TRANSPORTATION TECHNOLOGY SUPPORT
FOR DEVELOPING COUNTRIES

COMPENDIUM 8

Chemical Soil Stabilization

Estabilización química de suelos

La stabilisation chimique des sols

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Notice

The project that is the subject of this report was approved by the Governing Board of the National Research Council, whose members are drawn from the councils of the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine. The members of the committee responsible for the report were chosen for their special competence and with regard for appropriate balance.

This report has been reviewed by a group other than the authors according to procedures approved by a Report Review Committee consisting of members of the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine.

Cover photo: Shoulder is stabilized by using asphaltic emulsion (Central Africa).

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Project Description

The development of agriculture, the distribution of food, the provision of health services, and the access to information through educational services and other forms of communication in rural regions of developing countries all heavily depend on transport facilities. Although rail and water facilities may play important roles in certain areas, a dominant and universal need is for road systems that provide an assured and yet relatively inexpensive means for the movement of people and goods. The bulk of this need is for low-volume roads that generally carry only 5 to 10 vehicles a day and that seldom carry as many as 400 vehicles a day.

The planning, design, construction, and maintenance of low-volume roads for rural regions of developing countries can be greatly enhanced with respect to economics, quality, and performance by the use of low-volume road technology that is available in many parts of the world. Much of this technology has been produced during the developmental phases of what are now the more developed countries, and some is continually produced in both the less and the more developed countries. Some of the technology has been documented in papers, articles, and reports that have been written by experts in the field. But much of the technology is

Descripción del proyecto

En las regiones rurales de países en desarrollo, el desarrollo de la agricultura, la distribución de víveres, la provisión de servicios de sanidad, y el acceso a información por medio de servicios educacionales y otras formas de comunicación, dependen en gran parte de los medios de transporte. Aunque en ciertas áreas los medios de ferrocarril y agua desempeñan un papel importante, existe una necesidad universal y dominante de crear sistemas viales que provean un medio asegurado pero relativamente poco costoso para el movimiento de gente y mercancías. La mayor parte de esta necesidad se solucionaría con la construcción de caminos de bajo volúmen que generalmente moverían únicamente de 5 a 10 vehículos por día y que pocas veces moverían tanto como 400 vehículos por día.

El planeamiento, diseño, construcción y mantenimiento de caminos de bajo volúmen para regiones rurales de países en desarrollo pueden ser mejorados, con respecto al costo, calidad, y rendimiento, por el uso de la tecnología de caminos de bajo volúmen que se encuentra disponible en muchas partes del mundo. Mucha de esta tecnología ha sido producida durante las épocas de desarrollo de lo que ahora son los países más desarrollados, y alguna se produce contínuamente en estos países así como en los países menos desarrollados. Parte de la tecnología se ha documentado en disertaciones, artículos, e informes que han sido escritos por expertos en el campo. Pero mucha de la tecnología no está documentada y existe principalmente en la memoria de aquellos que han desa-

Description du projet

Dans les régions rurales des pays en voie de développement, l'exploitation agricole, la distribution des produits alimentaires, l'accès aux services médicaux, l'accès aux matériaux et aux marchandises, à l'information et aux autres services, dépendent en grande partie des moyens de transport. Bien que les transports par voie ferrée et par voie navigable jouent un rôle important dans certaines régions, un besoin dominant et universel éxiste d'un réseau routier qui puisse assurer avec certitude et d'une façon relativement bon marché, le déplacement des habitants, et le transport des marchandises. La plus grande partie de ce besoin peut être satisfaite par la construction de routes à faible capacité, capables d'accommoder un trafic de 5 a 10 véhicules par jour, ou plus rarement, jusqu'à 400 véhicules par jour.

L'utilisation des connaissances actuelles en technologie, qui sont accéssibles dans beauundocumented and exists mainly in the minds of those who have developed and applied the technology through necessity. In either case, existing knowledge about low-volume road technology is widely dispersed geographically, is quite varied in the language and the form of its existence, and is not readily available for application to the needs of developing countries.

In October 1977 the Transportation Research Board (TRB) began this 3-year special project under the sponsorship of the U.S. Agency for International Development (AID) to enhance rural transportation in developing countries by providing improved access to existing information on the planning, design, construction, and maintenance of low-volume roads. With advice and guidance from a project steering committee, TRB defines, produces, and transmits information products through a network of correspondents in developing countries. Broad goals for the ultimate impact of the project work are to promote effective use of existing information in the economic development of transportation infrastructure and thereby to enhance other aspects of rural development throughout the world.

In addition to the packaging and distribution of technical information, personal interactions with users are provided through field visits, con-

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rrollado y aplicado la tecnología por necesidad. En cualquier caso, los conocimientos en existencia sobre la tecnología de caminos de bajo volúmen están grandemente esparcidos geográficamente, varian bastante con respecto al idioma y su forma, y no se encuentran fácilmente disponibles para su aplicación a las necesidades de los países en desarrollo.

En octubre de 1977 el Transportation Research Board (TRB) comenzó este proyecto especial de tres años de duración bajo el patrocinio de la U.S. Agency for International Development (AID) para mejorar el transporte rural en los países en desarrollo acrecentando la disponibilidad de la información en existencia sobre el planeamiento, diseño, construcción, y mantenimiento de caminos de bajo volúmen. Con el consejo y dirección de un comité de iniciativas para el proyecto, el TRB define, produce, y transmite productos informativos a través de una red de corresponsales en países en desarrollo. Las metas generales para el impacto final del trabajo del proyecto son la promoción del uso efectivo de la información en existencia en el desarrollo económico de la infraestructura de transporte y de esta forma mejorar otros aspectos del desarrollo rural a través del mundo.

Además de la recolección y distribución de la

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coup de pays, peut faciliter l'étude des projets de construction, tracé et entretien, de routes à faible capacité dans les régions rurales des pays en voie de développement, surtout en ce qui concerne l'économie, la qualité, et la performance de ces routes. La majeure partie de cette technologie a été produite durant la phase de développement des pays que l'on appelle maintenant développés, et elle continue à être produite à la fois dans ces pays et dans les pays en voie de développement. Certains aspects de cette technologie ont été documentés dans des articles ou rapports écrits par des experts. Mais une grande partie des connaissances n'existe que dans l'esprit de ceux qui ont eu besoin de développer et appliquer cette technologie. De plus, dans ces deux cas, les écrits et connaissances sur la technologie des routes à faible capacité, sont dispersés géographiquement, sont écrits dans des langues différentes, et ne sont pas assez aisément accessibles pour être appliqués aux besoins des pays en voie de développement.

En octobre 1977, le Transportation Research Board (TRB) initia ce projet, d'une duré de 3 ans, sous le patronage de l'U.S. Agency for International Development (AID), pour améliorer le transport rural dans les pays en voie de développement, en rendant plus accessible la documentation existante sur la conception, le tracé, la construction, et l'entretien des routes à faible capacité. Avec le conseil, et sous la conduite d'un comité de direction, TRB définit, produit, et transmet cette documentation à l'aide d'un réseau de correspondants dans les pays en voie de développement. Nous espérons que le résultat final de ce projet sera de favoriser l'utilisation de cette documentation, pour aider au développement économique de l'infrastructure des transports, et de cette façon mettre en valeur d'autres aspects d'exploitation rurale à travers le monde.

ferences in the United States and abroad, and other forms of communication.

Steering Committee

The Steering Committee is composed of experts who have knowledge of the physical and social characteristics of developing countries, knowledge of the needs of developing countries for transportation, knowledge of existing transportation technology, and experience in its use.

Major functions of the Steering Committee are to assist in the definition of users and their needs, the definition of information products that match user needs, and the identification of informational and human resources for development of the information products. Through its membership the committee provides liaison with project-related activities and provides guidance for interactions with users. In general the Steering Committee gives overview advice and direction for all aspects of the project work.

The project staff has responsibility for the preparation and transmittal of information products, the development of a correspondence network throughout the user community, and interactions with users.

Information Products

Three types of information products are prepared: compendiums of documented information on relatively narrow topics, syntheses of knowledge and practice on somewhat broader

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información técnica, se provee acciones recíprocas personales con los usuarios por medio de visitas de campo, conferencias en los Estados Unidos de Norte América y en el extranjero, y otras formas de comunicación.

Comité de iniciativas

El comité de iniciativas se compone de expertos que tienen conocimiento de las características físicas y sociales de los países en desarrollo, conocimiento de las necesidades de transporte de los países en desarrollo, conocimiento de la tecnología de transporte en existencia, y experiencia en su uso.

Las funciones importantes del comité de iniciativas son las de ayudar en la definición de usuarios y sus necesidades, de productos informativos que se asemejan a las necesidades del usuario, y la identificación de recursos de conocimientos y humanos para el desarrollo de los productos informativos. A través de sus miembros el comité provee vínculos con actividades relacionadas con el proyecto y también una guía para la interacción con los usuarios. En general el comité de iniciativas proporciona consejos y dirección general para todos los aspectos del trabajo de proyecto.

El personal de proyecto es responsable de la preparación y transmisión de los productos informativos, el desarrollo de una red de corresponsales a través de la comunidad de usuarios, y la interacción con los usuarios.

Productos informativos

Se preparan tres tipos de productos informativos: los compendios de la información documentada sobre temas relativamente limitados, la síntesis del conocimiento y práctica sobre temas

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En plus de la dissémination de cette documentation technique, des visites, des conférences aux Etats Unis et à l'étranger, et d'autres formes de communication permettront une interaction constante avec les usagers.

Comité de direction

Le comité de direction est composé d'experts qui ont à la fois des connaissances sur les caractéristiques physiques et sociales des pays en voie de développement, sur leurs besoins au point de vue transports, sur la technologie actuelle des transports, et ont aussi de l'expérience quant à l'utilisation pratique de cette technologie.

Les fonctions majeures de ce comité sont d'abord d'aider à définir les usagers et leurs besoins, puis de définir leurs besoins en matière de documentation, et d'identifier les ressources documentaires et humaines nécessaires pour le développement de cette documentation. Par l'intermédiaire des ses membres, le comité pourvoit à la liaison entre les différentes fonctions relatives au projet, et dirige l'interaction avec les usagers. En général, le comité de direction conseille et dirige toutes les phases du projet.

Notre personnel est responsable de la préparation et de la dissémination des documents, du développement d'un réseau de correspondants pris dans la communauté d'usagers, et de l'interaction avec les usagers.

La documentation

Trois genres de documents sont preparés: des recueils dont le sujet est relativement limité, des subjects, and proceedings of low-volume road conferences that are totally or partially supported by the project. Compendiums are prepared by project staff at the rate of about 6 per year; consultants are employed to prepare syntheses at the rate of 2 per year. At least one conference proceedings will be published during the 3-year period. In summary, this project aims to produce and distribute between 20 and 30 publications that cover much of what is known about low-volume road technology.

Interactions With Users

A number of mechanisms are used to provide interactions between the project and the user community. Project news is published in each issue of ***Transportation Research News.*** Feedback forms are transmitted with the information products so that recipients have an opportunity to say how the products are beneficial and how they may be improved. Through semiannual visits to developing countries, the project staff acquires first-hand suggestions for the project work and can assist directly in specific technical problems. Additional opportunities for interaction with users arise through international and in-country conferences in which there is project participation. Finally, annual colloquiums are held for students from developing countries who are enrolled at U.S. universities.

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un poco más amplios, y los expedientes de conferencias de caminos de bajo volúmen que están totalmente o parcialmente amparados por el proyecto. El personal de proyecto prepara los compendios a razón de unos 6 por año; se utilizan consultores para preparar las síntesis a razón de 2 por año. Se publicará por lo menos un expediente de conferencia durante el período de tres años. En breve, este proyecto pretende producir y distribuir entre 20 y 30 publicaciones que cubren mucho de lo que se conoce de la tecnología de caminos de bajo volúmen.

Interacción con los usuarios

Se utilizan varios mecanismos para proveer las interacciones entre el proyecto y la comunidad de usuarios. Se publican las noticias del proyecto en cada edición de la ***Transportation Research News.*** Se transmiten, con los productos informativos, formularios de retroacción para que los recipientes tengan oportunidad de decir cómo benefician los productos y cómo pueden ser mejorados. A través de visitas semianuales a los países en desarrollo, el personal del proyecto adquiere directamente de fuentes originales sugerencias para el trabajo del proyecto y puede asistir directamente en problemas técnicos específicos. Surgen oportunidades adicionales para la interacción con los usuarios a través de conferencias internacionales y nacionales en donde participa el proyecto. Finalmente, se organizan diálogos con estudiantes de países en desarrollo que están inscriptos en universidades norteamericanas.

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synthèses de connaissances et de pratique sur des sujets beaucoup plus généraux, et finalement des comptes-rendus de conférences sur les routes à faible capacité, qui seront organisées complètement ou en partie par notre projet. Environ 6 recueils par an sont preparés par notre personnel. Deux synthèses par an sont écrites par des experts pris à l'extérieur. Les comptes-rendus d'au moins une conférence seront écrits dans une période de 3 ans. En résumé, l'objet de ce projet est de produire et disséminer entre 20 et 30 documents qui couvriront l'essentiel des connaissances sur la technologie des routes à faible capacité.

Interaction avec les usagers

Un certain nombre de mécanismes sont utilisés pour assurer l'interaction entre le personnel du projet et la communauté d'usagers. Un bulletin d'information est publié dans chaque numéro de ***Transportation Research News.*** Des formulaires sont joints aux documents, afin que les usagers aient l'opportunité de juger de la valeur de ces documents et de donner leur avis sur les moyens de les améliorer. Au cours de visites semi-annuelles dans les pays en voie de développement notre personnel obtient de première main des suggestions sur le bon fonctionnement du projet et peut aider à résoudre sur place certains problèmes techniques spécifiques. En outre, des conférences tenues soit aux Etats Unis, soit à l'étranger, sont l'occasion d'un échange d'idées entre notre personnel et les usagers. Finalement, des colloques annuels sont organisés pour les étudiants des pays en voie de développement qui étudient dans les universités américaines.

Foreword and Acknowledgments

This compendium is the eighth product of the Transportation Research Board's project on Transportation Technology Support for Developing Countries under the sponsorship of the U.S. Agency for International Development. The objective of this book is that it provide useful and practical information for those in developing countries who have direct responsibility for soil stabilization. Feedback from correspondents in developing countries will be solicited and used to assess the degree to which this objective has been attained and to influence the nature of later products.

Acknowledgment is made to the following publishers for their kind permission to reprint the selected text portions of this compendium:

Butterworth and Co. (Publishers) Ltd., London;
Building and Road Research Institute, Ghana Academy of Sciences.

Prefacio y agradecimientos

Este compendio es el octavo producto del proyecto del Transportation Research Board sobre Apoyo de Tecnología de Transporte para Países en Desarrollo bajo el patrocinio de la U.S. Agency for International Development. El objetivo de este libro es el de proveer información útil y práctica para aquellos en países en desarrollo quienes tienen responsabilidad directa para la estabilización de suelos. Se pedirá a los corresponsales en los países en desarrollo información sobre los resultados, para utilizarse en el asesoramiento del grado al cual se ha obtenido ese objetivo y para influenciar la naturaleza de productos subsequentes.

Se reconoce a los siguientes editores por el permiso dado para reimprimir las porciones de texto seleccionadas para este compendio.

Butterworth and Co. (Publishers) Ltd., London;
Building and Road Research Institute, Ghana Academy of Sciences.

Avant-propos et remerciements

Ce recueil représente le huitième volume du projet du Transportation Research Board sur la Technologie des transports à l'usage des pays en voie de développement. Ce projet est placé sous le patronage de l'U.S. Agency for International Development. L'objet de ce recueil est de réunir une documentation pratique et utile qui puisse aider les personnes responsables de la stabilisation des sols. La réaction des correspondants des pays en voie de développement sera sollicitée et utilisée pour évaluer à quel point le but proposé de ce projet a été atteint, et pour influencer la nature des ouvrages à venir.

Nous remercions les éditeurs qui ont gracieusement donné leur permission de reproduire les textes sélectionnés pour ce recueil:

Butterworth and Co. (Publishers) Ltd., London;
Building and Road Research Institute, Ghana Academy of Sciences.

Appreciation is also expressed to libraries and information services that provided references and documents from which final selections were made for the selected texts and bibliography of this compendium. Special acknowledgment is made to the U.S. Department of Transportation Library Services Division and to the Library and Information Service of the U.K. Transport and Road Research Laboratory (TRRL). Any photographs provided by TRRL have been reproduced by permission of Her Majesty's Stationery Office.

Finally, the Transportation Research Board acknowledges the valuable advice and direction that have been provided by the project Steering Committee and is especially grateful to R.G. Hicks, Oregon State University, Wilbur J. Morin, Lyon Associates, Inc., and W. G. Wilson, International Road Federation, who provided special assistance on this particular compendium.

También se reconoce a las bibliotecas y servicios de información que proveen las referencias y documentos de los cuales se hacen las selecciones finales para los textos seleccionados y la bibliografía en este compendio. Se hace un especial reconocimiento a la Library Services Division del U.S. Department of Transportation y el Library and Information Service del U.K. Transport and Road Research Laboratory (TRRL). Las fotografías proveídas por TRRL fueron reproducidas con la autorización de Her Majesty's Stationery Office.

Finalmente, el Transportation Research Board agradece el consejo y dirección valiosos provistos por el comité de iniciativas, con especial reconocimiento a los señores R. G. Hicks, Oregon State University, Wilbur J. Morin, Lyon Associates, Inc., y W. G. Wilson, International Road Federation, que prestaron ayuda especial para este compendio en particular.

Nos remerciements aussi aux bibliothèques et bureaux de documentation qui nous ont fourni les documents et les références utilisés dans les textes choisis et bibliographie de ce recueil. Nous remercions spécialement la U.S. Department of Transportation Library Services Division et les Library and Information Service of the U.K. Transport and Road Research Laboratory (TRRL). Les photos fournies par le TRRL ont été reproduites avec la permission de Her Majesty's Stationery Office.

Finalement, le Transportation Research Board reconnait la grande valeur de la direction et de l'assistance des membres du comité de direction et les remercie de leur concours et de la façon dont ils dirigent le projet, spécialement Messieurs R.G. Hicks, Oregon State University, Wilbur J. Morin, Lyon Associates, Inc., et W. G. Wilson, International Road Federation, qui ont bien voulu prêter leur assistance à la préparation de ce recueil.

Overview

Background and Scope

Compendium 7: Road Gravels states that low-volume roads are constructed mostly of soils. When these soils are coarse granular materials at the proper gradation, a road surface that has a load-bearing capacity can be constructed using mechanical or granular stabilization. The proper blend of particle sizes produces an interlocking effect, or mechanical strength, when it is properly compacted at or near optimum moisture content.

Coarse granular materials, or gravels, are not always available within an economic haul distance of a road construction site, especially in many developing areas of the world. When a simple track is constructed with in situ material, the problem of inferior materials can be solved by closing the road during periods when it is structurally unsound due to excessive moisture content, i.e., during the rainy season. During the dry season, road users grope their way through clouds of dust, following ruts and avoiding potholes because of the difficulty in maintaining a reasonable running surface at a justifiable cost on a road constructed of inferior material.

These conditions are accepted by the road user when the traffic volume is low. However, as the traffic increases — especially if the traffic is commercial in nature, i.e., trucks and buses trying to operate at a profit — added pressure is placed on the roadbuilder to improve the riding characteristics and all-weather capability of the roadway.

Vista General

Antecedentes y alcance

En el *Compendio 7: Gravas* se dice que los caminos de bajo volúmen se construyen mayormente de suelos. Cuando estos suelos son de material granular tosco en la correcta graduación, se puede construir una superficie de camino con capacidad de carga utilizando la estabilización mecánica o granular. La mezcla correcta de tamaños de partícula produce un efecto de engranaje, o resistencia mecánica, cuando se compacta correctamente con humedad óptima o casi óptima.

Los materiales granulares toscos, o gravas, no siempre se encuentran a distancias económicamente factibles para su transporte hasta la zona de construcción, especialmente en muchas de las áreas en desarrollo a través del mundo. Cuando se construye un sendero sencillo con materiales in situ, para solucionar el problema de los materiales inferiores se puede clausurar el camino durante los períodos cuando es estructuralmente defectuoso debido a excesiva humedad, es decir, durante la época de lluvia. Durante la época de sequía, los usuarios pasan por grandes nubes de polvo y sobre roderas y baches porque es difícil mantener una superficie de rodamiento razonable a un costo justificable en un camino construído con materiales inferiores.

Exposé

Historique et objectif

Dans le recueil no. 7, nous avons formulé que les routes à faible capacité sont, la plupart du temps, des routes de terre. Quand cette "terre" est constituée par des matériaux granuleux grossiers, de granulométrie adéquate, on peut obtenir, à l'aide de la stabilisation mécanique, une route dont la surface aura une bonne capacité de portance. La compaction d'un mélange adéquat de grains de dimensions différentes, résulte en un phénomène d'accrochage ou résistance mécanique, quand ce mélange est compacté à, ou presqu'à, sa teneur en eau optimale.

Les matériaux granuleux grossiers, ou graviers, ne se trouvent pas toujours à une distance de transport économique, surtout dans les pays en voie de développement. Quand il s'agit d'une simple piste de terre construite de matériaux in situ, on peut résoudre le problème de la qualité inférieure de ces matériaux en interdisant l'accès à cette piste durant la saison des pluies, quand l'humidité excessive la rend dangereuse.

Improving road surfaces made from inferior material is neither inexpensive nor simple. It is accomplished by the addition of some alien material or combination of materials that react chemically with the native soil to improve its physical properties. The selection of the chemical additives, commonly called stabilizers or binders, differs depending on the classification of the native soil to be stabilized.

Rationale for This Compendium

Many additives can be used to chemically stabilize soils. Some additives — for example, molasses, salt, fly ash, phosphates, and lignin sulfonate — are products of local industries. These additives are of limited interest because their use is restricted to those small areas where they are available at a reasonable cost. Although their use should certainly be investigated by highway engineers who have access to such products, Compendium 8 is concerned with more widely available stabilizers, i.e., lime, cement, and asphaltic or bituminous products.

Chemical stabilization involves a series of problems. All must be solved successfully to construct a suitable roadway. Because of the number and variety of these problems, many

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El usuario acepta estas condiciones cuando el volúmen de tránsito es bajo. Sin embargo, cuando el tránsito aumenta — especialmente si es comercial, es decir, camiones y autobuses utilizados con fines lucrativos — se presiona al constructor de caminos para que mejore las características de rodamiento y la capacidad para toda temporada del camino.

No es barato ni simple mejorar las superficies de camino hechas de material inferior. Se realiza añadiendo algún material ajeno o combinación de materiales que reaccionan químicamente con el suelo nativo para mejorar sus propiedades físicas. La selección de estos aditivos químicos, llamados estabilizadores o aglutinantes, depende de la clasificación del suelo nativo a estabilizarse.

Exposición razonada para este compendio

Hay muchos aditivos que se pueden utilizar para estabilizar los suelos químicamente. Algunos aditivos — por ejemplo, melaza, sal, ceniza volátil, fosfatos y sulfonato de lignina — son productos de industrias locales. Estos productos son de interés limitado ya que su utilización se limita a aquellos lugares donde se pueden obtener a costo razonable. Sin duda alguna deben ser investigados por aquellos ingenieros que tienen acceso a tales productos, pero el Compendio 8 se interesa en los estabilizadores obtenibles en cualquier lugar, es decir, cal, cemento, y productos asfálticos o bituminosos.

Pendant la saison sèche, les usagers de la piste avancent à tâtons à travers des nuages de poussière, en suivant les ornières, et en essayant d'eviter les nids de poule, tout cela car il est difficile de garantir une surface de roulement adéquate pour un prix justifié, quand cette piste est construite en matériaux de qualité inférieure.

L'usager acceptera ces conditions si la circulation est très modeste. Par contre, quand la circulation augmente, particulièrement si celle-ci devient commerciale, c'est à dire inclut des camions et des autocars essayant d'opérer à profit, la situation change et on va faire pression sur les responsables du réseau routier pour obtenir une route praticable en toutes saisons et ayant une bonne qualité de roulement.

L'amélioration des routes construites avec des matériaux de qualité inférieure, est une opération qui n'est ni simple, ni peu onéreuse. Cette opération consiste à ajouter un ou plusieurs matériaux, de façon à obtenir une réaction chimique quand ils sont mélangés au matériau in situ, qui améliorera ses propriétés physico-mécaniques. La sélection de ces adjuvants chimiques dépend de la classification du sol que l'on veut stabiliser chimiquement.

Objet de ce recueil

On peut utiliser toute une gamme d'adjuvants pour effectuer la stabilisation chimique des sols. Certains d'entre eux peuvent être les produits d'industries locales, tels que la mélasse, le sel, les cendres volantes, les phosphates, le lignosulfite de sodium (sous-produit de la fabrication de la pâte à papier). Ces adjuvants sont d'un intérêt limité car leur utilisation est restreinte aux endroits où on peut se les procurer à des prix raisonnables. Bien que, dans ces conditions, l'utilisation de ces produits devrait certainement être considérée par les ingénieurs routiers, nous allons examiner dans ce recueil, les stabilisants les plus usuels que l'on peut trouver partout: la chaux, le ciment, et les produits bitumineux.

La stabilisation chimique des sols implique une série de problèmes que l'on doit résoudre engineers have had unpleasant experiences with soil stabilization projects. This compendium identifies and offers solutions to some of these concerns.

The first problem relates to the identification of the soil or soils to be stabilized on a given project. Because various soils react differently with each stabilizer, poor identification of soils leads to the improper selection of a stabilizing agent. Furthermore, a few soils cannot be stabilized with any degree of assurance using any of the conventional stabilization methods. Once the soil has been identified, the stabilizing material can be determined.

A second problem is determining the amount of stabilizing agent to use. Various laboratory tests are outlined in the selected texts to determine the needed admixture rates. Inexperienced engineers who attempt to make use of these tests should watch for the following:

1. Any test procedure should be read carefully to determine whether the percentages referenced are by volume of dry material or by dry weight of material. A further precaution to be taken with bituminous cutback or emulsified material is to check whether the percentage specified (either by weight or volume) is for the entire material as applied or only for the asphaltic cement portion of the material.

2. The soils used in the test procedures must be representative of the soils in the field. The test results are not transferable to other soils.

3. The stabilizing agent, the water used, and the curing temperature affect the test results. They should be consistent throughout the testing procedures, and they should be representative

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En la estabilización química hay una serie de problemas que deberán resolverse antes de poder construir un camino adecuado. A causa de la gran cantidad y variedad de problemas, muchos ingenieros han tenido malas experiencias con relación a los proyectos de estabilización de suelos. Este compendio identifica y ofrece soluciones para algunos de estos problemas.

El primer problema se relaciona con la identificación del suelo o suelos a estabilizarse en un determinado proyecto. Ya que cada suelo reacciona diferentemente con cada estabilizador, la mala identificación de los suelos causa una incorrecta selección del agente estabilizador. Además, algunos suelos no se pueden estabilizar de modo seguro con los métodos convencionales. Una vez identificado el suelo, se puede determinar el material estabilizador.

Un segundo problema es la determinación de la cantidad de agente estabilizador a utilizarse. En los textos seleccionados se explican varios ensayos de laboratorio utilizados para determinar las cantidades necesitadas de aditivos. Los ingenieros de poca experiencia que tienen intención de utilizar estos ensayos deberán ocuparse de los siguientes detalles:

1. Todos los procedimientos de ensayo deberán leerse cuidadosamente para determinar si los porcentajes a los cuales se refieren son por volúmen de material seco o por peso seco de material. Además, con respecto al material bituminoso fluidificado ("cutback") o emulsificado, se deberá averiguar si el porcentaje especificado (por peso o volúmen) es para todo el material tal como se aplique, o únicamente para la parte de cemento asfáltico.

2. Los suelos que se utilicen en los ensayos

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avec succés si l'on veut aboutir à une qualité de roulement convenable. A cause du nombre et de la diversité de ces problèmes, beaucoup d'ingénieurs ont été soumis à des expériences désagréables lors de projets de stabilisation chimique. Dans ce recueil nous allons essayer d'abord d'identifier ces problèmes, puis de leur trouver des solutions.

Le premier de ces problèmes est relatif à l'identification du sol ou des sols à stabiliser. Comme différents sols réagissent différemment avec chaque stabilisant, une mauvaise identification d'un sol conduit à une sélection inexacte d'un stabilisant. En outre, il existe quelques sols qui ne peuvent pas être stabilisés en utilisant les méthodes conventionnelles de stabilisation. Une fois que le sol est identifié on peut déterminer le stabilisant à utiliser.

Un deuxième problème concerne la quantité de stabilisant nécessaire. Plusieurs essais en laboratoire, pour la détermination du dosage approprié des adjuvants sont esquissés dans les textes choisis. Les ingénieurs peu familiarisés avec ces essais devront faire très attention aux points suivants:

1. On doit lire très attentivement les instructions avant de commencer l'essai, pour déterminer si les dosages se réfèrent au volume de matériau sec ou au poids de matériau sec. De plus, avec les bitumes fluidifiés (naguère appelés les cut-backs) ou avec les émulsions de bitume, il faut vérifier si ce dosage, pondéral ou volumétrique, se rapporte au matériau dans son ensemble tel qu'il va être appliqué, ou seulement pour la partie ciment asphaltique du matériau.

of the material and conditions encountered in the actual construction of the road.

4. The test procedures used should realistically represent the construction techniques available in the field, especially the mixing and compaction operations.

5. The test results found in the laboratory will be more favorable than test results from the finished field product.

The proper application of the stabilizing agent in the field presents a third problem. The field engineer must also know if the percentage to be applied is by weight or by volume. The uniform application of the additive is very difficult unless specialized equipment is used. However, the proper rate of application is often a problem even with specialized equipment.

A fourth problem concerns the proper mixing of the stabilizing agent and the soil. This problem may also involve the preparation of the soil before the application of the stabilizer. Heavy clays must be pulverized before proper mixing can take place. If a granular material is to be added to an in situ material, it must be thoroughly blended before introducing the additive. The use of different types of construction equipment to mix various stabilizers with soil is fully explained in the selected texts. However, no amount of explanation will assist the engineer who tries to shortcut the mixing procedure to increase production rates.

A fifth problem is the proper use of water. A soil undergoing chemical stabilization does not react to moisture in the same way as a soil being mechanically stabilized, because the water is in-

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deberán ser representativos de los suelos en el campo. Los resultados de ensayo no son transferibles a otros suelos.

3. El agente estabilizador, el agua que se utiliza, y la temperatura de curado, afectan los resultados de ensayo. Deberán ser consistentes durante todos los procedimientos de ensayo y deberán representar el material y condiciones que realmente se encontrarán en la construcción del camino.

4. Los procedimientos de ensayo que se utilicen deberán representar, en forma realística, las técnicas de construcción disponibles en el campo, especialmente las operaciones de mezclado y compactación.

5. Los resultados de ensayo del laboratorio serán más favorables que los resultados de ensayo del producto terminado del campo.

La correcta aplicación del agente estabilizador en el campo es el tercer problema. El ingeniero en el campo deberá saber también si el porcentaje a aplicarse es por peso o volúmen. Es muy difícil aplicar el aditivo uniformemente, a menos que se utilice equipo especializado. Frequentemente la aplicación del aditivo es un problema aunque se tenga este equipo.

Un cuarto problema es el mezclado correcto del agente estabilizador y el suelo. Esto puede exigir la preparación del suelo antes de la aplicación del estabilizador. Las arcillas pesadas deberán pulverizarse antes de poder mezclarse correctamente. Si se agrega material granular al material in situ, éste deberá mezclarse bien antes de aplicar el aditivo. En los textos seleccionados se explica cómo utilizar los distintos tipos de equipo de construcción para mezclar

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2. Les essais en laboratoire doivent être faits sur des échantillons représentatifs des matériaux que l'on va utiliser dans la construction routière, prélevés directement et de façon adéquate dans les gîtes ou les emprunts, car les résultats des essais ne peuvent pas être transférés d'un sol à un autre.

3. Le stabilisant, l'eau, et la temperature de la cure, influencent les résultats des essais. Donc, il est important de les garder constants durant les essais, et de s'assurer qu'ils soient représentatifs des matériaux et des conditions tels qu'ils seront sur le chantier, lors de la construction de la route.

4. Les méthodes d'essais employées devraient refléter, de façon réalistique les procédés que l'on va utiliser sur le chantier, surtout en ce qui concerne le malaxage et le compactage.

5. Les résultats des essais faits en laboratoire seront plus favorables que les résultats des mêmes essais faits sur des échantillons prélevés sur la route une fois qu'elle a été construite.

L'application du stabilisant en chantier pose une troisième problème. L'ingénieur routier doit savoir si le dosage de stabilisant est calculé en poids ou en volume. Il est très difficile d'obtenir une couche uniforme d'enduit sans avoir recours à des engins spécialisés. Même avec ceux-ci, une vitesse d'application adéquate n'est pas toujours facile à obtenir.

Un quatrième problème est causé par le mélange du stabilisant et du matériau. Ce problème peut aussi mettre en jeu la préparation du matériau avant l'addition du stabilisant. Les argiles lourdes doivent être pulvérisées avant le malaxage. Si l'on doit ajouter des granulats au matériau in situ, il faut prendre la précaution de volved in the chemical reactions of most chemical stabilization processes. Quite frequently the optimum moisture content may not give the densest mix or the strongest end product. The rules-of-thumb that are often used to determine the proper moisture content for mechanical stabilization may result in failure of a chemically stabilized material. Because the water is involved in the chemical reaction, certain impurities in the water may also inhibit proper chemical stabilization.

A sixth problem focuses on the compaction of the stabilized material. Each stabilizing agent requires its own compaction technique. Chemical stabilization is time dependent. The necessary compactive effort, the optimum moisture content, and the maximum dry density vary depending on the time lag between application and compaction. Cement-soil stabilization is most sensitive to compaction time delay. The strength of the finished soil-cement product decreases very rapidly as the time between mixing and compaction increases. Increasing the moisture content during the mixing operation sometimes will reduce the loss of strength due to this delay. Many engineers, however, do not fully appreciate the critical nature of the mixing-compaction timing of soil-cement.

A seventh problem is the curing of the stabilized material. While the cation exchange and flocculation-agglomeration reactions occur very rapidly in lime stabilization, all other chemi-

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los diversos estabilizadores con el suelo. Sin embargo, no hay explicación que valga si el ingeniero trata de acortar los procedimientos de mezclado para aumentar la velocidad de producción.

El quinto problema es el uso correcto del agua. Un suelo que se está estabilizando químicamente no reacciona a la humedad de la misma manera que un suelo que se está estabilizando mecánicamente, ya que el agua toma parte en las reacciones químicas de casi todos los procesos de estabilización química. Frequentemente la humedad óptima no da como resultado ni la mezcla más densa ni el producto resultante más resistente. Los métodos empíricos que muchas veces se utilizan para determinar la cantidad correcta de agua en la estabilización mecánica podrían resultar en el fracaso de un material estabilizado químicamente. Asimismo, ya que el agua toma parte en la reacción química, ciertas impurezas en el agua pueden inhibir la correcta estabilización química.

Un sexto problema es la correcta compactación del material estabilizado. Cada agente estabilizador requiere su propia técnica de compactación. La estabilización química depende del tiempo. El esfuerzo necesario de compactación, la humedad óptima y la densidad seca máxima varían según la demora entre la aplicación y la compactación. La estabilización de cemento y suelo es la más sensitiva a esta demora. La resistencia del producto resultante disminuye rápidamente a medida que se aumenta el tiempo entre el mezclado y la compactación. Si se aumenta el contenido de humedad

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bien les mélanger avec celui-ci avant d'introduire l'adjuvant. Le mode d'emploi de plusieurs engins pour mélanger les différentes sortes de stabilisant, est expliqué en détail dans les textes choisis. Cependant aucune explication, si détaillée soit-elle, ne pourra venir à l'aide de l'ingénieur routier qui va essayer de prendre un raccourci dans cette opération pour soit-disant augmenter la vitesse de production.

Un cinquième problème à considérer est celui de l'eau. Un matériau en train de subir un traitement chimique, ne réagit pas de la même façon à l'eau qu'un sol qui subit la stabilisation mécanique, car l'eau elle même est associée aux réactions chimiques de la plupart des procédés de stabilisation chimique. Souvent la teneur en eau optimale peut ne pas résulter en le mélange le plus dense ou le matériau final le plus solide. Les moyens empiriques qui sont souvent utilisés pour déterminer la teneur en eau nécessaire pour la stabilisation mécanique peuvent avoir pour résultat la rupture d'un matériau stabilisé chimiquement. Le fait que l'eau est un des agents réactifs peut aussi signifier que si l'eau est impure, la stabilisation chimique ne se fera pas correctement.

Le sixième problème concerne le compactage des matériaux stabilisés. Chaque stabilisant exige sa propre technique de compactage. La stabilisation chimique se fait en fonction du temps. L'effort de compactage, la teneur en eau optimale et la densité sèche maximale varient selon les délais entre l'application et le compactage. La stabilisation au ciment des sols est particulièrement affectée par les délais de compactage. La résistance du sol-ciment décroît au fur et à mesure que le temps entre le malaxage et le compactage s'accroît. En augmentant la teneur en eau durant le malaxage on peut quelquefois réduire la perte de résistance causée par ces retards. Cependant, beaucoup d'ingénieurs ne sont pas assez conscients de la nature crucal reactions described in the selected texts continue over a considerable period of time along with a continued strengthening of the end product. Curing techniques vary from material to material. The recommendations given in the selected texts should be followed to ensure that the stabilization project will not be damaged by premature use.

An eighth problem is the maintenance of the stabilized material. The selected texts indicate that chemically stabilized soils are primarily used as base courses. One reason for this is that the chemical stabilization imparts structural strength to the soil, but it does not necessarily improve its ability to withstand the abrasive effects of traffic. Another reason is that chemical stabilization does not, by itself, ensure that the stabilized soil will be impervious. Thus, several of the selected texts recommend that all chemically stabilized soils receive at least a prime coat of bituminous material. This material will assist in curing and also will protect the stabilized soil from traffic and water.

The number and magnitude of the problems described here illustrate the engineering difficulties inherent in the chemical stabilization of soils. Many low-volume road engineers do not have the opportunity to become expert in the art of chemical soil stabilization. The costs involved and the avoidable mistakes made in the past usually cause inexperienced engineers to consider chemical soil stabilization as a last resort.

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durante la operación de mezclado, esto a veces reduce la pérdida de resistencia debida a la demora. Sin embargo, son muchos los ingenieros que no entienden totalmente la crítica importancia de la regulación del tiempo entre mezclado y compactación de suelo-cemento.

Un séptimo problema es el curado del material estabilizado. Mientras que el intercambio de catión y las reacciones de floculación-aglomeración ocurren rápidamente en la estabilización con cal, todas las otras reacciones que se describen en los textos seleccionados continúan durante un período considerable de tiempo junto con un aumento contínuo de resistencia en el producto resultante. Las técnicas de curado cambian de material a material. Se deberán seguir las recomendaciones en los textos seleccionados para evitar daño por el uso prematuro del proyecto de estabilización.

El octavo problema es el mantenimiento del material estabilizado. Los textos seleccionados indican que los suelos estabilizados se utilizan primordialmente para capas de base. Una de las razones es que la estabilización química le da resistencia estructural al suelo pero no necesariamente mejora su resistencia a los efectos desgastadores del tránsito. Otra razón es que la estabilización química por sí sola no asegura que el suelo sea impermeable. Por esta razón los textos seleccionados recomiendan que a los suelos estabilizados se les dé por lo menos una capa de imprimación de material bituminoso. Este material ayudará en el curado y protejerá el suelo estabilizado del tránsito y del agua.

La cantidad y magnitud de los problemas

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ciale de l'intervalle malaxage/compactage des sol-ciments.

Un septième problème concerne la cure du matériau traité. Les réactions cationiques et de coagulation/agglomération se produisent très rapidement dans le traitement à la chaux, mais les autres réactions chimiques qui sont décrites dans les textes choisis continuent à se produire pendant un temps considérable, et à augmenter la résistance du matériau traité. Les méthodes de cure sont différentes selon les matériaux. Les recommandations données dans les textes choisis devraient être suivies très attentivement pour s'assurer que l'opération de stabilisation ne soit pas ruinée par une circulation prématurée.

Le huitième problème à considérer concerne l'entretien des matériaux traités. Les textes choisis indiquent que les matériaux stabilisés chimiquement sont utilisés principalement comme couche de base. Une des raisons pour ce choix est que le traitement chimique augmente la résistance fondamentale du sol mais n'améliore pas nécessairement sa capacité de résistance à l'effet abrasif de la circulation. Une autre raison est que la stabilisation chimique par elle-même n'assure pas l'imperméabilité du sol traité. Par conséquent, plusieurs textes choisis recommandent que tous les sols traités chimiquement reçoivent un enduit d'imprégnation hydrocarboné. Ce traitement hydrocarboné facilitera la cure et protégera le sol stabilisé des effets de l'eau et de la circulation.

Le nombre et l'ampleur des problèmes dont nous venons de parler démontrent les difficultés inhérentes à la stabilisation chimique des sols. L'ingénieur qui construit des routes à faible capacité n'a pas toujours l'opportunité de devenir un expert en l'art de la stabilisation chimique des sols. Les frais en jeu, les erreurs qu'on aurait pû éviter, sont les raisons pour lesquelles les

However, soil stabilization, when done properly, can be a viable option for the road designer and builder.

Discussion of Selected Texts

The first text comprises two excerpts from *Low Cost Roads; Design, Construction and Maintenance* (UNESCO, 1967; English translation, 1971). The first excerpt describes soil stabilization using cement, lime, and bitumen. It presents a general description of soil types that are suitable for stabilization and of the physical factors that limit the use of these soils. The text discusses the ŋormal application percentages of the various stabilizers and the minimum or maximum proportion of fines required by each stabilizer. It also outlines some of the laboratory tests that can be used to evaluate the suitability of soils for stabilization and to determine the strength of stabilized soils.

The second excerpt describes the construction of the roadway. It stresses the shaping of the roadway base because the surface of the base determines the smoothness of the thin bituminous surface applied over stabilized base materials on low-volume roads. It also describes

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aquí presentados nos dan una idea de las dificultades ingenieriles inherentes en la estabilización química de suelos. Muchos ingenieros de caminos de bajo volúmen no tienen la oportunidad de convertirse en expertos de este arte. Por razón de los costos y las equivocaciones que se podrían haber evitado en el pasado, los ingenieros de poca experiencia consideran la estabilización química como último recurso. Sin embargo, la estabilización química correctamente realizada puede ser una opción viable en el diseño y construcción de un camino.

Presentación de los textos seleccionados

El primer texto se compone de dos extractos de *Low Cost Roads; Design, Construction and Maintenance* (Caminos de bajo costo; diseño, construcción y mantenimiento) (UNESCO, 1967; traducción al inglés, 1971). El primer extracto describe la estabilización del suelo utilizando cemento, cal, y betún. Presenta una descripción general de tipos de suelo que son aptos para la estabilización y los factores físicos que limitan el uso de estos suelos. El texto habla sobre los porcentajes normales de aplicación de los diversos estabilizadores y la proporción mínima o máxima de finos requerida por cada estabilizador. Resume algunos de los ensayos de laboratorio que se pueden utilizar para evaluar la aptitud de los suelos para la estabilización y para determinar la resistencia de suelos estabilizados.

El segundo extracto describe la construcción del camino. Se da importancia a la formación de la base del camino, ya que la superficie de la base determina la uniformidad de la fina superficie bituminosa que se aplica sobre materiales

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ingénieurs inexpérimentés considèrent le traitement chimique des sols en dernier recours. Malgré tout, le traitement chimique des sols peut être une option viable lors de la conception et de la construction des routes.

Discussion des textes choisis

Le premier texte est en réalité deux extraits du livre *Low Cost Roads; Design, Construction and Maintenance* (Routes dans les pays en voie de développement; conception, construction et entretien) (UNESCO, 1967; traduction anglaise, 1971). Le premier décrit la stabilisation ou traitement chimique des sols à l'aide de ciment, de chaux, ou de produits bitumineux. Ce texte présente une description générale des genres de sols propices à ce traitement, et les facteurs physiques qui limitent l'emploi de ces sols. Les dosages utilisés normalement, et la proportion minimale ou maximale d'éléments fins requis par chaque stabilisant, sont indiqués. Certains essais en laboratoire, qui sont utilisés pour juger des possibilités de stabilisation d'un sol et déterminer sa résistance après le traitement, sont expliqués.

Le second extrait décrit la construction de la route. On met l'emphase sur le profilage de la couche de base parce que la qualité de la surface de cette couche déterminera la régularité de l'enduit hydrocarboné mince qui constitue le revêtement de surface appliqué sur les matériaux stabilisés des routes à faible capacité. Le procédé de compactage est décrit, et le rapport entre l'énergie de compactage, la teneur en eau optimale et la densité sèche maximale, est expliqué. Le texte passe en revue les diverses façon de mesurer la densité sèche in situ pour déterminer le taux de compactage. Des directions quant au nombre, genre et location des essais nécessaires pour contrôler le compactage sont présentés.

the process of compaction and explains the relationship among compactive effort, optimum moisture content, and maximum dry density. The text discusses the use of field-density tests to determine the amount of compaction. It presents guidelines for the types, numbers, and locations of tests required to control the compaction operation.

This text also addresses the major components of the construction of stabilized soils in both mix-in-place and premixed construction. Special emphasis is placed on the proportioning of the stabilizer and its proper mixing with the material to be stabilized.

The second text, *Soil Stabilization: A Mission Oriented Approach*, is a report that appeared in *Highway Research Record 351* (Highway Research Board, 1971). It describes the Soil Stabilization Index System (SSIS) developed for the U.S. Department of the Air Force. This system assists engineers with limited experience in soil stabilization.

The text presents a series of flow charts that can be used in selecting the type and amount of stabilizer for a given soil. These charts include both chemical stabilization and mechanical stabilization; both alternatives should be evaluated before any decision is made. The SSIS deals with lime, portland cement, bituminous materials, and combinations of these materials. The soils criteria used in the system require determination of relatively simple soil properties.

Other inputs to the system include (a) use fac-

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de base estabilizados en caminos de bajo volúmen. También describe la operación de compactación y explica la relación entre el esfuerzo compactivo, la humedad óptima, y la densidad seca máxima. El texto habla sobre los ensayos de densidad realizados en el campo para determinar lá compactación. Presenta una guía para los tipos, números y ubicaciones de ensayos que se necesitan para controlar la operación de compactación.

Este texto también habla sobre los componentes principales de la construcción de suelos estabilizados, mezclados in situ tanto como premezclados. Se da especial importancia al proporcionamiento del estabilizador y su mezclado correcto con el material a estabilizarse.

El segundo texto, *Soil Stabilization: A Mission Oriented Approach* (Estabilización del suelo: un enfoque práctico), es un informe que apareció en *Highway Research Record 351* (Archivo de Investigación Vial 351, Highway Research Board, 1971). Describe el sistema de índice para la estabilización del suelo (SSIS) desarrollado para el Departamento de la Fuerza Aérea de los Estados Unidos de Norte América. Este sistema asiste a los ingenieros con poca experiencia en la estabilización de suelos.

El texto presenta una serie de diagramas que se pueden utilizar en la selección del tipo y cantidad de estabilizador para un suelo determinado. Estos diagramas incluyen la estabilización química y mecánica; se deberán evaluar ambas alternativas antes de llegar a una deci-

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Les composants les plus importants pour la construction de sols stabilisés soit sur place, soit en centrale, sont examinés. On met l'emphase sur le dosage du stabilisant et le mélange de ce stabilisant avec le matériau à traiter.

Le deuxième texte, *Soil Stabilization: A Mission Oriented Approach* (Stabilisation des sols: une approche systématique), est un rapport paru dans le *Highway Research Record 351* (Highway Research Board, 1971). Ce rapport décrit le Soil Stabilization Index System (SSIS) qui a été développé pour le U.S. Department of the Air Force. Ce système a été conçu pour aider les ingénieurs dont l'expérience est limitée quant à la stabilisation des sols.

Une série de tableaux est présentée pour la sélection du stabilisant et son dosage pour les différents sols. Ces tableaux sont à la fois pour la stabilisation mécanique et la stabilisation chimique puisque les deux alternatives doivent être considerées avant la décision finale. Le SSIS traite de la chaux, du ciment portland, de matériaux hydrocarbonés et de combinaisons de ces matériaux. Les critères de sols utilisés dans ce système éxigent une détermination des caractéristiques des sols assez simple.

Les autres données que l'on peut entrer dans ce système comprennent: (a) facteurs d'utilisation (couche de base ou couche de forme) (b) facteurs climatologiques y compris les températures et les chûtes de pluie (c) facteurs de construction y compris le matériel disponible et le temps de construction et (d) les conditions requises de performance in situ du sol stabilisé. Après avoir entré ces données, on peut obtenir: (a) le stabilisant à utiliser, (b) le dosage du stabilisant, (c) les sols qui ne se prêtent pas à la stabilisation, et (d) l'élimination de la possibilité de stabilisation à cause des conditions climatiques ou du manque de matériel adéquat.

tors (subgrade or base), (b) environmental factors including both rainfall and temperature, (c) construction factors including the type of equipment and the time available, and (d) field performance requirements for the stabilized soil. The output from the system includes (a) the stabilizer to use, (b) the amount of stabilizing material to use, (c) the soils unsuitable for stabilization, and (d) the elimination of the possibility of stabilization due to climatic conditions or lack of appropriate equipment.

It must be stressed that this system is not a substitute for structural pavement design. Furthermore, some of the output is airfield oriented. The complete list of references used to compile this system is included in the selected text. This list will assist the reader in identifying the data that were developed directly for highway construction practices.

The third text is ***State of the Art: Lime Stabilization*** (Circular 180, Transportation Research Board, 1976). This report represents the state of the art in lime treatment based on a comprehensive analysis of current practice and the literature. It contains four major areas of information — soil-lime reactions, properties and characteristics of lime-treated soils, soil-lime mixture design, and lime-stabilization construction.

Lime stabilization can be divided into two separate reactions: Cation exchange and flocculation-agglomeration occur quite rapidly when soil and lime are thoroughly mixed, and

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sión. El SSIS trata con la cal, el cemento Portland, los materiales bituminosos, y las combinaciones de estos materiales. Los criterios utilizados en el sistema requieren la determinación de propiedades relativamente simples de los suelos.

Otros datos de entrada utilizados por el sistema incluyen (a) factores de uso (subrasante o base), (b) factores del medio ambiente, incluyendo la lluvia y la temperatura, (c) factores de construcción, incluyendo el tipo de equipo y el tiempo disponible, y (d) los requisitos de rendimiento en el campo del suelo estabilizado. Los resultados proporcionados por el sistema incluyen (a) el estabilizador que se debe utilizar, (b) la cantidad de material estabilizador que se debe utilizar, (c) los suelos que no son apropiados para la estabilización, y (d) la eliminación de la posibilidad de estabilización debida a condiciones climáticas o falta de equipo apropiado.

Deberá subrayarse que este sistema no reemplaza el diseño estructural del pavimento. Además, algunos de los resultados proporcionados son para uso en campos de aviación. Se incluye en el texto seleccionado la lista completa de referencias que se utilizó para compilar este sistema. Esta lista ayudará en la identificación de los datos que se desarrollaron específicamente para las prácticas de construcción vial.

El tercer texto es ***State of the Art: Lime Stabilization*** (El estado del arte: estabilización con cal, Circular N° 180 del Transportation Research Board, 1976). Este informe representa el estado del arte en el tratamiento con cal, basado en un análisis comprensivo de prácticas actuales y literatura técnica. Contiene cuatro áreas principales de información — las reacciones entre la cal y el suelo, las propiedades y características de los suelos tratados con cal, el diseño de mezclas de cal y suelo, y la construcción con estabilización con cal.

La estabilización con cal se puede dividir en dos reacciones separadas: El intercambio de catión y la floculación-aglomeración ocurren

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Nous appuyons cependant sur le fait que ce système ne peut remplacer le calcul des chaussées. De plus, certains des résultats sont orientés vers la construction des champs d'aviation. La liste complète des références utilisées pour compiler ce système est incluse dans le texte et aidera le lecteur à identifier les données se rapportant directement à la construction routière.

Le troisième texte est *State of the Art: Lime Stabilization* (Etat de la question: stabilisation à la chaux) (Circular 180, Transportation Research Board, 1976). Ce rapport présente l'état de la question sur le traitement à la chaux, en prenant pour base une analyse d'ensemble des méthodes actuelles et de la littérature technique. Il contient quatre sujets principaux: réactions entre le sol et la chaux, propriétés et caractéristiques des sols traités à la chaux, le calcul des mélanges sol/chaux, et la construction avec le traitement à la chaux.

Le mode d'action de la chaux peut être divisé en deux parties: un échange cationique accompagné d'une réaction de floculation/agglomération, qui se produisent très rapidement quand la chaux et le sol sont mélangés intimement, et des réactions pouzzolaniques sol/chaux qui sont fonction du temps et sont influencées principalement par les propriétés du sol en train d'être traité.

Quand on leur adjoint de la chaux, pratiquement tous les sols fins démontrent une diminution de leur plasticité, une amélioration de leur soil-lime pozzolanic reactions are time dependent and are influenced primarily by the natural properties of the soil being stabilized.

Practically all fine-grained soils exhibit decreased plasticity and improved workability and volume-change characteristics when mixed with lime. However, not all soils exhibit improved strength, stress-strain, and fatigue characteristics. The properties of lime-soil mixtures depend on many variables. The most important variables are (a) soil type, (b) lime type, (c) lime percentage, and (d) curing conditions, including time, temperature, and moisture. More important, however, is the fact that these variables are interdependent. The effect caused by a change in a given variable depends on the levels of the other variables.

The major objective of the mixture design process is to establish an appropriate lime content for construction. Design-lime contents generally are based on an analysis of the effect of varying lime percentages on selected engineering properties of the soil-lime mixture. It is important to note that different design-lime contents for the same soil may be established depending on the objectives of the lime treatment, i.e., pavement structure or subgrade and the mixture design procedure chosen. Mixture design procedures should be flexible enough to allow the exercise of judgment when unusual stabilization objectives are contemplated.

There are essentially three recognized lime stabilization methods — in-place mixing, plant mixing, and pressure injection. In-place mixing

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rápidamente cuando el suelo y la cal se mezclan a fondo, y las reacciones puzolánicas de suelo y cal dependen del tiempo y son influenciadas principalmente por las propiedades naturales del suelo que se está estabilizando.

Casi todos los suelos de grano fino, cuando se mezclan con cal, demuestran una reducción de plasticidad y mejoría en manejabilidad y en las características de cambio de volúmen. Sin embargo, no todos los suelos demuestran mejoría en las características de resistencia, esfuerzo-deformación, y fatiga. Las propiedades de las mezclas de cal-suelo dependen de muchos factores variables. Los factores más importantes son (a) tipo de suelo, (b) tipo de cal, (c) porcentaje de cal, y (d) condiciones de curado, incluyendo el tiempo, temperatura, y humedad. Aún más importante es el hecho de que estos factores dependen el uno del otro. El efecto causado por un cambio en un factor determinado depende de los niveles de los otros factores.

El objetivo principal del proceso de diseño de la mezcla es el de establecer un contenido apropiado de cal para la construcción. Los contenidos de cal de diseño se basan generalmente en un análisis del efecto de porcentajes variables de cal sobre ciertas propiedades ingenieriles de la mezcla de cal-suelo. Es importante observar que se pueden establecer distintos contenidos de cal de diseño para el mismo suelo, dependiendo de los objetivos del tratamiento con cal, es decir, estructura de pavimento o subrasante, y el procedimiento escogido de diseño de mezcla. Los procedimientos de diseño de mezcla deberán ser lo suficientemente flexibles para permitir el uso del juicio profesional cuando se tienen objetivos excepcionales de estabilización.

Hay esencialmente tres métodos reconocidos de estabilización con cal — mezclado in situ, mezclado en planta, e inyección a presión. El mezclado in situ puede subdividirse en tres métodos: (a) mezclado de la cal con los

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maniabilité et de leurs caractéristiques de gonflement. Cependant, tous les sols ainsi traités ne montrent pas nécessairement une amélioration de leurs caractéristiques de résistance, tension-déformation et fatigue. Les propriétés des mélanges sols/chaux sont fonction de beaucoup de variables. Les plus importantes sont: (a) le genre de sol, (b) le genre de chaux, (c) le pourcentage de chaux, et (d) les conditions de la cure (temps, température et humidité). Plus important encore est le fait que ces variables sont fonction l'une de l'autre, et que l'effet causé par le changement d'une variable donnée est fonction du degré de sensibilité des autres variables. L'établissement de critères pour le mélange chaux/sol est expliqué. L'objectif principal en ce cas, est de trouver le dosage approprié de chaux pour la construction. Les critères de choix des chaux sont basés sur une analyse de l'effet de différents pourcentages de chaux sur certaines propriétés routières du mélange sol/chaux. Il faut signaler ici que différents critères de choix pour le même sol peuvent être nécessaires selon l'objectif du traitement à la chaux, c'est à dire si l'on va utiliser le mélange pour la fondation ou le revêtement, et selon la méthode de mélange choisie. Les méthodes de mélange devraient d'ailleurs être assez souples pour permettre l'exercice du jugement professionel quand on envisage le traitement à des fins inhabituelles.

Essentiellement, il existe trois méthodes de may be subdivided into three methods: (a) mixing lime with the existing materials already a part of the construction site or pavement, (b) off-site mixing in which lime is mixed with borrow and the mixture is then transported to the construction site for final manipulation and compaction, and (c) mixing in which the borrow-source soil is hauled to the construction site and processed. This section attempts to describe and summarize modern soil-lime construction procedures and equipment.

The fourth text, *Field Studies on the Pulverization of Black Cotton Soil for the Construction of Stabilized Soil Road Bases*, was published in *Highway Research Record 315* (Highway Research Board, 1970). It stresses the importance of properly pulverized soil as a prerequisite for proper soil stabilization. Black cotton soil (expansive black or dark gray clay) can be effectively improved using lime as a stabilizer. However, black cotton soil is particularly difficult to pulverize. This paper describes a number of methods that have been tried in the field to achieve an economical and effective pulverization method. It shows that an acceptable degree of pulverization can be attained using agricultural machinery, if the soil is within certain moisture content limits.

The fifth text is excerpted from *Soil-Cement: A Material of Construction for Road and Airfield Pavements* (Building and Road Research Institute, Ghana Academy of Sciences, 1967). This text states that the combination of soil with cement under controlled conditions of moisture and density produces a material of distinct physical and engineering characteristics. These

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materiales existentes que ya forman parte de la zona de construcción o el pavimento, (b) mezclado de la cal con suelo de préstamo en una ubicación fuera de la zona de construcción, transporte de la mezcla hasta la zona de construcción para la manipulación final y la compactación, y (c) mezclado en que el suelo de préstamo se transporta hasta la zona de construcción donde es procesado. Esta sección intenta describir y resumir los procedimientos y equipo modernos en la construcción con cal y suelo.

El cuarto texto, *Field Studies on the Pulverization of Black Cotton Soil for the Construction of Stabilized Soil Road Bases* (Estudios de campo sobre la pulverización de suelo "black cotton" para la construcción de bases de camino de suelo estabilizado), fué publicado en *Highway Research Record 315* (Archivo de Investigación Vial 315, Highway Research Board, 1970). Subraya la importancia de una correcta pulverización del suelo como requisito previo a su estabilización. El suelo "black cotton" (arcilla expansiva negra o gris oscura) puede mejorarse eficazmente utilizando la cal como estabilizador. Sin embargo, el suelo "black cotton" es particularmente difícil de pulverizar. El informe describe varios métodos que se han utilizado en el campo para obtener un método de pulverización económico y eficaz. Demuestra que se puede obtener un grado satisfactorio de pulverización con maquinaria agrícola si el suelo se encuentra dentro de ciertos límites de contenido de humedad.

El quinto texto fué extraído de *Soil-Cement: A Material of Construction for Road and Airfield Pavements* (Suelo-cemento: un material de construcción para pavimentos de camino y aeropuerto, Building and Road Research Institute, Ghana Academy of Sciences, 1967). Este texto afirma que la combinación de suelo con cemento bajo condiciones controladas de

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stabilisation à la chaux: le mélange en chantier, le mélange en centrale, et l'injection de cartouches de chaux. Le mélange en chantier peut être fait de trois façons: (a) on mélange la chaux au matériau existant et faisant déjà partie de la route, (b) on mélange la chaux au matériau d'emprunt et ensuite on transporte le mélange au chantier pour la construction finale et le compactage, et (c) on transporte le matériau d'emprunt jusqu'au chantier et on le traite à la chaux selon le procédé utilisé en (a). Les méthodes de traitement à la chaux, ainsi que le matériel utilisé à ces fins, sont passés en revue dans la dernière partie de ce texte.

Le quatrième écrit, *Field Studies on the Pulverization of Black Cotton Soil for the Construction of Stabilized Soil Road Bases* (Etudes sur le chantier de la pulvérisation des terres noires pour la construction de couches de base en sol stabilisé), fut publié dans le *Highway Research Record 315* (Highway Research Board, 1970). Cette communication met l'emphase sur la nécessité de pulvériser correctement le sol au préalable pour obtenir une stabilisation convenable. Les terres noires (argile expansive noire ou gris foncée) peuvent être améliorées efficacement quand on utilise la chaux comme stabilisant. Cependant, les terres noires sont particulièrement difficiles à pulvériser. Cette communication décrit un certain nombre de procédés qui ont été utilisé en chantier pour effectuer une pulvérisation efficace et économique. Il est démonproperties depend on four main factors: (a) the nature of the soil, (b) the proportion of soil, cement, and water in the mixture, (c) the compactive energy used for the molding of the soil-cement, and (d) the physical conditions, such as the curing temperatures and age of the soil-cement mixes.

The text discusses such properties of soil-cement as volume changes, thermal expansion, permeability, and strength characteristics. It emphasizes the necessity of a soil survey of the site in the planning of a cement stabilization project and presents the criteria for selecting soils that are suitable for cement stabilization. It describes the important factors in the stabilization of soil with cement, including the moisture-density relationship and the durability of the soil-cement mix. It defines soils requiring special treatment such as organic soils and soils contaminated with sulphates. The text also discusses the stress-strain characteristics of soil-cement. It analyzes the major factors affecting the strength of soil-cement mixtures, such as the influence of the cement, the effect of age, and the effect of curing temperatures.

The text describes the design of road and airfield pavements using soil-cement. It evaluates the basic concepts of design considering soil-cement as a rigid material and as a flexible material. It concludes that design, using either concept, is overly conservative because soil-cement — basically a flexible material — shares

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humedad y densidad produce un material de características distintivas físicas e ingenieriles. Estas propiedades dependen de cuatro factores principales: (a) la naturaleza del suelo, (b) las proporciones de suelo, cemento, y agua en la mezcla, (c) la energía compactiva utilizada para el moldeo del suelo-cemento, y (d) las condiciones físicas, tales como las temperaturas de curado y la edad de las mezclas de suelo-cemento.

El texto examina algunas propiedades de suelo-cemento tales como cambios de volúmen, expansión termal, permeabilidad, y características de resistencia. Subraya la necesidad de una investigación del suelo de la ubicación, en el planeamiento de un proyecto de estabilización con cemento, y presenta los criterios para la selección de suelos adecuados para la estabilización con cemento. Describe los factores importantes en la estabilización del suelo con cemento, incluyendo la relación humedad-densidad y la durabilidad de la mezcla de suelo-cemento. Define los suelos que necesitan un tratamiento especial, tales como los suelos orgánicos y suelos contaminados con sulfatos. El texto también examina las características de esfuerzo-deformación del suelo-cemento. Analiza los factores principales que afectan la resistencia de mezclas de suelo-cemento, tales como la influencia del cemento, el efecto de la edad, y el efecto de las temperaturas de curado.

El texto describe el diseño de pavimentos de camino y aeropuerto utilizando el suelo-cemento. Evalúa los conceptos básicos de diseño considerando el suelo-cemento como un material rígido y como un material flexible. Concluye que el diseño, si se utiliza cualquiera de estos dos conceptos, es demasiado restrictivo ya que el suelo-cemento — básicamente un material flexible — posee algunas de las características de un material rígido.

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tré que l'on peut arriver à un degré de pulvérisation satisfaisant en utilisant du matériel agricole, si la teneur en eau du matériau est entre certaines limites.

Le cinquième texte est extrait de ***Soil-Cement: A Material of Construction for Road and Airfield Pavements*** (Le sol-ciment: un matériau de construction pour routes et pistes d'aérodromes, Building and Road Research Institute, Ghana Academy of Sciences, 1967). Ce texte affirme que la combinaison de sol et de ciment, sous certaines conditions contrôlées d'humidité et de densité, résulte en un matériau aux caractéristiques physiques et de construction très précises. Ces caractéristiques sont en fonction de (a) la nature du sol, (b) les proportions de sol, ciment et eau dans le mélange, (c) l'énergie de compactage utilisée pour le moulage du sol-ciment, et (d) les conditions physiques, comme la température de cure et l'âge des mélanges sol-ciment.

Le texte discute certaines propriétés des sols-ciment telles que: les variations de volume, l'expansion thermique, la perméabilité et les caractéristiques de résistance. On met l'emphase sur la nécessité de faire une reconnaissance sur les lieux quand on envisage la stabilisation au ciment, et les critères de choix des sols qui se prêtent à ce traitement sont présentés. Les facteurs importants dans le traitement des sols par le ciment sont le rapport entre l'humidité et la densité, et la durabilité du mélange sol-ciment. Les sols qui ont besoin d'un traitement spécial sont les sols organiques et les sols contaminés some of the characteristics of a rigid material.

The text presents the criterion for mix design. It also describes soil-cement construction methods, including both mix-in-place and plant-mix construction. It includes tables on typical equipment requirements for different types of mixing machines and the steps in construction procedures for different types of mixing equipment. The text also includes a detailed section describing quality control in the field for soil-cement construction. Scientific control in the field should involve subgrade conditioning; selection of soil; checking the quality of cement and water; mix proportion; and mixing, laying, compacting, and curing processes.

The sixth text, *Variation in Laboratory and Field Strengths of Soil-Cement Mixtures*, appeared in *Transportation Research Record 560* (Transportation Research Board, 1976). It evaluates the effect of variations of laboratory

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El texto presenta los criterios para el diseño de la mezcla. También describe métodos de construcción con suelo-cemento incluyendo el mezclado in situ y el mezclado en planta. Incluye tablas sobre el equipo típicamente utilizado con diversos tipos de máquinas de mezclado, y las etapas en los procedimientos de construcción con diversos tipos de equipo de mezclado. El texto también incluye una sección detallada que describe el control de calidad en el campo para la construcción con suelo-cemento. El control científico en el campo debería incluir el acondicionamiento de la subrasante, la selección del suelo, verificación de la calidad del cemento y agua, la proporción del mezclado, y los procesos de mezclado, colocación, compactación y curado.

El sexto texto, *Variation in Laboratory and Field Strengths of Soil-Cement Mixtures* (La variación en las resistencias de mezclas de suelo-cemento en el laboratorio y en el campo), apareció en *Transportation Research Record 560* (Archivo de Investigación Vial 560, Transportation Research Board, 1976). Evalúa el efecto de variaciones de los procedimientos de diseño en el laboratorio y compara las resistencias verdaderas en el campo con las resistencias de diseño en el laboratorio. Concluye que pueden ocurrir inconsistencias en el diseño de laboratorio si no se adhiere cuidadosamente a factores como moldeo, temperatura, hidratación, y tiempo de fabricación. Asimismo, concluye que el mezclado in situ de cemento y suelo puede producir una variación de 5 por ciento del contenido teórico de cemento.

El séptimo texto, *Changes in the Characteristics of Cement-Stabilized Soils by Addition of Excess Compaction Moisture* (Cambios en las características de suelos estabilizados con cemento por la adición de excesiva humedad de compactación), apareció en *Highway Research Record 315* (Archivo de Investigación Vial 315, Highway Research Board, 1970). Indica que la humedad de compactación de los suelos estabilizados con cemento normalmente se especifica como la humedad óptima para ob-

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par les sulfates. Les caractéristiques "tension-déformation" des sols ciments sont aussi discutées. Les facteurs les plus importants qui influencent la résistance des sols-ciments sont analysés: l'influence du ciment, l'effet de l'âge, et l'effet de la température de la cure.

Ce texte décrit aussi la conception de routes et de pistes d'aérodrome à l'aide de sol-ciment. Les concepts de base de calcul en utilisant le sol-ciment comme un matériau rigide ou un matériau souple sont évalués. On conclut que les méthodes de calcul de l'un ou l'autre concept sont trop modérées, car le sol-ciment qui est au fond un matériau souple, partage quelques caractéristiques d'un matériau rigide.

Les critères pour le calcul du mélange sont présentés. Les méthodes de construction du sol-ciment sont décrites et comprennent le mélange en chantier et le mélange en centrale. Il y a un tableau sur l'equipement nécessaire pour différentes sortes de machines mélangeuses, et un autre qui éxplique pas à pas la marche à suivre pour la construction en se servant de ces machines mélangeuses. Une section détaillée sur le contrôle de qualité en chantier est incluse. Ce contrôle doit comprendre: la préparation du sous sol; le choix du sol et la vérification de la qualité du ciment et de l'eau; le dosage du mélange; et les procédés de malaxage, de répandage, de compactage et de cure.

Le sixième texte, *Variation in Laboratory and Field Strengths of Soil-Cement Mixtures* (Variations de la résistance des mélanges sols-ciment selon les essais au laboratoire et sur le chantier) a été publié dans *Transportation Research Record 560* (Transportation Research Board, 1976). Ce texte évalue l'effet des variations qui existent entre les méthodes d'essais des laboratoires et compare la résistance actuelle obtenue sur le chantier à la résistance obtenue lors des design procedures and compares actual field strengths with laboratory design strengths. It concludes that inconsistency can occur in laboratory design if factors such as molding, temperatures, slaking, and fabrication time are not closely adhered to. The text further concludes that in-place mixing of cement with soil can show a variation of ±5 percent from the theoretical cement content.

The seventh text, *Changes in the Characteristics of Cement-Stabilized Soils by Addition of Excess Compaction Moisture*, was reported in *Highway Research Record 315* (Highway Research Board, 1970). It indicates that the compaction moisture of cement-stabilized soils is usually specified as the optimum moisture content to obtain maximum density according to the standard Proctor test. In some instances maximum density may not correspond to maximum strength. If compaction of the soil-cement mix is delayed, the relationship between compaction moisture and the strength and den-

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tener la densidad máxima de acuerdo con el ensayo Proctor. En algunos casos la densidad máxima podría no corresponder a la resistencia máxima. Si se demora la compactación de la mezcla de suelo-cemento, la relación entre la humedad de compactación y la resistencia y densidad del suelo-cemento también cambia.

El texto llega a la conclusión de que la pérdida de resistencia y durabilidad del suelo-cemento causada por una demora en la compactación puede reducirse considerablemente en muchos casos con la adición de excesiva humedad de compactación. La resistencia de marga aluvial y marga arenosa mezcladas con cemento se incrementó en un 40-50 por ciento con la adición de un 2-4 por ciento de excesiva humedad cuando se demoró la compactación. La marga aluvial arcillosa y el limo estabilizados con cemento y compactados después de demoras, demostraron poca mejoría en resistencia y durabilidad cuando se añadió excesiva humedad de compactación. Sin embargo, sí mejoraron las margas aluviales arcillosas con la adición de excesiva humedad sin demora en compactación.

El texto indica que la cantidad de excesiva humedad que se requiere para máxima resistencia y durabilidad depende del tipo de suelo y el tiempo de demora entre mezclado y compactación. Excesiva humedad mejora la lubricación de los agregados granulares de suelo después de demorarse la compactación, con un aumento subsecuente de la densidad seca. La humedad excesiva mejora las propiedades de las mezclas de suelo-cemento de grano fino compactadas sin demora, porque aumenta la hidratación del cemento.

El octavo texto fué extraído de *Bituminous Bases and Surfacing for Low Cost Roads in the Tropics* (Bases y revestimientos bituminosos para caminos de bajo costo en los trópicos, Transport and Road Research Laboratory, G.B.,

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essais en laboratoire. En conclusion, des variations peuvent exister dans les résultats des essais en laboratoire si on n'adhère pas étroitement aux facteurs de moulage, température, extinction et temps de construction. Pour terminer, le mélange en chantier du sol-ciment peut différer de 5 pour cent du dosage théorique du ciment.

Le septième texte, *Changes in the Characteristics of Cement Stabilized Soils by Addition of Excess Compaction Moisture* (Modification des caractéristiques des sols-ciments par addition d'eau excédentaire pendant le compactage), a été publié dans le *Highway Research Record 315* (Highway Research Board, 1970). Il est indiqué que la teneur en eau de compactage pour les sols traités au ciment est d'ordinaire la teneur en eau optimale pour obtenir une densité maximale selon l'essai Proctor. Cependant, dans certains cas, la densité maximale ne résulte pas en la résistance maximale. Si le compactage du sol-ciment est retardé, le rapport entre la teneur en eau de compactage et la résistance et densité du sol-ciment, changent.

La perte de résistance et de durabilité qui résulte d'un d'un délai de compactage, peut être réduite considérablement si l'on ajoute un excédent d'eau au moment du compactage. La résistances des sols-ciment de marne limoneuse et de marne sableuse est augmentée de 40 à 50 pour cent quand on ajoute 2 à 4 pour cent d'excédent d'eau quand la compaction est retardée. Par contre la marne argilo-limoneuse et les silts traités au ciment et compactés après un délai ne montrent guère d'amélioration en résistance et durabilité quand on leur ajoute un excédent d'eau. Cependant, si l'on ajoute cet excédent d'eau et que l'on compacte immédiatement, leur résistance et durabilité se trouvent grandement améliorées.

Le texte indique que la quantité d'excédent d'eau exigée pour obtenir la plus grande résistance et durabilité, dépend du genre de sol et de la période de temps entre le malaxage et le sity of the soil-cement also changes.

The text concludes that the loss in strength and durability of soil-cement resulting from a delay in compaction can be significantly reduced in many instances by the addition of excess compaction moisture. The strength of silty loam and sandy loam soil-cements was increased by 40 to 50 percent by adding 2 to 4 percent excess moisture when compaction was delayed. Cement-stabilized silty clay loams and silts compacted after delays showed little improvement in strength and durability with excess compaction moisture. However, the silty clay loams were significantly improved in strength and durability by the addition of excess compaction moisture with no delay in compaction.

The text indicates that the amount of excess moisture required for maximum strength and durability depends on the soil type and the delay time between mixing and compaction. Excess moisture improves the lubrication of granular soil aggregates after delays in compaction with a subsequent increase in dry density. Excess moisture improves the properties of fine-grained soil-cement mixes compacted without delay by increasing the amount of cement hydration.

The eighth text is excerpted from ***Bituminous bases and surfacings for low-cost roads in the tropics*** (Transport and Road Research Laboratory, U.K., 1977). It describes full-scale experimental trials supported by laboratory research, which have prompted proposals of acceptance criteria for bitumen-stabilized sand bases for light-to-medium traffic. It stresses the fact that the success of bituminous soil stabilization depends on the correct selection of both the type and grade of bituminous binder. Selection of the process itself involves a consideration of other

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1977). Describe ensayos experimentales realizados en escala natural y apoyados por investigación en el laboratorio, que han instigado propuestas de criterios de aceptación para bases de arena estabilizadas con betún para tránsito liviano-mediano. Subraya que el éxito de la estabilización bituminosa del suelo depende de la correcta selección del tipo y clasificación de aglutinante bituminoso. La selección del proceso mismo exige la consideración de otros factores, tales como el tipo o tratamiento del material, la planta, y el medio ambiente. Menciona que los valores mínimos del CBR (California Bearing Ratio) empapado, que normalmente se utilizan como criterio para la estabilización de cal y cemento, presentan problemas en el diseño de la estabilización bituminosa de suelos. Esto es porque las mezclas bituminosas reaccionan viscoelásticamente y son susceptibles a la temperatura. El informe recomienda que el diseño de las bases estabilizadas con betún dependa de la estabilidad de la mezcla, y

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compactage. Un excédent d'eau augmente la lubrification des agrégats de matériau granuleux après les délais de compactage, avec une augmentation subséquente de la densité sèche. L'excédent d'eau améliore les caractéristiques des mélanges sols-ciment à grains fins qui sont compactés immédiatement en augmentant l'hydratation du ciment.

