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A laboratory investigation into the stabilization of natural soils using two waste materials

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A LABORATORY INVESTIGATION INTO THE STABILIZATION OF NATURAL SOILS USING TWO WASTE MATERIALS

Andrew John Barrett

A Dissertation Submitted to the Faculty of Engineering, University of the Witwatersrand, Johannesburg, in partial fulfillment for the Degree of Master of Science in Engineering.

Johannesburg, 1979.

DECLARATION

I, Andrew John Barrett hereby declare that this is my own unaided work and I have not submitted this dissertation to any other university for degree purposes.

A.J. Barrett.

ABSTRACT

Soil stabilization is the generic term for any process which has as its aim the upgrading or improvement of one or more soil properties. In South Africa, stabilization is usually effected using a combination of mechanical densification and the addition of a binder to the soil. Two waste materials, PFA (a power station waste) and phosphogypsum (a waste from the fertilizer industry) have been shown to be useful binders overseas. The purpose of this project was to investigate local samples of these waste materials to establish their potential as soil stabilizers.

A limited amount of research into the use of these two materials has been carried out in South Africa. The results of these programmes are discussed. Generally, however, none of these programmes were comprehensive enough to reach categoric conclusions regarding either binder.

The soils used in the investigation were taken from various locations in the Johannesburg Municipal area in which it is intended to construct surfaced roads for light traffic in the near future. The soils chosen cover a range of plasticity indices from 0 to 30, but unfortunately, were very similar mineralogically.

The phosphogypsum used was supplied by Triomf Fertilizer (Pty)

Limited. Two samples of PFA were used.
One of them was supplied by Darling and Hodgson, Limited, who are extracting a "selected" ash from Grootvlei power station, and intend to make it commercially available at some stage. The other ash was obtained from Kelvin power station.

Tests were done to establish both the short and long term effects of the binders on the soils. The short term effects were investigated by looking at changes in the plasticity and moisture density relationships for the soils immediately after the addition of the binders. The long term effects were investigated by looking at the changes in unconfined compressive strength of moulded soil samples with the addition of the binders. (Samples were moulded at 100% Mod AASHO density and optimum moisture content in a tapered cylindrical mould by static compaction). These effects were looked at from 3 different angles.

i) the effect of binder composition (i.e. different ratios of PFA/lime and PFA/cement)

ii) the effect of binder content

iii) the effect of age on strength gain

The results of the testing programme were not very encouraging. For the binder contents used, there were minimum short term effects for both binder types. The addition of phosphogypsum to the soils resulted in only very small strength increases. For the PFA binders, it appears that the lime content of many

of the PFA/lime binders fell in a range where only short term reactions could be expected. However, even where sufficient lime was present to satisfy the initial lime demand of the soil, the PFA appeared to act mainly as a diluent to the lime. Similar conclusions apply to the PFA/cement binders.

The general nature of PFA/lime stabilization was uncovered in this testing programme, but further tests will have conducted before conclusions regarding the quality of our local materials can be drawn.

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1 INTRODUCTION

1.1 Soil Stabilization

Soil stabilization is the generic term for any process which has as its aim the upgrading or improvement of a particular soil property, or soil properties. Stabilization may be accomplished in many ways, which include mechanical, chemical, thermal and other processes. The technique selected depends on both the purpose of the stabilization eg. modification of one or more of - volume stability, strength, permeability and durability, and the nature of the soil being stabilized. (Ingles and Metcalf [5])

Although soil stabilization is mainly associated with road construction, it is extensively used in other fields. These include slope protection, industrial and domestic foundations dust control and deep excavations. Research is currently in progress to examine the viability of stabilized earthfill dams (Robertson [29]) and this is undoubtedly only one of many further uses for stabilization.

1.2 Soil Stabilization in South Africa

In South Africa, stabilization is usually effected using a combination of mechanical densification and the addition of lime or portland cement or either of these in combination with blast furnace slag to the soil. (Other binders are also

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used but on a much smaller scale) The choice of which binder is used depends not only on soil type as mentioned above, but also on the availability and costs of the various binders, and in certain circumstances, peculiarities of the provincial road building authority. Details of methods used in the evaluation and selection of binders are discussed in texts such as Ingles and Metcalf[6].

1.3 Research Objectives

The idea of using waste materials in soil stabilization is not new. As stated above, blast furnace slag is already used extensively in South Africa. Other waste materials, of which sulphite lye and carbide sludge are two examples, have also been used locally. Pulverised fuel ash (more commonly known as flyash or PFA, a power station waste) has been used successfully in a number of countries, but although in excess of 6 million tonnes is produced per annum locally, no concerted effort has been made to use this material until recently.

The purpose of this research project is to investigate the possibility of using two local fly ashes, and a sample of phosphogypsum (a waste material from the fertilizer industry) as soil stabilizers.

1.4 Pulverised Fuel Ash

Pulverised fuel ash is a waste material produced by coal-fired power stations and is defined [7] as the "the finely

divided residue that results from the combustion of ground or powdered coal and is transported from the boiler by flue gases'.

In modern coal fired powerstations, the coal is finely milled and then blown by a jet of air into the combustion chamber of the boiler where it burns, forming an ash which consists of minute separate spherical particles, some of which are hollow. About 15-20% of this ash agglomerates and drops to the bottom of the combustion chamber. This ash is known as furnace bottom ash (FBA). The remaining ash is carried away in the flue gases, from which it is extracted by means electrostatic precipitators. (Nb. older installations still use mechanical precipitators) From this stage on, the ash is referred to as PFA. The FBA and PFA are then mixed with water and sluiced to dumps and dams.

The civil engineers' interest in PFA arises mainly from its pozzolanic properties. As the question of pozzolanic properties arises later in this chapter in the discussion on clay minerals, a short section will be devoted to this topic here.

1.4.1 Pozzolanic Materials

A pozzolan is defined[7] as 'a siliceous or alumino siliceous material that in itself possesses little or no cementitious value but that in finely divided form and in the presence of moisture will chemically react with alkali and alkaline earth hydroxides at ordinary temperatures to form or assist in

in forming compounds possessing cementitious properties[7]. Pozzolanic materials may be broadly classified as naturally or artificially occuring deposits and the various categories are summarised in fig. 1.1. The materials listed in this table may all be regarded as potential pozzolans. (i.e. some may require treatment, eg. calcination, before they show any response to lime.)

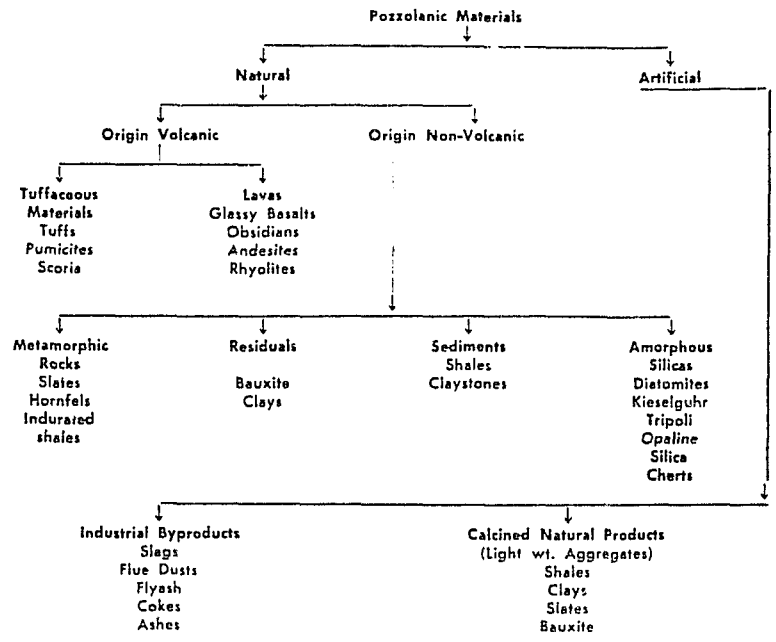


FIG 1.1 Classification of Pozzolans (after Croft[8])

The nature of the lime-pozzolan reaction has been studied by a number of researchers and details are given in references such as Croft [8,9].

1.4.2 PFA as an Engineering Material

The pozzolanic properties of PFA were first reported by Davis[28] in 1937 and utilization of PFA in construction in the USA began in the following decade. With the exception of South Africa, most industrialised countries have developed more and more uses for this waste material and now more than 50 million tonnes are used per annum on a world wide basis. Although many secondary uses for PFA exist (eg. as a filler for asphalt) its main uses are

1) In cement manufacture, both as an admixture and an additive.

2) As a part replacement of cement in mass concrete and concrete products.

3) In soil stabilization, usually in conjunction with cement or lime.

4) As a structural fill, sometimes with the addition of cement or lime.

1.4.3 PFA in South Africa

The use of PFA has been encouraged in foreign countries by high disposal costs (some being ten times the Escom figure[10]), environmentalists and anti-pollution organisations. In South Africa, very little motivation has been experienced

to find uses for this waste product. The main forms of ash disposal have been dumps and dams. The backfilling of worked out coal mines is one of the very few examples of local utilization of PFA.

Besides the reasons outlined above, the construction industry in South Africa has not exploited this cheap construction material because up to 1974, all research done on South African PFA indicated that the local product was inferior to its overseas equivalent. The building of new power stations (which use more modern equipment) and the updating of older ones has resulted in a higher quality of ash becoming available. A comprehensive research program initiated by Messrs. Safic Resources Utilization (Pty) Limited revealed that if ashes from the modern power stations could be classified,* a higher quality product could result.

*The flue gases pass from the boiler through a number of banks of precipitators. It was found that the coarser particles were removed in the initial banks and the third or final bank retained a finer, higher quality material. 'Classification' in the context used above refers to the selection of the finer ash.

Darling and Hodgson Limited have since installed an extraction plant at Grootvlei power station, with the object of making selected PFA commercially available. (nb. one of the ashes

tested in this program is a sample of the classified ash extracted at Grootvlei. The other ash comes from Kelvin power station and was removed from the outlet precipitator (i.e. 2 banks are used at this station, an inlet and an outlet bank) These ashes are compared in more detail in chapter 3)

1.5 Phosphogypsum

A typical phosphoric acid plant produces of the order of 50 to 60 thousand tons of wet phosphogypsum per month.[11] A total of approximately 2,5 million tonnes is produced in South Africa annually and this is currently disposed of in dumps and dams. The chemical analysis of this material has shown that it contains both hydrofluoric acid and phosphoric acid in small quantities in the pore fluid. Both these chemicals have been used successfully in soil stabilization. Initial investigations have shown phosphogypsum to be a potentially useful soil stabilizer.

1.5.1 Soil Stabilization with Phosphoric Acid

Phosphoric acid has been used for soil stabilization in America[6]. (mainly on an experimental basis). The quantities of acid required to effectively stabilize the soil were similar to normal cement and lime requirements (2-10%) As the cost of the acid is several times greater than that for cement or lime, this was not economical. The nature of the reaction of

phosphoric acid with the various clay minerals will be discussed later in this chapter.

1.5.2 Soil Stabilization with Hydrofluoric Acid

This chemical has not been used extensively as a binder because of its handling difficulties and high cost. Laboratory tests have shown, however, that it is an extremely effective stabilizer for all except the aluminium rich clays (kaolins and bauxites)

An investigation[12] into the effects of the addition of flouride (in the form of hydrofluoric acid) to phosphoric acid, on the stabilization with the latter, showed that highest strengths (immersed unconfined compressive strengths) of a silty clay loam were obtained with 2 per cent of an acid that contained 30 per cent of flouride (Strengths obtained were double those obtained with pure phosphoric acid). Results with a soil that had a lower silt content were far less impressive. It was concluded that a flouride content is most desirable in phosphoric acid stabilization of a clay soil with a high silt content.

1.6 The Mechanisms involved in Soil Stabilization

1.6.1.1 Soil Properties

As recognised by most modern soil classification systems, the properties of soils depend not only on the particle sizes and size distribution, but on the quantity and nature of the fine

material present. In concrete work, the filler materials (i.e. coarse and fine aggregates) may usually be considered inert. Aggregates that are not inert are usually regarded as undesirable. However, in soil stabilization the object is to improve a local material for economic or other reasons and it is very seldom that the soil is chemically inert. The object of this section of the report is to examine the nature of these "non-inert" materials, the clay minerals and their response to various stabilizers.

Metcalf and Ingles[6] summarise what has been stated in the previous paragraph as follows. "Perhaps the most important association after grading is recognition of the clay mineral type, since volume stability cohesiveness and especially the stabilization reactivity of a soil are directly dependent there on. Mineral type is therefore a more powerful diagnostic than plasticity for the engineering behaviour of a soil".

1.6.1.2 Review of Clay Mineralogy

The clay minerals, which are usually formed during the weathering of primary minerals, are a group of complex alumino siliciates. They are small in particle size, flaky in particle shape, and generally carry negative charges on the crystal surfaces. Electrochemical forces (ionic bonds, van der Waals forces, hydrogen bonds, and polar forces) are

very significant in determining the engineering properties.

In order to simplify the study of the mineral structures it is convenient to view each separate mineral as being formed from various basic building blocks.

The first of these units is the silicon - oxygen tetrahedron (see fig 1.2) This consists of four oxygen ions (O^{2-})surrounding a central silicon ion (Si^{4+}). These units combine and form layers with the general formula $n\ (Si_2\ O_5)^{2-}$. The oxygen ion at the apex of each unit carries one unsatisfied valency bond. This is then the tetrahedral silicon layer.

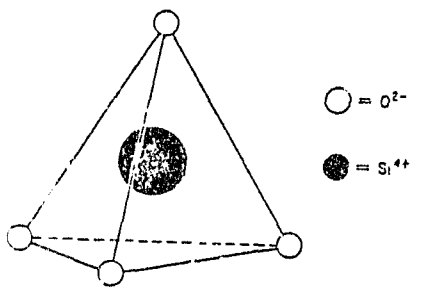


Fig.1.2 The Silicon Oxygen Tetrahedron

The second basic unit is the octahedral unit (see fig. 1.3) The octahedral unit consists of a central ion (which is usually divalent or trivalent (e.g. Al^{3+}or Mg^{2+}) surrounded by six monovalent hydroxyl ions. These units also combine to form

layers and the most commonly occuring octahedral layers in clay minerals are the gibbsite sheet (n $(Al_2 (OH)_6$) and brucite sheet (n $(Mg_3 (OH)_6)$)

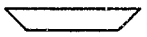


Fig. 1.3 The Octahedral Unit

These basic units may be modified by isomorphic substitutions and adsorbed ions. This is the situation in which the basic units described above are modified by the replacement of one of the ions by another different ion. (eg Fe^{3+} replacing Al^{3+}). The substitution has no effect on the crystal structure, but if the replacement ion is of different valency, the crystal structure will be left with a different charge. The structure of the various clay mineral groups, together with details of the isomorphic substitutions and various physical properties are detailed in table 1.1 For a more detailed description of the clay mineral properties and structure, readers are referred to texts such as Lambe and Whitman[13], Mitchell [14] and Scott [15].

The physical properties of the clay minerals are summarised in table 1.2

Mineral	Structure Symbol	Isomorphous Substitution (nature and amount)	Linkage between Sheets (type and strength)	Specific Surface (m^2/g)	$\frac{1}{\text{Charge Density}}$ ($Å^2$/ion)	Potential Exchange Capacity (me/100 g)	Actual Exchange Capacity (me/100 g)	Particle Shape	Particle Size
Serpentine		none	H-bonding + secondary valence			1	1	Platy or fibrous	
Kaolinite		Al for Si 1 in 400	H-bonding + secondary valence	10–20	83	3	3	Platy	d = 0.3 to 3 μ thickness = $\frac{1}{3}$ to $\frac{1}{10}d$
Halloysite ($4H_2O$)		Al for Si 1 in 100	Secondary valence	40	55	12	12	Hollow rod	OD = 0.07 μ ID = 0.04 μ L = 0.5 μ
Halloysite ($2H_2O$)		Al for Si 1 in 100	Secondary valence	40	55	12	12	Hollow rod	OD = 0.07 μ ID = 0.04 μ L = 0.5 μ
Talc		none	Secondary valence			1	1	Platy	
Pyrophyllite		none	Secondary valence			1	1	Platy	
Muscovite		Al for Si 1 in 4	Secondary valence + K linkage			250	5–20	Platy	
Vermiculite		Al, Fe, for Mg Al for Si	Secondary valence + Mg linkage	5–400	45	150	150	Platy	$t = \frac{1}{50}d$ to $\frac{1}{5}d$
Illite		Al for Si, 1 in 7 Mg, Fe for Al Fe, Al for Mg	Secondary valence + K linkage	80–100	67	150	25	Platy	d = 0.1 to 2 μ $t = \frac{1}{10}d$
Montmorillonite		Mg for Al, 1 in 6	Secondary valence + exchangeable ion linkage	800	133	100	100	Platy	d = 0.1 to 1 μ $t = \frac{1}{100}d$
Nontronite		Al for Si, 1 in 6	Secondary valence + exchangeable ion linkage	800	133	100	100	Lath	l = 0.4 to 2 μ $t = \frac{1}{100}l$
Chlorite		Al for Si, Fe, Al for Mg	Secondary valence + brucite linkage	5–50	700	20	20	Platy	

 Silicon layer

 Gibbsite

 Brucite

Table 1.1 Structure of Common Clay Minerals (after Lambe[13])

MAJOR SOIL COMPONENTS AND THEIR CHIEF PHYSICAL PROPERTIES

Group Designation	Minerals Included in the Group	Mean Size*	Chief Physical Properties
Very fine sand (silica)	quartz	$> 1\mu$	cohesionless, abrasive
Mica	muscovite, biotite	$> 1\mu$	cohesionless, weathers easily, resists compaction
Carbonate	calcite, dolomite	any	pulverizes readily
Sulphate	gypsum	$> 1\mu$	can disrupt cement work
Allophane	amorphous aluminosilicates, attapulgite, hydrous alumina and silica	any	high void ratio, high plasticity, often irreversibly reduced by air drying
Kaolin	kaolinite, halloysite	$\simeq 1\mu$	non-swelling, low plasticity, low cohesion
Illite	illite and partially degraded micas	$\simeq 0.1\mu$	expansive, medium plasticity, low permeability
Montmorillonite	montmorillonite, "bentonite", and mixed layer expansive clays	$\leq 0.01\mu$	highly expansive, very plastic, extremely low permeability
Chlorite	chlorite, vermiculite	$\simeq 0.1\mu$	limited swelling, low shear resistance
Organic matter	humic acid and humates	any	high permeability resists compaction, can degrade rapidly by air oxidation

* 1μ = 1 micron = 10^{-4} cms.

Table 1.2 (after Ingles[6])

1.6.2 Soil Lime Reactions

An understanding of the reactions between soil and lime is probably fundamental to any discussion on lime-PFA stabilization and to a certain extent, cement and cement-PFA stabilization. Thompson[16] states 'The addition of lime to a fine-grained soil generally (a) reduces the plasticity index, (b) increases the shrinkage limit, (c) improves workability, (d) promotes drying, (e) reduces swell potential, and (f) may increase strength'. The reasons for these effects can be understood if the four basic lime-soil reactions are examined.

A. Cation Exchange

The lyotropic[16] series of common cations associated with soils is given by Na^{+} K^{+} Ca^{++} Mg^{++} (any cation will tend to replace a cation to the left of it). Ion exchange is the short term reaction in which Ca^{++} ions (which are supplied by the lime) replace smaller sodium and hydrogen ions on the surface of the clay lattice. Ballantine[17] states that the sodium ion (Na^{+}) is capable of holding 79 water molecules, whereas the calcium ion only 2, and the lime thus converts the soil from a hydrophyllic to a hydrophobic condition. The extent of the exchange reaction depends on the amount of replaceable sodium ions in the soil.

B. Flocculation and Agglomeration

The second rapid physico-chemical reaction[17] is the crowding

of excess calcium cations onto the surface of the clay lattice. This, and the preponderance of divalent calcium ions which have replaced the monovalent sodium ions causes an attraction between the soil particles which results in their flocculation. This reaction results in an apparent change in texture of the clay[16].

