Quantitative criteria for the selection and stabilisation of soils for rammed earth wall construction

Author:
Burroughs, Van Stephan

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Quantitative criteria for the selection and stabilisation of soils for rammed earth wall construction

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Van Stephan Burroughs

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ABSTRACT

Modern building procedures and requirements demand that the selection and stabilisation of soils for the purposes of rammed earth construction be better quantified. This study examines the relationships between soil properties, stabiliser treatments, and stabilised strength and density for 111 soil samples taken from sites in New South Wales (Australia), and develops new quantitative criteria for soil assessment, selection, and stabilisation.

Laboratory measurements of soil particle size distribution, plasticity, and shrinkage were made for each soil. Various quantities from 0-6 % of lime, cement, and asphalt were added to the soil samples, and the resulting 230 specimens were compacted, and cured for 28 days. Determinations were made of the optimum moisture content, maximum dry density, and compressive strength of the stabilised material. The samples showed stabilised strengths ranging from 1.0-5.4 MPa, with a mean of 2.62 MPa, and densities from 1.44-2.21 t/m^3, with a mean of 1.86 t/m^3.

The results show that over 90 % of the variation in stabilised strength and density of the samples is due to variation in soil properties, with differences in stabiliser type or stabiliser quantity being relatively minor. The most important soil properties explaining stabilised strength are linear shrinkage and plasticity index. These properties have been used to categorise the soils into three groups on the basis of their suitability for stabilisation as measured against a compressive strength criterion of 2 MPa. Favourable soils have shrinkages of < 7.1 % and plasticities of < 16 %, and 90 % of these samples passed the 2 MPa criterion. Satisfactory soils have shrinkages of 7.1-13.0 % and plasticities of 16-30 %, and 65 % of these samples had strengths in excess of 2 MPa. Unfavourable soils have shrinkages of > 13 % and plasticities of > 30 %, and only 10 % of these samples exceeded the 2 MPa value. Soils in the favourable and satisfactory categories can be further discriminated using textural information. On that basis, all soils classified as favourable, and those classified as satisfactory and which also have sand contents < 60 %, are recommended as being suitable for stabilisation. Soils not fulfilling these criteria are unlikely to be successfully stabilised and should be rejected.

These results stress the importance of selecting a soil favourably predisposed to stabilisation. Field techniques to search for such soils could be refined on the basis of the new soil criteria presented. Use of the criteria should also minimise unnecessary laboratory testing of the density and strength of soils that subsequently prove unsuitable for stabilisation. A flow chart is presented to guide practitioners through the different stages of soil testing, assessment, and rammed earth stabilisation.

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CHAPTER 1: INTRODUCTION

1.1 BROAD THEME OF STUDY

Soil stabilisation is the modification of soil properties by adding another material (e.g. a chemical or another soil) and/or compacting it to improve its physical properties for a specified purpose. This thesis investigates the stabilisation of soil for rammed earth wall construction. In particular, it examines the effects of adding different stabilisers to a wide range of soils, with a view to establishing improved guidelines for the appropriate assessment, selection, and stabilisation of soil for the construction of rammed earth walls. Although rammed earth has been used for construction for many centuries, its use in modern societies requires that it be strengthened and otherwise modified by the addition of stabilisers in order to meet building code requirements, such as those of the Uniform Building Code (UBC, 1997). These regulations require that the methods and materials of earth construction be quantified. In addition, there is a need for an improved knowledge base on earth stabilisation with which practitioners of rammed earth construction are able to make better judgements about the soils and stabilisers used. This thesis addresses these problems by measuring soil properties and stabilisation data for samples taken from earth building sites in New South Wales, Australia.

1.2 EARTH WALL CONSTRUCTION BASICS

Rammed earth is the most widely used construction technology in human history, and has been used in nearly every part of the world for thousands of years (e.g. Heathcote, 1995; Ngowi, 1997). Rammed earth structures have ranged from their common use in housing (Figure 1.1), walls, and fences, to monumental fortifications, palaces, and temples. These structures have proven to have great longevity and show that earth can be a durable building material when construction materials and methods are sound. The benefits of earth construction and architecture were realised by the earliest civilisations: low cost, ease of construction, durability, fire-proofness, and buffering of temperature variations. Thirty percent of the world's population lives in earth-walled shelters (Coffman et al., 1990).

Figure 1.1: Rammed earth dwelling.

Earth walls can be constructed using mud bricks (bricks made of mud and dried); rammed earth (earth compacted by mechanical means); and poured earth (liquid earth poured then dried). Mud bricks can also be stabilised by additives. There are two basic stabilisation methods: physical and chemical. Physical stabilisation involves the addition or subtraction of material (either soil or substances such as straw) that affects the physical or mechanical properties of the earth. Chemical stabilisation involves the addition of chemicals such as lime, cement, and asphalt that react with the natural soil constituents, or otherwise lead to an increase in durability and strength of the earth (UN, 1992).

1.2.1 Methods of building with earth

1.2.1.1 Mud bricks

Also known as adobe or puddle brick, this method involves making bricks from mud and laying them with mud-based mortar. Suitable soil is mixed with water by manual or mechanical methods, waterproofing and stabilising agents are also often added, and the mixture is then moulded into bricks using simple wooden or steel forms. After drying in the sun, the bricks are laid using a mortar of the same mixture. Adobe brick walls are aesthetically pleasing and can be used to construct straight or curved earth walls.

Soil bricks can be compressed, a method that relies upon mechanical pressure to compress soil to a density that allows the bricks to be used as building units. The equipment used for the process can be either a manual lever-operated system or a high-production hydraulic press. Many brick designs are in modern use, including interlocking dry-stack shapes and partially hollow forms.

1.2.1.2 Rammed earth

Rammed earth involves the incremental compression of a mixture of soil, stabiliser, and water, by using pneumatic rams to compact earth within shutters (formwork) that are removed after ramming. Common stabilisers include lime, Portland cement, and asphalt. Compressed pneumatically to a density in excess of 1.8 t/m^3, the resultant 30-60 cm thick wall (Figure 1.2) is extremely strong. Rammed earth walls are monolithic in nature, unlike adobe brick walls. This study addresses soil stabilisation with a focus on rammed earth construction, rather than on dried or compressed brick methods, although many of the principles involved are common to the different methods. Ramming earth directly into the formwork eliminates the moulding and curing process required for mud bricks, and as walls are constructed in large sections, there is a lesser requirement for mortar.

Figure 1.2: Rammed earth wall.

1.2.2 Characteristics of rammed earth buildings

Earth is a versatile building material with many qualities to justify its use. The material is fire resistant and has a 4-hour rating (Byrne, 1982). It possesses the structurally sound properties and values of load bearing walls (CSIRO, 1987), and its thermal mass properties mean that earth walls provide constant internal environment with little temperature fluctuation (Robertson, 1987). Earth also has a low sound transmission rating making earth homes quiet dwellings in which to live (Narang & Demos, 1983). In addition, earth is water resistant when treated with a proper waterproofer or sealant (Selwitz et al., 1990).

1.2.2.1 Durability

Earth walls in Europe, Africa, and Asia have longevities of several hundred years. This is without the addition of modern stabilisers or reinforcing, or the use of pneumatic ramming of the earth. Therefore, modern earth walls and homes should last for several centuries if built properly. Earth walls are fireproof, a property that obviously adds to the longevity of an earth home (Byrne, 1982).

Rammed earth walls and the homes constructed with them do not wash away with rain, are resistant to weathering, and do not deteriorate in quality if well built. Walls may be saturated for a long time without jeopardising their integrity; although in wet climates, raised foundations are usually needed. Exterior surfaces of earth walls require no waterproofing or sealing in arid environments. However, as walls do have a limited porosity, prolonged rainfall in stormy regions may induce moisture to penetrate through the wall into the interior surfaces. Sealing exterior surfaces in homes located in such areas will prevent this problem.

1.2.2.2 Structural properties

Ideally, stabilised rammed earth walls have load-bearing compressive strengths in excess of 2 MPa (CSIRO, 1987; UBC, 1997). In earthquake-prone areas, modern earth homes may be designed with additional structural elements included. For example, earth walls may be reinforced with a network of vertical steel rods extending from the footing to the bond beam. Generally, two to three storeys are the maximum height for rammed earth constructions. In modern societies, engineers' computations usually accompany plans in an application for house building permit approval for an earth home.

1.2.2.3 Thermal benefits

Current research indicates that rammed earth walls have poor thermal resistance (Clark, 2000). However, the high mass of rammed earth walls does slow the rate of transfer of sensible heat from the outside to the inside, and vice-versa, by being able to store large amounts of energy over long times. Interior temperatures therefore remain constant within a range of about 5 ^{o}C . Diurnal variations in temperature are particularly well catered for by earth homes. In desert (continental) climates that are characterised by high day-time temperatures and cool night-time, the home stays relatively cool during the day, and during the night will be warmer than outside due to the release of the sensible heat created by solar energy absorbed by the walls during the days. This is in addition to the high density of the compacted earth wall itself, which traps inside any heat internally generated by whatever source. Thermal conductivity is closely related in a positive manner to the dry density of the stabilised earth (Adam & Jones, 1995). Therefore, proper compaction of earth during ramming will assist not only in the achievement of strength but also provide thermal benefits. Further research is needed to determine which other factors in rammed earth walls compensate for low R-values.

1.2.2.4 Indoor living environment

Homes with earth walls are characterised by good air quality, quietness, and steady temperatures. These desirable features stem from the properties of the earth as a natural material, as a sound insulator, and as a thermal store. Humidity levels are generally lower than outside. In addition, rammed earth walls are resistant to habitation and damage by insects such as ants and termites. Earth walls have a “natural” and worn look, an attractive feature for many people.

1.2.2.5 Environmental considerations

Earth is found in most places where people live and is therefore a widely available, local building material. Building an earth home consumes considerably fewer resources than constructing in concrete, timber, or other conventional materials (Ren & Kagi, 1995). Earth construction does not pollute or otherwise alter the surrounding environment, apart from the removal of limited amounts of earth from its source. Costs of maintenance, and of heating or cooling the interior, are less than other types of homes.

1.2.2.6 Construction techniques

Earth wall houses cost a little more (about 10 %) than a similar-sized conventional house made of wood. The cost will vary with the design of the house, its size, and the supply of earth (local or transported from a remote source). Earth wall construction, even with machinery, is a labour-intensive process (Figure 1.3). It involves obtaining the soil, stabilising and mixing it to form a suitable earth for ramming; then setting up formwork and filling it with earth and ramming; and finally removing the formwork and finishing the cosmetics of the walls. Formwork may consist of plywood strengthened with steel frames, with spacers separating the two sides at a distance equal to the desired wall thickness. The forms are held together by rods that are then removed after ramming, with the resulting small holes being filled with earth. The roof may be attached onto timber running around the tops of the walls, and window spaces and doorframes are usually defined with steel or timber.

Figure 1.3: Construction of earth home.

Earth walls have a footing that may be of brick, stone, earth (not recommended), or more commonly, concrete. On top of the footing is placed damp proof course to form the bottom 30-60 cm of the wall, primarily to provide protection against the effects of rising damp. The exterior walls themselves are generally around 30-60 cm thick.

1.3 STABILISATION OF SOIL FOR EARTH WALL CONSTRUCTION

1.3.1 Introduction

The United Nations Centre for Human Settlements (UN, 1992) defines the stabilisation of soils as “the modification of the properties of a soil-water-air system in order to obtain lasting properties that are compatible with a particular application.” The main objectives of soil stabilisation are to achieve improved mechanical characteristics of the earth (increase compressive strength, tensile strength, and shearing strength); to reduce porosity and changes in volume (minimise shrink and swell due to water); and to improve resistance to rain and wind erosion (waterproofing and reduction of surface abrasion) (Winterkorn, 1975; UN, 1992; Symons, 1999). All these objectives will increase the durability of structures built using rammed earth.

Stabilisation techniques can be categorised into three main groups: compaction (densification), granular stabilisation, and chemical stabilisation. Compaction, or densification, involves compacting the soil either manually or mechanically to increase its density and strength. Compaction may be the single stabilisation method, or used with either granular and/or chemical stabilisation. Granular stabilisation is the mechanical combination of two or more materials (or soils) possessing complementary physical characteristics (e.g. particle size distribution) in order to produce a material that is more favourable for construction. Chemical stabilisation is the addition of a chemical, such as lime, to a soil in small quantities and the resulting chemical reactions produce a material with increased strength and improvements in other properties such as plasticity and shrinkage (e.g. Croft, 1968).

A range of stabilisers can be added to soil to stabilise it. Commonly used additive stabilisers include sand and clay, Portland cement, lime, cement-lime combinations, asphalt, straw, fly-ash and lime combinations, and sodium silicate (Wolfskill et al., 1963). Additionally, cow-dung, rice husks, or ant-beds may be used (CSIRO, 1987). The selection and amount of additive to be added is dependent upon the characteristics of the soil and the degree of change in soil quality desired (see Chapter 2 for detailed discussion of stabilisers and soil properties). Generally, smaller amounts of additives are required when it is simply desired to alter soil properties such as workability and plasticity, than when it is desired to improve strength and durability. In the selection of a stabiliser additive, the factors that must be considered are the type of soil to be stabilised, the type of soil quality improvement desired, the required strength and durability of the stabilised rammed earth wall, the cost of mixing the materials, and the environmental (exposure) conditions to which the resulting structure will be subjected.

1.3.2 Soil suitability for stabilisation

The selection of soil to be stabilised is a crucial step in the stabilisation process. The most suitable type of soil for rammed earth construction is a well-graded soil, with a range of particle sizes from sands through to clays (CSIRO, 1987). These types of soil have been shown to produce satisfactory compressive strength and durability, and low shrinkage (e.g. Akpokodje, 1985; Bryan, 1988a). Coarse (sand/gravel) particles in the soil will impart frictional strength to the stabilised material, and finer particles will add cohesive strength. In addition, a range of particle sizes encourages efficient densification of the material during compaction. Soils favourable to stabilisation are also identified by correlating soil properties with stabilisation outcomes, enabling the suitability of different soils to be assessed usually on the basis of soil texture and plasticity. It has been generally recommended that soils unsuitable for stabilisation include organic soils, clean gravels and sands, and highly plastic clays (Wolfskill et al., 1963; UN, 1992; USACE, 2000). Research by others has determined that particular values of soil texture and plasticity can be used to define the favourability of soils for stabilisation. For example, Bryan (1988) found that soils that respond most favourably to cement addition have clay contents between 5-30 % and liquid limits of less than or equal to 40 %. Dumbleton (1962) recommended that soils should have a minimum clay content of 10 % and a plasticity index greater than 10 % to be suitable for lime stabilisation.

1.3.3 Stabilisation treatments and effects on soils

1.3.3.1 Compaction (densification)

Compaction is used whether or not stabilisers are added to a soil, and may be done manually or with machines. Compaction by manual or mechanical methods changes the density, strength, compressibility, porosity, and permeability of the soil in ways that make the material stronger and more stable (Winterkorn, 1975; UN, 1992; Ausroads, 1998). The increase in density is a function of the gradation of the soil compacted, the compaction method and effort used, and the moisture content of the material (USACE, 2000). A range of particle sizes will pack more efficiently than narrower gradations, and therefore the material will densify to a greater degree. Greater compactive effort will bring about higher densities, although the relationship is one characterized by "diminishing returns", whereby progressively smaller increases in density are achieved as compactive effort is progressively increased.

However, the primary influence on effectiveness of compaction is the moisture content of the material being compacted. For any soil, there is an optimum moisture content at which a maximum dry density is developed, and above or below which, inferior densities are achieved (USACE, 2000). Different soils have different optimum moisture contents, with sandy soils compacting to maximum density at lower moisture contents than clay-rich soils. Thus, the moisture content of the soil being compacted is a very important variable that must be quantified and monitored in order to achieve maximum effectiveness of compaction during earth wall construction.

1.3.3.2 Granular stabilisation

Granular stabilisation can be accomplished by mixing or blending soils of two or more gradations to obtain a material meeting the required specification. Granular stabilisation is used to improve particle size distribution, which will increase the effectiveness of compaction and thereby add to the frictional strength of the material, and also improve the effectiveness of chemical stabilisers if added. However, this method of stabilisation is technically difficult to accomplish (CSIRO, 1987), and should be avoided if possible. Methods of proportioning the two (or more) soils demand that the gradation of the soils be known. Trial combinations are made and the plasticity of the resulting material is tested against the desired specification—a very cumbersome process.

1.3.3.3 Chemical stabilisation

Lime

Lime (oxides and hydroxides of calcium and magnesium) has the greatest effect on clay-sand or clayey soils. It reacts with the clay particles in the soil, and forms a cementitious material. Lime is commonly used in two forms: quicklime CaO, and hydrated lime $Ca(OH)_2$.The addition of 6-12 % lime by weight of dry soil is used for the stabilisation of soils. With lime there is no specified optimum quantity for each soil (UN, 1992), except that the higher the plasticity of the soil, the more lime needed to stabilise it. Lime has been shown to increase the compressive strength of soils (Croft, 1963; Akpokodje, 1985; Ngowi, 1997), with higher strengths resulting from increased stabiliser quantities. It also reduces soil plasticity and shrinkage (e.g. Spangler & Patel, 1949), thereby enhancing workability of the soil-stabiliser mixture. Lime may be used to alter plasticity prior to adding another stabiliser such as asphalt or cement.

Cement

Nearly all soils can be stabilised with cement and have dramatic improvement in their properties, although cement should not be used when the liquid limit of the soil is greater than 30-40 % (Bryan, 1988; UN, 1992; Walker, 1995). The required quantities of cement depend on particle size distribution and structure. Generally, sandier soils respond better to cement stabilisation. Sandy soils require as little as 2.25 % of cement content by weight of dry soil (CSIRO, 1987). Soils that have high clay contents will require as much as 10 % cement. A maximum clay content of around 25-30 % (by dry weight of soil) is recommended by several studies including Fitzmaurice (1958), Bryan (1988), and Walker (1995). The compressive strength is generally a function of the quantity of cement used, and Akpokodje (1985) showed that the compressive strength increases with increasing amounts of cement in a distinctly linear relation.

The way in which cement stabilises materials differs between soil types. Non-cohesive (sandy) soils have particle sizes larger than cement grains and can therefore be coated with cement. The hydrated cement bonds soil particles at points of contact, leading to an overall increase in the strength of the soil. The increase in the strength is a function of the number of points of contact between soil particles (which itself is a function of the particle size distribution of the soil) and of the effectiveness of compaction in bringing soil particles into the closest possible contact (Bell, 1975). In clay soils, the cement hydrates to form cement gels, and the lime that is released during the hydration process reacts with clay particles. The cement gels then penetrate the altered clay aggregates and bond with them.

Asphalt

Asphalt stabilisation, which makes earth material resistant to water absorption, is very seldom used in Australia. Soil stabilisation using asphalt emulsion neither increases nor decreases the strength of the earth material being used for the earth wall construction (CSIRO, 1987). It is usually used, therefore, as a waterproofer of stabilised earth. In order to obtain uniform distribution of the asphalt with the soil it is best to mix the asphalt with large quantities of water to form an emulsion. This asphalt emulsion is suitable for sandy or sandy-gravel soils, although the use of very clean soils with the emulsion can lead to separation of the asphalt under the action of water. Moist soils are not suitable for use with asphalt emulsion due to the difficulty of mixing the material with the soil. Normally 2-3 % of asphalt by weight of soil is added, but this value can be as high as 8 % (UN, 1992). CSIRO (1987) recommends that higher amounts of asphalt emulsion be used for the stabilisation of soils that contain lower proportions of sand.

1.3.3.4 Other stabilisers

Lime, cement, and asphalt are the most common stabilising agents used in modern earth construction in developed countries. Materials such as straw, dung, and ants' nests can be used as alternatives if available and if allowed under the prevailing building codes. These materials are often used as stabilisers in developing countries. Chemicals such as phosphates (e.g. Rucker, 1965), asbestos (e.g. Kruker, 1964), and urea-formaldehyde (e.g. Lahalih & Ahmed, 1998) have been studied as potential stabilizing agents, but were found to be unsuitable based on performance, cost, availability and toxicity. Oil and industrial slag have also been suggested as potential stabilising agents.

1.3.4 Stabilisation outcomes and guidelines

The properties of stabilised material that may be measured in order to assess its suitability include compressive strength, density, durability, and shrinkage (CSIRO, 1987; Walker, 1995; Heathcote, 1995; UBC, 1997). Various measures of strength are available, but the most common is unconfined compressive strength, which measures the resistance of the material to an applied load. The unconfined compressive strength of a material may be measured when it is dry or saturated, and at various curing times after the mixing of the soil and stabiliser. Generally, the most common compressive strength measurements are those as recorded after 7 days of curing (7-day test) and after 28 days of curing (28-day test). Building codes and other specifications define criterion values of compressive strength that represent the minimum acceptable standard to be reached for samples of stabilised soil. UBC (1997) specify a criterion of 2 MPa for rammed earth building units.

Durability of earth buildings is an important aspect of stabilisation. Durability describes the resistance to surface deterioration of an earth structure, and can be measured by various tests involving spraying water onto samples, soaking samples in water for specified periods to measure absorption, scraping and scratching sample surfaces, and measuring resistance to freeze-thaw cycles (CSIRO, 1987; Heathcote, 1995; UBC, 1997). Most building codes use an absorption test as an indicator of durability (e.g. UBC, 1997; New Mexico Construction Bureau, 1991). Shrinkage is important for rammed earth walls on account of their monolithic structure. Shrinkage is measured by the characteristics of cracks, if any. CSIRO (1987) specify that cracks in the stabilised material must be no longer than 75mm, no wider than 3mm, and no deeper than 5mm.

Practices vary among professional earth practitioners in soil selection, stabilisation methods, testing procedures, and in the criteria for evaluation of what constitutes a “good” earth wall. At present, there are a number of guidelines and publications that address various aspects of earth wall construction including soil selection and testing, earth stabilisation, construction procedures, and structural regulations and advice. They include the guidelines contained within the Handbook for Building Homes of Earth (Wolfskill et al., 1963); The United Nations Centre for Human Settlements Earth Construction Technology book (UN, 1992); and CSIRO’s National Building Technology Centre Bulletin 5 “Earth-Wall Construction” booklet (CSIRO, 1987). The CSIRO guidelines are probably the most important guidelines for rammed earth wall engineering in Australia. A feature of these guidelines concerning soil assessment, selection and stabilisation is that the focus is on extensive testing and trialling procedures to assess and stabilise soils.

1.4 RATIONALE

1.4.1 History of rammed earth construction

Rammed earth construction continues to be used as a construction method in developed and developing countries, despite the availability and popularity of other building technologies and materials. However, the 19th and 20th centuries have seen a decline in the popularity of the method in modern societies, with modern construction products continuing to be the main threat to its continued use.

Earth building in Australia was introduced by members of the First Fleet in 1788 with the use of wattle and daub construction. By the early 19th Century, rammed earth construction had become a popular building method in rural areas. The general order made by Governor Macquarie in 1810 (Archer, 1987) that houses be made of either brick or weatherboard, and the condition placed on land grants that a minimum standard of building be required, combined to make earth wall construction less desirable in urban settlement areas. Due to this historic circumstance, and the ready supply of materials such as clay-fired brick and wood, Australia's architectural heritage of rammed earth wall construction underwent a marked deterioration starting from the early 19th Century and continuing well into the 20th. Through this time the use of rammed earth was confined to a large degree to remote or inaccessible areas. During the 20th Century, the rise of modern building products and technologies almost brought the use of rammed earth to an end. Although the utilisation of unbaked earth no longer needs any justification on technical or material grounds, resistance to it still persists from groups whose economic, psychological, cultural, institutional, and political well-being is threatened by it. Influential institutions such as industrial corporations that produce building materials, as well as the technical consultants responsible for employing them, occasionally seek to discredit unbaked earth in order to protect their own market (Dethier, 1982).

However, during the last quarter of the 20th Century, increasing public awareness about the environment in which we live generated considerable interest in sustainable living and development. Concern about large-scale problems such as global climatic change, through to local problems such as rubbish recycling, has increased dramatically in the last two decades years. One aspect of this revolution in environmental awareness has been the reawakening to alternatives to conventional building technologies; one such alternative is rammed earth construction. The natural state of earth and its obvious environmental compatibility, added to its thermal benefits, safety, durability, and competitive economic cost, are again being appreciated. New technologies (e.g. pneumatic ramming techniques), and a desire by practitioners to improve knowledge and understanding of the materials and processes used in soil stabilisation and earth wall construction, are further assisting the renewed interest in earth building.

1.4.2 Need for better understanding of soil-stabiliser relationships

The desire to understand more about soil stabilisation and earth construction stems from both a practical need (practitioners' desire to be better able to appropriately stabilise soils for successful earth construction), and an academic point of view (a better knowledge base will help in understanding the processes and effects of soil stabilisation).

The majority of research work on soil as a building material concerns its use as a material for roading or earthworks (e.g. Winterkorn, 1975; Akpokodje, 1985; USACE, 2000). Of the works that relate to housing or similar structures, most concentrate on examining the thermal properties, history, rendering, and architectural merits of rammed earth (e.g. Robertson, 1987; Adam & Jones, 1995). While some of the studies of stabilisation for earthworks or roading purposes are applicable to soil stabilisation for earth walls because of the similarity of materials or stabilisation methods used (e.g. Croft, 1968; Winterkorn, 1975; Akpokodje, 1985; Symons, 1999), some are not. Part of the reason for this is that the properties of earth desirable for roading and earthwork applications differ from those needed for earth wall construction. The principles of road design are at variance with those of rammed earth in that the former involves constructing and compacting various different layers and assessing their effect on the properties of the finished road. It involves manipulation of the respective thicknesses of pavement, base course, sub-base, and other imported materials, all of which must overlie the native soil.

A number of studies have produced useful results specifically aimed at soil stabilisation for earth construction (e.g. Bryan, 1988, 1988a; Walker, 1995; Osula, 1996). These studies have examined the influences of soils and stabilisers on the qualities of the stabilised material, and assessed the material for the purposes of constructing earth walls (for houses). However, this literature addresses stabilisation with respect to compressed soil bricks rather than rammed earth. However, the results of these studies are directly relevant, as the soil materials and stabilisers are common to both methods. Differences may arise from the different compaction methods, although modern mechanical brick presses are capable of producing earth bricks that have been subjected to equivalent pressures during compaction as rammed earth. A significant difference between soil brick and rammed earth wall construction lies in the size of the unit of construction—bricks are small and a brick wall is composed of many bricks adhered together using mortar, while a rammed earth wall is a single, monolithic unit.

There is a need, therefore, to generate good empirical data as a foundation for an improved knowledge base on soil stabilisation for earth wall construction, which should lead to a better understanding of the specific problems of soil stabilisation for earth wall construction. These problems include: defining which properties of soil are most important for proper stabilisation; determining the response of a soil to different stabilisers; and assessing the suitability of a soil for stabilisation.

1.4.3 Need for better guidelines

Macquarie's order of 1810, concerning the materials and standards of houses, was probably the beginning of building regulations in Australia. Now, concomitant with earth construction in modern times are modern construction regulations, building codes, and planning requirements. The specifications contained in these building codes and regulations demand a need to be able to quantify the methods and materials of earth construction technology for the modern era.

However, current guidelines for rammed earth construction are considered by the author to be insufficient in a number of ways to fulfill this need for quantifying earth construction methods and materials, and a vital requirement of viable rammed earth wall construction is the generation of improved guidelines. Present guidelines (e.g. CSIRO, 1987; UN, 1992) are often too vague and rely on an approach that involves testing varying amounts of different stabilisers on the same material to identify the suitable stabiliser treatment. This approach necessitates repeating an extensive array of experiments at every new construction site. For example, Wolfskill et al. (1963) suggests using an experimental system that involves the use of three trial sets of seven blocks with a range of percentages of stabilisers being tested, using small amounts of soil material and limited mixing. Once the desirable quantity of stabiliser(s) has been determined, this amount is increased in the field to compensate for lack of proper mixing. What is needed is a procedure that better assesses the suitability of a soil for stabilisation, and more accurately quantifies the stabilisation treatments able to be used for a particular soil, without the need for extensive experimental stabilisation tests on every new occasion. Which stabiliser to use, and how much, depends largely on the type of soil to be treated. Therefore, if better criteria can be developed to assess soil suitability for stabilisation on the basis of soil properties, this should reduce the need for extensive stabilisation trials.

Historically, practitioners of earth construction have learned what they know through a process of trial and error, with the experience of practitioners being handed on to others in an apprentice-like fashion. Despite the long history of rammed earth as a building material, there has been surprisingly little scientific effort directed specifically at investigating the properties and processes that influence the structural and other characteristics of rammed earth. There is a need to determine the properties that control the performance of rammed earth in construction, both for the efficiency of the practice of earth stabilisation and construction, and for the purpose of meeting structural and other specifications as defined in modern building regulations. Practitioners need to be able to make better, more informed decisions about soil suitability for stabilisation, and the most appropriate stabilisation treatments to use in particular circumstances. In addition, more information about soil stabilisation for the purposes of earth construction is needed not only by practitioners, but also by engineers, and those officials who scrutinise building permit applications and architectural plans. A better understanding of soil characteristics and behaviour will permit more thorough exploitation of stabilised rammed earth wall construction methods and their proper use.

The conditions under which rammed earth wall construction takes place vary so greatly with variations in individual soils, project requirements, environmental conditions, and work limitations, that it is probably impossible to set out a single formula or set of instructions that fits all sets of circumstances. However, the absence of clearer guidelines for builders is a major barrier to the wider use of rammed earth construction. Rammed earth is a difficult building material with which to work. Concrete blocks, bricks, and other modern building materials with which engineers, for example, are used to working, are somewhat standardised and uniform in material properties and behaviour, whereas rammed earth exhibits more variation and is consequently a more complex construction material.

1.5 RESEARCH OBJECTIVES

As a response to the issues discussed above, and which are detailed further in Chapter 2, the research objectives in this study are to:

1. Determine the relationships between soil properties (gradation, plasticity, and moisture), stabiliser types (asphalt, cement, lime), stabiliser quantities, and stabilised earth qualities (density and compressive strength), for a wide range of soil types.
2. Identify the appropriate quantities of stabiliser needed to satisfactorily stabilise different soils.

3. Quantify the minimum, optimum, and maximum values of soil properties required to promote successful stabilisation.
4. Assess the relative importance of stabilisers and different soil properties in controlling variation in the qualities of stabilised earth.
5. Establish specific guidelines for soil assessment, selection, and stabilisation that can be used for the construction of rammed earth walls, based on the measured relationships between soil properties, stabiliser treatments, and the qualities of the stabilised material.

As discussed below in Chapter 3, these objectives are addressed through the collection and analysis of data derived from 230 stabiliser experiments involving measurement of more than 1700 unique data about soil properties and stabiliser treatments for 111 different samples. These data have been accumulated over a period of some 8 years from soils collected from rammed earth construction sites in New South Wales, Australia.

1.6 STRUCTURE OF THE THESIS

This research thesis is divided into seven chapters as follows.

Chapter 1. Introduction. This chapter has introduced the research topic, constructed the rationale for the study, and developed the objectives of the research.

Chapter 2. Background and previous work. Previous research on the topic is covered in this chapter, focusing on the history of soil stabilisation, the properties of soil and its suitability for stabilisation, stabiliser treatments, and relationships between soils, stabilisers, and stabilisation outcomes.

Chapter 3. Methodology and experimental techniques. This chapter describes and justifies the methods used in the study. It addresses such issues as the selection of variables to be measured, the soils and stabiliser treatments used, and presents experimental and measurement techniques.

Chapter 4. Data analysis and interpretation. The data are analysed and statistically evaluated. Relationships between soil properties, stabilisers, and rammed earth properties are established, to determine the effects of stabilisers and soil variation on the properties of rammed earth.

Chapter 5. Results and discussion. The new results are developed and discussed in the context of previous research and the original research objectives.

Chapter 6. Guidelines for stabilisation. This chapter details guidelines for soil selection and stabilisation, developed using the relationships established between soils, stabiliser treatments, and stabilisation outcomes as established in previous chapters. A flow chart representation of the guidelines is presented.

Chapter 7. Summary and conclusions. The most important results of the thesis are summarised and commented upon, and directions for further research identified.

CHAPTER 2: BACKGROUND AND PREVIOUS WORK

2.1 HISTORY OF THE SCIENCE OF SOIL STABILISATION FOR EARTH WALL CONSTRUCTION

2.1.1 Introduction

The concept of soil stabilisation for rammed earth wall construction probably goes back to the dawn of human history, since soil is virtually ubiquitous and an obvious choice as a building material. The oldest recorded cases of earth brick construction date back to about 10,000 BC in Mesopotamia (Heathcote, 1995). The Great Wall of China, 7.5 metres high and 9 metres wide in places, was built of rammed earth and stone during the Third Century BC, and in Germany and France there are several buildings of rammed earth that are at least 400 years old. Perhaps the oldest existing rammed earth wall building is the Pueblo in Taos, New Mexico, which is 900 years old, although it requires ongoing maintenance.

The scientific understanding and engineering application of modern rammed earth stabilisation is a product of the 20th Century, and started during the early 1930s (Patty, 1936; Winterkorn, 1939). The study of rammed earth as a building material and construction technique is really a study of the performance of soil as a structural material. Included among the physical principles upon which it is based are those that control the deformation (strain) of both elastic and plastic bodies due to applied stress, the nature of soil constituents particularly clay particles, and the influence of moisture on material properties.

The concept of stabilising soil with cement is attributed to the trials using such material for road construction by Brooke-Bradley on Salisbury Plain (England) in 1917. However, it was not until the 1930s in South Carolina in the USA that the technique was used on a substantial scale. Because of the urgent need for airfields during the Second World War, rapid development in soil-cement engineering took place. Papers published shortly after the war contain valuable information about this important phase in the history of construction materials. Sparkes & Smith (1945) stated “The purpose of stabilisation is to maintain the moisture content and the mechanical properties of the soil at a satisfactory level so that it will retain its originally compacted state indefinitely under traffic and weather conditions. This is achieved to an appreciable extent by compacting the soil to an adequate density. The same object can also be secured by adjusting the grading of the soil, by adding certain cementitious binders or resin.” This statement has stood the test of time, and remains an accurate description of the purpose and methods of soil stabilisation.

2.1.2 The work of Ralph Patty (1930s)

Patty (1936), working from the South Dakota Experiment Station, performed field trials on rammed earth wall construction. The purpose of his study was to determine the characteristics of soils favourable to rammed earth construction and to determine the optimum clay and sand ratio with respect to weathering resistance in rammed earth walls. In an effort to find a reliable method for accurately identifying favourable soils for rammed earth construction, he built test walls for determining the resistance of various types of soils to weathering.

Patty constructed 29 experimental rammed earth walls from different soil types obtained from different areas of South Dakota. The walls were built to examine the weathering resistance of walls that contained varying proportions of sand. Twenty-four of the walls were built during the summer of 1930, with the remainder being built the following summer. The walls were 91 cm wide by 76 cm high and 30 cm thick, and were built on concrete footings and protected on the top by a metal capping. The walls were built in an east-west run leaving the north and south faces exposed to the elements.

After 4-5 years, these test walls were given a relative grading of deterioration (weathering) by visual appearance. Comparisons were then made between the measured particle distributions of the wall material and the variable resistance of the to weathering. Three of the best walls rammed by Patty contained over 75 % sand. Of these three walls, two contained fine sand, while the third contained coarse and gravelly material. Patty inferred that the size of the grains of sand have little, if any effect, on the resistance to weathering (Table 2.1). After examining and rating all the test walls, the clay content of the walls was the most observable difference between walls of varying quality as assessed by the degree of deterioration. Patty concluded that soils containing 40% or more clay should not be used for rammed earth wall construction, although these soils were useable if rendering coats were applied. The best rammed walls in Patty's yard had clay contents that ranged from 30-39%. However, the walls used in the tests were of lower compressive strengths than would be acceptable under modern structural standards, and the results are not alone sufficient for a reliable determination of soil suitability for stabilisation.

Wall No.	Rating (%)	Sand (%)	Clay (%)	Silt (%)
1	99	76.2	14.3	9.5
2	99	77.2	15.9	6.9
3	97.5	71.6	19.2	9.2
4	97	19.9	23.8	56.3
5	96.5	75.2	17	7.8
6	95.5	44.8	21.6	33.6
7	95.5	53.3	27	19.7
8	92	28.8	26.4	44.8
9	91.5	27.9	26	46.1
10	90.5	28	22.8	49.2
11	90	35.6	28.4	36
12	90	17	28.8	54.2
13	89.5	45.8	29.6	24.6
14	89	27.2	27.6	45.2
15	88.5	47.9	25.1	27
16	88	47.7	26.9	25.4
17	87.5	48	23.4	28.6
18	86	41.8	28	30.2
19	82.5	21.7	26.9	51.4
20	82.5	34.3	33.1	32.6
21	79	18.6	33.2	48.2
22	75	35	33.1	31.9
23	73.5	38.4	34.2	27.4
24	72.5	25.2	35	39.8
25	70	19.3	41.4	39.3
26	69	29.8	33	37.2
27	65	23	38.4	38.6
28	50	25.9	49.1	25
29	50	19.8	51.5	28.7

Table 2.1: Data about wall quality ratings and soil properties for the weathered rammed earth walls of Patty (Patty, 1936). The "rating" is the personal judgement of Patty and two associates and is based on a visual assessment of the deterioration of the wall, with a high score indicating a better (less deteriorated) wall.

2.1.3 The work of George Middleton (1940s)

In 1946, George F. Middleton was appointed as research officer at the Commonwealth Experimental Building Station, North Ryde, New South Wales, Australia. He set out to define a reliable method for accurately identifying favourable soils for rammed earth wall construction. In his first publication on rammed earth (Middleton, 1947), he reviewed a number of English and American reports which were added to his own observations of a number of rammed earth houses in Victoria. He noted that rammed earth construction was performed by “rule-of-thumb” having developed over time by trial and error of its practitioners. In this context, Middleton began his experimental study of rammed earth walls to secure more definitive and reliable information with respect to earth construction. The wide range of soil types over Australia made it impossible to make reliable recommendations as to their use for this construction method without a careful and detailed study. The purpose of the study was to determine the structural characteristics of soils that are favourable to rammed earth wall construction, to determine the optimum clay: sand ratio, and the optimum moisture content for both strength and weathering resistance in rammed earth walls. He expressed a hope that reliable guidelines and information might lead to increased use of rammed earth construction of buildings.

In the spring of 1947, Middleton built 23 walls for his study. Middleton’s approach of using experimental rammed earth walls was based on Patty’s work from North Dakota. A garage, temperature hut, and additional walls to be rendered were built the following winter in 1948. The experimental yard in which the test walls were built was located on the Experimental Building Station grounds located in North Ryde, New South Wales. Although there was some protection from trees, their exposure to the weather was comparatively uniform. The layout of the walls in the experimental yard is shown in Figure 2.1. The walls were 1215 mm long by 900 mm high and with a thickness of 305 mm. They were built on a concrete footing extending 305 mm below ground grade level and 150 mm above ground level. The walls were built with a north and south exposure to their broad sides.

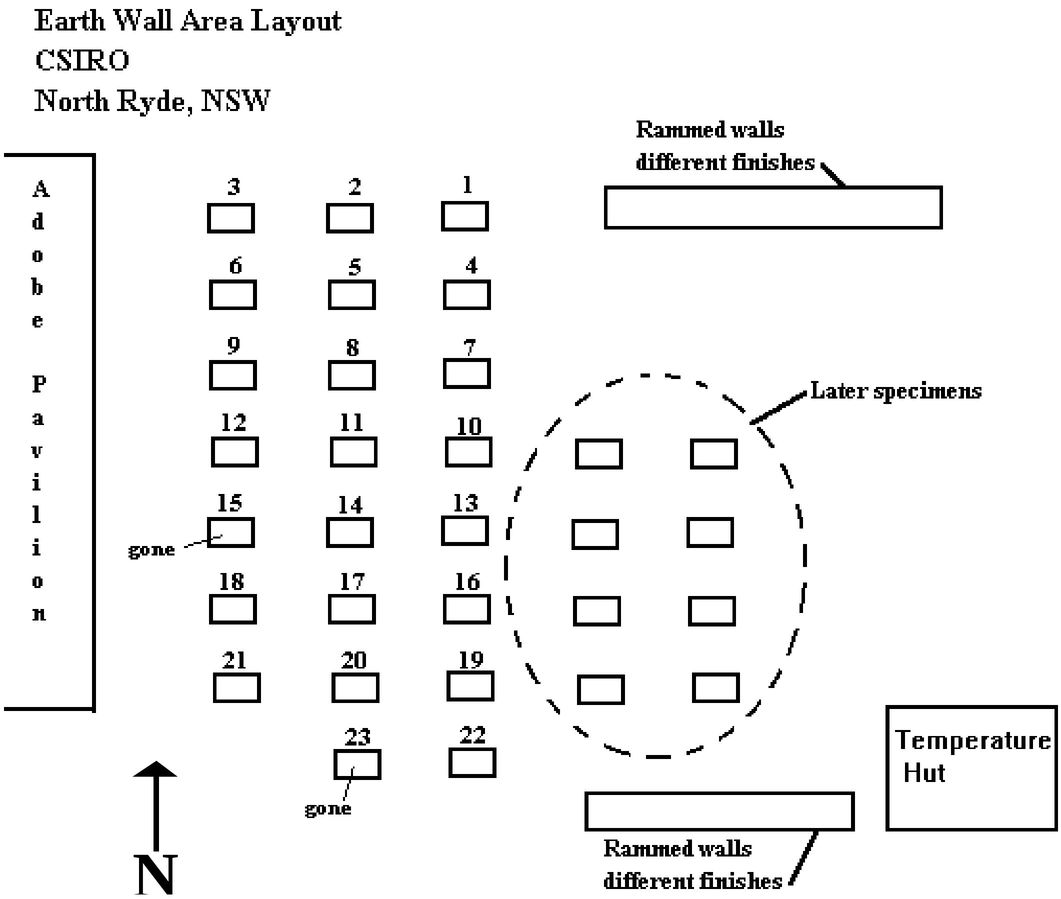


Figure 2.1: Layout of Middleton's (1947) test rammed earth walls at North Ryde, New South Wales.

Middleton found that the effect of sand in the soil was favourable to weathering resistance, without exception (Table 2.2). However, there were a few walls in which soils having a low sand content showed high resistance to weathering. Nevertheless, it was found that soils containing 40 % or more sand were favourable to weathering resistance. Both Patty and Middleton, therefore, had identified the importance of the particle size characteristics of the constituent soil to the quality of the rammed earth wall. However, their research did not address aspects of rammed earth wall construction such as chemical stabilisation, and material plasticity and shrinkage.

Wall No.	Date built	Sand/clay (%)	Damp Course	Finish on Wall	Remarks
1	26/8/47	70 / 30	Plastogel	None	Some attrition from rain - considered due to light ramming
2	25/8/47	70 / 30	"	Render	Hair line crack appeared south side
3	22/8/47	70 / 30	"	Emulsion	Excellent condition
4	27/8/47	70 / 30	"	Paint	Excellent condition
5	20/10/47	64 / 36	"	None	Some rain damage
6	21/10/47	64 / 36	"	Render	Bad cracks in all directions
7	21/10/47	64 / 36	"	None	To be painted
8	21/10/47	64 / 36	"	Emulsion	Excellent condition
9	24/10/47	61 / 39	"	Render	Extensive cracks on north and south sides
10	30/10/47	61 / 39	"		To be painted
11	31/10/47	61 / 39	"		To be emulsion coated
12	26/10/47	61 / 39	"		Some rain damage
13	29/10/47	70 / 30	Asphalt lead - core		Rammed earth blocks - some rain damage
14	28/11/47	39 / 61	"		Adobe earth blocks - some rain damage
15	29/11/47	70 / 30	"		Rammed earth block - some rain damage
16	5/11/47	70 / 30	"		Excellent condition
17	13/11/47	39 / 61	"		Wall demolished 17/11/47
18	11/11/47	70 / 30	"		Rammed earth, asphalt stabilisation, excellent condition, monolithic built
19	4/11/47	70 / 30	"		Same as wall #18
20	?/1/53	0/100	"		Rammed earth, cement stabilisation, excellent condition, monolithic built
21	?/1/53	0/100	"		Same as wall #18, built with sandy/loam

Table 2.2: Middleton's summary of test walls as at 31 December 1947.

2.1.4 Soil stabilisation research and literature 1950 onwards

The Second World War, regional military conflicts, and aid for developing countries, together brought about a great expansion in basic research about soil stabilisation and its application to road engineering. Since the 1950s the majority of the advances in knowledge and understanding of soil stabilisation, stabilisers, and material properties have been with regard to roading, paving, and construction. Much of the literature produced since the 1950s is characterised by laboratory tests and experiments that have generated data on the performance of materials, and most is directly or indirectly applicable to soil stabilisation for rammed earth walls. This research has been performed by transport organisations, such as the Transport Research Laboratory, London, and the Highway Research Board, USA; by engineering research institutions, for example the US Army Corps of Engineers; and by university construction and geotechnical engineering departments. Contributions to the field of soil stabilisation have also come more recently from research in the field of materials science, particularly regarding clays and their behaviour.

Of those works that relate specifically to housing or similar structures most concentrate on examining the thermal properties, history, rendering and architecture of rammed earth, rather than the selection of soil and its stabilisation for earth construction. Several published handbooks provide guidance for earth construction, including Wolfskill et al. (1963), CSIRO (1987), and UN (1992). These handbooks provide general guidelines for the earth building practitioner including soil selection and stabilisers. There is also a small body of research work performed by university academics, earth practitioners, architects, and engineers in both developing and developed countries aimed at examining (and improving) earth materials for construction of dwellings (e.g. Akpokodje, 1985; Webb, 1994; Lilley & Robinson, 1995; Bell, 1996; Osula, 1996; Ngowi, 1997).

2.2. SOIL PROPERTIES IMPORTANT FOR RAMMED EARTH STABILISATION

2.2.1 Introduction

A working definition of the term "soil" for the rammed earth practitioner denotes unconsolidated material that overlies (and is distinguishable from) bedrock and which provides the base material used for the construction of rammed earth walls. Some soils are less suitable than others, for example pure sands, organic soils, aggregates, and materials of high clay content are unsuitable for rammed earth stabilisation (CSIRO, 1987). Soil is distinct from the term "earth" which refers to the construction material that is formed by the stabilisation and ramming of soil, hence "rammed earth".

"Stabilisation" is the process of mixing physical or chemical additives with a soil, and compacting it, to improve certain properties of the soil. These properties are porosity, permeability, strength, compressibility, and density (UN, 1992; USACE, 2000). The process may include the blending of soils to achieve a desired gradation or the mixing of chemical additives that may alter soil properties, or act as a binder or matrix for the soil.

Stabilisers (also termed modifiers or additives) include commonly available products such as cement, asphalt, and lime, and others such as slag, fly-ash, and various combinations of these substances.

Although soil is composed of a mixture of solids, liquids, and gases, the normal diurnal and seasonal variations in the proportions of liquid and gas phases in a natural soil has led to the consideration of solid constituents as the actual soil material. The solid constituents of a soil are uncemented (or only weakly cemented) mineral particles from disintegrated rock with or without a mixed organic matter. The size of these particles may range from sub-micron scales to several centimeters. Engineering soils include aggregate, sand, and natural soils ranging from predominantly sandy texture to heavy colloidal clays, and mixtures and combinations of all the named materials. Chemically, the particles in these soils may represent a large variety of soil minerals and of organic matter of varied composition and different stages of decay. Not all soils are of use to the rammed earth practitioner, as sound rammed earth structures can only be made from a limited range of soil types (discussed further below). An important point for soil stabilisation and rammed earth construction is that the environmental processes that formed the natural soil continue to be active on stabilised soil systems themselves. An understanding of these factors and of the physical and chemical mechanisms involved in soil formation is needed for an understanding of the durability of stabilised soils.

2.2.2 Textural properties of soil

The texture (or gradation, or particle size distribution) of a soil is divided into four main size categories. Table 2.3 reports the soil particle size fractions as defined in Australian Standard 1289. Often, the silt fraction is combined with the clay fraction to give a "fines" category and a three-fold, rather than four-fold, broad division of particle sizes. The fraction of clay particles of size less than 0.001 mm dominates the inorganic colloid fraction of the soil (colloids also consist of other minerals, e.g. aluminium oxide, and organic colloids consisting of humus).

Gravel	Coarse	60 - 20 mm
	Medium	20 - 6 mm
	Fine	6 - 2 mm
Sand	Coarse	2 - 0.6 mm
	Medium	0.6 - 0.2 mm
	Fine	0.2 - 0.06 mm
Silt	Coarse	0.06 - 0.02 mm
	Medium	0.02 - 0.006 mm
	Fine	0.006 - 0.002 mm
Clay	< 0.002 mm	

Table 2.3: Soil particle size fractions as defined by Standard Method by sieving (Australian Standard 1289).

Clay particles are important in the stabilisation of soil for the construction of rammed earth walls. In particular, the clay particles control the cohesive bonding and plastic properties of soil, and the way in which it responds to moisture, and therefore control the shrinkage and swelling behaviour of earth walls as well as part of their strength (e.g. Walker, 1995). In addition, chemical reactions involving clay and lime (the pozzolanic reaction) to form cementitious products are a major contributor to the strength of earth stabilised with lime (and cement) (e.g. Akpodje, 1985; Bell, 1996).

Two broad groups of clay are recognised—the silicate clays (characteristic of temperate regions) and the iron and aluminium hydrous oxide clays (characteristic of tropical and semi-tropical regions). Silicate clays are derived from the weathering of common rock-forming minerals such as mica, feldspar, amphibole, and pyroxene. Their genesis may result from either direct alteration of these minerals, or by recrystallization of their weathering products. Silicate clays are laminated in structure, consisting of plates or flakes or more rarely rods or blades. At a finer scale, the silicate clay particles that comprise the laminated plate structures are crystalline. Most silicate clays are aluminosilicates, containing both aluminium and silicon in structures that are tetrahedral (silicon) or octahedral (aluminium) in nature. These two basic layers, of tetrahedra and octohedra, in different combinations and layering, form the structural basis of silicate clays. The clays can be divided into 4 groups upon this basis: 1:1 (silica: aluminium) type minerals (e.g. kaolinite, halloysite); 2:1 expanding type minerals that expand internally by the addition of water molecules and cations within the layers (e.g. montmorillonite, vermiculite); 2:1 non-expanding types (e.g. illite); and 2:2 type minerals (e.g. chlorites). Although the properties of clays are well known, it is only relatively recently that the differences between clay types have been shown in a quantitative way to influence aspects of soil stabilisation (e.g. Bell, 1996).

Sand particles are needed in a soil to be stabilised to help form the granular skeleton of the material. Although sand grains lack cohesion, their inter-grain contact gives the material high frictional strength. Sand particles also help to limit shrinkage and swell. Silt particles also contribute to frictional strength, and can act as a pore filler for sandier soils. The role of gravel constituents in soil stabilisation is somewhat uncertain, although it might be expected that small proportions of gravel would assist in contributing to frictional strength. The textural properties of a soil are important in determining how the soil responds to stabilisation, and are discussed further in other sections in this chapter.

2.2.3 Plasticity

Plasticity is the property of soil that describes deformation without elastic failure (cracking or breaking) (Brady, 1974; USACE, 2000). A fine-grained soil can exist in any one of several different states, depending on the amount of water in the soil. The boundaries between these different soil states are moisture contents called consistency limits or Atterberg limits after the Swede who first characterised them in 1908. The Atterberg limits are used to describe the transitions of soil material from semi-solid, to plastic, to fluid, using tests on the fine mortar size of the soil (<0.4 mm particles). The plastic limit is the proportion of water that corresponds to the transition between the semi-solid and plastic states, and the liquid limit similarly represents the transition between plastic and liquid consistencies, above which the soil exists in a liquid state. The difference between the two, the plasticity index, describes the range of moisture content over which the soil is in a plastic condition. The Atterberg limits are important properties of fine-grained soils, and are used in identifying and classifying soils (e.g. Section 2.3). In addition, they are utilised in specifications to control the properties, compaction, and behaviour of soil mixtures (USACE, 2000).

2.2.3.1 Plasticity index

The plasticity index (PI) of the sample mud brick material is defined as the numerical difference between its Liquid limit (LL) and its Plastic limit (PL). It also is stated as a percentage of dry weight. It measures, in combination, the fineness of the soil mortar and the interplay of the attractive forces tending to hold the clay-mineral flakes together and the thickness and lubricating properties of the water film. Soils in which adsorbed films on particle surfaces are relatively thick compared to particle size (such as clays) are plastic over a wide range of moisture contents. This is presumably because the particles themselves are not in direct contact with one another. Plastic deformation can take place because of distortion or shearing of the outside layer of viscous liquid in the moisture films.

For a coarse-grained soil, or for a fine-grained soil with few particles of clay or colloid size, a small increase in moisture above the plastic limit provides enough particle separation to destroy the attractive forces, which provide shearing strength. This means that the difference in numerical value between the plastic limit and the liquid limit is small, so that the plasticity index also is small. On the other hand, for a soil high in clays or colloids, considerable water will be required before the attractive forces are overcome and the strength of the mass is destroyed. In this case, the numerical value of the plasticity index is high. Thus the plasticity index is an indirect method for measuring the amounts and moisture affinities of the clays and colloids in the soil.

Sandy soils and silts have characteristically low plasticity index values, while most clays have higher values. Soils that have high PI values are highly plastic and are generally highly compressible and highly cohesive. A plastic soil is considered cohesive if its PI is 5 or more. These soils possess some cohesion or resistance to deformation because of the surface tension present in the water films. Thus, wet clays can be molded into various shapes without breaking and will retain these shapes. Gravels, sands, and most silts are not cohesive and are called cohesionless soils. Soils of this general class cannot be molded into permanent shape and have little or no strength when dry and unconfined, although they may be slightly cohesive when damp. This is attributed to apparent cohesion, which forms due to the surface tension in water films between the grains.

Coarse soils (such as clean sands and gravels) are nonplastic, and the plastic limit cannot be determined. In this case, the plasticity index is reported as zero (USACE, 2000). Silts also are essentially nonplastic materials, since they are usually composed predominantly of bulky grains; if platy grains are present, they may be slightly plastic. Non-plastic materials displace easily under wall loading and tend to crumble apart when made into brick shapes.

2.2.4 Shrinkage

The shrinkage limit is the boundary between the semi-solid and solid states of a soil, and is reported as a percentage corresponding to the moisture content of the soil at this soil state. Many soils undergo a marked reduction in volume as their moisture content is reduced, and the effect is most pronounced in clay soils. Sands and gravels, however, demonstrate negligible volume change with change in moisture content.

The shrinkage of a clay soil may be attributed to the surface tension existing in the water films created during the drying process (USACE, 2000). When the soil is saturated, a free-water surface exists on the outside of the soil mass, and the effects of surface tension are not important. As the soil dries out because of evaporation, innumerable meniscuses are created in the voids adjacent to the surface of the soil mass. Tensile forces are created in each of these boundaries between water and air. These forces are accompanied by compressive forces that act on the soil structure. For the typical, fairly dense structure of a sand or gravel, the compressive forces are of little consequence; very little or no shrinkage results. In fine-grained soils, the soil structure is compressible and the mass shrinks. As drying continues, the mass attains a certain limiting volume. At this point, the soil is still saturated. The moisture content at this stage is called the shrinkage limit. Further drying will not cause a reduction in volume but may cause cracking as the meniscuses retreat into the voids. In clay soils, the internal forces created during drying may become very large. The existence of these forces also principally accounts for the rocklike strength of a dried clay mass. Both silt and clay soils may be subject to detrimental shrinkage with disastrous results in some practical situations. For example, the shrinkage of a stabilised clay soil during curing may produce cracks in a structure such as an earth wall.

Swelling is the opposite process to shrinkage. If water is again made available to a clay soil that has undergone shrinkage, water enters the soil's voids from the outside and reduces or destroys the internal forces previously described. Thus, a clay mass will absorb water and expand or swell. If water is made available to the soil after it has dried below the shrinkage limit, the mass generally disintegrates or slakes. Slaking may be observed by putting a piece of dry clay into a glass of water. The mass will fall completely apart, usually in a matter of minutes. Construction problems associated with shrinkage and expansion are generally solved by removing the soils that are subject to these phenomena or by taking steps to prevent excessive changes in moisture content.

2.2.5 Compactibility

The compactibility of a soil is its capacity to be compacted to a maximum given a specific compacting force and at the optimum moisture content. As a material is compacted, the voids ratio decreases, porosity is reduced, and densification occurs (USACE, 2000). The Proctor compaction test is the most widely-used standard test used to assess the compactibility of a soil. Different soils have different optimum moisture contents so that different soils will require different conditions to be optimally compacted. Compaction is described in more detail in Section 2.5.

2.2.6 Mechanical properties of soil

Mechanical failure of soil systems usually takes place in shear. Therefore, the shear resistance of soils and its functional connection with compositional and other factors are of primary concern to the rammed earth practitioner. The shear resistance of soils can be expressed in first approximation by the formula developed by Coulomb:

$$\Upsilon = \sigma_n \tan\phi + c$$

in which

Υ = maximum shear stress or shear strength
σ_n = normal stress on the shear plane
tan ϕ = coefficient of internal friction
c = cohesive resistance

For systems without cohesion, i.e. for those composed of granular materials with no silt-clay binding, this equation reduces to:

$$\Upsilon = \sigma_n \tan\phi$$

Both ϕ and c for a particular soil can vary. They are both functions of the volumes and degree of dispersion of the various soil components. For a purely granular soil, tan ϕ is a function of the void ratio (the ratio of the pore space to total volume of material) in the form:

$$\tan\Phi_e = \frac{\tan\phi_{crit}(e_{crit}-b)}{(e-b)}$$

in which

tan Φ_e=coefficient of internal friction at the void ratio e
tan Φ_{crit} = coefficient of internal friction at the critical void ratio, e_{crit}
b = constant of the character of an effective minimum void ratio

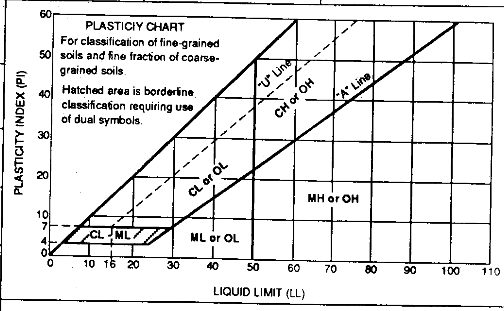
The critical void ratio is one at which shear results in neither an increase nor a decrease in the volume of the tested soil. For a limited range of normal stress on the shear plane, the product $(e_{crit}-b)\tan\Phi_{crit}$ becomes a constant. Denoting this constant as k, we can write:

$$\tan\phi_e = \frac{k}{(e-b)}$$

The cohesive contribution to the shear strength of a soil system is a function of the relative volume of silt clay material and, more especially, of the ratio of the volume of the water component to that of the silt-clay material (Winterkorn, 1955; Vees & Winterkorn, 1967).

2.3 SOIL CLASSIFICATION FOR ENGINEERING PURPOSES

There are many soil classification schemes, which vary according to purpose and from country to country. The most widespread classification for soil engineering purposes, the Unified Soil Classification System, is shown in Table 2.4. The classification is based on textural and plasticity measurements, as these are the primary determinants of the mechanical behaviour of soils. The soils are placed into one of three broad categories - coarse-grained, fine-grained, and highly organic. They are then further subdivided according to textural with symbols G for gravel, S for sand, M for silt, and C for clay. For example, a soil that meets the criteria for a silty gravel would be designated GM. Some soils are borderline and cannot be classified by a dual symbol, in which case four letters may be needed to describe them properly (Table 2.4). The fine-grained soils are further classified according to their plasticity and compressibility.

Major Divisions			Group Symbols	Typical Descriptions	Classification Criteria		
Coarse-Grained Soils More than 50% retained on No. 200 sieve*	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines	Classification on basis of percentage of fines: Less than 5% pass No. 200 sieve — GW, GP, SW, SP; More than 12% pass No. 200 sieve — GM, GC, SM, SC; 5% to 12% pass No. 200 sieve — Borderline classification requiring use of dual symbols	$C_u = D_{60}/D_{10}$ Greater than 4 $C_z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines		Not meeting both criteria for GW	
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures		Fines classify as ML or MH	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures		Fines classify as CL or CH	
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands	SW	Well-graded sands, and gravelly sands, little or no fines		$C_u = D_{60}/D_{10}$ Greater than 6 $C_z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			SP	Poorly graded sands and gravelly sands, little or no fines		Not meeting both criteria for SW	
		Sands with Fines	SM	Silty sands, sand-silt mixtures		Fines classify as ML or MH	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Fines classify as CL or CH	
Fine-Grained Soils 50% or more passes No. 200 sieve*	Silts and Clays Liquid limit less than 50		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands			
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL	Organic silts and organic silty clays of low plasticity			
	Silts and Clays Liquid limit 50 or more		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity			
Highly Organic Soils			PT	Peat, muck, and other highly organic soils	Visual-Manual Identification, see ASTM Designation D2488		

*Based on the material passing the 3-in. (75-mm.) sieve.

Table 3: Soil Classification Syst

Table 2.4: Soil classification for engineering purposes, based on measurements of particle size distribution and plasticity (from USACE, 2000).

2.4 INTRODUCTION TO SOIL STABILISATION

2.4.1 Soil stabilisation and its objectives

Soil stabilisation is the mechanical, physical, or chemical treatment of a soil designed to increase or maintain stability of the soil or otherwise to improve the engineering properties of the soil, enabling the material to serve as a better construction material (UN, 1992). Different engineering uses of soil pose different requirements in strength and other properties. Discussion here is placed on the use of soil stabilisation for the construction of rammed earth walls, although work on stabilisation of soil for roads and pavements is also relevant as many of the principles are similar, and is discussed where appropriate.

Stabilisation is a complex problem with a large number of variables involved. Proper solution of soil stabilisation problems involves a knowledge of the mechanical properties of the soil in question obtained by testing and measurement, and an understanding of how these properties are related to the physical properties of the soil. In addition, knowledge of chemical and physico-chemical reactions is required for the appropriate selection of a chemical stabiliser after the soil to be used for the construction has been characterised. Moreover, an understanding is needed of the climatic and other environmental processes to which the system will be exposed and its anticipated reaction to these processes. Soil stabilisation involves, then, more than an improvement in soil strength and other properties - it must also aim to produce a material whose integrity is resistant as far as possible to external processes (Winterkorn, 1975).

There are three broad types of stabilisation (e.g. Winterkorn, 1975; UN, 1992, Webb, 1994). Mechanical stabilisation is the general term for stabilisation by compacting the soil thereby decreasing soil porosity and increasing density. Physical stabilisation refers to the addition of particulate material to the soil, for example clay and sand. Chemical stabilisation involves the addition of chemicals to the soil, particularly cement, lime, asphalt, and industrial by-products. The selection and amount of material or chemical to be added is dependent upon the broad soil characteristics and the degree of improvement in soil quality desired (Wolfskill et al., 1963;

Akpodje, 1985). Generally, smaller amounts of physical or chemical additive are required when it is desired to alter soil properties such as gradation, workability, and plasticity, than when it is desired to improve strength and durability. In the selection of an additive, the factors that must be considered are the type of soil to be stabilised, the type of soil quality improvement desired, the required strength and durability of the stabilised rammed earth wall, the methods of mixing the materials, the cost of different stabilisers, and the environmental (exposure) conditions to which the resulting structure will be subjected.

Stabilisation involves direct modification of soil texture and structure. Interstitial voids may be reduced in volume by ramming, effecting a reduction in porosity. Voids may also be filled by adding particulate matter to the soil, thereby reducing porosity and permeability (USACE, 2000). Bonding between soil particles can be improved, mainly by chemical additives, resulting in increased mechanical strength. The most common soil quality improvements through stabilisation include better soil gradation, a reduction of the plasticity index or swelling potential, and increases in density, strength, and durability (UN, 1992).

Rammed earth wall construction is based on the premise that specified levels of quality will be achieved for each soil layer compacted into the formwork. Each soil layer compacted into the formwork must resist shearing within the layer, avoid excessive elastic deflections that would result in fatigue cracking within the layer or in overlaying layers, and prevent significant deformation through densification. The better each rammed earth layer, the greater the ability to distribute load over a greater area. This is usually achieved by reducing the thickness of the rammed earth layer. The tensile strength and stiffness of a soil layer in a rammed earth wall can also be improved through the use of additives and thereby permit an increase in the thickness of the stabilised soil layers in the rammed earth wall (UN, 1992).

Besides density and strength increases in soil, stabilisation should aim to waterproof the soil or otherwise enable mitigation of the effects of water. There are two stabilisation methods that reduce water erosion and the swell and shrinkage of soils that are exposed to successive wetting and drying cycles. The first method involves filling all the voids, pores and cracks and micro-cracks with a material that is unaffected by water; asphalt is probably the best example of a stabilising agent that acts in this way (CSIRO, 1987). The second method is to dispense in the soil a material that expands upon contact with water thus preventing the infiltration of pores (UN, 1992). Bentonite is typical example of a material used in this way. The dry weight of a highly cohesive soil compacted at its optimum moisture content to maximum density is usually less than 1600 kg/m^3. Assuming an average specific gravity of 2.65 for the soil solids, voids therefore make up 40 % or more of the total soil volume (Winterkorn, 1975).

The extent and rate of the penetration of stabilising agents into the soil and the case hardening, or waterproofing of the rammed earth walls, depend on such factors as soil size, permeability, and moisture content. Any differences between field and laboratory procedures and conditions of curing and testing must be taken into account when evaluating and comparing data derived from the field and the laboratory. Among factors often overlooked are the sequence of admixture, in the case of two or more reacting substances added; the time allowed for mixing; and the period between completion of mixing and beginning of ramming of walls (Arman & Saifan, 1967; West, 1959; Osula, 1996).

The method of stabilisation chosen depends on the properties of the soil requiring stabilisation, and the required properties of the resultant earth. Other factors that may need to be considered are the costs, as these differ between stabiliser treatments. The costs of chemical stabilisers, for example cement and asphalt, may need to be balanced against transporting onto the construction site suitable soil either to stabilise or to add to the local soil. Future costs that relate to earth wall maintenance may also need to be considered.

2.4.2 Types of stabilisation

2.4.2.1 Compaction/densification

Compaction by manual or mechanical methods changes the density, strength, compressibility, porosity, and permeability of the soil (Winterkorn, 1975; UN, 1992). The increase in density achieved is a function of the properties of the soil compacted, the compaction method and effort, and the moisture content of the material. Compaction is detailed in Section 2.5.

2.4.2.2 Granular stabilisation

Granular stabilisation is the alteration of soil properties by changing the gradation of the soil by the controlled addition or removal of particles (CSIRO, 1987; UN, 1992). Granular stabilisation can be accomplished by mixing or blending soils of two or more gradations to obtain a material meeting the required specification. The soil blending may take place at the construction site, or at a central plant or yard. The blended material is then moistened with water loaded in the formwork and compacted to required densities by ramming.

2.4.2.3 Chemical stabilisation

Chemical stabilisation is the alteration of soil properties by the use of chemical additives. When mixed into a soil these additives result in strength increases and changes in porosity, permeability and density. Chemical stabilisation is a term for all those methods in which chemical reactions play a predominant role and covers soil-cement and soil-lime stabilisation, and waterproofing with asphalt. There are two broad types of chemical stabilisation. First are those methods in which chemical reactions within additives provide the stabilisation mechanism by forming a matrix but which do not react with the soil. This includes the common stabilisation method of addition of cement. Second are those methods in which one or more chemicals are added to a soil and which react both with each other and with the soil components themselves, for example, the reaction between clay particles and lime or cement (pozzolanic reaction).

The general approach in cementitious stabilisation with chemicals is to form insoluble compounds that have enough affinity for the surfaces of the natural soil minerals or for the water-film held by them to adhere strongly. The theoretical strength of the dipole bond of two water molecules at most favourable orientation is about 20 kg/m^2. With regard to the kind of chemicals most likely to have a good affinity for the common soil, it is evident that the compounds most likely to have the greatest affinity for the surfaces of the soil minerals are those that contain a considerable amount of oxygen, such as hydrated silicates, aluminates, ferrates, phosphates, and sulphates. However, there exist certain restrictions with respect to the rate of formation of these insoluble compounds and also to the solution companions present at the time of their formation. It is best if the only other ions present are H^+ and OH^- since these can coalesce to form water; the least beneficial condition is when the solution companions form soluble salts, especially when these salts are present in concentrations that have a flocculating effect on those compounds serving as bonding agents (Winterkorn & Reich, 1962).

2.5 COMPACTION/DENSIFICATION

2.5.1 Introduction

Compaction is the oldest method of stabilisation and is a vital modern stabilising procedure for soils whether or not stabilisers are added to the mixture. Compaction is the process of mechanically densifying a soil, by pressing the soil particles together into intimate contact, as air is expelled from the soil. Achieving high density will generally provide high frictional strength in the material, although the two variables are not perfectly related (Ausroads, 1998; USACE, 2000). Densification brings the particles of the soil into more intimate contact, thereby increasing the frictional strength of the material (e.g. Winterkorn, 1975). Also, the smaller pore size decreases the rate of moisture entry into the stabilised material after curing, thereby adding to durability. Over compaction must be avoided, as there must remain sufficient water in the system to satisfy the hydration of both the soil minerals and the cement or lime that is added as a stabilising agent (Wolfskill et al., 1963; Winterkorn, 1975).

Well-graded soils are preferred because of their greater stability when compacted and because they can be compacted more easily. Maximum density is achieved for these soils when the packing is closest and the void volume is at a minimum, which is when (Winterkorn, 1975):

$$p = 100*[d/D]^n$$

Where p = % passing sieve of aperture d, D = maximum particle size, and

n = 0.45 to 0.50 for most materials

High densities can be achieved for some materials that are well graded from coarse to fine with n values of down to 0.33 (Ausroads, 1998). However, when n values are less than 0.33, the fines content is probably excessive. This will likely lead to undesirably low permeability and may lead to the development of positive pore water pressures (which increase internal stresses) both during compaction and afterwards. When n exceeds a value of 0.50 the material will tend to segregate and disintegrate. Thus, values of n between 0.33 and 0.50 are to be preferred (Ausroads, 1998).

Compaction may be used as the single method of stabilisation, or may be used in combination with granular stabilisation and/or chemical stabilisation. For rammed earth walls, soils are first mixed with chemical stabilisers (including lime, cement, and asphalt) before being compacted (rammed). However, compaction is used on its own commonly for the production of mud bricks (e.g. Osula, 1996; Ngowi, 1997), and can achieve reasonable compressive strengths. Lilley & Robinson (1995) studied the behaviour and ultimate strength of soil stabilised purely by manual compaction (ramming), with respect to its performance in load-bearing walls in construction in developing countries using local soils. Experiments on four types of lateritic soil from Ghana showed that the compressive strength of cubes of rammed earth increased within the first 7 days to about 2 MPa although in some cases this reduced to about 1.8 MPa after 28 days. Lilley & Robinson (1995) found that deformations within rammed earth under substantial maintained load (such as found in an earth dwelling) are likely to continue for at least 50 days after construction, although about 80 % of the final deformation is likely to occur within two weeks.

