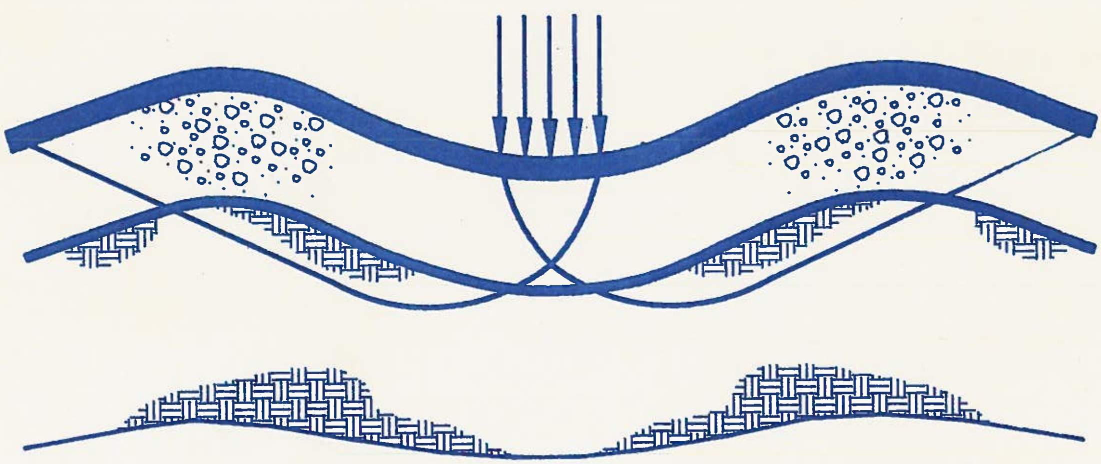
OHIO
RIVER
VALLEY
SOILS
SEMINAR

XIX

CHEMICAL AND MECHANICAL STABILIZATION OF SOIL SUBGRADES



PROCEEDINGS

October 21, 1988
Lexington, Kentucky

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PREFACE

Since 1970, the Kentucky Geotehnical Group and the Cincinnati Geotechnical Group, both affiliated with their local sections of the American Society of Civil Engineers, have been the principal sponsors of a series of one-day "Specialty Seminars". The geotechnical seminars are organized and presented for the purpose of providing a forum for the interchange of ideas and techniques among engineers engaged in the practice of geotechnical engineering, and particularly engineers engaged in the areas of soil mechanics, foundation engineering, rock mehanics, geomechanics, and geology. Past seminars and topics are summarized in the APPENDIX .

The 1988 Ohio River Valley Soils Seminar (ORVSS) was held on October 21, 1988, at the Holiday Inn North in Lexington, Kentucky. The seminar was principally organized and hosted by the Kentucky Geotechnical Group of the American Society of Civil Engineers. Co-sponsors of the seminar included the Cincinnati Geotechnical Group of the American Soicety of Civil Engineering, the University of Kentucky Department of Civil Engineering Office of Continuing Education and the Kentucky Transportation Center, the University of Louisville Department of Civil Engineering and the Center for Continuing Studies, and the University of Cincinnati Department of Civil and Environmental Engineering.

In October 1987, a task committee was appointed to select a seminar theme and organize the Nineteenth Annual Ohio River Valley Soils Seminar. The task committee consisted of the following members:

- Tommy C. Hopkins - Chairman -- University of Kentucky Transportation Center
- David L. Allen - Vice Chairman -- University of Kentucky Transportation Center
- Vincent P. Drnevich - Speakers and General Seminar Counselor - University of Kentucky
- Everett Gray - Exhibits and Patrons -- Kentucky Transportation Cabinet
- Paul Howell - Publications and Logistics - United States Soil Conservation Service
- John Storm - Publicity - Law Engineering, Inc.
- Don Armour - Brochure - Fuller, Mossbarger, Scott, and May, Inc.

The time donated freely by these individuals is gratefully acknowledged.

The theme selected for the 1988 seminar was **"Chemical and Mechanical**

Stabilization of Soil Subgrades". Geotechnical and pavement design engineers are frequently confronted with the problem of designing and constructing highway pavements, railroads, airport pavements, city streets, subdivision streets, parking lots, and haul roads on soils of poor or marginal engineering properties. Millions of dollars are spent each year in repairing roadways, railways, and airport runways because of the poor performances of soil subgrades. The purpose of the Nineteenth Annual Ohio River Valley Soils Seminar was to present a discussion on the techniques for improving the engineering properties of subgrade soils. Also, the seminar focused on this question: when should soil subgrade stabilization be considered?

As a means of developing the seminar subject and to encourage participation among the various geotechnical engineers, the seminar committee issued a call for abstracts in early 1988. Additionally, to ensure that the seminar subject was fully explored and developed from a technical viewpoint, some speakers with many years experience in soil subgrade stabilization were invited to participate in the seminar. As a result of these efforts, nine papers were submitted for publication and presentation.

The seminar committee divided the one-day seminar into three sessions. The morning program (Session I) was devoted to five formal presentations on the chemical admixture stabilization of soil subgrades. Douglas J. Keller, an environmental engineer with the United States Environmental Protection Agency, presided over this session. Following lunch, the afternoon program (Session II) consisted of three formal presentations on the mechanical stabilization of soil subgrades. Presiding over the afternoon session was Keith D. Combs, Senior Project Engineer of Fuller, Mossbarger, Scott, and May, Inc. Included in this session was a panel and comment session. Speakers in Sessions I and II were invited to participate in a panel and comment session. In this part of the program audience questions and comments were invited and encouraged. Professor Vincent P. Drnevich served as the moderator of the panel and comment session. The seminar committee would like to acknowledge the efforts of Professor Drnevich in performing this important task.

The evening program (Session III) consisted of a dinner and featured banquet speaker. Members of the seminar committee gratefully acknowledge Dr. Lars Forssblad, former Research Director of Dynapac, who traveled from Bromma, Sweden, for his presentation of the after-dinner speech to the seminar.

Geotechnical equipment and products pertaining to chemical and mechanical soil stabilization were exhibited throughout the seminar. Members of the seminar committee deeply appreciate the participation of exhibitors and patrons in the seminar. Their participation and generosity made the seminar more affordable to many engineering students. A listing of exhibitors and patrons is shown on the following pages.

The seminar committee deeply appreciates the contributions made by the audience, the panelists, presiding officers, and the authors of papers that were presented to this seminar. Also, the committee gratefully acknowledges Calvin G. Grayson, Director of the University of Kentucky Transportation Center, for the welcome and opening remarks he made to the seminar.

Tommy C. Hopkins

Seminar Chairman

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Admixture stabilization of subgrades

by

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Abstract: Subgrade stability relates to soil strength and deformation properties; with particular reference to repeated loading. Construction and pavement performance related subgrade stability requirements are considered. A minimum CBR of 6-8 is recommended to insure adequate subgrade support for high type pavement construction.

Factors influencing the in situ strength of fine-grained subgrade soils are established. Moisture content is the dominant factor for "wet of optimum" conditions. Many soils have in situ water contents greater than optimum and CBR's less than 6. Subgrade stability problems are thus frequently encountered.

Three remedial procedures (admixture stabilization, undercut and granular backfill, and moisture-density control) are evaluated. Admixture stabilization is frequently the best choice for fine-grained soils.

An approach to subgrade stabilization engineering is presented. The approach includes
1) Determine in situ subgrade strength,
2) select the appropriate admixture,
3) establish the admixture treatment level, and
4) determine the required treatment depth.

INTRODUCTION

Admixture stabilization is the physical mixing and blending of a powder, liquid, or slurry with a soil. Admixture stabilization is used to expedite construction, modify subgrade soils, and improve strength, stiffness, volume change, and durability (moisture and freeze/thaw resistance) properties of soils. The major thrust of this paper is the use of admixture stabilization as a remedial procedure for "SUBGRADE STABILITY" related problems.

The more widely used stabilization admixtures for fine-grained soil treatment are lime products (quick lime, hydrated lime, LKD-lime kiln dust), cement, CKD-cement kiln dust, and fly-ash. Several recent publications (1-4) consider in detail the technology and utilization of various types of admixture stabilization.

ADMIXTURE STABILIZATION MECHANISMS

Lime products, cement products, and fly-ash effect changes in many engineering properties of fine-grained soils. References 1-4 include in-depth information for various admixtures. Brief summaries relevant to subgrade stability problems are presented in this paper.

Lime Products

Strength, stiffness, plasticity, and durability properties are improved by the addition of lime. Compaction properties of fine-grained soils are also modified by lime treatment. Optimum moisture content is increased, which is desirable since high moisture content is the primary reason for most unstable subgrades. The maximum dry density is decreased. Typical moisture-density-CBR relations for natural and uncured lime-treated soils are shown in Figures 1 and 2.

The changes in compaction properties and immediate/uncured strength are achieved for all fine-grained soils with clay contents greater than 10-15%. With favorable curing temperature conditions, some cohesive soils (termed "lime reactive") will develop soil-lime pozzolanic reaction products and achieve high strength and modulus levels.

Cement Products

Portland cement treatment levels less than the amount required for "Soil-Cement" produce "Cement Modified Soil". Compaction properties (maximum

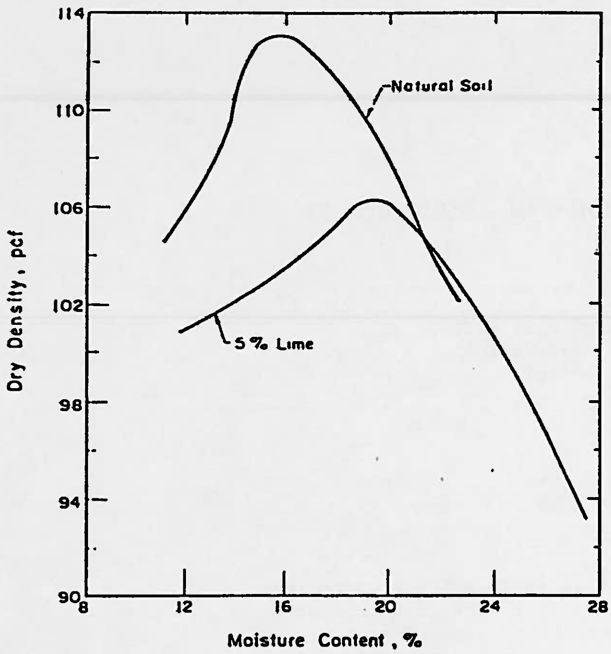


Figure 1. Moisture-density relations for natural and lime-treated A-6 soil

dry density/optimum moisture contents) and plasticity (PI, LL, PL) are not systematically changed by cement if evaluated shortly after the cement addition. With favorable curing conditions (moisture/temperature/time), the portland cement hydrates to produce strength/modulus increases. Increased cement contents and curing time effect larger strength/modulus increases.

The major problem in cement modification is construction related. It is very difficult (if not practically impossible) to adequately pulverize and mix wet cohesive soils with portland cement.

Fly Ash

Several stabilization mechanisms may be active in producing the beneficial engineering property improvements frequently noted in fly ash modified cohesive soils. The bulk drying effect (adding dry fly ash to a wet soil), the uptake of moisture in fly ash reactions, and the possible formation of "cementing type" agents may contribute to an overall subgrade stability increase. Type C (high CaO content) fly ashes are generally most effective. A higher fly ash treatment level is generally used compared to lime products.

THE SUBGRADE STABILITY PROBLEM

Subgrade stability is a term which relates to soil strength and deformation properties. Both properties significantly influence 1) the response of a subgrade to the heavy repeated loading of construction traffic and operations, 2) the ability to place and compact overlying layers, and 3) the long term performance of the pavement subgrade. Ideally, the subgrade should be strong enough to prevent excessive rutting and shoving, and sufficiently stiff to minimize resilient deflection.

To insure adequate stability, certain minimum levels of strength and stiffness must be achieved in the subgrade soil to the depth influenced by construction traffic as well as vehicles using the completed pavement. The depth of influence is a function of the magnitude of the wheel load, the tire pressure, and the relative stiffness of the various layers. Subgrade stability must be defined for given loading and traffic conditions.

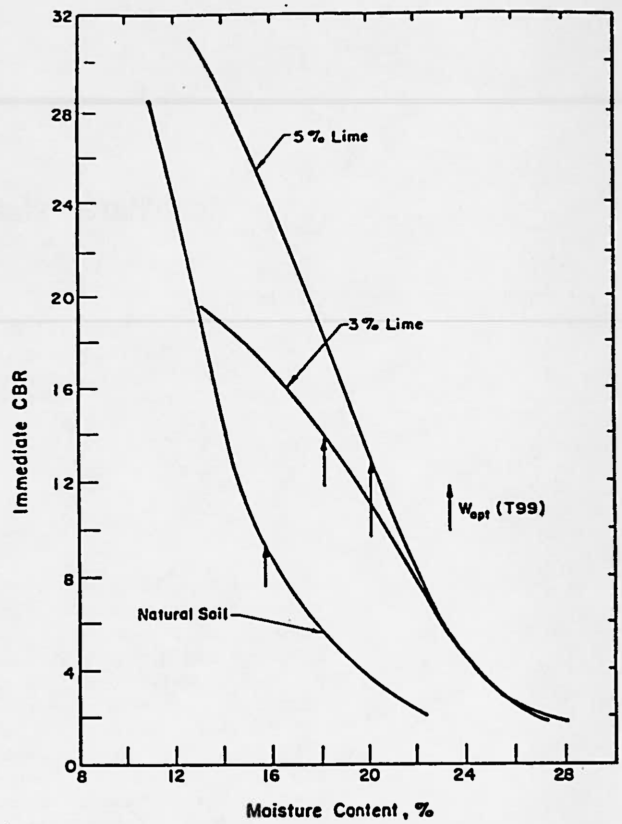


Figure 2. CBR - moisture content relations for a natural and lime-treated A-6 soil

CHARACTERIZATION OF FIELD CONDITIONS

Several studies (5,6,7) have demonstrated that for fine-grained soils the major factor that influences strength and stiffness is water content. An Indiana study (8) showed that, "water content appears as the dominant variable (with regard to strength) in the field and Standard Proctor regressions".

Inadequate subgrade stability is generally associated with moisture contents in excess of optimum as measured in the AASHTO T99 Compaction Test. In all cases of inadequate subgrade stability evaluated in an Illinois study (5), field moisture contents were significantly wet of optimum. Knight (9) has indicated that if the soil moisture content is greater than optimum, the Corps of Engineers cone index will generally be less than 300 (equivalent CBR of 6 or 7).

A study of placement moisture contents (6) for a variety of soils from two Illinois interstate highway sections, each approximately ten miles in length, revealed that sizable quantities were placed wet of optimum. The compaction moisture content averaged 97.2% of optimum with a standard deviation of 15.1% of optimum for the 1213 observations. On these projects 43% of the soil enbankment was placed wet of optimum, 20% above 110% of optimum, and 7% above 120% of optimum. Similar data developed by the Illinois DOT and others also indicate the potential of embankment construction with soils wet of optimum.

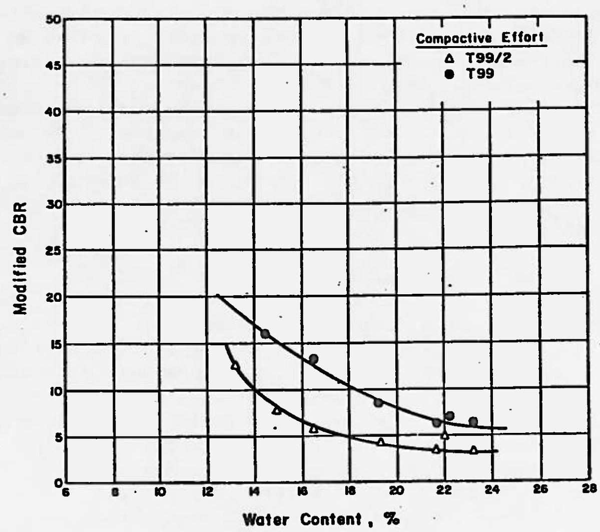
In many cases the field moisture contents at the borrow areas in the above studies were probably higher, but were reduced by aeration during placement and compaction. It is apparent that compaction wet of optimum occurs frequently, hence subgrade stability problems are common.

The field definition of potential subgrade stability problem areas requires knowledge of soil type and the moisture content. Density has a very minor influence on soil strength and stiffness at moisture contents wet of optimum, assuming relative densities on the order of 95% are achieved.

Soil Type Considerations

The current uses of pedologic soils information (county soil reports, etc.), previous engineering soil reports, geologic data, drilling, sampling, and testing activities for considering soil type are fairly well defined. It should be noted that soil type and distribution do not change as a function of time. If particularly bad soil types are not detected during the soils investigation, they can be easily located and identified during construction.

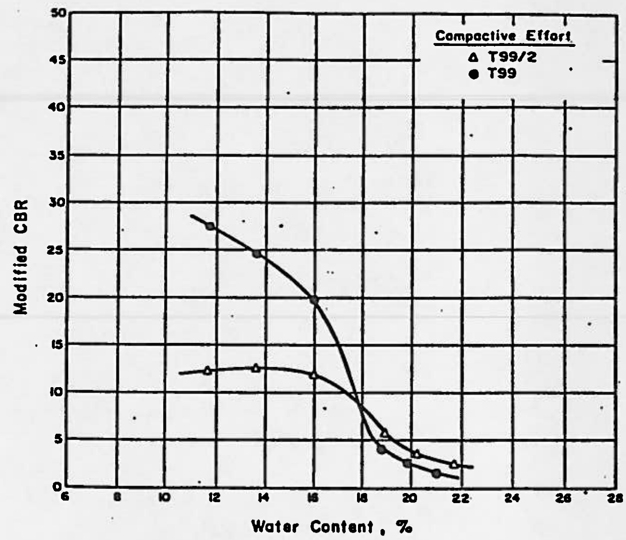
The adverse effects (loss of strength and reduction of stiffness) due to a high moisture content vary depending on soil type. Compare, for example, the CBR-moisture content relations in Figures 3 and 4. The high plasticity Drummer (Figure 3) is fairly insensitive to moisture content change while the low plasticity Fayette C (Figure 4) is quite sensitive. Illinois data (6) indicate that the resilient moduli of soils with high clay contents and high plasticity are less sensitive to moisture content increases than the soils of higher silt content and lower PI. Repeated loading permanent deformation data developed for typical Illinois soils indicate a similar effect of soil type on moisture sensitivity.



Drummer B - Champaign County, Illinois
AASHTO: A-7-6(28)

% Passing No. 200 - 100 T99/2 $W_{opt.}$ = 22.2%
% < 2μ Clay = 38 T99 $W_{opt.}$ = 21.5%
LL = 49
PI = 26

Figure 3. CBR-water content relations for Drummer B



Fayette C - Henry County, Illinois
AASHTO: A-4 (9)

% Passing No. 200 - 100 T99/2$W_{opt.}$ = 19.8%
% < 2μ Clay - 18 T99 $W_{opt.}$ = 16.6%
LL = 32
PI = 9

Figure 4. CBR-water content relations for Fayette C

It is important to have adequate soil characterization data for consideration of potential subgrade stability problems. Soil texture and plasticity (particularly PI) appear to be the two major factors that need to be considered.

Moisture Considerations

The most significant factor influencing the strength and stiffness of any fine-grained soil is moisture content. Unfortunately, soil moisture content in the field shows tremendous spatial variability and is constantly changing with time. The intricacies of moisture movement and moisture content changes in soils have been well documented by Dempsey and Elzeftawy (10).

Currently it is not practical on a design engineering basis to accurately predict field soil moisture content as a function of location, depth, and time. However, significant advances are being made, and the available technology certainly is of great value in rendering improved qualitative engineering decisions concerning field moisture conditions. It is very important to acknowledge that soil moisture content will change from the moisture content at placement.

The Soil Conservation Service of the U.S. Department of Agriculture uses seven natural soil-drainage classes (11). These soil drainage classes refer to the soil moisture equilibrium in the natural landscape. They should not be confused with surface drainage which is influenced by man's activities.

Figure 5 shows the general relationship of the water table depth to the natural slope of the ground surface for each of these soil drainage classes. In medium-textured, moderately permeable soils the depth to the water table varies directly with the slope. In finer-textured materials, more poorly drained soils occur on steeper slopes and in coarser materials well drained soils occur on gentler slopes.

Table 1. Equilibrium suction - CBR values for average construction conditions (Ref. 19)

Soil Type	PI	High Water Table (1) Thin Pavt. (3)	High Water Table (1) Thick Pavt. (4)	Low Water Table (2) Thin Pavt.	Low Water Table (2) Thick Pavt.
Heavy Clay	70	2	2	2	2
	60	2	2	2	2
	50	2	2.5	2	2.5
	40	2.5	3	3	3
Silty Clay	30	3	4	4	4
Sandy Clay	20	4	5	5	6
	10	3	6	4.5	7
Silt		1	1	2	2
Sand (poorly graded)		20	20	20	20
Sand (well graded)		40	40	40	40
Sandy Gravel		60	60	60	60

Notes:
1 - Water table depth within 12 inches of subgrade surface
2 - Water table 40 inches below subgrade surface
3 - Thin pavement is 12 inches or less in thickness
4 - Thick pavement includes a 22 inch granular "capping layer" and the total section thickness is equal to or greater than 47 inches.

Dempsey and Elzeftawy (10) have summarized the various rational procedures for predicting field moisture contents and also present a computer based model. The Transportation Road Research Laboratory (TRRL) procedure (12,13,14,15) was used (16) to demonstrate the effect of soil type and depth of water table on the field moisture content. The procedure has been described in an OECD publication (17), as well as a University of Illinois Report (10). For typical cohesive soils and water table depths within four to eight feet of the surface, the moisture content variation above the water table is small. For these shallow water table conditions, subgrades will frequently be near 100 percent saturation.

Langfelder (18) demonstrated that the suction-water content relations for compacted typical Illinois soils are basically independent of molding water content and density. Thus, equilibrium moisture contents above the water table are not dependent on initial placement conditions. Placing a soil dry of optimum, or at optimum does not insure that the moisture content will not subsequently increase.

The TRRL utilizes water table depth, soil textural class, and plasticity index values for estimating in-situ CBRs for pavement design. TRRL recommendations (19) are shown in Table 1.

SUBGRADE STABILITY REQUIREMENTS

Subgrade stability requirements are dictated by construction related and pavement performance considerations. The most pertinent construction aspects are rutting and shoving and the need to effectively and efficiently place and compact the various pavement layers. Pavement performance considerations (as related to subgrade stability) are primarily the resilient deflection of the pavement and permanent deformation accumulation in the subgrade.

Pavements can be designed to provide adequate performance for a broad range of subgrade support conditions. It is important that the subgrade support (subgrade stability) assumed for design be achieved at the time of construction and maintained throughout the desired design life. Adequate subgrade evaluation during the design phase should include careful consideration of both the soil type and the moisture conditions (current and future).

Construction subgrade stability requirements are more restrictive than the pavement performance requirements. Load induced stresses, strains and displacements are higher in the subgrade during construction than at any other time. The preceding statement should not be interpreted to mean that subgrade properties do not have an impact on the ultimate performance of the pavement. Quite to the contrary, the subgrade may be the most important factor influencing pavement performance.

Construction Related Requirements

Sinkage. Equipment sinkage (rutting) is an important construction consideration. Rutting creates an uneven grade, making it difficult to control the thickness of the subsequent pavement layer. Severe subgrade rutting causes a significant loss in equipment efficiency. Current specifications frequently indicate that rut depths in excess one to two inches are unacceptable. In reality, ruts of lesser depth are often intolerable because of strict controls on paving layer thicknesses.

Rutting is caused by a combination of permanent deformation by repeated loading at stresses near the shear strength of the material and bearing capacity failure. In terms of equipment mobility, the bearing capacity portion of the rutting is probably the most significant. A large portion of the permanent deformation is

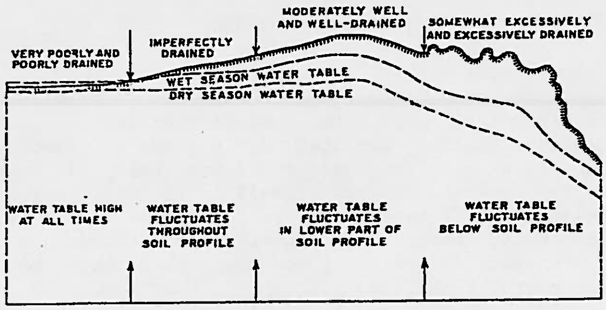


Figure 5. Water table depth - natural soil drainage relations (medium - textured soils)

accumulated during the first few load applications.

Traylor and Thompson (20) used two of the most promising procedures for predicting sinkage at varying subgrade strengths. Figure 6 illustrates the effect of subgrade strength on sinkage. To limit sinkage of a 9 kip wheel load with a 80 psi tire pressure to 0.5 inch or less, the subgrade strength should be in the CBR range of 5.5 to 8.5. For a 0.25 inch sinkage, the corresponding CBR strength range is 8 to 8.5. It is apparent that to minimize rutting damage of the finished grade, the subgrade CBR should probably be at least 6.

Compaction of Paving Materials. The shear strength and stiffness of the subgrade significantly influence the process of compacting pavement materials such as crushed stone, gravel, and stabilized bases or subbases. Compaction effectiveness and efficiency are both influenced by subgrade support. Results from controlled field compaction tests demonstrate that there are practical achievable density limits for a given type of equipment, layer thickness, and subgrade support.

If high surface deflections are experienced during the placement and compaction of asphalt concrete and high strength stabilized materials, cracking may be initiated at the bottom of the layer. These construction process induced cracks significantly decrease the "continuity" and "structural integrity" of the layer. Nondestructive surface deflection testing data support the preceding reasoning.

Field studies summarized by Heukelom and Klomp (21) lead to the conclusion that, "The degree to which layers of unbound materials can be compacted depends to a large extent on the reaction of the subsoil". When successive layers of materials were compacted over the initially placed granular layer, it was possible to achieve higher levels of compaction in the upper layers as indicated by the dynamic modulus of elasticity.

Heukelom and Klomp (21) suggested that the states of compaction, stability and decompaction of a granular layer over a subgrade can be considered by examining the tensile stress conditin at the bottom of the granular layer. They indicated that due to intergranular friction the vertical component of the stress would permit the granular material to withstant certain radial tensile stresss without decompacting or expanding. If the subgrade soil at the granular material-subgrade interface has a very low shear strength, it may not be possible to develop the full potential of the frictional stress needed to resist the radial displacement of the granular layer, and decompaction may follow. A low modulus subgrade results in high tensile stresses being developed at the bottom of the granular layer, also leading to decompaction.

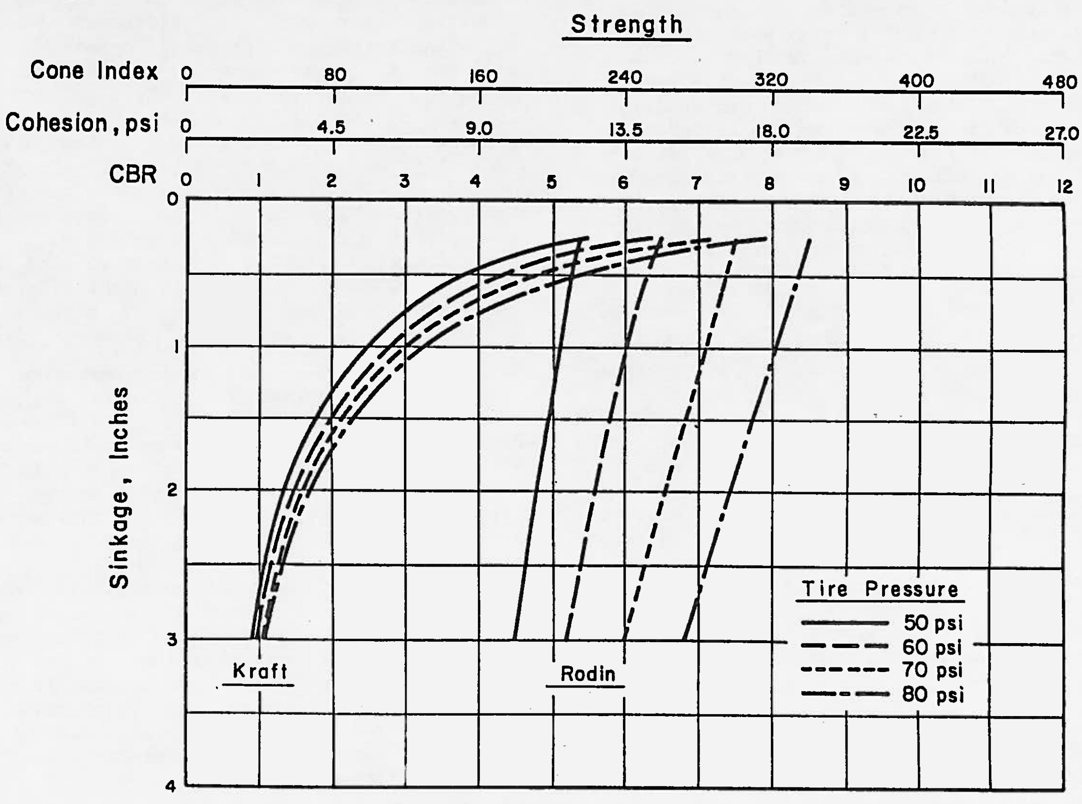


Figure 6. Soil strength - sinkage relations for a 9-kip wheel load

Barenberg's Shear Layer Theory (22) also demonstrates the importance of maintaining a high shear strength in the soil at the granular material- subgrade interface. The theory shows that the loss of shear strength at the granular material-subgrade interface causes a substantial decrease in load distribution capability and increases the deflection of the granular layer, thus preventing additional compaction.

Stress-dependent finite element analyses (ILLI-PAVE Model) have been conducted (16) to determine the effect of subgrade support on the compaction behavior of granular type layers (crushed stone, gravel, cement-aggregate mixture, bituminous aggregate mixture, pozzolanic aggregate mixture etc.). Realistic loading conditions for a pneumatic roller were used. If the system can not withstand sufficiently high tire pressures, field compaction, which involves shear failure in granular layer during densification, can not be accomplished. If tire pressures are too high, significant permanent deformation and shoving will develop in the subgrade.

Certain minimum levels of subgrade strength and stiffness are needed to insure that adequate compaction can be achieved. A minimum CBR of approximately 6 seems reasonable with regard to compaction operations and also checks favorably with the minimum stability required for construction sinkage control.

Performance Related Requirements

Subgrade stability in the completed construction must be at least equal to the value used in establishing the pavement design. The minimum acceptable level of subgrade support (based on thickness design considerations) should be stated in the project plans and documents. To insure adequate performance over the design life of the pavement, the subgrade support should meet those stability levels assumed in design.

Subgrade stability will vary with time. The major factors influencing the stability changes are moisture fluctuations and freeze-thaw action. The effects of moisture on strength and resilient properties have been discussed in a previous section. Freeze-thaw softening problems for typical Illinois soils have been considered by Robnett and Thompson (23). Typical detrimental effects (expressed as a reduction in resilient modulus upon freezing and thawing) may reduce resilient moduli by a factor of 2 or 3.

Summary

Construction operations and pavement performance factors should be considered in establishing subgrade stability requirements. In most situations, construction based stability requirements will control. It is interesting to note that in an Illinois study (6) the average CBR (immediate penetration) of typical fine-grained Illinois soils compacted at T-99 optimum moisture content to 100 percent of AASHTO T-99 maximum density was 8.6 with a standard deviation of 3. Approximately 20 percent of the soils had a CBR of less than 6. It is apparent that for moisture contents in excess of AASHTO T-99 optimum, many soils will have compacted CBR's less than 6.

REMEDIAL ACTIONS

Introduction

A comparison of subgrade stability requirements with the properties of typical cohesive soils compacted at a range of commonly found water contents indicates that in many instances the compacted soil will not possess adequate strength and/or stiffness. Thus, appropriate remedial procedures must be used.

Various remedial procedures that have been successfully utilized are:

A. Undercut and Backfill
B. Moisture Density Control
C. Admixture Stabilization (physical mixing of soil and admixture)

Undercut and Backfill

A popular remedial procedure is to cover the soft subgrade with a thick layer of granular material or to remove a portion of the soft material to a pre-determined depth below the gradeline and replace it with granular material. Geotextiles are frequently used to reduce granular layer thickness requirements and separate the subgrade from the granular layer. The granular layer distributes the wheel loads over the unstable subgrade and serves as a working platform on which construction equipment can operate.

Two conditions must be satisfied to provide a firm working platform. First, the thickness of the granular material must be sufficient to develop acceptable pressure distribution on the soft subgrade. Second, the strength of the backfill material must be able to limit rutting under the applied wheel loads to acceptable levels.

Moisture-Density Control

General. It has been shown in this paper that the stability (strength and stiffness) of a cohesive soil is influenced primarily by moisture content and, to a lesser extent, by density. Wet of optimum, moisture is the primary factor influencing stability. With a low density or excessively high moisture content (which is generally the problem), it is difficult to achieve an adequate working platform for efficient use of construction equipment and adequate subgrade support for the finished pavement.

A major problem in implementing moisture control is the proper establishment of permissible compaction moisture contents. Figures 3 and 4 illustrate the relations between compaction moisture content, CBR and compactive effort. Previously a minimum CBR of 6 was suggested for adequate subgrade stability. In Figure 7 it is obvious that the compaction moisture content must be less than 110 percent of optimum to insure a CBR of 6 in half of the soils tested. In many instances the in-situ soils or borrow materials are significantly wet of optimum, thus requiring extensive drying.

The use of density control in embankment construction as a means of improving subgrade stability is widely accepted as indicated by its overall use in specifications. To a lesser extent, the use of moisture control is accepted, generally as a qualitative requirement. The quantification of moisture control would contribute to increased subgrade stability, at least on a temporary basis.

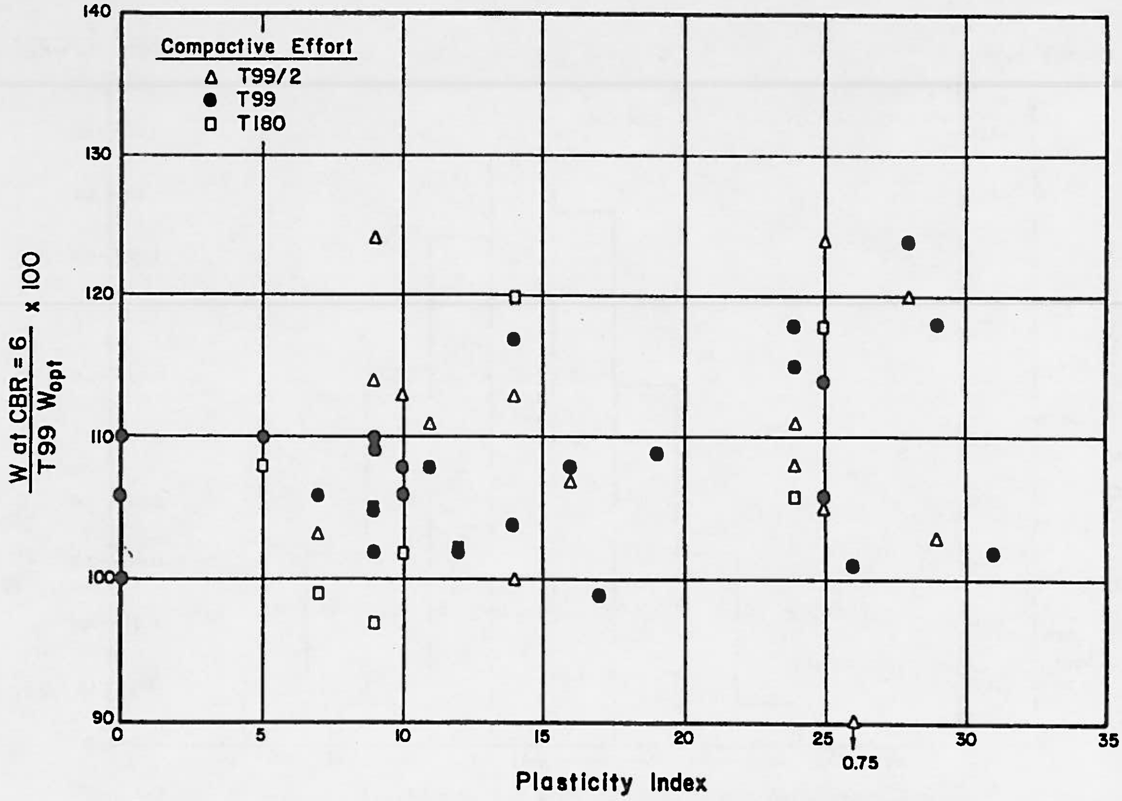


Figure 7. CBR - moisture content - PI relations for typical Illinois soils (CBR =6)

Water Content Control. To control excess moisture, many present specifications provide for drainage of the grade and drying of the top several inches of the subgrade. Drainage of the area will remove surface water, but will not significantly reduce the water content of fine-grained soils. Drying is accomplished through evaporation. Disking and manipulation of the soil increase the amount of exposed surface area available for evaporation and may speed drying.

To quantify the potential for drying by evaporation, Thompson et al (16) considered various models for predicting evaporation. Several models have been developed, but most of them use inputs which cannot be easily determined. Models which use commonly available inputs have been proposed by Thornthwaite (24) and Hamon (25,26) to predict evaporation from an open pool of water.

Hamon's (25,26) simplified expression for potential evapotranspiration is:

$$E_p = CD^2 P_t$$

where:

E_p = is the potential evapotranspiration (in./day)

D = possible hours of daily sunshine in units of 12 hours

P_t = the saturated water vapor concentration at the mean temperature (g/m^3)

C = 0.0055 (empirically developed constant)

Values of D^2 and P_t have been tabulated in Hamon's paper (26). For illustration purposes, potential evapotranspiration data (based on Hamon's equation) are presented in Figure 8 for Central Illinois. The theoretical percentage of moisture removed from a 1 ft^2 block of soil, 8 inches in depth is also shown.

Calculated evapotranspiration is likely to be more than the actual evaporation from a subgrade. The models are for ideal conditions, not allowing for precipitation during the calculated period. Both methods use mean temperature, which does not give as good an indication of potential as radiation. The methods also include removal of water by transpiration, which will not occur on an earthen surface. Neither method takes into account soil type effects.

The Hamon predictions are an optimistic appraisal of expected evaporation from the soil. Although drying can be expected from evaporation, it is questionable as to the possibility of large amounts of water being removed from the subgrade over a short time span. It is particularly important to note (see Figure 8), that during periods of low prevailing temperatures, very little drying can be achieved.

Maintenance of Subgrade Water Content. Many surficial soils are poorly drained. If a subgrade soil is in an area of a prevailing high water table, it may be possible to place the soil at or near the optimum water content but difficult to maintain that moisture condition. Moisture will be drawn up through the subgrade from the underlying soil due to higher suction in the dryer surface soil until an equilibrium condition is reached. The equilibrium water content with shallow water table conditions is generally considerably above the optimum water content. In addition, precipitation will cause surface wetting and subsequent moisture increases in the near-surface zone.

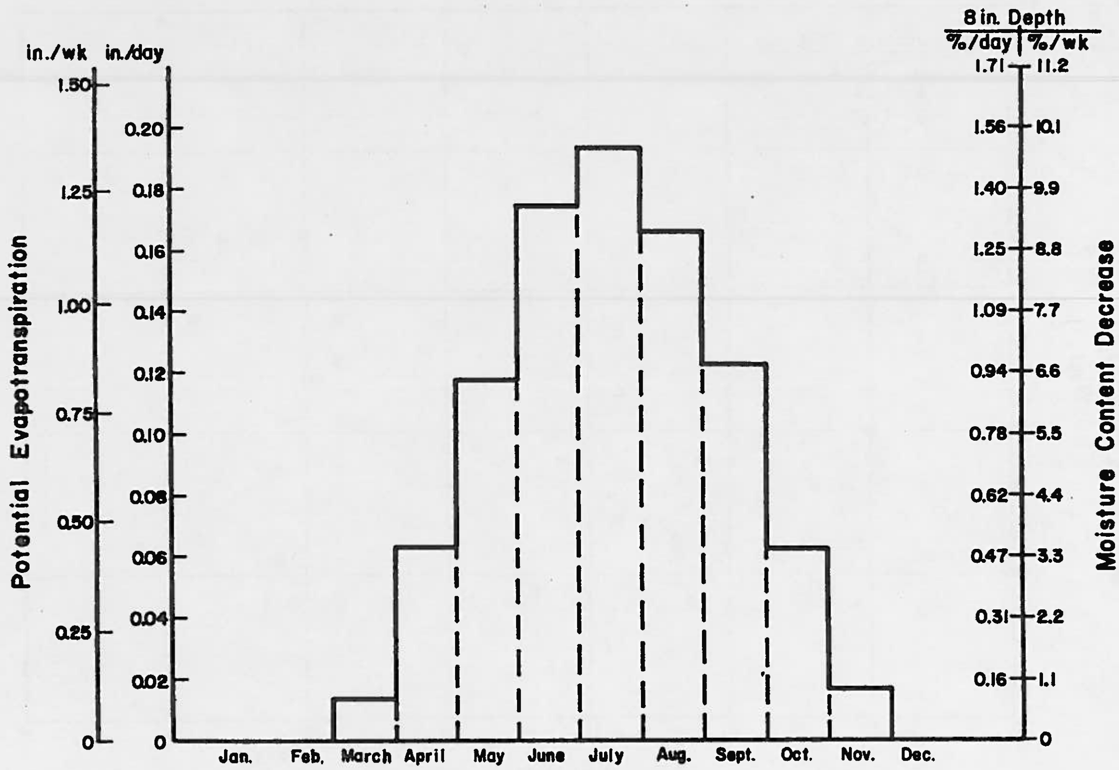


Figure 8. Potential evapotranspiration for Urbana, IL

Admixture Stabilization

Admixture stabilization (mixing and blending a liquid, slurry, or powder with the soil) is a technique which has been successfully used for improving soil strength and stiffness properties, thus improving subgrade stability. In some cases, particularly with lime products, immediate (uncured) strength increases are achieved while with other admixtures a curing period is required.

Those admixtures most widely used for remedial treatment of fine-grained subgrade soils are lime products (quick lime, hydrated lime, LKD-lime kiln dust), fly ash, lime-fly ash, cement, and cement kiln dust (CKD). Fly ash, CKD, and LKD are generally available at low cost. A particularly attractive characteristic of fly ash and kiln dusts is their low production BTU value.

Guidelines for selecting the appropriate admixture are presented in Reference 1. Selection criteria are gradation (primarily - #200 sieve) and plasticity index. Construction operation limitations (mixing and pulverizing) related to high clay contents and PI's and in-situ moisture contents in excess of optimum frequently limit the use of cement. The lime products are particularly well suited for these adverse conditions. In some applications, combination stabilization (e.g. lime pretreatment followed by cement) is utilized.

The length of curing period required to develop acceptable strength/modulus properties in the admixture modified soil is an important consideration in the admixture selection process. In some projects it is essential to have "immediate" results while in others a short curing period may be acceptable.

For cohesive soils (which are the most troublesome), lime products are currently the most widely used admixtures. In some areas flyash, particularly Type C, is quite popular. High treatment levels and mixing and pulverization problems associated with wet cohesive soils generally precludes the effective use of cement for subgrade stability applications.

With the increased use of the fluidized bed combustion process for burning high sulfur coal, BED-DRAIN or BED-DRAIN + fly ash materials (by products from the process) will become more readily available. These materials may prove to be effective soil stabilization admixtures.

General guides for stabilization construction are summarized in several References (1-4). Construction specifications and procedures for subgrade soil remedial treatment are frequently less stringent than those specifications used for "stabilization" where the stabilized material is to be used as a structural pavement layer. For example, the Illinois DOT has specifications for "LIME STABILIZED SOIL MIXTURE" (for structural layer applications) and "LIME MODIFIED SOILS". The Portland Cement Association (4) uses the term "cement modified soils" to describe a product not meeting "soil cement" quality requirements.

Several construction procedures have been utilized in "wet soil" treatment operations. Conventional rotary mixers can readily handle lifts up to approximately 12-15 inches. In some instances, discing may be adequate. Special procedures are needed to construct thicker layers. Deep plowing has been described in detail by Thompson (27). Lime-treated layers up to 24 inches in thickness have been constructed in one lift in Illinois. In some instances, "wet borrow soils" have been "100% treated" to form a stable embankment. Admixture stabilization can be accomplished using "borrow pit mixing" or the wet borrow can be spread on the embankment in a normal lift thickness and then stabilized.

Mixed-in-place admixture stabilization is in many instances the most cost effective remedial procedure. Undercutting and removal are not required and mixed-in-place construction procedures are relatively inexpensive. Improved job mobility, less loss of working days due to wet weather, and a general expediting of construction are frequently mentioned benefits of admixture stabilized subgrades. Significant energy savings (relative to other remedial procedures) may also be achieved.

The technologies associated with the various forms of admixture stabilization are fairly well established and can be used with confidence. Careful consideration should be directed to selecting the admixture, establishing admixture treatment levels, specifying construction techniques and operations, and exercising adequate construction control.

SUMMARY - SUBGRADE STABILITY

Soil type and moisture content are the major factors influencing subgrade stability (soil strength and deformation properties). For a given soil, the moisture content is the factor that primarily controls subgrade stability.

The current use of pedologic soils information, previous soil reports, geologic data, drilling, sampling, and testing activities for considering soil type are fairly well defined and seem adequate for subgrade stability evaluation purposes. Techniques and procedures currently used in practice for characterizing and predicting the field soil moisture regime are inadequate. It is essential to acknowledge that field soil moisture content is not static, but constantly varying as a function of time.

Subgrade stability requirements are <u>primarily dictated by pavement construction considerations</u>. Analyses of equipment sinkage and paving material compaction operations indicate that a minimum in situ CBR of 6 to 8 is required.

Many typical fine-grained soils do not develop CBR's in excess of 6 to 8 when compacted at or wet of AASHTO T-99 optimum water content. Thus, to provide adequate subgrades for pavement contraction, remedial procedures must frequently be used.

Three remedial procedures--undercut and backfill, moisture-density control, and admixture stabilization were described and evaluated. Undercut/ granular backfill and admixture stabilization offer the greatest potential and provide permanent solutions with significant carry over effects beneficial to the ultimate performance of the completed pavement.

SUBGRADE STABILIZATION ENGINEERING

Several issues must be considered in "engineering" an admixture stabilization treatment for a subgrade stability problem. They are:

1. Determine the range of in-situ subgrade strengths;
2. Select the appropriate admixture(s);
3. Establish the admixture treatment level (generally expressed on a dry weight of soil basis); and
4. Determine the required treatment depth.

Characterizing In-Situ Strength

Two conditions may exist. In the first, the subgrade stability problem is anticipated during project design activities. Thus, field exploration and sampling and laboratory testing data are available to characterize in-situ strength and determine the need for remedial treatment.

In the second condition, the problem develops during construction. Devices like the Corps of Engineers Cone Penetrometer or a Dynamic Cone Penetrometer (DCP - See Figures 9 and 10) are rapid and economical hand-operated portable devices for quickly characterizing the in-situ subgrade strength. The DCP is the preferred device since it can be used over a broad range of strengths and can penetrate to a significant depth (up to 40-45 inches).

Admixture Selection

This topic has been previously discussed.

Mixture Design

Establishing the appropriate admixture treatment level should be based on laboratory testing data. A moisture-density-CBR based procedure (AASHTO T-99/AASHTO T-180/AASHTO T-193) is a commonly used procedure. In this procedure, a CBR penetration test is immediately conducted on each specimen prepared in the moisture-density test. For fine-grained soils, the CBR's determined in a 4-inch or 6-inch diameter mold are similar. The 4-inch mold is more convenient to use. A typical moisture-density-CBR plot is shown in Figure 11. If warranted, a range of compaction efforts (see Figures 3 and 4) can be evaluated.

A minimum CBR of 10-12 is suggested as a general guide for admixture modified cohesive soils. An adequate thickness of CBR 10-12 quality material should provide a satisfactory working platform for routine pavement construction operations. If the immediate (uncured) CBR of the admixture-modified soil does not meet the CBR requirement, the effect of curing conditions (time/temperature) can be established using the procedure. Individual samples must be prepared for each moisture/density condition. Curing should be accomplished in "sealed containers". If substantial "cementing reactions" are achieved, very high CBR's may develop.

As an alternative, unconfined compressive strength data may be used to establish curing effects. An unconfined compressive strength of 40-50 psi is approximately equivalent to CBR 10.

The proposed procedure does not consider durability (moisture/freeze-thaw effects). Moisture effects can be considered by preparing specimens and subjecting them to soaking (complete immersion) prior to CBR penetration testing. A new specimen is needed for each test point.

The laboratory testing conditions <u>must be</u> a realistic representation of the expected field conditions (moisture-density-curing- "degree" of mixing and pulverization). Data developed from "idealistic and unrealistic" laboratory testing conditions is of limited value and may be misleading!

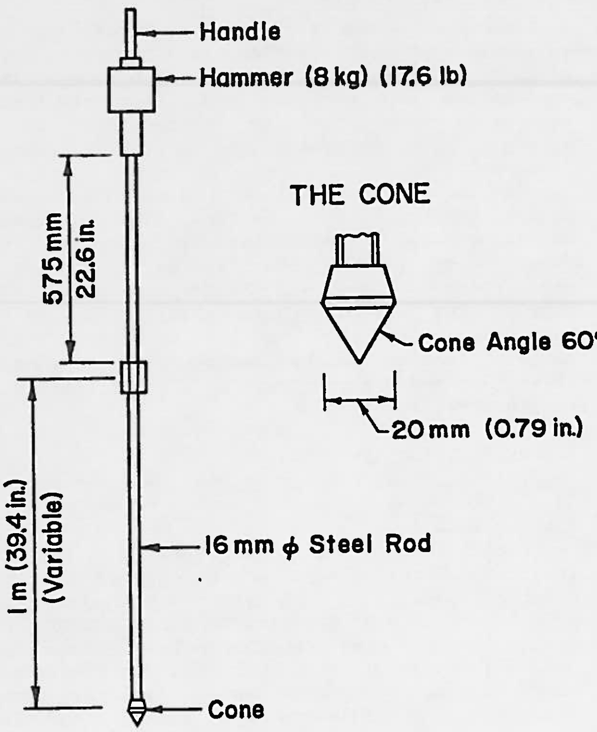


Figure 9. The dynamic cone penetrometer (DCP)

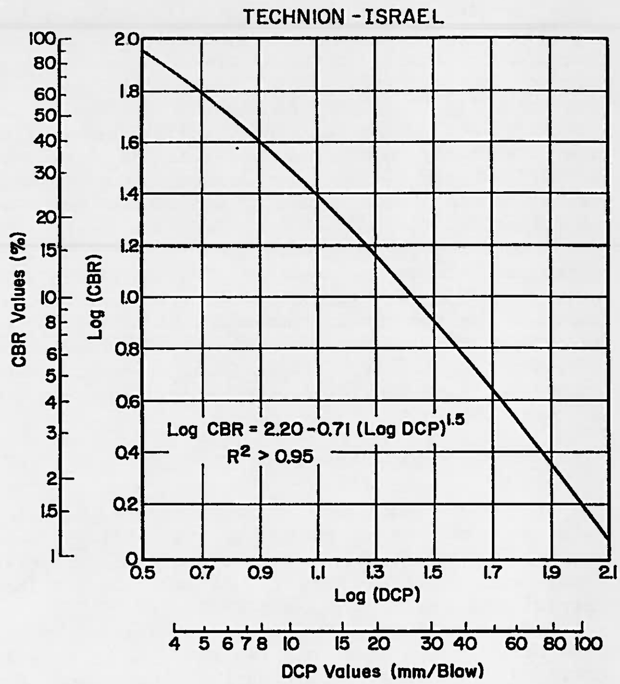


Figure 10. CBR - DCP correlation

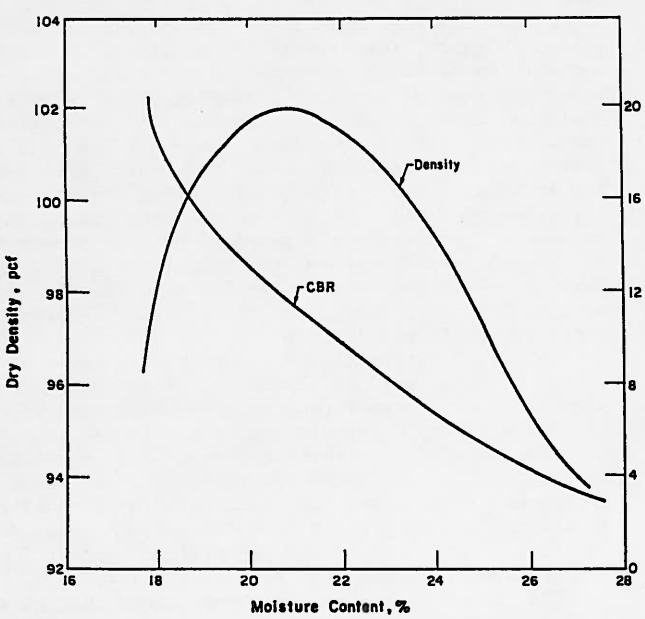


Figure 11. Typical moisture-density-CBR relations

Treatment Depth

Admixture modification of an in situ soil layer or the removal of the unstable subgrade material and replacement with a granular material creates a working platform that can accommodate repeated coverages of construction traffic. The admixture modified soil layer or granular backfill material distributes the stresses over a larger area of the subgrade, thus reducing the magnitude of subgrade stress and strain. The improved layer (admixture-modified soil or granular material) must have adequate strength to minimize rutting. Minimum CBR values are 10-12 for the admixture modified soil layer and about 20 for granular material. The minimum CBR values are for the in-situ material (field moisture content/field density) at the time of initial loading. The DCP (Dynamic Cone Penetrometer) is a good device for rapidly assessing the in-situ strength of the admixture-modified soil layer.

Thompson, et al, (16) adapted the Corps of Engineers' CBR based design approach (28,29) to develop the thickness requirements shown in Figure 12. Note that the thicknesses are for 500 coverages and 32 kip/tandem axle/dual wheel loading. Similar design charts can be easily developed for other numbers of coverages and wheel loading conditions.

Figure 12 thicknesses reduce maximum subgrade stresses to about 75% of the soil's shear strength (16). Most soils can withstand 500-1000 repeated stresses of this magnitude without experiencing permanent deformation strains in excess of 1-1.5%.

If the CBR of the admixture-modified soil is less than about 10, a granular surface layer may be necessary. The combined thickness of the granular layer and the admixture-modified soil layer is then considered the thickness of "improved material" above the underlying grade.

The minimum admixture-modified soil layer thickness should be 8 to 10 inches.

Proper consideration of the factors discussed above should provide a firm working platform for efficiently constructing high-type pavements. The process is obviously not refined and precise. However, it has worked well in many field applications.

SUMMARY

Fine-grained subgrade soils frequently lack adequate stability (in situ CBR's less than about 6) for supporting pavement construction operations. Inadequate stability is generally associated with moisture contents wet of optimum.

Admixture stabilization, undercut and backfill, and moisture-density control are commonly utilized remedial procedures. Admixture stabilization and the undercut/backfill techniques offer permanent beneficial carry-over effects to the completed pavement section. The admixture stabilization option offers many advantages and is frequently the best remedial treatment choice.

General guidelines for determining admixture treatment level and establishing required treatment thickness are provided. The guidelines and procedures are not "sophisticated and refined", but are considered appropriate for the rather ill-defined field conditions typically associated with subgrade stability problems.

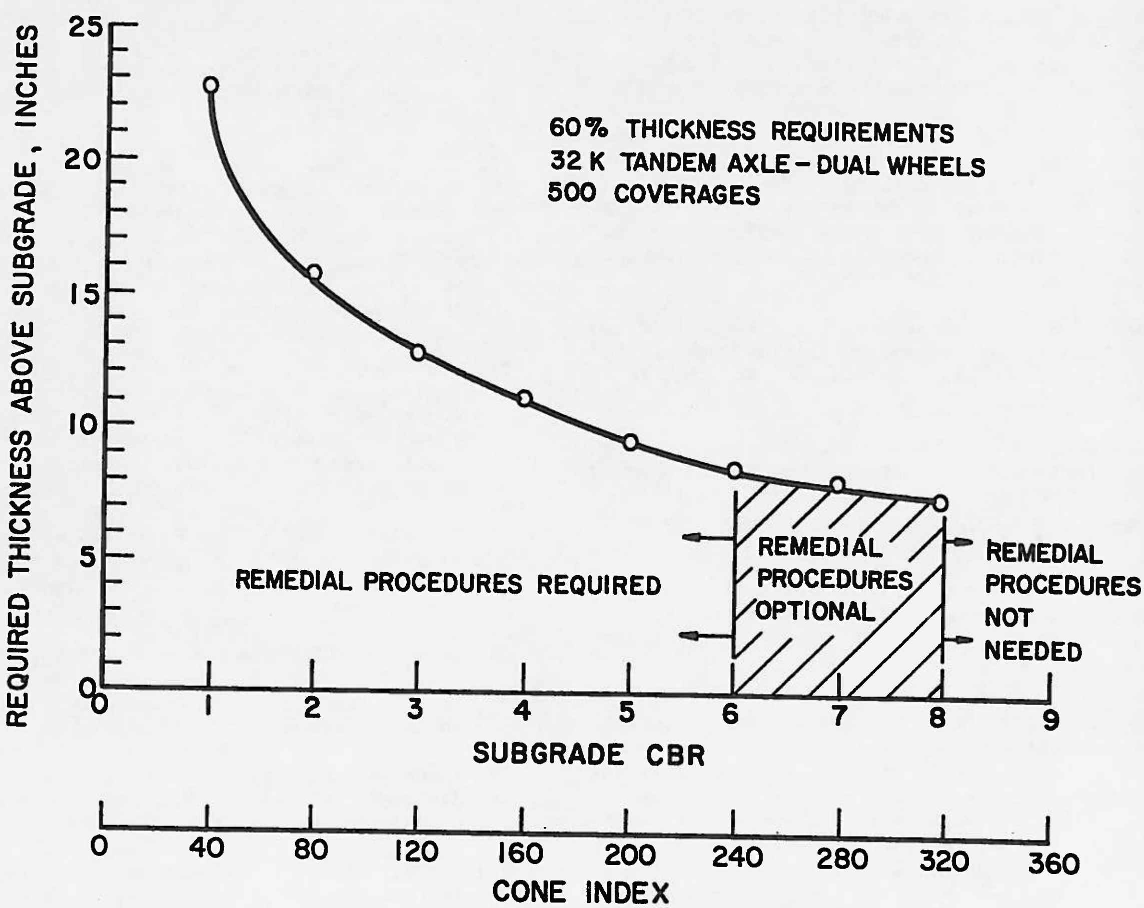


Figure 12. CBR-based thickness design procedure for granular backfill and admixture modified soil

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Case history of a cement stabilized coal transfer yard

by

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Abstract: A clean coal storage and handling facility was constructed adjacent to the Ohio River near Ghent, Kentucky in 1986. The storage yard was built by partial excavation and partial filling and the yard surface was stabilized using compacted soil-cement to provide a working surface. Shortly after the facility became operational, several areas of the soil-cement pad failed and surficial sloughing occurred in the fill outslope. Remedial repairs were made to the affected portions of the soil-cement yard by segmental removal and replacement of the failed areas with cast-in-place concrete. Moreover, the fill outslope was stabilized by installing shallow synthetic drains to intercept seepage before it reached the face of the slope. Based on the satisfactory performance of the repaired outslope and pad, a relatively simple and cost effective repair scheme has been identified that can continue to be used for any subsequent repairs. As described herein, the cost of the selected soil-cement stabilized yard surface was lower than other surface stabilization options which were considered even when the cost of the remedial repair work was included.

