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Mitigation of Expansive Soils in South Dakota

Study SD2014-13
Final Report

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DISCLAIMER

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16. Abstract

Expansive soils can cause considerable damage to pavements. Several localized areas on South Dakota highways have experienced significant heaving along fault gouges caused by differential shifting of discontinuous clay/shale deposits that intersect the road embankment and adjacent drainage features. The South Dakota Department of Transportation initiated this project to update its specifications and construction guidelines to more effectively mitigate the impact of these types of distresses on paved roads. A program that included a comprehensive literature review, laboratory tests on materials collected from six sites in South Dakota, and a benefit/cost analysis was used to meet this objective. Laboratory testing consisted of general soil characterization tests, standard CBR tests, free-swell tests, and large-scale CBR tests. Free-swell data were used to limit the type and application rate of chemical additives used to mitigate expansive soils. Applications of 9 percent lime and 12 percent Class C fly ash were selected for follow-on CBR tests used to evaluate swell and strength. A benefit/cost analysis was run using swell results from the large-scale CBR tests, indicating that 12 percent fly ash provided the most economical value based on a hypothetical construction project. Construction guidelines were outlined to help mitigate the effect and presence of water in and around expansive clay/shale soils and fault gouges, and a step-by-step process was outlined and summarized in a flow chart to guide practitioners on how to adequately identify and treat expansive soils in transportation applications using chemical additives.

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Acronym	Definition
AASHTO	American Association of State Highway Transportation Officials
ACAA	American Coal Ash Association
ASTM	American Society for Testing and Materials
CBR	California Bearing Ratio
CEC	Cation Exchange Capacity
CH	High-Plasticity Clay (Fat Clay)
CL	Low-Plasticity Clay (Lean Clay)
CPV	Cost-Performance Value
FHWA	Federal Highway Administration
GSD	Grain-Size Distribution
GTT	GeoTechTools
IBC	International Building Code
IRI	International Roughness Index
LL	Liquid Limit
LVDT	Linearly Varying Differential Transducer
MRM	Mileage Reference Marker
OMC	Optimum Moisture Content
PI	Plasticity Index
PL	Plastic Limit
ppm	Parts Per Million
SC	Clayey Sand
SD	South Dakota
SDDOT	South Dakota Department of Transportation
UCS	Unconfined Strength
USBR	United States Bureau of Reclamation
USGS	United States Geological Survey
UU	Unconsolidated Undrained

1.0 EXECUTIVE SUMMARY

The condition, composition, depth, moisture content and density of subgrade soils directly influences the design and performance of roadways. Fine-grained subgrades consisting of silts and clays are more susceptible to problems due to their sensitivity to the effects of water on their behavior. Expansive soils generally consist of clay and are most prominent in the central United States, and perhaps most problematic in Texas, Montana, North Dakota, South Dakota, Louisiana and Mississippi. Expansive subgrades create a challenging scenario for road designers and, if not properly addressed, swelling and shrinkage of subgrade soils can create situations that range from troublesome to dangerous, depending on the level of severity.

Coupled with general presence of expansive clays, many highways in South Dakota cross Cretaceous Pierre Shale deposits that experience localized faults and heaves. While the South Dakota Department of Transportation has remediated some of the localized faults and heaves by treating the subgrade with Class C fly ash, additional effort is needed to determine the most appropriate stabilizing methodology to save money and improve the quality of service to road users in South Dakota.

A comprehensive literature search was performed to determine appropriate practices related to the characterization of expansive soils, design and construction of roadways on expansive soils, and the costs and benefits of various stabilization techniques. The review also included an analysis of SDDOT records pertaining to previous use of stabilization agents. The wealth of literature available on this subject was thoroughly reviewed, summarized and synthesized to extract information relevant to the challenges faced by South Dakota road managers.

Six sites were selected by SDDOT for evaluation of expansiveness and appropriate mitigation recommendations. Four sites are on Interstate 90, one on US14/SD34, and one on SD34. Most are within the Upper Cretaceous formation, although Site 2 is in the White River formation. Materials were extracted and sent to the Materials Testing lab at Montana State University for evaluation.

A preliminary testing matrix and screening process was proposed to help determine the additive type and addition rate. Preliminary additives and rates were determined through the literature review and discussions with the project technical panel at SDDOT. Four rounds of testing were employed to evaluate each of the six clay shale soils provided by SDDOT. The first round of testing was used to characterize the general properties of the clay/shale soil samples using a series of standard soil tests (Atterberg Limits, Standard Proctor, California Bearing Ratio (CBR), sulfate, pH, and hydrometer tests).

The second round used the Free-Swell Index test (IS2720, Part 40) to narrow down which additives and rates show the greatest improvement. An analytical approach based on performance data derived from the free-swell tests coupled with costs to implement each mitigation strategy was used to narrow down the most efficient additive and application rates. These results indicated that 6 percent lime was the most effective treatment option, on average, based on cost and swell reduction potential. Free-swell samples treated only with

Class C fly ash tended to cause greater swell than those treated with only lime or a blend of lime and fly ash.

Round 3 consisted of standard soaked CBR tests on each of the six soils that were blended at 9 percent lime and 12 percent Class C fly ash prepared at optimum moisture content and at 2 percent above optimum. The swell results from soaked CBR tests indicated:

- Generally less swell from 9% lime at OMC when compared to the control (with the exception of Sites 1 and 4)
- Always less swell from 9% lime at OMC+2% when compared to the control and 9% lime at OMC
- Generally more swell from 12% fly ash at OMC when compared to the control (with the exception of Site 3)
- Mixed results when comparing swell from 12% fly ash at OMC+2% when compared to the control, but always less swell when compared to 12% fly ash at OMC
- Always less swell from samples prepared at OMC+2% when compared to samples prepared at OMC

The standard CBR strength results from soaked CBR tests indicated:

- Always greater strength from 9% lime at OMC or 12% fly ash when compared to the control
- Strength gains for 9% lime were greater than for 12% fly ash (with the exception of Site 2)
- Generally greater strength from samples prepared at OMC+2% when compared to samples prepared at OMC (with the exception of Site 4 for lime and Sites 3 and 4 for fly ash)

The fourth and final round of testing was used to determine the effect of large particles and mellowing time on the optimal blend/rate combination from Round 3 testing. Large-scale soaked CBR tests were conducted to determine this effect. These tests were designed and conducted to allow for larger particle sizes during the mixing, mellowing and testing of the six clays samples. Three large-scale CBR tests were conducted on each soil: a control (i.e., no treatment), 9 percent lime, and 12 percent Class C fly ash. All samples were prepared at optimum moisture content plus 2 percent based on the results from the standard CBR tests.

The swell results from the large-scale CBR tests indicated:

- Less than a quarter percent of swell or shrink during the mellowing period
- The swell was usually complete after one to three days of soaking
- Always less swell from samples treated with 9% lime when compared to the control
- Generally less swell from samples treated with 12% fly ash when compared to the control (with the exceptions of Site 5 and Site 6, which experienced significantly greater swell than the control)
- Mixed results when comparing swell from coarser particles treated with 9% lime and 12% fly ash when compared to treated finer particles

The large-scale CBR strength results indicated:

- Always greater strength from 9% lime or 12% fly ash when compared to the control
- Strength gains for 9% lime were always greater than for 12% fly ash
- Generally greater strength from coarser particles treated with 9% lime when compared to finer particles treated with 9% lime (with the exceptions of Site 1 and Site 3)
- Mixed results when comparing strength from coarser particles treated with 12% fly ash to finer particles treated with 12% fly ash

Benefit/cost analyses are often used to compare the overall effectiveness of a variety of improvement options. Benefits of chemical stabilization include reduced swell, increased strength, improved ride quality, increased longevity, reduced maintenance, and more. It was understood, based on input from the technical panel, that improvements to the strength of the subgrade were secondary and may not be used to modify the structural design. Therefore, for the purposes of this research, benefits were only quantified in terms of the effectiveness of each mitigation to reduce swell. Based on the test data from the large-scale CBR tests, the benefit/cost analysis indicated that the fly ash treatment at 12 percent is more cost beneficial when compared to 9 percent lime treatment for the soils at Sites 1, 2, 3 and 4. Fly ash treatment of the soils at Sites 5 and 6 caused the soils to swell more than the control resulting in a negative benefit and consequently a negative benefit/cost ratio. In these cases, the 9 percent lime treatment was more cost beneficial.

Construction guidelines and specifications were outlined based on the results of the laboratory testing and the literature review, which included information from other state specifications and guidebooks. Final parameters of interest are outlined, and a flowchart was created to help guide planning, construction, and rehabilitation of highways over expansive soils. The effect and presence of water must be reduced at the fault to adequately address problems associated with expansive soils. The effect of water can be controlled using a soil treatment such as lime or fly ash. The presence of water under and adjacent to the roadway can be controlled using proper drainage (e.g., adequate slope, edge drains, underdrains etc.) and/or by installing a protective barrier to keep surface water from infiltrating into the fault gouge. A step-by-step process was outlined and summarized in a flow chart to guide practitioners on how to adequately identify and treat expansive soils in transportation applications using a chemical additive.

The following recommendations were made based on the results of this effort.

1.1 Recommendation 1 – Adopt the suggested flow chart in Section 9.2 as a step-by-step process to guide practitioners on how to adequately identify and treat expansive soils in transportation applications using lime and Class C fly ash.

The flow chart outlined in Section 9.2 provides a concise and thorough process to identify and predict the swell potential of clay/shale deposits, evaluate swell and strength, determine sulfate content and perform benefit/cost analysis. Following this decision-making process will help practitioners determine the most economical and beneficial treatment to reduce the effects of expansive soils on South Dakota roads.

1.2 Recommendation 2 – Repair highway embankments and right-of-way borrow and drainage areas currently experiencing heaving along fault gouges caused by differential vertical shifting of discontinuous clay/shale deposits using the construction guidelines outlined in Section 9.0.

Heaving of faults in South Dakota is the result of differential vertical shifting of discontinuous clay/shale deposits. The effect and presence of water must be reduced at the fault to adequately address this problem. The effect of water can be controlled using a soil treatment such as lime or Class C fly ash. The presence of water under and adjacent to the roadway can be controlled using proper drainage (e.g., adequate slope, edge drains, etc.) and/or by installing a protective barrier to keep surface water from infiltrating into the fault gouge. Construction guidance is provided to rehabilitate areas where known or anticipated fault gouges are likely to cause damage to highway infrastructure due to expansion.

1.3 Recommendation 3 – Compact subgrades at 2 percent above optimum to reduce the effect of water on expansive soils, based on the information presented in Section 7.3.

It is generally well known that soils compacted wet of optimum will experience less swell than those compacted at or below their optimum moisture content. Test results from standard soaked CBR tests run on treated and untreated samples from South Dakota indicated, in all cases, that samples compacted above the optimum moisture content exhibited less swell than those compacted at optimum moisture content.

2.0 PROBLEM DESCRIPTION

The condition, composition, depth, moisture content and density of subgrade soils directly influences the design and performance of roadways. Fine-grained subgrades consisting of silts and clays are more susceptible to problems due to their sensitivity to the effects of water on their behavior. Expansive soils generally consist of clay and are most prominent in the central United States, and perhaps most problematic in Texas, Montana, North Dakota, South Dakota, Louisiana and Mississippi. The expansive nature of clays can also make the surface of the roadway heave and shove causing permanent damage to the pavement, structures, and other roadside infrastructure. The cost of damage to roadways is estimated to be greater than one billion dollars annually (Christopher et al., 2006). Expansive subgrades create a challenging scenario for road designers and, if not properly addressed, swelling and shrinkage of subgrade soils can create situations that range from troublesome to dangerous.

Coupled with general presence of expansive clays, many highways in South Dakota cross Cretaceous Pierre Shale deposits that experience localized faults and heaves. While the South Dakota Department of Transportation has remediated some of the localized faults and heaves by treating the subgrade with fly ash, additional effort is needed to determine the most appropriate stabilizing methodology to save money and improve the quality of service to road users in South Dakota. Many fault and heave areas require routine maintenance to maintain adequate ride quality. The South Dakota Department of Transportation seeks to update its specifications and construction guidelines to assist design and construction staff in managing these types of soil deposits more efficiently.

3.0 RESEARCH OBJECTIVES

The objectives of this project are listed below and were accomplished as described in the paragraphs that follow each objective. Tactical details related to how each of these objectives will be accomplished are provided in Chapter 4 of this report.

3.1 Determine the effectiveness of common soil stabilizing agents for mitigating localized faults and heaves of clay shale in South Dakota

Commonly used soil stabilizing agents include lime, Portland cement and Class C fly ash. Laboratory tests will be conducted to determine optimal application rates for each of the six soil samples provided by SDDOT using lime and/or fly ash. The results will be used to develop a flowchart for recommended type and application rate of stabilizing agent(s) based on soil properties from localized areas.

3.2 Determine construction guidelines and specifications for using stabilizing agents that include recommended construction methods, addition rates, density and moisture requirements

Based on a thorough review of common practices for constructing paved roadways effected by expansive soils, a clear and concise guideline will be written to outline the procedure for mitigating expansive soils, which will include the quantification of the expansive potential of the soil, applicable mitigations techniques, construction procedures and inspection recommendations. A flow chart will be developed based on laboratory tests to determine optimal mitigation chemicals and associated application rates for expansive soils at various field conditions.

3.3 *Determine the cost-benefit of using stabilizing agents to mitigate localized areas of faults and heaves in clay shale*

A benefit/cost analysis will be conducted using information associated with roadway construction and maintenance of localized areas plagued with faults and heaves from expansive soils. The costs of using stabilizing agents include purchase of the additives and construction time and effort. These costs will be compared to the anticipated benefits of reduced faults and heaves which affect roadway maintenance and reconstruction schedules and serviceability. Benefits that are directly quantifiable include degree of mitigation of expansion and improvement in subgrade strength, by which all other associated benefits are derived.

4.0 TASK DESCRIPTIONS

The objectives of this research were accomplished through a comprehensive literature review, laboratory testing, benefit/cost analysis, and synthesis of recommendations. From these efforts, recommendations for mitigating expansive clay-shale were developed to include construction specifications, optimal additive blend and application rate, and general guidance of the acceptable range in in-situ moisture content of the subgrade. Information was disseminated to the technical panel through detailed and timely quarterly reports. A final report and presentation was delivered to summarize the results of this research. Each of the tasks are detailed in the subsections that follow.

4.1 Meet with project's technical panel to review the project scope and work plan

The Principal Investigator met with the Technical Panel on May 6, 2015 to review the proposed scope and work plan. During this meeting, both parties had the opportunity to meet, discuss, verify and refine the methodology set forth in the proposal to ensure that it meets SDDOT's needs.

4.2 Review and summarize literature pertinent to mitigation of expansive soils with stabilizing agents

A comprehensive literature search was performed to determine appropriate practices related to the characterization of expansive soils, design and construction of roadways on expansive soils, and the costs and benefits of various stabilization techniques. Effort was made to learn as much as possible about the design, construction and mitigation techniques of roads relevant to the soils, climate, traffic and other characteristics of the Upper Great Plains region (South Dakota, North Dakota, Nebraska, eastern Montana, eastern Wyoming and Midwest Canadian Provinces). Best management practices for expansive soils were sought from journals, reports, databases, conference proceedings and other sources. This review documented methods and specifications for use of stabilizing agents with an emphasis on specific construction strategies, application rates, and state of the existing soils. The review also included an analysis of SDDOT records pertaining to previous use of stabilization agents. The wealth of literature available on this subject was thoroughly reviewed, summarized and synthesized to extract information relevant to the challenges faced by South Dakota road managers.

4.3 Develop a testing matrix for six different clay/shale samples supplied by SDDOT, varying stabilizing agent, addition rate, and soil moisture content. The testing should include index tests, compaction tests, California Bearing Ratio (CBR) tests, and possibly other relevant testing deemed necessary by the researcher and approved by the technical panel to determine the optimum soil stabilizer addition rate and soil moisture content.

A preliminary testing matrix and screening process was proposed to help determine the additive type and addition rate. Preliminary additives and rates were determined through the literature review and discussions with maintenance and construction personnel at SDDOT. A procedure by which an optimal material and application rate was established for each of the six clay shale soils provided by SDDOT is outlined in Figure 1. Round 1 testing was used to characterize the general properties of the clay/shale soil samples using a series of standard soil tests. The second round used the Free-Swell Index test (IS2720, Part 40) to determine which additives and rates show the greatest improvement. Sixteen combinations of lime and Class C fly ash were tested on each of the six soils during this round of testing – a total of 96 tests. A screening process using the results of the free-swell tests coupled with basic cost information was employed to determine the optimal level of chemical additive to use during the Round 3 testing. Soaked CBR tests were conducted on each of the six soils that were blended with the best two admixture combinations prepared at optimum moisture content and at 2 percent above optimum. Time for mellowing the mixtures was not used for this round of tests. The final round of testing (Round 4) was used to determine the effect of large particles and mellowing time on the optimal blend/rate combination from Round 3 testing. Large-scale soaked CBR tests were conducted to determine this effect because they allowed for larger particles without being negatively influenced by the smaller volume of the standard CBR tests, yet they were similar enough to standard CBR tests that the results could be directly compared to one another.

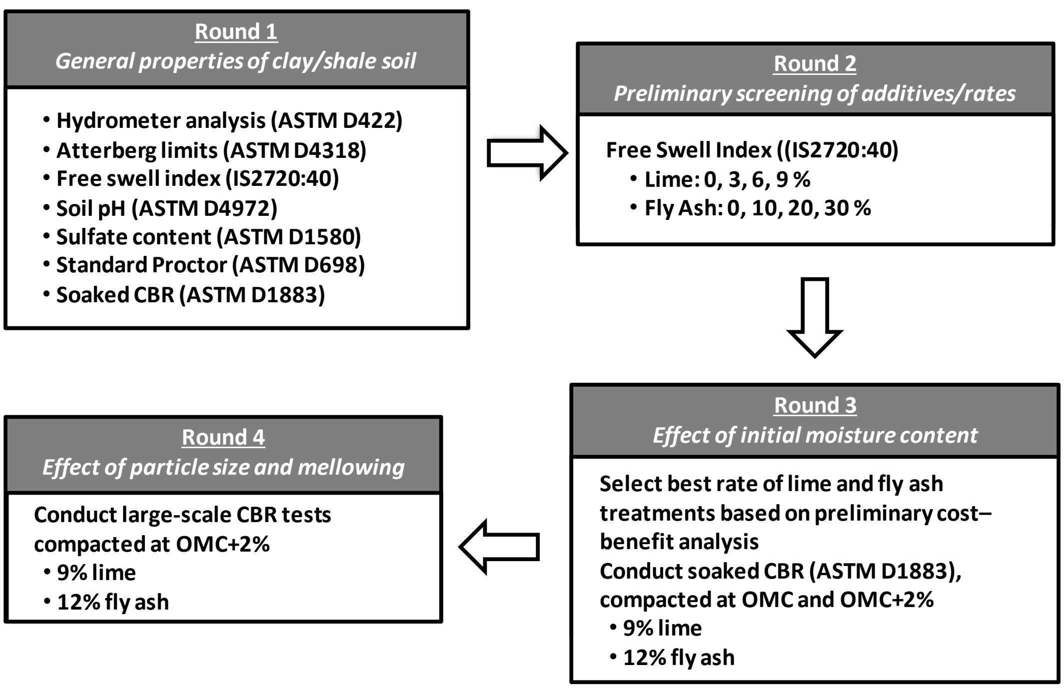

Figure 1: Testing sequence to determine optimal additives and application rate.

4.3.1 Round 1 Testing – General Properties of Clay/Shale Soil

General characterization of each soil sample was done using several standard soil tests, namely, grain-size distribution, Atterberg Limits, Standard Proctor compaction test, and California Bearing Ratio (CBR) test. A hydrometer test (ASTM D422) was conducted to determine the particle size distribution and percent clay content. Knowing the clay content is useful to determine the Activity of the soil, an indicator of the soil's expansion and contraction properties when wetted. Atterberg limit tests were used to determine the liquid limit, plastic limit, and plasticity index of the soils (ASTM D4813). This information was initially used along with the grain-size analysis to classify the soils, as well as to provide a baseline value by which to compare the effects of the various stabilizing agents used to mitigate soil expansion. Standard Proctor tests (ASTM D698) were conducted to determine the optimum moisture content to achieve the maximum density of the soil. This information was used to determine the relationship between water content and density as well as to determine the soil preparation protocol for the CBR tests. Soaked CBR tests (ASTM D1883) were used to characterize the bearing strength of each of the soils as well as determine the swell of the soil. The results of these tests provided a baseline measure of the bearing capacity and swell of the soils at their optimum moisture content. The CBR and plastic limit tests were used as the primary means of evaluating each of the soils under various stabilization treatment strategies. Sulfur content and pH are relevant to lime stabilization of soils and were measured following ASTM C1580 and ASTM D4972, respectively.

4.3.2 Round 2 Testing – Preliminary Screening of Stabilizing Agents/Rates

Due to the quantity of soil samples and the large number of tests that needed to be executed to select the type of stabilizing agent and application rate, an optimization technique was employed to narrow down the type and rate based initially on swell, prior to analyzing effects on strength and plasticity. This was accomplished by first using a procedure to determine the unconfined swelling potential of soils inundated with water (Free-Swell Index, Indian Standard 2720, Part 40). This test was relatively simple to perform allowing many tests to be conducted in a short amount of time to determine an initial range of appropriate application rates and stabilizing agent (or combination of agents). Lime and Class C fly ash were used as the stabilizing agents in these experiments. Portland cement has also been used in this application, but was not the focus of this investigation as suggested by the SDDOT Technical Panel. Various application rates were used for each of these constituents to create a matrix of tests: 0, 3, 6, and 9 percent lime, and 0, 10, 20 and 30 percent Class C fly ash. These rates were established from the literature review and discussions with the project's Technical Panel. A matrix of 16 possible combinations (summarized in Table 1) were used for each of the six soil samples, for a total of 96 tests. By varying each of the components, the effects of each combination were quantified in terms of their ability to mitigate swell in the soils which helped narrow down the type of stabilizing agent (or combination of stabilizing agents) and the application rate.

Table 1: Mix Designs Used to Evaluate Additive Type and Application Rate

Sample	% Lime	% Fly Ash
1	0	0
2	0	10
3	0	20
4	0	30
5	3	0
6	3	10
7	3	20
8	3	30
9	6	0
10	6	10
11	6	20
12	6	30
13	9	0
14	9	10
15	9	20
16	9	30

4.3.3 Round 3 – Effects on Strength and Plasticity

This round of laboratory testing was conducted to determine the effect that lime or fly ash had on the strength and swell potential of each of the six soils at varying rates of initial moisture content. Samples of each soil type were prepared to both optimum moisture content (as determined from the Standard Proctor test) and at 2 percent greater than optimum. Soils were prepared so that the final moisture content was at optimum or 2 percent above after the dry lime and fly ash materials were added. Application rates of 9 percent lime and 12 percent fly ash were used for these tests. Soaked CBR tests were run on all the prepared samples (5 tests per soil type, for a total of 30 tests). The results of these tests were used to determine whether to recommend a greater-than-optimum moisture content during construction.

4.4 Test samples should be prepared and tested following ASTM, AASHTO or SDDOT test procedures, as applicable. Additional test samples should be prepared in a manner that mimics field conditions by keeping the particle size close to field conditions, to determine optimum addition rates for stabilizing agents and compare to standard test methods.

Using the information from Task 3, further evaluation of the soils was conducted to evaluate the effect of soil particle size on soil strength. Strength was evaluated using the soaked CBR test so that the outcomes from this experiment could be compared to previous CBR results. Samples of soil were prepared to mimic grain sizes characteristic of field conditions, particle sizes of less than about 1 in. in size. A large-scale CBR mold setup was designed and constructed to accommodate the larger particle sizes and to facilitate multiple CBR penetration tests on a single sample. Compaction energies were utilized that mimicked previous testing. All samples were prepared to 2 percent above optimum moisture content. The same application rates of 9 percent lime and 12 percent fly ash were used to mitigate swelling. Samples were soaked to monitor swell then tested for CBR strength. Information from this part of the test program were used to determine the effect of particle size on its behavior.

4.5 Recommend the appropriate stabilization agents applicable to South Dakota clay/shales, including addition rates, soil moisture content, density, and any other property necessary to minimize expansive clay-shale faults and heaves.

Recommendations regarding which stabilizing agent (or combination of agents) and application rates were made to SDDOT using the information from the literature review (Task 2), the laboratory testing program (Task 3), and the verification experiments (Task 4). This recommendation was made while taking into consideration the moisture content of the soils during construction, swell potential, sulfur content, and pH of the soil. These recommendations also consider the types of construction techniques, availability and cost of the various constituents, and geographic location of the construction site. The final recommendations were made in consultation with SDDOT personnel to assure applicability of the results to the department.

4.6 Develop construction guidelines and specifications for using stabilizing agents in South Dakota.

Construction guidelines and specifications were developed to describe when mitigation of expansive clay shale should be considered and the construction methods and materials needed for successful mitigation. The results of the laboratory testing and stabilization recommendations were used to develop a flowchart to select the optimum stabilizing additive and its application rate based on the six soil samples provided by SDDOT. Construction guidelines were put together to provide details on the construction process specifically affected by the additional effort required to mitigate the effects of expansive soils in localized areas, collected primarily from the literature review and SDDOT construction personnel.

4.7 Perform a cost-benefit analysis of using expansive soil stabilizing agents to mitigate localized faults and heaves in South Dakota highways. The analysis should include a variety of stabilizing agents, construction equipment and methods, and mitigation of faults and heaves using standard construction methods without a stabilizing agent.

