

**TECHNICAL RECOMMENDATIONS
FOR HIGHWAYS**

Draft TRH13

**CEMENTITIOUS
STABILIZERS
IN ROAD
CONSTRUCTION**

1986

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PREFACE

TECHNICAL RECOMMENDATIONS FOR HIGHWAYS (TRH) are written for the practising engineer and describe current, recommended practice in selected aspects of highway engineering. They are based on South African experience and the results of research and have the full support and approval of the Committee of State Road Authorities (CSRA).

To confirm their validity in practice, TRHs are circulated in draft form for a period of trial before being submitted to the CSRA for approval. This document, Draft TRH13, will therefore be in use for a limited period during which you are welcome to send suggestions for improvement to the Chief Director, National Institute for Transport and Road Research, P O Box 395, Pretoria 0001. Eventually, a revised document, approved by the CSRA, will be issued as a full TRH in both languages.

NOTE

This document is published by the National Institute for Transport and Road Research for and with the approval of the Highway Materials Committee of the Committee of State Road Authorities.

SYNOPSIS

The most important engineering properties of stabilized materials are presented together with a brief description of the different types of cementitious stabilizers that are available for the treatment of road construction materials.

The advantages and uses of various stabilized materials in practice are discussed and the terms "modification" and "cementation" are explained. Laboratory test methods, design methods and design criteria are provided.

Construction procedures are recommended, which include comments on quality assurance and the cracking of pavements with cemented layers.

SINOPSIS

Die belangrikste ingenieurseienskappe van gestabiliseerde materiale word aangebied tesame met 'n kort beskrywing van die verskillende tipes sementerende stabiliseerders wat vir die behandeling van padboumateriale beskikbaar is.

Die voordele en die gebruike van verskeie gestabiliseerde materiale in die praktyk word bespreek en die terme "modifisering" en "sentering" word verduidelik. Laboratoriumtoetsmetodes, ontwerpmetodes en ontwerpkriteria word verskaf.

Konstruksieprosedures word aanbeveel wat kommentaar oor gehalteversekering en oor kraakvorming in plaveisels met gesementeerde lae, insluit.

KEYWORDS:

Stabilization, cement, lime, cementitious stabilizers, road construction, material strength.

There has been a steady increase in soil stabilization since it was first undertaken in South Africa more than 40 years ago. The improvement in the engineering properties of soils that can be achieved by stabilization and the economics of stabilization are such that some form of this process features in virtually every surfaced road constructed today.

All readers of this draft document are encouraged to submit comments. The document was prepared by a subcommittee appointed by the Highway Materials Committee, and consisted of engineers, scientists and technicians from both the private and public sectors.

CHAIRMAN
HIGHWAY MATERIALS COMMITTEE

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DEFINITIONS

For the purposes of this document, the following definitions apply:

Cement	Ordinary Portland cement (OPC), Portland blastfurnace cement (PBFC) or a mixture of OPC and milled granulated blastfurnace slag (MGBS).
Cementation	Stabilization with the object of increasing the compressive or tensile strength to a predetermined level. The term "cemented material" is also used.
Cementitious stabilizing agent	Cement or lime or a combination of either cement or lime with MGBS or fly ash (FA). Note: Other stabilizing agents, such as bitumen, tar and certain other chemicals, are not dealt with in this document.
Fly ash (FA)	The finely divided residue that results from the combustion of ground or powdered coal.
Lime	All types of slaked and unslaked lime.
Milled granulated blastfurnace slag (MGBS)	Granulated slag, a by-product of the processing of iron ore, milled to a fine powder.
m/m	Mass by mass.
Modification	The improvement in properties obtained when a material is stabilized and there is not necessarily a development of a compressive or tensile strength. Modification is usually employed to reduce the Plasticity Index (PI) and to increase the California Bearing Ratio (CBR). The term "modified material" is also used.
Soil	Any combination of rock (crushed or uncrushed), gravel, sand, silt and clay.
Stabilization	The treatment of soils by chemical or other means, in order to improve their engineering properties.
Stabilizing agent (also Stabilizer)	A chemical added to a soil to improve the engineering properties of the soil.

1 INTRODUCTION

The use of mixtures of lime and clay in building dates back to the beginning of man's recorded history. Mixtures of lime and clay were used in the construction of the pyramids of Shensi on the Tibetan-Mongolian plateau. In ancient China and India, lime treatment was frequently used in the construction of bridges and dams. The Romans used lime treatment in constructing their famous roads. In the USA lime treatment in road construction was first studied in Texas about 1930, but various difficulties delayed its general implementation until about 1952. Since then many kilometres of lime-treated pavements have been built in Texas and elsewhere in the USA.

The first recorded studies of mixtures of soil and cement date back to 1917 when a patent was taken out on soil-cement mixtures in the USA, but it is quite probable that work was done on soil-cement even before that date. Studies on the mixing of soil and cement were undertaken by several states in the USA during the 1920s, but it was not until 1935 that the South Carolina Highway Department demonstrated that soil and cement were compatible materials and could be mixed together to produce a usable pavement base.

The use of cement in the treatment of road construction materials in South Africa was first recorded in the early 1940s, and since then there has been a steady increase in the use of cement and lime. Other types of stabilizing agents were also developed, and at present some form of stabilization features in the construction of the great majority of both heavily and lightly trafficked pavements.

Naturally occurring and crushed materials have certain inherent limitations, which can have far-reaching effects on the performance of pavements. Many of these limitations can be overcome to a great extent by the addition of a stabilizing agent. Some of the most important advantages of stabilization are:

- the strength of the material is increased;
- durability and resistance to the effects of water are improved;
- wet soils can be dried out; and
- the workability of clayey materials can be improved.

In addition, cemented pavement layers possess good load-spreading properties that can be used effectively to improve the structural capacity of pavements.

The stabilization of pavement layers is a fairly straightforward operation and with sound construction techniques the properties of materials can be improved significantly by the addition of a relatively small quantity of stabilizing agent. The treatment is usually economical in relation to the benefits that can be obtained.

Stabilized pavement materials have thus become an important part of pavement design and construction. Test procedures, design methods and construction techniques have been developed that take into account the properties and be-

haviour of stabilized materials. Test methods for stabilized materials are given in TMH1¹ and its supplement. Recommended standards for stabilized materials are given in TRH14². The strength of stabilized layers and their position within the pavement are both important, and TRH4³ gives recommendations on the stabilized strength and pavement composition to provide a structurally balanced pavement. In the Catalogue of pavement designs given in TRH4³, some form of stabilization features in nearly every pavement, which shows how extensively stabilized materials are used in present-day road construction in South Africa.

This document, which is based on experience and philosophies developed in South Africa and elsewhere, deals with cementitious stabilizers only and does not include bituminous or other stabilizers. The various objectives of soil stabilization are stated and the materials suitable for treatment are described. The properties of treated materials are discussed and recommendations on test methods are given. Criteria for strength are summarized. Comments on construction techniques and quality assurance are included.

2 STABILIZING AGENTS

The following cementitious stabilizing agents are available for soil stabilization:

- (a) Cement
 - Ordinary Portland cement
 - Sulphate-resisting cement
 - Portland blastfurnace cement
 - Portland cement 15 SL
 - Portland cement 15 FA
 - Blends of milled granulated blastfurnace slag and ordinary Portland cement
 - Blends of ordinary Portland cement and fly ash
 - Masonry cement
- (b) Lime
 - Slaked lime
 - Unslaked lime
- (c) Blends of milled granulated blastfurnace slag and lime
- (d) Blends of fly ash and lime.

2.1 Cement

2.1.1 The nature of cement

Cement consists essentially of finely ground calcium silicates and aluminates with small percentages of magnesium oxide, gypsum and uncombined calcium oxide. When water is added to cement, a gel is formed, which has a much larger surface area than dry cement powder. The gel hardens owing to crystalline intergrowths formed by the hydration compounds, the predominating cementing compound being calcium silicate hydrate.

2.1.2 Types of cement

2.1.2.1 Ordinary Portland cement

Ordinary Portland cement (OPC) is a general, all-purpose cement that complies with SABS 4714. In South Africa, between 85 and 90 per cent of all cement used is OPC.

Note:

Should a cement with a low-alkali content be desired, this should be specified by the purchaser. In such an event SABS 4714 limits the total content of sodium oxide (Na_2O) plus $0,658 \times$ potassium oxide (K_2O) to 0,6 per cent (m/m).

2.1.2.2 Sulphate-resisting Portland cement

Sulphate-resisting Portland cement (SRC) is a Portland cement that complies with SABS 4714 but has a higher resistance than OPC to attack by sulphates. It achieves this greater resistance to sulphate attack by virtue of its low tricalcium aluminate content. Although SRC is reasonably resistant to calcium and sodium sulphate solutions, it is vulnerable to magnesium sulphate attack, and is extremely vulnerable to ammonium sulphate. It is not an acid-resistant cement.

Note:

Should a cement with a low-alkali content be desired, this should be specified by the purchaser. In such an event SABS 471⁴ limits the total content of sodium oxide (Na_2O) plus 0,658 x potassium oxide (K_2O) to 0,6 per cent (m/m).

2.1.2.3 Portland blastfurnace cement

Portland blastfurnace cement (PBFC) consists of an intimate mixture of mainly OPC or Portland cement clinker and granulated blastfurnace slag (SL). The granulated blastfurnace slag constitutes not less than 15 or more than 70 per cent (m/m) of the mixture; the SL content is usually about 50 per cent in practice. The cement content complies with SABS 626⁵.

This cement is characterized by longer initial setting times and a slower rate of development of compressive strength than OPC, which can be an advantage in practice.

2.1.2.4 Portland cement 15 SL

This cement comprises essentially an intimately interground mixture (or a uniform mixture of separately ground components) of not less than 80 per cent (m/m) of Portland cement clinker and not less than 5 or more than 15 per cent (m/m) of SL. Its behaviour is similar to that of OPC. The cement complies with SABS 831⁶.

2.1.2.5 Portland cement 15 FA

This cement comprises essentially an intimate interground mixture (or a uniform mixture of separately ground or processed components) of not less than 80 per cent (m/m) of Portland cement clinker and not less than 6 or more than 15 per cent (m/m) of fly ash (FA). Its behaviour is similar to that of OPC. The cement complies with SABS 831⁶.

2.1.2.6 Blends of milled granulated blastfurnace slag and ordinary Portland cement

Milled granulated blastfurnace slag (MGBS) can be mixed with OPC on site and, provided that an intimate mixture is obtained and used in the form of a 50/50 blend by mass of OPC and MGBS, the behaviour of the blend will approach that of PBFC.

2.1.2.7 Blends of ordinary Portland cement and fly ash

Blends of OPC and FA have recently been used in concrete in South Africa. Such blends have certain advantages over OPC, one of which is that they tend to limit alkali-aggregate reaction. This type of blend is used fairly extensively in concrete in the Western Cape. The OPC/FA blend usually contains 15 to 30 per cent of FA. At present there is no SABS specification for this product, but the suppliers state that it complies with ASTM C618⁷ of 1978. OPC/FA is suitable for soil stabilization.

2.1.2.8 Masonry cement

Cements that have been developed and marketed for masonry work have been found to be effective in the treatment of soils.

2.2 LIME

2.2.1 The nature of lime

Lime is either the oxide or hydroxide of calcium. Magnesium oxide or hydroxide may also be present. Lime is usually obtained by burning limestone or dolomite, thus converting the carbonate to the oxide, which is known as "unslaked lime". If water is added the hydroxide, which is known as "slaked lime", is formed. Since the carbonate has little effect on the soil, the more carbonate present in the lime the less effective it will be. Specifications for lime therefore give minimum requirements for the free calcium oxide or hydroxide content, known as "available lime". If lime is exposed it will combine with the carbon dioxide in the atmosphere, gradually revert to the carbonate and become less effective. Up to 5 per cent CO₂ is permitted in SABS 824⁸.

2.2.2 Types of lime

SABS 824⁸ gives the chemical requirements of three types of lime as follows:

Type of lime	CaO/MgO	CaO + MgO (% m/m)		Available lime (% m/m)	
		Slaked	Quick	Slaked	Quick
Calcium	≥14,0	75	85	50	80
Magnesium	≥2,0 – <14,0	75	75	35	35
Dolomitic	≥1,3 – <2,0	75	75	30	30

Lime should have at least 75 per cent of calcium and magnesium oxide on the basis of ignited mass. The available lime content (expressed as calcium oxide) requirements should also be met since it is the available lime which actually reacts with the soil. Although the available lime content is an important consideration, the amount of available lime in itself should not necessarily be a criterion for selecting a particular lime. Lower quality limes not complying with SABS 824⁸ can provide satisfactory results and the selection of a particular lime and the lime content should be based on laboratory tests on the soil concerned.

2.2.2.1 Slaked lime

Dry, slaked lime is the most frequently used type of lime. It is easy to handle, spread and mix.

2.2.2.2 Unslaked lime

Lime can be used for stabilization in the unslaked form and it is particularly effective for facilitating construction in wet or marshy conditions. Unslaked lime exhibits a high heat of hydration and during mixing and compaction special care must be taken to ensure that the slaking process is completed, otherwise the hydration may cause blisters in the compacted layer. The rate of slaking during

the watering and mixing process may vary, depending on factors such as the fineness and the magnesium oxide content. Some unslaked limes slake very rapidly and slaking is completed during the normal watering and compaction process, whereas others may require mixing and watering, with compaction delayed for about 24 hours. The volume of unslaked lime is up to one-third less than the volume of slaked lime with the same oxide content and the smaller volume is easier to transport and store.

2.2.2.3 Lime from the manufacture of acetylene

Slaked lime is also available as a by-product of the manufacture of acetylene from calcium carbide. When it is available as a wet product it is often referred to as "wet lime" but it is also available in the dry condition. Wet lime has been used successfully for stabilization, but there are disadvantages such as variable water content and the difficulties of spreading and mixing the wet material, particularly when cohesive soils are treated. For this and other reasons the dry slaked lime is preferred. It has the same properties as slaked lime manufactured by burning limestone or dolomite and has the advantage of a relatively high degree of purity, with the calcium hydroxide content usually exceeding 90 per cent.

2.2.3 Safety precautions

It is essential that precautions be taken to protect workers from the harmful effects of lime. Unslaked lime can rapidly cause severe burns when it comes into contact with moist skin, and can be dangerous if it comes into contact with the eyes and mucous membranes. Although slaked lime is less dangerous, it too can cause skin irritation and damage to the eyes and mucous membranes. Prolonged contact with hydrated lime may cause dermatitis, particularly during hot and humid weather.

The following safety precautions should be taken:

- suitable protective clothing should be worn;
- petroleum jelly or barrier cream should be applied to exposed parts;
- safety glasses with eye shields or goggles should be worn;
- respirators should be worn if there is a possibility of inhaling lime dust;
- after work a bath or shower should be taken; and
- if unslaked lime comes into contact with the skin it should be washed off immediately.

2.3 BLENDS OF MGBS AND LIME

MGBS alone does not have significant cementing properties since it does not always contain the calcium constituents necessary for cementation. When MGBS is mixed with lime, a cementing compound is obtained with properties similar to those of PBFC, and this is a very effective soil-stabilizing agent. The MGBS and

lime are often mixed in equal proportions. MGBS and lime are not available in premixed form, and the two materials must be mixed into the soil to provide an intimate mixture.

2.4 BLENDS OF FA AND LIME

FA is a by-product obtained from the burning of pulverized coal and is usually collected on electrostatic precipitators. If some FAs are mixed with lime, cementing silicates and aluminates are formed. The reactivity of FA from different sources can vary considerably and this will affect FA-lime-soil reactions.

2.5 CHOICE OF STABILIZING AGENT

Sometimes more than one stabilizing agent will provide the desired results, and in other cases one particular agent may be more suitable than any of the others. Some of the factors influencing the selection of a stabilizing agent are discussed in the following paragraphs, the main considerations being: design requirements, laboratory test results, site conditions, availability of stabilizers at the time of construction and economics.

3 THE PROPERTIES OF STABILIZED MATERIALS

A considerable amount of research has been done on the properties and behaviour of stabilized materials and in this section the most important engineering properties are summarized.

3.1 CEMENT-STABILIZED MATERIALS

3.1.1 The reaction between soil and cement

The lime released during the hydration of cement reacts with plastic soils and this results in a fairly rapid reduction in plasticity and an increase in strength. However, it is essentially the characteristic bonding of particles by the cement that provides early and long-term strength and thus determines the engineering properties of cement-treated materials.

3.1.2 Soils suitable for treatment with cement

Cement is particularly effective in stabilizing medium to low plasticity materials for use as pavement layers. Although it is possible to treat almost any soil with cement to improve its properties, in practice it is difficult to treat fine, clayey materials with cement owing to the high cement content required and the difficulty of pulverizing the soil and mixing in the cement. Where stabilization of fine, clayey materials is considered, as in difficult conditions in the subgrade, lime is a better stabilizing agent.

Fine, single-sized windblown or dune sands can be stabilized with cement but high cement contents are usually necessary to obtain the compressive strength required in pavement layers: 10 to 14 per cent of cement may be required to achieve an unconfined compressive strength (UCS) of about 2,5 MPa.

3.1.3 Compaction characteristics

The addition of a stabilizing agent to a soil usually results in an increase in optimum moisture content (OMC) and a decrease in maximum dry density. The increase in OMC is caused by the flocculating effect and by the water demand of the stabilizer. The reduction in density is caused by the development of early bond strength between particles that form loosely bonded aggregations. These aggregations may not be broken down by compaction and the longer the compaction is delayed after the cement comes into contact with water, the greater is the strength of the loose aggregations and the greater the reduction in density and strength of the pavement layer.

Some reduction in density is unavoidable owing to the normal duration of the compaction process in practice, and most specifications allow for a slightly reduced density. Most specifications also state a maximum permissible period between the time the cement is added and the time compaction must be completed. Efforts to obtain a high density after too long a mixing period may also result in degradation of the material in the top of the layer.

Laboratory density tests should be done with care and the procedures that allow for the reduction in density should be strictly adhered to. The Appendix to TMH1¹ Method A14 allows for a four-hour delay before compaction, but in practice this delay may be longer. Maximum densities may also be obtained by taking samples from the road after the stabilizer has been added and allowing for the delay due to mixing and compaction. The delay should be determined from the time the stabilizer is applied if the layer is pre-wetted or, if the stabilizer is mixed in dry, from the time water is added. Sampling of plant-mixed materials may be done at the plant immediately before water is added to the material.

Whatever method is used to obtain the maximum stabilized density, it is essential that sound judgement be exercised when accepting or rejecting a compacted layer on the basis of density, since the main objective is to achieve a suitable in-situ strength.