2.5.2 Influence of moisture on density

The density of a rammed earth soil depends many physical variables including the specific gravity of soil particles, particle size distribution, packing arrangement, moisture content, and compactive method and effort. However, the properties of a compacted soil mixture are influenced more by moisture content than by any other cause. Nearly all soils exhibit a similar relationship between moisture content and dry density when subjected to a given compactive effort (Figure 2.2). For each soil, a maximum dry density develops at an optimum moisture content for the compactive effort used. The optimum moisture content at which maximum dry density is obtained is the moisture content at which the soil becomes sufficiently workable under a given compactive effort to cause the soil particles to become so closely packed that most of the air is expelled (USACE, 2000). The maximum dry density corresponds to 100 % compaction for the given soil under the given compactive effort. For most soils (except cohesionless sands), when the moisture content is less than optimum, the soil is more difficult to compact. Beyond optimum, most soils are not as dense under a given effort because the water interferes with the close packing of the soil particles, and the air content remains constant even though the moisture content is increased.

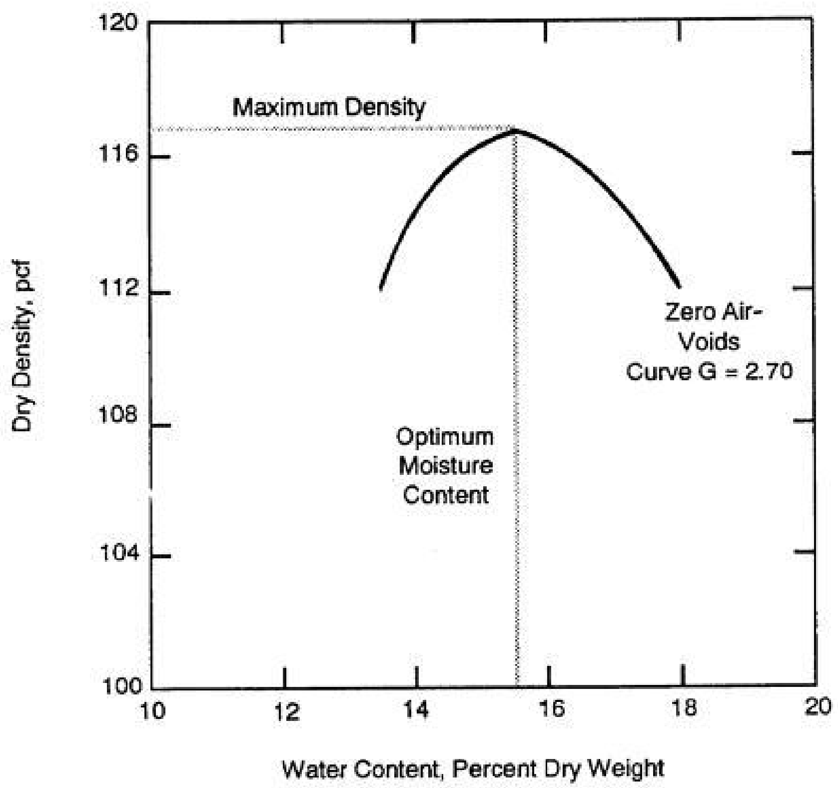
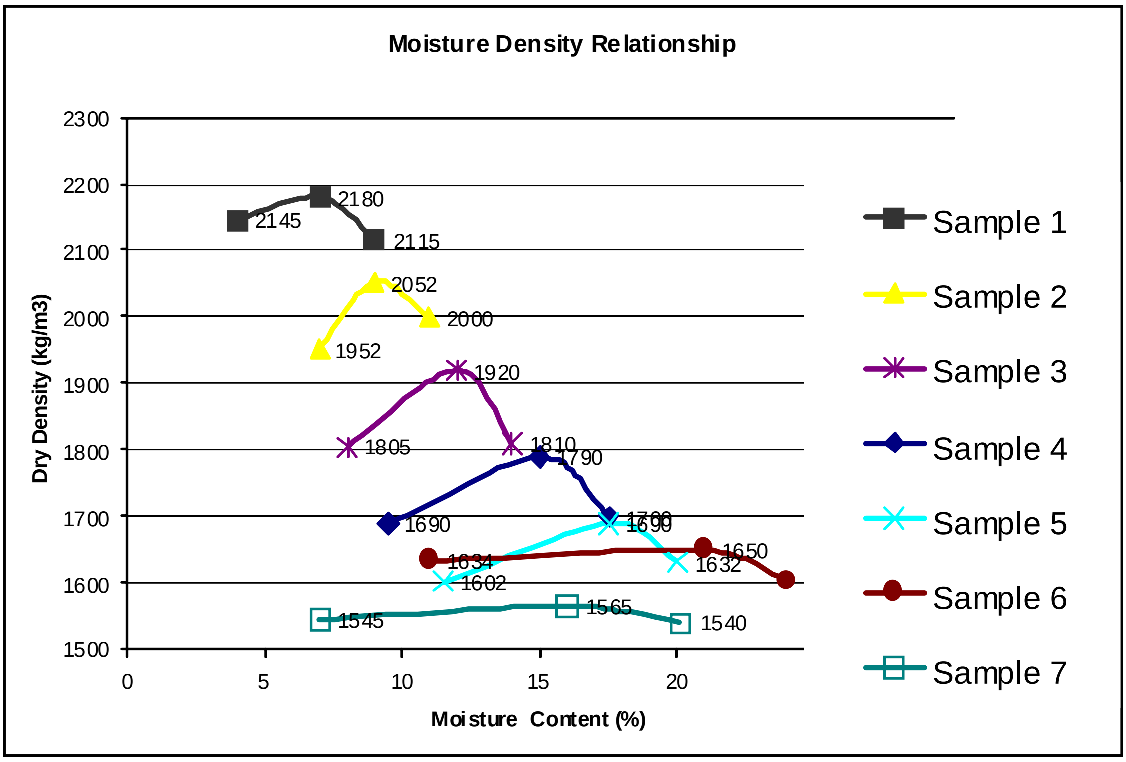


Figure 2.2: Typical moisture-density relationship, from USACE (2000).

Figure 2.2 shows the zero air-voids curve for the soil involved. This curve is obtained by plotting the dry densities corresponding to complete saturation at different moisture contents. The zero air-voids curve represents theoretical maximum densities for given water contents. These densities are practically unattainable because removing all the air contained in the voids of the soil by compaction alone is not possible (USACE, 2000). Typically, at moisture contents beyond optimum for any compactive effort, the actual compaction curve closely parallels the zero air-voids curve.

The nature of a soil itself has a great effect on its response to a given compactive effort. Moisture-density relationships for seven different soils are shown in Figure 2.3. Sandy soils generally have lower optimum moisture contents than clayey soils. The curves of Figure 2.3 indicate that soils with moisture contents less than optimum react differently to compaction. Moisture content is less critical for heavy clays (CH) than for the slightly plastic, clayey sands (SM) and silty sands (SC). Heavy clays may be compacted through a relatively wide range of moisture contents below optimum with comparatively small change in dry density. However, if heavy clays are compacted wetter than the optimum moisture content (plus 2 %), the soil becomes unworkable. The relatively clean, poorly graded sands also are relatively unaffected by changes in moisture. In contrast, granular soils that have better grading and higher densities under the same compactive effort respond to slight changes in moisture, producing more significant changes in dry density.



Sample No.	Description	% Sand	% Clay/silt	% LL	% PI
1	Well-graded loamy sand	88	12	16	NP
2	Well-graded sandy loam	72	28	16	0
3	Med-graded sandy loam	73	27	22	4
4	Lean sandy silty clay	32	68	28	9
5	Loessial Silt	5	95	26	2
6	Heavy clay	6	94	67	40
7	Very poorly graded sand	94	6	NP	

Figure 2.3: Moisture-density relationships for 7 different soils, adapted from USACE (2000).

Wright et al. (1996) examined the relationships between permeability, soil density, and moulding moisture content for soil used in earthworks. They performed laboratory permeability tests on four different soil types, and established that a minimum value of permeability is obtained at close to optimum moisture content and is maintained for moisture contents up to the liquid limit. Results obtained for predominantly granular soils showed that permeability is a function of density. However, for more cohesive soils, permeability is unaffected by changes in moisture content wetter than optimum. The implication for soil densification is that water content should be at or slightly above optimum moisture content. As the compressive strength of stabilised earth increases with density (e.g. Bryan, 1988), the moisture content and density of the soil-stabiliser must be carefully controlled during compaction. The density achieved in laboratory compaction cannot, however be used as the sole predictor of mechanical properties such as compressive strength, as these properties are also partially dependent on the type and quantity of stabiliser used.

Osula (1996) examined the effect of elapsed time after mixing on the optimum moisture content and density of mixtures stabilised with cement or lime. Optimum moisture contents as measured shortly after mixing were higher with greater percentages of stabiliser added, and maximum dry densities were lower. For the soil-cement mixes, the optimum moisture content increased on average by 7 % and maximum dry density decreased by 2 %. For lime-soil mixes, the equivalent figures were 14 % and 6 % respectively. Therefore, the quantity of stabiliser added to a soil will influence the optimum moisture content of the mixture and also the density. These results stress the importance of checking and controlling the moisture content of the soil-stabiliser mixture during compaction, to achieve the maximum density possible.

2.5.2.1 Compactive effort

The relationship shown in Figure 2.2 is valid only for one compactive effort. For each compactive effort used in compacting a given soil, there is a corresponding optimum moisture content and maximum dry density. If the compactive effort is increased, the maximum density is increased and the optimum moisture content is decreased (Winterkorn, 1975; USACE, 2000). If a soil is compacted under several different compactive efforts, a density-effort relationship can be developed for that soil. Also, different soils respond differently to varying compactive efforts. For example, a greater increase in compactive effort will be required to increase the density of a clay soil from 90 to 95 % of maximum density than is required to effect the same changes in the density of a sandy soil.

2.6 GRANULAR STABILISATION

Granular stabilisation refers to the improvement in one material by blending it with one or more other granular materials to effect a change in the gradation of the material (CSIRO, 1987; UN, 1992; Ausroads, 1998). Granular stabilisation is related to densification (discussed above), as the particle size distribution of the mixture is a prime determinant of the amount of compaction that can take place. Compaction creates an interlocking of soil-aggregate particles, and the gradation of the soil mixture must be such that a dense mass is produced when it is compacted (USACE, 2000). The strength of the material is a function of both the internal friction and cohesion of the resulting soil mixture. The internal friction component is generated by the size (> 0.075 mm) and shape of the coarser soil particles, with maximum frictional strength being achieved at maximum density (through compaction). Cohesion and associated properties such as shrinkage and swelling are due to the quantity and type of clay.

For the purposes of stabilisation, soils can be divided into those that when compacted have a granular bearing skeleton consisting of particles larger than 0.075 mm, and those without such a bearing skeleton (Winterkorn, 1975). Natural soils with a granular bearing skeleton yield stable structures of greater dependability than soils without a skeleton, because of their frictional strength component that is effective in both wet and dry conditions. Soils with enough of both granular constituents and silt-clay particles produce mechanically stable structures when compacted. Such materials can be artificially produced by combining granular soils that are deficient in clay binder with clay soils deficient in granular constituents.

Some natural soils have a combination of amount and activity of silt-clay constituents such that the tolerances for a stable bearing skeleton are slightly exceeded. In these cases, treatment by adding stabilising material to the soil prior to ramming, or spraying appropriate amounts of waterproofing agent onto the constructed walls, can decrease the swelling power of the silt-clay fraction sufficiently to avert the danger of instability under wet conditions (Winterkorn, 1975). On the other hand, if there is too small a filler-binder fraction for the desired cohesion and system permeability, this condition can be remedied by the addition of silt-clay or other binder materials.

CSIRO (1987) is cautious to recommend granular stabilisation on account that it is likely to be extremely labour intensive, and suggested it should be used only when there is no alternative solution to the stabilisation problem. However, granular stabilisation may be an option in developing countries due to low labour costs. The clay must be mixed in either wet or dry state but both are hard to achieve. If using the dry state all clods should be crushed down to a fine powder and then mixed with the sand to a desired mixture and samples of walls or bricks made for testing. If the wet state is selected then the clay must be placed in a tank of water and turned into a slurry. At this point the sand is added and mixed together with the liquid clay. After drying, the material is formed into a wall or bricks and testing is carried out to determine the proper mixture.

2.6.1 Proportioning

Mixtures of two or more different soils are difficult to generate in practical terms. A rough estimate of the proper proportions of available soils in the field is possible and depends on manual and visual inspection. The problem is to determine the proportions of the two (or more) materials that should be used to produce a satisfactory mixture. Trial combinations are usually made on the basis of the particle size analysis of the soils concerned, with calculations made to determine the gradation of the combined materials and the proportion of each component adjusted so that the gradation of the combination falls within specified limits (USACE, 2000). The plasticity index of the selected combination is then determined and compared with the specification. If this value is satisfactory, then the blend may be assumed to be satisfactory. If the plasticity characteristics of the first combination are not within the specified limits, additional trials must be made.

Table 2.5 and Figure 2.4 show an example (from USACE, 2000) of proportioning using two materials, materials "A" and "B". Material "B" is the local (coarse) soil and material "A" is a borrowed soil with a greater range of particle sizes. Table 2.5 reports the particle size qualities of these soils, together with the liquid limit and plasticity index of each, as well as the desired grading and plasticity of the combination. The data from Table 2.5 are graphed in Figure 2.4, which shows the gradations of the two materials plotted on the left and right hand axes, and the upper and lower limits of the gradation required of the resulting combination. A line is drawn across the graph, connecting the percent passing of material A with the percent passing of material B for each sieve size. The point where the upper and lower limits of the gradation requirements intersect the line for each sieve size is marked. The acceptable ranges of material A to be blended with material B is the widest range that meets the gradation requirements for all sieve sizes. The shaded area of the graph represents the combinations of the two materials that will meet the specified gradation requirements. The boundary on the left represents the combination of 44 % material A and 56 % material B. The position of this line is fixed by the upper limit of the requirement relating to the material passing the Number 200 sieve (15 %). The boundary on the right represents the combination of 21 percent material A and 79 % material B. This line is established by the lower limit of the requirement relative to the fraction passing the Number 40 sieve (15 %). Any mixture falling within these limits satisfies the gradation requirements.

Mechanical Analysis			
Sieve Designation	Percent Passing, by Weight		
	Material A	Material B	Desired
1 inch	100	100	100
3/4 inch	92	72	70-100
3/8 inch	83	45	50-80
Number 4	75	27	35-65
Number 10	67	15	20-50
Number 40	52	--	15-30
Number 200	33	1	5-15
Plasticity Characteristics			
Liquid limit	32	12	≤ 28
Plasticity index	9	0	≤ 6

Table 2.5: Gradation and plasticity data for two soils to be combined to produce the desired mixture.

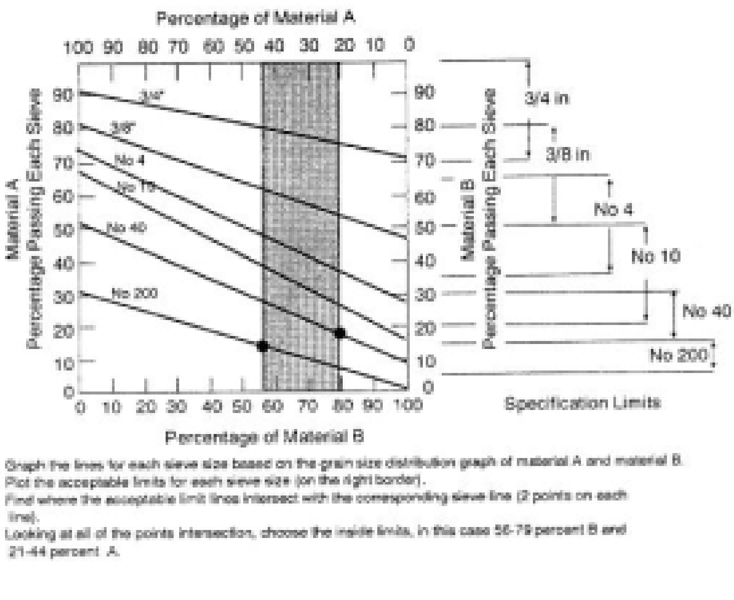


Figure 2.4: Graphical method of proportioning 2 soils to meet gradation requirements, from USACE (2000).

2.7 SOIL-LIME STABILISATION

2.7.1 Introduction

Lime is the oldest soil stabilising agent known, and was used as roadway stabiliser by the Romans and other early civilisations (Jarrige, 1989). Lime stabilisation of clayey base materials has been used in Australia and throughout the world for many years in the construction of airport runways, roads, and parking areas. Clay soils in particular can be stabilized by the addition of lime to enhance their engineering properties, thereby producing an improved construction material (e.g. Bell, 1996; Ausroads, 1998).

The term lime - as used by those in the building industry - refers to oxides and hydroxides of calcium and magnesium, but not to carbonates. There are various types of lime commercially available, which include the following: Calcia (high-calcium quicklime) CaO; Hydrated high-calcium lime $Ca(OH)_2$; Dolomitic lime $CaO + MgO$; Dolomitic lime normal hydrated or monohydrated $Ca(OH)_2 + MgO$; and dolomitic lime pressure hydrated or dihydrated $Ca(OH)_2 + Mg(OH)_2$. Quicklime is the most concentrated form of lime. Hydrated lime results from the addition to quicklime of sufficient water (about 30 % by mass) to satisfy its chemical affinity for H_2O.

The higher the magnesia content of the quick or hydrated lime, the lower is the water affinity and the heat developed in mixing with water. The quick as well as the hydrated limes absorb and react with carbon dioxide from the air and form $CaCO_3$. This reaction is no longer useful either for the making of common mortar or for the alkaline reaction with finely divided silica to product hydraulic cements. For this study, only hydrated lime $Ca(OH)_2$, has been used. This is because although various forms of lime have been successfully utilised as a soil stabilising agent for many years, the most commonly used product is hydrated, high calcium lime.

There is some debate as to whether calcitic lime or monohydrated dolomitic lime is the more effective additive for rammed earth wall stabilisation. Studies in USA by the Portland Cement Association have shown that high calcium limes are generally more effective for modifying soil plasticity (Thompson, 1967), and that dolomitic limes produce higher cured strength. The investigation, however, concluded that "most soils do not respond preferentially to dolomitic monohydrate or hydrated calcitic lime stabilisation for strength improvement." Therefore, it is likely that both high-calcium and monohydrated dolomitic lime is equally satisfactory for use to stabilise rammed earth walls.

2.7.2 Mechanisms of lime stabilisation

There may be as many as five mechanisms for the stabilisation of soil by lime (UN, 1992):

1. Water absorption: Quicklime undergoes hydration using added water or soil moisture, releasing heat in the process.
2. Cation exchange: Calcium ions from the lime are replaced by exchangeable cations from the soil constituents, including potassium and magnesium. The amount of exchange is a function of the soil's cation exchange capacity.
3. Flocculation and accretion: Soil particles flocculate and accrete due to the cationic exchanges and increased electrolyte activity in the pore water. In effect, particle size distribution may be modified.
4. Carbonation: Added lime reacts with atmospheric CO_2 to form weak carbonated cements.
5. Pozzolanic reaction: This is the dominant reaction in lime stabilisation of soil. The strength of the resulting material is largely a function of the dissolution of clay minerals in the alkaline environment created by the lime and the recombination of clay-derived Si and Al with Ca to form cementitious aluminium and calcium silicates. The pozzolanic reaction is discussed in more detail further below.

Once initiated, these reactions (apart from the pozzolanic reaction) take place rapidly and produce immediate changes in soil plasticity, workability, and the immediate uncured strength and load-deformation properties (e.g. Osula, 1996). These reactions allows for very easy ramming of the soil and a very good compaction rate. If the appropriate clay constituents are present in the soil, a pozzolanic reaction will occur over a longer time frame, as detailed in Section 2.7.3.

Reaction rates are proportional to the surface: volume ratio of the soil particles, and this ratio is inversely proportional to particle size. The rate of reaction is therefore partly a function of the amount of the smaller fractions of the siliceous particles of the soil. Moreover, the higher the proportion of silica and the smaller the proportion of aluminium and iron sesquioxides in the smaller particle size fractions, the greater is their reactivity with $Ca(OH)_2$. Because the rate of reaction is a function of the concentration of the components, and since the reactions under consideration occur in an aqueous medium, the solubility of $Ca(OH)_2$ in water is also of significance (Winterkorn, 1975). The solubility is relatively low; around 1.65 grams of $Ca(OH)_2$ per litre at normal temperature and pressure. In a normal clay soil, this amount of $Ca(OH)_2$ in solution is quickly exhausted to satisfy the cation exchange capacity of the clay fraction even though there is no excess solid $Ca(OH)_2$ stored in the system to balance the $Ca(OH)_2$ taken out of solution by the silicate surfaces. To assure the greatest possible rate of solution of the solid $Ca(OH)_2$, this substance should be in a colloidally dispersed state.

The reaction between soil silica and/or alumina and lime forms various types of cementing agents. These cementing agents are generally regarded as the major source of the strength increases noted in lime-soil mixtures. (Thompson, 1967). The pozzolanic reaction results in the formation of various cementing agents that increase mixture strength and durability. Pozzolanic reactions are time dependent. Strength development is gradual but continuous for long periods of time, amounting to several years in some instances. The rate of reaction depends not only on the effective concentration of the reactants but also on the temperature of the system (the rate of reaction doubles with every 10 °C increase in temperature).

The solubility of $Ca(OH)_2$ decreases with increasing temperature while that of SiO_2 increases. Therefore, there is a playoff between the two compounds. In the commercial production of sand-lime bricks, 4-10 % of $Ca(OH)_2$ is mixed with quartz sand and reacted at a steam pressure of 150 psi and a temperature of 185 °C. Only a few hours of such treatment are required to produce bricks of a strength in the order of 4000 psi, while lower temperatures (< 15 °C) retard the reaction. (Anday, 1963).

When a significant quantity (> 1 %) of lime is added to a soil, the pH of the soil-lime mixture is elevated to approximately 12.3, the pH of saturated lime solution. This is a substantial pH increase compared to the pH of natural soils (USACE, 2000). Soil reaction products are forms of hydrated calcium silicates and hydrated calcium aluminates (e.g. Keller, 1957). A wide variety of hydrate forms can be obtained, depending on reaction conditions, curing time, and temperature.

2.7.3 Soil-lime pozzolanic reaction

Lime is only effective when the material to be stabilised contains clay particles or pozzolanic materials that react with lime. The reactions between lime, water, and various sources of soil silica and alumina to form cementitious materials are referred to as soil-lime pozzolanic reactions. Long-term strengthening by the pozzolanic reaction occurs in a highly alkaline environment (pH >12.3) which encourages dissolution of the clay, particularly at the surfaces, enabling the formation of cementitious products that contain calcium silicates and aluminates (Ausroads, 1998). The process is quite slow. Possible sources of silica and alumina in typical fine-grained soils include clay minerals, quartz, feldspars, micas, and other similar silicate or alumino-silicate minerals, either crystalline or amorphous in nature. Since the clay minerals and amorphous materials are the only important sources in most soils, the simple addition of hydrated lime to water can suffice to establish the desired composition. The critical question is whether or not the rate of reaction is high enough for the final cementing compounds to be formed within a reasonable period of time. The answer is that they are high enough in mixtures of clay soils with quick or hydrated lime and water. There is thus a foundation for lime stabilisation of clay soils with or without an admixture of reactive siliceous compounds, such as pozzolans, infusorial earth, or fly-ashes.

The extent to which soil-lime pozzolan reaction proceeds is influenced primarily by natural soil properties. With some soils, the pozzolan reaction is inhibited, and cementing agents are not extensively formed. Thompson (1966) termed “reactive” those soils that react with lime to produce a substantial strength increase (> 0.345 MPa following 28-day curing at 22.8 ^{o}C), whereas soils were termed “non-reactive” if they displayed limited pozzolan reactivity (< 0.345 MPa strength increase).

The main factors that control the development of cementitious materials in a lime-treated soil are the inherent properties and characteristics of the soil (Harty, 1970). If a soil is non-reactive, extensive pozzolan strength development will not be achieved despite lime type, lime percentage, or curing conditions of time and temperature. Certain soil properties and characteristics influence the ability of the soil to react with lime to produce cementitious materials. These include soil pH, organic carbon content, natural drainage, the presence of excessive quantities of exchangeable sodium, clay mineralogy, degree of weathering, presence of carbonates, extractable iron, silica-sesquioxide ratio, and silica-alumina ratio (Harty & Thompson, 1973).

2.7.4 Construction considerations

The major objective of the mixture design process is to establish an appropriate lime content for construction. The primary variable that can be altered is lime percentage, since the inherent properties and characteristics of the soil are fixed (unless granular stabilisation is performed prior to the addition of lime). Because of the many varied applications of lime stabilisation of soils, design procedures have been developed by the lime industry. The general principle of soil-lime mixture design is that the mixture should provide satisfactory performance when rammed correctly. It would seem that a wide range of soil-lime stabilisation mixtures of varying quality could be successfully used to accomplish the goal of structurally sound rammed earth walls. Generally, lime stabilisation contents are based on an analysis of the effect of various lime percentages on selected engineering properties of the soil-lime mixture. For structural engineering considerations, cured strength is the most appropriate property to consider.

In the field, the most appropriate manner in preparing the soil-lime is by mixing the dry soil with lime. After thoroughly mixing the material, the water is added. The mixture should be a close as possible to optimum moisture content. A "speedy" moisture meter or other devices (as explained in Section 6.9.1.5) will adequately do the job in the field, keeping the rammed earth at the correct moisture content.

2.7.5 Influence on soil and earth properties

Treatment with quicklime or hydrated lime is very beneficial for clay rich soils in particular, as both strength and workability are improved. A substantial reduction in plasticity (a reduced plastic limit and plasticity index, and a decreased shrinkage limit) is effected by lime treatment (e.g. Osula, 1996). The addition of 2-3 % of quicklime to a soil quickly reduces plasticity of the soil by hydration (dries the soil), and breaks up lumps (UN, 1992). Generally, high initial plasticity index and clay content soils require greater quantities of lime for achieving the non-plastic condition, if indeed it can be achieved. The improved level of workability expedites subsequent manipulation and ramming of the treated soil in rammed earth wall construction (UN, 1992). However, lime is not effective in low cohesion materials unless pozzolanic additives are added.

Shrinkage associated with the loss of moisture from the stabilised soil is important when considering the problem of shrinkage cracking of the materials and reflective cracking through rammed earth walls. Testing by Dempsey & Thompson (1969) indicated that lime-treated soils decrease shrinkage potential. Their field moisture content data for lime-treated soils suggest that the moisture content changes in the stabilised material are not larger than 3 %, and that the in-situ water content stabilises near optimum. Further details on the effect of lime on shrinkage are described in Section 2.8.4.

Soil swell potential and swelling pressures are normally significantly reduced by lime treatment for rammed earth wall construction. The reduced swell characteristics are generally attributed to the decreased water affinity of the calcium saturated clay and the formation of a cementitious matrix that can resist volumetric expansion. In considering additive treatments for swell control, Thompson (1969) and Mitchell & Raad (1973) concluded that lime is the most effective additive for stabilisation of expansive soils. It is apparent that the immediate strengthening effects of lime treatment are substantial. In addition, as curing progresses and the soil-lime pozzolanic reaction proceeds, the soil-lime mixture will develop much higher levels of strength and stiffness characteristics.

For a given compactive effort, soil-lime mixtures have a lower maximum dry density and higher optimum moisture content than the untreated soil, due to the fine-grained characteristic of hydrated lime (Osula, 1996; Ausroads, 1998). Maximum dry density reductions of 0.1-0.2 t/m^3 and optimum water content increases of 2-4% are common. If a mixture is allowed to cure and thus gain strength in a loose state before ramming, further reductions in maximum dry density occur. Increases in optimum moisture content may also be noted as compared to untreated soil.

Unconfined compressive strength increases considerably with the use of lime to stabilise a soil (Metcalfe, 1963; Croft, 1968; Akpokodje, 1985; Symons, 1999). Akpokodje (1985) investigated the stabilisation of Australian arid zone soils using lime. The soils included a sandy loam (59 % sand, 28 % silt, 13 % clay), a silty loam (15 %, 75 %, 10 %), and a clay loam (36 %, 27 %, 37 %). For the clay loam and sandy loam, a lime content of 2 % gave a 7-day compressive strength of about 0.7 MPa, but increasing the percentage to 4 % gave a compressive strength of around 1.2-1.3 MPa (Figure 2.5). Further increases in lime up to 12 % appeared to yield no additional strength to the samples.

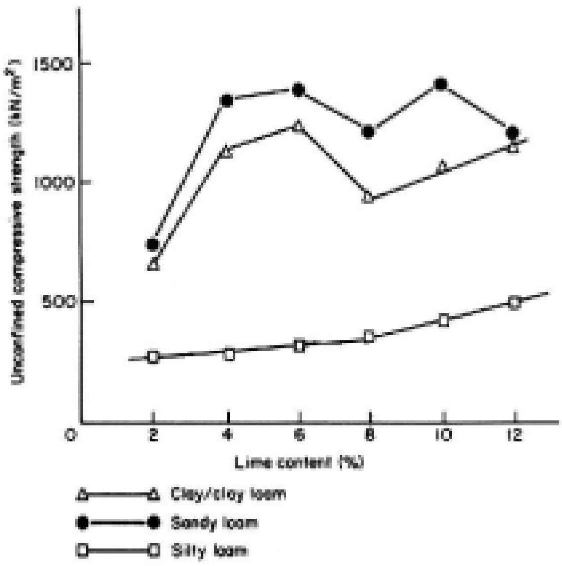


Figure 2.5: Relationship between compressive strength and lime content for 3 Australian arid zone soils after Akpokodje (1985).

Lime stabilised materials are usually assessed at 7 days and 28 days. However, high lime contents will not necessarily promote high early strengths. Figure 2.6 shows the variation in compressive strength as a function of lime percentage and time. It indicates that early strength gains (7-day strength) are likely to be small, but are significant by 28 days.

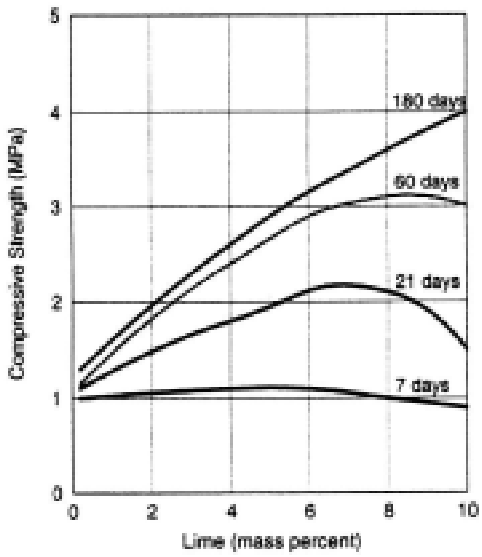


Figure 2.6: Compressive strength variation with lime percentage and time, from Ausroads (1998).

Bell (1996) assessed the improvements made to clay soils and glacial tills through the addition of lime as a stabiliser, with emphasis on three of the most frequently occurring minerals in clay deposits - kaolinite, montmorillonite, and quartz. With the addition of lime, the plasticity of montmorillonite was reduced whereas that of kaolinite and quartz was increased. Bell found that all materials experienced an increase in their optimum moisture content, a decrease in their maximum dry density, and an increase in their strength, on addition of lime.

2.8 CEMENT STABILISATION (SOIL-CEMENT)

2.8.1 Introduction

Soil-cement is the reaction product of a mixture of soil, portland cement, and water, compacted to high density (Winterkorn, 1975). Cement stabilisation resembles lime stabilisation in many ways, except with cement, pozzolan material is present in the cement initially and need not be derived from the soil itself. In predominantly coarse-grained soils, the cement paste bonds soil particles together by surface adhesion between the cement gel and particle surfaces. In fine-grained soils, the clay component may also contribute to stabilisation through solution in the high pH environment and reaction with the $Ca(OH)_2$ from the cement. However, cement stabilisation works best on sandy soils (Wolfskill et al., 1963), partly because portland cement does not mix easily with soil and breaking down the natural soil aggregates in a clay soil to ensure good distribution of stabiliser is very difficult.

2.8.2 Mechanisms of cement stabilisation

The precise way in which cement stabilises materials differs between soil types. Non-cohesive soils have particle sizes larger than cement grains and can therefore be coated with cement. The cement may react with itself or with the sandy granular skeleton (UN, 1992). The hydrated cement bonds soil particles at points of contact, leading to strength increase of the soil. The increase in strength is a function of the number of points of contact between soil particles (which itself is a function of the gradation of the soil) and of compaction to bring soil particles into the closest possible contact (Bell, 1975). Non-cohesive soils can be made more suitable for cement stabilisation by adding silt and clay to improve gradation.

In cohesive soils, many particles are finer than cement grains and thus cannot be coated by cement (Bell, 1975). However, following compaction the cement hydrates leading to strength increases by forming a skeletal structure of its own accord. In addition, a three-phase reaction with the clay occurs: 1. Hydration promotes formation of cement gels on the surface of clay aggregations, and the lime that is freed during hydration reacts with the clay; 2. Clay aggregates are disaggregated by hydration and are penetrated by cement gels; 3. Cement gels and the clay aggregates become intimately bonded (UN, 1992). This results in both an inert sand-cement matrix and a matrix of stabilised clay in the new structure.

Hydraulic cements are mineral powders of such composition that they react with water to form strongly cemented systems. The common hydraulic cements, which are mixtures of calcium silicates and aluminates, include portland, natural, slag, and alumina cements. The range in primary chemical composition (SiO_2, CaO, Al_2O_3, and Fe_2O_3) of these compounds are shown in Figure 2.7, which also shows the compositional ranges of quicklime, hydraulic lime, and pozzolan cement.

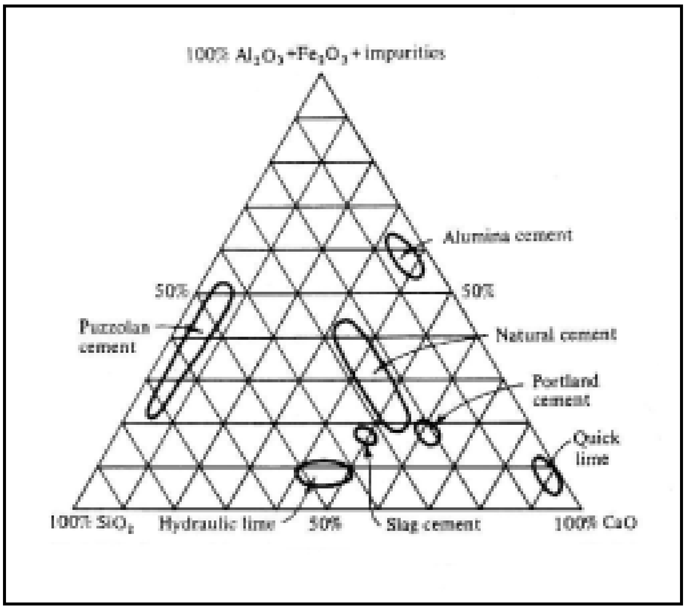


Figure 2.7: Composition of cements by weight, after Murphy (1957) in Winterkorn (1975).

From the location of their respective compositional ranges, it might be concluded that it is possible to make a Portland cement out of a mixture of hydraulic lime and pozzolan. However, Portland cement is legally defined not only by its elementary composition but also by the formation of definite Ca-silicate and aluminate compounds at sintering temperatures and by pulverisation of the hard clinker to a certain particle size range. Within this compositional definition of Portland Cement fall several types of cement that are used for particular purposes (Figure 2.8).

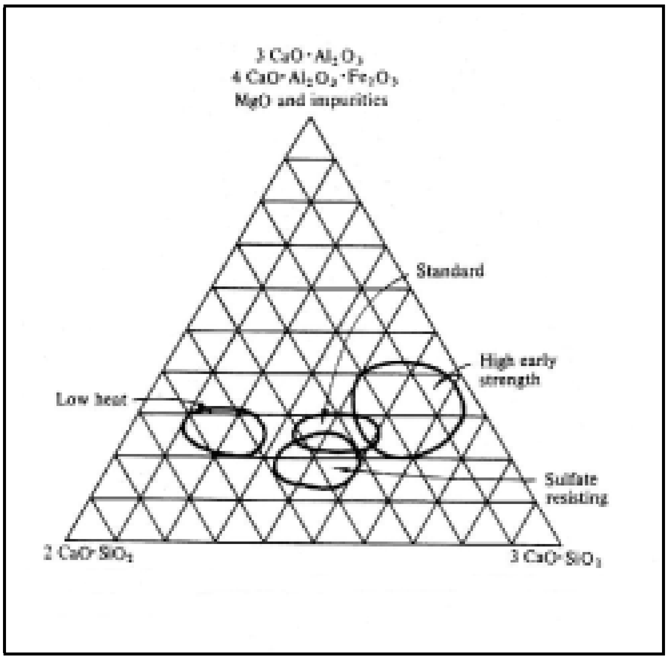


Figure 2.8: Composition of the various types of portland cements by weight, after Murphy (1957) in Winterkorn (1975).

Since the shear resistance of a purely granular soil is directly proportional to the normal stress on the shear plane of rammed earth walls and since this stress tends toward zero in the unloaded surface layer, cementation of the component particles is required for mechanical stability. For optimum strength properties, the volume of the cementing material should interfere as little as possible with the packing of the granular components. Cemented systems have the generic names of "mortar" if the particles that form the granular skeleton are of sand size and "concrete" if the majority (by volume) of the granular constituents are of gravel size and larger. Cementing agents can consist of any one or any combination of bonding and/or waterproofing materials, and comprise a large number of natural and artificial inorganic and organic substances. A sandy soil, possessing naturally or by admixture the right amount and quality of silt-clay material can be considered as a clay mortar and the corresponding clay-sand-gravel system as a clay concrete.

The relationship between the strength of a granular system and the ratio of the volume of cement to the volume of the pore space is given by (Feret 1892) :

$$S = k\left(\frac{c}{l - s}\right)^n$$

where:

S = compressive strength of the cemented system
k = essentially the strength of the cement but influenced by the granulometric and material factors
c = absolute volume of the cement
s = absolute volume of the inert granular components
n = constant depending on material and geometric factors

This relationship, which was originally discovered in studies of lime and cement mortars, became the water/cement ratio for plastic portland cement concrete (Abrams, 1918). The relationship holds for the stabilisation of all soils that possess or can be given a granular bearing skeleton.

Cementing agents such as portland cement have an average particle size of around 0.01 mm. This means that the soil should be in the form of aggregations that are larger than 1 mm if significant interference of the packing structure of the soil is to be avoided (Winterkorn, 1975). For a highly cohesive soil compacted at optimum moisture content to its maximum density, the porosity is about 40 %. A question arises as to what extent must this pore space be filled to promote effective stabilisation. Filling this space completely with stabiliser to produce a non-porous system would be both unrealistic and uneconomical due to the large amounts of cementing agents required. Therefore, the stabilised soil is likely to be of capillary or subcapillary porosity (Winterkorn, 1975).

The Portland cement compositional range gives the most desirable end product for building after reaction with water. Portland cement, containing the correct chemical components and the standardised particle size range, will attain desired strength and durability properties within a few days, or at most a few weeks. The ultimate equilibrium products depend only on the composition, including the respective concentrations and the temperature and pressure conditions of the system. However, the times required for reaching equilibrium can vary tremendously depending upon the particular components that make up the initial system. If given sufficient time, the hydration products of portland cement can be duplicated by combining, at normal environmental temperatures, two or more of the primary components, calcium oxide CaO, SiO_2, and Al_2O_3 Fe_2O_3 in the correct proportions in the presence of water.

2.8.3 Properties of cement-soil

The properties of cement-stabilised soils are strongly dependent on density, water content, and confining pressure. The development of generalised property relationships is further complicated by the fact that cement content, and curing time and conditions, are also important. Thus, measurement of properties under one set of conditions may yield data of limited value for other conditions. In general, for a given cement content, cohesionless soils or cement mixtures with greater densities are associated with greater strengths (e.g. Bryan, 1988; Walker, 1995). Both water content at compaction and compaction method may be important in cohesive soil and cement-soil stabilisation.

2.8.3.1 Optimum moisture content and maximum dry density

Cement additions to a soil generally cause some change in both the optimum water content and maximum dry density for a given compaction, although the direction of this change is not always predictable. The flocculating action of the cement tends to give an increase in optimum water content and a decrease in maximum density, which counteracts the higher density of the unhydrated cement compared with the soil, which promotes an overall increased density. The gradation of the unhydrated cement relative to that of the soil may also be important because it influences the packing of particles. A delay during construction between mixing the soil-cement and ramming the material may lead to a decrease in both density and strength for a fixed compactive effort (West, 1959). If, however, compaction effort is increased so that the original density is obtained, and provided no significant amount of cement hydration occurs during the delay period, then no strength loss is observed.

2.8.3.2 Shrinkage of cement-soil

Cement stabilisation requires sufficient water for hydration of the cement and silt-clay particles, and for workability of the material. As the stabilised material dries and cures, water is lost which results in shrinkage cracks, which are more severe with soils with high water affinity (Winterkorn, 1975). There are therefore limits to the types of soils that can be satisfactorily stabilised with portland cement.

Cement-soil stabilisation will usually exhibit some shrinkage on curing and drying in an amount that depends on several factors, including cement content, soil type, water content, degree of compaction with rammers, and curing conditions. Some amount of shrinkage cracking should be considered inevitable in soil-cement stabilisation. Field observations on many rammed earth projects show the cracks to be from three to six millimetres wide at a spacing of three to six metres. The smaller crack spacings are usually associated with the higher clay content soils. Because of the likelihood of shrinkage cracks in soil-cement stabilisation, it is important to consider the structural load requirements and sealing of rammed earth walls.

Akpokodje (1985) demonstrated that both swell and shrinkage of soils is decreased by the addition of cement. Lowest swell and shrinkage values are associated with higher cement contents. For the sandy loam tested by Akpokodje, a quantity of cement of 6 % reduced the natural soil swell from 2 % to 0.1 %, and the swell of a clay loam from 6 % to 0.7 %. The shrinkages of these soils for the same cement quantity were reduced from 1 % to 0.2 % and from 6 % to 2 % respectively.

2.8.3.3 Influence on compressive strength

The greatest compressive strengths using cement as a stabiliser are obtained with gravels and sandy soils rather than with silts or clay (UN, 1992). In general, the amount of cement stabiliser as recommended by UN (1992) is 6 %, although some soils may require only 3 % and others may need 12 %. Generally, the compressive strength is dependent on the amount of cement used (e.g. Croft, 1963; Akpokodje, 1985; Ngowi, 1997). Cement-soil should be compacted immediately after mixing, at a moisture content close to optimum. Soil with a high clay content should be compacted slightly above optimum moisture content, while sandy soils should be compacted slightly drier than optimum moisture content (UN, 1992; USACE, 2000).

Adding cement to a soil increases its compressive strength markedly (Croft, 1968; Akpokodje, 1985; Bryan, 1988a; Walker, 1995). Akpokodje (1985) examined the effect on soil strength of stabilising a clay loam, a sand loam, and a silty loam with various quantities of cement. He found that compressive strength increased as a linear function of cement content between 0 and 12 %. Figure 2.9 shows the results for the 3 soil types. The 7-day compressive strength for sandy loam is highest, with values of 1.3 and 2.7 MPa for 2 % and 6 % cement respectively. Figure 2.9 also shows that sandy loam strength increases more rapidly as a function of cement content than do clay loam and silty loam. Similar effects on stabilised strength have been reported by Croft (1968), Bryan (1988a), and Walker (1995), and further details on this aspect of cement stabilisation are given in Section 2.15.

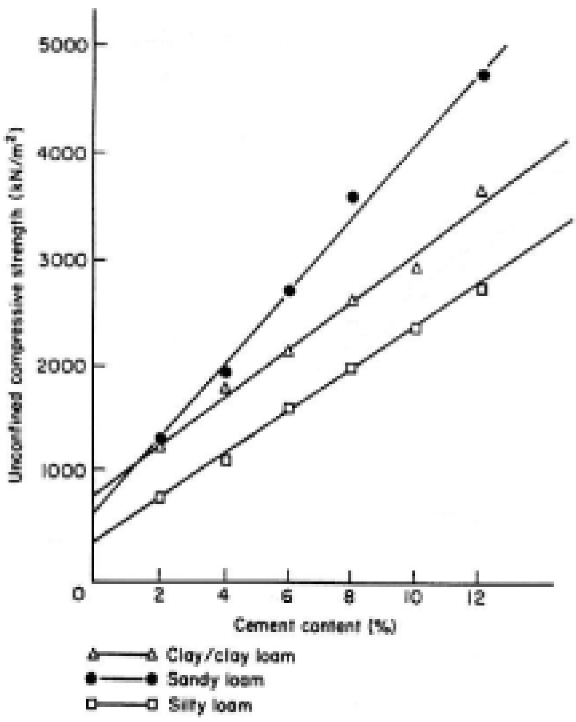


Figure 2.9: Relationship between unconfined compressive strength and cement content for 3 soils from the Australian arid zone (adapted from Akpokodje (1985)).

Cement addition to a soil also alters the particle size distribution of the material. Osula (1996) demonstrated that the gradation is altered chiefly by increasing the proportion of particles that are greater than 0.06 mm in size.

2.8.4 Lime-Cement comparisons

Spangler & Patel (1949) reported the results of an experimental study of the effect of various quantities of lime (CaO) and portland cement on the properties of a clay gumbotil soil from Iowa. Adding lime of 4 % by soil dry mass reduced the natural soil liquid limit from 69 % to 54 %, and the plasticity index from 42 % to 10 %. The equivalent amount of cement decreased the liquid limit to 55 % and the plasticity index to 20 %. Lower amounts of lime between 1-3 % were more effective than the equivalent amounts of cement at reducing both liquid limit and plasticity index. For linear shrinkage, the natural soil was 14.3 %, but adding 2 % lime and 4 % lime reduced this figure to 6.5 % and 4.0 % respectively. For soil mixed with 2 % cement the equivalent linear shrinkage was 12.5 %, and for 4 % cement was 7.5 %. These results also, therefore, suggest that lime is more effective for reducing the linear shrinkage than cement. Osula (1996) reported results in support of these earlier findings. He found that lime addition decreased plasticity index by between 1.3 and 2 times that of cement addition.

Osula (1996) also evaluated the performances of cement and lime with respect to the effect on the maximum dry density of stabilised material. The results showed that lime reduced the density of the soil material slightly more than did cement, both immediately after adding stabiliser and at various times (up to 3 hours) following. This result extended across soils of both high and low clay-content soils. Further details on lime-cement comparisons are given in Section 2.15.2.

2.9 ASPHALT STABILISATION

2.9.1 Asphalt as a stabiliser for soil

Asphalt is one of the oldest adhesives known. As early as 3800 BC, asphalt was being used as mortar for building stones and paving blocks. The use of asphalt stabilisation for rammed earth walls began in the early 1900's, but has never been popular with earth wall practitioners. This is because of a general lack of knowledge about soil requirements and mixing procedures, and an unfavourable reaction to the smell and colour of the additive, and the difficulty of cleaning equipment.

The mechanisms involved in the stabilisation of soils and aggregates with asphalt differ greatly from those involved in cement and lime stabilisation. The basic mechanism involved in the asphalt stabilisation of fine-grained soils is a waterproofing phenomenon (CSIRO, 1987). Soil particles of soil agglomerates are coated with asphalt, resulting in a membrane that prevents or impedes the penetration or adsorption of water, which, under normal conditions, would result in a decrease of compressive strength. In addition, asphalt stabilisation can improve durability characteristics. Since the soil particles or aggregates are coated with water-repelling asphalt film, the soil is resistant to the detrimental effects of water such as volume change due to alternating wet-dry cycles (Winterkorn, 1975; Al-Homoud et al., 1996).

In non-cohesive materials, such as sands and aggregates, two basic mechanisms are active, waterproofing and adhesion. The asphalt coating on the cohesionless materials provides a membrane that prevents or hinders the penetration of water and so reduces the tendency of the material to lose strength. In addition, the aggregate particles adhere to the asphalt and thus the asphalt acts as a binder or cement, acting to increase strength by increasing cohesion. Besides the benefits cited for increased compressive strength, asphalt stabilisation would prevent surface water from penetrating into the rammed earth wall structure.

Emulsified asphalt soil stabiliser consists of asphalt globules of microscopic size, which are surrounded by and suspended, in a water medium. A few drops of asphalt stabiliser perceptibly colours a large amount of clear water and remains suspended without any coalescence of the asphalt, however, the stabiliser must be stored and used at a temperature above freezing. Freezing causes the asphalt to settle out of emulsion and it becomes unusable for rammed earth wall construction. When the stabiliser is mixed into a clay-bearing soil in the presence of enough water, the water carries the asphalt globules into direct contact with the surfaces of the clay particles. Since the water-carrying capacity of the clay many times exceeds that of the sand and small rock particles, practically all of the asphalt in the stabiliser is brought into close contact with the clay (USACE, 2000). As the water evaporates, the asphalt globules are drawn into very thin films so dense that the asphalt forms a practically solid coating, which is not removable from the surface of each clay particle. The amount of asphalt required to coat the clay particles is minimal when compared to most other methods of coating particles.

Fine-grained soils may be stabilised with asphalt, depending upon the plasticity characteristics of the soil and amount of material passing the 0.075 mm sieve. Due to the extremely high surface area of the finer soil particles, a large % of asphalt would be required to coat all of the soil surfaces. Since this is virtually impossible, agglomerations of particles are coated with economical amounts of asphalt (Herrin, 1960). The amount of material passing through a 0.075 mm sieve should be less than 25 %. In addition, the plasticity index should be less than 10 % to ensure that adequate mixing is possible. If proper attention is not paid to mixing the soil-asphalt, the plastic fines may swell upon contact with water, resulting in a substantial loss of compressive strength.

Cohesionless soils (plasticity index < 6) are suitable for asphalt stabilisation (UN, 1992; USACE, 2000). These soils could be classified as sand, or sand-aggregate types. Asphalt-stabilised materials are made with dense-graded aggregates and have a very high strength. Specifications and criteria for bituminous-stabilisation rammed earth soils and aggregates are almost exclusively based on compressive strength (e.g. Ngowi, 1997; USACE, 2000).

Soil moisture levels affect the distribution efficiency of the asphalt emulsion through the soil. Dry soil causes the emulsion to break prematurely, resulting in asphalt blobs and an uneven distribution of emulsion. The tendency of the emulsion to break early reduces with increasing moisture content, and in addition, mixing times can be extended which favours a more even distribution (Ausroads, 1998). However, excessive moisture levels are unfavourable for stabilisation, and aeration or liming may be needed.

2.9.2 Types of asphalt emulsion

Asphalt emulsion consists of approximately 55% asphalt, 43-44% water, and 1-2% emulsifying agent used to hold the water and asphalt together. As the emulsions are fluid, they are easily mixed with soil. Australian Standard AS 1160 allows for two classes of asphalt, categorised on the basis of the charge of the suspended particles. These two types are anionic asphalt emulsion (negatively charged particles of asphalt, basic emulsion with pH > 7) and cationic asphalt emulsion (positively charged particles of asphalt, acidic emulsion with pH < 7). Each type is also prepared in two grades, slow setting (SS) and rapid setting (RS). The cationic type is more common and compatible with a wide variety of soil types. The type of asphalt emulsion used for stabilisation for rammed earth walls is an SS-1h or a cationic SS-1h, where 1h stands for one-hour setting time. Rapid setting emulsions cannot be used for rammed earth stabilisation as the water is released before the asphalt has completely spread through the interstices of the soil (CSIRO, 1987).

The type of asphalt to be used depends on the type of soil to be stabilised, and the weather conditions. Asphalt emulsion manufactured with Class 170 asphalt is satisfactory for soils with 0-10% silt-clay, but for soils with higher percentages of fine material (15-25 % silt-clay), softer asphalt (Class 50) may need to be used (Ausroads, 1998). Asphalts are affected to some extent by temperature, but a grade of asphalt suitable to the prevailing climate should be selected. Generally best results are obtained when the most viscous liquid asphalt that can be readily mixed into the soil is used (USACE, 2000).

2.9.3 Effect on soil properties

Ngowi (1997) examined the effects of various stabilisers, including asphalt, on the stabilised strength and disintegration of pressed earth bricks. The bricks stabilised with asphalt were only one third as strong as those stabilised with cement, and one half as strong as those stabilised with lime. In fact, the asphalt-stabilised bricks had lower compressive strength values than did the control bricks stabilised by pressing only. Asphalt-stabilised bricks also broke down more rapidly than cement-stabilised bricks when immersed in water. However, Ngowi postulated that the cracking of the asphalt-stabilised bricks might have been due to inadequate manual mixing, and not necessarily poor waterproofing performance of the asphalt.

2.10 COMBINATIONS OF STABILISERS

The use of combination stabilisers has not received widespread application in Australia or other parts of the world. Most rammed earth builders prefer to use only one stabiliser and avoid the handling and construction requirements of a two-component stabilisation system. However, the advantage in utilising combination stabilisers is that one stabiliser in the combination compensates for the lack of effectiveness of the other in treating a particular aspect or characteristic of a given soil. For instance, in clay areas that are devoid of base material, lime has been used jointly with other stabilisers, notably cement or asphalt, to provide stabilisation for rammed earth walls. Since cement or asphalt cannot be mixed successfully with plastic clays, the lime is first incorporated into the soil to make it friable, thus permitting the cement or asphalt to be adequately mixed. While such a stabilisation practice might be more costly than the conventional single stabiliser methods, it may still prove to be economical in areas where other soils must be trucked to the construction site.

2.10.1 Lime-cement combinations

Combinations of lime and cement are acceptable stabilisers. While cement cannot be used alone for heavy clays or highly plastic soils, lime can be first used to initiate cation exchange and flocculation-agglomeration reactions and to produce immediate changes by reducing the plasticity and improving the workability of these soils (Wolfskill et al., 1963). In general, lime reacts readily with most plastic soils, either fine-grained clays or clay-gravel types that range in Plasticity Index (PI) from 10 % to more than 50 %. Lime also reacts with some silts but normally will not react with sandy soils. Cement can then be mixed with the soil to provide rapid strength gain. This is especially advantageous when rapid strength gain is required under cooler weather conditions.

2.10.2 Cement-emulsion and/or lime-emulsion combinations

Portland cement and lime have been used to promote curing and increasing strength of emulsion-treated soils for rammed earth walls. The rate of development of strength in emulsified mixtures on curing is greatly accelerated by cement. Even small amounts of portland cement can enhance the early modulus gain for emulsified mixtures. Emulsion mixtures that might not cure to strength in a reasonable length of time (for instance, because of cool, damp weather) can be improved through the use of cement or lime. Soil-asphalt mixtures with significant amounts of clay/silt must have sufficient water available to satisfy their affinity for water. If the mixture is too sticky, it cannot be mixed properly. In this case, lime can be added as a pre-treatment of the soil (Winterkorn, 1975).

Although asphalt stabilisation is effective with many soils, the effect of moisture may have a significant influence on performance. It is known that the presence of moisture decreases the stiffness or modulus of emulsion mixtures. Mixtures of bituminous and cementitious stabilisers have the advantage of reduced moisture susceptibility as well as increased strength and cohesion. Hydrated lime percentages of 1-2 % have been shown to improve particle coating by asphalt emulsion (Ausroads, 1998). The lime addition may prevent stripping at the emulsion-soil interface as well as increase the stability of the mixture. Pre-treatment of the soil with lime has been quite effective not only in raising the modulus value in some cases but also in imparting almost complete water resistance.

2.10.3 Combinations with fly-ash

Fly-ash is the fine material released during the burning of coal and coke. Fly-ash stabilisation for rammed earth walls has not been used in Australia or the South Pacific Region. Its initial application in the United States was in road construction in combination with lime. If a soil does not possess finely subdivided, highly siliceous materials that are capable of reacting with $Ca(OH)_2$, then such material can be added in the form of volcanic ashes, defatted diatomaceous earth, or fly-ashes of proper composition (Symons, 1999). For proper reaction, an excess of $Ca(OH)_2$ over the amount of reactive silica is necessary. If the end product of the reaction is tobermorite, consisting of $3Ca(OH)_2 \cdot 2SiO_2$, then the weight-ratio of $Ca(OH)_2$ to reactive silica is 1.82:1, assuming that both components are of equal effective particle size and reactivity. The coarser the component that furnishes the SiO_2 or the less its surface: volume ratio, the more of it will be required. That amount which exceeds the theoretical ratio must be considered as an inert filler. Use of an excess of about 30% $Ca(OH)_2$ over the theoretical requirements for the fly-ashes or the pozzolanic ingredients is indicated; this would make the theoretical ratio of $Ca(OH)_2$ to reactive portion of pozzolanic ingredient equal to about 2.4:1. The amount of $Ca(OH)_2$ should be further increased if there is danger of diffusion of CO_2 into the system which forms the less desirable $CaCO_3$ compound and pre-empts the possibility of the $Ca(OH)_2$ reacting with the siliceous constituents.

These considerations indicate that in the case of fly-ash lime proportions of 5:1, which seems to be common in practice, a considerable amount of the fly-ash serves as inert filler. Fly ash and lime are normally used in mixtures as a pozzolan and as a filler for the voids. Since the particle size of the fly-ash is normally larger than the voids in the fine grained soils, it is not appropriate to use fly-ash as a filler in fine grained soils (Mateos, 1964) Thus, the only role for the fly-ash in stabilisation of fine grained soils is that of a pozzolan. Consequently, silts are generally considered the most suitable soil types for treatment with lime and fly-ash.

2.10.3.1 Properties of lime fly-ash stabilised soils

Pozzolanic reactions from which lime fly-ash mixtures derive their long-term strengths are influenced by many factors, including ingredient materials, proportions, processing, moisture content, field density, and curing conditions. For a lime fly-ash mixture to develop its maximum possible strength, the ingredients must be thoroughly mixed. Curing conditions have a profound influence on the properties of lime fly-ash mixtures. Both curing time and temperature greatly affect the strength and durability of "hardened" mixtures. Because of the combined effects of time and temperature on the strength development of the lime fly-ash mixtures, it is difficult to specify combinations of curing conditions that simulate field conditions. Curing at low temperatures retards the reaction process of lime fly-ash mixtures and almost entirely stops the reaction below 4°C. Reduced temperatures or even freezing of the mixture have no apparent permanent detrimental effect on the chemical properties of rammed earth mixtures. If the reaction is slowed during cold weather, increases in strength are again developed with rising temperatures during the subsequent warm months.

In an effort to accelerate development of early strength and improve the short-term durability characteristics of lime fly-ash mixtures, thereby permitting extension of the construction period later into colder weather, admixtures have been added to accelerate or complement the lime fly-ash reactions. Most of the work in this area has been with chemicals in liquid suspension or in powdered form. Portland cement is an effective admixture for use in lime fly-ash mixtures. The early strength development associated with hydration of portland cement complements the slower strength development associated with some lime fly-ash reactions.

2.10.3.2 Different ratios of fly-ash

Symons (1999) reported the influence of mixture of fly-ash and cement/lime on unconfined compressive strength of a soil in application to roading use. Although the soil used, a non-plastic poorly graded limestone gravel, would not be suitable for rammed earth walls, the information about stabiliser treatments is still relevant. Mixes of cement and fly-ash, totalling 4 % by dry mass of soil, showed a decrease in compressive strength as the proportion of cement to fly-ash decreased below 75:25. The 28-day compressive strength of a cement fly-ash mixture (20:80) was 1.0 MPa compared with an 80:20 mixture of 2.7 MPa. Increasing a fly-ash-hydrated lime mixture from 75:25 to 50:50 increased the 28-day strength of the soil from 1.1 to 1.6 MPa.

Symons (1999) also reported the effect of different proportions of cement: fly-ash as a stabiliser of a poorly graded gravel or silty gravel (GP-GM). He found that the maximum unconfined compressive strength is attained with a ratio of cement:fly-ash of 80:20 for samples stabilised with 4 % of stabiliser additive by mass of dry soil (Figure 2.10). This value declines as the proportion of cement replaced with fly-ash increases.

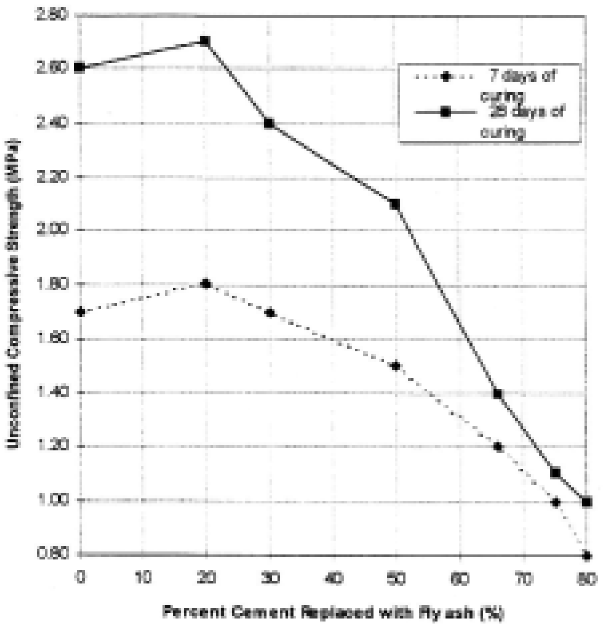


Figure 2.10: Unconfined compressive strength of soil stabilised with various proportions of cement and fly-ash (from Symons, 1999).

Indraratna et al.(1995) examined the effect of fly-ash, with lime and cement, on the strength and behaviour of a soft clay (Bangkok clay). Unconfined compressive strengths of the clay treated with 5 % lime and 18 % fly-ash, measured after 14 days, were 2 to 3 times higher than the natural clay. When lime was replaced by cement, the rate of strength development increased significantly. Excessive fly-ash proportions of the total mix (> 25 %) lead to tensile splitting of test specimens. Another feature of the fly-ash treatment was that the compressibility of fly-ash-cement treated soil was substantially lower than the natural clay. Triaxial compression tests showed that fly-ash-lime and fly-ash-cement treated owed their strength to an improvement in the apparent angle of friction, while the effect on the cohesion intercept was marginal.

2.11 CURING

Increased curing times generally bring about strength increases. Figure 2.10 shows the effects of curing time on cement-fly-ash mixes from Symons (1999). In all cases, the strength at 28 days is greater than at 7 days, and the effect is most marked for mixtures with higher proportions of cement. Akpodje (1985) examined the effects of curing time on both lime and cement stabilised soils. For 4 % cement stabilisation, for sand loam, clay loam, and silty loam samples, unconfined compressive strengths at 28 days were around 1.4 times higher than strengths at 7 days, and at 112 days were around 1.7-1.9 times higher (Figure 2.11). For 10 % cement, the 28-day strength was again around 1.4 times higher than the 7-day strength, and the 112-day strength was 1.7-1.9 times higher except for the sand loam, which was 3.0 times higher. For 4 % lime, the equivalent figures were 1.4 times and 1.7-2.2 times higher. Bell (1996) studied the effect of lime addition on clay soils, and concluded that both the curing time and the temperature at which curing took place had an important influence on the amount of strength developed.

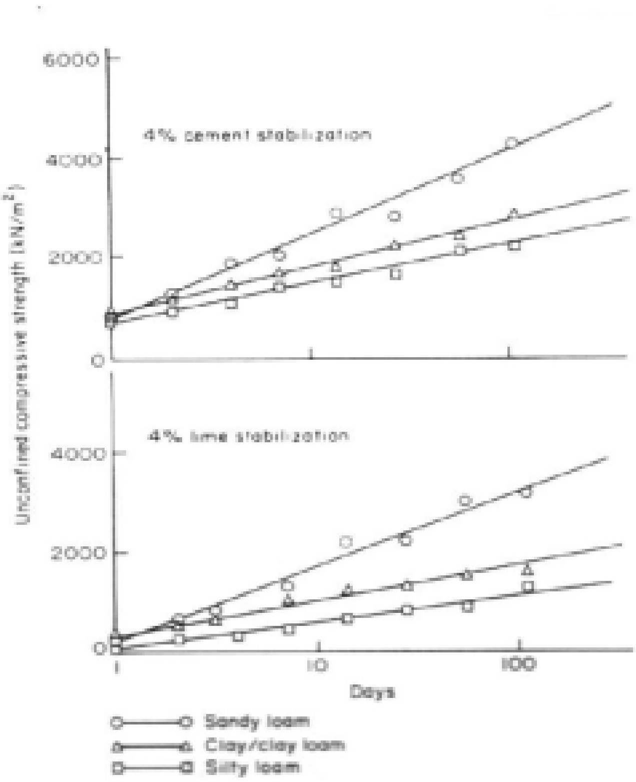


Figure 2.11: Effect of curing time on the compressive strength of soils stabilised with cement or lime, after Akpokodje (1985, p. 178).

Symons (1999) studied strength of a single soil as a function of curing time for a range of stabiliser treatments. The soil was a non-plastic poorly graded gravel. The results showed that the 28-day strength was higher than the 7-day strength by proportions ranging from 1.1 times for 4 % mixtures of cement-fly-ash 25:75) to 1.5 (for 4 % mixtures of cement-fly-ash 80:20). Symons (1999) reported a ratio of 1.6 for 4 % pure cement.

Jalali & Abyaneh (1995) examined the effect of curing temperature on the final strength of concrete, with particular reference to predicting long cure time strength of concrete in hot climates. Although their work relates to concrete, and not soil-cement, the broad conclusions may be relevant for rammed earth, particularly because most rammed earth construction takes place in hot localities (e.g. South Africa, Australia, Southwest USA). Concrete strength at long curing times is known to decrease with increased curing temperature. This is possibly due to the large differences in volumetric expansion of concrete constituents with temperature, and the increase in volume of water and air with higher temperatures. This creates internal stresses and strain in the form of increased porosity and microcracks in the curing material, thus reducing the strength of the concrete. Other possibilities include chemical ones, including compositional or structural changes of the hydrates with temperature, or that the degree of hydration at long curing times is temperature dependent. Jalali & Abyaneh (1996) suggested that the temperature dependence of final strength, could be explained by assuming that the hydration of cement follows a nucleation and diffusional growth process. They proposed a linear relationship between the natural logarithm of strength and the reciprocal of curing temperature, which was tested using published data for concrete strength and also lime-fly-ash mixtures. As hydration of cement and lime are key processes for rammed earth stabilisation, albeit at lower percentages (approximately 6 %), these results have implications for practical rammed earth construction if the long cure time strengths of soil-cement mixtures are temperature dependent.

Both West (1959) and Osula (1996) examined the effect of elapsed time after mixing on the properties of the stabilised mixture. West (1959) showed that a delay of several hours between the mixing and compaction of soils stabilised with cement slightly reduced the 7-day strength of the material dependent upon soil type. For a sandy soil and a clay soil, delays between mixing and compacting of six hours reduced the strength of the material by up to 10 %. Reductions in strength after 2 hours delay were minimal. However, the strength reduction for a sand clay soil was some 35 % for a delay of six hours. Osula (1996) showed that reductions in optimum moisture content and maximum dry density occurred over time for both lime and cement soil mixtures. Three hours after mixing, dry densities declined on average by 5 % of their initial values as measured very shortly after mixing. Optimum moisture contents were more affected, declining by between 9 and 22 %. Optimum moisture contents showed greater declines for cement-stabilised mixes than for lime-stabilised ones. These results that show significant changes in optimum moisture contents over periods of a few hours stress the importance of checking the moisture content of the mixture being rammed during earth wall construction.

2.12 OTHER METHODS OF STABILISATION

The stabilisers discussed below are those considered generally ineffective, or undesirable, or otherwise rarely used. In civil engineering applications of stabilised earth, over 90 % involves lime, cement, or asphalt and combinations thereof (USACE, 2000), a statistic that is also probably true for rammed earth stabilisation.

2.12.1 Resins

There are two main categories of resin stabilising systems: those in which resin functions only as a waterproofing agent and those in which the resin also acts as a cementing agent. For the former, natural resins in their acid form or neutralised to various extents are commonly employed. For both waterproofing and cementing purposes, synthetic resins, especially the condensation products of aromatic amines and furfural, are efficient stabilisers. A large number of long-chain aliphatic and resin amines, amides, and related organic cationic agents have become commercially available as adhesion-promoting additives for asphalt and as independent waterproofing agents for soil.

Within the two categories of (1) waterproofing and (2) waterproofing plus cementing stabilisation by resinous treatment, fall the same types of systems, which were previously treated in the case of asphaltic soil stabilisation. Thus, the following can be considered as waterproof, with or without additional cementation: (1) Clay concrete, or waterproofed clay-bound granular soil stabilisation. Complete replacement of the clay binder by resinous binders leads to resin concretes; (2) Clay mortar, or resin waterproofed sand-clay, in which complete replacement of the clay mortar by a cementing resin also leads to a resin mortar. The system can be open, as in beach sand stabilisation with A_2-F resin products, either without inert filler or with filler of low permeability (Winterkorn, 1965;

Eyraud et al., 1965).

There are also the corresponding waterproofed systems, with or without cementation, for soils without granular skeletons. If cementing action is desired, the condensation products of aromatic amines with furfural often compare favourably with portland cement as strong and durable bonding agents. Purely waterproofing, non-cementing resins are recommended only for soils that possess a granular bearing skeleton and enough of a silt-clay fraction that the strongly bound water barely fills the available pore space of the sand or sand-aggregate skeleton.

The natural resins employed for waterproofing soil stabilisation are produced by coniferous trees and can be harvested from living trees, from the stumps of dead trees (wood resin, vinsol), or mined as fossil remnants of decayed coniferous trees. Many of these resins are normally too costly to be considered for rammed earth wall construction. (Gieseking, 1955). However, urea-formaldehyde resins have shown to be of potential for soil stabilisation. Lahalih & Ahmed (1998) showed that a 2 % quantity of urea-formaldehyde and 0.25 % sodium chloride increased the compressive strength of dune sand from 0.02 MPa to 2.4 MPa. This stabiliser is water-based and non-toxic, and is therefore "environmentally friendly" and safe to use.

2.12.2 Inorganic chemicals

Inorganic chemicals have generally been shown to be of questionable effectiveness in stabilising soils.

2.12.2.1 Calcium Silicate

Laboratory results of treatments involving the simultaneous mixture of hydrated lime and powdered silicate with montmorillonite-rich clay soil have been reported by Ruff & Davidson (1961). No field applications of such treatment are known to have been attempted, probably because of the double difficulty of obtaining a proper dispersion or admixtures and of satisfactorily compacting the treated soil before its solidifying reaction progresses.

2.12.2.2 Sulphates of Calcium

Surface courses treated with gypsum ($CaSO_4.2H_2O$) displayed a remarkable improvement on secondary roads in the USA (Parcher, 1964). These tests showed that clays possessing liquid limits greater than 100% and gypsum with $Ca(OH)_2$ or NaOH as a secondary additive, gave about 50psi (0.35 MPa) wet strength at 28 days. Lime and gypsum in combination were also found superior in stabilising allophanous ($AlSiO_3.xH_2O$-type) clays in Japan.

2.12.2.3 Phosphates

Several years of laboratory investigations on the suitability of phosphorous compounds as primary additives for soil stabilisation showed that tetra sodium pyrophosphate ($Na_4.P_2O_7.10H_2O$) is superior to results obtained with a clayey silt and with ferric chloride as a secondary additive to deter leaching of the soluble phosphates. However, a comparison of lime-treated and phosphorous-treated field test in USA showed lime to be a better stabiliser. In addition, the phosphorus treated soil dried slowly, which provided an unstable working platform, and showed inconsistent gains in strength (Rucker, 1965).

2.12.2.4 Sodium and potassium salts

As secondary additives in laboratory soil-cement specimens made from silts and clayey silts of low plasticity, 0.5 to 1% of NaOH and Na_2CO_3 were the most effective. Other sodium and potassium salts tried included KCl, $K_2Cr_2O_7$, KOH, $KMnO_4$, NaCl, and Na_2SO_3. Whereas many of these secondary additives increased the strength the strength of the cement-treated soils, the economy of these additives might be borderline due to higher material costs and additional operations involved in mixing. Their strength-increasing effect was apparently due to greater densities achieved in the laboratory might not be representative of field results; in the laboratory, the cement was mixed with dry soil, after which the mixing water, in which the chemicals had been dissolved, was added during the final mixing (Moh, 1962).

