Modification of Highway Soil Subgrades

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MODIFICATION OF HIGHWAY SOIL SUBGRADES

by

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in cooperation with the
Kentucky Transportation Cabinet
The Commonwealth of Kentucky
and
Federal Highway Administration

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June 1995
December 12, 1995

Subject: Implementation Statement for Research Study KYHPR 88-132, Modification of Highway Soil Subgrades

Dear Mr. Toussaint:

This study originated from a desire to investigate what was considered premature failures of soil subgrades and flexible pavements during and shortly after construction. Problems encountered during construction, as noted by construction and geotechnical engineers of the Cabinet, included shoving and pushing of clayey subgrades under construction traffic loadings, the lack of a firm working subgrade platform for constructing and compacting base and asphaltic paving materials, and a loss of subgrade bearing strength during and after construction.

Premature failures and problems arising during construction, or shortly after construction, of subgrades and pavements can be chiefly attributed to the poor engineering properties of Kentucky soils. Approximately, 85 percent of Kentucky soils used to construct subgrades are fine-grained clays and silts. When these soils are initially compacted, bearing strengths are usually large. However, the bearing strengths of the compacted soils decrease greatly when they are exposed to moisture from surface runoff and subsurface seepage. Most of these compacted soils absorb water, expand, and decrease in strength.

Major objectives of this research study were to establish a highway pavement subgrade stabilization program, develop subgrade stabilization guidelines, and examine long-term benefits of chemical stabilizers, such as cement and hydrated lime. During this study, a stabilization program was established. However, to implement this program, several questions concerning laboratory testing, design, and construction of stabilized bases had to be addressed and resolved. These issues and recommended practices are described in the report.
**Implementation Statement**

For example, knowing when subgrade stabilization is needed was a key issue. To address this problem and develop guidelines, a detailed analysis of this problem was made using a newly developed, mathematical bearing capacity model. A full description of the model was presented previously by the geotechnical staff of the University of Kentucky Transportation Center in Research Report KTC-91-8, "Bearing Capacity Analysis of Pavements." As shown by results from this mathematical model, subgrade stabilization should be considered when the CBR strength of the subgrade is less than 6.5. This important recommendation was based on a tire contact ground stress of 552 kPa (80 psi). Relationships between CBR and tire stresses were developed for other anticipated stresses. This finding was also confirmed by published field data and experience. The CBR value of 6.5 is the minimum strength of the subgrade required to support construction traffic. However, to avoid failure it was shown that the subgrade CBR should be equal to nine, or greater. Cabinet engineers put these guidelines into practice during this study.

Factors, such as site geology, swelling of soil subgrades, and changes in subgrade moisture significantly influence the behavior and change the performance of subgrades and pavements. Changes in moisture content and in situ CBR strengths of untreated and chemically treated subgrades were monitored over a six-year period at several experimental highway sections -- KY 11 (10.5 km, or 6.5 miles) and Alexandria - Ashland Highway (Sections 12, 13, 14, 19, and 20). Although bearing strengths of the subgrades immediately after compaction at those locations were typically large (CBR values greater than 9), a dramatic decrease occurred in the bearing strengths shortly after construction. Typically, values ranged from one to four. However, CBR strengths of subgrades treated with hydrated lime generally exceeded 12 and increased over the study period. CBR strengths of experimental subgrades treated with cement generally exceeded 25 after seven years. Moreover, at four old sites ranging from 10 to 30 years in age -- highway sites discovered during the research study -- where cement had been used to treat the subgrades, CBR strengths exceeded 90. Rutting at all experimental sites where chemical admixtures had been used were nominal, or very small. Generally, rutting depths of flexible pavements on subgrades, which were stabilized chemically, were smaller than rutting depths of flexible pavements on untreated subgrades. Moreover, the strengths of chemically stabilized subgrades were several times larger than strengths of untreated subgrades.

When chemical stabilization is used, the optimum percentage of the chemical admixture, such as cement or hydrated lime, must be determined. A procedure for determining the optimum percentage of a chemical admixture was developed early in the study and was implemented by Cabinet engineers. A unique testing mold and compaction accessories and equations were developed. Working drawings of the compaction equipment and a PC® computer program were transmitted to Cabinet engineers for implementation.

In recent years, byproducts from industrial processes and coal-fired, power plants have been widely discussed and proposed as subgrade stabilizers. Two byproducts were used in subgrade trials at one highway site. One byproduct, called multicone kiln dust (MKD), obtained in the manufacturing of hydrated lime, was successfully used in treating a trial subgrade section on KY 11. CBR strengths of the MKD - soil subgrade generally exceeded 90 and rutting depths were generally less than 2.5 mm (0.1 in.) after seven years. This
Implementation Statement

Material may be very suitable for improving strengths of clayey subgrades. However, the availability of free CaO (Quicklime) and Ca(OH)₂ (Calcium Hydroxide) should be confirmed before using this material at a given site.

The other byproduct used in two subgrade trial sections on KY 11 was a material obtained from a process called atmospheric fluidized bed combustion (AFBC). Swelling problems of the AFBC-soil subgrades developed during construction when approximately three-fourths of the flexible pavement had been placed and shortly after a rainy period. The swelling nature of the AFBC-clayey mixtures did not occur in routine laboratory tests. Long-term swelling tests and analysis of data from those tests were used to estimate the time swelling would cease. Final paving was delayed until sufficient time had elapsed. Subsequent sampling and testing -- X-ray diffraction and Scanning Electron Microscopy -- showed that the swelling was caused by the formation and growth of minerals called gypsum and ettringite. Formation of these minerals might be dependent on the availability of calcium sulfate, which was present in the AFBC material. This compound is usually present in most flue gas desulfurization byproducts. It is not present in typical fly ashes, such as class C or F. Consequently, materials from coal-fired power plants, such as flue gas desulfurization byproducts, should not be used in subgrade stabilization unless it can be shown by long-term laboratory swelling tests that swelling is less than about 4 percent, or that the material can be treated in some fashion to reduce swelling. Caution must be exercised in any attempts to use FGD byproducts.

The swelling problem can also occur when soils containing sulfates are mixed with cement or hydrated lime. Although no cases of this type have been reported in Kentucky, a few cases have been reported in England, Texas, and California. Research is needed to develop standard techniques for identifying soils that may have high sulfate contents. Therefore, in those cases, hydrated lime or cement should not be used as subgrade stabilizers to avoid the swelling problem.

Although this study focuses mainly on chemical stabilization, some attention is given to mechanical stabilization. Based on analyses using the newly developed, bearing a capacity model mentioned above, geogrids placed at the bottom of granular bases appear to increase stability. The factor of safety against failure of reinforced bases is some 10 to 25 percent greater than the factor of safety of granular bases that do not contain reinforcement. To develop this bearing capacity model for routine use, much more research is needed. This type of analytical tool is needed when examining alternative pavement designs.

In situ mixing of granular material with clayey subgrades is another mechanical stabilization technique. Strength tests and stability analyses show that when the percent finer than the 0.002mm-size particle is greater than about 15 percent, the factor of safety decreases significantly. Here, the long-term performance of this technique is questionable.

Selection of the design strength of soil subgrades is an extremely important and complex issue that poses a major problem to pavement design engineers and geotechnical engineers. The future behavior and performance of a pavement, and the pavement design thicknesses, are very dependent on the design strength initially selected for the subgrade. Therefore,
the selection process is not a trivial matter. Selection of the design strength is complicated by the fact that usually several different types of soils exist in a highway corridor. Furthermore, these different types of soils may have different strengths after compaction. Additionally, design strengths of chemically treated subgrades were needed by the Cabinet’s engineers. Design values of soil-hydrated lime and soil cement subgrades proposed in the study were obtained from extensive field and laboratory testing at several highway sites. Guidelines for selecting the design subgrade strengths of untreated and treated subgrades were proposed. To facilitate use of these guidelines, algorithms were programmed for the PC® computer. These selection guidelines and a computer program are being used by Cabinet engineers on a trial basis.

Finally, the Cabinet’s engineers needed a rapid means of assessing the existing strengths of both untreated and treated subgrades in the field before paving. The dynamic cone penetrometer was recommended for field personnel because this device is very easy to operate. Also, as an alternative, the Clegg impact hammer was recommended. Correlations of dynamic cone penetrometer and Clegg impact hammer values and in-situ CBR and unconfined compressive strengths were developed. Many tests were performed at several new highway sites to develop those correlations. The correlations are being used by the Cabinet’s engineers on an experimental basis at this time.

In summary, the major objectives of this study were met. Major findings and recommendations have been implemented. Establishment of a subgrade stabilization program in Kentucky is playing a major role in reducing, and essentially eliminating, problems encountered in constructing subgrades and pavements. Furthermore, pavements constructed on chemically treated subgrades are performing better than pavements constructed on untreated subgrades. This should decrease maintenance requirements in the future. A long-term evaluation of pavements on chemically treated and untreated subgrades is needed to confirm this observation. Presently, the Cabinet is funding research to evaluate several different design sections on the Alexandria-Ashland Highway in northern Kentucky. This highway contains several subgrades and bases constructed in different ways.

To link our design procedures with state-of-the-art pavement design practices, a research study is in progress to determine resilient moduli of Kentucky soils. The major aim of this research is to develop a simple technique for predicting the resilient modulus of a given type of soil.

Sincerely,

J. M. Yowell, P.E.
State Highway Engineer
## Abstract

Major study objectives were to develop highway pavement subgrade stabilization guidelines, examine long-term benefits of chemical stabilizers, such as cement, hydrated lime, and two byproducts from industrial processes, and to establish a subgrade stabilization program in Kentucky. In developing a program, a number of design and construction issues had to be resolved. Factors affecting subgrade behavior are examined. Changes in moisture content and CBR strengths of untreated and chemically treated subgrades at three experimental highway routes were monitored over a 7-year period. CBR strengths of the untreated subgrades decreased dramatically while moisture contents increased. CBR strengths of subgrade sections treated with hydrated lime, cement, and multicone kiln dust generally exceeded 12 and increased over the study period. At four other highway routes ranging in ages from 10 to 30 years, CBR strengths of soil-cement subgrades exceeded 90. Knowing when subgrade stabilization is needed is critical to the development of an economical design and to insure the efficient construction of pavements. Bearing capacity analyses using a newly developed, stability model based on limit equilibrium and assuming a tire contact stress of 552 kPa show that stabilization should be considered when the CBR strength is less than 6.5. For other tire contact stresses, relationships corresponding to factors of safety of 1 and 1.5 are presented. Stability analyses of the first lifts of the paving materials show that CBR strengths of the untreated subgrade should be about 9 or greater. Guidelines for using geogrids as subgrade reinforcement are presented. Factors of safety of geogrid reinforced granular bases are approximately 10 to 25 percent larger than granular bases without reinforcement. As shown by strength tests and stability analysis, when the percent finer than the 0.002mm-particle size of a soil increases to a value greater than about 15 percent, the factor of safety decreases significantly. Guidelines are also presented for the selection of the design strengths of untreated and treated subgrades with hydrated lime and cement. Based on a number of stabilization projects, recommended design undrained shear strengths of hydrated lime- and cement-treated subgrades are about 300 and 690 kPa, respectively. A laboratory testing procedure for determining the optimum percentage of chemical admixture is described. Correlations of Dynamic Cone Penetrometer and the Olegg Impact Hammer values and in situ CBR strengths and unconfined compressive strengths are presented.

## Key Words

Highways, Pavements, Stability, Case Histories, Soil Subgrades, Mechanical Stabilization, Chemical Stabilization, Cement, Hydrated Lime, Byproducts, Guidelines, Design

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INTRODUCTION

Most pavements in Kentucky have been, and are being, constructed on fine-grained clay and silt. Some 85 percent of soils in Kentucky consist of clay and silt. When first compacted, these clayey soils usually have sizeable bearing strengths. If pavements are constructed immediately after compaction on the clayey soils, then major difficulties are normally not encountered in placing and compacting layers of paving materials. Problems may arise, however, when surface and subsurface water penetrates the compacted clayey subgrades. Water from a rainfall, snow melt, and groundwater seepage enters the clayey subgrades, causes swelling, and produces a loss of bearing strength. The most susceptible, adverse period occurs when the subgrade has been exposed to the wetting conditions of winter and early spring. During this period, before paving, rutting may quickly develop in the softened subgrade and slow, or even halt, the movement of construction traffic. Difficulties arise when attempts are made to compact the first lifts of pavements because of a lack of a firm subgrade. When these situations develop, delays occur which require costly remedial measures.

Even when the construction of the pavement is successful, the bearing strength decreases significantly and adversely affects the behavior of the pavement. This study originated because of problems encountered with highway pavement subgrades during and after construction. Problems, as noted by construction and geotechnical engineers, frequently included the shoving and pushing of clayey subgrades under construction traffic, the lack of a firm working platform for constructing and compacting base and paving materials, and a loss of bearing strength during and after construction.

The primary objective of this study was to devise and establish a subgrade stabilization program in Kentucky. Other objectives included the examination of the long-term benefits of using such commercially available chemical admixture stabilizers as cement and hydrated lime and two byproducts from industrial processes, and the establishment of highway subgrade design and construction standards. In establishing this program, many issues had to be examined and resolved. These issues are briefly described and discussed below.

FACTORS AFFECTING PAVEvMENT SUBGRADE BEHAVIOR

Some factors that significantly affect the behavior and performance of highway pavements and subgrades include the geologic setting and soil types existing at a
Physical properties of the subgrades, such as compaction degree, swelling tendencies, and the presence of moisture, may also affect the behavior and performance. Types of soils available at a given location in Kentucky for constructing subgrades are controlled by site geology since major portions of Kentucky's soils are residual -- soils that are the result of the weathering of bedrock. For example, soils derived from clayey shales, such as the Kope Geological Unit, in the northern regions of Kentucky, have very poor engineering properties. Pavements placed on subgrades constructed with these types of soils have notoriously performed poorly. In comparison, pavements constructed on soils derived from the New Albany Geologic Unit have generally done very well. Statistically some 85 percent of Kentucky soils consist of clay and silt -- materials that generally have poor engineering properties.

Although compaction of clayey soils increases shear strength, compaction alone will not, necessarily, insure that a subgrade will act properly throughout pavement life. Subgrades are subjected to the infiltration of water from surface runoff and subsurface seepage. Compacted clayey subgrades absorb water and swell. As swelling occurs, a loss of bearing strength occurs. Both field and laboratory data obtained during this study illustrate this condition. Moreover, the use of drainage measures, although desirable, will not prevent the development of this situation because the subgrade will be exposed to water during some period of the pavement's life. Therefore, compaction and drainage measures used alone will not totally insure good performance of clayey subgrades and pavements.

**MINIMUM SUBGRADE BEARING STRENGTH**

When should subgrade modification be considered? To resolve this question, a newly developed bearing capacity model (Hopkins 1994) was used to analyze this problem. Relationships between undrained shear strength (and California Bearing Ratio -- CBR) of the subgrade and different tire ground contact stresses were developed for different factors of safety against failure.

Therefore, if the tire contact stresses that may exist on the clay subgrade during construction are known, then the minimum strength necessary to sustain construction traffic may be found from the relationships developed in this study. Using these relationships, engineers of the Kentucky Transportation Cabinet can rapidly detect difficulties during construction of the pavement layers, or if the untreated or treated subgrade may fail under construction traffic. For example, if the anticipated tire stress is 552 kPa (80 psi), then the minimum CBR strength required to maintain incipient failure (factor of safety equals one) is about 6.5. However, to maintain good stability, the CBR strength should be about nine or greater (factor of safety equal to 1.5). Minimum strengths required when the tire contact stress is some value other than 552
METHODS FOR IMPROVING SUBGRADES

Using the above guideline, if subgrade modification is deemed necessary, then several techniques may be used to improve bearing strength. These methods can be broadly classified into two categories: mechanical and chemical. Mechanical methods include such traditional approaches as: controlling subgrade density-moisture, undercutting poor materials and backfilling with granular materials, proof rolling and rerolling of the subgrade, mixing of stone aggregate with the clayey subgrade, using granular layers, and using granular layers reinforced with geofabrics. A detailed laboratory examination of the technique of mixing stone aggregate into the soil subgrade was performed. As shown by the results of this study, a significant decrease in bearing strength occurs when the percent finer than the 0.002 mm-particle size is greater than about 15. Therefore, this stabilization technique may be ineffective in soils with a high clay content. The use of geofabrics, such as geogrids, to improve bearing capacity of granular bases was also examined using a newly developed bearing capacity model. Results of these analyses show that the factor of safety increases some 10 to 25 percent when geogrids are used. However, stability analyses of field case studies need to be done to confirm this result and to verify the newly developed stability model.

A major focus of this report is on chemical subgrade stabilization. Before 1987, chemical stabilization was used sparingly in Kentucky. Commercial chemical stabilizers include hydrated lime and cement. Only four sites, constructed before 1987, were found that used cement as the subgrade chemical admixture. No sites constructed before 1987 were found that used hydrated lime as the chemical admixture. Apparently, the first sites -- KY 11 and Section 19 of the Alexandria - Ashland Highway-- in Kentucky using hydrated lime as a subgrade stabilizer originated from this study. Experimental sites, established in this study, have been monitored for about seven years. CBR strengths of the soil - hydrated lime subgrades in the experimental sections are several times greater than the untreated subgrade. These strengths are increasing with time. The soil-cement subgrades at the four old sites ranging in ages from six to 30 years are extremely stiff. In situ CBR strengths generally exceed 90. Flexible pavements constructed on the soil-cement subgrades generally have done very well. Average overlay history is about 12 years.

Two byproducts were used at the KY 11 site near Beattyville, Kentucky. Two subgrade sections of this reconstructed route were treated with an Atmospheric Fluidized Bed Combustion (AFBC) spent-lime. Laboratory tests showed that the addition of the spent lime significantly increased the bearing strength. However, about two months after some asphalt layers had been placed, and after a rainy period, pavement buckling
occurred at several locations. None of the standard CBR laboratory data suggested that swelling was a problem. As shown by subsequent tests, a long delay occurred before swelling commenced. Based on laboratory swell tests, a theoretical estimate of the time for completion of primary swelling of the subgrade was made. Final surfacing, after pavement milling of buckled locations, was placed after the estimated time. After some seven years, in situ monitoring shows that CBR strengths generally exceed 10 and rutting is less than about 7.6 mm (0.3 in.). To determine the causes of the swelling, subgrade specimens were obtained. X-ray diffraction (XRD) and scanning electron microscopy analyses were performed on the collected specimens. Analysis showed that the swelling behavior of the AFBC-treated subgrade was caused by the formation of ettringite and anhydrite gypsum—types of minerals. Formation of these minerals and swelling appear to be closely related to the presence of calcium sulfate and sulfite. The recommendation was made to engineers of the Kentucky Transportation Cabinet that FBC-type of byproducts should not be used as chemical admixtures in soil subgrades unless it could be shown that the long-term swelling, as determined from long-term laboratory swelling tests, of the FBC-type material is less than about 4 percent and the CBR strength is greater than above nine after the total swelling has occurred.

A second byproduct, multicone kiln dust (MKD), was also used to treat a subgrade section of KY 11. After seven years, the in situ CBR strength of the MKD-treated subgrade generally exceeds 90. Rutting of the pavement after seven years is less than 0.25 cm (0.1 in.). Because of the superior performance of this pavement section, it was recommended that this byproduct could be used as a chemical admixture.

To decide if soaked, laboratory strengths represent long-term, field strengths, in situ CBR tests were performed at two highway routes over a period of about five years. The laboratory and field CBR values were graphed as a function of percentile test values; the laboratory strengths seem representative of field strengths. Therefore, it was recommended that soaked laboratory strengths could be used to select an appropriate design strength of untreated clayey subgrades. Although this has been done in the past, data to support this design approach was obtained in an attempt to justify using soaked laboratory strengths.

When should soil subgrade stabilization be considered? This question was posed by members of the Study Advisory Committee. Guidelines were formulated and recommended to the Cabinet's engineers for deciding when subgrade stabilization is needed. If the CBR strength of a subgrade is below about 6.5, and the tire contact stress is 552 kPa (80 psi), then subgrade stabilization, such as chemical stabilization with hydrated lime or cement, should be considered. This important principle was established from results obtained from the newly developed bearing capacity model described in the report cited above. This recommendation was carried out and established as a policy during this study by the Cabinet's geotechnical staff engineers.
If chemical stabilization is used, then two major questions arise: should the treated subgrade be considered merely as a construction, or working platform, or should it be considered a part of the pavement structure? How thick should the treated subgrade be to avoid failures during construction? To address the first question, core specimens were obtained at several highway sites from cement-and hydrated lime-treated subgrades. The specimens were obtained at the end of a 7-day curing period. Unconfined compression tests were done on these specimens. Also, laboratory specimens were compacted and unconfined compression tests were performed on those specimens. The compacted specimens had been aged for seven days before testing. Results from laboratory and field unconfined compression tests were graphed as a function of percentile test values. Based on the 90th percentile test value, it was recommended that reasonable undrained design strengths for soil-cement and soil-hydrated lime subgrades were 711 kPa and 331 kPa (103 and 48 psi), respectively. These values correspond to CBR values of about 25 and 12, respectively. Dynamic modulus of elasticity are about 297,487 kPa (43,114 psi) and 152,594 kPa (22,115 psi), respectively. By using these values, at least part of the subgrade strength gain may be used in design. Presently, this approach has been adopted by the Cabinet, although, as we understand, the lower value of 152,594 kPa (22,115 psi), is being used for both soil-cement and soil-hydrated lime subgrades. Nevertheless, this idea has been implemented.

Regarding the second question, a design chart relating the required thicknesses for soil-cement and hydrated-lime to the CBR strength of the untreated subgrade found below the treated layers was developed using the newly developed, bearing capacity model. A factor of safety of 1.5 and the undrained strength, or CBR, occurring at the 90th percentile test value (listed above), were used in those analyses.

**OPTIMUM PERCENTAGE OF CHEMICAL ADMIXTURE**

During the early portion of this study, a laboratory procedure for determining the optimum percentage of a chemical admixture that should be specified on a given project and for a given type of soil was developed. Unique laboratory compaction equipment was designed and constructed. Working drawings of this equipment were transferred to the Geotechnical Branch of the Kentucky Transportation Cabinet. This procedure, including mathematical algorithms and a PC computer program for doing the necessary calculations to remold specimens, was adopted by the Geotechnical Branch and has been used routinely since the early portion of this study. This procedure uses the unconfined compression test to determine the optimum percent of a chemical admixture.
SELECTION OF DESIGN STRENGTHS OF UNTREATED SOIL SUBGRADES
AND SUBGRADES TREATED WITH CEMENT AND HYDRATED LIME

An in-depth analysis of several approaches to this problem has been made; two case studies involving pavement failures were analyzed using a newly developed bearing capacity model. The case studies were very useful in establishing the most appropriate method for selecting the design strength of a soil subgrade. It was recommended that a least-cost approach -- proposed by Yoder (1969), be adopted by the Cabinet's engineers. This approach involves graphing the strengths (for example, CBR) as a function of percentile test values. If the cost ratio -- the unit maintenance cost to the unit initial cost -- is known or assumed, then the design percentile test value may be selected. Once this value is known, then the design strength is obtained. If the cost ratio is unknown, then the value of strength occurring at the 80th to 90th percentile test value may be selected for design purposes. It was shown that this is a good approach, as illustrated by the analyses of two case studies involving failures of pavements during construction. To implement and facilitate the use of this approach, a PC® (personal computer) computer program was developed for the Cabinet's engineers. The geotechnical staff of the Cabinet received training on the use of this program during the research study period and they are using the approach on a trial basis.

In situ moisture contents and field CBR values of clayey subgrades at two experimental highway routes were monitored over a period of about five years. A dramatic reduction in strengths of untreated clayey subgrades occurred with increases in moisture content and time. Such large decreases in strength must be considered in the design of pavements. Soaked laboratory strengths have been and are being used for predicting long-term field strengths. However, soaked strength from a laboratory test may not represent long-term field strength. This research study attempted to address this issue. Results obtained at two sites over a period of five years showed that the field CBR strengths were close to soaked laboratory CBR strengths.

POTENTIAL PROBLEMS IN CHEMICAL ADMIXTURE STABILIZATION

When the air temperature is below about 4.4 to 7.2 degrees Centigrade (40 to 45 degrees Fahrenheit) at the time of chemical stabilization, chemical reactions between soil particles and hydrated lime or cement may not occur. Consequently, improvement in bearing strength of the treated subgrade will not occur. Chemical admixture specifications include a stipulation that the temperature must be greater than 7.2° C (45° F) before chemical stabilization is allowed.

Previous published case studies show that when soils contain high levels of soluble
sulfates, large magnitudes of swelling may occur when hydrated lime or cement is used as chemical admixtures. Swelling of the treated subgrade adversely affects the pavement, that is, the pavement is prone to heave, or form "humps" that run perpendicular to the centerline. This condition may also occur if the chemical admixture contains high levels of soluble sulfates. For example, FGD byproducts produced from coal-fired power plants contain high levels of soluble sulfates. Those materials also contain calcium oxide (quicklime) which reacts with clayey soils when mixed and increases shear strength. In either case, five conditions must exist to initiate swelling. These are as follows:

- high pH conditions,
- adequate supply of alumina, silica, and carbonates -- sufficient clayey mineral content,
- presence of sulfates (either in the soil or FGD byproduct),
- correct temperature conditions, and
- availability of water.

When these conditions exist, the formation of the minerals, ettringite and thaumasite, occurs and the treated subgrade will swell. To date, no cases of pavement heave have been reported in Kentucky at sites where subgrades have been treated chemically at various locations in Kentucky. Swelling did occur on two sections of KY 11. However, high levels of soluble sulfates were present in the FGD byproduct admixture and not in the soils. Other subgrade sections on this route were treated with hydrated-lime and cement. No swelling occurred. Although no cases of pavement swelling have been reported to date, using hydrated lime and cement as chemical subgrade admixtures in certain geological regions of Kentucky could potentially cause swelling problems. For example, the residual soils of the New Albany Geologic Unit have the potential to pose problems. This unit contains pyrite, which is high in sulfur content. Identifying soils high in sulfate content was beyond the scope of this study. Additional research is needed for identifying suspect areas. Moreover, the use of FGD by products in highway applications will not be realized until the swelling nature of those materials is fully understood and methods developed to control swelling.

**RAPID FIELD EVALUATIONS OF IN SITU SUBGRADE STRENGTHS**

A final objective of this study consisted of developing rapid methods of evaluating the in situ bearing strength of untreated and treated subgrades. The dynamic cone penetrometer and the Clegg impact hammer were selected for evaluation. Many dynamic cone penetrometer tests, in situ CBR tests, and unconfined compression tests were performed on newly constructed highway subgrades. Correlations were developed between dynamic cone penetrometer values, unconfined compressive
strength, and CBR tests. Additionally, Clegg impact hammer values were correlated with unconfined compressive strengths. These correlations are being used by engineers of the Kentucky Transportation Cabinet to obtain a rapid evaluation of the strength characteristics of treated and untreated highway subgrades.

SUMMARY

A major subgrade stabilization program was established for the construction of highway pavements in Kentucky. Many issues are addressed and resolved in developing and implementing this program. Field monitoring of the performances of selected roadway sections will continue. To date, data show that flexible pavements constructed on untreated subgrades are performing very well. Moreover, the collected data show that the strengths of chemically treated subgrades are several times larger than the untreated layers.
INTRODUCTION

Problem Statement

Pavement subgrades must be stable during construction and perform throughout the design life of the pavement. Frequently, the subgrade is the weakest member of the pavement structure and is one of the most important factors influencing pavement performance. The subgrade during construction must be sufficiently stable to prevent rutting and shoving. The subgrade must also provide a stable platform to construct the various pavement layers effectively and efficiently. First, the subgrade must serve as a "working platform." Secondly, it must have sufficient strength so that large permanent deformations do not accumulate over time and affect the performance of the pavement. Therefore, design guidelines must not only consider the required thickness of the pavement layers but they also must consider the issue of whether the pavement can be safely constructed without failure. This latter consideration was for many years neglected -- that is, the question of constructing the pavement without failure was left to the field and geotechnical engineers to confront and solve.

Many pavements in Kentucky, and in other states, have been, and are continuing to be, constructed on soil subgrades of poor or marginal engineering properties. Typically, the pavement subgrades consist of fine-grained soils, such as clay or silty clay, of low-bearing strengths. This has lead to premature failures of the pavements and failures during construction. Occasionally, pavement failures during construction have been reported and have required expensive remedial solutions. Although pavements of sufficient thicknesses may be designed for low bearing subgrades, the question that arises is whether pavements can be constructed as designed.

The question posed above should perhaps, be viewed in another perspective. What are the cost and consequences if subgrade improvements are not made? The Geotechnical Branch of the Division of Materials (Kentucky Department of Highways) reportedly has investigated many subgrade problems on new pavement construction sites in past years (Smith 1989). According to Smith (1989), some forty highway projects (May 1986 to November 11, 1989) required some type of subgrade modification. Additionally, the University of Kentucky Transportation Center has investigated several pavement failures over the past several years (for example, Hopkins and Sharpe 1985; Sharpe 1988; Hopkins and Allen 1986; Graves 1989; Williams, et al, 1984; Sharpe and Deen 1987; Hunsucker 1989 (KY 94); Graves 1989; Hunsucker 1989 (US 60); Drake and Havens 1959; Graves and Sharpe 1989; Hopkins 1991). Most of these problems required some type of remedial treatment before the placement and compaction of pavement layers. So, considerable efforts were already being devoted to improving the subgrade during construction. Moreover, weak subgrades -- deep ruts and shoving of the subgrades -- cause considerable construction delays not only to the contractor but
to the state's engineers when remedial measures are required. Consequently, development of subgrade modification or stabilization design guidelines is a logical process to establish - that is, if unstable subgrades are frequently encountered, then why not establish a procedure to avoid this problem? Many pavements have failed during construction over the past several years requiring additional cost to repair. Also, many pavements have failed prematurely, or before their design lives have expired because of weak subgrades. Although exact figures cannot be assigned to this problem, the costs are believed to be substantial.

Objectives

The major objective of this study was to determine the long-term benefits of chemical and mechanical stabilization of soil subgrades. Other objectives include developing and evaluating laboratory testing procedures, developing a method of designing the thicknesses of soil subgrades treated with chemical additives, and developing criteria for determining when subgrade modification is needed or required. The objectives also included observing long-term field strengths of stabilized subgrades and comparing field and laboratory strengths of stabilized subgrades and strengths of untreated specimens.

Scope

Many immediate benefits are obtained from subgrade stabilization, especially chemical admixture stabilization. For example, by improving the bearing strength and stiffness of the subgrade, a good working platform is established for supporting construction traffic and for compacting paving materials. Additionally, subgrade soils that have poor engineering properties may be used effectively when chemical stabilization is used. Therefore, construction can continue efficiently.

Although short-term benefits of subgrade stabilization are readily apparent, more information regarding long-term benefits is needed. Before 1987, only a few chemically-treated subgrade stabilization projects were constructed in Kentucky, although many subgrades were stabilized by mechanical means. For example, when chemical admixture stabilization is used, a question arises concerning the durability and longevity of the treated subgrade. However, well-documented case studies were not published. Since 1987, several chemical and mechanical stabilization projects have been built. Major aims of this study were to examine several selected subgrade stabilization projects in more detail and consolidate information so that long-term benefits may be documented and evaluated. Other goals were to develop stabilization guidelines and establish a major subgrade stabilization in Kentucky. This report mainly focuses on the long-term benefits of chemical admixture stabilization.

To establish and implement a subgrade stabilization policy and program, many issues
These issues, which are discussed in this report, include the following:

- Factors that affect and influence the short-and long-term behaviors of untreated pavement subgrades.
- Minimum subgrade strength required to sustain construction traffic loadings and prevent bearing capacity failures of the subgrade.
- Use of laboratory strengths to predict long-term field strength of subgrades.
- Method of selecting design strengths of untreated and treated subgrades.
- Types of stabilization methods.
- Method of determining the optimum percentage of a chemical admixture when chemical stabilization is used.
- Treatment depth required to sustain construction traffic loadings when chemical admixture stabilization is used.
- Comparison of the long-term strengths of treated subgrades to the long-term strengths of untreated subgrades.
- Affect of wetting-drying behavior on strengths of untreated and chemically stabilized subgrades.
- Longevities of subgrades treated with hydrated lime and cement.
- Rapid methods for the assessment of the overall bearing strengths of untreated and treated subgrades.
- General performances of flexible pavements constructed on chemically treated subgrades and the potential for reducing maintenance.
- Cost of chemical admixture stabilization.
- Soil subgrade conditions where hydrated lime and cement should not be used.
Physical Properties

The mechanical behavior and performance of pavements are controlled by the physical properties of the materials used to construct the individual layers of the pavement structure. Some important physical properties include moisture content, compaction, and swelling. These factors affect the shear strengths of the materials that are available to resist stresses imposed by traffic loadings.

Moisture

The method normally used to stabilize fine-grained soil subgrades is mechanical compaction. Compaction specifications for soil subgrades usually require that placement dry density and moisture content conform to stated criteria. For example, many specifications require that the dry density of the soil subgrade at placement be 95 percent of the maximum dry density obtained from the standard laboratory compaction procedure (AASHTO T 99 or ASTM D 698). The placement moisture content should not be greater than 2 percent, nor less than 2 percent of optimum moisture content. Many soils, when initially compacted to conform to such criteria, may have adequate bearing strength to withstand, without failure, construction traffic loadings and traffic loadings shortly after the pavement is constructed.

However, the bearing strengths of fine-grained soils are very sensitive to changes in moisture content. Regarding moisture content of soil subgrades, two problems may arise. First, if the placement moisture content of the compacted subgrade exceeds the optimum moisture content of the soil, that is, the placement water content is too wet of optimum moisture content, then inadequate bearing strength may result. As the moisture content of a compacted, fine-grained soil subgrade increases, decreases in the undrained shear strength, or bearing strength, occur. The decrease may be explained by the principle of effective stress (Terzaghi 1943). This principle is useful in viewing the mechanical behavior of subgrades subjected to loadings of traffic. Simply stated, the principle is as follows:

\[ \sigma'_n = \sigma_n - u \]  
\[ \tau = c' + \sigma'_n \tan \phi' \] 
or,  

\[ \sigma'_n = \sigma_n - u \]  
\[ \tau = c' + \sigma'_n \tan \phi' \]  
or,
\[ \tau = c' + (\sigma_n' - u)\tan \phi' \]  

where  
\( \tau \) = shear strength of soil subgrade,  
\( c' \) = the effective stress parameter, cohesion,  
\( \sigma_n' \) = effective normal stress (the numerical value of this parameter depends on the stresses induced by traffic loadings, stiffness of materials, and pore water pressures),  
\( \sigma_n \) = total normal stress,  
\( \phi' \) = the effective stress parameter, angle of internal friction,  
\( u \) = total pore water pressure

As shown by Equation 3, the shear strength of the subgrade consists of two parts: the cohesion, which does not depend on pore water pressure, and the internal friction, which depends on pore water pressure. Effective stresses, or pressures, are transmitted through the points of contact between soil particles whereas the pore water pressure, or "neutral stresses," are transmitted through the pore water space between particles.

