

NATURAL, LOW COST ROAD  
CONSTRUCTION MATERIALS: THEIR  
OCCURRENCE AND STABILIZATION

by

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## PREFACE

This book is based on research carried out by the author, at the National Institute for Transport and Road Research (NITRR) of the South African Council for Scientific and Industrial Research (CSIR), Pretoria, Republic of South Africa during the research era of the establishment. Permission has been granted by the chief director of that institute to compile a text which brings together the results of research spread over a number of reports and papers during the past 27 years and to create a guide for soil stabilization work.

Its aim is to provide sufficient knowledge of soil/lime reactions to make stabilization design less of an art and more of a technical process.

I should like to thank the CSIR for assistance in producing the manuscript.

My thanks are also extended to all my colleagues in materials engineering for their interest and criticism.

The interpretation of test results and experiments is the sole responsibility of the author and does not necessarily coincide with that of the NITRR.

KAC  
PRETORIA, 1988



## TABLE OF CONTENTS

PREFACE	iii
List of tables	xi
List of figures	xii
List of plates	xvii
1 INTRODUCTION	1
2 DEPOSITS OF NATURAL CONSTRUCTION MATERIALS FOR ROADS	3
2.1 Introduction	3
2.2 Classification of materials	4
2.3 Occurrence of materials deposits	4
2.4 Materials assessment of stratigraphic units	11
2.5 Conclusions	17

3	SOIL STABILIZATION	19
3.1	Introduction	19
3.1.1	REVIEW	19
3.1.1.1	Soil mechanical phase	21
3.1.1.2	Quality of stabilizer phase	25
3.1.1.3	Soundness of aggregate and soil phase	25
3.1.2	PROJECTION OF SOIL/LIME REACTION PHASE	27
3.2	Possibly reactive compounds in stabilized soil	28
3.2.1	ROAD LIME	35
3.2.1.1	Chemical reaction of lime and water	36
3.2.1.2	Technical properties of lime as specified	36
3.2.1.3	Further technical properties of lime road lime	36
3.2.1.4	Aspects of particle size	43
3.2.1.5	Hydration and volume change	44
3.2.1.6	Lime carbonation	47
3.2.1.7	Economic aspects of lime	48
3.2.1.8	Conclusions	52
3.2.2	AMORPHOUS SILICA	54
3.2.3	AMORPHOUS ALUMINA	57

3.2.4	CLAY	60
3.2.4.1	Chemical properties	60
	(a) Kaolinite type soils	61
	(b) Montmorillonite type soils	68
	(c) Clay in gravels	74
3.2.4.2	Physical properties	88
3.2.5	CEMENT AND POZZOLANITY OF SOILS	95
3.2.5.1	Description of pozzolans	95
3.2.5.2	Description of pozzolanity	102
3.2.5.3	Definition of pozzolanity	105
3.2.5.4	Classification of pozzolans	105
3.2.5.5	Conclusions	111
3.2.6	ORGANIC MATTER IMPURITIES	111
3.2.6.1	Sugar type	111
3.2.6.2	Coal and plant matter	112
3.2.6.3	Conclusions	113
3.3	Soluble salts and high acidity	114
3.3.1	SOLUBLE SALTS	114
3.3.2	HIGH ACIDITY	114

3.4	Stabilization reactions	116
3.4.1	INTRODUCTION	119
3.4.2	SOIL MODIFICATION	121
3.4.2.1	Mechanical modification	121
3.4.2.2	Chemical modification	122
3.4.2.3	Soil modification in time	127
3.4.2.4	Compaction delay and seasoning	129
3.4.2.5	Conclusions	134
3.4.3	SOIL CEMENTATION	135
3.4.3.1	(Portland) Cement-cementing	136
3.4.3.2	Carbonation cementing	136
3.4.3.3	Hydraulic soil/lime reactions	141
3.4.3.4	Conclusions	145
3.5	Proportional analysis of strength factors	146
3.5.1	COMPACTION STRENGTH	146
3.5.2	CARBONATION STRENGTH	146
3.5.3	CEMENT REACTION STRENGTH (HYDRAULIC CALCIUM SILICATE HYDRATE STRENGTH)	148
3.5.4	CLAY REACTION STRENGTH (HYDRAULIC CALCIUM ALUMINATE AND SILICATE HYDRATE STRENGTH)	148



3.6	Guide to testing in stabilization work	148
3.6.1	INTRODUCTION	148
3.6.2	QUALITY OF LIME	150
3.6.3	OBJECTIVE: PLASTICITY CORRECTION	150
3.6.4	OBJECTIVE: CARBONATION STRENGTH	150
3.6.5	OBJECTIVE: POZZOLANIC HYDRAULIC SOIL/LIME REACTION STRENGTH	150
3.6.5.1	pH determination in lime stabilized soil	154
3.6.6	OBJECTIVE: ORGANIC-MATTER IMPURITIES	162
3.6.6.1	Introduction	162
3.6.6.2	Test method	165
3.6.6.3	Conclusions	174
3.7	The time factor in soil stabilization	174
3.7.1	OBJECTIVE OF FIELD EXPERIMENT	174
3.7.2	LAYOUT OF EXPERIMENT	175
3.7.2.1	Design	175
3.7.2.2	Materials	175
3.7.2.3	Construction	181
3.7.2.4	Test programme	181
3.7.2.5	Traffic	181

3.7.3	FAILURE OF UNSTABILIZED SECTION	181
3.7.4	PRELIMINARY CONCLUSIONS	190
3.7.5	REVIEW OF STABILIZATION EXPERIMENT (1982)	191
3.7.5.1	Test programme	191
3.7.5.2	Strength probe	191
3.7.5.3	Stabilization reactions and cracks	192
3.7.6	CONCLUSIONS	194
4	GLOSSARY	195
5	REFERENCES	210
6	INDEX	222

## List of tables

2.1	Specifications for determining the suitability of natural materials for road layers (according to TRH4).	5
2.2	Specifications for determining the suitability of natural materials for road layers (according to TRH4).	6
2.3	Land form groups that are considered important to the formation of materials deposits.	9
2.4	Legend for road-construction materials assessment tables.	12
2.5	Material assessment: K3 Beaufort Series.	18
3.1	Application of cement- and lime-stabilization in South Africa (mainly in Transvaal and Natal).	20
3.2	1971 standards for bases for asphalt pavements (untreated material).	23
3.3	Comparison of CBR and UCS.	32
3.4	Technical properties of various limes according to SABS 824: 1967 (as amended).	38
3.5	Undesirable coarse fraction particles in dry-processed carbide lime.	40
3.6	Technical properties of road lime.	41
3.7	Approximate chemical composition (% by mass) of Transvaal dolomite.	51

3.8	Amorphous silica in soil.	56
3.9	Technical properties of Silverton soil.	62
3.10	Indicator test results of Silverton soil treated with 10 % of different stabilizers.	67
3.11	Technical properties of black clay.	69
3.12	Solubility of selected compounds in g per 100 cm <sup>3</sup> of cold water.	75
3.13	Indicator test results of black clay treated with 10 % of different stabilizers.	76
3.14	Determination of the initial consumption of lime (ICL).	83
3.15	Total lime and silica content of some pozzolanic materials.	107
3.16	Weathering of pyrite in waste coal.	117
3.17	Salt concentration in road layers which consist of waste coal.	117
3.18	Effect of lime on the liquid limit (LL) of South African soils.	126
3.19	Development of carbonation strength in lime-stabilized, washed sand.	140
3.20	Abstract of reaction mechanisms in stabilized soil.	149
3.21	Effect of increasing sugar concentrations on the solubility of lime.	173

3.22	Construction record of an experimental section with one follow-up test.	177
3.23	ICL of a dolerite weathering succession from Morgenzon, Transvaal.	180
3.24	Gravel ICL of weathered dolerite from borrow pit as used in stabilization experiment (collected 1982).	180
3.25	Traffic counts on P48-2.	188
3.26	CBR of road layers of Morgenzon experiment as determined by the 60° Dynamic Cone Penetrometer (DCP) test.	193

## List of figures

2.1	Criteria used for the grouping of land forms.	7
3.1	Random reaction of different gradings of waste carbide lime and Marvello road lime with Silverton soil.	30
3.2	Regression analysis of strength development in Marvello and carbide lime-stabilized black clay (OMC, MDD).	31
3.3	Attempt at a best-fit curve of the (indicator) ICL of black clay and Silverton soil.	34
3.4	pH of saturated calcium hydroxide solution at different temperatures and cementation pH-ICL.	37
3.5	Dispersion of (waste) carbide lime and Marvello road lime in water during 10 minutes.	42
3.6	Clay reactivity of lime.	45
3.7	Initial consumption of lime (ICL) of amorphous silica in terms of saturation pH after 1 hour.	55
3.8	The effect of pH on the zeta potential of silica and alumina particles.	58
3.9	The arrangement of the $\text{SiO}_2$ molecule as tetraeder with a residual electrical potential equivalent to two oxide ions.	59
3.10	Solubility of Si, Al and Fe.	59
3.11	Particle-size distribution: Silverton soil.	63

3.12	XRD diagram of natural Silverton soil.	64
3.13	Cation and anion reactions of Silverton soil after treatment with 10 % of different additives in terms of liquid limits (4-day curing).	65
3.14	Cation and anion reactions of Silverton soil after treatment with 10 % of different additives in terms of liquid limits (4-day curing).	66
3.15	Particle-size distribution: Black clay.	70
3.16	XRD diagram of natural black clay.	71
3.17	Cation and anion reaction of black clay after treatment with 10 % of different additives in terms of liquid limit (4-day curing).	72
3.18	Cation and anion reaction of black clay after treatment with 10 % of different additives in terms of liquid limit (4-day curing).	73
3.19	ICL of different quantities of ferricrete.	81
3.20	ICL of different gradings in basic igneous rock.	82
3.21	ICL determinations of differently ground basecourse.	85
3.22	UCS of lime-stabilized Silverton soil, ground to $\leq$ 425 micron (OMC, MDD).	90
3.23	UCS of lime-stabilized black clay ground to $\leq$ 425 micron (OMC, MDD).	92

3.24	Diagram of cement formation.	96
3.25	Lime reaction of PFA in comparison with slagment UCS of different PFA-lime mixes.	103
3.26	XRD diagram showing amorphous matter humps in inert and reactive PFA and soil.	108
3.27	Critical pH of hydraulic cement reactions in H <sub>2</sub> O saturated, lime-stabilized black clay in terms of 90-day cured UCS.	110
3.28	Salt damage to roads.	115
3.29	Development of pH in a 1:1 mix of colliery spoil and ferricrete and its influence on the mass of a single ferricrete nodule.	118
3.30	Montmorillonite-type clay reaction. The effect of curing on the LL, PL, PI and LS of carbide lime-stabilized weathered norite.	123
3.31	Kaolinite-type clay reaction. The effect of lime treatment on weathered Pretoria shale.	124
3.32	Kaolinite-type clay reaction. Atterberg limits and linear shrinkage of OP cement-stabilized Silverton soil.	125
3.33	Determination of Ca(OH) <sub>2</sub> reacting with Silverton soil (0,8 %) and black clay (2,8 %) according to the sucrose method.	128
3.34	Atterberg limits of 18,5 % carbide lime-stabilized black clay over a period of 11 months.	130



3.35	UCS after delayed compaction of 22 % lime-stabilized black clay and 3 % lime-stabilized Silverton soil (OMC, MDD at mod AASHTO).	132
3.36	UCS after delayed compaction of 18 % lime-stabilized Silverton soil and 30 % lime-stabilized black clay (28-days, OMC, MDD).	133
3.37	UCS of lime-stabilized black clay and Silverton soil in the absence of air (water-saturated).	138
3.38	Strength gain of lime-stabilized Silverton soil at MDD with organic impurities (sugar) added.	144
3.39	Carbonation and hydraulic aluminate and silicate and Ca-Si-hydrate unconfined compressive strength (OMC, MDD, 50 x 100 mm cylinder, 28-day cured) in different soils.	147
3.40	Recording form for ICL test results.	157
3.41	Standard graph of different lime types according to CAS A43 (1974).	164
3.42	Graph of titration against g of lime.	169
3.43	Graph of titration against g of lime in sample.	169
3.44	Section of layout of experimental road.	176
3.45	Sketch of soil profile in quarry, Morgenzon.	179
3.46	Particle-size distribution of material used in the Morgenzon stabilization experiment.	182

3.47	Particle-size distribution of material used in the Morgenzon stabilization experiment.	183
3.48	Particle-size distribution of material used in the Morgenzon stabilization experiment.	184
3.49	Particle-size distribution of material used in the Morgenzon stabilization experiment.	185
3.50	Particle-size distribution of material used in the Morgenzon stabilization experiment.	186
3.51	Particle-size distribution of material used in the Morgenzon stabilization experiment.	187
3.52	Sketch of section 3 and 4 of Morgenzon experiment.	189

**List of plates**

- |     |  |     |
|-----|--|-----|
| 3.1 | +850 micron fraction (0,4 %) of dry-processed carbide lime.      | 39  |
| 3.2 | Electron microscope photographs of carbide lime.                 | 46  |
| 3.3 | Etching of pH glass electrode and intense scratching of surface. | 161 |

## 1 INTRODUCTION

In the light of experience gained during the past 28 years, two aspects of road construction work may be classified as "grey areas". These are:

- (a) the occurrence of deposits of low-cost construction materials and
- (b) the upgrading of substandard construction materials by a process of soil stabilization.

The first problem we encounter is the definition of 'construction materials'. Construction materials are usually judged by the performance of the roads for which they have been used; in other words by an assessment of their technical properties. They are documented in materials specifications, which are in turn the product of many years of experience by road authorities.

Generally, a material is required which can be worked well, is strong enough to stand the stress of traffic for a reasonable time and is available economically. Transferring these requirements into materials terminology, one can say that road construction materials must be a slightly plastic gravel, not too weathered, and available near the construction site. There are, of course, exceptions to this rule: for example, the materials of a crushed stone base layer. Gravel is the weathering product of fresh rock and is generally produced by natural forces. Depending on the type of road to be constructed, suitable gravel may be found as a surface deposit or may require quarrying; or may not be available economically at the construction site since it has been buried under a mass of in situ weathered material or transported material.

The problem now becomes one of comparing the cost aspects of transporting suitable material over long distances, of quarrying suitable materials from deep deposits or improving unsuitable material to such an extent that it becomes a suitable material. This last-mentioned alternative has, in the course of time, become known as soil stabilization.

Deposits of natural, suitable road construction material are of importance in any low-cost road construction work, because they are the cheapest construction materials available. Of equal importance is a knowledge of the interaction of natural, unsuitable material with stabilizing agents such as lime in order to improve naturally unsuitable construction material. The most widely used stabilizing agents in road work are Portland cement, lime and mixtures of these or mixtures with one or other pozzolan such as suitable pulverized fuel ash (PFA) or ground blast-furnace slag.

In the chapters that follow we discuss the occurrence of materials deposits, observe how materials may react upon stabilization, consider suitable test methods and give guidance on their application in stabilization work.

## 2 DEPOSITS OF NATURAL CONSTRUCTION MATERIALS FOR ROADS

### 2.1 Introduction

The search for natural, low cost road construction materials usually starts with a listing of the location of deposits of materials potentially suitable for pavement construction. The listing is based on materials specifications. Air photography interpretation and geological interpretation of existing maps with existing borrow pits as point of departure form the basis of such a materials survey. This advice is based on the philosophy that 'regardless of geographic distribution, soils developed from similar parent material under the same conditions of climate and relief are related and will have similar engineering properties' (Belcher, 1943). Since Belcher's statement, a number of data banks have come into being which have applied this relation and used terrain patterns that are more or less rigidly classified according to geology, climate and relief etc as technical information units, in an attempt to conserve the knowledge on materials deposits for later use. However, none of these data banks achieved any success.

One such terrain data bank, established in 1971, is the South African National Data Bank for Roads. It is based on the work of Brink, Partridge, Webster and Williams (1968). An attempt was made to increase its technical appeal by emphasizing materials data rather than terrain data (Clauss, 1980), but this new approach met with little success and was abandoned just as the earlier terrain data bank had been.

However, some two thousand borrow pits were processed during the data bank's period of operation and it is believed that the analysis of such a vast store of data may lead to a practical and systematic body of knowledge on the occurrence of deposits of low-cost, natural road-construction materials that can readily be applied.

## 2.2 Classification of materials

The sources of materials were investigated on a routine basis by the South African road authorities and the results were given to the National Data Bank for Roads for further processing. The materials were then classified according to their technical properties in relation to material specifications valid at the time (TRH4, NITRR, 1971). For easy reference, a summary of these specifications is given in Tables 2.1 and 2.2.

## 2.3 Occurrence of materials deposits

In addition to their technical properties, materials deposits were described according to the land forms in which the borrow pit (deposit) was found.

It should be remembered in this context that land forms are often applied parameters for a wide range of environmental conditions such as run-off, erosion, sedimentation, depth of weathering and, more recently, also terrain classification (Brink et al, 1968) and after some trial and error it was decided to use four land form groups for the description of materials localities. Figure 2.1 presents an idealized cross-section of terrain which illustrates the position of these land forms in any terrain. The four land form groups are defined as follows:

### (a) High position

High-position land forms are situated above (and including) the constant portion of a slope (convex slope type).

TABLE 2.1  
Specifications for determining suitability of  
natural materials for road layers (according to TRH4).

Layer	Test	Quantity	Limiting values
Bases	Strength	CBR (98 % comp) CBR swell	$\geq 80 \%$ $\leq 0,5 \%$
	Atterberg limits	Liquid limit Plasticity index Linear shrinkage	$\leq 25$ $\leq 6$ $\leq 3$
	Soil mortar grading	Coarse sand Fine sand Silt and clay	35-40 % 25-40 % 15-30 %
	Aggregate grading	Sieve 37,5 mm 26,5 mm 19,0 mm 13,2 mm 4,75 mm 2,00 mm 0,425 mm 0,075 mm 0,053 mm	100 % 82-95 % 70-85 % 58-75 % 34-55 % 22-40 % 10-25 % 5-14 % 4-12 %
Subbases	Strength	CBR (95 % comp) CBR swell	$\geq 45 \%$ $\leq 0,5 \%$
	Atterberg limits	Plasticity index	$\leq 10$ (if $> 30 \%$ pass 2 mm sieve)  $\leq 12$ (if $< 30 \%$ pass 2 mm sieve)
Selected subgrades	Strength	CBR (93 % comp) CBR swell	$\geq 10 \%$ $\leq 1,5 \%$
	Atterberg limits	Plasticity index	$\leq 12$
Lower layers of subgrade	Strength	CBR (90 % comp)	$\geq 7$
Fill	Strength	CBR (90 % comp)	$\geq 3$



TABLE 2.2  
 Specifications for determining the suitability of  
 natural materials for road layers (according to TRH4).

Layer	Test	Quantity	Limiting values
Bases (cement- treated only)	Strength	CBR (98 % comp)	$\geq 50 \%$
	Atterberg limits	Liquid limit Plasticity index Linear shrinkage	$\leq 25$ $\leq 6$ $\leq 3$
	Aggregate grading	Sieve 37,5 mm 26,5 mm 19,0 mm 13,2 mm 4,75 mm 2,00 mm 0,425 mm 0,075 mm 0,053 mm	100 82-95      100 70-85      84-100 58-75      71-89 34-55 or 42-63 22-40      27-47 10-25      12-28 5-14      5-16 4-12
Subbases (cement- and lime- treated)	Strength	CBR (95 % comp)	$\geq 150 \%$ (cement- treated) $\geq 70 \%$ (lime- treated)
	Atterberg limits	Plasticity index (if $> 30 \%$ pass 2 mm sieve) $\leq 10$ (if $< 30 \%$ pass 2 mm sieve) $\leq 12$	

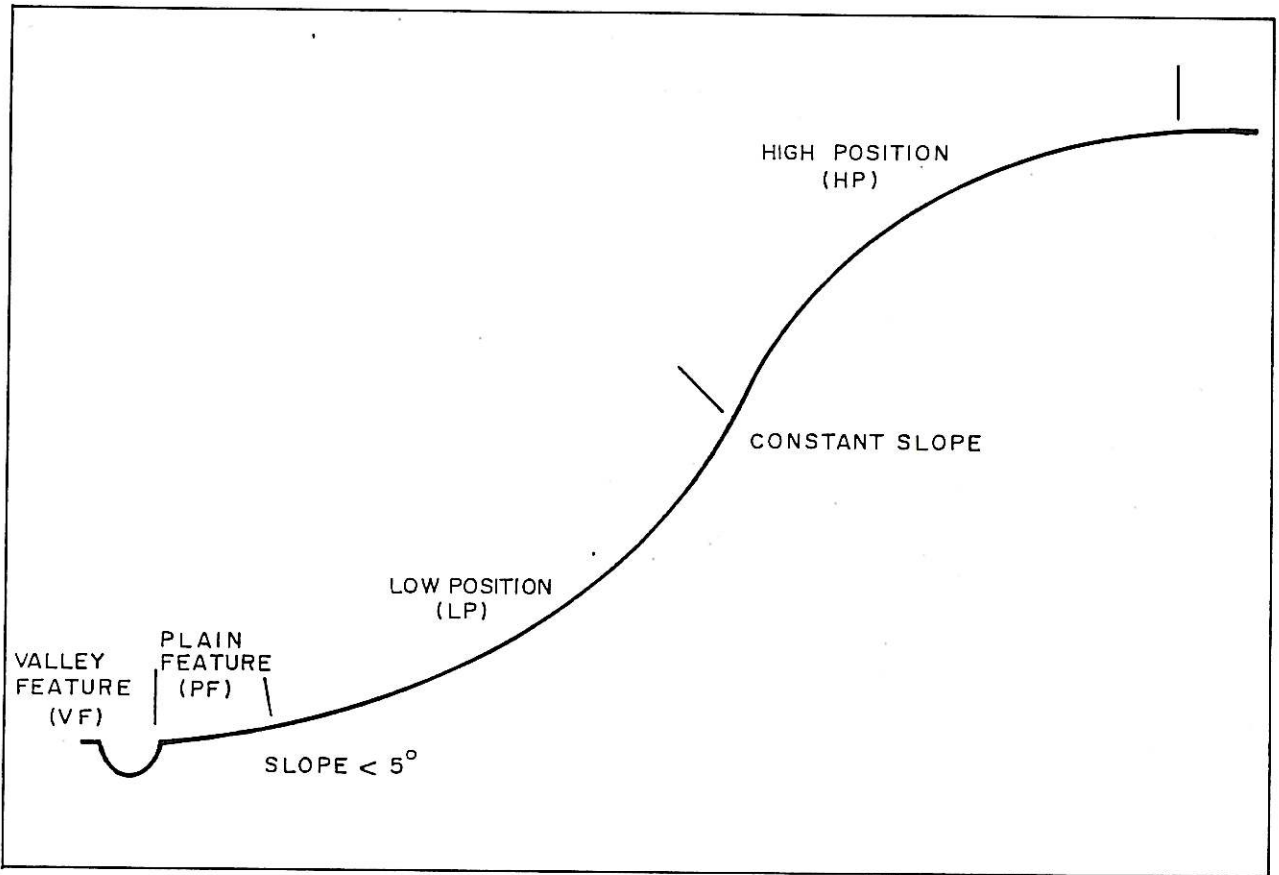


FIGURE 2.1

*Criteria used for the grouping of land forms.*

(b) Low position

These comprise mainly concave slope-type land forms and are often related to water seepage.

(c) Plain features

These incorporate pediments and plains below pediments with a slope of less than  $5^{\circ}$ .

(d) Valley features

These are land forms below plains and pediments and may be described generally as drainage features.

A useful summary of terms for land forms is given in Table 2.3.

Practical considerations demand that land forms be of a similar size to borrow pits and that the land forms quoted above (see Table 2.3) should have a minimum extent in the order of some 100 m x 100 m. This is equivalent to a borrow pit yielding some 20 000 m<sup>3</sup> of material to a depth of 2 m.

The identification of land forms is not always easy. Particularly in bush terrain or in flat terrain when vegetation camouflage smooths the relief or when the relief itself is poorly developed, problems may be encountered. However, many of these uncertainties may be overcome by a careful stereoscopic study and the following observations have been recorded in order to indicate the magnitude of error involved in land form identifications.

It is assumed in the following that the stereoscopic air photography interpretation and classification of land forms is correct within a

TABLE 2.3  
Land form groups that are considered important  
to the formation of materials deposits

Land form group	Land form No.*	Description of land forms
High position	1.1	Hillcrest
	1.3	Ridge crest
	1.4	Plateau crest
	1.8	Bump
	2.1	Cliff
	3.2	Convex slope
	3.4	Constant slope
	3.6	Ledge
Low position	3.1	Talus slope
	3.3	Concave slope
	5.1	Gulley head
	5.2	Gulley
Plain features	3.5	Pediment
	4.1	Plain
Valley features	3.8	Pan
	5.3	Pan side
	5.4	Pan floor
	5.5	River terrace
	5.6	River
	5.7	Flood plain
	5.11	River channel

\*The land form numbers given are those of the practical residue as indicated. The complete list is much larger.

margin of error of 10 %. This margin of error appears reasonable because a 10 % error was still found in the classification of land forms after a repeat air photo interpretation of the same area and after a time lapse of some 6 months. In comparison, the error of the first field exercise was some 60 %, which dropped during the second exercise to some 25 %.\*

Another factor which plays an important role in site prediction in the search for construction materials is the distance from one borrow pit to another. In other words, no materials survey ever represents a true distribution of construction-quality materials, even along communication corridors. The borrow pits eventually presented as possible sites are those found at a convenient distance from one other. It is again a matter of large numbers yielding in due course, at least a knowledge of the reliably true distribution of construction materials in communication corridors.

Strict statistical treatment would require some 50 individual approaches in order to establish a 'true situation'. The average number of test holes is estimated at 5 per borrow pit with 2 to 3 horizons in each test pit. It is consequently assumed that some 40 to 60 borrow pits are represented by approximately 500 test results. Note also that 'true materials situations' along road routes most probably tend to reflect the terrain preferred for road corridors (TRH2, NITRR, 1971) and may differ from the terrain outside such corridors.

It must therefore be kept in mind that each set of materials data (technical properties and locality) contains - a priori - four different types of error, ie

---

\*Fieldwork seems to indicate that comparatively little experience results in a sharp reduction of the margin of error, while thereafter much experience will be required to reduce it further. A margin of between 10 and 20 % should therefore be accepted in the application and extrapolation of land form classification although the accumulation of large quantities of data from a number of different sources may improve the position considerably.

- . individual classification error (individual operator);
- . general classification error (several operators);
- . distance selection error (specified distance);
- . true situation error (communication corridor).

It will also be recognized from the definition of these errors that all of them, except the individual classification error which is considered equal to the residual error, may be neutralized if large quantities of data become available.

## 2.4 Materials assessment of stratigraphic units

Although a large quantity of data was collected, the total was still too small to make possible meaningful analyses of all the proposed parameters, eg parent rock, climate, relief, erosion cycle, lithostratigraphy, physiography etc\*. It was therefore decided to use the stratigraphic units given in the legend of the 1 : 1 000 000 geological map of the Republic of South Africa (1970) as the basis of the materials assessment tables. This legend is presented in Table 2.4 for easy reference.

However, the stratigraphy of South Africa has since been reclassified between 1970 and 1980. The decisions and recommendations of the South African stratigraphy committee have been published in Handbook 8 (1980) of the Geological Survey of the Republic of South Africa. The new stratigraphic units were used in the most recent 1 : 1 000 000 geological map of South Africa, which was published in 1984. The reader is therefore advised to study the above references carefully since they provide the link between the old and the new lithostratigraphy.

The number of test results processed for each stratigraphic unit is summarized in column 3 of Table 2.4. The materials assessment tables are

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\*If these parameters have been meaningfully established by relevant surveys which is not always the case.

TABLE 2.4  
Legend for road-construction materials assessment tables.

List of terms for the naming of geological (stratigraphical) units according to the 1:1 000 000 geological map of the RSA (1970), with a code number as used in data banking.

Symbol	Code No.	Test results	Stratigraphical units	Lithology
-	1	300	Tertiary to Quaternary	Conglomerate, limestone, sandstone, marl
Cr2	2	-	Upper Series Cretaceous System	Sandstone, shale, limestone, marl
Cr1	3	33	Lower Series Cretaceous System	Conglomerate, sandstone, shale, limestone
K4b	62	-	Lebombo Stage	Pyroclast, andesite, rhyolite, syenite, granite conglomerate
K4v	5	22	Lebombo Stage	Rhyolite, dacite, pyroclasts
K4v	6	375	Drakensberg Stage	Basalt, limburgite, pyroclasts, minor sandstone
K4	7	104	Molteno, Red Beds, Cave Sandstone Stages	Sandstone, shale, mudstone, marl coal
K3	8	2 321	Beaufort Series	Shale mudstone, sandstone, limestone, coal
K2	9	7 124	Ecca Series	Shale, sandstone, grit, coal
K2	10	1 163	Dwyka Series	Tillite, shale, sandstone
C3	11	148	Witteberg Series	Quartzite, shale
C2	12	357	Bokkeveld Series	Shale, sandstone
C1	13	418	Table Mountain Series	Quartzite, shale, tillite

TABLE 2.4 (cont.)

Symbol	Code No.	Test results	Stratigraphical units	Lithology
K1	14	-	Klipheuwel Formation	Conglomerate, greywacke, shale
AS3	15	-	Syenite	
AG9	16	95	Cape Granite, Kuboos plutons etc	Granite, syenitic rock, quartz porphyry
N	17	112	Nama System (including Malmesbury, Gango, Numees, Ntingwe, Franschoek Beds)	Quartzite, arkose, limestone, shale, tillite, phyllite, lava, tuff
D	18	-	Damara System (including Kaigas and the lower two series of Gariep)	Tillite, limestone, quartzite, schist, conglomerate, grit
AG8	19	-	Richtersveld Igneous Complex	
Ns	20	-	Nosib System (including Stinkfontein and the upper two series of Gariep)	Phyllite, lava, quartzite, conglomerate, schist, tillite, hornfels
Ko	21	-	Koras Formation	Andesite, acid lava, sediments, pyroclasts
AG7	22	637	Namaqualand and Natal belts of metamorphism and granitization	Migmatite, gneiss, ultrametamorphic rocks
AS2	23	-	Igneous complexes of Pilanesberg, Spitzkop, Roodeplaar, Leeuwfontein etc	Alkaline, basic and ultrabasic intrusives, contemporaneous sediments and carbonatite
W2	25	89	Kransberg Series	Sandstone, siltstone, conglomerate
W1	26	223	Nylstroom Series (including Matsap beds)	Lava, sandstone, conglomerate, siltstone, greywacke
AG6	27	71	Bulai, Singelele, Palala etc plutons	Granite, grandiorite

Continued/...



TABLE 2.4 (cont.)

Symbol	Code No.	Test results	Stratigraphical units	Lithology
AG5	28	1 816	Bushveld Igneous Complex	Several types of granite porphyry
8Y	64	354	Bushveld Igneous Complex	Granophyre
AN3	65	460	Bushveld Igneous Complex	Gabbro, norite etc
Lf	66	461	Bushveld Igneous Complex	Felsite and pyroclasts
AS1	29	-	Palabora Igneous Complex and related intrusives	Carbonatite, pyroxenite, syenite, phoscorite
AG4	30	621	Limpopo belt of metamorphism and granitization	Migmatite, gneiss, ultrametamorphic rock
T3s	32	248	Smelterskop Stage	Quartzite, shale, hornfels, limestone, andesite tuff, conglomerate
T3m	33	542	Magaliesberg Stage	Quartzite, shale, hornfels, lava, pyroclasts, chert, limestone
T3d	34	278	Daspoort Stage	Quartzite, shale, tillite, andesite, jaspilite, limestone
T3t	35	373	Timoball Hill Stage	Quartzite, shale, conglomerate
T2	36	621	Dolomite Series	Dolomite, chert, shale, quartzite, conglomerate, tillite
T1	37	122	Black Reef Series	Quartzite, shale, conglomerate, andesite
V	38	179	Ventersdorp System	Andesite, acid porphyry, pyroclasts, sediments

Continued/ . . .

TABLE 2.4 (cont.)

Symbol	Code No.	Test results	Stratigraphical units	Lithology
Wd5	39	-	Kimberley-Elsburg Series	Quartzite, grit, conglomerate, shale, slate
Wd4	40	-	Main-Bird Series	Quartzite, conglomerate, lava
Wdc3	41	70	Jeppestown Series	Shale, slate, quartzite, andesite, agglomerate
Wd2	42	-	Government Reef Series	Shale, slate, quartzite, grit, conglomerate, tillite
Wd1	43	44	Hospital Hill Series	Shale slate, quartzite, grit
AG3	44	84	Pongola, Mpageni, Salisburykop Mashishimala, Uitloop, Dalmein, Gaberone plutons	Granite
Dr	45	74	Dominium Reef System (including Soetalief, Godwan and Uitkyk Beds)	Conglomerate, quartzite, shale, phyllite, limestone, tillite, lava
AN	46	10	Usushwana Complex and other basic intrusives	Gabbro, ultrabasic rocks, locally granophyre
Mo	48	-	Mozaan Series	Quartzite, shale, lava, conglomerate
Iz	49	-	Insuzi Series (including Nkandla Beds)	Lava, pyroclasts, quartzite, schist
AS	50	-	Bosmanskop Syenite	Syenite
AG2	51	2 201	Northern Cape-Transvaal belt of meta- morphism and granitization	Migmatite, gneiss, ultrametamorphic rocks
AG2	52	-	Ancient gneiss complex in Swaziland	Gneiss

Continued/...

TABLE 2.4 (cont.)

Symbol	Code No.	Test results	Stratigraphical units	Lithology
AG1	53	165	Kaap Valley and Rooiwatergranites	Granite, granodiorite
AN1	54	145	Jamestown and Rooiwater Igneous Complexes; other similar intrusives, probably of various ages	Ultrabasic and basic intrusives and their metamorphosed derivatives
Md	55	94	Moodies Systems	Conglomerate, quartzite, shale, limestone, lava
52	56	24	Fig Tree Series	Greywacke, shale, banded ironstone, volcanics
S1	57	40	Onverwag Series	Basic and acid volcanics, sediments
S	58	64	Kheis and Swaziland Systems (including the Kraaipan, Messina, Mfongosi and Nondweni beds)	Metamorphosed sediments, volcanics and intrusives, probably of various ages

Pedocretes such as ferricrete, calcrete, silcrete etc and/or intrusives such as dolerite etc are included in the tables.

summaries of where materials have been found in the past and represent a relation of frequency versus occurrence.

Table 2.5 presents an example of a materials assessment table. The land forms in which any construction materials were found are listed in the left-hand, vertical column. They are grouped into sub-units according to Figure 2.1. The materials quality found is listed on top from left to right. Each material tested is now recorded according to the technical properties determined and their occurrence in land forms.

The summaries in the example (Table 2.5) at the bottom and in the last column on the right-hand side indicate, then (for the assessment of the 'Beaufort Series'):

- (a) that the material tested was mainly of natural subbase quality (1 645 tests);
- (b) that approximately 75 % (1 218 tests) of all construction materials were found in high-position land forms (Figure 2.1); and
- (c) that 50 % (824 tests) of these materials were concentrated in land form 3.2 (convex slope, according to Table 2.3).

## 2.5 Conclusions

If, therefore, one were to search for road construction materials in the Beaufort Series, one would obviously concentrate on convex slopes.

Materials assessment tables of this type form a very useful first step in the systematic analysis of occurrences of natural materials deposits.

TABLE 2.5  
Material assessment: K3 Beaufort Series

Lithology according to Table 2.4											
Land form	Number of tests carried out on different materials										Number of tests carried out per land form
	Base		Subbas		Selected subgrade		LL sub-grade		Fill		
	Nat	Stab	Nat	Stab	Nat	Stab	Nat	Stab	Nat	Stab	
1.1	-	-	100	25	28	-	4	-	2	-	159
1.3	-	4	71	17	49	-	-	-	-	-	141
1.4	-	-	9	-	-	-	-	-	-	-	9
1.8	2	-	10	-	-	-	3	-	-	-	15
2.1	-	-	2	2	4	-	-	-	-	-	8
3.2	2	4	593	75	135	-	12	-	3	-	824
3.4	-	-	31	8	9	-	4	-	1	-	53
3.6	-	-	4	-	-	-	-	-	-	-	4
5.4	-	-	5	-	-	-	-	-	-	-	5
High position	4	8	825	127	225	-	23	-	6	-	1 218
3.3	2	-	202	28	61	-	4	-	2	-	299
5.1	-	-	2	-	-	-	-	-	-	-	2
5.2	-	-	16	-	3	-	-	-	-	-	19
Low position	2	-	220	28	64	-	4	-	2	-	320
3.5	1	1	266	28	83	-	9	-	-	-	388
4.1	4	-	195	-	28	-	17	-	-	-	244
Plain	5	1	461	28	111	-	26	-	-	-	17
3.8	-	-	17	-	-	-	-	-	-	-	17
5.3	-	-	4	-	-	-	-	-	-	-	4
5.5	-	-	13	-	-	-	-	-	-	-	13
5.6	-	-	96	6	-	-	-	-	-	-	102
5.11	2	-	9	-	4	-	-	-	-	-	15
Valley	2	-	139	-	10	-	-	-	-	-	151
	13	9	1 645	183	410	-	53	-	8	-	
	22		1 828		410		53		8		2 321
	2 321										

## 3 SOIL STABILIZATION

### 3.1 Introduction

Soil stabilization is a technique to improve the technical properties of natural soil by mechanical or chemical additives or by other means.

The improvement of soil properties in stabilization work generally refers to a reduction in the plasticity and an increase in the strength or bearing capacity of the soil so that the traffic may be carried safely. A mechanical additive, for example, is clean sand and the end product of this treatment is mechanically stabilized material. Chemical additives are lime and cement and, accordingly, we talk about lime-stabilized or cement-stabilized materials.

There are, of course, many more stabilizers generally, such as bitumina, tars, resins, heat or almost any salt, but their use is limited in comparison with the use of lime and cement.

An example of the wide use of soil stabilization is given in Table 3.1, which indicates its application in particular to the higher quality road layers such as the basecourse and subbase. It is only for the use in lower road layers where, often, the quality of natural materials is found to be sufficiently good to be of use untreated.

#### 3.1.1 REVIEW

Lime stabilization of soil has been applied as an art for thousands of years (McDowell, 1959), but it is interesting to note that even the 1959 edition of Soil Mechanics for Road Engineers (TRRL, 1959) described only mechanical and cement stabilization, but not lime stabilization. Although, therefore, its application as an art made great progress during the last 30 years or so especially, our understanding of soil reactions (ie soil chemistry) with lime has lagged behind. Soil mechanical

TABLE 3.1  
Application of cement- and lime-stabilization in  
South Africa (mainly in Transvaal and Natal)

Use		Dolerite- type gravel	Granite- type gravel	Shale- type gravel
Base	Nat	2 (4 %)	23 (13 %)	2 -
	Stab	48 (96 %)	157 (87 %)	5 -
Subbase	Nat	505 (63 %)	587 (60 %)	346 (85 %)
	Stab	291 (37 %)	391 (40 %)	59 (15 %)
Selected subgrade	Nat	316 (94 %)	230 (100 %)	133 (100 %)
	Stab	20 (6 %)	0 -	0 -
Lower layer subgrade	Nat	134 (98 %)	59 (100 %)	6 -
	Stab	2 (2 %)	0 -	0 -
Fill	Nat	46 (98 %)	15 (100 %)	2 -
	Stab	1 (2 %)	0 -	0 -
TOTAL	Nat	1003 (73 %)	914 (62 %)	489 (88 %)
	Stab	362 (27 %)	548 (38 %)	64 (12 %)

(physical) aspects, on the other hand, developed rapidly, and resulted in number of recipe books that made their appearance, the most important of which are listed below as recommended reading:

Kezdi, A. (1979)

Terrel, Epps, Barenberg, Mitchell and Thompson (1979)

There is sufficient evidence available to enable us to consider cement stabilization of soil as a special case of lime stabilization as far as soil reactions are concerned. The reason for this is that the process may be considered a combination of cement hardening on its own and a reaction of the soil with chemically released lime ( $\text{Ca(OH)}_2$ ). This is a by-product of the hardening process of cement.

Soil stabilization has gone through four very distinct phases of development and understanding during the past 30 years, such as

- . Soil mechanical phase
- . Quality of stabilizers
- . Soundness of aggregates and soils
- . Soil/lime reaction.

These are discussed in more detail in the following.

#### 3.1.1.1 Soil mechanical phase

This refers mainly to the grading and plasticity aspects of construction materials. The philosophy is that the density of the material has a direct influence on its load-bearing capacity. The presence of clay minerals was a complicating factor which was taken into account by lime treatment to 'kill' plasticity. Too much clay is considered physically detrimental in that it influences the workability of road construction materials, has no bearing capacity when wet and causes uncontrolled movement of the pavement by shrinkage and swell, depending on the prevailing moisture content. The soil mechanical approach is still dominating the



scene and an example of its application on construction materials for untreated road bases (TRH4, NITRR, 1971) is given in Table 3.2. It has been updated from time to time without many changes being introduced (TRH4, NITRR, 1985 and TRH14, NITRR, 1985). The technical terms are either self-explanatory or are explained in the glossary at the end of this work.

In short, the parameters for

plasticity	Atterberg limits
aggregate crushing value (ACV)	aggregate strength
shape of aggregate	workability
grading	density
California Bearing Ratio (CBR)	strength/moisture content

were believed to form a balanced soil system for a specified traffic load (say 30 000 equivalent 80 kN axle loads) over the lifetime of the road (usually 15-20 years). But this is a long time and it is important therefore to know the technical 'properties of stabilized materials and the techniques for achieving them' (Ingles and Metcalf, 1972) in order to manipulate this system over this period. The literature on stabilization has adopted the approach of analysing stabilization reactions in terms of the strength parameters mainly of

- . mechanical stabilization,
- . lime stabilization and
- . cement stabilization etc,

which approach has, with hindsight, proved to confuse the issue more than reveal the truth, because stabilization reactions are of a chemical nature and include also a time factor.

This is easily understood considering that the above materials parameters are mechanical factors which also describe the stage of physical weathering of a material. If any one of these parameters changes (including lifetime and traffic load), the system becomes unbalanced and the road

TABLE 3.2  
1971 Standards for bases for asphalt pavements  
(untreated material).

(a) Atterberg limits	
Maximum liquid limit (LL)	25
Maximum plasticity index (PI)	6
Preferred minimum plasticity index	2
Maximum linear shrinkage (LS)	3
(b) Group index	
Maximum group index	0
(c) Aggregate strength and resistance to crushing	
Aggregate crushing value (on -13,2 -9,5 mm fraction)	max 30 %
OR	
10 % Fines aggregate crushing test (FACT) value (10 % FACT) (on -13,2 -9,5 mm fraction)	min 110 kN
(d) Shape of aggregate	
Maximum flakiness index (determined on the -26,5 -13,2 mm fraction in accordance with SABS647)	35
Minimum percentage of the material retained on the 4,75 mm sieve having at least one fractured face	50
(e) Aggregate grading	
Sieve (square mesh) (mm)	Percentage passing
37,5	100
26,5	82-95
19,0	70-85
13,2	58-75
4,75	34-55
2,00	22-40
0,425	10-25
0,075	5-14
0,053	4-12

Table 3.2 cont/...

TABLE 3.2 (cont.)

The grading of the material after compaction should be as near as possible to the mean of the specified grading envelope and in two out of any four consecutive tests carried out on the completed basecourse, the percentage by weight of material passing the 2,00 mm sieve should not exceed 34 % and that passing the 0,425 mm sieve not more than 20 %.

(f) Soil mortar grading

The desirable grading limits for the soil mortar (fraction passing the 2,00 mm sieve) are as follows:

Designation	Passing sieve (mm)	Retained sieve (mm)	Percentage by weight of soil mortar
Coarse sand	2,00	0,425	35-40
Fine sand	0,425	0,053	25-40
Silt	0,053	0,005	10-20
Clay	0,005		5-10

In general, whereas the silt content should be about 15 %, the clay content should not exceed 8 %.

The total sand content, ie the coarse sand plus the coarse fine sand, medium fine sand, and fine fine sand, should not be less than 75 % of the soil mortar.

(g) Strength

Minimum CBR in place (ie compacted to 98 % Mod AASHO density at field water content and then soaked)	80 %
Maximum CBR swell at 100 % Mod AASHO compaction and optimum moisture content	0,5 %

(h) Compaction requirements

Compaction moisture during construction shall be that moisture content which is the optimum for the compaction equipment employed, provided that the moisture content is not so high as to cause instability or movement during the compaction process.

Minimum relative compaction in place shall be at least 98 % of Mod. AASHO density. This may alternatively be specified as a percentage of theoretical maximum density.

fails. It is relatively easy to correct these parameters, eg the plasticity index (PI) by adding lime, but a greater PI also indicates different grading, aggregate crushing value etc and such correction very seldom restores the total parameter balance. In fact, all technical properties must be rearranged to balance the system again for the projected lifetime and traffic load of the road. This interrelation was particularly shown by the work of Emery (1985) and Sampson, Emery and Rose (1985). It is also interesting to note that chemical aspects such as weathering products in construction materials seem to enter the picture only at a stage of weathering beyond the mechanical parameters specified above and it is in the area of these substandard materials that stabilization is applied most effectively.

The early concept of unsoundness and durability in construction materials, and of aggregates in particular was introduced during this phase of knowledge, and thus provided at least a term of reference for such material deficiencies.

#### 3.1.1.2 Quality of stabilizer phase

The problem, if one existed, of the quality of lime for stabilization was resolved by the adoption of quality specifications for lime such as SABS824: 1967 and by subsequent amendments. These standards placed the responsibility for any road failures squarely on materials and materials selection.

#### 3.1.1.3 Soundness of aggregate and soil phase

Until very recently it was believed that the strict specification of mechanical properties was all that was necessary to guarantee satisfactory road performance.

This belief is correct in principle for untreated materials as specified above, but since natural resources of high-quality, sound materials became exhausted the upgrading of lower-quality construction materials has come to be increasingly applied and therefore the diagnosis of what constitutes soundness has taken on a new dimension.

It was found, for example, that different construction materials such as granite, dolerite, sandstone etc require to be mixed with different quantities of lime in order to reach saturation pH (Clauss and Loudon, 1971). Saturation pH is the pH of a soil/lime/moisture mixture which does not increase if more lime is added. In this instance the pH is used as a practical parameter to measure the quantity of lime and it indicates how much dissociated lime is in solution in the soil moisture. By the same token, the pH also indicates how much of the lime is reacting with the soil and changing it into a relatively 'sound' and inert material. Different particle ranges which generally represent different stages of weathering also demand different quantities of lime for saturation pH and must be considered in any determination of the quantity of lime necessary for soil stabilization.

Furthermore, the permanence of stabilization products was recognized as a variable - except for carbonation strength - since the formation of calcium silicate depends on the pH of the mix, as does the permanence of these products. The reaction time of soil and lime also is not instantaneous and sometimes considerable time elapses before the reaction is complete. It was found, for instance, that the reaction time of black clay with lime exceeds 24 hours and hence the pH may change. The problem of durability is therefore of great interest. It seems to be a problem concerning the interpretation and extrapolation of stabilization reactions, although some of the more basic and common problems are encountered with pH instrumentation, eg the accuracy of reading and electrode efficiency.

The soundness of construction materials may therefore be related to the predictability of soil/lime reactions. It is considered as a transition

to the soil/lime reaction phase which succeeded the previous phase of knowledge.

The final word on soil/lime reactions and their significance in the achievement of stabilization strength has still not been pronounced, but some important aspects are discussed below by means of a projection.

### 3.1.2 PROJECTION OF SOIL/LIME REACTION PHASE

Diamond and Kinter (1966) eventually decided to analyse soil stabilization in terms of chemical reactions between the stabilizing agent, lime and soil or gravel; they opened the way to a systematic first approach towards these aspects.

Since then, lime stabilization has usually been discussed under the headings 'modification processes' and 'cementation processes' (Gregg, 1966: Clauss and Loudon, 1971: Blight and Barrett, 1979). These processes are also known as amelioration or rapid reactions and long-term reactions respectively (Diamond & Kinter, 1966). The effect of modification, ie cation exchange and flocculation agglomeration (TRB, 1976) and also anion exchange, is decreased plasticity and the development of uncured strength and improved load deformation properties. The effect of cement reaction, ie the soil/lime pozzolanic reaction (TRB, 1976), is the development of strength in time by the formation of cement minerals.

The development of strength in lime/clean sand mortar mixes in air (carbonation) ranges from between 345 and 2 069 kPa in 28 days (Boynton, 1975) and is well demonstrated in nature by the formation of hard calcrete.

However, the development of strength by carbonation in limestabilized soils has been rejected by Diamond and Kinter (1966) and is considered undesirable (TRB, 1976).

Although, therefore, the ingredients for lime stabilization were well known the problem was that the influence of soil and gravel on strength formation remained a largely unknown variable which a classification into parent rock such as granite, shale etc could not resolve completely (TRH13, NITRR, 1986). The stage and mode of weathering (Weinert, 1980) of these different parent rocks is another variable, which was even more elusive and the different soil mechanical classification systems simply do not have sufficient resolution power to facilitate the discussion of soil/lime reactions in detail.

The following section therefore concentrates on possibly reactive compounds in stabilized soil/gravel and describes their reaction.

### 3.2 Possibly reactive compounds in stabilized soil

Possibly reactive compounds in a lime-soil-water system may be listed as follows:

- (a) Lime.
- (b) Soil - clay, amorphous silica and alumina, amorphous glass.
- (c) Water - water, gases dissolved in water (such as  $\text{CO}_2$ ), soluble salts and organic matter.

The reactions dealt with cover a spectrum including ion-exchange, chemical reactions proper and precipitations and may broadly be described as chemical reactions. Physical aspects as a result of chemical reactions also enter the discussion although these are discussed in Section 3.4 Stabilization reactions and Section 3.6 Guide for testing in stabilization work. These physical aspects are the engineer's parameters by which the success or failure of some chemical reactions is assessed.

Thompson (1970) was probably the first worker to recognize that different soils react differently after lime treatment. He subdivided natural construction material into lime-reactive and lime non-reactive soils. The criterion applied was an increase of the unconfined compressive strength (UCS) in lime-stabilized material of 345 kPa (50 Psi) for lime-reactive soil or less in lime non-reactive soil. These two soils are, inter alia, discussed in the text that follows and, for easy reference, are briefly compared in Figures 3.1 and 3.2.

The problems were identified more clearly by using completely weathered basic igneous rock (see also Table 3.11) and shale (see also Table 3.9). These soils are also believed to have developed any unsuitable technical properties which may be disadvantageous to soil stabilization to a maximum degree and are of wide occurrence. The disadvantage was, of course, that roads are constructed with gravel and not with soil. The argument is, however, easily overcome by pointing out that every natural gravel, except washed deposits and granular derivatives, generally contains to a larger or lesser degree a completely weathered clay fraction of soil which also consists of clay. The difference between clay fraction and clay is that the former term denotes a particle size while the latter means that the material consists of clay. In terms of stabilization strength these two soils are described as follows:

The UCS of lime-treated Silverton soil and black clay was determined on 50 x 100 mm cylindrical specimens, otherwise following method A14 (TMH1, NITRR, 1979). The results presented in Figures 3.1 and 3.2 give essentially the same readings for carbide waste lime, Marvello road lime and pro analysi pure lime. A correlation between the UCS and CBR of soils as presented in Table 3.3 may be used for convenience in the interpretation of the test results given below. It was developed by Kozan and Fenwick (1965) for the US Corps of Engineers. It may be seen that the difference between the strength levels of lime stabilized Silverton soil and lime stabilized black clay was as high as 100 % and an explanation of this difference is not easy when one relies exclusively upon mechanical concepts.



