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THE INFLUENCE OF SOIL TYPES AND STABILIZING AGENTS ON THE ENGINEERING PROPERTIES OF STABILIZED SOIL FOR USE IN EMBANKMENT DAMS

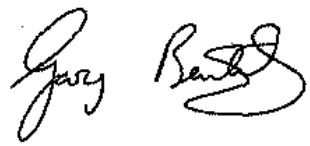
GARY MICHAEL BENTEL

A thesis submitted to the Faculty of Engineering, University of the Witwatersrand, Johannesburg, for the degree of Doctor of Philosophy.

Johannesburg 1989

DECLARATION

I declare that this thesis is my own unaided work. It is being submitted for the degree of Doctor of Philosophy in the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination in any other University.



GARY MICHAEL BENTEL

2nd Day of August, 1989.

To my mother, Gwyneth,
and the memory of my father, 'Speedy'.

Abstract

The thesis considers the feasibility of using marginal soils, stabilized with hydrated lime or ordinary Portland Cement as an alternative construction material for embankment dams.

An historical review discusses some prior successful applications of soil-cement, soil-lime and roller compacted concrete in hydraulic structures.

The principles of soil stabilization with lime and cement are presented as an introduction to the laboratory testing programme. The testing programme considers aspects of engineering properties fundamental to embankment dam construction. These include compaction characteristics, unconfined compressive strength, shear strength, flexural strength, shrinkage characteristics and permeability.

Since the stabilized material will require protection by a durable facing, the durabilities of the materials are considered, but to a limited extent, as are the thermal properties. Further research is required to investigate the suitability of the presently accepted durability test methods which appear to be too harsh for soil-lime.

The engineering properties of the untreated soils, which would probably not be considered suitable for conventional embankment dams construction, are significantly enhanced by the addition of the stabilizers. The resulting stabilized materials present the designer with a variety of alternative materials suitable for dam construction. Cost savings relative to conventional dam building materials result from a significant reduction in the construction time.

Although the properties of soil-lime and soil-cement may differ, e.g. in time rate of strength development, this is seen as an advantage in the consideration of alternative design options in arriving at a creative cost-effective solution.

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Chapter 1

INTRODUCTION

1.1 Developments in the Use of Compacted Materials in Embankment Dams and other Hydraulic Structures

In recent years, rapid increases in construction costs have motivated dam designers to research and develop construction techniques which reduce costs by utilizing less suitable materials and by reducing the construction time. Both stabilized soil and roller compacted concrete have become increasingly popular in dam building and other hydraulic applications where durable and erosion resistant materials are required.

The need for cost effective construction materials is especially apparent in Africa where many areas are subjected to alternating periods of severe drought and sudden high rainfalls. The third world economies are hard pressed to cope with the more immediate problems of disease and starvation, and thus cannot afford the costs of building structures to either prevent floods or to provide water during periods of drought. Although foreign aid attempts to partially solve the problem by providing much needed food and medical supplies, it does not help the third world countries to become self-sufficient.

Thus many dams must be constructed at the lowest possible cost, while at the same time providing sufficient spillway capacity and/or erosion resistance to prevent dam failure during a major flood. Rural communities could then develop agricultural projects without fear of their efforts being thwarted by flood or drought.

Robertson, Bentel and Blight [1] discuss the potential of the stabilized soil dam in the African context, and how the different materials and embankment configurations provide the designer with a number of feasible alternatives which can be used under a wide variety of circumstances.

The following sections present an historical review of some of the developments in the construction of embankment dams and other hydraulic structures which motivated this research project. The discussion highlights the areas where the use of stabilized soil and roller compacted concrete can provide benefits over traditional solutions, either by expediting construction or by cutting costs.

1.1.1 Soil-cement

Probably the most important development in dam building technology in recent years was the successful use of soil-cement as slope protection. A typical soil-cement slope protection design is shown in Figure 1.1. The protection layer of approximately 600 mm thickness is constructed by compacting horizontal layers of soil-cement typically 2 to 3 m wide and 150 to 300 mm deep in stairstep fashion up the embankment. It should be noted that the same technique is used in roller compacted concrete construction.

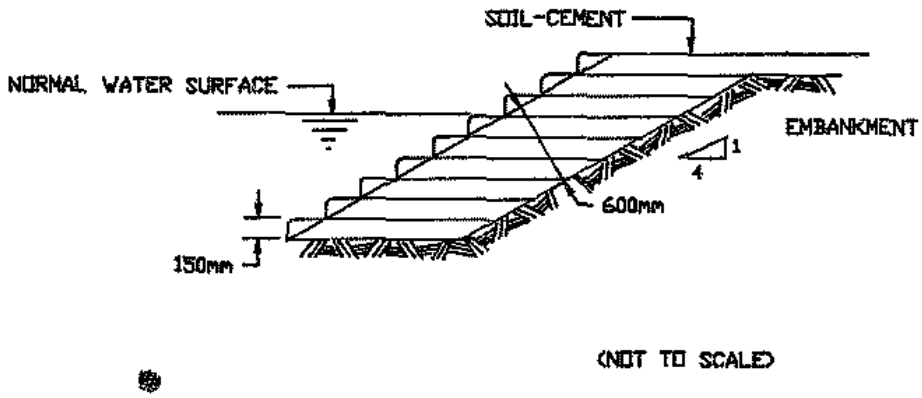


Figure 1.1: Soil-cement Slope Protection

The first significant use of soil-cement in dam construction occurred in 1951 when a trial embankment in the area of the Bonny dam was faced with soil-cement and asphalt concrete as substitutes for rip-rap slope protection [2,3]. The closely monitored trial proved successful and soil-cement rapidly gained popularity as an economically competitive slope protection. In 1976 the Portland Cement Association [4] reported that soil-cement had been used in some 150 projects for slope protection for earth dams or other embankments.

Although not yet widely used in South Africa, soil-cement was used by the Department of Water Affairs to line the upstream and downstream faces of several small containment dams in the Vaal-Grootdraai emergency scheme. The intermediate containment dams were built at "great speed" in 1983 to reverse the flow of the Vaal River between the Vaal Dam and the Grootdraai Dam during a particularly severe drought which began in 1978 [5].

The idea of a solid stabilized soil dam, i.e. a homogeneous embankment (Figure 1.2) was initiated by Nash, Jardine and Humphreys [6] and pursued by Blight [7], Robertson and Blight [8], and Robertson [9]. The philosophy behind the idea is that since stabilized soil has proved successful in 'hydraulic' applications, then under suitable conditions it can also be used in the body of the dam as the mass gravity component. By varying the stabilizer content, the specific 'zones' required in the embankment can be provided e.g. a cement rich upstream facing can be used simultaneously as an impermeable membrane and as slope protection.

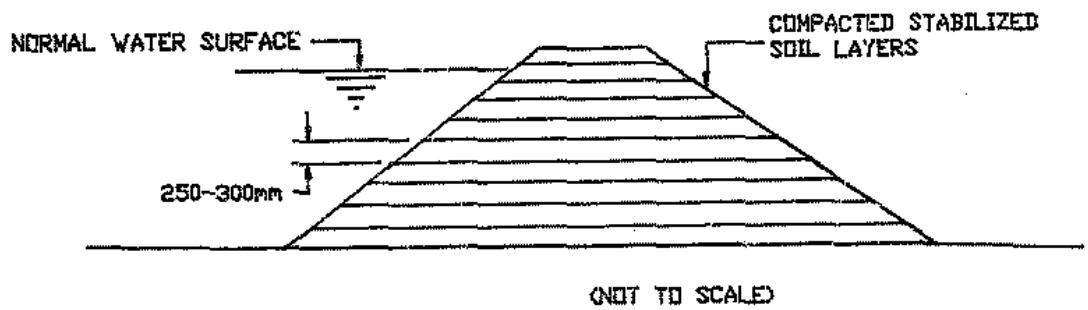


Figure 1.2: The 'Solid' Stabilized Soil Embankment Dam

In comparison with a typical zoned earth embankment dam, some advantages perceived in the use of stabilized soil are:

- The embankment slopes can be steepened as the strength of the material increases thereby decreasing the embankment volume. Depending on the height of the embankment, the cost saving through reducing the embankment volume and thereby the construction time can exceed the additional cost of the stabilizer.
- The need for slope protection can be eliminated if the stabilized soil is sufficiently durable to resist erosion.
- The reduced base width of the stabilized soil dam and the increased erosion resistance could reduce the lengths and thus also the costs of diversion and outlet works. For example, an emergency spillway could be lined with soil-cement rather than the conventional and more expensive concrete lining, while the length of the spillway would be less than that required for an earth dam because of the reduced width of the dam.
- Foundation preparation would be similar to that for an earth embankment dam. Although the width of the foundation would be reduced, the depth might in some cases have to be greater to provide an adequate foundation for the less flexible stabilized soil.
- The permeability of stabilized soil is generally low enough to eliminate the need of impervious zones and filters. For example, an impervious upstream soil-cement facing might eliminate the necessity for a clay core.

Robertson [9] conducted extensive research on various engineering properties of soil-cement, using a residual felsite located near to the new Rondebosch Dam site at Middelburg, Transvaal. He concluded that the solid soil-cement dam was indeed viable and in a comparison with the actual Rondebosch concrete buttress dam, he found that the soil-cement dam would provide many advantages which would result in a considerable cost saving.

Although a large soil-cement dam has yet to be constructed, this may be attributable to the fact that the material is still being researched and developed. Nevertheless, the material has been used in smaller hydraulic structures in a number of ways.

Dinchak [10] reports the construction of small solid soil-cement embankment walls (depths varying from 2.5 m to 6.5 m) forming the circumferential wall and the interior baffle dykes of a cooling water reservoir in Texas. Dinchak

[10] and Adaska [3] report the use of soil-cement for foundation construction, shoreline erosion protection and the use of low-permeability soil-cement as a lining material for seepage control and slope protection in reservoirs, channels, and sludge-drying and wastewater lagoons. Adaska [3] reports the use of soil-cement in more than 300 water resource projects over the 30 year period before 1985. He also mentions the development of a composite liner consisting of a synthetic liner placed between two layers of soil-cement for maximum seepage protection in the storage of hazardous wastes.

1.1.2 Soil-lime

The strength and durability of lime stabilized soils are generally lower than cement stabilized soils. A direct comparison is however pointless since the choice of *stabilizing agent for* a particular application is dependent on the soil type, and in particular the quantity and type of clay present in that soil. For a soil containing a heavy clay, one will often find that on adding a given quantity of lime, the *resulting material is superior* to that obtained by adding a similar quantity of cement.

Thus cement stabilization is not suitable for all applications, and Gutschick [11] reports a wide variety of projects where soil-lime has been used successfully in 'hydraulic' structures, i.e. structures that are submerged or partially submerged.

One such project is the Friant-Kern irrigation canal in California where the USBR re-lined sections of the canal with soil-lime. Initially, 85% of the canal was lined with concrete panels, the remainder of the canal being earth lined. The purpose of the remedial work was to overcome cracking and sliding problems along earth lined sections of the canal, and to prevent further cracking or "popping off" of concrete panels. These problems were caused by the expansive clay soil used to construct a large proportion of the canal. After twelve years, the soil-lime lining was reported to have performed "in an excellent fashion" [12], showing little deterioration, very good resistance to erosion and no loss in strength. Core samples indicate that the strength is increasing with age.

Lime stabilization has also been used extensively in repair work in levees and earth dams where piping problems in dispersive soils, and sliding problems in expansive clays have been encountered. As a result of these successful applications, soil-lime is being used as a 'shell' to face dams and spillways [13,14,15] and in dam cores to prevent erosion and piping [14,15].

1.1.3 Roller compacted concrete - RCC

Roller compacted concrete is a material which has gained rapid popularity as a construction material in embankment dams. Since the first RCC dam was constructed in 1982 [16], RCC has proved economically competitive as an alternative to both earth embankment and concrete gravity dams. Prior to August 1988, approximately eighteen RCC dams had been built worldwide [17]. In South Africa, two RCC dams have been constructed while at least two more are at the design or tender stage.

RCC originated from the idea of compacting conventional structural concrete with vibratory rollers in order to gain economical advantage by reducing the construction time. Since densification is achieved by compaction, less placement water and less cement is required to achieve similar strengths at similar cement/water ratios as conventional concretes. The cement content can be further reduced by replacing cement with flyash which improves the workability of the mix, and which can contribute to strength gains through pozzolanic reaction with lime liberated during cement hydration.

The unit cost of RCC is thus less than the unit cost of conventional concrete. However, the major cost savings emanate from the reduction in construction time, since increased material volumes may offset cost savings through reduced cement contents.

Successful mixes [16,18,19,20] have also been achieved using less select aggregates and a higher cement content to offset the lower strength resulting from the higher proportion of fine material. The aggregates may be a mixture of locally quarried materials ("as dug"), with some processed aggregate added to improve the particle grading distribution.

Prior to the addition of cement, these latter materials would appear to fall within the Unified and AASHTO soil groups, i.e. they can be classified as soils rather than concrete aggregates. The products however have concrete like properties. Thus the products can justifiably be classified as soil-cement, but are rather included in the so-called "range" of roller compacted concretes. The classification seems to result from the ability to obtain a fairly consistent material throughout the dam, thus minimizing the variability of the product.

There exists no single definition for RCC, and it appears that in order for such materials to be acceptable to dam builders, the definition of concrete has been 'stretched' to include soils successfully stabilized with cement. Schrader [16] contends that the soil in soil-cement has very little coarse material, whereas RCC

contains more coarse gravels. However, from descriptions given, the materials used at *Willowcreek [16] and Monksville [18]* dams (albeit manufactured by mixing constituents), might well be classified as well graded gravels rather than 'concrete aggregates'.

The author feels that such attempts to redefine stabilized soil and concrete are unnecessary. The search for alternative construction materials will lose momentum if it gets caught up in the concrete designers desire for materials with the predictability of conventional concrete. *The 'code' approach to concrete design* will certainly stifle the development of alternative materials if this approach is allowed to be the criterion for evaluating the success of alternative materials.

A combination of the arts of soil-mechanics and concrete technology provides sufficient understanding for (at least) trials to validate (or invalidate) proposed alternatives. Such trials would be expensive but could prove profitable to dam building, as did RCC which required the expenditure of *vast amounts of money* on field trials before becoming acceptable as a dam building material.

The successful applications of soil-cement quoted in Section 1.1.1 can thus be augmented by similar applications where the use of material referred to as RCC has not been prepared using conventional concrete aggregates.

1.2 Justification for Further Research

As the title of this thesis suggests, the primary aim of this research project is to extend earlier research into alternative dam building materials to include stabilizing agents other than cement, and simultaneously broaden the range of exploitable soil types. This idea arose from the desire to utilize the large quantities of plastic (clay) soils available in South Africa not generally suitable for use as a dam building material in an unstabilized form.

Legge [21] notes that in certain regions of South Africa, "it is not possible to obtain even semi-pervious soils within an economic haul distance from the dam site". A possible solution under such circumstances is to construct an homogeneous embankment, i.e. without semi-pervious or pervious outer zones, using the impervious clay material. This results in the embankment having steeper side slopes than the slopes which might be achieved in a zoned embankment. Costs could certainly be reduced significantly if it proved feasible to use such materials in a stabilized form to construct the embankment dam.

It was also envisaged that it would be advantageous to dam designers if it became possible to provide a more flexible stabilized material by using plastic soils and stabilizing agents other than cement. As a result the restriction on the use of soil-cement to sites with foundations stiff enough to prevent cracking in the relatively brittle soil-cement, could be eliminated or at least improved upon. Alternatively, the increased costs of providing adequate foundations for the less flexible stabilized soils (relative to conventional embankment dam requirements) might well be offset by the reduction in the transportation costs through utilizing nearby 'marginal' soils.

Since stabilized soil has been used so extensively in highway construction a vast amount of research has been conducted into the properties of stabilized soil resulting in a great deal of reference literature on the subject. However the past research is often selective, concentrating on the quantification of the properties specific to a particular application. This reference material is thus not easily transferable to the more elaborate requirements of embankment dam design, and thus further research is required to evaluate alternative materials in this context.

Furthermore, the behaviour of an embankment dam under varying boundary conditions is certainly different to the behaviour of road or airfield pavements. The consequences of the failure of a dam demand that a dam be designed with almost no risk of failure. This requires an intimate understanding both of the engineering properties of the materials involved, and of changes in boundary conditions (such as the presence of permeating water) on the engineering properties and on the integrity of the structure as a whole.

In order to use stabilized soil as a construction material in dams, there is a need for research which provides:

- An understanding of the mechanisms of cement and lime stabilization of various soil types.
- The quantification of the entire spectrum of resulting engineering properties pertinent to dam design, for a variety of soil-stabilizer combinations.

In planning the research programme, it was recognized that the scope of the project would not permit an in-depth investigation into all the properties which require quantification. For a number of soil/stabilizer combinations, each property could in fact form the basis of an individual research project. It was thus decided to differentiate between those properties which require laboratory investigation before the construction of a trial embankment, and those properties which could better be investigated under full-scale conditions.

A testing programme was then set up to investigate the following aspects of stabilized soils:

- The determination of the index properties of the selected soils, including particle size distribution and Atterberg limits. The results of these preliminary tests are presented in Chapter 3 together with information regarding selection of soil types, stabilizing agents, and stabilizer contents.
- Compaction characteristics.
- Strength - including triaxial shear strength, unconfined compressive strength and flexural strength.
- Permeability and erosion resistance.
- Shrinkage.
- Durability.
- *Thermal conductivity.*

The testing programmes for thermal conductivity and durability were limited thus allowing more effort to be concentrated on the other properties. Thermal conductivity would be more easily quantified through measurements on a large structure, but some value for stabilized soil was desired. The testing of durability according to the accepted wet/dry testing procedure appears to be too harsh for soil-lime mixtures which, although they may fail such a test, show evidence of being sufficiently durable for some hydraulic applications. Further research is required to develop a suitable laboratory test which provides a satisfactory comparison with field results.

Nevertheless, it was felt that the quantification of the listed properties would provide sufficient design data for the construction of a trial embankment. The trial embankment could then be used to quantify other properties and to evaluate the field behaviour of the various materials.

Chapter 2

THE PRINCIPLES OF SOIL STABILIZATION WITH CEMENT AND LIME

2.1 Introduction

In South Africa, both lime and cement are used to stabilize soils for use in pavement layers. The type of stabilizing agent depends on the constituents of the soil to be treated while the quantity of stabilizer normally depends on the end result required, i.e. modification or cementation.

A rule of thumb frequently used is that lime is suitable for treating materials with a plasticity index (PI) higher than 10 ([22]). Ballantine and Rossouw [23] recommend that for effective stabilization with lime, the treated soil should have a minimum clay content of 5% and a PI of at least 12 to ensure "an adequate cementing matrix". There are however many materials e.g. natural gravels such as certain sandstones, calcretes, decomposed granites and dolerites, whose strength may increases considerably when treated with lime, even though the PIs of these materials may be below 10 [22].

Although cement can be used to stabilize almost any soil, it is generally most effective in stabilizing medium to low plasticity materials. In practice it is sometimes difficult to obtain a uniform mixture when using cement with fine, clayey soils. In such cases, lime may prove to be a better stabilizing agent since it reduces the plasticity of the clay and can aid in the pulverization of a clay i.e. in

allowing desiccated lumps to be broken down more easily (see Section 2.4).

Research is currently being conducted at the University of the Witwatersrand which will hopefully delineate more clearly the boundary between soils that should be stabilized with cement, and soils that should be stabilized with lime. Nevertheless, the final choice of stabilizing agent for a particular project requires consideration of the design requirements, laboratory test results, site conditions, availability of stabilizers at the time of construction and economics.

In attempting to achieve a desired result, one can also consider the possibility of using a combination of stabilizing agents. By adding small quantities of lime to a clay soil, one can significantly reduce the plasticity of the soil (or "dry out" the soil) and produce a more friable material which is easier to work and compact. The technique of "preconditioning" clayey soils in this way is often used to expedite construction, particularly on wet sites or during rainy weather, or to prepare the soil either for further additions of lime or for stabilization with cement.

Clay mineralogy and the chemistry of lime-clay reactions and cement hydration are complex topics not fully understood at this stage. A critical revue of these topics is beyond the scope of this project. Some references dealing with soil stabilization use simplistic explanations to describe complex manifestations such as the self-aggregation or flocculation of clay resulting from the addition of lime.

An understanding of the fundamentals of soil stabilization according to these simplistic explanations has proved adequate for the successful use of stabilized materials in a variety of applications. However some points do require emphasis to prevent materials from being incorrectly used.

In the case of lime stabilization for example, a knowledge of the type of clay being stabilized is important since the extent and rate of the lime-clay reactions differ between clay minerals because of the different mineral structures. Thus successful lime stabilization is fairly sensitive to the quantity of added lime i.e. different clays require different quantities of added lime to achieve the same end result. Unlike cement stabilization where increasing the cement content almost inevitably results in an increase in the strength, excess lime may defeat the objective, with higher strengths being achieved at lower lime contents.

The rate of gain in strength of soil-lime may be very slow compared to the rate of gain in strength of soil-cement. Design considerations must therefore take into account the need for lengthier curing periods before a structure can be put into service.

In practice, it is not economically viable to pulverize hardened clay lumps into individual clay particles. Specifications generally require that the lumps be crushed to a maximum size of 4.75 mm (passing the #4 sieve). A particle size distribution curve obtained in the laboratory represents an ideal case, with the clay particles having been separated from each other (dispersed) by the addition of a deflocculant or dispersing agent. In reality, the clay lumps must be treated as larger particles inherently weaker than any cementing agent or silt or sand particles. Although the outsides of the lumps may be modified and thereafter coated with cementing material, it is not likely that the interior of the lumps will become stabilized unless the stabilizer is able to diffuse into the body of the lump. Care must therefore be taken not to treat clay soils as if they were made up of easily divisible minutely sized particles.

The basic structure of clay minerals is described in Appendix A, together with the structure of two clay types smectite and kaolinite. Smectite and kaolinite are common in naturally occurring soils and are frequently used to describe the extremes of the behaviour of the various clay minerals. They are used in this text to highlight the differences in the behaviour that one might expect on stabilizing various clay soils with lime and cement[1].

2.2 Lime Stabilization

2.2.1 Lime

Although 'lime' refers specifically to the chemical compound calcium oxide (CaO), it is also commonly used to refer to the calcination products of limestone ($CaCO_3$) and dolomite ($CaMg(CO_3)_2$) and other calcareous or dolomitic calcination products.

When heated to about 1100^{0}C [24], calcitic limestone decomposes into "burned" lime (also "quick" or "unslaked" lime - CaO) and carbon dioxide (CO_2), i.e.:

$$CaCO_3 \xrightarrow{heating} CaO + CO_2 \text{ - limestone}$$

The decomposition of dolomitic limestone is a two stage process. At a temperature of about 700^{0}C dolomite decomposes to form magnesium oxide (MgO), carbon dioxide (CO_2) and calcium carbonate ($CaCO_3$) i.e.:

[1] The clay minerals react with the free lime in the cement

$$CaMg(CO_3)_2 \xrightarrow{700^0C} MgO + CaCO_3 + CO_2 \text{ - dolomite}$$

It is necessary to continue heating the products to 1100^{0}C to decompose the $CaCO_3$, i.e.:

$$MgO + CaCO_3 \xrightarrow{1100^0C} CaO + MgO + CO_2$$

The products generally include small quantities of 'impurities' originally included in the limestone (see Table 3.2), but the two types of lime are easily distinguished by the quantities of calcium (as CaO) and magnesium (as MgO). 'High calcium' limes may contain 90-99% CaO while dolomitic limes may contain 30-40% MgO.

Dry hydrated calcitic lime is produced when CaO is reacted with just enough water to satisfy its affinity for moisture under conditions of hydration. High calcium quicklime reacts readily with water to produce calcitic hydrated lime i.e.:

$$CaO + H_2O \longrightarrow Ca(OH)_2 \uparrow$$

The hydration reaction, also known as 'slaking', is exothermic and is accompanied by an increase in volume. Logically, the hydrated lime has less available CaO per unit mass (typically 60-75% available CaO) than the unslaked lime.

Dolomitic quicklime does not hydrate as readily, the MgO component being less reactive as a result of the 'over burning'. Hydration at atmospheric pressure and low retention times results in dolomitic monohydrate lime ($Ca(OH)_2$ + MgO), while longer retention periods, elevated temperature and pressure results in dolomitic dihydrate lime ($Ca(OH)_2 + Mg(OH)_2$).

The products most commonly used for soil stabilization are hydrated high calcium lime ($Ca(OH)_2$), monohydrated dolomitic lime ($Ca(OH)_2$ + MgO), calcitic quicklime (CaO) and dolomitic quicklime (CaO + MgO).

2.2.2 Lime-clay Reactions

When lime[2] is mixed with a moist clay soil, two basic though complex reactions take place[22,23,25,26]:

[2]This thesis considers the use of high calcium hydrated lime, calcium hydroxide ($Ca(OH)_2$) only.

- Cation exchange reactions and *flocculation of the clay* which begin almost instantaneously and which produce rapid changes in the soil plasticity and workability - *referred to as modification* reactions.
- Depending on the characteristics of the soil being stabilized, a clay-lime pozzolanic reaction may occur. *The pozzolanic reaction*, which is time dependent, results in the formation of of various cementing agents which increase *the strength* and durability of the material - called *cementation*.

Ion exchange and flocculation

Clay particles adsorb cations[3] to neutralize the negative surface charge resulting from unbalanced isomorphous substitution (see Appendix A). The negative surface and the distributed charge in the adjacent phase are together termed *the* 'diffuse double layer' [27].

The type and size of the adsorbed cations, together with other influences such *as pH* and *electrolyte concentration*[4], influence the thickness of the double layer and thus the interparticle forces and the quantity of water adsorbed by the clay to hydrate the particle surface and the adsorbed cations.

The adsorbed cations are exchangeable, i.e. they can be replaced by other cations with the general rule for replaceability *being that multivalent* cations are adsorbed in preference to monovalent cations.

When lime is added to a moist clay soil, a base-exchange reaction occurs with Ca^{2+} ions replacing the H^{+} ions and other monovalent ions such as Na^{+} ions adsorbed to the clay.

Both the increase in the *electrolyte concentration* and the preferential adsorption of multivalent Ca^{2+} ions suppress the double layer, leading to reduction in *interparticle repulsion* [27,29].

As the water content of calcium smectite is increased, the interlayer expansion is restricted to a c-axis spacing of 19 $\mathring{A}$ leading to domains or aggregates of several unit layers [27]. On the other hand, sodium smectite can expand almost indefinitely due to strong interparticle repulsions, leading to fully separated *unit* layer particles (dispersed).

[3] The influence of adsorbed anions is generally not considered as being significant in lime stabilization

[4] *Lambe and Whitman*[28] and Mitchell [27] and Grim [29] describe the various influences on the adsorbed water.

If sufficient lime is added to the clay, crowding of the Ca^{2+} cations onto the clay particle surfaces results in the particles aggregating into silt sized 'flocs'.

It is thought [30] that the tendency towards flocculation arises from the increase in the pH of the soil-water. At higher pH values, hydrogen ions may dissociate from SiOH groups exposed at the surface and edges of the particle i.e.:

$$SiOH \xrightarrow{H_2O} SiO^- + H^+$$

resulting in the particle becoming more negatively charged. Thus the cation exchange capacity is dependent on pH, being greater at higher pH values [29].

Thus the phenomenon of flocculation appears to result from the combined effects of the reduced interparticle repulsions and from the bonding together of two negatively charged edges or surfaces by the positive valencies of calcium ions.

The ion exchange reactions and the flocculation of the clay begin almost immediately, resulting in most cases in an increase in the plastic limit and therefore an increase in the strength at the same 'in-situ' water content [31]. Generally the plastic limit increases, while the liquid limit either remains unchanged or increases or decreases slightly. The lime improves the texture of the clay, and the material becomes more friable and more workable.

The result of flocculation is in effect an increase in the size of 'particle' which will remain stable, i.e. will not disperse into smaller particles, when placed in water. Ingles [32] defines flocculation as the 'bridging' of clay particles into a loose random structure (Figure 2.1). The randomly oriented structure is more porous (and hence has a higher permeability) than the unflocculated structure. The bridged structure resists shear and thus yields a lower compacted density for the same compactive effort. He describes flocculation as "effectively converting the clay to the mechanical equivalent of a fine silt".

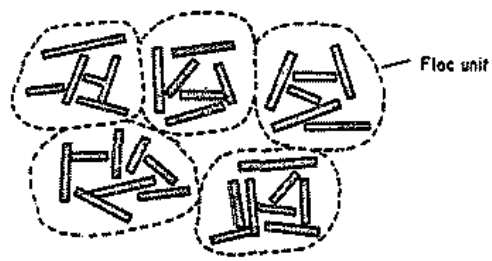


Figure 2.1: Floc Units in Random Orientation

Figure 2.2 shows the variation of the Atterberg limits with increasing lime content of a smectite clay soil investigated in this project. The limits were determined after the mixtures had been allowed to 'cure' in a loose state for a period of 48 hours. The trend is typical of smectite clays which show a significant reduction in plasticity resulting from an increase in the plastic limit and a reduction in the liquid limit.

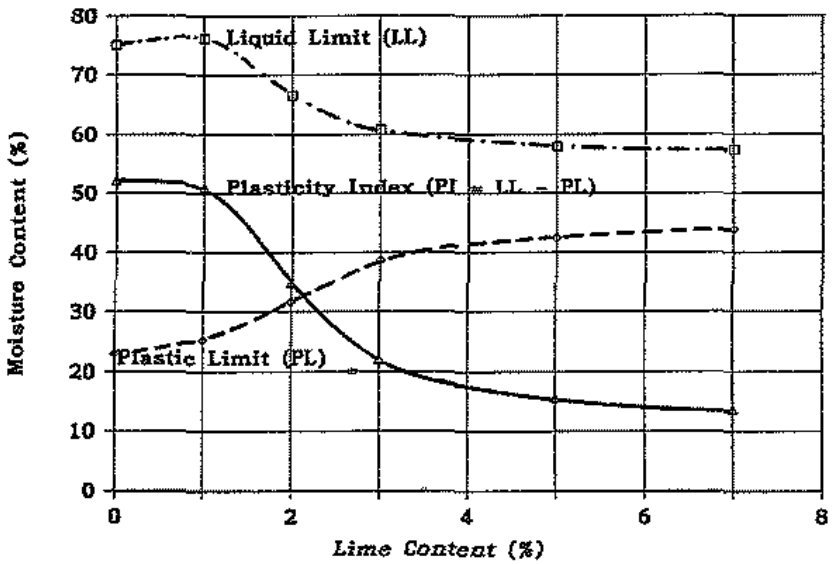


Figure 2.2: Variation of Atterberg Limits of Soil C with Lime Content

The increase in the plastic limit can be ascribed to the flocculation of the clay which behaves more like a granular material. Hilt and Davidson [33] explain the rise in plastic limit as being additional water required to break the stronger bonds between clay particles within the flocs. In other words, a higher water content creates a more dispersed structure which is easier to remould.

The stabilized material still exhibits a liquid limit i.e. some cohesive strength, but will exhibit little plasticity because of the overriding influence of weakly attracted silt size flocs. Although the soil may appear to be non-plastic, this does not mean that the plastic limit is zero, but that the plastic limit is very close to the liquid limit or cannot be measured. The higher the clay content (untreated), the further

the plastic and liquid limits diverge from one another because of the increasing influence of the clay.

Thus as the lime content is increased, more clay becomes flocculated, reducing the range of water contents over which the soil will behave plastically. The lime itself may also contribute to some extent to the raising of the plastic limit when it hydrates, i.e. additional water is required to plasticize the lime.

The same trend cannot be expected for all soils, and laboratory determinations of the Atterberg limits must be conducted to confirm any assumed changes. Kaolinitic clays often show increases in the liquid limit with increasing lime content, sometimes showing increased plasticity [31]. However, the increased plasticity can be shown to result when the liquid limit increases by a greater amount than the plastic limit[5], i.e. the liquid and plastic limit curves diverge with increasing lime content.

Rowlands, Fiht and Delpak [34] present results obtained by A. Leroux which show the PI of a kaolinite clay increases from 35 to 40% on the addition of 20% lime as a result of the liquid limit increasing from 71 to 87%, while the plastic limit increased from 36 to 47%. Although an increase in the plasticity may not be desirable in certain applications, the increase in the plastic limit still makes the soil stronger at the same 'in situ' water content [31]. Results of their own research programme [34] conducted on a soil with a PI of approximately 12% show that the PI decreases with increasing lime content, curing period and curing temperature.

The quantity of lime required to complete the ion exchange and flocculation reactions (known as the 'initial lime demand' or 'lime fixation capacity'), the rate of the reaction and the extent to which the properties of the clay are altered by the reaction, are dependent on the clay type.

Kaolinites have low cation exchange capacities and thus low initial lime demands (1 to 2% lime by dry weight). The ion exchange and flocculation reactions are normally completed fairly rapidly i.e. in less than 24 hours. Smectite clays have high cation exchange capacities and initial lime demands may range from 5% to as high as 12%. The ion exchange and flocculation reactions occur at slower rate and may take some days to complete.

Because of its small particle size and the expanding lattice structure of these particles, smectite clay is highly water 'sensitive'. Smectite is notorious for the large volume changes which occur when it is wet or dried. On the other hand,

[5] In the numerous texts consulted, the author found no instance where the plastic limit of a clay soil of any type decreased with increasing lime content.

kaolinite is relatively insensitive to changes in water content and does not swell or shrink very much. Thus the potential for modification of the engineering properties of smectite by lime stabilization is logically much greater than that for kaolinite.

Changes in the engineering properties can be effected rapidly, with the modified clay having higher shear strength, higher permeability and lower plasticity than the original material. As mentioned earlier, these beneficial effects can be utilized to expedite construction on wet sites where the modification of a plastic clay to a friable non-plastic material allows access to heavy construction equipment.

Cementation

After the short term ion exchange and flocculation reactions have occurred, any excess or 'free' lime reacts chemically with the clay minerals or with any other fine, pozzolanic component such as hydrous silica or alumina. These reactions produce *"tough, water-insoluble" hydrated calcium silicate and aluminate gels* which cement the soil particles [22,25,31,35,36].

The reaction products are similar to those present in hydrated Portland cement (see section 2.3). However the hydration of cement is independent of the soil type, while with lime the gel is formed only after attack and removal of silica from the clay minerals of the soil [25,31].

The type and amount of minerals in the soil that react with the lime (known as pozzolans) vary for different soil types and determine the long term properties of the soil-lime. Possible sources of silica and alumina include clay minerals, quartz, felspars, micas and other similar silicate or alumino-silicate minerals.

In addition to the formation of calcium and aluminium silicates, it is also believed that *the lime reacts chemically with the surfaces of the clay minerals* [25], with new phases nucleating directly upon the surfaces of the clay particle. The same reference suggests the possibility that cementation occurs as a combination of the two processes.

These reactions occur only under a condition of high alkalinity [23,25,33,37,38,39] (pH > 11) at which the solubilities of silica and alumina are greatly increased. This condition is achieved by adding sufficient lime (the pH of lime saturated water is approximately 12.4) to ensure that the pore water remains calcium saturated *for the projected duration of the pozzolanic reactions.* Ingles and Metcalf [31] note that the reactions proceed only whilst water is present and able to carry

calcium and hydroxyl ions to the clay surface. The reaction thus ceases on drying (just as cement hydration would cease on drying), and very dry soils will not react with lime.

Ingles and Metcalf [31] describe the cementing mechanism as shown in Figure 2.3 whereby the gel immediately begins to coat clay lumps in the soil and to block off the soil pores. In time the gel crystallizes on the sides of clay lumps into calcium silicate and aluminate hydrates. Water for the reactions is 'withdrawn' from the soil pores until there is insufficient water in the voids and the reaction ceases. Correct emphasis is placed on the fact that (in the short term) clay lumps are stabilized on their outside surfaces only. Thus together with the unused lime and the weaker uncrystallized gel, the interior of the lumps represent local weaknesses in the 'matrix' of stabilized soil. In the longer term (a number of years), the interior of the lumps may become stabilized through the diffusion of dissolved lime.

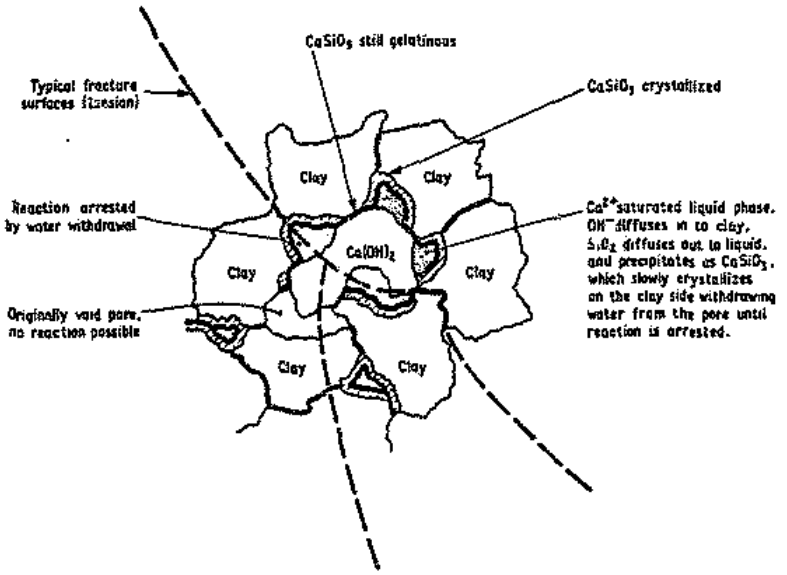


Figure 2.3: Mechanism of Lime Stabilization of Clay Soils (after Ingles and Metcalf [31])

Natural soils contain both granular (silt, sand and gravel) and clay materials. Some of the granular particles may be capable of reacting with the lime to form cementitious gel. It follows logically that there must be sufficient lime reactive clay or other materials to produce sufficient cementing material to both encase the clay lumps and to bind granular particles together at points of contact [32].

Although the cementing reactions may begin immediately, the rate of the reaction is very slow, and it may take some years before sufficient crystalline products form to produce strengths comparable with soil-cement. The cementitious products are permanent [22,25,39], and as will be demonstrated by the results of this project, large increases in strength occur when the material is submersed in water or when flowing water is passed through the material.

On the basis of this discussion, the author feels intuitively that water percolating through a compacted lime-clay material is advantageous since it can 'carry' calcium ions into the unstabilized body of clay lumps and other regions otherwise inaccessible to the calcium. In this way the material may be more effectively stabilized fairly rapidly. However, excess lime required for the slower pozzolanic reactions may be leached out of the material. It is thus important in a structure such as an embankment dam, to ensure that the flow of water through the stabilized material is very slow.

Not all clay types are lime reactive. The non-reactive materials show little or no increase in strength when lime stabilization is attempted. Short term (7 or 28 day) laboratory strength tests are of little value in determining 'optimum' lime requirements for strength, and may provide misleading results (see Chapter 5). Long term laboratory tests, which may be shortened by accelerating the reactions by curing specimens at elevated temperatures [25], are the only reliable means of identifying whether a soil is lime reactive or not.

Carbonation

If a soil-lime mixture is left exposed, then carbon dioxide from the air reacts with the calcium hydroxide to form calcium carbonate. Carbonation is undesirable because it robs the lime intended for pozzolanic reactions and fills the voids with a weak cement which prevents the bonding of soil particles by any pozzolanic cement that may be produced.

Paige-Green [40] found that not only does carbon dioxide retard the development of strength of lime-stabilized materials but is also capable of reversing the strength development in some materials, particularly lime stabilized calcretes, such that

major strength losses result.

Carbonation of lime prior to and after mixing must therefore be prevented in order to achieve the maximum strength gains through pozzolanic action and to prevent the deterioration of the stabilized material when subjected to service loads.

2.3 Cement Stabilization

2.3.1 Cement

Portland cement is manufactured by burning an intimate mixture of calcareous (mainly *limestone*) and *argillaceous (clay or shale)* and/or other silica, alumina or iron oxide bearing materials at approximately 1500^{0}C. The constituents combine by means of solid state diffusion to produce stone-hard dense lumps known as 'Portland cement clinker'.

The clinker is finely ground at which stage a small proportion (up to 5%) of *gypsum is added to retard* the initial setting of cement for sufficiently long to allow working of mixed materials (e.g. concrete) into a final desired position [41]. The manufactured cement consists essentially of finely ground calcium silicates and *aluminates with* small percentages of magnesium oxide, gypsum and uncombined calcium oxide. Table 2.1 lists the approximate compound composition of ordinary Portland cement.

The abbreviated notation of the compounds use one letter to describe each oxide, i.e. $CaO = C$, $SiO_2 = S$, $Al_2O_3 = A$, $Fe_2O_3 = F$, and $H_2O = H$.

Some of these compounds have hydraulic properties, i.e. when mixed with water they hydrate to form new compounds which interlock in a mixture of high physical strength. The hydration of the major constituents of portland cement i.e. tricalcium silicate and dicalcium silicate are largely responsible for the strength of hydrated cement.

The most commonly used cement is ordinary Portland cement, a general all purpose cement. Other cements with special properties are also used, generally for more specialized applications. These are manufactured either by altering the chemical composition of the clinker (e.g. sulphate resistant cement), finer grinding of the clinker (e.g. rapid-hardening cement) or adding other constituents e.g. blastfurnace slag or flyash.

Table 2.1: Approximate Compound Composition of Ordinary Portland Cement

Compound	Formula (Abbreviation)	Percentage (by weight)
Tricalcium silicate	$3CaO.SiO_2$ (C_3S)	35 - 55
Dicalcium silicate	$2CaO.SiO_2$ (C_2S)	20 - 40
Tricalcium Aluminate	$3CaO.Al_2O_3$ (C_3A)	5 - 12
Tetracalcium Aluminoferrite	$4CaO.Al_2O_3Fe_2O_3$ (C_4AF)	5 - 10
Gypsum	$CaO.SO_4.xH_2O$	4 - 7
Calcium oxide	CaO	0.5 - 2.5
Magnesium oxide	MgO	0.5 - 3

2.3.2 Soil-cement Reactions

The chemistry of hydrated cement is also not completely understood at this stage. As in the previous section, simplistic explanations are used to describe the complex reactions which produce hardened cement.

When water is added to cement, the cement-water reactions (hydration reactions) produce hydrates of the various compounds which are precipitated in gel form. The gel hardens owing to crystalline intergrowths formed by the hydration *compounds*.

The idealized hydration reactions of the major constituents are written as follows [42,43]:

Tricalcium Silicate

$$2C_3S + 6H \longrightarrow C_3S_2H_3 + 3Ca(OH)_2$$

Dicalcium Silicate
$2C_2S + 4H \longrightarrow C_3S_2H_3 + Ca(OH)_2$

Tricalcium aluminate
$C_3A + 6H \longrightarrow C_3AH_6$

Tetracalcium aluminoferrite
$C_4AF + 4Ca(OH)_2 + 22H \longrightarrow C_4AH_{13} + C_4FH_{13}$

The hydration compounds reach different strengths at different rates [41]:

- Tricalcium silicate hydrates rapidly and is chiefly responsible for the early strength of cement-water pastes.
- Dicalcium silicate hydrates slowly, contributing little strength before 28 days. It does however contribute significantly to the strength at later ages if sufficient water is available for continued hydration.
- Tricalcium aluminate does not contribute much to the cementing action. It accounts for the initial set and adds slightly to the early strength liberating large amounts of heat during the first few days of hardening.
- Tetracalcium aluminoferrite hydrates rapidly but develops very little strength.

When mixed with soil and water, the reactions which occur are similar to those presented in the case of clay-lime:

- The hydration of the calcium silicates releases lime as $Ca(OH)_2$. Reactive free lime (as CaO) present in the cement (i.e. CaO that is not over-burned) also hydrates to produce $Ca(OH)_2$. These $Ca(OH)_2$ products react with the clay in plastic soils as described in Section 2.2 resulting in a fairly rapid reduction in plasticity and an increase in strength [22].
- The major contribution to the strength of soil-cement comes however from the hardened cement which bonds the soil particles together providing early and long-term strength, and thus determining the engineering properties of cement treated materials [22].

Ingles [32] notes that granular soils require cementation at points of contact to maintain their dense state after compaction, while clay lumps must be encased by a 'shell' of cement which provides a moisture barrier and restricts volume changes.

He also points out however that a disadvantage of the high early strength type of structure is that once the structure has been ruptured it does not readily reform.

This implies that high cement contents may be required to effectively stabilize expansive or shrinkable clays. In this case, lime stabilization is certain to provide a cheaper solution with the added advantage of being capable of 'self-healing' early bond breakages because the cementing action is relatively slow.

2.4 Preconditioning Soils with Lime

The strengths of lime and cement stabilized soils strongly depend on the uniformity of the soil-stabilizer-water mixture before compaction, i.e. on how evenly the stabilizer has been distributed throughout the soil. However when either cement or lime is mixed with a highly plastic clay and the mixture moistened, clay lumps may stick together preventing a uniform mixture of soil and stabilizer.

'Preconditioning' such soils with lime is a fairly common method of achieving a more uniform mixture and thus obtaining higher strengths from plastic soils [25,31,33,37,44,45,46]. Firstly a small amount of lime (1-2%) is mixed with the soil. After water has been added, the mixture is allowed to stand for one or two days in a loose uncompacted state. The lime increases the plastic limit of the soil with the result that on subsequent wetting to the moisture content required for compaction, the clay lumps do not stick together. When further lime (or cement) is added for the purpose of cementation, it can therefore be more uniformly distributed throughout the soil, thereby improving the effectiveness of the stabilizer.

The same references also mention the 'helpful action of lime in breaking down clay clods'. Apparently it also becomes easier to break the clay lumps into smaller fragments after preconditioning with lime.

Chapter 3

SELECTION, PREPARATION AND MONITORING OF MATERIALS AND TEST SPECIMENS

3.1 The Soils

The particle size distributions of the four soils used in this project are shown in Figure 3.1. Throughout the text the soils are referred to as:

Soil A: A reddish brown clayey silt.

Soil B: A yellowish brown clayey silt.

Soil C: A light green silty clay.

Soil D: A greyish black clayey silt containing organic material.

The soils were obtained from the same site. Soils A, B and C appear to have originated by in situ weathering of the same parent material, with Soils A and B more highly weathered than Soil C. Soil D, a topsoil containing organic material, appeared to be alluvial.

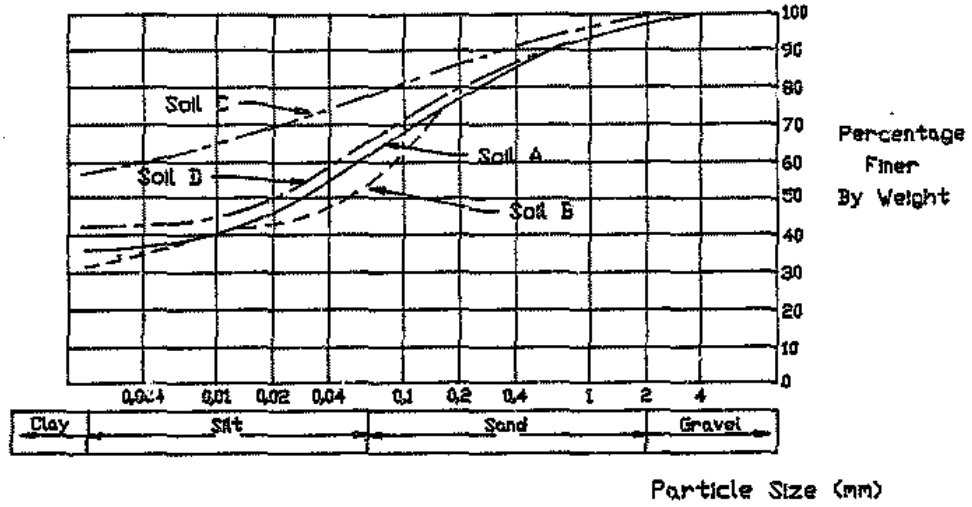


Figure 3.1: Particle Size Distributions of the Soils used in the Project

The predominant clay mineral was identified using X-Ray diffraction analyses as smectite, a highly expansive and lime reactive clay mineral. An X-Ray fluorescence scan indicated that the clay was a calcium smectite since a preponderance of calcium was indicated while little or no sodium was evident. All clays in inland South Africa are likely to be calcium clays since they were not deposited in a marine environment.

In accordance with the Portland Cement Association guidelines for the preparation of soils for laboratory testing [4] the soils were air dried, mixed thoroughly and passed through a # 4 sieve (4.75 mm opening). The material retained on the # 4 sieve was then crushed if in clod form, or discarded if granular. Soil C, a heavy clay, required a great deal of pulverising before passing through the No. 4 sieve. This resulted in lumps of clay (maximum size 4.75 mm) and clay 'powder'.

The soils were then thoroughly mixed to re-include the material that required pulverizing, and then stored in large steel drums covered with plastic sheeting. *The soils were not* oven dried since an elevated temperature can permanently alter the properties of a clay soil [47].

When soil was required for sample preparation, an amount of soil (approximately

5 kg) was taken from the drum and mixed thoroughly. After taking a moisture content sample from the mixed soil, the soil was sealed in a plastic bag and placed in an air tight bin for 48 hours before it was used. This preparation proved to be satisfactory for achieving accurate and uniform moisture contents in moulded samples.

Table 3.1 lists the results of the preliminary tests conducted on the soils to determine their index properties and compaction characteristics.

