CEMENTITIOUS STABILIZATION OF ROAD MATERIALS

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PREFACE

This document has been prepared by Prof P Paige-Green of Paige-Green Consulting (retired from CSIR Built Environment) to provide a state-of-the-art reference on chemical stabilization in South Africa. It has been based primarily on the information in the existing Draft TRH 13 published in 1986 and the more recent Gautrans stabilization manual produced by the CSIR in 2004 and includes other more recent developments in the field of chemical stabilization. The latter document attempted to incorporate information related to the re-classification of cements in South Africa, mainly from SABS 471 and SABS 626 to the SANS 50197-1 standard.

This document also summarises relevant recent local and international research to provide a manual to assist users with conventional soil stabilization and highlight problems that have appeared in recent times and means of overcoming them.

A separate document has been prepared simultaneously covering the use of non-traditional (or alternative) soil stabilizers.

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Definitions/Glossary

Agglomerate - To form into a mass.

Aggregation - The collection of fine units (clay-sized minerals) into larger single units (silt-sized or larger particles).

Biscuit layer – Colloquial term for lamination

Calcium lime - Lime in which the ratio of calcium oxide to magnesium oxide is 14.0 or higher.

Calcium silicate and aluminate hydrates - The cementitious substances produced by the hydration of cement and other cementitious materials or blends. Initially a gel is formed, then needle-like crystals form from the gel, interlocking into a matrix to give the rigid structure observed in stabilized layers.

Cement – Cement is a hydraulic binder , ie, a finely ground inorganic material which, when mixed with water, forms a paste which sets and hardens by means of hydration reactions and processes and which, after hardening, retains its strength and stability even under water.

Cementation – Cementation is the bonding of individual particles of soil by secondarily developed materials resulting in an increased compressive and/or tensile strength.

Colloidal gel - A mixture of one substance suspended in a different gelatinous medium.

Deliquescent - The property of a material to dissolve itself in water absorbed from the surroundings (air).

Dolomitic lime – Lime in which the ratio of calcium oxide to magnesium oxide is between 1.3 and 2.0.

Flocculation - The combination of dispersed units (clay minerals) due to the breakdown of repulsive forces between the units - similar to the curdling of milk on addition of vinegar.

Flyash – Flyash is obtained by electrostatic or mechanical precipitation of dust-like particles from the flue gases of furnaces fired with pulverised coal.

Grading coefficient (G_c) – a particle size parameter defined by the product of the gravel fraction (percentage between 2.0 and 26.5 mm) and the percentage passing the 4.75 mm screen ((P26.5 - P2) x P4.75/100). The value should be determined after normalizing the grading to 100 per cent passing the 37.5 mm screen.

Gravel – Particles of soil and rock passing a sieve with 63 mm square openings and retained on a 2 mm sieve.

Granulated blastfurnace slag – Granulated blastfurnace slag is made by rapid cooling of a slag melt of suitable composition, as obtained by smelting iron in a blastfurnace and contains at least two thirds by mass of glassy slag and possesses hydraulic properties when suitable activated.

Ground granulated blastfurnace slag (GGBS) – Ground granulated blastfurnace slag is produced by milling granulated blastfurnace slag to a fine powder.

Hydrated lime - Unhydrated lime (calcium oxide) treated with water and/or steam to convert from the oxide to hydroxide form (calcium hydroxide). Also known as slaked lime.

Hydraulic cements and materials – These are binders, ie, finely ground inorganic material, which when mixed with water, form a paste that sets and hardens by means of hydration reactions and processes which, after hardening retain their strength and stability, even under water.

Hygroscopic - Tending to absorb water from the surroundings without necessarily dissolving, eg, an increase in the number of molecules of waters of crystallisation.

Indirect Tensile Strength (ITS) - A measure (usually in kilopascals) of the tensile strength of a stabilized material when a cylindrical specimen is loaded to rupture on its diametrical axis.

Indirect Tensile Test (ITT) - A laboratory test to determine the ITS of a stabilized material.

Initial Consumption of Cement (ICC) - The minimum quantity of cement which, when added to a suspension of soil in water, raises its pH to a constant value after one hour.

Initial Consumption of Lime (ICL) - The minimum quantity of lime which, when added to a suspension of soil in water, raises its pH value to 12.4 at normal temperature (20 - 22°C) after one hour. Refer to Appendix D.

Initial consumption of stabilizer (ICS) – The minimum quantity of chemical stabilizer which, when added to a suspension of soil in water, raises its pH to a constant value after one hour, depending on the stabilizer. This test is probably more appropriate than the ICL when stabilizers other than lime are to be used.

Ion exchange - The replacement of cations and anions by other cations or anions, which tend to be more physically and chemically stable when attached to clays and less strongly charged.

Lamination - A thin layer of weaker material, usually at the top of a compacted layer, resulting from various construction or curing activities.

Lime - Calcined limestone. Calcium carbonate treated by heating to remove carbon dioxide, forming calcium, and after hydration, calcium hydroxide. Lime used for road construction should not be confused with agricultural lime, which is dominantly calcium carbonate.

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 and to increase the California Bearing Ratio (CBR).

Pozzolan - A siliceous or siliceous and aluminous material which in itself possesses little or no cementitious properties, but when finely ground react with calcium hydroxide in the presence of water to form compounds possessing cementitious properties.

Quicklime - Refer to unhydrated lime (calcium oxide).

Saturation moisture content - That content at which all inter-particle voids within the material are filled with water. No allowance is made in this definition for entrapped air occurring in the water or disconnected voids that cannot be reached by the water.

Slake - Combining with water or water vapour to cause partial disintegration (e.g., of lime or a rock).

Slaked lime - Refer to hydrated lime (calcium hydroxide).

Stabilization – Chemical or mechanical treatment of a mass of soil to improve its engineering properties.

Treatment - Any material that is changed by the application of some action, (e.g., stabilization), has been treated.

Unhydrated lime - Calcined limestone consisting essentially of calcium oxide or a combination of calcium oxide and magnesium oxide. Also known as Quicklime or unslaked lime.

Unslaked lime - Refer to unhydrated lime.

Abbreviations

The following abbreviations are used frequently in the text.

CBR - California Bearing Ratio
C&CI - Cement and Concrete Institute
FA - Flyash
GGBS - Ground granulated blastfurnace slag
ICL - Initial Consumption of Lime
ICS - Initial Consumption of Stabilizer
ITS - Indirect tensile strength
MDD - Maximum dry density
OMC - Optimum moisture content
OPC - Ordinary Portland Cement
PI - Plasticity Index
UCS - Unconfined compressive strength

1. INTRODUCTION

The use of mixtures of lime and clay in building dates back to the beginning of man's recorded history where they were used in the construction of buildings, bridges and dams. The Romans used lime treatment in constructing their famous roads, many of which still exist today (Figure 1).

Figure 1: 2000 year old Roman road in Jordan

In the USA lime treatment of natural materials for road construction was first studied in Texas in about 1930 but difficulties delayed its general implementation until about 1952. Since then many kilometres of lime treated roads have been built in the USA.

The first recorded studies of the treatment of soil with cement were carried out in the USA in about 1917 although it is probable that that work was done using this combination prior to these reports. It was only in 1935 that the South Carolina Highway Department demonstrated that soil and cement were compatible and could be used together as a pavement base.

The first cement manufactured in South Africa was produced in 1892 in the "Eerste Cement Fabrieken" at Daspoort, Pretoria. The first chemical stabilization trials were carried out in the Transvaal in 1941 when 5% of cement was mixed into a soil layer on the road between Pretoria and Delmas (P36/1) using the ripper teeth of graders and light ploughs. A year later, cement stabilization was also carried out on the national route between Standerton and Volksrust with more effective mixing methods. Since then stabilization has increased to such an extent that on virtually every sealed road, one or more of the pavement layers is now stabilized.

Naturally occurring and crushed materials can have certain inherent limitations, which can have far-reaching effects on their performance in 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 and stiffness of the material are 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.

Chemical stabilization can be used to:

- improve the bearing capacity and strength of pavement layers and temporary bypasses, especially during rainy periods to limit construction delays due to rain;
- delay certain chemical reactions such as the weathering of sulphides and other minerals that are detrimental to road soils or aggregates;
- dry out soil where the moisture content is too high for successful compaction;
- make soil less permeable where this is necessary;
- reduce the plasticity of soils used in road construction and thereby to reduce the effect of moisture variations;
- improve the compactability of clays by changing the clay to a more granular and workable material;
- reduce the swelling and shrinkage of clays; and
- neutralise the sulphuric acid and to reduce the solubility of the highly soluble sulphate salts in gold and certain other mine waste rock crushed stone base materials.

Chemical stabilization is defined as any treatment or method whereby a chemical is used to either change the soil properties and thereby increase the bearing capacity of the soil layer (modification) or increase the strength and stiffness of the layer (cementation).

The chemical stabilization of pavement materials is a relatively straightforward operation and with sound construction techniques the properties of the materials can be significantly improved using a relatively small proportion of stabilizing agent. The treatment is usually economical in relation to the benefits that can be obtained, especially when locally available materials are of insufficient or marginal quality and large quantities of suitable material may have to be hauled over long distances.

Stabilized materials have become an important part of pavement design and construction in South Africa. With the need to reduce environmental impacts of pavement construction and rehabilitation and increase material sustainability, the re-use of existing materials in pavements after stabilization is an almost ubiquitous rehabilitation practice. Over the years, test procedures, design methods and construction techniques have been developed and improved to take into account the behaviour of stabilized materials. These are discussed in this document but are being improved on an ongoing basis.

The document has been prepared to provide the current state-of-the-art of cementitious stabilization in South Africa. It covers the chemical stabilization of materials using cement and lime. Although the contents of this diocument are equally applicable to the use of ground granulated blast-furnace slag (GGBS) and pulverised fuel ash (PFA), the practice of blending them on site has been almost eliminated as a result of the wide range of extended cements (already blended with these extenders) commercially available for construction, classified by SANS 50197-1. These chemical stabilizers are marketed as a milled powder and are available in either bulk form or in paper bags.

The manual does not include stabilization with bituminous products, which are seeing increased application and are covered fully in Manual TG2 (Asphalt Academy, 2009).

Various proprietary products are promoted as soil stabilizers, many of which were originally developed as dust palliatives. Few of these have been fully researched and these are not discussed in this document but are covered in a separate manual (Paige-Green, 2015).

This document is based partly on the Gautrans Stabilization Manual (GDPTR&W, 2004), which was an update of the older Transvaal Provincial Roads Department Manual L3/84 (TPA, 1984) and various concepts and examples are extracted from this.

2. CEMENTITIOUS STABILIZERS

2.1. Introduction

This chapter discusses the types of cementitious stabilizers commonly in use, their manufacture, composition and properties and relevant specifications. It also provides guidelines on storage and handling, including some safety measures.

2.2. Types

2.2.1 Cement

The specification of cement in South Africa was reviewed in 1996 and the SABS 471, SABS 626, SABS 831 and SABS 1466 specifications in force at the time were replaced. Since then, cements in South Africa have been produced in accordance with the SANS 50197-1 (SABS, 2000) for common cements and SANS 50413-1:2004 (SABS, 2004) for masonry cements. The SANS 50197-1 cement classification allows for various compositions and the cement type and notation (eg CEM II A-W) and strength class (eg, 32.5 MPa) are based on the types and quantities of Portland cement clinker, the cement extenders and the strength obtained from the cement using a standard sand mortar test (SANS 50196-1, (SABS, 2006)). The cement types are summarised in Table 1 (SABS, 2000). The Table shows the notation and composition of the different cement types indicating the percentages of clinker and type and percentage of extender for each type.

It should be noted that **not all of the cements shown in Table 1 are currently manufactured or available in South Africa** and the cement types may come and go, may change brand names and the compositions of a brand may change within the limits specified in Table 1. Brand names should thus not be used in cement stabilization but the unique cement classification nomenclature should be used. A list of currently available branded names of South African common cements, identifying those available in each province individually, is available on the Concrete Institute website (http://www.theconcreteinstitute.org.za/) As these details change periodically, the latest information should be obtained from the Concrete Institute).

Laboratory testing using these cements should state the full nomenclature of the cement according to Table 1, and preferably the manufacturer as well, as these could all have significant effects on the results of the testing at different times.

Table 1: Common cement types according to SANS 50197-1

Main types	Notation of products (types of common cement)		Composition, percentage by mass										
			Clinker	Blast Furnace slag	Silica fume	Pozzolana		Fly ash		Burnt shale	Limestone		Minor additional constituents
						natural	natural calcined	siliceous	calcareous				
			K	S	D	P	Q	V	W	T	L	LL	
CEM I	Portland cement	CEM I	95-100	-	-	-	-	-	-	-	-	-	0-5
CEM II	Portland-slag cement	CEM II A-S	80-94	6-20	-	-	-	-	-	-	-	-	0-5
		CEM II B-S	65-79	21-35	-	-	-	-	-	-	-	-	0-5
	Portland-silica fume cement	CEM II A-D	90-94	- -	6-10	- -	- -	- -	- -	- -	- -	- -	0-5 0-5
	Portland-pozzolana cement	CEM II A-P	80-94	-	-	6-20	-	-	-	-	-	-	0-5
		CEM II B-P	65-79	-	-	21-35	-	-	-	-	-	-	0-5
		CEM II A-Q	80-94	-	-	-	6-20	-	-	-	-	-	0-5
		CEM II B-Q	65-79	-	-	-	21-35	-	-	-	-	-	0-5
	Portland-flyash cement	CEM II A-V	80-94	-	-	-	-	6-20	-	-	-	-	0-5
		CEM II B-V	65-79	-	-	-	-	21-35	-	-	-	-	0-5
		CEM II A-W	80-94	-	-	-	-	-	6-20	-	-	-	0-5
		CEM II B-W	65-79	-	-	-	-	-	21-35	-	-	-	0-5
	Portland-burnt shale cement	CEM II A-T	80-94	-	-	-	-	-	-	6-20	-	-	0-5
		CEM II B-T	65-79	-	-	-	-	-	-	21-35	-	-	0-5
	Portland-limestone cement	CEM II A-L	80-94	-	-	-	-	-	-	-	6-20	-	0-5
		CEM II B-L	65-79	-	-	-	-	-	-	-	21-35	-	0-5
		CEM II A-LL	80-94	-	-	-	-	-	-	-	-	6-20	0-5
		CEM II B-LL	65-79	-	-	-	-	-	-	-	-	21-35	0-5
	Portland-composite cement	CEM II A-M	80-94	◄------	------	------	------	6-20	------	------	------	------►	0-5
		CEM II B-M	65-79	◄------	------	------	------	21-35	------	------	------	------►	0-5
CEM III	Blastfurnace cement	CEM III A	35-64	36-65	-	-	-	-	-	-	-	-	0-5
		CEM III B	20-34	66-80	-	-	-	-	-	-	-	-	0-5
		CEM III C	5-19	81-95	-	-	-	-	-	-	-	-	0-5
CEM IV	Pozzolanic cement	CEM IV A	65-89	-	◄------	------	11-35	------	------►	-	-	-	0-5
		CEM IV B	45-64	-	◄------	------	36-55	------	------►	-	-	-	0-5
CEM V	Composite cement	CEM V A	40-64	18-30	-	◄------	18-30	------►	-	-	-	-	0-5
		CEM V B	20-39	31-50	-	◄------	31-50	------►	-	-	-	-	0-5

Notes:

a) The values in the table refer to the sum of the main and minor additional constituents

b) The proportion of silica fume is limted to 10%

c) In Portland-composite cements CEM II A-M and CEM II B-M, in pozzolanic cements CEM IV A and CEM IV B, and in composite cements CEM V A and CEM V B, the main constituents other than clinker shall be declared by designation of the cem

d) The shaded areas indicate those cements that are unlikely to ever be made in South Africa.

In addition to the cement compositions listed, each cement has a strength class (determined according to SANS 50196-1 (SABS, 2006) as well based on the unconfined compressive strength of the cement with a standard sand. This identified both the actual minimum strength as well as the rate of gain of the strength (**Error! Reference source not found.**).

Table 2: Compressive strength requirement of SANS 50197-1

Strength class	Compressive strength (MPa)			
	Early strength		Standard strength	
	2 days	7 days	28 days	
32.5N	-	≥ 16.0	≥ 32.5	≤ 52.5
32.5R	≥ 10.0			
42.5N	≥ 10.0		≥ 32.5	≤ 52.5
42.5R	≥ 20.0			
52.5N	≥ 20.0		≥ 32.5	-
52.5R	≥ 30.0			

Historically, the SABS 471 Ordinary Portland Cement (OPC) was usually the cement type specified for road stabilization. Under the SANS 50197-1 specification the term OPC is no longer used but has been replaced by the CEM I cements. While the standard allows for the production of CEM I cement at grades with minimum strengths from 32.5 through to 52.5 MPa, only grades 42.5 and 52.5 are currently produced in South Africa. The 42.5 grade CEM I gives slightly higher strengths than the old OPC previously used in South Africa, but the rate of strength gain may vary considerably from that of the old OPC. As the production of CEM I cements generates disproportionally high quantities of environmentally unfriendly gases, the production of CEM I is gradually being reduced. The choice of cement for road stabilization should be based on the cement type and the nominal strength class. In general cements with a 32.5 MPa strength class should probably be considered generally, while the impact of the changed specifications is being investigated. However, laboratory testing using different cement types and classes should be carried out to confirm the final choice of cement (Section 6.4.1).

In the past, masonry cement (e.g. Wallcrete) was a commonly used stabilizer for road construction in Gauteng. Masonry cements are made under SANS 50413 specification, but currently only non-air-entraining types MC12.5X and MC 22.5X are available. Otherwise the closest SANS 50197-1 equivalent would be Portland limestone cements CEM II A and B - L and LL cements. None of these are probably directly comparable with Wallcrete as it contained more limestone and contained the air entraining agent, which the CEM II type cements do not, although this was apparently omitted when Wallcrete was to be used for stabilization.

Composite cements (CEM V A) consisting of a lower proportion of Portland cement clinker and higher proportions of blast-furnace slag and flyash are available in South Africa.

Most of the cements are supplied in 50 kg bags or in bulk.

It should be noted that a number of independent cement blenders supply various cement blends, not necessarily complying with SANS 50197-1. Cognisance should be taken of this in terms of their performance and potential variability for road stabilization.

2.2.2 Lime

There are two types of lime available for stabilization of soils, namely, hydrated and unhydrated. Calcitic, magnesium or dolomitic lime can be used (See paragraph 2.4.2). Hydrated lime is supplied in 25 kg and 40 kg bags and in bulk. Unhydrated lime is only available in bulk.

- Dry hydrated lime is most commonly used for the lime stabilization of soils. It is easy to use and when used in small quantities (less than about 2 per cent depending on the ICS of the material) usually only results in a modification process. This is mostly restricted to cation exchange and flocculation of the particles with little cementation. When used in larger quantities (probably greater than 3 or 4%, depending on the ICS), pozzolanic reactions may also occur and significant cementation develops over time.
- Unhydrated lime is available as a high reaction grade product that is produced in rotating furnaces. Use of this material has significant safety hazards and it is thus not widely specified or used in South Africa.

2.2.3 Ground Granulated Blast-furnace Slag

Ground granulated blast-furnace slag (GGBS), sometimes known simply as slag, is a latent hydraulic binder and thus must be mixed with cement or lime to develop the high pH necessary to become reactive. The blended cement/GGBS mixture is marketed as CEM II A or B-S or CEM III A, B or C, depending on the percentage of GGBS included in the blend. The CEM V cements available in South Africa may also contain a high proportion of GGBS.

2.2.4 Flyash

Flyash, like GGBS is not a stabilizer on its own, but is a pozzolan that reacts with lime. It is also marketed as a blend in the CEM II A-V or CEM II B-V range (calcareous flyash (W) is not used in South Africa) and can be a component of the CEM IV and CEM V cements.

Fly ash is supplied in bulk and in 40 kg bags.

2.3. Manufacture of stabilizers

2.3.1 Cement

Cement clinker is manufactured by heating together a mixture of raw materials including limestone, shale and iron ore, which provide sources of calcium, silica and alumina and iron respectively. The clinker formed is then ground to a fine powder together with a small quantity of gypsum for set control, which is the basis of all the cements produced. The range of cements other than CEM I consists of clinker ground together or blended with various percentages of cement extenders (eg, GGBS, limestone, flyash, etc).

2.3.2 Lime

Unhydrated (unslaked) lime is produced by heating limestone (calcium carbonate - $CaCO_3$) or dolomite (calcium magnesium carbonate - $CaCO_3.MgCO_3$) to form calcium oxide (CaO) with varying percentages of magnesium oxide (MgO). This can be slaked by treatment with steam or water, and calcium hydroxide ($Ca(OH)_2$) or calcium and magnesium hydroxide ($Ca(OH)_2 + Mg(OH)_2$) is formed. The hydration of calcium oxide is normally much faster than that of magnesium oxide.

Hydrated lime is also produced as a by-product of the manufacture of acetylene from calcium carbide (carbide lime). The wet product is known as "wet lime", which has the disadvantage of variable water content and is thus difficult to mix with cohesive soils. It is also available in the dry state. This product has a high degree of purity, with the calcium hydroxide content usually exceeding 90 %. This can be used for stabilization.

Inert ground calcium carbonates (agricultural lime) have very little (if any) effect as a soil stabilizer. It is thus essential that the free calcium oxide and/or hydroxide content, known as the “available lime” is sufficiently high to provide the necessary stabilization potential.

2.3.3 Ground Granulated Blast-furnace Slag (GGBS)

When iron ore is reduced to iron in a blast-furnace, any silicates and aluminates in the ore react with the dolomitic flux to form a slag.

The slag is tapped off the furnace and quenched in water causing it to granulate. This granular blast-furnace slag is then finely milled to become GGBS, which should comply with SANS 55167 (old SANS 1491: Part 1). This specification is only relevant if the GGBS is to be supplied separately to a concrete producer – when blended with cement clinker to produce the cement types shown in Table 1, the requirements of SANS 50197-1 are applicable.

Milled air-cooled slag from a slag dump or steel plant is not acceptable as a cementitious slag. Material placed on slag dumps is allowed to cool slowly resulting in a loss of reactivity.

The COREX type direct reduction iron refining process used at the Arcelormittal plant in Saldanha Bay produces a granulated slag that is used as a cement extender in the Western Cape cements, with slightly different properties to conventional blast-furnace slags.

2.3.4 Flyash

Fly ash is a by-product of modern power stations. Fine material (flyash) is collected by electrostatic or mechanical precipitation in the flues. This flyash is classified and should comply with SANS 50450 (old SANS 1491: Part 2) and can be used by cement producers as a cement extender or blended with lime on site as a stabilizer. When used as a cement extender, the flyash should comply with the requirements of SANS 50197-1.

2.4. Composition and Properties

2.4.1 Cement

Composition

Cement consists mainly of the components given in Table 3, but small quantities of other elements and additives may also be present. The percentages provided are typical ranges (Owens, 2009), but can vary from supplier to supplier and even over time from one supplier. It should be noted that the added extenders in composite cements can fluctuate significantly within the ranges specified in Table 1.

Table 3: Typical compound composition of South African Portland cements (Owens, 2009)

Cement mineral	Composition	Percentage per mass in cement (%)
Tricalcium silicate	$3CaO.SiO_2$	45 - 65
Dicalcium silicate	$2CaO.SiO_2$	10 - 35
Tricalcium aluminate	$3CaO.Al_2O_3$	4 - 10
Tetracalcium aluminoferrite	$4CaO.Al_2O_3Fe_2O_3$	5 - 10
Magnesium oxide	MgO	0.3 - 4.0
Gypsum	$Ca SO_4.2H_2O$	3.5 - 7
Free lime	CaO	0.3 - 2.5

Hydration of the tricalcium aluminate results in the early set of cement-water pastes. The long-term strength is provided by hydration of the tricalcium and dicalcium silicates, which initially form calcium silicate hydrate gels. With time, these gels crystallise and form a strong interlocking matrix, cementing the soil and aggregate particles together.

The hardening of dicalcium silicate takes place slowly (only after about 28 days) and this material does not initially contribute much towards the early strength of the hydrated cement. The tri- and dicalcium silicates contribute about 95% of the overall strength of the final stabilized material.

Gypsum is ground together with the clinker to retard the rapid setting of the cement by coating the tricalcium aluminate, which reacts very quickly, early during the hydration process.

Small quantities of free calcium oxide are present in clinker and additional slaked lime (about 20% $Ca(OH)_2$) is released as part of the calcium silicate hydration reactions. The presence of this lime is essential to maintain the high pH in the stabilized materials necessary for long-term durability.

The properties of any cement are influenced by the amount and nature of the constituents and extenders present. The method of manufacture and source materials utilised also have a large influence on the properties of cement, particularly the cooling method that is used in the manufacture of the clinker.

The degree of grinding has a significant impact on the rate of setting of the cement, with finer cements setting much quicker than coarser cements.

Research has shown that the hydration properties of two cement samples with identical compositions are not necessarily the same. For this reason the chemical composition of cement can only be regarded as an indication of its probable behaviour. In addition to specific chemical properties, any cement must also possess certain definite physical properties, e.g. soundness, setting times and time-strength requirements, which are specified in the SANS 50197-1 requirements.

Setting and Hardening of Cement

- **Chemical reaction of cement with water:** Cement reacts with water to release hydrates of the constituent particles (mainly calcium-silicate hydrate: tobermorite). This reaction is known as the hydration of cement. It is, however, a complex process and will not be dealt with further: detail can be found in concrete textbooks such as Owens (2009) and Hewlett (1988). The calcium hydroxide released on the hydration of calcium silicates serves a number of useful functions:

  - It provides calcium ions for possible cation exchange reactions similar to those occurring during lime stabilization.
  - It reacts with any clays present in the soil or gravel as a pozzolan, which produces added strength and reduces the plasticity.
  - It maintains a high pH in the cemented material, which is necessary to ensure long-term durability of the cementitious products.

  In stabilized soils, the additional calcium hydroxide is an advantage unlike its presence in concrete, where it reduces the durability of concrete.

- **Factors that influence the rate of hydration:** Hydration of cement commences as soon as the cement makes contact with the water. The rate of hydration and thereby the rate of strength development slows as it becomes more difficult for the unreacted cement to come into contact with water.

  Reference has already been made to the fineness of the cement particles. Cement particles that are more finely ground have an increased specific surface area, i.e. the total surface area of the particles. The hydration rate is increased because of the greater opportunity for contact between cement and water.

  Temperature plays an important role in all chemical reactions. As the temperature falls, the cement reaction is slowed down, and when the temperature is below 5°C, no (or very little) reaction takes place. The opposite is also true: as the temperature rises the reaction speeds up and this can have serious implications when roads are stabilized in hot dry areas. Asphalt road surface temperatures in excess of 70°C have been measured in the western parts of South Africa although there is little knowledge of temperatures of gravel or crushed stone materials being treated with cement – high temperatures could, however, develop in the stabilized materials once primed.

- **Setting time:** The time taken for the neat cement paste to set or to harden is known as the setting time. Two stages are recognised, namely, initial set and final set. There is no well-defined physical meaning to these terms, which arbitrarily describe the gradual setting taking place (Hewlett, 1988). However, after the initial set the cement-water paste is regarded as unworkable (Owens, 2009).

Generally, but not necessarily in every case, the setting time increases as the main cement type changes from CEM 1 to CEM III and the strength class reduces from 52.5 to 32.5 MPa. The use of materials with strength classes higher than 32.5 MPa for soil stabilization is thus not recommended without careful testing. This fact was identified in Europe, where a pre-standard on hydraulic road binders (ENV 13282) has been released (ECS, 2000). This pre-standard has strength classes in the range 5.0 to 32.5 MPa and a minimum initial setting time of 120 minutes.