3.1.4 The strength of cement-treated materials

3.1.4.1 Elastic modulus

When a cemented material is loaded beyond a certain limit microcracking first develops at the interface between coarse particles and the matrix. The extent of the microcracking increases upon subsequent loading, and eventually the microcracks join up, the matrix disintegrates, and the treated material no longer behaves like a cemented material. Laboratory flexural and compressive tests have indicated that microcracking starts at about 35 per cent of the ultimate strength and at about 25 per cent of the strain at break. During initial loading the material is linear-elastic up to the onset of microcracking and thereafter it becomes non-linear and non-elastic. A general stress/strain curve for cemented materials can therefore be prepared (see Figure 1). The slope of the initial straight line portion represents the elastic modulus of the cemented material. Equations for estimating the elastic moduli of some cemented materials are given in Appendix A.

3.1.4.2 Factors affecting strength

The bond strength between the particles is influenced mainly by the type of soil, the amount of cement and the density of the compacted material. Compaction moisture content and curing conditions also influence the strength. Some of these effects are shown in Figures 2 to 5. They are examples and should not be used for estimating strength. The strength increases more or less linearly with cement content but at different rates for different soils (see Figure 2). The importance of rapid and efficient mixing and compaction is illustrated in Figure 3, which shows the loss in density and strength with increase in time between mixing and compaction. Figure 4 shows the increase in strength with age. It has been found that the 28-day strength is between 1,4 and 1,7 times the 7-day strength. For estimation purposes a factor of 1,5 may be used. The rate of gain in strength increases as the temperature increases and this effect has been used

to develop accelerated test methods, i.e. curing at high temperatures to give an early indication of the long-term strength. The effect of curing temperature on strength is shown in Figure 5.

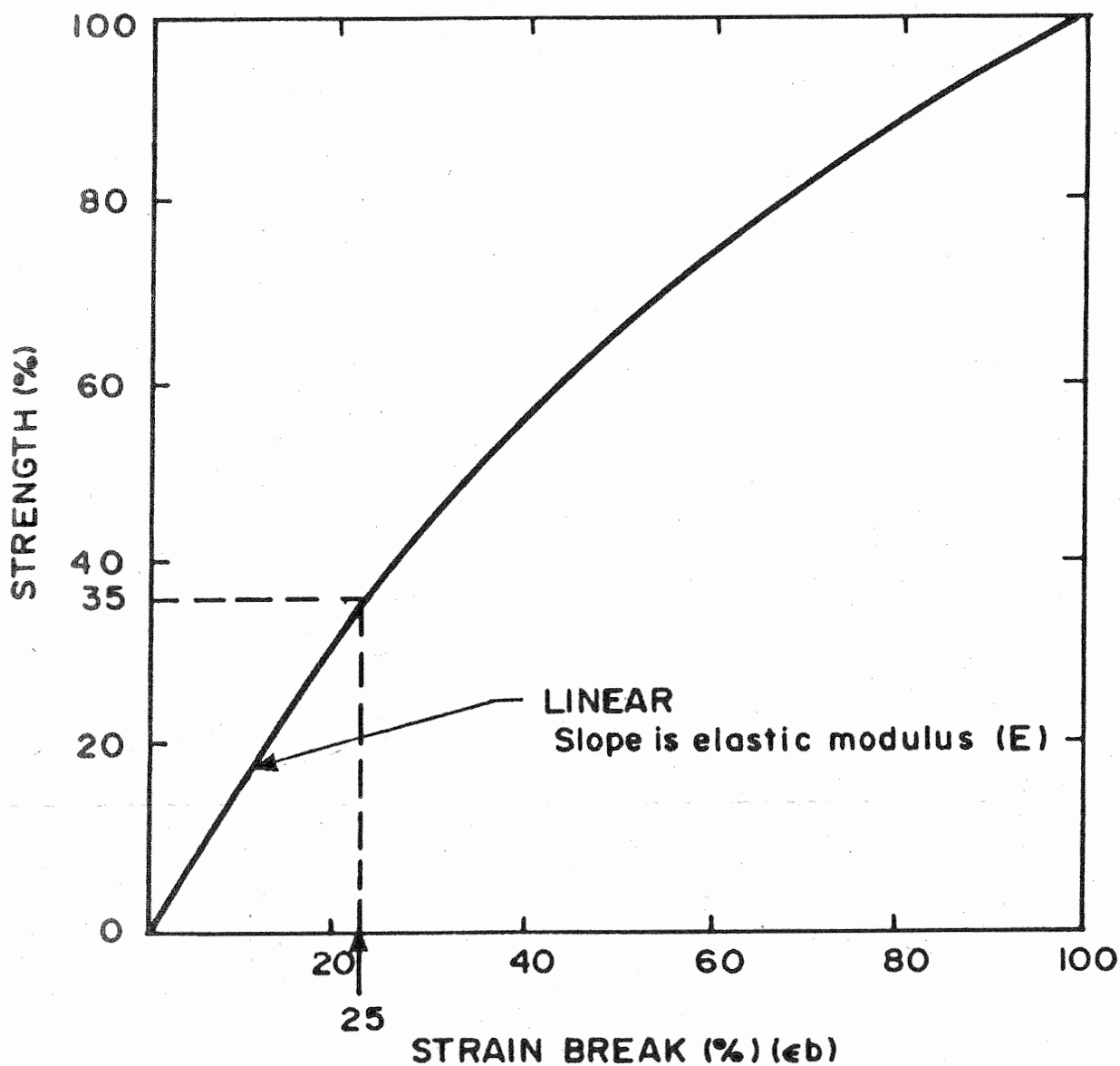


FIGURE 1

Typical stress/strain relationship of cemented materials

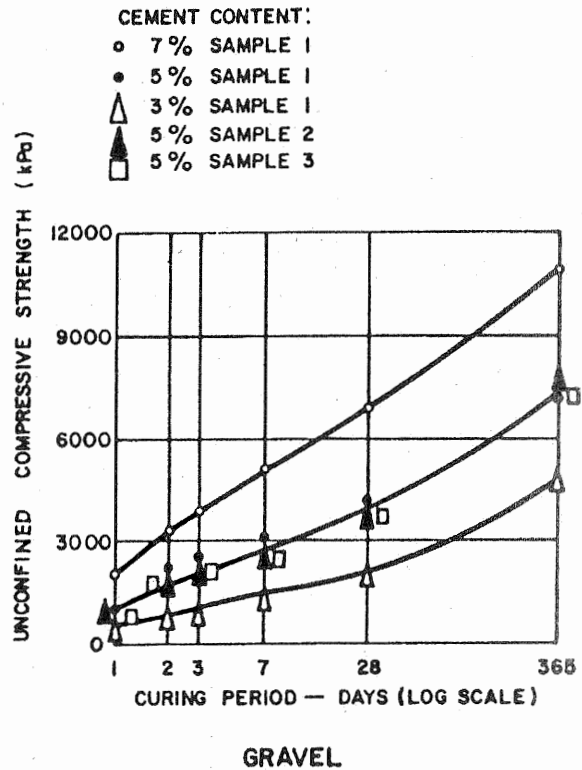
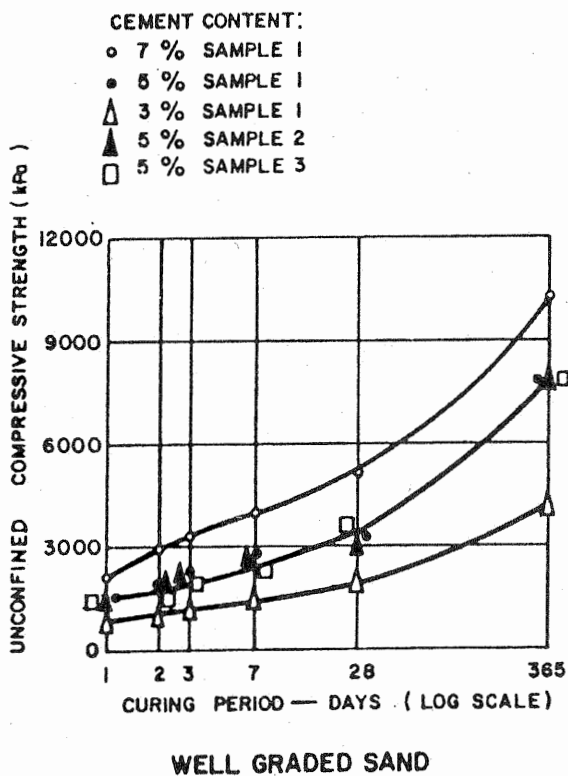
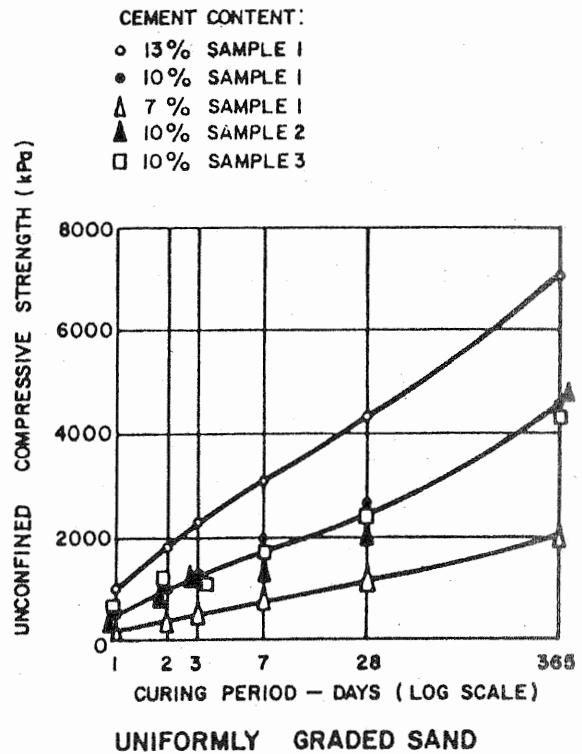
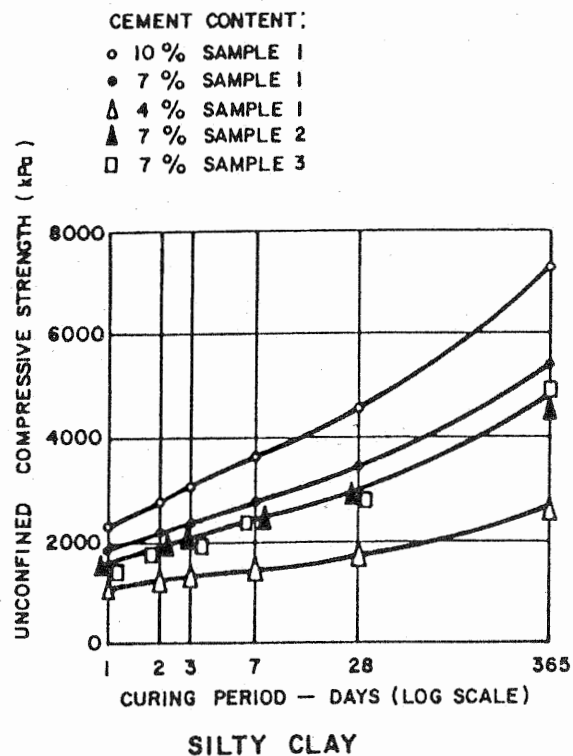
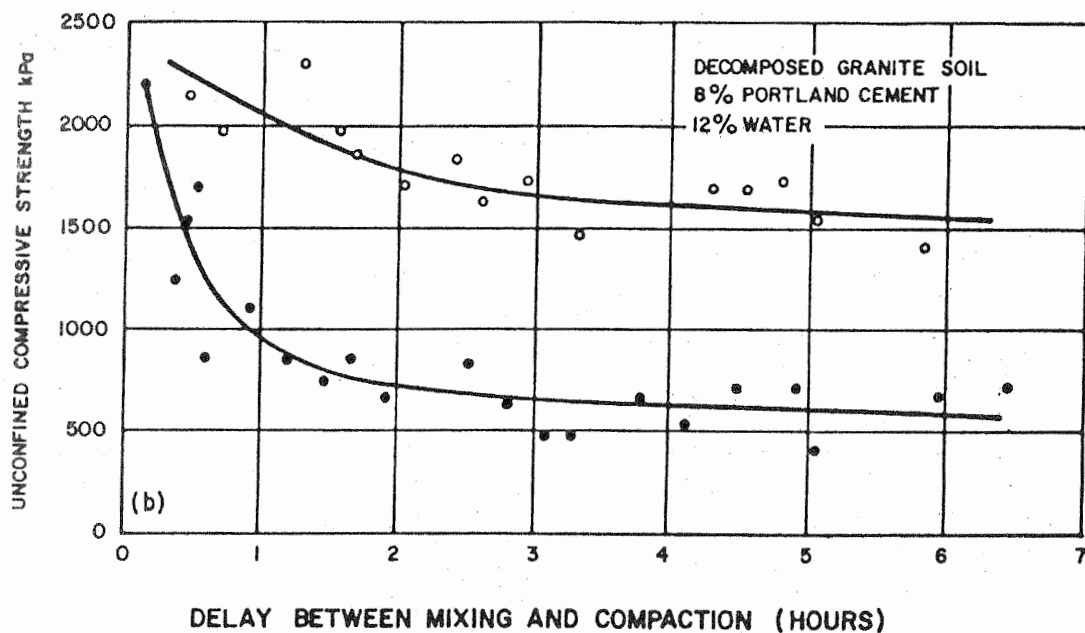
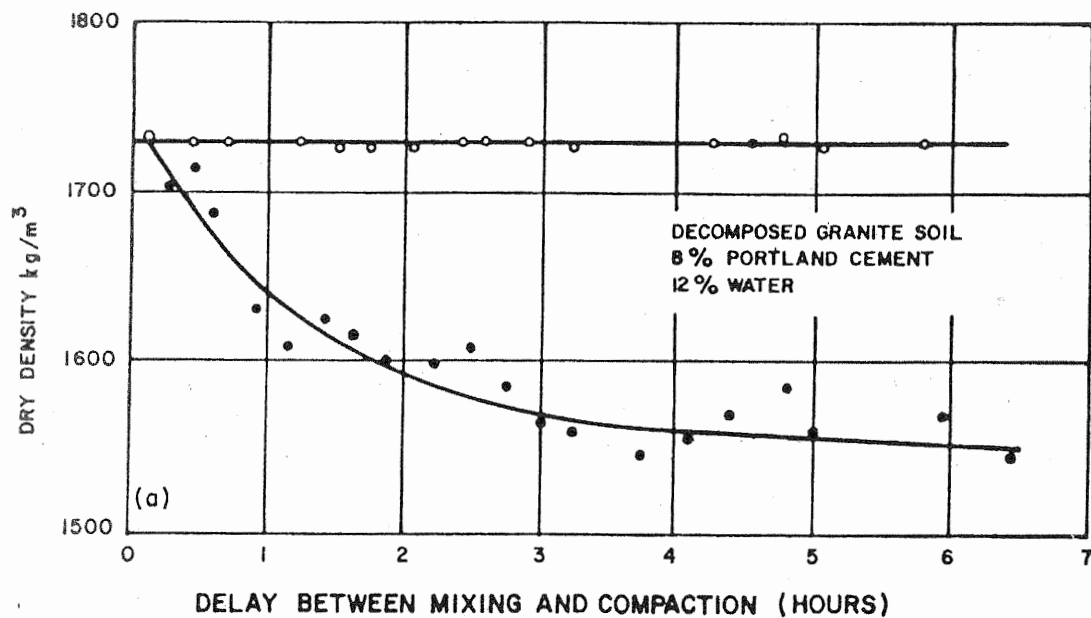


FIGURE 2

Relationships between unconfined compressive strength and curing period for various soils stabilized with various percentages of OPC

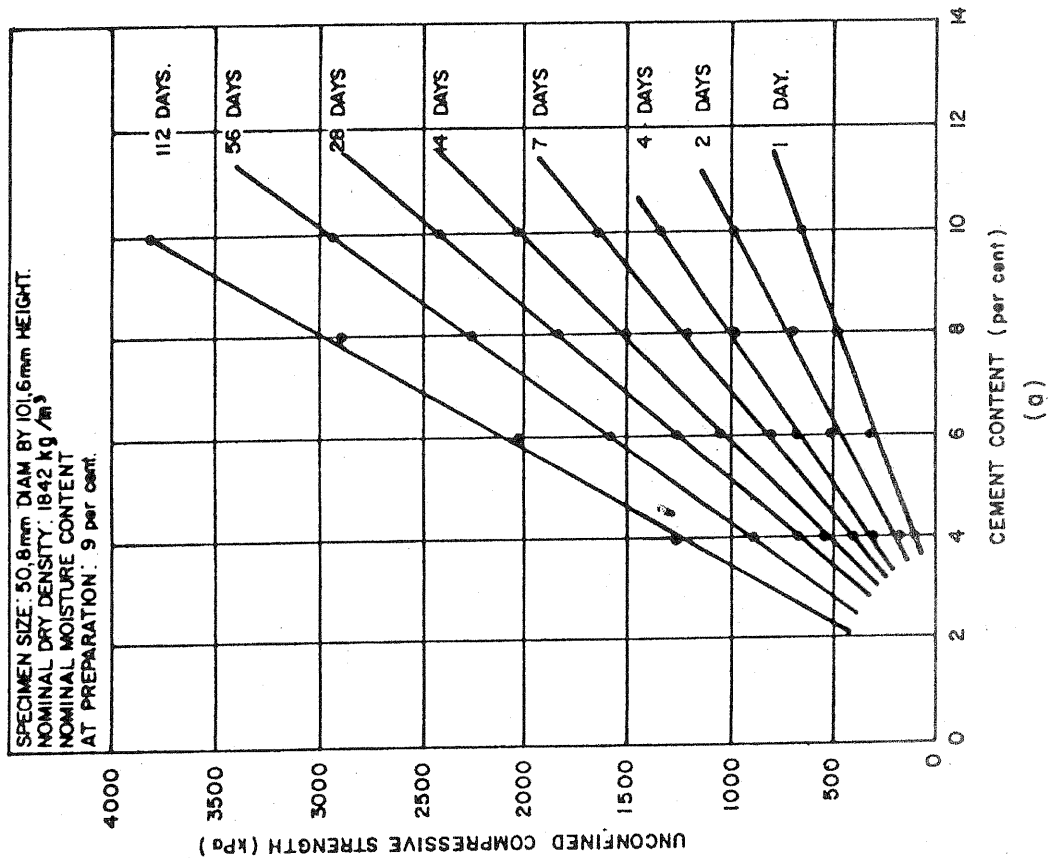


LEGEND:

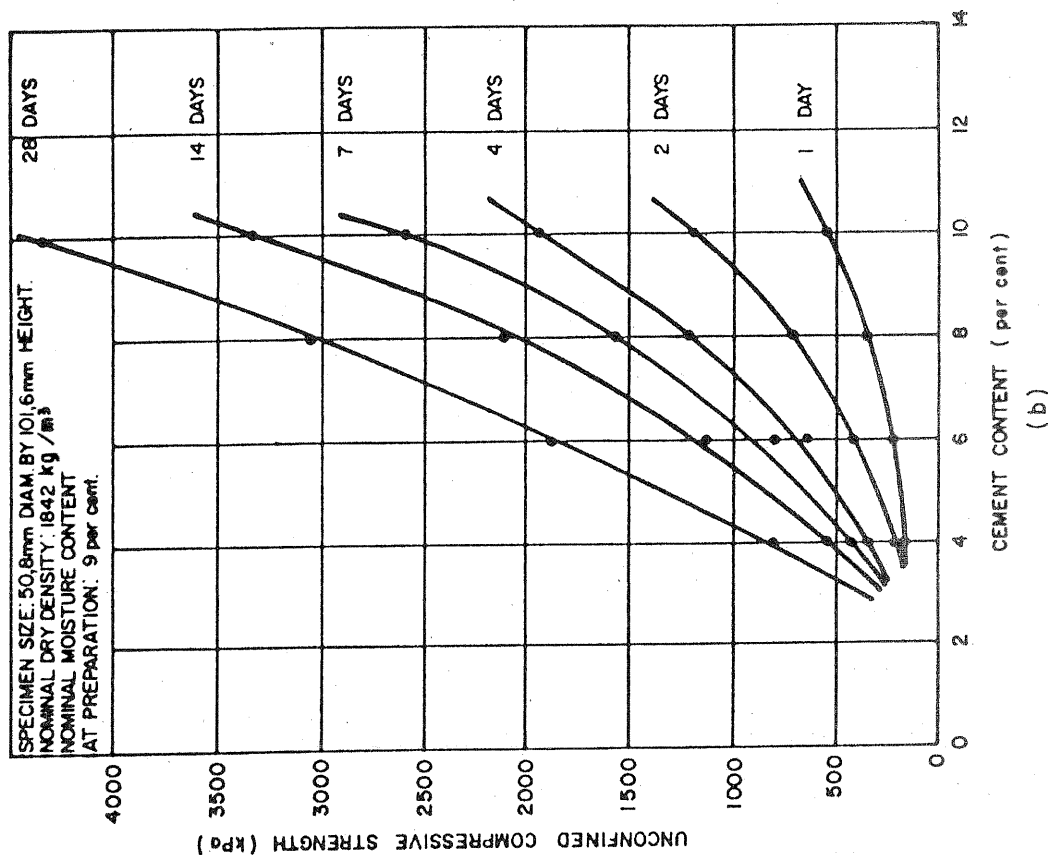
- SAMPLES PREPARED IN CYLINDERS, 50,8mm DIAMETER, 101,6mm HEIGHT, TO CONSTANT DRY DENSITY (1730 kg/m^3)
- SAMPLES PREPARED IN CYLINDERS, 50,8mm DIAMETER, 101,6mm HEIGHT, BY USING CONSTANT COMPACTION EFFORT (THE EFFORT, WHICH PRODUCED A DRY DENSITY OF 1730 kg/m^3 WITH NO TIME LAPSE, WAS KEPT CONSTANT)

FIGURE 3

The effect of a time lapse between mixing and compaction on dry density and unconfined compressive strength in laboratory testing



ORDINARY PORTLAND CEMENT



PORTLAND BLASTFURNACE CEMENT

FIGURE 4

Relationships between unconfined compressive strength and nominal cement content at various times of curing for a sand stabilized with ordinary Portland cement and Portland blastfurnace cement

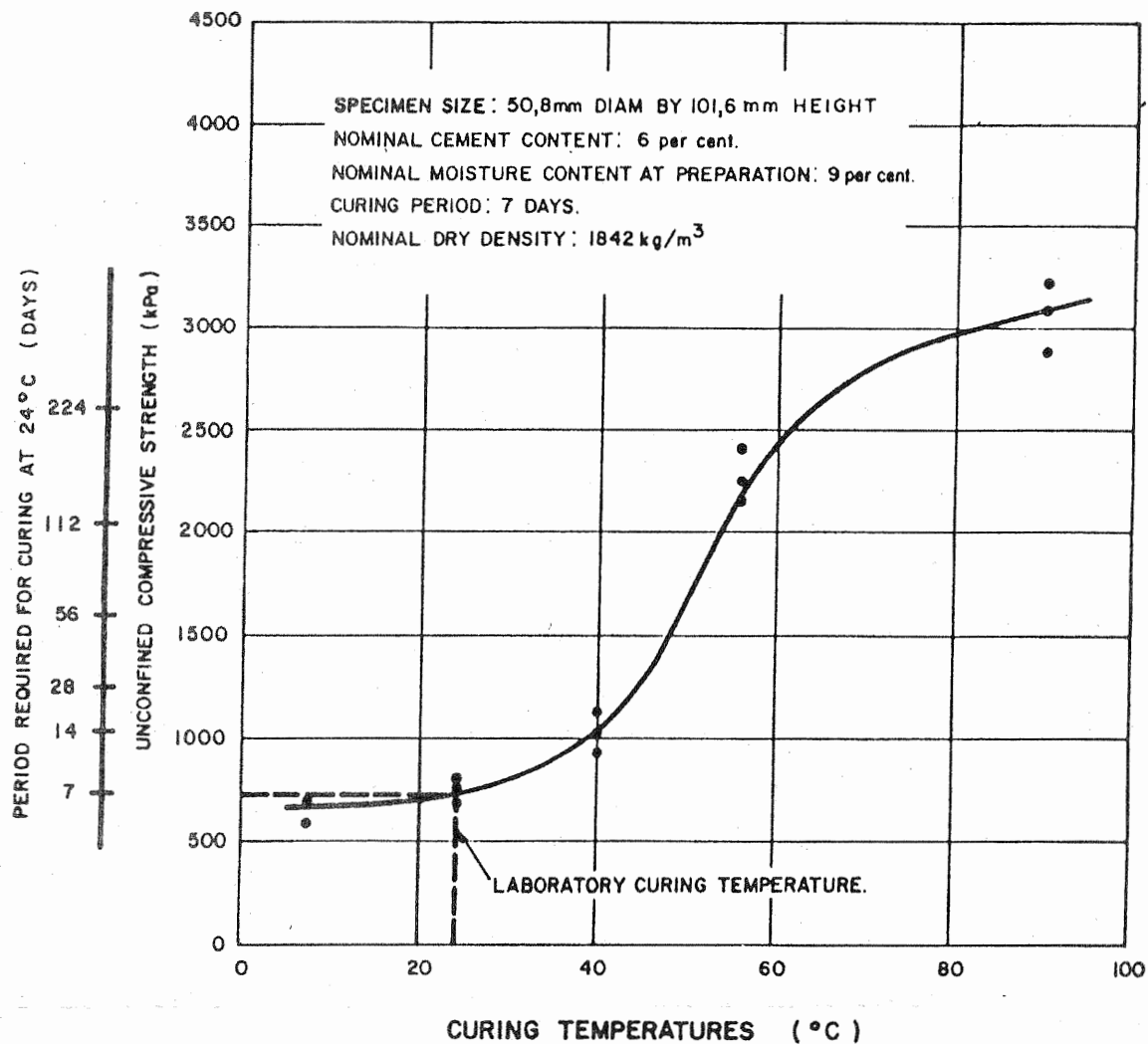


FIGURE 5

Relationship between unconfined compressive strength and curing temperatures for a sand stabilized with ordinary Portland cement

3.1.5 Fatigue behaviour

If a cemented material is subjected to repetitive loading within its elastic range and is not loaded beyond the stress (or strain) at which microcracking begins, then the material remains intact for an indefinite period since cracking does not develop. In structural pavement design this approach is not used since it is too conservative. In practice, cemented materials will suffer fatigue but it must be ensured that the fatigue life is not expended prematurely by the overstressing of the layer.

The fatigue properties of cemented materials show semi-brittle behaviour. Once microcracking has developed its growth can be rapid, particularly under controlled stress conditions, and when the microcracks interconnect fatigue occurs fairly rapidly. This is also demonstrated by the very small levels of strain the material can withstand and the extreme sensitivity of its fatigue life to small changes in the applied strain.

3.1.6 Cracking in cement-treated layers

Cracks in cement-treated layers cannot be avoided and must be accepted as an essential feature of cement treatment. However, cracking may cause structural and maintenance problems and there should be an understanding of the mechanism of crack formation and the means of controlling and accommodating cracks so that they do not have an adverse effect on the performance of the pavement.

There are essentially two types of cracks in cement-treated layers:

- (a) cracks that are not caused by traffic and that are usually referred to as "initial cracks"; and
- (b) traffic-associated cracks.

(Non-traffic-associated cracks such as cracking caused by expansive clays and unstable embankments are not discussed here.)

3.1.6.1 Initial cracking

Initial cracking is caused by drying shrinkage or thermal effects or both and is independent of traffic. In the majority of cases drying shrinkage is probably the main cause of initial cracking and volume changes due to temperature variations may be regarded as a contributory cause. The soil type, the compaction moisture, the rate of drying and the cement content are factors that influence the degree and nature of the cracking.

- (a) Shrinkage cracking

Shrinkage cracking is a natural characteristic of cemented materials and it provides evidence that a hardened material has been produced. When the cement-treated material shrinks, friction develops between the treated layer and the underlying material and consequently internal

stresses are induced. The stresses eventually exceed the tensile strength of the treated material and cracking occurs. The cracks usually form rectangles. Some cracks appear after a few days whereas others appear up to four months later (see Figure 6). Cracks are often 3 to 6 m apart but they may be as close together as 1 m or as far apart as 20 m.

The spacing and width of the cracks are largely determined by the rate of strength development relative to the rate of shrinkage stress development. If the shrinkage stresses exceed the tensile strength at a relatively low strength then the cracks will be more numerous, narrower and more closely spaced (see Material A in Figure 7). Such cracks will vary in width from fine hair cracks to 1 mm, and they are usually up to 2 m apart. If the material develops a greater tensile strength before the shrinkage stress exceeds the tensile strength, there will be fewer cracks, which will be wider and spaced further apart. They may be 2 to 3 mm wide and 4 to 6 m apart (see Material B in Figure 7; Material C has not cracked yet).

The cracks are wider at the top than at the bottom and the vertical faces are irregular, which ensures effective load transfer.

The control of shrinkage cracking is discussed in 7.4.

(b) Cracking due to thermal stresses

Once cracks have developed, the net effect of the thermal stresses becomes virtually negligible in comparison with that of the traffic-associated stresses. Thermal stresses may therefore be considered unimportant in cracked treated layers and may be disregarded for structural pavement design purposes⁹.

The amount of thermal movement at the cracks is about 0,1 to 0,3 mm, and the use of an untreated layer on top of the treated layer will help to insulate the treated layer against thermal stresses.

3.1.6.2 *Traffic-associated cracking*

Cracks associated with traffic are caused by traffic overstressing the cement-treated layer. Traffic-associated cracking may occur in a cemented layer in any position in the pavement, but in practice cracks in the bituminous surface have been observed mostly in pavements where the treated layer has been used as a base. The cracks are usually closely spaced and, with continual overstressing, connect to form circular or square blocks (see Figure 8). The cracks become progressively more numerous and the blocks become smaller, eventually ranging from about 50 mm to 1 m in diameter. The combined action of free water and traffic, i.e. the generation of positive porewater pressures, often results in fine material from the pavement layers being deposited in the cracks and on the surface. The fines discolour the surface along the cracks and make them clearly visible. This phenomenon is referred to as "pumping".

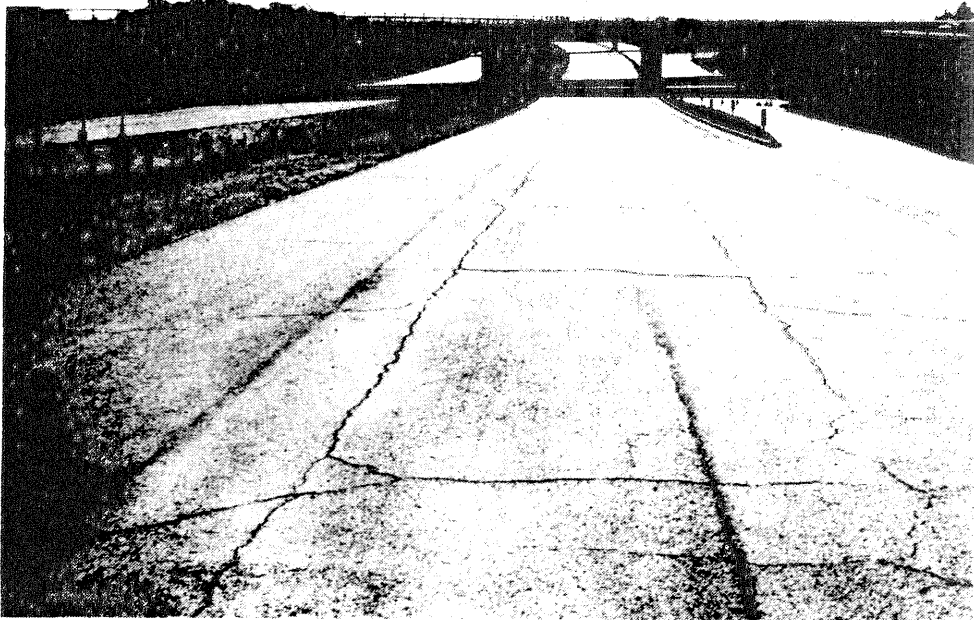


FIGURE 6

Shrinkage cracking in a pavement with a cement-treated base

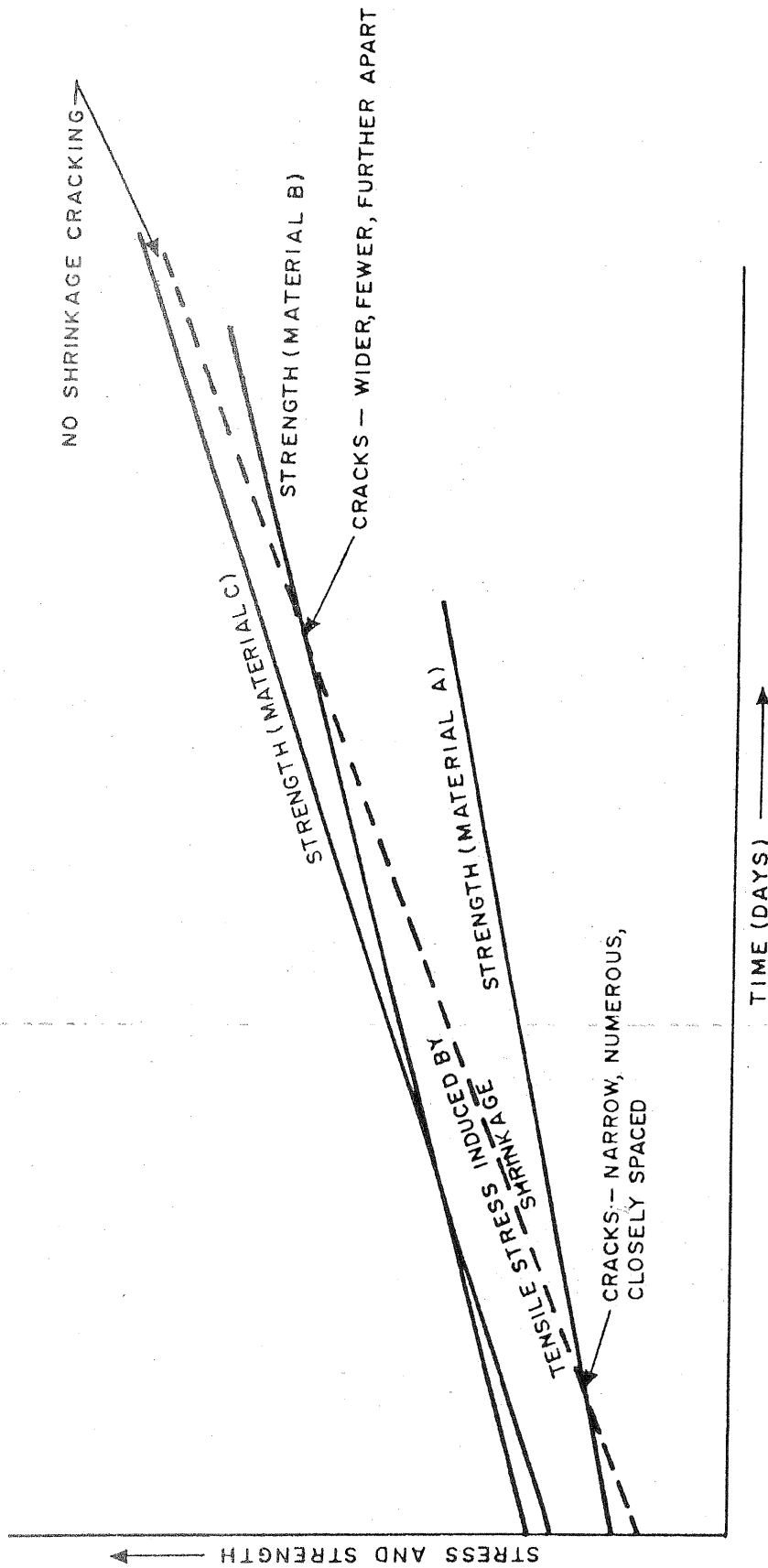


FIGURE 7

Cracking as a result of the interrelationship between shrinkage stress, strength and time

