DURABILITY TESTING OF FINE GRAINED STABILISED SOILS

MPhil Thesis

Nottingham University

Student ID Number: 4032733

Student: Craig Notman

Executive Summary

Lime and/or cement stabilised fine-grained soils have been successfully used in the construction industry throughout the UK since the early 1970's. Soil stabilisation has several economic, technical and environmental advantages. Although the vast majority of roads built upon stabilised soil foundations have resulted in durable pavements, a few case studies exist where expansive reactions have locally occurred, resulting in the requirement for extensive remedial works. Two high profile failures attributed to the expansion of stabilised capping layers were the M40 Banbury IV contract and the more recently constructed A10 Wadesmill Bypass. Both were Department of Transport (DoT) contracts in which the California Bearing Ratio (CBR) swell test was used as part of the quality control and/or investigation procedure. The Highways Agency (HA) and the Transport Research Laboratory (TRL) are still recommending the use of the CBR swell test as a means of determining a soils suitability for use within the stabilised process.

This thesis was undertaken at Nottingham University as part of an MPhil study programme conducted by Craig Notman. The main aspect of the research was to review the CBR swell test (B.S.1924-2: 1990) to determine its suitability as an appropriate laboratory test for assessing a soils volumetric change (as it is, and has been, previously recommended by the HA and the TRL).

The research focuses on the volumetric stability of stabilised soils, which require assessment under laboratory conditions. Various laboratory standards for determining the volumetric stability of stabilised soils were selected for comparative purposes. They included the CBR swell test (BS 1924-2: 1990), the European accelerated swelling test (BS EN 13286-49: 2004) and the loss of strength upon immersion test Manual of Contract Documents for Highways Works, Volume1 (MCHW1) Series 800 Clause 880.4.

When comparing the pass/fail criteria from the three test methods, all three resulted in differing recommendations. The research findings indicate that the pass/fail criterion of the CBR swell test (recommended by the HA74/00) is less stringent than the European accelerated swell test for the same material. That is, when assessing a material's suitability for stabilisation as a Capping material (foundation class 1: IAN73), the CBR swell test is more likely to deem a material suitable than if the European accelerated swell test was used. The loss of strength on immersion test is the most difficult pass/fail criterion to satisfy.

The author concludes that the BS 1924-1990 CBR swell test is an inappropriate test to be used as the sole determinant for the volumetric stability of stabilised soils (as recommended by TRL505), and that further research is required to develop appropriate guidance before this test is used again for assessing the volumetric stability of stabilised soils.

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MST Recycling Ltd – supplier of material

Lhoist – supplier of lime

Castle Cement – supplier of cement

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Glossary of Terms

Acid Soluble Sulfate

Normally expressed as % SO_4, and is a measure of sulfate in a material determined by acid extraction.

Capping Layer

Optional layer between the subgrade and subbase, which can be manufactured from stabilised soil or imported granular materials.

Cohesive Material

All material which by virtue of its clay content will form a coherent mass

CBR

California Bearing Ratio.

Chemical Durability

The resistance of a material to chemical reactions which results in a decrease in performance compared to its design performance.

Ettringite

Trisulfo calcium aluminate hydrate – formation of this mineral is generally accompanied by expansive forces due to the increase in volume associated with its crystallisation.

Hydraulic Binder

Material (or a combination of materials) that reacts with water to harden both under water and in air. These include cement, coal fly ash (known in the UK as Pulverized-Fuel Ash [PFA]), lime and granulated blast-furnace slag and factory produced hydraulic binders for roads known as hydraulic road binders.

Liquid Limit (W_L)

The liquid limit is the empirically established moisture content at which a soil passes from the liquid state to the plastic state. It is used with the plastic limit to determine the plasticity index of a soil which can then be used as a means of classifying a soil.

MCHW 1

Manual of Contract Documents for Highway Works: Volume 1 Specification for Highway Works.

MCV

Moisture condition value – relates the moisture content of a soil to the compactive effort required to give a defined level of site compaction of soils for use as fill materials.

MST

Mid Sussex Testing Services Ltd.

Normal Proctor

Laboratory reference density determined from the dry density/water content relationship obtained by the Proctor test with a specific energy of approximately 0.6 MJ/m^3.

Plasticity

The degree to which a material is physically malleable. The ability of a soil to undergo unrecoverable deformation at constant volume without cracking or crumbling. The upper and lower limits of the range of moisture contents over which a soil exhibits plastic behaviour are defined as the Liquid Limit (W_L), Plastic Limit (W_P) and the water content range itself as the Plasticity Index (I_P).

Plasticity Index (I_P)

The plasticity index is the empirical difference between the liquid and plastic limits of a soil.

$$I_P = W_L - W_P$$

Plastic Limit (W_P)

The plastic limit is the empirically established moisture content at which a soil becomes too dry to be plastic. It is used with the Liquid Limit to determine the plasticity index of a soil which can then be used as a means of classifying a soil.

Quicklime

Calcium oxide.

Softening

The process by which a material loses strength/stiffness and softens.

Swell

The process by which a material linearly expands, or increases in volume.

Subgrade

The made ground or naturally occurring soil that is found below roads.

Subbase

A layer of the pavement foundation.

Sulfates

Compounds or ions in which sulfur is in its oxidised state (i.e. $CaSO_4$).

Sulfides

Compounds or ions in which the element sulfur is in its reduced form (i.e. Fe_2S). They are easily oxidised when exposed to air.

Stabilisation

Incorporation of hydraulic binders into materials, thereby turning unsuitable or marginal materials into suitable construction materials. An operation that significantly enhances (generally in the medium or long-term) the characteristics of a material and renders it permanently stable.

Thaumasite

Hydrated calcium silicon carbonate sulfate hydroxide – formed from ettringite at low temperatures. It is associated with the deterioration of bound material due to its attack on the strength of cement pastes.

Total Potential Sulfate

Normally expressed as % SO_4, this is the total amount of sulfur available to form sulfate (SO_4) and is derived from the quantity of reduced and oxidised sulfur added together.

Total sulfate

Normally expressed as % SO_4, and is a measure of the total sulfate in a material.

Water soluble sulfate

Normally expressed as % SO_4, and is a measure of sulfate in a material that is soluble in water.

Workability

Duration of time counted from the end of the mixing, during which, the binder setting remains nil or very low.

1. Introduction

This thesis has been produced by Craig Notman as part of an MPhil study programme at The University of Nottingham, supervised by Dr Nick Thom.

In todays current economic and environmental climate, more and more pressures are put on Engineers to find suitable techniques wherever possible to re-use/recycle any locally available materials in order to reduce the costs of a project along with the impacts on the environment (i.e. reduce CO_2).

Soil stabilisation is a world wide ground improvement technique which is aimed at stabilising/strengthening local materials in order to create an environmentally friendly alternative to traditional dig and dump methods of construction, generally offering some economic and environmental advantages over traditional construction techniques. Although the vast majority of roads built upon stabilised soil foundations have resulted in durable pavements, a few case studies exist where expansive reactions have occurred within the stabilised layer causing swelling/softening of the layer, which resulted in costly and extensive remedial works. Against this background, the importance of an appropriate laboratory swell test, and associated guidance and acceptability criteria against which to assess the risk, were highlighted for further research.

The research focuses on the volumetric stability of stabilised soils, which requires assessment under laboratory conditions.

This project ran between May 2006 and April 2010.