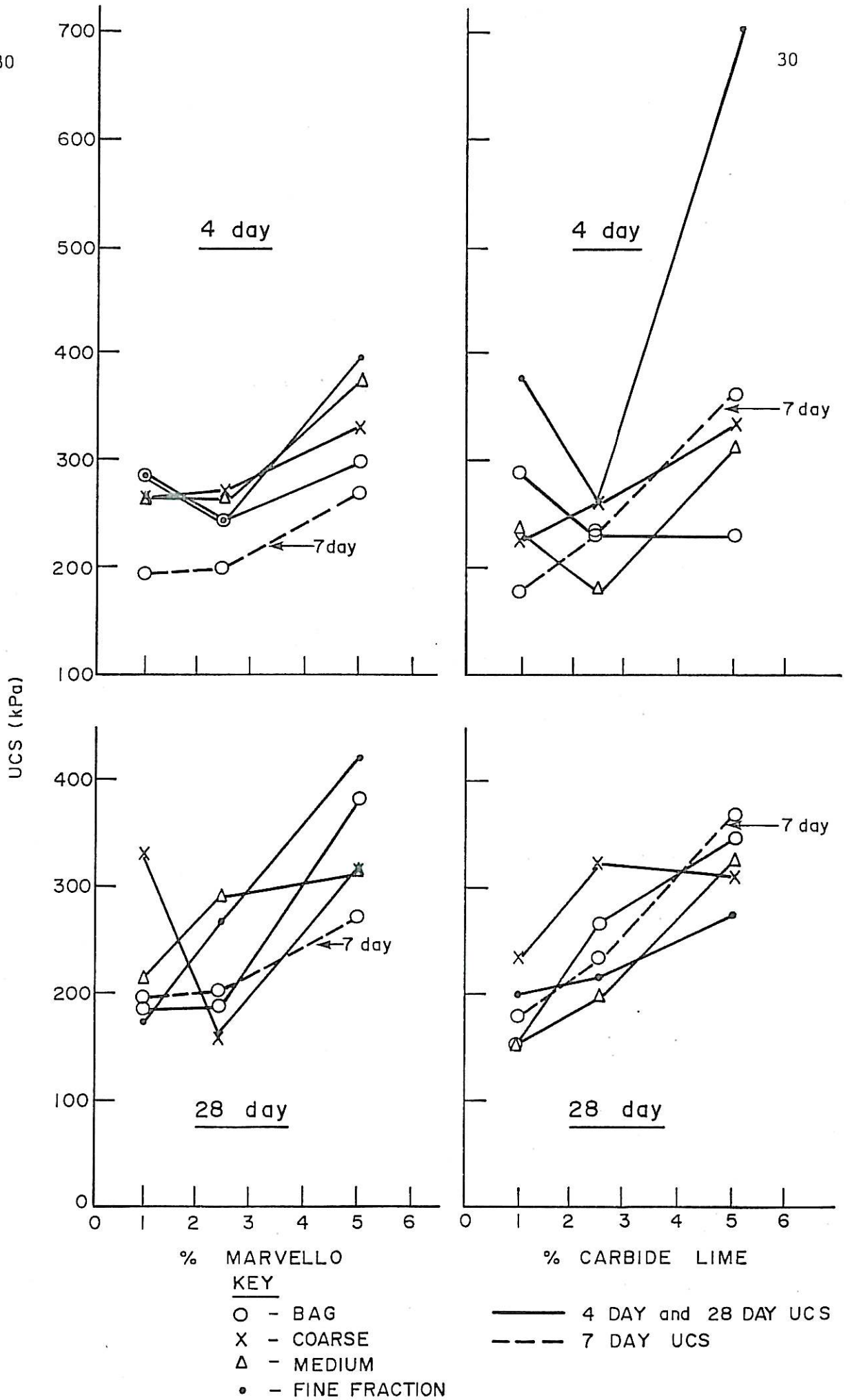


FIGURE 3.1

Random reaction of different gradings of waste carbide lime and Marvello road lime with Silverton soil. All particles are particle agglomerates.

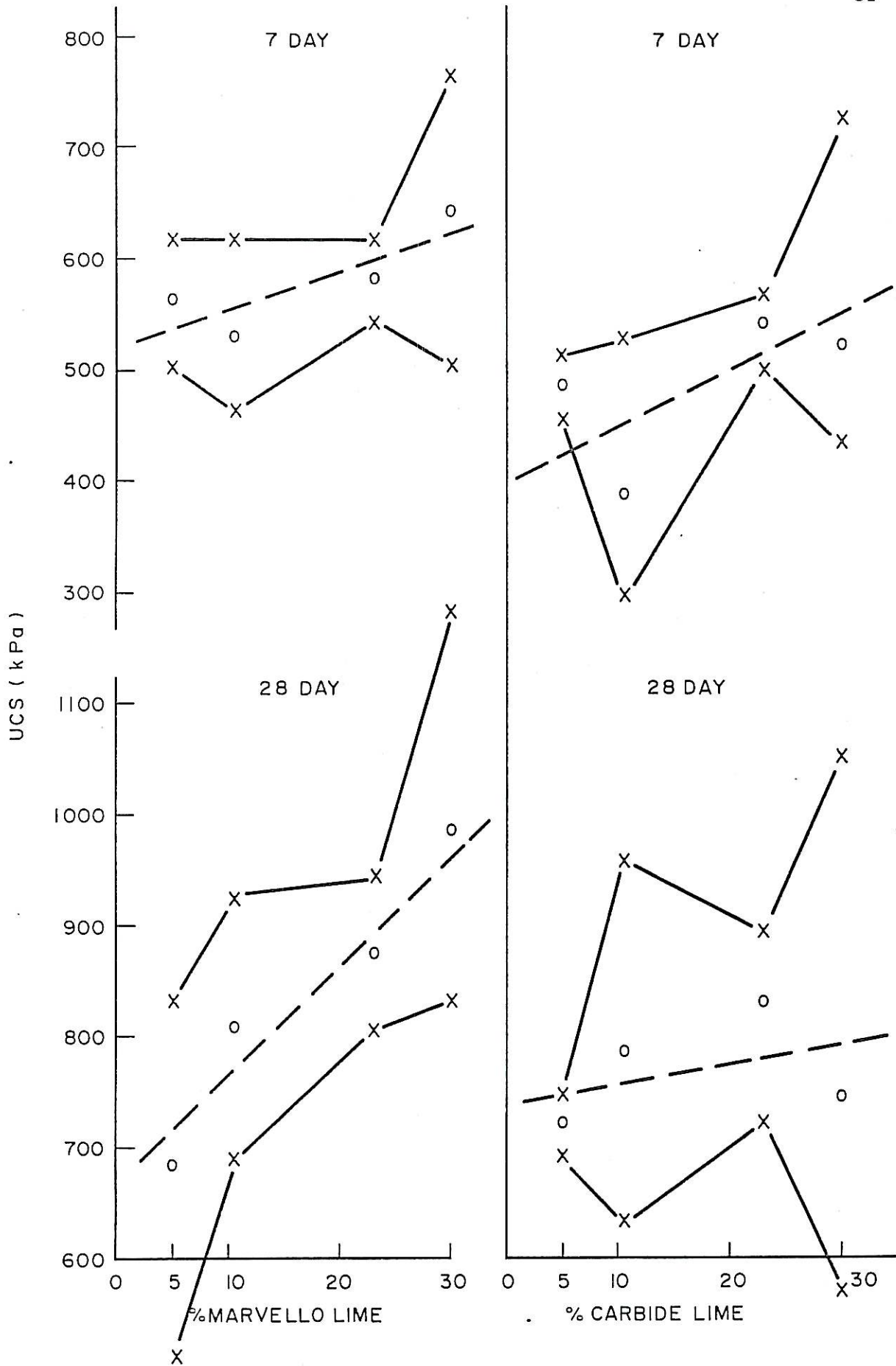


FIGURE 3.2

*Regression analysis of strength development in Marvello and carbide lime-stabilized black clay (OMC, MDD).*

14U 4 423276 N1111111

TABLE 3.3  
Comparison of CBR and UCS

CBR (%) (unsoaked)	(Psi)	UCS (kPa)
20	80-100	552-690
15	60- 80	414-552
12	50- 60	345-414
8	35- 45	242-311

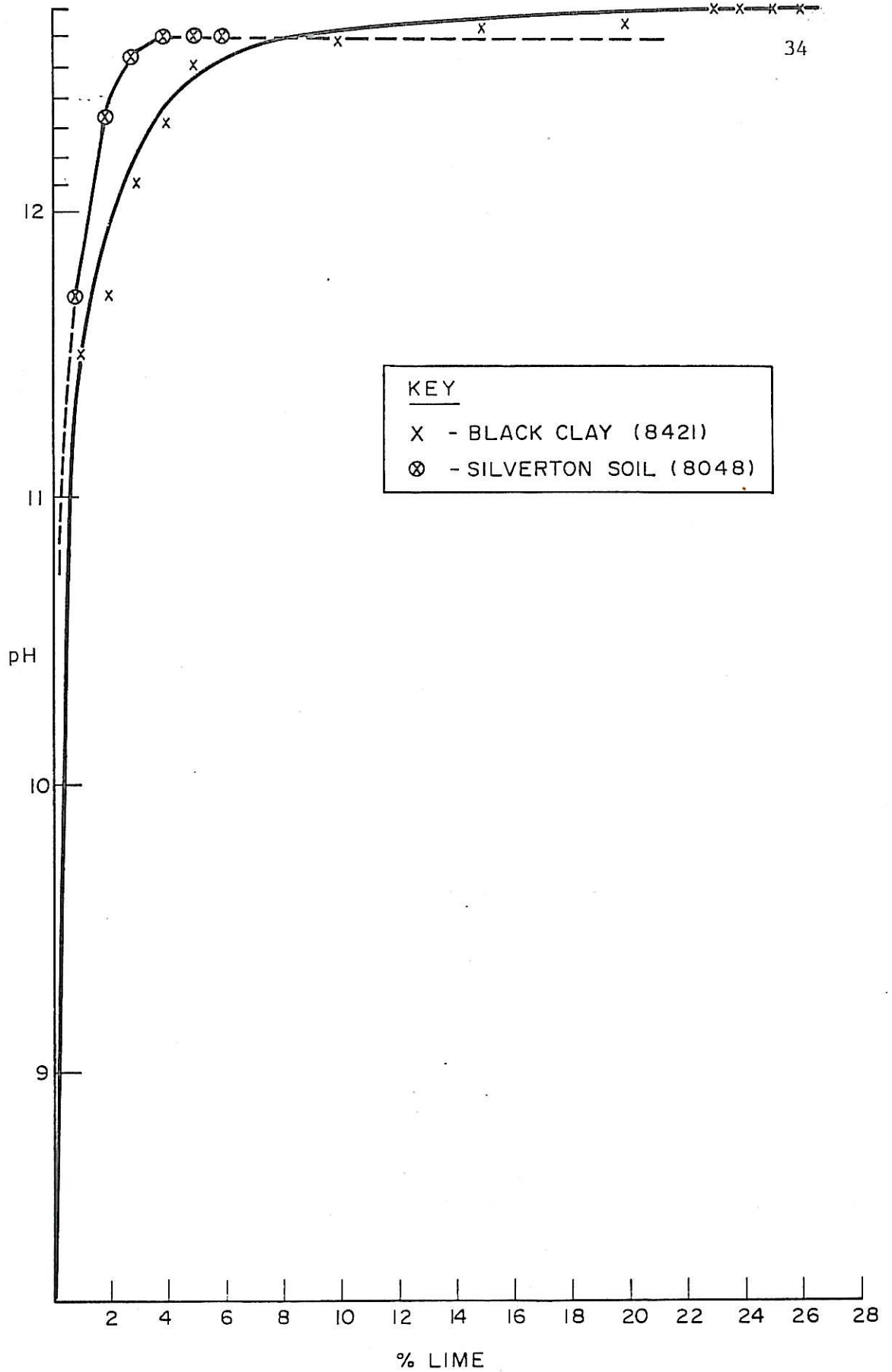
When road lime is added to these soils the resulting pH levels show very different reaction patterns, as shown in Figure 3.3. On the one hand may be seen the rapid lime saturation of Silverton soil which does not change with time, indicating no further reaction between lime and soil. On the other hand an ongoing reaction with lime may be postulated for black clay as indicated by the gradually changing pH level in the lime-soil water mixture.

Two particular schools of thought offering an explanation for these phenomena dominate the scene. They are:

- (a) The attack on clay minerals by the highly alkaline soil moisture in lime-stabilized soils. The result is believed to be the decomposition of the crystal lattice of clay minerals and the subsequent formation of cement-like minerals from the solution. This hypothesis is particularly favoured in the USA (Eades, Nichols and Grim, 1962; Diamond, White and Dolch, 1964), Europe (Brand, 1963), Australia (Ingles and Metcalf, 1972) and South Africa (De Wet and Skinner, 1967).
- (b) A chemical reaction of existing amorphous soil compounds with lime. This hypothesis is favoured in South Africa, mainly by Netterberg (1969) and Clauss and Loudon (1971), where research has been strongly dominated by a soils bias.

However, neither of these hypotheses succeeded in explaining all or even most stabilization reactions.

A case in point is a road failure near Stoffberg in the Transvaal, Republic of South Africa. As early as 1965 an at the time unsuccessful search was carried out in South Africa to isolate a soil compound with a high consumption of lime. A road constructed near the little settlement of Stoffberg had failed without apparent cause and all subsequent testing also failed to produce any explanation. The problem was eventually more or less shelved, with the name Stoffbergite being given to the unidentified and unknown, but none the less troublesome soil compound.



## KEY

X - BLACK CLAY (8421)

⊗ - SILVERTON SOIL (8048)

FIGURE 3.3

Attempt at a best-fit curve of the (indicator) ICL of black clay and Silvertown soil. Note the different saturation pH levels which were probably caused by different temperatures.

What follows is a discussion of possible reactive soil compounds in stabilized material.

### 3.2.1 ROAD LIME

Lime is manufactured by heating limestone ( $\text{CaCO}_3$ ) in furnaces. When a temperature of  $850^\circ\text{C}$  is reached the limestone decomposes and  $\text{CO}_2$  gas is released. The residue  $\text{CaO}$  (burned lime, quick lime, unslaked lime) is slaked with water, which results in the formation of lime powder ( $\text{Ca(OH)}_2$ ). Unslaked and slaked lime are used in road work.

This process may be described by the following formulae:



Quarried limestone is seldom pure and therefore the active or available lime is specified in SABS824: 1967 (as amended). Three types of lime for soil stabilization have been specified. These are -

The calcium type        in which the ratio of calcium oxide to magnesium oxide is 14 or higher,

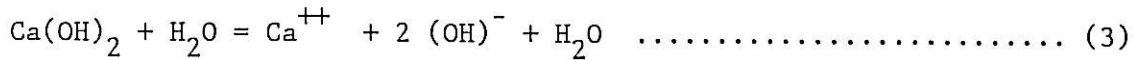
The magnesium type      in which the ratio of calcium oxide to magnesium oxide is 2 or higher, but less than 14,

The dolomite type        in which the ratio of calcium oxide to magnesium oxide is 1,3 or higher, but less than 2.

Limes from different sources have a different composition and in the course of time test methods have been developed to achieve comparable results. The parameters tested however, present, a rather limited description of the product. A wider spectrum of test parameters and descriptions - all or any of which may contribute to an explanation of different applications and even soil reactions - is therefore presented below.

### 3.2.1.1 Chemical reaction of lime and water

When lime ( $\text{Ca(OH)}_2$ ) is added to water it dissociates slightly according to the following formula -



The process is strongly alkaline and the effective quantity of lime for lime stabilization is often measured by its resulting pH level (see Figure 3.4) which depends on the ambient temperature. The pH is used as the parameter for measuring the stabilization potential of a soil, because the formation of some strength-producing minerals ceases at low pH levels as explained later in more detail below.

### 3.2.1.2 Technical properties of lime as specified

One test each of carbide lime, Marvello road lime and  $\text{Ca(OH)}_2$  pro analysi were carried out by the SABS in Pretoria. The test results are presented in Table 3.4.

The sieve analysis of a second sample indicated, however, that a coarser fraction than specified in SABS824:1967 (as amended) may in fact be found in carbide lime (Plate 3.1). It was also found in Marvello road lime, although to a much smaller extent (Table 3.5). The tests carried out so far show no measurable influence on the soil reactivity by this impurity if lime is apportioned in terms of initial consumption of lime (ICL) or cement reaction lime content (CLC) determinations, as shown below.

### 3.2.1.3 Further technical properties of road lime

Road lime also has technical properties of its own which are usually ignored in practical stabilization work. They are important when it comes to the interpretation of reactions of extreme mixtures of lime and soil, eg 5 % lime and 95 % soil or 95 % lime and 5 % soil. These technical properties are recorded in Table 3.6 and Figure. 3.5.

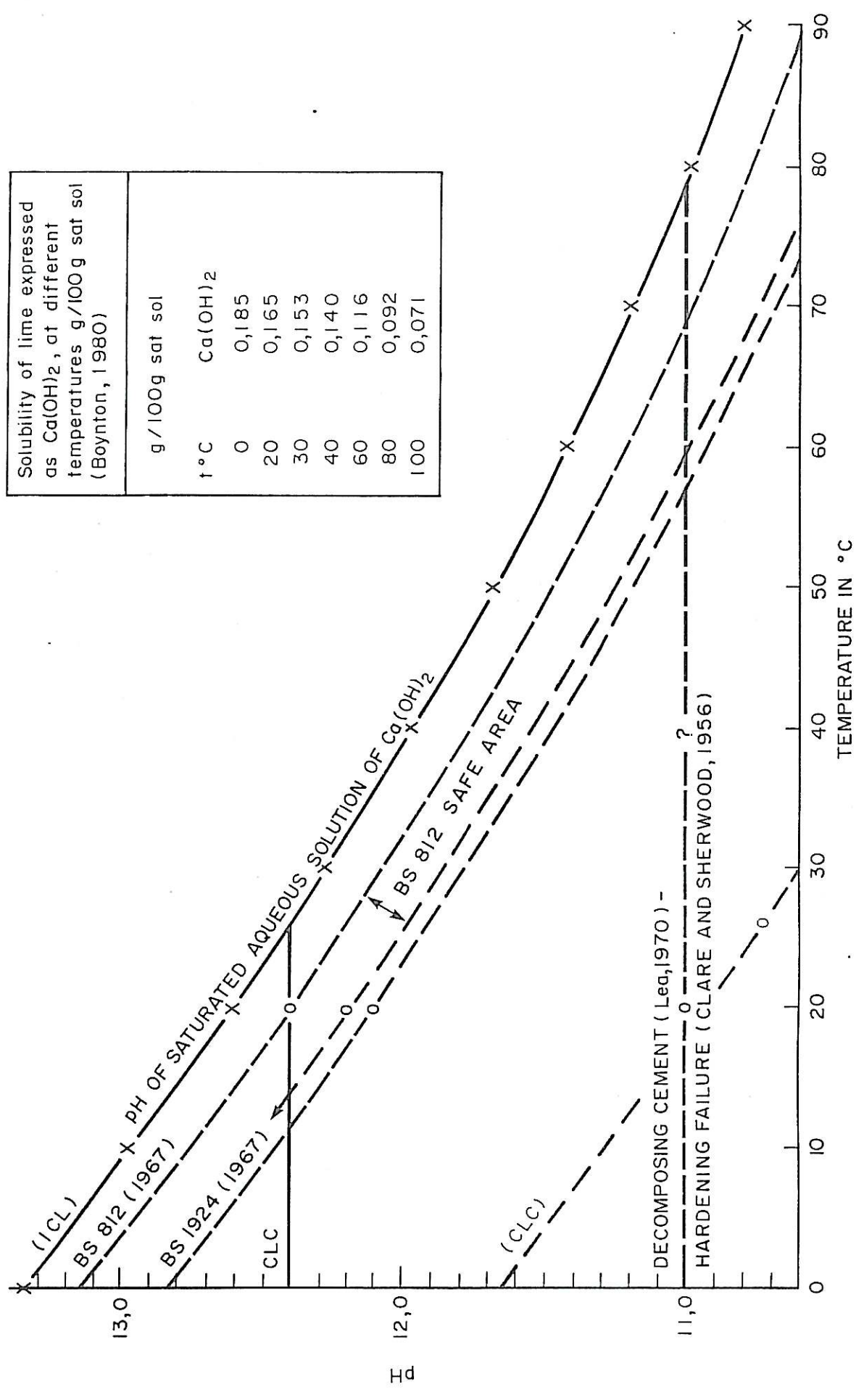


FIGURE 3.4

*pH of saturated calcium hydroxide solution at different temperatures and cementation pH - ICL (SABS 78/56109 : Draft proposal for general methods of pH Measurements, ISO, TC47, SCI, WG37).*



TABLE 3.4  
 Technical properties of various limes according  
 to SABS824: 1967 (as amended)  
 (Analyst: SABS, report No. 61118587/80 dated 22.5.80.)

Nature of test	Requirements of SABS 824-1967 (as amended)	Test results		
		8089 Ca(OH) <sub>2</sub> pa	8088 Carbide lime	8087 Marvello lime
Fineness				
% retained 850 micron	Nil	Nil	Nil	Nil
on sieve 600 micron	2,5 max.	Nil	1,0	Nil
75 micron	50,0 max.	8,0	30,0	11,0
Soundness factor	30 max.	6	11	7
CaO plus MgO content % mass (CaO)	75 min.	99,24	93,73	96,26
CO <sub>2</sub> content % mass	5 max.	0,44	0,62	1,67
Free water content, % mass	3 max.	0,42	0,76	1,03
Available lime content % mass (CaO)	Ca type: 50 min Mg type: 35 min Dolomite type: 30 min	71,47	65,11	61,04

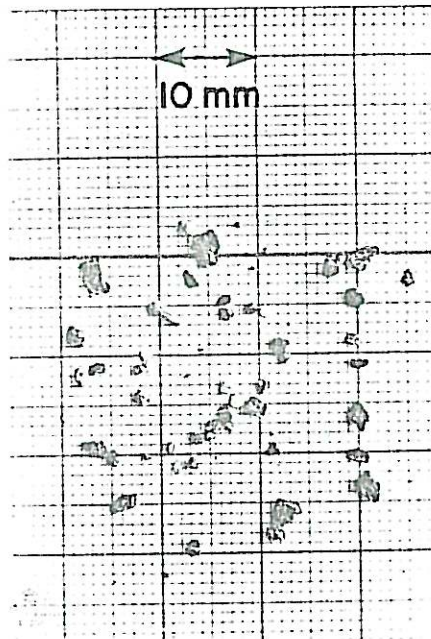


PLATE 3.1

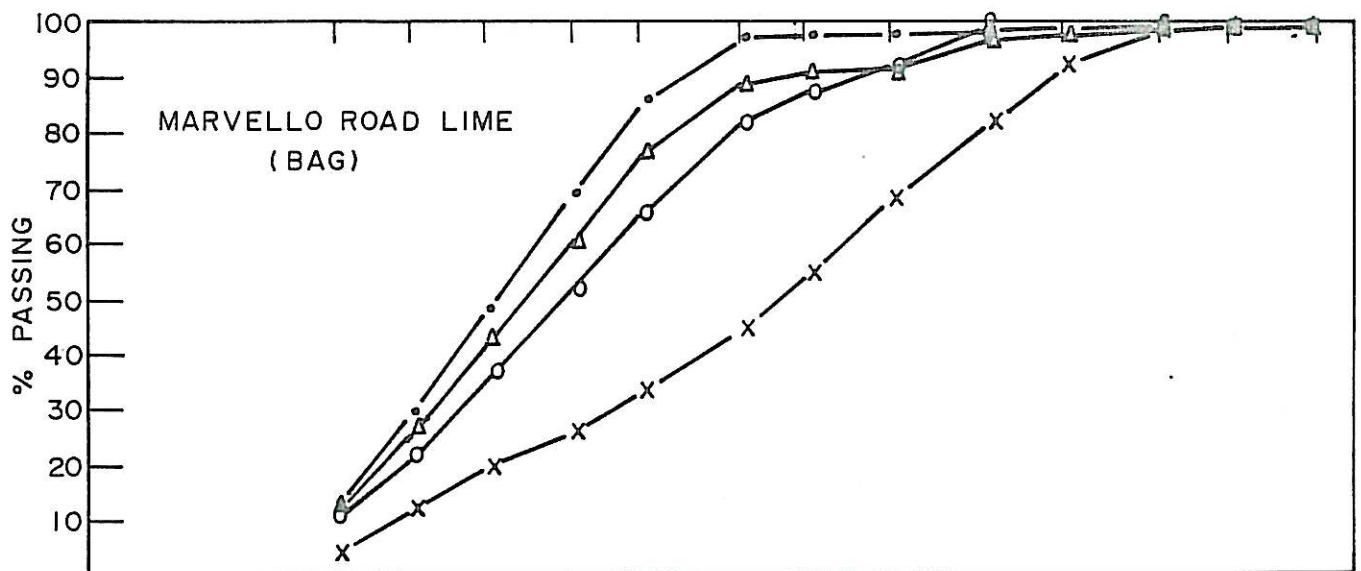
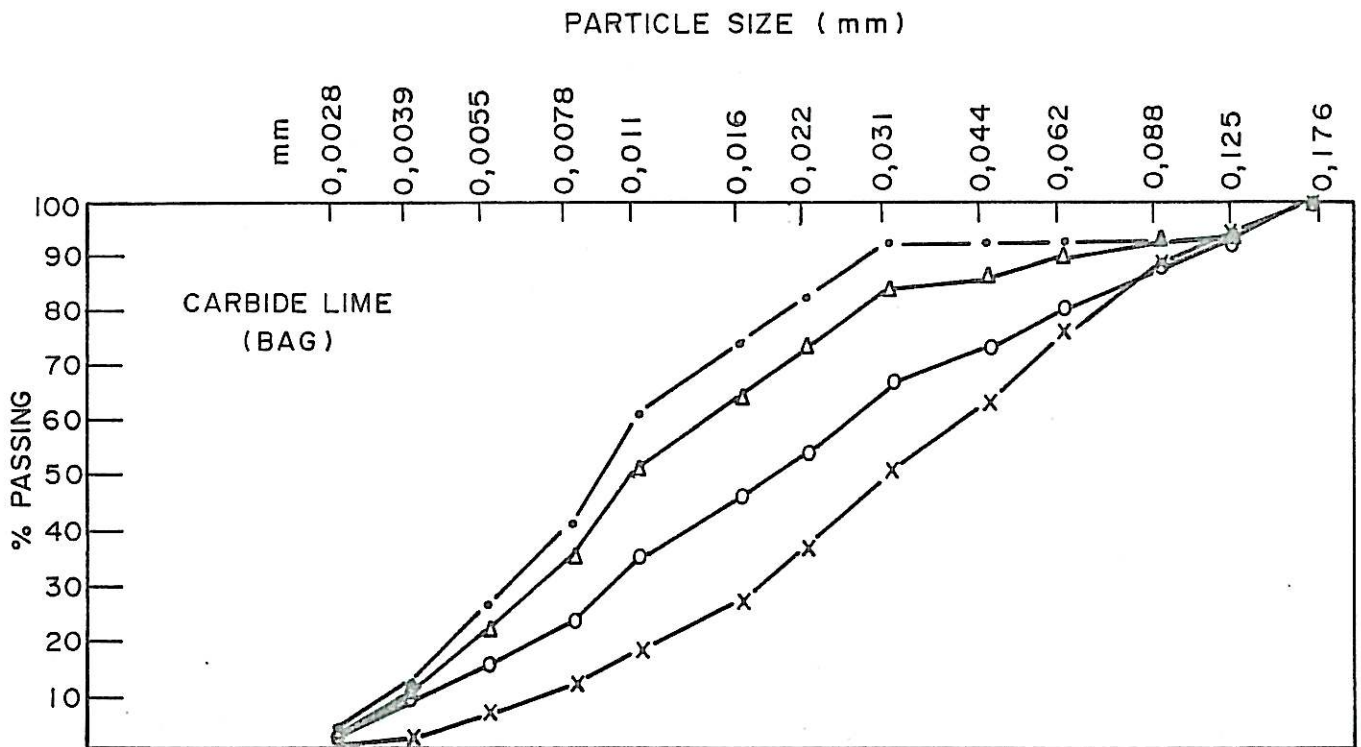
+ 850 micron fraction (0,4 %) of  
dry-processed carbide lime

TABLE 3.5  
Undesirable coarse fraction particles  
in dry processed, carbide lime

	Aperture size of sieve	Percentage retained on sieve	
		Marvello road lime	Carbide lime
Slaked lime	850 micron	0,03	0,41

TABLE 3.6  
Technical properties of road lime

Ref No.	8089	8088	8087	L 39
Origin	Ca(OH) <sub>2</sub> p.a. (Bag)	Carbide lime (Bag)	Marvello (road lime) (Bag)	Rotalym (road lime) (Bag)
Grading (percentage passing by mass) (mm)				
0,536			100	
0,250			100	
0,150			99	
0,075			98	
0,060			65	
0,020			64	
0,006			63	
0,002			1	
Plasticity				
LL (Atterberg method)	-	47	-	60
PL	-	42	-	40
PI	-	5	-	20
LS	-	1,3	-	6,0
Compaction and strength characteristics (Mod AASHTO)				
OMC	-	33	-	-
MDD	-	1 100	-	-
UCS (28-day)	-	1 491	-	-
Soluble salts and pH				
pH	-	12,6	12,7	-
Conductivity (Sm <sup>-1</sup> at 25 °C)		0,50	0,38	-
Specific surface area (Nitrogen adsorption, m <sup>2</sup> /g)	8,10	8,27	9,30	-



KEY

- |            |   |   |
|------------|---|---|
| X = FIRST  | } | READING ON MICROTRAC INSTRUMENT (NIM, 1981) |
| O = SECOND |   |   |
| Δ = THIRD  |   |   |
| • = FOURTH |   |   |

FIGURE 3.5

*Dispersion of (waste) carbide lime and Marvello road lime in water during 10 minutes (four successive readings of the same sample).*

#### 3.2.1.4 Aspects of particle size

The fineness of lime in particular is thought to be a major factor affecting its soil reactivity and this aspect must be considered in detail.

The result of standard sieve and hydrometer analyses with calgon as dispersion agent is shown in Figure 3.5. A microtrac instrument (manufactured by Leeds & Northrup), which applies a new principle in particle size determination, was also used.

'As the basis for analysis, the analyzer utilizes the phenomenon of low-angle forward-scattering of light in conjunction with a new optical filtering technique. Particles of any type scatter precise quantities of light through precise angles related to their sizes. The combination of a laser light source, proprietary optical filters and a powerful microprocessor makes it possible to use these relationships for reliable reproducible measurement.

In the analyser the sample is transported across a continuous laser beam. Particles in the sample scatter the light; the scattered light is collected by a lens, passed through a patented Compumask optical filter and focused by a second lens onto a sensitive photocell detector. The resulting detector output is proportional to selected functions of the particle diameter. These sequential signals are then processed by the microprocessor to provide measured and calculated values.'

Microtrac analysis allows in particular the fast repetition of determinations. Figure 3.5 shows four repeated particle-size analyses of the same sample with a time difference of approximately 2 minutes. A substantial reduction in particle sizes of the 0,2 mm fraction may be observed during the process. The small difference of 5-10 % in the mass of the 0,0028 mm fraction indicates, however, that although minor solubility

may not be discarded (see also Table 3.12), the different particle size curves indicate that a strong mechanical disintegration process takes place.

To date it has not been possible to establish any difference in the clay reactivity (Atterberg limits) of different particle sizes of lime. Coarse, medium and fine ranges of grain size of carbide lime controlled by determinations of their specific surface area (Figure 3.6) reduced the plasticity of treated soil in the same way.

Electron microscope photographs of carbide waste lime with different magnifications explain this puzzling result. A sequence of enlarged particles of carbide lime (Plate 3.2) shows that the originally sharply contrasted particles (x 600) in fact form particle agglomerates. Even the smallest particles of the x 600 picture resolve into a fluffy and lacey microstructure when enlarged 20 000 times; they are therefore much too fine to be distinguished into process- and reaction-related particle ranges by coarse test methods such as sieve analysis.

Marvello road lime and  $\text{Ca(OH)}_2$  pa were magnified in the same way and in each case the particles resolved into the same fluffy and lacey microstructure.

#### 3.2.1.5 Hydration and volume change

The hydration process of lime (slaking) involves a large reduction in density from 3,3 g/cm<sub>3</sub> to 2,2 g/cm<sub>3</sub> (Ingles and Metcalf, 1972). In terms of mass, it was found that 56 parts of calcium oxide are equivalent to 74 parts of calcium hydroxide (Ballantine and Rossouw, 1972) ie 24 % H<sub>2</sub>O is combined by the slaking process.

The use of unslaked lime in stabilization work - where lime is often transported over many kilometres from the supplier to the construction site - therefore has the distinct economic advantage of saving the cost (mass and volume) of transporting the water necessary for the slaking process.

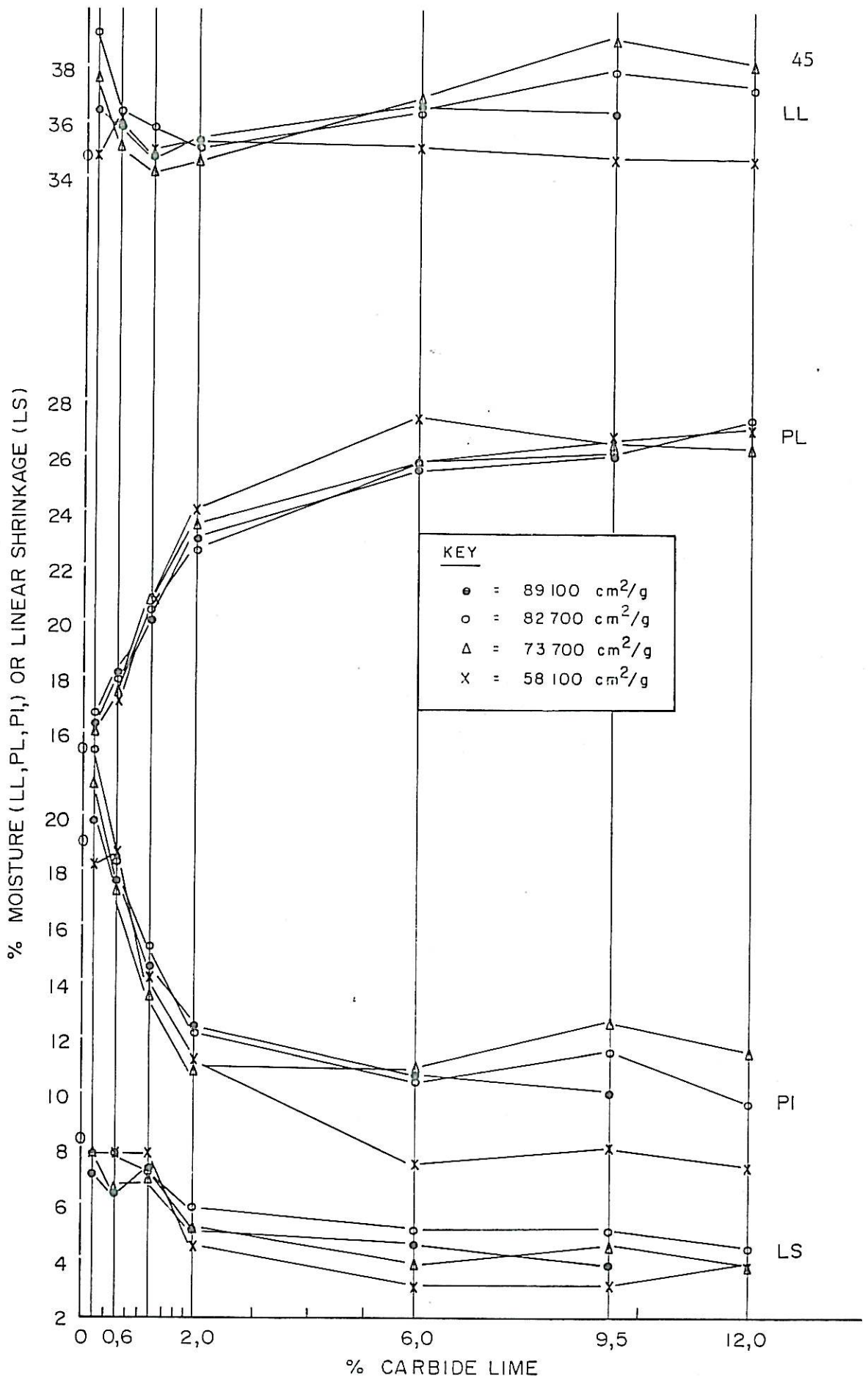
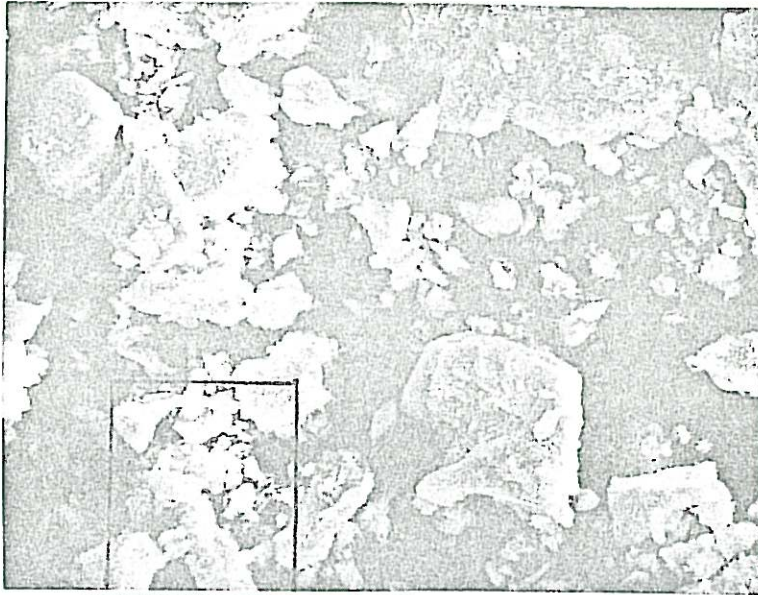


FIGURE 3.6

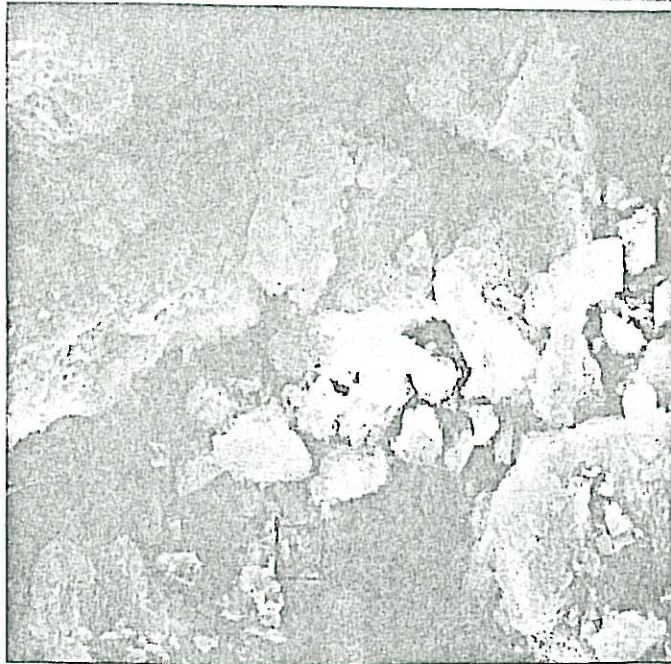
*Clay reactivity of lime.*

*The effect of carbide lime on weathered shale (Pretoria series) (4 day curing; ○ = natural soil, ● = carbide lime as supplied x = coarse carbide lime, ○ = fine carbide lime, △ = medium fine carbide lime) sample No 8 059.*

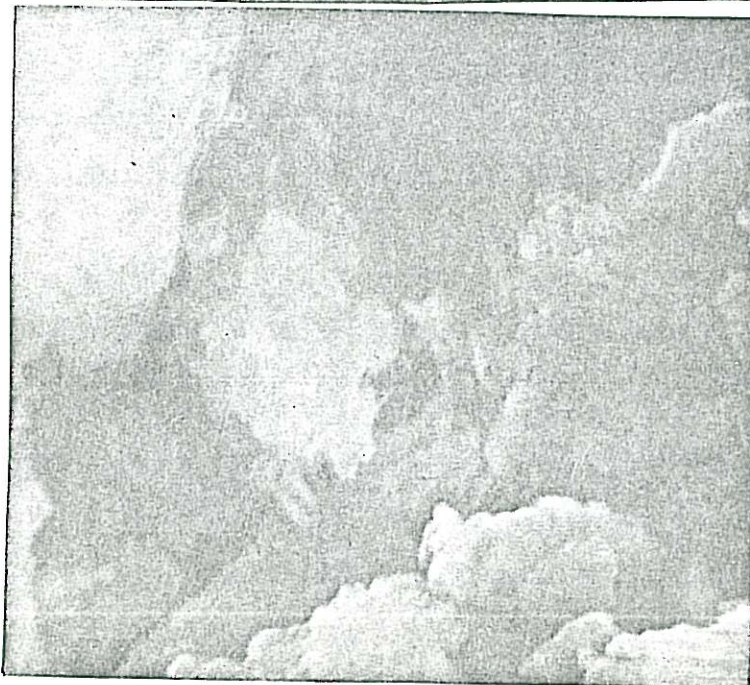




x 600



x 2 000



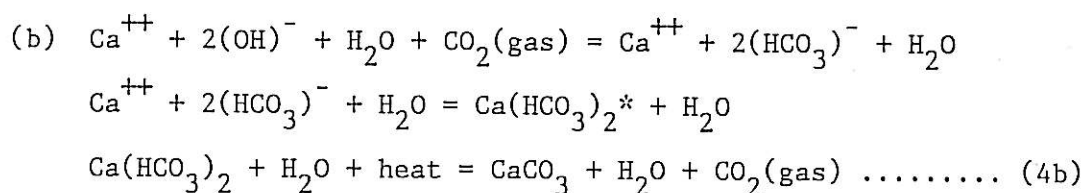
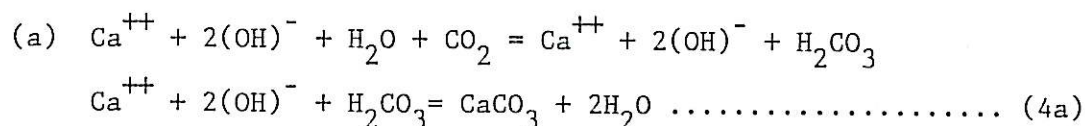
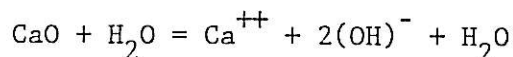
x 20 000

## PLATE 3.2

Electron  
microscope  
photographs of  
carbide lime.  
The framed area  
in photographs  
top and centre  
have been  
enlarged in  
photographs  
centre and  
bottom  
respectively.

## 3.2.1.6 Lime carbonation

The chemical reaction of lime carbonation may be summarized as follows:



\* High partial pressure of  $\text{CO}_2$ .

Wagner (1950) quotes the following data:

1 l $\text{H}_2\text{O}$	dissolves	14 mg $\text{CaCO}_3$
1 l $\text{H}_2\text{O} + \text{CO}_2$ (air saturation)	dissolves	60 mg $\text{CaCO}_3$
1 l $\text{H}_2\text{O} + \text{CO}_2$ (saturated)	dissolves	1 000 mg $\text{CaCO}_3$

The increase in temperature of  $\text{H}_2\text{O}$  from 0 to 20 °C reduces the  $\text{CO}_2$  dissolved in  $\text{H}_2\text{O}$  by approximately 50 per cent.

Experience seems to indicate that, depending on the physical conditions in the soil, either permanent or transient  $\text{CaCO}_3$  may form. The physical conditions in a South African soil were found to be such that a 15 cm cover of natural soil and grass increased the solubility of 1  $\text{cm}^3$  solid dolomite ( $\text{CaMg}(\text{CO}_3)_2$ ) cubes during 5 years' exposure from 0,03 g on the surface to 0,09 g at a depth of 15 cm. The same solubility (0,09 g) was also found at 30 cm soil cover, probably indicating  $\text{CO}_2$  saturation. It is believed that the high concentration of  $\text{CO}_2$  in rain also caused a high concentration of  $\text{CO}_2$  in the soil. The Gmelin Institute (1961) described the system  $\text{CaO-CO}_2\text{-H}_2\text{O}$  as follows:

'Stable precipitates during increasing  $\text{CO}_2$  partial pressure are, in sequence,  $\text{Ca(OH)}_2$ ,  $\text{CaCO}_3$  and probably  $\text{Ca(HCO}_3)_2$ . If  $\text{CO}_2$  is passed through lime water,  $\text{CaCO}_3$  precipitates but dissolves if more  $\text{CO}_2$  is added.  $\text{CaCO}_3$  forms then again on evaporation of  $\text{CO}_2$  from the oversaturated, heated solution.'

Most of the more spectacular and recent manifestations of carbonation strength indicate that  $\text{CaCO}_3$  rock seems to form from solution, more precisely from  $\text{CO}_2$  saturated solutions from which dissolved  $\text{CO}_2$  evaporates at increased temperature (Formula 4(b)). Examples are lime precipitation at springs, probably the lime encrustation of plants in lakes, the formation of stalactites and stalagmites in caves, lime mortar strength, probably some calcretes and boiler scale. This would indicate that, given the higher expectation of increased strength, critical moisture contents in stabilization such as optimum moisture content (OMC) should rather be approached from the wet side in order to make use of the possibly greater quantity of  $\text{Ca(OH)}_2$  dissolved in the greater moisture content and precipitated at OMC. One experiment which produced  $\text{CaCO}_3$  according to Formula 4(a) produced no strength at all.

It is concluded from this that  $\text{CaCO}_3$  may probably be formed under two different sets of physical conditions. One of these conditions seems to result in a  $\text{CaCO}_3$  powder without any significant strength-producing bond between  $\text{CaCO}_3$  crystals.

#### 3.2.1.7 Economic aspects of lime

The decision regarding the type of additive to be used in stabilization work cannot be made on test results alone; economic considerations such as transport distance, pricing, availability and prevailing technical skills must also be borne in mind.

It is difficult to understand the preference for calcitic lime in road construction except, perhaps, that its users hope to achieve hydraulic soil/lime reactions with it. The fact remains, however, that it is generally and widely favoured. The pricing of lime should be left to market forces, although quality (reactive  $\text{Ca(OH)}_2$ ) and production cost must be considered in arriving at a price. A comparison with the pricing of cement is made in the following:

- (a) It is assumed that the handling of the finished product costs as much as the handling of cement.
- (b) It is also assumed that the costs related to quarrying and transportation of the raw material before burning are similar to those of cement. There is, however, the additional cost factor of adding and probably producing separately the minor additives such as shale or clay or ash in the manufacture of cement, which would not be necessary for the production of lime.
- (c) A major cost factor is the energy consumption of 14-28 % of coal (by mass) in the production of clinker (Fulton, 1969) required for the manufacture of cement. The temperature required to burning cement is approximately 1 500 °C (Förster, 1928), while the temperature range of decomposing limestone is between 400 °C and 1 050 °C (Dammer and Tietze, 1927). The upper heat limit of 1 050 °C may not be increased because a higher temperature starts off the process of sintering and reduces the soil reactivity of the lime by the formation of dead-burnt lime.
- (d) A second major cost factor in the production of cement is the very intensive, multiple quality control of at least six compounds. The making of lime requires only one compound ( $\text{Ca(OH)}_2$ ) to be controlled.
- (e) The third major cost factor in the production of cement is the capital cost and running of some of the biggest and heaviest machinery in the world (Fulton, 1967), whereas lime may be produced cheaply in comparison.

These considerations indicate that the reasonable price of road lime should be approximately two-thirds of the price of cement - in accordance with the costs for raw materials, energy consumption during burning, quality control and capital items. The price of waste lime (with savings (b) to (e)) should be even cheaper.

The above comments refer to calcitic lime. The use of dolomite as road lime has even more advantages. The carbonation of calcitic lime is slow in the early phases, whereas that of dolomitic lime is fast in the early phases (Boynton, 1980). This is demonstrated if Transvaal dolomite (Table 3.7) and calcitic lime are burnt at 875 °C for 5 hours. Both limes were hydrated after cooling by adding (de-ionized) water. Calcitic lime remained powdery even after 3 weeks' dry/wet cycling. Dolomitic lime had turned into a more stony material after 3 days and remained so after 3 weeks of dry/wet cycling. After 3 weeks the pH was 12,0 for calcitic lime and 11,5 for dolomitic lime. Production is also cheaper: Dolomitic lime decomposes at about 200 °C below the calcination temperature of calcitic lime (Boynton, 1980) and the saving of energy during burning may reduce production cost by another 25 per cent when compared with calcitic lime.

A very detailed survey was carried out by Fossberg (1965) in which it was found that all researchers in the field recommend the use of dolomitic lime for soil stabilization because of its better strength producing properties (Lu, Davidson, Handy and Laguros, 1957; Laguros, Davidson, Handy and Chu, 1956; Remus and Davidson, 1961; Mateos and Davidson, 1962; Wang, Davidson, Rosaner and Mateos, 1962). It is suggested therefore that the use of dolomite lime in soil stabilization should be re-evaluated. It should be possible to market the superior-strength-producing dolomite lime at less than 50 % of the cost of cement.

However, its use should not be considered in connection with the cement stabilization of soil. This is because the quantity of lime ( $\text{Ca(OH)}_2$ ) released chemically during the hardening of cement must be replaced by calcitic lime only. Dolomite may even prevent the formation of cement minerals altogether because of the low pH level achieved by such lime.

TABLE 3.7  
Approximate chemical composition  
(% by mass) of Transvaal Dolomite  
(Hall, 1938).

SiO <sub>2</sub>	0,94
Al <sub>2</sub> O <sub>3</sub>	-
Fe <sub>2</sub> O <sub>3</sub>	-
FeO	1,35
MnO	1,18
MgO	19,71
CaO	29,61
CO <sub>2</sub> + H <sub>2</sub> O	46,69

The strength in dolomite-stabilized soil is essentially a carbonation strength and the rapid hardening of dolomite will lock the soil particles rigidly into their position as in the case of cement stabilization but without the great danger of secondary shrinkage by chemically released lime.

#### 3.2.1.8 Conclusions

The technical properties of lime that are of importance in stabilization work are as follows:

(a) The chemical composition

This ranges from calcitic lime to dolomite.

(b) The grading

The sieve grading was found to represent lime aggregates which disintegrate further when mechanically treated. The chemical reaction is based on these micro-particles of lime.

(c) Plasticity

Lime has a plasticity of its own. The plasticity of stabilized soil therefore reflects the dominance of the plasticity of soil or of lime, depending on the soil/lime proportion of the mix.

(d) Stabilization potential

The pH of a soil/lime/moisture may be used as a parameter of the stabilization potential, depending on the temperature and soil compounds that may react with lime. The pH (at 25 °C) of calcitic lime is 12,4 and that of dolomite 11,5.

(e) Hydration

The hydration of lime results in an increased mass and volume of lime.

(f) Carbonation

Carbonation results in the formation of crystals which either remain separate and give no strength or grow together to form a strong mass of cemented soil particles, depending on the mode of carbonation.

(g) Economic aspects

The production cost of calcitic lime is approximately two-thirds of that of cement. The production cost of dolomite lime is approximately 25 % less than that of calcitic lime.

(h) Strength aspects

Dolomite lime produces early strength; calcitic lime produces late strength.



### 3.2.2 AMORPHOUS SILICA

Amorphous silica ( $\text{SiO}_2$ ) reacts very rapidly with lime ( $\text{Ca(OH)}_2$ ) in water to form calcium silicate hydrate (CSH), a cement reaction product (see Formulae 5-8 in Section 3.2.5.1(b)). However, the compound is very sensitive to the pH level and may be transient at a pH below approximately 10,5, but it seems to be permanent as long as the pH is maintained above 10,5 (Lea, 1970).

Hardening was observed in 30 % lime-stabilized amorphous silica after 12 hours. During this period the pH dropped from 12,4 to 9 which indicates that too little lime had been added and that the semi-stable CSH I + II phases were decomposing. This may be expected in a low lime-content system of this type since the ICL of 31 % quoted below refers to a reaction period of 1 hour only (Lea, 1970).

Clauss and Loudon (1971) reported an Initial Consumption of Lime (ICL) of amorphous silica of 31 % and believed this to be the reason for unusual soil-lime reactions (Figure 3.7). Strong and rapid strength development in stabilized soil was also reported by Netterberg (in Clauss, Netterberg and Williams, 1969) when lime reacted rapidly with construction material in northern Natal and with diatomaceous earth in South West Africa (Table 3.8). The reaction was so rapid that the required compaction could not be achieved.

The amorphous silica content of two soils from South Africa was determined by the method of Eggimann, Mannheim and Betzer (1980). The test results are presented in Table 3.8 together with amorphous silica contents of weathering successions of dolerite and norite, trass and rhyolite for the purpose of comparison. The analytical method, used was that of Mellor and Thompson (1938). Silverton soil and black clay were stabilized with 5 % lime. It may be seen from Table 3.8 that the UCS of natural soil/lime mixes does not correlate with the amorphous silica content as determined.