Le huitième texte est tiré de ***Bituminous bases and surfacings for low-cost roads in the tropics*** (Bases et revêtements hydrocarbonés pour routes économiques des tropiques, Transport and Road Research Laboratory, U.K., 1977). Il décrit des essais expérimentaux à grande échelle, appuyés par la recherche en laboratoire, qui ont permis de proposer des critères de réception pour les couches de base en sable stabilisées avec un liant hydrocarboné pour les routes à circulation faible ou moyenne. L'emphase est mise sur le fait que la réussite du traitement aux liants bitumineux dépend de la sélection appropriées de la classe et de la nature de ce liant. Le choix du porcédé de stabilisation lui-même implique l'examen d'autres facteurs tels que le genre de matériau et de traitement, la fabrication et les conditions climatiques. Il est noté que les valeurs CBR minimales après imbibition qui sont normalement utilisées comme critères pour le traitement à la chaux ou au ciment, présentent des problèmes dans le calcul du traitement aux liants bitumineux. Ceci provient du fait que ces materiaux bitumineux sont susceptibles, c'est à dire que leur viscosité varie en fonction de la température. Le calcul des bases stabilisées aux liants bitumineux dépend de la stabilité du mélange. Ce rapport suggère comme valeurs minimales pour les routes à faible circulation:

Stabilité Marshall à 60° C .... 100 kg
ou
Stabilité Hubbard-Field à 60° C .... 300 kg

Le neuvième texte, ***Performance Study of*** features, such as material type or treatment, plant, and environment. It notes that the minimum soaked California Bearing Ratio (CBR) values, normally used as a criterion for lime and cement stabilization, present problems in the design of bituminous soil stabilization. This is because bituminous mixtures behave viscoelastically and are temperature susceptible. The report recommends that the design of bituminous-stabilized bases depend on the stability of the mixture and suggests these minimum values for lightly trafficked roads:

Stability, Marshall at 60° C .... 100 kg
or
Stability, Hubbard-Field at 60° C ... 300 kg

The ninth text, ***Performance Study of Asphalt Road Pavement with Bituminous-Stabilized-Sand Bases***, was published in ***Transportation Research Record 641*** (Transportation Research Board, 1977). It describes the construction of an experimental road using aeolian (windblown) sand stabilized with bitumen material as a base course. Several bituminous stabilizers were used, i.e., two cutback bitumens, a cationic bitumen emulsion, and cutback tar. The cutback bitumens and the emulsion were each incorporated in at least two different percentages. Also, a 15 percent (by volume) calcareous filler was added to the windblown sand before addition of the bituminous binders in some of the test sections. The setting up of the bitumen-sand mixtures on the road was studied over a period of 2 years under a light sand seal before a 30-mm-thick asphalt-concrete surface was placed.

The text describes (a) the details of the experiment, (b) the materials used, (c) the techniques of construction, (d) the laboratory control during

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sugiere los siguientes valores mínimos para caminos con volúmen liviano de tránsito:

Estabilidad, Marshall a 60° C ...... 100 kg
o
Estabilidad, Hubbard-Field a 60° C . 300 kg

El noveno texto, ***Performance Study of Asphalt Road Pavement with Bituminous-Stabilized-Sand Bases*** (Estudio de rendimiento de pavimento de asfalto con bases de arena estabilizadas con betún), fué publicado en ***Transportation Research Record 641*** (Archivo de Investigación Vial 641, Transportation Research Board, 1977). Describe la construcción de un camino experimental utilizando arena eólica (soplada por el viento), estabilizada con material bituminoso, como capa de base. Se utilizaron varios estabilizadores bituminosos, es decir, dos betunes fluidificados (cutback), una emulsión bituminosa catiónica, y alquitrán fluidificado. Los betunes fluidificados y la emulsión fueron incorporados cada uno en por lo menos dos porcentajes distintos. Además, se añadió un relleno calcárea de 15% (por volúmen) a la arena eólica antes de añadir los aglutinantes bituminosos en algunas de las secciones de ensayo. El establecimiento de las mezclas de betún-arena sobre el camino se estudió durante dos años bajo un sello liviano de arena antes de colocar una superficie de asfalto-hormigón de 30 mm de espesor.

El texto describe (a) los detalles del experimento, (b) los materiales utilizados, (c) las técnicas de construcción, (d) el control de laboratorio durante la colocación de las capas de base experimentales, (e) los estudios de campo realizados después de completarse la capa de base y después de la colocación del pavimento, (f) las conclusiones hechas al final

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Asphalt Road Pavement with Bituminous-Stabilized-Sand Bases (Etude du comportement des chaussées en asphalte avec base de sable stabilisé aux liants hydrocarbonés), fut publié dans le ***Transportation Research Record 641*** (Transportation Research Board, 1977). La construction d'une route expérimentale dont la couche de base est faite de sable éolien stabilisé aux liants bitumineux est décrite. Plusieurs stabilisants bitumineux ont été utilisés: deux bitumes fluidifiés ou cut-backs, une émulsion cationique et du goudron fluidifié. Les cut-backs et l'émulsion cationique ont été réspectivement mélangés à au moins deux dosages différents. De plus, dans certaines sections de la route, on a ajouté 15 pour cent (volume) de filler calcaire au sable éolien avant l'adjunction des liants bitumineux. Le comportement de prise des mélanges sable/bitume recouverts d'une mince couche de sable, a été étudié pendant deux ans, avant la pose finale d'un revêtement de 30 mm de béton asphaltique.

Le texte décrit (a) les détails de l'étude, (b) les matériaux utilisés, (c) les méthodes de construction, (d) les examens de contrôle au laboratoire pendant la construction des diverses couches de base expérimentales, (e) les études sur le chantier après la construction de ces couches de base, et aprés avoir revêtu la chaussée, (f) les conclusions de cette étude, et (h) l'extrapolathe laying of the experimental base courses, (e) the field studies carried out after completion of the base course and after the placing of the pavement, (f) the conclusions drawn from the study, and (h) the extrapolation of performance results to heavier traffic conditions.

The construction method, known as the wet-sand process of bituminous stabilization, included the addition of water to the sand and sand-filler blend before the additon of the binder to increase the density and shear strength of the finished product. The arbitrary compaction of this wet sand (moisture content approximately 10-12 percent before stabilizing with binder) also improved the inherently poor stability of the dry sand. As a result, the stabilization plant was able to move over the sand bed at the required speed without undue slippage. The optimum condition for compaction of the stabilized material was determined by means of a vane-shear apparatus.

Bibliography

The selected texts are followed by a brief bibliography containing reference data and abstracts for 24 publications. The first nine describe the selected texts. The other 15 describe publications related to the selected texts. Although there are many articles, reports, and books that could be listed, it is not the purpose

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del estudio, y (h) la extrapolación de los resultados de rendimiento a condiciones de tránsito de mayor volúmen.

El método de construcción, que se conoce como el proceso de arena mojada de estabilización bituminosa, incluye la adición de agua a la arena y la mezcla de arena-relleno antes de añadir el aglutinante, para aumentar la densidad y resistencia al esfuerzo cortante del producto terminado. La compactación arbitraria de esta arena mojada (cuyo contenido de humedad es aproximadamente 10-12 por ciento antes de la estabilización con el aglutinante) también mejoró la estabilidad pobre inherente de la arena seca. Como resultado, la planta estabilizadora pudo trasladarse sobre la base de arena a la velocidad requerida sin excesivo resbalamiento. La condición óptima para la compactación del material estabilizado fué determinada por medio de una veleta.

Bibliografía

Los textos seleccionados son seguidos por una breve bibliografía que contiene los datos de referencia y abstractos para 24 publicaciones. Los primeros nueve describen los textos seleccionados. Los otros 15 describen publicaciones que se relacionan a ellos. Aunque hay muchos artículos, informes, y libros que se podrían nombrar, no es el propósito de esta bibliografía con-

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tion des résultats obtenus à des conditions de circulation plus intense.

La technique utilisée pour la construction, connue sous le nom de "wet sand process" (procédé au sable humide) demande l'adjunction d'eau au sable et au mélange sable/filler avant l'adjunction du liant pour augmenter la densité et la résistance au cisaillement du matériau final. Le compactage, arbitrairement décidé, de ce mélange de sable humide (teneur en eau d'à peu près 10-12 pour cent avant stabilisation avec le liant) améliora aussi la stabilité, plutot médiocre, du sable sec, et permit de pouvoir faire avancer l'enrobeur automoteur-autochargeur à la vitesse désirée sans que les roues glissent à vide. La condition optimale de compactage du matériau stabilisé a été determinée à l'aide de l'appareil à palettes.

Bibliographie

Les textes choisis sont suivis d'une brève bibliographie contenant des données de références et des analyses de 24 publications. Les neuf premières décrivent les textes choisis. Les autres quinze décrivent des publications apparentées au thème des textes choisis. Bien qu'il y ait beaucoup d'autres articles, rapports et livres qui pourraient être inclus, l'objectif de cette biblioof this bibliography to contain all possible references related to the subject of this compendium. The bibliography contains only those publications from which a text has been selected or basic publications that would have been selected had there been no page limit for this compendium.

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tener todas las referencias relacionadas con el tema de este compendio. Contiene únicamente aquellas publicaciones de las cuales se seleccionó un texto o las publicaciones básicas que se hubieran seleccionado si no hubiera un límite al número de páginas para este compendio.

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graphie n'est pas d'énumérer toutes les références possibles ayant rapport au sujet de ce recueil. Donc, cette bibliographie, telle qu'elle, se rapporte seulement aux publications dont nous avons choisi des extraits, ou aux textes de base que nous aurions choisis aussi, s'il n'y avait pas de limites quant au nombre de pages de ce recueil.

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LOW COST ROADS

DESIGN, CONSTRUCTION AND MAINTENANCE

Drafted by a group of international experts
L. ODIER, R. S. MILLARD,
PIMENTEL dos SANTOS, S. R. MEHRA
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CONTENTS

NOTE: The deleted material does not pertain to chemical stabilization.

6 4.4.2.3 *Stabilised soil* (cement, lime and bitumen)

Soil stabilisation offers one of the best forms of base construction for use in tropical and sub-tropical countries. A wide variety of soils may be used. Apart from organic soils and some other rather rare soils that because of chemical contamination react unfavourably with cement and lime, the only physical factors limiting the use of soils for stabilisation are:

(i) that it must be possible to break the soil to a fine tilth in order to mix in the stabiliser, and
(ii) that the soil should have an adequately stable grading.

A suitable criterion is that the coefficient of uniformity should be greater than 5 and preferably greater than 10. The coefficient of uniformity is the ratio of the sieve size through which 60% of the material passes to the sieve size through which 10% of the material passes.

The important characteristic of stabilisation with cement and lime is that the relatively high strengths that may be obtained with dry compacted soils are retained by the mixtures when they become wet. The mixtures, particularly with cement, may attain strengths considerably in excess of those of the compacted dry soil. High strengths are not necessarily an advantage since they are accompanied by shrinkage and cracking to relieve internal stresses. Thus the proportion of cement or lime required normally ranges between 3 and 7% by weight of the dry soil.[10]

With lime stabilisation, clay minerals are necessary in the soil for the stabilisation reaction to proceed. A useful working limit is that the soil should contain at least 15% of fines passing the 425 μm (BS. 36) sieve and have a plasticity index of at least 10%. Cement may be used to stabilise both plastic and non-plastic soils.

The suitability of soils for stabilisation with cement or lime is judged on the results of laboratory tests. The simplest test is a simple wetting and drying test to judge whether mixtures retain their strength when wetted. Quite useful results can be obtained by subjective assessment using hand and eye. A form of this test has been recommended by the Portland Cement Association and adopted as standard by the American Society for Testing and Materials.[11]

In testing the strength of stabilised soils the C.B.R. test is widely used for testing gravelly soils whereas the unconfined compression test is often used with finer textured soils. Both tests are often made at an age of seven days comprising three days moist curing followed by four days immersion in water. Since the strength of stabilised soil mixtures is critically dependent on temperature, where strict comparability between results is needed the temperature should be maintained constant ($\pm$ 2°C) during these seven days. The curing temperature selected in tropical countries will normally be between 25°C and 35°C, the precise level being that at which it is possible to maintain an even temperature in a laboratory without refrigeration.

Guiding criteria for laboratory tests are as follows:

(a) *C.B.R. Test.* A C.B.R. value of 80–100% at the density to which the soil will be compacted in the field.

(b) *Unconfined Compression Test.* Criteria are more uncertain lying probably between 350 and 1700 kN/m^2 (50 and 250 lb/in^2) with the same qualification on the density of test specimens. Fig. 4.3 shows [12, 13] the relation between C.B.R. and unconfined compressive strengths for a wide range of soils and it is suggested that the unconfined compressive strength requirement for the different soils can be taken as that corresponding to a C.B.R. of 100%.

In all but very dry areas it is important to examine the effects of moisture on the stabilised materials by testing specimens that have been immersed in water. Specimens should be prepared at the maximum dry density obtained in the BS. compaction test,

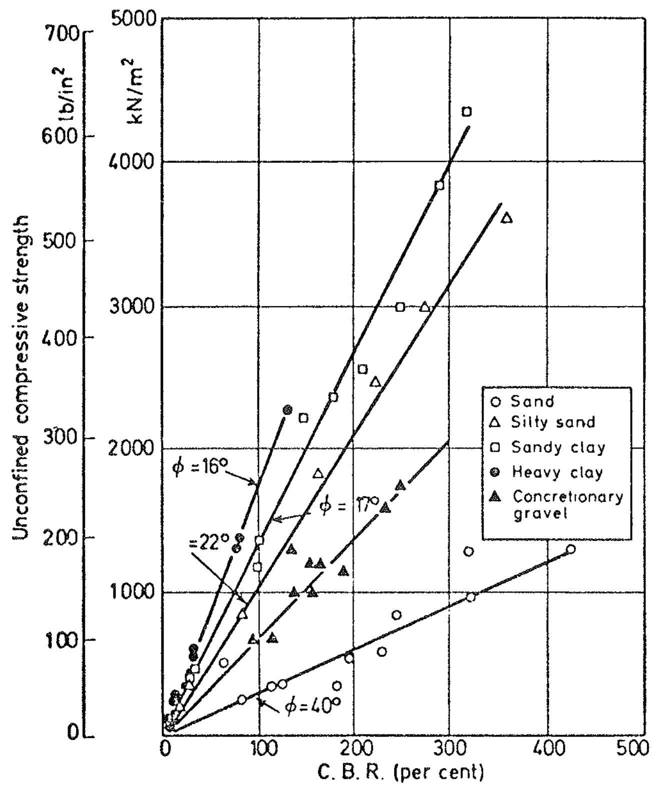

FIG. 4.3. *Relation between C.B.R. and unconfined compressive strength for soil-cement mixtures.*

2·5 kg (5·5 lb) rammer method since this is usually close to the density achieved in the field. Alternatively, a series of tests at a range of densities should be undertaken so that the strength at field density may be interpolated.

The stabiliser content indicated from the test may have to be adjusted from a knowledge of how effectively the stabiliser will be spread and mixed with the soil in the field. With hand spreading, amounts of cement of less than 8 kg/m² (15 lb/yd²) or of hydrated lime of less than 6·5 kg/m^2 (12 lb/yd^2) for a 15 cm (6 in) thick base are difficult to distribute uniformly.[5] With most soils the stabiliser requirement will be of the order of 11 kg/m^2 (20 lb/yd^2) for a 15 cm (6 in) thick base and amounts exceeding 16 kg/m^2 (30 lb/yd^2) are usually uneconomical. To perform the mixing operation effectively, plant should be capable of pulverising the soil so that 75% of all material other than stone or gravel will pass the 4·75 mm ($\frac{3}{16}$ in) sieve when mixing is complete[14]. If soils mixed with these proportions of stabiliser fail to harden satisfactorily they are not necessarily unsuitable for stabilisation. It may be possible to overcome the difficulty by modifying the stabilisation process and expert advice should be sought.

Bitumen finds its main use in stabilisation with sandy soils. Its function is in supplying the cohesion that is lacking in such non-plastic soils. Best results in the field are achieved with well-graded sands in which the proportion of material passing the 75 μm (BS. 200) sieve does not exceed 10% and is non-plastic. The cheapest method is to employ the sand without drying or heating. In wet areas this is accomplished by using up to 2% of hydrated lime as an adhesion agent in conjunction with a cut-back bitumen containing special acids to react with the lime[15]. In dry areas where the natural moisture content of the sand is low, a normal cut-back bitumen or bitumen emulsion may be used. With some sands, it may be necessary to use heated sand with a harder bitumen to obtain sufficient stability.

Generally the proportions of bitumen required range between 4 and 6% by weight of dry sand, the higher proportions being necessary with fine sands. Recent experience has indicated that with well-graded sands, adequate cohesion and stability may be obtained with bitumen contents as low as 2%.[10]

All stabilised soils require the protection of a bituminous surfacing. If not so protected, they are likely to be rapidly abraded by traffic. With cement- and lime-stabilised soils the first bituminous treatment is generally a prime-coat of fluid cut-back bitumen. This prime coat may also function to assist in the curing of the stabilised soil by inhibiting the evaporation of moisture.

NOTE: Text deleted for continuity.

6.4 SHAPING

The riding quality of a road surface depends on the smoothness of the upper surface of the pavement and it is this quality which determines the comfort or discomfort experienced by the road user. A poorly finished surface gives an uncomfortable ride, increases the maintenance cost both for the road itself and for the vehicles travelling over it and increases hazards by allowing puddles to form during periods of rain.

Although shaping is desirable at all stages of road construction, particular attention should be given to the shaping of the surface of the subgrade, sub-base, base and surfacing layer. In road-building, succeeding layers are placed in a loose state on top of the previously compacted layer and any inequalities in the depth of loose material placed leads to an uneven surface. Rectification of this unevenness is often a time-consuming operation and in many instances is wasteful of material, since it is thoroughly bad practice to attempt to scrape off the high spots and distribute the material
10 in thin layers in the hollows, especially on base layers. Such thin layers of material do not bond effectively with the mass of the base layer and will subsequently peel off under traffic.

Thin bituminous surfacings are the norm in developing countries and the riding quality must be built into the base layer. The motor grader is widely used to shape natural gravel and soil whether stabilised or not and is rapidly superseding manual methods. With harsh base materials of large particle size, such as crushed stone, spreading may be carried out by spreader boxes or by paver, but hand finishing may still be needed.

The motor grader is virtually indispensable nowadays both for road construction and for maintenance, its uses ranging from spreading and shaping pavement and embankment layers, and the trimming of cutting and embankment slopes, to the digging of ditches. A skilled operator is essential when high standards of finish are required and unskilled operation can appreciably diminish its usefulness.

In towns and cities, road surfaces must generally be finished to precise levels in relation to footpaths and adjacent property. On rural roads the surface must be built to precise levels on bridge approaches; elsewhere it is of no great consequence if final surface levels differ by 5 to 7 cm (2 or 3 in) from the design levels. The important consideration is to provide a true running surface of good riding quality and with the correct cambers and superelevation on bends. Standards of finish for the different layers of construction are indicated in Table 6.1. These standards are not difficult to achieve and, if they are combined with adequate design and care in compacting the different layers of construction, will provide roads that keep a good riding quality for many years.

Table 6.1 RECOMMENDED STANDARDS OF FINISH ON PAVEMENT LAYERS

Surface of layer	*Maximum depression under 3 m (10 ft) straight-edge*	
	mm	in
Subgrade	50	2
Sub-base	25	1
Base	13	$\frac{1}{2}$
Base with thin bituminous surfacing [less than 25 mm (1 in) thick]	10	$\frac{3}{8}$
Surfacing	10	$\frac{3}{8}$

On roads with the single or multiple bituminous surface dressings generally used in developing countries, particular care is necessary in shaping the surface of the base.

The following technique has been used successfully on gravel bases. The spread and processed loose layer of material is shaped before compaction starts. The initial compacting passes are given by light rollers, pneumatic-tyred rollers operating at low tyre pressures being very suitable, so that the surface of the layer is not distorted. Compaction by heavy rollers follows and then the final grading (sometimes called scalping).[4] Lastly a coverage with a smooth-wheeled roller is given to close the surface. Using such a sequence of operations, surface finishes complying with the above recommendations can be consistently attained[5].

6.5 COMPACTION

The aims in compacting soil and other materials in road building are:

(a) to increase the strength of the material and thus obtain the best use from it as a component in the road structure,

(b) to reduce within tolerable limits subsequent compaction under traffic, i.e. to prevent the road structure from being distorted and possibly damaged by differential compaction under traffic.

The process of compaction consists essentially of packing the particles of soil or other material closer together and expelling air.[6] Compaction is measured quantitatively in terms of dry density, i.e. the weight of solid material per unit of bulk volume.

The state of compaction achieved is determined by the amount and character of the energy applied and the moisture content of the material. With a given compactive effort there exists for each soil, as shown in Fig. 6.4, a moisture content termed the 'optimum moisture content' at which a maximum dry density is obtained. As the compactive effort increases so the maximum density is increased and the optimum moisture content decreases. The effects of moisture content on compacted density can be appreciated by considering the water as a lubricant. As the moisture content increases, the lubrication given by the water causes the soil to soften and become more workable. This results in higher dry densities and lower air contents. As the air content becomes less, the water and air in combination tend to keep the particles apart, and prevent any appreciable decrease in air content. The total voids, however, continue to increase with the moisture content, and hence the dry density of the soil falls.

An essential feature of the compaction process is thus to adjust the moisture content of the material so that the compacting plant can be fully effective. Soils as they are obtained from the excavation may contain sufficient moisture for compaction. It is important for compaction to be done before drying takes place. With experience, squeezing a lump of soil in the hand can provide a simple test to indicate whether the material to be compacted is in a moisture content range suitable for compaction.[4, 5] At the optimum moisture content, the lump so formed can be broken into two pieces without crumbling yet it is not sufficiently plastic to squeeze between the fingers or more than lightly stain the hands. The data from standard specimens made in the laboratory are a better guide. With experience, the moisture content of soils can be controlled within quite close limits by such methods. In addition, regular measurements of the densities achieved should be made so that consistently high standards of compaction will be achieved.[6]

Smooth-wheeled, pneumatic-tyred, vibrating and sheepsfoot rollers are commonly used on road works. On large embankments on which heavy equipment is used this equipment alone can often give sufficient compaction provided it is routed systematically over the whole area being constructed.

The various types of roller operate most effectively under different conditions but all are capable of achieving good states of

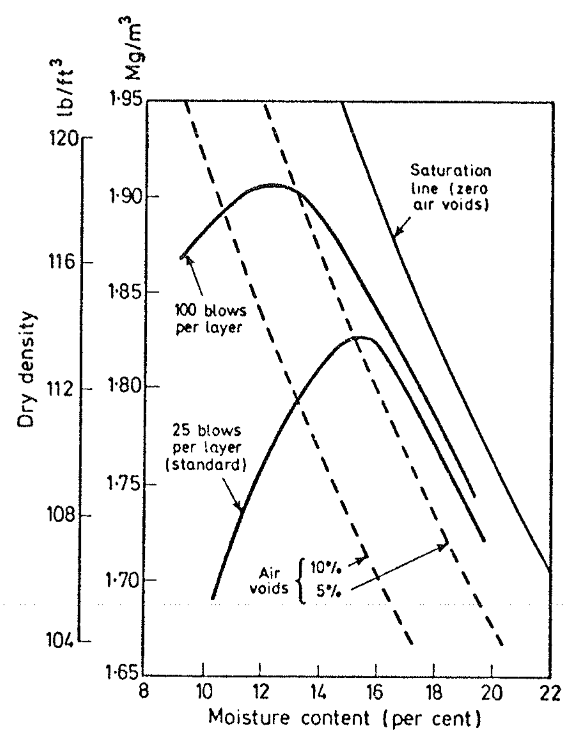

FIG. 6.4. *Effect of different amounts of compaction on dry density of sandy clay soil.*

compaction on a wide range of materials. Smooth-wheeled and pneumatic-tyred rollers are all-purpose pieces of plant although the heavier vibrating rollers can now be considered in this category also. Vibrating plates, grid and sheepsfoot rollers and the lighter vibrating rollers are less versatile but it is always well worthwhile to match the compaction plant to the job in hand especially on large works where maximum economy can accrue.

Data are available indicating the performance of compaction equipment with different soils and base materials over a range of moisture conditions.[7, 8, 9, 10] Fig. 6.5 shows typical compaction curves for commonly used items of compaction plant compacting a plastic clayey soil and a well-graded gravel-sand-clay base material. In general terms smooth-wheeled rollers are most useful in drier conditions compacting material spread in thin layers and for surface finishing operations; pneumatic-tyred rollers are more appropriate in wetter conditions and on thicker layers, and the self-propelled variety is especially useful in compacting bituminous surface dressings. Vibrating rollers and plates are seen to advantage on more granular well-graded materials, particularly on base courses. Over-stressing of the layer being compacted should be avoided; with well-graded material the range of moisture content can be critical and a spongy unstable condition can develop if it is exceeded.[9] With uniformly-graded cohesionless soils compaction pressures must be low initially and be increased gradually as the bearing capacity of the material improves.

In works of any magnitude rigorous control of the compaction process is always desirable and *in-situ* density determinations, normally using the sand replacement method, are added to the simple control techniques outlined above.[11, 12, 13] More rapid and sophisticated methods of measuring density and moisture content are at present being developed using radioactive sources. Particular care is required in calibration if these methods are to give accurate results[14]. The required state of compaction is normally specified relative to a laboratory compaction test. To obtain a reliable average value of the dry density achieved, at least 6–10 *in-situ* determinations must be made and the laboratory compaction test must be carried out on exactly the same material. With unstabilised materials this is easily achieved since the laboratory tests can be carried out on the soil extracted from the density holes.

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The amount of testing required to control the density will be determined by the magnitude of the works and by the nature of the material and its position in the road structure. To determine a reliable average figure, sampling points should be confined to an area where materials are substantially the same and which has been subjected to the same construction sequences under similar weather conditions. For example, on stabilised base construction, individual areas processed are commonly about 100–300 m (100–300 yd) long and in each such area the average state of compaction would be determined. About six dry density determinations would

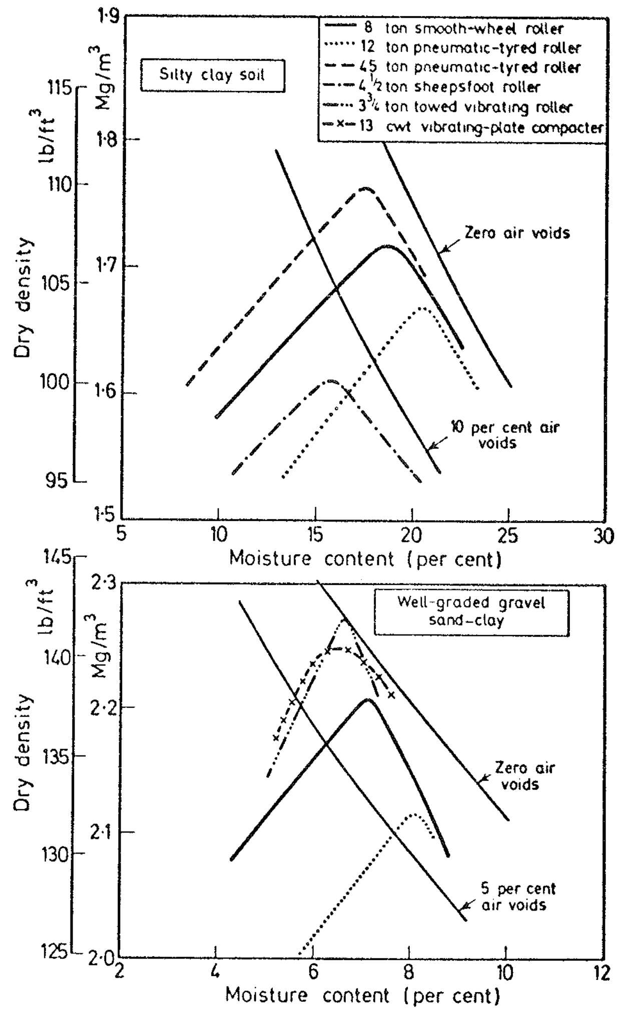

FIG. 6.5. *Typical relations between dry density and moisture content obtained after 8 passes on a silty clay soil and after 16 passes on a well-graded gravel-sand clay for the top 150 mm (6 in) of compacted soil.*

be appropriate for the smaller areas, increasing to ten on the larger. Similarly, on earthworks the material placed daily would often form a useful unit for measurement but on large contracts five dry density determinations for every 765 m^3 (1000 cu. yd) placed would be appropriate.[14]

Of greater importance than the amount of testing is the location of test points. While the location of these at chainage pegs is administratively simple it is liable to lead to optimistic results, since the pattern of testing will often result in greater care being taken with compaction at the known test locations. Test points should, therefore, be selected at random; simply drawing or casting lots for test locations would be quite appropriate.

Table 6.2 AVERAGE STATE OF RELATIVE COMPACTION THAT CAN BE CONSISTENTLY OBTAINED UNDER NORMAL WORKING CONDITIONS

Description of layer	*Minimum average relative compaction attainable** (%)	*Standard laboratory compaction tests*[12, 13]
Earthworks and subgrades	95 (with moisture control) 85 (without moisture control)	BS. 1377, 1967. Test 11 and ASTM.D. 698-64T
Base and sub-base layers: Granular and lime-stabilised materials	100 or 95	BS. 1377, 1967. Test 11 and ASTM.D 698-64T BS. 1377, 1967. Test 12 and ASTM.D. 1557-64T
Cement-stabilised materials	95	ASTM.D. 558-57

* Average relative compaction $= \dfrac{\text{Average field dry density} \times 100}{\text{Maximum dry density in laboratory compaction test}}$ per cent

As with all operations it is essential that the levels of compaction specified are realistic and the tolerances required for work in the field recognised. Table 6.2 summarises the states of compaction that are commonly specified in pavement and subgrade layers and which have been shown to be attainable in practice in climates ranging from moist temperate to arid tropical. It is particularly important to recognise that with cement-stabilised materials, delay between mixing and compaction may result in lower values of relative compaction being obtained than would be expected with the same soil without stabiliser.[15] Standard laboratory compaction tests making allowance for this phenomenon are available[12] and provide a measure by which the state of compaction of cement-stabilised materials can be judged.

6.6 QUARRIES, DRILLING, BLASTING AND CRUSHING

Stone is needed at some stage in almost all forms of road construction and potential quarry sites which can provide the quantities required should be located at an early stage. Geological surveys will be of assistance in locating stone deposits and indicating the type of rock likely to be available. A new quarry should be carefully planned to take full advantage of the terrain in constructing access roads and siting crushing and associated plant; differences in levels can provide gravity loading of the quarry products. Overburden and debris should be cleared from working areas and all-weather access provided.

Quarry operation requires skilled personnel and careful adherence to safety regulations. Drilling is normally done by rock drills operated by compressed air. Improved performance of operation will be achieved by the use of tungsten carbide bits which should be kept correctly sharpened. Modern blasting techniques using short time delay fuses, the delay being of the order of 10–1000 ms. (1 ms = 0·001 sec), enable large quantities of rock to be broken out with good fragmentation and minimum ground disturbance.[16] Blasting with short time delay fuses enables a large number of charges to be blasted virtually simultaneously, thus producing sufficient rock to match the weekly output of the quarry. Earlier methods of blasting involved the intermittent firing of smaller numbers of charges with considerable delay to other operations.

After blasting, the rock is crushed to sizes required on the road. The size and type of crushing plant will depend on several factors: the type of rock, the amount required and the specified gradings. The output of crushing plant should be assessed bearing in mind the amounts of stone likely to be required; too large a plant will involve heavy capital expenditure and only operate for short periods; too small a plant, however, will be unable to meet requirements on schedule and work may well be delayed.

The minimum requirement to produce chippings of reasonable shape and size is a plant with two-stage crushing, the first to reduce block stone from 20–30 cm (9–12 in) to 5–8 cm (2–3 in) and the second to produce chippings from the 5–8 cm (2–3 in) stone. The crushed stone will be stockpiled prior to use on the road and care should be taken to avoid mixing the various sizes and contamination with soil or other materials. Every effort should be made to remove the dust from crushed stone which is to be used for bituminous surface treatments. Even a small quantity of dust can affect adhesion between the stones and bituminous binders.

6.7 STABILISED SOILS

6.7.1 SPREADING AND MIXING OF STABILISED SOILS

Road bases consisting of natural gravels or soils of low plasticity and stabilised with small proportions of cement or hydrated lime have been described previously and are widely used in developing countries, especially in tropical and sub-tropical areas. Mix-in-place methods are generally used to incorporate cement and lime into the natural soil. Mix-in-place may be used with bitumen but premix plants are more commonly used.

When the mix-in-place method is used, the required amount of stabiliser is spread on the surface of the soil to be stabilised and then mixed in. In the premix method, the correct proportions of stabiliser and soil are mixed and then laid on the subgrade to the required depth.

6.7.1.1 *Spreading or proportioning the stabiliser*

In mix-in-place construction the bags of cement or hydrated lime are spotted on the layers of soil to be processed, split open and the stabiliser distributed over the surface by hand (Fig. 6.6). It is essential that this spreading should be carried out carefully since any lack of uniformity will be reflected in the finished base.[5] Bag spotting should be arranged so that each bag is located at the centre of an area which is as near a square as practicable and for the usual quantity of stabiliser, 11 kg/m^2/15 cm base thickness (20 lb/yd^2/6 in base thickness), this is achieved with bags spotted in three longitudinal rows on a 6 m (20 ft) wide carriageway. Where bulk supplies of stabiliser are available a mechanical spreader is often used and recent research has shown that mechanical spreaders can spread low proportions of cement and hydrated lime more uniformly than manual methods.[4] Where bituminous binders are being used these are normally applied through the metered system of specialised mixing plant.

With plant-mix, weigh-batching of the soil is usual. Cement and lime are supplied in bags of known weight and bitumen is proportioned by either weight or volume. It is essential that a close check is kept on the quantities of stabiliser used by regular checking of the invoices and stocks, so that no gross proportioning error is overlooked.

6.7.1.2 *Mixing of the stabiliser and soil*

Where soils are friable and free-flowing at the moisture content prevalent in the field little or no pulverisation is required. The description of soils as free-flowing, though qualitative, is fairly clear. It depends on the combined effects of the plasticity and grading of the soil and the prevailing moisture conditions.

The product of the plasticity index and the percentage passing a 425 μm (BS. No. 36, ASTM No. 40) sieve can usefully define soil properties for this purpose. Where the product does not exceed 1000, a wide range of plant has been found to be effective in mixing these soils;[5] this range includes agricultural plant which effects mixing by tumbling the constituents over and over (e.g. agricultural disc harrows and ploughs and motor graders) in addition to specialist machinery such as single or multi-pass machines of the rotavator or pugmill type. (Fig. 6.7). 19

In mix-in-place using cement or hydrated lime with heavier plastic soils, pulverisation of the soil becomes more important and machines of the rotavator type with a positive shredding action are needed. Even with these it is usually not practicable to stabilise a soil where its liquid limit and plasticity index exceed 50 and 30% respectively.[17] However, the friable red clay soils common in tropical areas have been successfully treated although exceeding these limits[18]. In any given instance, the quality of pulverisation attainable can be checked by sieve analysis and a common requirement is that soils should be pulverised until 75% of all material other than stone or gravel particles will pass a 4·75 mm ($\frac{3}{16}$-in) sieve.[19]

Water is normally added to soils being stabilised with cement and lime after some initial mixing passes of the dry soil and stabiliser, when multi-pass mixing methods are used. With agricultural plant, water is sprayed on the surface from bowsers, but

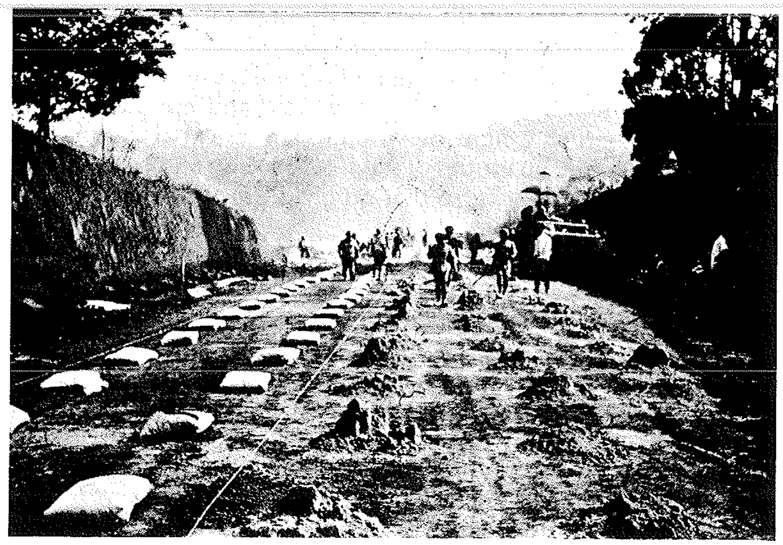

FIG. 6.6. *Spreading cement from bags for a stabilised road base. Sierra Leone.*

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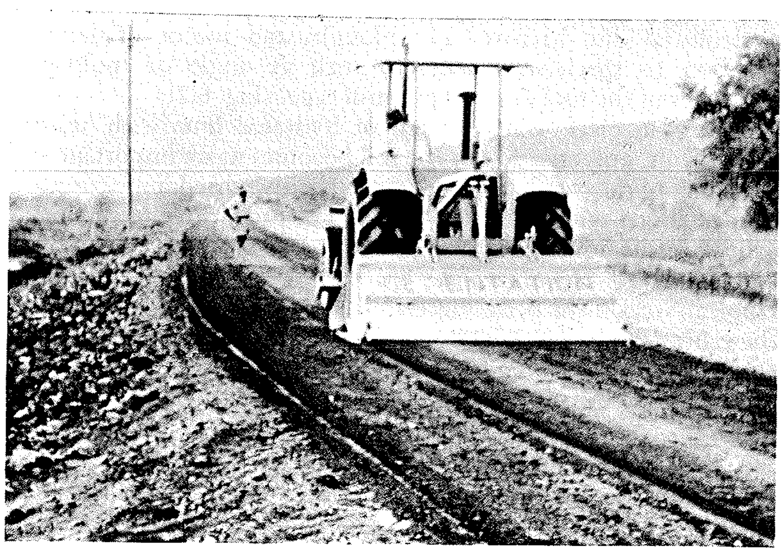

FIG. 6.7. *Mixing cement-stabilised road base with a multi-pass mixing machine. Kenya.*

most specialised mixing machines of the rotavator and pugmill type spray the moisture required into a mixing chamber, which greatly assists in quick dispersion through the soil and reduces evaporation losses. Where central plants are employed, the mixers used for cement, lime and bitumen stabilisation are commonly those used for premixed bituminous materials. With friable and free-flowing soils, concrete mixers can be used for cement and lime stabilisation.

Materials stabilised with cement and hydrated lime should be damp-cured until the bituminous prime coat is applied. This is simply achieved by spraying the surface of the stabilised base with water at least three times daily (morning, mid-day and evening); alternatively the surface may be covered with damp sand or polythene sheeting. At least three days should elapse between the construction of the base and the application of the prime coat.

NOTE: The details of surfacing are beyond the scope of this compendium.

Grader blade is used to mix lime-stabilized laterite soil (Honduras).

HIGHWAY RESEARCH RECORD

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Soil Stabilization: Asphalt, Lime, Cement

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CONTENTS

SOIL STABILIZATION: A MISSION ORIENTED APPROACH

J. A. Epps, W. A. Dunlap, and B. M. Gallaway, Civil Engineering Department, Texas A&M University, College Station; and
D. D. Currin, U.S. Department of the Air Force, Kirtland Air Force Base, New Mexico

The widespread use of chemical additives for improving the physical properties of soils and soil-aggregate systems has emphasized the critical need for a classification and indexing system to simplify the selection of the most desirable chemical to be used for the existing environmental conditions and service demands. Such a system is described in this paper. The soil stabilization indexing system is subdivided into parts dealing separately with lime, portland cement, bituminous materials, and combinations of these materials. The different criteria for the use of each of these stabilizers are described in detail with extensive references to the literature. A series of flow charts have been developed that can be used in selecting the type and the amount of stabilizer for a given soil.

•THE U.S. Department of the Air Force demands and utilizes a broad array of airfield pavement types, ranging from very austere temporary runways in forward combat zones to well-engineered, heavy-duty runways designed for the most up-to-date aircraft. Because many of the existing pavements were built in the early 1930's, a continual program of maintenance and reconstruction is carried so that the airfields can accommodate modern aircraft. New construction is also mandatory, and this includes permanent facilities as well as limited-life pavement systems, many of which are constructed within very severe time constraints. Expedient construction must take full advantage of on-site construction materials because all additional materials and equipment must be airlifted in to ensure rapid response.

The attractive engineering and economic benefits of soil stabilization make it necessary that this construction alternative be considered. Yet, in many cases, the engineer has no past experience or specialized training in soil stabilization techniques. To alleviate this problem, an index system is required that will allow the engineer to select the appropriate type and amount of stabilizer. The use of the index system in the field should require determination of relatively simple properties of the soil. These soil properties, together with suitable use factors and environmental data, should be used as input to the index system.

AIR FORCE SOIL STABILIZATION INDEX SYSTEM

An overall systematic approach was used in developing the Air Force soil stabilization index system (SSIS). The development of this system, shown in Figure 1, is discussed in this section (1).

Type of Stabilization

Chemical stabilization is of primary concern in the SSIS. However, both chemical and mechanical stabilization must be considered and the alternatives evaluated.

Sponsored by Section on Compaction and Stabilization and presented at the 50th Annual Meeting.

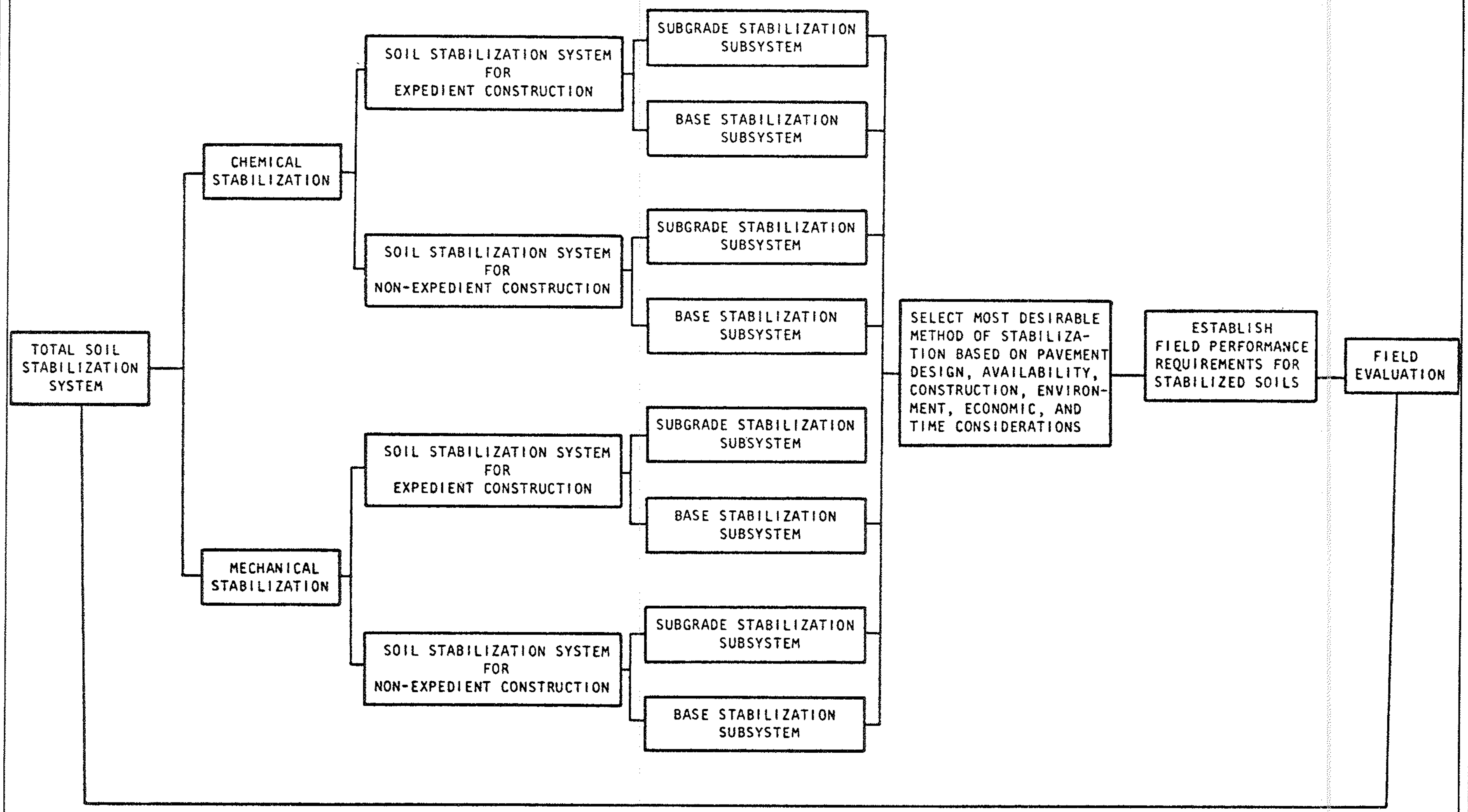

Figure 1. The Air Force soil stabilization index system.

Use Factors

The soil stabilization system should be capable of being utilized for (a) theater of operations use on both expedient and nonexpedient pavements and (b) zone of interior use on permanent pavements.

Expedient refers to short-lived, high-risk, rapidly constructed pavements, whereas nonexpedient and permanent pavements have a longer life and require an extended construction period. The major difference between nonexpedient and permanent pavements is that the latter would probably be constructed by civilian firms and the design lead time would allow more thorough and detailed investigation of stabilization alternatives. Permanent construction is identical to that used by state highway departments for primary roads, and the index system for nonexpedient construction supplies a "jumping-off" point for investigations in permanent construction.

Figure 1 shows another way in which use factors are entered in the index system by specifying different subsystems for subgrade and base course stabilization. Subbases are not considered directly, but they may fall either in the subgrade or base course subsystems depending on the material type and desired strength characteristics.

Environmental Factors

Environmental factors might influence the ultimate durability and suitability of the stabilized soil. They are based primarily on climatological effects. Both rainfall and temperature must be considered because either can significantly influence the type and amount of stabilizer used as well as the time of the year in which certain stabilizers can be used.

Construction Factors

Military engineers faced with hasty construction in the theater of operations usually are faced with limited equipment also. Knowledge of the type of equipment required for a certain stabilization task may prove to be a valuable planning tool not only in anticipating the type of equipment necessary to perform a stabilization task but also in eliminating the use of a particular stabilizer if adequate equipment and time are not available.

Field Performance Requirements for Stabilized Soils

The desired performance of the stabilized soils is established by the Air Force. In most cases, this is based on anticipated life of the structure and allowable time for construction. Examples of this information include the recent mobility concepts and various other operational requirements that have been developed by the Air Force.

Field Evaluation

The verification of the index system for soil stabilization must ultimately come from the user, i.e., the Air Force and its military partners. On pavement projects where stabilization has been used, adequate construction records and follow-up evaluations will be absolutely necessary to verify the adequacy of the stabilized sections. Continual evaluations of stabilized sections that are already in place will also aid in evaluating the ultimate performance of the index system.

Finally, it should be stressed that the SSIS is not a substitute for structural pavement design. In its present form, it will not indicate to an engineer whether a layer should be stabilized or whether there are structural advantages of stabilizing one layer instead of another. Rather, the role of the index system is this: If the engineer decides to use stabilization, then he should be able to use the index system to tell him what kind of stabilization to use and how much stabilizer he should use. Soils that are not amenable to stabilization can be so identified in the index system. If other circumstances, such as climatic conditions or lack of appropriate equipment, rule out the possibility of stabilization, the index system can also provide this information.

GENERAL REQUIREMENTS FOR SELECTING STABILIZERS

Several guides have been published that assist the engineer in the selection of a stabilizer for a particular soil (2, 3). These guides indicate that selection of the stabilizer is dependent on the location of the stabilized layer in the pavement as well as the soil type. Systems have been developed for both base course and subgrade stabilization (1), although only the base course stabilization system will be presented in this paper.

Both the Unified Soil Classification System and the AASHO Soil Classification System have been utilized to select soil stabilizers (4, 5). Because both grain size and Atterberg limits are necessary inputs to classify soils according to either system, these 2 parameters were used for the initial separation of the soils into specific categories. In particular, the percentage passing the No. 200 sieve and the plasticity index (PI) were selected.

Specific guidelines for stabilizer selection are also available from literature published by consumer, producer, user, and general interest groups. These guidelines are discussed here in detail for lime, cement, bituminous materials, and combinations of these stabilizers.

Criteria for Lime Stabilization

Lime will react with most medium, moderately fine, and fine-grained soils to decrease plasticity, increase workability, reduce swell, and increase strength (6). In general terms, the soils that are most reactive to lime include (7) clayey gravels, silty clays, and clays. All soils classified by AASHO as A-5, A-6, and A-7 and some soils classified as A-2-6 and A-2-7 are most readily susceptible to stabilization with lime. Soils classified according to the Unified System as CH, CL, MH, ML, CL-ML, SC, SM, GC, and GM should be considered as potentially capable of being stabilized with lime.

Robnett and Thompson (6), based on experience gained with Illinois soils, have indicated that lime may be an effective stabilizer with clay contents (< 2μ) as low as 7 percent; and, furthermore, soils with a PI as low as 8 can be satisfactorily stabilized with lime in certain instances (8). Armed forces criteria (2) indicate that the PI should be greater than 12, while representatives of the National Lime Association (9) indicate that a PI greater than 10 would be a reasonable lower limit to utilize.

In view of these suggested criteria, it is believed that the PI of the soil should have a lower limit of 10 to ensure that a reasonable degree of certainty will exist for successful stabilization with lime.

Criteria for Cement Stabilization

The Portland Cement Association (10, 11) indicates that all types of soils can be stabilized with cement. However, well-graded granular materials that possess a floating aggregate matrix have given the best results (12).

Limits on PI have been established by the armed forces (2), depending on the soil type. The PI should be less than 30 for sandy and gravelly materials and less than 20 for the fine-grained soils. These limitations are necessary to ensure proper mixing of the stabilizer in the soil.

Information developed by the Federal Highway Administration (5) indicates that cement should be used as a stabilizer for materials with less than 35 percent passing the No. 200 sieve and with a PI less than 20. Thus, this implies that A-2 and A-3 soils can be best stabilized by cement, while A-5, A-6, and A-7 soils can be best stabilized by lime.

The authors have selected a maximum PI of 30 for those soils to be stabilized with cement.

Criteria for Bituminous Stabilization

The majority of soil-bituminous stabilization has been performed with asphalt cement, cutback asphalts, and emulsified asphalts. For this reason, only these types of bituminous stabilizers are considered.

Some of the earliest criteria for bituminous stabilization were developed by the HRB Committee on Soil-Bituminous Roads. These criteria were revised and published by Winterkorn (13). Other criteria have been presented by the American Road Builders Association (14), The Asphalt Institute (15, 16), Herrin (17), Chevron Asphalt Company (18), Douglas Oil Company (19), and the U.S. Department of the Navy (20). Although these criteria were developed for particular types of bituminous stabilizers (i.e., soil-bitumen made with cutback asphalt), they are given in a single table (Table 1) for comparison purposes.

Current trends indicate that stabilization with asphalt cements is gaining widespread application. Requirements for aggregate grading and mixture properties of mixes containing asphalt cement have recently been summarized by the HRB Committee on Bituminous Aggregate Bases (21). This survey of criteria together with data published by the armed forces (22) suggests that soils that are nearly nonplastic and contain less than 18 percent passing the No. 200 sieve are most suitable for hot-mix asphalt cement stabilization.

Based on these criteria, a limit of 20 percent passing the No. 200 sieve, a PI less than 6, and the product of PI and the minus No. 200 material less than 60 have been utilized for selecting soils suitable for stabilization by asphalt. Less stringent requirements have been used in conjunction with the other stabilization subsystems developed for the Air Force (1).

Criteria for Combination Stabilizers

Combination stabilization is here defined specifically as lime-cement, lime-asphalt, and lime-fly ash. Because lime-fly ash stabilization is not expected to be a common stabilization method used by the Air Force, it will not be incorporated into the index system. The purpose of using combination stabilizers (lime and then one or the other stabilizers) is to reduce plasticity and increase workability so that the soil may be effectively stabilized by the second agent or additive. 29

Robnett and Thompson (23) have reviewed the literature and have suggested that soils that may be treated by these combination stabilizers are those classified by AASHO as A-6 and A-7 and certain soils classified as A-4 and A-5.

Based on these findings, it has been suggested that these combination stabilizers be utilized with materials that have greater than 35 percent passing the No. 200 sieve and that quantities of lime be used sufficient in magnitude to ensure that the PI is less than the established criteria for either cement or asphalt stabilization as appropriate.

These criteria together with appropriate environmental and construction precautions as given in Table 2 have been used to establish the base course stabilization system shown in Figure 2.

This stabilization system separates soils into various groups so the engineer may select the stabilizer suitable for use within these particular groups. This system will not, however, indicate the amount of stabilizer that must be used for a particular soil. The following discussion will suggest criteria that will allow the development of appropriate subsystems for the determination of stabilizer quantities.

TABLE 1

CRITERIA DEVELOPED FOR BITUMINOUS STABILIZATION

Developer	Percent Passing No. 200 Sieve	Plasticity Index	Plasticity Index × Percent Passing No. 200 Sieve
Winterkorn	8 to 50	18	
American Road Builders Association	0 to 35	10	
Herrin	0 to 30	10	
The Asphalt Institute, Pacific Coast Division	3 to 15	6	60
Chevron Asphalt Company	0 to 25	Nonplastic	72
Douglas Oil Company	0 to 30	7	

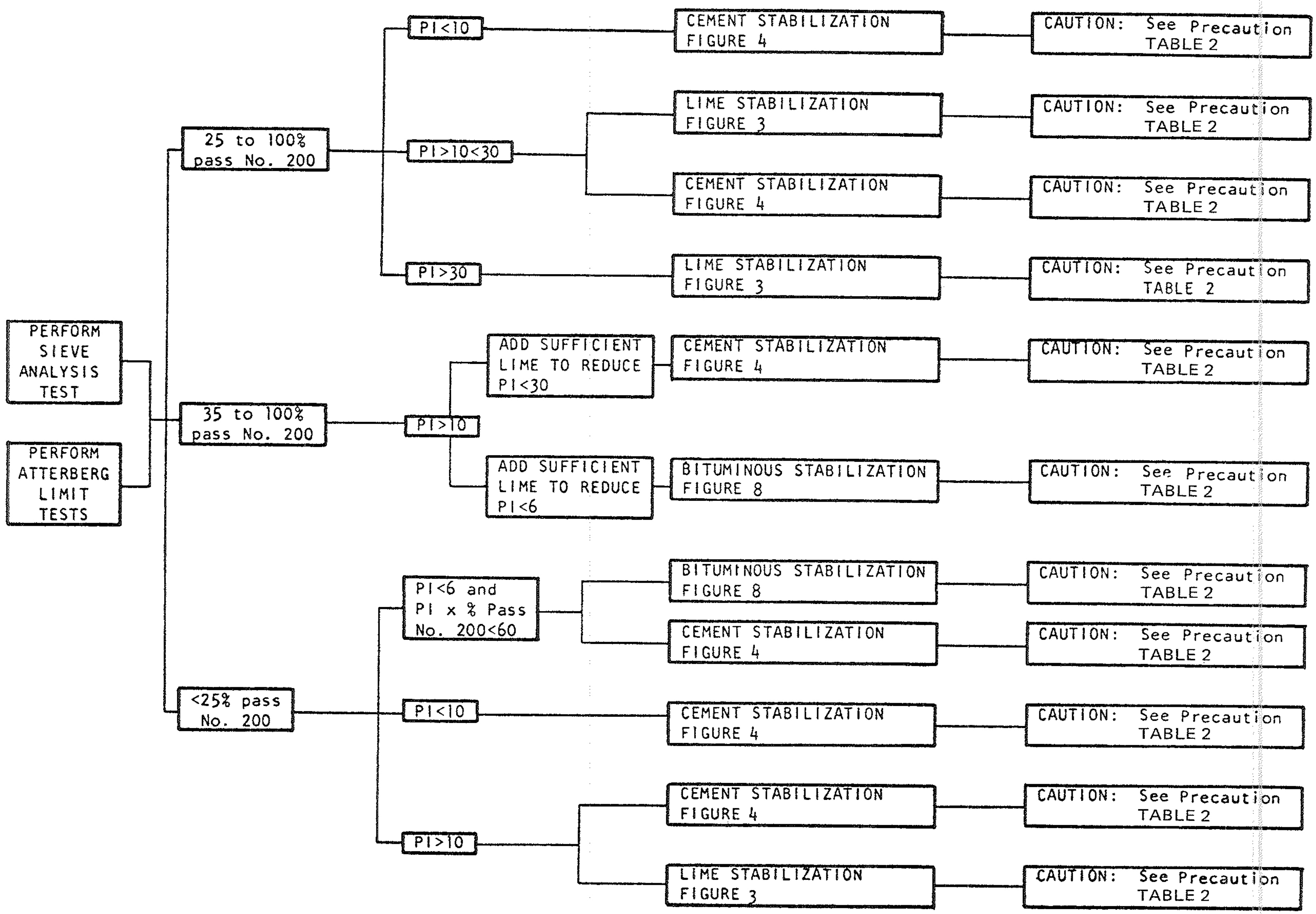

Figure 2. Selection of stabilizer for nonexpedient base construction.

TABLE 2

ENVIRONMENTAL AND CONSTRUCTION PRECAUTIONS

Stabilization	Environmental or Construction	Precaution
Lime	Environmental	If the soil temperature is less than 40 F and is not expected to increase for 1 month, chemical reactions will not occur rapidly and, thus, the strength gain of the lime-soil mixture will be minimal.
		Lime-soil mixtures should be scheduled for construction such that sufficient durability will be gained to resist any expected freeze-thaw cycles.
	Construction	Heavy vehicles should not be allowed on the lime-stabilized soil for 10 to 14 days after construction.
Cement	Environmental	If the soil temperature is less than 40 F and is not expected to increase for 1 month, chemical reactions will not occur rapidly and, thus, the strength gain of the cement-soil mixture will be minimal.
		Cement-soil mixtures should be scheduled for construction such that sufficient durability will be gained to resist the expected freeze-thaw cycles.
		Construction during periods of heavy rainfall should be avoided.
	Construction	Heavy vehicles should not be allowed on the cement-stabilized soil for 7 to 10 days after construction.
Bituminous	Environmental	When asphalt cements are used, construction should be allowed only when proper compaction is possible. If thin lifts are being placed, the air temperature should be 40 F and rising. Adequate compaction can be obtained at freezing temperatures.
		When cutbacks and emulsions are being used, the air temperature and soil temperature should be above freezing.
		Bituminous materials should completely coat the soil particles prior to compaction.
	Construction	Central batch plants, together with other specialized equipment, are necessary for bituminous stabilization with asphalt cements.
		Hot, dry weather is preferred for all types of bituminous stabilization.

DESIGN SUBSYSTEMS

Numerous research publications and technical guides are available to aid the engineer in the selection of criteria to determine the amount of stabilizer. A wide variety of test methods have been proposed; however, quantitative criteria are not always available. The criteria discussed here are for establishing the design subsystems aimed at determining appropriate stabilizer quantities for lime, cement, and bituminous stabilization. 31

Lime Stabilization

Selection of Appropriate Soils—The preceding section discussed the general requirements of the soil with respect to gradation and plasticity. However, there are other physicochemical features that must be considered in determining whether lime will react with a soil.

Thompson (24) has defined soils as being lime-reactive if they display significant strength increase (measured by unconfined compressive strength) when treated with lime. Soils that are not lime-reactive according to this definition are not necessarily unimproved by the addition of lime because lime may still decrease the plasticity, decrease the susceptibility to water, and enhance the overall engineering behavior of the soil. However, because improved load-bearing characteristics are desired in the stabilization index system, strength will be a major consideration here.

Factors that may prohibit soils from being lime-reactive include soil pH and the presence of organics and sulfates. Soils with a pH less than 7 may not be lime-reactive, although some soils with pH values as low as 5.7 have reportedly been effectively stabilized with lime (24). It has also been reported that soils with organic carbon contents exceeding about 1 percent are not satisfactorily lime-reactive (24). In addition, experience has shown that the presence of significant amounts of sulfates diminishes the effectiveness of lime.

It has been reported that A-horizon soils in Illinois do not satisfactorily react with lime (24), and similar reports have been made on other soils. This is probably the result of high organic contents in the upper horizon. Poorly drained soils often are the most reactive to lime, possibly because of the higher pH and the availability of lime-reactive constituents, such as unweathered soil minerals.

Selection of Type of Lime—Lime is generally used as an all-encompassing term to denote either slaked (hydrated) lime or quicklime. Both calcitic lime and dolomitic (high magnesium) lime are available in the United States. Although there is some disagreement as to whether the type of lime influences the strength of lime-soil mixtures (25), the selection of the lime type is usually predicated by availability of the stabilizer and safety requirements of the particular job.

Selection of Lime Quantity—There are fewer definitive criteria for evaluating the correct quantity of lime than for cement or bituminous materials.

Eades and Grim (26) have proposed a short-cut test where the appropriate lime content is that which will produce a minimum pH of 12.4 one hour after mixing. This test has not been validated for soils on a worldwide basis and should be used with caution.

Most authors have reported that a minimum of 3 percent lime is necessary to produce adequate reactions in the field (27). The Air Force (28) suggests that 2, 3, and 5 percent lime be used in coarse soils (those containing 50 percent or less passing the No. 200 sieve) while 3, 5, and 7 percent be tried for fine-grained soils (greater than 50 percent passing the No. 200 sieve). The National Lime Association recommends 3, 5, and 7 percent lime in trial mixtures (27). With the exception of the pH test described, the lime content must generally be determined by trial mixtures with the amount of lime being the minimum required to produce the desired reactions.

Methods of Evaluating Soil-Lime Mixtures—Several types of tests have been proposed for evaluating soil-lime mixtures. These include, but are not limited to, unconfined compressive strength, California bearing ratio, flexural fatigue strength, triaxial compressive strength, tests yielding elastic properties, cohesiometer values, and freeze-thaw and wet-dry tests. Most of these tests are not used routinely, and satisfactory criteria are not generally available. Some of the most reliable data are based on unconfined compressive strengths developed from research done by Thompson (29). Table 3 gives his results.

Durability, the ability of a material to retain stability and integrity over years of exposure to service and weathering, is perhaps the most difficult to determine. Of the many tests developed, only a modified freeze-thaw test shows substantial merit (30).

Figure 3, the lime stabilization subsystem, has been developed from these criteria.

TABLE 3

TENTATIVE LIME-SOIL MIXTURE COMPRESSIVE STRENGTH REQUIREMENTS

Anticipated Use	Residual Strength Requirement[b] (psi)	Strength Requirements for Various Anticipated Service Conditions[a]			
		8-Day Extended Soaking (psi)	Cyclic Freeze-Thaw[c] (psi)		
			3 Cycles	7 Cycles	10 Cycles
Modified subgrade	20	50	50	90 50[d]	120
Subbase					
Rigid pavement	20	50	50	90 50[d]	120
Flexible pavement					
10-in. cover[e]	30	60	60	100 60[d]	130
8-in. cover[e]	40	70	70	110 75[d]	140
5-in. cover[e]	60	90	90	130 100[d]	160
Base	100[f]	130	130	170 150[d]	200

[a]Strength required at termination of filed curing (following construction) to provide adequate residual strength.
[b]Minimum anticipated strength following first winter exposure.
[c]Number of freeze-thaw cycles expected in the lime-soil layer during the first winter of service.
[d]Freeze-thaw strength losses are based on 10 psi/cycle except for these 7-cycle values that are based on a previously established regression equation.
[e]Total pavement thickness overlying the subbase; requirements are based on Boussinesq stress distribution; rigid pavement requirements apply if cemented materials are used as base courses.
[f]Flexural strength should be considered in thickness design.

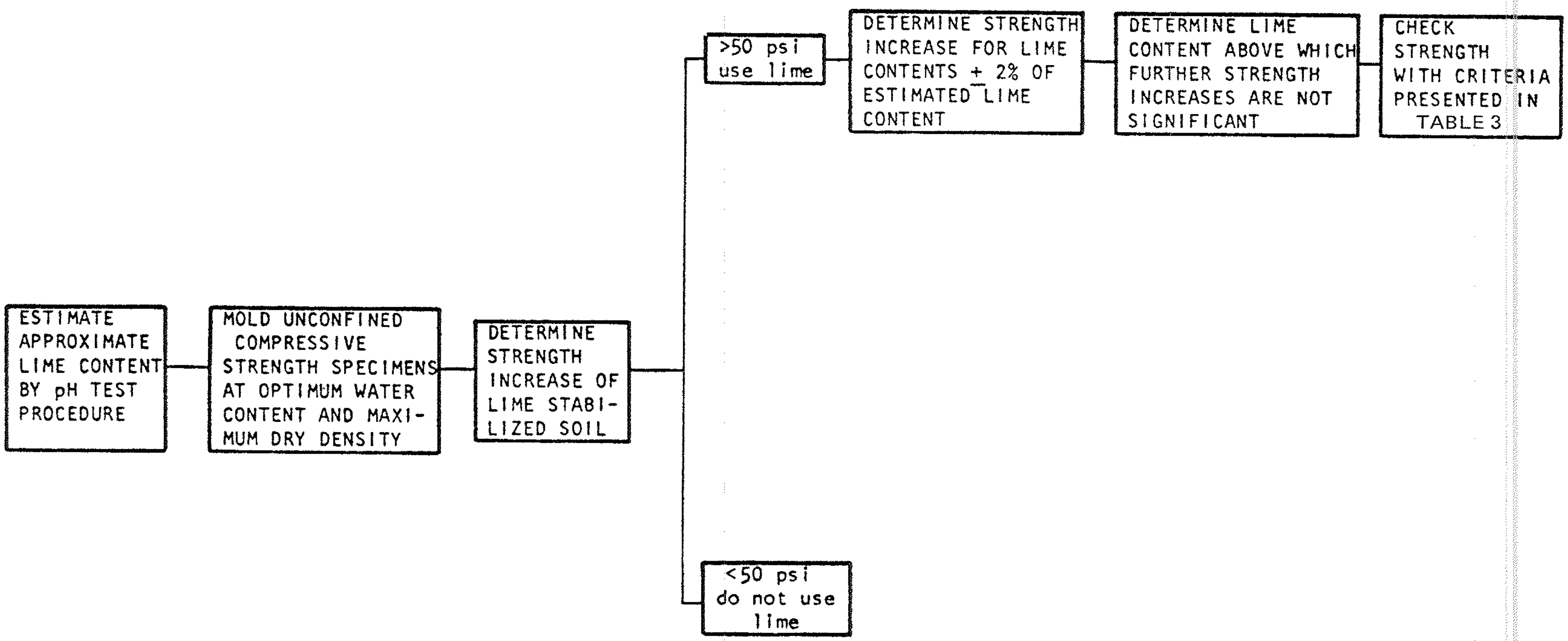

Figure 3. Subsystem for base course stabilization with lime.

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Cement Stabilization

Information as to general requirements such as gradation and Atterberg limits have been discussed previously. Most research and construction with cement-soil mixtures has been performed on soils that have been classified according to the AASHO classification system. Experience has shown that this approach is satisfactory; but, it does not include important soil properties such as clay type, soil pH, organic content, and soil sulfate content that may influence the suitability of a soil for cement stabilization. These effects are discussed in this section.

Effects of pH, Organics, and Sulfates—The Road Research Laboratory has shown a general trend of strength increase with soils possessing high pH values. For pH values greater than 7, no ill effects on strength were noted (31). The Portland Cement Association has conducted pH tests on soils, but it has found no general correlation between pH and performance (32).

Two tests have been proposed to assess the effects of organics on soil-cement strength. The Portland Cement Association (33) has suggested the use of the calcium adsorption test to determine the presence of organics in sandy soils, but this test should not be used for clay soils. Additional research conducted by the Portland Cement Association (32, 33) has shown that the standard colorimetric tests will not identify the presence of organics satisfactorily.

A satisfactory method for determining the presence of active organic matter, according to MacLean and Sherwood (31), is the pH test conducted on a soil-cement paste 15 minutes after mixing. This test essentially indicates the reactivity of the soil with cement; however, the reactivity is not solely a function of the organic content (32, 34), but it is dependent on both the organic content and the type of organics (35).

Studies conducted by Sherwood (36) have indicated that sulfate contents in soils in excess of 0.5 to 1.0 percent reduce the strength of soil-cement mixtures. Similarly, sulfate concentrations in water in excess of 0.05 percent create strength loss. For these reasons the sulfate content of the soil should be ascertained.

Type of Cement—The influence of the type of cement on the properties of soil-cement mixtures has been examined by several investigators (36, 37, 38, 39). In general, Types I, II, III, and V produce only small differences in behavior for most soils. Thus, because of its general availability and economy, it is recommended that Type I cement be utilized.

Selection of the Cement Quantity—Research performed by the Portland Cement Association (10, 40, 41, 42) on more than 2,000 soils provides data for determining cement contents for various types of soils. Cement contents for subsurface soils are given in Table 4 (10). Requirements for soils in various horizons are also specified by the Portland Cement Association.

TABLE 4

CEMENT REQUIREMENTS FOR VARIOUS SOILS

AASHO Soil Classification	Unified Soil Classification[a]	Usual Range in Cement Requirement[b]		Estimated Cement Content Used in Moisture-Density Test (percent by weight)	Cement Content for Wet-Dry and Freeze-Thaw Tests (percent by weight)
		Percent by Volume	Percent by Weight		
A-1-a	GW, GP, GM, SW, SP, SM	5 to 7	3 to 5	5	3 to 5 to 7
A-1-b	GM, GP, SM, SP	7 to 9	5 to 8	6	4 to 6 to 8
A-2	GM, GC, SM, SC	7 to 10	5 to 9	7	5 to 7 to 9
A-3	SP	8 to 12	7 to 11	9	7 to 9 to 11
A-4	CL, ML	8 to 12	7 to 12	10	8 to 10 to 12
A-5	ML, MH, OH	8 to 12	8 to 13	10	8 to 10 to 12
A-6	CL, CH	10 to 14	9 to 15	12	10 to 12 to 14
A-7	OH, MH, CH	10 to 14	10 to 16	13	11 to 13 to 15

[a] Based on U S Air Force recommendations (2).

[b] For most A horizon soils, the cement content should be increased 4 percentage points if the soil is dark gray to gray and 6 percentage points if the soil is black.

Methods of Evaluating Soil-Cement Mixtures—Various types of tests have been used to evaluate the properties of soil-cement mixtures (43). These methods include unconfined compressive strength, flexural strength, modulus of elasticity, California bearing ratio, plate bearing value, fatigue, R-value, and freeze-thaw and wet-dry tests.

Many of these test methods have not been used extensively, and satisfactory criteria are not available. However, the Portland Cement Association recommends the use of freeze-thaw and wet-dry tests and has established criteria (Table 5) for these tests.

The design subsystem for cement based on these criteria is shown in Figure 4.

TABLE 5

PORTLAND CEMENT ASSOCIATION CRITERIA FOR SOIL-CEMENT MIXTURES USED IN BASE COURSES

AASHO Soil Classification	Unified Soil Classification[a]	Soil-Cement Weight Loss During 12 Cycles of Either Wet-Dry or Freeze-Thaw Test (percent)
A-1	GW, GP, GM, SW, SP, SM	≤14
A-2-4, A-2-5	GM, GC, SM, SC	
A-3	SP	
A-2-6, A-2-7	GM, GC, SM, SC	≤10
A-4	CL, ML	
A-5	ML, MH, OH	
A-6	CL, CH	≤7
A-7	OH, MH, CH	

[a]Based on correlation presented by U.S. Air Force (2).

Bituminous Stabilization

A bituminous binder in 1 of 3 forms is generally used; the forms include cutbacks, emulsions, or cements. An indication of the type of bitumen to use for certain types of soils has been suggested by The Asphalt Institute (15), Herrin (17), the U.S. Navy (20), the Air Force (28), and Chevron Asphalt Company (18). Selection of the proper bituminous stabilizer should depend on the grain-size distribution in addition to the function of the stabilized layer in the pavement system. Table 6, adapted from Herrin and prepared by using the soil gradings also suggested by Herrin (17), and Table 7 give data regarding bitumen stabilization. 35

Asphalt Cement—Criteria used for selection of the binder viscosity and the quantity of cement for base stabilization vary among state highway departments (21), and a suitable method based on highway experience is not available. The armed forces, however, base the selection of asphalt cement viscosity or grade on the pavement temperature index. Their recommendations have been altered and are used in the design subsystem (Table 8).

The quantity of asphalt can be estimated on a surface area and particle surface characteristic concept such as the California CKE method, or the quantity can be estimated from experience. Data given in Table 9 can be used to obtain a preliminary estimate of asphalt content, but these quantities are a guide only. Final selection should be based on a test performed on the asphalt-aggregate mixture.

A recent summary of state practices (21) indicates that both Hveem and Marshall tests are popular evaluation methods among state highway departments and that criteria

TABLE 6

SELECTION OF A SUITABLE TYPE OF BITUMEN FOR SOIL STABILIZATION PURPOSES

Mix	Sand-Bitumen	Soil-Bitumen	Crushed Stones and Sand-Gravel-Bitumen
Hot	Asphalt cements 60 to 70 hot climate 85 to 100 120 to 150 cold climate		Asphalt cements 45 to 50 hot climate 60 to 70 85 to 100 cold climate
Cold	Cutbacks See Figure 5	Cutbacks See Figure 5	Cutbacks See Figure 5
Emulsions	Emulsions See Table 11 See Figures 6 and 7 to determine whether cationic or anionic emulsion should be used	Emulsions See Table 11 See Figures 6 and 7 to determine whether cationic or anionic emulsion should be used	Emulsions See Table 11 See Figures 6 and 7 to determine whether cationic or anionic emulsion should be used

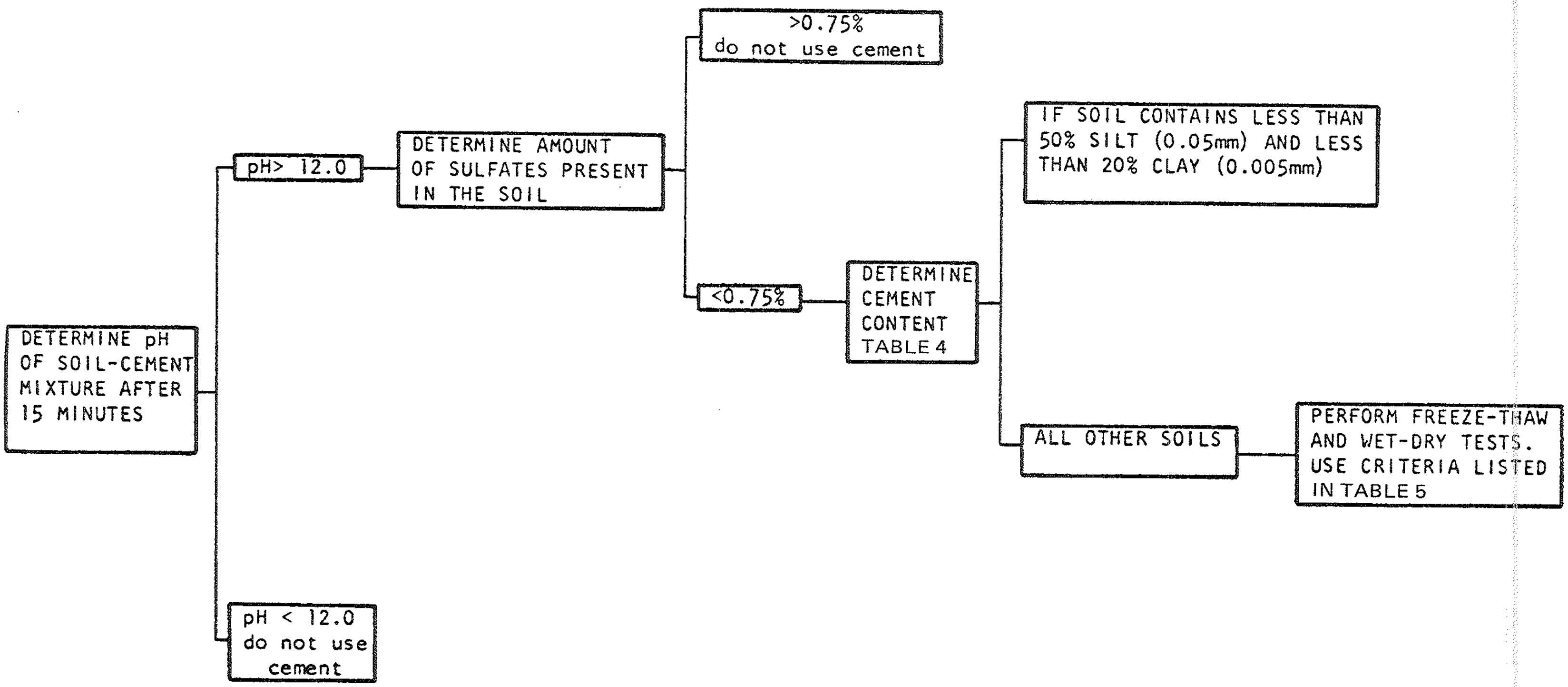


Figure 4. Subsystem for base course stabilization with cement.