Table 3.1: Basic Properties of the Soils

Soil	Atterberg Limits			ASTM Soil Group Classification	Particle Specific Gravity
	L.L (%)	P.L (%)	P.I (%)		
A	46.9	28.8	18.1	CL	2.72
B	50.3	26.3	24.0	CL-CH	2.70
C	75.1	23.0	52.1	CH	2.73
D	63.1	32.9	30.2	MH	2.67

Compaction Characteristics

Soil	Modified AASHTO		Standard Proctor	
	Optimum Moisture Content	Maximum Dry Density	Optimum Moisture Content	Maximum Dry Density
	(%)	(kg/m^3)	(%)	(kg/m^3)
A	20.7	1654	24.3	1550
B	20.3	1682	22.5	1578
C	21.0	1648	24.8	1535
D	22.8	1552	25.8	1475

3.2 Stabilizing Agents

Initially, in order to keep up with the current trends in soil stabilization, it was hoped to stabilize the soils with cement, lime, and combinations of these stabilizers with fly ash. At the time that the research was conducted however, the use of South African fly ash as a pozzolan in soil stabilization had not proved as successful as elsewhere in the world.

It was thus decided to limit the scope of the project to the stabilizing of the four soils with varying percentages of cement and lime.

3.2.1 Cement

Ordinary Portland cement was used throughout the project. Sufficient cement for the entire project was sealed in plastic bags which were placed together with large bags of desiccant in air tight containers. Cement retains its properties over fairly long periods time if correctly stored and sealed from the ingress of carbon dioxide [9,41].

3.2.2 Lime

A lime commonly used for base and sub-base stabilization was selected for this project. The lime was described as a "super air-separated dry hydrated lime". The chemical analysis of the lime is shown in Table 3.2.

To prevent carbonation after opening the bag of lime, the lime was divided into smaller quantities and stored with bags of desiccant in airtight containers. In this way, the lime was minimally exposed before use in sample preparation. Any lime that was felt to be overexposed was discarded.

In considering the type of lime to use, unslaked lime was rejected b[illegible] its high heat of hydration was felt to be undesirable in dam construction.

Table 3.2: Chemical Analysis of Hydrated Lime

Available CaO	67.00%
Total Calcium as CaO	73.05%
Total Magnesium as MgO	0.63%
Silica + Insolubles as SiO_2	2.08%
Other Metals as R_2O_3	0.94%
Carbon Dioxide at Works	0.25%
Loss on Ignition	23.23%
Residue on 0.075mm sieve	1.50%

3.3 Stabilizer Contents

Before the testing programme could begin, it was necessary to select stabilizer contents at which the engineering properties would be measured. The feasibility of stabilized soil as a dam building material depends strongly on the quantity of stabilizing agent and the relative strength at a particular stabilizer content.

Although the unit costs (per dry mass) are similar for cement and lime, the soil-stabilizer reactions for soil-cement and soil-lime differ considerably. The stabilizer contents must therefore be selected appropriately.

3.3.1 Cement

The selection of cement contents was based on the Portland Cement Association 'Guidelines for the preparation of soils for laboratory testing' [4]. Minimum cement contents for soil-cement to be used in 'water control' are specified according to ASTM soil group classification.

To use soils A to D for 'water control', the recommended minimum cement content is 10%[1] [4]. It was felt that by using the recommended minimum as the upper bound of the cement content for this project, it would be possible to establish why this limit was necessary. Thus it was decided to investigate the properties of soils A to D stabilized with 5%, 7.5%, and 10% cement.

Because of the high plasticity of Soil C, 2% lime was added at the same time as the cement to facilitate mixing which was otherwise not possible.

3.3.2 Lime

The selection of lime content depends on a number of factors including:

- The initial lime demand to satisfy ion-exchange and flocculation reactions.
- The reduction in plasticity required to facilitate construction procedures.
- Whether the soil is lime reactive - not all clay soils react with lime to produce a substantial strength increase.

[1] Throughout the thesis, the stabilizer content is expressed as a percentage of the dry mass of the soil to which it is added.

- The strength requirements, i.e. short or long term strength. For 'cementation' additional lime must be provided to satisfy *long term pozzolanic* reactions if these are possible.

Since the last three factors were not known at the start of the project, the selection of the lime contents was based on the initial lime demand as determined by the Eades and Grim 'Quick Test' [48].

The 'Quick Test' measures the amount of lime required to raise the pH of the soil-water mixture to 12.4 (i.e. lime saturated) after one hour. Figure 3.2 shows the results of the Eades test for Soil C, indicating that at least 5% lime should be added to this soil to satisfy the initial lime demand. The results for the other three soils were very similar, i.e. all four soils had an initial lime demand of approximately 5%.

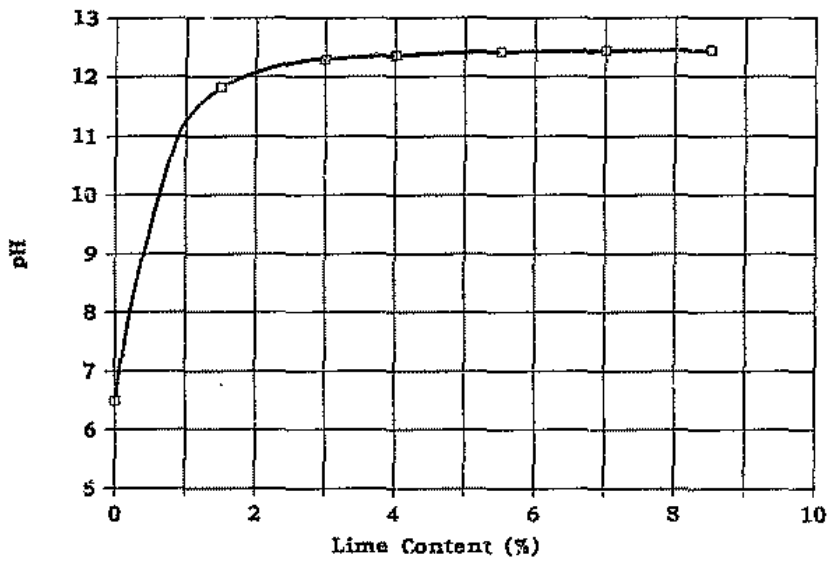


Figure 3.2: pH versus Lime Content (Eades and Grim 'Quick Test') - Soil C

This result was confirmed by the results of X-Ray diffraction analyses conducted on Soil C stabilized with increasing amounts of lime (Figure 3.3). The samples were wet-mixed to ensure maximum lime-clay reaction, and then cured for three months in sealed containers before the X-Ray analyses were conducted. The samples were air-dried and the mounts oriented to enhance basal reflections.

From the results it can be seen that at least 5% lime must be added before the clay is 'transformed', i.e. the peaks on the X-Ray diffraction trace at $2\theta \simeq 5^0$ and 12^0 (corresponding to the peaks normally obtained for smectite) are significantly suppressed. These results also agree with the results from the tests to determine the variation in plasticity with increasing lime content which show that 5% lime almost renders the material non-plastic. From the small peaks still evident at 7.5% lime, it is also apparent that significantly longer curing periods are required before the clay is 'destroyed'.

It should be noted that because of the variability of the properties of clay soils and the uncertainty which exists as to the exact reactions which may take place between the clay and the lime, a clear cut method for determining the minimum lime content does not exist. All the methods (reference [25] quotes nine possible methods) have limitations and thus either suggest or use supplemental strength tests to determine the lime content required to develop the strength desired for the particular application.

Although existing literature [23,25] indicates that there exists an optimum lime content above which the strength of the soil-lime mixture would decrease, it appears that this is applicable only to short term unconfined compressive strength (see Chapter 5). The amount of 'excess' lime required to "ensure a pH of 12.4 for sustaining the strength-producing soil-lime pozzolanic reactions" [23,25] can only be determined by testing the properties over longer periods, at lime contents higher than those required for the short term reactions.

In order to simplify the recording, analysis and presentation of data and the comparison of results with soil-cement, lime contents of 5%, 7.5% and 10% were selected.

3.4 Moulding and Curing Specimens

Standard cylinder specimens measuring 37.5 mm diameter by 75 mm long were used for the unconfined compressive strength tests, the triaxial tests and the permeability tests. The following procedure was used in the manufacturing of

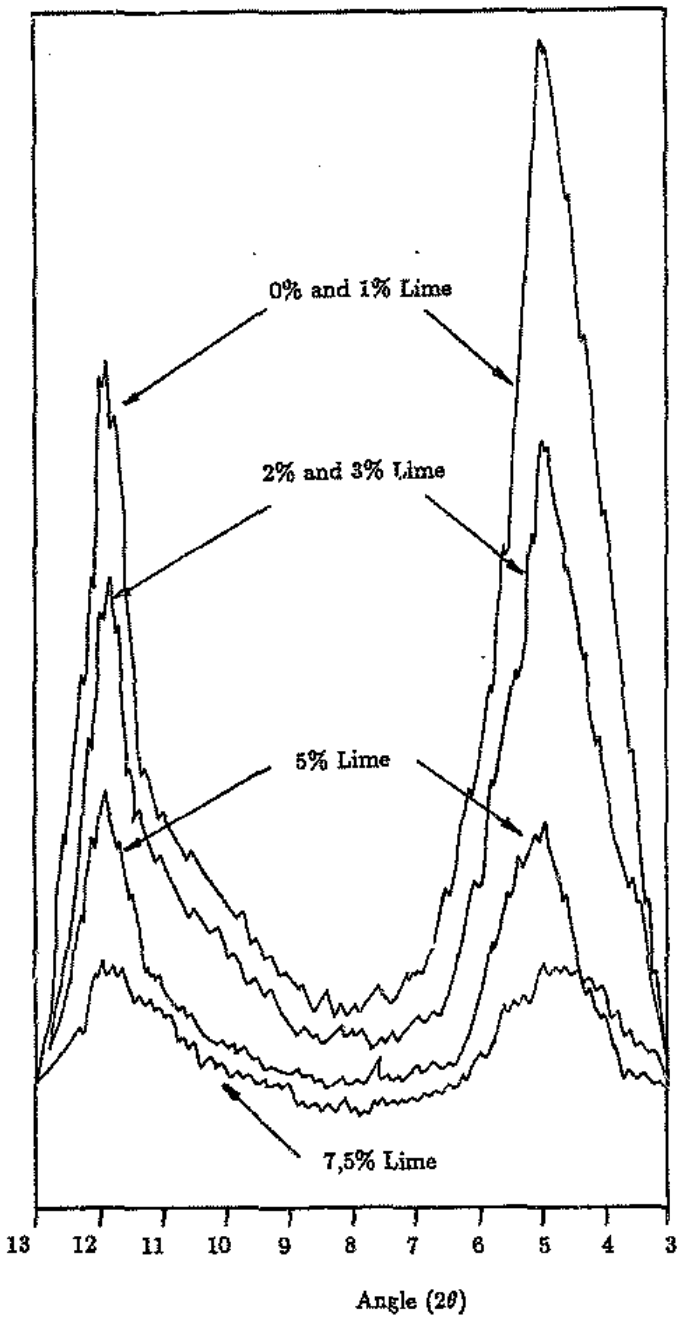


Figure 3.3: X-Ray Diffraction Trace for Soil C Stabilized with Lime (Specimens cured for six months at 30°C)

the specimens:

- The required quantities of stabilizer and soil were mixed in a dry state using a domestic food mixer with a dough hook.
- The required water was then added while mixing proceeded. It was found that because the mixtures initially remained plastic, they could not be adequately mixed in the mixer, and required supplemental hand mixing to break down lumps which stuck together. The total mixing time was approximately five minutes, although some materials required slightly longer mixing times.
- The required mass of stabilized soil was weighed and poured into a cylindrical mould designed to BS 1924 (1975). The material was continuously tamped into the mould with a tamping bar to obtain a uniform loose density.
- The mould was placed in a compression testing machine and the top and bottom 'ram' caps compressed until flange closure was obtained.
- The specimen was extruded, weighed and stored in a humid chamber (approximately 95% relative humidity).
- *Moisture content samples* were taken at the time that each specimen was being moulded. Moisture content determinations were made by oven drying these samples at approximately 110^0C.
- After the specimens had been cured in the humid chamber for 7 days, they were sealed by wrapping them firstly with a commercial thin plastic kitchen wrap, followed by a layer of aluminium foil. The wrapped specimens were then dipped in paraffin wax.
- The sealed specimens were cured for a further three weeks before testing, i.e. a total of 28 days curing.

Because of the size of the flexure specimens (300 mm long by 75 mm square), the materials for these specimens were hand mixed and the compacted specimens humid cured (95% relative humidity) for 28 days before testing.

3.5 Recording of Data and Analysis of Results

The primary objective of the research was to examine several properties of a variety of soil-stabilizer combinations with the intention of proving the feasibility

(or otherwise) of the use of alternative materials in dam construction.

In view of the large number of soil-stabilizer combinations to be tested (28 combinations per test type), and bearing in mind the fact that the results were not intended for design purposes, it was decided to conduct only one test of each type at each stabilizer content. The triaxial shear strength, flexure and shrinkage tests were time consuming and it was estimated that increasing the number of tests per property to three (for a statistically acceptable set of results) would have extended the laboratory programme by $1\frac{1}{2}$ to 2 years.

The number of specimens of a particular soil-stabilizer combination used for each test type was limited to:

- Triaxial shear strength test - four specimens per set, each specimen sheared at a different confining pressure.
- Unconfined compressive strength - three specimens per set.
- Permeability test - three specimens per set.
- Single specimens were used in the shrinkage and flexure tests.

The standard cylinders were moulded in sets of six or seven specimens, depending on the tests to be performed at the end of the curing period. Specimens for each test type were then randomly selected from within a set, e.g. four triaxial specimens and three UCS specimens.

The specimens were weighed before and after each procedure. A record of the mass of each specimen was kept to determine if any irregularities e.g. excessive drying through improper sealing, had occurred. No irregular moisture changes during the curing or testing periods were observed.

The majority of the test results are presented in graph form, using smooth curves to indicate the estimated trends. The range of the results for the UCS and permeability tests (three specimens per set) allowed these property trends to be estimated fairly accurately. When estimating the trends of the results obtained from single specimen sets e.g. the variation of peak shear strength with stabilizer content for a particular confining pressure, reliance was made on supportive trends i.e. similarities in trends between various stabilizer contents or similar soil-stabilizer combinations, to exclude apparently inconsistent data.

Chapter 4

THE COMPACTION CHARACTERISTICS OF STABILIZED SOILS

4.1 Introduction

Figure 4.1 shows the moisture-density curves for the untreated soils used in this project using Standard Proctor compactive effort, while Figure 4.2 shows the moisture-density curves using Modified AASHTO compactive effort.

Since the addition of a lime or cement to a soil results in changes to the index properties of the soil, the compaction characteristics of the material can also be expected to change. Figure 4.3 shows how the addition of cement and lime influences the compaction characteristics of Soil A [1].

Figures 4.4 and 4.5 show the variations of maximum dry density and optimum moisture content with cement and lime content respectively for all the soils. Note that 2% lime was added to the Soil C specimens stabilized with cement.

The results are similar to those commonly obtained for stabilized soils [25,31,49,50]. In general, the moisture-density curves for soil-cement show an apparent decrease in optimum moisture content and an increase in the maximum dry density, although for some soils the opposite may occur [49]. The majority of soils stabilized

[1] The stabilized soils were all compacted immediately after mixing using Standard Proctor compactive effort

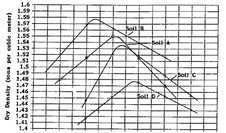


Figure 4.1: Moisture-density curves - Standard Proctor Compactive Effort

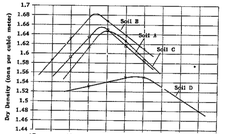


Figure 4.2: Moisture-density curves - Modified AASHTO Compactive Effort

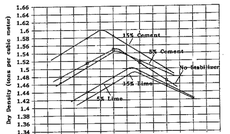
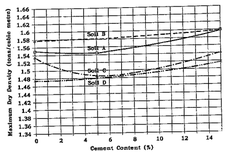


Figure 4.3: Moisture-density Curves for Soil A at Various Cement and Lime Contents - Standard Proctor Compactive Effort

with lime show an increase in the optimum moisture content and a decrease in the maximum dry density as in Figure 4.5.

Some aspects of the influences of stabilizing agents on the compaction characteristics of soils are discussed in the following sections. The determination of the magnitude by which a particular influence affects the compaction characteristics is beyond the scope of this project. However a discussion of the probable reasons for these changes is important, particularly to allay fears that a decrease in density resulting from the addition of a stabilizing agent might be deleterious to the engineering properties of the material.

Awareness of these influences is important in the determination of the compaction characteristics since the addition of cement or lime results in a different material with a new specific gravity, a different particle size distribution and a different "water demand". Some references (e.g. [49]) suggest using the unstabilized soil's optimum moisture content and maximum dry density for the compaction of the stabilized material. However, if for example the optimum moisture content for soil-cement mixtures are not corrected to allow for the consumption of compaction



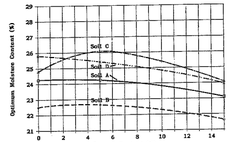
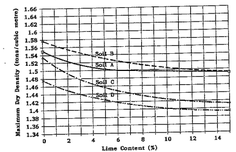


Figure 4.4: Maximum Dry Density and Optimum Moisture Content vs. Cement Content - Standard Proctor Compactive Effort



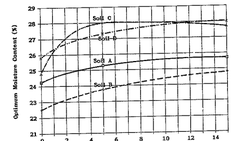


Figure 4.5: Maximum Dry Density and Optimum Moisture Content vs. Lime Content - Standard Proctor Compactive Effort

water by cement hydration (in laboratory specimens), then field compaction of the soil-cement would be carried out at a moisture content dry of the optimum moisture content (see Section 4.3).

4.2 Soil-lime

Lime has a lower specific gravity (s.g. = 2.2) than soil particles (s.g. = 2.6 to 2.8). A proportion of the decrease in the dry density may thus be attributable to the lower specific gravity of the 'composite' material. An accurate comparison would require that the volume of air voids be compared which in turn would require the determination of the specific gravity of the stabilized material at the time of the test.

As discussed in Chapter 2, the addition of lime causes clay to flocculate into a loose random structure which offers increased resistance to shearing. The stabilized material thus yields a lower maximum dry density at a higher optimum water content. Ingles [32] notes that a reduction in maximum dry density can serve as an indicator that a clay has in fact become flocculated.

From Figure 4.5, it can be seen that the reduction in density for amounts of lime in excess of 5% influence the compaction characteristics far less than amounts of lime up to 5%. For the soil (Soil A), the initial lime demand was determined to be approximately 5%, i.e. little further flocculation of the clay or increase in plastic limit is to be expected above the 5% lime content. Le-v, Abdelfatee and Mamdani [51] also found that the first increment of added lime (2%) caused the major proportion of the reduction in maximum dry density, while further additions of lime caused smaller reductions in density.

This result is important in that it indicates that lime in excess of the initial lime demand does not affect the compaction characteristics very much. Although short term strengths may decrease at lime contents above the initial lime demand, this results from the presence of unreacted lime which appears to reduce the short term cohesive strength, but not necessarily the shearing resistance (see Chapter 5).

Hydrated lime powder is approximately silt sized with approximately 75% by weight being smaller than 0.05 mm [28]. Since lime has a low solubility in water, it is possible that some of the lime behaves in a particulate fashion during compaction, and can be considered to increase the proportion of 'fines' to some extent. This does not seem to be significant as shown by the slight decrease in

density when the lime content is increased from 5% to 15%.

The flocculation of clay into silt-sized flocs does however modify the particle size distribution. The permanence of this modification can be demonstrated by comparing the rates of settlement in water of clay particles when flocculated and dispersed. The increase in the silt content may be beneficial to some soils in providing a void filler, while it may make other soils more single sized, yielding a larger void volume. However the overriding factor seems to be that the porous clay floc, being less dense than the unflocculated clay, will cause a decrease in the maximum dry density.

It is essential to note that the modification of the clay to a non-plastic state results in the material not having the capacity to maintain its remoulded shape even under small loads, when in an unconfined condition at early ages. The success of lime-stabilization depends to a large degree on the curing time allowed for the cementing action which will provide the required cohesion.

4.3 Soil-cement

Since the specific gravity (s.g) of cement is higher than that of the soil minerals (the s.g. of cement is approximately 3.14), part of the apparent increase may result from the new higher specific gravity of the combined materials. As with soil-lime, it would be necessary to compare the volume of voids in order to establish whether in fact an increase in density does occur due to this cause.

A further proportion of the increase in density can be attributed to the hydration products which begin to fill up the voids. The density of the compacted material continues to increase with time, as hydration proceeds and more cementitious material is produced [46].

A problem encountered with results reported by various researchers is that these results do not show the water content to which the mixture was brought before compaction, but the evaporable water after the cement has hydrated to some extent. Cement hydrates rapidly, and thus some of the added water is irreversibly consumed. This leads to lower water contents being measured than were actually added to the soil-cement mixture prior to compaction.

During this study, controlled tests which accounted for water evaporating to the atmosphere showed that moisture content specimens reflected lower values (0.5 to 1.0 percentage points or 2.5 to 5.0 percent of the water added) than they should

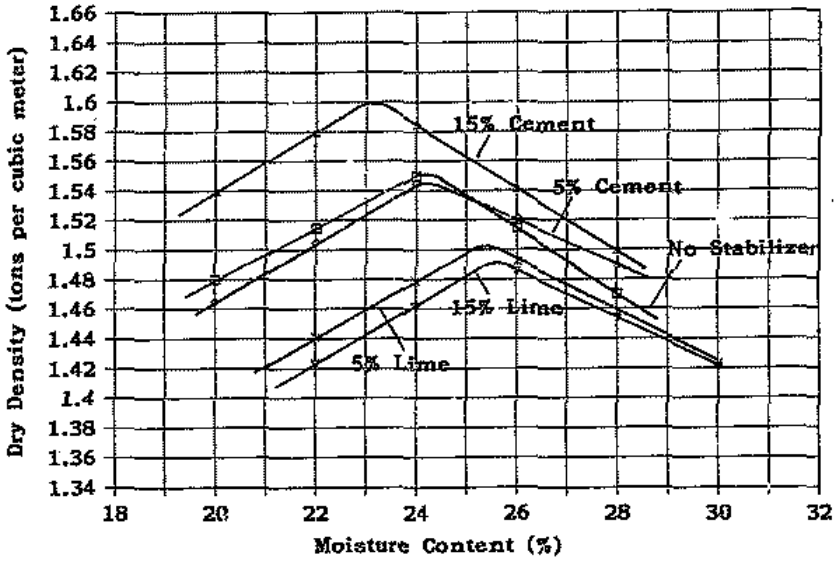


Figure 4.3: Moisture-density Curves for Soil A at Various Cement and Lime *Contents* - Standard Proctor Compactive Effort

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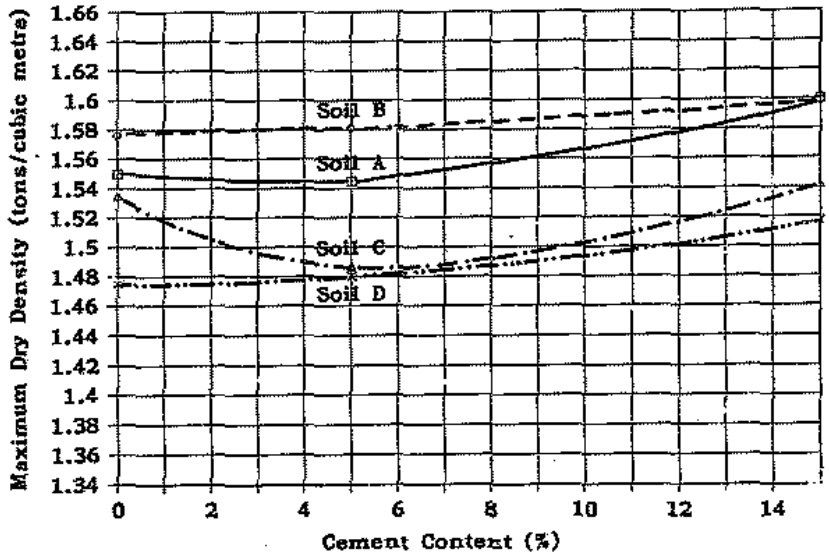

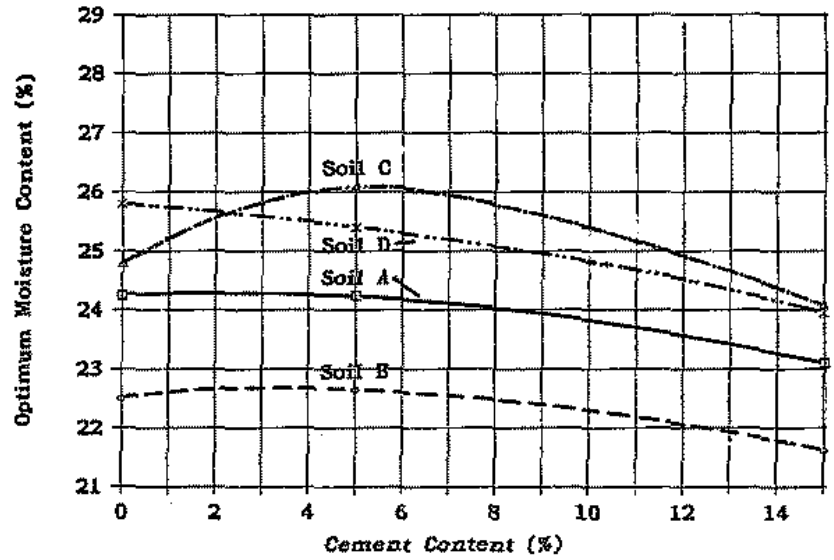

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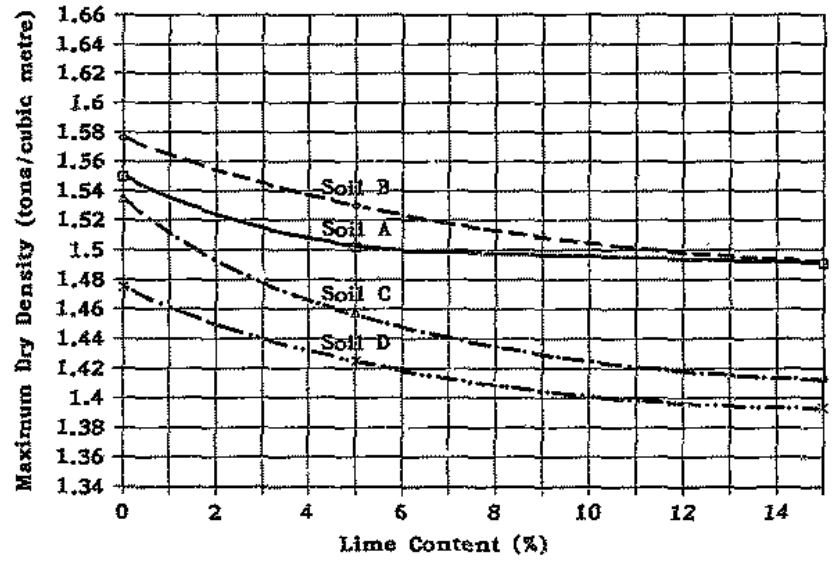

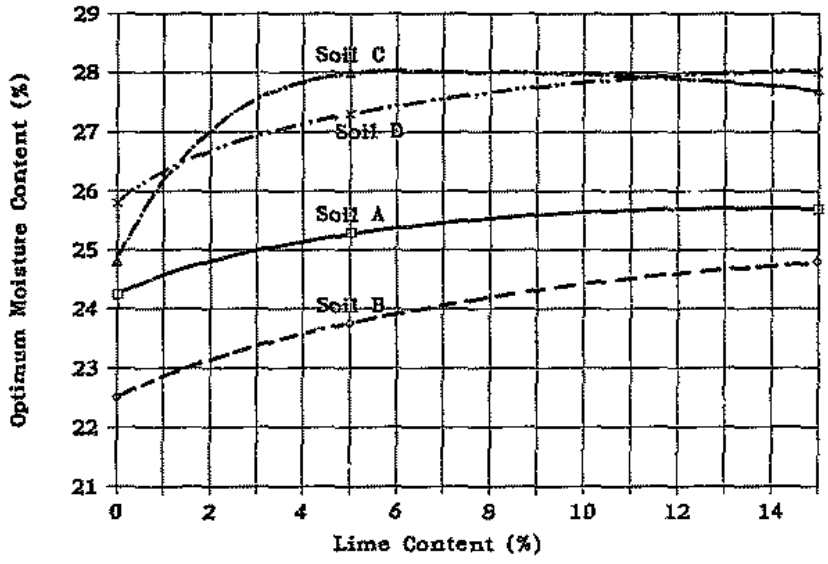

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A further proportion of the increase in density can be attributed to the hydration products which begin to fill up the voids. The density of the compacted material continues to increase with time, as hydration proceeds and more cementitious material is produced [49].

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During this study, controlled tests which accounted for water evaporating to the atmosphere showed that moisture content specimens reflected lower values (0.5 to 1.0 percentage points or 2.5 to 5.0 percent of the water added) than they should

have. The Highway Research Board [49] report that about 10% of the total water added to a clayey sand is used up during the first seven days in hydrating the cement.

The quantity of water consumed by hydration increases with cement content with the measured water content reduced by about 0.5% at 5% cement and 1.0% at 15% cement. If corrections for moisture consumed during hydration are applied to the moisture-density curves for soil-cement in Figure 4.4, then it becomes apparent that the optimum moisture content may be higher than the optimum moisture content for the unstabilized soil.

The hydration of cement can increase the mixture's resistance to compaction both as a result of flocculation of clay present in the soil, and as a result of the viscosity of the cement 'paste' [4]. These effects result in an increase in the optimum water content relative to the unstabilized material and a decrease in the maximum dry density. The influence of the increase in viscosity is time dependent, and thus delays between mixing and compacting can result in further increases in the optimum water content and decreases in density (see Section 4.4).

It would be highly impractical to attempt to determine the exact compaction characteristics i.e. based on void volume, immediately after compaction. The author believes however that the increase in specific gravity and the rapid formation of cementitious products (which can be assumed to continue to some extent during oven drying of water content specimens) tend to 'mask' the real influences of cement on the compaction characteristics. These may be opposite to what is shown by the moisture-density plot, i.e. the addition of cement actually increases the quantity of water required to achieve maximum density while the volume of voids may well be greater than the unstabilized material immediately after compaction.

Cement particles are also approximately silt sized and thus increase the silt content of the soil, together with any flocculation of clay by free lime and lime liberated during the cement hydration. As with lime, the influence of the cement depends on the original particle size distribution, on the quantity of added cement, and on how far the cement hydration has proceeded since the viscosity of the cement paste will determine how particulate is the behaviour of the cement particles within the paste.

The Highway Research Board [49] report increases in dry densities for uniform sands and clay while the densities of silts decrease on the addition of cement. However, it is not clear whether the measured increase in density arises solely from the change in the particle size distribution.

A practical implication of the above discussion is that specifications must include both the moisture content required for compaction and the moisture content to be expected at the time that moisture content determinations are made. Failure to do this may result in attempts to compact at a water content dry of optimum which will make it difficult for the required density to be achieved at the specified level of compaction. Compaction dry of optimum may also be deleterious to some soil-cement mixtures. Davidson et al [50] and the Highway Research Board [49] report that the water contents for maximum strengths of soil-cement mixtures are to the dry side of optimum for sandy soils and nonexpansive clays, but to the wet of optimum for expansive clays. Compaction of expansive clays dry of optimum may thus produce inferior soil-cement.

4.4 Delays Between Mixing and Compacting

4.4.1 Soil-cement

A delay between mixing and compacting can have negative influences on the compaction characteristics, the strength, and the durability of soil-cement [4,49,52]. The initial setting of cement occurs fairly rapidly and the viscosity of the cement paste increases with time, resulting in the need for more compaction water. Consequently the maximum dry density decreases for the same compactive effort.

To account for the discrepancy which may result between laboratory and field results, specifications for soil-cement construction require that moisture-density relationships be determined in the field, with soil-cement being taken directly from the area being constructed [4].

West [52] found that delays of up to six hours did not significantly alter the compaction characteristics of a uniformly graded sand to which 10% cement had been added, although slight decreases in strength were evident. On the other hand, the densities and strengths of both a sandy gravel and a medium plasticity clay decreased substantially even with short delays (one hour) between mixing and compacting.

West hypothesizes that the influence of the delay is dependent on the strength which the material is capable of developing (measured after 7 days in this case). The sandy gravel had the highest strength and showed the highest reductions in density and strength after delays between mixing and compaction, while the uniform sand had the lowest 7 day strength. The uniform sand-cement thus has a lower resistance to compaction than the other soils tested, and thus a

delay between mixing and compaction causes little change in the compaction characteristics and the strength.

West also found that remixing the material after a delay reduces the decreases in density and strength.

4.4.2 Soil-lime

When soil-lime is cured in a loose uncompacted state, cation exchange and flocculation reactions proceed, the rate of the reaction depending (among other factors) on the type of the clay, the clay content and the lime content.

These reactions may take some time to complete. Taylor and Arman [44] report that a major proportion of the change in plasticity occurs within the first 3 to 6 hours but that for higher clay contents, increases in the plastic limit are still evident after 42 days.

As discussed in Chapter 3, kaolinite has a much lower cation exchange capacity than smectite, and since cation exchange only occurs on the exterior surfaces of kaolinite particles, flocculation reactions will be completed quicker.

Moisture-density tests on soil-lime mixtures which are allowed to cure in a loose state before compaction reflect decreases in dry densities and increases in compaction water contents.

These results are attributed to [25,45]:

- The further flocculation and increase in plastic limit with time causing increased resistance to compaction.
- The reaction products from pozzolanic reactions begin to cement particles together, further increasing the resistance to compaction [45]. This is also time dependent i.e. the longer the material is allowed to cure before compaction, the more cemented it will become and the larger will be the resistance to degradation and deformation.

Using an expansive soil stabilized with 4% lime, Mitchell and Hooper [45] conclude that the delays in compaction are detrimental to density, strength and swell. The author does not feel that that their results are entirely conclusive since they do not establish the initial lime demand. Although their results indicate an increase in strength with time, it is not known whether sufficient lime was added to fully

satisfy the ion-exchange and flocculation reactions and to provide excess lime for cementation reactions, i.e. higher strengths may have been obtained at lime contents above 4%. Mitchell and Hooper did however find that if the density of the loose-cured material was increased to the same magnitude as the material which was compacted immediately after mixing, then the soaked and unsoaked strengths and the swell values were essentially the same.

If delays in compaction are likely to occur, for example if the plasticity of the clay must be reduced to render it workable, then it is suggested that:

- The lime fixation point be determined, i.e. the quantity of lime necessary to satisfy complete flocculation. The lime content should then be increased above this, the amount being determined by 'long term' strength tests (see Chapter 5) or after curing at elevated temperatures.
- The mixture must be 'sealed' by light compaction to prevent carbonation during the initial curing period.
- The highest economical compactive effort should be used for the final compaction since the lower the void volume, the less is the quantity of reaction products required to cement particles together. The increased cost of compacting to a higher level will be somewhat offset by the cost savings in first rendering the material workable.

The above process is sometimes carried out using two applications of lime. The first application (2 to 3% lime) is intended for the reduction in plasticity of the clay to expedite construction (modification), while the second application provides the additional lime for pozzolanic reactions (cementation).

4.5 Modification of Clays Prior to Cement Stabilization

If the strength of a soil-lime mixture is insufficient for a particular application, or if high early strengths are required, then it is possible to firstly modify the clay into a flocculated state and then use cement to stabilize the aggregated material.

The lime and cement can be added separately (double application) or together. The author found that it was unnecessary to use the double application method for soil-lime-cement mixtures prepared in the laboratory, and that a uniform mix

could be achieved with a heavy clay (Soil C) when the stabilizing agents were added together.

Figure 4.6 shows the moisture density curves for Soil C, which because of its high plasticity required a 2% supplement of lime before it could be stabilized with cement. The lime markedly improved the friability of the material indicating that flocculation occurred rapidly, although it required some 30 minutes of hand mixing before a satisfactory mix was obtained. On the other hand, the

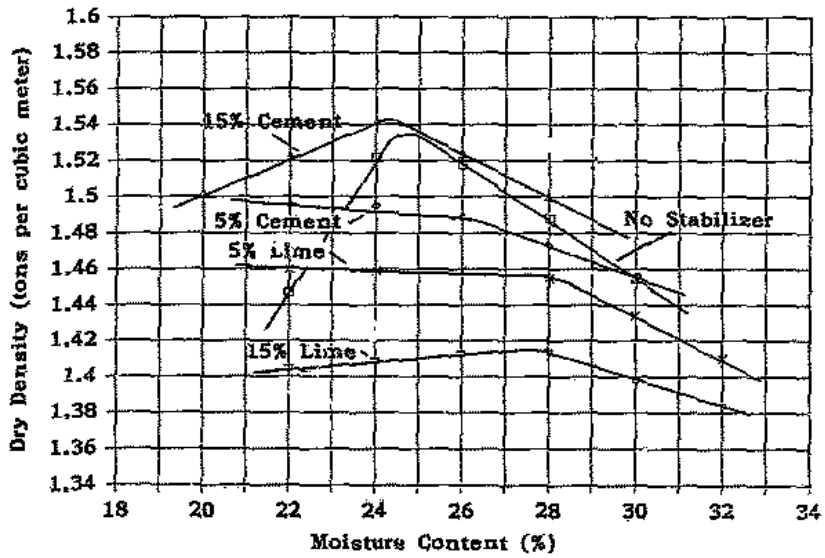

Figure 4.6: Moisture-density Curves for Soil C at Various Cement and Lime Contents

unstabilized material could not be mixed because of the clay's high plasticity, and uniform distribution of water throughout the unstabilized clay lumps could only be achieved by allowing the mixture to stand in a polythene bag for 2 to 3 days.

The flatness of the curve complicates the identification of the 'optimum' moisture content, and the selection was based on the appearance and feel of the soil. The optimum water content chosen for subsequent tests was at the point where the

decrease in dry density becomes pronounced. Below this water content, the soil felt too dry, and it was feared that using a moisture content below the chosen value might produce inferior soil-lime.

The addition of either lime or cement tended to flatten the moisture-density curves relative to the unstabilized material. The reason for the flattening of the curve which was also noted by Mitchell and Hooper [45] is not clear, although it appears to be linked to the degree of flocculation since the effect is more pronounced for lime stabilized material on the dry side of the optimum moisture content and for Soil C stabilized with cement and 2% lime.

Of interest to this section are the results of Pinto et al [53] who stabilized soils containing smectite with various percentages of lime and cement using a simultaneous application of both stabilizers. In addition to improving the engineering properties markedly, they found that the lime reduced the decrease in density caused by prolonged mixing i.e. the densities of samples stabilized with cement alone were lower than samples with similar cement contents, but containing lime.

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Chapter 5

THE STRENGTH OF STABILIZED SOILS

5.1 Introduction

The strength requirements of a stabilized soil are determined by its intended application. The more important design criteria for road and airfield pavement foundations are that the material has sufficient bearing capacity to sustain the predicted traffic loads, does not shrink or swell very much and is fairly permeable. The development of cohesive strength through cementation is of lesser importance since the layers are normally confined by surrounding material.

The economical use of stabilized soil in an embankment (where material might be in an unconfined condition or exposed to the elements) requires that the material develop sufficient frictional and cohesive strength so that the side slopes can be steepened to the extent that resulting cost savings are significantly greater than the cost of adding the stabilizer.

The following sections present the results of the laboratory strength determinations of unconfined compressive strength, triaxial shear strength and flexural strength of the four soils stabilized with varying quantities of lime and cement. The elastic properties are also discussed.

The triaxial testing programme was time consuming with one week required for the testing of each soil at a particular stabilizer content. It was thus decided to conduct triaxial strength tests at 28 days only, using UCS tests to examine

the variation in strength with time. The specimens used for UCS tests at the later age were the specimens used for the permeability determinations. It was felt that using these specimens would provide valuable information on the effect of leaching on the strength of the materials.

5.2 Unconfined Compressive Strength

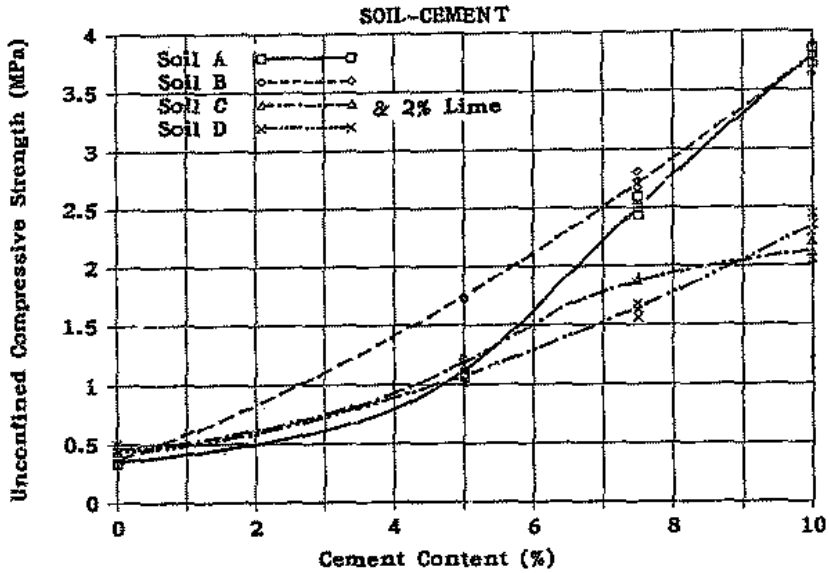
Figure 5.1 shows the variation of unconfined compressive strength at 28 days for each of the four soils stabilized with varying contents of cement and lime. The specimens were all tested in a dry condition since preliminary tests revealed that the soil-lime specimens had not developed sufficient permanent cohesive strength and crumbled when soaked.

It should be noted that in order to provide a quick visual comparison between the results for soil-cement and soil-lime for a particular set of data, the scale on the abscissa is kept the same for both stabilizers.

The soil-cement results all show that the UCS increases with increasing cement content. Soil C stabilized with cement appears to behave slightly differently to the other soils probably as a result of the 2% lime which was added at the same time as the cement. At higher cement contents it is possible that for this soil which has a high clay content, some of the added lime is not used up in ion-exchange and flocculation reactions and prevents the cementing together of soil particles.

In the case of the soil-lime the UCS of Soil A increases with increasing lime content up to 10%, but showed the lowest UCS at 5% lime. The strength of the other three soils appeared to reach a peak at 5% lime with Soils B and C showing firstly a reduction in strength as more lime was added (7.5%), and then slight increases in strength at 10% lime. The strength of Soil C at a lime content of 10% was higher than the strength at the peak at 5%.

Figures 5.2, 5.3 and 5.4 compare the UCS strengths of Soils A, B and C at 28 days (dry) and at 540 days (soaked) on a logarithmic time scale. After the normal 28 day curing period, the 540 day specimens were subjected to percolation for 240 days under a head of 50 m/m (see Chapter 8), and then stored in a water bath for 272 days. There was no specific reason for these time periods other than that sufficient permeability recordings had been obtained after 240 days, while it was felt that further storage would provide valuable information regarding long term strengths.



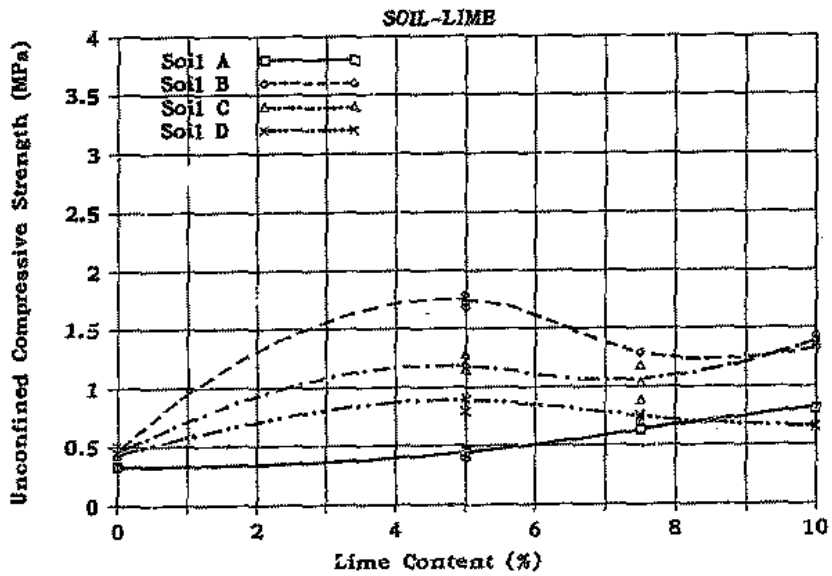


Figure 5.1: Unconfined Compressive Strength vs. Stabilizer Content at 28 Days

Soil D showed bad deterioration after percolation and wet storage, and UCS tests could not be performed on these specimens at any stabilizer content.

The 28 day dry strengths cannot be compared directly with the 540 day soaked strengths since part of the apparent reductions in strength can be attributed to the increase in the degree of saturation relieving negative pore water pressures. The plots are however useful in that they model the strengths of the materials under the different conditions to which they will be subjected in a dam, i.e. early dry strengths during construction and long term soaked strengths for normal operating conditions (dam full).

The results show trends which appear to be important to the choice of stabilizer and stabilizer content for an application such as a stabilized embankment dam.

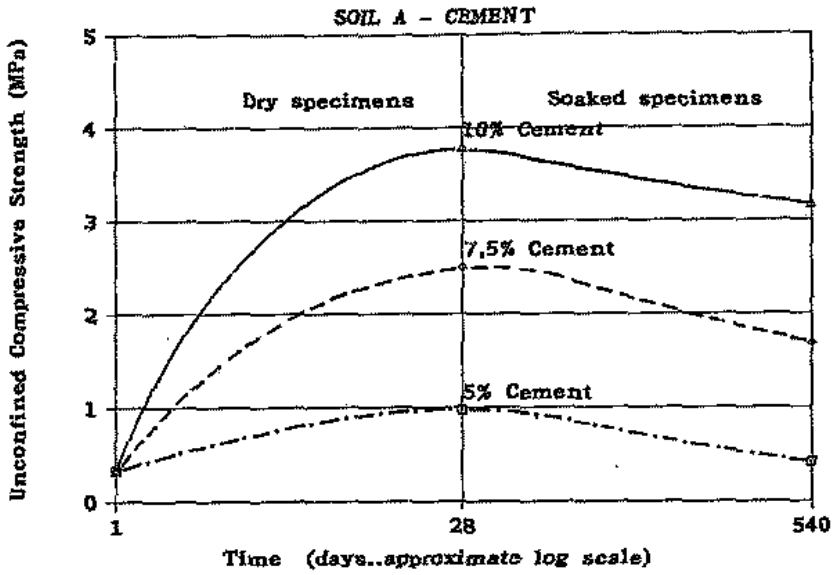
For Soils A and B, cement stabilization proved more effective than lime stabilization with higher strengths being achieved at all stabilizer contents and ages. Soil C stabilized with cement had higher early strengths than the lime stabilized material, but similar strengths at 540 days.

However the stabilizer contents of the Soil C-cement specimens are actually 2% higher than those indicated in the graphs (2% lime added to facilitate mixing). If this additional lime is taken into account, i.e. if the 7.5% lime is compared with the 5% cement (plus 2% lime), or the 10% lime compared with the 7.5% cement, then it is clear that Soil C appears to be more efficiently stabilized with lime.

Since the wet strength of Soil B stabilized with 10% cement was higher than the 28 day dry strength, it appears that some loss of strength through leaching has occurred in specimens at lower cement contents. The wet-dry tests (Chapter 10) showed that only Soils A and B stabilized with 10% cement appeared to retain a large proportion of their early strengths after leaching and soaking. This finding concurs with the Portland Cement Association's recommendation [4] that a minimum of 10% cement be added to Soils A and B if the stabilized material is to be used for water control structures.

The same trend of lower wet strengths at 540 days did not occur with Soil C even at the lowest cement content indicating that the 2% lime supplement appears to reduce strength losses in the soil-cement when subjected to percolation. The excess lime appears to reduce the strength loss by keeping calcium saturated water flowing through the specimen.

This agrees with Muller's [54] description for the corrosion of concrete where he notes that hydrated calcium silicates are stable only in highly alkaline conditions. With the removal of calcium hydroxide by flowing water, the pH is lowered and



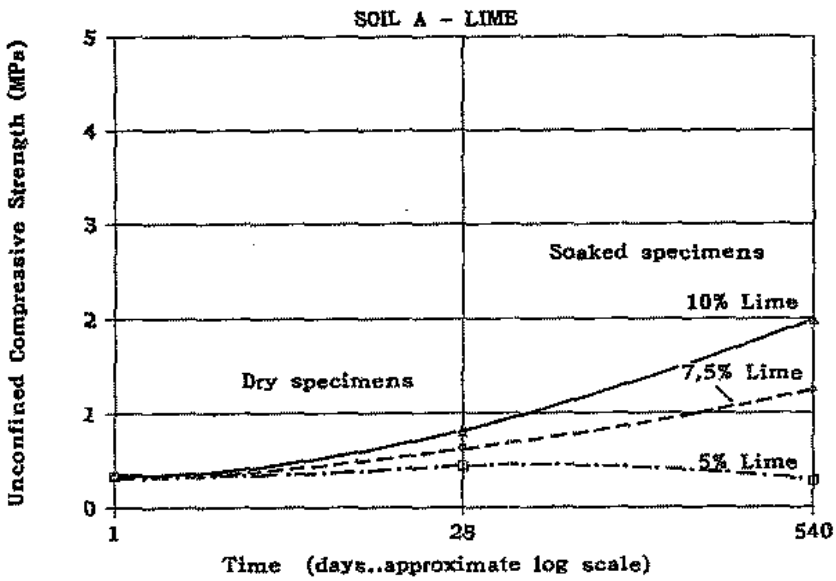
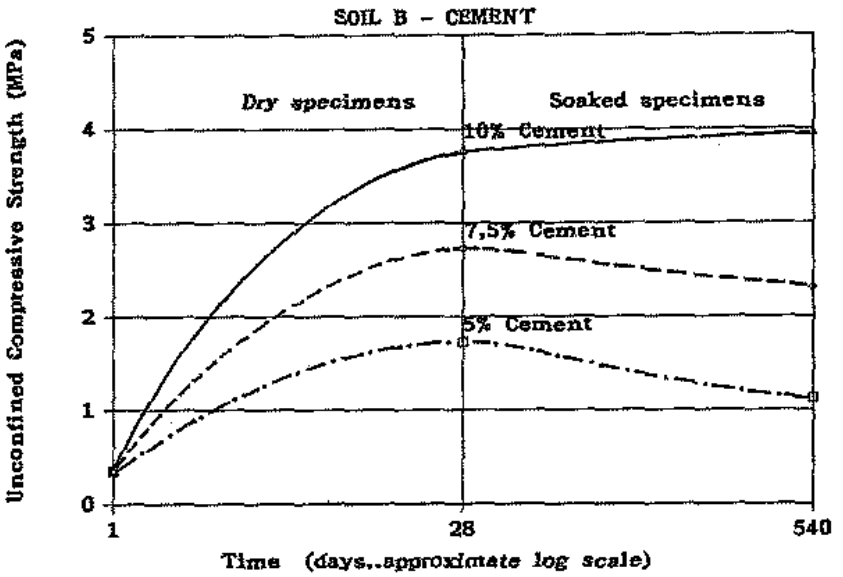


Figure 5.2: Unconfined Compressive Strength vs. Age - Soil A.