1.1 Research Aims and Objectives

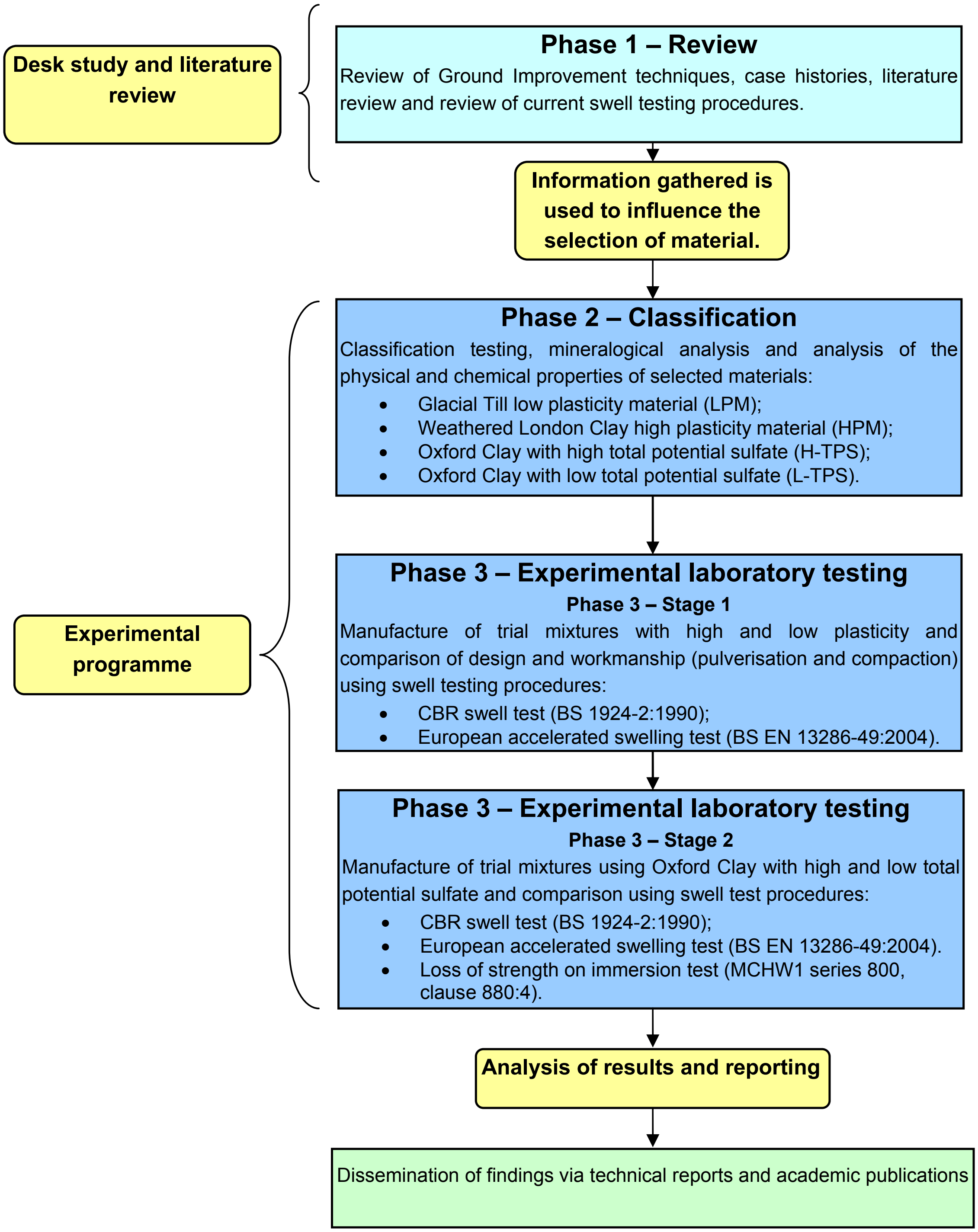
The aim of this project is to directly compare the California Bearing Ratio (CBR) swell test as set out in BS 1924-2: 1990 with the European accelerated swelling test (BS EN 13286-49: 2004), along with associated guidelines for use. This research also looks at other currently available test procedures, such as the loss of strength on immersion test (Manual of Contract Documents for Highways Works, Volume1 Series 800, Clause 880.4), specifically concentrating on fine grained stabilised soils.

The objectives of this project are:

- to review literature and case studies, and liaise with industry to establish current practice;
- to examine a range of materials with differing mineralogies and total potential sulfate contents;
- to compare the CBR swell test with the European accelerated swell test and the loss of strength on immersion test;
- to relate this testing to the current stabilisation process considering design versus screening and control, and;
- to disseminate the findings by inclusion within Highways Agency (HA) documents as appropriate and the production of industry guidance through Britpave and by academic publications.

1.2 Research Methodology

The research methodology comprises three phases as detailed below:



Phase 1 – Review

Review of Ground Improvement techniques, case histories, literature review and review of current swell testing procedures.

Information gathered is used to influence the selection of material.

Phase 2 – Classification

Classification testing, mineralogical analysis and analysis of the physical and chemical properties of selected materials:

- Glacial Till low plasticity material (LPM);
- Weathered London Clay high plasticity material (HPM);
- Oxford Clay with high total potential sulfate (H-TPS);
- Oxford Clay with low total potential sulfate (L-TPS).

Phase 3 – Experimental laboratory testing

Phase 3 – Stage 1

Manufacture of trial mixtures with high and low plasticity and comparison of design and workmanship (pulverisation and compaction) using swell testing procedures:

- CBR swell test (BS 1924-2:1990);
- European accelerated swelling test (BS EN 13286-49:2004).

Phase 3 – Experimental laboratory testing

Phase 3 – Stage 2

Manufacture of trial mixtures using Oxford Clay with high and low total potential sulfate and comparison using swell test procedures:

- CBR swell test (BS 1924-2:1990);
- European accelerated swelling test (BS EN 13286-49:2004).
- Loss of strength on immersion test (MCHW1 series 800, clause 880:4).

Analysis of results and reporting

Dissemination of findings via technical reports and academic publications

Phase 1: Review

To understand the background and current state of practice, a literature review is undertaken, relating to the various techniques/processes employed, historical data available, along with past and present specifications and best practice guidelines.

A summary of the pertinent points is given in the following sections. The review covers the background to soil stabilisation and current practice, outlines the volumetric expansion mechanism and the available standardised laboratory test procedures.

1.3 The Properties of Soil

Soil with the exception of peat is formed by the breakdown of rock masses, either by weathering or erosion.

The soils may accumulate in place, or undergo a certain amount of transport.

The soils characteristics may also have been effected by its geological past, i.e. being covered by ice, excessive heat, wind & rain, etc.

1.4 Problematic Soils

As there are many different types of soils, there can also be many different problems associated with each type of soil. Certain soils may shrink, expand, collapse or show a lack of strength/stiffness.

Soils are generally made up of a combination of up to four different groups of differing particle size:

Gravel = 60 – 2.0mm

Sand = 2.0 – 0.6mm

Silt = 0.06 – 0.002mm

Clay = <0.002mm

Clays themselves can be further divided into hundreds of different clay mineral types, of which three principle types are:

Group Designation	Minerals Included	Mean Size	Chief Physical Properties
Kaolin	kaolinite halloysite	~ 1μ	non-swelling, low plasticity, low cohesion
Illite	illite and partially degraded micas	~ 0.1μ	expansive, medium plasticity, low permeability
Smectite	montmorillonite, "bentonite" and mixed layer expansive clays	≤ 0.01μ	highly expansive, very plastic, extremely low permeability

1.5 Expansive Soils

Volume change in clay soils can be a major concern for the construction industry, because this change may result in substantial amounts of shrinkage (drying out) or swell (water absorption) which could cause damage to the highway, building or construction project in general.