A benefit/cost analysis was conducted to determine the overall effectiveness of lime and fly ash at various application rates to mitigate expansive soils in South Dakota. The decision to use stabilizing agents to mitigate expansive soils was based on whether the additional costs for materials and construction time and effort yield benefits that exceed the costs. Benefits of stabilization may include the following items: increased strength; reduced heaving, and cracking of faults; improved ride quality; increased longevity; increased safety; reduced maintenance (crack sealing, patching); and reduced reconstruction. However, benefits were only quantified in terms of each mitigation technique's effectiveness to mitigate expansive soils and its effect on the design strength of the subgrade because these properties are the only ones that could be directly quantified. Other subsequent benefits such as reduced maintenance, increased life and improved serviceability of the roadway are related to the stabilizing agent's ability to mitigate expansion of the soils.

4.8 In conformance with "Guidelines for Performing Research for the South Dakota Department of Transportation," prepare a final report summarizing the research methodology, findings, conclusions, and recommendations.

This final report was prepared to document the methodology, results and recommendations of the project. This document provides the detailed results of the literature review, laboratory tests, recommendations, guidelines and specifications generated during the course of this project.

4.9 Make an executive presentation to South Dakota Department of Transportation Research Review Board at the conclusion of this project.

The Principal Investigator presented the methodology and findings of the project to the SDDOT Research Review Board. The primary focus of the presentation was the literature review, laboratory tests, recommendations, guidelines and specifications associated with mitigating expansive soils on highway projects in South Dakota.

5.0 LITERATURE REVIEW

Damage from expansive soils is normally caused by water. Subgrade soils beneath roadways can become inundated with water through precipitation, rising ground water tables, or adjacent drainages. For soils that have become wet and have expanded, shrinkage becomes an issue when the soils desiccate. Shrinkage of soils can lead to cracking of the ground or road surface which then increases the opportunity for water infiltration, thereby exacerbating the problem. Another expansive phenomenon that occasionally occurs is the rebound of clay shales from the removal of overburden. Localized fault zones tend to swell and shift when overburden stresses are released and water can more easily infiltrate into the seam or gouge formed by adjoining clay shale formations. If removal and replacement of the problematic soils is not cost-effective, accounting for and controlling the ingress of water into the pavement structure can be accomplished by maintaining proper drainage of the road surface and base course, as well as maintaining the surface of the pavement through regularly-scheduled crack sealing, chip sealing, or surface restoration efforts. Another common method to control the effect of water on expansive soils is to use chemical stabilization such as lime, Portland cement, fly ash (most often Class C), or some combination of these constituents during construction of the roadway. Identification and mitigation are vital to properly managing the potentially costly effects that expansive soil can have on transportation infrastructure.

5.1 Expansive Soils

Clay mineralogy dictates the potential for clays to shrink and swell under various moisture levels. Although there are many types of expansive clays, most consist of kaolinite, illite, and smectite, with montmorillonite being a very common and important member of the smectites. The size of the clay particles is directly related to its sensitivity to water, with larger clay particles being less sensitive to water. Kaolinite has the largest clay mineral crystal, lowest specific surface area, and typically low activity. Activity is defined as the plasticity index of the soil divided by the percent of clay-sized particles (soil particles less than 2 micrometers). An activity higher than 1.25 indicates that a soil is susceptible to larger changes in volume when wetted and dried. Montmorillonites have the smallest clay mineral crystal, greatest specific surface, and highest activity. Montmorillonites are generally considered the most susceptible to swelling as their water content increases. The swelling potential of illites varies from inactive to active (Holtz et al., 2011, Nelson and Miller, 1992). Expansive clays analyzed within the Pierre shale region near central South Dakota predominantly consisted of mixed layer illite/smectite, smectite, illite and quartz minerals based on x-ray analysis tests conducted by the U.S. Geological Survey (Collins et al., 1988).

Clay particles contain naturally occurring cations, such as Na^{+}, Mg^{2+}, Ca^{2+}, which attract interlayer water and pore water to neutralize these positive ions. Clays with predominantly monovalent adsorbed cations (e.g., Na^{+}) have a much higher swelling potential than clays with divalent cations (e.g., Ca^{2+}) because half as many divalent cations are needed to balance the negatively charged clay surface, thus resulting in a smaller clay mineral–micelle structure with less adsorbed water. Besides having small particle size, montmorillonites also have a very strong attraction to water, which is used to balance the inherent charge deficiency of the clay crystal, and thus have an affinity for great increases in water content, the result of which is volumetric expansion with significant enough uplift forces to damage structures, including pavements (Holtz et al., 2011; ARA, 2004). Figure 2 shows a comparison of the relative volume of water absorbed by a kaolinite and montmorillonite crystal compared to the crystal size. According to Nichols et al. (1986), most of the Pierre Shale studied west of Pierre, South Dakota near Hayes contained 50 to 100 percent clay minerals of which 60 to 100 percent consisted of mixed layer illitic smectites or pure smectites. For sites near Pierre, South Dakota, Collins et al. (1988) showed similar results. X-ray diffraction testing indicated that the clay/shales from samples removed from ditches near faulted areas were illitic smectites that contain calcium and/or magnesium cations or sodium and/or potassium cations. Calcium and magnesium are divalent ions that are not as expansive as monovalent ions such as sodium or potassium. Addition of calcium-rich minerals such as gypsum ($CaSO_4 \cdot 2H_2O$) or lime (CaO) to these types of clays can significantly reduce their swelling potential. Class C fly ash contains about one-third of the CaO content compared to traditional Portland cement (ACAA, 2003).

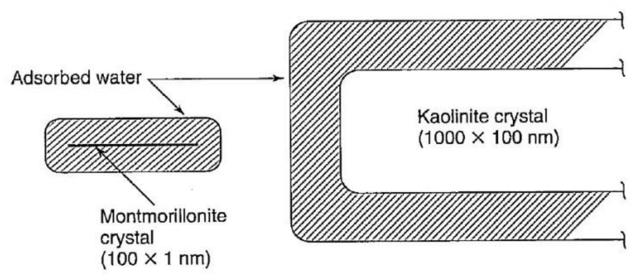

Figure 2: Relative sizes of absorbed water layers on sodium montmorillonite and sodium kaolinite (Holtz et al., 2011)

Naturally existing faults and/or seams in clay and shale deposits at times intersect the ground surface. These fissures allow water to infiltrate at the ground surface which causes the expansive clay to expand and move. This movement causes additional cracks and fissures which allow more water infiltration and increased expansion and movement over time. This repeated swelling concentrated near the fault opening causes heaving of material and, at times, damming and/or holding water in small temporary pools sometimes referred to as gilgais (Wikipedia, 2016a). The alternate swelling and shrinking, wetting and drying of soils near the faulted areas causes the soil to break down or mulch itself creating a material sometimes referred to as vertisol (Wikipedia, 2016b). This breakdown of the particles increases surface area and exacerbates the problem. Vertisol are subdivided into groups based on how long cracks formed by faulted surfaces are open and their global location. Vertisols in South Dakota are known as Torrerts. Torrert vertisols have cracks that are closed for less than 60 consecutive days for soil temperatures within the top 18 inches above 46 degrees F.

5.2 Laboratory Tests for Identifying and Quantifying Expansive Soil Characteristics

While clay mineralogy explains the mechanism of soil expansion, identifying the dominant type and amount of clay mineral crystals in a soil sample (using electron microscopy, X-ray diffraction, hydrometer analysis) has not been adequately correlated to the volume of expansion that is likely to occur. Thus, while clay type may help identify the potential for problems, other tests are needed to adequately identify and mitigate expansive soil (Snethen et al., 1977). In general, swell potential increases as the cation exchange capacity (CEC) of a clay increases. While not common in geotechnical laboratories, tests for CEC are routine in many agricultural soils laboratories and are inexpensive (Nelson et al., 2015). Snethen et al. (1977) identified many methods to predict or indicate the swell and/or shrinkage potential in soils in his review for the FHWA. This review found that LL, PI and soil suction at natural water content (τ_{nat}) are the recommended tests for expedient identification of potentially expansive soils (Table 2). Soil suction is generally not needed to identify these types of soils; however, both the accuracy and conservatism of the classification of potential swell can be improved by including this test. Soil suction can be measured with a thermocouple psychrometer (AASHTO T 273).

Table 2: Potential Expansiveness of a Soil (AASHTO T 258 and Snethen et al., 1977)

Degree of Expansion	LL	PI	τ_{nat} (ton/ft^2)
High	>60	>35	>4
Marginal	50–60	25–35	1.5–4
Low	<50	<25	<1.5

According to Section 1802.3.2 of the 2003 International Building Code (IBC), expansive soil can be identified using its plasticity characteristics. The IBC indicates that soils having the following four characteristics are considered expansive:

1. plasticity indices greater than 15
2. greater than 10 percent of the soil particles pass a No. 200 sieve
3. greater than 10 percent of the soil particles are less than 5 micrometers in size
4. expansion indices are greater than 20 (ASTM D4829 – Standard Test Method for Expansion Index of Soils)

Several standardized tests exist to quantify expansion or swell potential in soils – each focusing on a particular aspect of the behavior of these materials when inundated with water. Many tests use a consolidometer to measure vertical volume changes while loading and unloading soil samples that may be soaked or unsoaked. These tests can quantify swelling pressures in addition to changes in volume. The free-swell test simply indicates swelling of dry pulverized clay inundated with water.

The overburden swell test (AASHTO T258) requires an undisturbed soil sample be placed in a consolidometer and kept moist while a load equal to the sample's field overburden pressure for the sample is applied and the soil is allowed to readjust. The sample is then inundated with water and allowed to reach equilibrium before incrementally unloading the sample, both of which cause the sample to swell. At this point a conventional consolidation–rebound test is performed.

The expansion index test (ASTM D4829) requires soil be compacted in a consolidometer at 50 percent saturation and monitored for vertical deformation after inundating with water. Thus, the test does not mimic field density, water content, soil structure, or soil–water chemistry.

The one-dimensional swell test (ASTM D4546) can also be performed in a consolidometer using either reconstituted or undisturbed soil samples. Expected vertical stress or in-situ vertical stress is applied to the samples before inundating with water and monitoring vertical deformation, which causes the soil to swell, collapse, slightly swell and then collapse, or slightly collapse and then swell.

The soaked CBR test (ASTM D1883) also provides some indication of swelling by allowing the compacted soil access to water through perforated metal plates at the top and bottom of the sample. Weights are used to provide overburden to the sample during the soaking period. The soaking time is 96 hours (4 days) and vertical expansion is measured with a dial gage in contact with the top of the soil. Swell is calculated as a percentage of the change in sample height compared to its initial height.

The free-swell test was developed by the US Bureau of Reclamation (USBR) and was originally conducted by measuring the final volume of 10 cm^3 of dry clay (minus No. 40 sieve) put into a 100 cm^3 graduated cylinder filled with water. While it is no longer a standard USBR test, it is currently standardized by the Bureau of Indian Standards as IS:2720 Part 40, with several improvements. The test now uses 10 g of oven dry minus No. 40 clay, and also kerosene as a reference liquid, as its nonpolar nature merely "wets" the clay and does not cause swelling. The free-swell index, expressed as a percentage, is the difference in final swell volumes in water versus kerosene, referenced to the volume in kerosene. Highly swelling bentonites can have a free-swell index above 1200 percent. Soils with swell indices greater than 100 percent may damage pavements and light structures, whereas swell indices less than 50 percent are generally not considered expansive soils (Holtz et al., 2011).

5.3 Expansive Soil in South Dakota

The geology, geography and climate varies across South Dakota, but is generally similar within three regions associated with the eastern, central, and western portions of the state. The eastern portion of the state features lower elevations between about 600 and 2000 feet, the central portion has elevations approaching 3000 feet, and the elevations of the western side exceed 4500 feet (Geology.com, 2009). Most of the state receives between 10 and 20 inches of precipitation annually, although the eastern portion generally experiences slightly more (up to 25 inches) and the region of the Black Hills exceeds 30 inches (National Atlas, 2005). In relative terms, the northeastern portion of the state is cool and moist and the southeastern portion is warm and moist, whereas the western side is drier such that the northwest is cool and dry and the southwest is warm and dry – with the exception of the Black Hills (Tallman, 2009).

Geologically, portions of South Dakota east of the Missouri river drainage are dominated by late Wisconsin glacial sediments. These materials, which are rich in glacial outwash, provide abundant sand and gravel deposits useful for road building (Tallman, 2009). West of the Missouri river drainage, however, sedimentary sandstone, limestone, silt, clay, and shale are more common (Figure 3). A map of the contiguous United States, widely referred to in several reports about expansive clay soils, shows much of South Dakota being underlain by soils with expansive clays (Figure 4); however, the Upper Cretaceous shales and clays in central South Dakota are the focus of this research.

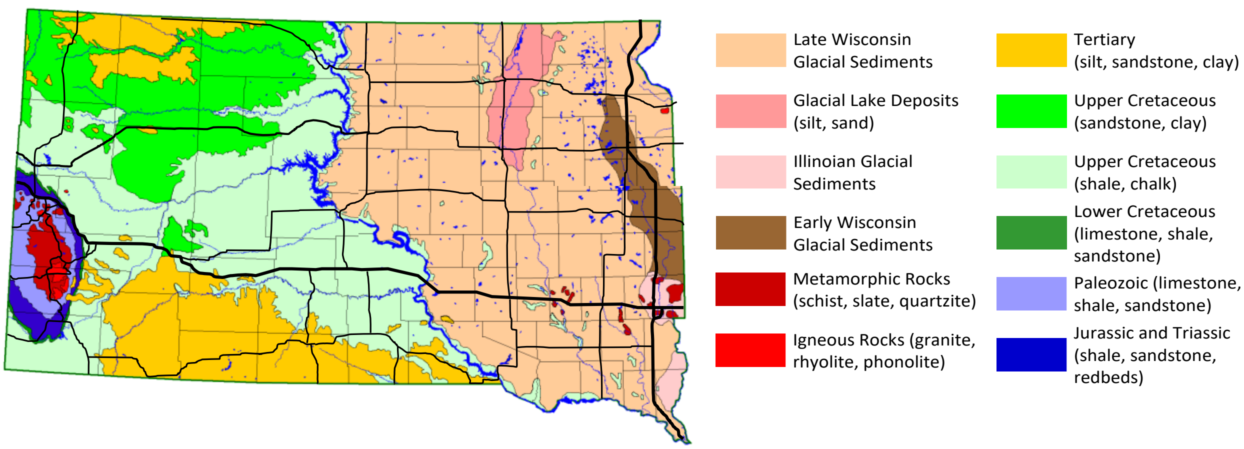

Figure 3: Geology of South Dakota with counties, U.S. Interstate and U.S. Highway system (geology information from South Dakota Geological Survey, 2009).

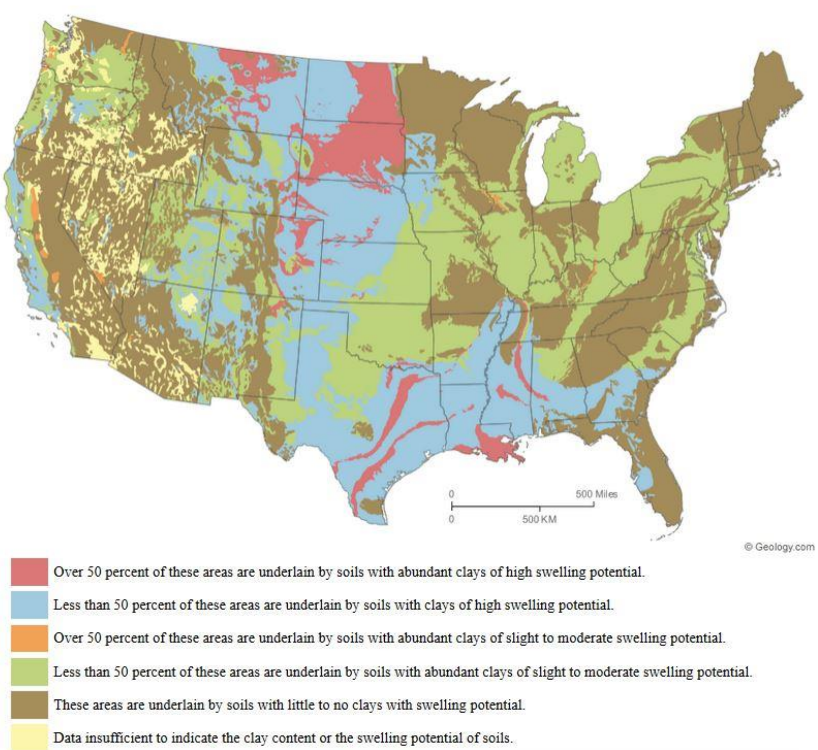

Figure 4: Location of swelling clays in the United States (Geology.com, 2014)

The Upper Cretaceaous geologic rock series formed during the Late Cretaceous period thought to be 100.5 to 65 million years ago when the ancient Western Interior Seaway covered South Dakota and many other North American interior states and provinces, dividing the continent into eastern and western halves (Figure 5). Pierre Shale is a geologic formation of overconsolidated smectite shale in the Upper Cretaceous of marine origin, deposited by the Western Interior Seaway (Wikipedia, 2017 and Nichols, 1992). Stratifications of thin layers of bentonite are common in the predominantly mixed layer illite–smectite clay (Underwood et al., 1964 and Nelson et al., 2015). Extensive field exploration and laboratory tests on outcropping Pierre Shale in South Dakota and Colorado by USGS indicates in-situ time-dependent weathering extends to about 75 ft. below ground surface. Compared to the unweathered parent Pierre Shale 75–600 ft. below ground, the weathered Pierre Shale has lower density, greater pore volume, higher water content, and lower saturation percentage. The slow natural weathering leads to geologic rebound, defined by Nichols as "the expansive recovery of surficial crustal material, either instantaneous, or over time, or both, and is initiated by the removal or relaxation of superincumbent loads" (Nichols, 1992) The weathering process of Pierre Shale described by Bjerrum is: "strain energy is locked in the shale by diagenetic clay bonds, which, upon disintegration, allow the relief of the energy, causing expansion of the fabric" (Bjerrum, 1967). Due to high lateral confinement, the expansion is typically vertical and causes an increase in voids (pore volume) and, thus the weathered soil has a lower density, as illustrated in results from porosity tests conducted by Nichols (1992) shown in Figure 6. Generally, more water is available near the surface, and the weathered soil has a higher gravimetric water content, but because of such a large increase in void volume, the surface soil is rarely saturated whereas the deep unweathered soil is usually saturated. Two compelling reasons suggest geologic rebound of Pierre Shale: 1) the increased void volume (expansion) "nearly always appears to precede the inflow of new pore water," and, 2) clay chemistry tests indicate predominantly divalent (e.g., Ca^{2+}) exchange ions, which have a lower swelling potential than monovalent exchange ions (e.g., Na^{+}).

Figure 5: Ancient Western Interior Seaway (from Wikipedia, 2017).

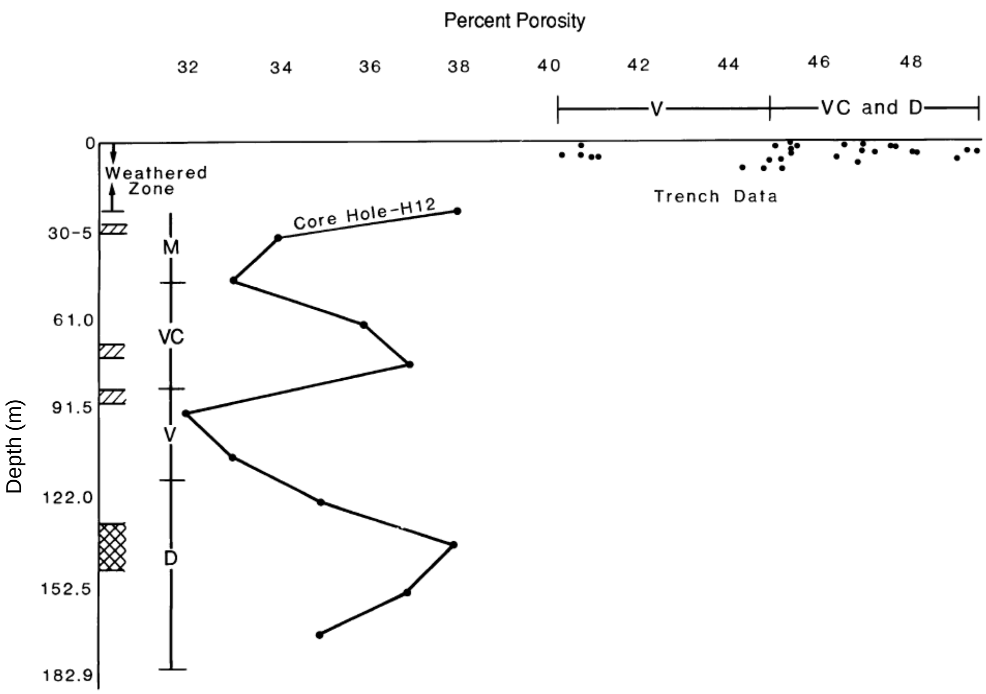

Figure 6: Porosity of Pierre Shale with depth (m) shows higher porosity in the weathered zone (from Nichols, 1992).

Significant overburden removal (up to 24,000 psf, or about 200 ft.) of the overconsolidated Pierre shale during construction of the Oahe Dam near Pierre, South Dakota was expected to cause rebound. However, significant differential rebound occurred along pre-existing fault surfaces, with the most significant being a 1.1 ft. abrupt (likely within a few hours) rebound, occurring near the time of completion of the excavation period (Figure 7). Instrumentation was almost immediately installed to monitor additional movement. Movement continued at slower, diminishing rates after the initial abrupt movement, and mostly within the upper few feet of shale. Laboratory tests on the soil (Atterberg limits and swell tests) indicate the Pierre shale is susceptible to swelling due to changes in moisture content. However, the permeability is so low that a long period of time and abundance of water would be needed for appreciable swelling, and swell with seepage water or "shale juice" was substantially lower than distilled water. Thus, swelling is not believed to be a factor in the initial abrupt movement, but could have contributed to the continued movement in the upper few feet, since water was usually available in the faults and joints. The unexpected differential rebound required an emergency redesign of the structure, which utilized a significant number of anchors installed at various angles to "sew" the faults (Underwood et al., 1964).

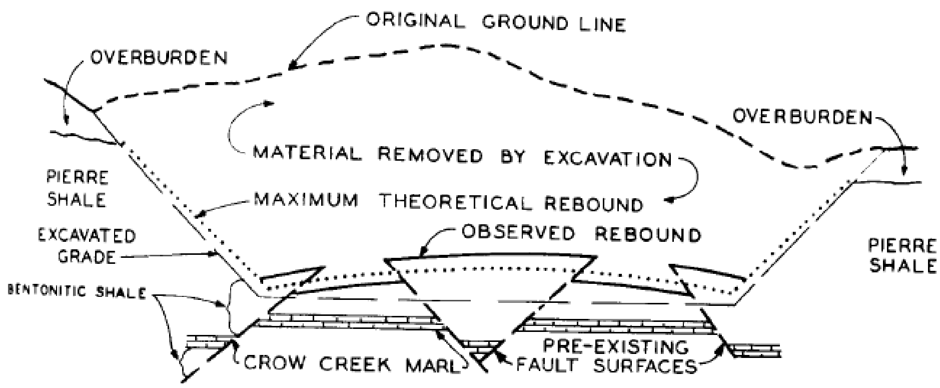

Figure 7: Cross section of excavation with differences in expected (theoretical) rebound and actual observed rebound (from Underwood et al., 1964).

Nelson et al. (2015) present several significant geologic profiles around the world that have contributed to structure damage from expansive soils. Two case studies presented involve Pierre Shale in Colorado (Figure 8 and Figure 9) with low initial water contents (typically near the PL) and high values of dry density, which contribute to high expansion potential, even if the swelling pressures are not particularly high. Notably, the presence of dips, coal seams, and other features cause significant variation in the behavior of the bedrock at different locations.

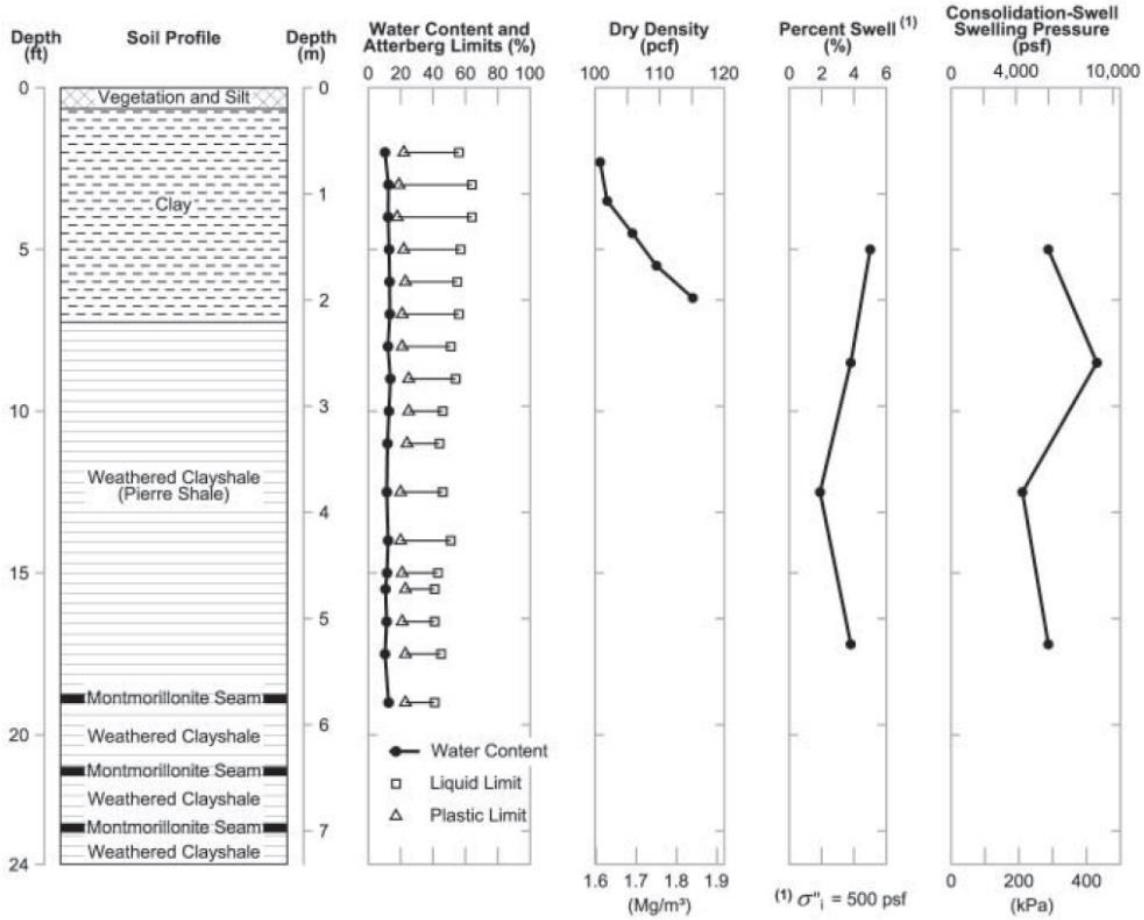

Figure 8: Profile of expansive soils near Fort Collins, CO (from Nelson et al., 2015).