Examples of setting times for various cements at different temperatures have been discussed by Paige-Green and Netterberg (2004).

2.4.2 Lime

Classification

SANS 824: 2006 defines three types of lime based on the ratio of calcium oxide to magnesium oxide, as shown in Table 4.

Table 4: Classification of lime (SANS 824)

Type	CaO/MgO Ratio (R)	Available lime (%)	
		As CaO	As $Ca(OH)_2$
Calcium*	$R \geq 14.0$	50	66
Magnesium	$2.0 \leq R < 14.0$	35	46
Dolomitic	$1.3 \leq R < 2.0$	30	40
* Quicklime minimum 80 % available CaO (106% $Ca(OH)_2$)			

Lime should have at least 75% 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 that actually reacts with the soil and not the “lime” incorporated in other minerals. Calcium carbonate for instance would show up in a chemical analysis in the CaO content but would not be available for reaction. 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 SANS 824 can provide satisfactory results and the selection of a particular lime and lime content should be based on laboratory tests on the soil concerned.

These categories of lime apply to both unhydrated and hydrated forms. The calcium hydroxide is mostly responsible for the positive effects of lime stabilization. Magnesium hydroxide apparently contributes little to the process. The available lime content (normally reported as CaO, but actually $Ca(OH)_2$) is therefore a most important property of lime for soil stabilization.

The presence of up to five per cent of carbonate calculated as CO_2 (a measure of the efficiency of burning and of subsequent carbonation) and up to 3 per cent of free water is permitted.

Carbonation of Lime

Lime has the ability to absorb carbon dioxide (CO_2) from the atmosphere and to react with it to form calcium carbonate ($CaCO_3$). This $CaCO_3$ reduces the amount of active CaO in the lime. The $CaCO_3$ also weakens the cementing action. Lime should therefore never be unnecessarily exposed to the air. The practice of spreading lime and leaving it unmixed or exposed for a number of days before mixing should be avoided.

Reaction of Sulphates with Lime

When soluble sulphates are present in the gravel or soil, they can react with the $Ca(OH)_2$ and alumina in the soil to form the mineral ettringite. This is accompanied by expansion, causing a reduction in the strength of the stabilized material. Severe damage in the form of heave has been reported overseas due to this mechanism when clayey soils containing excessive sulphates were stabilized with lime. This problem can be neutralised by the addition of GGBS to the lime (Celik and Nalbantoglu, 2013).

2.4.3 Ground Granulated Blast-furnace Slag (GGBS)

GGBS usually consists of about 34% calcium oxide, 33% silica and smaller fractions of alumina (16%), magnesia (14%) and other elements (Owens, 2009). The calcium oxide content is insufficient to give it true cementitious properties. As such, it must be mixed with an alkaline material (usually cement or lime), for use as a stabilizer. GGBS cannot be used on its own as a stabilizer. As indicated above, the addition of 6% GGBS to a lime treated soil containing 1% sulphate reduced the swell from 8% to 1% (Celik and Nalbantoglu, 2013).

2.4.4 Flyash

Flyash consists of spherical vitreous particles with the basic components being silica, alumina and lime. Like GGBS, it does not usually possess binding characteristics of its own, but reacts with CaO or $Ca(OH)_2$. Various percentages of pozzolanic flyash can be used as a standard cement extender as indicated in Table 1. Not all flyash is pozzolanic and for such use it must comply with the requirements of SANS 50540 and SANS 50197-1.

2.5. Specifications

Table 1 summarises the composition of possible cements. The compositions of other stabilizers as well as the relevant SABS specification are listed in Table 5.

Table 5: Applicable SABS specifications

Stabilizer Abbreviation	SABS specification	Composition
Cement:	See Table 3.1	
Lime	SANS 824 : 2012	Lime
Ground granulated blast-furnace slag (GGBS)*	SANS 55167 2011	GGBS
Fly ash (FA)	SANS 50450 : 2011	Fly ash
* Including COREX slag		

2.6. Storage and Handling

For convenience, it is preferable that the stabilizer is delivered and spread with a bulk tanker. Alternatively, where bags are used, they should be off-loaded directly onto the road and used immediately. Where these options are not feasible, the material should be handled and stored as follows.

2.6.1 Storage in Bags

Cement

Even when cement is correctly stored, it will deteriorate with time. In a damp environment, deterioration occurs even more rapidly.

In order to minimise the loss of strength, protection from dampness and humidity must be ensured. Storage sheds should be watertight and of solid construction. Floors should be waterproof and covered with a heavy-duty plastic sheet. Bags should be stored on the delivery pallets, which should be stacked closely together (to reduce the circulation of air) and away from any outside walls. Bags should be stored in such a way that older bags are used first. Vertical stacking of loose bags should not exceed 12 bags high. Doors and windows should be kept shut. Despite these precautions, cement should not be stored for longer than two months from the date of manufacture.

When cement is stored for a short period on site it should be protected with a waterproof cover.

Lime

Hydrated lime does not deteriorate as rapidly as cement in storage. When exposed to the atmosphere it carbonates with time to form calcium carbonate, which is non-reactive in stabilization. Due to its greater stability, lime bags can be stored on a raised wooden platform and covered with waterproof tarpaulins or heavy plastic for up to six months after manufacture. Other than this, similar storage conditions to those discussed above for cement should be applied for lime.

GGBS and Flyash

Limited exposure of flyash to moisture or the atmosphere will have little detrimental effect on it. However, it should be stored with some care in a dry place to avoid contamination. GGBS can be mildly hydraulic, however, and should thus be treated as for cement.

Where doubt exists as to the quality of any stabilizer as a result of extended or unknown storage conditions, laboratory testing should be carried out to check the effect the stored stabilizer has on a soil compared with a fresh stabilizer of the same type.

2.6.2 Bulk Storage

Portable silos are available for the storage of stabilizers on construction sites. These silos have a capacity of between 12 and 50 tonnes and are filled by means of special bulk tankers. There are several advantages related to the use of silos, namely:

- Silos are compact.
- The stabilizer is used in the sequence in which it is delivered.
- Wastage as a result of broken bags is eliminated.
- Loss of strength as a result of premature hydration is minimised.
- Handling costs are reduced.

2.6.3 Handling and Safety Measures

Bags of stabilizer will break if treated roughly. If a bag is broken during handling, the contents should immediately be transferred to two empty bags. Bags previously containing material other than stabilizers such as fertiliser or sugar should not be used for this purpose. If a bag is broken on the road, and mixing is not done within the specified time, the stabilizer must be removed and the bag replaced. It is preferable that bags are not left on the road for longer than one day as they may absorb moisture from the underlying material or be damaged during other construction activities.

Stabilizers should be handled with care. Cement and lime are strongly alkaline and can be harmful to eyes and skin, causing skin irritation (dermatitis) after prolonged contact. Particular care should be taken when working with unhydrated lime - contact with body moisture (perspiration, eyes and mucous membranes) results in hydration of the lime in a strongly exothermic reaction, which can cause severe burns. Safety data sheets from stabilizer suppliers should be referred to. Additional information in this regard is provided in Section 7.2.

The following safety precautions should be standard when working with cement and lime:

- Workers should wear the prescribed personal protective equipment (PPE) including overalls, boots, gloves, dust masks and goggles at all times.
- All exposed body parts especially the feet should be washed thoroughly after handling stabilizers. A bath or shower is recommended.
- Petroleum jelly or barrier cream should be applied to exposed parts of workers.
- If unslaked lime comes into contact with the skin it should be washed off immediately with copious cold water.

2.7. Bulk Distribution

Stabilizer is increasingly delivered directly to the road by bulk tanker, which spreads it where it is needed at the specified rate. The advantages of this system are that the stabilizer is always fresh, and it can be more easily and evenly spread than when using bags.

Previously, the cement industry used colour coded plastic seals on the tanker valves to identify the stabilizer type in the tanker. These were standardised across the industry. (NB: Although colour coded seals are still applied to bulk tankers, these are currently company specific and not industry specific).

The serial number on the seal identifies the supplier and depot, and should correlate with the information on the delivery slip.

The slag and flyash suppliers use similar seals but with company specific colour codes. The delivery notes should thus be used to confirm that the required stabilizer type is delivered.

It is important when designing lime-stabilized layers and placing the lime on the road that the low density is considered. The bulk density of hydrated lime is about one third that of soil and thus a 3% application of lime by mass will be equivalent to about 9% by volume. Transferring laboratory data (usually calculated on a mass basis) to the field (volume basis) must take tis into account.

3. MECHANISMS OF CHEMICAL STABILIZATION

3.1. Introduction

This chapter describes the mechanisms by which various stabilizers interact with the soil resulting in modification, cementation or a combination of the two.

The strength of stabilized soil and the rate of strength development depend upon the following:

- Soil type and properties
- Quantity of stabilizer added
- Type and fineness of stabilizer
- Uniformity of mixing
- Density to which the stabilized soil or gravel is compacted and duration of compaction
- Temperature during compaction
- Curing period and conditions

Although there are fundamental differences between cement and lime stabilization, the basic principles are similar in most respects.

3.2. Cement

Cement stabilization primarily results in cementation, but a secondary reaction related to the calcium hydroxide generated during hydration of cement also occurs (see Section 2.4.1). The end product is a cemented material consisting of the original soil, in which any clay minerals would have been partially or completely destroyed or altered (reduced plasticity).

Various chemical reactions take place during stabilization and are discussed in the literature (Owens, 2009; Hewlett, 1988; Ballantine and Rossouw, 1989). In essence, crystals of hydrated calcium silicates and hydrated calcium aluminates generated during these reactions join together and bind the individual soil particles providing significant compressive and tensile strength (Figure 2).

The main reactions that occur can be summarised as follows: (Note that the notation used is cement notation and not conventional chemical formulae).

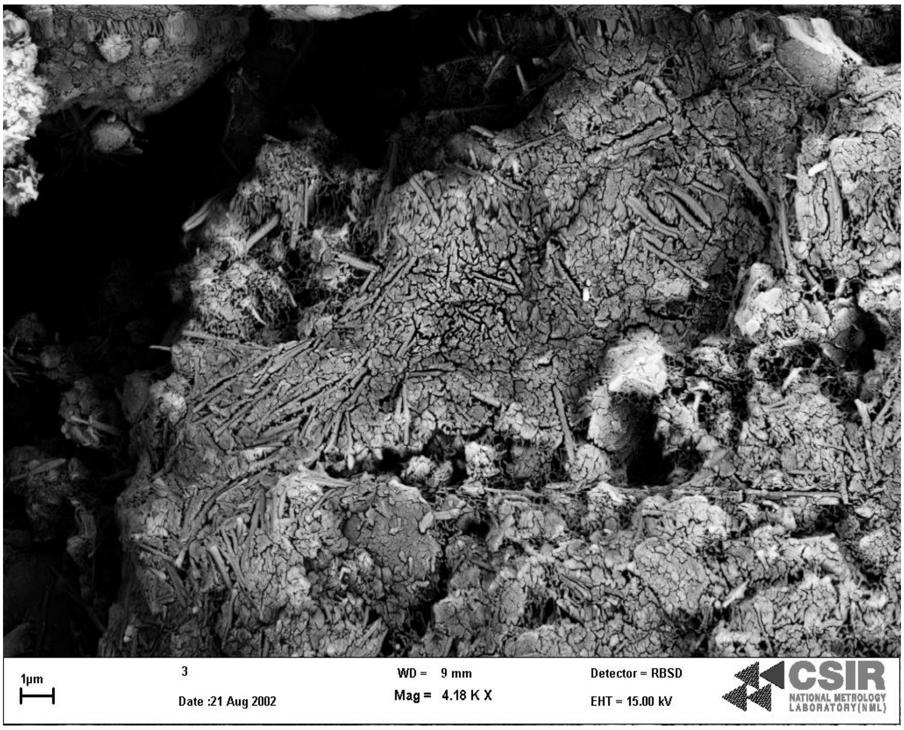

Figure 2 : Scanning electron micrograph of stabilized material

$2C_3S + 6H \rightarrow C_3S_2H_3 + 3CH$

$2C_2S + 4H \rightarrow C_3S_2H_3 + CH$

$C_3A + CH + 12H \rightarrow C_4AH_{13}$

$C_4AF + 4CH + 22H \rightarrow C_4AH_{13} + C_4FH_{13}$

where

C = calcium oxide (CaO)

S = silica (SiO_2)

A = alumina (AL_2O_3)

F = ferric oxide (Fe_2O_3)

H = water (H_2O)

and (see **Error! Reference source not found.**)

C_3S = tricalcium silicate

C_2S = dicalcium silicate

C_3A = tricalcium aluminate

C_4AF = tetracalcium aluminoferrite

CH = calcium hydroxide (hydrated lime $Ca(OH)_2$)

The reactions do not all proceed at the same rate nor to the same extent. Figure 3 provides an idealised diagram of the effects and timing of the different cement components on the overall strength development.

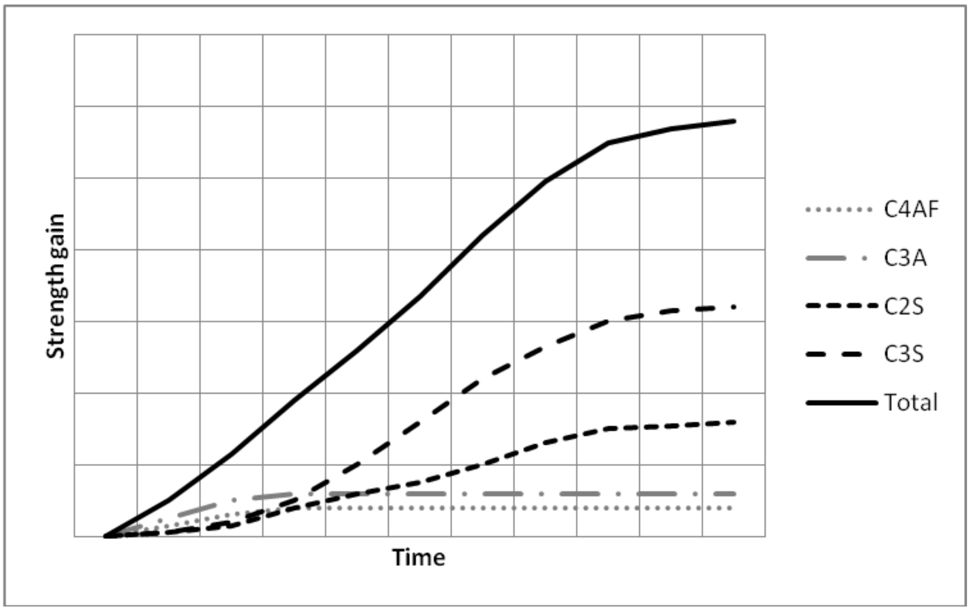

Figure 3: Idealised illustration of effect of different cementitious components

Some organic compounds are believed to interfere with the hydration of cement, which causes a reduction in strength, but not all organic materials affect the reaction. 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 mixture (by mass) is at least 12.1 fifteen minutes after mixing. Organic materials have not presented serious problems in soil stabilization in South Africa in the past but it should be borne in mind that problems could be encountered.

Sugar has serious deleterious effects on cement and this may affect cement stabilization in sugar farming areas. The presence of sugar in materials to be stabilized can be tested using SABS 5833.

3.3. Lime

The development of strength in soils treated with lime is usually slower than treatment with cement but is influenced by similar factors.

Lime stabilization entails the rapid (sometimes almost instantaneous) *modification* of the clay fraction of a material, and where sufficient stabilizer is available, continues with the development of hydrated calcium aluminates and silicates and eventual *cementation*. The absence of clay, however, does not necessarily always mean that a material will not react with lime as some materials contain non-plastic pozzolans.

3.3.1 Modification

The addition of lime to a clayey soil results in an early and rapid reaction accompanied by a change in the soil properties, primarily a reduction in plasticity and a small increase in CBR. There may, however, be no significant development of compressive or tensile strength and the material will be classified as unbound. This phase is known as modification and is the rapid phase of the mechanism of stabilization involving ion exchange and flocculation of the clay *mineral* fraction of the material. It is important to note that the clay mineral fraction i.e., the colloidal alumina-silicate minerals) may differ from the clay-sized particle fraction (i.e., those particles finer than 0.002 mm that are not alumina-silicate clay minerals). The former particles are necessary for lime reaction to occur.

The physico-chemical reactions involved are as follows:

<u>Ion exchange</u>

Clay particles (particularly plastic ones) can hold large amounts of water. They highly negatively-charged surfaces that attract free cations (positively charged well as water dipoles. This causes a highly diffused water layer around the clay separating the particles and causing clays to be weak and unstable. On the lime, the large Ca^{++} ion from the lime displaces the smaller H^+ and Na^+ ions on the surface of the clay particles (Ballantine and Rossouw, 1989). The Na^+ ion can hold molecules of water as opposed to the Ca^{++} ion, which can hold only two molecules water. As a result of the ion exchange process, the soil is changed from a condition (water attractive) to a hydrophobic (water repellent) condition and the water is released and can evaporate. This explains why the addition of lime unslaked) causes wet soil to dry out so dramatically and quickly. This also has the of reducing the liquid limit and consequently the plasticity index (

On the Witwatersrand, lime is often added to acidic, sulphate-contaminated crushed stone to prevent salt damage. This can be regarded as a form of ion exchange although clay minerals are not involved – the actual mechanism is probably more a reduction in the solubility of the salts in the high pH environment.

Table 6). This process can be almost instantaneous for small quantities of low activity clays (e.g. kaolinite and illite) but may take a number of days for higher quantities of more active clays (e.g. smectite). Although the clays attract mainly cations to their surface, anions in the material can also play a part in the ion exchange reactions.

Flocculation

Crowding of additional Ca^{++} ions on the surface of the negatively charged clay particles works together with the ion exchange described above to raise the positive electrical charge (Ballantine and Rossouw, 1989). The bond between clay particles is dependent upon this charge and the divalent calcium ions (Ca^{++}) replace the univalent sodium (Na^{+}) and hydrogen ions (H^{+}) and strengthen the attraction forces between the particles. They combine in a phenomenon known as flocculation. This is manifested as a decrease in the plasticity and the development of a more friable material (leading to a small increase in strength).

These two processes require the presence of clay minerals and thus will usually not occur in sandy materials unless non-plastic pozzolans (e.g., amorphous silica) are present. Some materials such as highly weathered basic crystalline rocks can react quickly (within one hour) with up to about 8.5 per cent lime (Clauss and Loudon, 1971). This quantity of lime that is utilised in these processes is referred to as the initial lime demand or initial consumption of lime (ICL) of the soil. In some materials with high concentrations of active clays, the lime may not be able to “reach” all of the clay and the demand determined will be too low. This is normally indicated by the pH dropping after a number of hours. If the lime added is not sufficient to satisfy the initial demand, the possibility of further release of plastic fines and subsequent weakening of the material exists. A further problem is that the pH of the stabilized material will not be maintained above about 12.0, the minimum pH required to ensure long-term hardening and stability (durability) of the cementitious products. It should be noted that the work by Clauss and Loudon (1971) was carried out on the material fraction passing the 0.425 mm sieve. Using the fraction passing 19 mm, as is current practice, the equivalent ICL would usually be somewhat lower, but the test result does not require the correction for the fraction tested.

An example of the reversion of the plasticity index in a stabilized base layer leading to premature failure of the road has been fully described (Pinard and Jackalas, 1987). A partly weathered basalt with an ICL of 4.5 was stabilized (modified) with 2% lime reducing 90% of the plasticity indices to less than 2%. Within 18 months of construction, more than 80 of the PIs were back in excess of 8%.

On the Witwatersrand, lime is often added to acidic, sulphate-contaminated crushed stone to prevent salt damage. This can be regarded as a form of ion exchange although clay minerals are not involved – the actual mechanism is probably more a reduction in the solubility of the salts in the high pH environment.

Table 6: 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, KZN	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 (Gauteng)	1.9	LL	40	36	-	-
			PI	11	5	SP	SP
	N3-12	2.0	LL	38	35	35	-
			PI	13	5	2	SP
Decomposed dolerite	N2-27/28 (KZN)	2.7	LL	34	27	-	-
			PI	14	4	SP	SP
	N2-27/28	2.3	LL	38	34	32	-
			PI	17	7	3	SP
	N2-27/28	2.3	LL	46	33	29	29
			PI	21	10	6	4
	N3-6	2.0	LL	31	26	-	-
			PI	9	3	SP	SP
	N3-6	2.5	LL	38	28	-	-
			PI	15	6	SP	SP
Decomposed lava (Ventersdorp)	N103	0.45	LL	48	30	28	-
			PI	21	10	4	SP
	N103	1.74	LL	53	46	40	-
			PI	24	13	5	SP
	N103	0.31	LL	65	46	40	-
			PI	36	10	6	SP
Silty sand (Berea Red)	KZN 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 h, soaked in water for 4 h, and tested in unconfined compression before being broken up for Atterberg limit testing. The first 4 materials were also cured at room temperature for 56 days and the same results obtained.

3.3.2 Cementation (pozzolanic reaction)

Cementation usually takes longer than modification and will continue for as long as there is available clay, moisture and a pH in excess of about 12.0. **The cementation reaction follows on from modification provided sufficient stabilizer remains and is defined as the development of a significant tensile strength resulting in a bound (albeit lightly bound) material**. There is no clearly defined boundary between modification and cementation and one state merges into the other. During cementation, the clay mineral structure is broken down (the solubility of alumina and silica is increased under strongly alkaline conditions) and forms colloidal gels of calcium aluminate and silicate hydrates (tobermorite), which have cementing properties similar to those of Portland cement when they crystallise and harden. High temperatures accelerate the process. For permanent cementation to occur, the quantity of stabilizer must exceed the initial stabilizer demand (in this case lime) of the material. In the absence of better information, this has been proposed as at least one per cent higher than the ICL (Sampson and Paige-Green, 1990).

In soils containing significant amounts of amorphous silica, lime can react with this forming calcium silicate hydrate without the presence of any clay minerals. This explains why some sandstones and calcretes can produce better and more rapid strengths with lime than with cement.

The most significant difference in the mechanism of cement and lime stabilization is that cement contains all the elements to generate the calcium silicate and alumina hydrates, while lime must react with clay minerals and/or other pozzolans in the soil to create these cementitious materials.

3.4. Ground Granulated Blast-furnace Slag and Flyash

With the commercial availability of composite cements, the use of separate GGBS and flyash for soil stabilization has reduced significantly. Both GGBS and flyash are inert unless activated by lime or the high pH created by lime or cement. When mixed with lime or cement and in the presence of moisture they become reactive and the same cementation mechanism that occurs for cement will take place. Although it is possible to blend lime with flyash and GGBS on site, it is generally much more practicable to use a commercial “off-the-shelf” composite cement, unless local materials or site conditions dictate otherwise.

3.5. Effect of Stabilizers on Soil Properties

The purpose of stabilization is to improve the properties of a soil such that the soil is suitable for use in the relevant pavement layer and will provide the necessary properties over the design life of the road. Stabilization can result in one or more of the following improvements in soil properties:

- improved strength;
- improved workability;
- improved durability;
- reduced permeability;
- reduced plasticity;
- reduced shrinkage, and
- reduced swell.

The actual results achieved are controlled by the type and quantity of stabilizer used and the physical properties of the natural soil (e.g. grading, plasticity, linear shrinkage). The outcome of this is either modification or a combination of modification and cementation, depending on the initial material properties and amount and type of stabilizer added.

3.5.1 The Effect of Cement

Cement will generally improve the strength and behavioural characteristics of most low plasticity soils to a state where they can be used as structural layers in pavements. Different soils have different stabilization characteristics as discussed below. The ultimate strength and rate of strength gain depend on various factors of which the following are the most important:

- Quantity of cement added
- Characteristics of the cement (specifically its fineness and setting and hardening times)
- Soil properties (whether the soil is coarse or fine and is plastic or not)
- Mixing time and temperature
- Compaction of the soil and the moisture content at which compaction takes place
- Curing conditions.

Cement affects different soils in various ways. These are summarised as follows:

- Cement contains calcium, silicates and aluminates - all of the components essential for cementation. Theoretically, any material can thus be treated with cement. However, in practice, components of the soil can affect the cementation process. **For example, in a soil with a high active clay content, calcium ions can be taken up by the clay during cation exchange reactions, thereby removing an important constituent of the cement-hydration reaction.** Cement is thus less effective on clayey materials if significant cementation is required.

- Cement contains a small percentage of free lime and during hydration further lime is released. A rapid but limited amount of modification (reduction in PI) can therefore take place in plastic soils when cement is used as a stabilizer. This lime is essential, however, for maintaining a high pH in order to keep the hydrated cement stable.
- Cement is most cost-effective for the stabilization of a sandy soil or gravel with a low active clay content (i.e. typically with a low Plasticity Index (PI), usually less than 10%).
- If clayey soils (PI > 10%) must be stabilized with cement, it is usual to treat them first with a small percentage of lime to reduce the plasticity (thereby improving the workability) and thereafter to strengthen them (through cementation) using cement.
- The durability of marginal quality aggregate particles can be improved by stabilization with cement. Durability is generally improved (Clauss, 1967) under high pH environments.

Certain deleterious minerals and soil components can have a negative effect on the stabilization process. These include soluble sulphates, acids, organic compounds and other active constituents.

3.5.2 The Effect of Lime

Highly plastic clays and water sensitive subgrade soils are difficult to work and compact, especially when wet. Where wet soil conditions exist, either as a result of rain, a naturally occurring perched water table or a marsh, certain measures must be taken to improve the workability of the soil. When lime is added to wet soil, the soil becomes more friable, its plasticity is reduced and it is partially dried out resulting in better compactability. When the lime-stabilized soil is compacted, the negative effect of later heavy rains is reduced. On many construction projects where prolonged rains have affected progress, the lost time has been made up through the use of lime (unslaked lime under extreme conditions).

Lime stabilization reactions take place mainly between the lime and the clay fraction of the soils. Sandy soils therefore do not usually stabilize well with lime. However certain non-plastic calcretes and sandstones containing amorphous silica undergo rapid and strong cementation with lime.

Lime affects various soils in different ways. These are summarised as follows:

- A small strength increase is usually achieved through modification due to the improved soil structure and the reduction in plasticity.
- During stabilization of moist clayey materials compaction can usually be delayed until the day after the lime has been mixed in. However, if there is a possibility of significant amorphous silica in the material, the stabilized material must be compacted within four to six hours.
- Lime stabilization reduces the water susceptibility and consequently the swelling and shrinking of clay.
- Lime can be used to raise the pH of acidic materials and prevent damage due to soluble sulphate salts.
- Lime treatment will reduce the plasticity of the soil more than an equivalent quantity of cement, due to the greater availability and accessibility of free lime for modification.
- When the same percentages of cement and lime are added to two samples of a soil, cement stabilization usually results in higher initial strengths. However, under controlled curing, the same order of strengths is obtained after three to six months in materials with a PI of 8 to 12%.

