Optimization and Management of Materials in Earthwork Construction

Final Report
April 2010

IOWA STATE UNIVERSITY
Institute for Transportation

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(InTrans Project 04-162)

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The preparation of this (report, document, etc.) was financed in part through funds provided by the Iowa Department of Transportation through its "Agreement for the Management of Research Conducted by Iowa State University for the Iowa Department of Transportation," and its amendments.

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Technical Report Documentation Page

1. Report No. IHRB Project TR-501	**2. Government Accession No.**	**3. Recipient's Catalog No.**
4. Title and Subtitle Optimization and Management of Materials in Earthwork Construction		**5. Report Date** April 2010
		6. Performing Organization Code
7. Author(s) Longjie Hong, Vernon R. Schaefer, Radhey S. Sharma, and David J. White		**8. Performing Organization Report No.** InTrans Project 04-162
9. Performing Organization Name and Address Institute for Transportation Iowa State University 2711 South Loop Drive, Suite 4700 Ames, IA 50010-8664		**10. Work Unit No. (TRAIS)**
		11. Contract or Grant No.
12. Sponsoring Organization Name and Address Iowa Highway Research Board Iowa Department of Transportation 800 Lincoln Way Ames, IA 50010		**13. Type of Report and Period Covered** Final Report
		14. Sponsoring Agency Code
15. Supplementary Notes Visit www.intrans.iastate.edu for color PDF files of this and other research reports.		

16. Abstract

As a result of forensic investigations of problems across Iowa, a research study was developed aimed at providing solutions to identified problems through better management and optimization of the available pavement geotechnical materials and through ground improvement, soil reinforcement, and other soil treatment techniques. The overall goal was worked out through simple laboratory experiments, such as particle size analysis, plasticity tests, compaction tests, permeability tests, and strength tests. A review of the problems suggested three areas of study: pavement cracking due to improper management of pavement geotechnical materials, permeability of mixed-subgrade soils, and settlement of soil above the pipe due to improper compaction of the backfill. This resulted in the following three areas of study:

- The optimization and management of earthwork materials through general soil mixing of various select and unsuitable soils and a specific example of optimization of materials in earthwork construction by soil mixing
- An investigation of the saturated permeability of compacted glacial till in relation to validation and prediction with the Enhanced Integrated Climatic Model (EICM)
- A field investigation and numerical modeling of culvert settlement

For each area of study, a literature review was conducted, research data were collected and analyzed, and important findings and conclusions were drawn. It was found that optimum mixtures of select and unsuitable soils can be defined that allow the use of unsuitable materials in embankment and subgrade locations. An improved model of saturated hydraulic conductivity was proposed for use with glacial soils from Iowa. The use of proper trench backfill compaction or the use of flowable mortar will reduce the potential for developing a bump above culverts.

17. Key Words culverts—geo-materials—select soils—unsuitable soils		**18. Distribution Statement** No restrictions.	
19. Security Classification (of this report) Unclassified.	**20. Security Classification (of this page)** Unclassified.	**21. No. of Pages** 124	**22. Price** NA

Form DOT F 1700.7 (8-72) **Reproduction of completed page authorized**

OPTIMIZATION AND MANAGEMENT OF MATERIALS IN EARTHWORK CONSTRUCTION

Final Report
April 2010

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Sponsored by
the Iowa Highway Research Board
(IHRB Project TR-501)

Preparation of this report was financed in part
through funds provided by the Iowa Department of Transportation
through its research management agreement with the
Institute for Transportation,
InTrans Project 04-162.

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ACKNOWLEDGMENTS

The Iowa Highway Research Board and the Iowa Department of Transportation sponsored this project under contract TR-501. The opinions, findings, conclusions, and recommendations expressed in this report are those of the authors and do not necessarily reflect the views of the sponsors.

EXECUTIVE SUMMARY

Natural earth materials play an important role in the design and construction of geotechnical systems. Different earth materials, such as soil and rock, have been used in the construction of various geotechnical systems, including foundations, retaining walls, embankments, road and airfield pavements, box culverts, and bridge abutments. The choice of particular geo-materials for a construction project depends on the type and purpose of the geotechnical system itself. Some geo-materials, such as peat, muck, expansive/swelling soils, and collapsible soils, however, cannot be used in any type of construction because the severity of post-construction damage they may cause can be disconcerting. Often, site soils are unacceptable for the intended function and must be improved or replaced with better quality materials.

As a result of forensic investigations of problems across Iowa, a research study was developed aimed at providing solutions to identified problems through a better management and optimization of the available pavement geotechnical materials and through ground improvement, soil reinforcement, and other soil treatment techniques. The overall goal was worked out through simple laboratory experiments, such as particle size analysis, plasticity tests, compaction, permeability, and strength tests. A review of the problems suggested three areas of study: pavement cracking due to improper management of pavement geotechnical materials, permeability of mixed-subgrade soils, and settlement of soil above the pipe due to improper compaction of the backfill. This resulted in the following three areas of study:

1. The optimization and management of earthwork materials through general soil mixing of various select and unsuitable soils and a specific example of optimization of materials in earthwork construction by soil mixing.
2. An investigation of the saturated permeability of compacted glacial till relation to validation and prediction with the Enhanced Integrated Climatic Model (EICM).
3. A field investigation and numerical modeling of culvert settlement.

For each area of study, a literature review was conducted, research data were collected and analyzed, and important findings and conclusions were drawn. It was found that optimum mixtures of select and unsuitable soils can be defined that allow the use of unsuitable materials in embankment and subgrade locations. An improved model of saturated hydraulic conductivity was proposed for use with glacial soils from Iowa. The use of proper trench backfill compaction or the use of flowable mortar will reduce the potential for developing a bump above culverts.

INTRODUCTION

All kinds of natural earth materials play an important role in the design and construction of geotechnical systems. Different earth materials, such as soil and rock, have been used in the construction of various geotechnical systems, including foundations, retaining walls, embankments, road and airfield pavements, box culverts, and bridge abutments. The choice of particular geo-materials for a construction project depends on the type and purpose of the geotechnical system itself. Some geo-materials, such as peat, muck, expansive/swelling soils, and collapsible soils, however, cannot be used in any type of construction because the severity of post-construction damage they may cause can be disconcerting. Often site soils are unacceptable for the intended function and must be replaced with better quality materials or improved. One method of improvement is to mix high-quality site materials with lesser quality site materials to provide an acceptable soil material.

A field investigation of some embankments, subgrades, and pavements in the state of Iowa revealed different kinds of site-specific problems, such as pavement cracking and pipe distress. These problems have been identified as the following during field investigation:

- Cracking of pavements in longitudinal and transverse directions
- Rutting of pavements, which is due to poor construction
- Poor compaction of the backfill material (sand and rock backfill) of the drain pipe, leading to uneven settlement of pavements
- Bumps experienced by vehicles on points where drain pipes lie due to depression

Most of the pavements have been found to develop some amount of depression where drain pipes lie, causing vehicles to experience an inconvenient bump. These problems are related to the poorly compacted backfill (crushed lime stone) of the pipe. The settlement of the backfill material under the traffic loads causes bumps on the pavement surface and distresses to the drain pipes buried in the sand backfill. These distresses might lead to cracking pipes, resulting in water leakage into the subsoil.

Soil permeability is a key parameter for the stability of subgrade soils. Permeability governs engineering problems, such as the flow of water through or around embankment and the consolidation of embankment soils under applied loads. Permeability is also of great importance in connection with problems of seepage, settlement, and stability of embankment and pavement structures.

A review of the problems suggested that they could be divided primarily into the following three groups:

- Pavement cracking due to improper management of pavement geotechnical materials
- Settlement of soil above the pipe due to improper compaction of the backfill
- Permeability of mixed-subgrade soils

Objectives and Scope

The research project aims at providing solutions to these problems through a better management and optimization of the available pavement geotechnical materials and through ground improvement, soil reinforcement, and other soil treatment techniques. The overall goal was worked out through simple laboratory experiments, such as particle size analysis, plasticity tests, compaction, permeability, and strength tests. The geotechnical applications and the various properties of the material serve as a basis for predicting its engineering performance and its suitability.

One of the primary objectives and plans of this research was to evaluate the engineering properties of embankment materials by mixing different soils, such as the select and unsuitable soils in different proportions. Grain-size distribution, plasticity, compaction characteristics, permeability, and shear strength characteristics were determined for various mixes. Based on the amount of improvement in the engineering behavior, the optimum mix was selected and suggested for the field conditions. Other objectives were to evaluate the permeability of mixed materials and to evaluate the use of flowable mortar in place of conventional backfill material around a drainage pipe.

Report Organization

This report consists of three sections describing the three areas of research:

1. The optimization and management of earthwork materials through general soil mixing of various select and unsuitable soils and a specific example of optimization of materials in earthwork construction by soil mixing.
2. An investigation of the saturated permeability of compacted glacial till relation to validation and prediction with the Enhanced Integrated Climatic Model (EICM).
3. A field investigation and numerical modeling of culvert settlement.

Each section has a literature review, research data, important findings, and conclusions. The report is concluded with a summary of the main findings of the research, suggestions for future research, and references. A brief introduction of each section is presented next.

Optimization and Management of Earthwork Materials through Soil Mixing

Three materials available for earthwork construction are, based on their suitability, classified as select, suitable, and unsuitable soils. Unsuitable soils are some expansive/swelling or collapsible soils with low density and low strength, which should be disposed of at least 3 feet (1 meter) below subgrade elevation. As the large amount of money involved in carrying out the remedial measures is a limitation, it is agreed that management and optimization of the available materials be focused on by mixing the materials among themselves in various proportions and observing the response. Because of the availability and the fact that unsuitable soils are much cheaper than select soils, making use of unsuitable soils can reduce the cost of earthwork construction. In this research, two unsuitable soils were mixed with six select soils at various proportions (unsuitable: select = 25%:75%, 50%:50%, and 75%:25%) to investigate how the engineering properties can be improved at different select-unsuitable ratios. Experimental results indicate that maximum dry density and unconfined compression strength both increase while the optimum moisture content decreases linearly with increasing select proportion. Moreover, regression analysis shows that maximum dry density decreases linearly with increasing optimum moisture content. To reduce the cost of earthwork construction, a mixture of select-unsuitable at a ratio of 3:1 can still be used as select soil and be placed on the top two feet (0.6 meter) of the subgrade.

The site-specific example was from the Highway 34 bypass near Fairfield, Jefferson County, Iowa. The typical stratigraphy of this area is, from top to bottom, clay pan, a loess, paleosol (gumbotil), and Kansan glacial till layer. The clay pan and gumbotil have high swell potential, while loess is a collapsible material. The glacial till layer was classified as a select soil. Due to the large amount of unsuitable soils along the route, the potential for soil mixing of select and unsuitable soil for improvement of the unsuitable soil was studied. The change in the engineering properties (such as plasticity index, unconfined compressive strength, compaction characteristics etc.) of select-unsuitable blends with various mixing proportions is presented.

Permeability of Compacted Glacial Till Related to Validation and Prediction with the Enhanced Integrated Climatic Model (EICM)

Moisture and temperature are two environmental variables that affect the performance of pavement structure and subgrade. These variables have been incorporated in the *Mechanistic-Empirical Pavement Design Guide* through a sophisticated climatic modeling tool called the Enhanced Integrated Climatic Model (EICM). Permeability of the subgrade soil is a required input for this model. One of the major tasks undertaken in developing the EICM is the development of improved estimates of saturated hydraulic conductivity, k_{sat}, based on soil index properties such as fine contents, P_{200}, effective diameter, D60, and plasticity index, PI. This estimation is used when field and laboratory data are not available and has been proved to have a good agreement with an extended database. However, EICM model has some limitations; it can only predict the permeability of compacted soils at optimum moisture contents under standard Proctor compaction. To expand this empirical model for practical purpose, a more comprehensive model was developed in this study.

Field Investigation and Numerical Modeling of Culvert Settlement

Culverts are commonly built to deal with the highway drainage needs. However, settlement adjacent to highway culvert has been found shortly after the new highway is open to the traffic. Although it is not perceptible to the naked eyes, it is noticeable in a vehicle driving though these locations. To address this issue, an investigation of the causes of the problem and the development of a solution was undertaken.

Settlement is a common problem with the use of culverts and is often due to poorly compacted sand backfill and rock backfill (crushed limestone) materials. It can also result from settlement of the culvert in soft foundation material, displacement of soft material, or piping along the culvert. The settlement of backfill materials causes bumps on the pavement surface and distress to the drain pipes buried in the sand backfill. This distress might lead to cracking of pipes, resulting in leakage of water into the subsoil. If the subsoil consists of problematic soils, such as expansive soils or loess collapsible soils, seeping of water into these soils could trigger further intricate problems of volume changes detrimental to the engineering performance of pavements.

Although there is considerable information available on the design and construction of new culverts, there is little information in the literature on how to repair culvert problems and even less on how to rehabilitate, strengthen, or retrofit upgrade culverts. Initiating any kind of remedial measures requires a thorough investigation of the soil profile and properties of different soils in different strata. The objectives of this study were to investigate culvert settlement problems in Iowa, review the remediation methods in the literature, and study the select and flowable mortar as backfill options.

OPTIMIZATION OF MATERIALS IN EARTHWORK CONSTRUCTION—PROPORTIONING OF FOUNDATION/SUBGRADE MATERIALS

Introduction

Earth materials in the form of soil and rock have been used in the construction of various geotechnical systems, including foundations, retaining walls, embankments, and road and airfield pavements. Performance of the system depends on both the properties of the soil and how the soil is processed (compacted). In general, most soils can be processed such that their engineering properties will be acceptable. Certain geo-materials, such as peat, muck, expansive/swelling soils and collapsible soils, however, need to be used carefully in any type of construction because the severity of post-construction damage they may cause can be disconcerting. In Iowa, for application to roadway construction, a rapid performance-based classification system, the Iowa Empirical Performance Classification (EPC), is used to classify soils into three groups: select, suitable, and unsuitable (White et al. 2002). The choice of particular geo-materials for construction depends on the type and purpose of the geotechnical system itself. Select soils are those placed directly under the pavement structure (0 to 0.6 m) as subgrade. The normal select materials are clay loam, loam, or sand. Select soils contain either predominantly sand or a mixture of sand, silt, and clay (Iowa DOT Specification 2001). Because of the composition of a select soil, the density and the shear strength are much higher when they are properly compacted. Suitable soils are placed under select soils (0.6 to 1.5 m) and are usually in the zone of seasonal freeze/thaw and wetting and drying cycles. Unsuitable soils are buried beneath the suitable soils (1.0 to 1.5 m below top of subgrade). These soils are materials that cannot be consolidated properly in the embankment, including highly plastic clays or highly compressible frost-prone silts. It should be noted that if there is excessive unsuitable, it needs to be removed before construction. Table 1 shows current Iowa Department of Transportation (Iowa DOT) specifications for “select,” “suitable,” and “unsuitable” soils.

Based on their limited availability, select soils are generally more expensive to use than unsuitable soils, while unsuitable soils usually have to be wasted, thus increasing costs when they are encountered on a project site. To improve the engineering properties of the soils and reduce the cost of construction, mixing unsuitable soils with select soils has been proposed in this research. Engineering properties of the blends have been evaluated and compared with those of the natural unblended materials. This technique of optimization of the available geotechnical materials, it is hoped, would serve the purpose because proportioning and mixing different embankment fill materials would improve the engineering properties of the blends. Hence, the pavement constructed on these blends would possibly give a better engineering performance.

Currently, soil mixed with various chemicals, such as cement, lime, and fly ash, has been used in the field (Hunter 1988; Winterkorn et al. 1991; Petry and Little 1992; Rollings et al. 1999; Acosta et al. 2003; Hoyos et al. 2004; and Phani Kumar and Sharma 2004). Of the various additives used for stabilizing soils, lime, fly ash (Chen 1988; Rao 1984; Sankar 1989; Cokca 2001), and calcium chloride (Desai and Oza 1997) have shown promise because they reduced the amount of volume change and improved the strength characteristics. Most nonexpansive clays pose the problem of large compression and low shear strength at high water contents. The engineering behavior of nonexpansive clays also showed improvement upon stabilization with additives like lime, cement, and fly ash (Broms and Boman 1977; Chen 1988; Kaniraj and Havanagi 2001).

Table 1. Current Iowa DOT specification for cohesive soil classification into “select,” “suitable,” and “unsuitable” categories (IDOT 2001)

Select soils	Suitable soils	Unsuitable soils
Must meet all conditions – typically used in top 0.6m of subgrade	Must meet all conditions – used throughout fill except for top 0.6m of subgrade	Requirements for use at different depths
1. 45 percent or less silt size fraction (0.075 - 0.002 mm)	a. 1500 kg/m^3 or greater density (AASHTO T99 Proctor density)	Slope dressing only - peat or muck - soil with plastic limit ≥ 35 - A-7-5 or A-5 having density < 1350 kg/m^3
2. 1750 kg/m^3 or greater density (AASHTO T99 Proctor density	b. Group Index < 30 (AASHTO M 145 - 91)	• Disposal 1 m below top of subgrade - All soils other than A-7-5 or A-5 having density <1500 kg/m^3 - All soils other than A-7-5 or A-5 containing < 3.0% carbon
3. Plasticity index >10		• Disposal 1 m below top of subgrade - A-7-6 (30 or greater) - Residual clays overlying bedrock regardless of classification
4. A-6 or A-7-6 soils of glacial origin		• Disposal 1.5 m below top of subgrade with alternate layers of suitable soils - shale - A-7-5 or A-5 soils having density from 1350 kg/m^3 to 1500 kg/m^3

Note: (I) Select soils need to meet all requirements 1 through 4; (II) Suitable soils need to conform to both (a) and (b)

In recent years, natural poor soils mixed with granular materials have been used in construction of base, subbase, and surface courses of paved facilities. Leelanitkul (1989) studied the properties of an existing active clay by adding various proportions of fine sand and found that a 20% sand admixture can adequately improve the properties of the active clay for highway embankment construction. Granular stabilization can obtain a well-proportioned mixture of particles with continuous gradation (well-graded) and the desired plasticity. The granular constituents form a bearing skeleton, while fine portions can provide effective cohesion and cementation. However, more frequently, the natural soils will lack some constituents needed to form a continuous bearing skeleton or to provide the necessary cohesion and cementation. In these cases, the desired mixture can be compounded by addition of proper proportions of the granular materials or fines.

The results of a general mixing study of 13 soils from 5 sites in Iowa as well as the results from a specific site in which the general principles were applied to the specific site soils (Fairfield Bypass, Jefferson County) are presented. The engineering property (such as plasticity index, unconfined compressive strength, compaction characteristics, etc.) changes of select-unsuitable blends with various mixing proportions are presented. An optimal mix design to ensure a select blend is also provided.

General Mixing Experimental Investigations

Materials

Thirteen soils were collected from five different sites within the state of Iowa: Highway 2 near Sidney, Highway 218 near Charles, Highway 20 near Webster, Highway 30 near Le Grand, and Highway 60 near Hospers. At all of these sites, highway embankment construction projects were in progress. The soils were collected from the embankment or nearby borrow pits. The locations of these sites are shown in Figure 1.

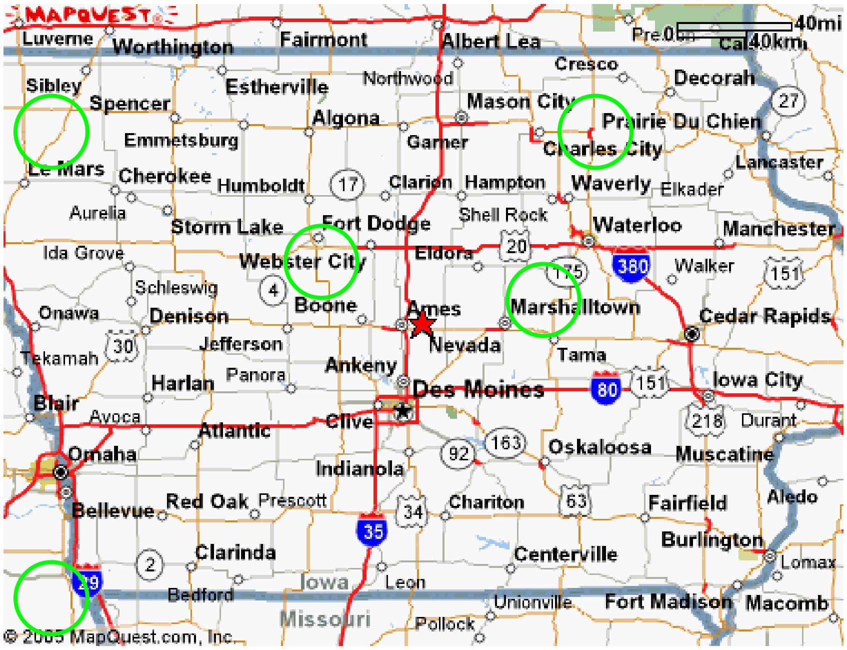


Figure 1. Locations of the sites

Tests Conducted

Laboratory gradation tests, Atterberg limits, compaction, and unconfined compression strength tests were performed in two series. In the first series, these tests were conducted to classify the 13 native soils into select, suitable and unsuitable according to the Iowa EPC. In the second series, the same tests were conducted on each select-unsuitable soil mixture in various proportions.

Gradation tests were conducted in general accordance to the ASTM D2487 "Standard Practice for Classification of Soils for Engineering Purposes." Air-dry soil (500 g) was pulverized and washed through a No. 200 sieve. The soil retained on the sieve was oven-dried and sieved through No. 10, 20, 40, 60, 80, 100, and 200 sieves. The 50 g of soil passing No. 200 sieve was collected and soaked in 125 ml of dispersing agent (40g/l of sodium hexametaphosphate) for 16 hours prior to a hydrometer test. A 152H hydrometer was used for all hydrometer tests.

Atterberg limits tests were performed according to ASTM D4318 "Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils." For the mixtures, select soils passing through the No. 40 sieve were mixed at various proportions with unsuitable soils passing through the No. 40 sieve.

Compaction tests were performed on 2 x 2 in. (0.05 x 0.05 m) cylindrical samples (O'Flaherty et al. 1963). A standard Proctor compaction sample is made in a 1/30 ft^3 (0.028 m^3) cylinder mold using three layers that are each compacted by 25 blows of the 5.5 lb (2.48 kg) rammer dropped 12 inches (0.305 m) for an energy input of 12375 ft-lb/ft^3 (59,942 m-kg/m^3). Another compaction test, the ISU 2 x 2 in. (0.05 x 0.05 m) compaction test method, utilizes specimens 2 inches (0.05 m) in diameter by 2 inches (0.05 m) (approximately) high compacted by 10 blows of the 5 lb (2.25 kg) hammer dropped 12 inches (0.305 m) for an energy input of 13751 ft-lb/ft^3 (66,606 m-kg/m^3), which only requires about one-tenth of the material and one-third of the time needed for standard Proctor specimens. However, maximum dry density and optimum moisture content obtained from two test methods are very close. It has been proven that the ISU 2 x 2 in. (0.305 x 0.305 m) compaction apparatus can be used in lieu of the standard Proctor compaction test to increase productivity.

Unconfined compression strength tests were performed on 2 x 4 in. (2 inches [0.05 m] diameter, 4 inches [0.10 m] height) cylindrical samples at different moisture contents in accordance to ASTM D 2166-00 "Standard Test Method for Unconfined Compressive Strength of Cohesive Soil." Soil was placed in two layers and the compaction energy was doubled compared to the energy for the 2 x 2 in. (0.05 x 0.05 m) samples.

For the 2 x 2 in. (0.05 x 0.05 m) and 2 x 4 in. (0.05 x 0.10 m) samples, air-dried select soils passing through the No. 4 sieve were mixed at various proportions with unsuitable soils passing through the No. 4 sieve. The required weight of the soil was determined based on the placement dry unit weight, water content, and the volume of the specimen. The required amount of water was added to the dry soil and thoroughly mixed.

General Mixing Results and Discussion

Properties of Individual Soil

The index properties of these 13 native soils and their classification are summarized in Table 2. As shown in Table 2, six native soils are classified as select soils (no. 1, 6, 8, 9, 10, and 11), five are suitable (no. 2, 4, 5, 7, and 13), and the other two are unsuitable (no. 3 and 12), according to EPC classification procedures. The gradation curves of the six select soils and two unsuitable soils are shown in Figure 2.

The six select soils are all well-graded glacial tills with coarse-grain proportions and silt and clay content ranging from 30% to 46%, 37% to 56%, and 10% to 21%, respectively. Soil no. 6, 10, and 11 are yellow in color, while no. 1, 8, and 9 are black. They all have a relatively high maximum Proctor density and low optimum moisture content. The maximum dry densities and optimum moisture contents of these six select soils range from 1,857 to 2,030 kg/m^3 and from 10% to 14%, respectively. The liquid limits of these 13 soils range from 25 to 41, and plasticity indices range from 10 to 21.

The unsuitable soils both have high silt contents. Soil no. 12 was collected from Hospers (northwestern Iowa), and it was dark black in color with high silt content (88%) and very little coarse-grain material (about 4% sand and gravel) and clay particles (8% clay). It also has a considerable amount of organic matters (peat and muck). It had low maximum dry density (1495 kg/m^3) and unconfined compression strength (187 kPa) at optimum moisture content of 22%. Soil no. 3 was collected from Sydney, and it was grey in color with high silt content (88%) and no coarse constituent. Its Atterberg limits fell in the low/medium plasticity zone. This soil is frost susceptible and should be buried at least 1 m below the top of the subgrade. The maximum dry density, optimum moisture content, and unconfined compression strength of soil no. 3 are 1,730 kg/m3, 19%, and 302 kPa, respectively. Soil no. 3 is a better material than no. 12, but they both are classified as unsuitable soils.

Except for soil no. 12, all other soils were low-medium plasticity and classified as CL by the Unified Soil Classification System (USCS) and A-6 by AASHTO.

Table 2. Index properties and classifications of individual soil

No.	G_s	LL	PI	% Gravel	% Sand	% Silt	% Clay	Proctor Dry Density	Optimum w%	GI	q_u (kPa)	EPC classification	USCS classification	AASHTO classification
1	2.62	35	17	11	25	45	19	1882	12	9	429	Select	CL	A-6
2	2.67	39	21	0	1	81	18	1746	17	21	341	Suitable	CL	A-6
3	2.58	34	12	0	0	88	12	1730	19	13	302	Unsuitable	CL	A-6
4	2.57	32	15	2	30	60	8	1872	12	8	487	Suitable	CL	A-6
5	2.70	33	13	1	30	58	11	1904	12	7	618	Suitable	CL	A-6
6	2.70	26	13	1	46	37	17	2030	10	4	356	Select	CL	A-6
7	2.65	40	16	0	5	78	17	1746	17	17	348	Suitable	CL	A-6
8	2.69	35	16	3	27	56	14	1938	12	9	336	Select	CL	A-6
9	2.74	34	16	5	29	56	10	1887	13	8	561	Select	CL	A-6
10	2.84	25	10	2	44	40	14	1997	11	2	507	Select	CL	A-6
11	2.71	33	15	3	31	45	21	1857	14	7	444	Select	CL	A-6
12	2.66	41	12	0	4	88	9	1495	22	14	187	Unsuitable	OL	A-4
13	2.63	41	21	0	23	46	31	1796	15	15	314	Suitable	CL	A-6

Note: qu was measured at optimum moisture content.

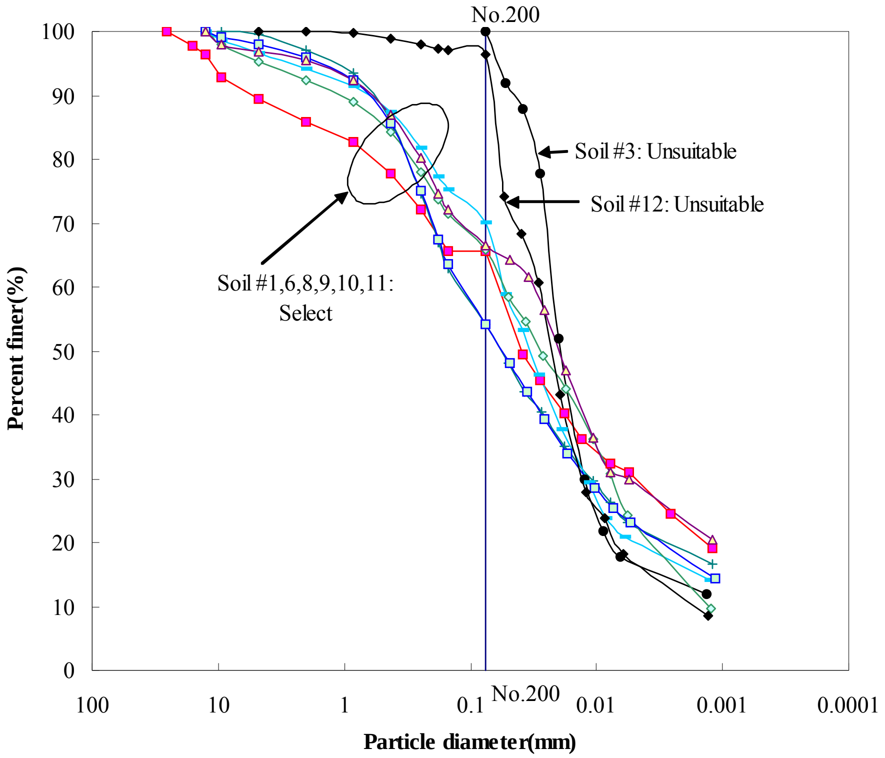


Figure 2. Grain size distribution of select and unsuitable soils

Proportioning and Mixing

When extensive quantities of unsuitable materials are encountered on a project site, the cost of removal and disposal of these materials can be quite high. An alternative is to manage and optimize the available materials by mixing the unsuitable materials with better materials in various proportions and observing the response. When select material is available from the project site or from a nearby borrow pit, the select material can be mixed with the unsuitable soil. The mix should be moisture-conditioned for uniform and easy compaction and compacted layer by layer until the required elevation is attained. Sheepsfoot rollers can be used for the compaction. Before actually deciding on the type of the mix to be used in the field, the physical and engineering properties of different mixes need to be investigated. One of the aims of this research project is to make use of mixing select and unsuitable soils and to determine different engineering properties. In this test program, each select soil (no. 1, 6, 8, 9, 10, or 11) is mixed with each unsuitable soil (no. 3 or 12) at different mixing proportions, as shown in Table 3. Five suitable soils were not used in mixing.

Table 3. Different proportions of materials for mix design

Material	Proportions (%)				
Select	0	25	50	75	100
Unsuitable	100	75	50	25	0

Index Properties of Mixtures

Figure 3 is the Casagrande plasticity chart for all of the mixtures at various proportions. Most of the soils are located in the CL zone (low/medium plasticity to medium plasticity). The soil index properties for two unsuitable soils are also shown in this chart. Because the variation of the liquid limit and plasticity index is small (25 to 41 and 10 to 21, respectively), it is hard to tell the trend of how the Atterberg limits of these mixtures change with the mixing proportions. However, conclusions can be drawn that the index properties of unsuitable soils have been changed after mixing with select soils.

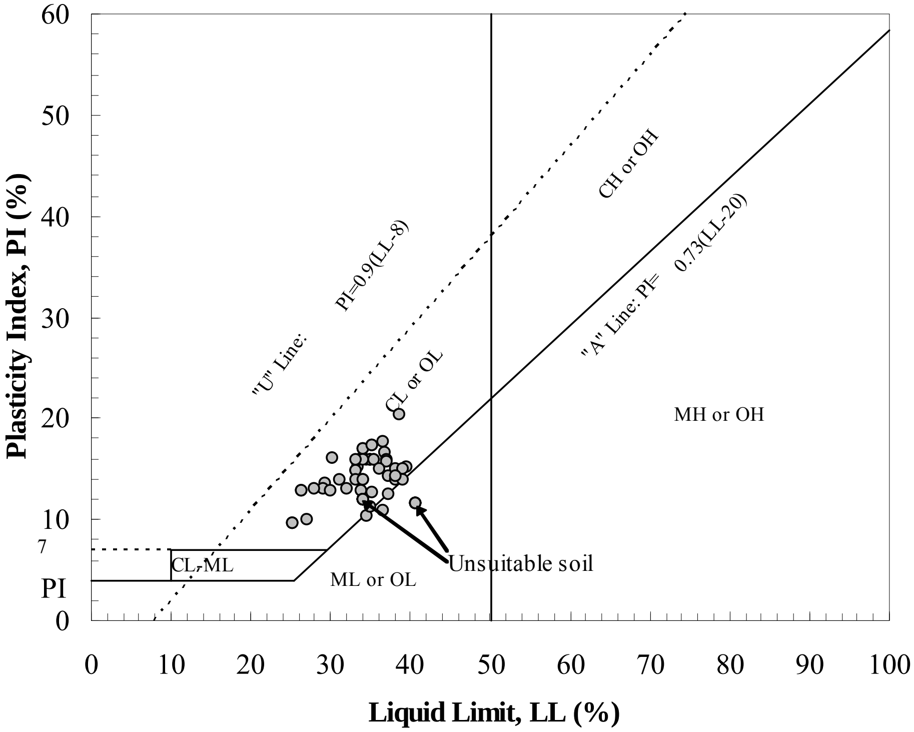


Figure 3. Casagrande plasticity chart for the mixtures and unsuitable soils

Compaction Characteristics

Figure 4 presents Proctor density curves at five mix proportions. Each density curve in Figure 4 represents the variation of dry density with moisture content at a specified ratio of the select-unsuitable mixture. For example, in Figure 4(a), select soil no. 1 was mixed with unsuitable soil no. 3 at proportions 100% and 0%, 75% and 25%, 50% and 50%, 25% and 75%, and 0% and 100%. At least five samples were prepared at different moisture contents for each compaction curve. The zero-air lines of unsuitable soil and select soil are also plotted in Figure 4. The upper most curve in Figure 4(a) is the result of the compaction test from soil no. 1, which is a select soil (proportion of select:unsuitable = 100%:0%), while the lowermost curve represents the density curve for unsuitable soil no. 3 (proportion of select:unsuitable = 0%:100%). These results indicate that select soil alone has the highest value of dry density at the lowest value of optimum moisture content. As the percent of unsuitable increases in the mixture, optimum moisture content increases from 12% to 18% and the maximum density decreases from 1,882 to 1,722 kg/m^3, the curves shift right and lower. The same trend was also found from other mixtures as shown in Figures 4(b) to 4(l). This effect is attributed to larger select soil particles that decrease the surface area and absorb less water to attain a higher density.

Observation during compaction of the mixtures revealed that the select soils improved the workability of unsuitable soils for compaction. Higher select content in the mixture was found to be easier to compact.