These two basic reactions account for effects (a) to (e) mentioned by Thompson.

C. Pozzolanic Reactions

A limited discussion on pozzolanic materials has already been given in section 1.4.1. The lime soil pozzolanic reaction is regarded as the major source of strength increase in lime stabilization[16]. It is a long term reaction and generally lime has to be present in quantities exceeding the requirements of the two reactions already discussed for this reaction to have any chance of occuring. The most significant soil properties influencing the pozzolanic reaction[16] are organic matter content, clay mineralogy, weathering status of the soil, natural soil drainage and the presence of carbonates. The pozzolanic reaction is also influenced by curing temperature, curing period and lime percentage.

D. Lime Carbonation

This reaction differs from the three already discussed in that

it is undesirable and should be avoided. It results from a reaction between lime and carbon dioxide in the air, and produces the weak cementing agents, calcium and magnesium carbonate. This reaction can be minimised by taking necessary precautions.

With the exception of carbonation, the extent of these reactions depends mainly on the clay mineral types present. The effect of various binders on the different clay mineral types has been studied by researchers such as Croft [8,9,18,19]. The procedure used by Croft was to test the effect of a binder on various clay minerals by comparing the unstabilized soil and clay-binder mixtures at various ages using :-

a) X-ray diffraction

b) differential thermal analysis

c) physical properties (eg plasticity index and shrinkage)

Compaction tests and immersed unconfined compression tests were also performed. In some cases, the reaction products were also examined.

As stated previously, the above tests were performed on pure clay minerals and not on natural soils. Croft did, however, reach some conclusions on the stabilization of natural soils and these will be summarised below.

1.6.3.1 Lime Stabilization of Natural Soils

"It is evident that for a soil to be satisfactorily stabilized by lime, it must possess an appreciable amount of clay. No significant reactions have been observed with quartz. The strength gain will be more rapid if the clay present in the soil is montmorillonite. The presence of both montmorillonites in small amounts and kaolinites in large amounts will ensure both long and short term strength increases. Clay soils containing illite and kaolinite as the essential clay minerals may appear, when tested in the laboratory at 7 and 28 days, to be non-responsive to lime. However, after further ageing it is quite probable that satisfactory strengths will be achieved. If high early strengths are absolutely essential, lime should not be used with such soils. It is suggested that a knowledge of the composition of a soil will permit a reasonable prediction of its likely response to lime action. Such a knowledge is not intended to replace existing testing procedures but merely to permit a more realistic estimate of the soil potential as well as to eliminate unnecessary testing "[9].

1.6.3.2 Lime - PFA Stabilization of Soils

Croft found the various flyashes he tested to be less responsive to lime than the pozzolanic clay minerals. In order for the flyash to be effective, therefore, sufficient lime has to be be present to satisfy the initial lime-soil reactions, and then

also the pozzolanic reactions of the PFA and clay minerals.

Croft's conclusions regarding clay soils were 'The addition of fly ash is probably only justified if the soil responds slowly to lime alone, eg. kaolinitic clays in which the improvement of strength is most pronounced in the long term' [19].

On the subject of natural soils, he reached the following conclusions[8]. If PFA is added to a clay soil, excess silt sized particles may cause compaction problems. The material most suitable for lime-PFA stabilization would be those deficient in particles in this size range and also some alluvial sandy soils, laterites, soil horizons at the base of residual profiles, natural and blended gravels, crushed stone, slags, etc. The presence of a clay fraction would also be advantageous. (If the clay minerals are active, both short and long term strengths would result) With non-active clays, short term strengths would depend largely on grading. The compacted material would have to be sufficiently strong to carry loads until the lime-PFA reaction had caused further strength increases.

1.6.3.3 Cement Stabilization

"For cement stabilization, a successful hydration of the cement minerals is not sufficient. The presence of organic matter, acid pH, expansive clays and possibly hydrated iron oxides may inhibit the 'setting-stage', and if this in inhibited it is

possible that no secondary cementation due to reaction of liberated lime with clay minerals takes place." [19]

1.6.3.4 Cement-PFA Stabilization

In concrete work, where elevated temperatures are not used in the curing period, about 20 percent of the cement is sometimes replaced by PFA, and sufficient lime is released during the hydration of the cement to cause the pozzolanic reaction to occur. In soil stabilization, where cement contents are much lower, and the possibility of active clay minerals being present does sometimes exist, the addition of PFA to the mix is not usually beneficial as free lime released by the cement reacts only with the clay minerals.

There may be cases, however, where the addition of PFA to cement is desirable for other reasons, e.g. as a diluent to facilitate mixing when very low cement contents are required.

1.6.3.5 Phosphoric Acid Stabilization

A research program, similar to those carried out by Croft[8,9,18,19] was carried out by Demirel and Davidson[20] to examine the reaction of phosphoric acid with clay minerals. The tests were performed and commercial samples of Kaolinite, Montmorillonite, Vermiculite, Chlorite and Illite. The authors reached the following conclusions. "Rapid and extensive reactions are shown by chlorite and vermiculite clays. The reaction with kaolinite was rather

slow and incomplete. Montmorillonite clay gave a rapid but incomplete reaction. Except for the conclusion reached for kaolinite and vermiculite, these results closely agree with the conclusions reached by the authors on analysing the strength characteristics of phosphoric acid stabilized soils[20].

1.7 Indentification of Clay Mineral Types

In order to identify clay mineral types present in a soil, it would obviously be desirable to perform X-ray diffraction or differential thermal analysis. Obviously, however, most soils laboratories do not have the necessary equipment. There are other simple methods of identification and a few of these will be mentioned below.

1) A knowledge of the geological origin of a soil will limit the number of possibilities of clay minerals present.

2) Tests such as the Emerson crumb test (see Metcalf and Ingles[6]) can be used to indentify the clay minerals present in a soil.

3) By examining the environment and soil profile, certain inferences regarding soil composition can be made. Tables 1.3 and 1.4 illustrate this statement.

Inferences From the Profile

Observation	*Dominant Clay Component*
mottled clays, red-orange-white mottle	kaolinites
mottled clays, yellow-orange-grey mottle	montmorillonites
medium to dark gray and black clays	montmorillonites
brown and red-brown clays	appreciable illite, some montmorillonite
white and light gray clays	kaolinites and bauxites
discrete microparticles of high light reflectance (micas)	micaceous soils
discrete microcrystals, easily crushed	gypsum-rich soils, or (rarer) zeolites
soft nodules, acid-soluble, disseminated	carbonates
hard nodules, red-brown	ironstones, laterite
extensive cracking, wide, deep and closely spaced at 5 to 6 cms. or less	calcium-rich illites and montmorillonites
up to intervals of 30 cms. and more	illites
open-textured friable loamy soils with appreciable clay content	usually associated with carbonate, allophane or kaolin, but *never* montmorillonite and seldom illite
open-textured friable loamy soils with appreciable clay content, black	organic soils, peats
open-textured friable loamy soils of low clay content	carbonate, silts and sands
wormy appearance on exposed pre-existing weathered profile	montmorillonites, plus soil salinity
relatively thin, strongly bleached horizon near the soil surface (up to 60 cms. from the top)	above the bleach, fine silt: below the bleach, dispersive clay. Probably a seasonal perched water table at the bleach level

Table 1.3 (after Ingles[6])

Inferences From the Environment

Observation	*Dominant Clay Component*
turbid waters of strong yellow-brown to red-brown colour	montmorillonites, illite plus soil salinity
clear waters	calcium, magnesium or iron-rich soil, highly acid soil, sands
clear waters with a bluish cast	non-saline kaolins
erosion gullies and/or field tunnelling in the natural soil	saline clays, usually montmorillonites
as above, mild	kaolinites
landslips	kaolinites, chlorites
surface microrelief (gilgai)	montmorillonites
country rock type granitic	kaolinites, micas
country rock type basaltic, poorly drained topography	montmorillonites
country rock type basaltic, well drained topography	kaolinites
country rock type sandstones	kaolinitic
country rock type mudstones and shales	montmorillonites or illite, often soil salinity
country rock type limestone	alkaline montmorillonites and chlorites of very variable properties
country rock type recent pyroclastics	allophanes

Table 1.4 (after Ingles[6])

1.8 Results of Previous South African Research

Beckwith and Schnuir[21] conducted a few tests on two types of soil viz. Black turf and Aeolian sand. Two types of binder were used :- Ash-lime and Ash-lime - gypsum mixtures.

They concluded:

1) That the addition of PFA increased the strength of the soil, possibly due to improved grading.

2) The plasticity index of the soil was reduced, possibly due to the lime in the stabilizer.

3) With the addition of gypsum to the binder, the pozzolanic activity of the PFA became apparent which increased the strength of the soil.

Figure 1.4 illustrates one of their results.

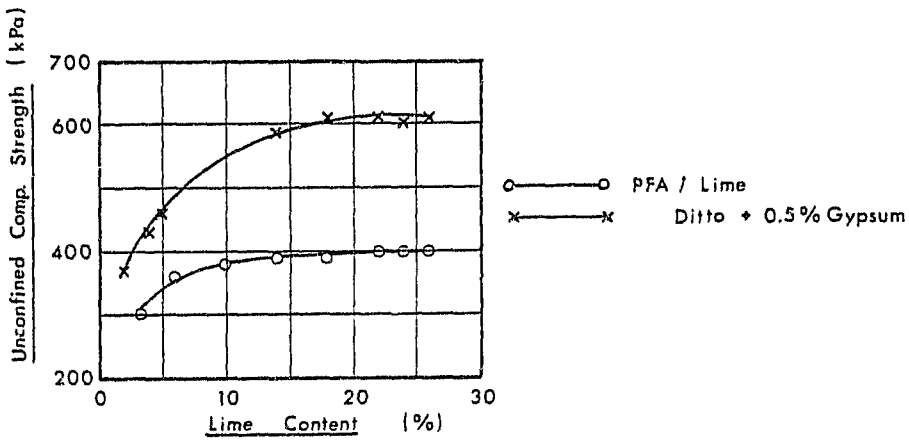


FIG 1.4 Strength of PFA / Lime mixes (after Beckwith[21])

A testing programme also began in the Johannesburg City Council's Laboratories in 1975. Tests were aimed at determining whether mine sand could be stabilized with PFA. A report covering the results of the above tests was never written. Fig. 1.5 is a copy of the results of compressive strength tests. CBR and compaction tests were also performed.

A series of tests investigating the stabilization of decomposed dolerite with lime-PFA mixtures was reported by de Wet[22]. Age strength curves from this investigation are shown in figures 1.6a to 1.6c.De Wet concluded that for the low plasticity material (material A in fig 1.6),lime-PFA gave better results than with lime alone. In the case of the higher plasticity material B, the clay was sufficiently pozzolanic, and the lime-PFA did not improve on lime alone. Slag-PFA mixtures showed poor results (see fig 1.6) De Wet could not explain the apparent strength drop from 56 to 90 days evident on both materials stabilized with lime-PFA.

1.8.2 Phosphogypsum Stabilization

The results of a limited series of tests using phosphogypsum as a binder are shown in fig. 1.7

	PFA %	Lime %	Gypsum %	Minesand %
A	88.7	10.7	0.6	
B	78.6	21.4	0.6	
C	89.3	10.7		
D	25	3	0.6	71.4
E	12	3	0.6	84.4

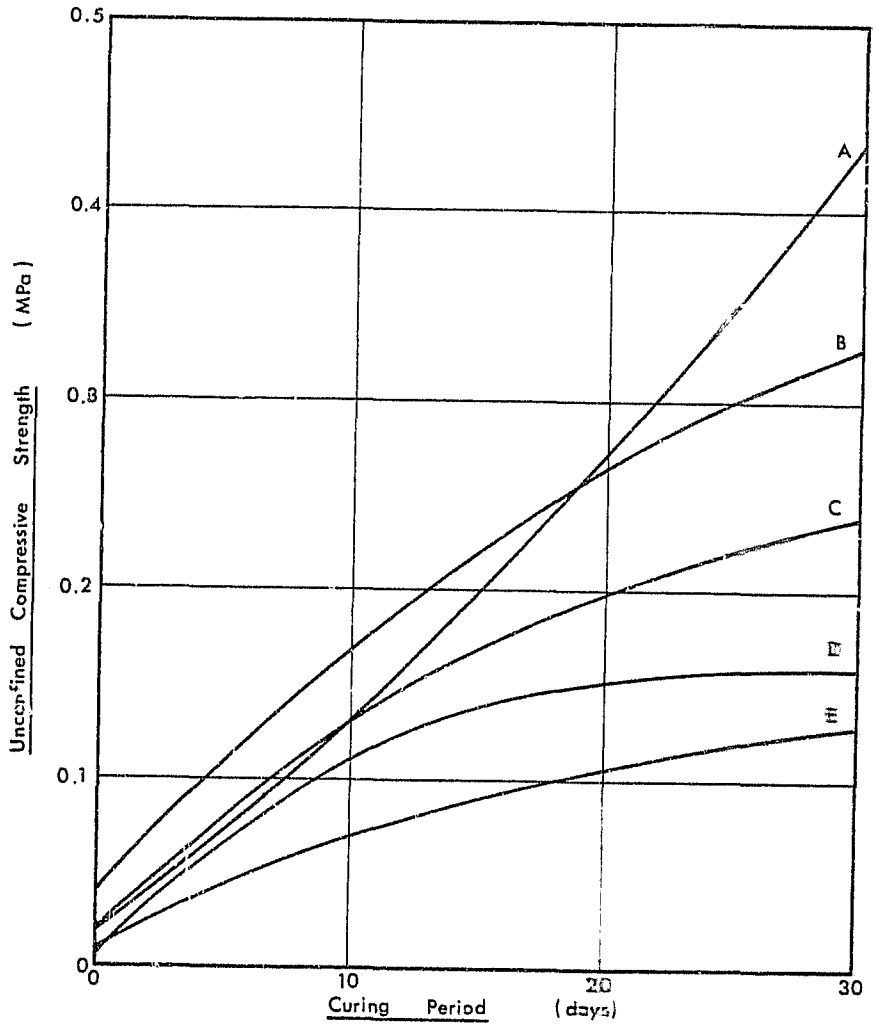


FIG 1.5 Results of Strength Tests (after City Council[23])

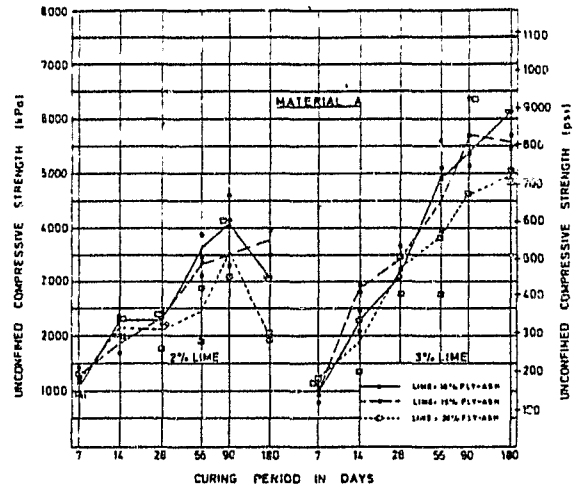


FIG 1.6a **Stabilized with lime/fly-ash** (after De Wet[22])

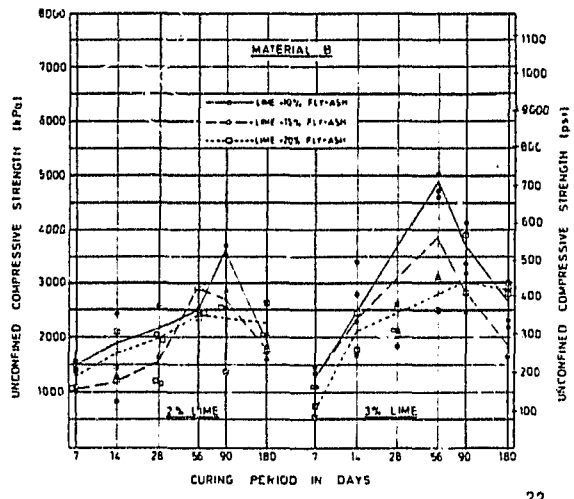


FIG 1.6b **Stabilized with lime/fly-ash** (after De Wet[22])

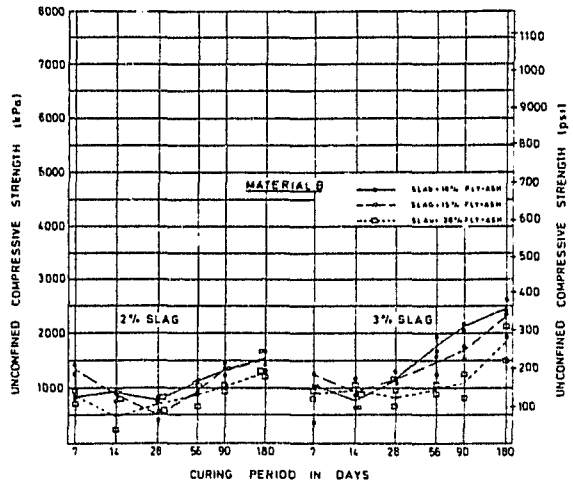


FIG 1.6c **Stabilized with slag/fly-ash** (after De Wet[22])

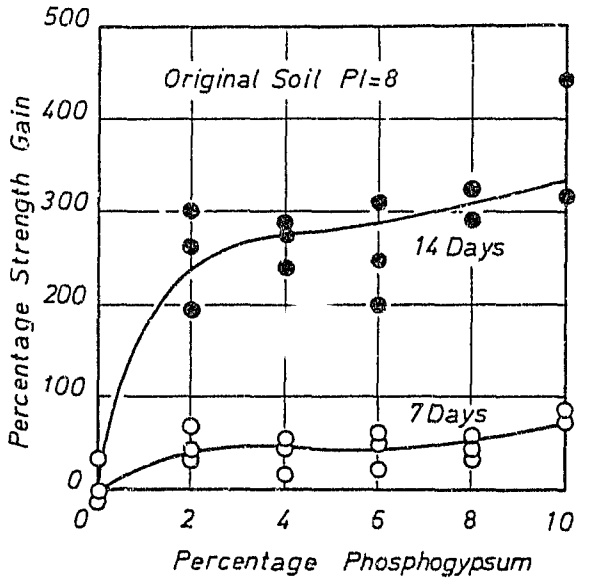


FIG 1.7 Stabilization of a clayey granite sand using phosphogypsum (after Blight[11])

1.9 Research Program

As stated previously, the object of this research program was to investigate the possibility of using PFA and phosphogypsum as soil stabilizers. It was decided at the outset to use a similar approach to that used by Blight[4] in the initial investigation into slag soil stabilization.

Five soils, all from the Johannesburg area were used in the testing program. The engineering properties of the unstabilized soils are compared in chapter 2.

The different ashes, Darling and Hodgson PFA (which is referred to as PFA in the results) and Kelvin PFA (KPFA in the results) are compared in chapter 3. A chemical analysis of the phosphogypsum is also given in this chapter.

In chapter 4 the results of the testing program are presented and discussed. Results fall into the following categories.

1) Effects of the binders on soil plasticity

2) Effects of the binders on the moisture-density relationship for the soils

3) Effects of the binders on the unconfined compressive strength of the soils.

   3a) Keeping the total binder content constant

at 3% the effect of binder composition (ie PFA/Lime and PFA/Cement ratios) was determined by performing unconfined compression tests at 28 days.

3b) Using only one of the five soils, the effect of binder content on compressive strength was determined.

3c) Using only one soil, the effect of curing period on strength was determined.

A limited series of tests were also performed to determine the influence of ash pulverization on strength gain.

In chapter 5, general conclusions are drawn and proposals for further research made.

2. PROPERTIES OF THE SOILS USED IN THE INVESTIGATION

2.1 Selection of Soil Types

In order that the investigation of the stabilizers should remain general, five soils with plasticity indeces ranging from 0 to 30 were chosen. These soils all originate from the Johannesburg Municipal area and samples of approximately 100 kg each were supplied by the City Engineers Department of the Johannesburg City Council.