3.5.3 The Effect of GGBS Blends

With the current cement classification, the differentiation between GGBS blends and Portland-slag and blast-furnace cements needs to be clear. Portland-slag cement (CEM II A-S or B-S), blast-furnace cement (CEM III A, B or C) and composite cements containing slag (CEM V A or B) are manufactured mixtures of cement clinker and GGBS complying with SANS 50197-1. GGBS blends with lime or cement are blended on site using GGBS complying with SANS 55167 and separate lime (SANS 824) or cement (SANS 50197-1). The following points are relevant:

- GGBS blends are generally cheaper than cement or lime on their own.
- Slag-lime and, to a lesser extent, slag-cement can be cost-effective stabilizers of clayey materials.
- Slag-lime, with up to 50 per cent GGBS, is effective in reducing plasticity.
- Slag-cement containing 50 per cent GGBS is a good stabilizer for sandy material, although the addition of more than 60 per cent GGBS usually results in a reduction in strength.
- Slag-cement does not perform as well as slag-lime in the reduction of PI.
- In the case of clayey materials the addition of GGBS:

    - to cement causes a slight reduction in strength compared with cement alone;
    - to lime improves the strength slightly compared with that of lime itself and gives early strength.
- The initial setting times of cement and slag-cement in the stabilization of sandy materials are similar, but for clayey materials the addition of GGBS appears to allow a slightly longer working time
- Stabilization of materials with a low PI (7%) can be done using lime if GGBS is added to provide the necessary pozzolans.
- GGBS blends are considered to perform better in sulphate rich environments

3.5.4 The Effect of Flyash Blends

In the SANS 50197-1 cement classification, Portland flyash cement (CEM II A or B) and Composite cement with flyash (CEM V A or B) can be manufactured. Flyash blends with lime and/or cement can also be mixed on site using flyash complying with SANS 50450 and separate lime (SANS 824) or cement (SANS 50197-1). The following points are relevant:

- Fly ash blends are generally cheaper than cement or lime, which make them the preferred stabilizer when found to be suitable. For fly ash to be suitable, it must comply with SANS 50450.
- The reaction products of flyash used in combination with cement or lime are similar to those of GGBS and cement or lime. With lime, however, flyash does not produce the same early strengths as slag.
- Mix compositions depend on the desired strength. Laboratory testing should establish which of lime or cement reacts better with any individual fly ash.
- Past experience suggests that 40 per cent fly ash combined with 60 per cent Portland cement (CEM I) reduces cracking in materials susceptible to this problem.

A number of cement manufacturers now produce cements sold as soil stabilizers for roads, which are formulated to gain strength somewhat more slowly than the normal cement blends. These are blends of Portland cement with various proportions of flyash and GGBS blends complying with SANS 50197-1 and mostly CEM II cements.

4. TESTING

4.1. Introduction

There are three objectives when testing stabilized material, namely:

- Material design testing to determine whether the material is suitable for use in a stabilized layer, both in the short and long term
- Process control testing to assure that the constructed layer meets the required standards
- Comparative testing to determine whether two laboratories obtain the same results when testing the same material. Special attention should be given to ensuring that, in these cases, only the test method is compared, with all other variables being constant.

Cementation (strength development) is an ongoing process, which must be taken into account when assessing the results of laboratory and field tests. Sampling techniques and test and curing methods will all influence the test results. The important test methods used in stabilization work are discussed below. The TMH 1 (DoT, 1979: 1986a) test methods that have traditionally been used have mostly been replaced by SANS methods (SANS 3001 series). As the SANS test methods are regularly reviewed, the latest versions of these test methods should be used.

4.2. Indicator Tests

4.2.1 Grading

SANS 3001 - GR1:2011 (wet) and GR2:2011 (dry)

The grading of the unstabilized material gives an indication of the workability, strength, permeability and compactability of a soil as well as providing an indication of the degree to which the soil can be stabilized. A grading on stabilized material is of questionable value as the stabilizer cements the particles together and the fines are typically highly under-estimated. The particle size distribution of stabilized materials thus depends more on the age of the stabilized layer and the techniques of sampling and preparation for testing than of any inherent material property. Thus, a grading analysis is not common when assessing stabilized materials. Grading analyses may still be relevant for material that is only modified, however.

4.2.2 Atterberg Limits and Bar Linear Shrinkage

SANS 3001 - GR10:2011, GR11:2010 and GR12:2010

Stabilization may be carried out to reduce the plasticity of natural soils in order that they meet the specification. If the plasticity index (PI) is tested immediately after the stabilizer is mixed in and before the modification reaction has taken place, it is possible that a higher PI may be obtained because the quantity of fines is increased by addition of the stabilizer. It is therefore recommended that the PI test is done on Unconfined Compressive Strength (UCS) briquettes after crushing or in the case of modified materials, on material removed from the CBR mould after testing. The PI test is therefore done on material that has been compacted and cured. Method GR12 (flow curve method) is recommended to give the most accurate results, although it is a little more time-consuming.

Evaluation of the PI is used in two phases during the project:

- Initial investigation of the borrow pit. An assessment of the PI is used to decide whether stabilization is necessary and which stabilizers are likely to be the most effective
- Assessment of whether the PI has been adequately reduced during construction of the stabilized layers. Typically, the material should be non-plastic after stabilization, but some materials may be modified to reduce the PI to within the specification for PI. Caution should be exercised when this is done as there is a possibility that the PI could increase during the service life of the road. Basic crystalline materials (Weinert, 1980) in particular should be treated until the material is non-plastic and not only slightly plastic).

The following issues should also be taken into consideration:

- For design and materials control, the PI after stabilization is usually done on fully cured UCS briquettes or other specially manufactured briquettes, not on samples removed from the road layer.
- A contractor is generally not held responsible for ensuring that the PI after stabilization meets the specification if the material and construction process has been approved by the Engineer.
- The onus is usually placed on the consultancy and its own technicians to determine the PIs from the UCS briquettes for reference purposes. Any significant variation should be taken as a warning that materials are changing.
- Because some limes are themselves highly plastic (Netterberg, 2004) it is advisable to determine the PI of the lime itself after similar curing conditions whenever the PI of the stabilized material must be reduced to a low level.

4.2.3 Maximum Dry Density (MDD) and Optimum Moisture Content (OMC)

SANS 3001 - GR30:2010 (untreated) and GR31:2010 (treated)

In order to ensure consistent construction and material control parameters when using stabilization, the method for determining MDD and OMC as specified in SANS- GR31 should be used. It is essential that the stabilizer of the type proposed and from the proposed source is used for the test. Unlike TMH1, which used a nominal four per cent stabilizer for the test, SANS 3001 provides for the use of any specified stabilizer content.

The old TMH1 Test Methods (A7 and A14 Appendix) had serious deficiencies in handling the material larger than 19 mm. Material larger than 19 mm was lightly crushed and returned to the sample - the result was that the material tested could bear little resemblance to the material actually being used in practice. SANS 3001 makes provision for this problem and only scalps the material at 37.5 mm, making use of the full grading smaller than this, but making note of the percentage of the sample scalped at 37.5 mm. The option of scalping at 20 mm, however, still exists but must be noted in the test report.

For the control of compaction during construction, both the laboratory MDD and the actual field dry density are required. SANS 3001-GR31 requires the laboratory material to “condition” for 3 hours (± 10 minutes) after addition of water to the cement (the old TMH method required 4 hours) and before compaction. For realistic comparison of the field and laboratory densities (quality control) the field compaction should not take much longer than 3 hours. However, the effect of time of compaction could vary considerably according to the cement type and ambient temperatures during construction. The effect of time delays on density and UCS needs to be assessed during the laboratory testing and stabilization design exercise (See section 4.2.5). This must also be considered when carrying out and assessing control test results (See section 8.2).

Typically the OMC of a material increases and the MDD decreases when a stabilizer is added. The OMC determined by SANS 3001-GR31 should be used to control compaction moisture during construction.

Figure 4 shows a typical example of the effect of a delay in compaction after mixing the cement (3%) and water for two different cements (CEM II A-V 32.5N and CEM V-A 32.5N) with a G6 quartzitic gravel. As expected the effect was greater on the CEM II cement with a reduction in MDD after 7 hours of 4%. The reduction in MDD with the CEM V was only 1%. A variation of 4% can have a marked effect on whether to reject or accept a layer during construction.

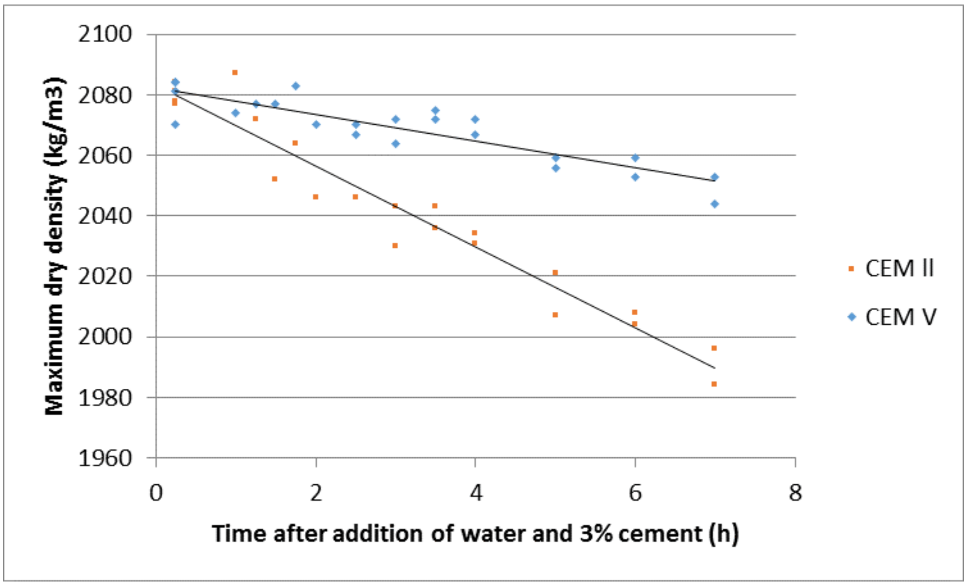

Figure 4: Effect of time lapse between mixing of cement and compaction on dry density

4.2.4 California Bearing Ratio (CBR)

SANS 3001-GR40

The CBR test is a poor measure of strength of a cemented soil. The CBR of an unstabilized soil is determined after specimens have been soaked in water for four days, by definition. When soil is stabilized using sufficient stabilizer to allow cementation to occur, the soaking facilitates and improves the curing process providing hydration water and preventing carbonation. The soaking therefore may result in an achievement of a strength that is unlikely to be achieved under field curing conditions which are usually less efficient. For lower percentages of stabilizer (modification), the soaking could have a weakening effect, although this is not always the case in the pavement.

When modification is utilised only to bring the PI into the specified requirement (assuming that all other requirements are met), the CBR should be determined to ensure that the design CBR is achieved. As carbonation may affect the lime in the material and the PI could return, it is suggested that a CBR and a PI test is carried out on the specimen after accelerated carbonation, to assess the long-term effect of the modification process on the strength (see section 9).

Various options regarding handling of the oversize are possible and any manipulation of the grading in accordance with the test methods should be reported with the test results.

4.2.5 Working time

No SANS method yet. Appendix B

As discussed in Section 4.2.3, the working time of different cements varies considerably. Based on work carried out in Australia (Vorobieff, 2006), the importance of controlling the compaction time in order not to over-compact already “partially set” stabilized material has led to a draft protocol being developed for use in South Africa This is provided in Appendix B. It is recommended that in addition to the actual method utilised, the testing should be carried out at the expected on-site working temperatures. If the cement type should change during construction, the testing needs to be repeated for the new cement type.

Ongoing monitoring of the working time of the cement/material combination should be carried out during construction, especially for projects extending over many months.

4.3. Total salts, sulphate and acids

Stabilized pavement layers may be damaged by salt crystallization, sulphate attack and acid, all of which could lead to a complete loss of cementation and/or excessive heaving and cracking. Limitation of these problems requires testing of the conductivity, the pH and where necessary the acid- and/or water- soluble sulphate content.

The electrical conductivity is a measure of the soluble salts that can go into solution in a soil paste (and thus act as an electrolyte in the soil water) and is used as a determination of the soluble salt content of the soil. This is tested using TMH 1 Method A21T.

Acid soils (pH below 6) can neutralise the high pH of the added stabilizer and should be avoided. The pH of the soil should be determined using TMH 1 test Method A20.

Sulphate and sulphides are known to react with cement (and to a lesser extent lime) to form Ettringite, a mineral that exhibits very high expansion during its formation. Sulphate minerals are usually produced during the weathering/oxidation of sulphide minerals (e.g. pyrite, marcasite, chalcopyrite, etc) and their contents are usually determined using the method for water-soluble sulphates, BS 1377 Method 9 (corrected for percentage passing 2 mm). The determination of the acid-soluble sulphate content follows BS 1377 Method 10.

4.3.1 Initial Consumption of Stabilizer (ICS)

SANS 3001-GR57

As discussed in the previous chapter, a series of reactions take place when a stabilizer is mixed with the soil. Within the first hour, initial reactions consume a part of the available lime. This does not affect GGBS or flyash, which contain little or no free-lime – instead they compete with the soil for the available lime. When these reactions are complete the addition of lime to the material will allow the pH to rise to approximately 12.4, before equilibrating at about this value.

The quantity of stabilizer that is consumed in the series of neutralising reactions is not available for strength development. This quantity of stabilizer is referred to as the initial consumption of stabilizer (ICS) and is that minimum quantity of stabilizer required by the soil-stabilizer mixture to maintain the lime saturated pH for one hour after the lime has been added (usually 12.4 at 25°C but can vary slightly depending on the lime chemistry and other compounds). The quantity of stabilizer required to satisfy the ICS varies from one soil to another. For example, soils derived from basic igneous rocks, such as dolerite, basalt and norite, can have a high initial stabilizer demand (i.e. they react rapidly with relatively large amounts of stabilizer) – any ICS value in excess of 3.5% can be considered high and would have significant economic implications. If insufficient stabilizer is available in the soil, complete modification or strength development will not take place (Pinard and Jackalas, 1987). Such material may fail in the road despite the normal control test results (e.g. PI, CBR, etc.) being satisfactory. Ferricrete can sometimes have a high ICS result and additional testing should be carried out to determine the cause of this.

For example, if the ICS is 1.5%, the relevant pavement layer should be stabilized with at least 1.0 % more stabilizer, i.e. 2.5% stabilizer, to make sure that sufficient residual stabilizer remains to retain a high pH and ensure durability. The following points should be noted:

- GGBS and flyash contain little or no free lime.
- Where the stabilizer is to be a mixture (eg lime-flyash), the ICS must be satisfied from the lime component alone.
- For a 50/50 blend of lime-flyash and an ICS of 1.5%, the minimum lime content should be 2.5% (1.5% + 1.0%), so the combined stabilizer content required would be 5.0%. Under normal conditions this blend would probably not be economical, and another stabilizer would be considered (e.g. lime for dolerite or chert, exhibiting a high ICS).

The pH of lime at 25°C is usually about 12.4 although that of cements can vary and often be higher, but small variations in the constant pH may be obtained, with the pH varying considerably with temperature. Therefore, the percentage of stabilizer required to reach a constant pH (ie, less than 0.01 pH value change for 1% increase in stabilizer) is more significant than actually attaining an absolute pH of 12.4, although the end-point should be near this value.

The test is highly sensitive to the condition of the electrode and calibration of the equipment with a high pH (about 12) standard solution is essential. It is also useful to test a saturated solution of the stabilizer (usually 10% stabilizer without any soil) to determine the “natural” pH of the stabilizer.

This test shows the initial consumption of stabilizer. However, by extending pH measurements over longer periods, changes in the stabilizer content at which the pH becomes constant will indicate ongoing consumption of the stabilizer, which could have a bearing on the durability of the material. A possible cause of this, for instance, is the penetration of the high pH solution into larger particles that have a component within them that will react with the lime. In general, the longer the period of curing the higher the ICL. In some laboratories, a combination of the one-hour and 28 day ICL is used to decide on a design value. It is not clear whether extension of the test when using cement as the stabilizer provides any useful results as hydration of the cement particles may change the pH more than the actual “consumption” of the lime component.

It is essential that the actual stabilizer/s that will be used during construction be used for the ICS and stabilization design testing.

4.4. Strength and Stiffness Tests

4.4.1 General

Strength tests are done for material control; firstly during the stage when the borrow pit is investigated and secondly when the soil is used in the layer. The borrow pit must be properly investigated beforehand and the material variability identified to minimise disruption during construction due to delivery of unacceptable or inconsistent material to site.

Often the gravel source cannot be investigated as extensively as required prior to construction or has significant variability and hence material must be sampled from the layer during construction. This can be either before or after the addition of stabilizer. The strength of this sample is then determined in the laboratory, adding stabilizer where it has not yet been added in the field. This could, however, lead to excessive costs if the material is only rejected after being processed or delivered to site.

Laboratory strength tests provide an indication of the strength of the stabilized soil as used in the road layer. This is only an approximation because of the influences of varying moisture, temperature, chemical conditions and curing. Laboratory strength tests are generally much better controlled in terms of uniform mixing, compaction and curing than the material in the actual layer. Laboratory strength testing is often protracted due to the specified curing time. Temperatures and times for accelerated curing are recommended in SANS3001-GR50 and can be considered for early results but the results need to be correlated/calibrated with those obtained from tests using conventional curing conditions. It is recommended that standard curing be used for both strength and durability tests during the initial design and testing phases with correlated accelerated testing being done for quality control.

The following tests are used to determine the strength of stabilized materials:

- Unconfined compressive strength (UCS)
- Indirect tensile strength (ITS)

The CBR test should not be used as it is typically inaccurate for cemented materials, and addition of stabilizer solely for modification is not usually recommended because of doubts concerning the long-term benefits. In specific instances, the CBR test can be used at the Engineers discretion. The CBR test is not used in the South African specifications for cemented materials (COLTO, 1998).

As the use of Mechanistic Empirical designs increases (and this is likely to increase significantly further with the introduction of the South African Pavement Design Method (SAPDM)) the need for the determination of materials stiffness (resilient modulus) and Poisson’s ratio will increase. Currently, there are limited facilities for testing these parameters in South Africa, but the number of testing locations is certain to increase with time. Protocols for these tests in South Africa are still being finalised, but they are currently based primarily on the AASHTO T307 test method.

An important property of a stabilized material is that its strength increases with age – perhaps for years, unless deleterious reactions such as carbonation affect the material.. However, the strength of a few materials, such as some ferricretes, may decrease with age. In cases of doubt, strength testing should be extended to 7, 14, 28 or even 56 days.

4.4.2 Unconfined Compressive Strength (UCS)

SANS 3001-GR31, SANS 3001-GR50 and SANS 3001-GR53

The standard specification for stabilized materials is primarily based on the UCS (COLTO, 1998). The minima and maxima for the laboratory UCS at 97 and 100 per cent of Mod AASHTO density are specified. The UCS test must be carried out according to SANS 3001-GR53 after preparation of the specimens according to SANS 3001-GR 31 and GR50, which specifies scalping of the material at 37.5 mm.

It should be noted that most material strength testing procedures make use of specimens with a height: diameter ratio of 2:1 but in South Africa a 152 by 127 mm specimen is used, i.e., the same dimensions as a CBR mould. One investigation has indicated that specimens prepared in the CBR-type mould give compressive strength values about 30 per cent higher than those 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. Various relationships between CBR and UCS are utilised from the simple UCS = CBR/100 (Gray et al, 2011) to that shown in Figure 5 (DoT, 1986b).

Although there is a definite trend, the variation is wide and the relationships depend on factors such as the material as well as the quantity and type of stabilizer and the relationship shown should be used with discretion.

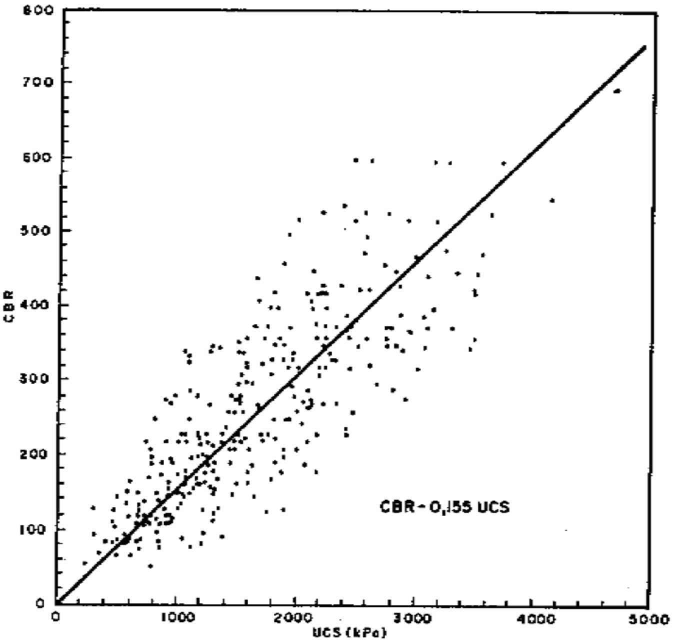

Figure 5: CBR UCS relationship

4.4.3 Indirect Tensile Strength (ITS)

SANS 3001-GR54

The latest COLTO specification (COLTO, 1998) for stabilized materials specifies minimum values for the ITS of C3 and C4 materials, based on the tensile strength providing a better representation of material behaviour in the pavement layer than the UCS. The SANS 3001-GR54 ITS test is a relatively simple test of tensile strength and involves applying a load to the curved sides of a UCS sample causing failure by splitting the cylinder along the axial plane.

The 4-point flexural beam test (Theyse, 2000) also gives an ITS result but, in addition provides more information, such as strain at break, which is an important parameter in assessing the behaviour of stabilized materials (DoT, 1986b; Theyse, 2000).

There are indications (Jordaan and Lewis, 1987) that the durability of stabilized materials is more closely related to the ITS than to the UCS as discussed in Chapter 9.

4.4.4 Accelerated curing

Standard testing for the UCS of a material requires 7 days of curing at a temperature of 23.5°C ± 1.5°C sealed in an airtight container, before soaking for 4 hours. This delay in getting results can often be too long for effective quality control and approval of constructed layer-works. For quicker results an accelerated reaction can be obtained by curing at a higher temperature.

The accelerated strength development at 72.5 ± 2.5°C for 24 hours for cement and 60.0 ± 2.0°C for 45 hours for lime has been successfully used 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 standard curing of 7-days at 22°C and it is advisable that these be used as far as possible. Where accelerated testing is used, parallel testing using the standard process should be carried out to confirm the results.

4.5. Durability Tests

4.5.1 Introduction

The large number of problems associated with stabilized layers in recent years has indicated that more attention needs to be given to the long-term durability of stabilized materials, particularly when the materials are of marginal quality.

The various durability tests discussed are:

- Wet-dry brushing test
- Erosion tests
- Residual UCS and PI after accelerated carbonation

4.5.2 Brushing Test for Durability

SANS 3001-GR55 (hand brushing) and SANS 3001-GR56 (mechanical brushing –still in preparation)

The wet/dry brushing test was developed in the United States in 1935, not only to measure the influence of moisture variation on the durability, but also to determine the influence of the surface chemistry of the soil.

The following weaknesses in this test have been identified in South Africa (Sampson and Paige-Green, 1990):

- The standard test is done on specimens compacted at Proctor compaction effort and not the modified AASHTO effort applicable to bases in South Africa.
- The method and degree of brushing is very operator-dependent.
- On gravelly materials, the loss of one or two large particles has a significant influence on the result. It is therefore important that the type of loss should be reported and that the Engineer should personally inspect the specimens after testing.

In response to these problems, it has been proposed that for the South African test, the following is implemented (Sampson and Paige-Green, 1990):

- Samples should be compacted to 100 per cent of Mod AASHTO density in a 152 mm diameter mould (now specified in both SANS 3001-GR55 and GR56).
- A mechanical brushing apparatus, developed at the CSIR, should be used to overcome operator susceptibility (SANS 3001-GR56) .

The procedures for preparation of the test specimens and carrying out the mechanical wet-dry brushing test have been reported by Sampson and Paige-Green (1990). However, due to the limited availability of the mechanical device, the hand-brushing technique is still taken as the standard.

The cycled wetting/drying process simulates poor curing practices in the field. This test has sometimes been carried out without brushing because of the significant loss (resulting from the low density in the standard Proctor compaction) with a UCS being determined after cycling (12-cycle wet/dry UCS) (De Wet and Taute, 1985). However, the mass loss from an unbrushed test should never be assessed against the loss criteria intended for a brushed test. Such practice has led to at least one major problem in southern Africa.

4.5.3 Erosion Test

Method in De Beer (1989a)

The erosion test has been developed at the CSIR (De Beer (1989a)) to simulate in the laboratory, the mechanical/hydraulic erosion of lightly cemented materials in the pavement. The test makes use of a loaded wheel running on a linear wheel track on the top of a compacted beam of stabilized and cured material. The beam is set in plaster-of-Paris in a sealed container and is tracked under water with a friction membrane between the top of the specimen and the wheel.

The test is less time consuming than the wet-dry durability test. The result of the test is expressed as the erosion index, which is related to the durability of the stabilized sample. The results are probably more applicable to the performance of stabilized materials beneath concrete pavements than conventional stabilized layers. The test should, however, be used to augment the wet-dry durability test and not to replace it.

4.5.4 Erosion test (C&CI)

Method in Van Wijk and Lovell (1986)

The use of a rotational shear device to identify stabilized materials prone to erosion beneath rigid pavements was proposed for use in the USA in 1985 (Van Wijk and Lovell, 1986). The test makes use of a rotating cylinder filled with water into which a relatively stationary specimen of the compacted and cured, stabilized material is placed. The mass loss after rotation of the cylinder relative to the specimen is used to indicate the degree of erosion of the specimen.

This test has subsequently been proposed for use locally and is currently being assessed for possible use in South Africa.

4.5.5 Residual UCS

Method in Sampson and Paige-Green, 1990

Many durability problems associated with stabilized materials have been attributed to carbonation of the stabilizer and stabilization products. A number of tests using accelerated carbonation have been investigated to assess the impact of this (Sampson and Paige-Green, 1990; De Wet and Taute, 1985) and the residual UCS after accelerated carbonation has been found to provide the most useful results. It is recommended that the residual UCS should comply with the standard design UCS and PI requirements (COLTO, 1998) to ensure durability, particularly where the material before stabilization does not meet the recommended G6 (COLTO, 1998) requirements. It is also recommended that the PI should be determined on the carbonated specimen and that the result should not exceed the maximum allowable PI for an unstabilized layer.

Investigation of samples after carbonation and UCS testing has shown that materials with stabilizer contents in excess of the ICS plus one per cent show minimal carbonation, even after accelerated carbonation. The residual UCS has also been shown to correlate with the 12-cycle wet/dry UCS and is a much quicker substitute test for this property.

4.6. Determination of the Cement Or Lime Content of Stabilized Soils

The stabilizer content of stabilized soils is determined according to Methods A15 (a), (b), (c) or (d) of TMH1 (DoT, 1986a). Methods (a), (b) and (d) are generally used:

- Method A15(a) - The determination of the cement or lime content of stabilized materials by means of the Ethylene Diamine Tetra Acetate (EDTA) test.
- Method A15(b) - The determination of the cement or lime content of cement-stabilized or lime-stabilized materials by means of a flame photometer.
- Method A15(d) - The determination of the cement or lime content of stabilized materials by means of the back titration (acid base) method.

EDTA has mostly been supplanted by Atomic Absorption (AA) analysis methods in central laboratories. The back titration method is an outstanding method for site laboratories, partly because of the larger sample tested. Atomic Absorption techniques have also replaced the use of the flame photometer in Method A15(b). The SANS 3001 - GR58 method currently in preparation follows the back titration (acid base) technique previously used in Method A15(d).

These tests need to be carried out to assess the uniformity of spreading and mixing of stabilizers during stabilization of road pavement layers. It should be noted, however, that none of the available methods is suitable for all materials. Various factors contribute to misleading results.

- When calculating the quantity of stabilizer, it is assumed that the layer after compaction is exactly the width and depth specified. Variations in thickness or width, or inclusion of part of the underlying layer, reflect as variations in the quantity of material stabilized. Although, in this situation the calculated volume of the layer, combined with the actual mass of stabilizer applied, would suggest correct stabilization, chemical tests may indicate a different stabilizer content.