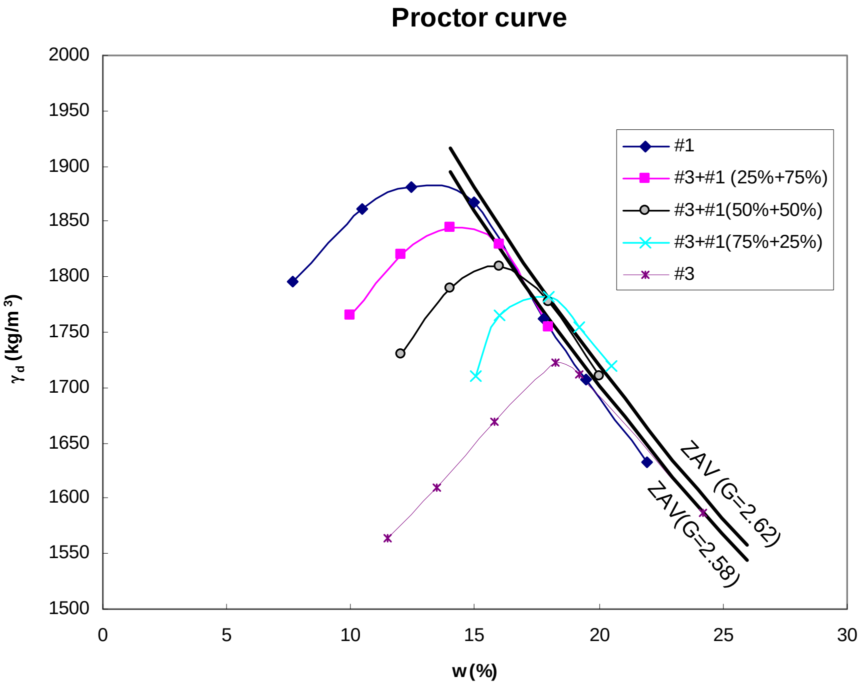


Figure 4(a). Compaction curves for mixture of soils 1 and 3

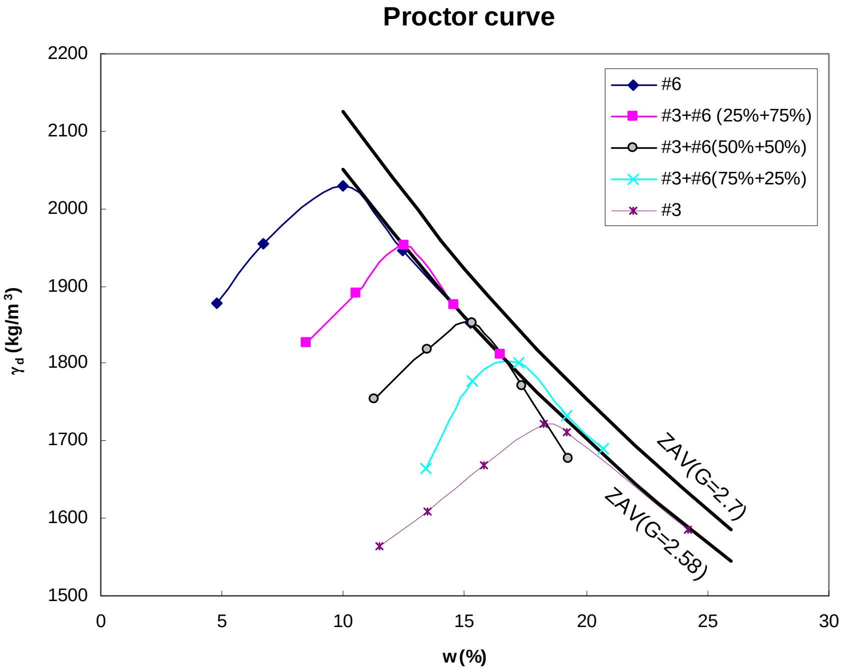


Figure 4(b). Compaction curves for mixture of soils 6 and 3

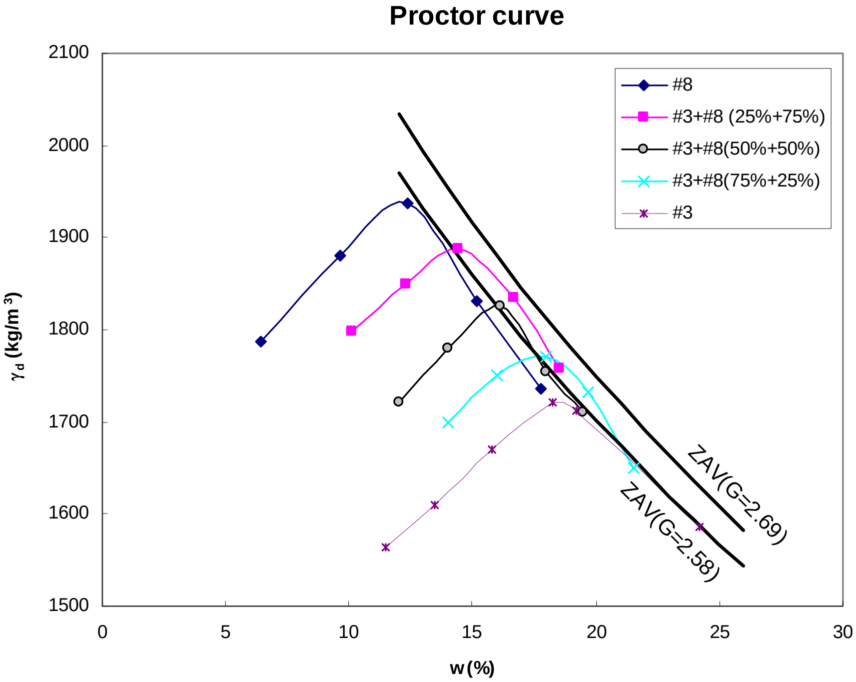


Figure 4(c). Compaction curves for mixture of soils 8 and 3

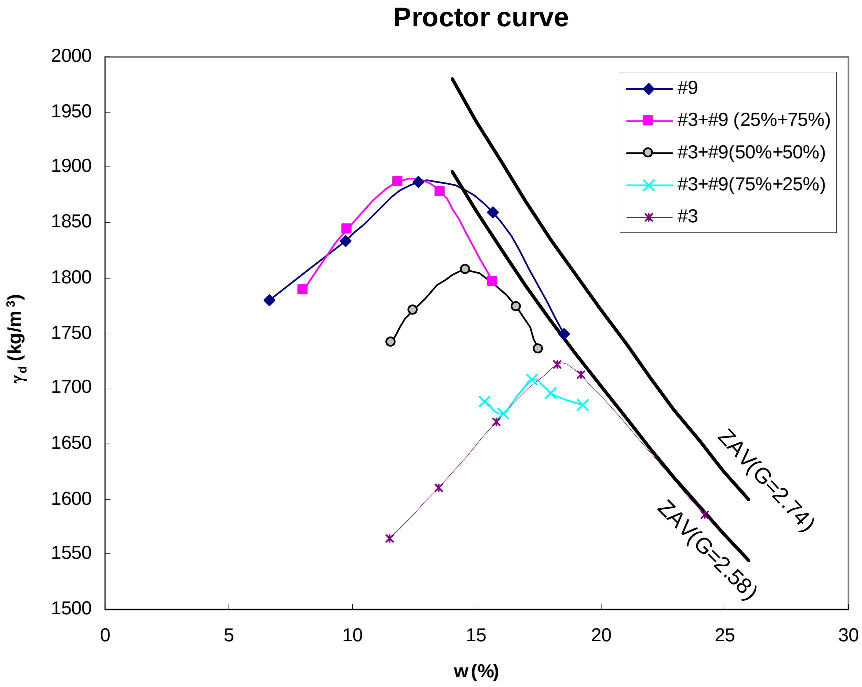


Figure 4(d). Compaction curves for mixture of soils 9 and 3

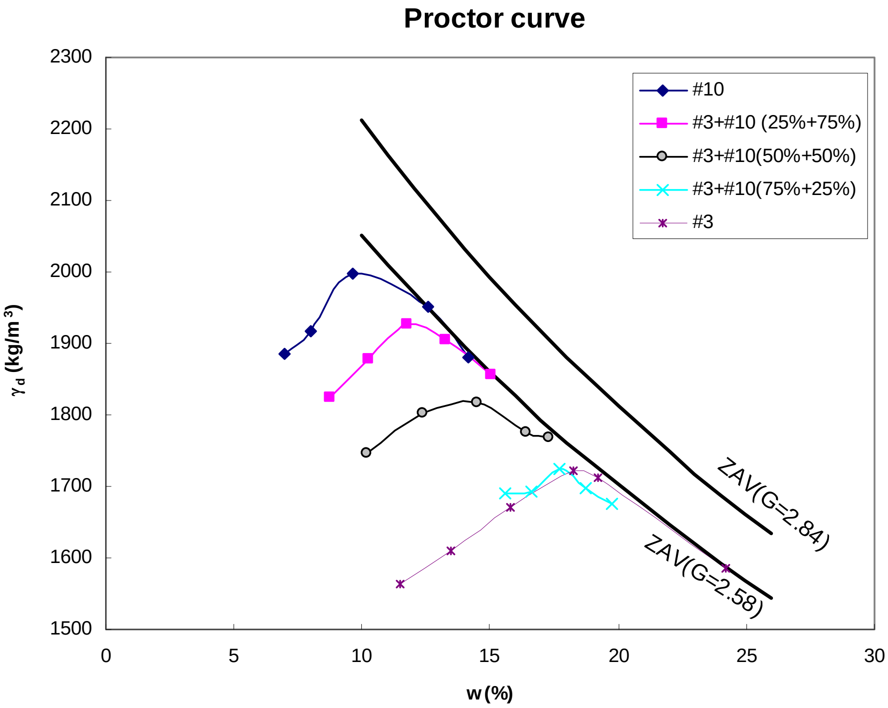


Figure 4(e). Compaction curves for mixture of soils 10 and 3

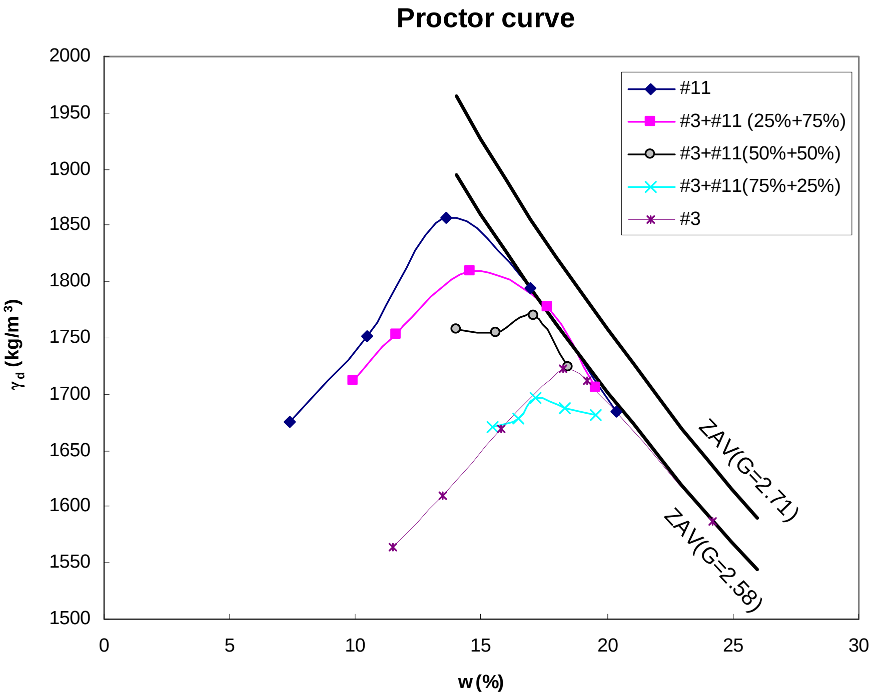


Figure 4(f). Compaction curves for mixture of soils 11 and 3

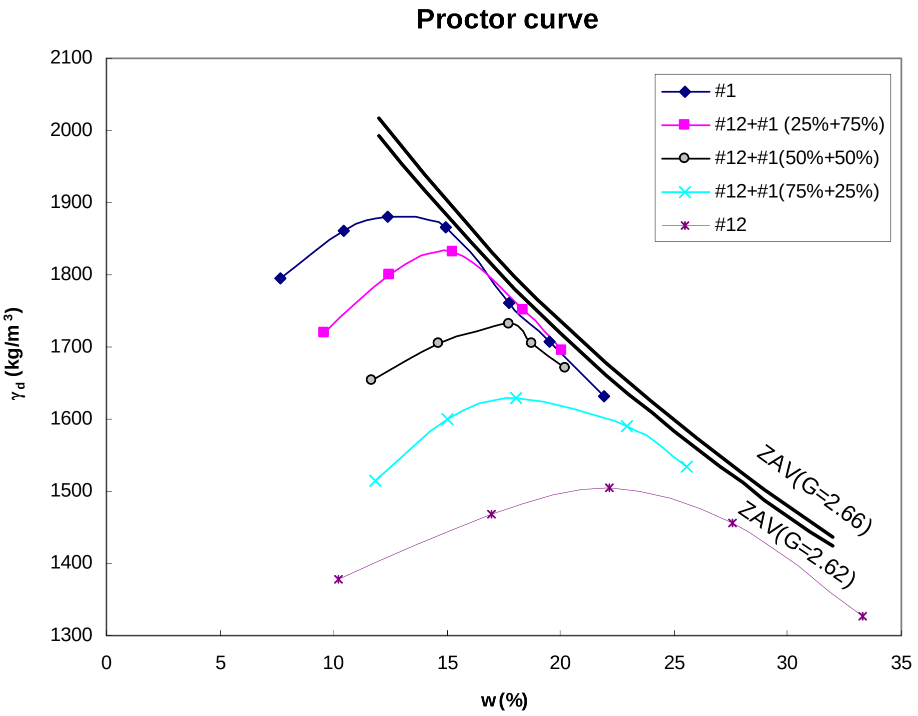


Figure 4(g). Compaction curves for mixture of soils 1 and 12

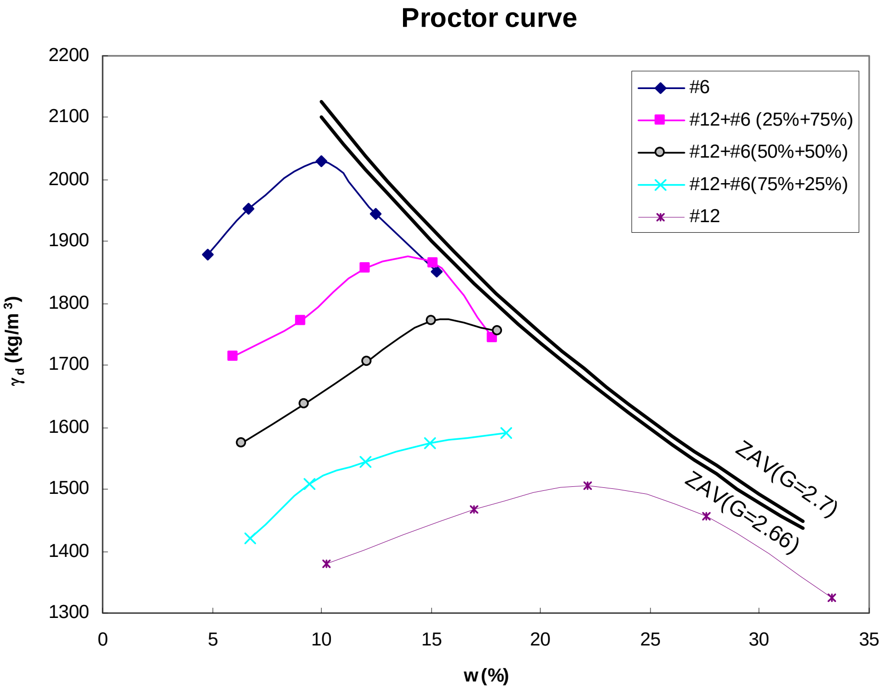


Figure 4(h). Compaction curves for mixture of soils 6 and 12

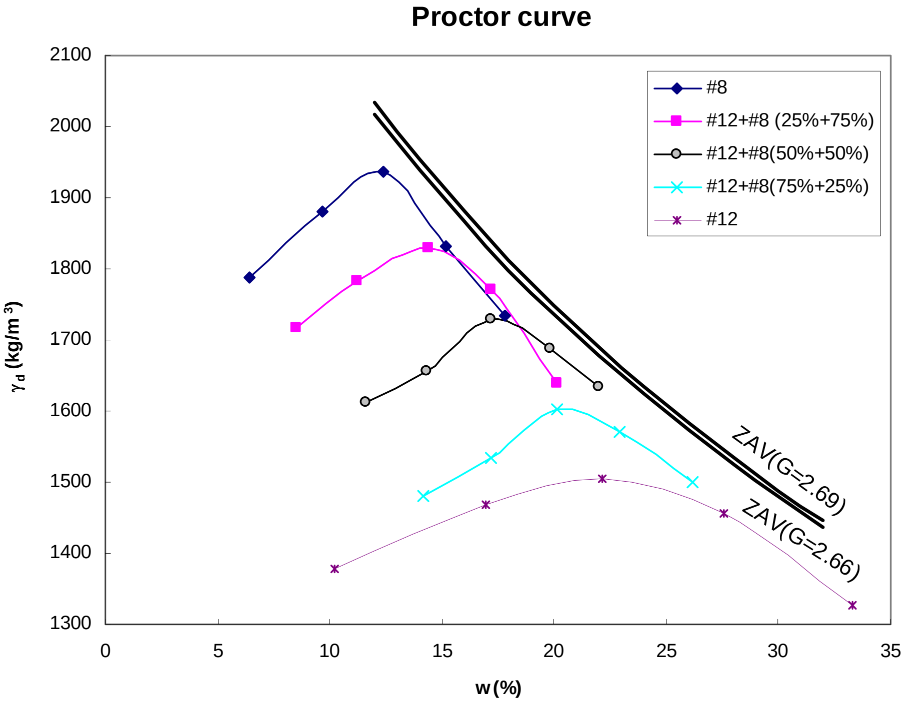


Figure 4(i). Compaction curves for mixture of soils 8 and 12

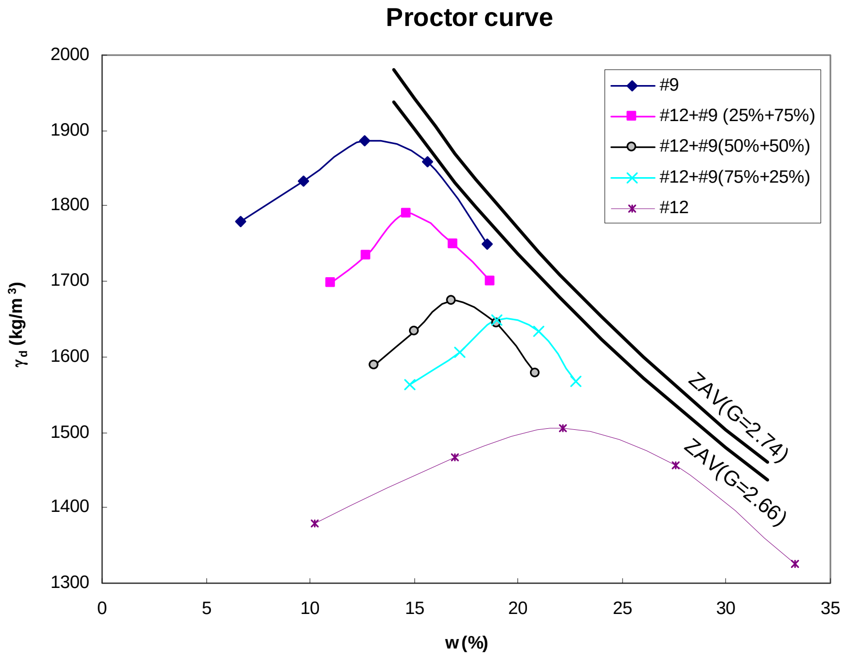


Figure 4(j). Compaction curves for mixture of soils 9 and 12

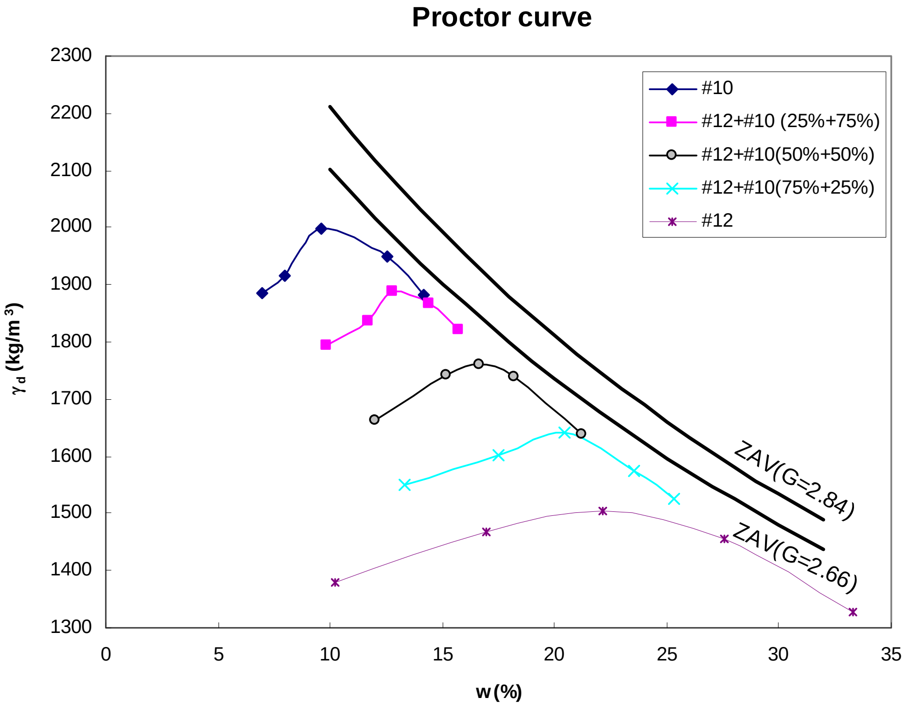


Figure 4(k). Compaction curves for mixture of soils 10 and 12

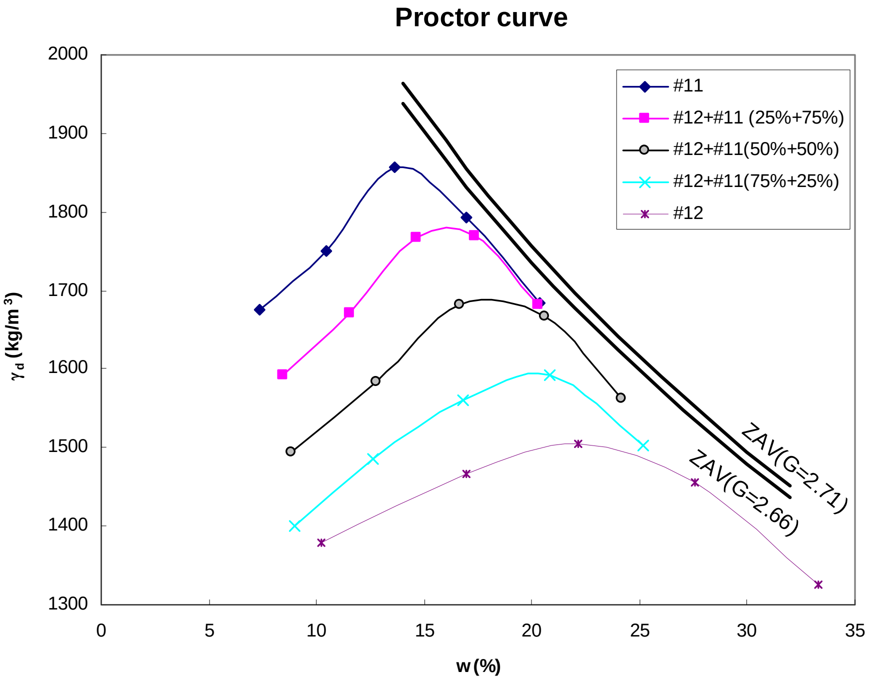


Figure 4(l). Compaction curves for mixture of soils 11 and 12

Maximum Proctor Density and Optimum Moisture Content

Figure 5 presents maximum Proctor density and optimum moisture content for various select-unsuitable mixtures. These figures show that as the proportion of unsuitable soil increases, the maximum dry density decreases and the optimum moisture content increases linearly. These results are attributed to the presence of larger select particles in the soil constituent. The larger select particles decrease the surface area of the soil and absorb less water. The select-unsuitable mixtures with higher content of the select soil exhibit higher values of the maximum dry density. This effect is attributed to the dense packing of soil particles because the workability in performing the compaction is significantly improved for the select-unsuitable mixture with higher amounts of select content.

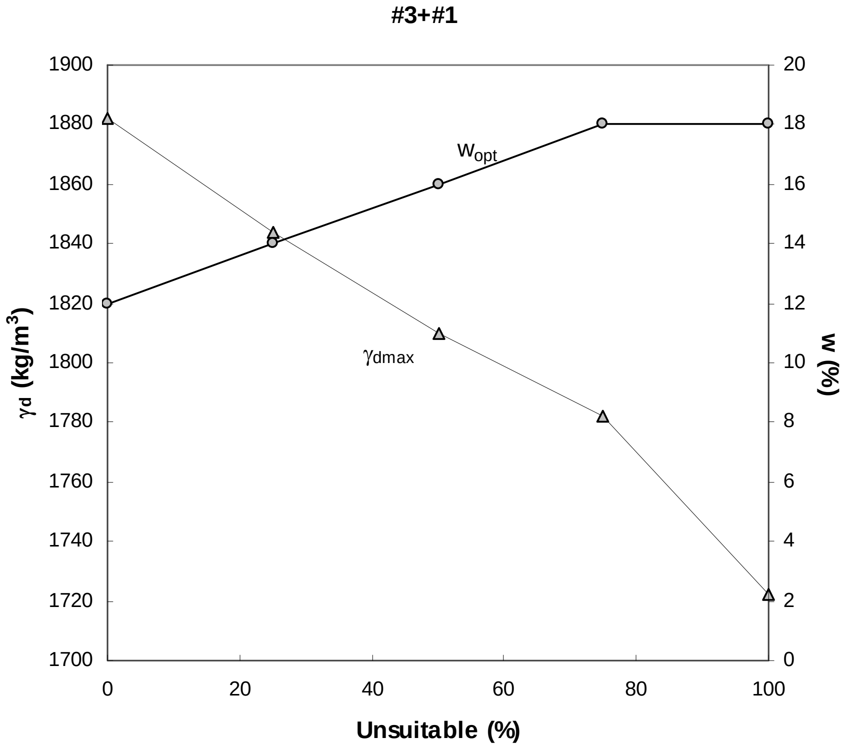


Figure 5(a). Dry density vs. moisture content for mixture of soils 1 and 3

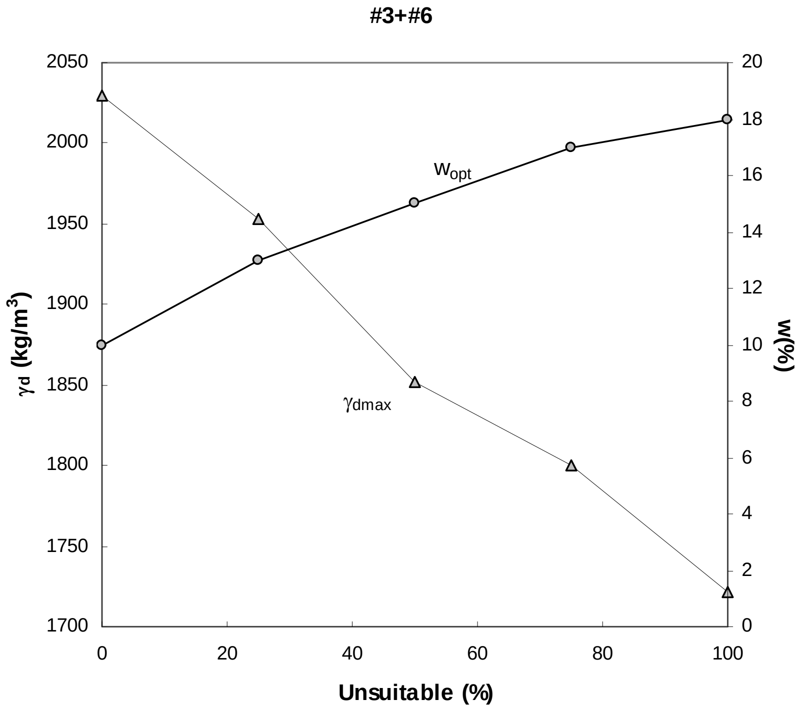


Figure 5(b). Dry density vs. moisture content for mixture of soils 6 and 3

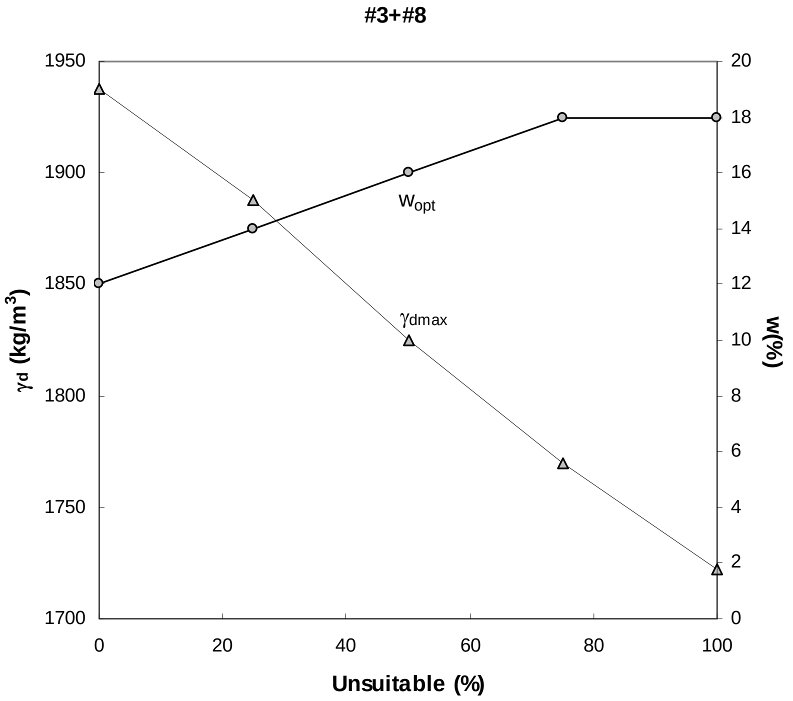


Figure 5(c). Dry density vs. moisture content for mixture of soils 8 and 3

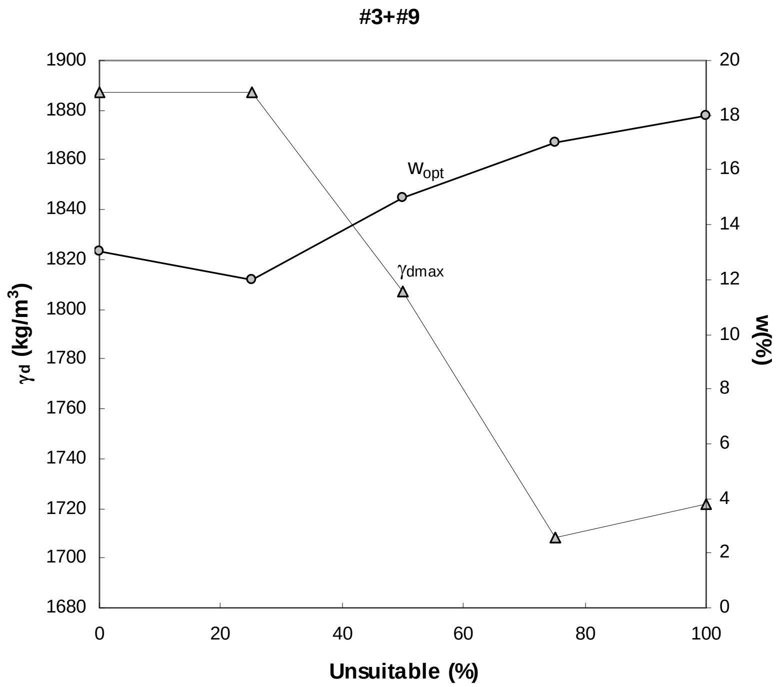


Figure 5(d). Dry density vs. moisture content for mixture of soils 9 and 3

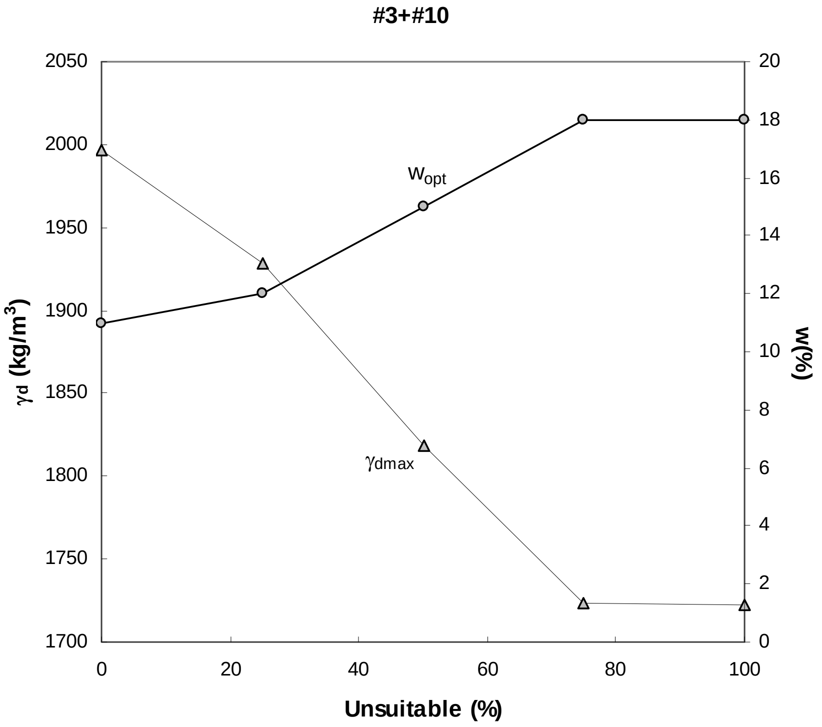


Figure 5(e). Dry density vs. moisture content for mixture of soils 10 and 3

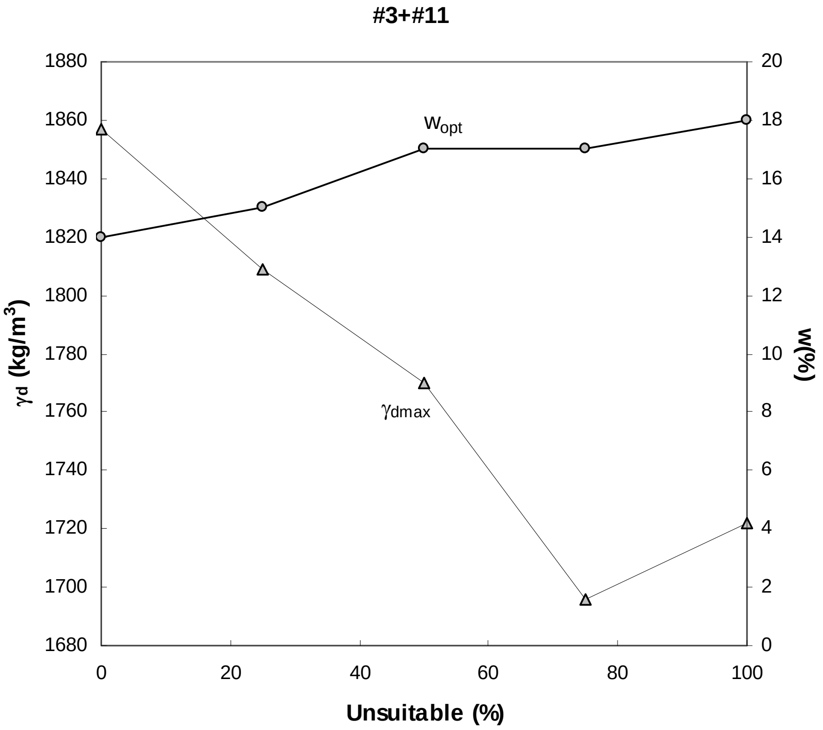


Figure 5(f). Dry density vs. moisture content for mixture of soils 11 and 3

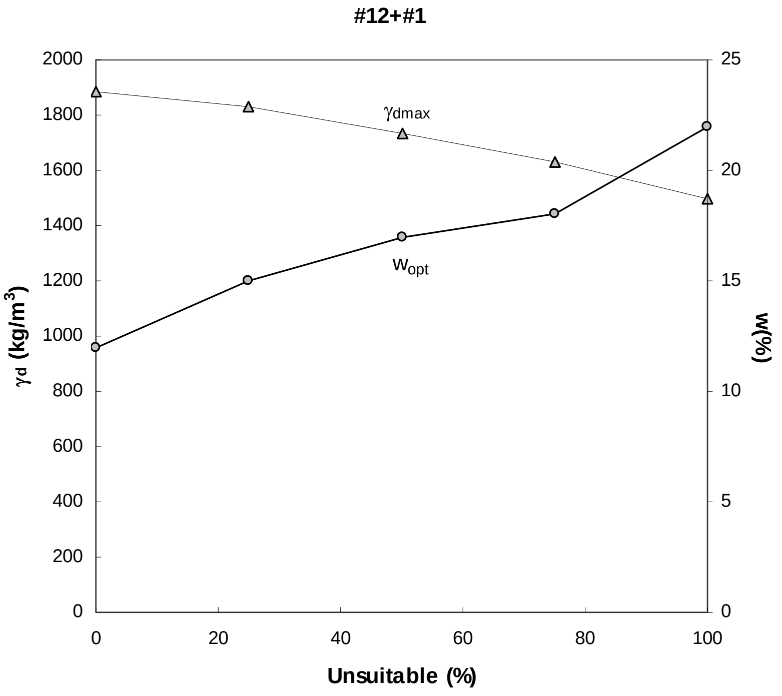


Figure 5(g). Dry density vs. moisture content for mixture of soils 1 and 12

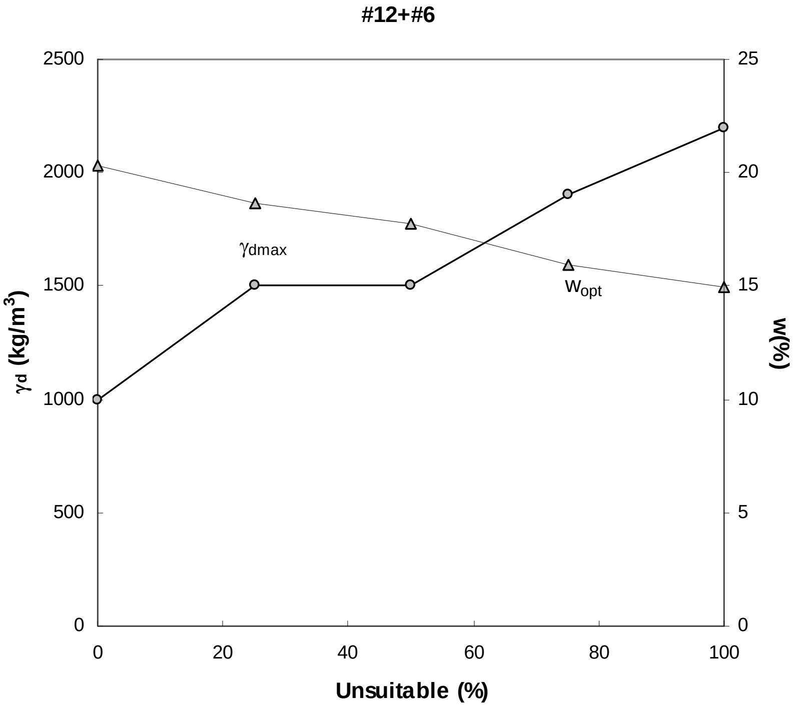


Figure 5(h). Dry density vs. moisture content for mixture of soils 6 and 12

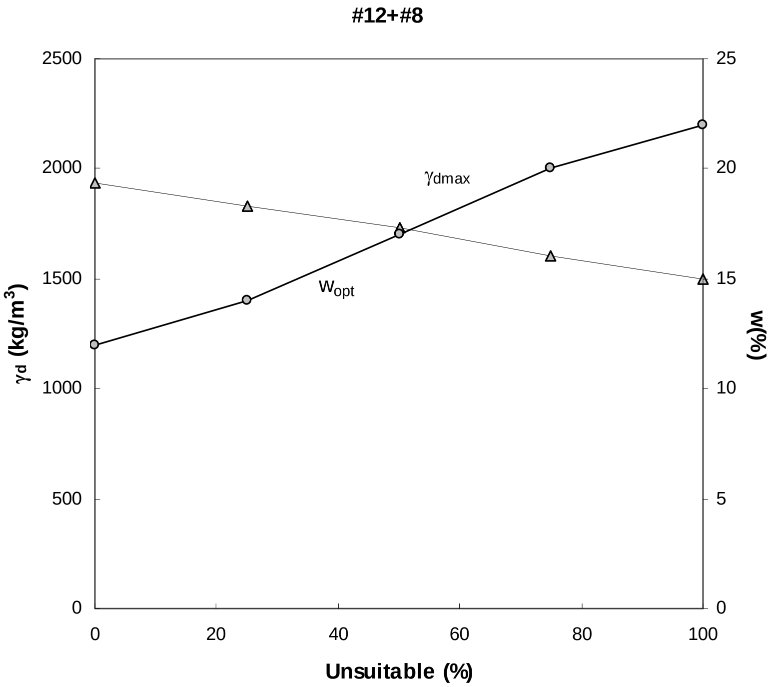


Figure 5(i). Dry density vs. moisture content for mixture of soils 8 and 12

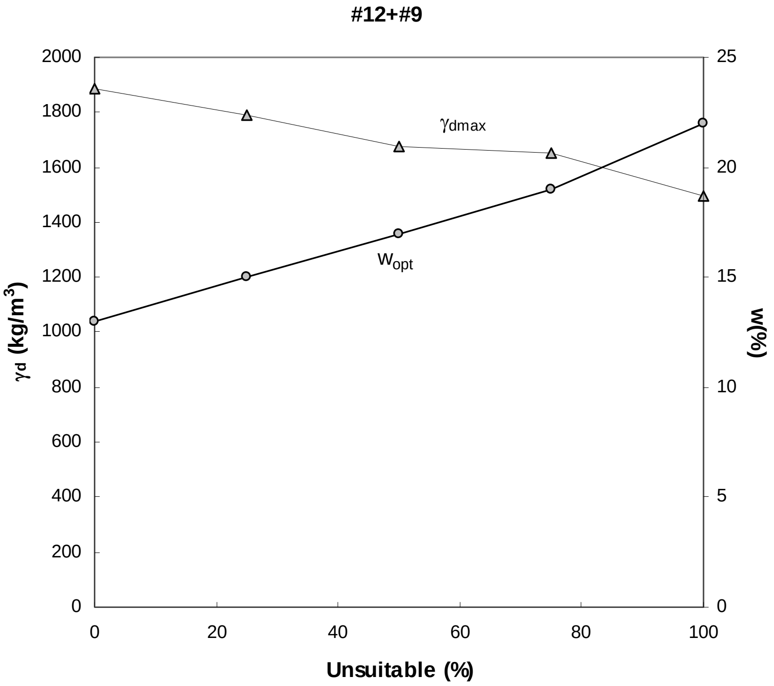


Figure 5(j). Dry density vs. moisture content for mixture of soils 9 and 12

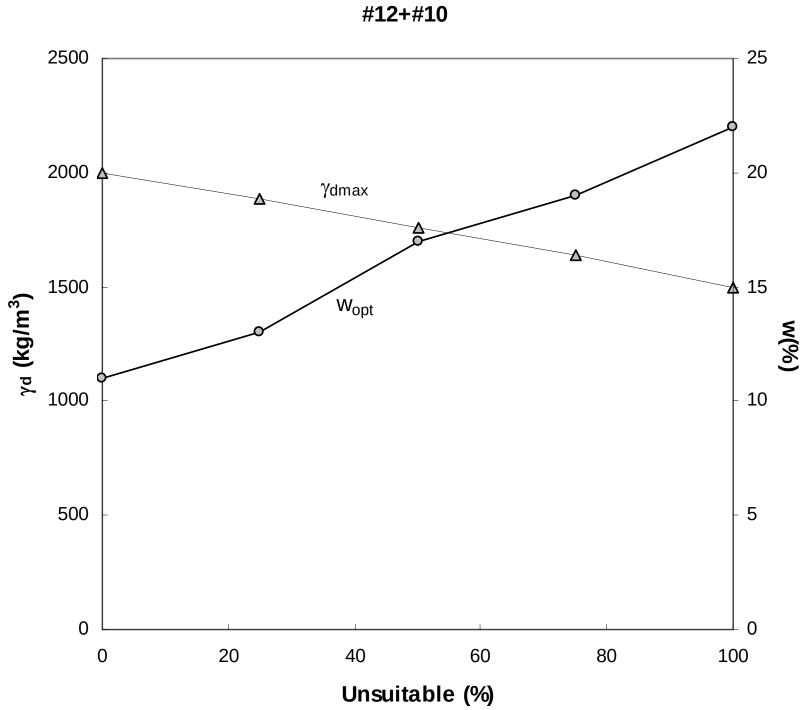


Figure 5(k). Dry density vs. moisture content for mixture of soils 10 and 12

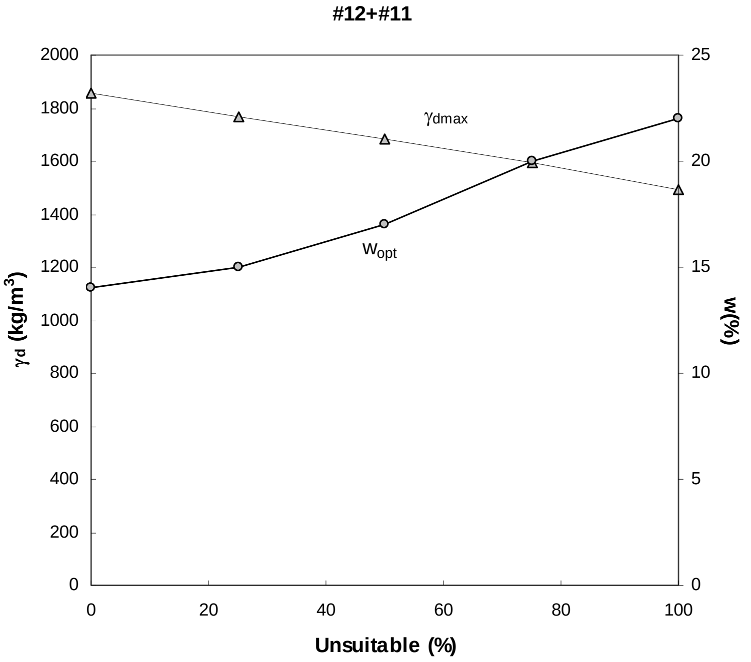


Figure 5(l). Dry density vs. moisture content for mixture of soils 11 and 12

Blotz et al. (1998) developed an empirical method to estimate maximum dry density (γd_{max}) and optimum moisture (w_{opt}) content of compacted clays from liquid limit (LL). It was found a linear relationship exists between γd_{max}, w_{opt}, and LL at any compactive effort. Figure 6 presents the relationship between optimum moisture content and maximum dry density of all mixtures. The compactive effort used for each sample was equal. A trend line is added to the data points and it shows that maximum density decreases linearly with increasing optimum moisture content, with R^2 of 0.93. The relationship between maximum Proctor density (γd_{max}) and optimum moisture content (w_{opt}) can be expressed by:

$$\gamma_{dmax}\,(\text{kg/m}^3) = -39.961\, w_{opt} + 2410.7 \qquad (1)$$

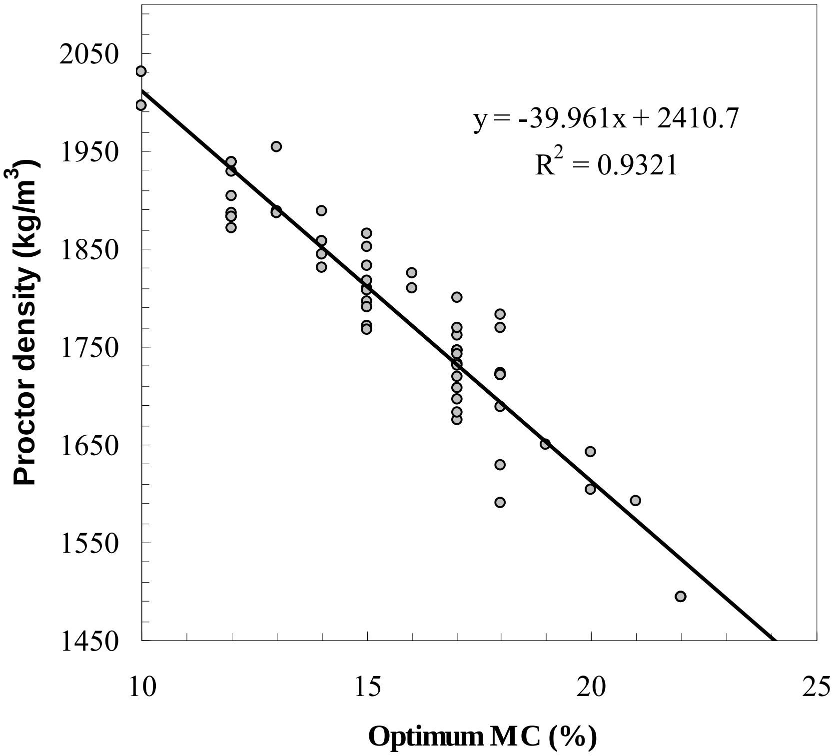


Figure 6. Relationship of proctor density and optimum moisture content

Strength of Mixtures

Figure 7 presents unconfined compression strength results at different moisture contents for each select-unsuitable mixture. When the moisture content is high, soil is very soft. From Figure 7(j), it can be seen that at moisture contents as high as 33%, soil no. 12 has an unconfined compression strength of 23 kPa. This compression strength is the lowest strength of all soils including mixtures and occurs at a compressive strain of 16%. While the maximum unconfined compression strength of these mixtures is 651 kPa, which is the strength of soil no. 9 at 10% moisture content, this soil is very strong and brittle fails at a compressive strain as low as 5%. Figure 7 shows a trend that as the moisture content increases, unconfined compression strength decreases. However, at very dry conditions (w% < 10%), the unconfined compression strength may increase with increasing moisture content. From Figure 7, it can also be seen that the maximum unconfined compression strength occurs at moisture content less than optimum moisture content of Proctor compaction. For example, Figure 7(a) shows that soil no. 1 has maximum unconfined compression strength of 531 kPa at 10% moisture content, which is less than 12% optimum moisture content.

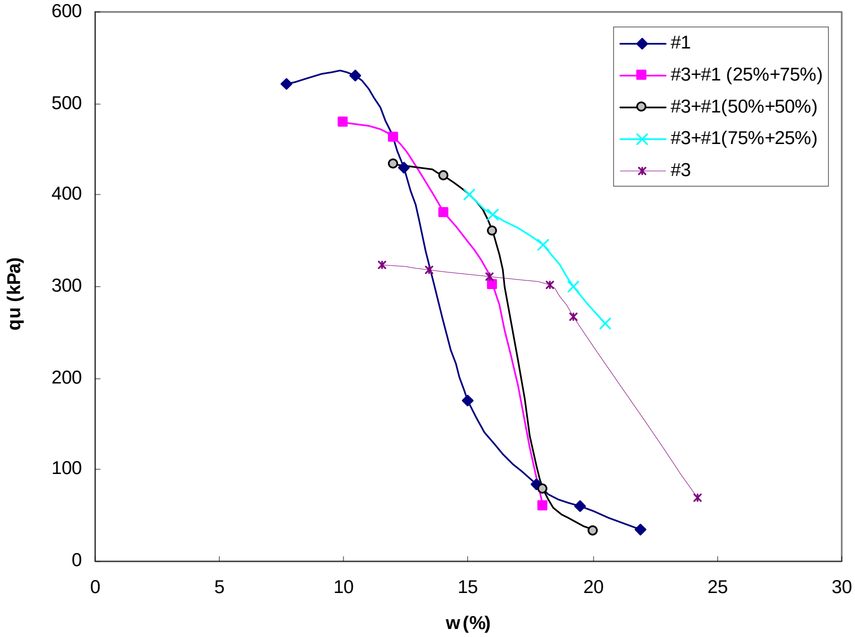


Figure 7(a). UCS vs. moisture content for mixture soils 1 and 3

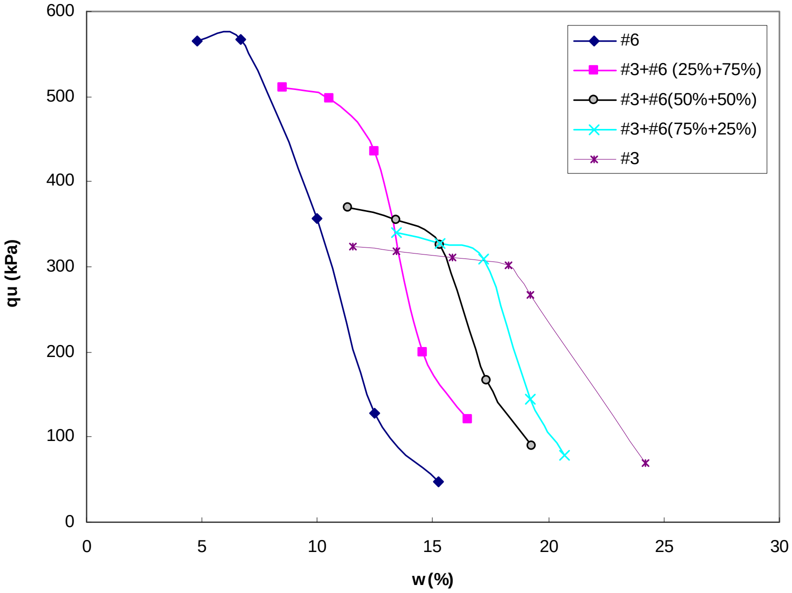


Figure 7(b). UCS vs. moisture content for mixture soils 6 and 3

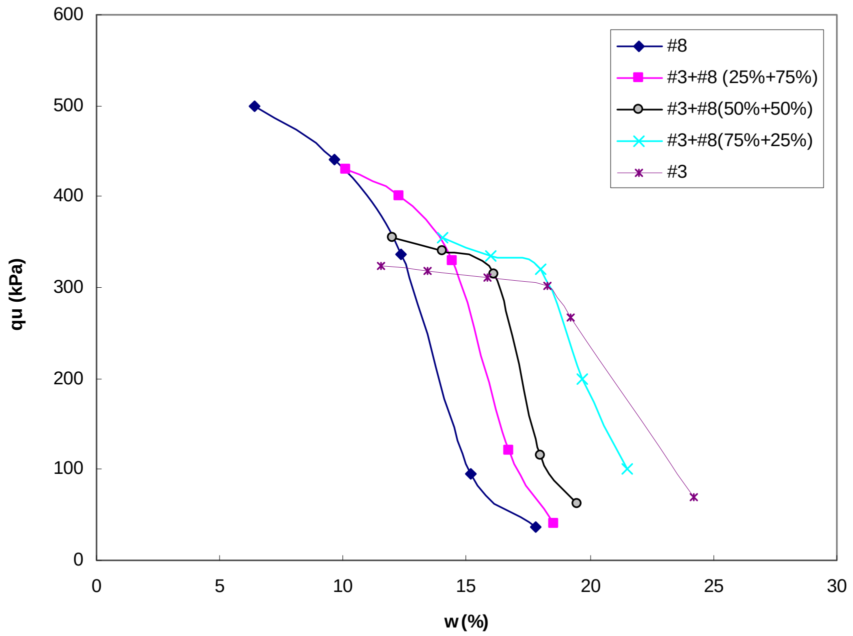


Figure 7(c). UCS vs. moisture content for mixture soils 8 and 3

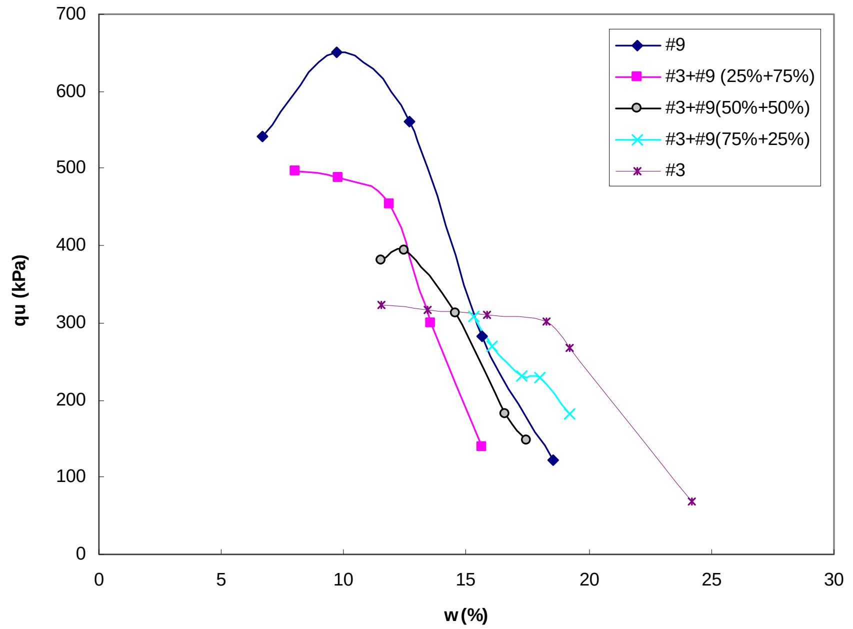


Figure 7(d). UCS vs. moisture content for mixture soils 9 and 3

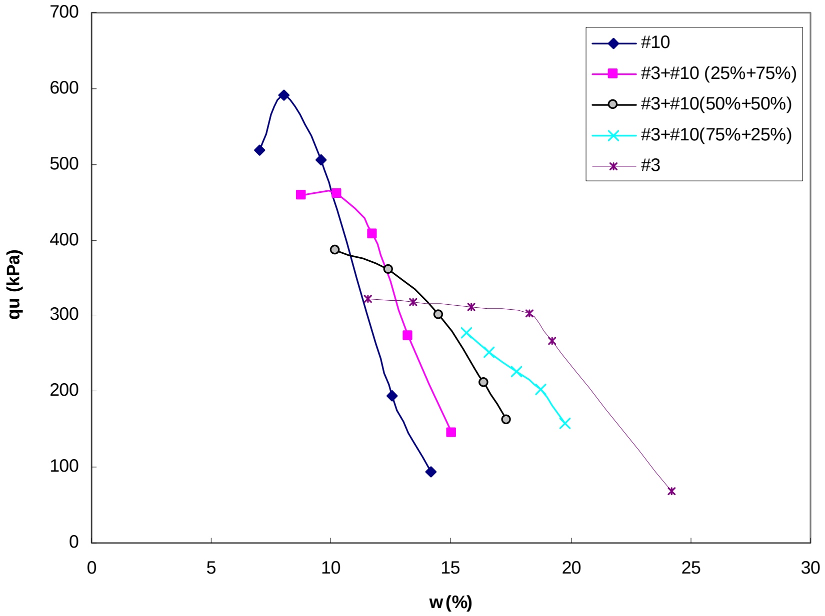


Figure 7(e). UCS vs. moisture content for mixture soils 10 and 3

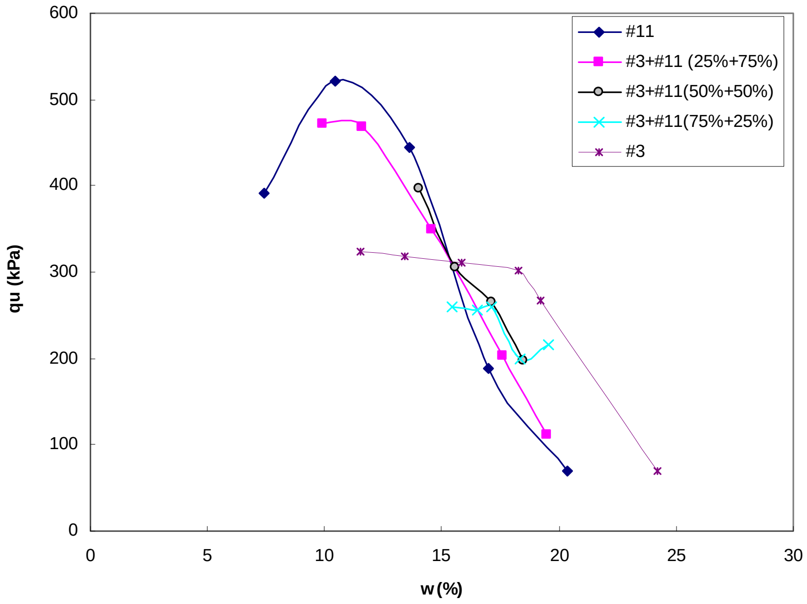


Figure 7(f). UCS vs. moisture content for mixture soils 11 and 3

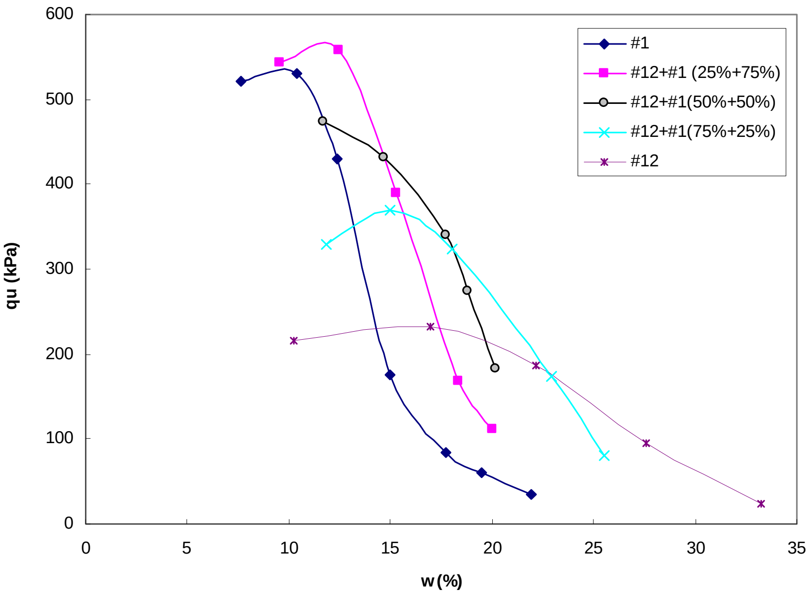


Figure 7(g). UCS vs. moisture content for mixture soils 1 and 12

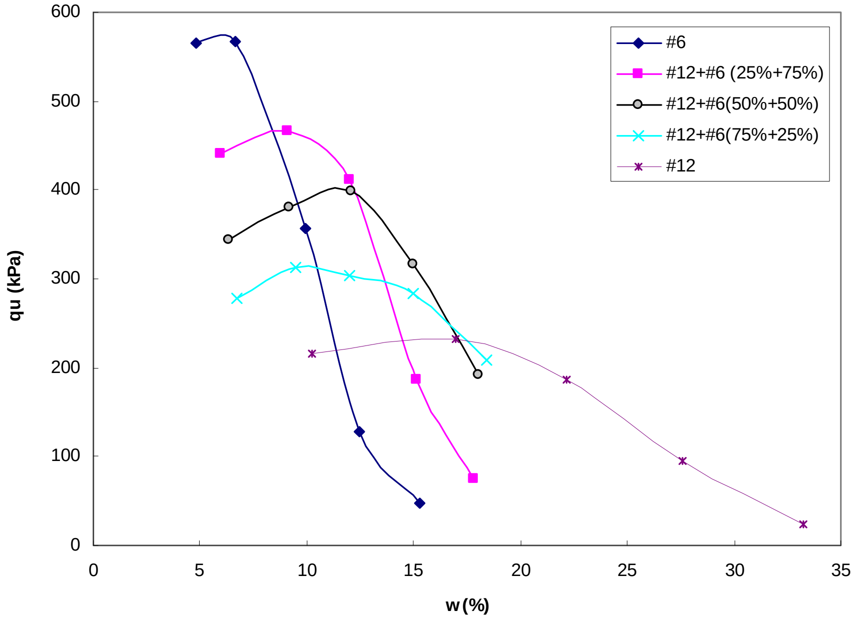


Figure 7(h). UCS vs. moisture content for mixture soils 6 and 12

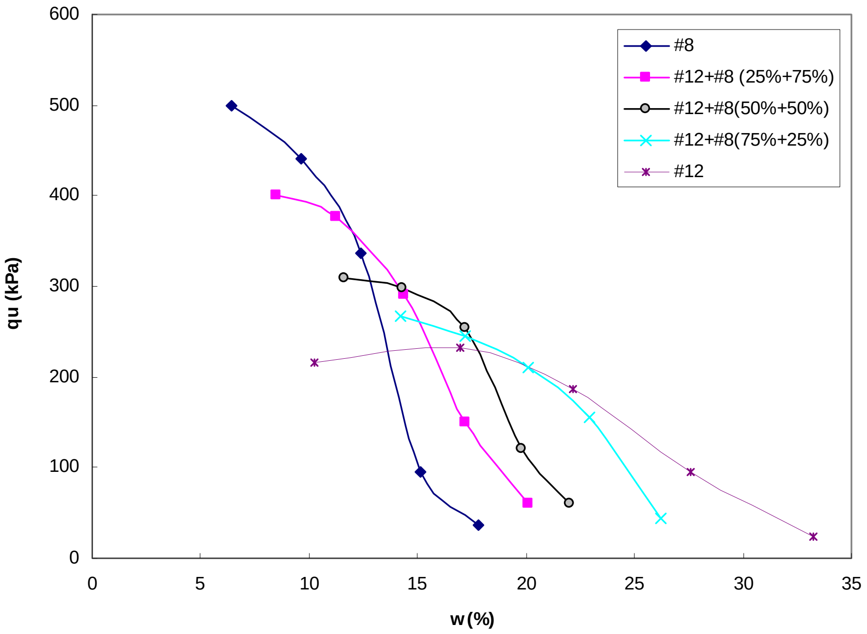


Figure 7(i). UCS vs. moisture content for mixture soils 8 and 12

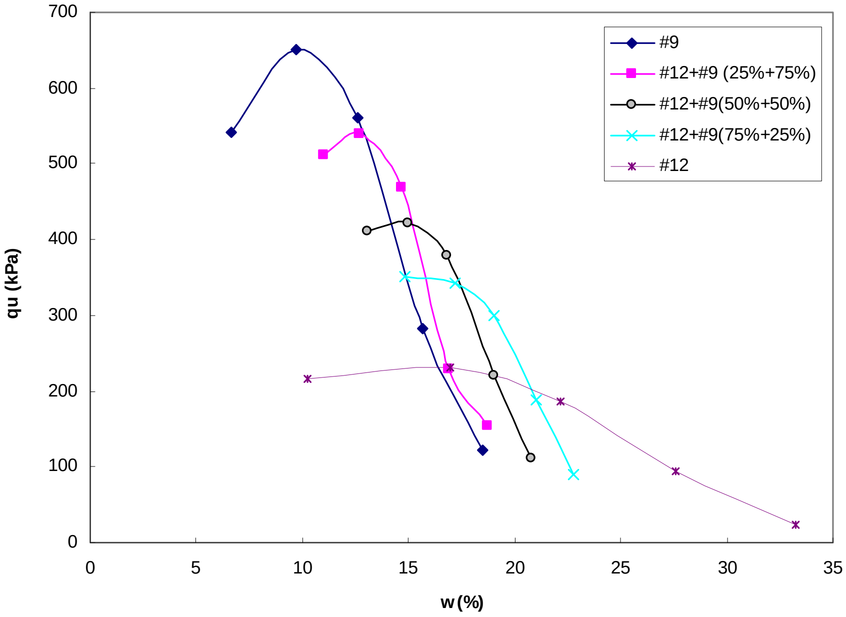


Figure 7(j). UCS vs. moisture content for mixture soils 9 and 12

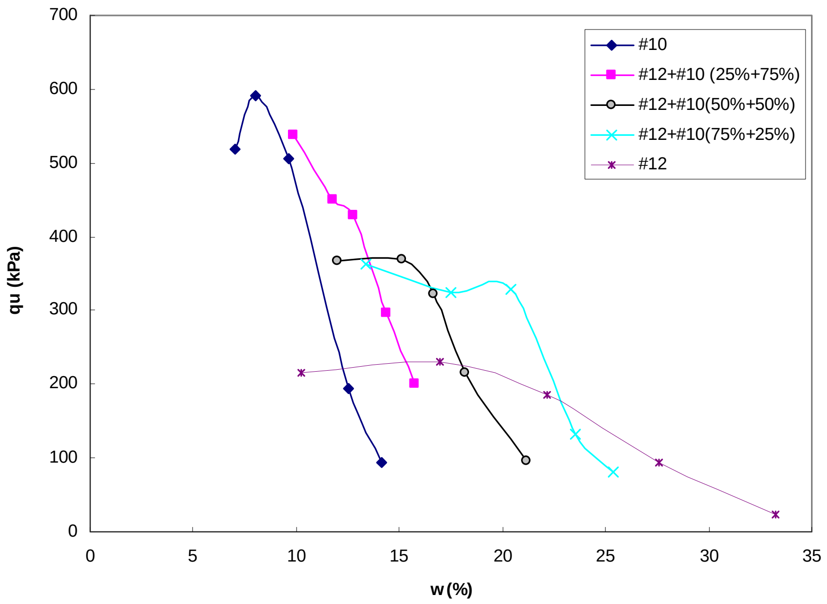


Figure 7(k). UCS vs. moisture content for mixture soils 10 and 12

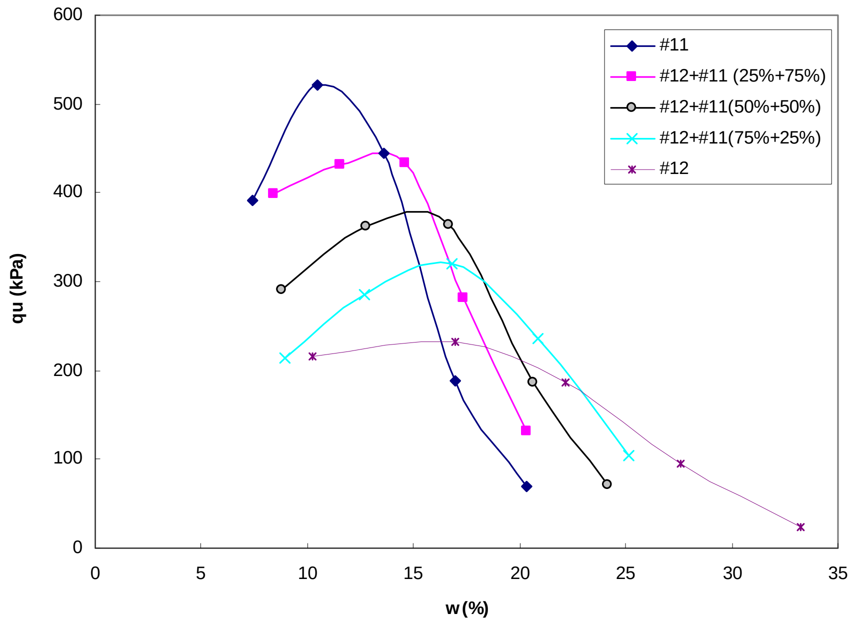


Figure 7(l). UCS vs. moisture content for mixture soils 11 and 12

Classification of Mixtures

Table 4 summarizes the index properties, compaction test results, unconfined compression test results, and classification of all mixtures. From the table, it can be seen that some of the mixtures still can be classified as select soil when the proportion of unsuitable is low (25% unsuitable). When percent unsuitable is as high as 75%, the mixture is either a suitable or an unsuitable soil. All the mixtures are poorer materials than select soil alone. However, to reduce the cost of construction, it is recommended that a mixture of select-unsuitable soils at a ratio of 3:1 be used as the top 0.6 m of subgrade.

General Mixing Conclusions and Recommendations

Improving the properties of unsuitable soils by mixing them with select soils were evaluated at various proportions of select-unsuitable soil mixtures: 0%:100%, 25%:75%, 50%:50%, and 100%:0%. Several conclusions were drawn from the results of this phase of the study.

1. The Atterberg limits variation of these soils is very small, with liquid limit and plasticity index ranging from 25 to 41 and 10 to 21, respectively. It is hard to see how the Atterberg limits change as percent unsuitable is increased. It is recommended that more unsuitable soils with high plasticity and more granular select soils be collected in the future research.
2. Experimental results indicate that maximum dry density and unconfined compression strength both increase while the optimum moisture content decreases linearly with increasing select proportion. The maximum dry density decreases linearly with increasing optimum moisture content.
3. When moisture content is greater than about 10%, unconfined compression strength decreases with increasing moisture content. When moisture content is less than 10%, however, the soil is brittle and may have a higher strength at higher moisture content.
4. The select-unsuitable mixture is a poorer material than select soil alone. However, from the test results, a 75% select and 25% unsuitable mixture can still be classified as a select soil. To reduce the cost of construction, a mixture of select-unsuitable soils at a ratio of 3:1 can still be used as the top 0.6 m of subgrade.

Table 4(a). Summary of properties of select-unsuitable (unsuitable soil no. 12) mixtures

Mixed soil	LL	PL	PI	% Gravel	% Sand	% Silt	% Clay	Proctor density	Optimum w%	GI	EPC classification	Gs	qu (kPa)	E50 (kPa)	% Unsuitable
6	26	13	13	1	46	37	17	2030	10	4	Select	2.70	356	9900	0
12&6(25%+75%)	30	14	16	1	36	50	13	1866	15	7	Select	2.69	411	13733	25
12&6(50%+50%)	33	18	15	1	25	63	11	1772	15	10	Select	2.68	316	7900	50
12&6(75%+25%)	37	20	17	0	15	75	10	1591	19	14	Suitable	2.67	209	5943	75
12	41	29	12	0	4	88	9	1495	22	14	Unsuitable	2.66	187	5813	100
1	35	18	17	11	25	45	19	1882	12	9	Select	2.62	429	9350	0
12&1(25%+75%)	37	19	18	8	20	56	16	1833	15	11	Select	2.63	390	9750	25
12&1(50%+50%)	37	23	14	6	14	67	14	1733	17	11	Suitable	2.64	341	8947	50
12&1(75%+25%)	39	24	15	3	9	77	11	1629	18	14	Suitable	2.65	325	10125	75
12	41	29	12	0	4	88	9	1495	22	14	Unsuitable	2.66	187	5813	100
8	35	19	16	3	27	56	14	1938	12	9	Select	2.69	336	10500	0
12&8(25%+75%)	37	21	16	2	21	64	13	1830	14	11	Select	2.68	290	9120	25
12&8(50%+50%)	38	24	14	2	15	72	11	1730	17	12	Suitable	2.68	254	8340	50
12&8(75%+25%)	39	25	14	1	9	80	10	1603	20	14	Unsuitable	2.67	210	7855	75
12	41	29	12	0	4	88	9	1495	22	14	Unsuitable	2.66	187	5813	100
9	34	18	16	5	29	56	10	1887	13	8	Select	2.74	561	18730	0
12&9(25%+75%)	35	19	16	4	23	64	10	1790	15	10	Select	2.72	468	16220	25
12&9(50%+50%)	38	23	15	3	16	72	9	1675	17	12	Suitable	2.70	378	13100	50
12&9(75%+25%)	39	24	15	1	10	80	9	1650	19	14	Suitable	2.68	300	8400	75
12	41	29	12	0	4	88	9	1495	22	14	Unsuitable	2.66	187	5813	100
10	25	16	10	2	44	40	14	1997	11	2	Select	2.84	507	18140	0
12&10(25%+75%)	29	16	14	2	34	52	13	1889	13	6	Suitable	2.80	429	15926	25
12&10(50%+50%)	34	21	13	1	24	64	11	1761	17	9	Unsuitable	2.75	321	8050	50
12&10(75%+25%)	37	25	13	1	14	76	10	1642	19	11	Unsuitable	2.71	329	10312	75
12	41	29	12	0	4	88	9	1495	22	14	Unsuitable	2.66	187	5813	100
11	33	18	15	3	31	45	21	1857	14	7	Select	2.71	444	13060	0
12&11(25%+75%)	34	18	16	2	24	56	18	1768	15	10	Select	2.70	434	12400	25
12&11(50%+50%)	37	21	16	2	17	67	15	1682	17	12	Suitable	2.69	365	11375	50
12&11(75%+25%)	38	24	14	1	10	77	12	1593	20	13	Suitable	2.67	236	5364	75
12	41	29	12	0	4	88	9	1495	22	14	Unsuitable	2.66	187	5813	100

Table 4(b). Summary of properties of select-unsuitable (unsuitable soil no. 3) mixtures

2	39	18	21	0	1	81	18	1746	17	21	Suitable	2.67	341	9444	0
3&2(25%+75%)	35	22	13	0	1	83	16	1742	17	14	Unsuitable	2.65	303	7550	25
3&2(50%+50%)	37	26	11	0	1	85	15	1720	17	13	Unsuitable	2.63	315	9813	50
3&2(75%+25%)	34	24	10	0	0	86	14	1689	18	12	Unsuitable	2.60	252	7412	75
3	34	22	12	0	0	88	12	1730	19	13	Unsuitable	2.58	302	7500	100
11	33	18	15	3	31	45	21	1857	14	7	Select	2.71	444	13060	0
3&11(25%+75%)	34	17	17	2	23	56	18	1809	15	11	Select	2.68	349	9511	25
3&11(50%+50%)	35	19	16	2	15	67	16	1770	17	12	Suitable	2.65	265	7389	50
3&11(75%+25%)	34	20	14	1	8	77	14	1696	17	13	Unsuitable	2.61	260	9286	75
3	34	22	12	0	0	88	12	1730	19	13	Unsuitable	2.58	302	7500	100
10	25	16	10	2	44	40	14	1997	11	2	Select	2.84	507	18140	0
3&10(25%+75%)	27	17	10	2	33	52	14	1928	12	4	Suitable	2.78	409	13600	25
3&10(50%+50%)	29	16	13	1	22	64	13	1818	15	8	Unsuitable	2.71	301	8780	50
3&10(75%+25%)	32	19	13	1	11	76	13	1724	18	11	Unsuitable	2.65	226	6011	75
3	34	22	12	0	0	88	12	1730	19	13	Unsuitable	2.58	302	7500	100
9	34	18	16	5	29	56	10	1887	13	8	Select	2.74	561	18730	0
3&9(25%+75%)	33	19	14	4	22	64	11	1887	12	9	Suitable	2.70	454	18160	25
3&9(50%+50%)	35	19	16	3	15	72	11	1807	15	12	Suitable	2.66	312	10540	50
3&9(75%+25%)	34	17	17	1	7	80	12	1708	17	15	Suitable	2.62	230	6686	75
3	34	22	12	0	0	88	12	1730	19	13	Unsuitable	2.58	302	7500	100
6	26	13	13	1	46	37	17	2030	10	4	Select	2.70	356	9900	0
3&6(25%+75%)	28	15	13	0	34	50	15	1953	13	6	Suitable	2.67	435	16440	25
3&6(50%+50%)	31	17	14	0	23	63	14	1852	15	9	Unsuitable	2.64	325	9205	50
3&6(75%+25%)	34	20	14	0	11	75	13	1800	17	12	Unsuitable	2.61	310	7890	75
3	34	22	12	0	0	88	12	1730	19	13	Unsuitable	2.58	302	7500	100
1	35	18	17	11	25	45	19	1882	12	9	Select	2.62	429	9350	0
3&1(25%+75%)	34	18	16	8	19	56	17	1844	14	10	Select	2.61	380	8890	25
3&1(50%+50%)	30	17	13	6	13	67	16	1810	16	9	Unsuitable	2.60	360	8630	50
3&1(75%+25%)	35	24	11	3	6	77	14	1782	18	11	Unsuitable	2.59	345	7980	75
3	34	22	12	0	0	88	12	1730	19	13	Unsuitable	2.58	302	7500	100
8	35	19	16	3	27	56	14	1938	12	9	Select	2.69	336	10500	0
3&8(25%+75%)	34	18	16	2	20	64	14	1888	14	11	Select	2.66	330	9730	25
3&8(50%+50%)	33	17	16	2	13	72	13	1825	16	12	Suitable	2.64	315	8810	50
3&8(75%+25%)	36	21	15	1	7	80	13	1770	18	14	Suitable	2.61	320	9430	75
3	34	22	12	0	0	88	12	1730	19	13	Unsuitable	2.58	302	7500	100

Site-Specific Example: Fairfield Bypass, Jefferson County

Presently, when soils are deemed unsuitable for a particular engineering purpose (e.g., fill, subgrade), the soils can be removed and wasted or the soils' engineering properties can be improved through mechanical compaction or chemical stabilization. When a project site has a considerable amount of unsuitable soils, it becomes costly to have to waste all the unsuitable materials. Additionally, borrowing materials from other sources may prove to be more expensive than using local available materials (Winterkorn et al. 1991). Some soils are not amenable to sufficient improvement using compaction alone. Chemical stabilization of soils is accomplished by mixing them with various chemicals, such as cement, lime, and fly ash. It has been shown that mixing soils with these chemicals can improve certain properties of soil to make the soil serve adequately an intended engineering purpose. For example, a soil-cement mixture can reduce the volume change tendency and plasticity and increase load-bearing capacity of the soil. However, high plasticity clays require larger amount of cements, which may not be economically viable. The liquid limit, plasiticity index, and percent passing No. 200 sieve generally are limited to 40%, 18%, and 35%, respectively (Winterkorn et al. 1991). In recent years, natural poor soils mixed with granular materials are also used in construction of base, subbase, and surface courses of paved facilities. Leelani (1989) studied the properties of an existing active clay by adding various proportions of a fine sand and found that a 20% sand admixture can improve the properties of the active clay adequately for highway embankment construction. Granular stabilization can obtain a well-proportioned mixture of particles with continuous gradation (well-graded) and the desired plasticity. The granular constituents form a bearing skeleton, while fine portions can provide effective cohesion and cementation. However, generally, the natural soils will lack some constituents needed to form a continuous bearing skeleton or to provide the necessary cohesion and cementation. In these cases, the desired blend can be compounded by adding proper proportions of the granular materials or fines (Winterkorn et al. 1991). One method to optimize and manage these available materials is to selectively mix the on-site soils—for example, mixing select soils with unsuitable soils. In the field, the blending of soils is accomplished by a pulvamixer or equivalent. Figure 8 shows a Multi-Terrain Mixer generally used for soil mixing (Gutzwiller 2006). The mix should be prewetted to specified moisture content and thoroughly mixed by a mixer to a homogeneous, friable mixture that is free from all clods or lumps. Compaction begins immediately after the mixing is completed. The mixtures are compacted layer by layer to the specified density until the required elevation is attained. Compaction is achieved using a vibratory padfoot roller for granular materials and sheepsfoot roller for cohesive soils.