2.12.2.5 Asbestos

An investigation of asbestos as the primary additive for stabilisation of a clayey silt and of clay gave negative results (Krukar, 1964). However, the use of asbestos fibre as a secondary additive in bituminous soil stabilisation has been shown to have merit (USACE, 1958). However, modern knowledge of the dangers to health caused by asbestos rule out its use in soil stabilisation on these grounds alone.

2.12.2.6 Other Mineral Salts

Trace (0.5%) amounts of sodium fluosilicate, Na_2SiF_6, were added to phosphate-treated soils after it was found that fluorides in natural phosphate rock were responsible for accelerated and improved strength. The fluoridic additive did not cure the leaching defect of phosphate treatment and was more costly. Various other mineral salts have been used but without success, except as occasional tertiary additives (USACE, 1959).

2.12.3 Organic Chemicals

2.12.3.1 Fatty Polyamides

More than 30 different chemicals of this type have been tested as secondary or trace primary additives for waterproofing, "hydrophobicizing", or strength improvement of various types of soils. Most of these chemicals have been added in the form of soluble acetates or chlorides, a few as emulsions or finely suspensions to achieve uniform dispersion. One of the more promising polyamides of this group is a fatty quaternary ammonium salt described as a di-hydrogenated tallow di-methyl ammonium chloride which has been tested for anti-stripping properties in asphalt stabilising, water-proofing and dust palliation, irreversibility in electrokinetic dewatering, density increase in soil-cement mixtures. As an anti-stripping additive in asphalt stabilisation, this ammonium salt showed a wet strength which was 32% of dry strength with 0.1% of dry soil weight in mixes containing 5.3% asphalt by weight of dry soil. In this regard, the stabiliser was only two-thirds as effective as a similar compound, Armeen 12D (lauryl), a 12-carbon amine chloride, which was also superior with the emulsified asphalt mixes. The treated soil was a clayey silt with a plasticity index of 15%, containing about 10% illite, with the balance principally quartz and feldspars (Hoover & Davidson, 1956). As a waterproofing agent, this amine chloride compared well with sodium methylethyl-propyl siliconate (SMEPS) and aniline-furfual in field weathering tests (Freitag & Kozan, 1961). As a secondary additive in soil-cement mixtures, SMEPS was ineffective. Its cationic, aggregate action did not tend to increase compacted density as did the sodium salts. The criterion of success in these tests was to increase the compacted density sufficiently to permit a substantial reduction in the amount of cement (Lambe & Moh, 1958).

2.12.3.2 Paper mill waste derivatives

The early use of lignin waste and spent sulphite liquor as a temporary soil stabilising agent led to numerous laboratory investigations concerning the effect of lignin derivatives. Lignosulfonates, lignin bichromates and lignosulfonic acid polymers have been tested with reputedly beneficial effects (Gow et al., 1961). Their effect is to adhere to soil particles and also acts to disperse clay to make the material more plastic and increase its density after compaction (Ausroads, 1998).

2.12.3.3 Others

TBC (4-Tert-Butylpyrocatecol) is an effective trace additive for excluding water from lean clayey silts and silts under favourable curing conditions. In laboratory tests, the waterproofing, wet-strength-preserving, and hydrophobic properties of the compound were much better when the specimens were cured at 40 to 50% relative humidity for a week or more. Since this type of curing does not always represent actual field conditions, the reported advantages of the treatment must be discounted considerably. However, the small trace amounts (0.05 to 0.25%) found to be effective warrant further investigation (Hemwall et al., 1962).

Davidson (1949) tested a plastic clay with 6 water soluble organic compounds including Armac T, Armac 18D, Armac 12D, Rosin Amine-D Acetate, Amine 220, and Ammonyx T. These organic cations when mixed with the soil reduced its plasticity index and shrinkage limit. Davidson (1949a) studied the effects of adding the fatty acide amine acetate to two soils of plasticity index about 12. The cationic mixture reduced the plasticity index of the soils, reduced the shrinkage limit, and slightly lowered the maximum dry density and optimum moisture content of the soils.

Attom & Al-Sharif (1998) assessed the use of burned olive waste as a soil stabilising agent. They found that the addition of 2.5% by weight of the burned olive waste increased the unconfined compressive strength and the maximum dry density of soil.

2.13 SOIL PROPERTIES AND RECOMMENDED STABILISER TREATMENTS FOR STABILISATION

2.13.1 Introduction

Both the properties of the soil to be stabilised, and the different treatments and quantities of stabilisation, are important to successful stabilisation of soil for rammed earth wall construction. The range of suitable soils for rammed earth is more limited than for mud bricks, because the walls are constructed in long sections and are more prone to shrinkage cracking from clay dehydration (CSIRO, 1987). Generally, the most suitable type of soil for rammed earth construction is a well-graded soil, with a range of particle sizes from sands through to clays (CSIRO, 1987; UN, 1992). The low moisture content used for the compaction of rammed earth will allow for the use of soils with higher clay content. However, an excess of clay will almost certainly cause excessive cracking of the cured rammed earth wall; but on the other hand, too much sand may cause crumbling during ramming.

2.13.2 General recommendations

Table 2.6 displays information about suitable soil types for stabilisation of rammed earth, from Wolfskill et al. (1963). It shows that soils at either end of the spectrum of particle size distributions, that is, clays and gravels, are unsuitable for stabilisation. Most other soils types are suitable, but some need more modification than others, with the general procedure being of altering the gradation of a soil by adding whichever particle size fraction is lacking. Table 2.7, from UN (1992), is a little more informative as it gives some additional information including density and void ratio of soil types. Table 2.8 is from USACE (2000), and gives broad recommendations for stabilisation for civil engineering purposes based on plasticity and particle size distributions of the soils. The feature of the tables is their general nature, with the majority of soils being able, apparently, to be stabilised to some degree. The tables give no specific guidelines as to the quantities of stabiliser to be used. Moreover, little or no guidance is given as to the relative merits of the potentially acceptable soils.

Soil type(s)	**Recommended stabilisation**
Very fine sands, silty fine sands, clayey fine sands, clayey silts, silts	Most appropriate stabiliser is portland cement. Asphalt emulsion also suitable.
Gravelly clay, sandy clay, silty clay	Adding lime and sand is beneficial.
Clays, fat clays, organic soils, clean gravel	Do not use.
Silty gravels, sand-silt-gravel mixtures	Portland cement.
Clayey gravels, gravel-sand-clay mixtures, clayey sands	Lime is best. Portland cement alright if soil easily mixed.
Silty sands	Portland cement, add clayey fines.
Clean sands	Must add clayey fines.

Table 2.6: Recommendations for soil suitability and appropriate stabilisations for soils, adapted from Wolfskill et al. (1963).

Soil symbol	Soil type	Shinkage and swelling	Density at OMC (t/m3)	Voids ratio*	Suitability for stabilisation
GW	Clean gravel well graded	Almost none	2.00	0.35	Not suitable. Fines should be added.
GP	Clean gravel poorly graded	Almost none	1.84	0.45	Not suitable. Fines should be added.
GM	Silty gravel	Almost none	1.76	0.50	Suitable but lacks cohesion. Add fines.
GC	Clayey gravel	Very slight	1.92	0.40	Suitable, fines sometimes needed.
SW	Clean sand well graded	Almost none	1.92	0.40	Not suitable. Fines should be added.
SP	Clean sand poorly graded	Almost none	1.60	0.70	Not suitable. Fines should be added.
SM	Silty sand	Almost none	1.60	0.70	Suitable but lacks cohesion. Add fines.
SC	Clayey sand	Slight to medium	1.70	0.60	Suitable, fines sometimes needed.
CL	Low-plasticity clay	Medium to high	1.52	0.80	Sometimes suitable, sand should be added.
ML	Low-plasticity silt	Slight to high	1.60	0.70	Suitable, but lacks cohesion.
OL	Organic silt and clay low plasticity	Medium to high	1.44	0.90	Not suitable.
CH	Highly plastic clay	High	1.44	0.90	Rarely suitable, add sand if using.
MH	Highly plastic silt	High	1.60	0.70	Rarely suitable.
OH	Highly plastic organic	High	1.60	0.70	Not suitable.

Table 2.7: Suitability of soils for stabilisation (adapted from UN, 1992). * vs = 2.17t/cm^2.

Soils Class	Type of Stabilizing Additive Recommended	Restriction on LL and PI of Soil	Restriction on Percent Passing No 200 Sieve	Remarks
SW or SP	(1) Bituminous			
	(2) Portland cement			
	(3) Lime-cement-fly ash	PI not to exceed 25		
SW-SM or SP-SM or SW-SC or SP-SC	(1) Bituminous	PI not to exceed 10	PI 30 or less	
	(2) Portland cement	PI not to exceed 30	PI 12 or greater	
	(3) Lime	PI not to exceed 12		
	(4) Lime-cement-fly ash	PI not to exceed 25		
SM or SC or SM-SC	(1) Bituminous	PI not to exceed 10	Not to exceed 30 percent by weight	
	(2) Portland cement	------*		
	(3) Lime	PI not less than 12		
	(4) Lime-cement-fly ash	PI not to exceed 25		
GW or GP	(1) Bituminous			Well-graded material only
	(2) Portland cement			Material should contain at least 45 percent by weight of material passing No 4 sieve
	(3) Lime-cement-fly ash	PI not to exceed 25		
GW-GM or GP-GM or GW-GC or CP-GC	(1) Bituminous	PI not to exceed 10		Well-graded material only
	(2) Portland cement	PI not to exceed 30		Material should contain at least 45 percent by weight of material passing No 4 sieve
	(3) Lime	PI not less than 12		
	(4) Lime-cement-fly ash	PI not to exceed 25		
GH or GC or GM-GC	(1) Bituminous	PI not to exceed 10	Not to exceed 30 percent by weight	Well-graded material only
	(2) Portland cement	------*		Material should contain at least 45 percent by weight of material passing No 4 sieve
	(3) Lime	PI not less than 12		
	(4) Lime-cement-fly ash	PI not to exceed 25		
CH or CL or MH or ML or OH or OL or	(1) Portland cement	LL less than 40 and PI less than 20		Organic and strongly acid soils falling within this area are not susceptible to stabilization by ordinary means
HL-CL	(2) Lime	PI not less than 12		

$$* \quad PI \le 20 + \frac{50 - \textit{percent passing No 200 sieve}}{4}$$

Table 2.8: Guide for selecting a stabilising additive on the basis of soil type (from USACE, 2000).

The selection of the soil to be stabilised is a critical step. Which stabiliser to use, and how much, depends on the type of soil to be treated and the aims of the stabilisation. These aspects are now examined below, with discussion confined to the three most commonly used stabilisers for rammed earth wall construction, being lime, cement, and asphalt.

2.13.3 Soils suitable for lime stabilisation

The effects of lime are highly dependent on the nature of the soil(s) to be stabilised, and much more so than cement. Lime is unlikely to successfully stabilise organic soils or those soils that are deficient in clay. Since the beneficial effects of lime stabilisation are the results of various reactions between the fine portion of the soil and lime, fine-grained soils respond most favourably. For clay-sand or clayey soils, lime stabilisation is just as effective, perhaps more so, than cement (UN, 1992). The addition of between 6 to 12 % quicklime is used for the stabilisation of soils, similar to the amounts required for cement stabilisation, although an optimum lime % is more likely to exist for a particular soil than for cement (Ausroads, 1998). Dumbleton (1962) suggested that a minimum clay content of approximately 10 % and a plasticity index in excess of 10 % are desirable for lime stabilisation.

NLA (1972), cited in USACE (2000), recommended lime stabiliser contents for civil engineering subgrades and bases (Figure 2.12). Although the values recommended may not be strictly applicable to rammed earth walls, the relative differences in the quantities recommended for different soils are relevant. For soils with plasticity index > 30, NLA recommended lime quantities of 5-8 %. For soils of plasticity index 15, the recommended level of lime was 3-5 %, and for soils of plasticity index < 10, lime quantities of 1-2.5 % were recommended.

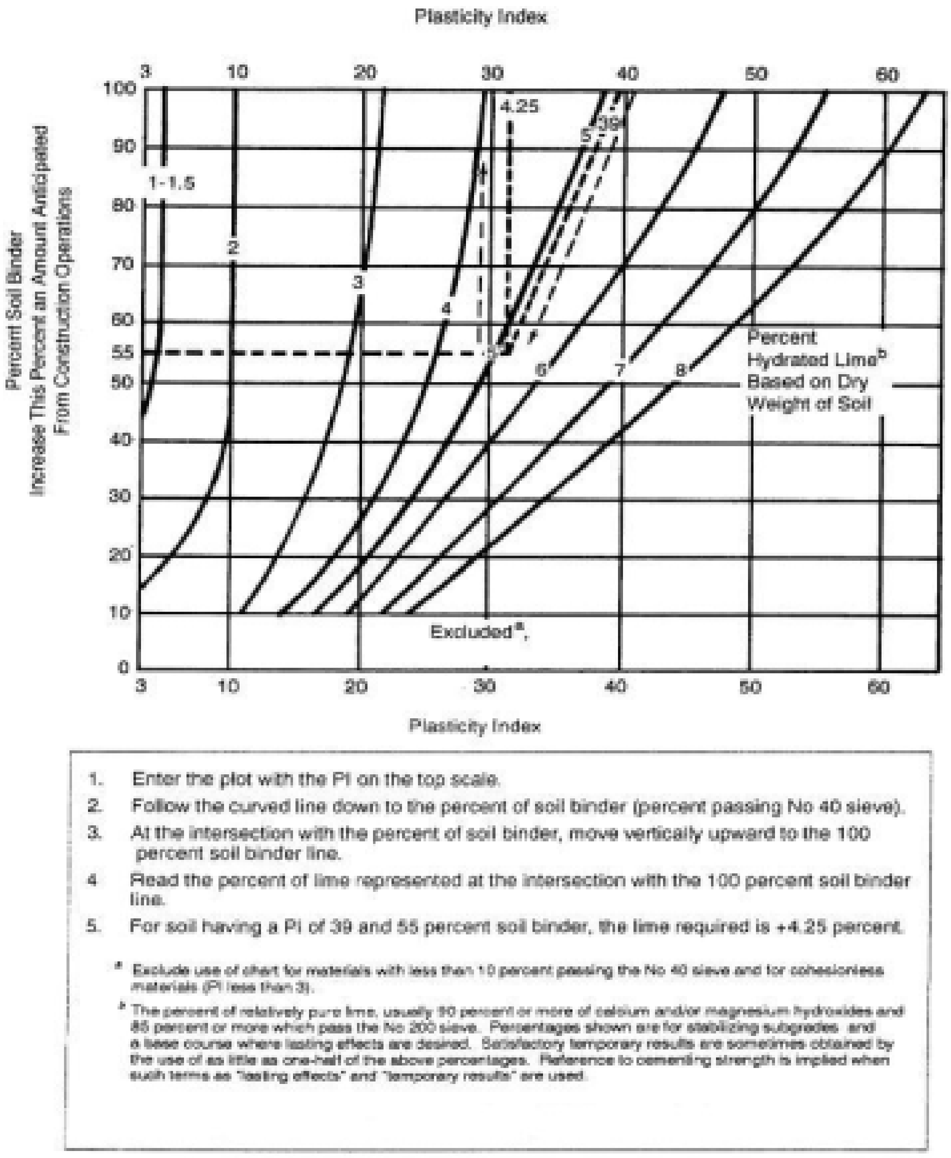


Figure 2.12: Method of determining initial design lime content (from NLA, 1972, in USACE, 2000).

Lime reacts at higher rates with montmorillonite clays than with the kaolinites, reducing the plasticity of soils containing the former substantially more than the latter (Bell, 1996). Natural pozzolan soils, not unexpectedly, react particularly favourably with added lime. However, lime stabilisation is not recommended by CSIRO (1987), who argue that because lime stabilisation is dependent on the type and amount of clay of the soil, complex tests are needed to determine whether lime should be used, and if so, in what proportions. In addition, they argue that the strength gains are much slower than with cement stabilisation, and that (in Australia) the cost of lime is virtually the same as cement. They suggest only two (uncommon) situations in which lime should be considered. The first is where limestone is available for mining and no other stabiliser is available at the time. The second is where the clay is too heavy for the production of mud bricks. Lime can be added to the heavy clay and left for several days, then cement is added for the stabilisation process and the material is formed into blocks (CSIRO, 1987).

2.13.4 Soils suitable for cement stabilisation

Nearly all soils can be stabilised with cement and have dramatic improvement in their properties. Cement stabilisation is used for two reasons, first to increase cohesion of soils and second to reduce the clay content of a particular material. The greatest effectiveness of cement in comparison to other stabilisers is with low clay content soils such as sands, sandy and silty soils, and clay soils of low to medium plasticity. These soils respond very well to the addition of cement, and require as little as 2 to 2.5 % of cement content (CSIRO, 1987). Other soils that have high clay contents will require as much as 10 % cement. The type of earth wall construction being performed also determines the amount of cement required. Compressed blocks or mud bricks will not cause problems but rammed earth will have major cracking so as much as 15% cement may have to be used according to CSIRO (1987), who also recommend that thorough testing should be carried out before construction begins.

If the plasticity index exceeds about 30 %, cement becomes difficult to mix with the soil. If cement stabilisation is to be used for highly plastic soils, then lime may be added first to reduce the plasticity index and improve mixing workability before adding the cement. A soil may be acid, neutral, or alkaline and still respond well to cement stabilisation. Although certain types of organic matter, such as undecomposed vegetation, may not directly decrease stabilisation, organic compounds of lower molecular weight act as hydration retarders and reduce strength. Organic matter within the soil is therefore undesirable.

UN (1992) provide several indicators of the suitability and proportion of cement. For linear shrinkage (Alcock test) values of < 15 mm, 5.3 % cement is recommended. For values of 15-30mm, the equivalent proportion is 5.9 %, for 30-45 mm is 6.7 %, and for 45-60 mm is 7.7 %. For the erosion test, the proportion of cement should reduce the depth of holes to 15 mm, and for the abrasion test should reduce loose material to 3 % after 50 cycles. Cement is not recommended for soils with liquid limits of higher than 30 %. These clayey soils can however be treated first with hydrated lime. Salt-rich soils are unlikely to be able to be satisfactorily stabilised using cement, but soils that contain 1 % or less organic matter are able to be stabilised with 10 % cement, as recommended by UN (1992). However, the higher the cement % in the soil, the more likely the soil-cement is to develop shrinkage cracks, which although would not be a problem in the production of earth bricks, is a serious problem for rammed earth wall construction given the monolithic nature of the walls.

The greatest compressive strengths using cement as a stabiliser are obtained with gravels and sandy soils rather than with silts or clay (UN, 1992). Winterkorn (1975) recommended adding cement in larger quantities to plastic soils than to sandy soils, with the most plastic soils requiring about three times as much cement as a sandy soil for effective stabilisation. Soil with a high clay content should be compacted slightly above optimal moisture content, while sandy soils should be compacted slightly drier than their optimal moisture content.

CSIRO (1987) suggest that the amount of cement required to stabilise a soil is a function of the gradation of the soil. Coarse-grained well graded sand silts probably require in the order of 2-3 % cement, whereas a poorly graded, fine-grained silt may need up to 10 %. Similar to these recommendations are the data from USACE (2000). Table 2.9 reports these data, and although they apply to stabilisation for roadworks and other civil engineering applications, show that coarser soils require less cement than clay soils to be satisfactorily stabilised.

Soil Classification	Initial Estimated Cement Requirement, Percent Dry Weight
GW, SW	5
GP, SW-SM,SW-SC GW-GM, GW-GC	6
GM, SM, GC, SC, SP-SM, SP-SC, GP-GM, GP-GC, SM-SC, GM-GC	7
SP, CL, ML, ML-CL	10
MH-OH	11
CH	10

Table 2.9: Estimated cement requirements for various soil types (USACE, 2000). Soil classification symbols are as defined in Table 2.4.

2.13.5 Soils suitable for asphalt stabilisation

Asphalt is a suitable stabiliser for most soils. Extremely clean soils can lead to separation of the asphalt under the action of water. Moist soils are not suitable for asphalt due partly to the difficulty of mixing the material with the soil, but moisture problems can be reduced by adding lime, as discussed above. The presence of soluble salts should not exceed 0.25 %, as their excessive presence in the soil interferes with the binding films between the asphalt and clays. In order to obtain uniform distribution of the asphalt with the soil it is best to mix with large quantities of water forming an emulsion. Normally 2 to 3% of asphalt is added but can be as high as 8% (UN, 1992). High sand content soils require less asphalt than do soils with lower sand contents or clayey soils. Winterkorn (1975) recommended an asphalt quantity of 1-2 % for soils with a good gradation of particles from coarse to fine.

Table 2.10 shows the recommendations for the usage of soils to be mixed with asphalt emulsions (CSIRO, 1987). The recommendations show that higher amounts of asphalt emulsion must be used for soils with lower proportions of sand. Asphalt emulsion mixes freely with soils containing moderate to high clay contents. Optimal stabilisation results are achieved using a soil that has high clay contents and in which the sand is well graded from fine to coarse (CSIRO, 1987). Although sandy soils may be stabilised using asphalt emulsion, the distribution of the stabiliser throughout the soil will be less even, and mixing is more difficult.

Soil type	% sand	% emulsion by weight
Soil - high sand content	> 50 % sand	4-6 %
Soil - medium sand content	Approx. 50 % sand	7-12 %
Fine clay	< 50 % sand	13-20 %

Table 2.10: CSIRO (1987) recommended quantities for asphalt emulsion stabilisation. The values do not include any extra water needed to bring the soil to a workable level of plasticity.

Figure 2.13 displays asphalt quantities recommended for stabilisation for roadwork applications by USACE (2000). The appropriate asphalt content is a function of the gradation of the soil, specifically, the percentage of material passing the No. 200 sieve (0.075 mm) and the percentage of material passing the No. 4 sieve (4.75 mm) and retained on the No. 200 sieve. The figure shows that the highest amounts of asphalt correspond to soils that have higher proportions of particles between 0.075-4.75 mm, and higher proportions smaller than 0.075 mm. The recommended upper limit to the percentage of the finer fraction is 30 %.

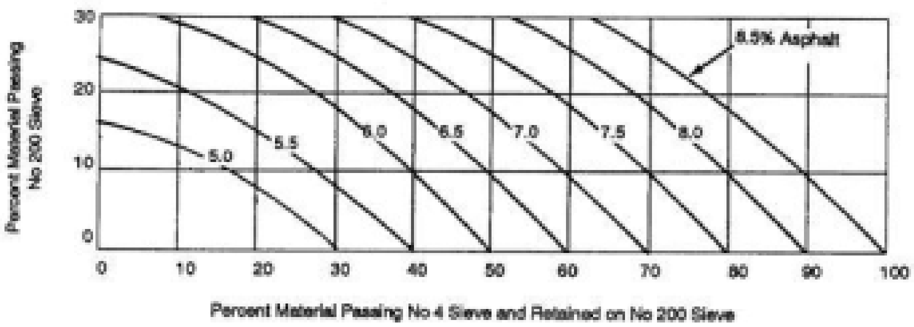


Figure 2.13: Selection of asphalt content (from USACE, 2000).

The type of clay in the soil to be stabilised influences the asphalt requirements (Winterkorn, 1975). In general, the larger the ratio of SiO_2 to (Al_2O_3 + Fe_2O_3) in the clay fraction, the greater the requirement for asphalt. As the base exchange capacity increases, so too does the need for asphalt.

Traditionally, testing has been used to determine the appropriate application rate for asphalt for particular soils. The following test is recommended to decide the correct soil/asphalt ratio when using asphalt as a stabiliser (CSIRO, 1987).

1. Take 500 g of the soil and place it into its plastic limit state.
2. Add 5 % asphalt and mix into soil thoroughly.
3. Take a small piece weighing about 15 g and place on a piece of blotting paper.
4. To the remaining sample add another 2 % asphalt, and mix.
5. Take a small piece weighing about 15 g again and place on blotting paper.
6. Add another 2 % asphalt to the remaining sample and repeat the process outlined.
7. Continue until there are samples of, say 5, 7, 9, 11 and 13 % asphalt
8. Dry the samples in an oven until dry.
9. Fill jars with water and place sample in water and shake jar.
10. The jar with no fines in the water and the lowest percentage asphalt is the recommended usage for the stabilisation.

The Asphalt Institute (1989) report the following equation as a guide to the amount of emulsion to form a heavily bound material in the absence of any other data:

$$\% \text{ emulsion (by mass)} = 0.75(0.05A + 0.10B + 0.50C)$$

where
A = % retained on 2.36 mm sieve,
B = % passing the 2.36 mm sieve but retained on the 0.075 mm sieve, and
C = % passing the 0.075 mm sieve.

UN (1992) recommend a test that involves progressively increasing the asphalt from 3 % by weight to 6 % by weight in several soil samples. The unconfined compressive strength is measured for each sample until a satisfactory result is obtained. Winterkorn (1975) also recommended an extensive testing method, involving measurements of soil gradation, plasticity, optimum moisture content and maximum dry density, strength and durability, for sets of test specimens at three different asphalt contents.

2.13.6 Assessing the stabilisation options

It is evident from the descriptions of the several methods of stabilisation presented that the ideal solution to the problem of strengthening a specific type of soil is often a combination of methods rather than a single type of treatment. It is also evident that many types of soil, from a theoretical standpoint, may respond to one or more combination of methods. Practical considerations, however, often restrict treatment to a single method or a single combination, which might be less than ideal. Whatever the approach to the selection of the most practical method of stabilisation, it is necessary to know the end result sought, or the type of requirement being placed on the rammed earth walls after completion. The degree of strengthening and extent of durability vary greatly with the intensity and type of treatment; the type of treatment is in part dependent on the field conditions during the ramming of walls, a point not often considered in laboratory tests or experiments of soil stabilisation.

2.14 MEASURES OF STABILISATION OUTCOME AND SUCCESS

The outcomes of stabilisation can be measured using a variety of properties including measures of strength, density, durability, and shrinkage. Criterion values of these properties can be assigned against which the measured qualities of stabilised earth can be evaluated. A determination measuring over the criterion value would be regarded as stabilisation "success", and one measuring under would be regarded as "failure". Recommended evaluation criteria are somewhat scarce in the academic literature, but more common in building codes.

2.14.1 Compressive strength

Compressive strength measures the load-bearing capacity of a material, and is therefore an important structural consideration in construction. The compressive strength test is used to determine the strength of stabilised material and is performed in laboratory conditions. The test involves subjecting a specimen to an increasing load without shock until failure occurs; the maximum load carried by the specimen is then recorded. This standard unconfined compressive strength test is detailed in Section 3.3.6. Strength determinations can be made at a variety of curing times between a few hours and many weeks. However, 7-day tests or 28-day tests are the most common (e.g., Akpokodje, 1985; Bryan, 1988; Walker, 1995).

CSIRO (1987) recommend a saturated unconfined compressive strength of 2 MPa for stabilised earth construction in Australia, although this is a 14-day strength measurement. Uniform Building Code Standard 71-1, a standard in the USA that controls earthen building materials and standard methods of sampling and testing earthen building materials (UBC, 1997), forms part of many county and city building codes. The UBC specifies a saturated compressive strength of 2.0 MPa, with 5 samples per 5,000 units to be randomly selected and tested. This is an average value of 5 samples, one of which may have a strength of not less than 1.7 MPa. The UBC assumes that the specimens have fully cured and therefore the strength measurements (and criterion value) are of "final" strength. Codes that include this standard include the New Mexico Adobe and Rammed Earth Building Code (New Mexico Construction Bureau, 1991), and the city of Boulder, Colorado structures building code (Boulder, 2000).

2.14.2 Density

At present, compaction (density) testing does not feature in earthen building codes. Compressive strength and durability are viewed as more important measures of stabilisation in both the academic literature and in building codes. DeaTech (1999) suggested that measurement of rammed earth wall density using a nuclear density guage could be a viable non-destructive way of testing wall strength. However, no criterion value of density was offered by DeaTech. Several studies have reported dry densities of stabilised earth of between 1.5-2.1 t/m^3 (e.g. Bryan, 1988; Walker, 1995; Ngowi, 1997). A criterion value should lie within that range. A value of about 1.8 t/m^3 correlates in some studies with a compressive strength of 2 MPa (e.g. Bryan, 1988).

2.14.3 Durability

Durability refers to the resistance of stabilised earth materials, and the structures made from them, to surface deterioration. Tests for durability include wire brush tests (e.g. Fitzmaurice, 1958), water spray tests (e.g. CSIRO, 1987), and water drip tests (e.g. Frencham, 1982). Other durability measures include the initial surface absorption rate and freeze-thaw dilatation (Bryan, 1988b).

CSIRO (1987) specify an accelerated erosion test that measures resistance of material to erosion. The soil erosion test consists of spraying the face of a prepared sample of the soil for a period of one hour or until the specimen is eroded through. A nozzle 470 mm from a 150 x 150 mm square section of sample sprays water under 0.05 MPa pressure at the surface. The maximum depth of erosion in one hour is measured in mm with a 10 mm diameter flat-ended rod. This value, divided by 60, gives the rate of erosion in mm/minute. In the event of the spray boring a hole through the specimen in less than one hour, the rate of erosion is obtained by dividing the thickness of the specimen by the time taken for full penetration to occur. The maximum permissible erosion rate is 1 mm/minute with nil water penetration into the remaining mass (CSIRO, 1987). However, it has long been the author's opinion that the soil erosion test is not an indication of the performance of the material when building rammed earth walls. The soil erosion test was designed to give an indication as to how a soil might respond to the elements once in the rammed earth wall format (and it is arguable whether it is even suitable for that purpose), not as a test of material to be used prior to construction. The author's experience and communication with other professionals indicate that the test has been used out of habit rather than good practice.

Heathcote (1995) suggested that the ratio between the dry and saturated compressive strengths may be a good measure of durability. The ratio indicates the effect of weakening the bond strength between particles by water absorption. Heathcote suggested a ratio of wet:dry strength of 0.33-0.50 may be appropriate for stabilised soil. Samples with a wet:dry ratio in excess of 0.33 tested by Heathcote (1995) with the accelerated erosion test of CSIRO (1987) all passed the test, indicating some correlation between the ratio and the CSIRO specification. Durability in the Uniform Building Code (UBC, 1997) is featured as a test of absorption. The test requires that the specimen not absorb more than 2.5 % moisture by weight when placed on a saturated porous surface for a period of 7 days.

2.14.4 Shrinkage

Building codes generally specify shrinkage criteria, on account of the weakening caused by the generation of shrinkage cracks. Stabilised earth with higher proportions of clay is predisposed to cracking. The problem is more significant for rammed earth walls than soil bricks, due to the monolithic nature of the rammed earth wall. CSIRO (1987) specify that no crack is to be longer than 75 mm, wider than 3 mm, or deeper than 5 mm. UBC (1997) specify that a 10 cm cube of material shall not contain more than three shrinkage cracks, and no shrinkage crack shall exceed 7.5 cm in length or 3 mm in width.

2.15 MULTIPLE INFLUENCES ON STABILISATION

2.15.1 Introduction

The outcome of stabilisation (as measured by strength and density) is a function of three influences: the properties of the soil used (soil type), the type of stabiliser used, and the quantity of stabiliser used. Although trying to isolate these different effects on stabilised material would appear to be important, less research than might be expected has been directed toward that end. The importance to stabilisation of the stabiliser used, and the quantity (percentage) of the stabiliser, has been generally assessed by experiments involving small numbers of different soils, sometimes just a single soil type. The experiments have involved a comparison of different quantities of a single stabiliser on one or more soils, or a comparison of types and quantities of stabiliser on one or more soils. As the number of soils studied has generally been small, the influence of soil properties with respect to the stabilisers has been treated qualitatively, being confined generally to statements concerning soil type, or sandy soils compared with clay soils. Hence, it has hitherto proved difficult to establish the relative importance of stabiliser variation and soil variation to the outcome of stabilisation.

The studies that have made a detailed examination of stabilisation with respect to the type of stabiliser, the quantity (percentage) of stabiliser, or soil type, are Croft (1968), Akpokodje (1985), Bryan (1988a), Walker (1995), and Ngowi (1997). Croft (6 soils), Bryan (5 soils), Akpokodje (3 soils) and Ngowi (5 soils) all used natural soils selected from a number of different sites, whereas 11 of 14 of the soils used by Walker were artificially modified and consisted of clay soil and river sand mixed in different proportions to effect textural and plasticity variations. Of these works, Croft (1968), Ngowi (1997), and Akpokodje (1985) examined all three effects (stabiliser type, stabiliser quantity, and soil type) on the strength of stabilised samples. Bryan (1988) used only cement at a single percentage level (10 %), and both Bryan (1988a) and Walker (1988) used only cement but at three different quantities. Therefore, specific information about the interactions between stabiliser type, stabiliser quantity, and soil properties, and the multiple effects on stabilisation outcomes and their relative importances to stabilisation, is actually rather disparate. Additionally, only the analyses of Bryan (1988a) and Akpokodje (1985) have had as their primary objective to properly disentangle the relative effects of these influences. Bryan (1988a) addressed aspects of stabiliser quantity and soil type, and Akpokodje additionally studied stabiliser type comparisons (cement and lime).

2.15.2. Author's reinterpretation of existing work on multiple influences on compressive strength

Akpokodje (1985) investigated the stabilisation of 5 Australian arid zone soils with various levels of lime or cement. The soils included a sandy loam, a silty loam, and a clay loam, as well as a highly calcareous soil and a highly gypsiferous soil, the results for which are of lesser interest here. For the clay loam and sandy loam, a lime content of 2 % gave a 7-day compressive strength of about 0.7 MPa, and increasing the quantity to 4 % gave a compressive strength of around 1.2-1.3 MPa (Figure 2.14). Further increases in lime from 4 % up to 12 % appeared to yield no additional strength to these samples. For the silty loam, the increase was a very gradual one, from 0.25 MPa at 2 % lime, to 0.30 MPa at 6% lime to 0.5 MPa at 12 % lime. The apparent small increase in compressive strength from 2 % to 6 % lime is probably not statistically significant. In contrast (Figure 2.15), for all 3 samples stabilised with cement, compressive strength increased as a well-defined linear function of cement content between 0 and 12 %. The 7-day compressive strength for sandy loam was highest, with values of 1.3 and 2.7 MPa for 2 % and 6 % cement respectively.

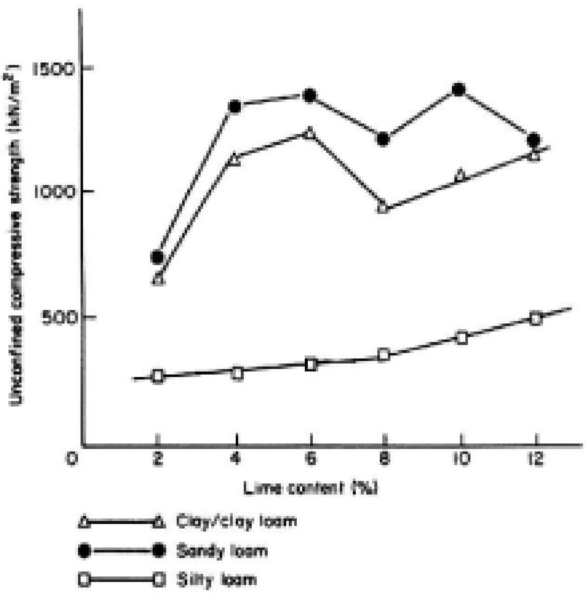


Figure 2.14: Relationship between compressive strength and lime content for 3 Australian arid zone soils after Akpokodje (1985, p.175).

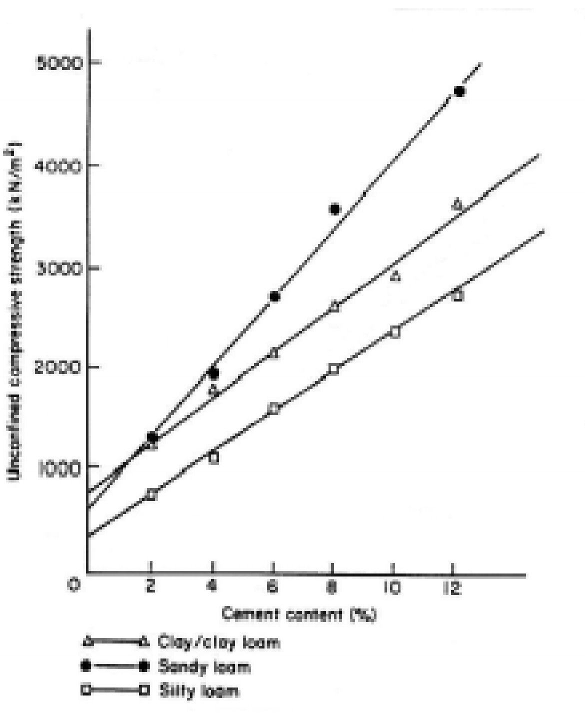


Figure 2.15: Relationship between unconfined compressive strength and cement content for 3 soils from the Australian arid zone, after Akpokodje (1985, p.175).

Akpokodje's (1985) results can be reinterpreted in terms of the relative importance to compressive strength of the type of stabiliser, the amount of stabiliser, and the soil type. The reinterpretation is confined to the information for quantities of stabiliser up to and including 6 %, to coincide with the range of stabiliser used in this study. All values are as read off Figures 2.14 and 2.15 as they were unreported in tabular form by Akpokodje. The effect of the stabiliser type (lime versus cement) is found by comparing the results for the same soil for the same percentage of stabiliser. At 6 % stabiliser, the average strength for the 3 soils for lime was 1.0 MPa and for cement was 2.2 MPa (difference of 1.2). At 4 %, the averages were 0.9 and 1.6 MPa respectively (difference of 0.7); and at 2 % were 0.5 MPa and 1.1 MPa (difference of 0.6). An approximate measure of the overall difference in effect of the stabilisers in the range between 2-6 % is the average of these differences, which is 0.8 MPa in favour of cement. The effect of the stabiliser is determined from these same values, and the figure is calculated using the increase in compressive strength from 2 % to 6 % of stabiliser. For lime, the average rate of increase in strength is 0.14 MPa per 1 % increase in lime as a proportion of dry soil; and for cement, the average rate of increase in strength is 0.28 MPa per 1 % increase in cement. It should that the increase in strength for cement (Figure 2.15) for a particular soil is almost perfectly linear, whereas the strength increase for lime added shows less trend. For the soil difference effect, this is dependent on the stabiliser used and the quantity. It is calculated by determining the average of the 3 pair differences in compressive strength between the soils for the same stabiliser quantity. For lime, the average difference between the soils is 0.8 MPa for 6 %; 0.7 MPa for 4 %; and 0.3 MPa for 2 % (lime average difference equals 0.6 MPa). For cement, the average difference between the soils is 1.0 MPa for 6 %; 0.6 MPa for 4 %, and 0.4 MPa for 2 % (cement average difference = 0.65 MPa). In summary therefore, for Akpokodje's data set for levels of stabiliser up to 6 %, the average difference between lime and cement stabiliser is about 0.8 MPa, the average difference between the soils is about 0.6 MPa for lime and 0.65 MPa for cement, and the strength increase per % of stabiliser added is 0.14 MPa for lime and 0.28 MPa for cement.

This same exercise can be repeated for the work of Bryan (1988a). Bryan examined the influence of cement quantity and soil type on compressive strength for 5 soils. These soils ranged in sand content from 30 % to 78 %, and plasticity index from

6.4 to 26.2 %. Figure 2.16 (from Bryan, 1988a, p.325) shows the variation in 28-day compressive strength for the 5 soils for 3 different quantities of cement (5 %, 7.5 %, and 10 %). The data are presented differently from Akpokodje (1985). The interpretation is confined to cement percentages of 5 % and 7.5 %, as these are closest to the levels used in this study. Using the values displayed on the graph, the average difference in compressive strength between the 2 different stabiliser levels (5 %, 7.5 %) for the same soil is 0.80, equivalent to a rate of 0.32 MPa per % of cement. This value, around 0.3 MPa increase per %, is in close agreement with the value obtained from the work of Akpokodje (1985). For the soil type effect, the average difference between the 5 soils is an average of the 10 possible pair differences in compressive strength, and is equal to 0.7 MPa. This value, based on a different data set, is in agreement with the value derived from Akpokodje's work.

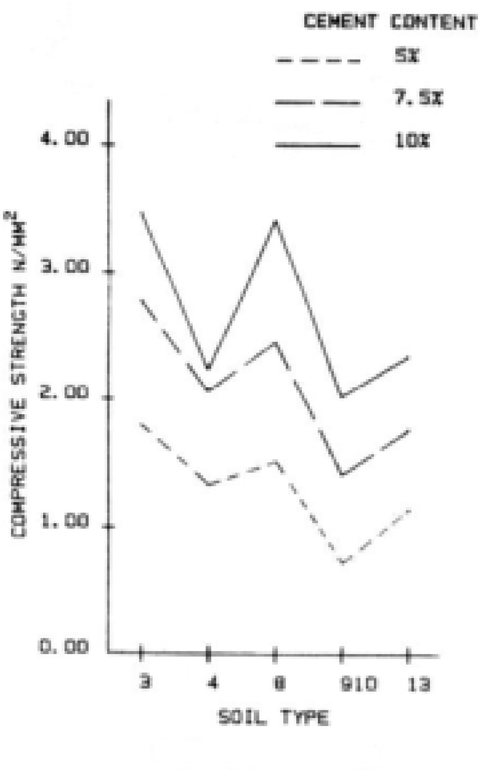


Figure 2.16: Variation in compressive strength by soil type for 3 levels of cement, after Bryan (1988a, p.325).

Croft (1968) examined the stabilisation of several soil types from two contrasting soil profiles using two quantities (5 %, 10 %) of both lime and cement. Figures 2.17 and

2.18 show the relevant 7-day compressive strength data as plotted by Croft, and from which the values were read as the data were not tabulated by Croft. The data for both stabiliser levels have been used, in order to ascertain differences in strength between levels, although it should be noted that the 10 % value is somewhat greater than the 6 % maximum used in this study. In each profile, three soils have been tested (data points in Figures 2.17 and 2.18) giving six different soils. There is no overall average difference between the stabilisers across the soils. The average difference between the 6 soil types for lime, using the 15 difference pairs, is 0.40 MPa; for cement, the average difference is 0.20 MPa, or about half that of lime. The increase in compressive strength between 5 % and 10 % lime is 0.28 MPa per %, and between 5 % and 10 % cement is 0.64 MPa per percent.

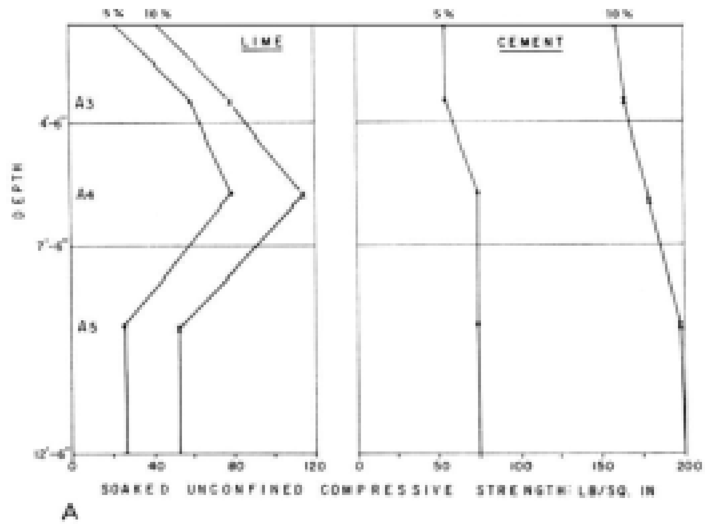


Figure 2.17: Typical 7-day compressive strengths developed in soils from a podsolic profile (from Croft, 1968, p.404).

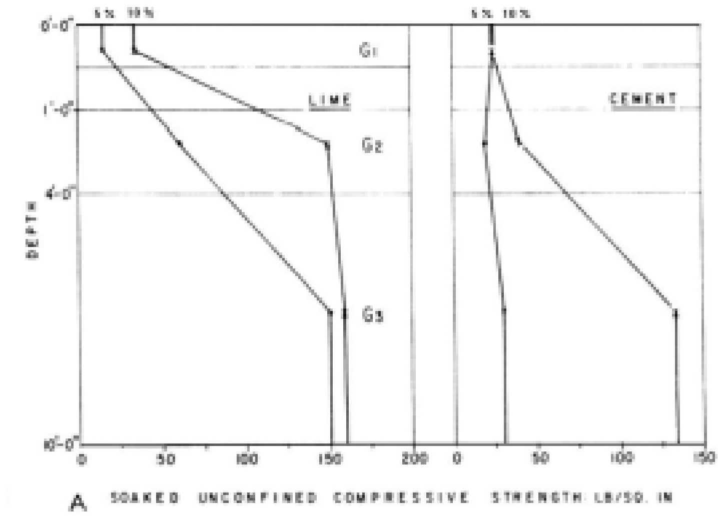


Figure 2.18: Typical 7-day compressive strengths developed in soils from a black earth profile, after Croft (1968, p.407).

Ngowi (1997) studied the stabilisation of soil bricks by pressing and stabilisation with several levels of cement and lime using two soils. The Mahalapye soil had 27 % sand and 73 % clay/silt, a liquid limit of 31 and plasticity index of 12. In contrast, the Tsabong soil had 63 % sand and 27 % clay/silt, a liquid limit of 50 and plasticity index of 26. Table 2.11 shows Ngowi's strength data for the stabiliser levels closest to those used in this study. The values in Table 2.11 are all averages of 5 determinations. The overall average compressive strength for cement is 5.36 MPa and for lime is 2.76 MPa, a difference of 2.6 MPa. This difference between cement and lime stabilisation is much higher than the other investigations examined above. The difference in compressive strength between stabiliser quantities is 0.47 MPa per % of cement, and 0.29 MPa per% of lime. This is consistent with the other studies to the extent that the ratio of the strength increase of cement-stabilised soil to lime-stabilised soil per % of stabiliser is about 2:1. The average difference in compressive strength between these two soils for cement is 0.28 MPa (Tsabong stronger) and for lime is 0.27 MPa (Mahalapye stronger).

Stabiliser (%)	UCS for cement (MPa)	UCS for lime (MPa)
Mahalapye soil		
5.0	4.55	2.65
7.5	5.90	3.14
Tsabong soil		
5.0	4.98	2.16
7.5	6.02	3.10

Table 2.11: Selected stabiliser and compressive strength data from Ngowi (1997).

Walker (1995) studied the stabilisation of soil blocks with cement, using soils modified texturally so as to produce a range of soil characteristics that were then related to block strength and other measures of stabilisation outcome. Figure 2.19 shows Walker's data for average compressive strength and plasticity index for 3 different quantities of cement of 5 %, 6.7 %, and 10 %, of which the two lower cement levels are closest to those used in this study. One feature of the data is that the compressive strength for both of the lower levels is very similar, at around 3 MPa, until a plasticity index of around 15 is reached (Figure 2.19). A second feature is that for values of plasticity index above 15, there is a marked decrease for both stabiliser levels in compressive strength to around 1 MPa for a plasticity index of 30. For the whole range of plasticity values, samples stabilised with 6.7 % cement were on average 0.41 MPa stronger than samples containing 5 % cement, in other words, an effective increase of about 0.25 MPa per % of cement. This value is very similar to those derived from the work of Akpokodje (1985) and Bryan (1988a). Although these studies used different compaction pressures, Bryan (1988a) showed that the rate of increase in compressive strength as a function of increasing cement content is unaffected by differences in compaction pressure. The influence of soil type is calculated using the 45 difference pairs that exist between the 10 determinations of compressive strength for the 5 % stabiliser quantity data.. The average difference between the soils is 1.25 MPa. This figure is higher than the other values reported above from other investigations, and probably reflects the greater number and range of soils studied by Walker (1995).

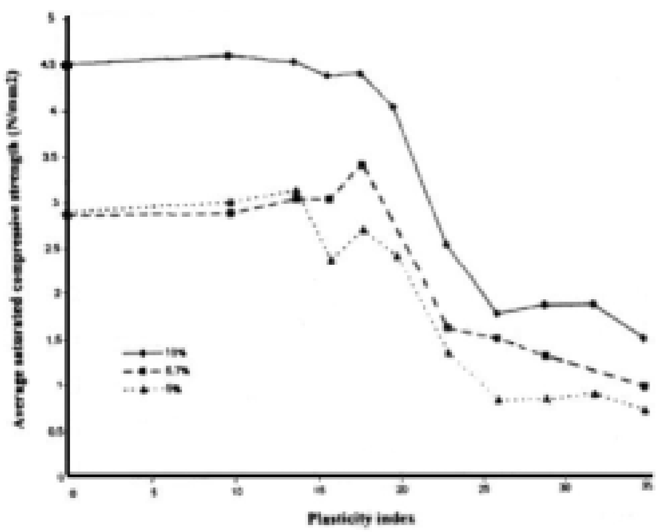


Figure 2.19: Variation of average 28-day compressive strength with plasticity index, after Walker (1995, p.307).

2.16 RESEARCH PROBLEMS AND OBJECTIVES

The literature relevant to soil stabilisation for rammed earth wall construction is characterised by several groups of publications:

1. Early soil stabilisation work as pioneered by Patty and Middleton. The research concentrated on the material properties of the soil, particularly the particulate properties and their influence on earth wall durability as assessed qualitatively (visually).
2. Civil engineering research. The research focuses on soil stabilisation by cement, lime, or asphalt, and in relation to the use of the material for roads, paving, and other earthworks. The work has provided experimental data relating stabiliser type and quantity to descriptors of the final stabilised product such as unconfined compressive strength and compressibility, with an emphasis on material performance and durability.
3. Manuals for rammed earth construction. The manuals give practical guidance on such aspects as soil selection, stabiliser treatments, and construction advice for earth construction practitioners. The guidance is general in nature, and often, pilot study testing of soils and stabilisers is the final recommendation.
4. Academic research. The research focuses on quantified comparative studies of the effect of different stabiliser treatments, or of different soils, on the properties of

stabilised material. The research is mainly in relation to earth bricks, not rammed walls. Also, it is in part tied to a theme of sustainability, which includes advancing knowledge of alternative construction materials.

In general, the most pertinent points to come from the literature are: the emphasis on the "test and see" approach in practical guidelines, which although giving broad recommendations, still recommend extensive testing procedures for every new construction situation; the emphasis in the civil/construction engineering and academic literature on the effect of stabiliser treatments on the qualities of stabilised material, with less focus on quantifying the influence of soil properties; and the paucity of research performed specifically regarding soil stabilisation for rammed earth wall construction.

From the rammed earth wall practitioner's viewpoint, then, there remain unresolved issues and problems. In particular, there is a need for the following:

- **The generation of a substantial database about soil properties and stabilisation treatments specifically for rammed earth wall construction.** The data are needed as a basis for improved understanding of soil stabilisation specifically for rammed earth building, and for the development of better guidelines for rammed earth construction. Most data available have been generated for civil engineering applications involving roading and paving, and these data are not necessarily appropriate for rammed earth building applications.

- **Better knowledge of the relationships between soil properties, stabiliser treatments, and the qualities of the resulting rammed earth.** The properties of soils and how they favour or disfavour stabilisation for rammed earth construction need to be better known, to improve the results of construction. The emphasis in previous studies has been on the effect of different types and quantity of stabiliser, rather than the influence of soil properties, and few attempts have been made to unravel the multiple influences on the properties of stabilised earth. Many of the studies have involved a very limited range of soils, sometimes just a single soil. In addition, most work has been done with respect to pressed earth bricks, not rammed earth. A major impediment to improving our understanding is that there are often problems of comparability between stabilisation studies due to experimental and other differences.

- **Improved guidelines for the stabilisation of soils for structurally sound rammed earth construction.** More specific guidelines are needed to assist practitioners in the field. An important consideration in developing the guidelines would be to avoid the need for extensive testing on soils that are unlikely to prove suitable for rammed earth wall construction. This demands a need to be able to predict the likely result of stabilisation, rather than the extensive demands of the "test and see" approach. Although some research has been performed on the changes in material properties that take place before/after stabilisation, (e.g. the effect of lime on strength or plasticity), very little work has been done on using measured soil properties to predict the likely outcome of stabilisation.

With these research needs now identified, the objectives of this study are to:

1. Determine the relationships between soil properties (gradation, plasticity, and moisture), stabiliser types (asphalt, cement, lime), stabiliser quantities, and stabilised earth qualities (density and compressive strength), for a wide range of soil types.
2. Identify the appropriate quantities of stabiliser needed to satisfactorily stabilise different soils.
3. Quantify the minimum, optimum, and maximum values of soil properties required to promote successful stabilisation.
4. Assess the relative importance of stabilisers and different soil properties in controlling variation in the qualities of stabilised earth.
5. Establish specific guidelines for soil assessment, selection, and stabilisation that can be used for the construction of rammed earth walls, based on the measured relationships between soil properties, stabiliser treatments, and the qualities of the stabilised material.

CHAPTER 3: METHODOLOGY AND EXPERIMENTAL TECHNIQUES

3.1 SITES AND SAMPLING

A total of 111 soil samples have been examined in this study, taken from 29 actual or proposed rammed earth construction sites within a 300-kilometre radius of Canberra, ACT. These sites are spread throughout the states of ACT and NSW as shown in Figure 3.1. From theses sites, a wide range of different soil types was sampled to examine the relationships between soil properties and stabiliser treatments. This was because a number of different soil types may potentially be available for stabilised rammed earth construction at any particular construction site. No attempt was made at the sites to assess the particular soils for their suitability for rammed earth construction, or reject possibly unsuitable soils, as the testing of a variety of soils possessing different characteristics would enable stabilisation suitability criteria to be established for as wide a spectrum of naturally-occuring soils as possible.

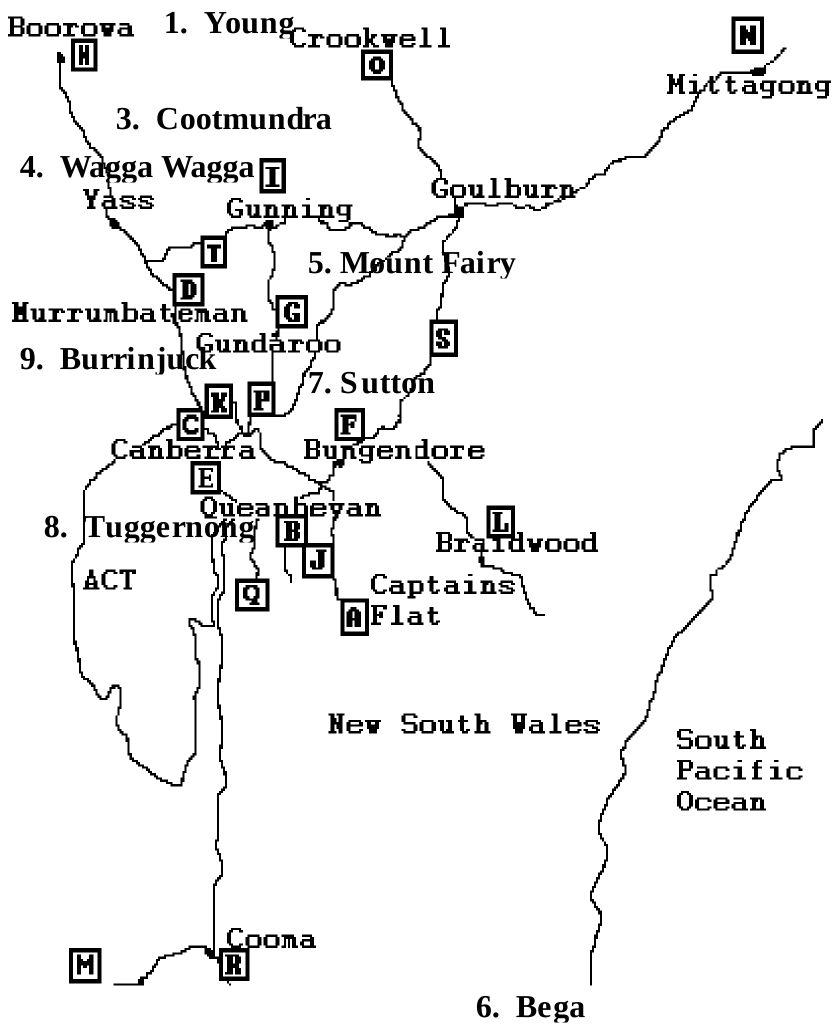


Figure 3.1: Locations of sites from which samples were taken.

At each site, soil was sampled either by test pits or hand borings. Test pits were dug using a backhoe to a depth of 1-3 metres, and boreholes made using an augur to similar depths. At each sample location, vegetation, if any, was cleared and the pit or bore made. With both pit and bore methods, the layer of soil below significant organic accumulation (e.g. leaves, humus) and above unweathered parent material, was sampled. This part of the soil is weathered, contains a range of particle sizes, and has negligible organic matter. Overlying material was not sampled as these layers contain too much organic matter and are not used for rammed earth work. The appropriate amount of soil, 40 kg, as recommended by SAA (1977), was collected at each pit or bore.

Locations of sample pits or bores at each construction site were considered representative of the area surrounding the site within a radius of 250 m, which represents a practical distance within which soil could be transported to the building site during actual construction. At each site, the surrounding area was reconnoitred and the vegetation, topography, rock outcrops, natural drainage, and soil colour noted and mapped. These indicators of soil type were assumed to represent a broad variation in soil properties over the area of interest around the site. Such indicators are those used routinely in earth stabilisation field reconnaissance, and are presented in detail in Chapter 6.

Once the broad soil variation had been established, sampling was performed to cover the range in soil properties as indicated by the mapping. Establishing this soil variation was important because it reflects the situation that a rammed earth practitioner would be presented with: a number of different soil types are likely to be available for consideration as sources for rammed wall construction at a particular site, and a decision has to be made as to the most suitable soil(s) for use in the construction. Between 1 and 12 samples were taken from each site, depending on the amount of variation in soil type as indicated during field reconnaissance and mapping. Further details of samples and experimental determinations are described below.

3.2 EXPERIMENTAL DESIGN

The overall experimental scheme was designed to investigate the influence on the properties of rammed earth of soils with different characteristics treated with various types and quantities of stabilisers.

3.2.1 Variables to be measured

The research reviewed in Chapter 2 suggested that the following variables are the most significant to the processes and outcome of soil stabilisation for rammed earth wall construction, and are therefore the most appropriate to measure.

3.2.1.1 Soil particle size distribution

The importance of soil gradation to stabilisation has been shown by Patty (1936), Winterkorn (1975), Akpokodje (1985), CSIRO (1987), Bryan (1988), UN (1992), and Walker (1995), the results of which have been discussed in Chapter 2. Croft (1968) used textural information for helping estimate the amount of cementitious stabiliser required for satisfactory soil stabilisation. Bryan (1988, 1988a) employed textural criteria to assess soils for their suitability for cement stabilisation, and his tests included information from the full range of fine, intermediate, and coarse particle sizes. Spence (1975) and VITA (1975) (reported in Bryan 1988), also utilised soil gradation criteria to assess soil suitability. Walker (1995), in his study of cement stabilisation of soil blocks, measured fine gravel, sand, silt, and clay in an attempt to ascertain textural influences on the strength and shrinkage of stabilised material.

For this study, three textural variables were measured from determinations of soil gradation, the method for which is described below. The three variables are % gravel (defined as the proportion of the soil retained on the 2.36 mm aperture sieve), % sand (defined as the proportion of the soil passing the 2.36 mm aperture sieve and retained on the 0.075 mm aperture sieve), and % clay/silt (defined as the material passing the 0.075 mm aperture sieve), which are fractions based on the Australian Standard sieve apertures. The upper size limit for gravel is 19 mm as explained below in Section 3.31. The traditional silt and clay fractions are combined for the sub-0.075 mm particle size fraction. By combining the silt and clay fractions, the soil particle size distributions can be obtained by sieve analysis without the need for complex or delicate methods (e.g. hydrometer for the sub-0.075 mm fraction). Studies of stabilisation with reference to developing countries have generally made no distinction between the silt and clay fractions (Bryan, 1988), and combining the two fractions should allow more widespread application of the results of this study.

3.2.1.2. Atterberg limits (plasticity)

The liquid limit, plastic limit, and plasticity index are fundamental physical properties of a soil. Their significance to stabilisation has been reported, amongst others, by Croft (1968), CSIRO (1987), Bryan (1988), UN (1992), Walker (1995), and Ausroads (1998). The results of these works have already been outlined in Chapter 2.

The Atterberg limits reflect the physical response of soil to water, and are therefore significant as indices of the behaviour of soils, particularly clay-rich soils. Bryan (1988, 1988a) used plasticity indices to assist in identifying soils suitable for cement stabilisation, and found liquid limit in particular to be a useful variable to help discriminate between suitable and unsuitable soils. Both liquid limit and plasticity index were used as indices of the suitability of soil for cement stabilisation by Webb et al. (1950), Ransom (1963), and Fitzmaurice (1958). Walker (1995) measured both liquid limit and plastic limit, and utilised plasticity index as an important x-axis variable in explaining patterns of both unconfined compressive strength and maximum dry density for soil stabilised with cement. Croft (1968) determined plasticity both before and after stabilisation in order to determine differences between natural and stabilised soil, and showed that soils with high natural shrinkage limits shrank less during curing having been treated with lime or cement.

The measurements of plasticity made in this study were liquid limit, plastic limit, and plasticity index, all measured on the natural soil. These measurements are standard ones and will enable comparison with previous stabilisation studies. Plasticity measurements were not made on stabilised material, because such tests made on curing material are difficult and their worth is uncertain. The criterion variables in this study are confined to measures of the density and strength of stabilised material.

3.2.1.3 Linear shrinkage

Bryan (1988) used linear shrinkage as one of the variables in his study of the selection of soil for cement stabilisation, and suggested that it may be employed as an indicator of the quantity of cement required for stabilisation. Croft (1968) measured linear shrinkage both before and after stabilisation in order to detect shrinkage changes resulting from the stabilisation process, and Walker (1995) measured shrinkage of the stabilised material only.

Linear shrinkage in this study was measured on the natural soil, and is treated as a predictive measure of stabilisation, rather than using the linear shrinkage of the stabilised material, which is a measure of the outcome of stabilisation.

3.2.1.4 Optimum moisture content, maximum dry density and unconfined compressive strength

The importance of optimum moisture content, the moisture content at which a material reaches maximum dry density under a given compactive effort, is recognised in stabilisation guidelines such as CSIRO (1987) and UN (1992). Optimum moisture content has been employed as an explanatory variable in several studies including West (1959), Bryan (1988), and Osula (1996). It was used by Bryan (1988, 1988a) as a predictor variable to assess soil suitability for cement stabilisation.

Maximum dry density and saturated unconfined compressive strength are well established in the stabilisation literature as measures of the outcome of stabilisation (West, 1959; Metcalfe, 1963; Akpokodje, 1985; Bryan, 1988, 1988a; Walker, 1995; Bell, 1996; Osula, 1996; Indraratna et al., 1995; Ngowi, 1997). Croft (1968) employed saturated compressive strength as the independent variable with which to gauge the influence of soil properties on stabilised strength. Both Akpokodje (1985) and Ngowi (1997) showed that differences in soils and stabiliser quantities generate differences in compressive strength of stabilised samples. Osula (1996) demonstrated that maximum dry density varied partly due to the stabiliser used (lime or cement) and partly to the amount of stabiliser used. Bryan (1988, 1988a) used both density and strength as the criterion variables against which he judged the suitability of various soils for stabilisation by cement. Dry density and compressive strength were treated by Bryan as independent variables influenced by three sets of factors: soil type, the quantity (percentage) of cement stabiliser, and the compaction pressure. Walker (1995) measured both density and strength of cement-stabilised blocks as a means to identify the types of soil most suited to stabilisation as defined primarily by plasticity index.

Although saturated unconfined compressive strength is the most common measure of material strength in stabilisation studies, other measures of strength and strength-related properties have been cited in the literature. Dry unconfined compressive strength is less favoured as a measure of strength as it yields higher (i.e. less conservative) values, although it was employed as a variable by Walker (1995). The modulus of rupture, a measure of flexural strength, was used by Walker as an indirect means of estimating compressive strength of stabilised soil blocks. However, the modulus shows considerable scatter when related to compressive strength, and was not recommended by Walker as a substitute for direct compressive strength testing.

In this study, optimum moisture content has been measured and treated as a predictor variable that influences values of maximum dry density and compressive strength. Saturated unconfined compressive strength and maximum dry density have been used as the dependent variables measured in this study, because they are appropriate stabilisation outcome variables that can be compared with previous stabilisation studies and with recommended construction-related standards as defined by such publications as CSIRO (1987) and other building regulations.

The two most widely used methods of determining the response of a soil to compaction (and thereby of determining the relationship between moisture content and maximum dry density) are the standard Proctor and modified Proctor tests. These tests involve dropping a rammer of specified weight from a specified height onto a cylinder of soil of specified volume. The standard Proctor test compacts soil in 3 layers by a specified number (25) of rammer drops per layer. The modified Proctor test compacts soil in 5 layers with a heavier hammer falling a greater distance than in the standard Proctor test. Various professional organizations concerned with standards for compaction, for example AASHTO (American Association of State Highway and Transportation Officials), ASTM (American Society for Testing and Materials), and SAA (Standards Association of Australia), each have their own versions of these tests that differ slightly in the dimensions of the apparatus and experimental techniques used. However, the result in all cases is that the compactive effort (quantum of energy) applied to the material is the same: for the standard Proctor test, the compactive effort is 596 kN/m^2 and is 2703 kN/m^2 for the modified Proctor test (SAA, 1977). Details of the different experimental apparatus dimensions used for the SAA and AASHTO standards are indicated in Table 3.1.

Variable	SAA test specifications (Tests AS 1289.E1.1 and 1289.E2.1)		AASHTO test specifications (Tests T-99 and T-180)	
	Standard	Modified	Standard	Modified
Weight of rammer (kg)	2.7	4.9	2.5	4.55
Height of drop (m)	0.300	0.450	0.305	0.457
Number of drops (total)	75	125	75	125
Internal diameter of cylinder (m)	0.1050	0.1050	0.1016	0.1016
Length of cylinder (m)	0.1155	0.1155	0.1164	0.1164
Number of soil layers	3	5	3	5
Volume of soil compacted (m^3)	0.001000	0.001000	0.000944	0.000944
Compactive effort (kN/m^2)	596	2703	596	2703

Table 3.1: Test specifications for standard and modified Proctor compaction tests for SAA (1977) and AASHTO.

The compaction test used to determine the moisture-density relationship in this study (details of which are presented in Section 3.3.5) was the modified Proctor test, using the specifications of Australian Standard 1289.E2.1 (SAA, 1977). The modified Proctor test was preferred to the standard Proctor for a number of reasons. The author's experience indicates that the modified Proctor is the test that is more acceptable to building professionals (including engineers and building inspectors) who monitor the density, strength and other properties of engineering structures, including rammed earth walls. Moreover, it is the author's experience that the density and strength of stabilised, rammed earth compacted in the laboratory using the modified test (compared with the standard test) are much better indicators of the density and strength of the actual rammed earth wall as constructed later at the site. CSIRO (1987) recommend that the laboratory compaction be the same as, or similar to, that used in the field. As the intention of this study was to produce results directly relevant to on-site construction practice, the modified test was preferred.

In addition, the modified Proctor test is a better laboratory simulation of on-site ramming conditions as assessed by the greater compactive effort applied to the earth compared with the standard test. This can be demonstrated by considering the calculation of compactive effort, which involves the following equation:

$$CE = (H*F*n_1*n_2)/V$$

Where,

CE = Compactive effort (kN/m^2)

H = height of drop (m)

F = force of hammer (kN) (mass in kg multiplied by 0.0098)

n_1 = number of drops of hammer

n_2 = number of soil layers compacted

V = volume of mould (m^3)

Although rammers vary in their dimensions, a typical hand rammer used on-site, with measurements of 0.1 m diameter, 15 kg weight, and with a drop of 0.4 m onto a soil layer thickness of 0.075 m, requires 27 blows (on the one spot) to achieve the compaction effort equivalent to the modified Proctor laboratory test (compared with 6 blows to achieve the standard Proctor equivalent). Typically, the higher number of blows of such a hand rammer is used during on-site construction. Pneumatic rammers used for compacting earth walls have a pneumatic force exerted as calculated by multiplying the cross-sectional area (0.00114 m^2) of the bore of the piston by the pressure of 620.5 kN/m^2, which equals 0.71 kN, per stroke (blow). With this force applied to a typical rammer head of 0.1 m diameter (a cross-sectional area of 0.007854 m^2), this pneumatic force translates into a compactive force of 90 kN/m^2 per blow (0.71 / 0.007854). Therefore, a compactive effort equivalent to the modified Proctor is achieved after around 30 blows, equivalent to only a few seconds ramming the same spot with a machine running at 600 strokes/minute. Although there are some dimensional differences between different models and makes of both manual and pneumatic rammers, it is clear that the compactive effort of on-site ramming using these tools is more comparable to the value of the laboratory modified Proctor test than that of the standard test.

3.2.1.5 Stabiliser treatments

For this study, the stabilisers of lime, cement, and asphalt were used as additives as their use is widespread in the rammed earth building industry. In addition, they are also the most researched stabilisers in the literature, the results of which have been reported in Chapter 2. The effect of lime as a stabiliser has been studied by researchers including Metcalfe (1963), Akpokodje (1985), Osula (1996), Bell (1996), Ngowi (1997). Cement as a stabiliser has been tested by workers including Croft (1968), West (1959), Akpokodje (1985), Bryan (1988, 1988a), Walker (1995), Osula (1996), and Ngowi (1997). These studies have tended to use a range of quantities of stabiliser treatments to ascertain the effect on the density and strength of the resulting stabilised material. Recommended percentages of lime for testing and construction range between 2 and 10 % (Winterkorn, 1975). Metcalfe (1963) used lime levels of 0, 2, 4, 6, and 10 %; Akpokodje (1985) used lime or cement levels of 2-12 % in intervals of 2 %; Bryan (1988a) used cement levels of 5, 7.5, and 10 %; and Walker (1995) used cement levels of 5, 6.7, and 10 %. Ngowi (1997) used cement and lime levels of 5, 7.5, 10, and 15 % in a study of stabilised soil bricks. Osula (1996) used somewhat lower levels of 0, 1, 2, and 3 % of lime or cement, perhaps reflecting better the requirements of developing countries. The ranges of lime and cement stabiliser percentage used have generally reflected the practical need for satisfactory stabilisation without incurring unnecessary and excessive costs. The range in percentage of lime and cement used in this study are similar to those used in previous studies, ranging from 0 to 6 % by weight of dry soil, for which treatment details are described further below.

There are fewer asphalt-related stabilisation studies than for cement or lime, and asphalt is used as an additive by rammed earth practitioners far less often than cement and/or lime. RTA (1995) recommended amounts of 2-3 % of asphalt by dry weight of soil for satisfactory stabilisation of soil for pavement work, a quantity with which UN (1992) concurred for the construction of rammed earth edifices. In contrast, CSIRO (1987) earlier recommended quantities from 4 % to over 15 % for earth wall construction. Given the economic and practical aspects of rammed earth stabilisation, which preclude the use of large quantities of asphalt stabiliser, the chosen quantities of asphalt used in this study were 0 and 3 %, to allow a comparison in the properties of samples stabilised with cement and/or lime and asphalt and those stabilised with cement and/or lime only.