- In the calculation of quantity of stabilizer, a density of the soil layer is assumed (a percentage of Modified AASHO maximum dry density). Any deviation from the estimated density as a result of material variability will manifest as a variation in the quantity of stabilizer. The following situations could thus arise:
    - The specified density is not achieved and therefore a smaller mass of soil exists in the layer. If the calculated quantity of stabilizer was mixed in, the chemical test will show a higher percentage of stabilizer compared with that specified.
    - Natural variation of the material may result in parts of the layer being above or below the laboratory determined MDD, often based on one test result. The situation could thus arise where the quantity of stabilization is rejected because of an incorrect MDD.
    - If a higher density is achieved, there is more soil in the layer than that specified in the design. The test will now register a lower percentage of stabilizer than that required.
- All of these test methods, except method A15(d) which determines the hydroxide alkalinity, determine the calcium content expressed as an oxide.. Frequently, natural materials (eg, calcretes, dolomites, dolerites, chert and other materials with high calcium contents) have calcium contents with greater variability than the percentage of stabilizer added. The control tests show up the difference between the calcium in the natural soil and the total calcium in the stabilized material. In these cases, the test is essentially meaningless.
- A similar problem has been encountered when using the EDTA method on lime-slag stabilized materials.

The nature of the test, in trying to measure very small percentages of calcium (as calcium ions or calcium oxide depending on the method), e.g. about 2% calcium for 3% cement added, and comparing this with the calcium content of the natural material (typically between 2 and 12% for acid and basic crystalline rocks respectively) is obviously problematic. The test is thus not ideal for quantity control and great care must be taken during interpretation of the results. **The quantity should therefore also be controlled physically by the counting of pockets, by weighing the tanker before and after spreading, ensuring that all of the cement has been pumped from the tanker and checking of actual distribution using canvas mats. It is important that the actual spread rates are reported back to the client.**

Another aspect to be considered is that it is assumed that the layer is absolutely uniform. In practice this is seldom the case:

(i) There are changes in material throughout the layer as a result of varying compactive effort resulting in a varying final density. From earlier discussions it should be clear that the cement or lime content, as tested chemically, also vary.

(ii) Variations with regard to the grading do occur, such that it is possible for instance to get a few large stones in a small sample. The density of these solid stones is very high and very little stabilizing agent can cling to the stones. The quantity and mixing results in such a case will be of no value.

The following situations in which the test (and most other chemical tests) cannot be used, are summarised:

(a) When variations in layer thickness and layer width arise without being quantified. (Note: It will be very difficult to show, for example, that 20 mm of the lower layer has been worked in with the layer to be stabilized).

(b) When the soil, for whatever reason, does not achieve the specified or calculated density. (Note: It must be realised, however, that a calculation can be made to determine the appropriate percentage. If the density is 5% higher than expected, the percentage of stabilizing agent that will provide good UCS results will be 5% lower. These calculations can become complex if the density of the layer varies. Variations in density are usually acceptable as long as they are above the required density.)

(c) Where the grading of the soil varies or where the CaO content of the natural material varies.

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5. PROPERTIES OF STABILIZED MATERIALS

5.1. Introduction

Changes to the fundamental soil properties such as plasticity and grading have been discussed previously (see Section 3). It is important that the differences between modification and cementation are remembered during the discussion in this Chapter.

5.2. Optimum Moisture Content (OMC) and Maximum Dry Density (MDD)

The optimum moisture content (OMC) and maximum dry density (MDD) of a material are affected by stabilization. Typically the OMC increases and the MDD decreases after stabilization. However, experience has shown that this is not always the case (GDPTRW, 2004). The increase in OMC is caused by the flocculating effect of the stabilizer on the clays and by the water demand of the stabilization reactions during hydration. The reduction in density is caused by the effective change in grading (flocculation and increased granularity of fine materials) as well as the development of early bonding between particles (tricalcium aluminates), which causes loosely bound aggregations. Compaction may not break these aggregations down and as time proceeds these become stronger and more to reduce the overall density.

It is important when comparing the MDD and the in situ density that the time between completion of mixing and compaction be noted. The time of processing will inevitably result in a reduction in density and this is exacerbated under high working temperatures. The standard laboratory test (SANS 3001-GR31) requires a three-hour period of conditioning before compaction. In practice, compaction time could be either less or greater than three hours and the laboratory determined MDD may not represent the actual field conditions (Paige-Green and Netterberg, 2004). The timing of the construction process should thus be simulated in the laboratory as closely as possible to assess the expected MDD, which ultimately is the basis of the quality control testing. This has been discussed more fully in Section 4.2.5**Error! Reference source not found.**.

Changes in OMC and MDD should be confirmed for any soil and taken into account when using stabilized materials. All soils should be tested with the actual stabilizer to be used and the specific behaviour identified.

5.3. Strength

The strength of stabilized materials starts increasing almost immediately after the stabilizer comes into contact with moisture, marginally for small quantities of lime (modification) and significantly for cement. This process may continue for many years provided moisture is present, although typically about one half (for cement and lime added in small quantities) of the ultimate strength is achieved after 28 days with about one-third after 7 days. The ultimate strength is a function of the natural material properties, the quantity and type of cement added, the density of the compacted material and the mixing and curing conditions.

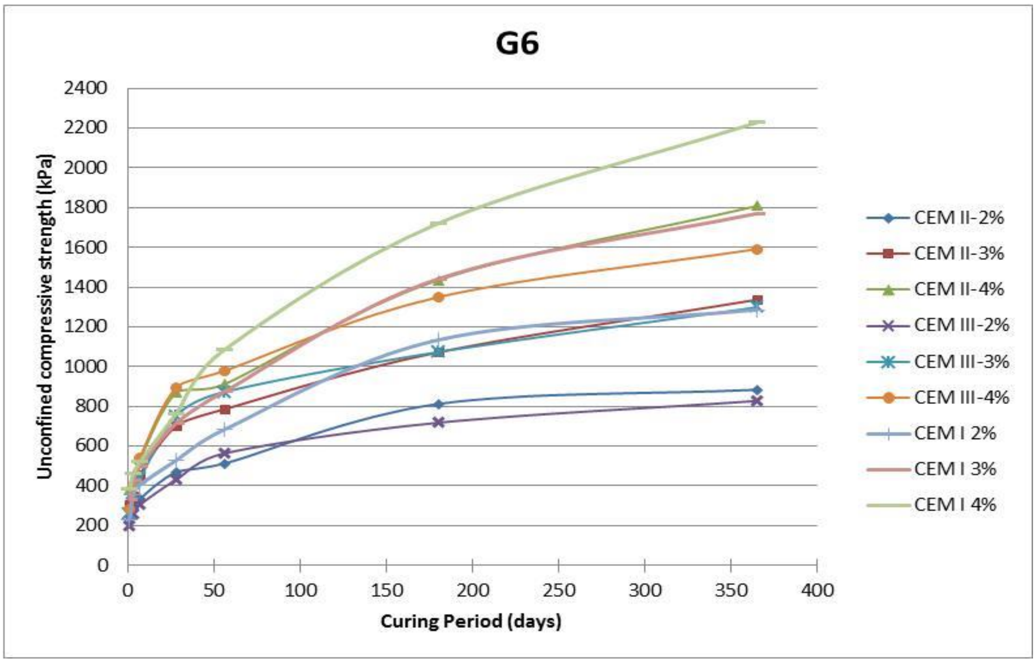
The bond strength between particles is influenced mainly by the type of material, the amount of cement and the density of the compacted material. Compaction moisture content and curing conditions also influence the strength. Typical examples of some of these effects using various percentages of current cement types on typical South African materials are shown in Figure 6 to Figure 11 but it is clear that the effects are highly material dependent. The following cements were used for the testing (their full nomenclature is provided here but in the remainder of the text only the main types are used):

CEM I 42.5 N
CEM II B-M (V-S) 32.5 N
CEM III A 32.5 N
CEM IV B-V 32.5 R
CEM V A (S-V) 32.5N

For the G6 sandstone material (Figure 6) the strength increases continuously with curing time but at different rates for different cements and generally at a decreasing rate with time (as shown on the non-logarithmic plot). The expected increase in strength with cement content is clear as is the overall increase in strength with equivalent cement contents as the cement type changes from a CEM V to a CEM I, although the CEM V performed marginally better than the CEM IV. After one year, all of the cements except the CEM IV were still causing an increase in strength of the cylinders.

The effect of the same cements on a weathered dolerite G6 material was entirely different (Figure 7). It is clear that the CEM I cement at all application rates and the CEM II and III cements at 4% provided an on-going cementation reaction up to one year of curing. The other cements and application rates resulted in small initial cementation reactions but no longer-term strength gains. The reasons for these differences between the two materials can probably be attributed to the ICC of the materials. The G6 sandstone had an ICC of 3.5% but the ICC of the dolerite varied between 3 and 6% depending on the cement type.

It appears that there were insufficient reaction components in some of the cements available for continued long-term reactions to continue.



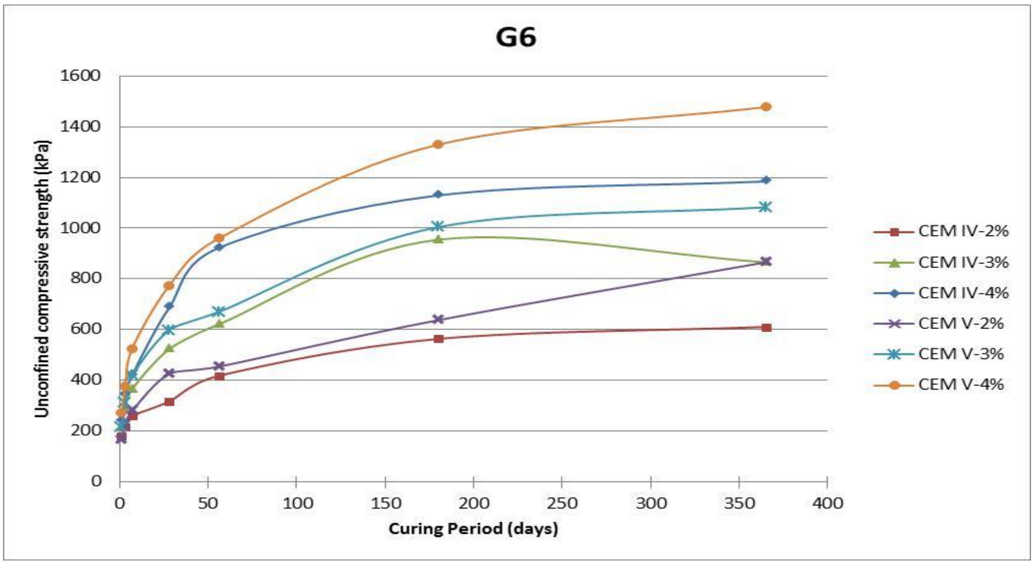
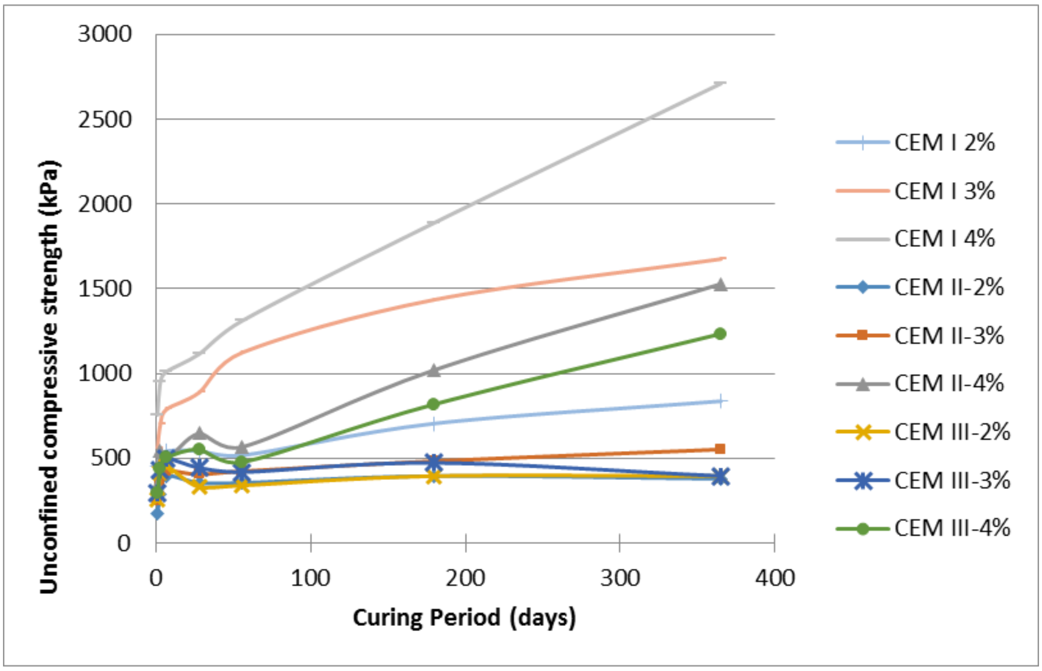


Figure 6: Relationships between unconfined compressive strength and curing period for a G6 sandstone stabilized with various percentages of different cements



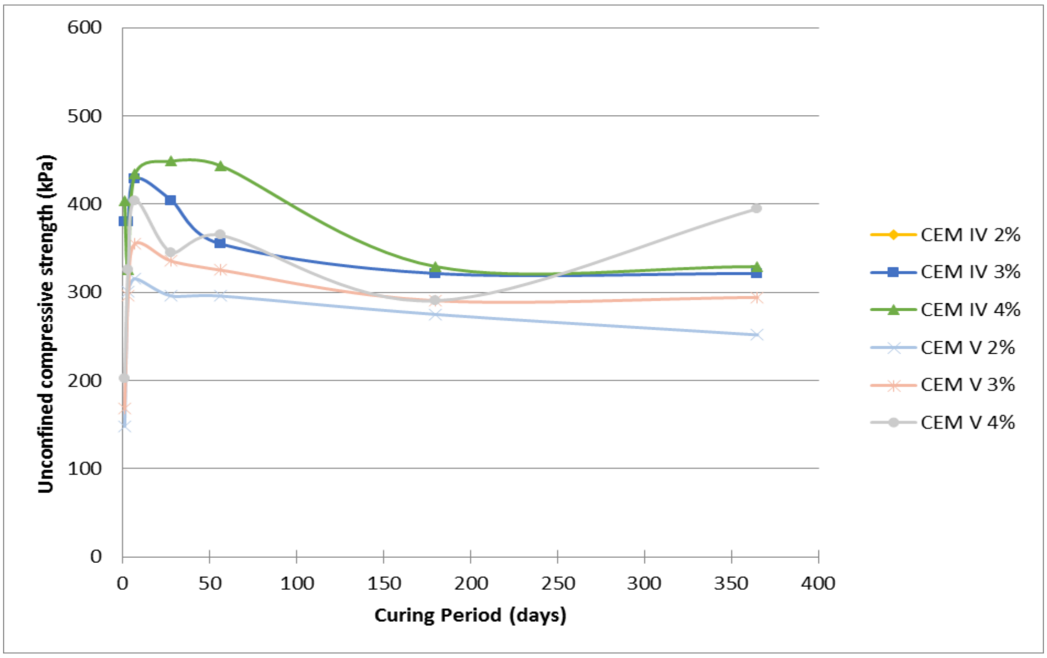


Figure 7: Relationships between unconfined compressive strength and curing period for a G6 dolerite stabilized with various percentages of different cements

The importance of rapid and efficient mixing and compaction and any time-delays on density is illustrated in Figure 4 and has been discussed earlier. The effects of time delays on strength are shown in Figure 8 and Figure 9, which illustrate the losses in strength for different materials with increase in time between mixing and compaction. Reductions of 25 and 30% were obtained on the norite and granite respectively and between 56 and 69% reductions for the dolerite. It is evident from the plot of the dolerite results that there is a wide scatter and not much trend in the first 4 hours after which a severe decrease in strength occurred. The first 4 hours would appear to thus be not that critical in the case of this material.

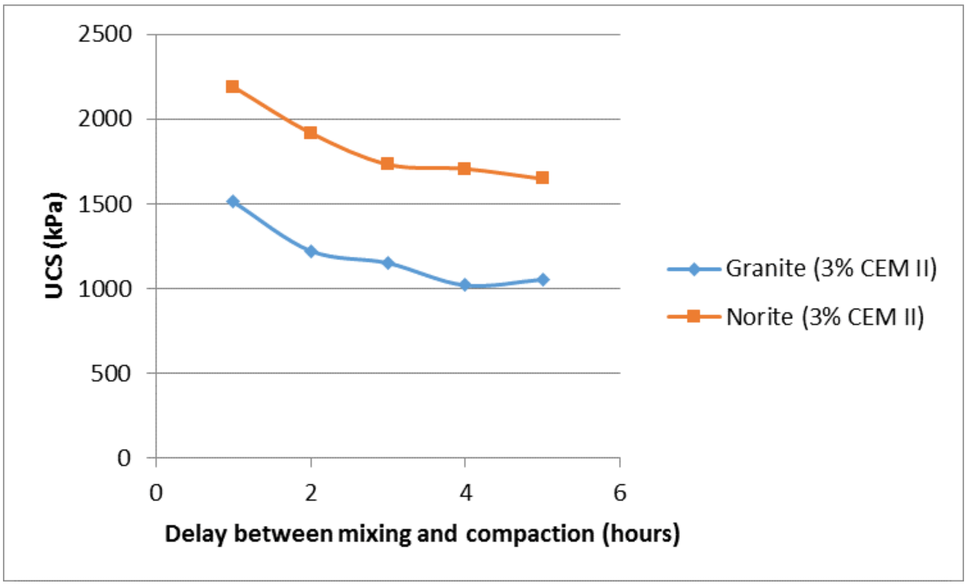


Figure 8: The effect of time lapse between mixing and compaction on unconfined compressive strength of two G5 materials in laboratory testing

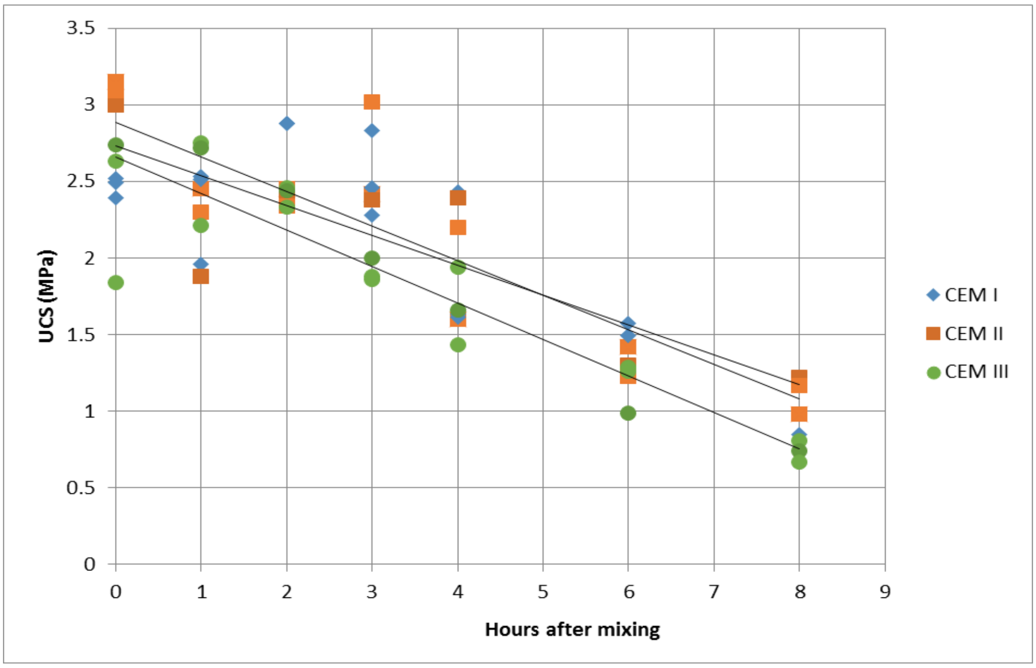


Figure 9: The effect of time lapse between mixing and compaction on unconfined compressive strength of G6 dolerite in laboratory testing

TRH 13 (DoT, 1986b) suggested that the ratio between the 28-day and 7-day strength is between 1.4 and 1.7 and proposes a factor of 1.5 for estimation purposes. From the results shown in Figure 6 it can be seen that provided the ICC of the material is satisfied an average ratio of 1.48 (range 1.22 to 1.63) is obtained. This is similar to the TRH 13 suggestion but will, however, probably depend on the material and the type of cement being used.

In general, the strength increases more or less linearly with cement content but at different rates for different soils (Figure 10). Although the CEM I achieves much higher strengths than the CEM V cement, the trends are similar.

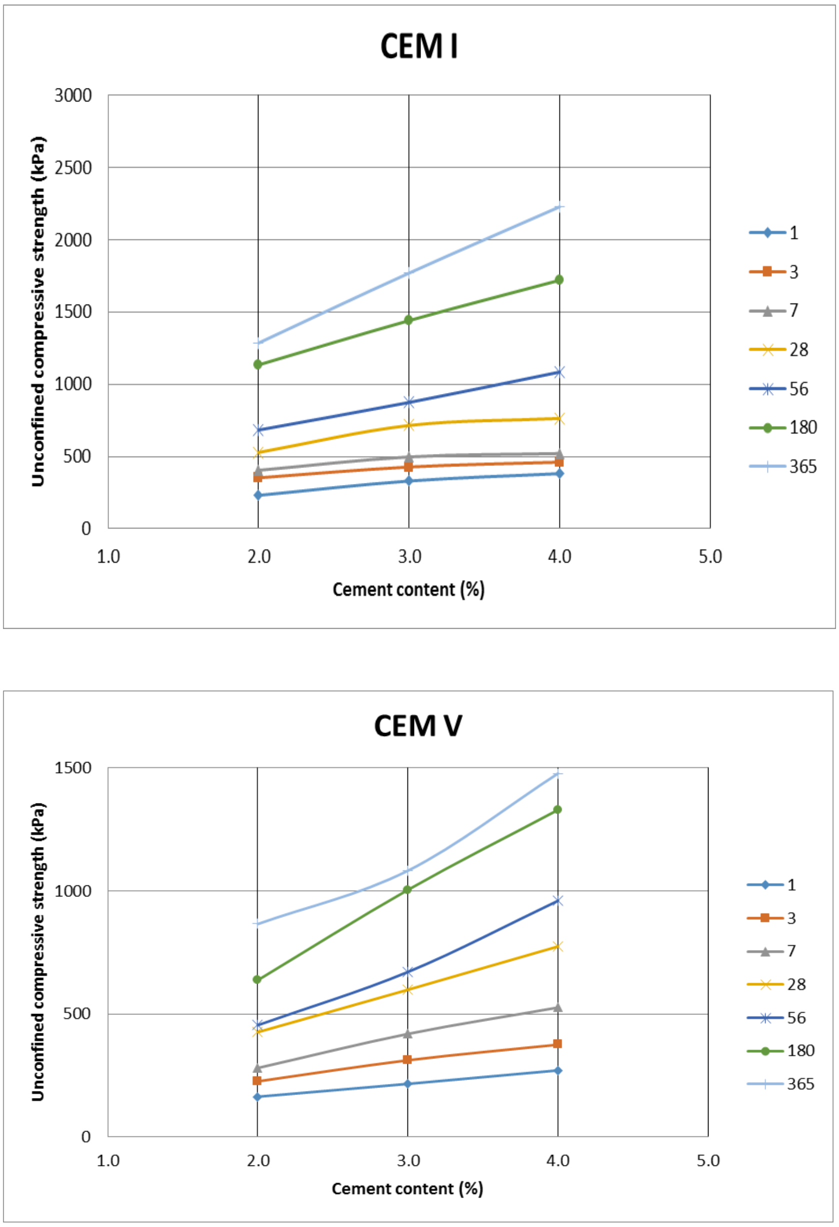


Figure 10: Relationships between unconfined compressive strength and nominal cement content at various times of curing (days) for the G6 sandstone stabilized with CEM I and CEM V cements

The development of strength in cemented materials is also strongly temperature dependent (Figure 11) and this property is used to advantage in accelerated testing – curing at high temperatures can reduce the time required for laboratory testing significantly. However, the ultimate strength is significantly reduced by both mixing and curing at higher temperatures. It is essential to investigate the impact of actual site temperatures on the properties of the stabilized material in the laboratory during construction.

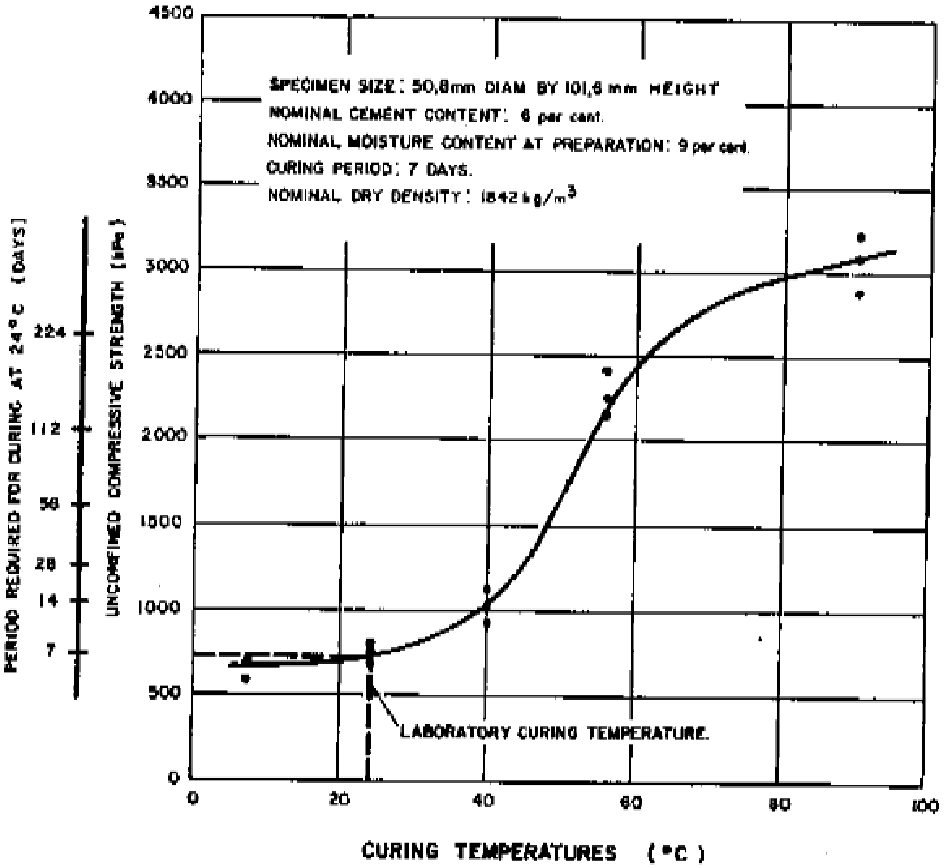


Figure 11: Relationships between unconfined compressive strength and curing temperatures for a sand stabilized with ordinary Portland cement (pre SANS 50197)

Limited work has been done comparing the effect of temperature using 7 different SANS 50197 cements. Figure 12 shows the results of the effect of temperature on the UCS of two materials (norite and dolerite) after 4 hours of conditioning at various temperatures.

It can be seen that although there are a few exceptions (probably experimental error) there is a general decrease in strength with preparation of the material at higher temperatures. This needs to be considered when working in hot arid areas.