For example, stability of the subgrade during construction is controlled by the amount of stresses imposed by traffic wheel stresses and the shear strength, \( \tau \), of the subgrade available to resist failure. As shown in Equation 3, the available strength depends on the magnitudes of \( \phi' \) and \( c' \) and the pore water pressure acting within the subgrade media. The total water pressure, \( u \), acting within the subgrade may be viewed as consisting of three parts, or

\[ u = u_s + u_{ss} + \Delta u, \]

where  
\( u_s \) = static pore water pressure,  
\( u_{ss} \) = pore water pressure due to flow, or artesian flow, and  
\( \Delta u \) = hydrostatic, or excess, pore pressure at a selected point in the subgrade due to transmitted stresses induced by the applied traffic stresses.

Usually, \( u_s \) and \( u_{ss} \) are small (however, \( u_{ss} \) may become large at the bottom of long, steep highway grades.) Typically, the major part of the total pore water pressure is due to the built-up of excess pore water pressure created by induced stresses of applied...
If \( \Delta u \) is large, (a situation created by the low permeability of the clay) then the shear strength available to resist failure may be reduced significantly, as shown by Equation 5. That is, since \( \Delta u \) is large, the term, \((\sigma_n - \Delta u)\tan\phi'\), decreases and the available shear, \( \tau \), decreases. Instability may occur when \( c' \) is small and the magnitude of \( \Delta u \) approaches the value of \( \sigma_n \). For clayey soils, which initially may have large values of \( c' \), the subgrade may be stable, although \( \Delta u \) may be large. However, when a clayey soil swells, the strength decreases because the value of \( c' \) decreases--a condition that is sometimes called "strain softening." Moreover, repeated loadings of the clayey subgrade will cause a gradual decrease in \( c' \). With a loss in cohesion and the creating of large values of \( \Delta u \), the available shear strength becomes very small and may even approach zero. That condition causes the subgrade to "pump," or liquefy. When this occurs, clay particles may intrude into the voids of a granular base, move into cracks in the pavement, or cause pavements to crack. With movement of the clayey particles, voids may be created in pavements, especially concrete pavements, and cause sections of the pavements to collapse.

Silty and sandy materials usually do not have a cohesive strength component, or Equation 5 becomes,

\[
\tau = (\sigma_n - \Delta u)\tan\phi' \tag{6}
\]

and the strength available to resist loading stresses depends entirely on the magnitude of \( \phi' \). Therefore, if induced stresses are sufficiently large, then \( \Delta u \) may approach \( \sigma_n \) and

\[
\tau \longrightarrow 0, \text{ or } \tag{7}
\]

the silty, or sandy, material may liquefy, (or "pump"), although the value of \( \phi' \) may be large. Although the permeability of silty and sandy materials is larger than the permeability of clays, the permeability under instantaneous loadings of traffic is still not sufficient to dissipate excess pore water pressures instantly. Therefore, excess pore pressures may build up rapidly. Even in granular bases, which contain only a small percentage of fines, this condition may occur--that is, large excess pore pressures build up and move the fines laterally and vertically through joints. In concrete pavements, the fines have been observed at the surface of joints.
When subgrades are stabilized with chemical admixtures, such as hydrated lime or cement, a large cohesive strength component is created. Strong bonding forces are created between the soil particles, and

$$\tau = c'_a + (\sigma - \Delta u)\tan \phi'_a. \quad (8)$$

Now, usually the cohesion, $c'_a$, of the treated material is much larger than the cohesion, $c'$, of the untreated soil. Some evidence (Hopkins, 1993) exists that $\phi'_a$ of the treated material is slightly (two to four degrees) larger than $\phi'$ of the untreated material. Although the value of $\Delta u$ (in Equation 8) may approach $\sigma_n$ and the frictional strength may approach zero, the large cohesive component, $c_a$, may be sufficient to prevent instability. Field strengths of hydrated lime-and cement-treated subgrades that have been in place for several years, as described herein, suggest that this may be the case.

Normally, soil compacted near optimum moisture content has a degree of saturation of above 80 to 85 percent. The size of $\Delta u$ in the compacted soil depends on the permeability and the size of the applied wheel stresses. If the applied stresses are sufficiently large, and the soil is wet of optimum, then the compacted soil may compress under the wheel stresses. As the soil compresses, the degree of saturation rapidly approaches 100 percent and $\Delta u$ increases dramatically in soils of low permeability. Consequently, if the degree of saturation is below 100 percent (unsaturated state), pore water pressures are small -- or $\Delta u$ is equal to zero-- and the value of $\tau_{us}$ is the shear strength of the unsaturated soil, then the shear strength of the compacted soil is large. Note that the term, $\tan \phi'$, in Equations 1 or 2, is multiplied by the term, $(\sigma_n-\Delta u)$, and

$$\tau = c' + (\sigma_n - \Delta u)\tan \phi' = \tau_{us} = c' + \sigma_n\tan \phi'. \quad (9)$$

The available shear strength of the saturated soil is less than the available shear strength of the unsaturated soil. Therefore, the shear strength decreases as the placement water content increases above optimum water content. According to Thompson (1988) and based on two Illinois interstate highway sections (each was about 16.1 km, or 10 miles, in length), sizable quantities of subgrades were placed wet of optimum. On these projects, 43 percent of the soil embankments were placed wet of optimum moisture content. According to Thompson, similar data developed by the Illinois Department of Transportation and others show that there is a tendency during embankment construction to place soils wet of optimum moisture content.

Secondly, when clay, or silty clay, subgrades are left exposed during construction to rainfall and snow melt for a considerable time before the pavement layers are placed,
they absorb water, swell, and increase in volume. As the degree of saturation approaches 100 percent and with an increase in volume, the shear strength, or bearing strength, as shown by Equation 2, decreases.

The moisture content of soil subgrades after construction of the pavement is not a static condition and does not necessarily remain the same as the placement moisture content. Therefore, the moisture content of the subgrade varies as seasonal changes occur in rainfall, snow melt, and temperatures. The complexities of moisture migration and changes in subgrade moisture content have been well documented by Dempsey and Elzeftryway (cf. Thompson 1988). Since bearing strength is related to moisture content, the bearing strength varies with seasonal changes as well. An important principle that must be acknowledged is that subgrade moisture content may change after pavement construction from the placement moisture content. Several studies (Traylor et al., 1976, cf. Thompson 1988, Liu, et al, 1964, Knight 1961) have shown that moisture content is the most significant factor controlling the strength and stiffness of fine-grained soil. However, Langfelder (1964, cf. Thompson, 1988) showed that placing a soil dry of optimum moisture does not necessarily insure that the moisture content will not increase. Therefore, mechanical compaction (based on standard compaction), when used alone, may not be sufficient to provide adequate bearing strength during construction and after placement of the pavement.

To insure sufficient bearing strength throughout construction and the life of the pavement, other methods, in combination with mechanical compaction should be considered. Some types of chemical stabilization admixtures include hydrated lime, quicklime, cement, lime kiln dust, and fly ash. Some benefits of chemical admixture stabilization are (Terrel, et al 1979):

- Speeds construction
- Improves bearing strength of subgrade soils
- Increases stiffness of subgrade soils
- Decreases swell potential of subgrade soils
- Improves subgrade durability
- Subgrade soils having poor engineering properties may be used effectively and represent a good economical alternative to the use of other materials.

Water may enter pavement layers in several ways. Surface water from rainfall or snow melt seeps into layers through surface cracks, joints, through the unpaved portions at the edges of shoulders, and through the pavement itself. Water may enter the pavement layers by subsurface seepage from water bearing rock strata in cut and fill transitional zones. Although the water table may be found at an elevation below the bottom of the base courses, the clay subgrade may increase in moisture content because of a capillary rise. It is generally recognized that water in pavement layers is detrimental to the performance of the pavement.
The greatest opportunity for damage to occur to a subgrade, and the future source of damage to the pavement, exists during construction. During this critical period, the subgrade is exposed to a rainfall, or snow melt, which seeps into the subgrade. Clayey subgrades absorb water and swell. With an increase in moisture content and volume, the degree of saturation approaches 100 percent. Both the cohesive shear strength component and angle of internal friction decrease, that is, the bearing strength is lowered. When the degree of saturation is 100 percent and the subgrade is loaded with construction traffic, excess pore pressures build up and, according to Equation 3, the shear strength available to resist failure is lowered. The stability of the subgrade is lowered. This situation occurs frequently when clayey subgrades remain exposed during the winter, or when heavy rainfalls (and/or snow melts) occur before placement of the pavement. Many damaged or soft subgrades, according to personnel of the Geotechnical staff of the Kentucky Transportation Cabinet, that have required remedial measures are of this nature (Smith 1989). The degree of saturation of clayey subgrades typically increases from an initial value of 80-85 percent to some value higher than the initial value.

Compaction problems occur when attempts are made to construct granular base courses and bituminous courses on subgrades that have increased in moisture content and saturation. Difficulties are encountered in achieving proper compaction of the granular base and bituminous layers because of the weakened or soft condition of the subgrade. Stresses due to heavy compactors are transmitted through the pavement layers to the saturated subgrade. These stresses cause an instantaneous increase in excess pore water pressure in the subgrade. As shown by Equation 3, the shear strength available to resist failure decreases when pore water pressure increases.

**Compaction**

Generally, many engineers assume that if a clayey subgrade is compacted to 95 percent of standard maximum dry density and close to optimum moisture content (AASHTO T 99 or ASTM D 698), then pavement construction problems and future pavement problems will not occur. This is true in many situations. Compaction increases the shear strength of soils. As the compactive effort increases, the cohesive component increases; the angle of internal friction increases slightly (Hopkins, January 1980). When fine-grained soils, such as clays or silts, are compacted in the subgrade, the degree of saturation of the compacted material is approximately 80 to 85 percent. The degree of saturation is a measure of the portion of void space in a soil mass filled with water -- the ratio of the volume of water to the volume of voids. When a subgrade is initially compacted to "standard compaction" and the volume of compacted subgrade remains unchanged under wheel stresses, the build up of excess pore water pressure cannot occur (Au equals zero) since the degree of saturation is below 100 percent, as shown by Equation 1. If the number of traffic load applications is increased, and the size of induced stresses is sufficiently large, then the fine-grained
subgrade may reduce in volume, or compress. With a reduction in the volume of material positioned under the wheel stresses, the degree of saturation increases and eventually may reach 100 percent. When this occurs, the situation is created under which large, excess pore water pressures, $\Delta u$, increase, and the shear strength available to resist failure decreases rapidly since the term, $(\sigma_n - \Delta u) \tan \phi'$, decreases. The available strength essentially decreases. Under repeated loadings, the cyclic action of excess pore pressures created by repeated loadings eventually destroys, or decreases, the cohesive component and the available shear strength to resist failure approaches zero. As a result, the subgrade may fail.

**Swelling**

Although a subgrade constructed of fine-grained soils may be compacted according to specifications, the subgrade soils may not remain in the same state as they were originally compacted. The likelihood that the original compactive state will change with increasing time, and load applications, is very probable. Fine-grained soils, especially clayey soils, in a compacted state have a large potential to absorb water and swell. When a compacted soil swells, the volume increases and the shear strength available to resist failure decreases. Swelling of clayey subgrades lowers the cohesive component rapidly and causes a slight decrease in the angle of internal friction (Equation 3). Therefore, the total shear strength available to resist failure is lowered. The shear strength available at some time after compaction may be much lower than the strength available at the time of initial compaction. Most clayey soils existing in a compacted state swell when exposed to a source of moisture.

The effect of swelling on the bearing, or shear, strength of clayey soils due to absorption of water is illustrated by data, Figure 1, obtained from the Kentucky Geotechnical Data Bank (Pfalzer, et al, 1995). These data illustrate how the bearing strength of many soils may be reduced when soaked in water and allowed to swell. In this figure, the KYCBR (Drake and Havens, 1959; Kentucky Methods
Manual 1987) values of unsoaked and soaked specimens are compared. Some 727 values of unsoaked and soaked values of KYCBR were available for comparison. In all these tests, each specimen was penetrated before and after soaking. The effect of swelling on bearing strength is also illustrated by data shown in Figure 2 (Hopkins 1983; Hopkins 1988). The values of CBR of unsoaked specimens of several different types of compacted shales are compared to CBR values of the same specimens soaked and given full opportunity to swell.

![Figure 2. Comparison of soaked and unsoaked values of KYCBR for a number of selected, typical Kentucky Shales.](image)

Except for specimens identified as "New Albany," "Hance," and "Drakes," a significant decrease occurred in the value of CBR for each specimen after soaking. The various types of shales represented in Figure 2 have been used often in Kentucky to construct pavement subgrades and have caused many pavement problems. A significant aspect of the data in Figure 2 is the large values of KYCBR of the compacted shales in an unsoaked state. The unsoaked values ranged from about 15 to 42. Materials that have bearing strengths of this size could easily withstand most construction traffic loadings without serious rutting or failure. When materials of this nature are initially compacted, serious problems are not normally encountered if they remain in this initial compactive state and the placement of water content is near or lower than the optimum water content. After soaking, the KYCBR values of the specimens range from about 0.5 to six when the KYCBR values of the specimens identified as New Albany, Hance, and Drakes are excluded. These three shales are very sandy and silty and do not degrade when exposed to water. The other specimens are clayey shales and degrade into flakes when exposed to water. Particles of these shales have a great affinity for water. The enormous decrease in the CBR values after soaking varies with the magnitudes of vertical swell (or strain) -- as measured in the CBR test -- and the clay-sized particles smaller than 0.002 mm, as shown in Figures 3 and 4, respectively. As shown in Figure 3, the trend of unsoaked values of CBR is essentially constant; it increases slightly as the vertical swell increases. After soaking, the soaked value of KYCBR decreases as the vertical swell increases. When the vertical swell is greater than about
4 percent, the soaked CBR strength is less than six. The magnitudes of KYCBR values of the compacted shales seem closely related to the percent finer than 0.002-mm size particles, as shown in Figure 4. As the percent finer than 0.002-mm size particles increase, values of both unsoaked and soaked values of CBR decrease. If the percent of clay-size particles finer than 0.002-mm size is less than about 15-25 percent or the value of swell is less than about 4 percent, the soaked CBR is greater than six. As shown later in this report, when the CBR of a subgrade is equal to or greater than six, sufficient strength is usually available to withstand most construction traffic loadings without failure.

When compacted shale or clayey soil absorbs water, the degree of saturation approaches 100 percent. When a source of water is readily available for a substantial period, the swelling soil eventually becomes completely saturated and the shear strength is lowered in two ways. First, as the soil swells, the volume increases as the moisture content increases. The cohesive component of strength (Equation 3) decreases. A slight change, or decrease, also occurs in the angle of internal friction. Secondly, when construction traffic loads clayey subgrades, sufficiently large, excess pore water pressures occur and the shear strength is further reduced as illustrated by Equation 3. Normally, since granular bases and bituminous layers of pavements do not swell, the shear strengths of these materials are not affected.
Geologic Setting and Soil Types

Soils and geology influence the behavior and performance of highway pavements. This observation is generally recognized by many engineers; however, this aspect is often ignored. The types of soils usually found in a given geologic setting are used to construct the pavement subgrades. The types of soils at a given highway site are usually controlled by the type of geological formation existing at a given location. For example, in certain locations in Kentucky, pavements constructed on compacted, clayey shales or residual soils from such geologic units as the Kope and Crab Orchard frequently undulate and show very visible signs of distress. Often, the pavements fail prematurely. Residual soils derived from the Kope geologic unit in the northern portion of Kentucky are highly plastic and swell when exposed to water.

Bearing capacity failures of many city streets in the Kope Shale region of Kentucky are extensive. In these townships, specifications permit placing four or five inches of concrete directly on subgrades constructed of the very plastic and weak residual soils of the Kope geologic unit. When concrete is used, no drainage courses or granular base materials are specified. Sections of these concrete city streets were observed to have completely collapsed. Compacted, residual soils of the shale formation of this area, absorb water, swell, and become saturated (the pores of the compacted clays are completely filled with water). Collapse of sections of the concrete streets in this area occurred because of three conditions. When the subgrade soils swell and change volume, the shear strength of the soil decreases. Secondly, when the subgrades become saturated and the thin concrete pavements are loaded with large wheel stresses (due to heavy garbage trucks, concrete trucks, etc.), large excess pore pressures, $\Delta u$, build up under the wheel stresses. According to Equation 5, with an increase in $\Delta u$, the shear strength decreases and promotes instability. With a build up of large excess pore pressures, a condition is created under which the dissipation of the excess pore pressures moves clay particles outward from beneath the concrete pavement, or upward at joints in the concrete pavement. With a loss of clay particles or material supporting the pavement, voids are created and the pavement collapses. Even pavements of Interstate 75 that pass through the area have required large remedial expenditures. Pavements of this interstate route that pass through this area characteristically undulated and were distressed, bumpy, and contained failures at joints (collapses) in the concrete sections before a thick bituminous overlay was placed.

Similar situations occur at other locations in Kentucky where pavements have been placed on highly plastic, clayey shales and residual clays from such geologic units as the Crab Orchard, Clays Ferry, and New Providence. The highly plastic clay areas around Elizabethtown (Midwestern Kentucky) have caused many pavement problems before and after construction. These residual soils were derived from limestone and shale geological formations. In a study by Hopkins and Sharpe, 1985, a section of the southbound lanes of Interstate 65 in this area failed during construction under
construction traffic loading (gravel trucks). The partially completed pavement and dense graded aggregate (DGA) cracked, rutted, and deformed under construction traffic. Large deflections were observed shortly after part of the pavement was placed. Analyses showed that failure occurred because of a bearing capacity failure of the plastic clayey subgrade. Values of undrained shear strengths were as low as 40 kPa (5.8 psi), and averaged 60 kPa (8.7 psi). Careful examination of several specimens (thin-walled, tube samples) obtained from the subgrade of the southbound lanes showed that the top 76 to 127 mm (3 to 5 in.) of the clayey subgrade was extremely soft—the material was easily indented with the finger. Material below the soft zone was very firm. Preconsolidation stresses obtained from consolidation tests of soil specimens from the soft zone were extremely low. Estimated stresses, due to traffic loadings and the weight of material above the clayey subgrade, showed that the preconsolidation pressures of the soft soils were smaller than the imposed stresses at the elevation of the soft layer of the subgrade. Both bearing capacity and punching shear failures occurred. Case studies such as this emphasize the need to analyze the bearing capacity of pavements before and during construction. Such cases illustrate the important role that geology and soil types play in the performance of pavements. Moreover, the consolidation characteristics of subgrades need to be examined. Other case studies similar to this case will be examined in depth later in this report.

Although courses of granular bases are used to drain water from the pavement structure, flowing water in the granular bases usually contacts the soil subgrade since the base rests directly on the subgrade. This condition exposes the subgrade soils to water, part of which seeps into the subgrade and is absorbed. During periods of flow, the soils of the subgrade have an opportunity to swell and soften. Consequently, shear strength decreases. The top portion of the subgrade becomes saturated. When this occurs and the pavement is subjected to stresses of traffic, the excess pore pressures build up. The shear strength available to resist failure decreases as shown by Equation 5. (One method that might be tried— at least experimentally— to prevent the water that flows through the base material from contacting a clay subgrade involves spraying the finished subgrade with an asphalt emulsion, or using a geomembrane cover. Currently, asphalt emulsion coatings are used to cure chemically-treated subgrades).

A view of the types of soils found in Kentucky, and the types of soils that are most likely to be used to construct pavement subgrades, may be obtained by analyzing engineering soils data contained in a geotechnical data bank developed for Kentucky. These data are the result of basic geotechnical tests performed on specimens obtained throughout Kentucky. The data bank contains some 20,000 records and the data have been accumulated over the past four decades. Examination of data in this data bank show (statistically) that about 70 percent of the soils in Kentucky classify as clays and silts as shown in Figure 5. About 16 percent of the soils are fat clays and silts. Only about 14 percent of the soils in Kentucky consist of clayey, silty sands and sands, or
clayey, silty gravel or gravel. About 86 percent of the soils in this state are materials of poor engineering quality and the likelihood of these poor engineering soils being used to construct pavement subgrades is very high. The likelihood of pavement construction problems occurring in Kentucky is considerable high. This problem occurs in many other areas of the country where clayey soils exist.

BEARING CAPACITY MATHEMATICAL MODEL

The mathematical bearing capacity model used herein is based on limit equilibrium and may be used to calculate the factor of safety against failure. This model is used to analyze the stability of initial construction conditions and the completed flexible pavement structure. Problems to be analyzed are visualized in Figures 6 and 7. Theoretical considerations and mathematical derivations of limit equilibrium equations for analyzing the ultimate bearing capacity of soil subgrades and partially completed asphaltic pavements, and the extension of those equations to the analyses of asphaltic pavements composed of multiple layers, have been presented elsewhere (Hopkins 1994) and are beyond the scope of this report. Each layer of material--subgrade, base, and asphaltic layers--in the pavement structure is described in the model using shear strength
parameters, the angle of internal friction, $\phi$, and cohesion, $c$, and unit weights. Problems involving total stress and effective stress analyses may be solved.

The assumed theoretical failure mass consists of three zones -- active and passive wedges connected by a central wedge whose shear surface is a logarithmic spiral curve. The shear surface assumed in the model analysis for a homogeneous layer of material consists of a lower boundary, identified in Figure 6, as abcd. This surface consists of two straight lines, ab and cd. The portion of the shear surface shown as line ab is inclined at an entry angle, $\theta_1$. Line cd is inclined at an exit angle, $\theta_2$. Angles, $\theta_1$, and $\theta_2$, are defined in Figure 6. The shear surface, bc, is determined from the properties of a logarithmic spiral. For a layered system, the shear surface is visualized as shown in Figure 7. Correct entry and exit angles are used in each material of a layered structure.

The approach is a generalized method of slices and is an adaptation of a slope stability method developed by Hopkins (1986). Vertical, horizontal, and moment equilibrium equations are considered for each slice. In the solution of these equations, the factor of safety appears on both sides of the final equation. Iteration and numerical techniques are used to solve for the factor of safety. To ease the use of the approach, all algorithms were programmed on the mainframe computer (IBM® 3090) at the University of Kentucky.

Although the theoretical bearing capacity model may be used to analyze a layered system, two classes of early construction problems are only considered below. In the first class, the soil subgrade is assumed to consist of a homogeneous layer of material of an infinite depth. The second class of problems involves two layers of different
MINIMUM BEARING STRENGTH OF SOIL SUBGRADES REQUIRED TO CONSTRUCT FLEXIBLE PAVEMENTS

In the construction of flexible pavements, the subgrade must possess some minimum bearing, or shearing, strength to support anticipated tire contact stresses of construction traffic, to avoid undesirable tire sinkage, and to avoid shoving and pushing of the subgrade. Periods when the subgrade is most susceptible to failure occur when the subgrade serves as a construction platform and when the first lifts of the pavement are placed. Inadequate bearing strength of the subgrade may lead to instability and cause costly delays and require remedial measures. Moreover, when bearing capacity failures occur in clayey subgrades during construction, weak zones, or shear planes, are created in the subgrades because the peak strength decreases to a residual shear strength. The existence of weakened shear planes has the potential to cause cracking of the pavement long after completion of construction and paving. One major objective of this report was to determine the minimum bearing strength required to avoid bearing capacity failures during construction for any given tire contact stress. Another major objective was to develop guidelines for determining when subgrade modification, or stabilization, may be needed. Knowing when to modify a soil to improve bearing strength is essential to the development of sound and economical plans before construction and to assure the efficient construction of the pavement. To determine a minimum strength for any given average, tire contact stress, theoretical bearing capacity analyses were performed. Results of these analyses are compared to published results obtained from field tests.

Minimum Subgrade Strength - Theoretical Analysis

Undrained Shear Strength

The minimum strengths of a subgrade required to maintain stability vary with tire contact ground stresses (Hopkins 1991; Hopkins et al, 1994). As the contact stress increases, the required strength increases. This situation, as visualized in Figure 6, was analyzed using the bearing capacity model described above. A range of undrained shear strengths of the soil subgrade, dual-wheel tires, and a range of tire contact stresses (uniformly distributed) were assumed. The relationships of undrained shear strength and tire contact ground stresses corresponding to factors of safety of 1.0 (incipient failure) and 1.5 (an assumed stable condition) were developed. For a
Relationships developed in this manner are shown in Figure 8.

Therefore, if the anticipated tire contact stress of construction traffic is known, then the required strengths to maintain an incipient failure condition ($F=1.0$) or an assumed stable condition ($F=1.5$) may be determined. For example, if the tire contact stress is 552 kPa (80 psi), then the undrained shear strength for an incipient failure is 94 kPa (13.6 psi) and about 144 kPa (20.9 psi) for an assumed stable condition.

**CBR Strength**

Considering that many agencies have used, and some continue to use the CBR (California Bearing Ratio) for expressing the bearing strength of subgrades and as a design parameter in pavement design schemes, and considering that the theoretical bearing capacity model uses undrained shear strength in total stress analysis as a design parameter, determination of the minimum subgrade CBR corresponding to a given tire contact stress is desirable. An approximate relationship between CBR
and cohesion, or undrained shear strength, $S_u$, is shown in Figure 9. Development of this relationship has been described elsewhere (Hopkins 1991). Based on the relationship in Figure 9, minimum CBR values of soil subgrades may be established as a function of tire contact stress as shown in Figure 10. For a tire contact stress of 552 kPa (80 psi), the required bearing ratio for an incipient failure is about 6.5 and 10 for an assumed stable condition. For other, selected values of tire contact stress, the required bearing ratios may be obtained from the relationships shown in Figure 10. Consequently, if the undrained shear strength, or CBR value, of the compacted subgrade at a selected tire contact stress is less than the values given in Figures 8 or 10, then soil subgrade modification should be considered for increasing bearing strengths to the minimum values required for stability.

**Dynamic Modulus of Elasticity (Modular Ratio)**

Minimum dynamic modulus of elasticity, $E_s$, required to maintain incipient stability and a stable condition may be approximated using the relationship developed by Heukelom et al. (1960 and 1962). Re-analyses of those data, as shown in Figure 11, yield

$$E_s = 17,914 \text{ CBR}^{0.874} \text{ (kPa)}$$ (10)
Inserting the CBR values of 6.5 and 10, which correspond to a tire contact stress of 552 kPa (80 psi), into Equation 10, the required values of dynamic modulus of elasticity to maintain an incipient failure state and an assumed stable condition are 91,977 kPa (13,349 psi) and 134,027 kPa (19,452 psi), respectively.

**Minimum Subgrade CBR Strengths -- Field Studies**

Thompson in 1988 cited two studies of field subgrade strengths and showed relationships among tire inflation pressures, field CBR values, and tire sinkage. Relationships between CBR and sinkage values for tire inflation pressures ranging from 345 kPa (50 psi) to 552 kPa (80 psi) are shown in Figure 12. For this range of tire pressures, the minimum CBR strength of the subgrade required to limit tire sinkage to 0.64 cm (0.25 inches), or less, must be between about six and nine, respectively. Data labeled as "Kraft" in Thompson's paper (1988) were re-analyzed (Hopkins 1991; 1994) to obtain an equation that relates tire inflation pressure ($T_e$), CBR, and tire sinkage ($S$). Results of these reanalyses are shown in Figure 12 and may be expressed in the approximate form as:

$$\begin{align*}
\text{CBR} &= \frac{1}{\frac{41.68}{T_e}(1+0.394S)+0.0787S-0.034}
\end{align*}$$

For example, the CBR of the subgrade required to limit tire sinkage to 0.64 cm (0.25 in.) is about 9.0 for a tire inflation pressure of 552 kPa (80 psi).

At small sinkage values, the two different sets of curves yield almost identical results. For example, at a sinkage of 0.64 cm (0.25 in.), the CBR values of both sets of curves range from about six to 8.5, or 9, for tire inflation pressures varying from 345 kPa to 552 kPa (50 to 80 psi). However, at larger sinkage values, the two different sets of relationships yield different results. For instance, at a sinkage of about 7.6 cm (3 in.), the curves identified as "Kraft" yields a CBR value of about one for different tire inflation pressures. The sinkage-CBR relationships identified as "Rodin" show that...
at a tire sinkage of 7.6 cm (3 in.), the CBR values range from 4.5 to 6.8 for tire inflation pressures ranging from 345 kPa to 552 kPa (50 to 80 psi). However, as shown by both sets of relationships, the CBR strength must be in an approximate range of six to nine to limit tire sinkage to a small value (say, 0.64 cm, or 0.25 in.).

To determine the factors of safety of subgrades subjected to different tire pressures in the field experiments, theoretical model analyses were performed. The intent of these analyses was also an attempt to relate a selected sinkage value to the factors of safety for tire inflation pressures ranging from 345 kPa to 552 kPa (50 to 80 psi). For example, at a selected sinkage value of 0.64 cm (0.25 in.), the field CBR strengths are approximately 8.5, 7.4, 6.5, and 5.5 as shown in Figure 12 (points 1, 2, 3, and 4). Undrained shear strengths for the model analyses were determined from the relationship shown in Figure 9 using those CBR strengths. The assumption was made -- which may not be strictly correct -- that the tire contact ground stress was approximately 90% of the tire inflation pressure. This assumption was based on data presented in Table 5 of the 1962 AASHO Road Test. Tire inflation pressure was about 518 kPa (75 psi). The tire contact stress was approximately 466 kPa (68 psi). The ratio of tire contact ground stress to inflation pressure is about 0.9. Tire contact stresses of 497, 435, 373, and 311 kPa (72, 63, 54, and 45 psi), which were used in the bearing capacity model analysis, correspond to tire inflation pressures of 552, 483, 414, and 345 kPa, (80, 70, 60, and 50 psi), respectively. If the subgrades were loaded with dual-wheels tires, factors of safety of 1.50, 1.54, and 1.57 were obtained for those contact stresses, respectively. CBR values associated with those factors of safety are shown in Figure 13 as a function of tire stress. These data are also compared to the curve shown in Figure 10 (F equals 1.5). The comparison shows that the factors of safety of the field tests (S equals 0.64 cm, or 0.25 in.) essentially coincide with the theoretical curve (CBR equals 0.0181T) in Figure 10.

Similar analyses were performed using points five, six, seven, and eight of Rodin's
curves (Figure 12) at a sinkage value of 7.6 cm (3 in.). Corresponding factors of safety of the field tests at that sinkage value were 1.20, 1.22, 1.24, and 1.27, respectively. CBR strengths associated with those factors of safety are shown in Figure 13 and compared to the curve obtained from the model analysis for a factor of safety of 1.23. A relationship obtained from the theoretical model analysis, which corresponds to a factor of safety of about 1.0, is also shown in Figure 13. This curve lies below the other two curves. Extrapolation of Rodin’s curves to larger sinkage values suggests that the sinkage associated with a factor of safety of 1.0 is about 13.2 cm (5.2 in.). The factor of safety from the theoretical bearing analysis is about 1.5 at low sinkage values of 0.64 cm (0.25 in.), or less. The required CBR strengths are about six to nine for tire contact stresses ranging from approximately 345 kPa to 552 kPa (50 to 80 psi).

For a tire contact stress of 552 kPa, the factor of safety, based on the above analyses using Rodin’s data, as a function of tire sinkage may be approximated as (Figure 14):

\[ F = 1.55 - 0.04S \]  

(12)

However, if the data identified as Kraft are analyzed, then the sinkage at a factor of safety of 1.0 is about 1.4 cm (0.55 in.). Nevertheless, both sets of relationships show that the CBR strengths must be about six to nine to limit sinkage to 0.64 cm (0.25 in.) and to maintain overall bearing capacity without large tire sinkage. The factor of safety should be about 1.5.

**Estimated Required Thickness of Modified Subgrades**

As shown in Figure 8 or 10, when undrained shear strength, or CBR, of the untreated subgrade is below a minimum value required to maintain stability, the bearing, or shear strength, must be improved to prevent instability during construction. Hydrated lime and cement are often used to improve the bearing strength of clayey subgrades of low strengths. However, treatment of the clayey subgrades with those chemical additives will not, necessarily, prevent instability. Bearing capacity of the treated layer depends on the thickness of the treated layer and the bearing strength of the
untreated layer found below the treated layer. To estimate thicknesses required to maintain an assumed stable condition, bearing capacity analysis was performed using the bearing capacity model described previously. In the analyses of this two-layered problem, the tire contact stress was assumed to be 552 kPa (80 psi). Unconfined compressive strengths of the hydrated lime-and cement-treated subgrades were based on 7-day, undrained shear strengths obtained from an analysis (Hopkins 1991; 1994) of several field stabilization projects. In the analysis, the bearing ratio of the untreated layer ranged from one to nine (or undrained shear strengths ranged from 15 kPa to 130 kPa, or 2.2 to 18.9 psi). Undrained strengths of the hydrated lime-and cement-treated subgrades assumed in the analysis were 333 kPa and 707 kPa (48 and 103 psi), respectively. Relationships between thicknesses of hydrated lime-and cement-treated subgrades and CBR of the untreated subgrade found below the treated layers are shown in Figure 15. For CBR values of the untreated layer ranging from one to nine, the required thicknesses of the hydrated lime-treated subgrade range from 41 cm to 11.4 cm (16.1 to 4.5 in.), respectively. Required thicknesses of soil-cement subgrades range from 21 cm to 7.6 cm (8.3 to 3 in.) when CBR values of the untreated layer range from one to seven, respectively.

Another technique often used in subgrade stabilization consists of placing a stone aggregate on the low-bearing subgrade soil. Relationships between thicknesses of the layer of stone aggregate, required to maintain factors of safety of 1.0 and 1.5, and CBR of the underlying untreated soil subgrade are shown in Figure 16. In these analyses, the tire contact stress was assumed to be 552 kPa (80 psi).
However, the theoretical model analyses may be performed for any assumed tire contact stress. The required thicknesses become very sizeable for very low-bearing soils. For example, if the CBR of the untreated layer is three, then the required thickness for an incipient failure state (F=1.0) is about 53 cm (20.9 in.). For an assumed stable condition (F=1.5), the required thickness is about 102 cm (40.2 in.).

Construction of the First Lift of Paving Materials

Sufficient shear strength of the untreated, compacted soil subgrade must exist to construct the first lift of a granular base of the flexible pavement or in constructing the first lift of a "full-depth," asphaltic pavement. If the soils of the untreated layer found below the first lift of paving materials are very soft, then instability problems may arise. Difficulties often arise in compacting the first lifts of paving materials when the subgrade is composed of clayey material. Clayey subgrades may become soft when they are exposed to rainy periods or when the subgrades may be exposed to winter conditions before paving. Moreover, bearing capacity failures may occur when construction traffic travels over the first lifts of paving materials.