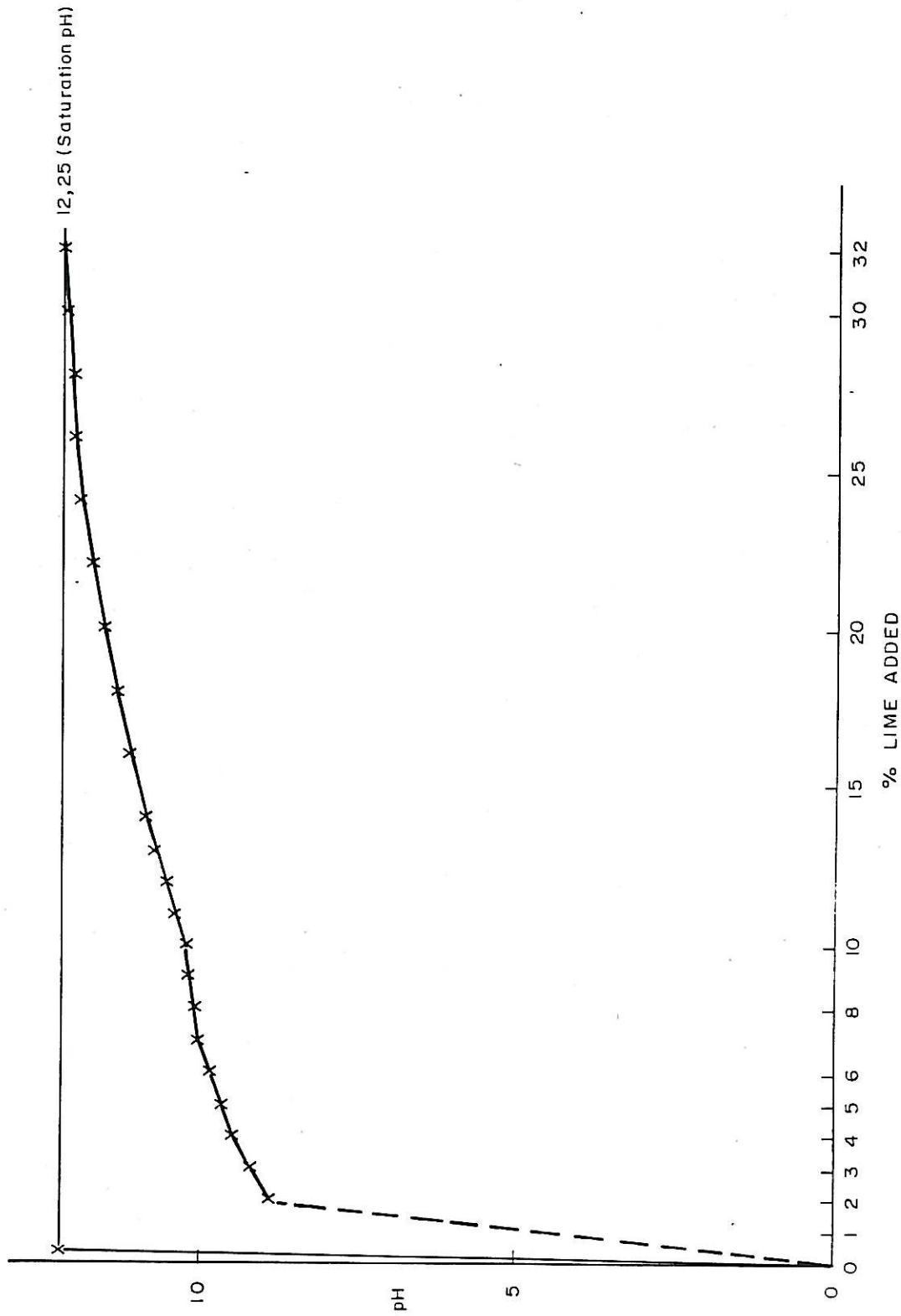


FIGURE 3.7

*Initial consumption of lime (ICL) of amorphous silica in terms of saturation pH after 1 hour.*

TABLE 3.8  
Amorphous silica in soil

(The strength given is the highest of various lime stabilized mixes determined at Mod AASHTO, OMC, MDD and after 28-day curing.)

Material	Source	Amorphous silica (%)	Stabilized strength# (MPa UCS)
Silverton soil	Pretoria	0,40+	0,3
Black clay	Pretoria	0,02+	0,7
Trass	Germany	11,80*	-
Rhyolite	Makatini	0,28*	-
Fresh norite	Stoffberg	0,36*	-
Weathered norite	Stoffberg	0,57*	-
Badly weathered norite	Stoffberg	0,40*	-
Residual soil on norite	Stoffberg	0,14*	-
Fresh dolerite	Heidelberg	0,24*	-
Weathered dolerite	Heidelberg	0,29*	-
Badly weathered dolerite	Heidelberg	0,35*	-
Residual soil on dolerite	Heidelberg	0,11*	-
Reactive sand	Matubatuba	0,09*	-

\*Mellor-Thompson method.

+Eggiman et al method.

#5 % lime-stabilized.

It is also believed that the ICL in quartzite is at least partly related to the reaction of lime with amorphous silica: amorphous silica forms the bond between quartz grains in quartzite.

A phenomenon that is demonstrable with particular ease in amorphous silica is the particle charge modulus of soils. This is defined as the modulus of the sum of zeta potentials of, for example, silica and alumina as shown in Figure 3.8 (Snel, 1984). It is probably also the catalyst for the development of cationic and anionic bitumen emulsions for making durable road seals with different types of aggregates. The best known example of this phenomenon is the negative charge of the silicium oxide molecule which consists of the configuration of four half-oxygen ions around the silica ion, as shown in Figure 3.9.

It is concluded that the amorphous silica content of soil does not, generally, seem to contribute to strength development in construction materials. A chemical reaction is to be expected, however. Moreover, the reaction product may be either lasting or transient, depending on the pH level maintained in the material. The zeta potential of soils and aggregates may be the cause for the preference of cationic or anionic emulsions for the treatment of different types of aggregates.

### 3.2.3 AMORPHOUS ALUMINA

The solubility of alumina, silica and iron in the presence of calcium was investigated by Acquaye and Tinsley (1964) and is presented in Figure 3.10. It may be seen from this that alumina has a marked solubility at high pH levels with a minimum between a pH of around 6 and 10,5. The solubility of silica, on the other hand drops sharply at a pH of 11,5 and that of iron at a pH of 4,5. The zeta potential of alumina ( $Al_2O_3$ ) is given in Figure 3.8.

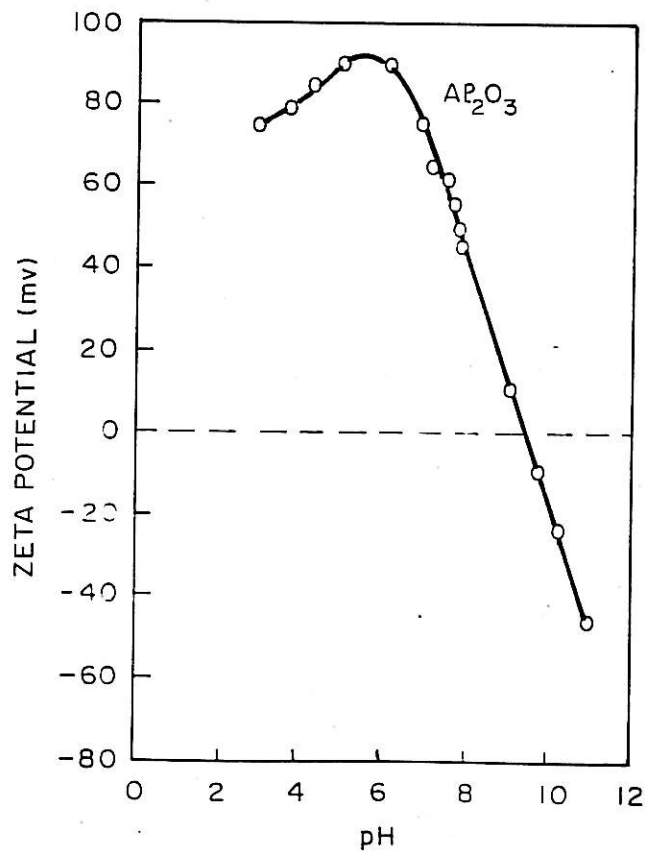
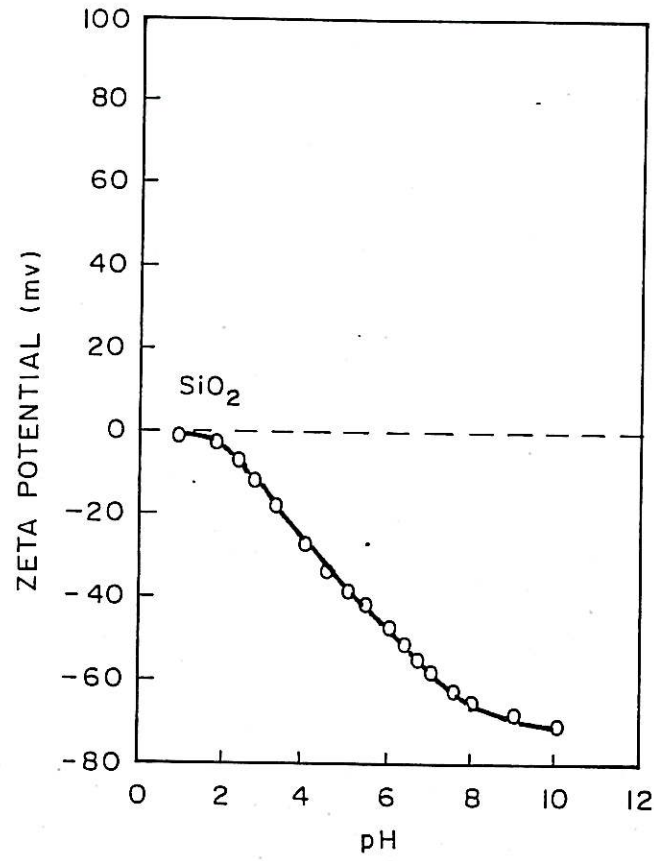
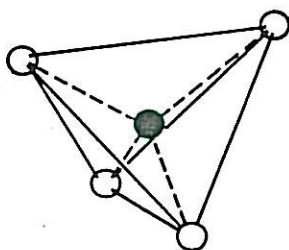


FIGURE 3.8

*The effect of pH on the zeta potential of silica and alumina particles (data reported by Hazel (1938) and Appleton (1973)).*



KEY	
●	= Si +++++
○	= O ---

FIGURE 3.9

*The arrangement of the  $\text{SiO}_2$  molecule as tetraeder with a residual electrical potential equivalent to two oxide ions.*

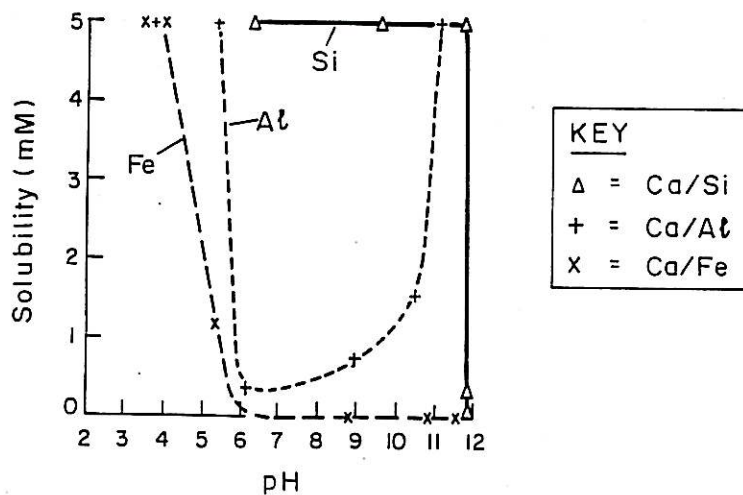


FIGURE 3.10

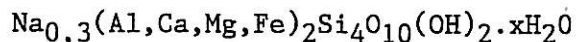
*The solubility of Si, Al and Fe.*

### 3.2.4 CLAY

Only those properties of clays which appeared important enough to the author to merit consideration in relation to stabilization work are discussed here. The interested reader is referred to the textbooks of Grim (1958, 1962) for more detailed information.

#### 3.2.4.1 Chemical properties

Considering that kaolinite-type clay consists largely of aluminium silicates and montmorillonite-type clays consist largely of aluminium-calcium-magnesium-iron silicates, and that both types have very small particle sizes and are therefore very sensitive to chemical reaction, it may be assumed that they are readily attacked in soil/lime mixes. The chemical composition of kaolinite is  $\text{Al}_2\text{Si}_2\text{O}_5(\text{OH})_4$ . That of montmorillonite is more variable and is, for example,



(International Centre for Diffraction Data, 1985).

Natural clays derived from decomposing rock also contain soil compounds which are, for the sake of convenience, grouped into the clay fraction. These are, however, not clays in the strict sense. This may be explained by the fact that in technical clay analyses the criterion used is one of particle size. In mineralogical analyses the criterion is one of crystallization. Neither would be suitable, however, for the determination of amorphous soil matter for which the term soil colloid has in the past been used.

A detailed soil analysis of Silverton soil as representative of the kaolinite-type soil and of black clay as representative of the montmorillonite-type soils is attempted in paragraphs (a) to (c) below.

(a) Kaolinite-type soil

This material is described in Table 3.9 and Figure 3.12. Here it is generally referred to as Silverton soil. It is being discussed in detail because it is the end result of the weathering process of shale and has not developed hydraulic strength during the period of observation when stabilized with lime. According to Thompson (1970) it may be classed as a lime non-reactive soil .

Of particular importance is the X-ray-diffraction (XRD) diagram (Figure 3.12), which shows an amorphous matter hump at position  $2\theta = 23,5^\circ$ . This is indicative of a lime-poor glass phase and is discussed later (Section 3.25).

This does not mean that hydraulic cementation does never takes place; but it may be delayed. Evidence of the formation of calcium silicate hydrates in lime-treated kaolinite was produced by Moh (1965). It is doubted, however, whether the slow formation of calcium silicates in Silverton soil - if they are formed at all - is of any importance in strength considerations of up to 7 months' curing (and possibly longer) or at low percentages of lime additives as practised in stabilization work.

The reaction in terms of LL of Silverton soil when treated with different additives was investigated in greater detail. The results are presented in Figures 3.13 and 3.14, where it may be seen that the different cations do indeed cause variations in the LL. However, these variations must be considered in comparison with the variations in the LL caused by anions such as  $(OH)^-$ ,  $(SO_4)^{--}$  and  $Cl^-$  for both calcium and sodium compounds. The complete results of indicator testing (Atterberg limits) are given in Table 3.10.

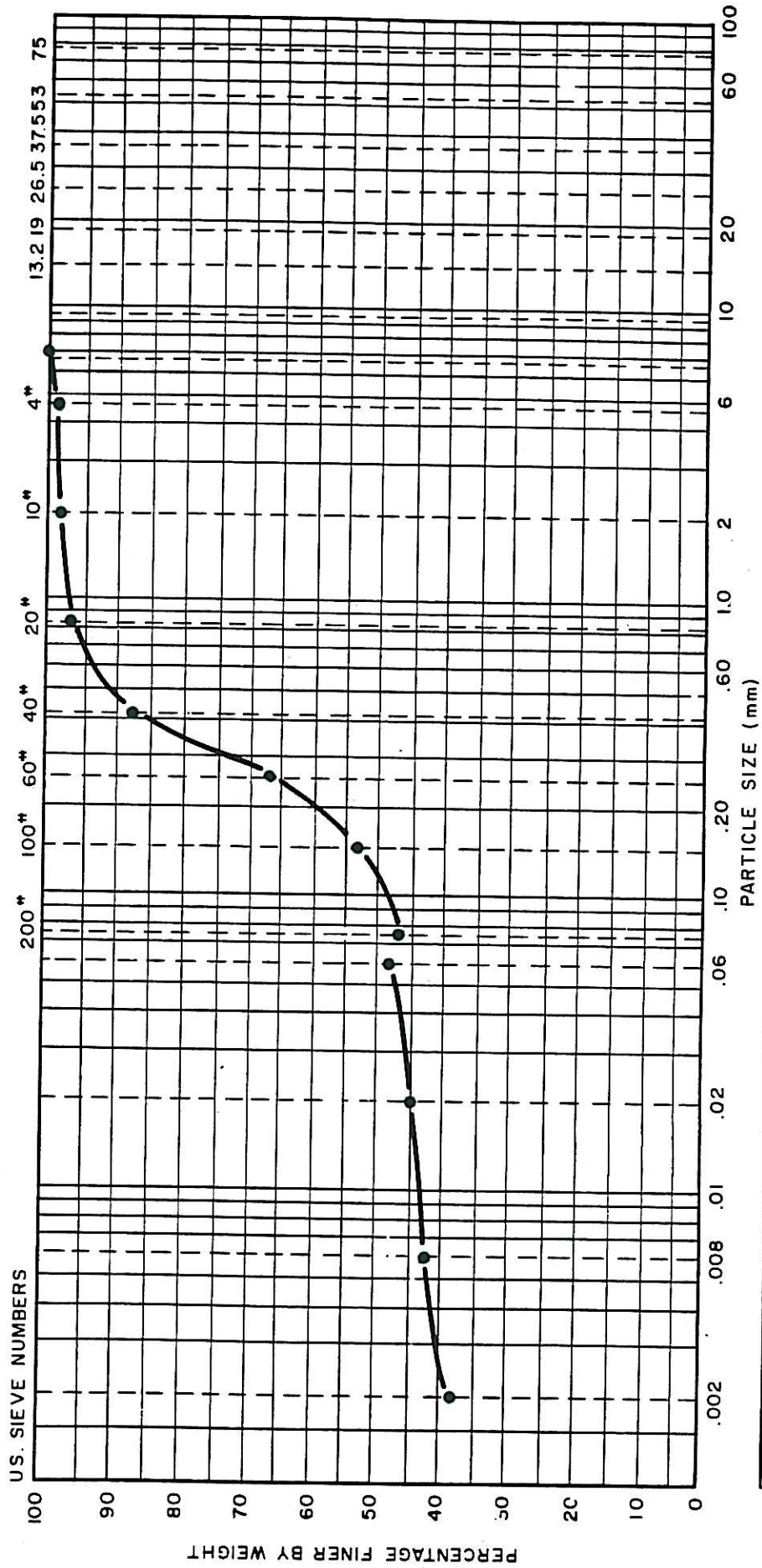


TABLE 3.9  
 Technical properties of Silverton soil, Ref. No.8048

Origin	: Weathered shale (T3m, Magaliesberg stage, Pretoria series, Transvaal)
Locality	: NITRR test site, Silverton, Pretoria, RSA
Depth	: 0-1,50 m
Grading	: See Figure 3.11
Plasticity	: LL 33,5 36,1 PL 15,8 15,6 PI 17,7 20,5 LS 7,3 9,3
Compaction and strength characteristics (Mod. AASHTO)	OMC 13,4 % MDD 1 855 kg/m <sup>3</sup> UCS (OMC, MDD, Mod. AASHTO, 28 days, 439 kPa SWELL Not determined
Stabilization potential	
Gravel ICL (Clauss, 1985)	: 2,5 %
Organic impurities (sucrose ICL, Clauss, 1985)	: 0,8 % lime equivalent
Soluble salts	
pH	: 6,95
Conductivity (Soluble salts)	: 0,01 % (0,01 Sm <sup>-1</sup> at 25 °C, -19 mm dia)
Spec. gravity	: 2,6945
Spec. surface area (nitrogen adsorption)	: 34,5 m <sup>2</sup> /g
Mineral and chemical composition	
Quartz	: 80 % (X-ray)
Kaolinite	: 20 % (X-ray)
Allophane (Method of Fields and Perrot, 1966)	: Nil (0,004 %)
Amorphous silica (Method of Eggiman et al, 1980)	: Nil (0,2 %)

NITRR DATE 23.1.80

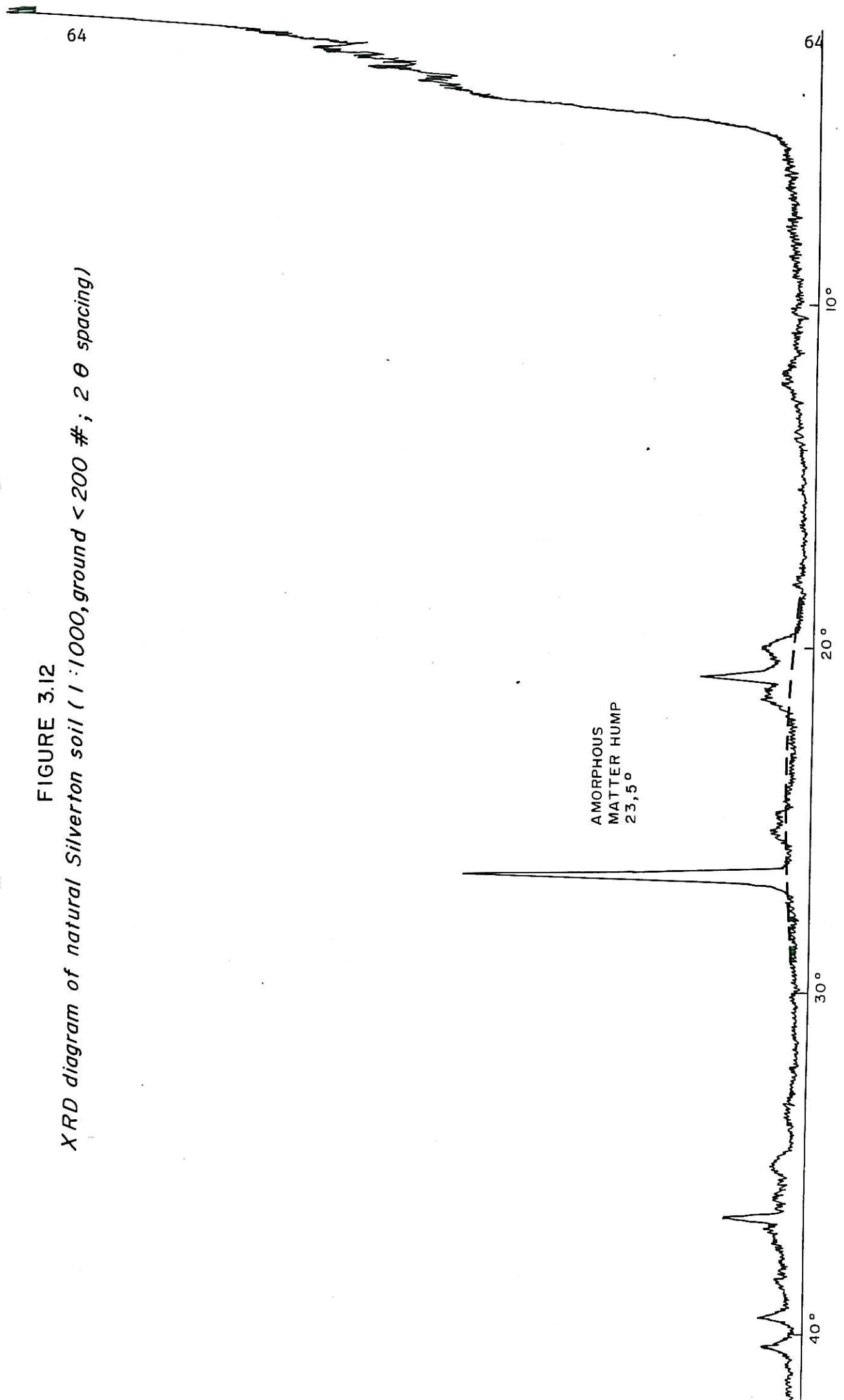
SAMPLE No.		8048	PROJECT 940 45804
SAMPLE LOCATION		—	
DEPTH		—	JOB No. 1536
CLASSIFICATION		SILVERTON SOIL	



CLAY FRACTION	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SILT FRACTION			GRAVEL FRACTION		

FIGURE 3.11  
Particle-size distribution : Silvertown soil

FIGURE 3.12  
*XRD diagram of natural Silverton soil (1:1000, ground < 200 #; 2  $\theta$  spacing)*



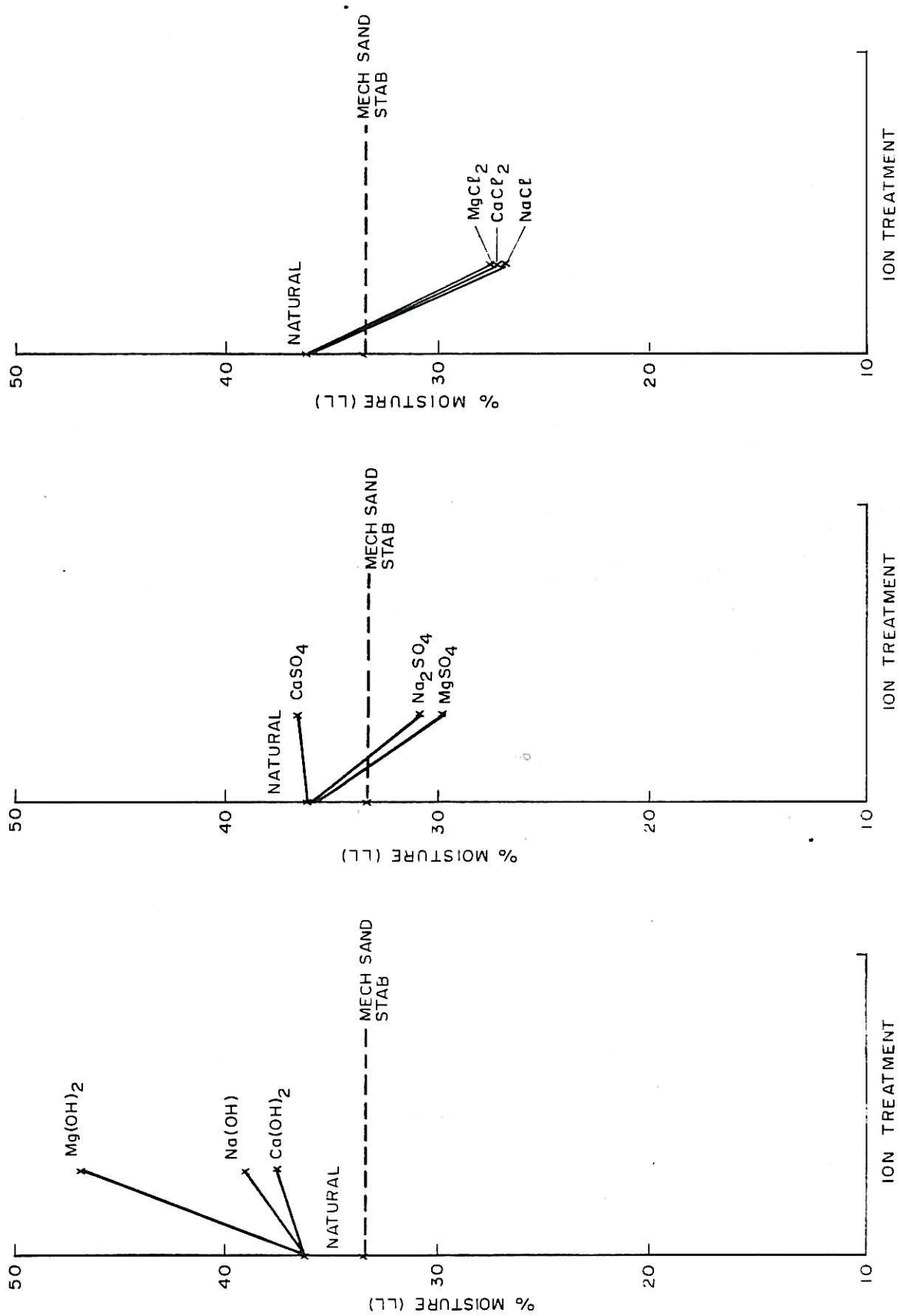


FIGURE 3.13  
 Cation and anion reactions of Silverton soil after treatment with 10 per cent of different additives in terms of liquid limits (4 day curing).

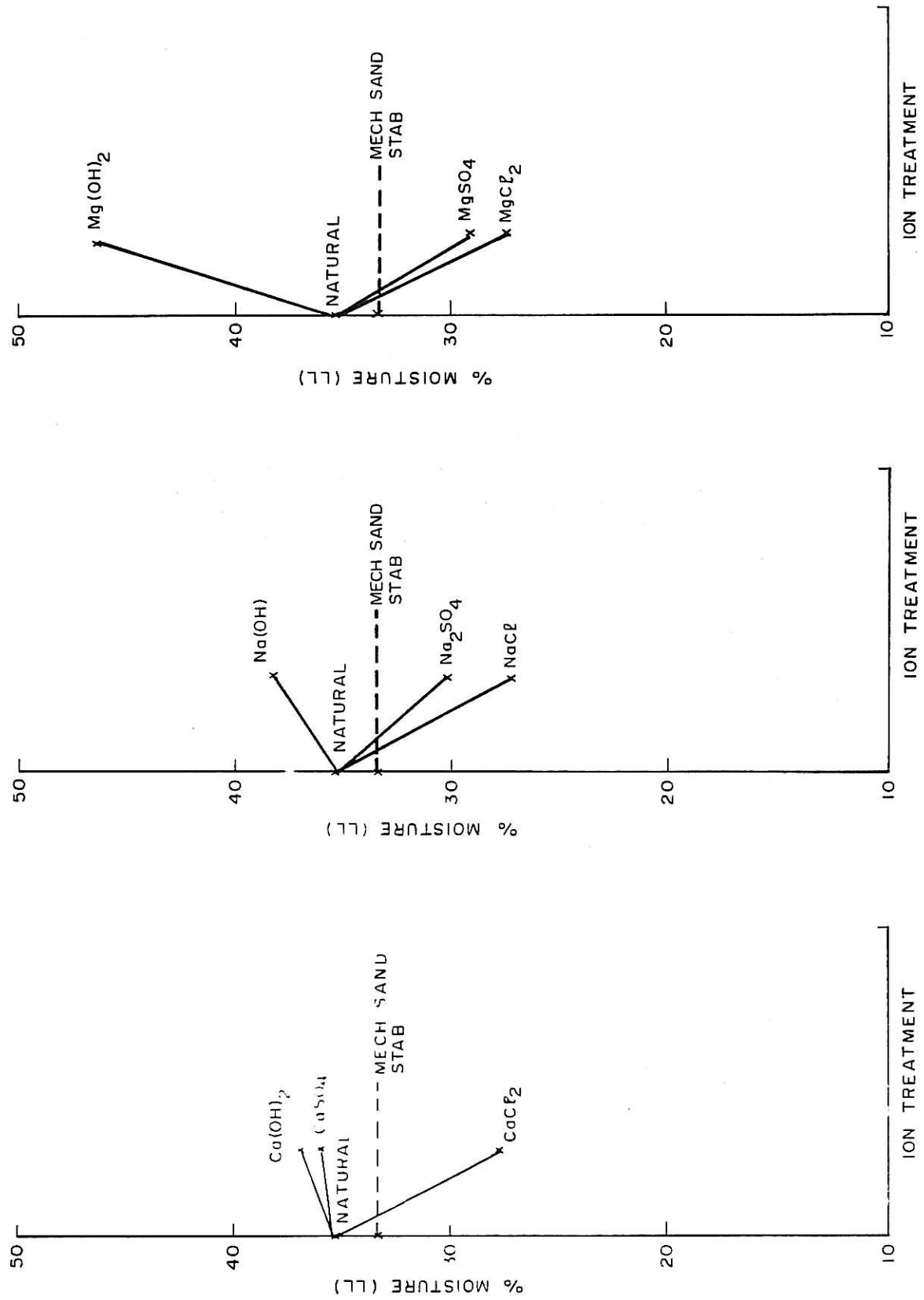


FIGURE 3.14

Cation and anion reactions of Silverton soil after treatment with 10 per cent of different additives in terms of liquid limits (4 day curing).

TABLE 3.10  
 Indicator test results of Silverton soil  
 treated with 10 % of different stabilizers.  
 The Atterberg limits of natural materials are means.

		LL	PL	PI	LS
Silverton soil  LL = 34 PL = 16 PI = 18 LS = 7	Sand	(33)*	(16)	17)	(8)
	Ca(OH) <sub>2</sub>	38	26	12	5,3
	CaSO <sub>4</sub>	22	16	6	2,0
	CaCl <sub>2</sub>	16	11	5	1,4
	Na(OH)	38	21	17	3,6
	Na <sub>2</sub> SO <sub>4</sub>	21	19	2	-
	NaCl	16	13	3	-
	Mg(OH) <sub>2</sub>	46	31	15	5,7
	MgSO <sub>4</sub>	16	11	5	0,7
	Mg Cl <sub>2</sub>	15	11	5	0,7
	5 % Ca(OH) <sub>2</sub>				
	+ 3 % sugar	16	4	2	1,3

\*Mechanical stabilization by sand cannot be compared directly with chemical stabilization and test results are therefore bracketed.

It is concluded that, compared with lime, more powerful plasticity manipulators are available. However, none of those tested is likely to produce a similar carbonation strength and very few can increase the pH sufficiently to start off the desired cement-reaction process.

It is further concluded from these test results that the LL reaction of stabilized soils also depends on the anion treatment of such soils.

(b) Montmorillonite-type soil

The technical properties of this material are given in Table 3.11 and Figure 3.16. Here it is generally referred to as black clay. It is being discussed in detail because it is the end result of the weathering process of a basic igneous rock and because it develops hydraulic strength when stabilized with lime. According to Thompson (1970), it may be classified as lime-reactive soil. Of particular importance is the amorphous matter hump at  $2\theta = 28,5$  when it is compared with Silverton soil (Figure 3.12, where this hump is at  $2\theta = 23,5^\circ$ ). It indicates by its position in the XRD diagram that it is a lime-rich glass phase, which is shown enhanced in Figure 3.16. The amorphous matter hump in Silverton soil is indicative of a lime-poor glass phase in that soil. The importance of this difference is discussed in Section 3.25.

The reaction of black clay to different additives is also presented for easy comparison of the different reaction patterns in terms of Atterberg Limits in these lime-treated soils (Figures 3.17 and 3.18). The solubility of a number of chemical compounds is listed

TABLE 3.11  
 Technical properties of Black clay (Ref. No 8421).

Origin	: Weathered basic igneous rock (Ng1 gabbro unit of the Bushveld Igneous Complex)
Locality	: NBRI test site, Onderstepoort, Pretoria, RSA
Depth	: 0-0,5 m
Grading	: See Figure 3.15
Plasticity	: LL 85,3 74,0 PL 29,6 26,9 PI 55,7 47,1 LS 24,7 15,3
Compaction and strength characteristics (Mod AASHTO)	OMC 30,0 % MDD 1 325 kg/m <sup>3</sup> UCS (OMC, MDD, Mod. AASHTO, 28 days) 296 kPa SWELL Not determined
Stabilization potential	
Gravel ICL (Clauss, 1985)	: 5,5 % (21,5 % in 24 hours)
Organic impurities (sucrose ICL, Clauss, 1985)	: 2,8 % lime equivalent
Soluble salts	
pH	: 7,6
Conductivity (soluble salts)	: 0,08 % (0,08 Sm <sup>-1</sup> at 25 °C, -19 mm dia)
Spec gravity	: 2,6435
Spec surface area (nitrogen adsorption)	: 96,4 m <sup>2</sup> /g
Mineral and chemical composition	
Quartz	: 25 % (X-ray)
Montmorillonite	: 70 % (X-ray)
Kaolinite	: 5 % (X-ray)
Allophane (Method of Fields and Perrot, 1966)	: Nil (0,0034 %)
Calcite	: Trace (inspection)
Amorphous silica (Method of Eggiman et al, 1980)	: Nil (0,04 %)



NITRR DATE 15.3.82

SAMPLE No.	8421	PROJECT 94045804
SAMPLE LOCATION	—	
DEPTH	—	JOB No. 1823
CLASSIFICATION	BLACK CLAY	

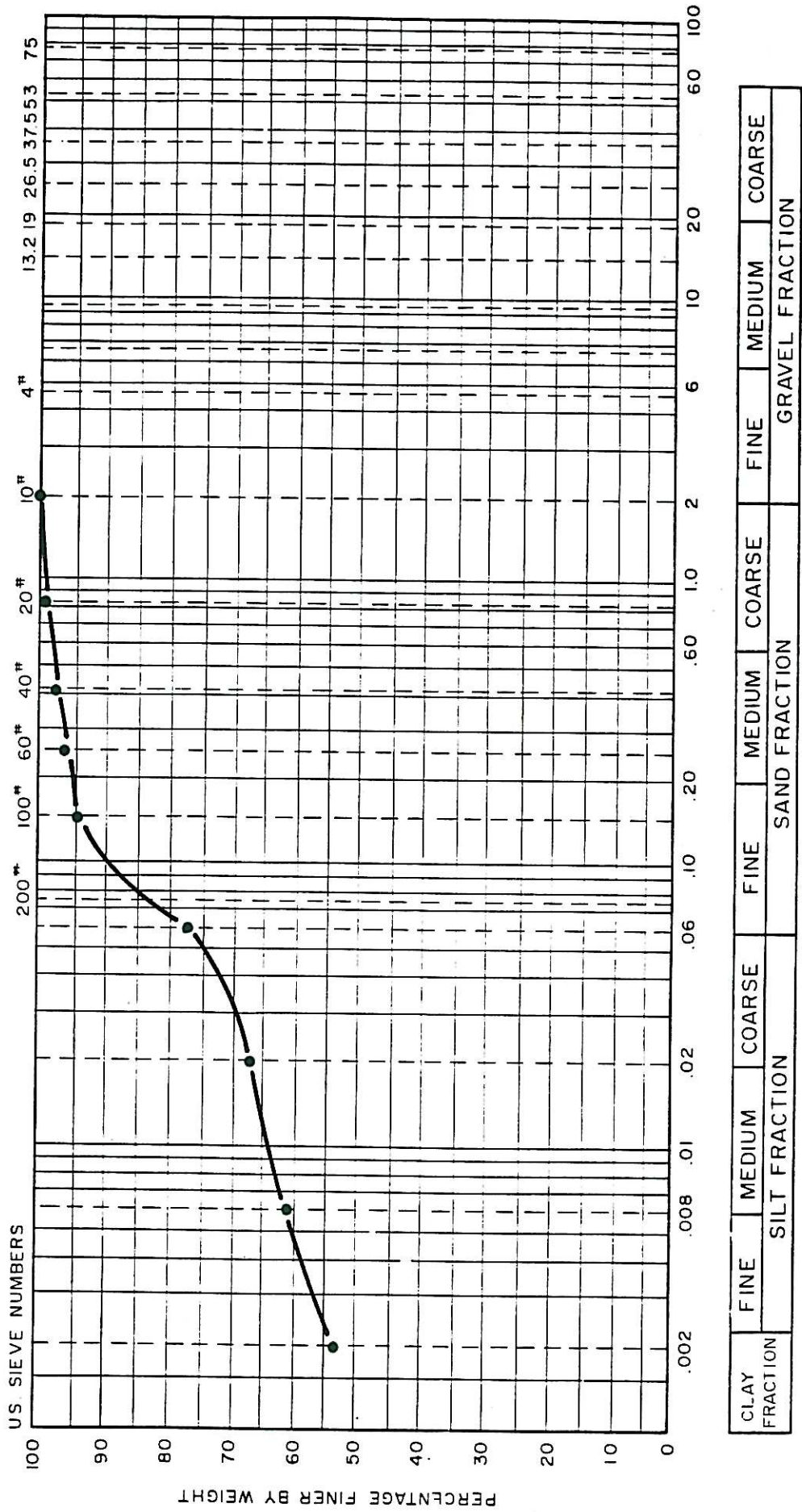


FIGURE 3.15 Particle-size distribution: black clay

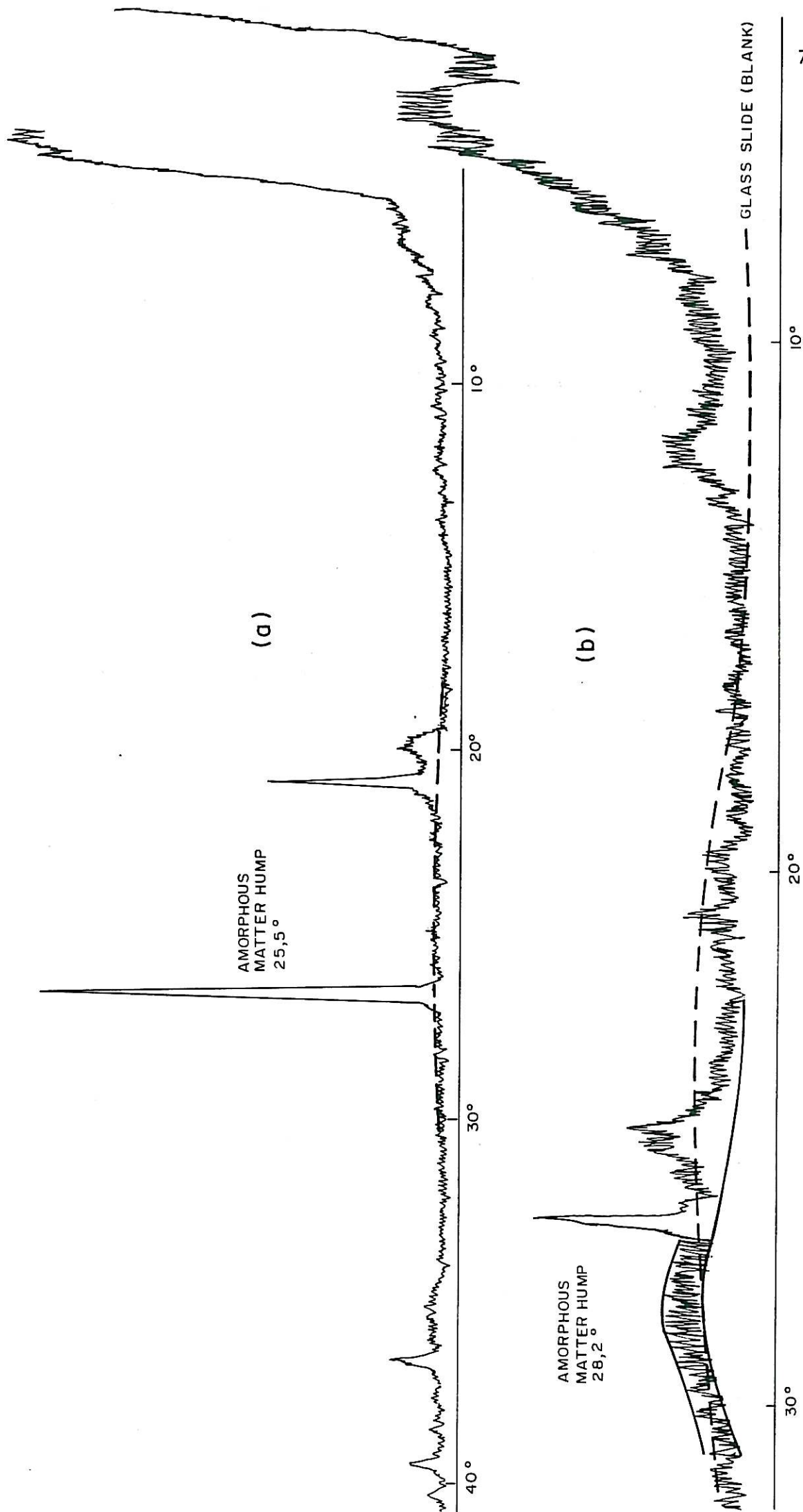


FIGURE 3.16

XRD of natural black clay (ground < 200 #; 2  $\theta$  spacing): (a) 1:1000 powder XRD; (b) 1:400 sedimented XRD.

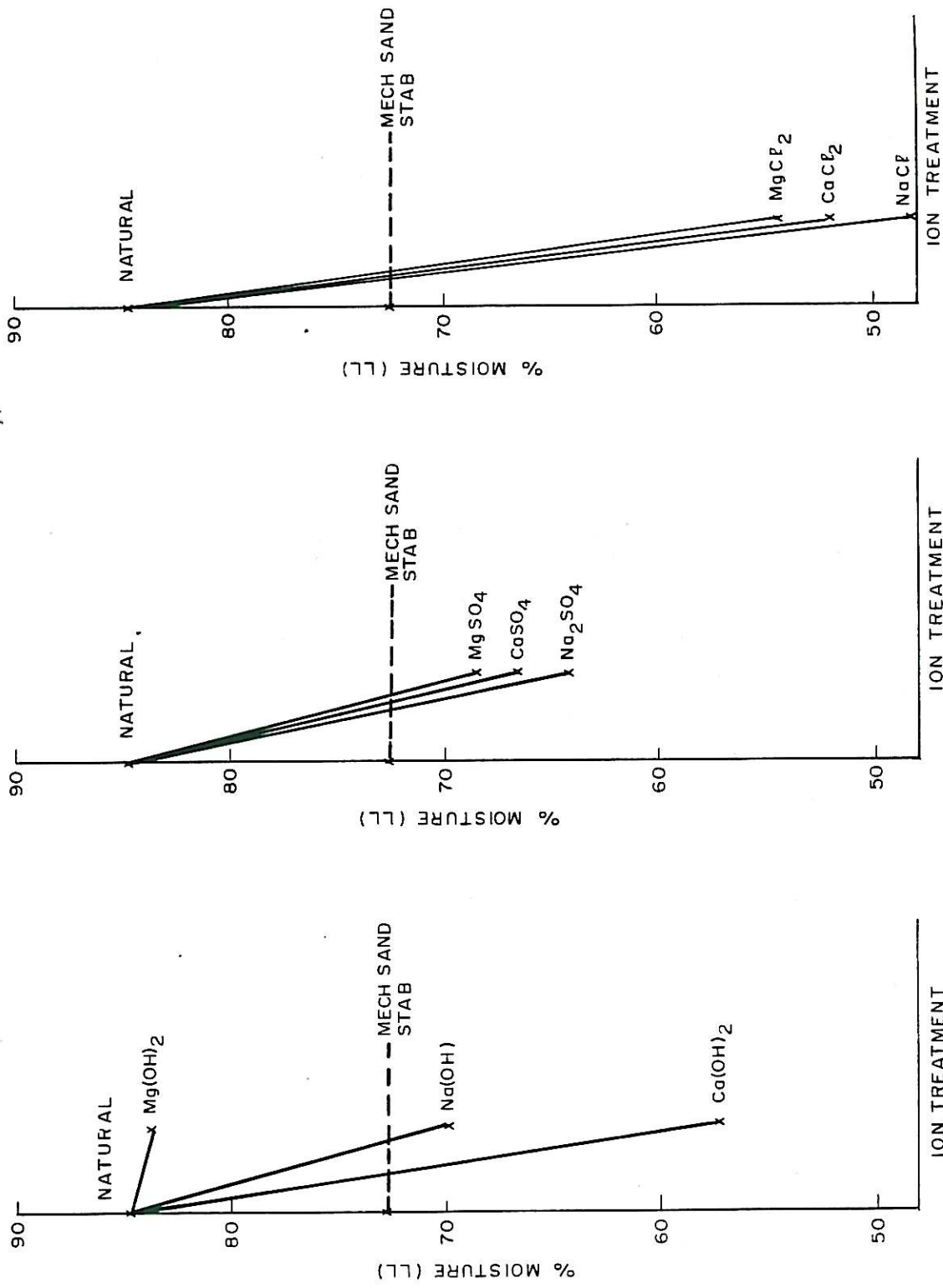


FIGURE 3.17  
Cation and anion reaction of black clay after treatment with 10 % of different additives  
in terms of liquid limit (4-day curing).

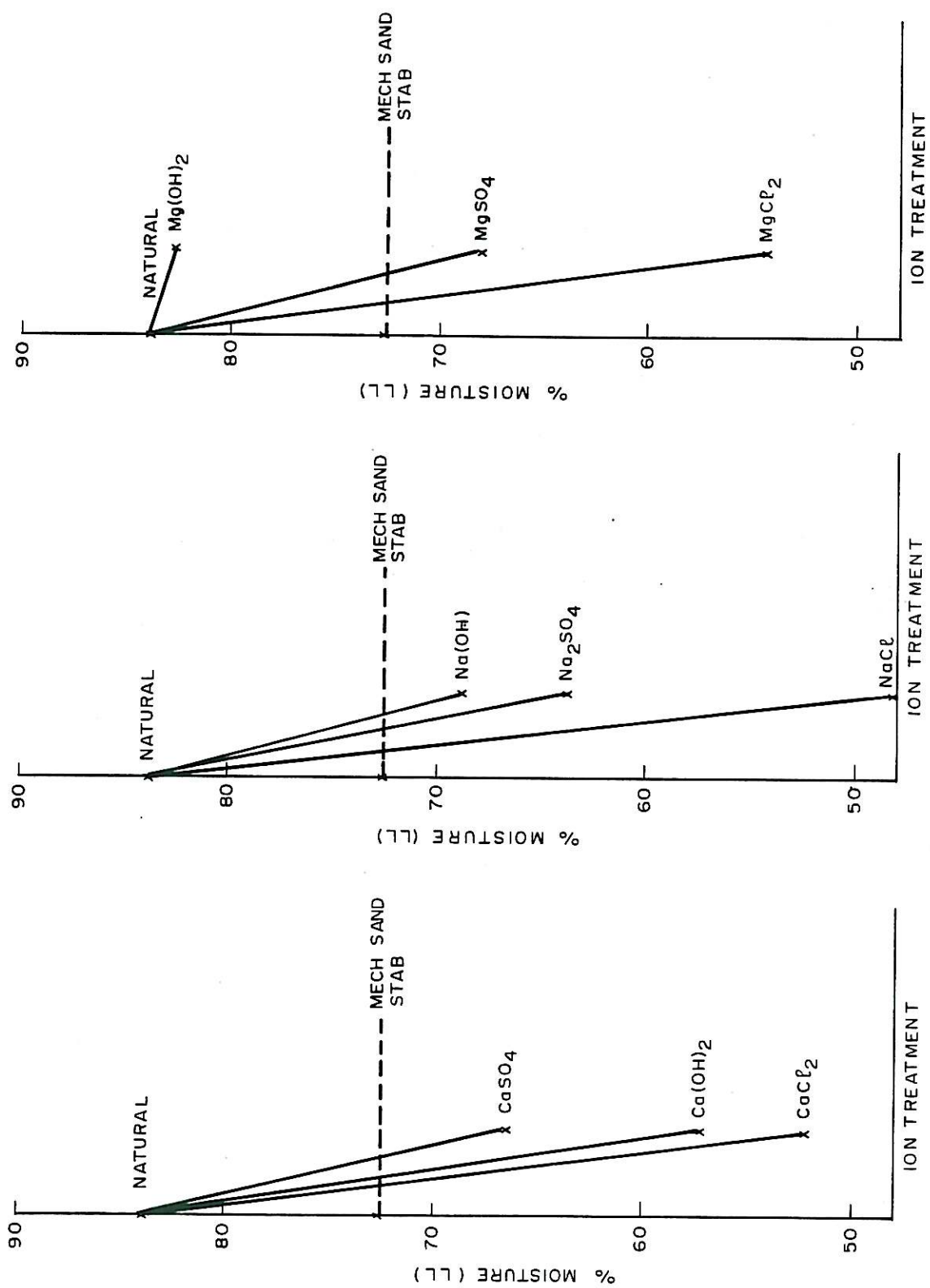


FIGURE 3.18

*Cation and anion reaction of black clay after treatment with 10 % of different additives in terms of liquid limit (4-day curing).*

in Table 3.12 to indicate the degree to which mechanical stabilization may have influenced the LL results, as given in Table 3.13.

It is concluded from these results that the LL reaction of stabilized soils depends on the cation and anion treatment of such soils.

The hypothesis of cat-ion exchange is often quoted to explain the reduced plasticity and uncured compaction strength in lime stabilized soil since  $\text{Na}^+$ -ions can hold 79 molecules of water,  $\text{Ca}^{++}$  ions only two (Kelly, 1956). It should be noted, however, that most of our South African soils are calcium saturated and this explanation may therefore be valid for a limited number of soils only.

It is further concluded from these test results that the LL reaction of stabilized soils also depends on the anion treatment of such soils.

(c) Clay in gravels

Much of the literature on soil mechanics has been devoted to the physical properties of mixtures of clays of both the kaolinite-type and the montmorillonite-type and gravel and over the years, the plasticity of road construction material became a criterion in its own right. A similar situation exists for clays in gravel with relation to chemical reactions such as stabilization work. The chemical properties of clays largely dominate the chemical properties of gravel-clay mixtures because their small particle size makes them react rapidly. The pH value in particular as a parameter for the lime demand of soils is of great importance and the discussion of the properties of clays would be incomplete without mention of these aspects. The test used for the determination of the lime demand of soils and clay containing gravels is the ICL test. It is performed on a natural material after a 1 hour reaction with lime, under controlled moisture conditions, grading and quantity of sample so that only the minimum of test variables need be considered in the inter-

TABLE 3.12  
Solubility of selected compounds in grams per 100 cm<sup>3</sup> of cold water  
(Weast, 1975-76).

Compounds	Solubility (g/100 cm <sup>3</sup> )
CaCO <sub>3</sub> (calcite)	0,0014
Ca(HCO <sub>3</sub> ) <sub>2</sub>	0,1
CaCl <sub>2</sub>	74,5
Ca(OH) <sub>2</sub>	0,185 (0 °C)
CaSO <sub>4</sub>	0,209
CaSO <sub>4</sub> ·2H <sub>2</sub> O	0,241
CaSO <sub>4</sub> ·H <sub>2</sub> O	0,3
CaCO <sub>3</sub> MgCO <sub>3</sub> (dolomite)	0,032
Mg (OH) <sub>2</sub>	0,0009
MgCl <sub>2</sub>	54,25
MgSO <sub>4</sub>	26,0
NaOH	42,0
NaCl	35,7
Na <sub>2</sub> SO <sub>4</sub>	4,76

TABLE 3.13  
 Indicator test results of black clay treated with 10 %  
 of different stabilizers. The Atterberg limits of  
 natural materials are means.

