not sufficient and may only be applicable to projects where waiting time will permit its use. Another feasible stabilization scheme is to use the GGBFS-portland cement mixture (GGBFS: portland cement=3:1) or the GGBFS-lime mixture (GGBFS:lime=5:1) at a 4% dosage, which had an adequate 7 day strength and intermediate deformation potential. Although some encouraging results for BCS stabilization were obtained in the laboratory, the performance of these stabilization schemes has yet to be confirmed in the field. The Louisiana Transportation Research Center is currently constructing several full-scale test sections for this purpose.

4.16 Conclusions from Literature Review

The first two tasks for this project were to: (1) identify stabilization materials and techniques of interest to Mn/DOT, and (2) identify past research projects relating to identified materials. Based on an initial list of materials and projects which was supplemented by additional literature review, a reasonably extensive list of stabilization products and techniques has been established. Task 3 has involved acquiring research reports and other references to obtain data from each of the materials identified in Task 2. In general, where data exist there is a large variability between stiffness and/or strength values due to moisture content, stabilization agent content, soil type and other variables. Now that this information has been collected, Task 4 will propose a methodology for obtaining project-specific ME design parameters to be used in future highway designs as the stabilization techniques considered are implemented in future projects.

CHAPTER 5.
TASK 4 – PROPOSE APPROPRIATE ME PARAMETERS AND TESTING METHODS FOR VARIOUS APPLICATIONS

This task is the final task associated with this project. Earlier tasks have built upon the Skok study (Skok et al., 2003) which identified a number of subgrade stabilization techniques being used at the time of that study, most of which are still being used in practice. Tasks 1 and 2 of this project identified subgrade stabilization techniques and materials which might be applicable to MnDOT practice (Task 1), along with projects and reports relating to these stabilization techniques and materials (Task 2). The technical advisory panel (TAP) for this project isolated the techniques and materials for which additional consideration by MnDOT was warranted. Task 3 involved obtaining and assessing the literature relating to these stabilization techniques and materials, initially in hopes of establishing ME design parameters appropriate to the stabilization material used.

Upon analysis of the available literature, the dependence of subgrade Mechanistic Empirical (ME) parameters for stabilized materials on various external conditions was made apparent. A number of projects for which stabilization was conducted showed that the factor of improvement relating to increased strength or stiffness was a function of soil type, the amount of the stabilization material added to the soil, and various other factors. In addition to having a number of factors contributing to the improvement of the subgrade material, the magnitude of improvement was also extremely variable. Thus, identifying one factor of improvement for a combination of material and method of stabilization was not deemed practical.

At the recommendation of the TAP group, the scope of this task was modified accordingly. Rather than recommending appropriate ME parameters for a stabilization technique, a recommendation was made to develop a procedure to be followed on a project-by-project basis to identify an appropriate parameter during the course of project pre-design and design. This recommended procedure/approach is provided in this report (Appendix A).

A similar procedural recommendation was provided through a recent Local Road Research Board (LRRB) project undertaken by Benson et al. (2009) entitled "Use of Fly Ash for Reconstruction of Bituminous Roads." While that study focuses primarily on the design for lower volume, local roadways (including the gravel-equivalency approach), a similar plan can be utilized for MnDOT applications in more general situations. Benson et al. (2009) recommend laboratory testing of the original base material, followed by similar tests on the "stabilized" material (fly ash stabilized material, for that project), providing a modified gravel equivalency factor to be used in the road design. The study tracked field performance of several projects over several years to note changes in properties with time. This long-term monitoring was not included in design/property determination recommendations.

5.1 Proposed Mix Design Procedure

The procedure outlined in Appendix A has been developed to quantify the degree of improvement obtained when using stabilization methods to improve subgrade properties. While there are many subgrade stabilization materials and techniques available for both subgrade and base stabilization, this procedure outlines a method appropriate when using (1) fly ash or (2) cement to stabilize the subgrade materials. As appropriate, a similar procedure may be followed to quantify the degree of improvement for other stabilization materials and techniques.

MnDOT uses R-value design and a Mechanistic-Empirical (ME) design method for roadway design. The ME design uses a stiffness parameter (the resilient modulus, M_r) for the subgrade material when designing the roadway. For the ME design, the "Stiffness Test" referred to throughout this mix design procedure will refer to the Resilient Modulus Test (as per the MnDOT modification of NCHRP 1-28a). For the R-value design, an R-value Test is to be conducted and the subgrade R-value is used for roadway design. For this approach, the "Stiffness Test" referred to throughout this procedure will refer to the R-value test.

This procedure was developed for MnDOT at the same time as a related project is underway through the National Cooperative Highway Research Program **(**NCHRP). Project NCHRP 04-36, entitled "*Characterization of Cementitiously Stabilized Layers for Use in Pavement Design and Analysis*"**,** is currently active and no final report has yet been published (30 May 2012). The objective of this project (NCHRP 04-36) is "to recommend performance-related procedures for characterizing cementitiously stabilized pavement layers for use in pavement design and analysis and incorporation in the MEPDG. This research will deal with material properties and related test methods that can be used to predict pavement performance. This research is concerned with subgrade, subbase, and/or base materials stabilized with hydraulic cement, fly ash, lime, or combinations thereof and used in flexible and rigid pavements" (http://apps.trb.org/cmsfeed/TRBNetProjectDisplay.asp?ProjectID=2494, accessed 30 May 2012). Once the final report for NCHRP 04-36 is made available, this MnDOT design procedure should be modified, if appropriate, to address the findings of that study.