The constituents and properties of each of the three stabilising agents used in this study are well-known and defined by product standards. The cement used for the stabilisation experiments, manufactured by Blue Circle Southern Cement Ltd, was general purpose, "GP", which is manufactured from Portland cement clinker and gypsum. This cement meets or exceeds the relevant product standard described in Australian Standard AS 3972 (1997). This product has been used widely in the construction industry, including rammed earth building, for many years. The cement properties of the GP cement used include a constancy of volume of 1.0 mm, a final setting time of 3-4 hours, a fineness index of 350-400 m^2/kg, and compressive strengths of 41-46 MPa (7-day) and 52-58 MPa (28-day) respectively (Blue Circle Southern, 1999).

The lime used for the stabilisation tests was hydrated lime $Ca(OH)_2$. Hydrated lime is produced by mixing quicklime with water at a controlled rate, which just satisfies the affinity of quicklime for water, thereby producing a dry powdered hydrate (Blue Circle Southern, 1997). The lime (manufactured by Blue Circle Southern Cement Ltd) used for the stabilisation experiments, meets or exceeds the requirements of the relevant Australian Standard for Building Limes AS 1672 (1974). The lime used had an available lime index (an index of how much lime is available for reaction) of 94.0 % as $Ca(OH)_2$; 99 % of its particles passing a 0.075 mm screen; a specific gravity of 2.2-2.3 t/m^3; a bulk density of 450-560 kg/m^3; and a solubility in water (at 20°C) of 1.65 g/L.

The asphalt used in the stabilisation experiments was an asphalt emulsion that conformed to the relevant Australian Standard AS 1160. Manufactured by CSR Emoicum Road Services, the asphalt emulsion contained 60 % asphalt and had a one-hour setting time.

3.2.1.6 Alternative variables

Other variables that have been measured in stabilisation studies include mineralogical and chemical properties of natural soil (e.g. Croft, 1968), and measures of stabilised material durability (e.g. Heathcote, 1995). Although no mineralogical-chemical properties or durability variables have been measured in this study, a brief discussion of them follows.

Ingles & Frydman (1966) showed that the cation exchange capacity (CEC) might be of use for predicting the strength of compacted and stabilised clay, at a given density. Croft (1968) used various measures of soil mineralogy and chemistry with respect to cement stabilisation of soil. The measures included the percentage of expansive clay in the clay fraction, the cation exchange capacity of the soil, and the pH. Croft found poor relationships between expansive clay percentage and strength of soils stabilised using cement. Croft concluded that pH was largely redundant in explaining strengths of soils stabilised using either lime or cement, and that CEC was not much better. Mineralogical and chemical properties therefore appear to be less important than textural and plasticity properties in respect of soil stabilisation using lime or cement.

Although measures of durability of stabilised material have been commonly used in some stabilisation studies, the scope of this study does not include such measures, focusing rather on strength and density as criterion variables. Within the literature, there is considerable discussion concerning the applicability of various measures of durability, summarised well by Heathcote (1995). The more common tests for surface durability have included wire brush tests (e.g. Fitzmaurice, 1958), water spray tests (e.g. CSIRO, 1987), and water drip tests (e.g. Frencham, 1982). Other durability measures include the ratio between the dry and saturated compressive strengths (Heathcote, 1995), initial surface absorption rate (Bryan, 1988), and freeze-thaw dilatation (Bryan, 1988). The different durability tests appear to measure different aspects of durability, and results between tests cannot generally be satisfactorily compared. Compressive strength is a better-defined property of stabilised earth than is durability, and is preferred in this study as an outcome variable.

3.2.2 Design details

In this study, 29 different sites were chosen from which between 1 and 12 samples were taken, with an overall total of 111 samples. Of these 111 samples, 53 samples were tested once yielding 53 determinations, and 58 samples were subdivided and tested with different stabiliser treatments (between 2 and 4 different treatments per sample) yielding 177 determinations. Overall, therefore, a total of 230 determinations of unconfined compressive strength resulted. Further details about the data generated from the experimental determinations are reported in Chapter 4.

The soil property variables measured (% gravel, % sand, % clay/silt, liquid limit, plastic limit, plasticity index, linear shrinkage, and optimum moisture content) define the variables by which a soil is able to be characterised. The soil property variables, together with the stabiliser treatments, are regarded as the independent or predictor variables in the study (e.g. Fitzmaurice, 1958; Spence, 1975; Bryan, 1988; Walker, 1995). These variables cause variation in the two dependent or criterion variables: the maximum dry density of the soil-stabiliser mixture under a defined compactive effort, and the unconfined compressive strength of the stabilised, cured sample (e.g. Bryan, 1988). The two dependent variables are the variables by which soil suitability for stabilisation, and stabiliser effectiveness, are judged. The values of compressive strength and dry density as measured in stabilisation experiments can be compared with the criteria for "successful" stabilisation (Section 2.14). The criterion values chosen for this study in light of the discussion in Section 2.14 are a minimum of 2.0 MPa for unconfined compressive strength, and a minimum of 1.80 t/m^3 for maximum dry density. Table 3.2 summarises the variables measured in this study.

Variable measured	Independent variable (predictor)	Dependent variable (outcome or criterion variable)	Test summary
Gravel (%)	✓		Natural soil before compaction/stabilisation
Sand (%)	✓		Natural soil before compaction/stabilisation
Clay/silt (%)	✓		Natural soil before compaction/stabilisation
Liquid limit (%)	✓		Natural soil before compaction/stabilisation
Plastic limit (%)	✓		Natural soil before compaction/stabilisation
Plasticity index (%)	✓		Natural soil before compaction/stabilisation
Linear shrinkage (%)	✓		Natural soil before compaction/stabilisation
Asphalt (%)	✓		Set quantity of stabiliser as % of natural soil
Cement (%)	✓		Set quantity of stabiliser as % of natural soil
Lime (%)	✓		Set quantity of stabiliser as % of natural soil
Optimum moisture content (%)	✓		Stabilised soil after compaction, before curing
Maximum dry density (%)		✓	Stabilised soil after compaction, before curing
Saturated compressive strength (MPa)		✓	Stabilised soil after both compaction and curing

Table 3.2: Summary of variables measured in this study.

Different combinations of asphalt, cement, and lime were applied to stabilise 230 sample specimens. The quantities of asphalt used were 0 % and 3 % of dry weight of soil, and the quantities of cement and lime were 0 %, 2 %, 3 %, 4 %, 5 %, and 6 % of dry weight of soil. These levels were chosen to cover a range of stabiliser contents, and the maximum quantities chosen correspond to practical and economic levels of additive. In addition, the relevant literature (Chapter 2) suggests that these stabiliser percentages should produce (on average) satisfactory stabilisation with respect to the "success" criteria as defined above. The combinations of stabiliser used in experimental determinations of density and compressive strength were: lime (29 determinations), cement (76 determinations), lime-cement (70 determinations), cement-asphalt (28 determinations), lime-asphalt (2 determinations), and lime-cement-asphalt (25 determinations). Table 3.3 shows the number of determinations of compressive strength and density made at particular levels of stabiliser.

	0 %	2 %	3 %	4 %	5 %	6 %
Asphalt	175		55			
Cement	31	27	11	58	16	87
Lime	104	50	12	34	8	22

Table 3.3: Number of experimental determinations made by stabiliser type and quantity. The information does not indicate the combinations of stabilisers used.

3.3 EXPERIMENTAL PROCEDURES

The tests on the untreated soil include particle size distribution, Atterberg limits, and linear shrinkage. The tests on the soil-stabiliser mixture include the optimum moisture content and maximum dry density (before curing), and the unconfined compressive strength (after curing).

3.3.1 Measurement of particle size distribution

The procedure for the determination of soil particle size distribution followed those described in the methods of testing soil for engineering purposes AS1289.C6.1 (SAA, 1977). The procedure covers the quantification by sieve analysis of the particle size distribution of a soil down to 0.075 mm. The combined clay/silt fraction can be obtained by difference. The Australian Standard sieve apertures used were 19 mm; 13.2 mm; 9.5 mm; 6.7 mm; 4.74 mm; 2.36 mm; 1.18 mm; 0.600 mm; 0.425 mm; 0.300 mm; 0.212 mm; 0.150 mm; and 0.075 mm.

3.3.1.1 Sample preparation

For each specimen, the sample was thoroughly mixed and subdivided by riffling to obtain an amount in accordance with the recommendations for minimum specimen mass reported in Table A2.3.6 of SAA (1977). In practical terms, the amounts used for the different soils ranged between 1 and 10 kg, depending upon maximum particle size characteristics. Sieving of the obtained mass generally proceeded in three stages: coarse, intermediate, and fine. Prior to sieving, any timber, roots, or any other visible organic material was manually removed from the sample.

3.3.1.2 Coarse sieving

Coarse sieving was used to check whether any samples contained individual particles greater than 19 mm sieve aperture. Samples were sieved on the 19 mm sieve and all material passing the sieve was collected after brushing the sieve with a bristle brush. The mass of material retained on the 19 mm sieve was recorded (m_1). However, in the event, no sample was recorded as containing any material in excess of 19 mm sieve aperture. The mass of material passing the 19 mm sieve was recorded (m_2), and this material was then mixed thoroughly and a subsample of about 7 kg taken by riffling. The mass (m_3) of this subsample was measured and the moisture content (w) of a separate representative portion of the material determined using a standard oven-drying procedure as described in AS1289.B1.1 (SAA, 1977), to later calculate the oven-dry mass (m_4) of the subsample.

3.3.1.3 Intermediate sieving

Intermediate sieving was used for samples that contained individual particles between 19 mm and 2.36 mm sieve apertures, and for the fraction passing the 19 mm sieve after coarse sieving as described above.

The mass (m_3) of the subsample was determined, and the moisture content of a separate representative portion of material was determined using a standard oven-drying procedure as described in AS1289.B1.1 (SAA, 1977), to later calculate the oven-dry mass (m_4) of the subsample. The air-dry subsample was spread out in a large tray, covered with dispersing solution and stirred thoroughly. The soil was allowed to soak for 2 hours, then agitated and washed with water on the 0.075 sieve and the process repeated until the wash water was sufficiently clear. The washed material retained on the 0.075 mm sieve was decanted of excess water and oven-dried at 110°C until constant mass was attained and recorded (m_5). The oven-dry sample (the fraction between 19 mm and 0.075 mm) was then sieved down through the 6.7 mm and 2.36 mm aperture sieves and the mass of material retained on each sieve recorded. The mass (m_6) of material passing the 2.36 mm sieve was also recorded. A subsample of about 100 g of the material passing the 2.36 mm sieve was obtained by riffling and its mass (m_7) determined.

3.3.1.4 Fine sieving

Fine sieving was used for samples that contained only particles smaller than 2.36 mm sieve aperture, and for the fraction passing the 2.36 mm sieve after intermediate sieving as described above.

The mass (m_3) of the subsample was determined, and the moisture content of a separate representative portion of material was determined using a standard oven-drying procedure as described in AS1289.B1.1 (SAA, 1977), to later calculate the oven-dry mass (m_4) of the subsample. The subsample was washed with water on the 0.075 sieve and the process repeated until the wash water was sufficiently clear. The washed material retained on the 0.075 mm sieve was decanted of excess water and oven-dried at 110°C until constant mass was attained and recorded (m_7). The oven-dry sample (the fraction between 2.36 mm and 0.075 mm) was sieved through the 0.600 mm, 0.212 mm, and 0.075 mm aperture sieves, and, the mass of material retained on each sieve was recorded.

3.3.1.5 Calculations

1. The mass of the sample/subsample passing the 19 mm sieve (m_4), after drying to constant mass was calculated as:

$m_4 = (100*m_3)/(100 + w)$

Where
m_3 = air-dry mass of sample/subsample
w = moisture content of m_3

2. The dry mass of the total sample (m_s) was calculated by:
(a) For samples containing particles larger than 19 mm sieve aperture:

$m_s = m_1 + m_2(m_4/m_3)$

Where
m_1 = sum of air-dry masses of the sample retained on the 19 mm sieve

m_2 = air-dry mass of the portion of the sample passing the 19 mm sieve

(b) For samples containing only particles smaller than 19 mm sieve aperture:

$m_s = m_4$

3. The mass of material retained on the 19 mm sieve was calculated as a percentage of m_s:

$\%_a = 100*m_a/m_s$

Where

$\%_a$ = % material retained on sieve of aperture a

m_a = mass of material retained on sieve of aperture a

4. The mass of material retained on each of the intermediate aperture sieves (2.36 mm and larger but less than 19 mm) was calculated as a percentage of m_s:

$\%_a = 100(m_a*m_2)/(m_s*m_3)$

5. The mass of material retained on each of the fine aperture sieves (0.075 mm and larger but less than 2.36 mm) was calculated as a percentage of m_s:

$\%_a = 100(m_a*m_6*m_2)/(m_s*m_7*m_3)$

Where

m_6 = the mass of sample/subsample after washing, drying to constant mass, and being passed through the 2.36 mm sieve

m_7 = the mass of sample/subsample after washing, drying to constant mass, and riffled from the material passing through the 2.36 mm sieve

The percentages of material retained on each sieve in order of decreasing aperture were tabulated and the cumulative percentages retained on each sieve calculated. The values for cumulative percentage retained were then plotted on a standard semi-logarithmic chart for recording particle size distribution (Figure 3.2).

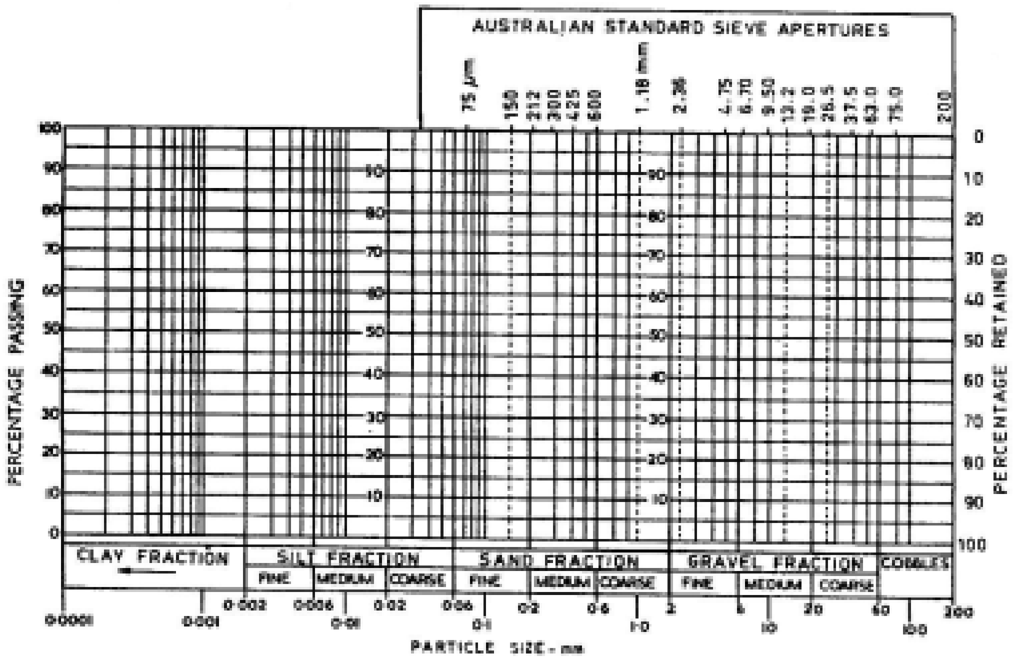


Figure 3.2: Semi-logarithmic chart for recording particle size distribution (from SAA, 1977).

3.3.2 Liquid limit

The method of liquid limit determinations followed those described in the methods of testing soil for engineering purposes AS1289.C1.1 (SAA, 1977).

3.3.2.1 Sample preparation

For each soil sample, an amount of approximately 300 g was taken from the material passing the 0.425 mm sieve after rubbing down larger aggregations to individual particle size. This fraction was then placed into a mixing bowl and mixed with distilled water added in increments until the soil became a thick, homogeneous paste. The soil was then covered and allowed to cure for at least 12 hours at room temperature.

3.3.2.2 Testing

After curing, the soil was then remixed and a portion placed into a standard Casagrande cup apparatus for determining liquid limits, with the rest remaining in the covered bowl. The soil was levelled off parallel to the base of the apparatus at a maximum depth of 10 mm, and a groove made through the whole depth of the soil across the diameter of the cup with a grooving tool and gauge of dimensions as specified in AS1289.C1.1 (SAA, 1977). The crank handle of the cup was then turned at a rate of 2 revolutions/second to lift and drop the cup repetitively to make the soil flow until the two parts of the soil came into contact at the bottom of the groove along a distance of 10 mm. The number of impacts of the cup on the base of the apparatus to close the groove to this extent was recorded. For this first test, if the number of impacts was between 35 and 45, the test was repeated until the difference in impacts between two consecutive tests was just one. At this point, about 10 g of soil was taken from the portion of soil just tested in the cup, and the moisture content determined using a standard oven-drying procedure as described in AS1289.B1.1 (SAA, 1977). If however the number of impacts for the first determination was above 45 or less than 35, then the soil was returned to the bowl, and having dried the whole 300 gram amount in air, or added water, was remixed and a portion retested in the cup, until the retest resulted in a figure of between 35-45 impacts. The test was then repeated until the difference in impacts between consecutive tests was just one, at which point the moisture content of the mixture was determined.

After this first determination, the mixture was then returned to the bowl and remixed thoroughly with the remainder. The test procedures described above were repeated at least 3 times, using portions from the 300 g sample to which an increment of water was mixed into the soil after each determination so that the final determination measured about 15 impacts of the cup.

3.3.2.3 Calculations

Moisture content % was plotted on an ordinal scale on the y-axis, against number of cup impacts on a logarithmic scale on the x-axis. A line of best fit was drawn through the 4-6 plotted points, and the liquid limit of the soil was measured as the moisture content that corresponded to 25 cup impacts (SAA, 1977).

3.3.3 Plastic limit and plasticity index

The method of plastic limit determinations followed those described in the methods of testing soil for engineering purposes AS1289.C2.1 (SAA, 1977).

3.3.3.1 Sample preparation

For each soil sample, an amount of approximately 50 g was taken from the material passing the 0.425 mm sieve after rubbing down larger aggregations to individual particle size. This fraction was then placed into a mixing bowl and mixed with distilled water added in increments until the material became homogeneous and sufficiently plastic to be shaped into a ball. The soil was then covered and allowed to cure for at least 12 hours at room temperature.

3.3.3.2 Testing

For each sample, a ball of soil of about 8 g was moulded in the hand and rolled between the palms until slight cracks appeared on its surface. The ball was then rolled between one palm and a frosted glass plate with just sufficient pressure to form a thread of soil of about 3 mm diameter, as compared with a steel rod of 3 mm diameter. If the thread crumbled prior to reaching 3 mm diameter, more distilled water was added to the whole soil mass, which was then remixed, and a new ball rolled, with the process repeated until the thread crumbled as it became 3 mm in diameter. If the thread rolled down to 3 mm diameter without crumbling, the 8 g soil portion was re-kneaded and re-rolled, again until the thread crumbled just as it reached 3 mm diameter. Three threads that crumbled at 3 mm diameter were rolled, and then collected into a container. The moisture content of this material was then determined using a standard oven-drying procedure as described in AS1289.B1.1 (SAA, 1977). A second group of three threads was then produced using the procedure described above, and the moisture content of this material measured.

3.3.3.3 Calculations

The plastic limit of the soil was taken as the average of the two moisture contents obtained. If the two values differed by more than 2 %, the test was repeated until the difference lay below this tolerance. The plasticity index was calculated as the difference between the Liquid Limit and the Plastic Limit in accordance with AS1289.C3.1 (SAA, 1977).

3.3.4 Linear Shrinkage

The linear shrinkage test measures the change in longitudinal dimension that occurs as a pasty mixture of soil mortar and water is dried at 110°C from near the liquid limit until the shrinkage ceases. The method of linear shrinkage determinations followed those described in the methods of testing soil for engineering purposes AS1289.C4.1 (SAA, 1977).

3.3.4.1 Sample preparation

For each soil sample, an amount of approximately 300 g was taken from the material passing the 0.425 mm sieve after rubbing down larger aggregations to individual particle size. This fraction was then placed into a mixing bowl and mixed with distilled water added in increments until the soil became a thick, homogeneous paste. The soil was then covered and allowed to cure for at least 12 hours at room temperature.

3.3.4.2 Testing procedure

After curing, each sample was remixed and water added to bring the material to a consistency near to the liquid limit. The material was then tested using the liquid limit procedure as described above, to obtain closure of the groove at 25 ± 3 impacts of the cup. With the soil at this consistency, it was placed inside a greased, brass shrinkage mould, shaped in the form of a semi-cylindrical trough of dimensions 250 mm internal length and 25 mm internal diameter. After over-filling the trough, air bubbles were removed from the soil by tapping, and excess material then removed with a knife. The specimen was allowed to dry for 24 hours at room temperature and then placed into an oven and dried at 105 °C until shrinkage ceased. The specimen was then cooled and its longitudinal shrinkage was measured to the nearest millimetre.

3.3.4.3 Calculations

The linear shrinkage percentage of the specimen was simply calculated as the longitudinal shrinkage divided by the length of the mould in millimetres, multiplied by 100.

3.3.5 Optimum moisture content and relationship between moisture content and density under compaction

The optimum moisture content of the soil-stabiliser mixture for each stabilised sample was determined in order to prepare the stabilised soil mixture at the appropriate moisture content for subsequent compressive strength testing. The optimum moisture content is determined by compacting the material at a number of different moisture contents, in order to establish the moisture content at which maximum dry density occurs. The procedure used is as described in AS1289.E2.1 (SAA, 1977).

3.3.5.1 Sample preparation

For each sample, the soil was brought to a state where it could be crumbled, if necessary by drying in air or in an oven at 40 °C. Aggregations of particles were broken up, so that if the sample were sieved using a 2.36 mm sieve, only discrete, uncrushed particles would be retained. The sample was then thoroughly mixed and subdivided by riffling until sufficient material to produce about 20 kg passing the 19 mm sieve was obtained. The sample was then sieved on the 19 mm sieve and the material retained on the sieve discarded. Five portions of the sieved soil were obtained by riffling of sufficient quantity, when wetted, to produce a compacted volume in excess of 0.001 m^3. Each portion was mixed thoroughly with the chosen stabiliser(s) to form a homogenous mixture, each with a different amount of water so that the optimum moisture content was straddled and the moisture steps remained reasonable. The increments of moisture content used for single samples ranged between 1.5 % for coarser samples to 3 % for clay-rich specimens, with the (previously determined) plastic limit of the soil being used to give an initial indication of the optimum moisture content.

3.3.5.2 Testing procedure

The specimen was then placed in stages into a collared, cylindrical mould of mass m_1 measuring 115 mm high and 105 mm inside diameter, whose inside surface was lightly oiled with castor oil. The material was then compacted by placing the material into the mould in five layers of equal thickness as determined using a gauge. Each layer was subjected to 25 uniformly distributed blows of a 4.9 kg rammer that was allowed to fall freely from a height of 450 mm above the compacting surface. The collar was removed and the soil levelled to the top of the mould using a straight edge, after which the mass (m_2) of the mould and soil was recorded. The specimen was then removed from the mould and a representative sample obtained using material from the full height of the specimen. The moisture content of this sub-portion of material was determined using a standard oven-drying procedure as described in AS1289.B1.1 (SAA, 1977).

The procedure described above was repeated for the other 4 portions of prepared soil. If subsequent calculations showed that the optimum moisture content had not been straddled or was not sufficiently precisely defined, further soil portions were prepared and tested.

3.3.5.3 Calculations

1. The dry density of each specimen was then calculated using:

$$\rho_d = [100(m_2\text{-}m_1)/v])/(100 + w)$$

Where

ρ_d = dry density of specimen
m_1 = mass of mould in g
m_2 = mass of mould plus specimen in g
v = internal volume of mould in cm^3
w = moisture content of specimen (as %)

For each specimen, dry density was plotted against moisture content for the five determinations. A smooth curve was drawn through the points and the position of the maximum dry density marked on the curve and recorded. The moisture content corresponding to the maximum dry density was also recorded as the optimum moisture content of the sample.

3.3.6 Unconfined compressive strength and maximum dry density

The method of determining the unconfined compressive strength of stabilised earth samples followed the procedure as specified in Test Method T116 of the Department of Main Roading, New South Wales (DMR, 1983), and is described below. It should be noted that the procedure for preparing and compacting the samples prior to strength testing is identical to that described and used in Section 3.3.5 above.

3.3.6.1 Preparation of samples

For each sample, aggregations of particles were broken up such that when screened on a 4.75 AS sieve only discrete, uncrushed particles were retained. The sample was then screened on a 37.5 mm AS sieve, and the material retained on the sieve was discarded. The sample was then reduced by riffling to provide a portion sufficient to yield not less than 3.0 kg passing a 19.0 mm AS sieve. The soil was then screened on a 19.0 mm AS sieve, and the material passing through was retained.

For each compressive strength determination, 3.0 kg of soil were used. The moisture content of a 300 g portion of the soil was determined using a standard oven-drying procedure as described in AS1289.B1.1 (SAA, 1977). The remaining portion was mixed thoroughly with the chosen stabiliser(s) to form a homogenous mixture, and sufficient water, as calculated, was added to this mass to produce the optimum moisture content appropriate for the sample as determined beforehand using the procedure described above for determining the optimum moisture content.

Each specimen was then placed in stages into a collared, cylindrical mould of mass m_1 measuring 115 mm high and 105 mm inside diameter, whose inside surface was lightly oiled with castor oil. The material was then compacted by placing the material into the mould in five layers of equal thickness as determined using a gauge. Each layer was subjected to 25 uniformly distributed blows of a 4.9 kg rammer that was allowed to fall freely from a height of 450 mm above the compacting surface. The mould was slightly overfilled, and then the specimen was then trimmed with a straightedge and smoothed to ensure the top surface was plane to within 0.1 mm. The mass of the mould and specimen was recorded as m_2. Using a quantity of 300 g of the excess mixture, the moisture content was determined using a standard oven-drying procedure as described in AS1289.B1.1 (SAA, 1977).

The specimen was then carefully ejected from the mould and inspected for deformities, cracks, or other defects. Non-perfect specimens were discarded. Each specimen was then stood on a filter paper, placed in a humidity cabinet, and allowed to cure at a temperature of 22 °C for 28 days.

After 28 days of curing, each specimen was immersed in water at room temperature for a period of 4 hours, and allowed to drain for 15 minutes. Each specimen was then inspected and any specimens that exhibited cracking between the different layers were discarded. Specimens whose end surfaces were not plane within 0.1 mm were capped with plaster of paris to produce a smooth surface. Each specimen was allowed to stand at constant moisture content for one hour before testing for compressive strength.

3.3.6.2 Compression testing

The compressive strength of each 28-day cured sample was obtained using a KingTest compression testing machine (Figure 3.3), which conforms to the apparatus requirements of TS116. Each specimen was placed on the lower bearing block of the machine with the vertical axis of the test specimen aligned with the centre of thrust of the upper block. The upper block was brought to bear on the specimen without shock, and uniformly seated on the specimen top surface. The continuous, applied loading rate during compressive testing was 0.10 ± 0.02 MPa/second. The applied load was continued until each specimen failed, and the load at failure recorded to the nearest 0.05 MPa.

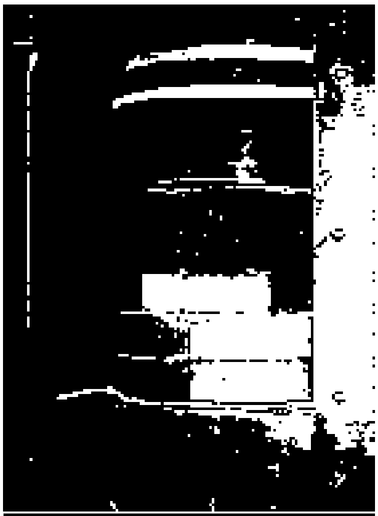


Figure 3.3: Machine used for measuring unconfined compressive strength of stabilised specimens in this study.

3.3.6.3 Calculations

1. The dry density of each specimen was then calculated using:

$\rho_d = [100*(m_2-m_1)/v])/(100 + w)$

Where

ρ_d = dry density of specimen

m_1 = mass of mould in g

m_2 = mass of mould plus specimen in g

v = internal volume of mould in cm3

w = moisture content of specimen

2. The unconfined compressive strength of each test specimen was calculated as the quotient of applied load and surface area, as follows:

C = W/A

Where

C = unconfined compressive strength (MPa)

W = applied load at which the specimen fails (kN)

A = area of test specimen top surface (= or^2 ,where r = specimen radius)

3. Each compressive strength determination was then adjusted for aspect ratio (specimen height divided by specimen diameter) as described in CSIRO (1987). As this ratio was constant for the tests (115/105 = 1.10), the aspect ratio correction factor used for all specimens was 0.72 (Table E1, CSIRO 1987). The adjusted compressive strength was calculated by:

$$Ca = kaC$$

Where
Ca = the adjusted unconfined compressive strength
ka = the aspect ratio correction factor (in this case 0.72)
C = unconfined compressive strength

All strength determinations generated in this study are adjusted saturated unconfined compressive strengths.

CHAPTER 4: DATA ANALYSIS AND INTERPRETATION

4.1 AIMS OF THE DATA ANALYSIS

The overall aim of the data analysis is to examine the data in such a way as to address the research objectives identified in Chapter 1. In brief, the overall objective is to determine the relationships between measured soil properties, stabiliser treatments (stabiliser type and quantity), and stabilised rammed earth properties. Ultimately, this should provide information regarding the construction of rammed earth walls that will help avoid poor stabilisation and establish guidelines for successful stabilisation that can be used in the future.

The aims of the data analysis are: to assess which variables are the most (and least) important in stabilisation; to separate the influence of soil properties on sample strength and density from the influence of stabiliser treatments; to identify threshold values of soil variables that define successful stabilisation; and to differentiate between poor samples and successfully stabilised samples on the basis of their soil properties and stabiliser treatments. These aims lead toward a goal of trying to predict the outcome of stabilisation based upon information about the properties of the natural soil.

4.2 TYPES AND QUALITIES OF THE DATA

The dependent variables of compressive strength and density are both continuous variables measured on an open scale and can theoretically take any value. They are ratio scale data, the best quality and most versatile data type (Swan & Sandilands, 1995). Although measured on a continuous scale, the stabiliser variables are regarded as categorical variables, as they have been used as treatments at set levels. These stabilisers and levels are asphalt (0 and 3 %), cement (0, 2, 3, 4, 5, and 6 %), and lime (0, 2, 3, 4, 5, and 6 %). In addition, there are five types of stabiliser combination: cement only; lime only, cement-lime; cement-asphalt; cement-lime-asphalt. Asphalt has not been used except in combination with cement and/or lime.

The soil property variables are all continuous variables but are closed data as they are measured as percentages. In some cases closed data can cause problems in analysis, as they are fundamentally interdependent. For example, in this study, the three textural properties (clay/silt, sand, gravel) sum to 100 %; also, plasticity index is calculated as the difference between liquid limit and plastic limit. This interdependency will tend to exaggerate the strength of relationships between variables, unless caution is taken in interpretation, or the most highly correlated variables are removed from statistical models (as done here).

The distribution of the data for each variable is also an important consideration in some statistical methods. Parametric statistical tests make some assumptions about the underlying data including the assumption of data normality. However, for large sample sizes (n >100), such as in this study (with over 200 samples), it can be assumed from the central limit theorem that the sample means will follow the normal distribution even if the respective variable is not normally distributed in the population (StatSoft, 2000). In addition, some parametric tests (e.g. analysis of variance) are resistant to deviations in normality, should they be present. The parametric measures used in this study include the mean and standard deviation; the statistical methods used include one-way analysis of variance (one-way ANOVA), analysis of covariance (ANCOVA), and multiple regression, which all fall under the category of General Linear Models (Bowerman & O'Connell, 1990).

Some of the variables have been coded; in other words, continuous data have been binned into categories or classes. There are two types of coding of the data, reported in Table 4.1:

1. The samples have been coded into "failed" (0) code and "successful" (1) with respect to their stabilisation. Successful for compressive strength code means ≥ 2.0 MPa; successful for density code means ≥ 1.80 t/m^3. They have been coded in order to assess the characteristics of samples that pass and fail according to the pre-defined success criteria (Section 3.2.2).
2. The samples have been coded into classes for each soil property, in order to help examine trends in stabilisation success rate across the range of values of soil property variables. The classes have been set to between 6 and 10 classes (groups)

for each variable, to retain reasonable group sizes but at the same time to effectively cover data variation through the range of each variable.

4.3 STATISTICAL METHODS USED

4.3.1 Summary statistics and graphical methods

Statistics such as the mean, median, and standard deviation provide useful summaries of trends and variations within the data set. Univariate and bivariate scatterplots and graphic expression of mean values also assist with qualitative assessment of data. Coded and classed data are used to examine the characteristics of samples of contrasting compressive strength or density.

4.3.2 Analysis of Variance (ANOVA)

ANOVA is one type of statistical design that falls under the class of General Linear Models (GLMs). A wide variety of types of designs can be analysed using the general linear model, including analysing linear (and non-linear, e.g. polynomial) effects of continuous and categorical predictor variables on a discrete or continuous dependent variable (StatSoft, 2000).

Analysis designs that contain only categorical predictor variables are termed ANOVA (analysis of variance) designs, designs that contain only continuous predictor variables are called multiple regression designs, and designs that contain both categorical and continuous predictor variables are called ANCOVA (analysis of covariance) designs. The most basic form of ANOVA is termed one-way ANOVA and involves just one independent variable and a single categorical predictor variable.

4.3.2.1 The purpose and basic logic of ANOVA

The purpose of analysis of variance is to test differences in means (of groups or variables) for statistical significance. This is accomplished by analyzing the variance, that is, by partitioning the total variance into the component that is due to true random error ("within-group sums of squares") and the components that are due to differences between means ("between-group sums of squares") (Bowerman & O'Connell, 1990).

These latter variance components are then tested for statistical significance, and, if significant, the null hypothesis of no differences between means is rejected, and the alternative hypothesis is accepted that the means (in the population) are different from each other.

4.3.2.2 Sums of Squares

The basis of ANOVA is the fact that variances can be divided up, that is, partitioned. The variance is computed as the sum of squared deviations from the overall mean, divided by n-1 (sample size minus one). Thus, given a certain n, the variance is a function of the sums of (deviation) squares, or SS for short. A fundamental principle of least squares methods is that variation on a dependent variable can be partitioned, or divided into parts, according to the sources of the variation. Stated formally, the total sum of squared deviations of the observed values on the dependent variable from the dependent variable mean is equal to (1) the sum of squared deviations of the predicted values for the dependent variable from the dependent variable mean and (2) the sum of the squared deviations of the observed values on the dependent variable from the predicted values, that is, the sum of the squared residuals (Bowerman & O'Connell, 1990). Stated another way, Total SS = Model SS + Error SS.

The Total SS is always the same for any particular data set, but the Model SS (also known as Effect SS) and the Error SS (within-group variability, or error variance) depend on the linear equation. The error term denotes the fact that it cannot be readily explained or accounted for in the particular design used. However, the Effect SS can be explained, as it is due to the differences in means between the groups (Swan & Sandilands, 1995).

4.3.2.3 Significance testing

Many statistical tests represent ratios of explained to unexplained variability. In ANOVA, this test is based on a comparison of the variance due to the between-groups variability (called Mean Square Effect, or MS_{effect}) with the within-group variability (called Mean Square Error, or Ms_{error}). Under the null hypothesis (that there are no differences between the means for groups in the population), minor random fluctuations would still be expected in the means for the two groups when taking small samples. Therefore, under the null hypothesis, the variance estimated based on within-group variability should be about the same as the variance due to between-groups variability ($F = 1$). These two estimates of variance can be compared via the F test which tests whether the ratio of the two variance estimates is significantly greater than 1. The test results in a measure of significance (the p-value). If the test were significant (e.g. $p < 0.05$ means the test is significant at better than 95 % level of confidence), it would be concluded that the means for two (or more) groups are significantly different from each other (StatSoft, 2000).

4.3.2.4 Assumptions and effects of violating the assumptions

Normal Distribution

As ANOVA is a parametric test, the dependent variable should be normally distributed within groups. Overall, the F test is remarkably robust to deviations from normality. Neither the skewness or the kurtosis of the distribution usually does not have a sizable effect on the F statistic. If the N per group is fairly large, as in this study, then deviations from normality do not matter much at all because of the central limit theorem, according to which the sampling distribution of the mean approximates the normal distribution, regardless of the distribution of the variable in the population (Bowerman & O'Connell, 1990).

Homogeneity of variances

It is assumed that the variances in the different groups of the analysis design are identical; this assumption is called the homogeneity of variances assumption. The error variance (SS error) is calculated by adding up the sums of squares of each group. If the variances in the two groups are different from each other, then adding the two together is not appropriate, and will not yield an estimate of the common within-group variance (since no common variance exists). However, the *F* statistic is quite robust against violations of this assumption (StatSoft, 2000).

Correlations between means and variances

The validity of significance tests may be threatened when the means for categories across groups are correlated with the variances (or standard deviations).

Intuitively, if there is large variability in groups with particularly high means, then those high means are not reliable. However, the overall significance tests are based on pooled variances, that is, the average variance across all groups. Thus, the significance tests of the relatively larger means (with the large variances) would be based on the relatively smaller pooled variances, resulting erroneously in statistical significance. In practice, this pattern may occur if one group contains a few extreme outliers, which have a large impact on the means, and also increase the variability.

4.3.3 Analysis of Covariance (ANCOVA)

The principles of ANOVA can be extended beyond single-predictor situations to situations involving multiple factors. Many natural systems are complex and multivariate in nature, and instances when a single variable completely explains a dependent variable are rare. ANCOVA provides a way to analyse the effect on the dependent variable of more than one factor, when there are both categorical variables and continuous variables. ANCOVA designs have traditionally referred to designs in which the first-order effects of one or more continuous predictor variables (here, soil property variables) are taken into account when assessing the effects of one or more categorical predictor variables (here, stabilisers). For situations with multiple predictor variables, the total variance can be apportioned into the variability due to each of the predictor variables, and the error. In ANCOVA it is possible to test each factor while controlling for all others; this is the reason why the technique is statistically very powerful (StatSoft, 2000).

4.3.4 Multiple regression

Multiple regression also falls under the category of General Linear Models (Bowerman & O'Connell, 1990). It is used when there are several continuous predictor variables (but not categorical ones) and the independent variable is continuous. The same general principles outlined above also apply to multiple regression.

4.3.5 Kruskal-Wallis test

The Kruskal-Wallis test tests for significant differences in medians between groups, and relies on ranking the values in each sample. It is the nonparametric equivalent of ANOVA. The null hypothesis tested is that all samples are drawn from populations with identical medians. This is rejected if at least one of the medians is rejected.

4.4 INTRODUCTORY DESCRIPTION OF DATA

Table 4.1 (contained in Appendix 1) reports the data generated in this study, which have been measured as described in Chapter 3. With more than 2500 individual measurements, the data set is a large one, and in order to introduce the data the following 2 sections explore the summary statistics and data distributions of the variables.

4.4.1 Summary statistics

Table 4.2 displays basic summary statistics for the 2 dependent variables (compressive strength and density). The mean compressive strength of the samples is 2.62 MPa, with a minimum of 1.00 and a maximum of 5.40 MPa. There is a reasonable amount of variation in compressive strength, as shown by the range and by the standard deviation of 0.97 MPa. The mean density of the samples is 1.86 t/m^3, and ranges between 1.44 and 2.21 t/m^3. The average particulate makeup of the samples is 24 % clay/silt (range 5-53 %), 68 % sand (range 30-94 %), and 8 % gravel (range 0-62 %). Liquid limit ranges between 18 and 95 %, and plasticity index between 0 and 70 %. The minimum linear shrinkage value is 1.0 % and the maximum 19.8 %. Optimum moisture content ranges between 5.4 and 28.0 %. This wide variation in values of the variables is favourable for examining relationships between them.

	N	Mean	Median	St Dev	Min	Max
UCS (MPa)	230	2.62	2.45	0.97	1.00	5.40
Density (t/m^3)	202	1.86	1.90	0.18	1.44	2.21

Table 4.2: Summary statistics for unconfined compressive strength and maximum dry density.

4.4.2 Distributions of dependent variables

Figures 4.1a and 4.1b (contained in Appendix 2) show frequency histograms and estimated population curves for the 2 dependent variables (compressive strength and density), using the same number of observations (N) for each graph as reported for the data in Table 4.2. These plots assist in obtaining a feel for the data, and help to examine how well the data approximate a normal distribution. Figures 4.1a and 4.1b indicate that the two criterion variables display reasonable approximations to normality.

4.5 VARIATION IN COMPRESSIVE STRENGTH AND DENSITY WITH RESPECT TO SOIL PROPERTIES

Both the properties of the natural soil and the type and quantity of stabilising additives have the potential to influence the compressive strength and density of rammed earth. An initial approach to assessing these influences is to examine the bivariate relationships between soil properties and the independent variables.

4.5.1 Variation in compressive strength and soil properties

Figures 4.2-4.5 show compressive strength plotted against the 3 soil particle properties, and are plotted regardless of stabiliser type or quantity. Figure 4.2, compressive strength versus % clay/silt, indicates that there is a range of values of compressive strength for any given clay/silt content. For example, for clay/silt content of 16 %, compressive strength varies from 1.2 MPa to greater than 5 MPa. The highest values of compressive strength are associated with clay/silt contents of between 10 to 35 %. For clay/silt contents between 5-20 %, the data are almost bimodal, with one group of data < 2.3 MPa and a second group > 2.6 MPa. Figure 4.3 indicates that these two groups of samples correspond to differences primarily in plasticity index, with the weaker samples having generally higher values of plasticity index. If this group of weaker samples in this group were ignored, there would be a distinct trend of decreasing sample strength with increasing % clay/silt over the whole range of clay/silt content. This is an example of how the patterns shown by bivariate graphs such as Figures 4.2, and 4.4-4.10, partially reflect the influences of the other variables, and for which caution must be taken in interpretation. ANCOVA is used further below as a technique to account for these inter-variable correlations.

The pattern of data in Figure 4.4 (compressive strength versus gravel) suggests that the highest values of compressive strength occur with gravel contents of between 20 to 45 %, and in this range there are very few samples with compressive strength < 2 MPa. However, this is probably a result of the sparseness of the data in this range of gravel content. In the range of gravel content < 20 %, where data are more numerous, there is a wide range of compressive strength values for any given gravel content. Sand content (Figure 4.5) ranges from 30 to 94 %., and the relationship with compressive strength appears rather complex. Samples containing less than 48 % sand exhibit high values with only very few samples below 2 MPa. However, this again may reflect the lower number of samples tested with sand contents in this particular range. Where the data are more numerous, the most interesting feature is the area between 64-75 % sand, where there is an absence of samples with high values of compressive strength. For sand values in excess of about 75 %, there are two groups of samples differentiated by compressive strength. Most of these samples correspond to the <25 % clay/silt samples, separated into two groups by differences in plasticity index, as described above and as featured in Figure 4.3.

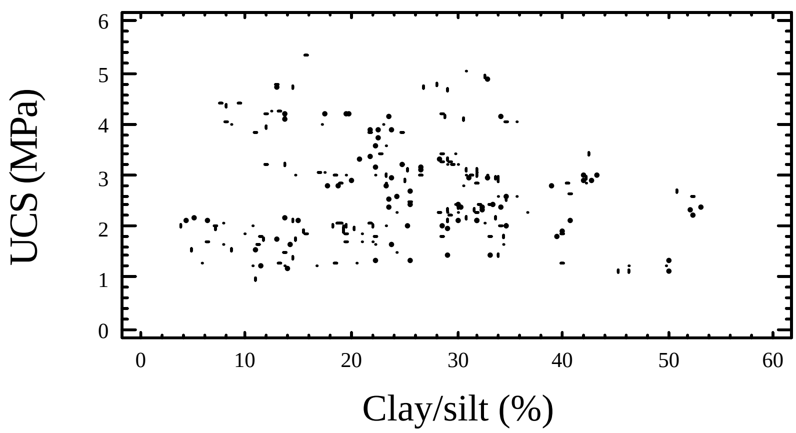


Figure 4.2: X-Y scatterplot of compressive strength versus clay/silt content.

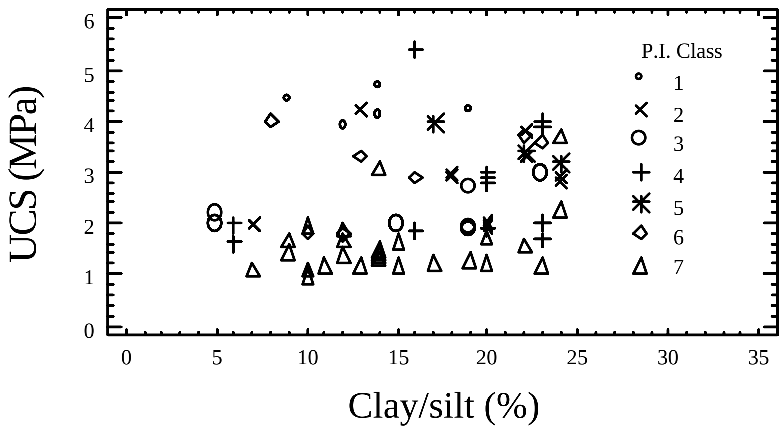


Figure 4.3: X-Y scatterplot for compressive strength versus clay/silt, coded by plasticity index class. Lower classes represent lower values of plasticity index The classes are defined in the caption of Table 4.1 (contained in Appendix 1).

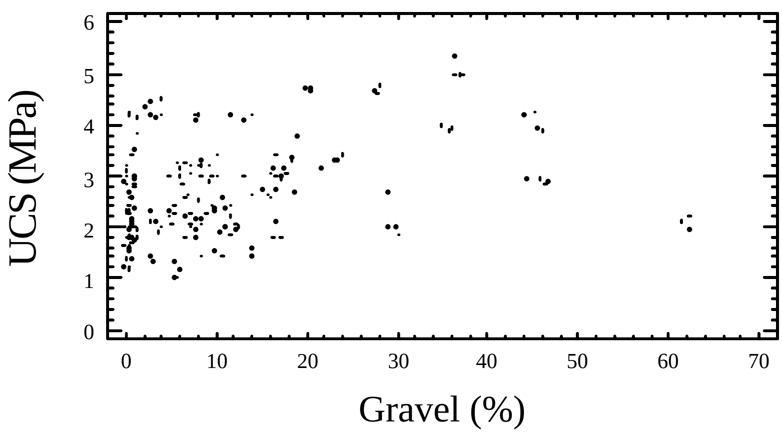


Figure 4.4: X-Y scatterplot of compressive strength versus gravel content.

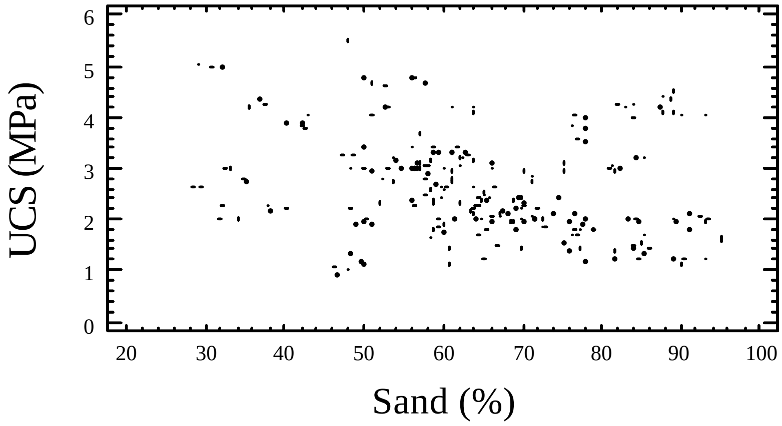


Figure 4.5: X-Y scatterplot of compressive strength versus sand content.

Figures 4.6 through 4.9 show bivariate scatterplots of compressive strength versus plasticity properties of the soil (liquid limit, plastic limit, plasticity index), and versus linear shrinkage. The relationship between compressive strength and liquid limit (Figure 4.6) is an interesting one. For values of liquid limit below 50 %, there is a very wide range of values of compressive strength ranging between about 1.5 to 5 MPa. For values of liquid limit in excess of around 60 %, all samples have a compressive strength below 2 MPa. This is in contrast to Figure 4.7, which shows that for any one value of plastic limit, there is a very wide range of values of compressive strength, and there is no trend to the data. For samples with plastic limit >28 %, the lack of any samples with compressive strength below 3 MPa is likely due to the paucity of samples with such plastic limits. As the relationship between compressive strength and plastic limit is featureless, the pattern of data in Figure 4.8 (compressive strength versus plasticity index) reflects the pattern of the relationship between compressive strength and liquid limit. For values of plasticity index between 0 to 35 %, there is a wide range of values of compressive strength for any one value of plasticity index. There is also a slight negative trend to the data as plasticity index increases to 35 %. Above this figure, no sample tested has a compressive strength of more than 2 MPa. The relationship between compressive strength and linear shrinkage differs from those of the 3 plasticity properties described (Figure 4.9). Although for any one value of linear shrinkage there exist a range of values of compressive strength, there is a distinct negative trend to the data with higher values of linear shrinkage generally being associated with lower values of compressive strength. For values of linear shrinkage up to 3.5 %, no sample has a compressive strength lower than 2 MPa. As linear shrinkage values increase, the maximum compressive strength decreases and a greater proportion of samples have a compressive strength below 2 MPa (Figure 4.9).

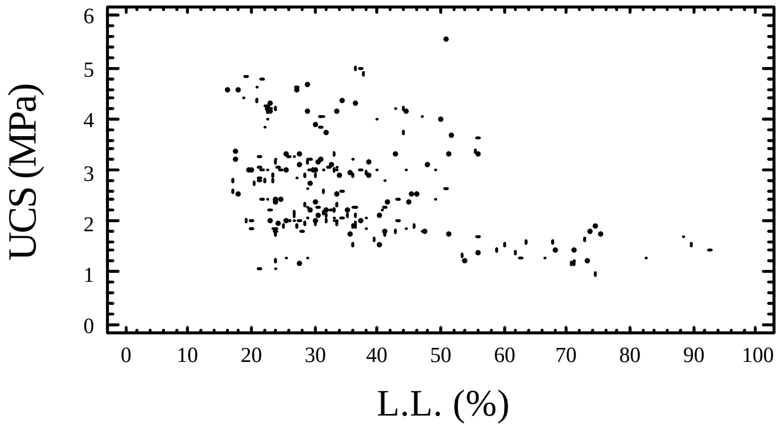


Figure 4.6: X-Y scatterplot of compressive strength versus liquid limit.

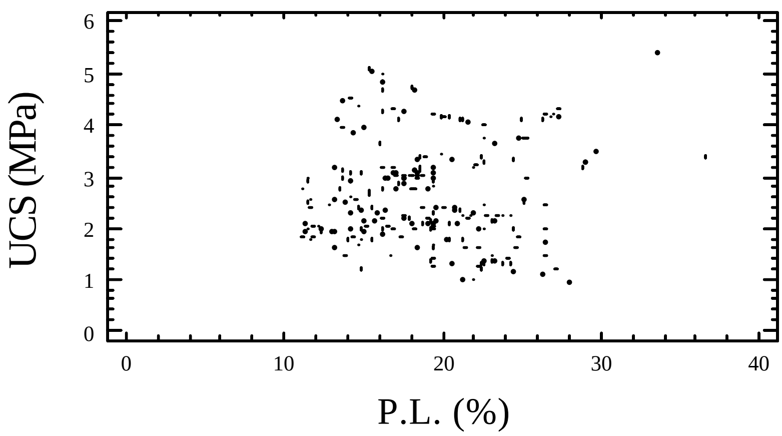


Figure 4.7: X-Y scatterplot of compressive strength versus plastic limit.

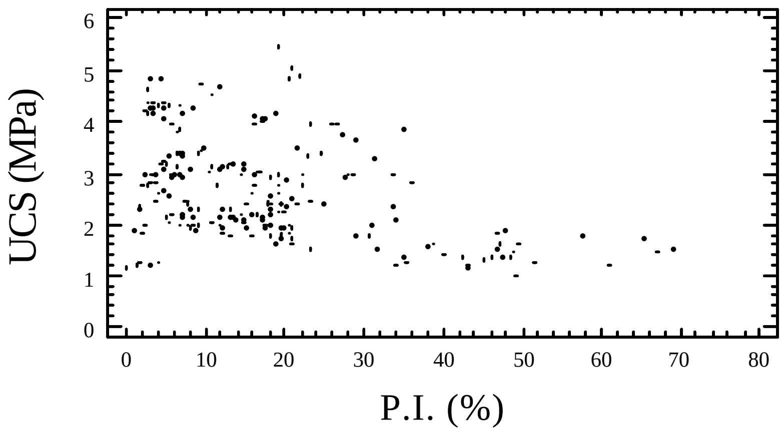


Figure 4.8: X-Y scatterplot of compressive strength versus plasticity index.

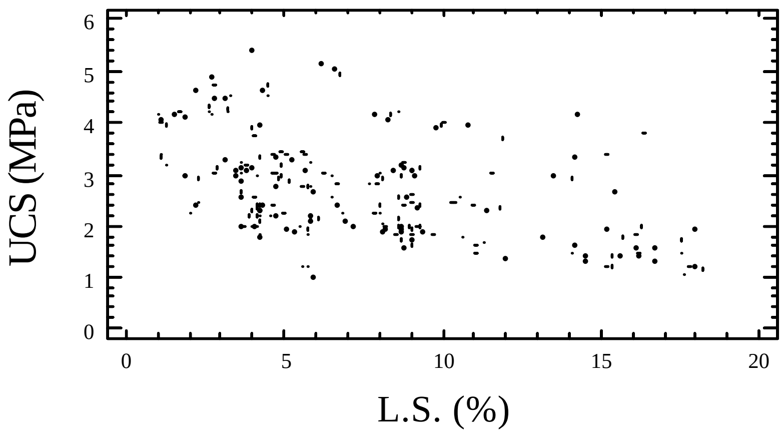


Figure 4.9: X-Y scatterplot of compressive strength versus linear shrinkage.

The moisture content of samples is related to compressive strength (Figure 4.10) in a manner not unlike liquid limit and plasticity index. Moisture values from 5 to 13 % are associated with values of compressive strength ranging from about 1.5 MPa to 5 MPa. Compressive strength values appear to peak at moisture contents of between 7 and 9 %. Moisture levels in excess of 13 % are associated with low compressive strength values generally below 2 MPa.

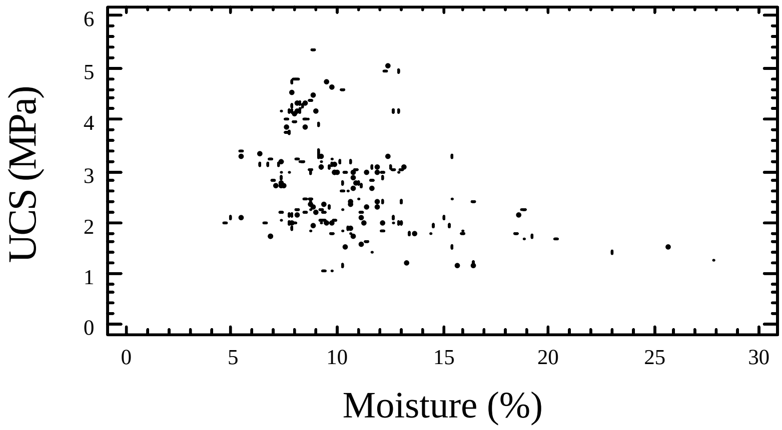


Figure 4.10: X-Y scatterplot of compressive strength versus moisture content.

4.5.2 Variation in density and soil properties

Relationships between density and soil properties are generally similar to those for compressive strength and soil properties. Figures 4.11-4.13 show density plotted against each of the 3 soil particle properties. Figure 4.11, density versus % clay/silt, shows no strong relationship between the two variables. Values of density are a maximum (2.2 t/m^3) when % clay/silt is around 10-14 %, and the maximum density declines as clay/silt content increases. The pattern of data in Figure 4.12 (density and gravel) reveals that low-density samples are very rare for values of gravel over 20 %, with just a single sample having a value lower than 1.9 t/m^3 in this range. The relationship between density and sand (Figure 4.13) is somewhat featureless, except that there is a lower proportion of low-density samples for sand values of less than 48 %, although this may reflect sampling sparseness.

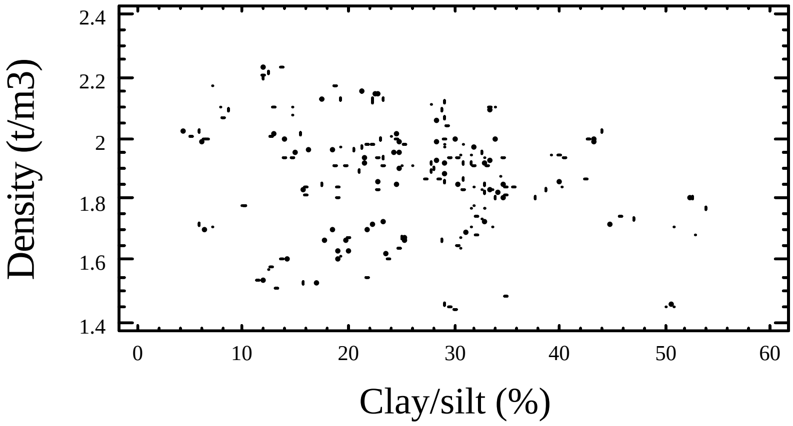


Figure 4.11: X-Y scatterplot of density versus clay/silt content.

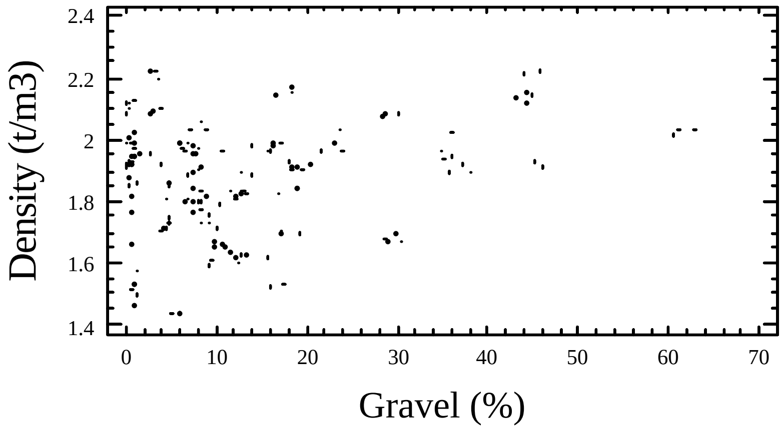


Figure 4.12: X-Y scatterplot of density versus gravel content.

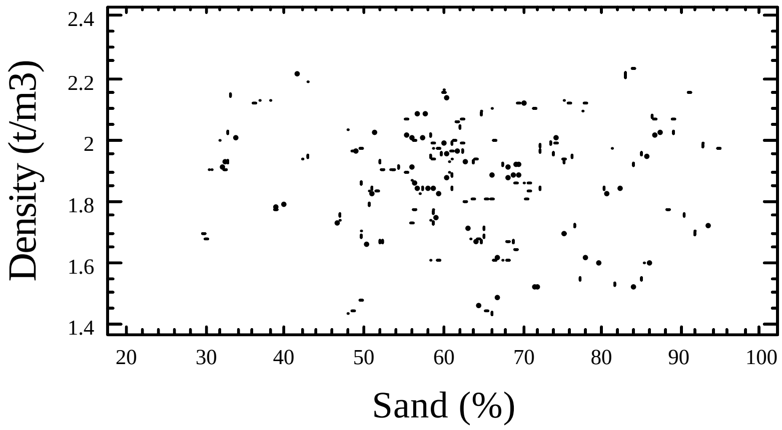


Figure 4.13: X-Y scatterplot of density versus sand content.

Figures 4.14-17 show bivariate scatterplots of density versus the plasticity measures and linear shrinkage. For values of liquid limit up to 55 % (Figure 4.14), there is a very wide range of values of density ranging between about 1.4 to 2.2 t/m^3. For values of liquid limit in excess of 58 %, all samples have low densities of less than 1.6 t/m^3. This is in contrast to Figure 4.15, which shows that for any one value of plastic limit, there is a very wide range of values of density, and there is no trend to the data. For samples with plastic limit greater than 25 %, the lack of any samples with low densities may be due to the paucity of samples with such plastic limits. As the relationship between density and plastic limit is virtually a trendless cloud of data points, the pattern of data in Figure 4.16 (density versus plasticity index) reflects the pattern of the relationship between density and liquid limit. For values of plasticity index between 0 to 35 %, a wide range of values of density are exhibited for any one value of plasticity index. Above a threshold of 35%, densities are around 1.6 t/m^3. Figure 4.17 shows that although for any one value of linear shrinkage there exist a range of values of density, there is a negative slope data trend with higher values of linear shrinkage generally being associated with lower values of density.

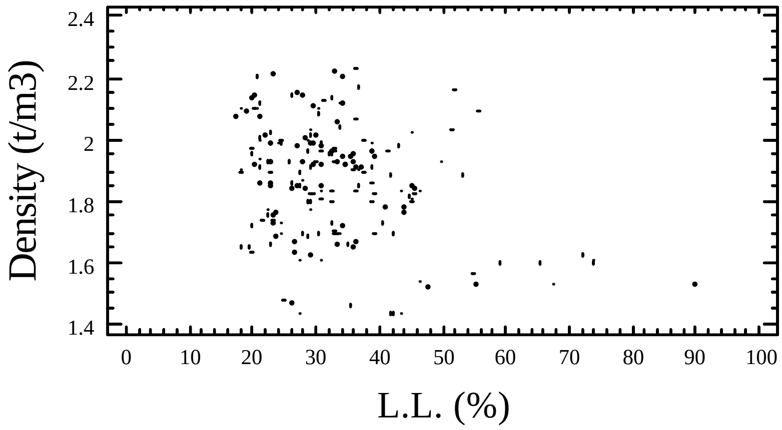


Figure 4.14: X-Y scatterplot of density versus liquid limit.

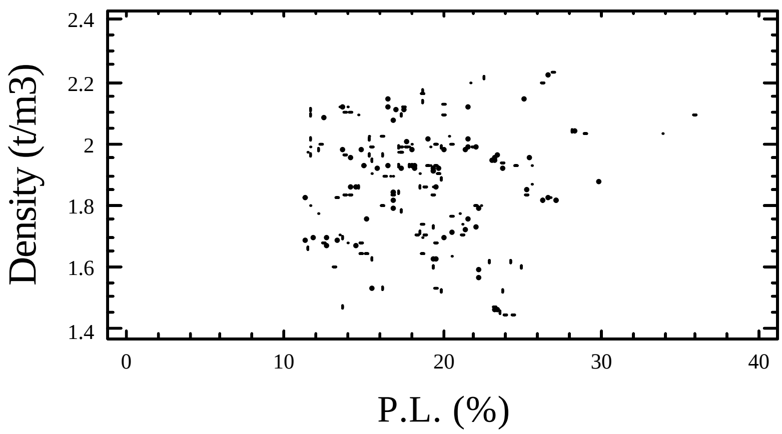


Figure 4.15: X-Y scatterplot of density versus plastic limit.

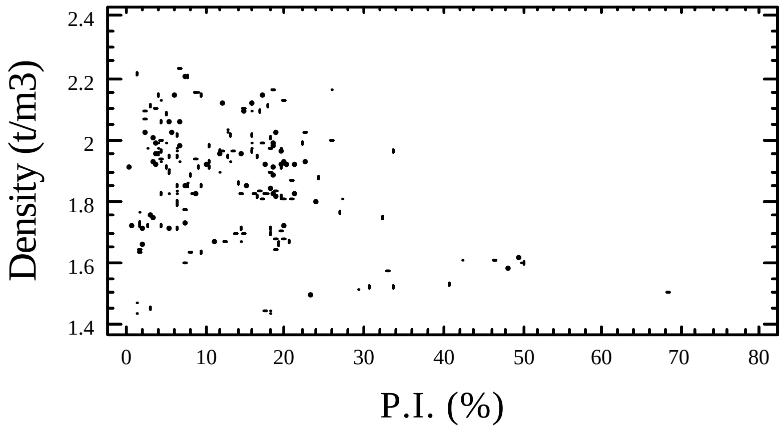


Figure 4.16: X-Y scatterplot of density versus plasticity index.

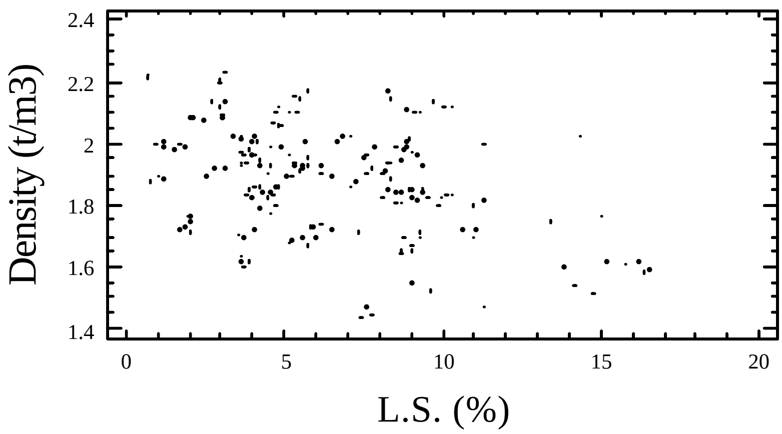


Figure 4.17: X-Y scatterplot of density versus linear shrinkage.

The density of samples is rather closely related to moisture content (Figure 4.18), which is expected because the maximum dry density for any one sample is well known to be a function of moisture. The relationship is characterised by peak densities (2.20 t/m^3) being achieved in the range of moisture contents from 7 to 9 %. Above this figure, peak densities decrease to reach 2.0 t/m^3 at moisture levels of 12 %, 1.80 t/m^3 at 16-18 % moisture, and just 1.65 t/m^3 at a moisture content of 22 %.

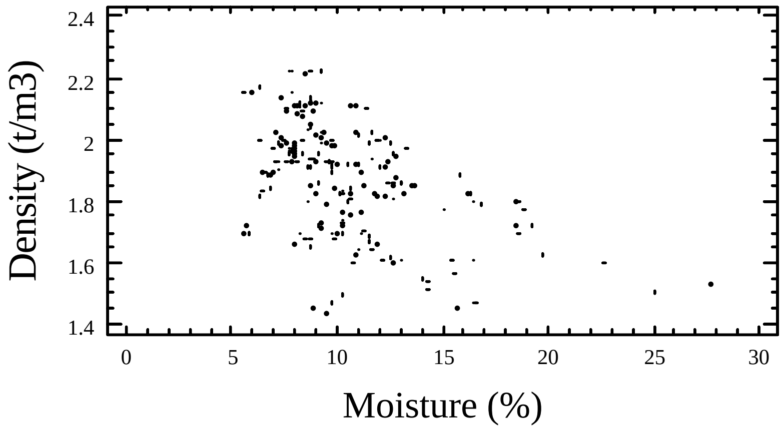


Figure 4.18: X-Y scatterplot of density versus moisture content.

4.6 CORRELATIONS BETWEEN CONTINUOUS VARIABLES

Correlations give a simple insight into the degree of (linear) relationship between two variables. Table 4.3 shows Pearson correlation coefficients between the soil property variables. Disregarding the correlations between the three textural variables, and between the 3 Atterberg limits, as these variables are correlated by definition, the highest correlations are demonstrated by: Linear shrinkage and plasticity index; linear shrinkage and liquid limit; moisture content and plasticity index; moisture content and linear shrinkage; moisture content and liquid limit; liquid limit and sand content; and moisture content and clay/silt content. All of these correlations are positive. Table 4.3 indicates that each variable is correlated to some degree with most other variables.

Table 4.4 reports Pearson correlation coefficients between the soil property variables and the criterion variables. Most of the soil properties are negatively correlated with compressive strength, with the highest negative correlations being between compressive strength and moisture content, linear shrinkage, and plasticity index. The highest positive correlation is with gravel, but the scattergraph of compressive strength and gravel indicates that this may at least partly be a function of the low number of samples that have higher gravel contents. Indeed, for all these correlation pairs, reference should also be made to the appropriate scattergraphs above (Figures 4.3-4.18). The highest negative correlations for density are with moisture content, plasticity index, and liquid limit. Interestingly, the correlation of density with linear shrinkage is positive. These correlations between the soil property variables and the criterion variables (Table 4.4) do not take into account the correlations between the different soil property variables (Table 4.3). The potential complication that this may cause for data interpretation is addressed below.

	Clay/ silt	Gravel	Sand	L.L.	P.L.	P.I.	L.S.	Moisture
Clay/silt		**-0.24 0.001**	**-0.47 0.000**	**-0.20 0.006**	0.04 0.61	**-0.19 0.010**	-0.009 0.21	**0.24 0.001**
Gravel	**-0.24 0.001**		-0.75 0.000	**-0.13 0.008**	-0.04 0.62	-0.10 0.18	**-0.15 0.041**	**-0.22 0.003**
Sand	**-0.47 0.000**	**-0.75 0.000**		**0.25 0.001**	0.01 0.91	**0.21 0.003**	**0.19 0.007**	0.03 0.68
L.L.	**-0.20 0.006**	**-0.13 0.008**	**0.25 0.001**		**0.37 0.000**	**0.89 0.000**	**0.76 0.000**	**0.38 0.000**
P.L.	0.04 0.61	-0.04 0.62	0.01 0.91	**0.37 0.000**		-0.06 0.44	-0.14 0.050	-0.03 0.68
P.I.	**-0.19 0.010**	-0.10 0.18	**0.21 0.003**	**0.89 0.000**	-0.06 0.44		**0.89 0.00**	**0.46 0.000**
L.S.	-0.009 0.21	**-0.15 0.041**	**0.19 0.007**	**0.76 0.000**	-0.14 0.050	**0.89 0.00**		**0.46 0.000**
Moisture	**0.24 0.001**	**-0.22 0.003**	0.03 0.68	**0.38 0.000**	-0.03 0.68	**0.46 0.000**	**0.46 0.000**	

Table 4.3: Pearson correlations between the soil property variables, using the 193 samples for which all variables were measured. The first figure in each cell is the correlation coefficient, the second figure is the significance level. Bold cells indicate a statistically significant linear relationship between two variables (significance level <0.05).

	UCS	Density
Clay/silt	-0.10 0.18	**-0.14** **0.040**
Gravel	**0.32** **0.000**	**0.16** **0.030**
Sand	**-0.22** **0.002**	-0.05 0.53
L.L.	**-0.27** **0.000**	**-0.37** **0.000**
P.L.	**0.16** **0.034**	0.07 0.36
P.I.	**-0.35** **0.000**	**-0.40** **0.000**
L.S.	**-0.46** **0.000**	**0.40** **0.000**
Moisture	**-0.34** **0.000**	**-0.47** **0.000**

Table 4.4: Pearson correlations between the soil property variables and the criterion variables of compressive strength and density, using the 193 samples for which all variables were measured. The first figure in each cell is the correlation coefficient, the second figure is the significance level. Bold cells indicate a statistically significant linear relationship between two variables (significance level <0.05).