2.2 Preparation of the Soils for Testing

The soil samples arrived in canvas bags. (each bag containing approximately 25 kg of soil). With the exception of sample E, the soil was removed from the bags and broken down into individual particles by hand and using a rubber mallet. Each soil was then sieved through a 4,75 mm sieve and material retained on this sieve was discarded. The remaining soil was then thoroughly mixed, quartered and returned to the canvas bags.

On arrival, soil sample E, a highly plastic black clay, was wet and the above preparation would have therefore been unsuitable. The soil was air-dried and then broken down in two stages, firstly using rolls and then a vertical spindle pulverizer

(subsequent inspection showed that the coarse fraction did not appear to have been affected by this pulverization) This soil was then also thoroughly mixed, quartered, and returned to the bags.

2.3 Description of the Soil Types

The origins and various properties of the soils are summarised in table 2.1

2.3.1 Plasticity Tests

These are standard laboratory tests and detailed experimental procedures are covered in texts such as Lambe[1] and Ackroyd[2].

2.3.2 Particle Size Analysis

Coarse Analysis

A sample of soil was carefully broken down into individual particles using a mortar and rubber tipped pestle. A representative 200 grams of this material was then placed in a beaker containing 500 ml of a 40 gram/litre sodium hexametaphosphate solution and left to stand for 24 hours. This slurry was then stirred for 10 minutes using a spatula, after which it was washed through a 75μ sieve. When no fine material (ie - 75μ)remained on the sieve, the remaining fraction was ovendried and then sieved through a nest of sieves. The combined results of this analysis and the hydrometer analysis (to be described next) are presented in graphical form in fig. 2.1.

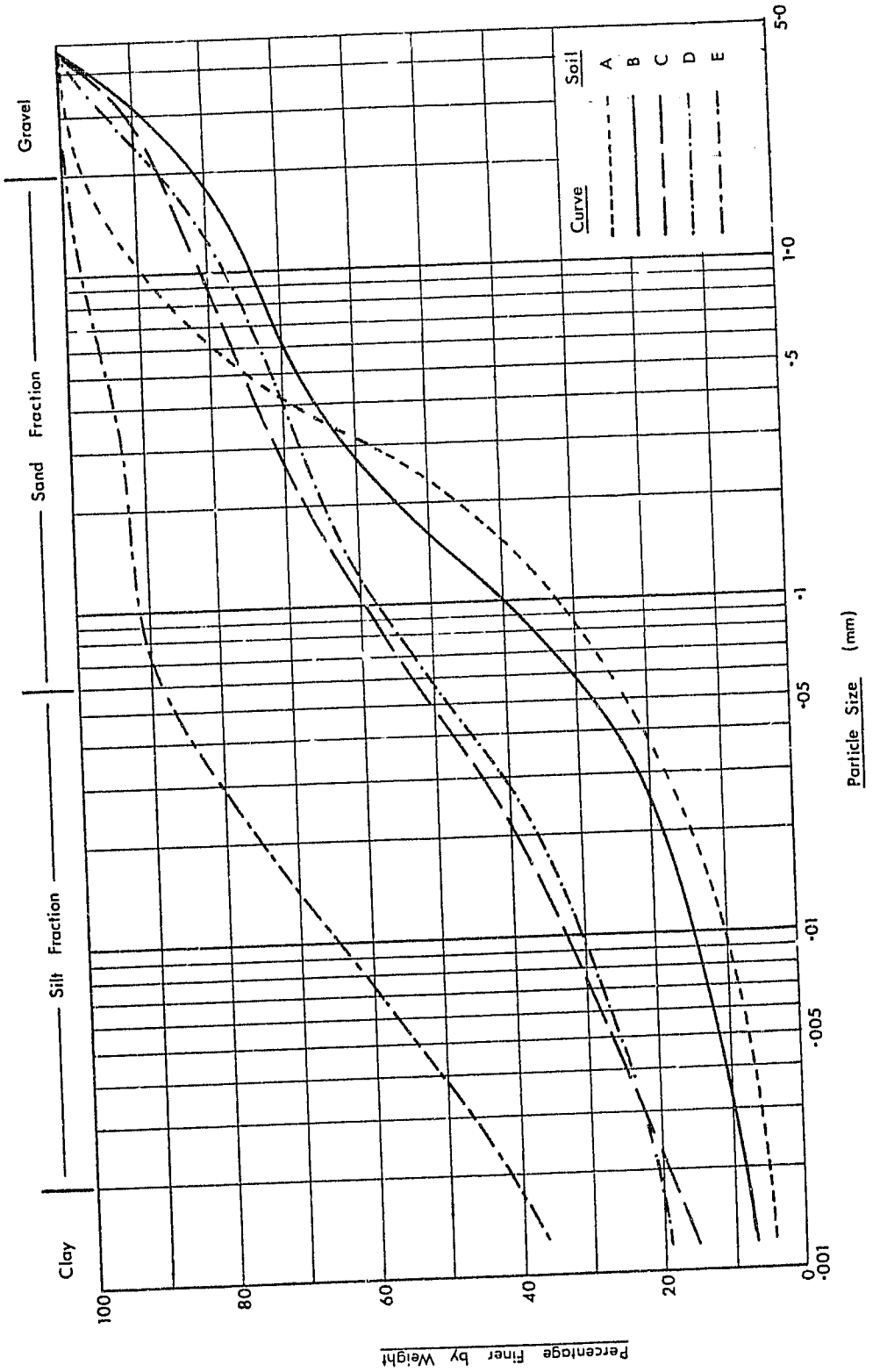


FIG 2.1 Grading curves for the Soils

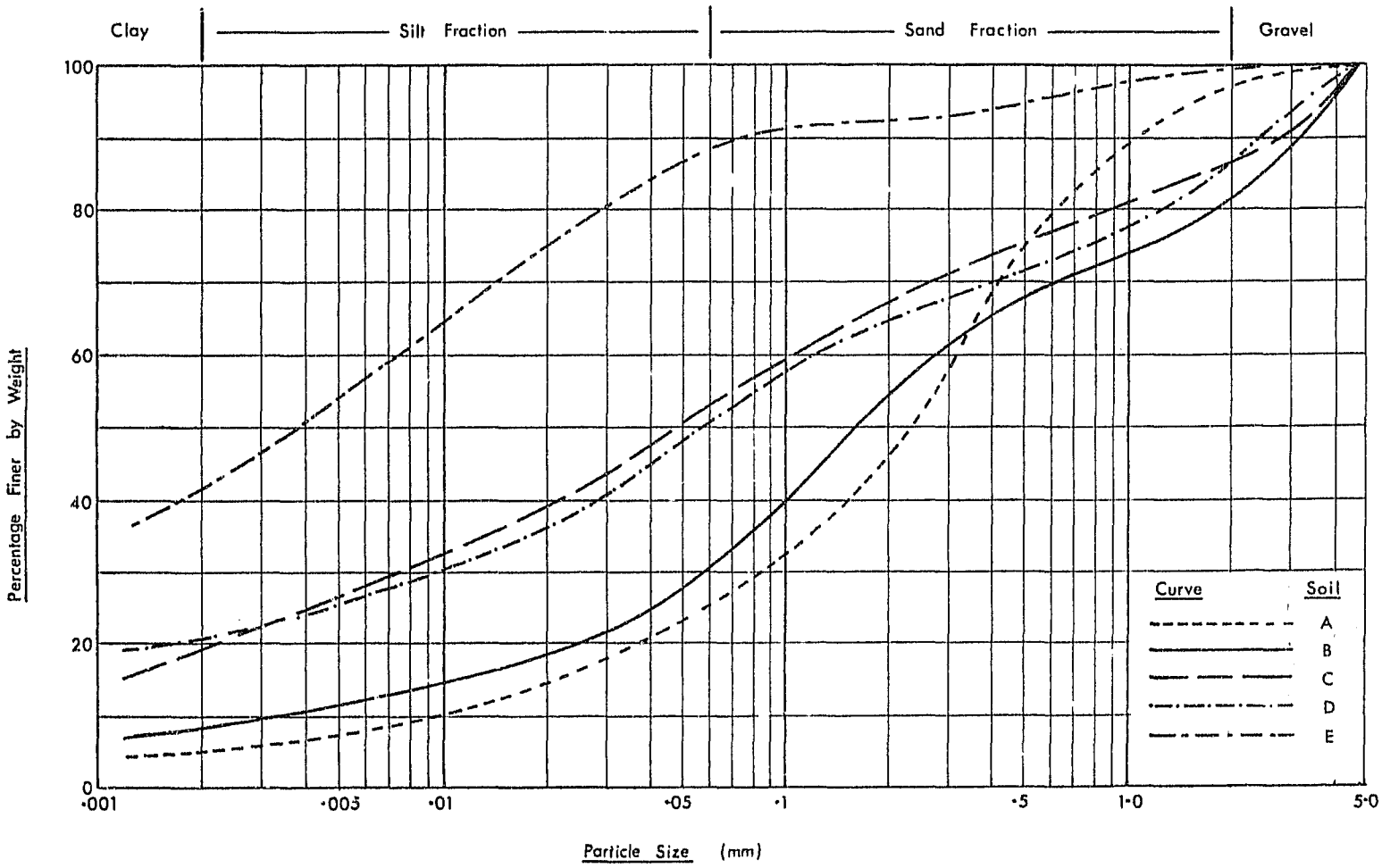


FIG 2.1 Grading curves for the Soils

Hydrometer Analysis

Only the preparation of the samples will be described here as the test is standard and as in 2.3.1, is covered in many texts.

A sample of soil was placed on the 75 μ sieve. Distilled water was then poured over this material and the fraction passing through the sieve was retained in a pan. The material on the pan was then oven dried 45 grams of this fraction were then placed in a dispersion cup with 20 ml of a 2% tetrasodium pyrophosphate solution, and approximately 400 ml of distilled water. This solution was then agitated for 15 minutes using a mechanical agitator, from where it was transferred to a graduated cylinder. Distilled water was added until 1 litre of solution was present. A standard hydrometer test and analysis were then performed.

2.3.4 Compaction Tests

The modified AASHO compactive effort was used in compaction tests. The tests were done in the 152 mm mould using a Farnell compaction machine. The optimum moisture content of the soils was estimated as 67% of the plastic limit and tests were carried out on fresh soil at moisture contents of approximately 1 and 3% above and below this value. The results of the tests are shown in graphical form on fig. 2.2

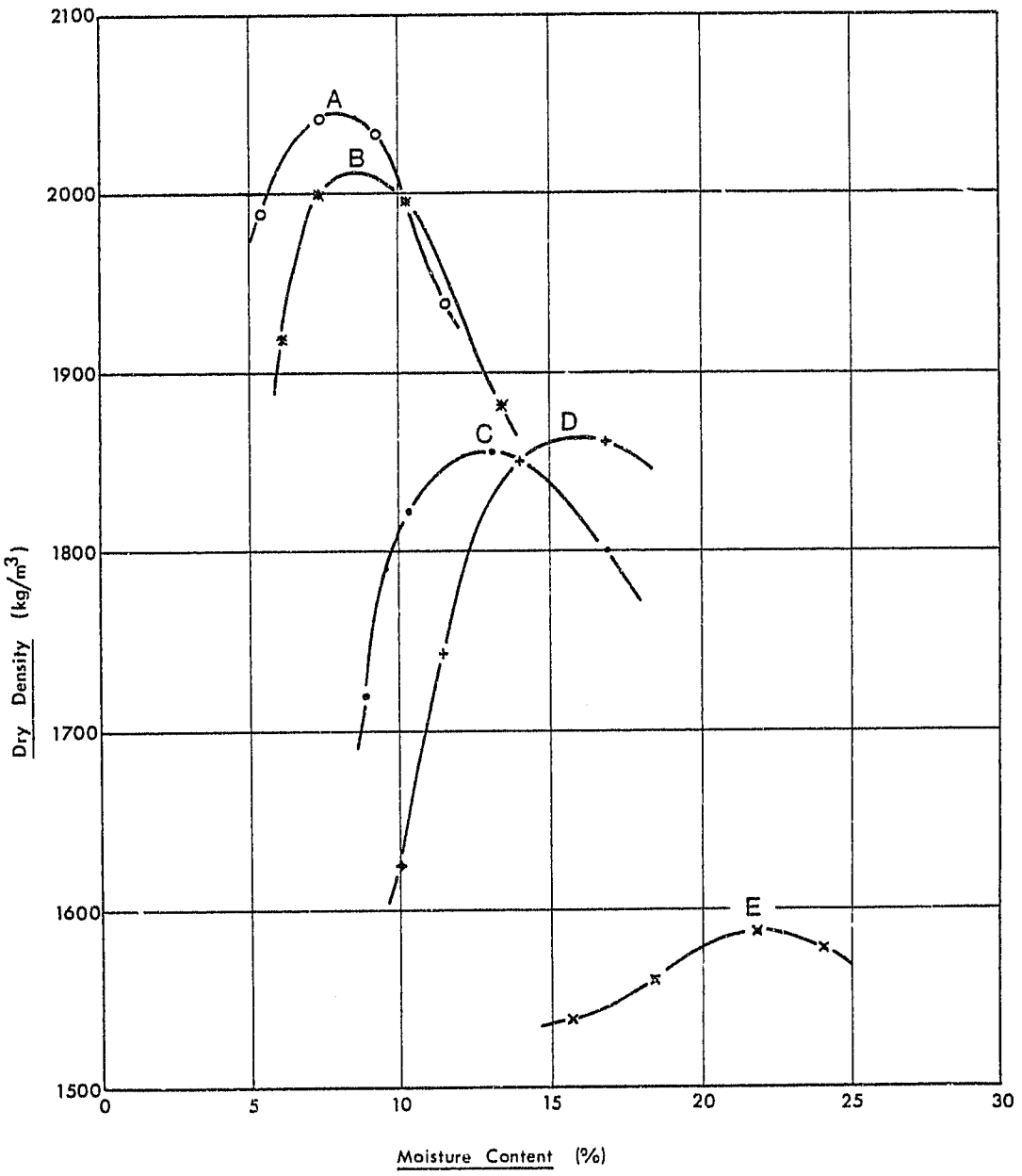


FIG 2.2 Dry Density – Moisture Content curves for unstabilized soils. (Mod AASHO effort)

2.3.5 Organic Carbon Content

The organic carbon content of each soil was determined by the Johannesburg City Council's laboratories. The method used was that described by P.R. Hesse in 'A textbook of soil chemical analysis'. (John Murray, London, 1971) The results of the analyses are shown on table 2.1.

2.3.6 X-ray Diffraction Analysis

20 grams of - 150 μ soil was mixed with 50 ml of a 0,04 per cent tetrasodium pyrophosphate solution and placed in a small glass bottle. The solution was then shaken by hand for 1 minute. Approximately 0,5 ml of this solution was then drawn from the centre of the container using a 1,5 mm internal diameter plastic tube. This was then dropped onto a glass slide and left overnight to dry.

Two slides were made for each soil type. One was glycolated, by leaving it for 24 hours in a desiccator containing a beaker of ethylene glycol, in a 60°C oven. (Glycolation is necessary to distinguish between some clay minerals that show the same diffraction patterns) Each slide was then placed in an X-ray diffractometer, and using Copper radiation, X-ray diffractograms recorded. The predominant clay minerals were identified using information from Mitchell[14] and these are listed in table 2.1.

Comments on Diffraction Analysis Results

It can be seen from table 2.1 that all soils contained koalinite and quartz, with only soils A and E containing other clay minerals. With the exception of soil A, the diffractograms for all the soils were very similar. The peaks were not at all sharp, and were approximately the same height for the four soils. The montmorillonite peak for soil E was extremely small. It is suggested that the soils probably originated from the upper horizons of the profile and had been slightly modified by organic matter, leaching or some other process.

2.3.7 Eades Tests

The Eades test[17] is a quick test used to determine the amount of lime necessary to satisfy the short term soil-lime reactions. It is assumed that these reactions are completed within one hour. The test is then based on the fact that when free lime is present is a soil-lime-water slurry, the pH of the slurry will be approximately 12,4. (i.e. the addition of further lime will not change this pH). The test procedure is then as follows[17]-

"1 Representative samples of oven-dried minus 0,425mm soil fines each of 20 gm, weighed to the nearest 0,1 gm, are poured into 150 ml (or larger) plastic bottles with screw tops.

2 Since most soil fines require between 3 and 7 per cent

lime, it is advisable to set up five bottles with dry hydrated lime percentages (based on soil fines) of 3, 4, 5, 6 and 7. This will ensure, in most cases, that the percentage of lime required will be included in the first test batch. Weigh the lime to the nearest 0,01 gm and add it to the soil fines. Shake to mix soil fines and hydrated lime. The lime used should meet the specification SABS 824-1967.

3 Add 100 ml of CO_2-free distilled water to each bottle

4 Shake the soil-lime and water until there is no evidence of dry material on the bottom. Shake for a minimum of 30 seconds.

5 Shake the bottles for 30 seconds every 10 minutes.

6 After one hour, transfer part of the slurry to a plastic beaker and measure the pH. The pH meter must be equipped with a "Hyalk" electrode and standardised with a buffer solution having a pH of 12,00.

7 Record the pH for each of the soil-lime mixtures. The lowest percentage of lime at which the pH remains constant (in most cases at pH = 12,4) is the percentage required to stabilize the soil fines".

8 The initial lime consumption for the whole soil is then calculated by multiplying the percentage found

in the Eades test by the percentage of minus 0,425 mm soil fines present in the original test sample.

The results of the Eades tests on the soils are shown on table 2.1. More detailed results for soil D are also shown on figs. 4.17 and 4.18 in chapter 4.

TABLE 2.1 Summary of Soil Properties

SOIL	A	B	C	D	E
Geological Origin	Main Bird Series (Residual Quartzite)	Black Reef Series	Ferricrete	Ventersdorp series (Residual Andesite)	Alluvium
Colour (Burland Colour Chart)	Light red	Dark Brown	Dark olive - Dark yellow	Light reddish orange	Black
Liquid Limit (%)	N.P.+	23,7	36,5	37,4	60
Plasticity Index (%)	N.P.+	7	14,1	15,2	30
U.S.C.S. Classification	SW	GW	CI	CI	CH
(%) Clay	5	8	19	21	42
Optimum Moisture Content*(%)	8	8,7	13	15,25	21
Maximum Dry Density*	2045	2017	1855	1860	1560
Organic Content (%)	0,22	1,2	0,51	0,55	1,4
Clay Minerals present	Kaolinite quartz felds,r	Quartz kaolinite	Kaolinite quartz	Kaolinite quartz	Quartz,kaolinite montmorillonite
Initial Lime Consumption (%)	2	2		3	5

* Modified AASHO Test

+ Non Plastic

3. PROPERTIES OF THE BINDERS USED IN THE INVESTIGATION

3.1 Pulverised Fuel Ash Binders

3.1.1 Particle Size Analysis

Sieve Analysis

The ashes were dry sieved to determine the coarser grading. It was noticeable with both ashes that the material retained on the sieves i.e. (coarser fractions) tended to be black, whereas the finer material which passed through the sieves was light grey. This indicates the presence of unburned coal in the coarser fractions.

Hydrometer Analysis

A sample of each ash weighing 45 grams was soaked in 125 ml of a 40 gram/litre sodium hexametaphosphate solution for 24 hours. This solution was then placed in a dispersion cup, approximately 300 ml of distilled water added and then agitated for 15 minutes using a mechanical stirrer. This solution was then transferred to a graduated cylinder and topped up with distilled water until 1 litre of solution was present. A standard hydrometer test and analysis were then performed.

The results of the tests on the Kelvin ash and Darling

and Hodgson ash, as well as pulverised+ samples of both ashes are shown in fig. 3.1 (nb. The curves for both samples of pulverised ash were very similar and so only one is shown.)

+ The pozzolanic properties of PFA depend partially on the fineness of the ash. In an attempt to upgrade the local PFA, the ashes were milled for 2 hours in a laboratory ball mill.