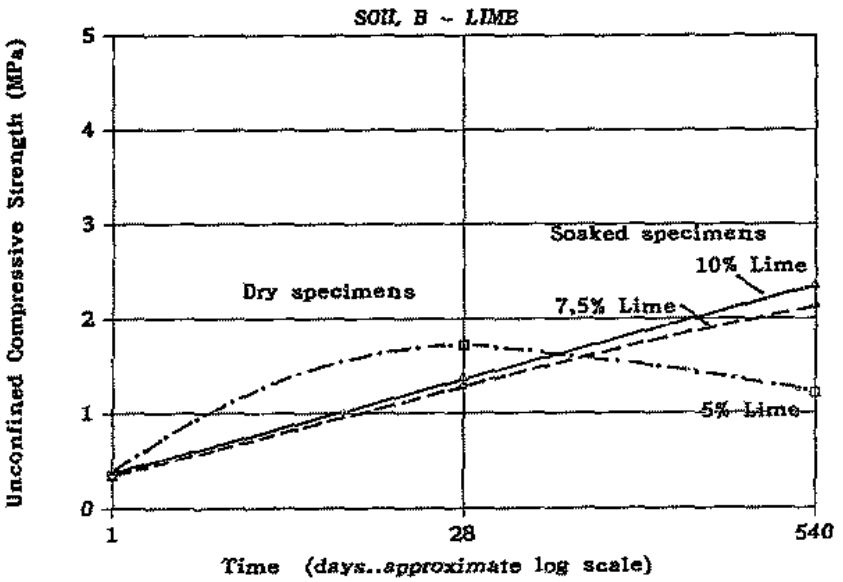
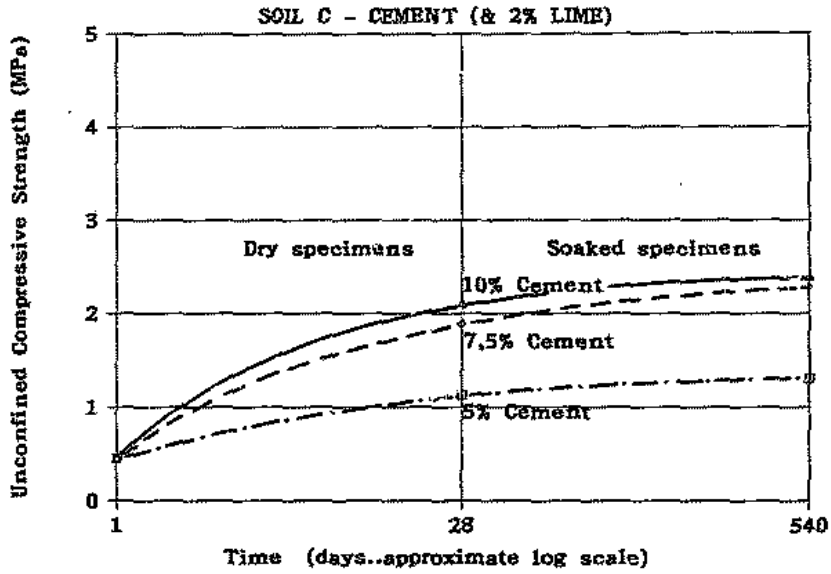


Figure 5.3: Unconfined Compressive Strength vs. Age - Soil B



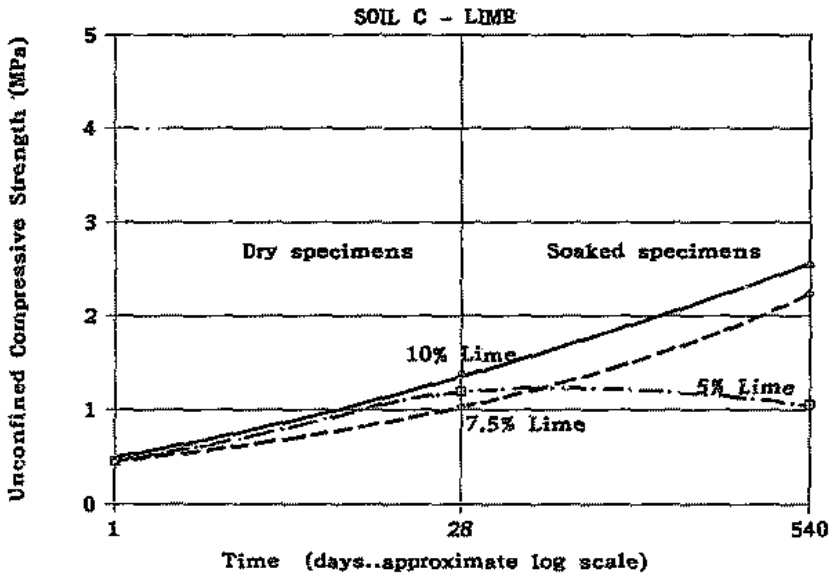


Figure 5.4: Unconfined Compressive Strength *vs.* Age - Soil C

the decomposition of calcium silicates takes place.

In the case of the soil-lime specimens, leaching apparently resulted in strength reductions in the 5% lime specimens for all the soils. Although a proportion of this loss in strength may be attributable to the relieving of negative pore water pressures through saturation, it might also indicate that insufficient lime is present for any pozzolanic reactions to occur. For the 7.5% and 10% lime stabilized specimens, the leached strengths were higher than the 28 day dry strengths, indicating that pozzolanic reactions occurred during the periods of leaching and soaking.

Of major significance is the trend of increasing long term wet strength with increasing lime content for each of the three soils which indicates that the lime content must not be selected on the basis apparent 'peak' or 'optimum' strengths from 28 day UCS results.

Since apparently excessive quantities of lime will actually reduce the cohesive strength of a clay soil, the use of the UCS test at early ages has lead to the belief that an optimum lime content for maximum strength exists since above a certain lime content the UCS decreases.

Although a peak appears in the UCS versus lime content curves (at 28 days) for Soils B, C and D at a lime content of 5%, the long term UCS tests (540 days) show that this peak does not represent the optimum lime content for maximum strength but that the strength continues to increase above the apparent peak as the lime content increases.

Davidson et al [50] make the same conclusions on the basis of the results of UCS tests conducted at later ages. They find that the rate of strength gain is influenced by the amount of lime and recommend that the highest economical percentage of lime be used where the project requires long term strength.

The results also substantiate that lime in excess of the initial lime demand (which was approximately 5% for each of the soils) must be added if long term strength gains through pozzolanic reactions are to be expected, and that the reaction products are in fact permanent. It is also apparent that higher lime contents yield higher rates of gain in strength.

Unfortunately a comparison was not made with specimens cured for long periods under sealed conditions, since at the time the above results were unexpected. Generally it has been found [9,22,31,49] that the strengths of both soil-lime and soil-cement specimens cured at constant moisture contents increase approximately linearly with the logarithm of age. Although not all soil-stabilizers follow

this trend exactly, the author found no other instance where the rate of gain in strength increased with the logarithm of time as occurred with the soil-lime specimens in this project.

It does seem possible that one of the factors contributing to the gain in strength of soil-lime is the calcium saturated percolating water which is able to stabilize the interior of clay lumps which would previously have acted as localized weaknesses in the stronger cemented matrix.

On examining the dry and wet unconfined compressive strengths of a highly plastic clay and a sandy clay stabilized with 4% and 7% cement or lime, Kennedy et al [55] found that lime stabilization produced higher dry strengths in the highly plastic clay, while cement produced higher dry strengths in the sandy clay. However on soaking the test specimens, the strengths of the soil-lime mixtures decreased far less than the strengths of the soil-cement mixtures (at 7% stabilizer content), with the wet strength of the sandy clay stabilized with lime being far higher than the wet strength of the cement stabilized specimens. They conclude that for both soils, lime is a more effective stabilizer in reducing the soils' susceptibility to moisture changes.

A notable point from their results is that the strengths of the soils stabilized with 4% lime appeared to remain constant as the curing time was extended, whereas the strengths of the 7% lime specimens increased with curing time, indicating that significant pozzolanic reactions are occurring only in the 7% lime specimens.

5.3 Shear Strength

The Mohr-Coulomb failure theory is widely used to characterize the shear strength of soils. It states that:

$$\tau = c' + \sigma' tan\phi' \text{.....(in terms of effective stresses)}$$

where τ is the shear stress at failure on the failure plane, c' is the cohesion intercept, σ' is the normal effective stress on the failure plane and ϕ' is the angle of shearing resistance.

To compare the variation of the components of shear strength i.e. c' and ϕ', four specimens of each soil-stabilizer combination were tested under triaxial loading conditions after a 28 day curing period. Each specimen was firstly saturated under a back water pressure. The specimens were then consolidated to effective confining pressures (cell pressure minus back pressure) of 100, 350, 600 and 850

kPa, after which they were sheared undrained (consolidated undrained (*CU*) triaxial test). The pore water pressure was recorded at each load/deflection reading.

The results at peak shear strength were used to plot Mohr failure envelopes using an approximate linear fit, from which the values of c' and ϕ' were obtained. The failure envelopes are not presented since the trends of the variations in c' and ϕ' are of particular interest, while the results from the plots of these trends provide sufficient information from which the Mohr envelopes can be constructed.

Difficulty was experienced in fitting a Mohr envelope to some of the results. This arose from an apparent 'instability' of these materials at higher confining pressures. This is discussed in Section 5.4 where the stress/strain behaviour of the soils is discussed.

The values of c' and ϕ' presented may thus vary to some extent since for example increasing the slope of the failure envelope, i.e. increasing ϕ', will lead to a decrease in the magnitude of c'. The author is confident however that the results describe the behaviour trends fairly accurately.

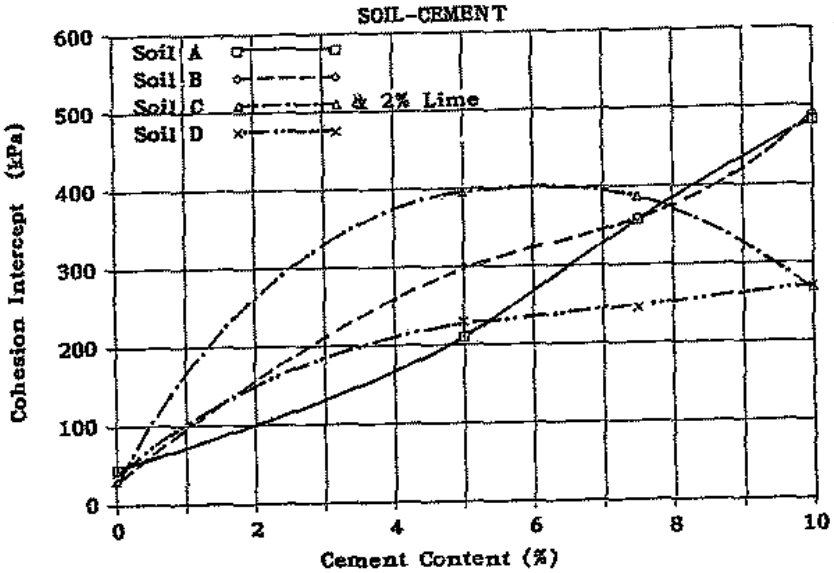
5.3.1 Cohesion

The cohesion intercept may have two components:

- 'True' cohesion which results from permanent interparticle bonding by cementing agents.
- 'Apparent' cohesion which results from negative pore pressures set up in the soil, for example in compacted soils which are always partially saturated immediately after compaction.

The addition of cement to any soil, and the addition of lime to a lime-reactive soil can thus be expected to increase the true cohesion of the soil, while the apparent cohesion may increase through self-desiccation when pore water is used up in the hydration of the stabilizing agent. Saturation of triaxial specimens largely removes the contribution of the apparent cohesion.

Figure 5.5 summarizes the values of the cohesion intercept obtained for the soils stabilized with cement and lime. For both stabilizers, the variations of cohesion with increasing stabilizer content are similar to the trends obtained for the unconfined compressive strengths (Figure 5.1).



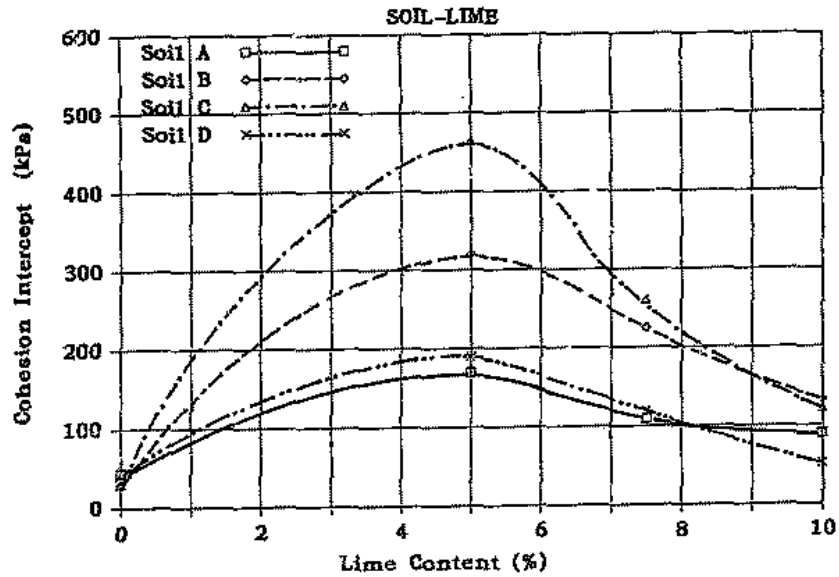


Figure 5.5: Cohesion Intercept vs. *Stabilizer Content* - 28 Day Curing Period

With the exception of Soil C to which 2% lime was added, the cohesion increases with increasing cement content. The decrease in cohesive strength for Soil C at cement contents above 5% may be caused by unreacted lime. The same trend occurs for all the soils when lime in excess of 5% is added, where significant reductions of cohesive strength occur.

The apparent decrease in cohesion may however have resulted from erroneous interpretation of the results caused by the instability of Soil C at higher confining pressures (see Section 5.4). Shear strength parameters of $c' = 375$ kPa and $\phi' = 34^0$ may be more appropriate for Soil C stabilized with 10% cement (and 2% lime), in which case the trends of increasing c' and ϕ' with cement content will be similar to the other soils.

The cohesion intercept of stabilized soils has been found to increase approximately linearly with the logarithm of age [9,25,31,56]. The same references find that the cohesion intercept also increases linearly with cement content.

Lees et al [51] obtain results similar to those presented in Figure 5.5 for soil-lime, where higher lime contents appear to cause a reduction in the cohesive strength or have little effect on the increase in cohesive strength at an early age (7 days).

5.3.2 Angle of shearing resistance

When a soil is sheared, a failure plane (or 'slip' plane) develops on which shear strains are concentrated. The frictional shear resistance developed along this plane is essentially derived from two components which contribute to the angle of shearing resistance of the material (ϕ'):

- Sliding friction where the relative movement of one soil particle to another is resisted through the development of adhesive friction forces at points of contact [28]. Microscopic interlocking because of surface roughness of particles may contribute to the sliding resistance between particles [57]. Negligible movement normal to the failure plane results from sliding friction.
- Macroscopic interlocking of particles which causes appreciable movement of particles normal to the failure plane during shearing, i.e. for shear deformations to occur, particles have to move up and over each other. This results in an increase in volume (dilatation) prior to failure [57].

Both cement [9,49,56] and lime [25,51,58,59] stabilization result in increases in the angle of shearing resistance.

Figure 5.6 shows the variation of the angle of shearing resistance with increasing stabilizer contents for the four soils at 28 days. The magnitude of the change in ϕ' is smallest in coarse materials which have initially high values of ϕ', and largest for stabilized clay soils having initially low values of ϕ'.

The rapid development of the angle of shearing resistance of soil-lime indicates that the change is largely attributable to the flocculation of the clay. Lees et al [51] and Fossberg [58] also observed significant increases in ϕ' after 28 days moist curing. Fossberg observed a slight decrease in ϕ' after 4 months moist curing, accompanied by a large increase in the magnitude of the cohesion intercept. It is possible that the 'instability' of soil-lime at higher consolidation pressures (which Fossberg observed in consolidation tests) leads to higher assumed magnitudes of ϕ' at 28 days (see 5.4).

5.3.3 'Peak' shear strength

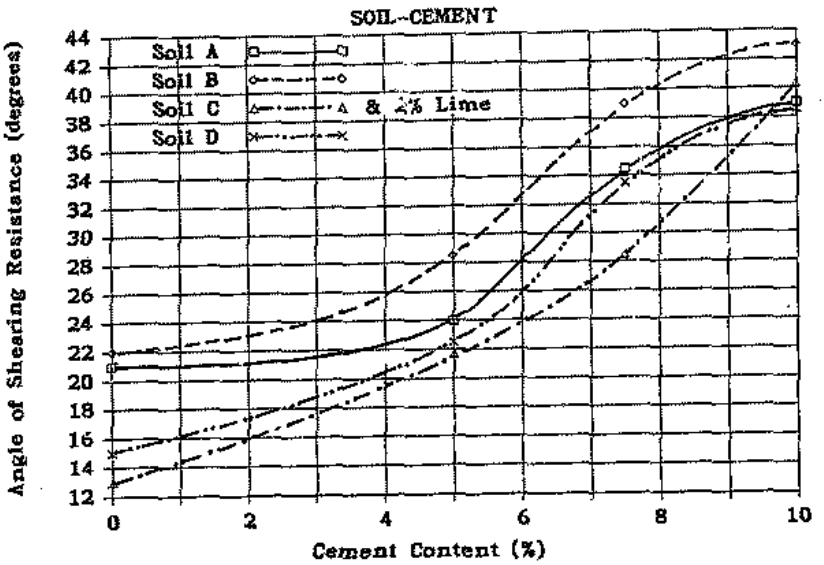
To observe the overall effect that the stabilizer is having on the shear strength it is necessary to look at the combined effects of the stabilizer on c' and ϕ'. Figures 5.7 and 5.8 show the variations of peak deviator stress with increasing stabilizer content as measured in the undrained triaxial tests at the various confining pressures.

These results indicate that in most cases, the shear strength increases with increasing stabilizer content after 28 days curing. The soil-lime results indicate that there is no significant peak at any stabilizer content although for Soils B and D there appears to be a slight drop in strength from 7.5% to 10% lime.

The results again confirm that at early ages, cement stabilization at stabilizer contents above 5% produced higher strengths than lime stabilization. From the UCS results, it appears that except for the highly plastic soil (Soil C) it is unlikely that the long term strengths of the soil-lime mixtures will approach those of the high stabilizer content soil-cement mixtures.

5.4 Stress-strain Behaviour

Since presentation of the large number of stress versus strain and pore water pressure versus strain curves would detract from the readability of the text, these curves are included in Appendix C. Sketch forms of these curves are used where required, while the results are summarized in abbreviated graph form.



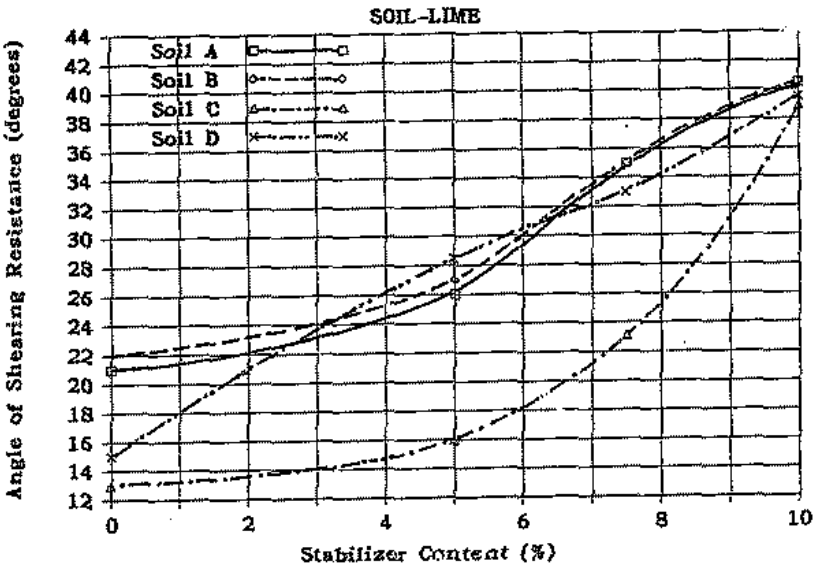


Figure 5.6: Angle of Shearing Resistance vs. Stabilizer Content - 28 Day Curing Period

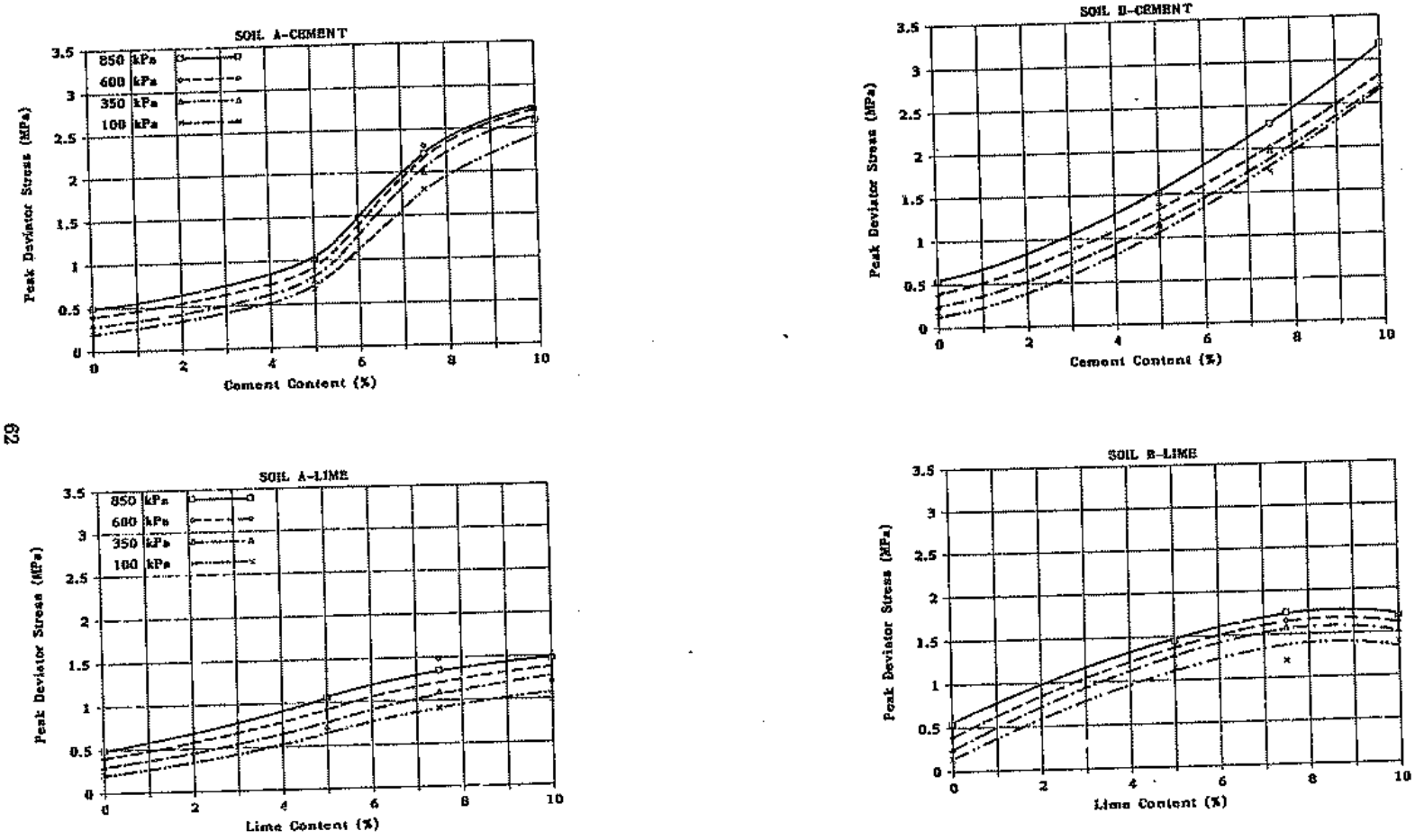


Figure 5.7: Peak Deviator Stress versus Stabilizer Content - Soils A and B

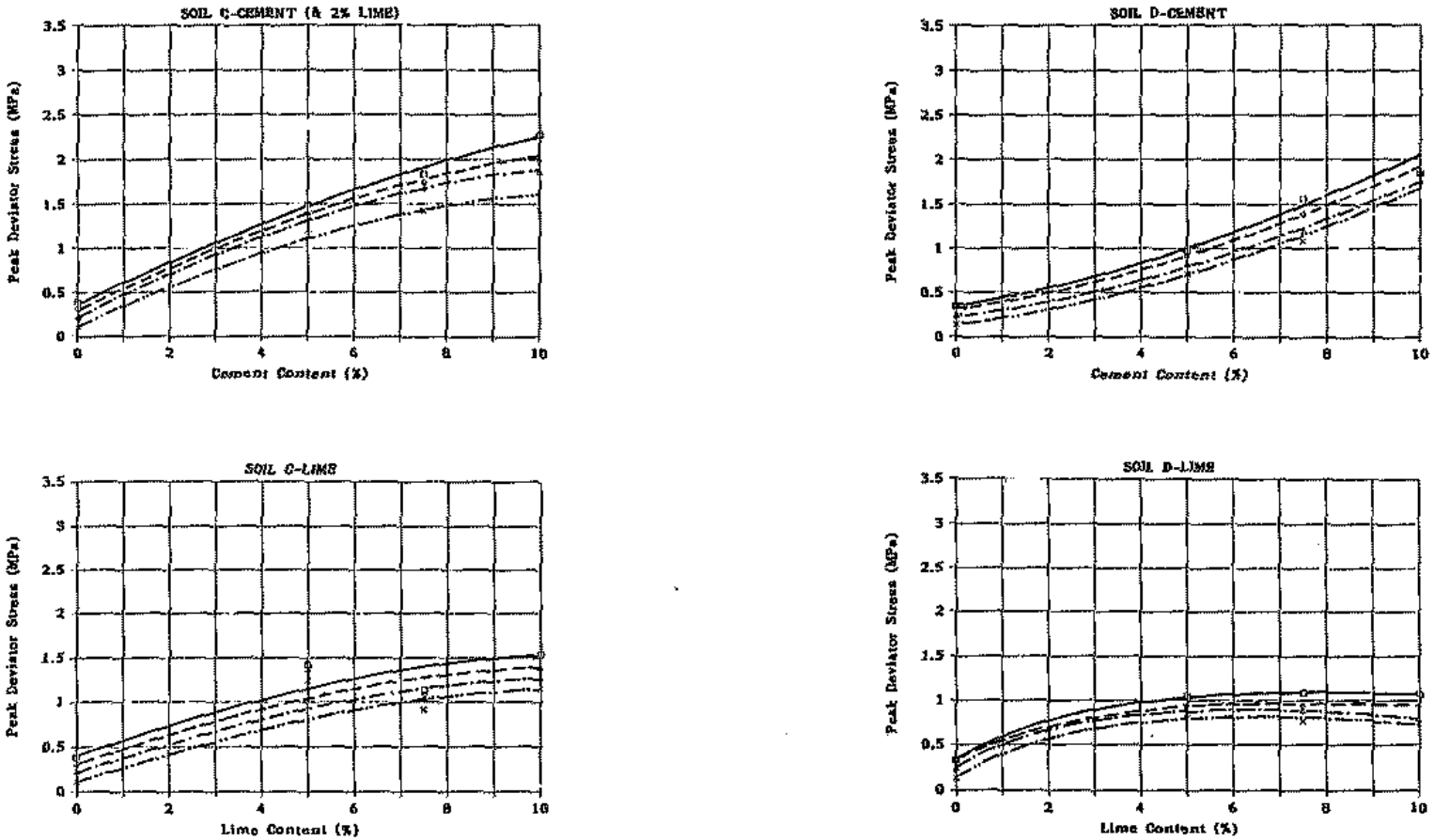


Figure 5.8: Peak Deviator Stress versus Stabilizer Content - Soils C and D

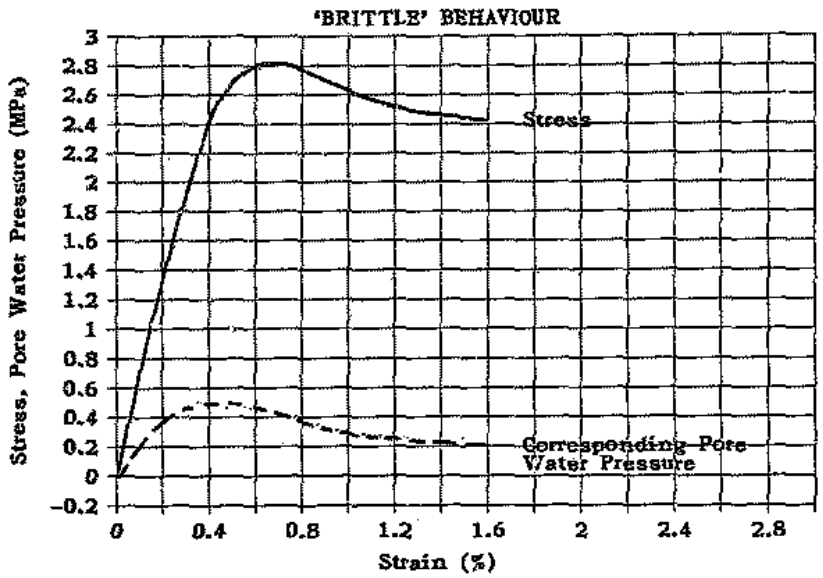
Figure 5.9 summarizes the shapes of the stress-strain curves which, together with visual observations indicate that:

- The failure of higher cement content specimens (7.5 and 10%) was generally of a brittle nature ('brittle' behaviour curve), with the development of a single shear 'fracture'. The pore water pressure increased to a maximum before the peak shear stress was reached and then decreased as the specimen was sheared further.
- The soil-lime specimens and the lower (5%) cement content specimens appeared to behave in a ductile fashion ('ductile' behaviour curve), the shear stress increasing at a decreasing rate with increasing strain, until the maximum shear stress was reached. These specimens 'barrelled' near the top or bottom. At the highest confining pressure (850 kPa) the pore water pressure in these specimens increased gradually (curve A) until a maximum at the peak shear stress, decreasing slightly as shearing continued. At the lower confining pressures (curve B), the pore water pressure reached a maximum before the peak shear stress was reached and then decreased as shearing continued.

Figure 5.10 shows the effective stress paths for these specimens, while Figure 5.11 shows corresponding plots of the pore pressure parameter ($A = \frac{\Delta u}{\Delta \sigma_1}$) versus strain. Figures 5.12 show the variation of the pore pressure parameter at failure with stabilizer content at a confining pressure of 850 kPa. Figure 5.13 shows the variation of the pore pressure parameter at failure at the various confining pressures for Soil B stabilized with cement and lime. The behaviour at the various confining pressures was similar for all the soils.

The pore pressure parameter decreases significantly on the addition of a stabilizing agent, and continues to decrease as the stabilizer content is increased. The magnitudes of the pore pressure parameter at failure reflected in Figure 5.11 e.g. A = 0.2 - 0.3 at 10% cement (confining pressure = 850 kPa) indicate that construction pore water pressures may become significant. However until the dam reaches a significant height, (850 kPa is equivalent to an embankment height of approximately 40 m) one would be justified in using A values obtained for low confining pressures (A = 0.0 to 0.15). As construction proceeds i.e. at later ages, it is envisaged that much of the pore water available at an early age will have been used up in cement hydration or clay/lime reactions, thus relieving construction pore pressures to some extent. This hypothesis will however require verification under larger scale conditions.

A slight but noticeable differ ce in behaviour patterns during the consolidation



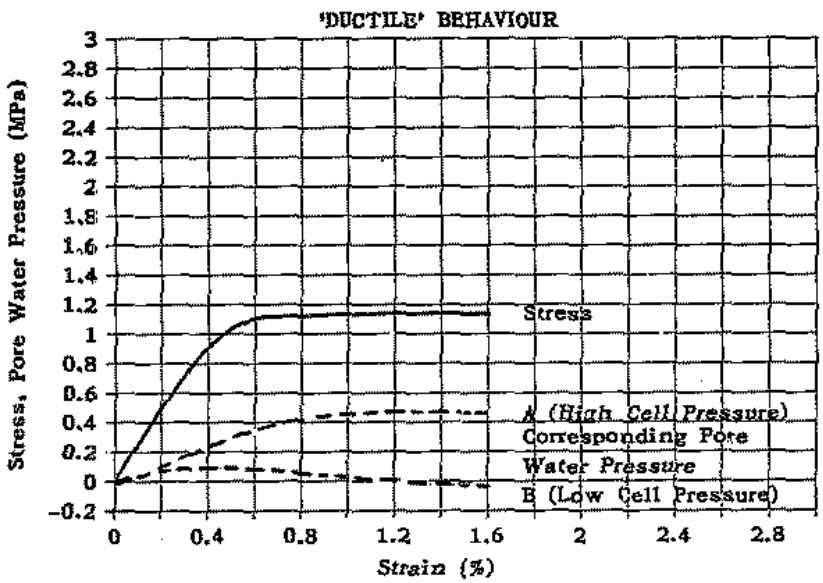


Figure 5.9: Typical Stress vs. Strain and Pore Water Pressure vs. Strain Curves

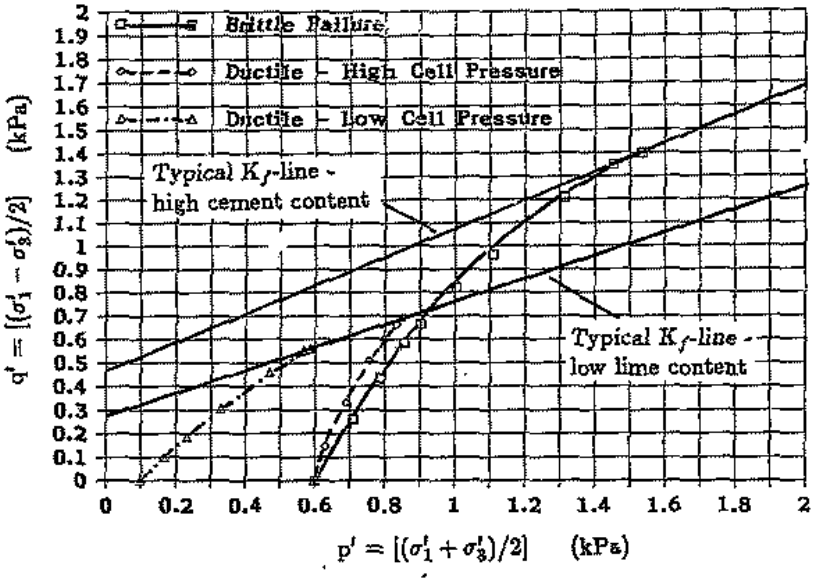


Figure 5.10: Typical *Effective Stress Paths*

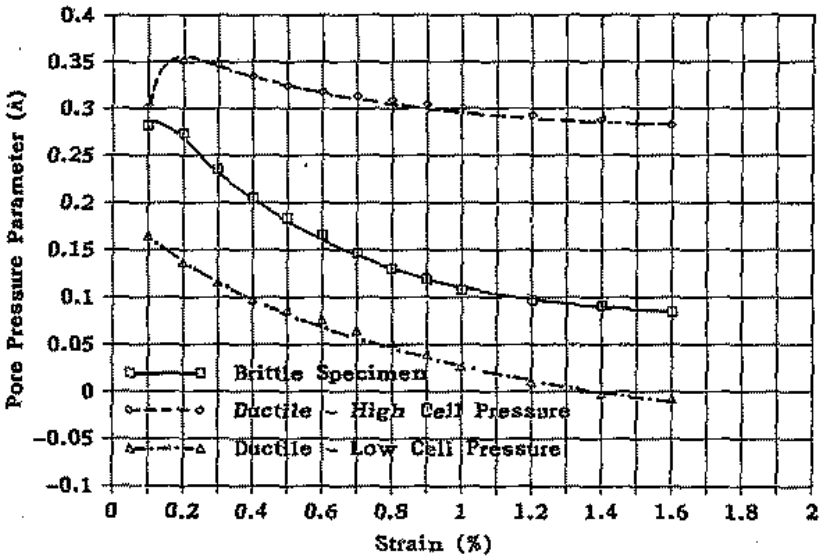
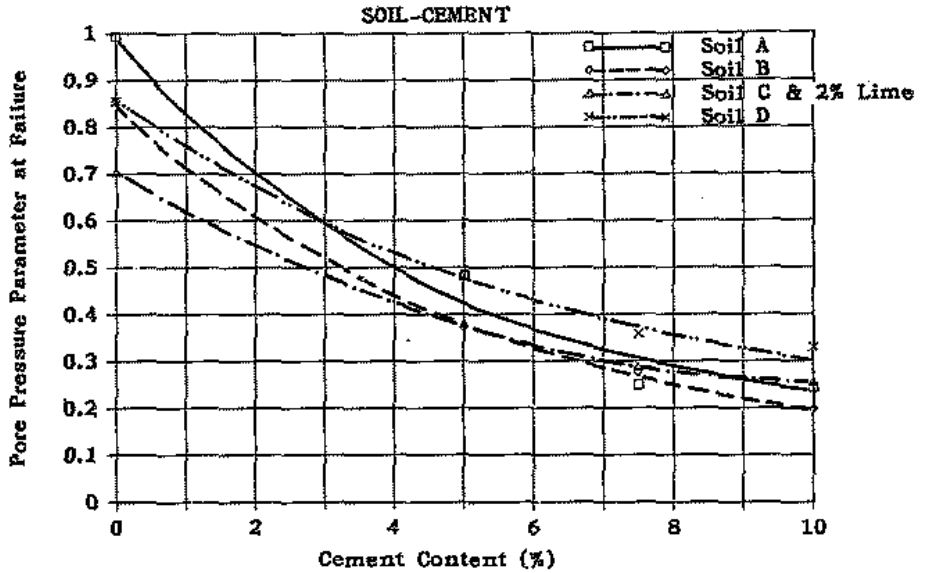


Figure 5.11: Typical Plots of Pore Pressure Parameter vs. Strain



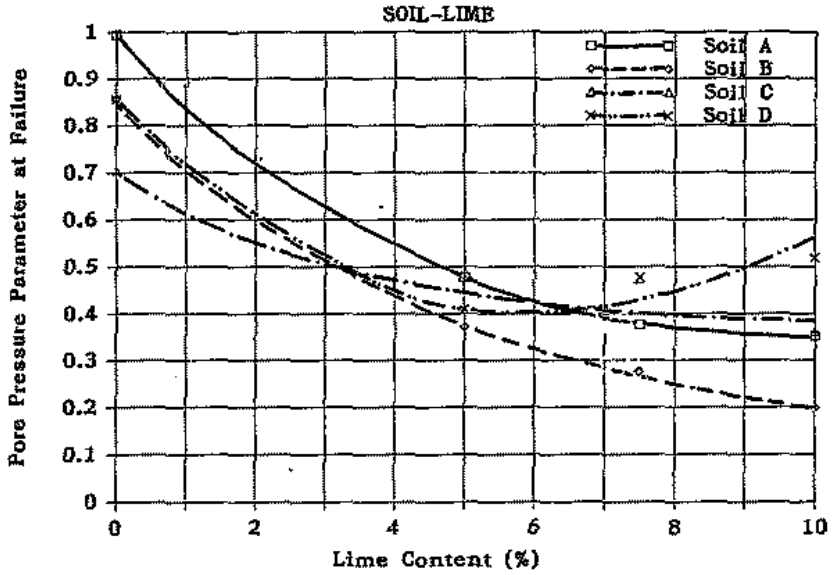
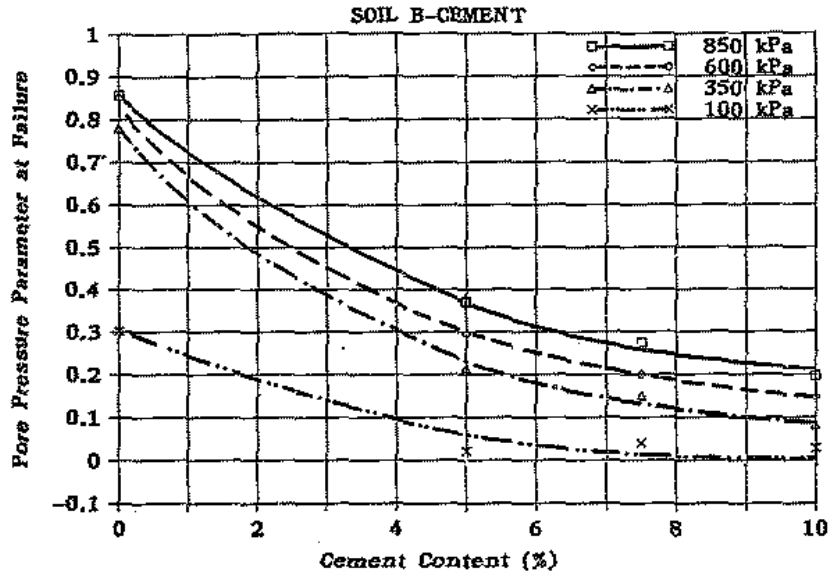


Figure 5.12: Pore Pressure Parameter at Failure vs. Stabilizer Content (Confining Pressure = 850 kPa)



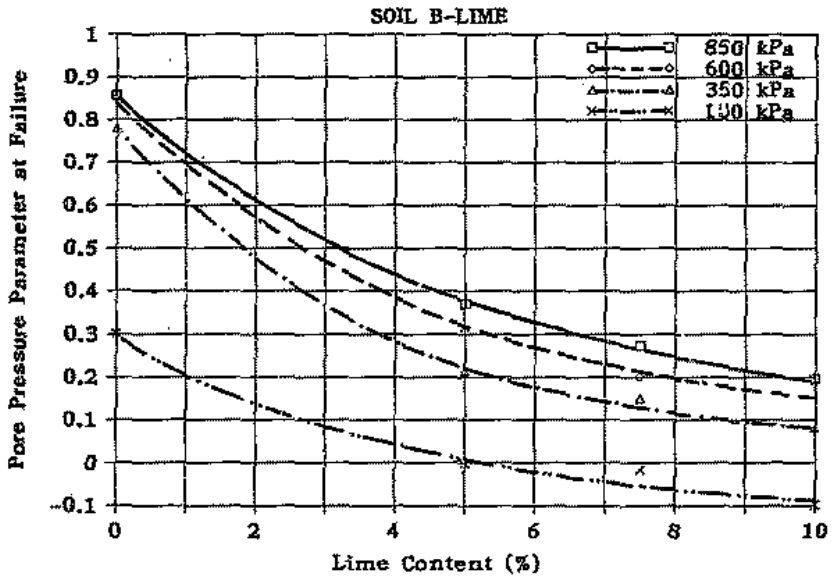


Figure 5.13: Pore Pressure Parameter at Failure vs. *Stabilizer Content Showing* Effect of Confining Pressure - Soil B

stage occurred for higher lime content specimens consolidated to high pressures. Figure 5.14 shows the void ratio versus confining pressure (e-log(P)) curves for Soil C at different cement contents, while Figure 5.15 shows the e-log(P) curves for Soil C at different lime contents. The curves were obtained from the three-dimensional consolidation of triaxial specimens, each data point representing the consolidation of a specimen at a constant confining pressure. The results indicate that the stabilization of a soil with either cement or lime reduces the volume changes occurring during consolidation significantly.

Insufficient results were gathered to be able to distinguish whether the slight difference in the trend of the e-log(P) curves for 7.5% and 10% lime between 600 kPa and 850 kPa resulted from experimental error, specimen variability or different behaviour to cement stabilized or low lime content specimens. However in all the high lime content specimens, there appeared to be a sudden change in slope of the e-log(P) curve between 600 kPa and 850 kPa.

A similar effect was reported by Fossberg [58] who noted that the soil-lime structure appeared to be unstable above certain consolidation pressures. Yong and Warkentin [57] attribute this 'instability' to the flocculation of the clay by the added lime. Consolidation of a clay soil that is not flocculated gradually reorients particles and pushes them closer together, yielding a fairly smooth e-log(P) curve (Figure 5.16 - curve (b)). On the other hand, the strong interparticle bonds in a flocculated clay resist reorientation until a sufficiently large load is applied to disrupt the bonds and break down the structure. Thus the initially flat e-log(P) curve (a) suddenly steepens after the preconsolidation load (c) is reached.

It appears that the degree of flocculation achieved when stabilizing Soil C with 5% lime is not sufficient to cause this instability.

This raises a question as to the accuracy of the fitted Mohr failure envelopes which sometimes indicated that a discontinuous failure envelope might be more applicable to a material with an unstable structure.

Figure 5.17 shows a hypothetical Mohr failure envelope (curve A) which becomes discontinuous at a state of stress sufficient to cause a breakdown of the unstable structure and a loss of cohesive strength. As curing proceeds (curve B), the cohesive strength increases as cementitious products are formed.

The identification of an unstable structure is important in the design of an embankment dam, since a sudden collapse of the material structure in the lower portions of the dam is certain to result in cracking of the upper material.

These observations point to the need for firstly determining whether a breakdown

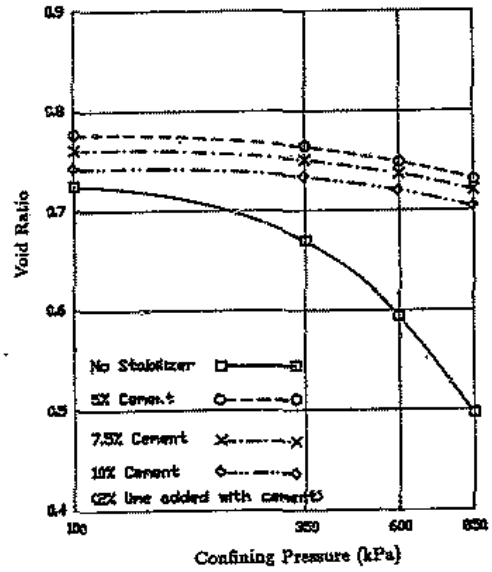


Figure 5.14: Void Ratio vs. log(Confining Pressure) - Soil C-cement (and 2% Lime)

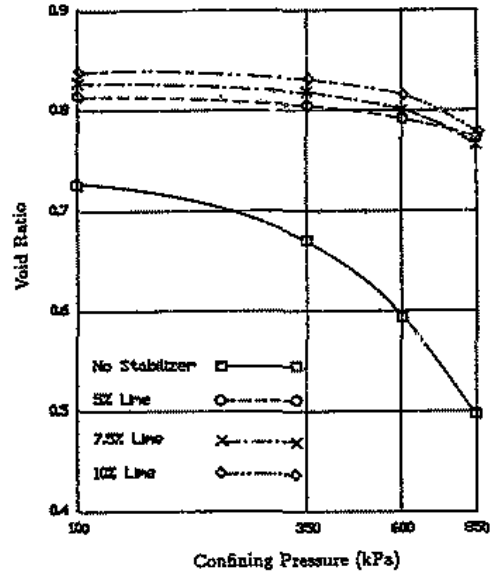


Figure 5.15: Void Ratio vs. log(Confining Pressure) - Soil C-lime

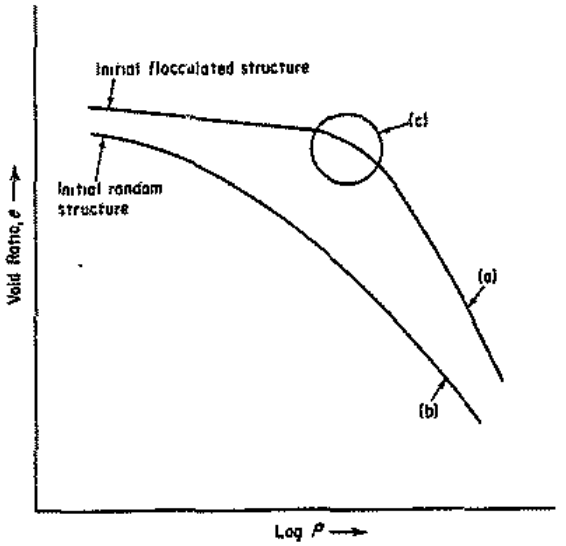


Figure 5.16: Change in Soil Structure Under Load (after Yong and Warkentin [57])

in the structure will occur during the consolidation phase and approximately at what consolidation pressure the onset of this instability occurs. *Fossberg* [58] found that the consolidation pressure at which the instability occurs is time dependent, i.e. the longer the material is cured prior to consolidation, the higher the stress required to break down the material structure. More information would also be provided by examining a larger number of specimens (6 or 8) sheared at smaller incremental confining pressures.