In Figure 17, the relationships between the factor of safety and the first lift thickness of a granular base and asphaltic materials are shown. In the stability analysis of the granular base, a lift thickness of 15.2 cm (6 in.) was assumed. Shear strength parameters, $\phi$ and $c$, of the granular material were assumed to be 43 degrees and zero, respectively. Unconsolidated-undrained triaxial compression tests were performed on asphaltic core specimens to establish $\phi$- and $c$-values (Hopkins 1991). These tests were performed at controlled temperatures of 37.8°C and 60°C (100°F and 140°F). Water, flowing from a water bath with a temperature control, was circulated through coiled copper tubing, which surrounded each specimen in the triaxial chamber. At a temperature of 60°C (140°F), $\phi$- and $c$-values obtained from the triaxial tests were 42.9 degrees and 55.2 kPa, respectively. At 37.8°C (100°F), the $\phi$- and $c$-values of the asphalt cores were 35.9 degrees and 189 kPa, respectively. Thickness of the first asphaltic lift was assumed to be 6.4 cm (2.5 in.). The undrained shear strength of the untreated subgrade was varied from 44 kPa to 129 kPa (6.4 to 18.7 psi). Corresponding values of CBR, as estimated from the relationship shown in Figure 9, ranged from three to nine. At
a factor of safety of 1.0, the minimum CBR strengths required to construct the first lift of an asphaltic pavement are about five to six for asphaltic temperatures ranging from 60°C to 37.8°C (140°F to 100°F). However, to avoid the possibility of failure, the CBR strength should be approximately equal to eight, or greater. To construct the first stone-aggregate lift, measuring 15.2 cm (6 in.) in thickness, at incipient failure, the minimum subgrade CBR strength must be about six (F equals 1.0). To prevent failure, the CBR strength should be near or greater than nine.

METHODS FOR IMPROVING SUBGRADES

Placement water content during construction and the infiltration of water into the subgrade by surface and subsurface seepage long after construction of the pavement significantly influences bearing strength of the subgrade. Consequently, subgrade modification should be an important design consideration to provide a sound working platform during construction and to maintain stability of the pavement long after construction. Methods of modifying the subgrade, which have frequently been used as remedial and routine measures in the past and should be considered during the design stage, may be divided broadly into two categories: mechanical and chemical. Some advantages and disadvantages of these techniques are discussed below.

Mechanical Stabilization

Moisture-density control

Compaction increases the shear strength of soils. As the compactive effort increases, the cohesive component increases and the angle of internal friction increases slightly. Usually, subgrade soils are compacted to 95 percent of standard maximum dry density and close to optimum moisture content (AASHTO T 99 or ASTM D 698). Compaction also decreases the compressibility. Compaction is necessary and is generally required by most agencies. By controlling the density and moisture content within prescribed limits, desirable strength and compressibility of the soil subgrade are initially insured. However, as shown in Figures 1 and 2, compacted clayey soils and shales swell when exposed to water and lose strength. Therefore, compaction (although desirable) will not insure that the initial compactive state will be maintained throughout the life of the subgrade.

Proof rolling or rerolling

Proof rolling of subgrades is sometimes used to discover "soft spots," or areas of low densities. This technique is useful for this purpose. Where a clayey subgrade has
been exposed to rainfall or melting snow, the subgrade may become soft and lose bearing strength. When rerolling or proof rolling is attempted and the subgrade is saturated, large excess pore pressures, \( \Delta u \), will build up. As shown by Equation 5, decreases in shear strength can occur. Consequently, subgrade bearing capacity failures are likely to occur under stresses of the compactor. Therefore, rerolling to improve the bearing strength of clayey subgrades can only be effective when the subgrade is unsaturated. Tests to determine the degree of saturation should be performed before rerolling. Results of those tests will determine when rerolling should be attempted. However, rerolling of the clayey subgrade will not insure that the strength will not decrease when exposed to moisture.

**Undercut and backfill techniques**

A common remedial approach of soft subgrades consists of placing granular material over the soft subgrade, or alternately, undercutting the soft material to some predetermined depth before backfilling with granular material. Certain disadvantages are associated with this technique, which the designer must consider. These are as follows:

- To provide a sound working platform, the granular material must be sufficiently thick to distribute stresses (due to wheel loadings) throughout the subgrade and to prevent severely rutting of the granular base.

- Tensile cracks may occur at the bottom of the granular material during initial construction.

- Clayey particles may intrude into the base of the granular base materials and lower the strength of the granular base.

- When a geosynthetic fabric (of low permeability) is used to separate the granular base and soil subgrade and placed on a saturated soft subgrade, construction traffic may create "mud waves," or bearing capacity failures. The subgrade is especially vulnerable during construction of the initial lifts of the granular base.

As shown in Figure 16, the thickness of the granular base required to withstand construction traffic without failure may be substantial for clayey subgrades that have very low CBR values. For example, at a CBR of two, the thickness of stone must be between about 90 and 135 cm (35 and 53 in.)--these values correspond to factors of safety of 1.0 to 1.5, respectively--to withstand failure. Factors of safety of 1.0 and 1.5 and tire contact stresses of dual-wheels were assumed in those analyses. Magnitudes of tire contact stresses -- therefore, the gross loads of construction vehicles -- may need to be smaller than permissible legal limits during construction of the first lifts of the
granular base. This is important aspect of those analyses. Otherwise severe rutting, subgrade shoving, and cracking of the granular base may occur -- conditions that may influence the future pavement performance. Regarding stone bases, a suggested guideline is as follows:

* Design of the pavement structure should include stability analyses of the early construction stages of the partially completed pavement using each stone lift thickness and anticipated construction traffic loadings.

These analyses may be accomplished in several different ways. Approaches that could be considered are as follows:

- The multi-layered, limit equilibrium method developed by Hopkins (1991) can analyze early construction stages. To use this approach, the shear strength parameters of the improved subgrade, or stone base material, and the untreated subgrade below the stone base must be defined. These parameters may be established from triaxial compression tests. Initially, triaxial testing of different stone base materials would be required to define the shear strength parameters. Shear strengths of the underlying subgrade may be defined from unconfined triaxial compression tests or unconsolidated-undrained triaxial tests. However, these strengths could be estimated from the relationship shown in Figure 9 (Hopkins 1991) -- since CBR values are usually available on soil and profile plans for new highway projects. The computer program has been made "user-friendly"; the analyses may be performed in a matter of minutes. Alternately, families of design curves could be developed from the computer model for various design scenarios. Additionally, the computer program could be used to analyze early construction stages involving placement of the asphaltic layers. Here, shear strengths of the asphaltic layers are obtained from triaxial tests. The tests are performed over a range of temperatures. Shear strength parameters, $\phi$ and $c$, of the asphaltic concrete are used in the bearing capacity model to determine stability of partially completed pavements subjected to construction traffic.

- Multi-layered, elastic methods, such as the Chevron computer model, could be used to analyze the initial stages of constructing the unbound stone or sand, sub-base lifts. This approach requires a knowledge of the dynamic moduli of elasticity of the stone base and underlying subgrade. From the relationships shown in Figure 11, the modulus of the underlying subgrade may be approximated.
These data, published by Heukelom and Foster 1960, show the variation of the dynamic subgrade modulus of elasticity, $E_s$, and the California Bearing Ratio (CBR). As shown in Figure 11, re-analyses of the data published by Heukelom and Foster yields:

$$E_s \approx 17.914 CBR^{(0.874)} (kPa),$$  \hfill (13)

where $E_s$ is in kilopascals (kPa). The relationship published by Heukelom and Foster (1960) is represented as

$$E_s \approx 10.795 CBR (kPa).$$  \hfill (14)

Yet later, Heukelom and Klomp (1962) represent the relationship as

$$E_s \approx 10.350 CBR (kPa).$$  \hfill (15)

These equations yield marginally different estimates. For example, at a CBR equal to three, Equations 13, 14, and 15 yield estimated values of subgrade modulus of 46,795, 32,389, and 31,050 kPa (6,791, 4,700, and 4506 psi), respectively.

As the modulus of the subgrade increases, the modulus of the base, or stone layer, increases. Since the modulus of the underlying subgrade influences the modulus of the unbound stone or sand, the improved dynamic modulus, $E_i$, of the stone may be estimated by the relationship shown in Figure 18 (data from Heukelom and Foster, 1962), or

$$E_i = 20 E_s^{(0.816)} (kPa)$$  \hfill (16)

Combining Equation 12 (or Equation 13 or 14 could also be used) and Equation 15,
then

\[ E_i = (14.023)[17,914CBR^{0.874}]^{0.816}, \]

(17)

and

\[ E_i = 41,443CBR^{0.7132}(kPa) \]

(18)

For example, at a subgrade CBR value of three, the estimated modulus of the subgrade is 46,795 kPa (6,791 psi)--Equation 13--and the estimated modulus of the stone is 90,727 kPa (13,166 psi)--Equation 18. For a subgrade CBR equal to seven, the estimated modulus of the subgrade is 98,132 kPa (14,241 psi) and the modulus of the stone is 166,026 kPa (24,093 psi).

Variation of the ratio, \( K_2 \) (ratio of the modulus of improved material to the modulus of the subgrade) with the modulus of the subgrade is shown in Figure 19. The scatter is evident in those data. However, the trend of the ratio, \( K_2 \), may be approximated by the following equation:

\[ K_2 \approx 1.986 + \frac{37,510.5}{E_s}, \]

(19)

and, as a function of CBR (Figure 20),

\[ K_2 \approx 1.986 + \frac{2.094}{CBR^{0.874}} \]

(20)

Figure 19. Variation of the ratio, \( K_2 \), with the modulus of the subgrade, \( E_s \).

Figure 20. Trend of ratio, \( K_2 \), in terms of CBR.
where $K_2$ is the ratio, $E_i/E_s$. The modular ratio appears to become nearly constant when the CBR is greater than about six to eight and it eventually approaches a value of approximately two. For CBR values ranging from one to eight, the modular ratio ranges from approximately four to 2.3, respectively.

Equation 20 relates modular ratio $K_2$, and CBR. However, and unfortunately, Heukelom and Klomp (1962) or Heukelom and Foster (1960) do not show thicknesses of the granular bases of their test sites. Therefore, the relationship of modular ratio and thickness of the granular base cannot be developed from their published data. Dormon and Metcalf (1965) published a relationship between modular ratio and thickness of granular base as shown in Figure 21. As noted by these researchers, the relation was arbitrarily chosen to give designs that correlate well with empirically developed CBR curves. By analyzing their data (Figure 4 in their report), an equation may be developed. Their curve may be expressed in equation form as:

\[ K_2 = \frac{E_i}{E_s} = 0.5553 t^{0.4617} \]  \hspace{1cm} (21)

or,

\[ 0.5553 t^{0.4617} = K_2. \]  \hspace{1cm} (22)

Solving for $t$,

\[ t = \left( \frac{K_2}{0.5553} \right)^{\frac{1}{0.4617}} \]  \hspace{1cm} (23)

and
Inserting the expression for $K_2$ (from Equation 20) into Equation 24, then

$$t = (1.8K_2)^{2.166}$$

(24)

The granular thicknesses obtained from Equation 25 range from about 75 cm to 21 cm (29.5 to 8.3 in.) for subgrade CBR values ranging from one to 10 as shown in Figure 22. The thickness-CBR curve obtained from Equation 24 approaches a thickness of 16.5 cm (6.5 in.) at a CBR of 100. When CBR values are less than about four, the thicknesses obtained from Equation 25 are smaller than thicknesses obtained from the HOPKIB model. That is, the thickness-CBR curve obtained from Equation 25 lies below the thickness-CBR (F=1.0) curve obtained from the HOPKIB model. When the CBR value is greater than four, the thickness-CBR curve from Equation 25 is positioned above the thickness-CBR curve (F=1.0) obtained from the HOPKIB model.

A curve similar to the modular ratio - granular thickness curve given by Equation 24 may be determined from relationships obtained from the HOPKIB model by using the equation shown in Figure 22 (factor of safety equal 1.5) and Equation 20, or

$$t_s = 158.2 - 71.8 \ln(CBR)(cm)$$

(26)

Solving for CBR and rearranging terms of Equation 26, then

$$CBR = e^{\frac{[158.2-t_s]}{71.8}}$$

(27)

Substituting Equation 26 into Equation 20, then
The relationship between modular ratio and thickness obtained from Equation 21 is shown in graphical form in Figure 23 and compared to the relationship, (Equation 28) obtained from the HOPKIB computer model.

**Protective filter requirements**

Early pavement research in Kentucky (Baker and Drake 1948) showed that, where clean, or open, stone bases had been used, fine particles of the clayey, or silty, subgrades infiltrated into bottom portions of the bases and often caused failure. The pavements in the early studies were typically thin. As fines intruded into the base courses, the shear strengths of the stone aggregate bases decreased. Also, as the fines increased, the permeability of the bases was lowered and caused the build up of large, excess pore pressures. With a build up of excess pore pressures, the shear strengths of the bases were lowered. Eventually, the pavements rutted and failed.

To prevent the intrusion of fine particles into the base aggregates, protective filter requirements (NAVFAC 1982) should be satisfied. This notion is very seldom mentioned in the literature on pavement design (Hopkins et al, 1991). More research and attention should be devoted to this aspect of pavement design. Concrete sand (ASTM C 33) suffices as a good filter material when the subgrade is constructed of fine-grained silty or clayey sands. When the subgrade is constructed of nonplastic, silts or clay, asphalt sand (ASTM D 1073) may be used as a filter material. In some states, a sand layer is placed directly on the clayey or silty subgrade to satisfy protective filter requirements before the clean aggregate is placed. In Kentucky, the problem apparently was solved using dense-graded aggregate (DGA). Five to 12 percent fines passing the No. 200 sieve were allowed in the stone (current practice limits the fines to two to 10 percent).
However, intrusion of fines into the base course creates a material that is more susceptible to the build up of excess pore pressures when the granular material becomes saturated. Moreover, with lower permeability, water retained in the base may freeze and expand. Additionally, the retained water and excess pore pressures keep the top portion of the compacted clayey and silty subgrades saturated. This situation creates a condition where the subgrade may "pump" and involves excess pore-pressure dissipation. Additionally, the top portion (approximately one to 4 inches, or 2.5 to 10.2 cm) of clayey subgrades, which have been in service for some time, have been observed (Hopkins, et al., 1985) to be very soft and weak. The situation is depicted in Figure 24. This condition occurs from water flowing along the bottom of the DGA base; part of this water is absorbed by the top portion of the clayey subgrade. When this occurs, the clayey subgrade absorbs the water, swells, and losses bearing strength. To protect the top portion of a clayey subgrade from flowing water in the base materials, consideration might be given (on an experimental basis) to covering the clayey subgrade immediately after compaction with an asphaltic emulsion.

In designing granular base courses for highways, protective filter requirements should be observed and properties of soils at a given site should be considered. Protective filter requirements (NAVFAC 1982) are described as follows:

1. To avoid segregation, the filter, or granular material of the base course should contain no sizes larger than 7.6 cm (3 in.).

2. To avoid head loss in the filter, or granular material, the permeability must be large enough to suffice for the particular drainage system, or

\[
\frac{D_{15\text{filter}}}{D_{15\text{subgrade}}} > 4,
\]

(29)
where

\[ D_{15\text{filter}} = \text{the diameter of the filter particles at 15 percent finer by weight, and} \]

\[ D_{15\text{subgrade}} = \text{the diameter of soil subgrade particles at 15 percent finer by weight.} \]

Values of \( D_{15\text{filter}} \) and \( D_{15\text{subgrade}} \) are obtained from sieve and hydrometer test data.

3. To avoid movement of particles from the soil subgrade into the filter, or granular base, the following criterion should be met

\[ \frac{D_{15\text{filter}}}{D_{85\text{subgrade}}} < 5 \]  

(30)

where

\[ D_{85\text{subgrade}} = \text{the diameter of the soil subgrade at 85 percent finer by weight.} \]

4. To avoid internal movement of fines within the granular base course, the granular material should have no more than 5 percent passing the No. 200 sieve, or

\[ P_{200} < 5\%. \]  

(31)

Therefore, in trying to meet these requirements for clayey and silty subgrades, the designer faces a dilemma. Sufficient fines (for example, older State specifications allowed five to 12 percent) may be added to the granular mixture to reduce, or prevent intrusion of fine clayey or silty particles into the granular base course. Including fines in the DGA base also had another beneficial effect. At low moisture contents during placement, the fine material acts as a binder. Thus, a cohesive component of strength develops. Clean stone has no cohesive strength component. However, the granular base containing fines may retain water (from surface and subsurface seepage) for long periods due to low permeability. Retention of water also may lead to the build up of large excess pore pressures because of large induced stresses from traffic loadings. Consequently, with an increase in excess pore pressures, the stability of the pavement structure is lowered because the strength is lowered. Additionally, if the granular base
course contains more than 5 percent fines, excess pore pressures may cause internal movement of the fines. When this occurs, part of the support provided by the base course under traffic loadings is lost. If the granular base course is "opened-up," or the fines are reduced to a level below 5 percent, then the fines of the clayey or silty subgrade may intrude into the granular base course.

Although the designer faces a dilemma, options are available that need to be explored. These are as follows:

- Protective filter calculations should be made for each specific project using geotechnical data that is normally available at a given site. The necessary data to make these calculations can be easily obtained from inexpensive, geotechnical laboratory tests.

- Adding a small amount of asphalt (one or 2 percent) -- lean mix -- to the granular drainage base course (for example, such a procedure was used on Section 12 of the Alexandria-Ashland Highway in northern Kentucky). However, protective filter requirements between the asphalt treated layer and the soils of the subgrade should be checked.

- When protective filter requirements are met, a thin (about 10 cm-- 4 in.--thick) layer of dense graded aggregate could be placed directly on the subgrade soils. This action would then be followed by adding a cleaner, granular blanket of stone (such as Number 57 stone) on top of the DGA. Protective filter requirements should be met between the clean stone and DGA, and the DGA and soil subgrade.

* Another approach might consist of placing an "open," or clean granular base (percent of fines less than 5) on geosynthetic filter fabric. However, the filter fabric-soil system should meet protective filter requirements (the possibility exists that the fabric may become clogged over time -- more information needs to be collected concerning this aspect).

- A protective layer of sand may be placed between the soil subgrade and clean stone. Concrete sand may suffice for soil subgrades constructed of plastic clays. For soil subgrades constructed of nonplastic silt, asphalt sand may be used. Protective filter requirements should be met between the sand and clean stone base material.

Although, presently, protective filter requirements may not always be met, the emphasis here is that efforts and attention should be directed toward performing the necessary calculations and attempting to meet these requirements. More research and data are needed to develop this idea fully.
When granular bases are placed on clayey subgrades that may soften and deflect under wheel stresses, tensile stresses develop at the bottom of the granular layer and may cause cracking. To lessen, or prevent, the development of base cracking, geosynthetics may be placed at the bottom of the granular base, or on top of the finished subgrade. Although charts have been published by manufacturers of geosynthetics for analyzing and designing reinforced granular bases, a limited, preliminary analysis was performed, as described below, using bearing capacity models developed by Hopkins (1986; 1991) and Slepak and Hopkins (1993; 1995). These models are based on limit equilibrium concepts. The purpose of those analyses was to obtain an indication of the general effectiveness of reinforcing granular bases with geosynthetics. Moreover, examples were analyzed to obtain general indications of how geosynthetic reinforcement might prolong, or increase, the life of a flexible pavement.

The stabilities of several example scenarios were analyzed. In subsequent analyses, geogrid material, identified as SR-1 and manufactured by the Tensar Corporation (1986 a, b), was used. Strength of this material is 907 kg (2000 lb.) per linear 30.5 cm (foot). In the first series of analyses, the thicknesses, corresponding to selected CBR strengths (or unconfined compressive strength), of granular base without reinforcement required to maintain factors of safety of 1.0 and 1.5 were determined, as shown in Figure 25. Those series of analyses were repeated using SR-1 geogrids. Factors of safety obtained when the geogrids were included in the sections are compared to the safety factors of 1.0 and 1.5 in Figures 25. Inclusion of the geogrid layer increases the factor of safety some six to 15 percent.

The effectiveness of geogrids may also be viewed in another manner. Relationships between the thicknesses of granular layers that were not
reinforced and values of subgrade CBR required to maintain factors of safety of 1.0 and 1.5 were determined as shown in Figures 26 and 27, respectively. These analyses were repeated using the same thicknesses and SR-1 geogrids. For CBR strengths ranging from one to 6.5, the thicknesses of granular layers with SR-1 geogrids required to maintain a factor of safety of 1.0 may be about 2.5 to 21 cm (1 to 8.3 in.) smaller than thicknesses of granular layers without SR-1 reinforcement, as shown in Figure 25. Between a CBR of about 1.5 and five, the thickness may be some eight to 21 cm (3.2 to 8.3 in.) smaller. To maintain a factor of safety of 1.5, and for CBR strengths ranging from one to three, thicknesses of granular layers with SR-1 reinforcement may be some 15 to 52 cm (5.9 to 20.5 in.) smaller than granular layers without reinforcement, as shown in Figure 27. When CBR strengths are greater than three, but less than nine, the thicknesses of the reinforced layer may be some seven to 23 cm (2.8 to 9.1 in.) smaller than the thicknesses of unreinforced granular base layers.

Based on the analyses described about, geogrid reinforcement provides some improvement to the overall bearing capacity of the subgrade. However, in those analyses, the assumption was made that the reinforcement was tightly placed so that any small deflections that occur in the granular layer mobilize stresses in the reinforcement layer. If the reinforcement layer is placed too loosely, then large deflections may occur before stresses are mobilized in the reinforcement layer. This situation occurs because strains of the geosynthetic and gravel and clayey subgrades are incompatible. Consequently, the reinforcement would be of no value there. Moreover, to construct the first lift of a granular layer reinforced with geosynthetic, the subgrade must
possess a certain minimum strength to avoid bearing capacity failures. For example, the stability of a 10.2 cm (4 in.) lift of stone reinforced with SR-1 geogrid was determined. Results of those analyses are shown in Figure 28.

To maintain a factor of safety of 1.0 (incipient failure) and 1.5 (assumed stable condition), CBR strengths of about 5.5 and 8.2 are required. If the subgrade is unsaturated, then the CBR strength would be greater than 5.5 and, normally, no difficulties would be encountered in constructing the first lift of a reinforced granular layer.

For estimating how geosynthetics can affect the life of a pavement, the stabilities of two typical pavement design sections were analyzed using the bearing capacity models developed by Hopkins (1991) and Slepak and Hopkins (1993; 1995). The section shown in Figure 29 is a typical flexible pavement design used in metropolitan areas and consists of 10.2 cm (4 in.) of asphalt pavement resting on 20.3 cm (8 in.) of granular base. The second section, Figure 29, is a typical section used on an interstate, parkway, or primary route in Kentucky and consists of 20.3 cm (8 in.) of flexible pavement resting on 40.6 cm (16 in.) of dense graded aggregate. The factors of safety of both sections were computed without and with a layer of geogrids. A strength (Tensar® SR-1) of 907 kg (2000 lbs.) per linear 30.5 cm (foot) was assumed in the analyses. A surface temperature of 60°C (140°F) was assumed for the flexible pavement. The flexible pavement was divided into increments of 2.54 cm (1 in.). The temperatures at the midpoints of each increment were estimated from relationships given elsewhere (Hopkins 1991). Strength parameters, \( \phi \) and \( c \), of each incremental layer of a flexible pavement were estimated from the relationships given by Hopkins (1991). From each series of calculations, the subgrade strength was varied from a CBR of one to seven. Corresponding undrained shear strengths were approximated from the equation in Figure 9. The calculations were performed using tire contact stresses of 469 kPa (68 psi) and 552 kPa (80 psi).

To estimate equivalent single axle loads corresponding to a factor of safety, the relationship developed from previous work by Hopkins (1991) was used. This relationship, Figure 30, is

\[
F = 0.095 \ln(ESAL) - 0.00463.
\]
The relationship in Figure 30 is based on stability analyses of 237 flexible pavement sections of the AASHO Road Test (1960) and corresponds to a serviceability index of 2.5. By rearranging terms of Equation 32, the value of ESAL may be computed when the factor of safety is known, or

\[ ESAL = e^{\frac{F - 0.00463}{0.095}} \]  \hspace{1cm} (33)

Factors of safety obtained from bearing capacity analyses of the thin, flexible pavement section (Figure 29) are compared in Figure 31. The section was analyzed with and without the geogrid layer (SR-1). The analyses were performed using tire contact stresses of 469 (68 psi) and 552 kPa (80 psi). CBR strengths were varied as shown in Figure 31. At a given tire contact stress, the factors of safety of the section containing geogrids were about 14 to 17 percent larger than factors of safety of the same section without geogrids. Moreover, the factor of safety of the section (with and without geogrids) decreases as the tire contact stress increases. Similarly, factors of safety of the primary route section (Figure 29) that contained a geogrid layer are compared in Figure 32 to factors of safety of...
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2.75

2.5

2.25

2.0

1.75

1.5

1.25

1.0

0.75

1

20.3 cm ASPHALT PAVEMENT 40.6 cm DGA

WITH GEOGRID

W/O GEOGRID

2.75

2.5

2.25

2.0

1.75

1.5

1.25

1.0

0.75

1

10.2 cm ASPHALT PAVEMENT 20.3 cm DGA

WITH GEOGRID

W/O GEOGRID

Figure 32. Factors of safety of the primary route section containing a geogrid layer compared to factors of safety of the same section without geogrids.

Figure 33. ESAL values of the city street containing geogrids compared to ESAL values of the same pavement section without geogrids.

Since the factor of safety of the pavement section containing geogrids are larger than the same sections without geogrids, an estimate of the extended pavement life obtained when geogrids are used is useful. Such estimates may be obtained by inserting factors of safety of the sections containing geogrids and factors of safety of the same sections without geogrids into Equation 33. ESAL values, obtained from Equation 33, of the city street containing a geogrid layer are compared in Figure 33 to values of ESAL of the same pavement section that did not contain a geogrid layer. By including the geogrid layer, the analyses show that the values of ESAL increase significantly. For example, at a tire stress of 469 kPa (68 psi) and a subgrade CBR of four, the estimated value of ESAL for the city section without geogrids is approximately 245,000; the value of ESAL of the same section with geogrids is 3.3 million. At a tire stress of 552 kPa (80 psi) and CBR equal to four the estimated ESAL value of the section without geogrids is 49,000. When a geogrid layer is included, the estimated value of ESAL increases to 390,000. Comparisons of values of ESAL obtained from Equation 32 for the primary route section are shown in Figures 34 and 35. At a CBR of three and a tire contact stress of 552 kPa, the value of ESAL is about 2.6 million when the geogrid

the same section that did not have geogrids. By adding the geogrid layer, the factor of safety increases some seven to 12 percent. Whether or not the geogrid layer is used, the factor of safety of the section decreases as the tire contact stress increases from 552 kPa (80 psi) to 828 kPa (120 psi). For a wide range of tire contact stresses, CBR subgrade strengths, and pavement sections, the factor of safety of the sections containing geogrid layers is some seven to 17 percent larger than the same sections without geogrids.
layer is excluded. When the geogrid layer is included, the ESAL value increases to 15.4 million. At a stress of 828 kPa (120 psi) and a CBR of four, the ESAL value increases from 264,000 to 783,000. Therefore, the theoretical analyses show that the inclusion of a geogrid layer into a pavement section can significantly increase the values of ESAL and, therefore, extend pavement life. However, stability analyses using the above ideas should be performed on a site specific basis. Moreover, field research is needed to verify these theoretical concepts.

Soil-Gravel Mixtures

Mixing a soil subgrade with a durable aggregate, such as limestone or sandstone, has been proposed and used as one means of improving the bearing strength of the subgrade. This mechanical stabilization technique has been proposed for situations where traffic must be routed onto the subgrade immediately after compaction. Use of chemical admixture stabilization may not in this situation be feasible because the time required for curing of the chemically-treated subgrade is not available or the type of subgrade soil may not be conducive to chemical treatment. In using this technique, the short-term bearing strength of the soil-aggregate subgrade and a partially completed pavement must be sufficient to support public and construction traffic loadings without rutting or bearing capacity failure. Furthermore, the bearing strength of the soil-aggregate subgrade must be sufficient so that proper compaction of the paving materials may be obtained. Moreover, the long-term bearing strength of the soil-aggregate subgrade must be sufficient to support traffic stresses long after completion of the pavement.
Support provided by the soil-aggregate subgrade mixture depends largely on the degree of saturation and the percentage of clay-size particles. The degree of saturation of the mixture largely controls the short-term bearing strength during construction. Although the bearing strength of the mixture immediately after compaction may be very high, a large decrease in bearing strength may occur if the mixture is exposed to extended periods of rainfall or melting snow during construction. Here, surface waters may penetrate the mixture and increase the degree of saturation. Consequently, the mixture may swell, increase in volume, and decrease in strength. The total strength, and the loss of strength, will depend on the percentage of clay-size particles in the mixture matrix and their affinity to absorb water. Although the loss of strength may not occur during construction, especially if construction occurs during a dry season, a long-term loss of bearing strength may occur after construction of the pavement. Here, the subgrade mixture may become saturated, or soaked, by penetration of surface and subsurface, ground water seepage. Premature failure, or faulting, of the pavement may occur sometime after construction as the mixture becomes saturated and loses bearing strength. Consequently, both the short-term and long-term, saturated (or soaked) bearing strengths are major design considerations because it cannot be predicted with any degree of certainty when rainfall or groundwater seepage may occur at a given location. Although side-drainage panels and granular bases may be used to convey water from the pavement layers, prolonged periods of rainfall or melting snow create water that will flow at the top of the subgrade. Therefore, during these flow periods water may enter the subgrade.

In designing the soil-aggregate mixture, the percentage of aggregate to be mixed with the soil subgrade must be determined. Additionally, the saturated bearing strength of the selected mixture must provide adequate bearing capacity to avoid deep ruts and tire sinkage of traffic. For anticipated ground contact stress of 552 kPa (80 psi), the CBR value must be about eight to 10 to avoid rutting (Hopkins 1991). The shearing strength of the soil-aggregate mixture is controlled largely by the percentage of clay-size particles (percent finer than the 0.005 mm- or 0.002 mm-size particles) present and the degree of saturation. The influence of these factors is illustrated in Figures 36 and 37. In Figure 36, values of Kentucky CBR are shown as a function of the percent finer, \( P_{0.05} \), than the 0.005 mm-size particles. The data used to obtain this approximate correlation...
Figure 37. Illustration of the loss of strength of clayey type soils when exposed to soaking for extended periods and when the clayey fraction increases.

Figure 38. Relationship between percent finer than 0.002mm-size particles and 0.005mm-size particles.

Vast differences that may exist between unsoaked and soaked bearing strengths were illustrated previously in Figures 1, 2, and 3. Figures 36 and 37 also illustrate the loss of strengths of clayey type soils when exposed to soaking for extended periods. For example, as shown in Figure 37, the soaked KYCBR at 17 percent finer than the 0.002 mm-size particles is about 10. However, in the unsoaked state the value is about 26. Therefore, when most soils are compacted to a dry density close to the maximum dry density obtained from standard compaction (ASTM D 698), the bearing strengths would provide adequate stability immediately after compaction. If the mixture remains dry during construction, then practically any combination of soil and aggregate are mixed, the percentage of clay-size particles \( P_{005} \) should not exceed a limiting value of approximately 27 percent. Similarly, as shown in Figure 37, the percentage of clay-size particles finer than the 0.002 mm-size particles should not exceed approximately 17 percent. If the percentage of clay particles, finer than 0.005 mm is known, then the percentage of clay-size particles finer than 0.002 mm can be estimated from the correlation given in Figure 38. The value of KYCBR value of 10 or greater is another limiting condition. These criteria should be viewed as very approximate limits to use in designing a soil-aggregate subgrade mixture.
aggregate would sustain traffic loadings. However, if the soil-aggregate base is exposed to extended periods of rainfall and seepage during construction or sometime after construction, then inadequate stability may develop. Therefore, the mixture must be designed for the soaked condition.

Although the very approximate limits established above may be used to design the mixture, the use of the KYCBR value may overestimate the long-term bearing strength of the subgrade. Additionally, if the subgrade mixture is exposed to extended periods of rainfall or melting snow during construction, then the bearing strength may be overestimated.

The compaction procedure in the KYCBR test generally produces a specimen that has an average dry density that is approximately 112 percent higher than the maximum dry density obtained from standard compaction (Hopkins, 1970). Even after soaking, the dry density of the KYCBR specimen generally averages some 107 percent higher than maximum dry density from standard compaction. Most specifications normally require that a subgrade should be compacted to 95 percent of maximum dry density normally produced in the field. Because dry densities obtained from the KYCBR procedure are larger generally than dry densities obtained from the ASTM D 698 or AASHTO T 99 compaction procedure, Kentucky bearing ratios are usually larger than those obtained from ASTM or AASHTO procedures. Studies by Hopkins in 1970 and 1991 and by Beckham and Allen in 1989 show that this is the case.

Consequently, a major objective of a laboratory testing program designed to perform strength testing should be to duplicate as close as practicable the dry densities and moisture content anticipated in the field. This involves both the short-term and long-term compactive states of the specimens produced from laboratory testing and the compactive state of the subgrade after a long-period of time. Obviously, different bearing ratios may be obtained for soil specimens that may exist in short-term and long-term, compactive states. The strength of the soil-aggregate or a soil immediately after compaction -- an unsoaked state -- is high. If the compactive states of soil subgrades could be maintained throughout the life of pavement structures, then most compacted subgrades would have ample strengths to provide good stabilities. Unfortunately, for clayey subgrades this condition cannot be maintained. The bearing strength decreases rapidly as the clay content and moisture content increases.

To examine the influences of the percentage of clay-size fractions and moisture content on the bearing strengths of compacted clay-aggregate mixtures, a laboratory testing program was devised and performed. Bag samples of a clay and a limestone aggregate were obtained from a highway construction site in Harrison County, Kentucky. Particle-size analyses of three bag samples and the aggregate are shown in Figure 39. Percent of particles smaller than the 0.002 mm-size particles of the clay
is approximately 55 percent. About 7 percent of the particles of the aggregate passes the U.S. Standard No. 200 sieve. Approximately 93 percent of the aggregate consists of gravel and sand particles. Testing was performed on the clay sample and the aggregate in an unmixed state. Additionally, testing was performed on mixtures of the clay and aggregate containing 25, 50, and 75 percent of aggregate. Variations of the percent finer than the 0.005 mm and 0.002 mm-size particles of the mixtures with the percent of aggregate present in a mixture are shown in Figure 40. Mixtures contained zero, 25, 50, 75, and 100 percent of aggregate. Index properties of the clay specimens are summarized in Table 1. Liquid and plastic limits of the clays are 58 and 28 percent, respectively. Classification of the clay is MH-CH and A-7-6 (27). The limestone aggregate is non plastic (NP) and is classified as GW-SM and A-1-A(O).

Variation of the maximum dry density obtained from standard compaction (AASHTO T-99) of the soil-aggregates with the percentage of aggregate present in the mixture is shown in Figure 41. As the percentage of aggregate increases, the maximum dry density increases. The maximum dry density increases from a low value of 1537 kg/m³ (96 lbs/ft³)–100 percent of clay–to 2172 kg/m³ (135.6 lbs/ft³)–100 percent of aggregate. Optimum moisture content as a function of the percentage of gravel is shown in Figure 42. The optimum moisture content at a value of zero percent of gravel is 24 percent and decreases to value of 8 percent when the percentage of gravel is 100.