Note. A hydrometer analysis is based on the assumption that all particles are solid and spherical and of the same specific gravity. Due to the presence of different amounts of carbon in the different size ranges, and the fact that the ash particles are hollow spheres, neither of these assumptions are met, but nevertheless, the test provides a means of comparing the ashes.

3.1.2 Specific Gravity Tests

The specific gravity of the ashes was determined (Lambe,[1] Ackroyd[2]) using phycnometers and distilled water. The results of these tests are presented in table 3.1 below.

ASH	Darling and Hodgson	Kelvin
SPECIFIC GRAVITY	2,27	2,21

TABLE 3.1 SPECIFIC GRAVITIES OF THE ASHES

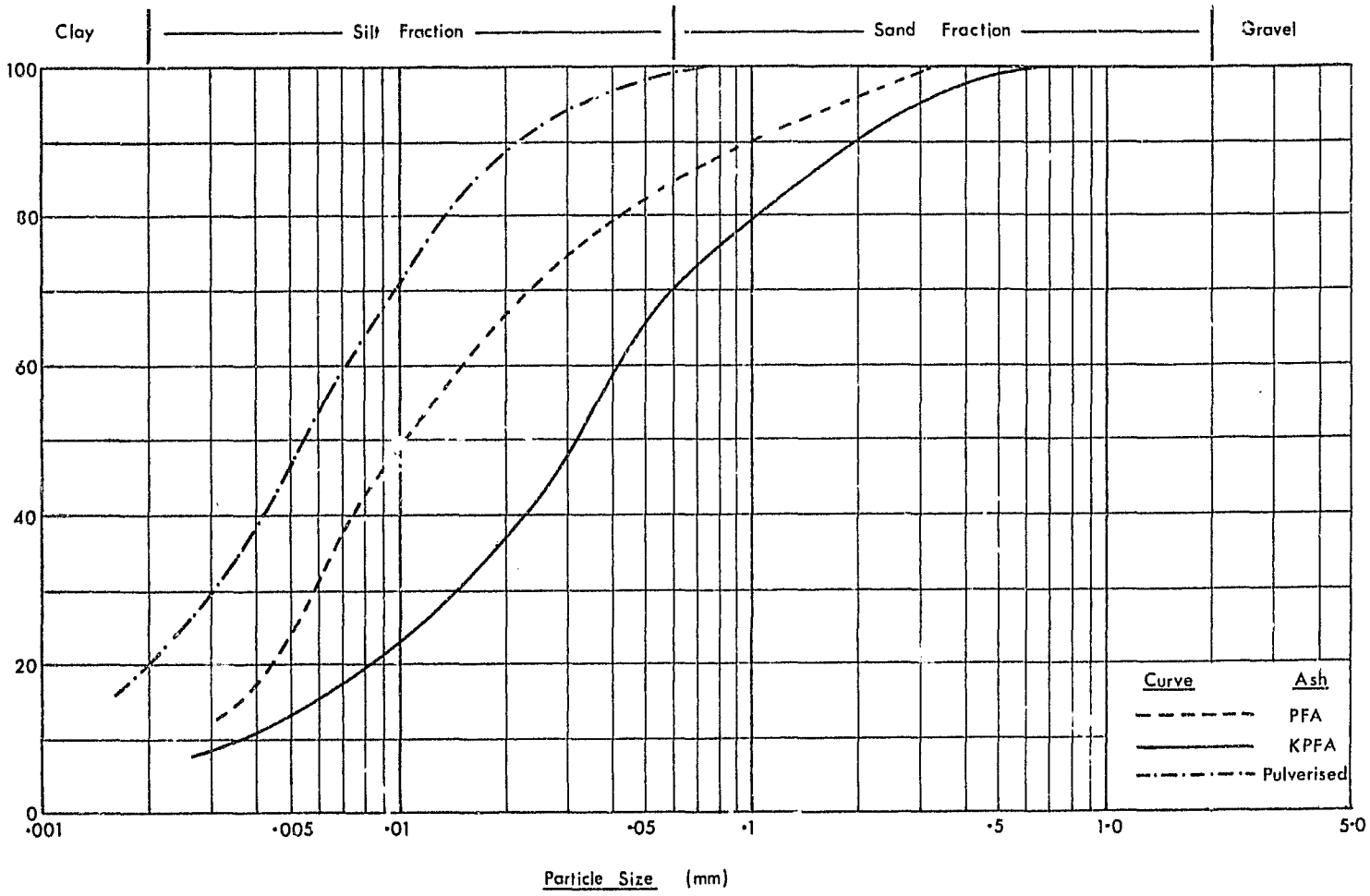


FIG 3.1 Grading curves for the Ashes

3.1.3 Chemical Analysis

Chemical analyses of the PFA samples were done by the Johannesburg City Council. The results of these analyses are presented in table 3.2 in which they are also compared to various international standards.

CHEMICAL CONTENT OR PHYSICAL PROP.	BS3892	ASTM C618/72	USBR	PFA	KPFA
min SiO_2(%)	-	>70,0	>75,0	53,3	43,41
min Al_2O_3(%)	-			18 >73	28 >73
min Fe_2O_3(%)	-			1,4	1,8
max SO_2(%)	2,5	5,0	4,0	0,3	0,7
max MgO(%)	4,0	-	5,0	1,4	1,3
max Na_2O(%)	-	>1,5	>2,0	0,1 >0,2	0 >0,1
max K_2O(%)	-			0,1	0,1
max loss on ignition (%)	7,0	12,0	5,0	1,9	15,9
min specific surface cm^2/g	4250 to 5780	6500 (cm^2/cm^3)	6500 (cm^2/cm^3)	3400	4400
max retained on 325 mesh(%)	-	20,0	15,0	15	30

TABLE 3,2 PROPERTIES OF PFA'S USED IN INVESTIGATION IN RELATION TO BS, ASTM AND USBR REQUIREMENTS

3.2 Phosphogypsum Binder

A chemical analyses of the gypsum and return water used in the tests are given in table 3,3 below.

This analyses was supplied by Triomf Fertilizer (Pty) Limited.

	GYPSUM	RETURN WATER
TOTAL P	0,43%	
Water Soluble P	0,36% (=1,14%H_3PO_4)	904%(=0,13%H_3PO_4)
Flourine	0,4% (=0,51%H_2SiF_6)	1,6% (=2,02%H_2SiF_6)
Free Water Content	10%	

TABLE 3,3 PHOSPHOGYPSUM ANALYSIS

4. INFLUENCE OF THE BINDERS ON SOIL PROPERTIES

4.1 Effect on Soil Plasticity

Liquid and plastic limit tests were performed on soil E immediately after the binders had been added to the soil. A 3 % binder content was used for each binder. The results of these tests are summarised in table 4.1 below.

BINDER	LIQUID LIMIT (%)	PLASTICITY INDEX (%)
NONE	60	30
PFA	57	26
KPFA	57	23
LIME	54	8
CEMENT	57	17
GYPSUM	56	23

TABLE 4.1 Effect of the Binders on Soil Plasticity.

Both the phosphogypsum and PFA binders had the effect of reducing the plastic index of the soil. It is expected that increased quanitities of PFA would further reduce the soil plasticity [27], until a minimum value was reached after which further addition of PFA would not be benificial (Joshi [27] reports that the beniefits of adding more than 15% PFA were

insignificant). The actual value would depend on the characteristics of both the soil and the ash.

The plasticity tests in 4.1 were performed immediately after mixing. The reduction in soil plasticity could be attributed to an increase of silt size particles in the soil, or possibly, due to the presence of trace amounts of lime in the PFA. If the effect was caused for the former reason, ash from dams could possibly be equally effective in reducing the soil plasticity.

4.2 Effect of the Binders on the Moisture-Density Relationship for the Soils

In order to determine the effect of the Binders on the moisture-density relationship for the soils, modified AASHO tests were performed on soil D, with a 3% binder content, for each binder.

The procedure adopted in these tests is slightly different to that described in 2.3.4 as only a limited quantity of soil was available. The test was now performed in a 102 mm diameter mould with appropriate changes to the energy input (ie number of blows per layer). Following compaction, trimming and weighing, the soil was extruded from the mould, a moisture content sample taken, and the remaining soil broken down and reused. (cf. 2.3.4 where fresh soil was used).

In order to check the accuracy of this method, a test was also

performed on some unstabilized soil and this was compared to the results in 2.3.4. The results obtained in both tests were virtually identical and the above method, therefore acceptable.

The results of the above tests are summarised in fig. 4.1 From fig. 4.1, the following conclusions can be drawn.

a) Both samples of PFA had the effect of increasing the maximum dry density and reducing the optimum water content. The effect was more evident with the Darling and Hodgson PFA. This is probably due to the presence of the sphercial PFA particles which increase the workability of the soil.

b) The phosphogypsum binder increased both the maximum dry density and the optimum water content.

c) Both lime and cement reduced the maximum dry density and increased the optimum water content.

Note

In the compaction test on the sample containing lime, the last few blows applied to each layer tended to break up the compacted soil and possibly reduce the density ie. overcompaction. The effect of this on the unconfined compression tests will be discussed in the following section.

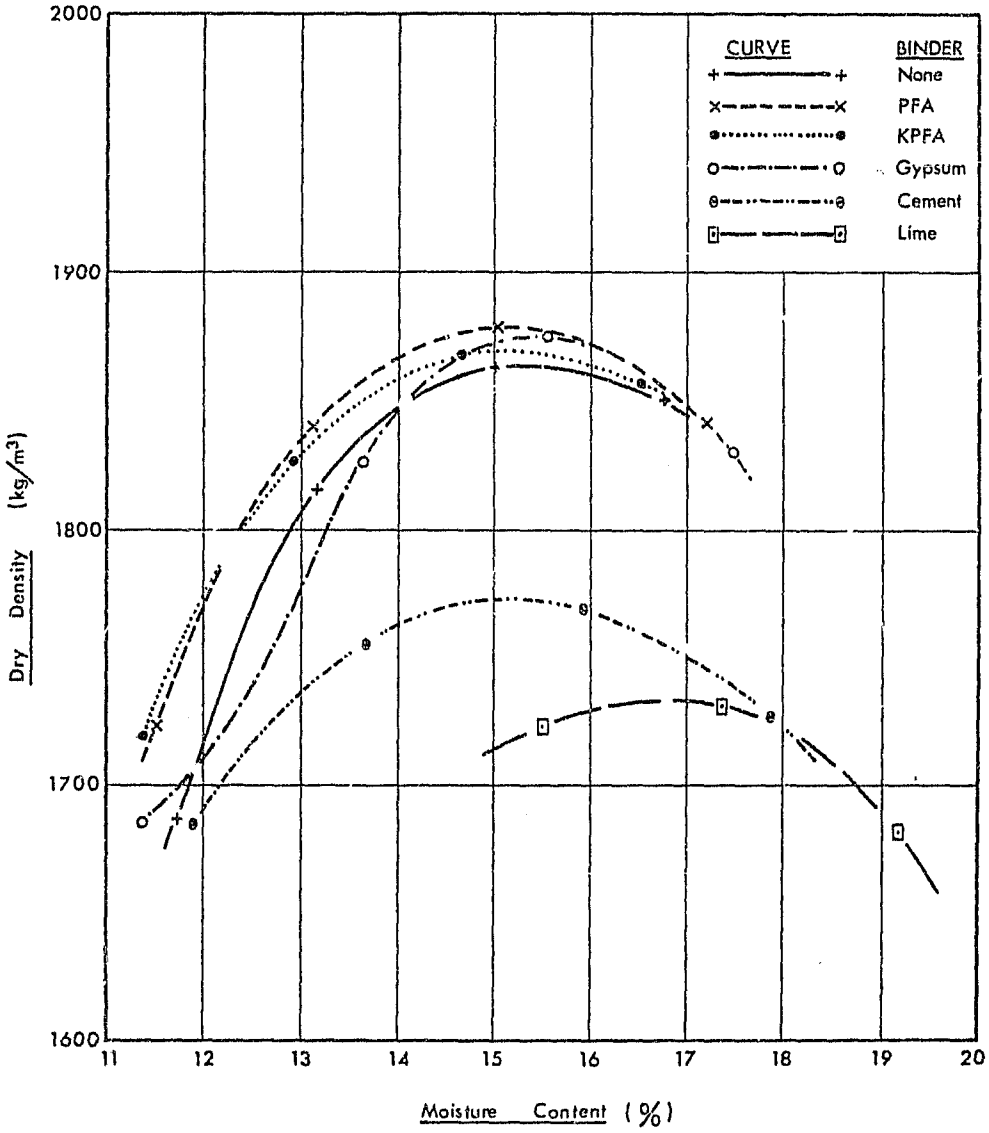


FIG 4.1 The effect of the various Binders on the Moisture Content - Dry Density curve for Soil D
(Mod AASHO effort ; Overall binder content = 3%)

4.3 Effect of the Binders on the Unconfined Compressive Strength of the Soils

4.3.1 Test Procedure

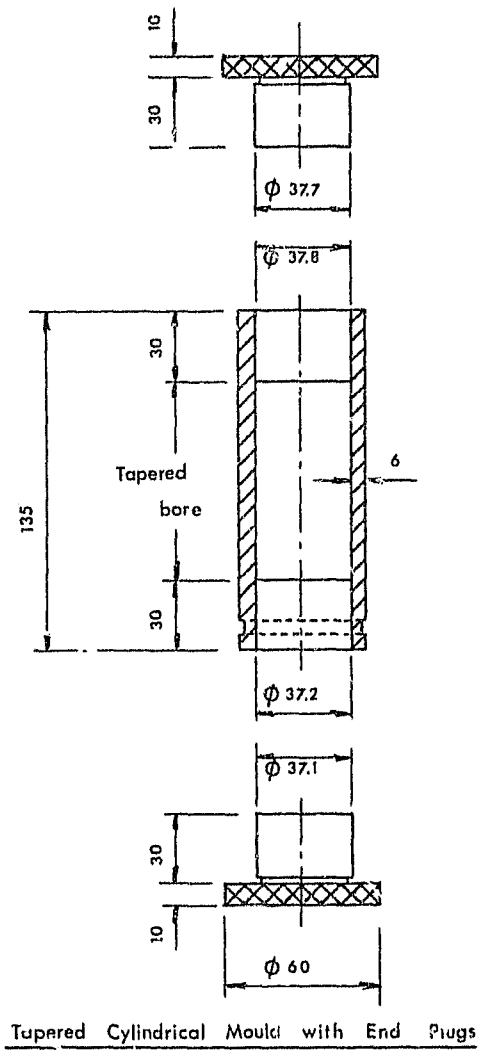
Cylindrical Samples, 36mm in diameter and 75 mm long were crushed in order to determine the unconfined compressive strength of the soils, and the changes in this strength due to the presence of various binders. All tests were carried out at the optimum moisture content and modified AASHO density established for the unstabilized soils.

4.3.2 Preparation of Specimens

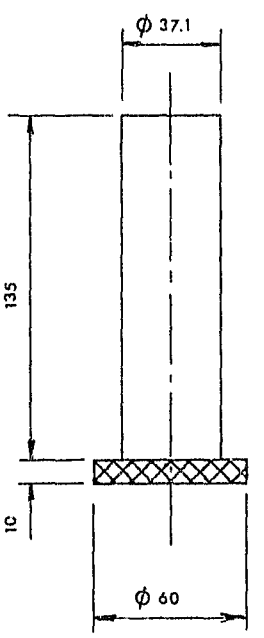
The specimens were prepared in a tapered cylindrical mould which is an adaptation of that illustrated in BS 1924[3] (fig. 20 and fig. 21). A detailed drawing of the mould used is shown in figure 4.2

General Notes on the preparation of Specimens

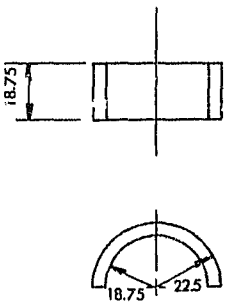
1) Specimens were prepared in groups of three and values given for unconfined compressive strength are the mean of three measurements.
2) Unless otherwise stated:
    a) The specimens were crushed at 28 days. The effect of age on strength gain is discussed in a later section.



Tapered Cylindrical Mould with End Plugs



Plunger

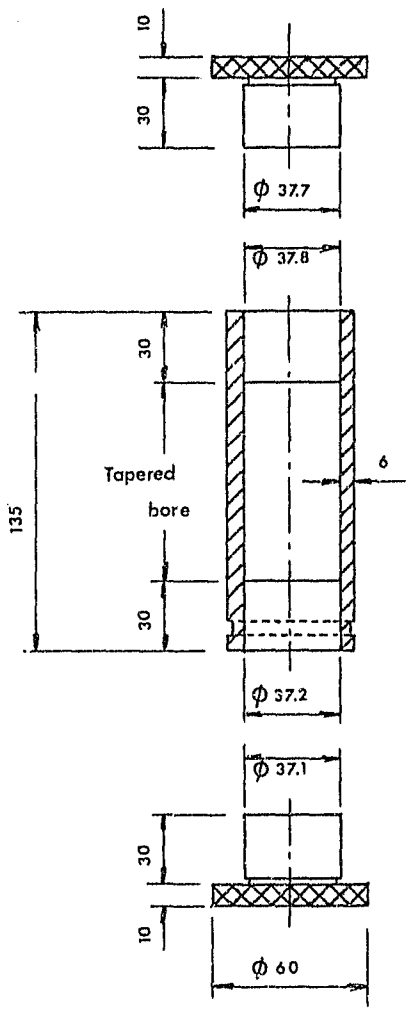


Plug Displacing Collar

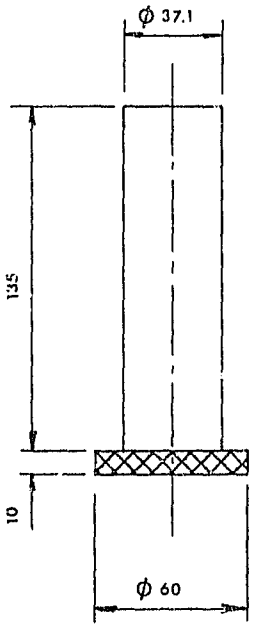
Material - Mild steel

All dimensions in mm

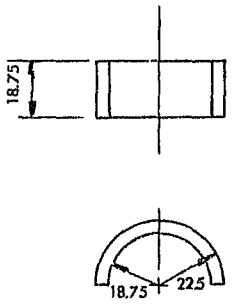
FIG 4.2 Tapered Mould with Attachments



Tapered Cylindrical Mould with End Plugs



Plunger



Plug Displacing Collar

Material - Mild steel

All dimensions in mm

FIG 4.2 Tapered Mould with Attachments

b) A binder content of 3% was used. The reason for choosing this particular content is that it is likely to be the lowest used in practice (Blight[4]). The effect of different binder contents is discussed in a later section.

Preliminary Tests and Calculations

In order that the quantities of soil, water and binder required for each mix could be calculated, the equilibrium moisture content (i.e. moisture content after pre-preparation and storage) of the soil was first determined. The mixes were then calculated as follows.

The known variables are:-

D - 100% modified AASHO dry density for the unstabilized soil.

M - optimum moisture content for the unstablized soil (%)

E - equilibrium moisture content of the soil

V - volume of the tapered mould

The dry mass of material required to make a sample is then DV

i.e. Dry mass of soil required = 0,97 DV

mass of soil required = 0,97 DV (1 + E/100)

mass of binder required = 0,03 DV

$$\text{mass of water required} = \frac{1}{100}(MDV - 0{,}97\ DVE)$$

$$= \frac{DV}{100}(M - 0{,}97\ E)$$

Although three samples were made from each mix, sufficient material was always prepared for four samples to allow for spillages or a possible breakage. For each mix, therefore, the quantities of material required were as follows.

mass of soil required = 3,88 DV (1 + E/100)

mass of binder required = 0,12 DV

mass of water required = DV/25 (M - 0,97 E)

The mass of each finished sample is DV (1+ M/100)

Procedure for making Samples

1) The required mass of soil and binder were placed in a large evaporating dish and thoroughly mixed with a spatula.