The peak shear stress versus stabilizer content curves were presented in the previous section. Figures 5.18 and 5.19 summarize (for all the soils) the strains at peak shear stress for cement and lime respectively, while Figures 5.20 and 5.21 summarize the variations of the secant modulus at one third peak shear stress[1] with stabilizer content.

The results reveal the following trends:

- The first increment of lime or cement results in an increase in the peak shear stress. In most cases, the peak shear stress continues to increase with increasing lime and cement content. The rate of increase is largest for soil-

[1]The majority of the stabilized specimens appeared to behave elastically up to this point

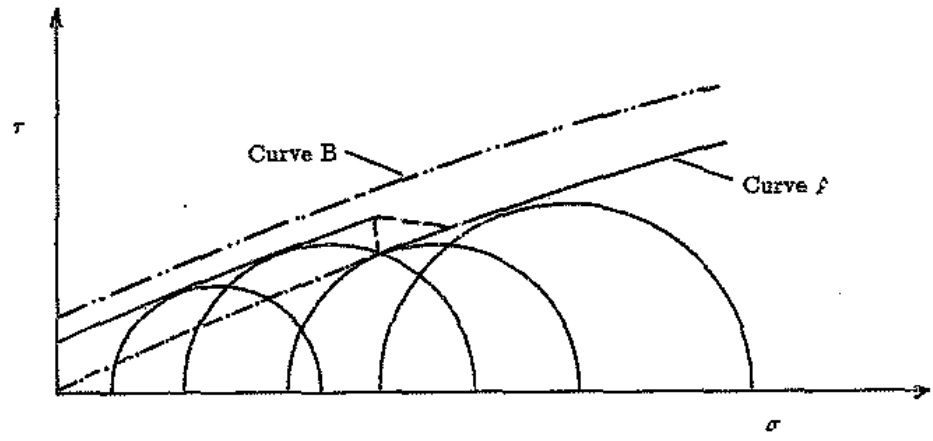


Figure 5.17: Hypothetical Mohr Failure Envelope for a Material with an Unstable Structure

cement specimens containing less clay, and with cement contents above 5%. In some cases, the shear strength decreases slightly at high lime contents.

- The peak shear stress generally increases with increasing confining pressure. It appears that in some cases, breakdown of the cemented or flocculated structure results in similar or lower peak shear stress at consecutive confining pressures.
- The secant modulus (to one-third peak shear stress) increases with the first increment (5%) of lime or cement. Thereafter, the modulus continues to increase with increasing cement content apparently at an increasing rate. The moduli for soil-lime specimens increase less rapidly at lime contents above 5% with Soils C and D showing a slight decrease in elastic modulus at higher lime contents (above 7.5%).
- The secant modulus also increases with increasing confining pressure.
- For soils with high strains to peak shear stress when unstabilized, the first increment of lime or cement reduced the strain to failure dramatically, e.g. Soil C with 5% stabilizer. The strain to peak shear stress remains approximately constant with increasing cement content, but increases significantly with increasing lime content.

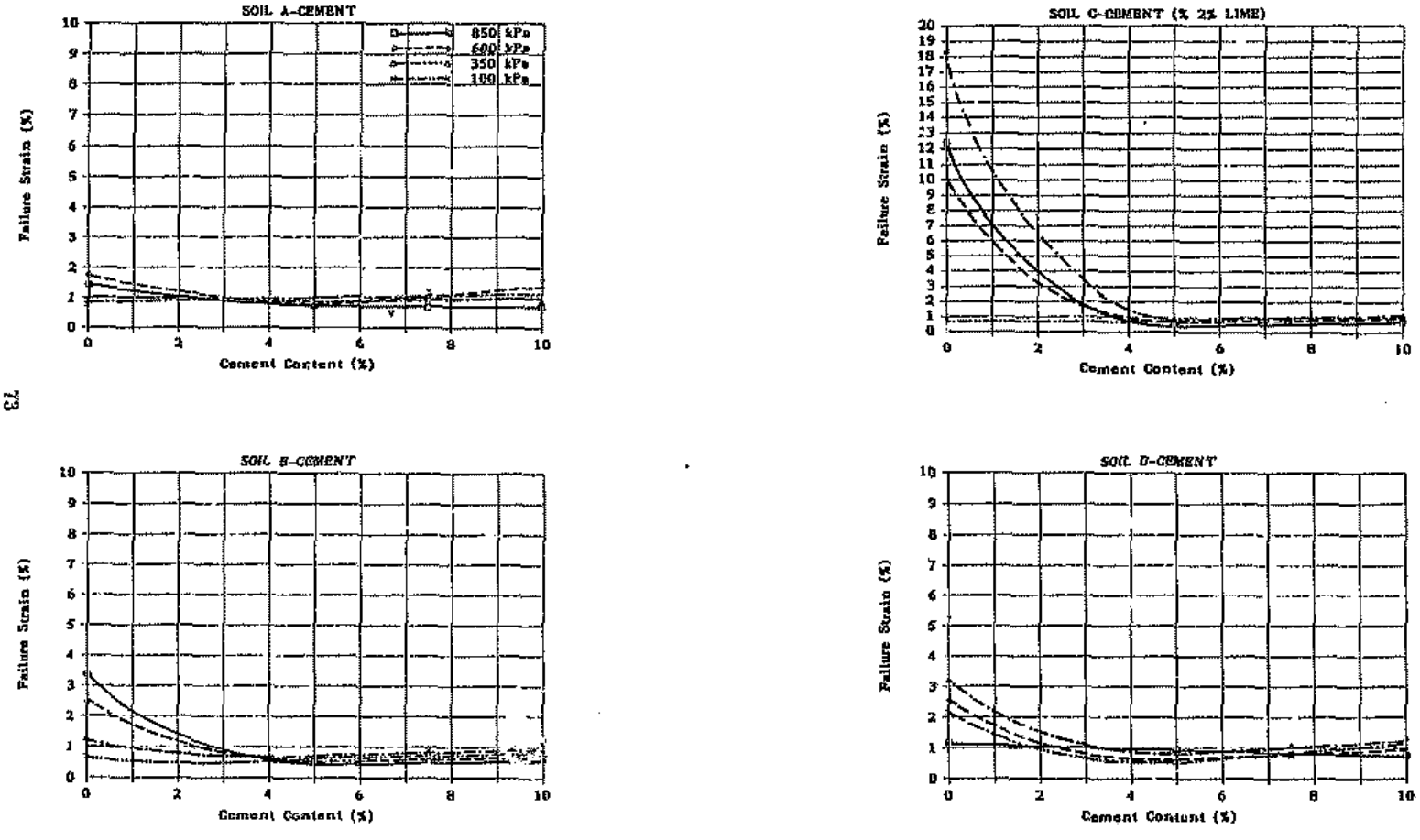


Figure 5.18: Strain at Peak Shear Stress vs. Cement Content

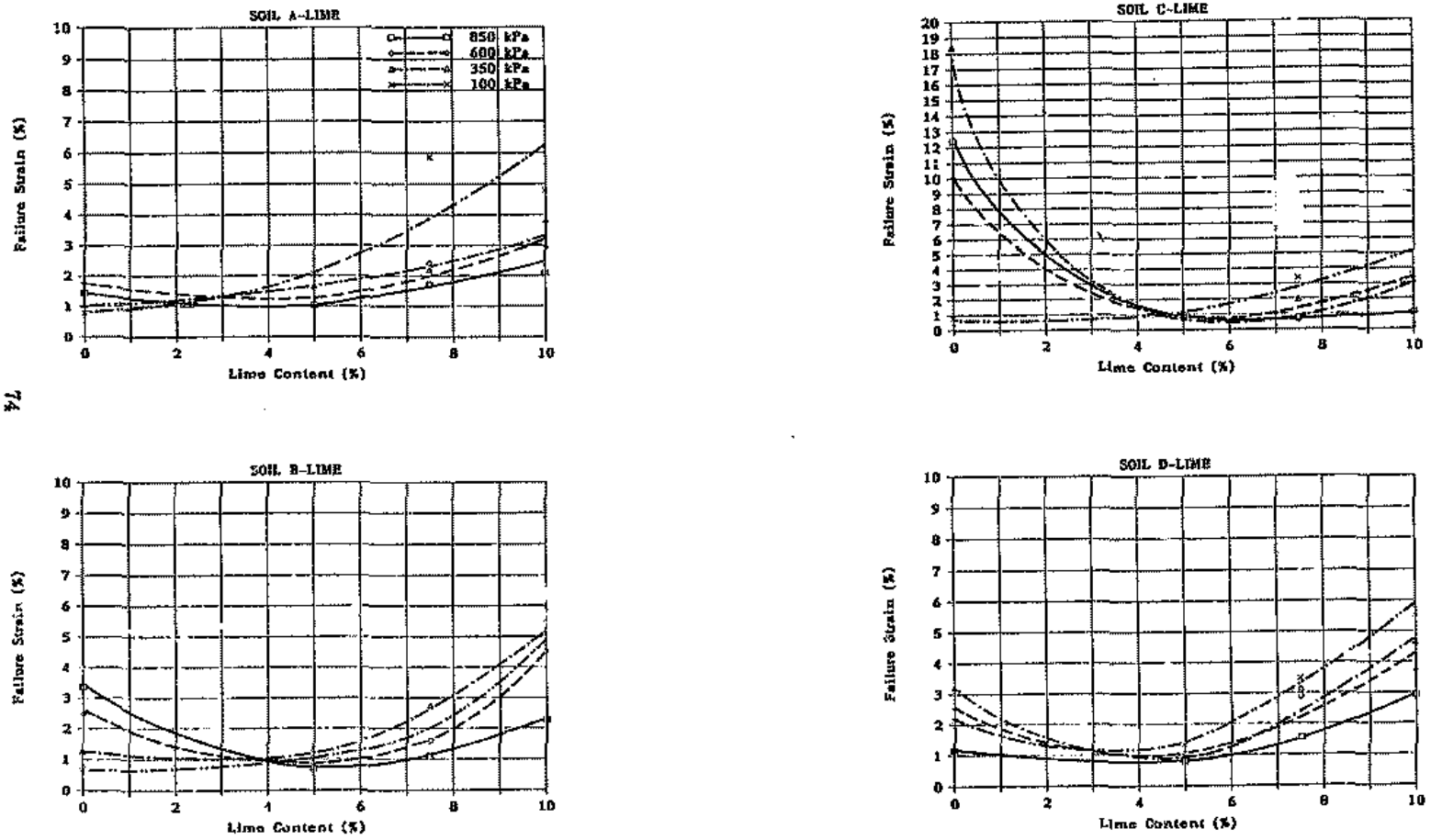


Figure 5.19: Strain at Peak Shear Stress vs. Lime Content

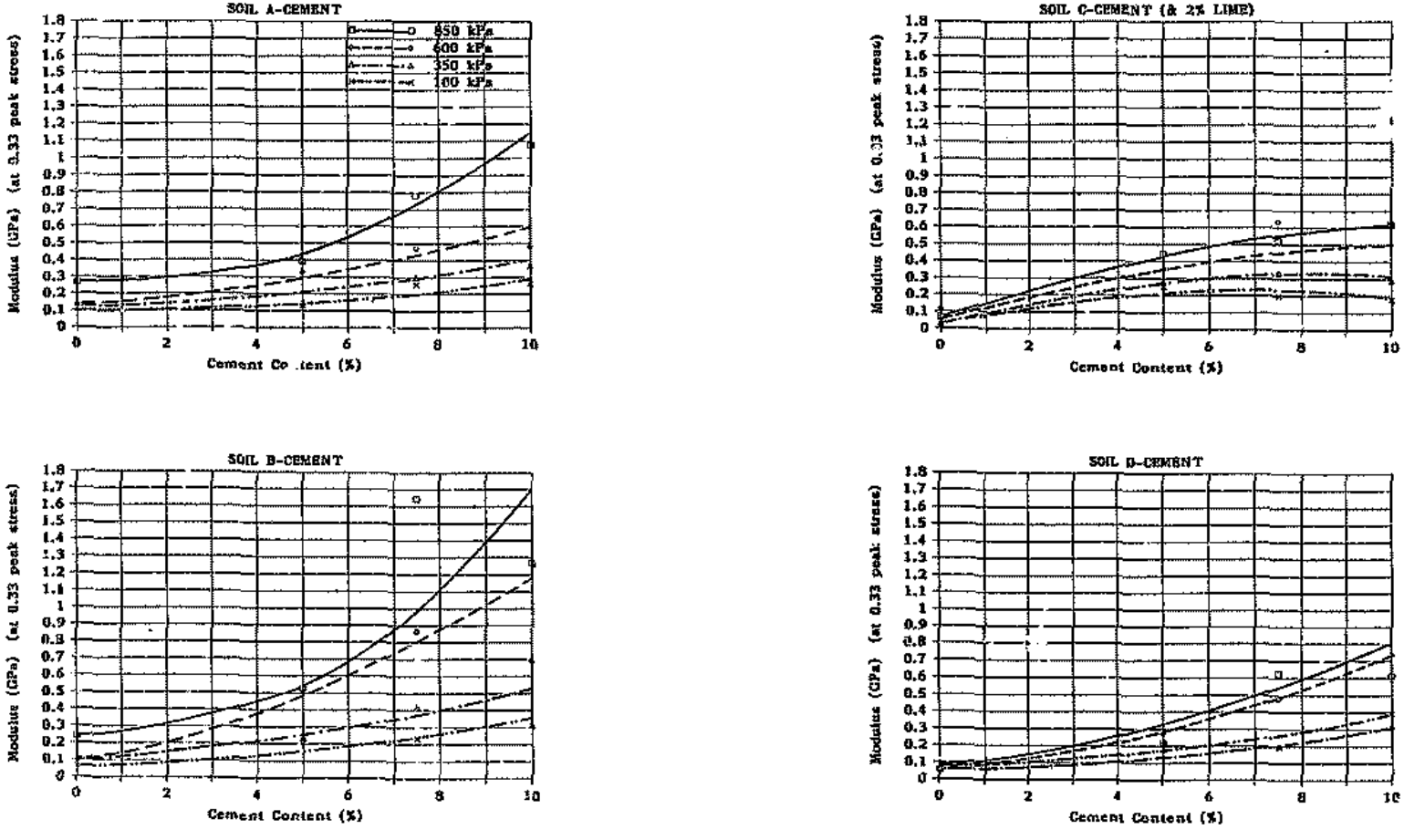


Figure 5.20: Initial Secant Modulus vs. Cement Content

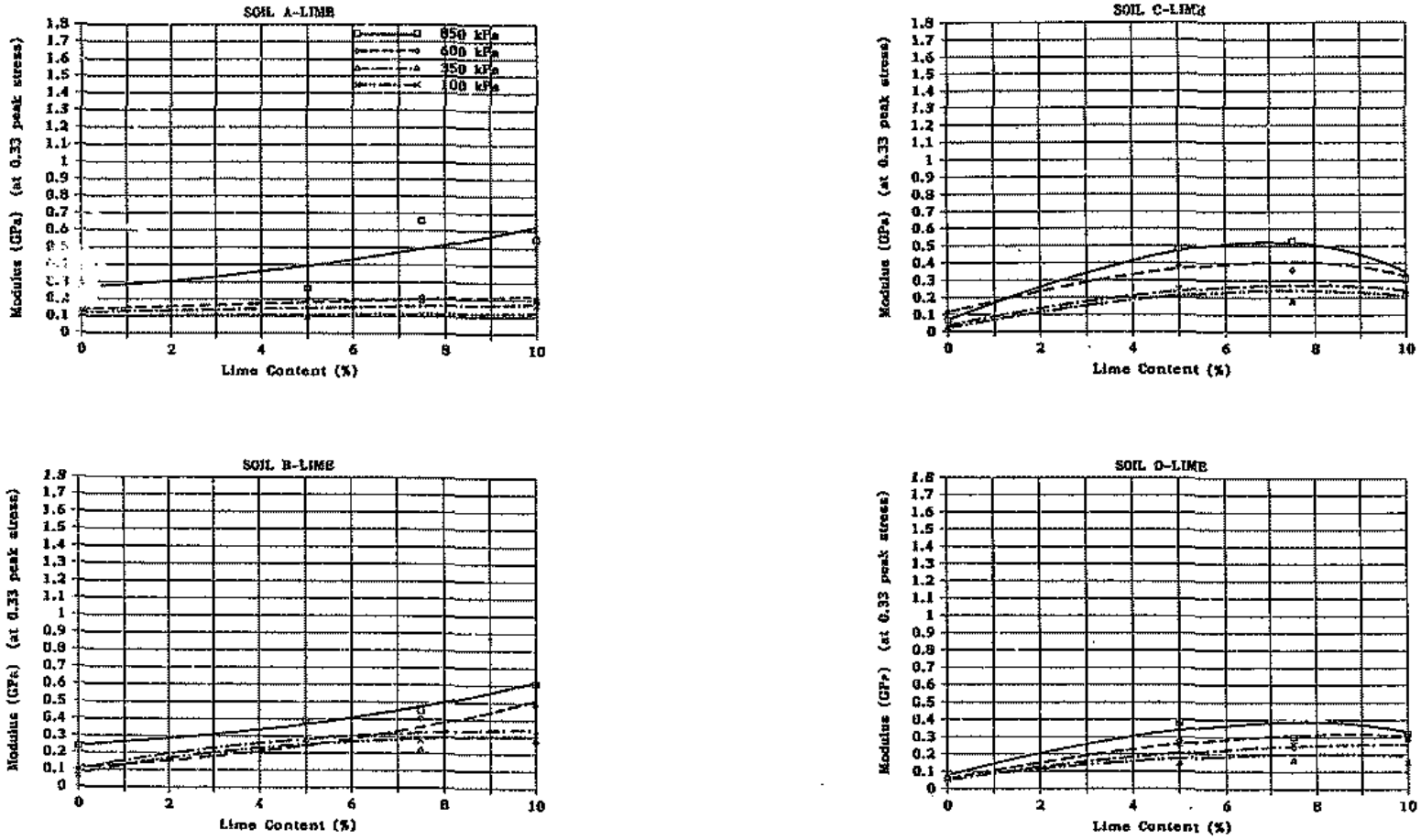


Figure 5.21: Initial Secant Modulus vs. Lime Content

- The strain to peak shear stress decreases with increasing confining pressure for both soil-cement and soil-lime.

The above observations differ from Robertson's results in that he found that at a particular cement content, the strain to peak shear stress generally increased with increasing confining pressure which is generally also the case with unstabilized soils. However, the soil that Robertson used was far more suitable for cement stabilization, yielding peak shear stresses 4 - 6 times those of the four soils used in this project at similar cement contents. These high strengths resulted from increases in the cohesion intercept, the angle of shearing resistance at 10% cement being approximately 37°.

It is thought that the effect observed in this project results from some breakdown in the cemented or flocculated structure whereby some cohesion is also lost. Progressive densification i.e. re-arrangement of the soil structure, as the consolidating pressure is increased might then modify the stress-strain behaviour from plastic to brittle. In such a case, the more dense material would reach peak shear stress at a strain less than that of the less dense material.

In comparison, such densification might not occur with Robertson's soil, since this material is capable of developing significant cohesive strength. Similar results to Robertson's might be obtained for the soils used in this project at cement contents higher than were used.

A notable point is the increase in strain to peak shear stress with increasing lime content. The implication of these results is that soil-lime might lend itself very well to embankment dam construction since it will be flexible while foundation settlement is occurring, but will gradually develop the strength required to sustain service loads.

5.5 Flexural Strength

The flexural strength of the stabilized soils was determined by conducting flexure tests in accordance with ASTM D 1635 - 63 - Standard test method for determining the flexural strength of soil-cement using simple beam with third-point loading. Figure 5.22 shows the loading configuration used.

The beam specimens were moulded by compacting the stabilized soil into a rectangular mould, using a square drop hammer. The beams were then moist cured for 28 days prior to testing.

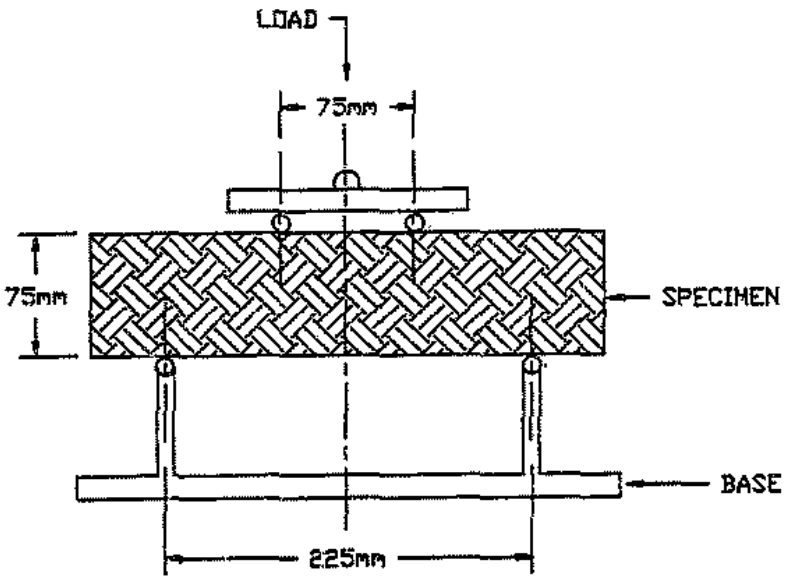


Figure 5.22: Loading Configuration for Flexure Tests

Since it was recognized that the strain at which a crack would begin to develop was also important to the consideration of shrinkage, strain gauges were glued to the bottom of the specimens so that a stress-strain curve could be obtained. The readings of load and strain were both electronically recorded by means of an XY pen recorder.

Figure 5.23 shows a typical set of stress versus strain curves (Soil B stabilized with cement). The stress versus strain curves for the other soil-stabilizer combinations are included in Appendix C. Figures 5.24, 5.25 and 5.26 summarize the variations with stabilizer content of the modulus of rupture (R), the flexural modulus (bending stress/strain) at 0.33 R_{max} and the failure strain.

Although in all cases, the specimens failed within the length of the central 50 mm strain gauge, suggesting that fairly uniform stabilization had been achieved, it is felt that because the failure is generally sudden, any observation of strain above the 'yield' point is unreliable, i.e. the apparent plastic behaviour is misleading.

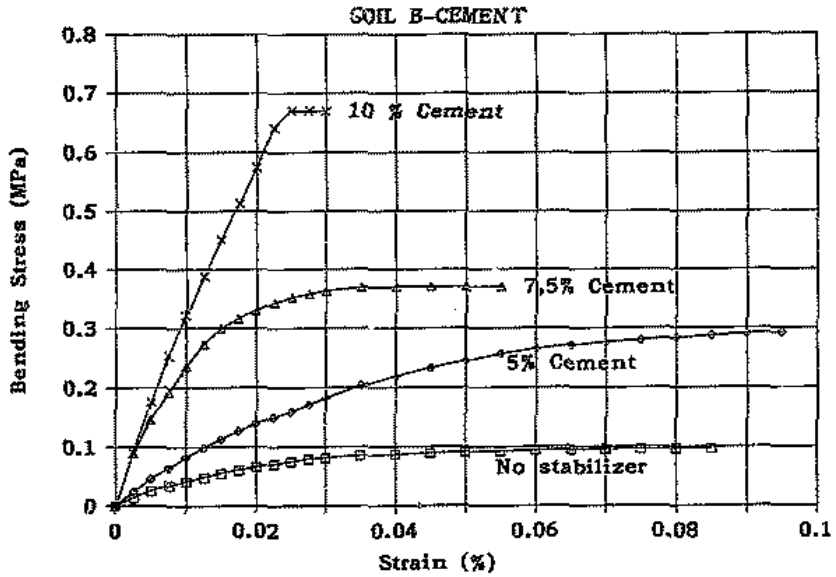


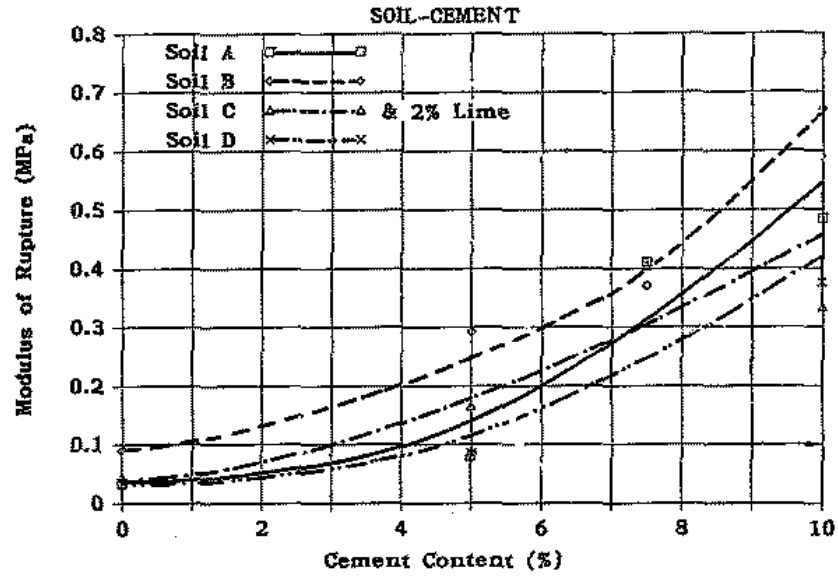
Figure 5.23: Bending Stress vs. Strain - Soil B-cement (28 days)

The failure strain was thus assumed to occur at the apparent yield point.

The results indicate that:

- Both the modulus of rupture and the flexu dulus increase with increasing lime and cement contents. The soil- specimens had higher moduli of rupture and flexural moduli than soil-lime specimens at the same stabilizer content.
- The failure strain generally appears to decrease with increasing stabilizer content.

The irregularity of the variation of failure strain with increasing stabilizer content is to be expected since these materials are not homogeneous in that they contain unstabilized clay lumps. These lumps, together with the air and water voids act as localized weaknesses from which cracks can propagate. Higher cement contents or longer curing periods in the case of the soil-lime might yield more regular results.



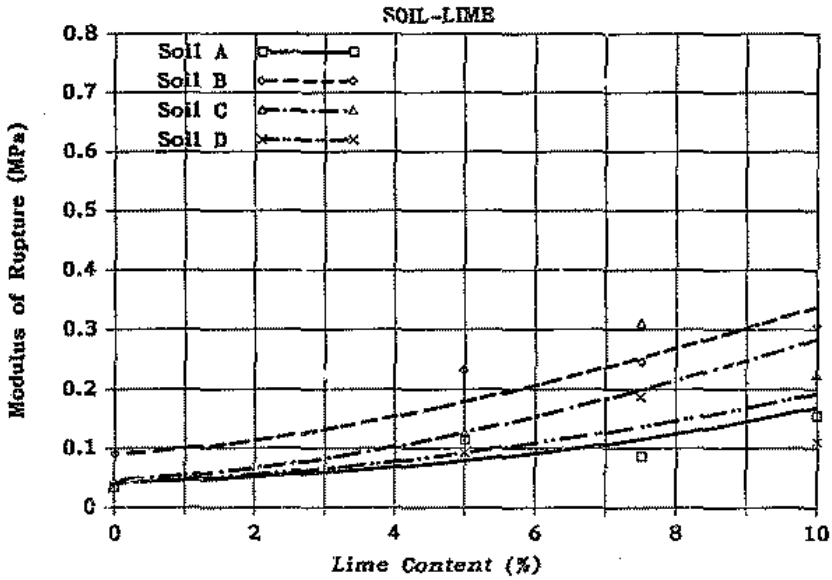
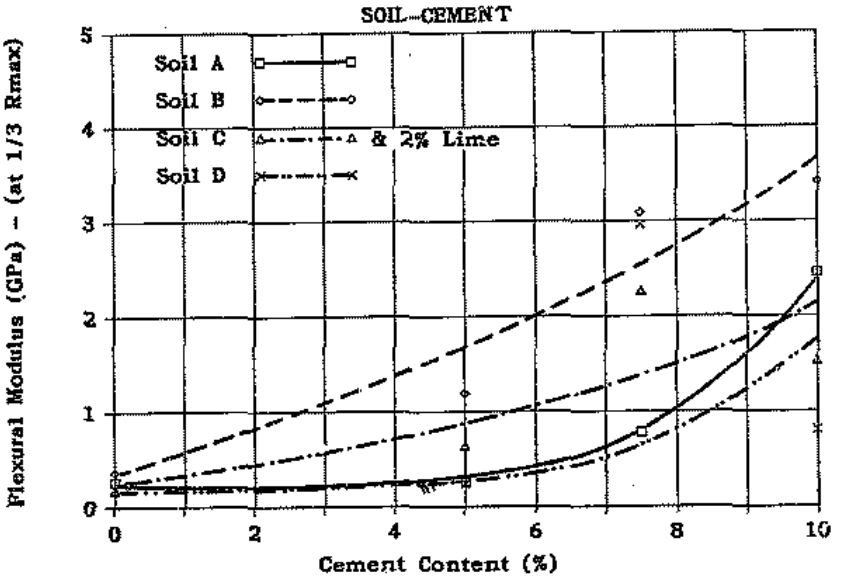


Figure 5.24: Modulus of Rupture vs. Stabilizer Content - 28 Day Moist Cure



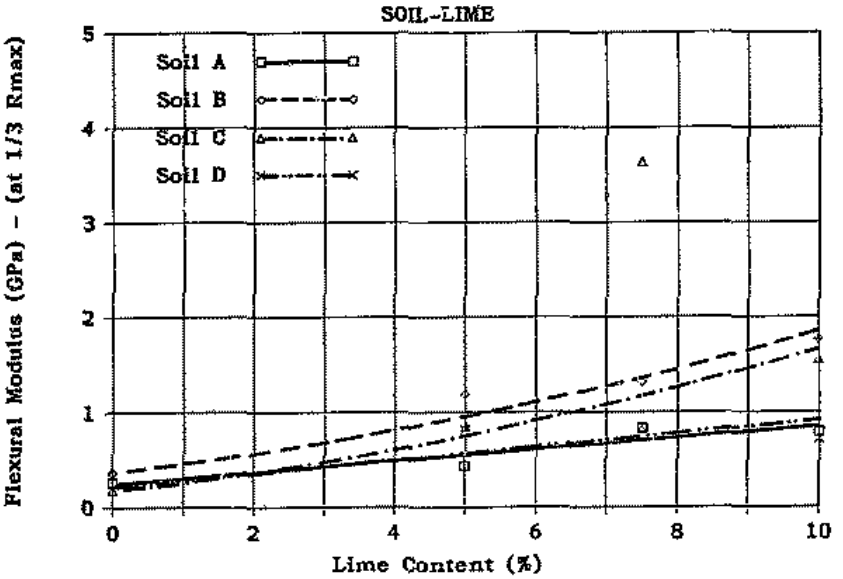
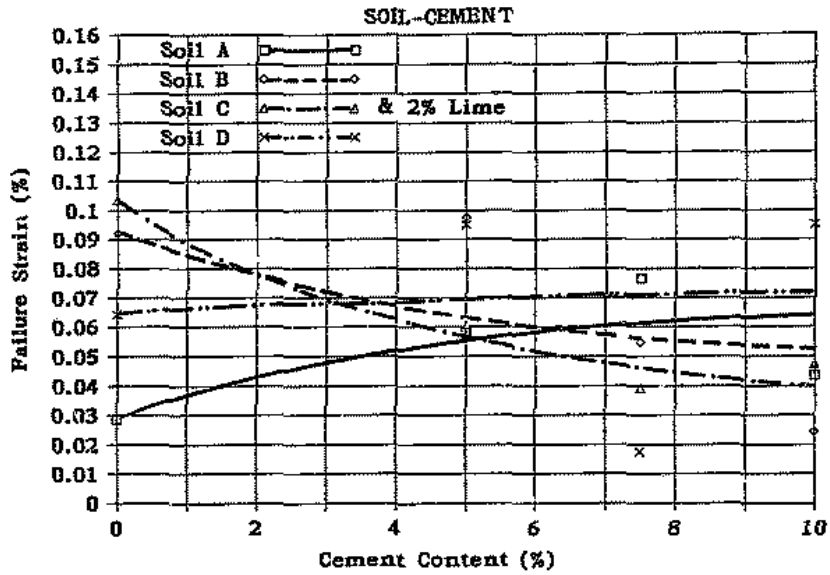


Figure 5.25: Flexural Modulus at 0.33 R_{max} vs. Stabilizer Content - 28 Day Moist Cure



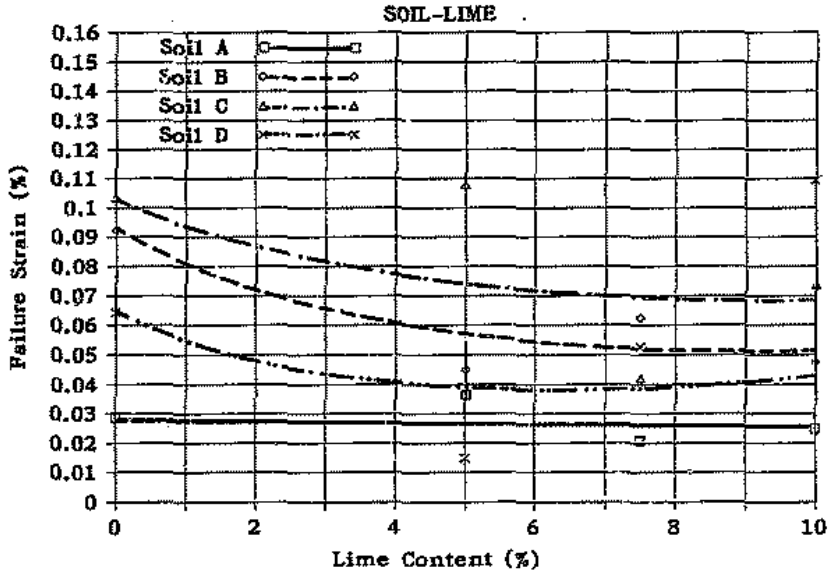


Figure 5.26: Failure Strain of Flexure Specimens vs. Stabilizer Content - 28 Day Moist Cure

Chapter 6

SHRINKAGE OF STABILIZED SOILS

6.1 Introduction

The major source of shrinkage in soils [60] (and concrete [41]) is drying shrinkage *where water from the voids evaporates into the atmosphere.* When the exposed surface of a stabilized soil layer begins to shrink, it is restrained from doing so by the material below it (which has not yet begun to lose moisture), thereby setting up differential tensile strains in the material. A surface crack will thus begin to form *when the tensile strain is sufficient to cause rupture.*

The development of cracks in dams is highly undesirable since these cracks can develop into leakage paths. Although it may not be economically feasible (nor possible) to attempt to *eliminate shrinkage cracking in stabilized soils* layers, through a knowledge of the shrinkage behaviour with time, the size and spacing of the cracks can be minimized, thereby minimizing the risk of the development of leakage paths.

In soil-cement, autogenous volume changes due to cement hydration can also lead to reductions in volume. The hydration of cement results in products which have less volume than the sum of the volumes of the original water and cement volumes taken separately [41]. Thus the total volume of the system *may decrease,* even though the absolute volume of solids is increased by the newly formed solid reaction products.

Autogenous shrinkage resulting from the internal consumption of water during hydration (self-desiccation) can also occur if additional water is not made available after mixing. However, the water cement ratio of the materials tested (approximately 2.75 at 10% cement) was far greater than the minimum water cement ratio required for the complete hydration of the cement (approximately 0.4). Thus autogenous shrinkage in these soils is only likely to contribute once the major proportion of drying shrinkage has occurred.

George [60] found that the shrinkage strain of a sand containing 4% clay increased from 0.05% (unstabilized) to 0.1% when 10% cement was added, this increase in shrinkage being attributable to the self-desiccation of the cement paste during hydration. Compared with the overall shrinkage strains of 2.5% for the soil-cement specimens tested in this project (see Section 6.2.2), it would not be possible to detect an autogenous volume change of 0.05%.

Both cement and lime stabilization reduce the shrinkage potential of a shrinkable soil. The reduction in shrinkage is attributed [25] to the reduced 'water affinity' of the calcium saturated clay and to the formation of a cementitious matrix which resists volume changes.

6.2 Laboratory Shrinkage Tests

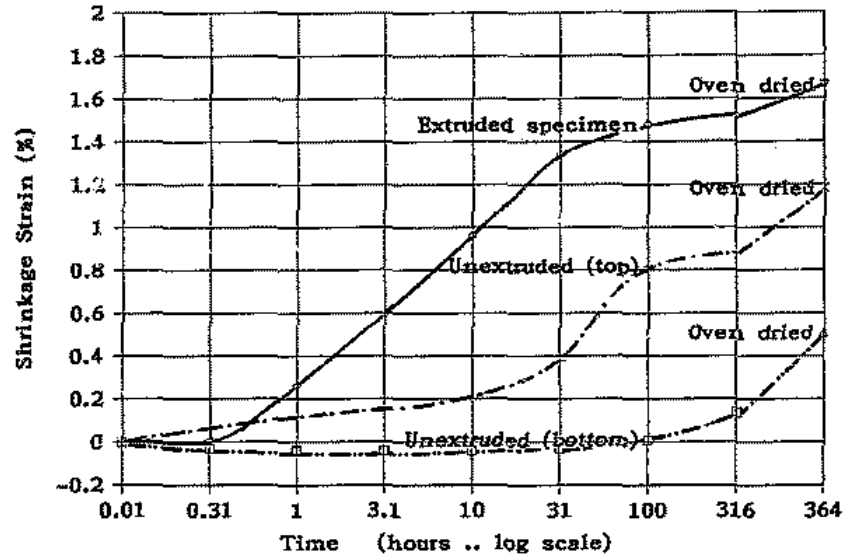
6.2.1 Preliminary tests and development of equipment

It was felt that the standard laboratory tests generally used for shrinkage testing were not adequate to provide meaningful data for the needs of this project.

On the one hand the shrinkage limit test, although providing valuable comparative data i.e. as an indicator test, would not model field conditions in any way since the material is uncompacted and oven dried. On the other hand the use of beams or cylinders extruded from moulds was also felt inadequate since the extrusion of the specimen from the mould allows it to expand to an equilibrium volume, therefore increasing the potential for shrinkage. Extrusion also allows drying from all sides of the specimen which rarely occurs in stabilized soil layers.

Figure 6.1 shows the result of a pilot test conducted to determine the effects of extrusion on the measured shrinkage, using a silty sand stabilized with 10% cement. The soil contained approximately 5% clay sized material.

The 'extruded' sample was compacted to Standard Proctor density, after which



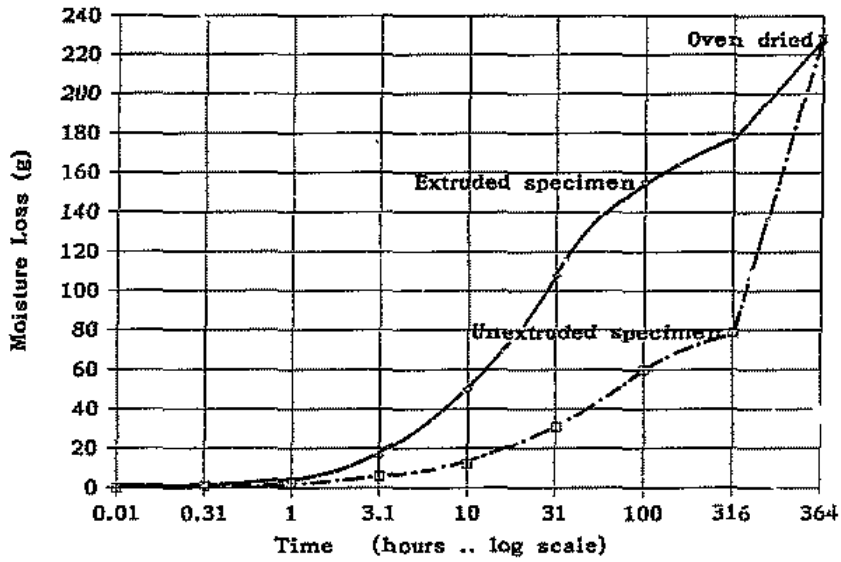


Figure 6.1: Pilot Shrinkage Test Results - Shrinkage Strain vs. Time and Moisture Loss vs. Time (Standard Proctor Specimens; 25^{0}C and 37% relative humidity).

it was extruded from the mould. Shrinkage strains were measured between two targets placed 100 mm apart on the circular surface. The unextruded sample was left in the Proctor mould, with the base kept in position to seal the bottom surface. Strain measurements were also taken on the bottom surface of the unextruded specimen.

To determine the maximum shrinkage that could occur, the specimens were oven dried for 48 hours once the shrinkage on the exposed surface had reached the maximum value under the testing conditions i.e. after 316 hours.

Measurements taken before and after extrusion showed that an expansion of 0.3% occurred on the circular surface. The measurements of shrinkage on the sealed surface of the unextruded specimen indicate that the material first expanded (0.03%), after which some shrinkage occurred.

The results indicate that extrusion of the specimen results in higher values of measured shrinkage. The difference arises both from the expansion of the extruded specimen increasing the shrinkage potential (the expansion of 0.3% represents 18% of the shrinkage potential of this specimen), and because the rate of moisture loss is higher in the extruded specimen (see graph of moisture loss versus time, Figure 6.1).

The results for the exposed and sealed surfaces of the unextruded specimen indicate that the slower the rate of moisture loss, the more the cement hydrates providing a stronger cementing matrix to resist shrinkage strains. Thus although approximately the same total mass of moisture is lost from both specimens (approximately 228g after oven drying), the exposed surface of the unextruded specimen shrinks less than the exposed surface of the extruded specimen, while the sealed surface shrinks the least.

Based on these results, and on the philosophy that a knowledge of, and the ability to control the shrinkage-time behaviour would be important to the minimizing of cracks in dams, a new testing procedure was developed.

A split-mould (mild steel) was used to 'secure' a 100 mm length of PVC tubing (150 mm outside diameter) while the stabilized soil was being compacted into the tube (Figure 6.2). After compaction, the specimen surface was trimmed flush with the top of the tube, and targets were glued across two diameters on either ends of the specimen using a rapid hardening, high strength glue.

The specimen was inverted from its compacted position so that the top surface appeared plane and uniformly densified. The bottom surface of the specimen was sealed using plastic sheeting secured in place with rubber bands. The sheeting

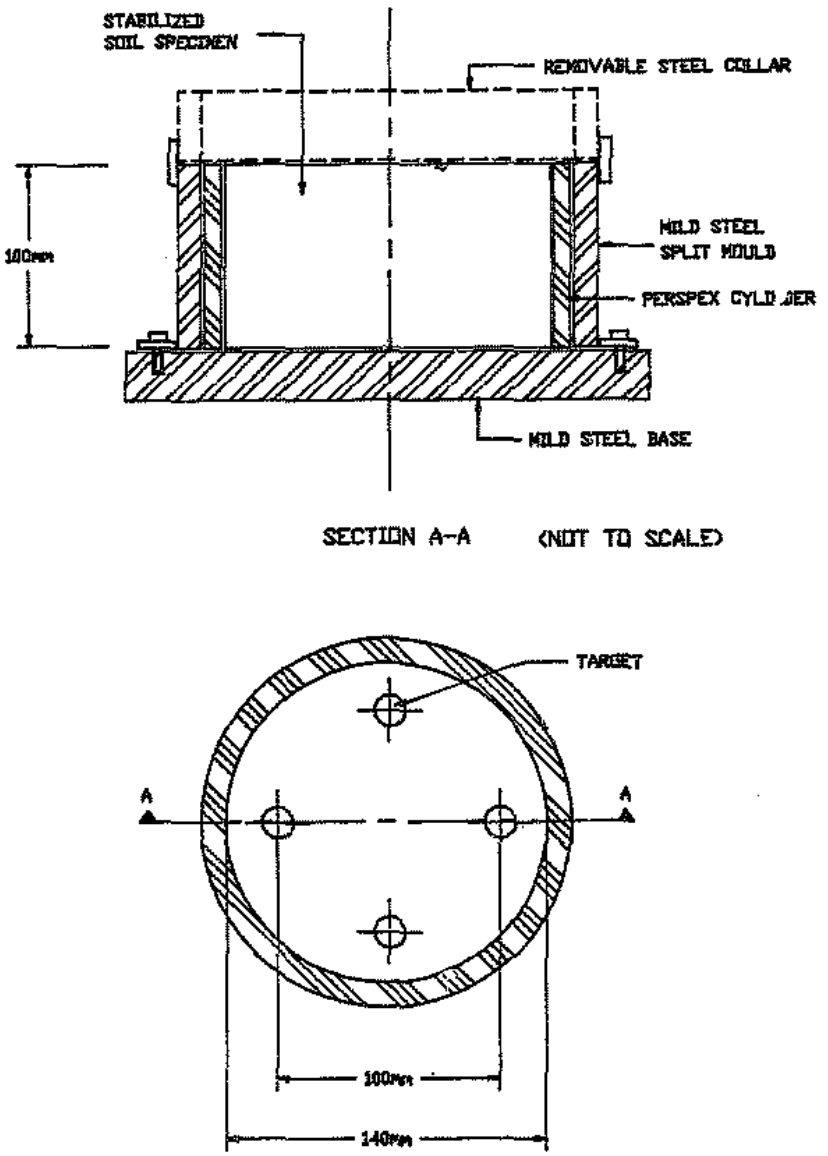


Figure 6.2: Apparatus for Compaction of Shrinkage Specimens

could be removed and replaced quickly to allow strain measurements.

A section of a laboratory was enclosed with plastic sheeting so that the specimens could be subjected to a constant temperature (25°C) and a constant humidity (55% relative humidity) during the tests.

The 'maximum' shrinkage measured after oven drying the pilot specimens at 110°C indicates that curing soil-cement specimens i.e. moist or sealed cure for say 28 days, would permanently reduce the shrinkage potential of the material. However since it is the period between the compaction of successive layers that any significant amount of shrinkage cracking on the horizontal exposed surface occurs (this is where cracks might develop into leakage paths through the body of the dam) it was decided to start the shrinkage observations as soon as the glue had set (approximately 15 minutes after compaction was completed).

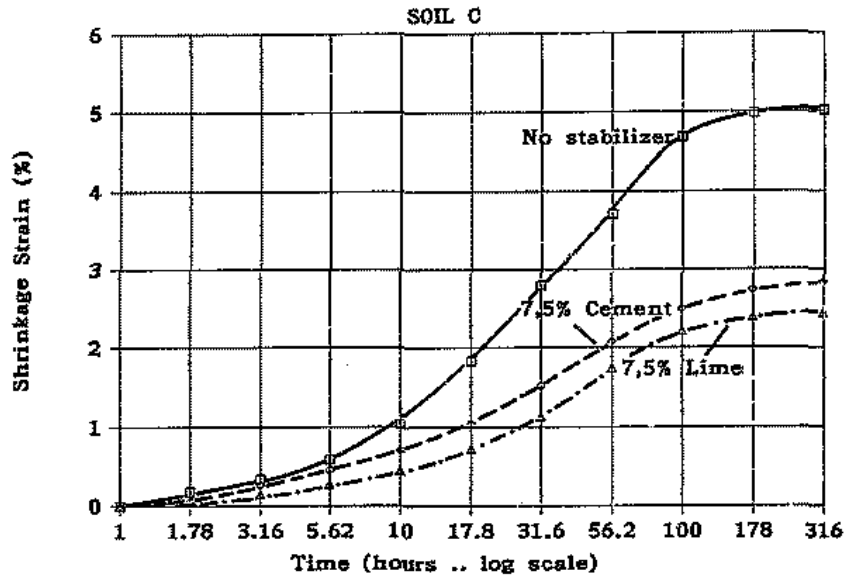
6.2.2 Shrinkage test results

The shrinkage versus time graphs for each soil at the various stabilizer contents are included in Appendix D. Figure 6.3 shows typical shrinkage time curves for the exposed surface for Soil C when unstabilized and when stabilized with 7.5% lime and cement. Figure 6.3 also shows the graph of moisture loss versus time for these specimens.

Figure 6.4 shows the trends of maximum shrinkage strain versus stabilizer content for the four soils. The shrinkage strains represent an average of the two strains measured on the exposed surface, while 'total' shrinkage strains were the maximum shrinkage magnitudes observed under the test conditions, i.e. the total shrinkage strains were not determined by oven drying as in the pilot tests.