The relationship between soaked values of KYCBR and percentages of gravel of the mixtures is shown in Figure 43 and compared to soaked CBR strengths obtained from the ASTM testing procedure. The ASTM CBR strengths were obtained for specimens remolded to 95 percent of maximum dry density and optimum moisture content (ASTM D 698 or AASHTO T-99).
Table 1. Index properties of limestone aggregate and clay.

<table>
<thead>
<tr>
<th>Test</th>
<th>Limestone Aggregate</th>
<th>Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid Limit</td>
<td>NP</td>
<td>58%</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>NP</td>
<td>28%</td>
</tr>
<tr>
<td>Classification</td>
<td>GW-GM and A-1-A (0)</td>
<td>MH-CH and A-7-6 (27)</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>2.65</td>
<td>2.84</td>
</tr>
</tbody>
</table>

Figure 41. Variation of the maximum dry density obtained from standard compaction of the soil-aggregates with the percentage of aggregate present in the mixture.

The KYCBR was some three to four times greater than the ASTM CBR. To obtain an ASTM CBR of 6.5, which corresponds to a factor of safety against failure of 1.0 obtained from the HOPKIN bearing capacity model, the required percentage of gravel of the soil-aggregate mixture must be about 93 percent. The percentage of gravel corresponding to a KYCBR of 6.5 is about 72 percent. However, since compaction specifications usually specify that subgrades are compacted to 95 percent of maximum dry density, the value of 93 percent is more realistic.

The CBR strength obtained from the ASTM
procedure is shown in Figure 44 as a function of the percent finer than the 0.002 mm-size particles. At a CBR of 6.5, the corresponding value of \( P_{0.002} \) is about 7 percent. Therefore, in designing a soil-aggregate subgrade mixture, the percent finer than the 0.002 mm-size particles must be equal to 7 percent or less to provide stability.

To insure good stability \((F=1.5)\), the mix must contain 5 percent or less of the particles finer than 0.002 mm. Concerning the KYCBR, Figure 45, the value of the percent finer than the 0.002 mm-size particles must be 17 percent to insure a KYCBR of 6.5 \((F=1.0)\). To obtain a KYCBR of 10 or greater, the percentage of particles finer than 0.002 mm-size particles must be equal or less than about 12 percent. Because the dry density used in determining the ASTM CBR is more representative of the compactive state of a field subgrade than the dry density obtained from the KYCBR procedure, the soaked ASTM CBR is more realistic. Therefore, the particle sizes in the clay-aggregate matrix finer than the 0.002 mm-size particles should be 7 percent or less, to insure good stability during and after construction.

Vertical swell of a compactive specimen of the clay is shown as a function of time in Figure 46 and compared to vertical swells of compacted specimens containing 50 percent clay and 50 percent aggregates. Also, vertical swell of a compacted specimen containing 100 percent aggregate is shown. These specimens were compacted to 95 percent of maximum dry density and optimum moisture content obtained from ASTM D 698. Vertical swell of the clayey specimens consists of primary swell followed by secondary swell. The clay specimen swelled some 5.7 percent. By adding 50 percent of aggregate to the soil-aggregate matrix, the vertical swell was
about 3.5 percent. Vertical swell of the aggregate was essentially zero. As the vertical swell increases, as shown in Figure 47, the CBR strength decreases. To obtain a CBR of 6.5 or greater, the vertical swell must not exceed about 1.5 percent.

Because the bearing capacity of a clay-aggregate mixture is dependent on the shear strength of the mixture, consolidated-undrained triaxial compression tests with pore pressure measurements were performed on compacted specimens of different mixtures of the clay and aggregate to determine the effective stress parameters, $\phi'$ (angle of internal friction) and $c'$ (cohesion). Specimens were compacted to 95 percent of maximum dry density and optimum moisture content obtained from ASTM D 698. Variations of $\phi'$ and $c'$ with the percentage of aggregate present in the compacted clay-aggregates matrices are shown in Figures 48 and 49. Values of $\phi'$ range from about 13 degrees for the clay to 42 degrees for the limestone aggregate. The cohesive strength component, $c'$, ranges from 32.5 kPa for the clay to zero for the aggregate. To obtain a value of $\phi'$ equal to or greater than 30 degrees, the percentage of stone in the mixture must be 75 or greater.

Stability analyses of subgrades constructed of different clay-aggregates proportions were performed using the shear strength parameters shown in Figures 48 and 49. In these analyses, a tire contact stress of 552 kPa (80 psi) was
assumed. To obtain a factor of safety against failure of 1.0 or greater, the percentage of aggregate present in the clay-aggregate matrix must be 80 or greater, as shown in Figure 50. Although the immediate mixing of aggregate with a soil subgrade may improve the short-term strength, the soaked strength is not improved unless the percentage of particles finer than 0.002 mm-size particles in the soil-aggregate is less than about 10 percent. Once the clay content of the mixture exceeds 10 percent (Figure 51), the long-term bearing strength of the clayey-aggregate is inadequate and this technique is ineffective.

Use of In Situ Native Materials

The use of in situ materials, or materials existing at a site, can be used effectively in constructing the top portion of highway subgrades. For example, this technique was used in 1987 to construct the top 0.61 m (2 feet) of the subgrade of Section 20 of the Alexandria-Ashland Highway in northern Kentucky. The 0.61-m subgrade was constructed of a mixture of dolomite and New Albany Shale. CBR strengths of the New Albany Shale in an unsoaked and unsoaked state are near values of 30, as shown in Figure 2. A slake-durability index of the material is 99 percent (Hopkins 1983 and 1988.) Moreover, the long-term durability of this shale is very good;
it has degraded only slightly over a seven 7-year period (Hopkins et al 1991). In situ CBR values at two different locations of the New Albany Shale- dolomite subgrade shortly after construction were 24.3 and 15.6. In 1994--seven years after construction--values of in situ CBR were 18.9 and 5.4. The lower value was obtained because some Crab Orchard clayey shale had, inadvertently, been mixed during construction with the New Albany - dolomite subgrade at the point of testing. However, the larger value obtained after seven years indicate that the mixture is performing very well.

**Synthetic Aggregates**

Over the past few years, attempts (Hopkins et al., 1993) have been made to use byproducts, produced from coal-fired, electric power plants, to develop synthetic aggregates, or pellets. Such pellets could eventually be used to construct highway subgrades and bases. While improved synthetic aggregates have been produced, more research will be necessary to develop a commercial product.

**Encapsulated Subgrades**

As shown previously in Figures 1 and 2, CBR strengths of unsoaked compacted soils and shales are several times larger than CBR strengths of the same soils after soaking. Therefore, if moisture could be prevented from entering the top portion of clayey subgrades, then the CBR strengths would remain large. One idea to prevent moisture infiltration into subgrades is shown in Figure 52. This technique involves encapsulation of the upper reaches of the clayey subgrade with an insulating material, such as an asphalt emulsion. More research is needed to make this a viable technique.

**Chemical Admixture Stabilization -- Commercial Products**

Chemical admixture stabilization is the physical mixing and blending of a powder, liquid, or slurry with a soil. Commercially available products commonly used to stabilize fine-grained soils include hydrated lime, quick lime, cement, and fly ash. These products may be used in combinations. For example, highly plastic clay may first be treated with hydrated lime and then mixed with cement. Hydrated lime is also

Attempts in recent years have been made to use byproducts produced by industrial processes for stabilizing subgrades. For example, each year coal-fired electric power plants and some oil refineries generate enormous quantities of byproducts. FGD (flue gas desulfurization) materials contain high percentages of lime, or calcium oxide. Some 89 million tons of fly ash, boiler slag, and FGD materials were produced in 1991. Although some 25 percent of those materials were consumed by industry in making cement and concrete products, baseboard, and other products, about 75 percent of the generated materials were disposed in landfills. Almost 98 percent of FGD materials, and such byproducts as atmospheric fluidized bed combustion (AFBC), were disposed in landfills. Production of FGD-type of byproducts will increase tremendously in coming years and major efforts are being made by various researchers to increase their usage in the highway industry. As another example of the use of a byproduct, multicone kiln dust (MKD), a byproduct obtained in the manufacturing of hydrated lime, has the potential to be used as a chemical admixture stabilizer. Other chemical additives, such as salts, sodium silicate, and polymers (anidine-furfural, epoxies, latex, and calcium acrylate), have been considered as stabilizers. Many of these materials are not widely used because of ineffectiveness, safety considerations, and expense (McCallister and Petry, 1990). Some aspects of chemical admixture stabilization are described below.

**Hydrated Lime**

When hydrated lime--Ca(OH)$_2$ or quick lime (CaO)--is mixed with clayey soils, complex chemical reactions occur (Hunter, 1988). Normal pozzolanic reactions occur and involve four stages. In the first stage, calcium oxide reacts with water, or

$$CaO+H_2O=Ca(OH)_2,$$  \hspace{1cm} (34)

which forms calcium hydroxide. In the second stage, ionization occurs, or

$$Ca(OH)_2=Ca^2^+2(OH)^-,$$  \hspace{1cm} (35)

and the pH increases to 12.3. Subsequently, a dissolution of the clay mineral occurs. For example, as illustrated by Hunter (1988), the dissolution of montmorillonite would take a form of
Finally, dissociation of silicic acid occurs, or

\[
2H_4SiO_4 = 2H_3SiO_4^- + 2H^+ = 2H_2SiO_4^3.
\]  

(37)

and siliceous cementation is formed. Overall, any type of clay will perform similarly, or

\[
SiO_2(\text{soil silica}) + H_2O - CSH(\text{calcium-silicate-hydrate}),
\]

(38)

and

\[
Al_2O_3(\text{soil alumina}) + Ca^{2+} + H_2O - CAH(\text{calcium-aluminate-hydrate}).
\]

(39)

The cementing action of hydrated lime changes the structural and physical properties of the clayey soil when the clay fraction is about 10 percent or greater (Epps and Little, 1994). Hydrated lime is effective in treating clayey soils with plasticity indices as low as about 8-10 percent. Properties affected include plasticity, moisture-density relations, textural changes, swelling potential, permeability, and strength (Hopkins and Allen, 1986 and Hopkins et al, 1988). To illustrate how index properties are affected, index test data and soil classifications of untreated and hydrated lime-treated soils of Section AA-19 of the Ashland - Alexandria Highway are summarized and compared in Table 2. Three specimens collected from different locations along this section classified as MH-CH, CH, and MH-CH, respectively, before treatment. After treatment with 6 percent by weight of hydrated lime, the specimens were classified as SM, SM, and ML, respectively. The texture of the clay changed to a silty or sandy nature. Liquid limits of the untreated clays ranged from 61 to 71 percent. After treatment, the liquid limits were reduced to 41 to 53 percent. Plasticity indices, which ranged originally from 29 to 41 percent, decrease to a range of two to 6 percent. The percent finer than 0.002 mm-size particles of the untreated clay ranged from 58 to 66 percent. After treatment, the clay fraction ranged from 21 to 38 percent. AASHTO Classification of the clay before treatment was A-7-5(32-44). The clay-hydrated mixtures were classified as A-2 and A-4 after treatment.

Treatment of clay with hydrated lime causes a substantial change in the maximum dry density and optimum moisture content as illustrated in Table 3. Typically, as the
maximum dry density decreases, the optimum moisture content increases. The change depends on the curing time before testing and the percentage of lime used in the mixture.

Generally, hydrated lime causes an immediate change in the shear strength and bearing strength of compacted clay-hydrated lime mixtures. The effects are illustrated by data shown in Table 4 and Figures 53 and 54. In this series of tests, the specimens were compacted to maximum dry density and optimum moisture content (AASHTO T 99). Strength depends on curing time. For example, the unconfined compressive strength of clays from Section AA-19 of the Alexandria-Ashland Highway increases with curing time, as illustrated in Figures 53 and 54. Moreover, stiffness of the treated, compacted specimens (and the modulus of elasticity) increases with curing time.

### Table 2. **Index test data and soil classification of untreated and treated soil specimens, Section AA-19.**

<table>
<thead>
<tr>
<th>Sample Number and Location</th>
<th>Atterberg Limits</th>
<th>Specific Gravity</th>
<th>Particle-Size Analysis</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Liquid Limit (%)</td>
<td>Plastic Limit (%)</td>
<td>Plasticity Index (%)</td>
<td>No. 10 (%)</td>
</tr>
<tr>
<td>A  STA. 1630+00</td>
<td>71</td>
<td>34</td>
<td>37</td>
<td>2.97</td>
</tr>
<tr>
<td>B  STA. 1495+00</td>
<td>71</td>
<td>30</td>
<td>41</td>
<td>2.80</td>
</tr>
<tr>
<td>C  STA. 1675+50</td>
<td>61</td>
<td>32</td>
<td>29</td>
<td>2.80</td>
</tr>
<tr>
<td>A  STA. 1630+00</td>
<td>53</td>
<td>47</td>
<td>6</td>
<td>2.94</td>
</tr>
<tr>
<td>B  STA. 1495+00</td>
<td>45</td>
<td>43</td>
<td>2</td>
<td>2.80</td>
</tr>
<tr>
<td>C  STA. 1675+50</td>
<td>41</td>
<td>37</td>
<td>3</td>
<td>2.81</td>
</tr>
</tbody>
</table>
Table 3. Maximum dry densities and optimum moisture contents of untreated and treated soils from Section AA-19 and KY Route 11.

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Site Name</th>
<th>Type of Chemical Admixture</th>
<th>UNTREATED</th>
<th>TREATED</th>
<th>Chemical Additive (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Maximum Dry Density (KN/m³</td>
<td>Maximum Moisture Content (%)</td>
<td>Maximum Dry Density (PCF)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>PCF</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>Section AA-19</td>
<td>Lime</td>
<td>19.15</td>
<td>90.1</td>
<td>31.00</td>
</tr>
<tr>
<td>B</td>
<td>Section AA-19</td>
<td>Lime</td>
<td>15.12</td>
<td>96.3</td>
<td>24.50</td>
</tr>
<tr>
<td>C</td>
<td>Section AA-19</td>
<td>Lime</td>
<td>15.48</td>
<td>98.6</td>
<td>14.30</td>
</tr>
<tr>
<td>Stock pile 1</td>
<td>KY 11 Site 1</td>
<td>Lime</td>
<td>16.66</td>
<td>106.1</td>
<td>18.00</td>
</tr>
</tbody>
</table>

Table 4. Results of unconfined compression tests performed on remolded, untreated specimens and specimens treated with 6 percent hydrated lime (bag sample A, Station 1630+00, Section AA-19).

<table>
<thead>
<tr>
<th>Specimen Number</th>
<th>Unconfined Compressive Strength (PSF)</th>
<th>Failure Strain (%)</th>
<th>MOLDING CONDITIONS*</th>
<th>STANDARD COMPACTION**</th>
<th>Curing Time (Days)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kPa</td>
<td>Water Content (%)</td>
<td>Dry Density (KN/m³)</td>
<td>Optimum Water Content (%)</td>
<td>Maximum Dry Density (PCF)</td>
</tr>
<tr>
<td></td>
<td>PSF</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>UNTREATED SPECIMENS</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A-6</td>
<td>148.5</td>
<td>3100</td>
<td>4</td>
<td>30.3</td>
<td>13.67</td>
</tr>
<tr>
<td>A-7</td>
<td>215.1</td>
<td>4490</td>
<td>4</td>
<td>27.9</td>
<td>14.26</td>
</tr>
<tr>
<td>A-8 (soaked)</td>
<td>94.2</td>
<td>1965</td>
<td>4.9</td>
<td>31.9</td>
<td>14.11</td>
</tr>
<tr>
<td>TREATED SPECIMENS (6% LIME)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A-1</td>
<td>319.1</td>
<td>6450</td>
<td>2.7</td>
<td>35.9</td>
<td>13.17</td>
</tr>
<tr>
<td>A-4</td>
<td>287.5</td>
<td>6000</td>
<td>1.7</td>
<td>35</td>
<td>13.44</td>
</tr>
<tr>
<td>A-2</td>
<td>527.1</td>
<td>11000</td>
<td>1.4</td>
<td>35.9</td>
<td>13.36</td>
</tr>
<tr>
<td>A-3</td>
<td>589.7</td>
<td>12160</td>
<td>1.3</td>
<td>34</td>
<td>13.38</td>
</tr>
<tr>
<td>A-5</td>
<td>757.1</td>
<td>15800</td>
<td>1.8</td>
<td>32.3</td>
<td>13.69</td>
</tr>
</tbody>
</table>

* Water contents and dry densities of all specimens were determined at the time of testing.
** ASTM D 698.
time, as shown in Figure 54. In this example, the unconfined strength of the clay treated with 6 percent of hydrated lime, after 14 days of curing, was some four to sixteen times greater than the untreated specimens. The substantial increases in shear strength of clay-hydrated lime mixtures is also illustrated by the large increases in CBR, as shown in Table 5. Soaked values of bearing ratios, based on ASTM D 1198, of untreated specimens identified as A, B, and C from Section AA-19 were 3.3, 2.7, and 0.8 respectively. Soaked ASTM Bearing ratio values of those specimens, which had been treated with 3 percent hydrated lime, were 38.0, 30.3, and 8.0, respectively. Bearing ratio values of the lime-treated clays were some 10 to 11 times greater than CBR values of untreated (soaked) soils. Kentucky CBR tests were performed only on sample A obtained from Station 1630+00. The soaked minimum Kentucky CBR value of specimen A, which had not been treated with lime, was 2.6. This value occurred at 12.7-mm (0.5-in.) penetration. At 2.5-mm (0.1-in.) penetration, the soaked Kentucky CBR was 3.7 for the untreated soil. Minimum, KYCBR values of soaked specimens of sample A, which had been treated with 6 percent hydrated lime, ranged from 7.1 to 42.4, as shown in Table 4. These values occurred at a penetration of 12.7 mm (0.5 in.).

Curing times at room temperatures (before immersion in the water tank) varied from zero to 14 days. At 2.5-mm (0.1-in.) penetration, the KYCBR values were 32.3, 58.0, 59.5, and 137.3, which corresponded to curing times of 0, 3, 7, and 14 days, respectively. Soaked CBR values (2.5 mm) of
treated specimens of sample A were some nine to 37 greater than the KYCBR value obtained from an untreated specimen of sample A. In each case, where the soils had been treated, the minimum, soaked KYCBR value occurred at 12.7-mm (0.5-in) penetration. The maximum CBR value occurred at 2.5-mm (0.1-in.) penetration. The CBR value decreased with increasing stress. Comparisons of the CBR values at 2.5-mm (0.1-in.) and 12.7-mm (0.5-in.) penetration for soil specimens treated with 6 percent hydrated lime and cured for different periods are compared in Figure 55. CBR values at 2.5-mm (0.1-in.) penetration are much larger than CBR values at 12.7-mm (0.5-in.) penetration. Also, the CBR values at 2.5-mm penetration increased with curing time while the CBR values at 12.7-mm penetration remain almost constant with increasing curing time. At 2.5-mm penetration, a bearing capacity failure had occurred during the CBR test. For brittle soils, such as lime-treated or cement-treated soils, peak failure loads will occur at small strains. Therefore, the CBR value at peak failure stress is the more valid value than the CBR value at 12.7-mm penetration, which occurs after the peak failure stress has been reached.

Table 5. Results of bearing ratio tests of untreated specimens and specimens treated with 6 percent hydrated lime (bag sample A, Section AA-19).

<table>
<thead>
<tr>
<th>Sample Number and Location</th>
<th>UNTREATED SPECIMENS</th>
<th>TREATED SPECIMENS (6% HYDRATED LIME)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Soaked ASTM CBR</td>
<td>Soaked Kentucky CBR</td>
</tr>
<tr>
<td></td>
<td>2.5 mm (0.1-inch) Penetration</td>
<td>Minimum Value</td>
</tr>
<tr>
<td>A 1630+00</td>
<td>3.3</td>
<td>2.6</td>
</tr>
<tr>
<td>B 1495+00</td>
<td>2.7</td>
<td></td>
</tr>
<tr>
<td>C 1675+50</td>
<td>0.8</td>
<td></td>
</tr>
</tbody>
</table>

* Values occurred at 0.5-inch penetration.

** According to ASTM bearing ratio test (ASTM D 1883-73(1978)). The bearing ratio value occurring at 0.1-inch penetration is normally reported.

One of the most beneficial effects of mixing hydrated lime with clays is the large reduction in swelling of the compacted clay. For example, values of total volumetric
strain (swell) of the CBR specimens of treated and untreated soils from Section AA-19 are compared in Table 6. Also, swell strains obtained from both ASTM and KYCBR tests are compared. Swell strains obtained from the ASTM bearing ratio tests for the untreated soils (A, B, and C) ranged from 2.1 to 5.0 percent. After treatment with 6 percent hydrated lime, the swell strains observed in the ASTM bearing ratio tests decreased significantly and ranged from 0.2 to 2.4 percent. Swell strains from ASTM tests of lime-treated soils were some six to 52 percent lower than swell strains observed from untreated soils. However, in the ASTM bearing ratio tests, no curing time was used before immersion in water. As shown in Table 6, swell strains obtained from the KYCBR test were reduced significantly when the soils were treated with hydrated lime. For the untreated soil (A), the swell was 4.4 percent. For four treated specimens allowed to cure at 0, 3, 7, and 14 days (before immersion in the water tank), the swell strains were 0.5, 0.2, 0.1, and 0.04 percent, respectively. As shown in Figure 56, the swell strains decreased with increasing curing time. The swell strain of the hydrated lime-clay mixture was less than 1 percent after curing for 14 days.

Table 6. Total volumetric strains observed during bearing ratio tests

<table>
<thead>
<tr>
<th>Soil Sample and Specimen No.</th>
<th>ASTM Bearing Ratio Test Total</th>
<th>KYCBR Test Total Volumetric Strain (%)</th>
<th>ASTM Bearing Ratio Test Total</th>
<th>Soil Sample and Specimen No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil A</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A(ASTM-U)</td>
<td>2.10</td>
<td>--</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A(KY-U)</td>
<td>--</td>
<td>4.37</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A(ASTM-6-0-T)</td>
<td></td>
<td>1.09</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A(KY-6-0-T)</td>
<td></td>
<td>0.51 (No Curing Time)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A(KY-6-3-T)</td>
<td></td>
<td>0.17 (3-Day Curing Time)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A(KY-6-7-T)</td>
<td></td>
<td>0.15 (7-Day Curing Time)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>A(KY-6-14-T)</td>
<td></td>
<td>0.04 (14-Day Curing Time)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil B</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B(ASTM-U)</td>
<td>2.54</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B(ASTM-6-0-T)</td>
<td></td>
<td>0.22</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil C</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C(ASTM-U)</td>
<td>5.00</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C(ASTM-6-0-T)</td>
<td></td>
<td>2.40</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: 1. All specimens allowed one hour mellowing time when prepared
2. ASTM - ASTM bearing ratio test (ASTM D 1883-73 (1978): 8 refers to percent lime: U - untreated soil: 0, 3, 7, and 14 - refers to curing time in days at room temperature before specimens immersed in a water tank: T - treated with 6 percent hydrated lime: and KYCBR test (KM-64-601-76)
Cement

Although cement can be used to treat a wide range of different soils, it is commonly and very effective in treating soils of low plasticity, such as gravel, sands and silts. The short time set of cement can create stabilization difficulties when mixing cement and plastic clays containing large percentages of clay fractions. Procedures for selecting the percentage of cement with a given type of soil and construction guidelines have been published by the Portland Cement Association (1979).

Treatment of soils with cement generally causes significant changes in index and structural properties. Data in Table 7 typifies changes that occur when clayey soils are treated with cement. This table summarizes the index properties of untreated and treated soils obtained from three stockpiles at the KY 11 reconstruction site. Liquid limits of the stockpiled soils ranged from 36 to 43 percent. Plasticity indices ranged from 12 to 15 percent. Based on the AASHTO Classification system, the soils were classified as A-6(10), A-7-6(11), and A-6(8), respectively. The stockpiled soils were classified as CL according to the Unified Soil Classification System. Percent passing the No. 200 U.S. Sieve ranged from 70 to 74. The clay fraction -- percent finer than 0.002mm- size, ranged from 10 to 33 percent. After treatment with 10 percent of cement, the soils were nonplastic (NP). The cement-soil mixtures were classified as A-
The percent passing the Number 200 sieve decreased from 70-74 to 39. The maximum dry density and optimum moisture content (ASTM D 698) of the untreated, remolded soil were 16.66 KN/m³ (106.1 lb/ft³) and 18 percent. After mixing with 10 percent of cement, the maximum dry density and optimum moisture content soil-cement remolded mixture were 16.91 KN/m³ (107.7 lb/ft³) and 16.8 percent—essentially unchanged.

Maximum unconfined compressive of untreated soil specimens (KY 11 Route) remolded to maximum dry density and optimum moisture content was 294.6 kPa (42.7 psi). Unconfined strengths of specimens remolded to maximum dry density and optimum moisture content, and containing seven and 10 percent were about 1725 and 3243 kPa (250 and 470 psi), respectively. Soaked CBR strengths of untreated soils from Ky 11 were 3.7. After the treatment with 10 percent of cement, the soaked CBR was 300—a dramatic increase in bearing strength. Additionally, the swell of the untreated, remolded specimen was 5 percent and after treatment, the swell was less than 0.002 percent.

**Chemical Admixture Stabilization - Byproducts**

An attractive feature of certain byproducts produced from industrial processes is that some of these materials contain significant quantities of CaO (quicklime) and Ca(OH)₂ (calcium hydroxide, or hydrated lime). Flue gas desulfurization—FGD—(wet and dry) processes used at some coal-fired power plants produce significant quantities of byproducts containing varying, but significant, quantities of CaO and Ca(OH)₂. Another byproduct produced in making hydrated lime is a material called multicone kiln dust. This byproduct contains a large amount of CaO. When either of these types of byproducts are mixed with clays, pozzolanic reactions occur (see Equations 34 through 39) when the CaO and Ca(OH)₂ in the byproducts are mixed with the clays and water. Strengths of compacted clay soils increase. For example, unconfined compressive strengths of clay specimens of soils from the KY 11 reconstruction route were about four times larger than untreated remolded specimens. Those specimens were mixed with about 10 percent of an Atmospheric Fluidized Bed Combustion (AFBC) byproduct—FGD type of material—produced by an oil refinery. Both untreated and treated specimens were remolded to maximum dry density and optimum moisture content. Unconfined strengths of the KY 11 soils treated with about 10 percent of multicone kiln dust were some four times the unconfined strengths of untreated, remolded specimens.

Unfortunately, as shown by field and laboratory data in this report, FGD byproducts swell when mixed with clays and exposed to water. The large magnitudes of swell are apparently caused by the formation of ettringite and gypsum—highly expansive minerals. However, compacted specimens of several different types of FGD byproducts that have not been mixed with soils swell to large magnitudes when mixed
with water. FGD materials tested included an FBC byproduct from a circulating fluidized bed combustor in Pennsylvania, an FBC material from an experimental retrofit scheme, which would be used on the cool side of a coal-fired power plant, and an atmospheric fluidized bed combustion (AFBC) material from an oil refinery. Results of long-term laboratory swelling tests and field trials of one FGD byproduct (described later in this report), show that compacted specimens of three different types of FGD byproducts swell excessively when exposed to water, as shown in Figure 57. All specimens used to obtain the results in Figure 57 were compacted to 95 percent of maximum dry density and optimum moisture content (ASTM D 698). Vertical swelling of compacted FGD-clayey specimens, characteristically, consists of two portions, which may be called primary swelling and secondary swelling. Primary swelling is usually a large portion of the total amount of swelling. Secondary swelling as a function of the logarithmic of time is linear, as shown in Figure 58.

Primary swelling of compacted clays, which are typical soils from Kentucky (medium to high plasticity), commonly range from five to 13 percent, as shown in Figure 57. Pavements on clayey subgrades, which exhibit swelling magnitudes of 13 percent or greater, generally have performed very poorly. Swelling of FGD byproducts appears to depend on sulfur in the material. The sulfur may be in CaSO₄, although this is not conclusive. For example, the size of primary swelling of a Coolside FGD specimen identified as CS-2 was much larger than the magnitudes of swelling of Coolside specimens identified as CS-388 and CS-1040. The former material contained some 8
percent sulfur while the latter materials contained only 2 percent. The AFBC material contained high levels (10%) of CaSO₄. FGD byproducts containing high levels of sulfur, or CaSO₄, appear to swell to values much greater than 13 percent. Therefore, magnitudes of swell of that order would be detrimental to pavements. Also, some FGD byproducts swell over long periods. For example, primary swelling of compacted specimens of an FBC byproduct obtained from a circulating fluidized bed combustor has continued over a period of some 600 days and shows no sign of reaching secondary swelling, as shown in Figure 59. Because of the swelling nature of FGD materials, the use of these materials as chemical admixtures in soil subgrades, or as base materials, should be restricted. They should not be used unless it can be shown by long-term swelling tests that large, excessive swelling does not occur. Generally, if vertical swelling exceeds about 5 percent, then the FGD material should not be used.

Strengths of compacted specimens of FGD byproducts may vary with swell. As the total amount of swelling increases, the shear strength decreases. This dependency

Figure 59. Primary swell of compacted specimens of a FBC byproduct as a function of the logarithmic of time.

Figure 60. CBR strength of compacted specimens of FGD byproducts as a function of the logarithmic of swell.
is illustrated in Figure 60. In these series of tests on the Coolside material, the CBR test specimens were allowed to swell until secondary swelling occurred. All specimens were compacted to 95 percent of maximum dry density and optimum moisture content (ASTM D 698). Also, in some tests, a small surcharge pressure of 3.5 kPa (0.5 psi) was placed on top of the specimen. Additionally, some specimens were aged, or cured seven days, before submerging in water. One Coolside material contained 2 percent of sulfur (CS-388) while the second Coolside material contained 8 percent of sulfur (CS-2). As shown in Figure 60, at a vertical swell of about 28 percent, the CBR of the Coolside material was 10. Although a CBR of 10 would provide good bearing strength for a highway pavement, the large amount of swelling would cause excessive damage to a pavement placed on the compacted material. Therefore, to use FGD materials in the construction of highways, means must be found to control the expansive nature of these materials.

**OPTIMUM PERCENTAGE OF CHEMICAL ADMIXTURE**

In the design of untreated and chemically treated subgrades, geotechnical laboratory tests, such as triaxial unconfined compressive strength tests, should be performed on specimens of the materials that closely simulate the compactive state of the materials as they may exist in an engineered subgrade. The laboratory compaction test (Hopkins et al, 1988 and Hopkins and Beckham, 1993) should produce specimens for physical properties tests that duplicate, the anticipated, or specified, field dry density and moisture content. For example, if field compaction specifications require that a subgrade be compacted to 95 percent of maximum dry density and ± 2 percent of optimum moisture content (ASTM D 698), then the laboratory compaction procedure should produce specimens that duplicate, as practically as possible, those target values of dry density and moisture content.

Moreover, since the maximum dry density and optimum moisture content of a clayey soil may change as a chemical admixture is added, then it is essential that the remolding procedure account for such a change. Otherwise, the physical properties as measured from laboratory tests may differ significantly from those that may exist in the completed engineered subgrade. Additionally, when chemical admixtures are used, a method is needed to determine the most economical amount, or percentage, of a chemical admixture to add to the soil subgrade to obtain some desired strength. The procedures developed during this study for compacting specimens that conform to specified values of dry density and optimum content and for determining the optimum percentage of a chemical admixture are described below. The procedure is applicable to a variety of fine-grained materials, such as soils, byproducts, and mixtures of byproducts and soils and chemical admixtures and soils.
Compaction Equipment

Equations used in the proposed procedure to compact specimens to selected, or target, values of dry density and moisture content are described below. When byproducts, or other chemical admixtures, are added to soils, the matrix of the mixture consists of air, water, soil particles, and admixture particles. To determine the weights of soil solids ($W_s$), water ($W_w$), and a chemical admixture ($W_p$), that must be mixed to form a specimen of a known volume ($V$), dry density ($\gamma_t$), and moisture content ($w_t$), the phase diagram of Figure 61, may be used in formulating the necessary equations.

By definition, the total density ($\gamma_t$), is

$$\gamma_t = \frac{W}{V} = \frac{W_s + W_w + W_p}{V}. \quad (40)$$

The target moisture content, $w_t$, is, by definition

$$w_t = \frac{W_w}{W_s + W_p}. \quad (41)$$

By definition, the dry weight of an admixture is expressed as a percentage, $P$, of the dry weight of soil particles, or

$$W_p = PW_s \quad (42)$$

Solving for $W_w$ in Equation 41 and inserting this quantity, and the quantity given by Equation 42, into Equation 40, then the dry weight of soil, $W_s$, is

$$W_s = \frac{\gamma_t V}{(1+w_t)(1+P)}. \quad (43)$$

The air-dried weight, $W_{ad}$, is related to the oven-dried weight, $W_s$ (or weight of soil solids) by the expression:

$$W_{ad} = W_s (1+w_{hs}), \quad (44)$$
where \( w_{hs} \) is the (hygroscopic) moisture content existing in the soil at the time of mixing. The total density is related to the dry density by the expression,

\[
y_t = \gamma_d (1+w_t).
\]  

(45)

Substituting Equations 43 and 45 into Equation 44, then

\[
W_{ad} = \gamma_d V (1+w_{hs})/(1+P)
\]  

(46)

Equation 46 gives the weight of air-dried soil that must be added to form the specimen of a known volume, \( V \), dry density, \( \gamma_d \), and moisture content, \( w_t \). The air-dried weight, \( W_{pad} \), of chemical admixture may be computed from the following expression.

\[
W_{pad} = W_p (1+w_{hp}) = PW_s (1+w_{hp}),
\]  

(47)

where \( w_{hp} \) is the hygroscopic moisture content of the air-dried (or moisture content at the time of mixing) admixture. Equation 47 gives the weight of an air-dried admixture that must be added to the mixture.

The amount of water (\( W_{wadd} \)), which must be added to the mixture depends on the amounts of moisture (or hygroscopic moisture content) existing in the soil and admixture at the time of mixing. If the materials have been air-dried, then the materials may contain only hygroscopic moisture. Hygroscopic moisture content (\( w_{hs} \)), of the soil, and \( w_{hp} \), of the admixture must be determined from laboratory tests. The total amount of water required to mix the materials and form the specimen of a known volume and selected dry density may be determined from Equations 40 and 41, or,

\[
W_w = w_t W_s (1+P).
\]  

(48)

From the phase diagram, Figure 61, the amount of water existing in the soil before mixing (by definition) is,

\[
W_{whs} = w_{hs} W_s,
\]  

(49)

and in the admixture,

\[
W_{whp} = w_{hp} W_p = w_{hp} PW_s.
\]  

(50)

The amount of water to be added at the time of mixing (See Figure 61) is

\[
W_{wadd} = W_w - W_{whs} - W_{whp}.
\]  

(51)
Substituting the expressions given by Equations 47, 48, and 49 into Equation 50, then

$$W_{wadd} = W_s \{P(w_t - w_{hp}) + w_t - w_{hs}\}$$

(52)

A spreadsheet computer program was developed to compute the weights of air-dried soil, water, and admixture needed to compact a specimen of a known volume and selected values of dry density and moisture content.