INTRODUCTION

Numerous industries require storage yards for the temporary holding or stockpiling of materials and products prior to distribution. Typically, the industrial machinery used in the particular storage area governs the design of the yard surface. Over the past several decades, the size of industrial equipment has increased dramatically to the point where the wheel loads from modern industrial vehicles can now exceed those of common highway trucks and even heavy aircraft (Portland Cement, 1975). Moreover, for reasons of economics, subgrade improvement techniques such as lime stabilization, soil-cement stabilization and aggregate stabilization using geotextile fabrics are now being used to provide the full working section of many large storage yards. Consequently, published information on the design of highway and airport pavements may not be applicable and designers of storage yards often must make decisions without historical performance data of the various alternatives for surfacing being considered. In addition, industries may choose to construct "interim" storage areas using low cost construction materials with the understanding that periodic replacement of damaged sections may be required. After the facility becomes operable, revenue which is generated can be used to systematically upgrade the yard surface to permanent status, thereby enhancing operational efficiency in an economically affordable manner.

A case history is presented of the coal storage yard constructed for Cleancoal Terminal Company near Ghent, Kentucky. Because of the availability of suitable on-site borrow and due to economic considerations, a cement stabilized sand and gravel surface was selected as a surfacing for the yard. Soon after the facility became operable, several areas of the soil-cement pad failed and surficial sloughing of the northern fill outslope occurred. Remedial repairs were made to the fill outslope and soil-cement pad while the facility continued to operate. Presented herein is a description of the design, construction, performance monitoring, and remedial measures used to repair the areas that failed.

BACKGROUND

Cleancoal Terminal is located between U.S. Highway 42 and the Ohio River approximately three miles northeast of Ghent, Kentucky. The area now occupied by the coal storage yard was previously farmland with the natural ground surface on a maximum slope of approximately 10 horizontal to 1 vertical. Surface elevations across the site ranged from about 510 feet near Highway 42 to about 460 feet near the north side of the proposed storage yard. As shown on Figure 1, the facility is comprised of several structures including a twenty (20) acre coal storage yard, a rail-mounted-stacker along with a reclaim conveyor belt, two (2) automatic sampling buildings, one (1) transfer building and a railcar dump bin. The site is underlain by glacial outwash deposits and more recently deposited alluvium as shown on the generalized cross-section of subsurface conditions included as Figure 2.

Clean coal is delivered to the site by rail, unloaded in the dump bin and conveyed to one of the sampling buildings. Thereafter, it can be loaded directly onto river barges for distribution to various power plants, or it can be stockpiled in the storage yard and reclaimed at some later date. Stockpiled coal is delivered to reclaim hoppers which traverse the center of the yard using Michigan 475-C front-end loaders, each with an operating weight of approximately 200,000 pounds.

DESIGN CONSIDERATIONS

During the initial planning phase for the proposed facility, geotechnical studies were performed by others (Converse Consultants, 1985 and Bishop, 1986) and surfacing options were examined as follows:

1. Lime stabilization;
2. Soil-cement stabilization;
3. Cement-aggregate stabilization; and,
4. Geotextile fabric-geogrid-aggregate stabilization.

Thickness design computations and cost comparisons for the various surface treatments were evaluated based on the anticipated wheel loads which would be exerted by the end loaders. For design purposes, a ground contact pressure of 65 pounds per square inch (psi) was used. The data presented in Table 1 was assimilated for evaluation.

Table 1. Comparision of alternative surface treatment costs

Surface Treatment	Recommended Thickness (inches)	Estimated Unit Cost ($/yd²/in)	Estimated Total Cost ($)
Lime Stabilization	51	0.30	1,240,000
Soil-Cement (6% Content)	30	0.60	1,440,000
Cement-Aggregate	23	0.78	1,432,000
Geotextile-Geogrid -Aggregate	30	0.40	952,000

Subsequent planning and research by the Owner indicated that a surface treatment which would produce a slab-like finish would be most suitable for Cleancoal Terminal in order to avoid inadvertent mixing of the different quality coals as well as to avoid contamination of the clean coal by upward migration of the subgrade. Considering this additional criteria, stabilization of the yard surface with geotextile-geogrid-aggregate was deemed unsuitable.

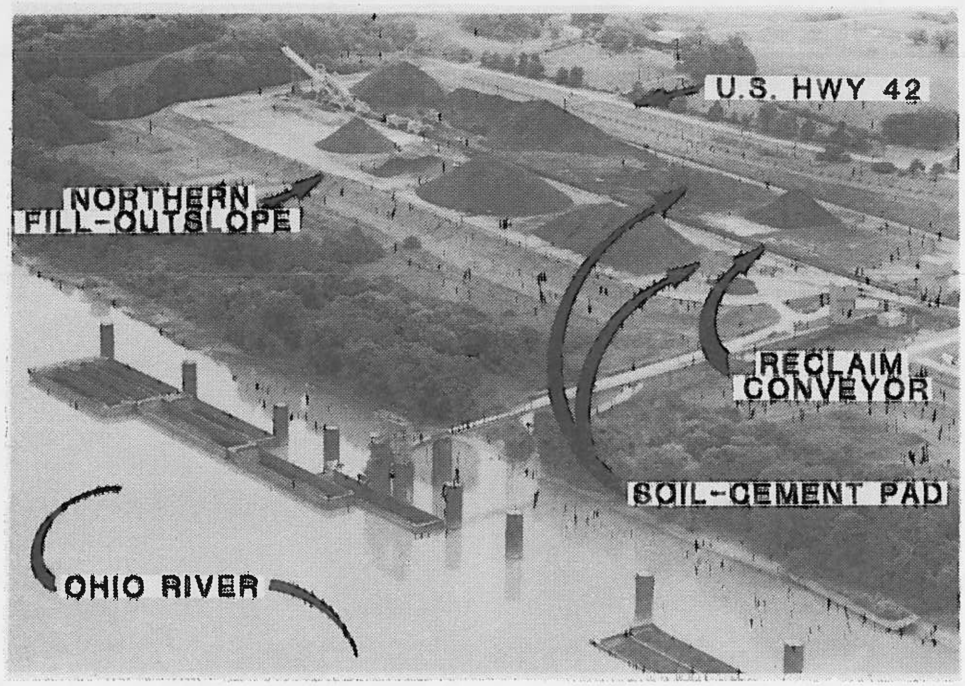

Figure 1. Aerial photograph of Cleancoal Terminal Facilities

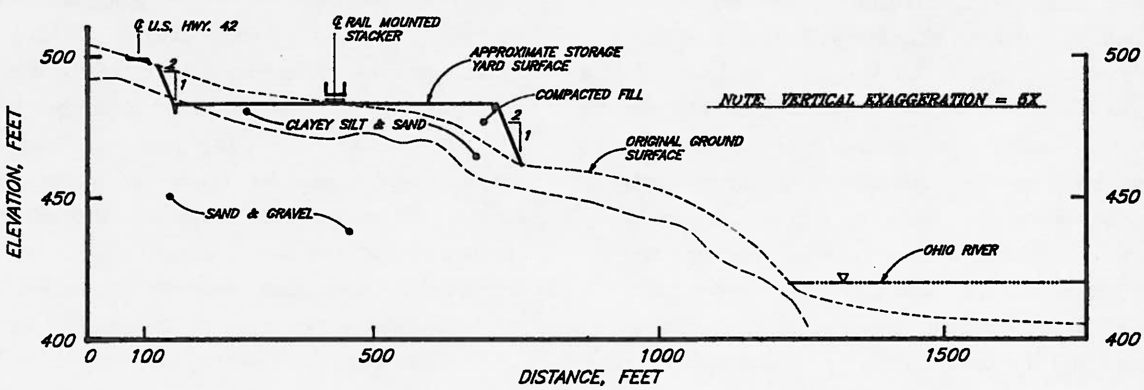

Figure 2. Sketch of generalized subsurface conditions

Realizing that a supply of sandy gravel borrow was available on-site, project personnel decided to investigate the soil-cement stabilization alternative in more detail. The Owner felt that considerable cost savings could be realized by utilizing on-site borrow and that the soil-cement would produce the desired slab-like surface. Additional studies were conducted whereby the previously recommended soil-cement thickness of 30 inches was reduced to 15 inches by providing a layer of synthetic reinforcing (Thacker, 1986). Laboratory testing indicated that increasing the cement content from 6% to 9% yielded a 47% increase in flexural strength. Moreover, an additional 60% increase in the flexural strength was observed for 9% cement content samples which were reinforced with Tensar SS-1 geogrid. Accordingly, based on these additional tests, the recommended surface treatment included an initial six (6) inch thick layer of compacted soil-cement containing 6% cement, covered with a single layer of Tensar SS-1 geogrid and finally surfaced with an additional nine (9) inch thick layer of soil-cement containing 9% cement. The cost for this surface treatment technique, according to a specialty contractor, was $0.70 per square yard per inch of thickness which resulted in a total estimated cost of approximately $840,000.

A design-build contractor, selected to perform the required construction activities, submitted an alternative yard surface detail whereby twelve (12) inches of soil-cement would provide an adequate working surface. The Contractor indicated that based on past experience and research, in-situ blending of the sand and gravel with 8% cement by weight, followed by compaction to 95% of the modified Proctor maximum dry density, would provide the desired surface. Due to the in-situ blending technique, no reinforcing could be installed since it would interfere with the mixing blades during the churning process. The Contractor provided a unit cost of $0.68 per square yard per inch thickness resulting in a total cost of approximately $650,000.

Discussions with operators of other large coal storage areas indicated that soil-cement was frequently used as a surfacing because of cost considerations even though it required periodic replacement of damaged sections. Considering the potential cost savings, the Owner elected to use the twelve (12) inch thick stabilized surface proposed by the Contractor. However, the Contractor had to agree to assume responsibility for the performance of his proposed alternative surfacing for a period of one year and to replace at his own cost any sections that failed during that period.

Specifications for placing and compacting subgrade material and construction of the soil-cement surface, as well as the quality control testing requirements were outlined and construction activities were initiated in August, 1986. As part of the design-build agreement, the Contractor was responsible for construction quality control monitoring. The concept of building several test sections, each with a different thickness of soil-cement with subsequent loading and monitoring was discussed. Unfortunately, because of the cost of mobilization of necessary equipment and the desired schedule for completion, construction of an elaborate test section field plot was not practical.

STORAGE YARD CONSTRUCTION

Based on construction records, silty clay overburden was excavated from the roadside of the yard and used as fill on the riverside following initial topsoil removal activities. As the yard surface was nearing final grade, it became apparent that insufficient fill was available from the roadside and the Contractor began hauling and placing sandy borrow obtained from other portions of the site to complete the fill. Consequently, in several locations the fill on the riverside of the yard consisted of interbedded layers of sandy silt and gravel within the silty clay layers. Throughout the earthwork activities, borrow material was air-dried by spreading prior to compaction due to the high natural moisture content of the material

Prior to full-scale surfacing activities, a small test pad was constructed on the roadside of the yard in an attempt to refine construction procedures and quality control monitoring activities. Although, the test pad was too small to be representative of full-scale construction, it enabled a general construction procedure to be developed. Afterwards, sandy gravel borrow was spread over the surface of the entire yard in a single twelve (12) inch thick layer. Thereafter, cement was spread over the surface in sufficient quantity to provide approximately 8% cement content by weight. The cement was mixed into the sandy gravel using a single pass of machinery capable of churning 12 inches deep. Afterwards, a small machine added the required water and churned the upper 8 inches of the sandy gravel layer. Finally, an additional pass with the larger equipment churned the wetted layer throughout the twelve (12) inch depth. Initial compaction of the cement treated soil was provided by several passes with a sheepsfoot roller followed by additional compaction with a smooth vibratory roller. Compaction criteria for the soil-cement was established as 95% of the modified Proctor maximum dry density at, or 2% above, optimum moisture content.

The completed yard surface was generally level along the yard length with each side sloped away from the centerline on a grade of 0.5%. A triangular shaped ditch three feet deep and lined with asphalt was constructed around the perimeter of the completed soil cement pad to collect surface runoff and direct it to a small sediment pond on the west side of the storage area. The area along the center of the yard beneath the rail-mounted stacker and conveyor was not stabilized with soil-cement. Instead, a three foot thick section of ballast was installed in order to provide a bearing medium for the rail ties. Perforated pipes were installed within the ballast section to provide drainage into the perimeter ditch.

PERFORMANCE MONITORING

General

Within six months after the storage area became operable, several localized areas of the soil-cement surface began failing. The failed areas exhibited servere cracking and rutting and in several of these locations, the distress was accompanied by a complete break-through of the slab as shown in Figure 3. For the most part, the failed areas were located on both sides of the central reclaim conveyor belt. Some adjacent

Figure 3. Photograph showing typical soil-cement pad failures

areas of the yard also exhibited deflection and pumping when traversed by the yard machinery. In addition to the soil-cement failures, surficial sloughing of the fill outslope on the north side of the yard began occurring. Instability of the outslope was noted along a 350 to 400 feet long section of the fill and progressive failures began undermining the triangular shaped perimeter ditch constructed around the storage area as shown on Figure 4. Although these problems did not threaten the continued operation of the coal storage facility, they did present troublesome working conditions and resulted in contamination of some coal stockpiles. In addition, breaches of the perimeter ditch created potential environmental problems by uncontrolled discharges of coal pile runoff water.

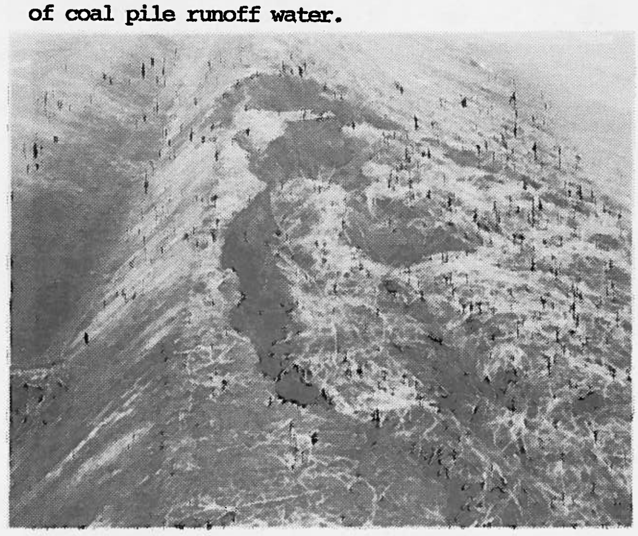

Figure 4. Photograph showing surficial sloughing of fill outslope

Causes of Failures

Failure of the soil-cement pad can be attributed to a variety of causes. The causes may have acted alone in some areas while acting in combination with one another at other locations. Generally, the following causes were identified:

1. Non-uniform and inadequate compaction of subgrade and presence of organic matter;
2. Weakly cemented localized areas indicative of non-uniform cement mixture;
3. Localized inadequate compaction of soil-cement surface and low compressive strength of test specimens; and,
4. Insufficient soil-cement thickness to carry service loads.

Specifically, test pits were excavated in several of the failed areas in order to assess the subgrade characteristics. A soft layer of gray clay containing organic matter from about 12 to 24 inches thick was observed at several locations. Moreover, in several areas, poorly compacted fill was encountered overlying organic soil. The poor subgrade was located primarily near the center of the site in areas which required little or no excavating or filling during the initial site grading activities (refer to Figure 2). At numerous locations, the sandy gravel was only weakly cemented, often accompanied by loose material at the pad surface. Construction monitoring records compiled during the soil-cement pad construction period indicated that about 9% of the field density tests failed to meet minimum compaction criteria. In addition, about 12% of the soil-cement test specimens failed to achieve the required minimum 7-day compressive strength of 1000 psi. Finally, observations of loading and unloading of coal with the end-loaders indicated that "dynamic" wheel loads were applied to the pad surface which exceeded the static loading of the machinery (Nussbaum, 1987). As such, the wheel contact pressures near the central reclaim conveyor were significantly higher than for other areas in the yard.

Instability of the fill outslope on the north side of the yard can be attributed primarily to a perched ground water level. Excavations into the slope indicated that perched water levels within the fill resulted in significant seepage at relatively high levels on the outslope face. The perched water levels appeared to be located in the interbedded layers of sandy gravel within the finer grained silty clay layers. The source of the perched water was apparently water that infiltrated through the surface of the 20 acre yard.

REMEDIAL REPAIRS

General

Remedial repairs were made to the affected areas approximately one year after initial construction of the yard. Initially, repairs were made to the fill outslope followed by segmental repairs to the failed soil-cement areas. All repair activities were performed while the coal storage area continued to function with only minimal disruption to yard activities.

Fill Outslope

Repairs to the affected outslope were made by installing seven (7) shallow synthetic drains at a spacing of about 50 feet where significant seepage was observed. As shown in Figures 5 and 6, the drains were installed down the face of the slope on an inclination of approximately 1½ horizontal to 1 vertical while the finished outslope was maintained at 2 horizontal to 1 vertical. Installation of the drains in this manner was intended to intercept seepage well behind the outslope face and safely discharge it near the toe of the slope. As shown by Figure 7, the drains dewatered the slope such that it could be regraded and reseeded.

Soil-Cement Pad

Identifying a single cause for failure of the soil-cement pad in any particular area was difficult considering the various deficiencies previously cited. Therefore, relatively small test repair areas were initially constructed using a variety of subgrade improvement techniques, replacement slab thicknesses and types of concrete. Subgrade improvements involved undercutting soft areas and organic material until a firm subgrade was encountered, then backfilling the excavation with compacted borrow. In several areas, the backfill was further stabilized using woven geotextile fabric or geogrid reinforcing. One test area was constructed using fiber reinforced concrete and

Figure 5. Photograph of excavated trench down fill outslope

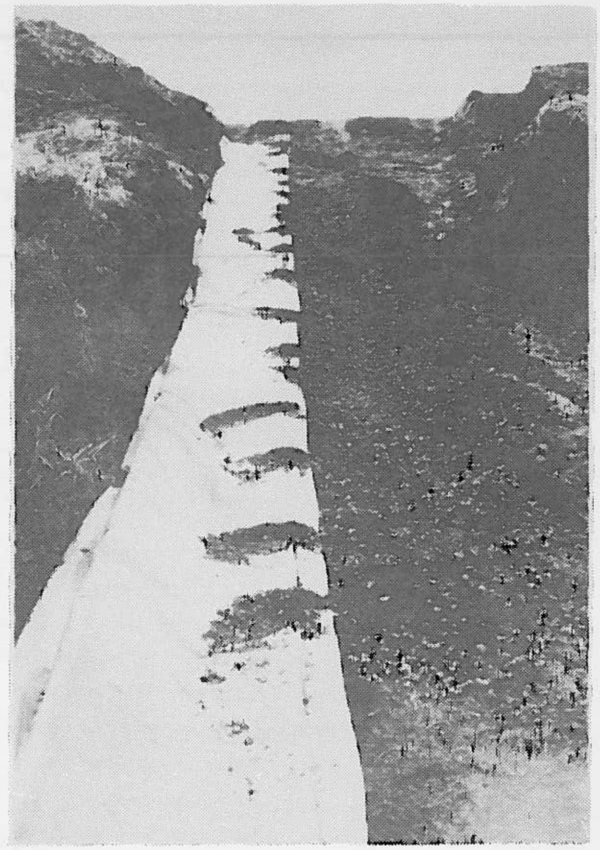

Figure 6. Photograph of shallow synthetic drain in prepared trench

Figure 7. Photograph of reconstructed fill outslope and ditch

another was constructed using a standard 4,000 psi concrete. Replacement slabs within each test area were constructed in 8, 12, and 18 inch thicknesses.

Following placement and curing of the concrete, each test area was proof-loaded by reclaiming several thousand tons of coal across each slab. The results of these activities indicated that performance of the 8 inch thick replacement slabs was marginal while the 12 and 18 inch thick slabs performed adequately with no signs of distress or deflection under load. Moreover, no significant performance differences were noticed between the differing concrete types, other than a lesser amount of shrinkage cracking within the fiber reinforced slabs.

Based on the performance of the test areas, a general repair procedure was established for subsequent areas. Initially, the soil-cement surface for a particular area was excavated and the exposed subgrade was proof-rolled to delineate soft areas. Those areas exhibiting deflections in excess of three (3) inches were undercut a maximum depth of three (3) feet. Thereafter, as shown in Figure 8, a single layer of stabilization fabric was installed and the excavation was backfilled with compacted cohesive borrow. As shown in Figures 9 and 10, areas adjacent to the central reclaim conveyor were replaced with an 18 inch thick slab and remaining areas were replaced with a 12 inch thick slab. A standard 4,000 psi concrete was used for all repair areas and tar filled expansion joints were saw-cut within 24 hours of concrete placement.

Throughout construction of the repair areas, concrete cylinders were molded and returned to the laboratory for compressive strength testing. As individual areas attained the specified strength according to the test specimens, they were released for resumed loading by typical yard activities. Subsequent periodic inspections of the replacement slabs indicated that all repair areas are performing adequately. More recently, several additional areas of the soil-cement surface have begun to exhibit distress and will require replacement in the near future. Nevertheless, based on the performance of the areas repaired to date, it appears that a relatively simple repair procedure has been established that can continue to be used in the future. Moreover, the repair techniques are such that they can be performed by company personnel on an "as-needed" basis.

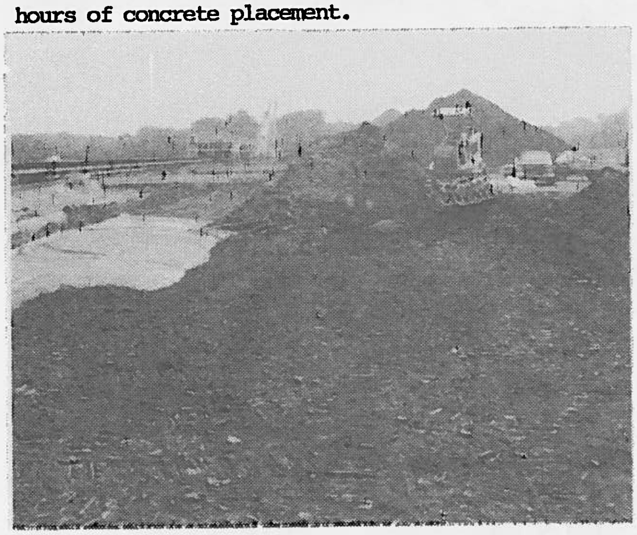

Figure 8. Photograph showing stabilization fabric and backfill within pad repair area

Figure 9. Photograph of concrete repair section

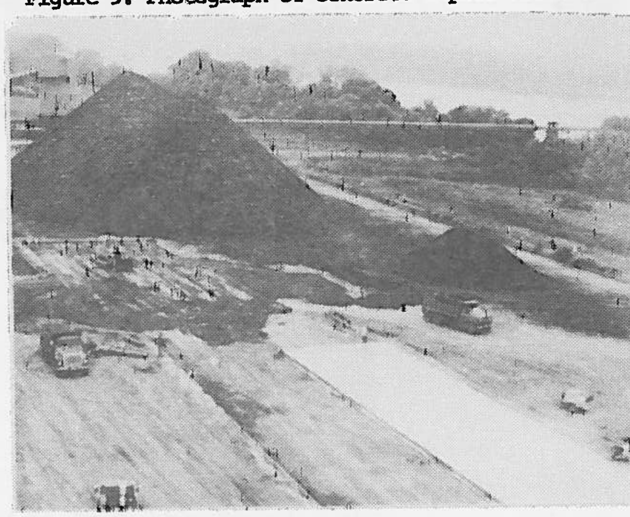

Figure 10. Photograph showing segmental pad repair activities

ECONOMIC ASSESSMENT

Thus far, repairs made to the soil-cement pad have involved replacement of approximately 100,000 square feet (14%) of the yard surface at a cost of approximately of $350,000. This amount includes the costs associated with the required outslope repairs. Considering that additional soil-cement areas are currently experiencing distress, it is reasonable to estimate that annual maintenance activities will include replacement of about 10,000 square feet of the yard surface for each of the next four years. Accordingly, the cost of the yard installed at Cleancoal Terminal in terms of present worth dollars, as of August, 1986, is about $1,090,000. This cost is still significantly lower than the other options included in Table 1.

CONCLUSIONS

Based on the information contained herein, the authors offer conclusions as follows:

1. Construction of stabilized industrial storage yard surfaces using cement treated soils can be performed in a cost effective manner, however, conservative design and stringent quality control practices are advised.

2. Shallow synthetic drains provide a cost effective means to stabilize surficial sloughing caused by seepage of perched water. Such a system can be readily expanded if additional seepage develops.

3. Repairs to soil-cement using cast-in-place concrete present a relatively simple repair scheme that can continue to be used for subsequent repairs without disrupting the operation of the yard.

4. The effect of "dynamic" loading caused by yard machinery, should be considered when determining the thickness of the soil-cement to be used.

5. During earthwork activities, special attention should be given to compaction of the subgrade in the area along the contact between excavation and fill.

6. Interim storage yard surfaces can often result in economically affordable facilities by reducing initial capital costs, however, the cost of subsequent repair and replacement should be considered in any decision making process.

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Utilization of incenerator ash

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Abstract: The past 35 years have seen a tug of war between land disposal of municipal solid waste, and incineration. It has been the cost and air pollution involved with the incineration process, versus the odor, unsightliness, and most importantly, public objection to having metropolitan waste sanitary landfills located close to their homes or their industries.

In the middle 1950's, the problem of disposal of the flue dust and bottom ash from rotary kiln incineration was brought to the author's attention by his attorney, who was also attorney for a group of disposal firms who had installed a rotary kiln incinerator, and were faced wtih 300,000 tons of residue sitting in their backyard.

After approximately a year of contemplation and experimentation, a method of stabilizing the material, creating unconfined compression strengths on the order of 300 to 600 psi, was developed. It has been improved through the years.

The Chempac process for producing a lime stabilized incinerator residue as a base course for parking lots and other pavements is the result of this work.

The first guinea pig for the new idea was in fact the author's own firm, which installed a Chempac base course in the company parking lot in 1962. The parking lot survived the life of the structure, which ended in 1987, when a new building was built on the site, into which the firm has moved. There were no instances of subbase failure, even though frequent passage of heavy trucks in the larger part of the parking lot was a daily occurrence. While the basic ingredient for stabilization has been hydrated lime, waste limes have also been used successfully as well as Type C flyash.

Research is presently under way to see what the effect of the lime additive will be on the heavy metals known to exist in incinerator ash. Preliminary data suggests that the toxicity factor for the most important heavy metals will be reduced by the lime treatment.

INTRODUCTION

The introduction for this paper should consist of copies of dozens of newspaper articles that are found almost daily, regarding opposition to landfills and/or solid waste incinerators.

Every so often, a local group generates pressure to create a recycling system, to avoid landfills or incinerators.

There is a big problem, and it is getting worse.

The author had an early exposure to part of the problem in the late 1950s, when he was introduced to the operators of the rotary kiln Volund incinerator at 2815 S. Laramie Avenue in Chicago, Illinois by his attorney and tennis partner, who also represented the Dutch scavengers who owned the incinerator. The incinerator occupied a piece of property owned by the 60 scavengers, while the residue was being deposited daily on five acres adjoining the incinerator property, a piece of property owned by one individual. It had accumulated to approximately 300,000 tons.

The residue consisted of fly ash and bottom ash in a mixed condition, having been placed in the pile over a number of years. The proposition to the author was a challenge to figure out what to do with the residue.

The assignment was uncompensated, therefore resulting in the obtaining of less data than would have been obtained were it to have been an adequately funded research project.

This paper describes some of the information that has been obtained with regard to basic data and on data related to using stabilized incinerator ash as a base course for flexible pavements and parking lots, and other light-use purposes.

The Big Problem

Everybody produces wastes in the form of garbage, generally known as municipal solid waste. People are willing to pay for getting rid of it, but refuse to have it buried or incinerated near their home or office.

Even if vaporization of municipal solid waste were found to be possible, that would fly straight into the sensitivities of those concerned with air quality.

In other words, you can't get rid of the problem easily, or even economically, considering the scarcity of landfills and incinerators.

Unfortunately, to top off the real technical problems of getting rid of wastes, enough people, private citizens, as well as corporations and solid waste disposal firms, have in the past been quite casual and indifferent toward the protection of the environment for years, resulting in almost a certain hostility toward disposal of solid wastes or incinerator ash on any property, in any form, by any of the fly-dumpers of the past, in spite of efforts by such firms to behave responsibly in recent years.

More recent concerns, since Earth Day, have dealt with the chemical components of solid wastes and/or residue from incineration. To make matters worse, the objectivity of some test procedures that have become national standards, to realistically portray the appropriate levels of risk, is recognized to be missing. In other words, there have been a large group of "objectors" who listen to no common sense since they were born with their minds made up and everyone else is wrong. Many such win-lose, linear thinkers are in governments at all levels, as well as private individuals.

Since that type of win-lose thinking enters into the legislative process, where few people know or understand the technology involved, there is obviously an almost impossible situation existing today. For example, even though heavy metals typically are below permissible maximums in residues from most incinerators, the problems of disposing even that residue are almost insurmountable because of interference tactics by the objectors, compliance with standards notwithstanding.

However, as someone once said, "tis better to light a candle than curse the darkness".

The lit candle, in this case, is the recycling of the incinerator ash as a component of flexible pavements, comprising the base course.

This is the idea that came to the author in 1958. However, even in those days, the toxicity issues were thought about in deliberations concerning Chempac, since they were a professional concern of a person who wants a clean environment as much as do the objectors. It was felt that the process provided a cementing action that would lock or bind any heavy metals into the structure of the Chempac base course material, as well as provide strength to the mass.

Furthermore, with asphaltic concrete surfacing on the Chempac base course, vertical percolation of water would be at such a minimal rate that leaching action would be essentially zero. The water would run off the pavement before it had a chance to penetrate through the pavement, except of course at cracks.

The tests performed during the initial use of Chempac were not as extensive as have been run recently, because the subject of heavy metals, etc., had not reached the public attention.

The Test Program - 1958

After all involved saw the cemented Chempac material, and the test results, it was agreed upon between Incinerator, Inc., the owners of the incinerator and STS, (Soil Testing Services in those days), that Incinerator, Inc. would install a test road on 38th Street, extending several hundred feet west of Laramie Avenue. The test strip was one-half the width of the street and consisted of 10 inches of Chempac.

It was, of course, surfaced with 1 1/2 inches asphaltic concrete.

In spite of the fact that heavy trucks crossed the test strip everyday, heading for an industrial facility toward the west, the test strip performed beautifully, with no evidence of any types of failure.

At this point enthusiasm on the part of Incinerator, Inc. as well as the author, led to their purchase of a lime bin and a pug mill mixer.

A local paving contractor collaborated in the installation of half a dozen parking lots, including one at the main office of STS, which was constructed in 1962.

While care was taken in most instances to select parking lots that did not have a probability of large truck loads, nevertheless, the performance was good.

Problems were related primarily to weak areas in the subgrade, improper asphaltic concrete surfacing, which resulted in shrinkage cracks, and use of fresh residue, which tended to create "bursts" as gas was generated by the chemical activity in Chempac, as the material hardens.

Weather, of course, played a part in some of the problems, as it does with all paving. There appears to be some sort of a hidden force that makes paving contractors try to pave jobs that you are involved with, immediately after rainfall or during freezing weather.

On one job in 1973, the site was prepared in June for paving. However, it was exposed to rain and sun for an entire summer and fall. Paving was started the day before Thanksgiving, during a snow storm, after 1/2 inch of rain the day before. As Murphy said, "If things could go wrong, they do!"

Some of the problems in scheduling might have resulted from pressure from Pozzopac contractors, who in those days had a good market for their product. They were, however, intent on proving, or at least asserting, that Chempac bore no relationship to their Pozzopac material. This went so far as to test the incinerator fly ash to determine how it's pozzolanic characteristics compared to similar properties of fly ash from the combustion of powdered coal. Conclusions at that time were that there were few similarities. The incinerator fly ash did not qualify under any circumstances under the criteria for fly ash to be used with Pozzopac.

There still seems to be some tendency in the industry to think of the fly ash from the incineration of municipal solid waste as being a material identical to, or certainly very similar to, that from the combustion of powdered coal. A test program to show the difference might answer the question, while not particularly serving any other useful purpose, except to perhaps permit extension of some of the experience with Pozzopac to the utilization of incinerator residue.

The attempts to use the Chempac material continued in Chicago into the middle '60s, with two new parties on the scene doing the actual installation work. There had been a change made because one job was installed by the first contractor omitting the lime. Obviously the author is convinced that no Chempac should be placed without the direct supervision of someone experienced with the material, for fear of a repeat of some of the undesirable experiences on past jobs.

Even in such cases, one project was installed under the direction of a person experienced with soil cement for 25 years, and a commercial testing laboratory who insisted on placing the material to 90% of the Standard Proctor density, instead of using 95% of Modified Proctor density as has been used in all of the work by the author. The inability of the experienced engineer to understand that the much greater density involved with the Modified Proctor test was necessary to achieve the hardening effects demonstrates the risk that the Chempac material would not be properly placed without supervision of somebody who cares about the performance and understands the technology. Typical civil engineering experience with soil-cement or soil-lime is likely to mislead that civil engineer in many ways.

For approximately five years, Great Lakes Asphalt Company represented the Chempac material. Several jobs were installed, one of which is the project that lay dormant during the hot summer months with a good subgrade and then was commenced at Thanksgiving time, due to the scheduling problems of the contractor picked for the project. Unfortunately, Great Lake's Asphalt interest in the matter ended with the death of the president and chief operating personnel in a plane crash.

At that point, arrangements were made with a minority contractor, Associated Contractors, who actively pursued projects for a period of a year or two, only to see the termination of incineration after the change in public attitude towards air and land pollution. Incinerators were found to be major pollutants and were shut down, eliminating the source of residue.

The 38th and Laramie plant and the City of Chicago 39th and Iron Street plant, both of which produced residue for Chempac, were both shut down, leaving only the Northwest Incinerator as a producing unit. Unfortunately, the operations at the Northwest Incinerator, perhaps due to the demand to feed solid waste faster than it could be properly combusted, resulted in improperly constituted residue which had too high a content of organic material.

Also, the metallic and large materials were not being separated from the bottom ash and fly ash.

Thus, the minority business venture with the Lawndale Peoples Planning and Action Council and Associated Contractors, after submitting a number of proposals, closed down operations because of lack of material to use.

In 1978, as a matter of major interest, the Lawndale High School project, let out for contract by the Public Building Commission, included use of Chempac for the parking lot. Since there was no contractor to do the work and no residue immediately available, it was necessary for the author to try to locate piles of residue from prior activities, which was readily accomplished.

Under the continuous supervision of the author, lime was mixed with the residue and trucked to the Lawndale High School site, where it was placed under, again, constant supervision.

Nevertheless, there were several small areas where the subgrade was not properly prepared, that caused some later cracking, though the performance of the entire project was satisfactory. A report on this project, citing the problems as well as the successes, is available, if one would like to receive a copy from the author.

A point of major interest regarding this project is that the architect, Andrew Heard, called the author to find out how the work would be done, since the contracts had been let. The author suggested that another material be used. This prompted another phone call from Mr. Heard stating that it would cost $28,000 more money to do the project using crushed stone than it would to use Chempac at the price agreed upon several years earlier.

Thus, the author was immediately aware of the economic advantage of Chempac, providing of course that there is material available to be used.

The Federal Highway Administration has taken a considerable interest in utilization of incinerator residue, including asphalt stabilization, aggregate for asphaltic concrete, and Chempac.

The use of the incinerator residue for aggregate in asphaltic concrete was complicated by: the need for much more asphalt than would be used for crushed stone aggregate; the variability of the material in terms of asphaltic content, resulting in slippery areas and dry areas; and the presence of nails and glass that certainly offered little in the way of an advantage to the material over other aggregate material. Furthermore, since the asphaltic content was the major cost of asphaltic concrete, not the residue, and because of the need to use higher than an average amount of asphalt, there were no cost advantages, only the advantage in disposing of the incinerator residue in a very good and useful recycling process.

The advantages of Chempac, however, involved a substantial reduction in the cost, including a reduction in excavating costs because the base course thickness using Chempac was generally 2 inches less than the typical crushed stone base course.

The contractor placing the 1978 Lawndale High School project had found the Chempac material much easier to spread and compact than crushed stone. The only disadvantage, he advised, was in the wear and tear on the tires, due to the nails, other metals and glass in the residue. While this one item was substantially increased percentage wise in its cost, it was a very minor item in terms of the total cost.

In 1975, a contract was negotiated with the Federal Highway Administration for additional laboratory testing of Chempac and a field demonstration in St. Charles, Illinois, all of which is reported in a paper available through the Federal Highway Administration entitled "Lime Treatment of Incinerator Residue".

However, in spite of this interesting project, which was funded by the Federal Highway Administration and which included the production of a video tape of the Chempac material being placed on the project, activities have ceased in the past 15 years, essentially because the incinerators were all shut down from a total of 4,000 incinerators in the country prior to Earth Day 1970, to less than 100 a few years ago.

Chempac Today

After more than 15 years of near dormancy regarding the Chempac process, the development of more efficient incinerators including walking grate incinerators as well a rotary kiln incinerators that were involved with the original projects, and the rapid decline in the availability in landfills to accept municipal solid waste, Chempac appears to have much greater potential than it did in the '60s. It serves as a method of recycling municipal solid wastes, much more economically feasible than the sorting of glass, cans, metals, and plastics in one's back yard. The Chempac process has been documented with data already available. However, much more is to be obtained using techniques resulting in a reduction in the lead and cadmium in leachate tests, as measured with the EP toxicity tests.

Even though the samples tested to date without the addition of lime have all shown levels of heavy metals below the maximum allowable set by the EPA, nevertheless, the above two heavy metals have been reduced due to the effect of the lime added in the Chempac process on the chemical activity in the residue.

While this effect was intuitively assumed during the early days with Chempac, test data now shows the lime additive to be a beneficial element of Chempac in reducing heavy metals.

Plans are presently being made to install a parking lot at the incinerator plant in Harrisburg, Pennsylvania under the management of William Strauss of the City of Harrisburg staff, and Leslie Davies of Nassaux-Hemsley as consultants.

Such testing and applications will be planned at several other locations, it is hoped.

Substantial numbers of tests were performed in the 1950s and early 1960s involving Volund rotary kiln incinerators around the country, which showed widely diverging results of Chempac effectiveness with the different materials. Where the fly ash was eliminated from the residue by other techniques, the material failed to harden. It is a necessary ingredient, although it is established that it is <u>not</u> the same fly ash that one would have if <u>one</u> were to purchase such from a coal fired power plant.

Nevertheless, the important thing is that all residues in different locals seem to have their own characteristics.

The highest strengths that have been measured were on residue from the Wheelobrater plant in Saugus, Massachusetts.

The important element appears to be the thoroughness of the burnout. Of course, the attitude of the operator of the plant to make sure that the burnout is good is all important.

At the present time there is a great debate going on at EPA and elsewhere in the various states about the requirements for disposal of incinerator residue. Although the author is not intimately familiar with these debates, he has heard discussions of the problems for hours on end, sufficiently to know that somewhere out of this debate should come some common sense regulations.

At the moment of preparation of this paper, it is understood that municipal solid wastes are not classified as toxic and hazardous, nor should their residues. But nevertheless, the residues should be subjected to special treatment in disposal according to regulators.

There are many that think the immunity of the basic municipal solid wastes from classification as toxic materials should extend to the residue.

Nevertheless, it appears that the utilization of the residue in combination with lime, and other additives that are available to increase the strength or change the properties, seems to put the Chempac material front and center as a potential candidate for recycling. This would not only reduce the cost of disposal which in many areas exceeds $15 per ton for the incinerator ash, but also would result in a reduction in the cost of paving, providing of course those involved have the good sense to use the material for parking lots that are not subjected to frequent and heavy, high speed traffic. The Harrisburg parking lot which is anticipated, hopefully will demonstrate performance under heavy wheel loads.

The author welcomes inquiries and suggestions regarding Chempac and its use.

Chemical and mechanical stabilization of railroad subgrades

by

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Abstract: Chemical and mechanical stabilization of railroad subgrades is an important part of the overall track maintenance program on Norfolk Southern. Engineering evaluations and investigations of problem locations determine the preferred method of stabilization to be employed and the work is accomplished so regular train schedules are maintained.

INTRODUCTION

The tracks of Norfolk Southern Corporation traverse twenty states in the Eastern United States and encounter many geologic settings and subgrade types. These systems supporting the track present a wide variety of subgrade problems which add to track maintenance. To combat the unstable track locations, several soil stabilization techniques, both chemical and mechanical, are being employed to improve the roadbed. Some of the subgrade stabilization programs are of large magnitude. For instance, more than 406,000 feet of rail piles are driven each year in order to enhance embankment stability. Moreover, the railroad injects more than 18,000 tons of various grout types per year. The techniques and requirements for the stabilization usually cannot be analytically analyzed but the railroad has a great deal of experience in applying these stabilization techniques on an empirical basis.

Sand cement injection grouting is being used to enhance the stability of railway embankments. This stabilization procedure is normally used on fills that are consolidating due to poor compaction, inferior fill materials, and increased traffic density. A slurry of sand and cement is injected into the fill slope in a regular pattern normally working from the base and proceeding up the slope. All work is done off-track so no interruption to train traffic occurs.

Lime flyash pressure injection is being performed on track subgrades where top of rail geometry is difficult to maintain. Excessive moisture, expansive clay soils and poor fill compaction often contribute to unstable track. The lime flyash injection stabilizes moisture content and provides internal bonding to increase bearing capacity. The injection stabilization work is performed by on-track equipment working on yearly programs in conjunction with other track maintenance work.

Although several methods of mechanical stabilization are being used, the most common method is the use of rail piling driven into embankments to reduce movement and provide adequate roadbed section. Lines of scrap rail are driven to refusal at the shoulder point on both sides of the track; sometimes these two lines of rail piling are mechanically tied together. In almost all cases, these rail piles are empirically specified by field personnel; however, on a few occasions, consultants to the railroad have been able to provide some analytical analysis of the effect of various spacings and lengths of the piles.

Nonwoven geotextiles are used extensively to provide separation, filtration and reinforcement in the railroad subgrade. On new construction, the geotextile is used to separate the finished subgrade from the subballast and ballast. The geotextile prevents the migration of subgrade fines into the ballast. The fines, if permitted to foul the ballast, reduce its permeability and shear strength, thus leading to increased track maintenance. In rehabilitation of existing track, geotextiles are installed on all switches, road crossings and rail crossings to extend the service life of these high maintenance areas. Any track undercutting to improve ballast and drainage is accompanied by geotextile installation to prolong the maintenance cycle.

The following sections discuss the stabilization methods described and trace the history of their development and use on Norfolk Southern.

SAND CEMENT GROUTING

Early stabilization files indicate that several types of subgrade treatments preceded conventional sand cement grout that is being performed today. As early as 1931, Southern Railway - an operating subsidiary of Norfolk Southern - owned and operated a one cubic foot Ransome Pneumatic Grout Mixer and Placer. This machine was used primarily for repairing defective masonry but was, on occasion, used to inject a mixture of cement and concrete sand through steel pipes into railroad fills for displacement of water and fill stabilization. This machine was used until about 1951.

Several other railroads, namely Baltimore and Ohio, Burlington, Sante Fe and the Virginian, began to use commercial equipment designed specifically for efficient volume grouting in the 1940's. In 1951 Southern Railway engineers visited these railroads and developed a grouting program to put into use on Cincinnati, New Orleans and Texas Pacific (CNO&TP) mainline tracks from Cincinnati, Ohio to Lexington, Kentucky.

At the same time, on other operating divisions of Southern, two other techniques utilizing sand were being used for roadbed stabilization. Portions of these processes were later incorporated into the sand cement injection grouting method.

In 1946 at two locations in Alabama, soft yellow underclay underlain by irregular rock formations was causing heavy maintenance expenditures and interference with train operation. A technique was developed to stabilize the track by driving a 12" x 12" timber spud, ten to twelve feet long, into the ground near the end of the crossties to a depth that would permit its withdraw with ease. Clean coarse sand was then placed in the hole created by the spud and the spud was driven again through the sand so that a mixture of the clay and sand was accomplished. It was determined that a 20% treatment of sand, calculated from surface area, proved to be most effective after tests at 10, 20, 30, and 32 percent sand were investigated. The work was done on fills up to ten feet in height with successful results (Brosnan, 1946).

At approximately the same time on the Washington Division in Virginia another type of roadbed stabilization was being done to correct locations that had to be surfaced two to three times per week during wet seasons. This procedure involved driving a 1 3/4" diameter rod into the track subgrade or fill surface to create a hole and then a small charge of dynamite was dropped into the hole and detonated. The cavity produced by the blast was then blown full of dry sand using an air compressor and sand blast machine. This procedure stabilized the trackbed by lowering the water table and consolidating the fill materials. Records indicate that an average of 1 cubic yard of sand was injected per three linear feet of track and shot holes averaged one per linear foot. (Fox, 1947). In order to utilize this method of stabilization, preparatory work was done consisting of cross sections and borings made along cross section lines. Soil samples were taken for laboratory testing to determine moisture and density. After two years work (1947-1949), approximately 41,000 linear feet of track had been stabilized on the Washington Division.

In the establishment of a long range sand cement grouting program, the three methods described above were merged using the most beneficial parts of each into a system of high production permanent stabilization.

From an extensive soils investigation program conducted in 1951 on Southern mainline track from Ludlow to Rogers Gap, Kentucky, it was believed that stabilizing fills composed of shale with shrink-swell clay and cinders would be dangerous using the dynamite technique or the spud driven sand pile method. It was decided that a sand cement grout mixture injected by hydraulic rather than air pressure would be most beneficial. Although this method was calculated to have a higher initial cost, the permanent reduction in long term maintenance costs gave it the best overall unit cost.

A four year grouting plan was formulated using high production equipment that was available and being used on other railroads. A Koehring mud jack was purchased and a gang established consisting of one foreman, two operators and fourteen laborers. As the mud jack had a high capacity, several laborers were required to drive and pull grout points on the fill slopes. The injection points varied in length from 6 to 16 feet and often had to be driven from a scaffold.

The sand cement ratio varied between 12 to 1 and 6 to 1 based on conditions of the fill and the weather, especially the temperature. Emulsified asphalt was also used to reduce the cement content, protect the machinery from sand scour, reduce friction in grout lines and prevent segregation of cement in the mix. The quantity of emulsified asphalt used was 0.1 gallon per cubic foot of sand. Emulsified asphalt was later omitted from the mixture when the cement factor was increased to offset variability in the quality of materials.

The slurry mixture was forced into the embankments to refusal, reaching poor material and water pockets a considerable distance from the injection point. Work was started at the toe of fills and progressed on a grid pattern of approximately 10 feet X 10 feet until the embankment was covered. The mud jack usually remained at the top of the embankment with slurry being piped to the injection points.

The cost savings of the sand cement grouting work were immediately realized as section track labor crews were reduced in the stabilized areas resulting in savings of $89,000 in 1953 (Wearn, 1954). Further mechanization occurred with the use of batch bins for sand and cement, and the addition of a second mud jack pump and refined methods of injection pipe installation.

From the work developed in Kentucky in the early 1950's, cement grouting work expanded to other railway divisions and progressed with the development of higher capacity grout machines, improved injection driving apparatus, and the utilization of better materials.

Current sand cement grouting work on Norfolk Southern is done by three gangs consisting of five men each. The equipment now in use was built by company personnel and consists of a grout machine with two - 1 cubic yard mix tubs. The volume of grout material injected averages approximately seven cubic yards per hour and is pumped with a Moyno screw pump that develops 600 psi. The injection pipe driver utilizes an air hammer mounted on a mast for driving 18 foot pipe into the embankment. The mast is mounted on a steerable rubber tired frame equipped with an air winch to facilitate moving.

The higher capacity machinery has required the development of grouting materials that are compatible with grouting operations and embankment materials encountered. Type I cement has been used for many years in grouting operations because it is readily available and reasonably priced. With the large volume of material being used, however, substitutes for cement have been investigated. A blend of cement and flyash was tested but was not acceptable in handling. The current product being used consists of 20% cement and 80% ground limestone dust blended in a bag mixture. A 2 to 1 ratio of sand to cementitious material is currently being used as a standard ratio with variations to meet local conditions. Test results performed on sand/cement cylinders produced strengths of 1240 psi for 7 day breaks and 2410 psi for 28 day breaks. The grout mix/sand cylinders produced strengths of 740 psi for 7 day breaks and 1000 psi for 28 day breaks. The grout strengths from the current product are sufficient for the strengthening of soil embankments. A copy of the Grouting Materials Specification is included in the Appendix. In 1987, 4270 tons of cement blend material was used by company forces.

Sand used for grouting is critical to the success of the injection. A coarse sand can easily cause grout lines to become plugged resulting in costly downtime. In areas where masonry sand is not available, substitutes of manufactured sand, ground limestone and agricultural limestone have been used. In 1987 over 8,000 tons of sand were used for grouting operations.

In addition to embankment stabilization, several other uses have been made of the sand cement process. In areas where roadbeds traverse limestone bedrock, sinkholes often develop and cause damage to the roadbed. Grouting the area of the sinkhole along with any backfill placed in the collapsed zone stabilizes the track roadbed. Underground fires have been extinguished by injection of sand cement grout into areas of underground coal mines and locations where coal waste and cinders were used for embankment. The grout seals off sources of air necessary for continued burning and provides an additional measure of strength to weakened embankment material. Railway causeways constructed adjacent to or across lakes often become unstable due to constantly changing water levels and removal of fine material from embankments. The use of grout has been successful in several such locations to protect the railway.

The development of a sand cement grouting program has spanned several decades and advanced with the advent of modern equipment. Increased emphasis placed on the engineering evaluation of materials and embankments has improved the techniques and has provided the railway a method to decrease maintenance and improve the roadbed.

LIME FLYASH PRESSURE INJECTION

The use of lime to stabilize subgrade soils is not a new process. Clay soils used in construction of subgrades in Texas and Louisiana were treated with lime in the 1940's and 1950's. The reaction of lime or lime flyash with fine grained soils involves cation exchange and agglomeration/flocculation and produces immediate changes in soil plasticity, workability swell and immediate (uncured) strength and deformation properties.

Depending on the characteristics of the soil being stabilized, a soil-lime pozzolanic reaction may commence. The addition of flyash aids in this reaction as flyash is a pozzolan. The pozzolanic reaction results in the formation of various types of hydrated calcium silicate and calcium aluminate cementing agents. Pozzolanic reactions are time dependent and strength development is gradual but continuous for a long period of time.

The reactions occur at positions where lime is in contact with the fine grained soils, such as around the periphery of a drill hole or along the seams created during pressure injection of lime. The migration and diffusion of the lime in the soil causes an increase in the volume of soil affected (Robnett and Thompson, 1975).

The use of lime and flyash for roadbed stabilization purposes has an interesting development on Norfolk Southern. The first recorded use was on the construction of Louisiana Southern trackage in 1959-1960. The lime was conventionally mixed into the roadbed and compacted on this new construction. Beginning in 1966, a process of drill lime treatment was begun on existing roadbeds where local shallow subgrade failures or "squeezes" were occurring at the ends of ties. In this process a pattern of 6-inch diameter holes was augered to a depth of 4 to 6 feet and filled with dry lime. In some areas flyash was also added with the dry lime in the drill holes to help with the stabilization of the subsoil. Water was then added and mixed to form a slurry. After mixing, the holes were loosely filled with cuttings (Farris, 1970).

A pattern of holes was developed to give equal treatment to the track structure through the failed area. Typical longitudinal spacing was 3'-6" and transverse spacing was 2'-0". A typical area was drilled in seven lines, parallel to the centerline of track for the distance of the subgrade distress. The holes were staggered to give better coverage.

An important part of the lime treatment of subgrade soils is a chemical analysis of the soil to determine the reactivity with lime. A simple ph test may be run to determine the reaction of the soil and the quantity of lime needed to stabilize that soil (Farris, 1970).

Atterberg limit tests can also be performed on soils in the raw condition, 3% lime by dry weight, 5% lime by dry weight and 7% lime by dry weight. A marked reduction in the plasticity index usually occurs with the addition of 3% lime and increased reduction with addition of the 5% and 7% weights. A small or negligible change in the plasticity index would indicate that the soil is not reactive to lime treatment. Several series of soil-lime reaction tests have been run on soils from various geographic regions and are shown in Table 1.

Table 1. Test results of Atterberg limits tests

	Atterberg limits							
Sample	Raw		3% Lime		5% Lime		7% Lime	
Location	LL	PI	LL	PI	LL	PI	LL	PI
Illinois	38	20	36	18	37	18	35	14
Illinois	47	27	48	25	37	20	35	17
Tennessee	58	33	50	28	43	25	39	20
Tennessee	35	16	30	12	31	12	28	8
Alabama	54	27	42	23	40	20	34	14
Alabama	72	44	62	38	51	31	39	20

In 1971 representatives of Southern Railway visited a site in Texas where a contractor was stabilizing track subgrade using pressure injection of a lime slurry with equipment specially modified to operate from the track. From subsequent inspections and soils investigations of sites on Southern, a program of lime and lime flyash injection was initiated in 1972. The areas first treated were between Greensboro and Sanford, North Carolina. Lime slurry pressure injection stabilization work has continued on the tracks of the Norfolk Southern until the present time.

Over this long period of injection treatment, all facets of the work have been refined. As early as 1974, specifications for equipment, production and labor were prepared and competitive bids were received for the work. The current specifications reflect experience gained in long-term work with contract and company personnel and address the work situation of this specialized operation. A copy of the Norfolk Southern Specifications for Railroad Stabilization with Lime Flyash Slurry Pressure Injection is included in the Appendix.

The equipment typically used to accomplish lime flyash injection consists of two on/off track vehicles. One is equipped with injection rigging capable of making three simultaneous injections to a minimum depth of twenty feet and having a 2000 gallon slurry tank onboard. The second truck is also equipped with a 2000 gallon tank and is used to shuttle slurry to the injection rig. A slurry tractor trailer truck with 4000 gallon capacity is required to transport slurry from the mix plant site to the injection or shuttle truck in order to keep the injection work in full production. A mix plant generally consists of three 17,000 gallon mix tanks with mechanical agitators, and material handling equipment-conveyors or vacuum rigs - along with pumps and power plants and is usually set up at a rail siding near the proposed stabilization locations.

Hydrated lime and class F flyash used in the pressure injection are supplied in covered hoppers by Norfolk Southern. The hydrated lime is specified to have a minimum purity of 90% $Ca(OH)_2$ and contain less than 5% MgO. A typical analysis of hydrated lime used is shown in Table 2.

Table 2. Hydrate lime analysis

LOI	23.55%
Available Lime Index	72.20% as CaO
Total CaO	74.32%
SiO_2	0.80%
R_2O_3	0.60%
MgO	0.56%
Fe	0.25%

The flyash used in the slurry injection may vary considerably in chemical analysis, since the flyash is a by-product of the coal burning process at power plants. The location of the flyash supply may have more influence on the use than the flyash class or chemistry. Class F flyash is most readily available on Norfolk Southern and is used for slurry injection. Typical physical and chemical analyses of flyash are shown in Tables 3 and 4.

Table 3. Flyash physical analysis

PARAMETER	RESULTS	ASTMC618 SPEC.F/C
Amount Ret. on No. 325 Sieve, %	16.4	34 Max
Pozzolanic Activity Index		
P.Cement at 7 Days, % of Control		
P.Cement at 28 Days, % of Control	92	75 Min
Lime at 7 Days, psi	1054	800/NA Min
Water Requirement, % of Control	98	105 Max
Autoclave Expansion, %		0.8 Max
Specific Gravity	2.31	
Increase of Drying Shrinkage, %		0.03 Max
Reactivity with Cement Alkalies		
Reduction of Mortar Expansion, %		
Mortar Expansion, %		0.020Max

Table 4. Flyash chemical analysis

PARAMETER	RESULTS	ASTMC618 SPEC.F/C
Silicon Dioxide (SiO_2), %	48.0	
Aluminum Oxide (Al_2O_3), %	28.0	
Iron Oxide (Fe_2O_3),%	11.6	
Sum of SiO_2, Al_2O_3 and Fe_2O_3, %	87.6	70/50 Min
Calcium Oxide (CaO), %	2.9	
Magnesium Oxide (MgO), %	1.28	
Sodium Oxide (Na_2O), %		
Potassium Oxide (K_2O), %		
Sulfur Trioxide (SO_3), %		5.0 Max
Moisture Content, %	0.19	3.0 Max
Loss of Ignition, %	4.7	6.0 Max
Available Alkalies as Na_2O, %		1.50 Max

In the preparation of lime slurry, 2 1/2 to 3 pounds of hydrated lime per gallon of water are mixed to obtain a specific gravity of 1.14 to 1.16. After the lime slurry is sufficiently mixed, flyash is added to the mixture until a specific gravity of 1.20 minimum is obtained. Since the flyash varies from source, to source a varying amount of flyash may have to be used to obtain the proper proportions.

Track sections experiencing shallow subgrade failures have responded well to lime flyash pressure injection. In areas of high maintenance, including track surface and line, the division officers request an inspection by the geotechnical engineer. A joint inspection is performed and areas for treatment are defined based on known soils information or in some cases additional investigations are conducted. Annual system contracts are obtained for stabilization and a schedule is prepared for the most critical track sections.

The lime flyash injection work is accomplished with on-track equipment so the production quantities vary with the amount of track time available per day. Typically train schedules will permit time to inject 300-400 track feet per day. Based on previous performance figures, approximately 50-60 gallons of slurry are injected per track foot. In 1987 over 49,000 track feet were injected using 3.2 million gallons of lime flyash slurry.

Certain areas of track may have to be injected more than one time to produce the desired results. Tracks in the "prairie" soils belt in Alabama have been treated several times due to the high content of expansive clay in track subgrades.

Maintenance personnel are required to evaluate all lime flyash slurry injection work performed on Norfolk Southern. Approximately six months after stabilization work has been completed in a particular area, a rating is made on the work. Maintenance work prior to and after treatment is discussed and the percentage of improvement noted. In most cases, maintenance work is reduced significantly and the percentage of improvement usually averages 70%. Based on this information, the lime flyash injection treatment program has been successful when properly applied to track areas experiencing shallow subgrade failures.

RAIL PILING INSTALLATION

The use of piling for roadbed stabilization is probably the oldest form of corrective action used on railroads. The use of trees, ties and timber planking preceded timber, steel and rail piling in the stabilization process. In many locations, driven piling is the recommended correction because the narrow corridor of railway right of way prohibits the use of slope or buttress construction.

A typical embankment location has been chosen to illustrate the use of rail piling to correct a failing slope. Southern Railway mainline trackage from Old Fort to Asheville, North Carolina traverses the structural and topographic escarpment of the Blue Ridge geologic province. Nine miles of track were required to connect points only 3.4 miles apart but having a vertical difference of 891 feet. Total curvature of 2,776 degrees was required in route location that passed through six tunnels. Deep mountain ravines were traversed by fills ranging to 150 feet high (Harshaw, 1977).

The track at milepost S-120.5 Coleman, North Carolina, is located on a through fill of approximately 60 feet high on the north side and 125 feet high on the south side. During a great storm in July 1916, this fill along with many others on the line was washed out. In an effort to reestablish rail service, the track was cribbed up on timber and filled by loose dumping of soil and rock from a nearby cut. Over the years, a sag developed in the track profile at this location. In an effort to maintain an acceptable grade, the track has been raised several times on ballast until shoulders no longer exist.

The embankment also shows signs of distress as a culvert through the fill has been pulled apart by moving fill. Leaning trees and communication poles on the slope indicate the slope has been moving for some time.