It was estimated that in Britain in a typical 10year period between 1995-2005 swelling and shrinkage in clay soils had caused over 3 billion pounds worth of damage.

Volume change in soils is generally a function of its moisture content, density, void ratio, stresses applied or released, along with the internal soil structure and mineralogy.

The principle cause of expansion in natural soils is the presence of swelling clay minerals such as montmorillonite.

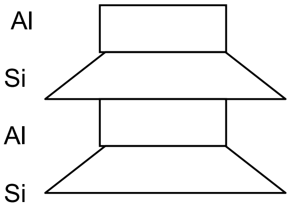
Grim (1962), set out two types of swelling characteristics in clay soils, namely intercrystalline and intracrystalline swelling.

Intercrystalline is when the uptake of water is restricted to the external crystal surfaces and the void spaces between the crystals.

Intracrystalline is when water enters not only between the crystals but also between the unit layers which comprise the crystals.

The different types of clay minerals present, dictate the materials capacity to take up (adsorb) water. Each clay mineral may have a similar structure to another clay mineral i.e. be made up of tetrahedral (Silicon atom-Si) and octahedral sheets (aluminium atom - Al), but it is the way that they are arranged that dictates their type:

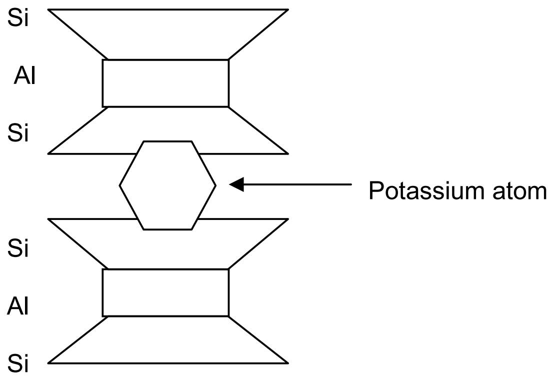
Kaolinite: No Inter layer (intracrystalline) swelling:



Surface area is typically 10-20m^2/gm

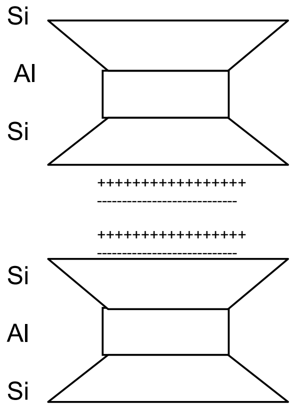
The bonding between the layers are Van der Waals forces (the sum of the attractive or repulsive forces between molecules) and hydrogen bonds, which are very strong and therefore they do not allow inter layer swelling.

Illite: Has a potassium atom which fits exactly into the hexagonal hole in the tetrahedral sheet and therefore forms a very strong inter layer bond. Illites may have lenses of water in the interlayer sites but not complete layers and do not show intracrystalline swelling.



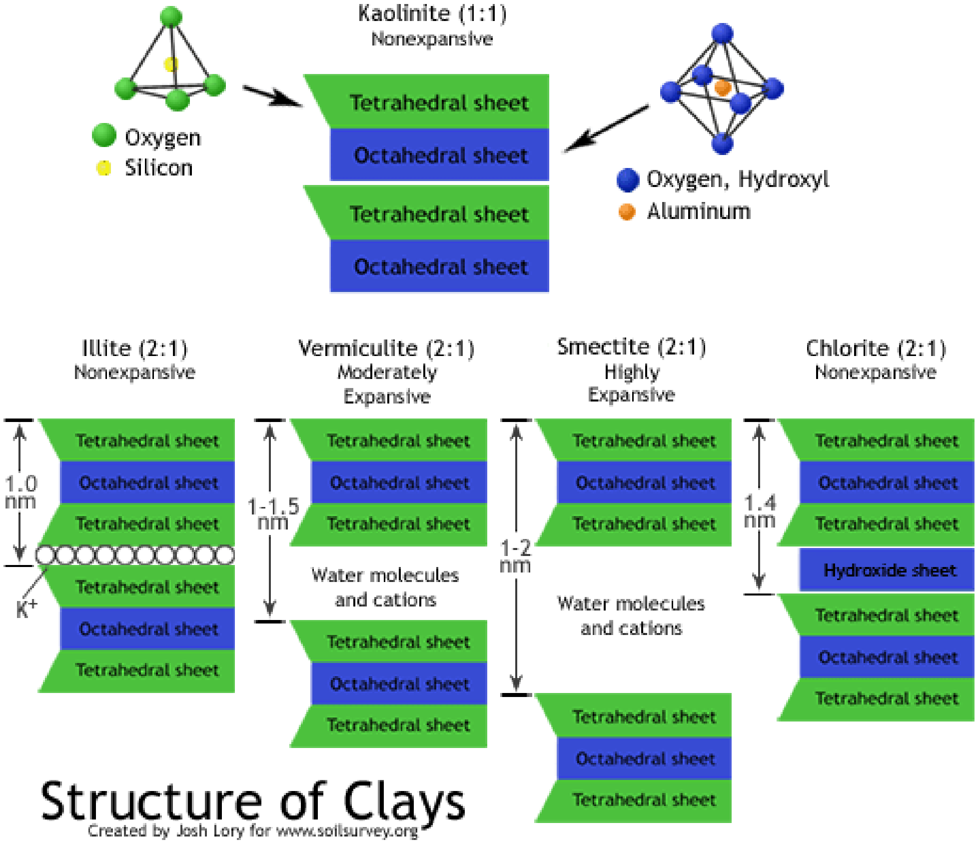
Surface area is typically 65-100m^2/gm

Montmorillinite: These particles can have a double layer (Gouy and Chapman theory) of charge surrounding the particle made up of water and ions which creates a very weak bond between the layers. Because of this attraction and repulsion of forces between the layers it creates a large surface area where cation exchange can happen which allows easy electron exchange and inter layer swelling.



Surface area is typically 50-120m^2/gm (external surface)

Surface area is typically 700-840m^2/gm (including the inter layer surface)



Bell & Cullshaw (2001), stated that generally kaolinite has the smallest swelling capacity, and nearly all of its swelling is of the intercrystalline type.

Illite may swell by 15%.

Intermixed illite and montmorillonite may swell by 60 – 100%.

Calcium (Ca) montmorillonite ranging between 50 – 100%

Sodium (Na) montmorillonite can amount to 2000% of the original volume, the clay then having formed a gel.

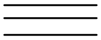
By adding lime to a potentially harmful clay such as Na-montmorillinte, the lime can sometimes reduce the swelling potential of the clay due to Ca^{2+} displacing Na^{+}, which can also result in an increase in the strength of the clay by dehydration and cementation.

The structure of a soil is taken to mean both the geometric arrangement of the particles or mineral grains, as well as the intracrystalline forces which may act between them.

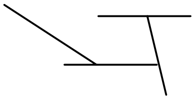
Soil fabric however, normally refers only to the geometric arrangement of the particles.

Inter particle arrangements of the fabric of the soil can be described in many different ways with particles having various arrangements as follows:

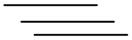
Face to Face:



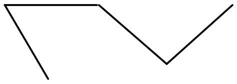
Edge to Face:



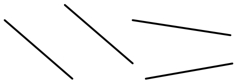
Shifted Face to Face:



Edge to Edge:



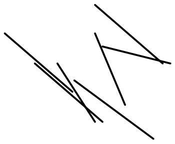
Dispersed Farbic:



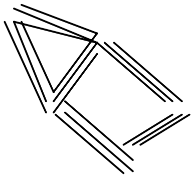
Soils can be composed of various mixed layers of clay minerals, normally made up of expanded water bearing layers and non water bearing layers which have been composed due to the interstratification and weathering over time.