**Compaction Method**

**Equipment**

Equipment required to compact a specimen includes some type of apparatus, or other means, for mixing the specimen, an electronic scale with a resolution of 0.01 grams, a split-type mold, and a specially-designed ram and slip rings. Although the split-type mold for compacting the specimens may be designed for any selected dimensions, a type of mold that is convenient for forming specimens for triaxial or permeability testing measures 20.32 cm (8 inches) in height and 7.11 cm (2.8 inches) in diameter. Specimens are compacted to a height of 15.24 cm (6 inches). The inside diameter of this mold is the same as the diameter (7.11 cm) of a commonly-used, thin-walled, field sampling tube. Another suitable size of a split-type mold for laboratory compaction purposes measures 15.24 cm in height and 5.08 cm (2 inches) in diameter. Specimens are compacted to a height of 11.43 cm (4.5 inches). By using a split-type mold, the specimen may be removed from the mold conveniently, the need to extrude the compacted specimen from the mold is avoided, and sample disturbance after compaction is reduced.

The function of the ram and rings, which slip over the ram, is to control the height of each layer of the compacted specimen. In the compaction standard, ASTM D 698, the specimen height is 11.6434 cm (4.584 inches); the specimen is compacted in three layers and each layer is 3.879 cm (1.527 inches) in height. In the proposed compaction procedure, each layer of the specimen is compacted to approximately the same height, or 3.81 cm (1.5 inches). For example, specimens measuring 15.24 cm in height are compacted in four layers but each layer is 3.81 cm in height. A schematic of the ram and slip rings used to compact specimens measuring 15.24 cm in height and 7.11 cm in diameter is illustrated in Figure 62. Views of the split mold, rings, and ram are shown in Figure 63.

**Procedure**

The purpose of the compaction procedure is to produce a specimen that has a dry density and moisture content that are near prescribed, or target, values of dry density and moisture content. For example, if field specifications dictate that a given material
Hopkins, Beckham, and Hunsucker—Modification of Highway Soil Subgrades

Figure 62. Schematic of the ram and slip rings used to compact specimens.

Figure 63. View of the split mold, rings, and ram.

must be compacted to 95 percent (or another choice) of maximum dry density obtained from a standard laboratory test, such as ASTM D698, then the target values for remolding the laboratory specimen would be selected according to the field specifications. After developing the specified moisture-dry density curve, target values of dry density and moisture content are selected. When only one material is involved (P=0), Equations 45 and 41 are used to calculate the weight of air-dried material and the volume of water that must be used to remold a specimen of a known (or selected) volume. However, the material does not necessarily have to be air-dried. The only requirement here is that the existing moisture content, \( w_{\text{ext}} \), in the material at the time of sampling, must be equal to or less than the selected, target moisture content. After a small sample is obtained to find the existing moisture content of the material, the material may immediately be placed and sealed in a zip-lock plastic bag to prevent any further loss of moisture. The material remains sealed until the time of mixing.
To mix the sample, the material is placed in a mixing bowl and the amount of water, as determined from Equation 51, is added to the material. When the specimen to be formed is 15.24 cm (6 in.) in height and 7.11 cm (2.8 in.) in diameter, the mixed material is divided into four parts of equal weight and stored in zip-lock bags. It is imperative that care is exercised in this portion of the procedure to avoid the loss of material when the material is weighed and transferred to the plastic bags. Normally, the material remains sealed in the plastic bags for about 24 hours before remolding to allow an even distribution of moisture. After the mellowing period, the specimen is compacted as illustrated in Figure 64. The contents of the first bag are placed in the split mold and the ram is hammered down until the collar of the ram rests against the top of the mold. When the collar touches the top of the mold, the first compacted layer is exactly 3.81 cm (1.5 in.) in height. The top of the first layer is scarified and the second bag of material is added to the mold. The first slip ring is slipped over the ram and the second layer is compacted. When the bottom of the first ring touches the top of the mold, the second layer is exactly 3.81 cm (1.5 in.). The procedure is repeated for the third and fourth layers, as shown in Figure 65, respectively. When the last layer is compacted, the specimen is exactly 15.24 cm (6 in.) in height. During the compaction procedure, the number of blows does not have to be counted because the exact amounts of materials and water are used to form the
specimen of a selected dry density, water content, and known volume.

When admixtures, such as a byproduct (Coolside, FBC, etc.) or hydrated lime, are added to soil, chemical reactions may occur. Maximum dry density and optimum moisture content derived from a given type of compaction test depend on the percent of an admixture used in the mix. If this percent of an admixture is known (or assumed), then the maximum dry density and optimum moisture content may be determined using the known percentage. However, if the objective of the testing program is to determine the optimum percent of an admixture, then the testing procedure must be altered because the maximum dry density and optimum moisture content vary with increasing percentage of an admixture. These variations are illustrated, for example, in Figures 66 and 67. This example involves mixtures of an AFBC byproduct (or admixtures) and a clayey soil. As the percent of AFBC byproduct increases, the maximum dry density decreases and the optimum moisture content increases. Therefore, to account for these variations, three to five compaction tests may need to be performed on the soil-admixture mixtures using different, selected percentages.
The percent of admixture is ranged from a low percent to a high percent. Since the objective of this testing program was to determine the maximum strength (unconfined compressive strength) and corresponding optimum percent of an AFBC byproduct, several compaction tests were performed on mixtures containing different percentages. Unconfined compressive tests were performed on specimens remolded to selected percentages of the AFBC byproduct. For a selected percent of an AFBC byproduct, the maximum dry density ($\gamma_d$), and optimum moisture contents were selected from the curves shown in Figures 66 and 67. Equations 45 through 51 are used to calculate the amounts of admixture, soil, and water that must be mixed to form a specimen of a known dry density, moisture content, and volume. Figure 68 shows the results of unconfined compressive tests performed on different remolded specimens of AFBC byproduct-soil mixtures. As these data show, the maximum strength occurred at 5 percent.

**Statistical Analysis**

To determine the reliability of the remolding procedure, actual dry densities and moisture contents obtained from the proposed compaction procedure were compared statistically to target values of dry densities and moisture contents. Seventy cases, which represented a variety of fine-grained soils, and different values of relative compaction, were analyzed. Deviation of the average dry density, or the difference between the actual dry density of the compacted specimen and target dry density, was $\pm 0.011 \, \text{kN/m}^3$ (0.07 lb/ft$^3$). Standard deviation was $\pm 0.121 \, \text{kN/m}^3$ (0.77 lb/ft$^3$). Deviation of the average moisture content, or the difference between the actual moisture content of the compacted specimen and the target moisture content, was -0.31 percent. Standard deviation was $\pm 0.88$ percent.

![Figure 68. Results of unconfined compressive strength tests performed on different remolded specimens of AFBC byproduct-soil mixtures.](image)
PERFORMANCES OF SOIL-CEMENT SUBGRADES

Although cement has been used in past years as a chemical admixture to improve bearing strengths of highway soil subgrades, little information has been published concerning the long-term bearing strengths of cement-treated soils and the long-term performances of flexible pavements on the cement-treated soils. Roberts (1986) reported that in 1938 the Oklahoma Highway Department investigated the use of cement-modified subgrades by constructing 11.3 km (7 mi) of test sections along U.S. 62. Plasticity indexes of the clayey subgrade soil were reduced from 39 to 13 percent at the time of construction. Plasticity indices of the cement-treated subgrade after 45 years of service ranged from nonplastic to 13 percent. Roberts showed that the pavements, placed on the treated subgrades, performed very well during the 45 years of service. McGhee (1972) described the use of cement-treated subgrades in Virginia and the long-term performances of pavements on treated subgrades. A major conclusion was that pavements constructed on cement-treated subgrades were much more resistant to rutting and other distortions when compared to most pavements ten or more years old. No long-term strength data are shown in the reports by Roberts or McGhee. Laboratory tests of remolded mixtures of soils and cement (Terrell et al., 1979) generally exhibit large shear strengths greater than 700 kPa (101.6 psi).

To determine the long-term strengths and field durability of cement-treated subgrades, and to examine the long-term performances of pavements constructed on cement-treated subgrades, four highway routes that have been in service for several years were selected (Hopkins et al., 1994). Two of the routes have been in service for some 30 years. Two sections of the third route have been in service for nine and 14 years, respectively. Two sections of the fourth route have been in service for about six years. A summary of the findings of geotechnical field and laboratory studies performed at the four highway routes and the general performances of flexible pavements on the soil-cement subgrades are discussed below.

Testing Procedures

Geotechnical field studies consisted of performing in situ bearing ratio tests on the top of the soil-cement subgrades and the top of the untreated soil subgrade found below the treated layer. These tests were performed through cored boreholes. Samples of the cement-treated subgrades and the untreated subgrades were obtained during the field testing for laboratory testing. Procedures of ASTM D 4429-84 were followed in performing the in situ bearing ratio tests. The reactive force necessary to push the penetrating rod was developed by jacking against the frame of a drill rig. Load and deflection measurements were obtained from a calibrated load ring and deflection dial. Efforts to obtain thin-walled tube samples were unsuccessful because of the hardness of the cement-treated subgrades. Core specimens of the pavement and cement-treated...
subgrades were obtained at each site. Diameters of the cores were 15 cm (6 in.). Measurements and visual inspections of the core specimens were used to find the actual *in situ* thickness of the flexible pavements and the number and thickness of overlays. Pavement performances were generally evaluated using overlay history (when available) and visual inspections.

Geotechnical laboratory tests were performed on the retrieved samples to determine the index properties of the cement-treated and untreated soils. Index tests, which consisted of liquid limits (ASTM D 4318-84), plastic limits (ASTM D 4318-84), particle-size analysis (ASTM D 422-63), specific gravity (ASTM D 854-83), and moisture content (ASTM D 2216-80), were performed. The materials were classified according to the Unified Soil Classification System (ASTM D 2487-85) and the AASHTO Classification System.

**Case Studies**

**KY Route 15**

KY Route 15, located between Campton and Jackson, Kentucky, is about 32 km (20 mi) in length. The flexible pavement of this stretch of highway has been in service for approximately 30 years. The original design section, consisted of 19.1 cm (7.5 in.) of flexible pavement, 15.2 cm (6 in.) of dense graded limestone aggregate, and 15.2 cm (6 in.) of a soil-(Portland) cement subgrade. Approximately 10 percent of cement (by dry unit weight) was mixed with the *in situ* soil subgrades. Details of the mixing operation were not available.

Index properties of the cement-treated and untreated subgrade soils at various locations are summarized in Table 8. At six locations, the untreated soils were classified as CL and A-4 to A-6. At other locations, the soils were classified as GC-GM, and SC. Plasticity indexes of the untreated soils generally ranged from six to 16, although one specimen was nonplastic. Classification of the cement-treated soils was SM and GM (after 30 years) and either A-1, A-2, or A-4. All specimens of the treated layer were nonplastic. Therefore, noticeable differences existed in the index properties of the treated and untreated soils.

*In situ* bearing ratios of the cement-treated layer and underlying soil subgrade are compared in Figure 69. Excluding the bearing ratio of 9.2 at Milepost 0.6, the bearing ratios of the cement-treated subgrades ranged from 54 to values greater than 100. *In situ* bearing ratios of the untreated soils ranged from 3.3 to 14.2. After 30 years, the *in situ* bearing ratios of the soil-cement were generally some four to 33 times greater than the untreated subgrade soils. Total pavement thicknesses ranged from 47.6 cm to 63.5 cm (18.7 to 25 in.), as shown in Figure 70.
Table 8. Index properties of soil-cement layer and untreated soils of Ky route 11.

<table>
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<td></td>
<td>No. 10</td>
<td>Classification</td>
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<td></td>
<td>No. 200</td>
<td>CBR</td>
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<td></td>
<td>AASHTO Unified</td>
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<tr>
<td>0.6</td>
<td>nonplastic</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
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<td></td>
</tr>
<tr>
<td>19.6</td>
<td>nonplastic</td>
<td>19.6</td>
</tr>
<tr>
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<td>24.8</td>
</tr>
<tr>
<td>26.7</td>
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<td>26.7</td>
</tr>
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</tr>
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<tr>
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</tr>
<tr>
<td>0.0</td>
<td>nonplastic</td>
<td>0.0</td>
</tr>
</tbody>
</table>

*No soil cement -- sandstone subgrade *Sections reconstructed (untreated sections)

Figure 69. In-situ bearing ratios of treated and untreated soils of KY 15.

Figure 70. Thicknesses of the pavement at several locations of KY 15.
At seven of the ten locations, which contained soil-cement subgrades, overlay thicknesses ranged from 3.2 to 6.4 cm (1.26 to 2.52 in.). At three of the ten locations, the overlay thicknesses ranged from 12.1 to 19.1 cm (4.76 to 7.52 in.). Since the pavements at those three locations were on side-hill-fills, overlays had been constructed when the fills had apparently settled over the 30-year period. The large overlay thicknesses observed at those three locations were a result of adding leveling courses, or patching, when the overlays were constructed. At two locations (MP 16 and 16.8), total thicknesses of the pavements ranged from 57.2 to 76.2 cm (22.5 to 30.0 in.). Overlay thicknesses were 10.2 and 14 cm (4.0 and 5.5 in.), respectively. At Milepost 16, the pavement was constructed on an untreated subgrade. At Milepost 16.8, the pavement was placed on a subgrade built with a select rock material.

The time between overlay construction generally ranged from 11 to 15 years for the flexible pavements and averaged 12.2 years. Based on visual inspections made in 1990, only nominal rutting was observed. Surface of the pavement contained some hairline cracks. However, these did not affect the riding quality of the pavements. Annual daily traffic (ADT) of KY 15, as reported in 1989, was 5,000 vehicles per day (VPD). About 13 percent (650 VPD) of the ADT consisted of trucks while 4 percent (200 VPD) of the ADT value consisted of coal trucks. Estimated equivalent single axle loads (ESAL)--converted from ADT values--observed each year at different locations from 1986 to 1992--ranged from about 0.8 to 4.0 million and averaged about two million per year.

**KY Route 79**

A stretch of Kentucky Route 79 was moved and reconstructed in 1959-60 when a dam was constructed to create Rough River Lake. Reconstruction began at the intersection of Kentucky Route 110 (MP 18.1) in Grayson County and extended to the edge of the dam at Milepost 19.7. Reconstruction resumed at the Breckinridge county line (MP 0.0) and extended north to the intersection with Kentucky Route 105 (MP 1.9) in Breckinridge County. Total length of reconstruction was 3.2 km (2 mi), not including the length of the roadway that traverses the dam (0.17 km, or 0.1 mile). This flexible pavement and cement-treated subgrade have been in service some 30 years.

The subgrade was constructed with a soil-cement-aggregate mixture designed by the U.S. Army Corps of Engineers. Proportions of the materials blended and construction specifications were not available. Thicknesses of the flexible pavement and stabilized subgrade, obtained from core measurements, are shown in Figure 71.

Total thicknesses of the pavements, as determined in 1991 and including the thicknesses of the treated layer, ranged from 25.4 cm to 35.6 cm (10 to 14 in.). Thicknesses of the treated layers (locations identified as two through 7) ranged from 16.5 to 22.9 cm (6.5 to 9.0 in.). Thicknesses of the flexible pavement, including
overlays, ranged from 7 cm to 19.1 cm (2.75 to 7.52 in.). A soil-cement layer was not used at location one and eight as shown in Figure 71. Locations identified as one and eight in Figure 71 are just beyond the limits of stabilization and are included for comparative purposes.

Index properties of the untreated and treated soils are shown in Table 9. The untreated soils were classified as CL, SM, and GC, and A-6, A-2-4, and A-2-7 at sampling locations. Plasticity indices of the untreated soils ranged from nonplastic to 44. Specimens of the cement-aggregate-soil subgrade obtained from three sampling sites were classified as SM and A-2-4 and A-1-B. The treated material was nonplastic. The untreated soils found below the treated layer of the section of roadway in Grayson County were nonplastic. Untreated soils below the treated layer of the section in Breckinridge County were generally plastic. Plasticity indexes ranged from 13 to 44 percent.

**In situ** bearing ratios of the treated layer and untreated layer are shown in Table 9 and compared in Figure 72. **In situ** bearing ratios of the treated materials ranged from 62 to values greater than 100. **In situ** bearing ratios of the untreated subgrade ranged from two to 7.1. The bearing ratios of the cement-treated materials were some seven to 50 times the bearing ratios of the untreated soils. After some 30 years, the bearing strength of the cement-aggregate-soil layer was much greater than the bearing strength of the underlying untreated subgrade. Annual daily traffic, as measured in 1990, was 1,950 vehicles per day. About 140 vehicles per day, or about 7.2 percent, of the total value of ADT, were classified as trucks. Values of
ESAL, beginning at Milepost 18.1 in Grayson County and ending at Milepost 1.9 in Breckinridge County, were 140,000 in 1989.

Table 9. Index properties of untreated soils and cement-aggregate-soil subgrade of Ky route 79.

<table>
<thead>
<tr>
<th>Location (MP)</th>
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<th>PI</th>
<th>Percent Passing</th>
<th>Classification</th>
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<tr>
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<tr>
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<td>72 28 44</td>
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<td>31.1</td>
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<td>71.0</td>
<td>69.3</td>
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Overlays were constructed on the section in Breckinridge County in 1979 and 1989. Intervals between overlay construction were about 10 years and 19 years, respectively. One overlay was constructed on the section in Grayson County in 1979, or about 19 years after construction. Reportedly, an overlay pavement was scheduled for 1991 or 1992, or about 14 years after the first overlay pavement. Therefore, the time between overlays ranged from about 10 to 19 years and averaged about 15 years. As shown in Figure 71, the thicknesses of the asphaltic overlays added to the Breckinridge section in a 31-year period were much larger than the thicknesses added to the Grayson section. In the former case, the overlay thicknesses ranged from 3.8 to 5.1 cm (1.5 to 2.0 in.) while in the latter case the overlay thicknesses ranged from 8.3 to 15.2 cm (3.3 to 6.0 in.). As shown in Figure 73, bearing ratios of the untreated soils found below the cement-treated materials of the Breckinridge section ranged from two to 2.8. Bearing ratios of the untreated soils of the Grayson section ranged from about two to seven.
Generally, the bearing ratios of the untreated soils of the Grayson section were greater than those of the Breckinridge section.

**U.S. Route 23**

U.S. Route 23 in Boyd County, Kentucky was reconstructed in the late seventies. Two sections containing cement-treated soil subgrades were included in the reconstruction. The first section, extending from Station 837+00 to 916+50 (2.4 km, or 1.5 mi), was reconstructed in 1977-78 and has been in service about 14 years. The original design cross section consisted of 28.6 cm (11.26 in.) of flexible pavement, 31.8 cm (12.52 in.) of dense graded (limestone) aggregate, and 15.2 cm (6.0 in.) of soil-cement. Total thickness of the pavement is 60.3 cm (23.74 in.). When the soil-cement layer is included, the thickness is 75.53 cm (29.74 in.). Observed thicknesses obtained from core specimens in 1990 are compared to the original design thickness in Figure 74. The second section, which extends from Station 916+50 to 1040+75 (3.9 km, or 2.44 mile), was completed in 1984 and has been in service about nine years. The design cross section consisted of 35.6 cm (14.0 in.) of asphaltic pavement and 15.2 cm (6.0 in.) of soil-cement. As shown in Figure 74, the actual thickness of a flexible pavement at three locations ranges from about 27.9 cm to 28.6 cm.

*Figure 73. In situ bearing ratios of treated and untreated materials of sections of US 23.*

*Figure 74. Pavement thickness of US 23 at different locations.*
Thickness of the soil-cement layer ranges from 15.2-20.3 cm (6.0-8.0 in.). This section contained no drainage layer. The top 15.2 cm (6.0 in.) of the soil subgrades of section 1 and 2 were mixed with 10 percent Portland cement. The third section extends from Station 1040+75 to 1151+36 and the design cross section consisted of 31.1 cm (12.24 in.) of asphaltic pavement resting on a rock subgrade. This section was included for comparative purposes. It was completed in 1985 and has been in service for about eight years. Actual thicknesses of flexible pavements at three locations were 29.8, 26.7, and 26.7 cm (11.73, 10.51, and 1.51in.).

Index properties of the cement-treated subgrades and untreated soil subgrades of Sections 1 and two are shown in the top portion of Table 10. The untreated materials were classified as SC and A-4 and A-6.

Table 10. Index properties of treated and untreated subgrades of U.S. Route 23.

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Untreated

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<th>CBR</th>
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<td>93.4</td>
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Rock Subgrade Section (Untreated)

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<td>27 18 9</td>
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</table>
The cement-treated subgrades were classified as SM and A-1-B and A-2-4. Plasticity indexes of the soil-cement materials ranged from nonplastic to 6 percent. Plasticity indexes of the untreated soils ranged from nine to 17 percent. The rock subgrade materials were classified as SC or GC and ranged from A-2 to A-6. Plasticity indices ranged from nine to 13.

_In situ_ bearing ratios, Figure 73, of the soil-cement subgrades were exceedingly large and ranged from 88 to values greater than 100. _In situ_ bearing ratios of the untreated soils of sections one and two ranged from 2.5 to 13.6. _In situ_ bearing ratios of the rock subgrade of Section 3 ranged from 13.2 to 191. Excluding the latter value, the bearing ratios generally ranged from 13.2 to 40. Bearing ratios of the soil-cement subgrades are greater several times than the bearing ratios of the untreated subgrades found below the treated layers and the rock subgrades of Section 3.

Based on core measurements, one overlay has been constructed on the pavement of Section 1 during its service period of about 14 years. Overlay thicknesses vary from about 2.54 to 4.11 cm (1.0 to 1.6 in.), as shown in Figure 74. No overlays have been constructed on sections two and three during their service periods of about nine and eight years, respectively. Annual daily traffic on the three sections, as obtained in 1989, is 8,890 VPD. About 3,645 VPD, or 41 percent, of the total ADT, or 2,578 vehicles per day were classified as coal trucks. Based on converted values of ADT and percentage of trucks, the estimated ESAL value is 16.2 million for 1989, as observed between mileposts four and nine. Between 1985 and 1991, the average ESAL was 7.2 million as observed at seven different locations between milepost 0.1 and 13.2. Although these sections carry a large volume of truck traffic, the three sections have performed well.

**KY Route 11**

Portions of Ky Route 11 were realigned and reconstructed in 1986-1987. The route is situated about 11.7 km (7.3 mi) north of Beattyville, Kentucky. Total length of the reconstructed route is 9.6 km (6 mi). The reconstructed route starts at Station 260+00 and ends at Station 576+50. Originally, the pavement was to consist of 19.1 cm (7.5 in.) of asphaltic pavement and 43.2 cm (17.0 in.) of dense graded limestone aggregate. A second option consisted of stabilizing the top 30.5 cm (12.0 in.) of subgrade with Portland cement for the entire route. However, the final plan, which was done by a change order, consisted of dividing the entire length of the roadway subgrade into sections and treating each subgrade section with different chemical admixtures. Chemical admixtures used in the different subgrade sections consisted of hydrated lime, multicone kiln dust (byproduct), a byproduct obtained from an atmospheric fluidized bed combustion process, and Portland cement (Type IP). Two sections of the subgrade were treated with cement. The first section extends from Station 317+00 to 348+00 (0.94 km, or 0.59 mi) while the second section extends from Station 429+00 to
522+00 (2.82 km, or 1.76 mi). The subgrade soils of the first section were mixed with 10 percent of cement while the soils of the second section were mixed with 7 percent of cement. The pavements of the two sections consisted of 15.2 cm (6.0 in.) of asphaltic concrete and 12.7 cm (5.0 in.) of dense graded limestone aggregate. To provide some basis for comparison, a section of the subgrade of reconstructed Route 11 was not treated. This section of the roadway extended from Station 522+00 to Station 532+00 (0.3 km, or 0.19 mi) and consisted 27.9 cm (11 in.) of asphaltic concrete and 12.7 cm (5.0 in.) of crushed stone.

Geology of the route consisted of interbedded layers of shales, sandstones, siltstones, and coal. The residual soils along the corridor are derivatives of those materials. Liquid and plasticity indexes of 25 corridor samples varied from nonplastic to 37 percent and nonplastic to 15 percent, respectively. The corridor soils were obtained before construction approximately at intervals of 152.4 meters (500 ft) along the route (Station 264 to 484). Generally, the soils were classified as CL and ML-CL.

At the 90th percentile test value of the 25 corridor samples, the liquid limit and plasticity indexes were 30 and eight, respectively. Liquid limit and plasticity indexes were 34 and 12 percent at the 50th percentile test value, respectively. Residual soils used to construct the subgrade were stockpiled at three locations (Sta. 233+00, 354+00, and 574+00), as shown in Table 11, along the route. Samples obtained from these stockpiles were classified as CL. Plasticity indexes ranged from 12 to 15 percent.

Optimum percentage of cement was determined following procedures described by Hopkins, et al, October 1988 and Hopkins and Beckham, 1993. In this approach, different percents of cement were mixed with the soil and compacted to a known volume, dry unit weight, and moisture content. After aging the specimens in sealed containers for seven days, unconfined triaxial compression tests are performed. The optimum percentage of cement is the point at which no increase (or only a slight increase) occurs in the unconfined compressive strength as the percentage of cement increases. Two soil-cement series of unconfined compressive tests were performed on soils from stockpiles at Stations 273+00 and 574+00. The maximum dry density and optimum moisture content of treated and untreated soils were essentially the same. Therefore, values of maximum dry density and optimum moisture content determined from the standard compaction test (ASTM D 698, Method A) on untreated soils were used to form specimens for unconfined compression testing. Unconfined compressive strengths are shown in Figure 75 as a function of the percentage of cement. The optimum percentage of cement occurs at eight to 10.

In situ bearing ratios of the soil-cement layers, the soil subgrades found below the treated layers, and the soil subgrade of the untreated section were obtained over a period of about six years. The field bearing ratios obtained for the two cement sections are shown in Figure 76. The lowest bearing ratio of the cement subgrades observed
during the 6-year period was seven. Excluding that value, bearing ratios ranged from 11 to values greater than 100 and averaged about 62. Bearing ratios of the untreated soil subgrade found below the cement-treated layers ranged from two to 14 and averaged 5.7.

Table 11. Index properties of treated and untreated soils of Ky route 11

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<th>Year</th>
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<th>PL</th>
<th>PI</th>
<th>Percent Passing</th>
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Cement-Treated Soil

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<td>433+15**</td>
<td></td>
<td></td>
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<tr>
<td>MP 14**</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>273+00***</td>
<td>39 25 14</td>
<td>100</td>
<td>91.2</td>
<td>74.0</td>
<td>A-6 (10)</td>
<td>CL</td>
<td></td>
</tr>
<tr>
<td>354+00***</td>
<td>43 28 15</td>
<td>100</td>
<td>92.4</td>
<td>73.0</td>
<td>A-7-6 (11)</td>
<td>CL</td>
<td></td>
</tr>
<tr>
<td>574+00***</td>
<td>36 24 12</td>
<td>100</td>
<td>89.7</td>
<td>70.0</td>
<td>A-6 (8)</td>
<td>CL</td>
<td></td>
</tr>
</tbody>
</table>

Untreated Soil

**10% **7% ***Stockpiles 1, 2, and 3
Figure 75. Optimum percentage of cement.

Figure 76. In-situ bearing ratios of soil-cement layers and untreated soil subgrades of KY 11.
Since the soils throughout the length of the reconstruction were essentially the same, the change in subgrade moisture content was measured during the study period. As shown in Figure 77, the moisture content of the untreated subgrade increased with increasing time. At the 90th percentile test value, moisture content increased from about 6.5 percent, as observed at the time of construction 1987, to about 16.5 percent in 1993. At the 50th percentile test value, the moisture content increased from 12 percent to 20 percent.

As the moisture content increased, bearing ratios decreased, as shown in Figure 78. While in situ bearing ratios of the soil subgrades during construction ranged from about 20 to 40, the in situ bearing ratios determined in March of 1993 ranged from one to six. Percentile test values as a function of laboratory CBR values of corridor soils and field CBR values of the untreated subgrade soils observed during the study period are shown in Figure 79.

At the 90th percentile test value, the field and laboratory CBR values are about three. The field and laboratory CBR values at the 50th percentile test value are six and seven, respectively.

Rutting measurements of the pavement obtained in 1993 for all experimental sections are compared in Figure 80. Depth of rutting of pavements on the cement-treated subgrades ranged from 0.00 to 0.15 cm (0 to 0.06 in.) after six years and was less than
pavement rutting depths of the other experimental sections. Depth of rutting of the pavement in the control section ranged from 0.33 to 0.48 cm (0.13 to 0.19 in.).

Annual daily traffic of Ky route 11, as measured in 1990, was 2,200 VPD. About 13 percent of the ADT value was classified as trucks. Three percent of the trucks were classified as coal trucks. Estimated values of ESAL observed in 1988 and 1989 were 434,000 and 365,000, respectively. Percentile test value as a function of in situ CBR of all soil-cement subgrades of all study sites is shown in Figure 81. At the 90th percentile test value, the CBR values are 25 and 90 respectively.

PERFORMANCES OF SUBGRADES MIXED WITH TWO BYPRODUCTS

To increase the use of byproducts in highway construction, the Kentucky Department of Highways elected to experimentally use a residue from an atmospheric fluidized bed combustion (AFBC) process from an oil refinery to modify a highway subgrade soil. Atmospheric fluidized bed combustion is an advanced
process that provides an environmentally acceptable method of reducing sulfur dioxide emissions in the refining process. Sulfur dioxide, an undesirable byproduct produced by the cracking of crude oil, is captured by calcium oxide (quick lime) during the combustion process and forms calcium sulfate. Construction and operation of fluidized bed combustion units in oil refineries and coal-fired electric generating plants in Kentucky represent high volume sources of byproducts that normally require disposal in landfills at substantial costs. Production of these types of materials, such as the AFBC byproduct, represents large liability and operating expense to many industries. By substituting the AFBC byproduct for commercially available chemical admixtures, mineral resources and construction materials may be conserved. A useful potential application for disposing of the AFBC spent lime may be identified, and useful lives of existing landfills may be extended.

A second byproduct selected for experimental use was multicone kiln dust (MKD). The generic name for MKD is lime kiln dust. MKD is a dry-collected byproduct generated in the production of hydrated lime. Approximately 18 million tons of this material are generated annually in the United States. Current disposal practices include use of the material in waste water treatment plants to treat sewage sludge and soil modification. Chemical analysis of the MKD byproduct, Figure 82, shows that the available CaO (quicklime) is about 23 percent (Source: Dravo Lime Company). Approximately, 63 percent of the material passes the U.S. Sieve No. 200 while 100 percent passes the No. 20 sieve, as shown in Figure 83.

In preliminary geotechnical laboratory tests, the shear and bearing strengths of remolded mixtures of AFBC spent lime and clayey soils and MKD and clayey soils were several times larger than strengths of remolded specimens of the untreated clayey soils. As shown by the
chemical analyses in Figures 82 and 84, the MKD byproduct and AFBC spent lime containing substantial quantities of CaO (quick lime), MgO, and Ca(OH)₂ (hydrated lime.) When the byproducts are mixed with clayey soils (source of alumina and silica) and water, soil-lime pozzolanic reactions occur to form cementing-type materials. Although the initial testing program was not specifically directed toward studying the swell characteristics of compacted AFBC-soil mixtures, no adverse swelling was observed. However, swell observations of the laboratory specimens were confined to periods not exceeding 10 days. Based on results of the preliminary testing, two experimental sections of the AFBC spent lime modified subgrade soils were proposed and constructed on a highway route (KY11) in Kentucky. One subgrade section was treated with the MKD byproduct. Additionally, other experimental subgrade sections were constructed using cement and hydrated lime (see Figure 85.) However, the main focus here is on the performances of the AFBC and MKD subgrade sections.

A summary of findings of the geotechnical laboratory and experimental field trial evaluations of using AFBC spent lime and multicone kiln dust byproducts as highway subgrade soil modifiers are given below. Approximately two months after construction of the AFBC spent lime-soil subgrade and placement of the asphaltic base courses, severe differential heave or swell occurred as shown in Figure 86. Specific field and laboratory mitigation measures devised after pavement heaving occurred are described herein. Performances of the asphaltic pavements on the AFBC spent lime-soil
subgrades and the MKD-soil subgrade after six years are discussed.

**Site Description and Geology**

Kentucky Route 11 was selected to construct two experimental sections of AFBC spent lime-soil subgrades and one section of MKD-soil subgrade. Some 11.2 km (6.6 mi) of this route involved reconstruction. The site is about seven miles north of Beattyville, Kentucky. An overall schematic of the layout of the Ky 11 experimental sections is shown in Figure 86. Geology of the area consists of interbedded layers of shales, sandstones, siltstones, and some coal. Soils at the construction site are residual and consist of derivatives of the shales, sandstones, siltstones, and coal. The first AFBC subgrade section is about 1.75 km (1.09 mi) in length and extends from station 260 to station 317+50. The second section extends from station 532 to 576+50 and is 1.35 km (0.84 mi) in length.

Other experimental sections constructed in the 9.6-km (6.0-mi) route included subgrades stabilized with hydrated lime and cement. Details of these sections have been presented by Hopkins, et al. (1988). A short section extending from station 522 to 532 (0.3 km or 0.19 mi) was not modified so that it could serve as a control section for comparative purposes. The pavement section, as initially proposed, for route 11 consisted of 21.6 cm (8.5 in.) of asphaltic concrete and 43.2 cm (17 in.) of dense graded aggregate (DGA) base. With the inclusion of a 30.5 cm- (12- in.) layer of AFBC spent lime-soil subgrade, and MKD-soil subgrade, thickness of the DGA was reduced from 43.5 cm to 12.7 cm (5 in.) in the two experimental sections. Thickness of the asphaltic concrete was not reduced. Configurations of the design sections are compared in Figure 87.

![Figure 86. View of heaved pavement.](image-url)
The main objective of the geotechnical laboratory testing program was to determine the suitability of using the AFBC spent lime and MKD byproducts as chemical admixtures. Other purposes of the laboratory testing were to classify the untreated soils of the KY 11 Highway route, determine changes, if any, in the engineering properties of the soils after treatment with the AFBC spent lime and MKD byproducts, and determine the optimum percentage of AFBC spent lime and MKD to mix with the soils. The testing consisted of finding select engineering properties of the soils in a natural state and in an altered state. An extensive laboratory testing program conducted prior to construction began by obtaining disturbed samples of the natural soils from three stockpiles (at stations 273, 334, and 574) constructed by the contractor. Additionally, the Geotechnical Branch of the Kentucky Department of Highways collected disturbed soil samples of
the completed subgrade every 152.4 meters (500 ft) along the entire length of the reconstructed roadway.