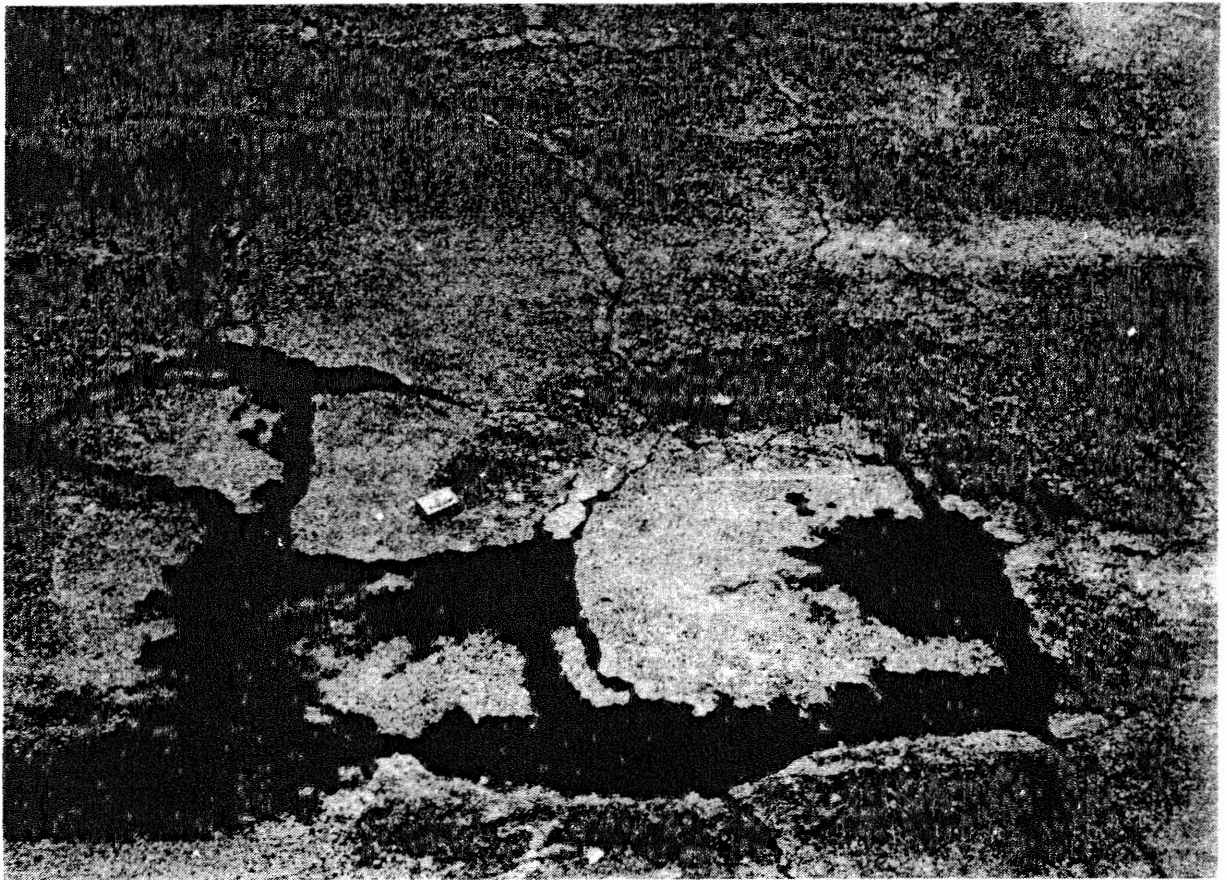


FIGURE 8

Traffic-associated cracking in a cement-stabilized base

In the past many design problems were experienced with cement-treated pavement layers, mainly because the layers were too thin and inadequately supported and because of excessive shrinkage. Such cement-treated layers are very sensitive to overloading and generally exhibit a value of $n \approx 6$ in the load equivalency formula (given below) in the pre-cracked phase. The value decreases as the degree of cracking increases.

$$\text{Load equivalency formula} = \left(\frac{W}{80} \right)^n$$

where W = axle load in kN

n = exponent

This high degree of sensitivity to overloading is taken into account in the mechanistic method and designs are produced that are thick enough to cope with both normal and, to some extent, overloaded vehicles. The degree of shrinkage is controlled by avoiding materials that are too strongly stabilized; this is done by controlling the upper limits specified for the unconfined compressive strength (UCS). These are given as a guide in Table 1, along with the most likely moduli in the pre-cracked and post-cracked phases¹⁰ (the control of cracking is further discussed in 7.4).

TABLE 1
Modulus of cemented material

Pavement material (cemented)	UCS	Pre-cracked phase		Post-cracked phase	
		Modulus range	Recommended modulus (min)	Under bound materials	Under a cracked or untreaded material
	(MPa)	(GPa)	(GPa)	(GPa)	(GPa)
Crushed stone	6-12	7-30	14	1,5	1,2
Stone or gravel	3-6	4-14	8,5	1,0	0,75
Gravel	1,5-3	3-10	6,0	0,75	0,50
Gravel	0,75-1,5	2-7	3,5	0,50	0,30

Poisson's ratio = 0,35

Recent work has shown that the post-cracked phase of a cement-treated sub-base adds substantially to the useful life of the pavement. Measurements of deflections at various depths within the pavement have demonstrated that the initial effective modulus of this material is of the order of 3 000 to 4 000 MPa, which is shown in Figure 9 (a).

This relatively rigid subbase generally suffers fatigue under traffic, or in some cases even under construction vehicles, and assumes a lower effective modulus (500 to 1 000 MPa). This change in modulus does not normally result in a marked increase in deformation, but changes in resilient deflection (as measured with the Benkelman beam) and radius of curvature are observed (see Figure 9 (b)).

In the mechanistic design¹¹ these phases have been designated the pre-cracked and post-cracked phases. The design accommodates these changes in modulus of the subbase and although the safety factor in the base will be reduced, it will still be well within acceptable limits.

The eventual modulus of the cemented subbase will depend on the quality of the material originally stabilized, the cementing agent, the effectiveness of the mixing process, the absolute density achieved and the degree of cracking. The ingress of moisture can significantly affect the modulus in the post-cracked phase. In some cases the layer may behave like a good quality granular material with a modulus of about 200 to 500 MPa, but in other cases the modulus will reduce to about 50 to 200 MPa. This change is also shown diagrammatically in Figure 9 (a). The net result is that the modulus of the cemented subbase decreases to very low values, which causes fatigue and high shear stresses in the base. Cracking of the surface will generally occur and the ingress of water may cause pumping from the subbase.

For high-quality, heavily trafficked pavements it is necessary to avoid materials that will eventually deteriorate to a very low modulus. Many of these lower class materials have, however, proved to be adequate for lower class traffic.

TRH4³ gives recommendations for the strength and thickness of cemented layers and their position in the pavement for various traffic categories.

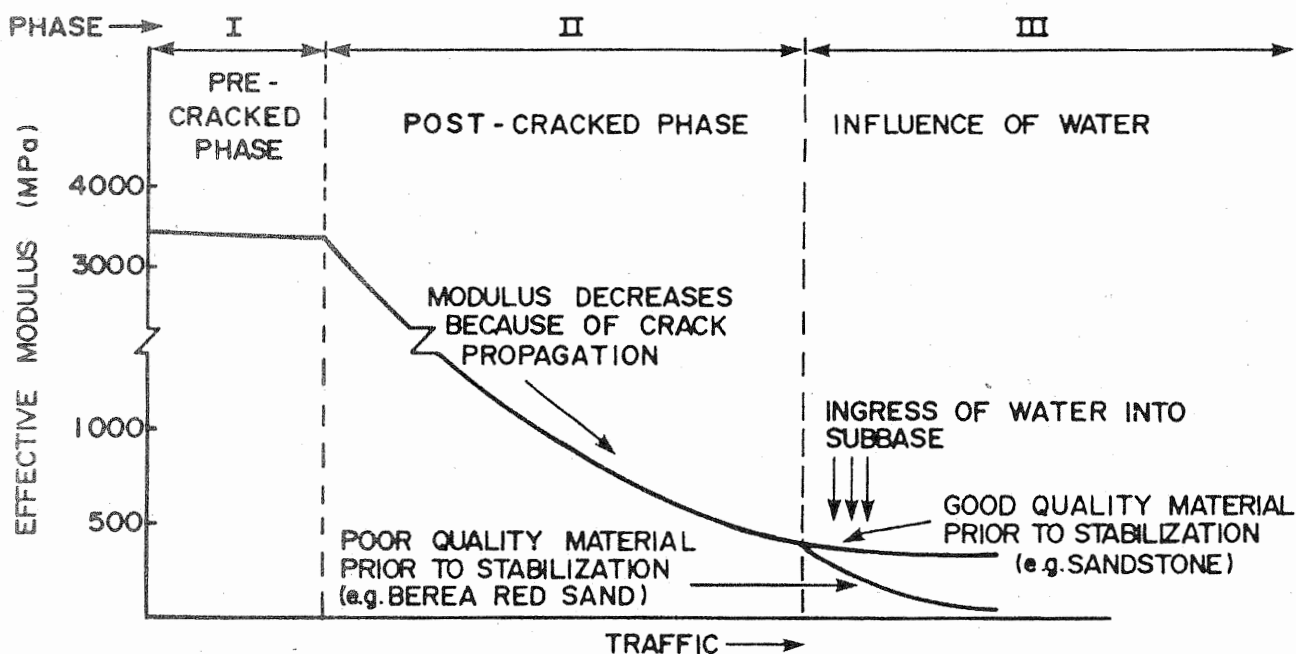
3.2 LIME-STABILIZED MATERIALS

3.2.1 The reaction between soil and lime

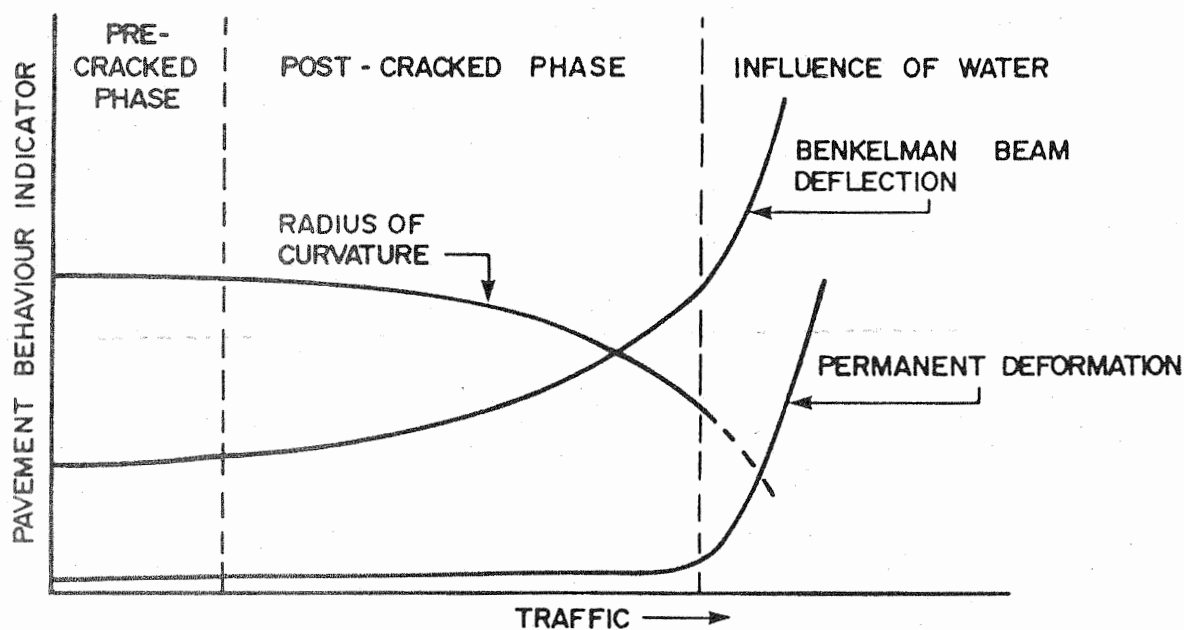
Two basic though complex reactions apparently take place when lime is mixed with a soil, namely:

- (a) a fairly rapid and sometimes almost instantaneous amelioration that may involve the exchange of ions; and
- (b) a pozzolanic reaction taking place over a period of time ranging from a few minutes to several months or longer.

In both cases there is a chemical reaction between the lime and the soil.



(a) History of cemented subbase modulus as a function of traffic loading



(b) Change in indicators of cemented subbase pavement behaviour with traffic

FIGURE 9

Cemented subbase behaviour with traffic

3.2.1.1 *Ion exchange*

This relatively rapid reaction usually involves both cations and anions and is accompanied by flocculation and the formation of agglomerations caused by clay particles adhering to one another. This increases the plastic limit and thus the Plasticity Index (PI) is reduced, whereas the liquid limit may remain unchanged, decrease or increase. The material becomes more friable and more workable and the strength is usually increased (see Table 2 and Appendix B).

On the Witwatersrand lime is often added to acidic, sulphate-contaminated crushed stone to prevent salt damage. This can be regarded as a type of ion exchange although clay minerals are not involved.

3.2.1.2 *Pozzolanic reaction*

If sufficient lime is added to a soil the pH is increased to about 12,4, which is the pH of saturated lime water at 25 °C. At this high pH reactions take place between lime and clay minerals and other pozzolans, such as amorphous silica, to produce cementitious hydrated calcium silicate and aluminate gels similar to those present in hydrated Portland cement. Crystallization and hardening of these gels are largely responsible for the strength developed.

3.2.1.3 *Initial consumption of lime*

The amount of lime required to satisfy the soil-lime reaction varies considerably with different soils and a test for the initial consumption of lime (ICL) has been developed. This is a quick test to determine the amount of lime required by the soil-lime mixture to maintain the lime-saturated pH for one hour after the lime has been added (usually a pH of 12,4 at 25 °C). If the ICL of the soil fines is greater than 3,5 per cent, the lime demand is considered to be high. It has been found that most weathered basic igneous rocks have a high demand for lime although certain other materials derived from sedimentary rocks, for example, may also have a high lime demand. In an analysis of random samples from various parts of South Africa, about 25 per cent of the samples had a lime demand that would be regarded as high¹².

Sufficient lime should be added to satisfy the ICL and additional lime may be necessary for the formation of cementing compounds (pozzolanic reaction). The amount of lime required for the development of a significant pozzolanic reaction is best determined by strength tests.

It should be noted that a high ICL is significant not only in lime stabilization but also in cement stabilization, since soils with a high ICL will consume part of the calcium in the cement and thus the strength of the cement-treated material may be reduced. Sometimes this can explain why unexpectedly high cement contents are necessary to satisfy strength criteria. If soils with a high ICL are treated with cement it may be necessary to increase the cement content or the ICL may first be satisfied by adding lime before the cement is added. The ICL of amorphous silica, which gives strong cementing reactions, is also very high.

TABLE 2

The effect of lime on the plasticity of some South African materials

Material	Region/ Route no.	GM	Lime content (%)				
				0	2	3	4
Decomposed sandstone (Cape supergroup)	Pinetown, Natal	1,2	LL	23	21	-	-
			PI	8	3	SP	SP
	N3-1	1,66	LL	24	24	-	-
			PI	10	6	SP	SP
	N3-1	0,90	LL	26	24	-	-
			PI	9	5	-	-
Decomposed granite (Basement)	N3-12 (Tvl)	1,6	LL	34	34	34	-
			PI	12	7	4	SP
	N3-12 (Tvl)	1,9	LL	40	36	-	-
			PI	11	5	SP	SP
	N2-28 (Natal)	2,0	LL	38	35	35	-
			PI	13	5	2	SP
Decomposed dolerite	N2-27 to 28 (Natal)	2,7	LL	34	27	-	-
			PI	14	4	SP	SP
	N2-27 to 28 (Natal)	2,3	LL	36	34	32	-
			PI	17	7	3	SP
	N2-27 to 28 (Natal)	2,3	LL	46	33	29	29
			PI	21	10	6	4
	N3-6 (Vaalkrans)	2,0	LL	31	26	-	-
			PI	9	3	SP	SP
	N3-6 (Vaalkrans)	2,5	LL	38	28	-	-
			PI	15	6	SP	SP
Decomposed lava (Ventersdorp)	N103 (Tvl)	0,45	LL	48	30	28	-
			PI	21	10	4	SP
	N103 (Tvl)	1,74	LL	53	46	40	-
			PI	24	13	5	SP
	N103 (Tvl)	0,31	LL	65	46	40	-
			PI	36	10	6	SP
Silty sand (Berea red)	Natal coast	0,62	LL	36		28	27
			PI	18		10	6

GM: Grading modulus

LL: Liquid limit

PI: Plasticity Index

SP: Slightly plastic

All samples were compacted into CBR moulds, cured at 70 °C for 24 hours, soaked in water for 4 hours, and tested in unconfined compression before being broken up for Atterberg limit testing. The first four materials were also cured at room temperature for 56 days and the same results obtained.

3.2.2 Soils suitable for treatment with lime

Lime is most effective when there is a sufficient amount of clay in the soil to react with the lime. A rule that has frequently been used is that lime is suitable for treating materials with a PI higher than 10, to reduce the PI and to increase the strength. If the PI is lower than 10, cement should be used. Although this rule can be used as a rough guide in so far as the PI is a rough indication of the amount of clay available for reaction with the lime, it should be stressed that there are many materials with PIs below 10 that react strongly with lime. Natural gravels such as certain sandstones, calcretes, decomposed granites and decomposed dolerites are examples of materials whose strength may increase considerably when treated with lime, in spite of PIs that may be below 10. Materials containing amorphous silica, such as some calcretes, react strongly and rapidly with lime and may even have higher strengths than when treated with cement. Strength tests can be used to establish whether materials with a low plasticity can be successfully treated with lime. Most plastic materials can be treated with lime although some clay minerals react more strongly than others. Materials containing montmorillonite usually react strongly with lime but lime also reacts well with kaolinite, illite and many other clay minerals.

3.2.3 Some features of lime stabilization

There are features of soil-lime reactions that have certain advantages in practice. The lime content and costs are based on mass and since the density of lime is about half the density of cement, the larger volume of lime is easier to mix than an equal mass of cement. The reaction that takes place when the lime makes contact with the damp soil usually has the effect of reducing the plasticity and making the soil more friable and more workable. These advantages are greater when clayey materials are treated.

Soil-lime reactions cause clay particles to move closer together and form small "aggregations". This results in a reduction in density, which also occurs in cement. However, the rapid soil-lime reaction improves the properties of the material even though the density has been reduced, so that it can be regarded as a "modified" material. It is also easier to use lime than cement under wet or damp conditions. The cementing or pozzolanic reaction of lime can be a relatively slow process and in such cases there is not the same urgency to complete the construction as in the case of cement. This can be an important advantage in practice. However, when high early strengths are developed in lime-treated materials similar to the strengths obtained with cement, the mixing and compaction procedures should be given more or less the same time limits as for cement stabilization. This aspect is recognized in the permissible construction time limits given in 7.2.4.

Treatment with lime can therefore offer certain advantages over cement treatment. However, cement is a more suitable stabilizer than lime when a specific

tensile or compressive strength is an essential requirement, since it is usually easier to control the strength of soil-cement than that of soil-lime.

3.2.4 The strength of lime-treated materials

The factors influencing the strength of lime-treated soils are similar to those affecting the strength of cement-treated soils, i.e. the soil type, the amount of lime added and the compacted density. Curing conditions and compaction moisture content are also contributory factors. However, the soil type and the clay mineralogy in particular are of importance. If conditions are favourable for a pozzolanic reaction the elastic modulus will increase and the material will develop a significant compressive and tensile strength. In contrast to cement, there may be cases where there is an optimum lime content for strength beyond which the strength remains constant or may even drop. If there is a significant increase in the modulus of elasticity and the lime-treated material behaves like a "cemented" material, the elastic properties and fatigue behaviour can be compared with the properties of cement-treated materials described in previous paragraphs.

Figure 10 shows the strength characteristics of lime-stabilized granitic soils containing kaolinite clay, which are typical of granitic soil in many parts of Southern Africa. The variables shown to influence the strength are lime content, age and clay content. Testing ceased when specimens reached an age of 12 months, but the curves indicate that the strength increase would continue provided that there was sufficient lime and kaolinite.