In 1986 two inclinometer holes were installed on the fill to monitor movement. One hole 85 feet deep was installed near the track shoulder and a second hole of 55 feet deep was installed on the downward slope approximately 60 feet lower than the top of rail. Both borings were drilled 10 feet into the rock to fix the inclinometer casing at the bottom of the hole.

Just prior to installation of the inclinometer, the track maintenance forces installed a line of rail piling on both sides of the track across the fill so that a ballast shoulder could be maintained. The rail was not driven to refusal and only penetrated the fill approximately 35 feet.

The readings of the inclinometer traced the movement of the embankment but also gave an indication of the effectiveness of the rail piling to resist embankment movement. For a period of two years, the inclinometers have been read on a regular basis and based on analysis by a consultant have produced several interesting results.

A well defined failure zone was detected in the embankment. Movement of approximately 0.8 inches was detected within the first year. Several stability analyses of the embankment were performed assuming a factor of safety of 1.0 for the embankment stability and back-calculating the soil strength parameters.

Without the installation of the rail piling, the most likely failure surface would impact the track superstructure. The surface is located through the track section. The most likely failure surface with the rail piling installed is to the field side of the rail piling and does not directly impact the track structure. A failure surface through the driven rail shows an increase in the factor of safety. Theoretically, an increase of 70% against shearing through the piles occurs but due to bending the increase in safety factors may be only in the 20% range.

Conclusions of the analysis indicate that driving of rail piling can increase the safety factor against slope instability. The rail must be strategically placed and extended to a depth that will force the potential failure surface below the tips of the piles (Darnell and Voor, 1987).

Recommendations to the Maintenance Department concerning the rail piling installation at Mile Post S-120.5 were to extend the piling previously installed to refusal and drive an additional row of piling downslope to counteract potential failure surfaces which occur in front of the piling driven at the shoulder. As of this writing, no additional piling has been driven at this site but the monitoring of inclinometers is continuing.

The use of rail piling on Norfolk Southern to reinforce failing embankments, provide adequate roadbed ballast sections and protect the track from sliding cuts is performed by five company crews using scrap rail and timber. In 1987 over 5,900 tons of rail or approximately 400,000 linear feet of rail piling was driven.

GEOSYNTHETIC APPLICATIONS

Geotextiles suitable for railroad subgrade applications were introduced into the United States in 1974. Southern Railway and Southern Pacific were pioneers in the use of geotextiles in maintenance operations involving undercutting for control of unstable track. Test installations and uses in normal maintenance programs expanded and with the help of geotextile manufactures the survivability rate of geotextiles in track soon improved. The railroads also improved their methods of undercutting to include geotextiles in the track rehabilitation.

Based on experience gained from test installations and regular maintenance uses, a set of guideline specifications was developed in the early 1980's. Tensile strength, puncture strength, and abrasion resistance are the major physical properties considered to be critical for the survival of a geotextile in railroad applications. The geotextile must also have chemical stability and be inert to commonly encountered chemicals and hydrocarbons. The geotextile must be resistant to ultraviolet light, rot and mildew.

Norfolk Southern specifications define three general categories of geotextiles for railroad use: Category A geotextiles are used for all general track maintenance in turnouts, railroad and highway crossings, in tunnels and all undercutting situations. The geotextile in the category is 16 ounces/square yard in weight and should be installed a minimum depth of 8 inches below the bottom of ties. A depth of 12 inches, however, is recommended where conditions permit. Category B geotextiles are used for all installations on new subgrade. This material is placed on top of the subgrade and below the subballast. A weight of 10 ounces/square yard is required for this use. Category C geotextiles are used for drainage purposes at depth over 2 feet below the bottom of ties. This category of geotextiles is also used in slope protections and silt fences. The weight of this material is 5 ounces/square yard.

Table 5 shows the specified physical properties of the three geotextiles categories. Values listed are average roll minimum value in the weakest principle direction. The specifications of Norfolk Southern are similar to those published by the American Railway Engineering Association but have higher specified values in many tests.

Table 5. Specified physical properties of geotextiles

Physical Property	ASTM Test	Category A	Category B	Category C
Grab Tensil Strength	D 4632 (lbs.)	350	250	115
Elongation at Failure	D 4632 (%)	60min. 115max.	60min. 115max.	60min. 115max.
Mullen Burst Strength	D 3786 (psi)	450	350	220
Trapezoidal Tear	D 4533 (lbs.)	150	100	60
Puncture Strength	D 751 (lbs.)	160	130	60
Coefficient of Normal Permeability	D 4491	.1	.1	.1
Permittivity	D 4491	.30	.25	.20
Weight	D 3776 (oz./sy.)	16	10	5

For general maintenance installations, a number of guidelines should be followed in the placement of the geotextile. The site preparation is very important as the use of a geotextile in a railroad subgrade is a harsh test of the material. The roadbed must be graded to the proper depth with positive drainage away from the track section. The geotextile must be the proper weight for the intended use as the railroad loads, and ballast placement and tamping can easily damage the material.

Test installations have shown decreases in grab tensile strength of 55% after 15 months service with approximately 20 million gross tons of traffic over the site. Puncture strength decreased approximately 36% during the same time frame. In all probability, most of these decreases occurred during initial installation and track surfacing.

The geotextile provides a barrier against the migration of subgrade fines into the ballast sections. The formation of a slurry has been observed at many installations on the underside of the geotextile. Permittivity tests run on geotextiles indicate a decrease of approximately 20% of initial values soon after installation (Williams and Grubert, 1988). The fines if permitted to penetrate into the ballast section will reduce the permeability and shear strength of the ballast and eventually lead to increased maintenance.

The specified physical property values must be sufficient to overcome the decreases that occur during installation and still provide a reasonable life for the geotextile.

Annual usage of geotextiles on Norfolk Southern is approximately 300,000 square yards for general maintenance use.

Geogrid use for maintenance functions has been limited and the only major use has been in the construction of new trackage through areas of poor natural soil conditions in which removal of unsuitable material was not economically practical. The geogrid was used through a swampy area in combination with a geotextile and placed between a soil raft mat and the subballast of the track structure. The geogrid was installed to spread the loadings transmitted through the subballast to the soil raft. From all indications, the track has maintained good line and surface since installation.

A geogrid is currently being tested in a ballast section to investigate the effects of the grid on top of rail profile. Laboratory tests have indicated that a geogrid may be used to lessen top of rail settlement (Bathurst, Raymond and Jarrett, 1985). This concept has been applied to a mainline track section recently constructed. The goegrid was installed 6 inches below the bottom of tie in a 12 inch ballast section and after tamping, top of rail elevations were recorded. At regular intervals of approximately 3 months, the top of rail profile has been checked. A comparison between the geogrid sections and control sections indicate a damping of rail deflections in the geogrid sections. The test results have been complicated somewhat by routine track maintenance through the test sections. The geogrid has also been damaged by the tamping. Final test analysis will address the potential for the geogrid to reduce the top of rail settlement and also analyze the effects of ballast abrasion on the geogrid under train loading.

As the railroad loadings continue to increase and maintenance costs escalate, the geotextile and geogrid specifications must be constantly reviewed to insure proper use of existing products and encourage the development of new systems.

CONCLUSIONS

Several methods of chemical and mechanical stabilization have been successfully applied to particular railroad subgrade problems. The selection of a particular stabilization process is based on a combination of empirical and analytical processes developed from within the railroad industry with the assistance of engineering and manufacturing representatives in allied organizations. The stabilization of railroad subgrades has decreased the maintenance effort required to maintain regular train operations.

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APPENDIX

GROUTING MATERIALS SPECIFICATION

This specification is to govern the cementitious material (20% minimum by volume Portland cement or approved equal) and its performance with sand to be used for the stabilization of railway embankments.

<u>Grout Consistency</u>

One part cementitious material is to be mixed with three parts mason's sand by volume with enough water added to obtain a fourteen second flow as determined with a flow cone meeting the Corps of Engineers Specification CRD C-79. The mason's sand shall have a fineness modulus of less than 2.0.

<u>Strength</u>

Strength testing shall be performed in an indoor laboratory where the ambient temperature remains between 65° and 85°F.

Testing shall be done using 6" high x 6" diameter nonabsorptive molds and tested with a Soiltest Pocket Penetrometer Model CT-421 or approved equal. Grout mixture shall be mixed in the laboratory using a mixer described in ASTM C305 and operated at slow speed (140 RPM ± 5 RPM).

The following strengths shall be obtained:

(1) Cementitious material and water mixed to a 14 second flow (no sand)

1 day	700 psi

(2) Cementitious material (1 part), 3 parts mason's sand and water mixed to a 14 second flow

7 days	300 psi
28 days	700 psi

<u>Other Requirements</u>

No air entraining cementitious products shall be permitted.

The cementitious material shall be formulated to prevent the sand from settling out of the mix while being pumped through a 1-1/4" diameter horizontal hose four hundred feet long.

SPECIFICATIONS FOR RAILROAD STABILIZATION WITH LIME/FLY ASH SLURRY PRESSURE INJECTION

A. <u>Materials</u>

1. The slurry shall consist of water, hydrated lime-$Ca(OH)_2$, fly ash and surfactant and shall be continuously agitated to insure uniformity of mixture.
2. The hydrated lime shall be sized to provide a uniform mixture, have a minimum purity of 90% $Ca(OH)_2$ and contain less than 5% MgO. The supplier shall

certify the percent CaO purity of each load of hydrated lime.

3. Fly ash shall test positive for pozzolanic activity and durability.
4. A nonionic surfactant (wetting agent) shall be used according to manufacturer's recommendations.
5. Clean fresh water shall be used and shall be supplied by the contractor.
6. Lime and fly ash shall be supplied by the Railway Company in covered hopper cars unless otherwise stated.

B. Equipment

1. Contractor shall furnish all equipment necessary to provide continuous lime/fly ash injection stabilization and all equipment will be maintained in good operating condition throughout the duration of the work. If equipment break downs or shortages affect production, then the contractor shall quickly repair or replace equipment to insure production minimums.
2. Equipment requirements listed below should be considered as minimum and the contractor shall be required to furnish any additional equipment necessary to complete the work.
   (a) On/off track injection rig with a 2000 gallon slurry tank capacity capable of making three simultaneous injections to a minimum depth of 20 feet.
   (b) On/off track slurry haul truck with 2000 gallon minimum capacity slurry tank.
   (c) Slurry transport trailer with 4000 gallon capacity and semi transport.
   (d) Portable slurry mixing tank(s) capable of handling at least 20 tons of the lime/fly ash mixture with a volume capacity of at least 17,000 gallons.
   (e) All slurry tanks are to be equipped with mechanical agitators to insure proper mixing and uniformity of the slurry.
   (f) Materials handlers (conveyors or vacuum rigs) capable of efficiently moving materials from rail cars to mix tanks with minimum spillage.

C. Personnel

Each crew shall be supervised by a seasoned, well-trained working foreman who is thoroughly familiar with all the stabilization equipment and recommended procedures and techniques for track stabilization using a lime/fly ash slurry.

D. Working Time

1. The contractor shall provide a crew to work five days a week, either Monday through Friday of each week or those days observed by local track maintenance personnel. Starting times shall be up to the discretion of local track maintenance personnel. The work day shall consist of eight hours of continuous lime/fly ash slurry injection as well as time spent performing operations incidental to the injection process. Mixing lime/fly ash slurry and hauling water will be done after work hours and will not be considered as part of the regular eight hours.
2. Moving equipment from one site to another and any delays or work stoppages due to train traffic, severe weather (weather situations that may be hazardous for safe working conditions as mutually agreed upon by contractor and Railway Company) or late material deliveries (when supplied by Railway Company) shall also be considered part of the eight hour continuous work day. Time lost to cold weather as defined in Section J shall be included as part of the regular 8-hour day.
3. Contractor guarantees to have equipment and personnel available to work 168 hours each calendar month. If the contractor does not meet the 168 hour guarantee for a particular month, the Railway Company shall receive a credit for each hour less than the guaranteed minimum. This credit shall be stated in the contractor's proposal. Railway Company observed holidays shall be considered as full 8 hour work days and will be credited towards the contractor's 168 hour work month. The contractor shall also supply rates for overtime (in addition to normal 8 hour days) work and for moving of equipment on weekends.

E. Lime/Fly Ash Mixing and Handling Procedures

1. Preparation of lime slurry: 2-1/2 to 3 lb. hydrated lime per gallon of water. Specific gravity readings of lime slurry shall range from 1.14 to 1.16. The contractor shall have the proper test equipment at the job site to verify the specific gravity readings.
2. Lime/fly ash proportions will vary, depending upon the characteristics of the fly ash to be used, as well as certain site conditions such as ballast depth, soil types, moisture contents, and the nature of the problem. Contractor shall provide the appropriate proportioning for each specific location. A minimum specific gravity of 1.20 should be maintained for the lime/fly ash mixture. Periodic specific gravity readings of lime and lime/fly ash slurries should be taken to verify compliance with specifications and should be noted on contractor's weekly report forms.
3. It is recommended that lime slurry be mixed after each work day in order to be ready for mixing with the fly ash the following morning, prior to the work day. Due to the pozzolanic (cementing) characteristics of the fly ash, it is not recommended that the fly ash be mixed with the lime slurry the day prior to injection.
4. If lime is supplied in pneumatic trucks (in lieu of hopper cars), deliveries shall be scheduled so as to not interfere with the regular work time established by the Railway Company.
5. If there is any lime/fly ash mixture remaining at the conclusion of the work day, part, or all of the material may have to be disposed of to prevent hardening of the mixture in the tanks. If material is remaining at the end of the work day due to railroad delays, the contractor shall be responsible for the material and its disposal as directed by the Railway Company.

F. Application

1. Phase I
   (a) Injections shall normally be continue to "REFUSAL" (i.e., until the pressure of the slurry running out of the previous injected holes becomes the same as that being injected or where the slurry breaks the roadbed or ground surface). In certain soil conditions, it is not practical or possible to achieve refusal. In these cases, slurry volumes should be controlled to inject no more than 100 to 150 gallons per hole, based on an injection depth of 20 feet. The contractor shall make every effort to minimize the amount of excess slurry deposited on the roadbed surface.
   (b) Injection pipes shall penetrate the soil in approximately twelve to eighteen inch intervals, injecting to refusal at each position. The injection pipe shall have a hole pattern that will uniformly disperse the lime slurry throughout the entire depth.
   (c) Proper spacing and depth for the injections should be determined by field examination of each site by Railway personnel and qualified personnel of the contractor. The maximum recommended spacing is every third tie (approximately five feet). Three injections will be made at each location; one at the centerline of the track and two spaced approximately five feet to either side. In cut sections, injections should be extended to a depth of ten feet where practical. In fill sections, injections shall be extended to natural soil.
   (d) Injection pressures should be adjusted to inject the greatest quantity of slurry possible within a pressure range of 50-200 PSI pump pressure.

Phase II

In certain locations, it may be necessary to inject the track more than once to achieve the desired stabilization. Railway personnel will recommend the appropriate sequence and material selection for each injection where applicable. A minimum curing period of 48 hours shall normally be allowed between each injection. The second injections will be spaced between the initial injections and made in the same manner as Phase I.

G. Production Minimums

Minimum Guranteed Production

(a) Inject a minimum of 50 track feet per hour of track time or inject a minimum of 2500 gallons of slurry per hour of track time.

(b) These minimums are an average and will be computed at the end of each month by dividing the respective totals by track time for the month.

Price Adjustment

(a) Contractor shall be required to achieve at least one of the above minimums in order to charge the normally computed monthly charge.

(b) If neither of these guarantee minimums are met, then based on computation of the percentages achieved for each minimum, the higher of the two percentages times the computed monthly rate will equal the adjusted charge.

(Example: If averaged 48 track feet/hour track time, and 2450 gallons/hour track time:

$\frac{48}{50} = 96\%$; $\frac{2450}{2500} = 98\%$

98% x computed monthly charge = adjusted charge)

(c) The guarantees shall not apply under the following conditions:

i) Hauling lime/fly ash slurry in excess of 15 miles one way by road.

ii) Hauling lime/fly ash slurry in excess of 2 miles by rail.

iii) Track time in increments of less than 1.5 hours.

(d) The price adjustment percentages will be computed monthly and submitted with the production reports. The actual price adjustments will be averaged over the period of the contract and, if applicable, credited to the Railway Company at the end of the contract period.

H. Quality Control

1. Detailed reports are to be furnished to Railway Company for each day's work at the end of each week on the contractor's form. On-site Railway personnel shall sign contractor's weekly report in order to confirm injection production.
2. Each crew shall have hydrometers or Baroid scales to control the lime and lime/fly ash slurry mix within the specification parameter. A report of readings (three from each tank mixed) shall appear on reports to Railway Company.
3. Periodic trips shall be made to the job site by contractor's superintendent to review the status of the work being performed and to receive direction for any adjustments desired by the Division Engineer.

I. Job Site Clean-up

Prior to moving equipment to a new site, the contractor shall clean up previous site to the Railway Company's statisfaction. Particular care should be taken to make certain that no hydrated lime or fly ash is left at the site.

J. Cold Weather Procedures

Due to the nature of the injection operations and the consistency of the slurry, injection cannot be effectively accomplished in sub-freezing temperature conditions. In the event that this becomes a factor, injection operations shall be cancelled, unless the weather forecast calls for temperatures to be above 32° for a minimum of five hours during the work day.

K. Contractor/Railway Coordination

The contractor's foreman shall meet with the Railway Company's representative each morning at the job site to insure proper coordination. In the event of poor weather conditions, a telephone conversation may be sufficient.

Highway field trials of chemically stabilized soil subgrades

by

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in cooperation with
Transportation Cabinet
Commonwealth of Kentucky

and

Federal Highway Administration
U.S. Department of Transportation

October 1988

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ABSTRACT: This paper documents the laboratory testing, construction and inital field performance of subgrade soils stabilized with four different chemical admixtures. This study was undertaken as part of a long-term effort to evaluate potential applications for by-product materials in highway construction. One potential application involves the use of by-product materials for soil stabilization. The paper presents preliminary information only and does not address long-term performance and benefits. The determination of long-term performance continues. Construction of several highway field trials of subgrade soils stabilized with four different chemical admixtures is described. Admixtures included hydrated lime, Type 1P cement, multicone kiln dust, and a waste by-product obtained from a process referred to as atsmospheric fluidized bed combustion (AFBC). Total mileage of the trial sections was about 9 miles. The purposes of the study were to construct experimental sections of soil subgrades with various types of admixtures, monitor the long-term performances of pavements placed on the treated subgrades, and determine the short-term and long-term benefits of admixture stabilization. Field and laboratory studies of the subgrade soils before and after admixture treatment and shortly after placement of the pavement are described. Problems that developed in two subgrade sections treated with the AFBC waste by-product, or spent lime, are described. A detailed description of a laboratory, procedure used to determine the optimum percent of admixture is described. Short-term benefits of admixture stabilization are discussed. Evaluation of the long-term benefits and performance of the pavements placed on the stabilized soil subgrades must wait future observations and tests since the field trial sections were only completed in late 1987. Future testing of these trial sections will be used to develop more detailed information for desiging admixture stabilized subgrades. Tentative conclusions and observations concerning the benefits of stabilizing soil subgrades are discussed.

INTRODUCTION

Construction of highway pavements on fine-grained soils, such as clays and silty clays, or soils having poor or marginal engineering properties, is frequently encountered by geotechnical and pavement design engineers. The purpose of this study is to evaluate the results of laboratory testing, construction of test sections and initial field performance of subgrade soils stabilized with various chemical admixtures. This study was undertaken as part of a long-term effort to evaluate potential applications for by-product materials in highway construction. More specifically, this paper presents preliminary information concerning initial laboratory testing, the construction of test sections utilizing the various admixtures for stabilization, and the results of initial field performance testing. All findings must be considered preliminary at this time and are subject to change as more long-term performance information is accumulated. Several cases of failures during construction and premature failures of pavements after construction have been documented (Hopkins and Sharpe, 1986; Hopkins and Allen, 1988; Hopkins, 1988; Hopkins, 1981; Sharpe, 1988; Sharpe and Deen, 1987; Sharpe, 1988). 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 placement dry density of the soil subgrade shall be 95 percent of the dry density obtained from the standard laboratory compaction procedure (AASHTO T 99 or ASTM D 698) and the placement moisture content shall not be 2 percent more or less than optimum moisture content obtained from the standard laboratory compaction test. 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 strength of fine-grained soils is very sensitive to changes in moisture content. With regard to moisture content of soil subgrades, two problems may arise. First, if the 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 the soil increases, there is a decrease in the undrained shear strength or bearing strength. Compaction of soils wet of optimum moisture content is not uncommon. Secondly, when clay, or silty clay, subgrades are left exposed during construction to rainfall and snowfall for a considerable time before the base stone and pavement are placed, they tend to absorb water, swell, and increase in volume. With an increase in moisture content and volume, the undrained shear strength, or bearing strength, decreases. Consequently, failures of the soil subgrade may occur under construction traffic loadings. In this situation, failures are built into the soil subgrade which may cause future problems after the pavement is placed. The built-in failure surfaces in the subgrade may take the form of punching-type shear surfaces or general-type bearing capacity shear surfaces. In either case, the shear strength along the failure surfaces has been reduced from some peak value to a residual value. Placement of pavements over areas of weakened shear surfaces in the soil subgrade increases opportunities for premature failures to occur in the pavement under traffic loadings; that is, opportunities for pavement shoving and settlement are increased. Moreover, compaction of the base stone and bituminous concrete is made difficult because of the softened and weakened subgrade soils.

Even when the soil subgrade possesses adequate bearing strength during construction and no problems are encountered in placing the base material and pavement, changes in moisture contents may occur after placement of the pavement. For example, subgrade soils may be exposed to water as a result of infiltration of surface waters and seepage of subsurface (or groundwater) waters. Also, when clayey soils are used in the subgrade, capillary action may pull water into the subgrade from the groundwater table below the subgrade. When any of these situations occur, then the subgrade may absorb water, swell, and lose bearing strength. Consequently, the moisture content of soil subgrades after construction of the pavement is not a static condition and it does not necessarily remain the same as the placement mositure content. Hence, the moisture content of the subgrade varies as seasonal changes occur in rainfall and snowfall. Consequently, since bearing strength is related to moisture content, the bearing strength varies with seasonal changes in rainfall and weather conditions. Therefore, mechanical 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. Admixture stabilization or the mixing of a powder, slurry or liquid with soil represents one approach to improving the engineering properties of fine-grained soils. Some types of stabilization admixtures include hydrated lime, quicklime, lime and cement kiln dust, and fly ash. Some benefits of admixture stabilization are:

- Expedites construction
- Improves the bearing strength of subgrade soils
- Increases the stiffness of the subgrade soils
- Decreases the swell potential of subgrade soils
- Improves subgrade durability

- ♦ Subgrade soils having poor engineering properties may be used effectively and represents a good economical alternative to the use of other materials.

In addition to lime and cement products, considerable interest has developed in the potential use of the waste by-products obtained from the atmospheric fluidized bed combustion process to stabilize highway soil subgrades. The fluidized bed combustion process is used to burn high sulfur coal and to remove sulfur so that the sulfur compounds do not enter the atmosphere. Increased use of this process will produce considerable amounts of the fluidized bed combustion waste material. Consequently, useful ways of disposing of this material, (other than storing in waste pits) such as for stabilizing soil subgrades, is attractive (Nebgen, et al, 1977).

PURPOSE OF STUDY

The purposes of this study were to construct experimental sections of soil subgrades treated with various types of admixtures, monitor the long-term performances of pavements placed on the treated subgrades, and determine the short-term and long-term benefits of admixture stabilization. Admixtures selected for the highway field trials included hydrated lime, cement, multicone kiln dust, and a waste by-product material obtained from a process referred to as atmospheric fluidized bed combustion (AFBC). This study is part of a much larger research effort aimed at determining potential benefits for the use of by-product materials in highway construction.

SCOPE OF STUDY

This paper describes the construction of several highway field trial sections of subgrade soils stabilized with various admixtures. Also, field and laboratory studies of the subgrade soils before and after admixture treatment and shortly after placement of the pavement are described in detail. A detailed description of a laboratory method used to determine the optimum percent of admixture is discussed. Additionally, short-term, potential benefits of the stabilized sections are presented. Problems that developed in two soil subgrade sections stabilized with the AFBC waste-by-product or spent lime are discussed. Since the highway field trial sections were completed in late 1987, evaluation of long-term benefits and the effectiveness of stabilized subgrades, if any, must await future observations and tests. Also, future testing of these sections will be used to develop more detailed information for designing admixture stabilized subgrades. Tentative conclusions and observations are made regarding the various admixtures used in the field trials.

LOCATION OF HIGHWAY FIELD TRIAL ADMIXTURE SECTIONS

Two routes were selected for the admixture trial sections and general locations are shown in Figure 1. The first section is located in northern Kentucky on the Alexandria-Ashland (AA) Highway. This section is located near the boundary of Mason and Lewis Counties and the town of Tollesboro. The section is referred to as Section AA-19 and extends from Station 1495+00 to 1675+50. The section is about 3.4 miles in length. The entire length of the subgrade soils on Section AA-19 were stabilized

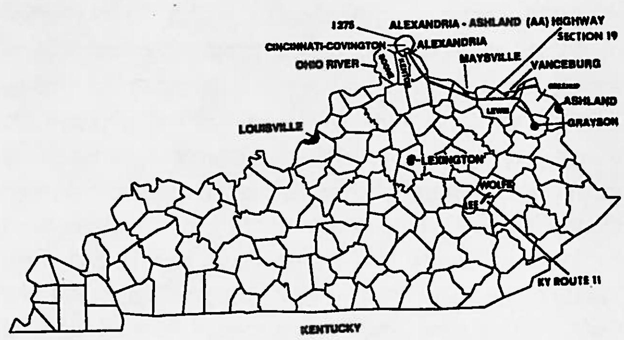

Figure 1. Locations of two highway routes selected for admixture subgrade stabilization

with hydrated lime (Hopkins and Allen 1986). Other admixture trial sections are located on KY 11 near Natural Bridge State Park and approximately 7 miles north of Beattyville. The route extends from Station 260+00 to 576+50 and it is about 6.0 miles in length. The sections of stabilized soil subgrades on KY 11, types of admixture stabilizers, beginning and ending station numbers, and lengths of the sections are shown in Table 1. Stabilization included two sections of soil-AFBC subgrades, two sections of soil-cement subgrades, one section of soil-lime subgrade, and one section of soil multicone kiln dust. One short section extending from Station 522+00 to 532+00 was left untreated on KY 11.

ADMIXTURES

The hydrated lime used in the subgrade section (AA-19) of the Alexandria-Ashland Highway and the subgrade section (Station 348+00 to 402+50) of KY 11 was produced and supplied by the Dravo Lime Company of Maysville, Kentucky. Chemical analysis of the hydrated lime (Dravo Lime Company 1988) is shown in Table 2. Total Ca0 is 72 percent and the amount available is 69 percent. Chemical analysis of the multicone kiln dust (supplied by the Dravo Lime Company), which was used to treat the subgrade soils of a section of KY 11

Table 1. General information pertaining to the sections of stabilized soil subgrades on KY Route 11

Beginning Station	Ending Mile Station	Length (miles)	Stabilizing Admixture
260+00	317+00	1.080	AFBC Spent Lime
317+00	348+00	0.587	Portland Cement
348+00	402+50	1.032	Hydrated Lime
402+50	429+50	0.511	Multi-clone Kiln Dust
429+50	522+00	1.752	Portland Cement
522+00	532+00	0.189	Non-Stabilized
532+00	576+60	0.845	AFBC Spent Lime

NOTE: Station 260+00 corresponds to Milemarker 9.169
Lee County Line ends at Milemarker 14.797

Table 2. Chemical analysis of hydrated lime and multicone kiln dust (courtesy of the Dravo Lime Company)

CHEMICAL ANALYSIS		PHYSICAL ANALYSIS	
COMPOUND	PERCENT	SIEVE SIZE	PERCENT PASSING
Total CaO	72.00	No. 20	100.0
Available CaO	69.00	No. 30	100.0
MgO	2.50	No. 50	100.0
SiO	1.60	No. 100	99.9
R O	0.75	No. 200	99.0
Fe O	0.15	No. 325	97.0
AL O	0.16		
Sulfur	0.045		
MULTICLONE KILN DUST			
CaCO	47.0	No. 50	90.1
CaO	28.0	No. 100	75.2
Available CaO	23.0	No. 200	63.0
MgO	4.6	No. 325	47.6
Sulfur	1.2		
SiO	8.8		
Fe O	0.7		
AL O	3.2		
CO	1.2		

(Station 402+50 to 429+50), is also shown in Table 2. Total Ca0 is 28 percent and the amount available is 23. Chemical analysis of the (1P type) cement admixture that was used to treat two sections of subgrade soils (Stations 317+50 to 348+00 and 429+50 to 522+00, respectively) are shown in Table 3. This type of cement contains about 20 percent fly ash and was supplied by the Kosmos Cement Company of Louisville, Kentucky. The AFBC waste by-product used in two subgrade

Table 3. Chemical analysis of the type 1P cement (courtesy of Kosmos Cement Company)

ELEMENT	PERCENT
C	1.67
Na	0.60
Mb	1.63
AL	6.84
Si	25.94
P	659.30 PRM
S	2.44
K	0.77
Ca	53.87
Ti	0.37
Mn	352.02 PRM
Fe	5.67
Sr	917.48 PRM

sections (Station 260+00 to 317+50 and Station 532+00 to 576+50) was obtained from the Ashland Petroleum Company of Ashland, Kentucky. Chemical analysis (Ashland petroleum Company 1987) of this material is depicted in Figure 2. Results shown in Figure 2 represent x-ray diffraction tests on 21 test specimens. Amounts of compounds in the AFBC material vary. The amount of Ca0 ranged from about 62 to 80 percent and averaged 70 percent. Varability, when considered alone, would not limit the use of this material. Admixture designs could be based on the strength of the AFBC-soil mixtures using the lower percentages of the compounds which improve stability and strengths.

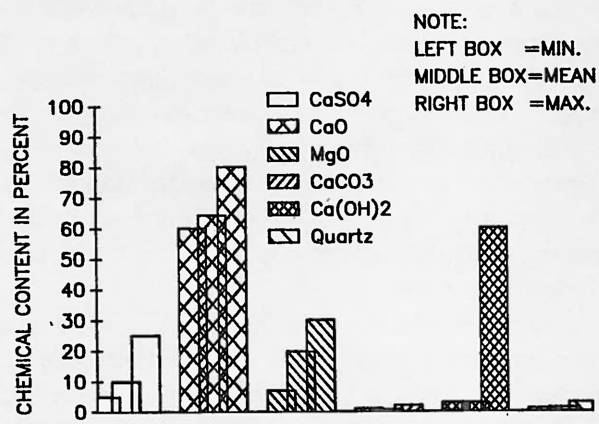

Figure 2. Chemical analysis of the waste by-product obtained from the atmospheric fluidized bed combustion (AFBC) process

CONSTRUCTION PROCEDURES

The construction procedures used on KY 11 varied for each admixture primarily because of the experimental nature of the project. Equipment used included three Ray-Go soil pulverizers, sheepsfoot and smooth wheel vibratory rollers, three spreader trucks for distributing the admixture stabilizers, and three water trucks. The subcontractor for the soil stabilization process was Mt. Carmel Sand and Gravel Company of Mt. Carmel, Illinois. Following is a short description of the construction processes used on each experimental section of Ky 11 and Section AA-19. All subgrade sections on KY 11 were treated to a depth of 12 inches. The subgrade section of hydrated lime on Section AA-19 was treated to a depth of 6 inches.

AFBC Spent Lime Subgrade - - Station 260+00 to Station 317+50 (Ky 11)

The AFBC spent lime was trucked to the site in tractor-trailer trucks, dumped into a storage pit, and covered prior to use. A front-end loader was used to load the material into modified spreader trucks. Tops of the trucks were removed during the loading operation. Stabilization began at Station 260+00 and proceeded in a northerly direction. AFBC spent lime was spread on the prepared subgrade in approximately 200-foot lengths. The amount of AFBC spent lime used in this subgrade section was 7 percent (by dry weight of soil). The prepared subgrade was virtually dry and had a very hard crust. Much of the water applied for mixing ran off the grade and into ditches because the subgrade was very hard. The soil pulverizers ground up the soil to a depth of 12 inches. A sheepsfoot vibratory roller was used for initial compaction immediately behind the soil pulverizers. A smooth-wheel vibratory roller was used for final compaction. Personnel of the Kentucky Department of Highways conducted moisture/density tests using a Troxler nuclear density gage to ensure proper moisture content and compaction of the stabilized soil subgrade. After determining that moisture and density requirements had been met, a Catepillar road grader was used to cut the stabilized subgrade to grade elevation.

Difficulties encountered on the first AFBC spent lime stabilization section included flow of the material, having to cut the stabilized subgrade to final grade elevation, and obtaining correct moisture/density readings. Because of the fined-grained nature of the AFBC spent lime, the material flowed much like a liquid. Because of this problem, the subcontractor eventually had to construct windrows along the subgrade shoulders to contain the AFBC spent lime and water on the subgrade. When water was placed on the AFBC spent lime, a large amount of steam was produced, reducing visibility to near zero. Another difficulty was the absence of cut-off valves inside the cab of the water trucks. On several occasions, the water truck became stuck and discharged excess water onto the subgrade. The subcontractor had to spend considerable time cutting the stabilized soil to grade elevation. Because the incorporation of the AFBC spent lime increased the soil volume, nearly four inches of the stabilized soil had to be trimmed to obtain proper grade elevation. The stabilized subgrade was easy to cut even 24 to 30 hours after final compaction. Department of Highways inspectors experienced some difficulties in obtaining correct moisture readings from the nuclear density gage. Difficulties were attributed to the high amount of hydrocarbons contained in the stabilized soil. After the problem was identified, the personnel determined the actual moisture content by applying a moisture-content correction factor. The correction factor was determined by field drying a soil sample. The correction factor for moisture content was entered into the nuclear density gage.

Portland Cement Subgrade Stabilization -- Station 317+00 to Station 348+00 (Ky 11)

The construction procedure was similar to that used for the AFBC spent lime section. The spreader trucks had their tops replaced and were loaded directly via pneumatic tanker trucks. The Portland Cement (type 1P) application rate was 10 percent for this section. The depth of stabilization was 12 inches. The cement was much easier for the subcontractor to work with than the AFBC spent lime. The setting time for the cement was low and the subgrade had to be cut to final grade elevation within five hours after the mixing operation.

Hydrated Lime Subgrade Stabilization -- Station 348+00 to Station 402+50 (Ky 11)

The stabilization procedure used on this section was similar to the procedure used on previous sections. The prepared subgrade was very dry and hard. The spreader trucks were loaded directly via pneumatic tanker trucks. The hydrated lime was spread by the spreader trucks and water was applied by the water trucks. The application rate for the hydrated lime was 7 percent. The depth of stabilization was 12 inches. Because of the fine grained nature of the hydrated lime, windrows had to be constructed along the subgrade shoulders to contain the hydrated lime and water on the subgrade. The setting time for the hydrated lime treated subgrade was very slow. The subcontractor had very little trouble in cutting the treated subgrade to grade elevation even 48 hours after incorporation of the hydrated lime into the subgrade.

Multicone Kiln Dust Subgrade Stabilization -- Station 402+50 to Station 522+00 (Ky 11)

The stabilization procedure was altered slightly for this section. The prepared subgrade was ripped with a road grader prior to placing the multicone kiln dust (MKD). By ripping the subgrade prior to placing the MKD and water, mixing of the stabilizing agent and water into the subgrade was easier than the method used on the other sections. Ripping also eliminated the need for constructing windrows to contain the material on the subgrade. The MKD material was brought to this section in pneumatic tanker trucks and pumped into the spreader trucks. The spreader trucks distributed the MKD material on the ripped subgrade. The rate of application was 10 percent. Water was applied and the soil mixed by the soil pulverizers. The depth of stabilization was 12 inches. After initial compaction, the soil was tested for moisture content. If there was sufficient moisture, then final compaction was completed. If there was not sufficient moisture in the subgrade, then additional moisture was added and the soil was re-mixed and re-compacted. The subcontractor had very little difficulty on this section. The MKD was easier to work with than the AFBC spent lime. The setting time of the MKD stabilized subgrade was similar to that encountered in the previous hydrated lime stabilized subgrade section.

Portland Cement Subgrade -- Station 429+50 to Station 522+00 (Ky 11)

The construction procedure was similar to that used on the previous portland cement section. Seven percent of portland cement (type 1P) was used on this subgrade section.

Non-Stabilized Subgrade -- Station 522+00 to Station 532+00 (Ky 11)

A 1,000-foot section of the subgrade was not stabilized and served as a control section for the project.

AFBC Spent Lime Subgrade -- Station 532+00 to Station 576+50 (Ky 11)

This section was conceived after construction difficulties were encountered on the initial AFBC spent lime section. Construction procedures were altered from those used on the previous AFBC spent lime section. The procedures used were similar to those used for the MKD section wherein the subgrade was ripped prior to placing the AFBC spent lime on the subgrade. The application rate for this section was 7 percent. After the AFBC spent lime was placed, the water was added and the soil was mixed. The depth of stabilization was 12 inches. The subgrade was checked for proper moisture content and dry density after initial compaction. If the moisture content was within a range of ∓2 percent of optimum moisture content and the dry density was equal to 95 percent (or greater) of maximum dry density, final compaction was completed. If there was not sufficient moisture in the subgrade, then additional moisture was added and the soil was re-mixed and re-compacted.

Hydrated Lime Subgrade -- 1495+00 to 1675+50 (Section AA-19 of the Alexandria-Ashland Highway).

The mixing procedure used on this section consisted of spreading hydrated lime (from spreader trucks) onto the subgrade and adding water from water trucks. The subgrade, lime, and water were mixed using a large disc pulled by a tractor. Treatment depth should have been 6 inches. The actual, completed treatment depth was about 4 inches. The completed depth was determined by using phenolphthalein on shelby tube specimens obtained after treatment. When this chemical agent is applied to soils treated with hydrated lime, the color of the treated soils changes to a deep red color. Untreated soils do not change color when this chemical agent is applied. Hence, by applying this chemical agent along the length of a field specimen, the depth of treatment may be established.

TESTING PROGRAM AND EQUIPMENT

Sampling

Both disturbed and undisturbed soil samples were obtained. Disturbed bag and (five-gallon) bucket samples of the natural soils were obtained from Section AA-19 of the (Alexandria-Ashland) Highway (Hopkins and Allen ,1986) at three different locations. The samples were obtained at Station 1630 (identified as Sample A), Station 1495 (Sample B), and Station 1675+50 (Sample C). The soil samples were obtained at the beginning, middle, and near the end of Section AA-19. The three sampling sites are directly underlain by the Crab Orchard Formation which consists of mainly clayey shales and some interbedded layers of limestone. The disturbed, residual soils collected from this site are derivatives of the clayey shales and limestones of the Crab Orchard Formation.

At the KY 11 site, disturbed samples of the natural soils were collected from three soil stockpiles constructed by the contractor. Soils from these stockpiles were used to construct the soil subgrades at this site. The stockpiles were located at Stations 274, 334, and 574. Also, the geotechnical branch of the Division of Materials (Kentucky Transportation Cabinet) obtained samples of the soil subgrade before treatment every 500 feet along the entire length of the reconstructed roadway. Geology of this site consisted of interbedded layers of shales, sandstones, siltstones, and some coal. The soils at this site are residual and consist of derivatives of the shales, sandstones, siltstones, and coal.

Undisturbed specimens of the soil subgrades, after treatment by admixtures were obtained using thin-walled, Shelby tubes. Inside diameter of the thin-walled tubes was 2.8 inches. Undisturbed samples were obtained from Section AA-19 of the Alexandria-Ashland Highway and the KY 11 site. A few undisturbed soil specimens of the treated subgrade were also obtained from the KY 11 site using a portable core drill.

Laboratory Testing Program

The laboratory testing program consisted of determining select engineering properties of the soil samples in an untreated, or natural, state and in a state treated by a chemical admixture. The purposes of the laboratory study were to:

* Classify the soils of Section AA-19 of the AA Highway and KY 11,
* Develop the necessary data so that an appropriate chemical admixture could be selected,
* Determine changes, if any, in the engineering properties of the soils at the two sites after treatment with chemical admixtures, and
* Determine the optimum percentage of a given chemical admixture to add to the soils at the two sites.

The laboratory study consisted of performing the following tests:

* Liquid and plastic limits
* Specific gravity tests
* Particle-size analyses
* Soil classifications
* Visual descriptions
* pH tests
* Moisture-density relationships
* Bearing ratio tests
* Swell tests
* Unconfined compression tests

Index tests. Liquid and plastic limit tests were performed according to procedures of ASTM D 423-66(72) and ASTM D 424-59(71). Particle-size analysis was performed according to ASTM D 854-58(79). The soil samples were classified using the Unified Soil Classification System, ASTM D 2487-69(75), and the AASHTO Classification System (M 145-82).

Moisture-density relationships. Moisture-density relationships of treated and untreated soils were determined according to ASTM D 698-78, Method A, or AASHTO T99. The purpose of these tests was to determine the optimum water content and maximum dry density of the various soils at the two sites. Also, these tests were used to study the variation (if any) of optimum moisture content and maximum dry density of the treated soils as the percentage of a given chemical admixture increases. The values obtained from these tests were also used to check field compaction of the chemically treated soil subgrades.

Bearing Ratio Tests. California Bearing Ratio tests (CBR) were performed using two slightly different procedures. A few tests on soils from Section AA-19 were performed following procedures of ASTM D 1883-(73). However, soils from both Section AA-19 and the KY 11 site were generally tested following procedures of Kentucky Method KM-64-501-(76). In the ASTM bearing ratio test, specimens are compacted dynamically at maximum dry density and optimum moisture content, as determined from ASTM D 698-(78). In the Kentucky method, CBR specimens were molded using the values of optimum moisture content and maximum dry density, as determined from ASTM D 698-(78). However, static compaction was used to mold the specimens (according to KM-64-501-76). A static pressure of 2,000 psi was maintained on the specimens for 2 minutes during the compaction stage. Actually, when this procedure is followed, the dry density and water content of the remolded specimen are not necessarily the same as values obtained from ASTM D 698-(78). Generally, based on studies by Hopkins, 1970, 1988, and 1984, the final remolded dry densities and moisture contents of the Kentucky CBR specimens after soaking and swell are usually slightly higher and lower, respectively, than maximum dry densities and optimum moisture contents as determined by ASTM D 698-(78). By specifying a static pressure of 2,000 psi, the height of the CBR specimen cannot be controlled and, therefore, the dry density cannot be controlled. Nevertheless, the procedure as outlined by KM-64-501-76 was followed because of its long-term usage in Kentucky and because the procedure is linked to pavement design in Kentucky.

In the ASTM procedure, the bearing ratio specimens are soaked (immersed) in a water tank for 96 hours. In the Kentucky method, the CBR specimens are placed (immersed) in a water tank and allowed to absorb water until consecutive swell deflection readings are equal to or less than 0.003 inch; specimens are soaked a minimum of 72 hours. Hence, in the Kentucky method, the CBR specimens are allowed to soak until most or all of primary swell ceases. In the ASTM method, swell may still be in progress when the specimen (Hopkins 1984) is removed from the water tank after 96 hours. In this case, the CBR value may be overstated; that is, the value obtained may be larger than the value obtained if swell was allowed to continue.

In both bearing ratio tests, penetration values, as recorded in the test, are 0.1, 0.2, 0.3, 0.4, and 0.5 inch. In the ASTM bearing ratio test, the bearing ratio value normally reported is the one occurring at 0.1-inch penetration. In the Kentucky method, the minimum CBR-value occurring at one of the five penetration values is normally reported. However, selecting a minimum value may be deceptive for soil specimens treated by chemical admixtures since the treated specimens are usually brittle and failure occurs at much smaller strains than strains obtained from the failure of untreated (clayey) specimens. The situation during penetration of the CBR specimen is depicted in Figure 3. The theoretical failure surface consists of an active wedge, central wedge, and passive wedge. The angle, θ_a, between the shear

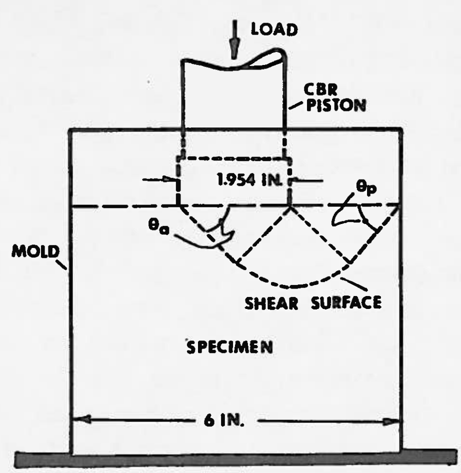

Figure 3. Theoretical bearing capacity shear surface in the CBR test during undrained loading

surface and a horizontal line (active wedge), and the angle, θ_p, between a horizontal line and the shear surface of the passive wedge is

$$\theta_a = 45 + \frac{\phi}{2} \qquad (1)$$

$$\theta_p = 45 - \frac{\phi}{2} \qquad (2)$$

The parameter, ϕ, is the angle of internal friction. The two wedges, or shear surfaces, are connected by a shear surface approximated by a logarithmic spiral, or the curve may be determined from the expression

$$r = r_0 e^{\theta \tan\phi} \qquad (3)$$

where r =radius vector of the logarithmic spiral,
r_0= initial radius vector of the logarithmic spiral,
e = base of natural logarithms, equal 2.71828,
ϕ= angle of internal friction, and
θ= angle between two radius vectors, r_0 and r, measured in radians.

Since the CBR specimen has been soaked and is saturated and since the specimen is penetrated rapidly, undrained conditions for clayey and silty clayey specimens prevail. Consequently, ϕ is 0, and equations 1, 2, and 3 become

$$\theta_a = \theta_p = 45 \text{ and } \qquad r = r_0$$

The logarithmic spiral becomes a circle. As shown in Figure 3, the bearing capacity shear surface of a specimen (undrained failure) in a 6-inch mold intersects the surface of the CBR specimen and the mold wall at point d.

For brittle specimens which have low values of failure strain, the maximum failure stress and CBR-value will occur early in the test and, subsequently, decrease as the penetration increases. With increasing penetration, the shear strength along the failure surface decreases from a peak value to residual value. Hence, at large penetration values, the value of CBR may decrease. Test values for 0.1-inch penetration (as well as the minimum value) obtained from the Kentucky CBR test are shown herein so that a true measure of the effectiveness of chemically treated, CBR specimens may be depicted.

Swell Tests. During the course of performing CBR tests, swell measurements are made according to test procedures outlined by Kentucky Method KM-64-501-76 and ASTM D 1883-(73). Additionally, a few selected swell tests were performed on the waste by-product, AFBC.

Optimum percentage of chemical admixture. Various methods may be used to determine the optimum percentage of chemical admixture (Terrel, et. al. 1979).

These methods are as follows:

* Unconfined compression tests
* Charts and tables by manufacturers of chemical admixtures
* pH tests.

Unconfined compression tests. One of the most widely used methods of determining the optimum percentage of chemical admixture to mix with a given type of soil is the unconfined compression test. In this approach, several soil specimens are remolded at different percentages of admixture and at optimum moisture content and maximum dry density (or at selected values of moisture content and dry density). Unconfined compression tests are performed on the specimens following procedures of ASTM D 2166-(79)--strain controlled technique. The unconfined compressive strengths are plotted as a function of the percentage of chemical admixture. The optimum percentage of chemical admixture is a point where there is no significant increase in the unconfined compressive strength as the percent of chemical admixture is increased.

One objective of this study was to develop a laboratory procedure for remolding laboratory soil specimens that have been treated with a chemical admixture and for determining the optimum percent of chemical admixture. For certain types of admixtures, such as hydrated lime (or quicklime) or the waste by-product, AFBC, the maximum dry density and optimum moisture content, as obtained from ASTM D 698, have been observed (Hopkins 1984) to decrease and increase, respectively, as the percentage of chemical admixture increases (to a certain point). Consequently, the maximum dry density and optimum moisture content should not be held constant for specimens remolded at different percent of chemical admixture. For example, maximum dry density and optimum moisture content of a specimen remolded at 10 percent of chemical admixture are different than the maximum dry density and optimum moisture content for a specimen remolded at 4 percent, or some other percent. The unconfined compressive strength is sensitive to changes in dry density and water content. Consequently, if the maximum dry density and optimum moisture content obtained from compaction tests of untreated material were used to remold specimens at different percent of a chemical admixture, then the unconfined compressive strengths would tend to be higher than the strengths of specimens remolded at maximum dry density and optimum moisture content of specimens at a given percent of chemical admixture. In the proposed procedure developed by Hopkins 1986 and described below, efforts are made to remold the specimens to maximum dry density and optimum moisture obtained at a given percentage of chemical admixture. This procedure should be considered tentative, although the method is being used on a trial basis by the Kentucky Department of Highways (Geotechnical Branch of the Division of Materials).

In molding the unconfined compression samples, the maximum dry density and optimum moisture content at a given percentage of chemical admixture must be determined. The procedure usually consists of performing three standard compaction tests (ASTM D 698 or AASHTO T-99) at 0, 5, and 10 percent of chemical admixture. (These percentages may be varied depending on the type of chemical admixture.) Values of maximum dry density are plotted as a function of the percent of chemical admixture (figure 4). Hence, the maximum dry density and optimum moisture content may be determined by interpolation for a given percent of chemical admixture other than 0, 5, and 10.

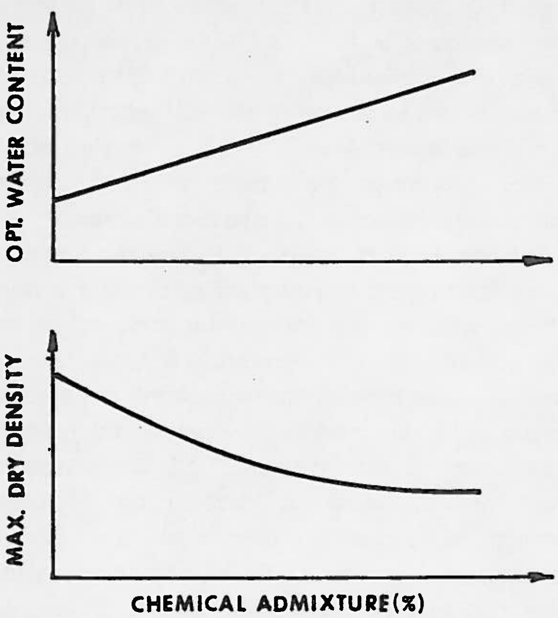

Figure 4. Maximum dry density and optimum moisture content as a function of the percentage of chemical admixture

Unconfined compressive specimens are molded at various percent of chemical admixture. The specimens are formed using a split-type, hinged mold. By using this type mold, the specimens do not have to be extruded. The total height of the mold is 8 inches and the inside diameter of the mold is 2.8 inches. Specimens are remolded to a height of 6.0 inches and a diameter of 2.8 inches.

To determine the amounts or weights of water, soil, and chemical admixture that are needed to form a specimen, use is made of a soil-water-chemical admixture phase diagram as shown in figure 5. Based on this diagram, the weight of dry soil, W_S, may be calculated from the following expression

$$W_s=\frac{\gamma_d V_t}{1+P} \tag{4}$$

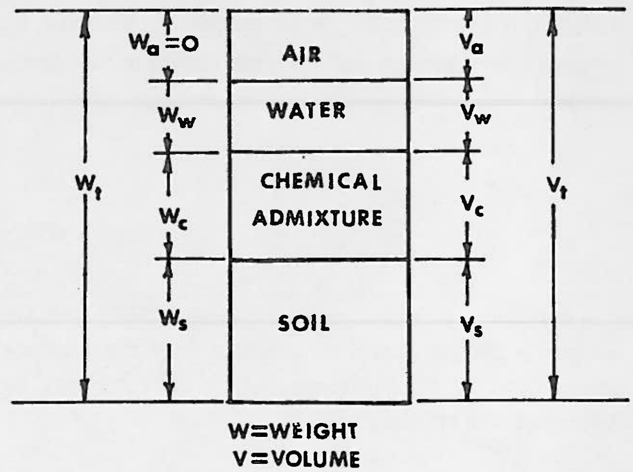

Figure 5. Soil-water- chemical admixture phase diagram

where γd = maximum dry density of the soil-chemical admixture specimen determined from ASTM D 698 or AASHTO T-99 (or for a selected percent of chemical admixture, the value may be obtained by interpolation in Figure 5. Alternatively, if field specifications require that the admixture-soil subgrade must be compacted to a certain minimum relative compaction, such as 95 percent, then a dry density should be selected that simulates the field condition. For example, if the relative compaction is 95 percent, then the dry density selected for use in equation 4 is obtained by multiplying the maximum dry density (from standard compaction) by 95 percent).

V_t = volume of specimen to be formed (predetermined values of height and diameter are used or assumed. In the tests reported herein, a height of 6.0 inches and a diameter of 2.8 inches were used), and

P = selected percent of chemical admixture.

Although oven-dried soil could be used, this is not a preferred method since oven-drying may alter the properties of the soil. Usually, the preferred method is to air-dry the soils. The oven-dried weight (or weight of soil solids) is related to the air-dried weight by the expression:

$$W_s=\frac{W_{ad}}{1+W_n} \tag{5}$$

Hence, to account for hydroscopic moisture in the air-dried soils, the expression from equation 5 is inserted into equation 4, and equation 4 becomes:

$$W_{ad}=\frac{\gamma_d V_t(1+W_h)}{1+P} \qquad (6)$$

where w_{ad} = weight of air-dried soil (Note: The soil does not have to be completely air-dried if the soil is sealed in a plastic bag to prevent loss of moisture after the hydroscopic water content is obtained).

w_h = hydroscopic water content of air-dried soil or the moisture content of the soil at the time the soil is sealed to prevent further loss in moisture).

The weight of chemical admixture needed to form the specimen is defined as:

$$W_c=PW_s \qquad (7)$$

or

$$W_c=\frac{PW_{ad}}{1+W_h} \qquad (8)$$

The amount of water to add to the mixture of soil-chemical admixture may be computed from the following expression:

$$W_w=(w_o-w_h)W_s+w_oW_c \qquad (9)$$

where w_o = optimum water content obtained from ASTM D 698 or AASHTO T 99 (the value may be interpolated from the plot in Figure 4) or some other value may be selected and used for w_o in equation 9. Equation 9 assumes that the water content of the admixture is zero.

Equations 6, 8, and 9 are programmed using a spread sheet computer program to facilitate the laboratory preparation of the specimens. Input parameters required for making calculations include:

* Maximum dry density, γ_d, (or some other selected value), corresponding to a selected percent, P , of chemical admixture;
* Optimum water content, w_o +1, (or some other selected value), corresponding to a selected percent of chemical admixture;
* Volume of specimen to be formed. (Height and diameter used in this study were 6.0 inches and 2.8 inches, respectively. The ratio of specimen length to diameter is slightly greater than 2). (See Bishop and Henkel 1964);
* Hydroscopic water content, w_h; and
* The selected percent chemical admixture, P.

The following parameters are obtained from the spread sheet computer program:

* The weight, W_{ad}, of air-dried soil needed to form the unconfined compressive specimen that corresponds to a selected percent chemical admixture, dry density, and water content,
* The amount or weight, W_c, of chemical admixture needed to form the specimen, and
* The amount of water, W_w, needed to form the specimen.

After the proper amounts of air-dried soil, chemical admixture, and water are calculated and weighed, the three ingredients are mixed using a mechanical mixer. The mixing procedure is similar to ASTM D 3551-(76). After mixing, the material is placed in plastic bags and sealed a 1-hour mellowing period. After mellowing, the specimen is divided into four equal parts by weighing. The specimen is compacted in four layers. The height of each layer is equal. For example, if the total specimen height is 6.00 inches, then the height of each compacted layer is 1.5 inches. The first layer is compacted by trial and error to a height of 1.5 inches. This is performed by compacting and measuring the height of the layer by trial and error until it is compacted to a height of 1.5 inches. This process is continued for each layer. In this approach, the number of blows do not have to be counted since the proper amounts of water, chemical admixture, and soils are known. However, having to stop and measure the height of each layer as it is compacted is inefficient. Consequently, a special steel ram and slip rings were designed and machined. Dimensions of the compaction ram are shown in Figure 6. The diameter of the ram is 2.75 inches or slightly less than the inside diameter (2.80

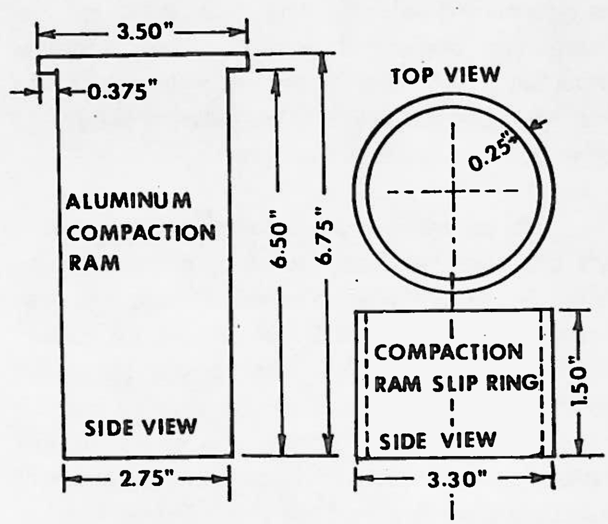

Figure 6. Dimensions of compaction and slip rings used to mold soil-chemical admixture mixes

inches) of the split-ring mold. The compaction ram has a collar at the top which has a diameter of 3.5 inches, or slightly larger than the outside diameter of the mold. The distance from the bottom of the collar of the ram to the bottom of the ram is 6.50 inches. When the first layer is compacted and the ram is thrust downward until the collar comes into contact with the top of the mold, the first layer is exactly 1.5 inches in height, since the height of mold is eight inches. The collar prevents over compaction or under compaction. As shown in Figure 6, three slip rings were machined . The height of each slip ring is 1.5 inches. The outside diameter of each ring is 3.3 inches. The inside diameter of each ring is 2.8 inches or slightly greater than the diameter of the steel ram. Each ring is machined so that it slides freely over the steel ram. To compact the second layer, a fourth of the mixture of soil-water-chemical admixture is added to the mold and the ram with one slip-ring is used. The material is compacted until the bottom of the first slip ring touches the top of the mold. Hence, the height of the second layer is 1.5 inches. The third and fourth layers are formed using 2 and 3 slip rings, respectively.

In using this technique, the amounts of soil, chemical admixture, and water must be weighed carefully. Also, care must be exercised in mixing the specimen so that material is not lost during mixing and placing the material in the mold. Generally, dry densities and water contents of molded specimens are within 0.5 pcf and 0.5 percent, respectively, of target values of dry density and water content.

After molding, each specimen is sealed by wrapping with plastic wrap and aluminum foil (alternately the specimens may be waxed) and stored in a humidity room. For most of the tests reported herein, the specimens were cured for 7 days at room temperature (about 72 degrees Fahrenheit). Unconfined compression tests were performed at the end of the 7 days. Data reduction was performed using LOTUS ® 123 computer software and programmed equations. The unconfined compressive strength was plotted as a function of the percentage of chemical admixture. The optimum percentage of chemical admixture may be selected from this plot.