2) The required mass of water was then added to the dry soil/binder mixture and this was then also thoroughly mixed.

N.B. In order to check that the correct masses of each constituent were being added to the mix, the cumulative weight was always checked. Spot moisture content checks were sometimes taken. The actual moisture content in these tests was always within 1% of the desired moisture content and the procedure, therefore, satisfactory.

3) The soil/water/binder mixture was then placed in a plastic bag which was kept tightly closed.

4) The required mass of material (i.e. DV (1 + M/100)) was then transferred from the plastic bag to a smaller evaporating dish.

5) The procedure then followed was in accordance with BS 1924[3], viz.

"The appropriate weight of material shall be placed in the mould into which, using a displacing collar, the lower plug has been inserted to a distance of ½in (1.3cm). During filling, the stabilized soil shall be tamped gently and uniformly so that the upper plug can be inserted a distance of ½in (1.3cm). The upper plug can be inserted and the mould assembly placed in the compression device or testing machine. After removal of the displacing collars, pressure shall be applied to the plugs until the flanges are in contact with the barrel of the mould. After the pressure has been maintained for about ½ minute, the load shall be released and the mould removed from the press. The plugs shall then be removed from the mould. The plunger shall then be inserted into the end of the mould having the smaller diameter and the specimen released from the taper by gentle hammering or pressure. The specimen shall then be removed from the mould and weighed to the nearest 1 g".

6) The mass, together with the number of the specimen and

date of casting were then recorded on a piece of paper which was attached to the specimen.

7) As it was necessary to store the specimens at constant moisture content, they were first wrapped in aluminium foil*and then sealed in a coating of wax.

*The wrapping consisted of two circular end discs 36 mm in diameter and a rectangular piece of foil 133 mm x 204 mm. The specimen was placed lengthwise along the shorter edge of the rectangular sheet which was then rolled around the specimens. The circular dics were then placed over the ends of the specimen and the projecting pieces of aluminium foil folded over these discs. A marking pen was then used to number and date the specimen.

8) The specimens were then stored in a constant temperature cupboard for the required curing period (temp. 25°C ± 2°C)

9) After the curing period had passed, the specimens were removed from the constant temperature cupboard and the wax covering and foil wrapping removed.

10) Each specimen was then reweighed and the moisture loss over the curing period determined. This loss was reasonably constant for each soil type. It was always less than 0,1% of the total specimen mass and therefore negligible.

11) The specimen was then tested in unconfined compression in a Losenhausen compression testing machine. This

machine does not have a strain-rate control facility, but it is possible to control the pump speed of the machine. The machine was set to the same pump speed for all tests and the actual strain rate then depended on the resistance of the specimen to deformation. It is estimated that the strain-rate during testing was always less than 5mm/minute.

12) The failure load of the specimen was recorded by a lazy pointer and this reading was noted for each test.

4.3.3 Results of Unconfined Compression Tests

4.3.3.1 Results of tests in which Binder Content = 3% and age at test = 28 days

4.3.3.1. a. Phosphogypsum Binder

The results of the unconfined compression tests in which phosphogypsum was used as a binder are summarised in table 4.2 below.

SOIL	STRENGTH OF UNSTABILIZED SOIL (kPa)	28 DAY STRENGTH OF STABILIZED SOIL (kPa)	STRENGTH GAIN (%)
A	316	407	28,8
B	422	511	21,1
C	1004	1007	0,3
D	313	644	105,8
E	1132	1317	16,3

TABLE 4.2 RESULTS OF TESTS USING PHOSPHOGYPSUM BINDER

The strength gain in all soils was very small with the exception of soil D where it was 106%. There does not appear to be any obvious explanation for this response. From what was said in section 1,5,it was possibly expected that some of the soils tested in this programme could be effectively stabilized using phosphogypsum. The phosphoric acid content of the material was far too low, and the hydrofloric acid would probably not be effective as all the soils contained kaolinite, an aliminium rich clay. Stabilization with phosphogypsum will have to be investigated in more detail,but this will be discussed later.

4.3.3.1 b. Pulverised Fuel Ash Binder

The results of the tests on which the Darling and Hodgson PFA (PFA) and Kelvin Power Station PFA (KPFA) are used in combination with cement and lime as binders are summarised in figures 4.3 to 4.7.

Blight[4] defined the optimum binder for a soil as "that additive, which when combined with a soil will produce the largest strength for a given cost". Keeping other variables constant (i.e. compaction cost, mixing and spreading costs etc.) he found the optimum binder for each soil type in the following manner.

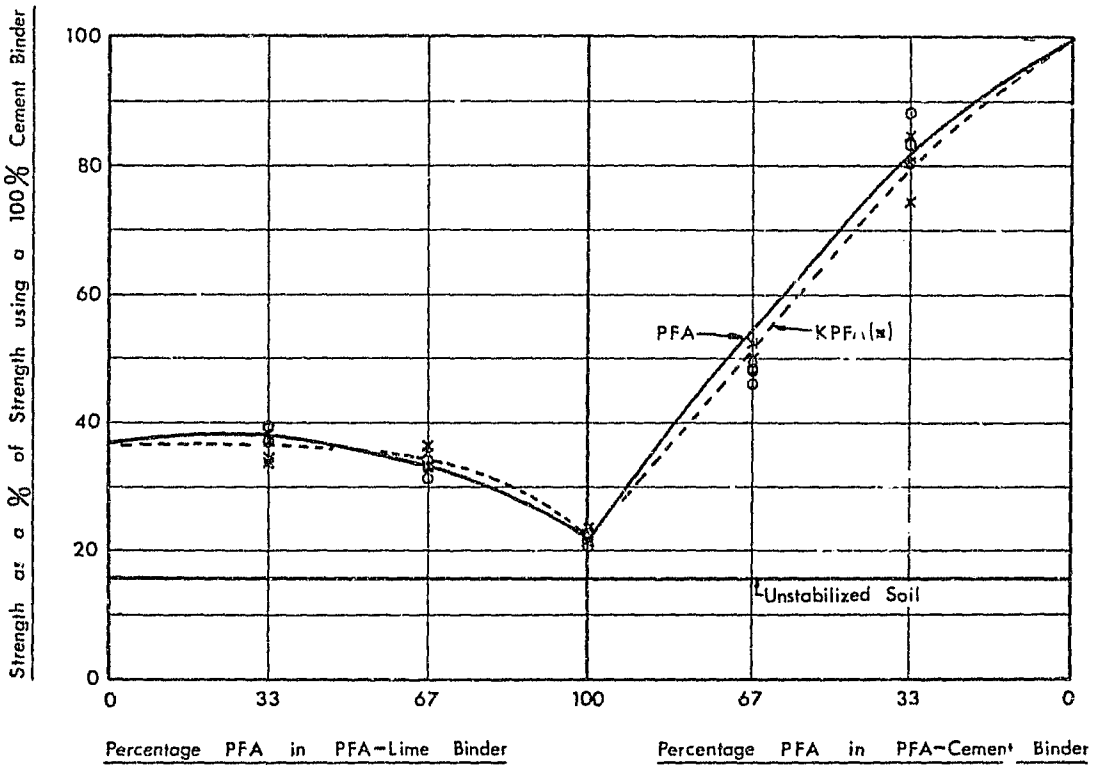


FIG 4.3 Relation between Strength and Binder Composition - SOIL A

(Overall binder content = 3%)

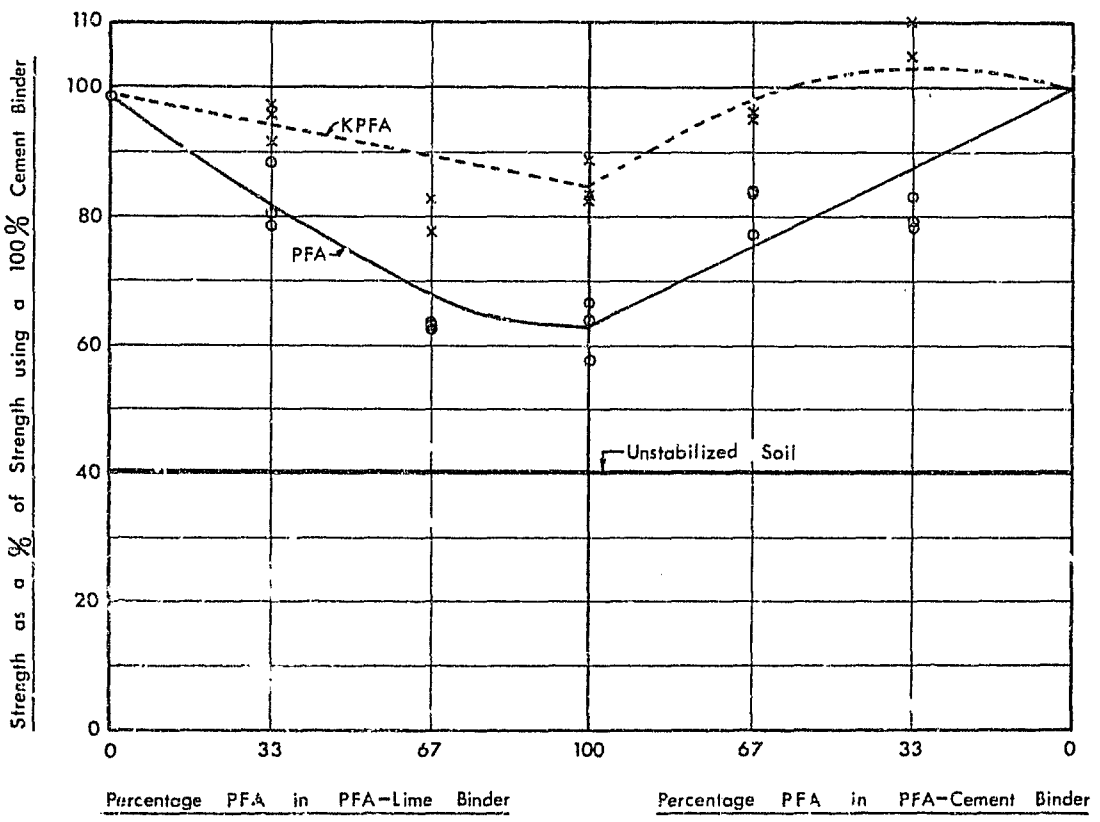


FIG 4.4 Relation between Strength and Binder Composition - SOIL B

(Overall binder content = 3%)

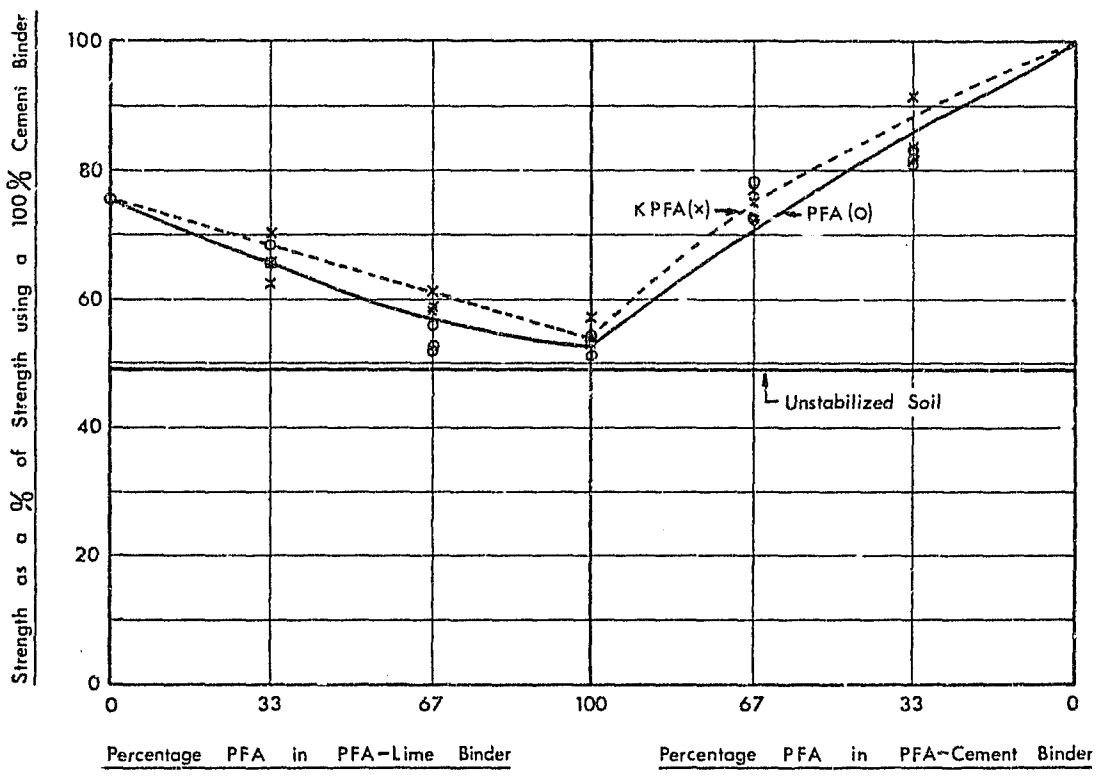


FIG 4.5 Relation between Strength and Binder Composition - SOIL C

(Overall binder content = 3%)

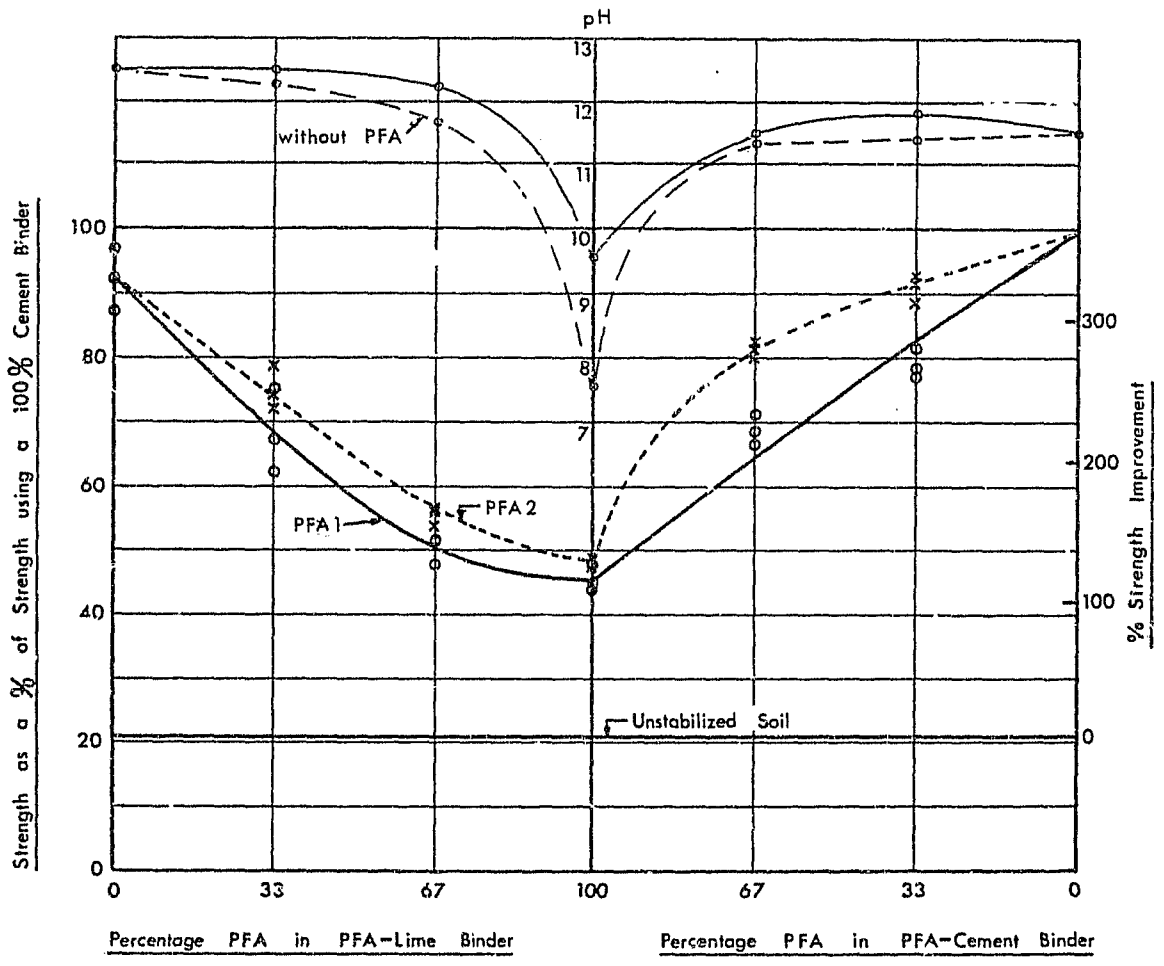


FIG 4.6 Relation between Strength and Binder Composition - SOIL D

(Overall binder content = 3%)

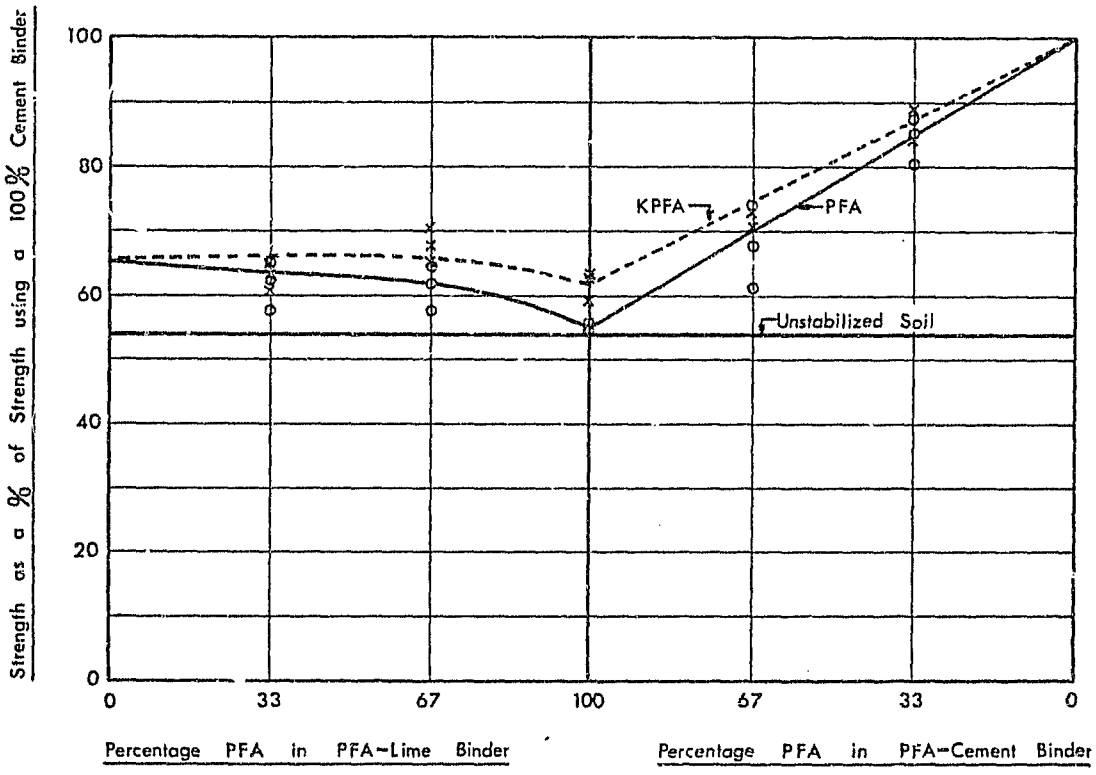


FIG 4.7 Relation between Strength and Binder Composition - SOIL E

(Overall binder content = 3%)

1) Curves relating binder composition to strength (similar to those shown in figs. 4.3 to 4.7) were plotted. (Blight used a 6 to 7 hour compaction delay which was not used in this testing programme).
2) He defined "relative strength" for any particular binder as the strength obtained using that binder expressed as a percentage of the strength obtained using the portland cement binder.
3) He defined "relative cost" for each binder composition as the cost of that binder expressed as a percentage of the cost of an equivalent mass of portland cement.
4) He then defined a strength-cost index as

$$\frac{\text{relative strength}}{\text{relative binder cost}}$$

For any particular soil, the optimum binder is then that binder for which the strength-cost index is a maximum.