Appendix B shows the effect of lime on the strength (CBR and accelerated UCS) of a variety of materials.

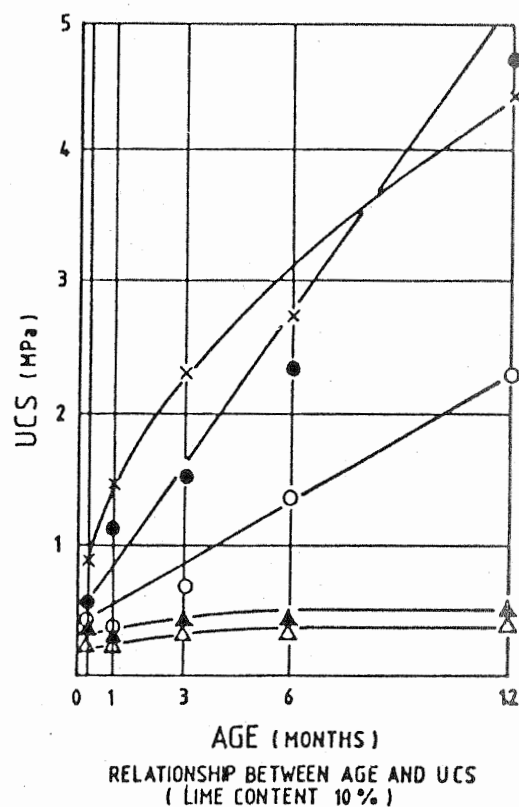
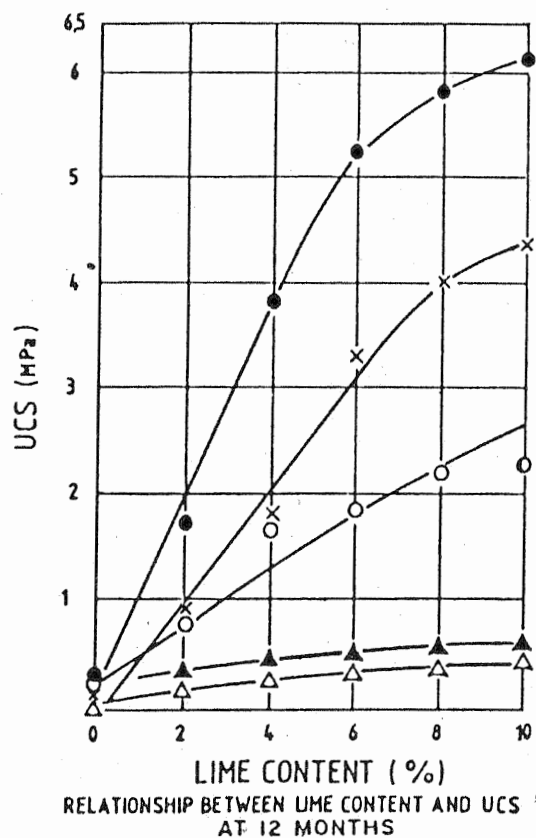
3.2.5 Cracking in lime-treated materials

Initial cracking develops in lime-treated pavement layers and the cracks form rectangles, like the cracks in cement-treated layers. The shrinkage characteristics and rate of strength development of lime-treated materials are generally different from those of cement-treated materials; usually the cracks in lime-treated materials are narrower and less extensive and therefore less significant than those in cement-treated materials. However, some lime-treated materials, such as some calcretes and sandstones, may crack as badly as cement-treated materials (see Figure 11).

It is believed that cracks in lime-treated layers may sometimes exhibit "self-healing" properties, but it must be stressed that this has not been proved, and this postulated feature of lime stabilization cannot be relied upon.

3.2.6 Carbonation in stabilized materials

After stabilization significant carbonation of the lime may take place in the treated layers¹³. Carbonation often commences on the surface and proceeds downwards into the treated layer, and may penetrate fairly rapidly (about 1 mm a day). It appears that stabilization ceases at a pH of less than about 11, and



	DEPTH SAMPLED (mm)	PI	PARTICLE SIZE (%)			KAOLINITE (%)
			SAND	SILT	CLAY <2μm	
△	0-225	SP	89	4	7	3
▲	225-450	SP	86	4	10	
○	450-650	8	81	4	15	
●	650-800	12	78	3	19	
×	800-1050	24	58	5	37	19

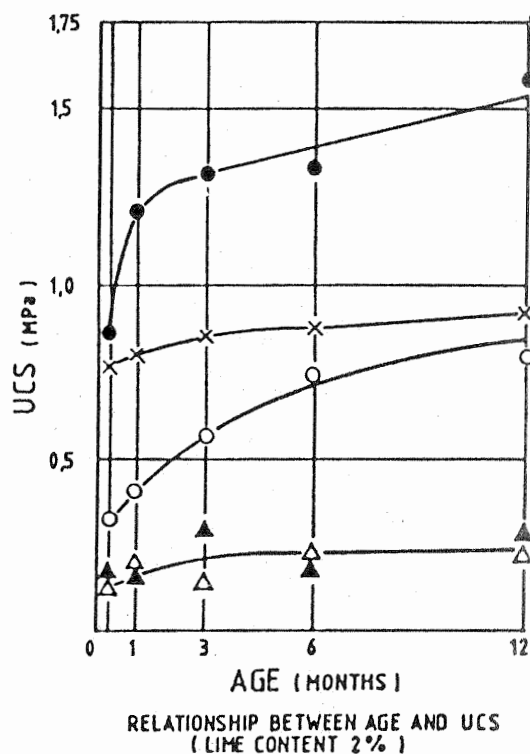


FIGURE 10

The effect of age, lime content and clay content on the strength of a granitic soil

The soil was sampled at different horizons in the same hole to obtain a systematic variation in soil properties (height:diameter ratio of UCS specimens 2:1)

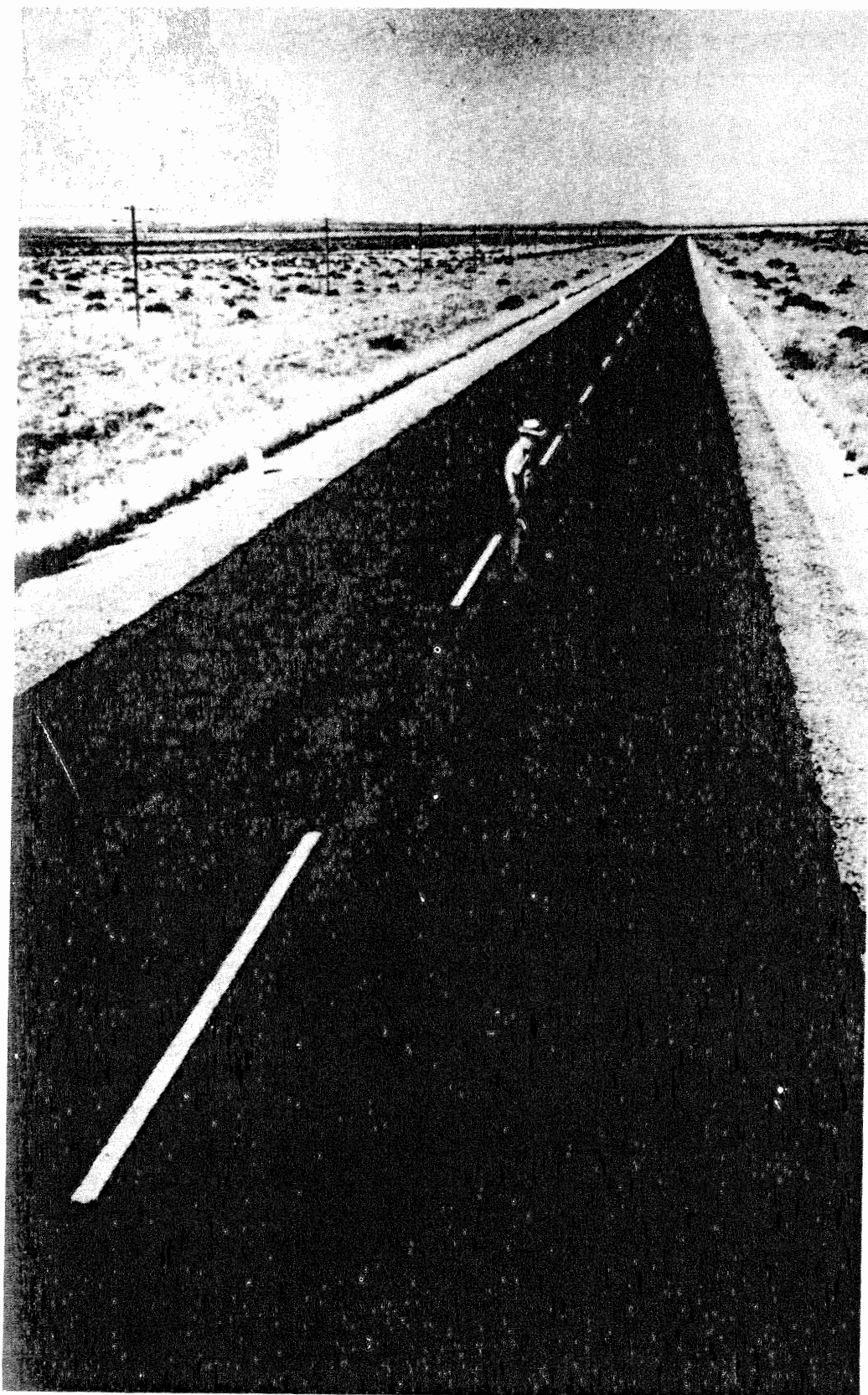


FIGURE 11

Block cracking exhibited by a lime-stabilized calcrete base

unstable products may be formed. This could be an explanation of failures due to surface disintegration that have occurred in some lime-stabilized bases. Carbonation may also occur in MGBS-lime-treated materials and in cement-treated materials. It has also been found that carbonation may progress from the bottom upwards. This appears to have been the cause of the loss of cementation in some cement and lime-stabilized bases. Carbonation may definitely be a factor in cases where a loss in cementation has occurred.

Present knowledge indicates that the best way of reducing carbonation from the top is effective curing and, in the base, the application of both the prime and surfacing as soon as possible after the base has been cured, but in any case not later than about two weeks after compaction (see also 7.2.5 (d)).

It is stressed that the possibility of carbonation should in no way prevent the use of lime and cement as stabilizing agents – the importance of proper curing and the addition of sufficient stabilizer is emphasized. Research on this problem is still in progress.

3.3 BLENDS OF MGBS AND LIME

MGBS has very poor cementing properties but, if it is mixed with lime, cementing compounds are formed with properties similar to those of cement, and similar strengths can be obtained. The rate of increase in strength of MGBS-lime is often slower than that of cement, and allowance for the slower increase in strength must be made when testing MGBS-lime as a stabilizer. A ratio of four parts of MGBS to one part of slaked lime often provides optimum strength, but when the two materials are mixed with the soil, the amount of lime may have to be increased to satisfy the lime demand of the soil. Equal parts of MGBS and lime are often used since this is a convenient ratio in practice even though more lime may be used than is needed.

The action of the lime on plastic soils and the slower cementing action of the MGBS-lime blends can be advantageous in practice. However, sometimes MGBS-lime may react as rapidly as cement.

3.4 BLENDS OF FA AND LIME

The amorphous glassy component in FA combines with the lime to provide cementing silicate and aluminate compounds. Since FA from different sources varies considerably and South African FAs do not appear to react strongly with lime, the reactivity of a specific FA should be determined carefully before it is accepted for soil stabilization.

Apart from a survey of some sources of FA and some laboratory investigations, very little experience has been gained in soil-FA-lime stabilization in South Africa.

3.5 MODIFICATION AND CEMENTATION

The terms "modification" and "cementation" are used to describe the degree or type of treatment. It was stated in 3.2.1.2 that, provided suitable clay minerals or other pozzolans are present, the addition of lime to a soil results in an early reaction accompanied by a change in the soil properties. The most significant changes are a reduction in plasticity and an increase in CBR, but there may be no significant development of compressive and tensile strength as a result of this rapid reaction. This change in soil properties is referred to as "modification". If conditions are favourable for the pozzolanic soil-lime reaction described in 3.2.1.2, the lime-treated material will develop compressive and tensile strength and it can then be regarded as a "cemented" material. Lime-treated materials can therefore also be used as cemented pavement layers provided they satisfy the criteria for cemented materials.

If a very small quantity of cement is added to a soil, the properties may also be modified without much hardening or the development of significant compressive and tensile strength. In such cases the degree of cementation is relatively poor, but the properties of a material can nevertheless be considerably improved in this way. This treatment is also referred to as "modification". An example of modification with cement is the treatment of natural gravel bases of low plasticity with 2 to 3 per cent of cement on roads carrying light or medium traffic.

When a material has developed a significant tensile strength it can be regarded as a cemented material, but there is no clearly defined boundary between cementation and modification. The one state merges into the other. Soil-lime and soil-cement mixes should be designed to comply with the relevant standards for construction materials and this is usually achieved by modification to comply with CBR and plasticity requirements and by cementation to comply with compressive or tensile strength requirements¹⁴ (see Figure 12).

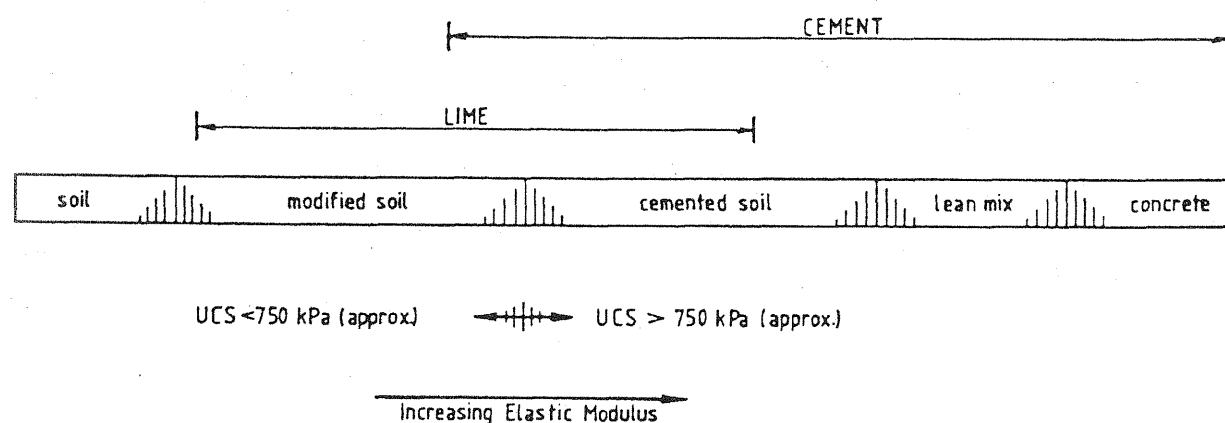


FIGURE 12

Stabilizing mode of lime and cement (adapted from Dunlop¹⁴)

(Boundaries are not clearly defined but represent a merging of one state into the next)

3.6 DELETERIOUS MATERIALS

3.6.1 Total salts, sulphates and acids

Pavement layers stabilized with lime or cement may be damaged by salt crystallization, sulphate attack and acid, all of which can lead to a complete loss of cementation and/or excessive heaving and cracking. Limits are uncertain, but the following are recommended as an interim measure and are applicable before stabilization:

Total salts (conductivity)

Base and subbase: 0,15 S/m (max) at 25 °C

Selected subgrade: 0,40 S/m (max) at 25 °C

Higher values than these may be permissible if special precautions are taken.

Sulphates (all layers)

- (a) The maximum acid-soluble sulphate content* of materials to be stabilized with cement or lime should not exceed 0,25 per cent calculated as SO_3 if the PI exceeds 8 or the percentage passing 2 μm exceeds 12. A maximum of 1,0 per cent is permissible for materials with lower PIs and clay contents. More sulphate is tolerable by sulphate-resisting cements (for low-clay and -PI materials only).
- (b) The maximum water-soluble sulphate content** of all materials within 500 mm of any stabilized layers should not exceed 2,5 g/ℓ of SO_3 . If a 2:1 water:soil ratio is used, a limit of 2,0 g/ℓ should be used.
(From the UK Department of Transport Specification¹⁶ slightly modified for South African conditions.)

NOTE:

- 1 Although BS 1377¹⁵ specifies a correction*, simply crushing the material to pass a 2 mm sieve is recommended instead.
- 2 As the effect of total salts, sulphates and acids on stabilized materials in South Africa is still subject to research it is recommended that, in cases of doubt, the National Institute for Transport and Road Research (NITRR) be approached for the latest recommendations.

Acid (all layers)

A minimum pH of 6,0 is required for all materials to be stabilized with cement or lime and the stabilized layer must not be closer than 500 mm to acidic materials with a pH of less than about 6.

*BS 1377¹⁵, Test 9, corrected for percentage passing 2 mm.

**BS 1377¹⁵, Test 10, no correction for size fraction.

If the sulphate or acid requirements are not met, the UCS should be determined after 7 days' soaking. If the specimens do not exhibit any cracking and the soaked UCS is not less than 80 per cent of the UCS design strength (after a similar curing period) the material should be satisfactory. This test may be used instead of the tests for sulphate requirements or as an additional guide.

3.6.2 Organic compounds

Some organic compounds are believed to interfere with the hydration of the cement, which causes a reduction in strength, but not all organic materials affect the reaction of cement. Deleterious organic materials may cause a fairly rapid reduction in the pH of stabilized materials and the measurement of the pH therefore provides a means of determining whether the cementation will be affected. It has been suggested that organic material is not likely to affect normal cement hydration or lime reaction if the pH of a 10:1 soil:cement mix (by mass) is at least 12,1 fifteen minutes after mixing. Organic materials do not present serious problems in soil stabilization in South Africa but it should be borne in mind that problems may be encountered.

Sugar has serious deleterious effects on cement and this may affect cement stabilization in sugar-cane areas. The presence of sugar in aggregates can be detected (SABS Method 833¹⁷).

4 LABORATORY TEST METHODS

Brief details are given of strength and other tests that can be carried out in well equipped central or field laboratories for the purpose of mix design and quality control.

4.1 CALIFORNIA BEARING RATIO

The California Bearing Ratio (CBR) can be used for **modified** materials since it gives an indication of shear strength when a compressive or tensile strength is not an essential requirement. Values of up to about 150 can be used to assess modified materials, but the CBR is not a sensitive test when high-strength materials are tested and it is not suitable for testing cemented materials.