		LL	PI	PI	LS
Black clay	Sand	(72)*	(29)	(43)	(19)
LL = 85	Ca(OH) <sub>2</sub>	57	50	7	3,3
PL = 30	CaSO <sub>4</sub>	67	43	24	14,5
PI = 56	CaCl <sub>2</sub>	52	31	21	15,8
LS = 25	Na(OH)	69	47	22	14,0
	Na <sub>2</sub> SO <sub>4</sub>	64	39	25	11,3
	NaCl	48	33	15	7,6
	Mg(OH) <sub>2</sub>	83	48	35	18,2
	MgSO <sub>4</sub>	68	35	33	15,3
	MgCl <sub>2</sub>	55	30	25	16,5

\*Mechanical stabilization by sand cannot be compared directly with chemical stabilization and test results are therefore bracketed.

pretation of results. The test method is described in paragraph 3.6.5.1(1) below; the relevant soil reactions are discussed immediately below.

(1) Material type

A maximum ICL of 3,5 % of the fines fraction for natural basic igneous rock as used in roads was recommended by Clauss and Loudon (1971) because it had been found during the investigation of stabilization failures in the past that the fines portion of natural material of all failed roads (doleritic materials) had an ICL of 3,5 % or more, and the limit was given to reflect this experience. However, today it is believed that, if sufficient lime ( $\text{Ca(OH)}_2$ ) is added to satisfy the lime demand of a soil as tested, most, if not all, soils can be stabilized successfully. The measured ICL of different soils and gravels as demonstrated in Figure 3.3 indicates a reaction potential in the lime-soil-water system which is worthwhile to pursue even for perfectly sound material such as quartzite (Clauss, 1982).

It is concluded from this that the test is sensitive enough to indicate different lime demands in different soils and gravels.

(2) Extrapolation in time of the 1 hour test results

Lea (1970) claims that the semi-stable, strength giving calcium-silicate hydrates (CSH) shift to stability as long as the pH remains above 10,5. It was therefore suggested that this pH limit be used as some kind of optimum lime content. However, the comparison of the ICL of a lime-reactive soil such as black clay (decomposed basic igneous rock) with that of Silverton soil, a weathered shale and lime non-reactive soil indicates an ongoing soil/lime reaction with time (Figure 3.3). This is shown by the very rapid lime saturation of Silverton soil compared with an almost asymptotic, gradual



approach of the ICL in black clay. The 1 hour ICL of black clay was measured to be 5,5 %. Continued measurements of the same sample over 48 hours established a lime consumption of 21,5 % at which time the experiment was terminated. This indicates that there is a difference between the CSH-stabilizing pH of 10,5 and the lime saturation pH of 12,4 (at 25 °C), which represents a safety factor for continued soil/lime reaction at least for all basic igneous rock gravels. It should also be noted that the ICL of 21,5 % refers to black clay, the end product of weathering in basic igneous rock. Gravel as used in road construction has much lower 48 hour ICLs, generally not more than 5 to 8 %.

It is concluded from this that the criterion of the lime saturation pH after 1 hour contains a safety factor for continued soil/lime reaction that is advantageous and should be used to its full potential.

### (3) Moisture content

Eades and Grim (1966) originally proposed in their test method a mixture of 20 g soil fines (-0,425), 100 cm<sup>3</sup> of water and 2,3 etc % lime. Clauss and Loudon (1971) retained this ratio, with the modification that the lime demand of the water be satisfied separately, ie the reading of a pH of 12,4 at 25 °C must be corrected for a quantity of lime to saturate 100 cm<sup>3</sup> of water. Ways and means were therefore sought to overcome this problem by reducing the moisture content to such a degree that directly comparable pH measurements were possible.

This was achieved by measuring the pH of the soil/lime mixture at moisture saturation of the soil (Jackson, 1969). Moisture saturation is reached when all soil pores are filled with water. The moisture content of the sample during testing is thereby reduced to approximately one-fifth of the water content required by the original test. The correction factor for the quantity of additive

(lime) would consequently have to be reduced from 1,5 % to 0,3 % and would therefore have been far below the technically achievable accuracy of 2 to 3 % in stabilization work.

A similar method was applied in the concrete industry when BS 812: 1967 specified the composition of the test sample as follows: 50 g sand, 5 g Portland cement and 12,5 cm<sup>3</sup> water. The composition of natural soil generally varies to such a degree that an absolutely defined water complement would render the test sample soupy in one instant but relatively dry in another. The suggestion of Jackson (1958) was therefore adopted. He proposed moisture saturation of soil to the extent of excluding air from the soil.

It was found in some 350 applications that this moisture content is very near the OMC (generally 4-6 % higher) so that the ICL reading of the wholegrading sample reflects near-construction conditions, and consequently no correction factor was necessary.

It is recommended, therefore, that the moisture content of material to be tested for saturation lime content (ICL) be not more than the saturation moisture content as defined by Jackson (1958, 1969).

#### (4) Size of sample

Clauss and Loudon (1971) originally recommended the use of a 1 kg sample each with 2, 3, 4, 5 and 6 per cent of lime and 5 l of distilled water for the determination of a whole-grading ICL. This would then be compared with a fines-only ICL and the difference used as a correction factor for extrapolating the fines-only ICL for the whole grading. The use of such a large sample was, however, not practical, even for the determination of a correction factor.

During further experimentation it was found that it made no difference to the test result whether 10 g, 200 g or 1 kg of

material was used (Figure 3.19). However, it was also found that it was not easy to obtain a reasonably representative sample of 20 g mass even with sample splitters since a 10-12 mm diameter particle itself weighs 20 g. No such problem was encountered after the quantity of the test sample had been increased to 200 g of mass (dry) or even to 1 kg.

It is recommended, therefore, that ICL testing be carried out on samples of 200 g mass (dry).

#### (5) Grading

##### . Fines fraction

Eades and Grim (1966) proposed the measurement of lime requirements of soil in the  $-0,425$  mm fraction. This is considered adequate for soil modification (plasticity manipulation). It is not adequate when strength aspects come into consideration as is common practice in South Africa. The ICL of different soil or gravel fractions was measured and it was found that the fines fractions generally produced a higher ICL than the coarser fraction.

The sample was a decomposed diabase. The grading and Atterberg limits are recorded in Figure 3.20. A number of particle size ranges were then isolated and tested separately, as indicated (Figure 3.20 and Table 3.14), and it can be seen from the test results that the finest fraction required the highest quantity of lime (7,5 %) to reach saturation pH (ICL). However, even the coarse fraction still had a measurable ICL and for practical reasons it is therefore proposed to use the whole grading for testing.

##### . Coarse fraction

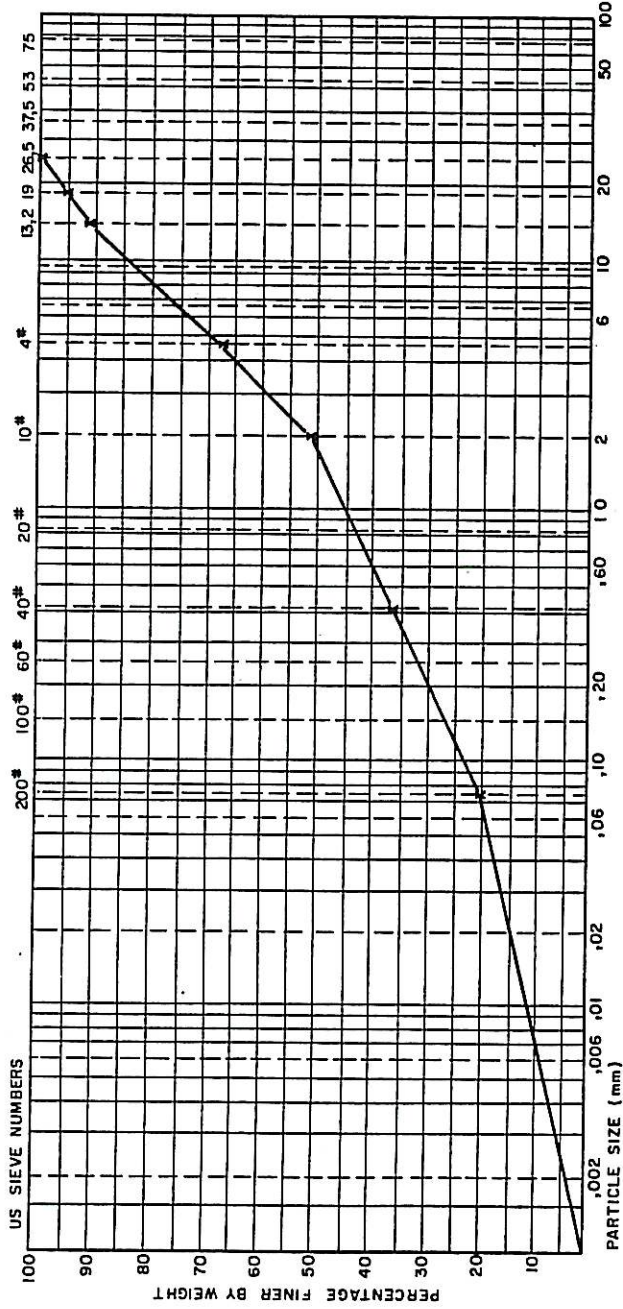
The coarse fraction needed only 4,5 %  $\text{Ca}(\text{OH})_2$  to reach saturation pH. This indicates so great a variation that, measuring

PARTICLE-SIZE DISTRIBUTION

DATE

NITRR

SAMPLE No.	8858	F 355 / 81	PROJECT 45804
SAMPLE LOCATION	P 966	(PIETERSBURG)	
DEPTH	0,0 - 1,5		
CLASSIFICATION			JOB No. 112



CLAY FRACTION	FINE SILT FRACTION		SAND FRACTION		GRAVEL FRACTION	
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE

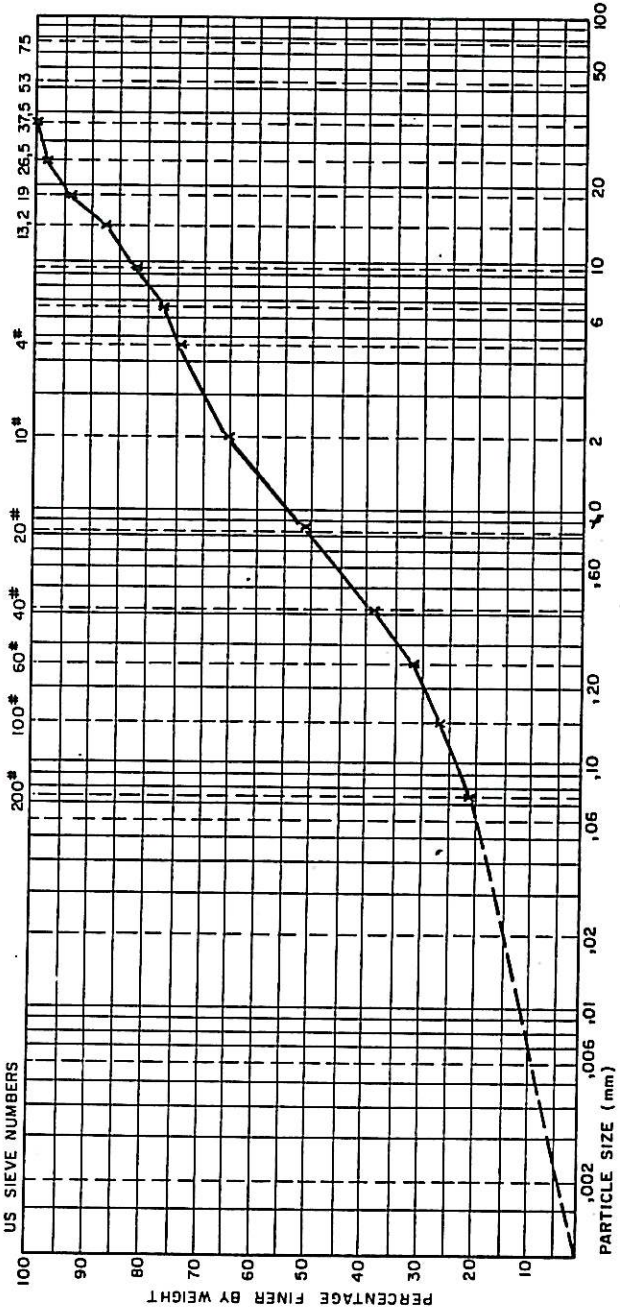
INDICATOR ICL (20 g SOIL, 100 cm <sup>3</sup> H <sub>2</sub> O)	2,5 %					
GRAVEL ICL (20 g GRAVEL, 100 cm <sup>3</sup> H <sub>2</sub> O)			2,5 %			
GRAVEL ICL (200 g GRAVEL, 1000 cm <sup>3</sup> H <sub>2</sub> O)			2,5 %			
GRAVEL ICL (1000 g GRAVEL, 5000 cm <sup>3</sup> H <sub>2</sub> O)			2,5 %			

FIGURE 3.19  
ICL of different quantities of ferricrete.

NITRR

DATE

SAMPLE No.	8872	PROJECT	45 804
SAMPLE LOCATION	10 km EAST OF CSIR,	N4	SOUTH LANE
DEPTH	ROAD CUT		
CLASSIFICATION	NAT. 1 + 13,2 mm, WEATHERED, BASIC, IGNEOUS ROCK		
	JOB No. 1 795 - 112		



CLAY FRACTION	SILT FRACTION		SAND FRACTION		GRAVEL FRACTION	
	FINE	COARSE	FINE	COARSE	FINE	COARSE

PREPARATION AND METHOD	ICL (%)			
	< 0,075 mm	+0,075 - 0,425 mm	+0,425 - 2,0 mm	> 2,0 mm
ICL	7,5	5,5	5,5	4,5
INDICATOR ICL	6,5 ?			
NATURAL -0,425 mm				
NATURAL WHOLE GRADING	6,5			

FIGURE 3.20  
 ICL of different gradings in basic igneous rock.

TABLE 3.14

Determination of the initial consumption of lime (Sample No. 8872).

Job Ref No: Natural grading		Technical : CMJ					
Date : 05/05/85		Checked by: KAC					
Temperature of soil : 20,4 °C							
Mass of sample : 20 g, 100 cm <sup>3</sup> H <sub>2</sub> O (1 hour curing)							
(%) Ca(OH) <sub>2</sub> *	< 0,425 mm (pH)	Gravel+ ICL (pH)	< 0,075 mm (pH)	+ 0,075 - 0,425 mm (pH)	+ 0,425 - 2 mm (pH)	> 2 mm (pH)	ICL
2						12,20	
3						12,45	
4	12,45	12,41			12,65	12,53	
5	12,51	12,52	12,61	12,50	12,67	12,57	
6	12,55	12,61	12,62	12,58	12,68	12,60	4,5
7	12,59	12,68	12,65	12,61	12,70	12,60	5,5
8	12,60	12,70	12,67	12,61	12,70		6,5
9			12,70	12,68	12,61		7,5
10		12,70	12,68				
11		12,70					

\*Dry mass

+200 g, saturation moisture content, whole-grading, 1 hour.

only the -0,425 mm (Eades and Grim, 1966), the -2 mm (Weinert, 1980) or the -4,75 mm (Standards Association of Central Africa, 1974) fraction would be too inaccurate for practical stabilization work without introducing correction factors.

It is concluded from this that only the whole particle range of a soil/gravel may be used for reliable results.

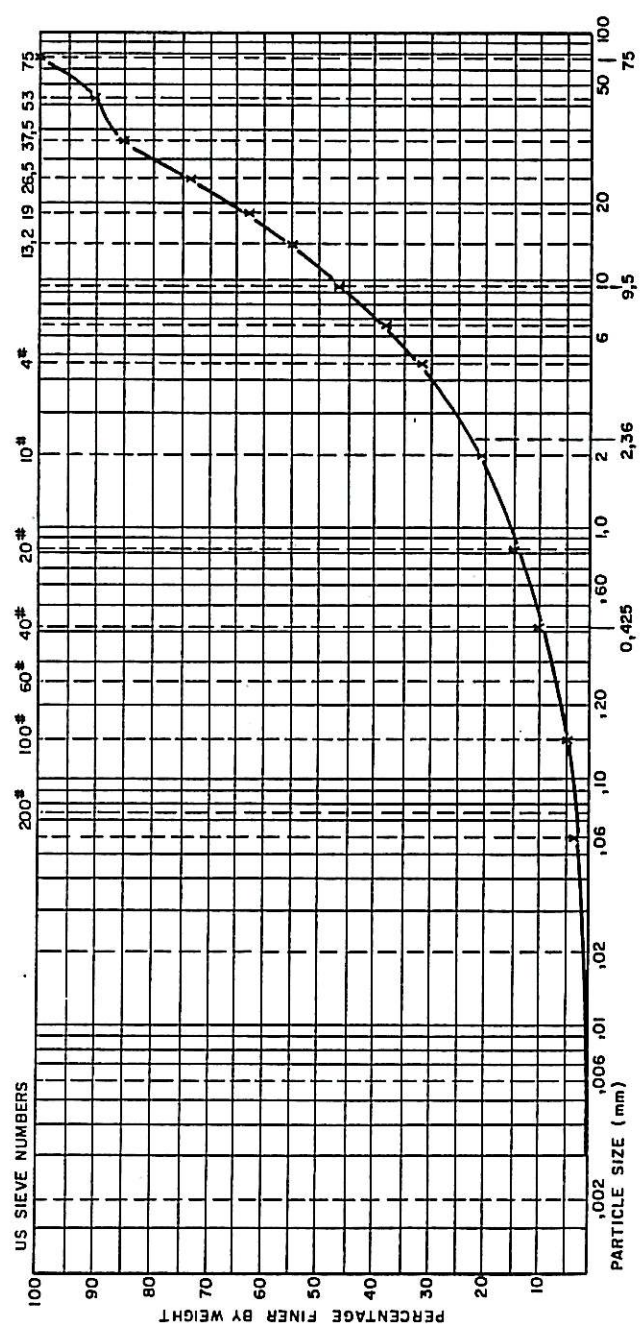
#### (6) Grinding

The results from Figure 3.20 show that the coarse fractions of weathered diabase also have an ICL, though one that is smaller than that for the finer fraction. The coarse material was therefore ground to pass 0,425 mm and the experiment was repeated. It was found that the ground material produced a larger ICL than the comparable, intact, coarse fraction (Figure 3.21). The question arises, therefore, whether the test material should not be ground generally to pass, say, the 0,425 mm sieve for a pH test in order to establish the worst possible conditions. This contradicts, however, the well-established and long-standing principle in stabilization work that in the test procedure one should simulate actual construction practices and not worst possibilities.

Natural road gravels may contain relatively large aggregates. It is, again, common practice, for example, for CBR testing either to remove aggregates larger than 19 mm in diameter before testing, or to break them up to a size smaller than 19 mm and then use them. The same treatment of the larger fraction (+ 19 mm) is suggested for this test (Buckle, personal communication, 1982).

It is concluded from this that the lime saturation of road gravel should be measured on samples of material as used in actual construction work.

SAMPLE No.	8637	PROJECT	SWAZILAND JOB
SAMPLE LOCATION	BP 2		WEATHERED BASALT
DEPTH			
CLASSIFICATION	BASECOURSE	MATERIAL	JOB No. 112



CLAY FRACTION	FINE SILT FRACTION		SAND FRACTION		GRAVEL FRACTION	
	FINE	MEDIUM	FINE	MEDIUM	FINE	MEDIUM
	6,5	—	—	—	—	—

BASECOURSE FRACTIONS OF TOTAL BASECOURSE, BUT LARGE AGGREGATES, BROKEN TO < 9,5 mm, 20 g + 100 ml	6,5	5,5	2,36-4,75	1,5	0,5
BASECOURSE FRACTIONS OF TOTAL BASECOURSE GROUND TO < 0,425 mm, 20 g + 100 ml	—	5,5	2,5	1,5	1,5
TOTAL BASECOURSE, BUT LARGE AGGREGATES BROKEN TO < 9,5 mm, 20 g + 100 ml		4,5			
TOTAL BASECOURSE, BUT GROUND TO < 0,425 mm, 20 g + 100 ml		6,5			

FIGURE 3.21  
ICL determinations of differently ground basecourse



### (7) Weathering

This was studied on a weathered basalt from Swaziland (Figure 3.21). An attempt was made to work on a mean grading, ie the grading indicated near the mass of 50 % of the sample, which is 9,5 mm for the sample used in Figure 3.21. Larger pieces were broken to pass the mean grading (-9,5 mm). More intense grinding (to  $< 0,425$  mm) was also carried out and this increased the ICL from 4,5 % to 6,5 %. In less weathered material the effect of grinding was to decrease the ICL. The effect of grinding on ferricrete was to increase the ICL.

It is concluded from these readings that breaking of the soil particles should be reduced to a minimum. Whole-grading samples are recommended for testing.

### (8) pH drift

A best fit curve of ICL readings in lime-stabilized black clay is presented in Figure 3.3, where it may be seen that the approach to saturation pH takes place very gradually. By contrast, the graph of ICL readings in lime-stabilized Silverton soil shows a very rapid and definite approach to saturation pH. It is believed that these two types of ICL curve are significant for two types of soil-lime reactions to take place, ie carbonation (Silverton soil) and the hydraulic cement reaction (black clay).

The drift in pH readings during ICL measurements was found to be caused by two factors:

- (a) an ongoing reaction in the lime/soil mixture, and
- (b) an instrumentation error caused by inaccurate calibration.

. Ongoing reaction drift

The gradual approach of the lime stabilized black clay towards saturation pH in particular is ascribed to the ongoing process of cement reactions which was not found in lime-stabilized Silverton soil (Clauss, 1982). It should also be noted that the satisfaction of the ICL can be considered safe and completed only for Silverton soil-type soils. Soils of the black clay-type may require a pH reserve for a continued cementation reaction which so far only the specification of the saturation pH may be able to provide safely.

. Calibration drift

The lime-saturation pH of soil suspensions as determined by a pH meter does not always coincide with predicted values and so far the practice has been to accept constant pH readings as saturation pH. In many cases, the drift was found to be related to temperature. Incorrect temperature compensation was, in fact, the reason for (a) pH readings far above theoretical values and (b) a substantial needle drift. This was found when the pH of a stabilized soil at 25 °C was 12,4 and, after being heated to 40 °C, reached the same pH value after some time (unadjusted pH meter). The reading of the pH at a specified time, say 60 seconds without visible drift, appears now to be a sensible proposition, although this practice was used originally to control the efficiency of the electrode but was discarded later on. A computerized version of this drift time control has been built into more modern instruments such as the Metrohm 654 pH meter.

(d) Conclusions

Problems related to pH determinations in lime-stabilized soil/gravel are related mainly to grading, sample mass, moisture content, preparation method (grinding effort) of the material and possibly ongoing soil/lime reactions.

The ICL test method is sensitive enough to determine the different lime demands of different construction materials.

Different grading ranges were found to have different ICL and the ICL test must therefore be carried out on whole-gradings if it is to be reasonably representative.

The quantity of sample tested was not found to influence the result if sufficiently representative and a 200 g mass was found sufficient and convenient.

The moisture content in stabilized road gravel to be tested for lime saturation should be not more than saturation moisture content.

The grinding of weathered material was found to increase or decrease lime consumptions in soil/gravel depending on soil type and it is suggested that this be avoided.

The pH drift was particularly difficult to analyse. It may be caused by a faulty instrument or calibration, damaged electrodes or even ongoing soil/lime reactions.

A whole-grading sample is recommended for testing.

The criterion recommended for the definition of lime saturation of a gravel is 'constant' pH readings in stabilized mixes.

#### 3.2.4.2 Physical properties

The physical properties of soil as mentioned below are discussed because they may influence the interpretation of the quoted test results. Soil physics has been exhaustively discussed by Kezdi (1979) and the interested reader is referred to that work.

The particle size of reaction agents has a marked influence on the speed of the chemical reactions (Motto and Melstead, 1966: de L Beyers and Mostert, 1977). It is also true, however, that the particle size of the material on which a chemical agent acts is of equal importance. Another aspect requiring consideration is the 'reaction' moisture for seasoning, the effect of air during compaction in addition to such general conditions of standard procedures as optimum moisture content (OMC), maximum dry density (MDD), compaction effort and curing.

(a) Clay lumps

The UCS of Silverton soil stabilized with different gradings of carbide lime and Marvello road lime was investigated. The analysis of all test results did not show a correlation between the strength gain and particle size of stabilizers already shown valid for plasticity test results (see Figure 3.7). Respectively, 83 % and 75 % of some 192 UCS test results showed no correlation between strength and particle size of the stabilizer (by sieve analysis) for four-day and 28-day curing.

The UCS of the carbide lime-treated black clay appears to be slightly less than its Marvello lime-treated equivalent. The UCS of Silverton soil was the same for both limes. However, the scatter for the UCS results of lime treated black clay (20 % out-layer results) was much higher than in Silverton soil (0 out-layer results) (Figure 3.2 and 3.1). The reason for this may probably be found in the sample preparation which produced a uniform mix for lime-stabilized Silverton soil. It produced a rather bad, non-uniform mix for lime-stabilized black clay showing little black clay lumps of approximately 3 mm diameter floating in the treated matrix.

Unconfined compression strength tests were therefore carried out with a material which had been ground to -425 micron. It may be seen that the UCS in Silverton soil remained at the same level (Figure 3.22) although the accuracy of test results of cured samples

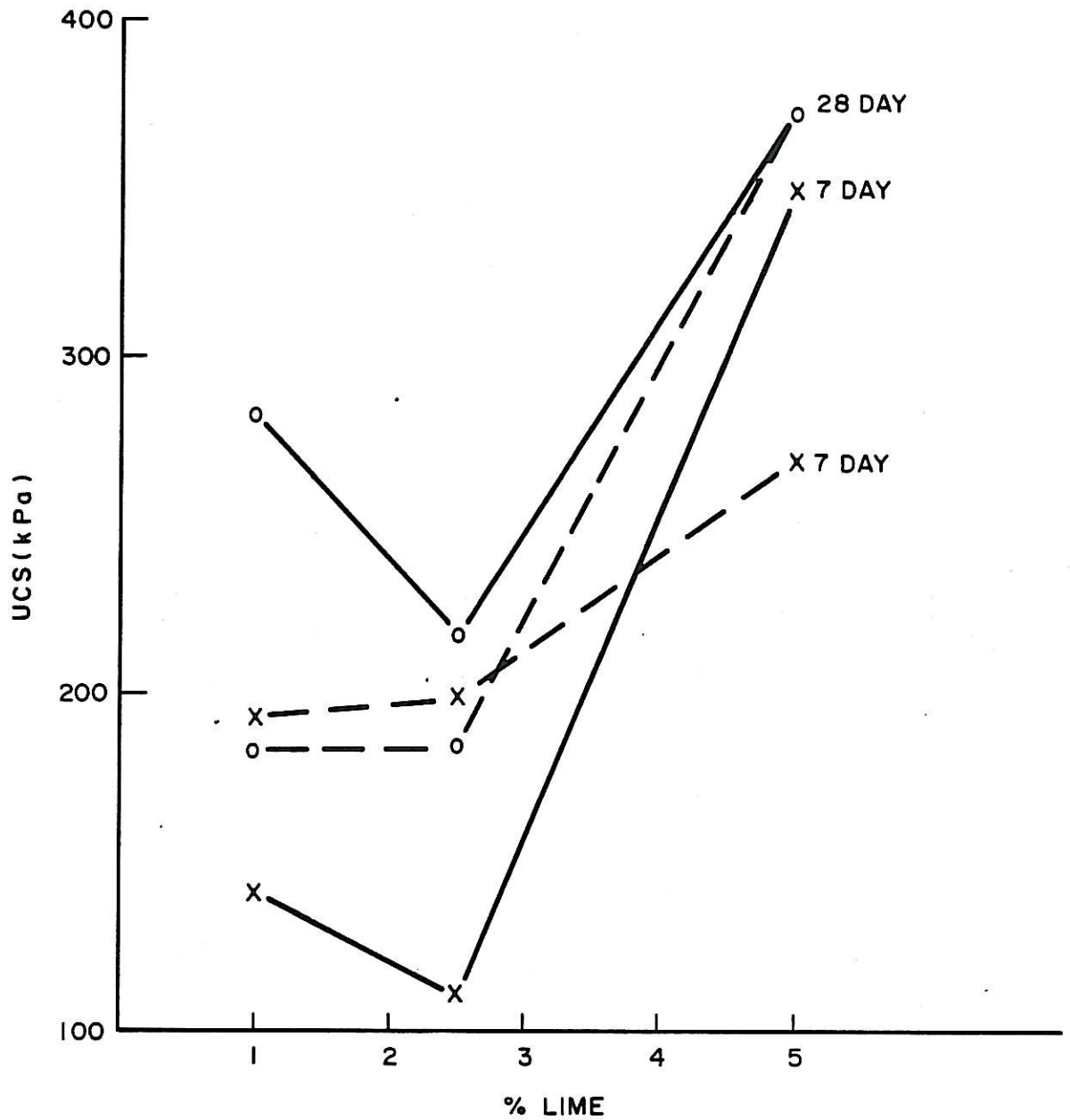


FIGURE 3.22

*UCS of lime-stabilized Silvertown soil, ground to  $\leq 425 \mu\text{m}$  (OMC, MDD). The strength of equivalent pellet mixes ( $\leq 3 \text{ mm}$ ) is given by the dotted line.*

seems to increase dramatically. The UCS of 18 % lime-treated, ground black clay increased, however, from the 750 kPa level to as much as 2 430 kPa after 28 days' curing (Figure 3.23). It is also interesting to note the response in terms of strength gain with increased quantities of lime which is more sensitive in fine-grained mixes.

It is also shown, however, that a better mixing of lime and soil - in effect finer soil - will result in an increased strength.

(b) Workability

Although the physical properties of stabilized soil are outside the scope of our consideration, it is worth mentioning that a small portion of clay in construction materials increases workability. The limit amounts are documented in the relevant specifications and should be adhered to.

(c) Conclusions

The technical properties of clay which are of importance in stabilization work are as follows:

(1) Clay type

The physical properties of clays depend largely upon their chemical composition but may be summarized under the headings of kaolinite-type soil and montmorillonite-type soil. Of particular importance for strength development is the amorphous matter content. In kaolinite-type soil it appears to be a lime-poor glass phase, while in montmorillonite-type soil it appears to be a lime-rich glass phase.

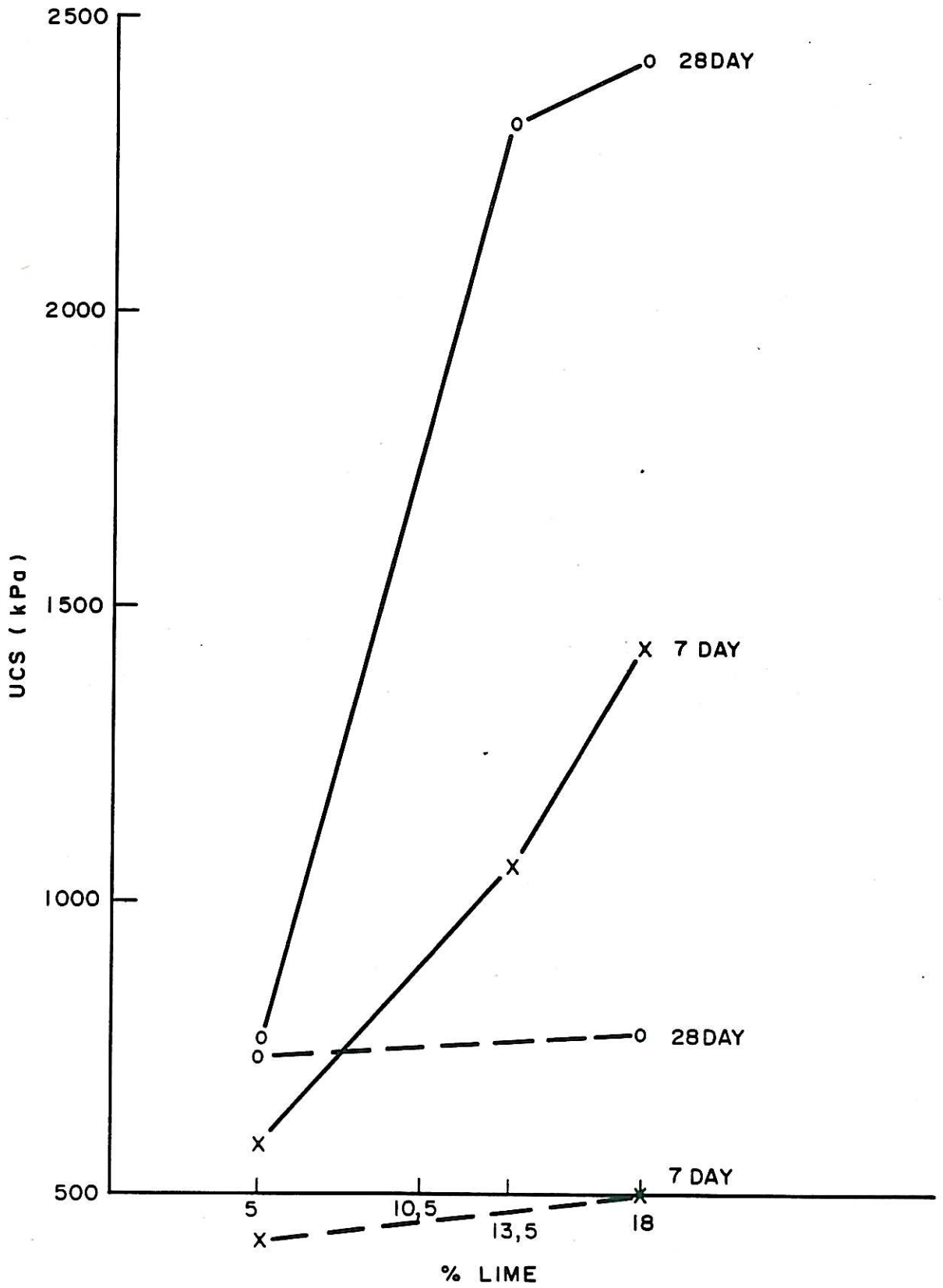


FIGURE 3.23

UCS of lime-stabilized black clay ground to  $< 425 \mu\text{m}$  (OMC, MDD). The strength of equivalent pellet mixes ( $< 3\text{mm}$ ) is given by the dotted line.

## (2) Grading

Clay is generally encountered as mineral aggregates which disintegrate further when treated mechanically. The chemical reaction is based on these micro-particles of clay. The finest fraction of soils is often referred to as clay fraction, since it may contain very small minerals of clay and of clay size. The clay-size fraction may also contain amorphous matter. The difference between amorphous matter and clay-size minerals is that, whereas minerals are crystallized, amorphous matter is not.

Most of the standard tests become unreliable when the clay aggregates are too large.

## (3) Plasticity

Clays are very sensitive to moisture and their plasticity is high because their moisture storage potential is high. The plasticity of montmorillonite-type clays is much greater than that of kaolinite-type clays.

The liquid limit in particular is a very sensitive parameter for chemical reactions upon lime treatment. It reflects plasticity changes after the treatment of clays with anions and cations.

## (4) Stabilization potential

Kaolinite-type soils were not found to react hydraulically when treated with lime. Montmorillonite-type soils were found to respond well hydraulically when treated with lime.



### (5) Hydration

Clays are very sensitive to moisture and swell considerably when wetted. The swelling may be manipulated by treatment with lime (and other chemicals).

### (6) Workability

Clay in too large or too small quantities make a soil unworkable.

## 3.2.5 CEMENT AND POZZOLANITY OF SOILS

The best known pozzolan is without doubt Portland cement. It is an artificial product and subject to world-wide quality control. It is composed in such a way that, unlike most natural pozzolans, all its components participate in a chemical reaction when water is added. However, in order to establish the meaning of pozzolanity, natural pozzolans have also to be considered since it is from them that the term derived. Much may be learned regarding the pozzolanity of soils by the extrapolation of knowledge from the well-investigated problems in the cement chemistry to soil/lime reactions. In particular, it is believed to be a useful yardstick for the engineering assessment of the previously described reaction compounds in stabilized soil.

### 3.2.5.1 Description of pozzolans

The making of pozzolans such as Portland cement may be described in a more systematic way, as shown in Figure 3.24.

A number of materials known to have pozzolanic properties are described below. These are

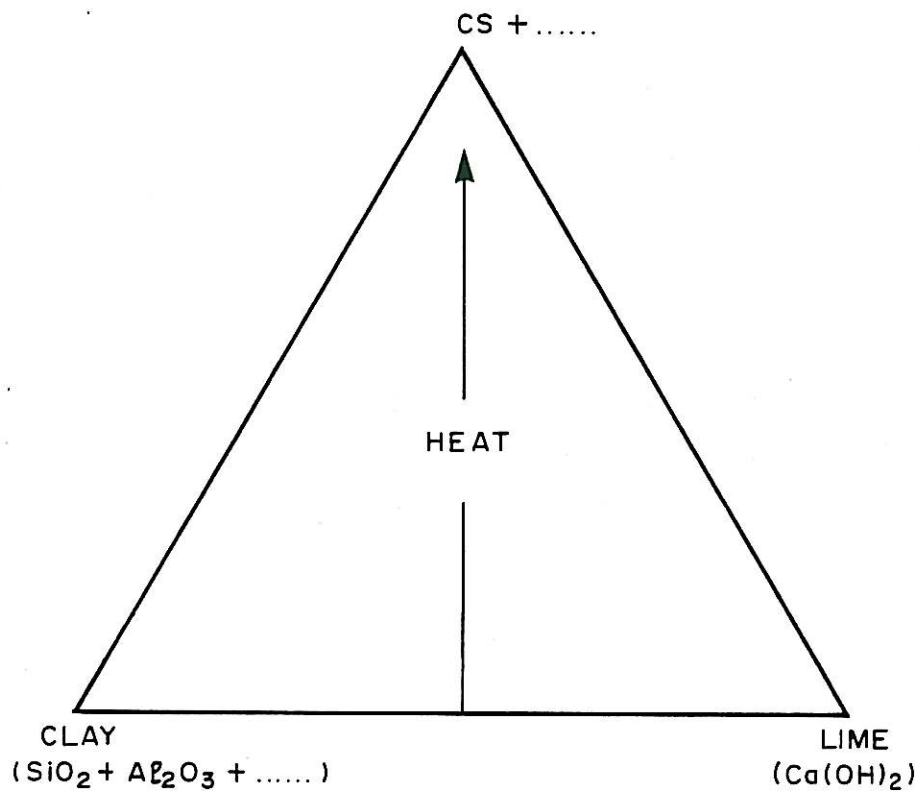


FIGURE 3.24

*Diagram of cement formation (1300 °C - 1400 °C for one or two minutes during the travelling time in the cement furnace).*

- (a) Pozzolanic mortars
- (b) Portland cement
- (c) Slagment
- (d) German trass

- (a) Pozzolanic mortars

'The Greeks and Romans made a pozzolanic mortar by mixing finely ground volcanic material with lime, sand and water, Pozzolana harden by reacting chemically with lime. The name derives from the Italian town Pozzuoli where a suitable volcanic tuff was found' (Encyclopaedia Britannica, 1971).

'Artificial pozzolanas are prepared by burning at suitable temperature certain clays, shales and diatomaceous earth containing a proportion of clay' (Lea, 1970).

'Current thinking is that calcium hydroxide combines with reactive silica and alumina compounds to form calcium hydrosilicates and aluminates substantially the same as those produced during the hydration of portland cement' (Le Sar, in Fulton, 1969).

- (b) Portland cement

Fulton (1969) has summarized the reactions that take place during the hardening process of cement, as follows:

'Tricalcium silicate has all the essential properties of portland cement and is chiefly responsible for the early strength of cement-water pastes. Dicalcium silicate hardens slowly, contri-

buting little to the strength until after twenty-eight days, but much to the strength at later ages, up to periods of many years.\*

Tricalcium aluminate<sup>†</sup> liberates a large amount of heat during the first few days of hardening, but makes no contribution to the cementing action other than accounting for the initial set, and adding slightly to the early strength.

It is responsible in a large degree for the susceptibility to sulphate attack of portland cement concrete, the higher the  $C_3A$  content of the cement, the lower being the sulphate resistance.

Tetracalcium aluminoferrite hydrates rapidly, and develops very little strength. It is valuable as a flux during manufacture.

Magnesia is a minor constituent of most raw materials.

Gypsum is interground with the clinker in order to control the rate of setting of the cement. Free lime is present in the clinker because of incomplete reaction in the kiln.

Of the other minor constituents of cement the alkalis, viz. the oxides of sodium and potassium, are of considerable interest. The deterioration of many concrete structures in the United States has been attributed to reactions between the potassium and sodium hydroxides in the set cement and reactive forms of silica contained in the aggregate. The reaction is expansive, resulting in extensive random cracking of the concrete.'

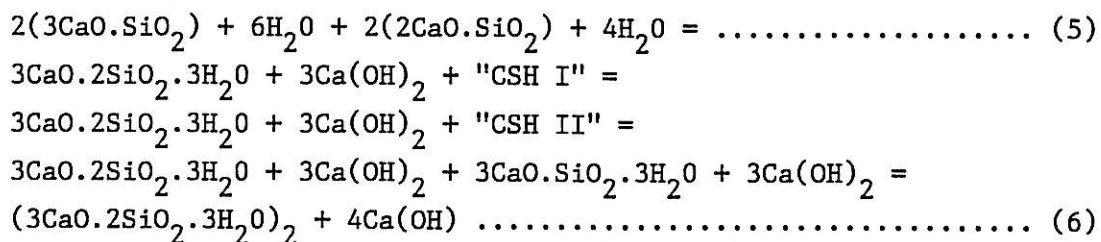
The main chemical reactions of calcium and silicate may be described as follows (after Lea, 1970):

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\*The tricalcium and dicalcium silicate phases in Portland cement amount to 70-80 % (Ramachandran, 1969). Lea (1970) estimates  $C_3S$  at 45 % and  $C_2S$  at 25 % respectively of OPC. Fulton (1969) quotes 55-95 %  $C_2S$  and  $C_3S$  in South African Portland cement.

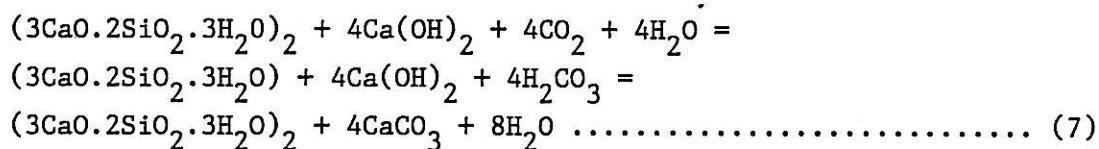
<sup>†</sup>Fulton (1969) estimates the  $C_3A$  compound in South African OPC to be 5-12 % and the  $C_4A$  compound at 5-10 %.

## Hydration



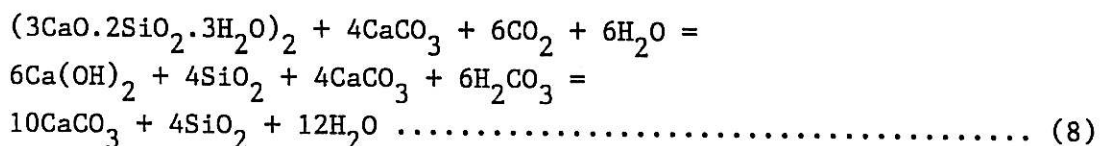
Formula (5) describes the hydration of tricalcium ( $\text{C}_3\text{S}$ ) and dicalcium silicate ( $\text{C}_2\text{S}$ ) to the tricalcium silicate hydrate ( $\text{C}_3\text{SH}$ ) and concurrent release of calcium hydroxide (Formula (6)). The dicalcium silicate hydrates in particular are passing through slow CSHI and CSHII phases, which appear to be responsible for final strength considerations. CSHI and II are reportedly also found in lime-stabilized montmorillonite as reaction products (TRB, 1976). The reaction is an intermediate phase and its direction of progress depends on the prevailing pH.

## Hardening (pH &gt; 12,2)



The system remains stable in terms of concrete technology if the pH is maintained at approximately 12,4 and the stabilization chemistry follows as nearly the cement chemistry as the soil/lime mixture will allow (Formula (7)) ie the necessary ions must be available from the soil-lime mix. Such pH level is usually not difficult to maintain during the hardening phase of the cement since a fresh cement paste was found to develop a pH of 13,45. This is sufficient pH reserve generally to correct for aggregate properties which would tend to consume  $(\text{OH})^-$  ions and depress the pH in the mix. It is, however, not sufficient in the case of some fine aggregates as described in BS812:1943, 1967 and is generally also not sufficient for South African natural gravels and soils as used in soil stabilization.

Hardening failure (pH < 11)



Most of the stabilization failures investigated in the past (1969-71) are believed to have been caused by the low pH of the stabilized material (Formula (8)), because reconstruction with the addition of more lime stabilizer was successful in all cases known to the author.

The collapse of existing (hydraulic) cementacious strength within 18 hours was observed in lime-treated slagment. Eighteen per cent lime-stabilized slagment was cured for 90 days under de-ionized H<sub>2</sub>O. After this period, the material was so hard that it could not be broken by hand. The colour of the fresh surface was uniformly dark green. The pH was 12,5. It was dried for 18 hours at room temperature and could thereafter easily be broken by hand. The fresh surface was still predominantly dark green, but showed many little white specks, approximately 0,1 mm in diameter. The pH was 9,5. This seems to confirm the earlier conclusion that CaCO<sub>3</sub> not precipitated from CO<sub>2</sub> oversaturated solution does not produce strength (Formula (4a)); it also appears to confirm the rapid decomposition of cement minerals at a pH below 10,5-11,0 (Formula (8)).

A pH control of natural soils and gravels must therefore be executed in general stabilization work in South Africa in order to allow the cement reaction in the form of calcium silicates and secondary calcium carbonates to take place. The decomposition of tricalcium silicate hydrates without the formation of dicalcium silicate hydrates - ie the loss of early strength without the formation of final strength (Fulton, 1969) - proceeds unchecked at a pH below 10,5-11,0 (Lea, 1970).

The release of lime ( $\text{Ca(OH)}_2$ ) during the hardening process of cement (concrete) has been estimated to be some 20 % of the cement added (Fulton, 1969). This means in effect that a 5 % cement-stabilized soil undergoes a secondary lime stabilization phase equal to the effect of a 1 % lime stabilization, but after hardening of the cement. The soil particles are, at this stage, already rigidly fixed in position by the hardened cement and the only conceivable way of accommodating the resulting shrinkage (modification) must be sought in the formation of shrinkage cracks. The remedy suggesting itself is the addition of lime to the cement additive before mixing. The quantity recommended is the quantity of lime released by the cement reaction. In the case of 5 % cement stabilized soil, this would be 1 % of lime. It has the effect of reducing the shrinkage of (plastic) soil by released secondary lime. The lime in this case must, of course, be calcitic lime in order to avoid unknown variables that may conceivably be expected when dolomite lime is used. It should also be noted that in this case the ICL by the soil treated may not be more than 2 % as shown later when the ICL is discussed in more detail.

The efficiency of a slagment/lime stabilizer may be analysed in a similar way. The mix proportion that is mostly used in slagment/lime stabilization is 1:1. However, the mix proportion giving the highest strength was found to be at a ratio of approximately 20 % lime to 80 % slagment (Clauss, 1984). This ratio leaves about four-fifths of the lime added free to react with the soil. In the case of 5 %, 1:1 slagment/lime stabilized soil, 2 % lime may react with the soil a possible pozzolan while 0,5 % (1/5) of the lime is consumed in the pozzolanic reaction with slagment. Two per cent of lime is a substantial quantity of stabilizer and, according to experience, generally sufficient to avoid secondary phase shrinkage cracking.

(c) Slagment

Slagment is the trade name for milled blast-furnace slag. It reacts hydraulically on its own and displays an increased strength development when mixed with lime (Figure 3.25). It is produced exclusively from granulated slag.

Granulation is the formation of grains by the rapid cooling of larger pieces of hot slag in water. It increases the reactivity of the slag by causing a larger portion of the molten slag to remain in the amorphous phase and by suppressing the formation of non-reactive minerals.

(d) German trass

Trass is a ground tuffa of volcanic origin. It is said to harden in the absence of air because 30-35 %  $\text{SiO}_2$ , which is soluble in weak acid, forms nearly insoluble calcium silicate when lime is added to it (Stegman, 1941).

### 3.2.5.2 Description of pozzolanity

(a) First approach

All the pozzolans previously mentioned in this text - except those involving the direct reaction of amorphous silica with lime - have undergone heat treatment and any definition of pozzolanity must therefore include the process of heat treatment of the original material and the formation of lime-reactive compounds by this heat treatment (which gives the material strength). Such a first approach makes it possible to base further discussion of pozzolanity on what happens during the reaction of Portland cement. A defini-



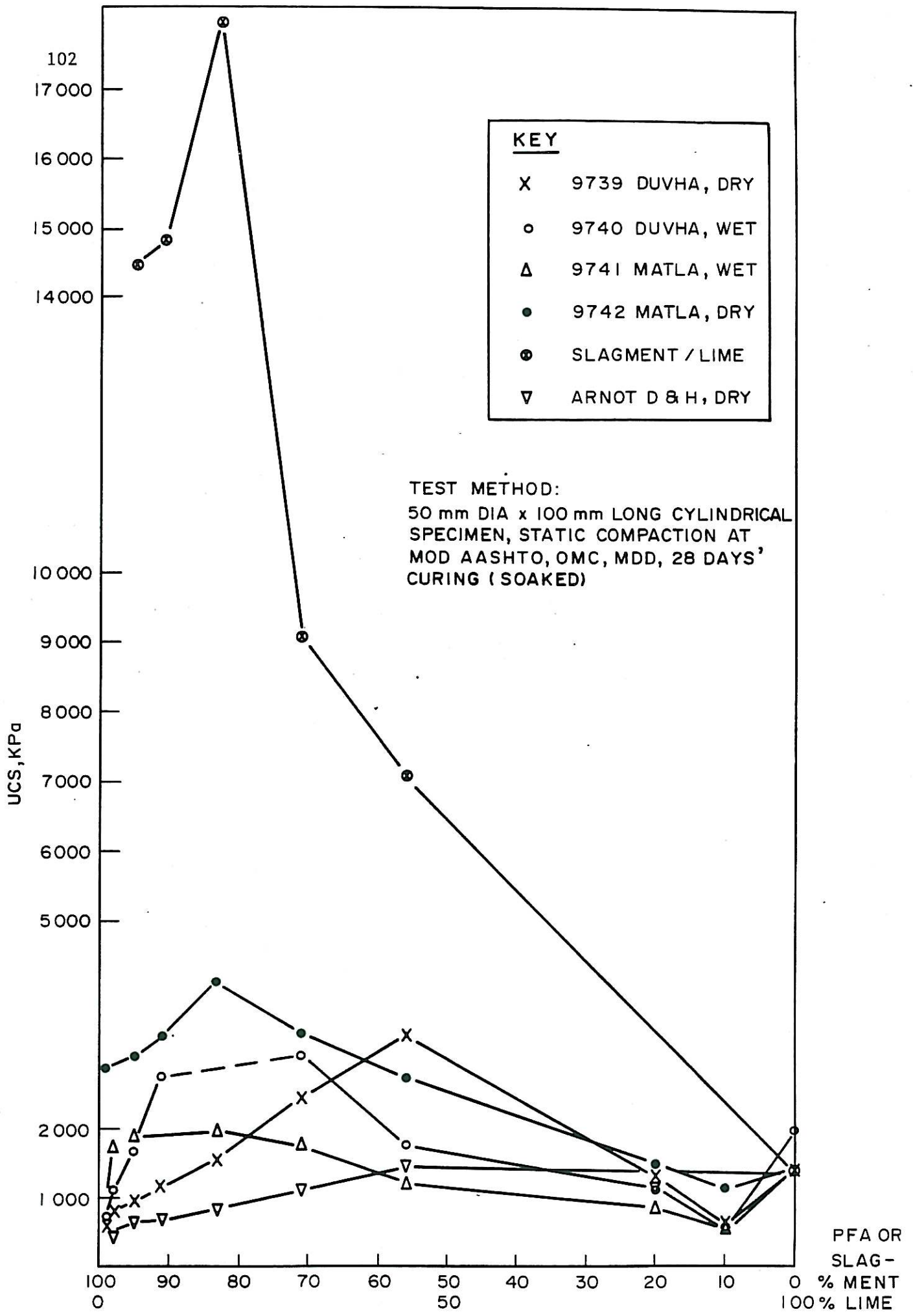


FIGURE 3.25

Lime reaction of PFA in comparison with slagment, UCS of different PFA-lime mixes.

tion of pozzolanity should therefore also include all the reactions described in Fulton's summary of cement reactions (quoted above).