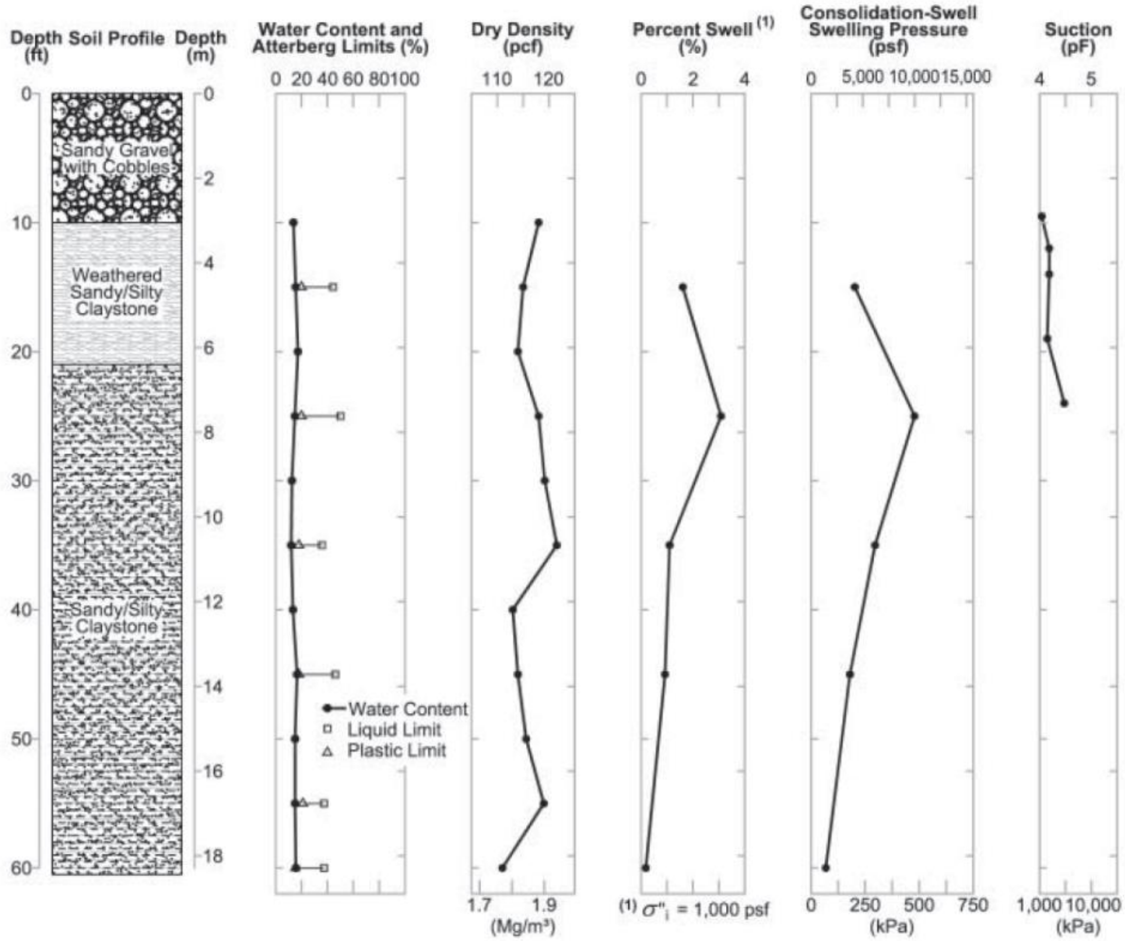

Figure 9: Profile of expansive soils near Canon City, CO (from Nelson et al., 2015).

The USGS in the late 1980s conducted a study regarding highway damage caused by faults in South Dakota (Collins et al., 1988). The study aimed to characterize the faults, describe the highway damage associated with them, and suggest possible solutions to avoid damage to highway infrastructure. Twenty-four sites located west of Pierre, South Dakota were selected for more in-depth review. Most of the failures associated with faults that intersect the road consisted of "swells or sags of a few inches to as much as a foot over a distance of 10 to 100 ft. along the axis of the highway" (Crandell, 1958). It was believed that these failures were the result of the accumulation of excess water in the subgrade, thereby weakening it and/or causing it to swell. When the road was constructed, overburden stresses were released, especially in areas with significant cuts. The release of overburden stress allowed the fault to rebound. This rebounding action opened up the surface of the fault gouge which allowed water to enter and accumulate. Nichols (1992) discusses the rebound effect at length, focusing in part on rebound in Pierre shale. Movement in the fault is not thought to be from tectonic activity (Collins et al., 1988). Faults were mapped throughout a 2,000 km^2 area west of Pierre, South Dakota and are prevalent throughout the region (Nichols et al., 1994). Movement at the surface due to these faults are related to the amount of overburden material excavated during construction, the degree of saturation of the clay/shale, and presence of other geologic inhomogeneities. Rebound from static nonequilibrium due to unloading may also be due to an increase in the coefficient of lateral earth pressure as fracturing of the clay/shale destroys bonds between the clay particles and thereby freeing "recoverable strain energy" (Bjerrum, 1967). Expanding clay within the gouge region expand and lose frictional strength allowing rebound movement along the fault plane. Related to this is the increased permeability of the soil within the faulted area, increasing by as much as 30 times materials located outside of the fault (Hammerquist and Hoskins, 1969). It was also speculated by Hammerquist and Hoskins (1969) that a half inch layer of dry material within the fault gouge would swell to 5 inches thick with swell pressures reaching 15 tsf, easily distorting a paved surface. As part of their research, trenches were dug within the fault zone revealing steep fault planes ranging from 41 to 65 degrees from the horizontal and intersecting the surface of the ground, as illustrated in Figure 10. The gouge is the natural crack that forms from the movement and subsequent deterioration of the soils within the fault zone. It is wider near the surface (on the order of up to 6 inches), but further down (beyond 10 ft. depth) the gouge zone was much narrower (on the order of an inch) and diminishing from there. Downward migration of surface water into the fractured soil surface causes further weathering and breakdown of the clay/shale soil matrix allowing substantial surface deformations to take place from expanding soils. Sealing the soil from these changes is the best way to restrict this breakdown. A sprayed asphalt liner was successfully used to seal the Oahe damsite clay/shale formation from continued deterioration and uplift.

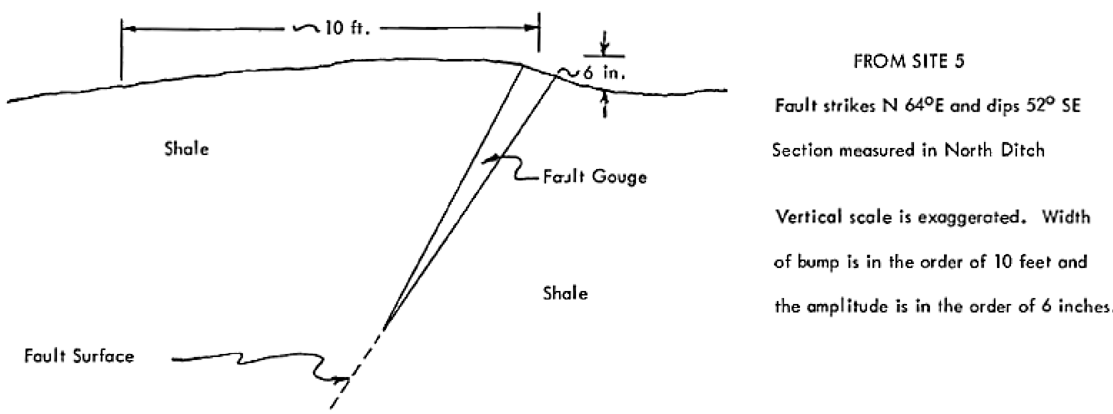

Figure 10: Illustration of a typical bump caused by exposed faults (from Hammerquist and Hoskins, 1969)

5.4 Mitigation of Expansive Clay

Expansive clay can lead to premature pavement failure and increased maintenance and rehabilitation costs for roads. Areas with significant moisture fluctuations and presence of expansive clays require special attention. Recommended strategies for mitigating expansive clay are discussed below (Christopher et al., 2006; ARA, 2004; Snethen et al., 1975), several of which have been tried in South Dakota with varying levels of success.

- Remove and replace—may be feasible for thin clay layers near the surface, but becomes less cost-effective for deeper deposits.
- Extend road width—reduces fluctuations in moisture content, but will likely not stop the threat of water ingress altogether. It may also be impossible due to right-of-way or budgetary constraints.
- Increase road crown—reduces infiltration but does not eliminate it. It also does not address water ingress from other sources such as ground water or nearby drainages.
- Partial or full encapsulation of the subgrade—the use of waterproofing membranes (e.g., sprayed asphalt emulsions, geosynthetics, etc.) to reduce fluctuations in moisture content in the subgrade has been used in South Dakota with low success.
- Compact at moisture contents above optimum with kneading compaction—produces dispersed clay structure which swells less than flocculated clay structure, but does not eliminate the swell potential of the soil. This method has successfully been used in the past in South Dakota.
- Scarify, stabilize and compact with lime, fly ash, or Portland cement—reduces volume change characteristics and may increase strength. This method has also been successfully used in South Dakota with success.

5.4.1 Chemical stabilization with lime

The most widely used mitigation technique for expansive clay in the U.S. is chemical stabilization, mostly using lime, although fly ash and Portland cement are also commonly used. Dozens of products have been tried (hundreds, according to Snethen et al. (1975)), including sodium chloride, magnesium chloride, phosphoric acid, kerosene, emulsified asphalt, soy flour, etc. In a physical and chemical sense, lime stabilization is an effective strategy to mitigate expansive soils for the following main reasons (Christopher et al., 2006):

- lime quickly absorbs water in the clay allowing it to dry, which can provide a suitable working platform for construction equipment, and
- cation exchange (calcium and magnesium from lime exchanges with the more active sodium and potassium in clay) reduces plasticity and changes the gradation (produces a coarser texture similar to silt or sand)

Lime is also relatively inexpensive, generally abundant and relatively simple to incorporate into traditional road construction activities. Adding lime for short-term modification of the soil to improve constructability is referred to as lime modification. Typical application rates for lime modification are generally 1–4 percent of the dry weight of soil.

The optimum application rate for a specific soil can be determined several ways. ASTM D6276 uses pH as an indicator for the minimum amount needed by testing the pH for several rates (typically 2, 3, 4, 5 and 6 percent lime) and choosing the lowest rate that has a soil pH of at least 12.4 one hour after application. This test does not always yield an optimum application rate for long-term stabilization, thus strength tests, such as unconfined compressive strength or CBR, may also be needed to determine the amount needed for stabilization.

Practical limits of in-place mixing are about 2–3 ft. and if significant moisture fluctuations are expected below that then drill hole, lime–slurry pressure injection, or other techniques for deeper stabilization may be considered.

5.4.2 Chemical stabilization with self-cementing products (fly ash, Portland cement)

While stabilization with lime is associated with increased subgrade strength, lower application rates may not provide the durability over cyclic wetting/drying or freezing/thawing. Insufficient lime may also not allow a reduction in pavement thickness. However, adding fly ash or Portland cement (often in combination with lime) generally provides strength increases that do allow a reduction in pavement thickness. Similar to lime, fly ash chemically stabilizes expansive soils through cation exchange of calcium, aluminum, and iron that helps clay particles bind together, thereby minimizing swell potential (Fusheng, 2008), as well as physically cementing the soil particles together. The optimum content of fly ash to be added to expansive soils typically ranges between 9 and 12 percent of the weight of treated soils (Fusheng, 2008) or from 8 to 16 percent according to the ACAA (2003). One potential disadvantage of using fly ash is that its chemical properties vary significantly between different production plants. It may be necessary to utilize fly ash from the same plant to maintain consistency while treating soils.

Compaction must occur immediately after mixing and within two hours (ideally within one hour) of applying Class C fly ash or Portland cement. Delays in compaction result in much lower strength gain (White et al., 2005).

5.4.3 Mitigation in localized faulted areas

Uplift or heaving due to the removal of overburden (a.k.a. rebound) is probable where highway cut sections intersect faulted clay-shale deposits. The level of heaving depends on the amount of overburden removed, the sensitivity of the clay and the extent of fissuring and faulting. Restricting water from entering these faulted areas and sensitive zones is the key to controlling the swelling of the clay-shale deposits. Collins et al. (1988) installed a drainage system parallel to the highway and sloping away from the road to allow surface water to drain from the road and away from the faulted zones; thereby preventing these zones from experiencing wet and dry cycles and causing disintegration of the particles which facilitates rebound movements in the faulted region through the gouge. In addition, it was recommended to add a calcium rich substance such as lime or gypsum to facilitate the exchange of sodium ions with calcium ions, thereby reducing swell potential. One final suggestion was to allow the rebound in the faulted area to occur and then fill in the downside of the fault scarp rather than releveling the road through continued excavation, as further excavation would likely facilitate continued heave of the faulted zone.

5.4.4 Summary of chemical stabilization

Significant research has been conducted on stabilization of expansive clay, particularly involving lime, fly ash, and/or cement. A summary of the research, including soil type, stabilizer, laboratory testing, and significant results is provided in Table 3.

Table 3: Summary of Research on Stabilization of Expansive Clay

Soils USCS Class. (location)	Stabilizing Additives	Lab Tests	Results & Recommendations	Reference
CH (Sudan)	• Lime, 3–15% • Fly Ash (F), 5–40%	• GSD (sieve) • Atterberg Limits • Modified Proctor • Free-Swell Index • Swelling Pressure • CBR • UCS	• Significant reduction in swell and increase in strength with stabilizers • Use fly ash for low to medium expansive soil • Use combination of lime and fly ash for highly expansive soils	Zumrawi & Hamza (2014)
SC (India)	• Fly Ash, 20–80% • Polymeric Fiber (0–1.5%)	• GSD (sieve) • Atterberg Limits • Standard Proctor • CBR	• Increased strength with up to 30% fly ash • Fiber provided only marginal improvement in strength	Sharma (2012)
CL, CH (Texas)	• Lime, 0–8% • Cement, 0–6%	• GSD (sieve) • Atterberg Limits • Static Compaction • Cation Exchange Capacity • Specific Surface Area • UCS • Swell with cyclic wetting/drying (ASTM D559, withdrawn)	• Soils with low percentages of Montmorillonite are effectively treated with current Texas practices of lime • Soils with high percentages of Montmorillonite need higher lime or cement application rates to survive repeated wetting and drying	Perdarla et al. (2011)
CL, CH (Cyprus)	• Fly Ash (C), 15–25%	• GSD (hydrometer) • Atterberg Limits • Standard Proctor • Linear Shrinkage • Swell • Cation Exchange Capacity	• Fly ash increases the particle size and reduces plasticity • High plasticity clay saw a greater reduction in plasticity • Fly ash reduces expansion and swell pressure	Nalbantoglu (2004)

Table 3: Summary of Research on Stabilization of Expansive Clay (continued)

Soils USCS Class. (location)	Stabilizing Additives	Lab Tests	Results & Recommendations	Reference
CH (India)	• CaCl2 + NaOH for in-situ lime precipitation, 2.5–16.4%	• GSD (sieve, hydrometer) • Atterberg Limits • Standard Proctor • Specific Gravity • Swell (using oedometer)	• Reduced plasticity index, reduced swell potential, increased strength • Strength increased with increased curing time	Thyagaraj et al. (2012)
CH (Australia)	• Lime, 1–6%	• Linear Shrinkage • CBR • UCS	• Significant decrease in shrinkage and increase in strength with increasing lime rates	Walter et al. (2012)
CH blended with 20% ¼ in. granulated scrap tire rubber (Colorado)	• Fly Ash (C), 14% • Fly Ash (off-spec) 14%	• Atterberg Limits • Proctor (Std. & Mod.) • UCS • CU triaxial	• Significant improvement in consolidation, shear strength and stiffness with fly ash stabilized mixes	Wiechert & Carraro (2011)
CH (India)	• Lime, 5% • Fly Ash (C), 5–25%	• Atterberg Limits • Standard Proctor • Free-Swell Index	• Significant reduction in plasticity (reduced LL, increased PL) and swell • Lime and fly ash is effective for reducing shrinkage, swelling and differential settlement	Malhotra & Naval (2013)
CH (Nigeria)	• Fly Ash, 0–3% • Cement, 0–12%	• Proctor • CBR • UCS • UU triaxial	• Significant improvement in strength with stabilization • Optimal mix was 9% cement and 3% fly ash	Amu et al. (2007)
Abbreviations: GSD = Grain size distribution; CBR = California bearing ratio, UCS = Unconfined Compressive strength, CU = Consolidated undrained, UU = Unconsolidated undrained t				

5.4.5 Leaching of Chemically Stabilized Expansive Clay

In a research study conducted on soils from Texas, four CH soils were stabilized with various percentages of lime or cement (6 percent lime, 8 percent lime, 3 percent cement, and 6 percent cement) with accelerated curing (drying at 104°F for 48 hours and backpressure saturation for 24 hours). The 4-in. diameter, 6-in. tall samples were placed in a constant-head leachate apparatus that subjected each sample to one leaching cycle per day where the volume of water leached equals the volume of voids in the soil sample. After 14 cycles, equivalent to about 5 years of rainfall infiltration in the field, nearly all samples lost less than 1 percent lime or cement (calculated from measuring calcium concentration in leachate). Unconfined compressive strength tests after the 14 leaching cycles showed average strength retention of 96 percent, with only one sample having less than 90 percent strength retention (Chittoori et al., 2011). From additional soils and stabilizer additive rates reported by Chittoori (2008), treating soils with lime at a lower rate than optimum (as determined by ASTM D6276) is associated with more significant strength loss due to leaching. Furthermore, whereas cement was associated with generally greater calcium leaching than lime, strength retention was high for all cement-stabilized soils.

Leach tests were performed on weathered clay shale soils from the Eagle Ford geologic formation in Dallas–Fort Worth, Texas treated with hydrated lime at 0 to 9 percent (McCallister & Petry, 1992). Based on Eades and Grim pH test (later formalized as ASTM D6276), the optimum lime additive rate for plasticity reduction is around 3–4 percent. However, optimum lime content for stabilization based on increased shear strength is 6–7 percent. Not surprisingly, greater calcium leachate was measured in samples with greater lime additive. Samples compacted dry of optimum also had greater calcium leaching (due to increased permeability that developed in these samples). Thus, this study provides another reason to compact samples at moisture contents greater than optimum.

6.0 SITE DESCRIPTIONS

Six sites were chosen by SDDOT for evaluation of expansiveness and appropriate mitigation recommendations. The sites are shown on the map below (Figure 11). Four sites are on Interstate 90, one on US14, and one on SD34. Most are within the Upper Cretaceous formation, although Site 2 is in the Tertiary White River Group and Site 6 is most likely in the Pierre Shale Group.

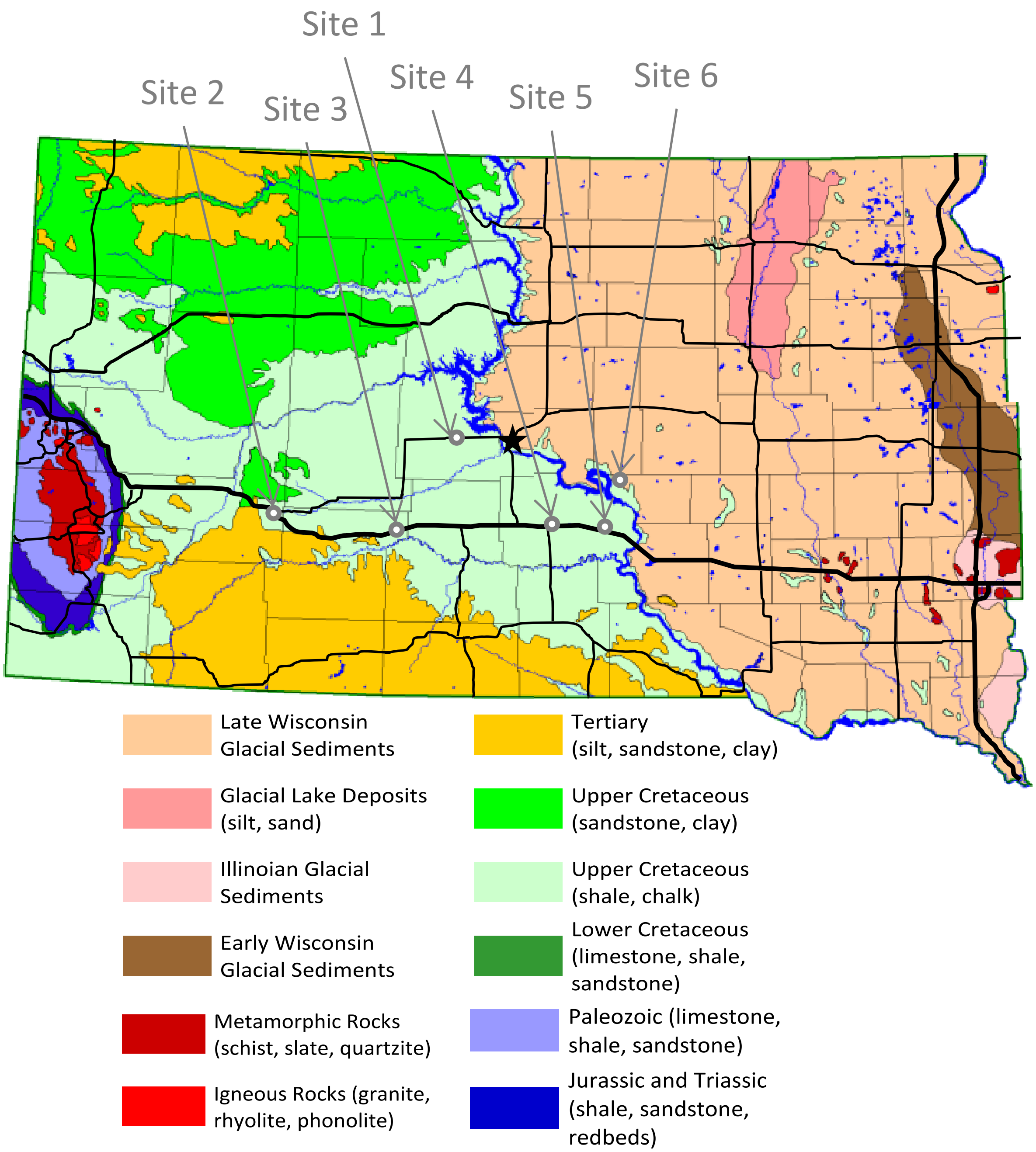

Figure 11: Location of six sites on a geologic map of South Dakota.

6.1 Site 1 – US14 MRM 207.75

Site 1 is located on US14 at MRM 207.75. Soil was collected on the north side of the East–West highway on the back-slope of the ditch cross-section within the SDDOT right-of-way. A fault heave is visible on the south and north sides of the highway, with a transverse crack running across the highway at the location of the fault (Figure 12). The road appears to have settled and heaved at the fault location (Figure 13), but may only be at the transitions of a previous fly ash fault heave repair. More recent photographs of the site reveal the fault extending along the cut face to the borrow area (Figure 14). The fault does not extend beyond the cut slope, as shown in Figure 15. The average rut depth and international roughness index (IRI) of US14 between MRM 209 and 211 are shown in Figure 16 and Figure 17.

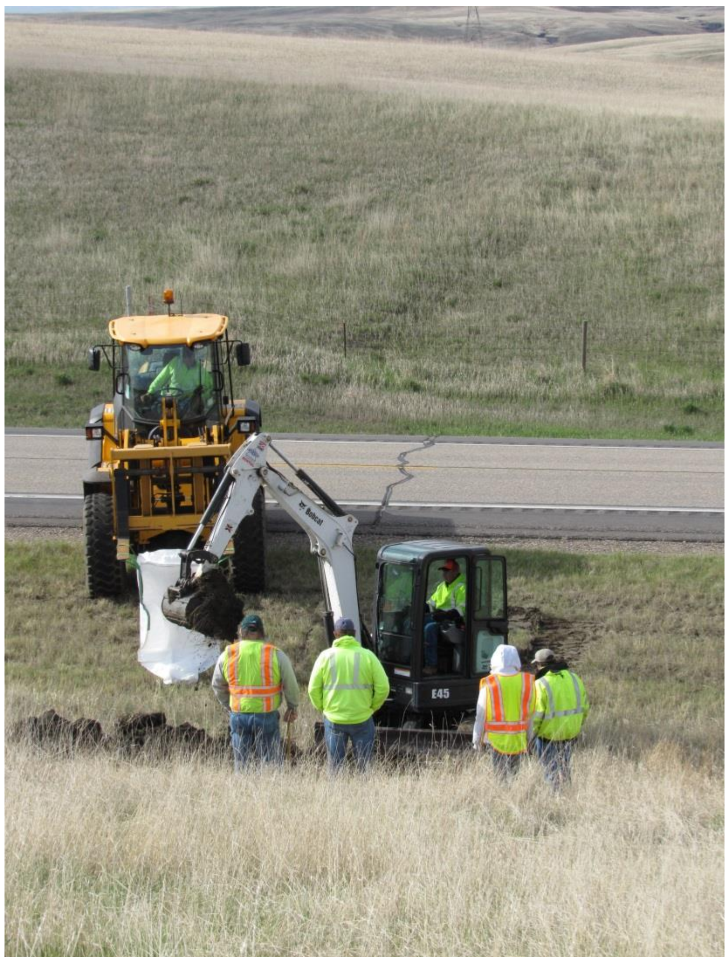

Figure 12: Site 1, looking south.