The first phase in the mix design procedure is to conduct Standard Proctor compaction tests on the native material. This step should follow the MnDOT test procedure (MnDOT Procedure 1305) to identify the moisture-density relationship for the native (unstabilized) subgrade material. This step provides two critical results. First – an optimum water content and maximum dry density for the unstabilized (native) material is obtained. Stiffness testing will be guided by this combination of optimum water content and maximum dry density to provide the baseline stiffness, allowing the relative improvement in stiffness due to stabilization of the subgrade to be obtained. Second – this optimum water content provides a starting point for the stabilized soil mix design.

The second phase in the mix design process is to conduct a series of Stiffness Tests on the native (unstabilized) subgrade material. Tests should be conducted on samples compacted to obtain dry densities similar to the maximum dry density identified in the previous phase. Stiffness Tests are to be conducted in accordance with appropriate test procedures for the unstabilized subgrade material.

Once moisture-density and stiffness tests have been conducted on the native (unstabilized) material, moisture density tests are to be performed on the stabilized material. Tests shall be performed on the stabilized material for one stabilization material content (either fly ash or cement), as provided in Table 5.1 below.

Table 5.1. Recommended stabilization material content for mix design testing

When Using Fly Ash Stabilization	When Using Cement Stabilization
Fly Ash Content (by weight)	Cement Content (by weight)
6%	2%

These values are in accordance with limited data from the literature and regional experience. Construction contents may be higher, as appropriate, but the degree of improvement allowed for design will be based on contents given in Table 5.1. Should other stabilization material contents (either lower or higher) be considered (based on the discretion and approval of the engineer), additional compaction tests should be conducted at the desired material content. For contents higher than those in Table 5.1, additional testing may be required to ensure that cracking problems will not develop in the stabilized material.

The optimum water content and maximum dry density will be identified for the stabilization material content required in Table 5.1. The maximum dry density will be target value for Stiffness Tests conducted. Note that this value will differ from the value obtained for the native (unstabilized) material.

For the stabilization material content tested, appropriate stiffness values will be obtained in accordance with MnDOT Stiffness Test standards. To accommodate curing of the stabilized material, samples will be allowed to cure for at least 24 hours between the time of compaction and the time of the stiffness test. Additional time between compaction and stiffness testing may be required for materials having a more extended time-dependent increase in stiffness due to curing.

Once the stiffness of the stabilized subgrade material has been obtained for the stabilization material content under consideration, roadway design can be completed. A Resistance Factor (RF) will be developed based on a ratio of the stiffness of the stabilized material (with stabilization material content as described in Table 5.1) to the stiffness of the native (unstabilized) material. This RF value shall have a value less than or equal to 2. With respect to ME design, Equation 1 is used to calculate the Resistance Factor:

$$RF = \frac{Mr(stab)}{Mr(native)} \leq 2 \qquad \text{Eq. 1}$$

Where:

RF = Resistance Factor for Stiffness

Mr(stab) = resilient modulus of the stabilized material

Mr(native) = resilient modulus of the native (unstabilized) material

A similar calculation/equation applies where R-value design applies, where the Resistance Factor is taken to be the ratio of the R-value for stabilized and native material.

With respect to design, this Resistance Factor will be used to account for the increased stiffness of the stabilized material with respect to the native material. A factored stiffness for the subgrade may be entered into MnPAVE or appropriate design software to address the degree of improvement.

Modified methods for moisture-density testing and for Resilient Modulus testing have been included in Appendix B and Appendix C, respectively. These modified procedures are to be utilized in cases where stabilization methods are to be used on a project, providing appropriate density and stiffness values for use in roadway design.

5.2 Recommendations for Additional Consideration

Both the ME design approach and R-value design approach require testing to be performed on original materials and the stabilized materials, allowing the factor of improvement to be obtained and allowing improved subgrade properties to be used in design. However, using the values obtained from lab tests without field validation of such values is not recommended. To the extent possible, field testing should be conducted on such projects, both during and subsequent to construction, in order to show that the stiffness values obtained from the lab testing are achieved in the field. Using Lightweight Deflectometer (LWD) readings, Dynamic Cone Penetrometer (DCP) relationships or other stiffness measurements can provide justification of parameters used in the pavement design. Falling Weight Deflectometer (FWD) testing should be performed in the short and long term. Some of the materials used for subgrade stabilization have been shown to have time-dependent property changes which will not likely be identified in the relatively short-term mix design testing. Having long-term data to show stiffness as a function of time will be valuable as pavement or road performance is monitored over the life of the roadway.

5.3 Conclusions from Mix Design Development

This task (and the overall project) was to propose a methodology for dealing with stabilized subgrades on road projects. Since the properties of stabilized materials were determined to be highly variable (as a function of the mix design, the subgrade properties, etc.), having a method for determining appropriate ME properties (or R-value, when appropriate) for a given project was highly desirable.