The soils used in this study were from a proposed bypass for Highway 34 around the city of Fairfield, Jefferson County, Iowa. The proposed bypass was approximately 10 miles in length and consisted of numerous cuts and fills some 5 to 6 meters in height. The soils along the route consisted of clay pan, loess, paleosol (gumbotil), and Kansan glacial till layer (NRCS 1992). The clay pan and gumbotil have high swell potential, while the loess is potentially collapsible, resulting in unsuitable ratings for these soils. The glacial till layer was classified as a select soil. Due to the large amount of unsuitable soils along the route, the potential for soil mixing of select and unsuitable soil for improving the unsuitable soil was studied.

Figure 8. Type MGM 250 soil mixer (Gutzwiller 2006)

This section presents how the engineering properties (such as plasticity index, unconfined compressive strength, compaction characteristics, etc.) of select-unsuitable blends change with various mixing proportions. Based on the test results, recommendations of proportions of select-unsuitable blends are provided.

Site-Specific Materials

Soils were collected from the site using a drill rig down to a depth of 5.7 m. Bag samples of each soil layer were collected from the auger, as well as three-inch Shelby tube samples. Based on the texture and color of the soils, the soil profile was divided into four layers, 0 to 1.5 m (clay pan), 1.5 to 2.7 m (loess), 2.7 to 4.2 m (weathered gumbotil), and 4.2 to 5.7 m (sandy Kansan glacial till) and named soil no. 1 to no. 4, respectively.

Laboratory gradation tests, Atterberg limits, compaction, and unconfined compression strength tests were performed on the four soils in two series. In the first series, these tests are conducted to classify the four soils into select, suitable, and unsuitable. In the second series, the same tests were conducted on select-unsuitable blends at various proportions.

Gradation tests were conducted in accordance to ASTM D 2487 “Standard Practice for Classification of Soils for Engineering Purposes” (ASTM 2003). Atterberg limits tests were performed according to ASTM D 4318 “Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils” (ASTM 2003). For the mixtures, select soils passing through the No. 40 sieve were mixed with unsuitable soils passing through the No. 40 sieve at various proportions.

Compaction tests were performed on 2 x 2 in. cylindrical samples in accordance with O’Flaherty et al. (1963). A standard Proctor compaction sample is made in a 1/30 ft^3 cylinder mold using three layers that are each compacted by 25 blows of the 5.5 lb rammer dropped 12 inches for an energy input of 12,375 ft-lb/ft^3. Another test, the ISU 2 x 2 in. compaction test method, utilizes specimens 2 inches in diameter by 2 inches high that are compacted by 10 blows of the 5 lb hammer dropped 12 inches for an energy input of 13,751 ft-lb/ft^3, which requires only about one-tenth of the material and one-third of the time needed for standard Proctor specimens. However, the maximum dry density and optimum moisture content obtained from the two test methods are very close. O'Flaherty et al. (1963) have shown that the ISU 2 x 2 in. compaction apparatus can be used in lieu of the standard Proctor compaction test to increase productivity.

Unconfined compression strength tests were performed on 2 x 4 in. cylindrical samples at different moisture contents in accordance with ASTM D 2166-00 "Test Method for Unconfined Compressive Strength of Cohesive Soil" (ASTM 2003). Soil was put in two layers and the compaction energy was doubled compared to 2 x 2 in. samples.

For the compaction and unconfined compression test samples, air-dried select soils passing through a No. 4 sieve were mixed with unsuitable soils passing through a No. 4 sieve at various proportions. The required weight of the soil was determined based on the dry unit weight, water content, and the volume of the specimen. The required amount of water was added to the dry soil and thoroughly mixed. The blends were allowed to hydrate for 24 hours prior to testing.

Site-Specific Results and Discussion

In Situ and Mixed Soils

The full-depth profiles of soil index properties and classifications are summarized in Table 5. Figure 9 shows the grain size distribution of the four soils. As indicated in Table 5 and Figure 9, the fines contents decrease from 98% of the top layer (0 to 1.5 m) to 64% of the bottom layer (4.2 to 5.7 m). Soils no. 1 and 3 are fat clays and potentially expansive, and their liquid limit and plasticity index are much higher than those of the other two layers. Soils no. 1 and 3 are classified as CH (A-7-5), while soils no. 2 and 4 are classified as CL (A-6). Compaction test results indicate that soils no. 1 and 3 have relatively high optimum moisture contents and low dry densities compared to soils no. 2 and 4. Soils no. 1 and 3 are weaker soils, as indicated by high Group Index (GI) and low unconfined compressive strength. According to Iowa DOT classification, soils no. 1, 2, and 3 are classified as unsuitable soils, while no. 4 is classified as select soil. The select soil contains considerably more sand than the three unsuitable soils.

To determine the amount of mixing to be used in the field, the physical and engineering properties of different mixes were investigated. In this test program, the select soil (no. 4) was mixed with each unsuitable soil (no. 1, 2, and 3) at 75:25, 50:50, and 25:75 proportions. The engineering properties are summarized in Table 6.

Table 5. Summary of properties of select and unsuitable soils

Soil	Depth (m)	LL	PL	PI	% Gravel	% Sand	% Silt	% Clay	Proctor density (kg/m^3)	Optimum w%	Group index GI	q_u (kPa)	USCS /ASSHTO	IDOT classification
1 (clay pan)	0-1.5	53	27	26	0	2	58	40	1716	19.0	30	376	CH/A-7-6	Unsuitable
2 (loess)	1.5-2.7	30.6	18.4	12.2	0	9	64	27	1856	14.0	10	366	CL/A-6	Unsuitable
3 (gumbo)	2.7-4.2	58.5	25.9	32.6	0	11	43	46	1668	21.0	32	295	CH/A-7-6	Unsuitable
4 (glacial till)	4.2-5.7	39	17.4	21.6	0	36	34	30	1912	13.0	11	406	CL/A-6	Select

Note: q_u was measured at optimum moisture content. U: Unsuitable, S: Select.

Table 6. Summary of properties of select and unsuitable blends

Soil Blend	U:S	LL	PL	PI	% Gravel	% Sand	% Silt	% Clay	Proctor density (kg/m^3)	Optimum w%	Group index GI	q_u (kPa)	USCS /ASSHTO	IDOT classification
1 and 4	75:25	47	23	24	0	10	52	38	1765	17.5	23	371	CL/A-7-6	Suitable
	50:50	44	22	22	0	19	46	35	1816	15.7	18	381	CL/A-7-6	Suitable
	25:75	40	19	21	0	27	40	33	1861	14.1	14	393	CL/A-6	Select
2 and 4	75:25	33	19.5	13.5	0	15	57	28	1861	14.1	11	369	CL/A-6	Unsuitable
	50:50	34.5	17.1	17.4	0	22	49	29	1866	13.7	12	375	CL/A-6	Suitable
	25:75	37	18	19	0	29	42	29	1893	13.2	12	390	CL/A-6	Select
3 and 4	75:25	48	24	24	0	17	41	42	1693	19.2	21	401	CL/A-7-6	Suitable
	50:50	45.4	22.5	22.9	0	23	39	38	1747	17.3	17	359	CL/A-7-6	Suitable
	25:75	40	21	20	0	29	37	34	1819	15.5	13	400	CL/A-6	Select

Note: q_u was measured at optimum moisture content. U: Unsuitable, S: Select.

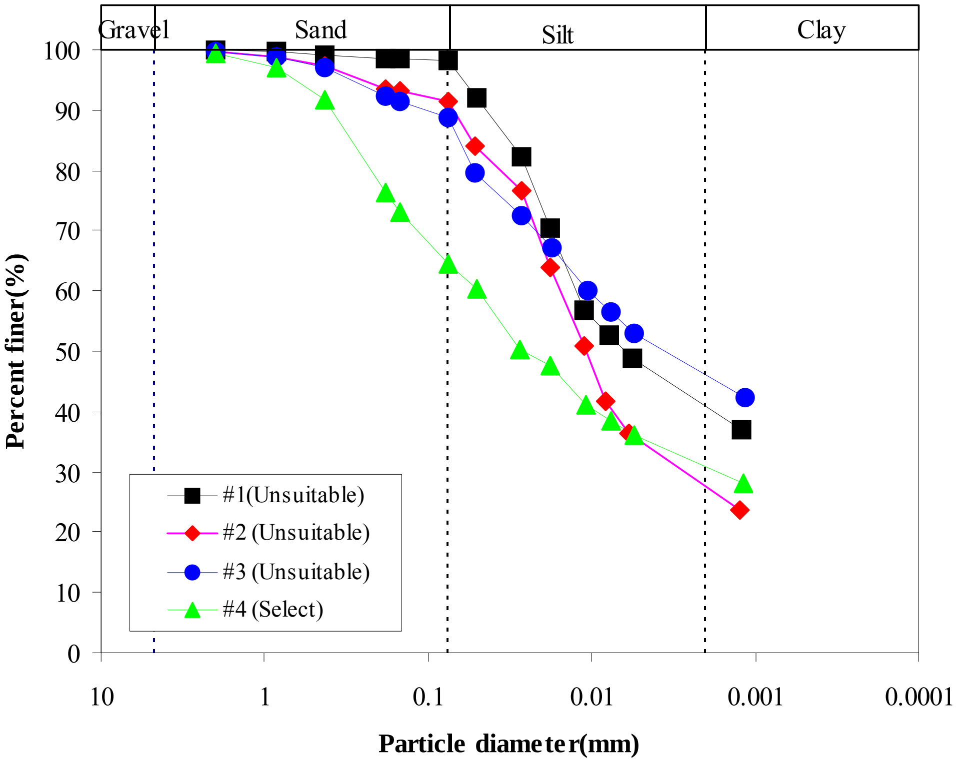


Figure 9. Grain size distribution of select and unsuitable soils

Index Properties of Mixtures

Index properties of all the mixed soils were determined at the various blends. The Casagrande plasticity chart for all the soils and blends at various proportions is shown in Figure 10. As can be seen in the figure, all the blends are located in the CL zone (low/medium plasticity to medium plasticity) and between the endpoints of individual select and unsuitable soils. As the select proportion increases, the liquid limit and plasticity index of the blends move closer to those of the select soil. Mixing with select soil reduces the plasticity and swelling potential dramatically, especially for blends of no. 4 mixed with no. 3.

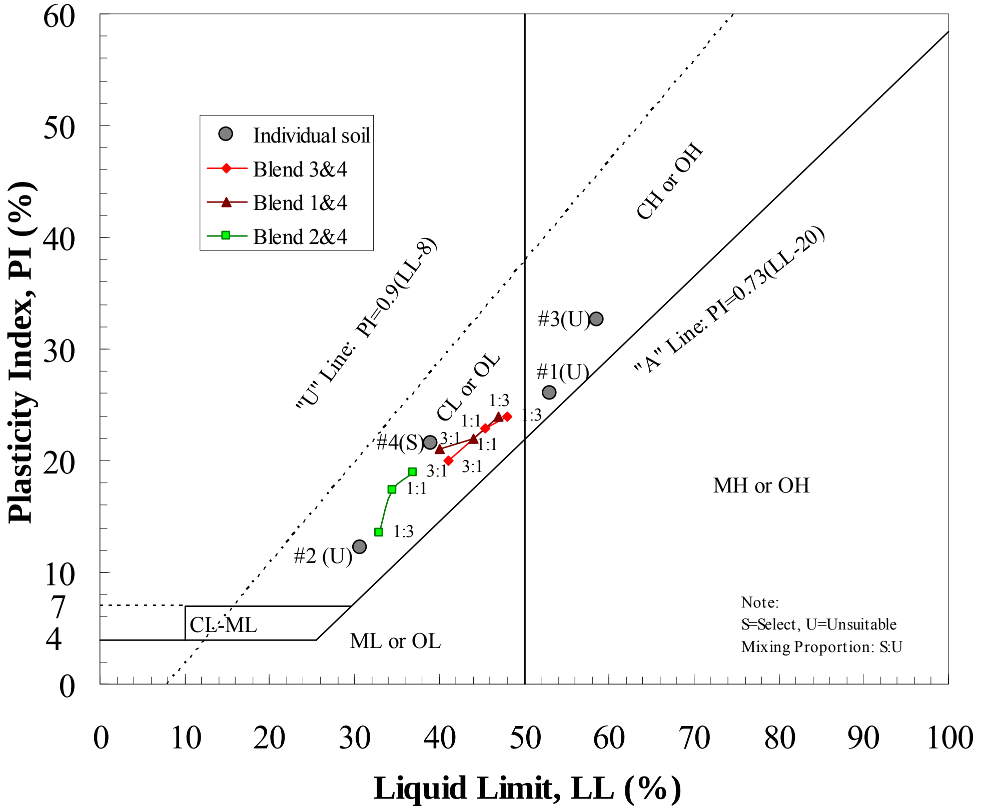


Figure 10. Casagrande plasticity chart of select, unsuitable, and blends

Compaction Characteristics

Figure 11 shows the Proctor curves of the four in situ soils. From this figure, it can be seen that soils no. 1 and 3 have much lower dry unit weights than the select soil, indicative of their higher plasticity. Soil no. 2 has a compaction curve very similar to the select soil (no. 4). Figure 12 presents the spectrum of Proctor density curves at five mix proportions. Each density curve in Figure 12 represents the variation of dry unit weight with moisture content at a specified ratio of the select-unsuitable blend. At least five samples were prepared at different moisture contents for each compaction curve. The zero-air line is also plotted in Figure 12. The uppermost curve in Figure 12(a) is the result of the compaction test from no. 4, which is a select soil (proportion of select:unsuitable = 100%:0%), while the lowermost curve represents the density curve for unsuitable soil no. 1 (proportion of select:unsuitable = 0%:100%). It indicates that select soil alone has highest value of dry density at the lowest value of moisture content. As the percent of unsuitable increases in the mixture, optimum moisture content increases from 13% to 19%, maximum density decreases from 1,912 to 1,716 kg/m^3, and the curves shift right and below. The same trend was also found from other blends, as shown in Figures 12(b) and 12(c). This effect is attributed to larger select soil particles that decrease the surface area and absorb less water to attain a higher density. The line of optimum in Figure 12 is approximately linear, which indicates a linear relationship exists between maximum dry density/optimum moisture content and mixing proportion.

Observations during the compaction tests indicate that the select soils improved the workability of unsuitable soils for compaction. Higher select content in the mixture was found to be easier to compact.

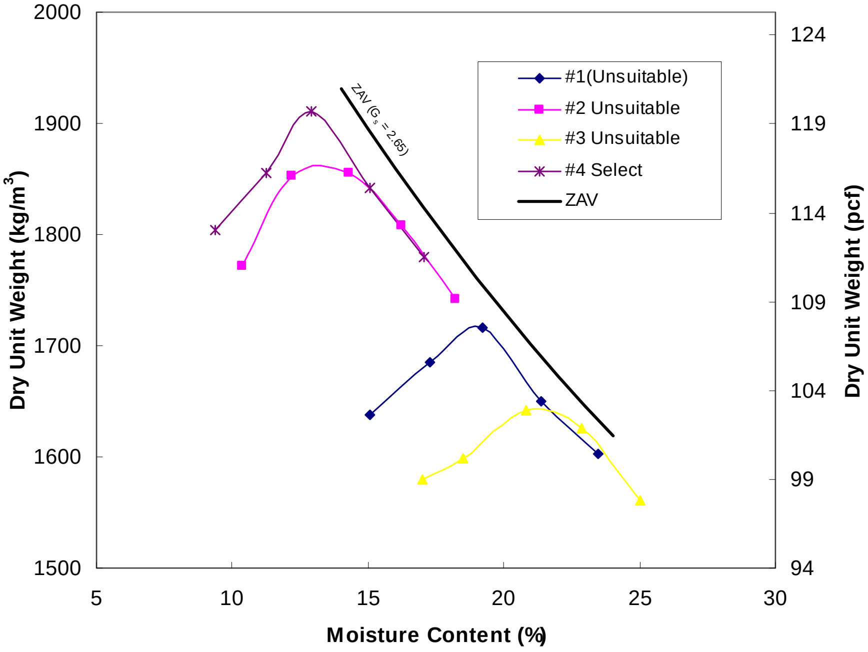


Figure 11. Compaction curves of soil no. 1, no. 2, no. 3, and no. 4

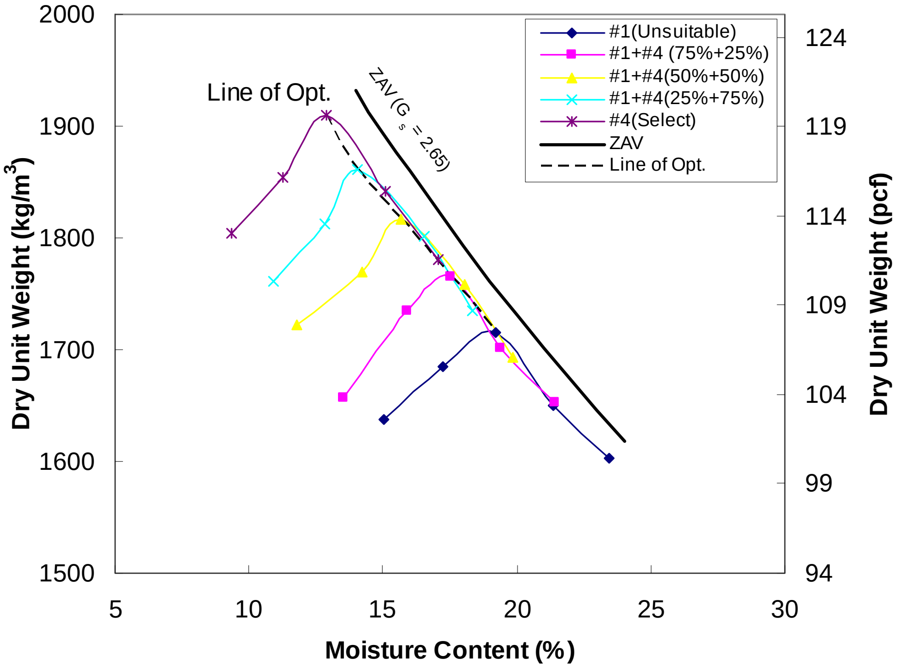


Figure 12(a). Compaction curves of no. 4 and no. 1 blends

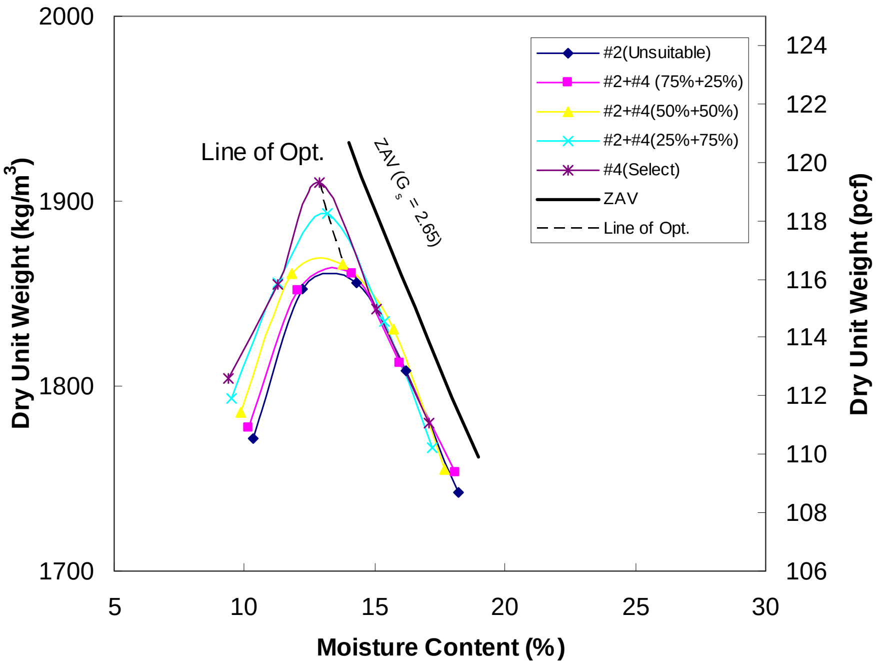


Figure 12(b). Compaction curves of no. 4 and no. 2 blends

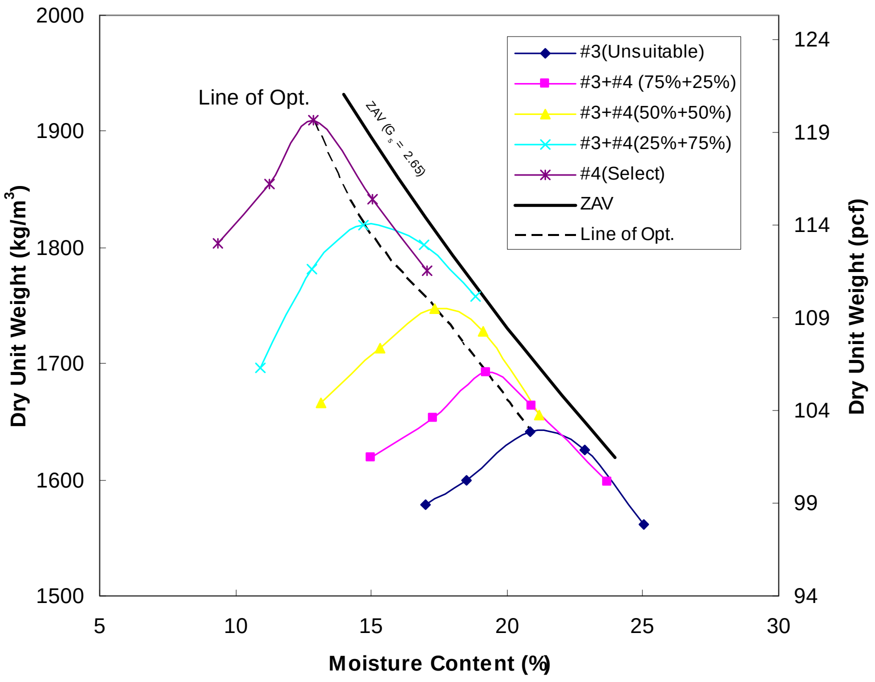


Figure 12(c). Compaction curves of no. 4 and no. 3 blends

Maximum Proctor Density and Optimum Moisture Content

Blotz et al. (1998) developed an empirical method to estimate maximum dry density (γd_{max}) and optimum moisture (w_{opt}) content of compacted clays from liquid limit (LL). It was found a linear relationship exists between γd_{max}, w_{opt}, and LL at any compactive effort. Figure 13 presents the relationship between optimum moisture content and maximum dry density of all blends. A trend line is added to the data points and shows that maximum density decreases linearly with increasing optimum moisture content, with an R^2 of 0.98. The relationship between maximum Proctor density (γd_{max}) and optimum moisture content (w_{opt}) can be expressed by:

$$\gamma_{dmax}\,(\mathrm{kg/m^3}) = -30.284\, w_{opt} + 2288.5 \qquad (2)$$

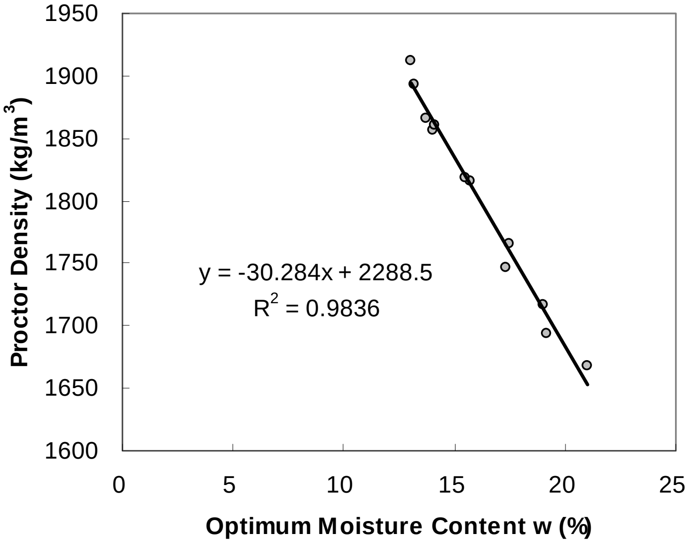


Figure 13. Maximum dry density vs. optimum moisture content

Strength of Mixture

Figure 14 presents unconfined compression strength (UCS) vs. moisture content for each select-unsuitable blend. The unconfined compression strength ranges from 100 kPa to 555 kPa and decreases with increasing moisture content even as the dry density increases. The peak strength occurs at approximately 2% to 5% dry of optimum. Meanwhile, at a specified moisture content w%, when the select proportion increases, the UCS of the blend increases. The UCS of the select soil and that of the unsuitable soil are at the endpoints of the spectrum. From Figure 14 the UCS of the blend can be determined by mixing proportion and compaction moisture content.

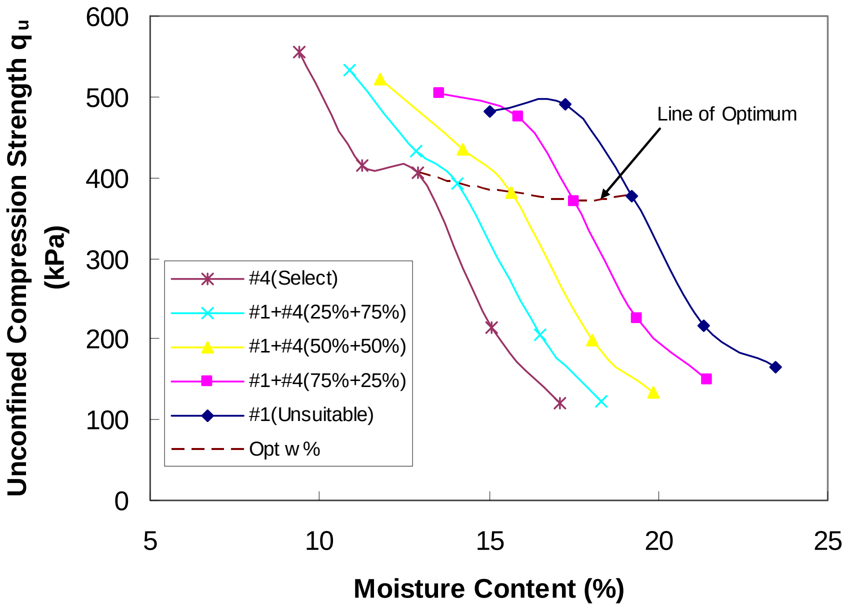


Figure 14(a). UCS of no. 4 and no. 1 blends

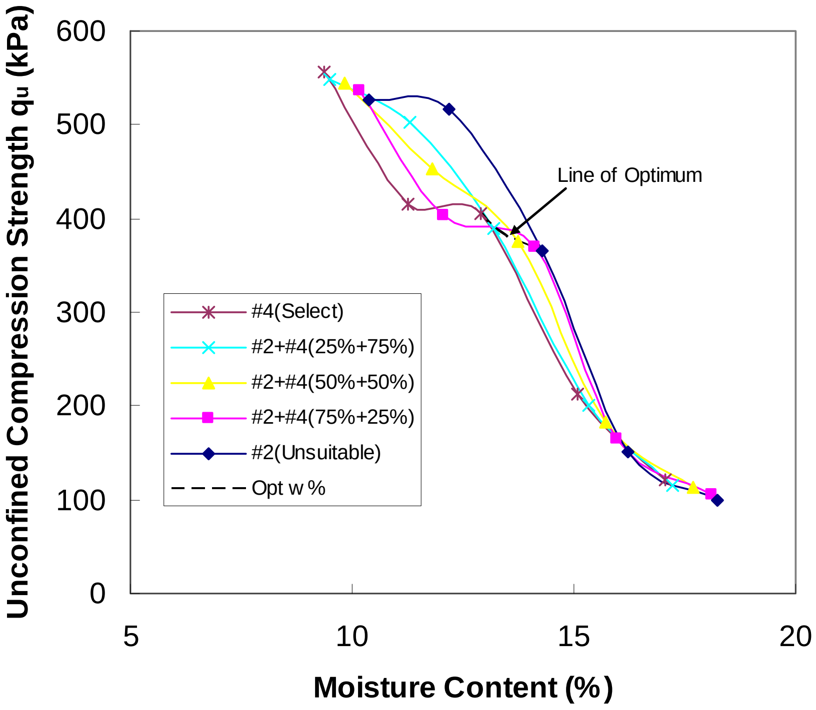


Figure 14(b). UCS of no. 4 and no. 2 blends

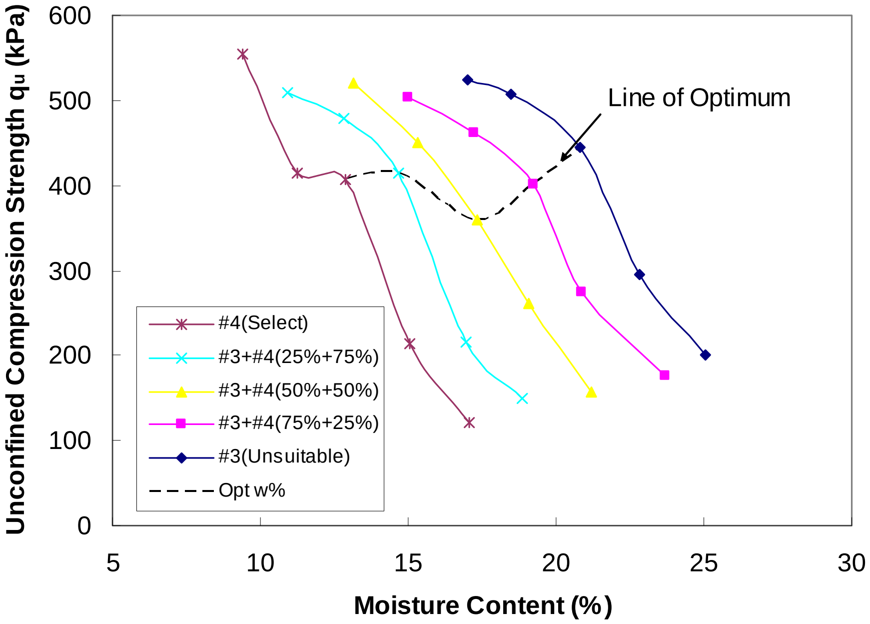


Figure 14(c). UCS of no. 4 and no. 3 blends

Classification of Blends

Table 6 summarizes the index properties, compaction test results, unconfined compression test results, and classification of all blends. From the table, it can be seen that some of the mixtures can still be classified as select soil when the proportion of unsuitable is low (25% unsuitable). Using 50:50 mixtures results in a suitable soil classification for the soils tested in this study. When the percent unsuitable is as high as 75%, the mixture is either a suitable soil or an unsuitable soil. Only in the case of soil no. 2, the loess, does the mixture result in an unsuitable classification. All the mixtures are poorer materials than select soil alone. These results provide a means of extending the use of on-site soils using soil mixing. Based on the results shown, 50:50 blends and 25:75 unsuitable to suitable soil blends provide acceptable soils for most purposes. Thus, rather than wasting site soils that are deemed unsuitable, the soils can be used in conjunction with select soils to increase the use of on-site soils and decrease the amount of off-site borrow.

Site-Specific Conclusions and Recommendations

Improving the properties of unsuitable soils by mixing them with select soils were evaluated at various proportions of select-unsuitable soil mixtures: 0%:100%, 25%:75%, 50%:50%, and 100%:0%. Several conclusions were drawn from the results of this study.

1. Mixing with select soil can improve the index properties of unsuitable soil by reducing the liquid limit and plasticity index.
2. Experimental results indicate that maximum dry density and UCS both increase while the optimum moisture content decreases linearly with increasing select proportion.
3. The UCS of the select-unsuitable blends decreases with increasing moisture content.

4. Select blends can be obtained by mixing up to 25% of an unsuitable soil with 75% select soil.
5. Suitable blends can be obtained by mixing 50% of unsuitable soil with 50% of select soil.
6. At 75% unsuitable with 25% select, some soils will be classified as suitable and some as unsuitable.
7. For the soils tested in this study, the addition of unsuitable soils to a select soil in the range of 25 to 50% provides acceptable soils for placement in embankment applications.

SATURATED PERMEABILITY OF COMPACTED GLACIAL TILL IN IOWA

Introduction

The performance of a pavement structure is dependent upon the stability and deformation resistance of the underlying subgrade. Subgrade performance in turn is a function of the permeability of the subgrade materials. Excess moisture in the subgrade soil can lead to degradation of material quality, strength reduction, deformation increase, and loss of bond between pavement layers. As a result, subgrade permeability has been incorporated into the *Mechanistic-Empirical Pavement Design Guide* through a sophisticated climatic modeling tool, the EICM (NCHRP 1-37A 2002). One of the major tasks undertaken in developing the EICM is the development of improved estimates of saturated hydraulic conductivity k_{sat} based on soil index properties, such as fine contents, P_{200}; effective diameter, D_{60}; and plasticity index, PI. This estimation is used when field and laboratory data are not available and has been shown to have a good agreement with an extended database. However, the EICM model has some limitations in that it can only predict the permeability of compacted soils at optimum moisture contents under standard Proctor compaction (Zapata and Houston 2008). To expand this empirical model for practical purposes, a more comprehensive model is presented in this paper.

The in situ soil permeability is difficult to measure accurately (Daniel 1989). Laboratory permeability tests also have limitations: (1) representative samples are not easy to obtain in the sense that samples prepared in the laboratory represent only soil matrix, primary conductivity characteristics but not structural defects such as fissures, desiccation cracks, etc. (Daniel 1989) and (2) laboratory tests often take a long time (Mitchell et al. 1965). Thus, it is of interest to predict the permeability from some simple laboratory tests for practical purposes. Since permeability is the measure of the ease with which fluid flows through porous material, certain relationships can be expected to exist between permeability and grain-size distribution, void ratio, and compaction characteristics, etc. Estimating permeability from these parameters takes less time and is less expensive than field testing. A number of factors influence the permeability of clay, such as particle size, void ratio, composition, fabric, and degree of saturation (Lambe and Whitman 1969). This number of factors makes the prediction of permeability more complicated unless some of the parameters are selected such that they are constant. In this study, the composition was held constant through the use of a set of cohesive select soils comprised of glacial till. The soils chosen were a subset of a larger number of soils studied in a soil mixing project to improve the engineering performance of soils deemed unsuitable for use in embankment construction as pavement subgrade. A number of permeability tests were performed on these soils that will be used at each layer of embankment. Permeability tests were conducted on these soils to (1) study permeability behavior of compacted cohesive select soils in Iowa, (2) provide a predictive method for permeability of the embankment soils, and (3) verify and compare this method to the EICM model.

Background

The permeability of soil is governed by many factors. For coarse-grained soils, the permeability was studied and related to grain-size distribution by Hazen (1892). Hazen was the first to correlate the permeability of coarse-grained soil to the representative grain sizes.

Other investigators (e.g., Krumbein and Monk 1942; Harleman et al. 1963; Masch and Denny 1966; and Wiebenga et al. 1970) also presented correlations between permeability and grain size of soil. Details on some of these methods and their applications and limitations have been reviewed by Egboka and Uma (1986) and Uma et al. (1989). The common aspect of these studies is the determination of an empirical statistical relationship between permeability and some index parameter, such as the geometric mean, mode, standard deviation (dispersion), or effective diameter, etc.

For fine-grained soils, the mechanism controlling permeability is more complex. The permeability is controlled by variables that may be classified as mechanical and physicochemical (Mesri and Olson 1970). The mechanical variables of main interest are the size, shape, and the geometrical arrangement of the clay particles. The physicochemical variables exert great influence on the permeability by controlling the tendency of the clay to disperse or to form aggregates. The more flocculated the soil is, the higher the permeability because there are more large channels available for flow in a flocculated soil (Lambe and Whitman 1969).