TABLE 7

ENGINEERING PROPERTIES OF MATERIALS SUITABLE FOR BITUMINOUS STABILIZATION

Item	Sand-Bitumen		Soil-Bitumen	Sand-Gravel-Bitumen
Gradation (percent passing)				
1½-in. sieve				100
1-in. sieve	100			
¾-in. sieve				60 to 100
No. 4 sieve	50 to 100		50 to 100	35 to 100
No. 10 sieve	40 to 100			
No. 40 sieve			35 to 100	13 to 50
No. 100 sieve				8 to 35
No. 200 sieve	5 to 12	Good	3 to 20	
		Fair	0 to 3 and 20 to 30	0 to 12
		Poor	>30	
Liquid limit		Good	<20	
		Fair	20 to 30	
		Poor	30 to 40	
		Unusable	>40	
Plasticity index	<10	Good	<5	
		Fair	5 to 9	
		Poor	9 to 15	<10
		Unusable	>12 to 15	

Note: Includes slight modifications later made by Herrin (17).

vary from state to state. Marshall method criteria utilized by the armed forces (2) are given in Table 10 (2). The criteria listed for asphaltic-concrete binder course are indicated for use with coarse-graded, hot-mix base courses, while separate criteria are given for sand-asphalt. The Air Force has also indicated that the asphalt content determined by the Marshall method should be altered depending on the pavement temperature index. However, this criterion, which was developed for surface courses, does not appear to be warranted for base courses. 37

The Asphalt Institute (44) recommends Marshall, Hveem, and Hubbard-Field criteria for use in hot-mix base course design. Specifically, The Asphalt Institute recommends the same criteria that are utilized for surface courses, but with a test temperature of 100 F rather than 140 F. This recommendation applies to regions having climatic conditions similar to those prevailing throughout most of the United States and to bases that are 4 in. or more below the surface.

Zoeph (45) recommends Marshall criteria based on studies conducted in Germany, while McDowell and Smith (46) have recently presented a design procedure based on unconfined compressive strength and air void criteria.

Recently, attempts have been made to develop a more rational approach to pavement design. Among others, Monismith (47) has indicated that elastic and fatigue properties of asphalt-treated base courses should be considered in pavement design. These more rational methods should allow engineers to better assess the engineering behavior of these stabilized materials.

TABLE 8

DETERMINATION OF ASPHALT GRADE FOR BASE COURSE STABILIZATION

Pavement Temperature Index[a]	Asphalt Grade (penetration)
Negative	100 to 120
0 to 40	85 to 100
40 to 100	60 to 70
100 or more	40 to 50

[a]The sum, for a 1-year period, of the increments above 75 F of monthly averages of the daily maximum temperatures. Average daily maximum temperatures for the period of record should be used where 10 or more years of record are available. For records of less than 10-year duration, the record for the hottest year should be used. A negative index results when no monthly average exceeds 75 F. Negative indexes are evaluated merely by subtracting the largest monthly average from 75 F.

TABLE 9

SELECTION OF PRELIMINARY ASPHALT CEMENT CONTENT FOR BASE COURSE CONSTRUCTION

Aggregate Shape and Surface Texture	Asphalt by Weight of Dry Aggregate (percent)
Rounded and smooth	4
Angular and rough	6
Intermediate	5

TABLE 10
CRITERIA OF MARSHALL METHOD FOR DETERMINATION OF OPTIMUM BITUMEN CONTENT

Test Property	Type of Mix	Point on Curve		Criteria	
		For 100-psi Tires[a]	For 200-psi Tires[a]	For 100-psi Tires[a]	For 200-psi Tires[a]
Stability	Asphaltic-concrete surface course	Peak of curve	Peak of curve	500 lb or higher	1,800 lb or higher
	Asphaltic-concrete binder course	Peak of curve[b]	Peak of curve[b]	500 lb or higher	1,800 lb or higher
	Sand asphalt	Peak of curve		500 lb or higher	
Unit weight	Asphaltic-concrete surface course	Peak of curve	Peak of curve	Not used	Not used
	Asphaltic course binder course	Not used	Not used	Not used	Not used
	Sand asphalt	Peak of curve		Not used	Not used
Flow	Asphaltic-concrete surface course	Not used	Not used	20 or less	16 or less
	Asphaltic course binder course	Not used	Not used	20 or less	16 or less
	Sand asphalt	Not used	Not used	20 or less	16 or less
Percentage voids in total mix	Asphaltic-concrete surface course	4 (3)	4 (3)	3 to 5 (2 to 4)	3 to 5 (2 to 4)
	Asphaltic-concrete binder course	5 (4)	6 (5)	4 to 6 (3 to 5)	5 to 7 (4 to 6)
	Sand asphalt	6 (5)	– (-)	5 to 7 (4 to 6)	– (---)
Percentage voids filled with bitumen	Asphaltic-concrete surface course	80 (85)	75 (80)	75 to 85 (80 to 90)	70 to 80 (75 to 85)
	Asphaltic-concrete binder course	70 (75)	60 (65)[b]	65 to 75 (70 to 80)	70 to 80 (55 to 75)
	Sand asphalt	70 (75)	– (--)	65 to 75 (70 to 80)	– (--)

[a]Figures in parentheses are for use with bulk-impregnated specific gravity (water absorption greater than 2.5 percent).
[b]If the inclusion of asphalt contents of these points in the average causes the voids to fall outside the limits, then the optimum asphalt content should be adjusted so that the voids in the total mix are within the limits.

Criteria currently used by the armed forces for binder course utilizing Marshall mix design methods have been suggested for use.

Cutback Asphalts—The U.S. Navy (20) has suggested that the grade of cutback can be selected based on the percentage of the soil passing the No. 200 sieve and the ambient temperature of the soil (Fig. 5). The Air Force (28) and The Asphalt Institute (15) recommendations are rather general in nature.

Several methods are available to the engineer for selecting the quantity of cutback asphalts. The California CKE method could be utilized as could equations developed in Oklahoma (48) and by The Asphalt Institute (15) based on the surface-area concept. The equation recommended by The Asphalt Institute (15) is

$$p = 0.02(a) + 0.07(b) + 0.15(c) + 0.20(d) \tag{1}$$

where

p = percentage of asphalt material by weight of dry aggregate;
a = percentage of mineral aggregate retained on No. 50 sieve;
b = percentage of mineral aggregate passing No. 50 and retained on No. 100 sieve;
c = percentage of mineral aggregate passing No. 100 and retained on No. 200 sieve; and
d = percentage of mineral aggregate passing No. 200 sieve.

Numerous laboratory tests have been used to determine asphalt contents for cutback and emulsified asphalts. These methods include Hubbard-Field, Hveem stability, Marshall stability, Florida bearing, Iowa bearing, extrusion, unconfined compression, triaxial compression, R-value, and elastic modulus. Mixing methods, curing conditions,

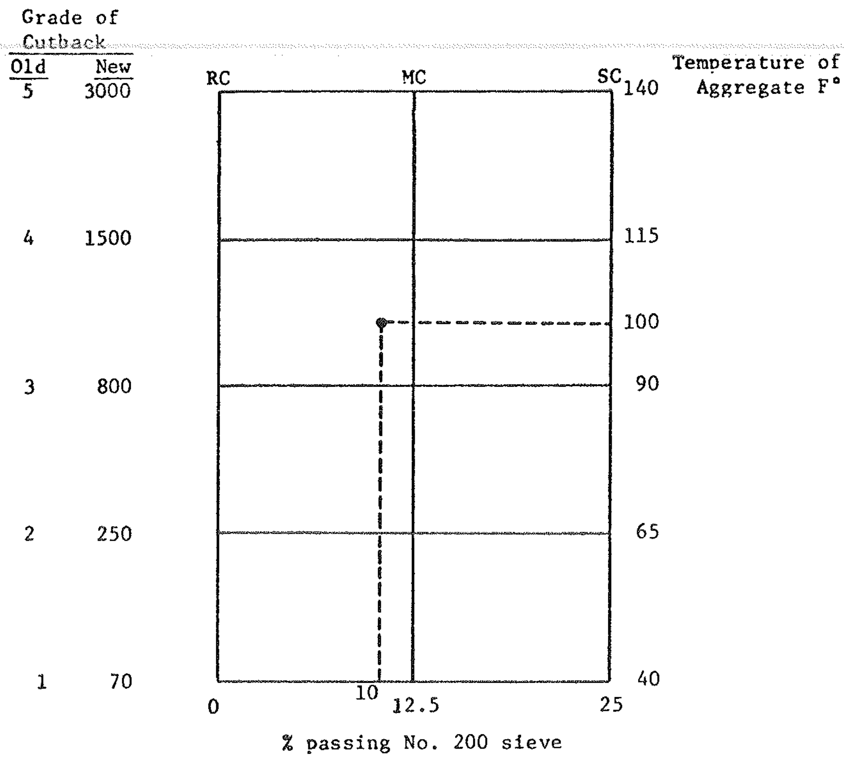
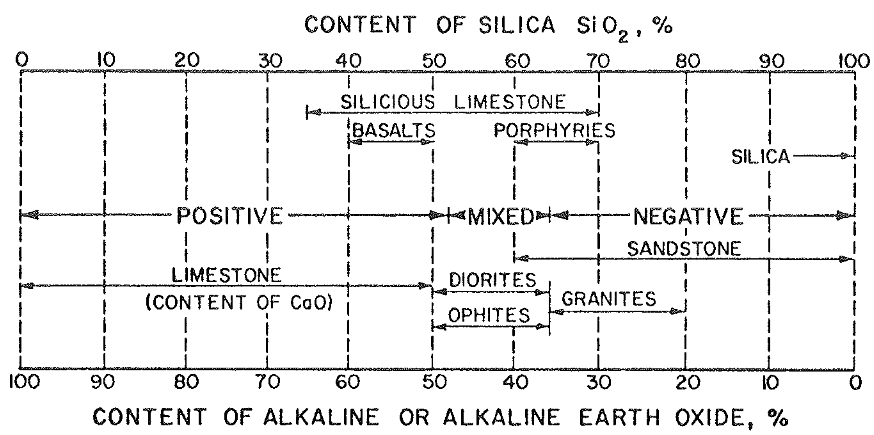


Figure 5. Selection of type of cutback asphalt for stabilization.

rate of loading, and temperature are important variables that must be carefully controlled when these tests are performed.

The Air Force is currently utilizing the extrusion test (28) for mixture design. The unconfined compression test is easy to perform, but sufficient experience to determine adequate criteria for its use is not available.



After Mertens and Wright (52)

Figure 6. Classification of aggregates.

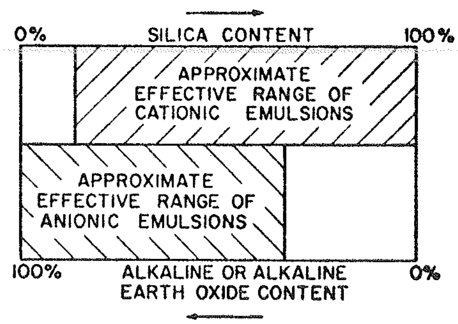
TABLE 11

SELECTION OF TYPE OF EMULSIFIED ASPHALT FOR STABILIZATION

Percent Passing No. 200 Sieve	Relative Water Content of Soil	
	Wet (5 percent or more)	Dry (0 to 5 percent)
0 to 5	SS-1h (or SS-Kh)	SM-K (or SS-1h)[a]
5 to 15	SS-1, SS-1h (or SS-K, SS-Kh)	SM-K (or SS-1h, SS-1)[a]
15 to 25	SS-1 (or SS-K)	SM-K

Note: Determine from Figures 6 and 7 whether an anionic or a cationic emulsion is to be used.

[a]Soil should be prewetted with water before using these types of emulsified asphalts.



After Mertens and Wright (52)

Figure 7. Approximate effective range of cationic and anionic emulsions on various types of aggregates.

It is important to note that not only are strength or stability criteria necessary for the determination of asphalt content but also a durability criterion is recommended by most agencies. Typical examples of durability tests are the immersion-compression test utilized by Winterkorn (13) and by Riley and Blumquist (49) and moisture vapor susceptibility that is utilized by Chevron Asphalt Company (18), The Asphalt Institute (50), and Finn et al. (51).

Emulsified Asphalts—The selection of the grade of emulsion can be conveniently determined from data given in Table 11, prepared by the U.S. Navy (20). Criteria are
40 based on the percentage passing the No. 200 sieve and the relative water content. The selection of either a cationic or an anionic emulsion should be based on the type of aggregate that is used. Mertens and Wright (52) have developed a method by which aggregate can be classified (Fig. 6) to indicate its probable surface charge and the type of emulsion (anionic or cationic) selected to satisfy particular aggregate surface characteristics (Fig. 7).

A preliminary selection of the quantity of emulsion can be obtained from data given in Table 12 (20). Other methods based on surface area concepts have been used by The Asphalt Institute (15) and Bird (53). The final selection of the quantity should be based on laboratory testing of the asphalt-soil mixture. Because the armed forces are equipped to perform Marshall tests, and apparently a better testing method with proven field performance is not available, the Marshall method with criteria suggested by

TABLE 12

EMULSIFIED ASPHALT REQUIREMENT

Percent Passing No. 200 Sieve	Pounds of Emulsified Asphalt per 100 lb or Dry Aggregate When Percentage Passing No. 10 Sieve Is					
	50 or Less	60	70	80	90	100
0	6.0	6.3	6.5	6.7	7.0	7.2
2	6.3	6.5	6.7	7.0	7.2	7.5
4	6.5	6.7	7.0	7.2	7.5	7.7
6	6.7	7.0	7.2	7.5	7.7	7.9
8	7.0	7.2	7.5	7.7	7.9	8.2
10	7.2	7.5	7.7	7.9	8.2	8.4
12	7.5	7.7	7.9	8.2	8.4	8.6
14	7.2	7.5	7.7	7.9	8.2	8.4
16	7.0	7.2	7.5	7.7	7.9	8.2
18	6.7	7.0	7.2	7.5	7.7	7.9
20	6.5	6.7	7.0	7.2	7.5	7.7
22	6.3	6.5	6.7	7.0	7.2	7.5
24	6.0	6.3	6.5	6.7	7.0	7.2
25	6.2	6.4	6.6	6.9	7.1	7.3

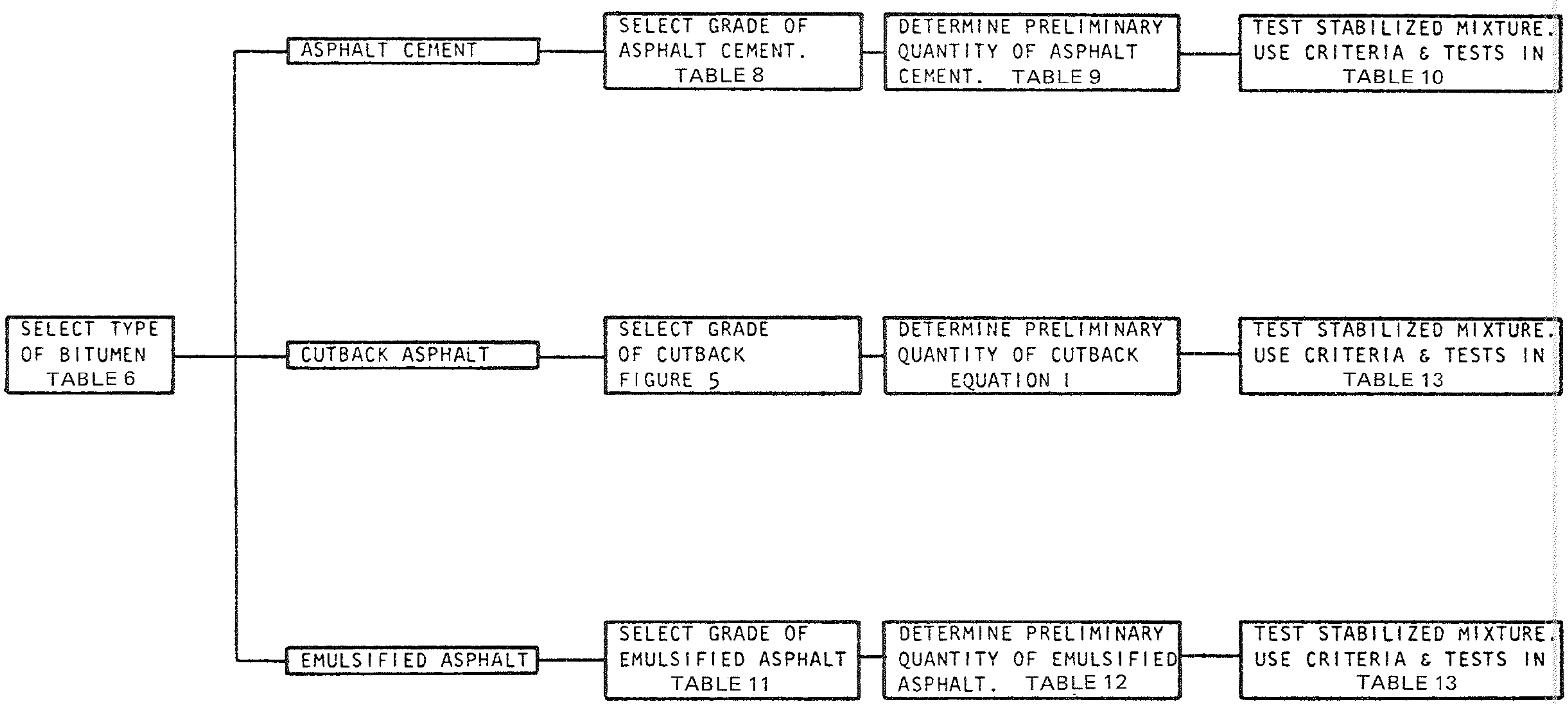


Figure 8. Subsystem for base course stabilization with bituminous materials.

Lefebvre (54) is suggested for use (Table 13). It should be recognized that this test is performed at 77 F.

The design subsystem for bituminous stabilization is shown in Figure 8.

TABLE 13

MARSHALL MIX DESIGN CRITERIA FOR CUTBACK AND EMULSIFIED ASPHALT MIXTURES

Marshall Test	Criteria for a Test Temperature of 77 F	
	Minimum	Maximum
Stability, lb	750	—
Flow, 0.01 in.	7	16
Air voids, percent	3	5

SUMMARY

A system utilizing currently available information has been developed to aid the engineer in the selection of a stabilizer or stabilizers for particular soil types. In addition, design subsystems have been developed to aid the engineer in the selection of the quantity of stabilizer for particular applications.

Many of the criteria utilized are based on observations and experience gained in constructing highway pavements. Because the Air Force is primarily concerned with airfield construction, validation or adjustment of these criteria may be necessary.

Equipment and environmental factors have not been included in the detail desired. In particular, field durability of the stabilized mixture is not well documented and, thus, suitable test methods are not always available to evaluate this important factor.

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STATE OF THE ART: LIME STABILIZATION

Reactions, Properties, Design, Construction

subject areas
25 pavement design
33 construction
62 foundations (soils)
64 soil science

Prepared by Transportation Research Board Committee on Lime and Lime-Fly Ash Stabilization

INTRODUCTION

Various forms of lime have been successfully utilized as a soil stabilizing agent for many years including products with varying degrees of purity. However, the most commonly used products are hydrated high calcium lime $Ca(OH)_2$, monohydrated dolomitic lime $Ca(OH)_2 \cdot MgO$, calcitic quicklime CaO, and dolomitic quicklime CaO·MgO. The use of quicklime for soil stabilization has increased during the past few years; in the United States it now accounts for more than 10 percent of the total stabilization lime while in Europe quicklime is the major type used.

Many significant engineering properties of soils are beneficially modified by lime treatment. Although lime is primarily utilized to treat fine-grained soils, it also can be used to modify the characteristics of the fine fraction of more granular soils.

There are several objectives for lime treatment of soils such as to expedite construction, modify subgrade soils, and improve strength and durability of fine-grained soils.

Lime-treated soils have been used in pavement construction as modified subgrades, subbase materials, and base materials. The position of the lime-treated soil layer in the pavement system is controlled by the quality of the lime-treated soil and other pavement design considerations. Railroad subgrades have also been successfully stabilized with lime.

In this report, the major aspects of soil-lime treatment are considered. The report represents the state-of-the-art in lime treatment based on a comprehensive analysis of current practice and the technical literature. For those desiring more detailed information, an extensive listing of references has been included.

This report was prepared by Transportation Research Board Committee A2J03, Lime and Lime-Fly Ash Stabilization. Various Task Groups prepared the different sections of the report. The final version of the report was reviewed by Committee A2J03 prior to publication.

SOIL-LIME REACTIONS

General

The addition of lime to a fine-grained soil initiates several reactions. Cation exchange and flocculation-agglomeration reactions take place rapidly and produce immediate changes in soil plasticity, workability, and the immediate uncured strength and load-deformation properties. Depending on the characteristics of the soil being stabilized, a soil-lime pozzolanic reaction may occur. The pozzolanic reaction results in the formation of various cementing agents which increase mixture strength and durability. Pozzolanic reactions are time dependent; therefore, strength development is gradual but continuous for long periods of time amounting to several years in some instances. Temperature also affects the pozzolanic reaction. Temperatures less than 13 to 16° C (55 to 60° F) retard the reaction and higher temperatures accelerate the reaction (Ref 4).

Lime carbonation is an undesirable reaction which may also occur in soil-lime. Construction should be carried out in such a fashion that lime carbonation is minimized.

Cation Exchange and Flocculation-Agglomeration

Practically all fine-grained soils display cation exchange and flocculation-agglomeration reactions when treated with lime. The reactions occur quite rapidly when soil and lime are intimately mixed.

The general order of replaceability of the common cations associated with soils is given by the lyotropic series, $Na^+ < K^+ < Ca^{++} < Mg^{++}$ (Ref 50). Cations tend to replace cations to the left in the series and monovalent cations are usually replaceable by multivalent cations. The addition of lime to a soil in sufficient quantities supplies an excess of Ca^{++} and cation exchange will occur, with Ca^{++} replacing dissimilar cations from the exchange complex of the soil. In some cases the exchange complex may be Ca^{++} saturated before the lime addition and cation exchange does not take place, or is minimized.

Flocculation and agglomeration produce an apparent change in texture with the clay particles "clumping" together into larger sized "aggregates". According to Herzog and Mitchell (Ref 55) the flocculation and agglomeration is caused by the increased electrolyte content of the pore water and as a result of ion exchange by the clay to the calcium form. Diamond and Kinter (Ref 39) suggested that the rapid formation of calcium aluminate hydrate cementing materials are significant in the development of flocculation-agglomeration tendencies in soil-lime mixtures.

Soil-Lime Pozzolanic Reaction

The reactions between lime, water, and various sources of soil silica and alumina to form cementing type materials are referred to as soil-lime pozzolanic reactions. Possible sources of silica and alumina in typical soils include clay minerals, quartz, feldspars, micas and other similar silicate or alumino-silicate minerals, either crystalline or amorphous in nature.

When a significant quantity of lime is added to a soil, the pH of the soil-lime mixture is elevated to approximately 12.4, the pH of saturated lime water. This is a substantial pH increase compared to the pH of natural soils. The solubilities of silica and alumina are greatly increased at elevated pH levels (Ref 60).

In an early study of soil-lime reactions, Eades (Ref 43) suggested that the high pH causes silica to be dissolved out of the structure of the clay minerals, thereby becoming available to combine with the Ca^{++} to form calcium silicates and that this reaction will continue as long as $Ca(OH)_2$ exists in the soil and there is available silica. Diamond et al (Ref 41) postulated that the reaction processes in the highly alkaline soil-lime system involved a dissolution at the edges of the silicate particles followed by the precipitation of the reaction products.

Although the work of Eades (Ref 43) and Diamond et al (Ref 41) generally suggest a "through-solution" mechanism in which clay lattice components are "dissolved" from the clay structure and reprecipitated as CSH and CAH, direct reaction of the lime at the surface of clay mineral particles has not been ruled out. Recent work in the adsorption of lime by kaolinite and montmorillonite (Ref 40) as well as electron optical work on clay-lime-water systems (Ref 85) tends to support the idea that surface chemical reactions can occur and new phases may nucleate directly upon the surfaces of the clay particles. It is also possible that the reactions may occur by a combination of through-solution (solution-precipitation) and surface chemical (hydration-crystallization) processes.

An oversimplified qualitative view of some typical soil-lime reactions is summarized below:

$$Ca(OH)_2 \text{ -------------> } Ca^{++} + 2(OH)$$

$$Ca^{++} + 2(OH)^- + SiO_2 \text{ (Clay Silica) -------> CSH}$$

$$Ca^{++} + 2\ (OH)^- + Al_2O_3 \text{ (Clay Alumina) ----> CAH}$$

A wide variety of hydrate forms can be obtained, depending on reaction conditions, e.g., quantity and type of lime, soil characteristics, curing time, and temperature. Typical soil-lime reactions are:

Kaolinite + lime ---> CSH (C/S = 0.2-1) + CAH + CASH (Ref 80)

Kaolinite + lime ---> CASH (prehnite) (Ref 93)

Montmorillonite + lime ---> CSH (gel) ---> CSH (II) (Ref 91)

Montmorillonite + lime ---> CSH (gel) + hydrogarnet C_4AH_{13} (Ref 112)

Montmorillonite + lime ---> CSH (gel) + CSH (1) + tobermorite + hydrogarnet

Clay + lime ---> CHS (gel) and/or CSH (1) + C_4AH_{13} + C_3AH_6 (Ref 41)

where

C = CaO ,
S = SiO_2 ,
A = Al_2O_3 , and
H = H_2O .

The extent to which the soil-lime pozzolanic reaction proceeds is influenced primarily by natural soil properties. With some soils, the pozzolanic reaction is inhibited, and cementing agents are not extensively formed. Thompson (Ref 101) has termed those soils that react with lime to produce substantial strength increase, i.e., greater than 34.5 N/cm^2 (50 psi) following 28 day curing at 22.8° C (73° F), as reactive and those that display limited pozzolanic reactivity, less than a 34.5 N/cm^2 (50 psi) strength increase, are called nonreactive.

Some of the major soil properties and characteristics which influence the lime-reactivity of a soil, i.e., ability of the soil to react with lime to produce cementitious materials, are soil pH, organic carbon content, natural drainage, presence of excessive quantities of exchangeable sodium, clay mineralogy, degree of weathering, presence of carbonates, extractable iron, silica-sesquioxide ratio, and silica-alumina ratio. Detailed summaries concerning the effects of soil properties on lime reactivity are contained in Refs 53, 54, and 101. It is emphasized that the main factors controlling the development of cementitious materials in a lime treated soil are the inherent properties and characteristics of the soil. If a soil is nonreactive, extensive pozzolanic strength development will not be achieved regardless of lime type, lime percentage, or curing conditions of time and temperature.

Those desiring more extensive and detailed background information on basic soil-lime reactions should refer to the "Interpretive Review" by Diamond and Kinter (Ref 39) and a recent comprehensive publication by Stocker (Ref 95).

Summary

Soil-lime reactions are complex and not completely understood at this time. However, sufficient basic understanding and successful field experience are available to provide the basis of an adequate technology for successfully utilizing soil-lime stabilization under a wide variety of conditions. Future research findings will further augment our technology and permit more refined engineering decisions to be made concerning lime treatment of soils.

PROPERTIES AND CHARACTERISTICS OF LIME-TREATED SOILS

In general, when mixed with lime all fine-grained soils exhibit improved plasticity, workability and volume change characteristics; however, not all soils exhibit improved strength, stress-strain, and fatigue characteristics. It should be emphasized that the properties of lime-soil mixtures are dependent on many variables (Refs 100 and 101). Soil type, lime type, lime percentage, and curing conditions including time, temperature, and moisture are the most important variables. More important, however, the effect produced by any given change in a given variable is dependent on the levels of the other variables.

At present only limited information is available concerning some of the properties of lime-treated soils. Nevertheless, in order to effectively utilize these treated soils as a structural material, it is necessary to evaluate and summarize the existing knowledge concerning the properties of soil-lime mixtures.

Compaction Characteristics

The compaction characteristics, i.e., maximum density and optimum moisture, are important for two basic reasons. First of all, an adequate level of compaction must be obtained in order to achieve satisfactory results. Secondly, and possibly more important, is the fact that density is used for field control.

When compacted with a given effort, soil-lime mixtures have a lower maximum density than the original untreated soil and the maximum density normally continues to decrease as the lime content is increased. In addition, the optimum moisture content increases with increasing lime contents (Fig. 1). Similarly, if the mixture is allowed to cure such that substantial cementing occurs the density would be further decreased and the optimum moisture increased.

Thus, moisture-density relationships are constantly changing, and it is important that the proper curve be utilized in field construction. Thus, if curing has occurred, it may be impossible to achieve density; however, it is important to realize that it is not necessary to achieve that density because the reduction is not due to poor compaction but rather to the fact that the material is different.

Plasticity and Workability

Substantial reduction in plasticity, i.e., reduced plasticity index PI, increased shrinkage limit, is produced by lime treatment, and in many cases the soil may become nonplastic. Generally, soils with a high clay content or exhibiting a high initial PI require greater quantities of lime for achieving the nonplastic condition, if it can be achieved at all. The first increments of lime addition are generally most effective in reducing plasticity, with subsequent additions being less beneficial (Ref 100). The reduced plasticity of the lime-treated soil and its silty and friable texture cause a significant improvement in workability and expedite subsequent manipulation and working of the treated soil. Figure 2 and Table 1 illustrate the manner in which lime influences the plasticity characteristics.

Volume Change

Swelling potential and swelling pressures normally are significantly reduced by treating clay with lime. These reduced swell characteristics are generally attributed to decreased affinity for water of the calcium saturated clay and the formation of a cementitious matrix which resists volumetric expansion. CBR-swell values of lime treated soils vary, but it is not uncommon to decrease swell to less than 0.1 percent compared to values of 7 to 8 percent for the untreated soil (Table 2). Typical expansive pressures (Ref 49) are shown in Fig 3.

Shrinkage due to moisture loss from the stabilized soil is of importance relative to the problem of shrinkage cracking. Lime treatment improves the shrinkage and swell characteristics of the treated materials. Figure 4 contains data (Ref 37) for typical Illinois soils.

Field moisture contents of lime treated soils suggest that the moisture content changes in the stabilized material are not large and the in-situ water content stabilizes at approximately optimum (Ref 106). Theoretical calculations based on laboratory shrinkage data as well as field service data from many areas indicate that for typical field conditions shrinkage will not be extensive (Ref 106).

Strength

The strength of lime-soil mixtures can be evaluated in many ways. The unconfined compression test is the most popular procedure while the stabilometer and CBR tests are used to a lesser extent. These methods, however, are definitely not the most applicable or desirable. Only limited data are available concerning the tensile properties of lime-soil mixtures (Refs 78, 82, 105, and 109), and additional effort is needed to evaluate the tensile characteristics of lime-treated materials.

It should be emphasized that the strength of a soil-lime mixture is dependent on many variables and that it varies substantially (Refs 100 and 101). Soil type, lime type, lime percentage, curing conditions of time and temperature, and the interactions between these variables are the major factors influencing strength (Refs 78, 82, and 109).

A distinction must be made with respect to curing. An immediate beneficial strength effect occurs with the addition of lime due to the immediate reactions, i.e., cation exchange, flocculation, and agglomeration. The long-term strength gain is primarily related to the pozzolanic reaction. Thus, it is necessary to separate the discussion into cured and uncured strength.

Uncured Strength

Immediately after the addition of lime a substantial improvement in strength and stability can be expected (Refs 84 and 110). These immediate effects can be considered to be an expedient for construction when soft, highly plastic, cohesive soils create mobility problems for wheeled vehicles (Fig 5) or do not provide satisfactory subgrade support for pavement construction operations.

Examples of the immediate effect of lime treatment on cone index, CBR, and unconfined compressive strength are shown in Fig 6. From this figure it is apparent that substantial improvements in strength can be realized. In some cases these increases may amount to several hundred percent.

Cured Strength

Unconfined Compression. Unconfined compressive strengths of typical fine-grained soils compacted at optimum moisture content and density (ASTM D2166) range from about 17 N/cm^2 (25 psi) to more than 207 N/cm^2 (300 psi) depending on the nature of the soil. Soil-lime mixture strength increases for Illinois soils cured 28 days at 22.8° C (73° F) range up to approximately 183 N/cm^2 (265 psi) with many soils displaying increases greater than 70 N/cm^2 (100 psi). Extended curing of 56 days at 22.8° C (73° F) of the same mixtures produced strength increases for some soil-lime combinations that exceeded 430 N/cm^2 (625 psi). Prolonged curing for 75 days at 48.9° C (120° F) of the AASHTO Road Test embankment soil treated with

Figure 1. Typical moisture-density relationships (Ref 84).

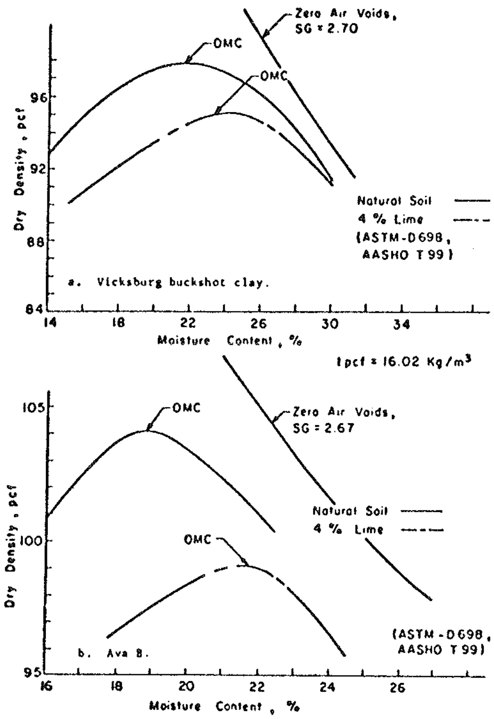


Figure 2. Effect of lime on plasticity characteristics of montmorillonite clays (Ref 56).

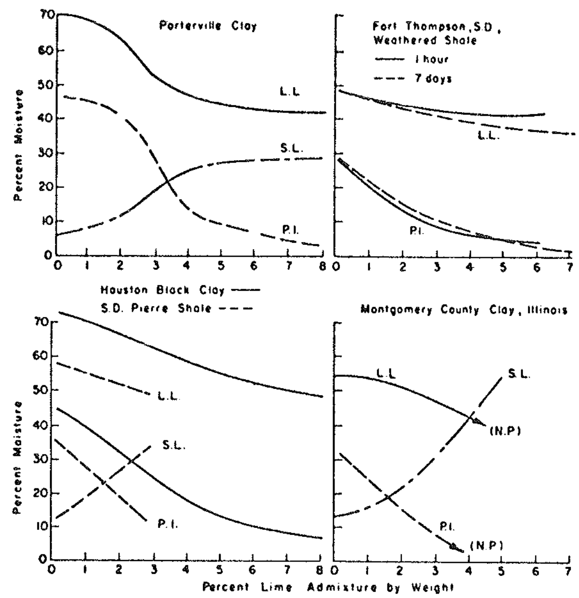
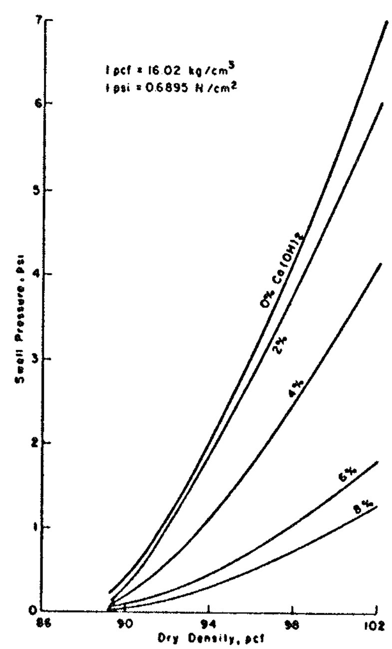


Table 1. Influence of lime on plasticity properties (Ref 100).

Soil	AASHO Class	Liquid Limit (LL) or Plasticity Index (PI), % — Percent Lime 0 LL	0 PI	3 LL	3 PI	5 LL	5 PI
Bryce B	A-7-6(18)	53	29	48	21	NP	
Cisne B	A-7-6(20)	59	39	NP		—	
Cowden B	A-7-6(19)	54	33	47	7	NP	
Drummer B	A-7-6(19)	54	31	44	10	NP	
Elliott B	A-7-6(18)	53	28	42	19	NP	
Fayette B	A-7-5(17)	50	29	NP		—	
Hosmer B_2	A-7-6(11)	41	17	NP		—	
AASHO Road Test	A-6(18)	25	11	27	6	27	5
Huey B	A-7-6(17)	46	29	40	9	NP	
Sable B	A-7-6(16)	51	24	NP		—	

NP = Nonplastic

Figure 3. Swell pressure-density relations for lime-treated Porterville clay (Ref 49).



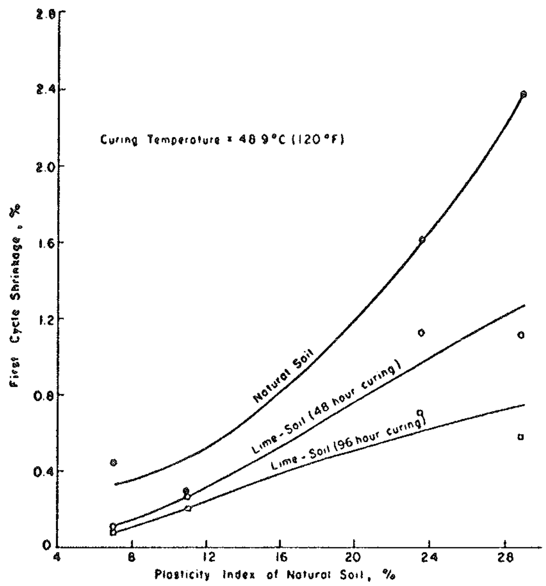


Figure 4. Influence of plasticity index of natural soil on first cycle shrinkage (Ref 37).

Figure 5a. Partially completed stabilized subgrade resists rutting during rain—Dallas-Fort Worth Airport.

Figure 5b. Comparison of ruts in untreated and lime-treated subgrade.

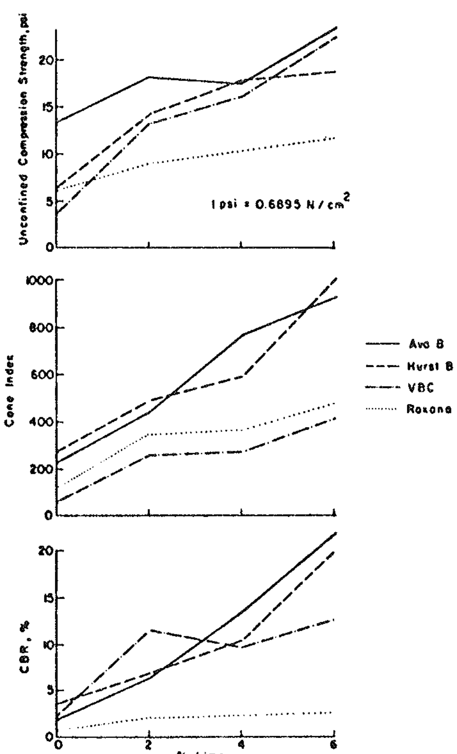


Figure 6. Immediate effects of lime treatment on strength (Ref 84).

5 percent lime produced an average compressive strength of 1090 N/cm^2 (1580 psi). Field data indicate that with some soil-lime mixtures, strength continues to increase with time up to and in excess of ten years. Typical results for various densities are shown in Fig 7.

The difference between the compressive strengths of the natural and lime-treated soil has been used as an indication of the degree to which the soil-lime pozzolanic reaction has proceeded (Ref 101). Substantial strength increase indicates that the soil is reactive with lime and can probably be stabilized to produce a quality paving material.

Shear Strength. Unconsolidated and undrained type triaxial testing have been utilized to partially simulate field service conditions.

The major effect of lime on the shear strength of a reactive fine-grained soil is to produce a substantial increase in cohesion with some minor increase in the angle of internal friction. At the low confining pressures normally considered to exist in a flexible pavement structure, the cohesion increase is of the greatest significance. For materials such as soil-lime mixtures which are characterized by very high cohesion, it is difficult to effectively evaluate the angle of internal friction.

For the typical lime reactive Illinois soils, the angle of internal friction for soil-lime mixtures ranged from 25° to 35° (Ref. 103). The cohesion of the mixtures was substantially increased compared to the natural soils and cohesion continued to increase with increased unconfined compressive strength. Using the linear regression equation shown in Fig. 8, cohesion values can be estimated from unconfined compressive strength results.

It is apparent that large shear strengths can easily be developed in cured soil-lime mixtures. It has been demonstrated that if high quality mixtures are used in typical flexible pavement structures, the strengths would be adequate to prevent shear failure (Ref. 103). Shear-type failures generally have not been observed and reported for field service conditions.

Tensile Strength. Tensile strength properties of soil-lime mixtures are of concern in pavement design because of the slab action that is afforded by a material possessing substantial tensile strength. Two test procedures, indirect tensile and flexural, have been used for evaluating the tensile strength of soil-lime mixtures.

The indirect tensile test is essentially a diametral compression test in which the material fails in tension along the loaded diameter of the cylindrical test specimen. Details and an evaluation of the test procedure for soil-lime mixtures are presented in Refs 2, 78, 82, 105, and 109.

Typical results (Fig 9) indicate that the mixtures can possess substantial tensile strength. The ratio of indirect tensile strength to unconfined compressive strength in one study (Ref 105) was found to be approximately 0.13, while in another study (Ref 109) it was found to be much lower as indicated by the following regression equation:

$$S_T = 6.89 + 50.6\ q_u$$

where

S_T = tensile strength, psi

q_u = unconfined compressive strength, ksi

The most common method used for evaluating the tensile strength of highway materials has been the flexural test. Typical flexural strengths of soil-lime mixtures (Ref 96) subjected to various curing conditions are shown in Table 3. Indirect tensile strengths are shown for comparison purposes. For a specific mixture, the ratio of the flexural strength to indirect tensile strength decreases as strength increases and the ratio is apparently not the same for all soil-lime mixtures.

If the ratio of flexural strength to indirect tensile strength is taken as approximately 2, a realistic estimate of flexural strength is 25 percent of the unconfined strength. The ratio is approximately equivalent to those reported for lime-flyash-aggregate and soil-cement mixtures.

California Bearing Ratio. The CBR testing procedures have been extensively used to evaluate the strength of lime stabilized soils. Many agencies have arbitrarily adopted this technique because of their familiarity with the test. In reality, however, the CBR test is not appropriate for characterizing the strength of cured soil-lime mixtures.

Extensive CBR tests have been conducted (Ref 99) with various representative Illinois soils including soils that reacted well with lime and also less reactive fine-grained soils.

Lime-treated soils were cured for 48 hours at 48.9° C (120° F) and companion specimens which had not been cured were placed in the 96-hour soaking cycle immediately after compaction. The 48-hour curing period is approximately equivalent to 30 days at 21.1° C (70° F) and the mixtures that were not cured prior to soaking had little opportunity to develop cementitious products from the soil-lime pozzolanic reaction. The improvements in engineering properties of the no-cure soil-lime mixtures were therefore primarily due to the cation exchange, flocculation, and agglomeration produced by the addition of lime. Test results for the natural soils and the soil-lime mixtures are presented in Table 2. The CBR increases of the no-cure soil-lime mixtures show the benefits that can be obtained from stabilization without prolonged curing. It is apparent that the no-cure specimens have not developed extensive cementing action.

The CBR values for many of the soil-lime mixtures cured for 48 hours at 48.9° C (120° F) are quite large and definitely indicate the extensive development of cementing agents. For those mixtures that display CBR values of 100 or more, the test results have little practical significance and are not meaningful as a measure of strength or stability. In general, these materials would also exhibit high compressive and tensile strengths, and these types of tests would provide a better strength evaluation. If extensive cementing action has not developed due to either lack of curing time or non-reactivity of the treated soil, CBR values may serve as a general measure of strength; but even in these cases the use of the CBR test is questionable.

It is evident that lime treatment of fine-grained soils produces increased CBR irrespective of the length of curing and lime-reactivity of the soil.

Stress-Strain Characteristics

Stress-strain properties are essential for properly analyzing the behavioral characteristics of a pavement structure containing a soil-lime mixture structural layer. The marked effect of lime on the compressive stress-strain properties of fine-grained soils is shown in Fig 10. The failure stress is increased, and the ultimate strain is decreased for soil-lime mixtures relative to the natural soil. As with strength it is necessary to separate the discussion with regard to whether the soil-lime has been cured or not since immediate beneficial effects

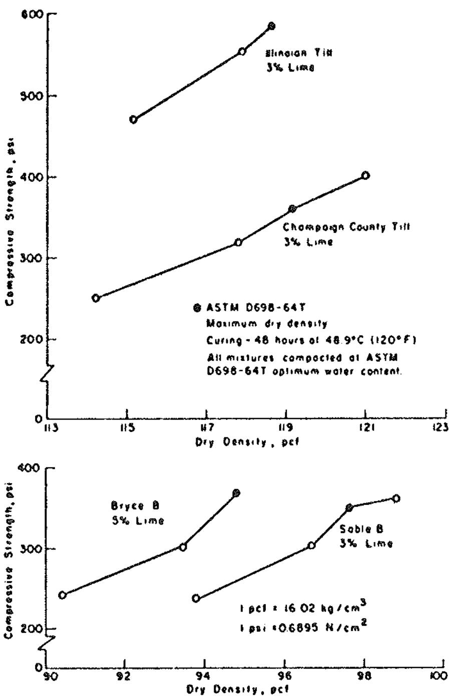


Figure 7. Influence of density on the strength of cured soil-lime mixtures (Ref 99).

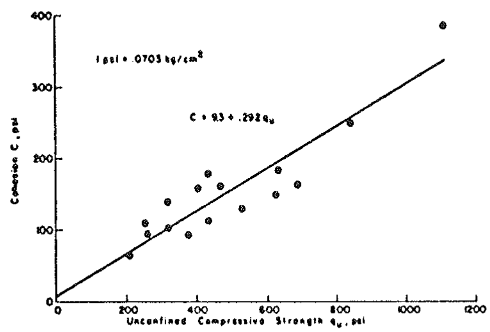


Figure 8. Cohesion vs unconfined compressive strength of soil-lime mixtures (Ref 103).

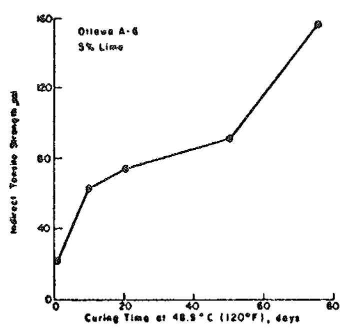


Figure 9. Indirect tensile strength of cured soil-lime mixture (Ref 105).

Table 2. CBR values for selected soils and soil-lime mixtures (Ref 99).

Soil	Natural Soil		Percent Lime	Soil-Lime Mixtures			
				No Curing[1]		48 Hours Curing, 48.9° C (120° F)	
	CBR, %	Swell, %		CBR, %	Swell, %	CBR, %	Swell, %
Good Reacting Soils							
Accretion Gley 2	2.6	2.1	5	15.1	0.1	351.0	0.0
Accretion Gley 3	3.1	1.6	5	88.1	0.0	370.0	0.1
Bryce B	1.4	5.6	3	20.3	0.2	197.0	0.0
Champaign Co. Till	6.8	0.2	3	10.6	0.5	85.0	0.1
Cisne B	2.1	0.1	5	14.5	0.1	150.0	0.1
Cowden B	7.2	1.4	3	—	—	98.5	0.0
Cowden B	4.0	2.9	5	13.9	0.1	116.0	0.1
Cowden C	4.5	0.8	3	27.4	0.0	243.0	0.0
Darwin B	1.1	8.8	5	7.7	1.9	13.6	0.1
East St. Louis Clay	1.3	7.6	5	5.6	2.0	17.3	0.1
Fayette C	1.3	0.0	5	32.4	0.0	293.0	0.1
Illinoian B	1.5	1.8	3	29.0	0.0	274.0	0.0
Illinoian Till	11.8	0.3	3	24.2	0.1	193.0	0.0
Illinoian Till	5.9	0.3	3	18.0	0.9	213.0	0.1
Sable B	1.8	4.2	3	15.9	0.2	127.0	0.0
Non-Reactive Soils							
Fayette B	4.3	1.1	3	10.5	0.0	39.0	0.0
Miami B	2.9	0.8	3	12.7	0.0	14.5	0.0
Tama B	2.6	2.0	3	4.5	0.2	9.9	0.1

[1] Specimens were placed in 96-hour soak immediately after compaction.

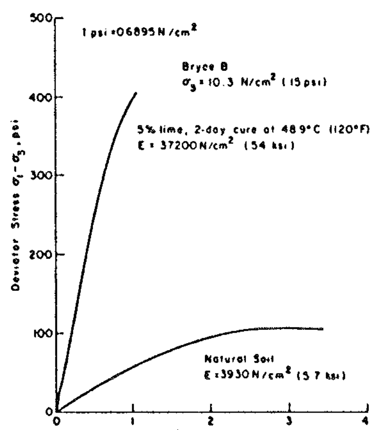


Figure 10. Typical stress-strain curves for natural and lime-treated soil (Ref 103).

occur which relate to improved workability and construction.

Uncured Soil-Lime

Figure 11 shows typical improved stress-strain characteristics which occur without curing and indicates the general nature of the modification attained from lime treatment.

As indicated in Fig 11, substantial increases in modulus of deformation can be expected. Figure 12 illustrates typical stress-strain relationships for soil and soil-lime which were compacted on the wet side of optimum to simulate a wet field condition during construction.

Cured Soil-Lime

As a result of an extensive study of representative Illinois soils stabilized with lime (Ref 103), it was possible to develop a generalized compressive stress-strain relation for cured soil-lime mixtures (Fig 13). The mixtures studied appeared to be strain susceptible, and the ultimate strain at maximum compressive stress was approximately 1 percent, regardless of the soil type or curing period.

Modulus of Deformation or Elasticity. It was found (Ref 103) that the compressive modulus of elasticity at a confining pressure of 1.05 kg/cm^2 (15 psi) could be estimated from the unconfined compressive strength of the lime-soil mixture according to the following relation:

$$E = 9.98 + .124\, q_u$$

where

E = compressive modulus of elasticity, ksi
q_u = unconfined compressive strength, psi

For soil-lime pavement layers possessing high shear strength, the flexural stresses in the mixture may be the controlling design factor. In view of this fact, flexural moduli of elasticity have been evaluated for typical cured soil-lime (Ref 103).

Typical Illinois soils were stabilized with lime, and beams with dimensions of 5.08 cm x 5.08 cm x 22.86 cm (2 in. x 2 in. x 9 in.) were prepared and cured for 48 and 96 hours at 48.9° C (120° F). After curing, strain gauges were attached to the midportion of the beams and the beams were tested under third-point loading conditions.

The modulus of elasticity in flexure was calculated from the moment-curvature relationships for the beams, and the relationship between the modulus of elasticity and the flexural strength was calculated (Fig 14). For the range of data considered, it was concluded that the regression equation shown in Fig 14 can be used to estimate the flexural modulus of elasticity. It should be noted that flexural moduli were substantially larger than compressive moduli for the same mixture.

Repeated compressive loading data for soil-lime mixtures are limited. Fossberg (Ref 45), utilizing a montmorillonitic clay treated with 10 percent lime, studied the influence of deviator stress and confining pressure on resilient modulus. The specimens were prepared at extremely high water contents and low densities. Consequently the data are not directly comparable with field conditions. The general relation between resilient modulus and principal stress ratio appeared to be linear and resilient moduli in excess of 69,000 N/cm^2 (100,000 psi) were noted for some test conditions, even under the rather unfavorable testing conditions involving high water content and low density.

Maxwell and Joseph (Ref 67) used a field vibratory testing procedure for evaluating the strength of an airfield pavement section containing a 15.2-centimeter (6-inch) lime-stabilized subgrade and an 20.3-centimeter (8-inch) lime-stabilized clay-gravel subbase. Based on periodic field-velocity measurements, computed elastic moduli for the stabilized subgrade ranged from 114,000 N/cm^2 (165,000 psi) following construction to 392,000 N/cm^2 (568,000 psi) approximately 2½ years after construction. Similar data for the lime-treated subbase were 135,000 N/cm^2 (196,000 psi) after construction and 696,000 N/cm^2 (1,010,000 psi) 2½ years later.

Poisson's Ratio. Only limited data are available for lime-soil mixtures. Reported values at stress levels less than 25 percent of ultimate compressive strength ranged from 0.08 to 0.12 with an average of 0.11 (Ref 99). These values are in agreement with those previously reported for rock, lime-flyash-aggregate mixtures, and soil-cement. At higher stress levels, greater than 50 to 75 percent of ultimate compressive strength, Poisson's ratio increased, ranging from 0.27 to 0.37 with an average of 0.31. A similar type of behavior has been noted for lime-flyash-aggregate mixtures. The influence of stress level, expressed as a percent of ultimate compressive strength, on Poisson's ratio for soil-lime mixtures is shown in Fig 15.

General. Since the properties of a soil-lime mixture change with increased curing time, it may not be justified to conduct elaborate tests to precisely evaluate properties that will change due to field curing effects. It may be more desirable to use unconfined compressive strength or the indirect tensile test for evaluating the quality of the mixtures. Use of correlations in place of testing is discouraged since these correlations depend on the conditions for which they were developed and can produce large errors. Correlations should be used only when there is no other alternative or the desired property cannot be measured and then only with caution.

Fatigue Characteristics

The flexural strength of soil-lime mixtures is important to its use in subbase and base courses. Flexural fatigue data developed for typical Illinois soils are shown in Fig 16.

The response curves are typical of fatigue in general and are similar to the curves normally obtained for similar materials such as lime-flyash-aggregate mixtures and concrete. The fatigue strengths at 5 million stress repetitions of the lime-soil mixtures varied from 41 to 66 percent of the ultimate flexural strength with an average of 54 percent.

More important is the behavior of lime-treated mixtures when subjected to repeated applications of tensile stresses such as in the indirect tensile test or the direct tensile test. Preliminary fatigue experiments using the indirect tensile test indicated that this test is quite applicable to the study of lime-treated materials (Ref 81).

Soil-lime mixtures continue to gain strength with time and the ultimate strength of the mixture is a function of curing period and temperature. The magnitudes of the stress repetitions applied to the mixture are relatively constant throughout its design life. Therefore, as the ultimate strength of the material increases due to curing, the stress level, as a percent of ultimate strength, will decrease and

Table 3. Tensile strength properties of soil-lime mixtures (Ref 99).

Soil	Percent Lime	Curing Time, Hours*	Flexural Strength σ_F, N/cm^2 (psi)	Indirect Tensile Strength σ_T, N/cm^2 (psi)	σ_F/σ_T
Bryce B	5	24	63 (92)	29 (42)	2.2
		48	72 (105)	37 (53)	2.0
		96	84 (122)	61 (88)	1.4
Champaign County Till	3	48	48 (69)	**	—
		96	64 (93)	**	—
Fayette C	5	24	45 (66)	32 (46)	1.4
		96	114 (166)	87 (126)	1.3
Illinoian Till, Sangamon County	3	24	59 (86)	24 (35)	2.5
		48	113 (164)	63 (92)	1.8
		96	139 (202)	73 (106)	1.9
Sable B	3	48	43 (63)	**	—
		96	53 (77)	**	—
Wisconsin Loam Till	3	24	57 (83)	24 (35)	2.4
		48	97 (140)	43 (63)	2.2
		96	108 (157)	54 (78)	2.0

* At 48.9° C (120° F).
**Test not conducted.

Figure 11. Immediate effects of lime treatment on modulus of deformation (Ref 84).

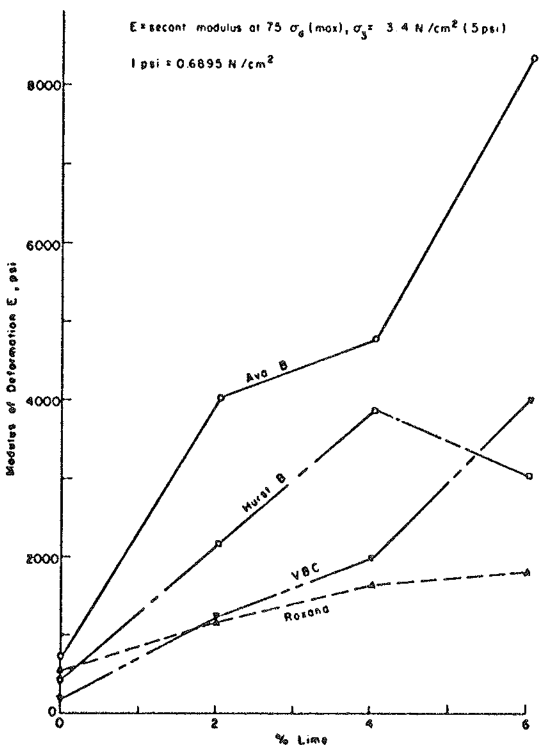


Figure 12. Typical stress-strain curve illustrating immediate effects of lime treatment (Ref 84).

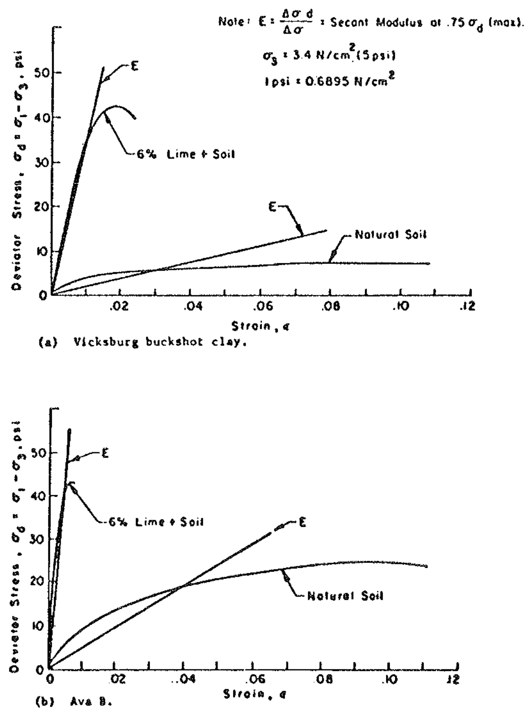


Figure 13. Generalized stress-strain relationship for cured soil-lime mixtures (Ref 103).

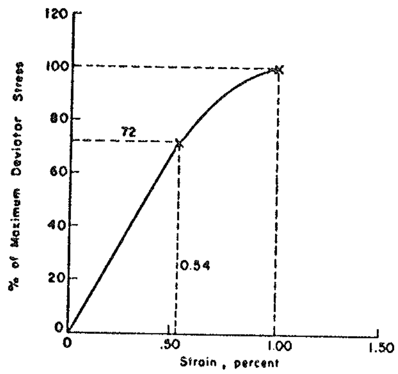


Figure 14. Relationship between flexural strength and flexural modulus for soil-lime mixtures (Ref 99).

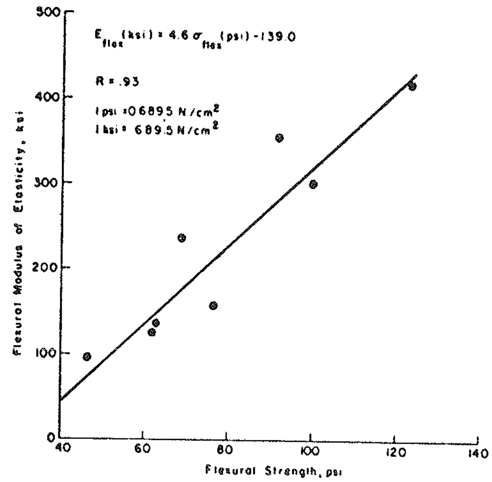


Figure 15. Influence of stress level on Poisson's ratio (Ref 99).

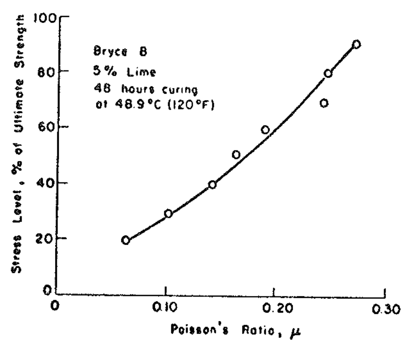
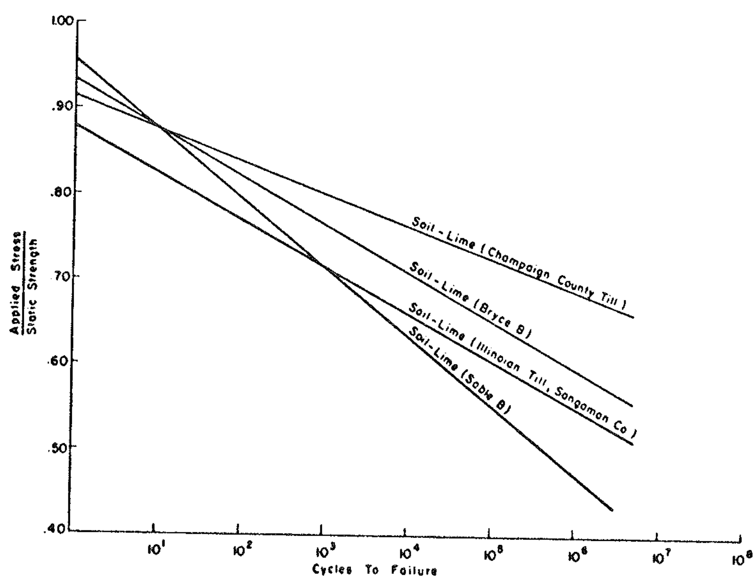


Figure 16. Flexural fatigue response curves (Ref 96).



the fatigue life of the mixture will increase.

Durability

The major durability consideration for soil-lime mixtures is the resistance to cyclic freezing and thawing. Prolonged exposure to water produces only slight detrimental effects and the ratio of soaked to unsoaked compressive strength is high at approximately 0.7 to 0.85 (Ref 106). Figure 17 shows the general relation between soaked and unsoaked strengths for typical lime stabilized Illinois soils. The soaked specimens seldom achieved 100 percent saturation, and in most cases the degree of saturation was in the range of 90 to 95 percent. Similar response to soaking has been noted in extensive studies conducted by the Road Research Laboratory, England (Ref 42).

In zones where freezing temperatures occur, rreeze-thaw damage may occur. The damage is generally characterized by volume increase and strength reduction as shown in Figs 18 and 19. The interrelation between length changes and compressive strength decreases is presented in Fig 20. The validity of using initial unconfined compressive strength as a measure of freeze-thaw resistance is demonstrated in Fig 21. Average rates of strength decrease for the typical mixtures were 6.21 N/cm^2/cycle (9 psi/cycle) and 12.4 N/cm^2/cycle (18 psi/cycle) for 48- and 96-hour curing at 48.9° C (120° F), respectively (Ref 36).

A study (Ref 107) has shown that some soil-lime mixtures display autogenous healing properties. If the stabilized soil has the ability to regain strength, or heal, with time, the distress produced during winter freeze-thaw cycles will not be cumulative since autogenous healing during favorable curing conditions would serve to restore the stability of the material. This phenomenon is illustrated in Fig 22. Confirming field data on autogenous healing have been presented by McDonald (Ref 68).

Durable soil-lime mixtures can be obtained when reactive soils are stabilized with lime. Although some strength reduction and volume change may occur, the residual strength of the stabilized materials is adequate to meet field service requirements. Durability considerations must be taken into account in establishing the mix design and selecting design strength parameters.

Variability of Properties

Analyses of testing error associated with repeat strength determinations of identical soil-lime mixture specimens have been reported by Liu and Thompson (Ref 63). The standard deviations for unconfined compression, indirect (split) tensile, and flexural strengths increased with increased strength and the average coefficients of variation. In general, the testing errors were approximately of the same magnitude, coefficients of variation of 11 to 12 percent, for the different testing procedures studied. This variation was for specimens prepared, cured, and tested in the laboratory. The variation for soil-lime mixtures constructed in the field would be substantially greater (Ref 66).

Factors contributing to testing variability include: a) heterogenous nature of soils; b) nonuniformity of mixtures; c) slight deviations in sample preparation and testing techniques; d) small variations in curing temperature and time; and e) density variations.

Moving from the laboratory to a field construction site, it could be expected that more variation would be introduced as a result of the relatively uncontrolled construction process as compared to the carefully controlled laboratory conditions. Additional variation may also be introduced with time during construction. The variation in material properties introduced along the roadway includes variation introduced by the environment, changes in the constituents of the mixture, changes in contractor or construction technique, and various other factors.

This variability should be recognized and considered in the evaluation of soil-lime mixtures.

Summary

An attempt has been made to summarize the basic characteristics and properties of soil-lime mixtures with respect to their engineering uses. These properties vary significantly depending on the type of soil, method and quality of construction, and type and length of curing. Thus, at this time it is not possible to define the actual properties. Only values can be provided. The use, evaluation, and mixture design procedures should be developed in terms of intended use, objectives, and test conditions. In addition, the evaluation should be based on meaningful tests which provide fundamental engineering properties rather than empirical test results. An attempt should also be made to recognize and consider the inherent variation in soil-lime mixtures.

SOIL-LIME MIXTURE DESIGN

General

The major objective of the mixture design process is to establish an appropriate lime content for construction. It is important to note that the primary variable that can be altered is lime percentage since the inherent properties and characteristics of the soil are essentially fixed. Because of the many varied applications of lime treatment of soils, several mixture design procedures have been developed which are described in this section. The general principle of soil-lime mixture design is that the mixture should provide satisfactory performance when constructed in a desired position in the pavement structure or the subgrade. It is apparent that a wide range of soil-lime mixtures of varying quality can be successfully used to accomplish differing lime treatment objectives. Design lime contents generally are based on an analysis of the effect of various lime percentages on selected engineering properties of the soil-lime mixture. Engineering properties which are considered, depending on the stabilization objectives, are Atterberg limits, i.e., liquid limit, plastic limit, and plasticity index, swell potential, and strength of cured or uncured mixtures.

Mixture design criteria are needed to establish the quantity of lime required to produce an acceptable quality mixture. For some stabilization objectives and soils, acceptable soil-lime mixtures may not be produced regardless of the lime percentage used.

Laboratory Testing Procedures

Many different laboratory testing procedures have been utilized in the various mixture design methods. Specific details of the various procedures have not been included in this report, however, general considerations are summarized below.

Test methods which have been used in the design of soil-lime mixture include (1) Atterberg limits, (2) California Bearing Ratio, (3) Hveem stabilometer or R-value, (4) swell tests, and (5) unconfined compression. Laboratory testing involves soil-lime mixture preparation, specimen preparation,

Figure 17. Influence of soaking on the strength of cured soil-lime mixtures (Ref 106).

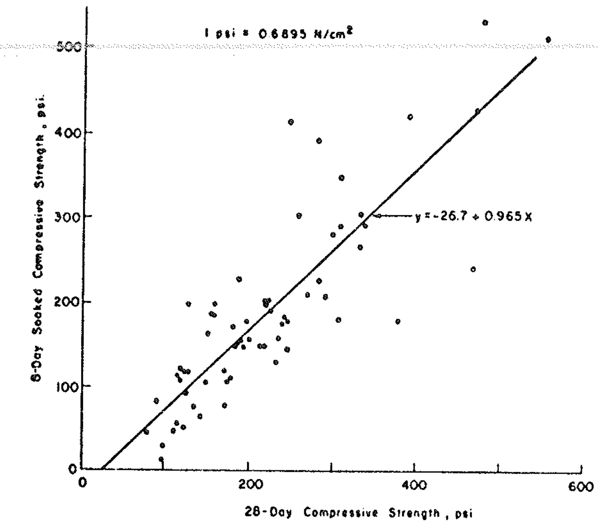


Figure 18. Influence of freeze-thaw cycles on unit length change (48-hour curing) (Ref 36).

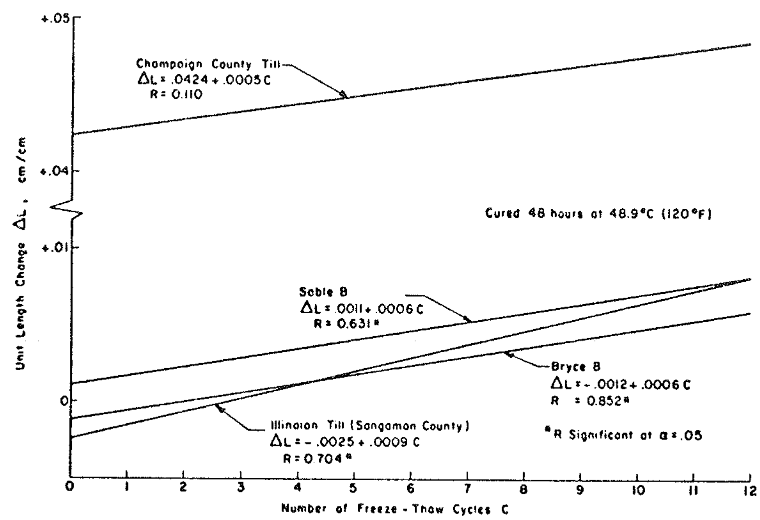


Figure 19. Influence of freeze-thaw cycles on unconfined compressive strength (Ref 36).

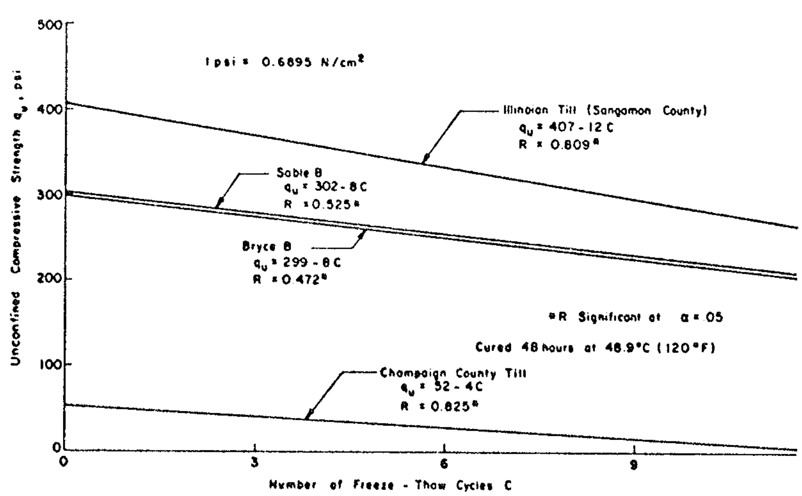


Figure 20. Relationship between unit length change and strength decrease with freeze-thaw cycles (Ref 36).

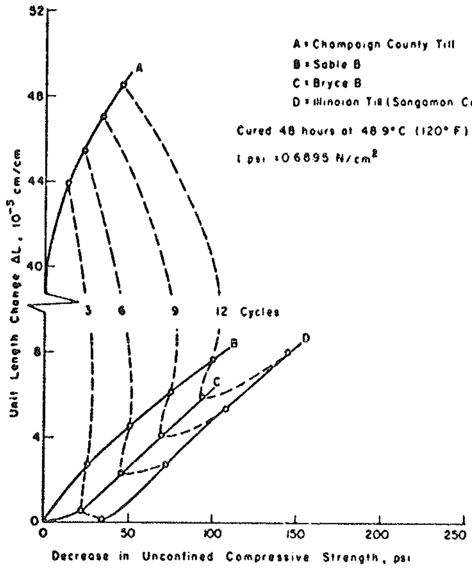


Figure 21. Influence of initial unconfined compressive strength on the residual strength after freeze-thaw cycles (Ref 36).

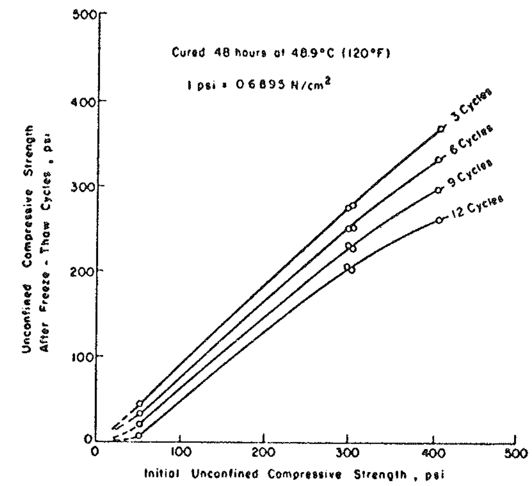


Figure 22. Influence on cylic freeze-thaw and intermediate curing on unconfined compressive strength (Ref 107).

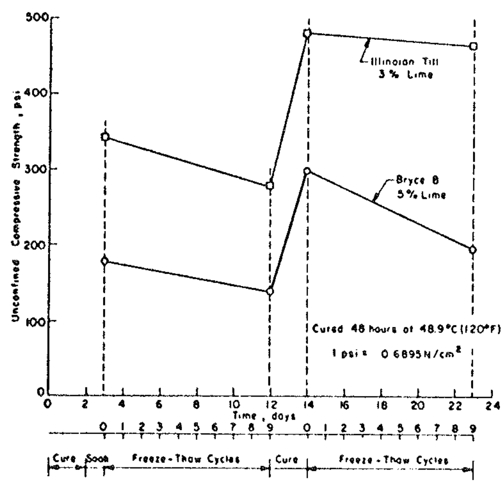
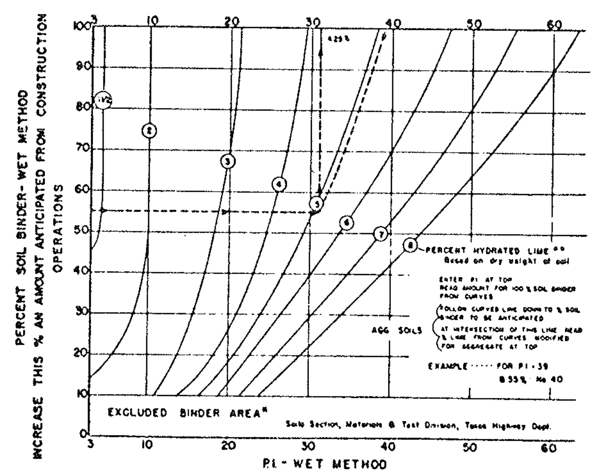


Figure 23. Recommended amounts of lime for stabilization of subgrades and bases (Ref 72).



* Exclude use of chart for materials with less than 10% - No 40 and cohesionless materials (PI less than 3)

** Percent of relatively pure lime usually 90% or more of Ca and/or Mg hydroxides and 85% or more of which pass the No 200 sieve. Percentages shown are for stabilizing subgrades and base courses where lasting effects are desired. Satisfactory temporary results are sometimes obtained by the use of as little as 1/2 of above percentages. Reference to cementing strength is implied when such terms as 'Lasting Effects' and 'Temporary Results' are used

Note: These percentages should be substantiated by approved testing methods on any particular soil material.

curing, and testing.

Mixture Preparation

Lime contents generally are specified as a percentage of the dry weight of soil, although a few agencies specify on a volume basis. Soil-lime mixtures are normally prepared first by dry mixing the proper amounts of soil and lime and then blending the required amount of water into the mixture. In most procedures, mixtures are prepared at or near optimum moisture content as determined by AASHTO T-99, T-180 or T-212.

Frequently the soil-lime mixture is allowed to mellow one hour or some other designated time prior to conducting Atterberg limit tests or preparing test specimens.

Specimen Preparation

Strength test specimens are generally cylindrically shaped. Diameter and heights vary substantially ranging from 35.6 mm (1.4 inches) in diameter by 71.1 mm (2.8 inches) high to 15.2 cm (6 inches) in diameter by 20.3 cm (8 inches) in height. Since the length to diameter ratios (ℓ/d ratios) vary, it is recommended that compressive strength values be corrected to an ℓ/d ratio of 2 for comparison purposes.

The density of the compacted specimens must be carefully controlled because the strength of a cured soil-lime mixture is greatly influenced by density (Fig 6) and small density variations may make it difficult to accurately evaluate the effect of other variables such as lime percentage and curing conditions. Thus, the compactive effort should always be specified since some test methods specify AASHTO T-99 or the equivalent and other procedures specify AASHTO T-180 or T-212.

Curing Conditions

Time, temperature, and moisture conditions during the curing period vary significantly. Some agencies cure at room temperatures while others cure at elevated temperatures, e.g., 48 hours at 48.9° C (120° F). Normally elevated temperature curing is of shorter duration than ambient curing. Many procedures specify that the specimens should be cured in a sealed container while others (AASHTO T-220) require a moist curing cycle followed by a drying and capillary wetting cycle. It should be noted that in some procedures no curing period is required.

The great disparity in curing conditions makes it very difficult to compare the results obtained from different testing methods. Thus, mixture quality criteria developed for a particular test procedure should not be arbitrarily adopted for analyzing test results obtained from a different test method.

Testing

Procedures used to evaluate soil-lime specimens usually involve conventional tests. For example, the Atterberg Limits (AASHTO T-89, T-90), California Bearing Ratio (AASHTO T-193), and R-Value (ASTM D2844) are used for many different types of materials. There is probably more variation in unconfined compression testing than any other procedure. Thus, details concerning specimen size, rate of loading, etc. should be specified in the description of any test method which is not standardized.

Mixture Design Criteria

Mixture design criteria are needed to evaluate the adequacy of a given soil-lime mixture. Criteria will vary depending on the stabilization objectives and anticipated field service conditions, i.e., environmental factors, wheel loading considerations, design life, etc. It is thus apparent that mixture design criteria may range over a broad scale and should be based on a careful consideration of the specific conditions associated with the stabilization project.