In order to accommodate using improved ME properties into roadway design, it is proposed that both the original subgrade properties and the representative stabilized subgrade properties be obtained. Having both the original and improved properties will not only allow the improved properties to be utilized in the pavement/roadway design (thus potentially reducing costs for the project), but also allow local data to be available showing the degree of improvement when comparing the original soil to the treated soil. As more data becomes available, more general recommendations may be developed for similar soils using a similar mix design for various stabilization methods. At this time, since the variability is so high for the projects analyzed, coming up with an improvement factor was not realistic. As additional local data is made available, the opportunity to revisit this option would be possible.

CHAPTER 6. CONCLUSIONS

The state and many counties throughout Minnesota are using a variety of subgrade stabilization techniques for various materials used in road construction. Such methods appear to improve constructability and lead to increased performance and reduced maintenance. While a number of studies have investigated such stabilization efforts (including materials and techniques, relative increases in strength and/or stiffness, etc.) no overall quantification and summary of the effects of material stabilization have been brought forward with recommendations of parameters to be used for design purposes. Although these techniques and materials are commonly used, minimal information has been obtained relating to the Mechanistic-Empirical (ME) properties of these improved materials such that the more cost-effective designs can be implemented. Not having recommendations for the ME properties of the improved materials, the designer is forced to use values for the non-stabilized material. While this does likely lead to extended road life, construction costs could be greatly reduced by taking advantage of the improved properties of the stabilized roadway materials.

This project has involved determining which types of subgrade stabilization are being used, identifying which of these stabilization techniques/materials are of interest to the Minnesota Department of Transportation (MnDOT), compiling the results of past research relating to these stabilization techniques, summarizing the results of past research and proposing a mix design procedure that obtains material properties for use in design. This proposed mix design procedure will allow the designer to account for improved stiffness due to stabilization, reducing costs and improving the efficiency of the design.

The initial effort of this project involved developing a fairly comprehensive list of potential stabilization materials and/or techniques that might be of additional consideration. Such materials included fly ash, cement, lime, emulsion, foamed asphalt, and a variety of recycled byproducts. A list was developed to show the options available along with a brief description of that material/technique. Based on the comprehensive list, the researchers developed a revised list with proposed materials for further investigation and consideration. The Technical Advisory Panel (TAP) for the project reviewed the comprehensive and abbreviated lists and focused the study on materials which they felt were most appropriate for MnDOT consideration.

Based on this list from the TAP group, a significant literature review was performed to obtain as much information as possible relating to these materials. The original hope was to collect project data, from both field and laboratory testing, which provided material properties for native (unstabilized) materials along with modified properties of stabilized materials. This would provide guidance as to the degree of improvement that would be anticipated for various materials, in hopes of proposing a factor that would apply to all soils for which that stabilization material was utilized.

Several facts became evident as the literature review progressed. First, while the number of research and construction projects relating to subgrade stabilization was significant, the number of projects for which both unstabilized AND stabilized soil properties were determined and reported was quite small. Not having both sets of data did not allow quantification of the effects of stabilization from enough projects to clearly establish a “factor” of improvement. Second, for the cases where soil property information was available for both the stabilized and unstabilized soils, it was found that the degree of improvement was highly variable as a function of soil type, stabilization material content, water content, and other parameters. It became clear that establishing one factor of improvement for a given stabilization method that applied to all conditions was not realistic.

With this finding, the project scope was altered somewhat, developing a procedure to be used to establish appropriate factors as part of the mix design process for the stabilized soil. This procedure was developed for the two most probable subgrade stabilization materials, namely fly ash and cement. The procedure involves performing compaction and stiffness tests on the native (unstabilized) material, followed by similar tests on stabilized soils at specified stabilization material contents (6% fly ash and 2% cement). Obtaining the combination of unstabilized and stabilized soil parameters allows the degree of improvement to be quantified, which allows the designer to account for this improvement in the design of the roadway. This will allow a more efficient and less expensive roadway design that takes advantage of the improved subgrade properties due to stabilization.

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APPENDIX A

GUIDELINES FOR STABILIZED SUBGRADE MIX DESIGN PROCEDURE

Guidelines for Stabilized Subgrade Mix Design Procedure

1. Overview

When dealing with some highway/roadway subgrades, it may be desirable (and sometimes mandatory to allow construction) to improve the properties of the subgrade material prior to construction of the upper layers. This may include increasing the density of the soil (through mechanical compaction), increasing the strength and/or stiffness of the soil (by mixing in various stabilization materials), or other methods of improving the subgrade soil properties.

This procedure has been developed to quantify the degree of improvement obtained when using stabilization methods to improve subgrade properties. While there are many subgrade stabilization materials and techniques available for both subgrade and base stabilization, this procedure outlines a method appropriate when using (1) fly ash or (2) cement to stabilize the subgrade materials. As appropriate, a similar procedure may be followed to quantify the degree of improvement for other stabilization materials and techniques.