Physicochemical variables are difficult to measure. However, they can be correlated to some soil index parameters. Samarasinghe et al. (1982) studied the permeability of remoulded fine-grained soils and found that at any void ratio, the permeability of soils is best correlated with shrinkage index, irrespective of the liquid and plastic limits. According to Taylor (1948), the relationship between the hydraulic conductivity and the void ratio could be expressed as:

$$k = C\left[\frac{e^x}{1+e}\right], \tag{3}$$

where *C* can be correlated to shrinkage limit and *x* is a constant depending on soil type. For normally consolidated soils, x is about 3 to 5, while for overconsolidated and compacted soils, this value is much greater, in the range of 10 to 30 (Samarasinghe et al. 1982).

The model shown in equation (3)has been found valid for a large number of soils (Franzini 1951; Lambe and Whitman 1969; Taylor 1948). Sridharan and Nagaraj (2005) studied this model by a number of tests and found that x = 4 and C = 2.5 x 10-4 (IS)-3.69, where *IS* is shrinkage index = LL-SL.

In the *Mechanistic-Empirical Pavement Design Guide*, the saturated permeability of compacted soil at optimum moisture content is estimated based on the percent finer than the No. 200 sieve (P_{200}), D_{60}, and the plasticity index (PI) as follows:

If $0 \leq P_{200}PI < 1$, then

$$k_{sat} = 118.11 \, x10^{[-1.1275 (\log D_{60} + 2)^2 + 7.2816 (\log D_{60} + 2) - 11.2891]} \text{ (ft/hr)} \tag{4}$$

Valid for $D_{60} < 0.75$ in. If $D_{60} > 0.75$in, set $D_{60} = 0.75$ mm. If $P_{200}PI \geq 1$, then

$$k_{sat} = 118.11 \, x10^{[0.0004 (P_{200} PI)^2 - 0.0929 (P_{200} PI) - 6.56]} \text{ (ft/hr)}, \tag{5}$$

where P_{200} = percent finer than No. 200 sieve, PI = plasticity index, D_{60} = grain size corresponding to 60% passing by weight (mm), and k_{sat} = saturated permeability (ft/hr).

These empirical equations correlate the saturated permeability directly to D_{60}, P_{200}, and PI. The mechanical variables are characterized by D_{60} and P_{200}, while physicochemical variables are reflected by PI. However, these equations are only applicable to soils under conditions of standard Proctor compaction at optimum moisture content.

As is well known, the permeability of compacted clay is also influenced by compaction method and molding moisture content. To take the compaction effort into account, Lambe (1958) investigated the relationship between the soil permeability and compaction effort characteristics on Jamaica sandy clay and Siburua clay, with the experimental results shown in Figure 15. It was found that when soil compacted dry of optimum, the permeability decreased with increasing molding moisture content. The permeability was about one to two orders of magnitude lower when 2% more water was added to the soil. The lowest permeability occurred when the soil was compacted at about 2% wet of optimum. Then, the permeability increased slightly with increasing moisture content.

Harrop-Williams (1985) derived an empirical equation to predict the permeability of soil from compaction variables and found the relationship presented in the form

$$\ln k = A + \alpha x, \tag{6}$$

where k = permeability of compacted soil and A, α = regression constants and

$$x = \ln\left(\frac{\gamma_d w G}{G\gamma_w - \gamma_d}\right), \tag{7}$$

where γ_d = dry unit weight of compacted soil, γ_w = unit weight of water, and G = specific gravity of soil.

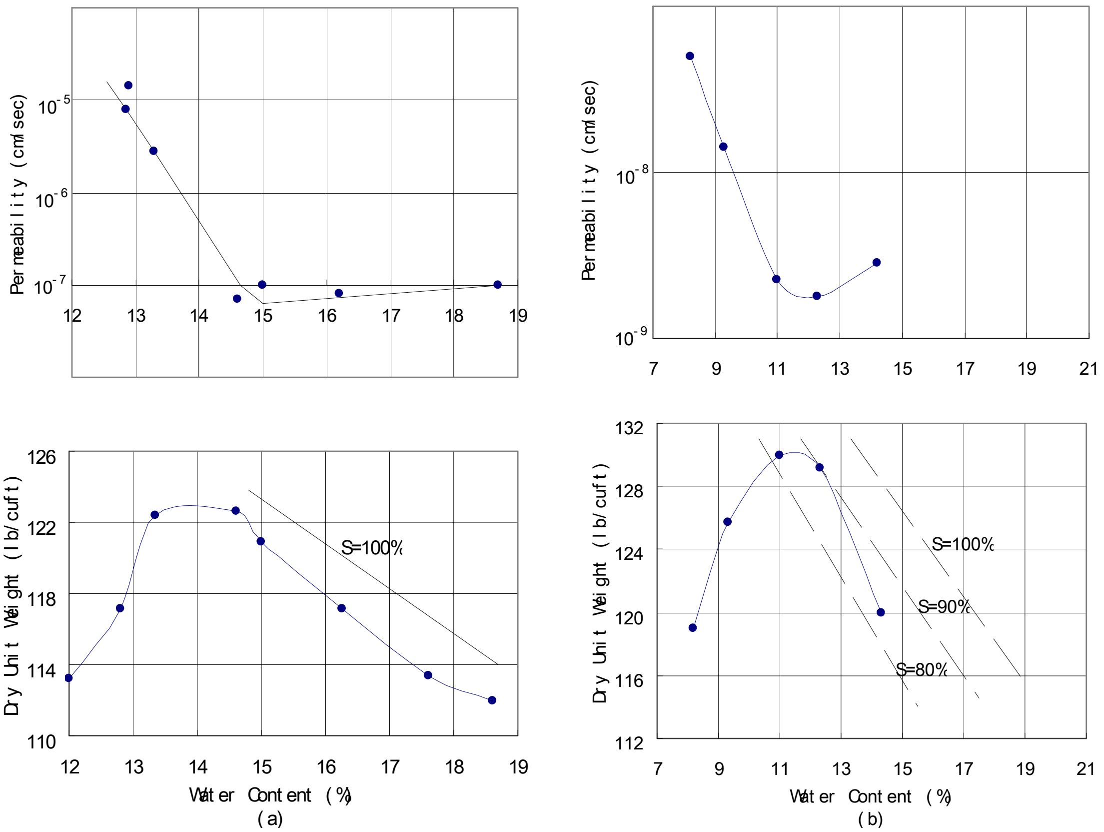


Figure 15. Permeability and water content, compaction characteristics relationship of compacted clays after Lambe (1958) (a) Jamaica Sandy clay and (b) Siburua clay

Equations (6) and (7) incorporate molding moisture content and dry density of compacted clay to predict permeability. Harrop-Williams (1985) stated that the regression constants A and α are related to soil type, method, and level of compaction. The typical value of A and α are -20 and -5, respectively.

Benson and Trast (1995) studied 13 compacted soils and found that the most significant factors affecting hydraulic conductivity are (in decreasing order of importance) (1) initial saturation, (2) compactive effort, (3) plasticity index, and (4) clay content. They developed a regression equation to estimate hydraulic conductivity, as shown in equation (8):

$$\ln K = -15 - 0.087 S_i - 0.054 PI + 0.022 C + 0.91 E + \varepsilon\,, \tag{8}$$

where K is hydraulic conductivity in m/s; C is clay content in percent; PI is plasticity index; E is the compactive effort index assigned as -1, 0, and 1 for modified, standard, and reduced Proctor compactive efforts; S_i is initial degree of saturation; and ε is a random error term.

Materials and Methods

Five cohesive select soils of glacial origin from Iowa were used in the permeability tests. These soils were selected from a group of 13 soils that were extensively studied for soil mixing. The soil information and index properties are listed in Table 7.

Gradation tests were conducted in accordance to ASTM D 2487 “Standard Practice for Classification of Soils for Engineering Purposes” (ASTM 2000a). Atterberg limits tests were performed according to ASTM D 4318 “Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils” (ASTM 2000b). The standard Proctor compaction tests were performed according to ASTM 698-78 “Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5-lb (2.49-kg) Rammer and 12-in. (305-mm) Drop” (ASTM 2000c). Prior to compaction, soils were air-dried and pulverized to sizes smaller than the No. 4 US sieve (4.75 mm). Specimens were compacted at five to nine different molding moisture contents, from dry to wet of optimum moisture content.

Hydraulic Conductivity Tests

Permeability tests were performed using a Tri-Flex 2 Permeability Test System. It is designed to have modular expansion capability, with the Master Control Panel capable of testing one sample while acting as a controller for up to two Tri-Flex 2 Auxiliary Control Panels. In this research, only one Auxiliary Control Panel was used. Three samples were tested simultaneously, each with its own pressure settings. For detailed information, refer to the Tri-Flex 2 Permeability Test System Owner’s Manual provided by ELE (Tri-Flex2 Permeability Test System 2003).

Table 7. Index properties of select soil used in permeability test

Soil No.	Gs	LL	PL	PI	% Gravel	% Sand	% Silt	% Clay	Proctor density (kg/m^3)	Optimum w%	Degree of Saturation at Optimum w%	Classification USCS, AASHTO	Source
A	2.7	26	13	13	1	46	37	17	1951	11	57	CL, A6	HW20, Webster County
B	2.69	35	19	16	3	27	56	14	1855	14	20	CL, A6	HW30, Le Grande
C	2.74	34	18	16	5	29	56	10	1893	13	22	CL, A6	HW30, Le Grande
D	2.84	25	15	10	2	44	40	14	1969	11.6	35	CL, A6	HW30, Le Grande
E	2.71	33	18	15	3	33	45	21	1816	16	45	CL, A6	HW60, Hopster

The hydraulic conductivity tests on remolded samples were tested in the triaxial cell under effective stresses and back pressures equivalent to the in situ condition according to ASTM Designation D 5084 "Standard Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter" (ASTM 2000d). Constant head tests were selected because it is easy to measure both the inflow and outflow of water and thus check the accuracy of the permeability measurement. Triaxial samples with 2.8 inches diameter and 5.6 inches high were prepared using standard Proctor compaction effort at various moisture contents (about 6% below optimum to 8% above optimum). Samples were cured in the moisture room for one week before tests. A vacuum was applied until the constant head tank and the burettes were deaired. Moreover, air was purged from porous stones and cell lines by applying a small pressure to flow the system until a steady state was reached. After samples were setup in the triaxial cell and the triaxial cell was filled with deaired water, the saturation procedure commenced. This procedure used the application of the same upper and lower backpressures, which are a few psi (one psi in this research) lower than that of the lateral pressure. Backpressure was applied in a five psi increment, alternating with corresponding increments of lateral pressure, until 95% degree of saturation was achieved (ASTM 2000d). During the saturation, the saturation percentage was measured by "B" value test by connecting a spare drainage line to an electrical pore pressure transducer and readout system. "B" value was calculated as the percentage of increase in pore pressure to the increase in cell pressure. It took about 24 hours to 48 hours for sample saturation. The permeability test was conducted after saturation was complete. A head differential was imposed across the specimen by applying unequal total heads at the top and bottom of the specimen. On the Tri-Flex 2 system, this was accomplished by adjusting the pressures applied to the upper and lower burette channels. The hydraulic gradients, *i*, used in the tests ranged from 10 up to 50, depending on the permeability of the sample (ASTM 2000d). Table 8 shows the selection of hydraulic gradient recommended by ASTM D5084.

The elevated hydraulic gradient was desired to reduce the test durations required to achieve equilibrium between effluent and influent. The permeability was calculated based on the following equation (Tri-Flex 2 Permeability Test System 2003).

$$K = \frac{V(t_1, t_2)L}{P_B A t} \quad \frac{cm}{\text{sec}}, \tag{9}$$

where $V(t_1, t_2)$ = volume of flow from t_1 to t_2 (cm^3), L = length of sample (cm), P_B = bias pressure psi x 70.37 cm/psi (cm – H_2O), A = area of sample (cm^2), and t = time from t_1 to t_2 (sec).

The permeability tests lasted one to two days until the measured permeability remained constant. At least four readings for each sample were taken, and the averaged value was reported as the permeability for each specimen.

Table 8. Recommended laboratory hydraulic gradients for various hydraulic conductivities (ASTM D5084 2000d)

Hydraulic Conductivity (m/s)	Recommended Maximum Hydraulic Gradient i
$1x10^{-5}$ to $1x10^{-6}$	2
$1x10^{-6}$ to $1x10^{-7}$	5
$1x10^{-7}$ to $1x10^{-8}$	10
$1x10^{-8}$ to $1x10^{-9}$	20
Less than $1x10^{-9}$	30

Test Results and Discussion

Discussions based on the permeability test results are provided in the following sections. These sections are ordered in such a manner (1) to verify the permeability test results are reliable, (2) to verify if the permeability behavior of theses soils complies with the literature, and (3) to develop a predictive equation for these soils and compare with the EICM model.

Reproducibility of Test Results

Hydraulic conductivity tests often give highly variable results, thus making it difficult to report values for K in a reliable manner. Identical samples of a single soil were prepared to investigate the reproducibility of the permeability test results. The soil used in these tests was soil no. A at moisture contents 7.6%, 9.5%, 11.5%, 13.4%, and 15.6%. Three to six samples were identically prepared at each moisture content. The averaged permeability values and deviation were computed and listed in Table 9.

From the results in Table 9, it can be seen that the permeability of soil no. A is in the order of 10^{-10} to 10^{-8} m/sec. Test results vary for identically prepared samples. As can be seen from the results in Table 9, the reproducibility in permeability for a given water content is quite good. In fact, the largest variation ranges from 1.5 times the average to 0.5 times the average, which is much less than one order of magnitude. The variation of the hydraulic conductivity is largest when the moisture content is 15.6%, but it is still within 50% of the averaged value. For other moisture contents, the variation of permeability falls within 25% of the averaged value. The relatively large scatter in data for samples at a moisture content of 15.6% may be due to the clay clods formed during mixing (Stephen and Daniel 1985) since the size of clay clods increases with increasing moisture content. The variation of the size of clay clods in samples compacted wet of optimum is large. Other errors may also occur due to sample preparation (samples are not exactly identical) and test procedures, such as test duration. Although readings were taken after the flow was constant, the hydraulic conductivity tended to decrease slightly with time. However, this difference was small and can be ignored. Overall, on the basis of these tests results, the reproducibility of permeability tests is good.

Table 9. Permeability test results for soil no. A

Sample	Moisture content w (%)	Permeability K (m/sec)	Average K (m/sec)	Deviation from Average(%)
1	7.6	5.42E-08	7.23E-08	-25
2		7.52E-08		4
3		8.75E-08		21
1	9.5	3.45E-08	3.53E-08	-2
2		3.45E-08		-2
3		3.68E-08		4
1	11.5	3.66E-09	3.61E-09	2
2		4.10E-09		14
3		4.02E-09		11
4		2.43E-09		-33
5		4.35E-09		21
6		3.08E-09		-15
1	13.4	6.99E-10	6.93E-10	1
2		6.69E-10		-3
3		7.44E-10		7
4		6.62E-10		-5
1	15.6	1.95E-09	1.31E-09	49
2		1.73E-09		32
3		6.32E-10		-52
4		9.21E-10		-29

Effect of Air-Drying after Compaction

The permeability of compacted soil in the field may be different due to moisture content change even though the dry density is the same. Previous work shows that wet-dry cycles affect the permeability (Lin et al. 2000). The permeability may change dramatically due to swell-shrinkage of soil. Stephen and Daniel (1985) found that permeability increased due to desiccation cracks that formed a few hours after samples were air-conditioned with a temperature of 78ºC. These cracks closed when the testing effective stress was as high as 8 psi. On the other hand, Mitchell et al. (1965) found that the permeability of samples cured in the moist room for a period of time is about an order of magnitude higher than those tested immediately after being extruded from the mold, especially when the samples are compacted wet of optimum. To evaluate the effect of moisture on permeability of compacted soil, two sets of identical samples of soil no. A compacted at various moisture contents were tested. One set of compacted samples was air-dried before testing, with the samples exposed to the air until the weight was constant, while the other set of samples was tested immediately after removal from the compaction mold. The measured permeabilities were plotted as a function of moisture content, as shown in Figure 16.

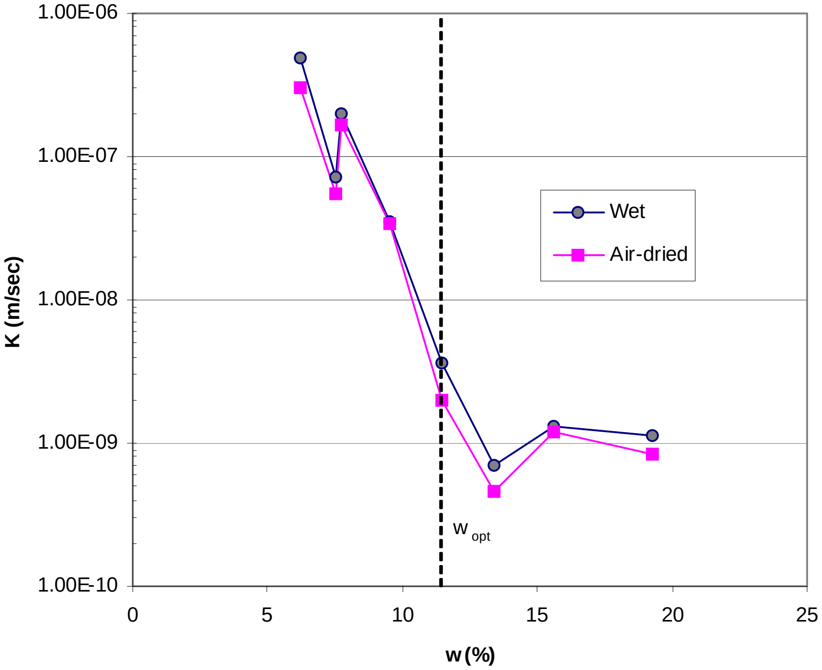


Figure 16. Effect of air-drying on permeability of compacted soil

From the results in Figure 16, it can be seen that the air-dried samples have lower permeability than the permeability of wet samples. The permeability of air-dried samples ranges from about 56% to 95% of that of wet samples. When the samples are dry of optimum, air-drying has little effect on the results. The effect of air-drying slightly increases the difference between air-dried and wet permeability results when the samples are wet of optimum. However, the decrease of permeability due to air-drying is not significant. This observation is different from that by Stephen and Daniel (1985) who reported that permeability was increased due to desiccation of samples. No cracks formed after the samples were air-dried. The decrease in permeability might be due to the shrinkage of soil after air-drying.

Effect of Compaction Moisture Content on Permeability

Permeability tests were performed to investigate the effect of compaction moisture content on permeability of soils no. A, B, C, D, and E. Samples were compacted at various moisture contents from about 5% dry of optimum to 8% wet of optimum. Drier samples than that range of moisture contents show cracks after being removed from the mold, while wetter samples experience local shear failure around the hammer during compaction. Standard Proctor compaction effort was selected due to its use in Iowa subgrade construction. Figures 17 to 21 show the dry density, the strength, and the permeability as a function of moisture content for these soils.

From Figures 17 to 21, it can be seen that the permeability of compacted soil decreases with moisture content when it is dry, while the permeability increases slightly with moisture content when the soil gets wetter. The results are similar to those found by many other investigators such as Lambe (1958), Seed and Chan (1959), and Mitchell et al. (1965). The minimum permeability occurs at a moisture content of about 2% to 4% wet of optimum. The difference in permeability between dry of optimum and wet of optimum is about two to three orders of magnitude. The lower permeability of samples compacted wet of optimum is often attributed to a more dispersed structure that leads to lower permeability (Seed and Chan 1959).

The strength curves show a trend that the maximum strength occurs at about 2% to 4% dry of optimum. Strength decreases as the moisture content increases when the samples were compacted wet of optimum. When compacted dry of optimum, moisture content has less effect on strength (Mitchell et al. 1965).

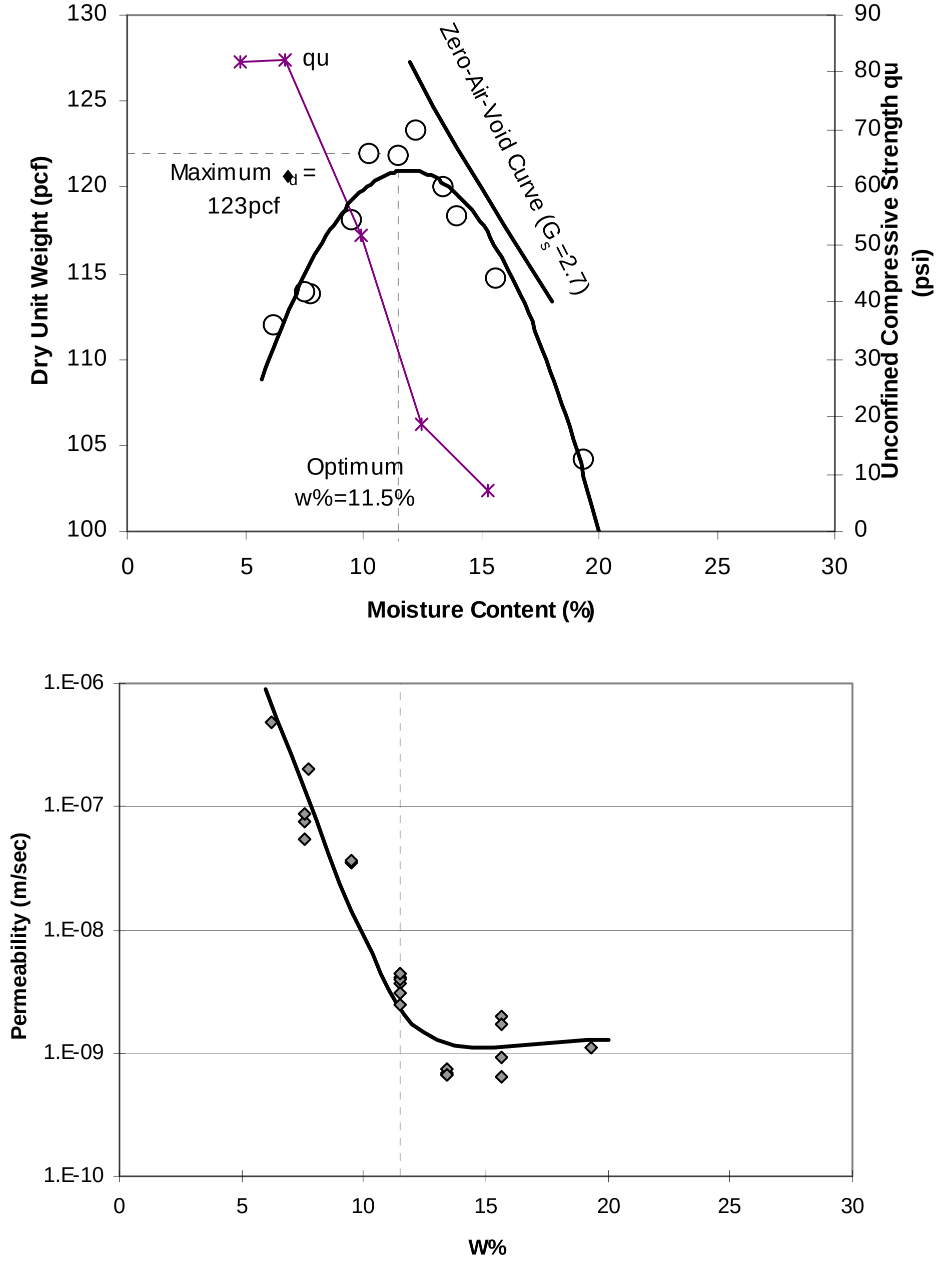


Figure 17. Compaction, permeability behavior of soil no. A

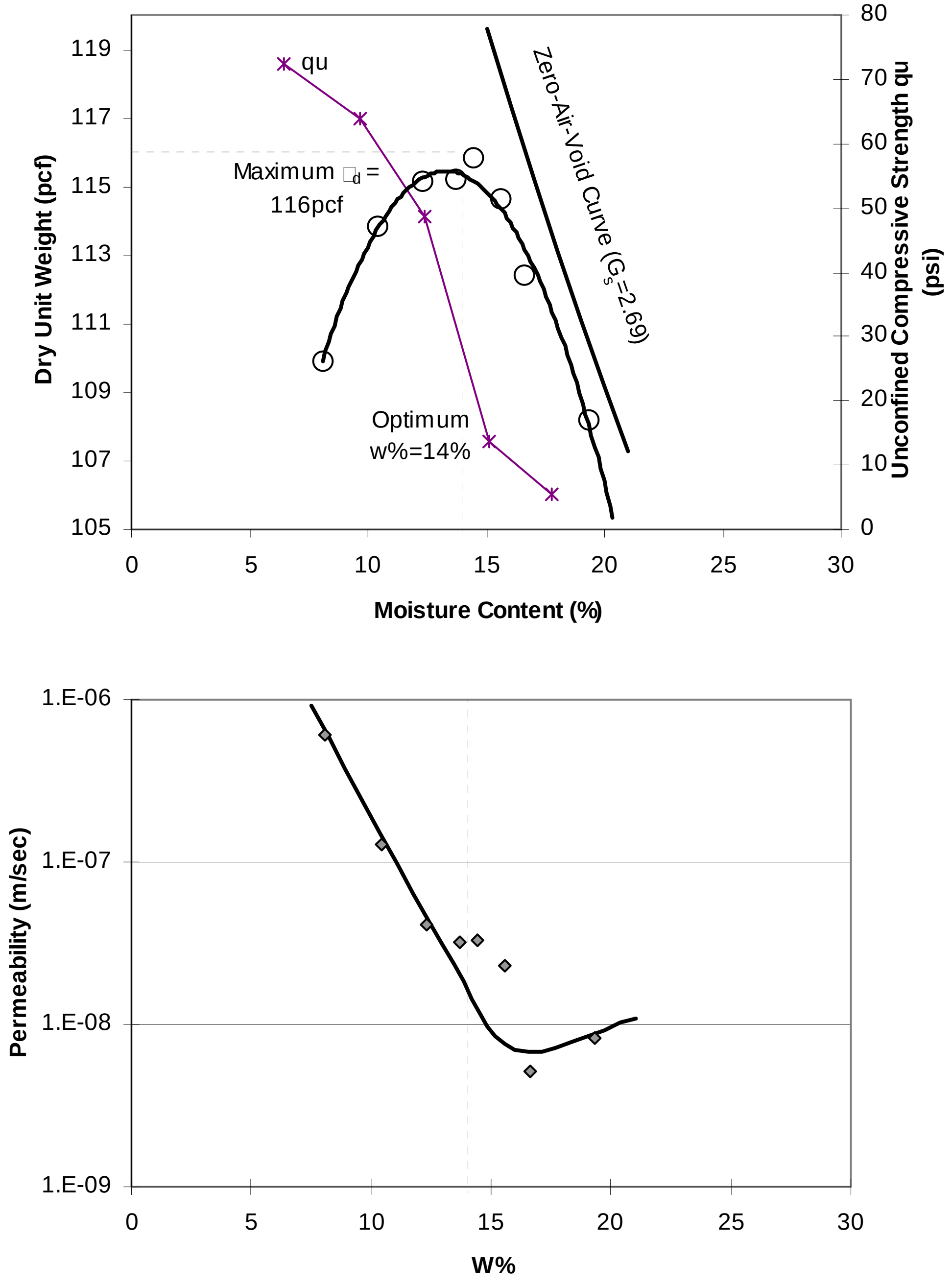
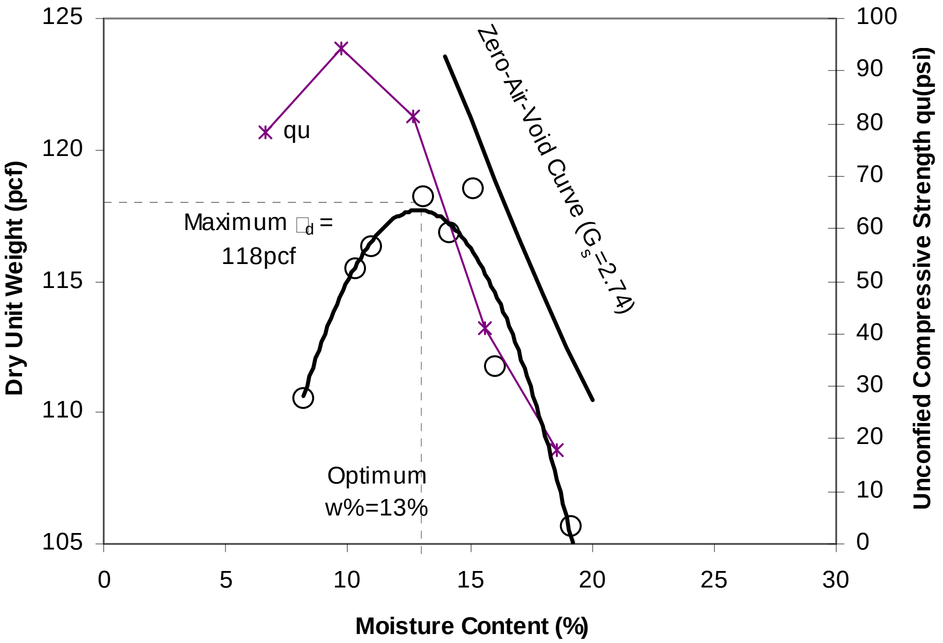


Figure 18. Compaction, permeability behavior of soil no. B



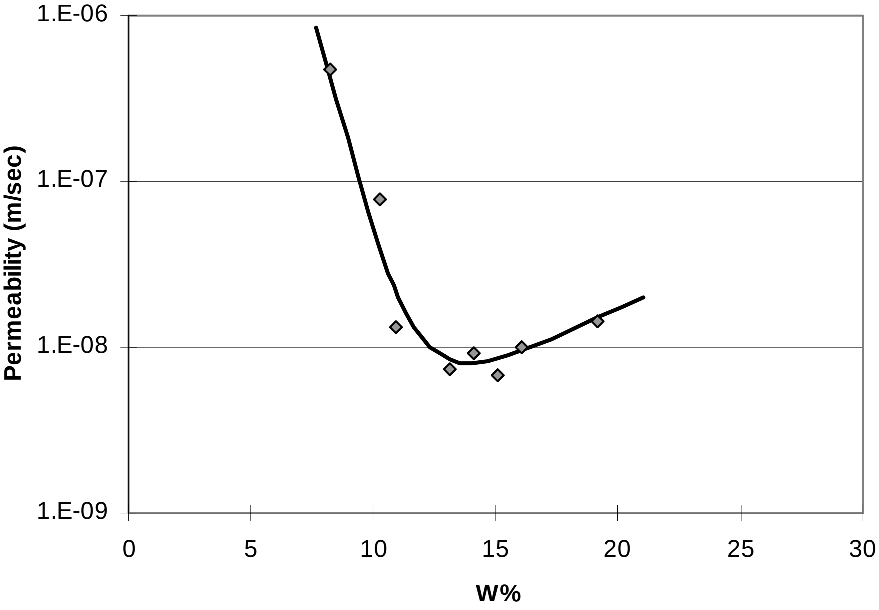


Figure 19. Compaction, permeability behavior of soil no. C

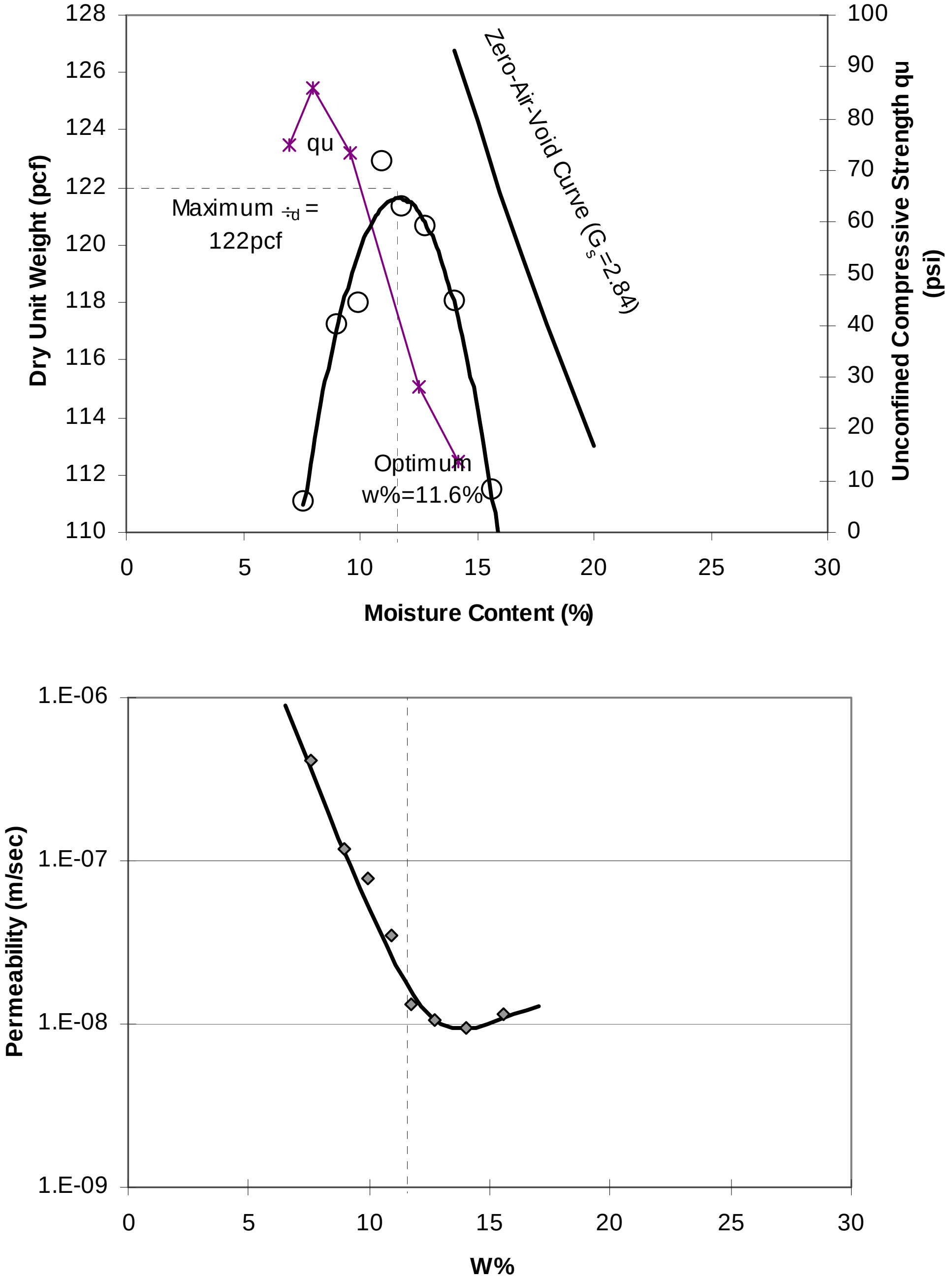


Figure 20. Compaction, permeability behavior of soil no. D

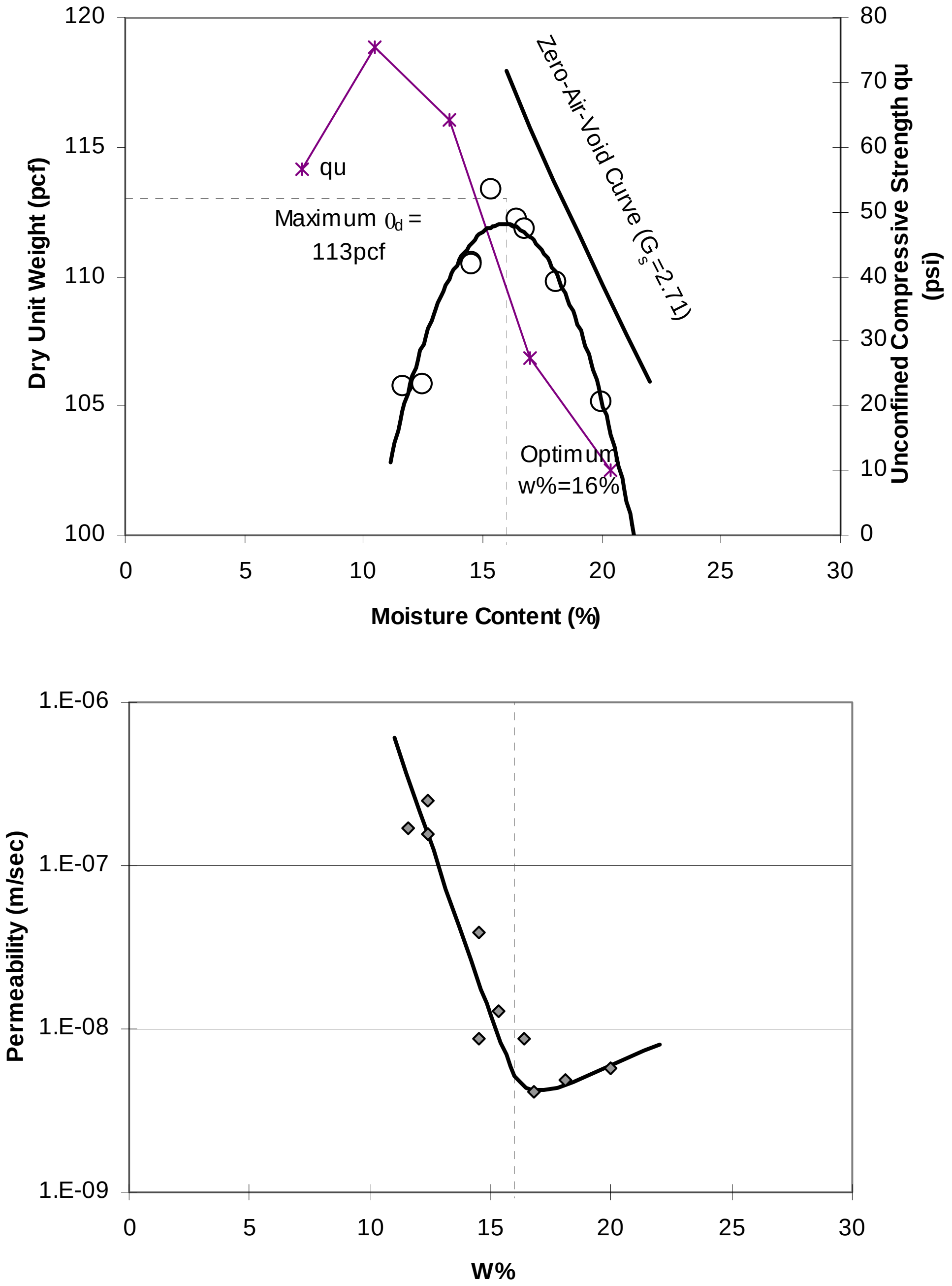


Figure 21. Compaction, permeability behavior of soil no. E

Permeability—Void Ratio Relationship of Soil Samples Dry of Optimum

The experimental data in Figures 17 to 21 were replotted in Figures 22 to 26 using void ratio versus the *log K*. The results in Figures 22 to 26 show that the *log K* vs. *e* plot is not strictly a straight line since the R^2 values are less than one. If the model expressed by equation (3) is applied to these data, the transformed plot *log[k(1+e)]* vs. *log e* can be approximated as a straight line. Figures 22 to 26 show *log K* vs. *e* plots and transformed plots *log[k(1+e)]* vs. *log e* for five cohesive select soils. From the R^2 values in these figures, it is clear that there is a good correlation between *log[k(1+e)]* and *log e* when the samples are compacted dry of optimum. However, the correlation on the wet side is very poor. The *C* and *x* values in the model $k = C\left[\frac{e^x}{1+e}\right]$ (equation [3]) can be obtained directly from these figures: *x* is the slope of the straight line and *C* is the intercept. Table 10 summarizes these parameters.