Figure 13: Site 1, settlement and/or heave in the highway near the location of the fault.

Figure 14: Site 1, fault extending up the cut slope.

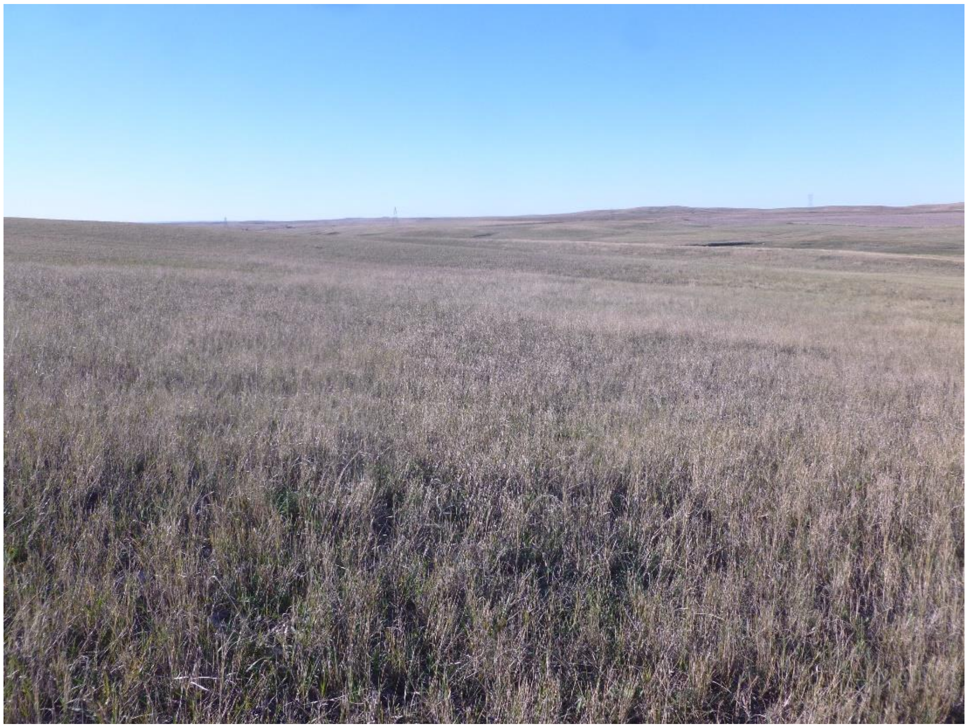

Figure 15: Site 1, no visible fault beyond the cut slope.

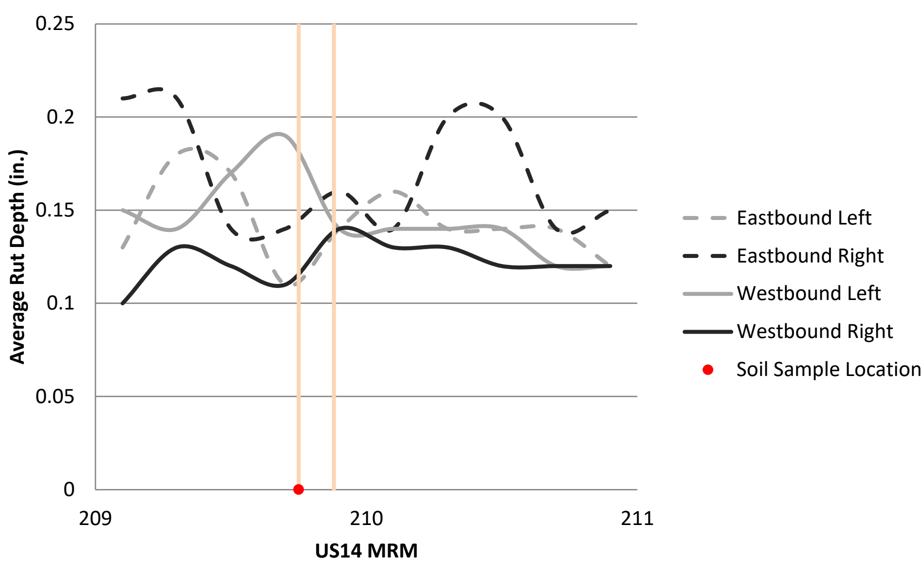

Figure 16: Site 1, rut depth along highway adjacent to soil sample.

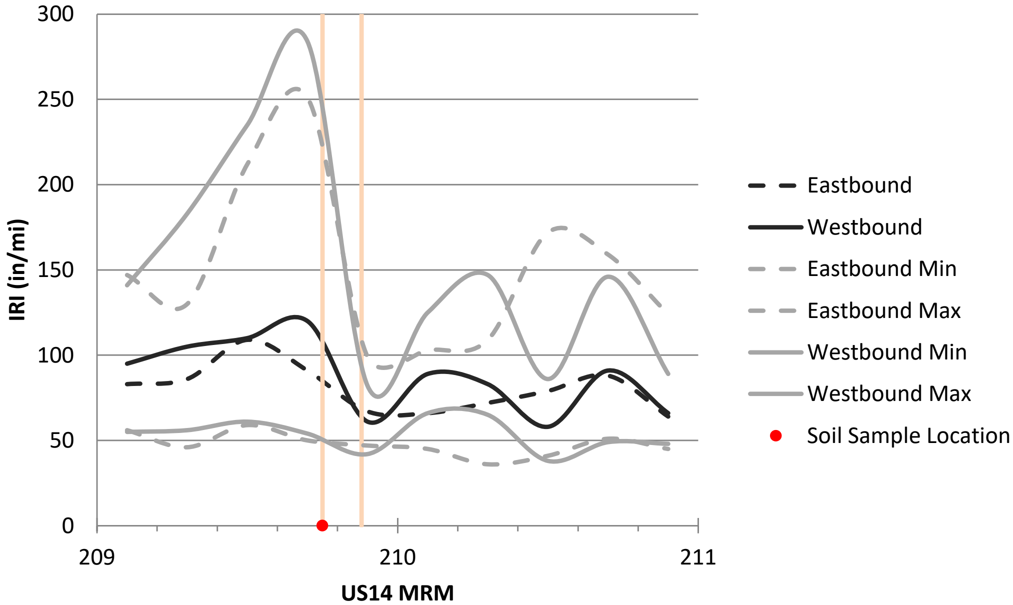

Figure 17: Site 1, IRI along highway adjacent to soil sample.

In 2009, this site received a fly ash fault heave repair between MRM 209.75 and 209.88 (633 linear feet). A shoulder width (about 36 ft.) undercut 3 ft. below the finished subgrade was excavated and stabilized with Class C fly ash at a rate of 12 percent of the dry density and at a moisture content range of 0 to 4% above optimum. It was replaced in one 12-inch and three 8-inch lifts (cross-sectional view is shown in Figure 18). A fault trace excavation procedure was also performed in which a trench about 50 ft. wide, 36 ft. long and 2 ft. deep was backfilled with uncompacted porous backfill (natural sand, no crushed material, with less than 2 percent fines) and overlain with a nonwoven geotextile (profile view is shown in Figure 19). The source of the fly ash was a coal combustion power plant in Port Neal, Iowa. The brown, wet, organic topsoil thickness was approximately 1–2 ft. The gray shale with iron staining was collected about 2–3 ft. below ground surface (Figure 20).

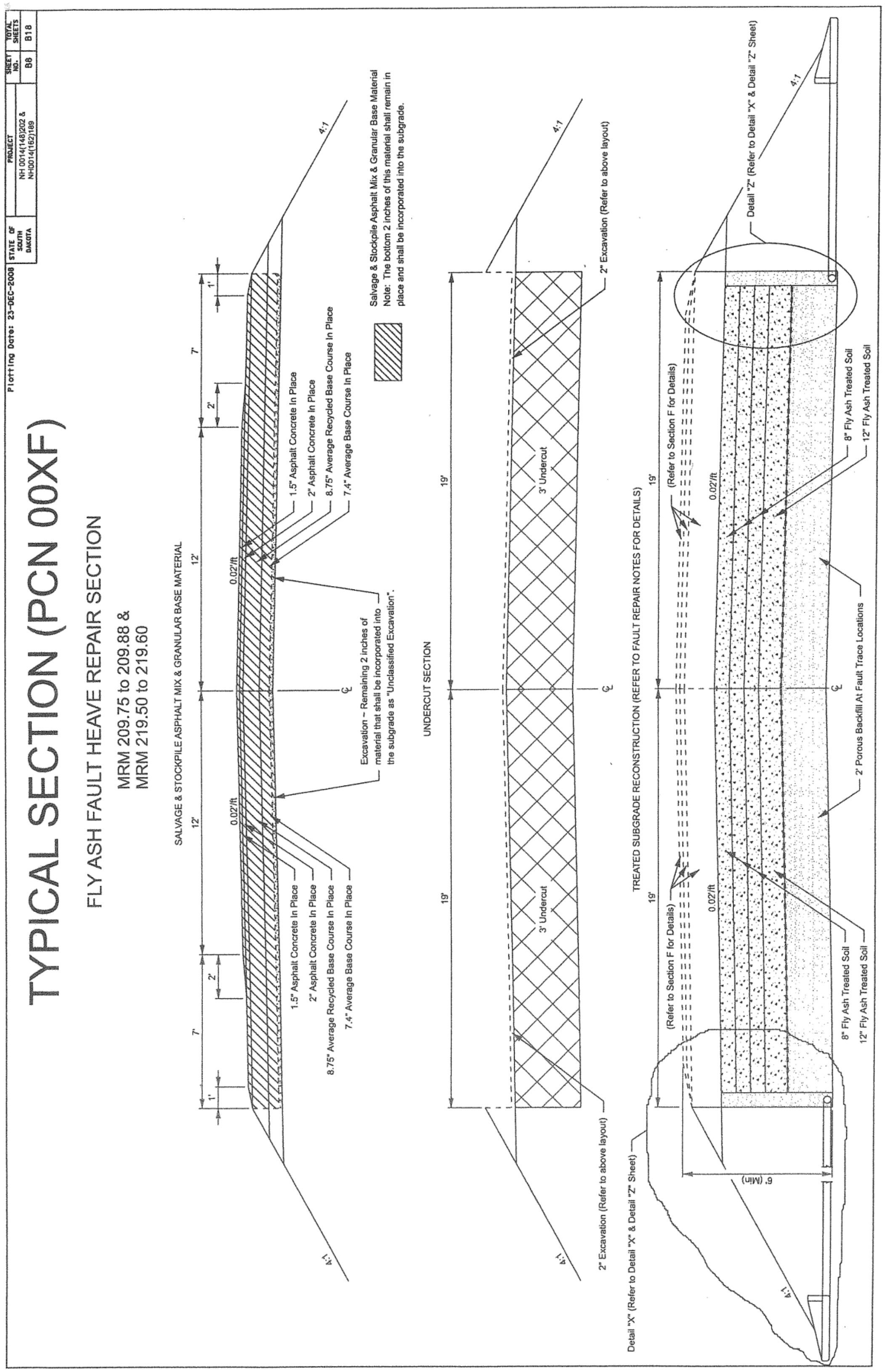

Figure 18: Cross-sectional view of Site 1 heave repair.

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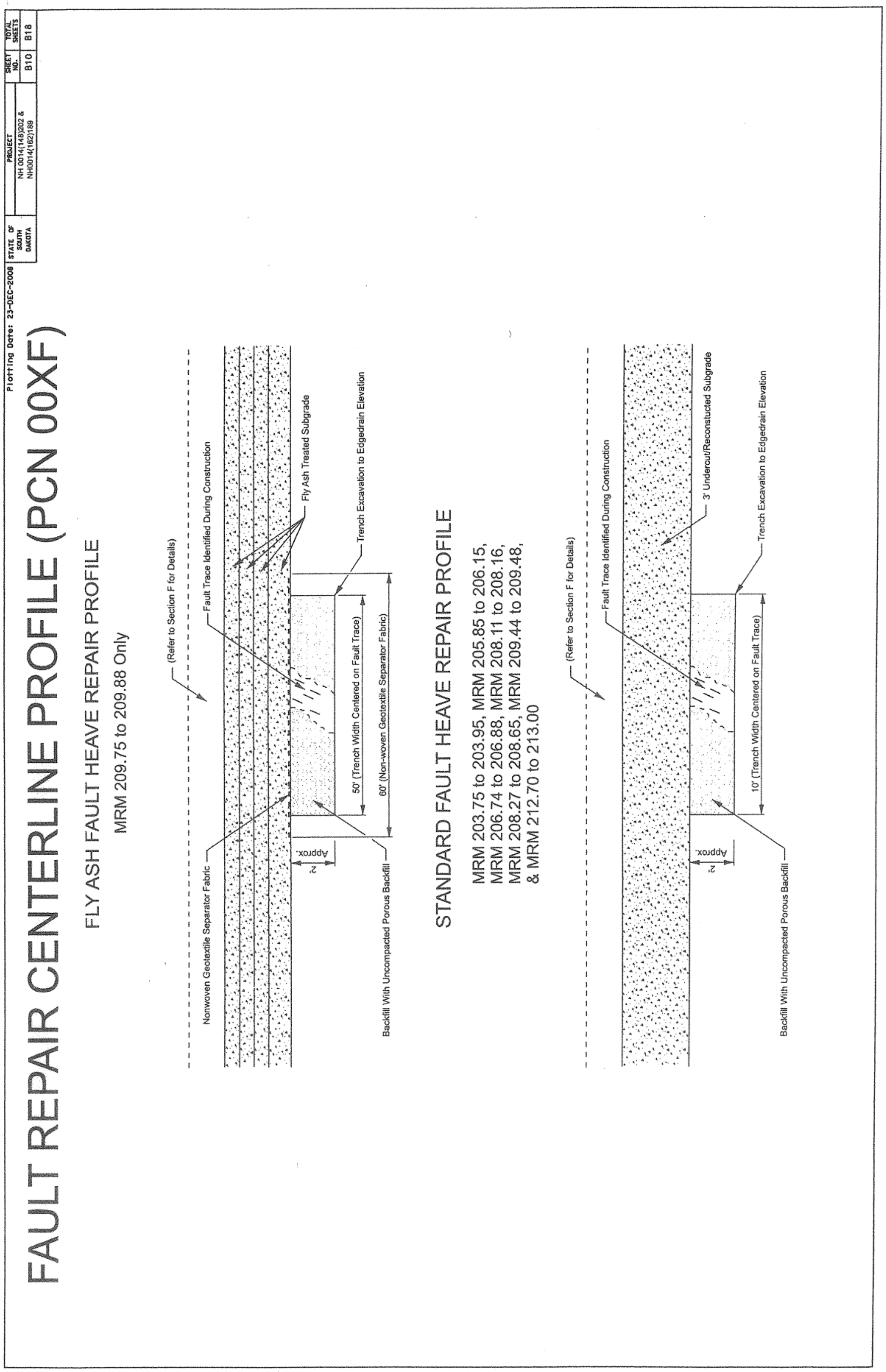

Figure 19: Profile view of Site 1 heave repair.

Figure 20: Site 1, soil sample test pit.

6.2 Site 2 – I90 MRM 118.35

Site 2 is located on Interstate 90 at MRM 118.35, about 7.35 miles east of SD240 at Wall, SD. Soil was collected on the south side of the Eastbound lanes on the back-slope of the ditch cross-section within the SDDOT right-of-way, near fencing (Figure 21). Based on the IRI of the driving lane adjacent to the soil sampling, the road appears to be in excellent condition (Figure 22). The light brown, very wet and sticky topsoil was about 1–2 ft. thick, above the light gray to yellow, dry, brittle shale collected between about 2–3 ft. below ground (Figure 23). Site 2 had concrete pavement with asphalt shoulders. No visible heaving was evident on the road surface near where the soil was collected (Figure 24). The topography was very flat making it somewhat difficult for runoff. There were several places near the site where the soil had heaved causing ponding, as illustrated in Figure 25 and Figure 26, indicative of faulting.

Figure 21: Site 2, soil sample location on the south side of I-90 east.

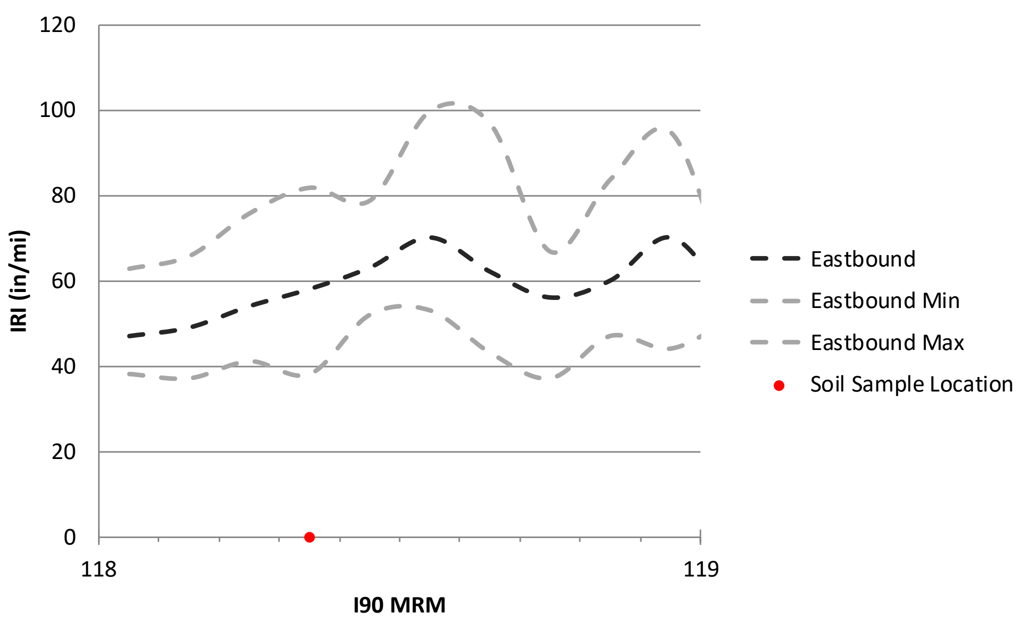

Figure 22: Site 2, IRI in driving lane along highway adjacent to soil sample.

Figure 23: Site 2, soil sample test pit.

Figure 24: Site 2, road profile.

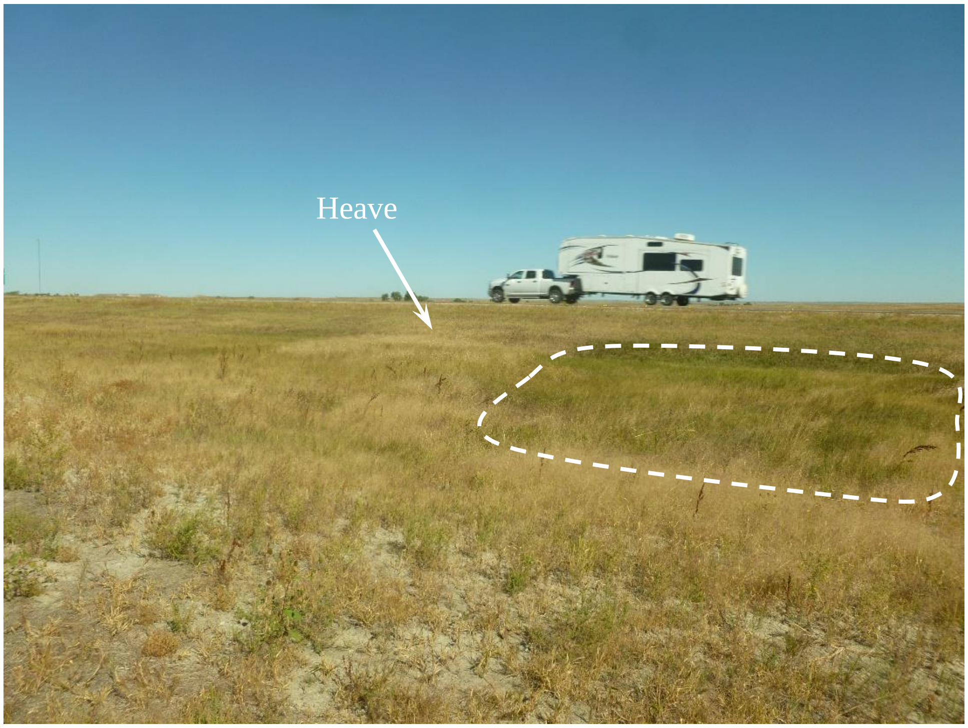

Figure 25: Site 2, heave in median with remnants of ponding.

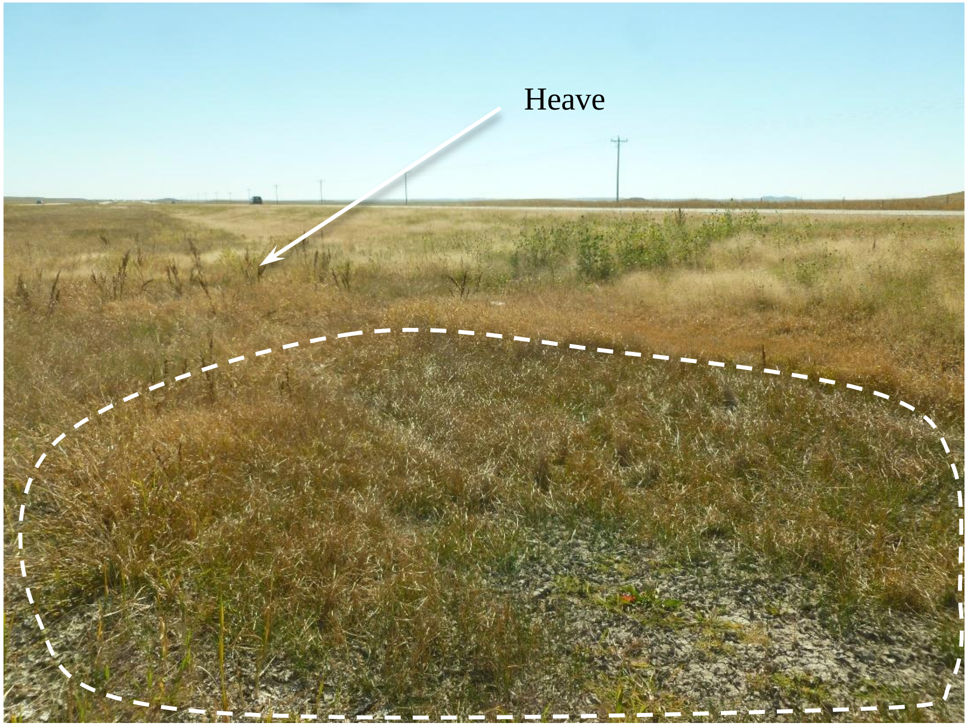

Figure 26: Site 2, another heave in center borrow with remnants of ponding in foreground.

6.3 Site 3 – I90 MRM 168.98

Site 3 is located on Interstate 90 at MRM 168.98, about 1.3 miles west of SD63. Soil was collected on the backslope of the north berm. Multiple heaves are visible in the in-slope and ditch on the north side of westbound I90 (Figure 27). Standing water was visible at many areas, often held back at a fault. At the soil sample location, some water was held back on the west side of the fault. Soil was collected on the east, heave side of the fault, away from standing water (Figure 28). There was 1.5–2 ft. of brown wet topsoil above the shale that was mottled brown to black with rust stains and red to brown concretions (Figure 29).

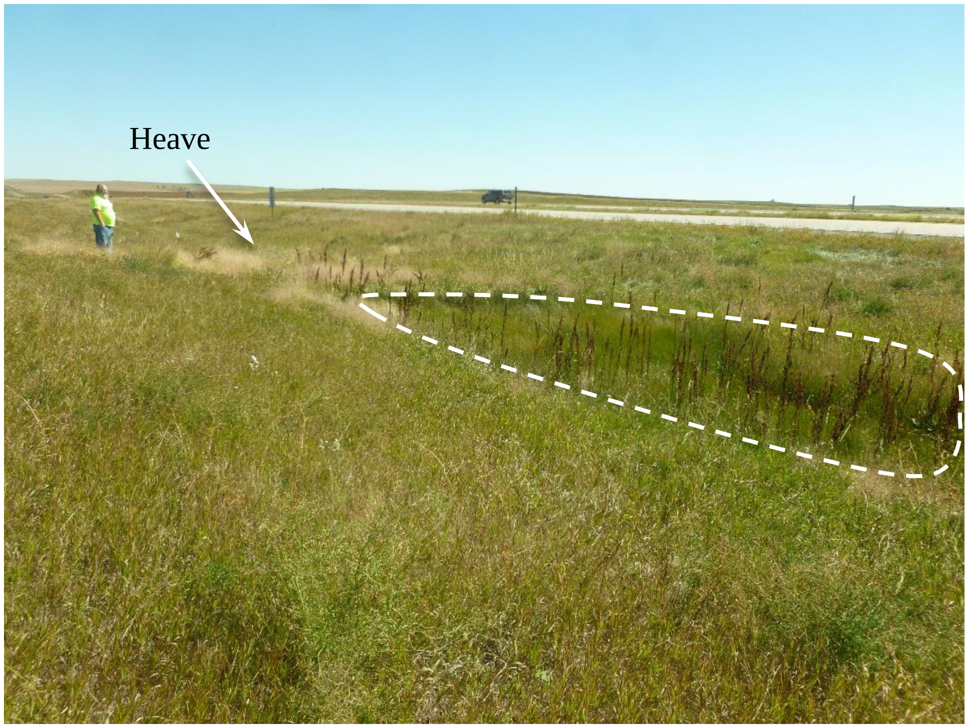

Figure 27: Site 3, looking east, prominent fault heave in ditch and inslope on north side of westbound I-90, ponded area highlighted.