4.7 INFLUENCE OF STABILISERS AND SOIL PROPERTIES ON UCS: ANCOVA

4.7.1 Introduction

There are two complications that preclude a simple determination of the relationship between the predictor variables and compressive strength or density. The first is that the soil property predictor variables are correlated amongst themselves (Table 4.3), meaning that examination of the influence of just a single predictor variable on compressive strength (or density) will not show the unique influence of that variable. For example, Figure 4.8 shows the relationship between plasticity index and compressive strength; however, % clay/silt is correlated with plasticity index with higher values of plasticity index generally being associated with lower values of clay/silt (Table 4.3 and Figure 4.19). Therefore, the pattern of Figure 4.8, which shows that higher values of plasticity index are associated with low values of compressive strength, may be partly due to the influence of clay/silt whereby some of the lower values of clay/silt may help explain some of the lowest values of compressive strength. In similar fashion, other predictor variables are inter-related to some extent (Table 4.3), and there is a need to take this into account in order to isolate the unique effect of each variable on compressive strength (and density).

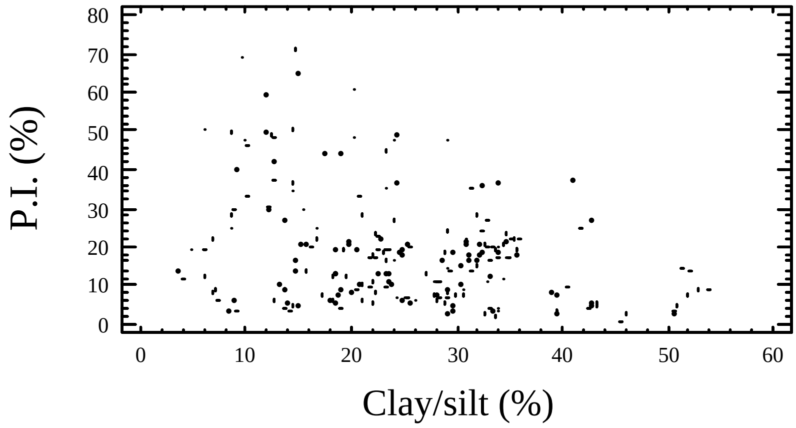


Figure 4.19: Bivariate scattergraph of plasticity index versus % clay/silt.

The second complication associated with isolating the unique effects of each predictor variable on compressive strength or density concerns the stabilisers. As the levels of stabiliser treatment have been applied across a range of values of soil properties, it is not possible to directly compare the treatments. Figure 4.20 illustrates an example of this. The graph displays the mean values of linear shrinkage for the different quantities of cement, with 95 % confidence limits marked about the means. It is clear that the mean values of linear shrinkage differ significantly between different quantities of cement. For example, samples with 2 % cement have higher values of linear shrinkage on average than samples treated with 6 % cement. It is therefore not possible to directly compare the effect of these 2 cement treatments on compressive strength, because strength is also influenced by linear shrinkage (Figure 4.9). Figure 4.9 shows that higher values of linear shrinkage are associated with lower values of compressive strength; therefore, if 2 % cement samples had lower strength on average than 6 % cement samples, this would partly be due to the difference in linear shrinkage between the two groups. Similar situations are noted with other stabilisers and soil variables; a second example is shown in Figure 4.21, which shows that liquid limit differs between lime treatment levels. Without allowing for the effects such as shown in Figures 4.20 and 4.21, it is not possible to identify the unique influence of each stabiliser and soil property variable on compressive strength and density.

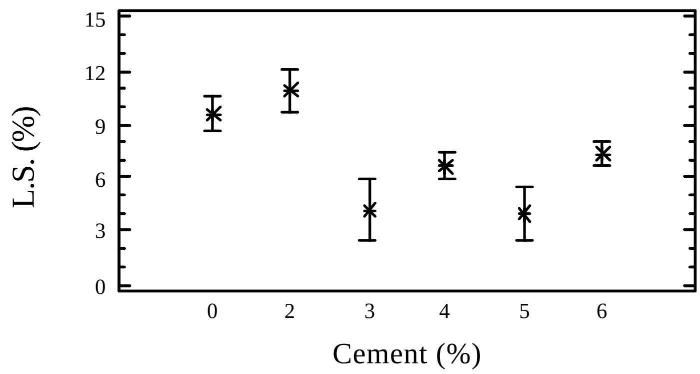


Figure 4.20: Mean values and 95 % confidence limits for linear shrinkage by cement percentage.

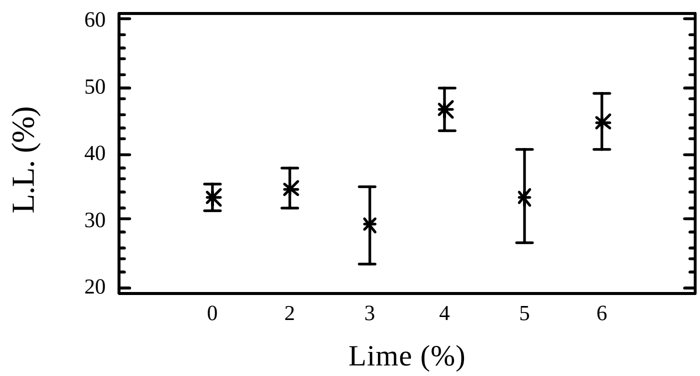


Figure 4.21: Mean values and 95 % confidence limits for liquid limit by lime percentage.

These two complications can be addressed by using ANCOVA to isolate the unique contribution of each of 9 predictor variables (3 stabilisers and 6 soil properties) in explaining variation in compressive strength and density, by examining the simultaneous effect of all predictor variables on the dependent variable. ANCOVA (analysis of covariance) is a linear model design that contains both categorical and continuous predictor variables. The soil property variables (the covariates) have been measured on continuous scales, and together with the categorical (fixed) levels of stabiliser treatments, can be analysed using ANCOVA.

Because through direct calculation there are some very strong correlations between some of the soil property variables, two variables have been omitted from the analysis. Gravel is calculated by subtracting the sum of sand and clay/silt from 100, and plastic limit is the difference between plasticity index and liquid limit. Both gravel and plastic limit have therefore been excluded from the model. Any one of the 3 particulate properties and 3 atterberg limits could potentially have been excluded; however, sand and clay/silt, and liquid limit and plasticity index, were retained as traditionally they have been more frequently used indicators of soil condition, and also have more favourable distributions of values for the purposes of linear analysis.

The statistical advantage of ANCOVA over that of examining the influence of each predictor variable at a time is that unique influence of each predictor variable in explaining variation in compressive strength (or density) can be identified while controlling for all the others. This includes separating the influences of the stabiliser variables from the soil property variables and disentangling the effects of asphalt, cement, and lime where combinations of stabilisers are used. ANCOVA produces information both about the means of compressive strength for the different quantities of stabiliser and about the continuous relationships between each of the soil property variables and compressive strength. For the stabilisers, the analysis gives information about the "main effect" of each stabiliser independent of the others, but no attempt has been made to ascertain interactions between different stabilisers, i.e. to ascertain whether the effect of one level of stabiliser on compressive strength is dependent on the level of another stabiliser used in combination with it.

An alternative to ANCOVA in this situation could be multifactor ANOVA, using categorical variables, which would involve binning the soil property variables into classes and treating them as categorical variables together with the stabiliser treatments. However, this alternative is not preferred here, as information is inevitably lost in the binning process used to convert continuous data into categories. In addition, the class intervals would be subjectively chosen and different results could arise depending on the number and size of classes for each variable. A second alternative, again using multifactor ANOVA, would be to use a classification system to classify the soils (e.g. using the Unified Soil Classification System). The soil type would then become a categorical variable. However, it is not clear that any soil classification system classifies soils according to the particular characteristics required for rammed earth walls. Subdivisions based on particular values of texture, plasticity, or other indicators in such classification systems may not necessarily be the most appropriate ones. Therefore, the soil properties have been retained as continuous data.

4.7.2 Results of ANCOVA

Table 4.5 (contained in Appendix 2) shows the results of fitting a linear ANCOVA model that relates compressive strength to 9 predictor variables (3 stabilisers, 6 soil properties). Only the samples that have values for all variables are used, totalling 193 out of the 230 samples. The relationship is significant at better than the 99.9 % level of significance. Table 4.6 decomposes the variability of compressive strength into contributions due to the 3 stabilisers and 6 soil property variables. The contribution of each variable is measured having removed the effects of all other variables, and the p-values reported test the statistical significance of each of the variables. Table 4.6 shows that, overall, there are no statistically significant differences in compressive strength between different percentages of stabiliser treatment for any of the 3 stabilisers, as the relevant p-values all exceed 0.05. Both clay/silt and sand content have a significant effect on variation in compressive strength, as shown by their significant p-values in Table 4.6. Linear shrinkage is the most significant variable by a reasonable margin. Moisture content, however, does not have a statistically significant influence (p-value = 0.20).

Source	**Sum of squares**	**Degrees of freedom**	**Mean square**	**F-ratio**	**P-value**
Asphalt	0.000318	1	0.000318	0.00	0.982
Cement	2.49248	5	2.49248	0.79	0.556
Lime	1.78084	5	0.356169	0.57	0.726
Clay/silt	4.03456	1	4.03456	6.42	0.012*
Sand	7.23152	1	7.23152	11.50	0.001*
L.L.	0.24227	1	0.24227	0.39	0.536
P.I.	0.325118	1	0.325118	0.52	0.473
L.S.	10.8401	1	10.8401	17.24	0.000*
Moisture	1.02133	1	1.02133	1.62	0.204
Residual	110.025	175	0.62871		
Total	165.825	192			

Table 4.6: ANCOVA for compressive strength using 3 stabilisers and 6 soil property variables. The statistical significance of each of the variables is assessed assuming it was entered into the model last (Type III sums of squares). Significant cases are starred. R-squared adjusted for degrees of freedom = 27.2 %.

Figure 4.22 shows the observed values of compressive strength plotted against the values predicted by the ANCOVA. The vertical deviation of each point from the line y=x is a measure of the residual of each point. Although there are more positive residuals than negative ones in the region compressive strength > 3.6 MPa, the model appears to be reasonably well-fitting overall. However, although it is significant overall, the model explains only 27.2 % (r^2 value adjusted for degrees of freedom) of the variation in compressive strength, a rather low figure.

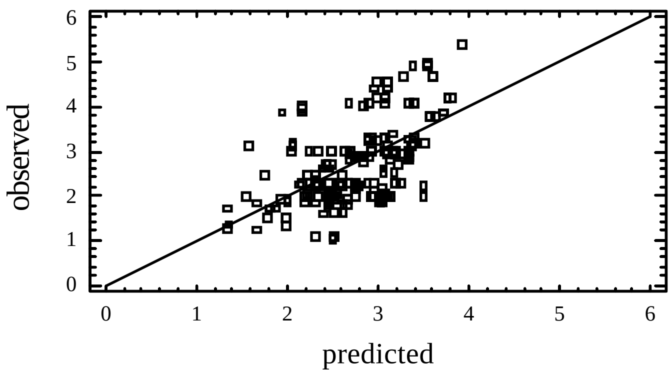


Figure 4.22: Observed values of compressive strength versus those predicted by the ANCOVA model.

Table 4.7 shows the mean compressive strength for each level of the stabilisers, and the upper and lower 95 % confidence limits about each of the means. The mean for each stabiliser level is the effect of that particular stabiliser level having accounted for the variation in the other stabilisers and soil property variables. Lime or cement quantities of 5-6 % appear to produce the strongest samples (overall weighted mean 3.00 MPa). Using no cement (i.e., using just lime or lime-asphalt) tends to yield samples of lower strength (mean 2.41 MPa), as does using no lime (i.e., using just cement or cement-asphalt) which produces samples of mean 2.61 MPa. The fact that these two mean figures are the lowest in Table 4.7 implies that a combination of lime and cement may be more effective than just one or the other used on its own.

The values of compressive strength for cement and lime for the same percentage of stabiliser from 2-6 % are quite similar, particularly for stabiliser quantities of 4, 5, and 6 %. This suggests that lime and cement have similar capability in generating stabilised strength at these levels. Indeed, the overall difference in compressive strength between lime stabilisation and cement stabilisation can be calculated as the difference between the weighted means of compressive strength at quantities of 2, 3, 4, 5, and 6 % lime/cement. The weighted mean for lime is 2.82 Mpa, and for cement is 2.87 Mpa; this difference of 0.05 MPa is not statistically significant given the wider confidence limits reported in Table 4.7. Table 4.7 confirms that the use of asphalt makes no difference whatsoever to the sample strength as the mean compressive strength figures for 0 and 3 % asphalt are identical at 2.78 MPa.

Level	**N**	**Mean**	**Lower limit**	**Upper limit**
Asphalt				
0 %	140	2.78	2.58	2.99
3 %	53	2.78	2.50	3.06
Cement				
0 %	25	2.41	2.00	2.83
2 %	15	2.69	2.19	3.20
3 %	11	2.85	1.99	3.71
4 %	50	2.68	2.26	3.09
5 %	15	3.06	2.45	3.66
6 %	77	3.00	2.51	3.48
Lime				
0 %	91	2.61	2.20	3.01
2 %	44	2.87	2.47	3.26
3 %	12	2.52	1.69	3.34
4 %	22	2.72	2.31	3.13
5 %	8	2.98	2.24	3.72
6 %	16	3.00	2.45	3.56

Table 4.7: Means for compressive strength with 95 % upper and lower confidence intervals about the mean, from ANCOVA.

Figures 4.23, 4.24, and 4.25 show means and 95 % confidence limits for different percentages of the three stabilisers asphalt, cement, and lime, using the values reported in Table 4.7. Figure 4.23 confirms that adding asphalt to samples stabilised with cement and/or lime does not result in any strength increase or decrease. Figure 4.24 shows the means for quantities of cement. There is an apparent increase in compressive strength with higher quantities of cement. However, the apparent trend is not statistically significant (Table 4.6). Lime (Figure 4.25, Table 4.7) shows an apparent trend of increasing compressive strength with higher quantities of lime, but again, any differences in the mean compressive strength values between different stabiliser amounts are not statistically significant. Residual plots (not displayed here) show that residuals from the model appear to be fairly evenly scattered around a residual of zero, and there are no untoward excess values. The patterns of residuals suggest that the ANCOVA model is appropriate for the stabiliser variables.

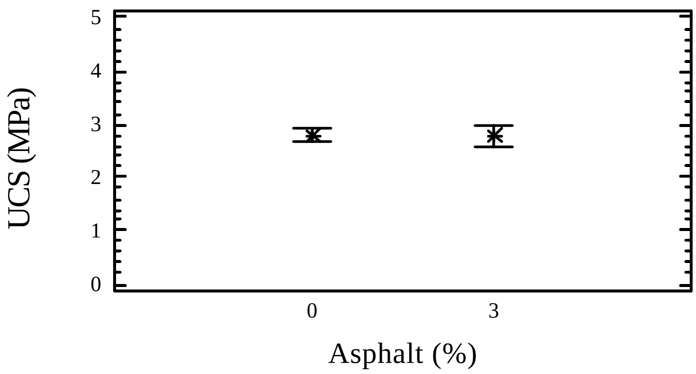


Figure 4.23: Mean, and upper and lower 95 % confidence limits about the mean, for compressive strength for different percentages of asphalt, using ANCOVA.

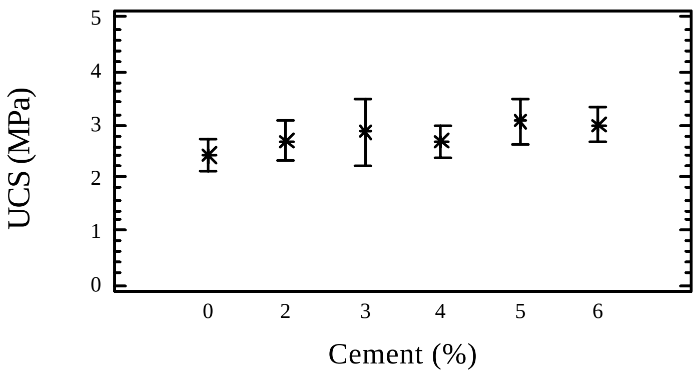


Figure 4.24: Mean, and upper and lower 95 % confidence limits about the mean, for compressive strength for different percentages of cement, using ANCOVA.

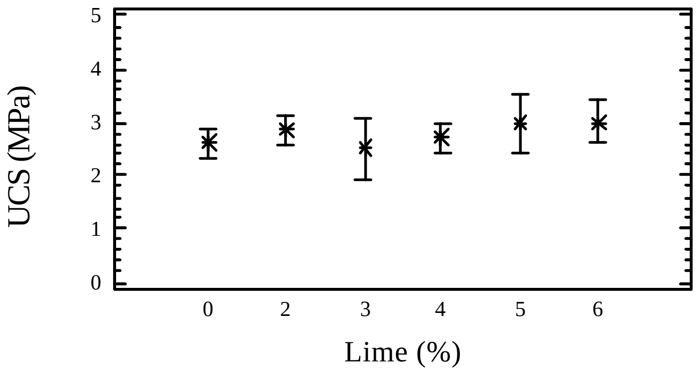


Figure 4.25: Mean, and upper and lower 95 % confidence limits about the mean, for compressive strength for different percentages of lime, using ANCOVA.

Figure 4.26a shows the estimated linear relationship between compressive strength and clay/silt content as established using ANCOVA. This relationship represents the unique influence of clay/silt on compressive strength, as the effect of all other variables in the model have been controlled by setting the stabiliser values to zero, and the other soil property variables to their mean values (Table 4.8). The line trends with a negative slope, which is significant (p=0.012, Table 6). It suggests that the relationship between compressive strength and clay/silt is best modelled with, on average, lower clay/silt contents producing stronger samples. The appropriateness of a linear model to the compressive strength-clay/silt relationship is displayed in Figure 4.26b, which shows the standardised residuals plotted against clay/silt content. The residual pattern is not too problematic, although the positive residuals generally have higher maximum absolute values than the negative ones. Figure 4.27a shows that compressive strength decreases with increasing sand content, and the regression slope is significant with a p-value of 0.001 (Table 4.6). The residual pattern plotted against sand is reasonable (Figure 4.27b), although the positive and negative residuals are rather unbalanced between sand contents of around 75-90 %. As both decreased clay/silt content and sand content are modelled to produce higher values of compressive strength, it is inferred that higher proportions of gravel should lead to an increase in compressive strength, at least up to a point.

Variable	Mean (%)
Clay/silt	25.8
Sand	63.1
L.L.	33.0
P.I.	13.9
L.S.	6.5
Moisture	10.5

Table 4.8: Mean values of continuous variables used in ANCOVA.

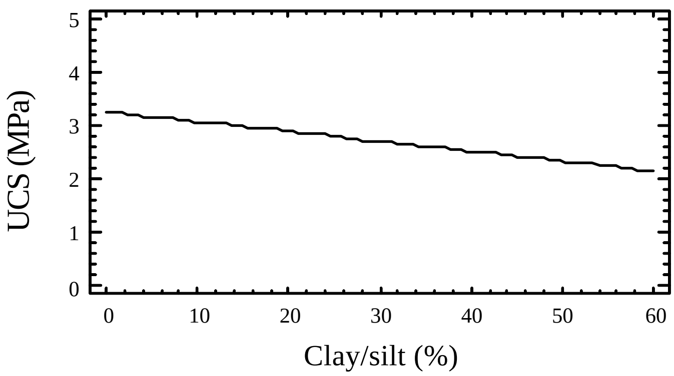


Figure 4.26a: Estimated compressive strength as a function of clay/silt content from ANCOVA. The values of the categorical stabiliser values have been set at 0. The values of the other continuous (soil property) variables are set at their mean values as reported in Table 4.8.

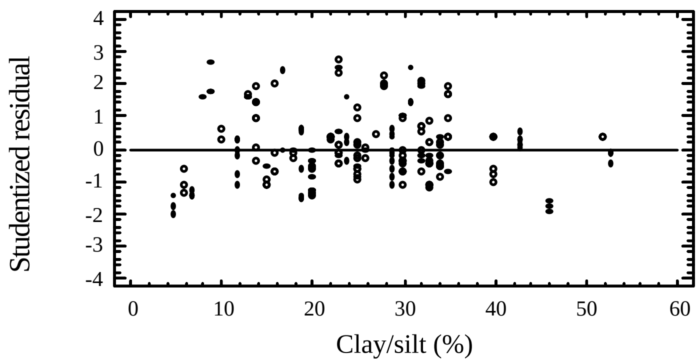


Figure 4.26b: Standardised residuals from ANCOVA for compressive strength, plotted against % clay/silt.

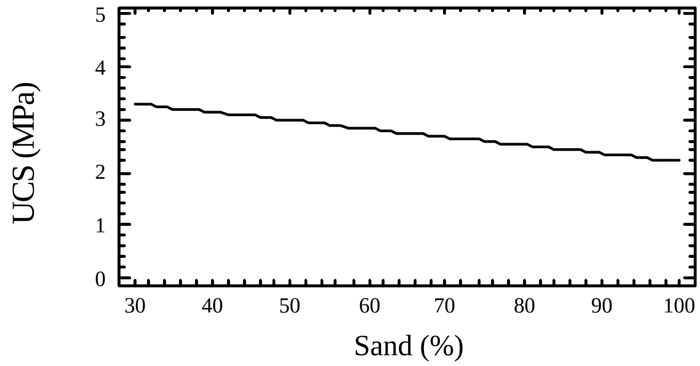


Figure 4.27a: Estimated compressive strength as a function of sand content from ANCOVA. The values of the categorical stabiliser values have been set at 0. The values of the other continuous (soil property) variables are set at their mean values as reported in Table 4.8.

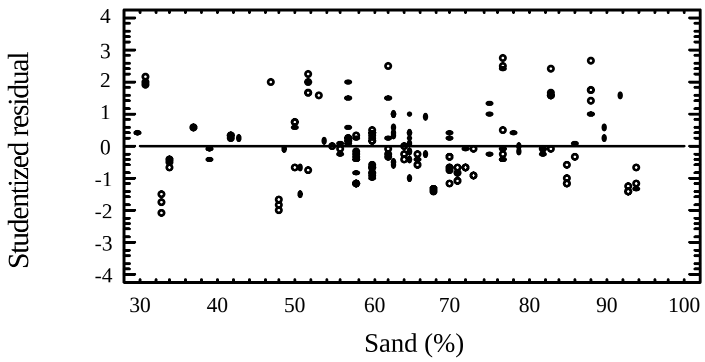


Figure 4.27b: Standardised residuals from ANCOVA for compressive strength, plotted against % sand.

The estimated compressive strength as a function of linear shrinkage (Figure 4.28a) shows a significant negative trend (p-value = 0.000), with higher linear shrinkage values modelled as producing weaker samples. There is some pattern to the residuals plotted against linear shrinkage (Figure 4.28b), which indicates some degree of misfit by the linear model.

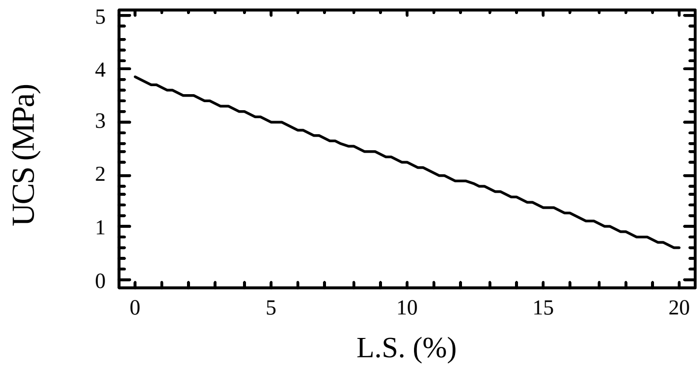


Figure 4.28a: Estimated compressive strength as a function of linear shrinkage from ANCOVA. The values of the categorical stabiliser values have been set at 0. The values of the other continuous (soil property) variables are set at their mean values as reported in Table 4.8.

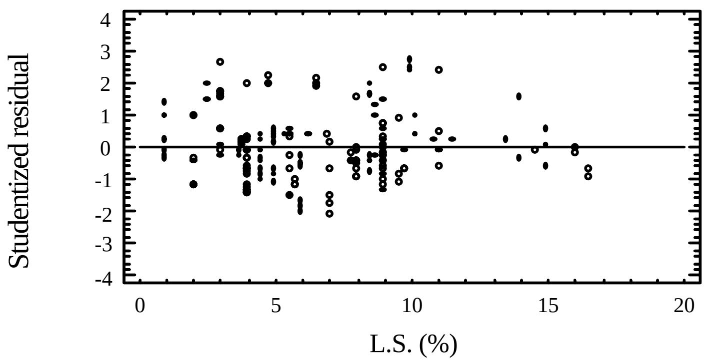


Figure 4.28b: Standardised residuals from ANCOVA for compressive strength, plotted against linear shrinkage.

The estimated compressive strength as a function of liquid limit, plasticity index, and moisture content respectively, is not shown as it is not significant for any of these three variables (Table 4.6). However, the residual patterns are still interesting and relevant as they constitute part of the statistical model. The residual pattern for liquid limit (Figure 4.29a) is interesting. The residuals apparently show decreasing variance about the zero mean with values of liquid limit over 35 %, technically referred to as "heteroscedasticity". Although this pattern would not be desirable, it is clear that it is actually due to a sampling phenomenon: fewer samples have been that have higher liquid limits, and there is therefore less chance to obtain a full range of residual values seen for lower values of liquid limit. The residual pattern for plasticity index (Figure 4.29b) is quite similar to that for liquid limit, with some higher positive residuals in the area around 20 %. Figure 4.29c shows that the residual pattern for moisture content is a reasonable one, although the residuals are unbalanced around moisture contents of around 10 %. The apparent reduction in residual variance with increasing moisture is due to the increasing sparseness of data with increasing moisture contents.

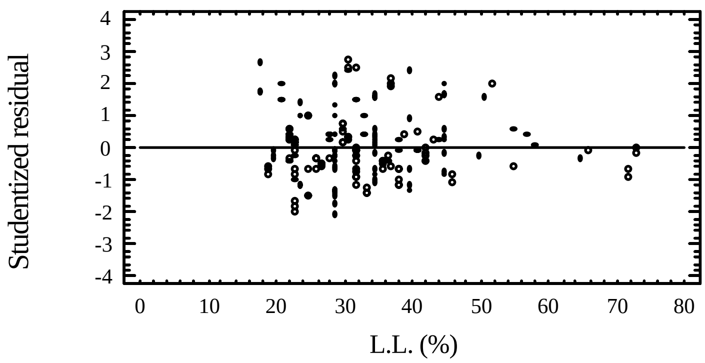


Figure 4.29a: Standardised residuals from ANCOVA for compressive strength, plotted against liquid limit

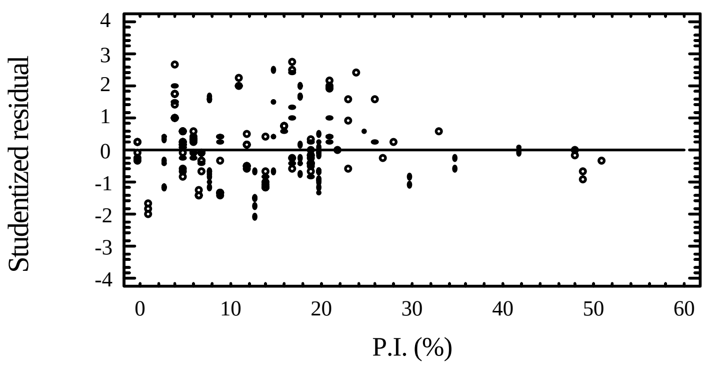


Figure 4.29b: Standardised residuals from ANCOVA for compressive strength, plotted against plasticity index.

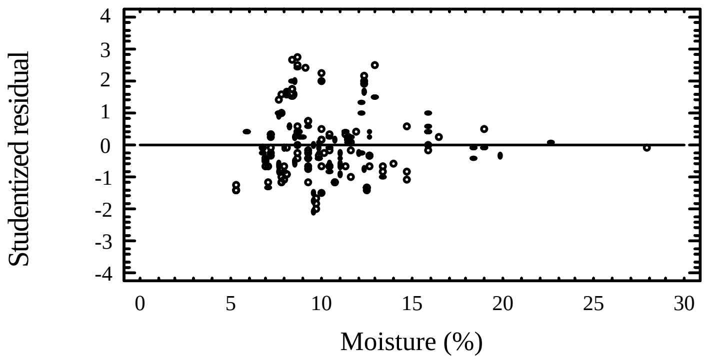


Figure 4.29c: Standardised residuals from ANCOVA for compressive strength, plotted against % moisture.

4.7.3 Relative importance of the predictor variables to compressive strength

One approach to rating the relative importance of the predictor variables to explaining variation in compressive strength is to use the p-values that measure the degree of significance of each variable using ANCOVA (Table 4.6). Table 4.9 shows the variables rated on that basis. One fundamental observation is that the different stabiliser quantities are much less important to explaining variation in compressive strength in stabilised samples than are the soil properties. The three most important properties identified using this method are linear shrinkage, sand content, and clay/silt content. The textural properties of the soil appear to be more important than the Atterberg limits.

Variable	Rank	P-value
L.S.	1	0.000*
Sand	2	0.001*
Clay/silt	3	0.012*
Moisture	4	0.204
P.I.	5	0.473
L.L.	6	0.536
Cement	7	0.556
Lime	8	0.726
Asphalt	9	0.982

Table 4.9: Ranking of relative importance of predictor variables to compressive strength, based on p-value of statistical significance from ANCOVA (Table 4.6).

ANCOVA examines the effect of the soil property variables on compressive strength using a linear model. As some of the residual plots show, some soil properties are not modelled perfectly using the linear model, and the r-squared value for the model as a whole is rather low given the number of predictor variables. However, given the nature of the data, it is the only viable way of isolating the influence of each variable separately.

4.8 INFLUENCE OF STABILISERS AND SOIL PROPERTIES ON UCS: ANOVA AND MULTIPLE REGRESSION

Given the somewhat surprising result of the ANCOVA model, which indicates that the level of stabiliser has a rather subordinate influence on compressive strength, two further analyses are now made on the data. The first is an ANOVA for the same-sample determinations identified in Table 4.1, which total 58 unique samples with an overall total of 177 observations. The second involves the regression of compressive strength on the 6 soil properties only (i.e., without stabilisers as variables), using the same dataset (193 observations) as the ANCOVA above.

4.8.1 Analysis of variance using same-sample determinations of compressive strength

The analysis of variance uses the "same-sample determination" column of Table 4.1 as a categorical variable to explain variation in compressive strength. The same-sample determination notation corresponds to those soil samples for which two or more determinations of compressive strength were made using different stabiliser treatments: the soil was the same but stabiliser treatments varied. Therefore, the between-group variation in this ANOVA describes the influence of soil characteristics on compressive strength, and the within-group variation describes the influence of the stabiliser treatments (the type and level of stabiliser). Therefore the variation will be split between these two influences - soil and stabiliser - and this effectively identifies the influence of the soils relative to the influence of the stabilisers. There are 58 unique samples that have different stabiliser treatments, with 177 observations in total in the design.

Table 4.10 (contained in Appendix 2) reports the results of the analysis of variance of compressive strength by same-sample determinations. There are highly significant differences between the mean values of compressive strength for same-sample determinations, and the r-squared value (adjusted for degrees of freedom) for the ANOVA model is 94.0 %. This figure is high because the variation in individual compressive strength values is due primarily to differences between unique samples (between-group variation), rather than between same-sample determinations for each sample (within-group variation). Figure 4.30 shows the mean compressive strength values for samples and their 95 % confidence limits. The main feature of this diagram is the wide variation in mean compressive strength between samples compared with the confidence limits that represent variation in compressive strength between the same-sample determinations for each sample. As the variations in compressive strength between samples are due to soil characteristics, and the variations in compressive strength between same-sample determinations are a result of different stabiliser treatments, 94 % of the total compressive strength variation can be ascribed to the influence of soil properties. Only 6 % of compressive strength variation is due to stabiliser treatment variations. The analysis confirms the intuitive feeling generated when perusing the same-sample determination compressive strength values in Table 4.1, which appear to vary far less substantially with stabiliser treatment than with soil type. This identified predominance of the influence of soil characteristics supports the ANCOVA above which showed that different quantities of asphalt, cement, and lime, made little difference overall to sample strength.

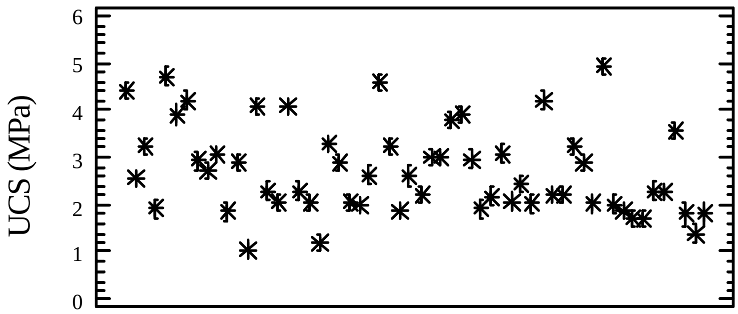


Figure 4.30: Graph showing the mean compressive strength and 95 % confidence interval for each of the 58 samples that had multiple determinations of compressive strength. Sample 1 lies at the far left and subsequent samples (Table 4.1) lie progressively to the right. Note the wide variation in mean compressive strength between samples (representing the large variation in compressive strength attributable to different soil characteristics), and the very small confidence intervals for the means (representing the small variation in compressive strength attributable to different stabiliser treatments).

4.8.2 Regression of compressive strength on soil properties

Table 4.11 (contained in Appendix 2) shows the results of fitting a multiple regression model to predict compressive strength using the 6 soil property variables as used in ANCOVA in Section 4.7. The model is significant at better than the 99.9 % level of confidence, showing that the it explains a significant amount of variation in compressive strength (r-squared adjusted = 28.7 %). This degree of explanation is actually slightly better than the ANCOVA (r-squared adjusted = 27.2 %) which additionally uses the 3 stabilisers. This supports the conclusion that the different stabilisers and treatment stabiliser amounts are subordinate to the soil properties in explaining variation in compressive strength.

The equation of the fitted model is:

Compressive strength = 5.05 - 0.017*Clay/silt - 0.016*Sand + 0.011*L.L. + 0.012*P.I. - 0.161*L.S. - 0.036*Moisture

Table 4.12 shows which variables in the model are of significance as judged by the p-value. The significant variables are clay/silt, sand, and linear shrinkage. These variables are precisely the same ones as are significant for ANCOVA. Again, this suggests that the different amounts of stabiliser make little difference to compressive strength.

Variable	P-value
L.S.	0.000*
Sand	0.000*
Clay/silt	0.013*
Moisture	0.098
L.L.	0.359
P.I.	0.525

Table 4.12: Significance of predictor variables
The statistical significance of each of the variables is assessed assuming it was entered into the model last (Type III sums of squares). Significant cases are starred. As with ANCOVA, both gravel and plastic limit have been omitted and for similar reasons.

Figure 4.31 shows the observed versus predicted values of compressive strength for the regression model. Overall it is quite a good fit, and can be compared with the corresponding diagram for ANCOVA (Figure 4.22). There is very little difference between the two diagrams, suggesting little difference in explanation of compressive strength between the ANCOVA and multiple regression models. Residual plots suggest the multiple regression model is no worse a fit than the ANCOVA model. One representative residual plot is shown in Figure 4.32 (residuals against liquid limit). Figure 4.32 should be compared with Figure 4.29a, which shows the ANCOVA residuals for liquid limit.

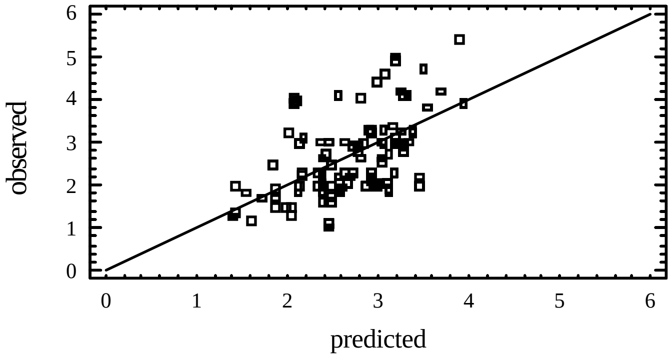


Figure 4.31: Observed verus predicted values for the multiple regression model relating compressive strength to the 6 predictor soil property variables.

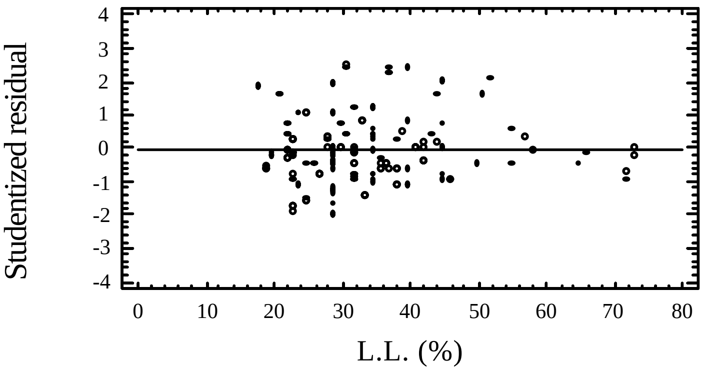


Figure 4.32: Standardised residuals from the multiple regression on compressive strength, plotted against liquid limit

4.9 INFLUENCE OF STABILISERS AND SOIL PROPERTIES ON DENSITY: ANCOVA

This ANCOVA follows exactly the same procedure as for compressive strength in Section 4.7 above, except that the dependent variable is now density. The comments from the introduction of that previous section apply also here. The main objective of the analysis is to isolate the unique effect of each predictor variable on density.

4.9.1 Results of ANCOVA

Table 4.13 (contained in Appendix 2) shows the results of fitting a linear ANCOVA model that relates density to 9 predictor variables. The relationship is significant at better than the 99.9 % level of significance. Table 4.14 decomposes the variability of density into contributions due to the 3 stabilisers and 6 soil property variables. The contribution of each variable is measured having removed the effects of all other variables, and the p-values reported test the statistical significance of each of the variables. Table 4.14 shows that overall, there are no statistically significant differences in density between different stabiliser quantities for any of the 3 stabilisers, as the relevant p-values all exceed 0.05. Neither are there any significant differences between particular pairs of means within cement, which contrasts with the results for compressive strength above.

Of the particulate soil properties, clay/silt is significantly related to density, but sand is not. Moisture is the only other variable to have a statistically significant effect on density, at better than the 99.9 % level of confidence (Table 4.14). These results contrast with those for compressive strength, for which the significant variables were modelled to be clay/silt, sand, and linear shrinkage (Table 4.6).

Source	Sum of squares	Degrees of freedom	Mean square	F-ratio	P-value
Asphalt	0.002132	1	0.002132	0.10	0.755
Cement	0.063883	5	0.063883	0.59	0.710
Lime	0.040773	5	0. 040773	0.37	0.866
Clay/silt	0.087878	1	0.087878	4.04	0.046*
Sand	0.014099	1	0.014099	0.65	0.422
L.L.	0.021416	1	0. 021416	0.98	0.323
P.I.	0.003715	1	0.003715	0.17	0.680
L.S.	0.003494	1	0. 003494	0.16	0.689
Moisture	0.271838	1	0. 271838	12.49	0.001*
Residual	3.80977	175	0.021770		
Total	5.54623	192			

Table 4.14: ANCOVA for density using 3 stabilisers and 6 soil property variables. The statistical significance of each of the variables is assessed assuming it was entered into the model last (Type III sums of squares). Significant cases are starred. R-squared adjusted for degrees of freedom = 24.6 %.

Figure 4.33 shows the observed values of density plotted against the values predicted by the ANCOVA. The vertical deviation of each point from the line y=x is a measure of the residual of each point. There are more positive residuals than negative ones in the region density > 2.0 t/m^3, and a few negative outliers around densities of 1.8 t/m^3, but overall the model appears to be a not unreasonable fit to the data. However, although it is significant overall, the model explains only 26.4 % (r^2 value adjusted for degrees of freedom) of the variation in density, a moderately low figure.

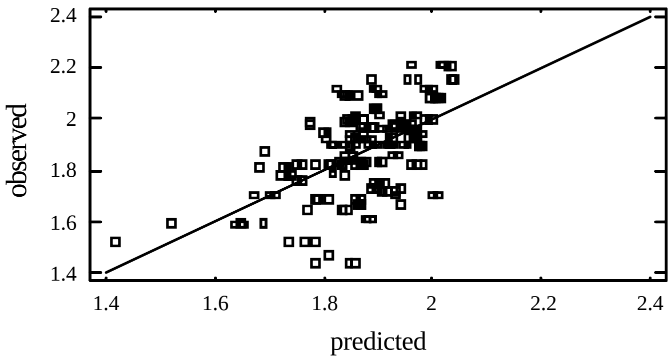


Figure 4.33: Observed versus predicted values of density using ANCOVA.

Table 4.15 shows the mean density for each level of the stabilisers. It also shows the upper and lower 95.0% confidence limits about each of the means. The major conclusion to be drawn from Table 4.15 (and Table 4.14) is that different percentages of any one stabiliser make no difference to sample density. The similarity of the mean densities for the different treatment levels (all are between 1.83 and 1.95 t/m^3) indicates that the different stabiliser types and amounts have an even more minor effect on density variation than they do on compressive strength variation.

Level	**N**	**Mean**	**Lower limit**	**Upper limit**
Asphalt				
0 %	140	1.88	1.84	1.91
3 %	53	1.87	1.82	1.92
Cement				
0 %	25	1.85	1.77	1.93
2 %	15	1.94	1.84	2.03
3 %	11	1.87	1.71	2.03
4 %	50	1.87	1.79	1.96
5 %	15	1.85	1.73	1.96
6 %	77	1.86	1.77	1.95
Lime				
0 %	91	1.89	1.81	1.97
2 %	44	1.87	1.80	1.94
3 %	12	1.95	1.80	2.10
4 %	22	1.86	1.79	1.94
5 %	8	1.83	1.69	1.97
6 %	16	1.83	1.73	1.94

Table 4.15: Means for density with 95 % upper and lower confidence intervals about the mean, from ANCOVA.

Figures 4.34, 4.35, and 4.36 show means and 95 % confidence limits for different percentages of the three stabilisers asphalt, cement, and lime, using the values reported in Table 4.15. The graphs confirm that there is no significant effect of different stabiliser percentage on density variation. Any apparent pattern in the mean values is insignificant and swamped by the magnitude of the confidence limits about the means. In other words, the variation within levels of stabiliser is much greater than the variation between levels of stabiliser.

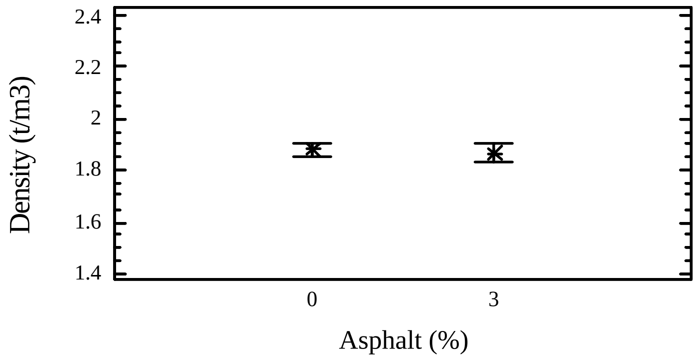


Figure 4.34: Mean, and upper and lower 95 % confidence limits about the mean, for density for different percentages of asphalt.

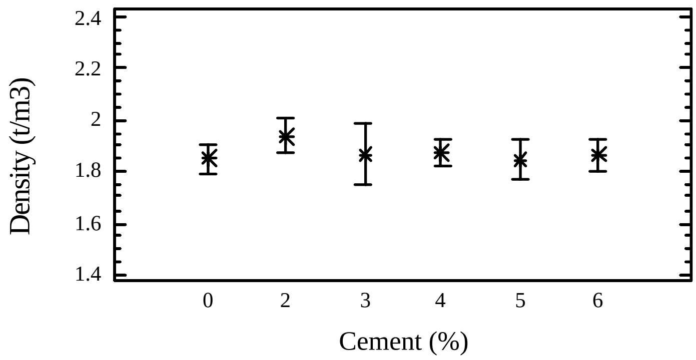


Figure 4.35: Mean, and upper and lower 95 % confidence limits about the mean, for density for different percentages of cement.

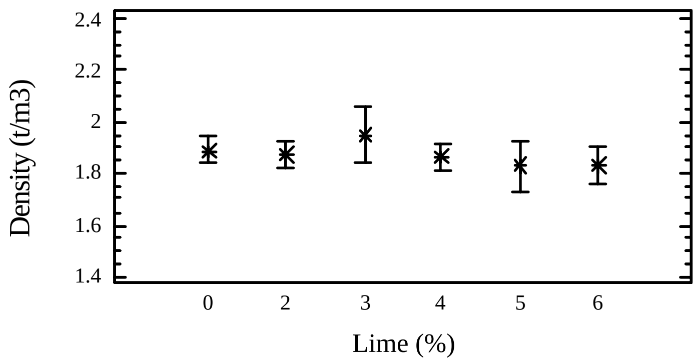


Figure 4.36: Mean, and upper and lower 95 % confidence limits about the mean, for density for different percentages of lime.

Figure 4.37a shows the estimated linear relationship between density and clay/silt content as established using ANCOVA. The line trends with a negative slope, and the linear relationship is significant with p = 0.046 (Table 4.14). The appropriateness of a linear model to the density-clay/silt relationship is displayed in Figure 4.37b which shows the standardised residuals plotted against clay/silt content. The residual pattern is acceptable despite a few negative outliers between clay/silt contents of 15-35 %.

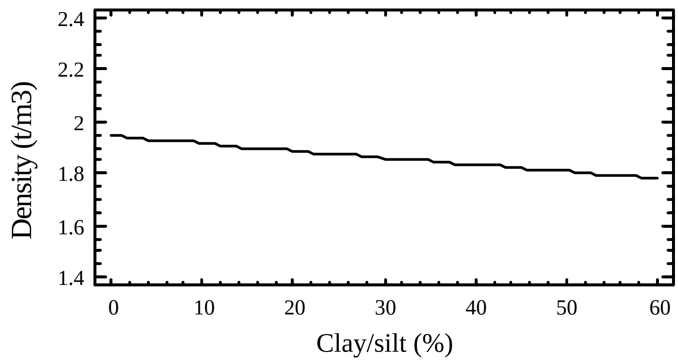


Figure 4.37a: Estimated density as a function of clay/silt content from ANCOVA. The values of the categorical stabiliser values have been set at 0. The values of the other continuous (soil property) variables are set at their mean values as reported in Table 4.8.

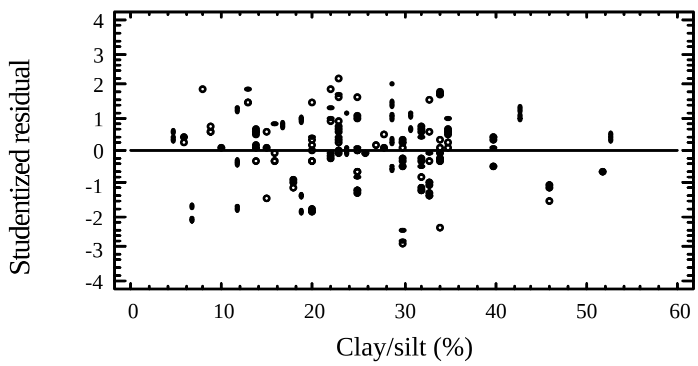


Figure 4.37b: Standardised residuals from ANCOVA for density, plotted against % clay/silt.

The relationship between moisture and density in Figure 4.38 shows a highly significant negative slope, with higher values of moisture being modelled to produce lower values of compressive strength in the range of the data. Figure 4.39 shows that the residual pattern is a reasonable one, with the apparent reduction in residual variance with increasing moisture again being due to fewer samples with higher moisture levels. The expected density as a function of % sand, liquid limit, plasticity index, and linear shrinkage respectively, is not shown as none of these variables is significantly related to density (Table 4.14).

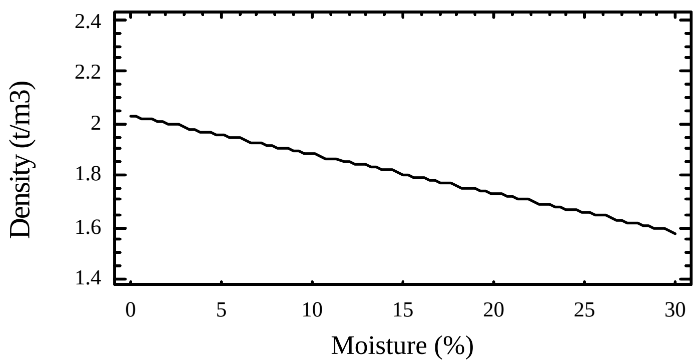


Figure 4.38: Estimated density as a function of moisture from ANCOVA. The values of the categorical stabiliser values have been set at 0. The values of the other continuous (soil property) variables are set at their mean values as reported in Table 4.8.

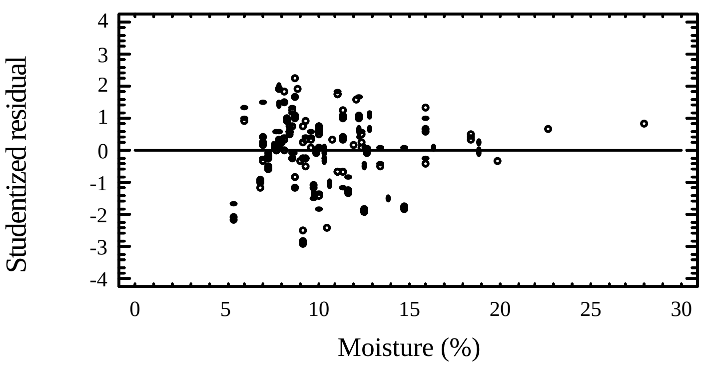


Figure 4.39: Standardised residuals from ANCOVA for density, plotted against % moisture.

4.9.2 Relative importance of the predictor variables to density

As for analyis of compressive strength above, the relative importance of the predictor variables to explaining variation in density is assessed using the p-values that measure the degree of significance of each variable using ANCOVA (Table 4.14). Table 4.16 shows the variables rated on that basis. The two most important variables as assessed using this rating, and the only statistically significant ones, are moisture level and clay/silt content. Liquid limit and sand are the next most important. Again, the least important appear to be the different quantities of stabiliser used. Differences with the rankings for compressive strength are that moisture is not significant for compressive strength, but both sand and linear shrinkage are; clay/silt is modelled as being important for both dependent variables. Linear shrinkage is the most significant variable for compressive strength based on a linear model, but the least significant for density.

Variable	Rank	P-value
Moisture	1	0.001*
Clay/silt	2	0.046*
L.L.	3	0.323
Sand	4	0.422
P.I.	5	0.680
L.S.	6	0.689
Cement	7	0.710
Asphalt	8	0.755
Lime	9	0.866

Table 4.16: Ranking of relative importance of predictor variables to density, based on p-value of statistical significance from ANCOVA in Table 4.14.

4.9.3 Influence of stabilisers and soil properties on density: ANOVA

As for the analysis of the influence of stabilisers and soils on compressive strength, an analysis of variance of density by same-sample determinations was made. The analysis followed the exact same procedure as for compressive strength (Section 4.8.1), except density was used as the dependent variable, and therefore only a summary of the results is reported here. The analysis of variance was highly significant with 98.7 % of the variation in total density variation being due to differences between samples, i.e., due to soil property differences. Just 1.3 % of the variability in density can be ascribed to differences between same-sample determinations, i.e., to differences in stabiliser treatments. This figure is even smaller than the 6 % reported for compressive strength.

4.10 INFLUENCE OF SOIL PROPERTIES AND STABILISERS ON STABILISATION SUCCESS

Another approach toward analysing successful stabilisation is to identify the characteristics of samples that are successfully stabilised and those that are not, and to examine the trends of stabilisation success with respect to the independent variables. Compressive strength code differentiates between two levels of compressive strength: those samples with compressive strength less than 2 MPa (compressive strength code = 0) and those samples with compressive strength more than or equal to 2.0 MPa (compressive strength code = 1). These two levels represent "failed" and "successful" stabilisation respectively as defined in Chapters 1 and 2. Overall, using this criterion, 163 samples were successfully stabilised, and 67 failed stabilisation, from a total of 230 samples. The approach of using a pass-fail analysis differs somewhat from the ANCOVA approach above. ANCOVA was used to model the unique effects of stabiliser treatments and soil properties on compressive strength and density, using actual measured values the criterion variables. In contrast, the binomial code analysis examines the pattern of successful or failed stabilisation with respect to the stabilisers and soil property variables.

An analysis of the properties of samples by compressive strength code is a fundamental requirement in identifying the material properties that are associated with both successfully stabilised samples and failed ones. Essentially, this is a problem of discriminating between two groups of samples on the basis of their characteristics. Two approaches are taken: one is to examine univariate plots and breakdowns by class to identify patterns of stabilisation success. The other is using one-way ANOVA of the soil variables using compressive strength code as the categorical variable.

4.10.1 Univariate analysis of soil property variables for compressive strength code

The aim of the compressive strength code analysis is to discriminate between successful and failed samples using the soil property variables. Univariate analysis is performed only for the soil property variables as they have continuous data. The patterns for density are not shown as they are broadly similar to the patterns displayed for compressive strength, and compressive strength is regarded as the more important criterion variable of the two. Figures 4.40 to 4.47 show univariate plots that show the association between compressive strength code and soil properties. Figure 4.40 shows that samples of moderate clay/silt contents (around 21-35 %) are more likely to be successfully stabilised than for higher or lower clay/silt content samples. Gravel is a useful discriminator (Figure 4.41), as all samples with gravel contents of greater than 35 % were successfully stabilised. In addition, most samples between 15 and 35 % gravel were successfully stabilised. Sand (Figure 4.42) is useful in discriminating between failed and successful samples, as all samples with sand content of < 48 % were successfully stabilised with respect to their strength. Above this value, it seems that the rate of failure increases slightly with higher sand contents, and high values of sand, above around 75 %, are rather mixed in terms of compressive strength stabilisation success or failure.

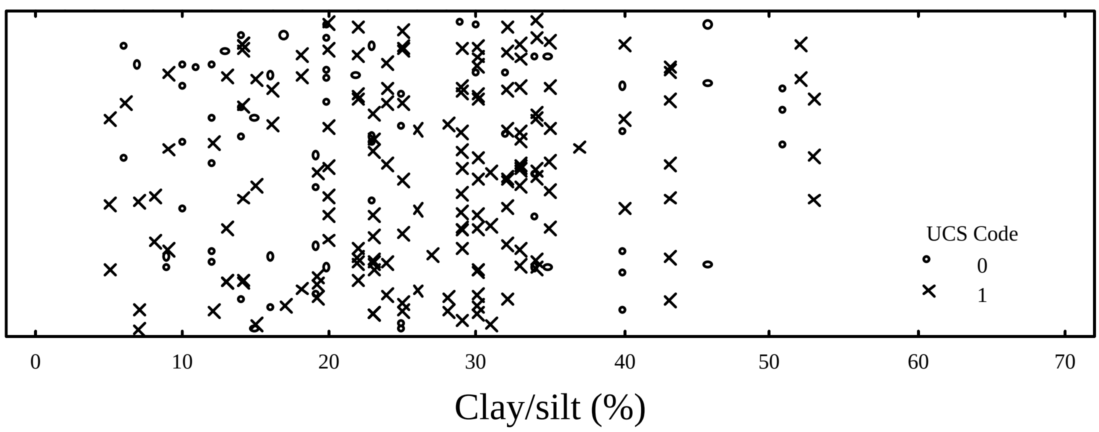


Figure 4.40: Univariate plot of clay/silt, by compressive strength code (230 values).

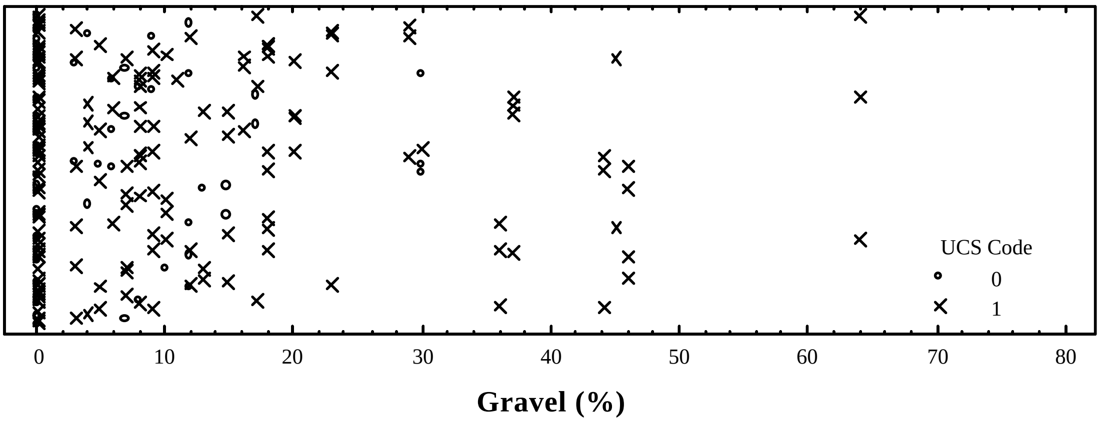


Figure 4.41: Univariate plot of gravel, by compressive strength code (230 values).

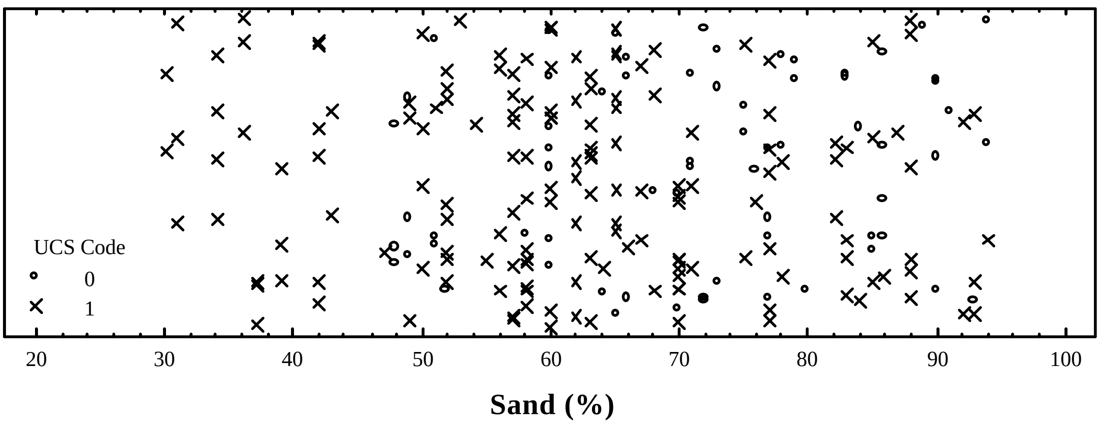


Figure 4.42: Univariate plot of sand, by compressive strength code (230 values).

The univariate plot of liquid limit is illuminating (Figure 4.43). It shows that no sample with a liquid limit greater than 57 % was successfully stabilised. Below this figure, the rate of failure appears to decrease with decreasing values of liquid limit down to the minimum measured of 18 %. Plastic limit (Figure 4.44) is a very good discriminator of compressive strength code in the middle region of around 16-19 % where successful samples dominate; the rate of failed samples appears to be about constant over the rest of the range of values of plastic limit. For the region of plastic limit above 28 %, where successful samples are dominant, the data are probably too sparse to be able to make a reliable conclusion. In contrast, plasticity index (Figure 4.45) is a very good discriminator of compressive strength code. No sample with a plasticity index higher than 36 % was successfully stabilised. Below this figure, the rate of failed samples with respect to plasticity index appears to decrease with decreasing plasticity index. Of the 118 samples that have plasticity indexes of less than or equal to 15 %, 103 were successfully stabilised. Therefore, plasticity index is an excellent discriminator both of unsuccessful, and successful, stabilisation. Linear shrinkage (Figure 4.46) is also an excellent discriminator between compressive strength codes. In addition, like plasticity index, it is able to discriminate at both ends of the range of its values. For linear shrinkage values of less than or equal to 4.0 %, 62 samples from a total of 63 were successfully stabilised. This is a very significant result. Toward the upper end of values of linear shrinkage, the variable is also an efficient discriminator. For values of linear shrinkage above 13.0 %, 26 samples out of 33 failed stabilisation. For intermediate values, the rate of stabilisation failure increases between the 4.0 % and 13.0 % values.

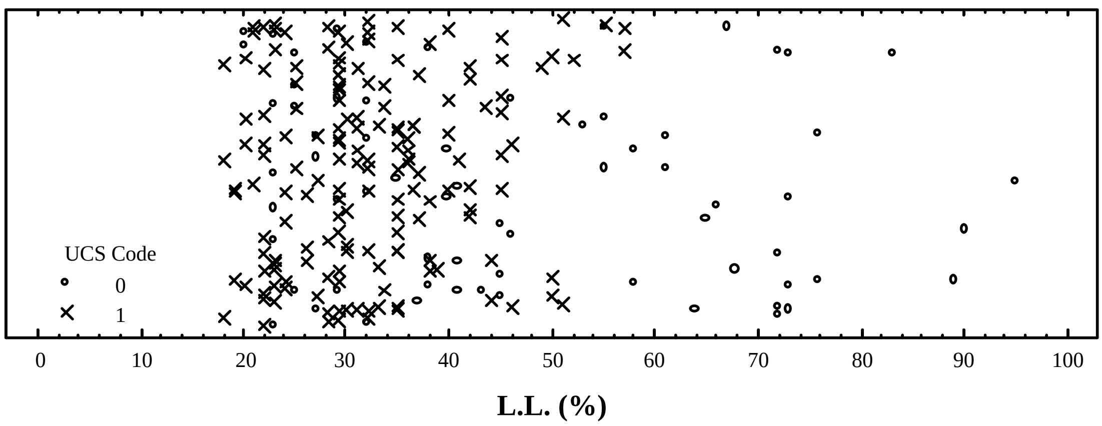


Figure 4.43: Univariate plot of liquid limit, by compressive strength code (230 values).

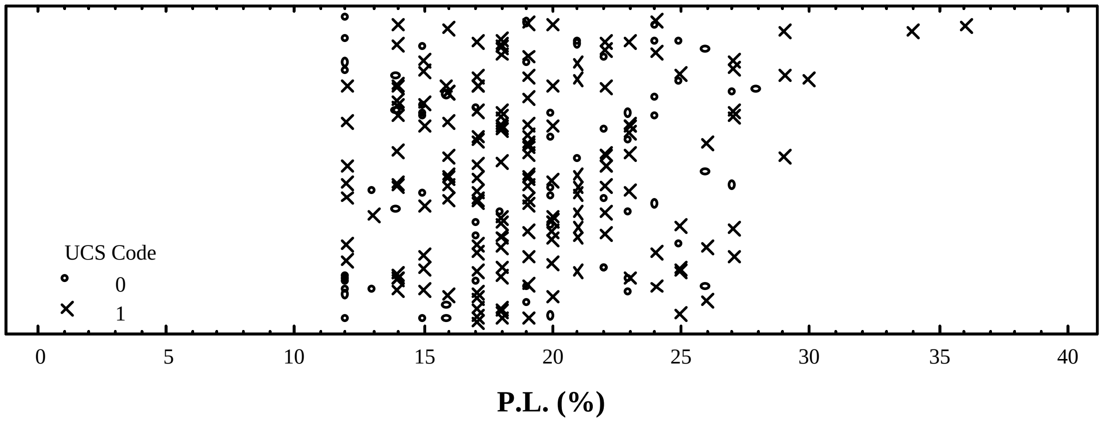


Figure 4.44: Univariate plot of plastic limit, by compressive strength code (229 values).

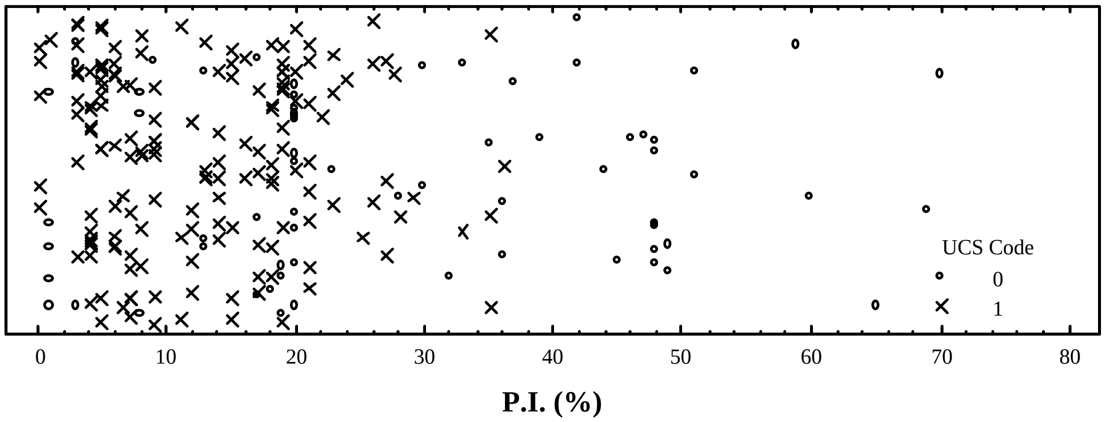


Figure 4.45: Univariate plot of plasticity index, by compressive strength code (229 values).

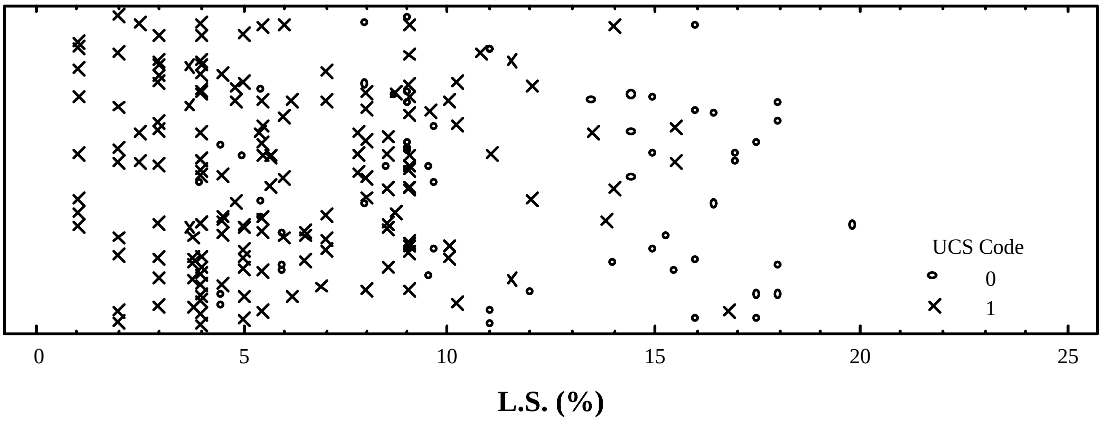


Figure 4.46: Univariate plot of linear shrinkage, by compressive strength code (220 values).

The failure rate of samples generally increases with increasing moisture content (Figure 4.47). Moisture is able to discriminate well between compressive strength codes near both ends of its range of values. For moisture contents of less than or equal to 10.0 %, 93 out of 105 samples were successfully stabilised. At the other end of values, for moisture contents over 13.0 %, 21 out of 28 samples failed stabilisation.

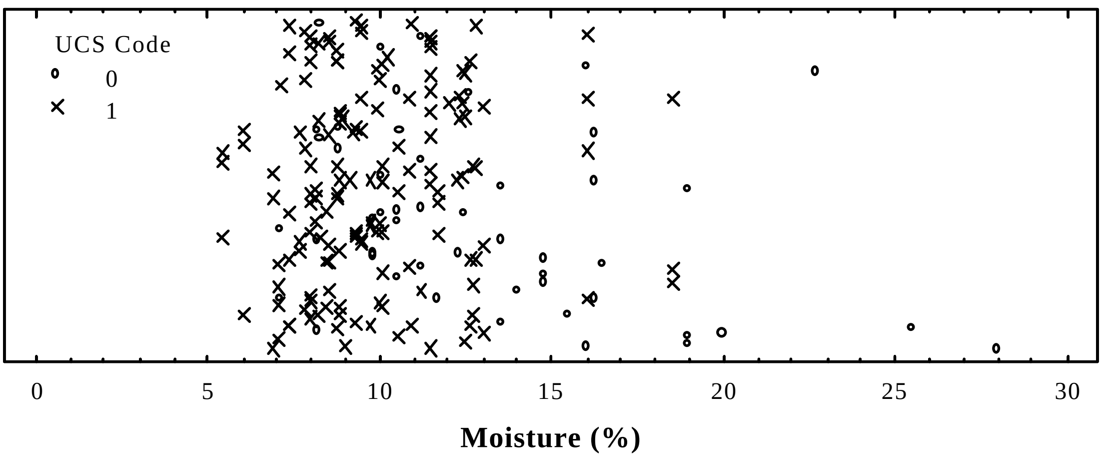


Figure 4.47: Univariate plot of moisture, by compressive strength code (202 values).

4.10.2 Compressive strength and density code analysis by soil property variable classes

Table 4.17 shows the numbers of samples in compressive strength code 0 and compressive strength code 1 for classes of each soil property variable. Clay/silt classes 4-6 are characterised by predominantly successfully stabilised samples, corresponding to clay/silt contents of 21-35 %. Clay/silt contents of less than or equal to 20 % (clay/silt classes 1-3) are unfavourable. Table 4.17 indicates that gravel contents between 3-5 % (gravel class 2) and of above 12 % (gravel classes 5-7) are likely to be more favourable for stabilisation. Sand classes 1, 3, and 4 exhibit a predominance of successful samples, which corresponds to sample sand contents of below 45 % and between 51 and 60 %. Sand classes 8-10 have high proportions of failed samples, and therefore sand contents in excess of 75 % should be regarded as unfavourable for successful stabilisation.

Table 4.17 shows that liquid limit classes 1-4 are characterised by higher proportions of successful samples, which corresponds to sample liquid limits of up to and including 35 %. Liquid limits of over 45 % (class 7) are associated with a high rate of unsuccessful stabilisation. Plastic limits of 17-19 % (plastic limit class 3) have the highest proportion of successfully stabilised samples out of the 6 classes by some margin, making plastic limit a reasonable discriminator in that range of values. Plasticity index classes 1-3 are associated with high rates of successful stabilisation, corresponding to values of less than 16 %. Classes 5 and 6 (plasticity index = 21-30 %) are also quite reasonable. However, samples with plasticity index values over 30 % (class 7) were very unlikely to be successfully stabilised. Values of linear shrinkage less than or equal to 7.0 % are dominated by successful samples (linear shrinkage classes 1-3), whereas values that exceed 11 % (Class 6) are much more likely to be characterised by failed samples. Moisture content classes 1-4 contain predominantly successfully stabilised samples, and therefore moisture contents of up to and including 10 % can be viewed as promoting successful stabilisation. Moisture classes 6 and 7 (11.1-13.0 %) are also associated with successful samples, although the intermediate classes 5 is not quite as favourable. Moisture values above 13 % (Class 8) are unfavourable for stabilisation.

CLASS	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Clay/silt Class										
UCS Code 0	4	6	6	4	1	3	5			
UCS Code 1	5	8	7	13	14	16	7			
Gravel Class										
UCS Code 0	16	2	3	3	2	1	0			
UCS Code 1	25	7	9	8	11	7	5			
Sand Class										
UCS Code 0	0	3	2	3	2	3	5	4	2	6
UCS Code 1	11	3	5	14	12	7	2	4	5	6
L.L. Class										
UCS Code 0	1	4	3	3	3	13				
UCS Code 1	4	14	17	8	7	6				
P.L. Class										
UCS Code 0	7	4	4	6	6	3				
UCS Code 1	10	8	26	13	7	7				
P.I. Class										
UCS Code 0	3	2	1	8	0	1	12			
UCS Code 1	18	16	11	14	6	4	2			
L.S. Class										
UCS Code 0	0	2	3	6	4	12				
UCS Code 1	15	22	13	15	4	5				
Moisture Class										
UCS Code 0	0	1	3	1	4	2	1	10		
UCS Code 1	6	12	16	12	8	8	10	3		

Table 4.17: Percentages (to nearest whole number) of samples in compressive strength code 0 (fail) and compressive strength code 1 (successful) for classes of soil property variable. Note that the classes for each variable are different, and no comparisons should be made between the same class number for different variables down the columns of the table. Number of observations is the same as for Figures 4.40-4.47.