Charts and tables by product manufacturers. Several charts and tables have been devised by manufacturers for selecting the percent of chemical admixture. These have been described by Terrel, et al 1979. For example, the Portland Cement Association presents a table showing the cement requirements for various types of soils based on the AASHTO and Unified Soil Classification Systems. Also, the National Lime Association has published a graph for determining the percent of hydrated lime to use for a given type of soil. This graph makes use of the index properties of the soil. Results obtained from these tables and graphs were used to compare to results obtained from unconfined compressive tests. Also, an approximate and quick means of selecting the percent of hydrated lime is shown in Figure 7. This graph represents a re-

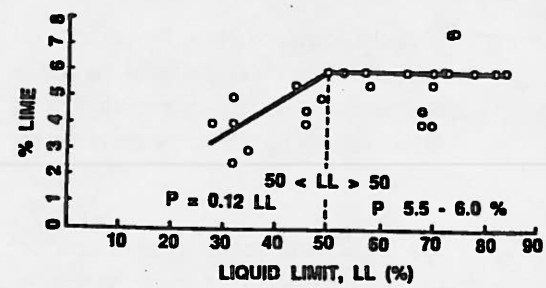

Figure 7. Percent of hydrated lime as a function of liquid limit

analysis (Hopkins 1986) of data presented by Haston, et, al, 1965 and is based on the liquid limit of the soil. If the liquid limit, w_L , is less than 50, then the percent of lime, P , may be approximated by the following expression

$$P = 0.12 W_L \tag{10}$$

If the liquid limit is greater than 50 percent, then the percent of lime needed for stabilization is approximately 5.5 to 6.5.

Use of pH tests. Tests to determine the pH of hydrated lime-soil mixtures were performed following a procedure proposed by Eades and Grim (1969). This is a quick method for determining lime requirements for lime stabilization.

FIELD TESTS

Field testing consisted of performing Road Rater deflection measurements, in-place CBR tests, moisture content - dry density tests, and pavement swell monitoring. Test procedures are described below.

Road Rater

The Kentucky Transportation Center Model 400B Road Rater was used for structural evaluation of each experimental admixture section and the control (untreated) section. section. The model 400B Road Rater is a dynamic pavement testing device which applies a steady state vibratory load to the pavement. The magnitude of the steady state vibratory load is a function of the frequency and amplitude of the vibrating mass. The mass for the Model 400B Road Rater is constant at 300 lbs. Frequency may be varied from 0 to 60 Hz and the amplitude may be varied from 0.0 to 0.1 inch. The steady state vibratory load applied by the Road Rater impulses the pavement. The forced motion of the pavement is measured by velocity sensors located as illustrated in Figure 8. The vibrating mass of the Road Rater is suspended by a system of rubber bellows which distribute the loading concentrically about the lifting cylinder. A second set of bellows is used to provide for equal distribution of the loading to the two load feet as shown in Figure 8. The Road

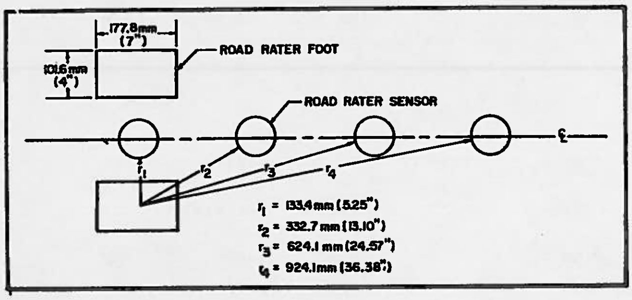

Figure 8. Schematic of road rater loading and sensor configuration

Rater is a hydraulically actuated system with both the raising and lowering of the system to the roadway surface and the vibratory motion of the mass controlled and actuated by an electro-hydraulic system. Data acquisition is computer controlled using a Hewlett-Packard 85B microcomputer with data storage on magnetic tape and also a paper tape. Override mechanisms are available which permit manual operation and manual data collection. A range of dynamic loadings is possible depending upon the selection of frequencies and amplitudes of vibration. Practically, the loading limits of the Road Rater 400B system are 0 to 2400-lbs force. A frequency of 25 Hz was selected for all testing for these pavement sections. Past experience has indicated that this frequency generally results in consistent response characteristics for all velocity tranducers. Amplitudes were varied to produce in dynamic forces of 600 and 1,200 lb-force for testing. Loadings are transmitted to the roadway surface by means of two "load feet" as shown in Figure 8. Road Rater deflection tests were obtained on the native subgrade prior to treatment with an admixture. Deflection tests were later obtained after placement and mixing of each subgrade section with a stabilizing admixture. Subsequent deflection tests were obtained after placement of the crushed stone layer and after placement of each course of the asphaltic concrete pavement. Additionally, long-term deflection testing is planned to quantify the long-term structural characteristics of each experimental section. Data have been collected after placement of each course of asphaltic concrete and after completion of the final pavement surface, but these data will not be included in this paper. Only deflection data obtained before treatment of the subgrade and after treatment of the subgrade are considered for this paper. The deflection data were used to estimate the elastic moduli of the subgrades.

In-place CBR tests.

In-place California Bearing Ratio (CBR) tests were performed before and after subgrade stabilization. Penetrations and calculations for in-place CBR tests were performed in accordance with ASTM D 1883-(73), "Bearing Ratio of Laboratory Compacted Soils", except that the tests were performed on the soil in it's actual in-situ conditions. Moisture contents of the soils were performed in accordance with ASTM D 2216-(80). "Laboratory Determinations of Water (Moisture) Content of Soil, Rock and Soil-Aggregate Mixtures".

Field density tests.

Field density tests of the compacted soil subgrade were performed by the Kentucky Department of Highways using a Troxler moisture-density nuclear gage. The manufacturer's procedures were followed in conducting the field density tests.

Pavement swell measurements.

The pavement surface placed on subgrade sections stabilized with the AFBC material of KY 11 exhibited signs of washboarding or non-uniform swelling in late September, 1987. Surface elevations were periodically obtained at selected locations to monitor elevation changes of the pavement surface. Initial measurements were made in early October at selected stations in each AFBC stabilized subgrade section and also at other sections (soil-cement subgrades, soil-hydrated lime subgrade, and soil-multicone kiln dust subgrade). The humps that formed on the pavement surface by the non-uniform swelling of the stabilized subgrade were generally transverse to the centerline. However, measurements were taken in both transverse and longitudinal directions on 2-foot centers. The equipment used included a Topcon AT-F6 leveling instrument and a leveling rod with a level bubble. Readings were estimated to 1/1,000 of a foot.

TEST RESULTS AND ANALYSIS

Laboratory index properties.

Index test data and soil classifications of the untreated and treated soils from Section AA-19 of the Alexandria-Ashland Highway are summarized and compared in Table 4. Three untreated bag samples, identified as A (Station 1630+00), B (Station 1495+00), C (Station 1675+00) obtained from Section AA-19, classified as MH-CH, CH, and MH-CH, respectively. Plasticity indices of the soils were relatively high and ranged from 29 to 37 percent. Liquid limits ranged from 61 to 71 percent. Specific gravities ranged from 2.80 to 2.97. The percent of soil passing the number 200 sieve ranged from 92.8 to 94.4 percent. The brown to greenish gray soils were highly plastic clays. A general range of soil properties of the natural soils occurring along Section AA-19 is shown in Figure 9. These data are from soil and profile plans developed during the corridor

Table 4. Index test data and soil classification of untreated and treated soil specimens, Section AA-19

SAMPLE NUMBER AND LOCATION	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	SPECFIC GRAVITY	PARTICLE-SIZE ANALYSIS PERCENT FINER THAN: NO. 10 (%)	NO. 200 (%)	0.002mm (%)	CLASSIFICATION AASHTO	UNIFIED
UNTREATED SPECIMENS										
A STA 1630+00		71	34	37	2.97	99.2	90.0	57.5	A-7-5(40)	MH-CH
B STA 1495+00		71	30	41	2.80	98.9	92.8	57.5	A-7-5(44)	CH
C STA 1675+50		61	32	29	2.80	99.5	94.4	66.0	A-7-5(32)	MH-CH
SPECIMENS TREATED WITH 6% LIME										
A STA 1630+00		53	47	6	2.94	97.2	39.4	21.0	A-4(0)	SM
B STA 1495+00		45	43	2	2.80	98.2	34.7	21.5	A-2-4(0)	SM
C STA 1675+50		41	37	4	2.81	98.9	85.4	38.0	A-4(0)	ML

study of section AA-19. Forty-two tests were performed. Figure 9, indicates 57 percent of the soils had liquid limits ranging from 50 to 77 percent and 43 percent of the soils had liquid limits that ranged from 25 to 50 percent. Fifty-four percent of the soils had plasticity indices ranging from 12 to 25 percent. Forty six percent had values of plasticity index ranging from 25 to 43 percent. The percent passing the number 200 sieve of 81 percent of the samples was equal to or greater than 87.

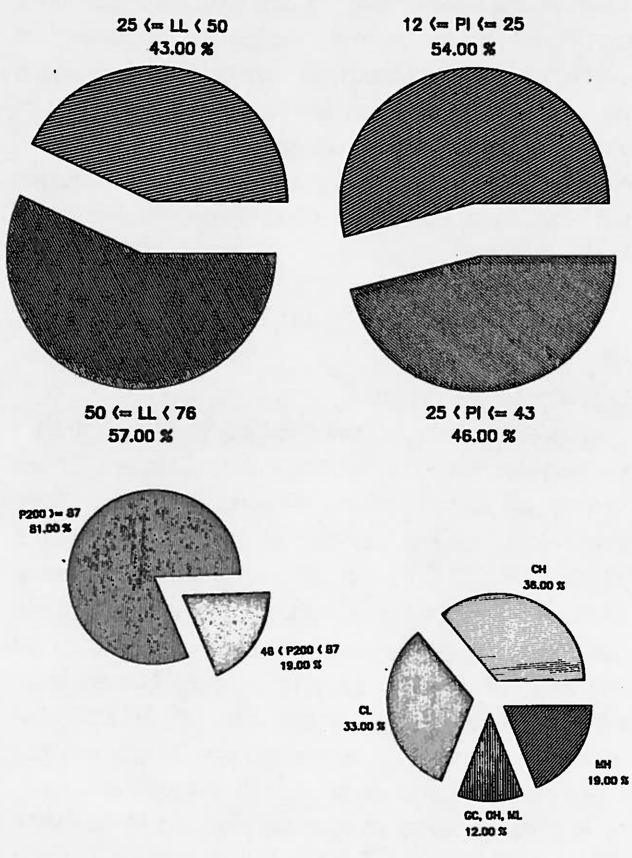

Figure 9. Index properties of soils from Section AA-19

Nineteen percent of the samples had percent passing the number 200 sieve that ranged from 46 to 87 percent. Approximately 55 percent of the samples classified as MH or CH. About 33 percent of the samples classified as CL and 12 percent classified as GC, OH, or ML. The bag samples, identified as A, B, and C, that were selected for detailed laboratory studies were reasonably representative of the natural soils occurring along section AA-19.

Based on criteria by Epps, Dunlap, and Gallway (chart) 1970 and the Air Force Manual of Standard Practice - Soil Stabilization 1976, the soils along Section AA-19 were very suitable for stabilization with hydrated lime. The effect of hydrated lime on the liquid limit and plastic limits of soils of Section AA-19 is shown in Table 4. The liquid limits of the untreated samples (A, B, and C) ranged from 61 to 71 percent. After treatment with hydrated lime, smaller values of liquid limits were obtained. Liquid limits of the treated soils ranged from 53 to 61 percent. The most noticeable change in the properties occurred in the plasticity indices of the soils. For the untreated soils, values of plasticity index ranged from 2 to 6 percent. The percent passing the No. 200 sieve of the untreated soils, as shown in Table 4, ranged from 90 to 94. After treatment, the percent passing the No. 200 sieve ranged from 35 to 85 percent. The percent finer than the 0.002-mm size of the natural soils, ranged from 58 to 66. After treatment, the percent finer than the 0.002-mm size ranged from 21 to 38. As shown in Table 4, the untreated soils classified as A-7-5 and MH and CH. After treatment, the soils classified as A-4 or A-2 and SM or ML, respectively. Hence, treatment with hydrated lime significantly changed the index properties of the soils from Section AA-19.

Index test data and classifications of the untreated soils obtained from the soil subgrade at 500-foot intervals along KY 11 are summarized in Table 5. Liquid limits of the untreated soils range from 27 to 48. Plasticity indices ranged from 7 to 21

Table 5. Index test data and soil classification of untreated soils obtained at 500-foot intervals along the Ky 11 subgrade

STATION NUMBER	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	GRAIN-SIZE ANALYSIS PERCENT FINER THAN: (%) No.3/8-in.	No.4	No.10	No.40	No.200	0.002mm	CLASSIFICATION AASHTO	UNIFIED
264	44	18	100.0	98.7	98.4	97.0	85.0	33.2	A-7-6(17)	ML-CL
269	32	9	100.0	98.0	97.7	96.4	82.7	17.6	A-4(7)	ML-CL
274	37	13	100.0	98.6	98.1	92.3	81.4	34.9	A-6(11)	ML-CL
279	42	17	100.0	98.0	97.2	91.7	80.6	31.5		
284	37	14	100.0	82.4	82.8	82.5	74.9	22.2	A-6(10)	ML-CL
289	43	18	100.0	98.4	97.9	94.2	85.1	36.8	A-7-6(16)	ML-CL
294	47	20	100.0	98.7	98.0	91.2	83.0	37.4	A-7-6(18)	ML-CL
299	48	21	100.0	99.5	99.0	95.4	86.8	37.5	A-7-6(21)	ML-CL
304	33	16	100.0	99.2	98.8	94.8	81.6	27.9	A-6(12)	CL
309	31	9	100.0	98.7	98.2	95.2	83.5	22.2	A-4(7)	ML-CL
314				79.8						
319				89.5						
324				95.3						
329				95.7						
334				79.9						
344				68.8						
349				75.8						
354				80.9						
359				90.0						
364				89.5						
369	30	9	100.0	70.5	54.8	52.7	43.8	10.6	A-4(1)	GM-GC
374	30	10	100.0	77.3	63.8	60.8	51.2	13.7	A-4(2)	CL
379	28	9	100.0	94.3	91.0	86.7	67.1	17.6	A-4(4)	CL
384	31	11	100.0	90.7	83.7	78.6	64.0	16.6	A-4(5)	CL
389	30	10	100.0	92.9	83.1	79.7	67.2	17.8	A-4(5)	CL
394	33	9	100.0	89.1	81.2	75.2	69.6	14.9	A-4(5)	ML-CL
399	29	8	100.0	93.3	86.4	81.6	66.4	14.8	A-4(4)	ML-CL
404	34	10	100.0	92.1	85.0	78.6	71.5	14.6	A-4(6)	
409	33	11	100.0	94.1	82.5	77.3	69.6	16.3	A-6(6)	ML-CL
459	36	8	100.0	95.4	95.4	84.4	75.8	15.0	A-4(6)	ML
464	43	15	100.0	94.3	80.2	76.4	72.6	24.4	A-7-6(11)	ML-CL
469	38	13	100.0	92.3	91.7	87.1	74.1	27.5	A-6(9)	ML-CL
474	40	16	100.0	92.6	89.3	80.6	69.9	25.4	A-6(10)	ML-CL
479	43	16	100.0	92.5	89.8	81.5	72.2	26.5	A-7-6(11)	ML-CL
484	30	8	100.0	92.9	91.3	84.1	60.6	15.3	A-4(3)	ML-CL

percent and averaged 12 percent. The percent soil passing the No. 200 sieve ranged from about 43 to 87 and averaged about 69. Based on a chart and guidelines by Epps, et al 1970, and index data, hydrated lime and cement are suitable admixtures for the soils of Ky 11. According to the AASHTO classification system, the untreated soils classify as A-4, A-6, and A-7-6. Based on the unified classification system, the soils classify mainly as ML-CL.

Index properties of the untreated and treated soils obtained from the three stockpiles (located at Stations 273+00, 354+00, and 574+00) are shown in Table 6. Liquid limits of the stockpiled soils ranged from 36 to 43 percent. Plasticity indices ranged from 12 to 15 percent. The percent passing the No. 200 sieve ranged from 70 to 74. Based on the AASHTO classification system, the soils classified A-6(10), A-7-6(11), and A-6(8), respectively. The stockpiled soils classified as CL according to the unified soil classification system.

Treatment of the clay soils at this site with cement significantly affected the index properties. The clay soils became non-plastic. Classification of the soils changed from A-6(10) to A-4(0). Classification by the unified classification changed from CL to GM. The percent passing the No. 200 sieve was reduced from 74 to 39. Treatment with lime also produced some changes in the index properties. There was some reduction in the plasticity index. The percent passing the No. 200 sieve changed from 74 to 54. The AASHTO classification changed from A-6(10) to A-5(4) while the unified classification did not change. Treatment with the waste by product, AFBC, produced mixed results. Plasticity indices showed little, or no change. The percent passing the No. 200 sieve changed from

Table 6. Index properties of untreated and treated soils from stockpiles located at stations 273, 354, and 574, Ky 11

Sample Number	Site Number	Station Number	Liquid Limit (%)	Plasticity Index (%)	Specific Gravity	Grain-Size Analysis Percent Finer Than: No.4	No.10	No.40	No.200 (%)	Classification AASHTO	Unified	Percent of Chemical Additive (%)
Untreated	1	273+00	39	14	2.69	100	91.2	82.4	74.0	A-6(10)	CL	0
Untreated	2	3	43	15	2.80	100	92.4	82.0	73.0	A-7-6(11)	CL	0
Untreated	3	574+00	36	12	2.72	100	89.7	81.0	70.0	A-6(8)	CL	0
Cement	1	273+00	NP	NP	2.65	100	91.2	60.0	39.4	A-4(0)	GM	10
Lime	1	273+00	45	10	2.80	100	91.2	65.7	54.0	A-5(4)	CL	6
AFBC	1	273+00	47	15	2.83	100	91.2	77.5	64.0	A-7-5(9)	CL	4
AFBC	1	273+00	51	13	2.80	100	91.2	75.6	64.4	A-7-5(9)	MH	7
AFBC	2	3	43	12	2.80	100	91.2	80.3	72.6	A-7-5(9)	CL	4
AFBC	2	3	49	14	2.80	100	91.2	63.6	48.5	A-7-5(5)	GM	7

74 to about 49 to 73. The AASHTO classification of the treated soils were CL, MH, or GM. Treatment with AFBC did not appear to improve the classification. Index tests were not performed on the multicone kiln dust because this experimental section was added to the study near the end of the laboratory testing program and sufficient time was not available to perform detailed laboratory testing before construction.

Moisture-density relationships. The maximum dry density and optimum moisture content of soils treated with hydrated lime and the AFBC spent lime changed significantly as the percent of either of these chemical admixtures increased. Maximum dry densities and optimum moisture contents of untreated and treated samples (A,B,C) from Section AA-19 are compared in the upper portion of Table 7. Six percent hydrated lime was used for the treated specimens. Treatment of the natural clays with lime yielded maximum dry densities and optimum moisture contents that were lower and higher, respectively, than maximum dry densities and optimum moisture contents of the untreated clays. The maximum dry density of the clay soils decreased 4 to 7 percent when treated with 6 percent of hydrated lime. The optimum moisture content increased 1 to 45 percent when treated with hydrated lime.

Maximum dry densities and optimum moisture contents of untreated and treated soils from the three stockpiles at the KY

Table 7. Maximum dry densities and optimum moisture contents of untreated and treated soils from Section AA-19 and Ky Route 11

SAMPLE NUMBER	SITE NAME	TYPE OF CHEMICAL ADDITIVE	UNTREATED MAXIMUM DRY DENSITY (PCF)	UNTREATED OPTIMUM MOISTURE CONTENT (%)	TREATED MAXIMUM DRY DENSITY (PCF)	TREATED OPTIMUM MOISTURE CONTENT (%)	CHEMICAL ADDITIVE (%)
A	SECTION AA-19	LIME	90.1	31.0	86.8	31.3	6
B	SECTION AA-19	LIME	96.3	24.5	89.8	27.7	6
C	SECTION AA-19	LIME	98.6	14.3	91.4	20.8	6
STOCKPILE 1	KY 11 SITE 1	LIME	106.1	18.0	103.9	18.7	6
"	"	CEMENT	106.1	18.0	107.7	16.8	10
"	"	AFBC	106.1	18.0	102.5	20.0	4
"	"	"	106.1	18.0	101.7	20.5	6
"	"	"	106.1	18.0	100.0	19.0	8
"	"	"	106.1	18.0	99.5	19.0	12
STOCKPILE 2	KY 11 SITE 1	AFBC	-	-	102.7	21.2	5
STOCKPILE 3	"	"	112.8	16.3	105.7	16.2	5
"	"	"	112.8	16.3	103.0	18.8	10

11 site are shown in the lower portion of Table 7. The maximum dry density and optimum moisture content of the untreated soil from stockpile 1 are 106.1 pcf and 18.0 percent, respectively. Treatment with 6 percent hydrated lime reduced the maximum dry density by 2 percent and increased the optimum moisture content by about 4 percent. Treatment with 10 percent cement increased the maximum dry density from 106.1 pcf (untreated) to 107.7 pcf -- a 1.5 percent increase. Optimum moisture content decreased from 18.0 to 16.8 percent, or a change of 6.5 percent. A noticeable change occurred in the maximum dry density and optimum moisture content of soils mixed with the AFBC spent lime. Variations of the optimum moisture content and maximum dry density of specimens from stockpile 1 and the percent of AFBC are shown in Figure 10. Optimum moisture contents and maximum dry density of soils from stockpile 3, as a function of the percent AFBC spent lime, are shown in Figure 11. As shown in Figures 10 and 11 (and Table 7), the maximum dry density decreased 6 to 9 percent as the percent AFBC spent lime approached 10 to 12. The optimum moisture content increased 6 to 16 percent as the percent AFBC approached 10 to 12.

Optimum percentage of chemical admixture. Results of the different methods used to determine the optimum percentage of chemical admixtures are described below.

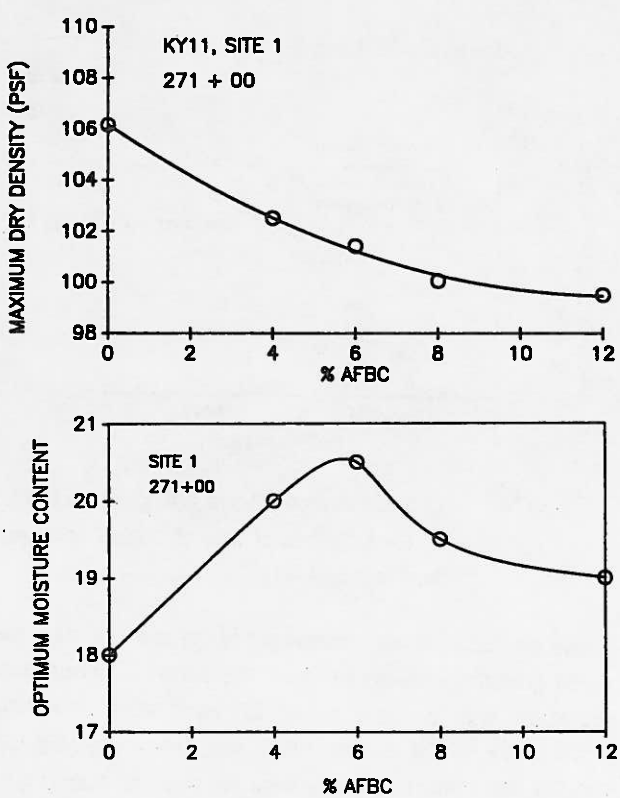

Figure 10. Variation of the maximum dry densities and optimum moisture contents of soils from stockpile number 1 with the percentage of AFBC spent lime

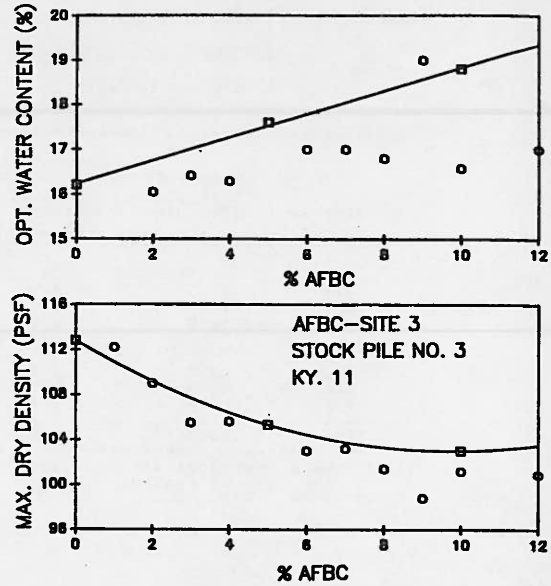

Figure 11. Variations of the maximum dry densities and optimum moisture contents of soils from stockpile number 3 with the percentage of AFBC spent lime

Unconfined compressive strengths. Results of unconfined compressive strengths of remolded soils from Section AA-19 are summarized in Table 8. All unconfined compressive tests were performed on bag sample A from Station 1630+00. The unconfined compressive strengths were performed on untreated, remolded specimens and remolded specimens treated with 6 percent of hydrated lime. Stress-strain curves obtained from the treated and untreated, remolded specimens are compared in Figure 12. The specimens were remolded to optimum moisture content and maximum dry density. Peak failure stresses (unconfined compressive strengths) of the treated specimens (A-1, A-2, A-3, A-4, and A-5) and untreated specimens (A-8, A-6, and A-7), as a function of curing time, are shown and compared in Figure 13. Specimen A-8 (untreated) was prepared by molding sample A in a CBR mold at optimum moisture content and maximum dry density. The specimen was allowed to soak and absorb water until vertical swell ceased. A specimen of the molded soil was obtained using a Shelby tube. Unconfined compressive strength of this specimen was 13.6 psi while the unconfined compressive stengths of two untreated, unsoaked specimens were 21.5 psi and 31.2 psi, respectively. Soaking the untreated soil reduced the remolded strength by about 37 to 56 percent. As shown in Figure 13, the 14-day strength of the treated specimen was some 3.5 to 5 times greater than the strength of the untreated, unsoaked specimens and about 8 times greater than the strengt of the untreated, soaked specimen.

Table 8. Results of unconfined compression tests performed on remolded, untreated specimens and specimens treated with 6 percent hydrated lime (bag sample A, station

SPECIMEN NUMBER	UNCONFINED COMPRESSIVE STRENGTH (PSF)	FAILURE STRAIN (PERCENT)	MOLDING CONDITIONS* WATER CONTENT (PERCENT)	MOLDING CONDITIONS* DRY DENSITY (PCF)	STANDARD COMPACTION** OPTIMUM WATER CONTENT (PERCENT)	STANDARD COMPACTION** MAXIMUM DRY DENSITY (PCF)	CURING TIME (DAYS)
UNTREATED SPECIMENS							
A-6	3100	4.0	30.3	87.1	31.0	90.1	1
A-7	4490	4.0	27.9	90.8	31.0	90.1	14
A-8(soaked)	1965	4.9	31.9	89.9	31.0	90.1	0
TREATED SPECIMENS (6% LIME)							
A-1	6450	2.7	35.9	83.9	31.3	86.8	0.1
A-4	6000	1.7	35.0	85.6	31.3	86.8	1.1
A-2	11000	1.4	35.9	84.7	31.3	86.8	5
A-3	12160	1.3	34.0	86.5	31.3	86.8	8
A-5	15800	1.6	32.3	87.2	31.3	86.8	14

* Water contents and dry densities of all specimens were determined at the time of testing.
** ASTM D 698.

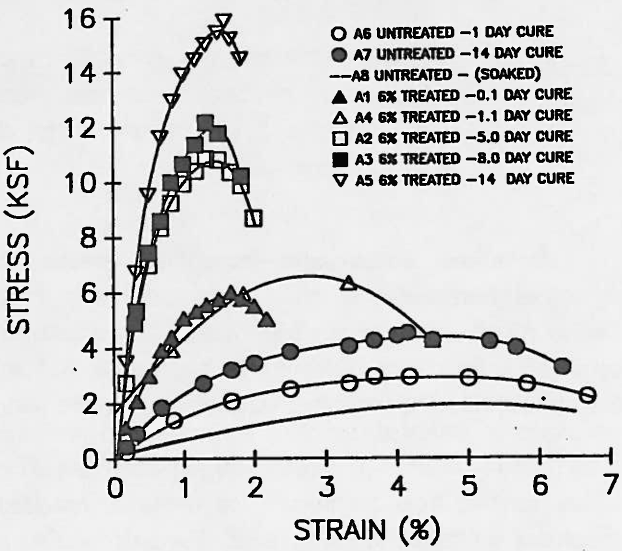

Figure 12. Unconfined compressive strength as a function of strain of treated and untreated soil specimens from section AA-19

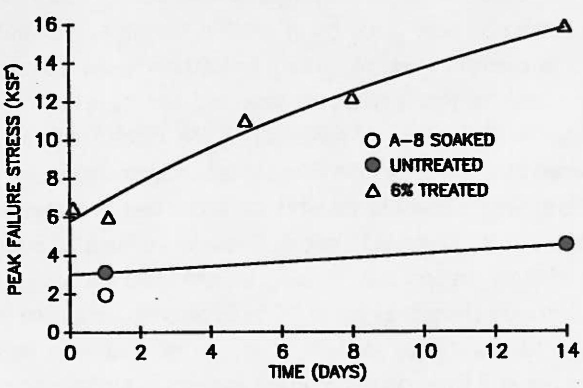

Figure 13. Variation of unconfined compressive strengths of untreated and treated specimens from section AA-19 and curing time

As shown in Figure 14, the strength of the lime treated soils continued to increase significantly with the logarithm of curing time (hours) while the untreated specimen did not increase significantly with the logarithm of curing time (hours). A trend analysis of these data (Figure 14) indicates that gain in strength may be expressed as (unconfined compressive strength, U , appears to be linear with the logarithm of time, t, hours)

$$U = 61.2 + 28.4(\text{Log } t) \qquad (11)$$

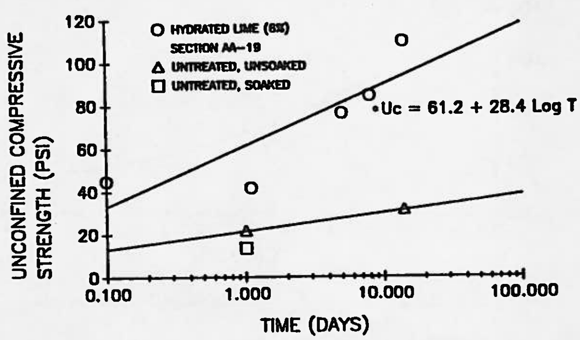

Figure 14. Unconfined compressive strengths as a function of the logarithm of time of treated specimens from section AA-19

Based on this equation, the strength of the soil-lime subgrade would gradually increase to about 146 psi at 2.7 years after placement and to 174.8 psi at 27 years after placement. Unfortunately, at the Section AA-19 site, specimens obtained from the lime-treated subgrade were too short for testing and unconfined strengths of field specimens could not be obtained. Although specifications had required a 6-inch compacted layer of lime-treated, soil subgrade, the treated subgrade was only 3.5 to 4 inches in thickness. Field unconfined compressive strengths were not available for comparison with the laboratory unconfined compressive strengths. Since the mixing efficiency of soil-lime mixtures in the field is usually lower than the effiency in the laboratory, the strengths given by equation 11 could be expected to be higher than field strengths.

The optimum percent hydrated lime selected for mixing with the subgrade soils of Section AA-19 was determined from pH tests performed by the manufacturer of the hydrated lime. Results of those tests are described in the section "Determinations based on pH tests". Unconfined compressive tests, as shown in Figure 12, were performed on the soil-lime specimens to confirm that significant strength gains could be obtained by using hydrated lime and that the engineering properties of the subgrade soils could be improved.

Unconfined compressive strengths of specimens from the KY 11 site were used to determine the optimum percents of hydrated lime, cement, multicone kiln dust, and the AFBC waste by-product or spent lime. In these series of tests, all treated, remolded specimens were cured for seven days. Variation of the unconfined compressive strength as a function of percent hydrated lime is shown in Figure 15. Soils for the lime tests were obtained from stockpile number 1. Based on the curve in Figure 15, the optimum percent hydrated lime is about 6 or 7.

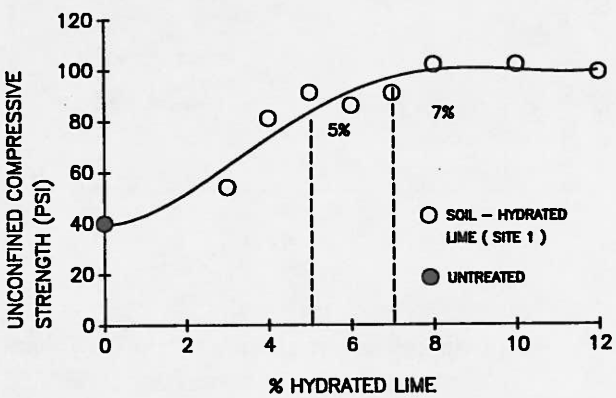

Figure 15. Unconfined compressive strengths of soil specimens from stockpile number 1, Ky Route 11, as a function of the percentage of hydrated lime

The value selected and used in field stabilization was 7 percent. As shown in Figure 15, the unconfined compressive strength of the untreated (unsoaked) specimen remolded to optimum moisture content and maximum dry density was about 40 psi. At an optimum value of 7 percent of hydrated lime, the unconfined strength of lime treated samples was some 150 percent greater than the strength of the untreated, remolded soil.

Two soil-cement series of unconfined compressive tests were performed on soils obtained from stockpiles number 1 and 3. Since the maximum dry density and optimum moisture contents of untreated and treated specimens were similar, all unconfined compressive specimens were molded to maximum dry density and optimum moisture content obtained from standard compaction tests on untreated specimens. For remolded specimens of soils from stockpile number 1, the variation of unconfined compressive strength and the percent cement is shown in Figure 16. The optimum percent cement is about 10 to 12. The average, unconfined compressive strength of three, untreated (unsoaked) specimens of soils obtained from stockpile number 1 and remolded to optimum moisture content and maximum dry density was about 40 psi. At the optimum percent cement, the unconfined compressive strength of the cement treated soils was about 265 psi, or the strength of the cement-treated soils was about 6 to 7 times greater than the

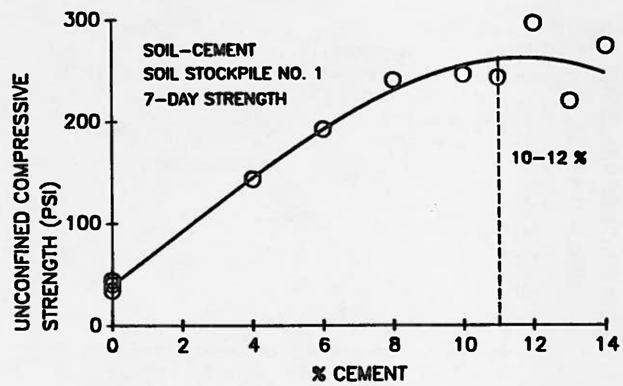

Figure 16. Unconfined compressive strength of soil specimens from stockpile number 1, Ky Route 11, as a function of the percentage of cement

strength of the untreated (unsoaked), remolded soil. The unconfined compressive strengths of cement-treated soils from stockpile number 3 are shown in Figure 17 as a function of the percent cement. Based on this plot, the optimum percent cement is about 10 to 12 percent. The unconfined compressive strength of a soil specimen remolded to optimum moisture content and maximum dry density and untreated (unsoaked) was 42.7 psi. At the optimum percent cement, the unconfined strength of the soil-cement specimens was about 470 psi. This strength was about 10 to 11 times the strength of the untreated strength. Values selected for two field trials of cement stabilized soil subgrades were 10 and 7 percent, respectively.

Unconfined compressive strengths of remolded soils obtained from stockpile number 1 and treated with the multicone kiln dust are shown in Figure 18 as a function of the percent of kiln dust. Based on these data, the optimum percent of kiln dust was about 6 to 8 percent. A value of 7 percent was used in the field. The unconfined compressive strength at optimum percent multicone kiln dust was about three times the strength of untreated, remolded specimens.

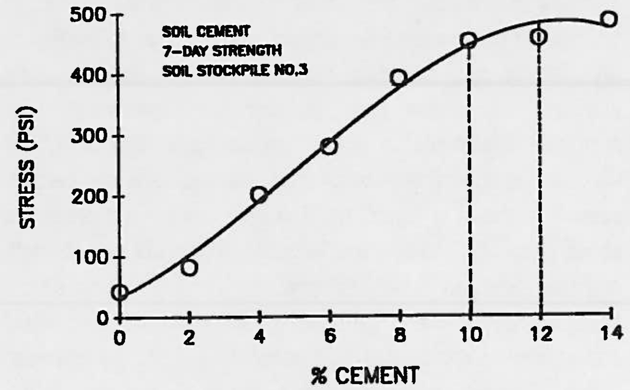

Figure 17. Unconfined compressive strength of soil specimens from stockpile number 3, Ky Route 11, as a function of the percentage of cement

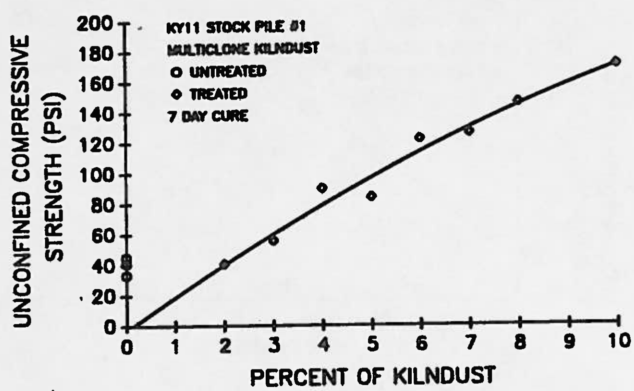

Figure 18. Unconfined compressive strength of soil specimens from stockpile number 1, Ky Route 11, as a function of the percentage of multicone kiln dust

Variations of unconfined compressive strength as a function of the percent AFBC spent lime are shown in Figures 19 and 20. The two series of unconfined compression tests were performed on remolded soils from stockpiles numbers 1 and 3. As shown in Figure 19, the optimum percent AFBC for soils from stockpile number 1 was about 5 or 6. For soils from stockpile number 3, the optimum percentage of AFBC was about 6. For the two trial sections constructed in the field, a value of 7 percent was used. Unconfined compressive strengths at the optimum percentage of AFBC spent lime were about 4 times greater than the strengths of untreated (unsoaked) remolded specimens.

Determinations based on pH tests. Results of pH tests of soil-hydrated lime mixtures for soils from Section AA-19 are shown in Figure 21. These test results indicated that 5 or 6 percent hydrated lime would be sufficient to stabilize the soils of Section AA-19. A value of 6 percent was used in the field. Since some admixture is lost during construction, then a normal practice is to add one percent to the optimum percent. Based on the relationship shown in Figure 7 and liquid limit values of soils of Section AA-19, the optimum percent lime is about 5.5 to 6.5.

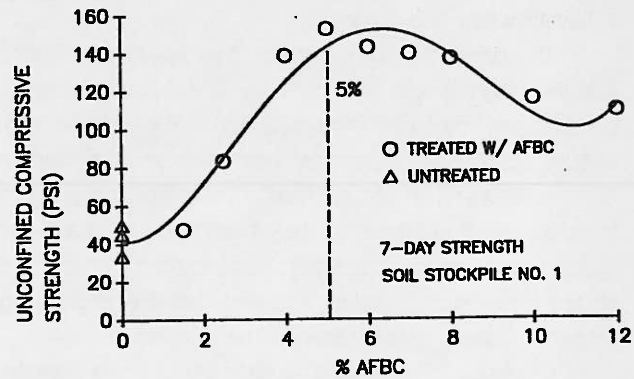

Figure 19. Unconfined compressive strength of soil specimens from stockpile number 1, Ky Route 11, as a function of the percentage of the AFBC spent lime

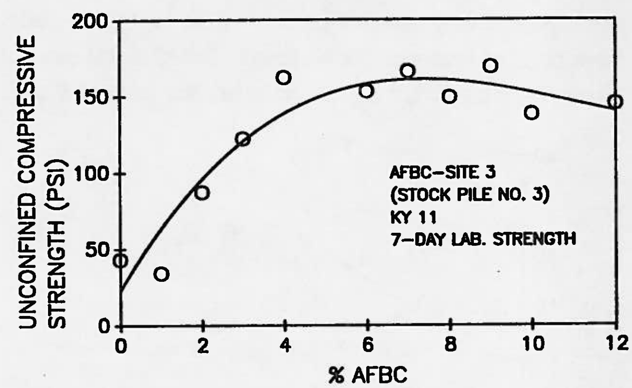

Figure 20. Unconfined compressive strength of soil specimens from stockpile number 3, Ky Route 11, as a function of the percentage of the AFBC spent lime

The two different methods (pH procedure and the method in Figure 7) gave essentially the same result.

Results of pH tests performed on mixtures of soils from KY 11 are shown in Figure 22. These results indicate that the optimum percent hydrated lime is about 5. Based on Figure 7, and using an average value of liquid limit of the soils from the Ky 11 subgrade, the indicated optimum percentage of hydrated limit is about 4.2 percent. Both of these values are slightly lower than the optimum percent hydrated lime (5 to 7) obtained from the unconfined compressive tests.

Although the procedure by Eades and Grimes was devised specifically for determining the optimum percent hydrated lime, the method was used with the AFBC spent lime material to determine if it was applicable to this material.

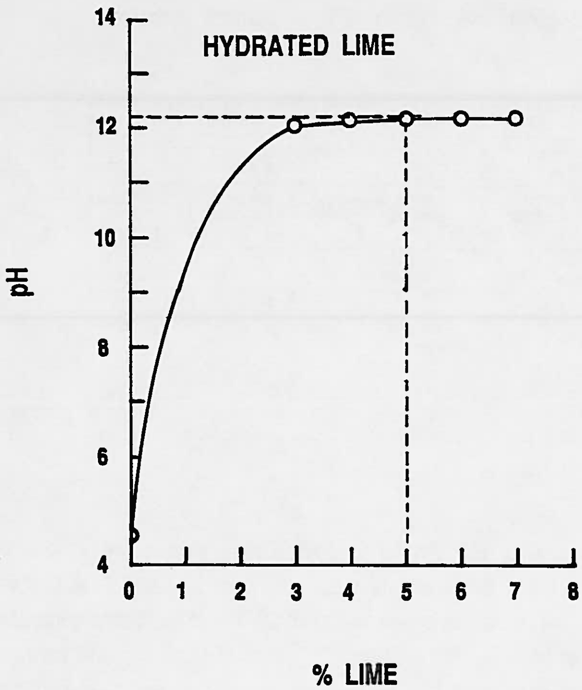

Figure 21. Variation of the pH-value of soil-hydrated lime mixtures with the percentage of hydrated lime, Section AA-19

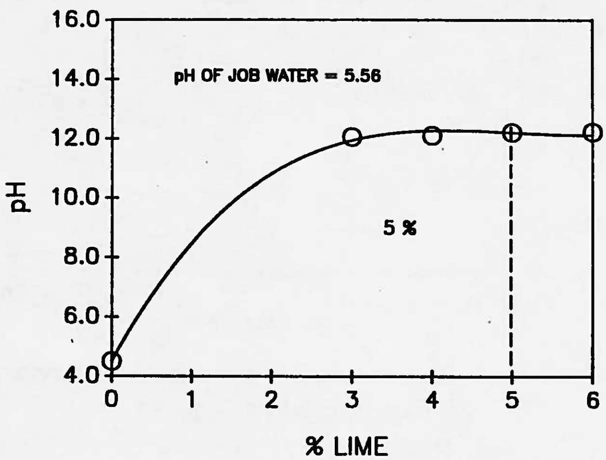

Figure 22. Variation of the pH-value of soil-hydrated lime mixtures with the percentage of hydrated lime, Ky Route 11

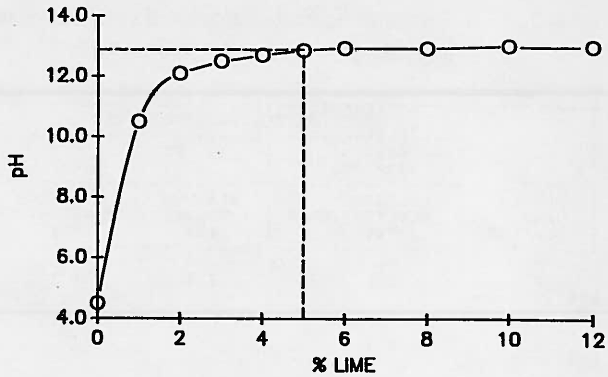

Figure 23. Variation of the pH-value of soil-AFBC spent lime mixture with the percentage of AFBC spent lime, Ky Route 11

Results of the pH tests on the KY 11 soils are shown in Figure 23. Values of pH as a function of the percent of AFBC spent lime indicate that 5 percent of AFBC spent lime is an optimum value. This value compares reasonably well with the optimum values obtained from the unconfined compression tests shown in Figures 18 and 19.

Bearing ratio and swell tests. Based on the ASTM bearing ratio test, the soaked CBR values of untreated specimens A, B, and C from Section AA-19 were 3.3, 2.7, and 0.8, respectively, as shown in Table 9. Soaked ASTM bearing ratio values of specimens A, B, C, which had been treated with six 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 from Station 1630+00. The soaked minimum Kentucky CBR value of specimen A without lime treatment was 2.6. This value occurred at 0.5-inch penetration. At 0.1-inch penetration, the soaked Kentucky CBR was 3.7 for the untreated soil. Minimum, soaked KYCBR values of specimens of sample A treated with six percent hydrated lime ranged from 7.1 to 42.4, as shown in Table 9. These values occurred at 0.5-inch penetration. Curing times at room temperatures (before immersion in the water tank) varied from 0 to 14 days. At 0.1-inch 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 (0.1-inch penetration) of treated specimens of sample A were some 9 to 37 times 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 0.5-inch penetration. The maximum CBR value occurred at 0.1-inch penetration. The CBR value decreased with increasing stress. Comparisons of the CBR values at 0.1-inch and 0.5-inch penetration for soil specimens treated with 6 percent hydrated lime and cured for different periods of time are compared in Figure 24. CBR-values at 0.1-inch penetration are much larger than CBR values at 0.5-inch penetration. Also, the CBR values at 0.1-inch penetration increased with curing time while CBR values at 0.5-inch penetration remain almost constant with increasing curing time. At 0.1-inch 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

Table 9. Results of bearing ratio tests of untreated specimens and specimens treated with 6 percent hydrated lime (bag sample A, Section AA-19)

SAMPLE NUMBER AND LOCATION	UNTREATED SPECIMENS: SOAKED ASTM CBR: 0.1-INCH PENETRATION** (%)	UNTREATED SPECIMENS: SOAKED KENTUCKY CBR: MINIMUM VALUE (%)	UNTREATED SPECIMENS: SOAKED KENTUCKY CBR: 0.1-INCH PENETRATION (%)	TREATED SPECIMENS (6% HYDRATED LIME): SOAKED ASTM CBR: 0.1-INCH PENETRATION (%)	TREATED SPECIMENS (6% HYDRATED LIME): SOAKED KENTUCKY CBR: MINIMUM VALUE (%)	TREATED SPECIMENS (6% HYDRATED LIME): SOAKED KENTUCKY CBR: 0.1-INCH PENETRATION (%)	CURING TIME (days)
A STA 1630+00	3.3	2.8	3.7	38.0	7.1*	32.3	0
					39.7*	58.0	3
					21.9*	59.5	7
					42.4*	137.3	14
B STA 1495+00	2.7			30.3			
C STA 1675+50	0.8			8.0			

*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.

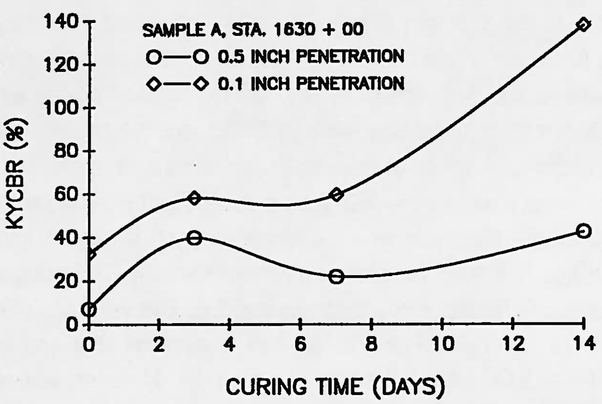

Figure 24. Comparison of KYCBR-values at 0.5-inch and 0.1-inch penetration for soil specimens from the Ky Route 11 site

small strains. Hence, the CBR value at peak failure stress is the more valid value than the CBR value at 0.5-inch penetration, which occurs after the peak failure stress has been reached.

Comparisons of values of total volumetric strain (swell) of the CBR specimens of treated and untreated soils from Section AA-19 are shown in Table 10. 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 six 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 6 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 10, swell strains obtained from the KYCBR test were reduced significantly when the soils were treated with hydrated lime. For the untreated soil(A), the strain was 4.4 percent. For four 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 25, the swell strains decreased with increasing curing time. The swell strains of the hydrated lime were only 1 to 12 percent (depending on the curing time allowed) of the swell strain obtained from the untreated specimen.

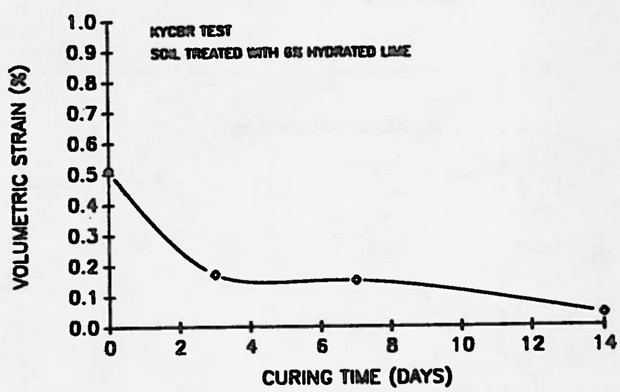

Figure 25. Swell as a function of curing time of hydrated lime-treated soils from Section AA-19

Results obtained from bearing ratio tests performed on soils from the KY 11 site and treated with hydrated lime, cement, and the AFBC spent lime are shown in Table 11. Also, the CBR value for the untreated soil was 3.7 (minimum value) and 4.1 percent, respectively, at 0.1- and 0.5-inch penetrations. Total swell strain of the untreated specimen was 3.9 percent (Figure 26). The KYCBR value of the soil-cement (10%) specimen was 300 at 0.1-inch penetration and 111 at 0.5-inch penetration. Swell strain of this specimen was essentially 0 (0.02 percent). The soil-hydrated lime specimen had KYCBR values of 67 and 45, respectively, at penetration values of 0.1-inch and 0.5-inch penetration values. The total swell strain of this specimen was 0.2 percent. Consequently, when the soils were treated with

Table 10. Comparisons of total volumetric strains observed from bearing ratio tests of treated and untreated soils from Section AA-19

SOIL SAMPLE AND SPECIMEN NUMBER	UNTREATED SOILS: ASTM BEARING RATIO TEST TOTAL VOLUMETRIC STRAIN (%)	UNTREATED SOILS: KYCBR TEST TOTAL VOLUMETRIC STRAIN (%)	SOILS TREATED WITH 6% HYDRATED LIME: ASTM BEARING RATIO TEST TOTAL VOLUMETRIC STRAIN (%)	SOILS TREATED WITH 6% HYDRATED LIME: KYCBR TEST TOTAL VOLUMETRIC STRAIN (%)
SOIL A:				
A(ASTM-U)	2.10	--		
A(KY-U)	--	4.37		
A(ASTM-6-0-T)			1.09	
A(KY-6-0-T)				0.51 (No Curing Time)
A(KY-6-3-T)				0.17 (3-Day Curing Time)
A(KY-6-7-T)				0.15 (7-Day Curing Time)
A(KY-6-14-T)				0.04 (14-Day Curing Time)
SOIL B:				
B(ASTM-U)	3.84			
B(ASTM-6-0-T)			0.22	
SOIL C:				
C(ASTM-U)	5.00			
C(ASTM-6-0-T)			2.40	

NOTE: 1. All specimens allowed one hour mellowing time when prepared.
2. ASTM - ASTM bearing ratio test (ASTM D 1883-73(1978)); 6 - refers to percent lime; U - untreated soil; 0, 3, 7, and 14 - refers to curing time in days at room temperature before specimen immersed in water tank; T - treated with 6 percent hydrated lime; and KY - KYCBR test (KM-64-501-76)

Table 11. Soaked, KYCBR-values of soils from the Ky Route 11 site in an untreated state and a state treated with lime, cement, and the AFBC spent lime

SAMPLE NUMBER	SITE NUMBER	CURING PERIOD (DAYS)	TYPE AND PERCENTAGE OF CHEMICAL ADMIXTURE	UNSOAKED PENETRATION VALUE 0.1inch	UNSOAKED PENETRATION VALUE 0.5inch	SOAKED PENETRATIO VALUE 0.1inch	SOAKED PENETRATIO VALUE 0.5inch	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	TOTAL SWELL (%)
U-1	1		NONE	--	--	3.7	4.1	115.6	16.9	110.1	19.9	5.1
C10	1	4	CEMENT-10%	--	--	300.0	111.1	123.6	15.9	116.6	13.0	0.02
HL1	1		HYDRATED LIME-6%	--	--	67.3	44.7	112.6	18.7	112.8	18.6	0.02
7	1	4	AFBC(7%)	--	--	47.7	32.7	114.0	17.0	107.4	20.2	3.1
7-1FB			AFBC(7%-FIELD BAG SAMPLE)			11.3	8.3	99.6	23.5		28.8	2.9
7-1FT		1	AFBC(7%-TRENCH)			57.7	39.5	93.6	25.9		31.30	0.8
15-2LAB			AFBC(15%)									
15-2LAB			AFBC(15%)			1.7						
30-1LAB			AFBC(30%)									
30-2LAB			AFBC(30%)									

cement or hydrated lime, there was a considerable reduction in swell strains (Figure 27) when compared to the swell strain of the untreated soil (Figure 26).

The values of KYCBR of an AFBC-soil specimen molded at 7 percent of AFBC spent lime were 48 and 33 percent at 0.1- and 0.5-inch penetrations, respectively. These values are some 13 to 9 times larger than KYCBR values of the untreated specimen. Total swell (strain) of the soil-AFBC mixture (7% admixture) was 3.1 percent. The value of swell was slightly lower than the swell strain (3.9 percent) observed for the untreated specimen. The KYCBR value of a specimen molded from a bag sample obtained during the field mixing of the AFBC and soil (near station 262+25) was 11.3 at 0.1-inch penetration. Total swell of this specimen was slightly less than 3 percent, as shown in Figure 28.

Approximately two months after construction of both AFBC spent lime - soil subgrade sections, severe differential swell or heave occurred as shown in Figure 29 and 30. The swell or humps occurred almost immediately after rainy periods. A close-up view of a swell area and pavement crack is shown in Figure 31. To investigate the pavement heave, a trench was excavated at Station 279+80. As shown in Figure 32, the soil-AFBC spent subgrade had heaved or swelled considerably. Both undisturbed and disturbed soil samples were obtained from

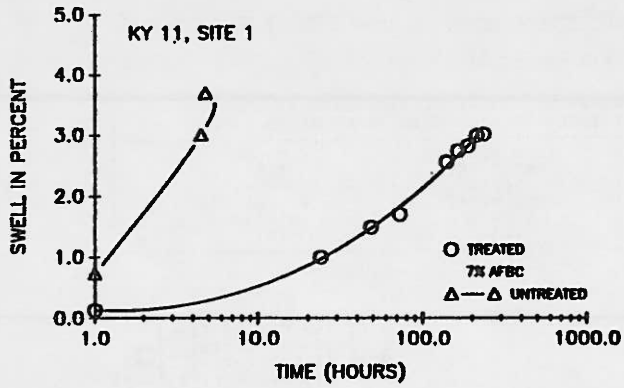

Figure 26. Swell of untreated soils and AFBC spent lime (7%)-treated soils from Ky Route 11 as a function of the logarithm of time

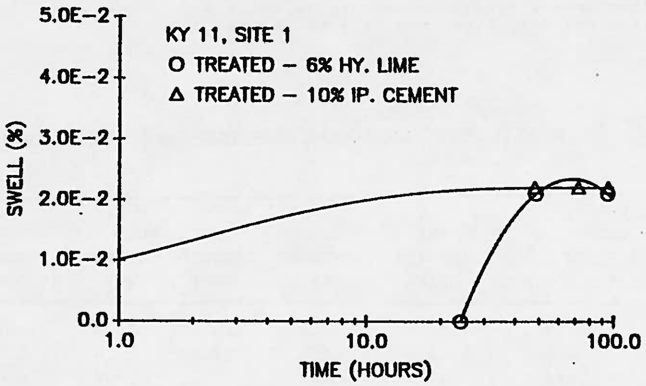

Figure 27. Swell of lime-treated and cement-treated soils from Ky Route 11 as a function of the logarithm of time

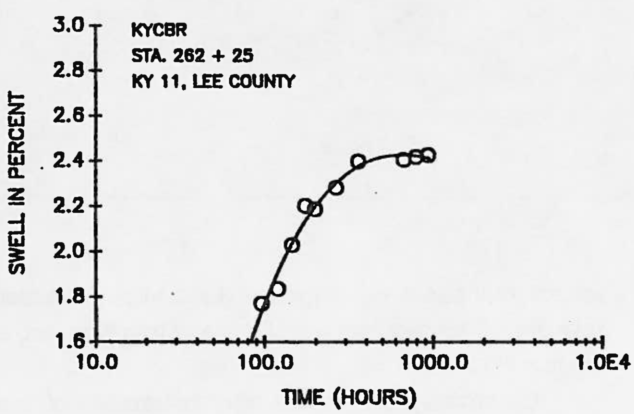

Figure 28. Swell of a specimen remolded from a field sample of AFBC spent lime and soil

the trench. Also, field moisture-density measurements were performed on the swollen AFBC-soil subgrade. To determine the swelling potential of compacted AFBC-soil mixtures, additional CBR-swell tests were performed.