4.2 UCS TEST

The UCS test is the most commonly used test for evaluating **cemented** materials. Overseas test procedures are usually based on specimens with a height:diameter ratio of 2:1 but in South Africa a 152 by 127 mm mould is used, i.e. it has the same dimensions as the CBR mould. One investigation has indicated that specimens prepared in the CBR-type mould give compressive strength values about 30 per cent higher than specimens with a height:diameter ratio of 2:1. Allowance has been made for this difference, and the strength criteria developed in South Africa are based on values obtained from specimens prepared in the CBR-type mould, suitably adapted for UCS tests. A relationship between CBR and UCS obtained in the Transvaal is shown in Figure 13. These relationships depend on factors such as soil type and quantity and type of stabilizer and should not be used to determine strength parameters.

4.3 UCS TEST WITH ACCELERATED CURING

In the standard UCS test on cement-stabilized materials, the specimens are cured for 7 days at a temperature of 22 °C and a humidity of 95 to 100 per cent, and then soaked for 4 hours. For quicker results an accelerated reaction can be obtained by curing at a higher temperature.

The accelerated strength development at 70 to 75 °C for 24 hours has been used successfully in practice both for design purposes and for field control to predict the 7-day strength of cement-stabilized materials. However, compressive strength criteria are based on the standard curing of 7 days at 22 °C and it is advisable to carry out a few comparative tests, particularly if there is little or no experience of the materials being treated.

Accelerated curing has also been used to provide an early estimate of the long-term strength of lime-stabilized materials and a curing temperature of 50 °C for 45 hours has been used. However, results can only be regarded as a very rough indication of the long-term strength of lime-stabilized materials, owing to the complexity of soil-lime reactions, and caution should be exercised when interpreting results.

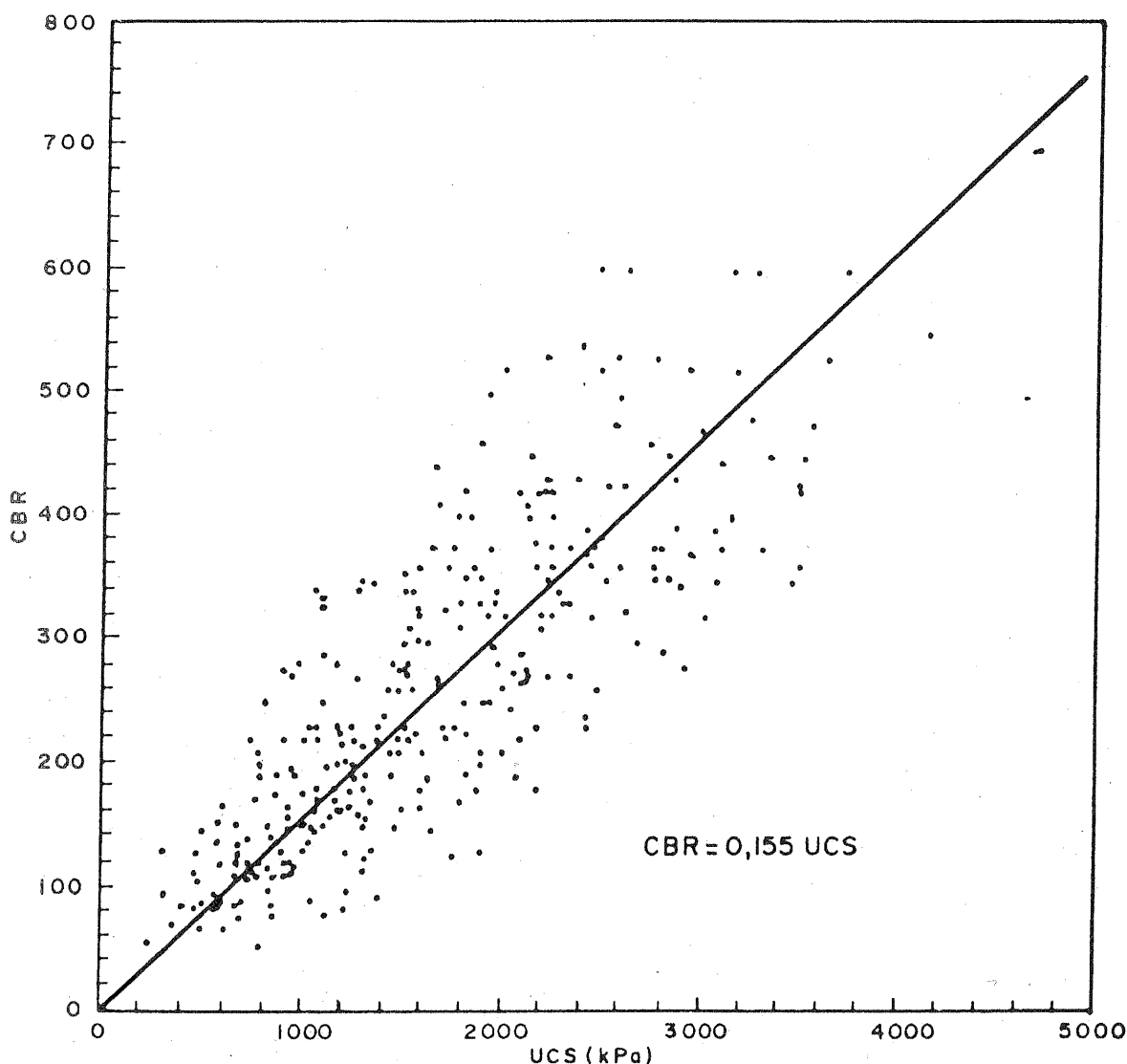


FIGURE 13

CBR/UCS relationship

4.4 WET/DRY TEST

(Sum 002 TRH14 ll. 24)

The durability of a cemented material is an important property especially if the material is to be subjected to severe wetting and drying and/or severe loading conditions. An example of such conditions is a cement-treated subbase subjected to pumping action under a concrete slab. There may also be instances where it is suspected that the properties of the natural material are such that the durability of the cemented bonds may be affected under normal wetting and drying conditions. When the durability is in doubt the wet/dry test (Method A19 of TMH11), which was originally developed in the USA to assess the durability of cemented bases, can be carried out. Specimens are subjected to 12 cycles of wetting and drying and brushed with a wire brush after each cycle. It is a controversial test and is often considered too severe for South African conditions. Generally acceptable criteria have not been established and it is also a lengthy and poorly reproducible test. However, it is the only readily available test to as-

sess **durability** and in practice it has been used successfully to explain cases of poor durability in South Africa. It should therefore be used inter alia to estimate the durability of weathered basic crystalline rocks and other soils with poor quality aggregates that are stabilized for the purpose of cementation (see also 5.1).

4.5 TENSILE TESTS

The direct tensile test, the indirect tensile test and the flexural test are three tests that can be used to determine the tensile strength of cemented materials.

The indirect tensile test is a relatively simple test and is easy to use in practice. It is therefore preferred to the direct tensile test, which is more difficult. In the indirect tensile test a load is applied to the curved sides of a cylinder and failure is obtained by splitting the specimen along the axial plane. The indirect tensile strength (ITS) is about 1,5 times greater than the direct tensile strength. An approximate relationship between the ITS and UCS, based on some tests carried out in South Africa, is $ITS = 0,13 UCS$, but the correlation is not good and this relationship should not be used to estimate values.

The flexural test is sometimes preferred because it represents the condition of a cemented layer in the pavement when it is subjected to a wheel load. The test is carried out on a beam supported at the ends and loaded at a third point. The flexural strength of cemented materials is about one-third of the compressive strength for low-strength materials and about one-fifth of the compressive strength for high-strength materials.

As stated in 4.2, overseas UCS values are usually based on specimens with a height:diameter ratio of 2:1, which differs from the specimen used in Method A14 of TMH1¹. The relationship between the UCS test and other tests will change as test procedures change.

4.6 CHOICE OF STRENGTH TEST

- (a) Modified materials: the CBR is a useful test for estimating the strength of modified materials.
- (b) Cemented materials: the UCS test is easy to perform and it has been used extensively for mix design and quality control. Considerable experience has been gained with the UCS and criteria for strength in TRH14² are based on the UCS. However, the tensile strength is important since it represents loading conditions in the field more closely, and attention should be given to the use of tensile tests for design and control purposes. The flexural test is particularly suitable for structural pavement design and should be used in central laboratories to study the properties of cemented materials and for mix design. At present relatively little experience has been gained with tensile tests and in the interpretation of test results, and it is recommended that greater use be made of tensile tests in practice.

4.7 TEST PROCEDURES

The procedures for the relevant tests are given as references below.

Test	Reference
California Bearing Ratio	TMH1 ¹ - Method A9
Unconfined compressive strength	- Method A14
Unconfined compressive strength after 7 days' soaking	- Method A14
Wet/dry durability test	- Method A19
Accelerated curing	- Method A13T ^{18†}
Indirect tensile strength	- Method A16T
Initial consumption of lime	- Method A17T ^{19†} or Method CA34-83 (1984) ^{20*††}
Cement and lime content	- Methods 15(a), (b), (c) and (d) (or Atomic Absorption Spectrophotometer)**
Plasticity Index (usually on material removed from CBR or UCS specimens after test)	- Method A3 (Standard PI test)
Total salts	- Method A21T
pH	- Method CA21-74 (1980) ²¹ on the fraction passing 6,7 mm
Sulphates	BS 1377 ¹⁵ (Tests 9 and 10)
Detection of the presence in soils of organic matter capable of interfering with the hydration of Portland cement (measurement of pH of soil-cement paste)	BS 1924 ²² (Test 18)
Detection of sugar	SABS Method 833 ¹⁷
Pick and click test	Weinert: <i>The natural road construction materials of Southern Africa</i> ²³ , p131.
Percentage of secondary minerals	Weinert ²³ : p95, p131.

Note: Sampling should be in accordance with TMH5²⁴.

*Method A17T¹⁹ (ICL determination on soil types) is being modified as Method CA34-83 (1984) to allow for better testing of gravels. Note that previously published limits derived from minus 0,425 mm material may therefore have to be modified.

**These tests for the cement or lime content are unsuitable for materials containing large or variable amounts of calcium. Other test methods are available for such materials but there is no local experience with these methods.

†This is a tentative method and has not been included in TMH1. It is available from the NITRR.

††Available from the NITRR.

5 CRITERIA FOR STABILIZED MATERIALS

The object of stabilization is either cementation or modification, and although there is a degree of overlap between the two states, it is appropriate that they should be considered separately when dealing with the uses of stabilizers in practice.

5.1 CEMENTED MATERIALS

Criteria for cemented materials are given in TRH14². There are four classes of cemented materials, namely C1, C2, C3 and C4. The cemented materials consist of crushed stone, crushed gravel or natural gravel, and detailed recommendations for the properties of the material before and after treatment are given in Subsection 3.2 of TRH14². Typical design strengths given in TRH14² for the four classes are shown in Table 3. The UCS is given at two densities (97 and 100 per cent Mod. AASHTO). The strength at 100 per cent Mod. AASHTO is given since it is easy to compact samples to 100 per cent density, and the strength at 97 per cent, which can be determined from a strength/density relationship, is equivalent to the strength at the field density usually specified for stabilized bases.

TABLE 3
Design strength for cemented materials (MPa)

	C1 Cemented crushed stone or gravel		C2 Cemented crushed stone or gravel		C3 Cemented natural gravel		C4 Cemented natural gravel	
	min	max	min	max	min	max	min	max
Laboratory-design, UCS at 7 days, 100% Mod. AASHTO density	6	12	3	6	1,5	3,0	0,75	1,5
Laboratory-design, UCS at 7 days, 97% Mod. AASHTO density	4	8	2	4	1	2	0,5	1

Criteria for the ITS have not yet been determined, but investigations carried out in Natal suggest the following:

Cemented material

Minimum ITS (kPa) (Test Method A16T¹)

C3	200
C4	120

Criteria for the wet/dry durability test are not well established in South Africa and it is not possible to make definite recommendations at this stage. TRH14² (paragraph 3.2.1(j)) states that the percentage loss of material should not exceed 14 per cent after 12 cycles of wetting and drying, but this is considered to be a very strict requirement for general use. In a recent investigation in Natal, wet/dry tests were correlated with results obtained with the Heavy Vehicle Simulator on a wide range of cemented subbases. This particular investigation indicates that a maximum loss of 40 per cent by mass could be a suitable criterion for cemented subbases.

Consideration of the durability of the **aggregate** in weathered basic crystalline rocks and some other materials is also necessary. For testing the durability of the aggregate in weathered basic crystalline rocks the "pick and click" test or the percentage of secondary minerals test should be applied. The allowable secondary minerals may be increased by 5 per cent if a material is stabilized. The ratio of soaked to dry 10 per cent FACT (fines aggregate crushing test) can also be applied to most materials. For satisfactory durability the soaked 10 per cent FACT should be at least 75 per cent of the dry 10 per cent FACT, but this is a general guide; additional guidance is available 23,25.

The load-carrying capacity of pavements can be increased considerably by the use of cemented layers but the strength of the treated layers, their thickness and their position in the pavement should be such that load-associated cracking does not lead to premature distress. This aspect is discussed in TRH4³, which includes a Catalogue of designs for cemented layers. The designs show the positions and thicknesses recommended for cemented materials in various pavements depending on the traffic and road category. It should be noted that normally only C3 and C4 cemented materials feature in flexible pavements since higher strengths could be both unnecessary and undesirable. C1 and C2 materials are usually confined to special cases such as subbases under concrete pavements.

5.2 MODIFIED MATERIALS

Since cementation is not an essential requirement for modified materials, the criteria are different. TRH14² mentions a large range of materials that are referred to as "untreated layered materials". A summary of the strength and Atterberg limit requirements for these materials is given in Table 4.

The grading and other specifications are given in TRH14².

If the CBR and Atterberg limits of the untreated material do not comply with these specifications, the material may be modified with a stabilizing agent in order to comply with the specifications. The CBR of a modified material will often exceed the specified minimum by a considerable margin, and for satisfactory modification the CBR should exceed the minimum values given in Table 4 by at least 20 per cent.

TABLE 4
Atterberg limits and CBR for untreated materials

	Material designation	Liquid limit (max)	Plasticity Index (max)	CBR (min) (at % Mod. AASHTO)
Graded crushed stone	G1	-	NP*	-
	G2	25	6	80 at 98%
	G3	25	6	80 at 98%
Natural gravel	G4	25	6	80 at 98%
	G5	30	10	45 at 95%
	G6	-	12**	25 at 93%
Gravel-soil	G7	-	12**	15 at 93%
	G8	-	-	10 at in-situ density
	G9	-	-	7 at in-situ density
	G10	-	-	3 at in-situ density

*In some cases the rock fines may be plastic in which case a maximum liquid limit of 25 and a maximum PI of 4 are permissible.

**For coarse materials the PI may be increased as follows:

Maximum PI = 3 x grading modulus + 10.

The modification should preferably reduce the PI to below the maximum equivalent specified for untreated material. The following values could be used as a guide:

Maximum PI after modification

Base	4
Subbase	6
Selected subgrade	10

When weathered basic rock is used as base or subbase the PI should preferably be reduced to slightly plastic (SP).

A substandard material can thus be modified to comply with the specification, which is a useful process in practice owing to the limitations and variability of many natural materials.

6 MIX DESIGN

The object of the treatment should be defined, i.e. whether it is modification to improve the properties of the material, cementation to increase the structural capacity of the pavement, or a combination of the two. This will influence the selection of the type and amount of stabilizer, the test method and also the method of construction. Figure 14 is a flow diagram of the stages in the design of stabilized pavement layers.

6.1 ESSENTIAL ELEMENTS OF MIX DESIGN

Once the object of the treatment has been defined, the essential elements of the mix design procedure are as follows:

- The most suitable type of stabilizer is selected. An investigation of more than one type of stabilizer is advantageous.
- The necessary tests are done with no fewer than three different stabilizer contents. Increments will normally vary from 0,5 to 2,0 per cent and must provide sufficient information for a suitable strength-/stabilizer content relationship to be determined.
- The most suitable stabilizer is selected from laboratory test results with due regard to site conditions and costs.

A mix design flow diagram is given in Figure 15.

There are certain constraints that should influence the selection of the type and amount of stabilizer.

- (a) For cemented materials the UCS given in Table 3 should, according to TRH14² (paragraph 3.2.1(f)) be "obtained with not more than five per cent by mass of stabilizer at the specified density and at optimum moisture content"¹². Table 3 also gives maximum strengths. The object of these recommendations is to guard against the use of unnecessarily high stabilizer contents in cemented layers.
- (b) Where possible, material used in cemented layers should comply with certain requirements before treatment. TRH14² gives the following:

Material quality before treatment	Cemented material
G 2	C 1
G 2 or G 4	C 2
G 5 or G 6	C 3 and C 4

It is also stated in TRH14² that the grading moduli for base and sub-base materials should not be less than 1,75 and 1,5 respectively. (See Subsection 3.2 of TRH14².)