The laboratory study consisted of performing liquid and plastic limits, specific gravities, particle-size analyses, soil classifications, visual descriptions, moisture contents, moisture-density relations, bearing ratio tests, swell tests, unconfined triaxial compression tests, and pH tests. Liquid and plastic limits were performed according to procedures of ASTM D 4318-84. Particle-size analysis and specific gravity tests were performed according to ASTM D 422-63 and ASTM D 854-83. Moisture contents were determined according to ASTM D 2216-80. The untreated and treated materials were classified using the Unified Soil Classification System, ASTM D 2487-85, and the AASHTO Classification System M 145. Moisture-density relationships of treated and untreated soils were determined according to ASTM D 698-78, Method A, or AASHTO T 99. The purposes of these tests were to establish the optimum water contents and maximum dry densities of the untreated soils, and to study variation in the optimum water contents and maximum dry densities of the residue-soil mixtures as the percentage of AFBC spent lime increased. Values from the moisture-density tests were used to check field compaction of the AFBC spent lime-treated subgrades.

Unconfined triaxial compression tests (ASTM D 2166-85), performed on remolded specimens, were used to establish the optimum percentage of AFBC. The general scheme for determining the optimum percentage has been described by Hopkins and Beckham (September 1993) and Hopkins, et al., (1988). In this approach, several soil specimens are remolded at different percentages of a chemical admixture and at selected values of moisture content and dry density. The samples were sealed and aged seven days at room temperature before unconfined compressive strength was determined. Unconfined compressive strength is plotted as a function of the percentage of the chemical admixture. Optimum percentage of chemical admixture is a point where there is no significant increase in the unconfined compressive strength as the percentage of chemical admixture increases. Since the AFBC residue contains significant quantities of lime, pH tests were done following a procedure by Eades and Grimm (1966). This is a quick method for finding the optimum percentage for lime stabilization.

California Bearing Ratio (CBR) tests were performed on the treated and untreated soils following Kentucky Method KM-64-501 (1987.) In the Kentucky method, CBR specimens are molded using values of optimum moisture and maximum dry density as determined from ASTM D 698. While performing the CBR tests, vertical swell measurements were made according to test procedures of KM-64-501 (Kentucky Methods, 1987.) Swell measurements are continued in this procedure until the difference in two consecutive readings for a 24-hour time period is less than or equal to 0.076 mm (0.003 inches). Additionally, a few selected swell tests were performed on compacted specimens placed in consolidometers.
Laboratory Test Results

Index properties of untreated and AFBC spent lime-treated soils from the three stockpiles are compared in Table 12. Liquid limits and plasticity indices of the natural soils ranged from 36 to 43 percent and 12 to 15 percent, respectively. The natural soils classified as CL and A-6(8) to A-7-6(10.) Approximately 70 percent of the particles passed the number (U.S. standard) 200 sieve.

Table 12. Index properties of treated soils and untreated soils from stockpiles.

<table>
<thead>
<tr>
<th>Sample Type</th>
<th>Stockpile Location</th>
<th>Liquid Limit (%)</th>
<th>Plasticity Index (%)</th>
<th>Specific Gravity</th>
<th>Grain-Size Analysis Percent Finer Than:</th>
<th>Classification</th>
<th>Chemical Additive (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Na. 4 (%)</td>
<td>Na. 10 (%)</td>
<td>Na. 40 (%)</td>
</tr>
<tr>
<td>Untreated</td>
<td>273+00</td>
<td>39</td>
<td>14</td>
<td>2.69</td>
<td>100.0</td>
<td>91.2</td>
<td>82.4</td>
</tr>
<tr>
<td>Untreated</td>
<td>334+00</td>
<td>43</td>
<td>15</td>
<td>2.80</td>
<td>100.0</td>
<td>92.4</td>
<td>82.0</td>
</tr>
<tr>
<td>Untreated</td>
<td>574+00</td>
<td>38</td>
<td>12</td>
<td>2.72</td>
<td>100.0</td>
<td>88.7</td>
<td>81.9</td>
</tr>
<tr>
<td>AFBC-Treated</td>
<td>273+00</td>
<td>47</td>
<td>15</td>
<td>2.83</td>
<td>100.0</td>
<td>91.2</td>
<td>77.5</td>
</tr>
<tr>
<td>AFBC-Treated</td>
<td>334+00</td>
<td>51</td>
<td>13</td>
<td>2.80</td>
<td>100.0</td>
<td>91.2</td>
<td>75.6</td>
</tr>
<tr>
<td>AFBC-Treated</td>
<td>334+00</td>
<td>48</td>
<td>12</td>
<td>2.79</td>
<td>100.0</td>
<td>92.4</td>
<td>80.3</td>
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<tr>
<td>AFBC-Treated</td>
<td>334+00</td>
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<td>14</td>
<td>2.80</td>
<td>100.0</td>
<td>92.4</td>
<td>83.6</td>
</tr>
</tbody>
</table>

Liquid and plastic limits of the samples obtained at 152.4 meter (500 ft) intervals of the completed (untreated) subgrade are shown in Figure 88 as a function of percentile test value. At the 50th percentile test value, the liquid and plasticity index are about 34 and 11 percent, respectively. Treatment of the soils with 4 percent and 7 percent of AFBC spent lime produced mixed results. Plasticity indices showed little or no change. Liquid limits appeared to increase. Unified classification of the treated soils ranged from CL to SM. Treatment with the AFBC spent lime did not
appear to improve index properties of the soils.

Maximum dry density of the soils treated with the AFBC spent lime decreased as the percentage of spent lime increased (shown previously in Figure 66.) The maximum dry density of soils from the stockpile at station 574 decreased from 16.66 kN/m$^3$ (106.1 pounds per cubic foot (pcf)) at zero percent to 15.62 kN/m$^3$ (99.5 pcf) at 12 percent. Similarly, the maximum dry density of soils from the stockpiles at stations 273 and 334 decreased from 17.71 kN/m$^3$ (112.8 pcf) at zero percent to 16.17 kN/m$^3$ (103 pcf) at 10 percent. Optimum moisture content of the treated soils increased as the percentage of AFBC increased (see Figure 67.) Optimum moisture content of soils from the stockpile at stations 574 increased from 18 percent at zero percentage of AFBC to about 21.2 percent at 5 percent of AFBC. Optimum moisture content decreased slightly when the percentage of AFBC was above five. Optimum moisture content for soils from the stockpiles at station 273 and 334 increased from 16.3 percent at zero percent to 18.8 percent at 10 percent of AFBC spent lime.

To establish the optimum percentage of AFBC spent lime to be added to the subgrade soils, unconfined triaxial compression tests were done on compacted specimens containing different percentages of the spent lime. At a selected percentage of AFBC spent lime, the maximum dry density and optimum moisture content used to form the specimens were obtained from the curves presented previously in Figures 66 and 67. Variations of the unconfined compressive strength as a function of the percentage of AFBC spent lime are shown in Figure 68. A value of 2 percent was added to the optimum percentage for use in the field mixture. Unconfined compressive strengths of the remolded specimens at 5 percent of AFBC spent lime were about four times greater than the strengths of untreated (unsoaked) remolded specimens.

Although the procedure by Eades and Grimes (1966) was devised specifically for finding the optimum percentage of hydrated lime, the method was used with the AFBC spent lime to determine if it was applicable. Results of the pH tests, as shown in Figure 89.
89, show that the optimum percentage of AFBC was about five. This value compares well with the optimum value obtained from the unconfined compressive test results given in Figure 68.

The values of a soaked CBR test performed on a compacted specimen of soil from one stockpile were 3.7 and 4.1 at penetrations of 25 and 12.7 mm (0.1 and 0.5 in.), respectively. A plot of percentile test values (a method proposed by Yoder, 1969) as a function of the soaked CBR values of the highway corridor soils (samples obtained about every 152.4 m (500 ft) before construction) are shown in Figure 90. At the 50th and 90th percentile test values, CBR values of the natural soils are 7.1 and 2.9, respectively. Values of CBR at penetration values of 2.5 and 12 mm (0.1 and 0.5 in.) obtained from a compacted specimen aged seven days before soaking and containing 7 percent of AFBC spent lime, were 48 and 33. These values were some nine to 13 times greater than values of CBR of the untreated specimen. Results of the unconfined and CBR testing showed that substantial strength gains could be obtained by mixing the AFBC spent lime with the subgrade soils of the KY 11 route.

Vertical swell measurements obtained from a compacted CBR specimen containing 7 percent of AFBC spent lime indicated that total swell (Figure 91) was about 3.1 percent. Unfortunately, this test was performed for the purpose of determining the value of
CBR and not swell. (In retrospect, the AFBC spent lime-soil specimen should have been allowed to soak for a longer period.) In the Kentucky CBR method, swell measurements are obtained during soaking until the differences between consecutive readings are equal to or less than 0.076 mm (0.003 inches) after a minimum soaking time of 72 hours. When this condition occurs, the soaking period is ended and the CBR penetration is done. Swell measurements of a CBR specimen of compacted soil indicated a total swell of about 3.3 percent.

Experimental use of multicone kiln dust was not considered for this project until after construction had started. Consequently, laboratory testing was limited to determining the optimum percentage of the MKD byproduct, as shown in Figure 92.

**Construction Procedures**

Preparation of the untreated soil subgrade of the entire project length was completed in May 1987. This work involved compacting and shaping the grade to subgrade elevation or to an elevation slightly below planned grade elevation to accommodate anticipated volume increases due to the incorporation of the AFBC spent lime.

**AFBC Spent Lime-Soil Subgrade Section 1 -- Station 260 + 00 to 317+50**

A special note was developed exclusively for the AFBC residue roadbed stabilization. The special note required primary and final mixing. The note required that two-thirds of the spent lime be placed initially and the moisture content of the modified subgrade should not be less than optimum and not more than 5 percent above optimum moisture content. After primary mixing, the modified layer is cured for at least 48 hours. This allows the spent lime and water to break down clayey clods. The surface during this period is kept moist. After the preliminary phase, the remaining spent lime should be spread and the stabilized layer is completely mixed and pulverized again. Final mixing continues until all clods are broken down so that 100 percent, exclusive of rock particles, passes a 2.54-cm (one-in.) sieve and at least 60 percent passes a No. 4 sieve (4.75 mm). However, it is said that if the pulverization requirement can be met during
the primary mixing phase, then the preliminary curing and final mixing steps may be eliminated. This was the situation on the two AFBC sections. A bituminous curing seal is required to prevent moisture loss. Vehicular traffic is prohibited for seven days.

After final mixing, the subgrade was compacted to a minimum of 95 percent of maximum dry density. Final moisture content specifications were minus four to plus 2 percent of optimum. The AFBC spent lime (unhydrated) was transported to the job site from a nearby oil refinery by covered tractor-trailer trucks. Initially, the spent lime was dumped into a storage pit and covered. A front end loader was used to load the material into modified spreader trucks (tops of the spreader trucks had been removed to allow loading). Approximately, 7 percent (by dry weight of the soil) of the spent lime was spread over sections measuring 61 meters (200 feet) in length. Much of the water applied for mixing ran off the grade and into ditches because of the hard, smooth subgrade. After spreading the spent lime, a Ray-Go® soil pulverizer, Figure 93, was used to pulverize and mix the spent lime, soil, and water to a depth of 30.5 cm (12 inches). The pulverization requirement was met on the first pass. A sheeps-foot vibratory roller was used for initial compaction and a smooth-wheel vibratory roller was used for final compaction. A grader was used to cut the subgrade to proper grade. A bituminous seal was placed after achieving grade elevation.

After completing approximately one-half of the first AFBC spent lime section, the material hauler began dumping the AFBC spent lime directly on the subgrade instead of placing it in a stockpile. A front-end loader was used by the subcontractor to spread the spent lime. Although this spreading method appeared to work as well as the use of the spreader truck, there was virtually no control on the amount (percentage) of spent lime used in the mixing operation.
Difficulties encountered on the first AFBC spent lime modification section included controlling the flow of the material, having to cut the modified subgrade to final grade elevation, and obtaining correct moisture-density measurements. Because of the fine-grained nature of the AFBC spent lime, the material flowed much like a liquid. It was necessary for the subcontractor to construct windrows along the subgrade shoulders to contain the AFBC spent lime material and water on the subgrade. When the water was placed on the AFBC spent lime, a significant amount of steam was produced, reducing visibility to near zero. Another problem was the absence of cut-off valves inside the cabs of the water trucks. Occasionally, a water truck would become stuck in the mud and discharge excessive water onto the subgrade. Cut-off valves were installed in the cab after the first day. The subcontractor spent considerable time cutting the modified soil to grade elevation. Because the incorporation of the AFBC spent lime increased the soil volume, nearly four inches of the modified soil had to be trimmed to obtain proper grade elevation. However, the modified soil subgrade was easily trimmed even 24 to 30 hours after compaction.

Inspectors experienced some difficulties in obtaining correct moisture readings from the nuclear density gauge because of the presence of hydrocarbons in the modified subgrade. After the problem was identified, the inspectors determined the actual moisture content by applying a moisture-content correction factor. The correction factor was found by field drying a soil sample to determine the correct moisture content. The correction factor was entered into the nuclear density machine.

**Soil-AFBC Subgrade Section -- Station 532+00 to 576+50**

This section was conceived after construction difficulties were encountered on the first AFBC spent lime modified subgrade section. Construction procedures were altered from those used on the initial section to include ripping the prepared subgrade before spreading the AFBC spent lime material. After the AFBC spent lime materials were spread over the ripped subgrade, water was added and the soil was pulverized. The application rate of the AFBC spent lime was 7 percent by dry weight of the soil for this section. The subgrade was checked for the proper moisture content and dry density after initial compaction with a sheeps-foot vibratory roller. Final compaction was completed using the smooth-wheel vibratory roller. After completing compaction requirements, the modified subgrade was cut to elevation grade and a curing seal of bituminous emulsion was sprayed. Generally, traffic was prevented from traveling on the subgrade for seven days.

**Untreated Soil Subgrade Section -- Station 522+00 to 532+00**

A 305-meter (1,000-foot) section of the subgrade was not stabilized and served as a control section for the project. Conventional (sheepfoot roller) compaction methods were used to construct the subgrade within this section.
Soil-Multicone Kiln Dust Subgrade Section -- Station 402+50 to 429+50

The soil-multicone kiln dust subgrade modification section was experimental. This was the first time a MKD byproduct was used to modify a soil subgrade in Kentucky. The construction requirements for the MKD subgrade modification are outlined in Kentucky Department of Highways' Special Note for MKD Roadbed Stabilization (Experimental). The Special Note was developed by engineers exclusively for this project. The Special Note requires primary and final mixing. Two thirds of the MKD is specified to be placed initially during the primary mixing phase. The moisture content of the modified soil during the primary mixing phase was required to be no less than optimum, and no more than five percent greater than the optimum moisture content. After primary mixing, the modified soil layer was required to be shaped to the approximate cross section and lightly compacted to minimize evaporation loss. Following primary mixing, the modified soil layer was required to be shaped to the approximate cross section and lightly compacted to minimize evaporation loss. Following primary mixing, preliminary curing (mellowing) of the modified soil layer was required for at least 48 hours. During the preliminary curing phase, it was specified that the surface of the subgrade be kept moist to prevent drying and cracking. Immediately after the preliminary curing phase, the Special Note required the remaining portion of the MKD be spread and the modified layer completely mixed and pulverized again. Final mixing continued until all clods were broken down so that 100 percent, exclusive of rock particles, passes a one-inch sieve and at least 60 percent passes a No. 4 sieve. However, if the pulverization requirement can be met during the primary mixing phase, then the preliminary curing and final mixing steps can be eliminated. After compaction and shaping to grade elevation, the Special Note requires that a bituminous curing seal be placed to prevent excessive loss of moisture. No vehicular traffic is to traverse the subgrade until after a period of seven days.

The spreader trucks were filled directly using pneumatic tanker trucks. A road grader with a ripper attachment, was used to rip the subgrade prior to placement of the MKD. After spreading the MKD and water, Ray-Go® soil pulverizers were used to mix the materials. A sheepfoot roller provided initial compaction. Inspectors checked for moisture content after initial compaction using nuclear moisture/density devices. When there was not sufficient moisture in the subgrade, additional water was added, the soil was re-pulverized and re-compacted. The working area for the MKD section from was shoulder to shoulder, and approximately 61 meters (200 feet) in length. The pulverization requirement was met with one pass, thereby eliminating the need for a mellowing period. Required dry density and moisture content of the compacted subgrade were easily achieved. The subcontractor indicated that the MKD was very easy to work with. The setting time was very similar to that of the soil-hydrated lime mixture. The subcontractor had no trouble cutting the treated subgrade to grade elevation 48 hours after incorporating the MKD into the subgrade. After cutting the subgrade to proper elevation, the bituminous curing seal was placed and the
experimental section blocked to traffic.

Construction Evaluations

Field evaluations of the AFBC modified soil subgrades consisted of performing moisture density tests, in situ CBR tests, and unconfined triaxial compression tests. Moisture-density field tests were used to learn if field compaction specifications were met. Field CBR tests and unconfined compression tests were used to assess the bearing strengths of the treated subgrade.

As shown in Figure 94, relative compaction, or the ratio of dry density obtained from field tests to dry density obtained from laboratory tests, was equal to or greater than 95 percent at various stations along the lengths of the two AFBC spent lime subgrade sections and the untreated subgrade. Hence, compaction of those sections met the dry density specification. Maximum dry densities obtained from standard laboratory compaction tests (ASTM D 698-78) were adjusted based on the percent material retained on the No. 4 sieve according to a nomograph (Kentucky Methods, 1987) when calculating the relative compactions. Adjustments were made in a few cases when the oversized material exceeded about 30 percent. Normally, adjustments of the field dry densities were not required. Values of relative compaction for the AFBC sections averaged 98.1 percent. For the entire 9.6 kilometer (six-mile) roadway subgrade at some 85 locations, relative compaction averaged 98.2 percent. Average relative compaction of the untreated section averaged 99 percent.

Deviations of field moisture contents from the optimum moisture contents obtained from standard laboratory compaction (ASTM D 698-78) are shown in Figure 95. The moisture content of the first section averaged about 1.4 percent greater than laboratory optimum moisture content and the subgrade was compacted "wet" of optimum moisture content. Field moisture contents at some 17 of 24 locations of the second section (70
percent) were smaller than laboratory optimum moisture content and, generally averaged about 1.1 percent lower than optimum moisture content. Moisture contents of the first section met specifications. Moisture contents at two-thirds of the locations selected for testing on the second section met specifications. Generally, from a practical viewpoint, both density and moisture content requirements were met at the two AFBC subgrade sections. Relative compaction of the untreated subgrade section was equal to or greater than 95 percent and averaged 99 percent. These soils generally were compacted on the "dry-side" of optimum moisture content.

**Field CBR Tests During Construction**

Before paving, in situ CBR tests were performed on the modified AFBC-soil subgrades to determine the immediate bearing strength improvements obtained during construction. A comparison of values of field CBR of the untreated and AFBC spent lime-treated subgrades is shown in Figure 96. In both cases, the values of CBR are exceptionally large. The high values of the untreated subgrade occurred in the hottest portion of summer (August) in Kentucky and the dried subgrade soils had low water contents. However, the average field CBR of the treated subgrade was about 41
while the average CBR of the subgrade before treatment was about 30. Therefore, the treatment increased the bearing strength.

Post-construction Evaluation

Since construction of the base courses of the asphalt pavement in the fall of 1987, monitoring of the experimental sections and control section has continued for about six years. The initial post-construction analysis involved a re-examination of the expansive characteristics of the AFBC spent lime-soil mixtures. Other components of the post-construction evaluation included performing in situ CBR tests, obtaining undisturbed samples of the treated subgrade for laboratory testing, performing optical surveys of points located on the pavement to observe swelling characteristics, and making visual inspections of the pavement. Moisture content, dry density, classification, and unconfined compressive strength tests were performed on thin-walled tube samples.

Expansive Characteristics of AFBC Spent Lime Subgrades

Approximately two months after construction, severe heave, or differential swelling, occurred, as shown previously in Figure 85. The swell, or humps, occurred almost immediately after rainy periods. To investigate the pavement disturbance, a trench was excavated in an area (station 279+80) where considerable heave had occurred. In situ CBR tests were done on the exposed subgrade and both undisturbed and disturbed samples were obtained.

To examine the long-term swelling potential of compacted AFBC spent lime-soil mixtures, six long-term, swell tests were performed on specimens compacted in CBR molds and mixed at AFBC percentages of 7, 15, and 30. Additionally, a long-term swell test was performed on a specimen of the untreated soil from stockpile one. Results of these tests are summarized in Table 13. Periods of swell monitoring ranged from some 48 days to 186 days. Generally, the tests were allowed to swell until primary swell was completed. Monitoring of swell continued after that time until a sufficient period had elapsed to establish the pattern of secondary swell. One (7-1FB) of the six AFBC-soil specimens consisted of material obtained from the field mixing operation. Another specimen (7-1FT) was composed of material from the trench excavated at station 279+80. Swell of those two specimens covered a period of some 48 days. Total amounts of swell of those two specimens were only 2.9 and 0.8 percent, respectively. This indicated that a large portion of the chemical reactions may have already occurred at the time the field specimens had been obtained. Amounts of swell measured in the field were much larger than the amounts of swell observed from the two laboratory tests.
Table 13. CBR and swell magnitudes of remolded soil-AFBC specimens

<table>
<thead>
<tr>
<th>Sample Number</th>
<th>Type and (%) of Chemical Admixture</th>
<th>Soaked Penetration CBR Value</th>
<th>At Compaction</th>
<th>After Test</th>
<th>Total Swell (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>2.5 cm (0.1 inch) 1.3 cm (0.5 inch)</td>
<td>Dry Density kNm$^3$ pcf</td>
<td>Moisture Content (%)</td>
<td>Dry Density kNm$^3$ pcf</td>
</tr>
<tr>
<td>7-1FB</td>
<td>AFBC (7%) Bag</td>
<td>11.3 8.3</td>
<td>15.64 (99.6)</td>
<td>23.5</td>
<td>28.8</td>
</tr>
<tr>
<td>7-1FT</td>
<td>AFBC (7%)</td>
<td>57.7 39.5</td>
<td>14.70 (93.6)</td>
<td>25.9</td>
<td>31.2</td>
</tr>
<tr>
<td>15-LAB1</td>
<td>AFBC (15%)</td>
<td>16.38 (104.0)</td>
<td>12.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15-LAB3</td>
<td>AFBC (15%)</td>
<td>15.37 (97.9)</td>
<td>13.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>30-LAB1</td>
<td>AFBC (30%)</td>
<td>14.82 (94.4)</td>
<td>15.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>30-LAB3</td>
<td>AFBC (30%)</td>
<td>1.7</td>
<td>14.29 (91.0)</td>
<td>11.5</td>
<td>12.53 (79.8)</td>
</tr>
</tbody>
</table>

Four additional CBR swell tests were performed on remolded specimens of soils from the stockpiles at stations 273 and 574. AFBC spent lime percentages used in these tests were 15 and 30 percent. These large percentages were used because it was originally believed that some heaved areas may have received more than the specified 7 percent of AFBC spent lime.

A typical plot of vertical swell (in percent) as a function of the logarithm of time for an AFBC lime-soil specimen (15-LAB3), which contained 15 percent of the AFBC spent lime, is shown in Figure 97 and compared to swell of a remolded clayey specimen of soil from stockpile three. The latter test was

Figure 97. Plot of vertical swell as a function of the logarithm of time for an AFBC lime-soil specimen.
performed in a consolidometer. Swell periods for the AFBC spent lime specimen and the untreated clayey specimen were 126 days and four days, respectively. Total primary swell of the AFBC specimen was 24 percent while the total primary swell of the untreated clayey specimen was only about 4 percent. If some secondary swell is included, the swell amounts are 26.3 and 4.6 percent, respectively. As shown in Table 13, total magnitudes of swell of the AFBC specimens, 15-LAB1, 30-LAB1, and 30-LAB3, were of the same order as specimen 15-LAB3. Swell values ranged from 24.3 to 26.5 percent. The swelling pressure was of such an extreme nature that in two of the CBR swelling tests the clamps that fasten the bottom of the mold to the CBR mold base were completely sheared. This occurred near the end of those two tests. In both cases, secondary swell was occurring when the clamps were sheared.

As a means of estimating the time required for completion of primary swell of the AFBC spent lime-soil mixture in the field, the swell curve in Figure 97 (specimen 15-LAB3) was analyzed to find a coefficient of primary swell (opposite of a coefficient of consolidation) based on Terzaghi's theory of consolidation (1943.) Based on the AFBC curve in Figure 97 and the equation,

$$C_{ps}=\frac{TH^2}{t_{50}}$$

(52)

where

- $C_{ps}$ = coefficient of primary swell (in$^2$/hour, or cm$^2$/hour)
- $T$ = dimensionless time parameter (Terzaghi, 1943)
- $H$ = length of drainage path of water into the absorbing layer, (at $t_{50}$)
- $t_{50}$ = time at which 50 percent of primary swell (hours) has occurred (T=0.198,)

the coefficient, $C_{ps}$, may be determined. If water is absorbed from both ends of the specimen, which is the case in the CBR mold and the original height of the specimen is 10.2 cm (4.0 in.), then $H$, at 50 percent of swell (12 percent), is equal to 6.3 cm (2.48 in.), which is $H=((4+0.24*4)/2)$in. Here, the coefficient of primary swell is

$$C_{ps}=(0.198)(2.48)^2\text{inch}^2/195 \text{hrs}=0.0062 \text{in}^2/\text{hr}=0.0400 \text{cm}^2/\text{hr}. \quad (53)$$

The time for completion of 95 percent of primary swell in the field was approximated (assuming that the 30.48-cm (12-in.) layer absorbed water from both top and bottom
and rearranging terms of Equation 52) as follows:

\[ t_{95} = \frac{(1.129)(7.368 \text{ in})^2}{0.0062 \text{ in}^2/\text{hr}} \times \frac{1 \text{ day}}{24 \text{ hrs}} = 413 \text{ days.} \quad (54) \]

The first sections of the AFBC spent lime subgrade, station 260 to 317+50, were constructed on August 8, 1987. Based on the above calculation, it was predicted that the completion of primary swell (Hopkins et al., 1988) would occur approximately at some date between October of 1988 and February of 1989. Similar calculations were made for the untreated specimen, Figure 97, or

\[ C_{ps} = (0.198)(0.32)^2 \text{ in}^2/0.3 \text{ hr} = 0.065 \text{ in}^2/\text{hr} = 0.4194 \text{ cm}^2/\text{hr}. \quad (55) \]

If a 30.48-cm (12-in.) layer of untreated layer is considered, then the time for completion is estimated to be

\[ t_{95} = \frac{(1.129)(6.5 \text{ in})^2}{0.065 \text{ in}^2/\text{hr}} \times \frac{1 \text{ day}}{24 \text{ hrs}} = 31 \text{ days} \quad (56) \]

However the primary swell of the clayey specimen was only about 4 percent, or only about 1.27 cm (0.5 inches) of swell for a 30.48-cm layer (12-in.). This swell occurred rapidly.

Another problem associated with the AFBC spent lime-soil mixture is illustrated in Figure 97. The treated soil exhibits secondary swell, which occurs after the completion of the primary swell. The relationship between secondary swell and the logarithmic of time is linear. Consequently, a coefficient of secondary swell, \( C_{ss} \), may be estimated using the following equation:

\[ H_{ss} = C_{ss} H \log(t_p/t_{100}) \quad (57) \]

where

\[ H_{ss} = \text{amount of secondary swell for a given time period (inches,)} \]

\[ H = \text{thickness of AFBC spent lime-soil mixture,} \]

\[ t_p = \text{selected time after completion of primary swell (days,)} \]

\[ t_{100} = \text{time of completion of primary swell (days).} \]
From the curve (linear portion in Figure 97,) the coefficient of secondary swell was estimated to be 0.062. From 413 days after the predicted time, $t_{ps}$, of completion of primary swell to 27.4 years (10,000 days), $t_p$, the estimated secondary swell of the pavement may be calculated as follows:

$$H_{ss} = (0.062)(12 \text{ in.}) \log(10,000/413)$$

$$H_{ss} = 1.02 \text{ in.} = 2.59 \text{ cm.}$$

This estimated swell indicated that secondary swell of the pavement after completion of primary swell may be a problem in the future. However, the problem may be controllable. If future differential heaving of the pavement is small, then an overlay may be feasible in mitigating the problem. These calculations indicated that the AFBC subgrades could be left in place and their removal could be avoided.

**In Situ Subgrade Strengths**

After non-uniform swelling of the pavement surface occurred in October of 1987, the asphaltic pavement was cored to the top of the AFBC spent lime-soil subgrade at two locations identified as a "humped area" and a "non-humped area," (station 279+40.) The in situ CBR of the humped area was 38 while the CBR near the edge of the lane was 40. Each had corresponding moisture contents of about 16 percent. During April of 1988, in situ CBR tests were conducted during the milling operation of the two AFBC sections. This operation provided distinctive locations of heaved and non-heaved areas. Figure 98. Comparison of in-situ values of CBR obtained during the six-year period.
area, a field CBR of 13 was obtained, while in a non-heaved area the CBR was 37. Moisture contents were 36 and 27 percent, respectively. CBR strengths in a "humped" area were not, necessarily, lower than the strengths of "non-humped" areas. However, field moisture contents were much higher than optimum moisture contents observed in laboratory tests.

Additional in situ CBR tests were performed through core holes on top of the two AFBC-treated subgrades in March of 1989, 1991, and 1993. In situ values of CBR obtained during the six-year period are compared in Figure 98. Although an occasional small value was observed, the in situ CBR generally exceeded 10 during the six-year period. Moreover, the in situ CBR strengths of the AFBC sections were several times greater than the CBR strengths of the untreated subgrade in the control section and the untreated subgrade located below the treated AFBC layers. In the latter case, the bore holes were advanced through the treated layer and in situ CBR tests were performed on top of the untreated subgrade. In Figure 99, the percentile test value is shown as a function of the in situ CBR strength of the AFBC treated subgrades, the laboratory CBR strengths of the corridor soils, and the in situ CBR strengths of the untreated soils of the control section and the untreated subgrade below the AFBC treated layers. In constructing the CBR-percentile test value curve of the treated layer, CBR values observed seven days after construction were excluded. At the 90th and 50th percentile test value, the CBR strengths are about five times greater than the field CBR strengths of the untreated subgrade soils. Unconfined compressive strengths of AFBC cored specimens obtained in 1989 and 1991 ranged from 75 to 269 kPa (10.9 to 39 psi). However, these strengths were suspect because the samples tended to crumble or come apart along horizontal
Moisture contents of both the untreated and treated layers increased with increasing time. Moisture contents of the untreated subgrade soils as measured during construction and at various times over the six-year study period were shown previously as a function of percentile test values in Figure 77. At the 90th and 50th percentile test values, the moisture contents in October of 1987 were about seven and 12 percent, respectively. In March of 1993, the moisture contents had increased to 16.5 and 20 percent at the 50th and 90th percentile test values. As shown in Figure 100, the observed moisture contents of the AFBC subgrade sections ranged from about nine to 20 percent during construction. However, subsequent measurements of moisture contents ranged from about 23 to 39 percent. Although the moisture contents of the AFBC layers increased, a significant decrease in CBR strengths has not occurred as shown in Figure 98. Field CBR strengths ranged from 34 to 53 seven days after construction in August of 1987. From October of 1987 to March 1993, the field CBR values range from about nine to 54, if the one value of two is excluded. However, the increase in moisture contents has apparently caused significant decreases in CBR strengths of the untreated subgrade soils as shown previously in Figure 78.

Index Test Results

Results of index tests performed on subgrade samples obtained in March of 1991 from the treated layer and the untreated layer found below the treated layer and the untreated subgrade of the control section are compared in Table 14. While the untreated soils were classified as CL, the two samples of treated material were classified as SM. The most noticeable change in the clays, after treatment, occurred in the percentages of particles passing the 40 and 200 sieves. The percentages of the natural clays passing the 40 and 200 sieves were about 70 and 60, respectively. After treatment, the corresponding percentages were approximately 59 and 39.
Hopkins, Beckham, and Hunsucker—Modification of Highway Soil Subgrades

Table 14. Soil classifications of thin-walled tube specimens (March 1991).

<table>
<thead>
<tr>
<th>Station Number</th>
<th>Liquid Limit (%)</th>
<th>Plasticity Index (%)</th>
<th>Specific Gravity</th>
<th>Grain-Size Analysis</th>
<th>Percent Finer Than</th>
<th>Classification</th>
<th>Type and % of Admix.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.91 mm (94% in.)</td>
<td>0.003 cm (30%)</td>
<td>No. 4  No. 10</td>
<td>No. 40</td>
</tr>
<tr>
<td>302+10 ft</td>
<td>48</td>
<td>8</td>
<td>2.75</td>
<td>100.0</td>
<td>98.9</td>
<td>93.1</td>
<td>86.9</td>
</tr>
<tr>
<td>522+75 ft</td>
<td>49</td>
<td>11</td>
<td>2.74</td>
<td>100.0</td>
<td>98.0</td>
<td>93.4</td>
<td>86.2</td>
</tr>
<tr>
<td>305+20 ft</td>
<td>38</td>
<td>14</td>
<td>2.74</td>
<td>100.0</td>
<td>96.4</td>
<td>89.7</td>
<td>79.8</td>
</tr>
<tr>
<td>622+50 ft</td>
<td>39</td>
<td>17</td>
<td>2.76</td>
<td>99.6</td>
<td>98.6</td>
<td>94.5</td>
<td>74.2</td>
</tr>
</tbody>
</table>

Measurements of Pavement Swell

Placement of the bituminous surface course in all sections was delayed after differential pavement heaving was observed in the two subgrade sections treated with the AFBC spent lime. Elevations of the uppermost base layer were monitored periodically at arbitrary locations selected within each chemically modified soil subgrade section to observe changes in the pavement surface profile. Initial measurements were obtained in early October of 1987 after noticeable differential heaving occurred. Optical survey points were also established on the pavement surfaces on subgrades chemically treated with hydrated lime, cement, and multicone kiln dust. Typical pavement profiles (AFBC subgrades) obtained at station 549 on four different occasions between October of 1987 and August of 1988 are shown in Figure 101. The maximum swell observed during this period was about 5.1 cm (2 in.) at that location.

Many profiles were obtained throughout the two AFBC sections and sections containing subgrades treated with hydrated lime, cement, and multicone kiln dust.
Maximum observed values of swell recorded at 25 different stations, between October of 1987 and August of 1988, are shown and compared in Figure 102. Maximum values of swell ranged from about 1.8 cm to 8.9 cm (0.7 to 3.5 in.) in the AFBC sections. In sections where the subgrades had been treated with cement, hydrated lime and multicone kiln dust, the maximum values of swelling were only 0.1 to 1.25 cm (0.04 to 0.5 in.). Heaving was not observed in the sections containing subgrades treated with hydrated lime, cement, and multicone kiln dust.