Figure 29. View of pavement and crack heave on Ky Route 11 near station 280+00

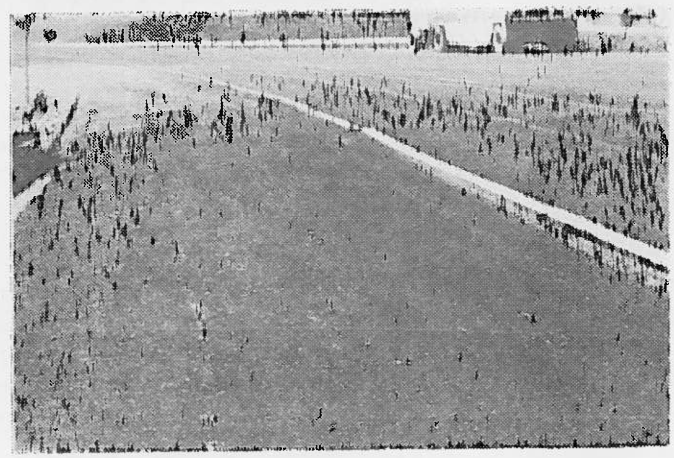

Figure 30. View of pavement humps on Ky Route 11 near station 560+00

Using a bag sample of the AFBC-soil mixture obtained from the trench at Station 279+80, a specimen, identified as 7-1FT in Table 10, was remolded in a CBR mold to average values of field moisture content and dry density measured on the AFBC-soil subgrade. These values were 26.4 percent and 98.1 pcf, respectively. The ASTM bearing ratio values at 0.1- and 0.5-inch penetrations were 57.7 and 39.5 percent, respectively. Swell (strain) of this specimen as a function the logarithm of time is shown in Figure 33. Total swell of this specimen in a period of about 48 days was only 0.8 percent. In the ASTM bearing ratio method, Designation D 1883-(78), specimens are soaked for 96 hours and then the bearing ratio test is performed. However, test 7-1FT was allowed to swell until the difference between consecutive readings was less than 0.003 inch. Based on the

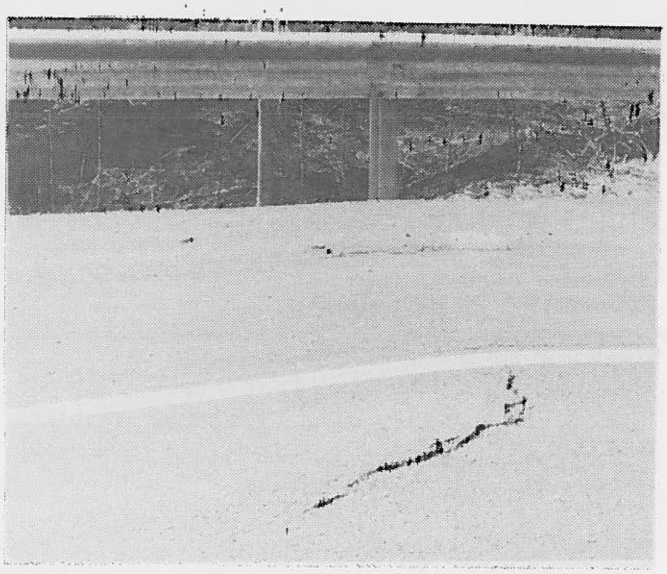

Figure 31. Close-up view of cracked pavement and hump on Ky Route 11

Figure 32.. View of pavement trench and AFBC-treated subgrade near station 279+80, Ky Route 11

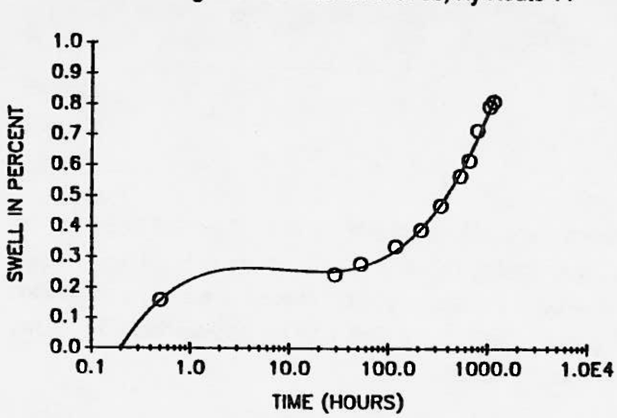

Figure 33. Swell of a remolded specimen from the trench at station 279+80, Ky Route 11, as a function of the logarithm of time

curve in Figure 31, the primary portion of the swell appears to have ceased. Unfortunately, secondary swell strain measurements were not obtained.

To examine the swelling nature of soil-AFBC spent lime mixtures, four additional CBR swell tests were performed on remolded specimens (stockpiles 1 and 3), using 15 percent and 30 percent of the AFBC material. These tests are identified in Table 12 as 15-LAB1, 15-LAB3, 30-LAB1, and 30-LAB3. The percent AFBC material used in these tests were higher than the 7 percent used in the field. Hence, swell strains measured in these tests may be higher than strains observed for a lesser percent the AFBC material. Swell strains as a function of the logarithm of time (in hours) are shown in Figures 34 through 37 for specimens identified as 15-LAB1, 15-LAB3, 30-LAB1, 30LAB3, respectively. In each of these tests, large swell strains occurred. The observed swelling of the four specimens has occurred over an 8-month period and is continuing. Volumetric swell of specimen 15-LAB1, molded from soils of stockpile number 1, is near 26 percent and primary swell has continued several months after the specimen was immersed in water. Results of a second swell test, performed on a remolded specimen (15-LAB3) of soils from stockpile number 3 and mixed with 15 percent of the AFBC material, is shown in Figure 35. Total swell of this specimen in a period of 4 months is 26.3 percent. This test was discontinued shortly after 4 months when it was observed that the swelling pressure had sheared one of the mold clamps (these clamps fasten the bottom of the mold to the mold base). Once the mold clamp had sheared, the vertical swell moved the bottom of the mold upward and invalid swell strains were obtained. However, as shown in Figure 35, the swell measurements showed that primary swell was completed at a time of about 1,487 hours (62 days) for specimen 15-LAB3 and secondary swell started before the clamp was sheared. A sufficient number of measurements of swell was obtained after completion of primary swell to establish the trend of secondary swell. As shown in Figure 35, the relationship of secondary swell and the logarithmic of time is linear.

Swell measurements of two specimens, which were remolded and mixed with 30 percent AFBC spent lime (30LAB1, 30LAB3) from stockpiles 1 and 3 are shown in Figures 36 and 37. In both cases, the total swell was 24 to 27 percent. The total swell in both cases was probably greater than the measured values because in each case the swell pressure of each specimen was sufficient to shear one of the mold clamps of each mold. Once this occurred, the measurements of swell were invalid since the bottom of the mold and top of the mold moved upward as the specimen swelled vertically. The CBR value of specimen 30-LAB3 was 1.7. The initial moisture content and dry density at compaction was 11.5 and 91 PCF, respectively. The moisture content in the top inch of the specimen was 73 percent. The water content increased

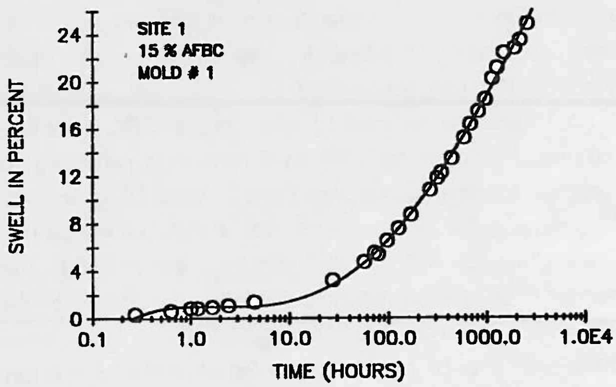

Figure 34. Swell- logarithm-of-time curve of a soil specimen treated with 15 percent of the AFBC spent lime

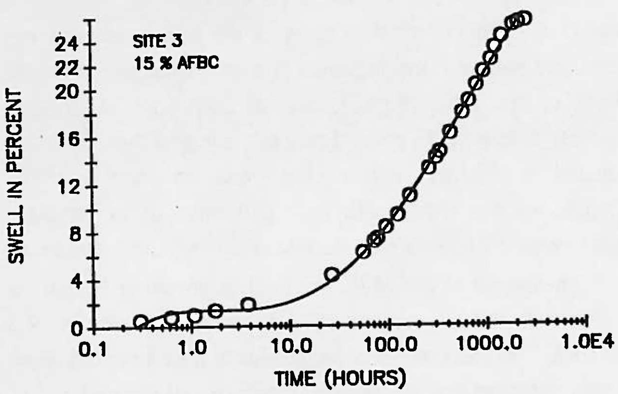

Figure 35. Swell- logarithm-of-time curve of a soil specimen treated with 15 percent of the AFBC spent lime

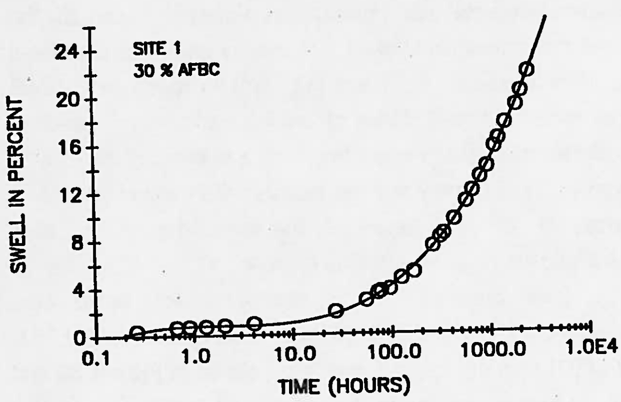

Figure 36. Swell-logarithm-of-time curve of a soil specimen treated with 30 percent of the AFBC spent lime

dramatically from the initial state of compaction to the final state. Also, the dry density decreased from 91 PCF to 79.3 PCF. As the moisture content increased, there was a decrease in dry density. As the dry density decreased (the volume increased), there was a large decrease in the shear strength and bearing ratio.

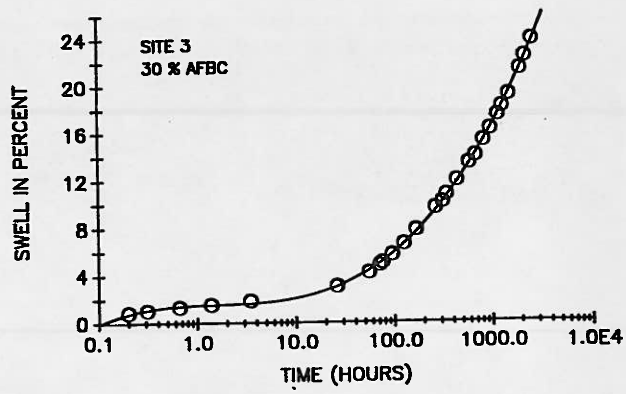

Figure 37. Swell- logarithm-of-time curve of a soil specimen treated with 30 percent of the AFBC spent lime

As a means of estimating the time required for completion of primary swell of the soil-AFBC spent lime mixture in the field, the swell-logarithm-of-time curves of specimens 7-FTLAB1 and 15-LAB3 were analyzed to determine a coefficient of swell (opposite of a coefficient of consolidation). Based on the curve in Figure 26 and the equation (Terzaghi 1936).

$$c_{ps} = \frac{TH^2}{t_{100}} \qquad (12)$$

where c_{ps} = coefficient of primary swell,
T = Dimensionless time parameters (see Terzaghi 1936)
H = thickness of laboratory specimen, and
t_{100} = time to completion of primary swell.

The coefficient of primary swell for the soil-AFBC mixture is

$$c_{ps} = \frac{(0.9)(4.504)^2 in^2}{1050 hrs}$$

$$c_{ps} = 0.018 in^2/hr.$$

Using the curve in Figure 35, the coefficient of primary swell of the soil-AFBC mixture is

$$c_{ps} = \frac{(0.9)(4.0)^2 \quad in^2}{1050 hrs}$$

$$c_{ps} = 0.0097 in^2/hr.$$

Hence, the coefficient of primary swell of the soil-AFBC mixture is approximately 0.018 to 0.0097 square inch per hour. Using the value of the later coefficient of swell, the time for completion of primary swell in the field may be approximated as follows (rearranging equation 11):

$$t_{100} = \frac{TH^2}{c_{ps}} \qquad (13)$$

and

$$t_{100} = \frac{(0.9)(12)^2 in^2}{0.0097 in^2/hr}$$

t_{100} = 13,361 hrs = 556.7 days = 1.53 yrs,

where H = thickness of soil-AFBC layer in the field = 12inches.

Section 1 of the soil-AFBC spent lime subgrade was constructed on August 8, 1987. From the above calculation, the date of completion of primary swell would be about February 15, 1989, or 557 days after construction of the AFBC spent lime-soil subgrade. These estimates should be viewed very cautiously and skeptically since laboratory behavior and field behavior may be entirely different. For example, laboratory specimens were subjected to a steady source of water while the source of water in the field varies or fluctuates due to wet and dry periods and some time is required during the early life of the soil-AFBC subgrade to reach a steady-state moisture environment. The completion of primary swell may occur over a longer time period than indicated by these thereotical calculations.

Another problem with the soil-AFBC mixture is illustrated in Figure 35. Swell of the soil-AFBC mixture exhibits secondary swell which occurs after completion of the primary swell. As shown in this plot, the relationship between the secondary swell and the logarithm of time (hours) is linear. Based on this curve, a coefficent of secondary swell, c_{ss}, may be approximately as

c_{ss} = 0.062 = slope of the swell-logarithm of time relationship.

Using this coefficient, the amount of swell that would occur between the time of completion of primary swell and some selected time after completion of primary swell may be approximated from the relationship

$$H_{ss} = C_{ss} H \log\left(\frac{t_p}{t_{100}}\right) \qquad (14)$$

where

H_{ss} = secondary swell over a given time period (inches),

H = thickness of soil-AFBC layer (inches)

t_p = selected time after completion of primary swell (days), and

t_{100} = time of completion of primary swell (days).

Let t_p equal 5 years (1825 days) and t_{100} equal 557 days (the estimated time to complete primary swell), then

H_{ss} = (0.062)(12in.)(log(1825/557)) = 0.4 inch.

From 5 years after completion of primary swell to 27.4 years after construction, the secondary swell is

H_{ss} = (0.062)(12 in.)(log(9444/2382)) = 0.45 inch.

From the time (557 days) of completion of primary swell to a time of about 27.4 years after construction, the total predicted secondary swell is 0.9 inch.

These calculations indicate that secondary swell of the soil-AFBC spent lime will be a problem in the future, but this problem may be controllable. However, estimates of secondary swell should be viewed cautiously since field and laboratory behavior may be completely different.

Field Moisture Content-Dry Density Compliance Data.

Results of field moisture-density tests obtained from the nuclear density gage are summarized in Tables 12 through 14. Maximum dry density and optimum moisture contents of subgrade samples collected at various station numbers are shown in the left portion of those tables. Maximum dry densities obtained from standard compaction tests (ASTM D698) may be adjusted on the basis of the percent material retained on the number 4 sieve and according to a nomograph in the Department's manual of Kentucky Test Methods (1983). Also, correction of maximum dry density for oversized material may be computed based on a formula given in NAVFAC 1982. These two methods yield essentially the same result (Hopkins 1988). In some instances, the Department's personnel adjusted the laboratory maximum dry density for oversized material. However, adjustments in laboratory maximum dry densities were not always made. The adjusted, or reference, values of maximum dry densities were used to compare to values of dry density obtained from the nuclear density meter. In the right-hand portions of Tables 12 through 14, values of relative compaction, or the ratio of adjusted, maximum dry density to field dry density, are shown. Based on 101 nuclear density tests, the relative compaction averaged 98.3 percent (standard deviation equals + 2.7 percent) for all subgrade sections on KY 11. Since specifications required that field dry densities must be 95 percent of maximum dry density, all subgrade sections were compacted according to the dry density specification.

Table 12. Dry density - moisture content compliance data for sections of soil subgrades treated with AFBC spent lime (7%), cement (10%), hydrated lime (7%), and multicone kiln dust (7%)

STATION NUMBER	AASHTO T-99 LABORATORY DATA MAXIMUM DRY DENSITY (PCF)	AASHTO T-99 LABORATORY DATA OPTIMUM WATER CONTENT (%)	ADJUSTED LABORATORY DATA +4 MATERIAL (%)	ADJUSTED LABORATORY DATA REFERENCE MAXIMUM DRY DENSITY (PCF)	LOCATION OF FIELD TEST (STA. No.)	FIELD MOISTURE-DRY DENSITY TESTS FROM NUCLEAR GAGE DRY DENSITY (PCF)	FIELD MOISTURE-DRY DENSITY TESTS FROM NUCLEAR GAGE WATER CONTENT (%)	RELATIVE COMPACTION (%)	FIELD WATER CONTENT MINUS OPTIMUM WATER CONTENT (%)
AFBC SPENT LIME SUBGRADE, STATION 260+00 to 317+50									
264+00	105.0	16.0	23.2	113.0	262	109.2	16.4	96.7	0.4
			23.2	113.0	264+10		17.1	95.4	1.1
			17.8	111.0	268		18.5	95.1	2.5*
			17.8	111.0	273		16.3	96.6	0.3
269+00	112.0	15.0							
274+00	109.0	15.0	22.1	116.2	276	110.7	17.1	95.3	2.1*
			22.1	116.2	277	110.5	17.2	95.3	2.2*
279+00	108.0	15.0	15.5	113.0	280	108.0	16.9	95.6	0.9
284+00	113.0	15.0	6.9	115.0	285	109.4	16.1	95.1	1.1
289+00	106.0	16.0							
294+00	108.0	18.0	12.5	112.0	296	107.8	17.9	96.3	-0.1
299+00	105.00	16.0	14.9	110.0	312	109.4	16.1	99.8	0.1
				110.0	315	104.5	20.3	95.0	4.3*
304+00	113.0	14.0							
309+00	116.0	14.0							
CEMENT (10%) SUBGRADE, STATION 317+50 to 348+00									
314+00	116.0	9.0	21.2	116.0	317+20	116.0	9.8	100.0	0.8
				116.0	318+50	113.1	9.8	97.4	0.8
319+00	116.0	12.0	11.5						
324+00	113.0	11.0	4.7	116.0	320	116.0	14.2	100.0	3.2*
329+00	113.0	11.0	4.3						
334+00	121.0	11.0	20.1						
344+00	126.0	10.0	31.2						
HYDRATED LIME (7%) SUBGRADE, STATION 348+00 to 402+50									
349+00	122.0	9.0	24.2						
354+00	123.0	9.0	19.1						
359+00	118.0	13.0	10.0						
364+00	121.0	12.0	11.5	-	-	-	-	-	-
multicone kiln dust (7%)									
404+00	110.0	18.0	7.9						
409+00	112.0	14.0	5.9						
414+00	112.0	19.0	5.4						
419+00	115.0	15.0	8.4	111.0	422+50	110.0	15.8	99.1	0.8
424+00	115.0	11.0	6.5	115.0	426	110.0	14.9	95.6	3.9*
429+00	117.0	10.0	15.4	112.0	430+25	117.2	12.4	105.1	-3.6*
	112.0	16.0	2.9	112.0	431+60	118.9	11.7	106.1	-4.3*

With regard to moisture content, compaction specifications require that the field moisture content shall not be 4 percent less or 2 percent above optimum moisture content. Differences between field and optimum moisture contents are shown in the right-hand portion of Tables 12 through 14. A negative sign in front of the difference indicates that the field water content was less than the optimum moisture content while a positive sign indicates that the field moisture content was greater than optimum moisture content. An average value of the differences between field and optimum moisture contents was -0.25 (2.3) percent. This means that generally the subgrades were compacted at moisture contents slightly dry of optimum moisture contents. In 78 cases, field values of moisture content met moisture content specifications. In 23 of 101 tests conducted along the subgrades of KY 11, the moisture content specification was not met. In 19 of 23 failure cases, the field water content was wet of optimum moisture content and was greater than a value of optimum moisture content plus 2 percent. In 4 of the 23 failure cases, the field moisture content was less than optimum moisture minus 4 percent. Although some adjustments were made in laboratory maximum dry density to account for oversized material, laboratory optimum moisture contents were not always adjusted for oversized material by personnel of the Department of Highways. When oversized material is present, both maximum dry density and optimum moisture contents may need to be adjusted (see NAFAC 1982).

Table 13. Dry density - moisture content compliance data for section of soil subgrade treated with 7 percent of cement, station 429+50 to 522+00

	AASHTO T-99 LABORATORY DATA		ADJUSTED LABORATORY DATA			FIELD MOISTURE DENSITY TESTS NUCLEAR GAGE			
STATION NUMBER	MAXIMUM DRY DENSITY (PCF)	OPTIMUM WATER CONTENT (%)	+4 MATERIAL (%)	REFERENCE MAXIMUM DRY DENSITY (PCF)	LOCATION OF FIELD TEST (STA. NO.)	DRY DENSITY (PCF)	WATER CONTENT (%)	RELATIVE COMPACTION (%)	FIELD WATER CONTENT MINUS OPTIMUM WATER CONTENT (%)
434+00	111.0	16.0	9.2						
	112.0	16.0	7.1						
439+00	111.0	15.0	9.2						
444+00	111.0	16.0	5.2	111.0	445	108.1	18.0	97.4	2.0
				111.0	443	108.9	15.2	98.1	-0.8
449+00	116.0	14.0	8.2	116.0	434	114.9	12.5	99.0	-1.5
				116.0	436+50	115.2	13.4	99.2	-0.6
				116.0	438+50	114.3	12.9	98.5	-1.1
				116.0	441	110.4	13.7	95.1	-0.3
				116.0	448+25	110.6	10.8	95.3	-3.2
				116.0	442+50	110.0	16.5	94.8	2.5*
				116.0	445+50	111.4	12.2	96.0	-1.8
				116.0	447+50	112.5	15.5	97.1	-0.5
454+00	113.0	15.0	4.8	113.0	453	111.7	14.2	98.9	-0.8
459+00	117.0	14.0	5.1	117.0	459	110.2	15.6	94.5	1.6
				117.0	456	110.6	14.9	94.6	0.9
464+00	110.0	16.0	1.5	110.0	466	112.8	14.1	102.6	-1.9
				110.0	462+50	106.4	13.9	96.2	-2.1
469+00	111.0	17.0	7.6	111.0	465	109.3	16.2	98.4	-0.8
				111.0	469	110.7	14.7	99.7	-2.3
				111.0	469+20	119.1	13.6	107.3	-3.4
474+00	112.0	15.0	1.8	112.0	473	107.4	18.0	95.9	3.0
				112.0	470	106.1	14.1	94.7	-0.9
479+00	112.0	18.0	6.9	112.0	473+50	106.9	18.5	95.4	0.5
				112.0	477+50	110.3	15.9	98.5	-2.1
484+00	116.0	14.0	8.2						
449+00	116.0	14.0	8.2	116.0	464	113.6	14.1	99.6	0.1
					461	115.8	14.0	99.8	0.0
					456	111.3	14.3	95.8	0.3
					453+50	115.4	14.9	99.4	0.9
					448+50	116.7	13.1	100.0	-0.9
484+00	116.0	14.0	8.2	116.0	490	110.5	14.3	95.0	0.3
				112.0	500+00	114.5	14.2	102.2	-2.8
				112.0	499+50	112.7	13.3	100.6	-3.7
				112.0	494+00	112.6	12.9	100.6	-4.1*
				112.0	496+50	112.5	11.3	100.4	-5.7*
509+00	112.0	15.0	18.5	117.0	512+00	111.5	19.6	95.2	4.6*
				112.0	505+00	114.9	12.2	102.6	-2.8
				112.0	506+25	112.2	13.2	100.2	-1.8
				112.0	510+75	117.1	13.0	104.6	-2.0
519+00	116.0	11.0	5.9	116.0	518+50	111.4	14.5	96.0	3.5*
				116.0	519+00	112.1	14.3	96.6	3.3*
				116.0	521+50	113.7	14.5	98.0	3.5*
				116.0	514+00	110.6	8.7	95.3	-2.3
				116.0	516+50	111.3	14.0	95.9	3.0*

Since the percent retained on the number 4 screen was small in many cases, no adjustments were made in laboratory optimum water contents. Based on the data in Tables 12 through 14, compaction specifications were generally met.

Pavement Swell Measurements.

After the pavement surfaces on the two sections of subgrade that had been treated with the AFBC spent lime showed noticeable signs of non-uniform heave, or swelling, survey points were established on those surfaces to monitor vertical movements. Initial elevations were obtained in early October, 1987 at several locations within the two experimental AFBC spent lime sections. A total of ten stations were monitored in the two distressed sections. Survey points at one location in each of the other experimental sections were established and monitored so that vertical movements of all experimental sections could be compared. Subsequent measurements were obtained in late October, November, and December of 1987 and March of 1988. Table 15 lists results of the optical surveys conducted at locations in the two AFBC spent lime sections, the hydrated lime section, one of the cement sections, and the multicone kiln dust section. Elevation differences shown in this table are in terms of minimum, maximum, and average changes in vertical movements. These vertical movements were recorded during a period from October 27, 1987 to March 1, 1988. Upward movements of the pavement in the cement and hydrated lime sections that were observed during the survey period were insignificant. The maximum change in elevation of the pavement in the cement section was 0.07 inches. In the hydrated lime section, the maximum change in elevation was 0.19 inch. In the multiclone kiln dust section, the pavement moved upward during the survey period a maximum value of 0.49 inch. To date, this amount of movement has not affected the performance of the pavement in this experimental section.

The most noticeable changes in pavement elevations that were recorded during the survey period occurred in the two AFBC spent lime subgrade sections as shown in Table 15.

Table 14. Dry density - moisture content compliance data for a section of soil subgrade treated with AFBC spent lime (station 532+00 to 576+00 and an untreated subgrade section (station 522+00 to 532+00)

STATION NUMBER	AASHTO T-99 LABORATORY DATA MAXIMUM DRY DENSITY (PCF)	AASHTO T-99 LABORATORY DATA OPTIMUM WATER CONTENT (%)	ADJUSTED LABORATORY DATA +4 MATERIAL (%)	ADJUSTED LABORATORY DATA REFERENCE MAXIMUM DRY DENSITY (PCF)	LOCATION OF FIELD TEST (STA. No.)	FIELD MOISTURE-DRY DENSITY TESTS FROM NUCLEAR GAGE DRY DENSITY (PCF)	FIELD MOISTURE-DRY DENSITY TESTS FROM NUCLEAR GAGE WATER CONTENT (%)	RELATIVE COMPACTION (%)	FIELD WATER CONTENT MINUS OPTIMUM WATER CONTENT (%)
524+00	113.0	16.0	6.8	113.0	526+00	108.1	21.1	96.4	5.1*
				113.0		115.1	14.9	101.8	-1.1
529+00	111.0	17.0	7.0	115.5	529+15	117.6	13.5	101.7	-2.1
				115.5	524+85	116.0	15.1	100.4	-1.3
				117.2	531+75	111.3	12.7	94.9	-1.1
534+00	116.0	14.0	9.9	116.0	547+25	114.6	13.9	98.8	-0.1
					545+50	111.9	15.7	96.5	1.7
					542	117.7	11.8	101.5	-2.2
539+00	110.0	13.0	5.7						
544+00	115.0	14.0	8.4						
545+00	113.0	14.0	7.9	113.0	540	112.1	13.5	99.2	-0.5
				113.0	537	116.5	17.0	103.0	3.0*
				113.0	532+75	113.6	13.8	99.6	-1.2
				113.0	537	112.9	15.2	99.0	0.2
				113.0	534+50	114.3	14.0	100.3	1.0
554+00	117.0	13.0	10.7	117.0	549+00	113.0	13.9	96.6	0.9
				117.0	554+25	113.0	12.8	96.6	-0.2
				117.0	551+25	119.2	8.8	101.8	-4.2*
559+00	110.0	17.0	3.8	114.0	561+50	108.4	18.6	95.1	1.6
				114.0	557+10	111.8	15.6	98.1	-1.4
				114.0	561+50	108.2	12.9	95.0	-2.1
				114.0	565+50	109.9	16.5	96.4	1.5
				114.0	574+00	109.7	14.6	96.2	-0.4
569+00	114.0	15.0	6.8	114.0	572+50	113.7	13.8	99.8	-1.2
				114.0	569+50	114.0	13.5	99.5	-1.5
				114.0	576	116.7	13.0	102.4	-2.0
				114.0	573	113.9	11.8	99.9	-3.2
				114.0	570+50	113.0	12.5	99.1	-2.5
				114.0	569	111.3	16.0	97.6	1.0
				114.0	565	111.4	10.4	97.7	-4.6*
				114.0	565+25	111.4	10.4	97.7	-4.6*

Table 15. Pavement swell measurements

Station/ Location	Admixture	Minimum Swell (in.)	Maximum Swell (in.)	Average Swell (in.)	Monitoring Station Located on:
270+00, T	AFBC	-0.132	0.696	0.204	Fill Section
270+00, 6	AFBC	-0.156	0.684	0.135	
270+00, 22	AFBC	0.000	0.780	0.301	
280+00, T	AFBC	0.384	2.316	1.372	Cut/Fill Section
280+00, 6	AFBC	0.612	2.088	1.306	
280+00, 20	AFBC	0.612	1.824	1.297	
285+00, T	AFBC	0.024	0.732	0.248	Cut Section
285+00, B	AFBC	0.060	0.948	0.338	
285+00, 22	AFBC	0.432	0.816	0.607	
300+00, T	AFBC	0.600	1.668	1.088	Fill Section
300+00, 24	AFBC	0.756	1.932	1.154	
334+00, T	CEMENT	-0.060	0.036	-0.008	Fill Section
334+00, 22	CEMENT	-0.048	0.072	0.014	
379+00, T	LIME	-0.024	0.144	0.044	Cut Section
379+00, 22	LIME	0.002	0.192	0.016	
406+00, T	MKD	0.192	0.456	0.272	Cut/Fill Section
406+00, 22	MKD	0.252	0.492	0.388	
549+00, T	AFBC	0.312	2.004	0.886	Cut/Fill Section
549+00, 24	AFBC	0.672	1.248	0.946	
555+00, T	AFBC	1.656	2.988	2.167	Fill Section
555+00, 24	AFBC	1.056	2.640	2.240	
555+00, 34	AFBC	2.220	2.940	2.408	
559+00, T	AFBC	0.204	3.456	1.487	Fill Section
559+00, 10	AFBC	0.336	2.028	1.289	
559+00, 30	AFBC	1.896	3.372	2.564	
564+00, T	AFBC				Cut/Fill Section
564+00, 24	AFBC				
569+00, T	AFBC	0.864	2.820	1.728	Cut/Fill Section

Figures 29 and 30 are typical views of pavement humps observed in the two sections. The humps generally were perpendicular to centerline and caused a very uneven ride. As shown in Table 15, the maximum upward movements observed during the survey period in the first AFBC spent lime section that was constructed on the project (Station 260+00 to 317+00) was 2.3 inches (Station 280+00). Considerable swell and development of pavement humps had occurred in this section before optical surveys were conducted. Actual upward movements for this section was larger than the movements shown in the top portion of Table 15. In the second section of AFBC spent lime subgrade (Station 532+00 to 576+60) the maximum upward movement was 3.5 inches (Station 559+00). Values shown in Table 15 for this section are very close to actual values since elevations were obtained soon after pavement construction and before movements were observed. The severity of the swell of the AFBC spent lime subgrade (and pavement) is illustrated in Figure 38. In this plot, the elevation changes that occurred during the survey period are plotted as a function of horizontal distance (perpendicular to centerline) at Station 555+00. This plot shows large increases in swell during the winter months, which is normally a wet period in the region. This indicates that the AFBC spent lime subgrades had a large affinity for water.

To confirm that the subgrades had swelled, a trench was excavated near Station 279+80 in a heaved area. A view of the heaved subgrade and pavement is shown in Figure 32. Moisture contents of samples from the trench subrade were some 6 to 8 percent higher than values obtained during construction. The AFBC absorbed water during the survey period. The locations of survey stations in cut sections or embankment areas may affect the elevation readings that were obtained to determine whether pavement heave, or swell, was occurring. The relative locations of the survey points in relation to cut or fill sections of the roadway are indicated in Table 15. If foundation and embankment settlement occurred during the survey period, then the elevation changes would have been less than the values shown in Table 15. Moreover, the foundations of embankments on KY 11 were very thin and generally were only about 0 to 5 feet in thickness. Foundation settlement was nominal (settlement measurements obtained from settlement platforms located on foundations at three locations showed no settlement during the survey period). Embankment settlement did not occur at these three locations. The embankments on Ky 11 were compacted with heavy compactors (60,000 pounds) at several embankment sites.

Field CBR Tests.

In-place California Bearing Ratio (CBR) tests were generally performed before and after subgrade stabilization. Unfortunately, this was not the case for all experimental sections. Penetrations and calculations for in-place CBR tests were performed in accordance with ASTM D 1883-(73). "Bearing Ratio of Laboratory Compacted Soils", except that the tests were performed on the soils in it's actual in-situ conditions. Table 16 lists results of CBR and moisture content testing on section AA-19 of the AA highway. Testing on treated subgrade indicated that in-place values of CBR ranged from about 9 to 38 percent. In-place moisture contents ranged from 16 to 30 percent. The average in-place CBR was 24 and the average moisture content was 24.9 percent for Stations 1495+00 to

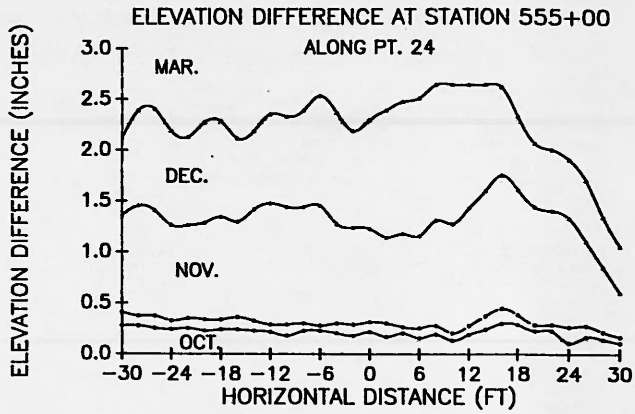

Figure 38. Pavement swell recorded at station 555+00 as a function of horizontal distance

1615+00. Unfortunately, in-place CBR values were not obtained prior to stabilization for direct comparison. Testing on the untreated subgrade (Stations 1652+50 to 1675+00) was performed in the fall of 1987 and again in the spring of 1988. The average in-place CBR and moisture content was 29 and 12.5 percent, respectively, for the October visit.

After the prepared subgrade had been allowed to sit through the winter, additional testing at the same locations revealed that the in-place CBR values had fallen to 2 while the moisture content of the soil had nearly doubled to 24.4 percent. The bearing strength of the untreated soils decreased significantly as the water content of the soils increased. This illustrates the relationship between bearing strength of fined-grained soils and changes in moisture contents. Although the moisture contents of the treated subgrade soils were nearly identical to the moisture contents of the untreated soils after exposure during the winter, the bearing strengths of the treated soils were nearly 12 times larger than bearing strengths of the untreated subgrade soils. Without stabilization, difficulties would have been encountered in construction of the pavements at this site. Table 17 lists results of CBR and moisture content testing on the first AFBC section of KY 11. Testing was performed prior to stabilization and again 7 days after treatment of the soil. Before treatment, CBR's ranged from about 20 to 42 while in-situ moisture ranged from about 6 to 18 percent. The average in-place CBR was 29.5 while the average in-situ moisture content was 11.6 percent. Seven days after treatment, the tests were repeated at the same locations. In-place CBR's ranged from 34 to 53 while in-situ moisture content ranged from 5 to 19 percent. Overall, the average in-place CBR increased to 40.8 while the in-situ moisture content decreased to 10.1 percent. The decrease in moisture content cannot be explained since the optimum moisture content for the section was about 16 percent. Five of the eight locations tested showed an increase in bearing strength after treatment. However, there was not a significant change in moisture content of the treated versus untreated soils.

Table 16. Results of field CBR tests performed on hydrated lime-soil subgrade of Section AA-19

STATION NUMBER	UNTREATED SUBGRADE CBR'S WATER CONTENT AND DATES 10/09/87 CBR	WC	04/05/88 CBR	WC	TREATED SUBGRADE CBR'S WATER CONTENT AND DATES 09/30/87 CBR	WC
1495+00					9	20.7
1505+00					24	27.0
1515+00					23	26.1
1525+00					23	21.1
1535+00					17	25.0
1545+00					38	25.5
1555+00					19	29.5
1565+00					20	19.7
1575+00					32	29.2
1585+00					15	29.1
1595+00					33	15.7
1605+00					32	25.6
1615+00					33	29.6
1652+50	NA	10.0	1	24.0		
1655+00	29	16.0				
1657+50	27	13.0	1	25.0		
1662+50	27	12.0	1	26.0		
1667+50	27	12.0	2	26.0		
1670+00	27	12.0	4	22.0		
1672+50	38	12.0	2	26.0		
1675+00	31	13.0	2	22.0		

Table 17. Comparison of field CBR values for AFBC-soil subgrade, (KY 11, Station 260+00 to Station 317+00)

Station Number	Age at Test (Days)	In-Place CBR (%)	In situ Moisture Content (%)
267+00 lt	before	37	7.6
267+00 lt	7	*	6.1
268+00 rt	before	26	6.1
268+00 rt	7	39	6.9
270+50 rt	before	*	7.4
270+50 rt	7	47	5.3
273+00 lt	before	*	13.5
273+00 lt	7	36	13.4
278+00 rt	before	42	18.1
278+00 rt	7	34	7.8
280+50 lt	before	20	11.8
280+50 lt	7	*	11.1
293+00 rt	before	22	12.5
293+00 rt	7	36	11.4
293+50 lt	before	30	16.1
293+50 lt	7	53	19.0

NOTE: * indicates insufficient data for CBR computation

The results of testing are inconclusive with respect to the benefits of stabilization at this time. In-place CBR tests also were performed on the prepared subgrade in front of the stabilization crew for the remaining experimental sections. Due to the rapid movement of the construction crew, in-place CBR's could not be obtained 7-days after treatment for all experimental sections. Results of these tests are presented in Table 18. On October 2, 1987, after the pavement surface of the AFBC sections began showing signs of non-uniform swelling, the asphaltic concrete pavement was cored to perform in-place CBR's and obtain moisture content samples. Two areas were targeted for testing and were identified as a "humped area" and a "non-humped area". The humped area (Station 279+79) had an in-place CBR of 38 and an in-situ moisture content of 23.9 percent. The non-humped area had an in-place CBR of 9 and

Table 18. Results of field CBR values, Ky 11

Station Number	In-Place CBR	In-situ Moisture Content (%)	Treated/ Untreated	Experimental Section
397+50 rt	*	6.6	U	LIME
398+00 lt	*	5.9	U	LIME
399+00 rt	*	6.4	U	LIME
400+00 lt	*	8.6	U	LIME
401+00 rt	*	7.3	U	LIME
401+50 lt	39	5.8	U	LIME
405+00 rt	33	6.8	U	MKD
406+00 rt	35	8.2	U	MKD
407+00 rt	*	9.7	U	MKD
409+00 lt	40	8.3	U	MKD
412+00 lt	34	8.1	U	MKD
415+00 rt	31	8.4	U	MKD
418+00 lt	10	12.0	U	MKD
515+00 rt	13	15.5	U	CEMENT
520+00 rt	11	14.2	U	CEMENT
525+00 rt	44	16.5	U	N/S
530+00 rt	15	24.9	U	N/S
535+00 rt	*	16.7	T	AFBC
540+00 rt	73	19.3	T	AFBC
552+50 rt	57	21.2	T	AFBC
552+50 lt	54	13.3	T	AFBC
555+00 rt	37	22.5	T	AFBC
555+00 lt	70	18.4	T	AFBC
557+50 rt	47	22.1	T	AFBC
565+00 rt	7	15.5	U	AFBC
565+00 rt	54	17.9	T	AFBC
572+50 lt	48	10.7	U	AFBC

NOTE: * indicates insufficient data for CBR computation
N/S indicates non-stabilized

an in-situ moisture content of 24.0 percent. As a follow-up, a trench was cut nearly one week later in an area where the pavement had formed a hump (near Station 279+82). Two in-place CBR's were performed in the trench. One test area was in the right wheel path of the southbound lane and the other test area was located near the edge of the lane. The in-place CBR for the right wheel path was 30 while the in-situ moisture content was 26 percent. The in-place CBR near the edge of the lane was 40 and the in-situ moisture content was about 24 percent. The two areas that were tested had similar moisture contents. It must be emphasized that the in-situ moisture contents were found to be 50 percent higher than the optimum moisture content used in construction of the AFBC spent lime treated subgrade (about 16 percent). Two in-place CBR's were performed during April 1988 on the first AFBC spent lime section. The pavement had been milled at that time so that it was easy to observe where the humped and non-humped areas were located. In-place CBR and moisture content measurements were obtained at each area. The thickness of each core was measured in an effort to determine how much thickness of asphaltic concrete base remained after milling. The humped area (Station 305+55) had an in-place CBR of 13 and an in-situ moisture content of 36.1 percent. The non-humped area had an in-place CBR of 37 and an in-situ moisture content of 27.0 percent. These measurements are the reverse of what was previously found, although the same area was not tested. During the fall, the humped area had an in-place CBR of 38 while the non-humped area had an in-place CBR of 9. The moisture contents of the non-humped areas were similar (27 versus 24). The important aspect of this is the fact that the moisture content in the humped area, tested the following spring, had increased by an additional 50 percent over that which was determined the previous fall. This increase in in-situ moisture is directly related to the decrease in bearing capacity (38 versus 13). Additional CBR and moisture content data were obtained in the fall of 1988. Test were obtained in each of the experimental sections with the exception of the non-stabilized and AFBC north section. The in-situ moisture contents for the first AFBC section remained quite high (34.5 and 29.8 percent). The in-place CBR's also were quite high (32 and 19 respectively). A summary of these results is presented in Table 19.

Table 19. Results of field CBR values obtained in the fall of 1988, KY 11

Station Number	In-place CBR	In-site Moisture Content (%)	Experimental Section
312+50	32	34.5	AFBC
316+50	19	29.8	AFBC
321+50	47	16.6	CEMENT
325+50	75	28.5	CEMENT
371+50	30	19.1	LIME
381+75	41	20.7	LIME
425+00	97	10.7	MKD
428+00	37	5.7	MKD
433+75	39	5.1	CEMENT

Comparison of Field and Laboratory Strengths.

Unconfined compressive strengths of specimens obtained from treated and untreated subgrades of KY 11 are shown in Table 20. The unconfined strengths of specimens obtained from the two AFBC spent lime treated subgrades ranged from 21.9 to 68.1 psi for curing periods ranging from 2 to 35 days. The unconfined strengths averaged 43.2 psi, or a cohesive value of 21.6 psi. For specimens that had cured for a period of 11 to 35 days, the unconfined compressive strength ranged from 25.3 to 65.8 psi and averaged 48.2 psi. Based on a relationship developed by Hopkins 1986, which relates the soaked CBR and cohesion, C,

$$CBR = (0.473)\, 10^{(1.01 \log C)} \qquad (15)$$

or the CBR values of the subgrade ranged from 6 to 16 percent for curing times ranging from 11 to 35 days. Based on average values of all specimens and equation 15, the average CBR value was 10.5 percent. This value is some 1.8 times greater than the recommended, minimum CBR value of 6 (Hopkins 1986). The field strengths are considerable lower than laboratory strengths. Unconfined compression strengths of laboratory AFBC spent lime - specimens were approximately 150 psi. Based on an average value of 43.2 psi, the field strengths were approximately 30 percent of values obtained from laboratory strengths.

There are at least two reasons for this difference. First, the mixing efficiency of admixture - soil materials in the laboratory is much better than the efficiency that may be obtained in the field. Field strengths will be lower than laboratory strengths. The unconfined compressive strengths

Table 20. Unconfined compressive strengths of field specimens from the treated and untreated subgrade sections of KY 11

STATION NUMBER	TYPE AND PERCENT OF STABILIZER	UNCONFINED COMPRESSIVE STRENGTH (psi)	FIELD MOISTURE CONTENT (%)	FIELD DRY DENSITY (PCF)	CURING PERIOD (DAYS)
320+50,6'RT.	CEMENT(10%)	73.9	19.4	108.0	7
320+50,CL	CEMENT(10%)	73.9	19.4	107.8	28
333+00,16'RT	CEMENT(10%)	127.2	13.8	107.4	28
342+50,CL	CEMENT(10%)	86.8	14.7	101.1	27
342+50,CL	CEMENT(10%)	87.6	14.5	102.9	27
343+00,CL	CEMENT(10%)	86.0	15.1	104.1	
274+00,CL	UNTREATED(0%)	63.3	17.8	111.5	35
340+50,CL	UNTREATED(0%)	45.6	16.2	115.1	
343+00,CL	UNTREATED(0%)	64.8	14.4	119.5	20
350+00,CL	UNTREATED(0%)	47.7	16.6	111.7	
398+00,CL	UNTREATED(0%)	59.2	19.4	111.81	
549+50,CL	UNTREATED(0%)	24.6	11.6	121.3	30
546+00,CL	UNTREATED(0%)	32.7	14.7	121.3	30
343+00,CL	LIME(6%)	86.0	15.4	104.1	28
348+50,CL	LIME(6%)	52.4	23.6	95.8	14
349+00,CL	LIME(6%)	32.1	15.8		9
349+00,CL	LIME(6%)	40.7	18.0	104.7	28
350+00,CL	LIME(6%)	55.8	19.8	104.3	28
399+00,CL	LIME(6%)	58.7	16.0	113.8	21
399+00,7'LT.	LIME(6%)	60.0	15.2	115.2	31
402+50,CL	LIME(6%)	20.9	18.4	102.3	21
405+00,CL	KILNDUST(6%)	41.7	23.5	102.1	7
405+00,3'LT.	KILNDUST(6%)	26.0	20.0	104.9	7
414+00	KILNDUST(6%)	13.0	15.5	109.0	7
417+00,9'LT.	KILNDUST(6%)	35.9	14.9	109.8	7
413+00,7'LT.	KILNDUST(6%)	86.3	17.6	113.8	28
	KILNDUST(6%)	42.8	14.7	106.5	28
261+00,CL	AFBC(7%)	43.8	15.2	111.7	2
261+00,CL	AFBC(7%)	67.0	15.0	109.1	7
264+00,10'RT	AFBC(7%)	68.1	15.1	108.3	15
264+15	AFBC(7%)	21.9	15.2	109.3	2

obtained in the laboratory were performed on specimens that had been remolded at a relative compaction of about 100 percent. In the field, the subgrade was compacted at a relative compaction less than 100 percent. With a decrease in relative compaction below 100 percent, there is a relatively large decrease in the unconfined compressive strength (Mathis 1988).

Unconfined compressive strengths of field specimens of soil cement ranged from 86 to l27 psi and averaged about 89 psi. The aging period of those specimens was 27 or 28 days. The soil-cement specimens were obtained from the cement section that extends from Station 317+00 to 348+00. This section of subgrade was treated with 10 percent cement. Based on equation 15, values of CBR ranged from 36 to 63 percent and averaged 44 percent. The average, unconfined compressive strength (89 psi) of field specimens was about 45 percent of laboratory strengths Unconfined compressive strengths of hydrated lime-soil specimens from the subgrade of KY 11 averaged 51 psi and ranged from 21 to 86 psi for curing periods ranging from 9 to 28 days. In terms of CBR values computed from equation 13, the CBR values averaged 12.5 percent and ranged from 5 to 21 percent.

The average value was about 2 times larger than the recommended value of 6 percent. The average field strength was about 50 percent of the unconfined strengths obtained from laboratory values. The average strength of field specimens of multicone dust-soil mixtures was 41 psi. The field specimens of mutlicone dust-soil mixtures was 41 psi. The field strengths ranged from 13 to 86.3 psi for curing times ranging from 7 to 28 days. The 28-day strengths were higher than the 7-day strength. Based on equation 15, the average, computed value of CBR was about 10, or about 1.7 times the minimum value (6 percent) recommended for field construction. Unconfined compressive strengths of specimens obtained from the untreated subgrade of KY 11 were fairly high and averaged 48 psi and ranged from 24.6 to 64.8 psi. The average value of CBR computed from equation 15 is 12. This value is about 2 times the recommended value of 6. An average value of soaked CBR values obtained from 31 laboratory tests of untreated specimens of the KY 11 subgrade soils was 7.7 percent. Slightly more than 42 percent of the tests gave CBR values that ranged from 1.4 to 6.1 percent. The high values of CBR and unconfined compressive strengths were a result of the dry conditions at the site during construction and moisture contents were relatively stable during construction and of the time of sampling. Future sampling and in-situ testing will be performed to determine whether the undrained strengths decrease with exposure to moisture content. Laboratory CBR tests indicate that the bearing strength of the Ky 11 soils may significantly decrease with exposure to moisture. CBR data in Table 16 illustrates how large decreases in bearing strength may occur as the moisture content of untreated soils increase. The data in Table 16 are a summary of results obtained from in-situ CBR testing performed at different stations on Section 19 of the AA Highway at different times. Initially, the in-situ CBR values of the untreated subgrade shortly after construction were very high. These values, which were obtained on October 9, 1987, averaged 29.4 percent and ranged from 27 to 38 percent. The water content of the subgrade soils averaged 12.5 percent. After the subgrade had been exposed to winter conditions, the in-situ CBR values, which were obtained some 7 months later on April 5, 1988 at the same station numbers, averaged only 1.9 percent. The moisture content of the subgrade soils had increased to an average value of 24.4 percent and nearly doubled in value. After treatment with hydrated lime, the in-situ CBR values averaged 24.5 percent and nearly doubled in value. After treatment with hydrated lime, the in-situ CBR values averaged 24.5 percent and the moisture content averaged 24.9 percent. Future in-situ CBR tests will be conducted to determine if there is a increase or decrease in bearing strength of the treated subgrades.

Interpretation of Road Rater Deflection Measurements

The Road Rater vibratory loading is approximately sinusoidal. The dynamic loading (sine wave) of the Road Rater has been approximated as a square wave as illustrated by Figure 39. Superposition principles may be used to compute the surface deflection at each velocity transducer location. Deflections are computed for the loadings associated with one of the load feet. By symmetry and superpositioning, the deflections for one load foot may be doubled to represent the deflections associated with the two load feet (Sharpe, et al, 1979)

The Road Rater applies a dynamic loading to the pavement. In theory, dynamic and/or wave propogation analyses techniques (Mamlouk, 1985 and 1987) should be used for analysis of deflection. However, for the sake of simplification, the measured deflection basins have been interpreted in terms of static analyses and layer elastic theory. More specifically, measured Road Rater deflections have been assumed to have resulted from a static load with a peak to peak vibratory load superimposed on the static load. In this situation, the static load used in analysis and interpretation of the dynamic deflections is the peak-to-peak magnitude of the square wave as illustrated in Figure 39.

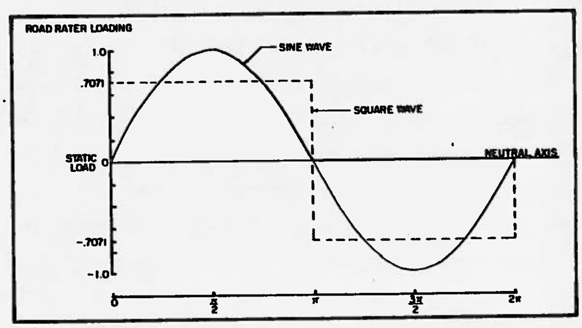

Figure 39. Approximation of dynamic loading of Road Rater using a square wave form

Elastic layer principles may be used to compute theoretical deflections for the applied loadings and the specific locations of each Road Rater velocity tranducer. There are a number of multi-layer elastic computer programs which may be used to compute deflections, stresses, and strains in pavements. The Chevron N-Layer computer program has been used in Kentucky to model pavement behavior (Deen, et al, 1971 and Sharpe, et al, 1979) was used for this analysis.

The following input parameters are required as input into the Chevron N-Layer computer program:

- o Thickness of each layer (inches)
- o Young's modulus of elasticity for each layer (psi)
- o Poisson's ratio for each layer
- o The coordinate of each required answer point (corresponding to the location each velocity transducer)
- o Loading applied to the road surface

o Contact pressure (applied load / contact area) for one loading foot of the Road Rater.

An array of layer moduli were used in combination with the constructed layer thicknesses and assumed values of Poisson's ratio. These parameter were entered into the Chevron N-Layer computer program to generate a matrix of simulated surface deflection basins corresponding to a number of combinations of layer moduli. The various combinations of layer moduli, layer thicknesses, and Poissons ratio are summarized in Table 21.

Table 21. Parameters for elastic layer simulations of Road Rater conditions

LAYER	LAYER THICKNESSES (IN)	DYNAMIC LOADINGS (LBF)	POISSON'S RATIO	ELASTIC MODULI (KSI)
Simulation No. 1				
1	6	600	0.45	3, 6, 12, 18, 24, 30, 36, 48, 60, 80, 100
2	Semi-infinite	600	0.45	6, 12, 18, 24, 30, 36, 48, 60, 80, 100
1	6	1,200	0.45	3, 6, 12, 18, 24, 30, 36, 48, 60, 80, 100
2	Semi-infinite	1,200	0.45	6, 12, 18, 24, 30, 36, 48, 60, 80, 100
1	12	600	0.45	3, 6, 12, 18, 24, 30, 36, 48, 60, 80, 100
2	Semi-infinite	600	0.45	6, 12, 18, 24, 30, 36, 48, 60, 80, 100
1	12	1,200	0.45	3, 6, 12, 18, 24, 30, 36, 48, 60, 80, 100
2	Semi-infinite	1,200	0.45	6, 12, 18, 24, 30, 36, 48, 60, 80, 100
Simulation No. 2				
1	6	600	0.15	3, 6, 12, 18, 24, 30, 36, 48, 60, 80, 100, 125, 150, 175, 200
2	Semi-infinite	600	0.45	6, 12, 18, 24, 30, 36, 48, 60, 80, 100
1	6	1,200	0.15	3, 6, 12, 18, 24, 30, 36, 48, 60, 80, 100, 125, 150, 175, 200
2	Semi-infinite	1,200	0.45	6, 12, 18, 24, 30, 36, 48, 60, 80, 100
1	12	600	0.15	3, 6, 12, 18, 24, 30, 36. 48, 60, 80, 100, 125, 150, 175, 200
2	Semi-infinite	600	0.45	6, 12, 18, 24, 30, 36, 48, 60, 80, 100
1	12	1,200	0.15	3, 6, 12, 18, 24, 30, 36, 48, 60, 80, 100, 125, 150, 175, 200
2	Semi-infinite	1,200	0.45	6, 12, 18, 24, 30, 36, 48, 60, 80, 100

Simulations have been determined for two different conditions:

o Simulation No. 1: Deflections on untreated subgrade (12" or 6") over a semi-infinite untreated layer;

o Simulation No. 2: Deflections on treated subgrade (12" or 6") over a semi-infinite layer of untreated subgrade.

Each simulation utilized a multi-layered elastic approach to compute the theoretically expected deflections. Simulation No. 1 was used in combination with measured deflections on untreated subgrades to determine the elastic stiffness of the untreated subgrade prior to stabilization with any adixture. Equations were determined for each Road Rater sensor location wherein deflection was related to elastic stiffness or modulus of elasticity. Measured deflections corresponding to each sensor location were used as input into appropriate equations to determine associated elastic moduli (stiffnesses) corresponding to each sensor. A mean, maximum, and minimum elastic stiffness (modulus) were determined for the overall data sample. These results of analyses of sections of KY 11 and the AA-Highway are are summarized in Table 22.

Table 22. Mean, maximum and minimum elastic moduli

SECTION	STATIONS TESTED BEGINNING	ENDING	MEAN (KSI)	MAXIMUM (KSI)	MINIMUM (KSI)
Ky. Route 11					
AFBC First Section	263+00	292+00	73	264	9
AFBC Second Section	532+00	540+00	77	204	5
Hydrated Lime	376+00	401+00	46	90	12
Multi-Cone Kiln Dust	422+50	429+00	93	279	19
Cement	326+00	338+50	137	279	52
AA Highway					
Hydrated Lime	1495+00	1615+00	112	125	150

Note: All test at 7 days.

Simulation No. 2 was used in combination with measured deflections on treated subgrade materials to determine the elastic stiffness of treated subgrade materials. Equations relating elastic stiffness and deflections were determined for each Road Rater sensor location. Measured deflections were used to determine associated elastic moduli (stiffness). Resulting mean moduli are summarized in Table 23 and compared with in-situ CBR tests. The results of these analyses were checked by comparing the deflection basins for the mean of measured deflections (for each section) versus the modelled deflection basins from the elastic layer simulations. The results of these analyses are presented in Table 24 and an illustration is presented in Figure 40.

Deflections also have been obtained after placement of the various layers of crushed stone and asphaltic concrete materials for the KY 11 and AA-Highway pavement sections. These data have not been analyzed at this time. The results will be presented in a subsequent report.

Benefits of Soil Stabilization

Information presented in this paper documents procedures used to determine the amounts and types of materials which may be used to stabilize subgrade soils. Information also is presented which documents the short term field performance of stabilized soils. Results of deflection testing for the KY 11 section indicates early elastic moduli (age 7 days) on the order of 127,000 psi for the soil-cement sections, 78,000 psi for the AFBC-soil, 93,000 psi for the multicone kiln dust-soil, and 46,000 psi for the hydrated lime-soil sections. At the time of testing, the in-situ (untreated) elastic modulus was a nominal 24,000 psi. Results from AA Highway, Section 19 indicated a nominal subgrade modulus of 9,000 to 21,000 psi before treatment with hydrated lime and a modulus of 100,000 to 125,000 psi after treatment with hydrated lime for the top 6 inches of the subgrade. Estimates of elastic modulus were obtained after seven days curing of the stabilized layer. These moduli appear reasonable relative to results from other studies

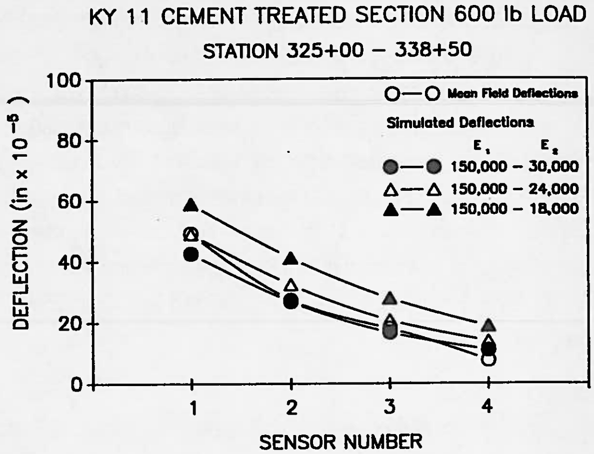

Figure 40. Comparison of Measured Deflection Basins and Theoretical Deflection Basins for 600 lb-force Dynamic Loading

Table 23. Comparison of stiffness by Road Rater versus other test methods

Section	Stations Tested Beginning	Stations Tested Ending	Mean Moduli Estimated From Road Rater Tests Treated (ksi)	Mean Moduli Estimated From Road Rater Tests Untreated (ksi)	Modular Ratio Treated/Untreated	Laboratory CBR Tests	Average Inplace CBR	Estimated Stiffness In-Place CBR Tests (ksi)
Ky 11								
Before Treatment								
Subgrade	262+00	562+00		24		4	32	47
After Treatment								
AFBC First Section	263+00	292+00	73	24	3.1	48	56	85
AFBC Second Section	532+00	540+00	77	24	3.2	48	46	69
Hydrated Lime	376+00	401+00	46	24	1.9	67		
Multicone Kiln Dust	422+50	429+00	93	24	3.9			
Cement	326+00	338+50	137	24	5.7	300		
AA Highway								
Before Treatment (fall)	1651+00	1675+00		16			36	53
Before Treatment (spring)	1652+50	1675+00		11				3
After Treatment								
Hydrated Lime	1495+00	1615+00	112	16	7.0		25	37

Table 24. Comparison of mean measured deflections with simulated deflections. Elastic modulus of layer 2 assumed equal to 24,000 psi

			Deflections (in x 10^{-5})			Layer Modulus (psi)	
Section		Sensor 1	Sensor 2	Sensor 3	Sensor 4	Layer 1	Layer 2
600 lb Dynamic Loading							
Ky 11							
AFBC	Measured Deflections	86.7	41.8	17.0	6.8		
First Section	Simulated Deflections	77.6	37.9	20.6	13.1	48,000	24,000
263+00 - 293+00		70.3	36.8	20.7	13.2	60,000	24,000
		62.3	35.4	20.7	13.4	80,000	24,000
AFBC							
Second Section	Measured Deflections	84.9	51.7	36.8	10.1		
532+00 - 540+00	Simulated Deflections	77.6	37.9	20.6	13.1	48,000	24,000
		70.3	36.8	20.7	13.2	60,000	24,000
		62.3	35.4	20.7	13.4	80,000	24,000
Hydrated Lime	Measured Deflections	83.4	25.3	23.0	22.3		
376+00 - 401+00	Simulated Deflections	88.9	39.1	20.5	13.0	36,000	24,000
		77.6	37.8	20.6	13.1	48,000	24,000
Multicone Kiln Dust	Measured Deflections	63.4	30.8	29.5	25.5		
422+00 - 429+00	Simulated Deflections	62.3	35.4	20.7	13.4	80,000	24,000
		57.1	34.4	20.7	13.5	100,000	24,000
Cement	Measured Deflections	51.2	30.9	17.6	8.4		
326+00 - 338+50	Simulated Deflections	52.4	33.3	20.7	13.7	125,000	24,000
		44.0	32.4	20.6	13.8	150,000	24,000
		46.4	31.6	20.5	13.9	175,000	24,000
1200 lb Dynamic Loading							
Ky 11							
AFBC	Measured Deflections	177.2	94.8	44.1	20.7		
First Section	Simulated Deflections	155.0	75.6	41.1	26.2	48,000	24,000
263+00 - 293+00		140.5	73.5	41.3	26.4	60,000	24,000
		124.6	70.9	41.4	26.8	80,000	24,000
AFBC							
Second Section	Measured Deflections	234.7	131.6	49.5	32.1		
532+00 - 540+00	Simulated Deflections	155.0	75.6	41.1	26.2	48,000	24,000
		140.5	73.5	41.3	26.4	60,000	24,000
		124.6	70.9	41.4	26.8	80,000	24,000
Hydrated Lime	Measured Deflections	205.6	65.3	44.9	63.1		
376+00 - 401+00	Simulated Deflections	177.7	78.3	40.9	25.9	36,000	24,000
		155.0	75.6	41.1	26.2	48,000	24,000
Multicone Kiln Dust	Measured Deflections	151.5	78.6	69.3	62.9		
422+00 - 429+00	Simulated Deflections	124.6	70.9	41.4	26.8	80,000	24,000
		114.2	68.8	41.4	27.1	100,000	24,000
Cement	Measured Deflections	117.8	74.0	38.5	20.6		
326+00 - 338+50	Simulated Deflections	104.9	66.6	41.3	27.4	125,000	24,000
		98.0	6.7	41.2	27.6	150,000	24,000
		92.7	63.1	41.1	27.8	175,000	24,000

(FHWA-IP-80-2). The effective changes for the elastic modulus are summarized in the form of modular ratios of after treatment versus before treatment conditions and are presented in Table 23. It may be seen from Table 23 that all stabilization materials and procedures result in an improved condition (Ea/Eb > 1).