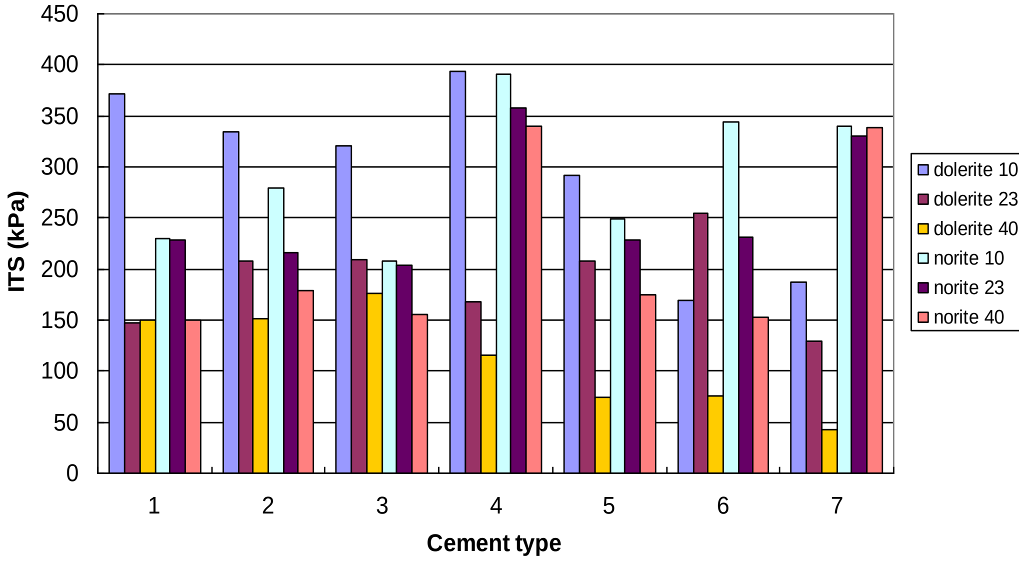


Figure 12: Effect of temperature on UCS of two materials (dolerite and norite)

The factors affecting strength development of lime stabilized materials are similar to those for cement described above. However, the soil type and clay mineralogy in particular are of greater importance. 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 above.

It has also been shown that lime itself may have a plasticity index. Netterberg (2004) found that the addition of lime to certain materials resulted in an increase in the PI. After some investigation, it was found that the lime had a PI of up to 17% which resulted in an increase in the PI of the material being treated after the optimum lime content was passed.

5.4. Stiffness

As the move towards mechanistic empirical (ME) pavement designs accelerates, the need to determine the resilient modulus (stiffness) and Poisson’s ratio is becoming more important.

The Resilient Modulus (M_r) is the relationship between the recoverable (i.e., non-plastic) strain and the load applied and is determined from repeated load triaxial testing. Unfortunately, facilities for this specialised and expensive testing are currently rather limited in South Africa. Poisson’s ratio is the ratio of the lateral to the axial strain and is also required in the ME design process. This is generally difficult to measure accurately and values for analysis purposes are usually assumed. These range between 0.15 and 0.25 for cemented materials and a value of 0.2 for cemented materials is usually used for analysis purposes.

When a cemented material is loaded beyond a certain limit, micro-cracking first develops at the interface between coarse particles and the matrix. The extent of the micro-cracking increases upon subsequent loading and eventually the micro-cracks join up, the matrix disintegrates, and the treated material no longer behaves like a cemented material. Laboratory flexural and compressive tests have indicated that micro-cracking 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 micro-cracking and thereafter it becomes non-linear and non-elastic. A general stress/strain curve for cemented materials can therefore be prepared (Figure 13). 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 material are given in Appendix C.

If a cemented material is subjected to repetitive loading within its elastic range described above, and is not loaded beyond the stress (or strain) at which micro-cracking 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 any overstressing of the layer.

The fatigue properties of cemented materials show semi-brittle behaviour. Once micro-cracking has developed its growth can be rapid, particularly under controlled stress conditions, and when the micro-cracks 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.

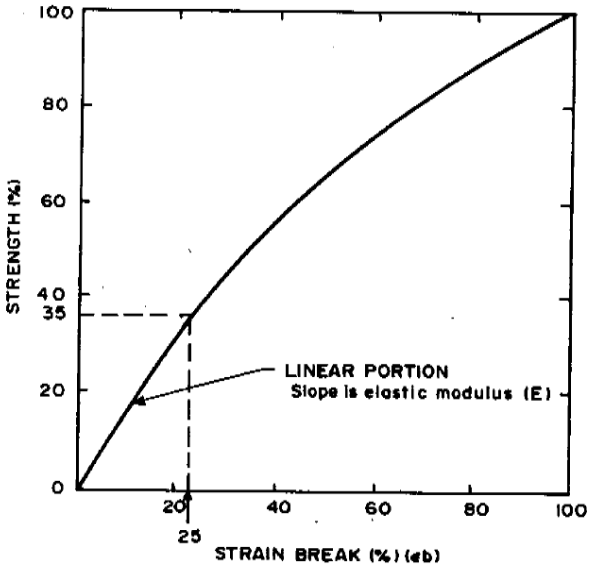


Figure 13: Typical stress/strain relationship of cemented materials

5.5. Cracking

Cracks in cement (and/or lime) stabilized pavement layers cannot be avoided and must be accepted as an essential feature of chemical stabilization. Other than the micro-cracking described above, cemented layers will usually show conventional cracking at some stage in their design life. This cracking allows the ingress of water and air into the layer and supporting layers, which can lead to deterioration through material weakening, erosion and carbonation if not timeously sealed. 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. This cracking can be either non-traffic or traffic associated.

5.5.1 Non-traffic Associated (initial) Cracking

This category of cracking is caused by environmental and material factors. It normally becomes evident within a few weeks or months of construction. It is generally accepted that the root mechanism is shrinkage due to thermal and moisture effects related to hydration of the stabilizer. The severity of the shrinkage is dependent on the:

- Type of stabilizer
- Presence of excessive stabilizer
- Type of material
- Presence of excessive compaction moisture
- Effectiveness of curing

- Wetting and drying cycles affecting curing
- Thermal stresses
- Presence of non-uniform materials
- Presence of black curing membrane (including prime)

In the majority of cases drying shrinkage is the main cause of initial cracking and volume changes due to temperature variations may be regarded as a contributory cause.

(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 (Figure 14). The cracks usually form rectangles or “blocks”. Some cracks appear after just a few days whereas others appear up to four months (or even longer) later. Cracks are usually 3 to 6 metres 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 15). Such cracks will vary in width from fine hair-line cracks to 1 mm wide, 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 15; Material C would not crack until much later).

Stabilization cracks are typically wider at the top than at the bottom and the vertical faces are irregular, which ensures active load transfer.

Figure 14: Typical pattern of stabilization cracking

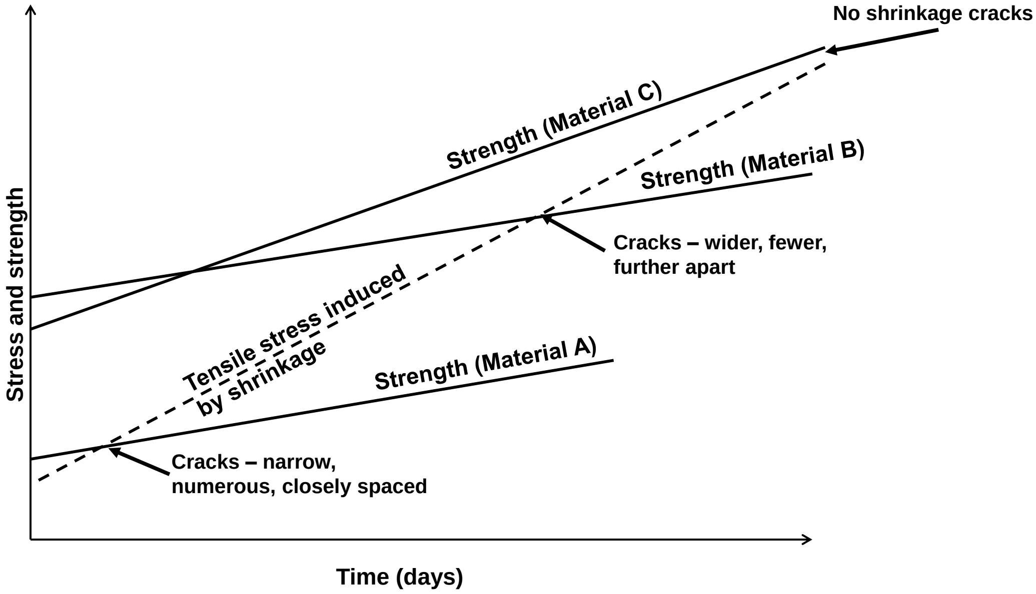


Figure 15: Cracking as a result of the interrelationship between shrinkage stress, strength and time

The use of an overlying unbound material to reduce the presence of cracking in the seal may not always be effective. Figure 16 shows an example of cracking in a stabilized subbase that has “reflected” through a 125 mm thick G1 base course and 25 mm asphalt.

Figure 16: Reflection cracking of stabilized subbase through 150 mm G1 base and 25 mm asphalt

There are several design and construction techniques that have been shown to minimise or eliminate shrinkage problems. These include:

- The compaction moisture content should never exceed 80 per cent of the saturation moisture content of the natural material without stabilizer, calculated at maximum dry density.
- Thorough mixing of the natural soil and of the soil/stabilizer mix should be done to minimise thermal stresses and to limit differential water absorption or drying out of the material mixture
- Thorough curing of the mixture, including the prevention of wetting and drying cycles or even just drying must be ensured
- The maximum quantity of cement-based stabilizers should be limited to 3.5 % although this depends on the cement type and strength class. In the case of lime the maximum should be 4 %
- The addition of certain proprietary additives can reduce cracking but the effects of these need to be ascertained on each material prior to use

Certain materials such as weathered sandstones, granites, quartzite and chert with low plasticity's are more susceptible to cracking when stabilized than other materials. In all materials the tendency to crack increases as the percentage stabilizer increases. This type of cracking (block) is, however, a normal occurrence for stabilized materials. This can be limited by using stabilizers with slower initial strength gain.

(b) Cracking due to thermal stress

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 design purposes.

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 changes and stresses.

5.5.2 Traffic-associated Cracking

Cracks associated with traffic are caused by traffic overstressing the cemented layer. Traffic-associated cracking may occur in a cemented layer in any position in the pavement structure, 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 overstressing, connect to form circular or square blocks (see Figure 17).

Figure 17: Typical cracking due to overstressing

The cracks become progressively more numerous and the blocks become smaller, eventually ranging from 50 mm to 1 m in diameter (crocodile cracking as shown in Figure 18). 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” and is also evident in Figure 18.

Figure 18: Typical crocodile cracking and pumping associated with overstressing of a stabilized base

Overstressing of the stabilized layer can occur in at least four ways:

- Construction vehicles travelling on the stabilized layer before it has developed sufficient strength or before placing a sufficiently thick protective layer over the newly stabilized layer. However, pre-cracking by judicious rolling with the appropriate roller at the right time can sometimes be advantageous if done carefully.
- Unbalanced pavement design, especially when the base is too stiff relative to the subbase or overall support, i.e., the modular ratio of the base to subbase is too high.
- Not providing sufficient cover to the stabilized layer to withstand the long-term traffic loading. Overloaded vehicles cause high strains in the stabilized material, which is then more susceptible to failure than natural gravel layers.
- General fatigue failure of the stabilized layer as a result of repetitive deflection under traffic.

In the past many design problems were experienced with cement treated pavement layers, mainly because the layers were too thin and stiff and inadequately supported and also because of excessive shrinkage. Such cement treated layers are very sensitive to overloading and generally exhibit a value of n of about 6 in the load equivalency formula below:

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

where W = axle load in kN

n = exponent

Initial cracking can also develop in lime treated materials and the cracks form rectangles, like the cracks in cement stabilized materials. The shrinkage characteristics and rate of strength gain 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, lime treatment of materials with high amorphous silica contents (e.g., some calcretes and sandstones) may behave in a similar way to cement stabilization and crack as badly as an equivalent cement stabilized material.

It is also believed that cracks in lime-treated materials may sometimes exhibit “self (autogenous) healing” properties, but it must be stressed that this has not been proved for lime treated materials as yet.

5.6. Behaviour of Stabilized Materials in Service

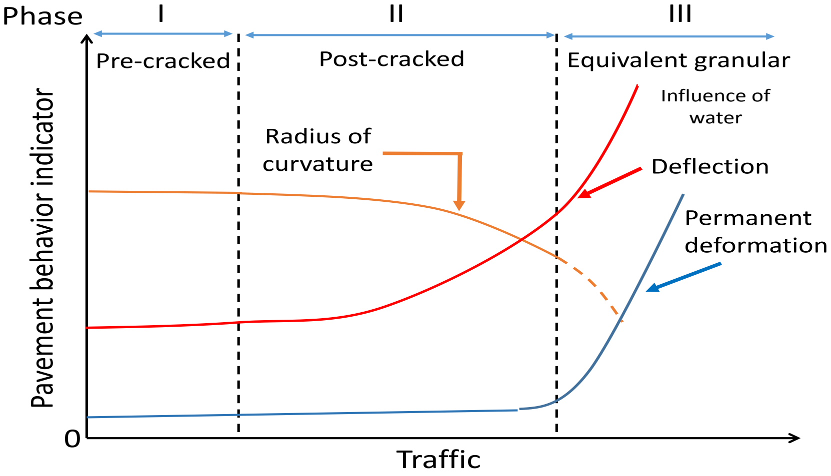
5.6.1 Traffic associated crack progression

As discussed previously, cemented materials are relatively brittle and if loaded beyond a certain limit will develop microcracks. This has an important influence on the behaviour of stabilized layers in flexible pavements. Microcracking is obviously related directly to traffic loads on the layer and dictates the ultimate road performance.

Every time a load is placed on the stabilized layer by traffic, the layer deflects (strains) infinitesimally. Any excessive strain will result in the development of microcracks, which eventually coalesce, become more numerous and result in small blocks of cemented material instead of a continuous layer. The size of the blocks continues to diminish with time until they may be as small as 50 mm in length or diameter. Ultimately the properties of the stabilized material will revert almost to the properties of the original unstabilized material (the plasticity will generally remain lower if the material was adequately stabilized). These three phases in the life of a stabilized material are termed the *Pre-cracked, Post-cracked* and *Equivalent Granular* phases and need to be taken into account in the analysis of long-term pavement performance.

It should be noted that the term Pre-cracked relates only to traffic associated cracking (mostly crocodile). Pre-cracked layers can still have substantial non-traffic related cracking and exhibit significant block cracking but do not contain any microcracks associated with traffic loading.

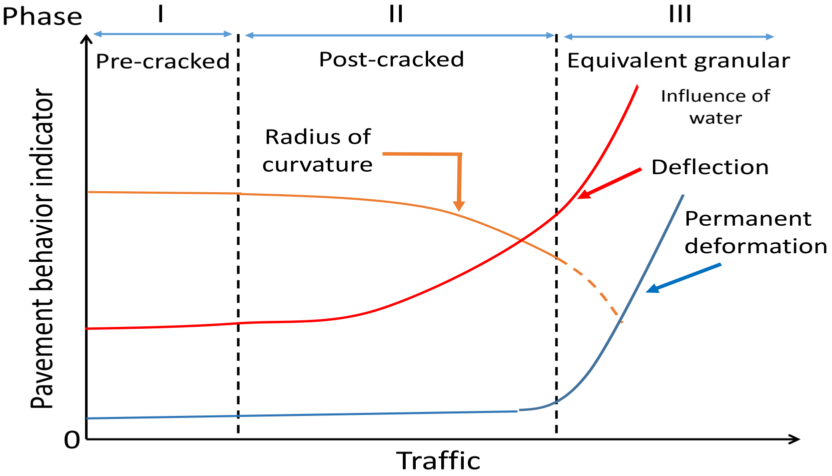
It has been shown that in its Post-cracked phase a stabilized subbase material substantially to the useful life of a pavement. Measurements of deflections at depths within the pavement have demonstrated that the initial effective modulus material is of the order of 3 000 to 4 000 MPa as shown in



a) Change in indicators of performance of pavement behaviour with traffic

Figure 19a. The performance of stabilized materials in the base course appears to follow similar trends, although the absolute moduli and duration of the different phases can differ significantly.

The relatively rigid stabilized layers generally suffer fatigue under traffic, or in even under construction vehicles, and assume lower effective moduli (500 to This change in modulus does not normally result in a marked increase in changes in resilient deflection and radius of curvature are observed



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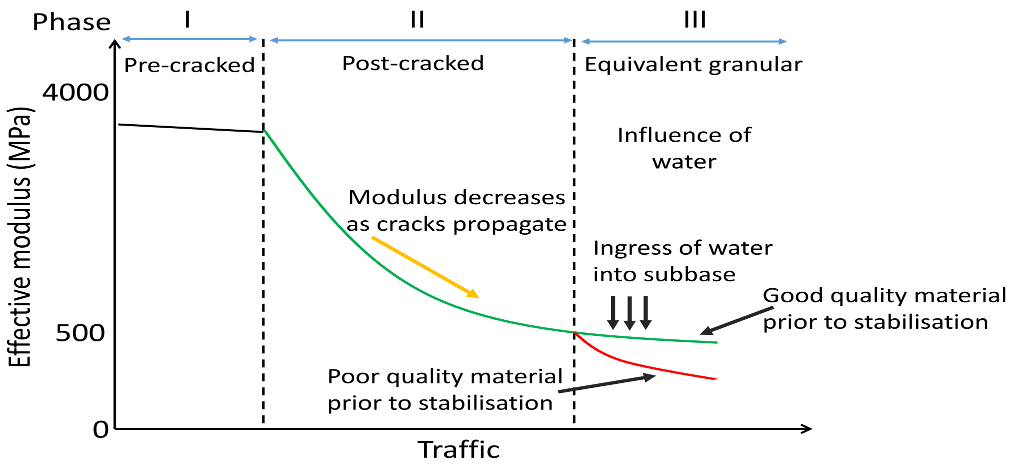
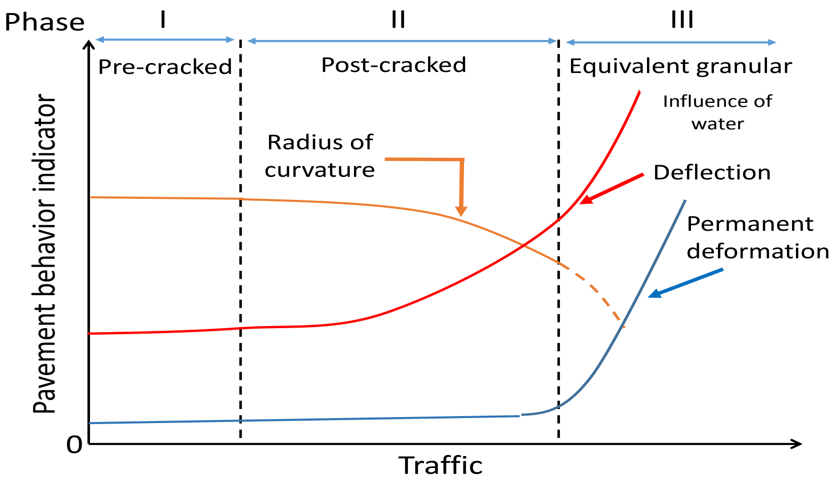
b) Change in indicators of performance of pavement behaviour with traffic

Figure 19b).



c) History of cemented material modulus as a function of traffic loading



d) Change in indicators of performance of pavement behaviour with traffic

Figure 19: Cemented subbase behaviour with traffic

In the mechanistic design procedure, these phases have been designated the pre-cracked and post-cracked phases and the design accommodates their corresponding changes in modulus.

The eventual modulus of the stabilized layer (Equivalent Granular phase) 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. Figure 20 shows the cracking exposed in the top of an old (> 30 years) cement stabilized base which was in an equivalent granular state on exposure. 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, while in other cases the modulus will reduce to between 50 and 200 MPa. This change is also shown diagrammatically in Figure 19a. The net result is that the modulus of the cemented material decreases to very low values which lead to deformation and failure. Cracking of the surface generally occurs and the ingress of water results in pumping and potholing.

Figure 20: Cracking of an old stabilized layer seen during rehabilitation (equivalent granular phase)

For high quality, heavily trafficked and strategic roads it is important to avoid stabilizing marginal materials that will eventually deteriorate to a low modulus. Many of these marginal quality materials have, however, proved to be adequate for lower class traffic.

The rate at which the stabilized material moves through the phases is a function of the initial strength of the material, the environmental conditions and the magnitudes of the traffic loading.

5.6.2 Surface crushing

Another aspect regarding the behaviour of stabilized materials is that of crushing or compression failure near the top of stabilized base layers (De Beer, 1989b: Litwinowicz and de Beer, 2013). The direct contact of tyres with thin bituminous surfacings allows little dissipation of the contact stresses between the tyre and the top of any stabilized base layer. It is thus possible that stresses in excess of 1000 kPa will affect the top of the stabilized base and if the strength of the stabilized material is insufficient, crushing of the material this contact area may occur. This will be exacerbated should there have been any carbonation of the upper portion of the base prior to sealing.

The relationship between tyre contact stresses and the strength of the upper portion of the base under repetitive loading is used to determine the number of repetitions before crushing failure occurs. The original models were based on HVS test results that were saturated with water to accelerate failure. These have subsequently been re-modelled under non-saturated (as-built) moisture conditions to provide more realistic predictions and are directly related to the stress ratios (i.e. ratio of applied load (vertical tyre contact stress) to in situ UCS of material). Various models have been developed (Litwinowicz and de Beer, 2013):

Crush initiation (N_{ci}) = $10^{8.2218(1 - (SR/1.245))}$
Advanced crushing (5 mm rut) $N_{ca5} = 10^{8.0160(1 - (SR/1.6233)}$
Advanced crushing (10 mm rut) $N_{ca10} = 10^{8.1759(1 - (SR/1.7984)}$
Advanced crushing (15 mm rut) $N_{ca15} = 10^{8.0614(1 - (SR/1.9785)}$
Advanced crushing (20 mm rut) $N_{ca20} = 10^{7.9941(1 - (SR/2.1410)}$

It is clear that the weaker the upper part of the base is, the more likely the stress ratio is to increase and crushing of the upper base is likely to occur. The effects of carbonation on the upper base resulting from poor curing will result directly in premature crushing as discussed later.

5.6.3 Weak interlayers

Another important issue that has recently been identified is the influence of the weak interlayers in stabilized materials (Netterberg and De Beer, 2012; De Beer et al, 2012) on the structural capacity of the pavement.

Netterberg and De Beer ((2012) have shown that a weak interlayer between the base and the surfacing (the typical result of carbonation of the upper surface caused by poor curing during construction) has a severe impact on the structural capacity of the pavement (Figure 21). The effect is exacerbated if the underlying layer remains strong.

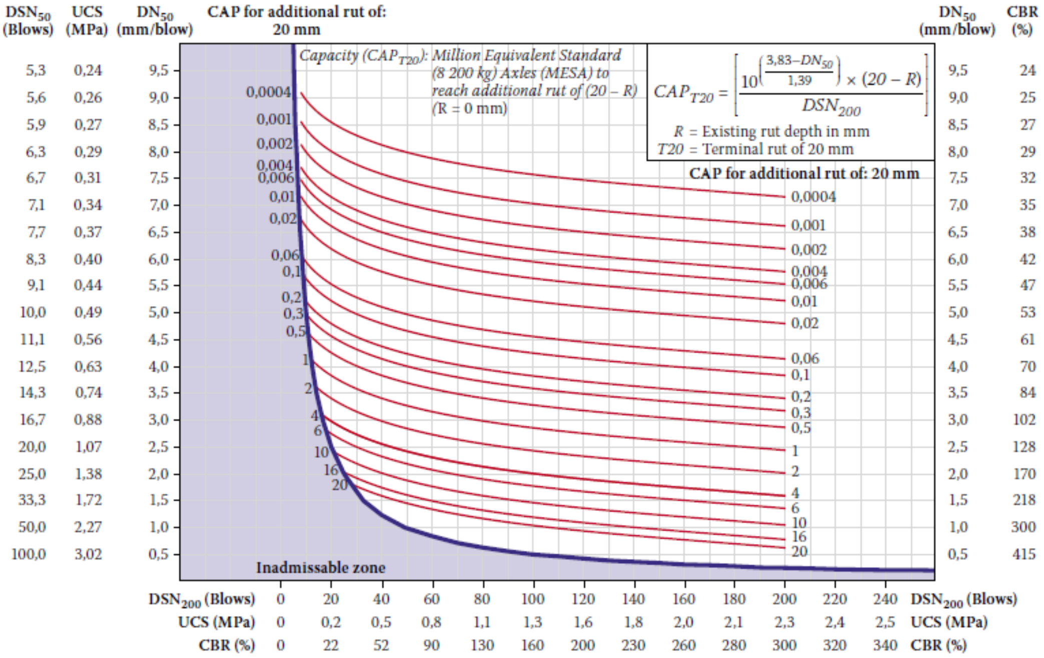


Figure 21: Nomogram showing estimation of structural capacity of weakly cemented pavements (Netterberg and de Beer, 2012)

As an example, if the strength of the upper 50 mm of a 200 mm thick C3 layer (UCS = 1500 kPa) is reduced to 700 kPa, the carrying capacity will decrease from 5.2 million equivalent standard axles (MESA) to about 0.3 MESA (Netterberg and de Beer, 2012). They also note that strengths as low as 300 to 500 kPa have been recorded in the upper 50 to 75 mm of stabilized roads within months after construction.

Further analyses investigating the effects of weak interlayers within pavement structures (carbonation at the top of a stabilized subbase beneath a granular base would be an example) have also been modelled by De Beer et al, (2012) and have been shown to have a severe impact on the structural capacity of a pavement.

6. DESIGN OF STABILIZED LAYERS

6.1. Introduction

The stabilization design process involves the following procedures:

i) Sampling and laboratory testing of the borrow or in-situ material to be used
ii) Determination of the required structural properties of the stabilized layer i.e., modification or cementation
iii) Initial proposal of types and quantities of stabilizer required
iv) Checking the proposal through appropriate laboratory tests leading to acceptance or adjustment of (iii) above, in accordance with the results achieved.

The flow diagram shown in Figure 22 gives this process in more detail and can be used to ensure that all of the necessary steps in the stabilization design are followed. It should be noted that this procedure is specifically for the design of the stabilized materials and differs significantly from the pavement design procedure, which is briefly discussed in Section 6.6.

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 sections, the main consideration being: design requirements, laboratory test results, site conditions, availability of stabilizers at the place and time of construction and economics.

6.2. Initial Sampling and Testing of Material

It is essential that the samples collected for testing are representative of the material to be used during construction. The number of samples tested is dependent on the variability of the source. Details are discussed in TMH5 (DoT, 1981). Each sample should consist of sufficient material to allow a full range of natural and stabilized soil tests to be performed. Approximately 150 kg is usually sufficient, but this needs to be increased for materials with a high percentage of coarse aggregate that will be discarded.