The capacity of a material to take on water and soften and/or swell, can also be down to the elementary particle arrangements and void spaces.

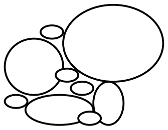
Individual clay arrangements:



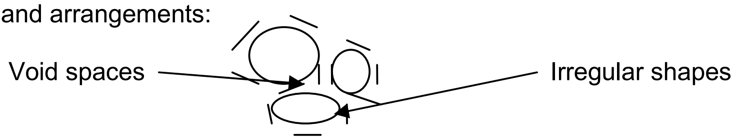
Clay platelet groups:



Individual silt or sand particles:



Clothed silt or sand arrangements:



Generally the swelling behaviour of a coarse grained soil depends on its particle size distribution, whereas the swelling behaviour of a fine grained soil can depend much more on its geological history and structure, rather than its particle size distribution.

In less dense soils expansion initially takes place three dimensionally into zones of looser soil (i.e. voids) before volumetric expansion takes place, (this could be seen as “softening” of the specimen/soil mass).

However in densely packed soil with low void space the soil mass has to swell more or less immediately to accommodate the volume change, and if confined will often swell in a one dimensional manner, i.e. upward, potentially causing serious damage.

It also follows that highly expansive clays, normally possess extremely low permeabilities, therefore moisture movement can be very slow, whereas moderately expansive clays with a smaller potential to swell, but with higher permeabilities may swell more during a single wet season (period) than the more expansive clays.

Therefore undisturbed expansive clay soils often have a high resistance to deformation and may be able to absorb significant amounts of swelling pressure, without swelling themselves.

It also follows that when the microstructural arrangement of an expansive clay is disturbed (re-mixed / re-moulded) during construction/testing, the expansion of the clay mass, may tend to swell more than that of their undisturbed counterparts.

As can be seen by some of the above comments, there are many reasons that may result in the volume change of a soil, without even considering at this point the chemical reasons.

It is therefore very important to have a good understanding of all the elements that can effect the swelling behaviour of a material in order to fully understand the engineering behaviour of each material before using it in construction.

The ingredients generally necessary for potentially damaging swelling to occur in natural soils are:

1 The presence of montmorillinite in the soil
2 There must be a source of water for the potentially swelling clay.

1.6 Volumetric Expansion (Formation of the Mineral Ettringite)

Other reasons for a soil to swell may be the presence of certain chemicals elements within the soil matrix which in the right conditions create crystalline structures and force the matrix apart as the crystals grow.

A common problem in uk soils is the presence of sulfate, which when mixed with, aluminas (a primary constituent of clay), calcium (from lime or cement) and water can create a highly expansive crystalline formation called ettringite (calcium-aluminate-sulphate-hydrate mineral). The following conditions must generally be met for this to occur:

1. a source of sulfates (including those derived from oxidation of sulfides);

2. the presence of mobile groundwater, and;

3. calcium hydroxide and calcium aluminium hydrate in the cementitious matrix.

Ettringite is a hydrous calcium alumino-sulfate mineral that precipitates in environments with high pH and sufficient sulfate concentration (Perkins et al, 1999). It forms in the early stages of hydration of calcium aluminate and has a large expansion potential, up to 250% (Britpave, 2005). Thaumasite attack is a secondary concern for stabilised soils and generally occurs as the pH decreases and at low temperatures.

Sulfur is widely distributed in the form of sulfides and sulfates.

Primarily the sulfides oxidise to form sulfate which can form aggressive (acidic) ground conditions, leading to chemical attack on construction materials such as steel and concrete.

When adding calcium oxide based binders such as lime and cement, a secondary sulfate based formation (ettringite or thaumastite) can occur which can cause ground heave.

Ettringite [$Ca_6Al_2(OH)_{12}(SO_4)_3.27H_2O$] is formed by the combination of soluble sulfates, from gypsum for example, and dissolved alumina, produced by the effect of the high pH associated with adding lime. The crystallisation of ettringite is expansive and, in addition, is subject to further large volume changes as it takes in water. Thaumasite ($Ca_3Si(CO_3)(SO_4)(OH)_6$) will also form from ettringite but without a change in volume.

P Sherwood (1993) explains the importance in understanding the chemical factors which are likely to affect Stabilised Soil.

"The reactions between sulfates and the hydrated silicates and alluminates lead to products that occupy a greater volume than the combined volume of the reacting constituents".

It is mentioned that with all forms of sulfate attack, water is an essential part of the reaction. And generally there will be insufficient water at the time of mixing to dissolve very much sulfate, so that unless extra water is able to enter the material no appreciable attack will occur, even if high concentrations of sulfate are present.

In addition to sulfates, sulfides in the form of iron pyrites may also be present in the soil, which when oxidised can form jarosite, or in the presence of calcium carbonate form gypsum. Both these formations are highly expansive, (Jarosite can form without the addition of calcium minerals, i.e. even if the material were not stabilised).

It is important therefore to test the material not just for sulfates, but also for sulfides, this is generally conducted by determining the total sulfur content of a soil and multiplying this result by 3. This then gives the approximate Total potential sulfate (TPS) content of the material, i.e.

if all the sulfides were oxidised. To then find the sulfide content you would need to test the original material for total sulfate content and simply subtract this from the total sulfur content.

The testing and investigation of soils for sufate and sulfide along with the acceptable limits recommended has been well documented in TRL447 and HA74/07 and will not be further discussed here.

1.7 The Principles of Ground Improvement

The principles of ground improvement are as the title suggests, to improve the ground in some way, so as to enhance the properties of the soil and therefore bring benefit to the contract in some way or another, be it financial, design life, ease of construction or durability, to name just a few.

Two forms of ground improvement technique are "**Modification**" and "**Stabilisation**".

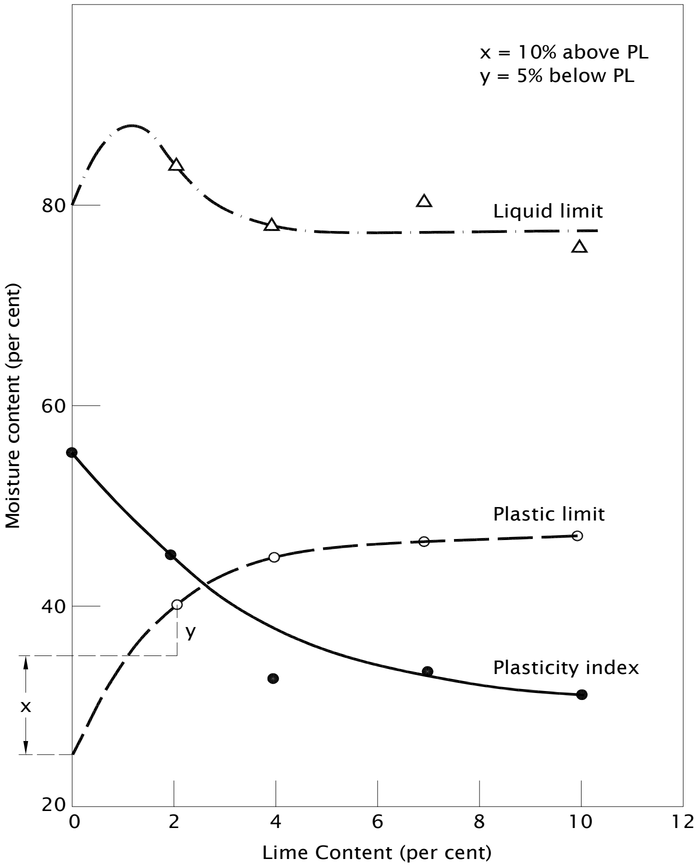
1.7.1 The Modification Process:

The Modification process can generally be described as an improvement of either the moisture content and/or the workability of the material.