The degree of improvement is a function of the proportion of material used for stabilization, the strength gain characteristics of the stabilized soil mixture, and the characteristics of the soil being stabilized. Nothing is known about the long-term characteristics of the by-product material (soil-AFBC, soil-kiln dust). Kentucky does not have a long history of documented performance data for soil and portland cement or soil and hydrated lime. These materials have been used at various times in Kentucky with apparently good results. Results have been promising in other areas as well. The success or failure of soil stabilization lies in the long-term performance of the stabilized layer. Premature failure of the stabilized layer will likely lead to premature failure of the pavement section. Conversely, if stabilization is successful, the pavement life will likely be extended given that all other factors remain constant.

Table 24. (continued) Elastic modulus of layer 2 assumed equal to 18,000 psi

Section		Sensor 1	Deflections (in x 10¬-5) Sensor 2	Sensor 3	Sensor 4	Layer Modulus (psi) Layer 1	Layer 2
600 lb Dynamic Loading							
Ky 11							
AFBC	Measured Deflections	86.7	41.8	17.0	6.8		
First Section	Simulated Deflections	91.1	48.6	27.5	17.7	48,000	18,000
263+00 - 293+00		83.1	47.2	27.6	17.9	60,000	18,000
		74.2	45.4	27.6	18.1	80,000	18,000
AFBC							
Second Section	Measured Deflections	84.9	51.7	36.8	10.1		
532+00 - 540+00	Simulated Deflections	91.1	48.6	27.5	17.7	48,000	18,000
		83.1	47.2	27.6	17.9	60,000	18,000
		74.2	45.4	27.6	18.1	80,000	18,000
Hydrated Lime	Measured Deflections	83.4	25.3	23.0	22.3		
376+00 - 401+00	Simulated Deflections	103.4	50.4	27.4	17.5	36,000	18,000
		91.1	48.6	27.5	17.7	48,000	18,000
Multicone Kiln Dust	Measured Deflections	63.4	30.8	29.5	25.5		
422+00 - 429+00	Simulated Deflections	74.2	45.4	27.6	18.1	80,000	18,000
		68.3	44.0	27.5	18.3	100,000	18,000
Cement	Measured Deflections	51.2	30.9	17.6	8.4		
326+00 - 338+50	Simulated Deflections	62.9	42.4	27.4	18.5	125,000	18,000
		58.9	41.1	27.3	18.6	150,000	18,000
		55.7	40.0	27.1	18.7	175,000	18,000
1200 lb Dynamic Loading							
Ky 11							
AFBC	Measured Deflections	177.2	94.8	44.1	20.7		
First Section	Simulated Deflections	182.2	97.3	55.1	35.6	48,000	18,000
263+00 - 293+00		166.2	94.5	55.2	35.7	60,000	18,000
		148.5	90.9	55.2	36.2	80,000	18,000
AFBC							
Second Section	Measured Deflections	234.7	131.6	49.5	32.1		
532+00 - 540+00	Simulated Deflections	182.2	97.3	55.1	35.6	48,000	18,000
		166.2	94.5	55.2	35.7	60,000	18,000
		148.5	90.9	55.2	36.2	80,000	18,000
Hydrated Lime	Measured Deflections	205.6	65.3	44.9	63.1		
376+00 - 401+00	Simulated Deflections	206.7	100.8	54.8	34.9	36,000	18,000
		182.2	97.3	55.1	35.6	48,000	18,000
Multicone Kiln Dust	Measured Deflections	151.5	78.6	69.3	62.9		
422+00 - 429+00	Simulated Deflections	148.5	90.9	55.2	36.2	80,000	18,000
		136.5	87.9	55.1	36.6	100,000	18,000
Cement	Measured Deflections	117.8	74.0	38.5	20.6		
326+00 - 338+50	Simulated Deflections	125.8	84.9	54.8	36.9	125,000	18,000
		117.8	82.3	54.5	37.2	150,000	18,000
		111.5	80.1	54.1	37.4	175,000	18,000

Table 24. (continued) Elastic modulus of layer 2 assumed equal to 30,000 psi

Section		Deflections (in x 10-5) Sensor 1	Sensor 2	Sensor 3	Sensor 4	Layer Modulus (psi) Layer 1	Layer 2
600 lb Dynamic Loading							
Ky 11							
AFBC	Measured Deflections	86.7	41.8	17.0	6.8		
First Section	Simulated Deflections	68.9	31.6	16.4	10.4	48,000	30,000
263+00 - 293+00		62.0	30.2	16.5	10.5	60,000	30,000
		54.7	29.2	16.5	10.6	80,000	30,000
AFBC							
Second Section	Measured Deflections	84.9	51.7	36.8	10.1		
532+00 - 540+00	Simulated Deflections	68.9	31.6	16.4	10.4	48,000	30,000
		62.0	30.2	16.5	10.5	60,000	30,000
		54.7	29.2	16.5	10.6	80,000	30,000
Hydrated Lime	Measured Deflections	83.4	25.3	23.0	22.3		
376+00 - 401+00	Simulated Deflections	79.8	32.2	16.3	10.3	36,000	30,000
		68.9	31.6	16.4	10.4	48,000	30,000
Multicone Kiln Dust	Measured Deflections	63.4	30.8	29.5	25.5		
422+00 - 429+00	Simulated Deflections	54.7	29.2	16.5	10.6	80,000	30,000
		49.9	28.3	16.6	10.7	100,000	30,000
Cement	Measured Deflections	51.2	30.9	17.6	8.4		
326+00 - 338+50	Simulated Deflections	45.7	27.5	16.6	10.8	125,000	30,000
		42.6	26.8	16.5	10.9	150,000	30,000
		40.2	26.2	16.5	11.0	175,000	30,000
1200 lb Dynamic Loading							
Ky 11							
AFBC	Measured Deflections	177.2	94.8	44.1	20.7		
First Section	Simulated Deflections	137.8	62.1	32.8	20.8	48,000	30,000
263+00 - 293+00		124.0	60.5	32.9	21.0	60,000	30,000
		109.3	58.4	33.0	21.2	80,000	30,000
AFBC							
Second Section	Measured Deflections	234.7	131.6	49.5	32.1		
532+00 - 540+00	Simulated Deflections	137.8	62.1	32.8	20.8	48,000	30,000
		124.0	60.5	32.9	21.0	60,000	30,000
		109.3	58.4	33.0	21.2	80,000	30,000
Hydrated Lime	Measured Deflections	205.6	65.3	44.9	63.1		
376+00 - 401+00	Simulated Deflections	159.5	64.4	32.6	20.5	36,000	30,000
		137.8	62.1	32.8	20.8	48,000	30,000
Multicone Kiln Dust	Measured Deflections	151.5	78.6	69.3	62.9		
422+00 - 429+00	Simulated Deflections	109.3	58.4	33.0	21.2	80,000	30,000
		99.7	56.7	33.1	21.4	100,000	30,000
Cement	Measured Deflections	117.8	74.0	38.5	20.6		
326+00 - 338+50	Simulated Deflections	91.3	55.0	33.1	21.7	125,000	30,000
		85.2	53.6	33.1	21.8	150,000	30,000
		80.4	52.4	33.0	22.0	175,000	30,000

The pavement sections of KY 11 were initially proposed for construction as 8 1/2 inches asphaltic concrete and 17 inches of dense graded aggregate base (DGA). The decision for stabilization was made later and it was determined to use the various materials documented in this paper for soil stabilization. Past experience of design personnel had indicated that the thickness of DGA could be reduced as the thickness of stabilized soil was increased. The thickness design was initially modified to include: 8 1/2 inches asphaltic concrete, 5 inches DGA and 12 inches of stabilized soil subgrades. Preliminary analyses of stabilized soil mixtures indicated the soil-cement on KY 11 appeared somewhat stiffer (stronger) than the other stabilized soil mixtures. It was decided during construction to further modify thicknesses as presented in Table 25 so as to have more equivalent thickness designs from a structural perspective. The long-term performance of these sections will provide data for determination of the potential benefits for soil stabilization and pavement performance.

Table 25. Thicknesses as modified by construction -- KY 11

Station Number Beginning	Ending	Chemical Admixture	Thicknesses Crushed Stone (in)	AC (in)
260+00	317+00	AFBC	5.0	8.5
317+00	348+00	Cement	5.0	6.0
348+00	402+50	Hydrated Lime	5.0	8.5
402+50	429+50	Multicone Kiln Dust	5.0	8.5
429+50	522+00	Cement	5.0	6.0
522+00	532+00	Non-Stabilized	5.0	11.0
532+00	576+60	AFBC	5.0	8.5

Summary and Conclusions

Construction of highway field trials of admixture stabilization of several sections of subgrades were described. Four admixtures used in the experimental sections included hydrated lime, cement (type 1P), multicone kiln dust, and a waste by product obtained from a process referred to as atmospheric fluidized bed combustion(AFBC). A detailed description of a laboratory procedure for determining the optimum percentage of admixture is described. Conclusions concerning the long-term benefits of admixture stabilization of these sections cannot be made at this time. Future testing and observations of the performances of pavements placed on the admixture stabilized subgrades will be required to establish long-term benefits. However, based on field and laboratory studies described herein the following conclusions, comments, and recommendations are offered.

A laboratory procedure was developed for determining the optimum percentage of admixture to add to a given type of soil. When this optimum percentage of admixture is added to a given type of soil, the maximum unconfined compressive strength is obtained. An increase in the percent admixture above the optimum percentage does not significantly increase the unconfined compressive strength.

Index properties of soils at the KY 11 site were improved significantly when mixed with cement (type 1P). Some improvement in the index properties was observed when hydrated lime was mixed with these soils. However, index properties of soils at the Section AA-19 site were improved significantly when mixed with hydrated lime. The soils of the Section AA-19 site contain higher perentages of clay particles than the soils at the KY 11 site. Hydrated lime appears to significantly improve soils that contain high percentages of clay particles. With regard to improvement of index properties of soils at the KY 11 site, the AFBC spent lime produced mixed results. A slight reduction was observed in the percentage of clay particles when this material was mixed with soils from the KY 11 site. However, plasticity index showed little or no change. Index data for the multicone kiln dust were not available.

As the percent hydrated lime and AFBC spent lime increases, the maximum dry density and optimum moisture content obtained from standard compaction (ASTM D698) decrease and increase, respectively. In construction, the volume change that occurs when the natural soils are mixed with these admixtures must be recognized so that proper grade elevations may be established. At the KY 11 site, and because of the change in volume, the finished subgrades that were constructed with hydrated lime and AFBC had to be cut to obtain design grade elevation. Conversely as the percent cement increased, no significant change was observed in the maximum dry density and otpimum moisture content.

Based on laboratory unconfined compression tests and CBR tests, the four admixtures significantly improved the shear strength and bearing strength of soils at the two study sites. Unconfined compressive strengths of field specimens of the four different soil-admixtures were lower than the unconfined compressive strengths of laboratory specimens of the four soil-admixtures. Generally, the field strengths were 40 to 50 percent of the laboratory strengths. This difference in strength is attributed to two factors. First, the relative compaction of the field specimens was slightly less than 100 percent while the laboratory specimens were compacted at or near 100 percent. A slight decrease in relative compaction will yeild lower values of strength. Secondly, the mixing efficiency of laboratory specimens is higher than what can be obtained in the field. Laboratory specimens should be compacted according to the relative compaction used or specified in the field. If a relative compaction of 95 percent is specified, then this value should be used in the laboratory to mold specimens. Generally, based on field density tests, the subgrades of KY 11 were compacted according to specifications. Field density tests were not available for Section AA-19.

Based on pavement elevations, no significant swell occurred in the subgrade sections stabilized with hydrated lime, cement, and multicone kiln dust. Laboratory swell tests also showed that there were no swelling associated with hydrated lime and cement. These two admixtures actually reduced the swelling the swelling of the natural soils. However, the AFBC spent lime -soil subgrade swelled significantly. Significant swell or heave, of the pavements placed on the two AFBC sections occurred shortly after construction. The swelling nature of this material when mixed with soils was not expected since a small qantity was mixed in the subgrades at the KY 11 site. Initial swell tests of soils from the Ky 11 site that were mixed with 7 percent of this admixture showed relatively small values of swell (about 3 percent). The untreated soils from the KY 11 site swelled some 3 to 4 percent. However, swell tests performed on specimens mixed with 15 and 30 percent of this admixture had swell values in excess of 20 percent. The humps that formed in the pavement at the KY 11 site may have been caused when the spreader trucks were stopping and starting, that is, excess AFBC material accumulated at the humped pavement areas when the truck was stopped. More research is needed to identify the mechanism that causes the swelling of AFBC spent lime - soil mixtures.

High values of CBR were obtained in each admixture stabilized subgrade shortly after construction and one year after construction. Generally, CBR values of the treated subgrades are in a range of 20 to 50 percent. Elastic moduli, as estimated from nondestructive deflection test, indicates substantial improvement when fine-grained soils are stabilized with admixtures addressed in this paper. Overall, Type 1P cement and hydrated lime gave the best results. These admixtures improved the stiffness of the soil six fold.

The ultimate success or failure of any admixture used for stabilization of subgrade soils rest with the long-term performance for each specific admixture. This can only be achieved by continued monitoring of performance with time. Additional research is needed in this area. Research also must continue in the area of quantifying the benifits of soil stabilization on improving pavement life and performance. This information will be useful for the development of improved pavement design methods incorporating stabilized subgrades as an integral structural component of the pavement layered system.

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Highway application of geosynthetics

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Abstract: The use of geosynthetics in highway engineering has increased by several hundred percent over the past few years. This increase in use results from a track record of many successful projects, cost savings, and the ability to place materials rapidly with relatively unskilled labor. The primary uses of geosynthetics in highway applications are drainage, separation, reinforcement, rehabilitation and erosion control.

1. INTRODUCTION

The use of geosynthetics for highway construction has greatly increased since they were first introduced in civil engineering 20 years ago. It is now commonplace to install geotextiles over soft subgrades, geogrids to steepen slopes and build walls, geocomposites for drainage, edgedrains along embankments, and geotextiles for pavement overlays. The reasons for the large increase in usage are simple: (i) geosynthetics are easy to install; (ii) geosynthetics save money; and (iii) geosynthetics improve the performance and extend the life of structures.

The design of any highway or related structure is a complex process which requires expertise in many technical disciplines. Each component of the structure is designed in detail. However, when geosynthetics are used in design, the same level of design is not always employed. The primary reason for this disparity is a general lack of familiarity with geosynthetics by engineers. This lack of familiarity results from the evolving status of geosynthetic design methods and the fact that many of these design methods have only been around for a short period of time.

The purpose of the paper is to review the primary uses of geosynthetics in highway engineering and to reinforce the importance of design of geosynthetics. The primary geosynthetic properties and common test methods used for design are discussed in Section 2. Drainage applications are discussed in Section 3. Separate applications are discussed in Section 4. Reinforcement applications are discussed in Section 5 and asphalt overlays are discussed in Section 6.

2. GEOSYNTHETIC PROPERTIES

2.1 Introduction

In order to design with geosynthetics, the geosynthetic properties must be known. The purpose of Section 2 is to present an overview of geosynthetic properties. The categories of tests used to measure geosynthetic properties are presented in Section 2.2. The material properties routinely measured and a reference to test procedures are presented in Section 2.3.

2.2 Test Categories

There are three primary categories of tests used to measure the properties of geosynthetics: (i) control tests; (ii) index tests; and (iii) performance tests. These tests are described below.

2.2.1 Control Tests

Control tests are typically performed during production to evaluate product integrity, quality an continuity, and to assess the impact of changes in production methodology or product properties. Typical control tests include thickness, density, fiber strength, fiber tenacity, polymer specific gravity and geotextile mass per unit area.

2.2.2 Index Tests

Index tests are standard tests which may be used to compare the relative, qualitative performance of several different geosynthetics under similar boundary conditions. These boundary conditions may be somewhat similar to field conditions, but do not incorporate the interaction of the geosynthetic composites with materials from the field such as soils nor account for variations in environmental conditions. Typical index tests include core compressive strength, tensile strength and compression creep, geotextile wide width tensile strength, tension creep, flexural rigidity, permittivity, puncture resistance, and composite transmissivity (between plates). Index test results can be given as typical values or minimum average roll values. Care should be taken to ensure comparisons are always made with similar values. Whenever possible, minimum average roll values should be used.

2.2.3 Performance Tests

Performance tests are used to evaluate the design properties of geosynthetics. These tests are typically performed under conditions which simulate field conditions. Typical performance tests include hydraulic conductivity ratio and compression creep in soil. In some cases index tests may be substituted for design tests. However, extreme caution should be exercised when using index properties for design because the standard laboratory boundary conditions of the index test may differ considerably from field conditions simulated in the performance tests. Only experienced engineers who understand how field conditions may affect the performance of the geosynthetic should attempt to design with index properties. In these cases, relatively high factors of safety should be used in the design process.

2.3 Material Properties

The geosynthetic material properties typically used in the design can be divided into four primary areas: (i) physical properties; (ii) mechanical properties; (iii) hydraulic properties; and (iv) durability properties. These properties are discussed below.

2.3.1 Physical Properties

Physical properties are those which identify a product by weight, geometry, material type, method of production or appearance. The primary physical properties used to identify synthetic drainage composites are:

- Physical Description (product name and number, type, manufacturing process, geometry, polymer types, geotextile types, color, weight, etc.);
- Polymer Specific Gravity (ASTM D-885-79);
- Thickness;
- Mass per Unit Area (ASTM D-3776);
- Porosity; and
- Apparent Opening Size, AOS (ASTM Draft).

2.3.2 Mechanical Properties

Mechanical properties describe the strength and deformation behavior of the composite for prescribed boundary conditions and states of stress. These properties typically differ in the main direction (MD) and cross direction (CD). The primary mechanical properties used in the design of synthetic drainage composites are:

- Wide width tensile strength of geotextile (ASTM D4595);
- Trapezoidal tear strength of geotextile (ASTM D-1117);
- Mullen burst strength of geotextile (ASTM D-751);
- Puncture strength of geotextile (ASTM Draft);
- Compressive strength (ASTM Draft);
- Compressive Creep (GeoServices Method); and
- Direct shear (ASTM Draft).

2.3.3 Hydraulic Properties

Hydraulic properties are used to measure the quantity of flow of a permeant, usually water, through a geotextile for various boundary conditions. Since these boundary conditions have a major impact on the flow capacity of the drainage composites it is very important to measure the hydraulic properties under conditions which simulate field conditions. The primary hydraulic properties which are used to design are:

- Hydraulic Transmissivity (ASTM Draft);
- Geotextile Permittivity (ASTM D4491);
- Geotextile Permittivity under Load (ASTM Draft);
- Gradient Ratio (ASTM Draft); and
- Hydraulic Conductivity Ratio (GeoServices Method).

Hydraulic Transmissivity

The hydraulic transmissivity is a measure of the conductivity under planer flow conditions [Williams, et al., 1984]. The hydraulic transmissivity, θ, is the product of the hydraulic conductivity and saturated thickness of the geosynthetic and may be evaluated by a modified form of the Darcy equation:

$$\theta = \frac{q}{iB} \quad \text{(Equation 1)}$$

where: q = flowrate (L^3/T); i = hydraulic gradient (dimensionless); and B = width of flow rate (L).

The hydraulic transmissivity of a synthetic drainage products is primarily a function of the boundary conditions, hydraulic gradient, compressive stress and duration of loading. In order to measure a value of hydraulic transmissivity which is indicative of field performance, the field boundary conditions should be simulated as closely as possible. For example, if the drainage layer is to be installed in a trench with soil on both sides, the transmissivity of the synthetic drainage composite should be measured under similar conditions in the laboratory; that is, with soil on both sides. A typical graph showing the relationship between hydraulic transmissivity, compressive stress and hydraulic gradient is given in Figure 2-1.

<u>Geotextile Permittivity</u>

The permittivity of a geotextile is a measure of flow perpendicular to the plane of the geotextile. The permittivity is the ratio of the hydraulic conductivity and the saturated thickness of the geotextile as defined below:

$$P = \frac{q}{HA} \quad \text{(Equation 2)}$$

where: P = permittivity (i/T); q = flow rate (L^3/T); H = hydraulic head above the geotextile (L); and A = cross sectional area of the flow path (L^2). The permittivity parameter is typically of little value in design because it does not account for the interaction between the geotextile and the adjacent soil. This interaction is discussed in Section 3.

2.3.4 <u>Durability Properties</u>

The durability properties describe the long-term response of the geosynthetic to chemical or environmental exposure. The primary durability properties used in the screening and selection of a geosynthetic are defined and described below.

<u>Chemical Compatibility</u>

The chemical compatibility is defined as the mechanical response of the geosynthetics when exposed to a chemical. Mechanical response usually refers to the grab or wide strip tensile strength and modulus, and shape, size and color of the geosynthetic. At the present time, there are no standard test methods for the evaluation of the chemical compatibility of geosynthetics. In some cases the EPA 9090 test method has been used a guide for length of time and temperature of exposure to leachates.

<u>UV Resistance</u>

The UV resistance is the mechanical response of the geosynthetics after exposure to ultraviolet light from the sun. The ultraviolet resistance is evaluated by measuring the change in tensile strength as a function of the duration of exposure to ultraviolet light. ASTM 4355 provides a standard testing procedure for measuring the effects of ultraviolet rays on geosynthetics.

3. DRAINAGE SYSTEMS

3.1 <u>Introduction</u>

Synthetic drainage composites have been used in numerous applications for highway construction. Highway edge drains are used to promote drainage from the embankment and prevent water from ponding at the interface between the embankment and the soil subgrade. In areas where highways pass over extremely soft subgrades, thick nonwoven geotextiles may be used in conjunction with strip drains and granular materials to promote vertical and lateral drainage. Geonets and sheet drainage products such as vacuum formed materials or cuspated core materials are often used behind walls or at the base of planter boxes.

Due to the wide variety of applications and requirements, there is no single type or family of synthetic drainage composites which performs better for all drainage applications. Each type of drainage product may have particular properties which are better suited for certain applications, but may not be suitable for other applications. For example, Continuous Injection Molding (CIM) materials are ideally suited for highway edge drains because of their high flow capacity at low gradients, resistance to deformation and structural stability. However, CIM drainage composites are manufactured in relatively narrow widths and require over-wrapping for stability. Therefore, the CIM drainage composites would not typically be appropriate for drainage layers where large areal coverage is required such as behind walls or at the base of a large planter.

Design of any drainage system, whether using conventional or synthetic drainage materials, is a complex process which must include an evaluation of the interaction between the drain and its environment. The drains are typically placed adjacent to soils. Therefore, the drains should be designed to carry the flow of ground water from the soil. In addition, the geotextiles used to separate the drains from the soil must be compatible with the adjacent soil materials. As water migrates through the soil into the drainage composite the geotextile should promote drainage while retaining the soil particles. The drains should also have adequate strength to resist the long-term compressive stresses imposed through the design life of the drain and they should be resistant to chemical degradation by chemicals which are commonly encountered in the soil or ground water. The drains should also be durable, relatively easy to install and cost-effective.

The design process for synthetic drainage composites is very similar to the design process for conventional granular drainage materials. However, the material properties used for the design of synthetic drainage composites are quite different from the material properties used to design granular drains. The interaction between synthetic drainage composites and soil is discussed in Section 3.2. A general design process for drainage systems is presented in Section 3.3.

3.2 Geotextile/Soil Interaction

In most synthetic drainage composites, geotextiles are used to separate a highly permeable control core of the drain from adjacent soils. As flow is initiated from the soil, through the geotextile and into the central core, a non-equilibrium condition is created. This is because a large increase in velocity occurs as water leaves the soil and travels through the geotextile. In most cases, the increase in velocity is high enough to reorient or dislodge soil particles at or near the interface between the soil and the geotextile. The degree of reorientation or transport of soil particles at the interface depends on a large number of variables. These variables can be divided into three areas: (i) soil variables; (ii) geotextile properties; and (iii) boundary conditions. Depending on these variables, three types of particle reorientations may occur.

- *Piping,* Small particles may be transported through the geotextile and into the drainage media;
- *Clogging.* Small to intermediate sizes soil particles may lodge in the pores of the fabric or my be electrostatically attracted to the fibers of the geotextile. The reduction in the pore volume of the geotextile caused by accumulation of soil particles is termed "clogging" and results in reduced flow capacity of the drain; and
- *Blinding.* A transitional filter may develop in the soil resulting from successive filtration of fine grained soils. This process is illustrated in Figure 3-1. The transitional filter which forms in the soil adjacent to the geotextile is called a filter cake [Lawson, 1982]. The formation of a filter cake at the soil/geotextile interface inevitably results in a reduced flow capacity across the interface which is called "blinding".

The proper design of a geotextile for drainage and filtration entails selection of a geotextile which prevents piping of fines into the core of the drain and clogging and induces the formation of a stable filter cake in the soil.

3.2.1 Evaluation of Geotextile Suitability

The methods used to design or evaluate the suitability of geotextiles used for filtration or separation in soils may be divided into three areas according to the fundamental properties used for design. These basic methods may be described as follows:

1. Methods based on the opening size of the geotextile;

2. Methods based on the permittivity of the geotextile; and

3. Methods based on the flowrate across the soil/geotextile interface.

3.2.2 Methods Based on Opening Size

A retention criterion based on the opening size of geotextiles has been presented in the Federal Highway Administration Geotextile Engineering Manual [Christopher and Holtz, 1984]. This criterion is given below:

The required apparent opening size (O_{95}) of the geotextile separator, if less than 50% of the soil particles by weight are finer than U.S. sieve No. 200 (0.076 mm), is:

$$O_{95} \leq B\ D_{85}$$

where: $B = 1.0$, if $C_u \leq 2$ or ≥ 8

$B = 0.5\ C_u$, if $2 \leq Cu \leq 4$; and

$B = 8/\ C_u$, if $4 \leq Cu \leq 8$.

If more than 50% of the soil particles by weight are finer than U.S. sieve No. 200 (0.076 mm), the required apparent opening size of the geotextile separator is:

$O_{95} \leq D_{85}$, for woven fabrics

$O_{95} \leq 1.8\ D_{85}$, for nonwoven fabrics

For the low-permeability soil, with 50% or more particles by weight passing the U.S. sieve No. 200 (0.074 mm) and $D_{85} = 0.15$ mm, the required apparent opening size of the geotextile separator underlying the soil is given below (the values were rounded down to the closest standard sieve size):

$O_{95} \leq$ U.S. sieve No. 100 (0.15 mm), for woven fabrics

$O_{95} \leq$ U.S. sieve No. 60 (0.27 mm), for nonwoven fabrics

Methods based on opening size provide a relatively good indication of the anticipated level of performance of a woven geotextile, but provide a relatively poor indication of the anticipated level of performance for nonwoven geotextiles. In addition, the methods based on opening size only provide some level of assurance that piping will not occur. These methods do not address clogging, nor is it possible to determine to what extent blinding will occur.

3.2.3 Methods Based on Permittivity

The methods which are based on permittivity or permittivity under load of the geotextile do not adequately address the interaction between the geotextile and the soil. Therefore, design methods based on geotextile permittivity are not recommended.

3.2.4 Methods Based on Flow Across Interface

Three primary methods have been developed which can be used to evaluate soil/geotextile interaction by monitoring flow across the interface: (i) Gradient Ratio; (ii) Long Term Column; and (iii) Hydraulic Conductivity Ratio. These tests are discussed in further detail below.

Gradient Ratio Test

Calhoun [1972] investigated piping and clogging potential by measuring head loss at various points through a laboratory-simulated soil/geotextile system. A clogging ratio based on the hydraulic gradients measured across the fabric and across the system was used as a basis to compare performance.

Soils used by Calhoun consisted of uniform Ottawa 20/30 sand (ASTM C-190) mixed with a low plasticity Vicksburg, Mississippi silt loess. Such soil mixtures (gap-graded mixtures) have maximum potential for internal soil piping and migration of fines and allow evaluation of fabric performance under worst case conditions.

Following the work of Calhoun, the Corps of Engineers established a direct measure of geotextile clogging potential, called the Gradient Ratio (GR) Test. Where GR is defined as the hydraulic gradient through the lower 25 mm (1.0 in.) of the soil plus geotextile divided by the hydraulic gradient through the adjacent 50 mm (2.0 in.) of the soil. Gradient ratio values exceeding 3.0 were believed to signify excessive geotextile clogging. Thus, a limiting value of 3.0 was established for testing with soil, geotextile, and hydraulic conditions of interest [Haliburton and Wood, 1982]. A Calhoun type apparatus is shown schematically in Figure 3-2.

Haliburton and Wood [1982] investigated clogging resistance of woven and nonwoven geotextiles. Results obtained showed that clogging potential increased for all fabrics as the silt content increased in the protected soil. Dramatic performance differences were observed between the fabrics tested. Typical results are shown in Figure 3-3.

The gradient ratio test is relatively inexpensive and can be performed by personnel with limited experience. However, the gradient ratio test has several very important limitations which have a major impact on the accuracy and validity of the test results:

- No control over the soil stress state;
- Samples are not fully saturated because of the absence of backpressure saturation;
- The test is not suitable for bidirectional flow conditions;
- Cohesive and plastic soils may experience shrinkage, swell, or local consolidation. This may result in consolidation of the soil or gaps between the soil sample and the wall of the cylinder, creating paths of minimum resistance through which bulk flow may occur. It is also difficult to achieve intimate contact between a cohesive soil and the surface of the geotextile;
- Soil placement does not model field conditions;
- There is no control over void ratio during the analysis.

The above mentioned limitations lead to the conclusion that since site conditions cannot be simulated properly using the gradient ratio device, the results of the analyses may not predict the behavior in the field.

Long-Term Column Tests

Long-term column tests such as those conducted by Koerner and Ko [1982] have the same basic limitations as the gradient ratio test. These tests do not adequately simulate field conditions and can give results which are inaccurate, misleading and unsafe.

Hydraulic Conductivity Ratio Analyses

The hydraulic conductivity ratio (HCR) analysis is a soil/geotextile interaction test which is performed under conditions which simulate the field conditions. The soil sample is prepared using standard laboratory of field sampling techniques which are designed to simulate field placement conditions. The state of stress, stress history and void ratio of the soil and geotextile are carefully controlled throughout the test. Since the soil sample may be fully saturated and the triaxial permeability device used in the HCR analysis provides control of the flow direction and hydraulic gradient, all of the primary variables listed in Table 3-1 which affect the filtration characteristics and flow properties of the geotextile/soil composites are either controlled or measured during the HCR analysis.

The hydraulic conductivity ratio is defined as the hydraulic conductivity of the soil, K_S, divided by the equilibrium hydraulic conductivity of a soil/geotextile composite, K_{GS}. Since the hydraulic conductivity of both the soil and the soil/geotextile composite are measured directly, the HCR analysis provides data which are appropriate for the design. A typical plot of hydraulic conductivity versus flow volume is given in Figure 3-4.

In cases where the geotextile will be placed against a clean granular backfill, the design methods based on apparent opening size give good results. However, when geotextiles are placed against soils with silt and clay-size particles, the HCR test is recommended.

3.4 Design Methodology

The design methodology for synthetic drainage composites is similar to the design methodology for conventional drainage systems. The primary components of the design process are outlined and discussed below.

- *Site Investigation.* The site investigation is a critical component of any design process. In the design of drainage systems it is imperative to have an understanding of the surrounding geology and hydrogeology. Borings should be advanced and samples obtained to evaluate the lithology and properties of site soils. Several of the borings should be completed as piezometers to evaluate the directions and rates of ground-water flow. Field testing to evaluate in-situ hydraulic conductivity may also be appropriate.

- *Laboratory Investigation.* A laboratory investigation should be conducted to evaluate the properties of the soils in contact with the drainage materials.

- *Concept Design.* A conceptual design should be performed so that performance and design criteria can be selected for the drainage system. A typical conceptual design would include the following:

    - calculate volume of flow and flowrate from soil for steady-state conditions,
    - calculate maximum flow from soil,
    - evaluate the range in hydraulic gradients,
    - establish outfall locations,
    - calculate required drain capacity, and
    - calculate required capacity of collector or trunk lines.

- *Select Candidate Materials.* Candidate synthetic drainage composites should be screened based on the performance and design criteria and index properties provided by the manufacturer. Two or three primary candidates should be selected for further evaluation.

- *Laboratory Testing of Geosynthetics.* The transmissivities and hydraulic conductivity ratios of the candidate materials should be measured for the same range of stresses and gradients anticipated in the field.

- *Final Design.* The final design includes detailed calculations regarding the anticipated performance of the drainage system for the conditions. These calculations vary depending on the type of drain. In general, final design includes the following:
    - calculate drawdown using HCR. Determine if geotextile provides adequate filtration,
    - calculate flow capacity of drain,
    - calculate flow through geotextile into drain,
    - calculate factor of safety for peak flow through the drain,
    - prepare plans and specifications.

4. SEPARATION

4.1 Overview

Traffic loads are distributed so that the stress on the subgrade soil is less than the subgrade soil bearing capacity. In pavement systems, load distribution is achieved by placing a layer of material ("base layer") between the load and subgrade soil. To achieve its load distribution function, the base layer should have: (i) adequate mechanical properties; and (ii) sufficient thickness.

To maintain its load distribution function under repeated loading, the base layer must remain intact, requiring that: (i) deformations of the subgrade soil be limited to prevent lateral or vertical displacements at the bottom of the base layer; and (ii) the mechanical properties of the base layer be such that it is not sheared or degraded by traffic loads.

Deterioration of pavement systems results from deterioration of the subgrade and deterioration of the base layer. Progressive deterioration of the subgrade results from the decrease of shear strength of the subgrade soil due to fatigue generated by repeated loading. Progressive deterioration of the base layer occurs through one or a combination of the following mechanisms affecting its thickness and/or mechanical properties:

- contamination of base layer by fine particles moving upward from subgrade; and

- sinking of base layer aggregate into subgrade soil.

These two mechanisms are shown in Figure 4-1a and 4-1b. Regardless of the mechanism, it can be observed that the effective base layer thickness is reduced. Both of these mechanisms can be prevented by using a geotextile to separate the base layer from the subgrade as shown in Figure 4-1c. The procedure for designing a geotextile separator is outlined below.

4.2 Design

4.2.1 Design Steps

The following steps are recommended to design a geotextile separator [Christopher and Holtz, 1984]:

- Establish the need for a separator.

- Establish the required geotextile filtration properties.

- Establish the required geotextile survivability properties.

- Design the pavement system using conventional techniques which do not account for geotextiles, or intrusion of fines into the base layer.

- Ensure that the base layer is thick enough to support construction equipment.

As described above, filtration characteristics and survivability properties of a geotextile must be addressed. Both of these design issues are addressed below along with secondary design issues.

4.2.2 Filtration

If the geotextile is not designed as a filter, then fines may pass into the base layer (piping) reducing the effective thickness of the base layer, or fines may block the geotextile openings (clogging) causing water to accumulate at the interface. The accumulation of water at the interface can soften the subgrade to a strength below that used in the design. Filter design was addressed previously in the discussion on drainage.

4.2.3 Survivability

In general, the geotextile will be subject to the harshest conditions during construction. Therefore the geotextile mechanical properties must be selected to ensure the geotextile *survives* construction.

Mechanical properties required for survivability will depend upon: (i) subgrade conditions; (ii) base layer material type; and (iii) construction equipment. An empirical approach accounting for these three factors [Christopher and Holtz, 1984] is presented in Tables 4-1 and 4-2. Table 4-1 relates subgrade conditions to construction equipment. Table 4-2 relates cover (base layer) material to construction equipment. Both tables express the required degree of survivability as low, moderate, high, or very high. The more severe degree of survivability obtained from Tables 4-1 and 4-2 should be used to establish mechanical properties. Mechanical properties corresponding to the various degrees of survivability defined in Tables 4-1 and 4-2 can be obtained from Table 4-3. It should be noted that the values in Table 4-3 are interim specifications put forth by the AASHTO-AGC-ARTBA Joint Committee. These values are empirical and are currently under review. The values reported in Table 4-3 are minimum average roll values, not typical values commonly presented in manufacturer's literature. Minimum average roll values can be obtained by contacting the manufacturer.

4.2.4 Other Design Factors

Two other factors to be considered in design are workability and water absorption. On extremely soft subgrades the geotextile will serve as an initial work surface. If the geotextile lacks adequate stiffness it may not support the weight of laborers unrolling the geotextile, which can lead to construction difficulties. If the subgrade conditions are extremely wet then a hydrophyllic geotextile can absorb water, making it heavy. In these cases a hydrophobic geotextile is preferred. The absorption properties of geotextiles can be evaluated by soaking them for one hour and measuring their weight change.

5. REINFORCEMENT

In using a geotextile as a separator, any load-carrying capacity of the geotextile is ignored. If a stiff geosynthetic (i.e., a high-strength geotextile or geogrid) is used then it is possible to account for the reinforcing effect of the geosynthetic. In order to develop adequate tension to reinforce the pavement system, rutting on the order of several inches is required. This magnitude of rutting is unacceptable for paved systems, however, it is acceptable for unpaved systems. Therefore, reinforcement application of geosynthetics are appropriate only for unpaved systems, which is beyond the scope of this paper. The reader is referred to the work of Stewart et al. [1977] for designing temporary reinforced unpaved roads and Giroud and Noiray [1981] for design of permanent, reinforced unpaved roads.

6. REHABILITATION

Open cracks or joints in pavement systems provide a path for moisture to enter the supporting subgrade. Moisture entering the soil subgrade will cause it to soften. Softening of the subgrade will result in a decrease in strength and modulus. Once this decrease has occurred, subsequent vehicle loading on the surface pavement will generate larger deflections of the pavement. As the deflection of the pavement system increases, the number of cracks will increase and existing cracks will grow larger. These new and larger cracks will provide additional paths by which moisture can enter the subgrade. This additional water will accelerate the process described above. A moisture barrier in surface pavement applications can be achieved through asphalt alone. However, asphalt is not a continuous media and is susceptible to cracking. The planar nature of geosynthetics helps form a continuous hydraulic barrier.

The use of asphalt overlays should be used with caution. If cracking of the pavement surface has already occurred, then it is unlikely that the geosynthetic will have any benefit because subgrade degradation has already started. Also, the geosynthetic may create an interface with low shear strength within the pavement system. This may lead to pavement breakup in areas where high shear stresses are applied (e.g., curves and areas of braking).

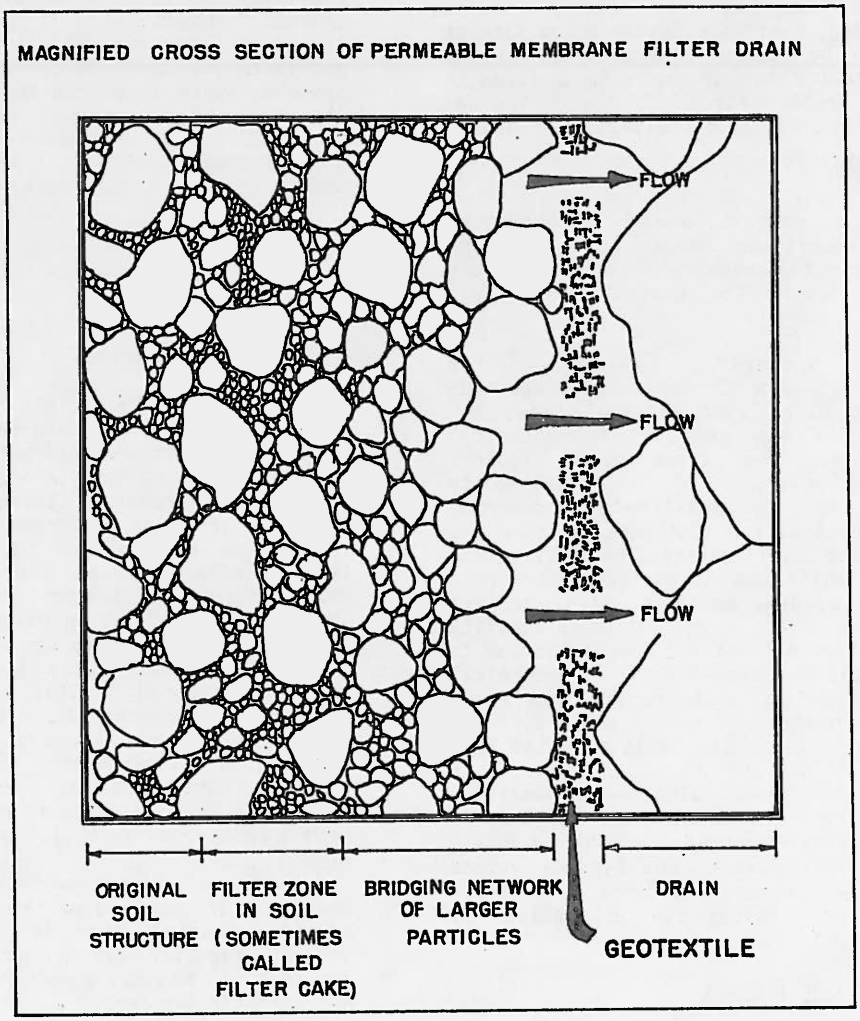


Figure 3-1. Formation of a filter cake at a geotextile interface.

Table 4-1.

REQUIRED DEGREE OF FABRIC SURVIVABILITY AS A FUNCTION OF SUBGRADE CONDITIONS AND CONSTRUCTION EQUIPMENT
Christopher and Holtz [1984].

SUBGRADE CONDITIONS	CONSTRUCTION EQUIPMENT AND 6-12 in. COVER MATERIAL INITIAL LIFT THICKNESS		
	Low Ground Pressure Equipment ($\leq$ 4 psi)	Medium Ground Pressure Equipment (> 4 psi, $\leq$ 8 psi)	High Ground Pressure Equipment (> 8 psi)
Subgrade has been cleared of all obstacles except grass, weeds, leaves, and fine wood debris. Surface is smooth and level such that any shallow depressions and humps do not exceed 6 in. in depth and height. All larger depressions are filled. Alternatively, a smooth working table may be placed.	LOW	MODERATE	HIGH
Subgrade has been cleared of obstacles larger than small- to moderate-sized tree limbs and rocks. Tree trunks and stumps should be removed or covered with a partial working table. Depressions and humps should not exceed 18 in. in depth and height. Larger depressions should be filled.	MODERATE	HIGH	VERY HIGH
Minimal site preparation is required. Trees may be felled, delimbed, and left in place. Stumps should be cut to project not more than ± 6 in. above subgrade. Fabric may be draped directly over the tree trunks, stumps, large depressions and humps, holes, stream channels, and large boulders. Items should be removed only if placing the fabric and cover material over them will distort the finished road surface.	HIGH	VERY HIGH	NOT RECOMMENDED

NOTE:

1. Recommendations are for 6-12 in. initial lift thickness. For other initial lift thicknesses:
   12 - 18 in.: Reduce survivability requirement 1 level
   18 - 24 in.: Reduce survivability requirement 2 levels
   > 24 in.: Reduce survivability requirement 3 levels
   Survivability levels are, in increasing order: low, moderate, high, and very high.

2. For special construction techniques such as pre-rutting, increase fabric survivability requirement 1 level.

3. Placement of excessive initial cover material thickness may cause bearing failure of soft subgrades.

Table 4-2

REQUIRED DEGREE OF FABRIC SURVIVABILITY AS A FUNCTION OF COVER MATERIAL AND CONSTRUCTION EQUIPMENT
Christopher and Holtz [1984]

	6-12 in. Initial Lift Thickness		12-18 in. Initial Lift Thickness		18-24 in. Initial Lift Thickness	> 24 in. Initial Lift Thickness
	Low Ground Pressure Equipment ($\leq$ 4 psi)	Medium Ground Pressure Equipment (> 4 psi, $\leq$ 8 psi)	Medium Ground Pressure Equipment (> 4 psi, $\leq$ 8 psi)	High Ground Pressure Equipment (> 8 psi)	High Ground Pressure Equipment (> 8 psi)	High Ground Pressure Equipment (> 8 psi)
Fine sand to ± 2 in. diameter gravel, rounded to subangular	Low	Moderate	Low	Moderate	Low	Low
Coarse aggregate with diameter up to one-half proposed lift thickness, may be angular	Moderate	High	Moderate	High	Moderate	Low
Some to most aggregate with diameter greater than one-half proposed lift thickness, angular and sharp-edged, few fines	High	Very High	High	Very High	High	Moderate

NOTE:

1. For special construction techniques such as pre-rutting, increase fabric survivability requirement 1 level.

2. Placement of excessive initial cover material thickness may cause bearing failure of soft subgrades.

Table 4-3

ASHTO-AGC-ARTBA JOINT COMMITTEE
(Interim Specifications)
MINIMUM[1] FABRIC PROPERTIES REQUIRED FOR FABRIC SURVIVABILITY
Christopher and Holtz [1984]

Required Degree of Fabric Survivability	Grab Strength (minimum values) (lbs)	Puncture Strength[2] (lbs)	Burst Strength[3] (psi)	Trap Tear[4] (lbs)
Very High	270	110	430	75
High	180	75	290	50
Moderate	130	40	210	40
Low	90	30	145	30

([1]) All values represent minimum average roll values (i.e., any roll in a lot should meet or exceed the minimum values in this table). Note: These values are normally 20% lower than manufacturers reported typical values.

([2]) ASTM D 751-68, Tension Testing Machine with ring clamp, steel ball replaced with a 5/16 inch diameter solid steel cylinder with flat tip centered within the ring clamp.

([3]) ASTM D 751-68, Diaphragm Test Method.

([4]) ASTM D1117, either principal direction

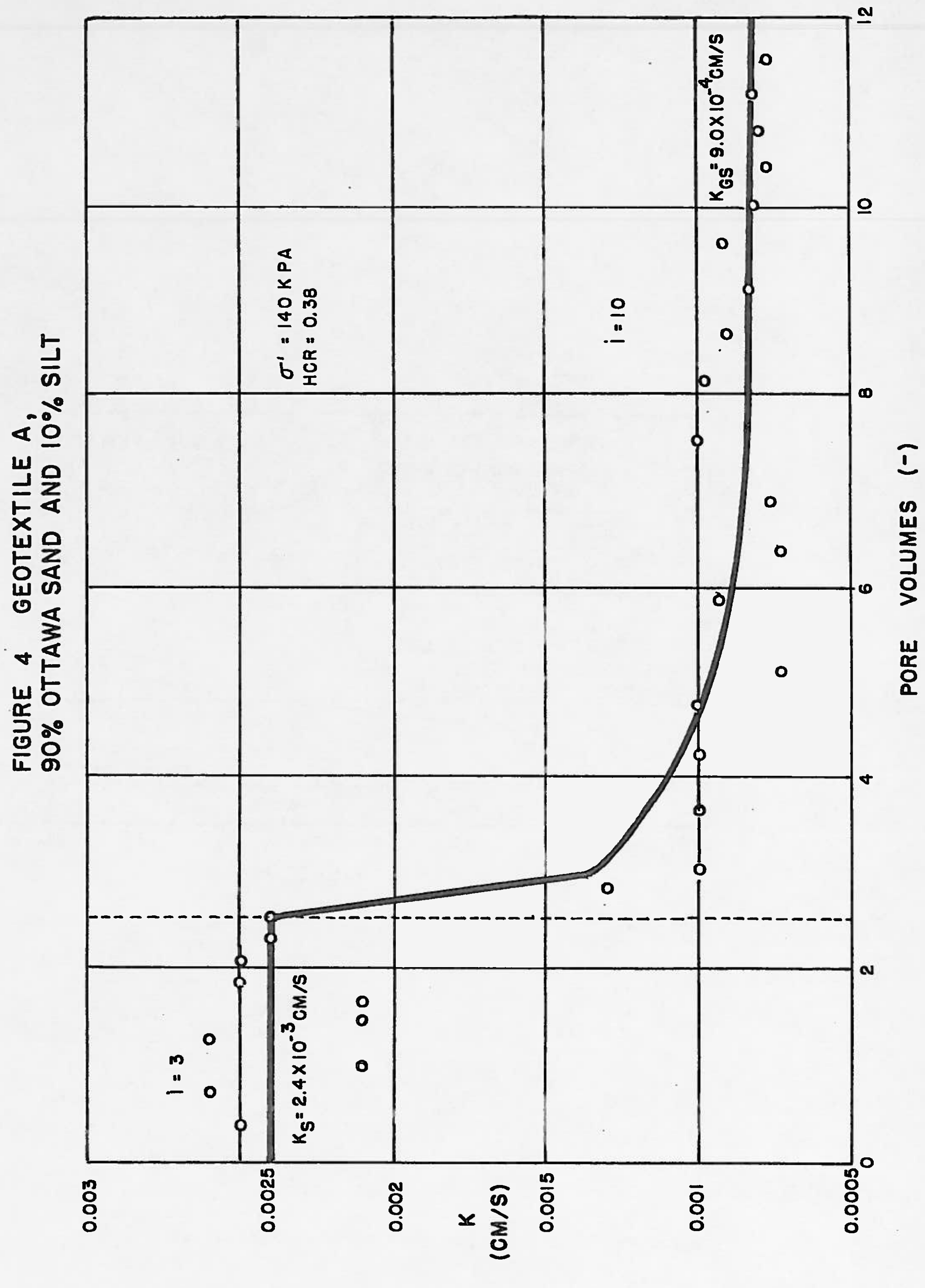


Figure 3-4. A typical plot of hydraulic conductivity versus flow content is measured in the HCR test.

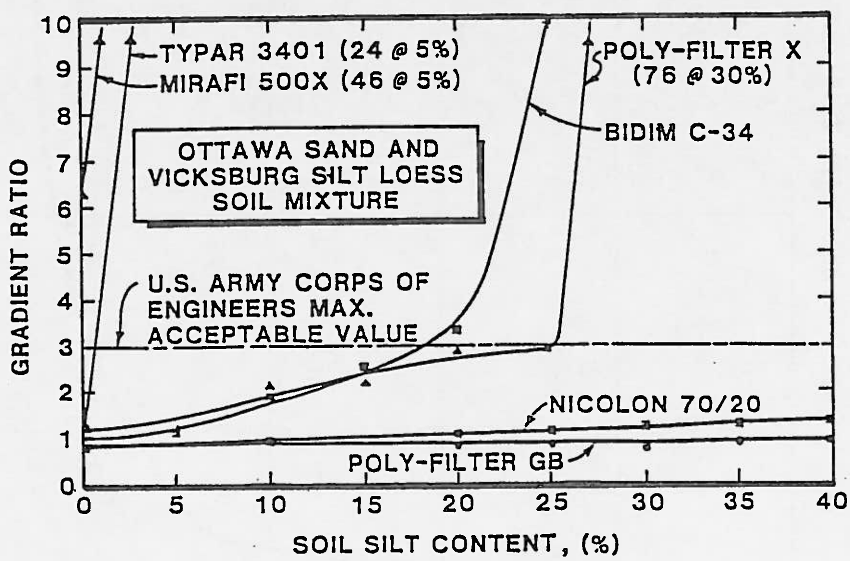


Figure 3-3. Variation in clogging potential with silt content.

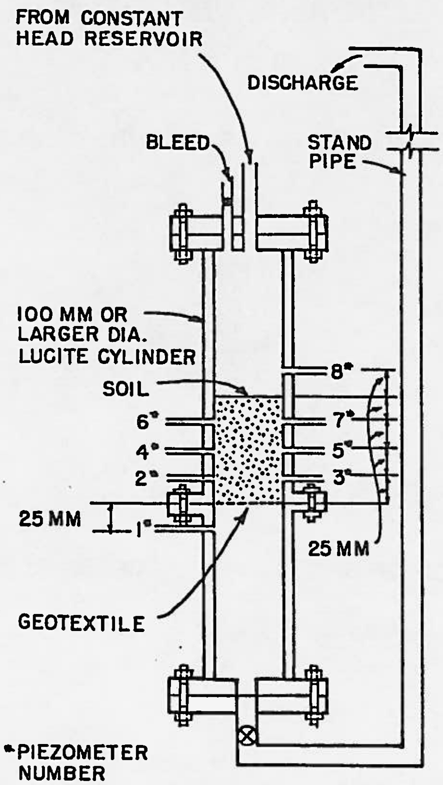
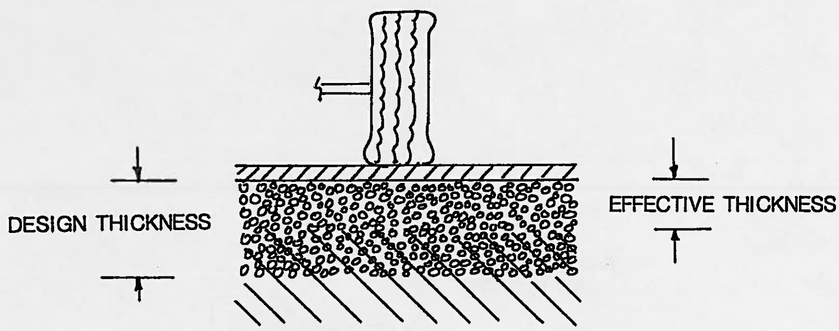
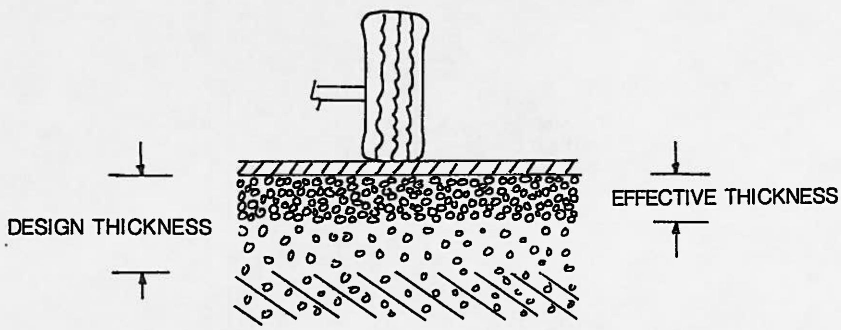


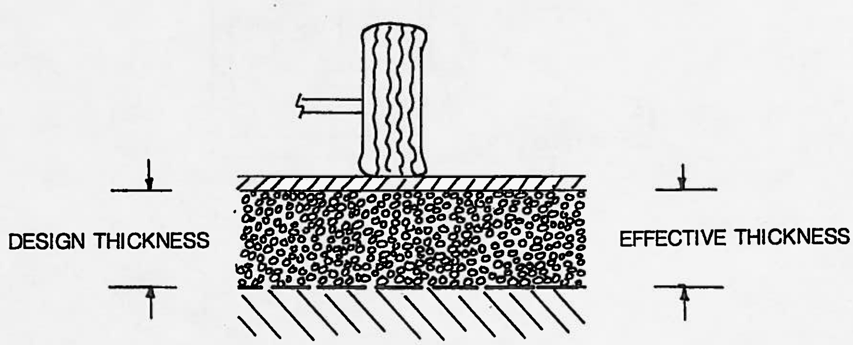
Figure 3-2. Schematic of Calhoun-type Gradient Ratio Device.



a) INTRUSION OF SUBGRADE INTO BASE LAYER



b) MIGRATION OF BASE LAYER AGGREGATE INTO SUBGRADE



c) PROPERLY DESIGNED GEOTEXTILE

Figure 4-1. Effective thickness of base layer.

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Improvement of subgrade support with blasted rock

by

John W. Storm PE
Project Engineer
Law Engineering, Inc.

D.J. Hagerty
Professor, Civil Engineering Dept.
University of Louisville

NOTE: The following two papers are case histories of rock-soil construction at the Toyota automobile plant near Georgetown, Kentucky. The first paper presents the construction of the test track at this facility. The second paper presents information from the initial assembly plant construction and has been reprinted with the permission of the ASCE (American Society of Civil Engineers) Construction Journal.

IMPROVEMENT OF SUBGRADE USING BLASTED ROCK

by

John W. Storm[1], M.ASCE and D.J. Hagerty[2], M.ASCE

ABSTRACT

Construction of an automobile test track required 30 feet of cut and fill earthwork under tight settlement restrictions. On-site materials were utilized as a rock-soil mixture to construct the track subgrade. The soil profile was monitored for settlement to determine actual compressibility characteristics of the rock-soil mixture and underlying native soil. The rock-soil fill performed excellently.

Introduction

As a part of the Toyota Automotive Plant in Georgetown, Kentucky, a test track was constructed to allow performance monitoring of new vehicles. It consists of a one-mile stretch of straight opposing lanes with high-bank 180-degree curves at each end. Because of the sensitivity of the monitoring equipment, it is of utmost importance to maintain a smooth, level driving surface. The designers specified that the asphalt surface course could experience no more than 1/2 inch total settlement.

The test track facility is located parallel to I-75 along the southwest boundary of the Toyota property. The gently rolling terrain characteristic of this area required that the test track span two valleys. Hilltop elevations varied from elevation 898 feet to 903 feet mean sea level (msl) and valley bottom elevations ranged from 860 feet to 845 feet msl. The design

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subgrade elevation varied from 873 feet to 870 feet msl, requiring maximum cut and fill depths of 20 feet and 30 feet, respectively.

The site geology, subsurface conditions and general situation of this project are described in an accompanying reprint "Shot-Rock Fill Construction: Case History." That reprint also describes construction procedures and experience at the main automobile assembly plant adjacent to the test track.

Design Considerations

Design preparations for the test track were fast-tracked during the summer of 1986. The use of an on-site rock-soil mixture had been developed for the main automobile facility platform. Our experience during construction of the main platform, which was on-going, had been successful. Therefore, the use of rock-soil fill was considered again for preparation of the test track embankment and subgrade. Other alternatives considered were to utilize crusher-run material for the embankment or to construct the test track pad of soil only.