A similar approach will be used to analyse the results plotted on figs. 4.3 to 4.9, but firstly the limitations of this approach will be discussed.

1) In all the tests, the moisture content used was the optimum moisture content at 100% modified AAHO effort for the unstabilised soil. Various researchers have shown that this is not usually the optimum moisture content for the stabilized soil, for maximum strength gain. In future testing programmes, the effect of initial

moisture content on strength gain should be investigated.

2) The above method also assumes that the cost of compaction would be equal for each soil binder combination. A weakness of the approach used in this testing programme is that soil-binder mixtures were compacted to a constant density and not using a constant compactive effort. As stated in 5.2 above, it is probably impossible to obtain the densities used with the lime stabilized soils by dynamic compaction due to compaction failures. With static compaction, however, no such problem arises. The method used, then, is probably biased against the PFA, and towards the lime and cement.

3) The cost of spreading and mixing binders is assumed equal in the above method. The efficiency required to effectively mix cement, cement/PFA or lime/PFA binders with soil is higher than that required for lime stabilization. (Davidson[25] states that due to lime migration large clods can be stabilized given sufficient time. The same effect would not be obtained with the alternative forms of stabilization.)

4) The "optimum binder" found in the method used by Blight is dependent on the age at test. This is shown in fig. 4.8 where instantaneous, 7 day, and 28 day strengths are plotted. The effect is more noticeable with the PFA binders due to the slow strength-gain reactions.

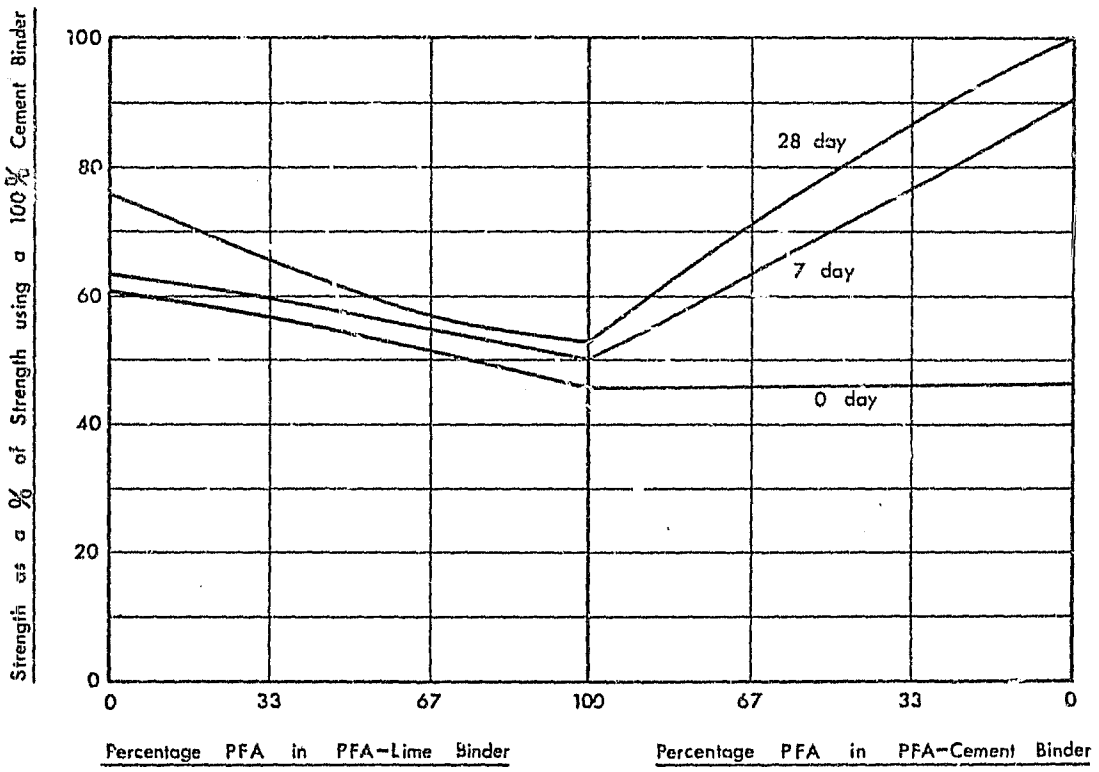


FIG 4.8 Relation between Strength and Binder Composition - SOIL C

(Overall binder content = 3%)

5) The "optimum binder" is also dependent on the overall binder content. This can be seen by comparing figs.4.6, 4.9 and 4.10. These are curves for soil D with different binder contents. (The effect is probably more evident in the lime - PFA stabilization. As the initial lime consumption of the soil is satisfied, other forms of strength gain come into play).
Another drawback is that until the initial lime consumption of the soil is satisfied, the PFA in the soil will not react chemically. The addition of lime to a soil until this critical value will not have any marked effect on the strength of the soil, and so the method may indicate that a binder containing less lime is optimum. A possible solution to this problem would have been to perform the unconfined compression tests on soaked samples (as opposed to samples tested at optimum moisture content for the unstabilized soils). The more plastic soils (i.e. soils containing less lime) would have been weaker and this problem would not have arisen.

6) As pointed out by Blight, the cost of the binders depends on the geographical location of the site, as transport costs are included in the binder price. An "optimum" binder for a particular soil will therefore only be optimal in a certain area.

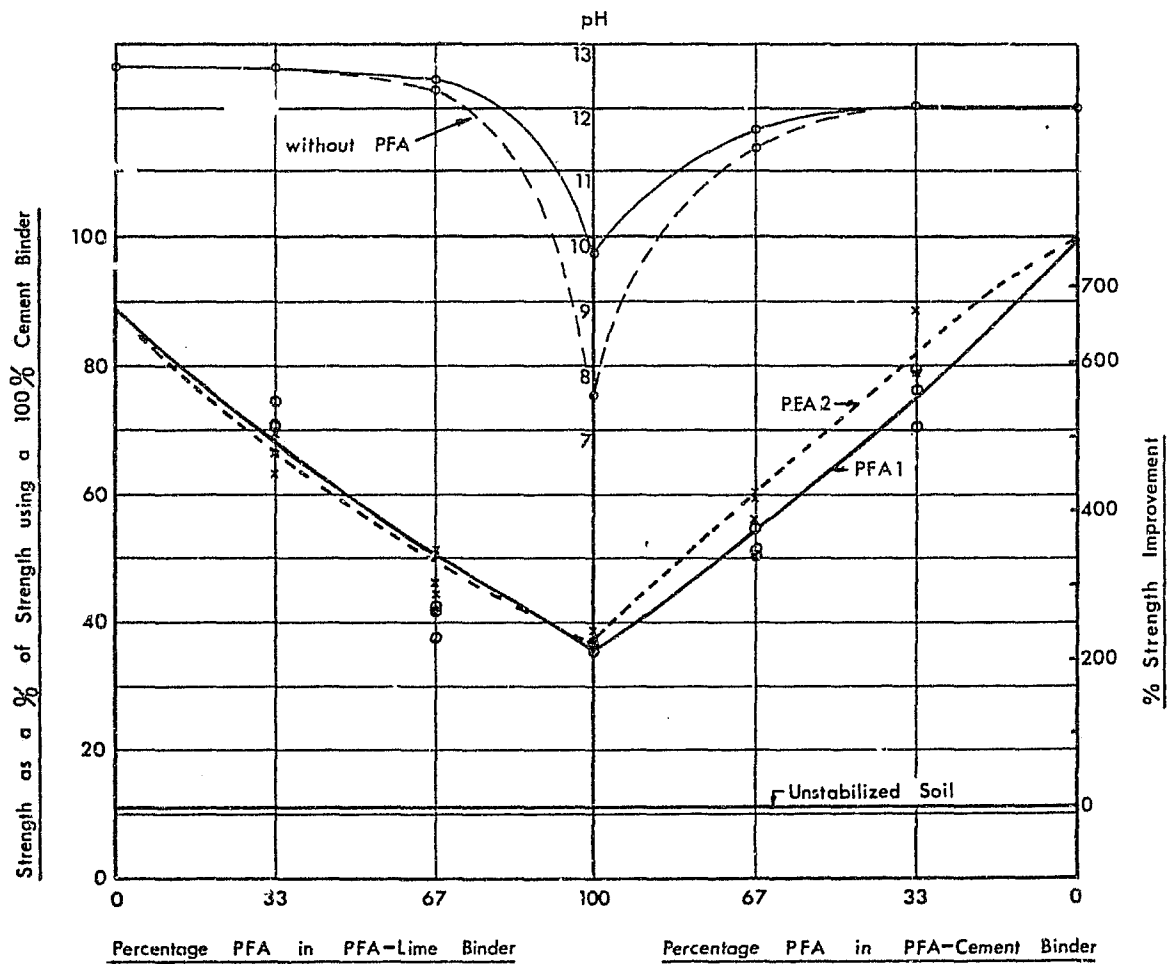


FIG 4.9 Relation between Strength and Binder Composition - SOIL D

(Overall binder content = 6%)

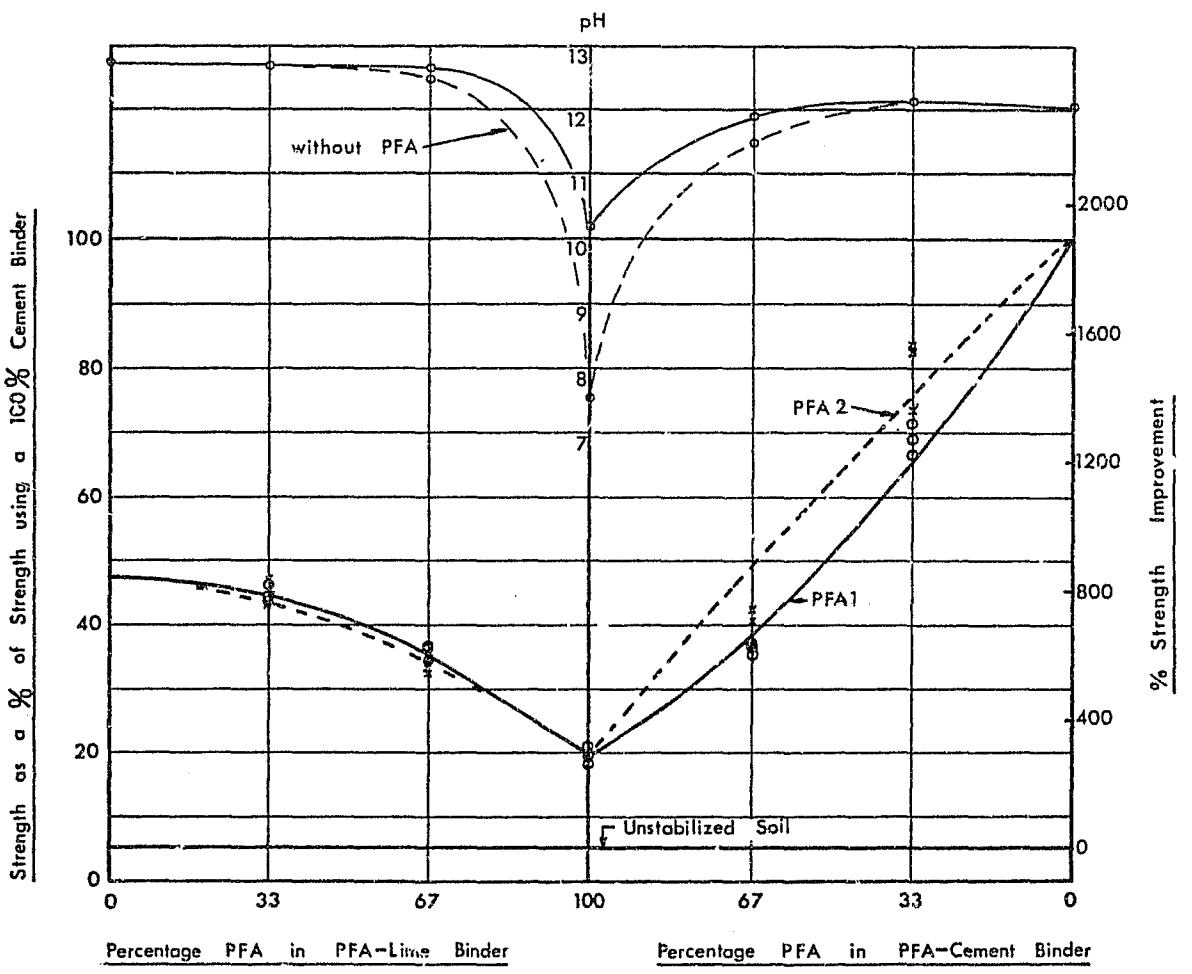


FIG 4.10 Relation between Strength and Binder Composition - SOIL D

(Overall binder content = 9 %)

Binder Costs

The selection of an optimum binder for each soil type will be limited to the Darling and Hodgson ash as no costs are available for Kelvin ash. The costs for the binders will be costs for the binder delivered to site in the Johannesburg area. (January, 1979).

BINDER	CEMENT	LIME	PFA
COST/TONNE	R35 *	R40 †	R17.50 °
RELATIVE COST	1	1.14	0,5

* Approximate cost obtained from Cement Marketing Organization.

† " " " " Northern Lime Company.

° Bates [10] says that PFA will probably sell at half the cement price.

TABLE 4.3 COST OF BINDERS

Relative costs of mixed binders are found by interpolating linearly between the costs of the two constituents. The relationship between strength-cost index and binder composition is plotted on fig. 4.11

Observations and Conclusions regarding optimum binder composition.

Fig.4.11indicates that the optimum binder for soils B, C and E is PFA and for soils A and D is cement. Re-examination of the results of the tests shows that the strength increases in soils B, C and E using a pure PFA binder were as follows

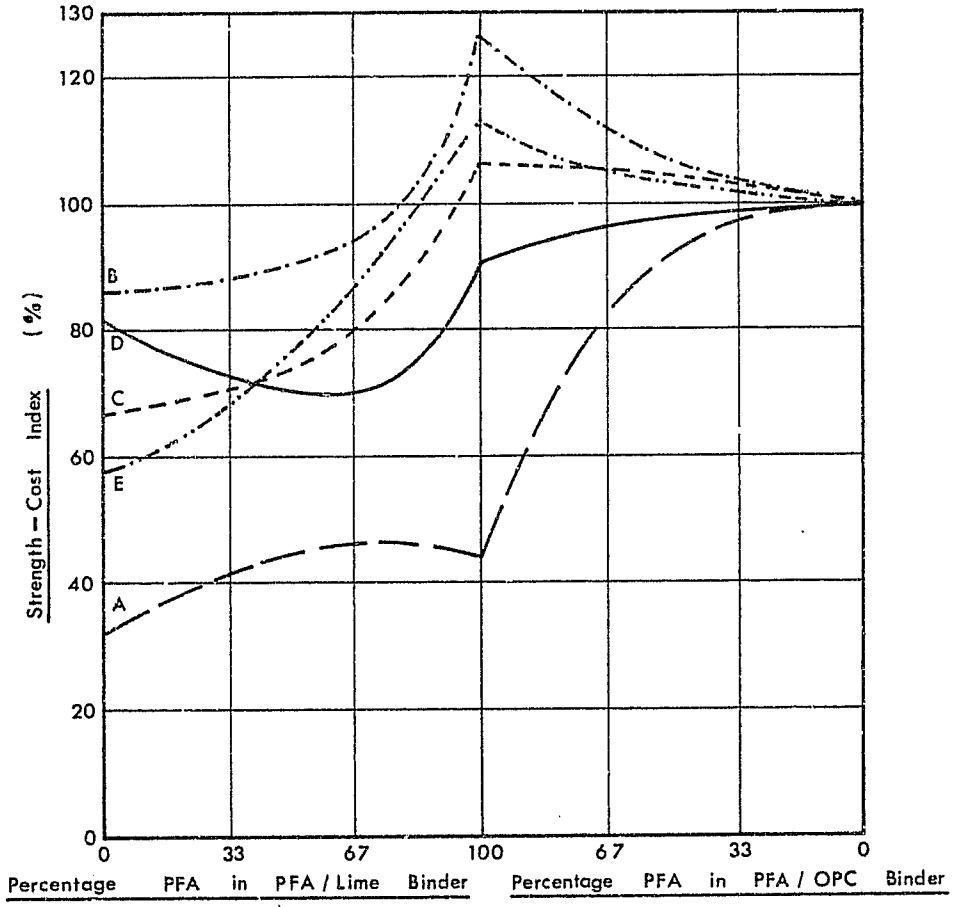


FIG 4.11 Relation between Strength – Cost Index and Binder Composition

SOIL	STRENGTH GAIN WITH 3% PFA (%)
B	56
C	8
E	2

Obviously, the PFA in the last two of these soils has produced negligible strength increases. Examining this extreme, if no binder was added, the relative cost would be zero and the unstabilized soil would always give the maximum strength-cost index.

Another method of analysing the results would be to redefine "relative strength" as strength increase, expressed as a percentage of strength increase using the cement binder. These results would obviously be more realistic, but the instantaneous strengths are only known for soil C (see fig. 4.8 - both instantaneous and 28 day strengths are plotted) and this cannot, therefore, be done here.

Results using Kelvin Ash

The strengths obtained using Kelvin ash were generally better than with the other ash. This is probably an indication of the different compactive effort used (i.e. in the static compaction, the machine load required to compact samples containing Kelvin ash was generally higher than for the other ash), rather than superior pozzolanic properties. With soil B (see fig. 4.4) an extremely good result seems apparent regarding cement/KPFA stabilization. All of the samples made with

with soil B, however, exhibited inconsistent results, possibly due to the presence of organic matter in the soil. In table 2.1, it can be seen that soil B had an organic content of 1,2% which, although higher than soils A, C and D, compares with that for soil E (1,4%). The soil B samples, however, had a foul smell after the 28 day curing period which was not as evident with soil E. The good results indicated of fig. 4.4, therefore, may not be significant.

4.3.3.2. The Effect of Binder Content on Unconfined Compressive Strength

4.3.3.2. a. Phosphogypsum Binder

The gypsum used in the tests described in 4.3.3.1 a was oven-dried gypsum, the moisture content of which had been adjusted to 10% by the addition of return water. In order to determine the relative importance of the gypsum and the return water, and to determine the possible effects of oven-drying, the tests in which binder content was varied were repeated using the following binders.

a) oven-dried gypsum

b) air-dried gypsum

c) gypsum return water

d) oven-dried gypsum + 10% return water.

With the solid binders i.e. a, b and d, binder contents of 3%, 6% and 9% were used. With C, where a liquid binder was

used, the return water was merely added to the water used to bring the soil to optimum moisture content in the following quantities

20%, 40% and 63% (of O.M.C.)

All tests were performed on soil E.

The results of these tests are presented in fig. 4.12

The reason that the air-dried gypsum shows a strength loss is that its return water content (25%) was neglected in the sample preparation. This increased the water content of the sample from 21% to approximately 24% (for a binder content of 9%). The same effect probably causes the leveling off of strength in the sample that contains 10% return water.

Once again, it would be advisable to determine the effect of initial water content on strength gain in future tests.

The results for the other binders, viz. gypsum return water and oven-dried gypsum, were not startling, but strength increased with increasing binder content. Otherwise, the same comments made in 4.3.3.1 a apply.