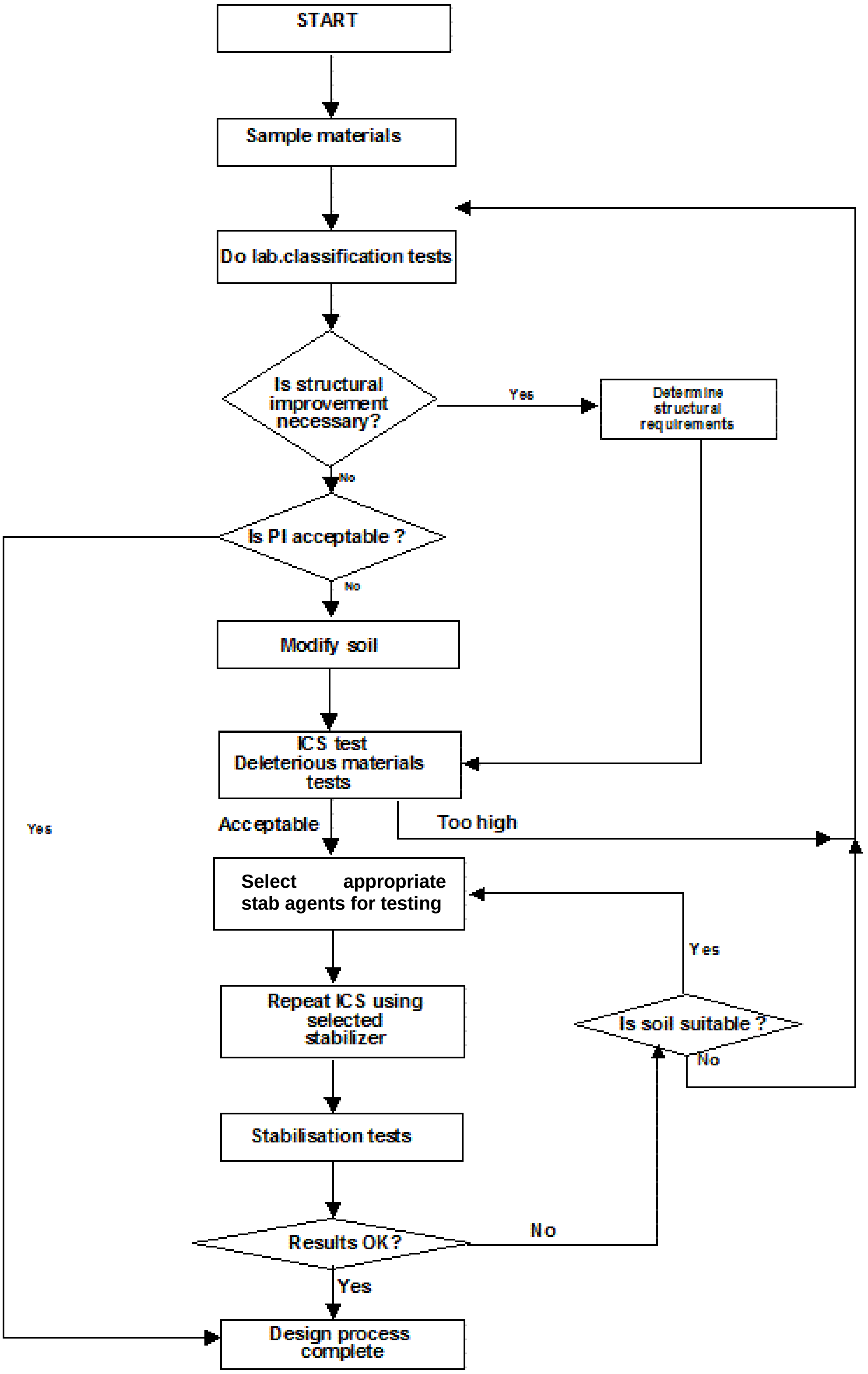


Figure 22: Stabilization design process

Initial testing of the material entails establishing the grading, plasticity and CBR of the natural soil. This provides sufficient information to determine whether the soil requires structural improvement or modification of the PI. Should the natural material require stabilization, ICS testing using lime should be carried out to determine whether chemical stabilization would be cost-effective? Typically, a high ICS indicates that excessive stabilizer is probably required: if cost analyses show that this is not cost-effective, alternative materials should be sought.

It is important that the natural material to be treated is of suitable quality to produce an appropriate cemented layer. Materials of highly variable quality are extremely difficult to stabilize and construct effectively – variability in the required stabilizer content and the compaction moisture can result in localised areas of poor quality. It is suggested that material of at least upper selected subgrade quality for the appropriate pavement class be selected for use in stabilized layers. Material inferior to this is more likely to be variable, contain deleterious components and require higher stabilizer contents. Typical deleterious components include:

- sulphates;
- soluble salts;
- sulphuric acid;
- secondary minerals, especially smectite clays, and
- organic compounds.

With the increasing implementation of cold in-place recycling, the stabilization of existing pavement materials is necessary. Although these layers should generally be fairly uniform (the original pavement should have been constructed to strict standards), the effects of patching and repairs, variable thicknesses of layers (different ratios of base and subbase may be recycled) and variable addition of "top-up" material can result in high variabilities (Paige-Green and Ware, 2006).

As the materials selected for stabilization will eventually revert to their equivalent granular state, the best possible materials only should be chosen. While certain authorities limit the quality to at least G6, others require at least a G5 material, in order to ensure that the equivalent granular state still provides sufficient strength (i.e. a CBR > 45%).

6.3. Pavement Strength Requirement: Cementation or Modification

The decision on whether to modify or cement or both depends on the function of the pavement layer in which the material is to be used. Stabilized base and subbase layers must have adequate strengths and use materials that generally contain some coarse aggregate. Cementation is used to achieve the required strengths, whilst modification may sometimes be used purely to reduce plasticity and/or improve workability. In the case of lime stabilization, higher quantities of lime may be required to ensure that cementation products can develop.

The practice of modifying a material and then cementing it, i.e., a double handling process, is a costly operation and the economics of the process should be carefully assessed before commencing such an operation. It can, however, in certain situations be the only option, e.g., on norites north of Pretoria and dolerites east of Benoni. On some recent recycling projects the use of pre-treatment with 1% lime followed by stabilization with 2 or 2.5% cement has been found to simplify construction, improve workability and produce the necessary tensile strength that the use of cement only was not developing.

Strengths specified for selected layers can usually be achieved using un-cemented finer grained materials with PI’s limited to acceptable levels. Modification of otherwise acceptable materials with marginal PI's, especially in base materials, would reduce the PI as well as improve the strength of that material. This strength gain, given similar compactive efforts, is normally not due to cementation, but results from the improved structure and increased density of the soil. It should be ensured that the strength of the natural material meets the required specification, as it is possible that the PI may return with time. The permanence of modification has still not been proved in practice.

Roadbed materials are not stringently specified, but in-situ materials can often have sub-standard strengths or high clay contents. The roadbed material can also be in a saturated condition during construction and require drying. In these cases modification could be (but is seldom) used to:

- Raise the CBR by improving the structure of the soil
- Reduce shrinkage and swell in clayey materials by reducing the plasticity
- Improve the consistency and workability of the material by the apparent drying effect of stabilizers

6.4. Initial Stabilization Proposal

6.4.1 Selecting the Stabilizer

When selecting a stabilizer, the following factors need to be considered:

- The physical properties of the material (e.g. liquid limit, plasticity index, clay content, ICS, etc.)
- The purpose of stabilization (e.g. for modification or cementation; for a rapid or slow increase in strength)
- The availability, costs and effectiveness of different stabilizers. Cost alone should, however, not dictate the use of a potentially inferior agent over a better but more expensive one.

The following can be used as a first guide to choosing a stabilizer:

Plasticity Index of the Material

Materials with a PI of about 10% or less (low to medium plasticity) are usually stabilized with cement. The modification reactions from the cement typically reduce the PI to less than six for these materials.

Materials with a PI of between 10 and 15% need modification as well as cementation. Lime or lime-cement mixtures should be evaluated.

Materials with a PI greater than 15% usually require both modification and strength improvement through cementation and contain sufficient clay to react with the lime. These materials would normally not meet the requirements for upper selected layers and should only be used if better material cannot be located. Lime is the recommended stabilizer but cements containing extenders can be successfully used after pre-treatment with lime. Lime should be used to stabilize basic crystalline materials at these PI levels.

Despite the above, many natural soils with low P'Is can react successfully with lime only as discussed previously. For this reason, various stabilizers should be tested for most materials.

Heavy clays or wet materials can be modified for improved workability and compactability. Under these circumstances lime is usually used as the modifying agent. Materials that contain kaolinite as the predominant clay mineral usually have a fairly low PI with a high liquid limit. These materials should also be stabilized with lime.

After stabilization of materials derived from basic crystalline rocks, it is important that the PI is reduced to "non plastic" (NP), and not only "slightly plastic" (SP).

Activity of the Material

In addition to the various physical properties of the material, Savage and Visser (1995) consider it beneficial to consider the activity of the material in the selection of a stabilizer.

The activity is defined as follows:

$$A = \frac{PI_{gross}}{P_{0.002}}$$

Where:

PI_{gross} = Weighted plasticity index of the sample (PI x P0.425)
P0.425 = Percentage passing 0.425 mm
P0.002 = Percentage smaller than 0.002 mm

When A > 0.5, modification is necessary and lime should be considered.
When A < 0.5, modification may not be necessary and cement, cement-lime blends should be considered.

Grading

Table 7 serves as a guide to the identification of appropriate stabilizers for use in laboratory testing based on the material classification. It should be borne in mind that material of poorer quality than G5 should only be treated for use as base or subbase where no other material is available and preferably only for low volume roads.

Table 7 : Suitable stabilizers for various soils

Grading	TRH14 Class	Requirement	Application	Stabilizer
Fine	G8, G9 G10	Reduce plasticity	Modification	Lime
Coarse	G4, G5, G6, G7	Increase strength	Cementation	Cement, cement blends, lime blends

Fine single-sized windblown or dune sands can be stabilized with cement but high cement contents are usually necessary to obtain the compressive and tensile strengths required for pavement layers. This may result in excessive shrinkage (block) cracking.

Initial Consumption of Stabilizer (ICS)

Where the lime demand of a material is found to be high (see Section 6.2), lime and lime blends with GGBS or fly ash could be considered. This is particularly relevant to soils derived from basic crystalline rocks. Where the use of cement is proposed, the ICS should be determined using the type of cement that would eventually be used on site.

Selection of cement type

The performance of cements complying with SANS 50197-1 appears to differ from those complying with the older SABS 471, 626, etc. The test methods are different and the strength classes of cements are now based on the strength after 28 days and not 7 days as previously.

A preliminary investigation (Paige-Green and Netterberg, 2004) carried out on behalf of the cement producers through the Cement and Concrete Institute (C&CI) has indicated that CEM II A and B cements using flyash or granulated blast furnace cement as extenders and CEM III A cements appear to allow greater flexibility during construction. The strength class of the cement should not exceed 32.5 (N and not R cements should be used). It is recommended, however, that preliminary testing using a range of cements within the boundaries described should be carried out during the stabilization design process.

As not all cements are produced country-wide and the behaviour of the cements from different suppliers can differ significantly, only those cements likely to be available on site during construction should be used for the stabilization design. It has also been noted that the duration between the pavement design and the actual construction can often be several months, during which time the cement properties may have changed. A new stabilization design with the new cement must be carried out in these cases. It must be remembered that the cement composition (Table 1) has quite wide limits (mostly up to 14%) and the manufacturers are free to adjust the composition within these limits, provided that the overall chemical compositions and strength classes are achieved.

Table 8: Tentative guidelines for the selection of stabilizer partially based on Gautrans experience

Rock type	PI	Suggested stabilizer type	Comments
General			• Low PI: 8% or less; High PI: 12% or more. • The PI range between 8 and 12% is a grey area and stabilizers for both "Low" and "High" can be considered. • Lime is normally used in subgrade layers, where modification is the priority • Cement should have a maximum strength class of 32.5 MPa and be of N type
Basic Crystalline Dolerite Andesite Diabase Norite Gabbro	Low High	Cement with strength class 32.5N Lime and GGBS blend should be considered where early strength is required	• ICS test essential. • Pre-treated soils should be stabilized with CEM II A or B rather than CEM III A • If the stabilizer does not reduce the PI rapidly, the materials should be allowed to condition for a longer period before testing
Acid Crystalline Granite Felsite Syenite	Low High	Cement with strength class 32.5N Lime and GGBS blend should be considered where early strength is required	• Cracking could be a problem with cement, especially with weathered granites
Arenaceous Sandstone Gritstone Conglomerate	N/A	Cement with strength class 32.5N Lime and GGBS blend can be considered	• Cracking could be a problem with sandstones
Argillaceous Mudrocks/Shales	Low High	Cement with strength class 32.5N Lime or lime and GGBS blend	
Pedogenic Ferricrete	Low High	Cement with strength class 32.5N Lime or lime and GGBS blend	
High Silica Chert Quartzite/Quartz	Low High	Cement with strength class 32.5N Lime or lime and GGBS blend or cement	• Cracks could form on non-plastic materials • Magaliesberg Quartzite can present problems when stabilized.

6.4.2 Quantity of Stabilizer

The quantity of stabilizer should be based on that required to achieve the specified standard for the pavement layer. This is probably the most important component of the design that ensures durability. Insufficient stabilizer will result in marginal strengths and an increased possibility of the pH of the material dropping and the cementation products of the material becoming unstable.

Strength tests (UCS and ITS) are carried out on the material at different stabilizer contents. The stabilizer content versus the strength curve indicates the quantity of stabilizer required to produce design strengths according to the national standard (COLTO, 1998) (Table 9).

Table 9: National requirements for strength and plasticity of chemically stabilized materials (modified after COLTO, 1998)

Criteria	C1	C2	C3	C4
Material classification before treatment	At least G2	At least G4	At least G5	At least G6
PI after treatment	SP	SP	≤ 6%	
Design strength (MPa)				
Lab UCS				
@ 100% Mod AASHTO	Min 6	Min 3	Min 1.5	Min 0.75
	Max 12	Max 6	Max 3	Max 1.5
@ 97 % Mod AASHTO	Min 4	Min 2	Min 1	Min 0.5
	Max 6	Max 4	Max 2	Max 1
Indirect tensile strength @ 100% Mod AASHTO (kPa)			Min 250	Min 200

Experience has shown that it is more important that for materials with strengths less than 1.5 MPa, the minimum tensile strength is achieved rather than the UCS. Problems associated with loss of stabilization are generally related to materials that do not achieve the specified ITS's and when the strains associated with any volume changes in stabilized materials exceed the tensile strength of the material, cracking is likely to occur (Paige-Green, 1991).

In fact Figure 15 is probably better depicted as shown in Figure 23.

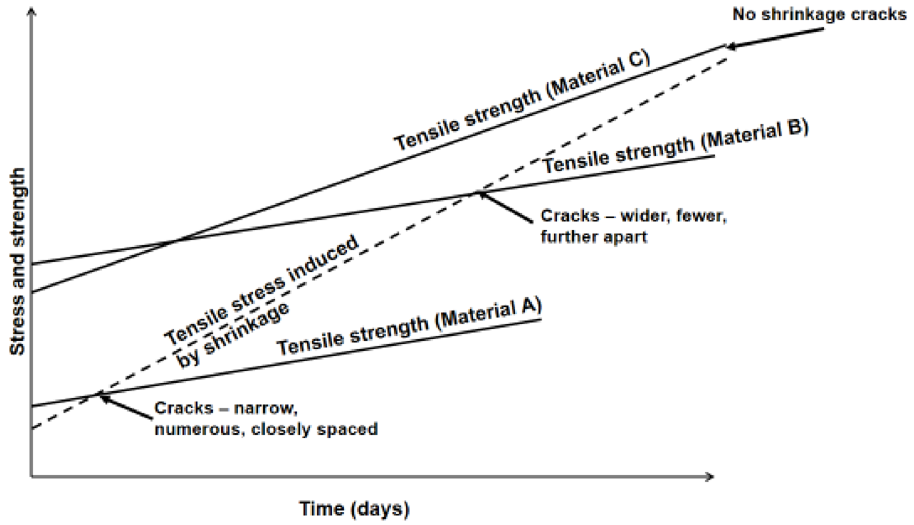


Figure 23 : Relationship between tensile strength and tensile stresses leading to cracking

A maximum limit is placed on all of the UCS values in Table 9. This was originally recommended to minimise cracking. In many instances, the maximum recommended UCS has been exceeded and yet problems unrelated to cracking (e.g. weakening and carbonation of the top of the layer) have been noted. In these cases, the specified tensile strength was invariably not met. On this basis, it is recommended that achieving the specified ITS should take precedence over exceeding the maximum UCS and the maxima be dropped (Table 10).

Table 10: Recommended requirements for strength, plasticity and durability of chemically stabilized materials

Criteria	C1	C2	C3	C4
Material classification before treatment	At least G2	At least G4	At least G5	
PI after treatment	SP	SP	≤ 6%	
Minimum design strength (MPa) Lab UCS @ 100% Mod AASHTO @ 97	 6.0 4.0	 3.0 2.0	 1.5 1.0	 0.75 0.5
Indirect tensile strength @ 100% Mod AASHTO (kPa)			Min 250	Min 200
Wet dry brushing loss (%) – (SANS 3001-GR55)	5	10	20	30

Investigation of numerous stabilized materials has shown that the strength of stabilized materials (in terms of the UCS) is not synonymous with durability (Netterberg, 1987). Materials that easily achieve the required UCS can deteriorate in service. It is thus essential that adequate durability be achieved during the design phase and the conditions that the materials are likely to be exposed to in service are simulated in the laboratory to give an indication of likely performance. The methodologies for these are described in Chapter 4.

It has, however, been proposed that in order to ensure adequate durability, the stabilizer content should at least exceed the ICS by one per cent (Paige-Green et al, 1990). It is also suggested that the stabilizer content is further increased appropriately to account for variations in spreading, mixing and material properties.

For subbase layers beneath rigid pavements, erosion resistance as well as conventional durability is required. For these materials, UCS, ITS, wet/dry brushing and erosion tests should be carried out as described in Chapter 4. These tests should be done at different stabilizer contents to determine the optimum cement content. ICS testing should also be performed to determine the lower limit for the stabilizer content.

This basic principle for determining the design stabilizer content should be followed for any stabilizer using the proposed stabilization agent for all tests.

Where mechanistic analysis is to be used in the design process, values for the E-modulus or stiffness need to be determined. These are best determined using repeated triaxial testing where protocols for this testing are currently being finalised by SANRAL. However, suggested values have been published (Theyse et al, 1996) and are summarised in Table 11.

Table 11: Suggested elastic moduli values for cemented material

Code	Pre-cracked condition		Post-cracked condition		
	Phase 1		Phase 2	Phase 3	
	Stage 1: Intact (GPa)	Stage 2: Shrinkage cracking (MPa)	Stage 3: Traffic and micro cracks (MPa)	Stage 4: Equivalent granular state (MPa)	
				Dry	Wet
C1	6 - 30	2500 – 3000	800 - 1000	400-600	50-400
C2	3 - 14	2000 – 2500	500 - 800	300-500	30-500
C3	2 - 10	1000 – 2000	500 - 800	200-400	20-200
C4	0.5 - 7	500 – 2000	400 - 600	100-300	20-200

6.5. Laboratory Testing of Stabilization Proposal

This chapter has discussed the considerations that influence the choice of stabilizer. The final choice of type and amount of agent that will satisfy the design requirements can only be made after laboratory tests have been carried out on the material with the proposed type and quantity of stabilizer. This has been part of the process of the stabilization design and permits the estimation of project costs and preparation of tender documents. However, once the project has commenced, the actual material to be used and stabilizer delivered to site should be comprehensively tested to confirm the original design.

It is often advisable to test with two different stabilizers, selected from those with the optimum results in the pre-design testing, based on strength, durability and the effect of time and temperature on density and strength. The types of cement selected for testing need to represent the availability of cements to the project. Laboratory tests using two or three different stabilizer quantities should also be carried out to confirm that the correct decision in this regard has been made. Large relative differences between different materials in their reaction to stabilizers may be found. Stabilizers should therefore not be changed during construction without repeating the necessary tests. Through the testing of the stabilizer-soil mixture, problem soils will be identified before they are used on the road.

It should be noted that in recent years, the use of lime stabilization has decreased significantly. The primary reason for this is probably that the cost of lime became higher than that of cement. There are, however, certain advantages of using lime that could be considered. Apart from the obvious use to improve the workability of clayey materials (and dry out wet clays) the lime content and costs are based on mass and since the density of lime is about half that of cement, the larger volume of lime is easier to mix than an equal mass of cement. The working time of lime is also usually considerably longer than that of cement, and where construction operations will necessitate longer processing times, lime could be considered. This is not a benefit where high early strengths are developed as a result of high amorphous silica contents in the soil, for instance.

Treatment with lime can therefore offer certain advantages over cement. 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.

6.6. Structural design

This document is not a structural design guide but a number of issues relating the stabilization design to the structural design are relevant and need to be considered. As the mechanistic empirical design process has developed over time, some changes to the stabilization design philosophy have evolved (Theyse, pers comm 2013).

The incorporation of a stabilized (lightly cemented) layer into the pavement structure is part of the design process and is a conscious decision made by the design engineer. The use of cemented bases in pavements will typically be based on the absence of sufficient materials of suitable quality for granular base construction. A cemented base would not normally be placed on an un-cemented subbase except for low volume roads or where there is a particularly strong subgrade. Cemented subbases are commonly used in many pavement designs, often mainly to provide a suitable platform on which to compact the overlying granular material or to improve the quality of the local materials. Such pavements are generally termed inverted, as the generally accepted gradual decrease in strength of the layers with depth is not achieved in such pavements. Cemented subbases are also invariably utilised in concrete and block-paved pavements.

The use of lightly cemented bases results in a need to ensure that maintenance is timeous and effective. Such pavement structures initially show little distress, but deteriorate rapidly once distress initiates. Remedial action on these pavements is therefore urgent, once signs of distress are noticed.

The South African Pavement Engineering Manual (SAPEM, 2014) has a useful table of where cemented layers can be successfully used in pavements, related to Road Category and traffic class (Table 12).

Table 12: Recommended pavement types for road category and design traffic class

Pavement type		Road category and design traffic class								Reasons for exclusion
		A		B			C			
Base	**Subbase**	**ES100**	**ES30**	**ES10**	**ES3**	**ES1**	**ES0.3**	**ES0.1**	**ES0.03**	
Concrete	Granular	x	x	√	√	√	√	√	√	Granular subbases prone to erosion at joints and cracks
	Cemented	√	√	√	√	√	√	√	√	
Granular	Granular	x	√	√	√	√	√	√	√	Uncertain behaviour for high traffic demand
	Cemented	√	√	√	√	√	√	√	√	
Hot-mix asphalt	Granular	√	√	√	√	x	√	x	x	Cost-effectiveness
	Cemented	√	√	√	√	x	√	x	x	
Cemented	Granular	x	x	x	x	x	x	x	√	Crushing, cracking, rocking blocks and pumping unacceptable
	Cemented	x	x	x	√	√	√	√	√	
BSM's	Granular	x	√	√	√	√	√	√	√	Cost-effectiveness and permanent deformation

The placement of a strong (stiff), thin stabilized layer over relatively weak support will result in the rapid development of fatigue cracking. The high degree of sensitivity of stabilized layers to overloading needs to be considered in the pavement design such that the layers are thick enough to cope with both normal and overloaded vehicles. It has been noted (DoT, 1996) that the damage exponent (n) used in estimating the equivalent traffic can vary between 3 and 10 depending on the type of pavement (cemented over granular) and the state of cracking, with a recommended value of 5 instead of the conventional 4.2. No further discussion regarding traffic calculations are included in this document, suffice it that the n-exponent should be carefully selected as a function of the pavement structure.

It has been suggested (Gray et al, 2011) that for cement stabilized layers, the tensile strength of the material should be at least double the tensile stresses developed at the bottom of the stabilized layer under traffic loading. In addition, the layer strains need to be managed in accordance with the observed behaviour.

The degree of shrinkage has in the past been controlled by avoiding materials that are too strongly stabilized; this is done by controlling the upper limits specified for the unconfined compressive strength (UCS). Although upper limits for the UCS are shown in Table 9 recent experience has indicated that it is more important to achieve the specified ITS, irrespective of the high UCS's that may be obtained in achieving the ITS.

7. CONSTRUCTION OF STABILIZED LAYERS

7.1. Introduction

This chapter describes the construction of stabilized layers and covers safety measures, materials, equipment, calculation of stabilizer spread rates and the construction procedures, including curing of the finished pavement layer.

7.2. Safety Measures

Lime and cement are strongly alkaline and can be dangerous if not handled carefully and reference should be made to the Material Safety Data Sheets available from suppliers. These agents can cause serious injuries.

The following precautions should be taken when working with stabilization materials:

- Workers should be trained on the use of stabilizers and the associated safety precautions and treatments for exposure.
- Wear a buttoned-up long sleeved shirt and trousers or alternatively overalls.
- Wear closed shoes or boots.
- Wear a hard hat and gloves.
- Use goggles and breathing masks
- When long periods of possible exposure are expected, petroleum jelly can be rubbed onto the exposed skin.
- Carefully wash immediately after the work is completed. A small amount of vinegar in the bath water will help to neutralise the alkali.
- Clothing exposed to lime and cement must be washed regularly.

Whenever hydrated lime or cement comes into contact with the skin it should be washed off as soon as possible.

In addition to the chemical hazards described above, physical hazards may also be encountered. These are mostly associated with handling of bags (typically 40 or 50 kg packages) and due consideration should be given to the possibility of back and other strains.

7.3. Stabilizers

The proposed stabilizer requirements will be shown on the gravel borrow pit sheet for the borrow pit concerned or on the pavement design sheets. Once on site, the actual materials to be used in the road with treatment need to be tested with the stabilizers delivered to site to confirm that the proposed stabilizer requirements are still appropriate.

7.4. Equipment

Besides the normal hand tools required for stabilization, the following equipment is usually used for conventional stabilization (cold in situ recycling makes use of different equipment):

7.4.1 Bulk Tanker with Spraybar

The tanker works on the principle that air is pumped into the tank causing the stabilizer to become fluid. This is then pumped out under pressure through the spray-bar.

The spray-bar distributes the stabilizer over a width of between 2.1 and 2.8 m. The amount of stabilizer that is distributed on the road is controlled by adjusting the speed of the tanker.

Bulk spreading of stabilizer should not be carried out in excessively windy conditions, as this leads to loss of stabilizer, poor distribution and environmental pollution.

7.4.2 Road Grader

A road grader is required to spread the heaps of imported soil, to mix in water and stabilizer and to blade and cut the layer to the final level. The mixing process can be expedited by using mixing vanes mounted on the ripper teeth at the rear of the grader.

7.4.3 Disc-harrow (plough)

Disc-harrows used for stabilizing, are heavy and run on rubber wheels with an adjustable depth of penetration. A road grader towing a disc-harrow can also be used to good effect.

7.4.4 Tractor

A powerful tractor that can pull a heavy disc-harrow at a speed of 6 to 10 km/h through loose soil, is essential for conventional stabilization.

7.4.5 Water Bowser

Adequate water bowsers to haul and spread water evenly over the road width must be available to allow completion of the watering, mixing and compaction process in the allotted time. Water bowsers should also be available for curing where water spray curing is utilised (See Chapter 9).

7.4.6 Compaction Equipment

Compaction equipment includes tamping, grid, pneumatic-tyred and vibratory rollers. Sufficient units should be available to allow completion of compaction to the specified requirements prior to development of cementation.

7.4.7 Recycler

Dedicated recycling equipment can be used to add and/or mix the stabilizer and water into the soil to the required depth. It should be ascertained whether or not the particular machine mixes transversely as well as vertically. The treatment of in situ materials and existing pavement layers (full depth reclamation) with the addition of cement or any other powder stabilizer is effectively accomplished with recycling machines (rotary mixers).

7.5. Calculation of Quantities

It is essential that the calculation of quantities and spread rates is done correctly and accurately. Formulae and examples for calculating these are provided in the Gautrans Stabilization Manual (GDPTRW, 2004).

7.6. Preparation of the Surface for Stabilization

The loose layer should be shaped and the levels checked. Excess soil must be removed and hollows filled in.

If the soil is very dry, it is advisable to water it well the previous afternoon so that it can soak overnight. The spraying of excessive water, however, will interfere with effective mixing and compaction during the stabilization operation.