In August of 1988, the pavements within the AFBC sections were milled to remove portions of the heaved asphaltic surfaces. Subsequently, an asphaltic surface course was placed on the asphaltic base courses of the AFBC sections and the base courses of the other sections of the project. Consequently, all surface points had to be reestablished in August of 1988. Subsequently, monitoring continued at all locations. Typical profiles, obtained between September of 1988 and March of 1993, at station 549 in one AFBC section are shown in Figure 103. These profiles were obtained after the placement of the surface course. Maximum swell measured during that period was about 0.51 cm (0.2 in.). Although swelling has continued after placement of the surface course, the rate of swelling has decreased significantly when compared to swelling before milling. This condition is illustrated in Figure 104. The portion of the field curve before milling represents primary swell. The rate of swelling was very rapid. Values of swell used to construct the curve in Figure 104 were obtained from Figure 101 (points one through 5) and Figure 103 (points six through 12). Using
these values and the dates of the survey, the curve in Figure 104 was developed by graphing the values of swell as a function of the logarithm of time. The shape of the field curve is much similar to the shape of the laboratory curve. The portion of the field curve after milling is linear on the semi-logarithmic graph. Therefore, the pavement swell that has occurred after placement of the asphalt surface course is secondary swell. Projecting the linear portion of the secondary swell curve, only about 0.64 cm (0.25 in.) of swell should occur between a time of two thousand days after October 1987 to a time of 27.4 years (approximately March 2015) after placement of the AFBC subgrades. Based on this projection, the swell of the pavement within the AFBC sections should be nominal in the coming years. However, in one isolated area of one AFBC section, substantial, non-uniform, swelling occurred and milling was required to eliminate surface humps. There also was a prominent crack within the milled area. The predicted theoretical swell-logarithm-of-time curve is shown in Figure 104. The predicted curve showed that the subgrades could be left in place and that magnitudes of swell would decrease with increasing time. Field measurements generally confirmed this estimate.

**Geochemical Analysis**

Thin-walled tube samples obtained in March 1993 were analyzed using x-ray diffraction (XRD) and Scanning Electron Microscope (SEM) methods. The results, Figures 105 and 106, showed the presence of ettringite, CaAl₂(SO₄)₃(OH)₁₂·26H₂O, thaumisite, CaSi(OH)₆(CO₃)·12H₂O, and gypsum, CaSO₄·2H₂O. All three minerals were present throughout the 30.5-cm (12-inch) depth interval of the subgrade samples. Ettringite was concentrated in the lower portions of the subgrade, although it was present throughout. Thaumisite and gypsum had approximately equal distribution throughout the depth. Exact amounts or percentages of these minerals could not be quantified. These minerals were not present in the original AFBC spent lime residue (Figure 84). Sulfate and portlandite, present in the spent lime can react with aluminum silicates present in the native soil in the presence of water to form ettringite and thaumisite. Gypsum can be formed by the reaction of anhydrite, CaSO₄, present in the spent lime with water.
Figure 105. X-ray diffraction of AFBC-soil sample from the KY 11 subgrade.
Ettringite is the main contributor to the compressive strength development of FGD byproduct mixtures. Excessive ettringite formation causes lateral and vertical swelling. (Graham et al., 1993). Ettringite, thaumisite, and gypsum formed after modification of the subgrade with AFBC and contributed to the swell of the AFBC modified subgrade.

**Pavement Rutting**

Many pavement rutting depth measurements were obtained in March of 1993, or some five years after the asphalt surface was placed. Typical rutting depths (left wheel path of the north bound lane) of the pavement surfaces of the AFBC sections are compared in Figure 80 (Page 83) to rutting depths of the pavement surfaces on the untreated subgrade and the other treated subgrades. Pavement rutting depths are generally equal to or below a value of about 0.33 cm (0.13 in.), except at a few isolated spots in the AFBC sections and the untreated section.

Figure 106. *Views of ettringite in AFBC-soil mixture from KY 11 subgrade.*
PERFORMANCES OF SOIL-HYDRATED LIME SUBGRADES

Hydrated lime has been used extensively for many years to treat expansive clays. According to McCallister and Petry (1990), hydrated lime has been used in more than 40 states as a primary stabilization technique in roads, runways, building and parking lots. Over the past 30 years, many studies have been performed to determine the durability, or long-term effectiveness, of lime-treated soils. Pavement conditions of roadways constructed with treated soils and aggregates throughout California were reviewed by Alexander (1976). This review with a detailed investigation of selected roadways was used to evaluate the performance of lime-treated materials incorporated into the roadway. Most of the roadways where lime-treated subbases were used were at least ten years old. Generally, most of the 14 sections reviewed were in good condition. Four sections were rated "fair to excellent" while two sections were rated "poor." Two sections were rated "extremely poor." Most of the observed distress was caused in part by construction variations, aggregate base deficiencies, and excessive water at time of construction, or after construction. Alexander concluded that lime treated material can be used effectively as a substitute for aggregate base. A comparison of the required thickness of the structural sections containing lime-treated aggregate base showed that all three loads met or exceeded design criteria. All but one road was found in "good to fair" condition after ten years. Just one section was evaluated. It showed that most of the distress was caused by a soil that was not responsive to lime treatment. Alexander concluded that low quantity aggregate containing fines that react with lime can be improved.

Alexander also focused on lime-treated bases. Twenty-two of thirty-five roads were judged to be in "good to fair" condition. Roads that did poorly were observed to be in a "poor or extremely poor" condition. He observed that those roads did poorly after only a few years after construction. Alexander showed that the poorly-performing roads were due to structural deficiencies -- that is, an insufficient thickness of an asphaltic pavement existed on those roads. Major factors leading to poor performance was attributed to attempts to treat soils that did not respond to lime treatment, poor lime distribution (or mixing), and deficiencies in thickness of treated base or asphaltic concrete. Alexander also noted that soils containing sulfates and organic materials are unsuitable for treatment with hydrated lime, cement, and fly ash. Initially, the soils may show good stabilization when mixed with lime. However, the quality deteriorates with increasing time.

Alexander's study strongly suggested that repeated spraying of water on the surface of a lime-treated soil may be detrimental to the long-term stability of the roadway section. Although aggregate is often placed over the lime-treated subgrade, Alexander recommended that the lime-treated subgrade should be sealed with an asphaltic emulsion curing seal when possible after mixing and compacting. Traffic should not
be permitted on the finished surface before sealing. Evidence suggested that repeated watering may leach lime from the uppermost portion of the treated layer and create a thin soft layer.

Alexander also noted that shrinkage cracks were more apparent in lime-treated aggregate than lime-treated soil. Shrinkage cracking was more evident in lime-treated soils having low plasticity indices. The study recommended that the maximum lift thickness of the lime-treated subgrade should be 30.5 cm (12 in.) and the minimum lift thickness should be 10.16 cm (4 in.).

Several other studies have been done that addresses the issues of lime stabilization longevity and durability. Eades and Grim (1960) concluded that the transformations that occur when clays are mixed with lime are permanent. Approximately 2,006,712 square meters (2,400,000 square yards) of runways and taxiways of the Dallas-Fort Worth International Airport were constructed on lime-treated subgrades (McAllister and Petry; 1990). Treated soil subgrades below runways and taxiways were 22.9 cm (9 in.) in thickness while treated subgrades below terminal aprons were 45.7 cm (18 in.) in thickness. A minimum of 6 percent was needed to treat the subgrade clays. A seal coat was applied to the treated subgrade. According to McAllister and Petry, maintenance and engineering personnel of that airport note that no major maintenance has been required in 15 years of service.

The long-term effectiveness of lime stabilized roads (Kelley 1977; cf McAllister and Petry, 1990) at several military posts found throughout Arkansas, Louisiana, New Mexico, Oklahoma, and Texas was conducted in 1977. After more than 25 years, the lime stabilized pavements were performing satisfactorily. Many soils stabilized at the military posts were highly expansive; plasticity indices ranged from 12 to 50 percent. A typical section consisted of 15.2 cm (6 in.) of a lime stabilized layer and a thin asphaltic cement overlay. About three to 8 percent of hydrated lime was used in those projects. Additionally, the strength of the lime-treated soils increased significantly with increasing time.

McAllister and Petry (1990) studied the long-term effects of leaching of hydrated lime-treated expansive clays. Their research showed that changes occurred in lime-treated soils after continuous leaching. Generally, as the leaching period increased, the change increased. The least changes in the treated soils occurred when the lime contents were above the lime stabilization optimum ("LSO" equal to six to 7 percent). The greatest changes occurred in treated soils containing lime content equal to three to 4 percent. Therefore, change was highly dependent on the lime content of the soil. They also noted that compaction moisture content was an important factor in controlling leaching effects. Specimens compacted “wet” of optimum moisture content exhibited the largest increase in plasticity and swell after leaching.
Hydrated lime has been used routinely in Kentucky since 1987 as a chemical admixture to improve bearing strengths of highway subgrades (Hopkins and Allen 1988; Hopkins, et al., 1988). For example, about one-third of some 137 kilometers (85 miles) of the clayey subgrades of the Alexandria-Ashland Highway was stabilized using hydrated lime, Figure 107. Generally, a vast majority of soils in Kentucky are potentially treatable using hydrated lime. According to one criterion published by Terrell, et al. (1979), soils with plasticity indices greater than or equal to about 10 percent are suitable for treatment with hydrated lime. Based on a search of the Kentucky Geotechnical Data Bank (Pfalzer et al., 1994), which contains index properties of soil samples collected from many locations throughout Kentucky, about 74 percent of Kentucky soils are, at least statistically, treatable with hydrated lime, as shown in Figure 108. Based on one published criterion (Epps and Little, FHWA, 1994), a soil containing a clay fraction (CF) equal to or greater than 10 percent is usually responsive to lime treatment. Here, some 87 percent of Kentucky soils are potentially responsive to lime treatment. Although each specific subgrade must be studied, the data in Figure 108 indicate that large portions of soil in Kentucky are

![Figure 107. Subgrade stabilization techniques used on the Alexander-Ashland Highway in Northern Kentucky.](image)

![Figure 108. Statistical percentage of soils in Kentucky that are potentially responsive to treatment with hydrated lime.](image)
potentially responsive to treatment with hydrated lime. Consequently, hydrated lime can be an effective method in subgrade stabilization in Kentucky.

To determine the durability and longevity of hydrated lime-treated subgrades in Kentucky, two highway subgrade sections were selected for detailed study over a period of about seven years. A summary of the findings of geotechnical field and laboratory studies performed at those sections and general performances of flexible pavements on the soil-hydrated lime subgrades are described below.

**Testing Procedures**

Geotechnical laboratory tests were performed on retrieved samples to determine index properties of the hydrated lime-treated soils and untreated soils. Index tests consist of liquid limits (ASTM D 4318-84) and plastic limits (ASTM D 4318-84), particle-size analysis (ASTM D 422-63), specific gravity (ASTM D 854-83), and moisture content (ASTM D 2216-80). The materials were classified according to the United Soil Classification System (ASTM D 2487-85) and the AASHTO Classified System.

Geotechnical field studies consisted of performing in-situ bearing ratio tests on the top of the hydrated lime-soil subgrades and the top of the untreated soil subgrade found below the treated layer. These tests were performed through cored boreholes. Procedures of ASTM D 4429-84 were followed in performing the in-situ bearing ratio tests. Core specimens of the subgrades were obtained at each site. Pavement performances were generally evaluated from visual inspections and rutting measurements.

**Case Studies**

*Sections of the Alexandria-Ashland Highway*

Several subgrade sections of the Alexandria-Ashland (AA) Highway were treated with hydrated lime. These included section B1, B2, B3, B4, C1, C2, C3, 6A, 6, 12, 17, 18, and 19. Each of these sections was treated to a depth of about 15.2 cm (6 in.) with about 5-7 percent of hydrated lime. Section 19 was selected for detailed study. This treated subgrade section was monitored over a period of about seven years. Additionally, in-situ CBR tests were performed on the treated subgrade of Section 12 shortly after construction and after the exposed subgrade has passed through a winter season. For comparative purposes, an in-depth study was made of sections 13 and 14. Subgrades of these two sections were not treated. These two sections failed during construction and required bituminous overlays, which ranged from 2.54 cm (1 in.) to 12.7 cm (5 in.) in thickness.
Except Section 19, the untreated roadway soils of the above sections of the AA Highway are residual soils of the Kope and Fairview Shale Formations -- clayey shale formations with some interbedded limestone. According to results tabulated on the soil and profile sheets, the soils of these sections typically are classified as A-7-6(10-35) and A-6(6-33), based on the AASHTO Soil Classification System, and CL and CH, based on the Unified Soil Classification System. Liquid Limits range from 34 to 61 percent and average 42 percent. Plasticity indices range from 13 percent to 38 percent and average 20 percent. Clay fraction, or the percent finer than 0.002 mm-size particles, ranges from 17 to 54 percent and averages 32 percent.

The subgrade of Section 19 was constructed with residual soil of the Crab Orchard Shale Formation. This formation is a clayey shale. Typical index properties of the residual soils of Section 19 were shown previously in Tables 2 and 3. These typically are classified as CL and CH. Both the Kope and Crab Orchard shales break down into fat and highly plastic soils when exposed to water. Average, soaked CBR values of the roadway soils of the above sections are compared to the CBR values occurring at the 90th percentile test value in Figure 109. Average CBR values for the subgrade sections range from 3.5 to 6.6, while the CBR value at the 90th percentile value ranges from 1.7 to 2.6 -- very low bearing strengths. Paving of Section 12 was not performed during the construction season. As a result, the hydrated lime-treated subgrade was left exposed to winter conditions. This situation presented an opportunity to check the strength of the treated subgrade after passing through the winter. Results of in-situ CBR tests performed on top of the treated subgrade after the winter season are shown in Figure 110 and compared to soaked laboratory CBR values in Figure 111. As shown in the upper portion of Figure 111, approximately 50 percent of the values of laboratory CBR of the untreated soils ranged from 1.7 to 3. About 50 percent of the values of the untreated soils ranged from 3 to 5.9. Maximum and minimum values ranged from 1.7 to 5.9 and averaged 3.5.
Values of in-situ CBR performed on the hydrated lime-treated subgrade exposed to winter ranged from 19 to 61 (CBR values of approximately 60 percent of the data set ranged from 40 to 61). Values of CBR of 40 percent of the data ranged from 19 to 40. A comparison of unconfined compressive strength before and after the winter season of the treated clays is shown in Figure 112. Generally, the strengths were larger after the winter season. Therefore, the treated subgrade maintained high strength after freeze-thaw cycles of winter.

In situ CBR tests were performed on AA Section 19 over a period of seven years to examine the long-term durability of the hydrated lime-treated subgrade. Tests were performed in 1991 and 1994, or three years and seven years, respectively after construction. The in-situ CBR tests were performed through boreholes in the asphalt pavement on top of the treated subgrade. Borings were also advanced through the treated subgrade and in-situ CBR tests, were performed on top of the untreated subgrade. Results of in-situ CBR tests of the untreated subgrade are shown in Figure 113. In-situ CBR values of the untreated subgrade found below the treated subgrade ranged from only 1.9 to 3.7 in 1991, while in 1994 the values ranged from 1.2 to 2.5. In 1991, the in-situ CBR bearing strengths of the treated subgrade layer ranged from 40 to 106. In 1994, some seven years after construction, the in-situ values ranged from 48 to 94. Therefore, the strengths of the hydrated lime-treated subgrade were exceptionally large.
Average rutting depths of east bound (EB) and west bound (WB) wheel tracks -- inner (I) and outer (O) wheel tracks -- are shown in Figure 114. Numbers of rutting measurements obtained for sections 12, 13, 14, 19, and 20 were 172, 140, 144, 142, and 160, respectively. The crosshatched bars represent the average rutting depth of all measurements for each section. Pavement thicknesses of sections 12, 13, 14, 19, and 20 are shown in Figure 115. Rutting of all sections is very small and ranges from about 0.26 cm to 0.58 cm. Average rutting depths of Sections 13 and 14 were 0.38 cm (0.15 in.) and 0.36 cm (0.14 in.), respectively. These rutting measurements were obtained on the overlays, which were constructed after these two sections failed. Average rutting depths of Sections 19 and 20 were 0.51 cm (0.20 in.) and 0.48 cm (0.19 in.), respectively. Average rutting depth of Section 12 was less than Sections 13 and 14, as shown in Figure 114. The subgrades of AA-13 and 14 were untreated. Total thicknesses of these two pavements ranges from 38.2 to 47.1 cm (including overlay thickness). Thickness of AA-12 was 36.9 cm (14.5 in.) and when the hydrated lime-treated subgrade is included, the thickness is 42.1 cm (16.6 in.). Therefore, Section 12 has performed as well as Sections 13 and 14. However, asphalt thicknesses of Sections 13 and 14 are some 1.3 to 10.2 cm greater than the asphalt thickness of Section 12. Moreover, Section 12 contained a very lean, asphalt
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Although the average rutting depth of Section 12 is less than the rutting depth of Section 19, the treated subgrade of Section 19 is some 5 to 7.2 cm (2 to 2.8 in.)—measured in the field—less than the treated subgrade of Section 12. Moreover, Section 19 has been in service about two years longer than the other sections. Generally, the rock subgrade of Section 20 has apparently performed as well as the other sections shown in Figures 114 and 115.

KY 11

This hydrated lime-treated subgrade is found on KY 11 near Beatyville, Kentucky and was part of an experimental scheme where over chemical admixtures were used to treat the subgrades of this reconstructed route. An overall schematic of the experimental scheme was shown previously in Figure 85. The hydrated lime-treated section is about 1.6 kilometers (one mile) in length. Geotechnical properties of the untreated soils are shown in the top portion of Table 12. After treatment with lime, the plasticity indexes of the corridor soils decreased form 12-15 to 10 percent. The percent passing a No. 200 U.S. Standard sieve decreased from 70-74 percent to 54 percent. As shown in Figure 116, the optimum percent of hydrated was about 6 percent. Soil classification changed from A-6(10)-A-6(14) to A-5(4) after treatment. Soaked,
During the six-year period of measurement, the pavement on the hydrated lime-treated subgrade has performed very well. As shown previously in Figure 80, rutting depths have only ranged from 0.2 cm (0.08 in.) to less than 0.4 cm (0.16 in.). These rutting depths are less than the depths observed in the untreated subgrade section of KY 11. No overlay has been placed at this section after almost eight years.

Laboratory CBR strengths of the corridor soils were 7.1 and 2.9, respectively, at the 50th and 90th percentile test value, as shown previously in Figure 90. In-situ CBR strengths of the hydrated lime-treated subgrades performed at various times over about a seven-year period are shown in Figure 117. In 1988, values were 30 and 41, while in 1989 values ranged from 16 to 26. In 1991 and 1993, CBR strengths ranged from 37 to values greater than 100. These data indicate that the treated subgrade gained in strength with increasing time. However, as shown in Figure 118, in-situ CBR strengths of the untreated subgrade in the control section and the untreated subgrade located below the hydrated lime-treated layer, have decreased with increasing time. In March of 1989 and 1991, the CBR strengths ranged from 3 to 14. In March of 1991, the in-situ CBR strengths ranged from one to six.
Along any highway corridor, before construction, a variety of soil horizons and soil types are normally encountered and a wide range of bearing strengths may exist when the different types of soils are used to construct pavement subgrades. To avoid bearing capacity failures during construction of the subgrade and placement of the pavement layers, a certain minimum subgrade strength (Hopkins, 1994 a) must exist to sustain construction traffic. Therefore, the design strength selected for pavement analysis should consider the issue of pavement construction. The method of selecting the design strength is complicated when different subgrade strengths exist along the route to be paved. Additionally, when the design analysis is based on a selected laboratory strength, the question arises whether the laboratory strength represents the long-term field strengths existing after paving.

When the actual subgrade strength is lower than the minimum strength required to sustain construction traffic, stabilization of subgrade soils with chemical admixtures, such as cement or hydrated lime, or by other means, may be necessary. When chemical stabilization is used, then a design strength of the treated layer and a design strength of the untreated layer found below the treated layer must be selected for the design analysis. If the improved strength created by chemical stabilization is ignored, then the pavement thickness obtained from the design analysis may be overconservative. Moreover, the long-term strength gain may be much larger than the subgrade strength existing at the time of construction. Consequently, the issue that arises is whether the stabilized layer should be treated merely as a working platform, with no allowance made in the pavement design analysis for the net strength gain obtained from chemical stabilization, or whether the stabilized layer should be considered an integral part of the pavement structure with the total, or a portion, of the net strength gain considered in the analysis. To examine and analyze the different issues posed, a pavement bearing capacity model, formulated based on limit equilibrium, is used (Hopkins, 1991). The selection scheme uses an approach described by Yoder (1969).

**Selection of Untreated Design Strength**

Different philosophies exist concerning the method of selecting the subgrade design bearing ratio (or strength parameters from other types of tests). Some approaches include using

- the lowest value,
• an average value,
• statistical methods of estimating the average values, or
• a value based on a least-cost analysis.

When the lowest value of bearing ratios of a data set is selected, the pavement may be over designed. If the average value of the data set is selected, then approximately one half the pavement (of a selected route) may be over designed while one half may be under designed (Yoder and Witczak, 1975). Another approach embraces the normal distribution curve and reliability concepts. This concept involves upper and lower limits for the selected confidence interval.

Another approach, based on a least-cost design, has been proposed by Yoder. He presented a series of curves that relate percentile test values to soil variability (measured by the coefficient of variance of the test data set), traffic (EAL), and a unit cost of the pavement. Unit cost of maintaining a highway is expressed as a cost ratio (CR), or unit maintenance cost divided by the unit initial construction cost. When detailed information is lacking, Yoder suggests using the bearing ratio occurring at the 80th or 90th percentile test value to obtain an optimum design.

To test and compare the results of the different approaches, an analysis of soaked laboratory CBR values of two adjacent sections of a highway route in Kentucky were performed. Total length of the two sections was about 12.2 km (7.6 mi). The planned pavement structure consisted of 26.7 cm (10.5 in.) of asphaltic pavement and 10.2 cm (4 in.) of dense graded aggregate. The design CBR and equivalent single-axle load (ESAL) were, reportedly, five and four million, respectively. During construction the partially completed pavements failed at many locations along the two highway sections.

Soaked laboratory values of CBR of corridor soil samples obtained before construction is shown graphically in Figure 119. The lowest CBR value of the data set (56 tests) is 1.3 and the average value is 3.4. Based on the assumption that the CBR data set is normally distributed, lower and upper-bound CBR values for a 95 percent confidence interval are

![Figure 119. Soaked laboratory CBR values of corridor soils.](image-url)
2.9 and 4.1, respectively. Percentile test value (as proposed by Yoder, 1969) as a function of the soaked laboratory CBR is shown in Figure 120. Cost ratios for the two highway routes were not available. As suggested by Yoder, the value of CBR occurring at the 90th or 80th percentile test value may be used. At the 95th, 90th, and 80th percentile test values, the CBR values are 1.4, 1.8, and 2.1, respectively.

To compare the different CBR selection approaches, factors of safety of the planned pavement section were computed using the bearing capacity model described above. Surface and air temperatures at the time of the failures were, reportedly, 60 and 26.7 degrees Centigrade, respectively. A temperature-depth model (Southgate, 1969) was used to estimate the temperatures at each midpoint of each 2.54-cm asphaltic layer. Using these estimated temperatures, φ-and c-values for each layer were estimated from the curves shown in Figures 121 and 122. Values of CBR were converted to undrained shear strengths using the relationship shown in Figure 9. A uniformly

![Figure 120. Laboratory percentile test values as a function of CBR.](image)

![Figure 121. Relationship of φ and temperature obtained from unconsolidated-undrained triaxial tests.](image)

![Figure 122. Relationship of c' and temperature obtained from unconsolidated-undrained triaxial tests.](image)
distributed, tire contact stress of 552 kPa (80 psi)--dual wheels-- was assumed in the analysis (see Figure 7).

Factors of safety, based on different CBR design assumptions, are compared in Figure 123. When the average CBR value of the data set is assumed to be the correct value, a factor of safety of 1.33 is obtained. If it is assumed that the CBR (equal to 5) used in the original design is correct, then a factor of safety of about 1.59 is obtained. If the CBR values obtained from reliability theory at a confidence interval of 95 percent are used, then factors of safety of 1.22 and 1.43 are obtained. This approach yields an unsafe design. In each of these three cases, the factor of safety is much greater than one. However, since the pavements failed, the factor of safety should be near one. Based on values of CBR (1.4, 1.8, and 2.1) corresponding to percentile test values of 95, 90, and 85, factors of safety of 0.91, 1.00 and 1.07 are obtained, respectively. The CBR value (1.8) corresponding to the 90th percentile test value, which yields a factor of safety of one, seems an appropriate design choice.

The problem of selecting a design CBR value may be illustrated in another manner using model analysis to determine the required thickness for a given design factor of safety. Based on an analysis (Hopkins, 1994) of some 237 asphaltic pavement sections of the AASHO Road Test (1962), an approximate relationship (Figure 124), corresponding to a serviceability index of 2.5, between a factor of safety and (weighted) equivalent single-axle load (ESAL) was developed, or

\[ F = (0.095) \ln(ESAL) - 0.00463. \]  

Inserting the design ESAL of four million into Equation 59, the design factor of safety is 1.44. The total pavement thickness corresponding to a selected subgrade CBR value and design factor of safety was obtained from the bearing capacity model by iteration. Thickness of the pavement is varied until the factor of safety is equal to the selected
Factors of safety obtained from Equation 59. The thickness of the DGA (10.2 cm, or 4 in.) was held constant so that the various thicknesses (based on different assumed CBR design values) could be compared to the thickness of the pavement sections after overlays were constructed.

Thicknesses obtained from the various analyses, based on different assumed design values of CBR corresponding to a factor of safety of 1.44, are shown in Figure 125. If the lowest value of CBR (1.3) is assumed to be the correct design value, than a total thickness of 53.1 cm (20.9 in.) is required. This thickness is some 16.3 cm (6.4 in.) larger than the planned thickness. If the average value of CBR (3.4) is used, a thickness of 40.1 cm (15.8 in.) is obtained. The average CBR value yields a
thickness that is only 3.3 cm (1.3 in.) greater than the original planned thickness. A value of 3.4 corresponds to a percentile test value of only about 40 to 50 (see Figure 120). Accordingly, many portions (spot-to-spot) of the pavement would require future maintenance. Required thicknesses obtained when the upper and lower bound values of CBR obtained from reliability theory are only 0.25 cm to 2 cm (0.1 to 0.8 in.), respectively, greater than the original design section, which failed. If the CBR value (1.8) occurring at the 90th percentile is assumed to be the correct design value, then a thickness of 50 cm (19.7 in.) is obtained -- a thickness that is some 13.2 cm (5.2 in.) greater than the original planned section. As shown in Figure 119, values of CBR less than 1.8 occur at only about 10 percent of the sampling sites.

Approximately 50 percent of the total length of the highway sections was repaired using an overlay thickness of about 12.7 cm (5.0 in.). Total thickness of the pavement at those locations after overlaying was about 49.5 cm (19.5 in.) -- a value that is nearly identical to the thickness (50 cm, or 19.7 in.) obtained when the value of CBR at the 90th percentile test value is used. The method proposed by Yoder may be a reasonable approach to the problem of selecting a design subgrade strength as strongly suggested by this case history analyses. Using the 1981 Kentucky design curves (8), a thickness of 47 cm (18.5 in.) is obtained. Proper selection of a subgrade design value of CBR (or other strength parameters) is vital to avoid construction failures and to insure good pavement performance.

**Effect of Moisture on Soil Subgrades**

Subgrades built of clayey soils and compacted according to standard compaction specifications generally possess large bearing strengths immediately after compaction. However, no assurance exists that the subgrade soils will retain their original strengths. Bearing strengths of the completed subgrade depend on the long-term density and moisture. The original compactive state of clayey soils is very likely to change with increasing time and load applications. Clayey soils absorb water and increase in volume. With an increase in volume, the shear strength available to resist failure decreases. The differences in bearing strengths of compacted soils in soaked and unsoaked states may readily be illustrated by analyzing the results of some 727 laboratory CBR tests (1) -- see Figure 1. Each specimen of the group of tests was penetrated before soaking and after soaking. Before soaking, and immediately after compaction, bearing ratios of 95 percent of the specimens were greater than six. After soaking, the bearing ratio of only 54 percent of the specimens exceeded six. As shown by the theoretical analysis, bearing capacity failures may occur in the subgrade when the CBR is less than about 6.5 and the tire contact stress of construction vehicles is about 552 kPa (80 psi).

Field observations also show that bearing strengths of clayey subgrades may decrease
significantly after construction (Hopkins, 1991; Hopkins et al., 1983). Field CBR tests were done on a clayey subgrade, at a highway construction site in Kentucky, immediately after compaction. Values of CBR ranged from about 20 to 40. A second series of field CBR tests was performed after the subgrade has been exposed to a winter season. Values ranged from about one to four -- a dramatic decrease in bearing strengths. Therefore, as noted by Yoder and Watczik (1975), pavement design analysis should be based on the characteristics of the completed subgrade. In areas where water may infiltrate the subgrade from surface and subsurface waters, the design should be based on the strength of the soaked condition of the completed subgrade. The strength may be very large if field tests are done on the subgrade immediately after compaction. When sufficient time has elapsed between the completion of subgrade compaction and paving, and the subgrade has been exposed to wetting conditions, then using the field strengths of the soaked subgrade may represent a valid approach. However, when the pavement is placed immediately after compaction, then the field strengths may be too large to assume for design purposes. Moreover, many projects are scheduled years in advance and it may not be convenient, or the opportunity may not be available, to perform field tests in a soaked condition before the design analysis. Therefore, the design analysis should be based on the soaked strengths of laboratory tests. When the design is based on laboratory tests, then a question arises concerning the similarity of field and laboratory strengths.

Comparison of Field and Laboratory Subgrade Strengths

To determine the similarity of laboratory and long-term field strengths, two highway routes were selected where several laboratory (soaked condition) bearing ratios had been performed on the corridor soils. Field bearing ratio tests were performed on top of the untreated subgrades through core holes over a period of six years. Testing did not commence until the pavement had been placed and at least one winter and spring season had passed. Because it was not certain where particular corridor soils would be placed in the subgrades of each route, curves of percentile test value as a function of laboratory and field bearing ratios were developed and compared. Soils of the first route (identified as the AA route) are residual soils of the Kope Geological Formation (clayey shales). Classification of these soils ranged from A-6 to A-7 and CL to CH. A comparison of percentile test values as a function of laboratory and field values of CBR of this route are presented in Figure 126. Average values of laboratory and field CBR were 3.5 (56 tests) and 4.1 (22 tests), respectively. At the 90th and 80th percentile test value, the laboratory strength is about 90 percent of the field CBR. Between 60 and about 10 percent, the laboratory CBR value was about 90 to 70 percent of the field value. Therefore, a reasonable agreement was obtained between the laboratory and field percentile test curves. Comparison of laboratory and field values of CBR of the second highway route (KY 11) are shown in Figure 127. Classifications of the soils on this route ranged from A-4 to A-7 and ML-CL to CL. From percentile test values of 100
to 90, the field and laboratory values are essentially the same. Between percentile test values of 90 and 10, the field value is some 100 to 75 percent of the laboratory CBR. At the 90th and 80th percentile test value, the field and laboratory values of CBR are nearly identical. Based on these comparisons, laboratory CBR values appear to provide a reasonable representation of the field CBR values of the completed subgrade after sufficient time has elapsed for soaking conditions to develop. Consequently, design strength of the untreated subgrade may be based on the soaked laboratory CBR test.

Stabilization Requirements

As shown by the theoretical bearing capacity analyses, Figure 6, bearing capacity failures may occur in the subgrade during construction when the CBR value is below about 6.5 and the tire contact stress is 552 kPa (80 psi). Consequently, to avoid bearing capacity failure of the completed subgrade during construction, to provide a firm platform for paving, and to insure efficient construction, stabilization may be necessary. Considering that a variety of strengths may exist in the completed subgrade, subgrade stabilization should be considered when the CBR value occurring at the 80th or 90th percentile test value is below about 6.5 - 10. Although the value of the design CBR may be selected at some percentile test value
that is smaller than the 80th or 90th percentile test value if cost ratios are used. By using the CBR value at the 80th or 90th percentile test value, adequate subgrade stability should be available to maintain efficient construction throughout.

**Design Strengths of Chemically Stabilized Subgrades**

Selection of the design strength of subgrades treated with cement or hydrated lime will be controlled by the time allowed for curing. At the end of the curing period, sufficient strength must exist to withstand construction traffic loadings and to avoid bearing capacity failures. If the strength existing at the end of a selected curing period can be estimated with some degree of confidence, then that strength may be used in the pavement design analysis. For example, in Kentucky, treated subgrades are allowed to cure for seven days and substantial strength gains occur in the treated layer during the curing period. This specified curing period appears acceptable to sponsoring agencies and contractors. Optimum percentages, as determined from testing (Hopkins and Beckham, 1993), of cement or hydrated lime are used to treat the subgrades.

General guidelines for selecting the design strengths of hydrated lime- and cement-treated subgrades were developed based on strengths of the treated layers existing at the end of a seven-day curing period. Several highway routes were selected and core specimens of the hydrated lime- or cemented- treated subgrades were obtained at the end of the seven-day period. Many types of soils, ranging from A-4 to A-7, were used to construct the subgrades at the selected routes. Unconfined compression tests were performed on the core specimens. Bag samples of the untreated soil subgrades were obtained at several, equally spaced, locations along each route of the completed subgrade before treatment. Specimens of these soils were remolded to optimum moisture content and 95 percent of maximum dry density (AASHTO T 99). Optimum percentages of chemical admixture were used in remolding the specimens. After aging the sealed specimens for seven days, unconfined compression tests were done.

Field and laboratory unconfined compressive strengths of the hydrated lime- and cement treated specimens, as a function of percentile test values, are shown in Figures 128 and 129. Unconfined compressive strengths of the field, hydrated lime-treated specimens were about 85 to 90 percent of the unconfined compressive strengths of the laboratory specimens for percentile test values ranging from 100 to about 10. This indicated that the hydrated lime and clayey soils were mixed very well in the field and suggested that the hydrated lime penetrated the clayey clods. Unconfined compressive strengths of the field, cement-soil core specimens ranged from about 75 to 50 percent of laboratory unconfined compressive strengths for percentile test values ranging from 100 to zero, respectively. If the 90th percentile test value is a reasonable working level, unconfined compressive strengths of about 333 kPa (48 psi) and 707 kPa (102.6 psi) --undrained shear strengths, $S_u$, of 167m (24.2 psi) and 354 kPa (51.4 psi), respectively)
are reasonable values to assume in the design of hydrated lime- and cement-treated soil subgrades, respectively. Corresponding values of bearing ratios, estimated from the equation in Figure 9, are about 11.6 and 24.9, respectively. Estimated values of dynamic modulus of elasticity (Equation 10) are about 152,590 and 297,489 kPa (22,143 psi and 43,171 psi), respectively.

**Approximate Required Thicknesses of Treated Subgrades**

By using the seven-day strengths, some portion of the total strength gain of the hydrated lime-or cement-treated subgrade may be considered in the pavement design analysis. However, use of these strengths does not, necessarily, assure that bearing capacity failure of the treated layer will not occur. The bearing capacity of the chemically treated layer depends on the thickness of the treated layer and the bearing strength of the untreated layer found below the treated layer. To estimate thicknesses required to maintain an assumed stable condition (say, F = 1.5), bearing capacity analyses were performed using the model described above. In the analysis of this two-layered problem, the tire contact stress was assumed to be 552 kPa, the unconfined compressive strengths occurring at the 90th percentile test value (Figure 10) were assumed for the treated layers. The

**Figure 128. Field and laboratory percentile test values as a function of CBR: Soil-hydrated lime subgrades.**

**Figure 129. Field and laboratory percentile test values as a function of CBR: Soil-cement subgrades.**
bearing ratio of the untreated layer was ranged from one to nine (or unconfined strength ranging from 15 kPa to 130 kPa). When CBR values of the untreated layer are varied from one to nine and the factor of safety is maintained at a constant value of 1.5, thicknesses of hydrated lime-treated subgrades range from about 40 cm to 11 cm (15.8 to 4.4 in.), as shown in Figure 15. For cement-treated subgrades and for CBR values ranging from one to seven, required thicknesses range from about 21 cm to 7.6 cm (8.3 in. to 3.0 in.).