We based our evaluation on:

- Long-term settlement performance
- Ease of placement and compaction
- Performance of fill as a road base

The use of only soil was rejected because of long-term settlement concerns and inferior subgrade characteristics. The native soils consisted of a reddish brown clay of moderate to high plasticity. Because of the cut and fill nature of the project, differential settlement between cut areas in rock and 30-feet fill zones was a major factor. The use of crusher-run rock obtained by processing the site rock had the advantages of relatively immediate settlement and no need for rock-soil mixing. The main drawback for rock-only construction was cost. A greater quantity of rock would have to be crushed for a rock-only fill, compared to a rock-soil fill; and overburden soils from the construction area would have to be wasted.

Rock-soil fill was considered the best alternative. Experience with the rock-soil mixture for the main facility indicated good subgrade bearing and compressibility characteristics, and placement of the mixture had been performed satisfactorily. Compressibility of the rock-soil fill was a question that had not been totally quantified. Our predictions based on plate load tests, consolidation tests and modified settlement rate calculations indicated that settlements could exceed project limits.

Settlement estimates for various thicknesses of rock-soil fill and native soil beneath the rock-soil fill indicated the following results:

Case	Post-Construction Probable Settlement (inches)	Worst Case Settlement (inches)
8 feet native soil 30 feet rock-soil fill	0.7	1.3
5 feet native soil 30 feet rock-soil fill	0.4	0.8
8 feet native soil 20 feet rock-soil fill	0.5	0.8
5 feet native soil 20 feet rock-soil fill	0.3	0.6

These settlement predictions indicate consolidation of the native soils under the weight of the rock-soil fill. The rock-soil itself was assumed to be negligibly compressible, and any compression in the rock-soil fill was assumed to take place during construction. However, this assumption also required verification during construction.

The worst case estimates exceeded project limits but were considered very conservative. We believed the probable estimates to be much more realistic. Therefore, several decisions were made to provide a level of assurance in the construction using rock-soil fill:

1) Native soil should be removed from the high-bank curves which contained the greater thickness of rock-soil fill;

2) A surcharge should be placed over the limits of the two drainage swales along the straight-away portion of the track; and

3) A settlement monitoring program should be performed to confirm settlement estimates.

Anticipated settlements were recalculated assuming that fill areas were surcharged with 7 to 8 feet of soil for a period of approximately 100 days. The results of this analysis are presented below:

Case	Post-Construction Probable Settlement (inches)	Worst Case Settlement (inches)
8 feet native soil 30 feet rock-soil fill	No longer applies- native soils in high bank curves removed	
5 feet native soil 30 feet rock-soil fill	0.4	0.6
8 feet native soil 20 feet rock-soil fill	0.3	0.5
5 feet native soil 20 feet rock-soil fill	0.3	0.4

The revised settlement estimates were within the project guidelines, and it was determined to proceed with rock-soil fill utilizing the previously described construction criteria.

Test Track Construction

Construction of the test track was initiated in October 1986. Hilltops above final grade elevation were blasted and the rock excavated. The subsurface exploration as well as previously performed seismic refraction surveys had indicated that some zones of weathered rock could be excavated by ripping. However, contractors who bid on the job preferred to perform all rock excavation by drilling and blasting. They felt that the overall excavation-processing operation would be easier if blasting were used for all rock zones. Blasting would fracture the rock and facilitate the crushing needed to satisfy maximum rock-fill particle size limits.

All blasting events were designed to significantly limit charge weight per delay. State regulations required that peak particle velocity in the ground at the nearest structure not exceed 2 inches per second (ips). It was recommended that the peak particle velocities at the site property line not exceed 1.0 ips, to ensure that the peak particle velocity at the nearest residence would be on the order of 0.5 ips, well below the threshold of cosmetic damage (1,6) at the vibration frequencies anticipated on this site (greater than 20 Hz).

In subgrade areas where blasting occurred, rock was overexcavated to a depth of 14 inches and replaced with rock-soil fill. Native soils in the areas to be filled were proofrolled, unsuitable material was removed, and rock-soil fill was placed. As previously mentioned, the overburden soils in the high bank curves were removed to bedrock (generally 4 to 6 feet of soil was removed).

The rock-soil fill was placed in approximately twelve inch-thick lifts using the following general procedure. The blasted rock was crushed to a 4 inch maximum size and placed in an 8 to 10 inch-thick lift. A 2 to 3 inch-thick soil layer then was placed, and harrowed into the rock lift. The mixture was compacted utilizing procedures developed on the basis of experience with two test pads. The test pads were constructed utilizing the above placement procedure. In-place density tests were performed after each pass of the compaction equipment. This data was evaluated to determine the effectiveness of compaction, the optimum number of passes to achieve compaction, and obtain guideline rock-soil densitites for spot checking during placement. This test pad procedure was performed for five successive lifts. Rock-soil samples also were obtained from each lift for grain size analysis and laboratory proctor moisture density tests. Details on the rock-soil mixture and placement procedures are presented in the accompanying paper "Shot Rock Fill Construction."

Fill placement in the two large drainage swales was accomplished without problems and completed prior to the winter season. The north and south high-bank curve fill areas were within two to three feet of final grade when cold, wet weather, and accompanying excessive moisture contents and freezing conditions, prompted construction to be delayed until spring. Transition zones between rock cut areas and rock-soil fill areas were particularly difficult to build during wet weather conditions. The rock-soil mixture was sufficiently pervious to absorb moisture readily; however, it tended to hold water in areas where it could not drain vertically into the underlying terrain. As is the case with typical soil fills, limited drying weather caused the rock-soil to compact poorly and exhibit pumping.

Approximately 7 feet of surcharge material was placed over the swale areas in January 1987. This material was loose dumped utilizing a nearby waste stockpile. The surcharge remained in-place until April. In late April of 1987, fill placement was completed. Problem areas dried sufficiently and, in most cases, tightened up under better weather conditions. The entire track was proofrolled using loaded scapers to identify localized soft subgrade areas. Field testing to determine subgrade parameters had been performed during previous studies for this project. Field California Bearing Ratio (CBR) tests indicated values of 16 to 40. Results varied due to the differing matrix of the rock-soil from location to location. In addition, numerous plate load tests were performed. Modulus of subgrade reaction ranging from 600 to 8,700 pci were achieved. Again, the results vary widely depending on site specific materials and mixing procedures. Variations can also be relatively large due to the rock-soil matrix from location to location.

Settlement Monitoring

A monitoring program was developed consisting of two methods of settlement measurement. The monitoring system consisted of a series of four buried steel plates installed during the fill construction, and a series of five Sondex casings installed in 6-inch diameter holes drilled after placement of the rock-soil fill material. The sondex casings and settlement plates were installed in pairs approximately ten feet apart, with the exception of Location 5 (see Figure 1). This location was in the south highbank curve where native soils were removed. Therefore, a settlement plate was not installed. The settlement plates measured the deflection of the residual soils during and after construction. The Sondex system measured incremental deflections in the rock-soil fill and in the residual soil at various levels after the casings were installed.

Settlement Plates: The settlement plates consisted of 3 feet by 3 feet by 1/2 inch-thick steel plates. They were installed on the native soil surface prior to rock-soil fill placement.

Plates 1 and 2 were located in the greater fill area of the northern swale, and plates 3 and 4 were located in the similar area of the southern swale. The plate areas were leveled manually and the steel plates were placed on the prepared surface. The survey party then determined the elevation and location of the plates from nearby benchmarks. Approximately 1.5 to 2.0 feet of soil was placed over the plates and compacted with passes of an unloaded scraper. The levelness of the plates was rechecked by probing through the soil fill with a rod.

After installation of the settlement plates, rock-soil fill placement was continued in the swale areas. When design road subgrade level was achieved, a rotary drilling rig was utilized, employing compressed air as the circulation fluid, to drill through the rock-soil fill and onto each steel plate. A 2-inch PVC casing then was installed from the surface to the plate, and the annulus around the casing was backfilled with small pea gravel.

After surcharge placement, a 1/2-inch diameter steel rod was placed through each PVC casing to rest on the settlement plates. Settlement plate elevations were taken on January 13, 1987 to measure the amount of settlement that occurred in the native soils during construction. Additional measurements of the settlement plates were performed on March 13 and April 8, 1987, with the results shown below.

Settlement Plate Measurements

Settlement Plate	Plate Elevation (As Placed)	01/13/87	03/14/87	04/08/87	Total Settlement
1	852.77	852.74	852.74	852.74	.03 (.36")
2	849.87	849.84	849.84	849.84	.03 (.36")
3	851.85	851.84	851.84	851.83	.02 (.24")
4	*	859.88	859.85	859.85	.03 (.36")*

*Plate #4 was disturbed during construction. The reported settlement is based on the difference of the 01/13/87 reading from the 04/08/87 reading. This does not reflect settlement which took place during construction.

Sondex System: The Sondex system consisted of a 3 inch diameter casing with metal sensing rings at 2-foot increments. The casings were installed by drilling a 6-inch (nominal) diameter hole through the rock-soil fill and residual soils, terminating 5 feet into the bedrock. The casing was grouted into the bedrock and the annulus backfilled with pea gravel. Settlement measurements were performed by lowering a probe which located the metal rings by means of electrical induction. Sondex readings were performed on a weekly basis for a period of approximately 2 months.

No trend of settlement could be developed in the sondex data for Locations 1 and 2. Sondex measurements oscillated about the initial reading. The oscillations were generally within the accuracy of the equipment and operator system - 1/8 inch. These data indicated negligible additional settlement occurred after installation. Readings from Sondex locations 3 and 4 indicated 1/8 to 1/4 inch of settlement during the sondex measurement period. These data are supported by the settlement plate data. The differences in settlement measurements were believed to be associated with soil variations between the north and south swales. Sondex location 5, located in the southern highbank curve, indicated no measurable settlement in the rock/soil fill. The residual soil had been removed to rock.

ettlement Evaluation: A settlement versus time plot was developed for each settlement measurement location with the exception of Location 4 (see Figures 2, 3, 4, and 5). The settlement plate for Location 4 was disturbed. The curves for the remaining locations were derived by using the settlement plate data (January 13, 1987) to obtain an initial data point at the completion of fill placement. A curve was extrapolated backward to the beginning of fill placement. The Sondex data points were then plotted to extend this curve forward with time. The range of anticipated settlement calculated for the test track described previously is also shown on the plots. The upper limit of the range was developed as the "probable case" settlement. The lower limit of the range represents the "worst case" settlement.

Conclusions

Use of rock-soil fill proved to be an economical and cost-effective means for constructing the required test track facility. Close control of blasting and crushing activities produced acceptable constituents for fill construction. Proper mixing and compaction procedures, developed through construction of test pads, were vitally important to the success of the construction process.

Comprehensive monitoring of settlement during and after construction of the test track facility indicated that the final post construction settlements would be well within the 1/2 inch limit set by track designers. Consolidation of the residual silty clay beneath the rock-soil fill occurred much more rapidly than was indicated by the results of laboratory oedometer tests. Worst-case estimates of settlement, based upon a combination of the unfavorable results of the laboratory and field tests, and a possible but not probable loading situation, exceeded actual measured settlements. "Most probable" estimates of settlement only slightly exceeded measured values. The rock-soil fill showed negligible compression, even under significant surcharge.

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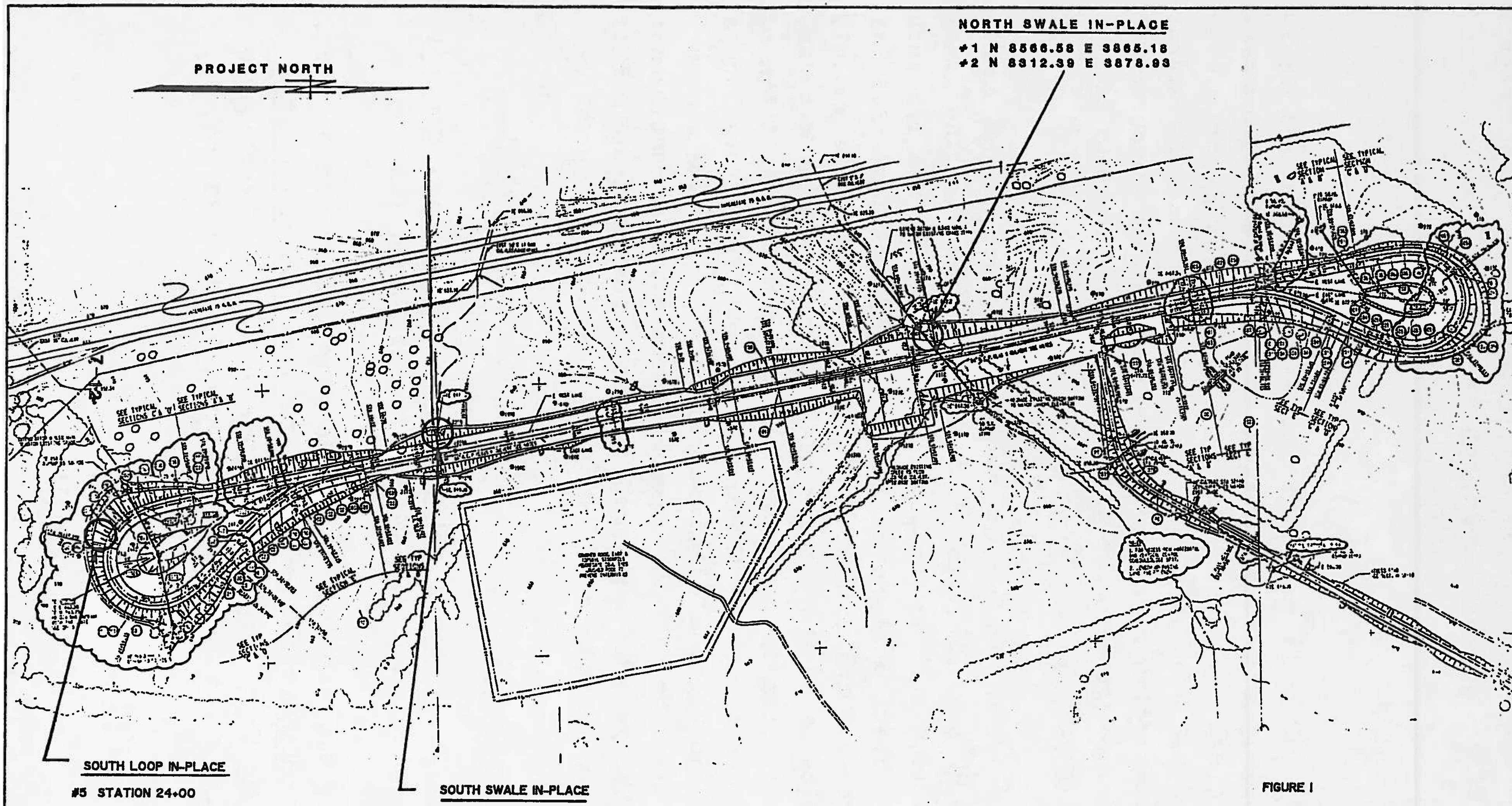


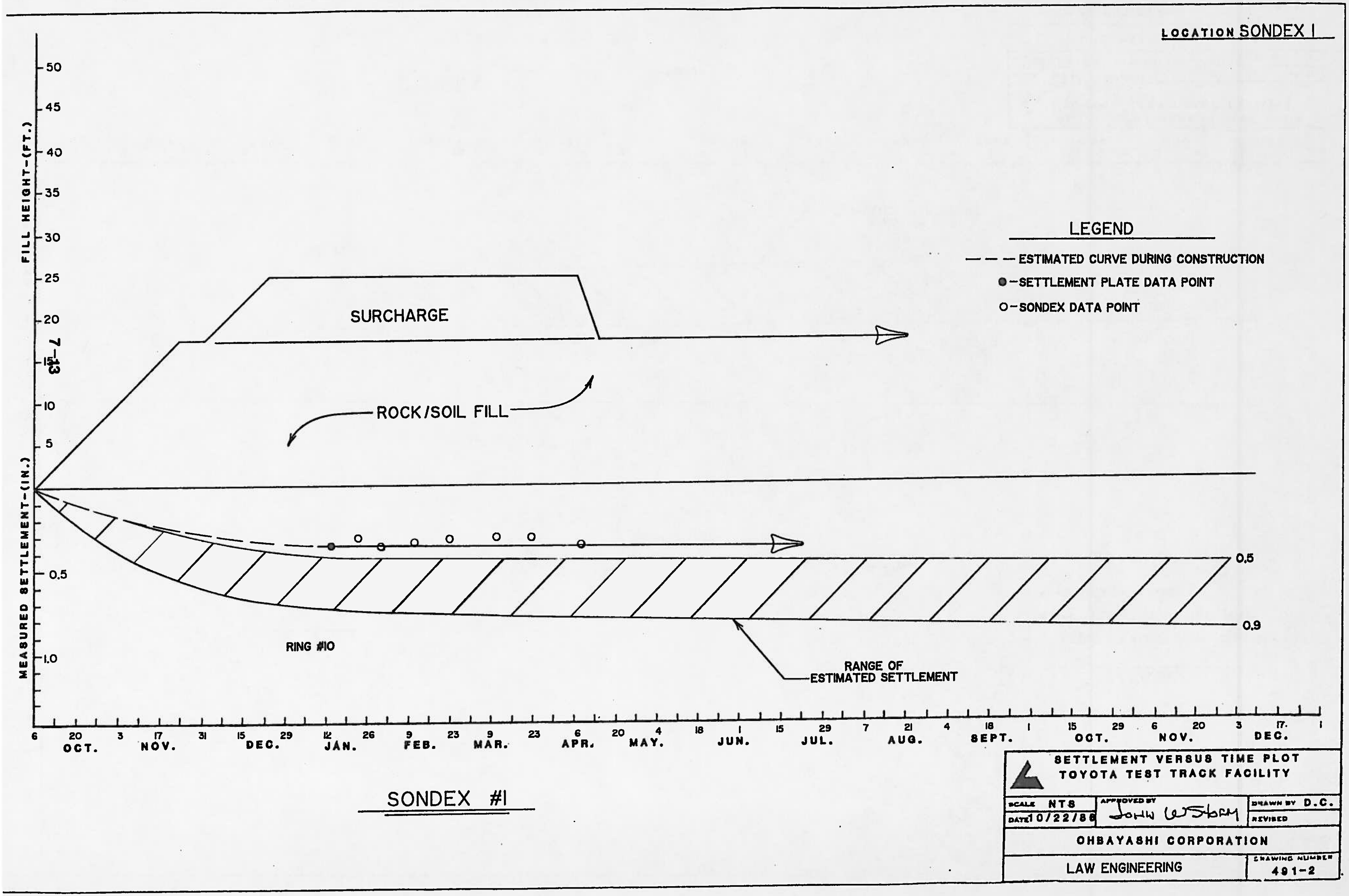
FIGURE I

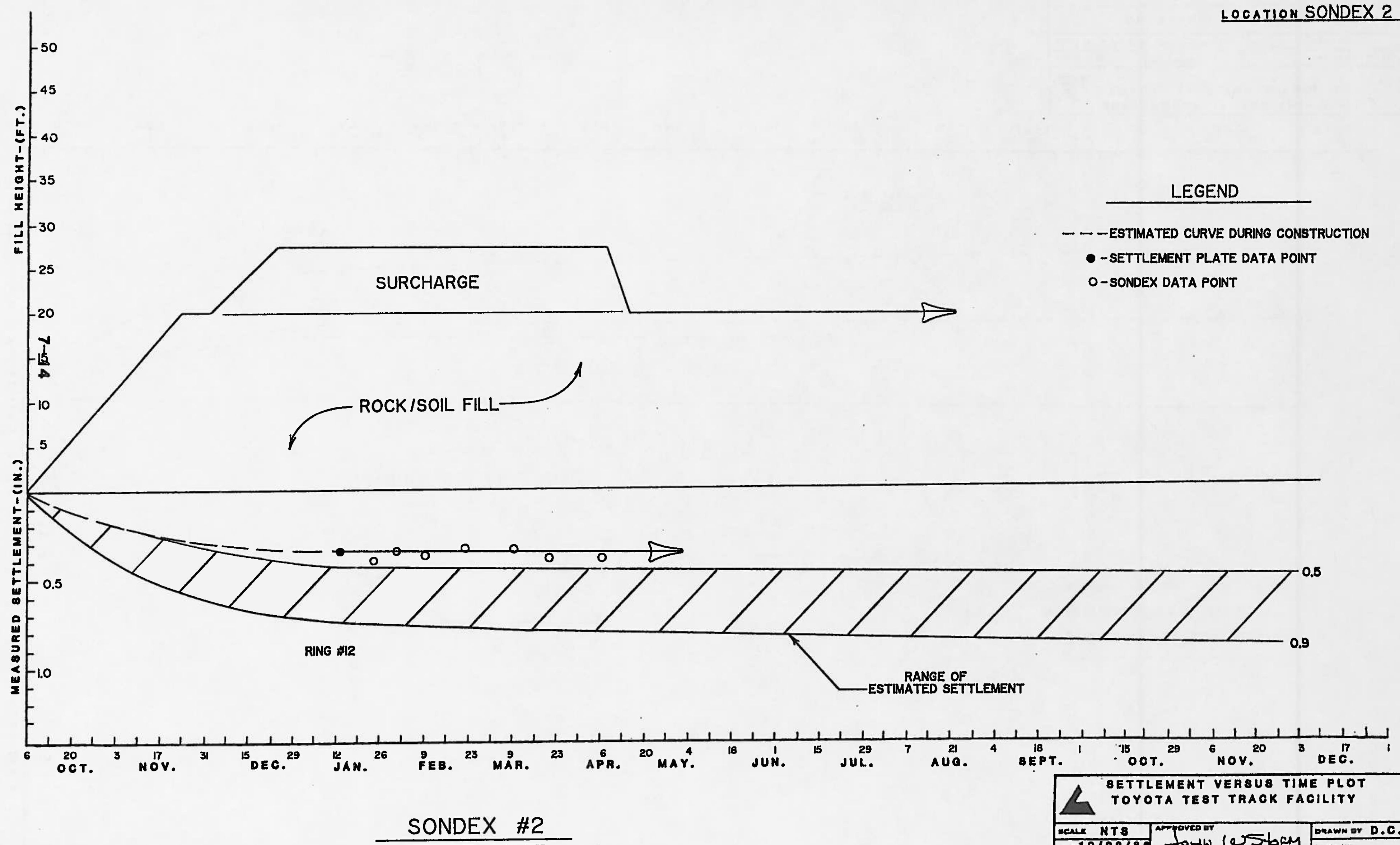
IN-PLACE SETTLEMENT MONITORING LOCATIONS
TOYOTA TEST TRACK FACILITY

SCALE NTS	APPROVED BY	DRAWN BY D.C.
DATE 10/22/86		REVISED 2-13-87

OHBAYASHI CORPORATION

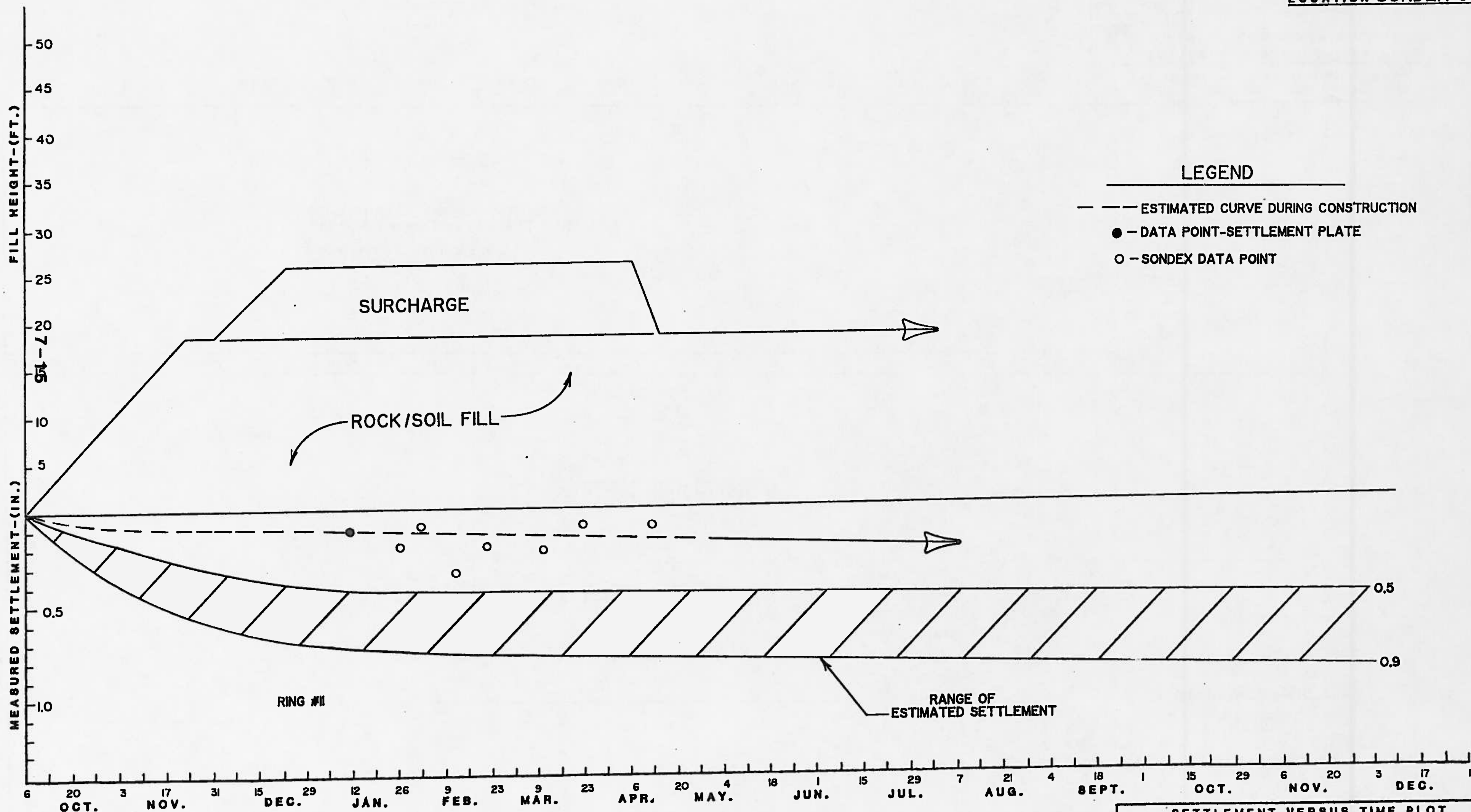
Law Engineering Testing Company — DRAWING NUMBER 94H-1


LOCATION SONDEX I
LEGEND
ESTIMATED CURVE DURING CONSTRUCTION
SETTLEMENT PLATE DATA POINT
SONDEX DATA POINT
FILL HEIGHT-(FT.)
MEASURED SETTLEMENT-(IN.)
50
45
40
35
30
25
20
15
10
5
0.5
1.0
SURCHARGE
ROCK/SOIL FILL
RING #10
RANGE OF ESTIMATED SETTLEMENT
0.5
0.9
6 20 OCT.
3 17 NOV.
31 15 DEC.
29 12 JAN.
26 9 FEB.
23 9 MAR.
23 6 APR.
20 4 MAY.
18 1 JUN.
15 29 JUL.
7 21 AUG.
4 18 SEPT.
1 15 OCT.
29 6 NOV.
20 3 DEC.
17 1
SONDEX #1
SETTLEMENT VERSUS TIME PLOT
TOYOTA TEST TRACK FACILITY
SCALE NTS
DATE 10/22/86
APPROVED BY John Wsborn
DRAWN BY D.C.
REVISED
OHBAYASHI CORPORATION
LAW ENGINEERING
DRAWING NUMBER 491-2



SETTLEMENT VERSUS TIME PLOT TOYOTA TEST TRACK FACILITY		
SCALE NTS DATE 10/22/86	APPROVED BY John W Sborry	DRAWN BY D.C. REVISED
OHBAYASHI CORPORATION		
LAW ENGINEERING		DRAWING NUMBER 491-2

LOCATION SONDEX 3



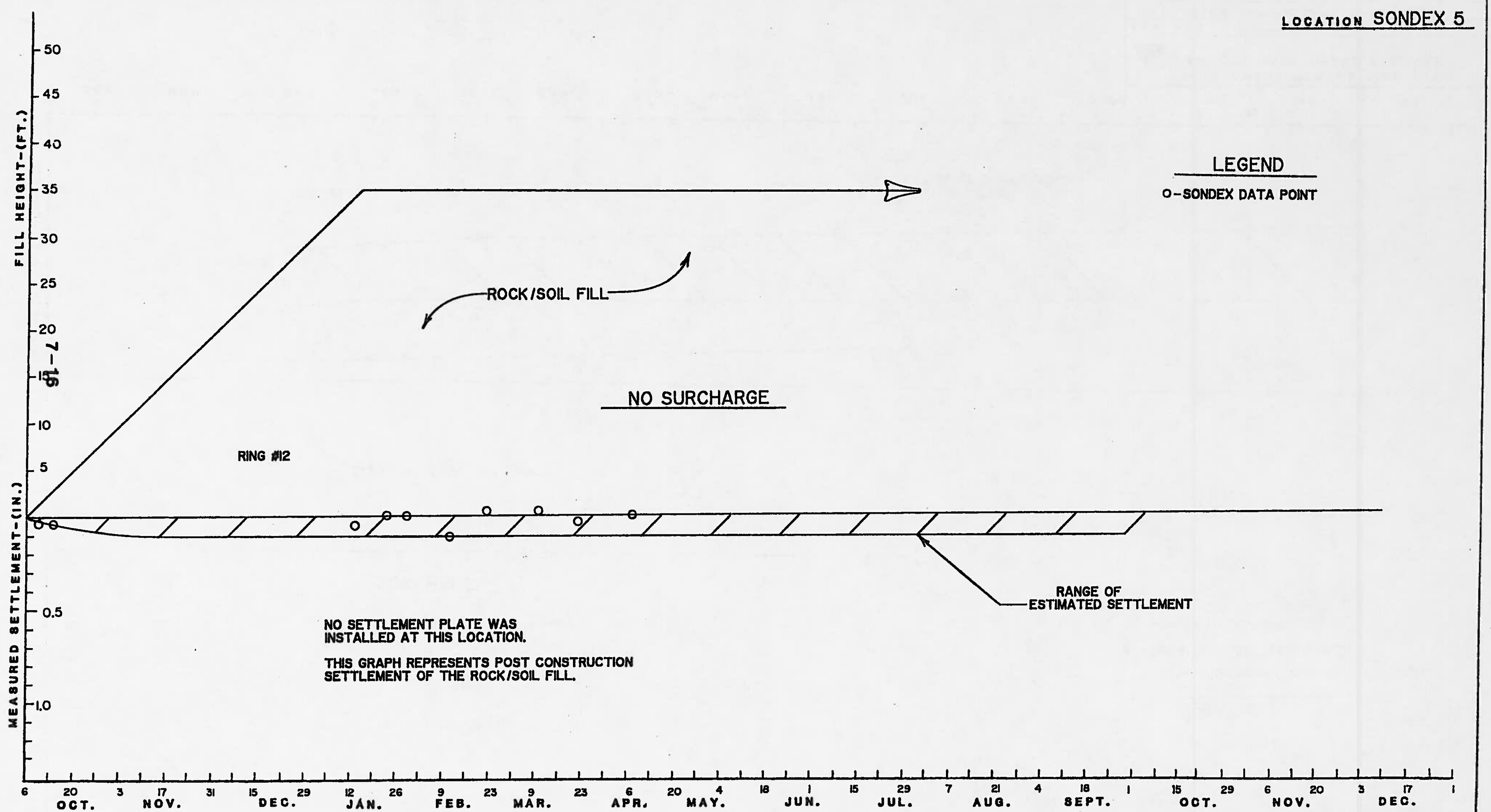
SONDEX #3

SETTLEMENT VERSUS TIME PLOT
TOYOTA TEST TRACK FACILITY

SCALE NTS | APPROVED BY John W. Storm | DRAWN BY D.C.
DATE 10/22/86 | REVISED

OHBAYASHI CORPORATION

LAW ENGINEERING | DRAWING NUMBER 491-2



SONDEX #5

SETTLEMENT VERSUS TIME PLOT TOYOTA TEST TRACK FACILITY		
SCALE NTS DATE 10/22/86	APPROVED BY John W Storm	DRAWN BY D.O. REVISED
OHBAYASHI CORPORATION		
LAW ENGINEERING		DRAWING NUMBER 491-2

Shot-Rock Fill Construction: Case History

By D. Joseph Hagerty,[1] Nicholas G. Schmitt,[2] and Gerald T. Vandevelde,[3]
Members, ASCE

Abstract: Construction of an automobile assembly plant with 3,000,000-sq ft (280,000-m^2) plan area required excavation of more than 2,700,000 cu yd (2,000,000 m^3) of soil and rock, and placement of more than 1,700,000 cu yd (1,300,000 m^3) of fill. Footings on rock and piers drilled through soil-rock fill were used to support column loads. Shale-limestone bedrock was excavated by blasting. Processed rock yielded sufficient fines to provide some cohesion to the fill. Fill performance was evaluated by field plate loading tests. Settlements of fill and underlying residual soils were monitored during construction. Construction was completed successfully within a compressed schedule. Plate load tests provided important data on soil-rock fill compressibility. Settlement readings confirmed laboratory and field test results, and verified predictions of settlement magnitude. Rate of settlement greatly exceeded estimated rates. Pier drilling through the soil-rock fill was accomplished without liners, with no significant ravelling of fill material.

Introduction

An industrial plant complex occupying more than 3,000,000 sq ft (280,000 m^2) was proposed for construction in an area of rolling terrain in central Kentucky. The major portion of the plant area was to have a uniform finished floor elevation of 920 ft (280 m) mean sea level (msl), but more than 360,000 sq ft (33,000 m^2) of pits were to extend to elevation 912 ft (278 m) msl, and approximately 400,000 sq ft (36,000 m^2) of pits were to reach elevation 902 ft (275 m) msl. In order to achieve the proposed main floor elevation, as much as 40 ft (12 m) of fill and 30 ft (9 m) of cut would be required, with deeper cuts/shallower fills in the pit areas. Approximately 3,600,000 cu yd (2,700,000 m^3) of primary excavation would be required on the site.

The plant structure was to be a metal-framed building with bay spacings from 48–96 ft (15–30 m). Column loads were to average about 300 kips (136 tonnes), with column loads of about 600 kips (272 tonnes) in some areas, and a few isolated column loads of 1,000 kips (450 tonnes). Exterior walls were to rest on grade beams supported by adjacent columns. Floor loads throughout most of the plant were to be less than 500 pounds per square foot (psf) (250 KPa) typically, but storage requirements near one of the pits would increase floor loading to 2,000 psf (1,000 KPa). Contact pressures below the pits were to range from 1,000–4,000 psf (500–2,000 KPa).

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Note. Discussion open until February 1, 1989. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on September 4, 1987. This paper is part of the ***Journal of Construction Engineering and Management***, Vol. 114, No. 3, September, 1988. ©ASCE, ISSN 0733-9364/88/0003-0489/$1.00 + $.15 per page. Paper No. 22784.

Structure designers wished to limit total settlement within the building area to 1 in. (2.5 cm), and imposed a limit on distortion of 1/1,200, i.e., differential settlement between any two points, divided by the distance between points, should not exceed 1/1,200.

Site Conditions

Site Geology

The central portion of the site is underlain by interbedded layers of shale and limestone of the Clays Ferry formation, generally at elevations above 910 ft (277 m) msl, with shaley limestone layers of the Lexington limestone formation below 910 ft (277 m) msl, near the surface under the outer portions of the site. All these rock units consist of interbedded layers of shale and limestone, from about 2–18 in. (5–46 cm) thick with shale components more significant above 910 ft (277 m) msl. The interbedded rock layers vary significantly in resistance to weathering. In some spots, test excavations uncovered first residual soil, then layers of weathered limestone, then zones of clayey residuum from weathering of more soluble/degradable rock, and then relatively competent rock.

The horizontally bedded units contain closely spaced vertical joints, but continuous open joints are virtually nonexistent because of the low strength/high deformability of the shale layers. The shaley nature of the limestones has prevented appreciable solutioning.

The rolling terrain, low permeability of surface soils, and low mass permeability of the rock units promote rapid runoff and minimize infiltration. Well yields are very low except in narrow belts of alluvium along streams on the site periphery.

Subsurface Conditions

Hollow-stem auger borings were made at 273 locations at spacings of 250 ft (76 m) both ways on a plan grid pattern superimposed on the site. Eighteen additional borings at spacings of 500 ft (152 m) were made in an area of proposed future plant expansion. In brief, three types of materials were found in the borings: surface residual soil; laminae of weathered rock and soil; and essentially continuous rock.

The residual soils on-site consisted of firm to hard, yellow-tan-brown, silty clays and clays, with property value ranges as follows: thickness range = 2–14 ft (0.6–4.3 m); thickness average = 8 ft (2.4 m); USCS classification = CL-CH; liquid limit = 37–80%; plasticity index = 12–50%; and water content = 12–38%. Natural moisture contents exceeded corresponding optimum moisture contents for compaction (ASTM D 1557) by about 10%.

The properties of the residual soils reflected the character of the rock units from which they developed, with more plastic soils developed from shale-rich units. Thin, discontinuous rock layers (floaters) typically 1–5 in. (2.5–12 cm) thick were found in the residual soil, particularly in thicker soil zones.

In some of the borings, interbedded layers of partially weathered rock [typically less than 3 in. (7 cm) thick] and soil were found, in total thicknesses up to more than 7 ft (2 m) but averaging about 2.5 ft (0.8 m) thick.

Below the zones described, relatively continuous layers of rock were found. Limestone layers were typically light gray, crystalline, and moderately hard to hard (Dept. of Navy 1982) with occasional yellow-brown fossiliferous layers. Shale layers were dark gray to gray and soft to moderately hard. Bed thicknesses ranged from 2–24 in. (5–64 cm), but most beds were 6–10 in. (15–25 cm) thick. Core recovery ranged from 19–100%, averaging about 90%. Rock quality designation (RQD) values ranged from 0–100%, averaging about 60%. Five-cycle slake durability indices (Commonwealth of Kentucky 1985) ranged from 63% (shale-rich samples) to 93% (limestone samples). Unconfined compression strength values varied from 11,400 psi (78.5 MPa) (average for limestone) to 6,200 psi (42.7 MPa) (average for shale).

Foundation Design

The overall plant design called for an essentially flat "table top" to support manufacturing operations. Column loads up to 1,000 kips (450 tonnes) were to be supported immediately adjacent to floor slabs with very light loads. Differential settlement considerations significantly affected the choice of foundation system.

Footings were rejected because of possible differential settlement between footings on rock and those on fill and because of uncertainties if all footings were placed on rock-soil fill. Footings were used only in cut areas in rock.

Insufficient soil existed on-site to permit construction of a soil-only fill, and serious questions arose concerning the compressibility of rock and rock-soil fills. Limited data on the behavior of rock fills in dams were available (Clements 1984; Cooke 1984; Wilson 1973; Wilson and Marsal 1979), but lift thicknesses, rock character, and required precision of compressibility estimates were quite different on this project compared with rock-fill dam projects. The properties of the rock fill produced on this site were seen to be highly dependent on the rock parent material (shale-rich rock versus limestone-rich strata), the degree of rock processing (crushing, screening, and washing), and the degree of compaction. The tight construction schedule also precluded preloading of fill to reduce settlement. Deep foundations were required in fill areas.

Driving piles through soil-rick fills would have required expensive processing of the rock to reduce rock size before fill placement, and seating of driven piles on bedrock would have been somewhat uncertain because of the presence of "floaters" above sound rock. Drilling of piers through a rock fill or a soil-rock fill appeared to be more practical, since larger rock pieces in the fill could be accommodated in the drilling (compared to pile driving), and "floaters" could be detected and removed. Required piers varied from three feet (0.9 m) to nine feet (2.7 m) in diameter. Drilling would be attractive, however, only if the holes through the fill would stay open without the use of expensive support techniques.

Constraints

Differential settlement considerations and enforced construction during wet weather virtually eliminated any soil-only fill. With regard to soil-rock or rock fills, differential settlement considerations also were significant.

The fill could not experience serious settlement under 2,000 psf (1,000 KPa) floor loads. Several soil samples from the site showed high potential in laboratory tests for volume change and generation of swell pressures in response to changes in moisture content. Soils developed from shales in the formation that underlies this site commonly exhibit significant shrink-swell potential. These factors favored rock-only fill construction. However, more than 20% of the floor area of the plant was to be occupied by pits either 8 or 18 ft (2.5 or 5.5 m) deep. Driving of sheet piles to retain pit sides during excavation, excavation of the pits as well as trenches for utilities in the plant, and possible future excavations of a similar nature to accommodate plant modifications all required that the maximum particle size of the rock fill be limited. Drilling for piers at column locations also favored a reduction in maximum rock particle size. If casings were required to stabilize the pier excavations during drilling, the pier construction cost would have been increased very significantly. Uncased drilling would require sufficient fines in the fill to act as a binder on the larger rock particles. However, if the fines content were furnished by adding soil to the rock fill, the compressibility of the fill would be increased. These constraints had serious effects on rock excavation methods, soil and rock processing requirements, and rate of fill placement. Since the rock component of a soil-rock fill could contain varying percentages of limestone and shale "gravel," as well as pulverized limestone and shale "fines," close control of processing and placement would be required if the fill were to have the required combination of strength, stiffness, and cohesion.

Finally, time constraints were severe on this project since the owner wanted all site preparation and filling work completed between February and June. This mandated high production rates for excavation, processing, and filling, as well as requiring that such accelerated operations be carried out during the (typically) wettest season of the year.

Construction Plan

The seismic refraction survey conducted on this site indicated that some zones of weathered rock could be excavated by ripping. However, all local site contractors who had indicated an interest in bidding on the work stated that they would perform all rock excavation by drilling and blasting. This decision was based partially on personal preference, but it also reflected the feeling that the overall excavation-processing operation would be easier if blasting were used for all rock zones. Maximum rock-fill particle size limits required crushing and grinding of the rock, and it was felt that this processing would be facilitated if excavation by blasting fractured the rock as it was removed.

Rock removal was to be carried to lower elevations under a press pit and an adjacent coil storage area, where all weathered rock was to be removed to reduce settlement. In all other areas where rock-soil fill was to be placed, the exposed natural soils were to be proofrolled before fill placement. A ten-wheeled truck with a total weight of at least 10 tons (9.1 tonnes) or other equipment producing a contact pressure of at least 4 ksf (2 MPa) was recommended for the proofrolling. All soft or excessively wet soils, as well as all organic zones and old fill areas, were to be removed.

TABLE 1. Suggested Rock-Soil Mixtures

Maximum nominal particle size (1)	PERCENT PASSING — Gradation Sieve Size									Maximum Lift Thickness		Static Weight of Sheepsfoot Roller or Smooth Roller		Projected number of compactor passes (15)	Allowable moisture range (16)
	12 in. (30.5 cm) (2)	8 in. (20.3 cm) (3)	6 in. (15.2 cm) (4)	4 in. (10.2 cm) (5)	2 in. (5.1 cm) (6)	No. 4 (7)	No. 40 (8)	No. 100 (9)	No. 200 (10)	in. (11)	cm (12)	tons (13)	tonnes (14)		
4 in. (10 cm)	—	—	100	98–100	—	30–80	20–40	10–30	10–30	12	30	20	18	6	± 5% of optimum for D-1557-C on −0.75-in. (1.9-cm) material
8 in. (20 cm)	100	98–100	—	—	40–100	30–80	20–40	10–30	10–30	16	41	20	18	6	± 5% of optimum for D-1557-C on −0.75-in. (1.9-cm) material

The character of the rock-soil fill was limited by the constraints of minimizing floor slab settlements while facilitating pier excavation. Recent research (Matheson 1986) has confirmed the writers' judgement that at least 30% of the fill material should pass the No. 4 sieve in order to get good compaction of the fine matrix between the rock pieces. Two gradations for rock-soil mixtures were developed with suggested guidelines for compaction, as shown in Table 1.

Considerable uncertainty existed about the "mix proportions" of the rock-soil fill because the character of the rock would vary during construction (shale-rich versus limestone-rich), and the behavior of the rock during blasting and processing was unknown. To determine how much soil should be added to the rock, in what way, and to develop procedures for rock excavation and processing, the writers recommended construction of a test strip and load testing of the compacted fill as described in the next section.

The main plant structures (columns and machine pads) were to be founded on rock-supported footings or piers, sized using a contact pressure of 40 ksf (20.2 MPa) for piers and 20 ksf (10.1 MPa) for footings. Because hard rock could be exposed at the founding level with soil seams at lower elevations, the writers recommended drilling probe holes at each pier/footing location. The probe holes were 1.5 in. (38 mm) in diameter and extended to a depth at least 1.5 times the pier bottom diameter (or footing width) or at least 5 ft (1.5 m). Pier bottom inspection was carried out in a temporary steel casing at least 30 in. (76 cm) in diameter.

Floor slabs for the plant were to be designed using a subgrade modulus for rock-soil fill of 200 pounds per cubic inch (pci) (5.5×10^6 kg/m^3) or a modulus of 400 pci (11×10^6 kg/m^3) for clean crushed stone over bedrock. The writers recommended that floor slabs be jointed around columns and along footing-supported walls to permit differential settlement. Because of the sensitivity of some soils at this site to changes in moisture content, it was recommended that construction of the floor slabs be delayed until the roof had been placed and the subgrade moisture had stabilized. Testing of the subgrade for density and moisture content was recommended prior to slab construction.

Construction Experience

Test Pad Construction

To investigate the suitability of available equipment and to determine the influence of soil-rock processing and mix proportions, two test fill strips were built. Before the test fills were placed, an area was cleared and grubbed, and all topsoil was stripped and stockpiled. Then 12–18 in. (30–45 cm) of crushed stone was placed over the exposed natural soils and compacted with a vibratory roller (Ingersoll-Rand SP5 6DD; 20,700 lb (143 MPa) static weight; 42,000 lb (290 MPa) maximum dynamic force; 0–14 Hz frequency). Then, successive lifts of various mixtures of soil and rock were placed and compacted over the base layer of crushed stone. The test pads were divided into four sections, and the compactive effort was varied from section to section.

The in-place density was checked with a nuclear density meter, as well as by a modified sand cone technique after each compactor pass. Samples

TABLE 2. Grain-Size Analysis: Percent Passing (by Weight)

Sieve size (1)	Crusher			Test pad 1, lifts 4 and 5 (5)	Design blend (6)
	1 (2)	2 (3)	3 (4)		
6 in.	100	100	100	100	100
4 in.	100	100	100	100	98–100
2 in.	88	76	75	85	—
3/4 in.	67	60	43	61	—
No. 4	34	24	17	46	30–80
No. 40	8	1	5	33	20–40
No. 100	3	—	2	29	10–30
No. 200	1	—	1	28	10–30

also were obtained from each lift for grain-size analysis and laboratory Proctor moisture-density tests.

In building the first test pad, the following procedure was used. In the first lift, eight inches of crushed rock were mixed with four to six in. (10–15 cm) of site soil using a caron wheel roller (A258 Caterpillar; 60,000 lb (415 MPa) gross weight) and a grader. This mix was compacted with five passes of the vibratory roller, operating at maximum frequency.

The second lift consisted of eight in. (20 cm) of crushed rock and four inches of soil. Mixing was done with the caron wheel roller and a crawler equipped with a root rake. Five passes of the vibratory roller were applied. The third lift was essentially identical to the second lift.

The fourth lift consisted of 8 in. (20 cm) of crushed rock and 2–3 in. (5–7 cm) of soil, mixed with a disc and compacted with the vibratory roller. The fifth lift was identical to the fourth, producing a combined compacted thickness of about 18 in. (45 cm) for lifts 4 and 5.

The blasted rock was processed on-site with three portable crushers; the texture of the crusher output, as well as that of the rock-soil mix in test pad 1 is shown in Table 2.

The second test pad was 20 ft (6 m) wide and 80 ft (24 m) long (top area dimensions), as was the first pad. However, in the second pad, the first lift consisted of 3 in. (8 cm) of loose soil mixed with 9 in. (23 cm) of crushed rock. One-half of the first lift was mixed with two passes of the disc and two passes of the caron wheel compactor; the other half of the lift was mixed with four passes of the disc. Six passes of the vibratory compactor were made over the entire lift. Only visual inspection was made for the first lift, since that lift was used only to test mixing techniques.

The second lift also consisted of one-fourth soil, three-fourths crushed rock. A grain size analysis was done on this lift, as were two Proctor moisture-density tests. The grain size analysis results were virtually identical to the results obtained for lifts 4 and 5 of the first test pad. The pad was divided into fourths, and the fourths were compacted with two, four, six, and eight passes of the vibratory compactor. In each fourth of the area, three field density tests were made using a six-inch diameter sand cone apparatus, and at one location in each fourth, a twelve-inch (30 cm) diameter sand cone apparatus was used near one of the six-inch (15 cm) cone test sites. At each cone test site, two density determinations were made using a nuclear density meter.

TABLE 3. Test Results, Test Pad 2

Test (1)	Two Passes: Dry density, pcf (tonne/m³) (2)	Two Passes: Moisture content (%) (3)	Four Passes: Dry density, pcf (tonne/m³) (4)	Four Passes: Moisture content (%) (5)	Six Passes: Dry density, pcf (tonne/m³) (6)	Six Passes: Moisture content (%) (7)	Eight Passes: Dry density, pcf (tonne/m³) (8)	Eight Passes: Moisture content (%) (9)
(a) Location 1								
12-in. sand cone	130.0 (2.08)	7.6	132.0 (2.16)	4.7	133.9 (2.14)	6.3	130.3 (2.09)	4.2
Nuclear meter	128.8 (2.06)	3.1	135.3 (2.17)	3.3	124.1 (1.99)	9.2	119.9 (1.92)	9.2
6-in. sand cone	125.6 (2.01)	8.4	134.6 (2.16)	2.9	119.9 (1.92)	10.0	128.8 (2.06)	9.8
Nuclear meter	128.6 (2.06)	3.9	121.8 (1.95)	3.8	122.9 (1.97)	9.6	124.3 (1.99)	5.3
(b) Location 2								
6-in. sand cone	136.6 (2.19)	4.1	127.0 (2.04)	10.2	119.3 (1.91)	9.4	104.3 (1.67)	11.1
Nuclear meter	132.5 (2.12)	4.4	128.9 (2.07)	6.9	121.8 (1.95)	5.1	122.9 (1.97)	5.8
(c) Location 3								
6-in. sand cone	106.6 (1.71)	10.4	123.4 (1.98)	10.5	127.9 (2.05)	4.6	119.8 (1.92)	7.3
Nuclear meter	124.4 (1.99)	6.9	123.8 (1.98)	5.9	133.6 (2.14)	5.3	129.4 (2.07)	3.8

The density-moisture content data obtained are shown in Tables 3 and 4. These data indicate that compaction of the soil-rock fill could be achieved with four passes of the vibratory compactor [95% of the Proctor maximum dry density was 124–127 pcf (1.99–2.03 t/m³)]. Based on the results of the test pad construction, the following placement procedure was used for rock-soil fill: (1) Crushed rock was placed to a loose thickness of approximately nine inches; (2) two to three inches of soil was spread on the crushed rock; (3) soil/rock mixing was done with a disc, using a minimum of four passes; (4) compaction was done with a minimum of four passes of the vibratory roller; and (5) moisture content of the mix was held within ±5% of the optimum moisture content of that portion of the material that passed the 3/4-in. sieve.

Excavation of drilled piers through the rock-soil fill disclosed no significant segregation of soil from rock. This lack of segregation indicated that mixing procedures were successful.

TABLE 4. Proctor Test Results (Minus 3/4-in. Material, Uncorrected)

Location (1)	Maximum dry density, pcf (tonne/m³) (2)	Optimum moisture content (%) (3)
1: near 12-in. sand cone test in two-pass area	131 (2.10)	8.0
1: near 12-in. sand cone test in six-pass area	134 (2.15)	7.0

Production Earthwork

After compaction and mixing procedures were developed from the test pad trials, full-scale production began. After some weeks of fill placement, it was noted that the rock coming from the crushers possessed sufficient fines to satisfy the gradation requirements without the addition of soil. However, it was decided to continue to add soil to the crushed rock, in a reduced ratio of one inch (2.5 cm) of soil to ten inches (25 cm) of loose rock. This decision was based on the fact that the moisture content of the crushed rock was too low for efficient compaction; addition of the reduced proportion of soil was found to provide the needed compaction moisture without exceeding the limits on fines set in the fill specifications. Moreover, a small test pad was constructed of crushed rock without soil, and an excavation was made into the test pad to simulate drilled pier excavation. Water was added to the rock to bring the moisture content to within 5% of optimum moisture content, but water loss during the hot, dry weather of test was severe. The test showed the crushed rock fill to be dry, cohesionless, and easily displaced from the sides of the trial excavation (which appeared to be marginally stable). To provide stable pier excavations, it was decided to continue to add soil to the crushed rock, and to add water, if necessary, to facilitate efficient compaction and pier construction. Fig. 1 shows a typical exposure of rock-soil fill.

Because of the usefulness of the test pad experiments, it was decided to construct a small test pad approximately 100 × 100 ft (30 × 30 m) each week to provide a better correlation between laboratory and field testing, and field performance. For each test pad, a series of tests were conducted: gradation tests on material in place before and after soil was added; nuclear density meter tests; sand-cone density tests; field moisture content determinations; and laboratory Proctor compaction tests.

In addition to this testing, in areas of parking lots and roadways, California bearing ratio tests were performed on mixed fill with the results shown in Table 5.

These fill areas contained soil mixed with rock fragments, but no effort had been made in these areas to satisfy specifications for "rock-soil fill" as given elsewhere herein. The presence of rock fragments undoubtedly accounted in part for the fact that the field CBR values were much higher than the laboratory CBR test values. For comparison, the results of several laboratory CBR tests are shown in Table 6.

The laboratory tests were performed on fill material from which all rock fragments not passing a No. 4 sieve had been removed. The low degrees of saturation and the presence of large rock fragments in the field indicated

TABLE 5. Field CBR Test Results

Test location (1)	CBR at 0.1 in. (2.5 mm) penetration (2)	CBR at 0.2 in. (5 mm) penetration (3)	Dry Density		Moisture content (%) (6)	Percent saturation for Gs = 2.70 (7)
			pcf (4)	kg/m³ (5)		
Area 1	16	16	111.0	1,777	11.0	58
Area 2	31	38	109.0	1,745	15.0	74
Area 3	16	16	97.3	1,558	14.1	52
Area 4	20	16	109.8	1,758	12.0	61
Area 7	40	33	109.0	1,745	10.0	50

FIG. 1. Typical Exposure of Rock-Soil Fill; Wallet at Left in Photo is Approximately 3.5 in. (5.3 cm) Wide by 5 in. (7.7 cm) High

that the field values would not be suitable for use in pavement design; the laboratory values were recommended for use in design.

Plate Load Tests

During the early stages of fill construction, a plate load test was conducted on an 18-in. (45-cm) diameter steel plate placed on a level area in an 18-in. deep trench. In this initial test, a load of 10 tons (9.1 tonnes) was placed on the plate; a total plate deflection of 0.009 in. (0.023 cm) was noted after ten minutes. These results correspond to a modulus of

TABLE 6. Laboratory CBR Test Results

Test location (1)	CBR at 0.1 in. (2.5 mm) penetration (2)	CBR at 0.2 in. (5 mm) penetration (3)	Modified Proctor Maximum Dry Density pcf (4)	Modified Proctor Maximum Dry Density kg/m^3 (5)	Optimum moisture content (%) (6)	Percent compaction (7)	Percent swell (8)
Area 1	2.4	2.4	113.0	1,809	13.5	95.1	6.0
Area 2	6.0	5.8	112.0	1,793	16.2	95.4	3.2
Area 3	10.8	10.6	112.0	1,793	15.5	94.6	0.9

TABLE 7. Plate Load Test Results

Location (1)	Modulus of Subgrade Reaction pci (2)	kg/m³ (3)	Modulus of Elasticity psi (4)	MPa (5)
1	800	22.1×10^6	13,300	91.6
3	4,000	100.4×10^6	98,700	680.0
4	600	16.6×10^6	8,600	59.3

subgrade reaction of about 8,700 pci (240,000,000 kg/m^3) or a modulus of elasticity of about 94,000 psi (650 MPa). Preliminary settlement estimates for the rock-soil fill had been based on a modulus value of 10,000 psi (69 MPa). The modulus of subgrade reaction is not a fundamental property but depends on the size of the loaded area, among other things (Terzaghi 1955). For example, the appropriate modulus of subgrade reaction for a surface footing 5 × 5 ft (1.5 × 1.5 m) in plan would be approximately one-half the value obtained for the plate used in these load tests. Because of the wide expanse of the loaded areas (floor areas) for which settlement estimates were required, the modulus of elasticity was used in calculations rather than the modulus of subgrade reaction.

After production earthwork operations began, three additional plate load tests were conducted with the results shown in Table 7.

The variation in these test results, including the first test results, can be attributed to the large size of the rock fragments in the soil-rock fill. Anyone desiring to use plate load test values from a soil-rock fill in foundation analysis and design should take great care to evaluate the effect of large fragments on plate load test results.

Settlement Monitoring

In an effort to verify design assumptions on compressibility of rock-soil fill and residual soil, five settlement plate gages were installed in the plant construction area. The gages were fabricated using 4 × 4 ft (1.2 × 1.2 m) plywood panels, one inch (2.5 cm) thick. In the center of each panel, a metal pipe flange was fixed, and a two-inch (5-cm) diameter pipe was screwed into the flange. A four-inch (10-cm) diameter polyvinyl chloride pipe was set over the metal pipe as a protective casing. At each monitoring location, the fill or soil surface was leveled to receive the plywood plate. The conditions at each monitoring location, as well as the settlements recorded during the first six months after fill placement began, are shown in Table 8.

The accuracy of these settlement readings was compromised when temporary benchmarks were destroyed and had to be replaced. However, the settlement measurements clearly demonstrated that residual silty clay settlement was essentially complete within three months after completion of fill placement. It appeared that the rock-soil fill was more compressible than had been predicted, a result in contradiction to the results of the plate load tests. To clarify this apparently contradictory situation, additional measurements of settlements were made during a later phase of this project.

In this subsequent investigation, settlement plates and a second method of settlement monitoring were used to measure settlement within the

TABLE 8. Settlement Readings

Plate (1)	Depth of Fill over Plate ft (2)	m (3)	Depth of Rock/Soil Fill under Plate ft (4)	m (5)	Depth of Residual Soil under Plate[a] ft (6)	m (7)	Settlement in. (8)	cm (9)
1	18	5.4	0	0	11	3.3	1.2	3.0
2	11	3.3	7	2.1	11	3.3	1.7	4.3
3	10	3.0	10	3.0	0	0	1.8	4.6
4	12	3.6	2	0.6	11	3.3	0.5	1.2
5	16	4.8	0	0	12	3.6	1.0	2.5

[a]Estimated from exploratory borings.

soil-rock fill and in underlying residual soil beneath an automobile test track built adjacent to the main plant building. It was of utmost importance to minimize post-construction settlement of this track so that electronic vibration monitoring equipment could be used to evaluate performance of automobile suspension systems and other structural components.