This works in two ways, firstly, by adding quick lime (CaO) to a material it has a drying effect created due to the exothermic reaction generated as the lime hydrates with the free water available in or around the host material, therefore driving off a certain amount of water (often seen as an improvement). Care does need to be taken to ensure that the Lime is fully hydrated and that the material is at the correct moisture content for compaction (normally determined by lab tests to assess the dry density/moisture content relationship of the soil).

Secondly there can be an immediate change in the workability of the material caused by cation exchange which occurs when calcium ions released by the lime, exchange with metal ions within or on the clay structure. This cation exchange can increase the plastic limit(PL) and therefore in turn reduce the plasticity index (PI) which can result in a more friable and more workable material.

This plasticity and moisture change can typically change a materials workability in typical cases by 3% moisture for every 1% CaO added.



In the above example the soil was originally at 35% moisture content (i.e. 10% wet of its natural PL). After the addition of 2% lime assuming no loss of moisture due to exothermic heat exchange, the clay would now be 5% dry of its treated PL (40%), therefore creating a larger workable moisture content range.

This modification process is generally only used with the minimum amount of binder necessary, making a material which was deemed to be unsuitable (e.g. too wet), into a material which can be deemed suitable (i.e. now within its moisture range). There is no requirement for a permanent increase in the strength over and above that of the natural material in the modification process, even though some stabilisation/strengthening may take place in the short term as a result of this modification treatment. This initial strength gain may later be reversible as and when the material reaches its long term equilibrium moisture content.

1.7.2 Stabilisation

Stabilisation occurs when enough CaO is added to raise the pH sufficiently, typically to 12.4: (*Eades & Grim: 1966*), so that an environment is created which promotes the dissolution of silica and alumina from the clay particles which react in turn with the calcium ions from the lime to form calcium silicate hydrates (CSH) and calcium aluminate hydrates (CAH). These hydrates eventually crystallise into calcium silicate/alluminate hydrate which are broadly similar to those in cement.

This reaction is relatively slow from a cementing perspective, and it is generally known as a pozzolanic reaction (the word pozzolana comes from the Italian town of Pozzuoli where the Romans found a siliceous volcanic ash, which when mixed with lime forms a hydraulic binder). Pozzolanic materials are additives which contain practically no lime but have a high silica content such as pulverised fuel ash (PFA). As explained above, the chemical effects of lime stabilisation does compare with those found in cement, but due to the use of the un-refined silica and alumina's in the soil, it may give you a lower 28 day strength. However as the full long-term pozzolanic reaction can continue for several years, the long term strength of the material can equal that or even be higher than that of cement bound materials.

The stabilisation process can therefore continue for years as long as there is enough lime still available to react with the clay minerals.

Stabilisation of silts can be conducted with cement, but cannot generally be conducted with lime only, they require the further addition of a more pozzolanic material such as clay, PFA, or ground granulated blast furnace slag (GGBS) to create the cementing effect.

Stabilisation as with modification can change the workability of a material, but unlike modification it is primarily used to permanently enhance the strength/stiffness of the material as well.

1.7.3 Other Ground Improvement techniques:

Each ground improvement technique must be assessed on its merits for satisfying the end requirement for the job. For example stabilisation and modification do involve an element of earth moving and in certain built up area's piling, be it with concrete or lime/cement recycled materials, vibro stone columns and dynamic compaction may be more appropriate.

The size of the contract (i.e. inner city locations having a small footprint) may favour a piling or vibro stone columns, and or dynamic compaction, as apposed to dig, treat and re-lay methods of construction such as stabilisation and/or modification.

The existing materials on a contract might not suit vibro or dynamic techniques (such as made ground or silts (which could be sensitive and become unstable with vibration), once again these materials may suit stabilisation and/or piling instead.

These other forms of ground improvement techniques, are outside the scope of this paper.

1.8 Durability

Durability can be defined in many ways, such as, the resistance to weathering, erosion due to trafficking and construction, as well as the resistance to soil chemistry, just to name a few.
As can be seen from the previous sections on expansive soils, there are two very different reasons for swelling in clays, and they are:

1. The natural soils ability to take on (adsorb) water and therefore change its volume due to its minerality, particle size distribution, voids ratio etc, and these have been classed under the ***design durability*** section of this paper.
2. The materials ***chemical durability***/resistance to chemical reactions such as ettringite formation, which may result in a volume change and a decrease/increase in performance compared to its design durability/performance.

As with all construction materials there is generally a desire to upgrade a materials strength/stiffness and in turn hopefully increase its durability.

Durability testing can vary, and there are numerous methods which can be adopted to establish whether or not a material can be defined as being durable.

It is important to know that there are at least two totally different types of durability issues when stabilising a soil (*Chemical and Design durability issues*). These can be occurring within a soil, and can be occurring at the same time in different ways.

1.8.1 Chemical Durability:

Chemical durability can be defined as the resistance of a material to chemical reactions due to the presence of certain chemical elements (i.e. such as sulfates) which result in a decrease in performance compared to its design durability.

There are other chemical elements such as organic matter content which can affect the durability of a soil. Organic content can use up (neutralise) some of the lime content added to the soil, therefore in turn reducing the amount of available lime content available to react with the soil elements and potentially making the material weaker and less durable.

Chemical durability tests have been well documented and publicised in the format of DMRB Volume 4, section 1, Part 6, HA74/07 and TRL447:

1.8.2 Design Durability:

Design durability can be defined as the effects on a material due to workmanship or design elements (such as inadequate compaction, frost, poor choice of binder, etc).

There are many different design durability tests conducted for stabilised soils, and each one has been specifically designed to look at a certain element of the materials engineering behaviour to help make an assessment on its end use engineering characteristics. As with the ground improvement techniques, each test has its own "*raison d'etre*", therefore the engineer requesting the test must seriously consider the individual benefits and drawbacks involved with each test before choosing which one or group of tests to use.

Simple classification tests such as moisture content, particle size distribution, atterberg limits, etc, may not seem at first a group of design durability tests, however each one may in one way or another create a good understanding of the materials long term durability. As shown before, the plastic limit test can reflect a materials workability range of moisture contents, as well as indicating its state of equilibrium (i.e. dry or wet of its optimum long term moisture content, showing its stability or affinity to take on or lose moisture).

Each test along with the parameters expected from the results can be a useful tool for determining a material's durability. A material with a high clay content may be deemed susceptible to frost (i.e. the material may show signs of heave under certain weather conditions). The silt content of a material may also deem a contract unsuitable for a certain type of ground improvement technique such as the use of dynamic compaction, due to the materials instability and sensitivity to vibration.

There are hundreds of tests which could be discussed as having a beneficial use into aiding the engineer in their design choice of material or use. There are also many papers showing how a material may perform or reflect how a material may perform in relation to other materials, or in comparison to other tests or conditions. It is therefore crucial for engineers to have confidence in the design tools chosen, along with the appropriate parameters for each test and process considered.

The CBR swell test is currently being used as a design tool to assist engineers in deciding whether or not a stabilised material is suitable for use in certain area's of construction works. This test is deemed by the TRL as being a good tool for determining a materials chemical durability as well as assessing the materials design strength. It is with this in mind that the test has been selected for comparison against the only other swell test (European Accelerated Swelling test) considered for use by the Highways Agency, when assessing the durability of a stabilised soil.