Table 4.18 is equivalent in construction to Table 4.17 and reports the results for density criterion success or failure by soil property classes, as a comparison with the results for compressive strength. Although there are some differences, the trends for density are broadly similar in style to those for compressive strength, but they tend tobe less marked.

Clay/silt content does not influence the passing or failing of the density criterion as much as for compressive strength. The clay/silt classes 1-3 (< 20 %) are not as unfavourable, and the most favourable class is 5 (26-30 %). The pattern of density code over the gravel classes is mixed; however, compared with compressive strength, class 1 (0-2 %) is more favourable. Gravel contents over 13 % are favourable as for compressive strength. The pattern across the sand classes is not as distinct as for compressive strength. Lower values of sand are not as favourable for successful density stabilisation as for compressive strength stabilisation, and neither are higher values so advantageous for unsuccessful stabilisation for density compared with compressive strength.

The pattern for liquid limit for the density criterion is similar to that for compressive strength. The major difference for plastic limit is the greater favourability of plastic limits of > 25 % for successful density stabilisation. Plasticity indexes of 10 % or less (classes 1 and 2) do not promote successful stabilisation for density to the same marked degree as for compressive strength. The pattern across the classes for linear shrinkage is similar in trend as for compressive strength (low values favourable, high values unfavourable), but the trend is not as marked. The trend across the moisture classes is very similar for both density and compressive strength, except that favourability for compressive strength stabilisation extends from low moisture contents to higher (10.0 %) values than for density (to 9.0 %).

CLASS	1	2	3	4	5	6	7	8	9	10
Clay/silt Class										
Density Code 0	2	4	4	5	3	6	6			
Density Code 1	5	8	9	11	14	15	6			
Gravel Class										
Density Code 0	10	3	3	8	3	2	0			
Density Code 1	26	4	10	4	11	7	6			
Sand Class										
Density Code 0	2	3	2	4	5	4	1	3	1	4
Density Code 1	10	3	5	14	9	7	6	3	5	5
L.L. Class										
Density Code 0	1	6	6	5	1	4	6			
Density Code 1	4	14	16	15	11	5	2			
P.L. Class										
Density Code 0	6	4	6	8	6	0				
Density Code 1	12	7	25	11	5	8				
P.I. Class										
Density Code 0	7	5	3	6	1	2	5			
Density Code 1	17	15	11	19	6	1	0			
L.S. Class										
Density Code 0	3	5	5	5	4	6				
Density Code 1	13	22	12	18	6	0				
Moisture Class										
Density Code 0	1	0	3	5	4	4	2	10		
Density Code 1	5	13	16	8	7	6	10	3		

Table 4.18: Percentages (to nearest whole number) of samples in density code 0 (fail) and density code 1 (successful) for classes of soil property variable. Note that the classes for each variable are different, and no comparisons should be made between the same class number for different variables down the columns of the table. Number of observations for each variable is 202, except for linear shrinkage, which has 193 observations.

4.10.3 Analysis of variance of variables by compressive strength code and density code

One-way ANOVA allows the differences in soil properties and stabilisers between samples of different compressive strength (or density) code to be formally calculated and tested. It should be noted that the ANOVA analyses below differ substantially from the ANCOVA used above, as it is a series of one-way ANOVA tests using one categorical variable (either compressive strength or density). Although compressive strength code (or density code) is treated as the predictor categorical variable in this case, and not the dependent variable, this is purely to examine statistically whether differences exist in the characteristics of two groups of samples. However, unlike ANCOVA, the one-way ANOVA tests do not isolate the unique effect of each variable on compressive strength or density, fundamentally because the criterion variables are now being used as predictor variables purely to characterise groups of samples. Therefore, the results cannot be used to rate the importance of each variable to variation in compressive strength/density, and should be interpreted only as values that characterise failed and successfully stabilised samples. As the variation in compressive strength attributable to stabiliser treatment variation has been shown to be very small compared with the effect of the soil properties (Sections 4.7.2, 4.8.1, and 4.8.2), only the soil properties are examined here.

Table 4.19 reports the results of analysis of variance of each variable by compressive strength code, using the 193 samples for which all soil properties have been measured (i.e., the same data set as for ANCOVA). All soil properties except clay/silt content differ significantly between the two levels of compressive strength code. Successfully stabilised samples are characterised by more gravel and less sand, slightly lower plastic limits, lower liquid limits and plasticity indices, and lower values of linear shrinkage and moisture (Table 4.19).

	Mean (UCS code = 0)	**Mean (UCS code = 1)**	**ANOVA P-value**
Clay/silt (%)	23.5	26.4	0.106
Gravel (%)	7.1	12.3	0.030*
Sand (%)	69.4	61.4	0.002*
L.L. (%)	40.1	31.1	0.000*
P.L. (%)	17.4	19.2	0.021*
P.I. (%)	22.6	11.5	0.000*
L.S. (%)	9.7	5.5	0.000*
Moisture (%)	12.8	9.8	0.000*

Table 4.19: Means of variables for the 2 levels of compressive strength code (193 samples). The ANOVA test examines whether the mean values of variables differ between the 2 levels of compressive strength code. P-values of the ANOVA F-test of < 0.05 indicate a statistically significant difference between the mean from one level of compressive strength Code to another at the 95 % confidence level. Starred values of the p-values indicate significant differences.

Table 4.20 displays the results of a series of one-way ANOVAs on each variable by density code, in order to characterise and compare samples that fail the density criterion (< 1.80 t/m^3) with those that pass (≥ 1.80 t/m^3). The textural properties do not show any significant differences between the two groups of samples. There are, however, significant differences in liquid limit, plasticity index, linear shrinkage, and moisture, all of which have lower values for samples that pass the density criterion. These results contrast with those for compressive strength, where textural properties are additionally of importance in distinguishing between successful and failed samples in terms of their compressive strengths.

	Mean (density code = 0)	**Mean (density code = 1)**	**ANOVA p-value**
Clay/silt (%)	26.7	25.4	0.455
Gravel (%)	8.9	12.0	0.152
Sand (%)	64.4	62.6	0.449
L.L. (%)	37.3	31.3	0.000*
P.L. (%)	18.2	19.0	0.281
P.I. (%)	18.6	12.1	0.000*
L.S. (%)	8.1	5.8	0.000*
Moisture (%)	12.5	9.7	0.000*

Table 4.20: Means of variables for the 2 levels of density code (193 samples). The ANOVA test examines whether the mean values of variables differ between the 2 levels of density code. P-values of the ANOVA F-test of < 0.05 indicate a statistically significant difference between the mean from one level of density code to another at the 95 % confidence level. Starred values of the p-values indicate significant differences.

CHAPTER 5: RESULTS AND DISCUSSION

5.1 MULTIPLE INFLUENCES ON STABILISATION

5.1.1 Relative importance of stabilisers and soils to compressive strength variation

This study has generated information to address aspects of the stabilisation process using a large data set on stabilisation of a wide range of soils. Several lines of evidence suggest that soil type, not stabiliser type or quantity, exerts the dominant influence on variation in compressive strength and density. First, the analysis of variance of a total of 177 compressive strength determinations made on 58 different soils using a variety of stabiliser treatments (Section 4.8.1) indicates that around 94 % of the variation in strength is due to differences in soil characteristics, and only 6 % to variation in the stabiliser type and amount (Table 4.10). This is a new and important result. Moreover, just 1 % of the variation in maximum density of stabilised soil is due to stabiliser variation. Second, analysis of covariance for both compressive strength and density using 193 samples shows that the stabilisers rate well behind the soil property variables when ranked for statistical significance (Tables 4.6 and 4.10). Third, when a multiple regression model is used to explain compressive strength or density variation in terms of the soil properties alone, the resulting level of explanation (an r^2adjusted of about 30 %) is little different from the analysis of covariance in which stabilisers are included as categorical variables (Section 4.8.2).

It is clear that the relative effect of the soil properties on compressive strength (> 90 %), compared with the effects of stabiliser types and quantities (< 10 %), is much greater than reported in previous studies (e.g. Croft, 1968; Akpokodje, 1985; Ngowi, 1997) as interpreted and discussed in Section 2.15.2 of Chapter 2. Although those researchers did not report their results explicitly in terms of the proportion of variation in strength attributable to soil variation and stabiliser variation respectively, the values shown in the summary Table 5.1 show that significant contrasts in compressive strength were generated by each of the three influences - differences in stabiliser type, differences in the quantity (percentage) of stabiliser, and differences in soil type. The effect of soils seems to be about the same as for the effect of the type of quantity of stabiliser.

For a comparison, the results of this study are shown in Table 5.1 in the same format as the earlier works. Overall, in this study, the difference in performance of cement and lime is 0.05 MPa, a difference that is statistically insignificant and which is comparable with the result of Croft (1968) for the stabiliser type difference. The effect of the quantity of stabiliser is similarly small (Table 5.1), and certainly much smaller an effect than found in previous work, which showed increases in compressive strength with increasing proportions of stabiliser of between 0.25 and 0.64 MPa per % of cement (average 0.40 MPa), and of between 0.14 and 0.29 MPa per % of lime (average 0.24 MPa) (Croft, 1968; Akpokodje, 1985; Bryan, 1988a; Walker, 1995; Ngowi, 1997). Cement in this study produced stabilised samples with average strengths ranging from 2.7 MPa (2 or 4 % cement) to around 3.0 MPa for 5-6 % cement (Table 4.7), and for lime, the contrast is even smaller.

The strength data of Croft (1968) and Akpokodje (1985) are 7-day values, compared with this study's 28-day data. Generally speaking, it is expected that any contrasts in strength between samples with different soil characteristics or treated with different stabilisers or quantities, would be greater for longer curing times, although the effect depends upon the particular soil type (e.g. Akpokodje, 1985; Symons, 1999). Therefore, the values in Table 5.1 pertaining to the work of Croft (1968) and Akpokodje (1985) are probably minimum values when compared with the data from this study. In addition, for reasons stated in Section 3.2.1.4, the samples tested for strength in this study have been compacted during their preparation using the modified Proctor test (an effective compaction pressure of 2700 kN/m^2), whereas Croft (1968) and Akpokodje (1985) compacted their samples using the standard Proctor test (600 kN/m^2), and Bryan (1988a), Walker (1995), and Ngowi (1997) used compaction pressures of 2000 kN/m^2. Increased compaction pressure generally increases the strength (and density) of a particular sample and decreases the optimum moisture content (e.g., Bryan 1988a), although the effect diminishes with increasing compaction. However, the effect of different compaction pressures on the relative difference in strength between different samples is unclear, although it evidently varies depending on the soil types involved and to a lesser extent upon stabiliser quantity (Bryan, 1988a). If differences in strength between samples do in fact increase under higher compaction pressures, the values in Table 5.1 pertaining to Croft (1968) and Akokpodje (1985) would be even larger (i.e., the contrasts with this study would be even greater), and the values pertaining to Bryan (1988a), Walker (1995), and Ngowi (1997) would be slightly over-estimated (i.e., the contrasts with this study would be slightly less). Overall however, Table 5.1 shows that the contrasts in results between this study and others, concerning differences in sample compressive strength between different soil types and stabiliser treatments, are considerable and overwhelm any stated differences in experimental technique.

Author	Number of soils studied	Lime-Cement difference (MPa)	Soil difference (lime) (MPa)	Soil difference (cement) (MPa)	Cement quantity (MPa per % added)	Lime quantity (MPa per % added)
Akpokodje (1985)	2	0.8	0.6	0.65	0.28	0.14
Croft (1968)	6	0.0	0.4	0.2	0.64	0.28
Bryan (1988a)	5	-	-	0.7	0.32	-
Ngowi (1997)	2	2.6	0.27	0.28	0.47	0.29
Walker (1995)	10	-	-	1.25	0.25	-
This Study	111	0.05(1)	1.12(2)		0.07(3)	0.03(3)

Table 5.1: Summary of results of previous work and of this study concerning effect of soil type, stabiliser type, and stabiliser quantity on sample strength. Refer to Section 2.15.2 in Chapter 2 for details on the figures derived from the previous studies cited. Strength values from Croft (1968) and Akpokodje (1985) are 7-day results; other results are 28-day strengths. Notes: (1) As calculated in Section 4.7.2. The value is not statistically significant. (2) Calculated using the 45 difference pairs of mean compressive strength for a random sample of 10 of the 58 samples used in the analysis of variance of Section 4.8.1 (Table 4.1: determination numbers 7, 13, 67, 83, 107, 119, 138, 156, 190, 219). (3) Calculated using the difference in mean compressive strength between 2 and 6 % of stabiliser from Table 4.7.)

Possible reasons for the lack of contrast between stabiliser quantities examined in this study are now discussed. One possibility is that lime and cement quantities of around 6 % do not produce significant increases in strength over quantities of 2 or 3 %. Many experimental analyses have used cement and lime at levels of 5 % and above, and contrasts between, for example, 5 % and 10 % stabiliser are apparent (e.g. Walker, 1995) (Figure 2.19). However, studies examining lower levels of stabiliser also indicate significant contrasts in compressive strength at these lower levels. Akpokodje (1985) demonstrated large 7-day compressive strength differences for cement between 2 and 6 % (Figure 2.15), and for lime (Figure 2.14) whose effect was more dependent on soil type. Metcalfe (1963) showed smaller 7-day compressive strength contrasts of 0.15 MPa for lime levels between 2 and 6 %, while Ausroads (1998) reported a difference in 21-day compressive strength of 0.7 MPa between lime contents of 2 and 6 %. Overall, therefore, the evidence suggests that significant contrasts in 28-day compressive strength may have been expected in this study, considering the range of lime and cement levels used.

A second possibility concerns soil type sample size and range. Walker (1995) found that differences in soil type were responsible for a large part of the variation in stabilised strength of soil blocks (Table 5.1), although the percentage of cement used also had an important influence. Walker used more samples than earlier workers, and his samples were modified so as to produce a very wide range of soil characteristics with clay content varying from 20 % to 100 % and plasticity index ranging from 10 to 35 %. The dominance of the soil effect found here may be similarly due to the wide range and large number of different soils examined in it. A large number and range of samples may allow the soil differences to approach the natural spectrum of soil characteristics. Studies reliant on a small number of samples have less soil variation; if the soil variation is indeed small as a result of a small number of soils tested, the stabiliser "signal" will be proportionally greater, and clear-cut differences between stabiliser types (e.g. the contrast between lime and cement) and between levels (e.g. the contrast between 2.5 % and 5 %) are more likely to be generated. Indeed, many investigations aimed at examining the effect of stabiliser percentage have used a single soil for this purpose (e.g. Indraratna et al., 1995; Symons, 1999). Statistically significant contrasts between different quantities of stabiliser may be beyond the limits of detection in this study given the major effect of variation in soil type as discussed. However, as the soil types tested in this study cover a large part of the spectrum of naturally occurring soils, this may provide a more accurate reflection of the effect of soil type variation on stabilised strength, compared with the effects of stabiliser type and quantity, than has previously been the case with studies that tested only small numbers of different soils.

Much of the limitation in inferring statistical differences between stabiliser types, and stabiliser levels, comes from the larger statistical confidence limits in this study, which are a function of the wide range of values of soil properties and their strong influence on causing variation in stabilised compressive strength. Thus, confidence limits in Figure 4.24, for example, are large compared with the variation in mean values, and indicate that the differences in strength between stabiliser levels is small compared with the variation within each stabiliser level. Confidence limits in Figure 4.30 are small compared with the differences between mean values. In different ways, these two diagrams graphically demonstrate the large amount of variation in compressive strength attributable to different soil characteristics, and the small variation in strength attributable to different stabiliser treatments. Generally, such statistical uncertainties as reported in this study have not been reported in previous work, and the uncertainties around mean values of variables such as stabilised strength are therefore not available for comparison.

5.1.2 Particular stabilisers for particular soils

Traditionally, particular soils have been regarded as being more suitable for particular types and levels of stabiliser, and much has been made of matching a stabiliser treatment to a soil. Most soils can be stabilised with cement (e.g. CSIRO, 1987; UN, 1992), although the greatest compressive strengths using cement as a stabiliser have been viewed as being obtained with gravels and sandy soils rather than with silts or clay (Wolfskill et al., 1963; UN, 1992). Sandy soils may require only 2-2.5 % of cement, whereas clay soils may require as much as 10 % (CSIRO, 1987). UN (1992) recommended that soils with lower values of linear shrinkage should be treated with lower values of cement. UN (1992) also recommended that cement is unsuitable for soils with liquid limits of higher than 30 %, although clayey soils can be treated first with lime. The effects of lime are more highly dependent on the nature of the soil(s) to be stabilised than cement, and in particular, lime is generally not likely to successfully stabilise soils with low or no clay content (Wolfskill et al., 1963). Dumbleton (1962) suggested that a minimum clay content of approximately 10 % and a plasticity index in excess of 10 % are desirable for lime stabilisation.

Despite the tradition of matching stabiliser type and quantity to soils, there is no evidence from the results of this study that particular stabiliser treatments are more beneficial than others for particular soils. This is primarily a function of the low amount of compressive strength variation effected by different stabiliser quantities compared with different soil types, as discussed above. Any effectiveness of particular stabilisers for particular soils is not detectable, and it is difficult on this basis to recommend stabiliser treatments for particular soils on the basis of strength. Contrasts in strength between higher levels of lime or cement are not statistically different from lower levels, due to the wide confidence limits around the mean values, which stem from the wide range of soils tested and the resulting wide range of strengths determined (Table 4.7). However, the results are at least suggestive of a strength increase, as the mean strength values for 5 and 6 % cement or lime are higher than for any other percentage quantities. A broad recommendation would therefore be to stabilise the chosen soil with 6 % of cement or lime (choice of the soil is the critical decision, not necessarily the stabiliser; soil suitability is discussed fully in Sections 5.2, 5.3, and 5.4 below). CSIRO (1987) did not recommend lime stabilisation on account of its effectiveness being dependent in a complex way on clay type and content, and its inferior strength gains compared with cement. However, the results of this study suggest that the performance of lime is similar to cement. The choice as to which of these two stabilisers to use may then depend on criteria other than strength. Lime should be considered for highly plastic soils that contain high percentages of clay and where workability is a problem. Lime could be added either 6 % on its own for these soils, or added prior in smaller amounts (say 2 %) in combination with 5-6 % cement (e.g. Akpokodje, 1985).

A different way to view matching stabilisers with soils is as follows. If the main contributor to compressive strength variation is variation in values of soil properties, then a corollary is that similar strengths could be achieved for different soil types by adjusting the stabiliser quantity according to the favourability of the soil. As discussed below in Section 5.3 and as shown in Figures 5.1 to 5.8, stabilised strengths differ substantially depending on the values of plasticity index, linear shrinkage, clay/silt content, and other soil properties. Different stabiliser treatments may be appropriate therefore depending on the soil characteristics. This is discussed more fully in Section

6.5 of Chapter 6. However, an example for illustration here is that stronger samples are produced by samples of lower plasticity index (e.g. Figure 4.45); therefore, samples with low plasticity could be treated with more conservative levels of lime or cement, of less than 6 %, and still stabilise at similar (acceptable) strengths to samples of higher plasticity but stabilised with levels of 6 %.

Another consideration concerning the amount of cement and/or lime to add to a soil is the shrinkage cracking that occurs in stabilised soil. Although not necessarily disadvantageous in the production of bricks or blocks due to their small size, shrinkage cracking can be a serious problem for rammed earth walls due to their monolithic nature. Clay-rich soils generally have higher shrinkage limits and are therefore more predisposed to cracking. Adding lime or cement to stabilise a soil reduces the linear shrinkage (Spangler & Patel, 1949; Akpokodje, 1985), at least in the range of up to 12 %, the maximum quantity examined by those studies. The higher the proportion of cement or lime used in the soil, therefore, the less likely the rammed earth wall is to develop shrinkage cracks. Spangler & Patel (1949) reported the results of an experimental study of the effect of various levels of lime (CaO) and portland cement on the properties of a clay soil. The natural soil had a linear shrinkage value of 14.3 %, but adding 2 % lime and 4 % lime reduced this figure to 6.5 % and 4.0 % respectively. For soil mixed with 2 % cement the equivalent linear shrinkage was 12.5 %, and for 4 % cement was 7.5 %. These results suggest that both stabilisers reduce linear shrinkage, with lime being more effective than cement. Akpokodje (1985) demonstrated that natural soils with linear shrinkage values of between 1 and 6 %, and stabilised with 6 % cement, have stabilised linear shrinkages of between 0.2 and 2.5 %. Also, his results showed for a range of soils that linear shrinkage reduces with increasing cement contents between 0 and 12 %. A common maximum acceptable value for shrinkage limit is 3 % (e.g, Akpokodje, 1985). Akpokodje's (1985) work showed that a natural soil with a linear shrinkage of 6 % needs 5 % cement to reduce this value to 3 % for the stabilised soil. Natural soils with shrinkages greater or less than 6 % will require correspondingly greater or lesser amounts of cement respectively to achieve the 3 % (or lower) stabilised soil shrinkage limit.

Although asphalt has been viewed as being better suited as a stabiliser for soils with high clay contents (CSIRO, 1987; UN, 1992), the results of this study suggest that asphalt can be used successfully in combination with either lime and/or cement on a wide range of soil types. The use of 3 % asphalt has no influence on the compressive strength or density of the stabilised soil when used in combination with lime and/or cement. Neither is there any indication of a particular affinity for asphalt by particular soils, or for preferential stabilisation for particular soils compared with others. Therefore, its use should be viewed as a broad-spectrum waterproofing agent, rather than an additive that imparts strength to stabilised soil when used in combination with either lime or cement.

CSIRO (1987) suggested that the amount of asphalt emulsion needed to effectively waterproof a sandy soil (70 % sand, 30 % clay) is about 5 %, compared with about 8 % for a clay soil (40 % sand, 60 % clay). However, the debate amongst rammed earth practitioners has not been about the additive's waterproofing effect, but rather the potential it may have to alter the strength of a rammed earth wall. The addition of asphalt to soil provides a membrane that impedes the penetration of water, and on this basis it has been suggested that asphalt stabilisation can improve both compressive strength and durability characteristics (Winterkorn, 1975; Al-Homoud et al., 1996). The evidence in this study however, based on 53 samples stabilised with asphalt in combination with cement and/or lime, and 177 without asphalt, suggests asphalt as a stabiliser does not affect compressive strength as the mean values are identical with or without asphalt. In addition, individual differences are small, generally 0.05 MPa or less, between same-sample determinations involving asphalt versus no-asphalt stabilisations (Table 4.1) over the whole range of soil types tested. Assuming that 3 % asphalt is effective for waterproofing purposes, the additional treatment of a cement and/or lime stabilised soil with asphalt emulsion does not affect either the strength or density of the stabilised material. However, over the long-term (in excess of the 28-day measurements made here), strength and durability characteristics of asphalt-stabilised walls over non-asphalt-stabilised walls could not be precluded.

5.1.3 Influences on maximum dry density

The variation of density of stabilised material as a function of stabiliser type, stabiliser quantity, and soil type, has received less attention in the literature than compressive strength. The data of Bryan (1988a) showed there to be no variation whatsoever in maximum dry density as a function of cement levels of between 5 % and 10 %, for any of the 5 soils that were measured. Similarly, Walker (1995) found that maximum dry density was largely independent of cement content, but was related to changes in textural properties of soil. Osula (1996) found only very small decreases in dry density of around 0.02 t/m^3 per % of stabiliser increase for lime and cement. The results of this study (Section 4.9.1) support these earlier findings. There are no differences in maximum dry density of stabilised samples across the three different stabilisers (Tables 4.14 and 4.15). Neither are there differences in density between different quantities for any one stabiliser. The mean densities for the different stabilisers and levels all lie between 1.83 and 1.95 t/m^3, and of these 14 determinations of mean density, 10 lie between 1.85 and 1.90 t/m^3.

5.2 IMPORTANCE OF DIFFERENT SOIL PROPERTIES TO STABILISATION

As soil type has been indicated in this study to be the dominant influence on stabilisation outcome, the remainder of this chapter is devoted to discussing soil property effects. This section examines the relative importance of the different soil properties to stabilisation outcome. Sections 5.3 and 5.4 below discuss soil suitability and discrimination/prediction of stabilisation.

The broad patterns of correlations between the outcome variables (strength and density) and the soil properties (Figures 4.2 to 4.18) show a larger amount of scatter than other studies, primarily on account of the larger number of samples examined. However, discernible trends are apparent in most cases. Although such relationships have been established before by studies using smaller sample numbers, there has been little attempt to separate the effects of the different soil properties or to rank or rate the soil properties in their importance. Information about the significance of each of the soil properties to compressive strength and density has been ascertained using two different methods in this study: analysis of covariance and multiple regression (Sections 4.7, 4.8.2, and 4.9; and discrimination between successful and failed stabilised samples (Section 4.10).

The results of analysis of covariance and multiple regression show that some soil properties are more important than others in influencing compressive strength of the stabilised soil. Although such linear models may not entirely suit the data patterns of some of the variables, they provide a useful insight into the relative importance of the soil properties to stabilisation. Linear relationships between the soil properties and compressive strength, when adjusted for inter-correlations between the soil properties themselves, are strongest for linear shrinkage, % sand, and % clay/silt (Section 4.7.3). Moisture content is middling in importance, and the plasticity measures are the least well related to compressive strength, based on a linear model. The importance of the textural properties is perhaps not surprising given that their influence on stabilised strength has been well researched in terms of concepts such as packing efficiency and granular structure of the constituent materials (e.g. Patty, 1936; Winterkorn, 1975; Bryan, 1988). Shrinkage during curing of a stabilised soil creates tensile stresses within the material mass, causing micro cracking (and visible cracks) which would tend to reduce strength. Therefore, natural soils with lower shrinkage may promote greater strengths, as the internal tensile stresses would be minimised. As the effect of adding cement or lime is to reduce shrinkage (e.g. Akpokodje, 1985), a natural soil with lower linear shrinkage has an advantage over soils with higher shrinkage.

For density, the results are somewhat different. Moisture content and % clay/silt are the most significant variables and show a negative correlation with density. These are followed by liquid limit, sand, plasticity index, and linear shrinkage, of which none is significantly related to density in a linear manner (Section 4.9.2). Optimum moisture content is intimately related to density as higher densities are achievable at lower moisture contents, due simply to the fact that water is less dense than soil particles and all other things being equal, the material with lower water content will be denser. Therefore, the importance of moisture content to density is not unexpected. The % clay/silt is also strongly related to maximum density, with soils having higher clay/silt contents being associated with lower maximum densities. The higher the clay/silt content, then the higher the optimum moisture content required for compaction due to the water requirements of the finer particles, and therefore the lower the maximum density achievable.

A rather different approach to ranking soil properties is to examine their discriminatory power in distinguishing between samples that pass or fail the pre-set criteria of 2 MPa for compressive strength and 1.8 t/m^3 for density. Discrimination using soil properties is discussed in more detail further below in Section 5.4, but the best overall discriminators appear to be linear shrinkage, plasticity index, and moisture content (Sections 4.10.1 and 5.4). Clay/silt content, sand content, and liquid limit are all moderately good discriminators, and gravel content and plastic limit are the least effective. The best and worst discriminators are the same for density as those for strength discrimination.

There are evidently some differences in the results between the two methods of rating the importance of the soil properties to compressive strength. For example, plasticity measures assume greater importance when used to discriminate stabilised soil with respect to pre-set strength (and density) criteria. The differences arise because the first method rates the unique statistical significance of each variable in terms of its linear relationship to compressive strength; whereas, the second method assesses how well each variable is able to discriminate between two groups of samples defined by a strength criterion. However, although the results are mixed, linear shrinkage emerges as the most important variable as judged by both methods. The soil properties are not effective at predicting actual values of strength: the combined soil properties explain only 29 % of the variation in compressive strength, using a linear model (Section 4.8.2). The relationships between soil properties and stabilised strength may be more complex than linear, or there may be other variables influencing strength that are not included in the model. However, the soil properties have been shown to be very effective at indicating soil suitability for stabilisation by being able to discriminate between groups of samples based on a criterion value of strength (or density) (Section 4.10). Therefore, these aspects of the results are developed and discussed in more detail in Sections 5.3 and 5.4.

5.3 SOIL SUITABILITY AND UNSUITABILITY FOR STABILISATION

The literature concerning soil suitability for stabilisation has focused on texture and plasticity as the primary properties of soil that determine its potential for successful stabilisation. The range of suitable soils for rammed earth is more limited than for mud bricks, because the walls are constructed in long sections and are more prone to shrinkage cracking from clay dehydration (CSIRO, 1987). Generally, the most suitable type of soil for rammed earth construction is a well-graded soil, with a range of particle sizes from sands through to clays (CSIRO, 1987; UN, 1992), although certain types of soils appear to produce more stronger and denser rammed earth material than do others. Patty (1936) provided data that showed the best rammed earth walls were those that contained about 75 % sand and 25 % clay/silt. The worst walls were those that had less than about 40 % sand and in excess of 60 % clay/silt. Middleton (1947) found similarly; his best condition experimental walls contained over 70 % sand, and the worst contained less than 40 %, with intermediate values of sand corresponding to middling quality walls. Akpokodje (1985) tested cement and lime stabilisation of soils that included a sandy loam (59 % sand, 28 % silt, 13 % clay), a clay loam (36 %, 27%, 37 %), and a silty loam (15 %, 75 %, 10 %). Overall, the sandy loam led to the strongest stabilised sample and the silty loam was the weakest. This result indicated that a higher proportion of sand to clay/silt content imparted greater strength to the stabilised sample.

Croft (1968) found that textural classification of soils was of little value in assessing their suitability for stabilisation, as correlations of textural indices with strength values were poor. However, Croft also found that Atterberg limits of the natural soils showed similarly poor relationships to the strength of stabilised soil, and were also deemed (by Croft) to be of limited value for predicting suitability for cement stabilisation as assessed against a criterion strength value of 1.7 MPa. Bryan (1988) concluded that plasticity is less sensitive than textural information for assessing the suitability of soil for cement stabilisation.

Other experimental examinations have provided more specific information concerning soil suitability for stabilisation, in addition to those outlined above. This information is reported in Table 5.2, which collates data on the values of soil properties as recommended by various authors for the satisfactory stabilisation of soil. Although different authors used slightly different criteria for suitability, and although most of the data are with reference to cement stabilisation of soil, and for earth bricks not rammed walls, the collation can be regarded as representing the best data with which to compare the data of this study. Some of the main features of the data include the following. The most popular indicators of soil suitability are liquid limit and plasticity index; however, no authors have offered recommendations concerning gravel content, plastic limit, or linear shrinkage of the natural soil. The data for textural and plasticity recommendations are consistent between authors to a degree. The average or modal values as recommended are < 30 % clay, < 35 % clay/silt, > 70 % sand, < 30-40 % liquid limit, < 15-20 % plasticity index, and 8-15 % optimum moisture content.

Author	**Clay (%)**	**Clay/ silt (%)**	**Sand (%)**	**Gravel (%)**	**LL (%)**	**PL (%)**	**PI (%)**	**LS (%)**	**OMC (%)**
NGS (1970)					≤ 30		≤ 12		
Walker (1995)	20-35		70-85		≤ 30		5-15		
Webb et al. (1950)	≤ 20				≤ 30		≤ 20		6-17
Fitzmaurice (1958)	5-20				< 40		2.5-22		10-14
Ransom (1963)	15-35				≤ 50		≤ 30		
Spence (1975)		< 35					< 20		
VITA (1975)		5-30	≥ 33						
Bryan (1988)	5-30				≤ 40				
Symons (1999)			> 73						
This study		21-35	30-60	3-5, > 12	< 36	16-19	< 16	< 7.1	5.0-10.0

Table 5.2: Summary of existing data relevant to soil suitability for stabilisation, and data from this study. Values are those recommended by the authors as most favourable for stabilisation, indicated on the basis of empirical research.

Regarding soil suitability derived from data in this study, the following can be reported as a summary from Sections 4.10.1 and 4.10.2. as regards to favourability for successful stabilisation based on the compressive strength 2 MPa criterion:

Clay/silt content: clay/silt contents between 21 and 35 % are best.
Gravel content: gravel contents between 3 and 5 % and above 12 % are best.
Sand content: sand contents from 30-48 % are best, and between 51 and 60 % are also good; sand contents above 75 % are worst.
Liquid limit: liquid limits less than 36 % are best; liquid limits above 45 % are poor and above 57 % are very poor.
Plastic limit: plastic limits between 17 and 19 % are good and between 23-25 % are poor.
Plasticity index: plasticity index values less than 16 % are best; values above 30 % are poor and over 35 % are worst.
Linear shrinkage: values of linear shrinkage less than or equal to 4.0 % are best, and between 4.0 and 7.0 % are also very good; values above or equal to 13.0 % are poor.
Moisture content: moisture values between 5.0 and 10.0 % are best, and between 11.0 and 13.0 % are reasonable; values above 13.0 % are poor.

The results regarding the most favourable soil property values for successful stabilisation are included in Table 5.2 to compare with existing information. One major difference in the overall data scheme is that new information is available for three variables previously disregarded as indicators of soil suitability for stabilisation - linear shrinkage, gravel content, and plastic limit - as well as for all other variables except clay content. A second major difference is that the recommended values of soil properties are more tightly constrained than earlier investigations. This may be due to narrower criteria as to what standard must be attained by a soil property to be regarded as "favourable", but is also likely to be a result of a substantially larger soil sample database from which more precise results have been able to be obtained.

Regarding the textural properties, the upper boundary recommended by this study for most favourable clay/silt content is similar to Spence (1975) and VITA (1975) at 35 %, but the lower bound is higher than those studies at 21 % (Figure 5.1). Gravel content is a variable not used previously for discriminating soils, and although ignored by previous workers in favour of a focus on sand and clay contents, has been shown here to provide useful information. Gravel values of 3-5 % or greater than 12 % are favourable for stabilisation (Figure 5.2) and minimal values or no gravel (< 3 %) is less favourable. Having some gravel in the soil does therefore appear to be important to stabilised sample strength, because it will impart frictional strength to the stabilised soil. Figure 4.41 indicates that all samples with gravel contents exceeding 35 % were successfully stabilised. However, there will be an upper limit to the amount of gravel able to be used in a soil for rammed earth stabilisation based on practical considerations. Usually, soils with less than 90 % passing the 2.36 mm sieve (i.e. > 10 % gravel) would not be used for stabilisation and construction by practitioners.

The recommended levels of sand differ somewhat from previous recommendations and traditional understandings, which have usually indicated the favourability of having higher proportions of sand, and in particular most favourable at > 70 % (e.g. Patty, 1936; Akpokodje, 1985; UN, 1992; Walker, 1995). This study favours sand contents between 30 and 60 % (Figure 5.3). Although this range does not preclude successful stabilisation for sand levels in excess of 60 %, Figures 4.42 and 5.3 clearly show that sand contents less than or equal to 60 % should be preferred. These new value ranges indicate that there are a variety of soil particle size distributions that can promote successful stabilisation.

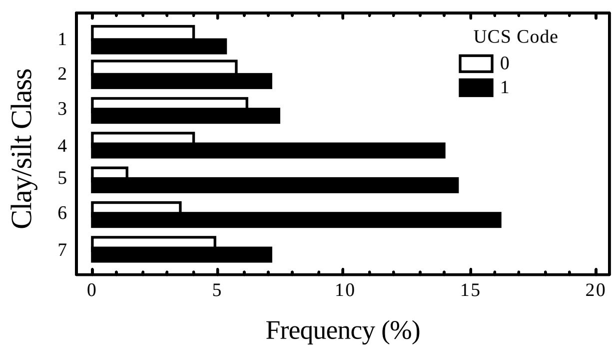


Figure 5.1: Data for clay/silt from Table 4.17 as percentages of the total of 230 observations.

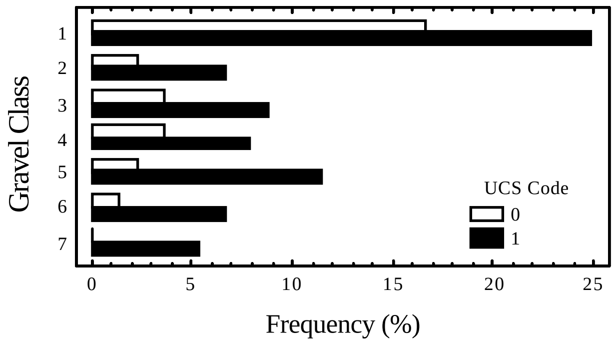


Figure 5.2: Data for gravel from Table 4.17 as percentages of the total of 230 observations.

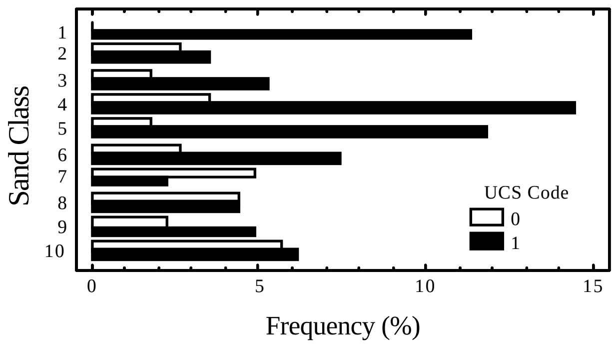


Figure 5.3: Data for sand from Table 4.17 as percentages of the total of 230 observations.

The two plasticity measures routinely used to assess soil suitability (Table 5.2) are liquid limit and plasticity index. The upper bound for liquid limit recommended in this study of 36 % (Figure 5.4) is very similar to that of the average of earlier studies, which have tended to settle on either 30 or 40 % as a threshold figure. Eighty-two percent of the 141 samples with liquid limits less than 36 % passed the strength criterion value of 2 MPa (Table 5.3). The figure of 15 % for plasticity index (Figure 5.5) is similarly consistent with previous determinations. For samples of plasticity index less than 16, a total of 103 samples out of 118 (87 %) passed the 2 MPa strength threshold. Other researchers have regarded plastic limit as a means toward calculating the plasticity index, and not in itself as a useful variable (e.g. Bryan, 1988a; Walker, 1995). However, it is evident from Figures 4.44 and 5.6 that plastic limit can provide useful information regarding soil suitability. There is a narrow range of plastic limit values between 17-19 % that promotes successful stabilisation. In this range, 87 % of the 68 samples exceed the 2 MPa strength criterion value (Table 5.3).

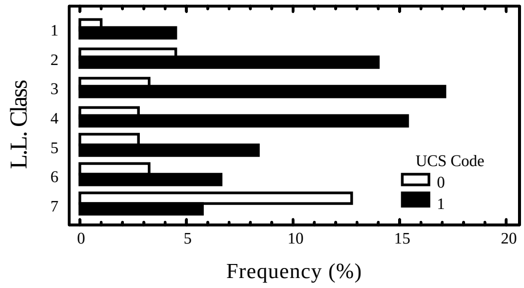


Figure 5.4: Data for liquid limit from Table 4.17 as percentages of the total of 230 observations.

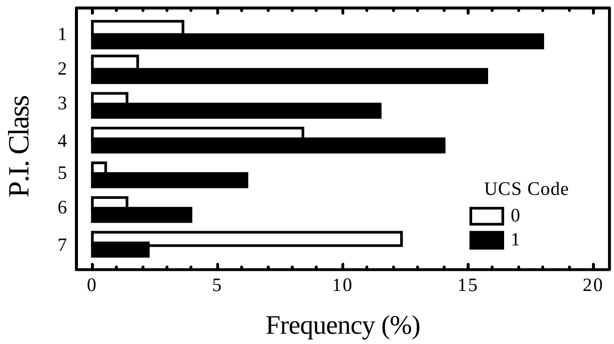


Figure 5.5: Data for plasticity index from Table 4.17 as percentages of the total of 229 observations.

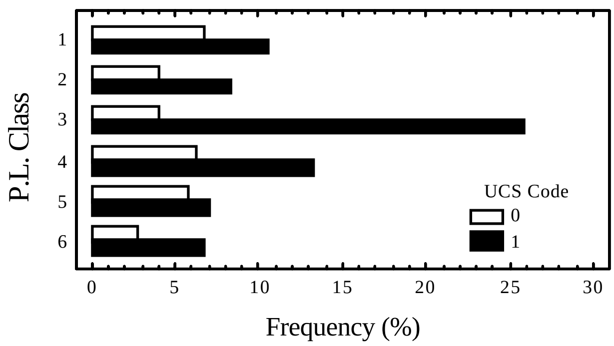


Figure 5.6: Data for plastic limit from Table 4.17 as percentages of the total of 229 observations.

The linear shrinkage of the natural soil has previously been used as a variable to predict the likely shrinkage characteristics of stabilised soil. For example, Akpokodje

(1985) demonstrated that natural soils with linear shrinkage values of between 1 and 6 % and stabilised with 6 % cement will result in stabilised linear shrinkages of between 0.2 and 2.5 %. However, as with gravel content and plastic limit, the property has been ignored as an indicator of suitability of a soil for stabilisation in terms of a compressive strength criterion. Linear shrinkage values of less than 7.1 % promote successful stabilisation (Figure 5.7), with 108 out of 119 samples (91 %) passing the 2 MPa strength criterion value in this range (Table 5.3). Linear shrinkage may be such a good indicator of stabilised strength because it represents the tendency of internal tensile stresses to pull apart the material from within and thereby weaken it; alternatively, linear shrinkage as a property may also encapsulate some of the other influences on stabilised strength including clay/silt content. Besides being a very good indicator of stabilised compressive strength, the linear shrinkage of the natural soil has an important bearing on the shrinkage behaviour of the stabilised soil, and therefore this property assumes even more significance in stabilisation. Stabilised soils with lower shrinkages favour less cracking of the earth wall, and Akpokodje (1985) showed that the shrinkage of stabilised walls is heavily dependent on the shrinkage of the natural soil used. The criteria set by CSIRO (1987) for assessing rammed earth walls include reference to the density, depth, and width of cracking, which must be met to meet their standards for successful rammed earth walls.

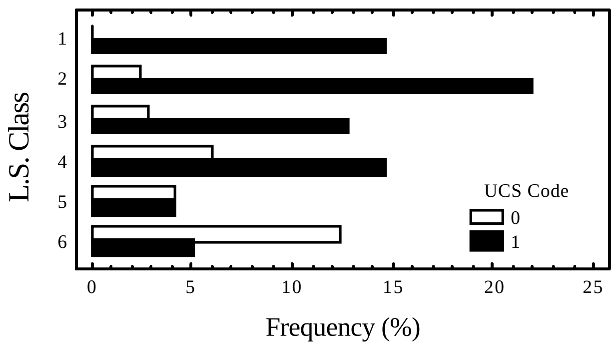


Figure 5.7: Data for linear shrinkage from Table 4.17 as percentages of the total of 220 observations.

Optimum moisture contents favourable for stabilisation determined in this study are more narrowly constrained and lower than either of Fitzmaurice (1958) or Webb et al. (1950) (Table 5.2; Figure 5.8).

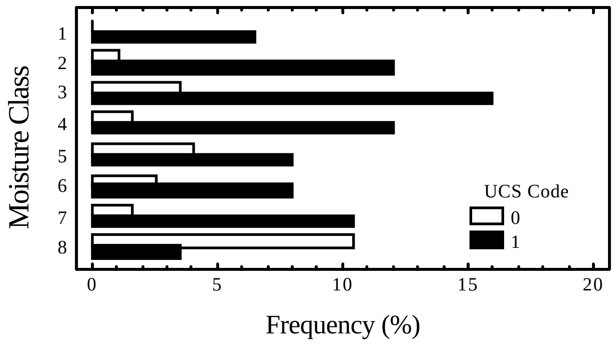


Figure 5.8: Data for moisture content from Table 4.17 as percentages of the total of 202 observations.

Further to the recommendations given for values of soil properties that favour stabilisation, Table 5.3 shows values of soil properties that are unfavourable to successful stabilisation in comparison. It also specifies values outside of the ranges of the most and least favourable values, here termed satisfactory, being intermediate in favourability (Table 5.3). Quantified recommendations for values of soil properties that disfavour stabilisation, or which are intermediate in character, have not previously been specified by other workers, who have confined their recommendations to favourable values only. It is clear that the information provided in Table 5.3 is very useful, as it differentiates between three sets of values of soil properties: those that should actively be avoided (unfavourable), those that should be actively sought (favourable), and those that are satisfactory and could be worked with depending on the values of the more important discriminating soil properties (Section 5.4.2).

	Favourable			Satisfactory			Unfavourable		
	Values	N	% ≥2.0 MPa	Values	N	% ≥2.0 MPa	Values	N	% ≥2.0 MPa
Clay/silt (%)	21-35	122	84	36-45	16	69	< 21 or > 45	92	54
Gravel (%)	3-5 or > 12	81	84	6-12	54	70	< 3	95	60
Sand (%)	30-60	97	81	61-75	70	70	> 75	63	56
Liquid Limit (%)	< 36	141	82	36-45	47	72	> 45	42	31
Plastic Limit (%)	17-19	68	87	<17 or 20-22 or > 25	132	67	23-25	29	55
Plasticity Index (%)	< 16	118	87	16-30	78	71	> 30	33	15
Linear Shrinkage (%)	< 7.1	119	91	7.1-13.0	68	66	> 13.0	33	21
Moisture content (%)	5.0-10.0	105	89	10.1-13.0	69	77	> 13.0	28	25

Table 5.3: Favourable, satisfactory, and unfavourable values of soil properties for successful stabilisation (based on strength criterion) of soil for rammed earth construction, from this study. Note that the lower boundaries for % sand and optimum moisture content for favourable stabilisation are based on the absence of any observations below those values. Columns marked N refer to the total number of samples in the specified ranges of values, and columns marked % ≥ 2.0 MPa refer to the percentage of these samples that passed the strength criterion of 2 MPa.

5.4 DISCRIMINATION AND PREDICTION OF STABILISATION OUTCOMES

5.4.1 Prediction of stabilised strength

A key requirement in the stabilisation process is to be able to predict the likelihood of successful stabilisation given the soil at hand, and to be able to modify the soil if necessary in a way that will increase the chance of success. The soil property variables explain less than 30 % of the variation in sample compressive strength when a multiple regression model is used (Section 4.8.2), and this result is similar for density variation. Given this rather low level of explanation, it is evident that to try to estimate or predict a particular compressive strength of new samples using measured values of soil properties, using a linear statistical model, would be rather unsatisfactory. However, given the ability of some soil properties to discriminate between samples using a threshold outcome criterion (2 MPa for compressive strength, or 1.80 t/m^3 for density), it seems more productive to use prediction of stabilisation outcomes in terms of success and failure, that is, as measured by values above and below a given criterion. This approach is now discussed.

With respect to the 2 MPa strength criterion, overall there was a 70 % pass rate for the samples stabilised in this study. Table 5.3 reports data for favourable, satisfactory, and unfavourable soils with respect to the 2 MPa criterion. It shows the relevant ranges of values, the total number of samples within those ranges, and the percentage of samples within those ranges that passed the 2 MPa criterion. The percentage of samples passing the 2 MPa criterion for favourable soil property values ranges from 81 % (for gravel as a discriminator) to 91 % (for linear shrinkage). These percentage figures drop for the satisfactory categories for which the figures range from 66-77 % passing the criterion of 2 MPa. For unfavourable soils, the percentage of samples passing the strength criterion is again lower at between 15 % (for plasticity index) and 60 % (for gravel).

It is evident that some soil properties are better than others at discriminating between strong and weak samples. The two poorest discriminators are probably gravel and plastic limit. Gravel content has a relatively small number of samples placed in the favourable range, and is not very good at discriminating the unfavourable samples (Table 5.3). Plastic limit has a small number of samples in the favourable category, and a preponderance of samples in the satisfactory category, and again the discrimination of unfavourable samples is poor. Clay/silt content could be used with some success as a discriminator of stabilised strength. The ability of any of the textural measures to discriminate between favourable and unfavourable soils is inferior to plasticity index and linear shrinkage. This is perhaps surprising given the emphasis on the importance of textural properties in the stabilisation literature. Measures of plasticity and shrinkage reflect both the constituent particle make-up of a soil, and how that soil responds to moisture. If these variables do reflect both textural and moisture effects, they are likely to be closely influence the physical processes of compaction and stabilisation, which would explain their effectiveness at assessing soil suitability for stabilisation.

Figure 5.9 shows that linear shrinkage and plasticity index complement each other in combining to differentiate between weak and strong stabilised samples, and the variables could be used with reasonable optimism to predict whether a new sample would be successfully stabilised. Other combinations of variables are less successful, as exemplified by gravel content and plastic limit (Figure 5.10). These variables show much less ability to discriminate between samples and have lower predictive potential. If a small number of soil properties can be used to successfully predict suitable (and unsuitable) soils for stabilisation, then an important consideration is whether field indices can be related to these attributes so that suitable soils can be identified efficiently in the field. This question is addressed in Chapter 6.

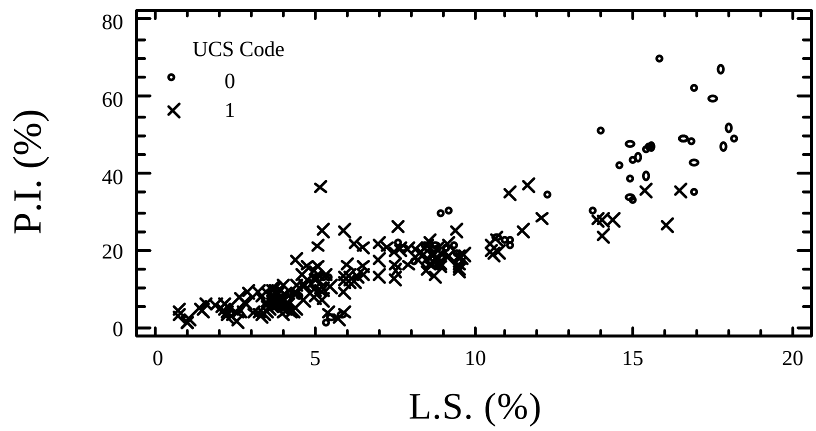


Figure 5.9: Good discrimination of weak (< 2 MPa) samples (compressive strength Code = 0) and strong (≥ 2 MPa) samples (compressive strength code = 1) using plasticity index and linear shrinkage.

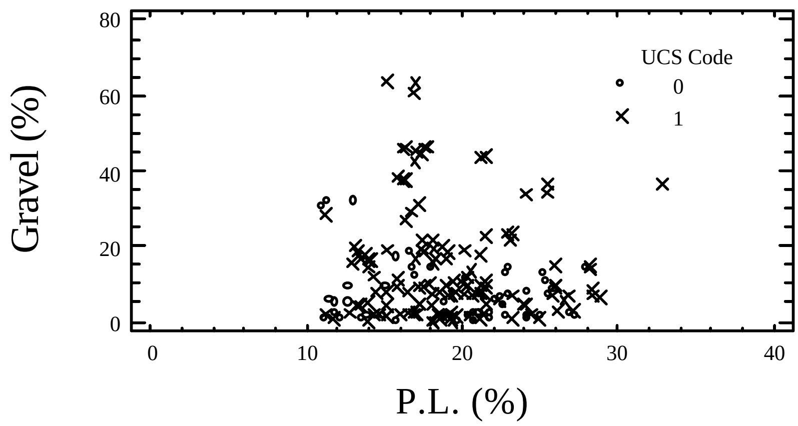


Figure 5.10: Poor discrimination of weak (< 2 MPa) samples (compressive strength Code = 0) and strong samples (≥ 2 MPa) (compressive strength code = 1) using gravel content and plastic limit.

5.4.2 The importance of linear shrinkage and plasticity index

5.4.2.1 Discrimination of favourable, satisfactory, and unfavourable soils

On the basis of the results of Table 5.3, the two best discriminators are linear shrinkage and plasticity index. These two variables are regarded here as primary discriminators of soil suitability for stabilisation. This contrasts with the results of Bryan (1988) whose results suggested that measures of plasticity were inferior to textural properties in discriminating soil suitability for cement stabilisation. Moisture content is also a very good discriminator, but on a practical basis can be put aside, as the determination of optimum moisture content is an involved and lengthy procedure whereas determinations of shrinkage and plasticity index are far more easily made. Moreover, the variables of linear shrinkage and plasticity index are sufficiently powerful at discriminating between failed and successful samples to obviate the need for using moisture content as a discriminator. Linear shrinkage has a large number of samples (119) in the favourable category, and 91 % of these passed the 2 MPa criterion (Table 5.3). There are 33 samples in the unfavourable category, and the discrimination is again good with only 21 % of samples passing the 2 MPa value. Plasticity index is not quite as efficient at discriminating favourable samples (87 % of 118 samples in the favourable category passed the strength criterion), but is slightly better at identifying the least favourable samples, with only 15 % of samples passing the 2 MPa threshold. The two variables generally classify samples in the same category. There are just three values of lineal shrinkage that are not in the favourable category when the plasticity index variable is favourable, and nine values of plasticity index that are not favourable when lineal shrinkage is favourable; all of these samples passed the 2 MPa strength threshold. In the unfavourable categories, 27 of 33 samples that had unfavourable values of shrinkage also had unfavourable values of plasticity, and the remaining 6 had satisfactory values; 29 of 33 samples of unfavourable plasticity also had unfavourable linear shrinkage values.

Of the 109 samples that have favourable values of both linear shrinkage and plasticity, 99 (90 %) passed the 2 MPa threshold; 40 of the 62 samples (65 %) that have satisfactory values of both variables passed the 2 MPa values; as did 3 out of the 29 samples (10 %) that have unfavourable values of both variables. If these figures can be applied to new stabilisation situations, then they can be regarded as likelihoods of soils that possess particular values of shrinkage and plasticity being successfully stabilised. It is evident that to maximise the likelihood of successful stabilisation, one should aim to use soils with favourable values of both plasticity index and linear shrinkage. For such soils, one may expect about a 90 % likelihood of stabilisation success. However, using a soil designated as satisfactory would still give about a 65 % probability of producing a stabilised material with strength in excess of 2 MPa, stabilised using cement or lime at 6 %. Importantly, soils with both unfavourable shrinkage and plasticity characteristics have very low likelihoods (10 % probability) of being satisfactorily stabilised, and should certainly be avoided.

5.4.2.2 Soils with favourable values of shrinkage and plasticity

As discriminators of soil favourability for stabilisation, the other soil properties are subordinate to linear shrinkage and plasticity index. Table 5.4 reports the numbers of samples that have favourable values of linear shrinkage and plasticity index but which failed the 2 MPa compressive strength criterion, broken down according to category of favourability for the other soil properties. There are 109 samples with favourable values of both shrinkage and plasticity, 10 of which failed. One overall result evident from Table 5.4 is that the great majority of samples with favourable shrinkage and plasticity values, but with unfavourable or satisfactory values of other variables, are successfully stabilised. Therefore, if lineal shrinkage and plasticity index values are favourable, then satisfactory or unfavourable values of the other variables can generally be tolerated without compromising stabilisation success. This is a most significant result for practical soil stabilisation.

	Number of samples in favourable category with UCS < 2 MPa	**Number of samples in satisfactory category with UCS < 2 MPa**	**Number of samples in unfavourable category with UCS < 2 MPa**
Clay/Silt	0/53 (0 %)	3/11 (27 %)	7/45 (16 %)
Gravel	3/48 (6 %)	4/29 (14 %)	3/32 (9 %)
Sand	9/62 (15 %)	1/29 (3 %)	0/18 (0 %)
LL	10/105	0/4 (0 %)	0/0 (0 %)
PL	0/41 (0 %)	10/58 (17 %)	0/10 (0 %)
Moisture	3/68 (4 %)	4/35 (11 %)	3/6 (50 %)

Table 5.4: Number of samples failing the 2 MPa strength criterion that have favourable values of LS (< 7.1 %) and PI (< 16 %), placed according to favourable, satisfactory, and unfavourable categories of other soil properties as defined in Table 5.4.

A closer examination of Table 5.4 reveals that some variables are more significant in helping explain why some samples with favourable shrinkage and plasticity values failed the strength criterion. The failed samples have sand contents and liquid limits overwhelmingly within the favourable ranges of these variables; for example, none of the 18 samples that have unfavourable sand content values but favourable shrinkage/plasticity values failed the strength criterion. Therefore, neither sand content nor liquid limit can explain stabilisation failure for samples with favourable shrinkage and plasticity. However, clay/silt content does exert some control on whether stabilised strength exceeds 2 MPa, as the majority of the 10 failed samples have unfavourable clay/silt contents (7 samples) or satisfactory values of clay/silt 3 samples). Importantly, unfavourable or satisfactory amounts of sand (> 60 % sand) in samples where linear shrinkage and plasticity index are favourable do not generate samples with low (< 2 MPa) strength (Table 5.4). Therefore, the key textural discriminator is clay/silt content rather than the percentage of sand. On a practical level, for soils with favourable shrinkage and plasticity, one should avoid soils with < 21 or > 45 % clay/silt, and be less concerned as to whether or not sand content is in the favourable range of values. Clay/silt content should be viewed as a useful secondary discriminator that is able to explain the small proportion (10 %) of samples that fail stabilisation when linear shrinkage and plasticity index values are favourable. The likelihood of stabilisation success when shrinkage and plasticity are favourable and clay/silt content is unfavourable/satisfactory is about 80 % (10 samples from 54), and is over 95 % (53/53) if clay/silt content is favourable (Table 5.4).

Although Table 5.4 shows that moisture content is also important, it does not need to be considered for practical soil discrimination due to the reasons discussed above. The significance of plastic limit is a little unclear. All the failed samples have values of plastic limit within the satisfactory range. Table 5.3 shows that this may be because of the very large relative size of the satisfactory category (132 samples) for this soil property, and the negative influence of satisfactory plastic limit values (Table 5.4) may be inflated due to this. However, it may be, as with clay/silt, satisfactory or unfavourable values of plastic limit should also be avoided if possible to increase the likelihood of stabilisation success.

Although 109 samples have favourable values of both linear shrinkage and plasticity index, several samples have favourable values of shrinkage but satisfactory values of plasticity (8 samples), and a few have favourable values of plasticity but satisfactory values of shrinkage (3 samples). All 11 of these samples passed the 2 MPa strength criterion. Therefore, it is probable that in new stabilisation situations, one or the other of the primary discriminators can be satisfactory when the other is favourable, and the soil is as likely to be successfully stabilised as when both discriminators have favourable values.

5.4.2.3 Soils with satisfactory values of shrinkage and plasticity

For soils that have satisfactory values of linear shrinkage and plasticity index, around 65 % (40 of 62 samples) were successfully stabilised. A comparison of the characteristics of the failed and successful samples in this category revealed no consistent differences between the two groups in clay/silt content, gravel content, liquid limit, plastic limit, or moisture. However, Figure 5.11 shows that there is an appreciable difference between the two groups in their sand content. This figure shows the variation in strength with sand content for samples with satisfactory values of shrinkage and plasticity. Just 2 samples out of 16 (13 %) with favourable sand contents failed the strength criterion; 15 from 30 (50 %) with satisfactory sand contents failed; and 6 from 16 (38 %) with unfavourable sand contents failed. Sand content, therefore, operates as a secondary discriminator when values of shrinkage and plasticity categorise a soil as satisfactory. For these samples, the key appears to ensure that the sand content is lower than 60 %, for which soils the likelihood of successful stabilisation is around 85 %. If sand content is higher than 60 %, then the likelihood of successful stabilisation drops to around 55 %. This likelihood is probably unacceptable for rammed earth practitioners, and therefore the "cut off" point for acceptable soils lies within the satisfactory category of soils, as defined, for the 2 MPa strength criterion.

Sand content as a secondary discriminator, therefore, operates in a different way for these samples compared with samples that have favourable values of shrinkage and plasticity (Section 5.4.2.2). Sand content is important to consider when shrinkage and plasticity are satisfactory, but not when shrinkage and plasticity are favourable; in that case, clay/silt is important as a secondary discriminator.

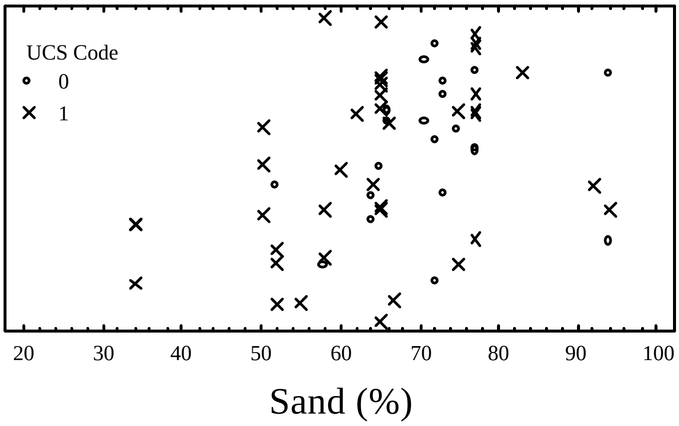


Figure 5.11: Variation in stabilisation success (UCS code 1 refers to samples ≥ 2 MPa; UCS code 0 refers to samples < 2 MPa) with sand content for samples with satisfactory values of linear shrinkage and plasticity index. Favourable values of sand are ≤ 60 %.

5.4.2.4 Soils with unfavourable values of shrinkage and plasticity

For samples that have an unfavourable value of linear shrinkage, 80 % failed stabilisation, and for samples with unfavourable plasticity, 85 % failed (Table 5.3). For those samples that have unfavourable values of both linear shrinkage and plasticity, 25 from 27 failed, or 90 %. Therefore, a soil with an unfavourable value of either linear shrinkage or plasticity index, or both, is very unlikely to be stabilised successfully, and a soil with both properties unfavourable is even less likely. In conclusion, no soil with an unfavourable value of shrinkage or plasticity should be used for rammed earth wall construction.

5.4.3 Compressive strength and density relationships

Another issue regarding discrimination or prediction concerns the relationship between density and compressive strength. Figure 5.12 displays this relationship, and the pattern of data shows that although there is a strong positive correlation between compressive strength and density, samples with higher density are not necessarily characterised by greater strength. Of the 202 determinations plotted in Figure 5.12, 61

% pass both the strength and density criterion (top right quadrant, Figure 5.12), 8 % just the density criterion (bottom right quadrant), 15 % just the strength criterion (top left quadrant), and 16 % neither criterion (top left quadrant). Of the 139 samples that pass the density criterion, 16 (12 %) did not subsequently pass the strength criterion. Therefore, if the maximum dry density of the compacted material prior to curing were used as a guide as to whether the sample would subsequently pass the 28-day compressive strength 2 MPa criterion, then the accuracy rate is quite high at 88 %. What is of concern however is the fact that of the 63 samples plotted that failed the density criterion, almost half (30) subsequently achieved stabilised strengths of ≥ 2 MPa. Therefore, the density criterion test is not a particularly good indicator of whether a sample will be stronger than 2 MPa, particularly if the sample failed the density criterion. Previous work (e.g. Bryan, 1988) has suggested a close, predictable relationship between density and compressive strength. However, those studies had smaller sample sets, and the more intricate relationship between these two variables as found here was probably not detectable because of the smaller sample bases. Because the density criterion test is not a good indicator of whether or not the sample will meet the strength criterion value, and because some of the soil properties (particularly linear shrinkage and plasticity index) give a very good indication of the likelihood of stabilisation success, the use of the maximum dry density of a sample as a guide to its subsequent strength is not recommended.

DeaTech (1999) suggested that a viable non-destructive method of measuring rammed earth wall strength would be to measure wall density (compaction) using a nuclear density gauge, and correlating these density measurements with compressive strength. However, given the variation in compressive strength for any one density value (Figure 5.12), using density to estimate compressive strength would be unlikely to produce reliable results.

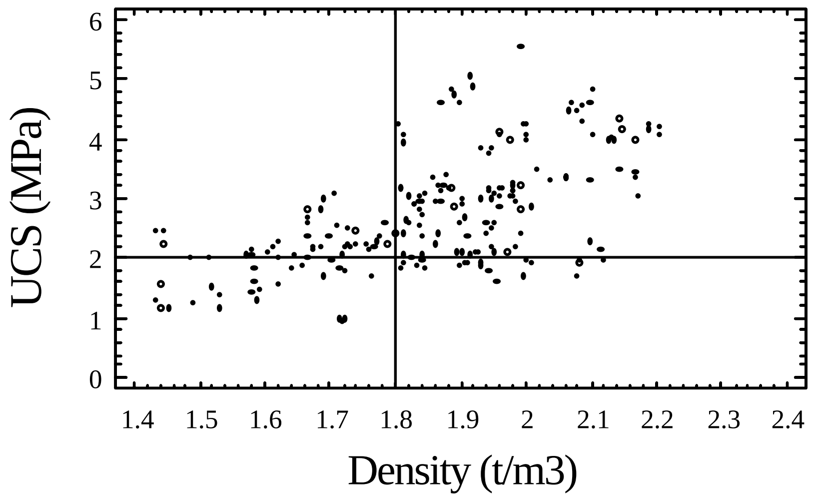


Figure 5.12: X-Y scatterplot of compressive strength versus density for 193 stabilised samples. The lines compressive strength = 2.00 MPa and Density = 1.80 t/m^3 represent the criterion values for successful stabilisation.

The pattern of data in Figure 5.12 suggests that the values of soil properties that favour successful strength stabilisation are not necessarily those that optimise density. What this means is that selecting a soil to pass the strength criterion will not necessarily optimise the likelihood of passing the density criterion. Given, though, that density is subordinate to compressive strength in terms of the structural and regulatory criteria applicable to rammed earth walls, soil characteristics should be chosen first and foremost to suit compressive strength requirements.

5.4.4 Alternative criterion values

Alternatives to the 2 MPa strength and 1.8 t/m^3 density criteria could be similarly investigated. Table 5.5 reports the success rates of samples for different strength criterion values using the favourable, satisfactory, and unfavourable categories as defined by shrinkage and plasticity for the 2 MPa criterion. If the criterion value is reduced to 1.5 MPa, both favourable and satisfactory soils as defined perform extremely well. For this more conservative criterion, the "cut off" point for acceptable samples lies at the boundary between the satisfactory and unfavourable soil categories, rather than within the satisfactory category as for the 2 MPa criterion (Section 5.4.2.3). As expected, the percentage of samples passing progressively higher criterion values decreases, with favourable soils outperforming satisfactory and unfavourable soils. An increase of the criterion value from 2 to 3 MPa dramatically reduces the success rate of favourable soils from 91 % to just 43 %. In order to better examine the characteristics of soils needed to pass the 3 MPa criterion, Table 5.6 reports information about the proportion of samples passing different strength criteria for two different value ranges of linear shrinkage and plasticity index. The value range in the far right hand column is the "favourable" category as already defined; the other column describes samples that have the most favourable values of shrinkage (< 4.1 %) and plasticity (< 7.1 %). These values are arbitrary. However, the results show that for a higher strength criterion of 3 MPa, even these most favourable samples have a failure rate of over 40 %. This suggests that under the stabiliser treatments used in this study, a strength criterion of 3 MPa may be a difficult threshold to reach consistently even with the most favourable soils.

		% ≥ 4.0 MPa	**% ≥ 3.5 MPa**	**% ≥ 3.0 MPa**	**% ≥ 2.5 MPa**	**% ≥ 2.0 MPa**	**% ≥ 1.5 MPa**
Linear shrinkage and plasticity index	Favourable (N=109)	19	24	43	62	91	97
	Satisfactory (N=62)	8	15	21	35	65	100
	Unfavourable (N=29)	0	0	0	10	10	38

Table 5.5: Percentages of samples passing different compressive strength criteria within the 3 categories of sample favourability as defined using plasticity index and linear shrinkage.

	L.S. < 4.1 and P.I. < 7.1 (52 samples)	**L.S. < 7.1 and P.I. < 16 (109 samples)**
% ≥ 2.0 MPa	100	91
% ≥ 3.0 MPa	58	42
% ≥ 4.0 MPa	29	19

Table 5.6: Percentage of samples passing different strength criteria for two different value ranges of linear shrinkage and plasticity index.

CHAPTER 6: STABILISATION GUIDELINES

There have been a number of manuals and guides written concerning the stabilisation of earth for the purposes of rammed earth wall construction. These include Wolfskill et al. (1963), CSIRO (1987), and UN (1992), which address not only soil and stabiliser issues but also construction practicalities. This chapter produces a flow chart, with extensive textual annotation, which guides a rammed earth practitioner through the stages of soil selection, preparation, and stabilisation. A flow chart has not previously been developed with respect to soil selection and stabilisation for rammed earth construction.

The novel contributions of the flow chart are that it is based on the results of a new, large data set, which has enabled new quantitative criteria to be applied over a wide range of soil types. It emphasises the importance of obtaining a soil with favourable attributes for stabilisation, and how such soils are identified; and it integrates the data and results of this study with the author's experience and knowledge gained in a 20-year career as a rammed earth builder. In addition, the flow chart enables unnecessary testing to be minimised. The most unfavourable soils (10 % likelihood of successful stabilisation) can be detected after measuring just linear shrinkage (Section 6.2) and plasticity (Section 6.3), and further testing on such soils (for gradation, optimum moisture content, dry density, and compressive strength) is a considerable waste of resources. Another group of soils with about a 1 in 2 likelihood of being successfully stabilised can be dismissed after additionally measuring soil gradation after shrinkage and plasticity (Section 6.4).

The flow chart should be examined with reference to Chapter 3 (for details on methods, which are not repeated in the flow chart stages), Chapter 4 (for data analysis and how the soil property values were established), and Chapter 5 (for discussion of the results of this study with respect to other work). The flow chart addresses the processes of soil selection and stabilisation but not construction. Aspects of rammed earth construction are addressed only as they relate to soil stabilisation including mixing and curing.

6.1 FLOW CHART STAGE 1: SOIL MAPPING AND SAMPLE SITE SELECTION (FIGURE 6.1)

6.1.1 Introduction

The concept of assessing field indices of soil properties is dependent on the indices being more easily observed than the soil characteristics of interest, and upon there being a strong relationship between these indices and the values of the relevant soil properties. If this is not the case, then it is more efficient to measure the soil properties directly rather than to measure the indices. Reasonable correlations are needed between field indices and the soil properties in the range of values of interest (Table 6.1). Some indicators of soil type are surficial (e.g. soil colour, vegetation), and therefore reflect the characteristics of the upper layers more than the lower layers from which soil for earth building is taken. Ideally, however, one should not rely on one or two indicators; a combination of surficial indicators, landscape factors, and subsurface testing is needed to best appraise an area and identify suitable and unsuitable soils.

Before commencing field exploration, any data available from previous subsurface investigations or which can be inferred from geological maps of the area should be studied. Topographical maps and aerial photographs, if available, should be studied. In Australia, much useful information in this regard is available from Australian Geological Survey, Soil Conservation Service, and the Department of Agriculture. The subsequent field visit to potential sites should obtain data on: access conditions for work forces and equipment; surface drainage patterns, seepage, and vegetation characteristics; surface geological features including rock outcrops and landforms, and existing cuts or excavations that may provide information on subsurface conditions; areas of potential instability such as deep deposits of organic soils, slide debris, areas of high ground water table and bedrock outcrops; and, of course, data on the soils themselves in the area of interest.