Figure 28: Site 3, soil sample location on east side of fault (heave) showing standing water on west side of fault.

Figure 29: Site 3, soil sample test pit and close-up of material.

6.4 Site 4 – I90 MRM 227.30

Site 4 is located on Interstate 90 at MRM 227.30, about 0.9 miles east of US183 at Presho, SD. Soil was collected on the south side of the Eastbound lanes on the back-slope of the ditch cross-section within the SDDOT right-of-way, near fencing. Multiple heaves are visible in the in-slope and ditch on the south side of eastbound I-90. Standing water was visible at many fault heaves (Figure 30), although some fault heaves were merely moist without standing water, such as the location of the soil sampling. Based on the rut and IRI of the driving lane adjacent to the soil sampling, the road appears to be in good condition (Figure 31 and Figure 32). The brown, wet topsoil was about 2 ft. thick, above the gray to brown, moist, rubber-like shale collected between about 2–4 ft. below ground (Figure 33).

Figure 30: Site 4, multiple fault heaves, some with standing water.

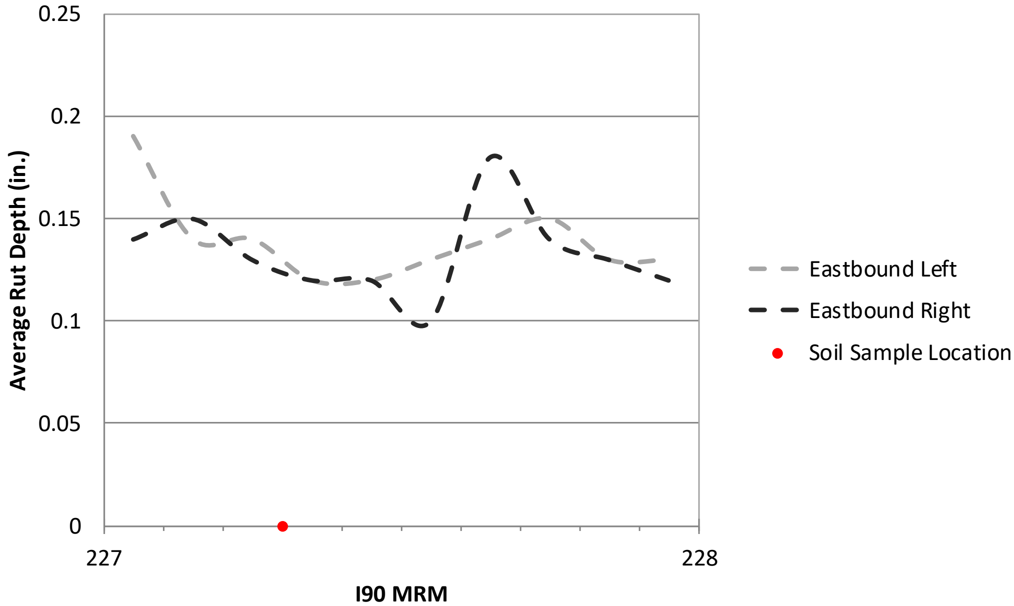

Figure 31: Site 4, rut in eastbound driving lane on highway adjacent to soil sampling.

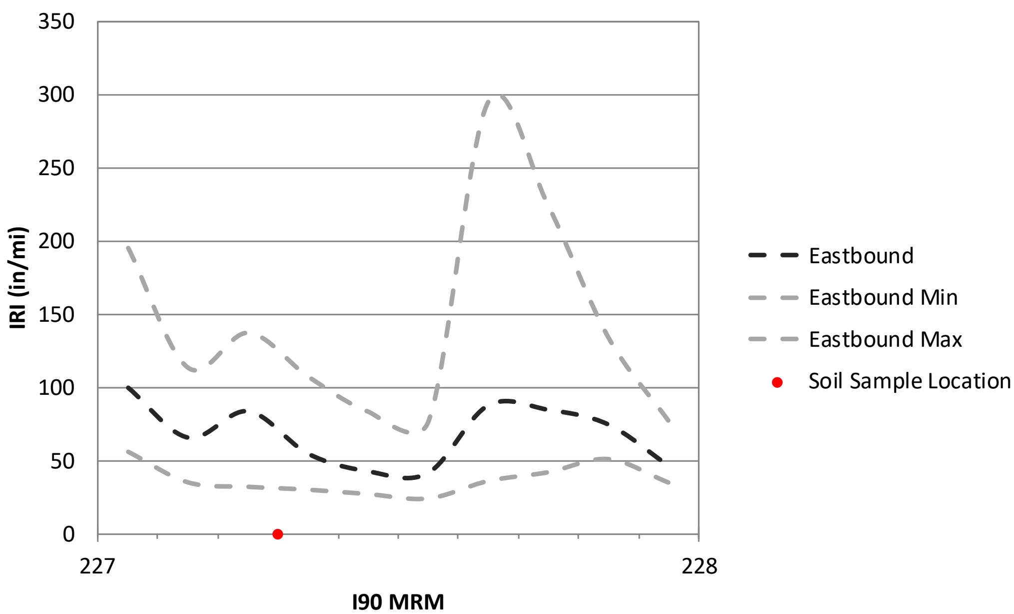

Figure 32: Site 4, IRI on eastbound driving lane on highway adjacent to soil sample.

Figure 33: Site 4, soil sample test pit.

6.5 Site 5 – I90 MRM 253.6

Site 5 is located on Interstate 90 at MRM 253.6, about 2.5 miles east of SD47 near Reliance, SD. Soil was collected on the south side of the Eastbound lanes on the back-slope of the ditch cross-section within the SDDOT right-of-way. Multiple heaves are visible in the in-slope and ditch on the south side of eastbound I-90 (Figure 34). A short length of road near the soil sampling test pit appears to have been overlaid (Figure 35). The sampling test pit is shown in Figure 36. The dark brown topsoil was about 5 ft. thick, above the wet black to dark gray mottled brown with orange and white precipitate collected about 5–10 ft. below ground. Based on the IRI of the driving lane adjacent to the soil sampling, the road appears to be in fair condition (Figure 37). An obvious heave intersects the roadway causing a hump and dip (Figure 38), and this heave follows the fault gouge across the median (Figure 39). Site was under construction during the time that the photos in Figure 38 and Figure 39 were taken.

Figure 34: Site 5, multiple fault heaves located in the ditch.

Figure 35: Site 5, evidence of road maintenance adjacent to soil sampling test pit.

Figure 36: Site 5 soil sample test pit.

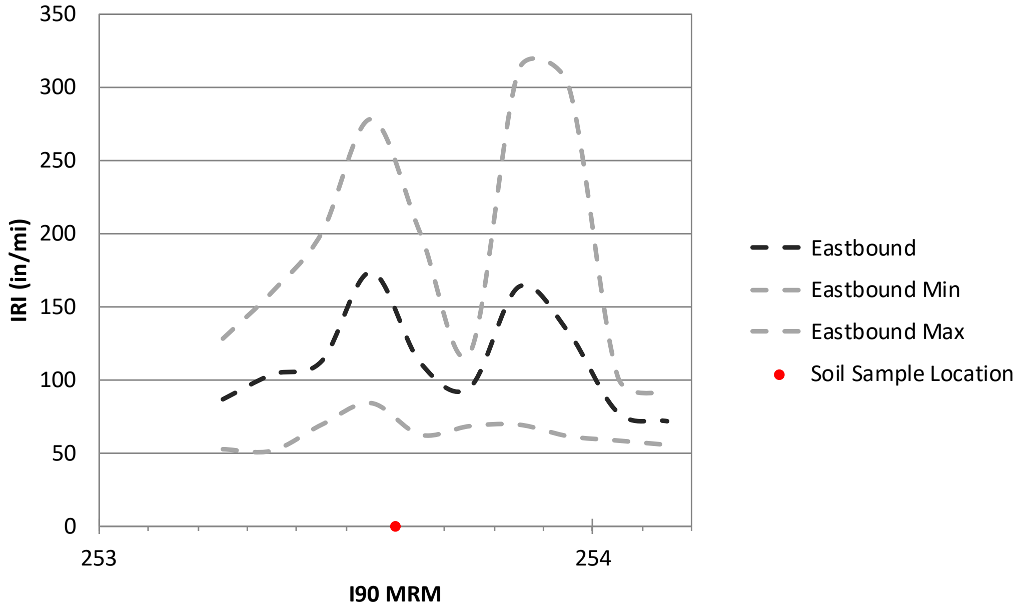

Figure 37: Site 5, IRI of driving lane on highway adjacent to soil sampling.

Figure 38: Site 5, heaved pavement surface with dip.

Figure 39: Site 5, heave evident across the median.

6.6 Site 6 – SD34 MRM 262.02

Site 6 is located on SD34/SD47 at MRM 262.02 along a north–south portion of the east–west highway. The pavement received an asphalt overlay in 2014, with grading and seeding in the ditch (Figure 40). The soil test pit was dug on the in-slope toe of the west berm of the highway. This is the only site with soil collected from the in-slope (other test pits were excavated on the backslope). The moist brown to light brown topsoil was about 6–7 ft. thick, with iron and white precipitate collected about 7–8 ft. below ground (Figure 41).

Figure 40: Site 6, borrow area.


Figure 41: Site 6, soil sample test pit.

7.0 LABORATORY TESTING

A preliminary testing matrix and screening process was proposed to help determine the additive type and addition rate. Preliminary additives and rates were determined through the literature review and discussions with the research technical panel at SDDOT. A procedure by which an optimal material and application rate was established for each of the six clay/shale soils provided by SDDOT is outlined in Figure 1. Round 1 testing was used to characterize the general properties of the clay/shale soil samples using a series of standard soil tests. The second round used the Free-Swell Index test (IS2720, Part 40) to narrow down which additives and rates show the greatest improvement. Sixteen combinations of lime and Class C fly ash were tested on each of the six soils during this round of testing for a total of 96 tests. A preliminary screening process based on basic cost information and the results of the free-swell tests was employed to determine the optimal level of chemical additive to use during the Round 3 testing. Soaked CBR tests were conducted on each of the six soils that were blended with the best two admixture combinations prepared at optimum moisture content and at 2 percent above optimum. Time for mellowing the mixtures was not used for this round of tests. The final round of testing (Round 4) was used to determine the effect of large particles and mellowing time on the optimal blend/rate combination from Round 3 testing. Large-scale soaked CBR tests were conducted to determine this effect.

7.1 General Characterization Tests

General characterization of these soils was done using several standard soil tests, namely, Atterberg Limits tests, Standard Proctor compaction test, and California Bearing Ratio (CBR) test. Hydrometer tests (ASTM D422) were conducted to determine the particle size distribution and percent clay content. Knowing the clay content is useful to determine the activity of the soil, an indicator of the soil's expansion and contraction properties when wetted. Atterberg limit tests were used to determine the liquid limit, plastic limit, and plasticity index of the soils (ASTM D4813). This information was initially used along with the particle-size analysis to classify the soils, as well as to provide a baseline value by which to compare the effects of the various stabilizing agents. Standard Proctor tests (ASTM D698) were conducted to determine the optimum moisture content to achieve the maximum Proctor density of the soil. This information was used to determine the relationship between water content and density as well as to determine the soil preparation protocol for the CBR tests. Soaked CBR tests (ASTM D1883) were used to characterize the one-dimensional swell properties and bearing strength of each of the soils. The results of these tests provided a baseline measure of the bearing capacity and swell of the soils at their optimum moisture content. Sulfur content and pH are relevant to lime stabilization of soils and were measured following ASTM D1580 and ASTM D4972, respectively.

7.1.1 Atterberg Limits

Atterberg limit tests (ASTM D4318) were used to determine the liquid limit, plastic limit and plasticity index on each of the six clay/shale materials. Using these properties, the soils were classified using the USCS system (ASTM D2487) and AASHTO system (ASTM D3282). The results of these tests are listed in Table 4.

Table 4: Summary of Atterberg Limits and Classifications

	Liquid Limit	Plastic Limit	Plasticity Index	USCS Classification	AASHTO Classification
Site 1	82	41	41	MH	A-7-5
Site 2	86	49	37	CH	A-7-5
Site 3	68	35	33	MH	A-7-5
Site 4	72	32	40	CH	A-7-5
Site 5	100	39	61	CH	A-7-5
Site 6	78	39	39	MH	A-7-5

7.1.2 Particle Size Distribution

A hydrometer analysis was used to determine the particle size distribution of each of the six soils. Individual particle size distribution curves are shown in Figure 42. The clay fraction was determined by the amount passing the 0.002 mm particle size. This information coupled with the Atterberg Limits results was used to determine the activity of each soil. Activity ranges from 0 to 2 and provides an indicator of the soil's expansion and contraction properties when wetted. Activity is determined by dividing the Plasticity Index by the percent clay fraction. Activity values approaching 0 are less prone to swelling and contracting while values approaching 2 are more prone. Clays are considered "Inactive" for values less than about 0.75, "Normal" for values between approximately 0.75 and 1.25, and "Active" for values for greater than about 1.25. Most of the activity values were around 0.8 or less meaning that they were on the high end of the inactive or low end of the normal range. The one exception was Site 5 which was on the high end of the normal range. A summary of the clay fraction and activity values for each of the sites is presented in Table 5.

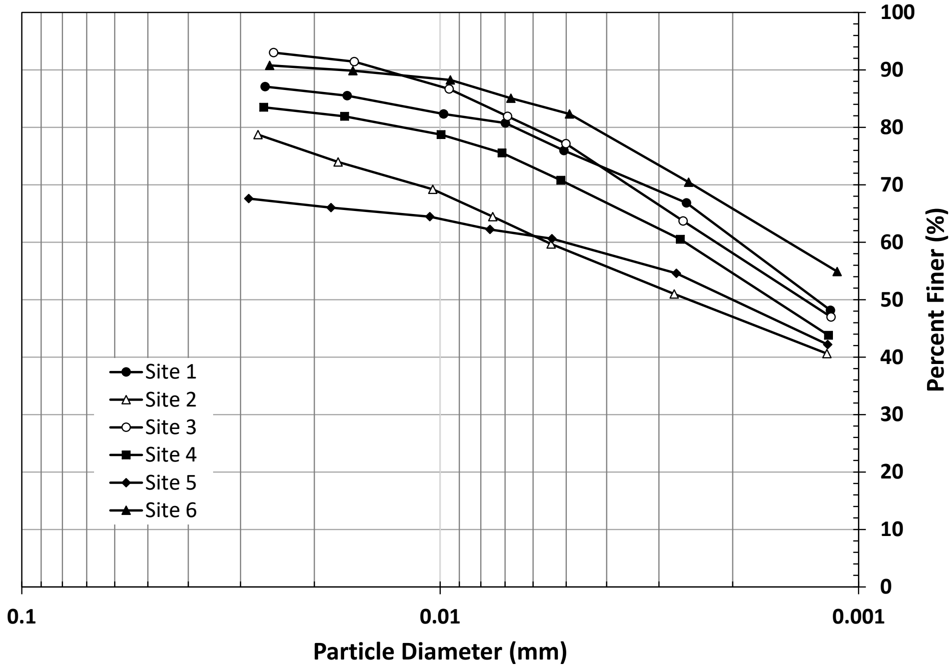

Figure 42: Particle-size distribution based on the hydrometer results.

Table 5: Summary of Clay Fraction and Activity

	Clay Fraction[1] (%)	**Activity[2]**
Site 1	61	0.7
Site 2	47	0.8
Site 3	58	0.6
Site 4	55	0.7
Site 5	50	1.2
Site 6	66	0.6

[1] percent finer than 0.002 mm from hydrometer analysis
[2] PI divided by the clay fraction

7.1.3 Chemical Attributes

The pH and sulfur content were determined by ASTM D4972 and ASTM C1580, respectively. pH has been used by some to estimate the lime proportion that will help mitigate expansive soils (ASTM D6276). This is done by adding lime to clays particles that pass the No. 40 sieve to determine the minimum percentage of lime that results in a pH of 12.4 of the soil mixture. Most of the soils collected from South Dakota had a pH of 7.7 to 8.0. The only exception to this was Site 1 which had the lowest pH of 7.2. Sulfur content is linked with the formation of ettringite in sulfur rich soils, which can cause large expansion of soils under the right conditions. It is believed that soils with sulfur contents greater than about

3000 ppm are more susceptible to this condition. Three of the soils (from Sites 1, 4 and 6) had sulfate contents greater than 3000 ppm. A summary of the pH and sulfate content values for each of the sites is presented in Table 6.

Table 6: Summary of pH and Sulfate Content

	pH	Sulfate Content (ppm)
Site 1	7.2	4477
Site 2	8.0	19
Site 3	7.8	634
Site 4	7.7	3595
Site 5	7.9	140
Site 6	7.7	4175

7.1.4 Standard Proctor and Soaked CBR

The standard Proctor test was used to determine the maximum dry density and optimum moisture contents of each of the soils. This information was used to prepare the CBR tests to the proper moisture content and density. The information in Table 7 summarizes the results from these tests. All sites had similar optimum moisture contents, ranging only from 28 to 31 percent, and maximum dry density, ranging from 80 to 86 pcf. CBR tests conducted on soil prepared near OMC (±0.8 percent moisture) demonstrated all clay/shale samples had low strength, with CBR values less than 2.5 percent. Vertical expansion swell after soaking for 96 hours was measured with dial gauges. Site 3 had the lowest swell (0.9 percent) and is also the soil with the lowest LL and PI. Sites 2 and 5 had the greatest swell (2.8 percent).

Table 7: Summary of Standard Proctor and Soaked CBR Test

	Standard Proctor		***Soaked CBR***			
	γ_{dmax} (pcf)	w_{opt} (%)	γ_d (pcf)	w (%)	*Swell (%)*	*CBR*[1] (%)
Site 1	82.9	31.2	81.8	31.8	1.4	1.9
Site 2	82.5	28.1	81.5	28.9	2.8	0.7
Site 3	83.9	29.7	82.8	30.0	0.9	1.9
Site 4	85.7	30.0	85.1	30.5	1.5	2.2
Site 5	80.1	31.1	80.1	30.5	2.8	1.1
Site 6	84.0	29.2	84.0	30.1	1.8	2.3

[1] post soaking at 0.1 in. penetration

7.2 Free-Swell Tests

Due to the quantity of soil samples and the large number of tests that need to be executed to select the type of stabilizing agent and application rate, an optimization technique was employed to narrow down the type and rate based initially on swell, prior to analyzing effects on strength and plasticity. This was accomplished by first using a procedure to determine the unconfined swelling potential of soils inundated with water, based on the results of the Free-Swell Index using Indian Standard 2720, Part 40. Since this test was relatively simple to perform, many tests were conducted in a short amount of time to determine an initial range of appropriate application rates and stabilizing agent (or combination of agents).

Each Free-Swell Index test uses 20 grams of oven-dried soil that passes the #40 sieve (0.425 in. opening size) and places 10 grams each into two 100 mL graduated cylinders. One cylinder is filled with distilled water and the other with kerosene oil, both to 100 mL volume. The bonding behavior of the kerosene is non-polar, so it will not cause the soil to swell in the same way as water, and thereby provides a control. The mixtures are carefully agitated to thoroughly blend the contents and set on a stable surface to settle. After sufficient time (at least 24 hours), the volume of soils in the cylinders is measured to determine the volumetric change from expansion of the clay. The Free-Swell Index is determined using these volumes, based on Equation 1.

$$\textbf{Free Swell Index} = \frac{V_d - V_k}{V_k} * 100\% \qquad \textbf{Equation 1}$$

where, V_d is the volume of soil in the graduated cylinder filled with distilled water, and V_k is the volume of soil in the graduated cylinder filled with kerosene.

Multiple application rates of lime and fly ash (and combinations of both) were used to evaluate their effect on free-swell behavior. Application rates for lime were 3, 6 and 9 percent, and 10, 20 and 30 percent for fly ash. Zero percent was also used as a control, totaling 16 free-swell tests for each of the six test sites. The matrix of free-swell tests that was conducted on each of the six soils is outlined in Table 1. Mixtures containing lime and/or fly ash were blended in such a way that 10 grams of soil were always used and the total weight was increased by the amount of lime or fly ash. This was done to be able to directly compare the performance of the treated samples with the controls. The results from these tests are summarized in Table 8. Larger numbers indicate greater swell, and numbers greater than the controls indicates worse performance when compared to the untreated soils. Soils from sites 2, 3 4 and 6 treated with fly ash tended to swell more compared to their untreated controls. Graphical results from each of the test sites are summarized in Appendix A.

Table 8: Free-Swell Test Results

Lime (%)	Fly Ash (%)	Free-Swell (%)					
		Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
0	**0**	84.6	50.0	35.7	46.2	73.1	61.5
0	**10**	73.1	153.8*	48.0*	58.3*	60.0	66.7*
0	**20**	46.7	146.2*	40.0*	56.7*	53.3	56.7
0	**30**	43.8	137.0*	34.4	43.8	40.6	40.6
3	**0**	60.0	48.0	33.3	42.9	57.7	33.3
3	**10**	48.3	64.3*	34.4	37.5	46.7	50.0
3	**20**	43.3	63.3*	29.4	33.3	43.8	50.0
3	**30**	35.3	62.5*	27.0	28.2	44.1	52.9
6	**0**	40.0	50.0	11.7	20.7	37.9	23.3
6	**10**	39.4	63.3*	28.6	40.6	37.5	37.5
6	**20**	37.1	61.3*	27.8	32.4	33.3	31.4
6	**30**	36.8	55.9*	23.7	27.8	27.8	27.0
9	**0**	26.5	50.0	11.8	27.3	35.5	24.2
9	**10**	41.9	45.5	31.4	22.2	30.6	32.4
9	**20**	37.1	40.0	27.0	21.1	29.7	27.0
9	**30**	36.8	37.8	22.5	20.5	25.6	23.1

* increased swell when compared to the control (i.e., no treatment)

Further analysis of the free-swell data was made by incorporating cost of the various treatment options along with the performance based on swell reduction. The cost of each option was based on an arbitrary project that treated 750 tons of material to reduce swell – a reasonable project size in South Dakota for this application. In this case, cost was based on the purchase of the treatment additive and the construction of the embankment. A cost of $175/ton for lime and $80/ton for fly ash was used in the estimate, based on the cost information found on the GeoTechTools website for these treatment options (GTT, 2016). Cost values were normalized on a scale of 1 to 10 based on where they fell within that range. The highest and lowest costs received normalized values of 1 and 10, respectively. The lowest cost was $0, for the untreated case, and the highest cost was $29,813, for 9% lime plus 30% fly ash treatment. Likewise, the performance based on the treatment's ability to reduce swell was also normalized on a scale of 1 to 10. In this instance, the best performance received a score of 10 and the worst received a score of 1. All other performance values fell between these two endpoints based on a linear scale. The final value used to evaluate the best option was determined by multiplying these two numbers together (normalized cost value * normalized performance value) to create a cost-performance value (CPV), which has a theoretical range of 1 to 100, with lower numbers indicating poorer value and greater values indicating better value. The results of this analysis are summarized in Table 9. Treatment options that performed worse than the control (i.e., no additive) were eliminated from the analysis and are shown as blank cells in Table 9 (denoted as asterisks in Table 8).

Table 9: Cost-Performance Values Based on Free-Swell Test Results

Lime (%)	Fly Ash (%)	Cost-Performance Value*						Average CPV	Rank
		Site 1	Site 2	Site 3	Site 4	Site 5	Site 6		
0	**0**	10.0	10.0	10.0	10.0	10.0	10.0	10.0	15
0	**10**	22.8				28.5		25.7	10
0	**20**	43.8				30.3	13.6	29.3	7
0	**30**	33.4		6.9	8.4	32.7	26.9	21.7	12
3	**0**	42.4	21.9	16.7	19.0	34.5	67.0	33.6	4
3	**10**	46.4		10.5	28.3	42.1	25.9	30.6	6
3	**20**	38.3		17.4	28.5	34.1	19.2	27.5	9
3	**30**	29.2		14.4	24.7	21.9	10.2	20.1	13
6	**0**	60.3	7.6	76.2	75.8	58.5	75.8	59.0	1
6	**10**	46.5		21.3	17.1	45.0	38.5	33.7	3
6	**20**	33.4		15.9	23.4	34.2	32.2	27.8	8
6	**30**	18.4		12.0	16.3	21.0	19.9	17.5	14
9	**0**	64.3	6.4	64.1	49.1	52.3	62.6	49.8	2
9	**10**	35.2	20.2	12.0	43.5	41.9	36.2	31.5	5
9	**20**	23.5	23.6	12.0	27.6	25.9	25.5	23.0	11
9	**30**	8.4	10.0	5.9	10.0	10.0	10.0	9.1	16

* Greater cost-performance values indicate better alternatives (theoretical range = 1 to 100)
Blank cells indicate CPV < 0 due to increased swell when compared to the control (i.e., no additive)

To further understand these results, an average was calculated for each of the treatment options across all six sites. This provided a method to determine which treatment option worked best for most of the expansive soils found in South Dakota. The ranking on the far right of Table 9 is based on the average values. This analysis shows that the top five cost-performance value are associated with the following blends:

1. 6 percent lime, 0 percent fly ash (CPV = 59.0)
2. 9 percent lime, 0 percent fly ash (CPV = 49.8)
3. 6 percent lime, 10 percent fly ash (CPV = 33.7)
4. 3 percent lime, 0 percent fly ash (CPV = 33.6)
5. 9 percent lime, 10 percent fly ash (CPV = 31.5)

These results indicated that 6 percent lime was the most effective treatment option, on average, based on cost and swell reduction potential. The results also indicated that many of the treatment options for Site 2 produced greater swell than the Control, and that most of the swelling came from samples treated with varying levels of fly ash. Furthermore, samples treated only with fly ash tended to cause greater swell than those treated with only lime or a blend of lime and fly ash. Finally, all treatments (with the exception of the 9% lime + 30% fly ash treatment) worked better than the Control, which ranked 15th overall out of 16.