The results were typical of all the soils and show that:

- The addition of the first increment of lime or cement reduces the shrinkage markedly. The shrinkage continues to decrease as the stabilizer content increases.
- Lime is more effective than cement for controlling shrinkage of these soils. This is especially noticeable in the clay soil (Soil C) where 10% cement and 2% lime reduced the shrinkage by 45%, while 10% lime reduced the shrinkage by 65%.
- Although the quantity of moisture lost in a given time is higher in the soil-lime specimens than in the soil-cement specimens at a particular stabilizer



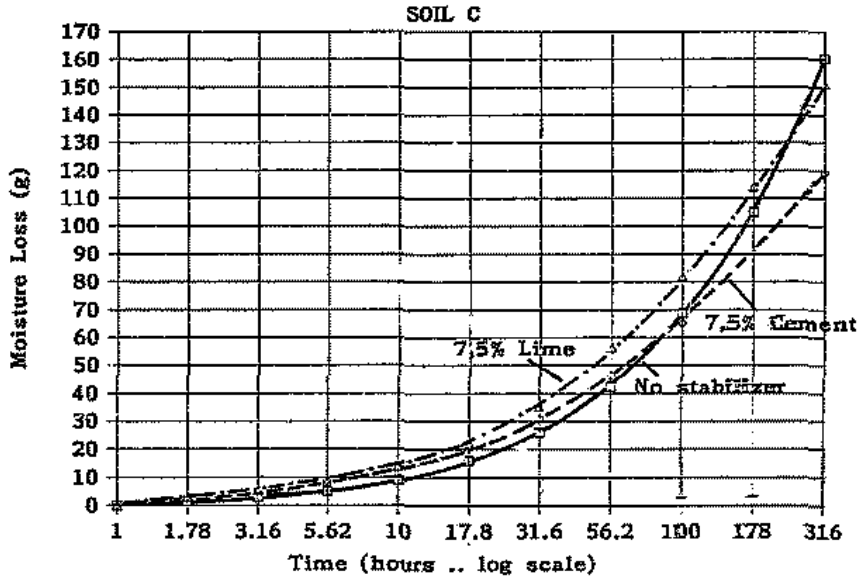
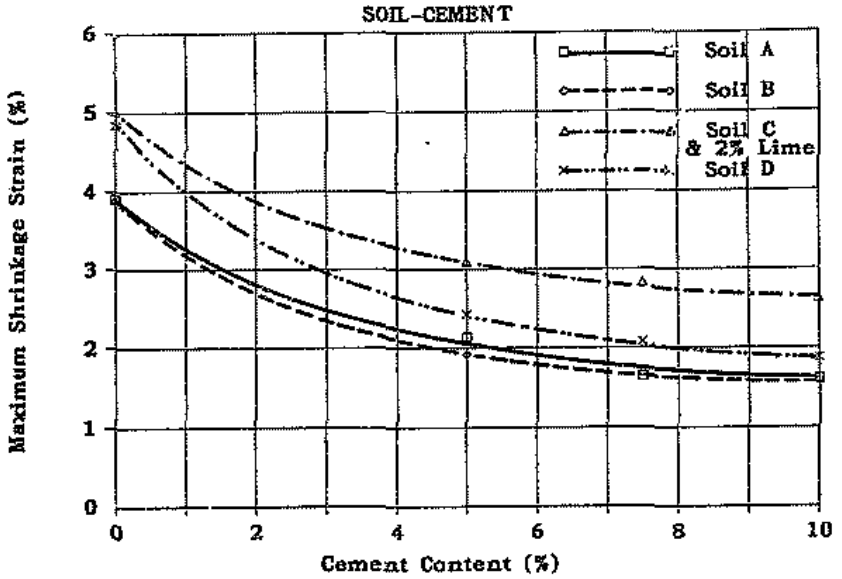


Figure 6.3: Shrinkage Strain vs. Time and Moisture Loss vs. Time - Soil C



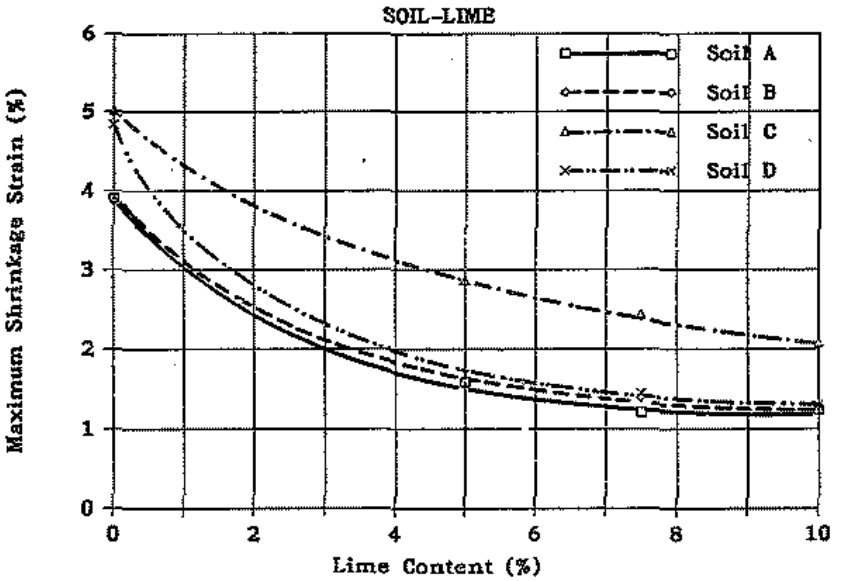


Figure 6.4: Maximum Shrinkage Strain vs. Stabilizer Content

content, the shrinkage of the soil-lime specimens is always less. Although only one example is given in Figure 6.4, this was always the case.

Figure 6.5 shows the graph of rate of moisture loss versus time for Soil C when *unstabilized and at all stabilizer contents. The same linear trend of decreasing* rate of moisture loss with the logarithm of time occurred for all the soils at all stabilizer contents. The rate of moisture loss from Soil C when unstabilized initially remained constant and then followed the same trend as the stabilized materials.

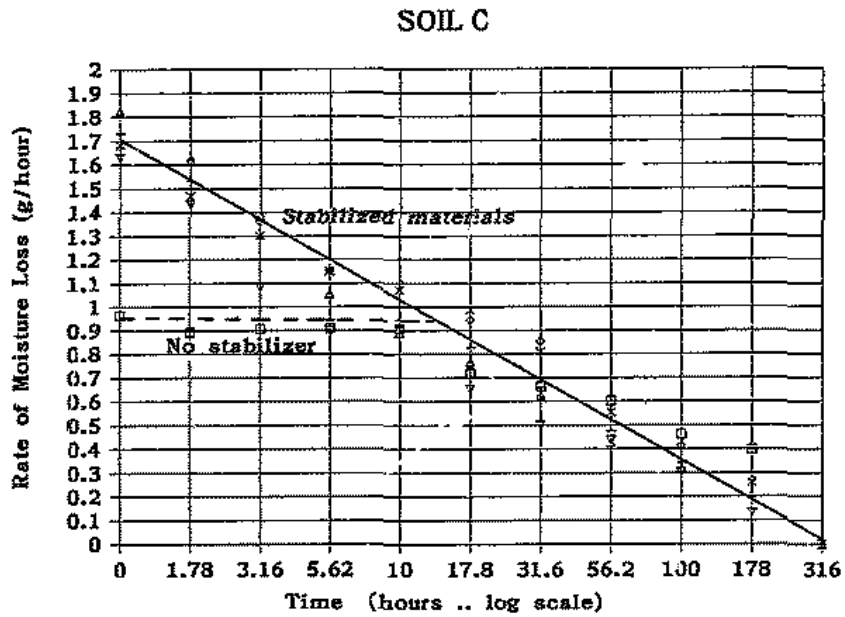


Figure 6.5: Rate of Moisture Loss vs. Time - Soil C

Although there was no definite relationship between the rate of moisture loss and the stabilizer content, it was noticed that the initial rate of moisture loss appears to be dependent on the permeability of the material, i.e. the lower the permeability, the lower the initial rate of moisture loss. This is however of academic value since the rate of shrinkage is dependent on the quantity and type

of clay present, i.e. Soil C unstabilized shrinks faster than when stabilized even though the rate of moisture loss is lower.

The initial rate of moisture loss is however important in determining the rate at which moisture needs to be replaced if shrinkage is to be minimized.

Figure 6.6 compares the total shrinkage on the top and bottom surfaces of the specimens of Soil B stabilized with cement and lime. The measurements of shrinkage on the bottom surfaces were begun once the shrinkage on the top surface had reached a constant value, at which stage the specimens were inverted and the previously sealed surfaces exposed. The final measurements were taken 21 days after the specimens had been compacted. Similar results were observed for the other soils.

The exposed surfaces of the soil-cement shrank more than initially sealed surfaces as occurred in the pilot tests. Although the specimens were not cured, it appears that since the rate of moisture loss from the lower part of the specimen is lower than the top, more cement hydration occurs in this region.

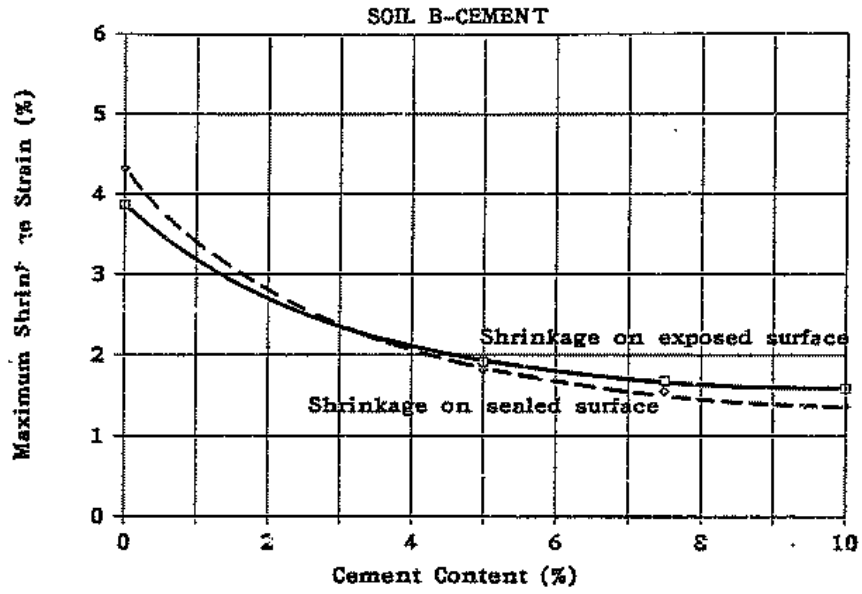
On the other hand, the sealed surfaces of the soil-lime specimens shrunk more than the exposed surfaces. This was not unexpected since it was noticed that for all the soils, the shrinkage of the unstabilized material was greater on the sealed surface.

It is thought that a density gradient is caused by the tamping of successive layers so that the bottom of the compacted specimen is more dense than the top. Since the specimens were inverted relative to their compacted positions, the exposed surface was more dense than the sealed surface and thus exhibited less shrinkage potential.

6.3 The Effect of Water Replacement on Shrinkage

If the amount of shrinkage can be controlled in the period between the placement of layers, then the amount of shrinkage cracking within the body of an embankment can be minimized. Since the shrinkage is largely caused by drying, it is logical that the shrinkage of an exposed surface can be reduced by replacing the lost water, until the surface is sealed by the subsequent layer.

Figure 6.7 shows the results of a water replacement test conducted on Soil B stabilized with 7.5% cement. Up to point B, the water lost through evaporation was replaced at 2 hourly intervals, by sprinkling water on the exposed surface. The



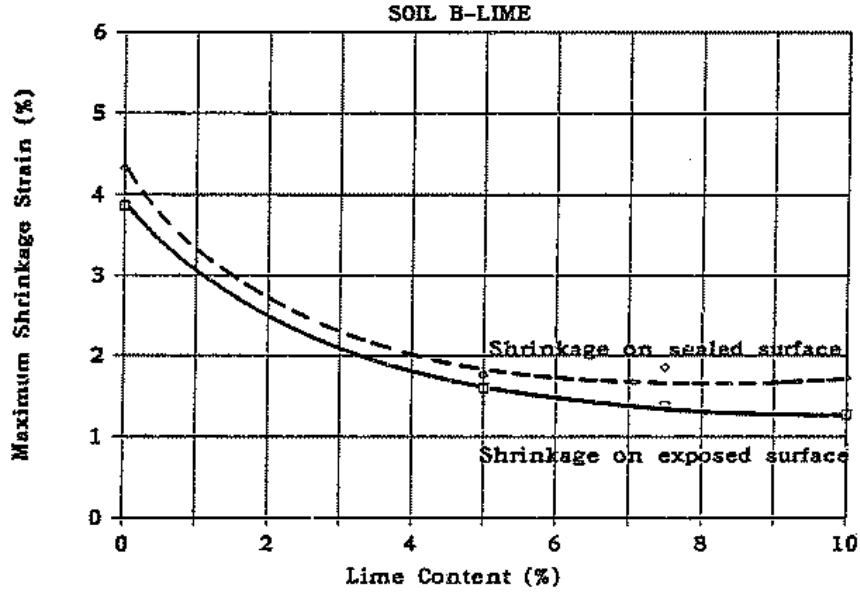


Figure 6.6: Comparison of Shrinkage Potential on Exposed and Initially Sealed Surfaces

specimen was then allowed to shrink overnight, after which the lost moisture was again replaced (point C). This was repeated (point D) after which the specimen was allowed to shrink with no further additions of water.

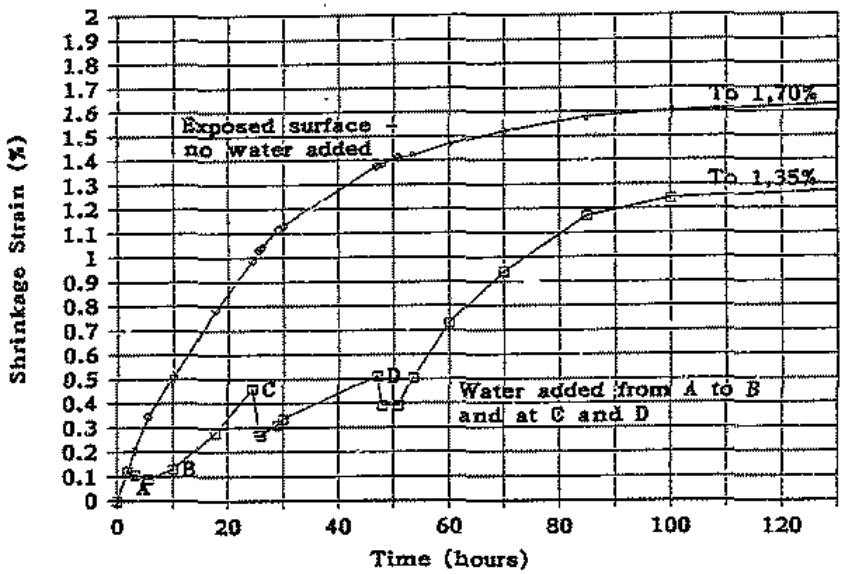


Figure 6.7: Effect of Moisture Replacement on Shrinkage-time Behaviour

The results show that not only can the shrinkage be successfully controlled, but there is also evidence that the shrinkage is partly recoverable, with the amount of recovery decreasing with time after compaction.

If the strain required to cause cracking is known, then it seems likely that the use of control measures such as sprinkling water on the surface at regular intervals would almost certainly allow good control over the development of shrinkage cracks.

From the flexure test results, it was observed that the strain at which a crack begins to devel may be as low as 0.02%. The shrinkage of all the specimens reached this strain very shortly after the readings began, indicating that it is

unlikely that shrinkage cracks can be entirely eliminated.

However, the following visual observations were made:

- 'Hairline' surface cracks were observed in the unstabilized and soil-cement specimens at very low shrinkage strains (less than 0.1%).
- These cracks grew as shrinkage continued, but were much smaller in the soil-cement than in the unstabilized specimens. The specimens also shrank away from the PVC tube.
- A few hairline cracks were observed in the 5% soil-lime specimens, but not in the 7.5% or 10% lime specimens. With continued shrinkage, these materials moved away from the PVC tubing.

It appears that the restraint offered by the material below the shrinking surface develops more rapidly in soil-cement than in soil-lime. Thus the soil-cement cracks, while the soil-lime appears to move radially inwards, i.e. material below the shrinking surface is pulled inwards by the shrinking material.

The NITRR [22] reports that "usually the cracks in lime-treated materials are narrower and less extensive and therefore less significant than those in cement-treated materials". Since the time rate of shrinkage is lower for soil-lime, it is apparent that it will be easier to control cracking of soil-lime than soil-cement.

To ensure adequate bonding at the interface between soil-cement layers, a fairly liquid cement paste may be spread on the exposed surface before construction of the subsequent layer begins [9]. This paste would certainly aid in filling any shrinkage cracks that may have developed.

Because of the slow rate of strength gain with time of soil-lime mixtures, some of these materials have been reported to exhibit 'self-healing' (or autogenous healing) properties [25,61,62]. Thus the possibility exists that if shrinkage cracks in soil-lime can be made to close up e.g. by water replacement, the material can regain some tensile strength. However this self-healing property is not well proven, and should not be relied upon [22].

Chapter 7

THE PERMEABILITY OF STABILIZED SOIL

7.1 Introduction

The permeability of the stabilized soil is an important consideration in the design of homogeneous stabilized soil embankment dams. It is desirable to minimize the permeability to reduce seepage losses through the embankment, while Robertson's [9] findings show that the distance into the material that calcium is leached from soil-cement in a given time is proportional to the permeability of the material.

The permeability of a soil is largely influenced by the volume of voids in the material and the continuity of flow channels. Shrinkage cracks can strongly influence the permeability by creating continuous flow channels. It is thus desirable to minimize the void volume, and to minimize shrinkage cracking to ensure and maintain a low permeability.

7.2 Permeability Test Results

The permeability tests were conducted using the laboratory apparatus designed by Robertson [9]. The percolation cell in which the specimen was placed is described in Figure 7.1. De-aired distilled water was used both to apply the confining pressure of 100 kPa, and for the percolating water which was at a pressure of 36 kPa at the inlet of the cell.

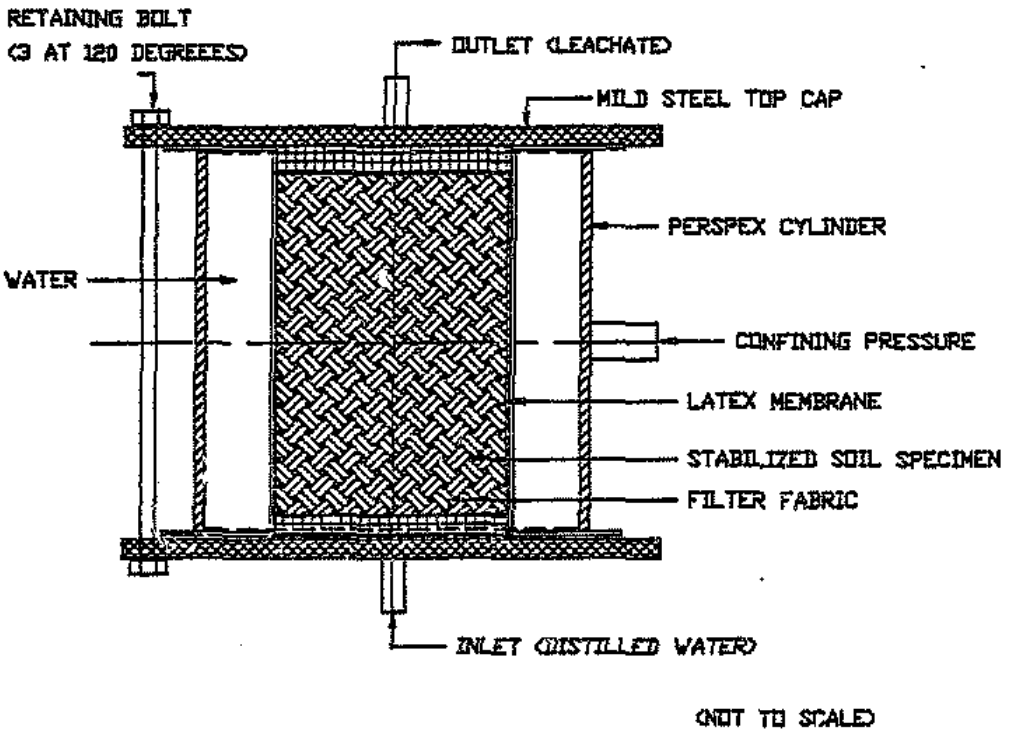


Figure 7.1: Permeability Apparatus

The gradient across the 75 mm long specimens was thus 50 m/m which may appear to be extreme for a homogenous embankment dam in which the gradient across the material is likely to be less than 1 m/m. However, the approach that Robertson took in the design of this test was to simulate the design life of the material using a higher gradient over a shorter test period. The test conditions for seepage over a period of one year are thus intended to represent a unit gradient (1 m/m) over a period of 50 years.

Three specimens of each soil-stabilizer combination formed a test 'set'. Once installed in the cells, the specimens were saturated firstly by flushing the specimen with the confining pressure off, and then by applying a vacuum at the outlet end for 30 minutes with the confining pressure on. This procedure was repeated until no air was visible in the outflowing water under vacuum.

During the tests, the water which had passed through the specimens was collected in plastic bottles. Moisture losses at the gap between the outlet piping and the bottle were effectively eliminated by sealing this gap with an impermeable tape. For each specimen, a continuous record was kept of the mass of water passing through the specimen in a given time, from which the permeabilities of the specimens were calculated at various intervals in time.

The tests were carried out over a period of 240 days following the standard 28 day cure at which stage, sufficient information on the permeabilities of the various sets had been obtained.

Figures 7.2 and 7.3 show the variations of permeability with stabilizer content. The figures have been presented separately for clarity.

In some cases, a single specimen within a set showed a significantly higher permeability than the average permeability for the set. This probably resulted from the development of shrinkage cracks during the setting-up of the cells. It was noticed that such high initial flows occurred in the majority of the 5% stabilizer specimens, which also showed the highest rate of shrinkage with time.

Figure 7.4 shows the variation in permeability with age for specimens showing high initial flows. The 'average permeability' represents the average permeability of the three specimens in the set after 240 days percolation. It is apparent from these curves that there is a tendency for the closure of leakage paths to occur, with the permeability decreasing towards the average value obtained for the specimen set.

The addition of either 5% cement or 5% lime resulted in increases in the permeability, this increase being larger for 5% lime. The increase in permeability is to

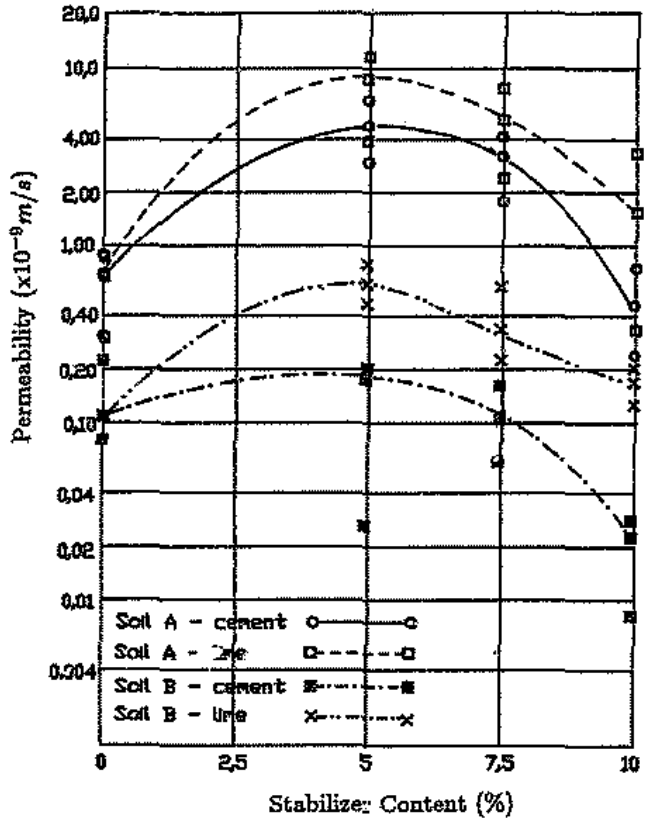


Figure 7.2: Permeability vs. Stabilizer Content - Soils A and B

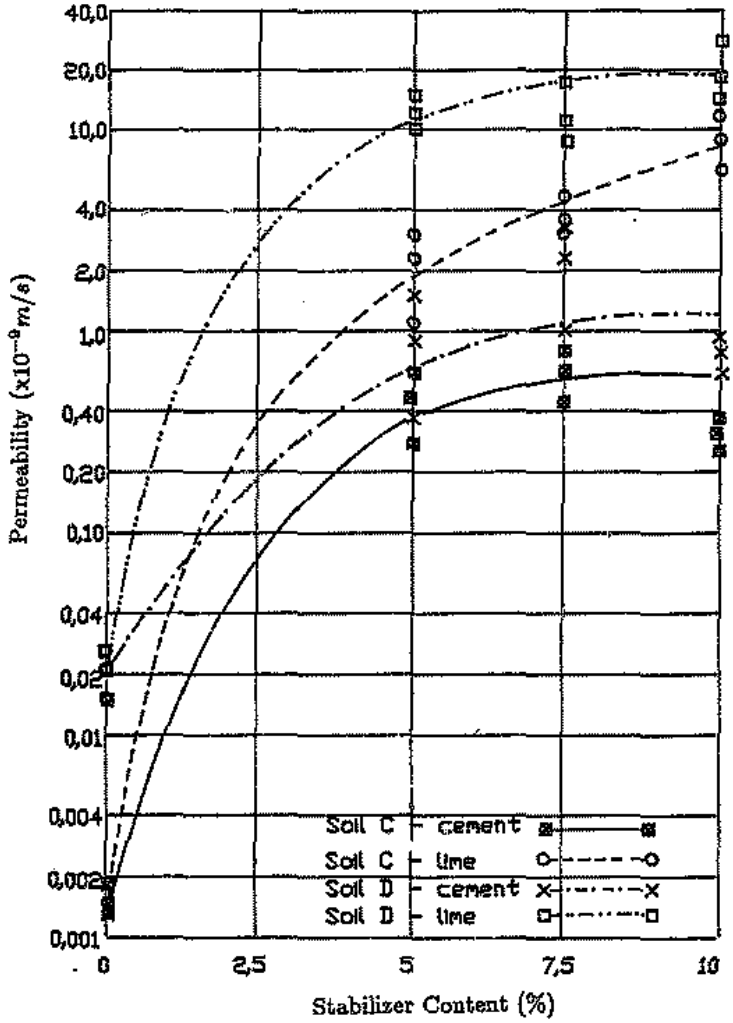


Figure 7.3: Permeability vs. Stabilizer Content - Soils C and D (Soil C-Cement has additional 2% Lime)

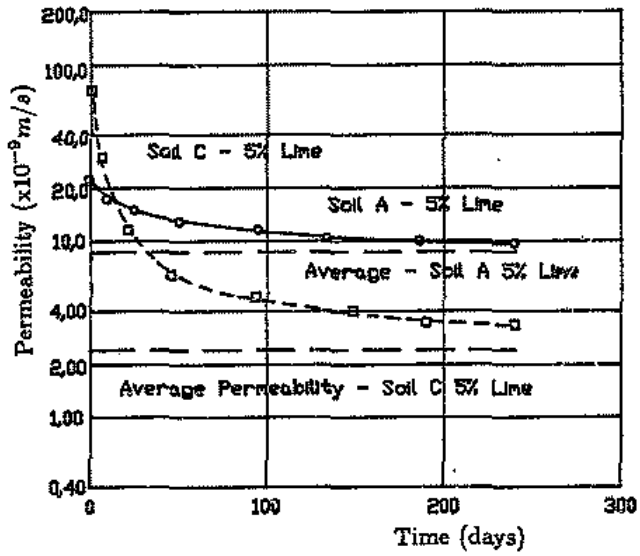


Figure 7.4: Permeability vs. Age for Specimens Showing Initially High Permeabilities

be expected since the flocculation of clay results in an 'open' less dense structure which results in a larger flow area and thus a higher permeability [28,57].

In the two less plastic soils, Soils A and B, further increases in either the cement or the lime content resulted in decreases in the permeability, with the cement having a larger influence than the lime at similar stabilizer contents. The permeabilities of both soils at a cement content of 10% were less than the permeabilities of the respective unstabilized materials.

The permeabilities of Soils C and D increased substantially on the addition of 5% cement or lime. Further increases in stabilizer content resulted in small increases in the permeabilities of these materials. The permeabilities at 10% cement were still far higher than the initially low permeabilities exhibited by these two soils when unstabilized.

As might be expected, the trend in permeability with stabilizer content resembles the inverse of the trend in maximum dry density with stabilizer content (Figures 4.4 and 4.5). Generally smaller increases in density occurred on the addition of 5% cement (reductions in density occurred in Soils A and C), while more significant increases in density occurred above the 5% cement level. This agrees with the initial increase in permeability at 5% cement and the reductions in permeability at higher cement contents.

Large increases in permeability can be associated with large decreases in maximum dry density resulting from the addition of 5% lime. The flocculation of the clay results in a less dense and hence more permeable structure.

Smaller decreases in density at lime contents above 5% correspond with smaller increases in permeability. At lime contents below the lime fixation capacity (approximately 5% for these soils), the added lime is used up in ion exchange and flocculation reactions. Since the degree of flocculation approaches a maximum at the lime fixation capacity, lime contents above this will result in free lime intended for the slower pozzolanic reactions. In a soft particulate form, this lime may act as a void filler, resulting in smaller increases in permeability and even decreases in permeability as observed for Soils A and B at lime contents above 5%.

Brandl [63] obtained similar results where the permeability increased on the addition of 1% lime, but decreased progressively at higher lime contents. He also found that the permeability decreases as the curing period is extended. After 180 days curing, 7% lim ample exhibited the same permeability as the unstabilized material.

As noted in Chapter 4, the specific gravity of the 'new' materials would have to be measured to be able to compare the void volumes more exactly. It is possible that the higher lime and cement content specimens were less susceptible to shrinkage cracking (generally the 'high' initial flowrates were observed in 5% specimens) and thus had lower permeabilities. It is also possible that the presence of excess lime in the lower plasticity soils increases the viscosity of the permeant thereby reflecting lower permeabilities. However since this testing programme considered only the quantification of the permeabilities, more precise tests will be required to determine the reasons for the observed behaviour.

Fossberg [58] also found that lime reduced the permeability of the clay soils he tested. The permeability decreased further when the specimens were cured for 4 months. Townsend and Klym [64] propose that Fossberg observed these results because he (Fossberg) compacted the soil-lime immediately after mixing (as the

author did in this project), which did not allow the clay to flocculate. Townsend and Klym maintain that "if the soil is allowed to cure for the recommended 48 hours in an uncompacted state to allow the reduction in plasticity necessary to expedite construction, then the flocculation of the clay will always result in an increase in permeability relative to the unstabilized material, if the two materials are compacted using the same compactive effort".

However the author found that it was not necessary to allow the material to cure before compaction without the material sticking to the compacting ram. There thus appears to be a compromise between curing time before compaction and the various properties obtained, i.e. some properties may be improved by curing before compaction while others may not.

For embankment dam design, Sherard et al [2] classify soils with permeabilities within the range 5.10^{-7} to 1.10^{-11}m/s as impervious to very impervious, while for an impervious core, Thomas [65] recommends that the permeability of the compacted core material should not exceed 1.10^{-7}m/s. The measured permeabilities of the stabilized materials ranged from 2.10^{-11} to 2.10^{-8}m/s. Since these permeabilities fall within the range of permeabilities acceptable for an impervious core, it would not be necessary to include other impermeable elements to reduce seepage losses if these stabilized soils were used in a dam.

7.3 Flow Through Artificially Created Cracks

An attempt was made to observe the erosion effects of fast moving water through an artificially created crack. Figure 7.5 shows the equipment used for these observations.

Cylinder specimens (37.5 mm diameter by 75 mm) cured for 28 days (standard cure) were cracked using cylinder splitting test equipment to yield two half cylinders. The two halves of the cylinders were held apart by thin steel wedges while they were glued in position by an epoxy normally used for concrete repairs.

After the epoxy had set, the wedges were removed and the area of the crack at each end measured. The cracked specimen was then installed in the cell, and a confining pressure applied. Water was then forced through the crack, the pressure applied to the flowing water being half the confining pressure. Once a consistent volume flowrate had been determined, the confining pressure was increased after which the flow pressure was also increased.

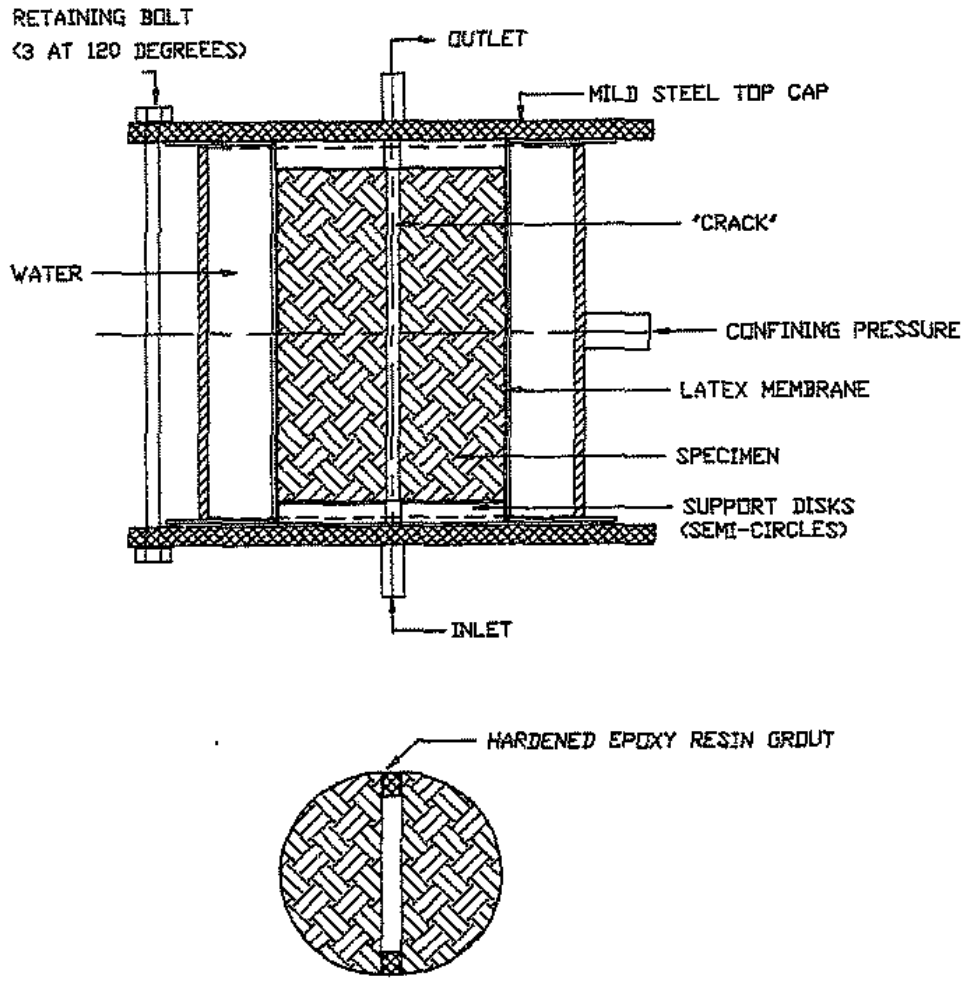


Figure 7.5: Apparatus for Measuring Flows Through Artificially Cracked Specimens

The velocity of flow was calculated by dividing the volumetric flowrate by the crack area. Since it was impossible to measure the crack area accurately, the results were adjusted so that the curve passed through the origin.

Figure 7.6 shows the results of the limited tests conducted. The equipment was calibrated using a specimen made from a granular soil with 20% cement (control specimen). For this specimen, the velocity increased linearly with increasing flow pressure. At higher flow pressures, the velocity increased at a decreasing rate, probably because of increased friction losses together with the closure of the crack at the higher confining pressures (the initial crack area was used for all calculations).

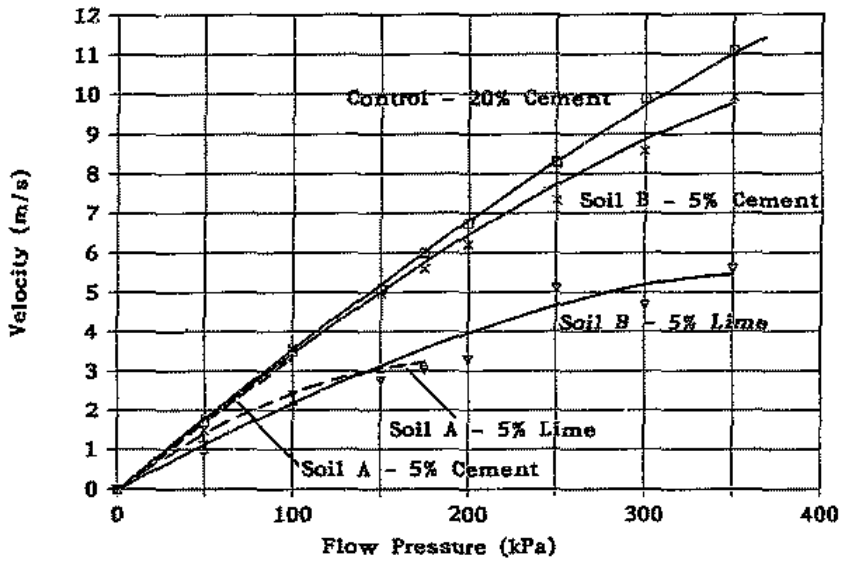


Figure 7.6: Velocity vs. Flow Pressure - Artificially Cracked Specimens (28 Day Cure)

Since a variable pressure continuous water supply was not available, the duration of flow for each test (i.e. for each data point) was limited to approximately 10

minutes.

Tests on unstabilized materials were not possible since the crack closed up under very low confining pressures. The observed outflows from the unstabilized specimens were extremely cloudy indicating that erosion of the material occurred at low flow velocities.

None of the stabilized specimens showed erosion in the form of cloudy outflow at any flow pressure, the specimens 'failing' when the crack collapsed under the confining stress. The graphs for the stabilized soil specimens are all similar to the control specimen with the rate of velocity increase decreasing to a larger extent. Early apparent failures appear to reflect the strength of a material rather than its erosion capacity, with the collapse or closure of the crack occurring earlier in the weaker specimens. The magnitudes of the unconfined compressive strengths (Figure 5.1) of these materials agree with this observation.

Although it is felt that the measurements are not highly accurate, the tests do reveal that the soil is far more erosion resistant when stabilized. A more sophisticated test will be required to determine whether such stabilized materials can be used without a protective layer to resist erosion.

The 'apparent' permeability of the specimens (for an approximate crack width of 1 mm), ranged from 1.10^{-4}m/s to 2.10^{-4}m/s. Compared with the permeabilities of the uncracked specimens which ranged from 2.10^{-11}m/s to 2.10^{-8}m/s it is evident that such cracks would result in unacceptably high seepage losses.

However is unlikely that extreme flows through shrinkage cracks will be problematic since in an embankment dam it is not likely that a shrinkage crack will develop through the entire width of the dam. Thus shrinkage cracks will probably not exist as open flow channels as they do in small specimens. With proper curing, the development of shrinkage cracks within the body of the dam can be almost entirely eliminated. Shrinkage cracks will then exist mainly on the surface of the dam and will not extend to any significant depth.

Since the durability of soil-lime materials will probably be a limiting factor in the design of an embankment dam using materials such as those tested in this programme, it is likely that a durable and relatively impermeable 'membrane', e.g. a cement rich granular soil-cement facing, will be used to face the dam. This would eliminate the risk of high flows through leakage paths and protect less durable materials from environmental effects.

Chapter 8

THE DURABILITY OF STABILIZED SOIL

8.1 Introduction

The durability of a soil can be broadly defined as that property of the material by which it *resists* environmental or other forces tending to cause the deterioration of the integrity of the material during its service life.

Destructive influences which can weaken cemented materials include:

- Repeated volume changes due to variations in temperature (freezing and thawing) or moisture content (wetting and drying). The tensile stresses developed can weaken or rupture the cementing bond, thereby causing a reduction in strength.
- The presence of deleterious materials. Salt crystallization, sulfate attack, acid and calcium carbonate can all lead to a loss of cementation and/or excessive heaving and cracking. Organic compounds are also believed to interfere with cement reactions.
- Leaching of cementitious products by percolating water.
- Abrasion and erosion e.g. by wave action or rainfall.

The durability of stabilized soil is not easily quantifiable and is normally investigated by conducting laboratory wetting and drying (ASTM D559) and freezing

and thawing (ASTM D560) tests. The ASTM tests correlate with the field performance of highway base courses in temperate climates, and thus may not be particularly relevant to the performance of stabilized soils in dams in Southern Africa.

The applied temperature ranges (71^0C drying and -23^0C freezing) are severe for South African conditions while the brushing of the specimens exerts stresses which are not realistic for all applications.

It was intuitively felt that the plastic stabilized soils used in this project would not be acceptable to dam builders as durable facing materials because of the high assurance levels required for this element of the dam structure. Better quality soil-cement with a fairly high cement content (approximately 10%) would be required to satisfy the criteria for soil-cement as a durable facing.

Since Robertson [9] has thoroughly investigated the leaching effects and the durability of such a material (a well graded silty sand with 4% clay sized material and PI = 5%), it was felt that repetition of such a testing programme was not warranted. It is worth noting that Robertson found for his soil that 2% cement was sufficient to satisfy the standard mass loss criteria for the wet-dry test. For slope protection, the PCA recommendations indicated a cement content of 4% while leaching tests indicated a cement content of 10% for minimizing permeability and cement loss.

Notwithstanding the above discussion, limited tests were conducted in order to make some comparison between the durabilities of the soils tested. This was motivated in part by the apparent contradiction arising from the inability of lime stabilized soils to satisfy the accepted durability tests, and the fact that soil-lime has performed successfully in hydraulic structures.

8.2 Durability Tests

A limited number of wet-dry tests were conducted on a number of standard specimens which had been subjected to percolation and soaked storage (see Chapter 7). Some of the soil-cement specimens at the lower stabilizer contents appeared soft at the 'inlet' end, this softening being evidence that some leaching of the stabilizer had occurred during the percolation test.

Since the specimens had been leached and wet stored for a prolonged period, a less harsh wet-dry testing procedure was adopted. The specimens were subjected

to cycles of wetting by soaking in a water bath followed by drying at 40^0C for 48 hours. No abrasion tests were conducted.

Soil D proved totally unsatisfactory with specimens at all stabilizer contents disintegrating after 1 or 2 wet-dry cycles. This was expected since Soil D contained a noticeable amount of organic material which appeared to affect all the engineering properties negatively.

Soils A and B behaved more favourably with 5% lime and cement specimens failing after 4 to 5 cycles, 7.5% specimens failing after 7 cycles, while both the 10% lime and cement specimens remained intact after 10 cycles. Soil C, which had a content of clay sized material of 60% (almost double that of Soils A and B) failed after 3 to 4 cycles at all stabilizer contents. Clearly a much higher stabilizer content would be required to render this soil as durable as Soils A and B.

Granular soils generally require low cement contents (5 to 7% by weight) to ensure satisfactory durability for hydraulic structures [4]. The durability of clay soil-cement depends on the strength of the cement matrix and on how effectively the volume changes in the clay can be minimized and restrained. In order to obtain a material sufficiently durable for hydraulic structures it becomes necessary to use a high cement content to enclose the clay lumps entirely with cement paste. A combination of lime and cement may prove to be a less costly method of rendering such a material less water sensitive and more durable.

For Soils A and B, lime appeared to be as effective as cement in improving the durability of the soils. It seems likely that the durability of soil-lime is questionable only in the short term when the material may exist in a non-plastic uncemented form and can disintegrate easily in the presence of water. Soaked strength tests could not be conducted on soil-lime specimens at 28 days because they disintegrated. In the long term, lime stabilization reduces the magnitude of volume changes (by reducing the water affinity) and by the formation of permanent cementitious reaction products which increase the tensile strength.

The size of the clay lumps also appears to influence the durability, i.e. the larger the lumps, the less durable is the material. Finer pulverization of the clay lumps would certainly allow more effective stabilization of the soil, resulting in higher strength and durability.

Throughout the thesis, the addition of lime in excess of the lime fixation capacity is emphasized. The durability results for Soils A and B again highlight that high lime contents are required to stabilize the material successfully, whereas poor results would have been obtained using the 'optimum' lime content for unconfined compressive strength at 28 days.

From the perspective of dam design, it is felt that the durability of stabilized soils should not be a deciding factor. As proposed in earlier chapters, the less durable material can be used in the body of the dam where moisture and temperature changes are small. More durable granular soil-cement can then be used to 'face' the external surfaces of the dam where temperature and moisture changes are large and durability becomes a critical design criterion. This type of approach is already used in the design of conventional and roller compacted concrete dams which use a lower cement content less durable material as the mass gravity component (termed 'hearting'), protected by a shell of cement rich durable material.

From the successes achieved in certain applications such as the Friant Kern canal, it is apparent that unprotected lime stabilized expansive clays can readily be used in hydraulic structures. Since such materials may be discounted for use because of their apparent lack of durability when compared with cement stabilized soils according to the standard tests, there is thus a need for more research into acceptable durability evaluation methods.

Chapter 9

SOME THERMAL PROPERTIES OF STABILIZED SOILS

9.1 Introduction

Generally, thermal volume changes are not significant relative to moisture related volume changes. Using a value for the coefficient of thermal expansion as quoted below, a temperature change of 10^0C may cause a strain of 0.01% which is small *compared to observed shrinkage strains* of the order of 1% for a change in moisture content of 1 to 2% (calculated for Soil C stabilized with 7.5% lime or cement). The thermal properties of stabilized soil are thus not dealt with in detail. Some information on the thermal properties of stabilized soil is presented below.

9.2 Coefficient of thermal expansion (α)

It is difficult to quantify the thermal expansions of soils in laboratory tests since a rise in temperature causes a loss of moisture which can offset the thermal expansion of the material. Should a quantitative measure of α be required it would probably prove more valuable to make determinations both in a dry state and sealed (preventing moisture loss) thus modelling exposed material and material in the body of the dam.

The Highway research Board [49] reports an increase in α with an increase in cement content and with an increase in density for all soil types. The rate of increase in α with density *is larger for clay soils.*

The coefficients of thermal expansion range from $5.5.10^{-6}$mm/mm^{0}C for low cement contents (2.5% by weight) to 11.10^{-6}mm/mm^{0}C for higher cement contents (10%). The variation between *soil types is small.*

9.3 Coefficient of Thermal Conductivity (k)

The thermal conductivity is defined as the rate of heat flow through a material under a unit thermal gradient.

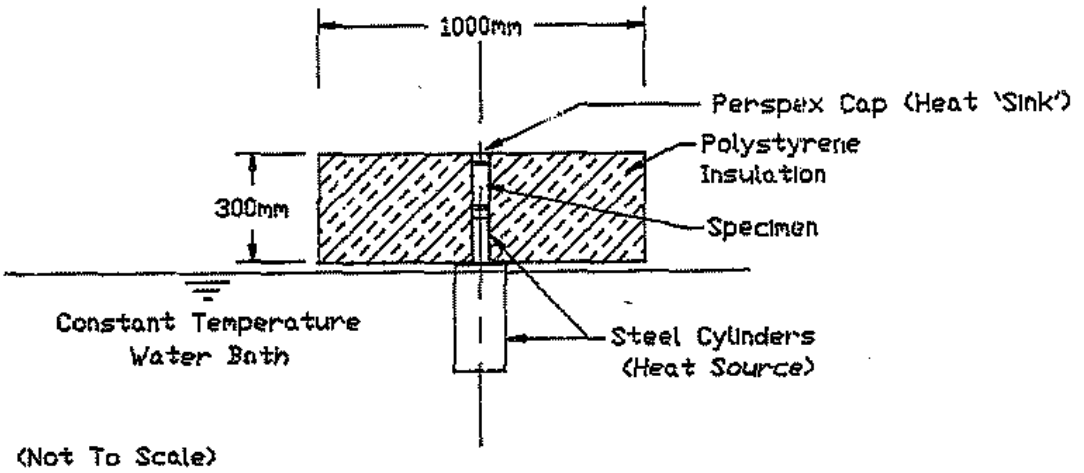
From data presented by Mitchell [27] it seems that at the same density and moisture content, silt and clay soils have lower thermal conductivities than sandy soils. Increasing the density at the same moisture content, or increasing the moisture content at a constant density both result in an increase in the thermal conductivity.

During the project, limited tests were conducted to determine the values of the coefficients of thermal conductivity (k) for Soils A, B and C unstabilized and stabilized with 5% cement and lime.

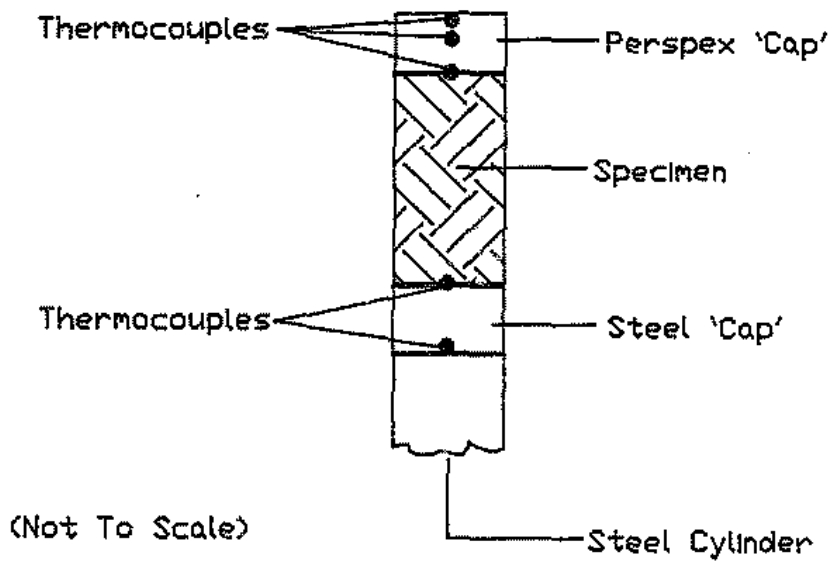
Figure 9.1 illustrates the components of the apparatus used in the tests. Although not sealed, the specimens were 'capped' at either end (the caps were glued to the specimen to ensure uniform heat transfer to the specimen), and enclosed within a large polystyrene foam block to minimize lateral temperature losses.

The bottom steel cap ($k \simeq$40 k.cal/h.m.^{0}C) acted as a heat 'source' while the perspex cap ($k \simeq$0,18 k.cal/h.m.^{0}C) acted as a heat 'sink' giving the thermal gradient across the specimen as shown in Figure 9.2. This configuration was chosen to allow a measurable temperature gradient to exist across the stabilized soil specimen and the perspex cap.