The cost of the rammed earth wall construction is heavily dependent on the availability on or near the site of appropriate soil. Investigations must therefore be conducted to locate and test locally available soil that may be used for rammed earth wall construction. Soils suitable for use in rammed earth walls are various types of loam with gradations from coarse to fine material in proportions as indicated within this research. These soils can be found in most localities in Australia. However, the composition of these materials may very enormously within the vicinity of a building site and sometimes appropriate material may be located only some distance away. Between 30 and 50 tonnes of processed earth are required to build a 150 m^2 home, and the transport of substantial masses of earthen materials should be kept to a minimum. Rammed earth homes are not always sited directly upon a source of appropriate material. It is therefore very important to locate a nearby raw material source with a minimum of time and expense.

Stabilisation, ramming, and construction cannot commence until detailed testing has shown that the soil or modified soil will produce a structurally sound wall. However, testing takes time, and exhaustive testing of soil from tens or hundreds of sites is clearly not feasible for commercial earth builders. Consequently, this first stage of the flow chart involves using topography, geomorphology, soil appearance, and some preliminary field tests to quickly identify areas of soil that are most likely to provide appropriate material.

Fieldwork for searching for suitable soils for earth building takes place in two phases. The first phase, reconnaissance, involves using physical features - topography, geomorphology and geology, soil colour, and vegetation - to select sample sites. The second phase involves excavation and soil sampling to allow preliminary testing of the subsurface conditions and soil properties. These preliminary tests are much faster and simpler than those (laboratory tests) in subsequent stages of the flow chart. Their purpose is to maximise the probability that the later tests will show that the soil is suitable to be used, and to minimise the probability of needlessly testing an unsuitable soil.

6.1.2 Ideal and non-ideal soils

The objectives of soil mapping and sample site selection are to identify promising soils and quantify their spatial extent. The guiding characteristics of soil suitability for stabilisation are reported in Table 5.3. The most important primary discriminators are linear shrinkage and plasticity index, and the most important secondary discriminator is clay/silt content (Section 5.4.2). Using these variables, it is possible to distinguish between soils that are favourable, satisfactory, and unfavourable for stabilisation.

Any mapping exercise should be able to discriminate between soils that are likely to be suitable for stabilisation and those that are not. A key question then becomes how soils with these qualities may be manifest and identified in terms of field indices and quick field-testing.

6.1.3 Field mapping of soil indicators

A map should be constructed of the area of interest that shows topographic features, geomorphology and geology, and soil colour and vegetation. From this information, a plan for sampling and testing of the soils in potentially more favourable areas can be made.

6.1.3.1 Topography, geomorphology, and geology

Topography, geomorphology, and geology are discussed together, as they need to be taken into consideration together in the field. Disregarding elevation, which is a large-scale topographic control, more localised surface morphometric measures relevant to seeking soils suitable for stabilisation include slope aspect, slope gradient, slope cross-profile shape, slope plan shape, and slope position. Slope aspect is the compass bearing that a slope faces; slope gradient is the angle of the ground surface; slope cross-profile shape refers to whether the slope is linear, convex, or concave in cross-profile; slope plan shape refers to whether the slope is divergent or convergent in cross profile; and slope position describes the position of the slope in the landscape, for example, summit or footslope or plain. Soils vary as a result of these local topographic controls, mainly in response to drainage patterns. Soils in poorly-drained areas, such as footslopes, and areas of concave and convergent topography which concentrate water, are likely to be more clayey and should be avoided in preference to better drained sites and sandier soils.

Features of terrain and geology narrow the task of selecting sites. Outcrops of rock, limestone shelves, and alkali and gypsum sands should all be avoided. Akpokodje (1985) showed that calcareous and gypsiferous soils do not perform satisfactorily when stabilised with lime or cement. Soils, including some lateritic soils, based on deep profiles formed by in situ chemical weathering, may be unsuitable for stabilisation due to the excessive amounts of clay contained in such soils. The additional likelihood of alkaline or saline soil chemistry (Northcote, 1986) would also quickly discount such soils. Hillslopes also should be avoided on account of the likelihood of undesirable colluvial stones (Northcote, 1986), and the logistical challenges of actually excavating the material later by tractor or hoe.

Other geomorphic features are indicators of suitable soils produced as a result of depositional processes and/or shallow weathering. Depositional landforms in Australia may receive material both eroded from deep weathering profiles, and from fresher parent material by detrital reworking (Northcote, 1986), therefore creating a mixture of particle sizes and types. Alluvial fans, built by the deposits of watercourses over time, are excellent places to search. Stream banks, gullies, and road cuts should be examined for profiles or strata of sand, clay, and rock. Such profiles can be sampled directly and indicate the depth at which the material can be relocated in surrounding terrain. Dry stream beds ought to be sampled when available, with particular attention paid to bends and turns in the course opposite the bank most sharply cut, for this is where sandy soils accumulate. Old river terraces are also favourable for locating suitable soils, given their mix of sand, silt, and gravel, particularly low terraces on undulating plains (Northcote, 1986). Any gently rolling to flat land, in general, should be favourable for locating promising soils. Old dune crests and slopes may be worth inspecting if they are sufficiently ancient to have some clay/silt derived from in situ weathering.

Parent rock type is important in terms of the textural properties to which the resultant soil may be predisposed. For example, where a sandstone weathers, the quartzite is very stable and will not undergo much chemical alteration, and therefore will form sandy soils. This would apply to both chemical and mechanical weathering. On the other hand, when a granite undergoes chemical weathering, the feldspar alters to form clay minerals. Where weathering is mainly mechanical the feldspar does not undergo a great degree of chemical breakdown and "granular" soils with less clay develop. As a rule, basic plutonic or intrusive rocks (e.g. dolorite) produce the most clay minerals because their components are crystallised out at temperatures and pressures very different to the surface temperatures and pressures. These components tend to break down chemically more easily to form clay minerals. Sedimentary rocks are derived from weathered parent material so they will have undergone a degree of alteration prior to formation and may already be weathered down to their base components. Sandstone for example may start as granite which is washed clean of the majority of clay and only quartz remains. In contrast, shale is formed in the lowest energy depositional environment in a lake or very slow moving stream by accumulation of clay, and when these rocks weather they tend to produce very clayey soils as a result.

6.1.3.2 Soil colour

The colour of a soil can be a useful indicator of its composition. Materials such as iron, for instance, impart a vivid red or dull rust hue. Soils with high organic content have a black to dark grey colour. Alkali and gypsum give the surrounding terrain a white, chalky appearance. Soils of the latter type absorb a great deal of water and acquire a jelly-like consistency during stabilisation. None of these soils are suited to stabilisation; hence, rusty, black, or white soils should be avoided. Favourable soils tend to be buff coloured, indicating areas of loam, sandy loam, silt loam, or clay loam.

6.1.3.3 Vegetation

There is a direct relation between the nature of soils and the vegetation that stands upon them. Deeper soil profiles tend to have more vegetation and less vegetation in drier areas indicates shallower soil profiles. Shallow soils in Australia are almost inevitably associated with excessive stoniness, and should be avoided. Certain plants prefer wet clayey soils that retain moisture, while others prefer sandy soils with less moisture.

Black soils, heavy in clay and organic compounds, support slow growing trees and a dense understorey of shrubs and grass. This soil is dense and often deep; water stands for long periods after rain. These soils should be avoided as they contain high clay contents, and access into forested or otherwise heavily vegetated areas is difficult logistically. Cracking clay soils, formed under rainfalls of 200-600 mm, support tussock grasslands dominated by Astrebla (Carnahan, 1986), and should be avoided.

Areas of open woodland, containing for example eucalyptus or acacia, may provide suitable sandy soils. Open woodlands with hummock grass understoreys are associated with sandy soils of sandplains and dunefields (Carnahan, 1986). Open scrublands and shrublands, often dominated by Acacia and Casuarina, or Eucalyptus species, with low sclerophyllous shrubs and hummock grass, also tend to develop on sandy soils.

Vegetation also indicates the chemical composition of the soils as some plants prefer saline conditions, alkaline conditions (in areas of limestone), or acidic conditions. Saline soils, or soils with highly alkaline or acidic conditions, are undesirable for stabilisation. Coastal saline soils support particular vegetation communities, often dominated by Sporobolus virginicus (salt-water couch).

6.1.4 Subsurface investigation and soil sampling

Material sampling is a vital step in the flow chart. Poorly chosen samples will jeopardise the subsequent tests of the flow chart, as they may not be representative of the remaining material. In most instances, soils for rammed earth wall constructions are blends of mixtures of particles of many sizes, shapes, and parent material. Considerable variation in these characteristics will be found in samples of apparently similar soils taken from almost adjacent locations. Soil variation occurs on a number of spatial scales, and soil attributes can change abruptly or in a gradual, continuous manner.

Once potentially suitable soils are identified on the basis of the field observations and mapping described above, the area should be sampled, and field tests made. However, the question of how many sample sites should be made over a potential source area is problematic. The spatial pattern of soil attributes is complex, and confident prediction of the spatial distribution of soil properties in unsampled locations on the basis of information from a limited number of sample sites requires complex modelling using geostatistics (e.g. Goovaerts, 1998). Geostatistical techniques are able, amongst other things, to determine the theoretical most efficient sample spacing in order to obtain soil property data without oversampling. This is beyond the scope of this research, and given time and cost constraints on the earth builder, oversampling is unlikely to be a problem.

It is advisable that an alternative, more concise approach be taken in line with observations that suggest that the soil types suitable for stabilisation are associated with particular landforms with clear boundaries. On the scale appropriate to the search for soils for rammed earth construction, which would take place over an area of around 4 km^2, abrupt changes in soil type are the rule rather than the exception; experience shows that it is not uncommon to find several distinct soil types along a two-kilometre search of a potential extraction area. The abrupt and numerous changes in soil characteristics are due to the fact that depositional landforms offer the best locations for favourable soils, and these landforms change abruptly from one form to another rather than in a gradational fashion. The corollary of this is that once an apparently suitable soil is found, and then that soil should be traceable over the extent of the landform upon which it is found, for example, a river terrace or old river bend. The question of sample spacing across a potential source soil is practically answered by the following method. Given that the tonnage of natural soil required to build one house is 30-50 tonnes, what area of soil is needed to provide that soil? Assuming that the layer of suitable soil within the soil profile is 0.7 m thick, and that the bulk density of a loamy soil is about 1.3 t/m^3 (Brady, 1974), then the area is estimated by dividing the required tonnage by the product of soil layer thickness and bulk density. The area calculated using the above assumptions equals 35-60 m^2, or say about 8 m by 8 m. A suitable plan therefore might be as follows. Several potential areas within the 4 km^2 search area are sampled and tested in turn. Once suitable results have been found from one sample site, then additional sample sites should be located at the corners of an 8 m by 8 m square surrounding the first sample site, giving 5 sample sites in all for the candidate soil. Field-tests of the soil at the 5 sample sites will reveal whether the soil properties are reasonably uniform and whether the soil contained within the square is sufficiently suitable for samples to be taken for further testing in the laboratory (Sections 6.2-6.7).

The subsurface exploration program should obtain all the information that could influence the design and the stability of the final rammed earth wall structure. Such information must include:

- precise location(s) of source site(s)
- estimation of sufficient material for the project
- analysis of characteristics of the material
- assessment of the uniformity of the material within the source area
- accessibility of material, including how much overburden must be removed

A soil sampling is performed as follows. At each ascertained sample site, clear an area of approximately 1 m^2 and dig a pit through the soil using a spade. Extraction of soil for testing (and for use for stabilisation) is made from the layer of soil below significant organic accumulation as evidenced by plant matter in various stages of decay (e.g. leaves, roots, humus) and above unweathered parent material. This part of the soil is weathered, generally contains a range of particles sizes and negligible organic matter. The depth to the top of this layer, and its thickness, should be measured. The field tests (Section 6.1.5) should be made on the spot.

If the field tests at a sample site indicate that the soil is likely to be favourable, approximately 25 kg of material must be removed from the soil layer as described above to be taken to the laboratory for testing as described in Sections 6.2 to 6.7. A 20-litre bucket makes an excellent container for transport and storage. A stake should be driven next to the sample hole and given an identification number; the same identification number should be attached to the container. Sample identification is very important given that a number of samples will be collected.

On the basis of the field tests and the landforms associated with the soil types found, the spatial extent of the potential source soil should be able to be (roughly) estimated. Obviously, there are errors involved in the estimation, deriving from uncertainties in the soil properties measured, in the assumed uniformity of the soil properties over the area considered, and the in the actual lateral extent of the soil. However, from the mapped information and field tests at sample sites, the following should be able to be calculated:

- The estimated potential tonnage of suitable soil, calculated by multiplying the estimated area of soil (in m^2) by the average thickness (in m) of the layer of suitable soil as found at the test pits, and multiplying this by the bulk density of this soil (in t/m^3) to obtain a value in tonnes.
- The estimated overburden to be removed, calculated by multiplying the estimated area of soil (in m^2) by the average depth (in m) to the top of the layer of suitable soil as found at the test pits, and multiplying this by the bulk density of the overlying soil (in t/m^3) to obtain a value in tonnes.

6.1.5 Field testing of soils

Each soil sample should be visually examined and appropriate quick tests performed to allow the soil to be characterised. The objective is to infer from the tests whether a soil is likely to be suitable and therefore whether a sample should be taken and its properties confirmed in laboratory testing. The field tests address the aspects of organic matter, particle size distribution, and plasticity. It is advised that the way for an earth builder to use the field-tests efficiently is to first prepare soils with known and variable organic content, texture, and plasticity. The tests should then be performed to associate the results of the tests with soils of particular attributes. For example, a series of soils could be artificially made with clay/silt increasing from zero upwards in 5 % steps for the textural field-tests, or with plasticity index increasing in 5 % steps for the plasticity tests. In this way, the earth builder can "correlate" the feel and behaviour of the known soils with their attributes, and through this experience estimate with some accuracy the organic content, texture, and plasticity properties of unknown soils. Details of the tests outlined below can be found in Wolfskill et al. (1963) or UN (1992). A good summary of manual field-test results for particular soils is given in Wolfskill et al. (1963, pp24-26).

6.1.5.1 Test for organic matter

The presence of organic material in the soil is undesirable, and soils with more than 1 % organic matter in the source layer should be rejected (e.g. UN, 1992). Very few inorganic soil components have any aromatic properties whatsoever, and therefore a soil that has no or little detectable odour will contain negligible organic material. If the soil smells musty or fetid, then the soil contains appreciable proportions of organic matter and should not be used.

6.1.5.2 Tests for texture and plasticity

Ball test

The ball test gives an indication as to the amount of clay/silt in the soil. An air-dry soil specimen no larger than a golf ball should be squeezed tightly in one fist, and the friability or coherence of the resultant ball should be observed under stresses manually applied, as described below. The test should be repeated with increasing increments of moisture until a drop of water placed on the squeezed ball is not absorbed within 30 seconds.

Manual manipulation of the ball should progress from the delicate to the severe. A ball that crumbles when lightly touched denotes a poorly-graded granular soil with an insignificant clay/silt fraction. A ball of well graded granular soil without a perceptible silt/clay fraction will crumble under light pressure but not as easily as a ball of poorly graded sand. When the squeezed ball at higher moisture contents leaves the fingers stained or muddy, it contains an appreciable silt/clay fraction. By opening the fingers slowly, after squeezing, the tendency toward stickiness can be appraised. When the maximum stickiness occurs at a water content less than the drop-absorptive upper limit of the ball test, the volume of the clay/silt fraction is less than the porosity of the squeezed granular fraction. This means that there is insufficient silt/clay and the soil should probably be rejected. As the stickiness-optimum approaches the drop-absorption limit the silt/clay volume approaches the volume of the granular pores.

Thread test for consistency

An olive-sized lump of soil should be taken and enough water added so that it can be moulded in the hand without being sticky. Roll the soil into a thread on a flat surface; if the thread breaks before its diameter is reduced to 3 mm, the soil is too dry and water should be added. The thread should break when its diameter is 3 mm. The results of the test are: 1. No thread able to be made - the soil contains no clay; 2. Weak thread - soil has a lot of silt and sand and little clay (favourable); 3. Medium strength thread - higher clay content (favourable); and 4. Very tough thread - highest clay content (not favourable).

Ribbon test for cohesion

This test gives similar information as the thread test, but is a good check on the latter. An amount of soil sufficient to produce a cigar-sized roll is made into a roll measuring about 12 mm diameter. The soil should be at a moisture content short of making the soil sticky. The roll is flattened to form a ribbon of between 3 and 6 mm in width as long as possible; the length of the ribbon prior to breaking is measured. The test results are: 1. No ribbon able to be formed - negligible amount of clay; 2. Short ribbon(s) (5-10 cm) - low to medium clay content (favourable); 3. Long ribbon (25-30 cm) - high clay content (unfavourable).

Sedimentation test

Another test for texture that could be used is a quick sedimentation test, for which a glass cylinder (e.g. a large jam jar) is used. The jar is quarter-filled with soil and then water is added to near the top and the mixture is allowed to soak for 15 minutes. The jar is then shaken vigorously to ensure all aggregates are broken into individual particles, and the particles allowed to settle. After one hour, the respective thicknesses of the layers of material can be measured, starting from gravel at the bottom to clay at the top. These measurements are converted to percentages of soil of gravel, sand, and silt/clay.

Visual test for gravel

The proportion of gravel in the soil could probably also be assessed visually using standard diagrams that show the percentage cover over an area, often at 2 %, 5 %, 10 %, 15 %, 20 % and upwards at 10 % intervals. Part of the cutting in the soil pit should be inspected, and the soil face compared with the diagrams to determine the percentage of area of the soil covered by particles between 2.4 and 20 mm. This percentage is an estimate of the gravel content of the soil.

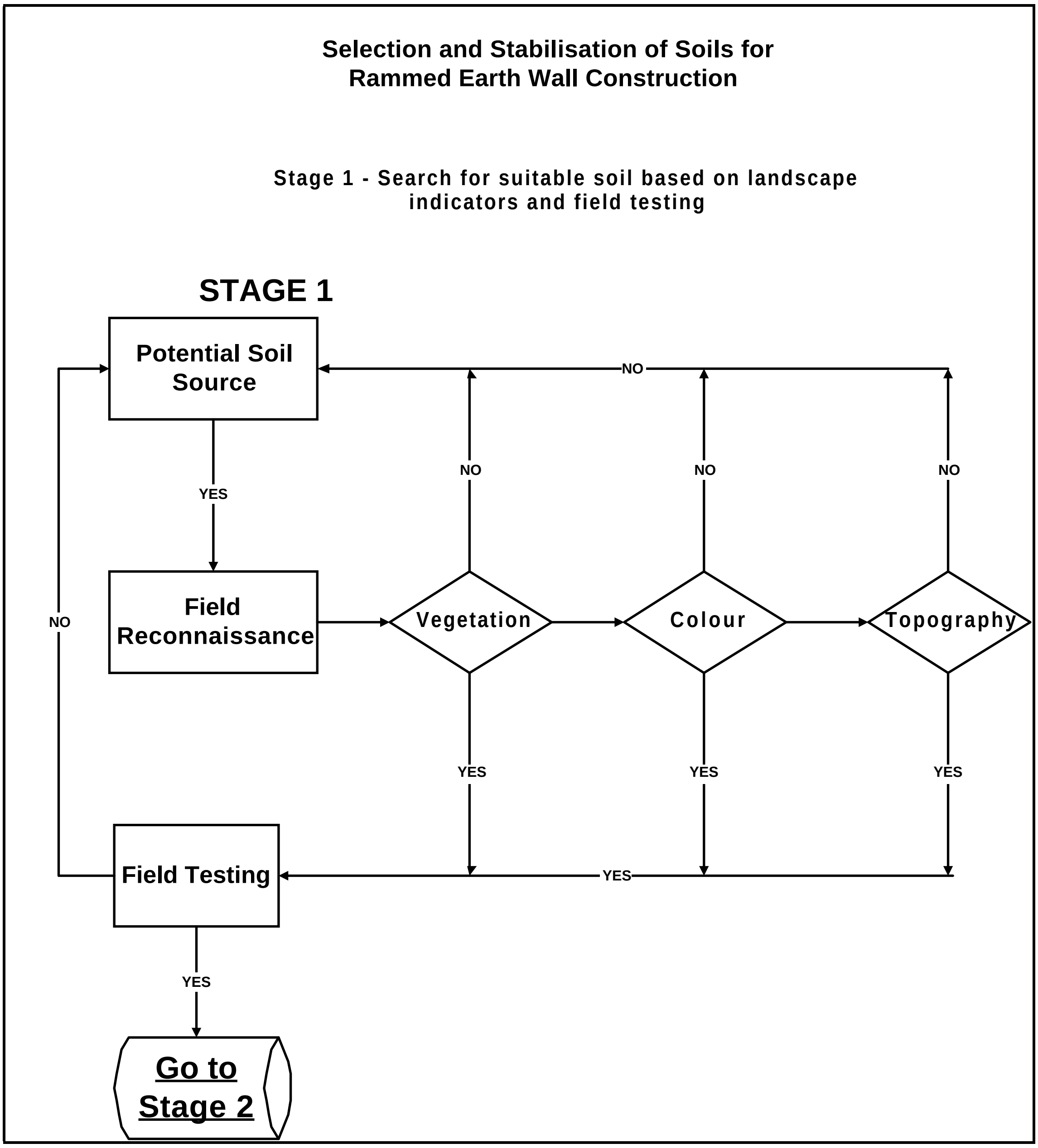


Figure 6.1: Selection of sample sites by field reconnaissance and identification of potentially suitable soils using field-tests.

6.2 FLOW CHART STAGE 2: DETERMINATION OF SOIL SHRINKAGE (FIGURE 6.2)

Favourable category: linear shrinkage < 7.1 %.

Unfavourable category: linear shrinkage > 13.0 %.

Satisfactory category: linear shrinkage 7.1-13.0 %.

Linear shrinkage should be measured using the procedure described in Section 3.3.4 in Chapter 3. Linear shrinkage is one of the two best soil property predictors of the suitability of a soil for stabilisation (Table 5.3 and Section 5.4.2), and great care should therefore be taken in determining an accurate value for this variable.

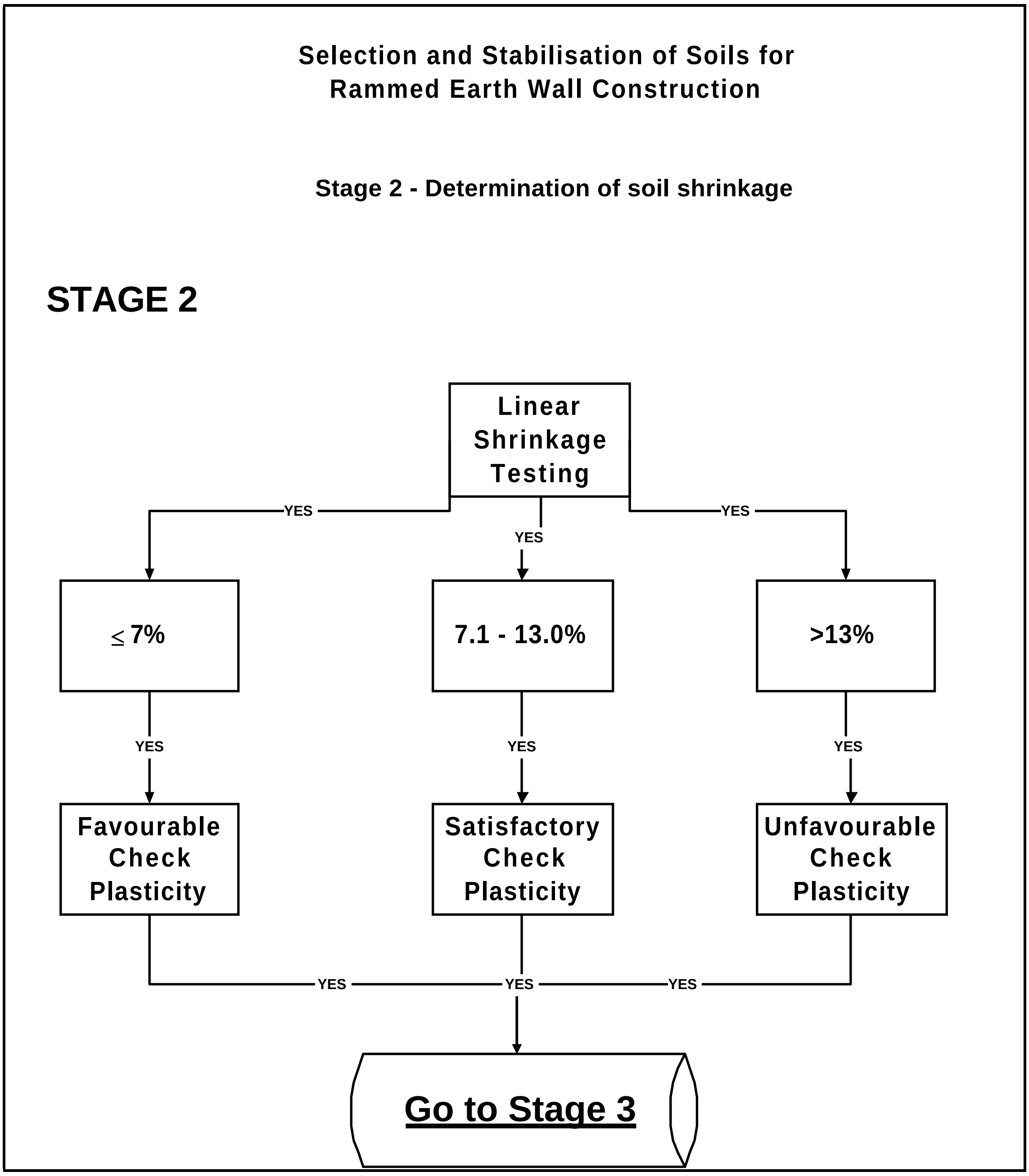


Figure 6.2: Determination of soil linear shrinkage.

6.3 FLOW CHART STAGE 3: DETERMINATION OF PLASTICITY (FIGURE 6.3)

Favourable category: plasticity index < 16 %; liquid limit < 36 %.
Unfavourable category: plasticity index > 30 %; liquid limit > 45 %.
Satisfactory category: plasticity index 15-30 %; liquid limit 36-45 %.

All soils, having been tested for linear shrinkage, should also be tested for plasticity (whether or not shrinkage values were favourable) to determine whether shrinkage and plasticity results are in accord with one another. Plasticity index is one of the two primary discriminators of soil suitability for stabilisation, and therefore great care should be taken in its accurate determination. Soils with favourable values of shrinkage and plasticity determined in stages 2 and 3 of the flow chart should certainly progress to the later stages; satisfactory soils should also progress and be tested for texture (Stage 4) to further assess soil suitability; however, soils with unfavourable values of shrinkage and plasticity should be rejected, as the likelihood of successfully stabilising such soils (Section 5.4.2.1) is so low (Figure 6.3).

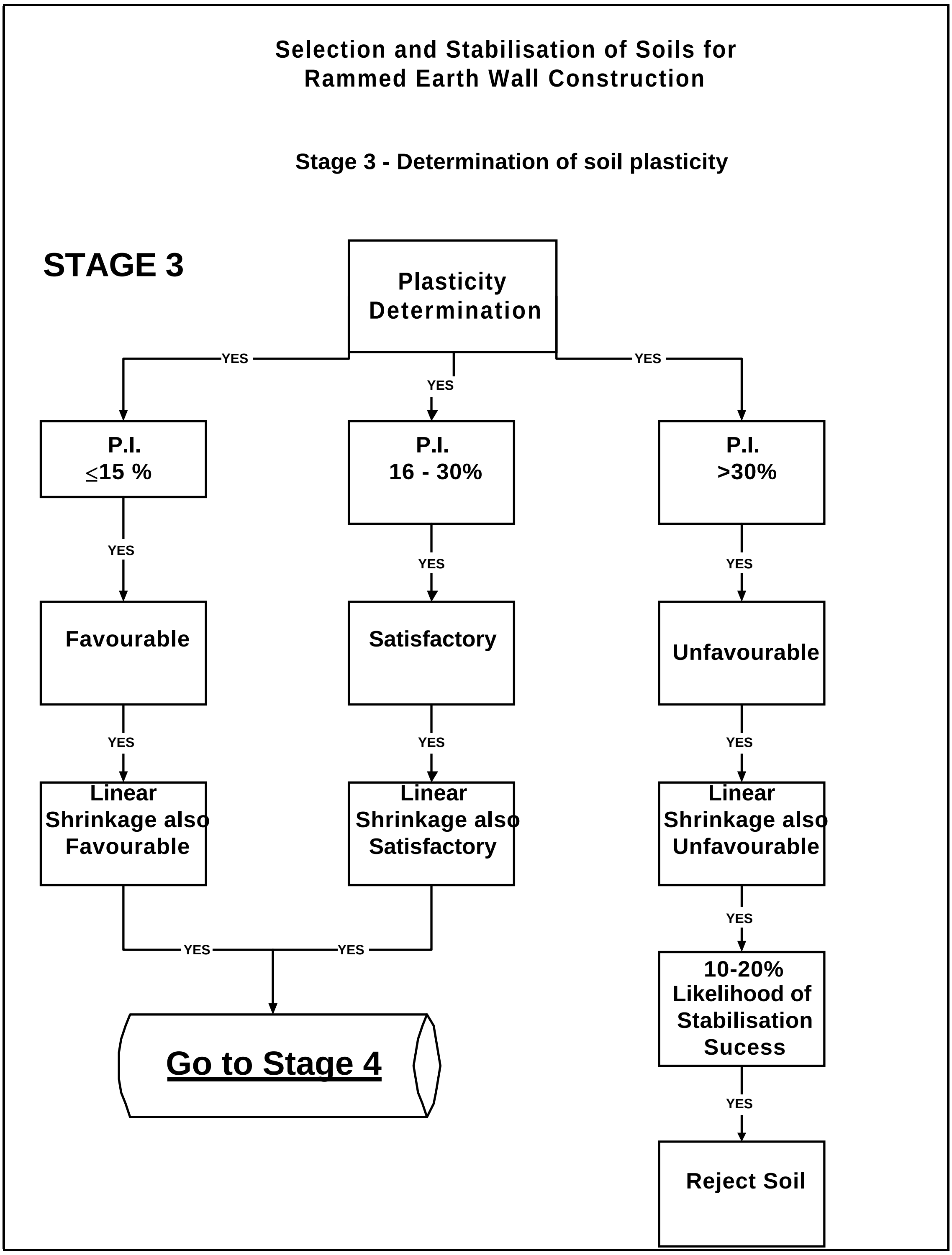


Figure 6.3: Determination of soil plasticity.

6.4 FLOW CHART STAGE 4: ASSESSMENT OF SOIL PARTICLE SIZE DISTRIBUTION (FIGURE 6.4)

Favourable category: clay/silt 21-35 %; or sand 30-65 %; or gravel 3-5 %.
Unfavourable category: clay/silt < 21 or > 45 %; or sand > 75 %; or gravel < 3 %.
Satisfactory category: clay/silt 36-45 %; or sand 66-75 %; or gravel 3-12 %.

The textural properties are subordinate to linear shrinkage and plasticity as primary discriminators of soil suitability for stabilisation. Therefore, if unfavourable values of shrinkage and plasticity have been determined in flowchart stages 2 and 3, then the soil should already have been dismissed and particle size measurements, and density and strength tests, would not need to be made. Soils categorised as satisfactory using shrinkage and plasticity measurements should have progressed to Stage 4 and particle size determinations should be made on these soils to decide on the basis of texture whether the soil will be suitable. Soils with favourable plasticity but satisfactory shrinkage, and vice versa, should also have progressed to Stage 4 (Section 5.4.2.2).

The textural properties are very important as secondary discriminators once shrinkage and plasticity are known. Soils with satisfactory or unfavourable values of sand (in excess of 60 %) can be tolerated if shrinkage and plasticity values are favourable (Section 5.4.2.2). However, when soil shrinkage and plasticity are favourable, unfavourable or satisfactory values of clay/silt content should if possible be avoided, because soils with otherwise favourable values of linear shrinkage and plasticity index will have an increased likelihood of producing weak stabilised samples (Table 5.4 and Section 5.4.2.2). If soil shrinkage and plasticity are satisfactory, then sand content should be used to further determine soil suitability (Section 5.4.2.3). For soils in this satisfactory category, the sand content should be < 60 % to allow a decent likelihood of stabilisation success.

The results of this study show that knowledge of the gravel (2.36-19 mm size particles), sand (0.075-2.36 mm particles), and clay/silt (< 0.075 mm size particles) proportions of the soil are able to give an accurate determination of the textural qualities of a soil that are favourable or unfavourable to successful stabilisation, and further breakdown of soil gradation is probably unwarranted. The procedure for sieve analysis should follow that described in Section 3.3.1, but using just 3 sieves with openings 19 mm, 2.36 mm, and 0.075 mm. The 2.36 mm, and 0.075 mm sieves are used to calculate the proportion of gravel, sand, and clay/silt in the soil. If the soil for testing contains stones greater than or equal to 2 cm in size, these should be removed prior to determining the particle size distribution; therefore, the particle size distribution effectively is that of the sub-2cm fraction of the soil. This equates with what would happen in the field during excavating the soil by tractor or rotary hoe, during which stones of around 2 cm or larger would be picked off, or screened, and thrown away. Soils with particle sizes > 2 cm are unsuitable for stabilisation, and therefore soils with appreciable amounts of stones should not be considered as removal of the stones will be too difficult. The use of just 3 sieves will not detect gap-graded soils that have deficiencies of particles of certain size within the three fractions (a comprehensive particle size determination would be needed to detect such soils). This is not considered a significant disadvantage given the relative rarity of gap-graded soils and the fact that shrinkage and plasticity are more important discriminators than textural properties in the assessment of soil suitability for stabilisation.

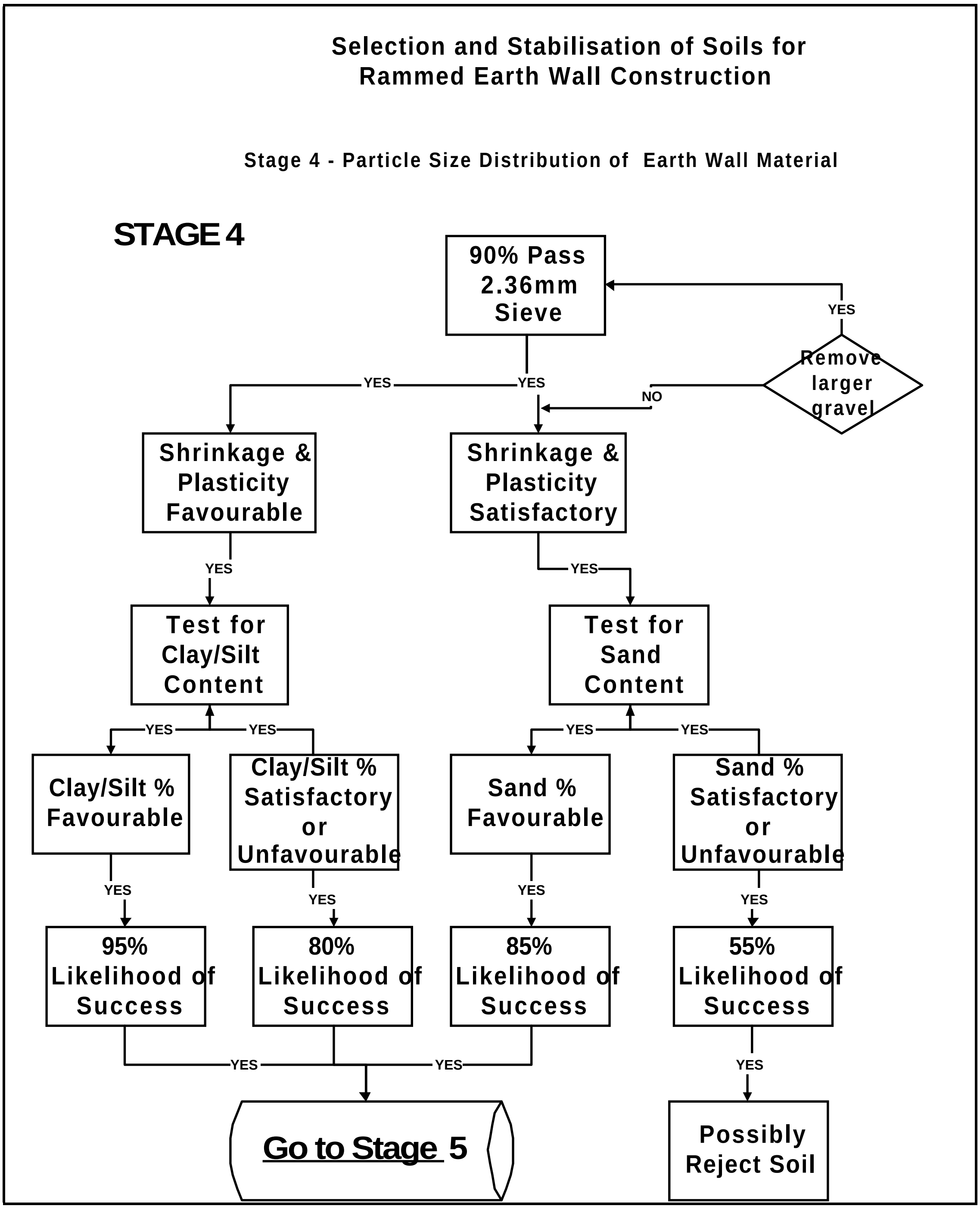


Figure 6.4: Determination of soil particle size distribution.

6.5 FLOW CHART STAGE 5: SELECTION OF THE STABILISERS (FIGURE 6.5)

The results of this study suggest that the attributes of the soil are more significant to stabilisation success (as measured by compressive strength and density) than the type of stabiliser (cement or lime, with or without asphalt) and percentage of stabiliser (2-6 %) used. Given the results of Chapters 4 and 5, the following discussion applies to the selection of a stabiliser. Particular reference for this stage of the flow chart should be made to Section 5.1.2 in Chapter 5.

6.5.1 Cement and lime stabilisation

Much has already been written about compressive strength, density, and stabilisers in Chapters 4 and 5, and is not repeated here apart from a short summary. In this study, lime and cement levels of 5-6 % achieved the highest mean compressive strengths having adjusted for the effects of different soils. There is no evidence to suggest that lime is more effective than cement in stabilising soils richer in clay/silt content. Nor is there any evidence that cement is substantially more effective than lime in stabilising sandy soils, although soils with negligible clay/silt contents of less than 5 % were not tested here. The percentage of stabiliser does not influence maximum density, and therefore density should not be a consideration in the quantity of stabiliser to use. A broad-spectrum stabiliser treatment recommendation based on strength is to treat a soil with 5-6 % cement or lime in the absence of more considered alternatives. Treating soils with lesser amounts of stabiliser down to 2 % will still stabilise some soils well but with increasing risk of failing the strength criterion, depending on the degree of favourability of soil characteristics. However, Figure 6.5 shows that stabiliser decisions should be made on the basis of the shrinkage and plasticity of the soil to be stabilised.

6.5.1.1 Using soil shrinkage and plasticity values to guide stabiliser treatment

Soil characteristics very strongly influence the compressive strength of the stabilised sample. As referred to in Section 5.1.2 of Chapter 5, if the main contributor to compressive strength variation is variation in values of soil properties, then a corollary is that similar strengths could be achieved for different soil types by adjusting the stabiliser quantity according to the favourability of the soil. Therefore, the best soils could be treated with quantities of cement or lime less than 6 % by dry weight of soil, to reduce costs, whilst retaining a strength that will pass the 2 MPa threshold. As demonstrated in Section 5.3 and Figures 4.45 and 4.46, soils that have lower plasticity index (< 16 %) and lower shrinkage (< 7.1 %) produce stronger samples than those soils that do not. Therefore, soils within these favourable ranges could be treated with lower levels of stabiliser, with the lowest levels being applied to the very best soils, perhaps with shrinkage of less than or equal to 3.0 % and plasticity index of less than or equal to 4 %. Soils within unfavourable value ranges of these properties (plasticity index > 30 %; linear shrinkage > 13.0 %) should not be used for construction. Satisfactory soils, defined as being in the range plasticity index 16-30 % and linear shrinkage 7.1-13.0 %, can be worked with, providing that sand content is also favourable (Figure 6.4), or if no better alternative is available. These soils should be stabilised using 6 % stabiliser. Using plasticity and shrinkage as properties to guide the choice of stabiliser treatment is the best option as these properties are the best predictors of both successful and unsuccessful stabilisation (Section 5.4). The properties work together rather well, as the upper limit of most favourable linear shrinkage of 7.0 % equates to the upper limit of most favourable plasticity index of 15 % for the samples in this study, and the lower limits of unfavourability equate at 13.0 % for shrinkage and 30 % for plasticity index (Table 5.3). Therefore, Stage 5 of the flow chart considers compressive strength requirements by reference to plasticity and shrinkage as the best indicators of the predisposition of a soil to stabilisation, and therefore the best indicators of the stabiliser treatment in terms of achieving satisfactory strength.

After consideration of the stabilised strength, attention could also be made to other aspects of stabilisation, including shrinkage reduction, plasticity reduction, and workability and cost, in choosing whether lime, or cement, or both, should be used and in what amounts.

6.5.2 Shrinkage and plasticity considerations

6.5.2.1 Shrinkage reduction

Shrinkage on curing of cement-stabilised soil depends on cement content, soil type, water content, degree of compaction with rammers, and curing speed (dependent primarily on humidity and temperature). The linear shrinkage of stabilised soil decreases with increasing levels of cement and lime (Spangler & Patel, 1949; Akpokodje, 1985), and therefore the potential for cracking is reduced by using these stabilisers. Shrinkage cracks should be considered inevitable in soil-cement stabilisation, and are generally from three to six millimetres wide at a spacing of three to six metres. CSIRO (1987) specify that shrinkage cracks in rammed earth walls should not be longer than 75 mm, nor wider than 3 mm, nor deeper than 5 mm. Therefore, the soil and stabiliser used should be able to meet these shrinkage criteria, although no guidelines have yet been generated by CSIRO which, when followed, would result in stabilised material having such shrinkage properties.

Reference to the work of Akpokodje (1985) should be made concerning the amount of cement to use to achieve a satisfactory stabilised shrinkage value given the linear shrinkage of the natural soil. As recommended in this study, a linear shrinkage of less than or equal to 7.0 % is most favourable for successful stabilisation (as defined using compressive strength). To achieve a stabilised linear shrinkage of 3 %, a common shrinkage criterion, a soil with natural shrinkage of 7 % would require 6 % of cement by dry weight of soil, which fits in rather well with the quantity recommended in Section 6.5.1 regarding compressive strength requirements. As lime is more effective than cement at reducing shrinkage (Spangler & Patel, 1949), then lime should therefore be used for soils with higher natural shrinkage. A soil of natural shrinkage of 4 % would require just 2 % cement to reduce the stabilised shrinkage to 3 %.

6.5.2.2 Plasticity reduction

A reduction in plasticity (a reduced plastic limit and plasticity index) is effected by lime treatment. The addition of 2-3 % of quicklime to a soil quickly reduces plasticity of the soil by hydration (dries the soil), and breaks up lumps (UN, 1992). Soils with high initial plasticity indexes and clay contents require greater quantities of lime.

Spangler & Patel (1949) reported the results of an experimental study of the effect of various levels of lime (CaO) and portland cement on the properties of a clay soil. Adding lime of 4 % by soil dry mass reduced the natural soil liquid limit from 69 % to 54 %, and the plasticity index from 42 % to 10 %. In comparison, adding 4 % cement decreased the liquid limit to 55 % but the plasticity index to 20 %. Lower quantities of lime, between 1-3 %, were more effective than cement at reducing both liquid limit and plasticity index. Osula (1996) came to the same conclusions - that both stabilisers reduce plasticity, but lime more so than cement. As natural soil samples with low plasticity indexes produce stronger stabilised samples than highly plastic soils, the inference is that reducing the plasticity with lime prior to stabilising with cement should help increase strength.

6.5.3 Other considerations

6.5.3.1 Workability

Addition of lime assists in workability of clay-rich soils. The improved level of workability expedites subsequent manipulation and ramming of the treated soil in rammed earth wall construction (UN, 1992).

6.5.3.2 Waterproofness

Although asphalt has been viewed as being better suited as a stabiliser for soils with high clay contents (CSIRO, 1987; UN, 1992), the results of this study suggest that asphalt can be used successfully in combination with either lime and/or cement on a wide range of soil types. The use of asphalt should be viewed as a broad-spectrum waterproofing agent, rather than an additive that imparts strength to stabilised soil, when used in combination with either lime or cement.

6.5.3.3 Cost

Cost may be a factor in choosing between cement and lime or choosing the proportions of each to be used for stabilisation, and this varies from country to country. At the time of writing, cement and lime are very similar in price in Australia, at around $8 per 40 kg bag, although in the mid-1990s, lime was at least 25 % cheaper. In South Africa, lime is currently about $8 per bag (20 cents/kg), but cement is cheaper at $5 per bag (12.5 cents/kg). However, a difference of 7.5 cents/kg would make a difference of only $112 in the cost of a typical 150 m^2 rammed earth dwelling, assuming 30 tonnes of soil is needed for such a construction and 5 % stabiliser (= 1,500 kg) is used.

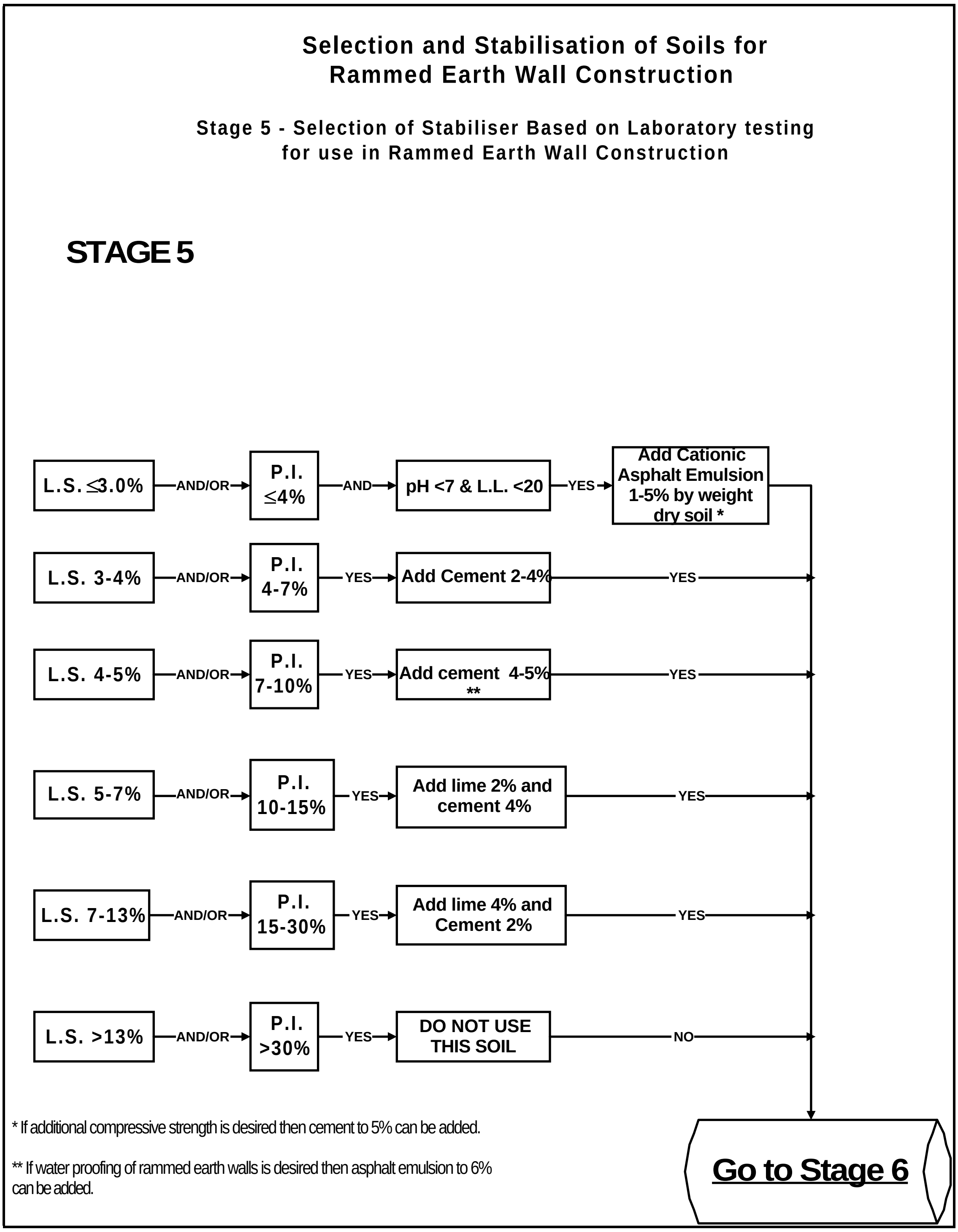


Figure 6.5: Determination of stabiliser type and quantity for soils of different shrinkage and plasticity.

6.6 FLOW CHART STAGE 6: OPTIMUM MOISTURE CONTENT AND MAXIMUM DRY DENSITY DETERMINATION (FIGURE 6.6)

Favourable category: optimum moisture content 5.0-10.0 %.
Unfavourable category: optimum moisture content >13.0 %.
Satisfactory category: optimum moisture content 10.1-13.0 %.

6.6.1 Density - an unsatisfactory indicator of strength

The primary objective of determining optimum moisture content is to compact the material to achieve the maximum density possible under the compaction specified. As pointed out in Section 5.4 of Chapter 5, density is not as good an indicator of strength as might be imagined, and a density criterion (1.80 t/m^3 in this study) should probably not be used as a filter to screen out samples that are apparently unsuitable (i.e., those samples < 1.80 t/m^3). The data pattern shown in Figure 5.12 demonstrates that this is probably true not only for a density criterion value of 1.80 t/m^3, but also any density criterion value between 1.6 and 2.1 t/m^3 on account of the scatter in the data and the distribution of data points within the four quadrants of Figure 5.12 and as discussed in Section 5.4. Moreover, the use of soil shrinkage (Figure 6.2), plasticity (Figure 6.3) and texture (Figure 6.4) offers a superior system of assessing soil suitability for stabilisation.

The result of the compressive strength test (Section 6.7) should take precedence over the density test as strength is the favoured measure of stabilisation outcome and is given greater importance in building regulations and material testing specifications.

6.6.2 Compactive method and effort

Compactive method and effort can vary the optimum moisture content of any soil. As the methods of compacting soils in the field and in the laboratory are different, it is important the testing reflect what happens in the field. For example, in the field, the modified compaction of a lean silty clay has moisture optimums ranging from about 80% to 85% saturation using the standard Proctor test. The optimums range from 84 to 92%

under laboratory compaction depending on the duration of ramming, 0.4 - 3.0 seconds, and the pressure exerted.

Soil density varies with compaction effort and method as well as with moisture content and the particular properties of the soil. The primary requirements for the density testing are that the compactive effort and method are specified, and that these reflect the compaction to be performed during actual construction.

6.6.3 Moisture content

Each rammed earth material has an optimum moisture content corresponding to the maximum density attainable under a certain compactive effort. Densities achieved by the same effort exerted on the same soil containing either more or less than optimum moisture are less than maximum unit weight. If a certain degree of compaction (percent of maximum density) is desired, it is usually better to compact the material with a water content on the wet side of optimum than on the dry side. The material will then be less vulnerable to volume change when water subsequently becomes available via infiltration, condensation, or capillary action. However, overcompaction should be avoided.

An additional aspect to moisture content concerns shrinkage of the curing stabilised material, which could potentially cause shrinkage cracks. The wetter the mix, the more likely will shrinkage cracking occur. Therefore, soils with lower optimum moisture contents are more favourable not only in terms of final stabilised strength, but also in terms of their lower predisposition to shrinkage cracking.

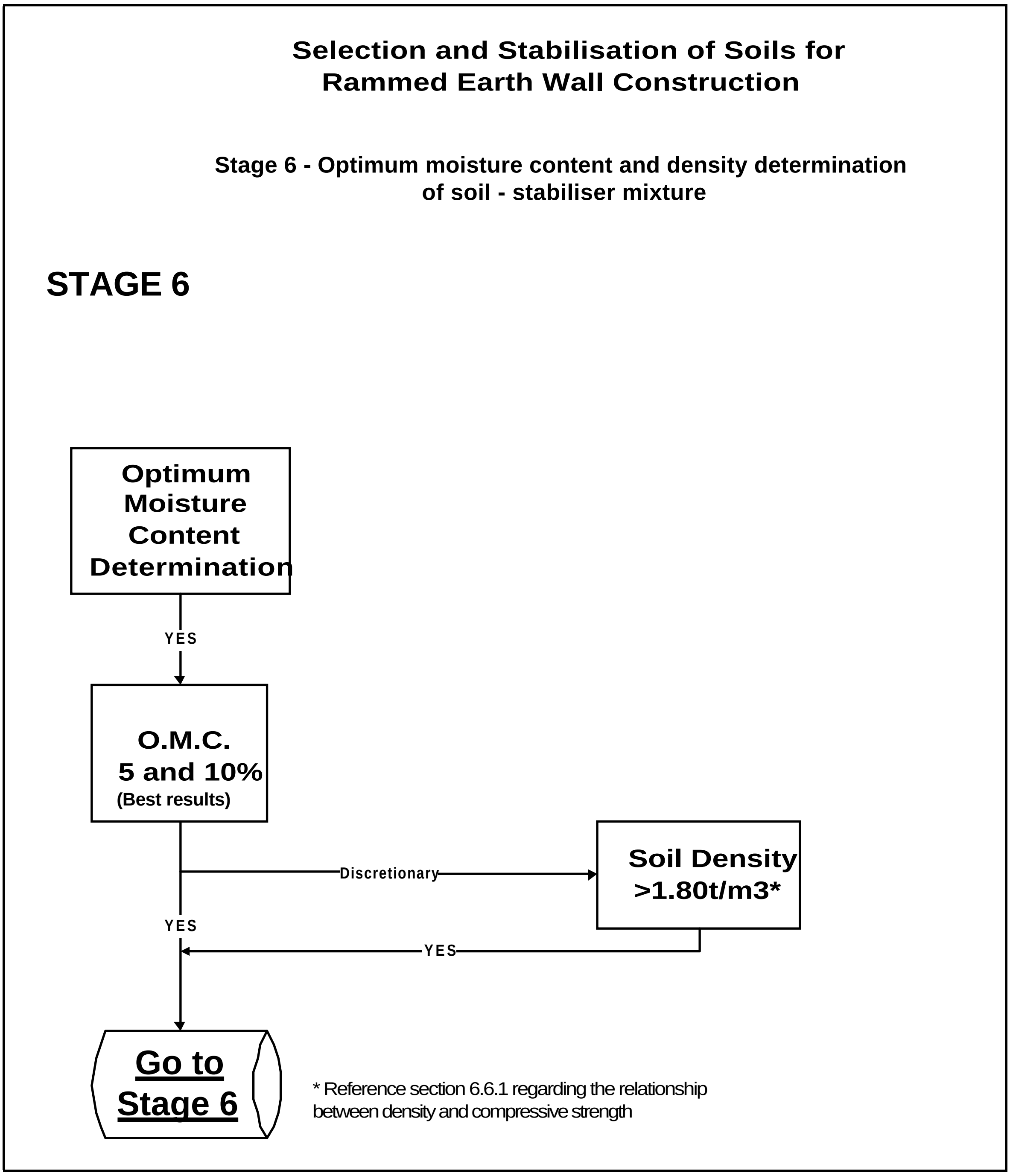


Figure 6.6: Determination of optimum moisture content and maximum dry density.

6.7 FLOW CHART STAGE 7: COMPRESSIVE STRENGTH TESTING (FIGURE 6.7)

The saturated, unconfined compressive strength of a sample, as performed using the method described in Section 3.3.6 must be greater than 2.0 MPa to pass the strength criterion. Judgement may have to be made concerning the number of samples taken from the field that fail or pass the criterion value. For example, if 5 samples are taken from the field, and the strength test results are mixed with 3 determinations passing but 2 failing, then the earth builder needs to consider whether this is good enough to proceed. It also needs to be remembered that the soil will be mixed thoroughly prior to construction, and therefore portions of slightly less favourable soil will be mixed with portions of slightly more favourable soil, so that the material finally used for the walls will be an overall average of the extracted soil. Guidance could be taken from the recommendations concerning strength-testing of CSIRO (1987) and New Mexico Construction Bureau (1991). CSIRO (1987) specify that for a soil to be used for rammed earth construction, stabilised specimens must have an average (or "characteristic") compressive strength of at least 2 MPa. This would enable an average value to be sufficient, so that in the above example, if the average value was greater than 2 MPa, the soil area from which the samples were taken could be excavated and used with some confidence. An alternative approach would utilise a combined average and pass/fail approach (New Mexico Construction Bureau, 1991). The New Mexico Rammed Earth Building Code specifies the test sampling of a completed wall. The test sampling involves a 5/25,000 ratio, meaning that 5 sample units should be tested per 25,000 units of material comprising the wall. Importantly, the test units must have an average compressive strength of 2 MPa, and one sample out of five may have a compressive strength of not less than 1.7 MPa. These regulations refer to the actual built rammed wall. However, it may be that similarly specific recommendations need to be made regarding the individual strengths and average strengths of soil samples taken from the field, in terms of assessing their suitability for rammed earth walls, particularly if the strength tests of these samples show inconsistent results.

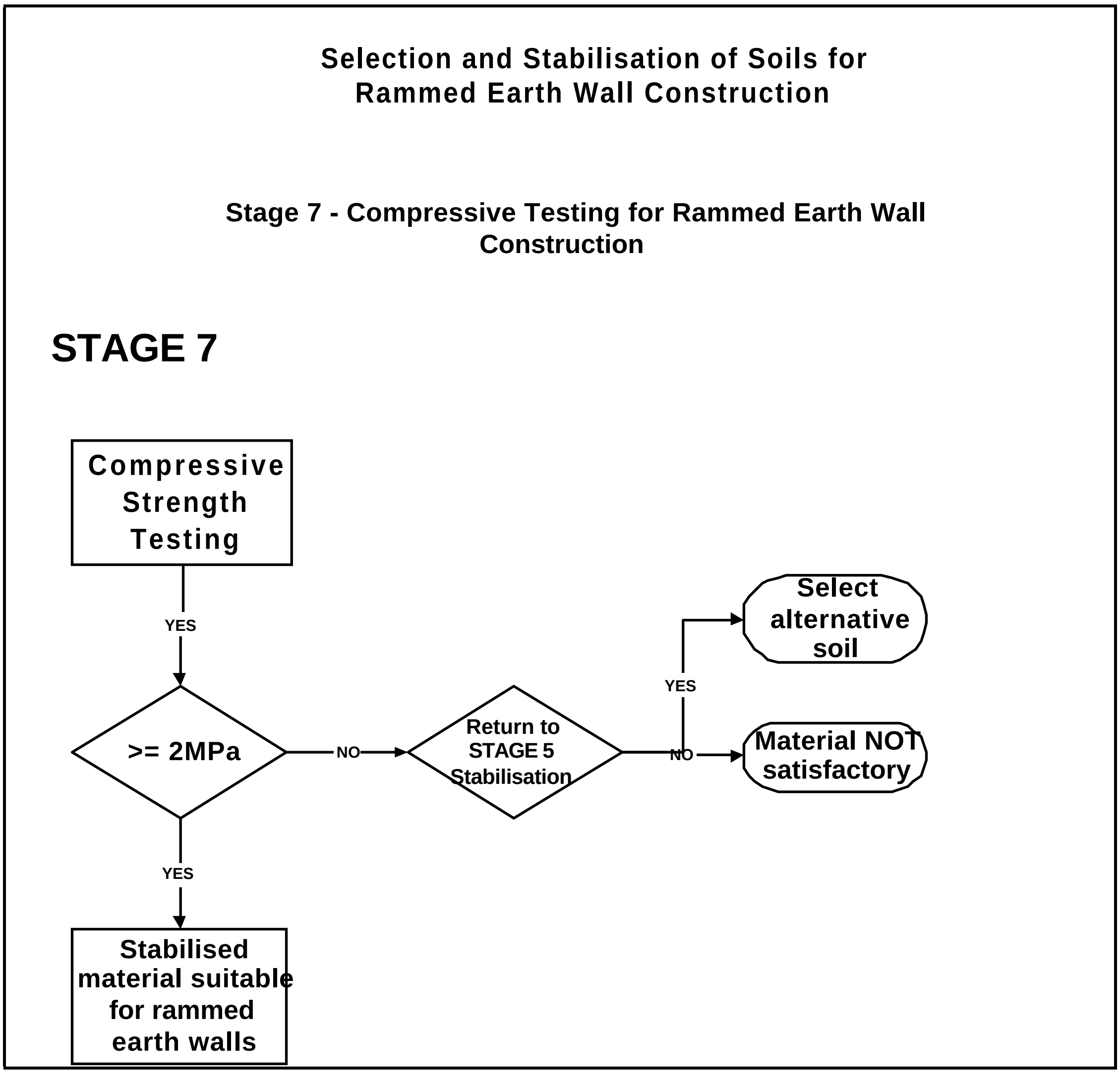


Figure 6.7: Determination of unconfined compressive strength.

6.8 FLOW CHART STAGE 8: ENVIRONMENTAL CONSIDERATIONS

The actual conditions existing at the time of ramming walls are not always accurately foreseen when preliminary tests are conducted. Environmental factors such as temperature, wind, humidity and precipitation need to be taken into account as they influence various aspects of stabilisation.

6.8.1 Temperature

An example of temperature influence on a method is seen in the generally accepted and commonly specified minimum permissible ambient air temperatures for mixing and ramming bituminous stabilised soils. A minimum of $20^{o}C$ for ramming and $25\text{-}28^{o}C$ for mixing is recommended from experience. Rammed earth stabilisation under extreme thermal conditions, such as in a frost region or in a low-latitude desert, would impose obvious limitations.

Secondary or tertiary additives used to retard, hasten, moderate, or intensify the interaction between other stabilising agents or between soil and stabilisers are often sensitive to temperatures. Proportions successful in one climate may or may not function too rapidly in another climate. For example, a catalyst system for polymarisation of calcium acrylate which induces reaction in 60 minutes at $19^{o}C$ would have the same effect in only 4 minutes at 41 ^{o}C. Part of this effect is due to the influence of temperature on water, and therefore on water solutions, suspensions, and emulsions. Also, as a general rule, the rate of chemical reactions is doubled for every temperature increase of $10^{o}C$.

Certain principles regarding temperature effects will help to evaluate the relative merits of different stabilisation methods applied to various soil types:

- Water expands about 9% on freezing at $0^{o}C$.
- The volume of air voids in a soil decreases with decreasing temperature.
- Soil water and soil water vapour migrate from warmer to colder horizons; some of the vapour condenses. If freezing temperatures occur intermittently, large ice lenses are formed at the cost of previously formed small ice lenses.
- The freezing point of water solutions of $CaCl_2$ or NaCl are lower than that of pure water or ocean water of lesser salt concentration.
- The viscosity of water approximately doubles between $25^{o}C$ and $0^{o}C$; with decreasing temperature, viscosities of water solutions and of most liquids parallel this behaviour.
- The pliabilities of asphalts, and resins increase with rising temperature; conversely, these materials approach brittleness at cold temperatures.
- Relative humidity decreases and evaporation rate increases as temperature rises.

It is known that the strength of concrete at long curing times (final strength) is adversely affected in hot climates (e.g. Jalali & Abyaneh, 1995). This is due possibly to the large differences in volumetric expansion of concrete constituents with temperature. A large increase in volume of air and water with high temperatures creates internal stresses in the curing concrete. If the tensile strength of concrete is not sufficient to withstand these internal stresses, porosity increases and microcracks are formed, resulting in the loss of final strength. Although this work relates to concrete, not cement-stabilised soil, it would not be safe to assume that hot curing conditions do not adversely affect the strength of rammed earth walls stabilised using cement.

6.8.2 Wind

When evaporation is necessary to attain optimum moisture for compaction or to lose excess water in curing, wind is an invaluable aid. Conversely, where retention of water to approximate optimum at the time of compaction or to create slow hydration (e.g. for cement stabilisation) is desirable, wind can be detrimental. The rate of diffusion of water vapor into still air, and thus the rate of evaporation, is very slow. In a dead calm, air immediately adjacent to the evaporational interface becomes saturated; additional evaporation cannot take place until some of the water vapour diffuses. If a wind springs up, fresh supplies of vapour-hungry air are continually fed to the interface. Undesirable and detrimental moisture loss from natural or stabilised soil layers can be prevented by use of membranes. To a lesser degree, heat transfer from air to soil or vice versa is aided by wind. In a calm, the exchange of heat between soil and air would be aided by convection currents set in motion by vertical temperature differentials. With wind, the heat dissipation or absorption would steepen the vertical temperature gradients and intensify the rate of heat transference.

6.8.3 Humidity

Absolute humidity is expressed either by the weight of water vapour per unit volume of air or by its expansive force. Relative humidity is the ratio of the actual amount of water vapour to the theoretical maximum at the same temperature; a relative humidity of 50 would signify a quantity of water vapour equal to half the maximum able to be held in the air volume. At constant moisture content relative humidity decreases as temperature rises. Conversely, for each quantity of water vapour present in the atmosphere, there exists a temperature, the dew point, at which saturation would be exceeded and excess water vapour would become liquid. In unturbulent air, water vapour can exist in a supersaturated state; precipitation formed there at ordinary temperatures is rain or dew, and at low temperatures is crystalline, as snowflakes in upper strata.

The interrelation of temperature and humidity is significant to soil stabilisation and therefore, rammed earth wall construction. Wide fluctuations in daily relative humidity often occur where the daily ranges in temperatures are large. In some coastal areas, air heated by sun-warmed ground blows oceanward in the daytime; at night the direction is reversed. Naturally, the humidity is lower landward and higher seaward. Heated air expands; as it expands, its density decreases; and it rises to "float" on heavier air. Sometimes land-warmed dry air, expanding laterally and upward because of lesser density at increased temperature, might spill seaward over the denser, landward moving ocean-cooled humid air, creating therein an inversion of the normal vertical temperature gradient. The temperature inversion fosters supersaturation of water vapour and the formation of fog.

Fog consists of visible, or at least light-refracting, minute globules of water held in suspension in air in which temperature increases with altitude. When the fog front nears the sun-warmed land, the air regains its normal temperature-altitude radient and its vapour capacity increases; the fog suspension then reverts to its invisible, vapour phase. As the angle of the sun's rays flattens, the expansion of air above the land diminishes, and the fog rolls in, sometimes to remain until next midday. Soil stabilisation of soils for rammed earth wall construction in coastal areas frequently encounters such daily extremes in humidity. Water control for hydration and for densification and the timing of compaction after mixing are more critical under such conditions, and curing times are erratic unless provisions are made against undesirable moisture loss or gain.

6.8.4 Precipitation

Less predictable than temperature, wind, and humidity at the time of construction is precipitation, the spoiler of many rammed earth wall construction operations and the curtailer of many projects. Local rain records can aid probability estimates. Seasonal probabilities of precipitation are fairly reliable in some locations. For example, in southern and southern Asia, the monsoon winds from the Indian Ocean bring much rain from late April to mid-October; practically no rain falls when the winds reverse during October to April. The actual period of monsoon prevalence varies in different parts of the Far East; the season can also vary from year to year. Climatic change may be making these patterns less predictable.

Rammed earth wall construction during the rainy season can entail such problems as achieving maximum density because optimum moisture content cannot be met. This moisture content cannot be controlled in the field if rain is falling. The soil pile will become increasing wetter and higher in moisture content as the rain falls and collects on the soil pile. Covering the pile may be an option but only works for a short period of time as the soil is still exposed during the moving to formwork, shovelling and ramming into the rammed earth wall itself. It is therefore advisable to stop ramming during rainfall and extreme moisture conditions.

If cement is used for stabilisation during the rainy season, storage, distribution, and mixing of bag cement are awkward. Lime is less critical and less vulnerable, either as primary stabiliser or in combination with fly-ash. Any type of bituminous work during the rainy season may be doubly vulnerable: Emulsions may break slowly and require excessive curing time; cutbacks may coat over wet clods and lose solvent too slowly; and liquid asphalt may form large blobs instead of enveloping films. Repeated wetting and drying of a silty clay soil are accompanied by alternate expansion and contraction. The effect of volume changes is to cause disruption of soil aggregate or clods into smaller units when the dried soil is wetted. The rapid intake of water causes unequal swelling throughout the clod and thus produces inferior rammed earth walls.

Some locations have relatively low mean annual and monthly precipitations but are characterised by frequent, sustained periods of mist or drizzle, a condition better suited to cement or lime treatment than to asphalt. All degrees of precipitation are favourable to bacteria with the exception of extreme dryness leading to accelerated oxidation.

6.8.5 Implications

The main effects of the environmental conditions outlined above relate to moisture: the moisture content of the soil-stabiliser mix, the rate at which it dries before and during compaction, and the rate at which it dries and cures. Given the importance of moisture content to maximum dry density and stabilised compressive strength, the moisture content of the mix must be carefully checked and controlled during compaction. Practically, this could be done with a speedy moisture gun, although the practice of monitoring moisture content on an almost continuous basis seems to be rare among the earth building community.

The earth builder must be aware of the geographic environment in which he or she is working, as it relates to environmental conditions during construction. For example, an earth wall construction site along an oceanfront would probably be faced with high humidity, little fluctuation in temperature, and medium but frequent precipitation. Rammed earth sites in hot desert areas are characterised by a wide fluctuation in daily temperature, especially at high elevations, and very low humidity. The consequences of environmental conditions for stabilisation and construction of rammed earth walls, particularly as regards moisture, need to be properly planned for prior to commencement of construction.

6.9 PRACTICAL SOIL STABILISATION

6.9.1 Rammed earth walls using lime

The aspects that seem most important to control during construction using lime-soil mixtures are: pulverisation and screening material; lime content; uniformity of mixing; time required in ramming material; compaction; and curing.

6.9.1.1 Pulverisation and screening rammed earth wall material

Prior to the application of lime, the soil is pulverized or screened. In order to assure the assure the adequacy of this phase of construction, a sieve analysis is performed. Most specifications are based upon a designated amount of material passing the 0.075 mm sieve. The depth of pulverisation is also a importance as it relates to the specified mixing of the lime stabilisation treatment. For heavy clays adequate pulverisation can best be achieved by pre-treatment with lime. When pre-treatment with lime is utilised, it is important to recognise that agglomerated soil-lime fractions may appear. These fractions can be easily broken down with additional time of mixing with the rotary hoe.

6.9.1.2 Lime content

When lime is applied to the premixed soil, the rate at which it is being applied is easily determined by the volume of soil for each bucket, which relates to the laboratory testing of dry weight volume versus percentage. Written instruction can be made available to field personnel for the correct lime content specifications.

6.9.1.3 Uniformity of mixing

Of concern here is obtaining a uniform lime content throughout the treated soil pile. This presents one of the most difficult factors to control in the field. It has been observed that mixed soil and lime has more or less the same outward appearance as mixed soil without lime. If the builder is concerned about proper mixing in the field a phenolphthalein indicator solution for control can be used. This method, while not sophisticated enough to provide an exact measure of lime content, will give an indication of the presence of the minimum lime content required for soil treatment. The soil will turn a reddish pink colour when sprayed with the indicator solution, indicating that lime is available in the soil.