Four steel plates were buried during fill construction. After the surcharge was placed, additional settlement monitoring devices were installed. The additional devices consisted of Sondex casings manufactured by Slope Indicator Company. These casings consisted of corrugated plastic pipe, four inches (102 mm) in inside diameter. Stainless steel wire loops were fixed at two-foot (61-cm) intervals around the outside of the casing. The casings were installed adjacent to the settlement plate sites in six-inch (7.5-cm) nominal diameter holes drilled through the surcharge, the soil-rock fill, and the residual soil, and 5 feet (1.5 m) into bedrock. A polyvinyl chloride pipe, two inches (5 cm) in diameter, was installed inside the casing. A fifth Sondex casing was installed through 30 ft (9 m) of soil-rock fill over bedrock. The annulus around each casing was filled with pea gravel. A Model 50819 probe was used to find the location of the wire loops on the casing after installation. When an internal coil in the probe is aligned with a wire loop, a maximum inductance is created in the probe system and can be detected easily with the surface meter furnished with the system. The location accuracy of the system is ±0.1 in. (2.5 mm). The Sondex casings were fitted with joints at final grade elevation to permit settlement monitoring after surcharge was removed.

Settlement plates 1 and 2 were installed 60 days before the Sondex casings; plates 3 and 4 were installed 50 days before the Sondex casings.

Table 9 shows the settlement data obtained from the plates and the Sondex casings 95 days after the Sondex casings were installed.

The differences between plate settlements and the compressions noted using the Sondex casings represent compressions of the residual soil during the 50–60 days between the beginning of fill placement and the installation of the Sondex casings after surcharge was placed. The Sondex readings indicated that no settlement occurred at any of the five locations later than 65 days after the Sondex casings had been installed. The very low compressibility and rapid rate of compression of the residual soil found at these locations were in good agreement with the information furnished by the earlier settlement plate measurements. These data also indicated that the rock-soil fill had very low compressibility.

TABLE 9. Settlement Readings

Site	Plate Settlement	Rock-Soil Fill		Residual Soil	
		Thickness	Compression	Thickness	Compression
	in (cm)	ft (m)	in (cm)	ft (m)	in (cm)
(1)	(2)	(3)	(4)	(5)	(6)
1	0.4 (0.9)	18 (5.4)	0 (0)	7 (2.1)	0.3 (0.8)
2	0.8 (2.1)	20 (6.0)	0 (0)	4 (1.2)	<0.1 (0.15)
3	0.8 (2.1)	18 (5.4)	0 (0)	6 (1.8)	0.2 (0.6)
4	Disturbed	10 (3.0)	0 (0)	7 (2.1)	0 (0)
5	N.A.	30 (9.0)	0 (0)	0 (0)	N.A.

To verify settlement predictions, settlement was monitored at 400 spots on the finished plant floor after construction. Preconstruction estimates of settlement under floor loads ranged from 0.2–1.1 in. (0.5–2.8 cm) for the range in soil and fill thickness and compressibility. Actual measured settlements ranged from 0.4–1.1 in. (1–2.8 cm).

Fill Deterioration in Transition Zones

Not all of the experience with the soil-rock fill on this site was satisfactory. In one building of the plant complex, the original contours of the rock surface trended diagonally across the building area. The transition between cut and fill zones likewise struck diagonally across the building footprint. After fill was placed in this transition zone in late summer, piers were drilled through the rock-soil material with no problems in autumn. In late fall and early winter, rainy weather coincided with intensive traffic on the transition zone fill (truck transfer of structural steel, roof decking, and other building materials). In December, a number of deteriorated areas 12–18 in. (30–45 cm) thick were found in the transition zone. Exploration of the softened areas disclosed distinct layering in the rock-soil fill, with inadequate mixing of materials. Soil-rich layers were supporting perched water; flow out of open rock-rich fill layers into test pits was appreciable in several spots. Throughout the entire thickness of the rock-soil fill zone exposed in each test location, the shale constituents of the fill had disintegrated completely into particles smaller than pea gravel. Observation of the test pits and borings indicated that water was flowing through the transition zone roughly parallel to the original bedrock contour trends.

The deteriorated zones were repaired on an area-specific basis. In each area of distress, obviously softened fill was removed, and the exposed surface was proofrolled with unloaded trucks. The subsurface was probed to determine the total thickness of rock–soil fill. In small pockets or deteriorated areas less than 2 ft (60 cm) deep, the softened fill was removed and replaced with compacted open-graded crushed stone (Kentucky grade No. 2 stone). Where the softened area was more than 2 ft (60 cm) thick, the deteriorated fill was removed, the surface was proofrolled, a layer of geotextile was placed on nonsoftened fill, and the area was filled to grade with compacted No. 2 stone. The backfill was compacted with a drum roller without vibration. In a second building where a small area of deterioration was found, a similar repair technique was used, except that a well-graded stone (Kentucky No. 57 grade) was used. All areas of repair showed no further deterioration prior to floor slab construction.

Drilled Pier Construction

More than 890 drilled piers were installed at this site, with diameters ranging from 3–9 ft (0.9–2.7 m). No difficulties were experienced during pier installation. For lightly loaded columns around the perimeter of the plant, only visual inspection of bedrock was done by inspectors lowered inside a temporary casing. For all other pier locations, probe holes were drilled as planned, and the sides of the probe holes were probed for cavities with a feeler gage.

Summary and Conclusions

The experience gained during all phases of this project provided valuable insights on the use of rock-soil mixtures as compacted fill:

1. The design mix gradation of crushed rock and site residual soil provided a fill with high strength and low compressibility, but which could be penetrated easily by drilling equipment during pier construction. The fill possessed sufficient cohesion that pier construction could be completed without the use of casings.
2. Plate load tests on the rock-soil fill indicated modulus of elasticity values that ranged from 8,600–13,300 psi (59–92 MPa), reflecting variations in crushed rock properties. These variations, documented with field density and gradation tests, also caused significant variations in field CBR test values.
3. Settlement monitoring programs showed that the rock-soil fill had variable but very low compressibility. Residual soils on-site were found to be very stiff also and to compress much more rapidly than had been indicated by laboratory consolidation tests.
4. Construction of test fills at least once each week was found to be very effective in demonstrating the compaction characteristics of the rock-soil fill, which changed as the properties of the bedrock varied during excavation in different portions of the site.

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Hot-mix asphalt stabilized railroad trackbeds

by

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Abstract: This paper documents a research project which has been ongoing since 1981 at the University of Kentucky in cooperation with several railroad companies. Numerous trackbeds containing a stabilizing layer of dense-graded hot-mix asphalt (HMA) have been constructed for experimental evaluations and the solution of soft roadway and pumping problems. The overall purposes are to develop a structural design procedure and to evaluate the applicability, performance, and long-term economic benefits of the system under widely varying traffic, roadbed, and environmental conditions.

INTRODUCTION

The use of Hot Mix Asphalt (HMA) as an integral railroad trackbed support material has received considerable attention in recent years. HMA has been applied under widely varying traffic, roadbed and environmental conditions in numerous experimental (test) trackbeds and has solved specific instability problems, including the rehabilitation of turnouts, crossings, bridge approaches, hump tracks, tunnel floors, and highway crossings (Rose, 1986; Rose, Huang, and Drnevich, 1986).

The most common method of incorporating HMA in trackbeds is known as the underlayment system as depicted in Figure 1. It consists of a layer (mat) of hot-mix asphalt, similar to that used for a highway base, which is placed directly on new subgrade or old roadbed. Mats ranging typically from 4 to 8 in. (100 to 200 mm) in thickness and 11 to 12 ft (3.3 to 3.6 m) in width have been used. A layer of ballast, typically 5 to 10 in. (125 to 250 mm) in thickness, is placed between the top of the HMA and below the ties. Variations in the surface smoothness of the HMA and ties can be compensated for with ballast.

This system represents little change from normal track construction and maintenance practices since the HMA layer merely serves as a subballast. Conventional surfacing equipment can be used for routine resurfacing and aligning activities to adjust geometry.

Utilization of HMA in trackbeds is not an entirely new concept. HMA has been used in other countries, most notably by the Italian Railways where several hundred kilometers are in service on several lines including the Rome to Florence high-speed line. Several significant

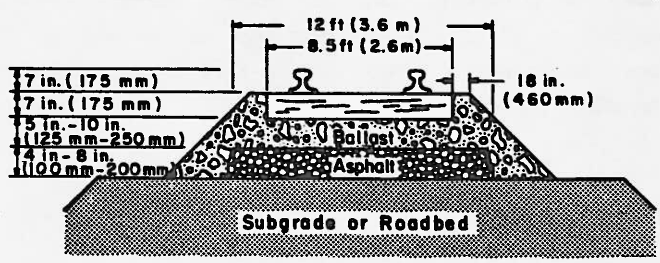

Figure 1. Typical sectional view of HMA underlayment trackbed.

installations were made in the U.S. during the 1960's and 1970's on the Cleveland Transit System, Sante Fe Railway, Southern Pacific Railroad and elsewhere. Recent inspections and evaluations indicate these sections are performing quite satisfactorily.

Renewed interest in the use of HMA for railroad trackbeds began in 1981. Since then over fifty projects have been built in Kentucky, Oklahoma, Kansas, and several other states. CSX Transportation (and predecessor companies), the Santa Fe Railway and the Union Pacific System represent railroad companies which have been primarily involved with HMA applications, although several other companies also have been involved.

All of the HMA test tracks and specific problem solving installations are performing extremely well. The increased cost of using HMA is most often minimal and at many sites indications are that long-term savings may be substantial when compared with conventional maintenance and rehabilitation techniques. Additional perfection and optimization of the field construction procedures represent activities of continuing interest.

The primary benefits of the HMA layer are to improve the load-distribution to the subgrade, waterproof the subgrade, and confine the ballast, thus providing consistent load-carrying capability for a trackbed even on subgrades of marginal quality. The waterproofing effects are particularly important since the impermeable HMA mat essentially eliminates subgrade moisture fluctuations, thereby improving and maintaining underlying support. Additionally, the resilient HMA mat provides a positive separation of ballast from the subgrade, thus eliminating subgrade pumping without substantially increasing the stiffness of the trackbed. The resultant stable trackbed has the potential of providing increased operating efficiency and decreased maintenance costs which should result in long-term economic benefits for the railroad industry.

STRUCTURAL ANALYSIS AND DESIGN

Computer Model

A computer model named KENTRACK (Huang, et al., 1984; Huang, et al., 1986; Huang and Rose, 1987) was developed for the structural analysis and design of railroad tracks. Figure 2 shows the various components of the track system to be analyzed. From top to bottom, the track is composed of rails, tie plates, ties, and the layered system. The finite element method was used in the analysis; the rails and ties were considered beam elements and the tie plates spring elements. The load is transmitted from the ties to the to the layered system through circular areas of equal diameter, thus Burmister's layered theory, so well known in the analysis of highway pavements, can be applied. By simply changing the properties of the layers, the program can be used for conventional ballasted tracks as well as hot-mix asphalt trackbeds. Major responses computed by the program include the deflections of rail, the bending and shear stresses in rail and tie, the contact pressure between tie and layered system, the horizontal tensile stress and strain at the bottom of hot-mix asphalt, and the vertical compressive stress and strain on the top of the subgrade. The rails, tie plates, ties, hot-mix asphalt, and subgrade are assumed to be linear elastic and the ballast and other granular layers as nonlinear elastic with an elastic modulus determined by

$$E_g = K_1\theta^{K_2} \tag{1}$$

in which E_g = elastic modulus of ballast or other granular materials, θ = sum of the three

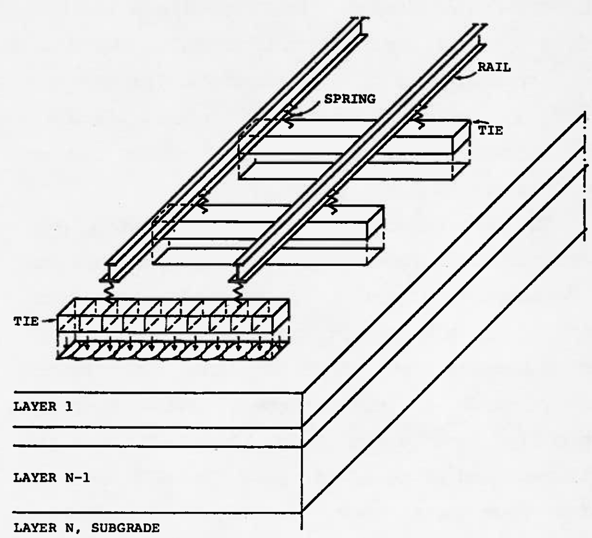

Figure 2. Modeling of railroad trackbed.

principal stresses due to loading and overburden, and K_1, K_2 = nonlinear coefficients.

By using the critical stress or strain in the trackbed, damage analysis can be performed by Miner's hypothesis (Miner, 1945). Because the elastic moduli of a layered system, particularly the hot-mix asphalt, vary with the time of the year, each year can be divided into 12 months or a number of seasons for damage analysis. Damage, computed on a monthly or seasonal basis for given traffic and material properties, is accumulated up to a damage ratio of 1.0 using the concept of linear summation of cycle ratios. Two criteria, similar to those used for highway pavements, are proposed for damage analysis: limiting the horizontal tensile strain at the bottom of hot-mix asphalt to prevent fatigue cracking and limiting the vertical compressive stress on the top of the subgrade to reduce permanent deformation. The reason for using the vertical compressive stress, instead of compressive strain, as is done in standard highway practice, is because the compressive stress is a much better and more sensitive indicator of structural adequacy, particularly for railroad trackbeds (Huang, Lin and Rose, 1984; Huang, Rose and Khoury, 1986).

The purpose of damage analysis is to determine design life. In highway pavements there is only one design life because the hot-mix asphalt must be replaced or overlaid whenever the design life for fatigue cracking or permanent deformation is reached. In railroad trackbeds there are two design lives, one involving the fatigue cracking of hot-mix asphalt and the other involving the permanent deformation of the track. The required design life for fatigue cracking should be much greater than that for permanent deformation because the latter can be corrected by adjusting the ballast. When permanent deformation has been corrected, the track is considered new and requires further correction only when the new design life for permanent deformation is reached.

Design Factors

Similarly to highway pavements, the design factors include traffic characteristics, material properties and environmental effects.

Traffic characteristics. The traffic information to be used for thickness design includes the magnitude of wheel loads and the number of repetitions per year. The load for the conventional freight trains consists of two 33,000-lb (147-kN) wheel spaced at 70 in. (1.78 m) on centers. Theoretically, there are four wheels in a group on each side: two at the back of one car and two at the front of the adjoining car. Because the critical stress and strain under the two wheels in one car are greater than those under the four wheels in two adjoining cars, only two wheels are used for design purpose.

In determining the number of load repetitions for freight trains, the passage of the group of four wheels is considered as one repetition because the two wheels at the back of one car are so close to the two wheels at the front of the adjoining car so these four wheels are considered as a unit. An approximate method is to count the passage of each car in a train as one repetition. For transit vehicles, where the two wheels in one car are quite far away from those in the adjoining car, the passage of one car should be counted as two repetitions.

Train traffic is generally expressed in terms of million gross tons per year, including both loaded and unloaded cars. For loaded trains with a standard wheel load of 33,000 lb (147 kN), each car weighs 132 tons (120 tonnes), so each load repetition is equivalent to 132 tons (120 tonnes). Because an unloaded car weighs only 25 percent of a loaded car and has practically no effect on thickness design, it should not be considered as a part of the train traffic. Since the unloaded car is included in the gross tonnage, one repetition of a loaded car should be equivalent to 160 gross tons (145 tonnes).

Material properties. The material properties include the nonlinear coefficients, K, and K_2 of each granular layer, as indicated by Eq. 1, the elastic or resilient modulus of subgrade, and the failure criteria for fatigue cracking and permanent deformation. Other properties of minor importance such as the Poisson's ratio of each layer, the lateral stress ratio, K_o, of each nonlinear layer, and the unit weight of each track component can be reasonably assumed.

The nonlinear coefficients of granular materials can be obtained from the repeated load triaxial tests. For the conventional ballast, it is reasonable to assume K_1 = 7500 psi and K_2 = 0.5. The elastic modulus of subgrade can be related to the California bearing ratio by (Heukelom and Foster, 1960)

$$E_s = 1500 \text{ CBR} \qquad (2)$$

in which E_s = subgrade modulus in psi and CBR = California bearing ratio of the saturated subgrade.

Based on the highway experience, the following two equations are proposed as failure criteria (Huang, et al., 1984):

$$N = 0.0795 \epsilon^{-3.291} E^{-0.853} \qquad (3)$$

$$N_d = 4.837 \times 10^{-5} \sigma_c^{-3.734} E_s^{3.583} \qquad (4)$$

in which N_c = allowable number of load repetitions to cause fatigue cracking, ϵ_t = horizontal tensile strain at the bottom of HMA, E_a = elastic modulus of HMA in psi, N_d = allowable number of load repetitions to cause excessive deformation, σ_c = vertical compressive stress on the top of subgrade in psi, and E_s = subgrade modulus in psi. It is true that, due to the differences in loading conditions, environmental effects, and performance requirements, the failure criteria for highway pavements may not be applicable to railroad trackbeds. Given the lack of railroad data, highway criteria can only be used as a guide and should be revised as more experience is gained. As will be indicated later, the use of the above criteria results a reasonable thickness commensurate with past experience.

Environmental effects. Due to the presence of a thick layer of ballast above the asphalt layer, the hot-mix asphalt and subgrade in railroad trackbeds are less affected by the environments as compared to highway pavements. The most important effect is the temperature of the HMA, which affects its elastic modulus. The effect of freezing and thawing and moisture change in the subgrade can be considered by selecting an appropriate elastic modulus for the subgrade.

Design Guides

The thickness of ballast and HMA depends on train traffic, subgrade support and climatic region. Table 1 shows the thickness required, as determined from KENTRACK. In the analysis, the following standard conditions are assumed: wheel load 33,000 lb (147 kN); rail 132 RE; wood ties 20 in. (508 mm) on centers; tie plate stiffness $7x10^6$ lb/in. (1.2 GN/m); and nonlinear coefficient of ballast, K_1, 7,500 psi (52 MN/m^2). The design life is 30 years for fatigue cracking and 5 years for permanent deformation. Design charts for nonstandard conditions, which require simple calculations and interpolations, are available (Huang, Rose and Khoury, 1987).

The train traffic is divided into four categories: light for 8 million gross tons (mgt) or 50,000 repetitions per year, medium light for 16 mgt or 100,000 repetitions per year, medium heavy for 32 mgt or 200,000 repetitions per year, and heavy for 48 mgt or 300,000 repetitions per year. The moduli of hot mix asphalt for cold, moderate and hot climate are assumed 1,000,000, 500,000 and 200,000 psi (6.9,

Table 1. Thicknesses of Hot Mix Asphalt Trackbeds

		Traffic			
Subgrade	Climate	Light	Medium Light	Medium Heavy	Heavy
Excellent CBR = 20	Ha(in.)	3	3	3	3
	Hb(in.) Cold	5	5	5	5
	Moderate	5	5	6	7
	Hot	5	5	6	8
Good CBR = 10	Ha(in)	4	4	4	4
	Hb(in.) Cold	5	5	6	8
	Moderate	5	5	7	9
	Hot	5	6	9	12
Fair CBR = 5	Ha(in.)	4	4	4	4
	Hb(in.) Cold	5	7	11	15
	Moderate	5	8	14	18
	Hot	6	11	20	27
Poor CBR = 2	Ha(in.)	6	6	6	6
	Hb(in.) Cold	15	22	-	-
	Moderate	17	24	-	-
	Hot	21	-	-	-

Note: Ha = thickness of hot mix asphalt; Hb = thickness of ballast; - ballast more than 30 in. and sub-grade should be improved; 1 in. = 25.4 mm

3.5 and 1.4 GN/m^2) respectively. The subgrade is divided into four categories with CBR values of 20, 10, 5 and 2, which correspond to subgrade modulus of 30,000, 15,000, 7,500 and 3,000 psi (207, 104, 52 and 21 MN/m^2).

Previous studies (Huang, Rose and Lin, 1984) on HMA trackbeds have demonstrated that the most effective way to reduce the horizontal tensile strain at the bottom of asphalt mat is to increase the thickness of ballast, rather than HMA. Therefore in the design of HMA trackbeds, a minimum thickness of HMA is selected and the ballast thickness is designed to satisfy both fatigue and deformation criteria. The minimum thickness of HMA depends on the subgrade support. A poor subgrade requires a thicker asphalt mat, so it can be properly compacted. The minimum thickness is 3 in. (76 mm) for excellent subgrade, 4 in. (102 mm) for good or fair subgrade, and 6 in. (152 mm) for poor subgrade. The use of thicker asphalt mat can cause stress concentration and may not increase the fatigue life.

A review of Table 1 indicates that the thickness obtained by KENTRACK are reasonable and commensurate with past experience. However, when the subgrade is poor, a very large ballast thickness is required. It may be more economical to improve the subgrade by laying a thick layer of subballast before the asphalt mat is placed. The table also shows that climate, or the modulus of HMA, has a small effect on thickness design when the subgrade is excellent or the traffic is light, but the effect increases as the subgrade support decreases as the traffic increases. The thickness increases as the climate becomes warmer.

Comparison with Highway Design Concept

The structural analysis and design of HMA trackbeds as described above are quite similar to those of asphalt highway pavements. Both consider traffic, materials and environments as the major design factors and apply the Burmister's layered theory and the Miner's damage hypothesis as a design tool. However, there is a major difference between a highway pavement and a railroad trackbed, namely the distribution of wheel loads to the layered system. On highway pavements, wheel loads are applied over small areas and the magnitude of loads on each area is a constant independent of the stiffness of the layered system. Whereas, on railroad trackbeds, wheel loads are distributed through rails and ties over a large area and the maximum load applied on one tie depends strongly on the stiffness of the layered system. Therefore, the use of thicker HMA for highway pavements is very effective in reducing both the tensile strain at the bottom of HMA and the compressive stress or strain at the top of subgrade but not very effective for railroad trackbeds. In fact, for a given combined thickness of ballast and HMA, the tensile strain increases as the thickness of HMA increases, which indicates that the use of ballast is more effective than the use of HMA in reducing tensile strains. That the replacement of ballast by HMA increases the tensile strain is due to the load concentration as indicated by the tremendous increase in contact pressure near the wheel load caused by the stiffer trackbed. For the same reason, the use of full-depth construction, which is so popular for highway pavements, is not recommended for railroad trackbeds.

CONSTRUCTION AND PERFORMANCE

Mixture Design

Table 2 contains the suggested composition ranges for the HMA mixture developed specifically for trackbed applications. The asphalt mix is a slightly modified ASTM D3515 (ASTM, 1988) dense-graded highway base mix

Table 2
Composition of Dense-Graded Hot Mix Asphalt Trackbed Paving Mix

Sieve Size	Amount Finer, Weight %*
1.5 in. (37.5 mm)	100
1.0 in. (25.0 mm)	90-100
0.5 in. (12.5 mm)	56-80
No. 4	29-59
No. 8	19-45
No. 50	5-17
No. 200	1-7**
Asphalt Cement (weight as a percentage of total mixture)	3-9**

* ASTM D3515 for medium traffic conditions.
** Selected to provide 1% to 3% air void content in the compacted mix and a minimum VMA (Voids in Mineral Aggregate) of 12%.

incorporating conventional mineral aggregates, 1 to 1.5 in. (25.0 to 37.5 mm) maximum size, and AC-10 or AC-20 viscosity-graded asphalt cement. It is proportioned slightly different from typical highway dense-graded base mixes and contains a little more asphalt cement, more mineral aggregate fines, and lower air voids (1 to 3%) after compaction. This type of mix will undergo minimal oxidation (hardening) and flushing of the asphalt in a trackbed because of the insulative value of ballast. The mix provides a stable, impermeable medium capable of carrying loads with minimum deflection but with sufficient resiliency to withstand the dynamic forces and prevent pumping and loss of track geometry. The low void and high asphalt cement content ensure a long fatigue life for the asphalt mat.

Installation Practices

Most of the HMA trackbeds placed to date have utilized conventional highway paving construction techniques. For existing trackbeds, the track first must be removed and the underlying material excavated to the desired grade. The asphalt mix is hauled by dump truck from a HMA facility and is either spread using a standard highway asphalt paver or backdumped from the trucks and spread with a dozer blade (only utilized for short sections). The HMA is normally placed in 4-in. (100-mm) lifts, although lifts 6 in. (150 mm) in thickness can be adequately compacted. Compaction is achieved with a standard roller, preferably a steel-wheel vibratory type. It is desirable to obtain a well-compacted mat with minimum air voids.

Immediately after compacting the HMA mat, the track is rebuilt or dragged back on the mat using rubber-tired equipment. After the rails are joined, the ballast is distributed using conventional unloading and spreading equipment. The track is pulled vertically to provide the specified ballast thickness below the ties, typically 5 to 10 in. (125 to 250 mm). Either No. 24 or No. 4 size ballast is generally used. The ballast also fills the crib areas between the ties and provides a 1 to 1.5 ft. (0.30 to 0.45 m) wide shoulder.

Cranes can be used to lift crossing panels, turn-outs, and crossovers. Snaking techniques are applicable for longer sections of track. Adequate space must be available to facilitate removal or replacement of the track and provide access for the HMA paving operation.

A HMA underlayment was successfully placed in 1986 under a raised track without removing the rail and ties on the Santa Fe Railway's mainline near Cassoday, Kansas (Hensley, 1987). The first step involved a single-pass 14-in. (350-mm) undercutting of the trackbed. Three track slewers were used to elevate the track structure 30 in. (750 mm) above the newly cut roadbed. The HMA was delivered by dumptrucks to a modified road widener positioned on the adjacent service road. A system of double augers distributed the HMA under the raised track to a strike-off blade and screed. Following was a plate compactor to densify the 6 in. (150 mm) thick, 850 ft. (260 m) long HMA mat. The track was immediately lowered onto the HMA to permit uninterrupted train traffic. Subsequently 8 in. (200 m) of ballast was added.

Application Considerations

New line construction and passing track extensions represent ideal conditions since the exposed subgrade is available for placing the HMA mat with conventional paving and spreading equipment prior to placing the ties and rails. Present construction procedures are not applicable for paving long sections of in-service single track because sufficient track time is normally not available for removing the track, excavating, paving, and rebuilding the track. Further modifications of equipment for placing HMA under the track without removing the track in conjunction with an undercutting or sledding operation would greatly decrease required track time.

Removing the track and paving short sections of in-service track, turnouts, crossovers, bridge and tunnel approaches, and crossings exhibiting poor soil, bad drainage, or subgrade pumping conditions can be accomplished with minor disruption to traffic. The use of an HMA base under highway crossings can provide a very economical means of obtaining adequate support for the combined highway and railroad loadings, thus reducing costly repairs to crossing surfaces. HMA should prove most advantageous in these areas.

Rapid transit and high speed passenger lines require substantial track structures to maintain accurate track geometry. The use of HMA in these track structures is very appropriate.

HMA is equally adaptable to the construction of intermodal yards. Heavy trucks and unloading equipment require substantial structural sections. Of particular advantage are the waterproofing characteristics of the HMA and the positive drainage systems which can be incorporated in the design of the unloading area.

Quantity and Cost Determinations

The cost of obtaining and placing HMA in a trackbed will vary depending on the following factors: the cost of the aggregates and asphalt cement in the local area, the length (time) of haul from the manufacturing facility to the site, the size (tonnage) of the project, the availability and cooperation of local contractors, and the ease of delivery access and construction maneuverability.

The prices typically quoted by HMA paving contractors represent in-place cost and include the cost of materials, mixing at a facility, hauling to the job site, and placing and compacting the mix to the specific dimensions. For example, a \$30 per ton in-place price (\$33 per tonne) could be representative of a fairly large tonnage project for average conditions.

In-place HMA prices for small installations could be considerably higher due to the fixed cost of dispatching the paving crew and equipment to the field. In instances where the HMA can be satisfactorily backdumped and spread with dozer-type equipment (generally already on site), a 30 percent or more reduction in the cost of the HMA could be expected.

HMA meeting the requirements of Table 2 and containing normal weight aggregates will compact to a nominal density of about 140 lb/ft^3 (2240 kg/m^3). Table 3 presents tons of HMA per track foot and per square foot for various thicknesses of the mat, assuming a 12-ft (3.6 m) wide mat having a nominal density.

For example, consider HMA compacted to a density of 140 lb/ft^3 (2240 kg/m^3) and placed 4 in. (100 mm) thick and 12 ft (3.6 m) wide. This requires 0.28 tons (255 kg) of mix per track foot. Assuming the in-place cost is \$30 per ton (\$33 per tonne), the cost would be \$8.40 per track foot (\$27.50 per track meter), or \$6.30 per square yard (\$7.50 per square meter) for the 4 in. (100 mm) lift.

Table 3. HMA Quantity Determinations

Mat Thickness (in.)	Tons of Mix* Per Track Foot	Tons of Mix* Per Square Yard
3	0.21	0.16
4	0.28	0.21
5	0.35	0.26
6	0.42	0.32
7	0.49	0.37
8	0.56	0.42

*Based on a HMA compacted density of 140 lb/ft^3 (2240 kg/m^3) and a mat width of 12 ft. (3.6 m).
1ton/track foot = 3.61 tonne/track meter
1ton/$ft.^2$ = 11.84 tonne/m^2

When comparing costs of constructing an HMA trackbed versus that of a conventional ballast/subballast/geosynthetic trackbed, the cost of the HMA can be partially or totally offset by the elimination of geosynthetics and the replacement of all the subballast and a portion of the ballast with a thinner HMA.

Observed Performances

Several of the HMA trackbed projects have been subjected to periodic instrumented tests and measurements. Adjacent control sections employing conventional ballasted track have been evaluated for comparison purposes. The performances of all the HMA sections have been excellent. Following is a summary of the various tests and measurements.

Trackbed moisture. Extensive studies were made at two sites to evaluate the waterproofing characteristics of HMA trackbeds. Samples of subgrade and old roadbed immediately underlying the HMA mat were obtained after removal of HMA cores. The results indicate that a HMA layer overlying either a fine-grained compacted soil or a granular mixture of old roadbed materials will maintain a uniform, low and near optimum moisture level in the underlying material even after several years.

This waterproofing and membrane effect will provide consistent load-carrying capability from the underlying material while preventing intrusion of subgrade into the ballast and subsequent fouling and pumping. These factors are considered to be primary benefits of HMA trackbeds.

Long-term track settlement. Top-of-rail elevations were established at five test installations soon after construction. Elevation changes along the test sections have been periodically measured using conventional leveling techniques. No significant changes in elevation have occurred, even after several years of service.

In new track construction on fills and embankments, some deep seated settlements are likely to occur regardless of whether or not HMA is used. However, if infiltration of surface water is a contributing cause of the settlement, then obviously a HMA layer will reduce this settlement due to its waterproofing characteristics.

Asphalt mix durability. The insulative effects of ballast can attenuate temperature fluctuations in the HMA layer. It also reduces the ultraviolet and oxidation-induced hardening of the HMA normally evident in highway pavements. The HMA retains resilient characteristics for a long period of time and there is a significantly reduced tendency for the HMA mat to become brittle and crack.

In typical highway applications, HMA undergoes large, temperature-induced changes in volume and stiffness during the year. Since the HMA in a trackbed is not exposed to ambient temperature, the range in winter and summer temperatures is reduced; thus the volume and stiffness of the HMA will remain more nearly constant throughout the year. This markedly increases the fatigue life of the HMA layer.

The hardening characteristics of HMA layers in insulated trackbed environments have been evaluated from two installations which were built during the late 1960's and from two recent installations built during the early 1980's. The asphalt cement was extracted and recovered from the cores. Viscosity and penetration tests were conducted to determine the hardness of the aged asphalt. Dynamic modulus tests were conducted on the extracted cores.

For the two 1980 installations, samples of the asphalt mix has been similarly evaluated during the construction phase. A comparison of test values indicates that minimal hardening of the asphalt cement and no deterioration of the mixtures have occurred. Similar test results were obtained from the two earlier installations. Apparently only minimal hardening has occurred and no deterioration of the mixtures is evident after 15 years of service.

Track stiffness. The stiffness of the HMA trackbeds under static loading conditions was evaluated using the track modulus obtained from beam on elastic foundation principles. The average track modulus value for the HMA underlayment trackbeds was 2500 lb/in./in. (17 N/mm/mm) and the average rail deflection was 0.18 in. (4.6 mm) under a loaded 100-tons (91-tonne) car on heavy rail and wood ties. These values are within the desirable range for wood tie and heavy rail track to provide the optimum track stiffness and flexibility. Little variation in modulus values for different HMA thicknesses were noted. Further, modulus values for the HMA sections have remained essentially constant. It is anticipated that HMA trackbeds will maintain an optimum stiffness level for a longer period of time and be less affected by variations in rainfall, water-table level, etc., than the typical ballast track.

Track geometry. Maintaining accurate track geometry is essential for safe and efficient train operations. Mainline, heavy tonnage HMA trackbed sections have had track geometry tests conducted periodically. No detectable changes in geometry have occurred.

CLOSURE

This paper documents current research and field studies aimed at developing design and field construction techniques for a trackbed system containing a layer of HMA paving mix. The applicability and performance of some forty HMA trackbed sections constructed during the past twenty years have been evaluated under widely varying traffic, roadbed, and environmental conditions.

The HMA trackbed system is applicable for constructing new trackbeds and for rehabilitating short sections of in-service trackbeds and special trackworks. Currently conventional HMA placement and spreading techniques are used. Further modification of highway paving equipment or mix composition to expedite the placement of the mix under a raised track will be required before its widespread use for rehabilitating long sections of in-service single track.

To design hot-mix asphalt trackbeds on a rational basis for specific design lives, a computer program named KENTRACK was developed. Design is based on two failure criteria; the horizontal tensile strain at the bottom of hot-mix asphalt, and the vertical compressive stress on the top of the subgrade. In the absence of experience (failures) on railroad applications, the highway failure criteria were used as a guide. These criteria should be updated as more experience is gained through their use.

Performance data indicate that the HMA layer improves load-distributing characteristics, provides proper ballast restraint and reduces subgrade (roadbed) moisture fluctuations relative to conventional ballasted trackbeds. No detectable changes in track geometric parameters or track settlements have occurred. Track deflection data confirm that asphalt trackbeds provide optimum flexibility and stiffness which should continue for a long period of time since no changes occur in the composition of the HMA in the insulated trackbed environment.

Construction costs for new trackbeds and for complete rehabilitation of short sections of existing trackbeds with HMA underlayments compare favorably with conventional subballast/ballast/geosynthetic systems. The use of HMA trackbeds has not been sufficiently widespread to provide conclusive documentation of long-term trackbed maintenance and operating cost reductions; however, no trackbed maintenance has been required and the serviceability remains excellent for all the HMA trackbeds under study. The use of a HMA trackbed system seems to offer the railroad and transit industries a viable option for achieving decreased trackbed maintenance costs and improved train operating efficiency.

ACKNOWLEDGEMENTS

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Roller-mounted compaction meters - principles, field tests, and practical experiences

by

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Abstract: Roller-mounted electronic compaction meters have been commercially available for about 10 years. Some different design principles have been used, but the models available up to now indicate a relative value of the bearing capacity of the ground.

This has been confirmed by a great number of field studies mainly carried out in Sweden and West Germany. On free-draining coarse-grained soils good relationships exist between bearing capacity and density.

On fine-grained soils the bearing capacity is to a high degree dependent on the water content, and compaction meters offer a unique opportunity to find weak spots which with very few exceptions are related to areas of fine-grained wet soil. On such types of soil there is, however, no direct relationship between bearing capacity and density which limits the usefulness of the compaction meter for a direct compaction control.

Very successful applications of compaction control with the help of compaction meters have been reported from West Germany. At some very large earth filling operations; at the construction of a new airport in Munich for example, the compaction meter registrations have been transmitted by radio from the roller to a field laboratory to obtain instantaneous values.

In many other cases the roller-mounted compaction meter has been a great help in the monotonous work of the roller operator to find weak areas and not least to avoid "overcompaction" leading to unnecessary wear of the roller.

During recent years roller-mounted documentation systems connected to the compaction meter have been developed. The systems display an overview of the compacted area on a screen which shows the bearing capacity of different sections.

INTRODUCTION

Since the rapid breakthrough of self-propelled vibratory rollers during the 70's, an on the whole international praxis has been established for methods and equipment used for soil and rock-fill compaction. For this application the self-propelled vibratory roller is today the dominating roller type. Self-propelled, high-speed tamping rollers represent, however, an economical and common alternative for large scale clay compaction jobs.

In contrast to the wide use of vibratory rollers for soil compaction, static smooth drum rollers, pneumatic tired rollers and vibratory rollers all keep substantial segments within the field of asphalt compaction.

An international praxis also exists for compaction control. For important soil compaction work, end-result specifications dominate. Density is very often checked with the help of the manually performed sand-replacement method. Nuclear gauges represent a more accurate and particularly a more rapid alternative. This method has, however, certain limitations.

The need for more advanced, rapid and economical methods for compaction control has, during recent years, been emphasized as the most urgent subject for further development within the compaction technology. Methods have been requested for indicating the strength and bearing properties of the compacted material better than those used at present.

ROLLER-MOUNTED COMPACTION METERS (1),(2),(3),(4)

The idea of equipping a vibratory roller with a sensor, where the registrations are related to the properties of the ground on which the vibrating drum is operating, is not new. A unit where the frame vibrations were measured was introduced in the USA by Tampo around 1960. The results were not very reliable and the unit soon disappeared from the market.

A very comprehensive study of different alternatives for roller-mounted compaction meters was started in Sweden in 1972 by the consulting firm Geodynamik in close cooperation with Dynapac. The study resulted in 1978 in the introduction of the Compactometer, described below.

During the 1980's other European roller manufacturers have introduced roller-mounted compaction meters based on other principles than the Compactometer. Among these the Terrameter developed by Bomag is best known.

Compactometer (Geodynamik/Dynapac)

The principle for the roller-mounted compaction meter called Compactometer is shown in Figure 1.

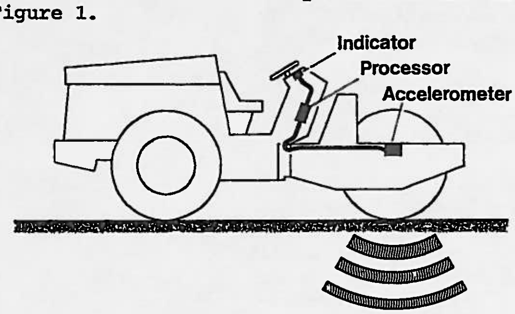


Figure 1. Compactometer mounted on vibratory roller

An accelerometer, rigidly mounted on the vibrating drum of the roller, continuously registers the vertical dynamic force which is developed when the drum works on the ground surface. The signals are treated in a special processor which continuously calculates a compaction meter value, called CMV which can be read on an instrument on the operator's panel, Figure 2.

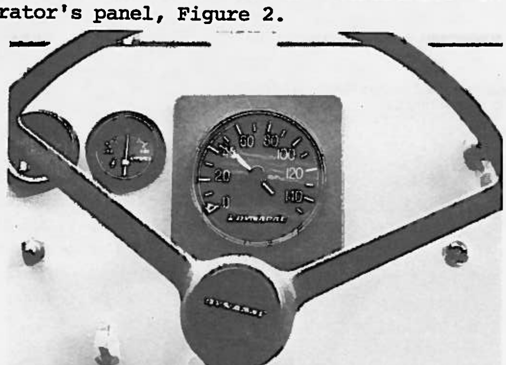


Figure 2. Indicator on the operator's panel

The processor evaluates by means of a filter system the difference between the regular sinus wave which is obtained in an uncompacted soft soil material and the more or less deformed sinus oscillation which is obtained on a compacted, more rigid material, Figure 3.

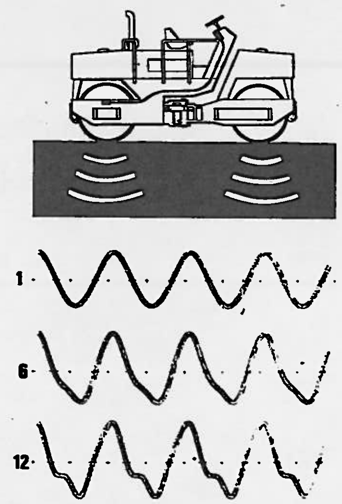


Figure 3. Acceleration for the vibrating drum during roller pass no. 1, 6 and 12.

The extent to which the regular sinus curve is deformed is proportional to the amount of overtones of the signals registered by the accelerometer and evaluated by the processor. Thus the compaction meter value represents a relative measure of the rigidity or bearing capacity of the ground.

For measure-technical reasons the CMV value is given as a relative, dimensionless value A_1/A_o based on the drum vibrations where:

A_1 = amplitude of the first overtone
A_o = regular amplitude (ground tone)

The processor is adjustable in steps with reference to the nominal frequency of the roller. To obtain reproducible CMV-values it is important that the frequency and roller speed are kept constant from pass to pass.

Terrameter (Bomag) (4)

The Terrameter is based on a different principle than the Compactometer. The Terrameter registers the dynamic force which is acting on the drum. This measurement is made with two accelerometers placed at an angle of 90^o. In this way both the magnitude and the momentary direction of the dynamic force is registered. In a micro-computer, the dynamic force between drum and ground is calculated as a function of the measured force and the centrifugal force generated by the rotating eccentric weight in the drum. The Terrameter also indicates if an increase of the value is obtained in two subsequent passes over the same compacted lane.

The dynamic force between the drum and the surface should logically be a function of the rigidity or bearing capacity of the ground. Comparative field tests between the Compactometer and the Terrameter have also shown good compatibility. The scales are however different and a CMV value of, for example, 40 corresponds to a Terrameter value of about 400.

Other types of compaction meters (4)

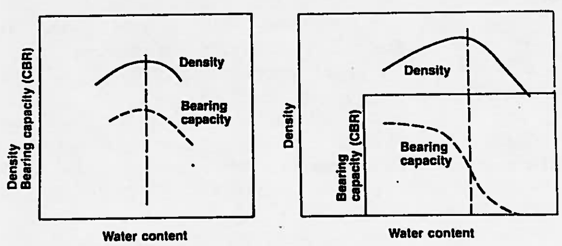
Case Vibromax introduced in 1981 a compaction meter where the acceleration of the vibrating drum was used as a reference value. The accuracy of this meter has, however, been less than for the Compactometer and the Terrameter.

Another type of compaction meter developed by ABG has not yet been described in detail in the available literature.
A new type of oscillating roller which transmits a solely horizontal vibration to the soil has been developed by Geodynamik and is now tested and evaluated primarily on Swedish job sites. The oscillating roller can be equipped with a special type of compaction meter called oscillometer.

PRINCIPAL DIFFERENCES BETWEEN DENSITY AND BEARING CAPACITY TESTS

It has already been stated that the Compactometer as well as the Terrameter indicate a relative value of the bearing capacity of the ground. A number of comparative tests between Compactometer measurements and static or dynamic load-bearing tests have also resulted in good correlations.

On coarse-grained permeable soils a good relation between density and bearing capacity exists, Figure 4.



a) Coarse-grained soils   b) Fine-grained soils

Figure 4. Principal relationships density/water content respectively bearing capacity/water content

On fine-grained soils the bearing capacity is to a high degree dependent on the water content. Increasing water content makes the material more and more soft and the bearing capacity decreases, see Figure 4. No direct correlation thus exists between density and bearing capacity.

On fine-grained soils the result of a determination of the bearing capacity, including a compaction meter test, is dependent to a high degree, on the water content of the soil. This will limit the usefulness of compaction meter tests on fine-grained soils. It should be noted, however, that even if the density cannot be directly checked, a knowledge of the bearing capacity can be of great value.

BASIC STUDIES (2),(3),(4),(5),(6),(7)

Several comprehensive studies of roller-mounted compaction meters including comparative tests with other control methods have been made, most of them in Sweden and West Germany.

Compaction meters show a high degree of reproducibility from pass to pass, Figure 5. Normally the values also show comparatively large variations over the tested area resulting in standard deviations of 10 to 15%, the same level as for static or dynamic load bearing tests. The standard deviation of a number of density tests normally falls below 5%.

Figure 5 also shows that the variations of the compaction meter values is still comparatively high at the end of the compaction procedure.

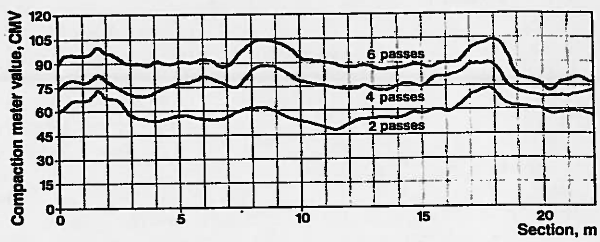


Figure 5. Example on recordings obtained on subbase with a Compactometer mounted on a 15-ton vibratory roller (6)

On coarse-grained, especially free-draining soils, good correlations have been obtained between compaction meter values and static as well as dynamic load bearing tests, Figures 6 and 7.

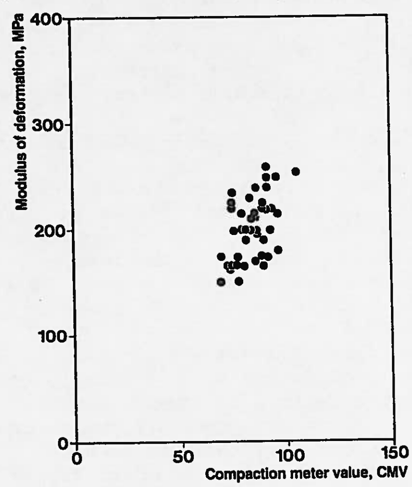


Figure 6. Relationship between bearing capacity (modulus of deformation) and compaction meter value on granular base course (6)

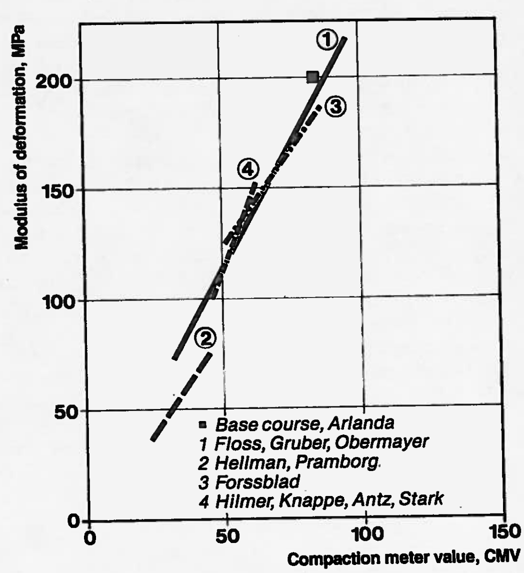


Figure 7. Compilation of relationships between modulus of deformation and compaction meter values (mean values) obtained in different studies on granular soils (6)

The dynamic load bearing tests have, in these cases, been performed with a falling weight deflectometer. Even if the spread of the individual values may be large (often larger than shown in Figure 6), an astonishingly good correlation between different studies were obtained, Figure 7. In the different studies different roller types and roller sizes have been applied and even if the soils have always been coarse-grained the gradations have varied. One explanation for this good correlation is the fact that the CMV-value is a dimensionless factor.

The following approximate relationship obtains: Modulus of deformation (obtained by static or dynamic load-bearing tests) ≅ 2.3 CMV.

On coarse-grained free-draining soils a good correlation exists between density and bearing capacity and consequently also between density and CMV.

The studies performed on fine-grained soils have confirmed the influence of the water content on the compaction meter value. At water contents above the optimum, bad correlations have been obtained between compaction meter values and density values. Further theoretical and practical studies within this area are very desirable.

LARGE-SCALE CONTROL PROJECTS (3),(4)

Comprehensive systematic studies of different types of roller-mounted compaction meters have been performed at the Institute of Soil and Rock Mechanics and Foundation Engineering, Technical University, Munich, West Germany. These studies have lead to a number of large-scale control projects.

The first large-scale project of this type was the construction of a foundation fill for a new BMW car factory in Regensburg, West Germany. The fill material replacing soils of inferior quality consisted of three million cubic meters of well-graded gravel, compacted in layers of 0.6 m (2 ft) using 10-ton self-propelled vibratory rollers.

After detailed pre-studies it was decided to mainly use compactometers for the compaction control. The compactometers were mounted on 10-ton vibratory rollers of the same type as were used for the soil compaction. The CMV-values were transmitted by radio to a field office where they were registered on diagrams and analyzed. The results were rapidly delivered to the contractor.

When a new international airport was to be built in Munich the same principles for compaction control were applied. Also here the embankment fills consisted of gravel which could have some silt content. The embankment fills were placed in layers of 0.6 m (2 ft) using 10-ton self-propelled vibratory rollers at maximum capacities of up to 30,000 cubic meters daily.

After compaction with four passes, two at high amplitude and two at low amplitude, a check was made with rollers of the same size equipped with compaction meters. As in Regensburg the compaction meter values were transmitted by radio to a field laboratory, Figure 8.

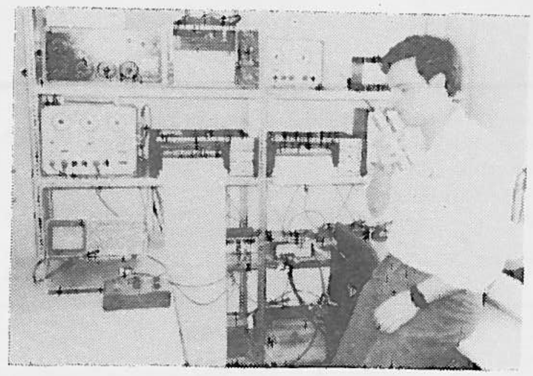


Figure 8. Radio transmission of compaction meter values to field laboratory, Munich airport

All values are stored in a computer system and thus documented for the future.

The contractor has expressed the opinion that the procedure for compaction control which was used, allowed the high working capacity.

New railway lines for high speed trains are being constructed in several countries. High requirements are put on the standard of the embankments.

For the construction of a new railway line Mannheim - Stuttgart in West Germany, vibratory rollers with Compactometers were used to check the uniformity and bearing capacity of the subgrade and the embankment fill.

The Wupper Dam in West Germany is a rock fill dam with an asphalt core. The rock fill compaction was checked with Terrameter measurements.

OTHER APPLICATIONS

Up to now, roller-mounted compaction meters have been used for regular compaction control only in a limited number of cases, some of them mentioned above.

In addition to this, roller-mounted compaction meters have, in a large number of cases, been used as a general aid to roller operators and field personnel to improve quality and economy. Weak sections are discovered, "overcompaction" with an unnecessarily high number of roller passes can be avoided.

A strong reason for the use of roller-mounted compaction meters is to make the monotonous work of the roller operator more interesting, constructive and efficient.

Up to now, about 1,000 Compactometers have been sold in different countries around the world.

PROOF-ROLLING

The method of checking the bearing capacity of a compacted subgrade or subbase using a heavy pneumatic tired roller with weights of 50 or 100 tons, was originally used in the USA. This method has also been applied in other parts of the world, often on airfield projects. With such a heavy roller, sections with low bearing capacity can be identified. The reasons are insufficient compaction and/or unsuitable materials.

A vibratory roller equipped with a compaction meter can, in the same way, identify weak sections.

A successful application was made at the construction of the railway line Paris - Lyons for high speed trains operating at speeds of up to 260 km/h.

Under the ballast bed a 200 - 500 mm subbase compacted to 100% Mod Proctor was placed above the embankment fill. On completion of the subbase the entire stretch of the line was checked with a 2-ton vibratory tandem roller equipped with a compaction meter. One example of the test results is shown in Figure 9.

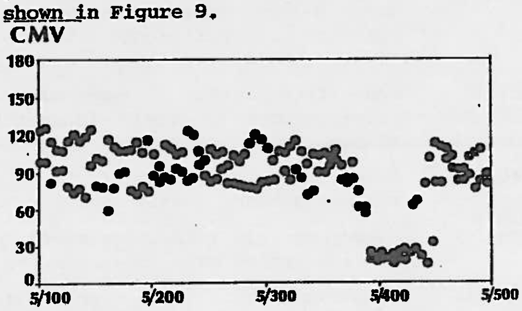


Figure 9. Example of control results on railway subbase (Paris - Lyons)

Figure 9 shows a section with CMV-values mainly on a high level, on which a weak area was found. Checks of such sections have always shown the presence of a high water content and/or surplus of fine-grained soil materials. Such unsuitable material has to be replaced.

A special measuring unit designed as a vibrating steel wheel equipped with a compactometer has also been designed and tested in Sweden, Figure 10.

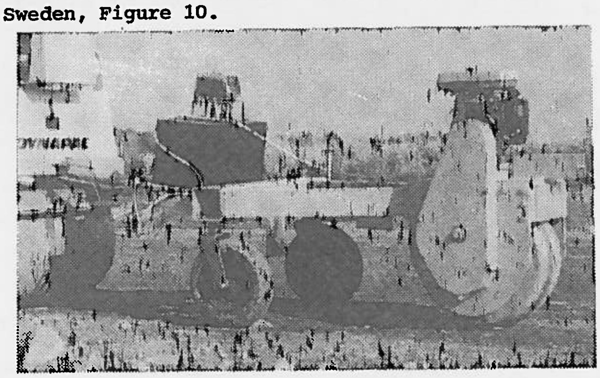


Figure 10. Measuring unit (vibrating steel wheel equipped with Compactometer)

NEW DOCUMENTATION SYSTEMS

Documentation of compaction meter values can be made with a strip chart recorder. Long paper strips with such registrations are in practice, however, difficult to handle and analyze.

New computerized roller-mounted documentation systems have, during the last years, been developed in Sweden. On a screen in front of the roller operator, compaction meter registrations are visualized in the form of numerical values and graphical presentations showing section with different CMV-values, Figure 11.

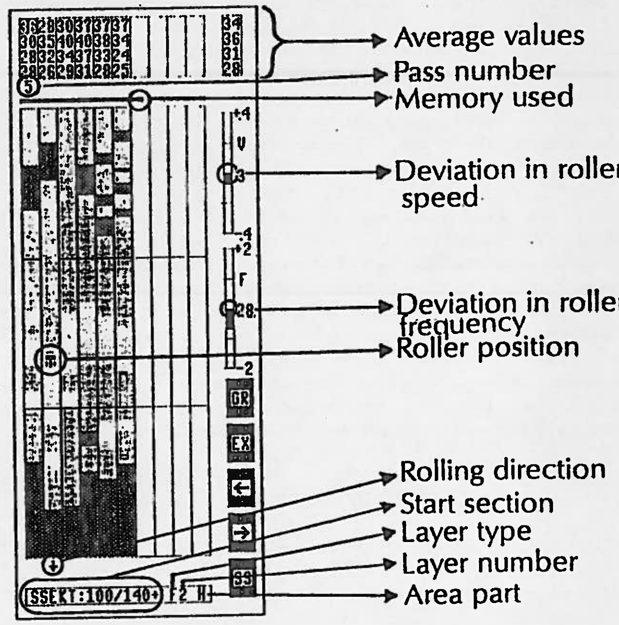


Figure 11. Documentation of compaction meter values on computer screen in front of the roller operator. Areas with different CMV-values are visualized

In the documentation system developed by Geodynamik, it is even possible to follow the movement of the roller on the screen. The values stored in the documentation system can be transmitted to a larger computer memory.

It is now planned to establish a central Swedish computer register in which different road projects are collected.

COMPACTION SPECIFICATIONS

Both in Sweden and West Germany the aim is to include roller-mounted compaction meters in future compaction specifications.

Up to now, detailed prestudies made in each special case have established which compaction meter values can be applied.

One possible line for future specifications based on the use of roller-mounted compaction meters is to prescribe procedures for a calibration of the compaction meter values against conventional density or load-bearing tests. After such calibration the compaction meter may then replace the traditional control methods.

It is still an open question in which way compaction meters can best be used on fine grained soils at water contents over the optimum. In such cases the compaction meter is best adapted for a check of the level and uniformity of the bearing capacity of embankment fills and subgrade surfaces.

It is certainly a difficult and time-consuming procedure to introduce new control methods replacing conventional methods established for many decades.

CONCLUSIONS

Roller-mounted compaction meters give dimensionless relative values indicating the bearing capacity of the ground. The values are very reproducible. On coarse-grained, and particularly free-draining types of soils, good correlations are obtained between the compaction meter values, and values on density and bearing capacity obtained with conventional test methods. On large-scale compaction jobs mainly in West Germany on coarse grained soils, compaction meters have been successfully used for a rapid compaction control covering the entire fill area.

On fine-grained soils the bearing capacity and thus the compaction meter value is, to a high degree, dependent on the water content. In this way the compaction meter cannot be used to check the density directly. A knowledge of the bearing capacity can, however, be of great value.

New computerized documentation systems represent a promising development.

Work is going on to include compaction meters in regular compaction specifications.

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LIST OF FIGURES

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APPENDIX

PAST SUBJECTS OF THE

OHIO RIVER VALLEY SOILS SEMINAR

The Kentucky Geotechnical Group and the Cincinnati Geotechnical Group, in cooperation with the Civil Engineering Departments of the University of Louisville, University of Cincinnati, and University of Kentucky, continued in 1988, with the assistance of the Office of Continuing Education at the University of Kentucky and Kentucky Transportation Center, the tradition of an annual Ohio River Valley Soils Seminar. These specialty seminars, begun in 1970, have been designed to provide an opportunity for interchange of ideas and information to practicing engineers and engineering students. The seminar topics are drawn from the fields of soil mechanics, foundation engineering, rock mechanics and engineering geology. Past seminar subjects are listed below:

ORVSS I: BUILDING FOUNDATION DESIGN AND CONSTRUCTION, October 16, 1970, Lexington, Kentucky

ORVSS II: EARTHWORK ENGINEERING, START TO FINISH October 15, 1971, Louisville, Kentucky

ORVSS III: LATERAL EARTH PRESSURES, October 27, 1972, Fort Mitchell, Kentucky

ORVSS IV: GEOTECHNICS IN TRANSPORTATION ENGINEERING, OCTOBER 5, 1973, Lexington, Kentucky

ORVSS V: ROCK ENGINEERING, October 18, 1974, Clarksville, Indiana

ORVSS VI: SLOPE STABILITY AND LANDSLIDES, October 17, 1975, Fort Mitchell, Kentucky

ORVSS VII: SHALES and MINE WASTES: GEOTECHNICAL PROPERTIES, DESIGN AND CONSTRUCTION, October 8, 1978, Lexington, Kentucky

ORVSS VIII: EARTH DAMS AND EMBANKMENTS: DESIGN, CONSTRUCTION, AND PERFORMANCE, October 14, 1977, Louisville, Kentucky

ORVSS IX: DEEP FOUNDATIONS, October 27, 1978, Fort Mitchell, Kentucky

ORVSS X: GEOTECHNICS OF MINING, October 5, 1979, Lexington, Kentucky

ORVSS XI: EARTH PRESSURES AND RETAINING STRUCTURES, October 10, 1980, Clarksville, Indiana

ORVSS XII: GROUNDWATER: MONITORING, EVALUATION, AND CONTROL, October 9, 1981, Fort Mitchell, Kentucky

ORVSS XIII: RECENT ADVANCES IN GEOTECHNICAL ENGINEERING PRACTICE, October 8, 1982, Lexington, Kentucky

ORVSS XIV: FOUNDATION INSTRUMENTATION AND GEOPHYSICAL

EXPLORATION, October 14, 1983, Clarksville, Indiana

ORVSS XV: PRACTICAL APPLICATION OF DRAINAGE IN GEOTECHNICAL ENGINEERING, November 2, 1984, Fort Mitchell, Kentucky

ORVSS XVI: APPLIED SOIL DYNAMICS, October 11, 1985, Lexington, Kentucky

ORVSS XVII: NATURAL SLOPE STABILITY AND INSTRUMENTATION October 17, 1986, Clarksville, Indiana

ORVSS XVIII: LIABILITY ISSUES IN GEOTECHNICAL ENGINEERING AND CONSTRUCTION, November 6, 1987, Cincinnati, Ohio