The bottom cap was heated using a large steel block partially immersed in a constant temperature bath, while the top cap was cooled by exposure to the atmosphere. Included in Figure 9.2 are typical temperature measurements, using thermocouples linked to a datalogger, for some of the specimens tested. The temperature gradients across the specimen and the perspex cap were then used to calculate the value of k for the soil specimen (a value of $k \simeq$0.18 k.cal/h.m.^{0}C was assumed [66] for the perspex) using the principle that the rate of heat flow

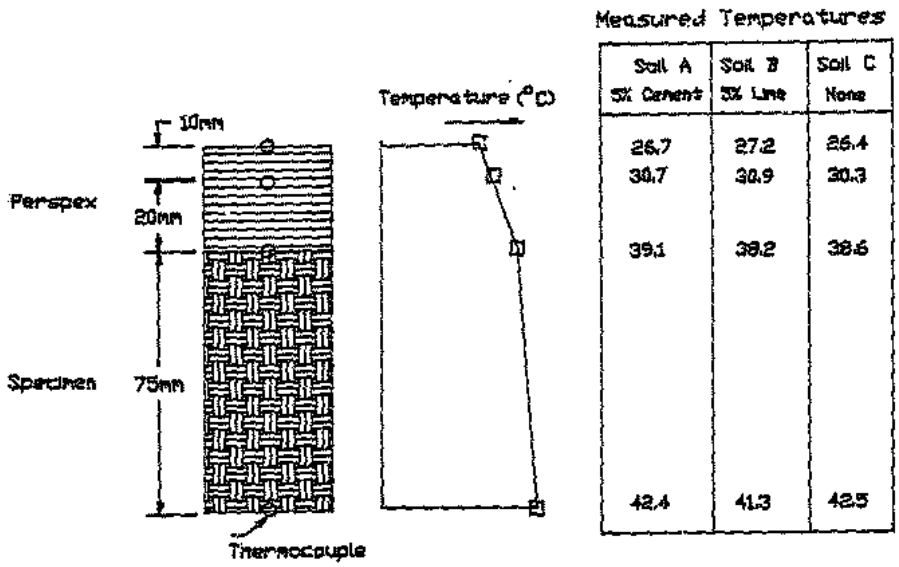


View of Assembled Thermal Conductivity Apparatus



Exploded View of Internal Parts

Figure 9.1: Thermal Conductivity Apparatus



Measured Temperatures

Soil A 5% Cement	Soil B 5% Lime	Soil C None
26.7	27.2	26.4
30.7	30.9	30.3
39.1	39.2	38.6
42.4	41.3	42.5

Figure 9.2: Typical Measured Temperature Gradients

across both materials is the same. The temperatures were logged continuously until equilibrium values were reached.

Table 9.1 presents the values for coefficient of thermal conductivity obtained from the tests. Also presented in the table are comparative values of k for concrete and various soil-cement mixtures as quoted by other references. Although the test results may not be highly accurate because of the simplicity of the equipment and the assumed value of k for the perspex cylinder, they appear to be near the values obtained from other sources for similar materials and of adequate comparative value.

The clay soil (Soil C) had a lower value of k than Soils A and B, while the lime treated materials showed lower values of k than their cement treated equivalents. For comparison, a silty sand stabilized with 20% cement was also tested. This material which was compacted to a higher density at a lower moisture content than Soils A, B and C exhibited the highest thermal conductivity.

The results also appear to follow the trends in thermal conductivity reported in the sources consulted [41,66] i.e.:

- The thermal conductivity increases with increasing density.
- The thermal conductivity increases with increasing moisture content since

Table 9.1: Thermal Conductivities of Stabilized Soils

Soil	Stabilizer *Content* (%)	Coefficient of *thermal conductivity* (k) (k.cal/h.m.^{0}C)
A	none	1.7
A	5% cement	1.7
A	5% lime	1.55
B	none	1.7
B	5% cement	1.7
B	5% lime	1.6
C	none	1.4
C	5% cement & 2% *lime*	1.35
C	5% lime	1.35
Silty sand	20% cement	1.9
Concrete [41]		1.0-2.3
Sand-cement [49]		2.25
Silt-cement [49]		1.2
Clay-cement [49]		1.1

the thermal conductivity of the water ($k \simeq 0.5$ k.cal/h.m.^{0}C) is greater than the thermal conductivity of the air it replaces ($k \simeq 0.02$ k.cal/h.m.^{0}C).

- Limestone aggregates yield concretes with lower thermal conductivities than concretes containing quartzite aggregates.
- An increase in the cement content results in an increase in the thermal conductivity.

Chapter 10

DESIGN OF STABILIZED SOIL EMBANKMENT DAMS

10.1 Introduction

The preceding chapters present the results of the laboratory investigations into the properties of stabilized soils relevant to embankment dam design. This chapter examines the validity of the premise of the thesis, i.e. whether the addition of the stabilizer increases the strength of these materials to the extent that a cost saving over conventional solutions can be achieved.

Robertson [9] used the Rondebosch Dam as a case study to compare the costs of alternative structures. Data were available for the concrete buttress dam constructed at the site, as well as for an alternative (proposed) earth embankment dam. Although Robertson compared costs essentially on the basis of material and construction costs per unit volume of material, he was able to show that a soil-cement embankment dam could have been considerably cheaper than the concrete buttress dam actually constructed at this site.

For the strength and stability analyses and the cost comparisons presented in the following sections, it was decided to assume the section dimensions of the 'proposed' Rondebosch earth embankment.

The section dimensions are:

Crest width	6.0 m
Height of spillway crest	35.4 m
Maximum reservoir depth at design flood	39.4 m
Maximum height	40.9 m
Volume of proposed earthfill embankment (for comparison)	890 000 m^3

The design of an embankment dam requires thorough consideration of numerous aspects such as foundation strength and suitability. Such a design is beyond the scope of this project, and the analyses are thus greatly simplified, the intention being more to demonstrate the feasibility of the use of the materials in terms of strength and cost.

A limitation on the use of stabilized soil embankment dams is that the foundation must be sufficiently incompressible to ensure that large transverse cracks do not develop as a result of differential settlement along the longitudinal axis of the dam. For the considerations which follow, it is therefore assumed that the foundations are sound and incompressible.

10.2 Material Properties for Design

From the results of the laboratory investigation presented in the previous chapters, it is clear that the most efficient stabilization i.e. highest strength per mass of added stabilizer, is achieved when cement is combined with lower plasticity soils having well distributed particle sizes.

For the strength and stability analyses it was decided to consider only Soil B stabilized with cement and lime, since this material gave the best results in terms of strength. From the results of these analyses, the requirements for other soil stabilizer combinations can be deduced.

The properties of Soil B at various cement and lime contents abstracted from results presented previously are shown in Table 10.1.

The triaxial shear strength parameters c' and ϕ' at 540 days were calculated from the unconfined compressive strength data at this age using the Mohr failure envelope ($\sigma'_1 = 0$ and $\sigma'_3 = \text{UCS}$) assuming that ϕ' remains constant with age.

Table 10.1: Summary of Compaction and Strength Properties - *Soil B*

	Compaction Characteristics		Shear Strength			
			28 Days		540 Days	
	Optimum Moisture Content (%)	Maximum Dry Density (kg/m^3)	c' (kPa)	ϕ' (degrees)	c' (kPa)	ϕ' (degrees)
No stabilizer	22.5	1578	30	22	30	22
5% *cement*	22.7	1580	300	28	330	28
5% lime	23.8	1530	320	27	300	27
7.5% cement	22.4	1583	360	39	600	39
7.5% lime	24.0	1520	225	35	550	35
10% cement	22.1	1586	485	43	925	43
10% lime	24.3	1510	130	40	550	40

10.3 Analysis of Strength Requirements

Since the properties of stabilized soils lie between those of concrete and unstabilized soils, it is reasonable to assume that the 'bulk' behaviour of the stabilized soil in the body of an embankment dam will also be somewhere between the rigid behaviour of concrete and the plastic behaviour of soils. In other words, a simplified design of a stabilized soil embankment dam can be carried out by satisfying the strength requirements for the extremes of the possible behaviour of the material.

10.3.1 Soil-cement

Strength requirements for gravity stability

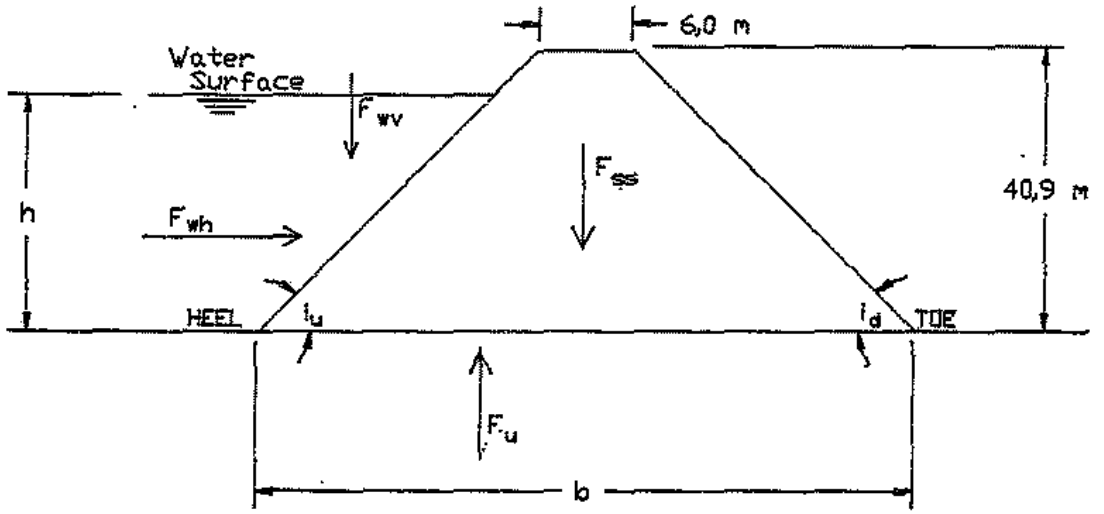
The gravity section is analyzed by the 'Gravity Method of Analysis' [67] where the following assumptions are made:

- The stabilized soil in the dam is homogeneous, isotropic, and uniform elastic material.
- All loads are carried by the gravity action of vertical, parallel-side cantilevers which receive no support from the adjacent elements on either side.
- Normal stresses on horizontal planes vary linearly from the upstream face to the downstream face.
- Horizontal shear stresses have a parabolic variation across horizontal planes from the upstream face to the downstream face.

Figure 10.1 shows the applied loading on the section under consideration. Equal upstream and downstream slopes are assumed throughout.

The factors of safety generally applied to concrete gravity dams [67] are:

- The factor of safety against overstressing (Sc) is normally specified as 3.0, 2.0 and 1.0 respectively for usual, unusual and extreme loading combinations.
- The shear friction factor of safety Q is specified as 3.0, 2.0 and 1.0 respectively for usual, unusual and extreme loading combinations.



F_{ss} = weight of stabilized soil
F_u = total uplift force
F_{wv} = weight of water on upstream slope
F_{wh} = horizontal water force on upstream slope
l_u = upstream slope angle
l_d = downstream slope angle
h = 35,4 m for normal dam operation
h = 39,5 m for design flood condition

Figure 10.1: Applied Loading on Gravity Section

Figures 10.2 and 10.3 show the results of the gravity analysis in relation to the recommended safety factors. The compressive stress safety factor reflects the critical stress at the toe of the dam while the analysis for the shear friction safety factor was performed for the base of the dam.

The 'critical gravity section' is defined as the section having zero vertical stress at the heel. For the section dimensions assumed, the limiting critical section occurs for the 'design flood' condition at a slope angle of 65^0.

The results show that the limiting condition according to the recommended safety factors is compressive stress (at the toe) under normal operating conditions, for which a cement content of 10% must be used in order to construct a dam of equal upstream and downstream slope angles between 54^0 and 62^0. Assuming that the foundation conditions permitted, the steepest slope would be used to minimize the cost of the embankment fill.

As pointed out by Robertson and Blight [8], since stabilized soils behave plastically under high confining pressures, it seems probable that plastic deformation will lead to a redistribution of stresses in highly stressed zones. Consequently the application of the rigid body analysis to stabilized soils may be conservative.

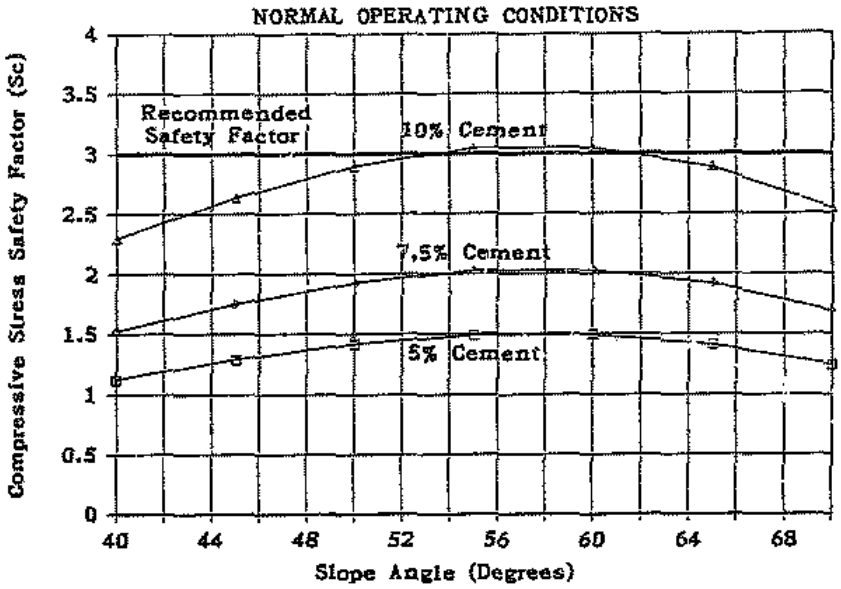
One would thus probably be justified in selecting the critical section (65^0) even though the safety factor for the critical section is slightly less than 3.0 (compressive stress, normal operating condition).

For the critical gravity section under the critical loading condition (design flood), the shear friction safety factor is 5.6. However, in the consideration of the shear strength on lift joints, the full shear strength of the material cannot be assumed to have developed between the faces of the joint, i.e. the safety factor must allow for the reduction in shear strength across bonded faces.

The shear strength can be considered (section 5.3) as comprising two components, i.e. cohesive and frictional shear resistance. Considering the lift joint to be a horizontal failure surface, the least resistance to shearing that can be expected is if the cohesive strength is zero, i.e. the 'residual' or 'frictional' strength is:

$$\tau = \sigma' tan\phi'$$

The shear friction safety factor on the lift joint can then be considered in terms of the residual (frictional) strength plus the proportion of cohesive strength assumed to have developed between the joint surfaces. Figure 10.4 shows the shear friction safety factor in terms of the residual strength on the joint ($c' = 0$) and the proportion of cohesive strength developed. From this graph only 28% of the



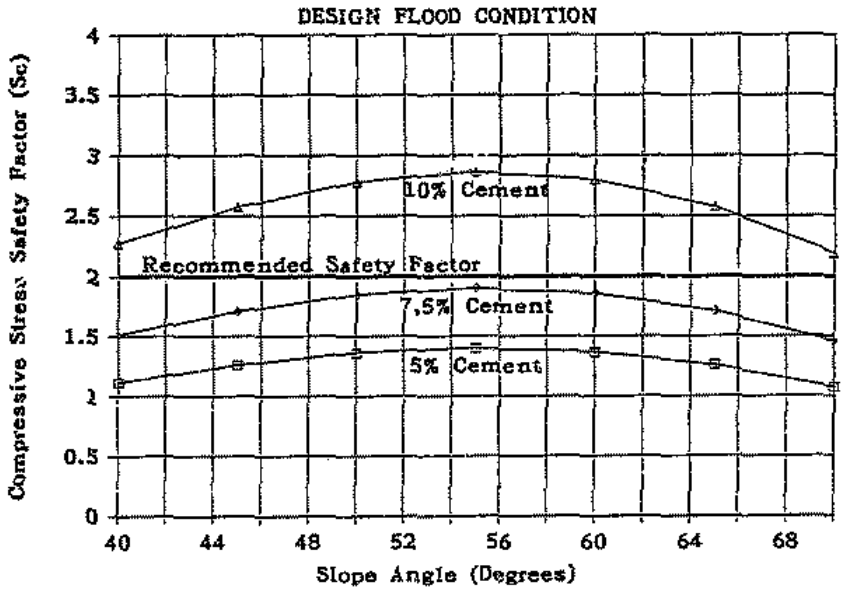
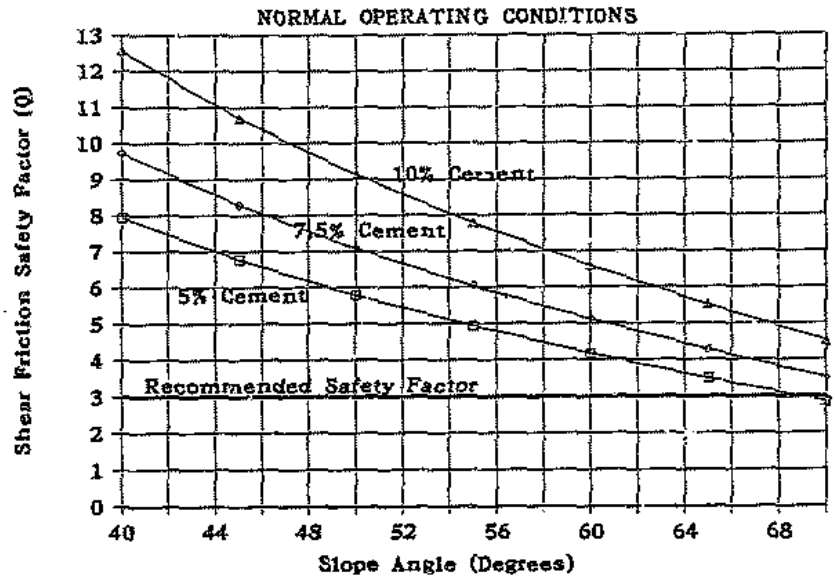


Figure 10.2: Compressive Stress Safety Factor vs. Slope Angle - Soil B-cement (28 days)



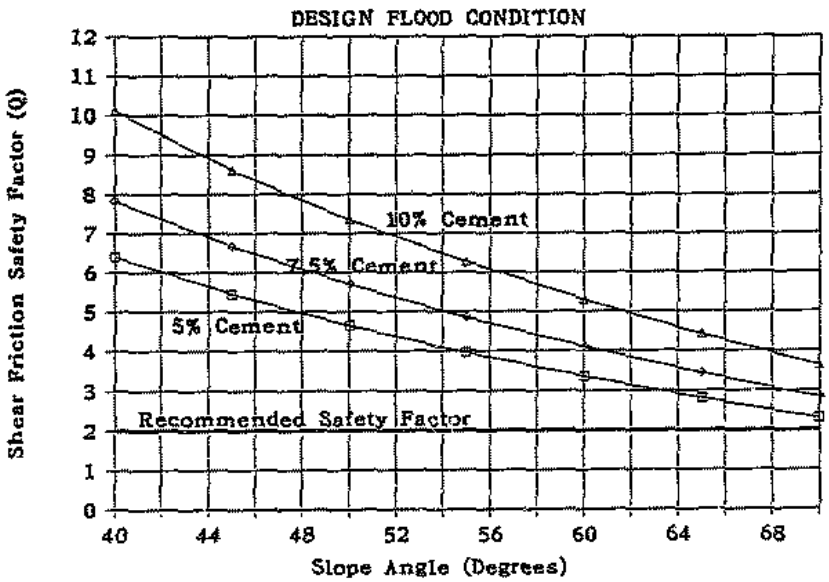


Figure 10.3: Shear Friction Safety Factor vs. Slope Angle - Soil B-cement (28 days)

cohesive strength need be developed to satisfy the safety factor of 3.0 on a lift joint at the base of the dam.

Robertson [9] points out that the safety factor of 3.0 is seemingly only applicable to conventional concrete gravity dams, since concrete has a high ratio of peak to residual strength. He proposes that design for shear friction for soil-cement should be based on the residual strength with a factor of safety of 1.2 applied for material variability, while the shear friction safety factor as previously determined should be a minimum of 2.0 for normal and unusual loading conditions.

Figure 10.4 shows that the residual strength is sufficient to satisfy these conditions for the normal operating condition, while development of 13% of the available cohesive strength would be required to satisfy the ultimate strength requirement.

Construction control measures such as brooming joint surfaces, treating the surface with a neat cement slurry, and reducing delays between construction of consecutive layers should improve the interlayer bond considerably should this be deemed necessary.

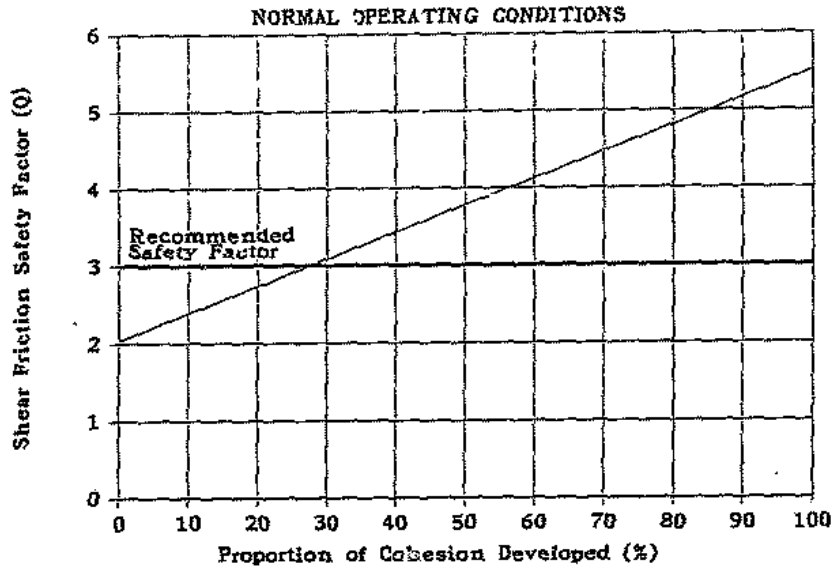
Figure 10.5 shows the compressive strength safety factor (S_c) for the same materials at an age of 540 days. At this age, a cement content of 7.5% would be sufficient to satisfy the recommended safety factors.

The loading conditions considered occur only after first filling and it will probably be justifiable to design on the basis of the cured strength at this age. Although the 540 day curing period of the tested specimens will probably be longer than the period required to construct the dam, these specimens had been subjected to percolation and soaking for 512 days. Thus the safety factors can be assumed to represent those for unleached material after a shorter curing period, although this may be conservative when the severity of the leaching tests is considered.

Thus the minimum cement content for the critical section can be set at 7.5%.

Slope stability analysis

Earth embankment dam design requires that the upstream and downstream slopes be stable under various operating conditions. The critical times [28] for the upstream slope are end-of-construction and during rapid drawdown whereas the critical times for the downstream slope are end-of-construction and steady seepage under normal (reservoir full) operating condition.



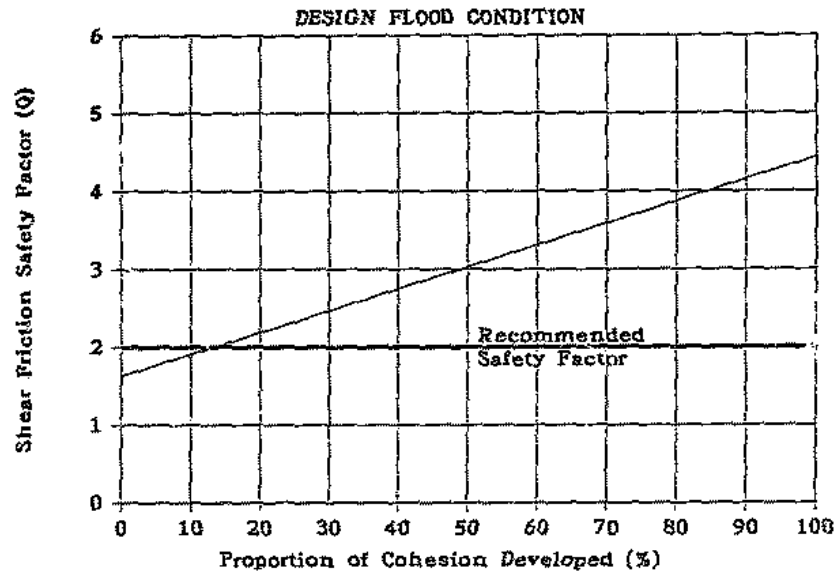


Figure 10.4: Shear Friction Safety Factor in Terms of Residual Strength Plus Developed Cohesive Strength - Soil B - 10% Cement (28 days)

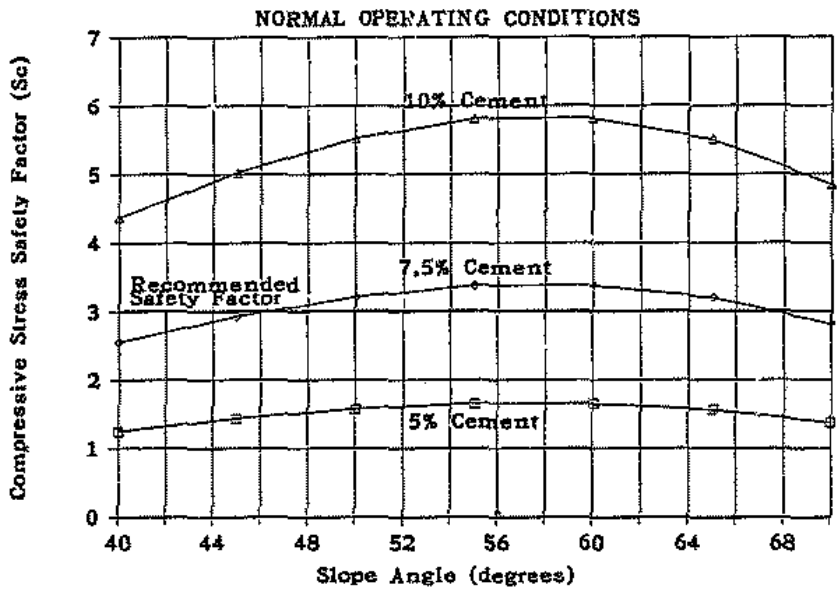


Figure 10.5: Compressive strength Safety Factor vs. Slope Angle - Soil B-cement (540 days)

Since it is likely that construction pore pressures will dissipate fairly rapidly since the pore water will be used up to hydrate the cement, the critical times considered in the following analyses are rapid drawdown (upstream slope) and reservoir full (downstream slope).

Slope stability analyses were carried out using a computer programme STABL, developed to handle general slope stability problems. The programme searches for the critical failure surface, using a random technique for generating circular failure surfaces. The author selected the Modified Bishop method for the analysis (method of analysis is optional).

Figure 10.6 shows the variation of the slope stability factor of safety for the upstream and downstream slopes. These results indicate that both upstream and downstream slopes are stable at the selected minimum cement content of 7.5% using the material strengths at 28 days.

10.3.2 Soil-lime

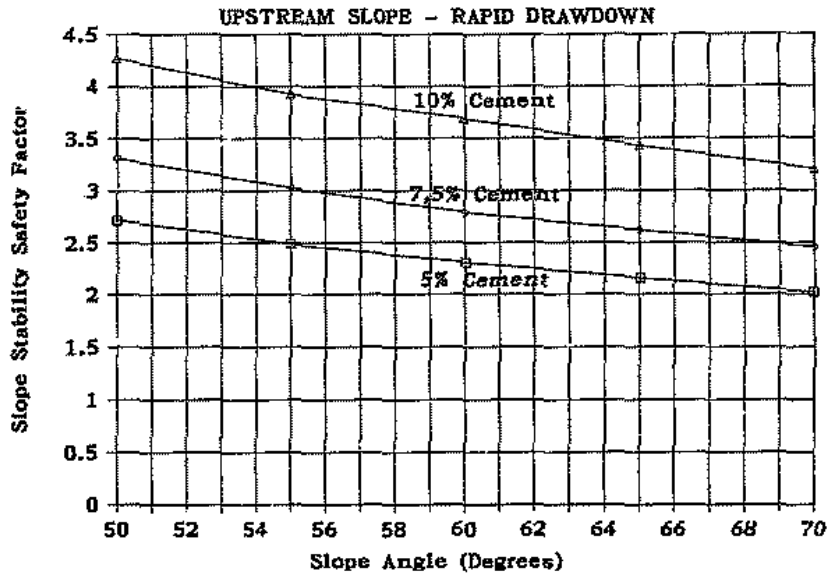
Figure 10.7 shows the variation of compressive stress safety factor with slope angle and lime content at ages of 28 days and 540 days for Soil B, while Figure 10.8 shows the variation of the shear friction safety factor with the same parameters.

Throughout the thesis it has been emphasized that soil-lime should not be judged according to 28 day strengths, unless short term strength is required. These graphs show that although the soil-lime may have been rejected at 28 days, sufficient strength gains have occurred at 540 days such that it might be possible to construct the dam using soil-lime with a lime content of 7.5%.

It will however be necessary to measure the strength gain with time so that the required strength is available at first dam filling.

For a 7.5% lime content, the safety factors (based on the shear strength parameters at 28 days) for the upstream and downstream slopes of the critical section are 1.8 and 2.1 respectively. At 10% lime (28 days) the safety factor for the upstream slope is 1.2. These safety factors can be expected to increase significantly with time.

Since the slope stability safety factors were obtained considering operating conditions and using 28 day strengths, it is unlikely that slope stability will be critical during construction. Pore pressures are also likely to dissipate rapidly since the pore water will be required to hydrate the lime.



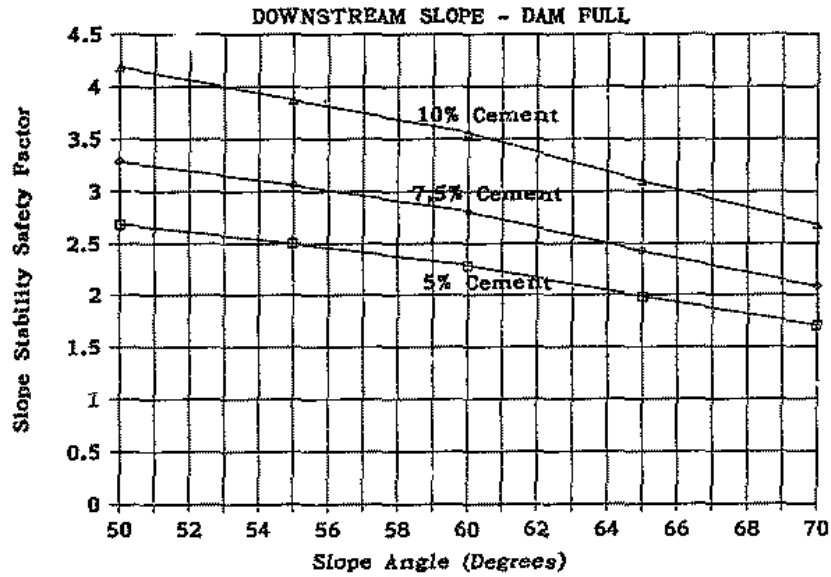
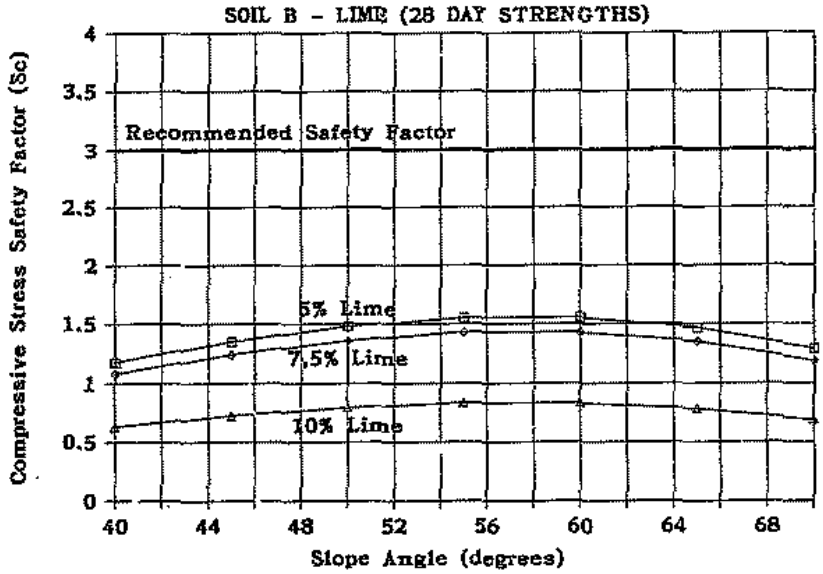


Figure 10.6: Slope Stability Factor of Safety vs. Slope Angle - Soil B-cement (28 days)



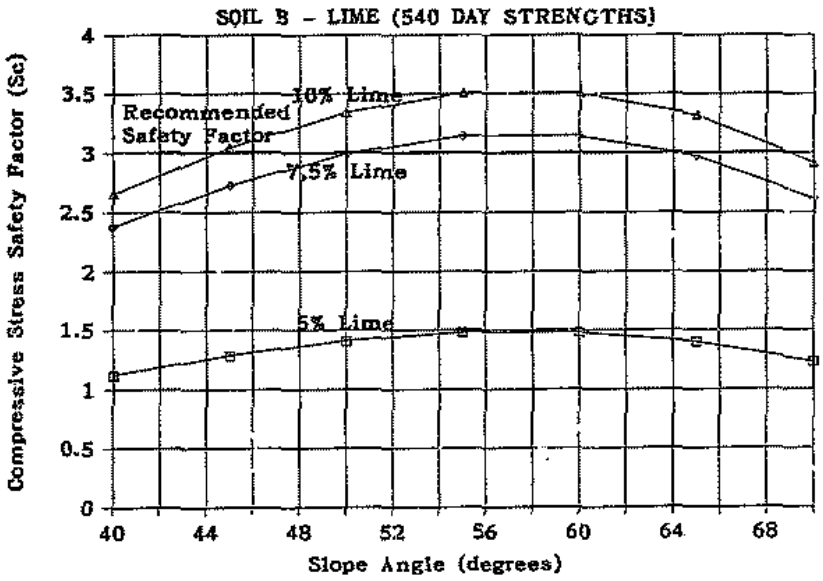
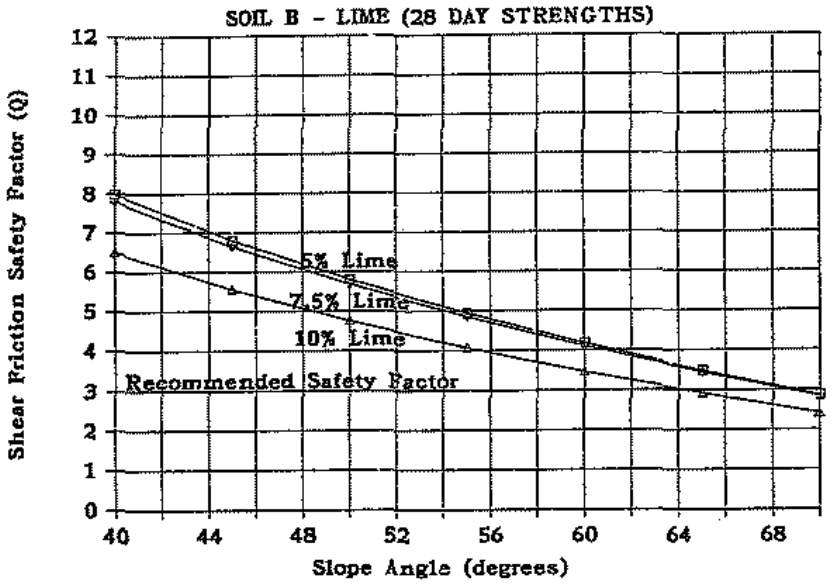


Figure 10.7: Compressive Stress Safety Factor vs. Slope Angle - Soil B-lime - 28 and 540 Days - Normal Operating Condition



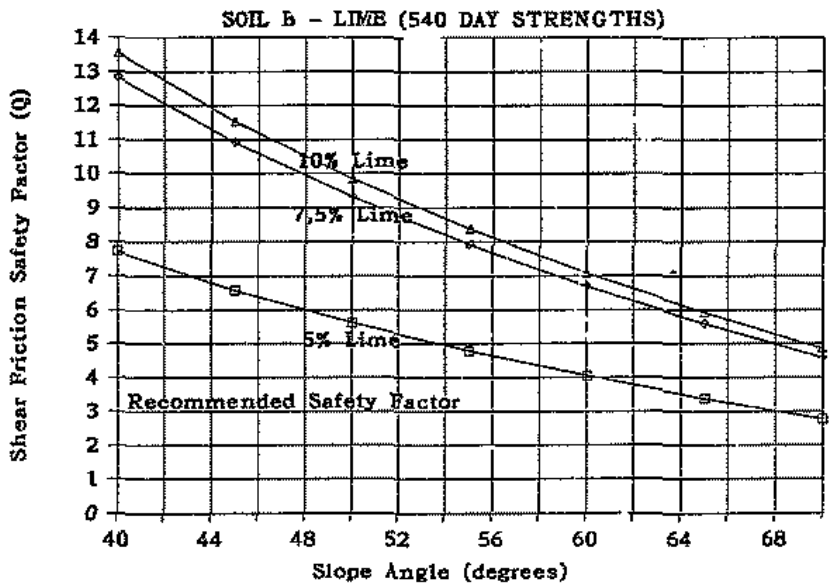


Figure 10.8: Shear Friction Safety Factor vs. Slope Angle - Soil B-lime - 28 and 540 Days - Normal Operating Condition

However since soil-lime has low cohesive strength at the time of compaction, it is probable that with the steep slopes proposed, problems will be experienced because of concentrated surcharge loads applied by compaction equipment. Furthermore it is likely that difficulty will be experienced in achieving the required degree of compaction at the edges of the layers where there is no confining support.

Thus it is likely that some form of confining support will have to be provided to prevent local slip failures and to provide a 'buttress' against which the soil-lime can be compacted.

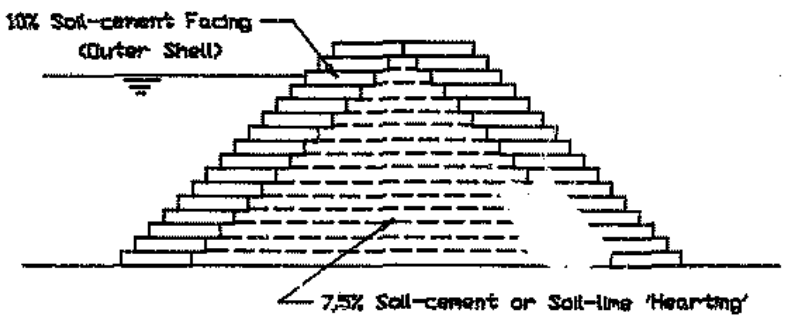


Figure 10.9: Composite Stabilized Soil Dam

These problems can probably be overcome by using a layer of better quality soil-cement at a fairly high cement content (say 10%) to form the outer 'shell' of the dam (see Figure 10.9). For the soil-lime dam, this shell would probably be required in either event for slope protection.

Due consideration will then have to be given to problems which may arise from differential settlement between the facing and the hearting. However from the

successes achieved with the facing of earth embankment dams with soil-cement, it does not seem that this will present a problem.

Since it will not be possible to prevent carbonation of excess lime near the surface of a compacted soil-lime layer in the period between the construction of successive layers, it will be necessary to institute control measures such as those listed below, to minimize carbonation or possible failures because of the presence of carbonated material.

- Delays between the construction of successive layers must be minimized to minimize exposure to atmospheric carbon dioxide.
- Areas where lengthy delays may have allowed carbonation to occur must be tested for carbonation. The 'rapid field test for carbonation of lime or cement treated materials' suggested by Netterberg [68] i.e. the treated material is sprayed with pH indicators to test for the presence of unreacted lime and the reaction with dilute hydrochloric acid solution is observed (effervescence indicates carbonation), will probably suffice as an indicator of carbonation. Carbonated material must be removed upon identification.
- The surface of each layer must be swept clean of all loose and possibly carbonated material prior to the compaction of fresh soil-lime to ensure maximum bonding between successive layers.

10.3.3 Soils A and C

By comparing the long term unconfined compressive strengths of Soils A and C (Soil D having been rejected) with the strength of Soil B for the chosen section (Figures 5.2 to 5.4), the following can be deduced:

- Soil A could only be used for a structure of the assumed dimensions when stabilized with 10% cement.
- Soil C, which would probably have been rejected early in the design on the basis of the indicator tests (PI = 52%) and the particle grading distribution, showed long term strengths comparable to Soil B stabilized with 7.5% lime or cement. The lime option would be chosen since the cement stabilization requires the addition of 2% lime to facilitate mixing, i.e. two or more stages of mixing.

Thus it is conceivable that an embankment dam could be constructed from material which probably would be classified as spoil. The normally encountered problems of placing such a plastic material during wet weather will be overcome by reducing the plasticity using lime.

A two stage process, i.e. preconditioning excavated material with 2% lime followed by stabilization with the remainder of the lime required as the material is placed, will probably prove the most practical method of ensuring high rates of placement.

10.4 Cost Comparison

In the absence of a factual design where various alternatives are thoroughly investigated, a simplified cost comparison must be viewed as approximate. The only costs which can be compared fairly accurately are the 'in place' costs of the materials used for the different structures. Even these may not be entirely realistic, since for example, quoted costs for bulk earthworks vary considerably while the price of stabilizers on site in South Africa vary depending on the proximity of the site in relation to the cement or lime plant.

The material unit cost will also vary according to the haulage distance. In arriving at a cost estimate, it is assumed that the soil for the stabilized soil embankment is closer to the dam site than soil for an unstabilized earth embankment (one would attempt to make use of spoil from foundation or spillway excavations) and that the earth embankment dam is likely to be 'zoned', i.e. more than one borrow material is required. The cost for 'borrow to fill' is thus assumed to be the same for both materials, currently approximately R 7-00/m^3, i.e. the cost of additional processing required for the stabilized soil is offset by the haulage cost and processing costs of the various materials for the zoned earth embankment.

The cost of the lime and cement per ton is fairly similar. The retail material cost 'ex plant' is approximately R 107/ton, but this cost is dramatically increased by the additional cost of transporting the stabilizer to the site. An average cost of R 215/ton (cement and lime delivered to site) is assumed.

The proposed earth embankment dam required approximately 890 000 m^3 compacted earthfill [9]. The cost of compacted earthfill for this dam is thus R 6.2 million.

An homogeneous stabilized soil dam with 35^0 upstream and downstream slopes

would require 201 000 m^3 compacted earthfill. This volume is based on an equivalent length of the dam wall of 220 m with the section dimensions given earlier. Based on an average stabilizer content of 7.5% by dry mass, the cost of the stabilizer is R 4.74 million, while the earthworks cost is R 1.4 million, i.e. a total cost of R 6.14 million.

These estimates do not take into account the cost of the slope protection. If a soil-cement facing using 10% cement is used in either case (upstream facing for earth embankment but both faces for stabilized soil dam), then the cost of the earth embankment increases to R 7.1 million, while the cost of the stabilized soil dam increases to R 6.5 million. Since soil-cement facings have not yet been widely used in South Africa, it is likely that preference would be given to rip-rap slope protection of the earth dam at an increased cost.

One could expect considerable cost savings to emanate from the reduction in construction time. The volume of placed stabilized material is approximately 25% of the earthfill volume. If the rate of placement of stabilized soil relative to unstabilized soil is halved because of the additional stabilization procedures, the construction period would also be halved. The rates of placement will probably be similar since the earth dam requires the placement of filters and zones.

Reducing the construction time reduces the costs in a number of areas including escalation, maintenance of site, wear and tear of equipment etc. Contractors will probably be able to offer lower rates because of the overall cost of the project relative to the construction period.

The costs of cement and lime in the large quantities required are likely to be less than those quoted above which were obtained from current prices for relatively small quantities for roadworks projects.

The extent of foundation preparation can be expected to be greater for the earthfill dam, but the foundation depth of the the stabilized soil dam would probably be greater. The zoned earth embankment dam requires filters while it is unlikely that the stabilized soil dam will require filters.

Further cost savings will result from the use of an integral soil-cement overflow spillway rather than a chute spillway commonly used for earth embankment dams. The chute spillway normally requires additional excavation along an abutment.

Thus under the appropriate circumstances, the stabilized soil dam could provide significant cost savings relative to a conventional earth embankment dam.

The present cost of the concrete buttress dam actually constructed, excluding

excavation, pipework and gates, is estimated at nearly R 20 million, a large contribution (approximately 21%) of this cost coming from escalation over the 3 year construction period [9].

The large difference in the costs of the earth embankment and the concrete dam suggests that the earth embankment costs would be significantly increased by other factors, or in fact that the materials were not considered suitable. Apparently only preliminary investigations were carried out for the earth embankment proposal which itself was not a comprehensive design [69].

However, there is no reason to suggest that a stabilized soil or rollcrete dam could not have been constructed in place of the concrete buttress dam, in which case the above cost estimates indicate that a significant cost saving could have been achieved, even if the stabilized soil costs are underestimated.

10.5 Alternative Dam Configurations

The preceding discussion considered only equal upstream and downstream slopes whereas other configurations may provide more economical solutions.

Large soil-cement dams could probably be constructed with a vertical upstream face, thereby reducing the volume of hearting material. This configuration is popular in the construction of RCC dams (Figure 10.10).

There are various options for the construction of the upstream face, including conventional concrete cast in place using vertical shutters, precast concrete panels, compaction of RCC against shutters and slipformed curbing. It is unlikely that one would attempt to use the soil-cement as the vertical facing, unless the resulting material had properties near those of conventional concrete or better quality RCC.

It is important that the facing and the hearting act as an integral structure, i.e. there must be adequate bonding between the facing and the hearting so that they do not separate under the applied loads or through differential settlements. If the materials deform independently under the horizontal water load, the less flexible facing will be subjected to bending. For materials where the strength of the bonding is doubtful, or where differential settlements may become problematic, the integral action can be achieved using precast concrete panels anchored into the hearting material.

For soil-lime dams, a similar configuration could be achieved using reinforced

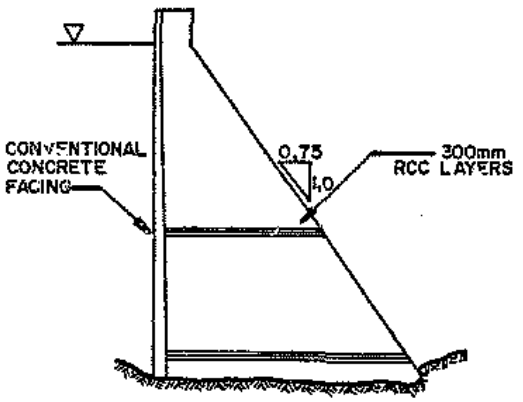


Figure 10.10: Soil-cement or Roller Compacted Concrete Dam with Vertical Upstream Face

fill. A vertical face could be formed by interlocking precast concrete panels tied back into the soil-lime with reinforcing straps. Smaller vertical faces or fairly steep slopes could also be constructed using fabric reinforced fill. Reinforced fill technology relies on the development of friction between the compacted soil and reinforcement, rather than the development of a cohesive bond. Thus the fact that soil-lime does not develop cohesive strength rapidly, becomes unimportant.

Since the frictional resistance of the soils tested increases significantly on the addition of small quantities of lime, the stabilizer content can be reduced to the content required to modify the soil adequately to expedite construction and to achieve the required frictional resistance. The use of lime might however provide an additional benefit in raising the pH of the fill, thereby reducing corrosion of steel reinforcing straps, although these are generally galvanized to prevent corrosion.

Reinforced fill structures are fairly flexible since elements are not fixed to one another. Thus the larger deformations expected in the soil-lime can be accommodated without the integrity of the structure being affected.

For a high vertical face structure, it is unlikely that any seal between interlocking panels would be required, since the soil-lime appears to be sufficiently impermeable to be used without any water barrier. It is worth noting that this type of structure has been used to form the vertical downstream face of a flip bucket spillway (Taylor Dam, U.S.A. [70]).

10.6 Other Design Considerations

Other design considerations such as permeability and durability have been discussed to some extent in the earlier chapters.

Although thorough consideration of such aspects will be crucial in gaining wider acceptance of stabilized soils in embankment dams, it is felt that further analysis requires the laboratory test results to be supported by results from the monitoring of small scale trial embankments.

Since laboratory testing apparatus limits the size of the tested specimens, unanswered questions remain as to certain practicalities e.g. since it will not be possible to pulverize clay lumps to the degree achieved by crushing in a small laboratory jaw crusher, will the properties of stabilized soils prepared using normal road stabilization equipment be satisfactory? The need for specialized equipment would increase costs, possibly making the use of such materials inappropriate.

Although it is felt at this stage that the use of stabilized soil embankment dams will be restricted to sites with relatively incompressible foundations, it may prove feasible to construct a soil-lime dam on fairly compressible foundation material such as that occurring in an alluvial valley. The soil-lime behaves more plastically and gains strength at a much slower rate than soil-cement and hence will accommodate settlements with less distress.

Both soil-cement and soil-lime show evidence that they are capable of self-healing, while they are sufficiently erosion resistant to eliminate the risk of piping failures. These behaviour characteristics suggest that it may be possible to assess the effects of transverse cracks in the design for stabilized soil dams on compressible foundations. These effects and the investigation of remedial measures such as the grouting of cracks either before filling the dam or during normal operation, can only be assessed on a larger scale structure.

The fact that soil-lime used to line channels subjected to fairly high flows and numerous wet-dry cycles has not shown any amount of distress [12], suggests that

the currently used durability test methods might be too harsh. Evaluation of the laboratory test methods would require simultaneous evaluation of the durability of the material under larger scale, realistic conditions.