6.9.1.5 Compaction

Of significance here is the proper control of moisture-density. Moisture content can be determined by either the nuclear method, speedy moisture meter method, or microwave oven method. In nuclear moisture determination, the small, safety-sealed americium 241-beryllium radioactive source emits neutron radiation into the test material. The high-energy neutrons are moderated by collision with the hydrogen atoms in moisture contained in the test material. Therefore only low energy moderated neutrons are detected by the moisture detector. If the test material is wet the meter will indicate a high response, if it dry the meter will indicate a low response for the same unit period of time. The speedy moisture meter technique is based on the fact that water will react with calcium carbide to form a gas and that the quantity of gas formed is directly proportional to the water present, providing a surplus of the chemical is used in the test. The quantity of gas is indicated on a built-in pressure gauge, which is calibrated in percentage of moisture. A simple conversion by graph, or table will give the corresponding moisture content based or dry weight. The microwave method involves mixing rammed earth material and placing it on an electronic scale and weighing until 100 g remains in the evaporating dish. The specimen is then placed in the microwave and heated on high for 3 to 5 minutes, after which it is removed and reweighed. The difference in weight is the moisture content, as a proportion of the dry weight.

The influence of time between mixing and compacting has been demonstrated in the field to have a pronounced effect on the properties of the stabilised soil. Compaction should begin as soon as possible after final mixing has been completed. If the stabilised material is to be held over night or well into the next day the indicator solution described in mixing should be used to determine the uniformity of lime. Stabilised material should be keep covered, correct moisture content, and remixed several times to insure proper compaction will be take place.

6.9.1.6 Curing

Curing is essential to assure that the soil lime mixture will achieve the final properties desired. Curing is accomplished by one of three methods:

1. moist curing (light sprinkling of water)
2. Plastic membrane covering walls
3. Sealant sprayed in walls allowing moisture in and out slowly

Regardless of the method used, the entire compacted wall must be properly protected to ensure that the lime will not become non-reactive through carbonation.

6.9.2 Rammed earth walls using cement

The aspects that are most important when using cement-soil for construction are: pulverisation and or screening; cement content; moisture content; uniformity of mixing; time sequence of operations; compaction; and curing.

6.9.2.1 Screening and pulverisation

Screening and pulverisation is generally not a problem in soil cement rammed earth wall construction. If clayey or silty soils are being stabilised then problems may arise. Sieve analysis is performed on the soil during the screening/pulverisation process with the 0.075 mm sieve used as the control. The percentage of screening/pulverisation can then be determined by calculations.

6.9.2.2 Cement content

Cement content is expressed on a volume of dry weight of soil basis. Tables can be made available to field personnel which will enable them to determine quantities of cement per cubic metre of soil.

6.9.2.3 Moisture content

The optimum moisture content determined in the laboratory is used as an initial guide when rammed earth wall construction begins. Allowance must be made for the in-situ moisture content of the soil when construction starts. The optimum moisture content and maximum density can then be established for field control purposes. Mixing water requirements can be determined on the raw soil or on the soil-cement mix prior to the addition of the mixing water. Tables can be made available to field personnel as an aid in determining the proper quantities of mixing water to be added. The moisture content should be checked during mixing.

6.9.2.4 Uniformity of mixing

To assure the uniformity of the rammed earth wall mixture throughout the ramming process, a visual inspection is made. Uniformity must be checked across the width of the rammed earth walls and to the desired depth of the treated soil pile. The pile can be back-bladed and then visually inspected. A satisfactory mix will exhibit a uniform colour throughout, while a straked appearance indicates a non-uniform mix. Special attention should be given to the edges of the soil pile.

6.9.2.5 Time sequence of operations

Soil being used for rammed earth construction that is being stabilised with cement should be mixed on site and brought to the correct moisture content and then rammed immediately. If not used within one hour then the soil should be discarded and not used for construction.

6.9.2.6 Compaction

Soil type and water content are the determining factors in the compaction of the rammed earth walls. The compacted density of the stabilised rammed earth material should be checked in the laboratory. It is important to give instructions as to the depth of compaction to be used and require special attention to be given to the compaction at the edges of the formwork.

6.9.2.7 Curing

To assure proper curing a plastic membrane is frequently applied over the completed rammed earth walls, other methods include keeping walls moist, and spraying walls allowing them to breathe but dry out very slowly. The surface to the rammed earth walls should be free of dry loose material and in a moist condition. Attention should be paid to the weather conditions because this can have a deleterious effect on the rammed earth walls. A delay during construction between mixing the soil-cement and ramming the material may lead to a decrease in both density and strength for a fixed compactive effort (West, 1959). If, however, compaction effort is increased so that the original density is obtained, and provided no significant amount of cement hydration occurs during the delay period, then no strength loss is observed.

6.9.3 Rammed earth walls using asphalt in combination with lime or cement

The aspects considered most important for construction using asphalt as a waterproofing agent are: surface moisture content; viscosity of the asphalt; uniformity of mixing; compaction; and curing.

6.9.3.1 Moisture content

The gradation of the aggregate has proven to be of significance as regards to moisture content in rammed earth walls. With dense graded mixtures of soils will require more water to achieve compaction. Too high a moisture content will delay compaction of the mixture of soil for ramming. Higher plasticity index soils require higher moisture contents.

6.9.3.2 Viscosity of asphalt

Emulsified asphalt soil stabiliser consists of asphalt globules of microscopic size, which are surrounded by and suspended, in water medium. A few drops of asphalt stabiliser perceptibly colours a large amount of clear water and remains suspended without any coalescence of the asphalt. However, the stabiliser must be stored and used at a temperature above freezing or standing for long period of time. Freezing and standing causes the asphalt to settle out of emulsion and it becomes unusable for mixing with rammed earth soils. When the stabiliser is mixed into a clay-bearing soil in the presence of enough water, the water carries the asphalt globules into direct contact with the surfaces of the clay particles.

6.9.3.3 Uniformity of mixing

Visual inspection can be utilized to determine the uniformity of the mixed soil. With emulsified asphalts, a colour change from brown to black indicates that the emulsion has broken. The mixing of soil for ramming is best achieved by using a rotary hoe. Turning the soil with a bucket will not brake the emulsion down and in turn will not mix into the soil. Once the soil pile is mixed the pile should be covered and the moisture content maintained.

6.9.3.4 Compaction

Prior to compaction, the dilutents that facilitated the cold-mix operation must be allowed to evaporate. If the mixed soil is not sufficiently aerated, it cannot be compacted to acceptable limits. Most aerating occurs during the mixing stage, but occasionally additional mixing may be required. If the emulsion has changed colour and the soil feels tacky in the hand it is ready for compaction into rammed earth walls.

6.9.3.5 Curing

Curing presents the greatest problem in asphalt soil stabilised rammed earth walls. The rate of curing is dependent upon many variables: the quantity of asphalt applied, prevailing humidity and wind, the amount of rain, and the ambient temperature. Initial curing, the evaporation of dilutents, occurs during the aeration stage. If ramming is started too early, the rammed earth walls will be sealed, thereby delaying dehydration, which lengthens the time before desirable strength is reached. Temperatures on very hot days cause the soil to soften, which prohibits the proper compaction to occur. One week minimum should be allowed before structural loading of rammed earth walls begins.

6.9.4 Mixing - general points

Rotary hoes and plug mill mixers are excellent choices for the mixing of soils, stabilisers, and water for the construction of rammed earth walls. Front-end bucket type loaders used in a manner of turning the soil, stabiliser, and into each other, can also be utilised; however, the desired uniformity of mixing is not always obtained. Mixing difficulty increases with increasing fineness and plasticity of the soils being treated with stabiliser. In-place mixing efficiency with a front-end loader, as measured by the strength of the treated soils, maybe only 60-80 % of that obtained in the laboratory. Rotary hoe and pug mill mixing operations afford the best opportunity to produce uniform stabilised material and can achieve close to 100 % mixing efficiency as measured by the strength of the treated soil measured after field versus laboratory mixing.

6.9.5 Compaction – general points

Compaction should commence as soon as possible after uniform mixing of water and the stabiliser when lime-fly ash, cement-fly ash, and cement are used as stabilisers. Materials to be compacted within four hours of mixing and always be complete on the same day the soil is mixed with the stabilisers.

For maximum strength, lime-stabilised soils should be compacted soon after mixing, provided uniform mixing is achieved. Since the reactions associated with lime stabilisation are longer-term compared to cement stabilisation, additional time is available for mixing and pulverising lime stabilised soils. This additional time is particularly useful when highly plastic soils are being treated and pulverization is difficult.

Experience has shown that breakdown of emulsified asphalt mixes should begin immediately before, or at the same time as, the emulsion starts to break. At about this time, the moisture content of the mixture is sufficient to act as a lubricant between the aggregate particles, but is reduced to the point where it does not fill the void spaces, thus allowing air void reduction under compactive forces. Also, by this time, the mixture should be able to support the rammers without undue displacement.

For coarse-grained soils, increased density and decreased moisture content improve the physical properties of a soil that are of primary importance in the construction of rammed earth walls and the manufacture of compressed earth blocks. Using pneumatic rammers or compressed block increases machinery strength, consolidating under loading and the rate of water movement through the soil is decreased. High compaction of coarse-grained materials to obtain these advantages is accepted practice when building rammed earth walls and manufacturing compressed blocks. On the other hand, problems may result from over compaction of clay materials that have a high affinity for water. These materials, if over compacted, will later take on water and expand. This could result in a possible failure of the constructed earth wall.

Layers of stabilised soil are normally spread to a thickness of between 30-50 % greater than the desired final thickness. Some experimentation may be necessary to determine the proper spread thickness of each lift within the formwork. The maximum recommended thickness for a simple stabilised layer of soil within the formwork is 150 mm. As a general rule, subsequent layers should be placed the same day so as to ensure the development of a bond between layers. Steps should be taken to ensure that there is no loose material on the lower layer and that the surface is moist before placing the stabilised material for the subsequent layers.

6.9.6 Curing – general points

After compaction, proper curing of walls containing lime, lime-fly ash, cement-fly ash, or cement stabilisation is important because the strength gain is dependent upon time, temperature, and the presence of water. Generally a 3- to 7-day curing period is required, during which time walls should not be loaded. Two types of curing are employed to ensure that the moisture is retained in the stabilised layers. These are sprinkling with water, and membrane. Sprinkling with water to keep the surface damp, has proven successful. However, the preferred method is membrane curing. In membrane curing, the stabilised soil is either sealed with a sealant, or covered with a plastic membrane. This should commence immediately after removal of formwork.

Time, temperature, and moisture conditions during the curing period for the compacted samples to be used in compressive strength tests vary significantly. Normally elevated temperature curing is faster than ambient temperature curing. Many procedures specify that the samples should be cured in a sealed condition while other procedures require a moist curing cycle followed by a drying cycle.

Laboratory curing and mixing conditions should, to some degree, be correlated with field conditions. The mixing of a stabiliser into the soil may take several hours and then set for several hours before it is compacted into the rammed earth wall. This should be duplicated in the laboratory testing if possible.

6.9.7 Practical construction details

In the construction of rammed earth walls the objective is to obtain a thorough mixture of a soil or aggregate material with the correct quantity of stabiliser and sufficient fluids to permit maximum compaction. Specifically, equipment must be selected, operated, and sequenced to provide the following:

- The proper water content (uniformly mixed),
- The proper stabiliser content (uniformly mixed),
- The attainment of a minimum density required,

- Favourable temperature and moisture conditions for strength development during the curing period, and
- Protection of the stabilised wall surface from rain to prevent abrasion and to ensure adequate time for strength development.

CHAPTER 7: SUMMARY AND CONCLUSIONS

7.1 OUTCOMES OF THE STUDY

This study has examined the relationships between natural soil characteristics, stabiliser treatments, and the strength and density of stabilised earth, based on a more extensive range of soils than used in previous studies. The major outcomes of the study are:

- **The generation of a comprehensive data set on soil properties and stabilisation treatments for rammed earth walls.** One hundred and forty nine unique soil samples representing a wide range of soil types have been collected and measurements of texture, plasticity, and shrinkage made on them. Two hundred and thirty experimental determinations of stabilised unconfined compressive strength have been made, and over 200 determinations of optimum moisture content and maximum density. In all, around 1,700 individual measurements have been completed. The data set is therefore a large one, and is directed specifically at soil stabilisation for rammed earth wall construction. The data have been analysed in order to establish relationships between soil characteristics, stabiliser treatments, and stabilisation outcomes, and for the development of improved stabilisation guidelines for rammed earth construction.

- **Better knowledge of the relationships between soil characteristics, stabiliser treatments, and stabilised earth properties.** Relationships have been established between soil properties (gradation, plasticity, and moisture), stabiliser types (cement, lime, asphalt), stabiliser levels, and stabilised earth qualities (density and compressive strength), for a wide range of soil types. Analysis of these multiple influences on stabilisation shows that using soils with particular characteristics, and avoiding those with contrary characteristics, are the keys to successful stabilisation. The ways in which soils with particular characteristics favour or disfavour successful rammed earth stabilisation is now better quantified, and an advance has been made in the use of soil properties to discriminate between good and poor soils for rammed earth stabilisation.

- **Improved guidelines for the stabilisation of soils for rammed earth construction.** Specific guidelines have been constructed to assist practitioners in the field and to improve the results of construction. Specific guidelines have been developed for soil selection, preparation, and stabilisation for rammed earth wall construction, based on the measured relationships between soil properties, stabiliser treatments, and the qualities of the stabilised product. Based on the differential suitability of soils for stabilisation, the stabiliser treatments have been recommended for the satisfactory stabilisation of these soils. Soil properties, especially plasticity index and linear shrinkage, can be used to discriminate between those soils that are likely to successfully stabilise and those that are not. The guidelines diminish the chances of selecting a soil that is unlikely to prove suitable for rammed earth wall construction.

7.2 RELATIVE IMPORTANCE OF SOILS AND STABILISERS

Stabilised strength and density are functions of three influences: differences in stabiliser type, differences in the quantity of stabiliser, and differences in soil type. Several statistical tests have been used to show that soil characteristics, not stabiliser type or quantity, exert the dominant influence on variation in stabilised compressive strength and density. Variation in soil type accounts for over 90 % of the variation in the compressive strength of tested samples, and over 95 % of the density variation. These figures are higher than those interpreted from a collation of previous studies, which overall suggested that the contributions to variation in compressive strength made by differences in soil type, stabiliser type, and stabiliser quantity, were roughly equal in importance (Table 5.1). Those previous studies reported differences between cement and lime of between 0 and 2.6 MPa, whereas this study determined that the overall difference in effectiveness between lime and cement is not critical (Table 5.1). The earlier studies indicated differences between stabiliser percentages whereby lime increased strength on average by 0.24 MPa per % of lime added, and cement increased strength on average by 0.40 MPa per % of cement added. In contrast, for this study, the increase in strength for lime added is at most 0.03 MPa per %, and for cement added is at most 0.07 MPa per % (Tables 4.7 and 5.2).

One reason for the difference in the relative importance of soils and stabilisers identified between this study and previous research is the much larger number of soil types tested here and the much greater variation in their characteristics. This wide range of soil variation tends to relatively increase the effect of soil characteristics and diminish the effects associated with stabiliser types and quantities, as discussed in Section 5.1.1. In addition, the wide variability in compressive strength resulting from soil variations means that statistical confidence limits around strength values averaged for particular stabiliser levels are quite large, making statistically significant contrasts between different stabiliser types and levels less likely to be achieved (e.g. Section 4.7.2).

7.3 SOIL SUITABILITY FOR STABILISATION

The major implication of the overwhelming importance of soil characteristics to stabilised strength and density is that the key to successful stabilisation lies in the selection of an appropriate soil. A second implication is that soils can be effectively assessed for their predisposition to successful (or unsuccessful) stabilisation. This information about the variation in the suitability of soils for stabilisation should be able to be applied at a practical level to future soil stabilisation situations.

Soil properties are able to be used to predict the suitability of a soil for stabilisation on the basis of how the properties relate to the compressive strength of the stabilised material. Three groups of soils have been identified according to their predisposition for stabilisation on the basis of their texture, plasticity, and shrinkage, and are designated unfavourable soils, satisfactory soils, and favourable soils (Table 5.3). Unfavourable soils, as defined, should be avoided, favourable soils should be sought, and satisfactory soils are intermediate in nature and should be assessed carefully for suitability. Recommendations by most other researchers have been confined to "favourable" soils. Soil property values that define unfavourable and satisfactory soils have not previously been quantified. In addition, the value ranges as recommended by this study for favourable soils have been able to be more tightly constrained than in previous studies. Regarding favourable soil recommendations, the data in this study broadly support previous recommendations of ≤ 15 % for plasticity index and ≤ 35 % for liquid limit. However, there are some disparities in the recommendations for other soil properties. Although the maximum clay/silt content of 35 % is in line with other studies, the lower limit is higher at around 20 %. Sand content recommendations also diverge: this study recommends sand contents in the range of 30-65 % as most suitable, whereas traditionally, sand contents in excess of 60 % have been regarded as most favourable for stabilisation.

This study has identified linear shrinkage as being a very good indicator of soil suitability for stabilisation. Linear shrinkage of the natural soil has been used previously for predicting stabilised shrinkage, but has not been used as an indicator of stabilised strength or density. Given the efficacy of linear shrinkage as a discriminator of soil suitability, it is unclear why this property may have been ignored by previous workers, considering its potential not only for indicating stabilised strength but also stabilised shrinkage and cracking potential.

Linear shrinkage and plasticity index are the best indicators of the stabilisation predisposition of a soil, and can discriminate between favourable, satisfactory, and unfavourable soils. Linear shrinkage and plasticity index should be regarded as the primary discriminators of soil suitability for stabilisation. Values of plasticity index of > 30 %, and linear shrinkage > 13 % indicate unfavourable soils; values of plasticity index 16-30 %, and linear shrinkage 7.1-13 % indicate satisfactory soils; and values of plasticity index < 16 %, and linear shrinkage < 7.1 % indicate favourable soils. Other soil variables are subordinate to plasticity and shrinkage for discriminating soil suitability. If the results of this study are applicable to new situations, then soils with favourable values of shrinkage and plasticity have a 90 % likelihood of being successfully stabilised, soils with satisfactory values have a 65 % likelihood, and soils with unfavourable values have just a 10 % likelihood of success. These values change if the strength criterion value is increased or decreased with respect to the 2 MPa value used, with higher criterion values being associated with markedly lower rates of stabilisation success even for the most favourable soils.

Soils classed as favourable on the basis of shrinkage and plasticity determinations should almost certainly be suitable for stabilisation, with an overall success rate of 90 %. The exception for this group of soils is where clay/silt and gravel contents are satisfactory or unfavourable, as this decreases the success likelihood to around 80 % compared to > 95 % when clay/silt is favourable. Satisfactory or unfavourable sand contents do not affect stabilisation success for this group of soils. Therefore, clay/silt and gravel contents are useful secondary discriminators for soils that have favourable shrinkage and plasticity values.

Soils classed as satisfactory on the basis of shrinkage and plasticity determinations are more likely than not to be suitable for stabilisation. Sand content is a good secondary discriminator for these soils. If sand content is favourable (< 60 % sand), then the likelihood of successful stabilisation is around 85 %, compared with a figure of around 55 % for soils with satisfactory or unfavourable sand contents of > 60 %. This likelihood of success of just over 1 in 2 would be rejected by most practitioners, and therefore the "cut off" point for an acceptable soil would be one with satisfactory values of plasticity and shrinkage and a favourable sand content. Given their low stabilisation success rate (10 %), soils with unfavourable (i.e. high) plasticity and shrinkage values should be avoided altogether for the purposes of stabilisation for rammed earth walls.

In summary, then, for discriminating soils for stabilisation, linear shrinkage and plasticity index should be used as primary discriminators, and the textural properties as secondary discriminators. Soils with unfavourable values of linear shrinkage and plasticity index should not be used. Soils with favourable values can be further differentiated using clay/silt and gravel content. Soils with satisfactory values of shrinkage and plasticity should be assessed further using sand content. The use of the textural properties as successful secondary discriminators, to better indicate the likelihood of stabilisation success once shrinkage and plasticity are known, is a novel approach.

7.4 STABILISER TREATMENTS

Overall, in this study, soil properties have been found to outweigh the importance of variation in stabiliser treatments. Although the differences in stabilised strength between different stabiliser types and quantities have been shown to be small (Sections 4.7.2 and 5.1.3), the demonstrated differential favourability of soils for stabilisation means that stabiliser treatments can be tailored to particular soils in response to this differential. The two best discriminators or predictors of stabilised compressive strength are linear shrinkage and plasticity index. These soil properties give an excellent indication of the predisposition of a soil towards stabilisation, and in the absence of gross differences in strength generated by different stabilisers, can be used to specify the type and quantity of stabiliser. Soils designated as unfavourable in this study should not be stabilised in any circumstances, as they are most unlikely to achieve satisfactory compressive strength (of minimum 2 MPa). Soils designated as satisfactory should be stabilised with a minimum of 6 % stabiliser - 4 % lime and 2 % cement. Soils designated as favourable can be stabilised with lower quantities of cement and lime as plasticity and shrinkage decrease progressively below 16 % and 7.1 % respectively to reach very favourable soils of plasticity less than or equal to 7 % and shrinkage less than or equal to 4 % (Section 6.5.1). When used in combination with either lime or cement, asphalt is effective only as a waterproofing agent as it has no influence on the compressive strength of the stabilised soil.

In addition to compressive strength, the shrinkage and cracking potential of stabilised soil should also be a consideration, given the monolithic design of most rammed earth walls. Adding quantities of lime and/or cement reduces shrinkage and is beneficial in this respect as well as imparting strength. Adding 6 % cement, or 4 % lime and 2 % cement, will reduce a soil with natural shrinkage of 7 % to a shrinkage criterion of 3 % when stabilised, which corresponds with the percentage also recommended to fulfil compressive strength requirements. Soils of shrinkage $\leq$ 7 % can be treated with lower quantities of stabiliser to achieve the same stabilised shrinkage of 3 %. As lime is more effective at reducing shrinkage and plasticity than cement, it should be added in greater proportions and in preference to cement for more clay-rich soils.

7.5 PREDICTION OF STABILISED STRENGTH AND DENSITY

7.5.1 Prediction of specific values versus discrimination using a criterion value

Two contrasting methods have been used in this study in an attempt to predict the strength (and density) of stabilised material on the basis of soil properties. Stabiliser treatment effects were found to be small in this study and have been disregarded in using soil variables as predictors of stabilisation outcomes. The basic aim of prediction is to minimise the chance of building an unsatisfactory rammed earth wall and maximise the chance of success; indeed, this was the intention behind the development of the stabilisation flow chart in Chapter 6.

The success of predicting an individual or average sample stabilised strength on the basis of natural soil properties has been shown in this study to be poor, with less than 30 % of the variation in compressive strength being explained using linear statistical models. It is clear that the way forward is to predict not an exact strength of a sample, but rather whether or not a stabilised sample is likely to pass or fail a criterion strength value. Predicting stabilisation success or failure with respect to a criterion value may make more sense in that building and professional regulations specify in terms of such criterion values. Given the efficiency with which a small number of soil properties can distinguish between suitable and unsuitable samples on the basis of the compressive strength criterion value in this study, it would be hoped that the soil criteria values established here could be used and reviewed by other practitioners.

7.5.2 Relationship between compressive strength and density

Surprisingly, soil samples with higher density are not necessarily characterised by greater strength. Density is therefore not a particularly good predictor of stabilised compressive strength, either on an individual value basis, or when criterion values are used. Of the 139 samples in this study that pass the density criterion ($\geq$ 1.8 t/m^3), 16 (12 %) did not subsequently pass the strength criterion of 2 MPa. %. Moreover, of the 63 samples that failed the density criterion, almost half (30) subsequently achieved stabilised strengths of $\geq$ 2 MPa. What is evident is that maximum dry density is an inconsistent predictor of stabilised strength. It is recommended therefore that the maximum density determination test be used not for predicting stabilised strength, but for identifying the optimum moisture content at which to compact the soil-stabiliser mix. The soil properties, particularly plasticity index and linear shrinkage, can be used prior to the density and strength testing to judge the suitability of a soil for stabilisation.

7.6 SUGGESTIONS FOR FURTHER RESEARCH

Some fruitful avenues for future research on soil stabilisation for rammed earth construction are able to be drawn from the results presented in this thesis, some of which have already been alluded to above.

The importance of soil properties in determining the outcome of stabilisation means that it is critical to find a suitable soil. Therefore, further research could be performed on the problems of mapping soil properties and sampling with respect to searching for soils specifically for rammed earth stabilisation. There is probably a need to better relate field soil indices to the known attributes of suitable and unsuitable soils, so that the search for suitable soils can take place on a speedier, more effective basis. Although it is known in general terms the sorts of landscapes, topographic features, and vegetation types that are loosely associated with certain soils, a more quantitative scheme may prove useful. Such relationships between the field indices and favourable (or unfavourable) soil characteristics could be better appraised by a database being developed that contained geomorphic, botanic, and soil information as collected by practitioners in the field. Such a database would, over time, be of enormous help in identifying more precisely the environmental features associated with good and poor soils. Additionally, field tests of texture and plasticity should be correlated experimentally with laboratory-measured textures and plasticities, so that a better "feel" for the soil in the field can be obtained. Improving methods of field survey and field tests should enable better decision-making to be made concerning soil suitability.

Although it is beyond the scope of this study, an interesting approach to the problem of soil mapping is discussed in McKenzie & Austin (1993). As an alternative to traditional soil mapping and survey, they utilised "parametric soil survey", which essentially aims to predict and map soil characteristics over the landscape, rather than define soil types. Using a mixture of aerial photo data and fieldwork, they were able to map textural properties with some success. Although such techniques may at present be out of the range of rammed earth practitioners, or deemed to be unnecessary, it is thought-provoking when considering how best to search for a suitable soil for stabilisation.

More research into the use of soil properties as indicators of a soil's predisposition to stabilisation should be developed. In particular, there is a need to further explore the significance of soil linear shrinkage as a new predictor of stabilised strength. In addition, further research on the use of textural properties as secondary discriminators for soils categorised using linear shrinkage and plasticity could be made, with a view to understanding better why clay/silt content and sand content are useful discriminators in different situations. Over time, further data on soil properties as discriminators of soil suitability could lead to a rammed earth statistical database being generated, which would be a valuable resource for the improvement of stabilisation methods and outcomes for a wide range of soil types and stabilisers. The flow chart developed in this study has been designed to facilitate feedback and direct input from practitioners who review and use it. In an iterative manner, it is hoped that the flow chart model could be refined and improved by the input of additional data from other practitioners.

Although it is not a focus of this study, the durability (as opposed to strength) of earth buildings is a topic of concern to some building authorities and academics (e.g. Heathcote, 1995). It remains unclear whether the durability of stabilised material should be, or is, controlled by the same variables as are compressive strength and density. In order to shed light on this problem, durability measurements could be made on samples of the soils stabilised in this study. This would enable the influences of different soils and stabilisers on durability to be compared with the results for compressive strength as established here, using similar methods of analysis.

7.7 RAMMED EARTH STATISTICAL QUALITY CONTROL

In recent years, the trend toward end result specifications has led to the implementation of statistical quality controls in various aspects of construction. Rammed earth is but another building medium that in Australia may soon be under the control of an Australian Standard, which will use some form of statistically-oriented end result specifications. These specifications should define the testing regimen of rammed earth units, in terms of the number of samples that should be tested, the average sample strength, and the acceptable strength variation in the samples (e.g. New Mexico Construction Bureau, 1991). To date no proper statistical survey has been completed to reveal the variation in quality of rammed, stabilised material at construction sites. There would be several advantages to having a system of statistical quality control for rammed earth. First, it would match the more general construction industry trend towards end result specifications. Second, "recipe type" specifications can have a significantly beneficial impact on the rate at which work is performed. Third, it would produce researched results that professionals could recommend and utilise, with an ever-increasing database of measurements of rammed earth qualities as more results became available.

A statistical quality control system for rammed earth wall construction, therefore, should specify the following:

1. The number and relative locations of sample sites in the field - this study has addressed some of these issues (see Section 6.1).
2. The natural soil properties that should be measured to indicate soil suitability for stabilisation - this study recommends linear shrinkage and plasticity index as primary discriminators of soils, and clay/silt and sand content as secondary discriminators (see Section 5.4).
3. The appropriate outcome variables and values of stabilisation - this study used unconfined compressive strength as the major outcome variable as this is specified in building codes and guide books; however, other outcome variables could potentially be specified such as swell/shrinkage, or various measures of durability;
4. The acceptable average and variation in individual values of stabilised strength as sampled, and the testing regimen required to assess such variation (see Section 6.7);
5. The expected stabilisation outcome given a soil with particular properties mixed with a particular type and quantity of stabiliser, for a wide range of soil types, stabiliser types, and stabiliser quantities (this study found that variation in stabiliser types and quantities were subordinate to soil characteristics in influencing stabilised strength).

It is evident that this study has contributed substantially to the development of such a statistical system, chiefly by developing quantitative criteria by which soil suitability for stabilisation can be assessed. It has also clarified and addressed some of the issues involved in such a system. For example, the prediction success of an actual, exact value for stabilised strength using soil properties and stabiliser quantities is poor (Sections 4.8.2 and 5.4). Indeed, less than 30 % of the variation in compressive strength is explainable by the soil properties using a linear model, which is nowhere near high enough to use as a model to predict individual values of compressive strength. Therefore, any statistical quality control system would probably need to be based on ranges of values of soils properties, rather than individual values, expressed in a form such as Table 5.3. It would also need to express likelihoods (probabilities) of successful stabilisation, as measured against a criterion value of stabilisation outcome, such as strength, exactly as have been developed in this study based on values of soil properties. The criterion value of 2 MPa as used in this study may be different for other situations, dependent on the needs of the builder, architect, or different building codes. Although the results of this study do not focus on different criterion values, Section 5.4.4 demonstrates that increasing the criterion value above 2 MPa requires that the desirable soil range be narrowed toward the most favourable soils as defined by very low plasticity and shrinkage.

In a statistical quality control system, engineers and architects monitor the builder's quality control procedures and periodically take samples for testing. This serves as a check on the builder and enhances the reliability of the results. In establishing a statistical quality control of rammed earth, it must be recognised that there are risks involved for the professional, builder, and client. One of the risks is the waste of time and resources spent on searching for soils and testing soils that prove to be unsuitable for stabilisation as indicated by failed strength and density determinations. This thesis has provided information to minimise these risks by establishing new quantitative criteria against which soil suitability may be assessed, by assigning likelihoods of successful (or unsuccessful) stabilisation to soils that exhibit particular values of shrinkage, plasticity, and texture.

7.8 EARTH BUILDING AND SUSTAINABILITY

The results of this study will allow rammed earth practitioners and new participants in the industry to more easily and successfully stabilise earth for use in earth building. If wider adoption of earth building were facilitated, there would be environmental benefits to be realised. However, wider adoption of earth building technologies would require significant changes in the attitudes of practitioners and the public at large. Using earth as a construction material with small percentages of stabiliser (< 10 %) is environmentally more sustainable and uses less energy than other construction methods such as concrete and steel. Building with earth consumes less energy than other methods, and also minimises the use of expensive, scarce resources such as cement, steel, and bricks in developing countries (Tiwari et al., 1996). In addition, CO_2 emissions associated with the production of cement could be reduced by using stabilised earth as an alternative building material.

While most rammed earth builders use cement exclusively as the stabiliser, there are alternatives that have the potential to further reduce environmental impacts. These include fly-ash, tall oil, slag, and sugar cane by-products, amongst others. Most of these materials are industrial by-products materials currently thought of, and treated as, waste. Alternative stabilisers would reduce landfilling of waste products and reduce embodied energy, while at the same time reducing expense. However, the alternative stabilisers would have to be shown to be of sufficient merit in terms of improving the strength of rammed earth walls to acceptable levels. Fly-ash, a coal combustion by-product, has been shown by several studies to be beneficial in this regard (e.g. Symons, 1999; Singh & Garg, 1999), and can be used as a replacement for up to 25 % of the Portland cement in stabilised earth walls or concrete foundations.

Rammed earth construction provides an opportunity to identify opportunities for revenue growth in environmentally sustainable products and technologies. Eco-efficiency research would involve mapping and measuring inputs and outputs from all by-product processes. The established criteria would provide direction for practitioners in developing a method to account for the total cost of rammed earth wall construction that includes the true environmental cost of producing, using, recycling, and disposing of by-products, compared with other construction technologies. Though recycled building products are becoming more common, there are still many barriers to overcome before by-products in rammed earth wall construction would be used routinely. Some caution is justified—the industry is responsible for public safety and is vulnerable to lawsuits. In addition, profit margins can be very slim. The construction industry as a whole tends to be conservative, relying on traditional products with long histories. However, many barriers can be overcome by education and access to information. Research into sustainable construction materials and methods will be most successfully applied when it is able to be incorporated into the framework of general construction industry practices.

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APPENDIX 1

DATA GENERATED IN THIS STUDY

Table 4.1: Data for 230 stabilised samples examined in this study. Abbreviations are: UCS is unconfined compressive strength (dry); L.L. is liquid limit; P.L. is plastic limit; P.I. is plasticity index; L.S. is linear shrinkage. UCS Code: 0 is <2 MPa; 1 is ≥2 MPa. Density Code: 0 is < 1.80 t/m^3; 1 is ≥1.80 t/m^3.

Site	Sample Number	Determination Number	L.L. (%)	L.L. Class	P.L. (%)	P.L. Class	P.I. (%)	P.I. Class	L.S. (%)	L.S. (Class)	Clay (%)	Clay Class	Sand (%)	Sand Class	Gravel (%)	Gravel Class	Moisture (%)	Moisture Class	Density (t/m3)	Density Code	UCS (MPa)	UCS Code	Lime (%)	Cement (%)	Asphalt (%)
n	1	1	18.0	1	14.0	1	4.0	1	3.0	1	9	1	88	10	3	2	8.5	3	2.08	1	4.40	1	0	6	3
n	1	2	18.0	1	14.0	1	4.0	1	3.0	1	9	1	88	10	3	2	8.5	3	2.08	1	4.40	1	0	3	0
n	1	3	18.0	1	14.0	1	4.0	1	3.0	1	9	1	88	10	3	2	8.5	3	2.08	1	4.40	1	0	6	0
d	2	4	19.0	1	14.0	1	5.0	1	4.0	2	25	4	60	4	15	5	7.8	2	1.96	1	2.54	1	2	4	0
d	2	5	19.0	1	14.0	1	5.0	1	4.0	2	25	4	60	4	15	5	7.8	2	1.96	1	2.54	1	0	5	0
d	2	6	19.0	1	14.0	1	5.0	1	4.0	2	25	4	60	4	15	5	7.8	2	1.96	1	2.60	1	2	4	3
c	3	7	20.0	1	20.0	4	0.0	1	1.0	1	29	5	62	5	9	4	7.3	2	1.89	1	3.20	1	2	4	0
c	3	8	20.0	1	20.0	4	0.0	1	1.0	1	29	5	62	5	9	4	7.3	2	1.89	1	3.20	1	0	5	0
c	3	9	20.0	1	20.0	4	0.0	1	1.0	1	29	5	62	5	9	4	7.3	2	1.89	1	3.30	1	2	4	3
g	4	10	20.0	1	19.0	3	1.0	1			30	5	70	6	0	1	8.3	3	1.65	0	1.90	0	0	5	0
g	4	11	20.0	1	19.0	3	1.0	1			30	5	70	6	0	1	8.3	3	1.65	0	1.90	0	0	4	0
g	4	12	20.0	1	19.0	3	1.0	1			30	5	70	6	0	1	8.3	3	1.65	0	2.00	1	0	4	3
s	5	13	21.0	2	17.0	3	4.0	1	2.5	1	14	2	57	4	29	6	8.5	3	2.08	1	4.70	1	0	6	3
s	5	14	21.0	2	17.0	3	4.0	1	2.5	1	14	2	57	4	29	6	8.5	3	2.08	1	4.70	1	0	4	0
s	5	15	21.0	2	17.0	3	4.0	1	2.5	1	14	2	57	4	29	6	8.5	3	2.08	1	4.70	1	0	6	0

i	6	16	22.0	2	22.0	4	0.0	1	1.0	1	12	2	43	1	45	7	8.7	3	2.20	1	3.90	1	0	4	3
i	6	17	22.0	2	22.0	4	0.0	1	1.0	1	12	2	43	1	45	7	8.7	3	2.20	1	3.90	1	0	4	0
q	7	18	22.0	2	17.0	3	5.0	1	3.0	1	19	3	37	1	44	7	8.4	3	2.12	1	4.20	1	0	6	3
q	7	19	22.0	2	17.0	3	5.0	1	3.0	1	19	3	37	1	44	7	8.4	3	2.12	1	4.20	1	0	5	0
q	7	20	22.0	2	17.0	3	5.0	1	3.0	1	19	3	37	1	44	7	8.4	3	2.12	1	4.20	1	0	6	0
c	8	21	22.0	2	19.0	3	3.0	1	2.0	1	33	6	58	4	9	4	9.9	4	1.72	0	2.95	1	0	5	3
c	8	22	22.0	2	19.0	3	3.0	1	2.0	1	33	6	58	4	9	4	9.9	4	1.72	0	2.95	1	0	6	0
c	8	23	22.0	2	19.0	3	3.0	1	2.0	1	33	6	58	4	9	4	9.9	4	1.72	0	2.95	1	0	5	0
g	9	24	22.0	2	19.0	3	3.0	1	5.5	3	40	7	60	4	0	1	11.4	6	1.92	1	2.70	1	0	6	0
g	9	25	22.0	2	19.0	3	3.0	1	5.5	3	40	7	60	4	0	1	11.4	6	1.92	1	2.70	1	2	4	3
g	9	26	22.0	2	19.0	3	3.0	1	5.5	3	40	7	60	4	0	1	11.4	6	1.92	1	2.70	1	2	4	0
p	10	27	23.0	2	18.0	3	5.0	1	3.0	1	26	5	56	4	18	5	10.0	4	1.90	1	3.00	1	4	4	3
p	10	28	23.0	2	18.0	3	5.0	1	3.0	1	26	5	56	4	18	5	10.0	4	1.90	1	3.10	1	4	4	0
p	10	29	23.0	2	18.0	3	5.0	1	3.0	1	26	5	56	4	18	5	10.0	4	1.90	1	3.10	1	0	5	0
l	11	30	23.0	2	15.0	2	8.0	2	4.5	2	40	7	60	4	0	1	13.5	8	1.83	1	1.80	0	2	4	0
l	11	31	23.0	2	15.0	2	8.0	2	4.5	2	40	7	60	4	0	1	13.5	8	1.83	1	1.80	0	3	3	0
l	11	32	23.0	2	15.0	2	8.0	2	4.5	2	40	7	60	4	0	1	13.5	8	1.83	1	1.95	0	3	3	3
d	12	33	23.0	2	18.0	3	5.0	1	3.8	2	43	7	57	4	0	1	11.5	6	2.00	1	2.90	1	2	4	3
d	12	34	23.0	2	18.0	3	5.0	1	3.8	2	43	7	57	4	0	1	11.5	6	2.00	1	2.90	1	4	6	0
d	12	35	23.0	2	18.0	3	5.0	1	3.8	2	43	7	57	4	0	1	11.5	6	2.00	1	2.90	1	2	4	0
d	12	36	23.0	2	18.0	3	5.0	1	3.8	2	43	7	57	4	0	1	11.5	6	2.00	1	2.90	1	0	6	0
d	12	37	23.0	2	18.0	3	5.0	1	3.8	2	43	7	57	4	0	1	11.5	6	2.00	1	2.90	1	0	6	0
f	13	38	23.0	2	22.0	4	1.0	1	6.0	3	46	7	48	2	6	3	9.8	4	1.73	0	1.00	0	0	6	3
f	13	39	23.0	2	22.0	4	1.0	1	6.0	3	46	7	48	2	6	3	9.8	4	1.73	0	1.10	0	4	2	0
f	13	40	23.0	2	22.0	4	1.0	1	6.0	3	46	7	48	2	6	3	9.8	4	1.73	0	1.10	0	0	6	0
1	14	41	24.0	2	20.0	4	4.0	1	1.0	1	14	2	88	10	0	1	7.7	2	2.00	1	4.10	1	0	6	3
1	14	42	24.0	2	20.0	4	4.0	1	1.0	1	14	2	88	10	0	1	7.7	2	2.00	1	4.10	1	0	4	0
1	14	43	24.0	2	20.0	4	4.0	1	1.0	1	14	2	88	10	0	1	7.7	2	2.00	1	4.10	1	0	6	0
o	15	44	24.0	2	21.0	4	3.0	1	2.0	1	33	6	58	4	9	4	10.8	5	1.75	0	2.30	1	2	4	3

o	15	45	24.0	2	21.0	4	3.0	1	2.0	1	33	6	58	4	9	4	10.8	5	1.75	0	2.30	1	0	6	0
o	15	46	24.0	2	21.0	4	3.0	1	2.0	1	33	6	58	4	9	4	10.8	5	1.75	0	2.30	1	2	4	0
d	16	47	25.0	2	12.0	1	13.0	3	5.5	3	19	3	51	3	30	6	10.1	5	1.67	0	1.86	0	5	0	0
d	16	48	25.0	2	12.0	1	13.0	3	5.5	3	19	3	51	3	30	6	10.1	5	1.67	0	1.86	0	6	0	0
d	16	49	25.0	2	12.0	1	13.0	3	5.5	3	19	3	51	3	30	6	10.1	5	1.67	0	1.90	0	6	0	3
d	16	50	25.0	2	12.0	1	13.0	3	5.5	3	19	3	51	3	30	6	10.1	5	1.67	0	2.70	1	0	6	0
k	17	51	25.0	2	21.0	4	4.0	1	2.0	1	30	5	63	5	7	3	7.9	2	1.98	1	4.10	1	0	6	3
k	17	52	25.0	2	21.0	4	4.0	1	2.0	1	30	5	63	5	7	3	7.9	2	1.98	1	4.10	1	0	5	0
k	17	53	25.0	2	21.0	4	4.0	1	2.0	1	30	5	63	5	7	3	7.9	2	1.98	1	4.10	1	0	6	0
p	18	54	25.0	2					5.0	2	40	7	60	4	0	1					1.85	0	4	2	0
1	19	55	26.0	3	19.0	3	7.0	2	4.0	2	30	5	70	6	0	1	12.7	7	1.86	1	2.30	1	3	3	0
8	19	56	26.0	3	19.0	3	7.0	2	4.0	2	30	5	70	6	0	1	12.7	7	1.86	1	2.30	1	3	3	3
8	19	57	26.0	3	19.0	3	7.0	2	4.0	2	30	5	70	6	0	1	12.7	7	1.86	1	2.30	1	0	5	0
4	20	58	27.0	3	16.0	2	12.0	3	6.0	3	34	6	63	5	3	2	8.7	3	1.93	1	2.05	1	3	3	0
4	20	59	27.0	3	16.0	2	12.0	3	6.0	3	34	6	63	5	3	2	8.7	3	1.93	1	2.05	1	3	3	3
4	20	60	27.0	3	16.0	2	12.0	3	6.0	3	34	6	63	5	3	2	8.7	3	1.93	1	2.05	1	4	2	0
g	21	61	27.0	3	24.0	5	3.0	1			51	7	49	2	0	1	16.3	8	1.45	0	1.20	0	2	4	3
g	21	62	27.0	3	24.0	5	3.0	1			51	7	49	2	0	1	16.3	8	1.45	0	1.20	0	4	2	0
g	21	63	27.0	3	24.0	5	3.0	1			51	7	49	2	0	1	16.3	8	1.45	0	1.20	0	2	4	0
g	22	64	28.0	3	19.0	3	9.0	2	5.5	3	22	4	60	1	18	5	6.0	1	2.15	1	3.30	1	2	2	0
g	22	65	28.0	3	19.0	3	9.0	2	5.5	3	22	4	60	1	18	5	6.0	1	2.15	1	3.30	1	2	2	3
g	22	66	28.0	3	19.0	3	9.0	2	5.5	3	22	4	60	1	18	5	6.0	1	2.15	1	3.30	1	2	4	0
r	23	67	28.0	3	19.0	3	9.0	2	4.5	2	24	4	70	6	6	3	12.8	7	1.84	1	2.80	1	4	4	3
r	23	68	28.0	3	19.0	3	9.0	2	4.5	2	24	4	70	6	6	3	12.8	7	1.84	1	2.93	1	2	5	0
r	23	69	28.0	3	19.0	3	9.0	2	4.5	2	24	4	70	6	6	3	12.8	7	1.84	1	2.93	1	4	4	0
3	24	70	29.0	3	16.0	2	13.0	3	7.0	3	5	1	33	1	62	7	9.7	4	2.01	1	2.00	1	2	4	0
3	24	71	29.0	3	16.0	2	13.0	3	7.0	3	5	1	33	1	62	7	9.7	4	2.01	1	2.00	1	4	0	0
3	24	72	29.0	3	16.0	2	13.0	3	7.0	3	5	1	33	1	62	7	9.7	4	2.01	1	2.20	1	2	4	3
a	25	73	29.0	3	20.0	4	9.0	2	4.0	2	20	3	68	6	12	4	12.6	7	1.61	0	1.95	0	0	6	3

a	25	74	29.0	3	20.0	4	9.0	2	4.0	2	20	3	68	6	12	4	12.6	7	1.61	0	2.00	1	2	4	0
a	25	75	29.0	3	20.0	4	9.0	2	4.0	2	20	3	68	6	12	4	12.6	7	1.61	0	2.00	1	0	6	0
a	25	76	29.0	3	20.0	4	9.0	2	4.0	2	20	3	68	6	12	4	12.6	7	1.61	0	2.05	1	0	6	0
2	26	77	29.0	3	12.0	1	17.0	4	8.7	4	25	4	75	7	0	1	12.3	7	1.99	1	1.98	0	0	6	3
2	26	78	29.0	3	12.0	1	17.0	4	8.7	4	25	4	75	7	0	1	12.3	7	1.99	1	2.96	1	5	0	0
2	26	79	29.0	3	12.0	1	17.0	4	8.7	4	25	4	75	7	0	1	12.3	7	1.99	1	2.96	1	0	6	0
h	27	80	29.0	3	18.0	3	11.0	3	4.8	2	28	5	52	3	20	5	10.1	5	1.90	1	4.60	1	2	4	3
h	27	81	29.0	3	18.0	3	11.0	3	4.8	2	28	5	52	3	20	5	10.1	5	1.90	1	4.60	1	6	0	0
h	27	82	29.0	3	18.0	3	11.0	3	4.8	2	28	5	52	3	20	5	10.1	5	1.90	1	4.60	1	2	4	0
t	28	83	29.0	3	22.0	4	7.0	2	4.0	2	29	5	49	2	23	6	8.1	3	1.97	1	3.25	1	0	6	3
t	28	84	29.0	3	22.0	4	7.0	2	4.0	2	29	5	49	2	23	6	8.1	3	1.97	1	3.25	1	0	5	0
t	28	85	29.0	3	22.0	4	7.0	2	4.0	2	29	5	49	2	23	6	8.1	3	1.97	1	3.25	1	0	6	0
p	29	86	29.0	3	14.0	1	15.0	3	7.0	3	32	6	50	2	18	5	11.4	6	1.69	0	2.20	1	0	6	0
2	30	87	29.0	3	12.0	1	17.0	4	9.0	4	34	6	66	6	0	1	11.2	6	2.09	1	1.82	0	6	2	0
2	30	88	29.0	3	12.0	1	17.0	4	9.0	4	34	6	66	6	0	1	11.2	6	2.09	1	1.82	0	0	6	0
2	30	89	29.0	3	12.0	1	17.0	4	9.0	4	34	6	66	6	0	1	11.2	6	2.09	1	2.00	1	0	6	3
p	31	90	29.0	3	14.0	1	15.0	3	6.2	3	52	7	30	1	18	5	11.4	6	1.69	0	2.62	1	4	0	3
p	31	91	29.0	3	14.0	1	15.0	3	6.2	3	52	7	30	1	18	5	11.4	6	1.69	0	2.62	1	4	0	0
e	32	92	29.0	3	22.0	4	7.0	2	4.5	2	53	7	39	1	8	3	18.5	8	1.78	0	2.20	1	0	4	3
e	32	93	29.0	3	22.0	4	7.0	2	4.5	2	53	7	39	1	8	3	18.5	8	1.78	0	2.20	1	2	4	0
e	32	94	29.0	3	22.0	4	7.0	2	4.5	2	53	7	39	1	8	3	18.5	8	1.78	0	2.20	1	0	4	0
b	33	95	30.0	3	18.0	3	12.0	3	5.0	2	23	4	60	4	17	5	10.1	5	1.97	1	3.00	1	4	4	0
b	33	96	30.0	3	18.0	3	12.0	3	5.0	2	23	4	60	4	17	5	10.1	5	1.97	1	3.00	1	4	4	3
b	33	97	30.0	3	18.0	3	12.0	3	5.0	2	23	4	60	4	17	5	10.1	5	1.97	1	3.00	1	4	0	0
1	34	98	30.0	3	14.0	1	16.0	4	9.0	4	32	6	50	2	18	5	9.4	4	1.83	1	3.00	1	2	4	3
1	34	99	30.0	3	14.0	1	16.0	4	9.0	4	32	6	50	2	18	5	9.4	4	1.83	1	3.00	1	6	2	0
1	34	100	30.0	3	14.0	1	16.0	4	9.0	4	32	6	50	2	18	5	9.4	4	1.83	1	3.00	1	2	4	0
m	35	101	31.0	4	25.0	5	6.0	2	4.0	2	22	4	42	1	36	6	7.3	2	1.94	1	3.80	1	0	6	3
m	35	102	31.0	4	25.0	5	6.0	2	4.0	2	22	4	42	1	36	6	7.3	2	1.94	1	3.80	1	0	5	0

m	35	103	31.0	4	25.0	5	6.0	2	4.0	2	22	4	42	1	36	6	7.3	2	1.94	1	3.80	1	0	6	0
2	36	104	31.0	4	14.0	1	17.0	4	10.0	5	23	4	77	8	0	1	8.8	3	2.12	1	3.90	1	5	0	0
2	36	105	31.0	4	14.0	1	17.0	4	10.0	5	23	4	77	8	0	1	8.8	3	2.12	1	3.90	1	0	6	0
2	36	106	31.0	4	14.0	1	17.0	4	10.0	5	23	4	77	8	0	1	8.8	3	2.12	1	4.00	1	0	6	3
1	37	107	32.0	4	17.0	3	6.0	2	3.7	2	18	3	82	9	0	1	6.9	1	1.82	1	2.95	1	6	2	0
1	37	108	32.0	4	17.0	3	6.0	2	3.7	2	18	3	82	9	0	1	6.9	1	1.82	1	2.95	1	0	6	0
1	37	109	32.0	4	17.0	3	6.0	2	3.7	2	18	3	82	9	0	1	6.9	1	1.82	1	3.00	1	0	6	3
g	38	110	32.0	4	12.0	1	20.0	4	8.0	4	20	3	73	7	7	3	8.2	3	1.96	1	1.93	0	0	6	3
g	38	111	32.0	4	12.0	1	20.0	4	8.0	4	20	3	73	7	7	3	8.2	3	1.96	1	1.93	0	3	0	0
g	38	112	32.0	4	12.0	1	20.0	4	8.0	4	20	3	73	7	7	3	8.2	3	1.96	1	1.93	0	0	6	0
g	39	113	32.0	4	24.0	5	8.0	2	4.0	2	30	5	70	6	0	1	9.3	4	1.92	1	2.15	1	3	3	0
9	39	114	32.0	4	24.0	5	8.0	2	4.0	2	30	5	70	6	0	1	9.3	4	1.92	1	2.15	1	0	5	0
9	39	115	32.0	4	24.0	5	8.0	2	4.0	2	30	5	70	6	0	1	9.3	4	1.92	1	2.20	1	3	3	3
c	40	116	32.0	4	17.0	3	15.0	3	9.0	4	31	6	62	5	7	3	13.0	7	1.95	1	3.10	1	4	4	3
c	40	117	32.0	4	17.0	3	15.0	3	9.0	4	31	6	62	5	7	3	13.0	7	1.95	1	3.10	1	3	0	0
c	40	118	32.0	4	17.0	3	15.0	3	9.0	4	31	6	62	5	7	3	13.0	7	1.95	1	3.10	1	4	4	0
d	41	119	32.0	4	13.0	1	19.0	4	9.0	4	32	6	64	5	4	2	8.8	3	1.69	0	1.98	0	6	0	0
d	41	120	32.0	4	13.0	1	19.0	4	9.0	4	32	6	64	5	4	2	8.8	3	1.69	0	1.98	0	2	4	0
d	41	121	32.0	4	13.0	1	19.0	4	9.0	4	32	6	64	5	4	2	8.8	3	1.69	0	2.30	1	2	4	3
a	42	122	33.0	4	12.0	1	21.0	5	10.2	5	35	6	65	5	0	1	16.0	8	1.81	1	2.45	1	2	6	3
a	42	123	33.0	4	12.0	1	21.0	5	10.2	5	35	6	65	5	0	1	16.0	8	1.81	1	2.45	1	6	0	0
a	42	124	33.0	4	12.0	1	21.0	5	10.2	5	35	6	65	5	0	1	16.0	8	1.81	1	2.45	1	2	6	0
d	43	125	33.5	4	18.0	3	6.5	2	4.0	2	7	1	93	10	0	1	5.4	1	1.70	0	2.01	1	0	6	3
d	43	126	33.5	4	18.0	3	6.5	2	4.0	2	7	1	93	10	0	1	5.4	1	1.70	0	2.01	1	5	0	0
d	43	127	33.5	4	18.0	3	6.5	2	4.0	2	7	1	93	10	0	1	5.4	1	1.70	0	2.01	1	0	6	0
8	44	128	35.0	4	27.0	6	8.0	2	3.0	1	13	2	83	9	4	2	8.2	3	2.21	1	4.20	1	3	3	0
8	44	129	35.0	4	27.0	6	8.0	2	3.0	1	13	2	83	9	4	2	8.2	3	2.21	1	4.20	1	3	3	3
8	44	130	35.0	4	27.0	6	8.0	2	3.0	1	13	2	83	9	4	2	8.2	3	2.21	1	4.20	1	0	4	0
h	45	131	35.0	4	15.0	2	20.0	4	9.0	4	25	4	65	5	10	4	11.7	6	1.65	0	1.60	0	2	4	0

h	45	132	35.0	4	15.0	2	20.0	4	9.0	4	25	4	65	5	10	4	11.7	6	1.65	0	2.30	1	5	0	0
h	45	133	35.0	4	15.0	2	20.0	4	9.0	4	25	4	65	5	10	4	11.7	6	1.65	0	2.30	1	0	6	0
h	45	134	35.0	4	15.0	2	20.0	4	9.0	4	25	4	65	5	10	4	11.7	6	1.65	0	2.60	1	0	6	3
3	46	135	35.0	4	21.0	4	14.0	2	5.0	2	29	5	71	7	0	1	8.0	2	2.10	1	2.10	1	0	6	3
3	46	136	35.0	4	21.0	4	14.0	2	5.0	2	29	5	71	7	0	1	8.0	2	2.10	1	2.27	1	0	6	0
3	46	137	35.0	4	21.0	4	14.0	2	5.0	2	29	5	71	7	0	1	8.0	2	2.10	1	2.27	1	5	0	0
e	47	138	35.0	4	29.0	6	6.0	3	5.0	2	29	5	63	5	8	3	8.8	3	2.04	1	3.21	1	2	4	0
e	47	139	35.0	4	29.0	6	6.0	3	5.0	2	29	5	63	5	8	3	8.8	3	2.04	1	3.21	1	0	5	0
e	47	140	35.0	4	29.0	6	6.0	3	5.0	2	29	5	63	5	8	3	8.8	3	2.04	1	3.30	1	2	4	3
4	48	141	35.0	4	17.0	3	18.0	4	7.0	3	34	6	54	3	12	4	10.9	5	1.89	1	2.87	1	0	6	0
g	49	142	36.0	5	17.0	3	19.0	4	8.0	4	20	3	34	1	46	7	7.0	1	1.90	1	2.80	1	6	0	0
g	49	143	36.0	5	17.0	3	19.0	4	8.0	4	20	3	34	1	46	7	7.0	1	1.90	1	2.80	1	0	6	0
g	49	144	36.0	5	17.0	3	19.0	4	8.0	4	20	3	34	1	46	7	7.0	1	2.15	1	2.90	1	0	6	0
g	49	145	36.0	5	17.0	3	19.0	4	8.0	4	20	3	34	1	46	7	7.0	1	1.90	1	3.00	1	0	6	3
d	50	146	36.4	5	19.0	3	18.0	4	8.5	4	23	4	77	8	0	1	9.4	4	1.92	1	2.02	1	0	6	3
d	50	147	36.4	5	19.0	3	18.0	4	8.5	4	23	4	77	8	0	1	9.4	4	1.92	1	2.02	1	5	0	0
d	50	148	36.4	5	19.0	3	18.0	4	8.5	4	23	4	77	8	0	1	9.4	4	1.92	1	2.02	1	0	6	0
b	51	149	37.0	5	16.0	2	21.0	5	6.5	3	32	6	31	1	37	6	12.4	7	1.90	1	4.90	1	6	2	0
b	51	150	37.0	5	16.0	2	21.0	5	6.5	3	32	6	31	1	37	6	12.4	7	1.90	1	4.90	1	2	6	0
b	51	151	37.0	5	16.0	2	21.0	5	6.5	3	32	6	31	1	37	6	12.4	7	1.90	1	5.00	1	2	6	3
7	52	152	37.0	5	14.0	1	23.0	5	11.0	5	34	6	66	6	0	1	10.6	5	1.47	0	1.50	0	2	4	0
1	53	153	38.0	5	23.0	5	14.0	3	5.7	3	15	2	85	9	0	1	7.9	2	1.93	1	2.00	1	0	6	3
1	53	154	38.0	5	23.0	5	14.0	3	5.7	3	15	2	85	9	0	1	7.9	2	1.93	1	2.00	1	6	0	0
1	53	155	38.0	5	23.0	5	14.0	3	5.7	3	15	2	85	9	0	1	7.9	2	1.93	1	2.00	1	0	6	0
g	54	156	38.0	5	17.0	3	20.0	4	9.7	5	16	3	72	7	12	4	10.5	5	1.82	1	1.88	0	0	6	3
g	54	157	38.0	5	17.0	3	20.0	4	9.7	5	16	3	72	7	12	4	10.5	5	1.82	1	1.88	0	4	0	0
g	54	158	38.0	5	17.0	3	20.0	4	9.7	5	16	3	72	7	12	4	10.5	5	1.82	1	1.88	0	0	6	0
6	55	159	38.0	5	17.0	3	21.0	5	10.8	5	33	6	60	4	7	3	9.1	4	1.78	0	2.30	1	2	4	0
a	56	160	39.0	5	25.0	5	14.0	3	6.9	3	27	5	65	5	8	3	12.0	6	1.86	1	3.00	1	0	6	0

l	57	161	40.0	5	20.0	4	20.0	4	9.0	4	6	1	94	10	0	1	7.1	2	1.98	1	1.61	0	6	2	0
l	57	162	40.0	5	20.0	4	20.0	4	9.0	4	6	1	94	10	0	1	7.1	2	1.98	1	1.61	0	0	6	0
l	57	163	40.0	5	20.0	4	20.0	4	9.0	4	6	1	94	10	0	1	7.1	2	1.98	1	2.00	1	0	6	3
7	58	164	40.0	5	15.0	2	24.0	5	11.0	5	17	3	83	9	0	1	9.2	4	1.97	1	3.98	1	0	6	0
7	59	165	40.0	5	18.0	3	23.0	5	9.5	5	33	6	67	6	0	1	7.8	2	1.92	1	2.99	1	2	4	0
7	60	166	40.0	5	17.0	3	23.0	5			37	7	52	3	11	4	10.9	5	1.78	0	2.30	1	2	4	0
g	61	167	41.0	6	21.0	4	20.0	4	11.0	5	23	4	77	8	0	1	19.0	8	1.70	0	1.69	0	0	6	3
g	61	168	41.0	6	21.0	4	20.0	4	11.0	5	23	4	77	8	0	1	19.0	8	1.70	0	1.69	0	4	0	0
g	61	169	41.0	6	21.0	4	20.0	4	11.0	5	23	4	77	8	0	1	19.0	8	1.70	0	1.69	0	0	6	0
8	62	170	41.0	6	14.0	1	27.0	6	13.8	6	43	7	52	3	5	2					2.90	1	2	4	0
d	63	171	42.0	6	23.0	5	19.0	4	7.8	4	30	5	65	5	5	2	9.3	4	1.44	0	2.30	1	0	6	3
d	63	172	42.0	6	23.0	5	19.0	4	7.8	4	30	5	65	5	5	2	9.3	4	1.44	0	2.30	1	5	0	0
d	63	173	42.0	6	23.0	5	19.0	4	7.8	4	30	5	65	5	5	2	9.3	4	1.44	0	2.30	1	0	6	0
7	64	174	42.0	6	20.0	4	22.0	5	8.0	4	30	5	55	3	15	5	9.7	4	1.86	1	2.79	1	0	6	0
8	65	175	42.0	6	15.0	2	27.0	6	6.0	3	33	6	67	6	0	1	12.2	7	1.98	1	2.96	1	2	6	0
d	66	176	43.3	6	15.0	2	28.0	6	13.5	6	10	1	90	10	0	1	16.5	8	1.76	0	1.82	0	0	6	0
l	67	177	43.5	6	15.8	2	27.7	6	12.0	6	23	4	77	8	0	1					3.60	1	0	6	0
7	68	178	44.0	6	22.0	4	23.0	5	14.0	6	24	4	53	3	23	6	8.7	3	2.01	1	3.20	1	0	6	0
7	69	179	44.0	6	18.0	3	26.0	6	11.5	6	33	6	62	5	5	2	8.9	3	1.79	0	2.25	1	2	4	0
9	70	180	45.0	6	12.0	1	33.0	7	15.0	6	10	1	90	10	0	1	14.8	8	1.76	0	1.98	0	0	6	0
e	71	181	45.0	6	26.0	6	19.0	4	9.0	4	34	6	58	4	8	3	10.5	5	1.82	1	1.75	0	2	4	0
e	71	182	45.0	6	26.0	6	19.0	4	9.0	4	34	6	58	4	8	3	10.5	5	1.82	1	2.45	1	6	2	0
e	71	183	45.0	6	26.0	6	19.0	4	9.0	4	34	6	58	4	8	3	10.5	5	1.82	1	2.45	1	4	2	0
e	71	184	45.0	6	26.0	6	19.0	4	9.0	4	34	6	58	4	8	3	10.5	5	1.82	1	2.50	1	4	2	3
j	72	185	45.0	6	27.0	6	18.0	4	8.5	4	35	6	52	3	13	5	12.5	7	1.82	1	1.90	0	2	4	0
j	72	186	45.0	6	27.0	6	18.0	4	8.5	4	35	6	52	3	13	5	12.5	7	1.82	1	4.10	1	2	5	0
j	72	187	45.0	6	27.0	6	18.0	4	8.5	4	35	6	52	3	13	5	12.5	7	1.82	1	4.10	1	2	5	3
j	72	188	45.0	6	27.0	6	18.0	4	8.5	4	35	6	52	3	13	5	12.5	7	1.82	1	4.10	1	6	2	0
9	73	189	46.0	7	18.0	3	27.0	6	14.0	6	8	1	92	10	0	1					4.00	1	0	6	0

e	74	190	46.0	7	16.0	2	30.0	6	9.5	5	12	2	71	7	17	5	14.8	8	1.52	0	1.80	0	0	6	0
e	74	191	46.0	7	16.0	2	30.0	6	9.5	5	12	2	71	7	17	5	14.8	8	1.52	0	1.82	0	4	4	0
g	75	192	46.0	7	17.0	3	29.0	6	13.5	6	16	3	84	9	0	1					2.90	1	0	6	0
d	76	193	49.0	7	15.0	2	33.0	7	11.5	6	14	2	86	10	0	1					3.10	1	0	6	0
7	77	194	50.0	7	24.0	5	26.0	6	15.5	6	13	2	87	10	0	1					3.30	1	0	6	0
n	78	195	50.0	7	16.0	2	35.0	7	12.0	6	24	4	76	8	0	1					2.30	1	0	6	0
a	79	196	50.0	7	15.0	2	35.0	7	5.5	3	32	6	58	4	10	4	10.2	5	1.94	1	3.00	1	4	2	0
7	80	197	51.0	7	25.0	5	26.0	6	8.0	4	8	1	92	10	0	1	7.9	2	2.15	1	4.02	1	0	6	0
g	81	198	51.0	7	23.0	5	28.0	6			22	4	78	8	0	1					3.70	1	0	6	0
b	82	199	51.0	7	15.0	2	36.0	7	15.5	6	33	6	67	6	0	1					2.70	1	2	4	0
6	83	200	52.0	7	34.0	6	18.0	4	4.0	2	16	3	47	2	37	6	8.7	3	2.01	1	5.40	1	0	6	0
e	84	201	53.0	7	14.0	1	39.0	7	16.0	6	9	1	91	10	0	1					1.70	0	0	6	0
g	85	202	55.0	7	20.0	4	35.0	7	15.0	6	15	2	85	9	0	1	14.0	8	1.52	0	1.20	0	0	6	0
d	86	203	55.0	7	23.0	5	32.0	7			22	4	78	8	0	1	15.5	8	1.55	0	1.57	0	0	6	0
b	87	204	55.0	7	19.0	3	36.0	7	12.0	6	40	7	60	4	0	1					1.20	0	6	0	0
7	88	205	55.0	7	30.0	6	25.0	5	5.5	3	43	7	57	4	0	1	16.0	8	1.87	1	3.30	1	0	6	0
7	89	206	57.0	7	36.0	6	21.0	5	5.4	3	22	4	78	8	0	1	9.0	3	2.10	1	3.40	1	0	6	0
a	90	207	57.0	7	22.0	4	35.0	7	16.8	6	24	4	56	4	20	5					3.70	1	0	6	0
n	91	208	58.0	7	16.0	2	42.0	7	15.0	6	14	2	86	10	0	1	22.7	8	1.60	0	1.35	0	6	0	0
c	92	209	58.0	7	22.0	4	36.0	7	17.0	6	35	6	65	5	0	1					1.30	0	6	0	0
d	93	210	61.0	7	24.0	5	37.0	7	14.5	6	12	2	76	8	12	4					1.40	0	2	4	0
e	94	211	61.0	7	15.0	2	45.0	7	15.3	6	23	4	77	8	0	1					1.20	0	6	0	0
e	95	212	64.0	7	20.0	4	44.0	7	17.5	6	19	3	78	8	3	2					1.30	0	6	0	0
f	96	213	65.0	7	14.0	1	51.0	7	14.0	6	14	2	86	10	0	1	20.0	8	1.60	0	1.50	0	4	2	0
n	97	214	66.0	7	24.0	5	42.0	7	14.5	6	17	3	83	9	0	1	28.0	8	1.52	0	1.25	0	6	0	0
e	98	215	67.0	7	18.0	3	48.0	7	14.5	6	29	5	71	7	0	1					1.45	0	4	2	0
b	99	216	68.0	7	20.0	4	48.0	7	17.0	6	14	2	86	10	0	1					1.45	0	4	2	0
f	100	217	72.0	7	26.0	6	46.0	7	19.8	6	10	1	90	10	0	1					1.15	0	6	0	0
e	101	218	72.0	7	23.0	5	48.0	7	15.5	6	20	3	75	7	5	2					1.25	0	4	2	0

d	102	219	72.0	7	23.0	5	49.0	7	16.5	6	25	4	60	4	15	5	11.2	6	1.60	0	1.30	0	2	4	0
d	102	220	72.0	7	23.0	5	49.0	7	16.5	6	25	4	60	4	15	5	11.2	6	1.60	0	1.50	0	2	4	0
g	103	221	73.0	7	22.0	4	51.0	7	18.0	6	7	1	93	10	0	1					1.15	0	4	2	0
d	104	222	73.0	7	27.0	6	47.0	7	18.0	6	11	2	89	10	0	1					1.22	0	6	0	0
l	105	223	73.0	7	25.0	5	48.0	7	16.0	6	12	2	79	8	9	4	16.0	8	1.59	0	1.70	0	2	4	0
l	105	224	73.0	7	25.0	5	48.0	7	16.0	6	12	2	79	8	9	4	16.0	8	1.59	0	1.90	0	0	6	0
b	106	225	76.0	7	28.0	6	48.0	7	18.0	6	10	1	90	10	0	1					1.00	0	4	2	0
h	107	226	76.0	7	17.0	3	59.0	7	18.0	6	20	3	80	8	0	1					1.76	0	4	2	0
i	108	227	83.0	7	23.0	5	60.0	7	17.5	6	13	2	84	9	3	2					1.20	0	4	2	0
l	109	228	89.0	7	19.0	3	69.0	7			14	2	86	10	0	1	25.5	8	1.51	0	1.40	0	4	2	0
j	110	229	90.0	7	25.0	5	65.0	7	17.5	6	15	2	85	9	0	1					1.65	0	4	2	0
n	111	230	95.0	7	26.0	6	70.0	7	16.0	6	9	1	83	9	8	3					1.45	0	4	2	0

APPENDIX 2

CHAPTER 4 SUPPORTING FIGURES AND TABLES

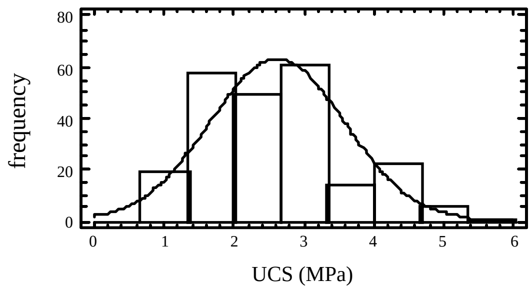

Figure 4.1a: Histogram and estimated population curve based on the sample data for compressive strength.

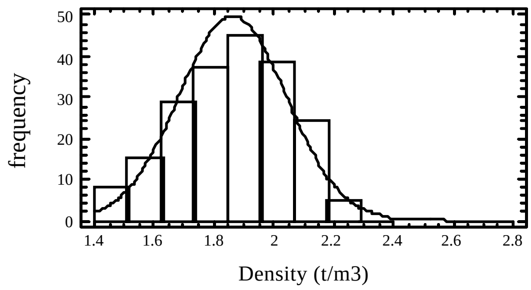

Figure 4.1b: Histogram and estimated population curve based on the sample data for density.

Source	**Sum of squares**	**DF**	**Mean square**	**F-ratio**	**P-value**
Model	55.7996	17	3.38233	5.22	0.000
Residual	110.025	175	0.628714		
Total	165.825	192			

Table 4.5: Results of fitting a general linear statistical model relating compressive strength to 9 predictive variables using ANCOVA. Since the p-value is less than 0.000, there is a statistically significant relationship between compressive strength and the predictor variables at better than the 99.9 % confidence level.

Source	Sum of squares	DF	Mean square	F-ratio	P-value
Model	145.55	57	2.554	49.83	0.000
Residual	6.098	119	0.051		
Total	151.648	176			

Table 4.10: Results of analysis of variance of compressive strength by same-sample determinations. There is a signficant difference between the mean values of compressive strength between samples at better than the 99.9 % confidence level.

Source	Sum of squares	DF	Mean square	F-ratio	P-value
Model	51.2198	6	8.5366	13.85	0.000
Residual	114.605	186	0.61616		
Total	165.83	192			

Table 4.11: Analysis of variance for a multiple regression model relating compressive strength to 6 predictive variables. Since the p-value is less than 0.000, there is a statistically significant relationship between compressive strength and the predictor variables at better than the 99.9 % confidence level.

Source	Sum of squares	DF	Mean square	F-ratio	P-value
Model	1.73645	17	0.102144	4.69	0.000
Residual	3.80977	175	0.021770		
Total	5.5462	192			

Table 4.13: Results of fitting a general linear statistical model relating density to 9 predictive variables using ANCOVA. Since the p-value is less than 0.000, there is a statistically significant relationship between density and the predictor variables at better than the 99.9 % confidence level.