There are many durability tests which could have been considered for use by the Highways Agency and some of them are described in brief as follows:

1.8.2.1 European Accelerated Swelling Test: BS EN 13286-49:2004

This standard requires a set of three number 50 mm diameter x 50 mm high specimens to be produced to 96±0.5% of the "Normal Proctor" wet density, manufactured using axial compression. The specimens must be manufactured using material passing the 6.3 mm sieve. They are then stored at 20±2 ^{o}C at more than 90% humidity for a period between 1.5 and 2 times the workability period of the mixture, then fully immersed for 168±4 h in water at 40±2 ^{o}C, prior to testing. The workability period of a material is that given in the producer's technical sheet, or that determined in accordance with BS EN 13286-45: 2003 - test method for the determination of the workability period of hydraulically bound mixtures. According to the MCHW Series 800, cement has a workability of approximately 2 hrs at 20^{o}C (or a period of 35 degree hours) from the time the cement was added. The degree hours are the summation of mean ambient air temperature over each hour period in degrees Celsius above 3^{o}C i.e. 2 x (20 - 3). The European accelerated swelling test is unconfined and allows expansion in all directions, as shown in Figure 1. As the manufactured specimen (shown in Figure 2) is smaller, it is also likely to require a shorter time period for saturation compared with a CBR specimen.

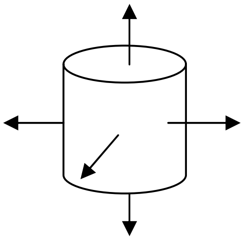


Figure 1. Test BS EN 13286-49:2004 allows expansion in all directions

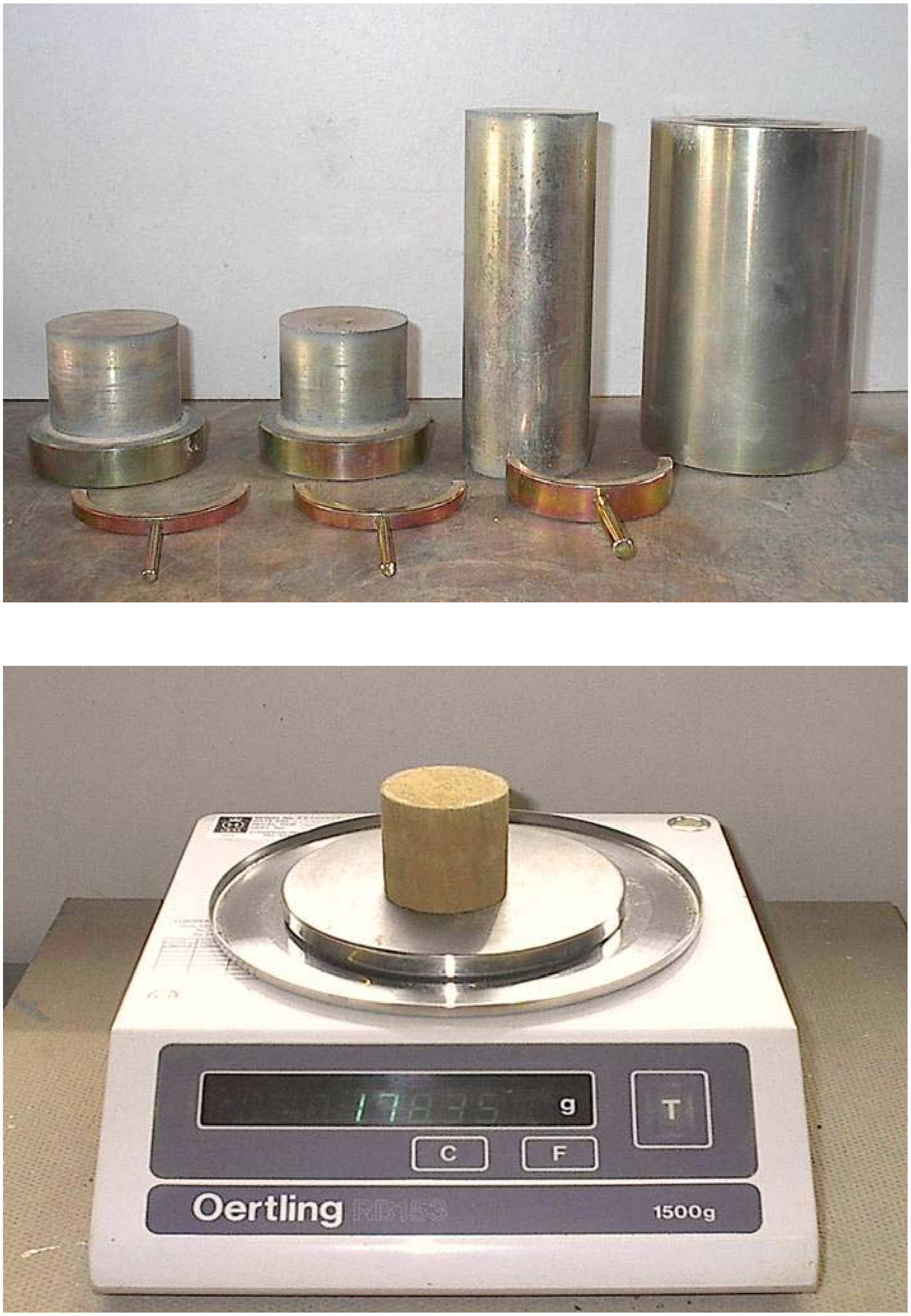


Figure 2. European accelerated swelling test compaction cylinders (above) specimen being weighed (below)

1.8.2.2 Laboratory Determination of the California Bearing Ratio: BS 1924-2:1990

A sample of stabilised material is compacted into a CBR mould (152 mm diameter × 127 mm high) in approximately three equal layers, using a 2.5 kg rammer (62 blows/layer), then sealed at 20±2 ^{o}C for three days.

The procedure also includes provisions for soaking the specimens, producing a soaked CBR value. This involves placing the specimens in a water bath at 20±2 ^{o}C. A collar is added to the top of the specimen and a perforated base plate is attached to the bottom to allow the ingress of water. The water level is kept just below the top of the collar (Figure 3).

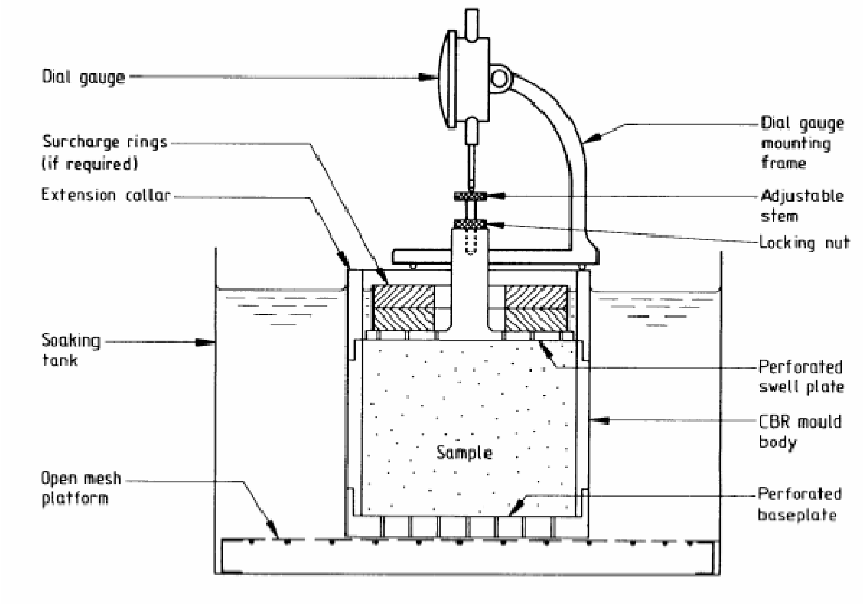


Figure 3. CBR Swell Test (BS 1924-2: 1990)

During immersion, water will flow into the sample due to capillary action. If after the first 3 days in the tank there is still little or no water at the top of the specimen, then water is added to the top of the specimen for the final 24 hrs prior to testing for strength. The standard curing regime is a 7 day CBR test and requires a 3 day air cure and a 4 day soak prior to testing. The 28 day swell test is cured in the same manner, but is soaked for 28 days instead of 4 days.