MnDOT uses R-value design and a Mechanistic-Empirical (ME) design method for roadway design. The ME design uses a stiffness parameter (the resilient modulus, M_r) for the subgrade material when designing the roadway. For the ME design, the "Stiffness Test" referred to throughout this mix design procedure will refer to the Resilient Modulus Test (as per the MnDOT modification of NCHRP 1-28a). For the R-value design, an R-value Test is to be conducted and the subgrade R-value is used for roadway design. For this approach, the "Stiffness Test" referred to throughout this procedure will refer to the R-value test.

This procedure has been developed for MnDOT at the same time as a related project is underway through the **National Cooperative Highway Research Program (NCHRP**). Project NCHRP 04-36, entitled "*Characterization of Cementitiously Stabilized Layers for Use in Pavement Design and Analysis*"**,** is currently active and no final report has yet been published (30 May 2012). The objective of this project (NCHRP 04-36) is "to recommend performance-related procedures for characterizing cementitiously stabilized pavement layers for use in pavement design and analysis and incorporation in the MEPDG. This research will deal with material properties and related test methods that can be used to predict pavement performance. This research is concerned with subgrade, subbase, and/or base materials stabilized with hydraulic cement, fly ash, lime, or combinations thereof and used in flexible and rigid pavements" (http://apps.trb.org/cmsfeed/TRBNetProjectDisplay.asp?ProjectID=2494, accessed 30 May 2012). Once the final report for NCHRP 04-36 is made available, this MnDOT design procedure should be modified, if appropriate, to address the findings of that study.

2. Acquisition of Subgrade Material

An adequate amount of material must be obtained from the project site to allow the laboratory testing to be completed. Laboratory work will require a number of Proctor compaction tests and Stiffness Tests, so a requisite amount of material will be needed to complete these tests. If the variability of subgrade materials throughout the project is high, more frequent sampling will be needed to ensure an adequate design.

3. Standard Compaction Testing

The first phase in the mix design procedure is to conduct Standard Proctor compaction tests on the native material. This step should follow the MnDOT test procedure (MnDOT Procedure 1305) to identify the moisture-density relationship for the native (unstabilized) subgrade material. This step provides two critical results. First – an optimum water content and maximum dry density for the unstabilized (native) material is obtained. Stiffness testing (Step 4) will be guided by this combination of optimum water content and maximum dry density to provide the baseline stiffness, allowing the relative improvement in stiffness due to stabilization of the subgrade to be obtained. Second – this optimum water content provides a starting point for the stabilized soil mix design (as per Step 5.)

4. Stiffness Test on Native (Unstabilized) Subgrade Material

The second phase in the mix design process is to conduct a series of Stiffness Tests on the native (unstabilized) subgrade material. Tests should be conducted on samples compacted to obtain dry densities similar to the maximum dry density identified in Step 3. Stiffness Tests, as defined in Section 1, are to be conducted in accordance with appropriate test procedures for the unstabilized subgrade material.

5. Compaction Testing on Stabilized Subgrade Material

Once moisture-density and stiffness tests have been conducted on the native (unstabilized) material, moisture density tests are to be performed on the stabilized material. Tests shall be performed on the stabilized material for one stabilization material content (either fly ash or cement), as provided in Table 1 below.

Table 1. Recommended Stabilization Material Content for Mix Design Testing

When Using Fly Ash Stabilization	When Using Cement Stabilization
Fly Ash Content (by weight)	Cement Content (by weight)
6%	2%

These values are in accordance with limited data from the literature and regional experience. Construction contents may be higher, as appropriate, but the degree of improvement allowed for design will be based on contents given in Table 1. Should other stabilization material contents (either lower or higher) be considered (based on the discretion and approval of the engineer), additional compaction tests should be conducted at the desired material content. For contents higher than those in Table 1, additional testing may be required to ensure that cracking problems will not develop in the stabilized material.

The optimum water content and maximum dry density will be identified for the stabilization material content required in Table 1. The maximum dry density will be target value for Stiffness Tests conducted in Step 6. Note that this value will differ from the value obtained in Step 3 for the native (unstabilized) material.

6. Stiffness Testing on Stabilized Subgrade Material

For the stabilization material content tested, appropriate stiffness values will be obtained in accordance with MnDOT Stiffness Test standards (defined in Section 1). To accommodate curing of the stabilized material, samples will be allowed to cure for at least 24 hours between the time of compaction and the time of the stiffness test. Additional time between compaction and stiffness testing may be required for materials having a more extended time-dependent increase in stiffness due to curing.

7. Stiffness Resistance Factor

Once the stiffness of the stabilized subgrade material has been obtained for the stabilization material content under consideration, roadway design can be completed. A Resistance Factor (RF) will be developed based on a ratio of the stiffness of the stabilized material (with stabilization material content as described in Table 1) to the stiffness of the native (unstabilized) material. This RF value shall have a value less than or equal to 2. With respect to ME design, Equation 1 is used to calculate the Resistance Factor:

$$RF = \frac{Mr(stab)}{Mr(native)} \leq 2 \qquad \text{Eq. 1}$$

Where:

RF = Resistance Factor for Stiffness

Mr(stab) = resilient modulus of the stabilized material

Mr(native) = resilient modulus of the native (unstabilized) material

A similar calculation/equation applies where R-value design applies, where the Resistance Factor is taken to be the ratio of the R-value for stabilized and native material.