Pozzolanity has also become known as a source of strength development or strength potential in suitable mixtures and may include any reaction of soil compounds with lime which results directly in greater strength, except compaction strength.

In line with its practical importance, pozzolanity denotes in particular the formation of calcium-silicate-hydrate strength, which is what is generally referred to when the term 'pozzolanity' is used in discussion.

(b) Second approach

Another aspect that the definition must incorporate is the property of pozzolans to react with lime in the absence of air. This is known as hydraulicity.

(c) Third approach

Recently, evidence has been found to show that heat treatment may not be necessary for the formation of lime-reactive calcium silicates (Clauss, 1982), because pozzolanity was also found in natural soil derived from basic igneous rock. It was also confirmed that the chemical attack on clay minerals by high concentrations of lime results in calcium aluminate and calcium silicate (De Wet and Skinner, 1967).

(d) Fourth approach

The reaction products of pozzolanity are permanent for most practical applications such as concrete. The permanence of calcium-

silicate hydrates depends on the pH level of the concrete mix in time (Lea, 1970). This becomes critical in the type of weak concentrations used in stabilization work.

Calcium aluminate hydrates are also not permanent compounds but change according to the concentration of lime in the mix (De Wet and Skinner, 1967). These compounds are, however, less important for strength development - in contrast to calcium-silicate hydrates - and may therefore largely be ignored.

#### 3.2.5.3 Definition of pozzolanity

Pozzolans are therefore materials that react with lime and/or water in the absence of air, to produce strength. Previous heat treatment is not essential. The reaction products are transient but long-lived if sufficient lime is available.

#### 3.2.5.4 Classification of pozzolans

Some of the pozzolans described previously can now be compared with one another if we use their strength potential as a comparative parameter. It appears that this is directly related to their lime:silica ratio. Note, however, that the quantities of calcium and silica also include non-reactive calcium and silica in the form of inert minerals.

The total lime content of some pozzolans, together with their total silica content, is summarized in Table 3.15. The materials listed have all undergone heat treatment and they all develop strength when water or water and lime are added. They are arranged in order of their strength potential, the highest being Portland cement and the lowest being natural pozzolans.

The importance of the total lime content in relation to the total  $\text{SiO}_2$  content in pulverized fuel ash (PFA) and PFA's strength potential was

first recognized by Diamond and Lopez-Flores (1981). The lime content in the amorphous matter phase of the XRD diagram of PFA was in fact used by them to distinguish between American PFAs of high and of low strength development and recommended for use in the ASTM classification of PFA (ASTM C-618-1978). The reactive materials listed in Table 3.15 may consequently be classified with respect to their silica content, as follows:

Portland cement	- high lime content
Slagment and lime-rich PFA	- equal lime content
Lime-poor PFA and natural pozzolans	- low lime content.

Similar XRD analyses were made of South African materials and Diamond and Lopez-Flores's findings were confirmed. The results are presented in Figure 3.26 where it may be seen that the American approach is also valid for some of our natural materials, such as black clay and Silverton soil. It could also be shown that the addition of lime to Matla PFA and subsequent melting at approximately 1 600 °C produced a shift of the amorphous matter hump in the XRD diagram in the direction of the more lime-containing and hence more reactive glass phase.

It is believed that the X-ray amorphous, lime-rich glass phase participates directly in a hydraulic reaction. However, no change in the diffraction intensity of the hump was observed for lime treated slagment and water after a 28-, 60- or 80-day reaction. This may be explained by the time delay known to exist between the onset of strength and the long period of strength gain in concrete. It may also be seen that the X-ray intensity of slagment producing a 28-day strength of some 18 MPa and that of 5 % lime-stabilized black clay producing a 28-day strength of 0,7 MPa is the same.

Costa and Massazza (1981) tested natural Italian pozzolans with a total lime content of traces to 15 %. The 28-day unconfined compressive strength of five 15 % lime-stabilized Italian pozzolans ranged from 0,0000115 Pa (11,5 N/mm<sup>2</sup>) to 0,0000056 Pa (5,6 N/mm<sup>2</sup>).

TABLE 3.15  
Total lime and silica content of some pozzolanic materials.

Material	SiO <sub>2</sub> (%)	CaO (%)	Source
Portland cement	19-24	63-68	Fulton, 1969
Slagment	33-36	32-36	Van Rensburg, 1981
Lime-rich PFA	31	31	Diamond and Lopez-Flores, 1981
Lime-poor PFA	38-67	2-15	Willis, 1982
Natural pozzolans	43-69	Trace-15	Costa and Massazza, 1981

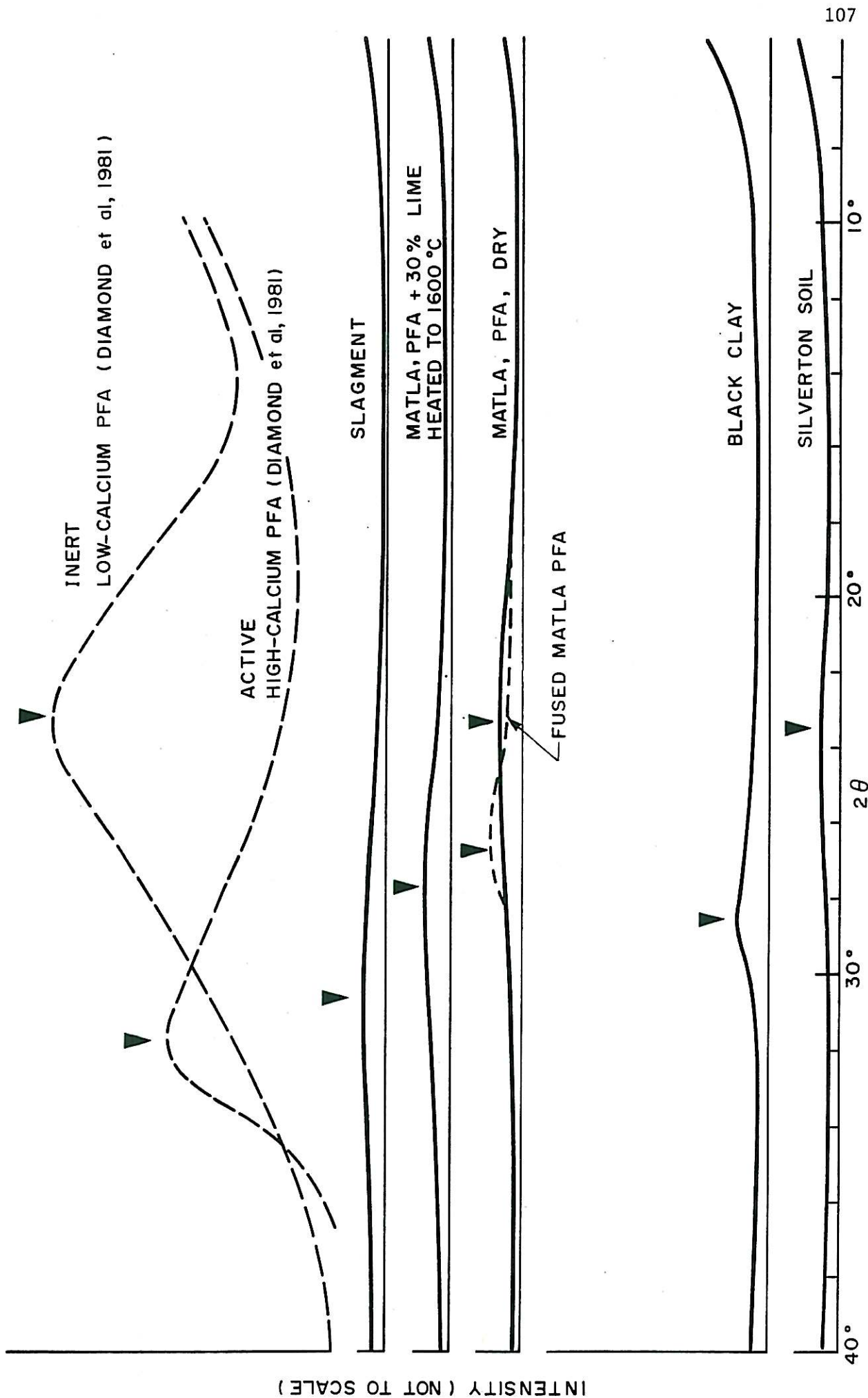


FIGURE 3.26 XRD diagram showing amorphous matter humps in inert and reactive PFA and soil.

South African PFA from Matla power station with a UCS of over 4 000 kPa and a total lime content of 6,5-15 % is therefore a highly reactive and well-developed pozzolan when compared with some natural pozzolans from Italy described previously. It was found, however, that even this high strength development of South African PFA is too poor to be of any use in stabilization work: the strength development required in soil stabilization must be comparable to a UCS of 18 000 kPa, such as that achieved by optimum lime/slagment mixes (Figure 3.25).

The difference between the amorphous matter present in kaolinite-type and montmorillonite-type soils is shown in Figure 3.12 and 3.16: a calcium-poor composition is shown for Silverton soil and a calcium-rich composition for black clay. Diamond and Lopez-Flores (1981), in particular, have drawn attention to the different lime reactivity of calcium-rich and calcium-poor glass in their analysis of pozzolanic potentials of PFA. Their findings are compared with the XRD diagrams of the two material (soil) types under discussion in Figure 3.26 and it may be seen that the evidence points towards the presence of a highly reactive, lime-rich, natural glass phase in black clay.

It is important at this stage to consider amorphous-glass reactions with lime since it was previously stated that black clay reacts hydraulically with lime. PFA does not contain any clay and it is concluded therefore that the hydraulicity of black clay is caused also by a lime-rich glass phase.

It must further be noted that the amorphous silica content of Silverton soil and black clay is 0,2 % and 0,04 % respectively, showing higher quantities of amorphous silica for the hydraulically non-cementing lime stabilized Silverton soil.

Moisture saturated, lime stabilized black clay was subsequently cured for 90 days. The UCS and the pH were determined after this curing period in order to establish the critical pH at which hydraulic cement reaction is being generated. The results are presented in Figure 3.27 where it may be seen that the critical pH level is 10,7 (interpolated) at 25 °C. No

HYDRAULIC STRENGTH (BREAKING, kPa)	pH (25 °C)	Ca(OH) <sub>2</sub> (%)
438	11,72	7,0
214	11,65	6,0
82	11,5	5,0
26	11,3	4,0
0-10 (FLUCTUATING READINGS)	11,2	3,0
0	10,15	2,0
0	9,3	1,0
0	8,35	0,5

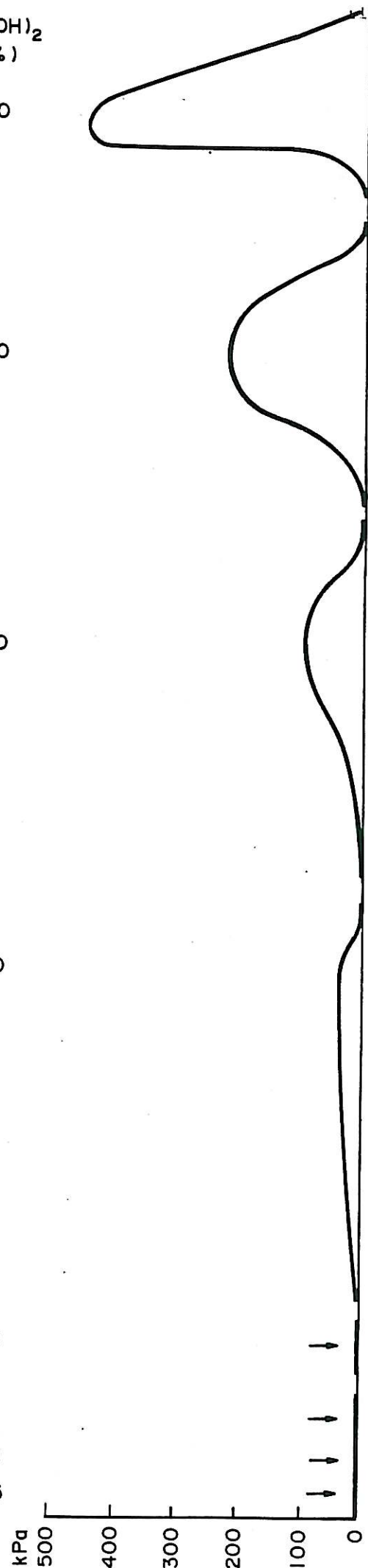


FIGURE 3.27  
Critical pH of hydraulic cement reactions in H<sub>2</sub>O saturated, lime-stabilized black clay in terms of 90-day cured UCS.



cement reaction strength is to be expected below this pH, only carbonation strength. This agrees well with the pH of 10,5 given by Lea (1970).

### 3.2.5.5 Conclusions

The pozzolanic strength development in cement, slagment, PFA and pozzolans is believed to be part of the same hydraulic 'cement' reaction, but it differs from one to the other according to the lime:silica ratio in each.

The formation of reactive calcium silicates seems to depend on the total lime content in relation to the total silica content either together with heat treatment or together with the length of time of weathering during which reactive calcium silicates are formed in the amorphous matter phase.

Black clay is considered a pozzolan but Silverton soil is not. The lime-reactive soils are also probably natural pozzolans as pointed out by Thompson (1970). Thus far the only way to show such pozzolanic potential is by XRD analyses.

The most important prerequisite for this reaction to take place in stabilization work is a high pH level in the stabilized material.

## 3.2.6 ORGANIC MATTER IMPURITIES

### 3.2.6.1 Sugar type

Lime ( $\text{Ca(OH)}_2$ ) dissolves in sugar solutions to a greater degree than in water (Smith and Halstead, 1971). The reason for this is the 100 times increased solubility of  $\text{Ca(OH)}_2$  in sugar-saturated moisture (Boynton, 1980). It was observed during experimental work that only a portion of the lime added to soil seems to take part in the stabilization reaction.

So much so that lime-stabilized soil is of a lighter colour than the relevant natural soil. It appears reasonable, therefore, when dealing with carbonation strength to consider organic impurities of the sugar type as strength-increasing additives in lime-stabilization work because they bring more lime into solution and hence into a state where subsequent precipitation produces strength (see Section 3.4.3.1). It is for the same reason that a sugar contaminated cement paste does not harden in time although the pH does not decrease during (at least) 7 days. Such hardening failure in concrete caused by organic impurities of the sugartype was described by Clare and Sherwood (1954, 1956) and would follow Formula (8).

Organic-matter impurities of the sugar type must be avoided, therefore, if the strength aimed at is a pozzolanic hydraulic cement reaction strength.

It should also be mentioned that, once achieved, the carbonation strength is permanent for all practical applications. The cement reaction strength, on the other hand, must be considered transient if the above hypothesis is correct and a powerful accelerator of the sugar type is present in the aggregate. The experimental evidence with a 1 % sugar-contaminated OPC points to a decomposition of the strength-producing but semi-stable CSHI and II within 4 weeks in surplus moisture.

#### 3.2.6.2 Coal and plant matter

The presence of coal and plant matter in construction materials is relatively rare.

The use of coal may be a proposition in mining-areas where material from waste dumps might be offered as a road aggregate. However, the problem is that spontaneous combustion, as well as the occurrence of soluble salts, must be considered as a possible hazard.

Investigations by the British National Coal Board have concluded that spontaneous combustion will be unlikely if the material has been well compacted since too little oxygen would be available for ignition. The British Ministry of Transport, however, does not allow unburnt coal to be used in road construction despite the Coal Board's assurance (Dawn, 1981).

Plant matter can, of course, not be allowed in roads because it will decompose.

### 3.2.6.3 Conclusions

The technical properties of organic-matter impurities which are of importance in stabilization work are as follows:

(a) Sugar-type impurities: Organic-matter impurities of the sugar type increases the solubility of lime in water by a factor of 100 and unbalances the chemical equation for the formation of calcium silicate hydrates in favour of  $\text{Ca(OH)}_2$ , according to Formula (8).

It must therefore be considered as a strong accelerator in the hardening of cement.

(b) Coal is a rather inert, but brittle material. The danger in its use in road construction is spontaneous combustion if it is not well compacted.

(c) Plant matter may decompose to form sugar-type compounds. It may also produce acid and attack soil aggregates such as ferricretes by dissolving the soil cement. It is not inert.

### 3.3 Soluble salts and high acidity

#### 3.3.1 SOLUBLE SALTS

Soluble sulphates will combine with the hydration products of cement and with clay/lime reaction products to form compounds of greatly increased volume such as Ettringite. This is particularly so when clay is present in quantity. The worst situation arises when sulphate enters the mix after it has hardened (or water enters a hardened material that already contains excessive sulphate) since the expansive reactions cannot then be accommodated. For example, the safe limit for sulphate in ground water in contact with concrete is considered to be only 0,03 %  $\text{SO}_3$ .

In South Africa it is also necessary to restrict the amounts of the highly soluble magnesium and sodium sulphates in order to minimize soluble salt damage to the road, which is a problem entirely separate from sulphate attack on treated materials. In this connection, a tentative limit of 0,05 %  $\text{SO}_4$  has been suggested for these two salts. Netterberg (1979) has investigated this problem in detail and recommends the application of criteria as presented in Figure 3.28.

Soluble salt failures in roads are generally restricted to artificial environments such as materials derived from waste dumps and desert conditions. They show as blisters and boils on the black top road surface.

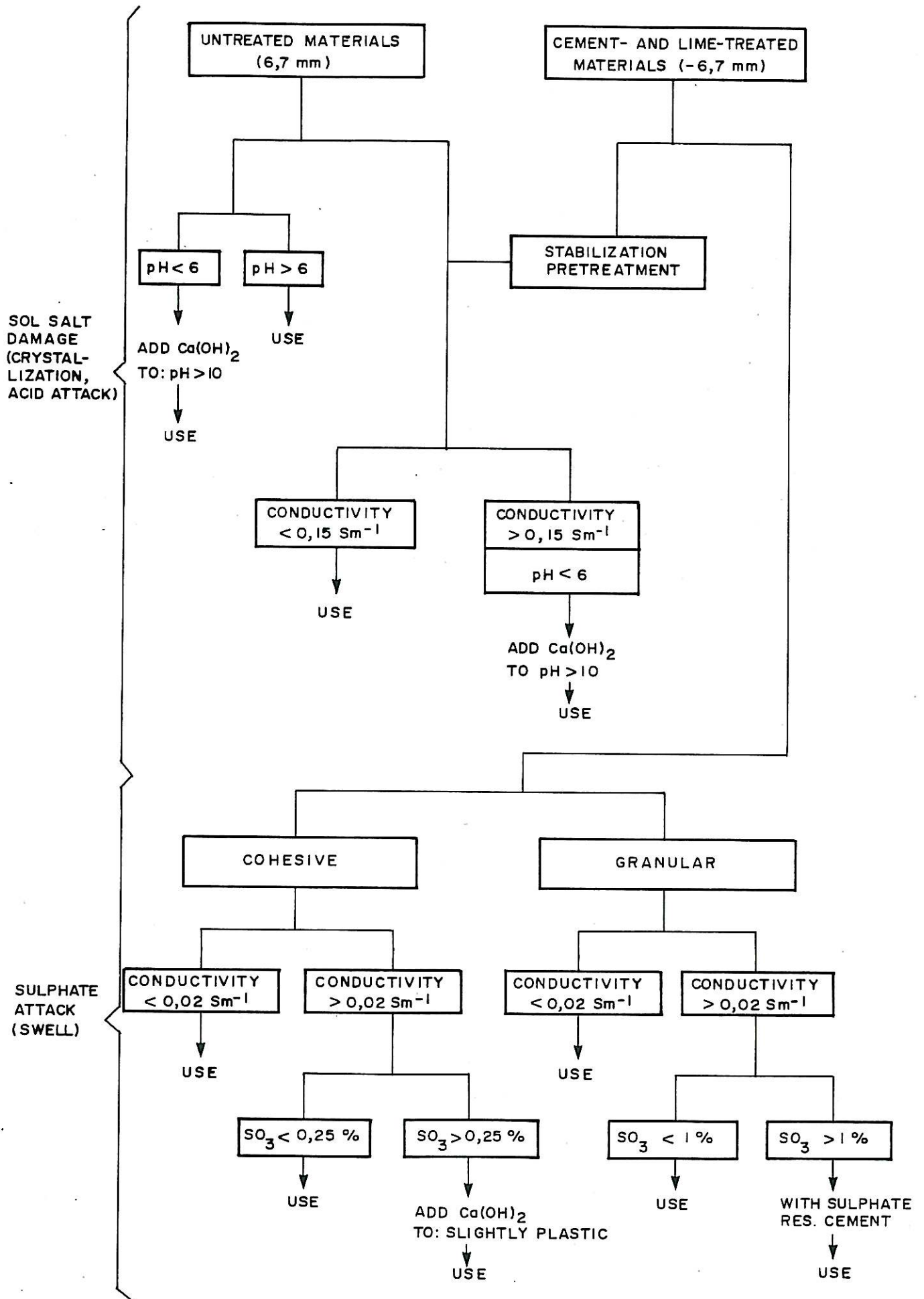
#### 3.3.2 HIGH ACIDITY

High acidity is very often found in an artificial environment eg a waste coal dump. It is caused by the decomposition of pyrite and attacks the bonds which cement soil aggregates together.

The acids in waste materials, eg mine dump sand or colliery waste are highly detrimental to cement hydration and carbonation strength. Such materials, if they have a pH below 6,0, must be pretreated with lime to

FIGURE 3.28

SALT DAMAGE TO ROADS (Netterberg 1979)– an interim guide to its diagnosis, prevention and repair (-6,7 mm). Figure to be used with report.



allow for carbonation strength and for cement reaction strength. The development of acidity is demonstrated in the following case of pyrite in colliery waste (Clauss, 1981).

Pyrite in colliery waste may cause problems, as a result of -

- (a) soluble salts and
- (b) the dissolution of ferricrete and lime cemented aggregates by acidic soil moisture.

The coal waste investigated consisted of low-grade coal with a carbon content of 43,8 % (four determinations, max 51,5, min 37,1).

Discarded coal dumps of different ages showed pH and soluble salts contents as given in Table 3.16, while the salt concentration in road layers is presented in Table 3.17. Note, however, that these data refer to a single case and may not represent typical conditions.

The pH in the compacted basecourse of a 50:50 mixture of ferricrete and colliery waste was found to be 4 and since ferricrete is soluble in an acidic environment it is possible that the Fe nodules in the base will dissolve over the years. The crocodile cracking observed might be interpreted as the first sign of dissolution instability. This was investigated in more detail and it was found that the pH dropped from neutral to 3,5 if the sample was stirred and was subjected to dry-wet cycling (Figure 3.29). The mass of an isolated ferricrete nodule increased initially and showed a loss of 0,8 % of the original mass after 125 days of reaction. The increase of mass was approximately 1,5 % in 20 days.

### 3.4 Stabilization reactions

The likely causes of the stabilization reactions have been discussed in Section 3.3 and we now discuss the engineering parameters and manifest-

TABLE 3.16  
Weathering of pyrite in waste coal.

	pH <sup>4</sup>	% soluble salts <sup>3</sup>
1 day	7,2 <sup>1</sup>	0,03 <sup>1</sup> (0,3 Sm <sup>-1</sup> , 25 °C)
3 weeks	7,4 <sup>2</sup>	0,04 <sup>2</sup> (0,3 Sm <sup>-1</sup> , 25 °C)
0,5 years	5,0 <sup>2</sup>	0,23 <sup>2</sup> (2,3 Sm <sup>-1</sup> , 25 °C)
1,5 years	5,1 <sup>2</sup>	0,24 <sup>2</sup> (2,4 Sm <sup>-1</sup> , 25 °C)

Notes

- 1 0,425 mm crushed
- 2 0,425 mm fraction
- 3 Conductivity method CX 21-74 (TMH1, 1979)
- 4 Two determinations each from one sample
- 5 LECO furnace

TABLE 3.17  
Salt concentration in road layers which consist of waste coal.

pH <sup>1,4</sup>	pH <sup>1,4</sup>	% soluble salts <sup>3</sup>
0-50 mm	3,5	0,28 (2,8 Sm <sup>-1</sup> , 25 °C)
50-200 mm	3,5	0,10 (1,0 Sm <sup>-1</sup> , 25 °C)
200-350 mm	2,8	0,15 (1,5 Sm <sup>-1</sup> , 25 °C)
350-500 mm	2,1	0,25 (2,5 Sm <sup>-1</sup> , 25 °C)

Notes

- 1 0,425 mm crushed
- 2 0,425 mm fraction
- 3 Conductivity method CX 21-74 (TMH1, 1979)
- 4 Two determinations each from one sample
- 5 LECO furnace



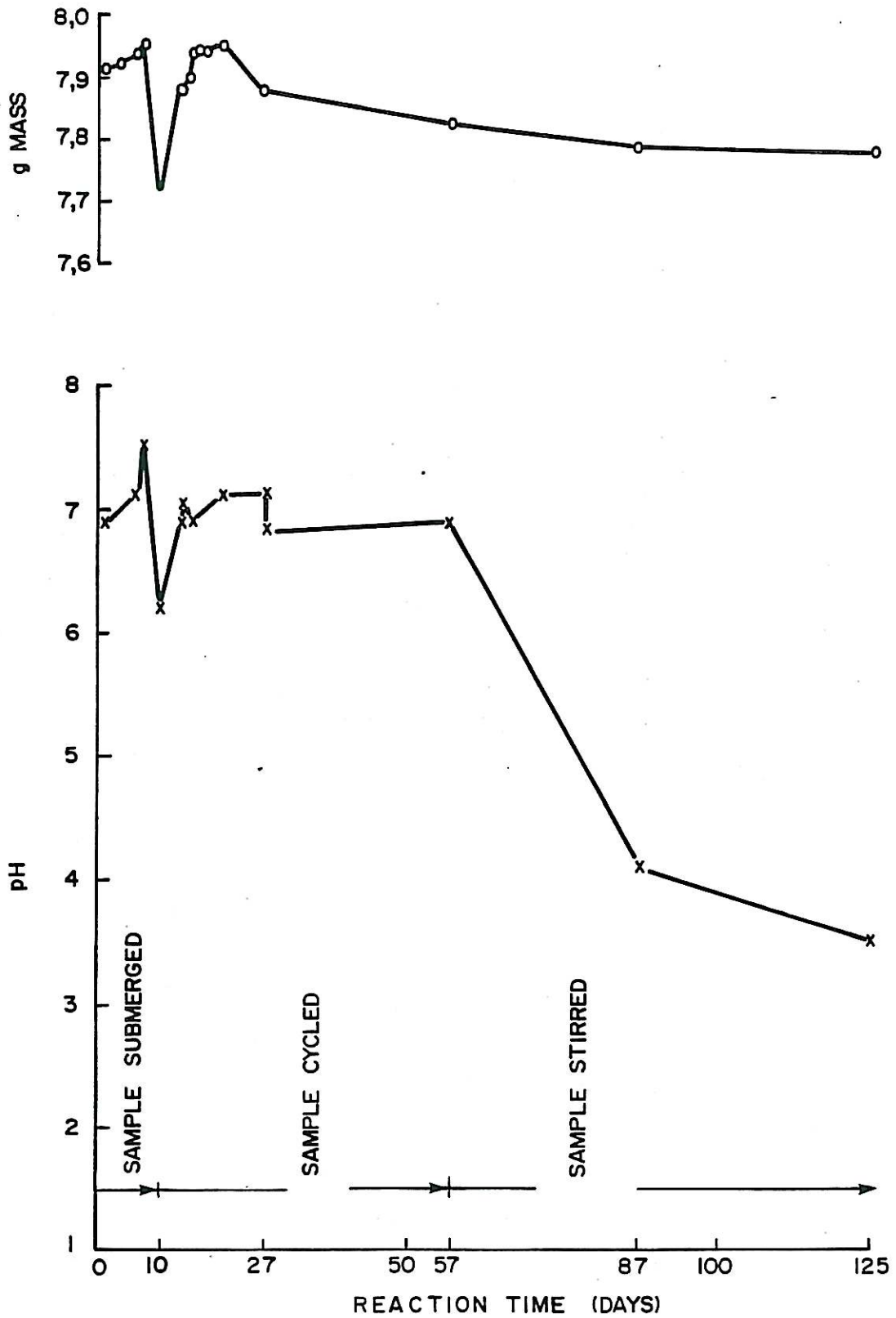


FIGURE 3.29

*Development of pH in a 1:1 mix of colliery spoil and ferricrete (bottom) and its influence on the mass of a single ferricrete nodule (top).*

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ations of these reactions in stabilization work. The problem is that the widely applied engineering parameters are physical parameters and can assist only in the soil mechanical considerations such as grading, ie density, and not in the assessment and analysis of the soil chemical stabilization potential of construction materials.

#### 3.4.1 INTRODUCTION

Under the old mechanical approach, stabilization was discussed in principle under cement stabilization, with mechanical and lime stabilization as mere appendices. It appears now, however, that strength gain by lime carbonation and cement hydration must be separated from lime/soil hydraulic reactions and again from mechanical and/or chemical modification reactions. The overlap of these reactions is in fact such that an understanding becomes possible only by describing these reactions as separate entities and synthesizing the resulting changes of technical properties accordingly. A good example of such an approach is the analysis of reactions taking place in a successfully cement stabilized soil. This may be described as follows:

- (a) Mechanical stabilization effect caused by adding cement which turns clayey soil into a more granular material after hydration.
- (b) Likely strength gain by compaction, ie higher density.
- (c) Mechanical cementation of soil particles by hydrated cement.
- (d) Chemical modification of soil by lime which is released during cement hydration and changes mainly the plastic properties of clay.
- (e) Carbonation of lime released during cement hydration.
- (f) Cement reaction of soil and lime released during cement hydration.

(g) Second-phase reactions of (a) and (c)-(f) resulting from secondary soil/lime reaction.

(h) Et cetera.

Cement-stabilized soil must therefore truly be considered as a reaction system of 'box within box within box', with the emphasis on soil/lime reactions.

The number of natural soils and gravels is very large. For the purpose of discussion, however, it appears sufficient to classify them into two technically significant groups, ie the kaolinite type soils derived from granitic rock and the montmorillonite-type soils derived from basic igneous rock. They differ from each other chemically in the elemental composition of their parent rock and hence in the soil compounds that are the end-products of weathering.

The difference in their significant mechanical (physical) technical properties - in particular their plasticity - has been known for decades and was discussed in detail by Kezdi (1979).

The difference in their significant chemical technical properties - in particular their stabilization potential - is discussed in the following. Also discussed are limited aspects of mechanical stabilization. This is necessary for the understanding and interpretation of test results and of the stabilization mechanism as a whole.

Briefly, the main difference between these two materials is that Silverton soil (kaolinite-type soil) is rapidly lime-saturated (1 hour reaction and 2-3 % lime), whereas black clay (montmorillonite-type soil) requires approximately 10 times as much lime and a long period of reaction (days). A second important difference is that lime-stabilized Silverton soil does not develop a measurable hydraulic strength while black clay does.

The best practical criterion of stabilized material is probably strength, ie its permanence and acceptable plasticity. It must be kept in mind, therefore, that a portion of the strength measured in samples is

invariably due to compaction and is referred to under the term 'compaction strength' in the following discussion.

We also discuss below the result of soil stabilization in terms of stabilization strength and plasticity, but refer to its causes only where necessary, since such causes have been dealt with earlier (see Section 3.2).

### 3.4.2 SOIL MODIFICATION

The term 'soil modification' refers strictly to the manipulation of the soil plasticity. An increase or a decrease in plasticity may be achieved in the following ways:

- (a) by adding less plastic soil (ie sand) to achieve a decrease in plasticity (mechanical modification) or
- (b) by adding more plastic soil (ie clay) to achieve an increase in plasticity (mechanical modification) or
- (c) by adding a chemical stabilizer such as lime or cement to achieve a decrease in plasticity (chemical modification).

#### 3.4.2.1 Mechanical modification

This is a very old method, but is today still as valid as it was in the past. The literature covering the subject is quite adequate and the interested reader is referred to Kezdi (1979) for further information.

Kezdi also discusses any necessary grading correction, although soil modification applies in its strict sense to the manipulation of plasticity, ie the fines fraction only.

### 3.4.2.2 Chemical modification

The influence of lime treatment on the plasticity of different soils is essentially the same except for the LL and the magnitude of reaction (Figures 3.30 and 3.31). This is valid for widely different soils such as Silverton soil (Table 3.9) and black clay (Table 3.11). The influence of cement treatment on the plasticity of Silverton soil is given for comparison in Figure 3.32. The liquid limit (of a clay) in particular is known to be much more sensitive to the type of cation present than the plastic limit (Diamond and Kinter, 1966). However, the anions appear to exercise an equally great influence on the technical properties as shown in Figures 3.13-3.14 and Figures 3.17-3.18.

It is believed that the modification of soil is caused by cation and anion exchange reactions of the soil and the dissociated lime, which some authors classify as physico-chemical changes in the clay fraction as opposed to soil/lime reactions or chemical reactions proper, when cement minerals form (Ballantine and Rossouw, 1972).

The reaction of soil with lime ( $\text{Ca(OH)}_2$ ) is of particular importance in stabilization work and the reactions of a number of South African soils when stabilized with 2-4 % lime are presented in Table 3.18. The criterion chosen was whether the liquid limit of the soil fines would increase (+ Delta LL), decrease (- Delta LL) or remain unchanged (0 Delta LL) when treated with lime. It is referred to as Delta-liquid-limit.

The modification of soil must not be confused with the stabilizer/soil proportional reaction. This becomes clear when considering the left-hand side of eg the plastic limit curve in Figure 3.30 which rises steeply on the addition of lime. A maximum is reached at approximately 6 %. A gradual decrease of the plastic limit with increasing lime content is observed thereafter until the diagram eventually links the plasticity values of pure lime.

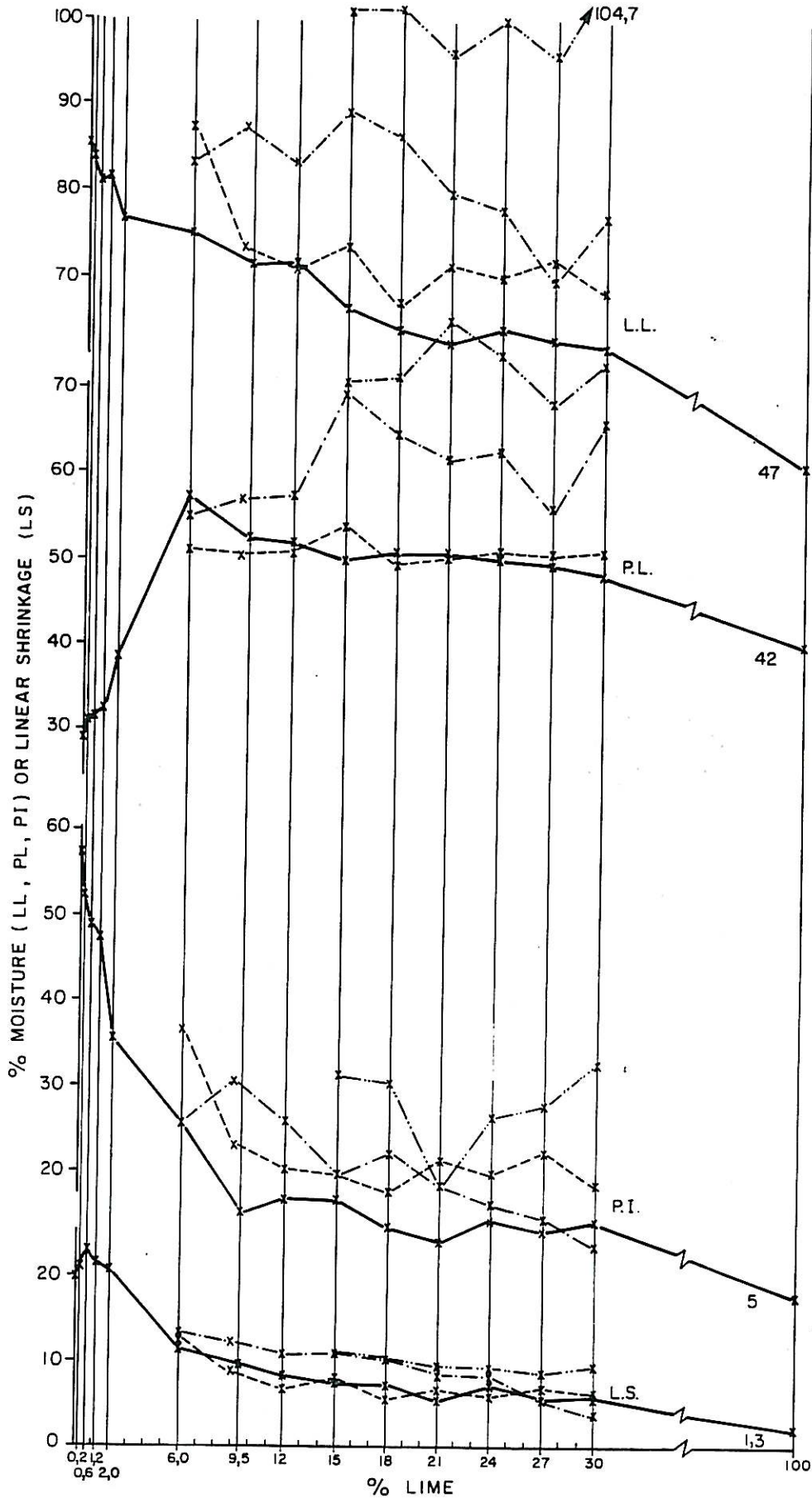


FIGURE 3.30

Montmorillonite-type clay reaction  
 The effect of curing (lime saturation of soil) on the LL, PL, PI and LS of carbide lime-stabilized weathered norite (black clay, sample B 421). (4 days cured = — ; 12 days = - - - ; 1 month = - · - · - ; 2 month = · · · · ·).

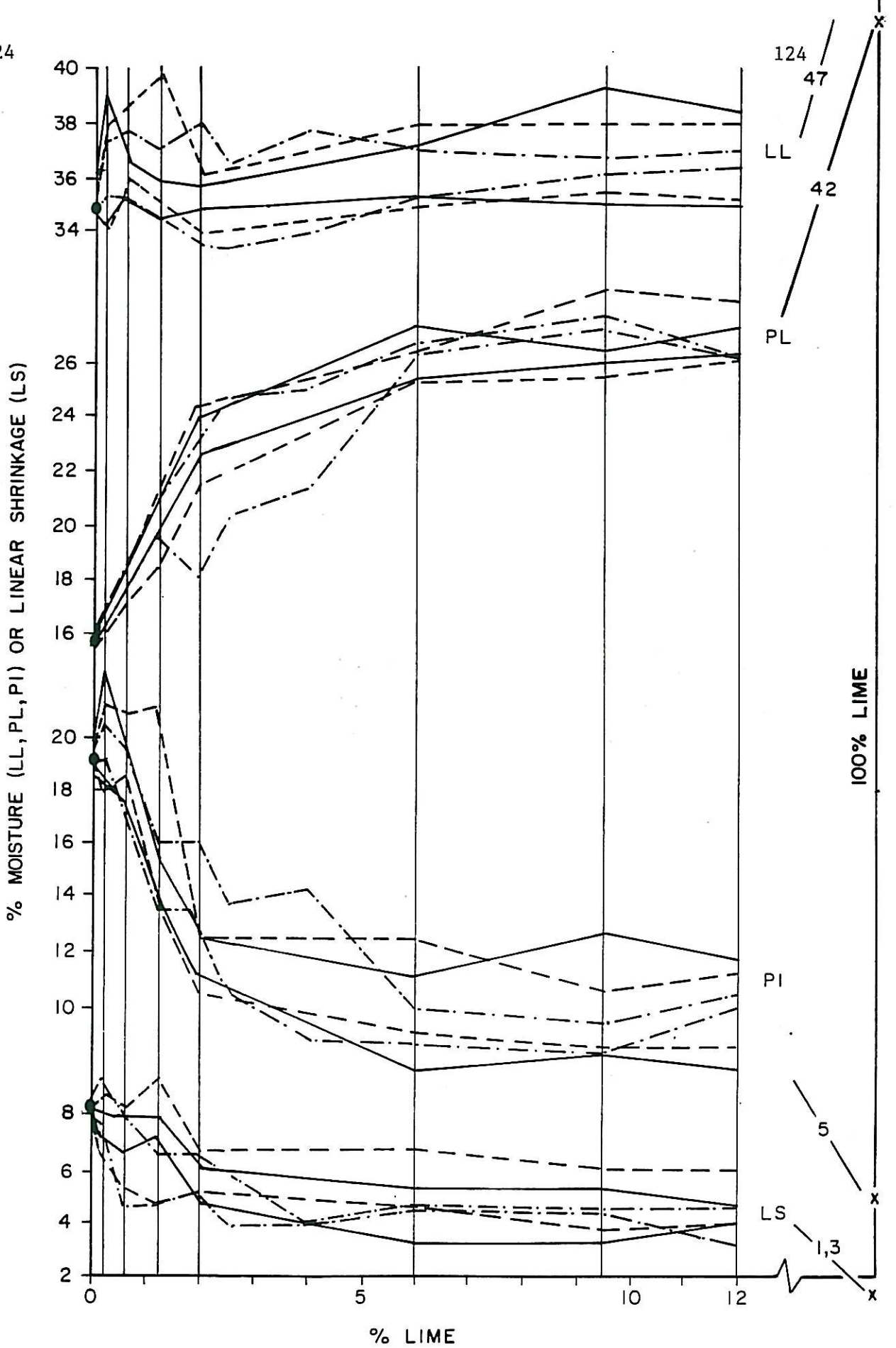


FIGURE 3.31

*Kaolinite-type clay reaction.*

*The effect of lime treatment on weathered Pretoria shale. The envelopes indicate the variation of four test results each (— carbide lime; - - Marvello road lime; - · -  $Ca(OH)_2$  pa).*

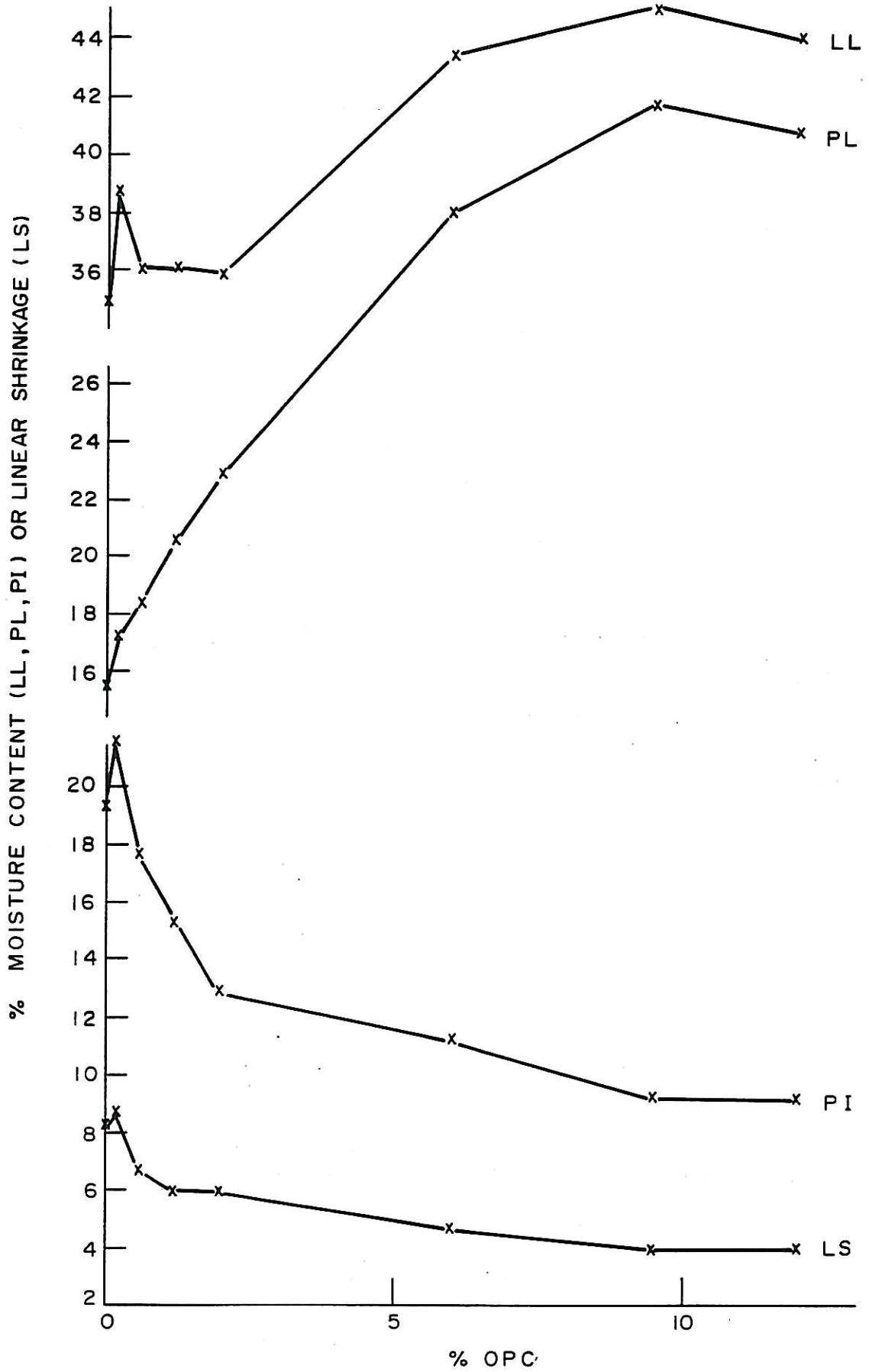


FIGURE 3.32  
 Kaolinite-type clay reaction.  
 Atterberg limits and linear shrinkage of OP cement-stabilized  
 Silverton soil after 4 days' curing (TMH1).



TABLE 3.18  
Effect of lime on the liquid limit of South African soils.

Soil type	Number of borrow pits	Reaction with lime Delta LL	Observations		Percentage
			Number	Total	
Beaufort shale	5	-	4	16	25
		0	2		13
		+	10		62
Ecca shale	21	-	50	69	73
		0	3		4
		+	16		23
Ferricrete	19	-	9	32	28
		0	5		16
		+	18		56
Granite ferricrete	31	-	15	55	27
		0	5		9
		+	35		64
Dolerite	24	-	71	84	85
		0	1		1
		+	12		14
Calcrete	4	-	2	6	-
		0	-		-
		+	4		-
Pretoria shale	4	-	-	4	-
		0	1		-
		+	3		-
Bokkeveld shale	1	-	1	1	-
		0	-		-
		+	-		-
TOTAL	113	-	156	276	57
		0	17		6
		+	103		37

Notes: - Delta LL = decrease in LL )  
 )  
 + Delta LL = increase in LL ) no curing, dry to wet (TMH,  
 ) 1979);  
 )  
 0 Delta LL = no change in LL )

Modification in its strict sense therefore refers only to the drastic reaction up to the maximum near the 6 % mark (for black clay). The change of plasticity thereafter must be considered as modification in its wider sense, ie the mechanical (sand) stabilization category. The same effect has been observed for Silverton soil (Figure 3.31).

It is interesting to note that the modification lime content of Silverton soil and black clay was found to be approximately twice the lime equivalent of the organic-matter impurities content as shown in Figure 3.33. The lime demand in terms of  $\text{Ca}^{++}$  ions of these two soils was determined and it may be seen that Silverton soil would require 0,9 % lime to saturate its calcium deficit. Black clay on the other hand would need some 2,8 % of lime to saturate its calcium deficit, as shown in Figures 3.30 and 3.31. However, this relation can be verified only by statistical analyses, which have not as yet been made.

A well-known phenomenon is that lime-stabilized soils produce a lower density compared with untreated soils for the same compaction effort. This may possibly be explained in part by the induced polarization of soil particles and is caused by the 'crowding' of  $\text{Ca}^{++}$  cations on their surfaces. It appears therefore that aspects of both electrostatic conditions such as flocculation and deflocculation and chemical reactions such as ion exchange must be considered in soil modification. In the one case we observe primary soil reactions and in the other we observe secondary soil reactions.

Another explanation may be that cementation reactions camouflage these results. No systematic experiments to identify and quantify these reactions in terms of plasticity have as yet been made.

#### 3.4.2.3 Soil modification in time

The modification of soils is not necessarily permanent as has already been established for London clay by Clare and Cruchly (1957) and Dumbleton (1962). The plasticity in time of 18,5 % lime stabilized black

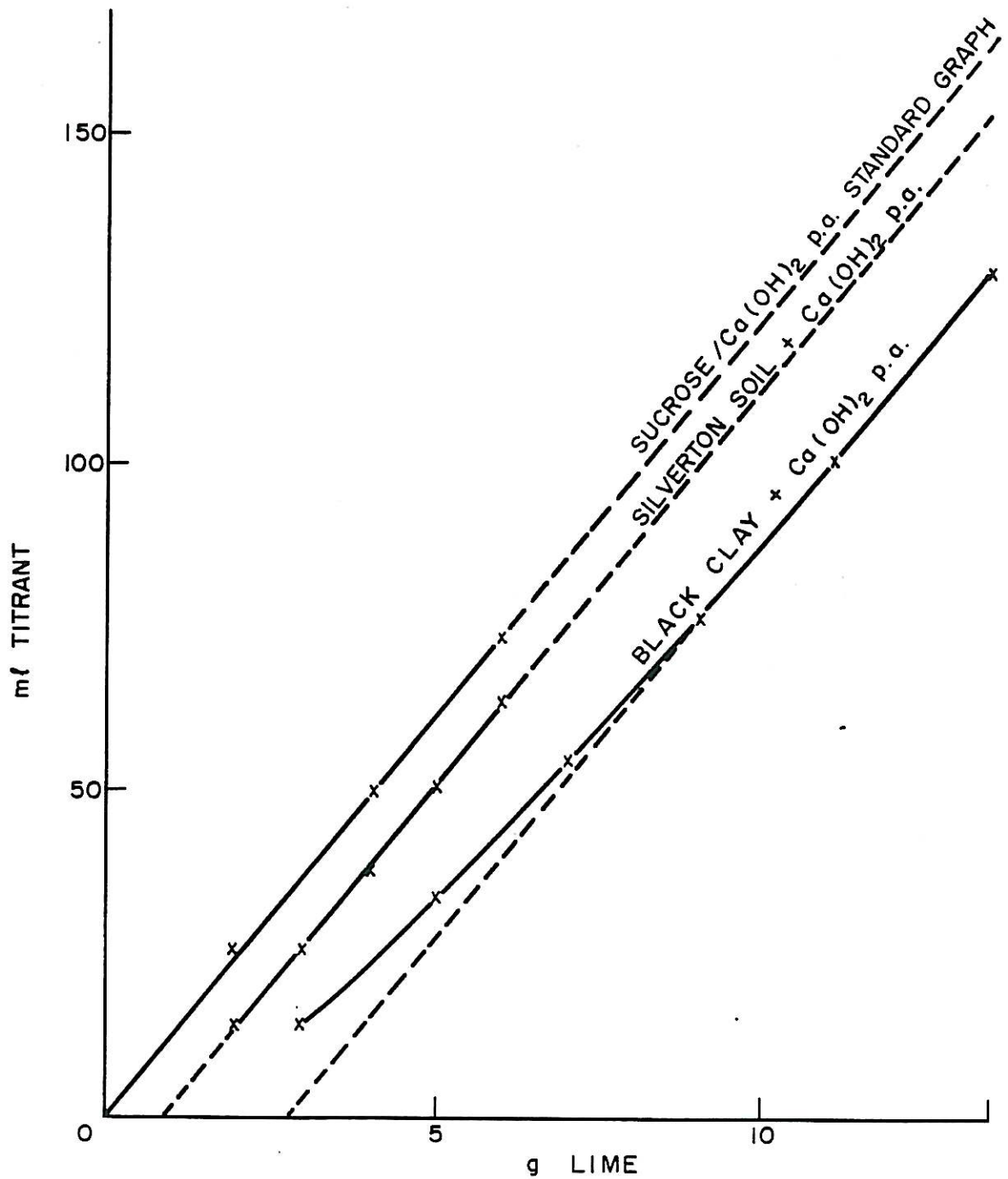


FIGURE 3.33

Determination of  $\text{Ca}(\text{OH})_2$  reacting with Silverton soil (0,8%) and black clay (2,8%) according to the sucrose method. Second determination on Silverton soil produced a reading of 1,2%. The ICL was 2,5% and 5,5% respectively (21,5% after 24 h).

clay and its variation with time are shown in Figure 3.30. It may be seen from Figure 3.34 that the LL and PL move through a maximum after 5 month curing. The quantity of lime seems to be important for this secondary swell since the LS of a 10 % lime stabilized black clay increased by 2 % only after 5 months' curing. The reason for these changes is believed to be related to dominating hydration/carbonation (cement) reactions of the lime treated black clay which resulted in the plasticizing/granulation of the soil. The effects of this phenomenon in particular on the completed road pavement or the possible cause for subsequent failure have not yet been considered.