In HA74/00, there was no requirement for specimens to be tested for strength (by CBR); however this has been amended in HA74/07, which also now stipulates that the soaked CBR specimen should be tested for both strength and swell. The material suitability criteria set out in HA74/07 are summarised in Appendix A. The specimen is considered durable if the average CBR value is >15% with no individual result <8%, and the heave is on average ≤ 5 mm with no individual result >10 mm.

The heave that can occur in the CBR test procedure is principally a function of the availability of water to the specimen. A simplified model is shown in Figure 4, showing frictional forces (including the potential for material specific adhesion) which could resist the initial linear expansion. Water flow is also thought to be inhibited, as when the initial void space in the specimen is filled, the specimen can only take on further water if it expands. In addition, micro-cracks and faults which would allow water ingress under site conditions may not be present, due to the use of a well made and relatively small laboratory test specimen. It is important to consider these practical limitations in the context of associated guidance and limits.

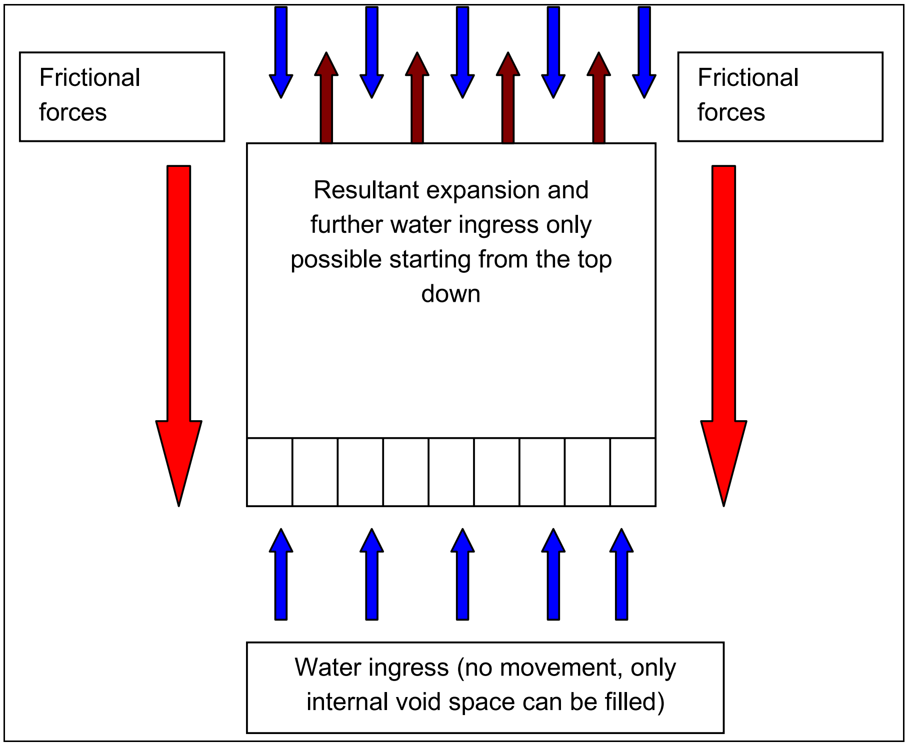


Figure 4. Simplified model of expansion and water flow during the CBR swell test

1.8.2.3 Loss of Strength on Immersion Test (Series 800, Clause 880.4, MCHW1)

The loss of strength on immersion test is defined in Series 800 (MCHW1, 2007) using the procedure given in Clause 880.4. Cylinders with a ratio of 1:1 (Height : Diameter) are prepared and cured for 14 days in air. They are then cured for a further 14 days immersed in water. The compressive strength of these immersed samples ($R_{C\ imm}$) is determined together with that of the control specimens ($R_{C\ control}$). The control specimens are cured for 28 days in a sealed condition. All curing is undertaken at 20 °C for the materials assessed in this project. The mixture is considered to be durable if the following applies:

$$R_{c\ vs} = \left(\frac{R_{c\ imm}}{R_{c\ control}} \right) \times 100 \ \geq 80\%$$

where:

$R_{c\ vs}$ is the relative volumetric stability (assumed to be durable if ≥ 80 %)

1.8.2.4 Frost Analysis: Tested in accordance with B.S.1924: 1990: Part 2.

The frost heave test has been designed to try and replicate what may actually happen to a material when subjected to low temperatures with a constant water source. UK roads normally have to be constructed using materials in the top 450mm (this may be varied dependent on the area’s frost index) of the pavement that are deemed to be non-susceptible to frost using the british standard test procedure: B.S.812-124. The test requires three test specimens 150mm high and 100mm diameter are to be placed vertically into a frost cabinet, which has a water source available at the base in order to allow the specimens to adsorb moisture therefore feeding the potential growth of frost lenses within the test specimen. If after 250hours the specimens have vertically expanded by more than 12mm they are deemed to be susceptible to frost, and therefore not allowed to be used within the frost susceptible layer.

1.8.2.5 Oedometer testing: Tested in accordance with B.S.1377: 1990: Part 5.

Along with the plasticity index, shrinkage limit and activity index, the Oedometer test can be used to predict the expansiveness of clay soils.

The test is generally used to measure the consolidation of a material (i.e. settlement) under a certain load. However it can be used to determine:

i) The “measurement of swelling pressure” (i.e. the vertical pressure on the specimen required to stop it from swelling.

ii) The “linear amount of swelling” (i.e. enabling the swelling characteristics of a laterally confined soil specimen to be measured when it is unloaded from the swelling pressure previously applied in the presence of water.

The Oedometer test and frost test are not tests that are currently being used by the Highways Agency or the TRL as an acceptability tool to measure a stabilised materials swelling behaviour. These tests may be very useful for determining certain aspects of a materials durability, however, they have not been chosen as tests requiring consideration for comparison in this paper.

1.8.2.6 Summary of Durability tests

Three standardised laboratory test methodologies taken from Series 800, and the test recommended in HA74/00 have been considered for the assessment of durability (specifically volumetric stability) of stabilised soils:

- CBR swell test (BS 1924-2: 1990);
- European Accelerated Swelling Test (BS EN 13286-49: 2004), and;
- loss of strength on immersion (Series 800, MCHW1, 2007).

The European CBR test method (BS EN 13286-47) adopted in HA74/07 has not been used in this study as the test procedure was only in draft format at the time the testing schedule was started, and the aim of the project was to investigate potential problems that have not been historically detected by the (then) currently used test procedure, (the BS EN 13286-47 CBR test method is discussed later in the final conclusions).

Recent experience would suggest that the soaked CBR test (BS 1924-2 1990) may not be suitable to highlight specific concerns. The alternative test methodologies are relatively untried in the UK for this specific application and there is currently a of lack experience to assess their suitability.

There have been numerous reports over the years where the CBR swell test has been directly compared to other tests as well as being used as a design acceptability tool, therefore if the CBR test method is being questioned by practitioners, it is crucial to future works and research that these questions are answered (see literature review conclusions for a summary of the questions seeking further clarification).