- (c) Materials with properties given under (b) above are not always economically available, and when the properties of the untreated material de-

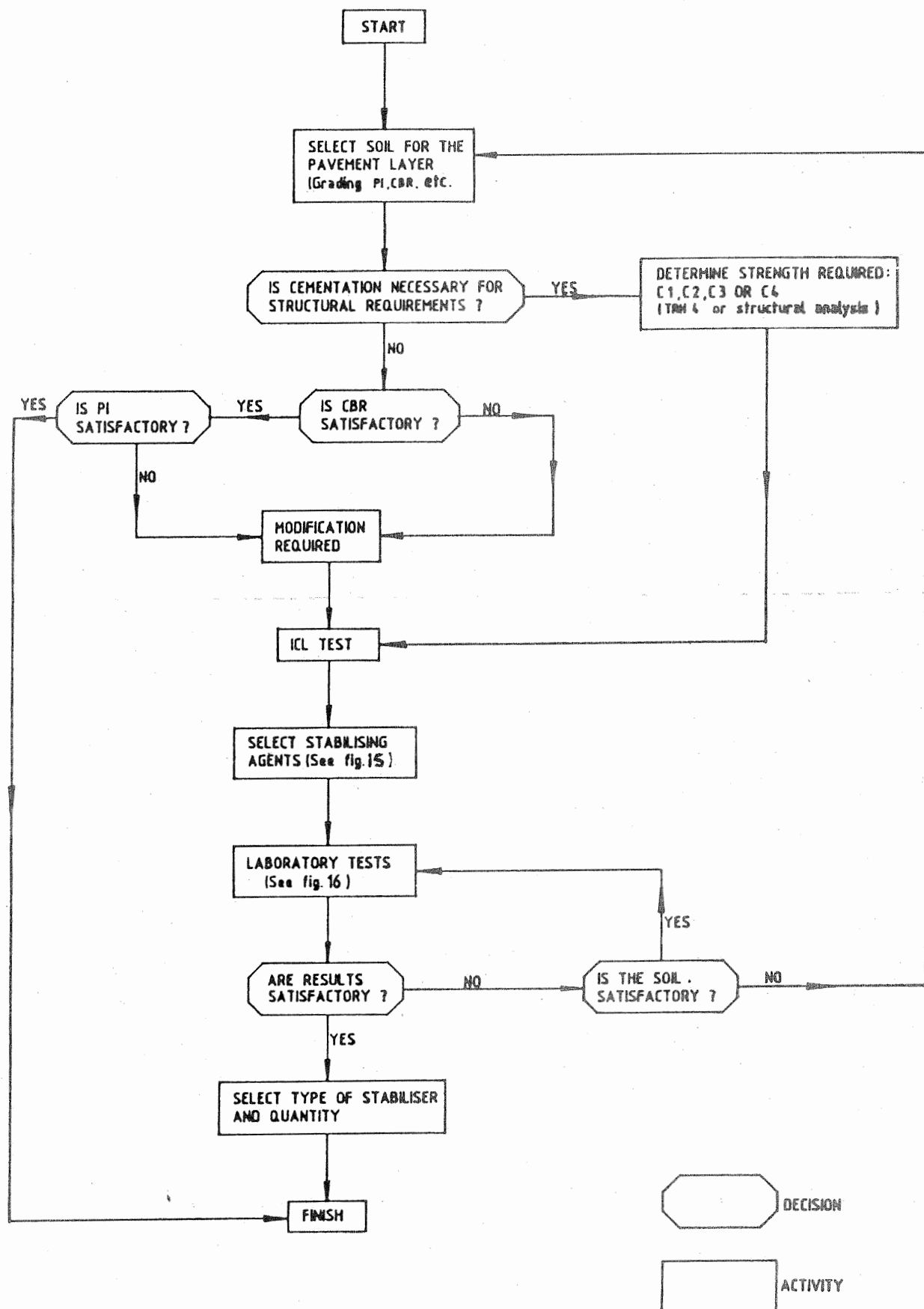


FIGURE 14

Stages in the design of stabilized pavement layers

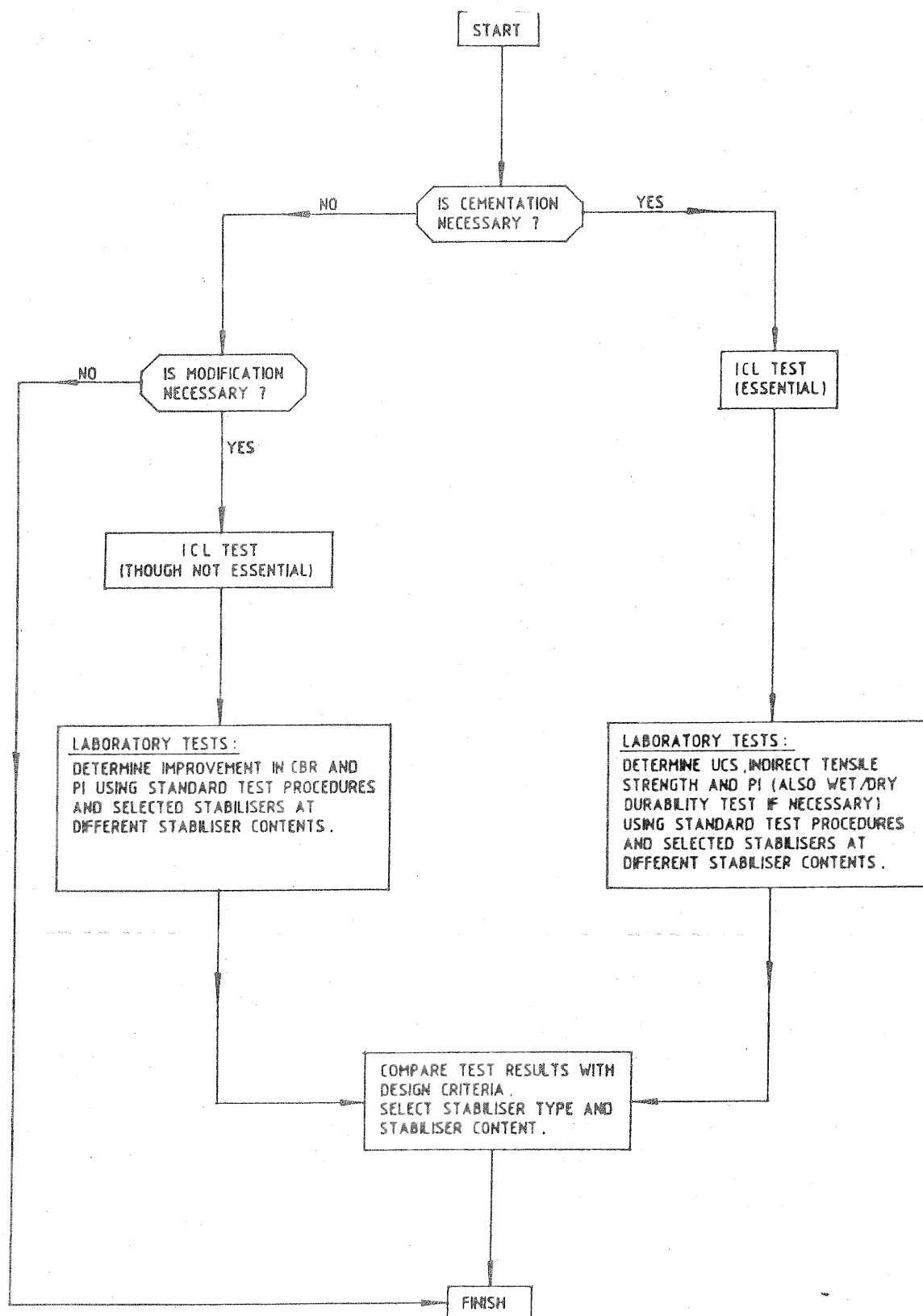


FIGURE 15

Essential elements of mix design

viate very far from the above criteria, the stabilization process becomes more critical, durability becomes more important and greater care should be exercised in judging results.

- (d) In all borrow pits, sufficient tests should be done to give an indication of the **variability** of the borrow pit material. One or two samples from the borrow pit should be tested by varying the type of stabilizer and also the stabilizer content. When the most suitable type of stabilizer and stabilizer content have been determined, further samples from the borrow pit should be selected and tested with the particular stabilizer and stabilizer content previously determined. The number of samples tested will depend on the size and variability of the borrow pit. The following rate of sampling is specified in the Design Manual of the Transvaal Roads Department²⁶:

Size of borrow pit (m ³)	Minimum number of samples
less than 20 000	5
20 000 - 40 000	6
40 000 - 80 000	8
more than 80 000	10

- (e) Materials such as basic crystalline rocks (basalts, dolerites, etc.), which are liable to weather in service, may require special consideration. Treatment to reduce the PI to SP may be desirable, and the secondary mineral content or stage of weathering should be considered in relation to the climate of the site. It is essential in such cases that the ICL be satisfied.
- (f) The ICL must always be satisfied in the case of cemented materials.
- (g) The coefficient of variation of the stabilizer content of random samples taken from the treated pavement layer will usually vary between 10 and 30 per cent. It should not be allowed to exceed 30 per cent, since this indicates poor mixing. A suitable design criterion is that not more than 10 per cent of the test results, as determined from random samples, should be below the specified stabilizer content. Stabilizer content variations should be taken into consideration when the amount of stabilizer to be added in the field is specified; for example, if the coefficient of variation is 30 per cent then the laboratory design stabilizer content should be increased by 1,4 to ensure that not more than 10 per cent of the values from random test results in the field are below the laboratory design value. In some circumstances the limiting factor is the minimum quantity of stabilizer that can be added in practice. This applies particularly to cement-treated pavement layers where a relatively low compressive strength is required and where the material reacts well

with cement. The cement content specified in the field may then be the minimum that can be mixed, which is 2 per cent.

A suitable criterion of both the quantity of stabilizer and the uniformity of mixing is that not more than 9 samples out of 50 should have a stabilizer content of less than 70 per cent of the specified stabilizer.

6.2 COMBINED STABILIZERS

If a plastic material must be used to provide a cemented layer, the strength specification for cemented materials may sometimes not be achieved with lime and the material may also be too plastic for satisfactory treatment with cement. The material could first be treated with lime to reduce its plasticity and then treated with cement to obtain the required compressive or tensile strength. Satisfactory results may also be obtained by the use of blends of MGBS and lime.

6.3 INNOVATIVE DESIGN

The recommendations summarized in the previous paragraphs are based on the criteria and the type of treatment most frequently used. A substantial improvement in the properties of almost any soil can be achieved, which is why treatment with cementitious stabilizers also lends itself to innovative design. This should always be borne in mind notwithstanding the guidelines given in the previous paragraphs, particularly in areas where good materials are costly or not available. Such treatment includes:

- (a) the cement stabilization of a large variety of sands, ranging from single-sized sands requiring high stabilizer contents to well graded sands, to provide durable base and subbase courses;
- (b) the stabilization of low-plasticity sands with lime, sometimes also with the addition of granitic soil;
- (c) the treatment of plastic materials with lime to provide good pavement layers including bases;
- (d) the provision of a cement-stabilized surface drain at an airport to prevent scour and erosion; and
- (e) the addition of fine sandy soil to a coarse gravel before stabilizing.

7 CONSTRUCTION

7.1 STORAGE OF STABILIZING AGENTS

Stabilizing agents deteriorate when they are in contact with air, the rate of deterioration depending on the amount of moisture in the air. Precautions should be taken to minimize contact with moisture during storage. It has been shown that if cement is stored in pockets there may be a 20 per cent loss in strength after three months even if the storage conditions are good. Lime does not deteriorate as fast as cement and can be stored for longer periods. However, calcium hydroxide changes to calcium carbonate and thus the "available lime" is steadily reduced during storage. The storage period should therefore be short.

When storage periods have exceeded two months in the case of cement or six months in the case of lime, it is advisable to check the efficacy of the stabilizer before it is used. If there has been a significant decrease in the efficacy then a corresponding increase in the stabilizer may be necessary.

7.2 CONSTRUCTION OPERATIONS

The main stages in stabilization construction are:

- (a) mixing
 - Mix-in-place, incorporating:
preparation of layer, application of stabilizer either mechanically or by hand and mixing and watering
 - or
 - central plant mixing;
- (b) compaction; and
- (c) curing

7.2.1 Mixing

7.2.1.1 *Mix-in-place*

- (a) Preparation of layer

Water should be added to the layer before the addition of the stabilizer for the following reasons:

- it facilitates the breaking down of the lumps;
- segregation of the stabilizer is reduced, which is of importance when coarse materials are treated;
- if the stabilizer is mixed with dry soil and the material is then watered, the stabilizer tends to absorb the water preferentially and this results in small lumps rich in stabilizer that may be difficult to break down; and

- it speeds up the compaction process, particularly if the moisture content is at or close to the OMC when the stabilizer is added.

The layer should be lightly rolled with at least one pass of a flatwheel roller to provide a firm surface for the spreading of the stabilizer.

The maximum and minimum layer thicknesses should be the same as those normally used in the construction of untreated pavement layers and the thickness of the layer should not be reduced to below the normal minimum in an attempt to achieve the density specification.

(b) Application of stabilizer

It is essential that the stabilizer be spread carefully and uniformly over the area to be treated since the uniformity obtained during the spreading operation will affect the amount of mixing necessary and possibly also the standard of mixing finally achieved. The minimum of cement that can be mixed efficiently is about 2,0 per cent by mass and the minimum of lime is about 1,0 per cent by mass. Hand spreading from pockets or mechanical spreading may be employed.

(i) Hand spreading from pockets

The pockets should be placed in three or four rows along the section to be processed so that the distance between the rows and the distance between the pockets in the rows are more or less equal. The pockets should be opened and the stabilizer spread uniformly in transverse rows. The transverse rows should then be spread with rakes or squeegees so that a uniform application over the whole area is obtained.

Efficient hand spreading provides better distribution than, and is therefore preferable to, spreading by grader. However, spreading by grader may be permitted provided the distribution is satisfactory.

(ii) Mechanical spreading

This is carried out by means of a mechanical spreader attached to a dump truck. As the truck moves forward the stabilizing agent flows through the spreader, which regulates the quantity of stabilizer placed on the prepared soil. To obtain a uniform spread, the spreader should be operated at a constant slow speed with a constant level of stabilizing agent in the hopper. A true line at the pavement edge should be maintained with a string line.

The mechanical spreader can also be attached directly to a bulk supply truck, in which case the stabilizing agent is moved pneumatically from the truck and falls into the hopper of the spreader. Forward speed must be low and even.

Mechanical spreading is more efficient and less labour-intensive than hand spreading.

(c) **Mixing and watering**

Mixing of the previously dampened material should commence as soon as the stabilizer has been spread. Effective mixing can be achieved by scarifying the material first by means of a grader fitted with mixing discs or blades, then by off-set disc-harrows or ploughs with specially designed mould boards. The best mixing action with a disc-harrow is obtained when it is towed at a speed of 8 to 10 km/h. Rotary mixers are especially effective if the stabilizer is uniformly and correctly spread and there is not too much coarse material, which tends to break the rotor teeth.

Watering should commence after the material has been fairly well mixed. There should be several uniform applications of water while the material is being turned over by grader and mixed with the disc-harrow. When the material is turned over by grader it should be ensured that the entire layer is turned over to its full depth, otherwise the stabilizer may be concentrated in the upper portion of the layer. The material should be mixed after each application of water to prevent the accumulation of an excessive amount of water on the surface, and cement-treated materials should not be allowed to remain undisturbed for more than 30 minutes.

7.2.1.2 Central plant mixing

Central plant mixing can be used as an alternative to mix-in-place. Either continuous-flow or batch pugmill mixers or rotary drum mixers have been used for this work. Facilities for efficiently storing, handling and proportioning materials must be provided at the plant. Mixing is continuous until a uniform mixture of soil, stabilizing agent and water is obtained. The basic essentials of thorough mixing, adequate stabilizer content and correct moisture content apply.

Central plant mixing provides better control of water and stabilizer content as well as more efficient mixing, but it may be more expensive owing to additional overhaul.

To reduce losses due to evaporation under hot, windy conditions and to protect against sudden showers, haul trucks should be equipped with protective covers. To prevent excessive haul time, not more than one hour should elapse between the start of the moist mixing of cement-treated materials and the start of compaction. The haul time of cement-treated materials is usually limited to 30 minutes.

7.2.2 Compaction

When visual inspection shows that mixing is satisfactory and that the correct moisture content has been reached, compaction should commence without delay.

(a) Density

The following values are usually specified for stabilized layers:

	% Mod. AASHTO (min)
Base (upper and lower)	97
Subbase (upper and lower)	95
Selected layers	90 - 93

Note: Stabilized crushed stone should be compacted to 97 per cent Mod. AASHTO.

(b) Processing time

Mixing and compaction should be efficient to minimize the decrease in density and strength due to the early reactions between stabilizer, water and soil. The operations should be completed within the following time limits:

Stabilizing agent	Maximum time (in hours) for completion of compaction and finishing after stabilizing agent comes into contact with soil
OPC, PBFC, OPC-MGBS,	
MGBS-lime	6
Lime (cementation)	8
Lime (modification)	48

In practice it is possible to complete the compaction in four to six hours, particularly if the material has been brought to OMC on the previous day.

Some materials, such as certain calcretes, sandstones, granites and red sands, may react so rapidly with lime that they should be treated like cement.

Some materials, particularly those with high PIs, may have to be left uncompacted to cure for several days if adequate density and freedom from excessive cracking are to be achieved. This period of curing may have to be determined by laboratory or field tests.

(c) Finishing

Some stabilized materials are difficult to compact. Stabilization tends to make some materials cohesionless, and this may result in the material sliding under the rollers, causing shear planes. Lighter rollers should be used in conjunction with a slightly lower compaction moisture content, and in such cases it is essential that finishing be done by shaving the surface and not by blading a thin layer back. If compaction planes have formed they should be removed by lightly scarifying the surface and re-compacting.

The surface should be kept damp during finishing operations. Slushing should not be allowed, but the surface may be finished by a light application of water followed by pneumatic rolling. Compaction and finishing (excluding spreading and mixing) should be completed in two hours. Additional recommendations for finishing and other aspects of construction are available ²⁷.

(d) Construction joints

After each day's construction, and when construction operations are delayed or stopped for more than two hours, a transverse construction joint should be formed by cutting back into the completed layer. This may be done last thing at night, or first thing the next morning, using the toe of a motor grader blade, for example. The joint should be vertical and perpendicular to the centre line.

In mixed-in-place construction, when construction is resumed, the transverse edge of the previous day's mixing must be cleared of all dry and unmixed material and retrimmed if necessary. Mixed material is then bladed into the area and compacted thoroughly. The joint is left slightly high until final rolling when it is trimmed with a motor grader and rerolled.

When a cemented layer has to be constructed adjacent to a hardened cemented layer, the joint face must be prepared at the end of the day's construction of the completed lane or the following morning by cutting to a string line with the toe of a motor grader blade. Some hand trimming may be needed. A disc mounted on the end of a blade makes a good edge cutter. With central plant mixers, this method of joint construction is used for all joints.

Regardless of the method of construction, it is essential that all joints be cut back to solid cemented material. The material next to the joint should be properly pulverized, mixed with stabilizer, moistened, compacted and finished flush with the adjoining section, care being taken to maintain the proper grade.

(e) Acceptance or rejection of a stabilized layer on the basis of density

If the specified density is not achieved the layer may be rejected and must then be ripped up after the initial reaction between soil and stabilizer. Additional stabilizing agent may then have to be added depending on the nature of the material and the object of the treatment, i.e. cementation or modification. If the object of the treatment was modification, additional stabilizer will probably not be necessary. However, if the object was to provide a cemented layer, additional stabilizer will have to be added before the material is recompacted. Satisfactory results have been obtained by adding about one-half of the quantity added originally, but the quantity of additional stabilizer required will

depend on factors such as the type of stabilizer, the interval between the first compaction and the second compaction and the strength required. If lime has been used to provide a cemented layer and the layer is ripped up, cement should be used for the second stabilization, unless it is known from laboratory testing that further lime will achieve the desired strength. Ripping up and retreatment will tend to reduce the Mod. AASHTO density even further and may lead to additional compaction problems. A new Mod. AASHTO density must be determined. If the compaction fails again it is possible that the material may be completely rejected since repeated reprocessing and density reduction may result in a weaker cemented layer.

The importance of correct and efficient mixing and compaction and accurate testing procedures cannot be over-emphasized.

7.2.3 Curing

After the treated layer has been finished it should be protected against drying for seven days and traffic should not be allowed on the layer during this period. The material near the surface can dry rapidly and it is important that even the top few millimetres be properly cured especially under thin surfacings. One of the following curing methods should be used:

- (a) Frequent light water spraying to ensure that the layer and even the surface remain **continuously** damp; a full, heavy water bowser may damage the layer and light or half-full bowzers or side spraying should be used. A layer of sand about 40 mm thick will improve the moisture retention. The surface of the layer must not be subjected to wetting and drying cycles since this has a detrimental effect on the treated layer.
- (b) Watering and covering with plastic sheeting; this is an effective method, but may be expensive, and can be difficult under windy conditions.
- (c) Application of a curing membrane consisting of spray-grade emulsion.
- (d) Covering the treated material while it is still damp with material that may be required for the next layer; this must be carried out in such a way that the treated layer is not damaged.