With respect to design, this Resistance Factor will be used to account for the increased stiffness of the stabilized material with respect to the native material. A factored stiffness for the subgrade may be entered into MnPAVE or appropriate design software to address the degree of improvement.

8. Report

The mix design report shall have the following information:

- The name of the road and other pertinent project information
- A general description of the materials received, their locations, and how samples were obtained
- Density and OMC from Proctor compaction on native material
- Stiffness values at OMC (for NATIVE material) on native material
- Density and OMC from Proctor compaction on stabilized material (at prescribed material content given in Table 1)

- Stiffness values for stabilized material (at prescribed material content given in Table 1) at design moisture content (240 psi exudation pressure) and as-compacted dry density
- Proposed field mix design (stabilization content, moisture content, target stiffness) for subgrade soils relative to the mix required in Table 1
- Proposed stiffness resistance factor, RF, to be used in design

APPENDIX B

MODIFIED MOISTURE-DENSITY TEST METHOD

1305 THE MOISTURE-DENSITY RELATIONS OF SOILS USING A 2.5kg (5.5 LB) RAMMER AND

A 305mm (12 INCH) DROP

AASHTO Designation T 99, Method "C" (MnDOT Modified-

Stabilized materials addressed)

1305.1 SCOPE

This method of test is intended for determining the relation between the moisture content and density of soils compacted in a mold of a given size with a 2.5kg (5.5 lb.) rammer dropped from a height of 305mm (12 ").

1305.2 APPARATUS

A. Molds - The molds shall be solid-wall, metal cylinders manufactured with dimensions and capacities shown in 1305.2A.1 and 2A.2 below. They shall have a detachable collar assembly of, approximately,

60mm (2 3/8") in height, to permit the preparation of compacted specimens of soil-water mixtures of the desired height and volume. The mold and collar assembly shall be so constructed that it can be fastened firmly to a detachable base plate made of the same material.

NOTE 1: Alternate types of molds with capacities as stipulated herein may be used, provided the test results are correlated with those of the solid-wall mold on several soil types and the same moisture-density results are obtained. Records of such correlations shall be maintained and readily available for inspection, when alternate types of molds are used.

1. A 101.6mm (4") mold having a capacity of 0.000943 ± 0.000008m^3; (1/30 [0.0333] ± 0.0003 cubic feet) with an internal diameter of 101.60mm ± 0.41mm (4.000 ± 0.016 inches) and a height of 116.43mm ± 0.13mm (4.584 ± 0.005 inches).

2. A 152.4mm (6") mold having a capacity of 0.002124 ± 0.000021m^3; (1/13.33 [0.07500] ± 0.00075 cubic feet) with an internal diameter of 152.40mm ± 0.66mm (6.000 ± 0.026 inches) and a height of 116.43mm ± 127.0mm (4.584 ± 0.005 inches).

B. Rammer

1. Manually Operated - Metal rammer having a flat circular face of

   50.80mm ± 0.25mm (2.000 ± 0.01 inches), a manufacturing tolerance of ± 0.25mm (0.01 inches) and weighing 2.495kg ±

   9g (5.50 ± 0.02 pounds). The rammer shall be equipped with a suitable guide-sleeve to control the height of the drop to a free fall of 305 ± 2mm (12.00 ± 0.06 inches) above the elevation of the soil. The guide-sleeve shall have at least 4 vent holes no smaller than 9.5mm (3/8") in diameter which are spaced approximately 1.57 radians (90°) apart and approximately

   19mm (3/4") from each end; and shall provide sufficient clearance so the free fall of the rammer shaft and head is unrestricted.

2. Mechanically Operated - A metal rammer which is equipped with a device to control the height of drop to a free fall of 305 ±

   2mm (12.00 ± 0.006 inches) above the elevation of the soil and uniformly distributes such drops to the soil surface. The rammer shall have a flat circular face 50.80mm ± 0.25mm (2.000 ± 0.01") in diameter and a manufactured mass of

   2.495kg ± 9g (5.50 ± 0.02 pounds).

   NOTE 2: The mechanical rammer shall be calibrated with several soil types and the mass of the rammer adjusted, if necessary, to give the same moisture-density results as with the manually operated rammer.

   It may be impractical to adjust the mechanical apparatus so the free fall is

   305mm (12.00") each time the rammer is dropped, as with the manually operated rammer. To make the adjustment of free fall, the portion of loose soil to receive the initial blow should be slightly compressed with the rammer to establish the point of impact from which the drop is determined. Subsequent blows on the layer of soil being compacted may all be applied by dropping the rammer from a height of 305mm (12") above the initial setting elevation. Or, when the mechanical apparatus is designed with a height adjustment for each blow, all subsequent blows should have a rammer free fall of 304.8mm (12.00") measured from the elevation of the soil as compacted by the previous blow.

3. Rammer Face - The circular rammer shall be used but a sector face may be used as an alternative provided the report shall indicate type of face used other than the 50.8mm (2") circular face. It shall have an area equal to that of the circular face rammer.