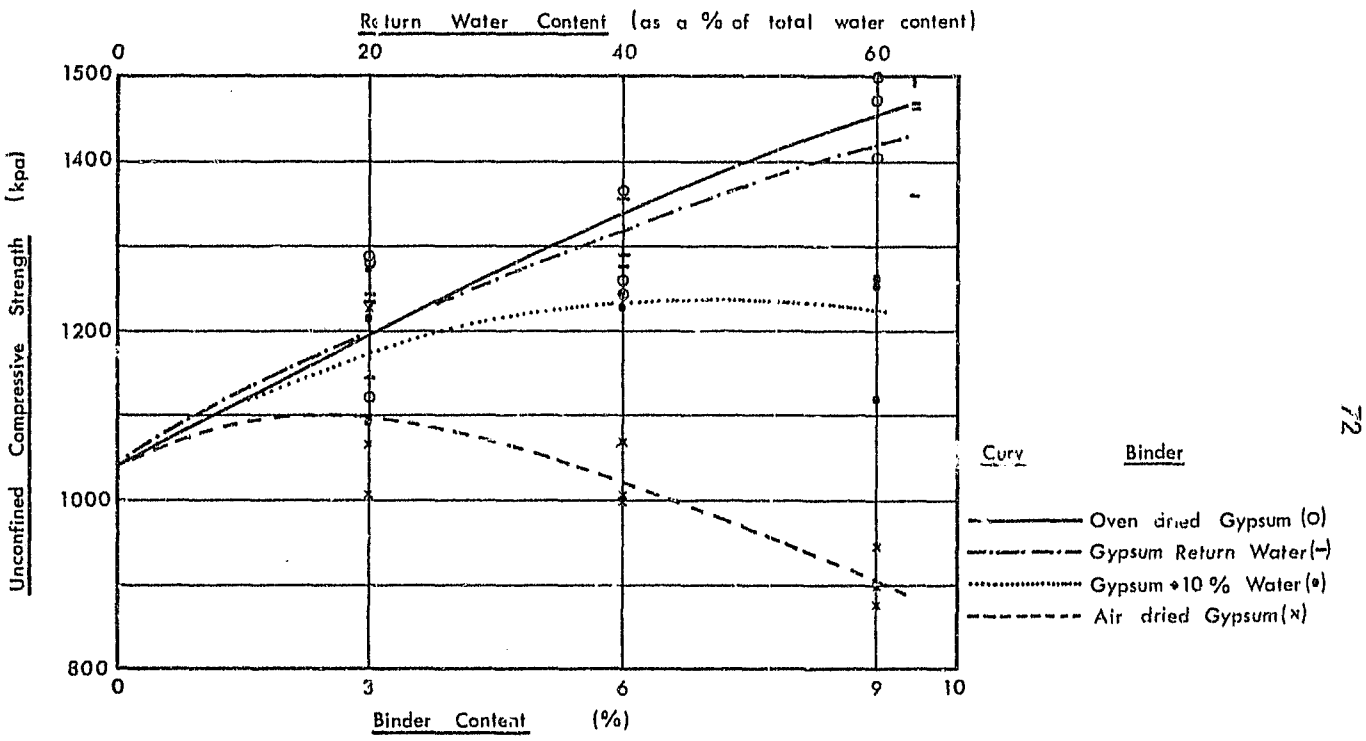


FIG 4.12 The effect of Binder Content - Phosphogypsum Binders

4.3.3.2 b Pulverised Fuel Ash Binder

The effects of binder content on unconfined compressive strength were studied in a series of tests on soil D. The results of these tests are presented in fig. 4.13 to 4.18. The pH values as determined by the Eades test procedure are also plotted on figures 4.17 and 4.18.

From the above mentioned figures, the following conclusions can be drawn.

1) The addition of PFA to the soil appears to be beneficial up to a binder content of approximately 9% where the curve begins to level off.

2) The increase in strength with binder content is extremely marked for the cement and cement/PFA binders. This is probably due to a reduction in the cement/water ratio. This is an example of a case in which a lower initial water content would probably be beneficial. (particularly with low binder contents). The lower density achieved in compaction would be offset by increased strength gain due to cementitious activity.

3) From fig. 4.14 and 4.16 it can be seen that the soil is becoming saturated with lime at lime contents of approximately 9%. (Eades test indicate an initial lime consumption of 3%). The shape of the curve in which the

lime ash ratio is 1:2 is promising. If the strength obtained using a 3% lime binder (1,3 MPa) is compared to the strength of the soil with 3% lime and 6% PFA (2 MPa), it can be seen that the PFA in the soil is having some effect. (The nature of this strength increase, however, is not apparent) It can also be noted from fig. 4.18 that in two of the samples batches tested, the amount of lime present in the soil exceeded the initial lime requirement. No additional strength increases were measured in the tests, however.

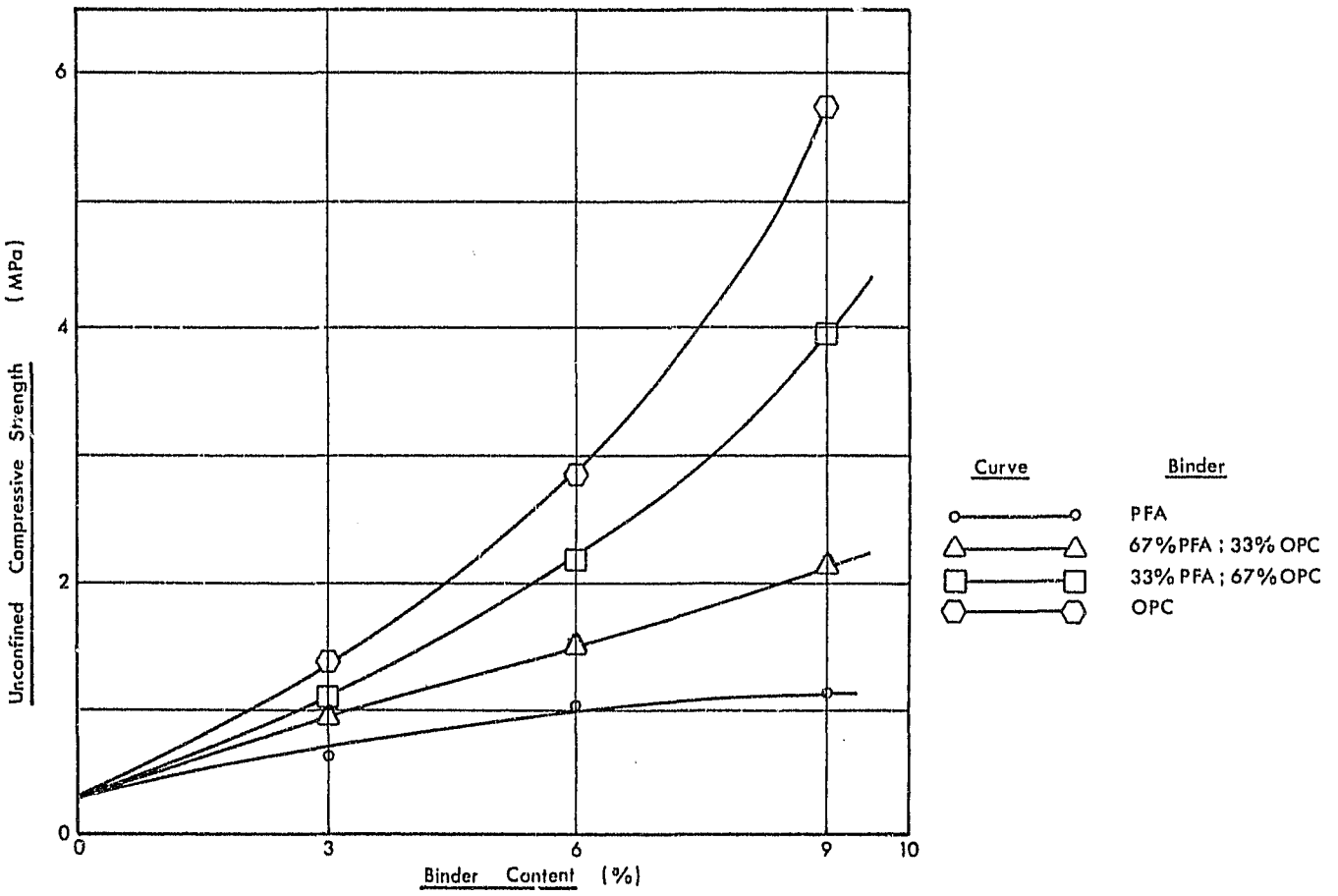


FIG 4.13 The effect of Binder Content on Strength - PFA / OPC Binders
(Soil D)

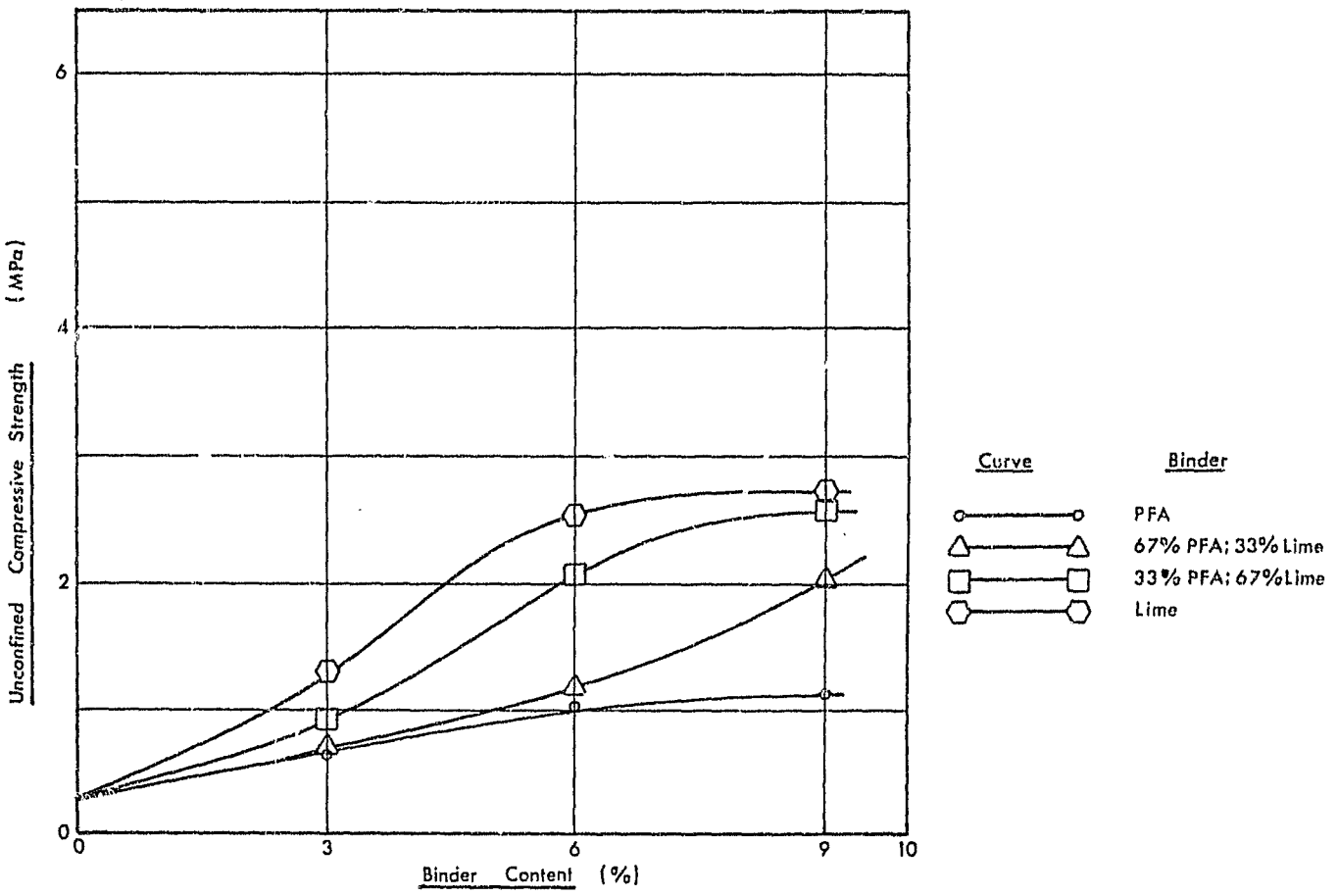


FIG 4.14 The effect of Binder Content on Strength PFA / Lime Binders (Soil D)

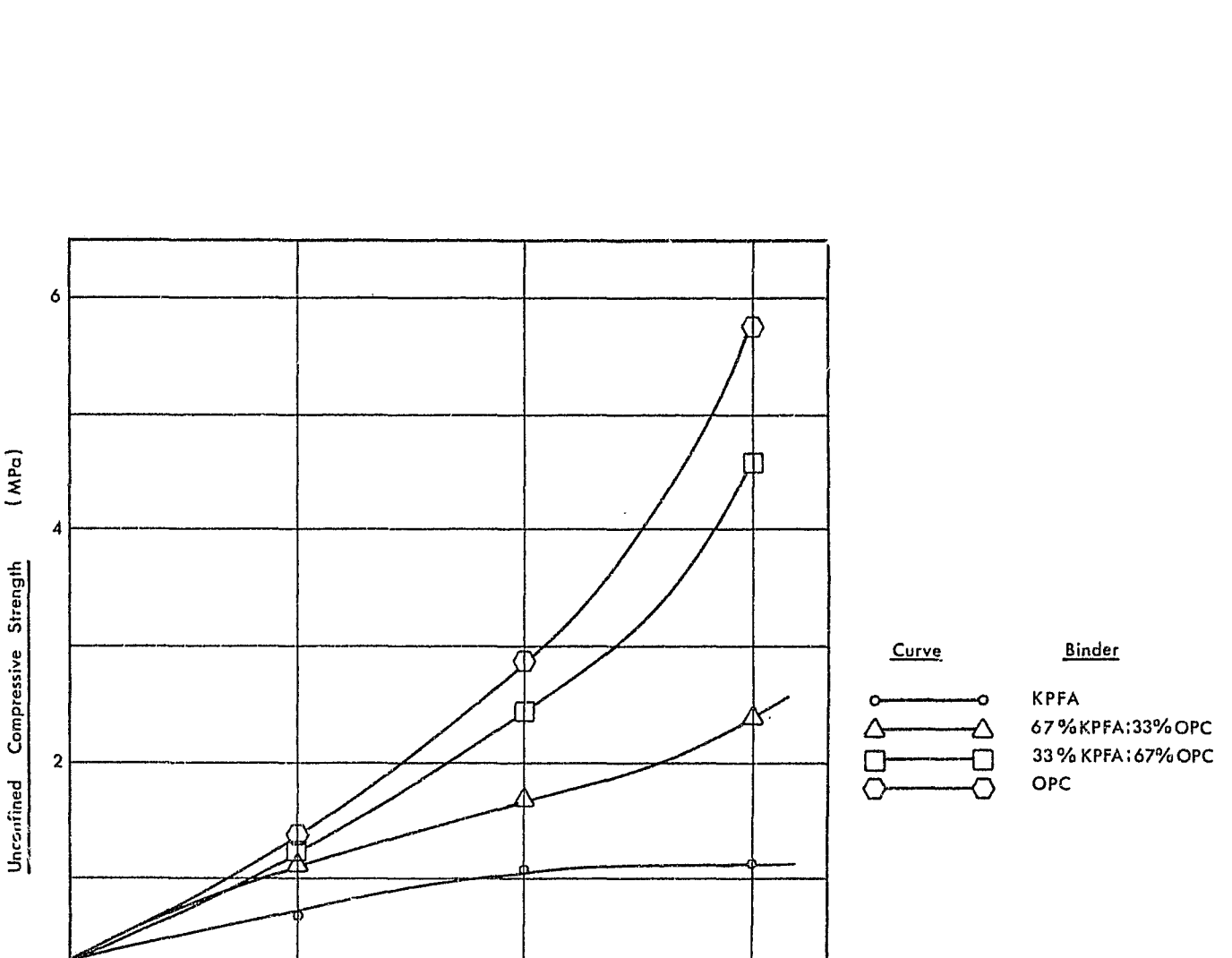


FIG 4.15 The effect of Binder Content on Strength KPFA / OPC Binders (Soil D)

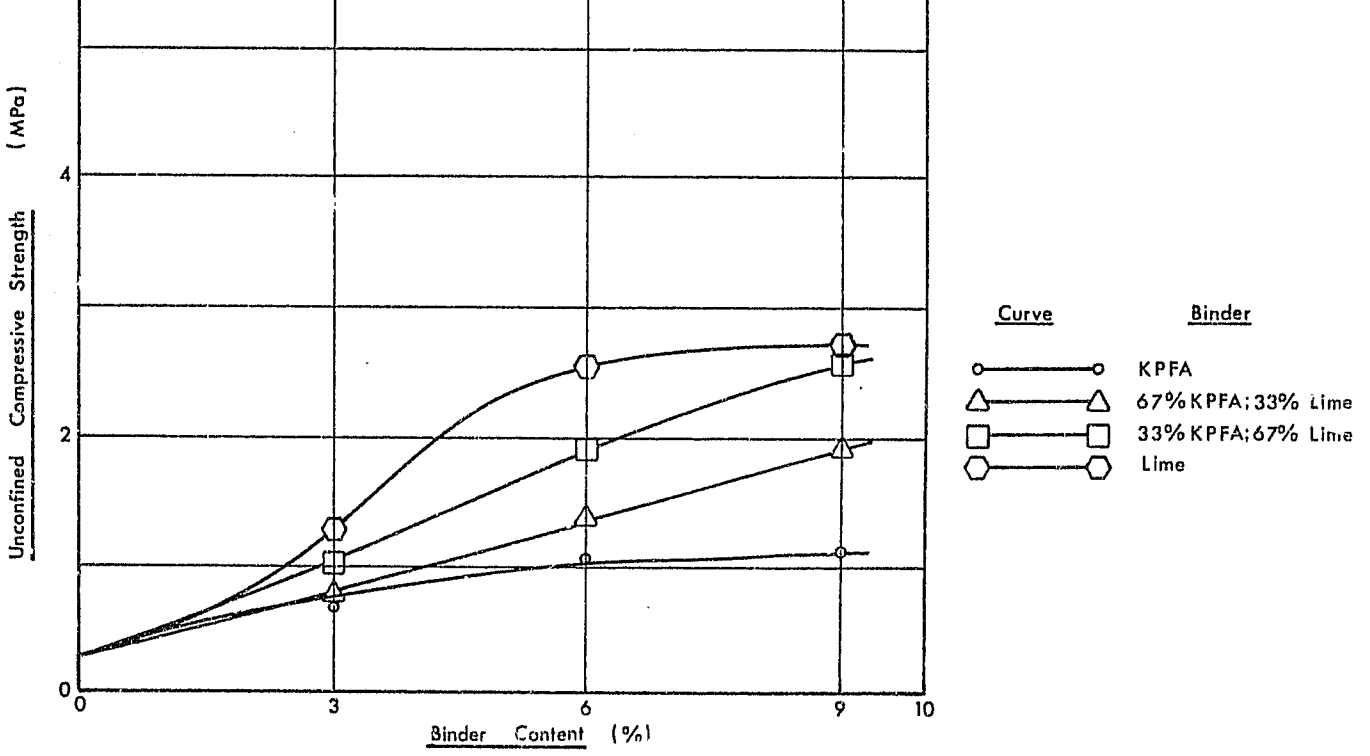


FIG 4.16 The effect of Binder Content on Strength KPFA / Lime Binders
(Soil D)