When the stabilizer is spread from bags by hand, the layer should first be rolled once with a steel wheeled roller (without vibration) to ensure that the surface is reasonably even.

When lime is used for the stabilization of a clayey material, spreading and partial mixing may be done the afternoon prior to final mixing and compaction. In the case of all other stabilizers it is important that the entire operation be completed within the allotted time (Table 13). Where lime and slag or lime and flyash are applied separately, the lime can be mixed in on the previous day, but the maximum time allowed for cement once the pozzolan (GGBS or flyash) is added, will apply.

It is important that the laboratory testing should reflect the construction sequence as closely as possible.

7.7. Methods of Spreading Stabilizers

7.7.1 Bulk Spreading

The use of stabilizers from bulk tankers is more convenient than pockets. Larger quantities can be handled daily with less labour. Spreading is also more uniform. Where labour-intensive construction is a project requirement, however, stabilizer in bags should be used.

Before spreading takes place two calculations must be made:

- Required mass of stabilizer per square metre
- Length of road that can be covered by the particular load.

The width of the spray-bar and/or the number of spray runs must be adjusted so that the full section width can be covered with three or four passes, and with minimum overlap or gaps occurring between the strips of stabilizer. The length of road to be covered by the load must be clearly marked.

The application rate should be checked during spreading by setting out canvas mats (1.0m x 1.0m) along the section and weighing the quantity of stabilizer distributed over each canvas mat. The spread rates so measured should be recorded and checked against the allowable deviation from the specified spread rate (± 10%). The correct mass of stabilizer should be replaced on the road once the mats have been removed and the stabilizer weighed.

An additional check on the spread rate is whether the consignment of stabilizer covered the required length of road without a shortfall or surplus. It is most important that this length be correctly calculated and that this requirement be met.

If a blend of two stabilizers is supplied separately in bulk (e.g. cement-slag or lime-flyash), the slag or flyash should be spread first, in the case of cement mixes, followed immediately by the cement. For lime mixes, the lime is applied first, unless otherwise specified by the designer. Mixing should take place only after both stabilizers have been spread.

For materials with a PI greater than 15, especially for weathered basic crystalline materials, initial lime treatment followed by the addition of cement or extender (GGBS or flyash) the following day may be employed. This allows time for the preliminary modification process to take place.

7.7.2 Spreading by Hand from Pockets

It is essential that the pockets are inspected to ensure that they all contain the correct cement and incorrect cement types have not been mixed with the correct stabilizers (Figure 24). The pockets of stabilizer should be neatly laid out in three or four rows, evenly spaced over the width of the road. The distance between the rows must be such that their spacing is approximately the same as the spacing between pockets along the road.

Figure 24: Two different types of cement being laid out for spreading

After the positions for the pockets have been marked on the prepared surface of the layer the pockets can be laid out on these marks. A team of labourers should open the pockets using shovels and spread the stabilizer evenly over the surface. After the stabilizer has been spread with shovels, it is advisable to spread the material more evenly using hand brooms or squeegees. A grader blade should only be used to spread the stabilizer on even surfaces.

If two stabilizers are specified, special attention must be given to ensure that each is spread evenly. If uneven spreading of the two stabilizers occurs, then the extender should be spread and mixed in first after which the surface should be evenly graded and lightly rolled before the cementitious stabilizer is spread and mixed in. This is unnecessary if lime modification is done first.

7.8. Mixing of Stabilizer and Water

7.8.1 Conventional Construction

When spreading is complete, the stabilizer should be partly worked into the layer using the ripper teeth of the grader. Considerable mixing can be achieved if use is made of mixer blades mounted on the ripper teeth at the back of the grader.

The second stage of mixing is done with a disc-harrow. The harrow must be adjusted so that it mixes to the required depth without damaging the underlying layers. This should not be controlled by using the cable that lifts up the plough. An independent depth control must be provided and set to the correct depth of working for each material type and layer depth required. Care must be taken to avoid a thin layer of unstabilized material between the newly constructed layer and the underlying layer. A layer such as this would have significant consequences on the performance of the road (Netterberg and de Beer, 2012). Tests with the heavy vehicle simulator have shown that such layers (even as thin as 10 mm) at the bottom of the treated layer can result in pumping under heavy traffic. Mixing too thin a layer also results in over-stabilization of the stabilized layer.

The speed with which the disc-harrow is drawn is important. A speed of 6 - 10 km/h is regarded as ideal for mixing.

Once the disc-harrow has made at least three passes over the full width of the layer, water can be added and mixing can continue. The water must be applied evenly over the width of the layer using pressure sprays. The required water must not be applied in one single operation but gradually after each application has been mixed in. Care must be taken to ensure that excessive water is not applied, to ensure that the stabilizer does not form balls or lumps.

It may be necessary to move the material to one side with the grader to ensure that the water and the stabilizer is mixed through the full depth of the layer.

It is important to determine that complete mixing has been achieved by opening the layer at a minimum of six points over the length and width of the job-lot for a visual inspection. An indication of the effectiveness of mixing can be obtained by feeling the soil texture. A visual inspection of the surface will not indicate whether effective mixing has taken place. Phenolphthalein sprayed at different points on the surface can be used to determine whether stabilizer is present (see 9.4.1).

If unhydrated lime is used, more water will be necessary than where hydrated lime is used due to the slaking process. The layer should be moistened the previous day. The normal procedure can then be followed except that additional water should be applied.

The best compaction moisture content (MC) should be determined from trial sections, as it will depend on the plant available and the material properties. At no time, however, should the "saturation" MC be exceeded and it is recommended that compaction should be carried out at not more than 80 per cent of saturation moisture content.

Materials with a high lime demand may require that the stabilizer be added and mixed in two or more processes. This must be indicated on the design or borrow pit sheets.

7.8.2 Recycling equipment

The use of customised recycling equipment in cement stabilization is increasing rapidly, as a result of the numerous benefits associated with this plant. These include environmental, energy, time and cost advantages as well as potentially better control of the application and mixing of the stabilizer. The equipment is particularly useful for recycling existing roads during rehabilitation, with the addition of cement (lime is seldom used as the primary stabilizer in recycling) but has also been used for mixing cement into new materials imported from borrow.

In conventional use, the plant mills and lifts portion of the existing road material, mixes it and places it back on the road. Cement (or lime, flyash, GGBS or combinations of these) can be added through the plant during the mixing process, but this tends to be extremely dusty. Usually, the stabilizer is spread on the road prior to milling and is mixed into the recycled layer material by the equipment at the same time that water is added.

Although recycling plant provides very good mixing vertically, there is evidence that the lateral or longitudinal movement of the material is not as good and variations in initial spreading of the cement are reflected in the final placed material.

The plant moves relatively quickly and compaction needs to follow the recycler immediately. **Care must be taken in handling the wheel tracks left by the recycler, which are heavily compacted already and can result in variable densities and uneven compaction.** For more information on avoiding such problems, reference can be made to for instance, Wirtgen, (2010). The mixing time in the recycler is very short and it is possible that unusual processes such as “brief-mix set”, which is a form of rapid setting, could take place.

Most problems related to this type of construction are related to poor construction control and attempts to blend, for example, good base materials with marginal quality shoulder materials during road widening. Problems with overlapping and application of uniform and sufficient rolling are also common, but good supervision and training can avoid these. The fundamental principle of using this plant is, however, sound and numerous problems associated with conventional construction can be minimised.

7.9. Compaction

As soon as mixing is complete, compaction must be carried out. Standard procedures are widely described in the literature and these should be followed.

Except for lime modification; treated layers should be compacted as quickly as possible to maximise the benefits of stabilization. The processing and compaction of stabilized layers should be completed within the times provisionally given in Table 13.

Table 13: Total stabilization processing time

Type of stabilizer	Maximum time* (hours)
Common cement	To be determined using working time test
Lime (Modification)	24 - 48
Lime (Cementation)	6 – 8 (depending on setting rate)
Maximum time* for completion after stabilizer has made contact with material on the road.	

These times are provided as guidelines. However, it is possible that cementation may occur within these time limits, particularly under hot working conditions. When this occurs, problems with density and strengths will be noted and corrective action must be initiated. This may include the processing of shorter lengths of construction, the utilisation of extra equipment or the possible use of a different stabilizer. Under no circumstances should construction continue until the cause of the problems has been identified and a viable alternative implemented. Additional efforts to achieve the specified density usually result in degradation of the upper portions of the stabilized layer and should not be attempted.

High temperatures accelerate the hydration reactions. When this occurs during layer construction it leads to a greater number of cement bonds being created and broken down again during mixing and compaction. The result is a weaker and less dense final product. The use of strength class 42.5 cements is suspected to also result in a faster set with similar problems. When compaction cannot be completed in the recommended processing time, shorter lengths must be constructed at a time.

Most soils contain some water, which will react with the stabilizer - it is seldom that a soil is so dry that it can be assumed that no moisture will come into immediate contact with the stabilizer. Therefore, the time allowed starts when the stabilizer first comes into contact with the soil.

During lime modification of clayey materials, compaction can be delayed until the day after the lime has been mixed in. In the case of heavy montmorillonite clays, up to 72 hours may be necessary for the modification reaction to be largely completed.

The speed of the compaction plant is important. Rollers moving either too fast or too slowly will not compact to the required density. As each material has specific rolling requirements, proof rolling should be carried out to identify a suitable range of rolling speeds for each material.

During the final rolling and cutting to level, it is imperative that no dry or untreated material is graded into any slacks or depressions in the surface. Filling instead of cutting will cause the formation of weak spots in the completed road. This is particularly relevant to fine materials and the top of the base.

When rolling material that has been recycled, it is important that the rollers do not get one side “perched” on the un-recycled portion of the layer, as this reduces the rolling force on the material adjacent to the “join”. All passes must be on the recycled material.

7.10. Reworking

When a stabilized layer cannot be satisfactorily compacted within the prescribed time, both the dry density and the final strength are reduced. In this case the layer must be ripped, re-stabilized and reconstructed. This is also necessary when the layer has been rejected for some other reason (eg, inadequate density). If lime is used alone, this is not usually necessary as cementing with lime is usually a slower process.

When reworking, not more than half of the original stabilizer application must be added (most exchange and flocculation reactions will already have occurred and there will usually be some residual stabilizer). The effects of this should be checked by laboratory testing prior to the final decision on the addition of extra stabilizer being made. The layer must not be reworked more than once except when lime is used on its own, to minimise cracking – lime treated material is usually unlikely to crack to the same degree as cement. When reworking material with additional stabilizer, the MDD must be retested.

It is also suggested that reworking of rejected material should not be permitted more than 7 days after the initial construction.

7.11. Curing

The hydration reactions that cause cementation during chemical stabilization require a constant source of moisture. These reactions commence when the stabilizer is applied and continue slowly for a long period, in excess of 12 months provided moisture is present. The first seven days after construction, however, are the most critical, during which effective curing is essential. This is not the case where only lime is used.

Drying out (even once) of the stabilized layer inhibits the hydration reactions. Throughout the stabilization process and thereafter, moisture is lost from the treated layer by evaporation and the hydration reactions. Curing is necessary to limit or compensate for this loss and is therefore a key element in the quality of the finished layer. However, a conflict arises as moisture is required for strength development of the stabilized layer, but construction specifications usually require that the layer should be dried out prior to application of the seal and trafficking. Effective curing, however, should take precedence.

Various curing methods, their application, advantages and limitations are discussed below (Netterberg, 1987; Netterberg et al, 1987; Paige-Green et al, 1990 and Netterberg, 1991) but these should be assessed in conjunction with durability aspects discussed in section 9.6.

7.11.1 Regular Application of Water

In this process, the stabilized layer is kept continuously moist by regular spraying with a water bowser. This is often applied to all stabilized layers, including the base. The disadvantage of this technique is that the top few millimetres of the layer tends to dry out rapidly between water applications. This usually results in repetitive cycles of wetting and drying, which creates a weak layer in the pavement, and in the case of a base can result in de-bonding of a thin asphalt surfacing layer or a bituminous seal.

During the curing period, the initial strength of the stabilized layer is developing, and a fully laden water bowser could overstress it. Water should thus be sprayed using a side spraybar or hose, or else only half-filled bowsers should be used.

It is clear that the water spraying technique has more disadvantages than benefits and should be avoided.

The use of an effective micro-irrigation system would overcome the disadvantages of spraying with a water bowser, but these tend to be expensive, time consuming to install and susceptible to damage by equipment movements.

7.11.2 Covering with Impermeable Sheeting

An impermeable sheet or cover can be placed over the finished layer immediately after compaction, with stones (or preferably a sand-berm to minimise wind ingress) to hold it down and it should be left in place for at least seven days. Effective overlap of sheets and close stone placement is essential to ensure a good seal. This method is suitable for all pavement layers, including the base and ensures a constant, humid environment. It also negates the need for, and prevents, vehicles from travelling on the layer. Unfortunately, the method is expensive, the sheeting can be difficult to apply in strong winds and is susceptible to vandalism or theft.

7.11.3 Application of a Curing Membrane

The application of a good bituminous membrane is the most effective curing method. A surface treatment grade of bitumen emulsion is probably the most effective, but has practical problems in terms of adhesion to tyres during follow-up sealing operations. Blinding with sand or crusher dust can overcome this problem and provide a temporary riding surface until sealing commences. It is still important that no other traffic is permitted on the surface during the curing period. This is obviously a costly operation for layers other than the base. It is also not advisable from a pavement drainage perspective to have an impermeable layer within the pavement structure.

The use of a cutback bitumen is probably the most common curing technique after water spraying. However, these contain too much cutback agent and tend to penetrate most materials without forming an effective impermeable membrane at normal application rates. An MC70 or MC250 that provides good balance between penetration and a surface membrane is far more effective than MC30. MC30 becomes ineffective as a curing membrane as soon as it is dry enough to walk, drive or seal on. In critical areas, the use of a concrete curing membrane could be considered. Tar primes (where used taking into consideration their carcinogenic properties) should not be applied within 7 days of compaction (DoT, 1986b).

Curing membranes should be applied as soon as possible after completion of the layer, but not later than 48 hours with the layer being kept continually moist during this period.

The curing membrane technique is cost-effective and prevents evaporation through the action of sun and wind. As the membrane is applied on completion of the layer, little damage is caused through overstressing by the tanker.

A disadvantage of this method (or covering with black plastic sheeting) is that the black mat causes additional heating of the layer, accelerating the hydration process on the one hand, but also setting up thermal stresses which could lead to cracking. The high temperatures in the upper base will also reduce the ultimate strength of the upper base.

7.11.4 Covering with the Subsequent Layer

The material for the subsequent layer can be tipped and spread over the stabilized layer and kept moist. This has been found to be one of the most effective curing techniques. Care should, however, be taken to minimise vehicle movements on the stabilized layer, and an end-tipping technique should be used where feasible. Material from the verge can be used as it is less costly than importing material specifically for curing.

This method can also be used to cure stabilized base layers using material from the verge, loose gravel or sand up to 100 mm thick depending on water spraying frequency, but requires careful removal and brooming prior to sealing. Trials should be conducted at the start of the project to determine the optimum loose layer thickness and watering frequency.

This is the preferred method where appropriate, due to the moisture retention capacity of the material and its cost effectiveness. It is also suitable for the longer curing periods required for lime and in some cases lime/slag stabilization.

7.12. Modification Method

Modification of a soil in a pavement layer should be carried out in the following manner:

i) The soil for the pavement layer is shaped and the lime should be spread and mixed into the material as discussed in Sections 7.6to 7.8.
ii) The layer should be lightly compacted to seal the surface and limit moisture movement, and then left to stand for a minimum of 12 hours. During this period the majority of the cation exchange reactions occur and the fine particles flocculate.
iii) After the required standing time, the layer should be ripped and constructed conventionally.
iv) The MDD is also then determined as it will usually differ from that of the un-modified material.

Modification would normally be done on crushed-stone or natural gravel base material to reduce a PI slightly outside the standard specification to within the required limits or on a clayey subgrade to improve workability. There is a strong likelihood that some of this PI could return after some time and it is important that the natural strength of the material meets the required specification. The addition of 1% lime to many crushed basic crystalline materials in an apparent modification reaction has proved to be effective for improving the durability and appears to remain present for significant periods.

7.13. Weak Interlayers

The importance of avoiding the building-in of weak interlayers – especially between the base and surfacing – cannot be overstressed (Netterberg and de Beer, 2012). For example, consideration of the work of de Beer et al (1981) shows that a reduction in the UCS of the upper 50 mm to 0.7 MPa (= CBR of 80) of a 200 mm thick base layer with an overall UCS of 1.0 MPa would reduce the structural capacity (to a rut depth of 20 mm) from 2 million to 0.3 million E80s. Such layers are not always obvious and have been the cause of many cases of premature distress. They have many causes, including the filling of slacks, over compaction, carbonation, drying out and the use of a tar prime (Netterberg, 1987).

Many – especially overloaded – heavy vehicles have tyre pressures in excess of 1.0 MPa and de Beer (1989b) also shows the extreme importance of this aspect with respect to crushing of the upper portion of the base course.

8. CONSTRUCTION CONTROL

8.1. Introduction

Stabilization can be a complex and expensive operation and significant wastage can be caused by carelessness. It is, therefore, essential that good and continuous supervision and strict control be diligently maintained. The Standard Specifications for Road and Bridge Works (Colto, 1998) provide guidelines on quality control of pavement layers. However, aspects such as testing frequencies, sample and lot sizes and even required tests are generally at the Engineer's discretion. It is therefore important that the control of required properties such as ITS and plasticity after treatment is specified in the project specification.

Part of the quality assurance process should include accurate and complete recording of the as-built data. This can provide valuable information for the assessment of future performance of the pavement.

Various properties should be measured in order to ensure that the material selection is correct and construction has been carried out to specification.

8.2. Relative Compaction

The MDD of the natural soil is altered by stabilization. In order to control compaction in the field, it is necessary to determine the maximum dry density on the stabilized material. Optimally, a material sample should be taken immediately prior to addition of the cement at the randomly selected point identified for compaction testing. This material should be tested, after the addition of the equivalent field cement percentage and using the same cement as the project, according to SANS 3001: GR40, except that the material is conditioned for a period equivalent to the expected construction time (time between addition of cement and final compaction). This maximum dry density should be used when checking field compaction. The typical variabilities of materials to be stabilized are such that this process should be followed at every point identified for compaction testing. At the same time confirmation of the optimum moisture content will be obtained.

It is nowadays common practice to obtain the sample for MDD determination immediately after mixing (or from directly behind a recycling machine). This sample should be returned to the laboratory immediately (not allowed to lie at the side of the road in the sun in a black bag as is commonly seen) and tested in the same time frame as the actual on-site compaction process.

The relative compaction of chemically stabilized layers should be controlled as specified in Clauses 8200 and 8300 of the COLTO Standard Specification (1998) or the relevant project specification.

The MDD usually decreases as cementation takes place and if the laboratory test does not simulate the field condition, spurious compaction results are likely to be obtained. A layer may pass or fail the quality control requirements depending on exactly when and how the MDD is determined in the laboratory. Samples collected in the field should be compacted in the laboratory after the same time delay that affected the material in situ and after handling at the same temperatures.

It should be noted that the material tested in the laboratory usually differs from that in the field as particles larger than 19 mm have been removed. This should be considered when interpreting the results. The new specification (SABS 3100:GR40) allows for material up to 37.5 mm in size, however, more closely simulating the field condition.

Bearing all these points in mind, whatever method is used to obtain the maximum stabilized density in the field and laboratory, 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. Although it is important to achieve a high degree of compaction, this can, however, often be attained by following a method specification based on proof rolling trials. The use of Intelligent Compaction equipment on the rollers can also ensure that the optimum compaction is applied to the layer.

8.3. Stabilizer Content

The primary purpose of these tests is to assess the uniformity of mixing and spreading and not to control the actual stabilizer content (refer to Paragraph 3506 and 8305(d) of COLTO, 1998). The discussion in Section 4.6 regarding problems with the determination of stabilizer contents is relevant. It should be noted that at the maximum coefficient of variation of 30% allowed for good mixing, with 20 samples, 20% of the results could have stabilizer contents equal to or below 1.2%.

Spraying the sides of a hole evenly with phenolphthalein solution (see 9.4.1) right to the bottom will show the presence or absence of stabilizer but not the amount. An indication of the mixing efficiency can also be obtained by sampling after mixing, splitting the sample, determining a UCS and PI on one half without further mixing and a UCS and PI on the other half after thorough mixing in the laboratory.

The minimum cementitious binder content in chemically stabilized layers should be controlled as specified in Clauses 8200 and 8300 of the Standard Specification (COLTO, 1998) or the relevant project specification.

8.4. Material Strength

The UCS and ITS values on material sampled from the layer after the addition of cement and water and mixing should be determined. Samples shall be prepared according to SANS 3001-GR51 ensuring that the conditioning time in the laboratory (between mixing and compaction) corresponds as closely as possible with the time taken for compaction in the field, but should not exceed 3 hours.

These tests should be done according to SANS 3001 Methods GR53 and GR54, but using two compaction efforts as follows, both at Mod AASHO OMC:

- Modified AASHO with 55 blows
- Standard Proctor with 55 blows

Three briquettes should be prepared at each effort for the UCS and the ITS. The average of the three results for each of the two energies is plotted against the density and joined with a straight line. The UCS and ITS values at 100% mod AASHO effort should be compared with the specified design strengths. Lower than specified strengths indicate that the gravel quality is inadequate or that there is insufficient stabilizer for that material. It should be remembered that the laboratory UCS and ITS are determined on well-mixed, well-compacted and well-cured specimens, which may not simulate the actual field conditions, which are likely to be not as well mixed or cured.

The crushed briquettes should be retained for PI testing as discussed in the following section.

Where rapid (accelerated) curing is used in the laboratory, higher strengths will often be obtained than when using conventional curing. In these cases, duplicate specimens using normal curing should be tested to provide a correlation with the accelerated curing results.

8.5. Grading and Plasticity

Regular testing of designated borrow pits (guideline testing) helps to ensure proper utilisation of these sources. Where the PI or grading results differ significantly from those appearing on the borrow pit design sheets, a warning that the wrong horizon is being exploited or excavation is extending outside the designated area is given.

The PI after stabilization should be determined on the crushed UCS briquettes as described in Section 4.2.2 and above and should comply with the requirements of Table 6.????.

8.6. Visual Inspection

Once the borrow material has been tipped onto the alignment, and prior to spreading, the material should be visually inspected for conformance to specification. Attention should be paid to the grading and PI, and where this appears to be substandard the tipped material should be removed. It may be necessary to confirm the visual observations with laboratory testing before any material is removed from the road.

Aspects that should be looked for during the visual inspection include:

- segregation of material;
- slacks and depressions filled with dry or untreated material;
- particularly moist or soft areas;
- changes in material characteristics.

9. DURABILITY OF STABILIZED LAYERS

9.1. Introduction

It is expected of all pavement layers that they will provide the desired service for the design life of the road. Except when pavements fail prematurely through poor construction, overloading, excessive pavement moisture or poor maintenance, poor performance is commonly related to unsatisfactory durability of one or more of the materials or layers in the pavement. In order to avoid durability problems, it is essential that both the aggregate and the cementation in the stabilized layers are durable.

Poor durability is normally manifested as either:

- Disintegration or weakening of the upper portion of stabilized bases leading to loss of the seal, or
- Partial or even complete loss of cementation of the treated layer leading to reduced strength, rutting, cracking and/or shearing
- Erosion of stabilized layers, particularly under rigid pavements.

These problems can be attributed to various causes, including damage by soluble salts or acids, degradation of the aggregate or carbonation of the stabilized layer. Loss of cementation of a cement-treated mine sand subbase attributed to soluble salts or acids has been reported in Gauteng (Netterberg, 1979), but provided the normal specifications for the electrical conductivity, pH of the raw materials and sulphate content (COLTO, 1998) are applied, damage to stabilized layers is unlikely.

Volume increases associated with degradation of marginal aggregates, particularly basic crystalline rocks, in stabilized layers could lead to problems within stabilized layers. The retention of a high pH in these materials assists in the prevention of material breakdown (Clauss, 1967; Clauss and Loudon, 1971).

Extensive research and field-testing and experimentation has indicated that the predominant cause of poor durability in stabilized materials in South Africa is carbonation (Netterberg, 1987; Paige-Green et al, 1990).

9.2. Carbonation

9.2.1 Background

It was established as long ago as the early 1960s that carbonation of stabilized materials is possible and can adversely affect the cementation. Carbonation is the process by which lime and cementation products added to a material or developed during the hydration reactions of cementitious stabilization of a material react with carbon dioxide in the atmosphere or soil air (Paige-Green et al, 1990). Calcium carbonate is produced and the pH of the stabilized material reduces to that of calcite (about 8.3) resulting in instability of the remaining cementitious products.

The common practice of curing stabilized layers by “keeping it continuously wet” frequently results in weakening or loosening of the top 6 to 25 mm as the material is seldom kept continuously wet. This loose layer, weakly cemented by calcium carbonate, must be removed before a subsequent layer or surfacing can be applied. Numerous investigations of this loose layer have shown that it has a low pH, an increased reactivity with hydrochloric acid and generally contains more calcium carbonate than material deeper in the layer that remains well cemented. This is undisputed evidence that carbonation has affected the material and the process can easily be simulated in the laboratory.

Lime and, to a lesser extent, cement are unstable under normal environmental conditions and are readily susceptible to carbonation. In concrete technology it is generally accepted that complete carbonation of cement is chemically possible even with the low concentration of carbon dioxide (0.03%) present in the atmosphere, provided the air permeability is sufficient. Through carbonation, the stabilization process is reversed, the active component of the stabilizer is neutralised and any cementing action is inhibited or even reversed as the pH drops.

Disintegration of stabilized layers in roads during carbonation is caused by the volume increase (about 10%) as lime ($Ca(OH)_2$) changes to calcium carbonate ($CaCO_3$) and to a lesser extent, the volume decrease (about 2%) as the cementation products revert to silica and calcium carbonate. If the stresses generated by these volume changes exceed the tensile strength of the stabilized material, microcracking will occur (Paige-Green, 1991). As these microcracks coalesce, the material loses strength and deteriorates. It is for this reason that the tensile strength is so important. The number of fully carbonated materials found in the field that have retained their strength also provides confirmation of this. In these cases, sufficient stabilizer would have been added to develop a tensile strength sufficient to resist microcracking at the time of carbonation.

9.2.2 Rate of Carbonation

The following depths of penetration of carbonation have been determined from laboratory and field tests (Netterberg, 1987; Paige-Green et al, 1990; Netterberg et al, 1987; Netterberg, 1991 and Sampson et al, 1987):

- Material exposed to the atmosphere; 0.5 to 2.0 mm per day on all exposed surfaces (most rapid penetration during curing and before sealing).
- Material exposed to soil air (upwards and sideways); 2.0 to 50.0 mm per annum (notably on the bottom and sides of stabilized layers; also from the top to the bottom of stabilized subbases beneath unstabilized base courses).

9.3. Extent of the Problem

Research has identified many cases in South Africa where stabilization has failed (Netterberg, 1987; 1991). It must be noted that a large proportion of South African roads include at least one stabilized layer.

Although most pavements that are stabilized with lime, cement or combinations with extenders perform well, problems occur predominantly with weak or marginal quality materials, especially with residual basic crystalline and calcrete materials.