C. Sample Extruder - A jack, lever, frame, or other device adopted for the purpose of extruding specimens from the mold.

D. Balances - Balance conforming to the requirements of AASHTO M 231 (Classes G2 & G20) having a sensitivity and readability to 0.1 grams and an accuracy of 0.1 grams or 0.1%. Balances shall be appropriate for the specific use.

E. Drying Oven - A thermostatically controlled drying oven capable of maintaining a temperature of 110 ± 5 °C (230 ± 9 °F) for drying moisture samples.

F. Straightedge - A hardened-steel straightedge at least 250mm (10") in length. It shall have one beveled edge, and at least one longitudinal surface (used for final trimming) shall be plane within 0.250mm per 250mm (0.01" per 10") (0.1 percent) of length within the portion used for trimming the soil.

G. Sieves - 50, 19.0, 9.5 and 4.75mm (2", 3/4", 3/8", and #4) sieves with bottom pan conforming to the requirements of AASHTO M 92.

H. Mixing Tools - Miscellaneous tools such as mixing pan, spool, trowel, spatula, etc., or a suitable mechanical device for thoroughly mixing the sample of soil with increments of water.

I. Containers - Containers for moisture content samples shall made of metal or other suitable materials, with close-fitting lids to prevent loss of moisture prior to or during weighing.

1305.3 SAMPLE (Refer to Sections 1301.4 and .5)

A. If the untreated/unstabilized soil sample is damp when it comes from the field, dry it until it becomes friable under a trowel. Drying may be in air or in a drying apparatus such that the temperature of the sample does not exceed 60 °C (140 °F). Then thoroughly break up the aggregations in such a manner as to avoid reducing the natural size of the individual particles. See Section 1301.4 for details.

B. Sieve an adequate quantity of representative, pulverized soil. Discard the +50mm (2") material. Weigh and discard the portion retained on the 19.0mm (3/4") sieve. Replace the discarded 50 - 19.0mm (2 - 3/4") material with material that passes the 19.0mm (3/4") sieve and is retained on the 4.75mm (#4) sieve that is from the original sample or from another sample having similar characteristics. See Section 1301.5 for details.

1305.4 PROCEDURE

A. This test typically involves compaction testing on natural/unstabilized materials. This revised specification allows for testing of stabilized materials, as well. The unstabilized soil sample will be tested in the traditional manner, obtaining the moisture-density relationship for the untreated/natural soil. In addition, tests will be conducted on the stabilized soil in an effort to obtain the moisture-density relationship for the stabilized material. As numerous stabilization materials and techniques are available, this specification is generic in nature. As the testing procedure is implemented, more specific procedural recommendations will be incorporated.

The initial test consists of compacting a portion of the natural soil sample in a mold at different moisture contents ranging from dry to wet. At least four samples will be run. The samples will differ in moisture content by one or two percent with the driest sample being about four percentage points below optimum moisture. This would result in two of the samples being below optimum, one near optimum and one over optimum moisture. A valid test will have two points below optimum.

Additionally, when stabilized soils are to be addressed, as a subsequent test several stabilized soil samples will be tested over a range of water contents for a prescribed stabilization material content.

For solid stabilizing agents (i.e., cement, fly ash, etc.) the amount of stabilization agent and the range of water contents should be in accordance with recommended mix design procedure for stabilized subgrades. The soil should be tested over a range of water contents comparable to those used for the initial (unstabilized) testing. Due to required hydration demands for the stabilization agent, the water content range may extend lower or higher than the range tested during the initial compaction tests of the untreated soil.

For solid stabilization agents, a fixed amount of stabilization material should be used for each test, as per mix design recommendations, with a range of water contents tested to obtain an appropriate moisture-density relationship for the stabilized material. More than four data points may be required to obtain the complete moisture-density curve for the stabilized material.

As per the mix design procedure, the mix should be allowed to hydrate over an appropriate period of time when time effects will be present, with samples prepared to various moisture contents and tested after adequate hydration has occurred.

NOTE 3: If heavy clay or organic soils exhibiting flat elongated curves are encountered, the water content increments may be increased to a maximum of 4 percent.

B. For natural/untreated soil, thoroughly mix the selected representative sample with sufficient water to dampen it to approximately 4 percentage points below optimum moisture content. A good indication of the soil being nearly right for the first point is if the soil barely forms a "cast" when squeezed.

For stabilized soil, thoroughly mix the selected representative sample with sufficient water to dampen it to an initial water content similar to the low-end of the natural/untreated soil. Once the soil and water have been adequately mixed, add the stabilizing agent to the soil and mix thoroughly. As mentioned above, if hydration time is required for the stabilizing agent, seal the soil sample in a covered container (as per NOTE 4) and allow the soil to hydrate for an appropriate time period. Follow the alternate procedure given below.

NOTE 4: Soils that are plastic and cohesive or friable may be (after additional moisture has been added) placed in a suitable covered container to keep from drying out and soaked overnight.

C. After the soil is thoroughly mixed and dampened, push the soil to the edge of the mixing pan to form a circle or ring. Scoop out a portion of the soil ring from four opposite sections. This will provide a good, representative sample for one layer. This should be done for every layer.