From the data in Table 10, it can be seen that the exponent *x* ranges from about 11 to 22, which is similar to values determined for overconsolidated clay (Samarasinghe et al. 1982) and is much greater than that of normally consolidated clay, as presented by Sridharan and Nagaraj (2005). The exponent x on the wet side is much different than the values for normally consolidated clay, likely due to the low R^2 values obtained.

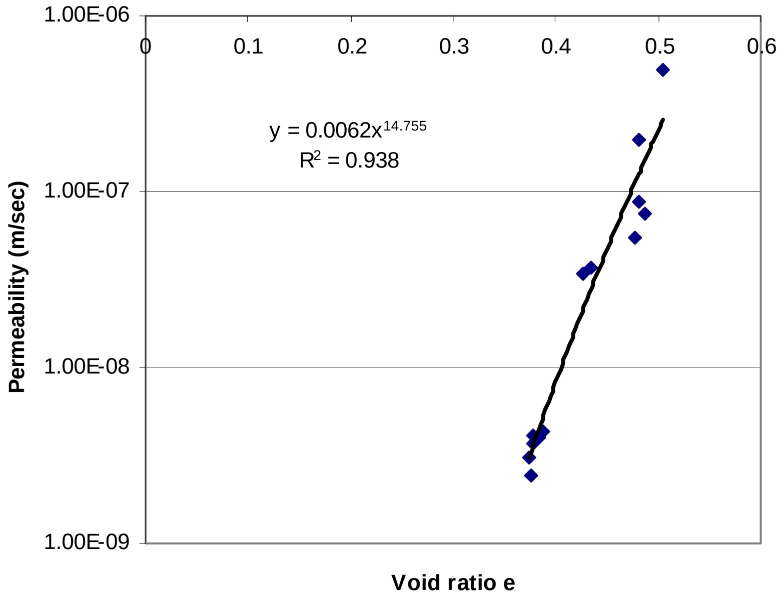
Permeability—Compaction Variables Relationship

Using the empirical model suggested by Harrop-Williams (1985) as shown in equation (6), the $\ln k - x$ curves are plotted in Figures 27 to 32. The regression constants *A* and α were obtained from these figures and are shown in Table 11.

From Table 11, it can be seen that constant *A* ranges from -18.58 to -21.16 and α ranges from -4.97 to -7.1 with a relatively high correlation coefficient R^2 (0.88 - 0.96), meaning 88% to 96% of the variance in permeability is explained by this model. The variation of these two constants is small because the five soils are all glacial till and their compaction methods are the same. By applying Harrop-Williams's model to all the five soils, the regression constants for these Iowa cohesive select soils are obtained with *A* equal to -20 and α equal to -5.63. These constants are very similar to those suggested by Harrop-Williams (1985) (*A* = -19.873, α = -5.19) for compacted clay.

As suggested by Harrop-Williams (1985), *A* and α depend primarily on the soil type and level of compaction. In this study, the level of compaction is the same for all soils so that the soil type is the only factor that influences *A* and α. As discussed previously, the permeability of fine-grained soils is influenced by specific surface area, which can be correlated to some soil index properties, such as liquid limit, plasticity limit, plasticity index, and fine contents. These correlations were investigated, and it was found that strong correlations exist between constant *A* and P_{200}*PI and α and LL. Figures 33 and 34 show the plots *A* versus P_{200}*PI, and α versus LL, respectively. From Figures 33 and 34, it is clear that there is a good correlation of *A* with P_{200}*PI ($R^2 = 0.95$), and α with LL ($R^2 = 0.80$).

These results indicate that a model using the liquid limit and the percent finer than the No. 200 sieve can estimate the permeability of these compacted clays.



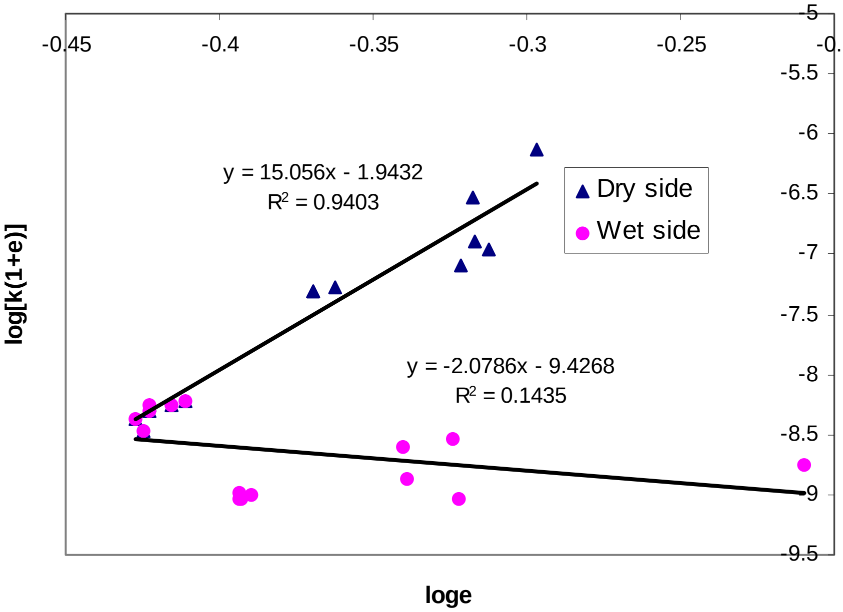
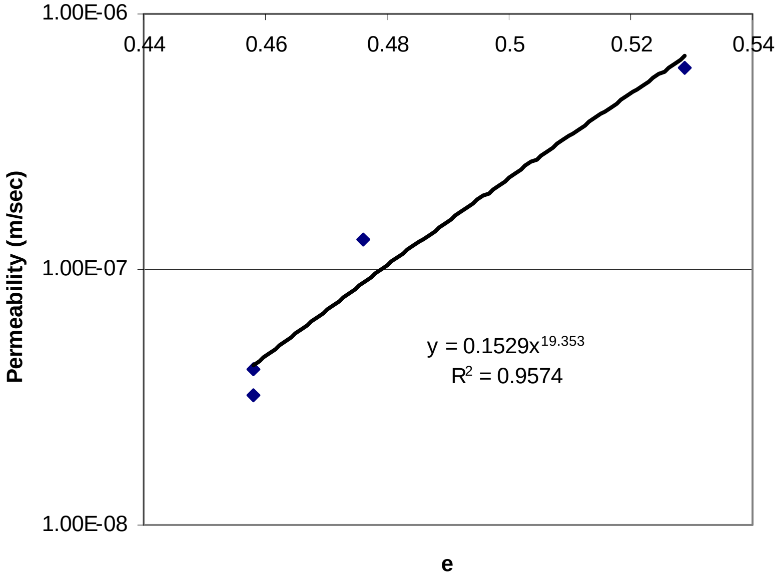


Figure 22. Permeability versus void ratio relationship for Soil No. A



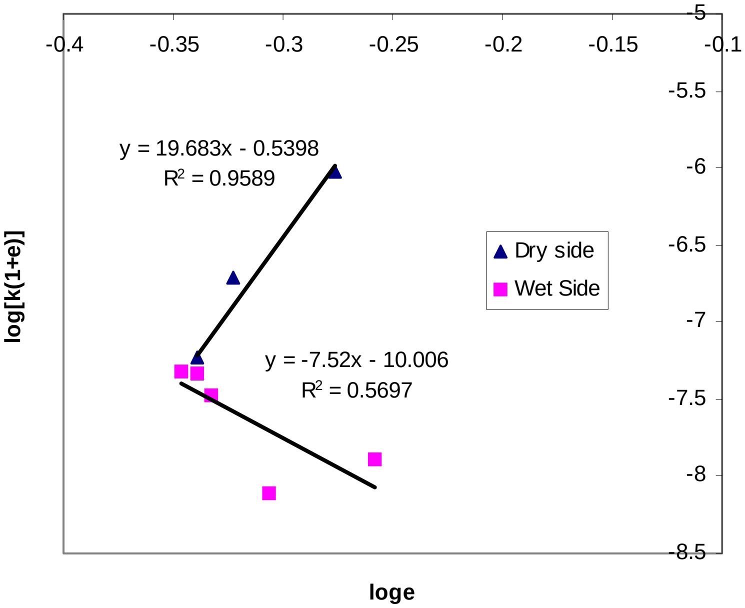
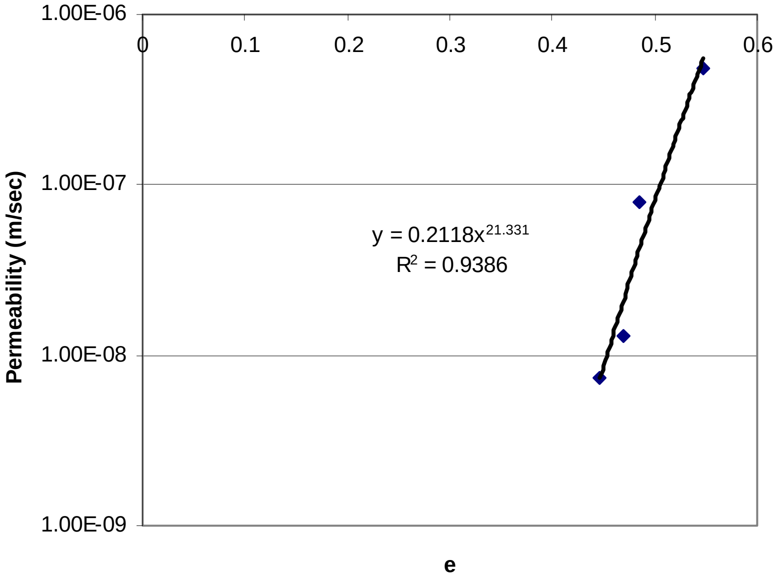


Figure 23. Permeability versus void ratio relationship for soil no. B



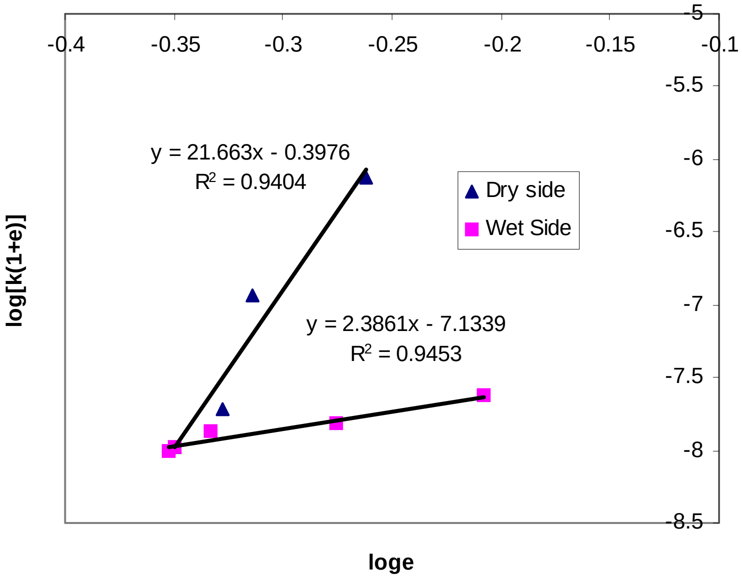
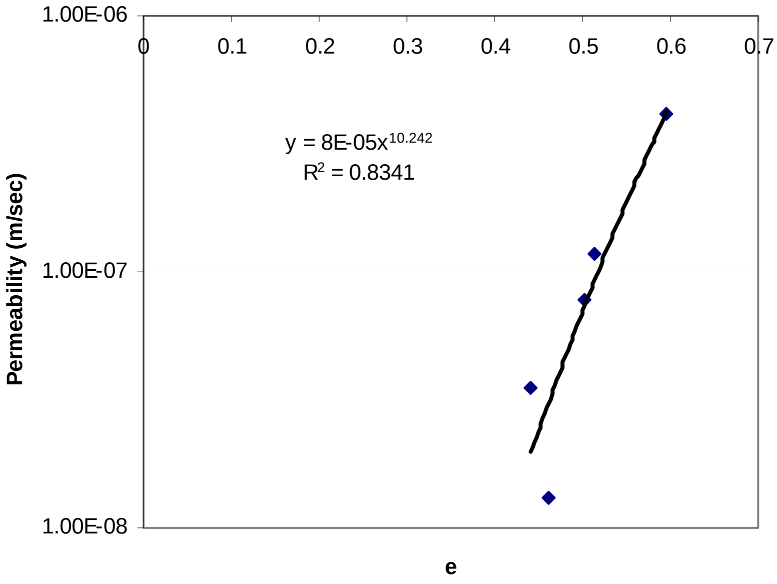


Figure 24. Permeability versus void ratio relationship for soil no. C



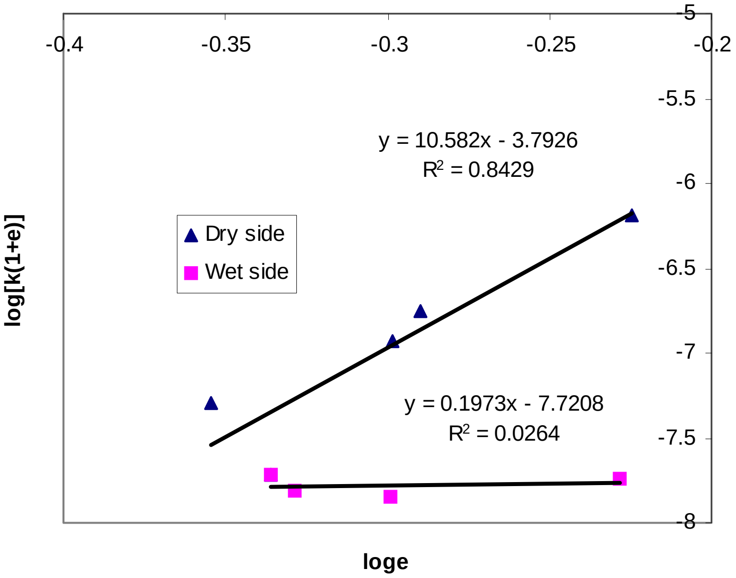
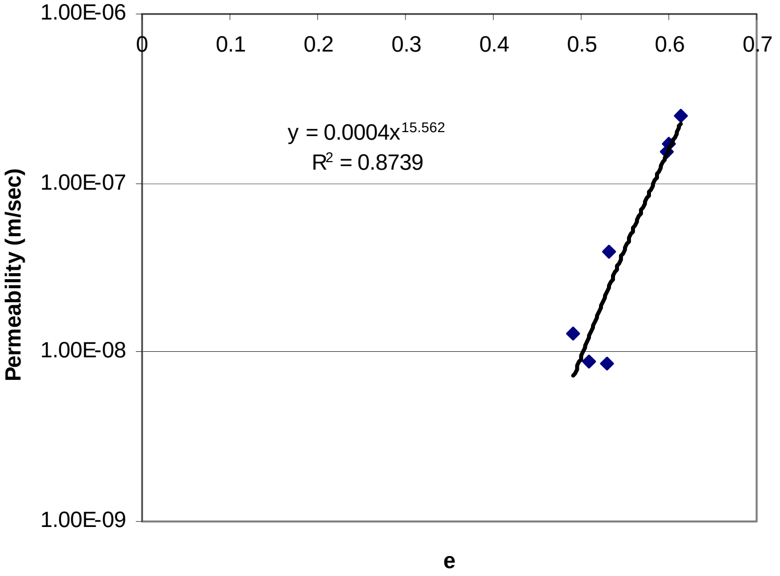


Figure 25. Permeability versus void ratio relationship for soil no. D



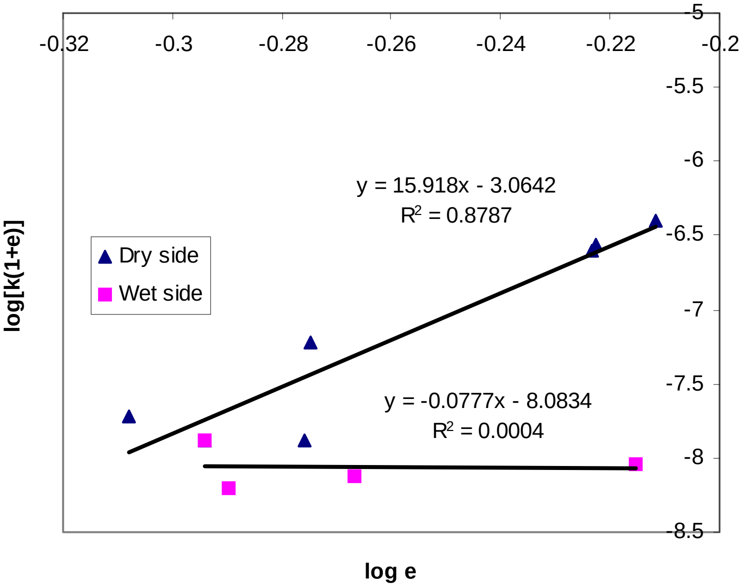


Figure 26. Permeability versus void ratio relationship for soil no. E

Table 10. Permeability parameters for five cohesive select soils in Iowa

Soil No.	Optimum w%	Maximum dry unit weight γd(pcf)	Dry side		Wet side	
			x	C (m/s)	x	C (m/s)
A	11.5	122	15.06	1.14E-02	-2.08	3.74E-10
B	14	116	19.68	0.29	-7.52	9.86E-11
C	13	118	21.66	4.00E-01	2.39	7.35E-08
D	11.6	122	10.58	1.61E-04	0.20	1.90E-08
E	16	113	15.92	8.63E-04	-0.08	8.25E-09

Table 11. Regression constants of five cohesive select soils from Iowa

Soil No.	optimum w %	Maximum dry unit weight γd (pcf)	Variables							
			A	α	R^2	LL	PL	PI	P200 (%)	PI*P200
A	11.5	122	-21.16	-6.24	0.92	26	13	13	54.06	7.03
B	14	116	-18.58	-4.97	0.88	35	19	16	70.21	11.23
C	13	118	-19.42	-5.14	0.88	34	18	16	65.87	10.54

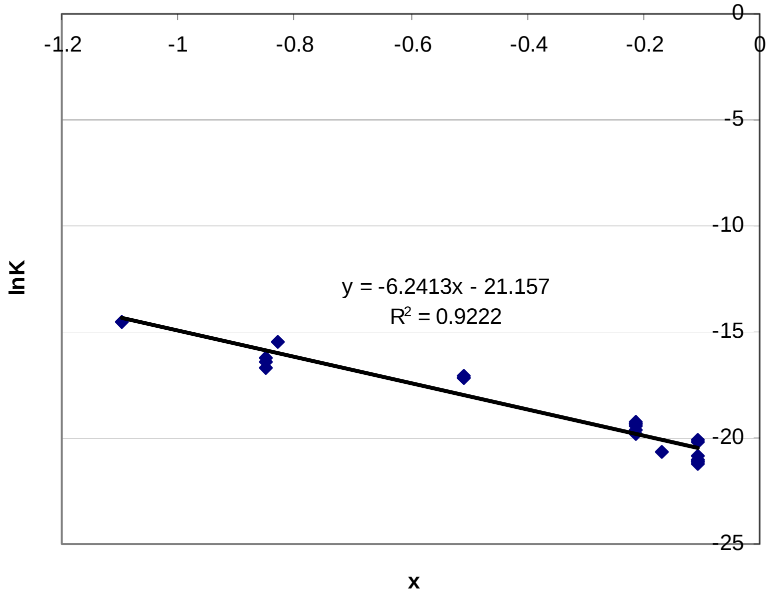


Figure 27. Permeability versus compaction variables for soil no. A

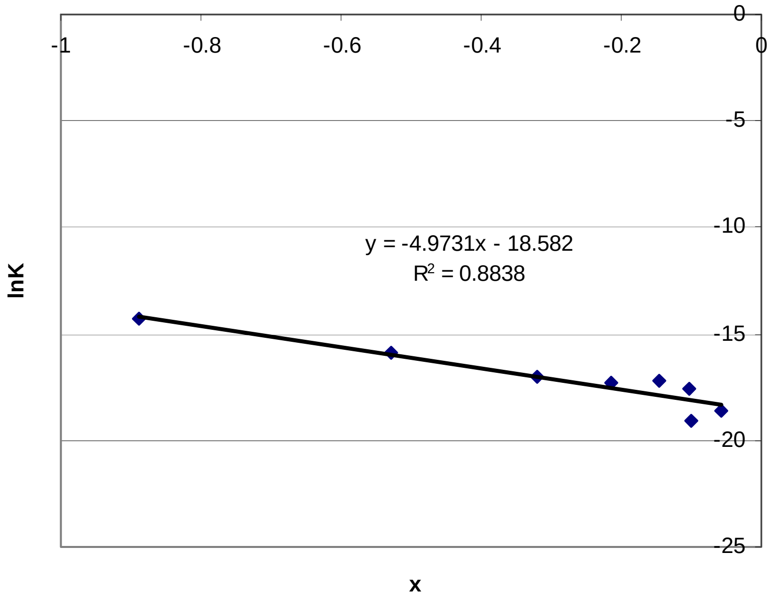


Figure 28. Permeability versus compaction variables for soil no. B

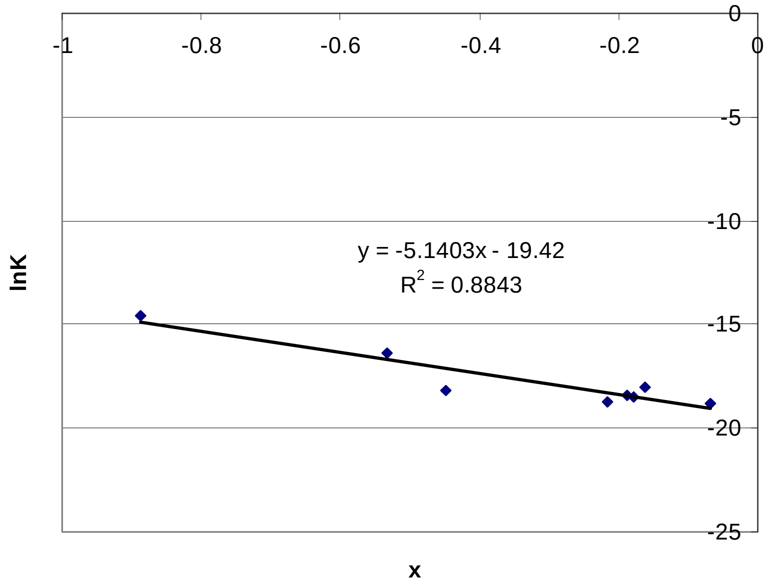


Figure 29. Permeability versus compaction variables for soil no. C

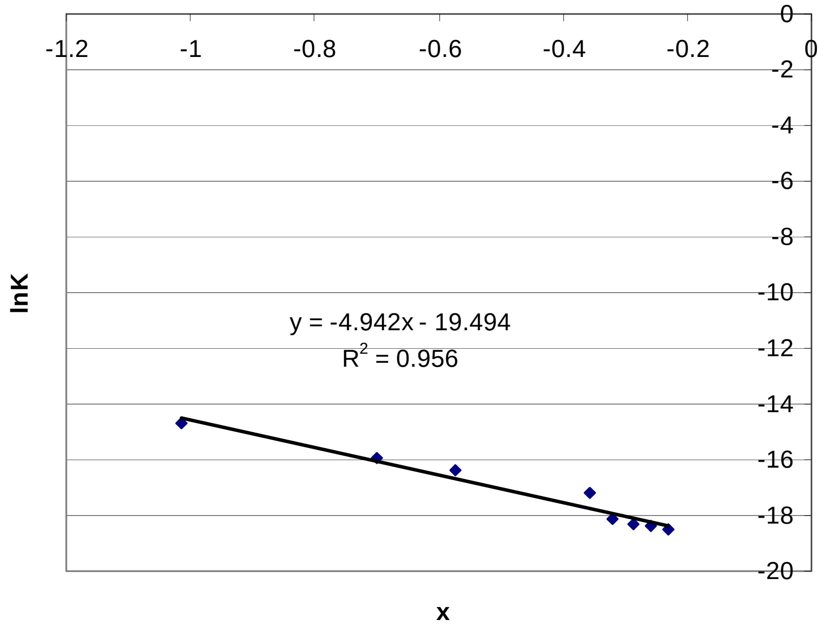


Figure 30. Permeability versus compaction variables for soil no. D

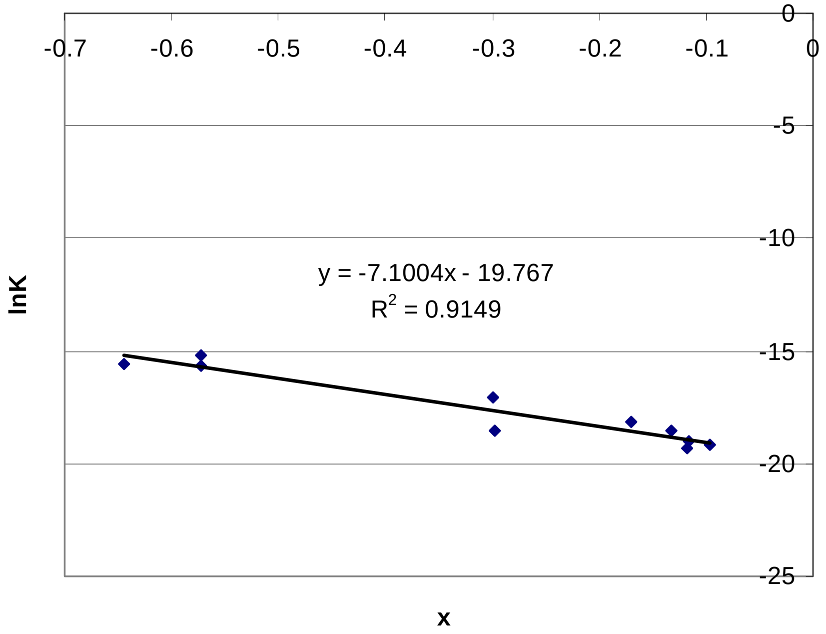


Figure 31. Permeability versus compaction variables for soil no. E

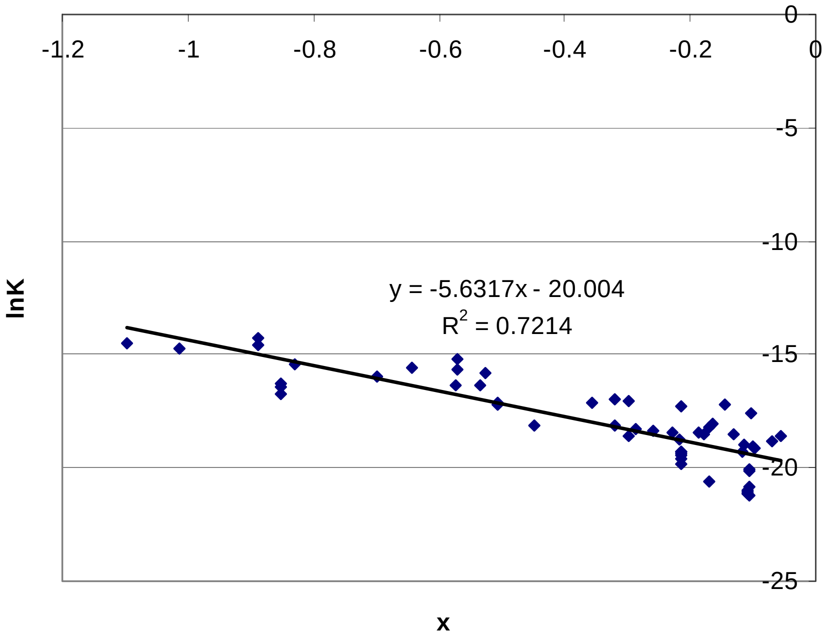


Figure 32. Permeability versus compaction variables for all five soils

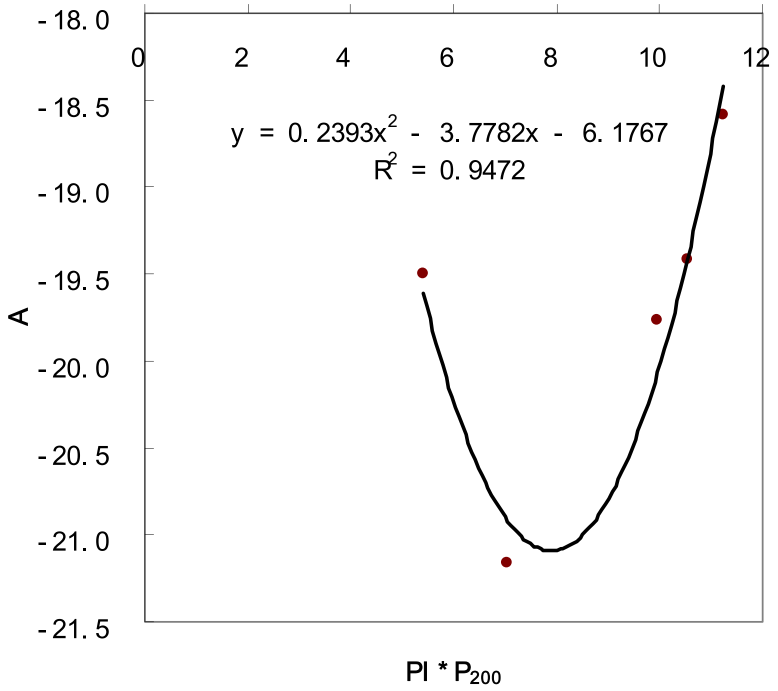


Figure 33. A versus PI*P_{200} for all five Iowa cohesive select soils

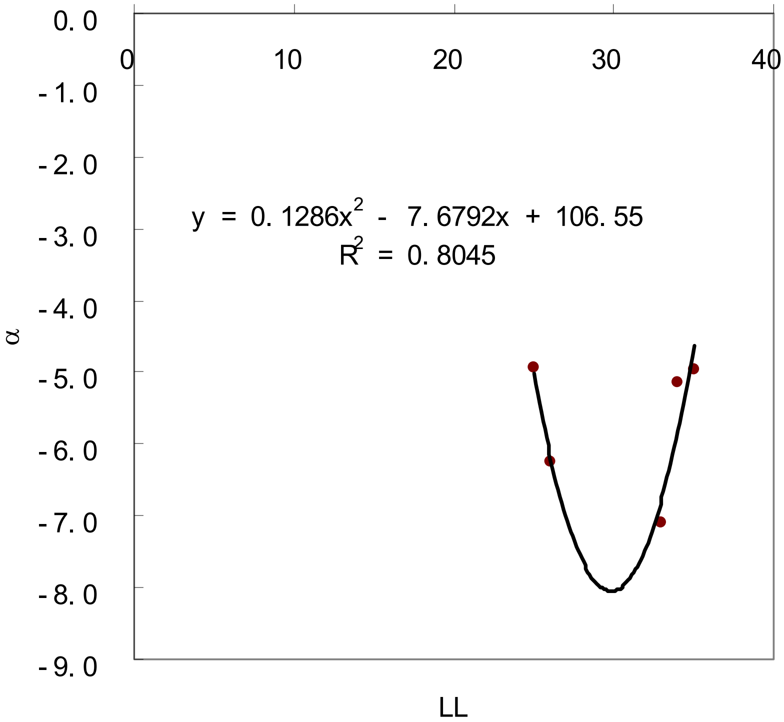


Figure 34. α versus LL for all five Iowa cohesive select soils

Comparison of Measured Permeability to NCHRP 1-37A Empirical Equations

As mentioned previously, equations (4) and (5) are incorporated in the EICM model to estimate the saturated permeability of compacted soil at optimum moisture content. The soil parameters used in these equations are P_{200}, PI, and D_{60}. To evaluate these equations, the k_{sat} was calculated and compared to the measured values, as shown in Table 12 and Figure 35.

Table 12. Comparison of measured permeability to NCHRP 1-37A empirical equations

Soil No.	D_{60} (mm)	PI	P_{200}	calculated k_{sat} (ft/hr) (from eq. (3))	calculated k_c (m/sec) (from eq. (3))	Measured k_m (m/sec)	k_c/k_m
A	0.13	13	54	7.58E-06	6.42E-10	3.61E-09	0.18
B	0.06	16	70	3.33E-06	2.82E-10	3.33E-08	0.01
C	0.06	16	66	3.77E-06	3.19E-10	7.33E-09	0.04
D	0.1	10	54	1.05E-05	8.91E-10	1.31E-08	0.07
E	0.04	15	66	4.28E-06	3.63E-10	8.73E-09	0.04

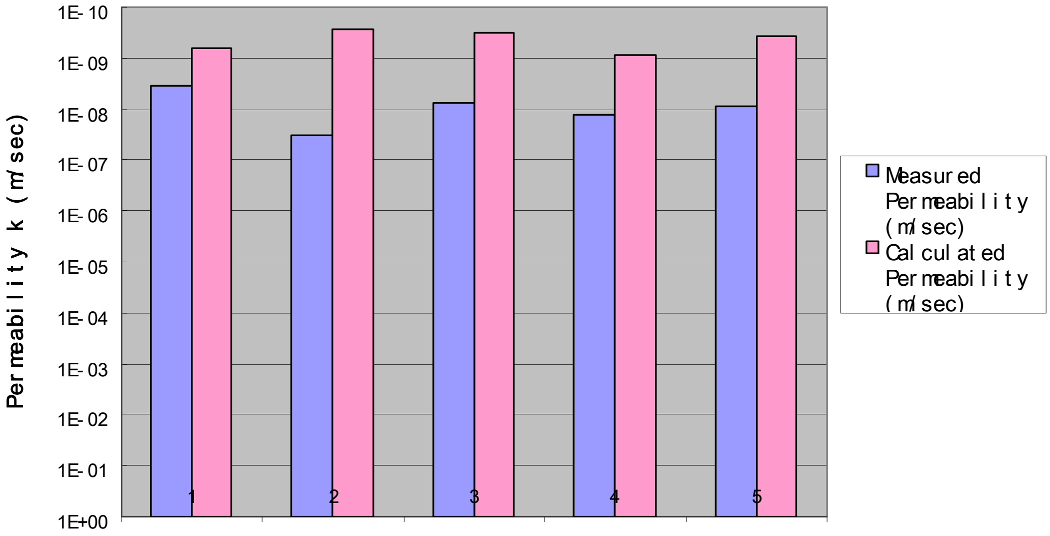


Figure 35. Comparison of NCHRP 1-37A calculated permeability and measured permeability

It can be seen from Figure 35 that the difference of measured and calculated permeability using equations (4) and (5) is between 1 to 2 orders of magnitude. One of the objectives of this study is to find an improved model to estimate permeability. A regression analysis from measured permeability and PI*P_{200} is given in Figure 36.

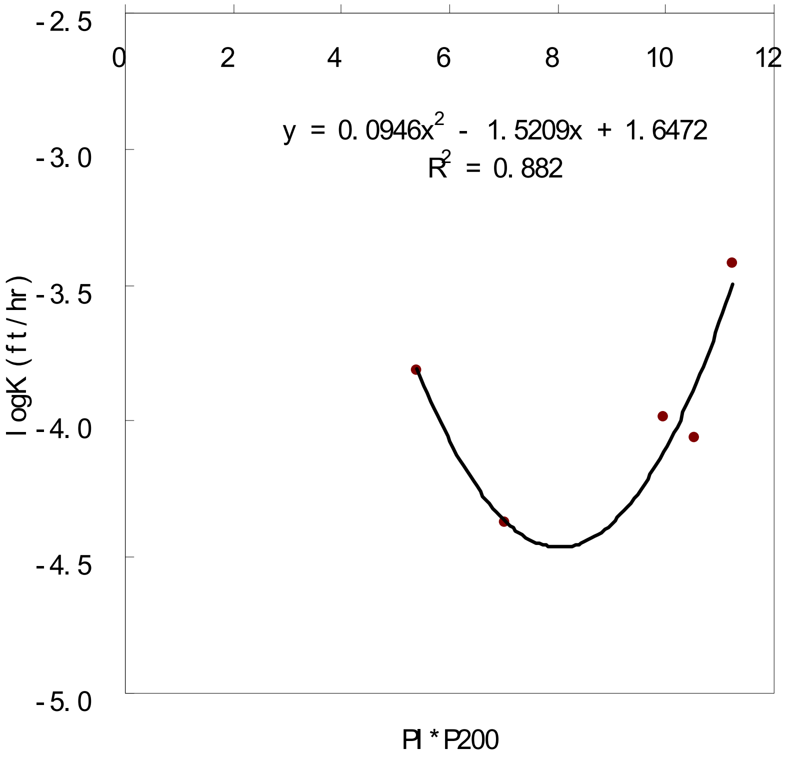


Figure 36. log K vs. PI*P_{200}

Figure 36 shows that permeability of compacted soils can be expressed by PI*P_{200}. Since PI*P_{200} is greater than one, the regression model is provided in a similar form to equation (5):

$$k_{sat} = 10^{[0.095\,(P_{200}\,PI)^2 - 1.52\,(P_{200}\,PI) + 1.65]} \text{ (ft/hr).} \qquad (10)$$

Although equation (10) has a similar form as equation (5), the permeability value obtained from each equation has one to two orders of magnitude difference, which indicates that k_{sat} may as well depend on other factors.