#### 3.4.2.4 Compaction delay and seasoning

Compaction delay or seasoning of material during construction may be considered as a special type of soil modification. The purpose of this construction technique is twofold, ie

- (a) to achieve a higher uniformity of the grading with resultant better and more intimate mixing and
- (b) to increase the workability of heavy clay soils by allowing time, after adding the lime stabilizer to the soil, for the modification to take place.

It is not modification in the accepted sense of bringing down the plasticity to the specified level, although both objectives - reduction of plasticity to the specified level and suitable workability - may be achieved. The result may be considered equivalent to an additional, mechanical mixing effort or grinding as already demonstrated in Figure 3.23. Consequently the combination of all three principles of soil stabilization, ie plasticity correction, workability and strength gain, must be considered in any discussion. The influence of compaction delay on strength is of particular interest and is discussed in greater detail below.

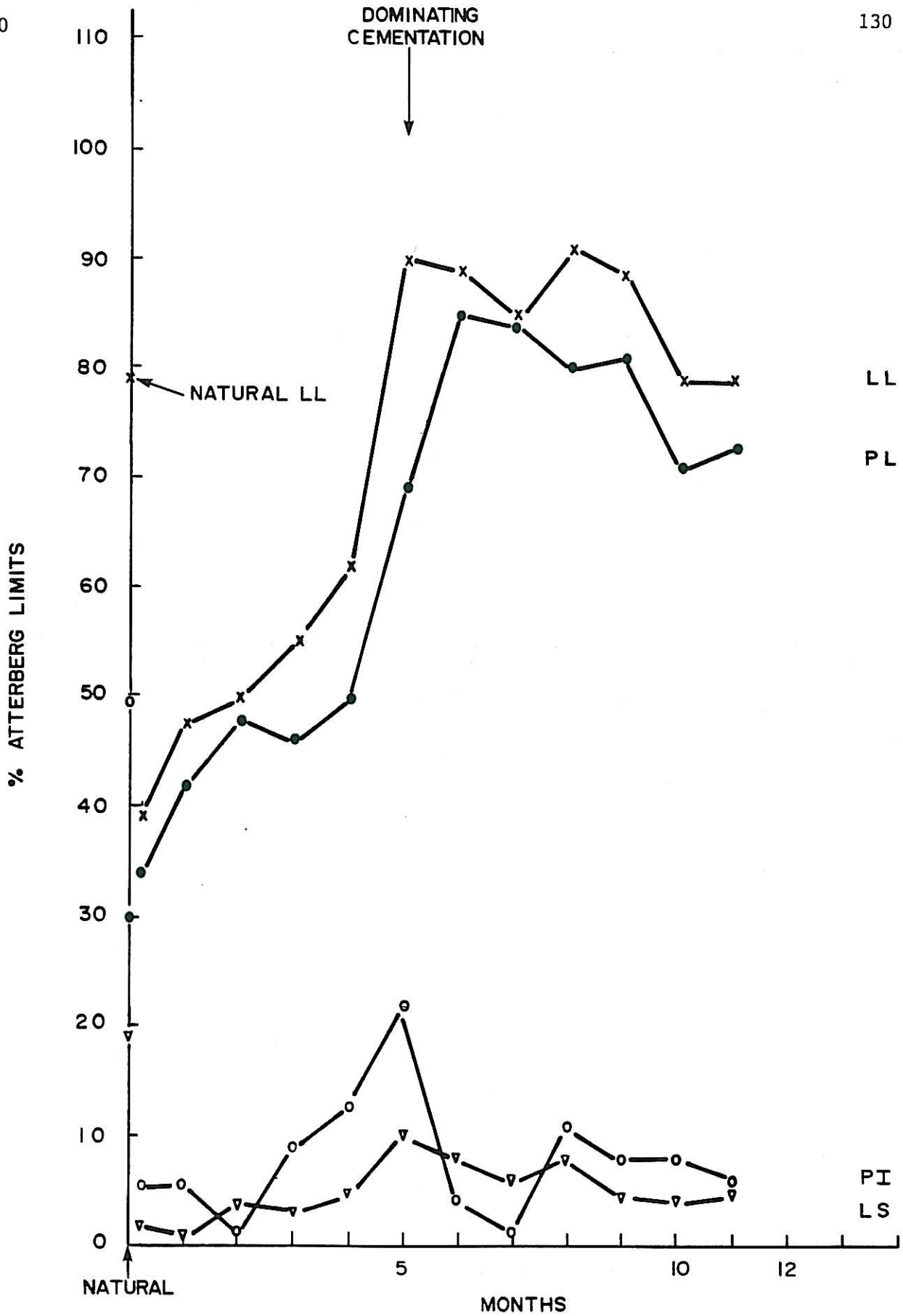


FIGURE 3.34

*Atterberg limits of 18,5 % carbide lime - stabilized black clay over a period of 11 months (JR 1669). The Atterberg limits of the natural material have been recorded at 0 months .*

The UCS in 22 % lime-stabilized black clay and 3 % lime stabilized Silverton soil was determined after different compaction delays. The results are presented in Figure 3.35 and it may be seen that compaction delays of 18 hours may be tolerated in Silverton soil without excessive loss of strength (longer periods have not been investigated). Compared with the strength gain of 3 % lime-stabilized Silverton soil, a decidedly beneficial effect may even be expected from compaction delays since an increase of the UCS from 250 kPa to some 570 kPa could be observed. Similar results were achieved with 22 % lime stabilized black clay which increased from a UCS of some 900 kPa in the immediately processed sample to almost 1 400 kPa after the 18 hour compaction delay. However, in the case of black clay there appears to be a rather marked 'kick' at a compaction delay of some 3 hours.

When considering these strengths it should be noted that the different compaction delays were also used to mix the sample well on each occasion and bring a fresh supply of air ( $\text{CO}_2$ ) in contact with the lime for further carbonation. The immediately compacted samples used for the results in Figures 3.35 and 3.36 had no fresh air ( $\text{CO}_2$ ) supply.

It has also been stated previously that cement reaction and carbonation effects may be expected in lime-stabilized black clay. Any disturbance in the cementation process should therefore interrupt and even decrease the strength gain in time. It is believed that the 'kick' as shown in Figure 3.35 (black clay) represents such cementation interruption. The critical point may be the indicated 3 hour compaction delay, at least for the 7-day strength. A second critical phase may be expected when all or most of the calcium hydroxide has reacted to form cement minerals or has carbonated and no further strength improvement may be expected as a result of chemical inertia. This is shown in Figure 3.36 where the UCS of 18 % lime-stabilized Silverton soil drops after a 5 month compaction delay. The UCS of the 30 % lime stabilized black clay drops after a 7 month compaction delay. The difference in time is probably caused by the difference in lime content which provides for longer reaction for more lime. Compaction delays were also found beneficial in (6 out of 8) Oklahoma lime stabilized shales (Laguros and Iha, 1977), where con-

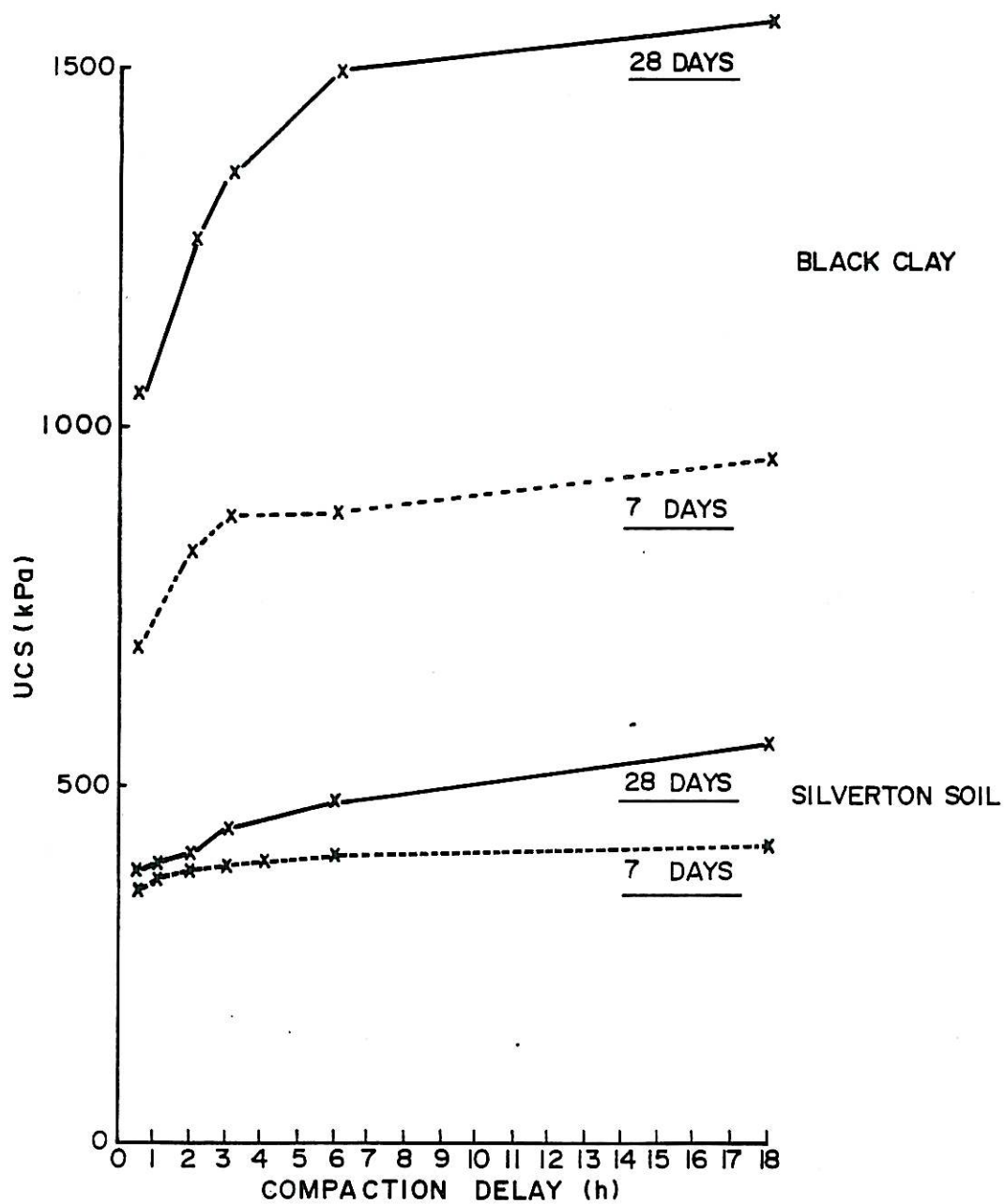


FIGURE 3.35

*UCS after delayed compaction of 22% lime-stabilized black clay and 3% lime-stabilized Silvertown soil (OMC, MDD at Mod AASHTO).*

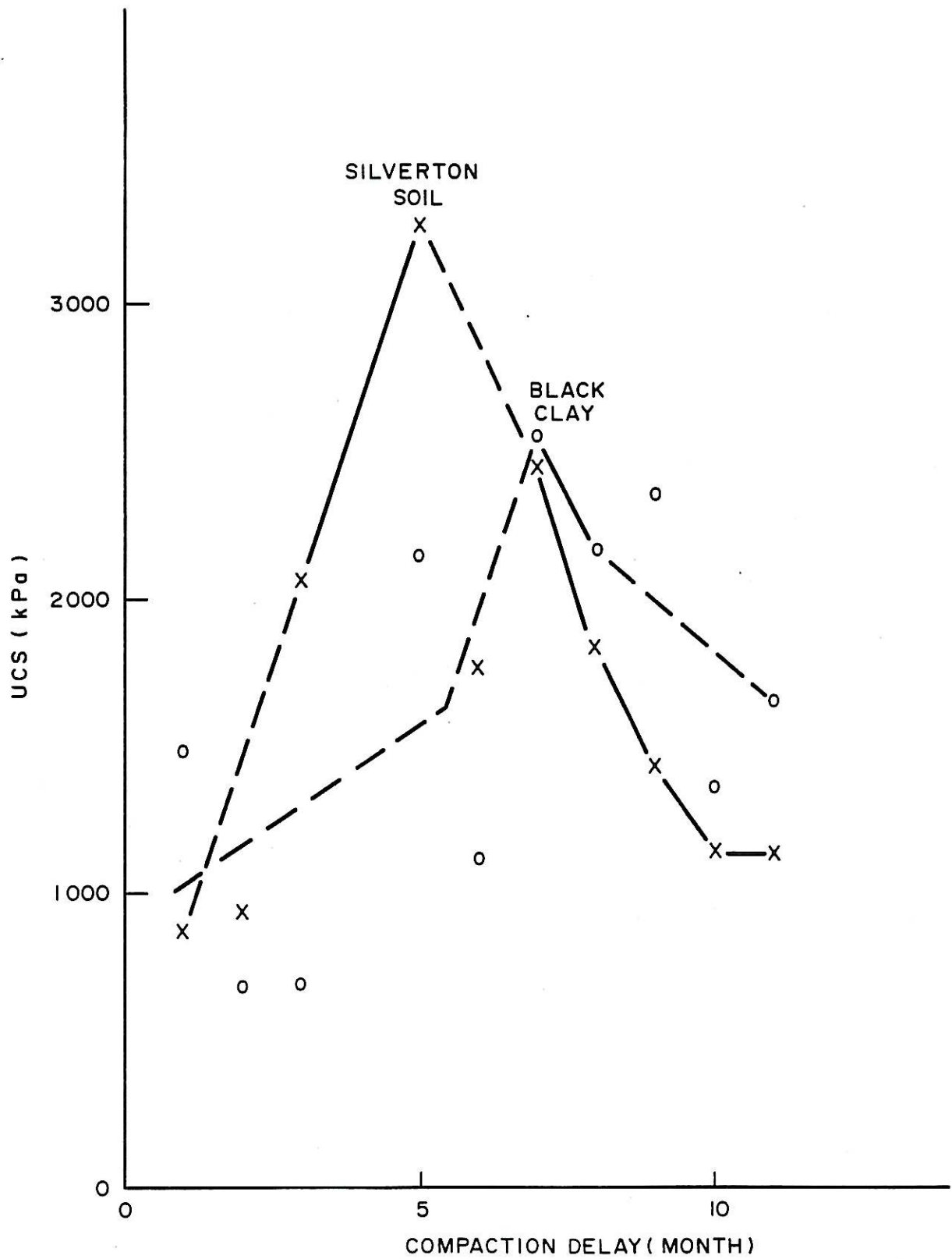


FIGURE 3.36  
*UCS after delayed compaction of 18% lime-stabilized Silvertown soil and 30% lime-stabilized black clay (28 days, OMC, MDD).*



siderable increases in the UCS of 28 and 90 day cured samples were observed.

The term 'beneficial' as used above has been used in relation to strength (UCS) increases. It should be noted, however, that this might be achieved at the cost of a loss of total strength: the quantity of the lime added must be considered as a source of limited strength gain; the frequent mixing of lime-stabilized soil (the provision of more  $\text{CO}_2$ ) is only a means of accelerating the carbonation reaction; and the destruction of particle bonds by repeated mixing results in an overall decrease of the final strength.

#### 3.4.2.5 Conclusions

It is concluded from these considerations that chemical soil modification describes the chemical changes in the plasticity of soil caused by lime. However, it is also concluded that chemical soil modification in the case of lime stabilization extends to changes in the plasticity of lime caused by soil. The intersection of these two curves represents the saturation of soil with lime and is an optimum condition for plasticity consideration. The relation allows us to define chemical soil modification by lime *sensu stricto* as the intersection of two curves which represent the decrease of the soil plasticity by lime and the increase of the lime plasticity by soil. It follows from this definition that this intersection is not a constant, but depends on the plasticity (composition) of the original soil and of the original lime.

The particle size of soils has a marked effect on strength development and a smaller particle size gives rise to a greater strength for well-mixed soils. The repeated mixing of lime-stabilized soils may accelerate (carbonation) strength at the cost of total (carbonation and cement reaction) strength gain.

### 3.4.3 SOIL CEMENTATION

Soil cementation strictly means the cementation of soil particles by means of a cement which has been added to the soil (such as Portland cement or lime or precipitates from the ground water) provided no reaction takes place with soil compounds.

It has also come to mean the cementation of soil particles by means of a cement which develops in situ by the reaction of lime with soil compounds either as result of chemically released lime from the hardening process of eg Portland cement or added plainly as lime in lime-stabilized soil, or a combination of both. It does not include, in this context, cementation due to cohesion, eg by clay minerals.

It is therefore important during this discussion to remember that in stabilization work we deal with -

- (a) primary reactions eg strength gained from the formation of carbonate from lime or from the formation of cement minerals, sensu stricto, from the hydration of, eg Portland cement added;
- (b) secondary reactions of stabilizer and soil, eg lime/soil where lime was added as a stabilizer or where it was added by chemical release from the hydration of cement; and
- (c) third-generation reactions involving the chemical release of yet other compounds which may react with natural soil compounds still available. One such example would be the lime released during the hydration of cement minerals formed by the reaction of soil compounds and free lime released again.

Soil cementation in the wider sense, also includes the chemical attack of lime on clay minerals which, in addition, results in calcium aluminates and also precipitation-bonding of eg iron oxides. These bonds of soil cementation manifest themselves as strength.

#### 3.4.3.1 (Portland) Cement-cementing

This is simply the result of the hydration of commercial (Portland) cement and has been well covered by Lea (1970). No further discussion is therefore necessary.

The best definition is probably as follows:

Cement-cementing strength in cement-stabilized soil is caused by the cementation of soil particles by primary cement minerals.

#### 3.4.3.2 Carbonation cementing

Carbonation is the formation of calcium carbonate ( $\text{CaCO}_3$ ) from calcium hydroxide ( $\text{Ca(OH)}_2$ ) when the latter is moistened and left in air to dry.

Carbide lime, Marvello road lime and chemically pure  $\text{Ca(OH)}_2$  were wetted with 10 % of water. By the time they dried again, lumps had formed. Very slight pressure powdered the lumps. The grading, formation of carbonate and moisture content are therefore a problem in limes which are stored in the open air.

Calcium carbonate is inert for all practical purposes and does not participate in the stabilization reaction of the soil (except, of course, in the case of carbonation in situ which might produce remarkable strength).

The most sensitive (qualitative) test known so far is the simple wetting of the lime with hydrochloric acid. Bubbling is clearly visible, even after exposing moist lime ( $\text{Ca(OH)}_2$ ) to the air for only 4 hours.

As explained earlier, the carbonation of  $\text{Ca(OH)}_2$  does not invariably result in cementing strength, but may also lead to the formation of  $\text{Ca CO}_3$  in the form of a loose powder.

(a) Definition

Carbonation strength of soil is the strength development of lime- and cement-stabilized soil caused by the cementation of soil particles due to the formation of calcium carbonate bridges between soil particles.

(b) Discussion

The literature is not unanimous on the formation of strength from the carbonation of lime and it is therefore necessary to consider these aspects.

The experiment described in Figures 3.1 and 3.2 was repeated with a greatly increased moisture content of 98 % (of dry mass) for black clay (OMC = 18 %) in order to establish whether the UCS was due to pozzolanic-hydraulic reaction (exclusion to air) or lime carbonation (access of air). The results are recorded in Figure 3.37, where it may be seen that cement reactions (hydraulic reaction in the absence of air) took place in lime-stabilized black clay. No strength development in lime-stabilized Silverton soil could be observed in the absence of air. The gain in strength of air-cured samples which cannot be achieved in the absence of air must therefore be ascribed to the carbonation of the lime stabilizer. This is confirmed by the work of Lu et al (1957), who found that high-calcium lime-stabilized montmorillonite produces high strength, where high-calcium lime-stabilized kaolinitic clays produce low strength (Ingles and Metcalf, 1972).

In the above experiment the strength achieved with surplus moisture was very low because of the low densities involved. This was found in particular for the UCS of lime-stabilized Silverton soil at OMC but at a density equivalent to some 65 % moisture content (Figure 3.37). The UCS of this (single) sample was some 6,1 kPa (28 days). Further evidence in support of the development of carbonation

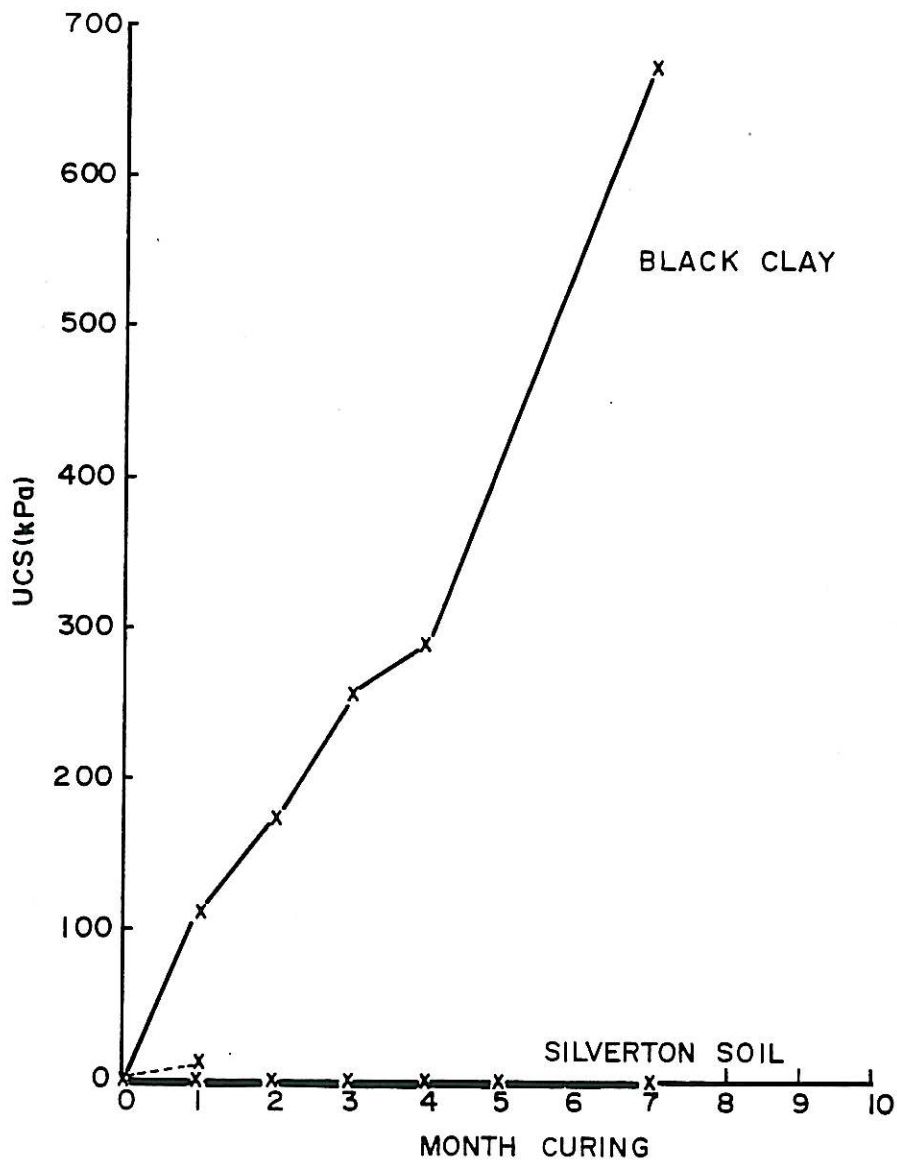


FIGURE 3.37

*UCS of lime-stabilized black clay and Silvertown soil in the absence of air (water-saturated). The broken line represents lime-stabilized Silvertown soil at OMC but at the same density as the water-saturated Silvertown soil (18,5% carb. lime).*

strength in lime-stabilized soil was produced in strength tests carried out on compaction-delayed materials which produced an increasing UCS (Figure 3.35). This can probably be explained only by the renewed supply of  $\text{CO}_2$  (from the air) during repeated mixing. An increased UCS was also obtained in 6 % lime-stabilized Oklahoma shales in 0-4 hour compaction delays and after a curing period of 90 days (Laguros and Iha, 1977). The development of carbonation strength has also been observed in CBRs of lime-stabilized, washed sand (Table 3.19). It is of particular interest to consider the 28-day strength increase from 39 % in 3 % lime-stabilized soil to a CBR of 122 % in 18 % lime-stabilized soil. The strength increase is three-fold, although it should be six-fold according to the quantity of lime added, and the reason for this might well be either the limited supply of  $\text{CO}_2$  (air) in the pores of the sample or partial powder formation during the process of carbonation. The pH at 16,5 °C was 12,2 and 12,6 respectively (28 days). The CBR of a 3 % fines-stabilized sand and strong alkaline treatment was also determined and the results are added for comparison.

The interpretation of Figure 3.36 in terms of strength appears to indicate maxima for both stabilized Silverton soil and black clay after delayed compaction. There is little doubt that the decreasing strength of the 5 and 7 month delayed compaction is caused by disrupted cementation bonds. The increasing strength of the first 5 and 7 months respectively is probably caused by the increased access of air during mixing for compaction and the subsequent rapid formation of carbonation bonds. An X-ray diffraction analysis, moreover, showed no  $\text{Ca(OH)}_2$  trace, while the  $\text{CaCO}_3$  trace increased from zero to some 58 mm in intensity at  $\theta = 14,65$ . Meanwhile, the pH dropped from saturation to 11,5 (Silverton soil) and 10,9 (black clay) after 12 months.

Brand (1963) in particular showed the formation of the  $\text{CaCO}_3$  skeleton in lime-stabilized soil by direct microscopic observation of thin sections (carbonate phase I). The carbonate skeleton had

TABLE 3.19  
Development of carbonation strength in lime-stabilized, washed sand.

Ref. No.	Material	Treatment	CBR (%)		
			7 day	28 day	90 day
9888	Washed sand	OMC, MDD, mod AASHTO. 2,54 mm, 3 % lime	28	39	82
9888	Washed sand	OMC, MDD, mod AASHTO. 2,54 mm, 3 % fines	38	39	-
9888	Washed sand	OMC, MDD, mod AASHTO. 2,54 mm, 18 % lime	-	122	153
9888	Washed sand	OMC, MDD, mod AASHTO. 2,54 mm, 15 g NaOH/ 100 cm <sup>3</sup> H <sub>2</sub> O	44	50	-

developed only in samples which were exposed to air after 1 year of curing under a paraffin skin cover. He also warned of a carbonate phase II which developed after a relatively short curing period under paraffin followed by intensive and lengthy curing in air. He described carbonate phase II as a powdery accumulation of carbonate crystals without any strength potential (Taylor and Arman, 1960).

The pH of a saturated  $\text{CaCO}_3$  solution is 8,3-8,4 and the formation of calcium carbonate at low pH levels has been investigated recently in more detail in stabilization failures in roads by Netterberg (1982) and Netterberg and Paige-Green (1984).

#### (c) Conclusion

It is concluded from these experiments that carbonation may produce strength and that the difference in strength between lime-stabilized Silverton soil and black clay is caused by a hydraulic, pozzolanic cement reaction of black clay. It is tentatively concluded that the OMC should be approached from the wet condition even if the MDD is specified to be on the dry side of the OMC.

#### 3.4.3.3 Hydraulic soil/lime reactions

As explained earlier, these were, considered for a long time to be the only strength-producing factors in stabilized soil. In the following discussion the reactions include only the formation of calcium silicates and calcium aluminates by a reaction between lime ( $\text{Ca(OH)}_2$ ) and soil compounds. The criterion applicable to practical stabilization work should be wider, however, and should, according to current knowledge, include anything that increases or produces strength in stabilized soil. Hydraulic soil/lime reactions must therefore be remembered as only one of a number of strength-producing factors and may be defined as the in situ formation of cement minerals, ie calcium-silicate hydrates and calcium-aluminate hydrates. The primary hydraulic reaction products of eg



Portland cement are not included in this definition since it is a self-contained reaction of the cement and does not require soil to proceed. The reaction between chemically released lime ( $\text{Ca(OH)}_2$ ) from the cement hydration or lime additives and soil is, however, included.

(a) Calcium silicates

It has been shown earlier that the formation of cement minerals such as calcium silicate hydrates (CSH) is strictly dependant on a high pH level in stabilized soil and is for all practical purposes, also dependant on the presence of a lime-rich natural glass phase in soil. If such a reaction is to take place it is necessary that the pH level of the mix remain above 10,5. If the pH level drops below 10,5, CSH minerals proceed to decompose to silica and calcium carbonate without appreciable strength formation.

It is suggested, however, that a working pH level of 12,2-12,4 be used for stabilization work because this has been recommended in BS812: 1943, 1967, is far enough removed from the critical pH of 10,5-11,0 and can be determined easily since it is also the lime saturation pH in water at a temperature of 25 °C (SABS78/56109: 1978). BS1924: 1975 considers a pH of 12,1 as still safe. The cement reaction pH values which have become known in the course of time are presented in Figure 3.4 for reference.

It is interesting to note that Eades and Grim (1966) reported maximum strength in lime-stabilized soil at a pH of about 11,0 when the strong formation of carbonates from solution commences (Lea, 1970).

The high UCS in lime-stabilized black clay is caused by a hydraulic reaction, because it takes place under water and in the absence of air. The Transportation Research Board lists no less than five calcium silicate hydrates in lime-treated soils which were described in the course of time by as many authors (Diamond et al, 1964; Moh,

1965; Sloane, 1965; Ruff and Ho, 1966; Wang and Handy, 1966). Calcium silicates are type-minerals in Portland cement and lime stabilization may be compared, at least partly, with possibly rudimentary cement reactions.

The cement reaction does not exclude the carbonation of the lime stabilizer in air and it is assumed that both reactions take place unhindered alongside each other - until the  $\text{Ca(OH)}_2$  is consumed. A comparison of the UCS in 5 % lime-stabilized black clay (cement reaction- plus carbonation strength) and 5 % lime-stabilized Silverton soil indicates that the strength gain in black clay from both cement reaction and carbonation might approximate 50 % each after 28 days (OMC, MDD), according to Figures 3.1 and 3.2. However, an uncured compaction strength of approximately 50 % of the Silverton soil UCS must be deducted, according to Figure 3.38.

The reason for hydraulic strength in natural black clay is, according to the XRD analysis, a lime-rich amorphous glass phase as demonstrated in Figure 3.16 and discussed in the section on the pozzolanicity of soils.

The type of clay mineral in a natural soil may therefore be used as an indicator of the stabilization potential of soils, although pure clay minerals seem to have lost the above amorphous glass phase during the purification process.

It is concluded from these considerations that the criteria for soil/lime reactions of the cement type differ substantially from those of soil modification and carbonation by the necessity of a high pH level in order for the cement reaction to proceed. The stabilization formulae ((5)-(7)) indicate that suitable cations and anions and the correct pH level are necessary for a pozzolanic lime/soil reaction to take place. The cement reaction lime content (CLC) may therefore be defined as that quantity of lime which produces a pH in a lime/gravel/water mix (after 1 hour at 25 °C) that allows a possible pozzolanic cement reaction to take place.

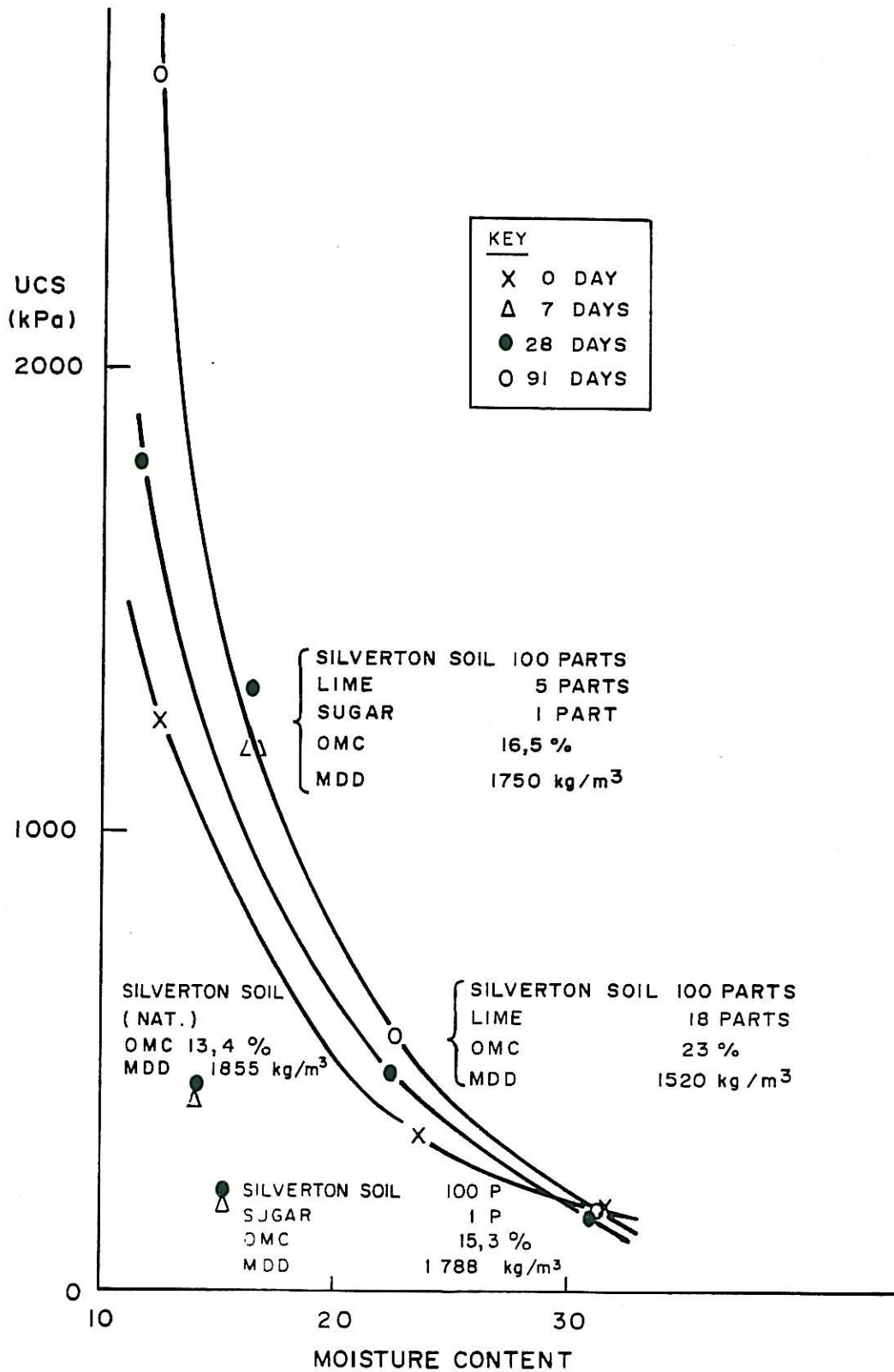


FIGURE 3.38

*Strength gain of lime-stabilized Silverton soil at MDD with organic impurities (sugar) added.*

(b) Calcium aluminates

De Wet and Skinner (1967) investigated kaolinite-lime and montmorillonite-lime reactions in detail. They found in 30 % lime-stabilized kaolinite and -montmorillonite a predominant development of calcium aluminate hydrate (CAH) in kaolinite mixes and of calcium silicate hydrate (CSH) in montmorillonite mixes. The strength development in montmorillonite mixes seems slightly higher than that in kaolinite mixes. This is in agreement with the higher amorphous matter content found in montmorillonite.

(c) Third-generation cement-type reactions

This group of stabilization reactions comprises those which result from secondary stabilization reactions, eg the chemical release of lime from the formation of rudimentary CSH when lime and soil compounds react for the first time. Too little is known about these aspects for detailed analysis.

#### 3.4.3.4 Conclusions

Soil cementation may be caused on the one hand by a number of chemical reactions of the stabilizer itself and stabilizer/soil compounds; it may also be listed in terms of strength for stabilization work, as follows:

(a) Carbonation strength

Primary carbonation of lime, chemical precipitation and lime residue carbonation.

(b) Hydraulic strength

Primary cement hydraulic reaction strength.

Secondary cement hydraulic reaction strength in lime/soil mixtures such as calcium silicates and calcium aluminates.

Third-generation cement hydraulic reaction strength.

- (c) A pH level of above 10,5 is the criterion for the permanence of CSH minerals.

### 3.5 Proportional analysis of strength factors

An attempt has been made in Figure 3.39 to analyse proportionally the various factors which probably make up stabilization strength for the two soil types discussed here. Reference is again made to lime reactive and lime non-reactive soil and it may be seen that the overall strength potential in lime-reactive soil is considerably higher than that in lime non-reactive soil.

#### 3.5.1 COMPACTION STRENGTH

This is generally the result of higher densities. It may, however, also be derived in part from the granulation effect of the modified clay.

#### 3.5.2 CARBONATION STRENGTH

Carbonation strength results from the mechanical cementation of soil particles by the formation of calcium carbonate. The same effect is achieved by the mechanical cementation of soil particles by the formation of cement minerals in cement-stabilized soil, as is also known in concrete.

Calcium carbonate may also form through the hydration of cement in which case additional carbonation strength may be expected - if the chemically released lime does not react with soil compounds.

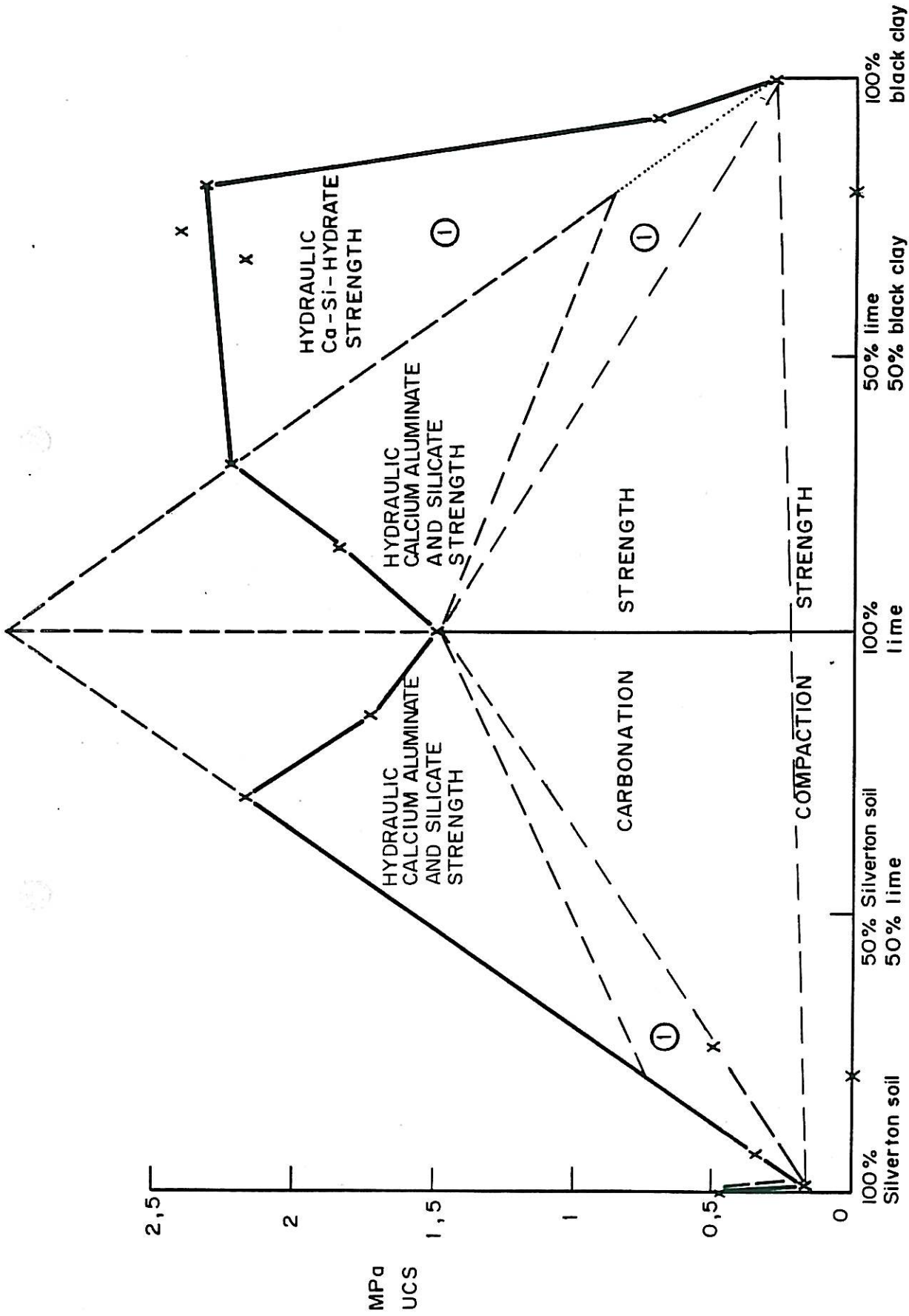


FIGURE 3.39

Carbonation and hydraulic aluminate and silicate and Ca-Si-hydrate unconfined compressive strength (OMC, MDD, 50 x 100 mm cylinder, 28 day cured) in different soils.

### 3.5.3 CEMENT REACTION STRENGTH (HYDRAULIC CALCIUM SILICATE HYDRATE STRENGTH)

This is a reaction between lime and the lime-rich glass phase in soil and may be expected only in soils derived from basic igneous rock with montmorillonite clay in the natural soil as indicator.

### 3.5.4 CLAY REACTION STRENGTH (HYDRAULIC CALCIUM ALUMINATE AND SILICATE HYDRATE STRENGTH)

Although substantial strength gains may be expected from the attack of lime on clay minerals and the subsequent reaction of lime with the dissociated aluminium and silica molecules the reaction seems to be restricted to high lime contents in the region of greater than 20 %.

## 3.6 Guide to testing in stabilization work

### 3.6.1 INTRODUCTION

The mechanism of lime stabilization - as discussed separately in previous sections - is further detailed and summarized in Table 3.20. It follows from these considerations that a test method to establish the stabilization potential directly is essential.

A reluctance to enter the field of chemical testing for establishing the stabilization potential in construction materials has prevailed during the past 15 years. This is perhaps so because, although testing is simple, the interpretation of test results is complex and requires as detailed a knowledge of the subject as that dealt with above. This can hardly be expected of engineers. Therefore an attempt is made in this chapter to make the application of test results more of a routine procedure. It must also be pointed out that the test variables of pH

TABLE 3.20  
Abstract of reaction mechanisms in stabilized soil.

	Stabilization mechanism	Reaction of stabilized soil			
		Interpretation	Criterion	Failure	
MODIFICATION	Mechanical	Uncured compaction strength	Packing density		
		Mechanical stabilization	Addition of coarse particles		
	Chemical	Reduced plasticity and swell, followed, possibly, by increased plasticity and swell after two months	Anion/cation exchange	Cracking caused by phase reactions of different soil compounds <sup>1</sup> ; superficial crocodile cracking	
			Stabilizer/soil relation reaction		
		Friction increase (Chem solubility)	High pH (NaOH)	Compaction failure	
	CEMENTATION	Carbonation	Calcite carbonation	Stabilizer/strength relation	Disintegrating curing carbonation <sup>2</sup>
Dolomite carbonation			Stabilizer/strength relation		
Cement reaction		Hydraulic (CSH) cementation	Ca(OH) <sub>2</sub> saturation pH (gravel ICL)	Ca(OH) <sub>2</sub> satisfaction of organic impurities and soil reactions	Hardening (reaction) failure: pH below CSH formation pH, isolation of Ca(OH) <sub>2</sub> in sugar like organic impurity solutions and soil reactions
			CSH modification		
		CSH carbonation	Carbonation of Ca(OH) <sub>2</sub> released during CSH formation		
		Hydraulic (CAH) <sup>3</sup> cementation	No test designed, strength development probably at > 20 % lime only		

Notes

<sup>1</sup>Davidson, pers comm, 1983.

<sup>2</sup>Netterberg and Paige-Green, 1984.

<sup>3</sup>De Wet and Skinner, 1967.



determinations in lime-stabilized soil are discussed in more detail because of their new-found importance in stabilization work.

### 3.6.2 QUALITY OF LIME

The quality of lime in terms of available lime may be measured by making a sucrose determination (SABS 824: 1967, as amended). This is important in order to ensure a uniform product to work with, but in stabilization work it does not have much meaning in comparison with the soil/lime reactivity, which is the critical issue. In other words, the quantity of available lime required for soil stabilization depends on the objective to be achieved and must be determined accordingly.

### 3.6.3 OBJECTIVE: PLASTICITY CORRECTION

The quantity of lime necessary to achieve the specified plasticity is determined experimentally by performing Atterberg tests on different soil/lime mixtures and specifying the most economical mix.

### 3.6.4 OBJECTIVE: CARBONATION STRENGTH

The quantity of lime necessary to achieve the specified strength is determined experimentally by performing the UCS or CBR or other suitable strength tests on different soil/lime mixtures and specifying the most economical mix.

### 3.6.5 OBJECTIVE: POZZOLANIC/HYDRAULIC SOIL/LIME REACTION STRENGTH

It is believed from stabilization work in the past that most, if not all, tested strength was derived from rapidly generated carbonation strength. The objective of cement reaction strength testing would consequently be

reasonably to guarantee a (pozzolanic) cement reaction strength potential. This may, according to current knowledge, only be achieved by a pH determination of the lime-stabilized soil mixture or, in terms of road design, by determining that quantity of lime which maintains a minimum pH of 10,7 during the lifespan of a road and preferably a pH of 12,4 at 25 °C after 1 hour curing and sufficient lime to overcome organic impurities in stabilized gravel/soil for 1 hour.

BS812: 1960 contains a test to deal with this contingency. The increased solubility of lime in sugar-like, organic-impurity solutions was used by the Zimbabwe Roads Department to establish the lime demand of soils for stabilization. The sucrose (organic impurity) gravel·ICL test is therefore also recommended for use in stabilization work.

Experience seems to indicate, however, that a pH level, after 1 hour, of a  $\text{Ca(OH)}_2$ -created saturation pH is sufficient to provide generally for both organic impurities and pH level.

The pH test method proposed by BS812: 1943, 1967, Eades and Grim (1966) and Clauss and Loudon (1971) was not found satisfactory in practical stabilization work and the trial and error approach suggested by Thompson (1970) was generally preferred. However, the latter test procedure does not determine whether the pH is high enough for a cement reaction to take place, since its only criterion is strength, but not the type by strength of which there are several, eg:

- (a) Modification strength (caused by granulation).
- (b) Compaction strength (uncured, denser packing of particles).
- (c) Carbonation strength (carbonation of lime with cementation effect).
- (d) Cement reaction strength (soil-lime-water reaction).
- (e) Clay reaction strength (clay-lime-water reaction).

Test results also require correction for grading and moisture content.

Attempts have been made in the course of time to develop the pH test approach further in order to secure cement reaction strength in lime-stabilized soils. Methods of grading extrapolation and changes in test parameters were proposed, but none of these changes were good enough for wide acceptance and supplementation of the Thompson approach.

Eades and Grim (1966) proposed a quick test to determine the lime requirements of a soil for lime stabilization. It was designed to measure the quantity of lime that may react with a soil by way of absorption on the exchange sites of clays, or by way of chemical reaction with amorphous silica, alumina, sulphates and phosphates. The reaction of amorphous silica, in particular, takes place only under high alkaline conditions. The success of satisfying the lime requirement was measured in terms of reduced plasticity in soils and, since the main source of soil plasticity is the fines fraction ( $<0,425$  mm), this fraction was used for testing in a slurry of 20 g soil fines and 100 ml water.

Clauss and Loudon (1971) improved this test method by introducing a correction factor for the satisfaction of the lime requirements of 100 ml of water in addition to that of soil and the extrapolation of the lime requirements of the soil fines fraction to include the whole grading. They also stressed the time factor of the original test method by naming it the 'initial consumption of lime (ICL) Test' method. This was necessary because the South African approach to soil stabilization aimed at strength in addition to reduced plasticity and it was believed that lime-stabilized material had an inherent crack-healing and strength-gaining potential with time far in excess of 1 hour.

Clauss (1982) investigated the use of waste lime in soil stabilization. He came to the conclusion that it is reasonable to subdivide lime stabilization into a number of practically significant stabilization reactions as listed in Table 3.20. In particular he distinguished between soil modification to achieve a reduction in plasticity, on the one hand, and cementation to achieve an increase in strength on the

other. It was further possible to distinguish between a strength gain caused by the carbonation of lime and one caused by the formation of cement minerals such as calcium-silicate hydrates. The test method differs, of course, depending on whether stabilization is aimed at reducing plasticity (Atterberg test), carbonation strength (UCS or CBR) or cement reaction strength (pH + UCS or CBR).

De Wet and Skinner (1967) investigated the reaction of clay in high lime concentrations (> 50 %) and came to the conclusion that hydraulic strength may also be expected from calcium-aluminate hydrates at high concentrations of lime. However, since such high lime concentrations rarely occur in soil stabilization, this aspect is not discussed further.

The cement reaction strength, ie the stability of the strength-giving cement minerals of calcium-silicate hydrates (CSH), is particularly sensitive to the maintenance of a pH of greater than 10,5 (Lea, 1970). CSHs decompose if the pH in the soil/lime mixture decreases below a pH of 10,5. A pH test method is therefore essential in stabilization work if the formation of cement reaction strength is aimed at.

The maintenance of the correct cement reaction pH and sufficient lime for organic impurities has been the objective of concrete technology, as documented in BS812: 1943 which, in turn, is based on a test suggested by Abrams and Harder (1917). An updated, suitable test procedure has been under consideration for the past 2 years because the (indicator) ICL test (Clauss and Loudon, 1971) and the lime requirement test (Eades and Grim, 1966) for pH control had the following shortcomings when applied in stabilization work:

- (a) The extrapolation from the fines fraction to the whole-grading material,
- (b) the correction factor for the lime saturation of the water required for testing and

- (c) a sample quantity too small (too large in the case of ICL testing) for representing the whole-grading material conveniently.

The latest specification for a suitable pH test is presented below, followed by the discussion of some of the test variables of a pH test for stabilization work and also for organic impurities.

#### 3.6.5.1 pH determination in lime-stabilized soil

pH testing has only recently been introduced for stabilization work and little practical experience seems to be available. Experience is therefore discussed in what follows. It should be noted, however, that a high standard of accuracy is required since even small errors may increase the cost of stabilizing a road considerably.

##### (a) Test method

###### (1) Description

Determination of the ICL in soil (gravel ICL test procedure).

###### (2) Definition

The test is known as 'gravel ICL' and deviates from the original ICL (soil fines ICL (Eades and Grim, 1966) and corrected soil fines ICL (Clauss and Loudon, 1971) in the following ways:

- . it tests construction material as a whole (crushed to pass a 19 mm sieve) instead of only the -0,425 mm fraction;
- . it uses a 200 g sample instead of one of 20 g and
- . it reduces the water content to just above saturation moisture content (pore moisture) and so dispenses with the necessity of a correction factor for lime saturation of water.

(3) Objective

The objective of the gravel ICL test is the control of the pH in lime- and cement-stabilized soil in order to allow the possible formation of cement minerals, in particular CSH.

(4) Equipment

Balance (accuracy: 0,1 g)

pH meter (accuracy: 0,02 units)

6 plastic beakers (150 ml, tall)

Spatula or palette knife

Soft tissues

Glass beaker (200 ml)

Distilled water

Calcium hydroxide and/or lime to be used

Jet bottle (for cleaning electrode)

Sample splitter (25 mm and 5 mm approximately are sufficient in general)

Drying oven (105-110 °C)

Thermometer (accuracy 0,5 °C)

(5) Preparation of soil samples

. The test is carried out on 2 kg of material finer than 19 mm. Any oversize material is crushed to pass 19 mm as described in TMH1, Method A7, Section 3.1 (NITRR, 1979).

. Oven-dry sample at 105-110 °C.

. The sample is further reduced to 200 g quantities by means of sample splitters and placed into 150 ml (or larger) plastic containers.

(6) Testing procedure

. Since most materials require between 2 and 5 % lime, it is advisable to set up six beakers with lime percentages of 1, 2, 3, 4, 5 and 6 of the dry soil mass, ie 200 g soil plus 2, 4, 6, 8, 10 and 12 g of  $\text{Ca(OH)}_2$ . This will ensure, in most cases, that the

- percentage of lime required can be determined in one hour. Weigh the lime to the nearest 0,1 g and add it to the soil. Mix soil and dry lime.
- . Slightly over-saturate samples with distilled water. The material shall be judged over-saturated 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/lime and water until there is no evidence of dry material on the bottom. Mix for a minimum time of 30 seconds.
  - . Mix for 30 seconds every 10 minutes.
  - . After 1 hour, measure the pH by inserting the pH electrode gently into a hole made in the material with the spatula to a depth of about 20-30 mm and gently covering the inserted part of the electrode with the material. Tap the beaker gently for contact between electrode and material.
  - . Record the pH of each of the lime-soil-water mixtures. The lowest percentage lime from which on the pH remains constant is the saturation lime content of this particular gravel.

(7) Recording of results

The results are recorded to the nearest 0,5 % lime required to produce the maximum pH in the lime-soil-water mixture as indicated by the pH of the mix. Since the pH depends on the temperature (Figure 3.4), this must also be recorded. A form as shown in Figure 3.40 was found to be convenient for recording results.