D. Form a specimen by compacting the prepared soil in the 101.6mm (4") mold (with collar attached) in three approximately equal layers to give a total compacted depth of about 127mm (5"). Compact each layer by 25 uniformly distributed blows from the rammer dropping free from a height of 305mm (12") above the elevation of soil when a sleeve type rammer is used, or from 305mm (12") above the approximate elevation of each finally compacted layer when a stationary mounted type rammer is used. During compaction, the mold shall rest firmly on a dense, uniform, rigid and stable foundation.

NOTE 5: The following has been found to be a satisfactory base on which to rest the mold during compaction of the soil: A block of concrete weighing not less than 91kg (200 lbs.), supported by a sound foundation, or a sound concrete floor.

E. Following compaction, the top or final layer should be about 12.5mm (1/2") over the top of the mold when the collar is removed. Carefully trim the excess 12.5mm (1/2") of compacted soil even with the top of the mold by means of the straight edge. Holes developed in the surface by the removal of coarse material shall be filled with finer material hand-pressed into place. Carefully trim around any stones that are at least half buried and solidly seated.

F. Clean all loose material from the mold and then weigh the mold and moist soil in kilograms to the nearest 5 grams or pounds to the nearest 0.01 lb. For molds conforming to tolerances given in 1305.2 and masses recorded in kilograms, multiply the mass of the compacted specimen and mold, minus the mass of the mold by 1059.43, and record the result as wet density, W_1 in kg/m^3. For molds conforming to tolerances in Section 1305.2, and masses recorded in pounds, multiply the mass of the compacted specimen and the mold, minus the mass of the mold by 30, and record the results as the wet density, W_1, in lbs/ft of compacted soil.

G. Remove the material from the mold and slice vertically through the center. Take a representative sample of the material from one of the cut faces, weigh immediately, and dry in an oven at 110 ± 5 °C (230 ± 9 °F) for at least 12 hours or to a constant weight to determine the moisture content. The moisture content samples shall weigh not less than 500g. A 12.5 - 19.0mm (1/2 - 3/4") wide section sliced from the center usually provides the 500 grams.

NOTE 6: A representative section must consist of material from all three layers.

H. Thoroughly break up the remainder of the material until it passes a 19.0mm (3/4") sieve and 90% of the soil aggregations will pass a 4.75mm (#4) sieve as judged by eye, and add to the remaining portion of the sample being tested. Add water in sufficient amounts to increase the moisture content of the soil sample by one or two percentage points (90 cc, ml, or grams of water will increase the moisture content of 4.5kg [10 lbs.] of material about 2%). Additional water may be needed to replace moisture lost by evaporation during mixing and between points. Repeat the above procedure for each increment of water added. Continue this series of determinations until there is either a decrease or no change in the wet mass, W_1, per cubic meter or cubic ft. of compacted soil.

NOTE 7: In each repetition the material shall be thoroughly mixed before compaction to assure uniform dispersion of the moisture throughout the sample.

1305.5 ALTERNATE PROCEDURE

If the material being tested is fine-grained and cohesive, it is difficult to mix and to break up after compaction; if the material is soft and fragile it may change gradation during compaction. These qualities require slightly different procedures to obtain reliable moisture-density information.

A. Prepare the original sample as outlined in Section 1305.3.

B. Select 11 - 14kg (25 - 30 lbs.) of the prepared material.

C. Moisten or dry the sample to about 4% below the estimated optimum moisture content. At this point when the soil is squeezed in the hand, a "cast" is barely formed. For granular soils (less than 20% passing the 75µm [#200 sieve]) this "cast" should crumble easily when touched.

D. This alternate method requires preparing a separate portion for each compaction test rather than using the same material over again; therefore, divide the sample into 4 or 5 portions, about 2.25kg (5 lbs.) each.

E. Place each portion into a watertight container.

F. Cover and set aside one portion. Mark it "Point #1".

G. Uniformly increasing increments of water shall be mixed with the separate portions to obtain a series of 3 or 4 additional moisture contents beyond Point 1, ranging from dry side to the wet side of optimum moisture.

H. Add enough water to one of the remaining portions to increase the moisture content about 2% over Point #1. Thoroughly mix, cover and mark this portion "Point #2".

NOTE 8: Forty-five cc or grams of water added to 2.25kg (5 lb.) of material will increase the moisture content about 2%.

I. Add 90 cc of water to another portion, thoroughly mix, cover and mark it "Point 3".

J. Add 135 cc of water to another portion, thoroughly mix, cover and mark it "Point 4".

K. Add 180 cc of water to another portion, thoroughly mix, cover and mark it "Point 5".

L. Allow the "points" to soak overnight to permit the moisture to disperse through the soil.

M. Each portion shall then be remixed the following day and compacted following steps B through H of 1305.4.

1305.6 CALCULATIONS

A. MOISTURE CONTENT of the material, as compacted for each trial, calculate as follows:

$$\text{Percent Moisture} = \frac{E - F}{F - H} \times 100$$

WHERE:

E = Weight of container and wet soil. F

= Weight of container and dry soil. H =

Weight of container.