Commonly encountered problems are:

- Loss of strength, especially in the top part of the stabilized layers
- Return of plasticity index to that of the original material
- Flaking and blistering of the primed base courses
- Formation of a loose layer of material between the base and surfacing
- Movement of the surfacing on the base course (loose surfacing) with resultant arcuate cracking of the surfacing

Loss of cementation and the presence of carbonation does not always result in the failure of roads. Possible reasons for this are that the roads carry light traffic, are well stabilized with good quality stabilizer and good quality gravel was used. The quality of material used plays an important role - the poorer the material before stabilization, the greater the possibility that carbonation and loss of cementation will lead to failure of the road. Materials that perform well, even after carbonation, are generally inherently adequate for their purpose or had sufficient stabilizer added to produce and retain adequate strength, even after carbonation.

9.4. Identification of Carbonation

9.4.1 Tests to Confirm Carbonation

Phenolphthalein and hydrochloric acid are useful field indicators to determine whether carbonation has occurred. The solutions are sprayed onto the soil being tested, generally on the profile of a freshly exposed face in a test-pit, and the results are compared with the reactions listed below (Netterberg, 1984).

Phenolphthalein Test

This test indicates which strata in a pavement layer have been carbonated, e.g. where the top 20 mm of a layer does not change colour relative to the remaining depth, carbonation of the top surface of the layer is likely (Table 14) (Netterberg 1984).

Table 14: Interpretation of results of phenolphthalein test

	Sprayed area remains colourless	**Sprayed area turns pink**	**Sprayed area turns red**
Probable pH interpretation	pH ≤ 8.4 Carbonation is complete or no stabilizer was present	pH = 8.4 - 10 Significant but incomplete carbonation	pH ≥ 10 No or little carbonation

Where the phenolphthalein test shows no reaction it must be established whether a stabilizer was originally added to the material. This is achieved using dilute hydrochloric acid.

Dilute Hydrochloric Acid Test

Diluted hydrochloric acid **Error! Reference source not found.** reacts (in the form of vigorous effervescence) with the following materials commonly found in pavement layers:

- Carbonated stabilizer
- Carbonates occurring naturally in the soil
- Old lime (fresh lime may show some effervescence)

- Old cement (fresh cement may show a weak effervescence, except for the limestone cements (CEM II A-L), which contain between 6 and 20 per cent limestone and effervesce strongly.

Where no effervescence is observed it can be concluded that no carbonation has taken place, or no stabilizer was added to the material. The hydrochloric acid test cannot generally be used on calcretes, limestones and dolomites (effervescence is very slow) as the acid reacts with the carbonate soil, which predominates, masking any lesser reaction with the stabilizer. However, careful comparison between different portions of the stabilized layers, together with phenolphthalein testing, can provide an indication of whether carbonation has affected the material.

Safety

Phenolphthalein is a strong laxative and a suspected carcinogen and care should be taken when working with it. Hydrochloric acid is corrosive and should also be handled carefully. Suitable protective equipment should be worn when using these chemicals.

9.4.2 Determination of the pH

The pH as well as the solubility of lime reduces with an increase in temperature. For example the pH of a saturated solution of calcium hydroxide ($Ca(OH)_2$) reduces significantly as shown below (Table 15):

Table 15: pH and solubility/temperature relationship of calcium hydroxide

Temperature (°C)	0	10	25	40	60	80
pH	13.4	13.0	12.4	12.0	11.5	11
Solubility (g/100g)	0.185	0.174	0.159	0.140	0.116	0.092

In hot areas where high road temperatures occur beneath dark primes and surfacing, carbonation can accelerate as a result of the drop in the solubility and pH of the stabilizer. The stability of the stabilizing action can also be negatively influenced. The influence of moisture condensation beneath the surfacing as a result of daily temperature fluctuations can affect the pH as well as causing a reduction in the available lime in the basecourse directly below the surfacing by dilution.

9.5. Consequences of Carbonation

Carbonation can have a number of negative effects on the stabilization process. Several of these effects are theoretically possible, but it is difficult to actually prove their occurrence in practice.

The theoretically possible effects include (Netterberg, 1987; 1991):

- Destruction of $Ca(OH)_2$, and $Mg(OH)_2$ and replacement by $CaCO_3$ and $MgCO_3$
- Loss of cementation
- Reduction of lime content
- Reduction of the pH from approximately 12.4 (lime) and sometimes higher (certain cements) to about 8.3 (pH of calcium carbonate)
- Decreased solubility
- Expansion, leading to densification of the material under traffic. Stabilized material generally also has a lower maximum dry density and a higher optimum moisture content than the equivalent unstabilized material.

The effects seen in the field and laboratory include (Netterberg, 1987; 1991):

- Reduction of $Ca(OH)_2$ content and increase in $CaCO_3$
- Reduction of the pH to between 8.3 and 10
- Increase in PI
- Microcracking of the cemented matrix
- Loss of strength (UCS and ITS) and development of loose layers
- Rutting
- Decrease in electrical conductivity

These effects can lead to distress in the surface or total disintegration of the stabilized layer before or after the seal/asphalt has been placed. The seal stone can often be easily pushed into the base course under traffic causing the surfacing to bleed, or the seal/base course interface to move and fail (stripping, rutting). The problem is often manifested during the sealing operation when after brooming and priming, the upper portion of the base fails during rolling of the chips. The consequences of this are only seen some time later when bleeding and “rutting” occur.

Loss of cementation of the entire stabilized layer can occur and has been observed on a number of projects. Carbonation usually starts at surfaces exposed to the atmosphere, at the base of stabilized layers or adjacent to shoulders or stabilization cracks (Figure 25). Rutting and deformation develop rapidly in carbonated materials.

Figure 25: Carbonation along cracks in top of the base

Material can lose, on average, about 40% of its unconfined compressive strength on carbonation regardless of it having been stabilized with lime, cement and/or lime/slag (Paige-Green, 1991; Netterberg, 1987; 1991). Lime stabilized samples that have been cured for seven days under controlled conditions, showed strength losses of between 45 and 75 per cent after carbonation (Paige-Green, 1991).

In some cases, the plasticity of stabilized materials returned to the initial plasticity of the untreated materials after carbonation. The plasticity should not increase after it has been effectively treated with lime during the life of a road, unless the ICS was not fully satisfied. In cases where the plasticity has returned, it can be attributed to one or more of the following factors (mostly after Netterberg, 1987):

- Poor mixing of the stabilizer with the material
- Insufficient stabilizer added to the material
- Destruction of the stabilizer before it reacted with all the clay particles in the material, e.g. premature wetting/drying
- Stabilizer or soil types that form clods
- The stabilizer reacted only with the outer surface of clay lumps or weathered rock. Disintegration of these lumps or rock after carbonation has released untreated clays leading to an increase in the PI.

Marginal materials with poor gradings and soft aggregate particles tend to be more prone to carbonation than for example good G5 or G6 materials and the influence on the material properties is more significant. Such materials are usually more moisture sensitive and the loss of stabilization causes the material to lose its strength to a greater degree when wet.

9.6. Construction Control to Limit Carbonation

The following construction procedures must be carefully controlled to avoid or at least minimise the possible carbonation of susceptible materials.

- Layers must be compacted to as high a density as possible (without disturbing the grading excessively) as soon as possible after mixing with the stabilizer, to expedite cementation and to prevent carbonation reactions. This action helps by sealing the layer and reducing the voids (not applicable to lime modification).
- If necessary, the bottom half of stabilized layers must be compacted separately to ensure that good compaction is obtained throughout the layer. Less than 5% air voids is desirable, which may require additional compaction effort. Layers that are too thin may result in “biscuits” and the related consequential problems.
- Where modification is required and the material is to be worked in two events with significant delays, slightly higher stabilizer contents should be applied to compensate for carbonation. Care should be taken to seal the surface after the initial application of stabilizer.
- Unnecessary delays in the placing of the next layer on a stabilized layer must be avoided. Stabilized base courses must be sealed as soon as possible after construction.
- The stabilizer must be as fresh as possible and must not be stored for long periods before use.
- Where difficulties in achieving a well-compacted and cured layer are envisaged, the thickness of the layer should be increased to allow skimming of the upper layer prior to sealing. Success of this will depend to a large degree on the grading of the material in the layer as large aggregate will result in an unacceptably uneven surface after skimming.

9.6.1 Curing and Surface Protection

For effective curing and the prevention of carbonation of a stabilized layer, it must be kept moist and be compacted to as high a density as practically possible. Methods of maintaining the layer in a damp condition are discussed in Section 7.11 and expanded on below.

When carbonation of the top of a base is suspected, carry out checks using phenolphthalein solution on a freshly exposed surface, before the surfacing is placed. Where less than about 5 mm of carbonation has occurred the loose layer must be swept off before applying a sealing membrane or constructing the surfacing. It is important to check that the broom has actually removed all carbonated material and exposed a hard uncarbonated layer.

Where more than about 5 mm has occurred, the layer should be lightly watered and skimmed, before being rolled with a light pneumatic tyred roller, primed and sealed. This is usually not possible on material with significant aggregate. Brooming, however will usually result in an unacceptably rough surface, which will not provide the required finish under a single or double seal. In these cases, reconstruction or the use of a bituminous slurry levelling layer may be required.

Other methods that can indicate whether a weak layer is present are:

- The ball penetration test, or
- A DCP can be used when deep carbonation is suspected (more than 10 to 20 mm). The penetration after each blow should be measured.
- By dragging a chain or geological hammer on the surface, a hollow sound will usually indicate carbonation, or at least a weak or biscuit layer.

9.6.2 Effectiveness of Curing Techniques

Table 16 gives an indication of the effectiveness of various curing methods to limit carbonation. These methods have been listed in order of effectiveness. The results are based on laboratory, small-scale and full-scale field experimentation (Netterberg et al, 1987).

Table 16: Effectiveness of curing method to limit carbonation (Netterberg et al, 1987)

Apparent order of merit	Curing method	No of days for 5 mm carbonation*	Remarks
1	Bitumen emulsion	> 18-150	
2	150/200 Pen Bit on MC-30	> 52 ?	150/200 Pen Bit applied within 24 hours
3	MC 3000 on MC-30	> 52 ?	MC 3000 applied within 24 hours
4	Covered with clayey layer	46-150	
5	Kept 100 per cent moist	> 30-90	
6	50 mm thick loose sand layer wetted once per day	55	
7	MC-800	29-32	
8	MC-70	35	
9	Covered with bags	27	
10	Wetted 4 times per day	21	
11	40-50 mm thick loose sand layer (dry)	20? - 24	
12	40-45 mm thick loose sand layer wetted once per day for 7 days	18-45	
13	150-200 mm thick loose soil layer	15- > 97	
14	Wetted twice per day	15	
15	Emulsion (MSP1 with diesel oil)	13	
16	RTH 3/12P	10-97 ?	More than 22 days unlikely
17	RC 70	10-20	
18	Wetted twice per day for 7 days, followed up with MC-30	10	
19	Wetted every 2nd day	10	
20	RTL 3/12P	8-10	
21	Wetted twice per day for 7 days	7-9	
22	MC-30	4-35 ?	
23	Do nothing - allow to dry	4-31	
* - the figures in this column provide the typical ranges of time (number of days) for 5 mm of surface carbonation to take place from completion of the stabilized layer.			

It is clear that emulsions and penetration grade bitumens used as a prime coat are more effective in limiting carbonation than the conventional cut-back bitumen and tar primes.

9.6.3 Treatment of Stabilized Base Courses

Various means of treating stabilized base courses to minimise the effects of carbonation have been employed. The following are probably the most effective (Netterberg, 1987; Netterberg et al, 1987):

i) Spray an appropriate bituminous prime directly after stabilization (not later than 48 hours). The layer must be kept ***continuously*** wet between construction and priming and must be relatively damp at the time of priming so that penetration of the prime coat is minimised and a surface membrane is formed. Bitumen primes must be used and tar primes avoided. The road must be sealed as soon as the prime coat has dried, keeping traffic off the road during this curing period.
ii) If a prime and/or sealing membrane cannot be applied immediately, the base course must be kept continuously moist. It is important that the layer is not subjected to wet/dry cycles during this period. After a sealing membrane or prime is applied, the road must be surfaced as soon as possible.
iii) Use a prime coat seal (1 or 2 layers MC-70 with a 5 mm layer of graded sand/crusher-dust) as soon as possible after the layer has been stabilized.
iv) Apply a spray grade emulsion with coarse sand or fine stone blinding as soon as the layer has been stabilized.
v) To minimise the penetration of carbonation from the sides of the base layer, the prime coat can be extended one metre wider than the stabilized layer.
vi) As a last resort a layer of material from the side of the road can be spread on the base and kept moist with regular water spraying. This will need to be totally removed and the layer carefully broomed before applying the surfacing.

9.6.4 Treatment of Stabilized Subbase and Selected Layers

Although the effects of carbonation on stabilized subbase or selected layers is not as critical as that on stabilized base courses, the structural capacity of a pavement can be significantly reduced through reductions in support (Netterberg, 1987; Netterberg et al, 1987). It is, therefore, still important that carbonation is minimised in these layers.

The next layer should be placed as soon as possible (within 48 hours) on the stabilized subbase and/or selected layer, ensuring that no vehicle damage is done to the stabilized layer. The exposed layer should be kept wet during the operation.

9.7. Repair of Layers in which Carbonation has Occurred

Where carbonation has occurred and is detected, urgent repair is necessary to retain the structural integrity of the pavement. Rehabilitation methods will depend on whether carbonation has occurred from the top, bottom or on the sides of the layer.

9.7.1 Carbonation from the Top

Carbonation from above influences any of the stabilized layers (base, subbase, selected layer or stabilized shoulders). It is usually caused by ineffective curing techniques or the exposure of stabilized layers to the atmosphere for excessively long periods before being sealed or covered.

When carbonation has penetrated less than about 10 mm into the layer, a loose layer can be swept off with road brooms and/or a mechanical broom, before the next layer is applied. If the layer is a base course and carbonation has occurred deeper than 10 mm or the weak layer is too strong to be removed by brooming, the loose material can possibly be skimmed with a road grader. The application of a coarse slurry will usually be necessary to remove the unevenness that typically results from this procedure. Care must be taken in this operation not to disturb or weaken the remainder of the layer. Unless provision is made for skimming the layer, the structural capacity of the pavement can be significantly affected by the reduced layer thickness.

9.7.2 Carbonation from the Bottom

Carbonation from below usually starts immediately after construction and it is difficult to predict how long it will take for the full layer thickness to carbonate in the absence of durability test results. However, it is likely to be at least several years (Sampson et al, 1987). Where full layers have carbonated and the pavement has failed it is recommended that the road or parts thereof be reconstructed. It should be noted that carbon dioxide contents as high as 13% have been measured in road subgrades – 400 times that of the atmosphere. Such high carbon dioxide is highly aggressive towards the bottom of the stabilized layer.

9.7.3 Carbonation from the Sides of the Road or through Cracks

Carbonation from the sides usually takes the form of a horizontal wedge penetrating towards the centre of the road. A sign of this is increased rutting in the outer wheelpath relative to the inner. This effect can be limited by surfacing the road shoulders.

Carbonation can also affect the material adjacent to any cracks in the road, especially the typical wide block cracks resulting from stabilization. It is thus important to seal cracks as soon as they become visible so that the penetration of carbon dioxide can be minimised.

9.8. Sulphate damage

The durability of stabilized layers can also be detrimentally affected by sulphate damage. This could be in the form of salt crystallization, sulphate attack or excessively acid materials or pore water, all of which could lead to a complete loss of cementation and/or excessive heaving and cracking.

The following limits for selected test results have been suggested:

Total salts (maximum conductivity)

Base and subbase: 0.15 S/m at 25°C

Selected subgrade: 0.40 S/m at 25°C

Sulphates (all layers)

(a) The maximum acid soluble sulphate content of materials to be stabilized with cement or lime should not exceed 0.25 % calculated as SO_3 if the PI exceeds 8 or if the percentage passing 0.002 mm exceeds 12. A maximum of 1.0 % is permissible for materials with lower PIs and clay contents.

(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.

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 or significant swell and the soaked UCS is not less than 80% of the UCS design strength (after a similar curing period) the materials should be satisfactory. This test may be used instead of the tests for sulphate requirements or as an additional guide.

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APPENDIX A

NEW TMH 1 NOMENCLATURE

APPENDIX A: DETERMINATION OF FIELD WORKING TIME

1. SCOPE

This protocol covers the laboratory procedure used for the determination of the maximum allowable working time for a cement stabilized granular pavement material. This should be carried out for each material and cement combination to provide an estimate of the likely time available for construction of the layer. Recent experience in Australia has shown that the UCS is a better indicator than MDD for working time limitations, but it may be prudent during this data collection process to assess the influence of both the UCS and MDD for selected materials. The protocol described can be adapted for the latter. The UCS may be substituted by the Indirect Tensile Strength (ITS) or complemented with the ITS as necessary.

2. DEFINITIONS

Maximum Allowable Working Time for UCS

The working time for unconfined compressive strength is defined as the time measured from the commencement of the addition of the stabilising agent to the compaction of the stabilised material, which corresponds to 80% of the mean value of three determinations of UCS, for samples compacted one hour after incorporation of the stabilising agent." The specified temperature is:

- May to September 10 to 15°C (or the maximum anticipated air temperature)
- October to April 20 to 25°C (or the maximum anticipated air temperature)

Maximum Allowable Working Time for MDD

The working time for maximum dry density is defined as "the time measured from the commencement of the addition of the stabilising agent to the compaction of the stabilised material, which corresponds to 97% of the mean value of three determinations of maximum dry density, for samples compacted one hour after incorporation of the stabilising agent.

All samples shall be cured in a loose condition in airtight containers at 23 ± 2 °C (or anticipated field air temperature).

3. APPARATUS

a. For grading - as detailed in SANS 3001-GR1.

b. For unconfined compressive strength - as detailed in SANS 3001-GR50 to 53.

c. For maximum dry density and optimum moisture content as detailed in SANS 3001-GR31.

d. For indirect tensile strength – as detailed in SANS 3001-GR54. (Can be substituted for UCS in the following text)

4. MATERIAL SELECTION

a. Obtain a representative sample of the material to be used on the road.

b. Prepare and precondition the material in accordance with Clauses 3.1 and 3.2 of SANS 3001-GR1.

Note: Obtain sufficient material to determine the maximum dry density, optimum moisture content and for the preparation of 12 UCS moulded test samples.

5. PROCEDURE

5.1 General

If maximum allowable working time is to be determined for construction being carried out from October to April inclusive, the test shall be performed between 20 to 25 °C.
If maximum allowable working time is to be determined for construction being carried out from May to September inclusive, the test shall be performed between 10 to 15 °C.

However, if it is envisaged that the working temperature will be higher than 25 °C, then the test shall be carried out at that temperature.

5.2 Maximum Allowable Working Time for UCS

5.2.1 Mix sufficient quantity of the material with the design cement content for the material for the determination of maximum dry density and optimum moisture content according to SANS 3001-GR31. The maximum size of the material shall be 37.5mm, with no compensation for any oversize material.

5.2.2 Place the mixed material in sealed plastic bags and allow to stand for one hour at the required temperature (see 5.1 above).

5.2.3 After 1 hour break up the cured material over a 37.5mm screen and recombine material passing and retained on the screen.

5.2.4 Determine the maximum dry density and optimum moisture content of the recombined material in accordance with SANS 3001-GR31.

5.2.5 Mix sufficient quantity of material with the design percentage of the cement to carry out the Unconfined Compressive Strength (UCS) tests in accordance with SANS 3001-GR50 to 53.

5.2.6 Place the mixed material in sealed plastic bags and allow to stand for one hour at the required temperature (see 5.1 above).

5.2.7 After the 1 hour standing time break up the cured material over a 37.5mm screen and recombine material passing and retained on the screen.

5.2.8 Add water if necessary and mix the material to achieve a laboratory moisture ratio of 95% to 105% of OMC.

5.2.9 Determine the UCS of the material in accordance with SANS 3001-GR53 and the following:

a. Compact the specimens in accordance with method SANS 3001-GR50. Complete compaction of both specimens within 30 minutes of mixing in step 5.2.8.
b. Cure the compacted test specimens for 7 days at the required temperature.
c. On completion of curing, immediately perform the procedure for compression testing as described in SANS 3001-GR53.

5.2.10 Repeat steps 5.2.5 – 5.2.9 for 2, 4, 8, 12 and 24 hours standing time after addition of the cement.

5.2.11 Plot UCS versus standing time. Draw the line of best fit to the points and determine, to the nearest hour, the maximum allowable working time for the cement (see Figure B1 as an example).

Note: All reference to compaction and strength testing in this protocol makes use of the proposed new method using material screened at 26.5 mm with no added back crushed oversize or other compensation for oversize material.

5.3 Maximum Allowable Working Time for MDD

5.3.1 Mix sufficient quantity of the material with the design cement content for the material for the determination of maximum dry density and optimum moisture content according to SANS 3001-GR50 and 51. The maximum size of the material shall be 37.5mm, with no compensation for any oversize material.

5.3.2 Place the mixed material in sealed plastic bags and allow to stand for one hour at the required temperature (see 5.1 above).

5.3.3 After 1 hour break up the cured material over a 37.5mm screen and recombine material passing and retained on the screen.

5.3.4 Determine the MDD of the material in accordance with method SANS 3001-GR31.

5.3.5 Repeat the process to determine the MDD after conditioning for 2, 4, 8, 12 and 24 hours after addition of the cement.

5.3.6 Plot MDD versus standing time. Draw the line of best fit to the points and determine, to the nearest hour, the maximum allowable working time for the cement.

6. REPORT

Report the following information:

6.1 The type and amount of cement used, including SANS type, make and brand.

6.2 The maximum allowable working time for the cement to the nearest hour (in terms of UCS/ITS and/or MDD).

6.3 The temperature range at which the value was determined.

6.4 The working time will be the lesser of the times determined for UCS, ITS or MDD.

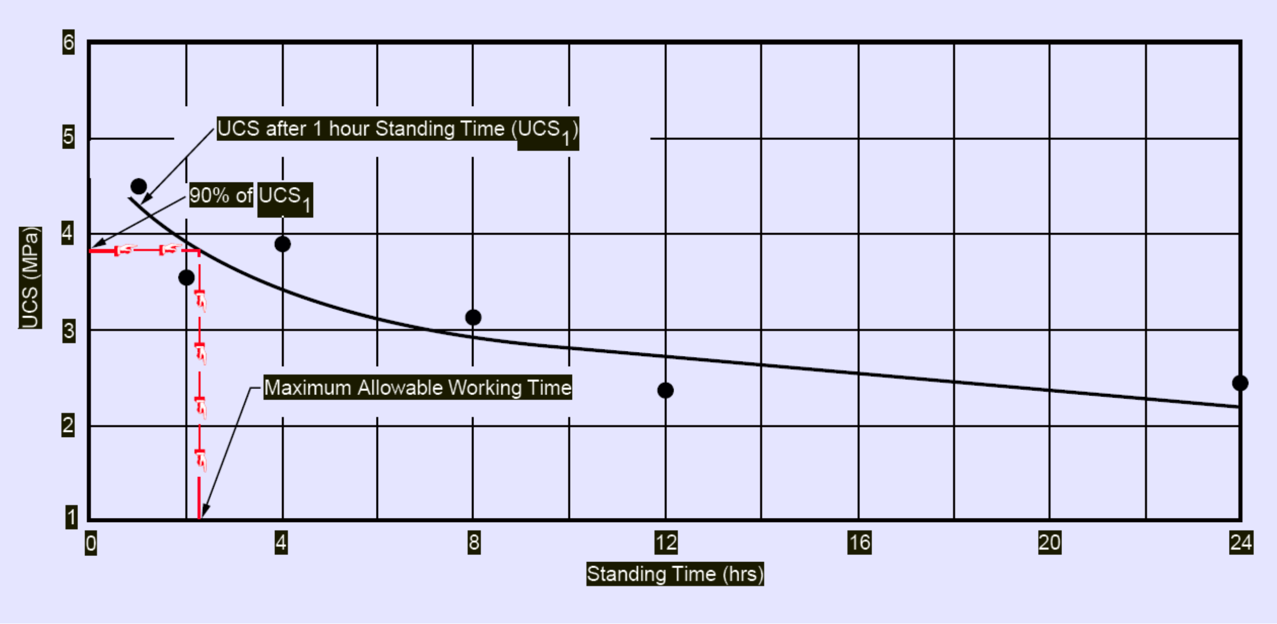

Figure B1: Unconfined Compressive Strength versus Standing Time (can be modified for ITS and/or MDD)

APPENDIX B: INITIAL CONSUMPTION OF STABILIZER TEST

A.1 Method

Determination of the initial consumption of stabilizer in soil (gravel ICS)

A.2 Scope

This method tests the construction material as a whole, and not only the -0,425mm fraction. The objective of the test is to control the pH of chemically stabilized materials in order to allow the possible formation of cementitious materials and to ensure the stability of the reaction products. During the initial testing, lime would normally be used for the ICS test. Once the type of stabilizer to be used on the project has been decided, this stabilizer will be used for all further testing for that project.

A.3 Equipment

1. Balance (accuracy : 0.1 gram)
2. Calibrated pH meter (accuracy : 0.02 units)
3. 5 plastic beakers (150 ml, tall)
4. 200 ml glass beaker
5. Distilled water
6. Calcium hydroxide or cement
7. Water bottle
8. Spatula or pallet knife
9. Soft tissues
10. Distilled water
11. Sample splitter (25 mm)
12. Jet bottle (for spraying electrode)
13. Drying oven (105 – 110°C)
14. Thermometer (accuracy 0.5°C)

A.4 Method

Sample preparation

The test is carried out on material finer than 19 mm. Any oversize material is lightly crushed to pass the 19 mm sieve. The samples are dried overnight in the oven at 105-110°C. The samples are then reduced to 200 g quantities using a sample splitter and placed into 150 mℓ or larger plastic containers.

Testing procedure

Since most materials require between 2 and 5 per cent stabilizer, it is advisable to set up six beakers with stabilizer percentages of 1, 2, 3, 4, 5 and 6 per cent respectively, of the dry soil mass. Weigh the lime to the nearest 0.1 g and add to the soil. Mix the soil and dry lime.

Slightly oversaturate the samples with CO_2-free distilled water. The material shall be judged oversaturated when the pores of the material are water-filled and free water can be observed on the surface of the mix. The surface particles need not be submerged completely.

Mix the soil-stabilizer and water until there is no evidence of dry material on the bottom of the beaker. Mix for a minimum time of 30 seconds.

Mix for 30 seconds every 10 minutes.

After one hour, measure the pH of the paste by inserting the pH electrode gently into a hole made in the material with the spatula to a depth of about 20 to 30 mm and gently covering this part of the electrode with the material. Tap the beaker gently to ensure contact between the electrode and material.

Record the pH of each sample to the nearest 0.02 of a unit. The lowest percentage stabilizer at which the soil paste remains constant is the saturation stabilizer content of this particular material. The saturation pH of lime at 25°C is usually 12.4.

A.5 Recording of Results

The results are recorded to the nearest 0.5 per cent stabilizer required to produce the maximum pH in the paste.

A.6 Preparation of pH Meter

The calibration procedures of the manufacturer of th pH meter must be strictly followed. The temperature-asymmetry and slope adjustment are of particular importance. The efficiency of the electrode must be assured.

APPENDIX C: PREDICTION MODELS FOR VARIOUS PROPERTIES

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 known[28]. Equations (1), (2) and (3) are recommended, since the use of the flexural strength is preferred.

Cement-treated crushed stone: $E = 8\,\sigma_b + 3\,500$ .... (1)

Cement-treated natural gravel: $E = 10\,\sigma_b + 1\,000$ .... (2)

Lime-treated natural gravel: $E = 17\,\sigma_b - 900$ .... (3)

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

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

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

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

Cement-treated crushed stone: $\varepsilon_b = 145$ .... (7)

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

Lime-treated natural gravel: $\varepsilon_b = 89{,}9 + \dfrac{182\,870}{E}$ .... (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.