Types of Criteria

Current mixture design criteria can be classified into two broad categories. The first category relates to those situations where the major stabilization objectives are PI reduction, improved workability, immediate strength increase, and reduced swell potential. To a large extent, these property improvements are produced by the cation exchange and flocculation-agglomeration reactions which occur quite rapidly. Mixture design criteria for this category of stabilization might typically include some of the following requirements:

(1) no further decrease in PI with increased percentage of lime,
(2) acceptable PI reduction for the particular stabilization objective,
(3) acceptable swell potential reduction, and
(4) CBR and R-Value increase sufficient for anticipated uses.

It is difficult to establish actual quantitative values for the above requirements because in many cases they must be established relative to the properties of the untreated soil and the specific job conditions.

The second category of criteria is concerned with strength improvement produced by the pozzolanic reaction between the soil and lime. For example, if the mixture is to be used as a subbase or base course in the pavement structure, it must possess minimum strength and durability. Thus, mixture design criteria normally specify that the cured mixture meet a minimum strength requirement and the design lime content is that percentage which produces maximum strength for given curing conditions.

Most current minimum strength criteria are specified in terms of compressive strength. The minimum strength requirements generally are higher for base materials than for subbase materials since stress and durability conditions differ for various depths in the pavement structure.

Typical current mixture design criteria are presented in the section entitled Current Mixture Design Procedures.

Experience and Evaluation

Mixture design criteria can be validated only on the basis of actual field performance. McDowell's extensive publications (Refs 72 through 77) concerning Texas experiences, Anday's summary (Ref 3) of Virginia projects, and McDonald's recent reports (Refs 68 through 71) are examples of extensive validation activities for widely separated geographic areas with drastically different climatic conditions.

Mixture design criteria developed for use with a particular mixture design procedure and geographic location must not be applied indiscriminately to other areas. Careful consideration should be given to all aspects of the problem before adopting any criteria.

Current Mixture Design Procedures

Selected current mixture design procedures are summarized below. As discussed, mixture design procedures consider specimen preparation, curing conditions, testing procedures, and mixture design criteria. More detailed information concerning the mixture design procedures can be obtained by consulting the various references listed in this section.

California Procedure

California's current design procedure is based on stabilometer test data developed for mixtures containing various lime percentages. The general procedure is as follows:

1. Soil-lime mixtures are prepared at various lime percentages. The mixture moisture content is adjusted to approximately optimum (AASHTO T-180) and the moist mixture is loose cured for 24 hours.
2. Stabilometer samples are compacted using the California kneading compactor (California Test Method 301). The compacted specimens are not cured.
3. The compacted specimens are tested using the stabilometer (California Test Method No. 312) to determine the R-value.
4. Depending on the intended use of the mixture, the lime percentage required to develop an R-value in the range of 60 to 80 is determined.
5. The lime percentage is increased approximately 1 percent to compensate for field construction variability.

Eades and Grim Procedure

The pH mixture design concept developed by Eades and Grim (Ref 44) involves, to a certain extent, a strength based criterion. The basic thrust of the pH procedure is to add sufficient lime to the soil to insure a pH of 12.4 for sustaining the strength-producing, lime-soil pozzolanic reaction. The pH procedure, as developed by Eades and Grim, is summarized below.

1. Representative samples of air-dried, minus No. 40 soil to equal 20 g of oven-dried soil are weighed to the nearest 0.1 g and poured into 150-ml (or larger) plastic bottles with screw tops.
2. Since most soils will require between 2 and 5 percent lime, it is advisable to set up five bottles with lime percentages of 2, 3, 4, 5, and 6. This will insure, in most cases, that the percentage of lime required can be determined in 1 hour. Weigh the lime to the nearest 0.01 g and add it to the soil. Shake to mix soil and dry lime.
3. Add 100 ml of CO_2-free distilled water to the bottles.
4. Shake the soil-lime and water until there is no evidence of dry material on the bottom. Shake for a minimum of 30 seconds.
5. Shake the bottles for 30 seconds every 10 minutes.
6. After 1 hour, transfer part of the slurry to a plastic beaker and measure the pH. The pH meter must be equipped with a Hyalk electrode and standardized with a buffer solution having a pH of 12.00.
7. Record the pH for each of the soil-lime mixtures. If the pH readings go to 12.40, the lowest percent lime that gives a pH of 12.40 is the percent required to stabilize the soil. If the pH does not go beyond 12.30 but at least two consecutive percentages of lime give the same reading, the lowest percentage which gives a pH of 12.30 is that required to stabilize the soil. However if only the highest percentage checked gives a pH of 12.30, additional test bottles should be started with larger percentages of lime.

Thompson and Eades (Ref 108) have demonstrated that for typical Illinois soils, the lime percentage determined by the pH test was approximately the same as the lime percentage producing maximum compressive strength. Recent work by Harty (Ref 53), however, indicates that the lime percentage obtained from the pH procedure does not produce maximum cured compressive strength for tropical and subtropical soils. There are limitations to the pH procedure in that, (1) the technique does not establish whether the soil will react with lime to produce a substantial strength increase, and (2) strength data are not generated for use in evaluating mixture quality.

Eades and Grim (Ref 44) recognized the need for supplemental strength data and have stated, "The one-hour pH or 'Quick Test' can be used only to determine the lime requirements of a soil for stabilization. Since strength gains are related to the formation of calcium silicates, and their formation varies with the mineralogical components of the soil, a strength test is necessary to show the percentage of strength increase."

Illinois Procedure

The Illinois procedure considers two types of stabilization objectives:

1. Soil-lime stabilization in which the mixture will be utilized as a base or subbase material in the pavement system, and
2. Subgrade modification and expediting construction.

The procedures are outlined below.

Soil-lime Stabilization. The mixture design procedure is based on unconfined compressive strength test data. Specimens with a 5.1 centimeter (2 inch) diameter and a 10.2 centimeter (4 inch) height of the natural soil and soil-lime mixtures are prepared at optimum moisture content and maximum dry density (AASHTO T-99). The soil-lime specimens, prepared at various lime treatment levels, are cured for 48 hours at 48.9° C (120° F) prior to testing.

The compressive strength of the soil-lime mixture with 3 percent lime must be at least 34.5 N/cm^2 (50 psi) greater than the compressive strength of the natural soil. The design lime content is designated as the lime percent above which further increases do not produce significant additional strength. For field construction, the lime content is increased 0.5 to 1.0 percent to offset the effects of field variability. Minimum strength requirements are 69 N/cm^2 (100 psi) for subbase and 103 N/cm^2 (150 psi) for base course. These minimum strengths relate to AASHTO coefficients of relative strength of 0.12 for subbase materials and 0.11 for base course materials.

Subgrade Modification. The mixture design procedure for lime modification is based on the effect of lime on the plasticity index of the soil. Optional CBR testing can also be conducted if desired.

AASHTO Methods T-89 and T-90 are utilized to determine the liquid limit, plastic limit, and plasticity index of the soil treated with various percentages of lime. The lime-soil-water mixture is loose cured for one hour prior to testing. A plot of plasticity index versus lime content is prepared. The design lime content may be designated as (1) that lime content above which no further appreciable reduction in PI occurs, or (2) a minimum lime content which produces an acceptable PI reduction.

Depending on the stabilization objectives, CBR tests may also be conducted to evaluate the stability and/or swell properties of the lime-treated soil. Curing and soaking of the CBR specimens prior to testing is optional depending on the stabilization objectives. If appropriate, the design lime content may be changed based on the CBR data, stability values, or swell properties.

For field construction, the design lime content is increased 0.5 to 1.0 percent to offset the effects of field construction variability.

Louisiana Procedure

Lime contents for soil-lime mixtures to be used as base or subbase courses are determined in accordance with LDH Designation TR 433-70, "Determining the Minimum Lime Content for Lime-Soil Treatment." Quality requirements, expressed in terms of minimum unconfined compressive strength, are 69 N/cm^2 (100 psi) for base course and 34.5 N/cm^2 (50 psi) for subbase courses.

Soil-lime mixtures of various lime contents are prepared at optimum moisture content (LDH TR 418) and specimens 15.2 cm (6 in.) in diameter and 20.3 cm (8 in.) in height are compacted to maximum dry density (LDH TR 418).

The curing cycle for the compacted soil-lime mixture is:

1. seven days in moist room,
2. air dried 8 hours at 60° C (140° F),
3. eight hours of cooling, and
4. ten day capillary soaking at a confining pressure of 0.69 N/cm^2 (1 psi), AASHTO T-212 procedure.

Following curing, the specimens are tested in unconfined compression at a rate of 3.81 mm/minute (0.15 in./minute). The minimum lime content providing adequate unconfined compressive strength, i.e., 69 N/cm^2 (100 psi) for base, 34.5 N/cm^2 (50 psi) for subbase, is determined from the test data.

Oklahoma Procedure

Oklahoma's standard procedure for determining the optimum lime content is the Eades and Grim Procedure. As an alternate, a plasticity index reduction test procedure, as outlined below, is used. The basic objective of Oklahoma's lime treatment is to modify subgrade soils without any specific strengthening objective.

1. Soil-lime mixtures with lime contents of 3, 5, 7, and 10 percent are prepared at the AASHTO T-99 optimum moisture content for the soil.
2. The soil-lime mixtures are loose cured in a moisture room for 48 hours.
3. The cured soil-lime mixture is then dried in accordance with AASHTO T-87 paragraph 4(a).
4. The liquid limit, plastic limit, and plasticity index PI are determined in accordance with AASHTO Methods T-89 and T-90, respectively.
5. A plot of PI versus percent lime is prepared. The percent lime which reduces the PI by two points per one percent increase in lime is considered to be the optimum lime content for the soil-lime mixture. Any lime content at or below the optimum lime content which gives the desired modification may be recommended by the engineer. The PI should be reduced to a maximum value of 10.

South Dakota Procedure

Initial lime requirements are established based on a pH procedure (Test No. SD 128) which is similar to the Eades and Grim Procedure. Supplemental strength data are developed by evaluating the CBR of various soil-lime combinations compacted at optimum moisture content (AASHTO T-99) to maximum dry density.

The South Dakota technique (Test No. SD 107) is similar to AASHTO T-193. If the CBR of the soil-lime mixture with no curing except for the 96-hour soaking period is 3 to 4 times greater than the CBR of the natural soil, the soil-lime mixture is considered to be of adequate quality for use as a pavement layer (AASHTO coefficient of relative strength = 0.05).

Texas Procedure

The soil-lime mixture design procedure used by the State Department of Highways and Public Transportation is AASHTO T-220 which provides for the determination of the unconfined compressive strength of soil-lime mixtures. The procedure suggests strength criteria of 69 N/cm^2 (100 psi) for base construction and 34.5 N/cm^2 (50 psi) for subbase construction.

Details of the procedure are included in AASHTO T-220, however, a general outline of the procedure is presented below.

1. Based on the grain size and plasticity index data, the lime percentage is selected from Fig. 23.
2. Optimum moisture and maximum dry density of the mixture are determined in accordance with appropriate sections of AASHTO T-212 or Tex-121-E. The compactive effort is 50 blows of a 4.54 kg (10 lb) hammer, with a 45.7 cm (18 inch) drop.
3. Test specimens, 15.2 cm (6 inches) in diameter and 20.3 (8 inches) in height are compacted at optimum moisture content to maximum dry density.
4. The specimens are placed in a triaxial cell (AASHTO T-212 or Tex-121-E) and cured in the following manner:

   a. seven days at room temperature,
   b. remove cells and dry at a temperature not to exceed 60° C (140° F) for about six hours or until one-third to one-half of the molding moisture has been removed,
   c. cool the specimens for at least 8 hours, and
   d. subject the specimens to capillarity (section 6 of AASHTO T-212 or Tex-121-E) for 10 days.

5. The cured specimens are tested in unconfined compression in accordance with sections 7 and 8 of AASHTO T-212 or Tex-121-E.

Relative to Fig 23, the Texas Procedure notes that the percentages should be substantiated by approved testing methods on any particular soil material. The results of the unconfined compression strength testing can be used for the purpose of substantiation.

Thompson Procedure

Thompson (Ref 104) has developed a mixture design process for lime-treated soils in which different procedures are proposed for lime modified soils and soil-lime mixtures.

The lime modification procedure is utilized when the stabilization objectives are to expedite construction and produce subgrade modification, e.g., CBR increase, decreased swell potential, and decreased plasticity. Soil-lime mixtures which display significant compressive strength increases, 34.5 N/cm^2 (50 psi) minimum, can be utilized as base and subbase materials depending on the soil-lime mixture properties and pavement service requirements.

A flow diagram illustrating the mixture design process is shown in Fig 24. Quality criteria for the soil-lime mixtures were established based on considerations of pavement structural behavior and durability requirements. The soil-lime quality criteria are summarized in Table 4.

The development of the mixture design process and the detailed testing procedures are contained in Ref 104. It is emphasized that the lime-modified soil mixture design process can be utilized for reactive soils if the stabilization objectives are primarily to expedite construction or modify the subgrade.

Virginia Procedure

Virginia's mixture design procedure, VTM-11 Virginia Test Method for Lime Stabilization, is based on the cured compressive strength of soil-lime mixtures stabilized with various amounts of lime. The procedure is summarized below.

1. Proctor sized specimens at various lime percentages are prepared at approximately optimum moisture content and maximum dry density (AASHTO T-99), compaction test conducted with 6 percent lime.
2. Specimens are cured in sealed containers at high humidity for 72 hours at 48.9° C (120° F).
3. The soil-lime specimens are tested in unconfined compression using a loading rate of 1089 kg/minute (2,400 pounds/minute) or approximately 1.3 kg/cm^2/minute (19 psi/minute).
4. Virginia criteria require a minimum compressive strength of 10.5 kg/cm^2 (150 psi) for soil-lime mixtures tested in accordance with the above procedure.

Summary

Design lime contents generally are based on an analysis of the effect of varying lime percentages on selected engineering properties of the soil-lime mixture. The basic components of a mixture design procedure generally are:

1. method for preparing the soil-lime mixture,
2. procedure for preparing and curing specimens,
3. testing procedures for evaluating a selected property or properties of the cured soil-lime mixture, and
4. appropriate criteria for establishing the design lime content.

It is important to note that different design lime contents for the same soil may be established depending on the objectives of the lime treatment and the mixture design procedure utilized. Mixture design procedures should be flexible enough to allow the exercise of judgment when unusual stabilization objectives are contemplated.

LIME STABILIZATION CONSTRUCTION

Introduction

The modern version of lime stabilization is less than 30 years old, but considerable advancement has been made in construction procedures during these past three decades. This progress of this method of stabilization was due to the efforts of many engineers and scientists and can be summarized as follows:

1. basic and applied research by numerous state highway departments, governmental agencies, and universities;
2. education by worldwide publication of research studies and construction reports of actual lime stabilization projects; and
3. equipment manufacturer's recognition of the potential of lime stabilization and development of equipment to meet the needs of the contractor.

Twenty-five years ago there were two basic stabilizers that could be adapted to lime stabilization, but today there are approximately twenty different types of equipment designed primarily for this stabilization.

With the growth of lime stabilization throughout the world for many climatic conditions, a diversity of applications has developed and a variety of construction techniques has evolved. This variation has been due to such factors as type of soil, degree of stabilization required, complexity of project, ecological restraints, and type of pavement design. For example, some heavy clay soils are very reactive to lime and can be completely pulverized with only one pass of a traveling mixing unit; however, this is the exception to the rule, as the stabilization of a heavy gumbo clay usually requires much more manipulation and curing than a low-plastic granular material. Modification of soil, e.g., drying out of wet soil with lime to expedite construction, is less involved than completely stabilizing a heavy clay to be used as a part of the pavement structure.

Projects may range from maintenance activities for which only a few bags of lime are required (Fig 29) to vast interstate highways or airfield pavements requiring thousands of tons of bulk lime. Because of dusting, projects located in urban areas generally require the use of lime slurry rather than dry lime. Thus, with the growing emphasis on ecology the trend is toward a greater use of a lime slurry and some engineers are now discouraging the use of dry lime except in very localized areas.

Finally, when lime is used in pavement design to reduce overall thickness, the stabilized layer must be built under tight construction specifications, whereas requirements are more lenient when lime is merely used to form a working table.

Regardless of the specific application of soil-lime, the following basic steps are involved in the construction procedure: soil preparation, lime spreading, mixing and watering, compaction and finishing, and curing. These basic steps, along with the more significant variations, are discussed in detail. Since this is a state-of-the-art report, undoubtedly there will be more variations as lime stabilization continues to expand throughout the world.

Lime Stabilization Methods

Basically, there are three recognized lime stabilization methods, including in-place mixing, plant mixing, and pressure injection.

In-place Mixing

In-place mixing may be subdivided into three methods:

1. mixing lime with the existing materials already a part of the construction site or pavement (Fig 25),
2. off-site mixing in which lime is mixed with borrow and the mixture is then transported to the construction site for final manipulation and compaction (Fig 26), and
3. mixing in which the borrow source soil is hauled to the construction site and processed as in method number 1.

The following procedures are utilized for in-place mixing:

1. One increment of lime is added to clays or granular base materials that are easy to pulverize. The material is mixed and compacted in one operation, and no mellowing period is required.
2. One increment of lime is added and the mixture is allowed to mellow for a period of 1 to 7 days to assist in breaking down heavy clay soils.
3. One increment of lime is added for soil modification and pulverization prior to treatment

Table 4. Tentative soil-lime mixture compressive strength requirements (Ref 104).

Anticipated Use	Residual Strength Requirement,[1] N/cm² (psi)	Strength Requirements for Various Anticipated Service Conditions[2]: Extended Soaking, 8 days, N/cm² (psi)	Cyclic Freeze-Thaw[5]: 3 Cycles, N/cm² (psi)	7 Cycles, N/cm² (psi)	10 Cycles, N/cm² (psi)
Modified subgrade	13.8 (20)	34.5 (50)	34.5 (50)	62.1 (90) 34.5 (50)	82.7 (120)
Subbase					
Rigid pavement	13.8 (20)	34.5 (50)	34.5 (50)	62.1 (90) 34.5 (50)*	82.7 (120)
Flexible pavement					
Thickness of cover[3]					
25.4 cm (10 in.)	20.7 (30)	41.4 (60)	41.4 (60)	68.9 (100) 41.4 (60)*	89.6 (130)
20.3 cm (8 in.)	27.6 (40)	48.3 (70)	48.3 (70)	75.8 (100) 51.7 (75)*	96.5 (140)
12.7 cm (5 in.)	41.4 (60)	62.1 (90)	62.1 (90)	89.6 (130) 68.9 (100)*	110 (160)
Base	68.9 (100)[4]	89.6 (130)	89.6 (130)	117 (170) 103 (150)*	138 (200)

[1] Minimum anticipated strength following first winter exposure.

[2] Strength required at termination of field curing (following construction) to provide adequate residual strength.

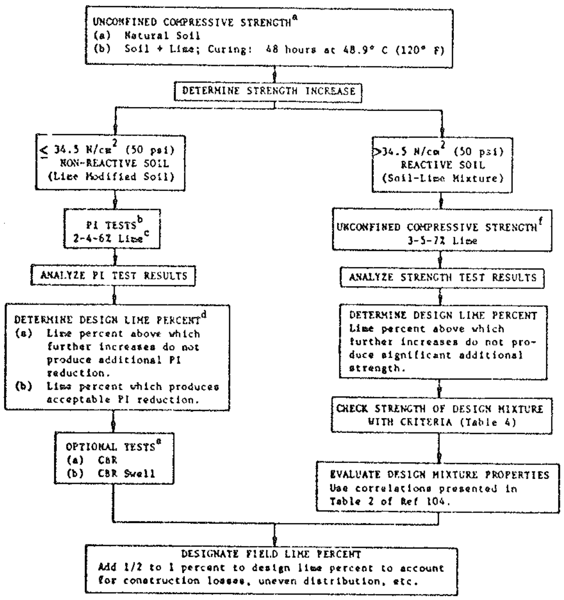
[3] Total pavement thickness overlying the subbase. The requirements are based on the Boussinesq stress distribution. Rigid pavement requirements apply if cemented materials are used as base courses.

[4] Flexural strength should be considered in thickness design.

[5] Number of freeze-thaw cycles expected in the soil-lime layer during the first winter of service.

*Freeze-thaw strength losses based on 6.9 N/cm²/cycle (10 psi/cycle) except for 7-cycle values indicated by an * which were based on a previously established regression equation.

Figure 24. Proposed mixture design process for lime-treated soils (Ref 104).



[a] All specimens compacted at optimum water content to maximum dry density. Lime treatment level for b may be 5 percent or as determined by "pH procedure." See footnote f if desired.

[b] PI Tests conducted one hour after mixing lime-soil-water. Mixture is not cured prior to testing.

[c] In some cases more closely spaced treatment levels may be appropriate.

[d] Criteria a or b may be applied depending on the stabilization objective.

[e] Conduct tests at design lime content. Curing of CBR specimens prior to soaking is optional depending on stabilization objective. If swell is not reduced to a satisfactory level, additional CBR tests may be conducted at higher lime contents. Design lime content may be increased if further swell reduction is obtained. Swell considerations are of great importance for lime modified subgrades.

[f] Specimens compacted at optimum moisture content to maximum dry density. Additional and/or different lime percentages may be required for some soils. An estimate of approximate optimum lime content may be obtained by applying "pH test procedure" developed by Eades and Grim (Ref 44).

Figure 25. In-place mixing of lime-stabilized subgrade—Oregon.

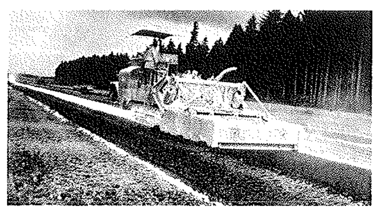


Figure 26. Off-site mixing pads for Mississippi River levee repair project—Arkansas.

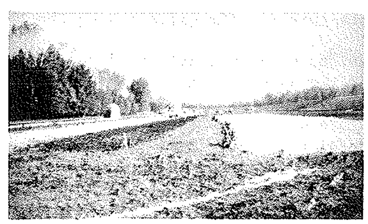


Figure 27a. Deep stabilization of access road—Dallas-Fort Worth Airport. After lime spreading the plow cuts to a depth of 61 cm (24 inches).

Figure 27b. Tractor plow pulled by a second tractor for deep stabilization.

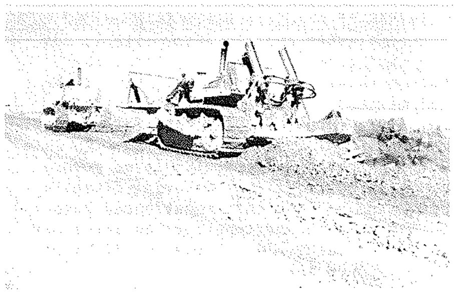


Figure 27c. Root plow for scarifying to a depth of 46 cm (18 inches).

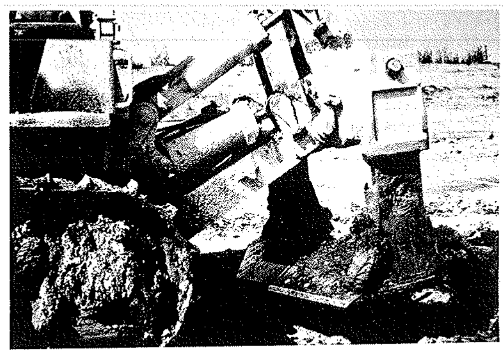


Figure 28. Plant mixing pug mills.

(a) Lime-treated gravel with lime fed by screw conveyor—South Dakota (Ref 21).

(b) Lime-cement-flyash-aggregate base course—Newark Airport.

Figure 29. Application of lime by the bag for a small maintenance project-Texas.

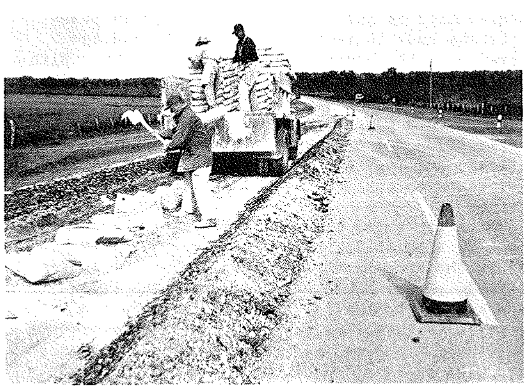
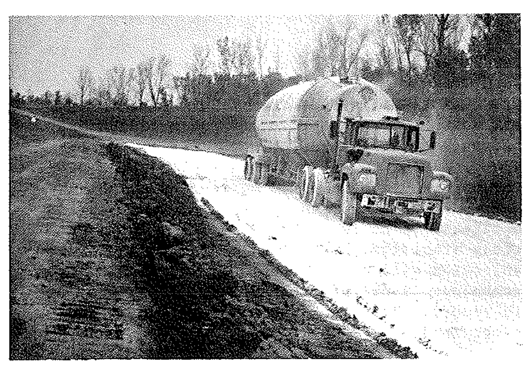


Figure 30. Application of lime by a bulk pneumatic truck for levee repair project.



with cement or asphalt.

4. One increment of lime is added to produce a working table. Proof rolling is required in lieu of pulverization and density requirements.

5. Two increments of lime are added for soils which are extremely difficult to pulverize. Between the applications of the first and second increments of lime, the mixture is allowed to mellow.

6. Deep stabilization which has been accomplished by one of two approaches (Ref 98).

a. One increment of lime is applied to modify soil to a depth of 24 inches (Fig. 27). Greater depths are possible but to date have not been attempted. A second increment of lime is added to the top 15 to 30 cm (6 to 12 in.) for complete stabilization. Plows and rippers are used to break down the large clay chunks in the deep treatment. Heavy disc harrows and blades are also used in pulverization of these clay soils. In frost zones, the use of small quantities of lime for soil modification under some circumstances may result in a frost susceptible material which in turn can produce a weak sublayer.

b. One increment of lime is applied for complete stabilization to a depth of 46 cm (18 in.). Mechanical mixers are now available to pulverize the lime-clay soil to the full depth by progressive cuts as follows: first pass cut to a depth of 15 cm (6 in.), second to 23 cm (9 in.), third to 30 cm (12 in.), fourth to 38 cm (15 in.), and then a few passes to a depth of 46 cm (18 in.) to accomplish full pulverization. The full 46 cm (18 in.) is compacted from the top by vibratory and conventional heavy rollers.

Plant Mixing

The plant-mix operation usually involves hauling the soil to a central plant where lime, soil, and water are uniformly mixed and then transported to the construction site for further manipulation (Fig 28).

The amount of lime for either method is usually predetermined by test procedures. Specifications may be written to specify the actual strength gain required to upgrade the stabilized soil, and notations can be made on the plans as to the estimated percent of lime required. This note should also stipulate that changes in lime content may be necessary to meet changing soil conditions encountered during construction.

Pressure Injection

Pressure injections of lime slurry to depths of 2 to 3 meters (7 to 10 feet) for control of swelling and unstable soils on highways and under building sites are usually placed on 1.5-meter (5-foot) spacings, and attempts are made to place horizontal seams of lime slurry at 20- to 30-cm (8- to 12-in.) intervals. The top 15- to 30-cm (6- to 12-in.) layer should be completely stabilized by conventional methods. Pressure injection will not be considered in detail since the technology is different and is outside the scope of this report. Additional information and detail can be obtained from Refs 58 and 113.

Construction Steps

Soil Preparation

The in-place subgrade soil should be brought to final grade and alignment. The finished grade elevation may require some adjustment due to the potential fluff action of the lime stabilized layer resulting from the fact that some soils tend to increase in volume when mixed with lime and water. This volume change may be exaggerated when the soil-lime is remixed over a long period of time, especially at moisture contents less than optimum moisture. The fluff action is usually minimized if adequate water is provided and mixing is accomplished shortly after lime is added. For soils that tend to fluff with lime, the subgrade elevation should be lowered slightly or the excess material trimmed. Trimming can usually be accomplished by blading the material onto the shoulder of embankment slopes.

This blading operation is desirable to remove the top 6.4 mm (¼ inch) since this material often is not well cemented due to lime loss experienced during construction. Excess rain and construction water may wash lime from the surface, and carbonation of lime may occur in the exposed surface.

If dry lime is used, ripping or scarifying to the desired depth of stabilization can be accomplished either before or after lime is added. If the lime is to be applied in a slurry form, it is desirable to scarify prior to the addition of lime.

Lime Application

Dry Hydrated Lime. Dry lime can be applied either in bulk or by bag. The use of bagged lime is generally the simplest but also the most costly method of lime application. Bags, 22.7 kg (50 lb), are delivered in dump or flatbed trucks and placed by hand to give the required distribution (Fig 29). After the bags are placed, they are slit and the lime is dumped into piles or transverse windrows across the roadway. The lime is then leveled by either hand raking or by means of a spike-tooth harrow or drag pulled by a tractor or truck. Immediately thereafter, the lime is sprinkled to reduce dusting.

The major disadvantages of the bag method are the higher cost of lime because of bagging costs, greater labor costs, and slower operations. Nevertheless, bagged lime is often the most practical method for small projects or for projects in which it is difficult to utilize large equipment.

For large stabilization projects, particularly where dusting is no problem, the use of bulk lime has become common practice. Lime is delivered to the job in self-unloading transport trucks (Fig 30). These trucks are large and efficient, capable of hauling 13,300 to 21,800 kg (15 to 24 tons). One type is equipped with one or more integral screw conveyors which discharge at the rear. In recent years pneumatic trucks have increased in popularity and are preferred over the older auger-type transports. With the pneumatic units the lime is blown from the tanker compartments through a pipe or hose to a cyclone spreader or to a pipe spreader bar mounted at the rear (Fig 31). Bottom-dump hopper trucks have also been tried, but they are undesirable because of difficulty in unloading and obtaining a uniform rate of discharge.

With the auger trucks, spreading is handled by means of a portable, mechanical-type spreader attached to the rear or through metal downspout chutes or flexible rubber boots extending from the screw conveyors. The mechanical spreaders incorporate belt, screw, rotary vane, or drag chain conveyors to distribute the lime uniformly across the spreader width. When boots or spouts are used instead, the lime is deposited in windrows; but due to lime's lightness and flowability, the lime becomes distributed rather uniformly across the spreading lane. Whether mechanical spreaders, downspouts, or boots are used, the rate of lime application can be regulated by varying the spreader opening, spreader drive speed, or truck speed so that the required amount of lime can be applied in one or more passes.

With the pneumatic trucks, spreading is generally handled with a cyclone spreader mounted at the rear, which distributes the lime through a split chute or with a spreader bar equipped with several large downspout pipes. Fingertip controls in the truck cab permit the driver to vary the spreading width by adjusting the air pressure. Experienced drivers can adjust the pressure and truck speed so that accurate distribution can be obtained in one or two passes.

When bulk lime is delivered by rail, a variety of conveyors can be used for transferring the lime to transport trucks; these include screw, belt or drag-chain conveyors, bucket elevators, and screw elevators. The screw-type conveyors are most commonly used, with large diameter units, 25.4 to 30.5 cm (10 to 12 inches) being recommended for high speed unloading. To minimize dusting, all types of conveyors should be enclosed. Rail car unloading is generally facilitated by means of poles and either mechanical or air-type vibrators.

Lime has also been handled through permanent or portable batching plants, in which case the lime is weigh-batched prior to loading. Generally, a batching plant set-up would only be practical on exceptionally large jobs.

Obviously, the self-unloading tank truck is the least costly method of spreading lime, since there is no rehandling of material and large payloads can be carried and spread quickly.

Dry Quicklime. Quicklime may be applied in bags or bulk. Due to its higher cost, bagged lime is only used for drying of isolated wet spots or on small jobs. The distribution of bagged quicklime is similar to that of bagged hydrate, except that greater safety emphasis is needed. First the bags are spaced accurately on the area to be stabilized, and after spreading, water is applied and mixing operations started at once. The fast watering and mixing operation helps minimize the danger of burns. Quicklime may be applied in form of pebble, approximately 9.5 mm (3/8 inch), granular, or pulverized. The first two are more desirable as less dust is generated during spreading.

Bulk quicklime may be spread by self unloading auger or pneumatic transport trucks, similar to those used for dry hydrate. In addition, however, due to its coarser size and higher density, quicklime may also be tailgated from a regular dump truck with tailgate opening controls to assure accurate distribution (Fig. 32).

Because quicklime is anhydrous and generates heat upon contact with water, special care should be taken during stabilization to avoid lime burns. Where quicklime is specified, the contractor should provide the engineer, for review, a detailed safety program covering precautions to be exercised and emergency treatment to be available on the jobsite. The program should include protective equipment for eyes, mouth, nose, and skin as well as a first-aid kit with an eyeball wash. This protective equipment should be available on the jobsite during spreading and mixing operations. The contractor should actively enforce this program for the protection of the workers and others in the construction area.

Slurry Method. In this method lime and water are mixed into a slurry. Historically, hydrated lime has been used in slurries. Nevertheless, quicklime could potentially be used providing that adequate equipment was developed for preparing the slurry. At the present time, this process involves a two-step operation in which a quicklime paste is first prepared and then additional water added to form the slurry. The hydrated lime-water slurry is mixed either in a central mixing tank (Fig 33), jet mixer (Fig 34), or in a tank truck. The slurry is spread over the scarified roadbed by a tank truck equipped with spray bars (Fig 35). One or more passes may be required over a measured area to achieve the specified percentage based on lime solids content. To prevent run-off and consequent non-uniformity of lime distribution that may occur under certain conditions, it may be necessary to mix the slurry and soil immediately after each spreading pass (Fig 36).

A typical slurry mix proportion is 907 kg (1 ton) of lime and 1.9 m^3 (500 gallons) of water which yields about 2.3 m^3 (600 gallons) of slurry containing 31 percent lime solids. At higher concentrations there is difficulty in pumping and spraying the slurry. Forty percent solids is a maximum pumpable slurry.

The actual proportion used depends upon the percent of lime specified, type of soil, and its moisture condition. Where small lime percentages are required, the slurry proportions may be reduced to 907 kg (1 ton) of lime per 2.6 to 3.0 m^3 (700 to 800 gallons) of water. Where the soil moisture content is near optimum, a stronger lime concentration normally would be required.

In plants employing central mixing, agitation is generally accomplished by using compressed air and a recirculating pump, although pug mills have also been used. The most typical slurry plant incorporates slurry tanks large enough to handle whole tank truck loads of hydrated lime, approximately 18,100 kg (20 tons). For example, on one job two 57-cubic meter (15,000 gallon) tanks, 3-meter diameter by 8-meter length (10-foot diameter by 26-foot length) were used, each fitted with a 20-centimeter (8-inch) perforated air line mounted along the bottom. The air line was stopped 46 cm (18 inches) short of the end wall, thereby providing maximum agitation in the lime feeding zone. A typical batch consisted of 38 m^3 (10,000 gallons) of water, charged first, and 18,100 kg (20 tons) of lime, producing about 45 m^3 (12,000 gallons) of slurry in less than 25 minutes. Loading of the tank trucks was handled by a standard water pump, with one slurry tank being unloaded while the slurry was being mixed in the other tank.

On another job the contractor used a similar tank and air line, but in addition made use of a 10-cm (4-inch) recirculating pump for mixing; the same pump loaded the tank trucks. To keep the lime from settling, the contractor devised a hand-operated scraper fitted with air jets.

Still another job involved a much smaller tank, approximately 9 m^3 (2400 gallons), mounted below a lime bin and weigh-batcher. A typical batch consisted of 1800 kg (2 tons) of lime and 4 m^3 (1000 gallons) of water, producing enough slurry for one tank truck. Mixing was accomplished with air jets and a 7.6-centimeter (3-inch) recirculating pump.

The newest and most efficient method of slurry production which eliminates batching tanks involves the use of a compact jet slurry mixer. Water at 5 kg/cm^2 (70 psi) pressure and hydrated lime are charged continuously in a 65:35 (weight) ratio into the jet mixing bowl, where slurry is produced instantaneously. The mixer and auxiliary equipment can be mounted on a small trailer and transported to the job readily, giving great flexibility to the operation (Ref 16).

In the third type of slurry set-up, measured amounts of water and lime are charged separately to the tank truck, with the slurry being mixed in the tank either by compressed air or by a recirculating pump mounted at the rear. The water is metered and the lime proportioned volumetrically or by means of weight batchers. Both portable and permanent batching plants are used. Mixing with air is accomplished at the plant. The air jets are turned on during the

Figure 31. Distribution of lime from bar spreader-Wisconsin.

Figure 32. Spreading of granular quicklime on canal relining project—California (Ref 18).

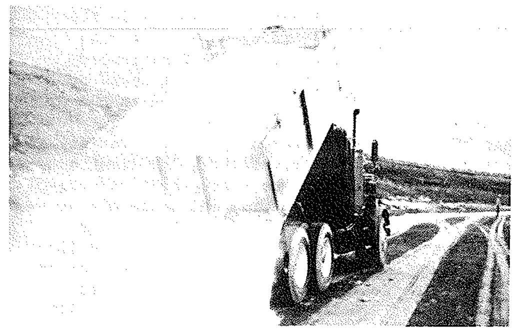


Figure 33. Slurry mixing plant using recirculating pump for mixing.

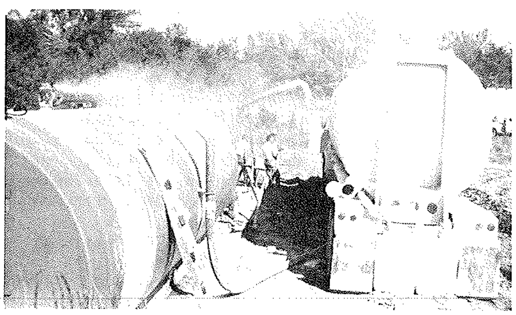


Figure 34. Jet slurry mixing plant-Dallas County, Texas (Ref 16).

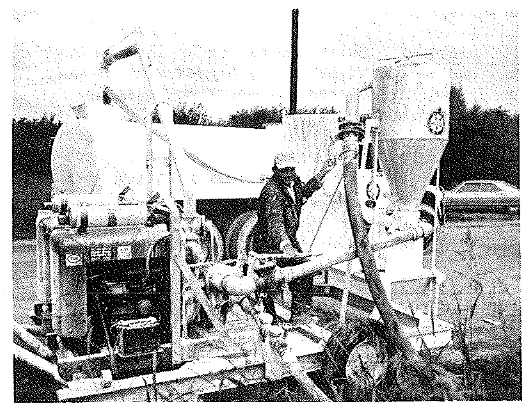


Figure 35. Spreading of lime slurry.

(a) Stabilization of stone base (Ref 16).

(b) Recirculation pump on top of 2.3 m^3 (6000 gal) wagon agitates slurry.

Figure 36. Grader-scarifier cutting slurry into stone base.

loading operation, and remain on until the slurry is thoroughly mixed which takes about 10 to 15 minutes. The use of a recirculating pump, however, permits mixing to occur during transit to the job. Generally, 5, 8 or 10-centimeter(2, 3, or 4-inch) pumps are used in this operation, with the slurry being recirculated through the tank by means of a perforated longitudinal pipe extending the length of the tank and capped at one end.

Spreading from the slurry distributors is effected by gravity or by pressure spray bars, the latter being preferred due to better distribution. The use of spray deflectors is also recommended for good distribution. The general practice in spreading is to make either one or two passes per load. However, several loads may be needed in order to distribute the required amount of lime. The total number of passes will depend on the lime requirement, optimum moisture of the soil, and type of mixing employed. Windrow mixing with the grader generally requires several passes.

Double Application of Lime. In some areas where extremely plastic, gumbo clay (PI of 50+) abounds, it may prove advantageous to add the requisite amount of lime in two increments to facilitate adequate pulverization and obtain complete stabilization. For example, 2 or 3 percent lime is added first, partially mixed, then the layer is sealed and allowed to cure for up to a week. The remaining lime is then added preparatory to final mixing. The first application mellows the clay and helps in achieving final pulverization and the second application completes the lime-treatment process.

Advantages and Disadvantages of Different Types of Application

Listed below are some of the advantages and disadvantages of the various lime application procedures.

1. Dry Hydrated Lime. Advantages: (a) dry lime can be applied two or three times faster than a slurry; and (b) dry lime is very effective in drying out soils. Disadvantages: (a) dry lime produces a dusting problem which makes its use undesirable in urban areas; and (b) the fast drying action of the dry lime requires an excess amount of water during the dry, hot seasons.

2. Quicklime. Advantages: (a) more economical as it contains approximately 25 percent more available lime; (b) greater bulk density so storage silos can be smaller in size; (c) faster drying action in wet soils; (d) faster reaction with soils; and (e) due to faster drying, construction season can be extended, both spring and fall. Disadvantages: (a) field hydration less effective than commercial hydrators, producing a coarser material with poorer distribution in soil mass; (b) quicklime requires more water than hydrate for stabilization, which may present a problem in dry areas; and (c) greater susceptibility to skin and eye burns.

3. Slurry Lime. Advantages: (a) dust-free application is more desirable from an environmental standpoint; (b) better distribution is achieved with the slurry; (c) in the lime slurry method, the lime spreading and sprinkling operations are combined, thus reducing job costs; and (d) during summer months slurry application pre-wets the soil and minimizes drying action. Disadvantages: (a) application rates are slower; high capacity pumps are required to achieve acceptable application rates; (b) extra equipment is required and thus costs are higher; (c) extra manipulation may be required for drying purposes during cool, wet, humid weather, which could occur during the fall, winter, and spring construction season; and (d) not practical for use with very wet soils.

Pulverization and Mixing

To obtain satisfactory soil-lime mixtures adequate pulverization and mixing must be achieved. For heavy clay soils two stage pulverization and mixing may be required while for other soils one stage mixing and pulverization may be satisfactory. This difference is due primarily to the fact that the heavy clays are more difficult to break down.

Two-stage Mixing. Construction steps in two-stage mixing consist of preliminary mixing, moist curing for 24 to 48 hours (or more) and final mixing or remixing. The first mixing step distributes the lime throughout the soil, thereby facilitating the mellowing action. For maximum chemical action during the mellowing period, the clay clods should be less than 5 cm (2 inches) in diameter. Prior to mellowing the soil should be sprinkled liberally to bring it up to at least two percentage points above optimum moisture in order to aid the disintegration of clay clods. The exception to excess watering would be in cool, damp weather when evaporation is at a minimum. In hot weather, however, it may be difficult to add too much water.

After preliminary mixing, the roadway should be sealed lightly with a pneumatic roller as a precaution against heavy rain, since the compacted subgrade will shed water, thereby preventing moisture increases which might delay construction. Generally, in 24 to 48 hours the clay becomes friable enough so that desired pulverization can be easily attained during final mixing. Additional sprinkling may be necessary during final mixing to bring the soil to optimum moisture or slightly above (Fig 37). In hot weather more than optimum moisture is needed to compensate for the loss through evaporation.

Although disc harrows (Fig 38) and grader scarifiers are suitable for preliminary mixing, high-speed rotary mixers (Fig 39) or one-pass travel plant mixers (Fig 25) are required for final mixing. Motor graders are generally unsatisfactory for mixing lime with heavy clays.

One-stage Mixing. Both blade and rotary mixing or a combination have been used successfully in projects involving granular base materials. However, rotary mixers are preferred for more uniform mixing, finer pulverization, and faster operation. They are generally required for highly plastic soils which do not pulverize readily and for reconstructing worn-out roads in order to pulverize the old asphalt.

Blade Mixing. When blade mixing is used in conjunction with dry lime, the material is generally bladed into two windrows, one on each side of the roadway. Lime is then spread on the inside of each windrow or down the center line of the road. The soil is then bladed to cover the lime. After the lime is covered, the soil is mixed dry by blading across the roadway. After dry mixing is completed, water is added to slightly above the optimum moisture content and additional mixing is performed. To assure thorough mixing by this method, the material should be handled on the mold board at least three times.

When blade mixing is used with the slurry method, the mixing is done in thin lifts which are bladed to windrows. One practice is to start with the material in a center windrow, then blade aside a thin layer after the addition of each increment of slurry, thereby forming side windrows. The windrowed material is then bladed back across the roadway and compacted, provided that its moisture content is at optimum.

A second practice is to start with a side windrow, then blade a thin, 5-centimeter (2-inch) layer across the roadway, add an increment of lime, then blade this layer to a windrow on the opposite side of the road. On one job this procedure was repeated several times until all the material was mixed and bladed to the new windrow. Since by this time only half of the lime had been added, the process was repeated, moving the material back to the other side. This procedure is admittedly slow, but it provides excellent uniformity.

Rotary Mixing. When high-speed rotary mixers or one-pass travel plant mixers are used, the lime is generally spread evenly on the entire roadway, and mixing starts from the top down. Depending upon the type of equipment used and the soil involved, complete mixing can normally be accomplished in one to three passes. If needed, water is added during mixing to obtain the desired moisture content, generally optimum. The water may be added by sprinkling trucks or by spraying into the mixing chamber of the mixer. The latter method has considerable merit, since the intimate contact of lime, water, and soil facilitates chemical breakdown and pulverization.

The traveling windrow-mixing type machine, commonly referred to as the soil-through-machine type, may also be used for one-stage mixing if adequate pulverization and mixing can be achieved in one pass.

Central Mixing. Pre-mixing of lime with granular base materials is becoming popular on new construction projects, particularly where submarginal gravels are utilized. Since the gravel has to be processed anyway to meet gradation specifications, it is a relatively simple matter for the contractor to install a lime bin, feeder, and pug mill at the screening plant. On one project a small pug mill was installed at the head pulley of the collecting belt conveyor (Fig 28a) and at another operation a larger pug mill plant was utilized (Fig 28b). The general practice is to add the optimum moisture at the pug mill, thereby permitting immediate compaction after laydown.

Pulverization and Mixing Requirements

Pulverization and mixing requirements are generally specified in terms of percentages passing the 1-1/2-inch or 1-inch screen and the No. 4 sieve. Typical requirements are 100 percent passing the 1-inch and 60 percent passing the No. 4, exclusive of nonslaking fractions. However, in some applications the requirements are relaxed. For example, the South Dakota Department of Transportation only requires 100 percent passing the 1-1/2-inch screen with no requirement for the No. 4 sieve. Other specifications may only require 40 to 50 percent passing the No. 4 sieve.

In certain expedient construction operations formal requirements are eliminated, and the "pulverization and mixing to the satisfaction of the engineer" type clause is employed.

Compaction

For maximum development of strength and durability, lime-soil mixtures should be properly compacted. Many agencies require at least 95 percent of AASHTO T-99 density for subbases and 98 percent for bases. Some agencies have required 95 percent AASHTO T-180 maximum density. Although such densities can be achieved for more granular soil-lime mixtures, it is difficult to achieve this degree of compaction for lime-treated fine-grained soils.

If a thick soil-lime lift is to be compacted in one lift, many specifications require 95 percent of AASHTO T-99 maximum density in the upper 15 to 23 centimeters (6 to 9 inches) and lower densities are accepted in the bottom portion of the lift. To achieve high densities necessitates compacting at approximately optimum moisture content with approved compactors. Granular soil-lime mixtures are generally compacted as soon as possible after mixing, although delays of up to two days are not detrimental, especially if the soil is not allowed to dry out and lime is not allowed to carbonate. Fine-grained soils can also be compacted soon after final mixing, although delays of up to four days are not detrimental. When longer delays, e.g., two weeks or more, cannot be avoided, it may be necessary to incorporate a small amount of additional lime into the mixture, e.g., 1/2 percent, to compensate for losses due to carbonation and erosion.

Various rollers and layer thicknesses have been used in lime stabilization. The most common practice is to compact in one lift, using the sheepsfoot roller (Fig 40) until it "walks out," followed by a multiple-wheel pneumatic roller (Fig 41). In some cases, a flat wheel roller is used in finishing. Single lift compaction can also be accomplished with vibrating impact rollers (Fig 42) or heavy pneumatic rollers, with light pneumatic or steel rollers being used for finishing. When light pneumatic rollers are used alone, compaction is generally done in thin lifts, usually less than 15 centimeters (6 inches). Slush rolling of granular soil-lime mixtures with steel rollers is not recommended.

During compaction light sprinkling may be required, particularly during hot, dry weather, to compensate for evaporation losses.

Curing

Maximum development of strength and durability also depends on proper curing. Favorable temperature and moisture conditions and the passage of time are required for curing. Temperatures higher than 4.4 to 10° C (40 to 50° F) and moisture contents around optimum are conducive to curing. Although some specifications require a 3 to 7-day undisturbed curing period, other agencies permit the immediate placement of overlying paving layers if the compacted soil-lime layer is not rutted or distorted by the equipment. This overlying course maintains the moisture content of the compacted layer and is an adequate medium for curing.

Two types of curing can be employed, moist and asphaltic membrane. In the first, the surface is kept damp by sprinkling (Fig. 43) with light rollers being used to keep the surface knitted together. In membrane curing, the stabilized soil is either sealed with one shot of cutback asphalt at a rate of about .45 to 1.1 liters/sq m (0.10 to 0.25 gal/sq yd) within one day after rolling or primed with increments of asphalt emulsion applied several times during the curing period. A common practice is to apply two shots the first day, and one each day thereafter for four days, at a total rate of .45 to 1.1 liters/sq m (0.10 to 0.25 0.25 gal/sq yd). The type of membrane used, amount, and number of shots vary considerably. Generally, it is difficult to apply more than 0.76 liters (0.2 gal) of asphalt prime because the lime stabilized layer is relatively impervious after compaction.

Measurement and Payment

Measurement and payment considerations in the contract documents are typically incorporated in the manner illustrated below. Particular attention should be given to the water item since abnormally large quantities are used in soil-lime construction operation.

Figure 37. Watering of lime-treated clay on airport project—Kansas City, Missouri (Ref 9).

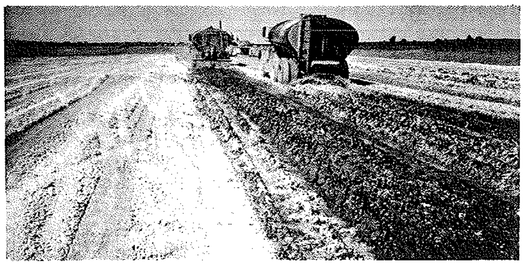


Figure 38. Mixing with disc harrow.

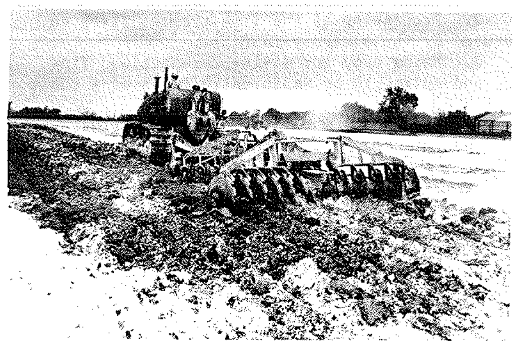
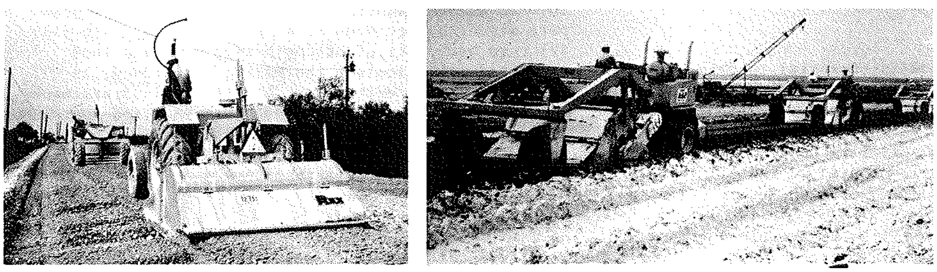


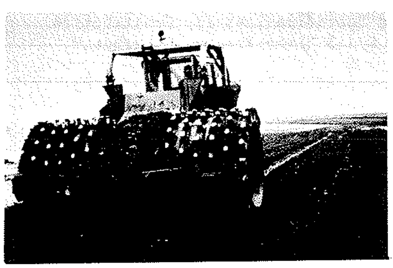
Figure 39. Mixing with rotary mixers.



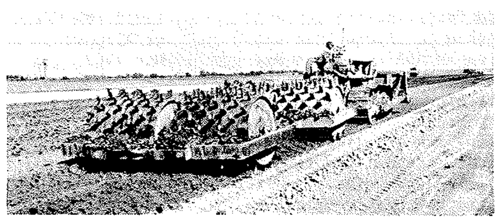
(a) Rotary mixers on project in Dallas County, Texas.

(b) Train of rotary mixers on Dallas-Fort Worth Airport.

Figure 40. Compacting lime-treated materials with sheepsfoot roller.

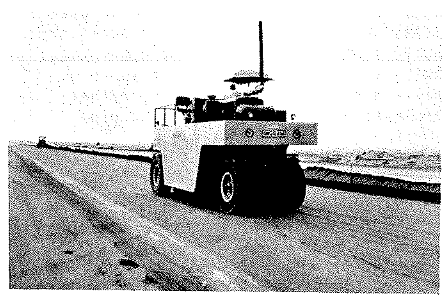


(a) Self-propelled sheepsfoot roller.



(b) Double sheepsfoot roller.

Figure 41. Pneumatic roller completes compaction of lime-cement-flyash base, Newark Airport.



Method of Measurement

1. Lime to be measured in tons.
2. Processing of the lime-treated layer to be measured by the square yard.
3. Water used for mixing, compacting, finishing, and curing to be measured in units of 4 cubic meters (1000 gallons).
4. Bituminous materials used for curing seals to be measured by the ton or gallon.

Basis of Payment

1. Lime to be paid for at unit bid price per ton of material accepted in place.
2. Processing of the lime-treated material to be paid for at the unit bid price per square yard of material completed in place.
3. Water to be paid for at unit bid price per 0.38 cubic meters (100 gallons) of material used on the project.
4. Bituminous membrane to be paid for at unit bid price per ton or gallon of material used for curing purposes.

Field Quality Control

Adequate quality control is essential to obtain a soil-lime mixture which will meet the stabilization objectives and provide the desired performance. There are many factors which should be considered in the quality control of soil-lime construction.

Factors typically considered in soil-lime construction and procedures for field use are listed below.

1. Depth of Lime Treatment. Since lime elevates the pH of the soil, phenolphthalein, a color-sensitive indicator solution can be sprayed on the soil to determine the presence of lime (Fig 44). If lime is present, a reddish-pink color develops.

2. Pulverization. The degree of pulverization attained in field mixing is evaluated using selected sieve sizes. Most specifications are based on the 1-inch and the No. 4 sieves. The processed material is "dry sieved" to determine the percent passing. Care should be taken to insure that the plus No. 4 material fraction is not really an agglomerated soil-lime mixture which can be easily broken down by a simple kneading action to pass the No. 4 sieve.

3. Lime Spread Rate. In dry lime spreading operations, the spread rate is established in terms of pounds of lime per unit area of surface. A simple procedure for measuring the actual field spreading rate is to place a 1-square meter or 1-square yard piece of canvas or other suitable material on the grade and then after the lime has been spread determine the weight of lime on the 1 square meter or square yard.

4. Slurry Composition. To accurately determine the quantity of lime slurry required to provide a desired amount of lime solids, it is necessary to know the slurry composition. The most convenient method of checking lime-slurry composition is to determine the specific gravity of the slurry, either by using a hydrometer or a volumetric-weight procedure.

5. Lime Content. Lime content is specified in all soil-lime construction. An ASTM procedure (ASTM D3155-73) has been developed for determining the lime content of uncured soil-lime mixtures. The procedure is rapid and easy to conduct. Other methods of determining lime content are also used and are discussed in the proceeding section on design.

6. Density. Conventional procedures, i.e., sand cone, rubber balloon, nuclear, (Fig 45) are used to determine the in-situ density of compacted soil-lime mixtures. It is very important to recognize that the proper moisture-density relation for the soil-lime mixture be used in the density control operation. The moisture-density relation for a soil-lime mixture may change relative to such factors as curing time. For example, if a soil-lime layer is reworked at some later date following initial construction the maximum dry density and optimum moisture content for the mixture probably will be different from the original mixture.

7. Moisture Content. Conventional procedures, oven drying and nuclear methods (Fig 45) can be used for moisture determinations. In calibrating the nuclear equipment consideration should be given to the presence of the lime in the mixture.

8. Mixing Efficiency. The thoroughness and efficiency of the field mixing operation is of interest. A simple procedure for evaluating mixing efficiency is: (a) secure a sample of the field mixed soil-lime material; (b) halve the sample; (c) prepare strength specimens (unconfined strength is normally satisfactory) from one portion; (d) completely "re-mix" the other portion of the field mixture to insure almost "100 percent mixing;" (e) prepare strength specimens from the "re-mixed" material; (f) cure both sets of strength specimens and test them; and (g) calculate the mixing efficiency, as follows: mixing efficiency, % = field mixed strength/lab mixed strength x 100. For mixed in-place operations mixing efficiencies normally range from 60 to 80 percent. In some types of soil-lime mixing operations lower values may be acceptable.

Specification References

Many agencies have developed specifications and special provisions for soil-lime construction. A comprehensive listing of current specifications and special provisions is presented below.

1. AASHTO - Guide Specifications for Highway Construction, 1968, (Sec. 307 on lime-treated subgrade).
2. U. S. Department of Transportation (FAA) 150/5370A "Standard Specifications for Construction of Airports," Item P-155 "Lime-Treated Subgrade," May 1968.
3. U. S. Corps of Engineers, "Engineering and Design Manual - Soil Stabilization for Roads and Streets" (also AFM 88-7, Chapter 4), June 1969.
4. U. S. Corps of Engineers, "Guide Specification for Military Construction - Lime Stabilized Base Course, Subbase or Subgrade for Roads and Streets," CE 807.32, December 1961 (partly revised February 1971).
5. National Lime Association, "Lime Stabilization Construction," Bulletin 326, 1972.
6. State Specifications or special provisions for the following states: Alabama, Arkansas, Arizona, California, Colorado, Florida, Georgia, Idaho, Illinois, Iowa, Kansas, Louisiana, Maryland, Minnesota, Mississippi, Missouri, Nebraska, New Mexico, New York, North Carolina, North Dakota, Ohio, Oklahoma, Oregon, South Dakota, Tennessee, Texas, Utah, Virginia, Wisconsin, Wyoming, etc.

Field Variability

Complete soil-lime construction will display variations in engineering properties such as strength and modulus of elasticity. Such variability is typical of all field construction operations. Major factors contributing to field variability in soil-lime construction are:

1. variations in properties of the soil encountered along the grade,

Figure 42. Vibrating roller completes compaction of subgrade—Virginia.

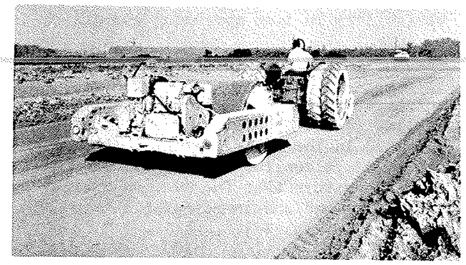


Figure 43. Moist curing of lime stabilized subgrade—Blytheville, Arkansas.

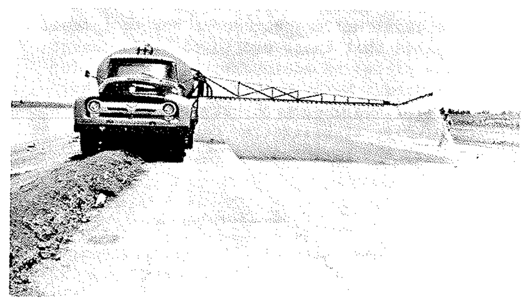


Figure 44. Checking uniformity of mixing with phenolphthalein solution.

Figure 45. Determination of moisture and density with nuclear gauge—Wisconsin.

2. variability in lime spreading and distribution,

3. variability in pulverization and mixing, and

4. moisture and density variations in the compacted soil-lime layer.

As indicated in the Field Quality Control Section, it is essential to monitor all aspects of soil-lime construction to assure that the desired quality of construction is secured and an acceptable level of uniformity is achieved (Fig. 44).

Summary

This section attempts to describe and summarize modern soil-lime construction procedures and equipment. It is anticipated that these procedures will change rapidly as new pieces of equipment and new uses for lime are developed. Nevertheless, it is felt that this section provides a comprehensive summary and description of lime stabilization construction at the time this report was prepared.

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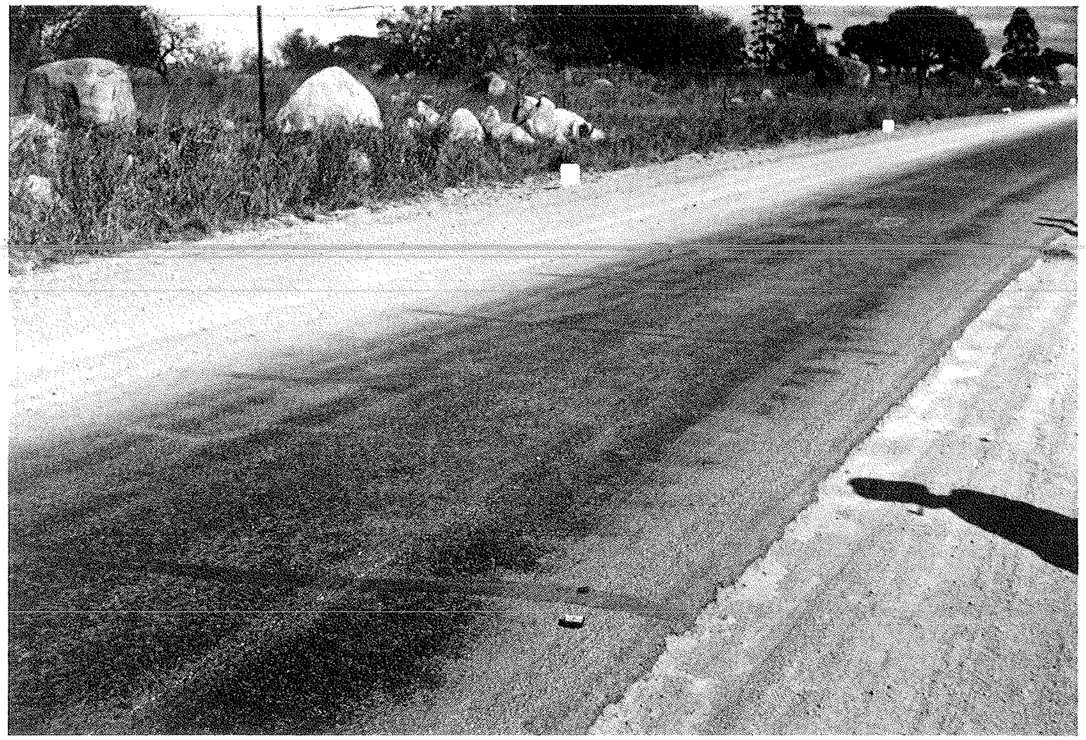

Shrinkage cracks in soil-cement base do not mean that pavement is unsatisfactory (Central Africa).

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Contents

Field Studies on the Pulverization of Black Cotton Soil for the Construction of Stabilized Soil Road Bases

H. L. UPPAL, L. R. CHADDA, and P. K. DHAWAN, Central Road Research Institute, New Delhi, India

Stabilization of black cotton soil with lime has been found effective in improving the engineering properties of the soil. Consequently, this has led to the increasing use of lime-stabilized black cotton soil in subbases or bases of road pavement. A properly pulverized soil is, however, a prerequisite for successful stabilization of soil. This paper describes a number of methods that have been tried in the field to achieve an economical and effective pulverization method. It has been shown that an acceptable degree of pulverization can be attained when the soil is handled mechanically at a particular moisture range by using agricultural machinery.

EXPANSIVE SOILS (1) occur in different parts of the world. One such soil, commonly termed black cotton soil, has similar characteristics and forms one of the major soil groups (2) of India, covering an area of about 500,000 km^2 (Fig. 1). The soil is predominantly montmorillonite (3, 4) having high base exchange capacity. It is characterized by high swelling on wetting and excessive shrinkage on drying. When swelling is restricted, it results in the development of swell pressure (3). On account of these peculiar properties, the black cotton soil presents serious problems in the construction of roads. Even at places where the conditions for the development of swell pressure do not exist, the roads still fail because of poor supporting power of the subgrade in wet condition. 79

It has been observed that the waviness at the road surface is mostly due to the working up of the soft subgrade soil into the crevices of the stone soling, thereby dislodging the soling stone from its original position. The sinking of the stone soling into the soft subgrade is a continuous process, and no amount of strengthening of the existing pavement at the surface would remedy this defect. It is, therefore, very necessary that a compacted layer of nonexpansive material having low voids be provided to prevent the movement of subgrade soil into the crevices of the stone soling. Such materials that could be considered suitable are light-textured soils, sands, or gravelly soils. In India, sandy soils or sands are not generally found in black cotton soil areas. At places, however, granular material mixed with soil fines, locally called moorum, occurs and a compacted layer of this laid over the subgrade before placing the stone soling has led to satisfactory results.

There are, however, still very large areas where none of these materials occurs and the only alternative is to improve the existing soil for use as a subbase between the subgrade and the stone soling. One of the known effective methods (4, 5) to improve the engineering properties of black cotton soil is the stabilization with lime. To achieve these requirements, it is necessary that the soil, which is generally in the form of hard clods, be brought to a reasonable degree of fineness to facilitate uniform mixing of lime with soil.

Paper sponsored by Committee on Lime and Lime-Fly Ash Stabilization.

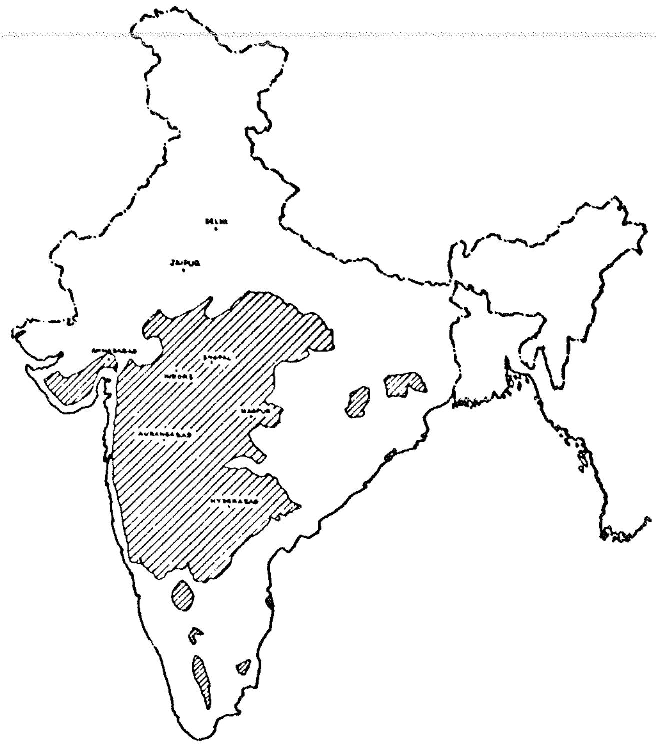


Figure 1. Black soil region of India.

Whereas light-textured soils are generally in a friable state when removed from the fields, it is not so in the case of expansive black cotton soil, which is very soft and sticky in the wet condition but very hard in the dry state.

OBJECT OF THE STUDY

The object of the present study, therefore, is to evolve a technique for an effective and economic pulverization of black cotton soil, which is a prerequisite for the uniform mixing of soil with lime and the subsequent development of strength.

Before taking up the field study, it was considered necessary to define the degree of fineness of soil. According to unpublished literature from the British Road Research Laboratory, the degree of fineness, which is also commonly termed as degree of pulverization, is determined from the formula

$$\frac{W_1 - W_2}{W_1 + W_3} \times 100$$

where

W_1 = total weight of the sample,
W_2 = weight of the sample retained on $^3/_{16}$-in. sieve, and
W_3 = weight of the sample passing $^3/_{16}$-in. sieve and retained on No. 8 sieve (British standard sieve or B. S. S.) 2 mm size.

It will appear from this formula that the fineness is not only controlled by the percentage of material passing the $^3/_{16}$-in. sieve but also by the fraction smaller than 2 mm.

For stabilization of light-textured soils with cement, about a 65 percent degree of pulverization (6), according to this formula, is considered suitable; but for hard clods of black cotton soil, such a high degree of pulverization may not be a practical possibility

TABLE 1

EFFECT OF DEGREE OF PULVERIZATION OF SOIL ON THE SOAKED CBR WHEN TREATED WITH 3 PERCENT COMMERCIAL HYDRATED LIME (PURITY 40 PERCENT) AND COMPACTED TO 1.5 gm/cc DENSITY

Sample	Percent Passing			CBR Soaked (percent)	Moisture Absorption (percent)
	1-in. Sieve	$\frac{3}{16}$-in. Sieve	No. 8 Sieve (B.S.S.)		
1	100.0	0.0	0.0	2.4	27.5
2	100.0	50.0	15.0	14.2	26.3
3	100.0	100.0	30.0	14.3	26.9
4	100.0	100.0	100.0	14.7	25.3

Note: Soil characteristics are liquid limit = 75.3 percent; plasticity index = 34.7 percent; and fraction coarser than No. 200 sieve (U.S. sieve) = 5.0 percent.

(7), 8). It was, therefore, considered necessary to relax the limits of fraction finer than 2 mm from the calculations. Before finally accepting the modification, it was essential to know how the clods of black cotton soil pulverized to particles of varying sizes will affect the resultant strength in saturated condition when the soil is stabilized with lime. Toward this objective, a preliminary laboratory study was carried out with soil samples having varying clod sizes. To achieve a limited increase in strength, trials were made using a low concentration of lime.

In the laboratory trials, the soil having a varying degree of fineness was compacted at optimum moisture with 3 percent commercial lime of known purity. After curing the treated specimens for 10 days, these were tested for soaked CBR. The results obtained are given in Table 1. 81

It will be observed from the data given in Table 1 that if the soil to start with consists of at least 50 percent passing the $\frac{3}{16}$-in. sieve then the strength attained is practically the same, irrespective of the fraction passing the No. 8 sieve (B.S.S.). It may be due to the fact that the process of mixing lime with soil and subsequent compaction may have resulted in further improving the degree of pulverization.

PULVERIZATION BY MANUAL LABOR

It is a common experience that during the rainy season the black cotton soil is sticky and difficult to handle. The field trials on the pulverization of soil by manual labor were, therefore, restricted to the dry season; details are given in the following.

Crowbar and Pickax

The field trials for the pulverization of black cotton soil were carried out on Berasia-Sironj Road near Bhopal (central India) in 1964. After removing the top vegetation, the dry soil crust for a depth of about 8 in. was loosened with crowbars. The soil thus obtained was comprised mostly of 5- to 6-in. clods. These were broken with pickaxes or rammers but the output was very poor, which raised the cost to 20 rupees per 100 cu ft for getting soil of acceptable degree of pulverization, i.e., about 100 percent passing the 1-in. sieve and about 50 percent passing the $\frac{3}{16}$-in. sieve. With a view to economizing on cost, an attempt was made to use a country plow drawn by a bullock, instead of digging manually, but this did not work well on account of the soil being dry and hard.

Wetting and Drying of Soil Clods

It has been observed that, when black cotton soil shrinks in the process of drying, high stresses are produced that lead to the disintegration of soil at the surface. Advantage was taken of this phenomenon in the pulverization of the soil. In actual practice, the soil dug from the fields was stacked and water was sprinkled on the clods. In the process of drying, the shrinkage of the soil took place and led to its disintegration at the surface. The process was frequently repeated to get more and more of the soil fines. It gave satisfactory results as almost 50 percent of the material passing the $^3/_{16}$-in. sieve could be obtained. This process would, however, require water within economic reach. Besides being slow, such operations will cost about 12 to 15 rupees per 100 cu ft.

PULVERIZATION BY MECHANICAL MEANS

Power Roller

The soil in the dry condition was dug from the adjoining fields, and clods were broken with pickaxes so as to reduce them to a size not bigger than 2 in. The soil clods were spread over a hard subgrade, and a power roller passed over them a number of times with frequent raking of the crushed material. It was found that about 8 passes of the roller and frequent raking of the rolled soil resulted in grading as follows:

Sieve No.	Percent Passing
$1\frac{1}{2}$ in.	100.0
1 in.	83.0
$^3/_8$ in.	80.0
$^3/_{16}$ in.	60.0

The cost of soil pulverized according to this method worked out to 10 to 12 rupees per 100 cu ft.

Heavy Agricultural Machinery

82 In the absence of a specially designed plant for the purpose, agricultural machinery available in the country was used. Field trials were, therefore, carried out with the following heavy agricultural machinery normally used for plowing the field and breaking clods (Fig. 2):

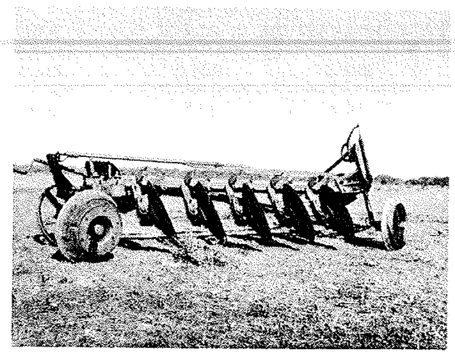
1. International Caterpillar tractor, 110 hp;
2. Moldboard plow consisting of 4 plowshares that can plow to about 15 in. in depth;
3. Disc plow consisting of 5 discs 28 in. in diameter with a working width of 10 ft; and
4. Offset notched disc harrow consisting of 18 discs 22 in. in diameter arranged in 2 gangs with a working width of 10 ft.

A field trial was initiated at Sehore (central India) by using this machinery in April 1964 when the ground surface was hard and badly cracked because of a hot, dry summer. To start with, the moldboard plow with a working width of 6 ft was used. This could plow up to a depth of about 15 in., giving clods of varying sizes with a maximum of about 8 in. After the moldboard plow was used, the disc plow was operated on the excavated soil. This reduced the size of the big clods to about 4 in. in the process of slicing. The soil was further subjected to the action of the offset notched disc harrow to improve pulverization. It was observed that, even with 6 passes of the disc harrow, there were still many clods 4 in. in size that resisted pulverization and were therefore removed manually. The sieve analysis of the resultant soil is as follows:

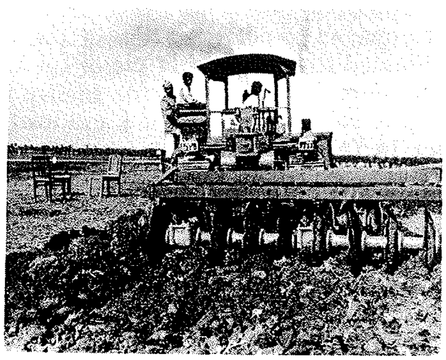
Sieve No.	Percent Passing
$1\frac{1}{2}$ in.	100.0
1 in.	81.4
$^3/_8$ in.	57.6
$^3/_{16}$ in.	33.5

During this operation, it was observed that those clods that were dry and consequently hard resisted breaking up, whereas slightly wet clods could be pulverized easily. It was, therefore, inferred that the pulverization of black cotton soil with heavy agricultural machinery required the soil to exist within a certain range of

Moldboard plow



Disc plow



Offset notched disc harrow

Pulverized black cotton soil being carried by the laborers

Figure 2. Agricultural machinery.

moisture for effective pulverization. With a view to finding out the range of moisture that would facilitate pulverization, soil samples at various depths of the natural subgrade were taken and subjected to moisture tests. The results are as follows:

Depth, in.	Moisture, Percent
6	7.1
12	15.6
18	18.2

It can be inferred that the clods that could not be pulverized had a moisture content of about 7 percent and those that could be readily pulverized had a moisture content ranging between 15 and 18 percent. It can, therefore, be stated that it will be more economical to pulverize black cotton soil after the rainy season when uniform field moisture conditions are likely to prevail.

Experiments were also made on the pulverization of black cotton soil in a moist state. Before operating the machinery, the moisture distribution to a depth of 2 ft was checked at 3 different stretches on the Sehore-Bilquishganj Road (central India) where the experiments were carried out. The data are given in Table 2.

It will be observed that the moisture throughout the depth is very uniform except at the top where there is a slightly lower moisture content.

The field trials were carred out in December for a length of about 440 yards using the same machinery.

TABLE 2
PERCENTAGE OF MOISTURE IN BLACK COTTON SOIL

Depth, in.	Site 1	Site 2	Site 3
1	14.7	15.1	14.8
6	19.8	19.5	19.8
12	20.4	19.9	20.9
18	20.5	21.6	21.9
24	20.5	22.0	21.9

TABLE 3
SIEVE ANALYSIS OF BLACK COTTON SOIL PULVERIZED IN THE MOIST STATE WITH AGRICULTURAL MACHINERY

Sieve No. (B.S.S.)	Percent Passing		
	2 Passes	4 Passes	6 Passes
1½ in.	100.0	100.0	100.0
1 in.	82.6	88.3	89.6
⅜ in.	70.6	69.9	79.0
$^3/_{16}$ in.	44.8	49.6	55.8
No. 8	29.1	34.3	38.6
No. 36	5.2	6.8	8.2

It was observed that, after the soil had been dug out with the moldboard plow, the disc plow could conveniently cut down the big clods to a smaller size at this moisture. The operation was also smoother as compared to pulverization of the dry soil. The soil thus pulverized was further subjected to the action of the offset notched disc harrow, with a view to determining the minimum number of passes needed to achieve the required degree of pulverization. The soil was tested for sieve analysis after 2, 4, and 6 passes. The results are given in Table 3.