In the example (See Section 1305.8):

$$\text{Percent Moisture} = \frac{270 - 243}{243 - 13} \times 100 = 11.7$$

B. WET DENSITY (wet weight in pounds per cubic foot of material compacted) calculate as follows:

Wet Density (kg/m^3) = (A - B) X 1059.43

WHERE:

A = Weight of wet soil and mold

B = Weight of mold

In the example (See Section 1305.8) the following calculation is made for "Point 1":

Wet Density (kg/m^3) = (7.189 - 5.488) X 1059.43 = 1802

C. DRY DENSITY (dry weight in kg/m^3) calculate as follows:

$$\text{Dry Density } (kg/m^3) = \frac{\text{Wet Density}}{\text{\% Moisture} + 100} \times 100$$

Using the above calculated values for % Moisture and Wet Density, the following calculation can be made for "Point 1" (See Section 1305.8):

$$\text{Dry Density } (kg/m^3) = \frac{1802}{11.7 + 100} \times 100 = 1613$$

1305.7 MAXIMUM DENSITY and OPTIMUM MOISTURE CONTENT

A. The Maximum Density and Optimum Moisture are determined plotting the information obtained by compacting the sample at various moisture contents. Each moisture content relates to a Wet and a Dry Density.

B. Plot (on the Moisture-Density Relationship graph form, see Section 1305.9) the calculated Dry Densities against the Moisture Contents that were calculated previously and shown in the example (see Section 1305.8). In the examples the following points are plotted:

POINT NUMBER	1	2	3	4
% MOISTURE	11.7	13.8	16.6	18.4
DRY DENSITY	1613	1651	1682	1653

C. Draw a smooth curve through the points.

D. The moisture content corresponding to the peak of the curve shall be termed the "Optimum Moisture Content" of the soil under compaction. In the example, 16.5% is the "Optimum Moisture".

E. The "Maximum Density" also corresponds to the highest point on the "Dry" curve. In the example 1682kg/m^3 is the "Maximum Density".

1305.8 EXAMPLE - COMPUTATION SHEET for the PROCTOR MOISTURE - DENSITY CURVE

	Point Number	1	2	3	4	6	7
A	Weight of Wet Soil + Mold	7.189	7.262	7.339	7.335		
B	Weight of Mold	5.488	5.488	5.488	5.488		
C	Weight of Wet Soil	1.701	1.774	1.851	1.847		
D	Wet Density (kg/m^3)	1802	1879	1961	1956		
	Can Number	1	2	3	4		
E	Weight of Wet Soil + Can	270	287	349	376		
F	Weight of Dry Soil +Can	243	254	301	320		
G	Moisture Loss	27	33	48	56		
H	Weight of Can	13	14	11	15		
I	Weight of Dry Soil	230	240	290	305		
K	Percent Moisture	11.7	13.8	16.6	18.4		
	Dry Density (kg/m^3)	1613	1651	1682	1651		

REMARKS:

APPENDIX C

MODIFIED RESILIENT MODULUS TEST METHOD

MNDOT RESILIENT MODULUS (MR) TESTING PROTOCOL AND DATA QUALITY CONTROL CRITERIA

The current MnDOT resilient modulus test, in general, follows the protocol developed from NCHRP 1-28a project. One of the objectives of NCHRP 1-28a was to recommend an enhanced protocol to ASTM, SHRP, AASHTO and NCHRP 1-28 procedures. This protocol was intended to support MEPDG. The protocol can be found in the Appendix 2 of the NCHRP Research Results Digest 285.

MnDOT has made an improvement in the test configuration. The NCHRP 1-28a protocol only uses two LVDTs, but MnDOT requires three on-specimen LVDTs to measure the specimen deformation. The three LVDTs are mounted between two aluminum rings and equally spaced around the specimen to directly measure axial displacement of the specimen. In terms of data acquisition, MnDOT recommends the sampling rate of at least 400 points/second for data acquisition. The baseline values for Mr calculation is determined by taking the average load/deformation values of baseline from the last 60% of a time history cycle.

Mr test of a specimen should represent element responds in field. However, the specimen is not homogenous, the specimen deformation is not always uniform through the specimen height. Also, the testing system could contribute errors to the results, such as system noise. Therefore, controlling Mr data quality is important. NCHRP 1-28 recommends an acceptable two vertical LVDT measurement ratio of 1.1 as a quality control criterion. But we found that this criterion cannot adequately address test quality, such as rotation of the specimen. Based on our testing data, MnDOT has developed a set of data quality control criteria to replace the NCHRP recommended 1.1 ratio criterion. There criteria are Angle of Rotation (AR); Signal-to-Noise ratio (SNR) and Coefficient of Variation of resilient modulus (COV). With application to stabilized subgrade materials, the same criteria with respect to Angle of Rotation (AR); Signal-to-Noise ratio (SNR) and Coefficient of Variation of resilient modulus (COV) will apply, as given at http://www.dot.state.mn.us/materials/mr/pdfs/mrlabtesting.pdf (accessed 18 June 2012).

In general, when considering subgrade materials to be stabilized with fly ash or cement, an identical procedure will be followed for resilient modulus tests as with native/unstabilized soils. Soils will be prepared to water contents and stabilization material contents as dictated by the mix design procedure (as per Appendix A), at which point the same testing procedure will be followed as per a test performed on the unstabilized material.