It appears that neither the rigid body analysis nor the slope stability analysis is ideally suited to the analysis of the bulk behaviour of stabilized soil. Although a select, high cement content soil-cement content may have concrete-like properties, stabilized soils have lower elastic moduli than concrete indicating that the rigid body analysis is conservative.

The monitoring of trial stabilized soil embankments would provide data for the development of analytical models (probably of the finite element type) to predict the bulk behaviour of a variety of materials under the appropriate loading conditions.

Numerous ideas as to dam configuration can be generated to suit the variety of products obtained through stabilization, such as the proposed soil-lime reinforced fill dam. Although theoretically feasible, the appropriateness of such ideas can only be evaluated through the attempted design and construction of such structures.

10.7 Approximate Design Parameters for the Preliminary Appraisal of Stabilized Soil Dams

During the early stages of a design, it is desirable in considering the different alternative dam types and construction materials available, to eliminate those alternatives which are not suitable for reasons such as prohibitive cost, insufficient quantities of construction materials or inadequate engineering properties.

The intention of this section is to provide the designer with approximate design parameters in order to be able to carry out a preliminary appraisal of stabilized soil alternatives after only minimal testing of the locally available soils.

10.7.1 Subdivision of Soil Groups

For soil-cement, the soils are approximately divided (in line with the AASHTO classification system) into:

Granular soils - less than 35% by weight finer than 0.075 mm (silt sized).

- Fine-grained soils - more than 35% by weight finer than 0.075 mm.

For soil-lime, the soil is assumed to be fine-grained, with a clay content (finer than 0.0025 mm) of more than 5% and a PI of 10 or more.

In the case of soil-lime, it will be necessary to determine as early as possible whether a soil is lime reactive. However this need only be done if the preliminary assessment shows that soil-lime might yield a promising solution.

10.7.2 Design Parameters

Approximate design parameters required to perform gravity and/or stability analyses are presented below. The values of the parameters have been extracted from references [9], [25], [56] and from the results of this thesis.

Comments on parameter trends are included to aid in the selection of appropriate design parameters.

Compaction Characteristics

Table 10.2 lists approximate parameters for the compaction characteristics of stabilized soil using Standard Proctor compactive effort.

If the particle grading distribution and the plastic limit are known, the compaction characteristics can be estimated [71] using the following equations:

$$ln\text{OMC} = 0.784 ln\text{PL} + 1.378 ln(\text{FA} + 100) - 6.586$$

and

$$0.0625 ln\text{MDD} = 7.247 - 0.567 ln(\text{PL} + 20) - 0.11 ln(\text{FA})$$

where ln is natural logarithm, OMC the optimum moisture content (%), MDD the maximum dry density (kg/m^3), PL the plastic limit, and FA the fineness average and:

FA = $\frac{1}{6}$(Sum of % finer by weight than mm particle sizes 2.0, 0.42, 0.074, 0.02, 0.005, 0.001)

On applying these equations to the soils used in this project and the soil used by Robertson [9] it was found that the equations predicted the optimum moisture

Table 10.2: Compaction Characteristics of Stabilized Soils - Standard Proctor *Compactive Effort*

	Maximum Dry Density (tons/m^3)	Corresponding Moisture Content (%)
Soil-cement		
Granular	1.6 to 2.2	21 to 8
Fine-grained	1.4 to 2.0	28 to 10
Soil-lime	1.3 to 1.8	32 to 15

content to within 1% and the maximum dry density to within 30 kg/m^3. The predicted moisture contents were consistently less than the actual moisture contents while the predicted dry densities were greater than the actual dry densities.

Since the preliminary analysis is approximate, it is not necessary at this stage to allow for the influence of the stabilising agent on the compaction characteristics.

Strength Parameters

Approximate UCS and shear strength parameters are listed in Table 10.3 while stress-strain parameters are listed in Table 10.4.

As might be expected, the soils which give higher strengths when stabilized with cement are those with a larger proportion of coarse material and with good compaction characteristics. Because of the higher density and lower proportion of fines, such materials require less cement to bond particles at points of contact, and will thus achieve higher strengths than poorly graded less dense materials at the same cement content.

The strengths of soil-lime mixtures are not as predictable as soil-cement mixtures,

Table 10.3: Strength Parameters

	Soil-Cement		Soil-Lime
	Granular	Fine-grained	
28 day UCS (kPa)	500 to 1000C	200 to 600C	50 to 150L
	C = cement content (%)		L = lime content
UCS at age d (days)	$UCS_d = UCS_{d0} + klog_{10}(\frac{d}{d0})$		
	k = 500C	k = 70C	k = 80L
Shear Strength			
c'	0.22UCS	0.23UCS	0.27UCS
ϕ'	40 to 45^0	30 to 40^0	25 to 35^0

since the development of cohesive strength is primarily dependent on the lime reactivity of the clay, and not all clays are lime reactive.

The Eades and Grim 'Quick Test' can be used to determine the minimum lime content required to satisfy ion-exchange and flocculation reactions. Based on this result, UCS tests at lime contents above the lime fixation capacity can be used to determine whether the soil is lime reactive, i.e. whether lime-clay pozzolanic reactions are capable of occurring.

The Transportation Research Board [25] proposes the use of accelerated cures (curing at an elevated temperature (48.9^0C) for 48 hours) to establish whether the soil is lime reactive. However, the criteria adopted for acceptable strengths are based on the strength requirements for base and sub-base pavement layers. The strength requirements (e.g. UCS greater than 0.7MPa at 28 days for a base layer) are significantly less than the strengths (UCS from 2 to 3MPa) required for soil-lime to be used economically as a construction material in dams.

The use of UCS tests after accelerated curing at elevated temperatures therefore needs to be investigated further to correlate the test with the strengths of soil-lime mixtures cured under field conditions for long periods i.e. 1 to 2 years.

Table 10.4: Stress-Strain Parameters

	Soil-Cement		Soil-Lime
	Granular	Fine-grained	
Compressive Modulus (GPa)	5 to 35	1 to 7	0.1 to 0.5
Failure Strain (%)	0.5	0.7	1 to 5

For both soil-cement and soil-lime, the compressive modulus increases with both stabilizer content and confining stress. For soil-cement, the failure strain is approximately constant at all cement contents and at all levels of confining stress.

For soil-lime, the failure strain increases with lime content and is inversely proportional to the confining stress.

Other Design Parameters

Other design parameters such as permeability, durability and shrinkage are more specific to the later phases of a design. For the detailed design, the designer will obtain accurate design parameters from in-depth laboratory investigations into all the pertinent material properties.

At this stage, should the material not meet one or more of the design requirements, the designer is able to zone the dam e.g. to satisfy the applicable criteria for seepage or durability, while controls can be specified to limit shrinkage to acceptable levels during construction.

The designer should however allow for increased costs, should it be suspected during the preliminary appraisal that such zoning might be required.

Chapter 11

CONCLUSIONS

The objective of this research project was to investigate the engineering properties of marginal soils stabilized with cement or lime to determine the feasibility of using these materials for embankment dam construction. The major benefit would be that cost savings could be achieved through shortening construction time and through using readily available soils close to the site of the dam wall.

From the results of the tests conducted, it can be concluded that both lime and cement stabilization enhance the engineering properties of marginal soils to the extent that the resulting materials can indubitably be used as construction materials for large embankment dams.

The addition of the stabilizing agent results in a significant increase in the shear strength of the original soil. The addition of cement or lime results in increases in both the cohesive strength and in the angle of shearing resistance. For soil-lime, increases in the cohesive strength are slow and depend on the type of clay present.

Although it is generally recommended that cement stabilization is preferable for soils with a plasticity index of less than 10%, it was found that soils with higher plasticity indices could be successfully stabilized with cement. The addition of 10% cement by dry mass to a soil with a plasticity index of 18% yielded a material with a long term strength of 4 MPa. This strength was obtained after the specimens had been subjected to percolation and soaking for a period of 512 days after an initial 28 day curing period.

It is also possible to stabilize highly plastic soils with cement, although the simultaneous addition of 2% lime by dry mass is required to facilitate mixing.

The addition of lime to a clay soil results in an almost immediate reduction in the plasticity of the soil, while the lime/clay pozzolanic reactions result in long term increases in cohesive strength. Lime can thus be used to expedite construction by reducing the plasticity of clay soils (modification), to increase the strength of clay soils (stabilization), or to modify clay soils prior to cement stabilization (preconditioning).

Unconfined compressive strength tests showed that the strength of soil-lime increased significantly during an extended period of leaching and soaking (512 days), indicating that the lime/clay reaction products are definitely permanent. It seems possible that the calcium saturated percolating water contributes to gains in strength by stabilizing the interior of clay lumps which would otherwise be weaker than the cementitious products formed on the outsides of the lumps.

Both lime and cement reduce the shrinkage of soils significantly. The addition of 7.5% cement reduced the maximum shrinkage strain of a highly plastic soil from 5% to 2.8%, while the addition of 7.5% lime to the same soil reduced the shrinkage strain to 2.4%.

The addition of a stabilizer also reduces the rate at which the soil shrinks. It is possible to control the shrinkage to a certain extent by replacing lost moisture. Such control can be utilized to ensure that no large cracks across the width of the dam (which could later become leakage paths) are developed.

Although the addition of either lime or cement may result in an increase in the permeability relative to the unstabilized soil, the permeabilities of the stabilized soils still fall within the range of permeabilities acceptable in impervious cores. Thus an homogeneous stabilized soil dam can be constructed without incorporating seepage control elements.

Gravity and slope stability analyses show that the strengths resulting from the addition of 7.5% by dry mass of either lime or cement to the best of the marginal soils tested were adequate to construct an homogeneous stabilized soil embankment dam 41 meters in height with equal upstream and downstream slopes of 65^0. Since similar long term strengths could be obtained from a highly plastic clay (PI = 52.1%) stabilized with between 7.5 and 10% lime, it is conceivable that a similar structure could be constructed by stabilizing soils which would previously have been rejected as spoil.

At a stabilizer content of 7.5% by dry mass, the total in place material cost is similar to that required for an earth embankment dam of the same height, although it is envisaged that significant cost savings will emanate from the use of an integral spillway and from the reduction in construction time. The cost

of the material in the stabilized soil dam is significantly less than the estimated material cost for a concrete buttress dam.

At this stage, the laboratory studies show that stabilized soil is theoretically a feasible alternative construction material. However, there is a need to investigate the behaviour of the material under field conditions. Small scale trial embankments would allow the various ideas as to possible dam configurations to be tested, as well as highlight the practical difficulties which might be encountered when constructing a stabilized soil dam. Monitoring of the behaviour of the trial structures would provide valuable information which cannot be gleaned from the laboratory tests.

This will become possible only if dam designers view such materials as 'alternatives' rather than 'competitors'. If seen as a competitor, stabilized soil can be easily criticized out of context, and hence disregarded. However the dam designer is obliged to consider as many options available to him as possible, not only to find the solution which best satisfies safety and economic criteria, but also to utilize modern technology and natural resources in an aesthetically pleasing and environmentally protective manner.

In the light of the findings of this research project, stabilized soil appears to be a promising alternative dam building material. The variety of materials which can be utilized to yield a diverse range of products, considerably augments the number of design options open to the dam designer.

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Appendix A

CLAY MINERALS

A.1 The Structure of Clay Minerals

The clay minerals commonly found in soils are alumino-silicates. The structures of the common layer silicates can be considered in terms of two simple structural units[1]. The silica tetrahedral unit (Figure A.1) has a silicon ion (Si^{4+}) tetrahedrally coordinated with four oxygen ions (O^{2-}). The octahedral unit (Figure A.2) has an aluminium cation (Al^{3+}) (or magnesium (Mg^{2+}) octahedrally coordinated with six hydroxyls ($(OH)^{-}$).

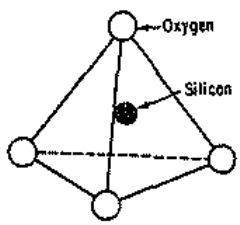


Figure A.1: Schematic Diagram of a Single Silica Tetrahedron

The different clay mineral groups are characterized by the stacking arrangements of sheets of these units, and the manner in which two successive two- or three-sheet layers are held together.

[1]The information in this section was extracted from references [28],[27] and [57]

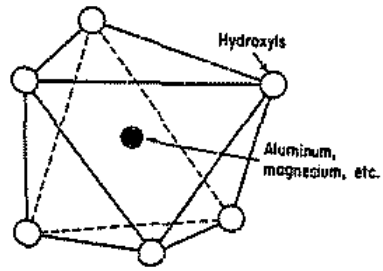


Figure A.2: Schematic Diagram of a Single Octahedral Unit

The silica tetrahedra form sheets where each of the three oxygens at the base of the tetrahedron are shared by two silicons of adjacent units, forming a hexagonal net (Figure A.3). The structure can repeat indefinitely and has the composition $(Si_4O_{10})^{4-}$.

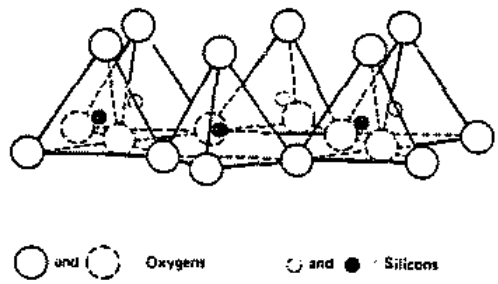


Figure A.3: Schematic Diagram of a Sheet of Silica Tetrahedra

Figure A.4 shows the sheet structure of aluminium or magnesium octahedral units. When the octahedrally coordinated cation is trivalent e.g. Al^{3+}, only two-thirds of the available spaces are normally filled (called a dioctahedral structure). If the octahedrally coordinated cation is divalent e.g. Mg^{2+}, all possible cation sites are normally filled (trioctahedral structure).

In the case of aluminium, the composition is $Al_2(OH)_6$ (the mineral gibbsite), while with magnesium the composition is $Mg_3(OH)_6$ (brucite). In clay mineral structures, aluminium and magnesium octahedral sheets are often referred to as gibbsite and brucite sheets respectively. The schematic representations for the

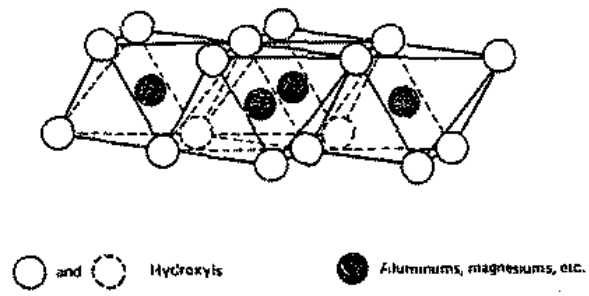


Figure A.4: Schematic Diagram of a Sheet of Octahedral Units

different structural units are shown in Figure A.5.

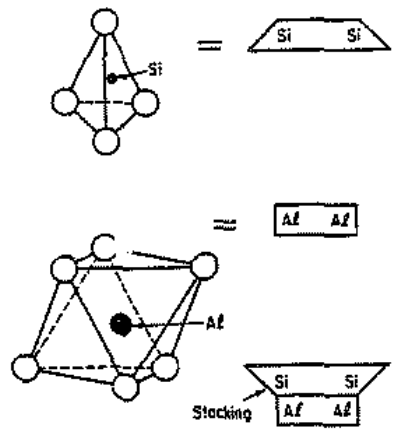


Figure A.5: Schematic Representation of Structural Units

In naturally occurring clay minerals, some of the tetrahedral and octahedral spaces are occupied by cations other than those in the 'ideal' structure, This isomurphous (same form) substitution occurs during initial formation of the mineral.

If a cation is replaced by another cation with a lower valence, e.g. Al^{3+} replaces Si^{4+} in the tetrahedral unit, then a net unit charge deficiency results for each substitution. Isomorphous substitution occurs in all clay minerals, giving clay

particles a net negative charge.

In nature, soil particles attract ions to neutralize their net charge. These ions are usually weakly held on the particle surface and can be readily replaced by other ions, hence they are called *exchangeable ions*. The quantity of exchangeable cations required to balance the charge deficiency of a clay is called the "cation exchange capacity" (*cec*) usually expressed as milliequivalents per 100 grams (me/100g) of *dry clay*.

A.1.1 Kaolinite

The basic kaolinite layer consist of a gibbsite sheet on top of a silica sheet with a shared layer of oxygen atoms between them (a 1:1 mineral). The kaolinite particle consists of stacked layers (Figure A.6) and has a structural formula $(OH)_8Si_4Al_4O_{10}$.

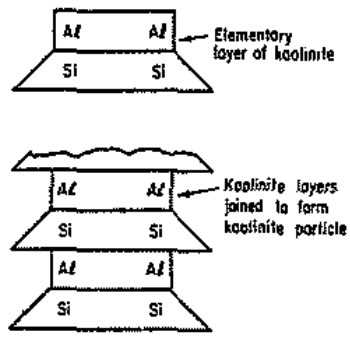


Figure A.6: Schematic Representation of Typical Kaolinite Structure

In kaolinites, bonding between successive layers is both by van der Waals forces and hydrogen bonds. This bonding is of *sufficient strength that layers will not* separate in the presence of water.

Kaolinites undergo little isomorphous substitution if any, but cation exchange capacities of 3 to 15 me/100g have been measured. Since interlayer hydration is prevented by strong interlayer bonding, balancing cations are adsorbed onto the exterior surfaces of the clay particles.

A.1.2 *Smectite*

Figure A.7 shows the typical 2:1 smectite structure, with each layer consisting of a *gibbsite sheet sandwiched* between two silica sheets (Figure A.7). The structural formula for smectite is $(OH)_4Si_8(Al_{3.34}\ \underset{\downarrow}{Mg_{.66}}\ )O_{20}{\cdot}nH_2O$.

$Na_{.66}$

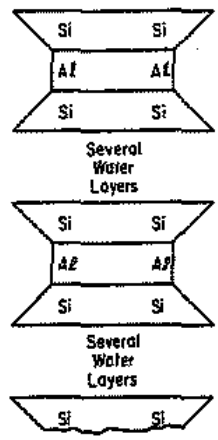


Figure A.7: Schematic Representation of Typical Smectite Structure

The term $\underset{\downarrow}{Mg_{.66}}$ indicates that the substitution of Mg^{2+} for Al^{3+} in the oc-
$Na_{.66}$

tahedral sheet is the source of charge deficiency, and that the sodium (Na^{2+}) is adsorbed to satisfy the charge deficiency. In this example, the clay would be called sodium smectite. Measured ion exchange capacities range from 80 to 150 me/100g).

The interlayer bonding in smectite is by van der Waals forces and by exchangeable cations that may be present to balance charge deficiencies in the structure. These bonds *are weak and can easily be separated by cleavage* or adsorption of water, and balancing cations take up positions between the unit cell layers as well as on the surfaces of the particles.

Figure A.8 compares the sizes, specific surface (surface area per unit mass) and

cation exchange capacities (the quantity of exchangeable cations required to balance the charge deficiency) of typical smectite and kaolinite particles.

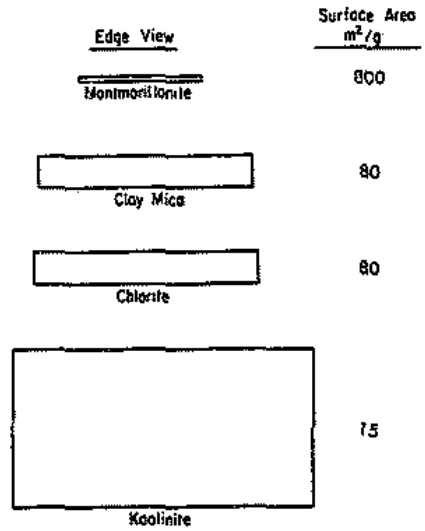


Figure A.8: Relative Sizes (Edge View) of Clay Particles

The properties of clay minerals are related to their particle size and their structure, for example smectite clays undergo large volume changes on wetting and *drying since cations* and water are easily adsorbed between layers. Smectite is said to have an 'expanding lattice structure' since the unit layers can be easily separated. On the other hand kaolinites are relatively water resistant since cations and hydration water are adsorbed onto the outside surface of the particle. Thus kaolinites do not swell or shrink very much.

The type of adsorbed cation also influences the properties for example, sodium *smectite swells and shrinks more than calcium smectite.*