It will appear from the data given in Table 3 that the acceptable limits of pulverization, i.e., 50 percent passing the $^3/_{16}$-in. sieve, could be achieved with 6 passes of the offset harrow. It will be observed further that insofar as the upper limits are concerned the conditions are just satisfied. It will be found more economical to accept about a 10 percent fraction coarser than 1 in. than to make additional passes of the offset disc harrow to pulverize it further. It will also be noticed that with this type of machinery the moisture for effective degree of pulverization ranges between 15 and 20 percent. The cost of pulverization as worked out by this machinery is 1 rupee per 100 cu ft as given in Table 4.

Light Agricultural Machinery

As heavy agricultural machinery is not easily available at most of the sites, further field trials were carried out using light agricultural machinery, which is readily procurable. The trials were made at Sidhantam in Andhra Pradesh (south India) where the construction of 5½ miles of the right approach road to Vasista Bridge, forming a part of the National Highway, was undertaken.

The light machinery consisted of the following:

1. Tractor, 50 hp;
2. Moldboard plow consisting of 3 plowshares;
3. Disc harrow consisting of 20 saucer-shaped discs 10 in. in diameter; and
4. Offset disc harrow consisting of 10 discs 20 in. in diameter arranged in two gangs.

The operation with this machinery was carried out, as in the case of heavy agricultural machinery, at varying moisture contents. It was found that effective pulverization could be economically achieved when the moisture content of the soil ranges between 10 and 22 percent, as against 15 and 20 percent in the case of the heavy machinery. It was further observed that it required about 6 passes of the disc harrow and about 10 passes of the offset harrow to achieve an acceptable degree of pulverization.

The cost of pulverization worked out to 2 rupees per 100 cu ft of loose soil as given in Table 5.

The higher cost of pulverization with light agricultural machinery as compared to heavy machinery resulted from the fact that the moldboard plow fitted with a light tractor could not plow more than 8 in. deep.

As a result of conducting a number of trials, it was found that, with the light machinery, it was possible to pulverize only about 8,000 cu ft of the soil in 8 hours as against 40,000 cu ft with heavy machinery during the same period. Although heavy

TABLE 4

COST ANALYSIS OF PULVERIZATION OF BLACK COTTON SOIL WITH HEAVY AGRICULTURAL MACHINERY

Item	Amount
Cubic feet of soil pulverized in 8 hours	40,000
Hire charges of the machinery at 45 rupees per hour	360.00
Labor charges, rupees	20.00
Miscellaneous charges such as for tools and repairs, rupees	20.00
Total rupees	400.00
Cost (in rupees) of pulverizing 100 cu ft $= \frac{100 \times 400}{40,000}$	1.00

TABLE 5

COST ANALYSIS OF PULVERIZATION OF BLACK COTTON SOIL WITH LIGHT AGRICULTURAL MACHINERY

Item	Amount
Cubic feet of soil pulverized in 8 hours	8,000
Hire charges of the machinery for 8 hours, rupees	120.00
Labor charges, rupees	20.00
Miscellaneous charges such as for tools and repairs, rupees	20.00
Total rupees	160.00
Cost (in rupees) of pulverizing 100 cu ft $= \frac{160 \times 100}{8,000}$	2.00

machinery is more economical to use, it is not readily available in the market. Therefore, even though the cost of pulverization with light agricultural machinery is slightly more, it is still preferred on account of its easy procurement either from the market or on hire from the agriculturists. The additional cost of pulverization is not likely to have any significant effect on the overall cost of road construction.

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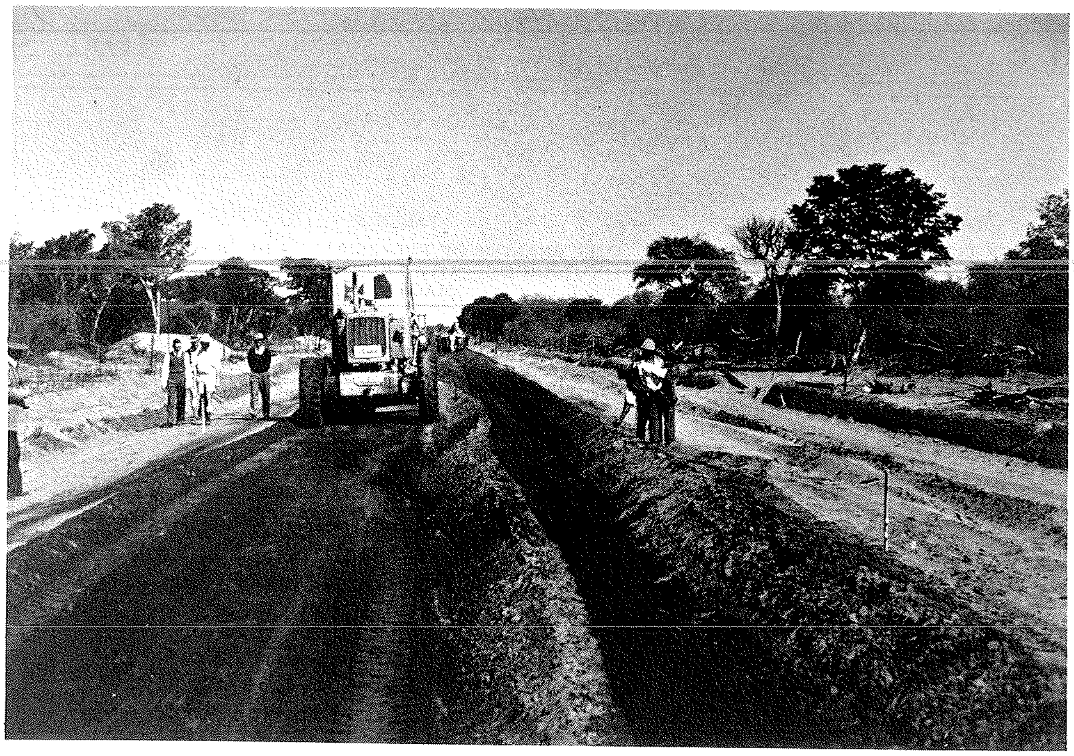
Kalahari sand was stabilized with tar (Central Africa).

BUILDING AND ROAD RESEARCH INSTITUTE
(GHANA ACADEMY OF SCIENCES)

TECHNICAL PAPER NO.1

S O I L - C E M E N T
A Material of Construction for Road and Airfield Pavements

BY

H. S. BHATIA
B.Sc., D.I.C., M.A.S.C.E.,
A.M.I.C.E., M.I.H.E., AMIE.
Principal Research Officer

Building & Road Research Institute
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SOIL-CEMENT, A MATERIAL OF CONSTRUCTION FOR ROAD AND AIRFIELD PAVEMENTS

I INTRODUCTION

1. During the World War II and the subsequent post war road development in Western Europe and the United States, considerable use was made of local materials with certain binders for pavement construction. It was during this period that an extensive programme of research on the use of soil-cement for road construction was initiated in various parts of the world. This was followed by large scale experimental tests, to understand fully the limitations of soil-cement as a material of construction. A number of other binders such as bitumen, lime, chemicals, industrial wastes, etc., were also tried for the stabilization of soils and some of these have given satisfactory performance in the field. However the use of cement for stabilization of soil, under varied conditions of climate and soil, has found wide acceptance among highway engineers, due to the ease of handling such a construction, in addition to satisfactory performance in the field.

2. During the post war road construction era, many orthodox civil engineers considered soil-cement as a second grade engineering material for road construction, as compared to stone and concrete. This group of engineers considered soil-cement as a suitable material only for roads carrying light traffic and temporary airfield pavements. Large scale field experiments backed by systematic research in this field steadily convinced all shades of professional opinion, that soil-cement

could effectively replace the conventional methods of road construction, without impairing the quality of construction, in addition to introducing a certain amount of economy.

II BRIEF HISTORY OF THE DEVELOPMENT OF SOIL-CEMENT

1. The South Carolina State Highway Department in the United States of America, initiated a scientific study on the use of soil-cement for pavement construction as far back as 1932, and this was followed by an extensive programme of research by the Portland Cement Association in 1935. In the same year the U.S. Bureau of Roads in collaboration with Portland Cement Association put up 1.5 miles of an experimental road length near Johnsonville, to check the laboratory results produced by various research organisations in the United States. The success of this experiment attracted many states towards the idea of using soil-cement for road construction.

2. The use of soil-cement for airfield construction was made during the World War II and it was felt that soil-cement was the only answer to speed up construction, even under very abnormal conditions. It is estimated that during 1941-44, about 22 million square yards of airfield pavements were built with soil-cement.

3. For the post war road development, soil-cement was very extensively used in many countries of the World, in addition to U.S.A. and Western Europe.

4. The object of this monograph is to review the development of soil-cement as an engineering material for road and airfield construction, with special emphasis on tropical and subtropical conditions.

III SOIL-CEMENT AS A CONSTRUCTION MATERIAL

1. The combination of soil with cement under controlled conditions of moisture and density produces a material of distinct physical and engineering characteristics. These properties depend on four main factors:-

(i) Nature of soil.

(ii) Proportion of soil, cement, and water in the mixture.

(iii) The compactive energy used for the moulding of soil-cement.

(iv) Physical conditions such as the curing temperatures and age of the soil cement mixes.

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2. In the absence of any consolidated data showing general mechanical, physical, elastic and strength characteristics of similar soil-cement mixes, tables I and II[1, 2] are presented to give some idea of the type of engineering material, soil-cement mixes make. It may also be of some interest to make a few general remarks on the inter-relationship between various properties of soil-cement mixes, before the limitations of various properties are discussed in detail.

3. Volume Changes

It is generally observed that an increase in cement content tends to reduce the shrinkage of a clay, due to the integranular cohesion produced by the hydration of cement. It may however be pointed out that in using soil-cement an optimum quantity of cement is required to cut down

TABLE I

SHOWING THE ELASTIC AND STRENGTH CHARACTERISTICS OF SOIL CEMENT MIXTURES. (REF.1)

Soil Type	ATTERBERG'S LIMITS		COMPACTION CHARACTERISTICS		CEMENT CONTENT %		COMPRESSIVE STRENGTH PSI (28 days)	FLEXURAL STRENGTH Psi	MODULUS OF ELASTICITY (Psi x 10^6)	
	L.L.	P.I.	O.M.C. %	Max. Density LBS/C.ft.	By wt.	By Vol.			$E_{(d)}$	$E_{sc.}$
Sand	16	N.P.	9.2	132	3.8	5	450	110	2.05	-
					6.0	8	800	180	2.75	-
					8.5	11	1,225	260	3.30	-
Sandy loam	17	N.P.	10.0	130	3.8	5	300	80	1.40	0.90
					6.1	8	650	145	2.00	1.25
					8.6	11	1,025	215	2.60	1.65
Clay Sand	28	15	12.2	123	5.7	7	475	105	1.30	-
					8.3	10	625	150	1.50	-
					11.0	13	800	195	1.75	-
Silty loam	26	7	15	113	8.0	9	525	125	0.90	0.55
					11.0	12	725	155	1.05	0.65
					14.2	15	900	190	1.25	0.75

E_d = dynamic modulus of elasticity

E_{sc} = static modulus of elasticity

TABLE II

DATA ON THE CO-EFFICIENT OF THERMAL EXPANSION OF SOIL CEMENT MIXES (REF. 2)

Soil Type	Atterberg's Limits L.L.	P.I.	Fraction < 200 sieve	Cement Content (% by wt.)	Co-efficient of thermal expansion (In/in deg F) x 10^{-6}
Sandy loam	22.0	5.00	65	2.5 - 10	4.5 to 5.8
Silty loam	28.0	9.0	19	2.5 - 10	4.1 to 5.6
Silty clay loam	37.0	17.0	8	2.5 - 10	3.9 to 6.1
Loam	25.0	10.0	43	2.5 - 10	4.6 to 6.3

the shrinkage to a minimum. Normally high concentrations of cement tend to produce shrinkage cracks in the soil-cement and sometimes these are detrimental to the strength of a pavement.

4. Thermal Expansion.

In the tropics where temperatures are high, the thermal expansion is an important physical property of soil-cement. A study conducted in India (2) on four types of soils, revealed that for the same density, the increase in percentage of cement increased the thermal expansion. In addition the increase in densities with the same quantity of cement also increases the thermal expansion. The curves as obtained in the study under reference are shown in figure I.

5. Permeability

Generally all types of soils respond very sharply to the decrease of permeability with the addition of cement. The permeability depends on the texture quantity and quality of cement. Some studies conducted in the United States indicate that addition of eight percent cement in a sand sample reduced its coefficient of permeability from 1500 to 6 ft./year. In the case of clay-sand and sandy clay samples, with the same percentage of cement the permeabilities were reduced from 182 to 6 and 1 to 0.5 ft./year respectively.

6. Strength Characteristics

The compressive strength, flexural strength, and modulus of elasticity of a soil-cement mix increases with the concentration of cement. In addition the strength with the

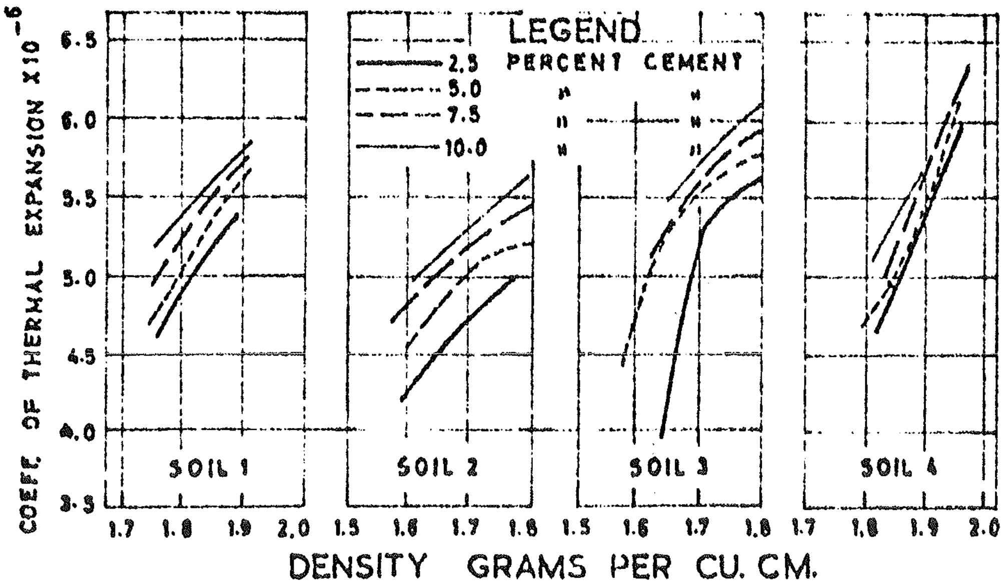

Fig. 1 Effect of density on thermal expansion of soils

same concentration of cement, is higher in a soil with a better grading. The various strength properties of soil-cement are correlated and it is generally accepted that flexural strength of soil cement mixes is approximately 20-25 percent of the unconfined compressive strength. The relationship between compressive strength, and the flexural strength, as also the modulus of elasticity are linear, except for lower strengths of silty and clay soil.

The curing temperature and period of curing have pronounced effect on the increase in strength of soil-cement mixes and this has been covered in more detail in the following sections.

IV SOIL SURVEY

1. The soil forms about 90-95 percent of a soil-cement mix and acts as an aggregate, and cement imparts to the mix cohesion. It is thus important that a proper selection of the soil is made to get the best results with a minimum proportion of cement.

2. Before carrying out a detailed soil survey along a proposed alignment of a road, it is important to have a visual reconnaissance of the area to know in a general way the nature of soil, topography, water table, availability of streams or rivers for the supply of water etc. Such an information is very useful at the time of detailed planning of soil-cement construction.

3. It may be important to say a word on the Soil Classification System to be adopted for the identification and

classification of soils, in order to get uniform nomenclature of soils in the country. Many systems of soil classifications have been developed during the past three decades. The most widely accepted system at present is a modification of the original Casagrande's System, which is known as "Unified Soil Classification System", in the United States of America.

4. The various groups of the Unified Soil Classification System are shown in table III. In addition table IV gives broad indication of the engineering properties of various groups, when the system is used for road and airfield construction.

5. In almost all the advanced countries of the world, soil engineering maps are available, which are a great help in planning soil surveys for road projects. In Ghana also the Central Materials Laboratory (5) of the Public Works Department, did some preliminary work on the distribution of surface soils in the country. The map as produced by the Central Materials Laboratory is shown in figure 2. Another map shown in figure 3 was also produced by the same organisation, showing the general distribution of gravels for road construction in the country. These two maps can be useful in the initial planning of a soil-cement road project. These maps however give only a general guidance and can not replace detailed soil and gravel survey on a project. Tables V and VI give additional information on the formation and engineering properties of different groups of soils in Ghana, as provided in figure 2.

V CRITERION FOR THE SELECTION OF A SOIL

1. A question has often been asked as to which are the

NOTE: Refer to Compendium 6, Selected Text 1 for details refered to in Table III which is not reproduced here.

TABLE IV CHARACTERISTICS PERTINENT TO ROADS AND AIRFIELDS

Major Divisions (1)	(2)	Symbol: Letter (3)	Symbol: Hatching (4)	Symbol: Color (5)	Name (6)	Value as Subgrade When Not Subject to Frost Action (7)	Value as Subbase When Not Subject to Frost Action (8)	Value as Base When Not Subject to Frost Action (9)	Potential Frost Action (10)	Compressibility and Expansion (11)	Drainage Characteristics (12)	Compaction Equipment (13)	Unit Dry Weight lb per cu ft (14)	Typical Design Values: CBR (15)	Typical Design Values: Subgrade Modulus k lb per cu in. (16)
COARSE-GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW		Red	Well-graded gravels or gravel-sand mixtures, little or no fines	Excellent	Excellent	Good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	125-140	40-80	200-300
		GP		Red	Poorly graded gravels or gravel-sand mixtures, little or no fines	Good to excellent	Good	Fair to good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	110-140	30-60	200-300
		GM d		Yellow	Silty gravels, gravel-sand-silt mixtures	Good to excellent	Good	Fair to good	Slight to medium	Very slight	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	125-145	40-60	200-300
		GM u		Yellow		Good	Fair	Poor to not suitable	Slight to medium	Slight	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	115-135	20-30	100-200
		GC		Yellow	Clayey gravels, gravel-sand-clay mixtures	Good	Fair	Poor to not suitable	Slight to medium	Slight	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	130-145	20-40	100-300
	SAND AND SANDY SOILS	SW		Red	Well-graded sands or gravelly sands, little or no fines	Good	Fair to good	Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller	110-130	20-40	200-300
		SP		Red	Poorly graded sands or gravelly sands, little or no fines	Fair to good	Fair	Poor to not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller	105-135	10-40	200-300
		SM d		Yellow	Silty sands, sand-silt mixtures	Fair to good	Fair to good	Poor	Slight to high	Very slight	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	120-135	15-40	200-300
		SM u		Yellow		Fair	Poor to fair	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	100-130	10-20	100-200
		SC		Yellow	Clayey sands, sand-clay mixtures	Poor to fair	Poor	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	100-135	5-20	100-300
FINE-GRAINED SOILS	SILTS AND CLAYS LL IS LESS THAN 50	ML		Green	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Poor to fair	Not suitable	Not suitable	Medium to very high	Slight to medium	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	90-130	15 or less	100-200
		CL		Green	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Poor to fair	Not suitable	Not suitable	Medium to high	Medium	Practically impervious	Rubber-tired roller, sheepsfoot roller	90-130	15 or less	50-200
		OL		Green	Organic silts and organic silt-clays of low plasticity	Poor	Not suitable	Not suitable	Medium to high	Medium to high	Poor	Rubber-tired roller, sheepsfoot roller	90-105	5 or less	50-100
	SILTS AND CLAYS LL IS GREATER THAN 50	MH		Blue	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Poor	Not suitable	Not suitable	Medium to very high	High	Fair to poor	Sheepsfoot roller, rubber-tired roller	80-105	10 or less	50-100
		CH		Blue	Inorganic clays of high plasticity, fat clays	Poor to fair	Not suitable	Not suitable	Medium	High	Practically impervious	Sheepsfoot roller, rubber-tired roller	90-115	15 or less	50-200
		OH		Blue	Organic clays of medium to high plasticity, organic silts	Poor to very poor	Not suitable	Not suitable	Medium	High	Practically impervious	Sheepsfoot roller, rubber-tired roller	80-110	5 or less	25-100
HIGHLY ORGANIC SOILS		Pt		Orange	Peat and other highly organic soils	Not suitable	Not suitable	Not suitable	Slight	Very high	Fair to poor	Compaction not practical	-	-	-

Note:

1. Column 3, division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is on basis of Atterberg limits; suffix d (e.g., GMd) will be used when the liquid limit is 25 or less and the plasticity index is 5 or less; the suffix u will be used otherwise.
2. In column 13, the equipment listed will usually produce the required densities with a reasonable number of passes when moisture conditions and thickness of lift are properly controlled. In some instances, several types of equipment are listed because variable soil characteristics within a given soil group may require different equipment. In some instances, a combination of two types may be necessary.
   a. Processed base materials and other angular materials. Steel-wheeled and rubber-tired rollers are recommended for hard, angular materials with limited fines or screenings. Rubber-tired equipment is recommended for softer materials subject to degradation.
   b. Finishing. Rubber-tired equipment is recommended for rolling during final shaping operations for most soils and processed materials.
   c. Equipment size. The following sizes of equipment are necessary to assure the high densities required for airfield construction:
      Crawler-type tractor -- total weight in excess of 30,000 lb.
      Rubber-tired equipment -- wheel load in excess of 15,000 lb, wheel loads as high as 40,000 lb may be necessary to obtain the required densities for some materials (based on contact pressure of approximately 65 to 150 psi).
      Sheepsfoot roller -- unit pressure (on 6- to 12-sq-in. foot) to be in excess of 250 psi and unit pressures as high as 650 psi may be necessary to obtain the required densities for some materials. The area of the foot should be at least 5 per cent of the total peripheral area of the drum, using the diameter measured to the faces of the feet.
3. Column 14, unit dry weights are for compacted soil at optimum moisture content for modified AASHO compaction effort.
4. In column 15, the maximum value that can be used in design of airfields is, in some cases, limited by gradation and plasticity requirements.

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TABLE II-3

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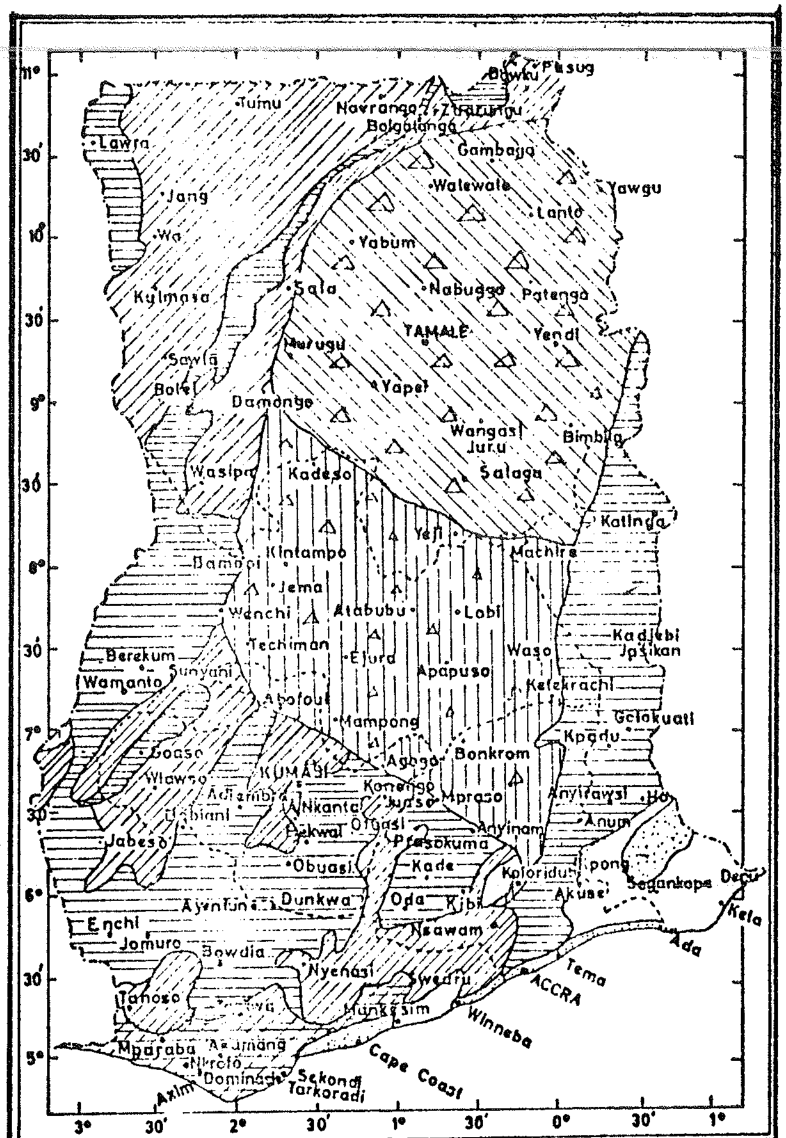
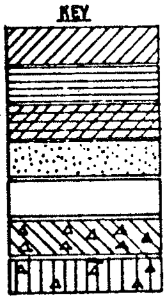


Fig. 2 TENTATIVE SURFACE SOIL ENGINEERING MAP OF GHANA.

KEY



- Direct Weathering products of Igneous & Sedimentary rocks.
- Intermediate valley rocks & adjoining out wash.
- Soils of filled valleys & outwash of plains.
- Coastal sands & silts.
- Tropical Earths
- Ground water Lateritic clayey sands
- Ground water Lateritic silty clays

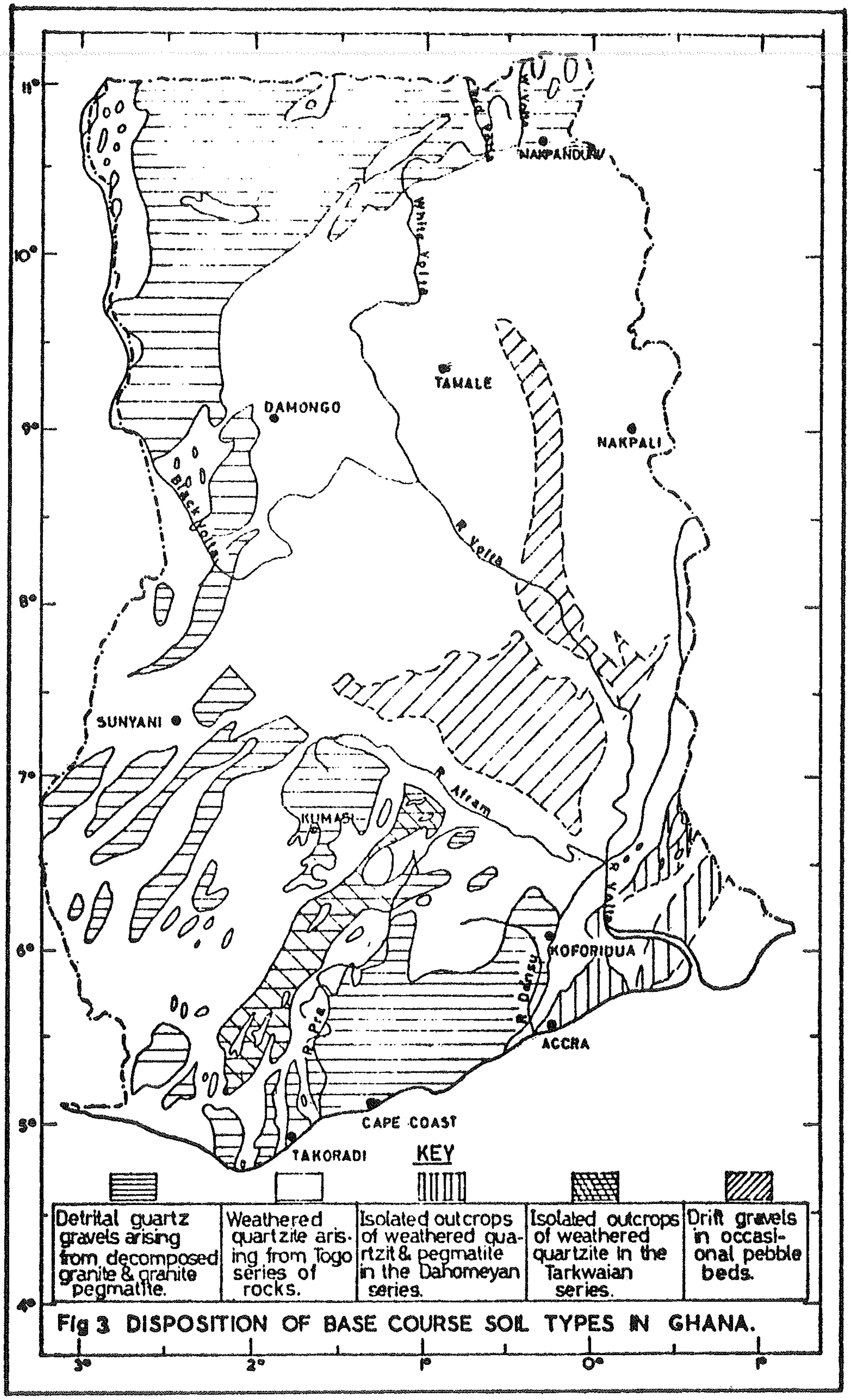


Fig 3 DISPOSITION OF BASE COURSE SOIL TYPES IN GHANA.

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TABLE V

FORMATION OF SUBGRADE SOILS IN GHANA (REF.5)

Group No.	Description of Groups	Parent Rock	Description of soil	Topography	Mean Rainfall/Annum	Mean Max. Temp. °F	Predominant Clay Mineral
1.	Direct Weathering products of igneous and sedimentary rocks.	In the North, from Sawla to Tumu, and in Ashanti, Eastern and Western Regions - granite Additionally, in Ashanti over phyllites and schists.	Sedentary formations - poorly graded sandy or silty gravel of coarse texture. Drift formations finer textured silty and sandy clays with little gravel.	In North at altitude 500 1300' and Ashanti at altitude 1000 2000' on rolling ground. In Eastern and Western Regions on plains and rolling topography	Northern Region 40"-45" Ashanti 55"-60" Western Region 80"-85" Eastern Region 30"-35"	Northern Region 90-95. Ashanti & Eastern Regions 85-90. Western Region 80-85	Kaolinitic
2.	Intermediate Valley rocks and adjoining out wash	In North-West-metamorphosed lavas and phyllites. Phyllites and schists on Northern border and in Ashanti. Shale and fine textured sandstone in remainder of Northern Region.	Variable texture from medium to fine, fineness increasing with depth. Sandy, gravelly clays, lean clays, etc. with 20-30% gravel	Rolling Altitudes of 300-500ft. in in N/R 500-800ft. in Ashanti and 150-200ft. in E/R.	As above	As relevant Regions above	Kaolinitic
3.	Soils of filled Valleys and out wash	Shales, sandstone and Limestone in W/R from Takoradi to Nakrofo to Tarkwa. Granite in isolated Cases.	Fine textured sedimentary clays and silts. Heavy clay near surface 10-25% gravel at 1-2ft. depth.	Plan Altitude 50-150 ft.	80-85"	85-90	Kaolinitic

Group No.	Description of Groups	Parent Rocks	Description of Soil	Topography	Mean Rainfall/Annum	Mean Max. Temp. °F	Predominant Clay Mineral
4.	Early stages of weathering of various rocks	Various sandstone, shale, granites and phyllites.	Medium texture at surface to coarse at depth. Sand to silty sand. Micaceous where derived from muscovite granite. No gravel.	Plain Maximum Altitude 200 feet.	In Eastern Coastal belt 30-35" Towards Cape Coast 60-65"	85-90	Not known
5.	Residual tropical earths	Ash/R.-metamorphosed lavas, phyllites and granites. Coast-phyllites, schists and gneisses	Texture very fine on surface cousening with depth. 15-20% gravel at depth 2-4 ft. expansive.	Slightly rolling A/R. Altitude 1000 ft. Coastal plains.	Ash/R.55-60" Coast (Eastern 30-35"	85-90	Montmorillonite
6.	Alluvial deposits subsequently laterised	Iron bearing rocks formed over sandstone shale and mudstone beds where poor drainage and rainfall/evaporation balance permits laterisation.	Coarse textured, rounded gravel 70-80% in top 1-2' fineness increases depth	Rolling maximum altitude 1500 ft.	40-45"	90-95	Kaolinite Possibly with Montmorillonite
7.	Alluvial deposits in process of laterisation.	Iron bearing rocks. Formed over shale and mudstone beds rarely over sandstone in conditions as for No. 6 above	Medium to fine texture 10-20% gravel at depths greater than 2 feet.	Gentle Maximum altitude 500 ft. Central Ghana.	50-55"	85-95	As above

TABLE VI

Showing the general engineering properties of different soil groups of Ghana. (Reference 5)

Soil Group and General description	Consistency Index L.L.	Consistency Index P.L.	Consistency Index P.I.	Gravels 2-5m.m.	Insitu den. lbs/c.ft.	Nat. Moisture Content	Lab/Comp. Ghana Density lbs/c.ft.	Lab/Comp. Ghana OMC.	C.B.R. % Compacted at OMC. & soaked for 48 hours.	Sp. Gravity of the particles	Base Exchange capacity
GROUP I Direct Weathering products of igneous & sedimentary rocks a) Sedentary	20-35	15-25	5-10	10-30	85-110	3-10	100-120	6-8	10-20	2.60-2.65	6-8
b) Drift	30-40	10-20	10-20	5-10	90-110	5-15	105-120	8-11	5-10	2.55-2.60	8-10
GROUP II Intermediate Valley rocks and adjoining out wash	20-50	10-25	10-25	No gravel up to 2' pea gravel about 10-15% at places in lower layers	90-105	4-15	105-120	8-15	10-20	2.55-2.60	8-12
GROUP III Soils of filled valleys and out wash of the plains	50-75	20-30	30-45	Up to 10% gravel at places in lower level	85-100	5-25	100-115	10-15	5-15	2.60-2.65	10-15
GROUP IV Early stages of weathering	20-30	15-20	5-10	Very little in various layers Soils on muscovite granite contain a high % of mica	105-120	3-12	115-120	8-12	4-10	2.7-2.8	8-12

TABLE VI (CONTD.)

Soil Group and General description	Consistency Index			Gravels 2-5m.m.	Insitu den. lbs/c. ft.	Nat. Moisture Content	Lab/Comp. Ghana		C.B.R. % Compacted at OMC & soaked for 48 hours.	Sp. Gravity of the particles	Base Exchange Capacity m.e./100gms. (-36 B.S.S.)
	L.L.	P.L.	P.I				Density lbs/c.ft.	OMC.			
GROUP V Residual Tropical Earths	60-100	20-40	30-60	Nil	85-95	20-45	95-105	15-20	5-8	2.60-2.65	20-45
GROUP VI Alluvial deposits subsequently laterised	30-45	20-25	10-20	10-30% in the top layers getting 40-50% with depths	120-150	3-8	125-140	6-8	30-50	2.65-2.70	10-30
GROUP VII Alluvial deposits in process of laterisation.	40-60	15-25	20-35	20-30% up to 3' gets less with depth	95-110	5-18	115-125	8-10	20-30	2.55-2.60	10-30

% Ghana compaction. 5" layers compacted in a C.B.R. mould each layer rammed with 25 blows of a 10 lbs. rammed dropped 18".

most suitable type of soil for stabilization with cement, and also if a demarcation line can be drawn between the soils which can be stabilized and which cannot be stabilized.

2. There is sufficient data available now to suggest that almost all types of soils can be stabilized with cement, the only consideration being economics, which has to be worked out in relation to similar type of construction with conventional materials. No hard and fast rules can be laid down regarding the relative economics of soil-cement construction in relation to conventional methods and this is left to the discretion of the Project Enginner. The choice of one method or the other depends on the availability and cost of different road materials on a given site.

3. It is however fairly well known that very clayey soils like tropical earths and also organic soils need special treatments before these can be stabilized with cement. Many attempts to stabilize such soils with cement alone have proved rather uneconomical. A lot of work has been done in Rusia, Anmerica, United Kingdom and India on using two or more stabilizers for such soils, and the results reported with lime/cement particularly are very satisfactory.

4. For all types of soils the quantity of cement required for their effective stabilization depends on their plasticity characteristics and gradings. Normally after a preliminary survey, only such soils are selected for detailed study which have a medium plasticity and appreciable quantity of sand. Some attempts

(6)

have been made to assess the requirements of cement for stabilization in a given soil on the basis of its specific surface area, but the method is not very convenient for application in the field. According to the Highway Research Board,(7) an ideal soil for economical stabilization with cement should fall within the following grain size limits.

Maximum size		3 inches
Passing No. 4 sieve A.S.T.M.	>	50%
Passing No.40 sieve A.S.T.M.	>	15-[illegible]%
Passing No. 200 sieve	<	50%

In addition the liquid limit should preferably be less than 40 and plasticity index not more than 18. This does not suggest that soils not conforming to these characteristics cannot be stabilized, except that the cost of stabilization in such cases will be relatively high. There are a number of soils highly silty and clayey, which not falling within the limits specified above are known to have been stabilized successfully, but the requirements of cement in such cases is as high as 10-15 percent. 107

5. In certain cases blending of soils to bring them near or within the recommended Highway Research Board limits, can reduce the quantity of cement. Therefore in places where blending is possible within an easy reach, it should be tried.

6. It has been observed that soils near the surface usually need a higher percentage of cement for stabilization, due to the presence of certain amount of organic matter. If however the upper 6-12 inch layer of soil is removed, it may require much less cement. In desert areas or areas of low rainfall,

such a possibility is rare and therefore the surface soil can directly be stabilized.

VI IMPORTANT FACTORS IN THE STABILIZATION OF SOIL WITH CEMENT

1. Moisture-Density Relation

Like raw soil, the soil-cement also has an optimum moisture at which it gets maximum density with a given compactive effort. Different laboratory methods employed for laboratory compaction of soils are, Proctor's Compaction, AASHO, mod. AASHO, Ghana Compaction etc. The selection of a given method for laboratory compaction will depend on the nature of compaction plant available in the field. The laboratory moisture density curve of a mix should be comparable to a certain degree to the field moisture density curve, obtained with the compaction plant. Therefore to achieve the best control and satisfactory results in the field, an appropriate laboratory test should be selected in determining the optimum moisture content.

2. The effect of increasing concentrations of cement on the optimum moisture-density relation of a mix, is the same, as would be the effect of adding fines in a soil. The effect of increasing compactive energy will also follow the same trend, as in a raw soil, i.e. the increase in compactive energy decreases the optimum moisture content and increases the dry density. Figure 4 shows the moisture-density relation of a typical Ghanaian soil stabilized with increasing concentration of cement. The soil is a lateritic gravel with liquid limit and plasticity index of fine fractions as 32.5 and 17.40 percent respectively.

LATERITIC GRAVEL SPECIFICATIONS			
GRAVEL Larger than No 7 BSS	34%	LIQUID LIMIT	32·5%
MAXIMUM SIZE	3/4"	PLATICITY INDEX	17·4%
FRACTION Passing No 200 BSS	28%	SPECIFIC GRAVITY	2·75

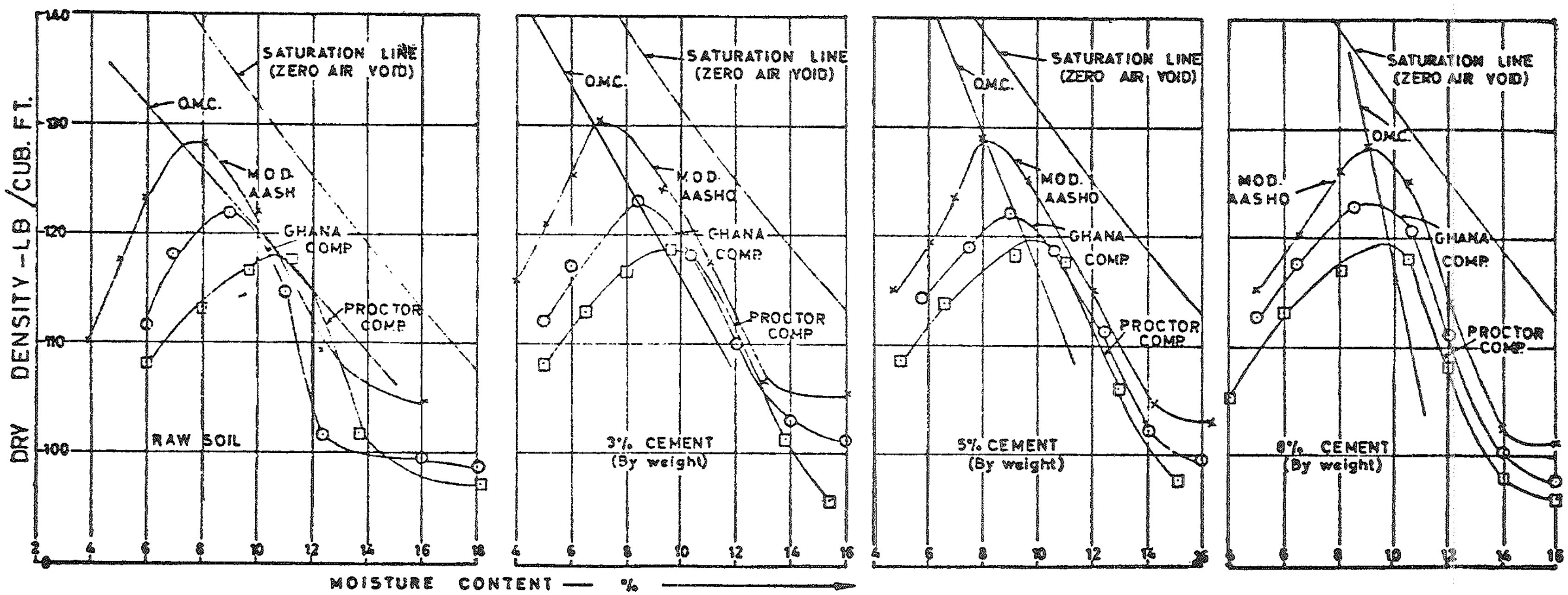


Fig. 4 MOISTURE DENSITY RELATION OF A LATERITIC GRAVEL (TAKINTA BARROW PIT HALF ASSINI) WITH DIFFERENT COMPACTIVE EFFORT AND INCREASING PERCENTAGE OF CEMENT

3. It may however be pointed out that the optimum densities in soil cement mixes do not necessarily produce maximum strengths, as is the case in raw soils. The work[8] on the stabilization of sandy and clayey soils, indicates that in the case of stabilized sand and sandy soils the maximum strengths are achieved on the dry side of the optimum moisture. On the other hand clayey soils give maximum strengths at densities slightly above the optimum moisture. This tends to point out that higher densities in a soil cement mix does not necessarily mean proportionately higher strengths. This is due to the fact that sands and sandy soils have a surface area much less than the clayey soils, and for this reason such soils do not absorb moisture on the surface of soil particles, with the result that most of the moisture is available for hydration of cement. At optimum moisture content therefore the amount of water required for cement hydration is more than needed, thus causing a reduction in strength. In the case of clays and clayey soils, the surface area is very much greater compared to sandy soils. A higher surface area will keep a lot of moisture tied up as absorbed water around the clay particles, thus leaving a deficiency of moisture for hydration at optimum moisture content.

4. Durability Tests

Before a soil-cement mix is finally accepted for use in the field, it must satisfy certain criteria of durability tests. According to American standards, soil-cement specimens 4 inches diameter, 4.6 inch in height are prepared in a Proctor's mould having removable base and collar, and cured for 7 days.

These are subjected to 12 cycles of wetting and drying, freezing and thawing tests, as per A.S.T.M. standards and should conform to the following:-

(i) Losses during 12 cycles of either wet and dry tests or freeze and thaw tests (ASTM designated D 559-44 and D 560-44) for different groups of soils shall conform to the following standards.

Soil Group			Loss %
U.S.P.R.A.	classification	A_2 & A_3	14
U.S.P.R.A.	"	A_4 & A_5	10
U.S.P.R.A.	"	A_6 & A_7	7

(ii) Maximum volume changes during the wet and dry test or freeze and thaw test should not exceed more than 2 percent of the value at the time of moulding.

(iii) Maximum moisture content at any time during the test shall not exceed the quantity which is required to completely fill the voids of the specimen, at the time of moulding.

(iv) The compressive strength of soil-cement specimens soaked for 1-4 hours prior to compressive strength test should increase with age and also with increase in cement content.

5. In United Kingdom, the Road Research Laboratory[(9)] in an attempt to establish some simple tests for finding out the weathering qualities of soil-cement, recommended a laboratory compressive strength of 250 psi, after 7 days curing, at its moulding moisture as a suitable criterion, to take care of all

the durability tests, specified in the A.S.T.M. standards. This criterion resulted out of an investigation carried out in 1939 by the Road Research Laboratory and could not be published due to war. In this study, it was found that cubes 3 inch size should have a minimum compressive strength of 250 psi at 7 days, to stand the requirements of the A.S.T.M. durability tests of wetting and drying, freezing and thawing. It was further observed that the criterion of 250 psi compressive strength, is applicable to cylindcrical specimens 2 inch diameter, 4 inch height and the results fairly correspond with 3 inch cube specimens.

112 6. A detailed study in the Central Road Research Institute of India[10] has shown that in the tropical and sub-tropical conditions, where freeze and thaw tests are not essential, a compressive strength of 150 psi at 7 days curing is sufficient to stand 12 cycles of wetting and drying, which do satisfy weathering conditions in the tropics.

VII SOILS REQUIRING SPECIAL TREATMENT

1. Organic Soils

The studies conducted at the Road Research Laboratory, U.K.[11, 12] indicate that an organic content of more than 0.5 percent normally retards the setting of Portland Cement, and results in lower strengths of soil cement mixes. It is further suggested that if the organic content in a soil is of an active nature, which can not be extrated with sodium hydroxide solution, then the hardening of cement can be accomplished by the addition of 0.2-0.3 percent calcium chloride.

2. Similar studies in the United States[13] on sandy soils suggest that the presence of all types of organic matter did not affect the setting of cement. It was further observed that the type of organic matter was more important than the percentage present. A study on a number of organic compounds, such as starch, sugars, cellulose etc., suggested that poly-saccharide compounds such as hemi celluloses and uronides which are the result of a microbial decomposition of soil organic matter, and also amino sugars present in certain soils, are definitely harmful for stabilization of soils. Organic matter with high molecular weight, such as lignin, starch, cellulose etc., does not appreciably affect the strength of soil cement mixtures. It has now been confirmed that retardation in the setting of cement is not brought about by the whole compound of organic matter, but due to certain active fraction of the compound.

3. The effect of different organic compounds on the development of strength of soil-cement is shown in figure 5 (a) and (b). In addition table VIII (16) shown the percentage of various organic compounds likely to retard the hardening of cement.

4. A considerable amount of work[11, 12, 13] has been done to show that the addition of small concentration of calcium chloride, 0.2 - 0.5 percent helps in the stabilization of organic soils with cement. The soils which generally give best results with calcium chloride belong to B horizon, according to pedological classification of soils.

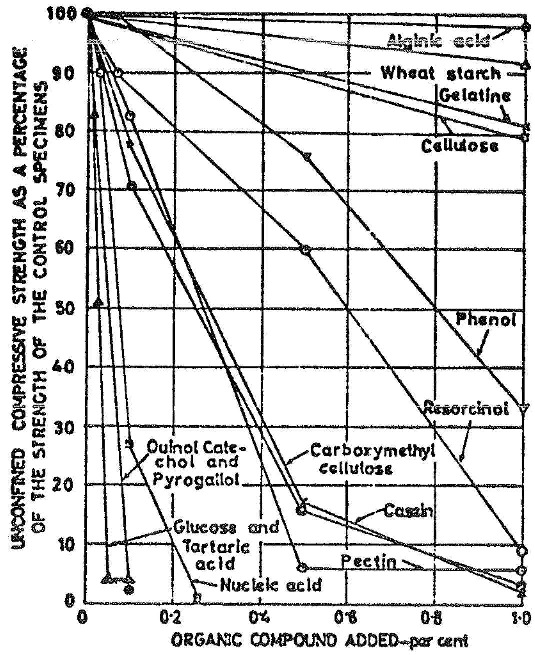


Fig.5(a) Effect of different proportions of organic compounds on the compressive strength at 7 days of an inorganic sand stabilized with 10 per cent of cement (Ref. 12)

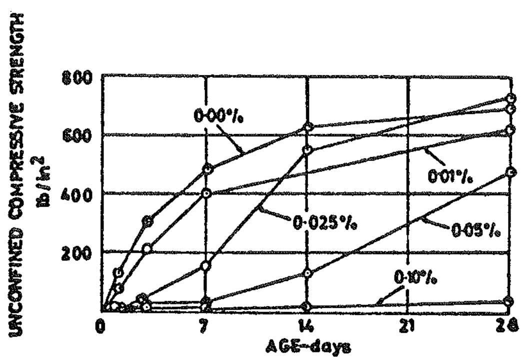


Fig.5(b) Strength/age relations for specimens of an inorganic sand, containing different proportions of glucose, stabilized with 10 per cent of cement (Ref. 12)

TABLE VII

Organic compounds that retard the setting of cement
(Ref. 14)

Retarding agent	Range of concentration % by weight of cement	Author	Reference
Oxidized cellulose	0.03 - 0.27	Ludwig	U.S.P.2,471,632/1949
Carboxymethylcellulose	0.05 - 0.75	Ludwig	U.S.P.2,427,683/1947
Hydroxyethylecellulose	0.05 - 0.50	Ludwig	U.S.P.2,427,683/1947
Oxidized corn starch	0.08 - 0.20	Andes and Ludwig	U.S.P.2,429,211/1949
Gums (acacia, arabic tragacanth, etc.)	0.15 - 0.90	Weiler	U.S.P.2,006,426/1935
Agar-agar	0.15 - 0.90	Weiler	U.S.P.2,006,426/1935
Dextrin, lactose, beet and invert sugars	0.50 - 1.00	Gresley	French Patent 391,711/1908
Tetrahydroxyadipic acid	0.10 - 0.20	Winkler	U.S.P.2,174,051/1939
Magnesium saccharate	-	-	U.S.P.1,637,321/1927
Tartaric acid	0.10 - 0.25	Brown	U.S.P.2,374,581/1945
Maleic acid	0.04 - 0.40	Ludwig	U.S.P.2,470,505/1949
Lignin-sulphonic acids, lignins and tannins	0.15 - 0.75	Cannon and Foster	U.S.P.2,188,767/194[illegible]
Casein	0.20 - 0.40	Gruewald Durbin and Tillie	U.S.P.2,290,956/1942

+U.S.P. = United States Patent

5. An attempt was also made to use lime in place of calcium chloride to accelerate the setting of Portland Cement, but it was observed that calcium chloride was more effective than ordinary lime, 0.2 percent calcuim chloride produced the same strengths, as produced by about 2 percent lime. The strengths obtained with the same quantity of calcium chloride is usually about twice as much as that obtained with lime. The maximum increase in strength in most of the cases is by adding 0.5 percent calcium chloride, after which further increase in the concentration of calcium chloride helps little in the development of strength. Figure[15] 5 (e) shows the relative effect of calcium chloride and calcium hydroxide on the strength of a cement treated organic sand. It is now established that the effect of calcium chloride on soil-cement mixes is more due to the accelerating effect of hydration of cement, rather than its neutralising effect on organic matter, as is the case with lime. During the reaction of lime with organic matter, certain calcium compounds are liberated, which get precipitated around cement grains, thus blocking the hydration of cement further. In addition it is observed[16] that the degree of hydration of cement in a saturated solution of lime is very much less than in water. If therefore excess lime is present in soil cement mixes, it will act as a retarder for the hydration of cement. This may explain to a certain degree the greater effectiveness of calcium chloride, as compared to lime in stabilizing organic soils with cement.

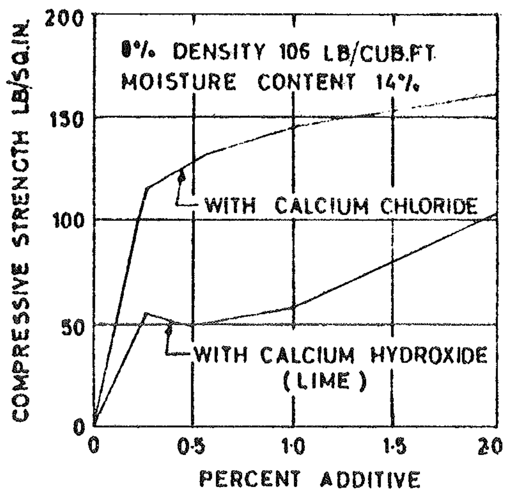


Fig.5(c) EFFECT OF THE ADDITION OF CALCIUM COMPOUNDS ON THE 7-DAY COMPRESSIVE STRENGTH OF CEMENT-TREATED SAND CONTAINING DELETERIOUS ORGANIC MATTER. (Ref. 15)

6. As organic matter has a definite deleterious effect on the hydration of cement, it may be essential to refer to some methods of organic matter determination, and the commonly employed[17] methods are, loss on ignition, dichromate oxidation, hydrogen peroxide oxidation etc. In addition the Portland[18] Cement Association, developed a calcium absorption method, for quick identification of sandy soils, containing deleterious organic matter likely to affect the setting of cement.

7. It is now possible to obtain special cements, such as 417 Portland Cement, for the stabilization of organic soils. The cement contains about 1.0 percent calcium chloride which helps in accelerating the hydration of cement in soils contaminated with organic matter.

2. Soils Contaminated with Sulphates

1. Sulphates are usually present in a number of soils in tropics and the most common form in which they exist in nature are as calcium sulphates. The presence of sodium and magnesium sulphate in subsoil in many tropical countries is also noticed. The effect of these salts on cement soil mixtures is important, as some of these salts tend to loosen the soil structure when they come in contact with moisture. It has been observed[19] that sulphates are usually present in higher concentrations in clay soils as compared to sands and sandy soils, due to the fact that sandy soils get easily leached, by the percolating rain water. The detrimental effect of sulphates on soil cement mass is due to the tendency of sulphates to absorb moisture from wet soil and get crystallized

with a large number of water molecules. The process of crystallization brings in very large volume changes and high pressures within the soil cement mass. The reverse process starts when the sulphates lose moisture during the hot dry season. In addition the sulphates react with calcium aluminate of cement and this reaction is also accompanied by high volume changes. Such high volume changes, leave the soil-cement loose and weak. It is for this reason that soil and water free from sulphates are used for soil-cement construction. It has been observed that sulphates as small as 0.2 percent can affect the soil-cement structure, reducing its strength to about 40-50 percent.

-2 The detrimental effect of magnesium sulphate on the soil-cement strengths is much more than that of calcium sulphate 119
and this is shown for one ~~[illegible]~~ particular soil in figure 6 (a) and (b).

VIII STRESS - STRAIN CHARACTERISTICS OF SOIL-CEMENT

1. In an engineering material, the modulus of elasticity and Poisson's Ratio are two important parameters for design work. In soil-cement mixes also these parameters are equally important to understand the behaviour of soil-cement under conditions of stress. The stress is usually caused by the application of external load, called force per unit area on a sample. The strain is the consequent change in the dimensions of a member as a result of stress, per unit length. Hooke's law of proportionality between stress and strain is true only on lower stress range, in most of the engineering materials.

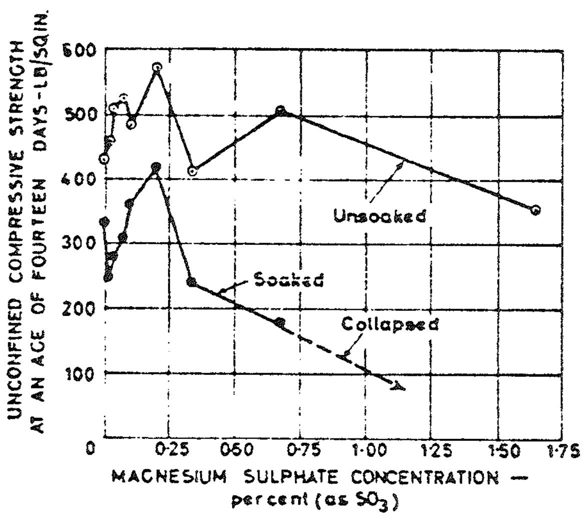


Fig. 6(a) EFFECT OF MAGNESIUM SULPHATE CONCENTRATION ON THE STRENGTH OF A CLAY STABILIZED WITH ORDINARY PORTLAND CEMENT. (Ref. 18)

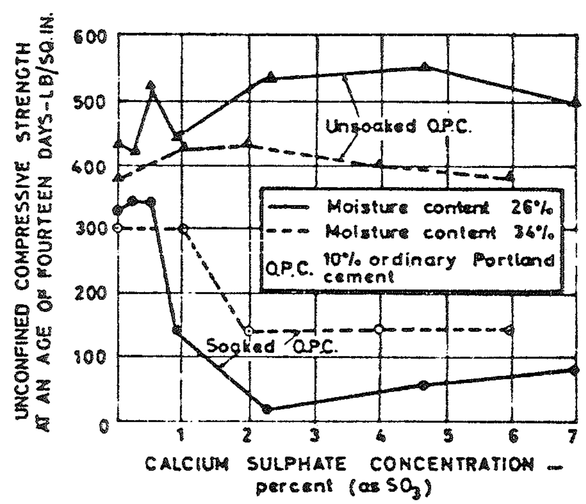


Fig. 6(b) EFFECT OF MOISTURE CONTENT AND CALCIUM SULPHATE CONCENTRATION ON THE STRENGTH OF A CLAY, STABILIZED WITH ORDINARY PORTLAND CEMENT. (Ref. 18)

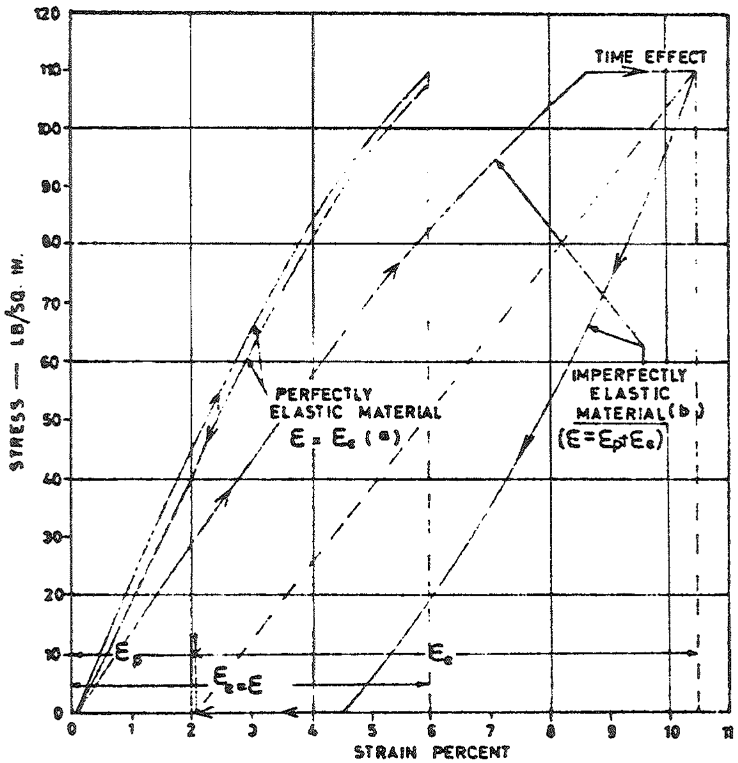


Fig. 7 TYPICAL BEHAVIOUR OF PERFECTLY AND IMPERFECTLY ELASTIC MATERIALS. (Ref. 19)

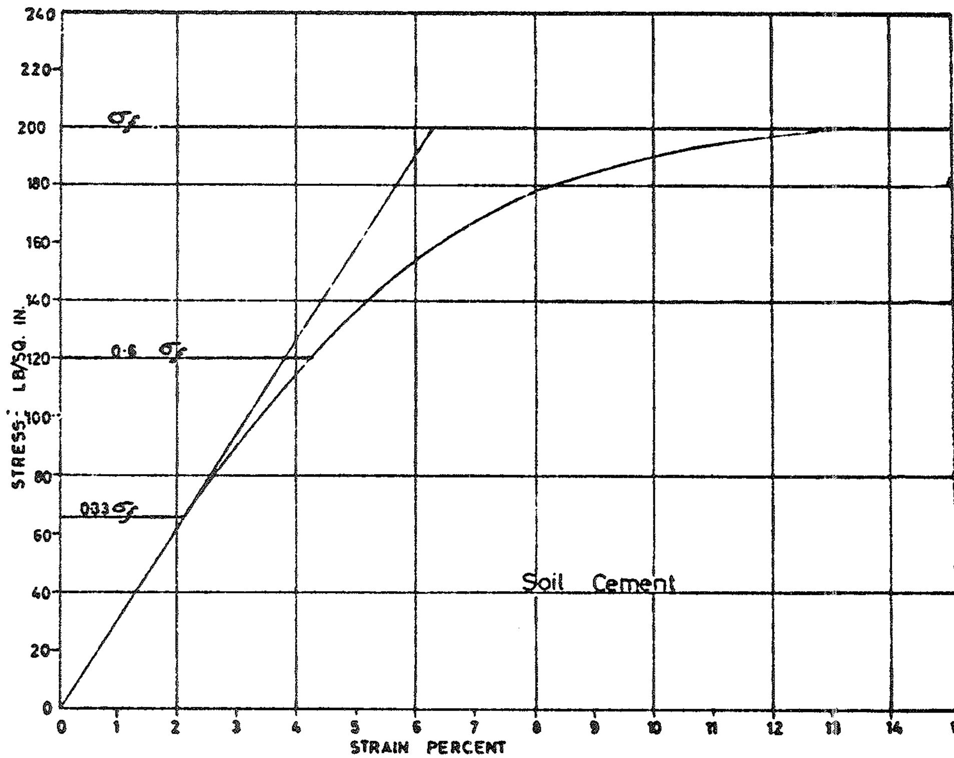


Fig. 8 CHARACTERISTIC STRESS STRAIN DAIGRAM OF A SOIL CEMENT MIX (Ref.19)

In addition, on a perfectly elastic material, the stress-strain relationship in this range is reversible. For semi-elastic materials, strains are not only a function of stress intensity but also the rate of loading its repetition, and the time for which the load is imposed on a material. Therefore there is difference between stress and strain curves of elastic and semi-elastic materials. The typical stress strain curves of two types of materials are shown in Figure 7. Curve (b) in figure 7 shows a typical semi-elastic material with a residual strain after the stress is removed and the loop strain in such a curve is known as hystersis.

2. In the light of the above basic properties, soil-
122 cement is to be considered as an engineering material with the point of view of assessing its structural values. The term with which an engineer is normally concerned in soil-cement mix is its degree of elasticity, which is defined as the elastic strain over the total strain. In terms of stress-strain relations the soil-cement normally possess the following general characteristics[20] as shown in figure 8.

(i) The stress strain curve is more or less concave with respect to strain axis.

(ii) The curve up to one third of the failure load, is almost a straight line. The plastic deformation in this range is extremely small and the material can be considered perfectly elastic.

(iii) When the stress increases to 0.6 of the failure load, the curvature starts increasing rapidly.

3. In a general sense it can be said that the elastic behaviour of soil-cement is a function of its strength. All the factors that influence strength also influence the elastic behaviour of soil-cement. Therefore the following can be accepted if the failure stress is high.

(i) The stress strain relation will be less curved.

(ii) The linear range of the stress-strain relation shall be higher.

(iii) The plastic deformation is small, and the degree of elasticity is higher.

4. From the above discussion it can be concluded, that any factors which help to give higher strength are apt to make the material more elastic, therefore high percentage of cement in soil and high densities do help to achieve greater elasticity. It has also been observed (20) that clay content too increases the elasticity of soil cement mixes, and the optimum quantity of clay to produce maximum elasticity is about 25 percent.

IX STRENGTH OF SOIL CEMENT MIXTURES

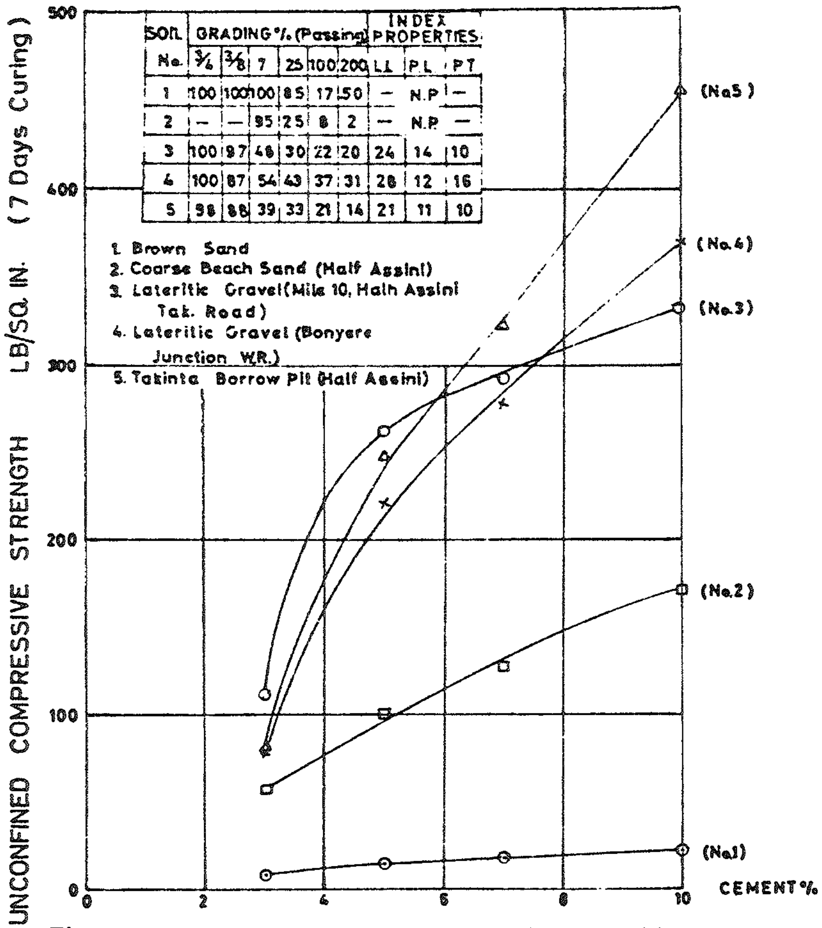
1. In order to use soil-cement as a material of construction, it is important to know the strength characteristics of the material. The strength of soil cement mixes is primarily required to resist weathering and also to with stand the imposed stresses without undergoing excessive deformations. An unconfined compressive strength test being a simple test has found popular acceptance for the assessment of strength characteristics of soil-cement mixtures.

C.B.R. test though popular for raw soils is used less frequently, than the compressive strength test. It may therefore be interesting to study in more detail the factors influencing the compressive strength of soil-cement.

2. Influence of Cement on Strength

It is well known that in a given soil, the increase in the concentration of cement is accompanied by an increase in the compressive strength. Figures (9) and (10) show the values of compressive strength and C.B.R. for few typical Ghanaian soils. It will be noted from the trend of these curves that in most of the soils an increase in cement content initially increases the compressive strength rapidly, and this increase gets less rapid at higher concentrations of cement.

3. The development of strength with the same concentration of cement for different soils is different and this is influenced by the grading and the mineral composition of the clay fraction in a soil. The results of an interesting study, carried out by Iowa State University (22) are represented in figure (11). It is seen in this figure that in a mechanically graded soil, it is possible to achieve sufficiently high strengths with as small a percentage of cement as 4-6. Such graded soils usually contain about 25 percent clay and 75 percent sands, and they attain higher densities with the same compactive effect as compared to other soils. As a result of this study it was noted that for clay content greater than 25 percent, the mixture containing montmorillonite clay gave higher strengths than the same concentration of cement with illite and kaolinite clay.



SOIL	GRADING % (Passing)						INDEX PROPERTIES		
No.	3/4	3/8	7	25	100	200	LL	PL	PT
1	100	100	100	85	17	50	—	N.P	—
2	—	—	95	25	8	2	—	N.P	—
3	100	97	48	30	22	20	24	14	10
4	100	87	54	43	37	31	28	12	16
5	98	88	39	33	21	14	21	11	10

Fig. 9 EFFECT OF CEMENT ON THE COMPRESSIVE STRENGTH OF SOME GHANAIAN SOILS.

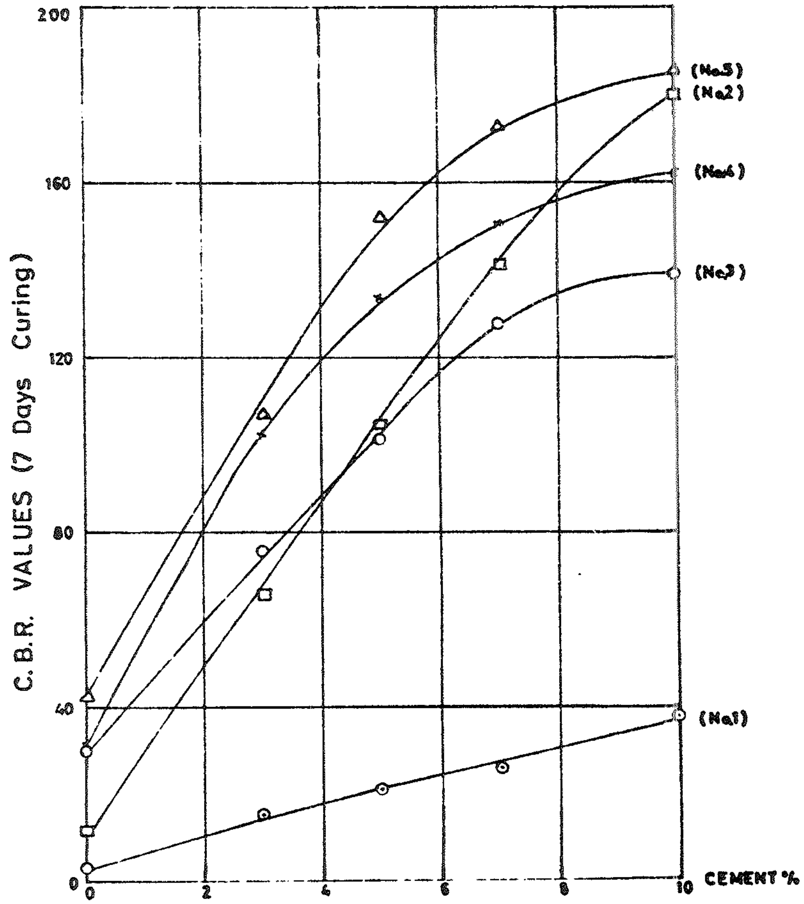


Fig 10 EFFECT OF CEMENT ON C.B.R. VALUES OF SOME GHANAIAN SOILS.

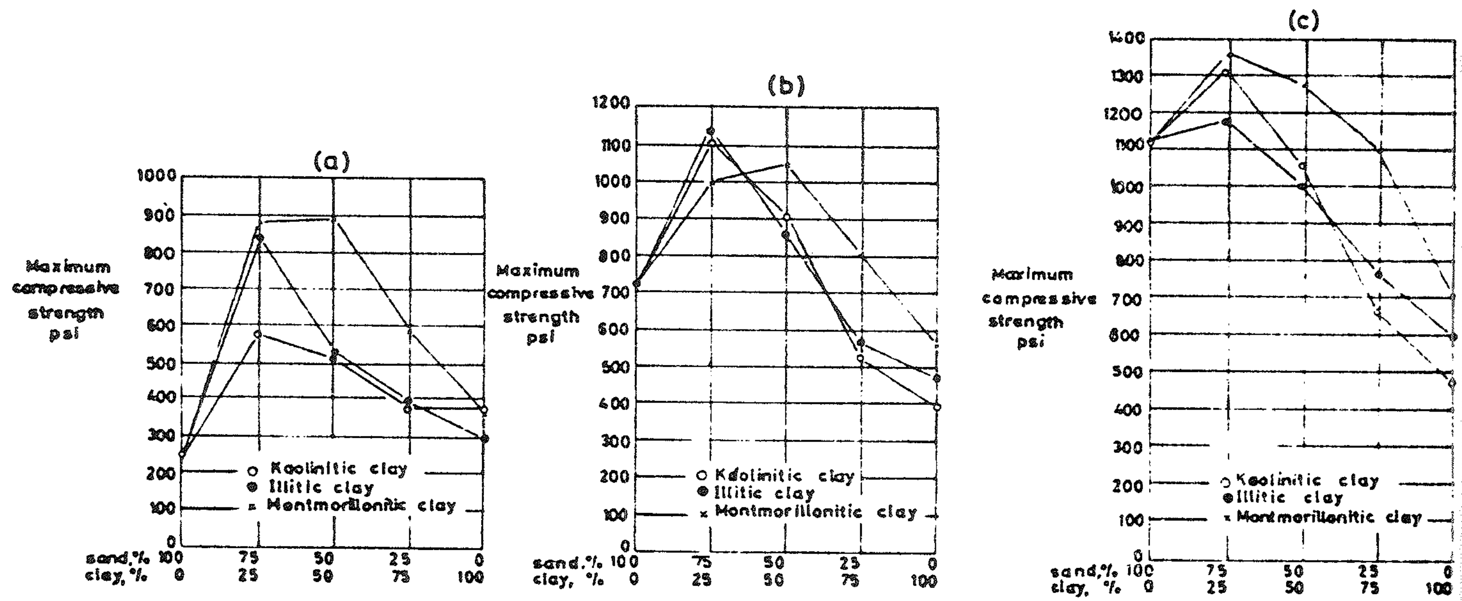
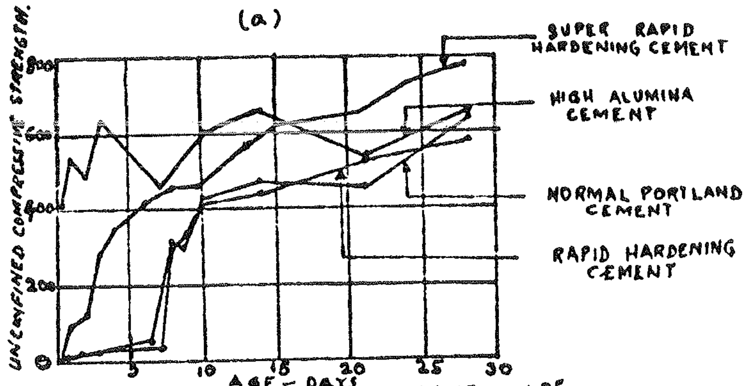
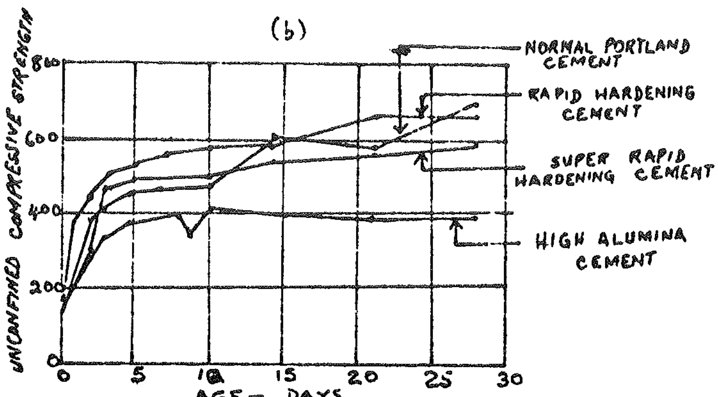


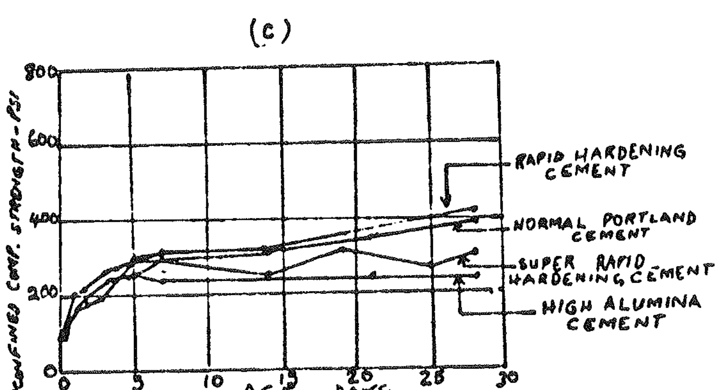
Fig. 11 Maximum strengths of sand-clay mixtures compacted with the standard AASHO compactive effort and treated with (a) 8 percent cement (b) 12 percent cement (c) 16 percent cement (Ref. 21)



COMPRESSIVE STRENGTH/AGE RELATIONSHIPS FOR SPECIMENS OF SAND STABILIZED WITH FOUR DIFFERENT TYPES OF CEMENT



COMPRESSIVE STRENGTH/AGE RELATIONSHIPS FOR SPECIMENS OF SILTY CLAY STABILIZED WITH FOUR DIFFERENT TYPES OF CEMENT



COMPRESSIVE STRENGTH/AGE RELATIONSHIPS FOR SPECIMENS OF CLAY STABILIZED WITH FOUR DIFFERENT TYPES OF CEMENT

FIG. 12

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This can be explained due to the greater surface activity of montmorillonite, which is likely to produce more cementitious compounds with cement, as compared to less active minerals.