7.3 Standard CBR Tests

Standard soaked CBR tests were conducted on soils from each of the six sites in substantial accordance with ASTM D1883. It was decided in consultation with the technical panel that treated CBR tests for the project were to use 9 percent lime or 12 percent Class C fly ash to be consistent with what was done in the past on SDDOT projects. No blended treatments were used. All soils were pulverized to fit through a #4 sieve, brought to the target moisture content (either optimum moisture content or 2 percent above optimum, OMC or OMC+2%), and treated with either lime or fly ash. A photo of pulverized soil treated with lime is shown in Figure 43. Four new tests were conducted on the soil from each site, as listed below. Earlier CBR tests conducted on untreated samples prepared at optimum moisture content were also compared to the results from treated samples.

- 9 percent lime at OMC
- 9 percent lime at OMC+2%
- 12 percent fly ash at OMC
- 12 percent fly ash at OMC+2%

A standard Proctor hammer was used to compact the samples. Compacted soil samples were soaked immediately after compaction. Soaking lasted 96 hours and total swell was measured using a dial gauge (Figure 44). Percent swell was determined from these measurements and a summary of these values is provided in Table 10. CBR tests were conducted on each sample immediately after the soaking period, the results of which are summarized in Table 11. Post-soaking moisture content measurements were made from the top inch of soil. CBR was determined from penetrations of 0.1 in. and 0.2 in. The values of CBR at 0.2 in. were oftentimes greater than the values at 0.1 in. indicating that this result was not an anomaly for these types of soils. The greater of these two CBR values are the ones reported and used in the analysis.

Figure 43: Pulverized, lime-treated soil prepared for CBR test.

Figure 44: Compacted CBR soil samples in soaking tank.

Table 10: Swell Results from CBR Tests Conducted on Finer Particle Sizes

Treatment/Moisture	Swell (%)					
	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
Control @ OMC	1.4	2.8	0.9	1.5	2.8	1.8
9% Lime @ OMC	2.4	1.8	0.2	1.8	0.9	0.3
9% Lime @ OMC+2%	0.1	0.7	0.1	0.7	0.1	0.1
12% Fly Ash @ OMC	3.1	5.4	0.3	5.4	5.3	5.2
12% Fly Ash @ OMC+2%	1.4	4.4	0.2	4.4	3.1	3.6

Table 11: Strength Results from CBR Tests Conducted on Finer Particle Sizes

Treatment/Moisture	CBR (%)					
	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
Control @ OMC	1.9	0.7	1.9	2.2	1.1	2.3
9% Lime @ OMC	10.5	2.9	15.9	15.3	7.8	5.7
9% Lime @ OMC+2%	10.8	6.1	21.4	10.6	12.0	8.1
12% Fly Ash @ OMC	7.1	3.1	14.7	12.2	2.4	3.0
12% Fly Ash @ OMC+2%	9.0	9.0	12.9	7.1	6.4	4.5

Summary of swell results from soaked CBR tests, referring to the data presented in Table 10:

- Generally less swell from 9% lime at OMC when compared to the control (with the exception of Sites 1 and 4)
- Always less swell from 9% lime at OMC+2% when compared to the control and 9% lime at OMC
- Generally <u>more</u> swell from 12% fly ash at OMC when compared to the control (with the exception of Site 3)
- Mixed results when comparing swell from 12% fly ash at OMC+2% when compared to the control, but always less swell when compared to 12% fly ash at OMC
- Always less swell from samples prepared at OMC+2% when compared to samples prepared at OMC

Summary of CBR strength results from soaked CBR tests, referring to the data presented in Table 11:

- Always greater strength from 9% lime at OMC or 12% fly ash when compared to the control
- Strength gains for 9% lime were greater than for 12% fly ash (with the exception of Site 2)
- Generally greater strength from samples prepared at OMC+2% when compared to samples prepared at OMC (with the exception of Site 4 for lime and Sites 3 and 4 for fly ash)

7.4 Large-Scale CBR Tests

Large-scale soaked CBR tests were designed and conducted to allow for larger particle sizes during the mixing, mellowing, and testing of the clay samples from the six test sites. Three large-scale CBR tests were conducted on each soil: a control (i.e., no treatment), 9 percent lime, and 12 percent Class C fly ash. All samples were prepared at optimum moisture content plus 2 percent based on the results from the standard CBR tests. Particle sizes were not manipulated from what was provided naturally from each of the sites. Particle sizes prior to moisture conditioning and treatment were generally less than about 2 in. These particles partially disintegrated as they were prepared for testing, resulting in particle sizes around ¾ in. to 1 in. These larger particle sizes are more indicative of what is expected in the field after the mixing process. The methodology and results of these tests are described below.

The large-scale CBR tests generally followed the lab-scale CBR test procedure, other than two primary differences: 1) the maximum allowable size of the particles, and 2) the size of the mold. Three large molds were fabricated to accommodate a 12-in. diameter sample having a final compacted depth of 8 in. The 8-in. depth was selected to simulate anticipated depths of compacted layers during construction. The molds were made of ⅛-in. thick steel pipe mounted to a ⅜-in. thick steel base. The base was perforated with $^{3}/_{32}$-in. holes on a ¾-in. grid spacing to allow water infiltration during the soaking period. All-thread rods were welded to the baseplates to accommodate ears mounted to the sides of the mold used to hold the mold in place during compaction and soaking. A 4-in. collar was used to facilitate compaction of final lift. A comparison of the relative size difference between the standard and large-scale CBR molds is shown in Figure 45.

Figure 45: Photo of standard 6-in. CBR mold compared to the large-scale 12-in. CBR mold.

Two hundred pounds of virgin material was prepared for each site. Preparations began by thoroughly mixing the soils to 2 percent above optimum moisture content. Sufficient moisture was added to account for the additional dry material added (lime or fly ash) during the treatment process. In other words, final moisture contents were 2 percent greater than optimum based on the total mass of dry materials (soil plus lime or fly ash). Mixing was done in a large pan using a hard-tined garden rake and hoe. Typical particle sizes after mixing is shown in Figure 46. Soils were mixed for 5 minutes to blend in the lime or fly ash stabilizer. Soils mixed with lime and fly ash are shown in Figure 47 and Figure 48, respectively.

Figure 46: Typical particle sizes after the moisture conditioning process.

Figure 47: Close-up of soil mixed with 9% lime.

Figure 48: Close-up of soil mixed with 12% fly ash.

Compaction of the soil into the larger molds was done using a modified Proctor hammer (Figure 49). Using Equation 2, it was determined that 87 blows and five layers were necessary to achieve the same total compactive energy as the standard CBR test (12,400 ft-lbf/ft^3). Filter paper was placed on bottom and top of the sample to prevent migration of soil particles during soaking. Several water content measurements were taken throughout the entire process to ensure proper moisture levels. After compaction, the soil samples were allowed to mellow for 24 hours in a sealed plastic container that had a shallow layer of water in it. The compacted samples were elevated above the water so no infiltration into the compacted sample took place during the mellowing period. Linear variable displacement transducers (LVDTs) were used to monitor swell and/or shrinkage on some of the samples (Sites 2, 3, 4 and 6) during the mellowing period (see Figure 50). These measurements generally showed very little movement, with the greatest being less than a quarter percent (negative values indicate percent shrinkage, positive values indicate percent swell).

$$\boldsymbol{Compactive\ Energy} = \frac{\left(\frac{\#of\ blows}{layer}\right)*(\#of\ layers)*(wt.of\ hammer)*(drop\ height)}{Volume\ of\ mold} \qquad \textbf{Equation 2}$$

Figure 49: Compaction of soil into large-scale CBR molds.

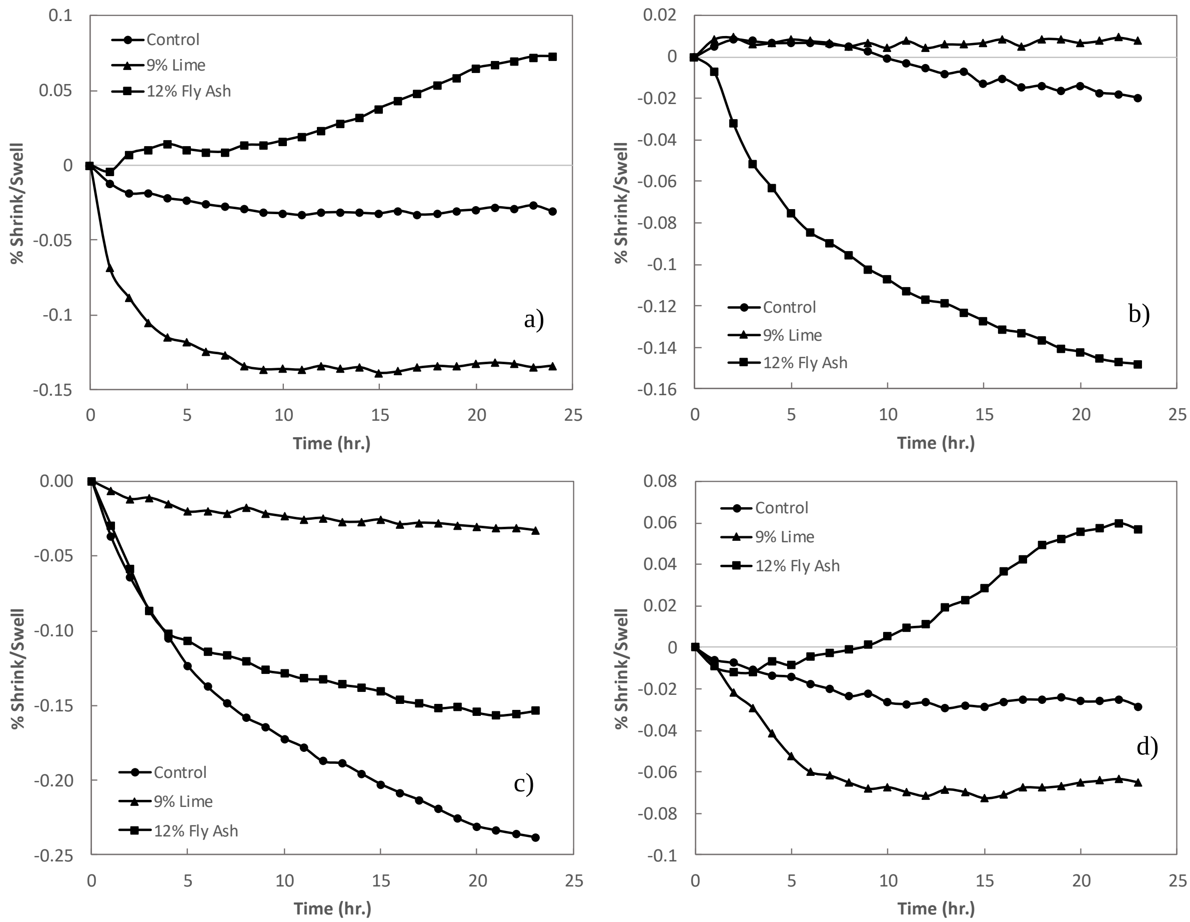

**Figure 50: Percent shrink/swell during 24-hour mellowing period for a) Site 2, b) Site 3, c) Site 4,
and d) Site 6 (negative values indicate shrinkage, positive values indicate swell).**

All the compacted samples were soaked for 96 hours immediately after the mellowing period by filling the plastic bins with water (setup is shown in Figure 51). An equivalent pressure to what was used for the standard CBR tests (0.354 psi from a 40-lb. weight) was used on the upper platen during the soak period. The final values of swell (after 96 hours of soaking) are summarized in Table 12. Daily measurements of swell were also made during the soaking period using dial gauges. The results of these measurements are summarized in Figure 52, and indicate that the swell was usually complete after one to three days of soaking.

Figure 51: Setup for large-scale CBR tests during the 96-hour soak period.

Table 12: Swell Results from Large-Scale CBR Tests Conducted on Coarser Particle Sizes

Treatment/Moisture	Swell at 96 Hours (%)					
	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
Control @ OMC+2%	3.6	2.3	0.6	1.3	2.0	2.1
9% Lime @ OMC+2%	1.8	0.6	0.2	0.3	0.1	1.3
12% Fly Ash @ OMC+2%	2.3	0.4	0.04	0.2	4.3	5.7

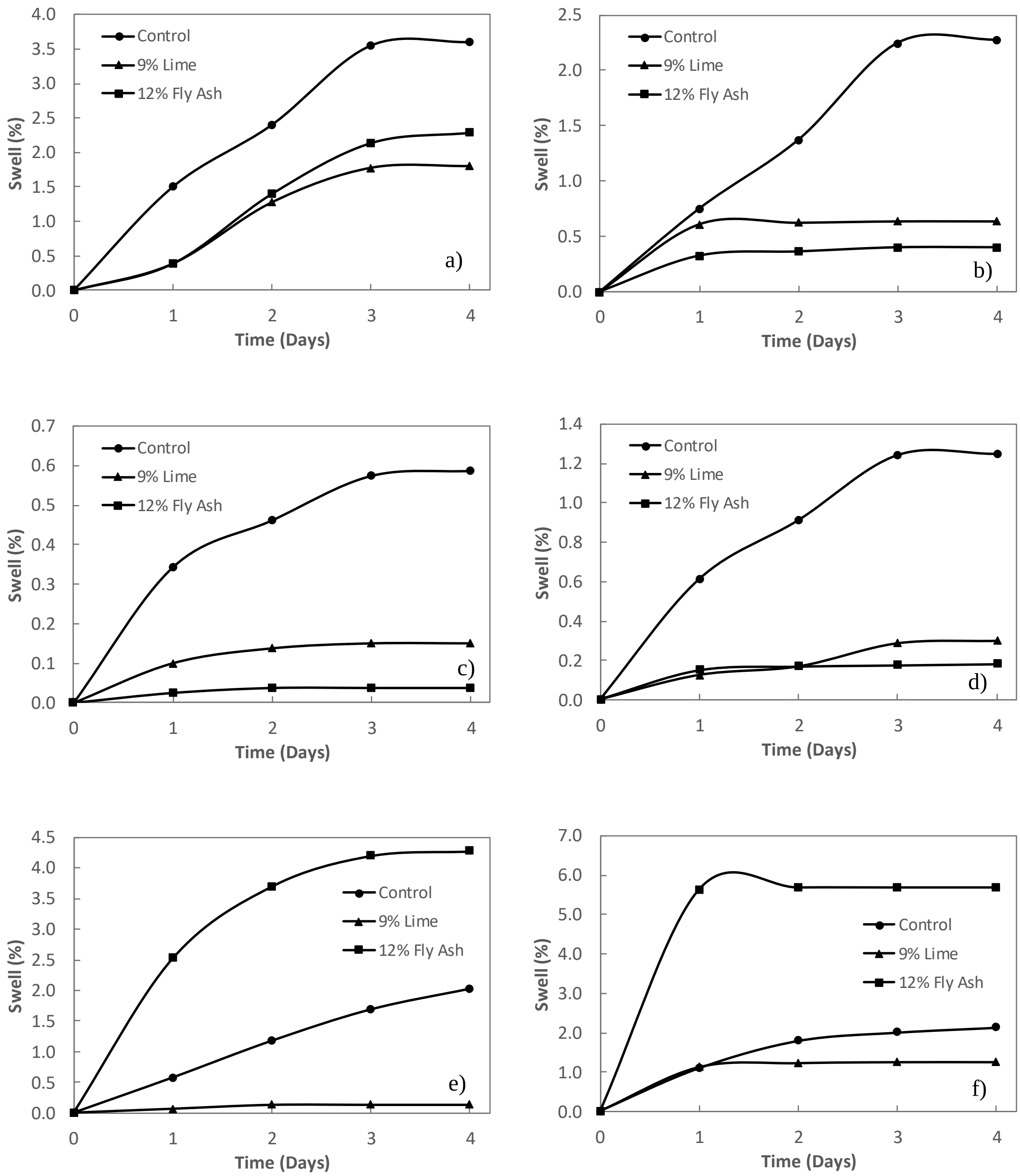

Figure 52: Swell during 96-hour soaking period for a) Site 1, b) Site 2, c) Site 3, d) Site 4, e) Site 5, and f) Site 6.

Due to the larger size of the large-scale CBR tests, multiple bearing tests (in this case, three) could be conducted on a single sample (see Figure 53). The values of the three bearing tests were averaged together and reported as a single value, as summarized in Table 13. Because of the large size of these samples, a hand-crank device was used to deliver the 0.05 in./min. penetration rate for the plunger (see test setup in Figure 54). A ⅛-in. thick steel plate, having a hole to accommodate the CBR plunger, was used on top of the soil surface to distribute the normal stress evenly across the entire soil surface and prevent upheaval of soil during testing. A 40-lb. load was place on this plate during the bearing test to provide the same normal stress as the standard CBR tests.

Figure 53: Photograph of the soil surface after CBR testing large-scale sample.

Table 13: Strength Results from Large-Scale CBR Tests Conducted on Coarser Particle Sizes

Treatment/Moisture	CBR (%)					
	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
Control @ OMC+2%	1.8	2.1	2.6	2.5	1.2	1.1
9% Lime @ OMC+2%	10.3	14.0	19.0	13.7	14.0	9.7
12% Fly Ash @ OMC+2%	9.7	12.7	10.6	8.7	2.5	2.8

Figure 54: Testing setup for large-scale CBR tests.

Summary of swell results from soaked CBR tests, referring to the data presented in Table 12 for the large-scale samples prepared at OMC+2%:

- Less than a quarter percent of swell or shrink during the mellowing period
- The swell was usually complete after one to three days of soaking
- Always less swell from samples treated with 9% lime when compared to the control
- Generally less swell from samples treated with 12% fly ash when compared to the control (with the exceptions of Site 5 and Site 6, which experienced significantly greater swell than the control)
- Mixed results when comparing swell from coarser particles treated with 9% lime and 12% fly ash when compared to treated finer particles

Summary of CBR strength results from soaked CBR tests, referring to the data presented in Table 13 for the large-scale samples prepared at OMC+2%:

- Always greater strength from 9% lime or 12% fly ash when compared to the control
- Strength gains for 9% lime were always greater than for 12% fly ash
- Generally greater strength from coarser particles treated with 9% lime when compared to finer particle treated with 9% lime (with the exceptions of Site 1 and Site 3)

- Mixed results when comparing strength from coarser particles treated with 12% fly ash when compared to finer particles treated with 12% fly ash

7.5 Summary of Laboratory Testing

The main results of the laboratory testing are summarized in Table 14, which lists the general material characteristics of the soil from each of the six sites, free-swell data, and swell and strength data from standard and large-scale CBR tests.

Table 14: Summary of Relevant Results from Laboratory Tests

	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6	Information Source
General Characteristics							
LL	82	86	68	72	100	78	Table 4
PI	41	37	33	40	61	39	Table 4
Clay fraction (%)	61	47	58	55	50	66	Table 5
Activity	0.7	0.8	0.6	0.7	1.2	0.6	Table 5
Sulfate Content (ppm)	4477	19	634	3595	140	4175	Table 6
$\gamma_{d,max}$ (pcf)	82.9	82.5	83.9	85.7	80.1	84.0	Table 7
w_{opt} (%)	31.2	28.1	29.7	30.0	31.1	29.2	Table 7
Free-Swell							
Control (%)	84.6	50.0	35.7	46.2	73.1	61.5	Table 8
6% Lime (%)	40.0	50.0	11.7	20.7	37.9	23.3	Table 8
9% Lime (%)	26.5	50.0	11.8	27.3	35.5	24.2	Table 8
10% fly ash (%)	73.1	153.8	48.0	58.3	60.0	66.7	Table 8
Standard CBR Swell at 96 Hours							
Control @ OMC (%)	1.4	2.8	0.9	1.5	2.8	1.8	Table 10
9% Lime @ OMC+2% (%)	0.1	0.7	0.1	0.7	0.1	0.1	Table 10
12% Fly Ash @ OMC+2% (%)	1.4	4.4	0.2	4.4	3.1	3.6	Table 10
Large-Scale CBR Swell at 96 Hours							
Control @ OMC+2% (%)	3.6	2.3	0.6	1.3	2.0	2.1	Table 12
9% Lime @ OMC+2% (%)	1.8	0.6	0.2	0.3	0.1	1.3	Table 12
12% Fly Ash @ OMC+2% (%)	2.3	0.4	0.04	0.2	4.3	5.7	Table 12
Standard CBR Strength							
Control @ OMC	1.9	0.7	1.9	2.2	1.1	2.3	Table 11
9% Lime @ OMC+2%	10.8	6.1	21.4	10.6	12.0	8.1	Table 11
12% Fly Ash @ OMC+2%	9.0	9.0	12.9	7.1	6.4	4.5	Table 11
Large-scale CBR Strength							
Control @ OMC+2%	1.8	2.1	2.6	2.5	1.2	1.1	Table 13
9% Lime @ OMC+2%	10.3	14.0	19.0	13.7	14.0	9.7	Table 13
12% Fly Ash @ OMC+2%	9.7	12.7	10.6	8.7	2.5	2.8	Table 13

8.0 BENEFIT/COST ANALYSIS

Benefit/cost analyses are often used to compare the overall effectiveness of a variety of improvement options. The decision whether to use stabilizing agents to mitigate expansive soils should be based on whether the cost for materials and construction yields tangible and worthwhile benefits. Benefits of chemical stabilization include reduced swell, increased strength, improved ride quality, increased longevity, reduced maintenance, and more. It was understood, based on input from the technical panel, that improvements to the strength of the subgrade were secondary and may not be used to modify the structural design. Therefore, for the purposes of this research, benefits were only quantified in terms of the effectiveness of each mitigation to reduce swell. Other benefits such as reduced maintenance, increased life, and improved serviceability are related to the ability of the stabilizing agent to mitigate potential soil expansion.

8.1 Benefit

Benefits are determined based on the results of the laboratory tests conducted on soils that have been treated with additives to reduce swell potential, as compared to a control where no mitigation is used. In this way, benefit can be calculated as a percent reduction in swell as compared to the control, as summarized in Equation 3:

$$\%SR = \frac{\%S_{control} - \%S_{treated}}{\%S_{control}} * 100\% \qquad \text{Equation 3}$$

where,

$\%SR$ is the total percent swell reduction,

$\%S_{control}$ = is the percent swell of the control sample (i.e., no treatment), and

$\%S_{treated}$ is the percent swell from a treated sample.

8.2 Costs

The cost of each treatment option was based on the purchase of the treatment additive and processing these materials into the subgrade soils that make up the embankment. A cost of \$175/ton to purchase lime and \$80/ton to purchase Class C fly ash was used in the estimate, based on the cost information found on the GeoTechTools website for these treatment options. Processing costs listed on the GeoTechTools website (from Colorado) indicated that it cost approximately \$2.29/yd^2 to process lime and \$2.18/yd^2 to process fly ash into the soil (GTT, 2016). Using this information, a benefit/cost ratio could be determined to compare multiple options to one another. Costs from SDDOT bid documents should be used in future benefit/cost analyses done by the department.

8.3 Benefit/Cost Ratio

A ratio of the benefits to the costs can be calculated to compare treatment options to one another. Alternatively, comparisons can also be made between chemical treatment and other design options such as undercutting and backfilling with granular fill. The equation is simply the benefit divided by the cost. In and of itself, the value that results from this ratio is not meaningful other than to directly compare two or more alternatives to one another. As benefits increase and/or costs decrease, the ratio becomes greater indicating better performance. Likewise, as benefits decrease and/or costs increase, the ratio becomes smaller, indicating that a particular option is less beneficial.

8.4 Example Calculation

An arbitrary yet realistic example is used to illustrate the benefit of this technique. In this example, 750 cubic yards of material are to be mitigated to prevent damage to the roadway due to swelling. The design indicates that 12 in. of subgrade should be processed to effectively mitigate this problem. Based on the Proctor results, the dry unit weight of the clay/shale is 85 pcf at 95 percent of maximum dry unit weight. Two alternatives are compared: 9 percent lime and 12 percent Class C fly ash. The swell results from large-scale CBR tests are used to provide realistic data to directly compare the benefits of lime and fly ash treated samples to a control, where no treatment was used. The steps in the process are:

1. Calculate the total weight of each additive needed to treat the soil.
2. Calculate the cost to purchase each additive (in dollars per square yard).
3. Calculate cost to process each additive into the soil (in dollars per square yard).
4. Determine the total cost of each treatment option, the addition of items 2 and 3 (in dollars per square yard).
5. Use swell results from soaked CBR tests to determine benefit (percent swell reduction).
6. Calculate benefit/cost ratios by dividing item 5 by item 4.
7. Make comparisons between the various alternatives and select the alternative with the greatest benefit/cost ratio.

The amount of additive materials needed (in tons) can be determined using Equation 4:

$$W_{add} = Vol * \gamma_d * R_{add} * \left({}^{27}/_{2000}\right) \qquad \text{Equation 4}$$

where,

W_{add} = the weight of the additive needed to treat the soil (in tons),

Vol = the volume of soil to be treated (in cubic yards),

γ_d = the dry unit weight of the compacted soil (in pcf),

R_{add} = the application rate of the treatment (in decimal), and

(27/2000) = the conversion factor to convert the result into tons.

Using Equation 4, the weight of lime needed to treat 750 cubic yards of soil compacted to a dry unit weight of 85 pcf and treated at 9 percent is <u>77.5 tons</u>. The weight of Class C fly ash needed to treat the same soil at 12 percent is <u>103.3 tons</u>.