(b) Instrumentation problems

The lime saturation pH in soil/lime mixtures is usually measured with an electrical pH meter and a glass electrode.

The manufacturer's calibration procedure must be strictly followed. The adjustment of the temperature, asymmetry and slope are of particular importance. The electrode efficiency must be assured.

(1) pH meter

The most important problem to be overcome is that of the pH drift. This describes the movement of the needle during measurement. It is rapid when the electrode is lowered into the sample and gradually decreases with time but does not seem to cease completely in gravel of the basic igneous rock type (Figure 3.3). It may be overcome to a degree by selecting a pH instrument with a drift indicator which correlates the drift with time and reports by means of a pilot light when a predetermined drift:time ratio is maintained, eg 0,009 pH units/60 seconds. It does not eliminate the error but rather ensures that the drift error remains constant and that comparable results are obtained.

(2) Electrodes

It was found over the years that glass electrodes have a very short reliable lifetime in high alkaline mixtures such as those with a pH of 12,4. This refers to Metrohm, Orion and Philips electrodes. The glass bulb was in all cases badly etched and is believed to be the cause of electrode failures.

The suitability is now being investigated of metal electrodes which promise longer life but operate at a lower efficiency (75 % for bismutum compared with 98 % for glass). However, such electrodes also have a reduced sensitivity and the pH readings may be considered reliable to the second decimal only when the layout of the instrument allows third-decimal accuracy. No final recommendation can be made as yet.

The electrode must be washed and dried with soft paper tissue after each measurement. Do not wipe the electrode because doing so may create disturbing static electricity.

An electrode has two sensitive areas. These are:

- . The membrane (bulb, often coloured). It consists of two layers: a solid inner layer and a gel-like outer layer. The outer layer is created by dipping the glass electrode into hydrofluoric acid during manufacture, which hydrates the glass (Van der Merwe, pers comm, 1983,). The thickness of such a layer is measured in microns. The electrode must be kept in water to retain the jelly nature of the outer layer; it is activated by soaking when left dry (see manufacturer's instructions for use attached to each electrode). The sponge properties of the outer layer of the membrane may be reduced during use (etching), and reactivation according to the manufacturer's instructions therefore becomes necessary. The usual procedure uses a weak hydrofluoric acid which removes the inactive outer layer and exposes a fresh and highly active surface of the membrane.

The following methods for reconditioning electrodes have become known in the course of time:

- In a plastic beaker, keep the electrode membrane for not more than 20 seconds in a solution of 50 ml of 1 M acetic acid and 50 ml of 0,1 M  $\text{Na}_2\text{F}_2$ . Remove and wash in distilled water. Soak for 24 hours in a buffer solution with a pH of 4.
- Keep the electrode membrane at room temperature for 2 minutes in a 2 % solution of  $\text{NH}_4\text{HF}_2$ . Remove and rinse carefully in  $\text{H}_2\text{O}$ . Soak for 5 hours in  $\text{H}_2\text{O}$  at 59-60 °C.

Neither treatment was satisfactory.

It has also not been established yet whether the above recipes are for specific glass types or whether they have a general application.

Another reason for electrode failures is scratches on the electrode surface. Both etching and scratching have been observed (Plate 3.3).

- . The diaphragm. This is a porous ceramic window in the glass mantle of the electrode. It shows as an approximately 1-2 mm diameter black spot and separates the sample solution or liquid from the electrode liquid. The electrode liquid flows very slowly from the electrode out into the sample liquid. The contact between the two liquids is very easily and frequently blocked by dirt, reducing the efficiency of the electrode. The manufacturer recommends dipping the electrode in Na(OH) solution to clear any clogging (and also generally for cleaning the membrane).

However, the method has its limitations and mechanical removal (filing) of the dirt layer may become necessary, specifically if the dirt consists of a carbonate cake. It is also recommended that the electrode be dipped into a weak HCL solution after lime determinations in order to remove any carbonate that may have formed on the surface of the membrane.

It was found that electrodes used in lime stabilization work ceased to function after 2 weeks of use, while the design life of electrodes is approximately 2 years (Philips, manufacturer's instructions, 1982).

There is currently no established way around the problem of electrode etching, although thick-wall electrodes may delay the breakdown and increase service life somewhat. A solution may also be offered by some of the metal electrodes that are designed to measure the pH in etching liquids such as hydrofluoric acid.



## PLATE 3.3

Etching of pH glass electrode (dark area, top) and intense scratching of the surface of the membrane (criss-cross of scratches on "intact" surface). Enlarged x50.

Note, however, that most metal electrodes are designed to be ion-specific (redox potential) and are not suitable for pH work at this stage.

It was also noted that the distance between reference electrode and measuring metal electrode must be constant during calibration and measurement of the pH of lime-stabilized soil mixes in order to achieve repeatable readings.

### (c) Conclusions

The test programme in stabilization work depends on the purpose to be achieved by the process ie plasticity manipulation (Atterberg tests), carbonation strength (UCS or CBR) or cement reaction strength (pH and UCS or CBR). pH determinations in lime-stabilised soils are considered to be routine tests, although the interpretation of some test results indicated problems related to the pH meter, its calibration or even faulty electrodes.

pH drift during testing may be neutralized by the taking of readings at a predetermined drift:time ratio.

The parameter measured by the ICL test is the pH of the soil moisture.

### 3.6.6 OBJECTIVE: ORGANIC-MATTER IMPURITIES

#### 3.6.6.1 Introduction

Organic impurities of the sugar type are not always bad. This is already shown in Figure 3.38 where sugar is shown to reduce the OMC (30 %) and increase the MDD (15 %) of lime-stabilized soil. The UCS of 5 % lime-stabilized and 1 % sugar-stabilized soil is twice that of an 18 % lime-stabilized soil. The reason for this improvement is not yet known,

but may be the greater solubility of lime in a sugar solution. The extent of this reaction may be observed in a hardening experiment with OPC. Two equal quantities of OPC were measured off and one of them was contaminated with 1 % of sugar. Distilled water was added to make a smooth paste in both cases. More water was added so that a surplus of 1-5 mm water was standing on top of the pastes. This was maintained for 4 weeks. It was found after this period that severe spalling and disintegration had occurred in the sugar-contaminated sample. The control sample showed no irregularities and it appears that a reaction acceleration according to Formula (7) was taking place.

The test results seem to indicate that organic impurities may be tolerated and even be beneficial for achieving carbonation strength because of the approximately 100 x higher solubility of lime in sugar solution (Boynton, 1980).

When no surplus moisture is available, as may be assumed in road construction generally the decomposition of CSHI and II results in the formation of  $\text{CaCO}_3$  and a relevant carbonation strength may be expected only if sufficient  $\text{CO}_2$  is available and the powder phase of  $\text{CaCO}_3$  is avoided.

This is important because sugar-type impurities are considered in the literature as retarding concrete additives. Formulae (5) to (8), however, clearly indicate that it more likely acts as a powerful accelerator.

A sucrose method of testing was introduced by the Zimbabwe road authorities in the place of the ICL test for determining the lime required for stabilization (Standards Association of Central Africa, 1974). However, the two different methods measure different soil parameters, ie pH in ICL testing and the solubility of lime in organic-matter contaminated soil moisture in the sucrose test.

The standard graphs for different limes are presented in Figure 3.41, showing the soluble  $\text{Ca}^{++}$  ions in a quantity of lime (free lime) in a

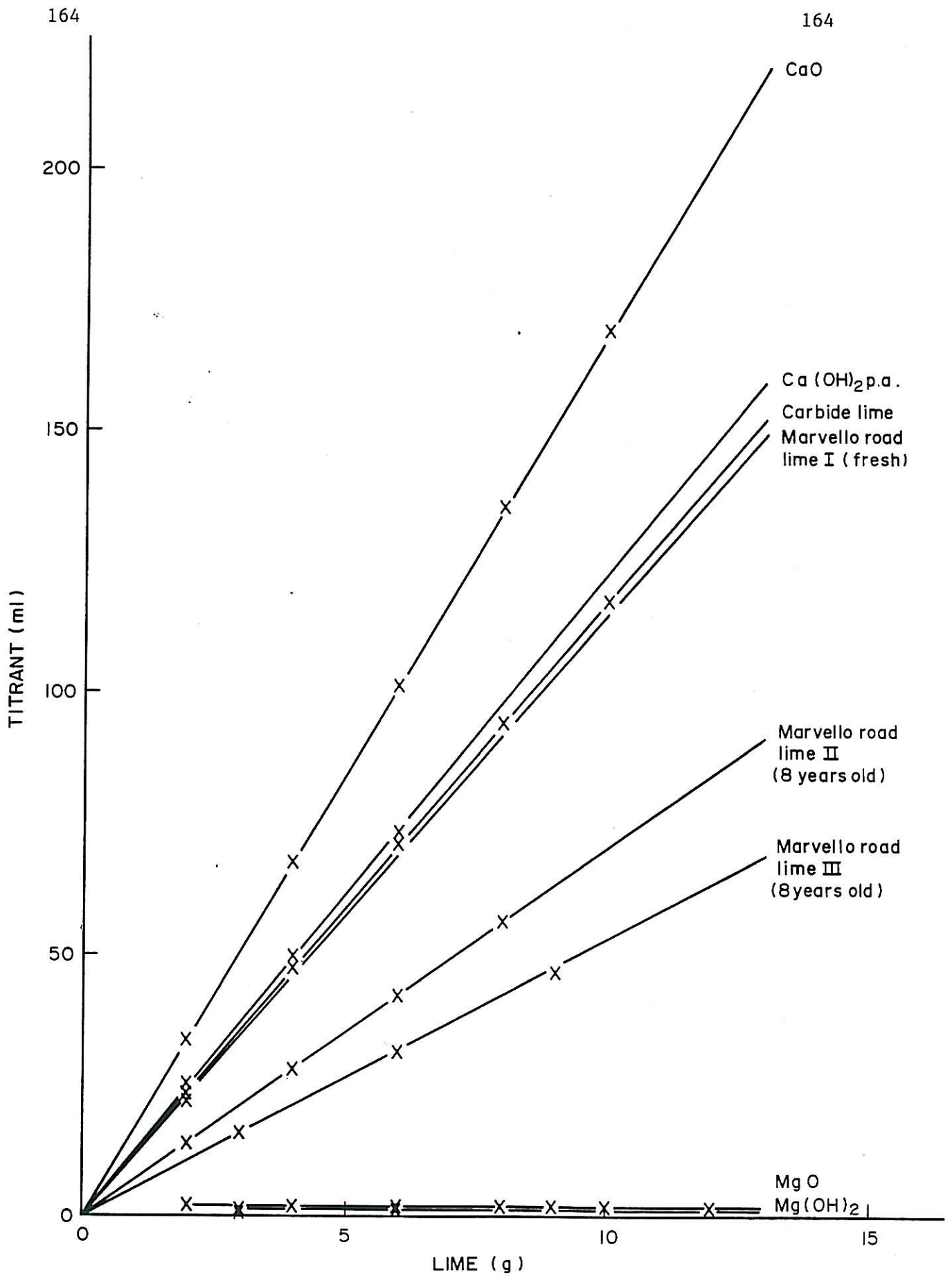


FIGURE 3.41

Standard graph of different lime types according to CAS A43 (1974)-ICL by sucrose method (SABS 824:1967, as amended) - showing the free lime content in different types of stabilizers relative to each other.

sugar solution of specified concentration. It may be concluded from this that the method determines the quantity of calcium ions ( $\text{Ca}^{++}$ ) only in different brands of lime. Magnesium ions ( $\text{Mg}^{++}$ ) or  $(\text{OH})^-$  ions are not determined.

The solubility of lime ( $\text{Ca}(\text{OH})_2$ ) in sugar solution is approximately 100 x higher than in water (Boynton, 1964) and lime will consequently concentrate in such sugar solutions. The pH of lime-saturated sugar solutions was found to be the same as in the lime water suspensions (Clauss, 1982). Although, therefore, the pH level in stabilized soil may be above the critical pH of 10,5, the formation of cement minerals is prevented because no lime ( $\text{Ca}(\text{OH})_2$ ) is available for the reaction with the soil.

#### 3.6.6.2 Test method

##### (a) Description

Organic impurities gravel ICL.

##### (b) Definition

The test is known as organic impurities gravel ICL and deviates from the original 'Determination of the initial consumption of lime by the sucrose method' as published by the Standards Association of Central Africa (1974) and called, for short, sucrose method ICL.

- (1) by testing construction material as a whole in place of the -4,75 mm fraction only and
- (2) through interpretation.



(c) Purpose

The purpose of this test is to determine the  $\text{Ca(OH)}_2$  dissolved and deactivated in soil moisture during the first hour of reaction in a lime-stabilized soil mixture.

(d) Apparatus

- (1) Five 1 000 ml glass measuring cylinders fitted with caps;
- (2) ten 500 ml Philips beakers;
- (3) five 250 ml Philips beakers;
- (4) 50 ml pipette;
- (5) 1l measuring cylinder;
- (6) 100 ml burette, accurate to 0,2 ml;
- (7) a balance, capacity 200 g, accurate to 0,001 g;
- (8) spatula or palette knife;
- (9) a shaker or slowly rotating mechanical mixer;
- (10) plastic wash bottle.

(e) Reagents

- (1) Weak solution of HCl (N/10 approximately);
- (2) Pure white cane sugar;
- (3) Phenolphthalein;
- (4) Chemically pure calcium oxide, best obtainable grade (not less than 97 %);
- (5) Concentrated hydrochloric acid, chemically pure, of approximately 35 % concentration.

(f) Preparation of reagents

- (1) N/10 hydrochloric acid  
Add 100 ml of chemically pure, approximately 35 % concentration, hydrochloric acid to 500 ml of distilled water in a 1l

measuring cylinder. Make up to 1 000 ml with distilled water. Shake. Dilute 1 volume of this solution with 9 volumes of distilled water to make the N/10 (approx.) solution.

Make a sufficient quantity to ensure that the same batch is used for all tests in the series.

(2) Phenolphthalein indicator

Dissolve 1 g of phenolphthalein powder in 100 ml of ethyl alcohol and 100 ml of distilled water and mix well.

(g) Determination of standard graph

The purposes of a standard graph are:

- . to ensure that the mass of sugar used in each test, usually 150 g, is sufficient to absorb 10 g of lime;
- . to determine the increments per millilitre titration for each 1g gram of lime;
- . to prove the accuracy of titration.

The determination of a standard graph is described below.

- (1) Take 5 samples of lime of a mass of 2, 4, 6, 8 and 10 g and place each sample into a Philips beaker.
- (2) To each beaker add approximately 50 ml of distilled water and allow to stand for 30 minutes.

- (3) Add  $\pm$  150 g of pure white cane sugar to each sample, transfer quantitatively using a wash bottle and distilled water, into a mixing device and mix thoroughly. Mix for 1 minute in a mechanical mixer, or 5 minutes in a hand mixer.
- (4) Transfer each individual sample quantitatively into a 1 000 ml measuring cylinder, and fill to the 962 ml mark (see Note (a)).
- (5) Close each measuring cylinder with a cap; shake thoroughly by turning upside down and back at least 20 times. Allow to stand until any sediment has settled.
- (6) Draw off exactly 50 ml of the clear liquid from each sample and titrate against N/10 HCl, using phenolphthalein as an indicator.
- (7) Plot a graph of millilitres of titrant against grams of lime, as shown in Figure 3.42.
- (8) The results should produce a straight line graph, thus enabling the millilitres of titrant (1) required for a 1 gram increment of lime to be determined.

If insufficient sugar has been used there will be a tendency for the upper end of the graph to curve downwards, as indicated by the dotted line.

Any other deviations from a straight line indicate inaccuracies in the measuring of the samples or in the titrations and the determination of the standard graph shall be repeated.

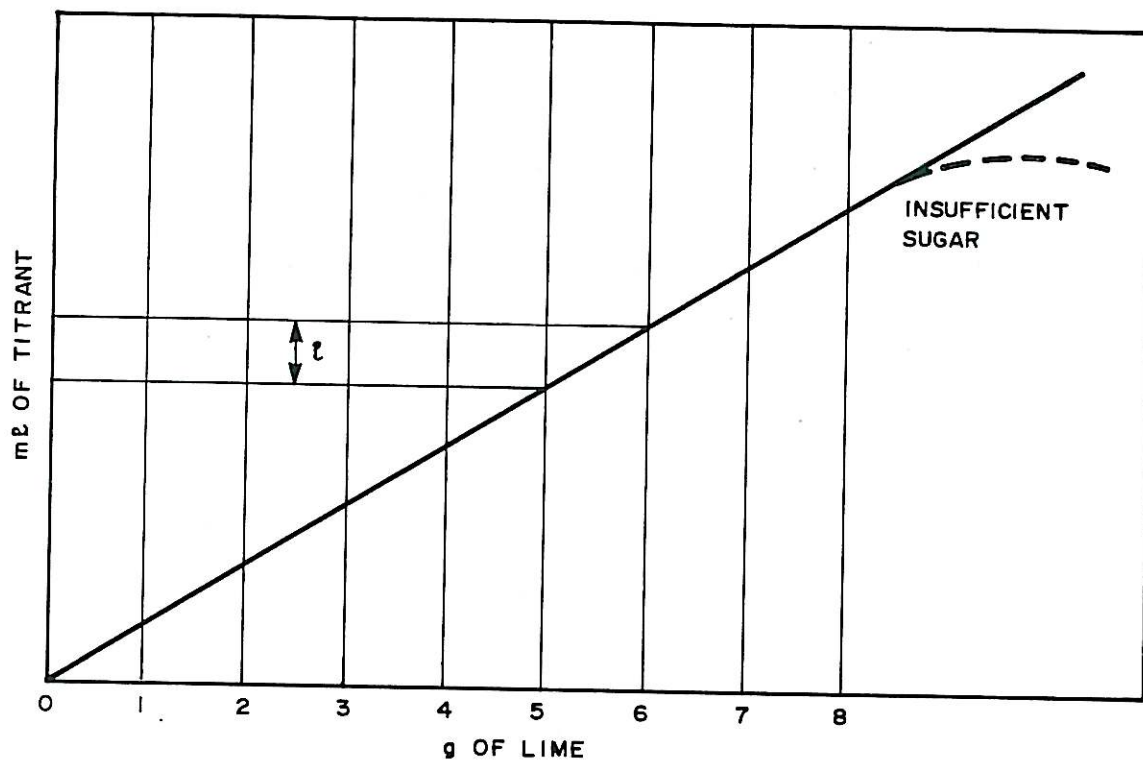


FIGURE 3.42

*Graph of titration against gram of lime.*

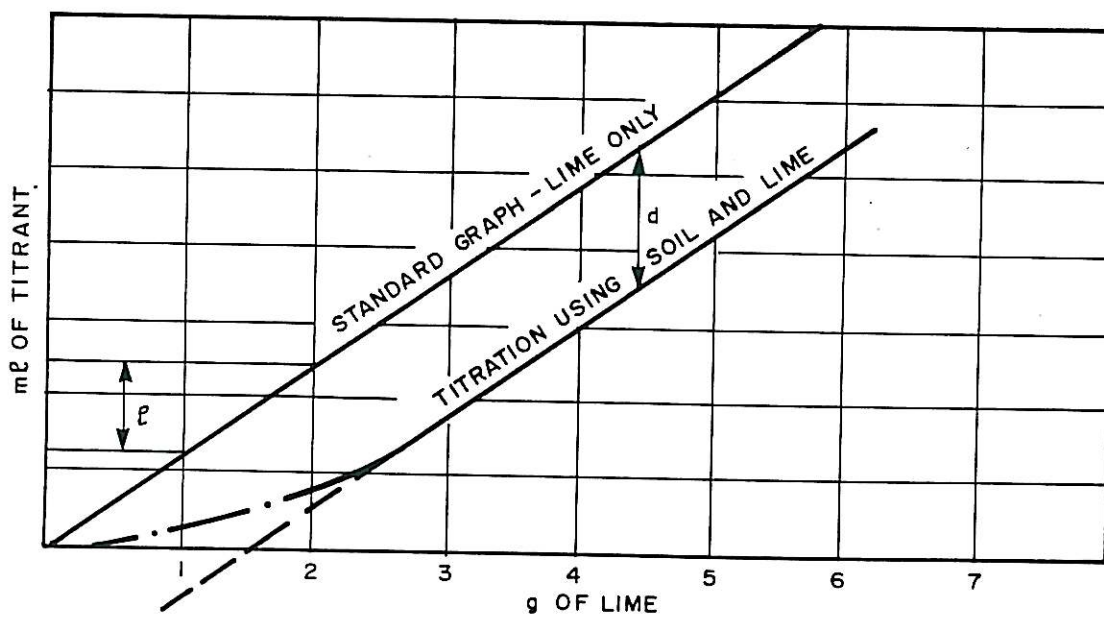


FIGURE 3.43

*Graph of titration against gram of lime in sample.*

(h) Preparation of soil samples

- (1) The test is carried out on 2 kg of material finer than 19 mm. Any oversize material is crushed to pass 19 mm as described in TMH1, Method A7, Section 3.1 (NITRR, 1979).
- (2) Oven dry sample at 105-110 °C.
- (3) The sample is further reduced to 100 g quantities by means of sample splitters and placed into 150 ml (or larger) plastic containers.

(i) Testing procedure

- (1) Since most materials require between 2 and 5 % lime, it is advisable to set up six beakers with lime percentages of 1, 2, 3, 4, 5, and 6 of the dry soil mass. This will ensure, in most cases, that the percentage of lime required can be determined in 1 hour. Weigh the lime to the nearest 0,1 g and add it to the soil. Mix the soil and dry lime.
- (2) Place each of the dry soil/lime specimens in the mixing bowl and add approximately 50 ml of distilled water. Mix thoroughly. Any material adhering to the mixing instrument shall be washed into the bowl with distilled water. Transfer each specimen to a beaker of adequate capacity and allow to stand for 30 minutes.
- (3) To each beaker add 150 g of pure cane sugar. Return each specimen to the mixing bowl and mix as follows:

mechanical mixing: 1 minute

hand mixing: 5 minutes

- (4) Transfer each specimen to a 1 000 ml measuring cylinder; fill to the 1 000 ml mark with distilled water. Close the cylinders with caps and shake thoroughly by turning upside down and back at least 20 times. Allow to stand until any material in suspension has settled.
- (5) Draw off 50 ml of clear liquid from each specimen. Using phenolphthalein as indicator, titrate against the same batch of N/10 hydrochloric acid as was used in the preparation of the standard graph.
- (6) Plot grams of lime against millilitres of titrant for each specimen on the standard graph (see Figure 3.43). The result should produce a straight line running below and parallel to the straight line of the standard graph.

(j) Calculations

$$\text{ICL} = \frac{\frac{d}{l} \times 100}{100} = \frac{d}{l} \%$$

where

d = millilitres of titrant required for a given mass of lime minus millilitres of titrant required for the same mass of lime mixed with soil.

l = millilitres of titrant required for 1 g of lime, read from the standard graph.

(k) Reporting of results

Report the results to the nearest 0,1 %.

## (1) Notes

$$(1) \quad 1\,000 - \frac{105}{2.65} = 962 \text{ ml}$$

where  $2.65 \text{ g/cm}^3$  = average density of soil grains.

If the soil grains are considered to have a significantly different density, then the measurement may be calculated from the expression:

$$1\,000 - \frac{100}{\text{density}}$$

The reduction  $\frac{100}{\text{density}}$  ml from 1 000 ml represents the volume occupied by the soil described in the test procedure.

The reduction is necessary to ensure that the concentrations are kept constant for both the 'standard' and "test" as the available lime is measured indirectly in terms of millilitres of hydrochloric acid.

- (2) The cementation lime content is the quantity of lime which allows a pozzolanic reaction to take place in lime stabilized soil. This depends on both the cations ( $\text{Ca}^{++}$ ) and on the anions ( $(\text{OH})^-$ ) supplied by the lime additive and available for a cement reaction and on the pH.

The method is based on the fact that calcium hydroxide ( $\text{Ca}(\text{OH})_2$ ) is more soluble in sugar solution than in aqueous solution as a result of the acidic alcohol groups in sugar reacting with calcium hydroxide to form the saccharate ( $\text{C}_{12}\text{H}_{22}\text{O}_{11} \cdot 3\text{CaOH}$ ). The saccharate has a limited solubility, however, and precipitates out at a definite concentration in water (Table 3.21). Evidence seems to indicate that lime is progressively covered with a layer of

TABLE 3.21  
Effect of increasing sugar concentration on  
the solubility of lime (Boynton, 1980).

25 °C			80 °C		
g/100 g sugar	Sat. sol. CaO	Solid phase	g/100 g sugar	Sat. sol. CaO	Solid phase
0,0	0,122	Ca(OH) <sub>2</sub>	0,0	0,071	Ca(OH) <sub>2</sub>
2,1	0,242	Ca(OH) <sub>2</sub>	4,90	0,117	Ca(OH) <sub>2</sub>
4,2	0,461	Ca(OH) <sub>2</sub>	9,90	0,189	Ca(OH) <sub>2</sub>
6,6	0,750	Ca(OH) <sub>2</sub>	11,75	0,230	Ca(OH) <sub>2</sub>
8,6	1,11	Ca(OH) <sub>2</sub>	19,50	0,358	Ca(OH) <sub>2</sub>
11,8	1,86	Ca(OH) <sub>2</sub>	21,60	0,518	Ca(OH) <sub>2</sub>
15,4	2,76	Ca(OH) <sub>2</sub>	29,70	1,017	Ca(OH) <sub>2</sub>
21,1	4,53	Ca(OH) <sub>2</sub>			
27,2	6,72	Ca(OH) <sub>2</sub>			
31,4	8,39	Ca(OH) <sub>2</sub>			
35,2	9,8	Ca(OH) <sub>2</sub> Saccharate			
35,0	10,1	Saccharate			
36,0	9,8	Saccharate			
43,7	8,84	Saccharate			
53,2	7,87	Saccharate			
68,3	4,08	Saccharate			



saccharate, which impedes the eventual dissolution of lime and a very high sugar concentration causes the solubility of lime to retrogress. Some soils extract available lime from lime and thus the addition of lime to these soils does not stabilize them until this 'available lime' demand has been met.

The 'available lime' content of lime is therefore reduced in the presence of soils and the 'available lime' content determination above can thus be used to determine the amount of lime needed to provide the minimum 'available lime' demanded by the soil.

- (3) See gravel ICL for the determination of the pH of lime stabilized soil mixtures.

#### 3.6.6.3 Conclusion

The sucrose test determines the solubility of lime in soil moisture. The solubility of lime is influenced by sugar-like organic impurities contained in the soil and by lime reacting with soil particles.

### 3.7 The time factor in soil stabilization

The Morgenzon stabilization experiment which was initiated some 14 years ago is described in what follows in order to demonstrate the long-term aspects of stabilization work.

#### 3.7.1 OBJECTIVE OF FIELD EXPERIMENT

The objective of the experiment was twofold:

- (a) to find out, whether the weathering of dolerite in road pavements can be retarded as might be expected from laboratory experiments (Clauss, 1967), and
- (b) to find out which has more advantages in road construction: cement-stabilization or lime-stabilization.

### 3.7.2 LAYOUT OF EXPERIMENT

#### 3.7.2.1 Design

The experiment was subdivided into four test sections, each approximately 60 m in length, over the full width of the road and under a triple seal black top (tar).

The road was designed to consist of a total cover of 550 mm on black clay, as given below.

Surfacing	50 mm
Base	125 mm of a material with a PI of < 6 and a CBR of 80 % at 98 % Mod AASHTO.
Subbase	125 mm of a material with a PI of about 7 and a CBR of 45 % at 95 % Mod AASHTO.
Fill	300 mm of a material with a PI of about 7 and a CBR of 15 % at 93 % Mod AASHTO.

Stabilization was carried out as indicated in Figure 3.44.

#### 3.7.2.2 Materials

The material for all road layers was obtained from borrow pit no 7 near Morgenzon. Its technical properties at the time of construction are

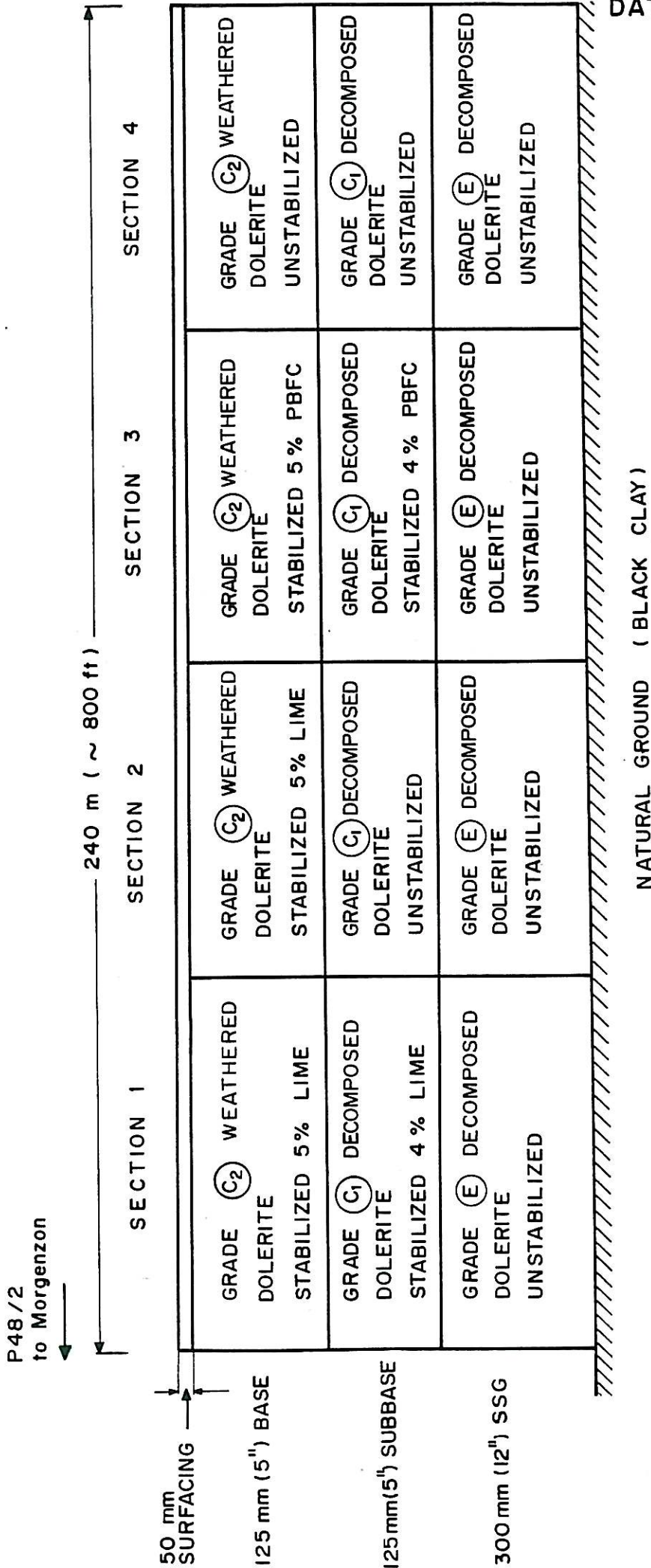


FIGURE 3.44  
 Section of layout of experimental road. The letters refer to different materials locations in the dolerite quarry, Morgenzon. (see Fig. 3.45).

TABLE 3.22  
Construction record of an experimental section with one follow-up test 3 and 4 after 3 years

Description	Test	Section 1		Section 2		Section 3			Section 4		
		TPA 1968	NITRR 1968	TPA 1968	NITRR 1968	TPA 1968	NITRR 1971	NITRR 1968	TPA 1968	NITRR 1968	NITRR 1971
50 mm black top	Ref No:		2661	-	2665	-	2669	RR 1400	-	2673	RR 1270
	LL	NP	NP	NP	SP	NP	SP	NP	33	34,1	36,0
125 mm base	PL	-	-	-	-	-	-	-	22	23,0	21,8
	PI	NP	NP	NP	SP	NP	SP	NP	11	11,1	14,2
	LS	ND	ND	ND	ND	ND	ND	1,3	5,3	7,0	9,3
	ICL	-	-	-	-	-	-	-	-	-	-
	<0,425 mm	33	4,8	33	13,3	38	11,1	15	30	23,1	17
	CBR*	217	-	218	-	453	-	(42)	100	-	(56)
	TAB	5 % L	5 % L	5 % L	5 % L	5 % PBFC	5 % PBFC	5 % PBFC	Nat	Nat	Nat
130 mm below top	OMC	10,6	-	10,6	-	-	-	14,4	-	-	7,9
	MDD	2177	-	2177	-	-	-	1969	-	-	2285
	Field density	2235	-	2212	-	2156	-	-	2262	-	-
	Rel comp	103	-	102	-	102	3,7	-	104	3,6	-
	Ref No	-	22662	-	2662	-	2670	RR 1275	-	2674	RR 1282
125 mm Subbase	LL	NP	39,8	37	37,2	NP	NP	39,2	37	36,1	40,0
	PL	-	20,8	26	23,8	-	-	37,6	26	22,0	23,2
	PI	NP	19,0	11	13,4	NP	NP	1,6	11	14,1	16,9
	LS	ND	9,3	6,7	7,7	0,7	ND	2,7	8,7	9,3	-
	ICL	-	-	-	-	-	-	-	-	-	-
	<0,425 mm	46	7,4	44	22,3	45	21,9	4	244	21,8	28
	CBR*	205	-	71	-	350	-	(45)	72	-	(21)
255 mm below top	Stab	4 % L	Nat	Nat	Nat	4 % PBFC	4 % PBFC	4 % PBFC	Nat	Nat	Nat
	OMC	12,6	-	12,2	-	12,6	-	11	12,2	-	9,4
	MDD	2028	-	2067	-	2025	-	2100	2067	-	2160
	Field dens	2053	-	2047	-	2017	-	-	2134	-	-
	Rel comp	101	-	99	-	100	-	-	103	-	-

Continued/...

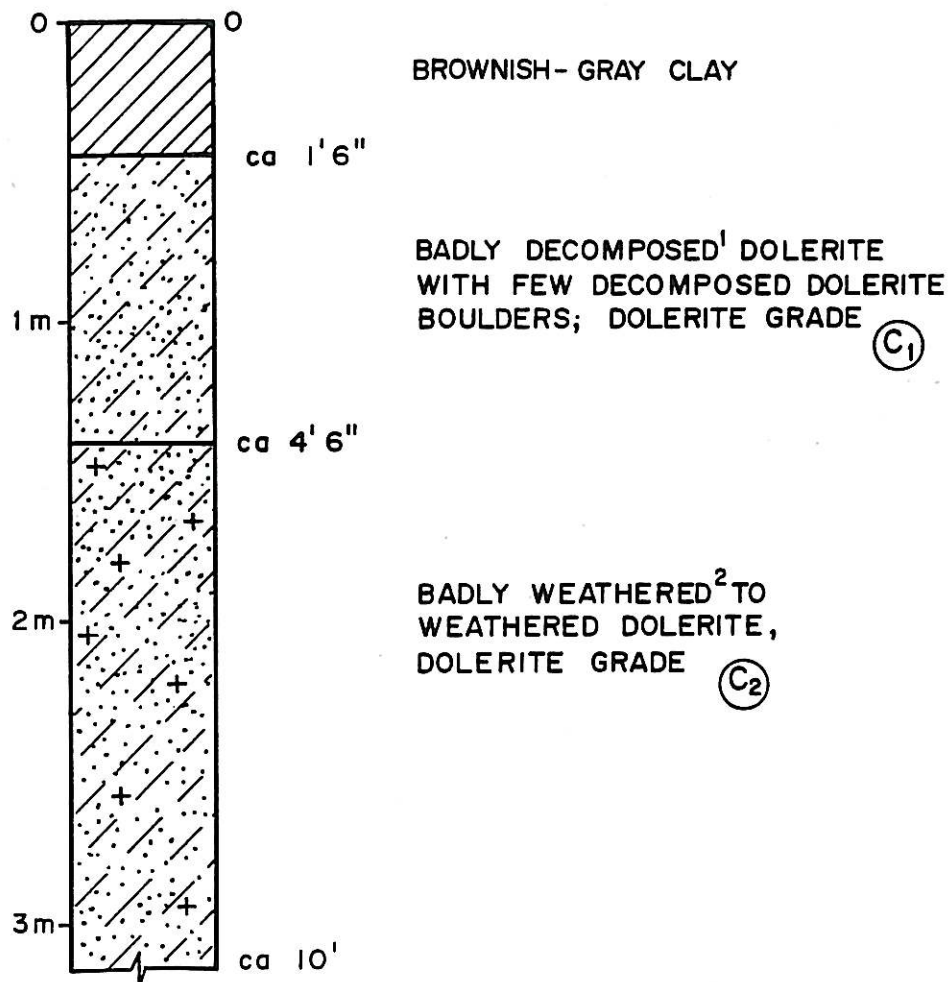
TABLE 3.22 (cont.)

Description	Test	Section 1		Section 2		Section 3			Section 4		
		TPA 1968	NITRR 1968	TPA 1968	NITRR 1968	TPA 1968	NITRR 1971	NITRR 1968	TPA 1968	NITRR 1968	NITRR 1971
	Ref No:	-	2663	-	2667	-	2671	RR 1247	-	2675	RR 1375
Top 125 mm of 300 mm fill (lower subbase)	LL	40	39,6	-	33	-	39,9	38,8	-	37,4	41,8
	PL	24	22,2	-	26	-	23,1	27,1	-	21,7	23,8
	PI	16	17,4	-	7	-	16,8	11,7	-	15,7	18,0
	LS	6,7	9,7	-	9	-	7,7	7,2	-	7,3	10,0
	ICL	-	-	-	-	-	-	-	-	-	-
	<0,425 mm	17	10,2	-	24,5	-	26,0	19	-	27,0	22
	CBR*	67	-	-	-	92	-	(3)	-	-	(24)
	Stab	Nat	Nat	-	Nat	-	Nat	Nat	-	Nat	Nat
	OMC	10,16	-	-	-	-	-	8,3	-	-	11
	MDD	2134	-	-	-	-	-	2290	-	-	2210
Field dens	2043	-	-	-	-	2089	-	-	-	-	
Field MC	8,2	4,2	-	3,8	-	25,7	-	-	3,9	-	
Rel comp	96	-	-	-	-	98	-	-	-	-	
	Ref No:	-	2264	-	2668	-	2672	-	-	2676	-
Natural ground	LL	-	57,5	-	60,2	-	48,2	-	-	50,8	-
	PL	-	21,7	-	25,9	-	21,2	-	-	21,7	-
	PI	-	35,8	-	34,3	-	27,0	-	-	29,1	-
	LS	-	-	-	18,7	-	13,3	-	-	15,3	-
	ICL	-	-	-	-	-	-	-	-	-	-
<0,425 mm	-	72,6	-	90,2	-	74,1	-	-	85,4	-	

\*Brackets refer to reworked samples

PROJECT: Dolerite stabilization experiment, Morgenzon, T.V.I.

DATE: 23.2.68



Note

- 1 Chemical weathering mainly.
- 2 Chemical and physical weathering combined.

FIGURE 3.45

*Sketch of soil profile in quarry, Morgenzon*

TABLE 3.23  
Initial consumption or lime (ICL)<sup>1</sup> of a dolerite weathering  
succession from from Morgenzon, Transvaal.

NITRR Ref No	Material description	ICL (%) (-0,425 mm)
2656	Fresh	0,5
2657	Weathered	1,5
2658	Badly weathered <sup>2</sup>	2,5
2660	Badly decomposed <sup>3</sup>	2,5
2659	Residual soil	3.5

Notes

- 1 20 g, -0,425 mm, 100 ccm H<sub>2</sub>O, 1 h.
- 2 Chemical and physical weathering combined.
- 3 Chemical weathering mainly.

TABLE 3.24  
Gravel ICL\* of weathered dolerite from borrow pit as used in  
stabilization experiment (collected 1982).

NITRR Ref No	Material description	Gravel ICL
9648	C <sub>1</sub> type (chemically) weathered	5 %
9838	C <sub>2</sub> type (chemically and physically) weathered	3 %

\*200 g, whole-grading, moisture saturated, 1 hour.

given in Table 3.22. Later test results are also given in Table 3.22. An attempt was made to vary the borrow material by concentration on working horizons as indicated in the soil profile (Figure 3.45). The ICL was determined at a later stage and is presented in Tables 3.23 and 3.24.

### 3.7.2.3 Construction

Control testing consisted of grading (Figures 3.46-3.51), laboratory CBR/field density and measuring Atterberg limits on each road layer (Table 3.22).

### 3.7.2.4 Test programme

No test programme was proposed.

### 3.7.2.5 Traffic

Three traffic counts were carried out during the course of the experiment and it may be seen that the traffic load in 1980 was more than three times that at the time of construction (Table 3.25), although probably not in terms of axle loads.

## 3.7.3 FAILURE OF THE UNSTABILIZED SECTION

The unstabilized section 4 of the experimental road was reported to have failed in 1971 and a field inspection was carried out by Messrs Fulger, Holleman and Schiller (21/9/71). Of all the sections only 3 and 4 showed any distress. Using the visual evaluation method (Figure 3.52) (Curtayne, 1971), the distress rates were described as follows:



PARTICLE SIZE DISTRIBUTION

NITRR DATE: SEPT. 68

SAMPLE No.	1584 / 2661	2662	2663	2664	PROJECT .9431 / 4 231
SAMPLE LOCATION	MORGENZON	(SECTION I)			
DEPTH	BASE	SUBBASE	SSG	NAT.GROUND	JOB No. 184
CLASSIFICATION					

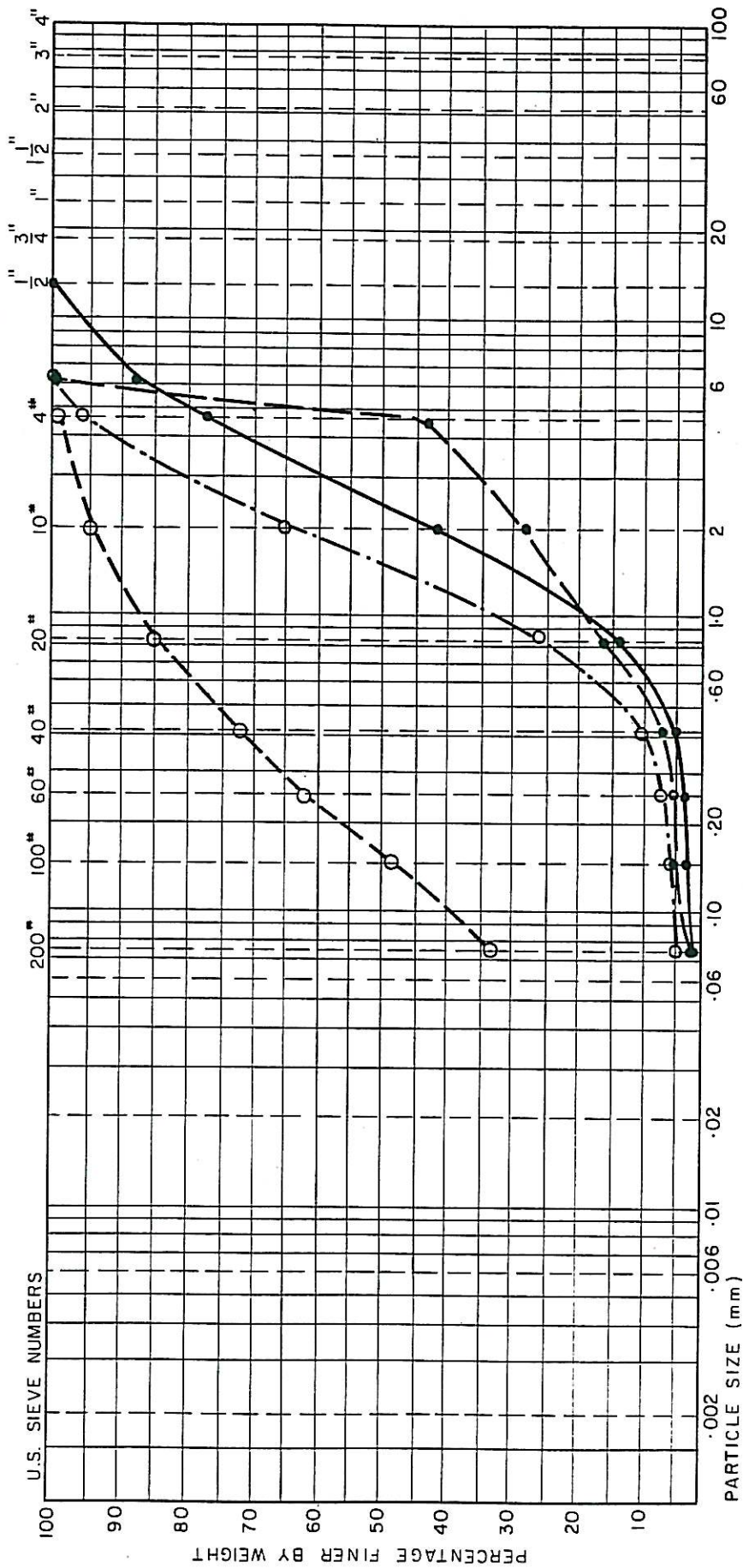


FIGURE 3.46

Particle - size distribution of material used in the Morgenzon stabilization experiment.

CLAY FRACTION	FINE	MEDIUM	COARSE	GRAVEL FRACTION		
	SAND FRACTION			FINE	MEDIUM	COARSE

PARTICLE SIZE DISTRIBUTION NITRR DATE: SEPT. 68

SAMPLE No.	1588 / 2665	2 666	2 667	2 668	PROJECT 9431 / 4231
SAMPLE LOCATION	MORGENZON	(SECTION II)			
DEPTH	BASE	SUBBASE	SSG	NAT. GROUND	JOB No. 184
CLASSIFICATION					

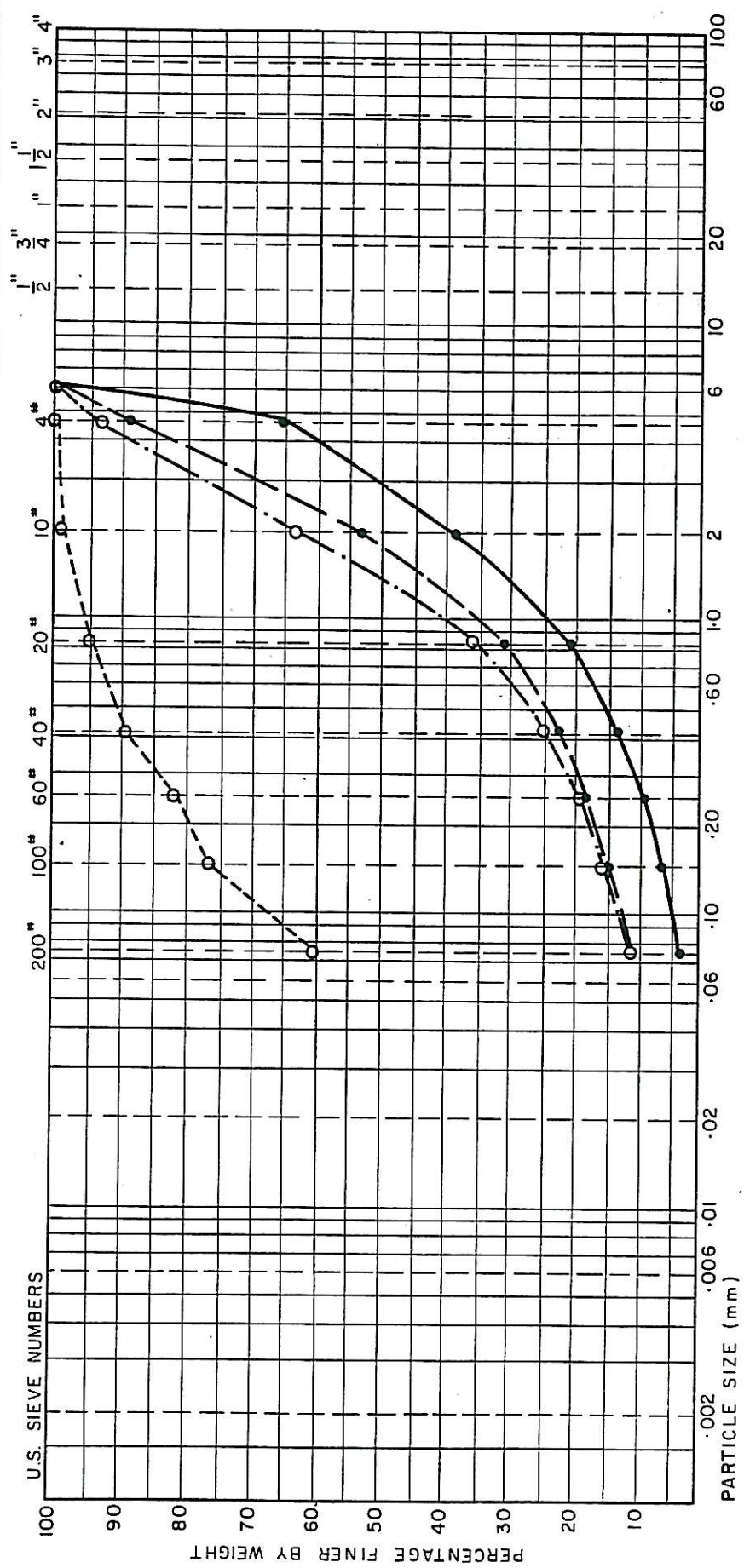


FIGURE 3.47

Particle - size distribution of material used in the Morgenzon stabilization experiment.

CLAY FRACTION	FINE SILT FRACTION		SAND FRACTION		GRAVEL FRACTION	
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE

PARTICLE SIZE DISTRIBUTION NITRR DATE: SEPT. 68

SAMPLE No.	1592 / 2669	2670	2671	2672	PROJECT 9431/4231
SAMPLE LOCATION	MORGENZON	(SECTION III)			
DEPTH	BASE	SUBBASE	SSG	NAT.GROUND	JOB No. 184
CLASSIFICATION					

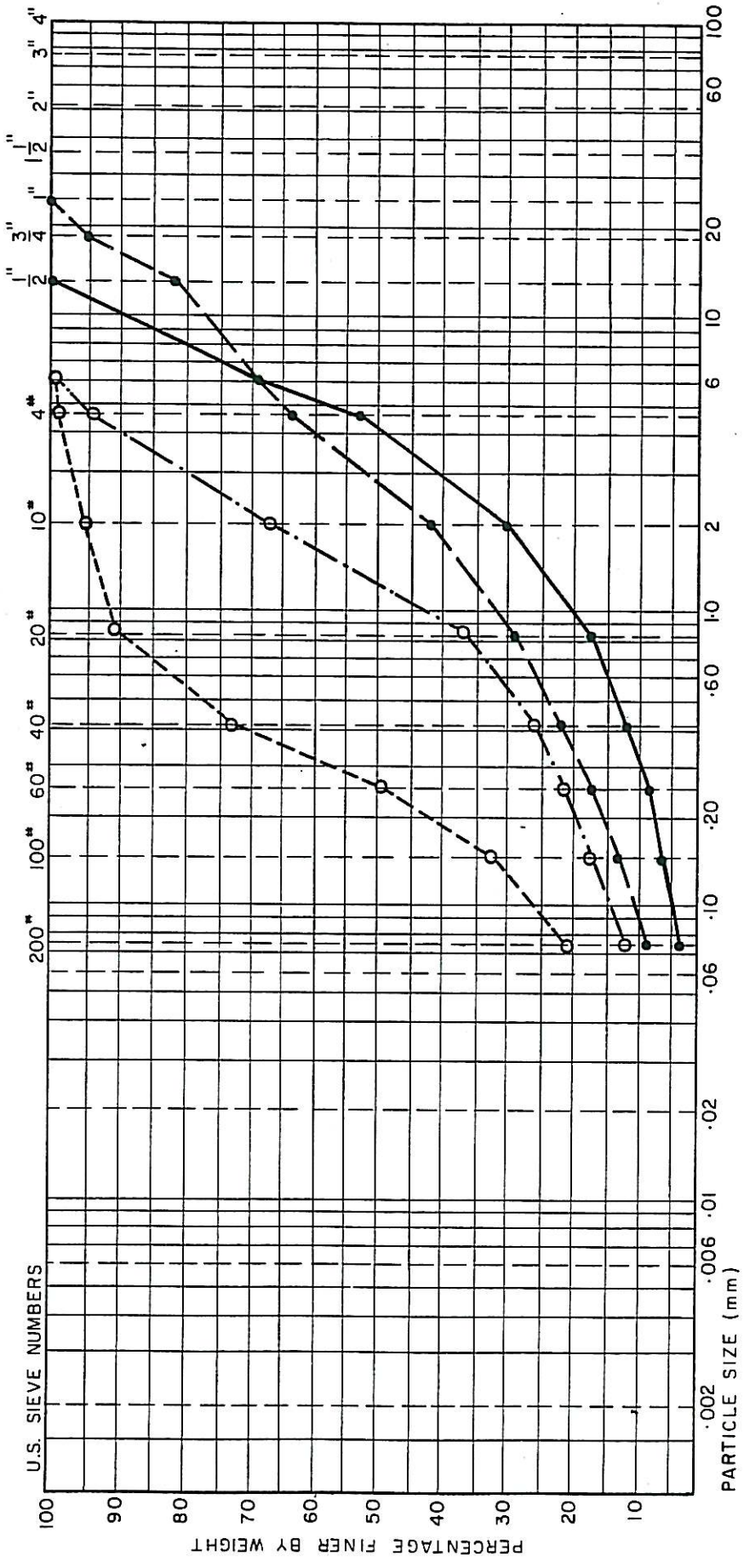


FIGURE 3.48

Particle-size distribution of material used in the Morgenzon stabilization experiment.