4. Effect of Age on the Development of Strength

The Road Research Laboratory (23) in the U.K. did some useful work to find out the increase in compressive strength of soil cement with age. Three soils, a neat sand, a silty clay and a clay sample were tested with four different types of cements, namely Portland, High Alumina, Rapid Hardening and Super Hardening Cements, and the strengths were studied up to 28 days. The specimens were cured at a constant moisture and temperature of 25°C by waxing and placing them in an oven. For sand and silty clay the cement used was 10 percent, whereas for clay it was 15 percent. The curves as obtained in this study are shown in figure 12. It would be seen that although sand has initially lower strength than silty clay and clay, the sand strengths progressively increases after 28 days curing.

5. A soil having no organic matter would normally go on increasing in strength with age. The increase in strength in the first one week is considerable, but the strength goes on increasing up to 28 days at a steady rate, after which the increase is very small. Normally for design work, 7 days strength is made the criterion for evaluating quality of soil-cement mixes, but the increase in strength with age can also be made use of in the economical design of pavements. It is for this reason that the development of soil-cement strengths over

long periods is important to study. The results of two investigations[24, 4], giving the relative strength of soil-cement for a period of one year and five years are shown in figures (13) and (14) respectively.

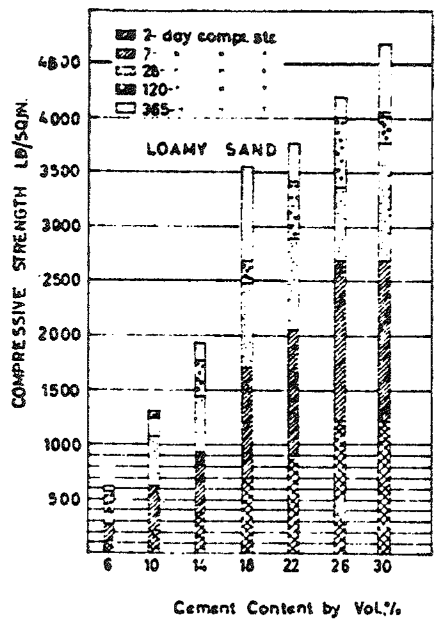
6. <u>Effect of Curing Temperatures</u>

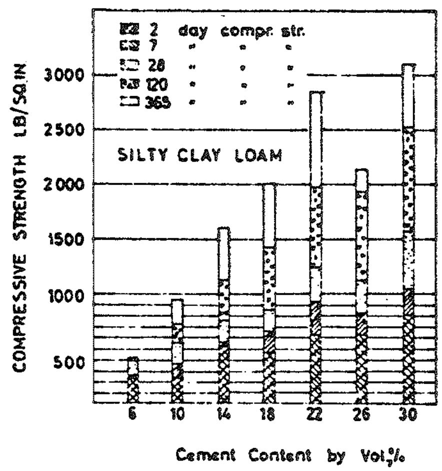
1. The effect of curing temperatures on the development of strength in concrete is fairly well known. It has been shown[25, 26, 27] that an increase in the curing temperature of concrete from 10° C to 30°C, increases the compressive strength by about 800 - 1000 psi. The studies on soil-cement also reveal that curing temperatures have very pronounced effect on the strength of soil cement mixes, and this is shown in figure 15. This figure gives the effect of curing temperatures between 0 - 60°C, for three types of soils, sand, silty clay and a clay. 129

2. The results of a rather short study carried out in Ghana[28] on the development of strength with age, under tropical temperatures, both for laboratory and the field conditions are shown in figure 16. The study gives fairly good relations for the strength of soil-cement pertaining to different curing periods and at temperatures typical of the West Africa Coast. In addition it gives a relation between the field and laboratory strengths for similar mixes, when mix in place method is employed for construction.

3. The following general remarks can be made on the effect of curing temperatures in soil-cement.

SOIL TYPE	GRADATION — %			PHYSICAL CONSTANTS		
	Sand	Silt	Clay	L.L.	P.L.	S.L.
Loamy Sand	83	6	11	14	N.P.	20
Silty Clay Loam	9	67	24	34	14	16
Clay	17	36	47	37	18	16





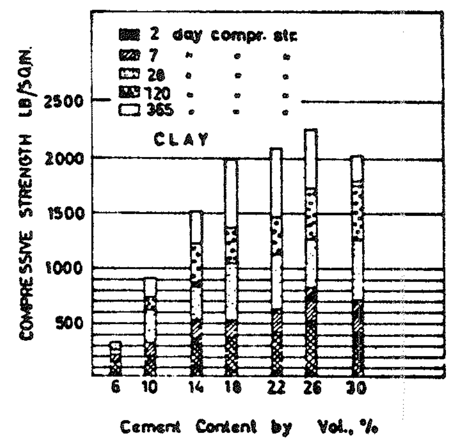


Fig 13 EFFECT OF CEMENT CONTENT AND AGE ON THE COMPRESSIVE STRENGTH OF SOIL CEMENT MIXES. (Ref. 23)

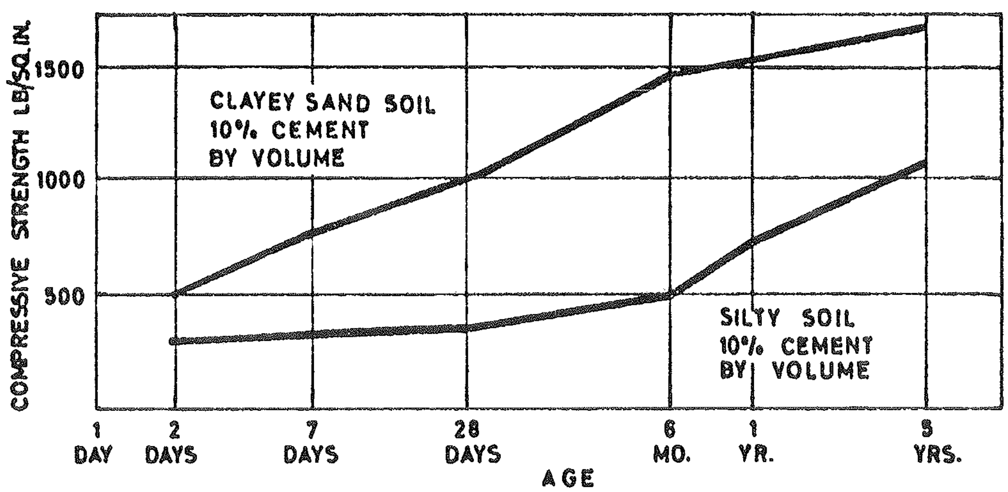
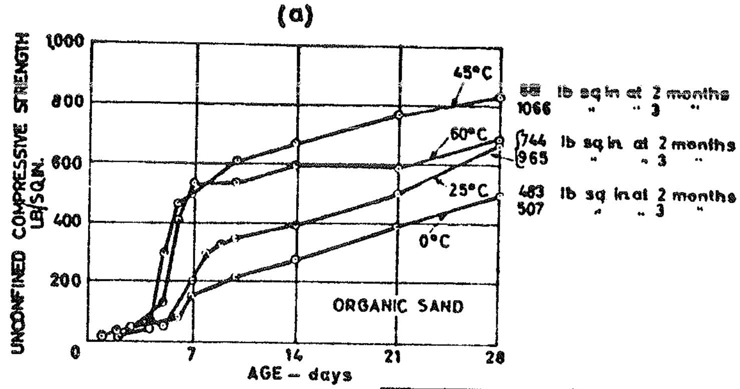


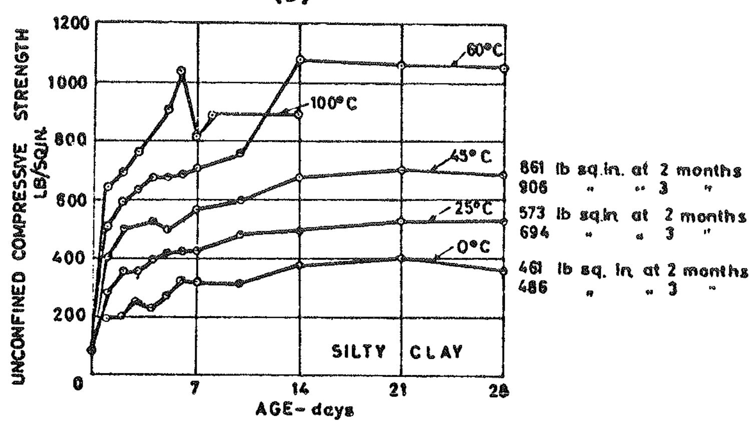
Fig.14 COMPRESSIVE STRENGTHS OF LABORATORY-MOLDED SPECIMENS OF SOIL-CEMENT (Ref.24)

(a)



Soil Type	Cement Content %	Moisture Content %	Dry Soil Cement lb.cu.ft.
Organic Sand	10	12	112
Silty Clay	10	15	106
Clay	10	24	91

(b)



(c)

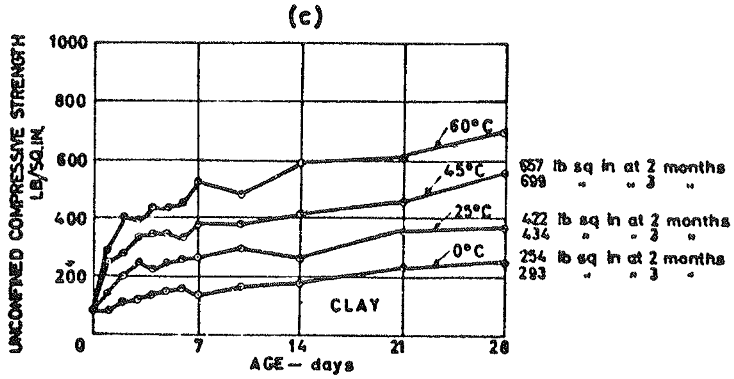


Fig.15 THE RELATIONSHIPS BETWEEN UNCONFINED COMPRESSIVE STRENGTH AND AGE AT DIFFERENT TEMPERATURES FOR SPECIMENS OF DIFFERENT SOILS STABILIZED WITH 10 PERCENT OF ORDINARY PORTLAND CEMENT (Ref. 27)

(i) If a design is based on the compressive strength of soil-cement mix, less cement would be required for development of a given strength under tropical conditions than in a temperate climate.

(ii) In tropics for temperatures between 85-95°F, the 7 days compressive strength in soil-cement is about 0.80-0.85 of the 28 days compressive strength. This is found to be true both for the laboratory and field strengths on the basis of a limited data available in Ghana.

(iii) Soil-cement gets hard with time at all temperatures except below 0°C.

(iv) The strength of soil-cement at 7 days curing changes by 2.0 to 2.5 percent for each 0°C rise in curing temperature, in the temperature range of 25-50°C.

(v) With clays and clayey soils, the strength obtained after 7 days curing at 25°C could be developed within 24 hours, if the temperature of curing was raised to 45°C. If the temperature was lowered to 0°C, then the same strength would be developed after 28 days.

(vi) It was observed in U.K., that soil-cement road base constructed in spring would have a strength 50-100 percent higher after three months, than the base constructed in autumn with the same mix.

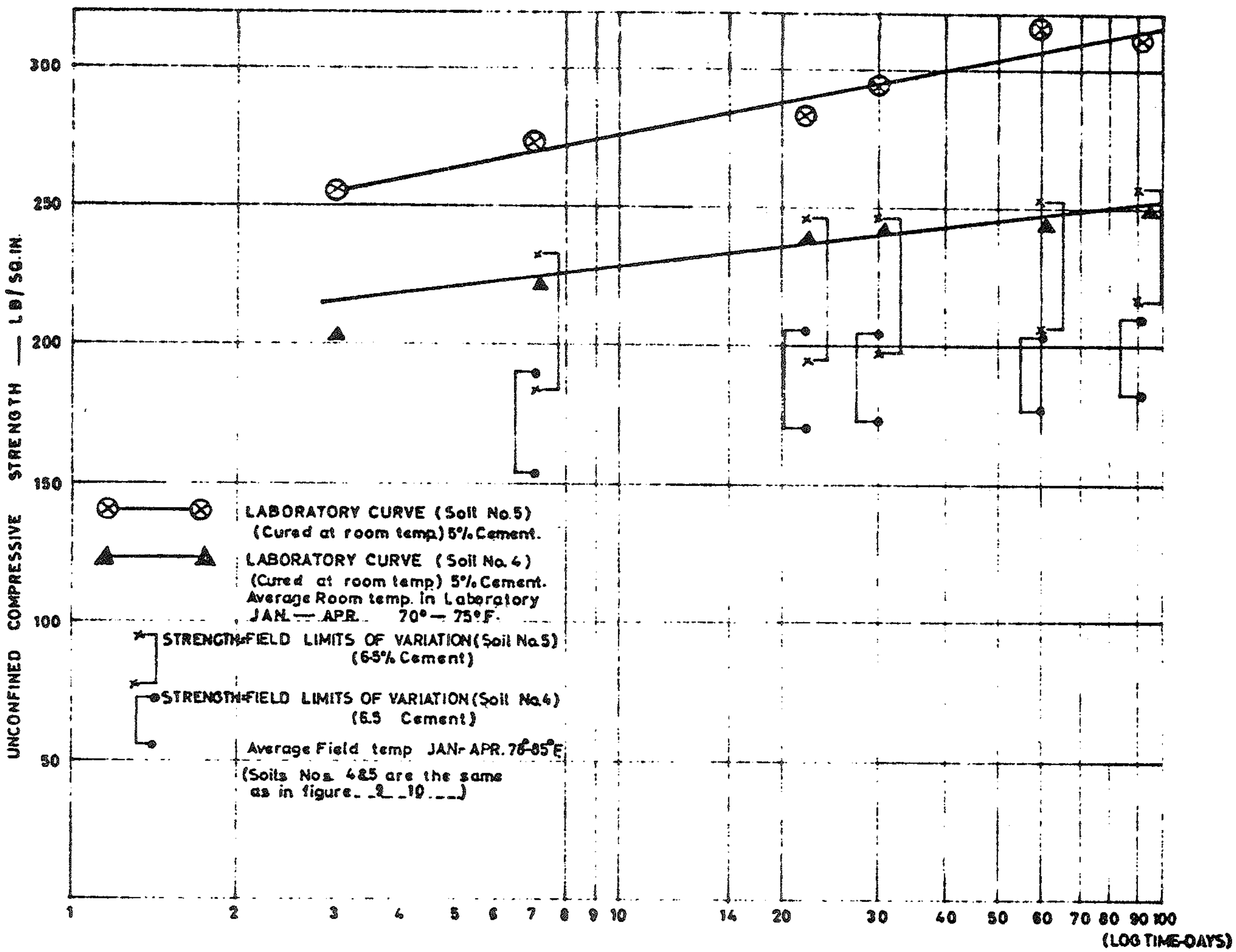


Fig. 16 EFFECT OF AGE ON THE DEVELOPMENT OF STRENGTH OF TWO GHANAIAN SOILS IN THE LABORATORY AND THE FIELD (HALF ASSINI TOWN ROADS PROJECT).

X DESIGN OF ROAD AND AIRFIELD PAVEMENTS

1. Before design aspects of soil-cement pavements are discussed, it is important to know the basic requirements of a good road material, which are as follows:-

(i) The road material should have adequate strength to withstand stresses imposed by the traffic; both under dry and wet conditions.

(ii) The material should have good weathering qualities.

(iii) The plastic deformations in the material, due to the repetitive load of traffic should not be excessive.

2. Any material which could be accepted for road works should fulfil the above referred conditions, ~~[illegible]~~, in addition to its cost being low. The question of providing economical road pavements has attracted the attention of Road Engineers at all times. There are generally two choices for road construction:-

(i) To construct a road pavement employing conventional methods, using hard broken stone (Macadam or Telford Construction) or Cement concrete.

(ii) or in places where stone is not available or available at high cost, the use of local materials such as soil, gravel, industrial wastes, such as fly ash, cinder etc., be made, with or without binders.

3. In using locally available materials. the question of studying the stability of these materials under worst moisture conditions, assumes a very great importance. It is for this reason that most of these locally available materials need some sort of binders to give them stability under different moisture conditions. This monograph only deals with the use of cement as a binder and discusses the limitations of soil cement as a road material.

4. There are two categories of conventional road pavements, namely the flexible and the rigid type: The former type has no resistance to bending, whereas the later has flexural strength and resistance to bending. A flexible pavement normally undergoes deformation with the deformation of the subgrade, whereas rigid pavement on account of its flexural strength does not get deformed, even if the subgrade at places loses contact with the pavement.

5. In the light of the above facts, it is important to study whether for design purposes, soil-cement should be treated as a flexible or a rigid pavement. Commenting on the flexural strength of soil-cement the Road Research Laboratory (9), in the U.K., suggest that soil-cement with an unconfined compressive strength of 250 psi, has a flexural strength of about 50 psi. It may however be interesting to work out the comparative thickness of pavements under similar conditions of loading, treating soil cement as a rigid and a flexible material.

6. Soil Cement As a Rigid Material

In considering soil-cement as a rigid material, the

Westergarrd's method of pavement design is used. Assumming the modulus of subgrade reaction 'K' as 100 psi/in, a material with a flexural strength of 50 psi, would need a pavement thickness of 24 inches for wheel loads of 9000 Lbs and with a contact pressure of 80 psi. The design thickness for a concrete pavement, with a flexural strength of 500 psi for similar loads would be only 6 inches. It has however been observed[29] in U.K. that soil-cement pavements with thickness of 9-12 inches, have given satisfactory service under a wheel load of 9000 lbs. and contact pressure of 80 psi. It can therefore be concluded that for economical design of soil-cement pavements, the rigid pavement design methods safeguarding flexural failure can give very conservative results.

7. Soil Cement As a Flexible Material 137

As discussed earlier soil-cement has a certain amount of flexural strength, whereas a flexible material has none. A very important effect of flexural strength is the greater load dispersing qualities of a material. This means that on application of a certain load on a pavement, the stresses transmitted to the subgrade by the same thickness of flexible and rigid pavement are widely different. This is one reason a concrete pavement needs a much less thickness as compared to flexible pavement.

If soil-cement is treated purely as a flexible material then the flexural strength of soil-cement is ignored, meaning that the thickness provided for the pavement is higher than what is actually needed.

It may therefore be said that to treat soil-cement as a completely flexible material, would be to overdesign the pavement. Soil-cement though very near to being flexible material shares some characteristics of a rigid material, and this should not be ignored in an economical design.

8. The difference between a completely flexible material and semi flexible material like soil-cement, is judged from the ratios of modulus of elasticity of their base and subgrade. This ratio may be about 5, for crushed rock and the subgrade and 20 or more in the case of soil-cement, having a compressive strength of 250-400 psi. The effect of higher modulus of elasticity ratio of base and subgrade will tend to decrease the shear stresses in the subgrade and therefore need a lesser base thickness.

9. A number of methods normally used for the design of flexible pavements are currently employed for the design of soil-cement pavements, either as such or with slight modifications.[32] In United States a Committee of the Highway Research Board on flexible pavement design, suggested that soil-cement displays all the characteristics of a flexible material or in certain cases semi-flexible material, and therefore the design method for soil-cement should be limited to the selection of a proper mix design only. In addition the thickness of pavement may arbitrarily be selected from experience on various types of subgrades. The recommended Highway Research Board thickness of pavement for various types of subgrades are shown in table VIII.

TABLE VIII

RECOMMENDED THICKNESSES OF SOIL-CEMENT BASE COURSE FOR
18,000-LB AXLE LOADS AND CORRESPONDING RECOMMENDED
THICKNESSES OF GRANULAR-TYPE STABILIZED BASE COURSES
(Ref. 32)

Subgrade Soil Group Classif.	Soil-Cement Base Course Thickness(in.)[a]	Granular-Type Stabilized Base Course Thickness(in.)[a]
A-1-a	0	0
A-1-b	5	5
A-3	5	5
A-2-4	5	5
A-2-5	5	6
A-2-6	5	6
A-2-7	5	6
A-4	6	8
A-5	6	8
A-6	6	8
A-7	6	8

[a] The soil-cement thicknesses were recommended at that time for highway pavements with average traffic that did not exceed 100 trucks, which range from 4,000-lb gross load to 18,000-lb axle load, per day or a total of 1,000 vehicles per day including the aforementioned truck traffic. Base courses of the thicknesses given in Table VIII may have to be supplemented by subbase when traffic exceeds the loadings for which the recommended base courses are intended.

10. In California, the design thickness of flexible and semi-flexible pavements are worked out by a method developed by Hveem and Sherman. The method takes into consideration what is called resistance values 'R', of soil and granular layers, and the cohesive resistance of all structural elements, such as soil-cement base and the asphalt surface. The resistance and cohesive resistance of various elements is measured by means of Hveem's Stabilometer. The effect of traffic, in terms of wheel load and repetition of loading is integrated into a single factor called traffic index. The method seems to be rational and takes care of the rigidity of the material. It is hoped that more data will be available, in the near future to prove the suitability of this method for the most economical design of semi flexible pavements.

11. In Britain the Road Research Laboratory, on the basis of a fairly extensive study in the field on experimental lengths has suggested, that for a soil cement mix having a compressive strength of 250 psi and above, the pavement thickness can safely be curtailed to 70 percent of the thickness obtained by C.B.R. method of flexible pavement design. However a mix having compressive strength less than 250 psi is treated as a completely flexible material, and for this mix the thickness as obtained from a flexible pavement design method is used without reduction. Usually C.B.R. method is very extensively employed for the design of soil-cement pavement, with necessary modifications, and therefore standard C.B.R. curves for road and airfield pavement designs are included in the appendix, for ready reference.

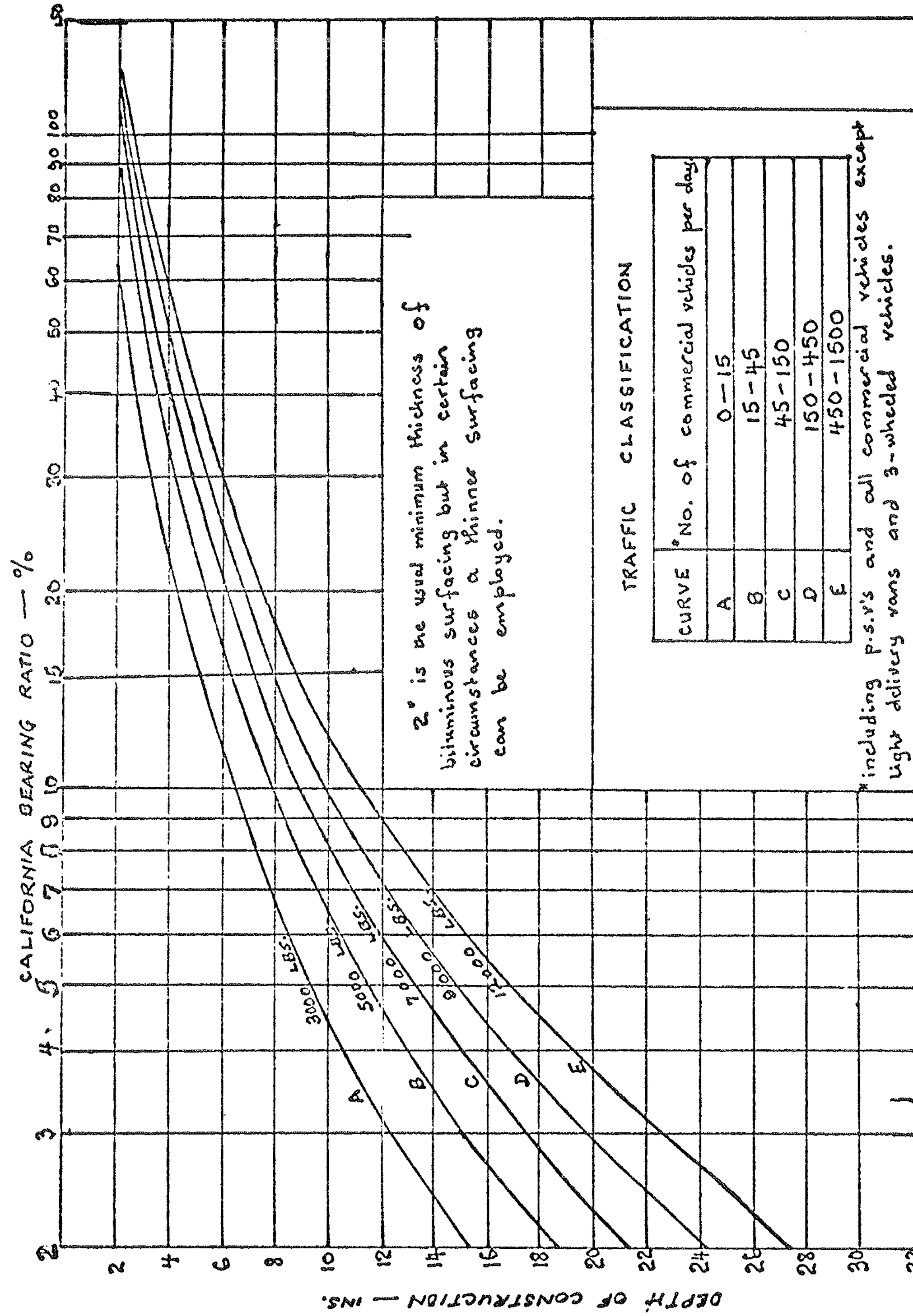


FIG. 23. C.B.R. DESIGN CURVES FOR DIFFERENT CLASSES OF ROAD TRAFFIC. (R. R. L., U.K.)

XI CRITERION FOR MIX DESIGN

1. Strength Requirements for Road Pavement

In addition to the total thickness of a pavement, to provide an adequate cover on the subgrade, it may be important to consider the quality of pavement material in terms of its strength, so that it is capable of withstanding stresses (9) imposed by the traffic. To give an idea of how strong a soil cement pavement material should be the case of a pavement 8 inches thick, composed of 6 inches base and 2 inches bitumen surface, under a $5\frac{1}{2}$ tons pneumatic wheel with contact pressure of 80 psi, is considered. For pavement materials having modulus of elasticity ratio of base to subgrade as 10, the calculations according to elastic theory suggest, that the maximum stresses in the base at any point shall not exceed 50 percent of the stresses on the surface of the pavement. This indicates that for a wheel load under discussion, the base will not be subjected to a shear stress of more than 40 psi. Giving an allowance for the impact, and repetitive action of traffic, it could be safely assumed that the pavement base in soil-cement for such a load should possess a minimum shear strength of 80 psi. For lighter traffic the shear strength requirements of a base are much lower than 80 psi.

2. Strength Requirements For Airfield Pavement

(29) The field studies carried out in the United Kingdom on the performance of some airfield pavements suggest that for an aircraft wheel, with a contact pressure of 100 psi, the base and subbase field compressive strength in soil-cement should be

a minimum of 250 and 125 psi respectively, under 4 inches thick bitumen carpet. For tire pressures higher than 100 psi, a minimum field compressive strength of 450 psi is recommended, under 6 inch bitumen carpet.

In the case of temporary airfields a minimum compressive strength of 200 psi, is considered a suitable criterion, under prefabricated bitumen surfacing.

3. Limitations of Unconfined Compressive Strength Test

The use of unconfined compressive strength test as a criterion for the design of soil-cement mixes is quite justified in clays and clay soils, but in the case of granular soils this can be misleading and results in uneconomical designs. A material having only cohesion and no internal friction will not show much increase in strength when tested under confined conditions, but the increase is considerable in materials having internal friction. It may therefore be quite uneconomical for sands and sandy soils to use the same criterion of compressive strength of 250 psi, as is recommended for clays.

The Road Research Laboratory in U.K., studied[9] the effect of confinement on the strength of various types of soils. The study was carried out on four types of soils, sand, silty sand, sandy clay and heavy clays, by checking their unconfined compressive strength and C.B.R. values under similar conditions after seven days curing. The results are illustrated in Figure 17. It would be seen in the figure that for clays and sand samples a compressive strength of 100 psi, meant C.B.R.

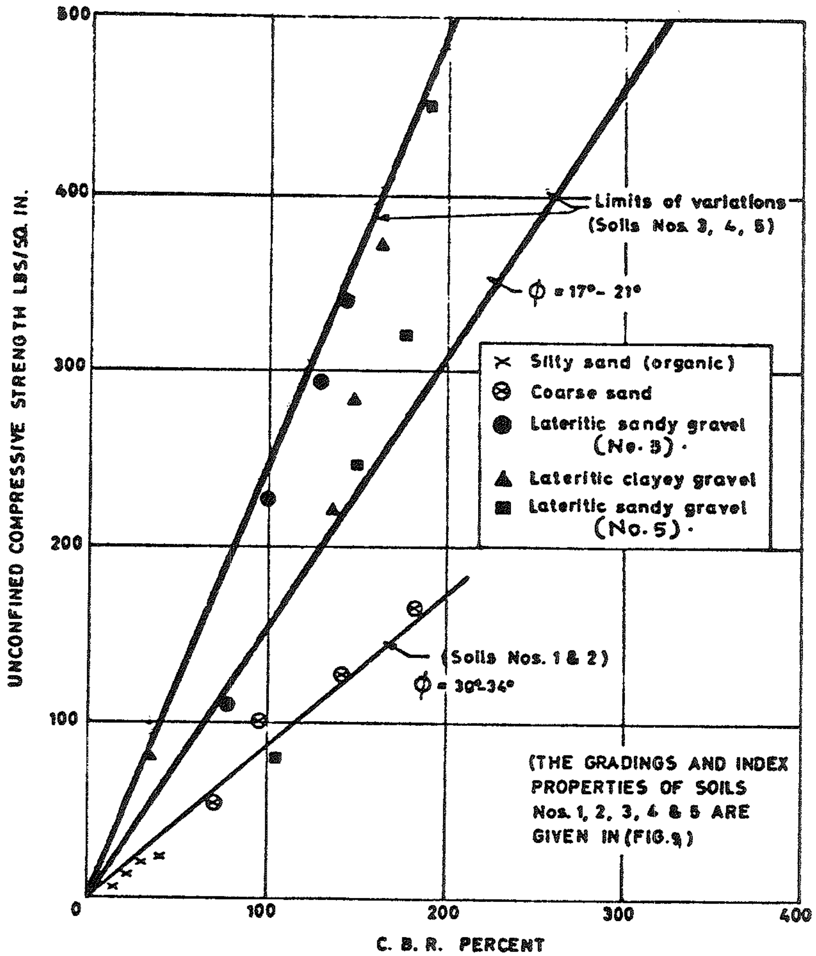


Fig.18 RELATION BETWEEN C. B. R. AND COMPRESSIVE STRENGTHS FOR SOME GHANAIAN SOILS.

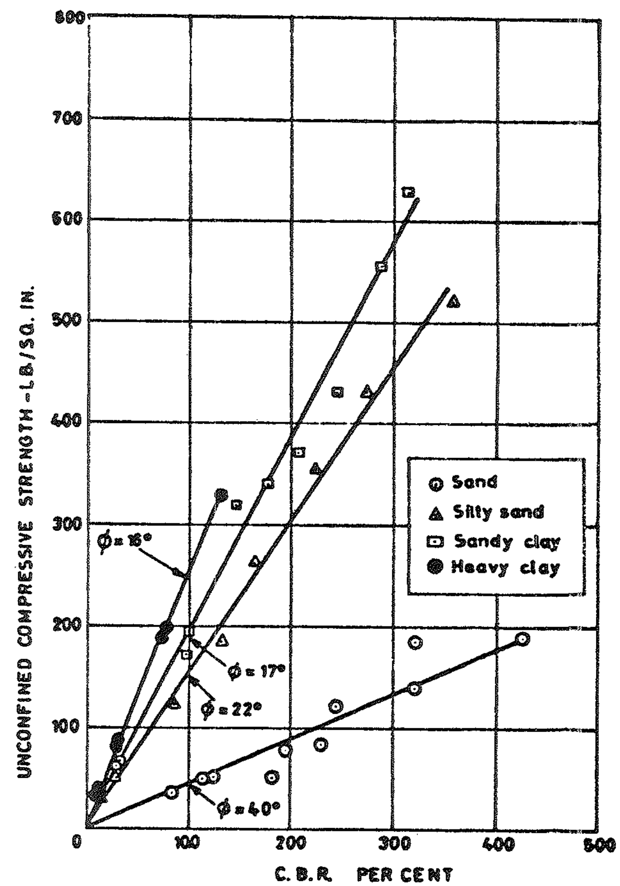


FIG.17 RELATION BETWEEN C. B. R. AND UNCONFINED COMPRESSIVE STRENGTH FOR SOIL-CEMENT MIXTURES. (Ref. 9)

values of 30 and 215 percent respectively. This leads to the conclusion that to aim at the same value of compressive strength for all types of soils is very unrealistic and the unconfined compressive strength can be a useful criterion, for the mix design in clays and clay soils alone. This test can not form a reasonable basis for studying the strengths of sands and sandy soils. The lateral confinement has a marked effect on the axial failure load of a material, which has a high angle of internal friction and therefore sands and sandy soils are capable of mobilizing much higher strengths under confined conditions than given by unconfined compressive strength test. The results of a similar study carried out on Ghanaian soils are shown in figure 18.

4. California Bearing Ratio Test

The California Bearing Ratio test or what is known as Cylinder Penetration Ratio test in Britain is a simple test to study the strength of a subgrade. The test is a measure of load required, to cause a cylindrical plunger, 2 inch diameter to penetrate a specimen of soil at the rate of 0.25 inch per minute. The C.B.R. or cylinder penetration ratio is calculated by expressing the load on the plunger for a penetration of 0.1 inch, as a percentage of standard load of 3000 lbs.

The original C.B.R. method of pavement design is silent on the quality of base required for road and airfield pavement. It only gives the total thickness of the sub base, base, asphalt carpet, required over a sub-grade of a certain strength. This is due to the fact that originally hard broken stone was the only material used for road construction and stone had little or no chance of failure under stresses imposed on the pavement. With the advancement of science of road construction and the use of a number of locally available materials for road construction, it became important to study the strength of a base material before it could be employed for construction purposes. The C.B.R. method was therefore subsequently employed to rate the quality of base and sub-base material.

The work carried out in Britain (29) and the United States (30) suggests that a laboratory C.B.R. value of 120 percent is a reasonable criterion for accepting soil-cement as a base material for road pavement. The field studies further indicate that with mix-inplace methods, the laboratory C.B.R. value of 120 percent would approximately give a C.B.R. value of 80 percent in the field.

In case of temporary airfields, likely to take aircrafts with wheel contact pressures up to 250 psi, a laboratory C.B.R. value of 140-150 percent has proved satisfactory. For permanent airfields which are likely to take aircrafts with wheel contact pressures of more than 100 psi, a laboratory C.B.R. value of 230-250 percent is considered adequate. Such a base should in addition be covered with 4-6 inches of asphaltic concrete or any other type of bitumen carpet.

For sub-base materials in an airfield pavement, the C.B.R. values should normally be higher than the C.B.R. values of subgrade, but should be less than the base material.

The thickness of various layers of the pavement can be worked out from the standard C.B.R. curves, on the basis of C.B.R. values of various layers of the pavement.

XII SOIL CEMENT CONSTRUCTION

1. The methods of soil cement construction can be divided into two distinct groups:-

(i) Mix-in-place Methods
(ii) Plant mix methods. (Travelling Plant and Stationary plant).

In recent years, a lot of plant and equipment is developed for the stabilization of soil with cement. The merits of each type of plant and the conditions under which each plant is most useful, are given by the manufacturers. A good deal of information[34, 35, 36] is also available in various pamphlets, papers and Civil Engineering Construction Hand Books, which give the relative merits and demerits of various plants, on the basis of practical experience.

2. The insitu mixing methods normally employ some form of light rotary tilling machines to till the soil in-situ. The soil is pulverized with the rotary machine itself. A layer of cement corresponding to a known percentage of cement is spread over the loosened soil and mixed by means of a rotary machine. At this stage water is added and mixed with soil-cement to bring up the moisture of the mix near the optimum moisture. After levelling the layer of soil-cement, it is rolled to bring it to its maximum density. The use of this type of methods limits the thickness of a compacted layer to about 4-6 inches, at one single time.

3. The types of plant used in the mix-in-place method are P & H single-pass stabilizer 10ft. wide, which cuts the soil, pulverizes and mixes it with the stabilizer in a single pass. The stabilizer is spread in front of the machine. The machine is driven by 250 H.P. power unit

and can give outputs as high as 7,000 - 8,000 sq. yards per day. The seaman pulvimixer or rotavators are about 5-6 feet, wide and have high speed rotors driven by a separate power or in the case of a rotavator, through a power take off from a tractor. These machines are very effective for fine grained soils, but have limited application on gravelly and hard lateritic soils.

4. In the plant mix methods, the soil is excavated either along the alignment of a road or from barrow areas, and mixed with the appropriate quantity of cement and water. The soil cement mix from the plant is laid on the site to a uniform thickness and compacted by rollers.

5. The travelling plants usually employed are, the Barber-Greene mixer, the Gardner Type mixer, the H & B Motopaver etc. In the case of Barber-Greene mixer the excavated soil and stabilizer are lifted by means of bucket elevators and stored in a hopper. The material from the hopper is passed into a pug-mill and water added. The pulverized soil-cement mixture is then spread on the road, levelled by a grader and compacted with rollers. In the Gardner type mixer the windrowed soil is put in a pug mill to pulverize the material. The stabilizer and water are added and the mixture depoisted back in a windrow. The mixture is spread and levelled by a grader and compacted with rollers.

6. In Stationary Plants, there are two types of machines, the continous mixers and the batch mixers. In the continous mixers, as in travelling plants, an elevator bucket lifts the soil to a hooper, and this is discharged into a pug mill by a belt conveyor. The stabilizer and water is added at this stage and the material mixed thoroughly. The mixed material is collected in trucks and laid on the site levelled and rolled.

The Batch mixers are nothing else but concrete mixers. These mixers are normally employed on small jobs. The soil, stabilizer and water are added in the concrete mixer and mixed till a uniform mixture is obtained. The best results in pulverization are obtained in double-paddle mixers, pug mills, etc. The mixed material is carried in trucks to the site, spread and rolled as discussed earlier.

7 The choice of one or the other method of construction is dictated more by the availability of plant and site conditions than anything else. The stationary mixing plant with a fleet of trucks for transporting soil, needs a far greater capital investment on a project, as compared to the in-situ methods of construction. On the other hand the in-situ methods are useful and economical only where a single layer of soil cement is to be laid. In places where a layered system of construction is adopted stationary plant method is more convenient and economical. In the in-situ methods there is a limit up to which a plant can excavate and pulverize a soil and therefore if a layered construction is required, the top layers after excavation need stock piling. The construction is then carried out by laying loose soil layers 8-9 inch, thick and stabilization carried out after mixing the appropriate quantity of cement and water. This process is rather inconvenient and uneconomical.

8. In order to illustrate the relative cost of layered construction using in-situ and plant mixed methods the following table (9) relating to relative costs in Britain is given as a guide.

Method of Construction	Thickness of Construction	Layers	Unit Cost per sq. yard
Mix in place	6 in.	Single	Sh 6/-
" " "	8 "	"	" 6/8
" " "	10 "	Double	" 12/4
" " "	12 "	"	" 13/-
Plant mixed	6 in.	Single	Sh 7/11
" "	8 "	"	" 10/5
" "	10 "	Double	" 11/4
" "	12 "	"	" 14/2

The two sets of costs are worked out by using the locally available soil in both the cases and employing 5 and 10 percent cement in the lower and upper layers.

9. A point which needs some emphasis is, that the quality control, in terms of uniformity of cement and water in the mix, in the stationary plant method, is much better than that obtained in the in situ process. In addition the in situ method tends to weaken the subgrade through a direct exposure of subgrade to weather and traffic. The excavation in the in situ method in addition leaves a thin loose layer of soil in between the subgrade and the soil-cement base; and these limitations should be kept in view before finally choosing this method of construction. Many machines such as single pass soil stabilisation plant, windrow type mechanical spreaders, multipass rotary mixer, etc. are in current use in various parts of the world, and each plant has its own advantages under given set of conditions.

T A B L E IX

TYPICAL EQUIPMENT REQUIREMENTS FOR DIFFERENT TYPES OF MIXING MACHINES

Windrow-Type Travelling Mixer	Flat-Type Travelling Mixer	Multiple-Pass Rotary Mixer	Stationary Mixing Plant
FOR PREPARATION 1 Pulverizer - if required 1 Motor Grader with scarifier 1 Windrow Evener or Spreader box FOR HANDLING BULK CEMENT 1 Cement conveyor 1 Cement Tanker 1 Portable Truck scale 1 Windrow-type mechanical cement spreader FOR MIXING AND WATER APPLICATION 1 Windrow type Travelling Mixing Machine with motive power 1 Water Pump at water source 1 Motor Grader for spreading mixied windrow FOR COMPACTION- See Note 1 FOR CURING - See Note 2	FOR PREPARATION 1 Motor Grader FOR HANDLING BULK CEMENT 1 Cement Conveyor 2 or more cement truck as required 1 portable truck scale 1 Mechanical cement Spreader of proper width FOR MIXING AND WATER APPLICATION 1 flat-type travelling Mixer 1 Water Pump at source 2 or more water supply trucks as needed FOR COMPACTION- See Note 1 FOR CURING - See Note 2	FOR PREPARATION 1 Motor Grader with scarifier rotary mixers for pulverizing, as needed 1 Water Truck for prewetting, if needed FOR HANDLING BULK CEMENT 1 Cement Conveyor. 2 or more cement Trucks as needed 1 Portable Truck scale 1 Mechanical cement spreader of proper width FOR MIXING AND WATER APPLICATION Rotary Mixers as needed 1 Water Pump at source 2 or more water pressure distributors or water supply truck as needed FOR COMPACTION- See Note 1 FOR CURING - See Note 2	FOR PREPARATION Motor Grader rollers as needed FOR MIXING 1 Stationary Mixing plant, batch-type or continuous flow type with facilities for storing, handling and proportioning soil, cement and water. FOR PLACING Haul Trucks as needed 2 Spreader boxes FOR COMPACTION- See Note 1 FOR CURING - See Note 2

Note 1: Compaction equipment depends on type of soil-vibratory compactors, vibratory rollers, sheepsfoot-type rollers, three-wheel rollers and pneumatic-type rollers as needed.

Note 2: If Type RC-2, MC-3 cutback asphalts, RT-5 road tars, or asphaltic emulsions are used, pressure distributors as needed.

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TABLE X

STEPS IN CONSTRUCTION PROCEDURES FOR DIFFERENT TYPES OF MIXING EQUIPMENT

Windrow-Type Travelling Mixer	Flat-Type Travelling Mixer	Multiple-Pass Rotary Mixer	Stationary Mixing Plant
A. PREPARATION With in-place Soil 1. Shape roadway to crown and grade 2. Scarify roadway soil 3. Pulverize soil-if necessary 4. Windrow soil and even windrow With borrow Soil 1. Shape subgrade to crown and grade 2. Compact subgrade 3. Place borrow soil 4. Windrow soil and even windrow B Soil-cement processing 1. Spread portland cement 2. Mix and apply water 3. Spread mixed windrow 4. Compact 5. Finish 6. Cure	A. PREPARATION With in-place Soil 1 Shape roadway to crown and grade 2. Loosen soil to design depth when necessary and reshape With borrow Soil 1. Shape subgrade to crown and grade 2. Compact subgrade 3. Place borrow soil 4. Shape borrow soil B Soil-Cement processing 1. Spread portland cement 2. Mix and apply water 3. Compact 4. Finish 5. Cure	A. PREPARATION With in-place soil 1. Shape roadway to crown and grade 2. Scarify roadway soil 3. Pulverize soil-if necessary 4. Pre-wet soil as needed 5. Shape prepared soil With borrow soil 1. Shape subgrade to crown and grade 2. Compact subgrade 3. Place borrow soil 4. Shape borrow soil B Soil-Cement processing 1. Spread portland cement 2. Mix, apply water, and mix 3. Compact 4. Finish 5. Cure	A. PREPARATION With borrow soil 1. Shape subgrade to crown and grade 2. Compact subgrade B. Soil-Cement processing 1. Mix, soil, cement and water in plant 2. Haul mixed soil cement to roadway and spread 3. Compact 4. Finish 5. Cure

10. In order to give a general guidance, tables ~~VIII~~ IX (37) and (~~IX~~ X) have been introduced, to give the requirements for different types of mixing machines, steps in construction for different types of mixing equipment, finishing procedures related to the type of soils, compaction equipment, etc., and this information is considered handy as well as useful in selecting the plant on a project.

XIII FIELD CONTROL

1. The quality of soil-cement, both from the point of view of durability and stability depends to a large extent on the scientific control in the field. The quality control in the field for soil-cement construction can be divided into four groups:-

(a) Condition of subgrade

(b) Selection of soil; checking the quality of cement and water.

(c) Mix proportion

(d) Mixing, laying, compaction and curing.

2. Subgrade

In the preparation of subgrade, care should be taken that wet subgrade is not covered with a pavement. Any weak, loose, and soft subgrade section, should be brought to appropriate moistures and compacted. In certain cases it may be necessary to remove and replace the weak and wet subgrade with suitable materials.

3. Selection of Soil

The soil should be checked thoroughly for its grading, and Atterberg's Limits.

Only soils falling either very near or within the specified HRB Limits should be selected for stabilization. In case the soils available on the site do not conform to the specified limits, efforts should be made to blend the soil to improve the grading. This will help in reducing the percentage of cement required for the attainment of a certain strength.

Organic soils should be avoided but in case no other soil is available appropriate steps should be taken to use either 417 cement or cement mixed with about 1 percent calcium chloride. The quantity of cement and the type of cement, for such soils, as specified by the laboratory should be very strictly adhered to. Any soils containing more than 0.5% surphate should be avoided. Micaceous soils should ordinarily be avoided for stabilization, due to their high swelling characteristics.

4. Pulverization

The degree of pulverization has a very marked effect on the development of strength in soil-cement. The degree of pulverization can be defined as the percentage of material passing 3/16 inch sieve or No. 4 A.S.T.M. sieve, to the total quality of soil taken for a test. The higher the degree of pulverization the better are the results in the fields. Normally pulverization above 65-70 percent is considered very satisfactory. In clays and clay soils better pulverization can be achieved if the clay is broken at a moisture, which is about half of its optimum moisture content ~~cement~~. After initial pulverization, cement is added and mixing continued with the addition of water. The mixing process and the slacking action of cement and water aids further pulverization very effectively.

5. Cement

The cement content of a mix is either specified in terms of percentage by weight or by volume. The percentage by volume of cement, can be converted into percentage by weight or visa versa, for any density by simple computations. In order to facilitate computations in the field, a monograph[38] is given in figure 19. On the basis of the percentage of cement specified, and knowing the thickness of the base, the quantity of cement or number of bags required per sq. yard, are worked out. This can be done either by simple calculations or with the help of figures[39] 20, 21, prepared by Portland Cement Association. In case cement is spread by mechanical spreaders ahead of the rotary mixing plant, the accuracy of spreading is checked from time to time by placing a one sq. yard canvas or a metal plate on the ground and collecting cement over it.

It may be essential to determine the cement content and uniformity of mix in the base from time to time, and this may require a certain amount of accurate testing in the laboratory. A number of methods have been developed by the Washington[40] Department of Highways, California[41] Department of Highways, and Road[42] Research Laboratory, United Kingdom, to assess the percentage of cement in a soil-cement mix. Any method which is quick enough in the field can be adopted for control work.

6. Moisture Content

After determining the optimum moisture content in the laboratory, it may be essential to check, if this moisture content and density have any relation to the optimum moisture content and density.

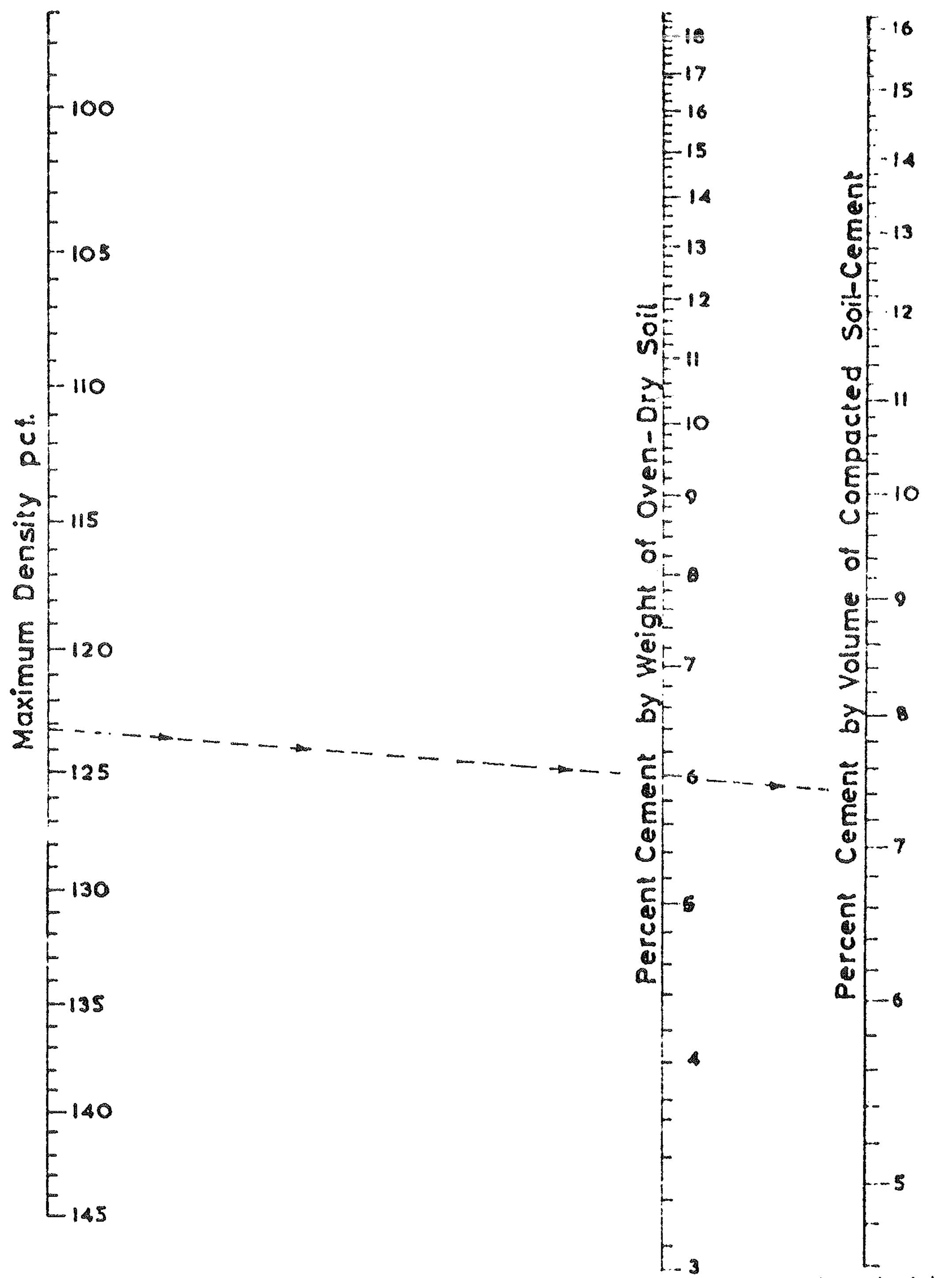

Fig 19 Relation of cement content by weight of oven-dry soil to cement content by volume of compacted soil-cement mixture (Ref 38)

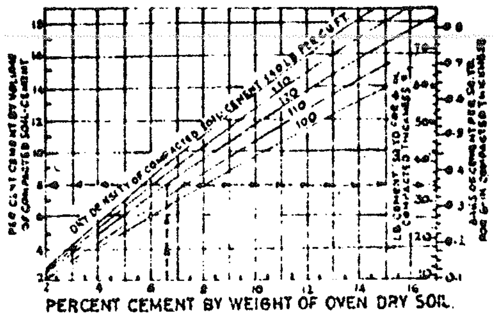

Fig. 20

Cement factor conversion chart. Percent cement by volume of compacted soil cement percent cement by weight of oven dry soil vs quantity of cement per sq yd for a 5 in compacted thickness for known dry densities of soil cement. (Ref. 39)

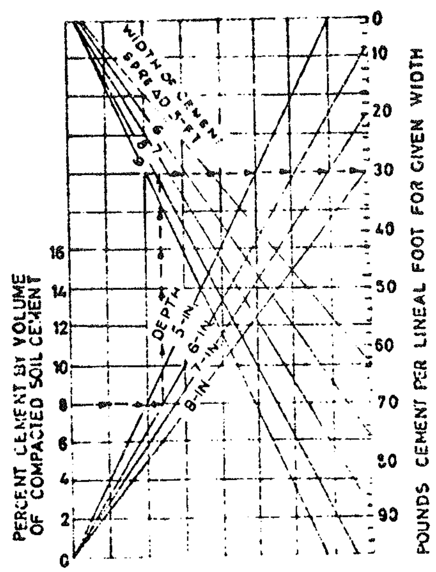

Fig. 21

Pounds of cement needed per lineal foot for various widths and depths for specified cement contents by volume (Ref. 39)

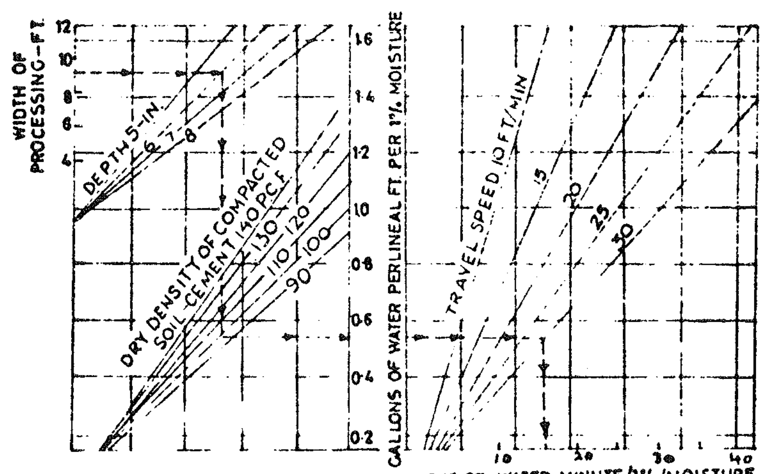

Fig. 22

Water required to raise the moisture content one percentage unit (Ref. 39) in the field, for the compaction plant available for use. This is done by laying small lengths in the field at various moistures, and rolled. The final densities are checked after the rolling is complete. The field moisture-density curve is very essential for setting up standards of compaction in the field.

The control of moisture content is very essential during compaction and this is done by taking samples of soil from time to time and checked for moisture content with quick methods like the speedy moisture meter, alcohol method, etc. It has been pointed out from field experience that approximately 2 percent additional moisture is needed for 5-10 percent cement added to the soil, if the optimum moisture content is determined on the untreated soil. Figure 22 gives a graph for converting the moisture percentage into gallons, for various thickness and densities of the soil cement base. The soil, cement and moisture, should be mixed as thoroughly as possible so that the uniformity of mix is obtained throughout the width and depth of a layer. This is judged by visual inspection only.

7. Compaction and Curing.

The compaction is carried out to bring the soil cement to a maximum density, after the process of mixing is complete. The rolling is started initially by light rollers 3-5 tons, followed by heavy compaction plant. The field density tests are carried out at several places to check the compaction. In case the densities achieved do not fall within the specified limits, the rolling is continued, after giving a light sprinkling of water on the surface of soil cement layer.

The soil cement base, after rolling should be cured for at least seven days, to permit the hydration of cement. Several methods of curing are adopted such as sprinkling of water from time to time, putting wet empty bags of cement, use of 4 inches thick hay or straw and keeping it moist and spraying of asphalt cutback such as M.C.2 or M.C.3, on the pavement soon after compaction at the rate of 0.20-0.25 gallons/sq. yard. The strength of soil cement will largely depend on the efficiency of curing, specially in tropical countries, where the chances of loss of moisture during summer month are very great.

XIV FUTURE STUDIES IN SOIL CEMENT

1. Use of soil cement for the construction of roads and airfields is now accepted as a standard practice. The laboratory studies over a period of thirty years, backed by extensive field experience has gone a long way to explain a good deal about the behaviour of soil cement under different physical conditions. It is now felt that even under very difficult conditions, it is possible to use soil cement as a pavement material with economy and speed.

2. It is however important to bear in mind that soil cement is produced out of two materials namely soil and cement, out of which cement is a standard material, whereas soil is usually very much variable in texture, grading, type of mineral etc. It is for this reason that soil cement as an end product can have very variable properties, and therefore it is essential to understand thoroughly the limitations of soil cement as a material of construction.

3. Although considerable research work has been done on soil cement for use under temperate climate for road and air-field construction, there is still much scope for research on the use of soil cement, in tropics. Some of these fields which do need further studies are listed below:-

(i) Studies on the shrinkage and expansion of soil cement mixes, due to the hydration of cement, as well as temperature changes. Provision of joints in the pavement to avoid indiscriminate and random cracking in soil cement.

(ii) The optimum strengths of soil cement in terms of modulus of elasticity, compressive strength, C.B.R. etc., to give best performance under a given set of loading conditions.

(iii) Development of some rational method for the design of soil cement pavement as semi-rigid material, by making use of its flexural strength.

(iv) The development of strength with age, at different curing temperatures, and relative humidities, under tropical conditions. This can help in an economic mix design of soil cement.

(v) The inter relationship of moulding moisture content, density and compressive strength in soil cement mixes.

(vi) Development of quick tests, for the determination of cement content, prediction of compressive strength, in situ densities and moisture, for effective control of construction in the field.

(vii) Maintenance of adequate records of all soil cement construction under tropical conditions, and correlating the field behaviour with some of the basic properties. Such records can assist a good deal in the formulation of future specifications for soil cement construction in tropics.

(viii) Review of different methods of construction and suitability of various plants under different conditions. This can help a good deal in economising the capital and running cost of projects, thus bringing down the unit cost of soil cement construction.

(ix) Study of the behaviour of various types of asphalt surfacings on soil cement construction.

4. It is felt that scientific investigations of many more aspects, in addition to the above suggested, can go a long way in improving, economising and standardizing soil cement as a material of construction, under tropical conditions. Some of these aspects are already under study in the Building & Road Research Institute, Kumasi and it is hoped that the results of these investigations will be made available to highway engineers in the country from time to time.

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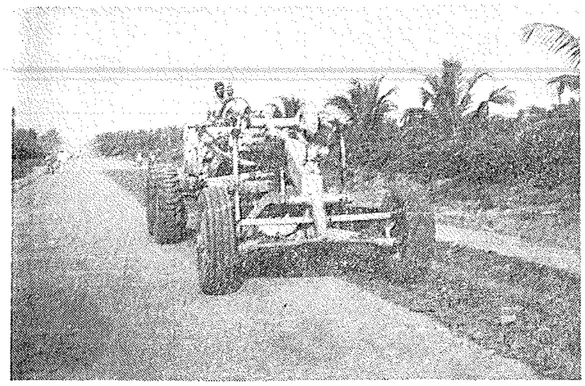


Plate 1—Excavation of Soil with a grader, for in-situ Cement Stabilization.

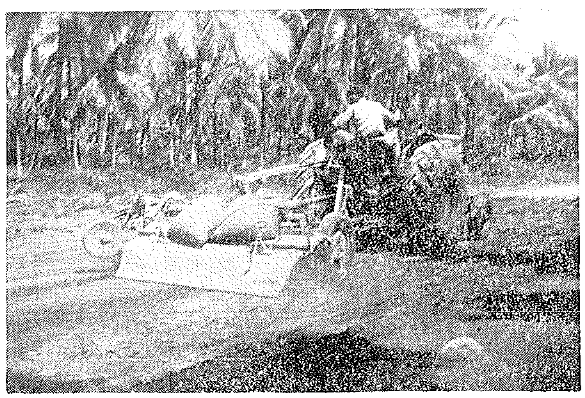


Plate 2—Mixing of Cement with a Rotavator

Plate 3—Final Mixing of Soil Cement and water with a Rotavator, toed to a tractor

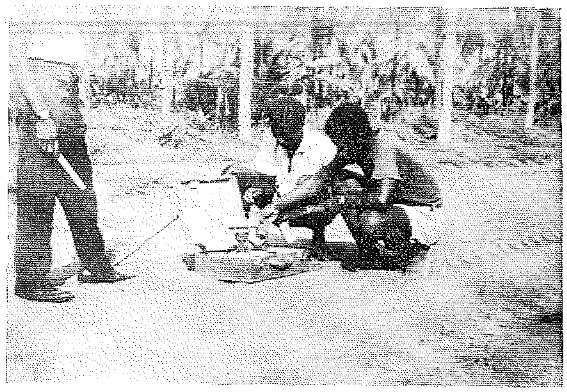


Plate 4—Checking of moisture with speedy moisture meter before starting Compaction.

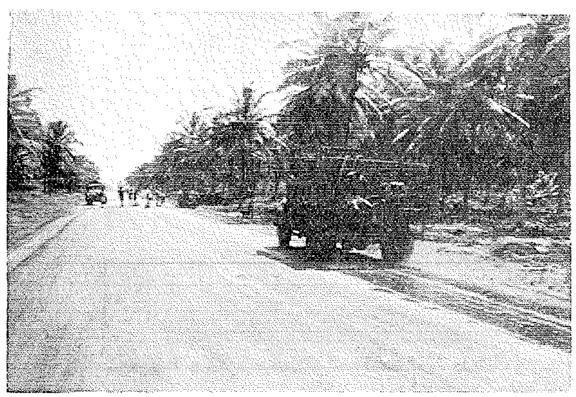


Plate 5—Curing of Soil-Cement road, with water

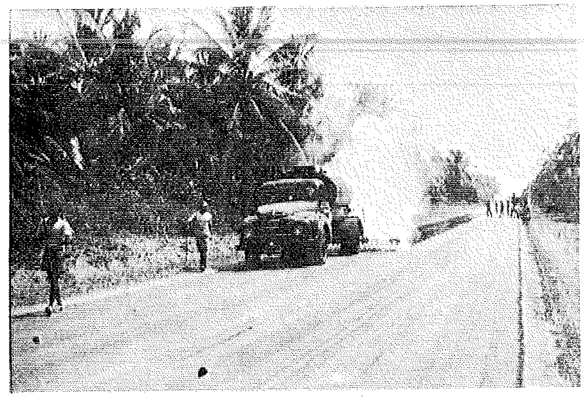


Plate 6—M. C. 2 being sprayed on a Soil Cement road soon after Compaction to act as a membrane for Curing, and a primer Coat

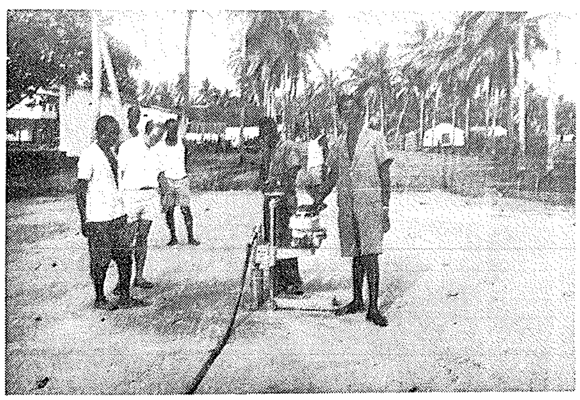
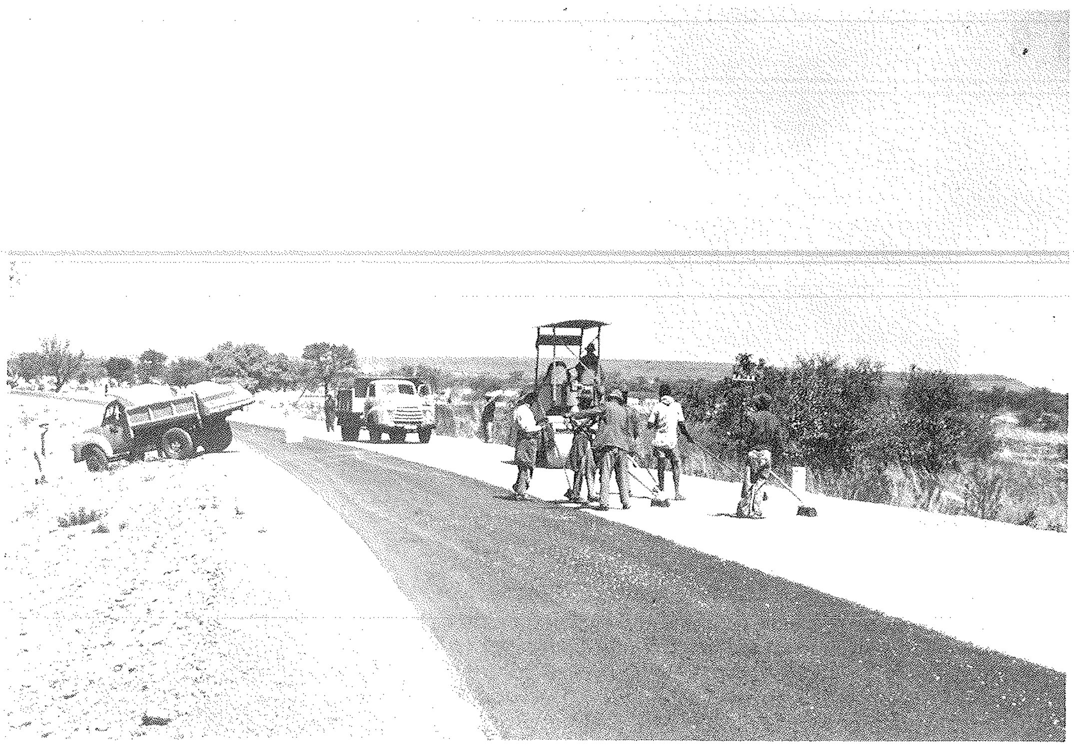


Plate 7— Cutting Cores from a finished Soil Cement road, to study the effect of age on its strength, under field Conditions.



Surface treatment is applied to a lime-stabilized lateritic gravel (Central Africa).

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Stabilization

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TRANSPORTATION RESEARCH BOARD
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CONTENTS

VARIATION IN LABORATORY AND FIELD STRENGTHS OF SOIL CEMENT MIXTURES

James L. Melancon and S. C. Shah, Louisiana Department of Highways

This report evaluates the variability in compressive strengths of stabilized in-place soil cement mixtures from the standpoint of design and actual field conditions. The findings are based on 15 projects with soils ranging from high silt to high sand content and 8 to 14 percent cement by volume. The data indicate considerable variation in the laboratory and field-molded specimens. In general, under the present construction techniques of cement application and density and moisture control, the product is within 75 percent of the 28-day design strength (225 psi or 1550 kPa). The data also indicate a need for pug mill mixing of soil and cement to reduce cement content variation.

•THIS report, which is an abridgment of a comprehensive study (1), evaluates the variability in compressive strengths of stabilized in-place soil cement mixtures from the standpoint of design and actual field conditions. The evaluation is based on 15 projects with soils ranging from high silt to high sand content and plasticity indexes of up to 15. The cement content varied from 8 to 14 percent by volume.

PROCEDURE

For strength evaluation, specimens were prepared and cured both in the laboratory and in the field. The strengths of these specimens were compared to the 7- and 28-day strengths of roadway cores. The variability of the laboratory design was studied in four phases. Each phase was designed to provide data on the variability among and within laboratories.

TEST RESULTS

Figure 1 shows the mean percentage of all tests that achieved specified compressive strengths in each mode of sampling and curing.

The means for the first bar chart (laboratory-molded and laboratory-cured, 7 day) include compressive strength results of materials in which the cement quantity recommendations were originally based on the wet-dry brush test, as well as those actually based on 300 psi (2070 kPa). The projects in which compressive strength was used for cement recommendations show substantial verification of the materials laboratory design, with only one of these projects having soil types in which less than 300 psi was obtained at the recommended cement percentage.

Variability of Cement Content of Bases

Cement contents of field-molded specimens and cores were determined on all 15 projects.

Publication of this paper sponsored by Committee on Soil-Portland Cement Stabilization.

Figure 1. Mean percentages of specimen compressive strengths.

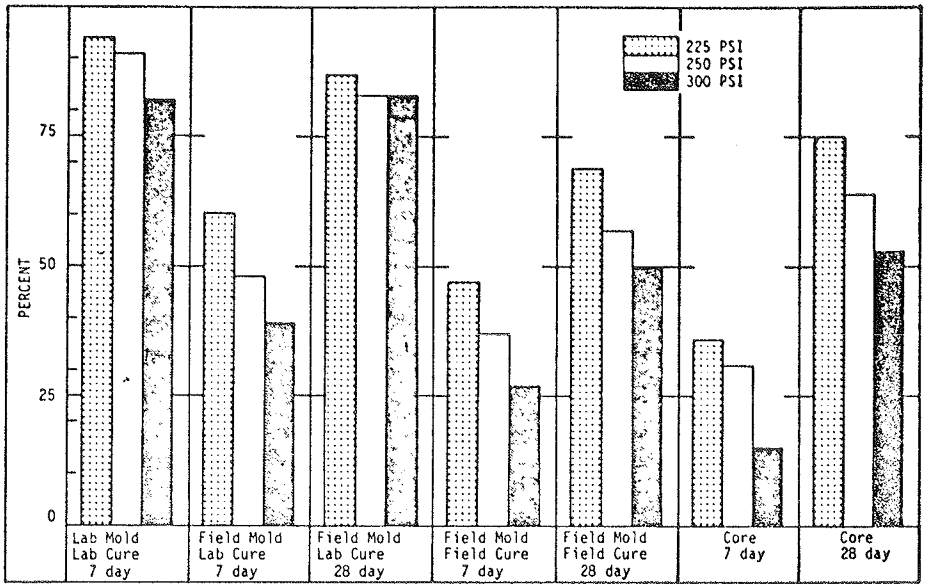

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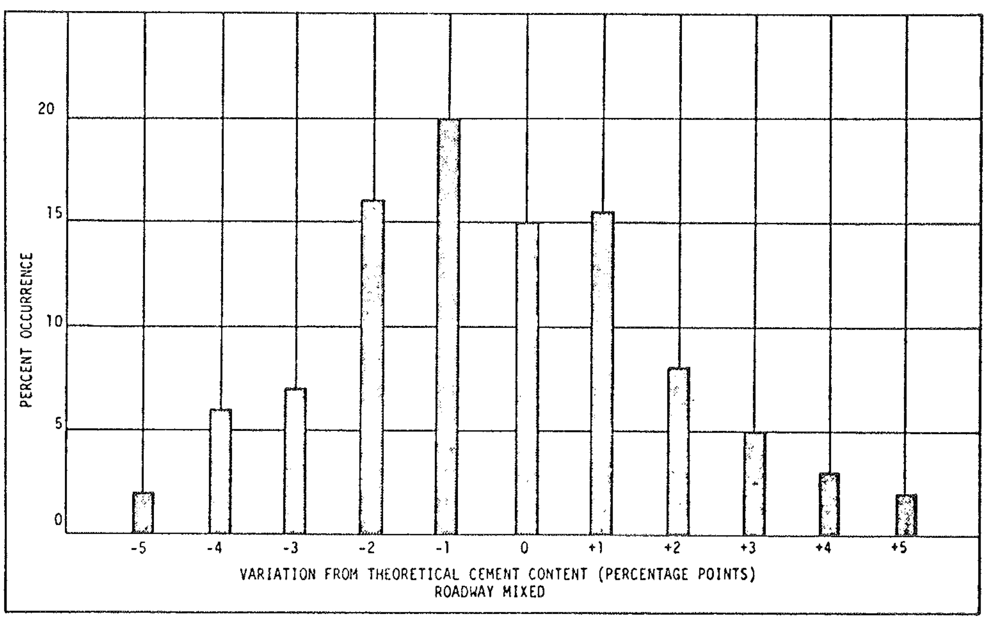
Figure 2. Frequency of occurrence of actual and theoretical cement contents.



An attempt was made to correlate the cement content of field specimens with strength; however, a definite trend could not be established because of the variation in specimen density and curing. These test results, however, did show the wide variation of cement within the soil cement bases as a result of in-place mixing. This is shown in Figure 2, which is a composite of all test runs (311 observations) and shows the percentage of observations in which the roadway-mixed cement content varied from the theoretical cement content.

Variability in Laboratory Design

In the process of obtaining the laboratory data, it was discovered that the laboratory design procedure for soil cement, based on compressive strength, exhibited a greater amount of variability than was previously acknowledged. At first, procedural errors were blamed, but repeated tests under strictly controlled circumstances confirmed the degree of variability. It was found that a difference of 100 psi (6890 kPa) between identical specimens could occur.

To evaluate the variability of the test procedure, interlaboratory tests were conducted. Such test procedures are used to ascertain whether a product meets the specifications set down for the products, or they may be performed for design purposes as was the case here. Regardless of the purpose, the information desired is whether the test procedure as set forth is capable of yielding acceptable agreement among results from different laboratories.

Table 1 gives the statistical parameters for the first phase of the cooperative test. Laboratory designations used in Table 1 are as follows:

Designation	Laboratory
1	Research
2	Materials
3	District 07

The variation for each series of soil cement data is expressed by the standard deviation. In the comparison of variability among laboratory-soil cement series data, the relative measure of the dispersion is given in the table as the coefficient of variation, which is the ratio of standard deviation to the mean of a given series. This measure is particularly useful when widely differing means are encountered.

The magnitude of the coefficient of variation for the district laboratories was considerably higher than that indicated by the research or materials laboratories. This was because they were unfamiliar with the test procedure. Furthermore, the magnitude of this variation was considerably higher than would be expected because of chance alone. Therefore, an effort was made to isolate the causes of variation before the second round of cooperative testing commenced.

It was found that the temperature of the three components—soil, cement, and water—varied widely within and among the three laboratories. The soil and the cement were stored, in some instances, in areas where temperatures were not controlled. That is, the temperature of the storage area fluctuated with the season: high in summer and low in winter. This could result in the use of hot cement and soil for some specimens molded in the summer.

Tap water was used in the molding of all specimens. This in itself did not seem to cause any problems; however, the temperature of one laboratory's tap water was close to 100 F (38 C) because its pipes were adjacent to the building's steam lines.

A check of specimens immediately after molding revealed many dry particles. The existing procedure required the full incorporation of water and cement immediately

Table 1. Statistical evaluation of laboratory data before standardization.

Soil	Cement Content (percent)	Mean 1	Mean 2	Mean 3	Standard Deviation 1	Standard Deviation 2	Standard Deviation 3	Coefficient of Variation 1	Coefficient of Variation 2	Coefficient of Variation 3
A	6	289.33	220.00	254.13	14.36	9.26	21.72	0.05	0.04	0.08
	8	414.00	344.25	344.88	26.41	34.31	22.63	0.06	0.10	0.07
	10	548.67	439.13	492.88	20.13	19.06	25.09	0.04	0.04	0.05
	12	678.33	698.25	647.00	59.80	36.62	65.42	0.09	0.05	0.10
	14	789.25	830.78	871.88	60.29	60.33	136.43	0.08	0.07	0.16
	16									
B	6	211.63	—	—	10.41	—	—	0.05	—	—
	8	278.38	207.88	208.00	14.60	14.20	29.39	0.05	0.07	0.14
	10	315.88	219.63	289.25	8.87	5.90	25.19	0.03	0.03	0.09
	12	387.25	303.63	304.00	17.19	17.53	24.91	0.04	0.06	0.08
	14	442.88	335.88	357.25	30.76	28.82	28.85	0.07	0.09	0.08
	16	465.63	411.88	427.38	11.06	16.47	46.49	0.02	0.04	0.11
C	6	277.75	206.00	231.38	12.12	15.55	26.75	0.04	0.08	0.12
	8	344.38	279.13	283.75	23.65	14.37	30.65	0.07	0.05	0.11
	10	398.25	309.25	349.63	15.94	13.11	53.03	0.04	0.04	0.15
	12	445.00	356.50	479.55	24.04	23.40	55.75	0.05	0.07	0.12
	14	535.63	399.65	469.50	49.51	37.40	36.15	0.09	0.09	0.08

Note: 1 psi = 6.9 kPa.

Table 2. Statistical evaluation of laboratory data after standardization.