**Significance of Treated Subgrades to Pavement Structure**

The use of hydrated lime or cement not only increases the shear strength of a soil subgrade but it also improves the overall bearing capacity of a flexible pavement. The value of stabilizing subgrades with hydrated lime or cement may readily be shown by an example design problem. Assume, for instance, that a flexible pavement is to be designed for an equivalent single-axle load (ESAL) of 18 million and the subgrade soils are the same as those used at the 1960 AASHO Road Test (1962). Percentile test values as a function of field CBR values (from the trenching program --Table 2 of AASHO Road, 1962) for spring and summer seasons were determined. At the 90th percentile test value, bearing ratios, corresponding to spring and summer, are 2.5 and 3.0, respectively. Average CBR values are 3.6 and 5.3, respectively. The design is to consist of one-third asphaltic concrete and two-thirds crushed stone. Coefficients, $a_1$ and $a_2$, are 0.44 and 0.14, respectively, terminal serviceability index is 2.5, and tire unit contact stress is 466 kPa. The soil support value is three.

Structural number, SN, is 5.6. Total pavement thickness is 59.2 cm (23.3 in.) -- 19.8 cm (7.8 in.) of asphaltic concrete and 39.4 cm (15.5 in.) of crushed stone base. Using the CBR of the untreated subgrade at the 90th percentile test value -- 2.5 or an undrained shear strength of 36.7 kPa (5.3 psi) -- model analysis yields a factor of safety of 1.29. If the average CBR (3.6) is used, then a factor of safety of 1.55 is obtained. From Equation 31, the estimated ESAL is only 800,000 -- a value that is much lower than the design ESAL of 18 million. If the average CBR of 3.6 is used in the analysis, then the estimated ESAL value is about 16 million, which is near the design value of 18 million. However, the percentile test value, is only about 40. Therefore, if the value of 3.6 is used, much maintenance would be required.

Since the CBR value at the 90th percentile test value and the average CBR at the 40th percentile test value are below 6.5, stabilization of the soil subgrades should be considered to avoid bearing capacity failures. Moreover, difficulties may be encountered during placement of the first lift of crushed stone base if treatment was not done. Bearing capacity analysis of the untreated soil subgrade, based on the undrained shear strength at the 90th percentile test value, yield a factor of safety of only 0.46. Using the average CBR value of 3.6, or an undrained shear strength of 49
kPa (7 psi), the factor of safety is only 0.65. Now if the subgrade soils remained free of water (an unsoaked condition) during construction, then the CBR strength may be greater than 6.5 and construction difficulties would not be encountered during paving. The designer cannot rely on this unlikely condition. Subgrade stabilization should be done.

In the design analysis, both hydrated lime- and cement-treated subgrade layers were considered. For the hydrated lime-treated subgrade, an undrained shear strength occurring at the 90th percentile test value was used in the analysis. A strength value of 36.7 kPa (5.3 psi) at a CBR of 2.5 was used for the underlying untreated layer. For an assumed thickness of 30.5 cm (12 in.), a factor of safety of about 1.36 was obtained. A factor of safety of this amount should be sufficient to avoid bearing capacity failures and deep rutting during construction. If a 12.7-cm (5-in.) subgrade layer of soil-cement is assumed, then a factor of safety of about 1.35 is obtained.

Model analyses were performed to determine the factor of safety of the full 59.2 cm (23.3 in.) of pavement resting on the 30.5-cm (12-in.) layer of hydrated lime-treated subgrade or the 12.7-cm (5-in.) layer of cement-treated subgrade. In both cases, the values of undrained shear strength for the treated and untreated layers at the 90th percentile test value were used. When the lime-treated layer is included in the design, a factor of safety of 1.85 is obtained. Therefore, the factor of safety increases from 1.29 (no treatment) to 1.85, or about 31 percent. Predicted values of ESAL (Equation 3) are much more than 18 million. Similarly, when a 12.7-cm (5-in.) layer of cement-treated subgrade is used, a factor of safety of 1.85 is also obtained. Based on Equation 3, a design factor of safety of 1.57 is required. Accordingly to this approach the thicknesses of the asphalt layer and crushed stone could be reduced. Thickness of the asphaltic layers can be reduced from 19.8 cm to 12.7 cm (7.8 to 5 in.). The crushed stone thickness could be reduced from 39.4 cm to 25.4 cm (15.5 to 10.0 in.) when a 30.5-cm (12-in.) layer of a hydrated lime-treated subgrade or 12.7-cm (5-in.) layer of soil-cement is used. In both cases, the factor of safety is about 1.57 -- the required value that satisfies Equation 3.

Layer Coefficients of Hydrated Lime-- and Cement-- Soils

The coefficient, $a_3$, may be estimated for the hydrated lime-treated subgrade and the soil-cement layer for the example described above.

The structural number, $SN$, is

$$SN = a_1d_1 + a_2d_2 + a_3d_3,$$  \hspace{1cm} (60)
where
\[ a_1, a_2, a_3 = \text{layer coefficients representative of surface, base, and subbase (in this case, the treated layer), respectively, and} \]
\[ d_1, d_2, d_3 = \text{actual thicknesses, centimeters, of surface, base and subbase courses, respectively.} \]

The coefficient, \( a_3 \), equals 0.17, since \( a_1 \) and \( a_2 \) are equal to 0.44 and 0.14, respectively, the structural number is 5.6, the thickness of the asphalt is 12.7 cm (or \( d_1 = 5 \text{ in.} \)), the crushed stone thickness is 25.4 cm (10 in.), and the hydrated-lime layer is 30.5 cm (or \( d_3 = 12 \text{ in.} \)). Similarly, \( a_3 \) equals 0.34 when the 12.7-cm (5-in.) layer of soil-cement is considered.

**RAPID FIELD EVALUATIONS OF SUBGRADE STRENGTHS**

Several approaches may be used to evaluate the bearing strengths of subgrades after compaction and before paving. In past years, the sole control criterion was based on the assumption that if the subgrade soils were generally compacted to at least 95 percent of maximum dry density and near optimum moisture content, then bearing, or shear, strength would be adequate to sustain construction traffic stresses and avoid failure. Although meeting those requirements may provide adequate strength for paving immediately after compaction, no assurance exists that the bearing strength (unsoaked) prevailing immediately after compaction will be available at the time of paving. As noted previously, if the subgrade is exposed to rainfall, melting snow, and subsurface seepage, then the subgrade soils may absorb water, swell, soften, and loss strength before paving. Therefore, basing strength control solely on density control may be insufficient to avoid problems during pavement construction.

Two other approaches that may be used to evaluate the bearing strength of subgrades before paving consists of performing in situ bearing ratio tests or obtaining thin-walled tube samples of the subgrade and performing unconfined compression tests. Another approach consists of performing Falling Weight Deflectometer. Although any of these techniques may be used to evaluate the bearing strength of subgrades before paving, those methods are time-consuming, very specialized, and costly. Furthermore, sufficient time may not be available to conduct such testing before paving. As alternatives to those approaches, rapid methods, which do not require specialized personnel for evaluating subgrade strengths have emerged. Two such methods include the Dynamic Cone Penetrometer (Sowers and Hedges, 1966) and the Clegg Impact Hammer (Clegg, 1980; 1977).
Dynamic Cone Penetrometer

The dynamic cone penetrometer is a lightweight, portable device. As shown in Figure 130, the device consists of a steel ring weighing 6.804 kg (15 lbs) that slides on an E-rod. In operation, the sliding ring weight is dropped for 50.8 cm and strikes an anvil. The falling weight drives a cone, which is attached at the bottom of the driving anvil, into the soil. The cone point is enlarged to reduce shaft resistance. Although the original intended (Sowers and Hedges, 1966) use of this device was shallow in-situ penetration testing in boreholes, the device appears readily adaptable for testing subgrade surfaces. Normally, the number of blows required to advance the cone for a distance of 4.45 cm (1.75 in.) is counted. This is repeated for a second and third increments or 4.45 cm, as shown in Figure 131. The dynamic cone penetrometer value (DCP) is stated in units of mm/blow.

To develop relationships between CBR and unconfined compressive strength, $Q_u$, and dynamic cone penetrometer value, several highway routes under construction were selected. The selected routes included both untreated subgrades and

Figure 130. Dynamic cone penetrometer characteristics.

\[ DCV = \frac{1.75 \text{ in.}}{\text{No. Blows}} \times \frac{25.4 \text{ mm}}{\text{In.}} = (\text{Value})(\text{mm})(\text{Blow}) \]

Figure 131. Dynamic cone penetrometer calculations and increments.
hydrated lime- and cement-treated subgrades so that a representative set of values could be selected. In situ CBR tests were done at several locations. Thin-walled tube samples were obtained at locations where the dynamic cone penetrometer tests were performed. Unconfined compression tests were performed on the tube specimens. A relationship between in situ CBR and Dynamic Cone Penetrometer Value (DCP) is shown in Figure 132, or

\[ CBR = 67.2 \times DCP^{-1.164} \]  

A relationship (Figure 133) between unconfined compressive strength and DCP is

\[ Q_u = \frac{6894.8}{(8 \times DCP - 3.744)} (kPa). \]  

In developing the correlations given by Equations 61 and 62, the average value of the DCP values measured in the last two penetration increments was used. A relationship between Clegg reading and dynamic cone value is shown in Figure 134. Based on the curve in Figure 132, when the value of DCP is below about 5 mm/blow, the CBR of the roadway is less than 10 and the unconfined compressive strength--Figure 133--is less than about 190 kPa (27.6 psi)
or the undrained shear strength is equal to 95 kPa (13.6 psi). When this occurs, stabilization may be required.

Clegg Impact Hammer

The Clegg impact hammer provides another means of performing strength tests rapidly. The device consists of a 4.5 kg (10.1 lb) compaction hammer, a tube for guiding the hammer, and an electronic meter. An accelerometer is attached to the hammer. At impact, a signal from the accelerometer is transferred by cable to a readout meter. The meter filters, amplifies, and converts the signal to a digital readout form. At several highway routes, Clegg impact hammer readings were obtained at the same locations where undisturbed samples were obtained for unconfined compression tests. A correlation of the Clegg impact value (CIV) and unconfined compression strength, \( Q_u \), is shown in Figure 135 and may be expressed as

\[
Q_u = 22.287(CIV)^{0.958} \text{(kPa)} \quad (63)
\]

The undrained shear strength, \( S_u \), may be expressed as

\[
S_u = 11.144(CIV)^{0.958} \text{(kPa)} \quad (64)
\]
Variation of in-situ CBR and Clegg impact hammer value is shown in Figure 136, or

$$\text{CBR} = 0.487 \times (\text{CIV})^{1.348}.$$  \hspace{1cm} (65)

The relationship given by Equation 65 is compared to a relationship given elsewhere (Clegg, 1985). To develop a relationship between the Clegg impact value and the dynamic modulus of elasticity, Equations 65 and 10, cited previously, may be used. At a selected value of CIV, the CBR value may be computed from Equation 65. Inserting the CBR value determined from Equation 65 into Equation 10, the dynamic modulus of elasticity may be estimated. A relationship developed in this matter is shown in Figure 137, and

$$E_s = 9552 \times (\text{CIV}^{1.178}).$$  \hspace{1cm} (66)
Most of the soils used to construct highway pavement subgrades in Kentucky are fine-grained clays and silts. Many of these soils when initially compacted have large bearing strengths. However, as shown in this study, these compacted soils have a large affinity for water, swell, and lose bearing strength. Although extensive drainage measures are used in pavements, clayey subgrades are continuously, or periodically, exposed to surface and subsurface seepage during a pavement's life. Consequently, the opportunity for a loss of bearing strengths of clayey subgrades will exist during the life of a pavement despite compaction and drainage measures that may be invoked. Therefore, use of compaction and drainage measures alone will not, necessarily, prevent problems that may arise from a loss of subgrade bearing strength.

This study originated because of problems frequently encountered with highway pavement subgrades during and after construction. These problems, as noted by construction and geotechnical engineers of the Kentucky Department of Highways, included the shoving and pushing of clayey subgrades under construction traffic loadings, the lack of a firm working platform, or subgrade, for constructing and compacting base and asphaltic paving materials, and a loss of bearing strength during and after construction. Shoving and pushing, or bearing capacity failures, of the subgrade prevent the efficient movement of construction equipment and creates costly delays. The lack of a firm working platform creates costly delays and produces inferior pavements because they cannot be compacted properly. The loss of bearing strength of the subgrade causes premature pavement failures and requires early and costly maintenance.

The major objective of this research study was to establish a highway pavement subgrade stabilization program in Kentucky. During this study, a major subgrade stabilization program was implemented. However, to start a subgrade stabilization program, several design, laboratory, and construction issues had to be addressed and resolved. These issues are described and addressed in this report. A key issue is to recognize when subgrade stabilization is needed. Knowing when to modify a subgrade to improve bearing strength is essential to the development of sound and economical plans before construction and to assure the efficient construction of the pavement. As shown in the report, a detailed analysis of this problem was made using a newly developed, mathematical bearing capacity model. Based on those analyses, if the CBR of the subgrade is less than 6.5, then subgrade stabilization should be considered. This important recommendation was based on a construction tire stress of 80 psi (552 kPa). For other tire stresses, the minimum CBR strength required to prevent failure may be determined from charts shown herein. Findings of the model analyses were confirmed by published field data. This finding and recommendation were fully implemented
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during this study.

If stabilization is deemed necessary, then many methods may be used to improve the bearing strength of subgrades. The methods are broadly divided into two categories: mechanical and chemical. Mechanical methods include such historical techniques as density-moisture control, proof rolling (and rerolling), mixing of stone aggregate with the clayey subgrade, granular layers, and undercut and backfill techniques. Use of the first three methods does not insure success because the fundamental problem of the loss of strength when the subgrade is exposed to water is not fully addressed. A detailed laboratory study concerning the third technique was done by mixing a typical clay with stone aggregate in different proportions. Results of triaxial shear strength and bearing ratio tests shows that this technique may be of little benefit when the percent of particles finer then the 0.002 mm-sized particles are greater than about 15 to 20 percent. This method must be used cautiously. Although the fourth method has been used successfully, it is noted that the thickness of granular layer or backfill required to avoid bearing capacity failures, directly depends on available strength of material found beneath the backfill and tire stresses. Based on model analysis, a design chart for selecting an appropriate thickness of a granular layer is presented.

Other mechanical methods include geofabric - granular mattresses, use of in situ native materials for constructing a granular layer, and encapsulation. A detailed examination of placing geofabrics at the interface of the granular base and clayey subgrade is presented. These analyses, based on a newly developed, bearing capacity model, show that geofabric may be beneficial when the CBR strength of the subgrade is less than about six. More research using the newly developed model and field trials using geofabric is needed to confirm the theoretical analyses and to confirm proposed design procedures published by manufacturers of geofabrics.

Large portions of this study focuses on chemical admixture stabilization. Before about 1987, chemical admixture stabilization of soil subgrades was used only occasionally in Kentucky. During this study, only four sites were found that contained subgrades stabilized with a chemical admixture. Portland cement had been used at those sites. No highway sites could be found where hydrated lime had been used. Consequently, laboratory studies at two selected sites were initiated to determine the benefits of using hydrated lime, to develop laboratory procedures for determining the optimum percentage of chemical additive, and to develop some interim, chemical admixture specifications. The first successful trials using hydrated lime were performed on a stretch of KY 11, near Beattyville, Kentucky, and Section 19 of the Alexandria - Ashland Highway in Northern Kentucky. Results of these trials aided the development of admixture construction specifications. A detailed laboratory testing procedure to find the optimum percentage of chemical admixtures was developed during the early portion of the study. This procedure was implemented and is used routinely by the Geotechnical Branch of the Kentucky Department of Highways.
Although it is shown that chemical admixtures increase the bearing strengths of soils, a major issue arises concerning the long-term durability and longevity of subgrades stabilized with chemical admixtures. Four old highway subgrade sites stabilized with cement were found. Ages of these sites ranged from about nine to 30 years. Detailed field and laboratory studies showed that the CBR strengths of the soil-cement subgrades were exceptionally large (generally the 90th percentile CBR test value of all four sites was about 90). The flexible pavements on the soil-cement subgrades have performed well. Overlay maintenance at two of the older sites generally averaged about 12 years. Each of those sites has been exposed to many winter and spring seasons. Therefore, those subgrades have been exposed to several freeze-thaw cycles without adverse effects.

Because hydrated lime had not been used in Kentucky before this study, no long-term data concerning durability could be developed. However, at three experimental highway sites established during this study and after seven years, the CBR strengths of subgrades stabilized with hydrated lime are very large and are several times larger than the CBR strengths of the untreated subgrades. Although this study has been completed, field studies of the three experimental subgrades stabilized with hydrated lime will continue under a long-term monitoring program. To insure the long-term strength and durability of soil-hydrated lime stabilized subgrades, it was recommended that the optimum percentage of hydrated lime, as determined in this study, be used to prevent leaching of the lime from the upper portion of the treated soil. If lower percentages are used, then the danger exists that leaching will occur and create a "soften zone" in the upper reaches of the subgrade. The optimum percentage of hydrated lime is currently used on stabilization projects in Kentucky.

Each year coal-fired electric power plants generate enormous quantities of byproducts. For example, some 89 million tons (80.74 million Mg) of fly-ash, boiler slag, and flue gas desulfurization (FGD) materials were produced in the United States in 1989. Although some 25% of those byproducts were consumed by industry in making cement and concrete products, baseboard, and other products, about 75% of the generated materials were disposed in landfills. Almost 98% of FGD materials (atmospheric fluidized bed combustion (AFBC), FBC, etc.) were disposed in landfills. Production of FGD-type byproducts will increase tremendously in coming years. Major efforts have been made by various researchers in recent years to increase the usage of these byproducts.

One effort attempted in this study is described. An AFBC byproduct from an oil refinery (very similar to FGD byproducts from coal-fired power plants) was used as a subgrade chemical admixture at two experimental sections on KY 11. Laboratory studies show that the AFBC admixture increased the shear strength of the soils at that site several fold. However, about two months after partial completion of the pavement, and after a rainy period, pavement buckling occurred at several locations. None of the
original CBR laboratory data showed that swelling was a problem. Several swelling tests were subsequently performed on AFBC-soil mixtures. However, in contrast to allowing the specimens to swell for periods specified by standard procedures, the specimens were allowed to swell for several months. The swelling problem was not identified in the original tests because there was an initial delay in the swelling. Parameters obtained from the long-term tests were used to estimate when primary swelling would cease in the field. Based on those estimates, the pavement was milled and the final surface was placed. Since then (about six years) no swelling problems (except one, small localized area) has been observed. Because of this problem, it was recommended to Departmental engineers that no FGD byproducts be used as chemical admixtures unless it could be shown that a given FGD byproduct had no long-term swelling behavior as observed from long-term swelling tests. The long-term behavior of the subgrades treated with AFBC has been monitored for some seven years. These data show that the in situ CBR strengths generally exceed about 10; rutting depths are generally small—less than about 0.3-0.4 cm.

Moreover, additional studies were performed to learn the causes of the swelling behavior. Both x-ray diffraction (XRD) and scanning electron microscopy analysis was performed on field specimens of the AFBC-soil mixture. Analysis showed that the swelling behavior of the AFBC-treated subgrade was caused by the formation of ettringite, gypsum, and thamusite. Based on this research (and research performed on other FGD byproducts), the formation of these minerals and the amount of swelling is closely related to the presence of calcium sulfate and sulfite in the FGD byproduct. Therefore, the future use of FGD-type byproducts in highway construction will require much more research to determine the exact swelling mechanism of those byproducts and to develop means of preventing, or minimizing, adverse swelling behavior.

As another example of the use of a byproduct, multicone kiln dust (MKD), a byproduct obtained in the manufacturing of hydrated lime, was used as a chemical admixture on a subgrade section of KY Route 11. After seven years, in situ CBR strengths of the MKD-soil subgrade mixture are exceptionally large and generally exceed a value of 90. Rutting depths are less than 2.5 mm. Based on the performances of the treated subgrade and flexible pavement, it was recommended that MKD could be used as a chemical admixture at locations where laboratory studies show improved bearing strengths.

In the design of flexible pavements, the strength selected for an untreated subgrade will dictate the thickness of the structure. Along any given highway corridor, before construction, a variety of soils normally exist and a variety of bearing strengths may exist after compaction. Different philosophies exist concerning this choice. These include using the average value, the lowest value, a statistical value about the mean, and a least-cost approach. Based on bearing capacity analysis of case studies, it was recommended that the least-cost method of analysis be used to select strengths of
untreated subgrades. To simplify this approach, a PC computer program was written. The Geotechnical Branch of the Kentucky Department of Highways is using this program and the least-cost method on a trial basis.

When chemical admixtures such as hydrated lime or cement are used, the improved subgrade may be treated merely as a working platform, with no credit given in the pavement design to the improved bearing strength. The improved strength may be included in the structural thickness design of the pavement. If the improved strength is included in the design, then a design strength must be selected. In analyzing this problem, the time allowed for curing of the chemically treated subgrade, and the fact that construction traffic will operate on the treated subgrade at the end of the curing period, were factors that must be considered in the selection scheme. The strength and thickness of the treated layer must be adequate to sustain traffic stresses and to prevent bearing capacity failures. Bearing capacity failures at this stage of construction would leave failure zones in the pavement structure and cause failures, or cracking, of the pavement after construction. In Kentucky, a seven-day curing period is used; construction traffic is not allowed on the treated layer during this period. To develop values of design strengths, core specimens of hydrated lime and cement treated subgrades were collected from many stabilization projects near the end of the seven-day curing period. Unconfined compression tests were done on the collected specimens. Percentile test value as a function of the unconfined compressive strength for each admixture was developed. It was recommended that a value of unconfined compressive strength of about 80 psi (550 kPa) for hydrated lime-soil subgrades could be used in the pavement design analysis. When cement is used to improve bearing strength, it was recommended that a value of about 160 psi (1100 kPa) could be used in the design analysis. Both recommended values correspond to the 90th percentile test value. The recommended design values correspond to CBR strengths of about 11.5 and 25, respectively. Since the strengths of both hydrated lime--and cement--treated pavements usually increase after the seven-day curing period, the recommended values are conservative. Nevertheless, at least part of the strength gain is included in the design analysis. This approach and recommendation were implemented during the study period, although the lower value--CBR equal to 11.5--is currently used in the design analysis.

When hydrated lime or cement is mixed with soils containing high levels of soluble sulfates, the treated subgrades are subject to large magnitudes of swell when exposed to water. Swelling occurs because of the formation of calcium sulfate based minerals. The swelling, or heaving, of the subgrade, which is three-dimensional heave, can occur and cause heaving and buckling of the pavement. As noted above, use of FGD byproducts with high levels of soluble sulfates can produce heaving when the byproducts are mixed with soils. Although this potential problem exists, no cases of pavement heaving due to this problem have been reported, or observed, to date in Kentucky. However, it is recommended that additional research be performed to avoid
this potential problem. The methodology used to measure soluble sulfates needs to be standardized; subgrade soils considered for treatment with cement, hydrated lime, and fly ash should be tested to detect sulfate levels.

A final objective of this study consisted of recommending rapid methods of evaluating the in situ bearing strength of both untreated and treated subgrades. The dynamic cone penetrometer was recommended to the Department. This portable device is very simple to use and requires little training. This device should be very useful to the Department's engineers and technicians in the twelve districts of the state. To expedite the usage of this device, correlations of dynamic cone penetrometer values, in situ CBR strengths, and unconfined compressive strengths, were developed during this study. Many dynamic cone penetrometer, in situ CBR, and unconfined compression tests were performed on newly constructed subgrades of several selected highway subgrades. Additionally, a correlation of values obtained from the Clegg Impact Hammer and unconfined compression strengths were developed. Those correlations are currently being carried out by the Department's engineers.

As noted above, several important findings and recommendations were made regarding design and construction of highway pavement subgrades. These recommendations were transmitted to the Department's engineers during the study. The recommendations were offered formally at meetings of the Study Advisory Committee and during informal discussions with the Department's engineers. Essentially these recommendations have been implemented and this research study has established a major subgrade stabilization program in Kentucky. This program has had a major affect on the manner of designing and constructing pavements in Kentucky. According to the Department's engineers, the number of problems encountered in constructing pavements has decreased dramatically wherever chemical admixture stabilization has been used. Construction and paving operations are also more efficient and there are fewer delays. Moreover, there are indications that the time required for maintenance overlays are extended when subgrade stabilization is used. Therefore, this program may provide economical savings in the future.

FUTURE RESEARCH NEEDS

Although this study has been completed, at least five topics should be considered for future research studies. These topics are, as follows:

1. Long-term performances of pavements on treated subgrades, mainly those treated with hydrated lime.

2. Sulfate testing of various soils located throughout Kentucky and standardization of a sulfate testing procedure.
3. More detailed theoretical and case history studies of the use of geofabrics to strengthen pavements.

4. A detailed theoretical and field examination of the performances of flexible pavements at intersections constructed on chemically treated subgrades.

5. Encapsulation of the top portion of subgrades as means of stabilization.

6. A detailed examination of the chemical and physical swelling mechanism of FGD byproducts and the use of those materials for highway applications.

A laboratory procedure for compacting specimens for physical properties testing was described. Statistical analysis of seventy cases shows that actual dry densities and moisture contents of specimens compacted according to the procedure were near target values of dry densities and moisture contents. Consequently, the suggested laboratory compaction procedure provides a means of duplicating, within reason, the anticipated compactive state of soils as they may exist in an engineered facility.

Minimum shear strengths of soil subgrades required to construct flexible pavements and avoid instability were determined using a theoretical bearing capacity model. The model is based on limit equilibrium and calculates the factor of safety against failure of a layered system. Parameters used in the model analysis to describe each layer of material include the angle of internal friction, $\phi$, cohesion, $c$, and unit weight. Relationships between undrained shear strength and tire contact ground stresses of dual wheels were developed for factors of safety of 1.0 (incipient failure state) and 1.5 (an assumed stable condition). Also, relationships between CBR and tire contact stresses were developed.

For a typical tire contact stress of 552 kPa (80 psi), the undrained shear strengths required to maintain an incipient failure state and an assumed stable state are about 94 kPa (13.6 psi) and 144 kPa (20.9 psi), respectively. Those strengths correspond approximately to CBR values of 6.5 and 10, respectively. Corresponding values of dynamic modulus of elasticity, as determined from a published relationship, are about 92,000 kPa (13,351 psi) and 134,000 kPa (19,446 psi). Results of published field experiments show that to limit tire sinkage to small values -- for example, 0.64 cm (0.25 in.) -- subgrade CBR values must range from about six to nine for tire inflation pressures ranging from 345 kPa to 552 kPa (50 to 80 psi). Assuming a factor of safety of 1.0 -- a state of failure -- CBR values obtained from the theoretical model are only four to 6.6 for tire contact stresses ranging from 345 kPa to 552 kPa (50 to 80 psi). To maintain a stable condition, CBR values should range from about 6.5 to 10.
Consequently, there was reasonable agreement between results of field experiments and results of the theoretical model analysis. Moreover, if the CBR strength of the subgrade soil is smaller than the minimum CBR strength, as determined from the theoretical bearing capacity model, for a selected tire contact stress, than methods for increasing bearing capacity should be considered. For example, if the anticipated tire contact stress is equal to 552 kPa (80 psi), and the soaked laboratory CBR values of soils that will be used to construct the subgrade generally are lower than about 6.5, then subgrade stabilization should be considered.

To assure the safe construction of the first lift of a granular base or asphaltic pavement, the CBR strength of the subgrade should be approximately eight, or greater, when the anticipated tire contact stress is 552 kPa (80 psi). However, this value will vary depending on the tire contact stress of construction traffic. For a given tire contact stress, the required CBR strength may be determined from relationships presented herein.

Guidelines for selection of design strength of untreated soil subgrades and subgrades treated with cement or hydrated lime were proposed. Theoretical bearing capacity analysis showed that a minimum subgrade strength must exist to avoid bearing capacity failures during construction. To maintain an incipient failure state (factor of safety \( F = 1.0 \), and an assumed stable state (\( F = 1.5 \)), the undrained shear strength should be 94 kPa (13.6 psi) and 144 kPa (20.9 psi), respectively. These values correspond to CBR values of about 6.5 and 10, respectively. Corresponding values of dynamic modulus of elasticity were 92,000 kPa and 134,000 kPa (13,351 to 19,446 psi). Based on a case history involving the failure of a partially completed pavement, the method proposed by Yoder (3) in 1969 may be a reasonable approach for analyzing strength data of corridor soils and in selecting design strengths based on percentile test values.

It was proposed that if the minimum strength for a selected percentile test value is less than the minimum strength required to avoid bearing capacity failures during construction, then chemical stabilization (or other stabilization methods) of the subgrade should be considered. For example, if the tire contact stress of construction equipment is 552 kPa (80 psi) and the CBR is 2.5 at a selected percentile test value, then subgrade stabilization should be performed since the CBR strength of 2.5 is less than the CBR strength of 6.5 required to maintain an incipient failure condition. However, to avoid bearing capacity problems during construction, the subgrade CBR strengths should generally be greater than about 6.5.

Field CBR values of untreated subgrades obtained at two highway sites over a period of about 5.5 years were compared to soaked laboratory CBR values of corridor soils. Soaked laboratory CBR strengths appeared to represent the long-term field CBR strengths of the clayey subgrades of the two routes. Use of soaked laboratory CBR
strengths appears to provide a reasonable approach for selecting design CBR strengths of clayey subgrades.

Unconfined compressive strengths of core specimens from several soil subgrades treated with hydrated lime and cement were compared to strengths of laboratory specimens mixed with hydrated lime and cement for percentile test values ranging from 100 to 10. Strengths of core specimens mixed with hydrated lime were about 85 to 90 percent of the laboratory strengths. Strengths of soil-cement cores were about 50 to 75 percent of laboratory strengths for percentile test values ranging from 100 to zero. Based on a 7-day curing period and strengths of core specimens occurring at the 90th percentile test value, unconfined compressive strengths of about 333 kPa (48.3 psi) and 707 kPa (102.6 psi) seem reasonable values to assume in the design of hydrated lime- and cement-treated soil subgrades, respectively. Corresponding CBR values are 11.6 and 25. Dynamic modulus of elasticity are 152,000 kPa and 298,000 kPa (22,058 psi and 43,245 psi), respectively. Bearing capacity model analysis of an example problem showed that treated subgrades, based on these values, increased the overall bearing capacity of flexible pavement.

Although adverse swelling of the subgrades treated with an AFBC spent lime occurred during construction and required milling of the surface of the base course to remove heaved areas before placement of the final asphaltic surface course, the pavements have done well over a period of about five years. Pavement rutting has been nominal as observed after five years. Rutting depths have generally ranged from zero to about 0.2 cm (0.08 in.), although pavement rutting depths as large as about 1 cm (0.4 in.) were observed in a few isolated locations.

Moisture contents of the AFBC spent lime-soil subgrades and the untreated subgrades found below the treated layers and in a control section increased with increasing time. While bearing strengths of the untreated layers decreased substantially, the bearing strengths of the AFBC subgrades decreased only very slightly over the six-year study period. The CBR strengths of the AFBC subgrades generally were some four times greater than the CBR strengths of the untreated subgrades. CBR strengths of the AFBC subgrades generally exceeded 10.

Theoretical concepts, originally proposed by Terzaghi (1943) for analyzing consolidation of clay layers under imposed loadings, were very valuable in estimating the size and rate of swelling of the AFBC spent lime-soil subgrades and in devising a remedial scheme. Predicted and observed magnitudes and rates of swelling were similar. Future application of those concepts may prove valuable in evaluating the swelling characteristics and potential uses of such byproducts as the AFBC spent lime. Field and laboratory data show that swelling of the pavement should be less than about 0.64 cm (0.25 in.) over the next several years at the two AFBC sites. However, it is not certain whether these small amounts of swelling will cause future heaving of
the pavement since expansion of the spent lime subgrades probably occurs in three-dimensional directions. Because the subgrades are constrained in expanding in a direction parallel to centerline, the subgrades may heave or "buckle" at intermittent locations. Because of the uncertainty, long-term monitoring of the two experimental subgrades is scheduled.

Future research needs to focus on developing a full understanding of the physical and geochemical swelling mechanisms of byproducts such as the AFBC spent lime. Ettringite, thaumisite, and gypsum found in the subgrade were formed by the reaction of sulfates, calcium oxide, and calcium hydroxide with alumina silicates present in the native soil and water. These three minerals contributed significantly to the swell of the subgrade. Not only must an understanding of the conditions that cause these types of materials to expand but the means of controlling the expansion must be developed.

Long-term in-situ bearing ratios of cement-treated soil subgrades of four highway routes were measured. Ages of the routes and cement-treated subgrades ranged from six to 30 years. Thicknesses of the treated subgrades varied from 15.2 to 30.5 cm (6 to 12 in.). Cement content used to treat the subgrades was 10 percent, although in one section of one route a cement content of 7 percent was used. Excluding the thickness of the cement-treated layer, pavement thickness of the various sections of three of the four routes ranged from 25.4 to 48.3 cm (10 to 19 in.). Thickness of the fourth route was about 58.4 cm (23 in.). Subgrade soils were classified as CL or ML-CL. Plasticity indexes of those soils were low to moderate. All specimens obtained from the various cement-treated layers during the study period were generally classified as SM, or silty sand.

Bearing strengths of the cement-treated subgrades were generally very large. The relationship between percentile test value and in-situ bearing ratios of all treated layers of all sections is shown in Figure 82. At the 90th and 50th percentile test values, the bearing ratios are 24 and 90, respectively. These values compare very well with bearing ratios of crushed stone. The strengths of the cement-treated subgrades of the four routes were long-lasting. Based on these data, the large bearing strengths of cement-treated soils could be expected to prevail throughout a 20-year design life that is typically assumed in flexible pavement design. Moreover, at two sections of one route, rutting depths of pavements placed on cement-treated subgrades were nominal after six years. Based on visual inspections of the other three routes, rutting was nominal.

Typically, flexible pavements constructed on the cement-treated subgrades had required an overlay about once every 11 to 14-year period. However, the overlays were generally thin, that is, less than about 6.4 cm (2.5 in.). Annual daily traffic of all routes ranged from about 2,130 to 8,000 VPD. Truck traffic ranged from about 800 to 3,200 vehicles per day.
Findings of this study show that the use of cement-treated subgrades is a valuable technique for stabilizing low-bearing soil subgrades and is a good design alternative when compared to other stabilizing methods and design alternatives.

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