CLAY FRACTION	FINE SILT	MEDIUM FRACTION	COARSE SAND	FINE SAND	MEDIUM FRACTION	COARSE GRAVEL	MEDIUM FRACTION	COARSE FRACTION
---------------	-----------	-----------------	-------------	-----------	-----------------	---------------	-----------------	-----------------

PARTICLE SIZE DISTRIBUTION

NITRR DATE: SEPT. 68

SAMPLE No.	1596 / 2673	2674	2675	2676	PROJECT 9431/4231
SAMPLE LOCATION	MORGENZON	SECTION (IV)			
DEPTH	BASE	SUBBASE		NAT. GROUND	
CLASSIFICATION		SSG		JOB No. 184	

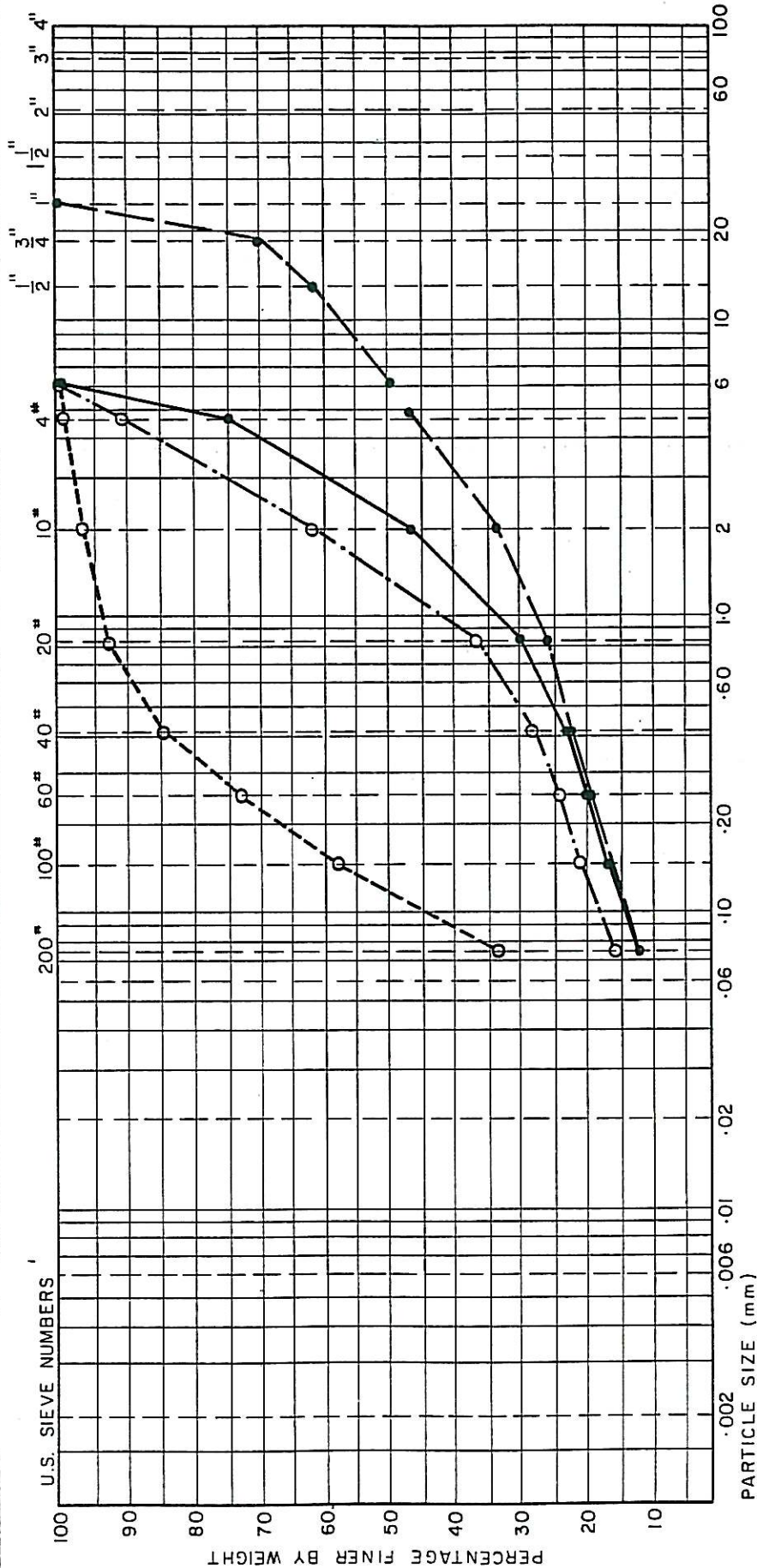


FIGURE 3.49 Particle - size distribution of material used in the Morgenzon stabilization experiment.

CLAY FRACTION	FINE	MEDIUM	COARSE	FINE GRAVEL FRACTION		COARSE
	SILT	FRACTION	SAND	FRACTION	GRAVEL	FRACTION

PARTICLE SIZE DISTRIBUTION

NITRR

DATE: 18.4.71

SAMPLE No.	RR 1400	RR 1275	RR 1247	PROJECT	4561
SAMPLE LOCATION	MORGENZON	(SECTION 3)			
DEPTH	BASE	SUBBASE	SSG	JOB No.	361
CLASSIFICATION					

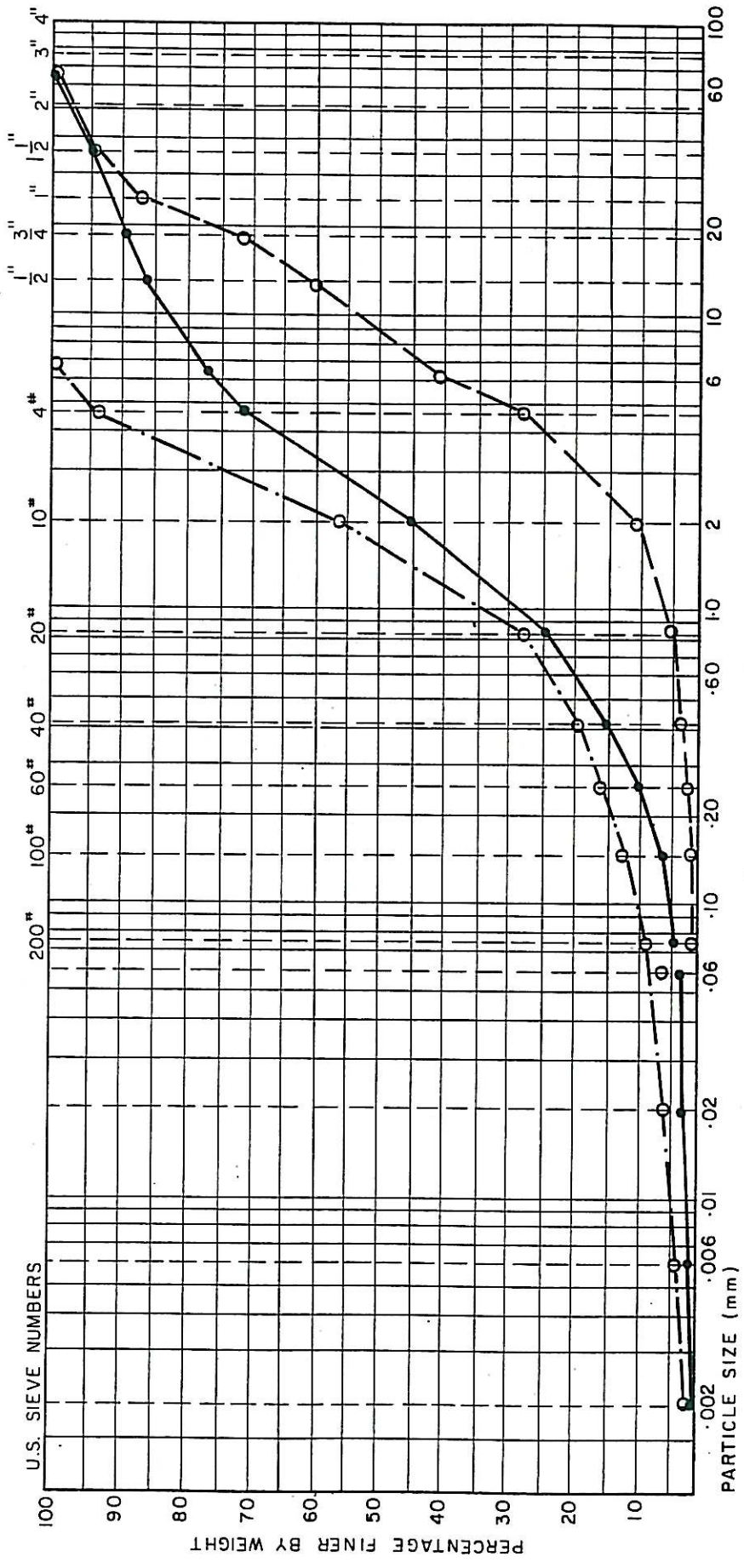


FIGURE 3.50  
Particle-size distribution of material in the Morgenzon stabilization experiment

CLAY FRACTION	SAND FRACTION			GRAVEL FRACTION		
	FINE SILT	MEDIUM SILT	COARSE SILT	FINE SAND	MEDIUM SAND	COARSE SAND

PARTICLE SIZE DISTRIBUTION

NITRR

DATE: 18-4-71

SAMPLE No.	RR 1270	RR 1282	RR 1375	PROJECT	4561
SAMPLE LOCATION	MORGENZON (SECTION 4)				
DEPTH	BASE COURSE	SUBBASE	SSG	JOB No.	361
CLASSIFICATION					

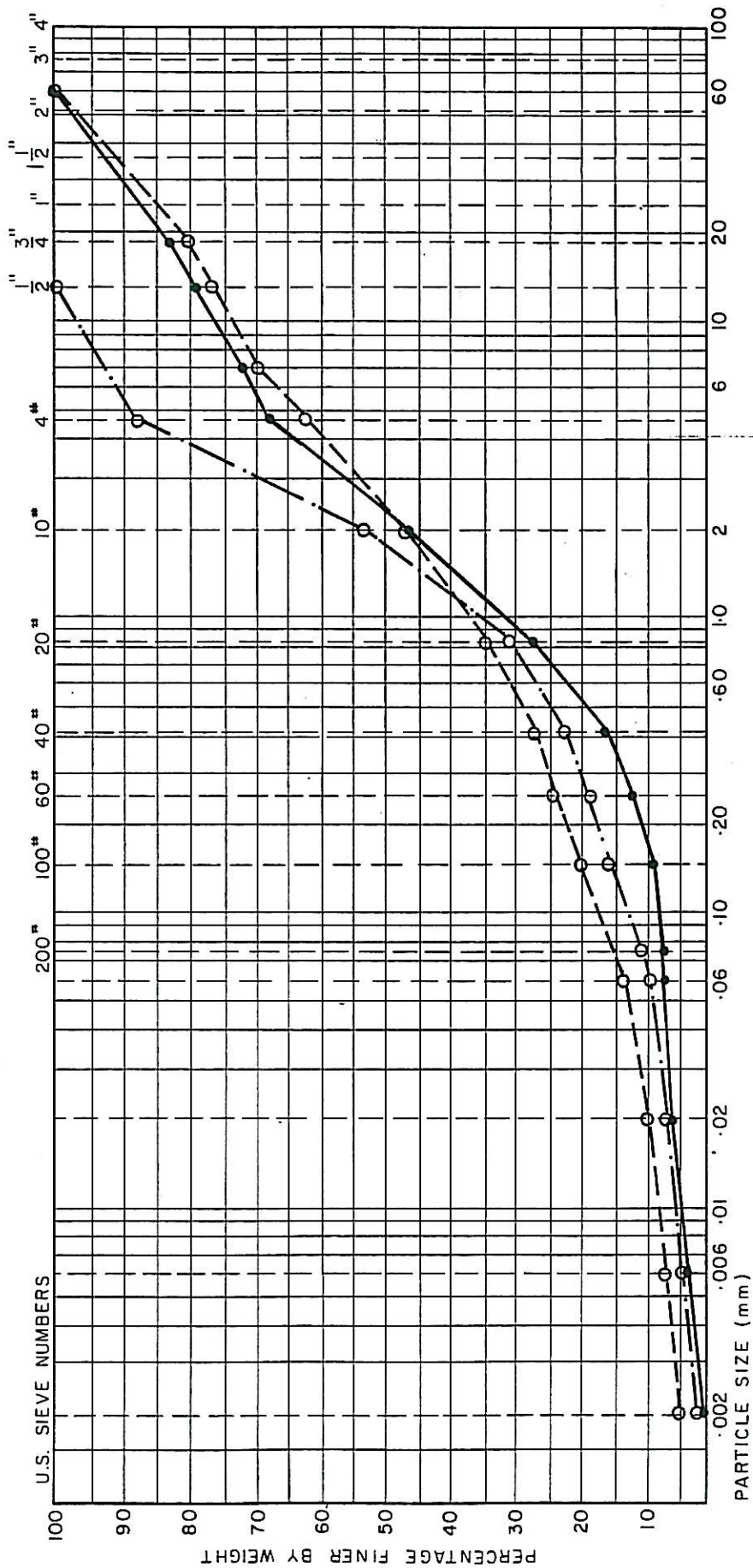


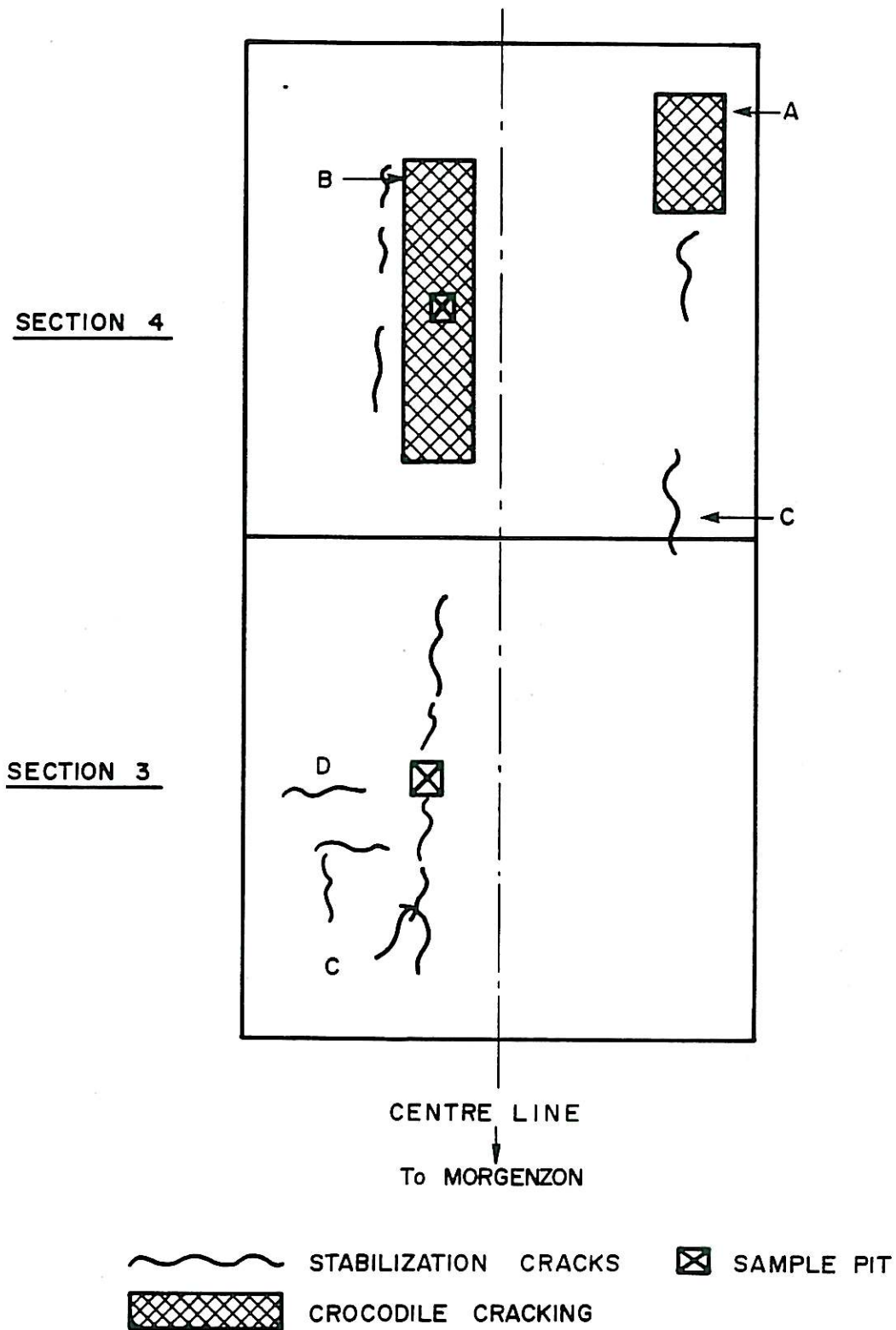
FIGURE 3.51

Particle - size distribution of material used in the Morgenzon stabilization experiment.

CLAY FRAC- TION	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SILT FRACTION			SAND FRACTION			GRAVEL FRACTION		

TABLE 3.25  
Traffic counts on P48-2

Year of counting	Means of counting	Period (hours)	Total traffic (VPD)	Heavy vehicles %
1968 (Sept)	?	24	150	14,4
1977 (Aug)	Instrument	24	337	?
	Instrument	24	485	?
1980 (Aug)	Instrument	24	478	17,3



CENTRE LINE  
 ↓  
 To MORGENZON




 STABILIZATION CRACKS       SAMPLE PIT  
 CROCODILE CRACKING

FIGURE 3.52

*Sketch of section 3 and 4, Morgenzon experiment, showing where distress occurred.*



- (a) Crocodile and distinct longitudinal cracks over an area of 10 % and with a spacing of 300 mm.
- (b) Open crocodile cracks of more than 3 mm width over more than 10 % but less than 90 % of the area with a spacing of 150 mm. A few longitudinal cracks were also observed.
- (c) Open longitudinal cracks with more than 3 mm width over an area of less than 10 %.
- (d) As in (c).

The cracking occurred generally in wheeltrack positions. The crocodile cracks in distressed area (b) penetrated the subbase. The longitudinal crack (c) penetrated the base.

Samples were taken from the places indicated. Test results are given in Table 3.22. It was concluded that the distress in section 3 was due to over-stabilization and that this section need not be rebuilt. The unstabilized section 4 showed, however, such a degree of distress that, after sampling, reconstruction was thought necessary. This took place in 1972. The material used for reconstruction was not from borrow pit No 7 and the new section was no longer considered as portion of the experiment.

#### 3.7.4 PRELIMINARY CONCLUSIONS

It was concluded from the experiment that weathered dolerite may safely be used as a construction material if stabilized with lime or cement to correct the PI from 11-19 % to 1,6 % to non-plastic, even if the PI were specified to be approximately 7 %.

It was further concluded that the addition of lime or cement arrests further deterioration of weathered dolerite for the design life of a road

(13 years in the case of Morgenzon), provided sufficient lime or cement is been added. It appears, however, that cement stabilization is prone to severe cracking whereas lime-stabilization is not.

10.1  
///

### 3.7.5 REVIEW OF STABILIZATION EXPERIMENT (1982)

The experimental site near Morgenzon carried its traffic without excessive maintenance and it was decided in the early eighties to review the experiment, which had been temporarily suspended in 1971, in order to arrive at conclusions regarding the long-term performance of dolerite. Of particular interest were, in the first instance, the formation of early stabilization cracks without apparent subsequent deterioration of the road in cement stabilized material and secondly the advantages or disadvantages of both cement-stabilized and lime-stabilized dolerites.

#### 3.7.5.1 Test programme

A great number of parameters were studied of which, however, only the DCP strength probe was of direct importance.

#### 3.7.5.2 Strength probe

The strength was determined by means of the Dynamic Cone Penetrometer/CBR relationship (Kleyn, 1975). The results are given in Table 3.26, where it may be seen that the CBR of the base was considerably lower in the wheeltracks than it was in the centreline positions. This effect does not apply to the subbase or selected subgrade. The strengths vary from 1 200 % to 2 400 %, with one exceptionally low value of only 540 %.

It is believed that this variation is due to the shortness of the test section, which is 60 m. The shortest test length now recommended is approximately 100 m (Netterberg, 1980), since a contamination zone of 20-30 m at the start and end of each section must be considered as a result of material displacement when road machinery is used.

The cement-stabilized section 3 produced the highest overall strength compared with the lime-stabilized sections 1 and 2 (Table 3.26).

It is concluded from these measurements that cement-stabilized dolerite is superior to lime-stabilized dolerite as far as its residual strength is concerned.

### 3.7.5.3 Stabilization reactions and cracks.

The cracks in section 3 (Figure 3.52) - believed to be cement-stabilization cracks in 1970 - had still not penetrated the lime-stabilized sections. The mechanism responsible is probably a two-phase reaction of cement such as:

- (1) the hydration and hardening of cement releasing  $\text{Ca(OH)}_2$  chemically (approximately 20 % of the cement additive), and
- (2) the reaction of this chemically released lime with the soil, causing shrinkage.

Cementation may also be expected in the lime-stabilized, hydraulically reacting soil (sections 1 and 2). However, the lime released during such cement reaction phase cannot be more than a fraction of the quantity of lime added. No delayed reaction phase is expected since the chemically released lime would not significantly alter the nature of the soil lime mixture.

A situation leading to cracking might, however, be introduced by rapid surface carbonation before the shrinkage is completed. In this case dissolved  $\text{Ca(OH)}_2$  may be carried to the carbonated surface, where it induces renewed shrinkage of the still lime reactive portion of the soil (superficial crocodile cracking). It was further found that lime-stabilized black clay decreased plasticity but showed an increase again of 30 % after 2 months (Clauss, 1982). This may also lead to superficial crocodile cracking.

TABLE 3.26  
 CBR of road layers of Morgenzon experiment as determined by the  
 60° Dynamic Cone Penetrometer test (1982).

Section	Position	CBR		
		Base	Subbase	SS grade
1/4	CL <sup>1</sup>	1 580	120	170
1/1	CL	540	205	39
1/3	OWT <sup>2</sup>	220	120	33
1/2	OWT	90	155	39
2/3	CL	2 400	-	-
2/2	CL	1 200	37	33
2/4	OWT	220	48	33
2/1	OWT	78	90	33
3/4	CL	1 650	785	39
3/3	CL	1 650	290	27
3/3	OWT	510	510	48
3/2	OWT	400	135	33

<sup>1</sup>Centreline position

<sup>2</sup>Outer wheeltrack position

Two cases are on record where the lime stabilizer and soil reacted in two distinct phases as follows:

- (a) Stabilization cracks in the Pietersburg goods yard. These were explained at the time as delayed reactions of two differently reacting clay types in the soil (K Davidson, pers comm, 1983).
- (b) The rapid reaction of soil compounds indicated by lime-rich amorphous matter (XRD) followed by a secondary reaction which is caused by the reaction product of the first reaction, ie cement hydration and hardening.

These considerations sound rather theoretical, but they may provide the means of achieving strong, cement-stabilized bases without cracks if lime is added in a quantity of at least 20 % of the quantity of cement added. This quantity is specified because 20 % of the quantity of cement (OPC) is released as  $\text{Ca(OH)}_2$  during the hydration and hardening process of cement and consequently becomes available for soil reactions. 1/18

### 3.7.6 CONCLUSIONS

If stabilized with 5 % lime or 5 % PBFC, weathered to badly weathered dolerite (PI = 11-19) was successfully used as basecourse material for 14 years. Similar material, unstabilized, failed after 3 years of service.

Cement-stabilized weathered dolerite developed stabilization cracks; lime-stabilized weathered dolerite did not. It is believed that these stabilization cracks are caused by a delayed two-phase reaction between cement and soil.

The residual 14-year strength of cement-stabilized dolerite is superior to that of lime-stabilized dolerite.

#### 4 GLOSSARY

(The definitions of this section are by various authors and not all of the terms may have been used in the text.)

**ACID SOIL:** A soil having a pH value of less than 7,0.

**AGGREGATE:** Broken stone and the like, ranging in size generally from 215 mm to 50 mm.

**AGGREGATE CRUSHING VALUE:** A test result to compare the strength of aggregates.

**ALKALINE SOIL:** A soil having a pH value greater than 7,0.

**AMORPHOUS SOIL MATTER:** A soil compound that is not crystallized.

**APPARENT DENSITY:** The same as bulk specific gravity; the ratio of the weight of a given volume of dry soil to the weight of the same volume of water.

**ATTERBERG LIMITS:** These are soil properties which help to identify a given soil in terms of its water retentivity and plasticity. They are liquid limit (LL), plastic limit (PL) and plasticity index (PI). The test method was proposed by Atterberg.

**AVAILABLE LIME:** Free  $\text{Ca(OH)}_2$  plus chemically released lime. Cement (OPC) contains approximately 20 % available lime.

**BALLAST:** Stone or gravel mixtures of irregular unscreened sizes which may also contain smaller material and sand.

**BASE:** That part of the pavement resting upon and through which the load is transmitted to, in turn the subbase, the subgrade and supporting soil.

**BINDER:** Any soil-cementing agent such as clay, humus, cement, lime, bitumen, and certain synthetic resins.

**BITUMEN EMULSION:** A colloidal suspension of bitumen in water.

**BLACK COTTON SOIL:** A brown or black clay soil in which volume changes due to swelling or shrinkage are particularly marked.

**BORROW PIT:** A small site next to the road from which construction material is borrowed for use in the road.

**BOULDER CLAY:** A deposit of unstratified clay or sandy clay of glacial origin containing subangular stones of various sizes scattered irregularly throughout its mass. The stones are not necessarily all of "boulder" size.

**BRICKEARTH:** A soil containing clay, silt and sand usually of a buff or biscuit colour, homogeneous, without any structure, and suitable for brickmaking. Found mainly in the Thames estuary and SE England.

**BULK DENSITY:** The weight of a material (including solid particles and any contained water) per unit volume, including voids.

**CALCIUM CARBONATE:**  $\text{CaCO}_3$ .

**CALCIUM HYDROXIDE:**  $\text{Ca(OH)}_2$ .

**CALCIUM OXIDE:**  $\text{CaO}$ .

**CALCIUM REACTION LIME CONTENT:** See CLC.

**CARBONATED LIME:**  $\text{CaCO}_3$  which forms from  $\text{Ca(OH)}_2$  in the presence of air and moisture (see also Quick lime). So far, four different types of carbonation are known:

- (a) Strength-producing carbonation in a humid atmosphere (beneficial).
- (b) Cement carbonation from cement hardening: pH > 11 (beneficial).
- (c) Hardening failure carbonation of cement: pH < 11 (malignant).
- (d) Disintegrating carbonation in a dry atmosphere (malignant).

**CARBONATION STRENGTH:** Soil strength produced by the carbonation of lime ( $\text{Ca(OH)}_2$ ) and subsequent cementation of soils by  $\text{CaCO}_3$  in lime-stabilized soils.

**CALIFORNIA BEARING RATIO:** A test result for the load-carrying capacity of soils, expressed as a percentage of a standard material.

**CBR:** See California Bearing Ratio.

**CEMENT:** Portland cement, Portland blast-furnace cement (PBFC) or a mixture of Portland cement and slagment etc.

**CEMENTATION:** The process during which calcium-silicate-hydrates form, binding the particles of the soil together and hence increasing the strength of the soil.

**CEMENTATION STRENGTH:** The strength of materials which is derived from the cementation of particles. The cementation may have been caused by different means, eg cement, lime, iron oxide etc.

**CEMENT-BOUND MATERIAL:** A term used in the United Kingdom for one of a group of three cement-treated materials, viz lean concrete, soil-cement and cement-bound materials.

**CEMENTED MATERIAL:** A soil treated with any of the stabilizing agents mentioned below and in which cementation has developed.



**CEMENT REACTION LIME CONTENT (CLC):** That quantity of lime at which soil cement reactions proceed. The determination is made on 200 g of whole grading (-19 mm dia material) after 1 hour at slightly above saturation moisture content.

**CEMENT REACTION pH:** The pH of a soil-lime-water mix at which cement minerals, CSH in particular, may form (possibly  $\text{pH} > 11,0$ ).

**CEMENT-STABILIZED/BITUMEN-STABILIZED/TAR-STABILIZED/CHEMICALLY**

**STABILIZED SOIL:** Soil in which stabilization has been assisted by the addition, respectively, of cement, bitumen, tar or chemicals.

**CEMENT-TREATED MATERIAL:** An intimate mixture of pulverized soil, cement and water.

**CHEMICALLY RELEASED LIME:**  $\text{Ca}(\text{OH})_2$  which is formed during a chemical process such as the hardening of cement. It may react as free lime. According to Fulton (1969), cement (OPC) produces some 20 % of chemically released lime.

**CLAY:** Fine-grained, natural, earthy, argillaceous material. Often arbitrarily classified by engineers as material having a particle size of less than 0,005 mm. Clays consist largely of complex silicates and are formed by the decomposition of igneous rocks.

**CLAY FRACTION:** That fraction of a soil composed of particles smaller in size than 0,002 mm.

**CLAY MINERALS:** Common minerals found in most soils.

**CLC:** See Cement reaction lime content (CLC).

The cement reaction lime content is the quantity of lime ( $\text{Ca}(\text{OH})_2$ ) that produces a pH at which cement minerals - CSH in particular - may form, if formation is possible, in a

soil-lime-water mixture after 1 hour. This pH is possibly > 11,0.

**COBBLES:** Stones between 60 and 200 mm in size.

**COEFFICIENT OF UNIFORMITY:** A term indicating the grading of a material. It is the ratio of the sieve size through which 60 % of the material passes to the sieve size through which 10 % passes.

**COMPACTION:** The process by which the soil particles are constrained, by rolling or other means, to pack more closely together, thereby increasing the dry density of the soil.

**CONSOLIDATION:** The process which soil particles are packed more closely together by the application of continued pressure over a period of time, eg an embankment under its own weight or the soil under a building.

**COURSE:** Prepared material placed to form a continuous layer of the pavement.

**CUT-BACK BITUMEN:** Bitumen which has been rendered fluid at atmospheric temperature by the addition of a suitable diluent such as white spirit, kerosene or creosote.

**DEPOSITS OF MATERIALS:** See Material deposits.

**DRAINAGE:** Natural or artificial means for the removal of water from the surface or subsoil of an area, usually by means of gravitation.

**DRY DENSITY:** The mass of the dry material, after drying to constant weight at 105 °C (221 °F), contained in a unit volume of moist material.

**DRY DENSITY/MOISTURE CONTENT RELATIONSHIP:** The relationship between dry density and the moisture content of a soil when a given amount of compaction is applied.

**EQUILIBRIUM MOISTURE CONTENT:** The moisture content at any point in a soil after moisture movements have ceased.

**FILL:** Excavated soil, rock or refuse when dumped for the purpose of filling a depression or raising a site above the natural surface level of the ground.

**FLEXIBLE PAVEMENT:** See Pavement, flexible.

**FORMATION:** The surface of the ground in its final shape after completion of the earthworks, and of consolidation, compaction or stabilization in situ.

**FREE LIME:** The  $\text{Ca(OH)}_2$  content of eg road-building lime. Cement contains 1-2 % free lime.

**GRADE:** The design surface of the in-place or fill material acting as a foundation upon which the pavement is constructed.

**GRADING:** See Particle-size distribution (grading).

**GRAVEL:** Rounded or water-worn stones of irregular shape and size that occur in natural deposits with or without some finer material.

**GRAVEL FRACTION:** That fraction of a soil composed of particles between 2,0 and 60 mm in size.

**GRAVEL ICL:** ICL made on whole-gradings.

**GRAVITATIONAL WATER:** The water which moves downwards under the action of gravity, from the soil surface to the water table.

**GROUND WATER:** The water contained in soil below the water table.

**GROUP INDEX:** An empirical value devised to measure a soil's usefulness for road-building purposes and based on the ratio of specified particle-size ranges.

**HARDPAN:** A horizon of accumulation that has been thoroughly cemented to an indurated, rock-like layer that will not soften when wet.

**HELD WATER:** The water retained in the soil structure above the water table by surface tension and adsorption forces.

**ICL (INITIAL CONSUMPTION OF LIME):** The lime-saturation pH of a soil-lime-water mix after 1 hour at the relevant recorded temperature, ie the quantity of lime ( $\text{Ca(OH)}_2$ ) necessary to maintain a lime-created pH of 12,4 in a lime-soil-water mix at 25 °C after 1 hour.

**INCIDATOR ICL:** An ICL test made on soil fines (-0,425 mm) only.

**LATERITIC SOIL:** Tropical soil in which the weathering processes have resulted in an accumulation of sesquioxides particularly of iron.

**LEACHING:** The process by which the soluble material in soil is removed by the percolation of water.

**LEACHING:** The process by which the soluble material in soil is removed by the percolation of water.

**LIME:** A general term for the product yielded by the calcination of limestone or dolomite or mixtures of these, with or without subsequent hydration. Quicklime, 'hot' lime, calcium oxide ( $\text{CaO}$ ), and unslaked lime refer to the calcined product, while slaked

lime, hydrated lime, calcium hydroxide ( $\text{Ca(OH)}_2$ ), air-separated lime ('air sep') and hydrate are used for the product resulting from the combination of quick lime and water.

The term is often used with reference to any of the quoted or other meanings.

**LIME IN CHEMICAL ANALYSES:** This is generally expressed as CaO. This calcium is usually not all available for reaction. Cement analyses mention some 63-68 % CaO for OPC, of which 20 % changes into chemically released (available) lime during hardening, while 1-2 % consists of free lime.

**LIME SATURATION (OF SOIL):** This is achieved by a quantity of lime added to the soil which produces a pH that does not increase further if more lime is added.

**LIME-TREATED MATERIALS:** An intimate mixture or pulverized soil, lime and water.

**LIQUID LIMIT (LL):** The moisture content, expressed as a percentage of the dry mass of soil, at which the soil passes from the plastic to the liquid state under the test conditions for determining this value.

**LOW-COST ROAD-CONSTRUCTION MATERIALS:** These are found and used locally, ie near the construction site and without large transportation expenditure attached to it.

**MATERIAL DEPOSITS:** Sites at which soil and gravel with technical properties suitable for use in road construction are found. Natural accumulations of road-construction materials. Their extent varies but is usually 100 m x 100 m x 2 m deep.

**MAXIMUM DRY DENSITY(MDD):** The dry density of a soil obtained by a specified amount of compaction at the optimum moisture content.

**MAXIMUM pH:** See Saturation pH.

**MDD:** See Maximum dry density.

**MECHANICALLY STABILIZED SOIL:** Soil to which imported soil or aggregate has been added to obtain a desired particle-size distribution or plasticity and which has been compacted to a desired density.

**MODIFICATION:** The process of improving the physical properties of the soil, eg the plasticity and grading.

**MOD AASHTO:** Compaction effort as specified by the American Association of State Highway and Transportation Officials and subsequently modified.

**MOISTURE CONTENT:** The loss in mass, expressed as a percentage of the dry material, when a soil is dried to constant mass at 105 °C (221 °F).

**NATURAL ROAD-CONSTRUCTION MATERIAL** is found in nature and has technical properties which deviate not too far from those specified as suitable. It may often be used in road construction, as found or may require improvement by stabilization.

**NON-COHESIVE SOIL:** Soil consisting of the coarser products of rock weathering in which the cohesive bonds mainly associated with the smaller fractions are largely absent.

**OMC:** See Optimum moisture content.

OPTIMUM MOISTURE CONTENT (OMC): That moisture content at which a specified amount of compaction will produce the maximum dry density.

PARTICLE-SIZE DISTRIBUTION (GRADING): The percentages of the various grain sizes present in a soil as determined by sieving, sedimentation or other means.

PAVEMENT: The whole of the artificial load-carrying structural unit built over the subgrade and consisting, in general, of subbase, base and surface courses.

PAVEMENT, FLEXIBLE: A pavement of inconsiderable flexural rigidity or tensile strength.

PAVEMENT, RIGID: A pavement developing considerable local flexural rigidity by reason of the 'tensile' strength of one or more of its courses.

PFA: See Pulverized fuel ash.

pH: The pH of a soil is the negative logarithm of the hydrogen-ion concentration in an aqueous suspension of the soil.

PI: See Plasticity index.

PL: See Plastic limit.

PLASTICITY INDEX (PI): The numerical difference between the liquid limit and the plastic limit of a soil ( $LL - PL = PI$ ).

PLASTIC LIMIT (PL): The moisture content, expressed as a percentage of the dry soil mass, at which the soil passes from the semi-solid to the plastic state under the test conditions for determining this value.

**PLASTIC SOIL:** A soil which, when containing the appropriate amount of moisture, can be rolled on the palm of the hand into a thread of less than about 3 mm dia.

**PORE WATER PRESSURE:** The pressure of the water in the voids of a saturated soil.

**POROSITY (POROSITY RATIO):** The ratio of the volume of voids to the total volume of a material including voids.

**POZZOLAN:** A siliceous or siliceous and aluminous material which in itself possesses little or no cementitious value, but which will in finely divided form and in the presence of moisture react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties. Clays are pozzolanic in varying degrees.

**PULVERIZED FUEL ASH (PFA):** Ash powder of ground coal.

**QUICK LIME:** Also known as burned lime (CaO). It reacts with water and changes into  $\text{Ca(OH)}_2$  (free lime); 56 parts of CaO are equivalent to 74 parts of  $\text{Ca(OH)}_2$  by mass (Ballantine and Rossouw, 1972). On hydration, the volume of quick lime increases by a factor of 1,92 (La Technique Routiere, 1982, No. 1, p 53). The molecular mass of  $\text{CaCO}_3$  is 100,8.

**RELATIVE COMPACTION:** The percentage ratio of the dry density of the soil in situ to the maximum dry density of that soil as determined by the standard compaction test.

**RIGID PAVEMENT:** See Pavement, rigid.

**SAND:** Small mineral particles from natural sources usually regarded as being of such a size that all will pass a 2,5 mm (No 7 BS) sieve. They are free from appreciable amounts of clay and silt and are normally sized between 2,0 and 0,06 mm.



**SAND FRACTION:** In the particle-size analysis of soils, mineral particles between the sizes 2,0 and 0,06 mm.

**SATURATED SOIL:** A soil in which the voids are entirely filled with water.

**SATURATION LIME CONTENT:** See Lime saturation

**SATURATION pH:** Is a pH that does not increase further when more lime is added.

**SETTLEMENT:** The downward movement of a soil or of the structure which it supports, resulting from a reduction in the voids in the underlying strata.

**SILT:** Mineral particles naturally deposited as sediment in water and usually regarded as of such a size that all will pass a 200 BS sieve. The particles are free from appreciable amounts of clay.

**SILT FRACTION:** In the particle-size analysis of soils, mineral particles between the sizes 0,060 and 0,002 mm.

**SITE PREDICTION:** Statistically highest occurrence of material deposits according to material assessment tables for different lithostratigraphical units.

**SLAGMENT:** Trade name of ground blast-furnace slag.

**SLAKED LIME:**  $\text{Ca(OH)}_2$ ; quick lime to which water had been added.

**SOIL:** Any combination of gravel, sand, silt and clay. Also included are crushed or uncrushed sand and gravel, crushed stone and other materials such as poorly graded or waste products from stone quarries. It usually consists of naturally occurring rock decomposition products that contain all particle sizes below

about 75 mm down to single micron sizes and less, but it includes anything that can be used for road-construction purposes such as junk, ash, refuse, etc.

**SOIL-CEMENT:** A high-quality cement-treated soil which complies with the wetting/drying and freeze/thaw criteria specified by the Portland Cement Association (PCA).

**SOIL CEMENT REACTIONS:** Soil strength produced by cement reactions, ie the pozzolanic/hydraulic reaction of lime and soil in lime- or cementstabilized soil. Very often combined with carbonation effects of the lime additive/compound.

**SOIL MODIFICATION:** Plasticity changes as caused by lime (or cement) additives to soil.

**SOIL PROFILE:** A vertical section showing the soil strata at a given site.

**STABILIZATION:** The treatment of soil in such a manner as to render its properties less affected by water or to increase its load-bearing capacity.

**STABILIZATION MECHANISM:** Reactions between lime/soil/gravel and water and classified into soil-modification, lime-carbonation and soil-cement reaction.

**STABILIZED MATERIAL:** A term used in Australia, New Zealand, South Africa and parts of the United States which is synonymous with treated material (qv).

**STABILIZING AGENT (OR STABILIZER):** The material used to treat a soil, eg lime, Portland cement, Portland Blast Furnace cement (PBFC), a mixture of either Portland cement or lime with ground granulated blast-furnace slag (eg 'slagment') etc. Other pozzolanic materials (eg fly-ash) may also be used in such a mixture.

**STABILIZED SOIL:** Soil treated in such a manner as to render its properties less affected by water or to increase its load-bearing capacity.

**STRATIGRAPHY:** Description of the sequence of layers.

**SUBBASE:** A layer of material placed between the subgrade and the base, often for a special purpose, such as drainage or support for the construction.

**SUBGRADE:** The natural foundation or the fill which directly receives the loads from the pavement.

**SUBSOIL:** The undisturbed strata lying immediately below the topsoil.

**SURFACING:** The course or courses above the base laid in the form of a continuous layer or layers to provide a wearing surface, to protect the base or to add strength to the pavement.

**TOPSOIL:** The loose top layer of soil that can support vegetation.

**TREATMENT:** A general term for the process of improving the engineering properties of a soil through either modification or cementation.

**TREATED MATERIAL:** An inclusive term for an intimate mixture of pulverized soil, stabilizing agent and water.

**UCS:** See Unconfined compressive strength.

**UNCONFINED COMPRESSIVE STRENGTH (UCS):** The load in kPa required to crush a cylindrical specimen  $\pm$  100 mm high and  $\pm$  50 mm dia to total failure at a rate of application of the load of 140 kPa/sec.

**VOIDS:** The spaces in a material occupied by water or air or both water and air.

VOIDS RATIO: The ratio of the volume of voids to the volume of solids in a material.

WATER-TABLE: The horizon in soil at which the pore water is at atmospheric pressure.

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## 6 INDEX

- Acid soil, 114
- Active lime, 35
- Aggregate soundness, 25
- Amelioration, 27
- Amorphous, 64, 71
  - alumina, 57
  - glass phase, 143
  - matter phase (hump), 105
  - silica, 54
  - silica content, 54
- Anion exchange, 65, 66, 72, 73
- Aspects of particle size of lime, 43
- Atterberg limits, 22
- Available lime, 35
  
- Balanced materials system, 21-25
- Black clay, 69-74
- Blast-furnace slag, 101
- Borrow pit, 10
- Burned lime, 35
  
- Calcium
  - aluminates, 145
  - carbonate, 35, 36
  - hydroxide, 35, 36
  - oxide, 35, 36
  - silicates, 142
  - type lime, 35
- Carbonation, 53, 131
  - cementing, 136
  - of lime, 119
  - strength, 137-141, 146, 150
- Cation exchange, 65, 66, 72, 73

- Cement, 95-111
  - and lime application for stabilization, 20
  - and pozzolanity of soils, 95-105
  - Portland, 96
  - pozzolanic mortars, 96
  - slagment, 101
  - trass, 101
- Cementation, 95-111
- Cement-cementing, 136
- Cement reaction, 141
  - in black clay, 131
  - lime content (CLC), 36, 141
  - of soil, 119
  - pH, 100
  - strength, 148, 151
  - 2nd, 3rd generation, 142-145
  - treated soil, 122
- Cement stabilization, 22, 149
  - stabilization mechanism, 149
  - CSH carbonation, 149
  - CSH modification, 149
  - hydraulic (CSH) cementation, 149
- Chemically released lime, 101
- Chemical modification, 119, 122
- Chemical reactions, 28, 36
- Chemical stabilization mechanism, 149
- Clay, 29, 57, 60
  - chemical properties, 60
  - fraction, 27, 29
  - in gravel, 74
  - kaolinite type soil, 60, 61
  - lumps, 89
  - montmorillonite type soil, 68
  - reaction strength, 148, 151
  - reactivity of lime, 46

CLC - see Cement reaction lime content, 36

Compaction,

    delay, 129

    strength (uncured), 119, 121, 146, 151

Delta liquid limit, 122

Deposits of materials, 3-18

Dicalcium silicate, 96

Dolomite-type (dolomitic) lime, 35

Durability, 25

Economic aspects of lime, 48, 53

Efficiency of stabilizer, 100

Electrode, 158

    failure, 160

    glass, 158

    metal, 158

Fines ICL test, 150-154

Glass phase 64, 71, 108

    lime-poor, 64

    lime-rich, 71

    natural amorphous, 108

Grading, 22, 52

    ICL test, 121

Grading correction, 121

Gravel ICL test, 154

Grinding, 84

    ICL test, 154

Guide for testing in stabilization, 148

Gypsum, 97

Hydration and volume change (lime), 44, 53

Hydraulicity, 103

Hydraulic lime/clay reaction, 141

Si, Al strength, 68

soil/lime reactions, 141-145

ICL - see Initial consumption of lime,

ICL test, 154-162

fines fraction, 77, 80

grading, 80

gravel ICL, 154

grinding, 84

indicator ICL, 87, 88

material type, 77

moisture content, 78

ongoing soil/lime reactions, 86

pH drift, 86

quantity of sample, 79

safety factor, 87, 88, 142

sucrose ICL, 165

weathering, 86

whole-grading, 80

Impurities, organic-matter, 110-112, 153, 162

coal and plant matter, 112

sugar-type, 111

Indicator ICL, 77, 80

Inert material, 26

Initial consumption of lime (ICL), 36, 152

Kaolinite type clay, 61-68

Land forms, 4, 9

classification, 7, 9

dimension of size, 8

## Lime, 35-53

- active, 38
  - aspects of particle size, 43
  - available, 38
  - burned, 35
  - calcitic, 35
  - carbonation, 47
  - dissociation, 36
  - dolomite-type (dolomitic), 35, 51
  - economic aspects, 48
  - furnace temperature, 35
  - hydrated, 35
  - hydration and volume change in, 44
  - magnesia-type, 35
  - non-reactive soil, 29
  - particle, 44
  - particle-size distribution in, 41, 43
  - pH, 37
  - plasticity, 41
  - pricing, 48
  - quality, 150
  - quick, 35
  - reactive soil, 29, 45
  - road, 35
  - saturation, 47
  - slaked, 35
  - stabilization, 53
  - technical properties 38, 41
  - unslaked, 35
- Lime/clay reaction, 146-148
- Si,Al strength, 147
  - soil/lime reactions, 149
- Lime non-reactive soils, 29
- Lime-poor glass phase, 64
- Lime-reactive soils, 29
- Lime-rich glass phase, 71, 143

- Limestone, 35
- Lime-treated material, 22
- Liquid limit (LL), 41
- LL - see Liquid limit
- Long-term reactions, 27
- Low-cost road construction material, 1
- Low density in stabilized soil, 127
  
- Magnesia, 98
- Magnesia-type lime, 35
- Materials, 1
  - assessment, 11
    - tables, 18
  - classification, 4
  - deposits, 4
  - description of deposits, 4
  - lithostratigraphy, 11, 12
  - stratigraphic units, 11
  - substandard, 23
- Microstructure of lime particle, 46
- Modification, 121-134
  - processes (amelioration), 121
  - soil stabilization versus modification, 127
  - strength, 151
- Moisture content,
  - ICL test, 154
- Montmorillonite-type clay, 68-74
  
- Non-reactive soil, 151
  
- Occurrence of materials, 4-10
- OMC - see Optimum moisture content,
- Optimum moisture content (OMC), 48

- Organic matter impurities, 110-112, 153, 162
  - coal and plant matter, 112
  - gravel ICL, 165
  - sugar type, 111
  
- Particle charge modulus of soil, 57
- Permanence of stabilization product, 26
- pH, 37
  - control, 99
  - determination, 154
  - drift, 86, 87
  - electrode, 158
  - extrapolation, 77
  - instrument, 158
  - lime, 37
  - maximum, 37
  - meter, 158
- Physical aspects of soil, 28, 88
- Physico-chemical changes in soil, 122
- Plasticity, 93
  - correction, 150
    - of lime, 52
    - of soil,
- Portland cement, 96
  - hardening, 98
  - hardening failure, 99
  - hydration, 98
- Possibly reactive compounds in stabilized soil, 28
- Potassium oxide, 97
- Pozzolanity, 102-105
- Pozzolans, 95, 105
- Pozzolanic/hydraulic soil/lime reaction strength, 150
- Pricing of lime, 48
- Primary stabilization reaction, 135

- Pulverized fuel ash (PFA), 103
  - strength development of, 103
  
- Quality of lime, 150
- Quality of stabilizer phase, 25
- Quick lime, 35
  
- Reactions, 27
  - long-term, 27
  - pattern, 33
  - rapid, 27
  - short-term, 27
- Reaction time of soil/lime, 26, 87
- Reactive soil, 28
- Reactive glass phase, 103-111
- Release of lime (chemical), 101
- Resolution power of soil-mechanical classification systems, 28
- Road-construction material, 1
  - definition, 1
  - description, 1
  - technical properties (specified), 5
- Road lime, 35
  
- Safety factor in stabilization (ICL test), 101
- Sample quantity of ICL test, 79
- Saturation lime content, 79
- Seasoning, 129
- Secondary stabilisation reaction, 135
- Second phase (stabilization) reaction, 120
- Shape of aggregates, 22
- Short-term reactions, 27
- Silica content, total, 107
- Silverton soil, 61-68
- Site prediction, 10
- Slagment, 101
- Sodium oxide, 97



## Soil,

- cementation, 135-145
  - cement-reaction, 122
  - lime non-reactive, 29
  - lime-reactive, 29
  - mechanical classification systems, 4
  - modification (definition), 121
  - modification in time, 127
  - profile,
- Soil/Lime reaction phase, 27
- Soil mechanical phase, 21
- Soil stabilisation, 19-27, 136-146
- carbonation, 136-141
  - cementation, 136-
  - cement reaction, 141-146
  - modification, 121
  - quality of stabilisers, 25
  - soil/lime reaction phase, 27
  - soil mechanical phase, 21
  - soundness of aggregates and soils, 25
  - stabilizer phase, 25
  - time factor, 174
- Solubility of alumina, 57
- dolomite, 47, 74
  - lime, 47, 74
- Soluble salts, 114
- Sound material, 26
- Soundness, 26
- Soundness of aggregate and soils phase, 25
- Stabilization,, 1, 2, 19, 25-29, 98-100, 108-112, 119-122, 134-160
- carbonation, 149
  - chemical, 149
  - CSH carbonation, 149
  - CSH modification, 149
  - cement reaction, 149
  - extrapolation of 1 hour test results, 77

- failures, 99
- grading, 80
- grinding, 84
- guide for testing, 148
- hydraulic cementation, 149
- material types and ICL, 77
- mechanical, 22, 119, 149
- mechanism, 149
- moisture content, 78
- permanence, 174
- pH drift, 86
- potential of clays, 93
- potential of lime, 53
- primary reaction, 119
- quantity of sample, 79
- secondary reaction, 119-120
- strength; 146
- third generation reaction, 120
- Stabilizer efficiency, 101
- Stabilizer phase, 25
- Stabilizer/soil proportional reaction, 127
- Stage of weathering, 22
- Stoffbergite, 33
- Strength,
  - factors, 146-148
  - probe, 191
- Substandard materials, 25
- Sucrose ICL, 163
- Synthesis of strength factors, 146-148
  
- Technical properties of lime, 36
  - carbonation, 47
  - chemical composition, 52
  - economic aspects, 48
  - grading, 43
  - hydration, 44
  - plasticity, 41

- stabilization potential, 53
- strength aspects, 53
- Temperature of lime solution, 37
- Tetracalcium aluminoferrite, 97
- Third generation cementing reaction, 145
- Third generation stabilization reaction, 135, 145
- Time factor in soil stabilization, 174
- Total lime content, 104
- Total silica content, 104
- Tricalcium aluminate, 97
- Tricalcium silicate, 96
  
- Unslaked lime, 35
- Unsoundness, 25
  
- Volume change (lime), 44
  
- Weathering, 80
- Whole-grading, 86
- Workability, 22, 91
  
- Zeta potential, 57