Soil	Cement Content (percent)	Mean 1	Mean 2	Mean 3	Standard Deviation 1	Standard Deviation 2	Standard Deviation 3	Coefficient of Variation 1	Coefficient of Variation 2	Coefficient of Variation 3
Sandy loam	6	353.00	237.25	219.00	17.98	8.61	18.40	0.05	0.04	0.08
	8	476.50	302.50	281.75	45.54	15.69	2.36	0.10	0.05	0.01
	10	538.00	345.75	322.00	47.22	5.12	20.49	0.09	0.01	0.06
	12	654.25	445.75	440.25	22.31	29.68	27.40	0.03	0.07	0.09
	14	820.25	506.75	556.75	48.04	8.83	52.71	0.06	0.02	0.09
	16	899.00	557.00	540.25	24.22	30.60	25.32	0.03	0.05	0.05
Clay loam	6	254.25	149.75	181.25	15.73	6.65	5.19	0.06	0.04	0.03
	8	333.00	192.25	220.50	8.60	13.40	16.38	0.03	0.07	0.07
	10	359.50	220.00	227.00	16.10	8.83	12.06	0.04	0.04	0.05
	12	391.00	234.50	224.75	19.04	17.06	16.03	0.06	0.05	0.07
	14	440.50	276.50	273.75	19.33	9.25	17.04	0.04	0.03	0.06
	16	510.00	275.50	322.75	4.81	21.13	27.29	0.01	0.08	0.08
Silty loam	6	189.50	136.75	140.75	5.67	5.91	16.50	0.03	0.04	0.12
	8	214.00	167.00	159.50	18.78	12.83	14.11	0.09	0.08	0.09
	10	271.50	199.50	195.50	15.46	12.37	37.12	0.06	0.06	0.19
	12	329.00	218.00	216.50	13.29	11.52	6.35	0.04	0.05	0.03
	14	403.00	265.00	266.00	12.38	12.73	27.64	0.03	0.05	0.10
	16	429.75	310.00	292.25	13.33	16.79	15.31	0.03	0.05	0.05

Note: 1 psi = 6.9 kPa.

prior to mixing. The soil particles did not adequately absorb the water immediately, causing density variations. Later, during the curing process, these soil particles possibly competed with the cement for the available water.

Another possible cause of variation was the cement itself. The cement used by the three laboratories came from different sources. Seven-day compressive strength (AASHO T-106) varied from 2,100 to 4,500 psi (14 480 to 31 030 kPa).

To alleviate these possible causes of variation the following steps were taken:

1. Each component in the fabrication of soil cement specimens was brought to the same temperature, 75 ± 5 F (24 ± 3 C), before the specimens were molded;
2. Water was added to the raw soils, and the mixture was allowed to slake overnight before addition of cement;
3. Cement from the same manufactured batch was used; and
4. The time involved in fabrication of specimens was held uniform.

The densities and moisture contents of the specimens were closely controlled among the three laboratories by using the same density and optimum moisture for specimen design for each material tested.

On the basis of standardization, a second set of soil samples was distributed to the same laboratories. The soils belonged to the same classification group. The same experimental design as the first one was used in this phase except that there were four replications instead of the previous eight. The improvement in the variability is clearly evident from data given in Table 2. With the exception of two series of data, the relative dispersion was 0.10 or 10 percent or less. Overall there was a decrease in the variability of the test procedure as a result of standardization.

The effect of soil samples stored for some period of time and then mixed for cement content determination was another aspect studied. To test whether there is a difference due to time in the strength property of specimens mixed and compacted at different times by the same laboratory, the statistical t-test for unpaired data was run. The mean for each soil group data obtained at time A was compared to the mean of the same soil data obtained at time B. On the basis of the calculated t-values, none of the differences in the means was significant at the 0.05 level.

CONCLUSIONS

The primary conclusions on the basis of this study are as follows:

1. The inconsistency in laboratory design can be minimized if factors such as molding, temperatures, slaking, and fabrication time are closely adhered to.
2. Under the present construction techniques of cement application and density and moisture control, a fair product is produced with 75 percent of the stations having achieved 75 percent (225 psi or 1550 kPa) of the design strength at 28 days. The compressive strength of cores taken on eight projects after 3 months or more was well over 300 psi (2070 kPa).
3. For those projects in which the laboratory design criteria were based on compressive strength, the raw soils sampled and tested in the laboratory showed substantial verification of the materials laboratory design. Only one project had soil types in which the 7-day strength was less than 300 psi (2070 kPa) at the recommended cement percentages.
4. In-place mixing of cement with soil appears to be somewhat undesirable. Results of 311 observations show a variation of ±5 percent from the theoretical cement content in the soil cement bases studied.

ACKNOWLEDGMENT

This report reflects the views of the authors and does not necessarily reflect the official views or policies of the Louisiana Department of Highways or the Federal Highway Administration.

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1. Soil-Cement Study. Louisiana Department of Highways, Research Rept. 72, Nov. 1973.

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WASHINGTON, D.C. 1970

Contents

Changes in the Characteristics of Cement-Stabilized Soils by Addition of Excess Compaction Moisture

GEORGE R. LIGHTSEY, ARA ARMAN, and CLAYTON D. CALLIHAN,
Louisiana State University

The compaction moisture of cement-stabilized soils is usually specified as the optimum moisture content to obtain maximum density as determined by the standard Proctor test. Previous investigations have shown that in some instances maximum density may not correspond to maximum strength. If compaction of the soil-cement mix is delayed, the relationship between compaction moisture and the strength and density of the soil-cement also changes. This study investigates the relationship between compaction moisture content and the strength, density, and durability of cement-stabilized soils in which compaction is delayed after mixing to correspond to typical highway construction practices. Four types of soil suitable for cement stabilization were investigated. The compaction moisture content was varied from 4 percent below to 4 percent above the optimum moisture content obtained by standard Proctor tests with no delay between mixing and compaction. At each of the moisture contents, and at the optimum cement content, specimens were compacted 0, 2, 4, and 6 hours after mixing with no intermittent mixing. Specimens were prepared for unconfined compressive strength and durability tests. The results of this investigation show that the loss in strength and durability of soil-cement resulting from a delay in compaction can be significantly reduced in many instances by the addition of excess compaction moisture. The soils most benefited after a delay in compaction by excess moisture were the silty loams and sandy loams. Strength increases of 40 to 50 percent were achieved with these soils by the addition of 2 to 4 percent excess moisture when compaction was delayed. Cement-stabilized silty clay loams and silts compacted after delays showed little improvement in strength and durability with excess compaction moisture. Without delay in compaction, only the silty clay loams were significantly improved in strength and durability by the addition of excess compaction moisture. A study of the data has indicated that the amount of excess moisture required for maximum strength and durability depended on the soil type and the detention time between mixing and compaction. In granular soils the addition of excess moisture improved the strength and durability after delays in compaction. This improvement resulted from the improved lubrication of the soil aggregates and subsequent increase in dry density. With fine-grained soils excess moisture improved the properties of soil-cement mixes compacted without delay by increasing the amount of cement hydration.

•THE STANDARD PROCTOR METHOD is recommended to determine the compaction moisture requirements for soils stabilized with cement. Several investigators have found that this moisture content does not necessarily produce maximum density and

Paper sponsored by Committee on Soil-Portland Cement Stabilization.

maximum strength and durability. Felt (1) reported as early as 1955 that for maximum effectiveness portland cement and sand mixtures should be compacted slightly below the optimum compaction moisture content for maximum density and that silty and clayey soils stabilized with cement should be compacted 1 to 2 percent above the optimum.

Davidson et al. (2) also reported that the moisture contents for maximum strength are generally on the dry side of the optimum for predominantly sandy soils, and above the optimum moisture content for soils rich in clay. They postulated that the variation between optimum moisture content for maximum density and maximum strength was related to the particle sizes in the soil. They proposed that soils having a large surface area absorb much of the added water for lubrication so that insufficient water is available for hydration of the cement.

In the field, the time between the mixing of the soil and cement and the compaction of the soil-cement mixture is normally from 2 to 4 hours. Investigators studying the effects of delaying the compaction of soil-cement mixtures have reported large losses in the compressive strength, density, and durability if the delay in compaction exceeds 2 hours (3, 4, 5). They also found that the moisture content to obtain maximum density changes with delay in compaction. Data taken by West while working with sandy soils showed that after delay in compaction the moisture-density curves tended to change from the usual convex parabola to a concave parabola with the largest reduction in density occurring near the optimum moisture content.

The purpose of this study was to investigate the relationship between molding water content and the strength, density, and durability of cement-stabilized soils both with and without delay in compaction.

TESTING PROCEDURE

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Ten natural soils considered suitable for cement stabilization were selected from various locations in Louisiana. The properties of these soils are given in Table 1. Each soil was stabilized with the optimum amount of cement as determined by the criteria developed by the Portland Cement Association based on freeze-thaw and wet-dry tests. Standard Proctor test ASTM D 558-57 was used to determine the optimum moisture content of the soil-cement mixture. Specimens were molded at the optimum moisture content and at 2 and 4 percent above and below the optimum.

Samples were not compacted at moisture contents higher than 4 percent above the optimum. Soils at higher moisture contents were not workable because they were above the plastic limit. Samples were molded according to ASTM D 1632-63 for unconfined compression tests and according to ASTM D 559-57 for wet-dry durability tests.

TABLE 1

SOIL PROPERTIES

Soil Identification	AASHO Group	Optimum Percent		Liquid Limit[c]	Plasticity Index[c]	Composition[d] (percent)		
		Cement[a]	Moisture[b]			Sand	Silt	Clay
Silty loam L-1	A-4	10	17.0	27	8	28	60	12
Silty loam L-2	A-4	10	17.0	28	8	7	79	14
Silty loam L-3	A-6	12	15.5	31	11	32	51	17
Silty clay loam L-4	A-4	10	17.5	28	9	26	53	21
Silty clay loam L-5	A-6	12	17.0	37	19	19	62	19
Silty clay loam L-6	A-6	12	20.0	38	14	28	51	21
Sandy loam L-7	A-2-4	7	15.0	25	7	71	16	13
Sandy loam L-8	A-2-4	7	10.5	18	1	67	21	12
Silt L-9	A-4	10	17.5	28	6	5	80	15
Silt L-10	A-6	12	19.0	36	11	0	87	13

[a] According to criteria established by the Portland Cement Association.
[b] Standard Proctor test, ASTM D 558-57.
[c] Atterberg limits, ASTM D 423-61T and ASTM D 424-5.
[d] Grain size analysis by ASTM D 422-63.

In order to study the effects of moisture content on the strength, density, and durability after a delay in compaction, samples were molded approximately 12 minutes after mixing had begun and at 2, 4, and 6 hours after initial mixing. All specimens prepared for strength studies were cured for 7 days in a 100 percent humidity chamber and then immersed in water for 4 hours prior to compression testing. Samples prepared for durability studies were tested according to ASTM D 599-57. All tests were run in triplicate, and over 1,200 samples were molded and tested during the course of this investigation.

At various detention times and moisture contents, portions of selected soil-cement mixtures were freeze-dried to prevent further hydration of the cement. Electron and optical microscopic studies and hydrometer and sieve analysis were conducted on the dried samples to study changes in particle shape and size. Isopropyl alcohol was used in the hydrometer and sieve analysis to prevent further cement hydration during the course of the experiment.

Chemical analysis and X-ray diffraction studies were also conducted to compare the extent of cement hydration at various moisture contents and detention times. The chemical analysis consisted of a spectrophotometric determination of the amounts of acid-soluble silica formed by the hydration of portland cement. Following the procedure developed by Ruff and Ho (6), the acid-soluble silica formed as the cement hydrate was extracted with 0.2N HCl during a half hour of vigorous shaking. The soil particles were removed from the liquid by centrifuging at 30,000 rpm for 5 minutes. Ammonium molybdate was then added to the solution and the absorption measured at 400 mu with a Beckman DU spectrophotometer.

The extent of cement hydration in soil-cement mixes at various moisture contents was also measured by X-ray diffraction. Powder specimens were X-rayed from 2 deg 2 theta to 40 deg 2 theta in a Philips Norelco diffractrometer using copper radiation. The reflections at 2.77 and 4.93 Å where chosen for study. The strong reflection at 2.77 Å

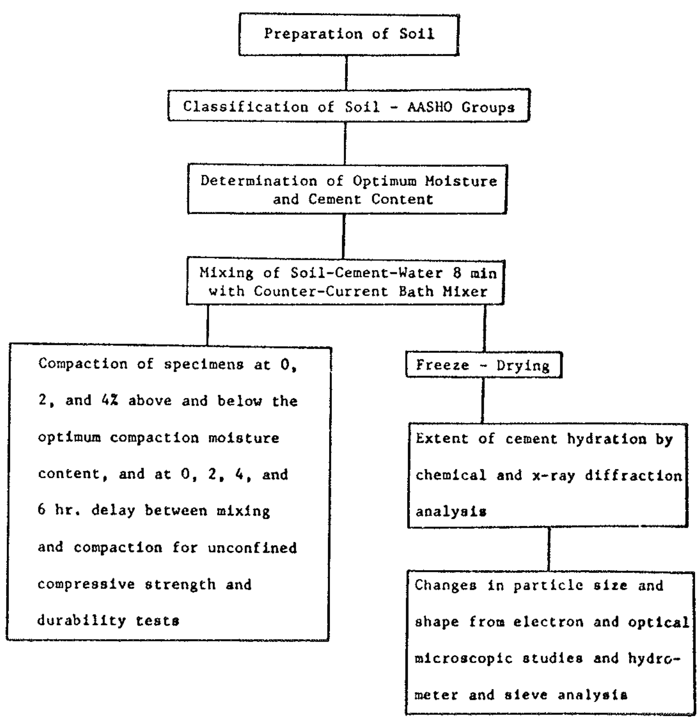


Figure 1. Flow diagram of experimental procedures.

is characteristic of unhydrated cement containing tricalcium silicate and should decrease as the cement is hydrated. The diffraction line at 4.93 Å results from the presence of the calcium hydroxide formed during the hydration reaction. The quartz peak of the soils at 2.45 Å was used as the internal standard.

The testing procedures are shown by the flow diagram in Figure 1.

DISCUSSION OF RESULTS

Silty Loam

The soils identified as L-1, L-2, and L-3 are classified as silty loam. The effects of percentage of compaction moisture on the strength, density, and durability of these soils stabilized with optimum cement content and compacted after detention times of 0, 2, 4, and 6 hours are shown in Figure 2.

With no delay in compaction, these soils developed maximum compressive strength and durability at the optimum moisture content. However, a delay in compaction drastically changed the moisture-strength, -density, and -durability relationships. The compressive strength of samples compacted at the optimum moisture content, obtained with no delay, was reduced 40 to 60 percent after a 2-hour delay in compaction and 60 to 70 percent after a 6-hour delay. Increasing the percentage of compaction moisture 4 percent above the optimum reduced the loss of strength after a 2-hour delay to only 15 percent for silty loam L-2. There was also considerable improvement in strength of soil L-2 compacted after delays of 4 and 6 hours. Increasing the compaction moisture content above optimum improved the strength of soils L-1 and L-3 only a small amount.

The moisture content producing maximum density was increased more than 4 percent after a delay in compaction for soils L-1 and L-2 and decreased 2 to 3 percent for soil L-3.

The durability of the soil-cement mixes compacted after a delay showed tangible improvement with excess compaction moisture.

Silty Clay

The effects of compaction moisture on the behavior of the silty clay loams L-4,

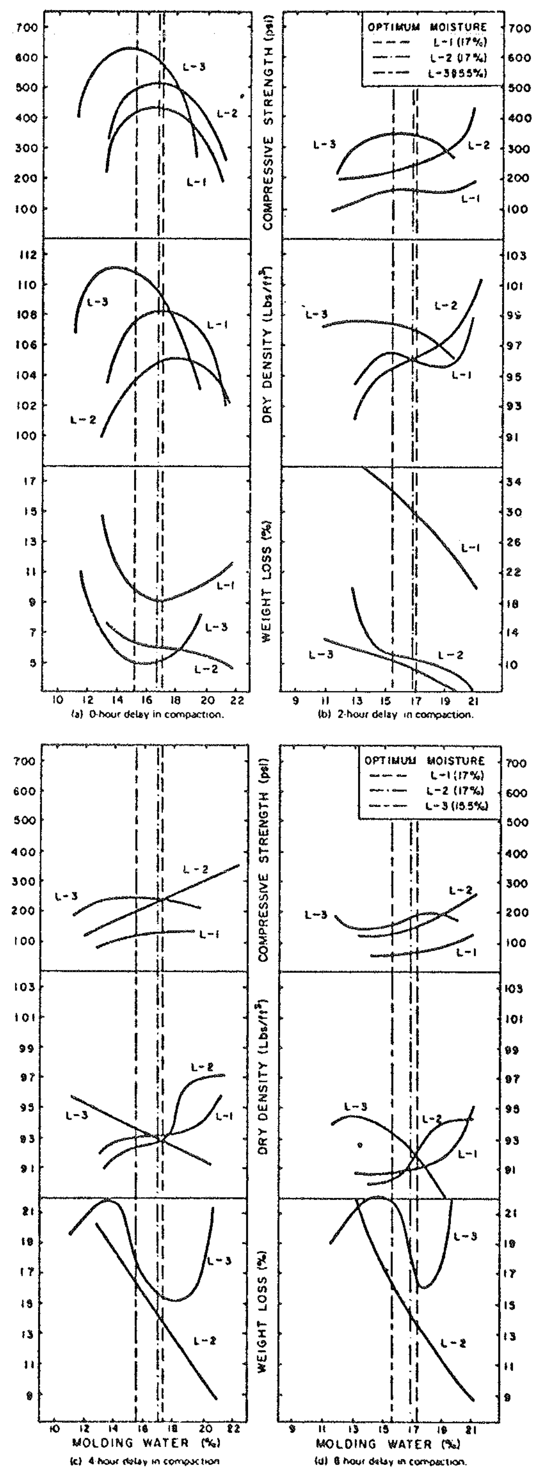


Figure 2. Effect of compaction moisture on silty loam soil-cement mixes at various detention times.

L-5, and L-6 are shown in Figure 3. The silty clay loams developed maximum strength at higher than optimum moisture content even for samples compacted without delay. With delay in compaction, the strength generally increased and then decreased as the molding water content was increased above the optimum.

After a delay in compaction, the density consistently decreased with increasing moisture content.

Silty clay loams compacted without delay showed improved durability at moisture contents higher than the optimum. The durability of samples compacted after a delay was poor regardless of the molding moisture content.

Sandy Loam

The sandy loam soils L-7 and L-8 had varied results as shown in Figure 4. The strength of soil L-8 at optimum moisture content was reduced over 50 percent by a delay in compaction of 2 hours or more. Varying the percentage of compaction moisture did not improve the strength a significant amount. On the other hand, increasing the percentage of molding water 2 percent above the optimum reduced the loss of strength after a 2-hour delay of soil L-7 from 23 to 3 percent.

The moisture-density curves of the sandy loam soils were concave after a delay in compaction instead of the normal convex shape. Maximum densities were obtained above the optimum moisture content with no delay.

With no delay in compaction, maximum durability was achieved at optimum water content. The durability of samples compacted after a delay was greatly improved by the addition of excess compaction moisture.

Silt

The variations of strength, density, and durability with moisture content for silts L-9 and L-10 are shown in Figure 5. The optimum moisture content for maximum strength and density did not change significantly with a delay in compaction.

These 2 soils differed appreciably in durability after delay in compaction. In general, the durability improved with increasing compaction moisture. Soil L-9 proved to be a very durable material for soil-cement stabilization even after delays in compaction of up to 6 hours. Soil L-10, although giving higher initial strength at 1-hour delay, had lower durability than L-9 at all moisture contents.

General

The results obtained for the 10 soils show that, when compaction of the soil-cement mix is delayed, maximum strength and durability in most instances are not obtained at the optimum moisture content for maximum density as determined with no delay but developed at moisture contents above the optimum. Even with no delay in compaction, the silty clay loams developed maximum strength and durability at higher than optimum moisture contents. A study of the data indicates that the increase in strength and durability with excess compaction moisture is due to a combination of increased dry density and improved cement hydration.

The reaction of cement with soil in the soil-cement matrix is very vigorous and is a function of time and moisture content. If compaction is delayed after mixing, cementation of the loose soil grains into larger aggregates takes place. This conglomeration effect increases the resistance of the soil to compaction so that the density obtained at a given moisture content and compactive effort is reduced.

As mentioned earlier, delay in compaction also tends to increase the moisture required to obtain maximum density. The necessary increase in compaction moisture with delay in compaction seems unusual because large soil particles normally require less water for lubrication than small particles. However, consideration of the shape of the soil aggregates explains why more moisture is required for lubrication. Figure 6 shows the configuration of the soil aggregates as seen with an optical microscope. The extremely irregular shapes of the aggregates increase the resistance to compaction by mechanical interlocking. If sufficient water is added to form a continous film around

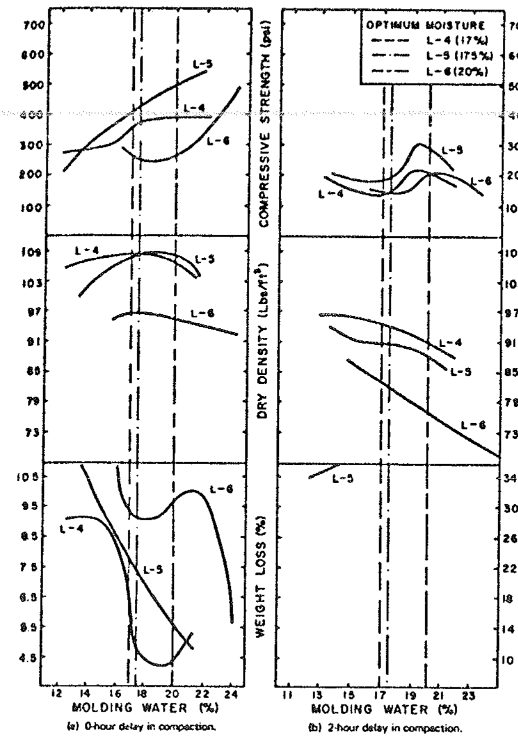


Figure 3. Effect of compaction moisture on silty clay loam soil-cement mixes at various detention times.

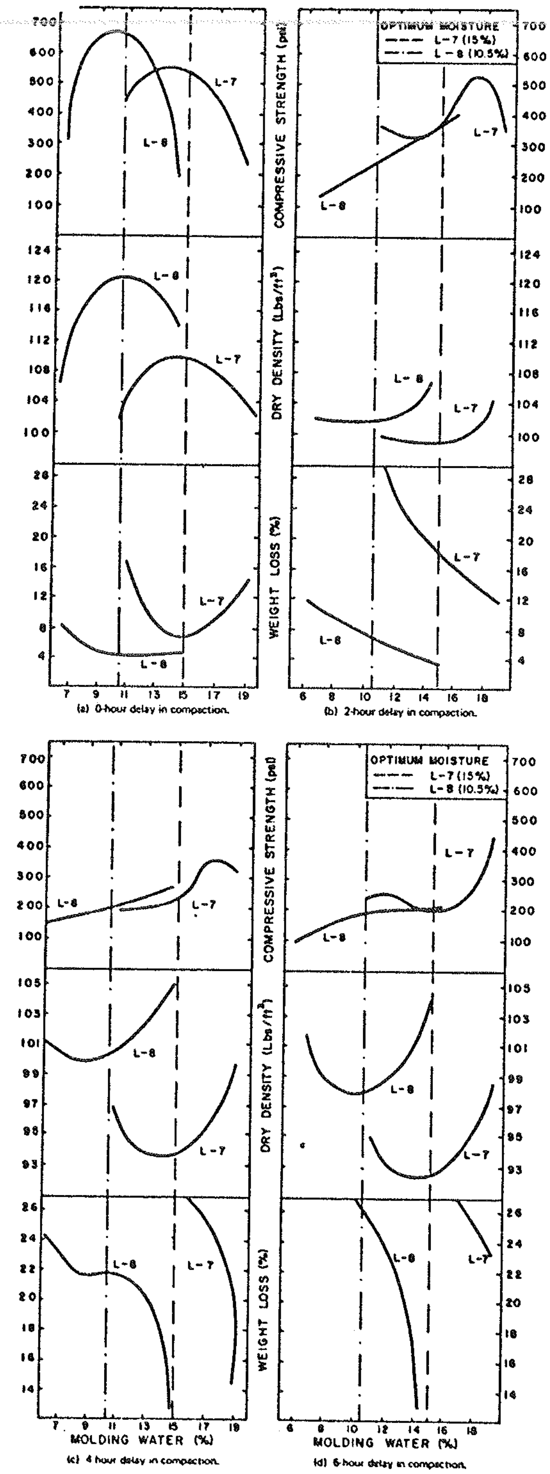


Figure 4. Effect of compaction moisture on sandy loam soil-cement mixes at various detention times.

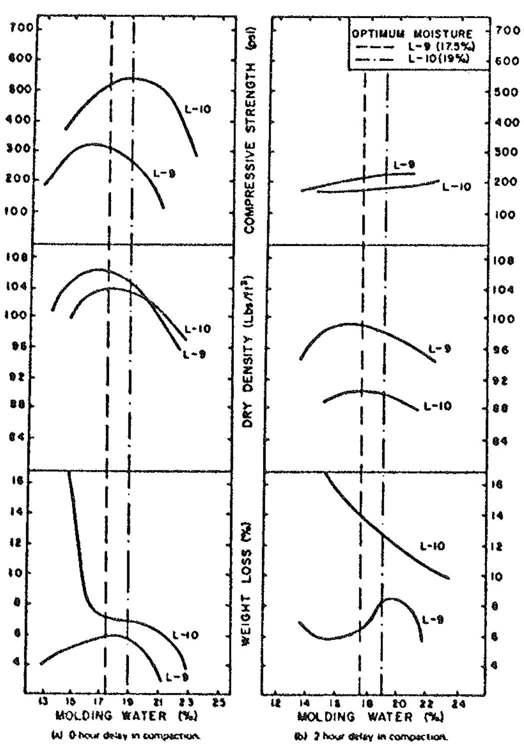
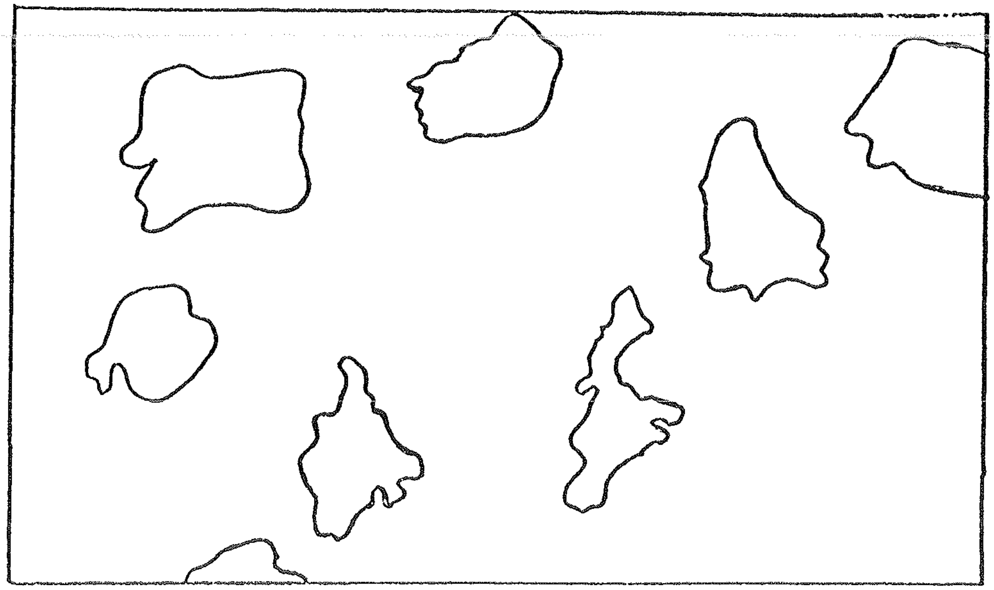
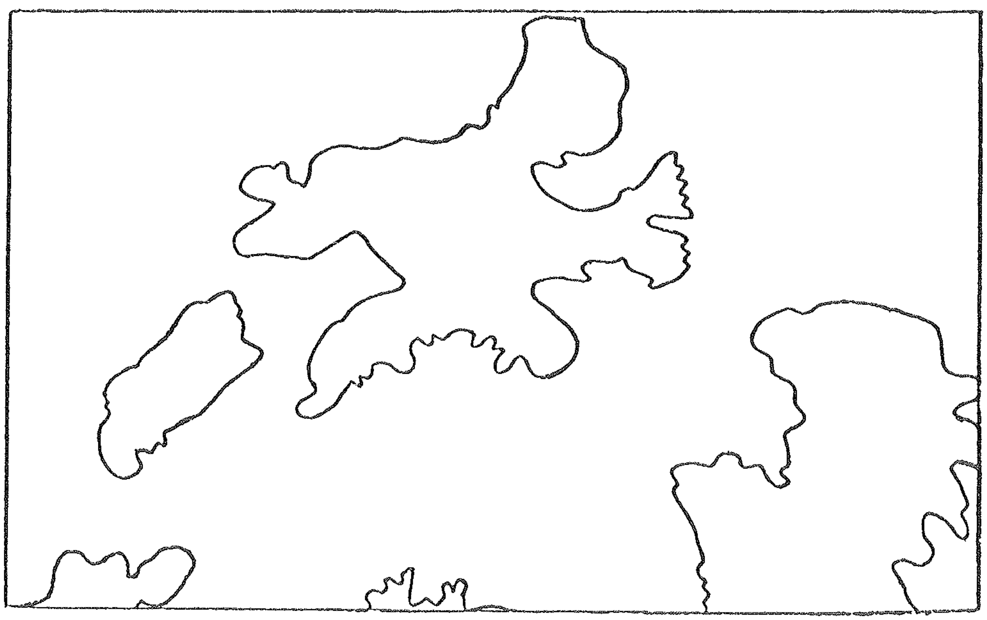


Figure 5. Effect of compaction moisture on silty soil-cement mixes at various detention times.



(a) 0-hour after mixing (magnification of 750x)



(b) 6-hours after mixing (magnification of 750x)

Figure 6. Effect of delay in compaction on the size and shape of soil particles.

the aggregates to prevent excessive interlocking, the resistance to compaction will be lower and the dry density will be increased. A schematic representation of this effect is shown in Figure 7.

The extent of soil particle aggregation with detention time was measured by a hydrometer and sieve analysis of the soil-cement mixture. Typical data for a silty clay loam are shown in Figure 8. These data indicate that considerable aggregation does occur

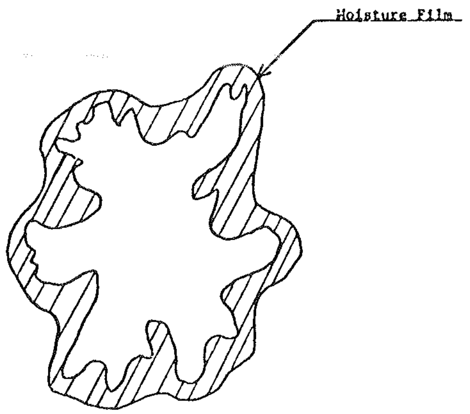


Figure 7. Hypothesis of reduction in mechanical interlocking at high moisture contents.

TABLE 2

EFFECT OF PERCENTAGE OF COMPACTION MOISTURE ON THE EXTENT OF CEMENT HYDRATION IN SILTY CLAY LOAMS

X-Ray Diffraction of Specimens Cured for 7 Days

Compaction Moisture (percent)	Tricalcium Silicate Line (2.77 Å)	Calcium Hydroxide Line (4.93 Å)	Quartz Line (2.45 Å)
10	15.9	3.2	100
12	7.0	3.0	100
14	6.0	4.0	100
16	11.0	2.0	100
18	8.0	7.0	100

Chemical Analysis of Acid-Soluble Silica

Compaction Moisture (percent)	Acid-Soluble Silica (g/100 g soil)		
	6-Hour Curing	1-Day Curing	7-Day Curing
10.0	0.68	0.86	1.04
12.5	—	1.03	1.08
15.0	0.74	1.02	1.07
17.5	0.74	1.06	1.05
20.0	—	1.11	0.98
22.5	—	0.95	0.96
25.0	—	1.31	1.20

with a delay in compaction. A 6-hour delay in compaction reduced the weight of particles less than 100 microns by 23 percent.

The increase in strength and durability at higher than optimum moisture contents after delay in compaction of the silty loam and the sandy loam soils can be attributed to the increase in dry density. The danger of directly relating the dry density and the strength and durability is readily apparent in Figure 4 (c and d). After a delay in

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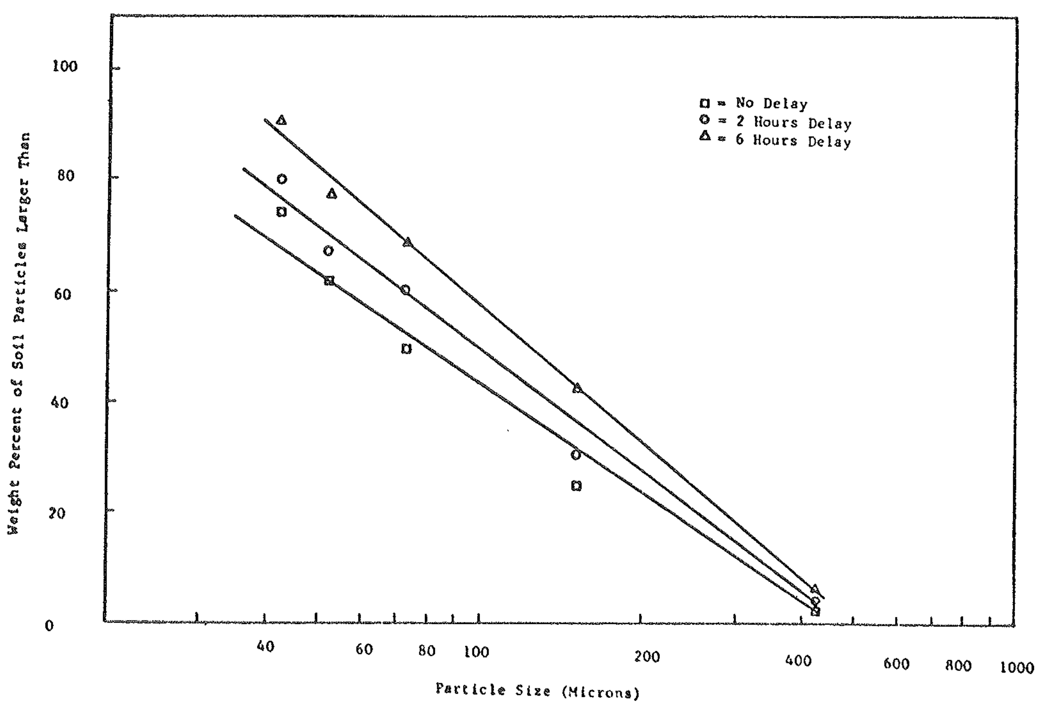

Figure 8. Effect of delay in compaction on soil particle size.

TABLE 3

COMPACTION MOISTURE FOR OPTIMUM SOIL-CEMENT PROPERTIES

Soil Identification	Composition (percent)			Compaction Moisture for Maximum Strength and Durability[a]			
	Sand	Silt	Clay	0-Hour Delay	2-Hour Delay	4-Hour Delay	6-Hour Delay
L-1	28	60	12	17.0	21.0+	21.0	21.0
L-2	7	79	14	17.0	21.0+	21.0+	21.0+
L-3	32	51	17	15.5	15.5	17.5	17.5
L-4	26	53	21	19.0	19.0	19.0	19.0
L-5	19	62	19	21.5+	19.5	19.5	19.5
L-6	28	51	21	24.0+	20.0	20.0	22.0
L-7	71	16	13	15.0	17.0	17.0	19.0+
L-8	67	21	12	10.5	14.5+	14.5+	14.5+
L-9	5	80	15	17.5	19.5	21.5	19.0
L-10	0	87	13	19.0	23.0+	23.0+	19.0

[a] + after moisture content denotes continuous increase in strength and durability with increasing moisture content.

compaction, the density of the sandy loam soils is significantly increased at moisture contents lower than the optimum moisture content obtained at 0-hour delay. However, at the low moisture contents, the strength and particularly the durability are much less than at higher moisture contents. The higher densities at low moisture content result from the small amount of cement hydration and subsequent small amount of soil aggregation.

The explanation for the behavior of the silty clay loams is not the same as for the silty loams and sandy loams. The strength of the silty clay loams continued to increase with an increase in the moisture content up to 4 percent above the optimum with no delay in compaction. When compaction was delayed, the strength of the silty clay loam soils tended to increase with increasing molding water content even though the density consistently decreased.

One explanation for this behavior as suggested by Davidson et al. (2) is that the moisture content gave maximum density and much of the water added for lubrication is absorbed by the clay particles in the soil, resulting in insufficient water for complete hydration of the cement. Additional compaction moisture above the optimum increases the amount of cement hydration and hence the strength, even though the density is reduced.

To determine if the addition of excess moisture above the amount necessary for maximum density did in fact increase the amount of cement hydration, silty clay loam soil-cement mixtures were compacted at various moisture contents. X-ray diffraction studies and spectrophotometric analysis were conducted as outlined earlier.

The results of these studies are given in Table 2. The results, although inconclusive, indicate a general trend to increased cement hydration in the silty clay loams compacted at moisture contents above that required for maximum density.

Table 3 gives the data collected during this investigation. The results indicate that the amount of excess compaction moisture required for maximum strength and durability depends on the soil type and the detention time between mixing and compaction. If the detention time was 2 to 6 hours, the strength and durability of the more granular soils were significantly improved by the addition of 2 to 4 percent excess compaction moisture. With no delay in compaction, only the fine-grained silty clay loam showed improvement with excess compaction moisture.

CONCLUSIONS

The following conclusions were drawn from the results of this investigation.

1. The moisture content for maximum strength and durability is not necessarily equal to the moisture content that gives maximum density.

2. The rate and direction of change in moisture content, which gives maximum strength and durability after a delay in compaction, are often not the same as the rate and direction of change of the moisture content required for maximum density.

3. The amount of excess compaction moisture required for maximum strength and durability depends on the soil type and the detention time between mixing and compaction. Two to 4 percent excess compaction moisture significantly improves the strength and durability if (a) the detention time is greater than 2 hours and the soils is granular and (b) the detention time is less than 2 hours and the soil is fine-grained.

4. After delays in compaction, the strength and durability of silty loams and sandy loams are significantly improved at moisture contents 2 to 4 percent above the optimum determined at 0-hour delay. The change in moisture content for maximum strength and durability with delay in compaction is due to aggregation and mechanical interlocking of soil particles. The addition of excess moisture lubricates the soil aggregates and increases the density, strength, and durability.

5. The moisture content giving maximum strength and durability with silty clay loams is 4 percent above the optimum with no delay in compaction, and 0 to 2 percent above the optimum with delay in compaction. The increase in strength at moisture contents up to 4 percent above optimum when compaction is not delayed appears to be due to more efficient cement hydration.

6. The moisture-density relationships for silts change little with delay in compaction. However, improvement in the durability of silts is achieved at moisture contents 2 to 4 percent above the moisture content for maximum density with no delay in compaction. Little improvement in the strength is obtained at higher moisture contents.

7. It is recommended that, if delays in compaction of 2 to 6 hours are expected, the compaction moisture of cement-stabilized silty loams, silty clay loams, sandy loams, and silts should be 2 to 4 percent above the amount required for maximum density as determined by Proctor test performed with 0-hour delay. The compaction moisture content of silty clay loams should be 3 to 4 percent above the optimum moisture content even if no delay in compaction is expected.

8. The use of additional moisture does not completely counteract the detrimental effects of delay in compaction; however, it will significantly improve the properties of compacted soil-cement mixtures with very little extra cost. Additional studies are being conducted to determine methods of completely overcoming the deficiencies in soil-cement as a result of delay in compaction. Until methods of eliminating the adverse effects of delayed compaction are perfected, time restrictions between mixing and compaction should be strictly enforced.

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SUPPLEMENTARY REPORT 284

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BITUMINOUS BASES AND SURFACINGS FOR LOW-COST ROADS IN THE TROPICS

by

L S Hitch and R B C Russell, BSc MPhil MIHE FGS

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CONTENTS

BITUMINOUS BASES AND SURFACINGS FOR LOW-COST ROADS IN THE TROPICS

ABSTRACT

Mechanically stable materials for road bases are often not obtainable in developing countries and the technique of soil stabilisation has therefore been developed. In the Middle East, aggregates are often scarce but oil products are readily available. The region has therefore provided some of the earliest examples of bituminous stabilisation, which originally consisted of thin running surfaces over compacted sand. Bituminous stabilisation can also enable local sand to be used for base construction, and various tests and design criteria have been proposed for such applications.

The report describes full-scale experimental trials supported by laboratory research, which have enabled acceptance criteria for bitumen-stabilised sand bases for light/medium traffic to be proposed. Construction methods for bituminous stabilisation are also described.

Details are given of methods of surface dressing, which is important both as an initial running surface on new bases and as a maintenance treatment.

Premixed bituminous materials, both as bases and surfacings, might perhaps be considered as inadmissible for low-cost roads. Such roads, however, usually require progressive improvement because of the traffic growth which accompanies development. There is a growing use of strengthening overlays and the report briefly discusses premixed materials and their application.

1. INTRODUCTION

Much of the accumulated knowledge gained from the construction of roads over the last 150 years relates to the use of mechanically stable roadmaking materials such as crushed rock. These materials are not always available, particularly in the remoter parts of many developing countries, but in recent years the development of the technique of soil stabilisation provides practical alternative roadmaking materials in such situations.

Stabilisation of soil and gravels with cement and hydrated lime is now well-established,[1,2,3] notably in Africa, but in the more arid regions, particularly those with an abundance of non-cohesive superficial materials, bitumen is an appropriate stabilising agent.

The scarcity of aggregates in parts of the Middle East, coupled with the plentiful supply of oil products and the generally arid climate, has encouraged the use of bituminous stabilisation for road construction in the region.

194 This report discusses appropriate design criteria for sand-bitumen roads carrying light and medium traffic in hot climates.

Bituminous surface dressings are a very appropriate type of surfacing for low-cost surfaced roads. If well-executed they can provide a more satisfactory and cost-effective surfacing than some types of hot-mixed bituminous surfacings. This report discusses the design of surface dressing and briefly describes appropriate construction procedure.

Premixed bituminous overlays are appropriate for strengthening low-cost surface-dressed roads when this is needed because of growth in traffic. The performance of premixed bituminous materials in temperate climates is well established, but less information is published about the design and performance of different mix types in hot climates. This report reviews specifications of premixed bituminous materials for use in hot climates and discusses appropriate construction techniques.

2. CONSTRUCTION METHODS AND THE SELECTION OF BINDERS

Bituminous binders include materials derived from the destructive distillation of coal, naturally-occurring asphalts and petroleum bitumen. The first two are seldom available in developing countries; only petroleum bitumen will be considered in this report.

Bitumen can function in one or more of the following ways, ie as a

(a) lubricant
(b) sealant
(c) adhesive.

These three functions are the basis of all types of bituminous construction, which can be successful only if the type and grade of binder are correctly selected. Selection of the process itself involves a consideration of other features, such as material type or treatment, plant, environment, etc. This is perhaps best illustrated in Table 1, which attempts to summarise several processes, with particular reference to binder viscosity.

3. CONSTRUCTION METHODS FOR BITUMEN-STABILISED BASES

3.1 *Principles of bitumen-bound bases*

Bitumen can be used as a stabilising agent in two ways:

(a) as a waterproofing agent (sealant)
(b) as a binding agent (lubricant/adhesive).

In the waterproofing application, bitumen is mixed with a cohesive soil which has a useful mechanical strength at a given moisture content. The bitumen merely seals the system against moisture changes, thus preserving soil strength. Such applications are rare; more usually bitumen is applied as a binding agent for non-cohesive materials; these can range from a dense continuously-graded crushed rock system (as in asphaltic concrete[4]) to local sand. The former is relatively expensive, the latter relatively cheap; indeed it is becoming increasingly common for road engineers in developing countries to be compelled to use local sand. Comparatively little is known about design criteria for bitumen-stabilised sand, particularly for roads carrying heavy axle loads. The Overseas Unit has cooperated in one full-scale experiment[5] which has given some valuable information, and laboratory research is continuing. (See Sections 3.2.2 and 3.2.3).

3.2 *Experience and research*

It is interesting to consider briefly work done some 40 years ago, mainly in America[6,7,8] and in the Middle East[9], in both of which extensive arid areas occur and through which roads have to be constructed. This work is valuable in that it demonstrates the techniques found to be necessary at a time when construction plant was usually simple and bitumen technology was in its infancy. Basically, little has changed since then, especially in construction methods. The blade grader, for example, is as useful now as before but one is now able to quantify performance criteria to some extent: criteria are discussed later (see Section 3.2.3).

Some early work in the Middle East[9] in the 1930s provided sand-bitumen roads of a very simple construction; sand bitumen produced in simple paddle mixers was spread, and aerated if necessary, by blade graders before compaction with smooth-wheeled rollers. In the absence of mixers, binder was sprayed directly on to the sand and mixed in by grader. Binder contents were evidently sufficiently high to provide adequate cohesion under the abrading action of traffic: this practice would almost certainly be unacceptable today on both economic and engineering grounds. Current practice is to stabilise with sufficient binder to provide adequate shear strength and to seal the surface against traffic abrasion, usually with a surface dressing.

Some valuable precedents were established by the first users of bituminous stabilisation who soon learned that binder viscosity had to be related to the sand temperature and the efficiency of the mixing plant; the importance of cut-back bitumen for this process was thus clear.

3.2.1 The design problem: The study of pavement design may be said to have developed from around the 1940s. Much valuable work was done at this time and has formed the basis of current practice. The California Bearing Ratio method due to Porter[10] is perhaps still the most widely used: the pavement design chart (Fig 1) from the Transport and Road Research Laboratory's Road Note 31[11] relates the strength of subgrade soils to thickness of pavement layers and design life in terms of cumulative standard axles[12]. Minimum soaked CBR values of 80 for crushed stone bases and 100 for cement and lime stabilised material are normally specified. A problem arises, however, in the application of these criteria when constructing bases with bitumen-stabilised soils. Bituminous mixtures behave viscoelastically and are temperature susceptible. In 1954 Nijboer[13] proposed the concept of "stiffness" of a mix, which reflects loading time, acceptable strain (1 per cent max) and maximum temperature experienced in practice. It was shown that this stiffness could be expressed as a ratio of the stability to the flow value (load and strain at failure respectively) measured in the Marshall test[14]. A high ratio may ensure resistance to deformation under traffic but at the expense of a brittle mix. The problem of selecting an appropriate ratio for surfacing materials is considered later (Section 5.1). The application of the concept to bituminous stabilisation is perhaps less important and its relevance probably depends upon the type of traffic using the road and the thickness of the surfacing. The stiffness concept proposed by Nijboer is "rational"; empirical methods, notably the Marshall, Hubbard-Field and the cone penetrometer methods, have also been used for the design of bitumen-stabilised soil bases. A bibliography[15] has been compiled which provides some examples of the use of different design methods and the design criteria adopted.

3.2.2 A full-scale experiment in Africa[5]: Abrasion-resistant sand-bitumen surfaces with relatively high bitumen contents have already been referred to in contrast to the leaner sand-bitumen materials for bases which are more common today. The conflict between abrasion resistance and stability has been recognised for many years. In 1960 the Tropical Section (now Overseas Unit) of the TRRL participated in the construction of experimental sections on a road in Northern Nigeria. These sections were incorporated into a new bitumen-stabilised road, the construction of which has been described elsewhere[16]. It was found that cut-back bitumen with a viscosity of up to approximately 8 x 10^4 centistokes at 60°C (S.125) could be mixed in a simple paddle mixer with the well-graded sand available. The temperature of the sand was 26-28°C and of the binder when added was 120°C (ie at a viscosity of approximately 200 centistokes).

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These findings raise the question of whether it is necessary to use sophisticated mixing plant in order to heat sand simply to enable a viscous penetration grade bitumen to be used, even though the superior stability of such mixes on cooling is unquestionable.

The experimental mixes were laid without aeration to remove volatiles and were surface dressed before curing could occur. Materials stabilised with MC2 grade cut-back failed quickly but were the only sections to fail. The remainder have, in general, performed as satisfactorily as those with the hot mix material.

The road has carried mainly light traffic in its 14 years of use and the number of heavy commercial vehicles probably does not exceed 100 per day; this volume of traffic, however, is characteristic of many developing countries.

It is interesting to record that samples of bitumen removed from slabs and taken from several of the experimental sections and from the hot-mixed main contract material after 13 years were all found to have essentially the same viscosity, ie 10-20 pen. A limited number of small samples taken after 18 months showed that the cut-back bitumens had, even at that early stage, cured to the consistency of 80-100 pen.

3.2.3 Research at TRRL on bituminous stabilisation: The principal objectives of the research undertaken by the Overseas Unit were:

1. To evaluate different design methods for bituminous stabilisation.

2. To establish design criteria.

3. To study the behaviour of a variety of sand-bitumen mixtures under different shear conditions.

The first two of these have so far been studied in the full-scale experiment in Nigeria and the results have been reported in full elsewhere[17]. Briefly, sand-bitumen mixtures were prepared using the sand from the Nigeria experiment and three different binders. Mixtures were cured for one year at 45°C and four tests were used to determine stability. Table 2 shows the results of stability tests at 60°C after one year and Table 3 shows some inter-relations of test values for material tested at 45°C.

Of the five design methods originally included in the study, two (Marshall and Hubbard-Field) are well documented[14]. Alexander and Blott[18] proposed the use of the cone penetrometer for sand-bitumen mixtures and it has been frequently used although, as with some other methods, design criteria have been little more than recommendations. It was found that the test did not correlate well with three other methods examined. The deformation wheel-tracking test[19], designed at TRRL and used mainly for studying deformation resistance in surfacings, was the fourth method studied. Tracking rates were found to correlate well with the Marshall and Hubbard-Field methods. The fifth method (CBR) was rejected as inapplicable early in the study.

The following design criteria for sand-bitumen bases for lightly-trafficked roads were deduced from the above study: 197

Stability, Marshall at 60°C .... 100 kg(min)
Stability, Hubbard-Field at 60°C .... 300 kg(min)

In a recent investigation of the causes of some failures in a road carrying heavy traffic in the Middle East it was found that the type of sand used for the sand-bitumen base varied considerably from place to place along the road and the sand particle texture varied from very rough to very smooth. A limited initial investigation appeared to show a link between failure and bitumen content, and the low residual bitumen contents found (2-4 per cent) were especially noteworthy in relation to the fineness of the sands, most of which passed a 300 μm mesh sieve. The bitumen film thicknesses were thus minimal, and mixes were suspected of being prone to shear failure.

The shear properties of the sands used in this road when stabilised with different percentages of bitumen were studied using a shear box. The effects of binder viscosity and shear rate were also studied. Figure 2 illustrates a typical stress/strain relationship for three sands, showing how the behaviour becomes more plastic as the normal load decreases, ie with increase in depth of construction. Inspection of site failures confirmed Prandtl-type failure,[20] and Fig 3 illustrates the effect of binder viscosity and shear rate on Prandtl bearing capacities for one of the sands.

Calculations based upon the most severe conditions likely to be experienced in a sand-bitumen base showed that the Coulomb equation[21] becomes:

$$\psi = c + 1035 \tan \phi$$

where ψ = shearing stress (kN/m^2)
c = apparent cohesion (kN/m^2)
and ϕ = angle of shearing resistance (degrees).

The results obtained using the shear box are in good agreement with this expression.

Figure 4 shows the relationship between ψ and estimated bitumen film thickness (EBFT) for three sands. The above formula is much simpler to use and to express graphically than the formula derived by Prandtl:

$$\text{Prandtl bearing capacity} = \frac{c}{\tan \phi}\left[\frac{(\sin \phi + 1)}{(\sin \phi - 1)} . e^{\pi \tan \phi} - 1\right]$$

where c = apparent cohesion
ϕ = angle of shearing resistance from the Coulomb equation.

It was found that all of the sands possessed optimum ψ values at an EBFT of 0.7 to 0.8 microns, given by

$$\text{EBFT} = \frac{x.10^4}{(100-x)S}$$

where x = bitumen content (% wt)
and S = Specific surface area (cm^2/g).

This recent work has also shown that optimum ψ values for different sands are related to Hveem's centrifuge kerosene equivalent (CKE)[14], the angle of shearing resistance measured by the Angle of Repose method[22], and the Efflux Rate[22] of the dry sand. Results for three sands are plotted in Fig 5.

Whilst this study has been by no means exhaustive, the following conclusions relating to sand-bitumen bases for medium to heavily trafficked roads can be tentatively drawn:

1. (a) For natural dry sand, an angle of shearing resistance of at least 30 degrees and/or CKE of at least 1.5 is required, and

   (b) for the sand-bitumen mix, an EBFT of 0.7 to 0.8 microns with a ψ value of at least 1200 kN/m^2 is required at 25°C and rate of strain of 2.22×10^{-2} sec^{-1}.

2. Comparative criteria for the Marshall and Hubbard-Field stabilities are as follows:

   Marshall stability of 300 kg(min) at 60°C, or

   Hubbard-Field stability of 700 kg(min) at 60°C, both accompanied by an EBFT of 0.7 to 0.8 microns.

3.3 *The mix-in-place process*

It is worthwhile perhaps to consider carefully what is involved in any mixing process: the requirement may perhaps be expressed in two parts:

1. the components must be brought together
2. an adequate mixing action must be provided.

The first statement may seem obvious but deserves discussion. The stabilisation process involves natural soil as the major component and the stabilising agent less than 10 per cent of the final product. It should therefore be rational to bring the stabilising agent to the natural sand; for example, in cement stabilisation the stabiliser is very often spread on the soil and mixed in by machine in one or more passes. This type of process, mix-in-place, often requires only simple plant, eg trucks, graders and water bowsers, and high output is possible. There are several drawbacks to this process however; in particular, difficulties are often experienced in controlling bitumen content, completeness of mixing and processing depth. Whilst these difficulties are lessened by the use of purpose-built single-pass stabilisation machines, mechanical failure usually results in complete stoppage of work. For this reason simpler multi-pass equipment has much to commend it.

Single-pass machines require the soil to be spread in a windrow such that it can be picked up by the machine as it travels slowly forward; the stabilising binder is added to the soil and mixed within the machine and is discharged from the rear, usually again as a windrow, ready for subsequent spreading and compaction.

An arrangement used on one contract in the Arabian Gulf consisted of a tractor-mounted mixer equipped with a spray bar above the tines inside the mixing hood and supplied with cut-back bitumen by a tanker which preceded it. A metered quantity of bitumen was thus delivered to the soil under the mixing hood but mixing was incomplete during the first pass: subsequent passes of the machine, without the bitumen supply, and assisted by a blade grader were necessary to complete the mixing process. In its simplest form multi-pass work must often be done using only a bitumen distributor and blade grader.

3.4 *The premix process*

Mix-in-place work is only possible where low viscosity binders, ie cut-backs or emulsions, are to be used. If it is necessary to stabilise with penetration grade binders, premix plant is required, although techniques are now available for stabilising with foamed penetration grade bitumen; Mobil Oil (Australia)[23] describe equipment for both mobile and static processes and Bowering[24] discusses the properties and behaviour of foamed bitumen mixtures.

Several points should be made concerning the role of static premix plants for bituminous stabilisation:

(a) Continuous type mixers, which are often capable of high outputs, are well suited to this type of work, ie production using a cold feed of constant gradation; materials are metered by volume. For stabilisation work the process becomes one of soil and binder only in most cases. Production of uniform quality is therefore more feasible than for surfacing materials, which demand close control. Batch-type mixers are essential for surfacing work and can naturally also be used for stabilisation.

(b) Pre-mixed stabilised material can be laid by paver to uniform depth and regularity; shaping by blade grader is thus cut to a minimum.

(c) Modern mixing plants are often constructed as several mobile units, capable of disconnection and re-assembly within a few hours. Such plants can follow the progress of work, thus keeping haul distances to a minimum.

3.5 *Compaction*

The thickness of bitumen coating in soil-bitumen systems is relatively thin, conferring cohesion rather than providing lubrication, and in this respect such systems resemble dense continuously-graded surfacing materials, such as asphaltic concrete, in which the interlocked mineral particles resist compaction. In common with these materials, soil bitumen is compacted most effectively by the kneading action of rubber or pneumatic-tyred rollers.

NOTE: The deleted text deals with road surfacings.

6. ACKNOWLEDGEMENTS

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TABLE 1

Some features of different bituminous processes and materials

Process/Material	Use	Binder function	Binder viscosity (as constructed)	Aggregate system
Priming	binds surface of new base in preparation for surfacing	sealant	low	nil
Tack coating	provides bond between existing surface and bituminous premix overlay	adhesive	high	nil
Surface dressing	re-sealing re-texturing	sealant and adhesive	low/medium	single-size chippings
Slurry sealing	sealing open/'hungry'/cracked bituminous surfacings	sealant/adhesive	low	dense, very fine
Macadams (includes asphaltic concrete)	bases, surfacings	lubricant/adhesive	medium/high	angular, interlocking (very open - very dense)
Mortar type mixes (includes rolled asphalt)	bases, surfacings	adhesive/sealant	V. high	very dense; may include stone

TABLE 2

Properties of sand-bitumen mixes after 1 year (+) storage at 45°C
Stability tests at 60°C (all compacted densities, CDM, in Mg/m^3)
(Sand: as for construction of experimental sections, Maiduguri-Bama road)

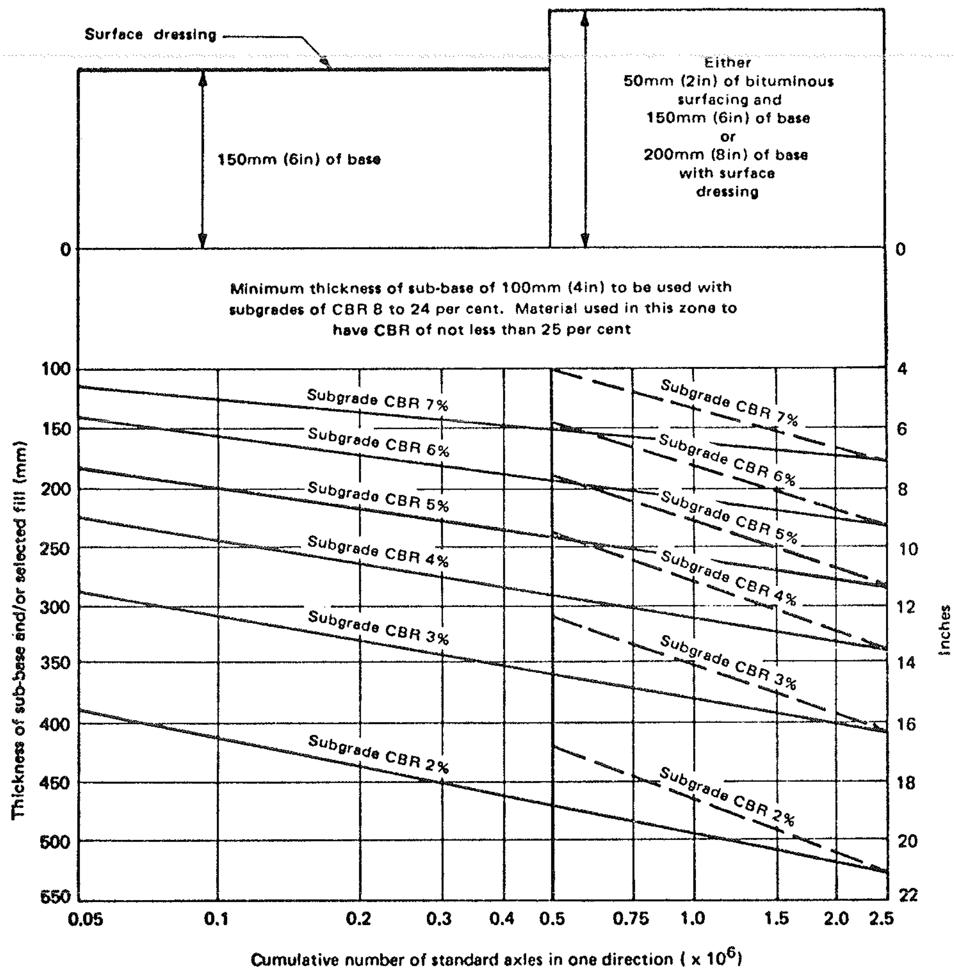
Binder	% wt	Cone stability				Deformation wheel tracking test		Hubbard-Field		Marshall			Properties of recovered binder		
		S	m	y	CDM	Rate of deformation (mm/min)	CDM	Stability (kg)	CDM	Stability (kg)	Flow (mm)	CDM	Penetration at 25°C	Viscosity (absolute) (poises x 10^4) at 45°C	Viscosity (absolute) (poises x 10^4) at 60°C
MC2	3	2.7	1.8	0.9	1.91	0.036	1.91	293	1.93	206	2.0	1.87	92	5.89	0.66
	4	2.2	1.3	0.9	1.92	0.098	1.92	304	1.93	170	1.8	1.89	88	16.2	0.12
S.125	3	2.7	1.8	0.9	1.91	0.036	1.91	413	1.92	337	2.0	1.89	58	13.6	2.2
	4	2.4	1.5	0.9	1.92	0.038	1.92	468	1.93	327	2.0	1.91	65	18.5	2.2
80/100 pen	3	1.7	0.9	0.8	1.90	0.094	1.90	343	1.91	256	2.0	1.88	44	16.7	4.0
	4	1.6	0.8	0.8	1.92	0.041	1.92	333	1.93	219	2.0	1.91	42	16.0	1.8

TABLE 3

Hubbard-Field/Marshall stability ratios and cone stability/equivalent tyre pressure (imperial) relationships; uncured and cured material; tested at 45°C

(Sand: as for construction of experimental sections, Maiduguri-Bama road)

Binder	Uncured						Cured 1 year (+) at 45°C					
	Binder content (% wt)	Hubbard-Field (kg)	Marshall (kg)	Hubbard-Field/ Marshall ratio	Cone stability (S) (kg/cm^2)	Equivalent tyre pressure (lb/in^2)	Binder content (% wt)	Hubbard-Field (kg)	Marshall (kg)	Hubbard-Field/ Marshall ratio	Cone stability (S) (kg/cm^2)	Equivalent tyre pressure (lb/in^2)
MC2	3	SPECIMENS UNSTABLE			2.9	41	3	671	382	1.76	11.0	156
	4				3.0	42	4	578	421	1.37	8.9	126
S125	3	198	79	2.51	3.1	44	3	587	473	1.24	9.6	136
	4	197	44	4.48	3.4	48	4	637	425	1.49	9.0	128
80/100 pen	3	420	232	1.81	2.7	38	3	507	384	1.32	7.9	112
	4	517	192	2.69	3.4	48	4	586	391	1.50	6.0	85



If it is desired to provide at the time of construction a pavement capable of carrying more than 0.5 million standard axles, the designer may choose either a 150mm (6in) base with a 50mm (2in) bituminous surfacing or a 200mm (8in) base with a double surface dressing. For both of these alternatives, the recommended sub-base thickness is indicated by the broken line.

Alternatively, a base 150mm (6in) thick with a double surface dressing may be laid initially and the thickness increased when 0.5 million standard axles have been carried The extra thickness may consist of 50mm (2in) of bituminous surfacing or at least 75mm (3in) of crushed stone with a double surface dressing. The largest aggregate size in the crushed stone must not exceed 19mm (¾in) and the old surface must be prepared by scarifying to a depth of 50mm (2in). For this stage construction procedure, the recommended thickness of sub-base is indicated by the solid line.

Fig. 1 PAVEMENT DESIGN CHART FOR FLEXIBLE PAVEMENTS

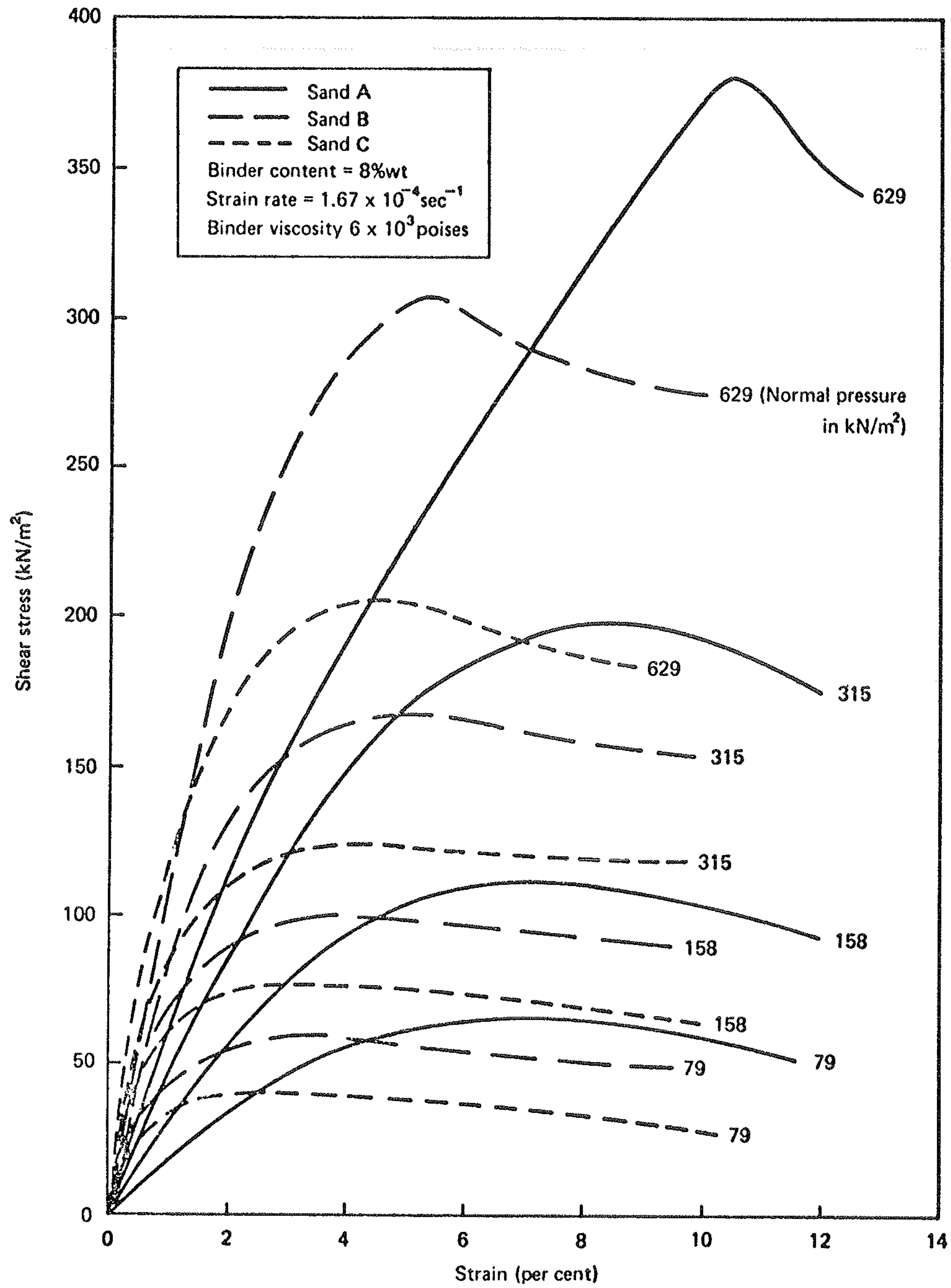


Fig. 2 TYPICAL STRESS/STRAIN RELATIONSHIPS FOR SAND/BITUMEN IN SHEAR BOX TEST

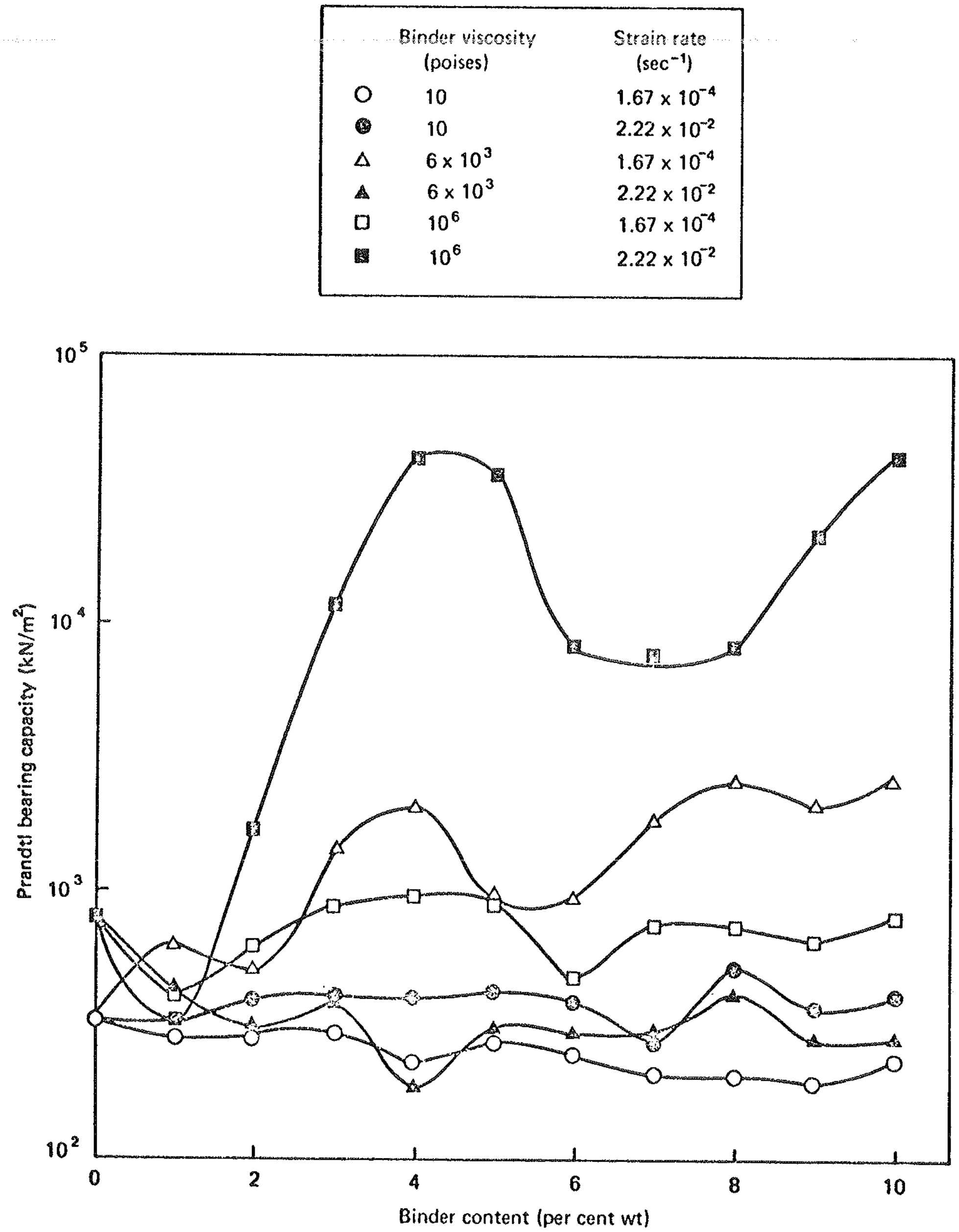


Fig. 3 PRANDTL BEARING CAPACITY VALUES FOR TYPICAL SAND/BITUMEN

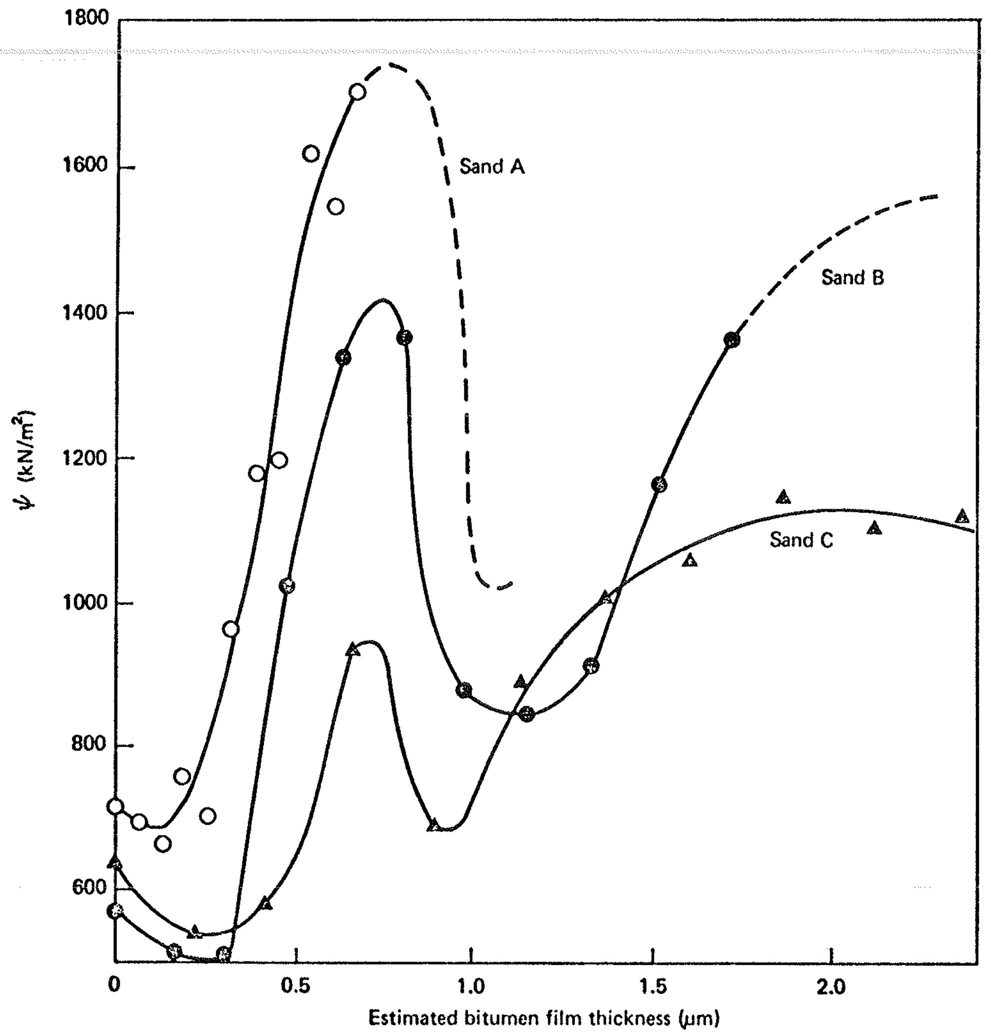


Fig. 4 RELATIONSHIP BETWEEN EBFT AND Ψ AT 2.22×10^{-2} SEC^{-1}, 10^6 POISES

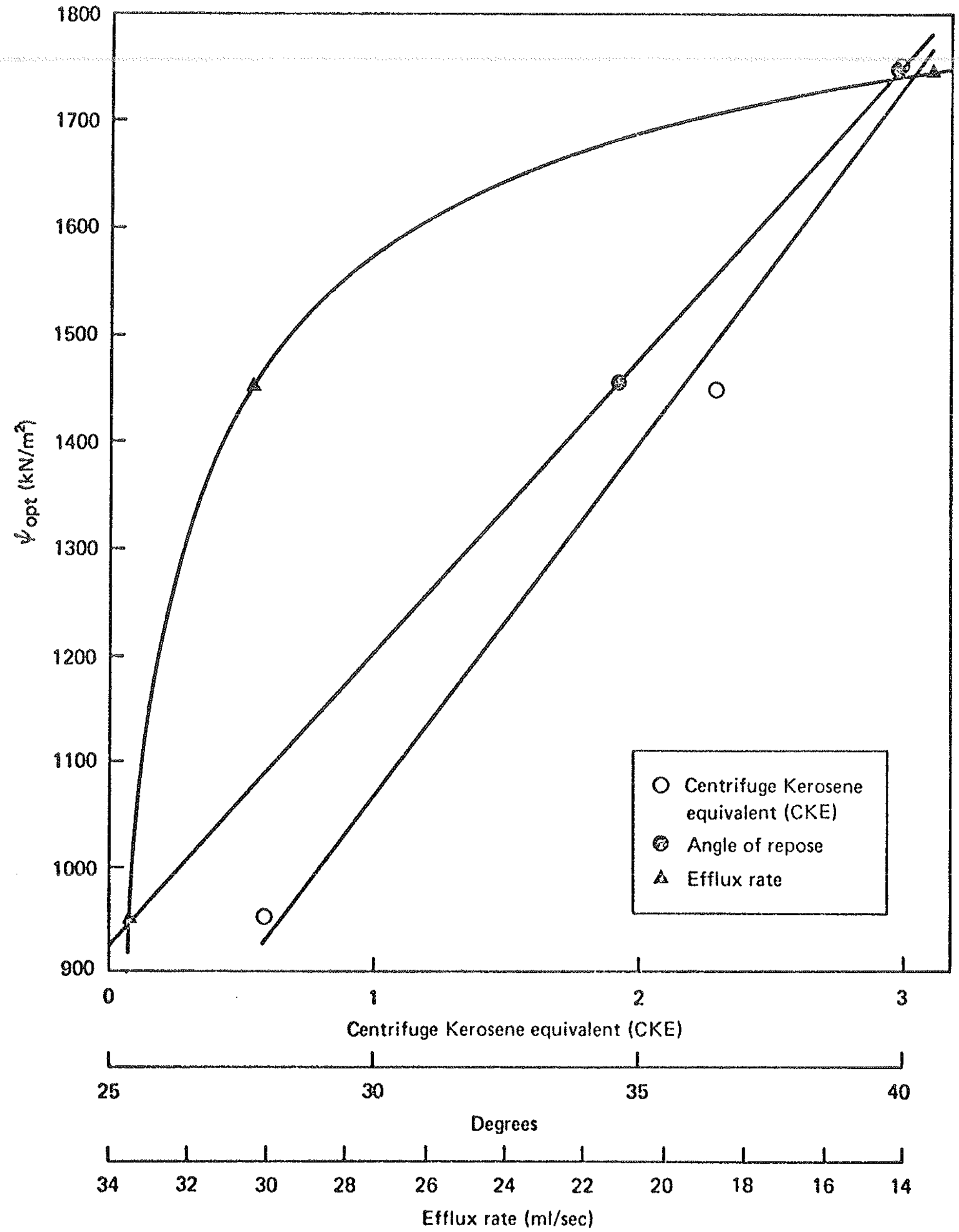


Fig. 5 RELATIONSHIP BETWEEN Ψ_{opt} AND SOME PHYSICAL CONSTANTS OF THREE SANDS (DRY)

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Contents

Performance Study of Asphalt Road Pavement With Bituminous-Stabilized-Sand Bases

Claude P. Marais and Charles R. Freeme, National Institute for Transport and Road Research, Council for Scientific and Industrial Research, Pretoria, South Africa

The possibility of using the windblown sands that occur in the northern areas of South West Africa for the construction of all-weather roads to carry heavy truck traffic has been investigated. Laboratory investigations and field trials in Pretoria, South Africa, showed that bituminous stabilization of these sands was promising, and a full-scale road experiment to test a limited number of bases of bituminous-stabilized sand was constructed in the homeland of Owambo, South West Africa. This paper describes the laying of the experiment and the construction techniques and control measures used. A new technique that establishes the optimum time for the compaction of a cutback bituminous-stabilized sand mixture after aeration by using a vane shear apparatus is described. The vane shear apparatus was also used to measure the in situ shear strengths of the various experimental bituminous-stabilized sand bases after compaction and during service; the results of these measurements, together with performance data after 8 years service with respect to deformation and cracking, are discussed. Laboratory and field studies are described and predictions about the performance of a bituminous-stabilized sand base under varying traffic conditions are made by using the best known techniques available at this time.