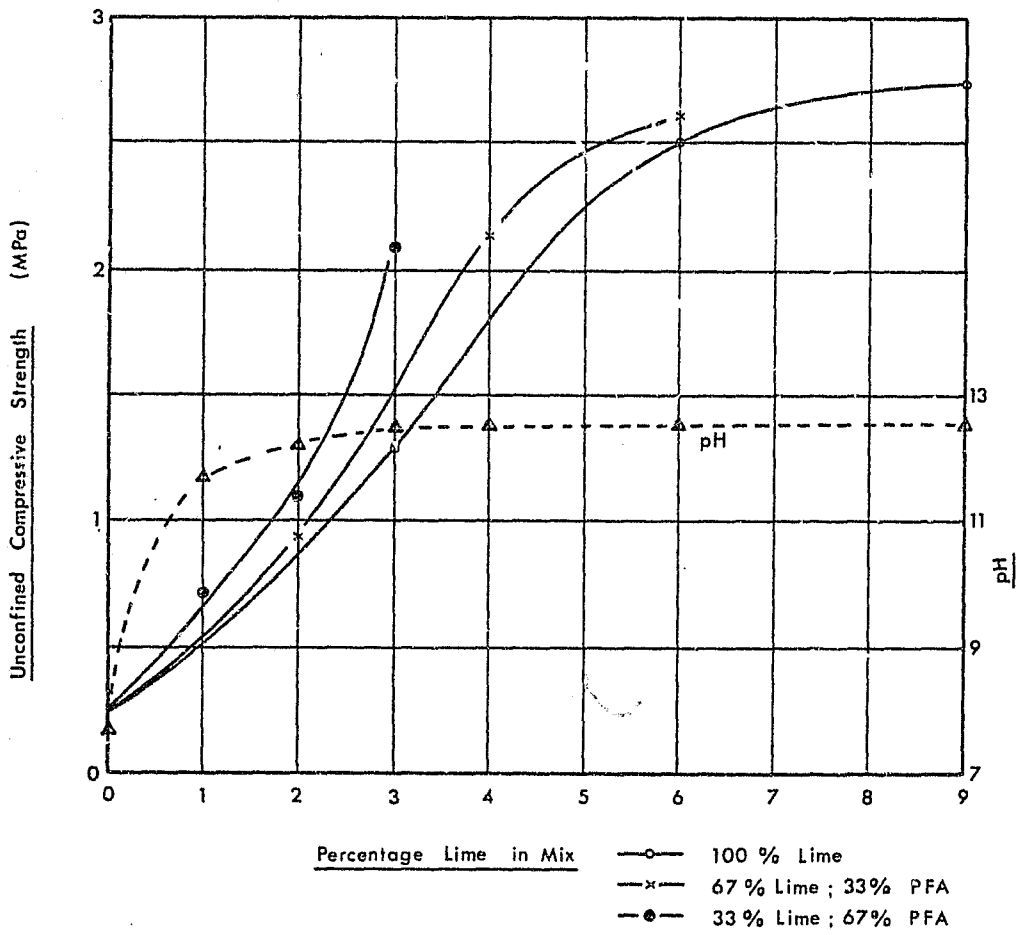


FIG 4.17 The effect of Lime Content on Strength

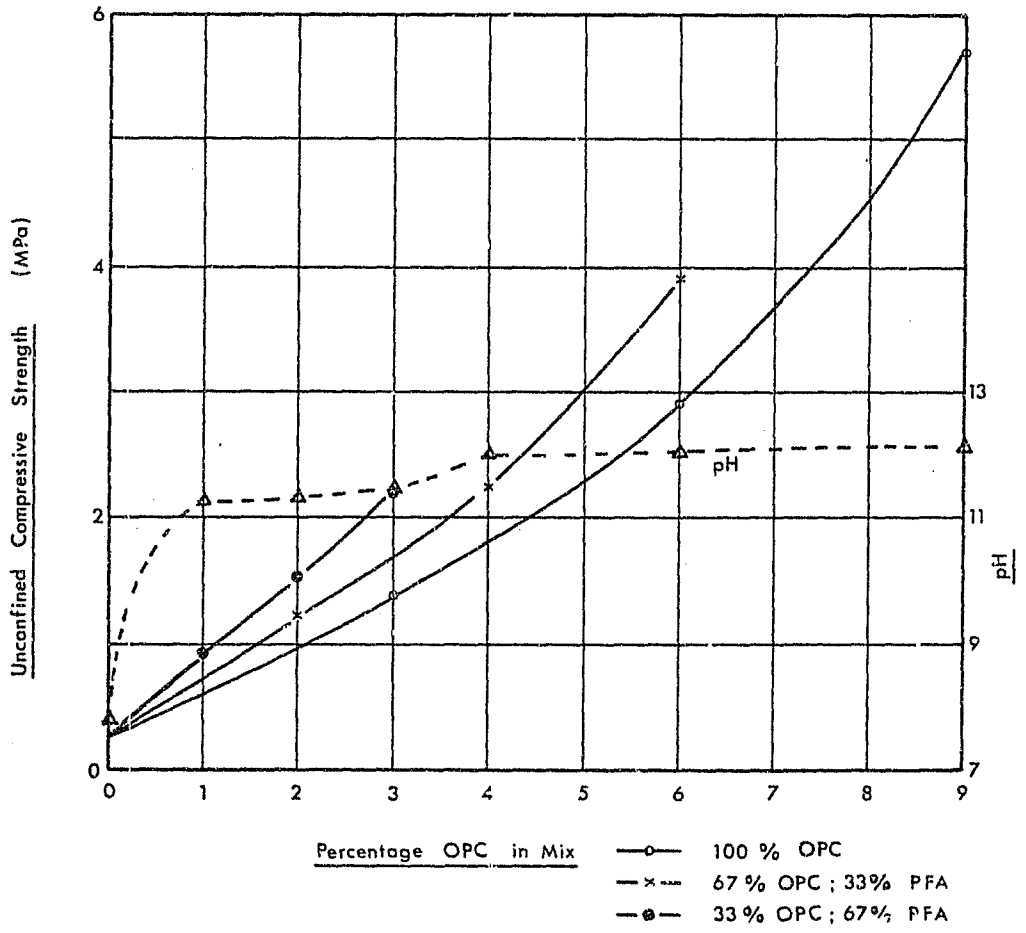


FIG 4.18 The effect of Cement Content on Strength

4.3.3.3 The Gain in Strength with Age for the Various Binders (binder content = 3%)

4.3.3.3 a Phosphogypsum Binder

In order to study the gain in strength with time, a number of identical samples were made using soil E which were tested at various ages. The resulting age-strength curve as shown in fig. 4.19.

It can be seen from fig. 4.19 that after the soil had cured for 3 days, no further strength increases were observed. This result is somewhat surprising when compared with those plotted on fig. 1.7 where there was a marked increase in strength from 7 to 14 days.

4.3.3.3 b Pulverised Fuel Ashes

In order to study the gain in strength with time, a number of of identical samples were made using soil C and tested at various ages. The age-strength curves for the Darling and Hodgson ash binders are shown in fig. 4.20, and for Kelvin ash binders in fig.. 4.21

The following conclusions can be drawn

1) For the pure ash binders, strength increases were small and ceased after 7 to 14 days.

2) The lime and cement binders continued to gain strength right up to 60 days.

3) The lime/ash binders gained strength up to 28 days. After this period, the Darling and Hodgson ash/lime stabilized soil maintained a consistent strength, whereas the Kelvin ash/lime stabilized soil appeared to lose strength. This strength loss is not significant as an examination of the earlier strengths (3,7,14 days) showed that the results appeared to be inconsistent there as well.

4) The Darling and Hodgson ash/cement stabilized soil gained strength up to 28 days. The Kelvin ash/cement stabilized soil continued to gain strength after 28 days.

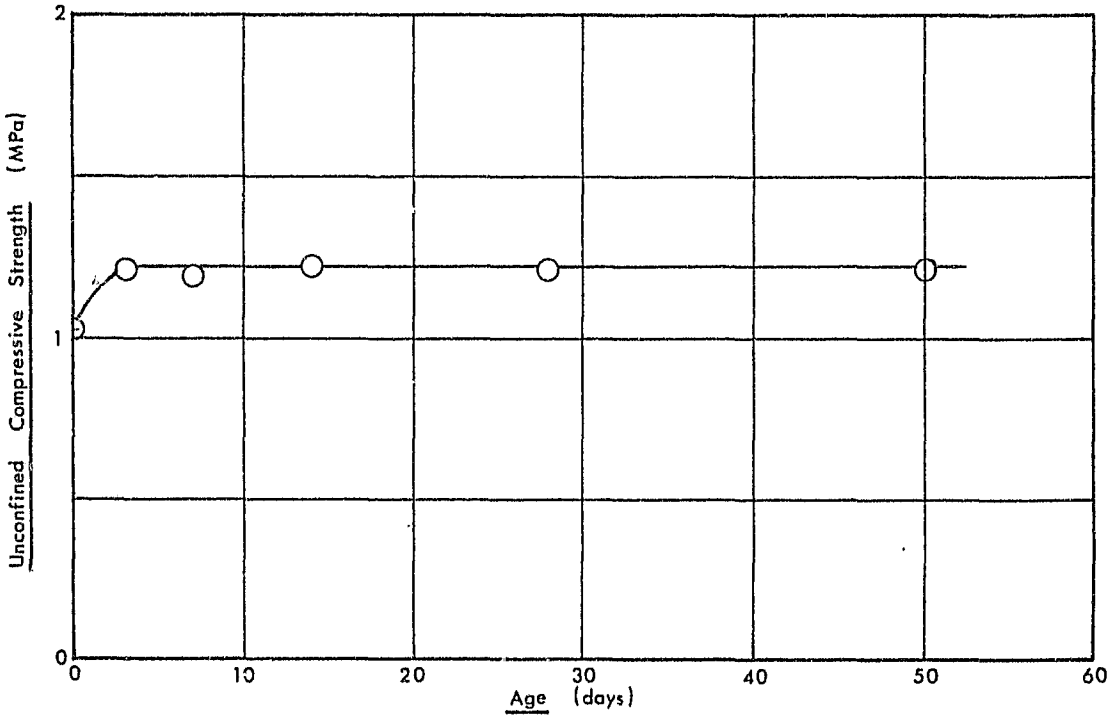


FIG 4.19 The effect of Age on Strength Phosphogypsum Binder (Soil E)

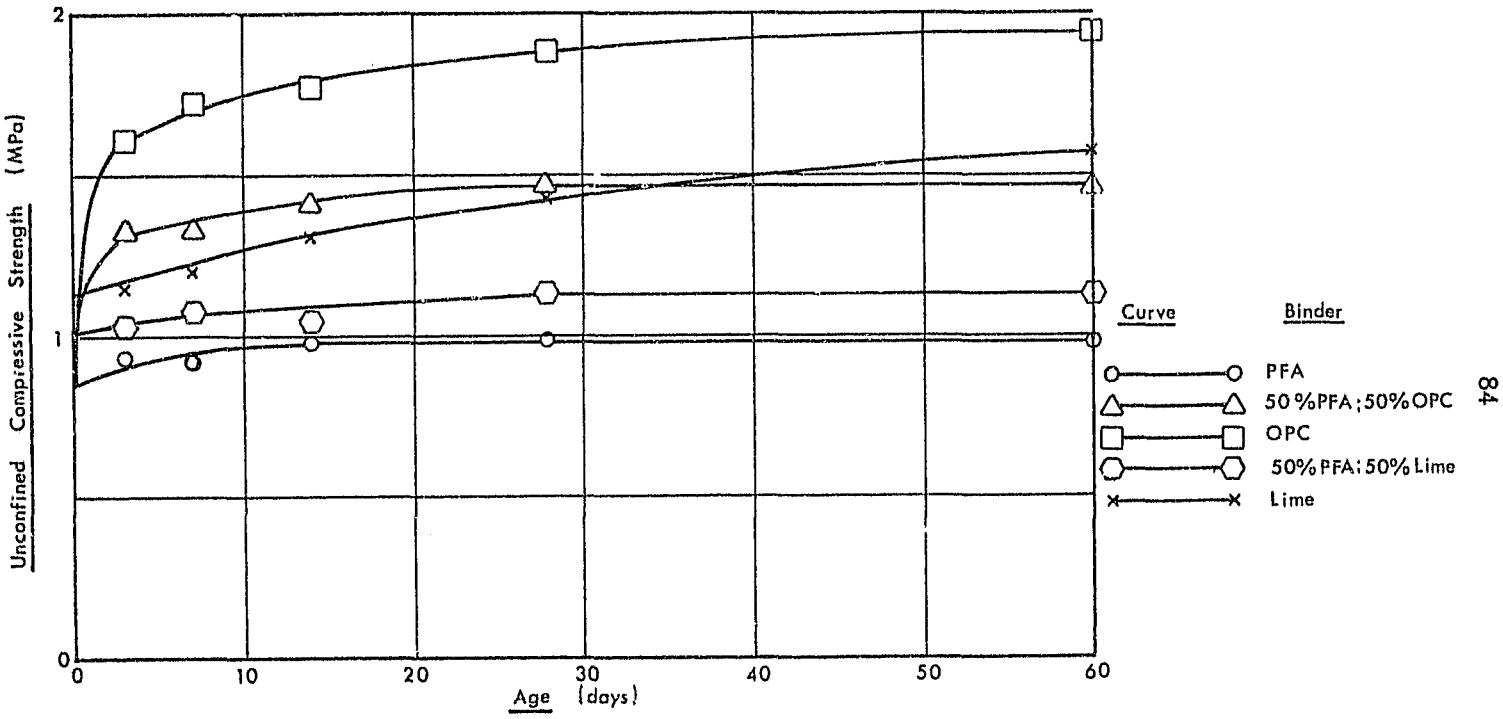


FIG 4.20 The effect of Age on Strength PFA Binders
(Soil C)

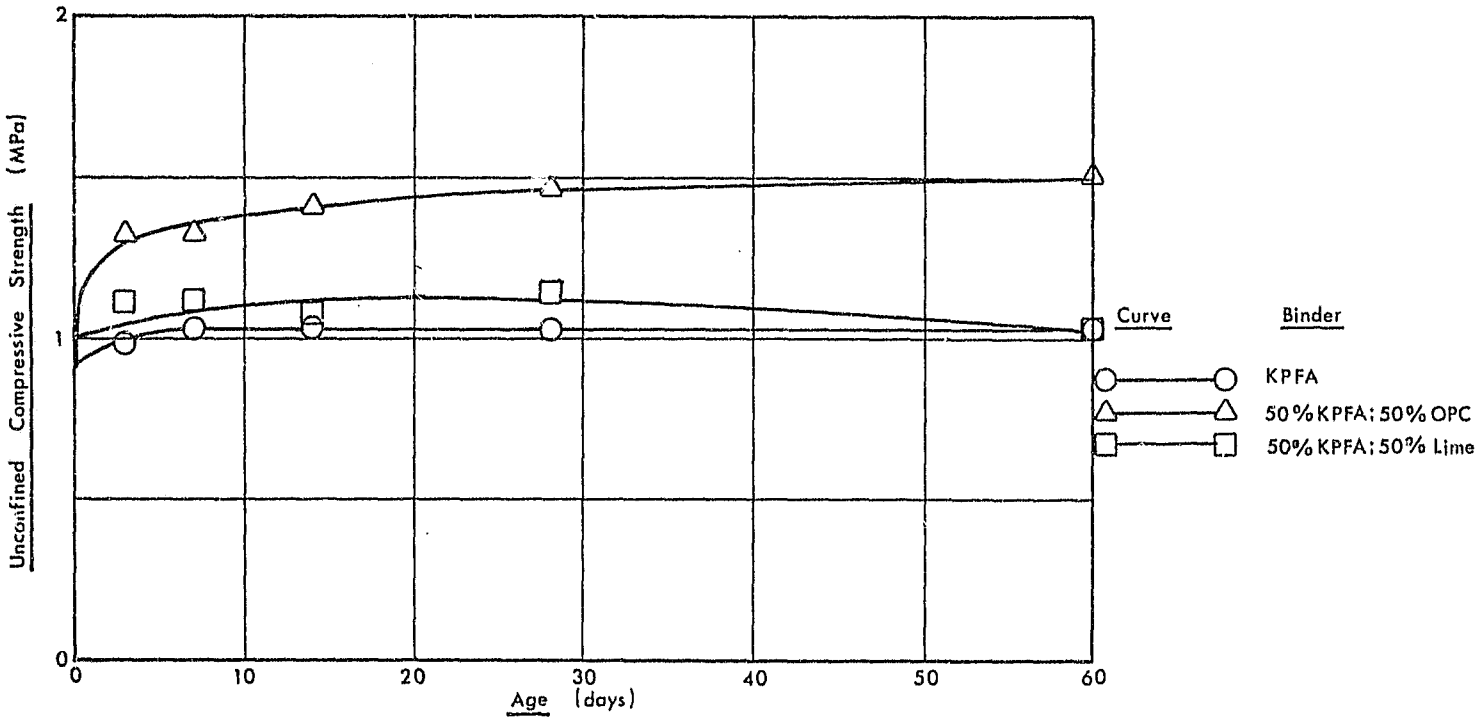


FIG 4.21 The effect of Age on Strength KPFA Binders
(Soil C)

4.4 The Effect of Milling PFA on strength gain

As stated in 3.11, samples of both ashes were milled for 2 hours in a laboratory ball mill in an attempt to upgrade them. A limited no. of tests were carried out on soil C using these milled ashes and the results are shown below. (binder content = 3%)

	BINDERS (all mixtures 50:50)			
AGE AT TEST	PFA/CEMENT	PFA/LIME	KPFA/CEMENT	KPFA/LIME
14 days	1519(1414)	1146(1041)	1492(1412)	1093(1035)
28 days	1563(1476)	1266(1134)	1437(1476)	1172(1134)

(all strenghts in kPa. Strengths in parenthesis are those for samples with un-milled ash)

TABLE 4.4 THE EFFECT OF MILLING PFA ON STRENGTH GAIN

These results can be summarised as follows.

BINDERS	PERCENTAGE STRENGTH GAIN DUE TO ASH MILLING	
	14 day	28 day
PFA / CEMENT	7	6
PFA / LIME	10	12
KPFA / CEMENT	6	4
KPFA / LIME	6	3

TABLE 4.5 PERCENTAGE STRENGTH GAIN DUE TO ASH PULVERIZATION

The pulverization of the ash was beneficial for all binders, especially the Darling and Hodgson ash/lime binder. A cost-benefit analysis will have to be performed to determine whether the additional strength gain will offset the cost of pulverising the ash.

5.0 CONCLUSIONS

5.1 a Phosphogypsum Binders

The results obtained in this testing programme using phosphogypsum binders were extremely dissappointing. The soils tested where, however, similar mineralogically and it is possible that this binder will give better results with soils containing other clay minerals.

5.1.b PFA Binders

The lime content of many of the PFA/lime binders used on the programme fell in a range where only short term reactions could be expected. However, even where sufficient lime was present to satisfy the initial lime demand of the soil, the PFA appeared to act mainly as a diluent to the lime. Similar conclusions apply to PFA/cement binders. It is not certain whether the lack of success resulted from chemical defiencies of the PFA, the coarseness of the PFA, or some other weakness in the testing programme (e.g. testing at too early an age)

5.2 Proposals for Further Research

Recomendations for future testing programmes are :-

1) Samples should be compacted to a constant compactive effort, and not a constant density

(nb. In the static compaction of the samples, samples containing lime binders always required a higher machine load for compaction than samples containing only PFA.
There were also differences in th load required to compact the two different PFA's). This will possibly eliminate some of the differences in instantaneous strength.

2) As stated previously, the initial moisture content of the stabilized soil samples should be varied, and the effect of this on strength at various ages determined.

3) The unconfined compression samples should be soaked and de-aired before testing. Although this may only be of practical significance in areas of high rainfall, it will result in lower instantaneous strengths. (probably zero). The strengths after a certain curing period will then probably be an indication of cementitious activity rather than favourable grading.

4) A realistic curing period should be found. The period used in this programme, viz. 28 days may be too short to allow pozzolanic soil-lime and lime-PFA reactions to take place. The period selected would probably have to bear some relation to practical actualaties e.g. time to the opening of the road to traffic.

5) Another possible alteration is to change the nature of the tests, from finding the response of the soil to a predetermined quantity of binder, to finding the binder which can most economically lead to a predetermined strength. This approach would probably be more realistic from a road design point of view. It would also eliminate the consequence of effects such as initial lime consumption from the test results. (they would appear as increased costs).

6) From what has been stated above, it is considered that 'plasticity index' cannot be successfully used to predict the optimum stabilizer composition for a soil. The soils used in this programme, although covering a wide range of plasticity indeces, all contained the same predominant clay minerals, and should, therefore, all have been able to have been stabilized with the same type of binder. (obviously proportions of the binder constituents would be changed eg. different PFA/lime ratios) Possibly, if the clay mineral types in a soil are known, the plasticity index could be successfully used as a "stabilizer requirement" indicator. In future testing programmes, an attempt should be made to obtain soils containing different clay minerals.

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