A cost of $175/ton for lime and $80/ton for fly ash is used in this example, based on cost information found on the GeoTechTools website for these treatment options (GTT, 2016). Using this information, the cost of purchasing 77.5 tons of lime is $13,562.50, and the cost of 103.3 tons of fly ash is $8,264.00, which translates into costs of <u>$6.03</u> and <u>$3.67</u> per square yard, respectively (based on 2,250 square yards of area, that is, 750 cubic yards of material processed to 1 ft. depth). The cost of processing lime and fly ash were also found on the GeoTechTools website. Based on cost data collected from Colorado DOT, processing lime costs $2.29 per square yard and processing fly ash costs $2.18 per square yard. The slightly larger cost of processing lime may have to do with the additional passes necessary to thoroughly blend the material. Altogether, the total cost of lime treatment is <u>$8.32 per square yard</u> and the total cost of fly ash treatment is <u>$5.85 per square yard</u> (not including other construction costs such as mobilization, etc., assuming that these costs would be the same between the two alternatives).

Benefits are based on the results of the large-scale CBR tests conducted as part of this research and summarized in Table 15. Using this information, the swell reduction for each treatment type and soil source was calculated. The swell reduction (benefit) was divided by the cost per square yard (cost) for the treatment options. The results of the benefit/cost analysis for each of the six sites in South Dakota are summarized in Table 15.

Table 15: Results of Example Benefit/Cost Calculations

Site	% Swell @ 96 Hours			Swell Reduction (%)		Benefit/Cost Ratio	
	Control	9% Lime	12% Fly Ash	9% Lime	12% Fly Ash	9% Lime	12% Fly Ash
Site 1	3.6	1.8	2.3	50	36	6.0	6.2
Site 2	2.3	0.6	0.4	74	83	8.9	14.1
Site 3	0.6	0.2	0.04	67	93	8.0	16.0
Site 4	1.3	0.3	0.2	77	85	9.2	14.5
Site 5	2.0	0.1	4.3	95	-115†	11.4	-19.7†
Site 6	2.1	1.3	5.7	38	-171†	4.6	-29.3†

† negative numbers indicate worse performance (i.e., increased swell) of treated samples when compared to the Control

The analysis indicates that the fly ash treatment at 12 percent is more cost beneficial when compared to 9 percent lime treatment for the soils at Sites 1, 2, 3 and 4. Fly ash treatment of the soils at Sites 5 and 6 caused the soils to swell more than the control resulting in a negative benefit and consequently a negative benefit/cost ratio. In these cases, the 9 percent lime treatment was more cost beneficial.

9.0 CONSTRUCTION GUIDELINES AND SPECIFICATIONS

Construction guidelines and specifications are outlined based on the results of the laboratory testing and the literature review, which included information from other state specifications and guidebooks. Final parameters of interest are outlined and flowcharts are shown to help guide planning, construction, and rehabilitation of highways over expansive soils.

The goals of chemical treatments applied to the subgrade are to reduce or control swelling in expansive soils, increase strength of the existing subgrade, and/or improve the construction platform by drying out wet areas. Distresses caused by expansive soils tend to manifest in one of two primary ways. The first type of surface distress from expansive soils consists of larger humps and/or abrupt shifts in the surface followed by a relatively smooth transition back to the original grade. These distresses are primarily the result of geologic faults within the clay/shale formation that have shifted due to 1) reduction in overburden due to excavation, and/or 2) exposure of the fault gouge to water from the ground surface. These types of distresses are the primary focus of this research. The second type of distress from expansive soils is evident as less severe yet somewhat frequent undulations with relatively smooth transitions into and out of the distressed zone, negatively affecting ride quality. Flexible pavements are better able to absorb these changes when compared to rigid pavements, which tend to concentrate the movement at the joints or form cracks. These surface distresses are primarily caused by differential expansion of clay/shales along a roadway due to variable water inundation and soil properties and/or sensitivities to water, and tend to affect ride quality without necessarily posing safety hazards. These two types of distresses, coincident with the way they manifest themselves on the roadway, are the basis of the guidelines presented below.

Heaving of faults in South Dakota is the result of differential vertical shifting of discontinuous clay/shale deposits. In this case, movement is concentrated along weaker slip planes between various natural clay/shale deposits common in South Dakota. This movement can be initiated due to release of overburden pressures from excavation (e.g., borrow areas, grade excavations, cut slopes), water inundation along the slip plane, or other geologic events. Differential movement along the slip plane causes breakdown of the clay/shales on either side of the fault gouge. Wetting and drying of the fault gouge causes heaving and contracting, respectively, further disintegrating the soil particles, which are more susceptible to water and swelling. Fault zones can extend into the ground several feet, and the depth depends on several geologic and geotechnical factors, such as soil makeup, stress state, natural fractures and fissures present in the clay/shale formation, etc. Depth of the fault zone may also be related to the number of wet/dry cycles it has experienced, and the amount of water present. Several sites that were visited along South Dakota highways as part of this research had shallow drainage paths due to the flat topography. Standing water at these sites caused the fault gouges to be inundated with water for longer periods of time. Leveling the bumps that form because of the differential swelling will provide short-term drainage benefits; however, because additional overburden will be removed, which will decrease confining stress in the fault area, it will only be a matter of time until the fault takes on water and reinitiates the swelling process. The effect and presence of water must be reduced at the fault to adequately address this problem.

The effect of water can be controlled using a soil treatment such as lime or fly ash. The presence of water under and adjacent to the roadway can be controlled using proper drainage (e.g., adequate slope, edge drains, etc.) and/or by installing a protective barrier to keep surface water from infiltrating into the fault gouge.

The following construction guidance can be used in areas where known or anticipated fault gouges are likely to cause damage to highway infrastructure due to expansion, such as the typical section depicted in Figure 18 and Figure 19 in Section 6.1.

1. Excavate trench centered on the fault trace. The depth of the excavation depends on how deep the gouge extends into the earth. This is evident by the severity of the weathering of the soils within the fault gouge. Excavations should be made deep enough to remove most of the disintegrated and friable slip plane material within the fault gouge, usually 2 to 6 ft. The width of the excavation should be sufficient to provide construction access.
2. Replace the materials within the excavation beginning with compacted layers treated with lime or fly ash to reduce swell potential. The type of treatment and mixing rate can be determined using the procedure outlined in Section 9.2. The final mixture of materials, including lime or fly ash, should be compacted 2 percent wet of optimum. Open graded backfills should not be used atop the fault zone as it will allow water to infiltrate into the area causing future wetting of the fault zone and subsequent expansion.
3. Use an impermeable membrane to keep water from migrating into the fault area. It is recommended that the membrane extend sufficient distance around the fault zone and be keyed on either side a depth of approximately 2 ft. deeper than the initial excavation coincident with the fault area. Drains should also be installed to keep water from infiltrating into the fault area.
4. Final grading of the borrow areas adjacent to the fault should be uniform as to not cause any water ponding or accumulation within this zone.

In areas that are prone to movement or are predicted to move based on the results of laboratory tests but have not experienced more severe or abrupt displacements at fault joints, subgrade materials should be worked and treated to prevent or slow differential movements along the highway surface. This will help slow the expansive nature of the soil that will be exposed during the construction process as overburden stresses are released. Decreasing the expansive nature of the soil in general is accomplished by using treatments such as lime and/or Class C fly ash.

9.1 Lime and Fly Ash Stabilization

Lime and Class C fly ash are commonly used to impede the effects of water on expansive soils. Geosynthetics are also beneficial in this application. The information in this section provides guidance on how to effectively select the proper stabilization method and incorporate it into the construction sequence.

9.1.1 Lime Stabilization

There are several existing methods to determine the amount of lime to use as soil stabilization. The method utilized as part of this research made use of the Free-Swell Index test using Indian Standard 2720, Part 40. Another common method is ASTM D6276 (Standard Test Method for Using pH to Estimate the Soil-Lime Proportion Requirement for Soil Stabilization), where the amount of lime is determined by calculating the lowest rate of lime treatment needed to bring the pH of the soil to 12.4.

There are various types of lime available for soil stabilization purposes: quicklime (a dry material made of calcium oxide (CaO), usually in powder form but can be in pebble form), hydrated lime (a dry powdered material made of calcium hydroxide, $Ca(OH)_2$), or lime slurry (a liquid mixture of hydrated lime solids and water). Lime slurry mixes can be advantageous during windy conditions, where the liquid slurry is distributed on site using a spray bar system similar to a water truck. The chemical properties of lime can be verified using the following two ASTM test standards ASTM C25 (Standard Test Methods for Chemical Testing of Quicklime, Hydrated Lime, and Limestone), and ASTM C110 (Standard Test Methods for Physical Testing of Quicklime, Hydrated Lime, and Limestone). Common guidelines include the total active lime content, unhydrated lime content, and free water content (ASTM C25) and particle size based on wet or dry sieve methods (ASTM C110).

Distribution of the lime on the ground surface should be monitored so that the proper amount of material is used. Too little lime will result in less effective treatment, too much will result in wasted material and possibly the need for additional water for hydration purposes. Depending on the type of lime and distribution method used, a monitoring method can be employed to verify application rates. Sampling the amount (mass) of lime over a known area is an effective and simple method to confirm quantities.

Mixing the lime and soil should be done in a way that breaks down the materials and distributes the lime throughout the entire depth being mixed. Typical depths of lime treatment are 6 to 12 inches, depending on the equipment available to mix and prepare the soils. The depth and mixing capabilities of the mixing equipment should be specified in the construction plans. Pulverization and mixing equipment should be able to grind and crush soils uniformly to the specified depth across a level surface. It is advantageous to mix water into this soil mixture as it is being processed to ensure that the lime has sufficient water to hydrate. It is common to make at least two passes with a rotary mixer over the lime treated area to ensure proper mixing and particle size breakdown. Only apply lime treatment in areas that can be completely mixed within the same day. Suggestions for particle size (based on information from the Kansas Department of Transportation) are 95 percent passing the 1.5 in. sieve and 40 percent passing the #4 sieve. A phenolphthalein solution can be used to verify the presence of lime within the soil, but is not suitable to determine the specific amount. A uniform blend of lime treated soil will have similar moisture content, color and consistency across the construction site.

Light compaction should be applied to “seal” lime treated soils after mixing. This protects the lime treated soils from excessive oxidation and carbonation (a chemical process that occurs when carbon dioxide from the atmosphere combines with lime to form calcium carbonate, which causes the lime to be less effective). A mellowing period of 24 to 72 hours should follow the initial compaction to allow the lime to react with the clay prior to final mixing, grading and compaction. Quicklime generally requires less mellowing time than hydrated lime. After mellowing, the soil should be remixed once again, and brought to a moisture content of 2 percent above optimum for the entire treated soil mixture. Compaction should be done with a sheepsfoot/padfoot roller, potentially followed by a smooth drum roller or lightly back-dragged with blade, and graded to meet specified elevations. It is at this second mixing stage that other treatments (e.g., cement, fly ash) be added, if desired. Final compaction after grading is generally done with a smooth drum roller.

9.1.2 Class C Fly Ash Stabilization

Similar to lime, Class C fly ash also acts as a water reducer and may also satisfy negative charges on clay particles; however, its primary benefit to expansive soils is that it cements the soil particles together to increase strength and helps create a matrix of soil particles that are larger and less susceptible to water effects. Fly ash is always applied in a dry state; therefore, it may be necessary to restrict installation during windy conditions so that it isn’t blown away during construction. Fly ash must be kept dry to prevent hydration from taking place prior to mixing with the soil, so provisions must be made to prevent rain, etc. from wetting the mixture prematurely. Mixing fly ash with the soil and water should be done relatively quickly to control the pozzolanic reactions (the chemical reactions associated with cementitious materials) during the construction process. These reactions are rapid, and bonds formed through this process should not be broken once they are created. Otherwise, the effectiveness of the fly ash treatment will be reduced. Optimally, mixing and compaction of fly ash treated soils should be done within one hour, but not more than two hours. No mellowing period is required for fly ash so grading and reshaping should begin within the one to two-hour timeframe. Grade corrections after mixing and compaction should be minimized (ACAA, 2003). Distributing, measuring and mixing fly ash should be done in a similar manner to lime.

The following two ASTM test methods can be used to characterize the properties of fly ash: ASTM D5239 (Standard Practice for Characterizing Fly Ash for Use in Soil Stabilization) and ASTM D7762 (Standard Practice for Design of Stabilization of Soil and Soil-Like Materials with Self-Cementing Fly Ash).

9.1.3 Geosynthetic Stabilization

Geosynthetics may also prove to be beneficial in this application. Geogrids and geotextiles placed properly in the embankment stratigraphy will help reduce stress concentrations, more evenly distribute water infiltration, stabilize weak subgrades, separate finer grained soils from engineered fill, provide stability during construction on weaker soils, and even wick water away from water-rich zones. Fine-grained soils such as silts and clays tend to migrate upward into the base coarse layer under traffic loads when wet. Geotextiles are commonly used as barriers to maintain the integrity of the structural base coarse over time by preventing this migration. Geogrids and woven geotextiles can additionally provide structural stability and stiffness to weak ground through high strength interaction with the gravel fill. Specialized geotextiles can also be used to wick water away from the roadway, directing it toward drainage features. It is highly recommended to use geosynthetics in this application to provide added stability and longevity to the road structure. Holtz et al. (2008) can be referred to for guidance on proper design, construction and specification of geosynthetics.

9.2 Flow Chart

A step-by-step process is outlined below and summarized in a flow chart in Figure 55 to guide practitioners on how to adequately identify and treat expansive soils in transportation applications using lime and fly ash. The information contained throughout this document is geared toward providing the practitioner with sufficient knowledge to judge whether this is the case. If current distresses evident in the road surface are indicative of expansive soil issues, then this analysis may be unnecessary; however, if during the investigative stage of road design and planning, it is desired to proactively address any potential problems associated with expansive soils, then the following steps should be followed.

The lighter, blue rectangular boxes in Figure 55 coincide with the four main steps outlined below. The yellow diamonds indicate decision points, the darker blue rectangles with rounded corners indicate where an alternative or additional step is needed, and the two green ovals are terminations to the process.

Step 1 – Predict Soil Swell Potential

The first step in the process is to conduct relatively simple and quick laboratory tests to determine whether or not there is potential for damage to the roadway infrastructure from expansive soils. If there is a history of damage from expansive soils, then a shortcut in the process may be used to bypass laboratory tests. Otherwise, soil swell potential may be determined using Atterberg limits, the free-swell test (other tests may be substituted to determine swell potential based on the experience and preference of the engineer).

Determine the percent fines, liquid limit, plasticity index and/or free-swell properties of the soil. If the fines content is greater than 25 percent, the liquid limit is greater than 60, and the plasticity index is greater than 35, there is a high chance that the soils will exhibit expansion when inundated with water (marginal expansion is expected for soils with liquid limits between 50 and 60 and plasticity indices between 25 and 35). Another alternative is to conduct a free-swell test on a virgin sample of the soil. If the free-swell index is greater than 40 percent, the soil is considered to exhibit sufficient swell characteristics as to warrant mitigation, based on the results of free-swell tests conducted as part of this study. If either of these criteria are met (fines > 25% and LL > 60 and PI > 35, <u>or</u> fines > 25% and free-swell index > 40%), continue to Step 2 to determine swell and strength properties. Otherwise, the soil poses relatively low risk of expansion.

Step 2 – Determine Optimum Chemical Treatment

Conduct soaked CBR tests (ASTM D1883) on samples that have been treated with lime and/or fly ash and on untreated samples to determine swell and strength characteristics. Lime and fly ash treatment levels for expansive soils in South Dakota were most beneficial at around 6-9 percent for lime and around 12-20 percent for fly ash, so these are good initial treatment values to evaluate. A list of potential sub-steps is provided below as an example of how to determine the optimum chemical treatment method.

a) Conduct soaked CBR tests on soil samples prepared at the following five treatment types and levels: no treatment (control), 6% lime, 9% lime, 12% Class C fly ash, and 15% Class C fly ash. The number and level of initial treatment levels should be determined based on the experience of the engineer. All samples should be prepared at 2 percent above optimum moisture content (taking into consideration the additional dry mass from lime or fly ash treatments), as determined by the Standard or Modified Proctor test (ASTM D698 or ASTM D1557, respectively). Assuming a dry treatment is selected, Equation 5 can be used to determine the weight of additional water that needs to be added to bring the soil to the specified final water content, taking into consideration the additional weight of the dry treatment.

$$W_{w,add} = \left(w_f - w_n\right) V_t \gamma_d + w_f W_{treat} \qquad \text{Equation 5}$$

where:

$W_{w,add}$ = weight of additional water needed to reach target water content {lb}

w_f = final water content of entire mixture (likely OMC+2%), in decimal

w_n = natural water content of soil prior to construction, in decimal

V_t = total volume of soil to be treated {ft^3}

γ_d = target dry unit weight of soil to be treated {lb/ft^3}

W_{treat} = weight of soil treatment {lb}

b) Soak samples for 96-hours to determine swell potential (as outlined in ASTM D1883).

c) Test strength of samples by performing CBR bearing test.

d) Determine reduction in swell potential by comparing the amount of swell in the treated versus untreated samples (refer to Equation 3). If soil swell potential is less than 1 percent then mitigation may not be necessary, otherwise proceed to sub-step e).

e) Potential load bearing improvements can be evaluated using the CBR strength values of the treated and the untreated samples. Determine increase in strength by comparing the strength of these samples using the same form as Equation 3, but substituting strength improvement for swell reduction.

f) If the reduction of swell through the use of lime or Class C fly ash is greater than 50 percent when compared to the untreated sample (indicative of the benefit of treatment) for soils that have at least 1 percent swell from the soaked CBR test, then the soil poses a significant risk of expansion and the treatment provides an adequate remedy. Otherwise, an additional treatment level may need to be evaluated or an alternative mitigation method selected. Thresholds on strength gains from treatments may also be used in tandem with or as a substitute for the swell results (significance of strength gain was not established as part of this effort). If these criteria are met, continue to Step 3 to determine the potential for ettringite formation.

An alternative method to determine the optimum lime content is the pH method (ASTM D6276).

Step 3 – Determine Potential for Ettringite Formation

The formation of ettringite from lime treatments can be detrimental to the swell characteristics of the soil. The potential for ettringite formation can be assessed by determining the sulfate content of the soil using ASTM D1580. Sulfate contents less than 3000 ppm indicate low severity of excessive soil expansion due to the formation of ettringite during lime treatment. Sulfur content values between 3000 and 5000 ppm indicate low to moderate severity, values between 5000 and 7000 indicate moderate to high severity, and above 7000 ppm indicates high severity. Potential issues with sulfate may also be qualitatively evaluated during Step 2. A non-cementitious, non-calcium based mitigation technique can be used in areas of higher risk. A sufficient mellowing period (from at least 1 day to up to a week) is also thought to decrease the risk of ettringite formation in sulfate-rich soils. Using a lime slurry can also help ensure that the mixture is homogeneous which helps minimize the risk of ettringite formation.

Step 4 – Calculate Benefit/Cost Ratio

Calculate a benefit/cost ratio for each of the improvement options evaluated in the lab (refer to the benefit/cost calculation example in Section 8.4 for procedure). This step can only be used if more than one additive and/or application rate is evaluated. Benefits of chemical stabilization are primarily reduced swell and increased strength. Costs include purchase of the treatment additive and processing these materials into the subgrade soils that make up the embankment. Select the treatment option that provides the greatest benefit at the lowest cost (highest benefit/cost ratio).

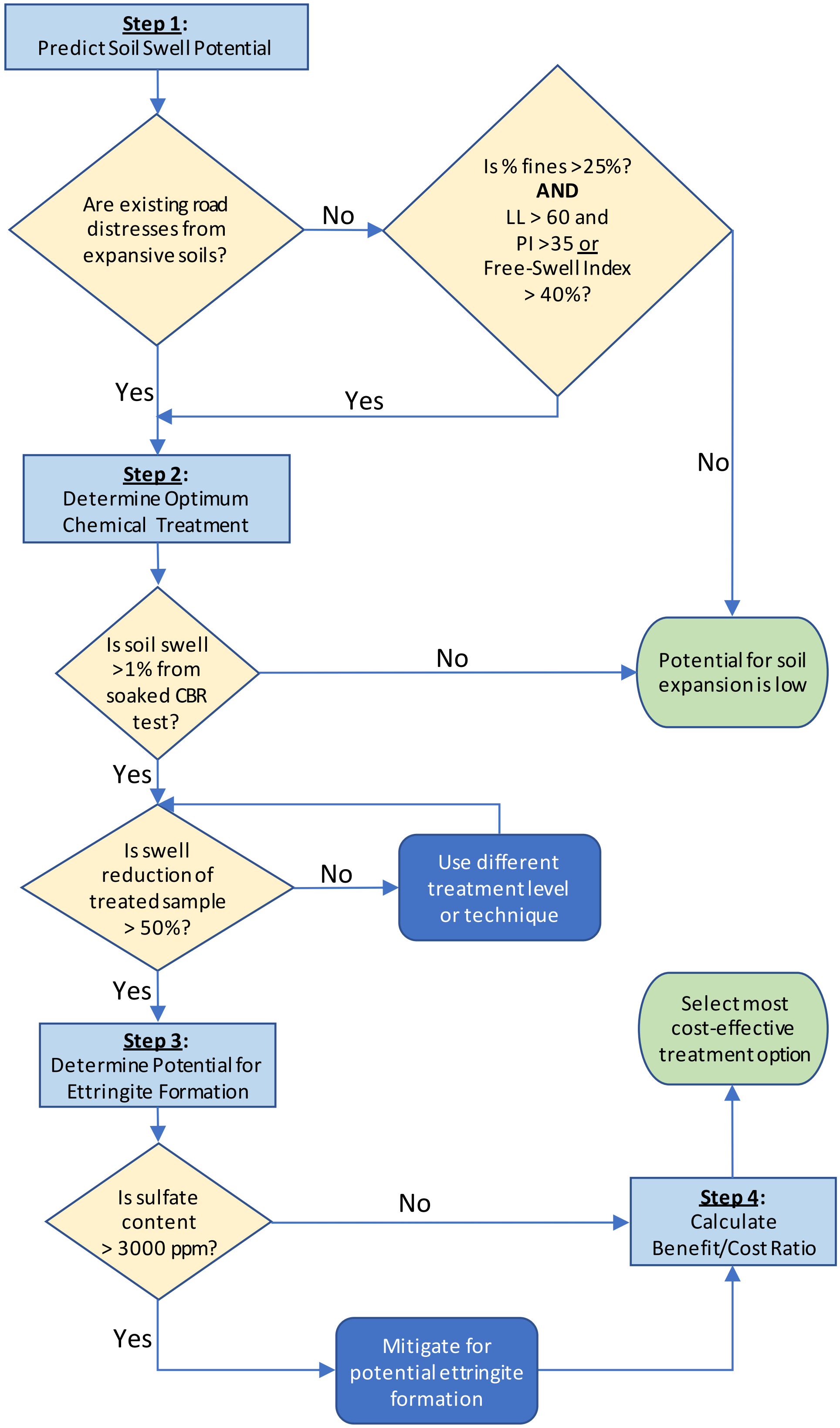

Figure 55: Expansive soil mitigation flow chart.

10.0 RECOMMENDATIONS

The objective of this research endeavor funded by the South Dakota Department of Transportation was to update its construction guidelines and associated specifications to more adequately mitigate the effects of expansive soils on paved roads. A program which included a thorough literature review, laboratory tests on materials collected from six sites in South Dakota, and a cost-benefit analysis was used to meet this objective. The following recommendations were made based on the results of this effort.

10.1 Recommendation 1 – Adopt the suggested procedure in Section 9.2 as a step-by-step process to guide practitioners on how to adequately identify and treat expansive soils in transportation applications using lime and Class C fly ash.

The process outlined in Section 9.2 provides a concise and thorough process to identify and predict the swell potential of clay/shale deposits, evaluate swell and strength, determine sulfate content and perform benefit/cost analysis. Following this decision-making process will help practitioners determine the most economical and beneficial treatment to reduce the effects of expansive soils on South Dakota roads.

10.2 Recommendation 2 – Repair highway embankments and right-of-way borrow and drainage areas currently experiencing heaving along fault gouges caused by differential vertical shifting of discontinuous clay/shale deposits using the construction guidelines outlined in Section 9.0.

Heaving of faults in South Dakota is the result of differential vertical shifting of discontinuous clay/shale deposits. The effect and presence of water must be reduced at the fault to adequately address this problem. The effect of water can be controlled using a soil treatment such as lime or Class C fly ash. The presence of water under and adjacent to the roadway can be controlled using proper drainage (e.g., adequate slope, edge drains, etc.) and/or by installing a protective barrier to keep surface water from infiltrating into the fault gouge. Construction guidance is provided to rehabilitate areas where known or anticipated fault gouges are likely to cause damage to highway infrastructure due to expansion.

10.3 Recommendation 3 – Compact subgrades at 2 percent above optimum to reduce the effect of water on expansive soils, based on the information presented in Section 7.3.

It is generally well known that soils compacted wet of optimum will experience less swell than those compacted at or below their optimum moisture content. Test results from standard soaked CBR tests run on treated and untreated samples from South Dakota indicated, in all cases, that samples compacted above the optimum moisture content exhibited less swell than those compacted at optimum moisture content.

11.0 RESEARCH BENEFITS

The results of this project provide a better understanding of the behavior and mitigation techniques applicable to localized distresses in highways due to differential expansion near fault gouges. The information contained within this document will help SDDOT personnel identify expansive soils, optimize chemical additives used to treat soils susceptible to expansion, implement construction guidelines and specifications to better address problems associated with expansive soils, and determine the most efficient treatment for expansive soils based on performance and cost. Implementation of the methodologies outlined in this report will likely provide the following benefits.

- <u>Increased longevity of roads</u>—roads that utilize better design methods and construction techniques require less maintenance, fewer natural resources, less time, and less money from state transportation agencies, and ultimately citizens, by reducing the need for frequent rehabilitation and replacement. Pavement analysis software used to manage roads can be used to monitor and evaluate the life of roadways over time.
- <u>Improved serviceability of roads</u>—reduced cracking, heaving, and buckling of the road surface will improve ride and reduce the need for frequent maintenance and rehabilitation. An assessment of ride can be made by monitoring and analyzing IRI on various road segments over time.
- <u>Improved safety</u>—eliminating heaves and undulations on the road surface will allow the public to travel at posted speeds without the need to alter their driving patterns to avoid the effects of rough pavements. Reducing speeds causes increased traffic interactions resulting in a higher likelihood of accidents. Similarly, roads that need less maintenance will require less lane closures or work zones, thereby reducing the potential for secondary collisions. Safety records can be collected and analyzed to determine the potential benefits of these methodologies over time.
- <u>Cost savings</u>—the methodologies outlined in this report can be used to determine the most efficient chemical treatment for expansive soils based on performance and cost. The benefit/cost analysis outlined herein can be used by designers to optimize their choice for the most effective treatment method.