Appendix B

MOISTURE DENSITY CURVES

B.1 Moisture Density Curves for Soils B and D

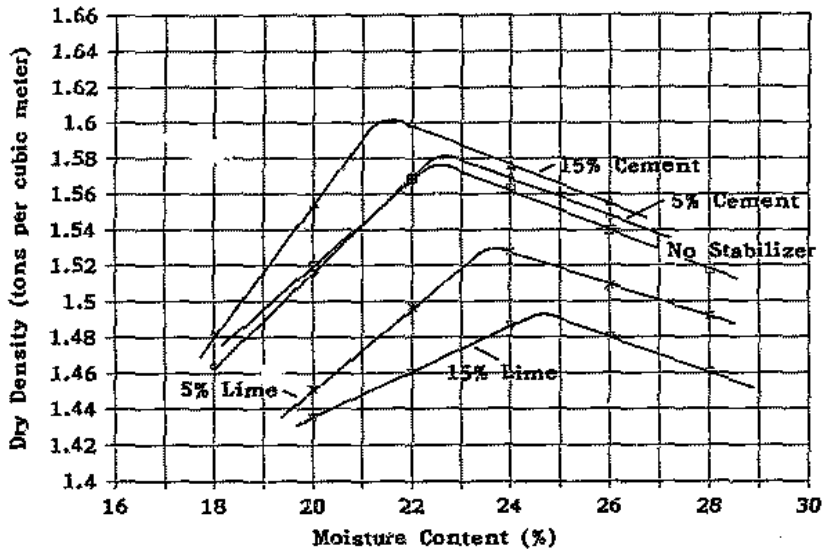


Figure B.1: Maximum Dry Density vs. Moisture Content - Soil B - Cement and Lime

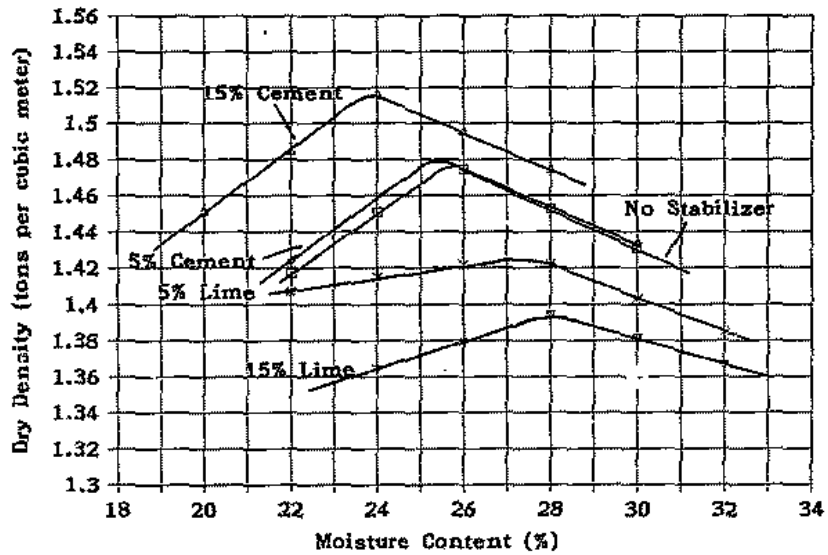


Figure B.2: Maximum Dry Density vs. Moisture Content - Soil D - Cement and Lime

Appendix C

STRENGTH TEST RESULTS

C.1 Triaxial Tests

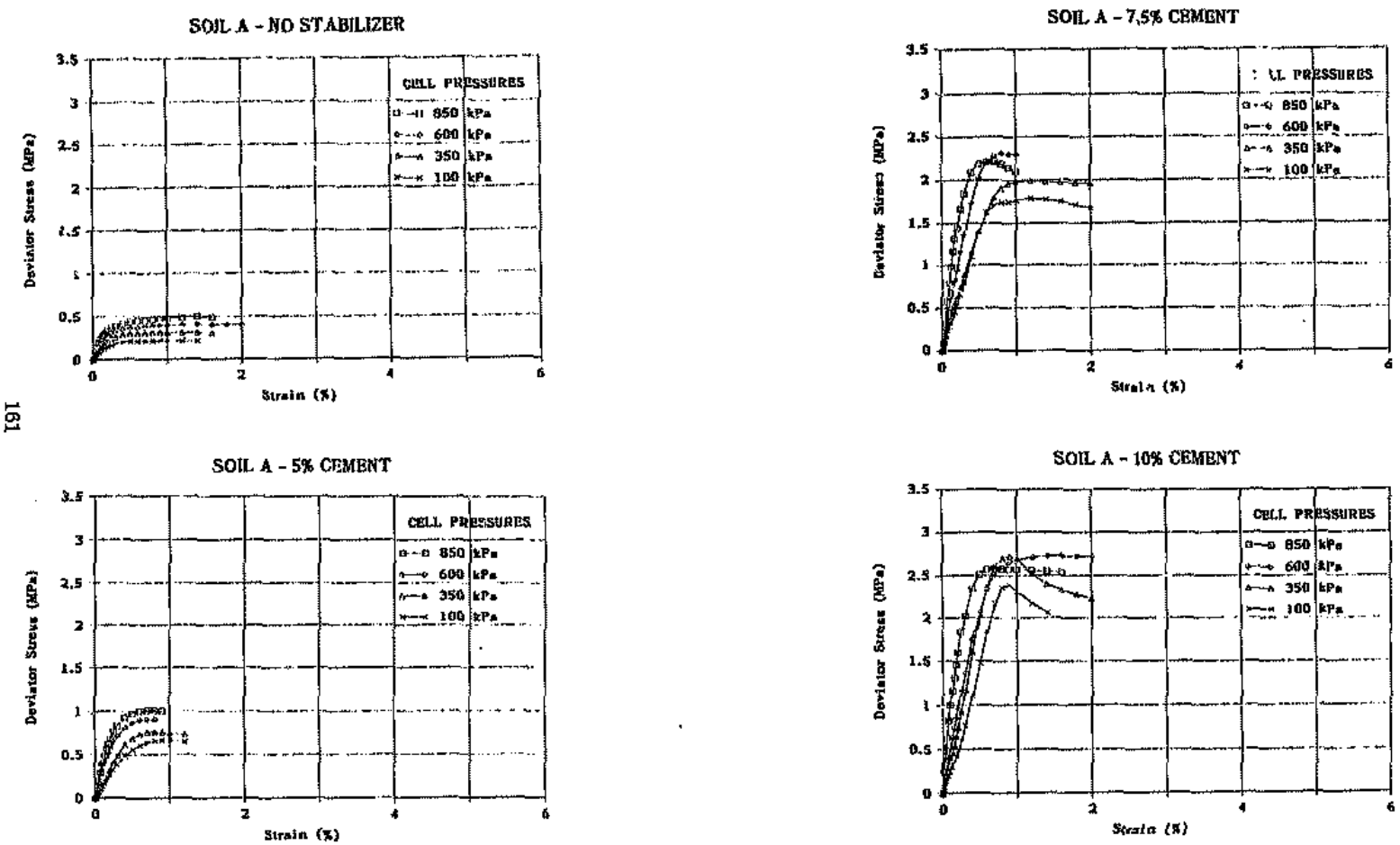


Figure C.1: Deviator Stress vs Strain - Soil A-cement

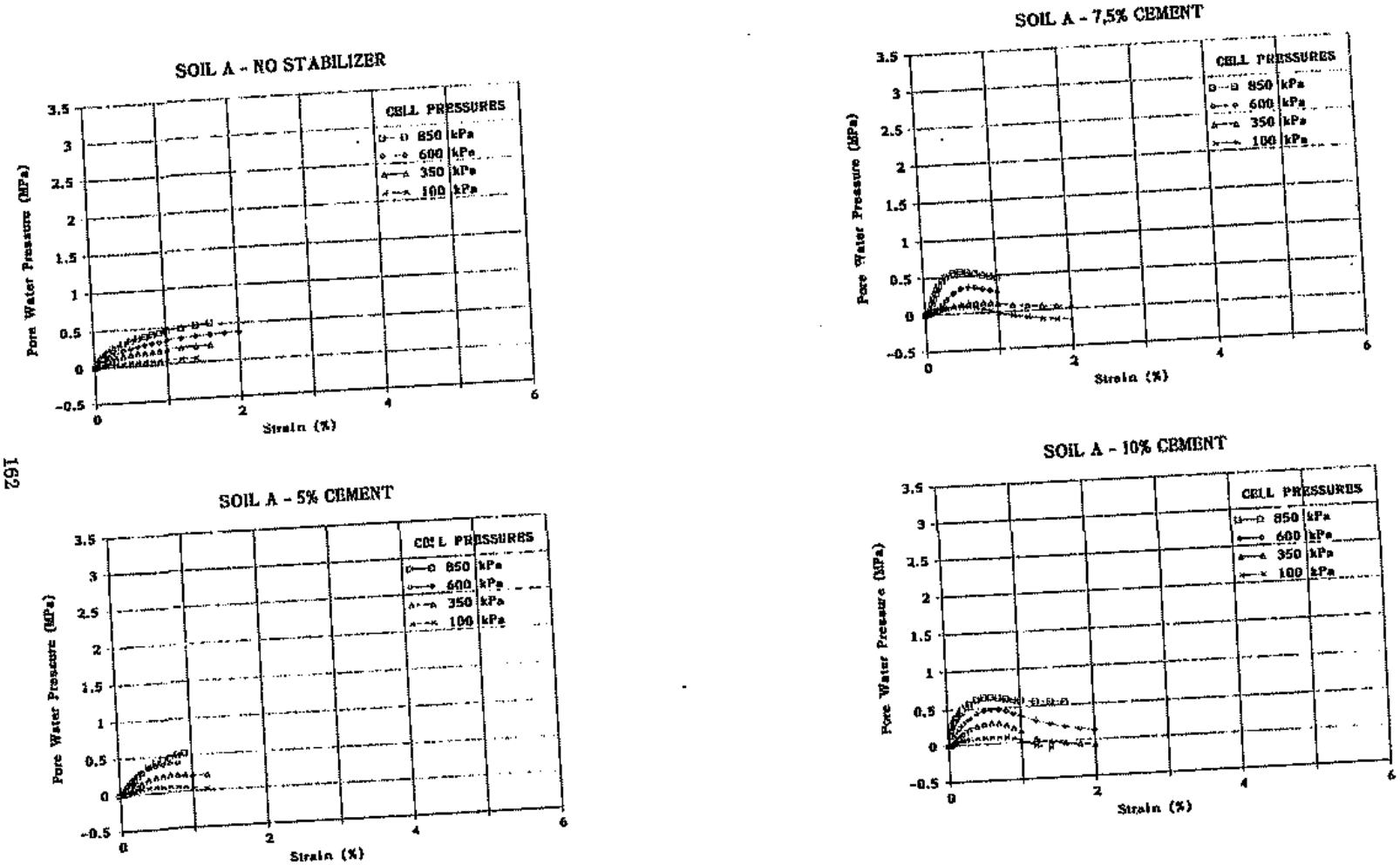


Figure C.2: Pore Water Pressure vs Strain - Soil A-cement

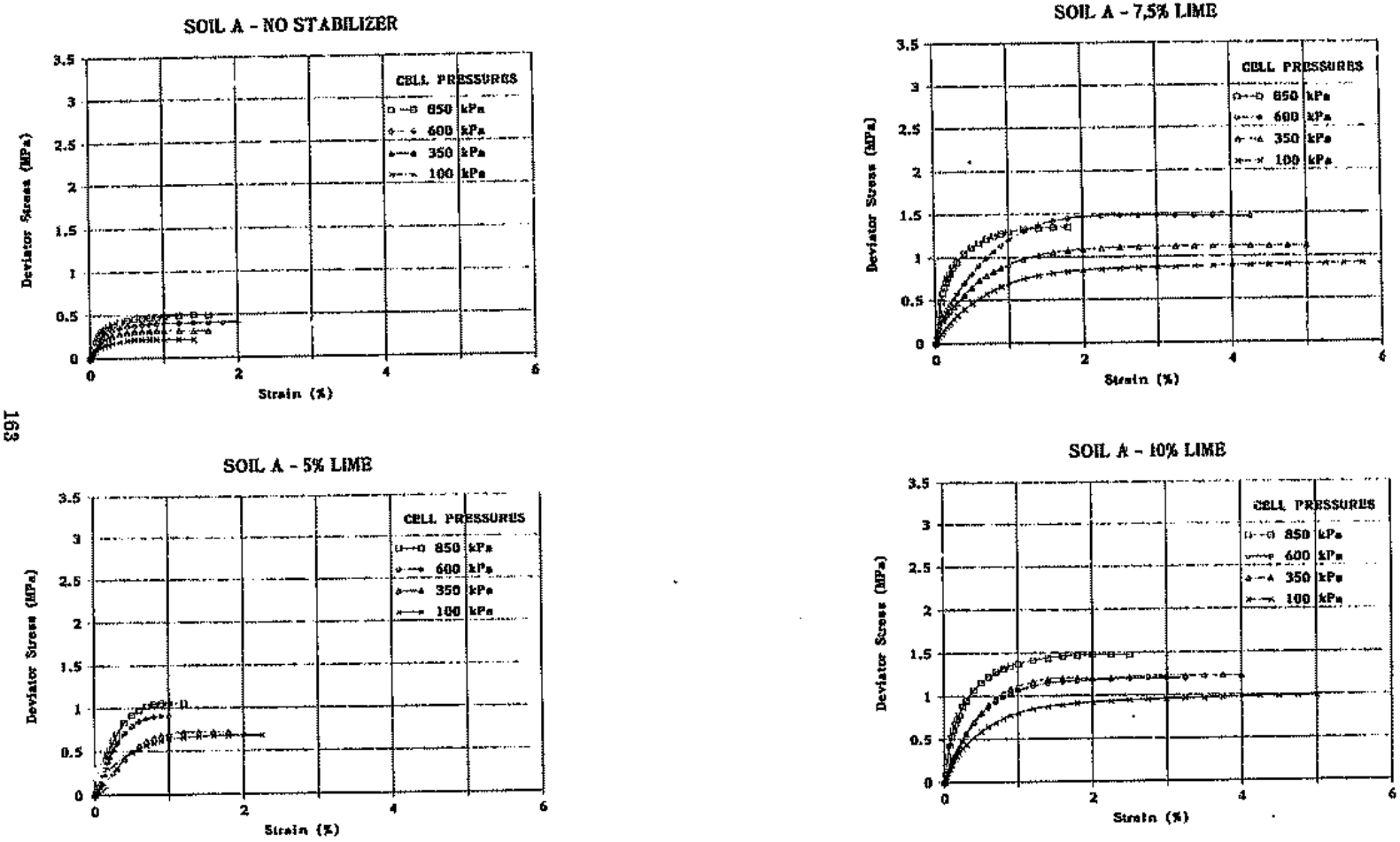


Figure C.3: Deviator Stress vs Strain - Soil A-lime

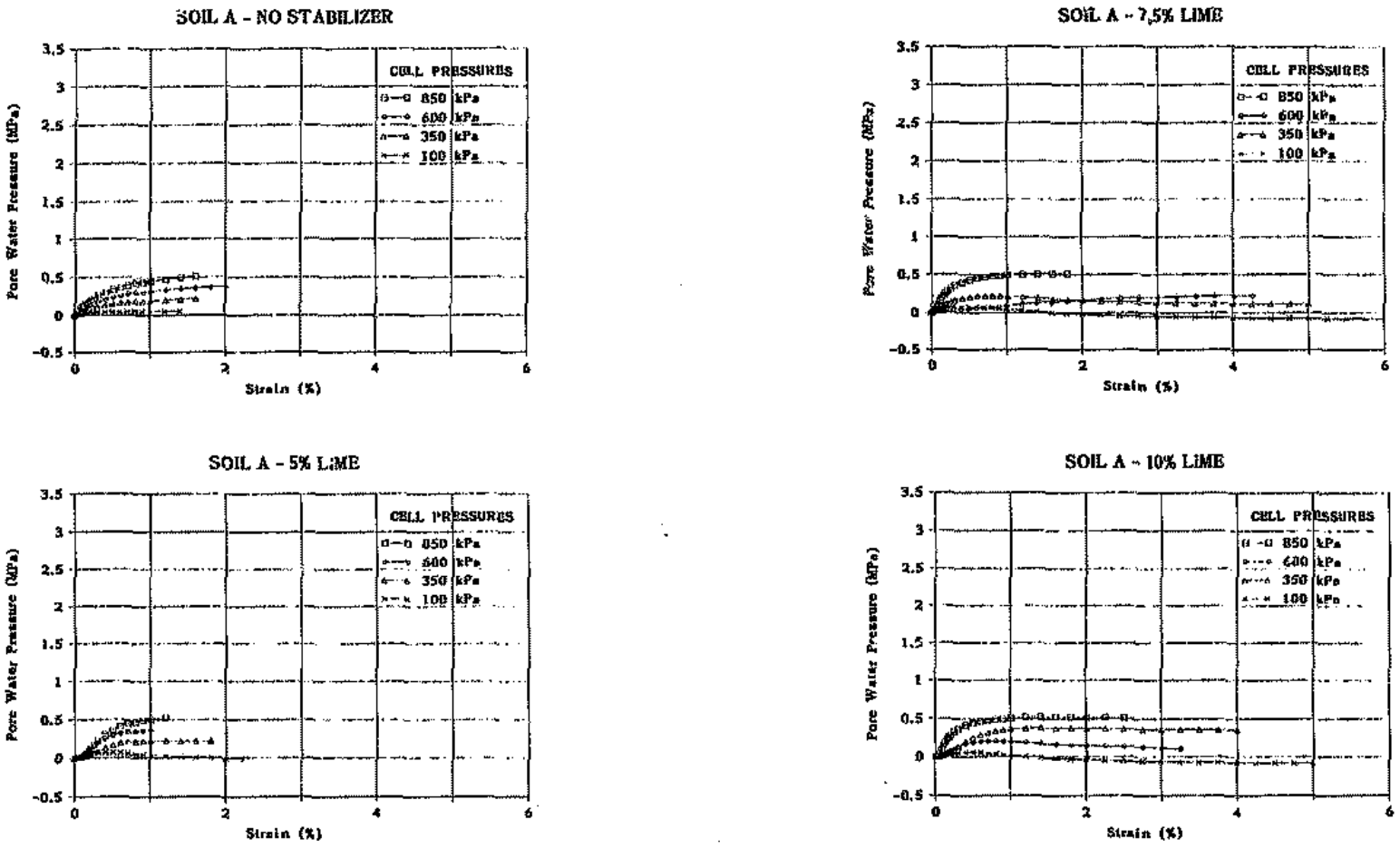


Figure C.4: Pore Water Pressure vs Strain - Soil A-lime

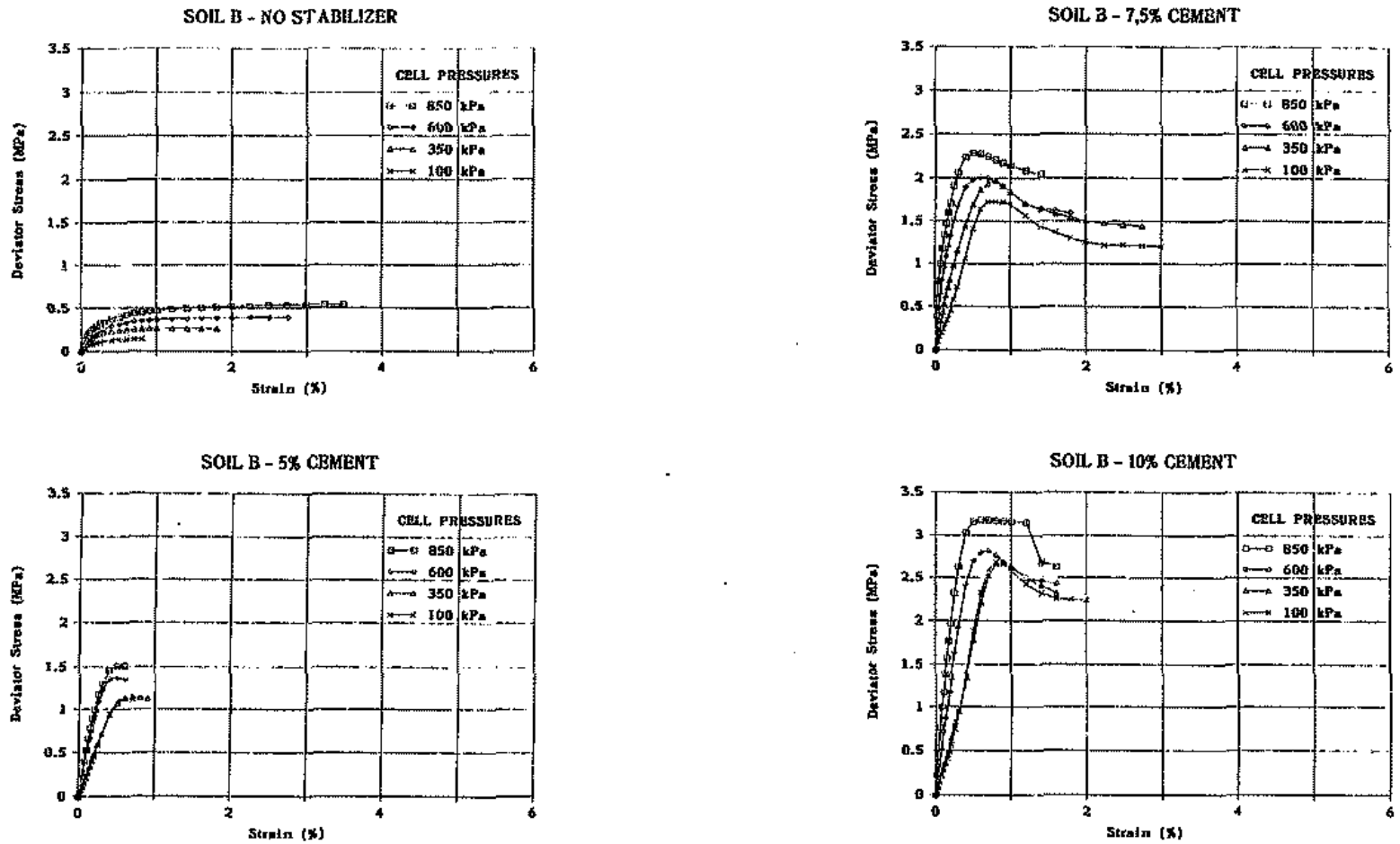


Figure C.5: Deviator Stress vs Strain - Soil B-cement

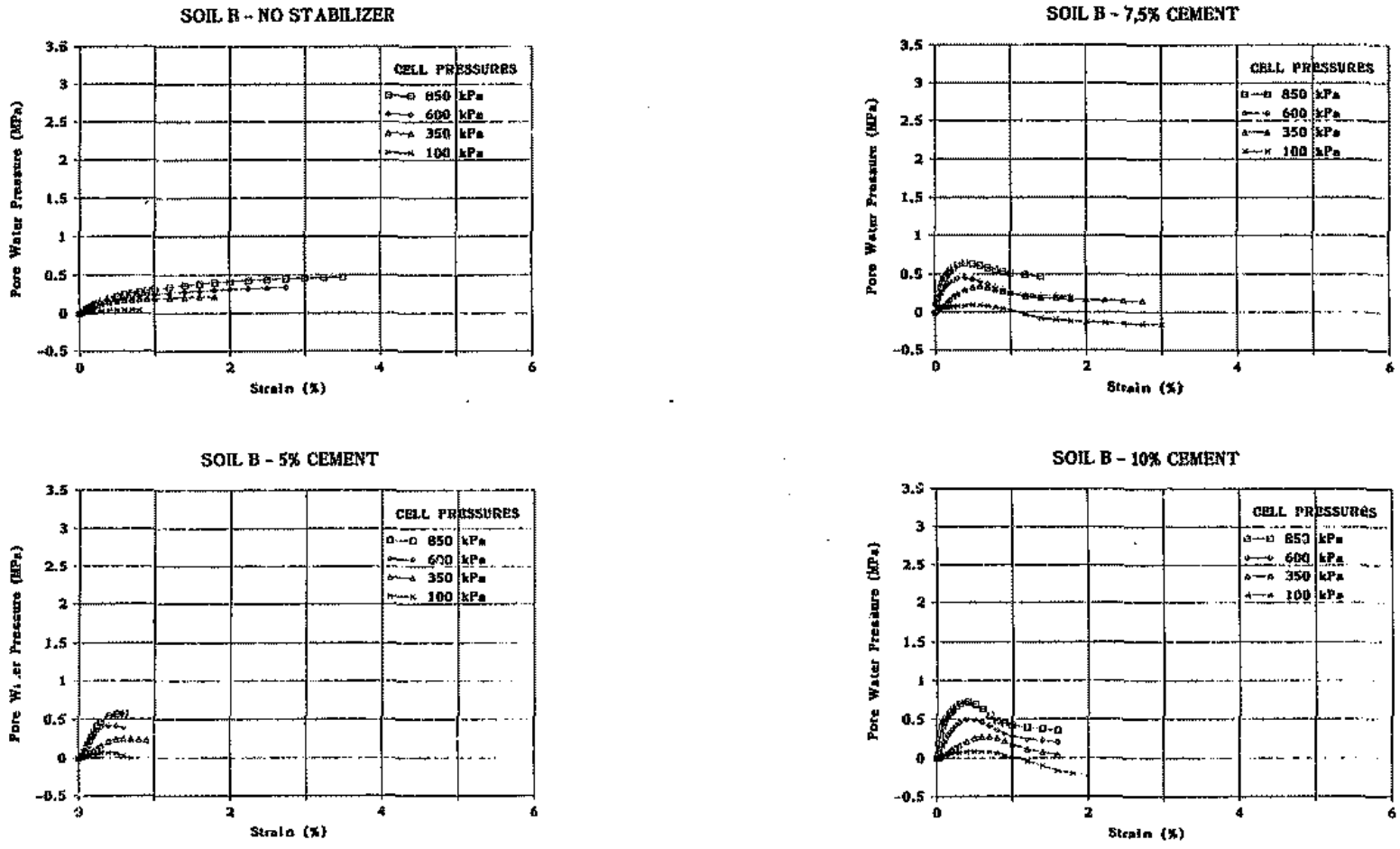


Figure C.6: Pore Water Pressure vs Strain - Soil B-cement

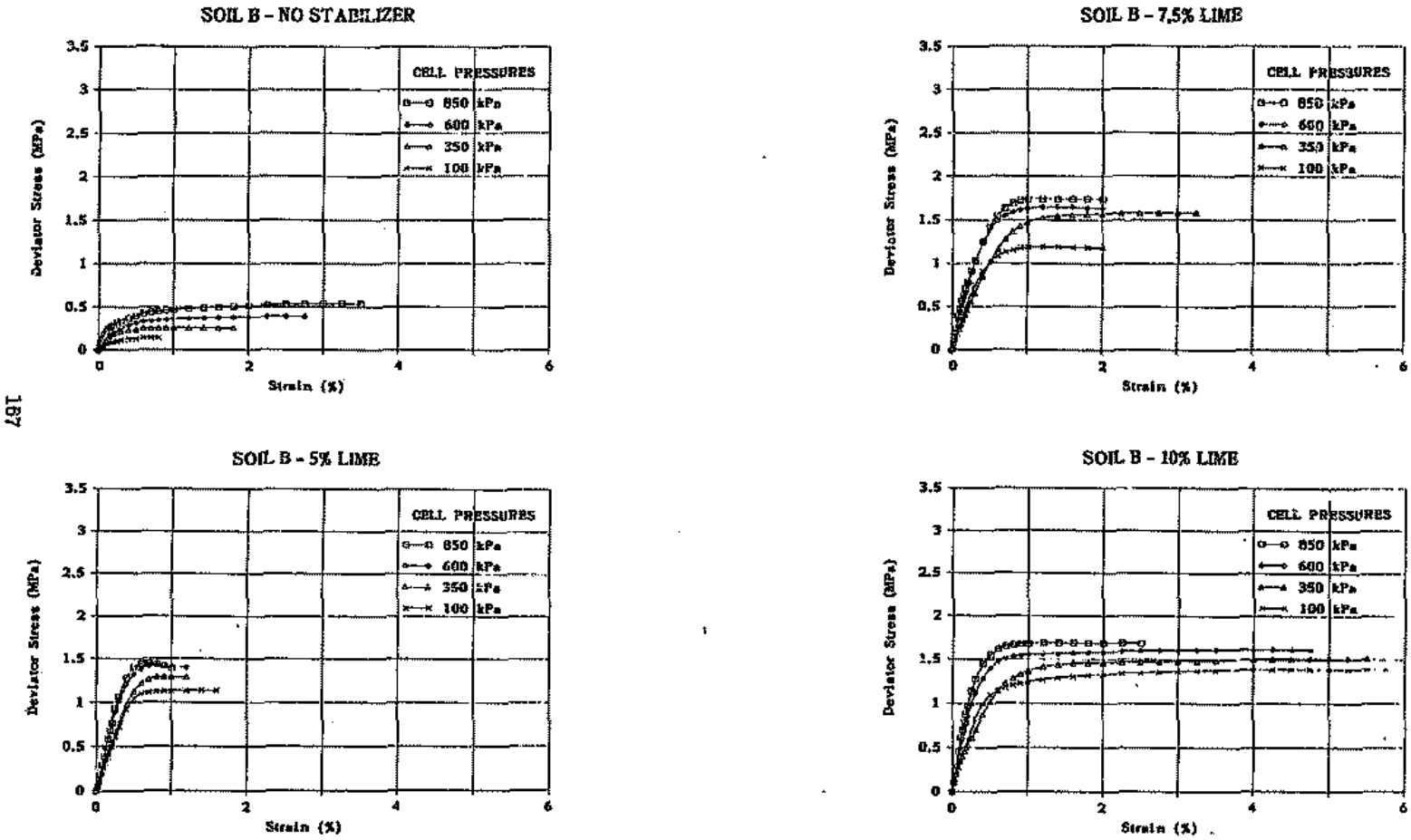


Figure C.7: Deviator Stress vs Strain - Soil B-lime

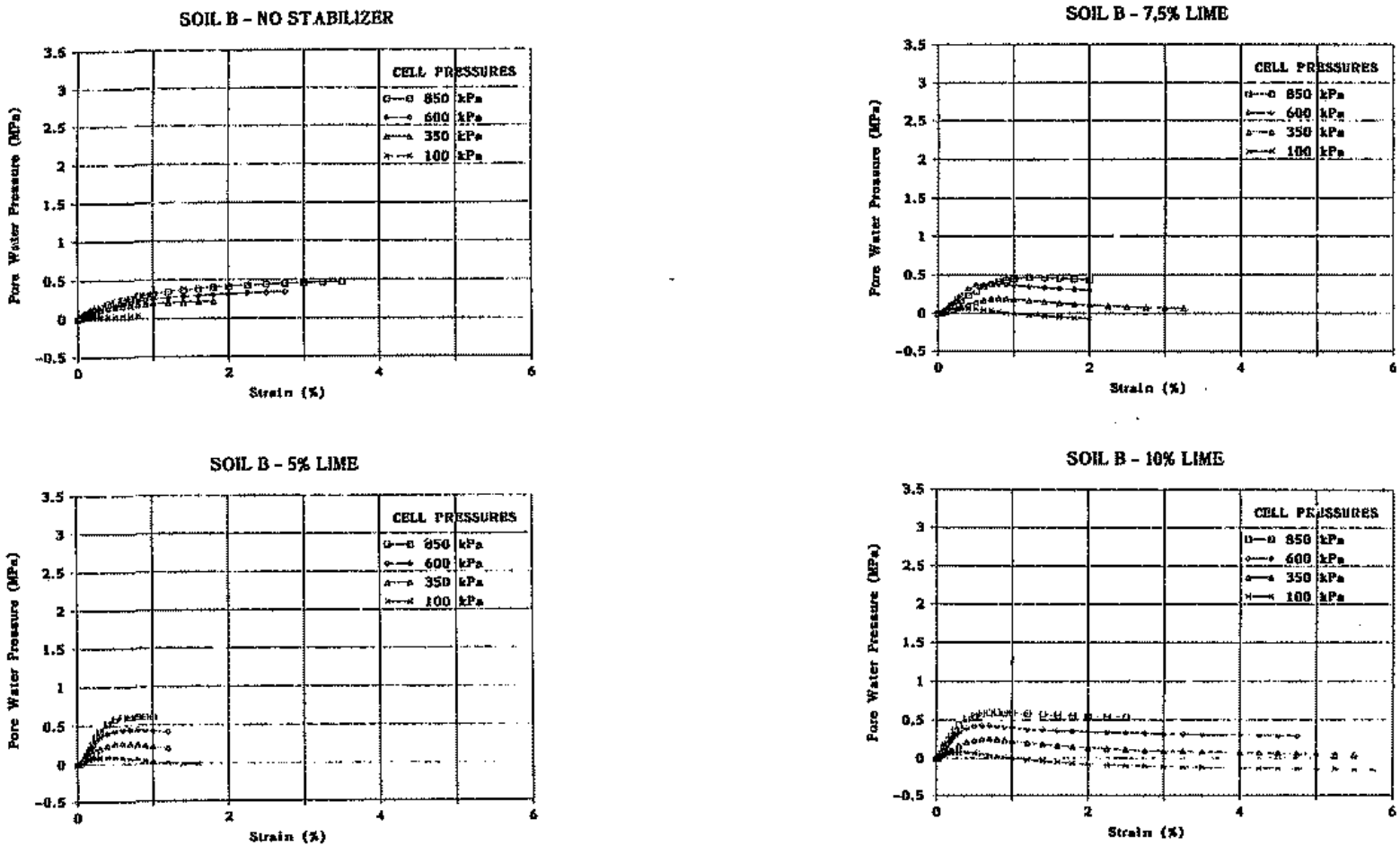


Figure C.8: Pore Water Pressure vs Strain - Soil B-lime

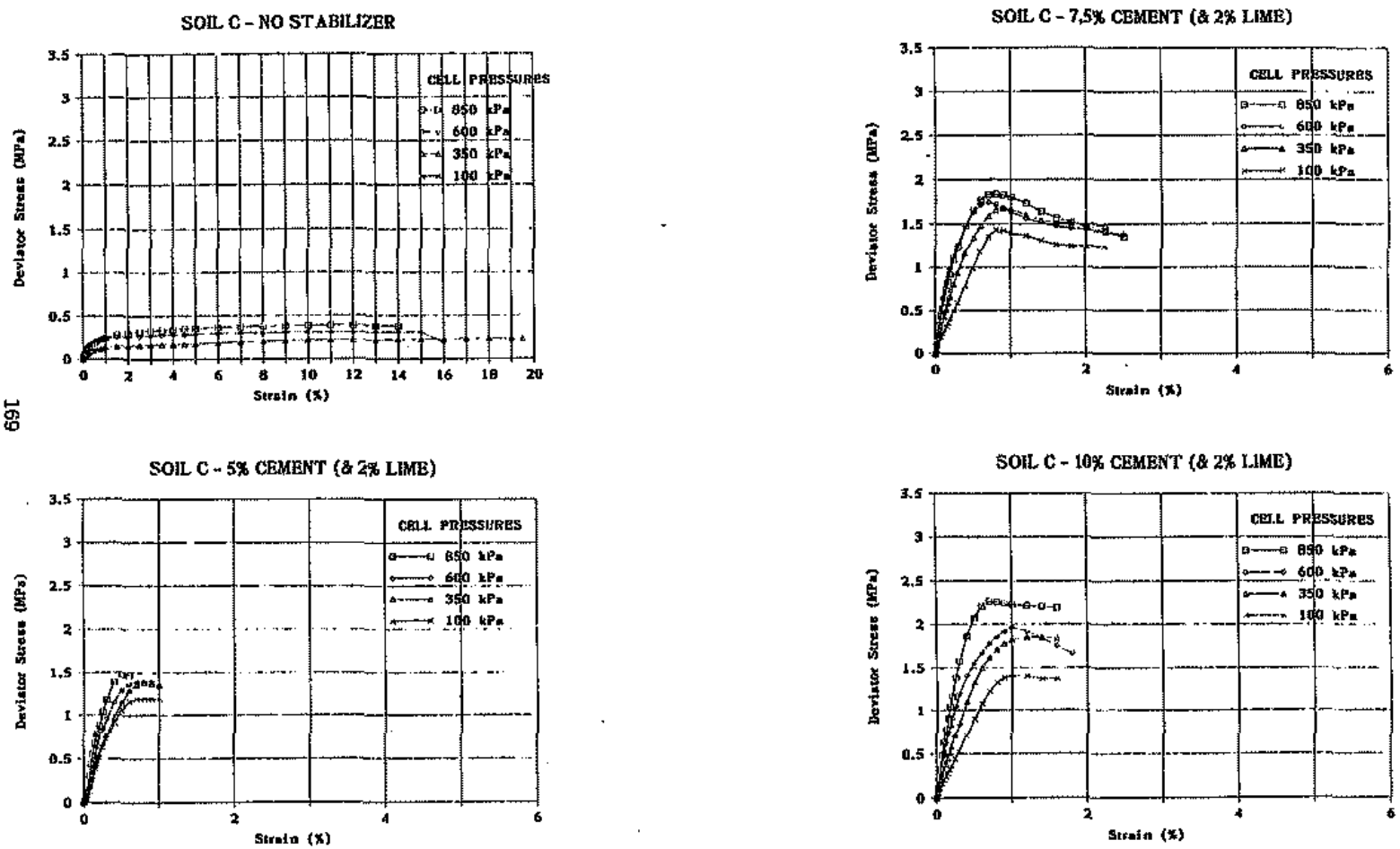


Figure C.9: Deviator Stress vs Strain - Soil C-cement

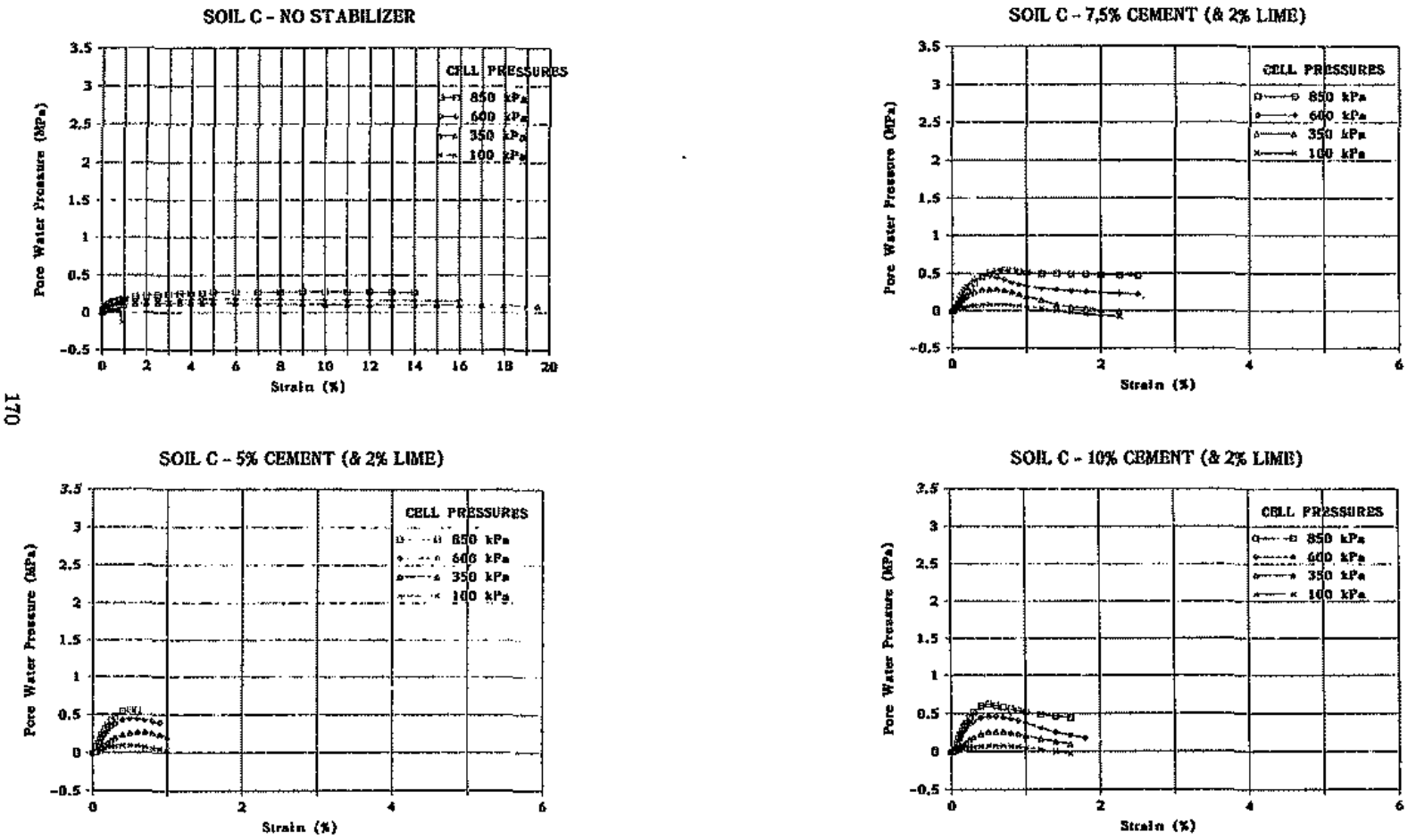


Figure C.10: Pore Water Pressure vs Strain - Soil C-cement

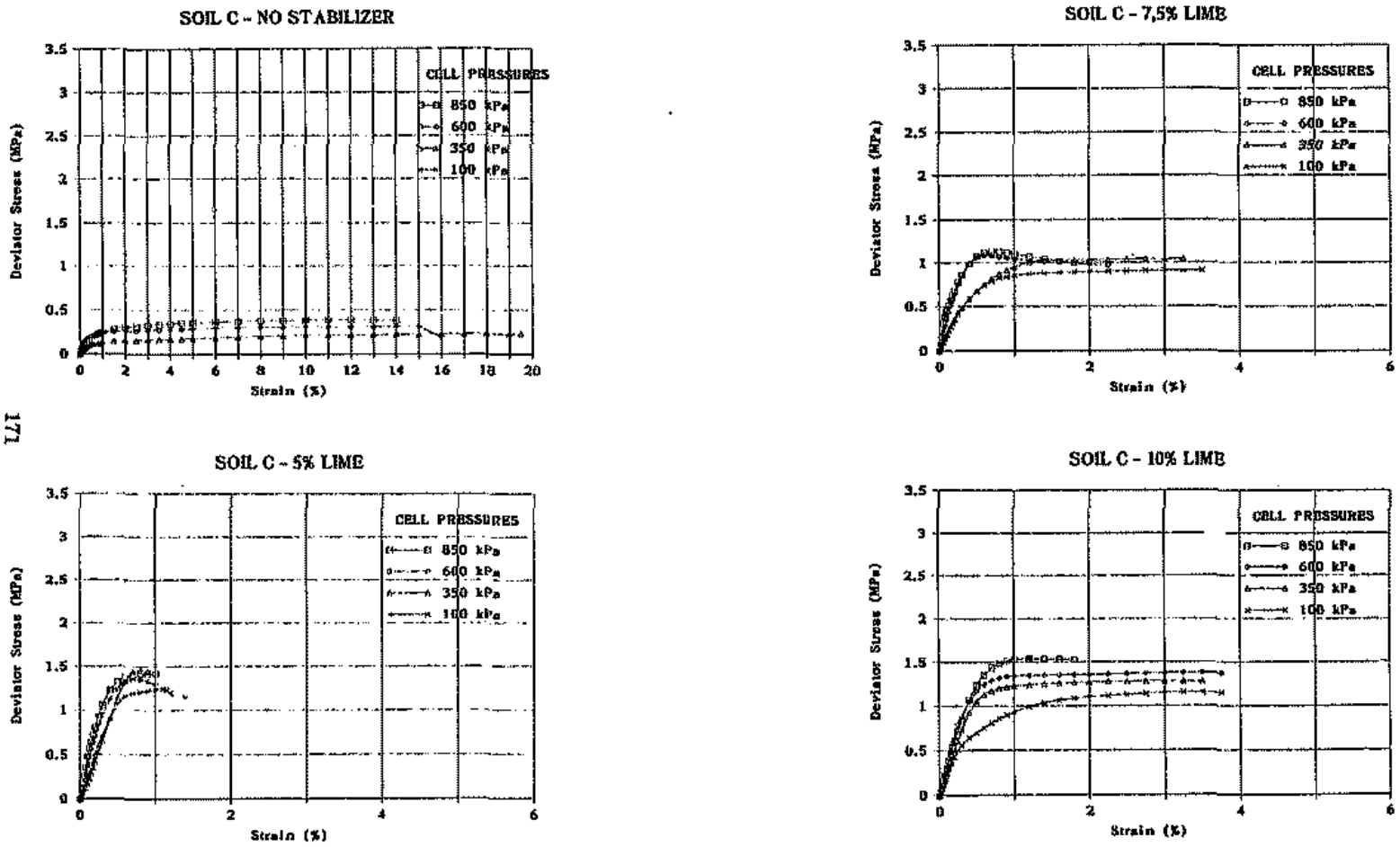


Figure C.11: Deviator Stress vs Strain - Soil C-lime

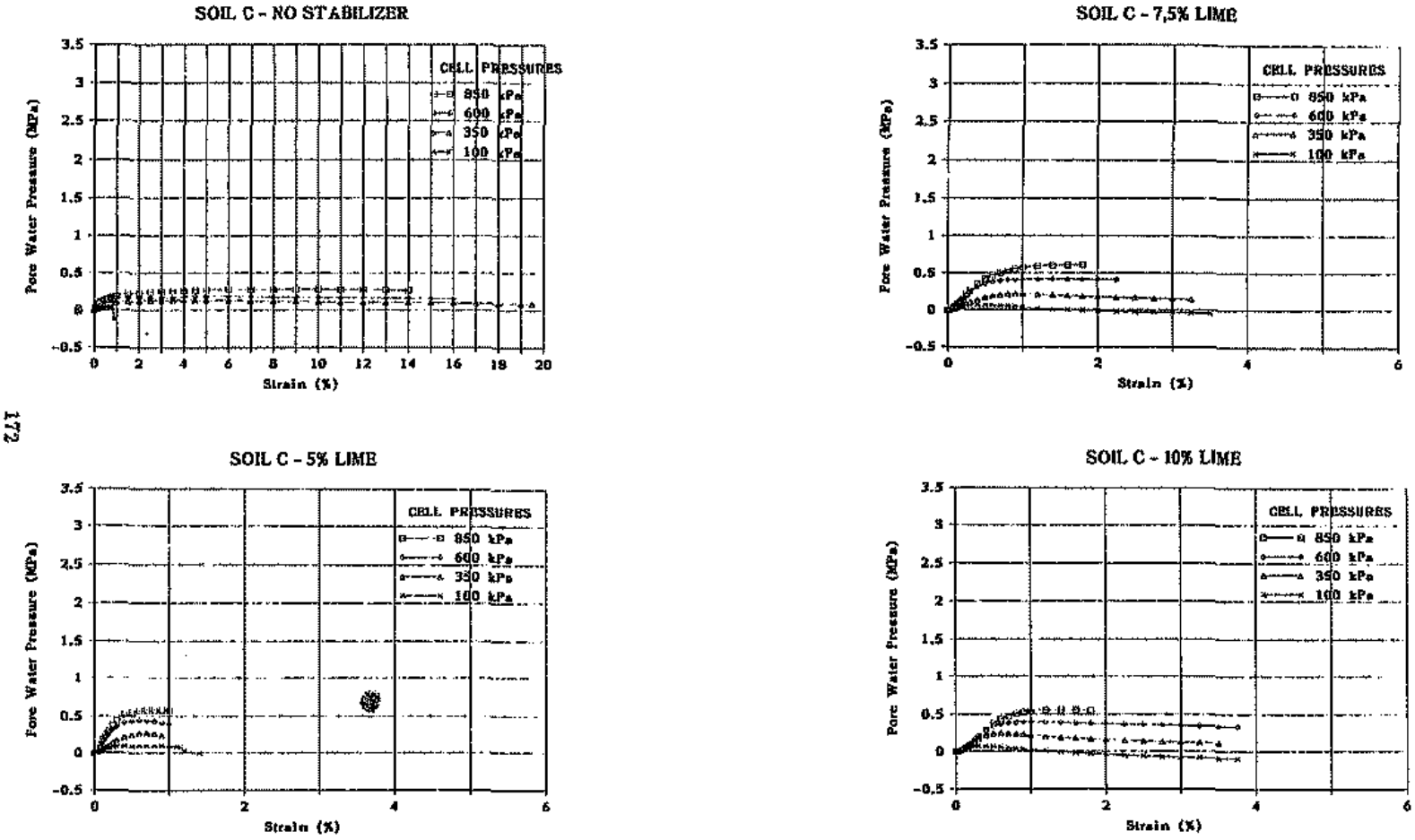


Figure C.12: Pore Water Pressure vs Strain - Soil C-lime

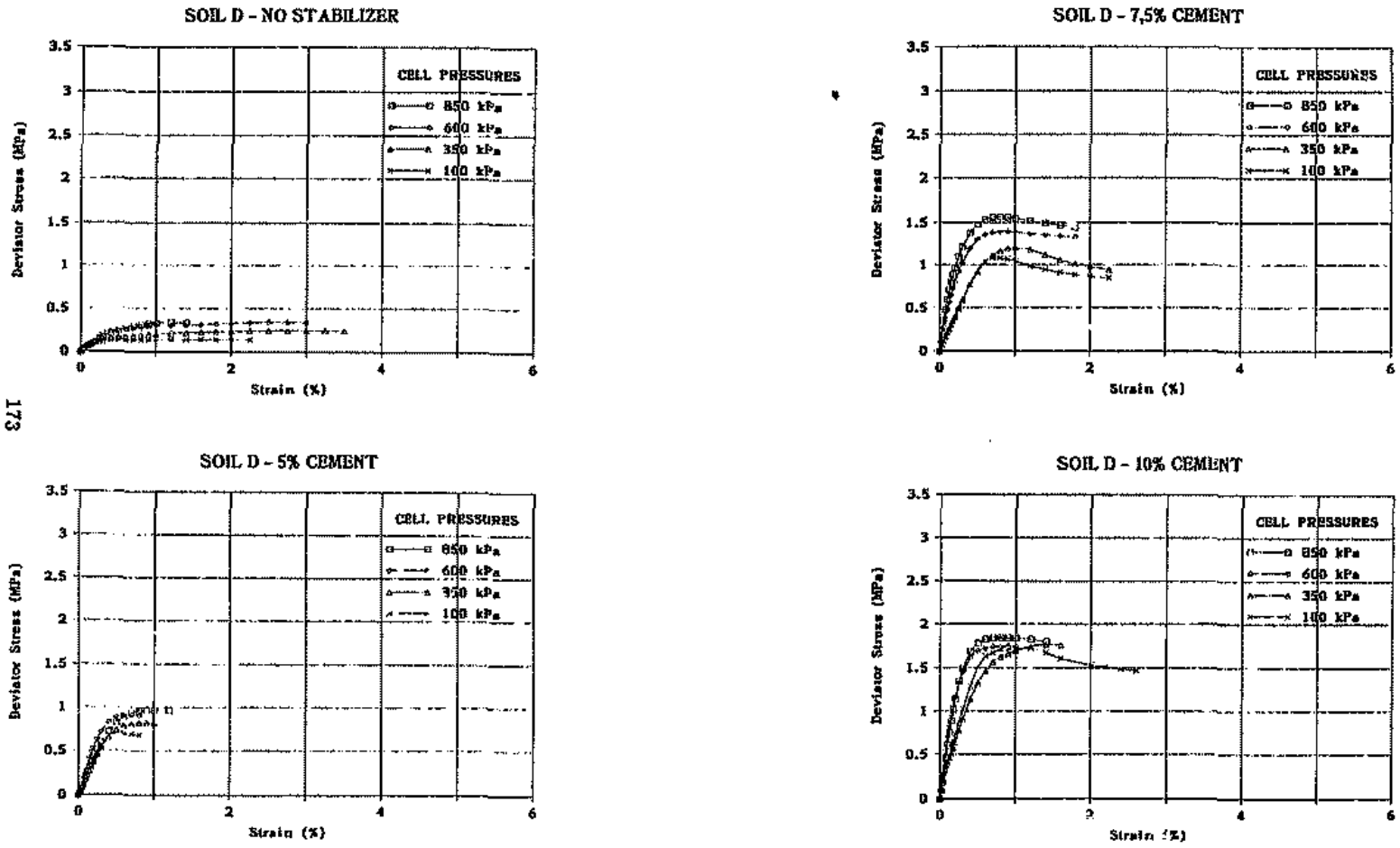


Figure C.13: Deviator Stress vs Strain - Soil D-cement

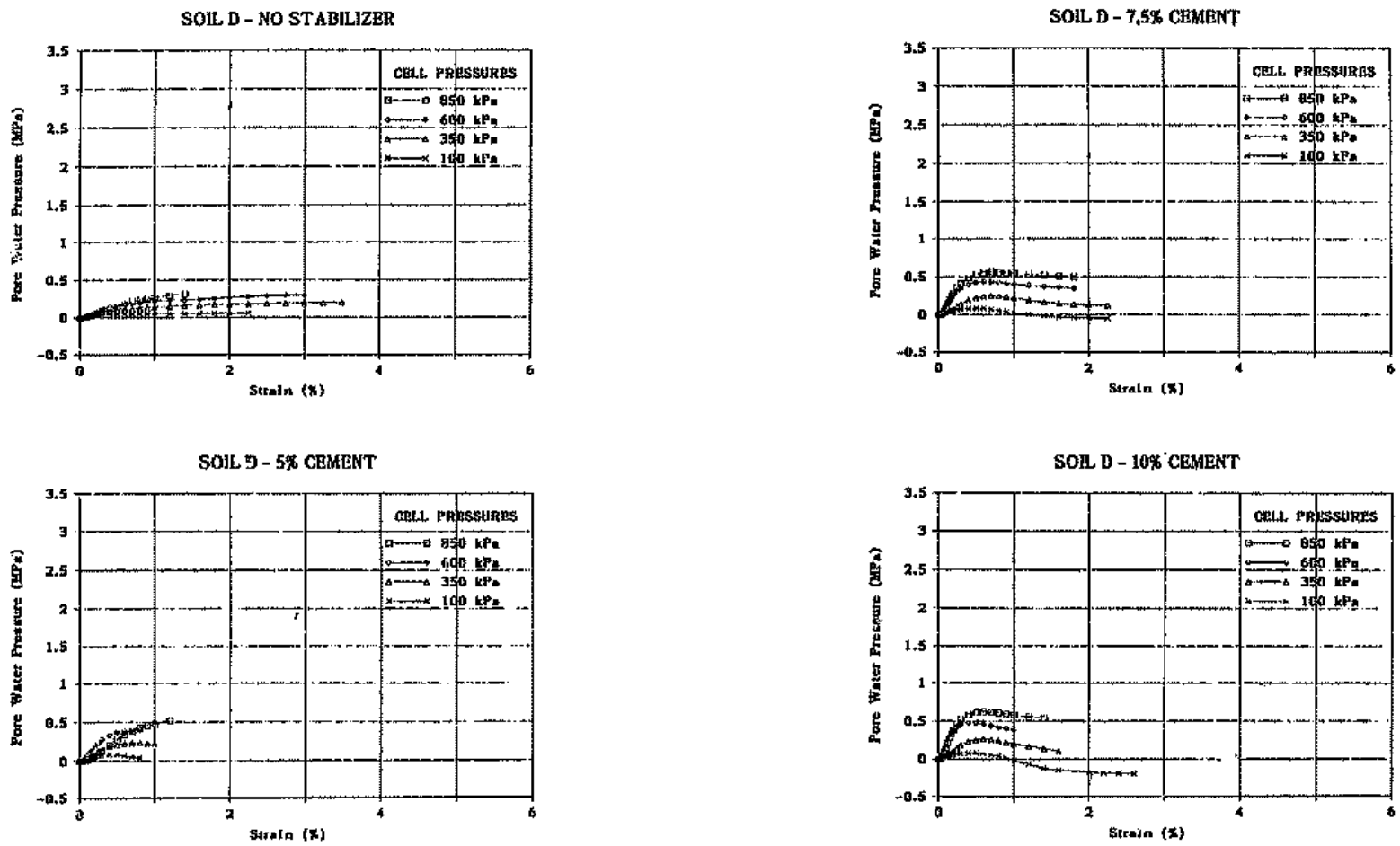


Figure C.14: Pore Water Pressure vs Strain - Soil D-cement

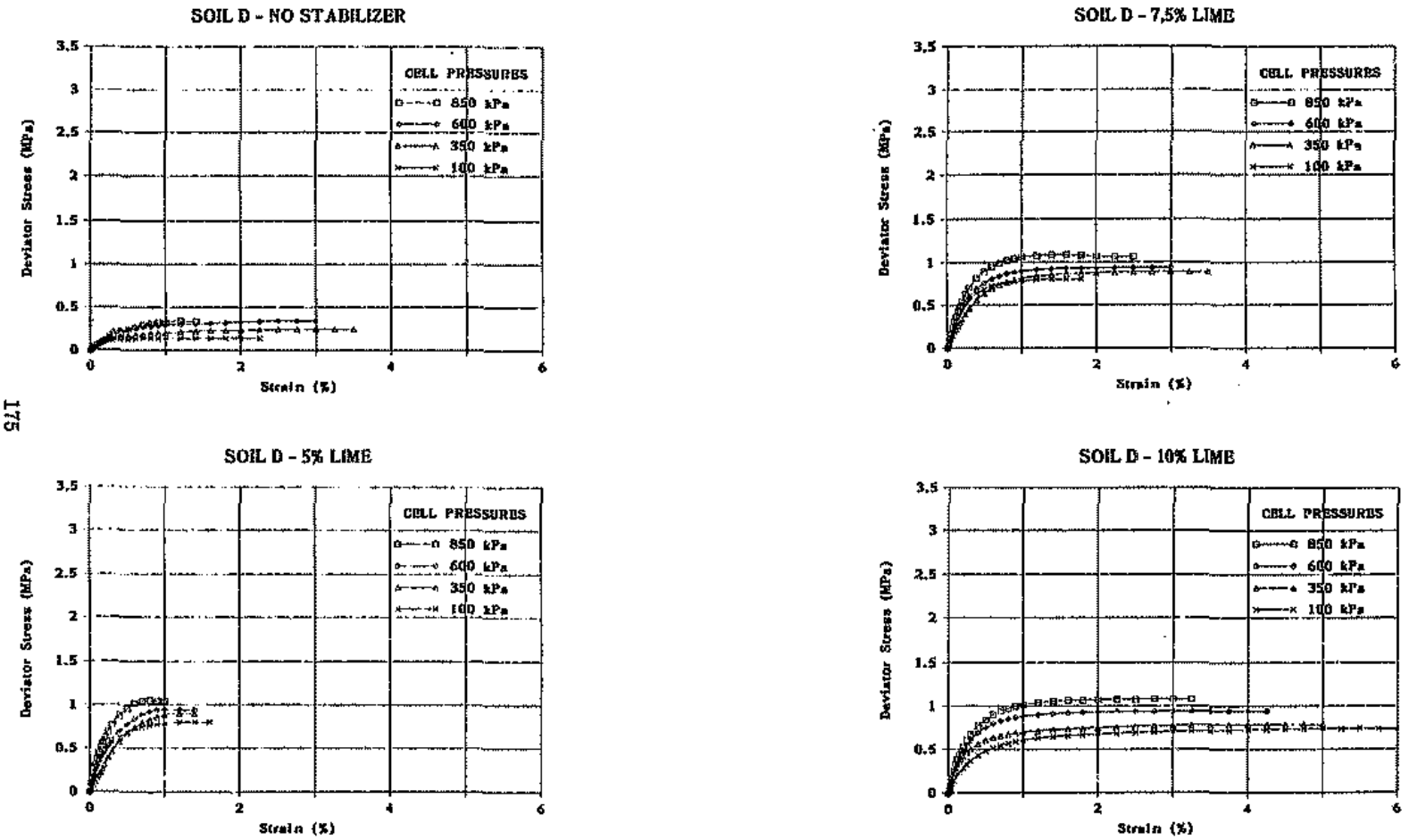
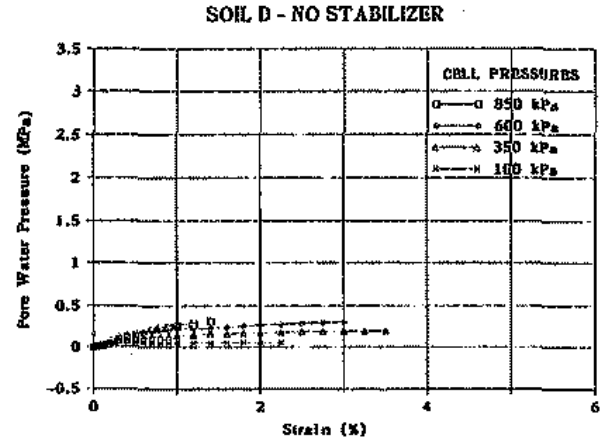
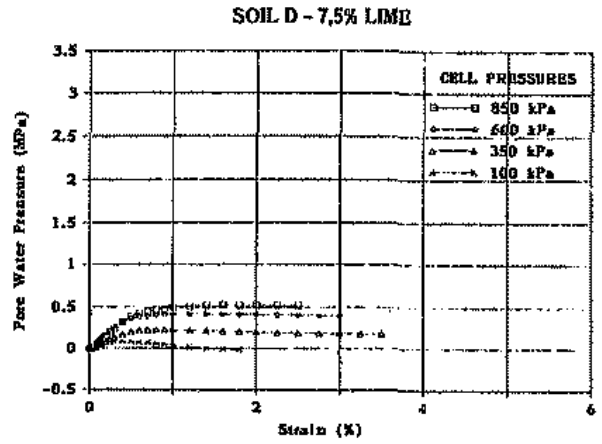
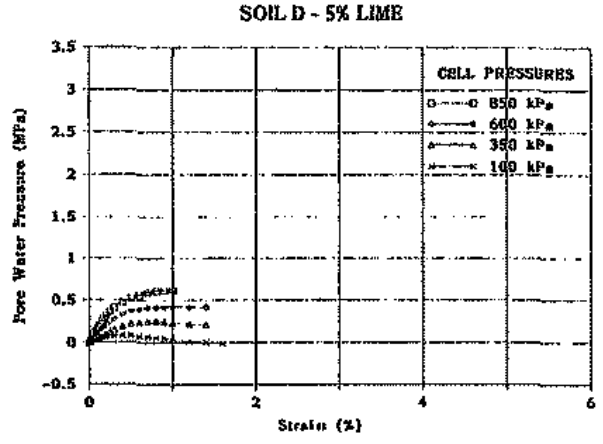


Figure C.15: Deviator Stress vs Strain - Soil D-lime







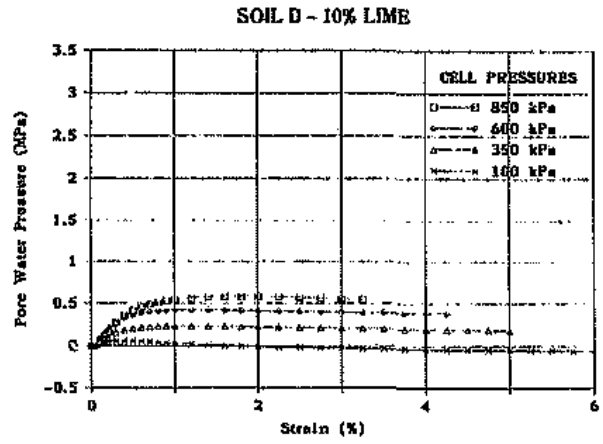


Figure C.16: Pore Water Pressure vs Strain - Soil D-lime

C.2 Flexure Test Results

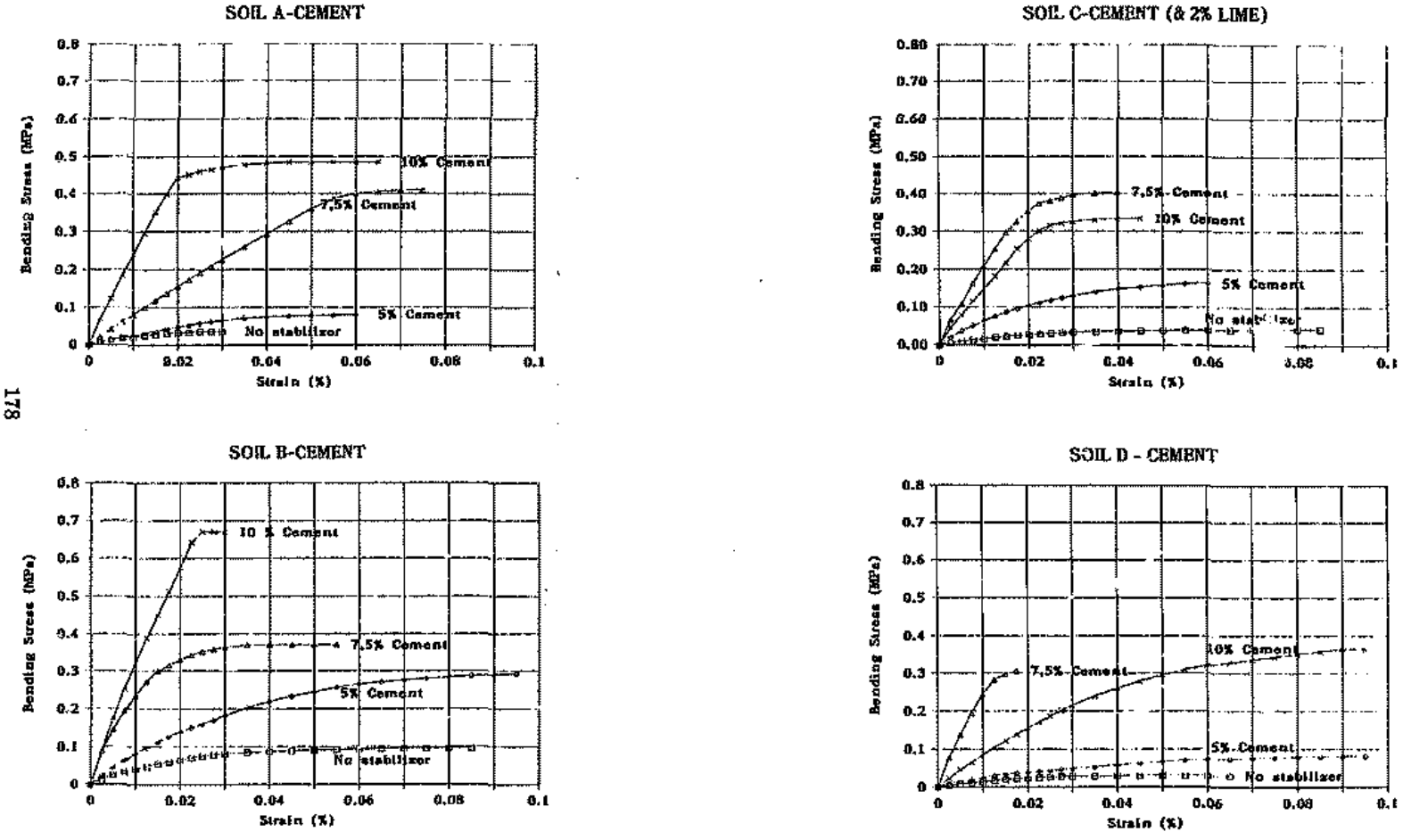


Figure C.17: Bending Stress vs Strain - cement

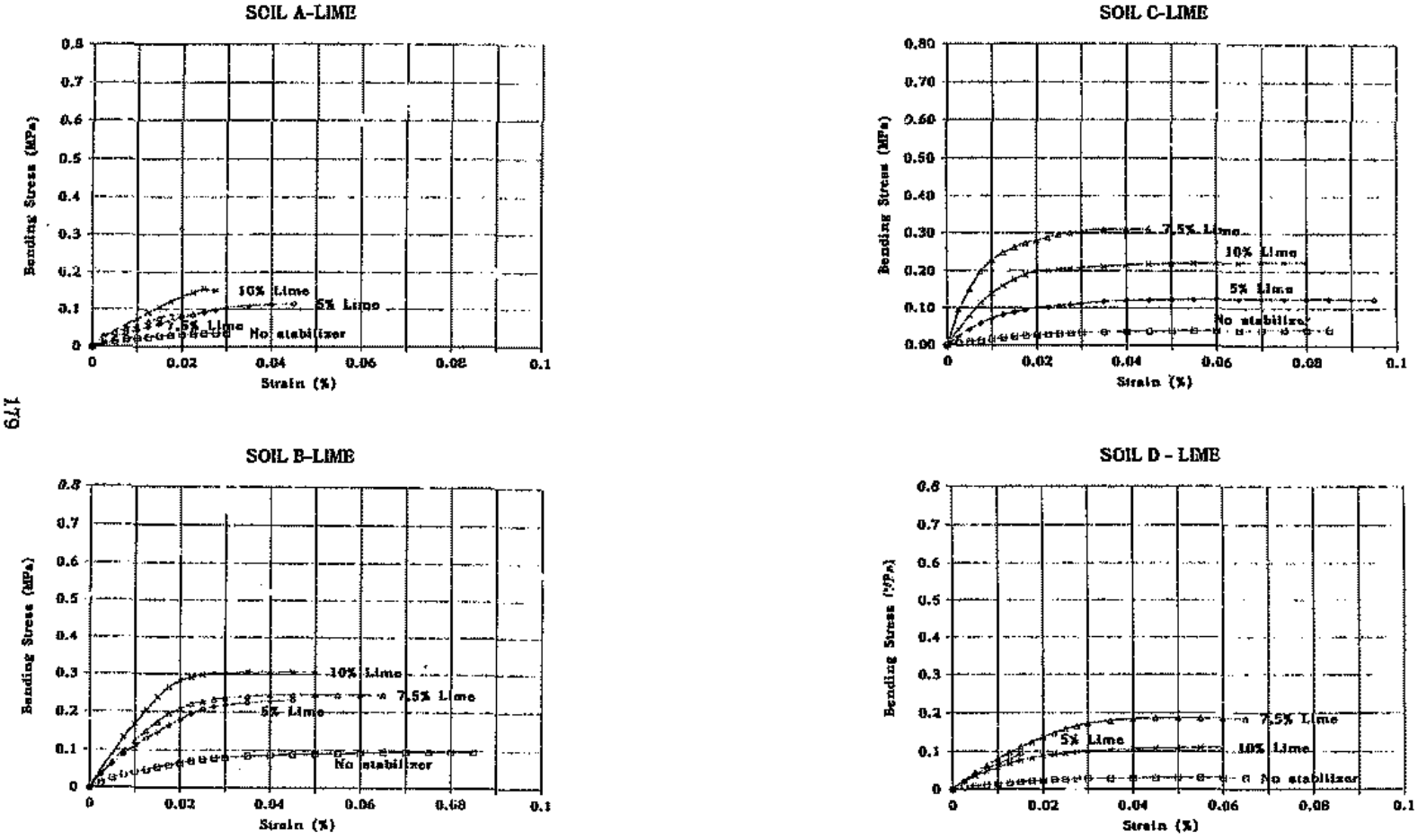


Figure C.18: Bending Stress vs Strain - lime

Appendix D

SHRINKAGE TEST RESULTS

D.1 Shrinkage vs. Time Curves

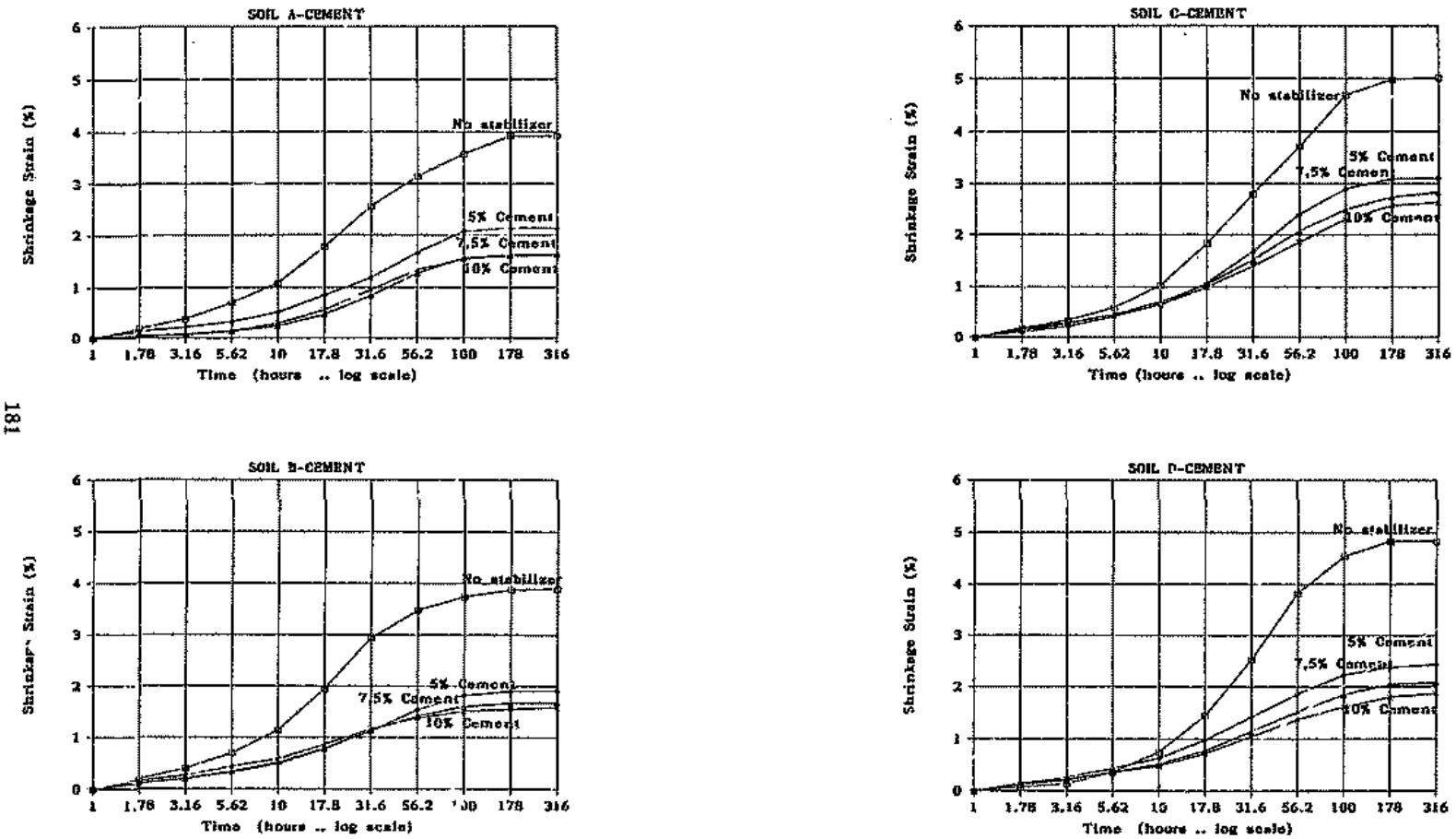


Figure D.1: Shrinkage versus Time - Cement

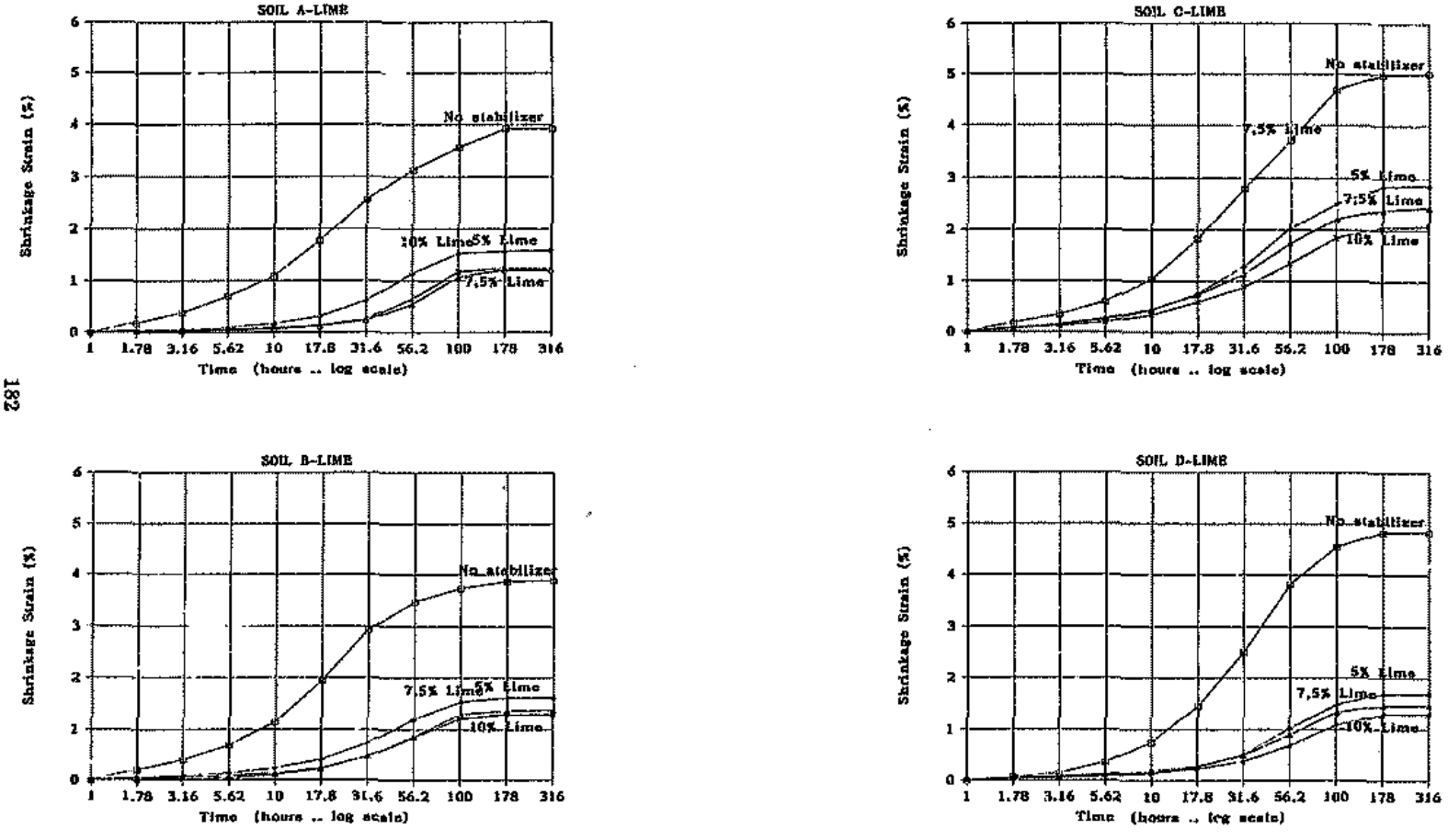


Figure D.2: Shrinkage versus Time - Lime

25×

25x10

Author Bentel Gary Michael
Name of thesis The Influence Of Soil Types And Stabilizing Agents On The Engineering Properties Of Stabilised Soil For Use In Embankment Dams. 1989

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