If a prime such as MC 30 or 3/12 tar is used and good penetration is obtained, it should not be applied before the material has been cured in accordance with (a) or (b). If little penetration takes place the prime can be applied sooner. However, tar prime should not be applied sooner than seven days after compaction. Primes are ineffective as curing membranes once they have dried and do not provide long term protection against carbonation or salt damage. In order to prevent such damage it is good practice to apply the surfacing as soon as reasonably possible after the prime or emulsion has dried sufficiently to permit this.

7.3 QUALITY ASSURANCE

A check-list is given below.

7.3.1 Process control

	Check	Notes
(a) Materials		
Quality of soil	Untreated soil to comply with requirements	See 5.2, 6.1(b) and (c). Stabilizer quantity based on correct layer thickness and width.
Quality of stabilizer	Compliance with relevant SABS specification	
Layer	Thickness and uniformity	
(b) Preparation	Material should be thoroughly pulverized (if cohesive), dampened and lightly rolled	
(c) Application of stabilizer		
By hand	Spacing of bags	Indication of quantity as well as uniformity of application. Ensure that stabilizer is not wasted and that mixing is confined to the prescribed dimensions both vertically and horizontally.
Mechanical	Determine mass on 1 m ² canvas sheets placed every 200 m and/or determine mass of tanker before and after application	
(d) Mixing and watering	Inspection holes at no fewer than 6 points per construction length to assess uniformity of mixing of stabilizer and water and to ensure mixing is to the full depth	Visual inspection. Sides of the hole from top to bottom should turn red when sprayed with phenolphthalein. Note: phenolphthalein shows the presence of cement or lime but does not indicate the amount.

- | | | |
|-----------------------|--|--|
| (e) Compaction | Prescribed maximum period for mixing and compaction
Compaction planes | |
| (f) Curing | Material to be continuously damp for 7 days | With lime-stabilized materials excessive carbonation will not take place if curing is efficient. The material should turn red if sprayed with phenolphthalein. |

7.3.2 Acceptance control

	Check	Notes
(a) Treated material	PI, CBR or UCS (or ITS)	Strength tests: The strength obtained from samples taken from the road will not necessarily be the same as that obtained in the laboratory owing to differences in mixing, compaction and curing techniques. However, tests on samples taken from the road after the stabilizer has been mixed will give an indication of the strength of the treated material and provide a means of checking the consistency of the strength obtained in the field; they may also detect a change in the quality of the material.
	Compacted density	The reduction in density due to the time lapse during compaction should be correctly applied.

(b) Stabilizer distribution	Efficiency of mixing Co-efficient of variations to be less than 30%	Random samples should be taken from the completed work in accordance with TMH524, and the stabilizer content should be determined in accordance with one of the A15 test methods given in TMH11, by means of the Atomic Absorption Spectrophotometer or by other approved methods. Between 20 and 50 samples per construction length should be tested.
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7.4 THE CONTROL OF CRACKING IN PAVEMENTS WITH CEMENTED LAYERS

Although cracking must be accepted as an unavoidable feature of cemented layers, the cracks should nevertheless be controlled so that they do not have an adverse effect on the performance of the pavement. Water penetrates into large cracks, which reflect through the bituminous surfacing; pumping may occur and the maintenance of cracked pavements can be difficult and costly. Premature maintenance may be necessitated by both shrinkage cracking and traffic-associated cracking.

Traffic-associated cracking will not be discussed in this document apart from the comments made in 3.1.6.2, since uncontrolled traffic-associated cracking, which results in premature maintenance, should be avoided by carrying out a structural analysis or by following the recommendations given in TRH4³.

Not all shrinkage cracking is necessarily detrimental and cracks of less than about 1 to 2 mm do not normally require special maintenance treatment. However, shrinkage cracking should be controlled during the design and construction stages to ensure that wide cracks, which require maintenance, do not reflect through the bituminous surfacing.

7.4.1 Control of shrinkage cracking during the design stage

(a) Design strength

Materials with a high stabilizer content are more susceptible to cracking than materials with a low stabilizer content. The design strength should therefore be as low as possible but still be consistent with the structural and durability requirements of the pavement. It is often possible to achieve adequate strength with a low stabilizer content.

(b) Position of treated layer in pavement

Cracks that reflect through the bituminous surfacing can usually be prevented if the cemented material is confined to the subbase layers and the base consists of untreated material. The initial cracks in the subbase do not usually reflect through an untreated base (G1, G2 or G3) of 150 mm or more. This is probably the most effective way of preventing shrinkage cracks from reflecting through the surfacing. However, instances have occurred where cracking in the subbase has reflected through 200 mm of an untreated base and the bituminous surfacing. On lightly trafficked pavements natural gravel bases with a low plasticity can be treated with cement to provide a better quality material. This results in fine narrow shrinkage cracks that reflect through the surfacing to a varying degree and that are acceptable. In such cases it should be ensured that the base is modified and not too strongly cemented since a cemented base that is on an untreated subbase and that has a very high strength may be susceptible to traffic-associated cracking, i.e. it may not be a correctly "balanced" pavement.

(c) Material properties

Some materials shrink more than others. Chert gravel is an example of a material that shrinks considerably when treated with cement. Many lime-stabilized calcretes and sandstones exhibit shrinkage cracking.

7.4.2 Control of shrinkage cracking during the construction stage

7.4.2.1 Stabilizer content

The stabilizer content should not be increased excessively during construction simply to ensure that strength requirements are met. Spreading and mixing, and also compaction and curing, should be of a high standard so that a uniform mix is obtained both vertically and horizontally and so that the specified strength can be achieved with the lowest possible stabilizer content.

7.4.2.2 Compaction moisture content

The degree of cracking is proportional to the amount of moisture lost on drying and thus the wetter the material on compaction, the greater the degree of cracking. The compaction moisture content should not be unnecessarily high.

7.4.2.3 Delay between mixing and compaction

Cracking in plastic materials stabilized with lime can sometimes be reduced by mixing the lime with moist material and then delaying the compaction until tests show that the plasticity has been reduced to acceptable limits. This delay should also be incorporated in test procedures for determining density and strength.

7.4.2.4 Delay of surfacing

If it is convenient to delay the construction of the bituminous surfacing until some or most of the shrinkage cracking has taken place, then such a delay should be considered since the tendency of cracks to reflect through the surfacing may be reduced. The layer must, however, be cured. This delay is not recommended for lime-stabilized materials owing to the likely increased risk of carbonation.

7.4.2.5 Settlement

If excessive settlement takes place in the road beds or in fills, cracking of cemented layers can be aggravated. Road beds and fills should be compacted efficiently so that subsequent settlement and cracking are reduced to a minimum. Although this is a normal construction requirement, settlement under a pavement with cemented layers may result in more serious cracking.

8 COSTS

8.1 COMPARISON OF DIFFERENT DESIGNS

A procedure for the cost analysis of different pavement designs is given Section 8 of TRH4³. This analysis can be used as a guide to the selection of the most economical pavement structure within the same road category. The Catalogue of pavement designs in TRH4³ gives examples of alternative designs with approximately the same structural capacity. It will be noted that cemented and/or modified materials feature in almost every catalogue design. By using unit costs for available materials on a particular project the cost of different pavement types (e.g. granular base with cemented subbase, bituminous base, etc.) can be compared.

Cost is an important consideration but it should not necessarily override all other considerations.

8.2 UPGRADING OF SUBSTANDARD MATERIALS

If a readily available material does not comply with the specification but can be stabilized to comply with the requirements, the total cost of the stabilized material should be compared with the cost of a material that may not be as readily available, but either requires no treatment at all or can be treated more economically. As in the case of different pavement designs, the cost analysis should be an aid to decision making. Preference should be given to better quality materials even if they cost slightly more than stabilized substandard materials.

8.3 TYPE OF STABILIZER AND STABILIZER CONTENT

A particular material should be tested with more than one type of stabilizer to determine which stabilizer not only gives the best reaction but is also the most economical. Availability of the stabilizer at the time of construction is also important since delays in delivery will increase construction costs.

APPENDIX A

ESTIMATE OF ELASTIC MODULI FROM UCS AND FLEXURAL STRENGTH

Equations (1), (2) and (3) may be used to estimate the elastic moduli of three types of material if the flexural strength is known, and equations (4), (5) and (6) may be used if the unconfined compressive strength is known²⁸. Equations (1), (2) and (3) are recommended, since the use of the flexural strength is preferred.

$$\text{Cement-treated crushed stone:} \quad E = 8 \sigma_b + 3\,500 \quad (1)$$

$$\text{Cement-treated natural gravel:} \quad E = 10 \sigma_b + 1\,000 \quad (2)$$

$$\text{Lime-treated natural gravel:} \quad E = 17 \sigma_b - 900 \quad (3)$$

$$\text{Cement-treated crushed stone:} \quad E = 4,16 (\sigma_c)^{0,88} + 3\,484 \quad (4)$$

$$\text{Cement-treated natural gravel:} \quad E = 5,13 (\sigma_c)^{0,88} + 1\,098 \quad (5)$$

$$\text{Lime-treated natural gravel:} \quad E = 8,56 (\sigma_c)^{0,88} - 927 \quad (6)$$

The strain at break of the cemented materials can be measured, but equations (7), (8) and (9) may be used to obtain approximations.

$$\text{Cement-treated crushed stone:} \quad \varepsilon_b = 145 \quad (7)$$

$$\text{Cement-treated natural gravel:} \quad \varepsilon_b = 76 + 7\,160 E^{-0,567} \quad (8)$$

$$\text{Lime-treated natural gravel:} \quad \varepsilon_b = 89,9 + \frac{182\,870}{E} \quad (9)$$

where

E	=	Elastic modulus (MPa)
σ_b	=	Flexural strength (kPa)
σ_c	=	Unconfined compressive strength (kPa)
ε_b	=	Strain at break ($\mu\varepsilon$)

Figures 16 and 17 show that the stresses, and particularly the strains, induced in cemented materials in a typical pavement structure are only marginally affected by relatively large changes in the Poisson's ratio of the cemented material. Poisson's ratios from 0,1 to 0,50 have been reported and 0,35 is normally used in mechanistic design.

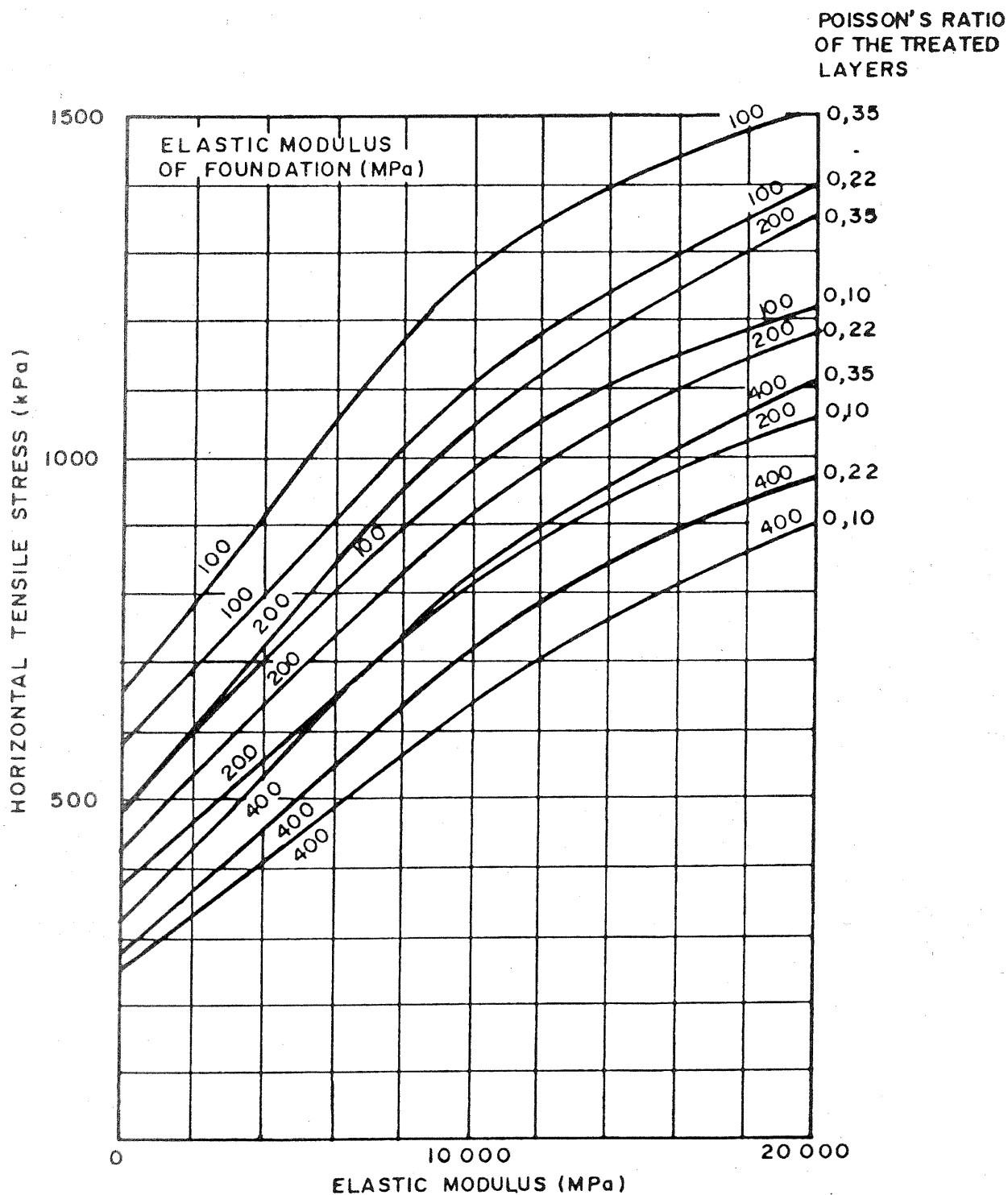


FIGURE 16

Influence of Poisson's ratio on induced stress

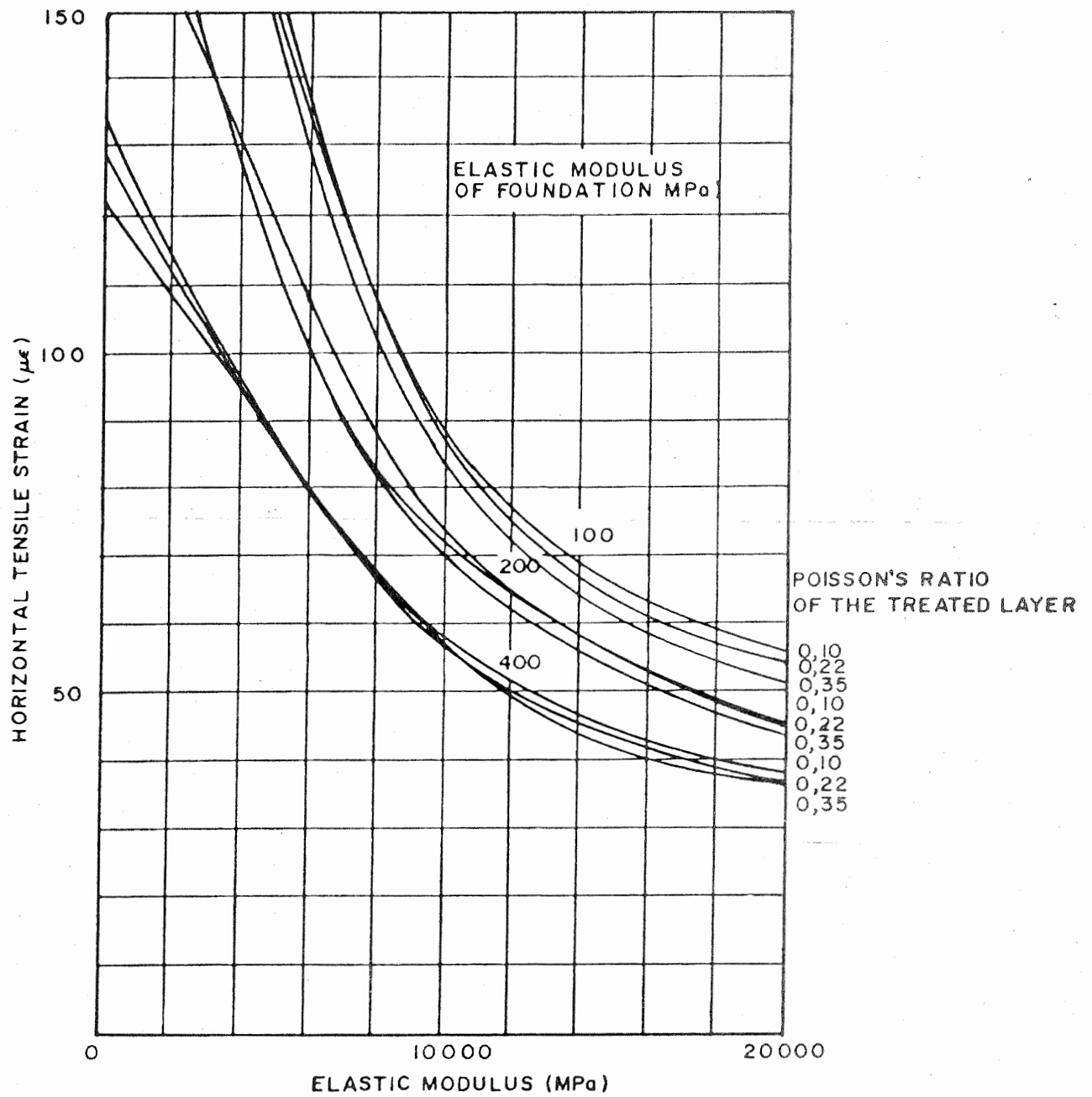


FIGURE 17

Influence of Poisson's ratio on induced strain

THE EFFECT OF LIME TREATMENT ON THE ATTERBERG LIMITS, CBR AND ACCELERATED CURING UCS OF SOME SOUTH AFRICAN SOILS

Cementitious stabilizers
TRH13, Pretoria, South Africa, 1986

DESCRIPTIONS:

WD = Weathered dolerite
WB = Weathered basalt
WG = Weathered granite
WF = Weathered felsite
SS = Sandstone
GS = Grey sandstone
SH = Shale
FC = Ferricrete

NOTES: 1. CBR and Atterberg limits after stabilization were determined according to Method A9 of TMH¹, i.e. after 7 days of curing and 4 days of soaking.

2. Accelerated curing (UCS) was done according to tentative Method A13T¹⁸ of TMH¹.

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