Vast areas of the southern subcontinent of Africa are covered with a deep blanket of aeolian sand. Probably these sands were originally derived from preexisting sedimentary rocks in the general area and first emplaced by wind during the lower most Pleistocene epoch (approximately 2 000 000 years ago). They were subsequently redistributed by wind and water during the Pleistocene; the latest major redistribution was brought about by wind action, probably some 10 000 to 15 000 years ago, although some minor redistribution is still occurring (1).

Because of their widely spread occurrence, these sands, apart from various types of calcrete (caliche), are sometimes the only natural building material available to the civil engineer. From an economic point of view, they are therefore extremely important and have been studied for use in concrete structures, building construction, and, more recently, pavement construction by the National Institute for Transport and Road Research (NITRR) of the Council for Scientific and Industrial Research in Pretoria, South Africa (2, 3, 4).

This paper describes the use of these aeolian sands as the base layer of a road pavement in the recently proclaimed homeland of Owambo, in the northern part of South West Africa (SWA). It discusses the performance results of the experiment and relates these to the probable performance that might be expected under much heavier traffic on a normal freeway.

The accelerated development of the infrastructure of Owambo during the past decade necessitated upgrading the existing gravel road linking Owambo to the more developed, southern part of SWA to an all-weather, 8-m-wide, black-topped facility. The construction of the R60 000 000 ($84 000 000) hydroelectric facility at Ruacana Falls and other major building schemes in Owambo have resulted in a significant increase in heavy freight vehicles using this, the only surface transportation route to the south.

Initial laboratory work by the NITRR in the early 1960s showed that the most suitable method of improving the engineering properties of the in-place sand was to blend it with 15 percent calcareous filler (mechanical stabilization) and then to bind the blend with a bituminous binder. Both the hot-mix and cold wet-mix processes were studied; the latter was adopted as the more practical because of the length of road required and the problems associated with the establishment of a hot-mix facility in this remote area.

After extensive preliminary research into the wet-sand process of bituminous stabilization of fine-grained wind-blown sands, a full-scale road experiment was carried out in May 1965 in Owambo to test the techniques developed during the preliminary study (3, 4).

DETAILS OF EXPERIMENT

The experiment was designed and constructed with the following objectives:

1. To demonstrate in the field the feasibility of in situ bituminous stabilization of sand by using cutback binders and a cationic bitumen emulsion;
2. To investigate the stability and durability, under the traffic conditions and climatic environment of the site, of various bituminous-sand mixtures containing cutback bitumens, a cutback tar, and a cationic bitumen emulsion at binder contents considered suitable from

laboratory work and previous experimental trials;

3. To investigate the effect on stability of adding a proportion of calcareous filler to the natural sand before stabilization;
4. To investigate the effect of laying the bituminous-stabilized sand mixtures over a compacted sand-clay subbase of low strength [California bearing ratio (CBR) approximately 30 percent];
5. To investigate the relative performances of 76.2 and 152.4-mm compacted layers of bituminous-stabilized sand bases;
6. To obtain data on the temperature distribution throughout a bitumen-sand mixture on the road under the climatic conditions of Owambo; and
7. To study the setting up of bitumen-sand mixtures on the road over a period of 2 years.

The experimental pavement was constructed in Owambo from April 27 to May 26, 1965, on the alignment of the road route from Oshivelo to Oshakati. The experiment consisted of eighteen sections each 91.4 m long by 7.3 m wide (width of carriageway). The stabilized sand bases were supported by 3.7-m-wide shoulders on either side of the carriageway. The layout of the experiment with details of the base compositions and compacted thicknesses is given in Figure 1. All of the experimental sections, with the exception of sections 1 and 2, were laid over a firm foundation comprising a number of layers of approved subbase and base material consisting of silcrete and calcrete to which binding material was added. These layers were designed to obviate any deformation distress below the bituminous-sand base layer.

The area in the vicinity of the experiment consists of flat to slightly undulating plains with shallow, localized depressions that have been prospected for calcrete (5). As distinct from river channels, there is a network of shallow watercourses (oshanas). These watercourses drain the level country and do not reach the sea. The center of the drainage system is the Etosha pan. During years of good rainfall, large quantities of slowly flowing water are carried southward along this network of oshanas; this phenomenon is of great importance for the underground water of Owambo (6). The annual rainfall of the area is approximately 500 mm, but it is very variable and results in frequent droughts. Rain falls during a period of less than 60 d/year, mainly during January to March. The area is about 1000 m above sea level, has a subtropical climate, and is composed of open grasslands with scattered palms (Hyphaene ventricosa) in various stages of development in the area near Ondangwa; it changes to Mopani veld near the experimental site. In this region, Mopani (Colophospermum mopani) displays an interesting tendency to form large copses of even-sized trees varying from scrub bush 1.2 m high to large trees 6 to 12 m high (7).

Since the opening of the experimental sections, the traffic pattern has increased significantly, as was anticipated because of the large development program initiated in Owambo at about the same time as the experiment. The table below gives the traffic counts recorded over the 8-year period and the calculated equivalent 80-kN axle loads per day (8).

Date of Survey (year)	Vehicles per Day	Equivalent 80-kN Axles per Day
1965 to 1967	100	4
1971	147	6
1972	195	8
1973	226	9

This traffic is light by normal standards in developed countries, where the equivalent 80-kN axle loads per day would be expected to vary between 200 and 500 for freeways carrying medium to medium-heavy traffic.

MATERIALS

Windblown Sand

The local sand used for the stabilization was cohesionless and had an average particle-size distribution as given in Figure 2. The particle shape, as seen by microscopic examination, can be described as subrounded to subangular with relatively few well-rounded grains. The color is grey-white to a light reddish-brown caused by iron oxide stains on the grains. In the dry state, the sand has very poor inherent stability. Its apparent relative density is 2.60.

Blend of Windblown Sand and Calcareous Filler

The calcareous filler added to the windblown sand was selected from a natural powder-calcrete deposit in the vicinity of the experimental site. Its maximum particle size was generally 4 mm and approximately 50 percent passed a 0.074-mm sieve (Figure 2). The properties are as follows:

Property	Value
Liquid limit, %	50.9
Plasticity index, %	17.0
Linear shrinkage, %	8.0
Apparent relative density	3.61

The blend of windblown sand and 15 percent (by volume of dry sand) of calcareous filler was nonplastic, with an average particle size distribution as shown in Figure 2.

Bituminous Stabilizers

The following bituminous binders were used:

1. Cutback bitumens—A special cutback bitumen manufactured from an 80 to 100 penetration bitumen (MX) and a cutback bitumen manufactured from a 40 to 50 penetration bitumen and cutback to an intermediate grade of rapid-to-medium cure (250) were used. Both were straight-run bitumens, refined in South Africa from Middle East crudes (9, 10). The nominal binder contents were 4.0 and 6.0 percent (by mass of dry sand) for the 152.4-mm-thick compacted bases and 6.0 and 8.0 percent for the 76.2-mm-thick compacted bases (MX only).
2. Cationic bitumen emulsion—The cationic bitumen emulsion used was manufactured from an 80 to 100 penetration bitumen that was fluxed with 10 percent (by mass of emulsion) of a fluxing oil having a boiling point of 160°C. The base bitumen was straight-run and refined in South Africa from a Middle East crude. The nominal binder contents were 4.5 and 6.5 percent.
3. Cutback tar—The cutback tar was manufactured from a high-temperature coke-oven tar and cut back to a 30 to 35°C Evt grade. The nominal binder content was 6.0 percent.

All of the binders were tested with the windblown sand only and with a blend of wind-blown sand and 15 percent calcareous filler. The results of laboratory tests on samples of the binders used are reported elsewhere (4).

Borehole Water Used for Wetting Sand and Sand-Calcrete Filler Blend

The local borehole water used for wetting the sand and the sand-calcrete filler blend before the addition of the bituminous stabilizers had a total dissolved solids content of 52 800 ppm with an alkalinity of residue as carbonate of 3980 ppm.

TECHNIQUES OF CONSTRUCTION

A working platform was constructed to correct the longitudinal and transverse levels and ensure that, once the stabilized sand base was laid, the final road profile would be in accordance with the design requirements. Two 3.66-mm-wide shoulders were constructed to give a 7.32-m-wide trough into which the sand could be spread before treatment. The full experimental length of working platform was primed with a medium-cure (30) cutback bitumen at a rate of 0.72 L/m^2.

In situ CBR measurements were made on the primed platform. The length of platform constructed with calcrete had an average value of 142 percent (dry), and that constructed with sand-clay had an average value of 78 percent (dry) and 28 percent after 24 h soaking.

Where a blend of sand and calcareous filler was required, the correct quantity of sand was spread over the section, and then the calcareous filler was spread uniformly over the sand layer with a mechanical gritter. The sand and calcareous filler were mixed with a disc harrow and a motor grader to form a homogeneous blend.

Watering of Sand and Sand-Filler Blend Before Addition of Binder

The laboratory work carried out before the beginning of the experiment showed that water in the fine sand aids in coating of the sand particles by the cutback binder. It also showed that an initial excess of fluids gave a mixture that, after a specified period of aeration, had the highest density and shear strength when compacted (11).

For these reasons, the moisture content of the sand was increased to approximately 10 to 12 percent before stabilizing with binder. At this range of moisture content, after arbitrary compaction of the wet sand with a pneumatic-tired vehicle, the inherently poor stability of the dry sand improved to such an extent that the stabilization plant was able to move over the sandbed at the required speed without undue slippage.

The watering was done with a water tanker fitted with a gravity-feed spray bar. The moisture content of the sand was controlled by a nuclear gauge that was very useful in obtaining a rapid result. After the required quantity of water had been added, motor graders mixed the water into the sandbed to uniformly distribute the moisture throughout the sand layer.

Heating of Binders

The binders were supplied in drums. The contents of the drums were transferred to four 2300-L binder heaters, which were fired with liquid petroleum gas.

The heating of cutback binders is a fire hazard, and because of this, the temperature to which the cutback bitumens were raised in the binder-heating tanks was limited to approximately 5°C below that required for spraying. This procedure was satisfactory, and no fires occurred.

The heated binder was then pumped into a 4500-L distributor and further heated to a temperature that gave a Saybolt-Furol viscosity of 30 to 40 s, which was satisfactory for spraying.

Spraying and Mixing of Binder Into Wet Sand

The stabilization train consisted of a tractor, pulvimixer, and 1370-L binder-storage tanker. The 4500-L distributor pumped hot binder into the storage tanker during the stabilization process, thus enabling the work to proceed with a minimum of delay.

The binder was sprayed through a specially fitted spray bar located in front of the rotor of the pulvimixer. The depth of cut of the pulvimixer rotor blades was set

Figure 1. Diagrammatic layout of bitumen-sand stabilization experiment in Owambo, South West Africa.

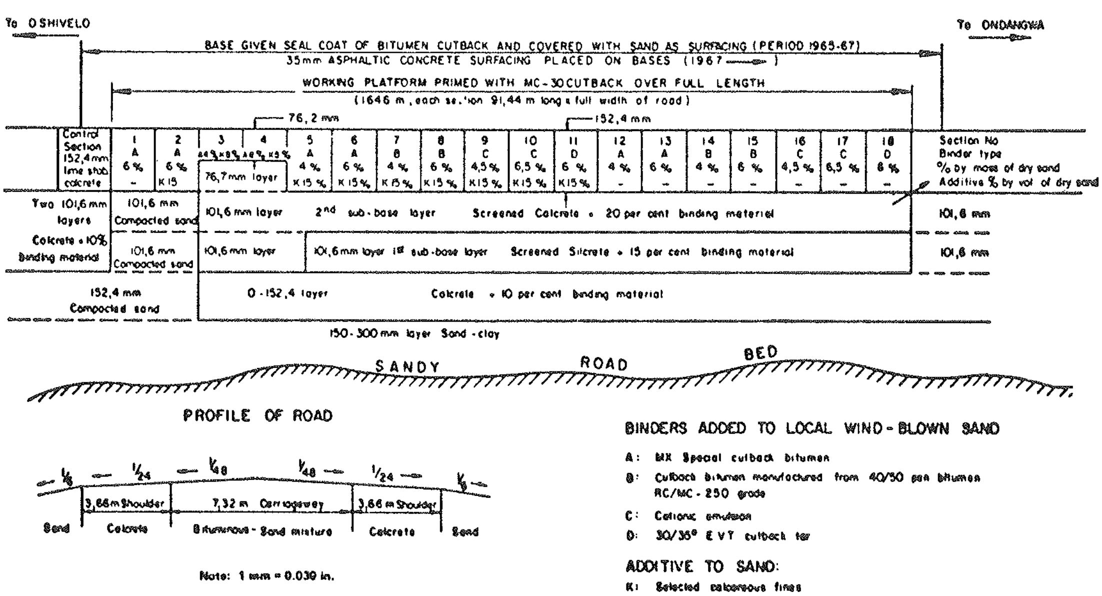
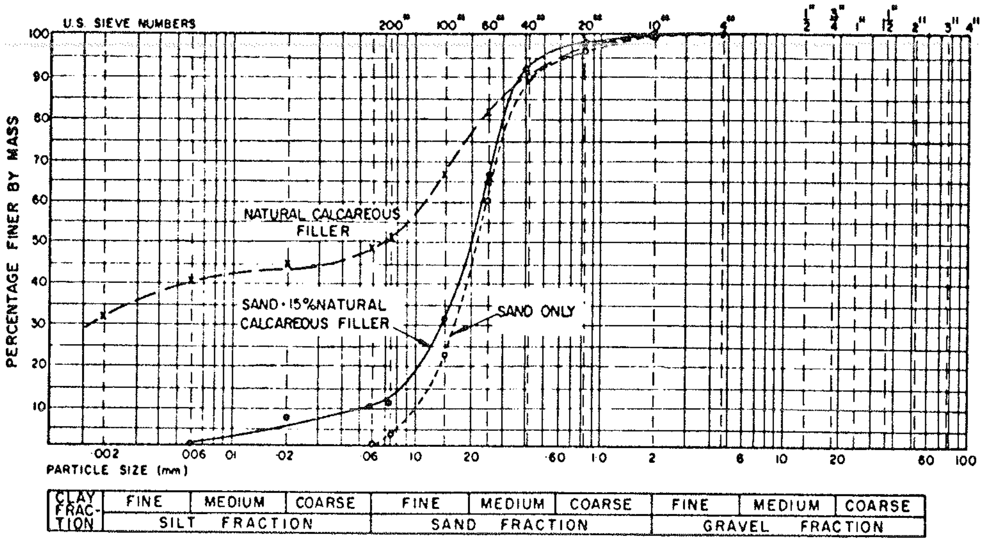


Figure 2. Grading analysis of materials.



to just touch the working platform, thus ensuring that the full depth of sand layer was mixed with the binder. The spray bar nozzles were adjusted so that the hot binder was sprayed just ahead of the rotor to obtain an intimate mix while the binder was still in a relatively fluid state. The distribution of the binder after this spraying pass was inadequate, and a second mixing pass was necessary to improve the overall distribution of the binder through the sand mass.

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A single pass of the pulvimixer covered a width of approximately 2 m, which made five passes over the full width of the carriageway necessary to give an adequate overlap between successive passes.

Control of the quantity of binder introduced was achieved by calibrating the spray bar to give a known output at a particular pump speed and by operating the train at the forward speed required to give the desired binder content. The relative density of the hot binder was used to convert the amount sprayed from a volume to a mass basis. The forward speed of the mixing train was controlled by charts relating the true forward speed to the various tractor gears and engine revolutions that could be selected. The times taken over measured distances during each mixing run were determined with a stopwatch and, where necessary, adjustments were made to the forward speed. This method of speed control gave very satisfactory results.

At the outset of the experiment, it was intended to dilute the cationic emulsion with 30 percent (by volume of emulsion) of water so that it could be sprayed in a cold condition because a trial experiment had shown that the best distribution and coating were obtained with this dilution (after a spraying and mixing pass of the pulvimixer alone). The highly alkaline water available on the site, however, precluded any form of dilution because the acidic emulsifier reacted immediately with the water, which caused the emulsion to break. The viscous, cationic bitumen emulsion was therefore heated to lower its viscosity and sprayed in the same manner as the cutback binders.

Mixing of Stabilized Sand With Motor Grader

An inspection of the stabilized sand after the mixing operation with the pulvimixer showed that the coating of the sand particles was still not adequate. Further mixing with the pulvimixer would have delayed the progress of the work and so mixing by means of a motor grader was begun as soon as possible after the pulvimixer had completed the last mixing run. The high shearing action of the motor blade, producing a spiraling motion of the material during the cutting operation, was very effective in improving the coating of the sand particles. There was a significant improvement in the appearance of the mixture after each passage of the grader, and adequate coating resulted after two movements of the material from one side of the carriageway to the other.

Finally, the grader leveled the now homogeneous mixture to an even, loose thickness.

Aeration of Mixtures

The laboratory control measures used during this experiment indicated that aeration was essential to obtain the high stabilities required for the traffic that would use the road.

Controlled aeration was therefore carried out with a disc harrow pulled by a pneumatic-tired tractor. The discs were set to cut to the full depth of the loose layer. During aeration, the resistance of the mixture to the movement of the discs increased; this became evident when the tractor required more power to maintain a constant forward speed.

The aeration was continued until laboratory tests showed that the mixture had reached the condition at which maximum stability would be obtained on compaction.

Compaction of Mixtures

The most satisfactory compaction plant was a 30-Mg pneumatic-tired compactor (fully ballasted), used with a sheepsfoot roller. The sheepsfoot roller was fitted with a cleaning device to avoid excessive pickup of material during compaction. The individual feet measured

21 by 13.5 cm and were arranged in 24 rows of four, around the periphery of each roller drum. If it is assumed that four feet made contact at one time, the pressure per foot was 1.31 MPa when the roller drums were fully ballasted.

Various methods of compaction were tried, but since they all resulted in similar densities, the most practical method was chosen. This consisted of compacting in approximately 50-mm lifts by using the pneumatic-tired compactor and the sheepsfoot roller while a motor grader spread uncompacted material over the already compacted layer. It is important to obtain a good bond between the successive lifts, and the impressions left by the sheepsfoot roller assisted in this. Compaction was achieved by a continuous operation with a final leveling of each section with the motor grader. Each section was given a minimum of eight complete coverages with each compactor.

The sections containing calcareous filler and sand were compacted to such high stabilities that the motor grader had difficulty in trimming the material to the final profile required and, in some cases, an imperfect finish resulted. This difficulty was not experienced where only sand was stabilized, as these mixtures were more workable.

On completion of the experiment and before opening the sections to traffic, in situ density measurements of each section were made in duplicate by the sand replacement method. The densities obtained on the 152.4-mm-deep sections were fairly consistent, varying generally between 1794 and 1826 kg/m^3. Sections 3 and 4, which were 76.2 mm deep, however, had densities of between 1700 and 1715 kg/m^3, i.e., significantly lower than those for the 152.4-mm-deep sections.

Surfacing and Opening of Sections to Traffic

On completion of the stabilized base sections, the type of surfacing to be used to protect the base material was considered. Because of the high stability of most sections, it was decided that initially only a light sand seal should be provided; the position could be reviewed if the traffic caused serious rutting of the weaker sections.

Sufficient cutback bitumen binder was available on site for all of the surfacing and so a blend of equal quantities of the two types of cutback bitumen used for the stabilization work was used for the sand seal. The rate of application varied between 1.37 and 1.76 L/m^2. The application of the sand to the binder film was delayed to enable the cutback binder to penetrate into the stabilized sand base and to allow the viscosity of the binder remaining on the surface to increase, which gave a more stable surfacing layer. The same wind-blown sand that was used for the base stabilization was used for the surfacing and was spread at approximately 0.0065 m^3/m^2 and then well-rolled with a 15-Mg self-propelled pneumatic-tired roller.

Traffic was not allowed onto the sections until June 21, 1965, so that the sections laid toward the latter part of the experiment could set sufficiently and thus not be at a disadvantage during the initial trafficking. This light surfacing was overlaid with a 30-mm thick asphalt-concrete surfacing in mid-1967; it was therefore in service for a period of 2 years.

Placing of Level Pegs for Future Observations

Purpose-made pegs were placed in lines transversely across each section at distances of 30.5 and 61 m from the beginning of each section. The pegs were spaced at 304-mm intervals.

Level observations were made before trafficking and at regular intervals after trafficking. A number of permanent bench marks were installed so that all precise level observations could be compared in relation to a common datum. When the asphalt-concrete surfacing was placed in 1967, the level pegs were replaced in exactly the same positions as the original pegs and releveled.

LABORATORY CONTROL DURING LAYING OF EXPERIMENT

Sampling Mixtures

Representative samples of the mixed material were taken with a sampling tool that consisted of a 75-mm-diameter thin-walled aluminum tube that was introduced vertically into the loose mixture to make contact with the firm working platform. On withdrawal, the mixture remained in the tube and could be extruded into a suitable container. Samples were taken at the following times: (a) after blade mixing (before the beginning of the aeration) and (b) before compaction (after the aeration was complete). These samples were analyzed for the residual binder and fluid contents, and generally close agreement with the designed binder content was found.

A certain amount of intermixing of material from adjacent sections was unavoidable during construction; the first and last 15 m of each section were therefore not sampled.

Field Test to Establish When to Compact Mixtures

In the field, the strength of the mixtures produced could be evaluated by a vane shear apparatus. These tests also made it possible to establish when a particular mixture was in the optimum condition for compaction (11).

Accelerated Aeration of Field Sample

The technique followed was to first obtain a representative sample from the road as soon as the blade-mixing operation was complete. This sample was then divided into at least eight portions, each weighing approximately 6 kg. Each portion was placed in a tray of approximately 1-m^2 area and left to aerate in the sun. Gentle agitation of the mixture in the trays accelerated the aeration process. The material from each tray was tested after various periods of aeration by the following procedure:

1. The mixture from the tray was compacted in a CBR mold under modified AASHO compaction.
2. After compaction, the wet density of the compacted material was determined.
3. The compacted sample in the mold was introduced into the vane shear apparatus, which was fitted with a special base plate to retain the CBR mold.
4. The shear strength of the mixture was determined with the vane shear apparatus, and the temperature of the mix was measured at the middepth of the vane (Figure 3a).
5. After shearing, a representative sample of the mixture from the mold was analyzed for fluid content by evaporating the volatile oils and water.

The binder content of the mixture tested was determined from field measurements. The constitutions of the cutback binders and emulsion were known from previous laboratory tests in terms of residual binder and volatiles (oils or water) on a mass basis. From these data, it was possible to calculate the dry density of the compacted bitumen-sand mixture.

The dry densities and the vane shear strengths converted to 40°C were then plotted against the fluid contents. Typical results for mixtures with 6.0 percent (nominal) cutback bitumen (80 to 100 penetration base) MX, sand, and 15 percent calcareous filler (section 6) are given in Figure 4. Both the density and the shear strength pass through maximum values. To achieve maximum stability of the mixture, the highest possible shear strength and density should be obtained when compaction takes place. However, the peak of the shear strength curve always occurs on the dry side of the maximum density, so that it is not possible to obtain both maximum shear strength and maximum density at a particular fluid content. Eighty to 90 percent of the maximum shear strength and 95 to 100 percent of the maximum density were chosen as a suitable compromise for the sand used, and this criterion was used to establish the time when compaction of the mixture on the road should take place.

Field Aeration Control and Subsequent Compaction

Control of the aeration process on the road was achieved by taking regular samples from the road mix and testing these for shear strength and density. The shear strengths were converted to 40°C values by using factors determined from laboratory and field tests. The values for section 6 shown in Figure 4 indicate the good correlation between field and accelerated test values. The fluid content at which compaction was begun on the road is also shown.

Temperature Records at Experiment Site

A clockwork temperature recorder was installed during the experiment and again later for obtaining long-term temperature records at the site. Thermocouples were

Figure 3. Measurement of vane shear strength of bituminous-stabilized sand: (a) in field laboratory and (b) on pavement.

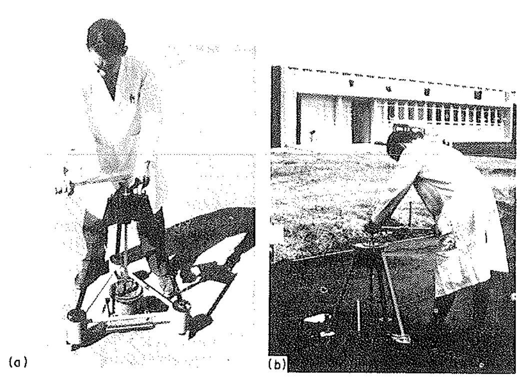

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Figure 4. Dry density, vane shear strength, and fluid content relationships (section 6).

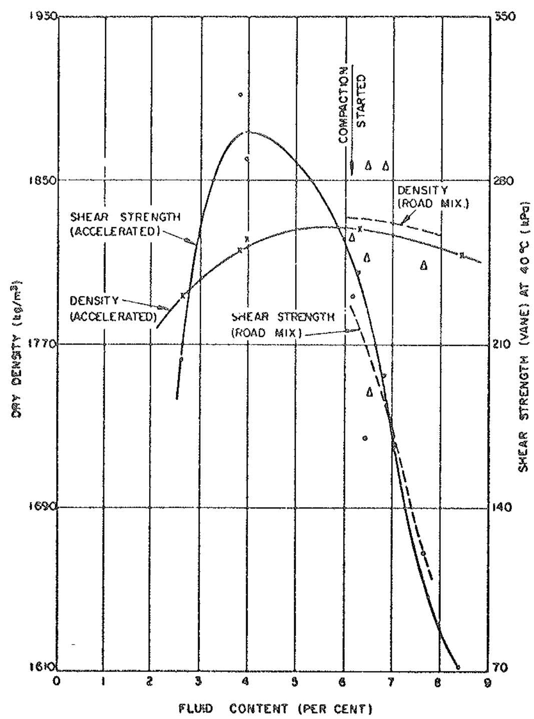


Table 1. Vane shear strength of experimental bituminous-stabilized-sand bases.

Stabilizer	Section No.	Binder Content (%)	Filler Content (%)	Vane Shear Strength (KPa) at 40°C After: Compaction	1 Year	2 Years
MX cutback bitumen	12	4.0	0	80	230	305
	5	4.0	15	125	346	415
	1	6.0	0	60	185	224
	13	6.0	0	60	178	255
	2	6.0	15	64	242	315
	6	6.0	15	90	286	405
Cutback bitumen (rapid-to-medium cure (250))	14	4.0	0	68	240	318
	7	4.0	15	230	373	495
	15	6.0	0	54	210	305
	8	6.0	15	175	255	390
Cationic bitumen emulsion	16	4.5	0	50	170	235
	9	4.5	15	168	280	345
	17	6.5	0	25	180	242
	10	6.5	15	130	280	314
Cutback tar (30 to 35°C Evt)	18	6.0	0	44	150	226
	11	6.0	15	98	380	555

placed at depths of 0.75, 150, and 300 mm below the surface of section 6. Maximum and minimum road surface temperatures on a hot summer day in February 1966 were 70 and 24°C. On the same day, the maximum and minimum temperatures were 52 and 34°C at a depth of 150 mm.

FIELD STUDIES

Setting Up of Bituminous-Stabilized Sand Mixtures

To study the setting-up pattern of the bituminous-stabilized-sand mixtures, shear strength measurements were made on the sections with the vane shear apparatus (Figure 3b). These measurements were carried out at intervals after completion of the work, up to the time of laying the asphalt-concrete surfacings (1967), and all vane shear strengths were converted to values at 40°C (standard temperature).

The vane shear strengths of the mixtures for various periods of time are given in Table 1. All of the mixtures gained significantly in shear strength with time. This gain in strength was fairly linear over the 2-year period. On the average, the mixtures containing sand only increased in shear strength by a factor of approximately 5.3 and those with sand and calcrete filler by a factor of approximately 3.6, i.e., annual increases in shear strength of 225 and 125 percent respectively.

The change in binder content did not have a significant effect on either the shear strengths of the various mixtures or their rate of gain in strength with time. However, the addition of 15 percent powder calcrete to the sand had a significant effect on the shear strengths of the mixtures; mixtures with calcrete filler had shear strengths from 1.1 to 3 and about 1.5 times those of the sand-only mixtures after compaction and after the 2-year period respectively.

Structural Tests

Various structural tests were carried out on a selected number of these experimental sections toward the beginning of 1966; those results were reported by Gregg and others (13) and will therefore not be covered in this paper.

Deformation Measurements

The precise level observations for the 8-year service period have shown that, significantly, the deformation that occurred during the 6-year period after the laying of the 30-mm-thick asphalt-concrete surfacing was very much less than the deformation that occurred during the preceding 2-year period, particularly in those sections that deformed excessively (>10 mm), viz., sections 16 and 18.

To investigate whether there was any relationship between the laboratory, vane shear strengths of the mixtures at the time of construction and the subsequent permanent deformation (the average maximum rut depth between the inner and outer wheel tracks) measured after an 8-year in-service period, these results were plotted as shown in Figure 5. These data gave a reasonable envelope of laboratory, vane shear strength versus rut depth. The data also indicated which mixtures were satisfactory, critical, or unsatisfactory with respect to

Figure 5. Relation between vane shear strength and deformation under traffic after 8-year service of bituminous-stabilized-sand bases.

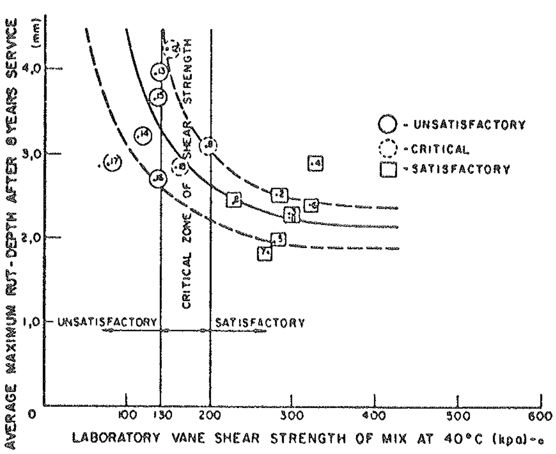


Figure 6. Crack indexes of sections.

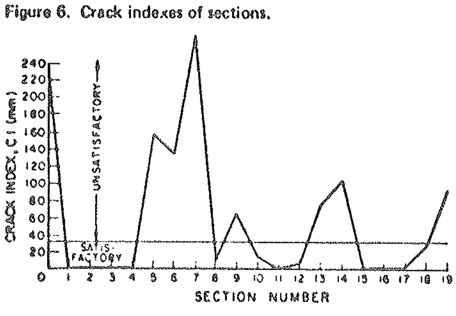


Figure 7. Extrapolation of measured rut depths to higher axle-load regimes.

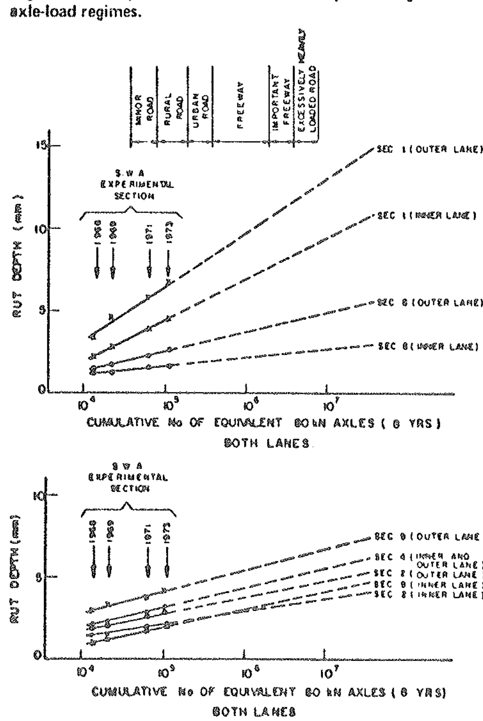
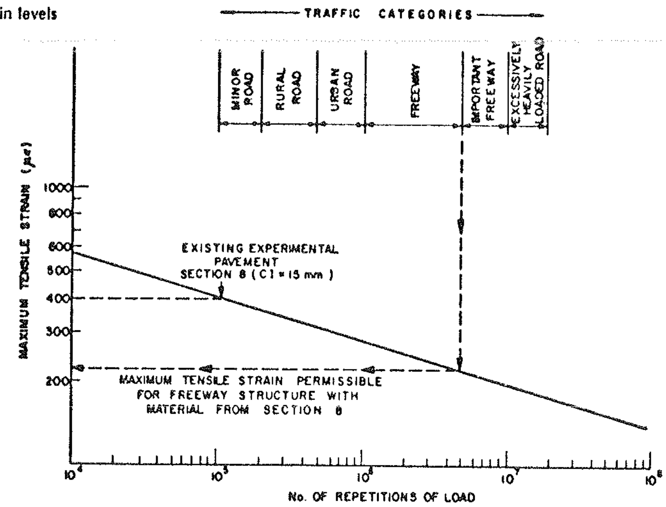


Figure 8. Prediction of maximum permissible tensile strain levels for different traffic categories.



permanent deformation: Those with laboratory, vane shear strengths between 130 and 200 kPa were regarded as critical, those above 200 kPa as satisfactory, and those below 130 kPa as unsatisfactory. The major external factors affecting permanent deformation on a particular bituminous material are traffic, rate of loading, and temperature, and it is not a simple matter to extrapolate the results from a given condition to a new situation if one or more of these major factors is different. However, techniques have recently become available for this purpose if the material properties are well characterized. These techniques were applied to this experiment in an attempt to predict the deformation likely to occur in similar materials under the heavier traffic conditions of a rural freeway; they are discussed later.

Cracking of Sections

A visual survey of the sections was made in 1973 (after 8 years service) to assess the extent of cracking on the different sections. A crack index (CI) (14) was developed for this purpose where

$$CI = \sum_{\text{all patterns}} \text{(percentage of area covered by cracks) (average width of cracks in mm)} \quad (1)$$

The parameters of the spacing and the length of the cracks are not included in this definition. In the case of block and chicken-mesh cracking the area affected can usually be determined. However, a difficulty arises in the case of longitudinal and transverse cracks: When only a few such cracks appear, engineering judgment must be used to assess their effect on the pavement structure.

The CI gives an indication of the severity and area covered by cracks, regardless of the pattern. The CIs for the various sections, including the control sections (0 and 19) on which a lime-treated calcrete base was used, are shown in Figure 6.

It is interesting to compare the results shown in Figures 5 and 6 and to note the opposite trends in performance of some sections with respect to deformation and cracking; e.g., section 12 has the highest deformation and a very low CI and section 7 has a low deformation and a very high CI. This type of behavior is, of course, expected and points to the problem facing the highway engineer in selecting a suitable material that will give good performance in both deformation and crack resistance. Sections 2, 3, 4, 8, 10, and 11 performed acceptably and may be used with confidence for a pavement carrying similar traffic under a similar climatic environment.

CONCLUSIONS FROM EXPERIMENTAL PAVEMENT

In terms of the objectives of the experiment, it can be stated that

1. It is feasible to construct in situ bituminous-stabilized-sand bases that will give satisfactory performance for a highway pavement by using the following binders: (a) MX special cutback bitumen, (b) cutback bitumen manufactured from a 40 to 50 penetration bitumen [rapid to medium cure (250)], (c) cationic emulsion, and (d) cutback tar (30 to 35°C Evt grade), [if the sand is mechanically stabilized by the addition of 15 percent powder calcrete (calcareous filler)];
2. The two 76.2-mm-thick bituminous-stabilized-sand sections (3 and 4) gave excellent performances and stand out as the most successful sections laid, from the points of view of both economy and performance;
3. The performance of the section containing a blend of calcareous filler and sand laid on a sand-clay subbase (section 2) was acceptable, and for similar traffic conditions, this type of structure is worth serious consideration because it would be economically advantageous; and
4. All of the bituminous-stabilized-sand bases increased in shear strength with time (for the 2-year period that measurements were made).

EXTRAPOLATION OF PERFORMANCE RESULTS TO HEAVIER TRAFFIC CONDITIONS

The design and particularly the construction of existing pavements have already provided a wealth of experience and information. The extrapolation of analyses of existing material to new pavements should not be neglected simply because the test methods used are considered inadequate to characterize the pavement by current standards. In general, extrapolation has not been widely used, primarily because new and more advanced methods of testing and evaluating materials are continually being developed.

An alternative approach is to use the results of in situ tests to characterize existing pavements as well as possible. These data can then be extrapolated to other conditions on the clear understanding that the extrapolations are guidelines with which engineers can supplement their existing knowledge. In this context, existing experimental and prototype test pavements are normally the best to use, because a number of variations are usually incorporated in the experiment and a well-defined program of testing is carried out. The experimental sections described in this paper are an example of this type. One of the main aims of these experimental sections was the determination of the behavior of bituminous-stabilized-sand bases under the environmental and traffic conditions pertaining to this road.

Pavement Deformation

Precise level determinations of permanent deformation were made for each of the experimental sections of the road. These measurements were originally made in 1967 to establish a datum line and then repeated in 1968, 1969, 1971, and 1973. The maximum rut depths in the inner and outer lanes were determined from the measurements. Semilog plots of rut depth versus cumulative number of equivalent 80-kN axle loads for some of the experimental sections are shown in Figure 7.

The amount of heavy traffic on these sections was small, about 0.1 million equivalent heavy axles over the 8-year test period. However, the few heavy axles borne by the section during this period were usually loaded more than the legal limit.

In Figure 7, the rut depth has been extrapolated to a higher number of 80-kN axles by using a linear-log relationship (14). From this relationship between rut depth and number of equivalent 80-kN axles, it appears that the base mixtures used would give reasonably acceptable values of rut depth even under freeway traffic conditions (i.e., 1 million to 10 million equivalent axles).

The conditions under which the extrapolation will remain valid are

1. That no major variations in the environmental conditions occur, other than those that have already occurred over the previous 8 years,
2. That the behavior of the bituminous materials does not go into the shear failure zone, and
3. That the deformation behavior of the bituminous materials used is not as dependent on their stress history as is that of, e.g., unbound granular materials.

The first condition is perhaps the most unpredictable. There are also other unpredictable factors, e.g., cracking of the surfacing and base, which could allow ingress of surface water to the subbase and result in excessive deformation.

Three zones can be identified in the deformation behavior of bituminous-based mixtures to repetitions of load. In zone 1, there is initial deformation, but no further increase; in zone 2, the deformation increases at a constant rate; and in zone 3, excessive shear begins to take place.

The conditions under which zone 3-type deformation will occur can be studied by investigating the cohesion (c) and the angle of internal friction (ϕ) of the material under the appropriate test conditions.

A study of this nature was carried out by using an extreme case of dual wheel loading with double the legal axle load (2 times 40 kN). Under these conditions, the base material should have a c-value greater than 35 kPa and a ϕ-value greater than 27° (at 40°C) to prevent zone 3 deformation. An analysis of the stress states of the 150-mm-thick bituminous layer showed that, except for a small area under the wheel loads at the 150-mm depth, the material remained in the satisfactory zones of deformation. This condition is not considered important from a practical point of view because of the extreme loading conditions chosen.

Fracture (Fatigue Cracking)

It is also desirable to estimate the cracking that can be expected if a similar base mixture is used in another pavement with different loading conditions. At present, the degree of cracking is known for specific sections of the SWA experiment after 8 years of trafficking. Section 8, for example, has a CI of 15 mm, which means that 15 percent of the area has cracks 1-mm wide, after 100 000 load repetitions. This degree of cracking is acceptable even for freeways. However, to achieve this level where there is a greater volume of equivalent 80-kN axles, the maximum tensile strain at the bottom of the base layer must be reduced. The amount by which it must be reduced can be determined from the fatigue behavior of the material, as shown in Figure 8. From knowledge of the tensile strain computed for the conditions at the pavement section (in the case of section 8, 400 microstrains) and of the traffic, a point on the strain-versus-load-repetitions (ϵ-N) line can be established. The slope of the (ϵ-N) line representing the material used for the base was taken to obey the formula given by Brown and Pell (15). This relationship can be determined either by measurement or by the average values used (14, 15). In Figure 8, the average value for the slope of the line was used. In the case of section 8, the maximum permissible strain level is approximately 210 microstrains.

A reduction in strain at the bottom of the base can be achieved either by increasing the modulus of the subbase (i.e., by increasing the CBR or by stabilization) or by increasing the thickness of the base layer. Structural analysis using elastic theory is an effective method of determining the upgrading of the pavement that is necessary to avoid fatigue cracking of the base material.

ACKNOWLEDGMENTS

The investigation was carried out as part of the National Institute for Transport and Road Research (South Africa) approved research program, and the paper is published by permission of the director.

The assistance of the South West Africa Roads Department is gratefully acknowledged.

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Publication of this paper sponsored by Committee on Soil-Bituminous Stabilization.

Bibliography

The following bibliography contains two sets of references. The first set consists of a reference for each selected text that appeared in the preceding part of this compendium. The second set consists of references to additional publications that either were cited in the selected texts or are closely associated with material that was presented in the overview and selected texts. Each reference has five parts that are explained and illustrated below.

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(c) Données bibliographiques

(d) Availability information
(d) Disponibilidad de la información
(d) Disponibilité des documents

(e) Abstract
(e) Resumen
(e) Analyse

Reference 5
A REVIEW OF HIGHWAY DESIGN PRACTICES IN DEVELOPING COUNTRIES

Cron, Frederick W. Washington, DC: International Bank for Reconstruction and Development; 1975 May. 57 p.

Order from: International Bank for Reconstruction and Development, 1818 H Street, N.W., Washington, DC 20433.

The design standards of some 150 highway projects financed by the International Bank for Reconstruction and Development between 1960 and 1970 are reviewed, and areas of agreement between the standards of the 63 countries studied are identified; practical highway standards based on these areas of agreement are sketched for the guidance of planners in developing countries. The roads discussed here, fall into three functional categories: a small group of expressways, freeways and toll roads carrying large volumes of traffic; a very large group of 2-lane highways carrying a wide range of traffic volumes serving both local and long distance traffic; and a smaller group of low-traffic tertiary or special purpose roads existing primarily for land service. Comments are made on the problem of classifying highway standards, and on the comparison of standards. Conclusions regarding standards for the capacity- related elements of design and standards for the velocity-related elements of design (radius of curvature, stopping sight distance, passing sight distance) are discussed, as well as the horizontal and vertical clearances for bridges. The standard live loadings for bridges, the structural capacity of pavements and legal load limits are covered, and conclusions relating to pavement design, design standards for 2-lane highways, incremental development of highways, and levels of service are presented.

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SELECTED TEXT REFERENCES

Reference 1
LOW COST ROADS; DESIGN, CONSTRUCTION AND MAINTENANCE

Odier, L.; Millard, R.S.; dos Santos, Pimentel; Mehra, S.R. London: Butterworths; 1971. 158 p. (Sponsored by UNESCO).

Out-of-print; may be consulted at U.S. Department of Transportation, Library Services Division, Room 2200, 400 Seventh Street, S.W., Washington, DC 20590.

Engineering codes relating to road planning, design, construction, materials, and maintenance for use in the developing countries are presented here. Basic principles of road construction and maintenance policy, including social and economic aspects, master plan and feasibility studies, as well as stage construction, are covered. Traffic and design speeds, design related to vertical alignment and horizontal alignment, and cross-section elements are discussed; design principles for unimproved roads, improved roads, roads with permanent surfaces, and flexible pavements are set forth. The drainage of the road is considered, including the control of erosion and the stability of embankments and cuttings. Defensive measures during wet-weather construction are noted. The location and waterway requirements for bridges and culverts are discussed, and the principal factors to be considered in the design of bridge foundations and structures are indicated. Notes are provided on construction operations and plant that include preliminary and detailed surveys, setting out, earthworks, compaction, quarrying, soil stabilization, bituminous surfacing, and concreting. The discussion of road maintenance distinguishes between short-term, largely manual maintenance and long-term maintenance usually involving the use of mechanical equipment. Methods of estimating costs are outlined and special consideration is given to the choice between manual and mechanized methods or combinations of the two.

Reference 2
SOIL STABILIZATION: A MISSION ORIENTED APPROACH

Epps, J.A.; Dunlap, W.A.; Gallaway, B.M.; Currin, D.D. Soil Stabilization: Asphalt, Lime, Cement. 10 Reports. Highway Research Board, Washington, DC 1971; pp. 1-20 (Highway Research Record Number 351).

Order from: Transportation Research Board, Publications Office, 2101 Constitution Avenue, N.W., Washington, DC 20418.

The widespread use of chemical additives for improving the physical properties of soils and soil-aggregate ystems has emphasized the critical need of a classification and indexing system to simplify the selection of the most desirable chemical to be used for the existing environmental conditions and service demands. Such a system is described in this paper. The soil-stabilization indexing system is subdivided into parts dealing separately with lime, portland cement, bituminous materials, and combinations of these materials. The different criteria for the use of each of these stabilizers are described in detail with extensive references to the literature. A series of flow charts has been developed that can be used in selecting the type and the amount of stabilizer for a given soil.

Reference 3
STATE OF THE ART: LIME STABILIZATION. REACTIONS, PROPERTIES, DESIGN, CONSTRUCTION.

Transportation Research Board Committee on Lime and Lime-Fly Ash Stabilization. Transportation Research Board, Washington, DC; September 1976; 31 p. (Transportation Research Circular Number 180).

Order from: Transportation Research Board, Publications Office, 2101 Constitution Avenue, N.W., Washington, DC 20418

This state-of-the-art report on lime stabilization is based on a comprehensive analysis of current practice and technical literature on soil-lime treatment. Many engineering properties of soils are optimized by lime treatment. Lime-treated soils are used in pavement construction as subgrades, subbase materials, and base materials. Topics included in this report are soil-lime reactions, soil-lime mixture design and lime-stabilization construction. A list of references is also included for more detailed information.

Reference 4
FIELD STUDIES ON THE PULVERIZATION OF BLACK COTTON SOIL FOR THE CONSTRUCTION OF STABILIZED SOIL ROAD BASES

Uppal, H.L.; Chadda, L.R.; Dhawan, P.K. Soil Stabilization: Multiple Aspects. 11 Reports. Highway Research Board, Washington, DC; 1970; pp 29-35 (Highway Research Record Number 315).

Order from: Transportation Research Board, Publications Office, 2101 Constitution Avenue, N.W., Washington, DC 20418.

Stabilization of black cotton soil with lime has been found effective in improving the engineering properties of the soil. Consequently, this has led to the increasing use of lime-stabilized black cotton soil in subbases or bases of road pavement. A properly pulverized soil is, however, a prerequisite for successful stabilization of soil. This paper describes a number of methods that have been tried in the field to achieve an economical and effective pulverization method. It has been shown that an acceptable degree of pulverization can be attained when the soil is handled mechanically at a particular moisture range by using agricultural machinery.

Reference 5
SOIL-CEMENT: A MATERIAL OF CONSTRUCTION FOR ROAD AND AIRFIELD PAVEMENTS

Bhatia, H.S. Kumasi, Ghana: Ghana Academy of Sciences, Building and Road Research Institute; June, 1968. 70 p. (Technical Paper No. 1).

Out-of-print; may be consulted at the Transportation Research Board, Room 513, 2100 Pennsylvania Avenue, N.W., Washington, DC 20037.

This monograph reviews the development of soil-cement as an engineering material for road and airfield construction, with special emphasis on tropical and subtropical conditions. The physical and engineering properties of soil-cement depend on the nature of the soil; the proportion of soil, cement, and water in the mixture; the compactive energy used for the molding of the soil-cement; and the physical conditions such as the curing temperatures and age of the soil-cement mixes. The following topics are discussed: properties of soil-cement (volume changes, thermal expansion, permeability, strength, etc.), criteria for selecting soils that are suitable for cement stabilization, important factors in the stabilization of soil with cement (including moisture density relationship and availability of the soil-cement mix), soils requiring special treatment, the stress-strain characteristics of soil-cement, major factors affecting the strength of soil-cement, and basic concepts of pavement design using soil-cement as a rigid material and as a flexible material. The monograph also presents the criterion for mix design and describes soil-cement construction methods and equipment requirements. Quality control in the field for soil-cement construction is detailed.

Reference 6
VARIATION IN LABORATORY AND FIELD STRENGTHS OF SOIL CEMENT MIXTURES

Melancon, James L., Shah, S.C. Stabilization. Transportation Research Board, Washington, DC; 1976; pp. 69-74 (8 reports prepared for the 54th Annual Meeting of the Transportation Research Board; Transportation Research Record 560).

Order from: Transportation Research Board, Publications Office, 2101 Constitution Avenue, N.W., Washington, DC 20418.

This report evaluates the variability in compressive strengths of stabilized in-place cement mixtures from the standpoint of design and actual field conditions. The findings are based on 15 projects with soils ranging from high silt to high sand content and 8 to 14 percent cement by volume. These data indicate considerable variation in the laboratory and field-molded specimens.

Reference 7
CHANGES IN THE CHARACTERISTICS OF CEMENT-STABILIZED SOILS BY ADDITION OF EXCESS COMPACTION MOISTURE

Lightsey, George R.; Arman, Ara; Callihan, Clayton D. Soil Stabilization: Multiple Aspects. 11 Reports. Highway Research Board, Washington, DC; 1970; pp. 36-45 (Highway Research Record Number 315).

Order from: Transportation Research Board, Publications Office, 2101 Constitution Avenue, N.W., Washington, DC 21418.

The compaction moisture of cement-stabilized soils is usually specified as the optimum moisture content to obtain maximum density as determined by the standard Proctor test. Previous investigations have shown that in some instances maximum density may not correspond to maximum strength. If compaction of the soil-cement mix is delayed, the relationship between compaction moisture and the strength and density of the soil-cement also changes. This study investigates the relationship between compaction moisture content and the strength, density, and durability of cement-stabilized soils in which compaction is delayed after mixing to correspond to typical highway construction practices. Four types of soil suitable for cement stabilization were investigated. The compaction moisture content was varied from 4 percent below to 4 percent above the optimum moisture content obtained by standard Proctor tests with no delay between mixing and compaction. At each of the moisture contents and at the optimum cement content, specimens were compacted 0, 2, 4, and 6 hours after mixing with no intermittent mixing. Specimens were prepared for unconfined compressive strength and durability tests. The results of this investigation show that the loss in strength and durability of soil-cement resulting from a delay in compaction can be significantly reduced in many instances by the addition of excess compaction moisture. The silty loams and sandy loams benefited most after a delay in compaction by excess moisture. Cement-stabilized silty clay loams and silts compacted after delays showed little improvement in strength and durability with excess compaction moisture. Without delay in compaction, only the silty clay loams were significantly improved in strength and durability by the addition of excess compaction moisture. In granular soils the addition of excess moisture improved the strength and durability after delays in compaction. This improvement resulted from the improved lubrication of the soil aggregates and subsequent increase in dry density. With fine-grained soils excess moisture improved the properties of soil-cement mixes compacted without delay by increasing the amount of cement hydration.

Reference 8
BITUMINOUS BASES AND SURFACINGS FOR LOW-COST ROADS IN THE TROPICS

Hitch, L.S.; Russel, R.B.C. Great Britain Road Research Laboratory, Overseas Unit; 1977. 33 pages plus charts. (TRRL Supplementary Report 284).

Order from: Transport and Road Research Laboratory, Crowthorne, Berkshire RG 11 6 AU, U.K.

Mechanically stable materials for road bases are often not obtainable in developing countries. In the Middle East, aggregates are often scarce but oil products are readily available. The region has therefore provided some of the earliest examples of bituminous stabilization, which originally consisted of thin running surfaces over compacted sand. Bituminous stabilization can also enable local sand to be used for base construction, and various tests and design criteria have been proposed for such applications. The report describes full-scale experimental trials, supported by laboratory research, which have enabled acceptance criteria for bitumen-stabilized sand bases for light/medium traffic to be proposed. Construction methods for bituminous stabilization are also described. Details are given of methods of surface dressing, which is important both as an initial running surface on new bases and as a maintenance treatment. Premixed bituminous materials, both as bases and surfacings, might perhaps be considered as inadmissible for low-cost roads. Such roads, however, usually require progressive improvement because of the traffic growth that accompanies development. There is a growing use of strengthening overlays, and the report briefly discusses premixed materials and their application.

Reference 9
PERFORMANCE STUDY OF ASPHALT ROAD PAVEMENT WITH BITUMINOUS-STABILIZED-SAND BASES

Marais, Claude P.; Freeme, Charles R. Stabilization of Soils. Transportation Research Board, Washington, DC; 1977; pp. 52-61 (Transportation Research Record 641).

Order from: Transportation Research Board, Publications Office, 2101 Constitution Avenue, N.W., Washington, DC 20418.

The possibility of using the windblown sands that occur in the northern areas of South West Africa for the construction of all-weather roads to carry heavy truck traffic has been investigated. Laboratory investigations and field trials in Pretoria, South Africa, showed that bituminous stabilization of these sands was promising, and a full-scale road experiment to test a limited number of bases of bituminous-stabilized sand was constructed in the homeland of Owambo, South West Africa. This paper describes the laying of the experiment and the construction techniques and control measures used. A new technique that establishes the optimum time for the compaction of a cutback bituminous-stabilized sand mixture after aeration by using a vane shear apparatus is described. The vane shear apparatus was also used to measure the in situ shear strengths of the various experimental bituminous-stabilized sand bases after compaction and during service; the results of these measurements, together with performance data after 8 years' service with respect to deformation and cracking, are discussed. Laboratory and field studies are described and predictions about the performance of a bituminous-stabilized sand base under varying traffic conditions are made by using the best known techniques available at this time.

ADDITIONAL REFERENCES

Reference 10
SOIL AND SOIL-AGGREGATE STABILIZATION

Highway Research Board, Washington, DC; 1955. 175 p. (A Symposium presented at the Thirty Fourth Annual Meeting January 11-14, 1955; Highway Research Board Bulletin 108).

Order from: University Microfilms International, 300 Zeeb Road, Ann Arbor, Michigan 48106.

This bulletin reviews the state of the knowledge of soil stabilization in 1955. The 14 papers presented in this bulletin emphasize the scientific facts and principles of soil stabilization, and the factors and phenomena that must be considered in the engineering application of soil-stabilization methods. The papers consider the importance of the inheritance factor in the origin of clay minerals; describe the new discipline, engineering pedology, and its use in scientific soil stabilization; describe colloid science and its application to soil stabilization; discuss the prediction of the consistency limits of soils and soil mixtures; and comment on the exchange absorption of large cations by clays. Microbial factors in soil stabilization, the stabilization of laterite soils, and the stabilization of silty and clayey soils with lime-fly ash mixtures are considered. Soil stabilization with resins and chemicals is also discussed. Other aspects covered include the effect of calcium on the electro-osmotic flow rate, the elastic behavior of soil-cement mixtures, the factors influencing physical properties of soil-cement mixtures, and the stabilization of Tennessee gravel and chert bases.

Reference 11
SOIL STABILIZATION WITH ASPHALT, PORTLAND CEMENT, LIME AND CHEMICALS

Highway Research Board, Washington, DC; 1960. 126 p. (Presented at the 38th Annual Meeting January 5-9, 1959; Highway Research Board Bulletin 241).

Order from: University Microfilms International, 300 Zeeb Road, Ann Arbor, Michigan 48106.

Six papers are published in this bulletin. The first paper presents the results of a series of tests performed on four Iowa soils stabilized with two organic, cationic chemicals in combination with lignin. The second paper describes a study of various mixtures of soil, cutback asphalt, and water. A compromise moisture content (CMC) for mixing was found, at which the variance of various properties (the maximum strength, maximum standard Proctor density, minimum moisture absorption during immersion, and minimum swelling) is minimized. Chemical treatments for surface hardening of soil-cement and soil-lime-fly ash are discussed in the third paper, and the improvement of soil-cement with alkali metal compounds in the fourth paper. The fifth paper reports a study of the effectiveness of phosphoric acid as a stabilizer for two fine-grained soils (a clayey silt and a heavy clay) with particular emphasis on the use of fluorine compounds as cure-accelerators and of amines as waterproofers. The sixth paper reports work on the use of calcium chloride for soils base stabilization.

Reference 12
INFLUENCE OF STABILIZERS ON PROPERTIES OF SOILS AND SOIL-AGGREGATE MIXTURES

Highway Research Board, Washington, DC; 1961. 159 p. (Presented at the 39th Annual Meeting January 11-15, 1960; Highway Research Board Bulletin 282).

Order from: University Microfilms International, 300 Zeeb Road, Ann Arbor, Michigan 48106.

Ten papers are presented in this bulletin. The first paper reports the results of a study which used compressive strength to evaluate the stability of five selected soils of widely varying physical properties stabilized with portland cement, a lime and fly ash mixture, phosphoric acid, and asphaltic cutback. The second paper reports an investigation of soil waterproofing and dustproofing materials. The improvement of phosphoric acid stabilization of fine-grained soils with secondary additives is discussed in the third paper. The effect of phosphoric acid treatments on the stability of compacted calcareous and non-calcareous clay soils is discussed in the fourth paper. Other papers describe sodium-chloride-stabilized roads in Iowa; the relative effects of chlorides, lignosulfonates, and molasses on properties of soil-aggregate mix; bituminous stabilization of Wyoming heat-altered shale; preliminary findings and future programming of a basic research project involving calcium chloride with pure clays; current interpretation of stability measurements on two experimental projects in Maryland; and a study of the occurrence of potholes and washboards on soil-aggregate roads.

Reference 13
SOIL STABILIZATION

Bezruk, V.M. Moscow, U.S.S.R. Izdatel'stov; 1965. 373 p. plus literature (TT 70-57767; PB-280 290-T/ST).

Order from: National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

This book describes the origin, composition, and properties of soils and the basic principles and methods of soil stabilization. The various chapters discuss the general concept of soils as dispersed, multiphased systems, the granulometric composition of non-coherent and coherent soils, and the physical properties of soils (hydrophily, plasticity, swelling and shrinkage, wetting, and adhesiveness). The structural-mechanical properties and the calculation of strength and deformation properties are also discussed. The methods of field investigation and determination of the basic soil characteristics are reviewed. The main features of soil distribution in the USSR and the typical clay and detrital soils and their distribution in the USSR are reviewed. The scientific basis of soil stabilization is set forth. The improvement of soil strength by artificial compaction and granulometric additives is covered. Details are given of soil stabilization with portland cement, with lime, with organic binders, and with synthetic polymer resins. Clay soil stabilization with phosphates and thermal soil stabilization are also detailed. The basic technological requirements and means for the mechanization of soil-stabilization operations are reviewed. Life test and the results of testing road surfaces with stabilized soil structural layers are given, as well as recommendations for road-surface designs.

Reference 14
LIME STABILIZATION: PROPERTIES, MIX DESIGN, CONSTRUCTION PRACTICES AND PERFORMANCE

Highway Research Board, Washington, DC; 1961. 147 p. (Presented at the 40th Annual Meeting January 9-13, 1961; Highway Research Board Bulletin 304).

Order from: University Microfilms International, 300 Zeeb Road, Ann Arbor, Michigan 48106.

Nine papers are presented in this bulletin. The paper, Accelerated Curing for Lime-Stabilized Soils, describes the preliminary phases of a study to develop specifications for the required compressive strength of lime-stabilized base and subbase materials. The influence of time between mixing and compaction on properties of a lime-stabilized expansive clay is considered in the second paper. The evaluation of promising chemical additives for accelerating hardening of soil-lime-fly ash mixtures is described in the third paper. The fourth paper describes the isolation and investigation of a lime-montmorillonite crystalline reaction product. Other papers describe the relation of strength to composition and density of lime-treated clayey soils, the lime and sodium silicate stabilization of montmorillonite clay soil, and the study of an old lime-stabilized gravel base. Lime-soil mixtures are reviewed, and the gravels used in lime-fly ash-aggregate composition are evaluated.

Reference 15
LIME STABILIZATION CONSTRUCTION MANUAL
SIXTH EDITION

National Lime Association, Washington, DC; 1976. 48 p. (Bulletin 326).

Order from: National Lime Association, 5010 Wisconsin Avenue, N.W., Washington, DC 20016.

This bulletin defines soil stabilization, describes the effect of lime on soil, and proceeds to sequentially cover the construction steps. This is followed by a more detailed treatment of the same steps, including all significant modifications, alternatives, and comparative conclusions. A discussion of subbase, subgrade, and base stabilization covers scarification, pulverization, lime spreading, mixing, watering, curing, compaction, and final curing. The use of bagged lime and bulk lime is covered, as well as the pros and cons of dry vs. slurry methods. The double application of lime is also discussed. Comments are made on miscellaneous considerations such as maintaining traffic, the need for a wearing surface, climatic limitations, lime safety precautions, bulk density, and the use of quick lime and of other stabilizers with lime. The appendix provides further information on specifications for hydrated lime and other pertinent specification references.

Reference 16
THE GEOLOGY AND GEOTECHNICAL PROPERTIES OF THE BLACK COTTON SOILS OF NORTHERN NIGERIA

Ola, S.A. Engineering Geology: An International Journal. Elsevier Scientific Publishing Company, Amsterdam, Netherlands. Vol. 12, No. 4: December 1978; pp. 375-391.

Order from: Elsevier Scientific Publishing Company, P.O. Box 330, 1000 AH Amsterdam, The Netherlands.

The paper gives a brief review of the geology of the black cotton soils of northeastern Nigeria. Mineralogical analysis shows the soils to be predominantly kaolinite and montmorillonite. Results presented show low strength characteristics, a soaked CBR of only 1.5, and a residual strength parameter of 12 deg. The plasticity index decreases from 47% to 5%, linear shrinkage decreases from 11 to 3.6% for an increase in lime content of 0 to 9%, respectively. Results presented indicate that a mixture of both lime and cement is necessary for adequate stabilization of road bases for heavy wheel loads on the black cotton soils of northeastern Nigeria. These results are in general agreement with results previously presented for other African and Indian black cotton soils.

Reference 17
SOIL CEMENT CONSTRUCTION HANDBOOK

Portland Cement Association. Skokie, Illinois; 1979. 40 p. (Engineering Bulletin).

Order from: Portland Cement Association, 5420 Old Orchard Road, Skokie, Illinois 60077.

The details are given of procedures for building a soil-cement base course. The booklet shows how high-quality soil-cement can be built rapidly and easily under a wide variety of conditions by adhering to the principles of thorough pulverization and mixing, adequate cement content, proper moisture content, and adequate compaction. The uses, properties, laboratory tests, and general construction procedures are detailed. The details of preparation and processing are given. Joint construction and multiple-layer construction are also covered. Street construction and airport paving are detailed. Other areas covered include storage and parking areas, widening and shoulders, the recycling of flexible pavements, and cement-modified soils. Cement-modified granular soils, silt, and clay soils are covered.

Reference 18
SOIL CEMENT LABORATORY HANDBOOK

Portland Cement Association. Skokie, Illinois; 1971. 61 p. (Engineering Bulletin).

Order from: Portland Cement Association, 5420 Old Orchard Road, Skokie, Illinois 60077.

This handbook describes laboratory test methods of determining the primary requisites for producing soil-cement with satisfactory characteristics and serviceability. These requisites are that an adequate quantity of portland cement be incorporated with pulverized soil, that the proper amount of water be mixed uniformly with the soil-cement mixture, and that the moistened soil-cement mixture be compacted to proper density before cement hydration. The booklet discusses the following subjects in detail: methods for testing soil-cement, the selection of cement contents for tests, soil-cement test methods, compressive-strength and other supplementary tests, the establishment of cement factors for construction, shortcut test procedures for sandy soils, rapid test procedure, the testing of unusual sandy soils, the testing of plastic soil-cement, and the modification of soils with portland cement.

Reference 19
STABILIZATION OF SOILS WITH PORTLAND CEMENT: DESIGN, TESTING, PROPERTIES, ADMIXTURES

Highway Research Board, Washington, DC; 1962. 123 p. (Presented at the 41st Annual Meeting, January 8-12, 1962; Highway Research Board Bulletin 353).

Order from: University Microfilms International, 300 Zeeb Road, Ann Arbor, Michigan 48106.

Seven papers are presented in this bulletin. The paper, A Cement-Treated Base for Rigid Pavement, presents the results of laboratory tests of various mixes of the natural materials with cement, and discusses the selection of the design mix. Methods of working the borrow pit and constructing the cement-treated base are discussed in detail. The second paper describes three alternate methods for measuring freeze-thaw and wet-dry resistance of soil-cement mixtures; length change, compressive strength, and pulse velocity. Moisture-density, moisture-strength and compaction characteristics of cement-treated soil mixtures are described in the third paper. Other papers discuss the effect of lime on the cement stabilization of montmorillonite soils, the strength-maturity relations of soil-cement mixtures, and the effect of sulfates on cement- and lime-stabilized soils. The seventh paper reports on the use of fly ash and sodium carbonate as additives to soil-cement mixtures. Three soils (dune sands, friable loess, artificial sand-loess mixture), fly ashes from three different sources, cement type 1, and regent grade sodium carbonate were used.

Reference 20
CEMENT-TREATED SOIL MIXTURES: 10 REPORTS

Highway Research Board, Washington, DC; 1963; 208 p. (Presented at the 42nd Annual Meeting January 7-11, 1963; Highway Research Record Number 36).

Order from: University Microfilms International, 300 Zeeb Road, Ann Arbor, Michigan 48106.

Ten reports are presented in this bulletin. The development, purpose and history of use of standard laboratory soil-cement tests are described in the first paper, and a California mix design for cement-treated bases in the second paper. The latter paper notes the increasing emphasis on the use of granular materials, resulting in a decrease in cement content and a minimization of shrinkage cracks. The third paper describes British practice in the design and specification of cement-stabilized bases and subbases for roads. The fourth paper reports a laboratory test for evaluating the durability of stabilized fine-grained soils subjected to repeated freeze-thaw cycles. Further papers describe developments in durability testing of soil-cement mixtures, and the determination of the cement content of soil-cement mixtures. The reactions accompanying stabilization of clay with cement are discussed, and comments are made on the effect of chemicals on soil-cement stabilization.

Reference 21
ROAD NOTE 31: A GUIDE TO STRUCTURAL DESIGN OF BITUMEN-SURFACED ROADS IN TROPICAL AND SUBTROPICAL COUNTRIES

Transport and Road Research Laboratory. Crowthorne, U.K.; 1977 26 p. (Third Edition).

Order from: Her Majesty's Stationary Office, 49 High Holborn, London WCJV 6 HB.

This guide covers design requirements of most non-urban roads in developing countries. Traffic is defined in terms of the cumulative equivalent number of 8200-kg (18000-lb) axles to be carried during the design life of the roads. Thus, this guide may be used to design pavements for traffic flows with widely different axle-load distributions and gives the designer greater flexibility in selecting the design life of the pavement. The guide emphasizes the importance of detailed consideration of the influence of tropical climates on moisture conditions in road subgrades, and the desirability of adopting a stage construction approach to road building where traffic growth rates are high, or long-term predictions are uncertain. The details are given of the three main steps to be followed in designing a new road pavement: estimating the amount of traffic (and its axle-load distribution) that will use the road over the selected design life; assessing the strength of the subgrade soil over which the road is built; and taking into consideration the influence of subgrade moisture and the stage-construction approach, in selecting the most economical combination of pavement materials and layer thickness that will be sufficient to provide satisfactory service over the design life of the pavement.

Reference 22
ASPHALT COLD-MIX MANUAL

Asphalt Institute. College Park, Maryland; February 1977. 106 p. (Manual Series No. 14; MS-14).

Order from: The Asphalt Institute, Asphalt Institute Building, College Park, Maryland 20740.

This publication is designed to assist and guide engineers in preparing a specification for the analysis, design, and control of asphalt cold-mix construction. The text describes typical mixes that are recognized and proven satisfactory for preparation by cold-mix methods. With the information provided, it will be possible to adapt specific conditions to the general specifications presented in the text. Basic information on asphalt, aggregates, and mix design are presented, and equipment for mixed-in-place and for construction are described. Equipment for plant mixes and central plant mix construction is also described. Appendices provide information on suggested plant mix and mixed-in-place specifications, as well as specifications for stockpile patching mixtures.

Reference 23
A BASIC ASPHALT EMULSION MANUAL

Asphalt Institute. College Park, Maryland; March 1979. 260 p. (Manual Series No. 19; MS-19).

Order from: The Asphalt Institute, Asphalt Institute Building, College Park, Maryland 20740.

This manual attempts to impart a basic understanding of asphalt emulsions, and is designed to be of use in choosing the emulsion that best fits a project's specific conditions. It is also designed to help in evaluating pavement systems for construction and maintenance. The chemistry of asphalt emulsions is described, and the storage, handling, and sampling of asphalt emulsions are reviewed. Emulsified asphalt tests are detailed. Selection of the right type of asphalt emulsion is discussed, and asphalt emulsion and aggregate application, as well as asphalt emulsion and aggregate mixes, are considered in detail. Miscellaneous asphalt emulsion applications, maintenance mixes, and various aspects of recycling are also covered. Emulsified asphalt-aggregate mix design methods detailed include the Asphalt Institute Method and the proposed Illinois method.

Reference 24
A GUIDE TO SHORT-CUT PROCEDURES FOR SOIL-STABILIZATION WITH ASPHALT

Vallerga, B.A. Oakland, California. Materials Research and Development Incorporated; April 1968. 38 p. (TN-955; sponsored by Naval Civil Engineering Laboratory, Port Hueneme, California; report #AD 668 699).

Order from: National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

Short-cut laboratory and field procedures for the use of asphalt in soil stabilization are detailed. These procedures are applicable only when the plasticity index (PI) is 6 or less, the sand equivalent is 30 or more, no more than 25% of the soil passes the No. 200 sieve, and when the product of PI times percent passing the No. 200 sieve does not exceed 72. The laboratory determinations covered here include gradation of soil, the sand equivalent or plasticity index, the asphalt requirements, selection of asphalt type for emulsified asphalts, mixing water content, compacting water content, laboratory density, asphalt requirement for cutback asphalts, the selection of the asphalt type for cutback asphalts, and the laboratory density for cutback asphalts. The field operations covered here include site preparation for in-place mixing, preliminary processing of material, application of asphalt for in-place mixing, mixing operation for in-place mixing, site preparation for stationary plant-mixing preliminary processing before mixing by stationary plant, mixing operation by stationary plant, placing plant mix, checking plant mix, spreading and compacting, checking density, and finishing and curing.

Index

The following index is an alphabetical list of subject terms, names of people, and names of organizations that appear in one or another of the previous parts of this compendium, i.e., in the overview, selected texts, or bibliography. The subject terms listed are those that are most basic to the understanding of the topic of the compendium.

Subject terms that are not proper nouns are shown in lower case. Personal names that are listed generally represent the authors of selected texts and other references given in the bibliography, but they also represent people who are otherwise identified with the compendium subjects. Personal names are listed as surname followed by initials. Organizations listed are those that have produced information on the topic of the compendium and that continue to be a source of information on the topic. For this reason, postal addresses are given for each organization listed.

Numbers that follow a subject term, personal name, or organization name are the page numbers of this compendium on which the term

Indice

El siguiente índice es una lista alfabética del vocablo del tema, nombres de personas, y nombres de organizaciones que aparecen en una u otra de las partes previas de este compendio, es decir, en la vista general, textos seleccionados, o bibliografía. Los vocablos del tema que aparecen en el índice son aquellos que son necesarios para el entendimiento de la materia del compendio.

Los vocablos del tema que no son nombres propios aparecen en letras minúsculas. Los nombres personales que aparecen representan los autores de los textos seleccionados y otras referencias dadas en la bibliografía, pero también pueden representar a personas que de otra manera están conectadas a los temas del compendio. Los nombres personales aparecen con el apellido seguido por las iniciales. Las organizaciones nombradas son las que han producido información sobre la materia del compendio y que siguen siendo fuentes de información sobre la materia. Por esta razón se dan las direcciones postales de cada organización que aparece en el índice.

Los números que siguen a un vocablo del tema, nombre personal, o nombre de organización son los números de página del compendio donde el vocablo o nombre aparecen. Los números romanos se refieren a las páginas en la vista general, los números arábigos se refieren a páginas en los textos seleccionados, y los números de referencia (por ejemplo, Ref. 5) indican referencias en la bibliografía.

Algunos vocablos del tema y nombres de organizaciones están seguidos por la palabra ***see.*** En tales casos los números de página del com-

Index

Cet index se compose d'une liste alphabétique de mots-clés, noms d'auteurs, et noms d'organisations qui paraissent dans une section ou une autre de ce recueil, c'est à dire dans l'exposé, les textes choisis, ou la bibliographie. Les mots-clés sont ceux qui sont le plus élémentaires à la compréhension de ce recueil.

Les mots-clés qui ne sont pas des noms propres sont imprimés en minuscules. Les noms propres cités sont les noms des auteurs des textes choisis ou de textes de référence cités dans la bibliographie, ou alors les noms d'experts en la matière de ce recueil. Le nom de famille est suivi des initiales des prénoms. Les organisations citées sont celles qui ont fait des recherches sur le sujet de ce recueil et qui continueront à être une source de documentation. Les adresses de toutes ces organisations sont incluses.

Le numéro qui suit chaque mot-clé, nom d'auteur, ou nom d'organisation est le numéro de la page où ce nom ou mot-clé parait. Les numéros or name appears. Roman numerals refer to pages in the overview, Arabic numerals refer to pages in the selected texts, and reference numbers (e.g., Ref. 5) refer to references in the bibliography.

Some subject terms and organization names are followed by the word ***see.*** In such cases, the compendium page numbers should be sought under the alternative term or name that follows the word ***see.*** Some subject terms and organization names are followed by the words ***see also.*** In such cases, relevant references should be sought among the page numbers listed under the terms that follow the words ***see also.***

The foregoing explanation is illustrated below.

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pendio se encontrarán bajo el término o nombre alternativo que sigue a la palabra ***see.*** Algunos vocablos del tema y nombres de organizaciones están seguidos por las palabras ***see also.*** En tales casos las referencias pertinentes se encontrarán entre los números de página indicados bajo los términos que siguen a las palabras ***see also.***

La explicación anterior está subsiguientemente ilustrada.

écrits en chiffres romains se rapportent aux pages de l'exposé et les numéros écrits en chiffres arabes se rapportent aux pages des textes choisis. Les numéros de référence (par exemple, Ref. 5) indiquent les numéros des références de la bibliographie.

Certains mots-clés et noms d'organisations sont suivis du terme ***see.*** Dans ces cas, le numéro des pages du recueil se trouvera après le mot-clé ou le nom d'organisation qui suit le terme ***see.*** D'autres mots-clés ou noms d'organisations sont suivis des mots ***see also.*** Dans ce cas, leurs références se trouveront citées après les mots-clés qui suivent la notation ***see also.***

Ces explications sont illustrées ci-dessous.

Illustration (from Comp. 1) **Ilustración** (del Comp. 1) **Illustration** (du Recueil 1)

Selected Text page numbers
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Vocablo del tema y términos *see also* (ver también)
Mot-clé et *see also*

Organization name and address
Nombre y dirección de la organización
Nom et adresse de l'organisation

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Subject term and *see* term
Vocablo del tema y término *see (ver)*
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mountainous terrain (see also degree of curvature; design speed; maximum gradient; radius of curvature; shoulder width): 11, 17, 34, 35, 38, 173, 175, 217, 234, 238

National Association of Australian State Road Authorities(P.O. Box J141, Brickfield Hill, N.SW. 2000, Australia):
publications, xxiii, Ref. 8, Ref. 9

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