According to Benson and Trast (1995), the initial saturation (S_i) and compaction effort (E) are more important factors than P_{200} and PI. Therefore, S_i and E were studied in this research. Equation (8) incorporated both factors and is applied. The comparison of calculated k from equation (8) and measured k is summarized in Table 13, which shows that if the initial degree of saturation is included in the model, the difference between calculated permeability and measured permeability is within one order of magnitude.

Overall, if PI, P_{200}, LL, and compaction variables, such as γ_d and *w*%, are all incorporated in the model, according to equation (4) and Figures 33 and 34, permeability can be expressed as follows:

$$\ln k = 0.24(PI * P_{200})^2 - 3.78(PI * P_{200}) - 6.2 + [0.13 * (LL)^2 - 7.68 * LL + 106.6]\ln\left(\frac{\gamma_d wG}{G\gamma_w - \gamma_d}\right). \qquad (11)$$

Equation (11) provides a general prediction model for permeability of compacted clay.

For Iowa cohesive select soils, *A* is about -20, α is about -5.63. This model can be simplified as follows:

$$\ln k = -20 - 5.63\ln\left(\frac{\gamma_d wG}{G\gamma_w - \gamma_d}\right), \qquad (12)$$

where k = permeability of compacted soil (m/s), PI = plasticity index, P_{200} = percent finer than No. 200 sieve, γ_d = dry unit weight of compacted soil, γ_w = unit weight of water, and G = specific gravity of soil.

To validate equations (11) and (12), all the test data is presented in Table 14.

It can be seen from Table 14 that the new model predicts the permeability very well; the ratio of predicted permeability and the measured permeability ranges from 0.16 to 1.78. Compared to equation (5), equation (11) is a more comprehensive model in that it incorporates the index properties (PI, P_{200}, and LL), compaction characteristics, and initial degree of saturation (which is implied by moisture content, specific gravity, and dry unit weight). For Iowa soil, the permeability of select soil can be predicted by equation (12), even in the absence of the index properties.

Table 13. Comparison of measured permeability to Benson and Trast's regression model (1995)

Soil No.	D_{60} (mm)	PI	P_{200}	S_i	calculated k_c (m/sec) (from eq. (7))	Measured k_m (m/sec)	k_c/k_m
A	0.13	13	54	57	1.39E-9	3.61E-09	0.38
B	0.06	16	70	20	6.5E-8	3.33E-08	1.26
C	0.06	16	66	22	3.23E-8	7.33E-09	4.41
D	0.1	10	54	35	1.11E-8	1.31E-08	0.85
E	0.04	15	66	45	4.61E-9	8.73E-09	0.53

Table 14. Validation of permeability model equations (9) and (10)

Soil No.	Optimum W%	Maximum dry unit weight γ_d (pcf)	G	LL	PL	PI	P_{200} (%)	$PI*P_{200}$	calculated k_c (m/sec) using eq(9)	calculated k_c (m/sec) using eq(10)	Measured k_m (m/sec)	k_c/k_m eq(8)	k_c/k_m eq(9)
A	11.5	122	2.70	26	13	13	54.1	7	2.37E-09	6.44E-09	3.61E-09	0.66	1.78
B	14	116	2.69	35	19	16	70.2	11.2	1.71E-08	5.35E-09	3.33E-08	0.51	0.16
C	13	118	2.74	34	18	16	65.9	10.5	1.01E-08	7.50E-09	7.33E-09	1.38	1.02
D	11.6	122	2.84	25	15	10	54.1	5.4	1.11E-08	1.22E-08	1.31E-08	0.85	0.93
E	16	113	2.71	33	18	15	66.5	10	4.04E-09	4.37E-09	8.73E-09	0.46	0.50

Conclusions

The permeability of five compacted cohesive select soils from Iowa were evaluated. Several conclusions were drawn from the results of this study.

1. Permeability test is reproducible, provided that the variance of test results is between 49% below the averaged value and 52% above the averaged value. The higher the moisture content, the higher the variance.
2. Air-dried samples have lower permeability than that of wet samples tested right after extruded from compaction mold. The permeability of air-dried samples is from about 56% to 95% of that of wet samples. Air-drying has less effect on the results when the samples are compacted dry of optimum than when they are compacted wet of optimum.
3. Permeability of compacted soil decreases with moisture content when it is dry, while the permeability increases slightly with moisture content when the soil gets wetter. The lowest permeability occurs at a moisture content of about 2% to 4% wet of optimum.
4. There is a good correlation between $\log[k(1+e)]$ and $\log e$ when the samples are compacted dry of optimum. However, the correlation in the wet side is very poor. When equation (3) applies, the x value ranges from 11 to 22 for overconsolidated clays.
5. Permeability can be predicted using Harrop-Williams (1985) model (equation [6]), where A ranges from -18.58 to -21.16 and α ranges from -4.97 to -7.1 for Iowa cohesive soils.
6. The difference between the measured permeability and the calculated permeability using EICM model is one to two orders of magnitude.
7. Equations (11) and (12) provide a more comprehensive model based on the Harrop-Williams (1985) model to predict compacted soil permeability, which incorporated PI, LL, P_{200}, w%, and γ_{dry}.

FIELD INVESTIGATION OF SETTLEMENT ADJACENT TO HIGHWAY CULVERTS

Introduction

Culverts are commonly installed to deal with the highway drainage needs. Installation of a culvert generally necessitates an excavation in natural or fill materials, with subsequent compaction of soil materials to fill back in the excavated materials. Unfortunately, settlement adjacent to highway culverts has often been found shortly after the road is open to traffic. Although not perceptible to the naked eye, the settlement near a culvert is noticeable in a vehicle driving though these locations. To address this issue, an investigation of the causes of the problem and the development of a solution was undertaken.

Settlement is a common problem following culvert installation and is often due to poorly compacted sand and rock backfill (crushed lime stone) materials. Displacement of soft material or piping along the culvert can cause significant damage to the culvert. Poorly compacted backfilled soils are subject to substantial volume reduction once saturated (Selig 1990). This phenomenon is called collapse compression. Collapse occurs because the backfill soils lose capillary tension when they become saturated, causing the soil particles to settle into a denser packing. Selig (1990) noted that collapse compression strain decreases as the degree of compaction increases and diminished to an insignificant amount once the compaction reaches 85% to 90% of standard Proctor unit weight. The settlement of backfill materials causes bumps on the pavement surface and distress to the drain pipes buried in the sand backfill. This distress might lead to cracking of pipes, resulting in leakage of water into the subsoil. If the subsoil consists of problematic soils, such as expansive or collapsible loess soils, seeping of water into these soils could trigger further intricate problems of volume changes that are detrimental to the engineering performance of pavements. Although there is considerable information available on the design and construction of new culverts, there is little information in the literature on how to repair culvert problems and even less on how to rehabilitate, strengthen, or retrofit upgrade culverts. Initiating any kind of remedial measures requires a thorough investigation of the soil profile and properties of different soils in different strata.

Figure 37 shows the general nature of the problem. The two-foot diameter pipe under the poorly compacted rockfill is subjected to distress owing to the settlement of the rockfill under traffic loads. The effect of soil settlement is shown in Figure 38. Field investigation indicates that settlement occurs near the pavement surface right above the pipe. Settlement of backfill material and movement of the structure can have serious structural consequences in pipes, such as misalignment or rupture of the pipe system. A stable soil envelope around pipes is necessary for side support that will reduce settlement of rigid pipe. Figure 39 shows the remediation alternatives. It is proposed that, to ward off large amounts of settlement, the backfill above the center of pipe is replaced by flowable mortar, the major components of which are fly ash, cement, sand, and water.

The objectives of this portion of the study were to (1) investigate culvert settlement problems in Iowa, (2) review the remediation methods in the literature, and (3) study the select and flowable mortar as backfill options.

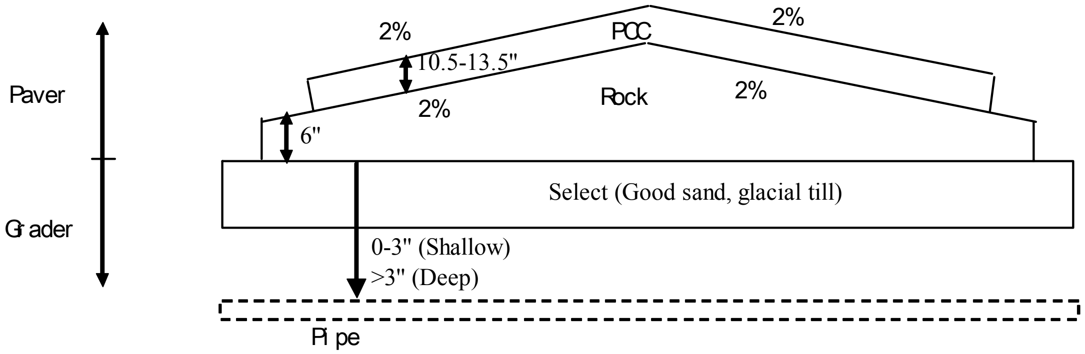
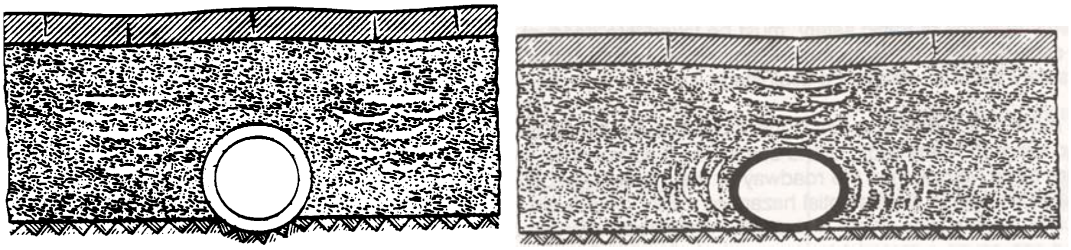
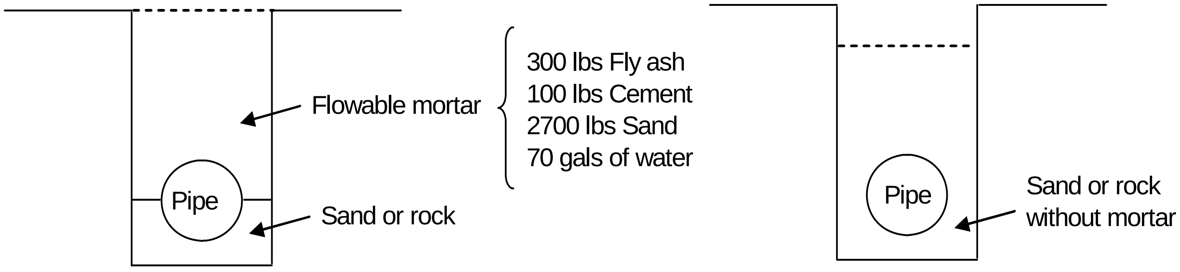


Figure 37. Cross-section of pavement



(a) Rigid pipe **(b) Flexible pipe**

Figure 38. The effect of soil settlement on (a) rigid and (b) flexible pipes (US Army 1959)



(a) Shallow Pipe (b) Deep Pipe

Figure 39. Proposed treatment of materials surrounding the pipe (Iowa DOT specification 2001)

Background

The pipe (culvert) and the soil surrounding it work as an integral system, which means that weakness in the surrounding soil affects the performance of the drainage pipe. Thus, the quality of backfill materials is of vital importance to the pipeline safety regarding bearing capacity and settlement issues. For culvert systems, compacted, well-graded, angular, granular backfill materials provide the best structural support. These backfill materials comprise primarily selected soils restricted to A-1, A-2, and A-3 classifications and are compacted to a density of more than 90% of standard Proctor maximum dry unit weight (US Army 1959). However, the use of conventional backfill materials is hard to meet the compaction criteria due to site restriction, soil conditions, equipment limitation, and workmanship, especially in the area right next to the pipe and below the spring line. Therefore, materials that can be compacted easily or possess a high stability to resist the deformation from the repeated loading have recently been used. These materials are granular backfill, concrete sand, well-graded crushed stone, recycled portland cement concrete, flowable mortar, etc.

The following factors are considered when selecting a backfill material: ease of compaction, labor requirements, degree of inspection, and cost (US Army 1959). Granular backfill or sand and flowable mortar would be the best choices. Currently, flowable mortar is becoming more and more popularly used as pipeline backfill (ACI Report 229R-94 1994). As the American Concrete Institute (ACI) Report 229R-94 (1994) outlines, this material possesses some potential advantages over conventional soil fill. A literature review of this material is provided in the following section.

Controlled Low-Strength Material

Flowable mortar is also called “Controlled Low Strength Material-Controlled Density Fill” (CLSM-CDF) by ACI Committee 229 (ACI Report 229R-94 1994). It is a low-strength material mixed to a wet, flowable slurry and used as an economical fill or backfill material. In the case of CLSM-CDF, the material components, when mixed and placed, must possess the following properties: flowability, removability, strength, and a competitive price. (ACI Report 229R-94 1994) The basic components for CLSM-CDF are portland cement, fine aggregate, fly ash, and water.

Portland Cement

Type I or Type II portland cement that conforms to ASTM C 150 (ASTM 1995) is usually used. The amount is approximately 3% of the total mixture’s weight. The purpose of the cement is to provide cohesion and strength control in the mixtures. For typical backfills where future removability is anticipated, the compressive strength (c’) should be less than 100 psi at 28 days (ACI Report 229R-94 1994).

Fine Aggregate

The fine aggregate, known as filler, makes up the major portion (72%) of a typical CLSM-CDF mixture. This material was utilized because of its availability and it proved to be an excellent CLSM-CDF filler. The filler should possess adequate gradation similar to the requirement as set forth in ASTM C 33 (ASTM 1995) to insure proper flowability. Another filler material consideration is the material’s particle angularity. Naturally, a material containing particles with sharp edges will result in less desirable flow characteristics. Aggregates, such as pea gravel with sand, sand with 10% silt, and quarry waste products, have been proved to be working well in CLSM.

Fly Ash

Fly ash used in portland cement concrete usually complies with ASTM C 618 (ASTM 1995). Fly ash makes up approximately 8% of a typical CLSM-CDF mixture (ACI Report 229R-94 1994). The majority of the CLSM research has been conducted using Class F fly ash (ACI Report 229R-94 1994). In addition to finding a suitable use for an industrial by-product, fly ash used in CLSM mixtures also provides several benefits, such as better workability, lower cost, reduced hydration heat, reduced shrinkage upon drying, and greater strength gain beyond the first 28 days (Sargand et al. 2001).

Water

Water is used in a CLSM mixture for flowability and hydration. Water increases in CLSM mixtures do not affect its compressive strength. However, water reductions below the design level will increase the compressive strength. A typical CLSM-CDF mixture would consist of approximately 17% water.

Flowable Fill Design

The purpose of flowable fill is to provide a structural substitute for compacted granular soil. As such, it needs sufficiently low strength to allow for future excavation while providing adequate support for the pipe (Hitch et al. 2002). Discussions with producers of flowable fill and literature research yielded the flowable fill specifications shown in Table 15. The recommended starting point for lab investigations for CLSM mix design for strengths of 100 psi or less is shown in Table 16.

Flowability

CLSM is able to flow into places that are hard to reach (haunches) or in narrow trenches where space limited, thus eliminating all labor requirements for placement. Flowability tests were conducted on all trial batches by placing a freshly mixed sample of CLSM in a 75 mm diameter (3 in.) by 150 mm (6 in.) high open-ended tube and quickly lifting the tube vertically, allowing the CLSM sample to slump into a circular mound. The circular sample spread was then measured. A minimum acceptable spread of 200 mm (8 in.) and no segregation of water were adopted acceptance criteria based on guide specifications of the Texas Aggregates and Concrete Association. To achieve this number, a water/solid ratio of 0.4 to 0.6 can be used for flowability design.

Table 15. Flowable fill specifications (Hegarty et al. 1998)

Property	Value
Unit weight	142 pcf (2275 kg/m^3)
Slump (max)	9 in. (22.8 cm)
Water/cement ratio	0.68
28-Day compressive strength (max)	100 psi (70310 kg/m^2)

Table 16. Flowable fill mix design

Property	Value
Portland cement (Type I) ASTM C150	100 lbs/yd^3 (59.32 kg/m^3)
Fly ash, ASTM C618 Class F	300 lbs/yd^3 (177.96 kg/m^3)
Aggregate, ASTM C33	2600 lbs/yd^3 (1542.32 kg/m^3)
Water	584 lbs/yd3 (346.43 kg/m^3)
Unit weight	127.8 pcf (2050 kg/m^3)
Compressive strength: 14 days	60 psi (0.41 MPa)
Compressive strength: 28 days	60 psi (0.41 MPa)
Compressive strength: 90 days	160 psi(1.10 MPa)
California Bearing Ratio (CBR), %: 0.1 in.	19.7
California Bearing Ratio (CBR), %: 0.2 in.	23.7

Removability

If the CLSM is to be excavated in the future, removability must be considered. As stated in ASTM D 4832 (ASTM 1995), the only structural requirement for the flowable mortar is that its minimum compressive strength is slightly higher than the surrounding soil; 50 psi is a typical value. However, to ensure removability, unconfined compressive strength of less than 100 psi is required.

Advantages of CLSM Compared to Conventional Backfills

First, CLSM does not form void spaces during placement like soils and will not settle under loads. CLSM's load-carrying capacity is higher than compacted soils, and its relatively low long-term strength (50 to 100 psi) makes it easy to remove in the future. Second, using CLSM is cost-effective because its placement procedure is less labor intensive. Brewer et al. (1991) made an illustrative comparison based on the material prices in Ohio and found that for a roadway trench with the dimensions of 3 ft wide, 6 ft deep, and 40 ft long, the total cost of the CLSM and conventional backfill is about equal. However, every one-foot reduction in trench width in this example represents a backfill cost reduction of $255.64 when using CLSM (Brewer et al. 1991). The trench width is reduced because conventional compaction requires additional access width around the culvert, and CLSM does not require significant testing and inspection during placement, both of which reduce labor cost.

The end use of the CLSM mixture must be known to design the proper mixture. For example, if removability is not to be a factor, then the compressive strength 100 psi or more would not be a factor. Similarly, if the mixture was to be pumped, then flowability could be reduced.

Flowable Mortar in Iowa

Iowa DOT specifications for highway and bridge construction have specified that flowable mortar can be used as backfill material around culvert pipes. The materials, including cement, fly ash, fine aggregate, and admixtures, must meet the following requirements: cement should be type I (ASTM C 150), fly ash should be either Class F or Class C (AASHTO M295), and fine aggregate should be natural sand with the gradation of 100% passing ¾ inch and 0%-10% passing No. 200 sieve.

Flowable mortar is being used by the Iowa DOT for culvert backfill with the following requirements: granular backfill for half the height of the culvert and flowable mortar for a maximum of five feet above the culvert. The granular backfill ensures that the culvert does not float and acts as a filter to keep from plugging the drainage system. The required subgrade treatment must be between the pavement and the flowable mortar.

Forensic Investigation

The pavement surface of Highway 18 (mile post 195), Highway 218 (mile post 211), and Highway 330 (mile post 12) were investigated in this research (see Figure 40). The locations of these sites are listed in Table 17. When driving through these sites, bumps could be felt on the pavement surface above the pipe. Due to limited time, a pavement surface topographic profile was taken only for Highway 18 and was plotted in Figure 41. The investigation revealed bumps across the lanes on the pavement surface. The "bump" shown in Figure 41 is about 0.2 in. deep and it is 20 to 30 ft away from the pipe. Figure 42 shows the pipe location and the pavement surface. Highway 330S was selected for a detailed investigation because it had maximum settlement of 0.5 in. Soil samples were collected for laboratory testing. One-dimensional consolidation tests and consolidated undrained triaxial tests (CU) were performed. For comparison, flowable mortar strength was obtained from Mr. John Vu from the Iowa DOT (Vu, personal communication 2005).

Figure 40. Locations of three investigated sites

Table 17. Locations of investigated sites

No.	Highway	Mile post	County
1	HW18E	195.65	Cerro Gordo (near Mason City)
2	HW18E	195.73	Cerro Gordo (near Mason City)
3	HW18E		Cerro Gordo (near Mason City)
4	HW18E		Cerro Gordo (near Mason City)
5	HW18E		Cerro Gordo (near Mason City)
6	HW218N	211.65	Bremer (Waverly City)
7	HW330S	12.15	Marshall
8	HW330S	12.4	Marshall
9	HW330S	12.9	Marshall
10	HW330S	13.05	Marshall
11	HW330S	13.3	Marshall

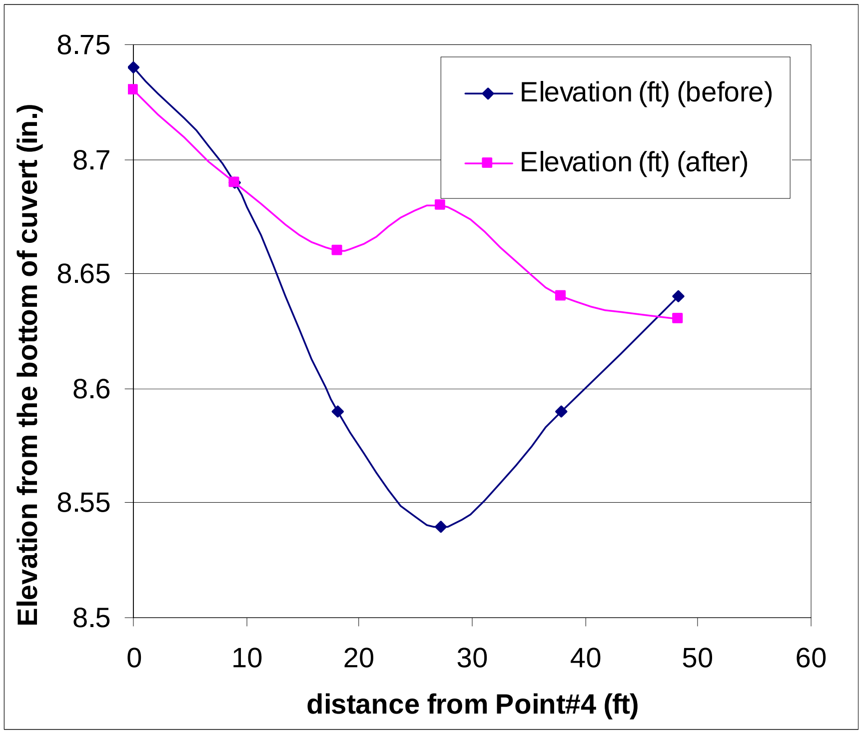


Figure 41. Topographic profile at mile post 195, Highway 18, Iowa

Figure 42. Pavement surface distortion, Highway 18 east (mile post 195), Iowa

Laboratory Testing

Soil parameters that are usually considered in the design of pipe are soil type, soil density, moisture content, modulus of elasticity, coefficient of lateral earth pressure, friction angle, etc. Shelby tube samples were collected from three bore holes of the embankment at Highway 330, mile post 13.30. Borehole 1 is 2.5 meters south of the culvert; samples were collected up to a depth of 13 ft (4 m). Borehole 2 is right above the culvert; samples were collected up to a depth of 7 ft (2.2 m), which is the top of pipe. Borehole 3 is 5 meters north of the culvert and drilled down to 15.4 ft (4.7 m). The boring logs are provided in Table 18; the boring location is schematically shown in Figure 43.

Moisture Content, Compaction, and UCS

It can be seen from Figure 44 that the moisture content is consistent at about 20%-25% of BH1 and BH3, while it is much dryer at BH2 at the range of 13%-18%.

The moisture content at compaction greatly affects the compaction density and unconfined compressive strength of select soils. A 2% moisture content difference can lead to about 50 kg/m^3 compaction density change. The select materials have an averaged unconfined compressive strength of about 400 kPa at optimum moisture content. The difference in strength over 2% moisture content is quite significant (more than 100 kPa). The information confirms the importance of moisture content during compaction. According to the specifications 2102.04 (Iowa DOT Specification 2001), the moisture content limits for select soil is from 2.5% dry of optimum to no upper limit during compaction. Since there is no upper limit during compaction, it is sometimes believed that the contractor may compact the select when it is too wet, which is hard to compact adequately and will result in the low strength of backfill. The stability decreases very quickly once the moisture content exceeds the optimum. The soil is so unstable that it should be obvious that the roller would not walk-out. Because there currently is no upper limit on moisture content for the select soils, the specifications should be changed to ensure proper moisture content during compaction Also, this change would help field staff enforce the discing and drying before compaction. This is essential to obtain good density and stability.

Consolidated-Undrained Triaxial Tests (CU)

For comparison purposes, CU tests were conducted in general accordance to ASTM D 4767-95 "Standard Test Method for Consolidated-Undrained Triaxial Compression Test for Cohesive Soils" (ASTM 1995). Three samples were tested for each bore hole at different confining pressures (25, 45, and 65 kPa for BH1; 15, 30, and 45 kPa for BH2) to access the shear strengths of the soils. The confining pressures are equal to the effective overburden pressure in the field. The specimen was initially fully consolidated to an isotropic confining pressure. A loading rate of 0.05 in. (approximately 1% axial strain) per minute was used in performing the tests. This led to testing times of approximately 20 minutes because the tests were continued to 20% axial strain. Peak deviator stress was used as the failure criterion if the peak was reached at 10% axial strain or less. If the peak did not occur below 10% axial strain, the deviator stress at 10% axial strain was used as the failure criterion. Figures 45 and 46 show the stress-strain and pore water pressure characteristics. During the compression, the pore pressure increased and then decreased to below the original value. It shows that the soils are overconsolidated—contracted (pore pressure increased) at the beginning of compression and then dilated (pore pressure decreased).The strength ranges from 115 to 221 kPa, which is much lower than the averaged unconfined compressive strength (400 kPa) at optimum moisture contents. It is believed that the soil around the pipe was not compacted at optimum moisture content. The stress path of these two bore holes materials is shown in Figures 47 and 48. It is evident from Figures 47 and 48 that, at the confining pressures at which the soils were tested, the soils exhibited both cohesion and friction angle. Confining pressure (σ_{3}), peak deviator stress ($[\sigma_1-\sigma_3]_f$), cohesion (c), and friction angles (ϕ) are summarized in Table 19.

Table 18(a). Boring logs of Highway 330S—borehole 1

Boring Log

Project:	Optimization and Management of Earthwork Materials (Culvert, at HW330S, MP 13.30. 2.5m south of the culvert)	**Boring No.:**	1
Report Date:	6/12/2006	**Start:**	10:00am, 6/12/2006
Location:	Fairfield, Jefferson, IA	**Finish:**	3:00pm, 6/12/2006
Logged By:	Longjie Hong	**Equipment Type:**	Drill Rig
Drilling Crew:	Schaefer, Brian, Longjie	**Sampling Methods:**	1. Shelby Tube-2.8 inch diameter, 2.5ft segment

Depth below surface (in)	Graphical Log	Shelby Tube#	Top Depth, (in)	Length, (in)	Bottom Depth, (in)	Material Description	γt (kN/m^3)	σv (kPa)		σ3 (kPa)	Moisture Content (%)
33								Top	Bottom		
64	░	1	33	31	64	2 triaxial samples, 1 consolidation sample	20.85	17.5	33.9	**25.0**	20.8
86		2	63	23	86	2 triaxial samples, 1 consolidation sample	18.37	29.4	40.1	35.0	21.9
116	▨	3	86	30	116	3 triaxial samples, 1 consolidation sample	19.05	41.6	56.1	**45.0**	22.4
144		4	116	28	144	3 triaxial samples,	19	56.0	69.5	55.0	22.4
158	■	5	114	44	158	2 triaxial samples, 1 consolidation sample	19.57	56.7	78.5	**65.0**	22

Table 18(b). Boring logs of Highway 330S—borehole 2

Boring Log

Project:	Optimization and Management of Earthwork Materials (Culvert, at HW330S, MP 13.30. above the culvert)	**Boring No.:**	2
Report Date:	6/12/2006	**Start:**	10:00am, 6/12/2006
Location:	Fairfield, Jefferson, IA	**Finish:**	3:00pm, 6/12/2006
Logged By:	Longjie Hong	**Equipment Type:**	Drill Rig
Drilling Crew:	Schaefer, Brian, Longjie	**Sampling Methods:**	1. Shelby Tube-2.8 inch diameter, 2.5ft segment

Depth below surface (in)	Graphical Log	Shelby Tube#	Top Depth, (in)	Length, (in)	Bottom Depth, (in)	Material Description	γt (kN/m^3)	σv (kPa)		$\sigma 3$ (kPa)	Moisture Content (%)
24								Top	Bottom		
31	░	1	24	7	31	1 consolidation sample	20.93	12.8	16.5	**15.0**	14
64		2	31	33	64	2 triaxial samples, 1 consolidation sample	20	15.7	32.5	**25.0**	17.2
85	▨	3	64	21	85	3 triaxial samples, 1 consolidation sample	19.96	32.4	43.1	**35.0**	19.2

Table 18(c). Boring logs of Highway 330S—borehole 3

Boring Log			
Project:	Optimization and Management of Earthwork Materials (Culvert, at HW330S, MP 13.30. 5m north of the culvert 175 inches south of station 1861)	**Boring No.:**	3
Report Date:	6/12/2006	**Start:**	10:00am, 6/12/2006
Location:	Fairfield, Jefferson, IA	**Finish:**	3:00pm, 6/12/2006
Logged By:	Longjie Hong	**Equipment Type:**	Drill Rig
Drilling Crew:	Schaefer, Brian, Longjie	**Sampling Methods:**	1. Shelby Tube-2.8 inch diameter, 2.5ft segment

Depth below surface (in)	Graphical Log	Shelby Tube#	Top Depth, (in)	Length, (in)	Bottom Depth, (in)	Material Description	γt (kN/m³)	σv (kPa)		σ3 (kPa)	Moisture Content (%)
24								Top	Bottom		
46	░	1	24	22	46	2 triaxial samples, 2 consolidation sample	20	12.2	23.4	**20.0**	23
56		2	40	16	56	2 triaxial samples, 1 consolidation sample	20.3	20.6	28.9	25.0	22.5
78	▨	3	64	14	78	2 triaxial samples,	20	32.5	39.6	35.0	23.3
114		4	94	20	114	3 triaxial samples, 1 consolidation sample	19.58	46.7	56.7	**50.0**	22.2
146	█	5	120	26	146	4 triaxial samples, 1 consolidation sample	19.53	59.5	72.4	65.0	23.2
185	█	6	160	25	185	4 triaxial samples, 1 consolidation sample	19.66	79.9	92.4	**80**	21.9