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13.0 APPENDIX A: FREE-SWELL TEST RESULTS

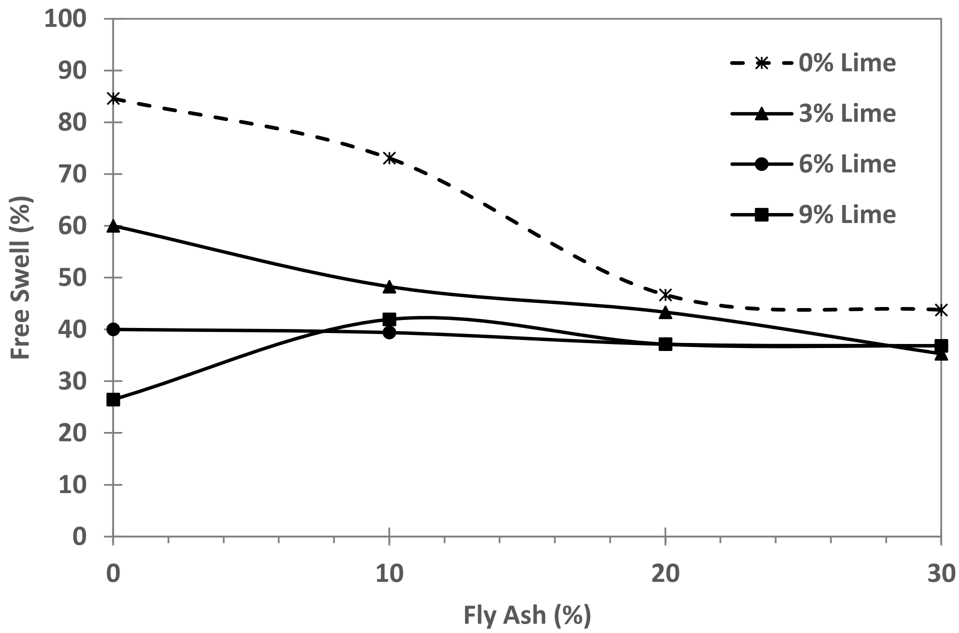

Figure 56: Free-swell results for Site 1, shown with respect to fly ash treatment.

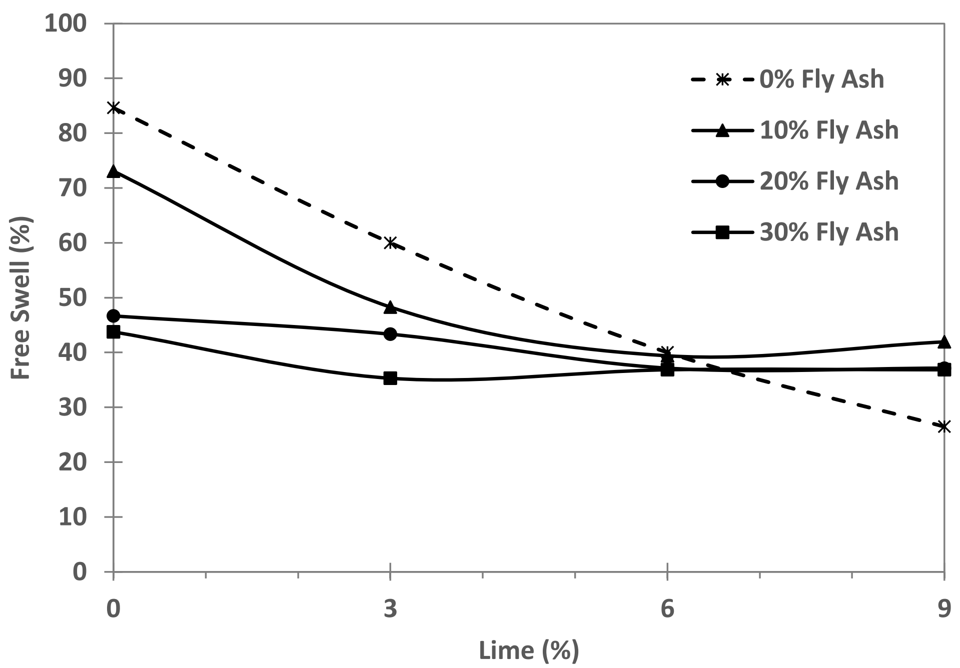

Figure 57: Free-swell results for Site 1, shown with respect to lime treatment.

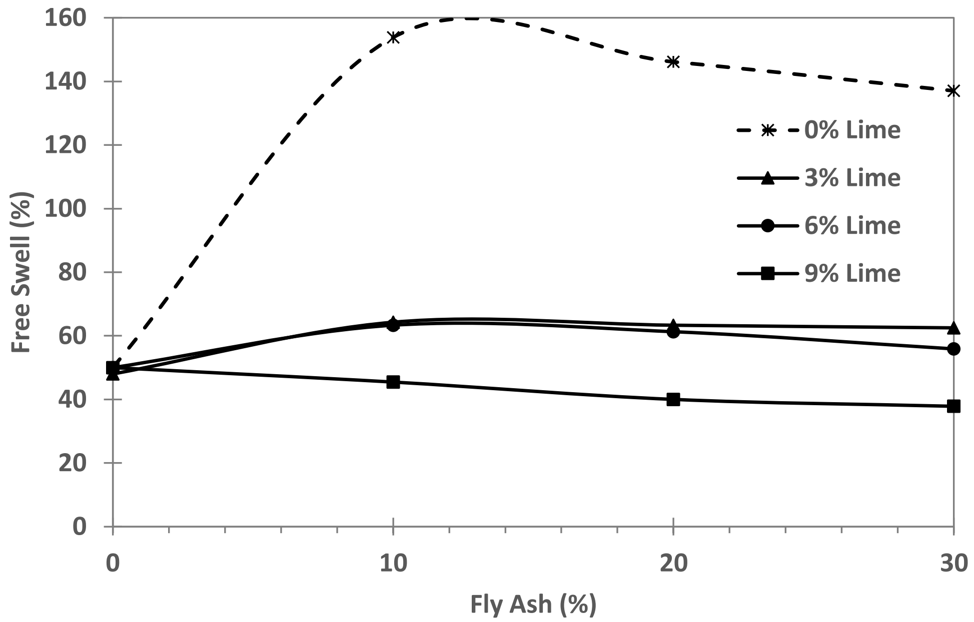

Figure 58: Free-swell results for Site 2, shown with respect to fly ash treatment.

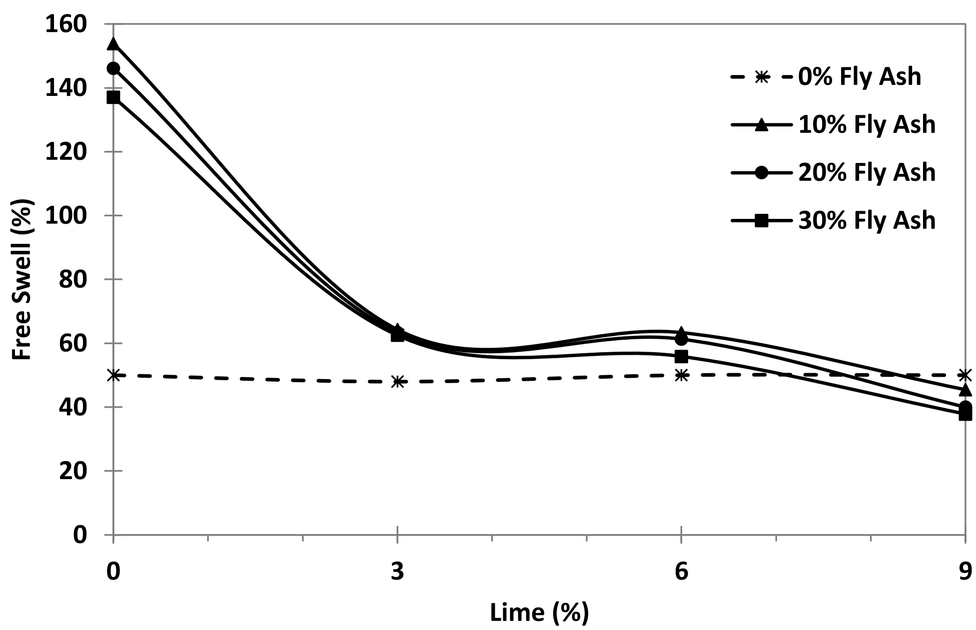

Figure 59: Free-swell results for Site 2, shown with respect to lime treatment.

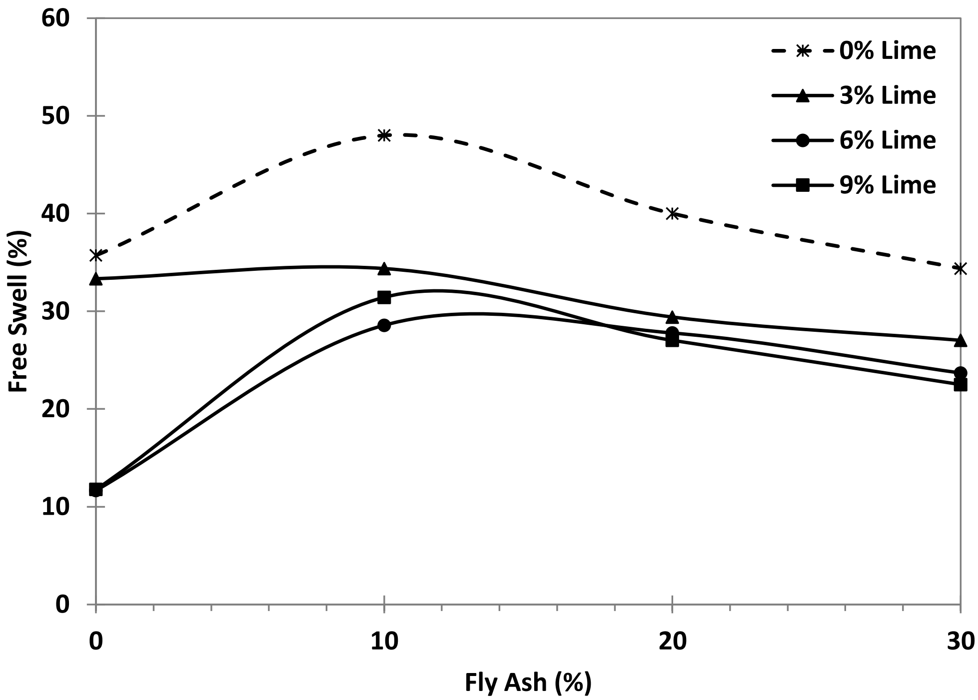

Figure 60: Free-swell results for Site 3, shown with respect to fly ash treatment.

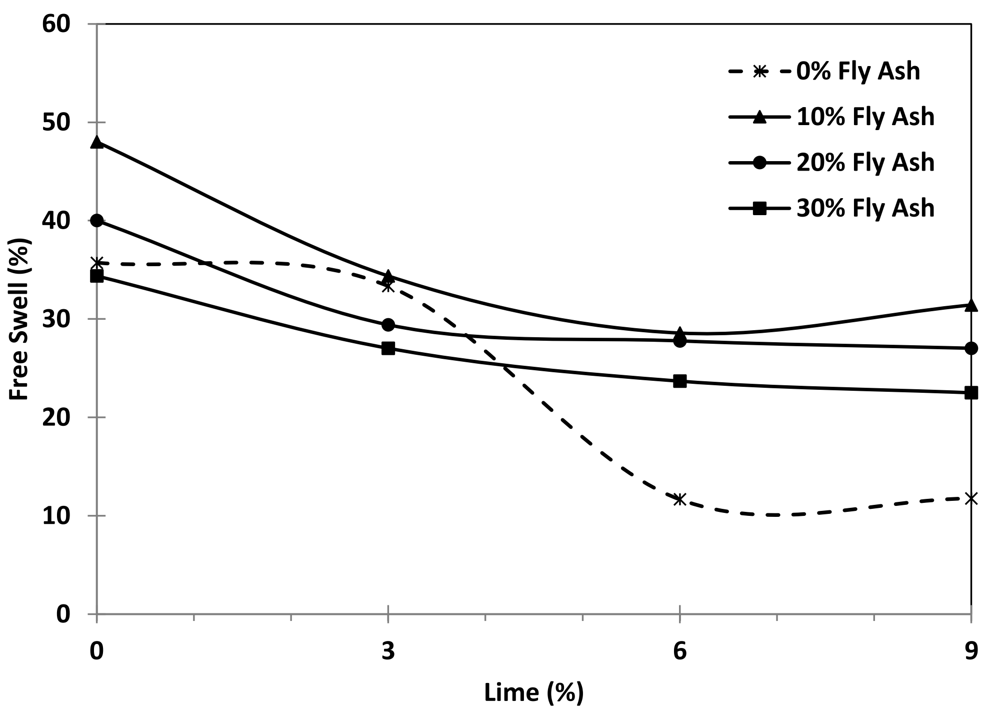

Figure 61: Free-swell results for Site 3, shown with respect to lime treatment.

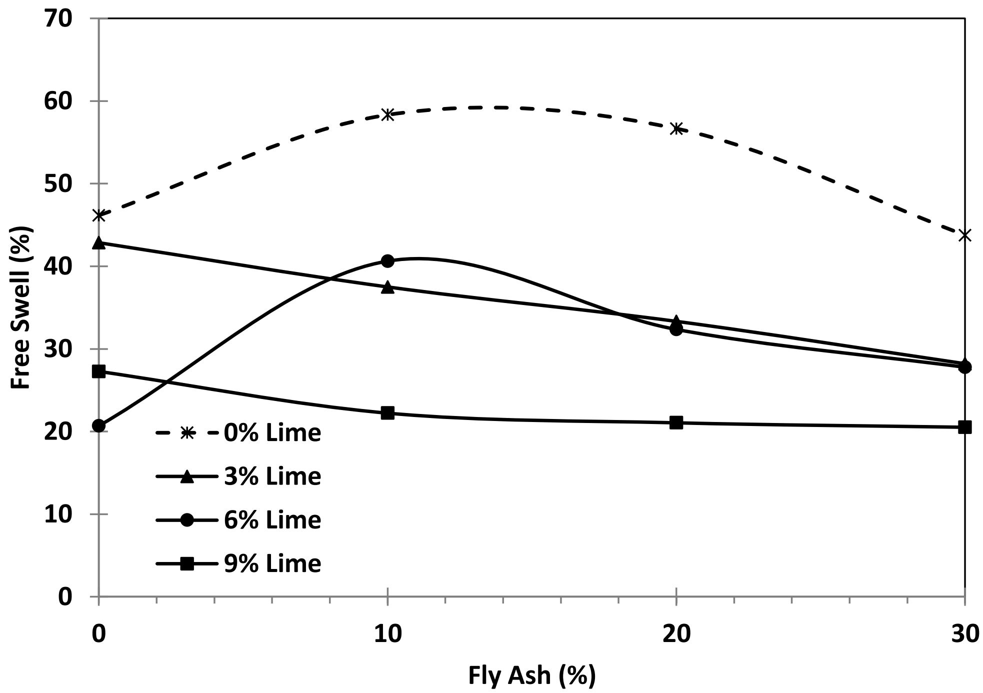

Figure 62: Free-swell results for Site 4, shown with respect to fly ash treatment.

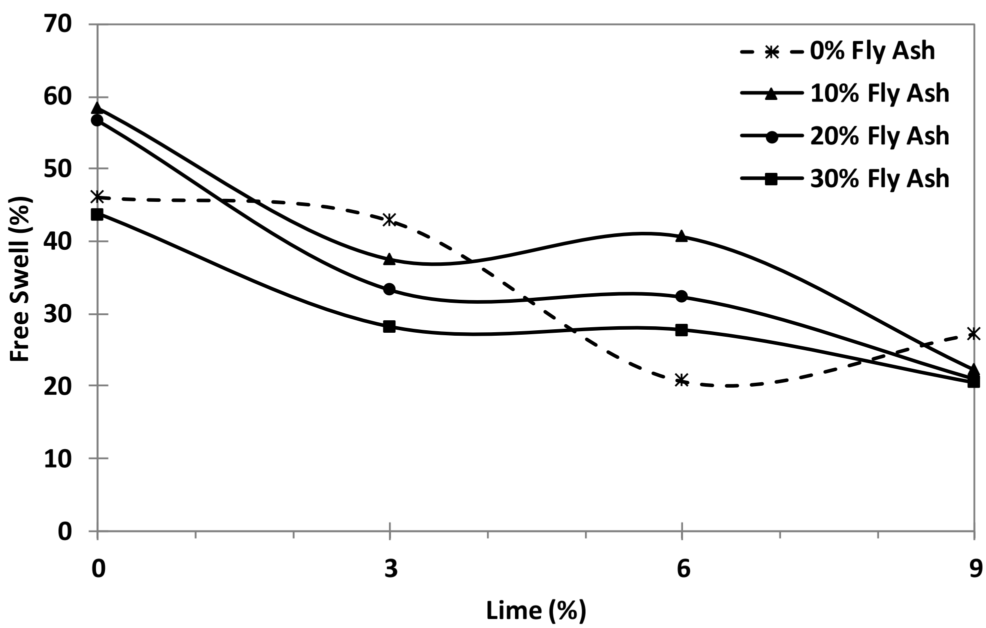

Figure 63: Free-swell results for Site 4, shown with respect to lime treatment.

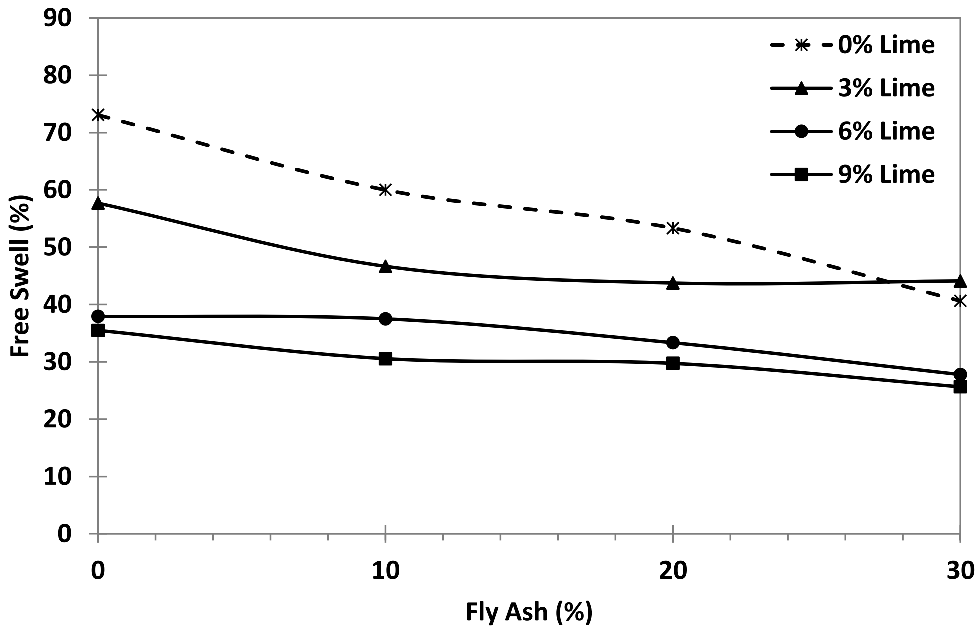

Figure 64: Free-swell results for Site 5, shown with respect to fly ash treatment.

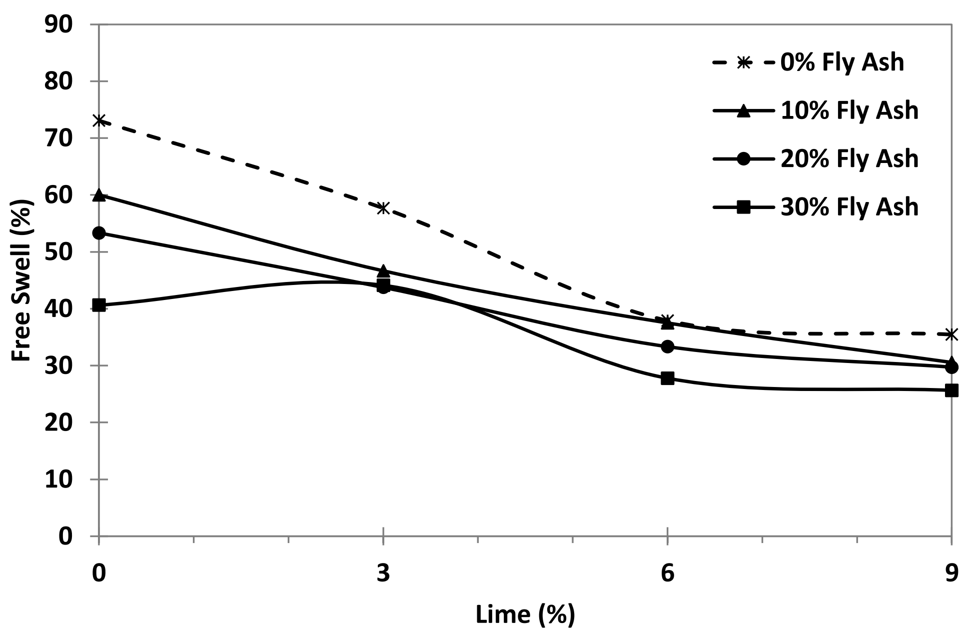

Figure 65: Free-swell results for Site 5, shown with respect to lime treatment.

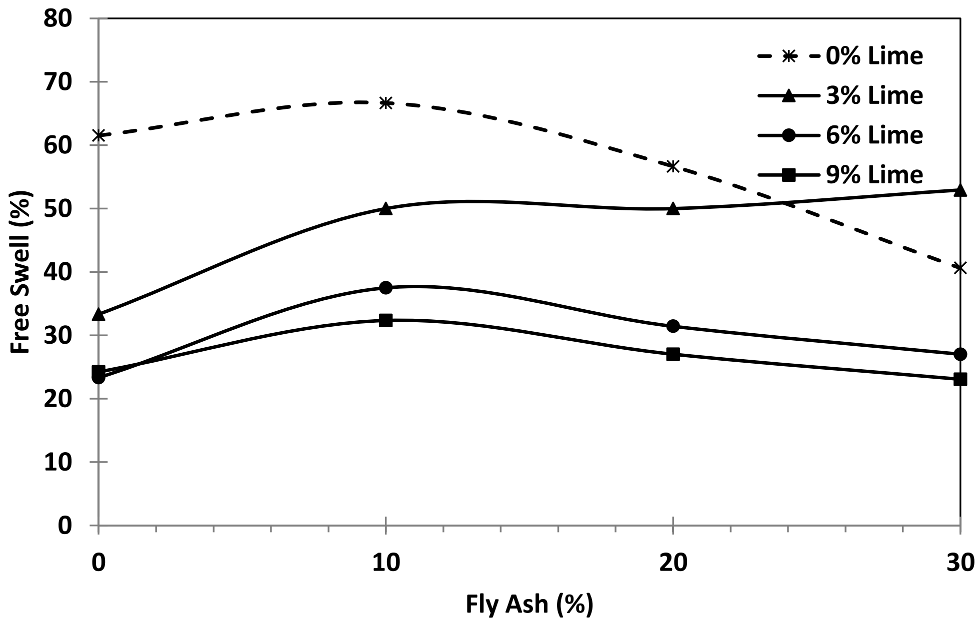

Figure 66: Free-swell results for Site 6, shown with respect to fly ash treatment.

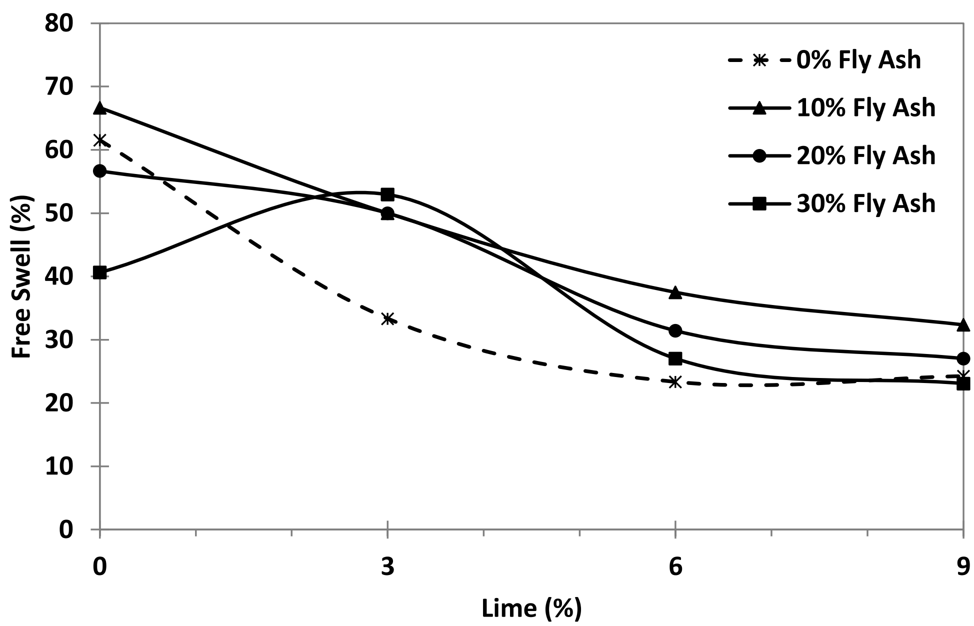

Figure 67: Free-swell results for Site 6, shown with respect to lime treatment.