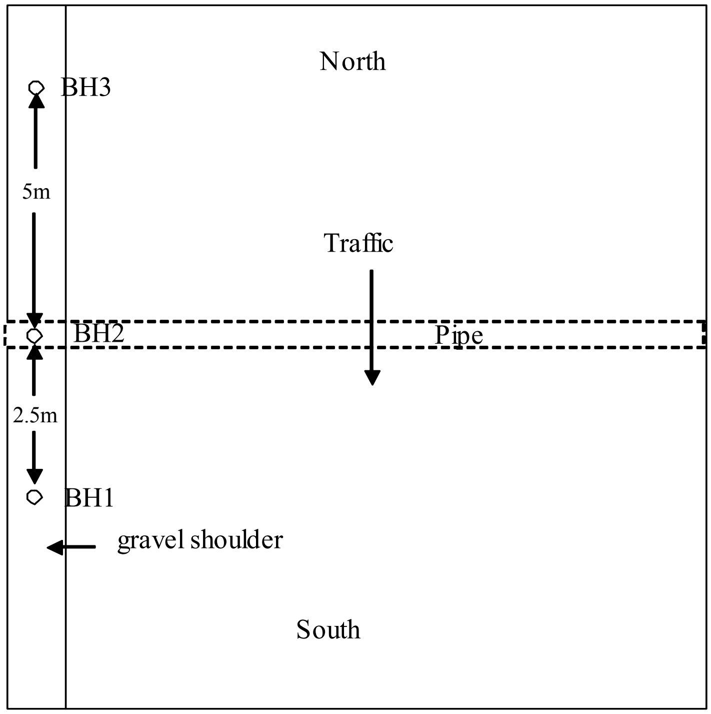


Figure 43. Schematic boring locations on Highway 330S

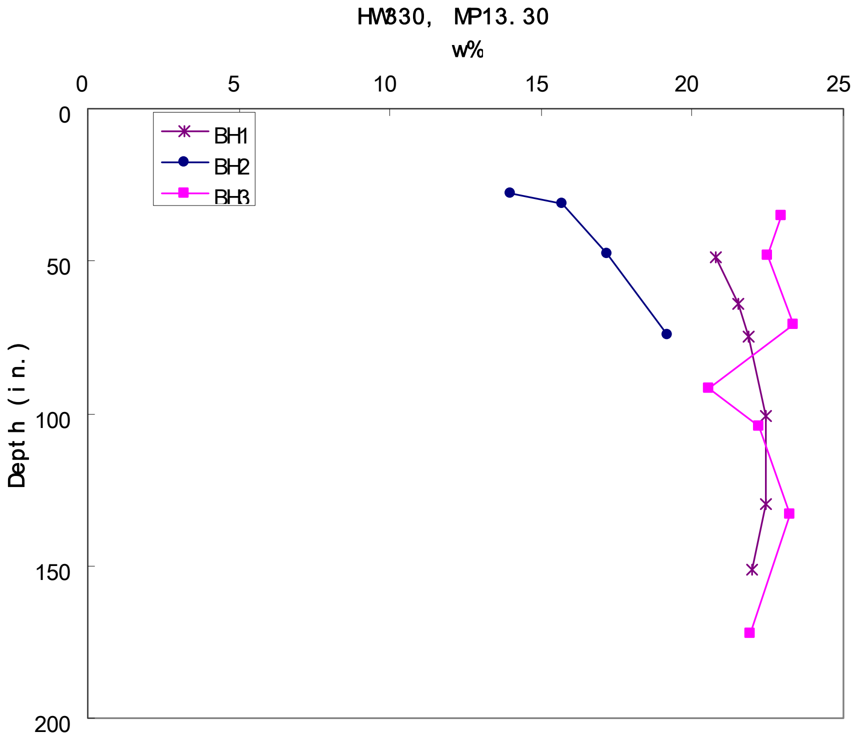


Figure 44. Moisture content profile of the embankment

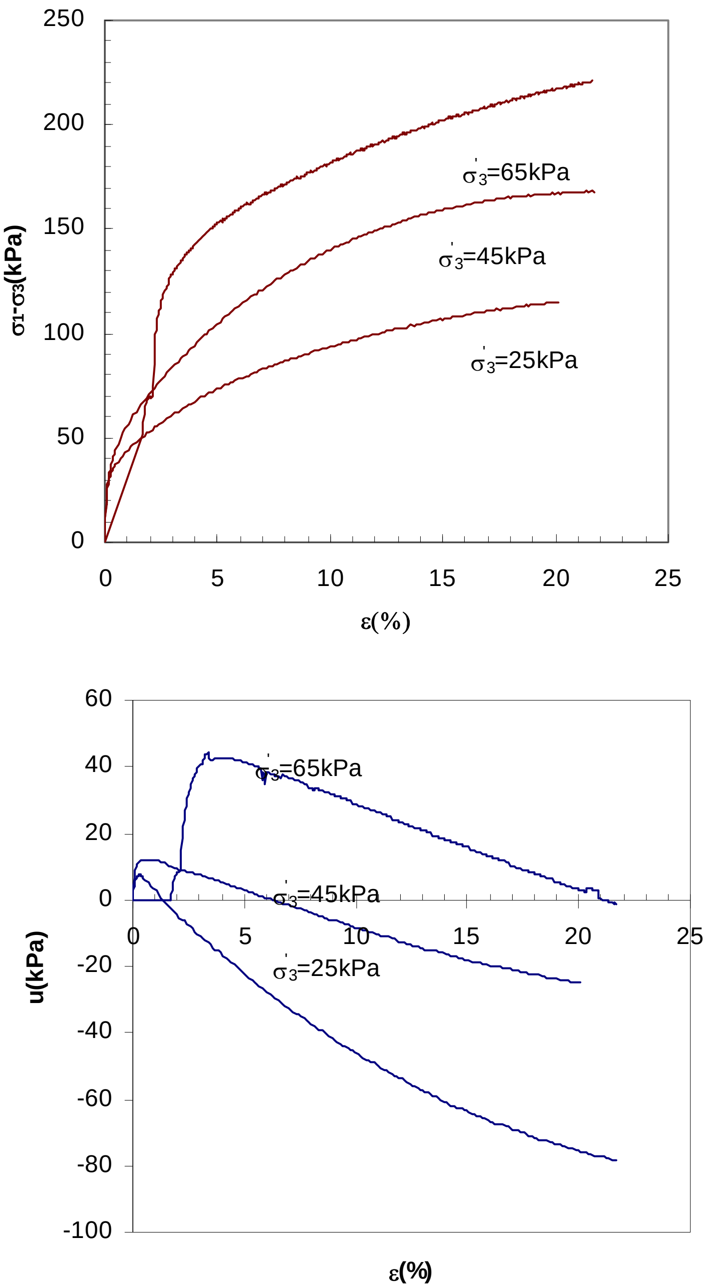


Figure 45. Stress-strain curves and pore pressure curve for BH1

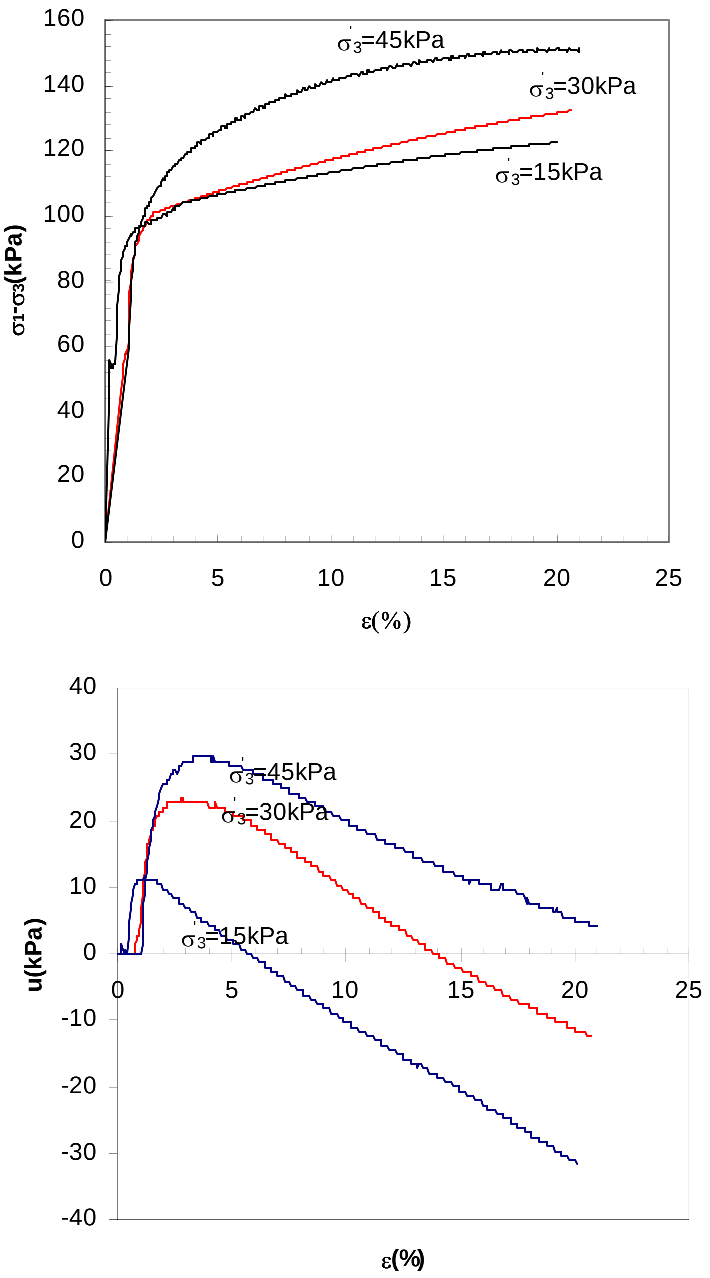


Figure 46. Stress-strain curves and pore pressure curve for BH2

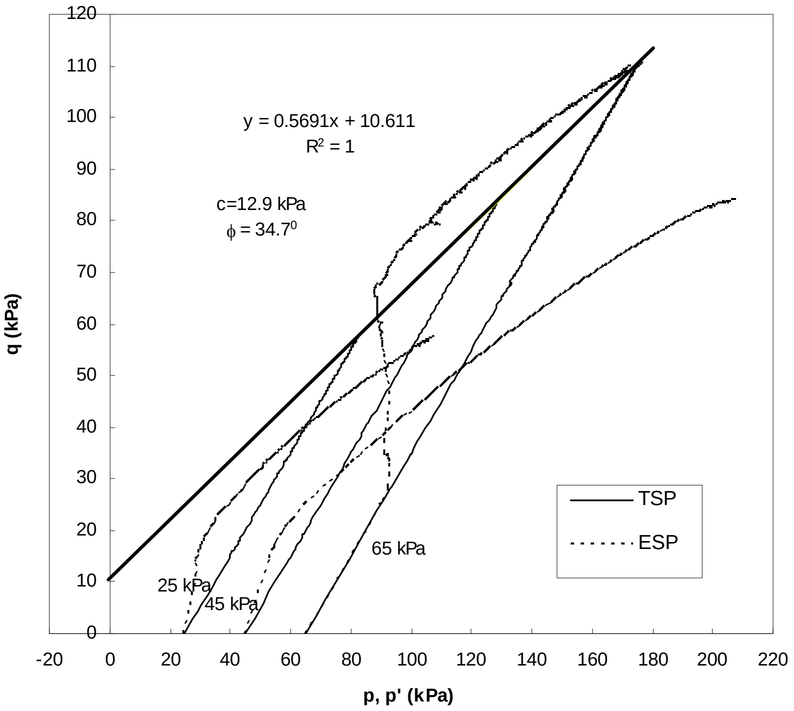


Figure 47. Stress path for BH1

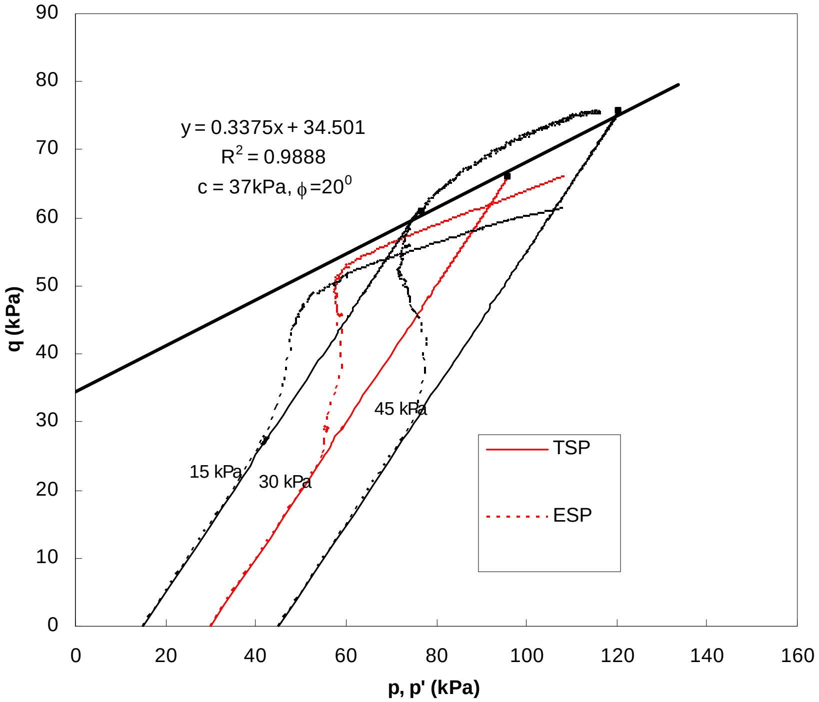


Figure 48. Stress path for BH2

Table 19. CU triaxial tests summary

Bore Hole	σ_3 (kPa)	$(\sigma_1\text{-}\sigma_3)_f$ (kPa)	c (kPa)	ϕ (^{0})
BH1	25	115	13	35
	45	168		
	65	221		
BH2	15	123	37	20
	30	132		
	45	151		

Consolidation Tests

Consolidation tests were conducted according to ASTM D 2435-96 "Standard Test Method for One-Dimensional Consolidation Properties of Soils" (ASTM 1995). The purpose of these tests was to obtain the parameters for settlement analysis. The results are presented in Table 20. From Table 20, it is noted that the soils are normally consolidated or slightly overconsolidated, with an OCR of 1 to 8. The coefficient of consolidation, C_v, ranges from 2 to 13 (x10^{-3} cm^2/sec).

Table 20. Summary of consolidation tests

Boring No.	Shelby Tube Sample No.	Depth (m)	p_c' (kPa)	Compression Index C_c	γt (kN/m^3)	overburden pressure p' (kPa)	Coefficient of Consolidation C_v (x10^{-3} cm^2/sec)	OCR
BH1	ST1	1.23	85	0.14	21	26	3.4	3
	ST2	1.89	60	0.173	18	35	12.9	2
	ST3	2.57	78	0.103	19	49	2.9	2
	ST5	3.45	75	0.127	20	68	3.3	1
BH2	ST1	0.7	110	0.139	21	15	8.3	8
BH3	ST2	1.22	110	0.136	20	25	2.9	4
	ST5	3.38	70	0.147	20	66	2.5	1
	ST6	4.38	88	0.196	20	86	4.7	1

Flowable Mortar Tests

Article 2506.02 (Iowa DOT specification 2001) allows up to 70 gallons of water per cubic yard of flowable mortar. The amount of water in the mix will be an important factor for fluidity and strength of the mix. In order to obtain the compressive strength that would be close to that of the cohesive select soil, a few mix designs were evaluated by John Vu (Pipe backfill report 2005). Samples were cast in the plastic molds with holes at the bottom, and then the molds were set on a layer of granular backfill to determine the compressive strength. The strength for three different mixes is summarized in Table 21.

Table 21. Summary of the flowable mortar strength of different mixes

Age, days	Mix Design 1	Mix Design 2	Mix Design 3
	67 gal. (psi)	45 gal. (psi)	55 gal. (psi)
5	101	140	161
7	107	174	252
14	158	333	388
28	259	723	613

Mix Design 1

This mix design meets Iowa DOT requirement for both critical and noncritical applications with a time of efflux of 11 seconds when 67 gallons of water per cubic yard was used to backfill the pipe from the spring-line to the top of the pipe. There was no problem with the efflux time for this mix. It flowed very well. Figure 49 shows the construction of pipe using flowable mortar as backfill.

Figure 50 shows that the flowable mortar was strong enough after 24 hours to support the inspector's weight. The stability reading measured with the Clegg Hammer showed that the strength was equal to a layer of six inches of crushed stone.

Figure 49. Pouring the flowable mortar

Mix Design 2

This flowable mix design did not flow. The efflux time was not tested, but it was very obvious that it would not flow through the testing cone. It was also noted that the strength was too high. For an open trench application, fluidity is not a problem. However, the strength is now a concern.

Figure 50. Strength after 24 hours

Mix Design 3

In order to determine what the minimum amount of water in the mix should be, another flowable mortar mix with 55 gallons was used. This mix, when tested for the efflux, did not flow at all. As before, flowable mortar cylinders were cast for compressive strength.

The compressive strength for this mix was still too high when compared with the UCS data for the cohesive select soil. The recommendation for the flowable mix design is the current design in the Iowa DOT specifications. There are two main reasons for this recommendation. The first reason is the lower strength, and the second reason is the fluidity and the ability to penetrate into the granular backfill layer. It will solve the "dip" problem and not create a "bump" problem.

Cost

The average cost per location as designed is about $2,100. However, depending on the contractor's operation, the cost may be higher. For example, when the contractor placed the pipe in the cut situation, the contractor cut the trench wide enough to use the pipe bedding machine. If the contractor gets paid for only the plan quantity for granular backfill (GB) and flowable mortar (FM), the contractor may use some of the material that was excavated to backfill the trench. This may lead to a settlement problem.

Numerical Analysis of Pavement Settlement

Settlement analysis was performed using Sigma/W provided by Geo-Slope. (Geo-Slope 2007) The purpose of this analysis was to analyze the amount and shape of settlement on pavement surfaces using different backfill materials, including conventional backfill soils, sand and gravel, and flowable mortar. Comparisons were made based on the analysis.

Input parameters are listed in Table 22. The elastic modulus of soil was obtained by taking the slope of stress strain curves in Figures 47 and 48. The averaged value was about 10 MPa, which was in the range of soft clay, as shown in Table 23. The elastic modulus of gravel (96 MPa) was taken directly from Table 23. The elastic modulus of flowable mortar was obtained from equation (13) (ACI 318-07 2007).

$$E_c = w_c^{1.5} 0.043 \sqrt{f_c'} , \tag{13}$$

where E_c = modulus of elasticity (MPa), w_c = unit weight of flowable mortar (kg/m^3), and f_c' = compressive strength of flowable mortar (MPa).

Table 22. Summary of the materials properties in Sigma/W analysis

Materials	Elastic modulus E (MPa)	unit weight γ (kN/m^3)
Soil	10	20
Flowable mortar	8400	21
Gravel	100	21

Table 23. Typical elastic moduli of soil (EM 1110-1-1904 1990)

	Soil	**Es, MPa (tsf)**
Clay	Very soft clay	0.48-4.8 (5-50)
	Soft clay	4.8-19.2 (50-200)
	Medium clay	19.2-48 (200-500)
	Stiff clay, silty clay	48-96 (500-1000)
	Sandy clay	24-192 (250-2000)
	Clay shale	96-192 (1000-2000)
Sand	Loose sand	9.6-24 (100-250)
	Dense sand	24-96 (250-1000)
	Dense sand and gravel	96-192 (1000-2000)
	Silty sand	24-192 (250-2000)

Based on the tested compressive strength of flowable mortar shown in Table 21, which is 613 psi (4.23 MPa) from flowable mortar mix design 3, and the unit weight of flowable mortar of 130 pcf (2085 kg/m3), the E_c is 8,420 MPa.

From the field investigation, the pipe was buried 7 feet (2.1 m) deep from the surface and was 2 feet (0.6 m) in diameter. The model was set in Sigma/W for three different materials: soil, gravel, and flowable mortar. Based on the Iowa DOT specifications, granular backfill was placed to half the height of the culvert and flowable mortar was set to a maximum of 5 feet (1.5 m) above the culvert.

The trench width was about 6.6 ft (2 m). A soil matrix of 4 m wide was also analyzed in order to see the “dip” problem. From the parametric study, the width of the trench has no effect on the settlement result (see Figure 51[a] and [b]).

The edges of the trench were assumed to be fixed in the x direction and free in the y direction. From 4 m deep below the pavement surface, soils were assumed to be fixed in both directions. Analysis results are shown in Figure 51.

It is noted from Figure 51 that the surface settlement is about 1.4 in. (0.035 m) for soil backfill, while it is only about half (0.018 m) for flowable mortar or gravel backfill (0.02 m). The settlement contour also shows that there is a "bump" on the pavement surface for soil backfill, but that the "bump" does not exist for flowable mortar or gravel backfill.

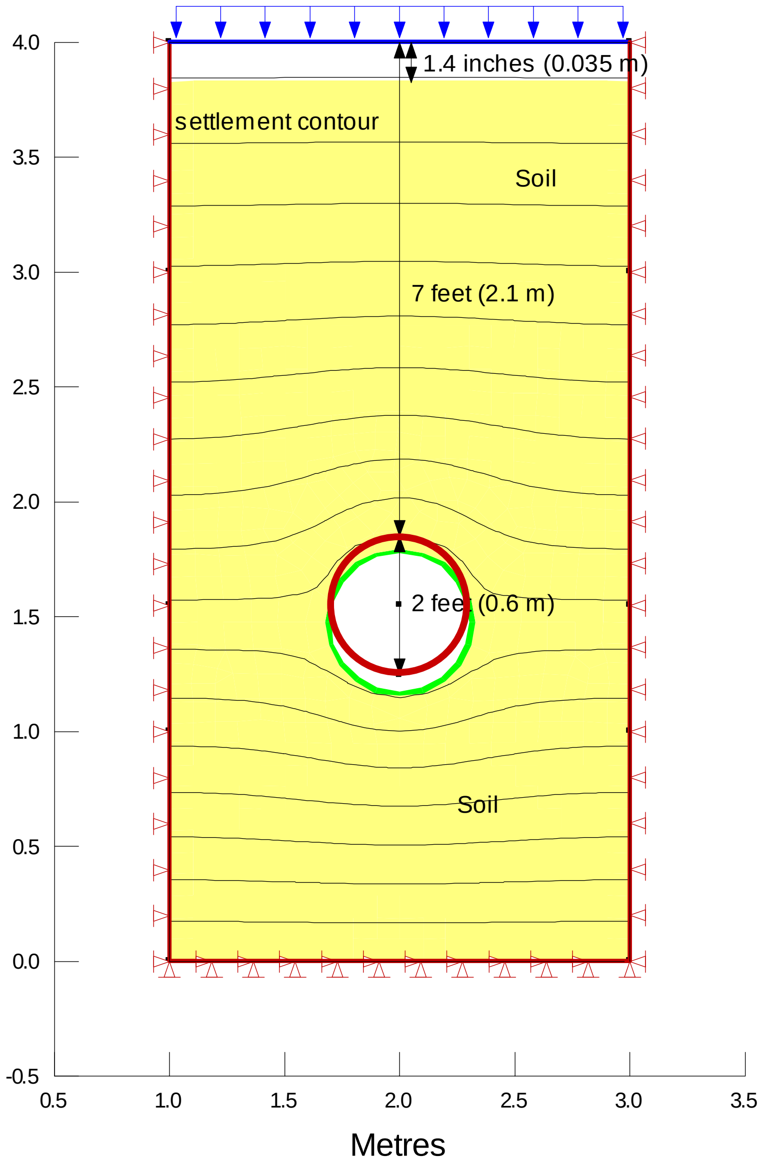


Figure 51(a). Settlement contour of pipe buried in soil (2 m wide trench)

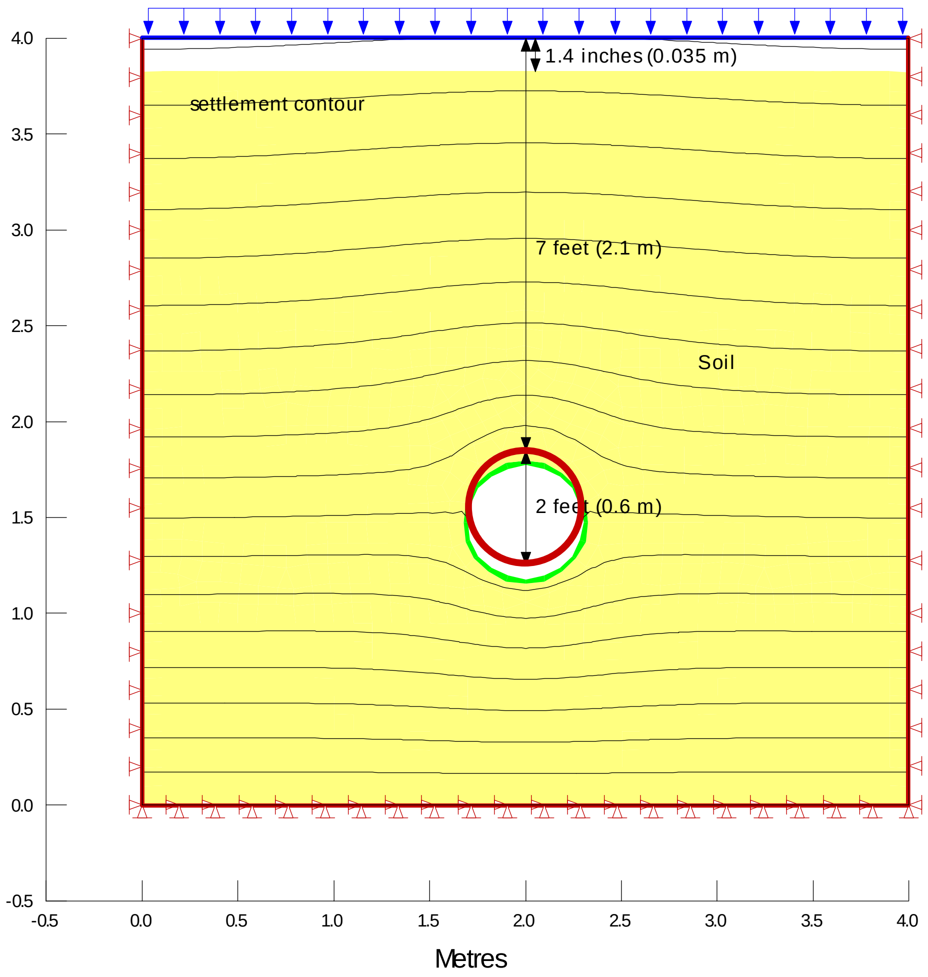


Figure 51(b). Settlement contour of pipe buried in soil (4 m wide trench)

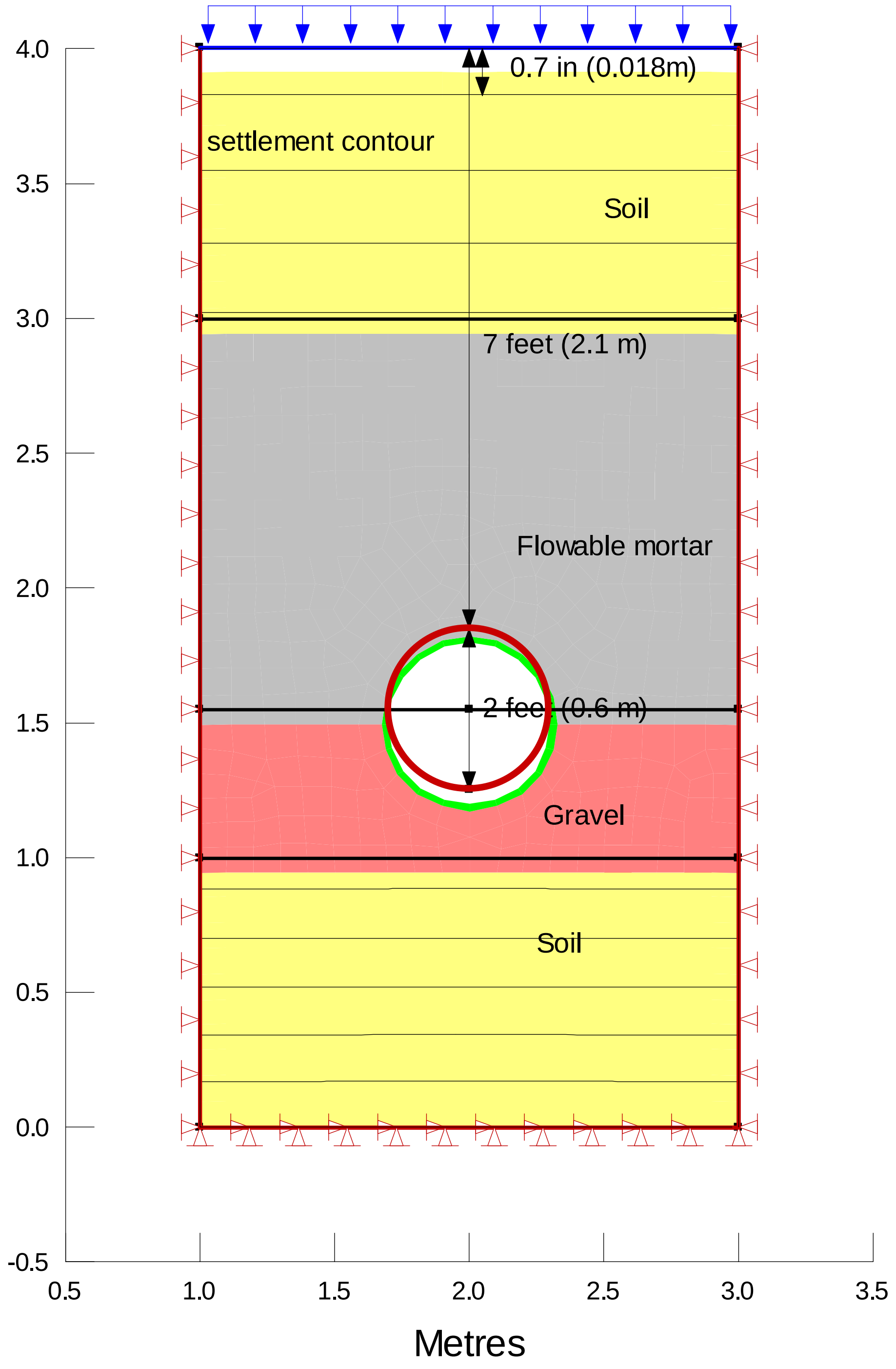


Figure 51(c). Settlement contour of pipe buried in gravel and flowable mortar (2 m wide trench)

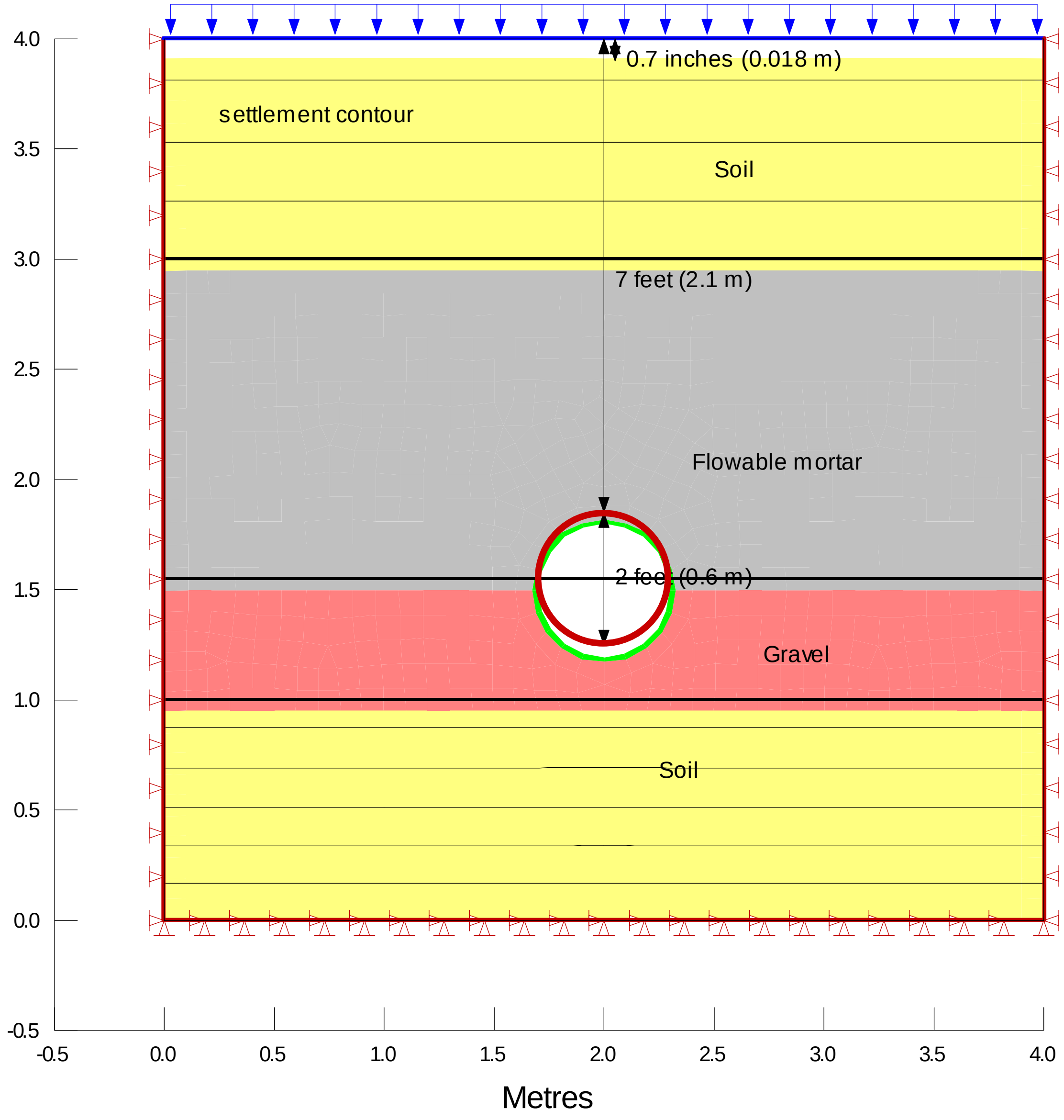


Figure 51(d). Settlement contour of pipe buried in gravel and flowable mortar (4 m wide trench)

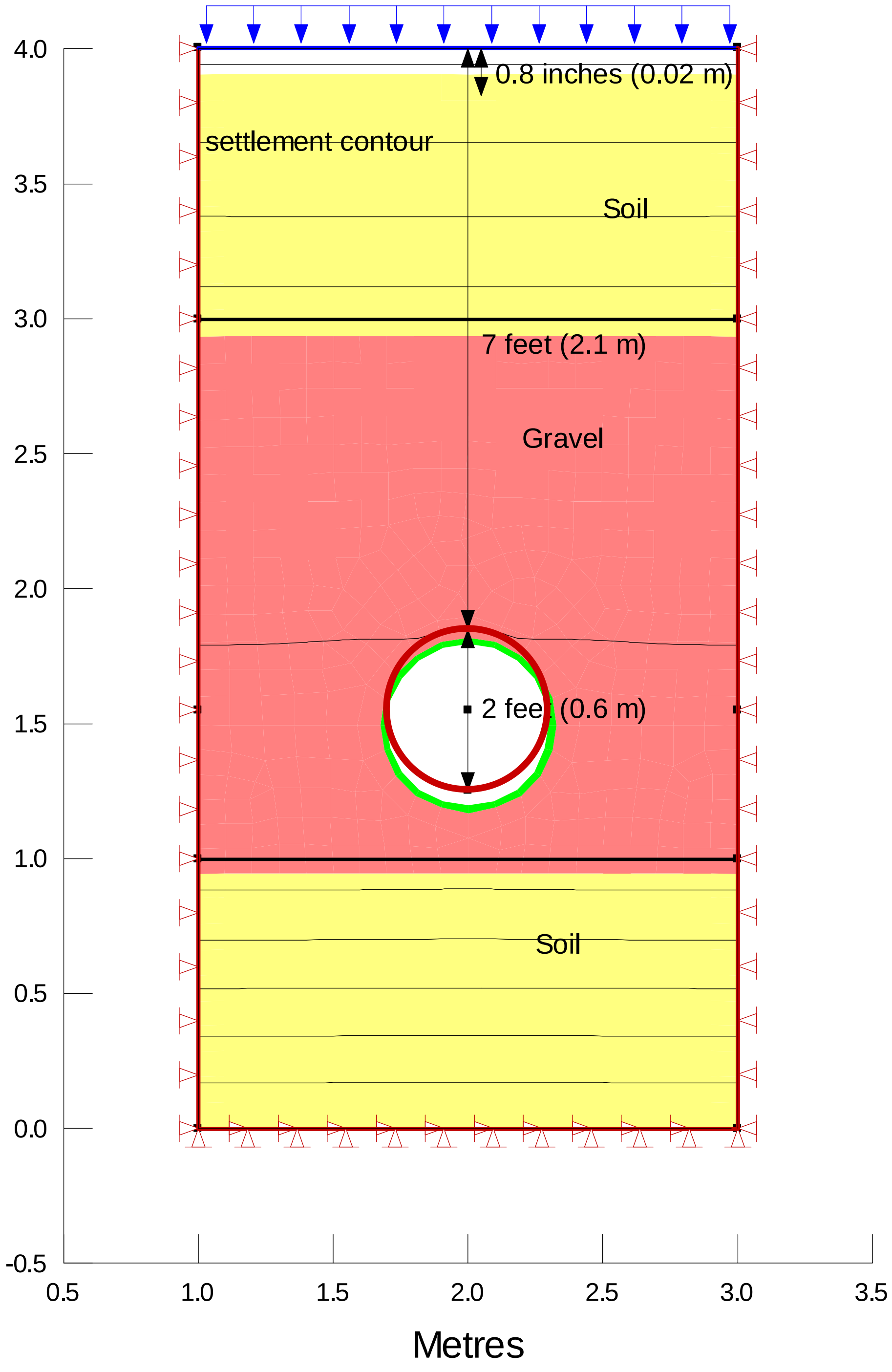


Figure 51(e). Settlement contour of pipe buried in gravel (2 m wide trench)

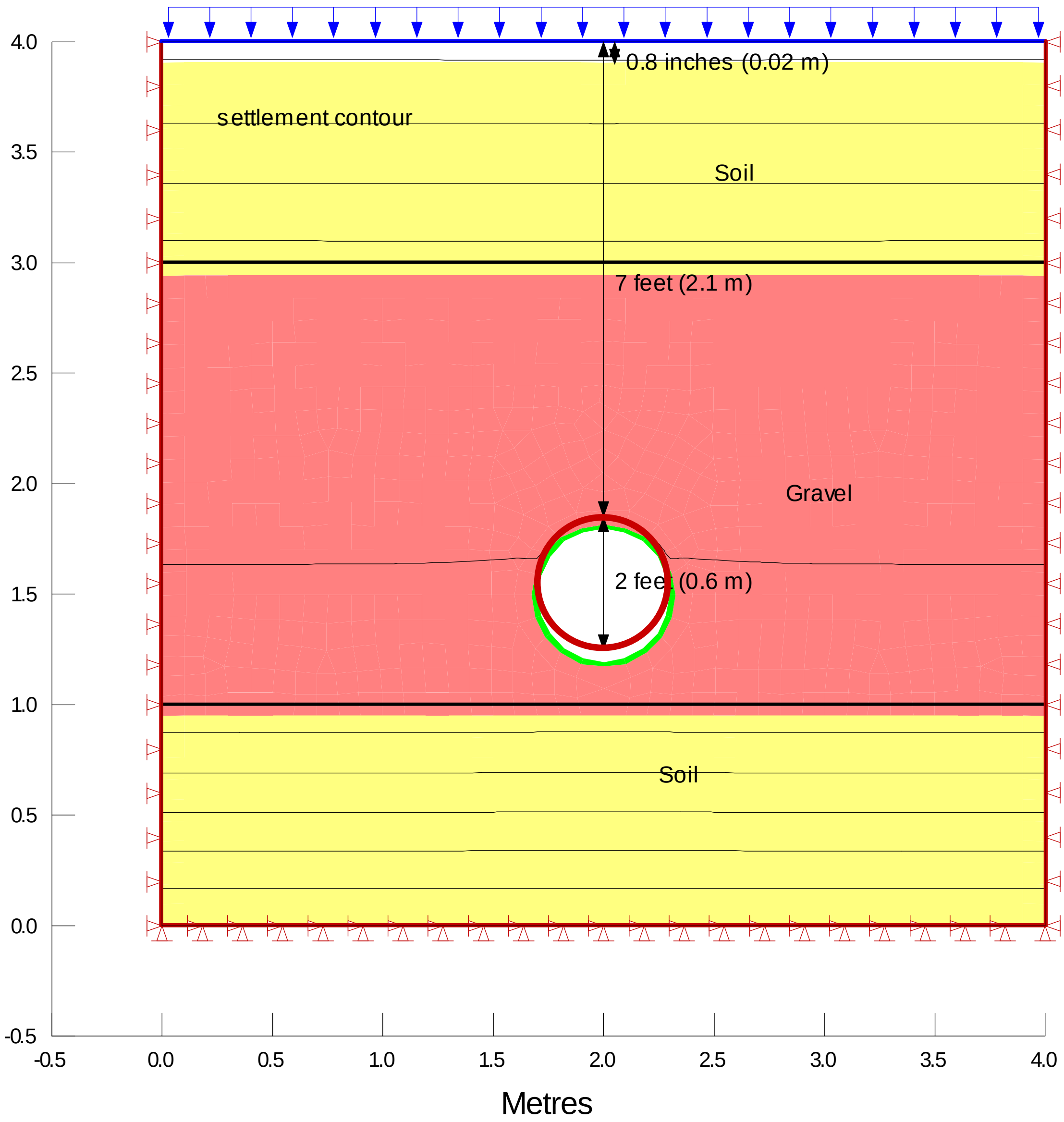


Figure 51(f). Settlement contour of pipe buried in gravel (4 m wide trench)

Conclusions and Recommendations

Conclusions from the study of select backfill materials and flowable mortars are drawn as follows:

1. Moisture content greatly affects the compaction density and strength of select backfill. Improper compaction moisture content can lead to low strength and density, thus causing the dip problem of pipe.
2. If the compaction is done properly, the minimum unconfined compressive strength of the good clay select should be around 700 kPa (100 psi) shortly after compaction. However, this select is not readily available throughout the entire state. The averaged tested strength of select materials is about 400 kPa (58 psi), which may be too low for backfilling purpose.
3. Based on the numerical analysis, the use of flowable mortar as an approach backfill material appears to be a simple and reasonably cost-effective method to reduce the potential for developing the bump above the drainage pipe.
4. Because there is no completed project on flowable mortar backfilling options, there is no information available to evaluate the pavement performance. Thus, it is recommended that a follow-up be done to monitor the performance.
5. Currently, there is no upper limit on moisture content for select; this specification should be reviewed to ensure good density and stability.

GENERAL CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER STUDY

This study was focused on developing information and solutions for managing geotechnical materials through characterization of soil mixing of select and unsuitable soils, an enhanced method for predicting the permeability of compacted soils in Iowa, and the understanding of settlement around culverts. Laboratory testing, field testing, and numerical modeling were conducted to help study the problems and propose solutions. Based on the test results and analysis, the most important conclusions and recommendations drawn from the research are as follows.

General Conclusions

1. Mixing with select soil can improve the index properties of unsuitable soil by reducing liquid limit and plasticity index.
2. Experimental results indicate that maximum dry density and UCS both increase, while the optimum moisture content decreases linearly with increasing select proportion.
3. The UCS of the select-unsuitable blends decreases with increasing moisture content.
4. Select blends can be obtained by mixing up to 25% of an unsuitable soil with 75% select soil.
5. Suitable blends can be obtained by mixing 50% of unsuitable soil with 50% of select soil.
6. At 75% unsuitable with 25% select, some soils will be classified as suitable and some as unsuitable.
7. For the soils tested in this study, the addition of unsuitable soils to a select soil in the range of 25% to 50% provides acceptable soils for placement in embankment applications.
8. Permeability test is reproducible, provided that the variance of test results are between 49% below averaged value and 52% above averaged value. The higher the moisture content, the higher the variance.
9. Air-dried samples have lower permeability than that of wet samples tested right after they are extruded from compaction mold. The permeability of air-dried samples is from about 56% to 95% of that of wet samples. Air-drying has less effect on the results when the samples are compacted dry of optimum than when they are compacted wet of optimum.
10. Permeability of compacted soil decreases with moisture content when it is dry, while the permeability increases slightly with moisture content when the soil gets wetter. The lowest permeability occurs at moisture content of about 2% to 4% wet of optimum.
11. There is a good correlation between $\log[k(1+e)]$ and $\log e$ when the samples are compacted dry of optimum. However, the correlation in the wet side is very poor. When equation (3) applies, the x value ranges from 11 to 22 for overconsolidated clays.
12. A new model of predicting permeability was developed based on the Harrop-Williams (1985) model. The difference between predicted permeability and measured permeability is less than 50%.

13. Moisture content greatly affects the compaction density and strength of select backfill. Improper compaction moisture content can lead to low strength and density, thus causing the dip problem of pipe.
14. If the compaction is done properly, the minimum UCS of the good clay select should be around 700 kPa (100 psi) shortly after compaction. However, this select is not readily available throughout the entire state. The averaged tested strength of select materials is about 400 kPa (58 psi), which may be too low for backfilling purpose.
15. Based on the numerical analysis, the use of flowable mortar as an approach backfill material appears to be a simple and reasonably cost-effective method to reduce the potential for developing the bump above the drainage pipe.

Recommendations for Further Study

1. The Atterberg limits variation of these soils is small, with liquid limit and plasticity index ranging from 25 to 41 and 10 to 21, respectively. As a result, it is difficult to discern how the Atterberg limits change as the percent unsuitable is increased. It is recommended that more unsuitable soils with higher plasticity and more granular select soils be collected and mixed to further study the mixing results.
2. The difference between measured permeability and the calculated permeability using the EICM model suggests that compaction energy of the soil sample is different. The proposed model may be improved by considering the relationship between compaction energy and soil permeability.
3. Due to the lack of a completed project on the flowable mortar backfilling option, there was no information available with which to evaluate the pavement performance. Thus, it is recommended that a follow-up study be conducted monitoring a completed flowable mortar backfilled project.

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