1.9 Historical Literature Review

The aim of the literature review was to target practitioners area's of concern which may be effecting the overall use and confidence in using stabilised materials in the construction industry. The uk industry leaders are seen to be the Highways Agency (HA), and due to fairly recent historical failures on a few Highways Agency contracts, questions are now being asked as to why the currently adopted specifications and testing are not high lighting the potential areas of risk prior to the commencement of the contract.

The Government via the Secretary of State for Transport employ the Highways Agency to operate and steward the maintenance of England's strategic road network. Together with the Department of Transport (DTp), the HA review, develop and set out specifications, and guidance for constructing these road schemes.

The HA employ academic research bodies such as the Transport for Research Laboratory (TRL), to investigate technical issues with a view to developing further the specifications and guidance used. Three of the main documents currently being employed by the HA and other uk practitioners for help and guidance in using stabilisation techniques are:

1. Design Manual for Roads and Bridges (DMRB), Volume 4; section 1; Highways Advice note; HA 74/00: Treatment of Fill and Capping materials using either Lime or Cement or both.

2. Manual of Contract Documents for Highway Works (MCHW); Volume 1; Specification for Highway Works(SHW); Series 600(Earthworks) and Series 800 (Road Pavements).

3. Transport for Research Laboratory (TRL) Report 505; Swell test requirements for Lime stabilised materials.

Each of these documents refer to various British Standards such as BS1924: 1990: Methods of test for cement-stabilised and lime-stabilised materials, and have been used or referred to before, during or after some of the Highway contracts today being questioned.

Since the 1950s thousands of miles of state highway and major airports have been stabilised in America, South Africa, and some European countries.
It wasn't until the early 1980s that lime/lime & cement stabilisation was really used in the UK, the British Airports Authority (BAA), chose to adopt a lime & cement process for stabilising their car-parks, taxiways and run-ways, and a lime stabilisation treatment for modifying the lake area to be used as their fire training ground. (*first trialled at Gatwick Airport, HEATH D C -1992. The application of lime and cement soil stabilization at BAA airports. Proceedings of the Institution of Civil Engineers (Transport), 95, February, 11-49)*

The effects of sulfates on stabilised materials had previously been investigated
(P T Sherwood 1962: the effects of sulphates on Cement and Lime stabilisation).

This paper describes the effects of sulfates and the mineralogy on lime or cement stabilised soils. One part of the paper looks at the effects of immersion and percentage clay fraction against sulfate content, and it concludes from this, that, under certain conditions lime or cement treated soils are more likely to disintegrate when the soil contains a high clay fraction when in the presence of sulfate solution.

It also shows that when the clay fraction is removed the risk of disintegration is also removed.

Throughout the paper cylindrical specimens were totally immersed in an unconfined state, and some were found to disintegrate whilst others did not. Each specimen performed as expected when subjected to an increase in sulfate solution.

However, interestingly, when the sulfate levels increased above 1% with some specimens, the amount of strength loss levelled off.

It was also concluded from this paper that some lime clay specimens disintegrated within a few days of immersion in sulfate solution, whereas cement treated sand specimens containing the same quantities of sulfates were un-affected, even after one years immersion.

It also states: “The degree of disintegration was proportional to the amount of clay present in the soil, and this is attributed to a reaction that occurs between clay and sulfate ions in the presence of lime and excess water”.

The paper suggests that it is the soil grain size that affects the durability and not the sulfate content. The larger the grain size, the better the durability.

M J Dumbleton (1962) “ Investigations to assess the potentialities of Lime for Soil Stabilisation in the UK” demonstrated that the effects of lime on plasticity vary with clay type, percentage lime addition, and time.

These findings were backed up by *Rogers & Glendinnings paper “Modification of Clay Soils using Lime”; Lime Stabilisation seminar at Loughborough University 1996.* They showed evidence that there are immediate changes as well as long term changes in the plasticity of materials over time (some quite considerable). It is suggested that these changes are also likely to be delayed due to the degree of pulverisation (or lack of it), involved when mixing on site.

In 1975 the Specification for Roads & Bridges (DoT SRBW:1975) did not include a specification for lime stabilisation. However it did have a specification for cement Stabilisation and it specified an upper limit of 0.25% total sulfate.

In 1986 the *Department of Transport Specification for Highway Works (Sixth Edition)* decided to drop the sulfate content as tool to control the use of cement stabilised materials, and put in its place was an immersion test. The immersion test was clearly set out in BS1924:1975, and is still being used today as the preferred suitability / durability test for hydraulically bound materials (MCHW 2005: series 800).

The basic principle of the test is as follows:
10 No. specimens (cubic or cylindrical) are prepared and made from the same lab mix. 5No. of these specimens are air-cured for 14 days prior to testing for unconfined compressive strength (UCS). The other 5No. specimens are air-cured for 7 days, then totally immersed (unconfined) in water for a further 7 days prior to testing for UCS. The soaked specimens have to achieve over 80% of the strength of that achieved by the un-soaked specimens. If they achieve this with no visual signs of deterioration, they are deemed to durable (All curing temps are set at 20^{o}C).
These specimens were deemed to be strong enough to handle after 7 days, and large enough to be representative of the mixed material.

The total sulfate test at this time was deemed too variable a test due to the possible sampling problems likely to be incurred whilst obtaining a small representative sample, i.e. only a 10g test portion taken from a large area of material likely to be considered.

Please note that the MCHW 2005; series 800 differs only from B.S.1924 (1975), in the respect that the curing periods have now been extended to 28days. (14air cure + 14days in soak)

Also in 1986 the *Department of Transport Specification for Highway Works (Sixth Edition)* included for the first time, a specification for the lime stabilisation of sub-grade materials for use as a capping. This first major step opened doors for lime stabilisation to be used throughout the UK on DoT contracts.

By the late 1980s there had been a couple of failures on major contracts,
A12 Saxmundham bypass *(Reported by Caerns & Noakes at the BACMI symposium 1988),* and the M40 Motorway Banbury IV contract *(Reported by Snedker &Page 43 of 95 - 43 - Temporal 1990, M40 Lime Stabilisation experiences; Snedker; Lime Stabilisation seminar at Loughborough University 1996).*

At the BACMI symposium questions were asked of the quality control checks conducted during the stabilisation process, which were unanswered therefore leaving a lot of doubt as to what created the heave. It was therefore recommended that on future works a more stringent quality control system be put in place to monitor such things as the spread rate of binder, the depth of mixing, the need for water to be added under the hood of the mixer, as well as the initial site levels. As can be seen from this symposium there was already a need for HA guidance. No soaked testing (either un-confined or confined) had been conducted.

Both these contracts raised questions into the viability of stabilising UK soils.

However when the report was written on the experiences of lime stabilisation on the M40 contract, some fairly key issues had been raised.

Initially, because of the very hot summer during the period that stabilisation was conducted, it had been hypothesised that there had been insufficient water available to slake the quicklime, and therefore during the investigation soil samples were prepared with high moisture deficit, mixed with quicklime, compacted and soaked in a CBR mould to simulate a very extreme case of delayed hydration. Although CBR swell tests were conducted they did not show any considerable amounts of swell.

It was noted during the investigation, that the specimens trialled, did not saturate throughout, as had occurred in the field. More water had been absorbed at the top of the specimen than at the bottom.

It was also noted that when the temperature of the curing tank fell below 15^{o}C (due to a power failure) each specimen showed a small but sudden increase in swell.

It was evident at the end of the tests that the conditions which had occurred in the field had not been replicated in the trialled laboratory CBR swell tests.

It was considered that neither the water access to the CBR specimen nor the field temperature changes had been replicated in the laboratory.

When summarised, the report showed that the geological history and mineralogy of the soil, had the potential to develop additional sulfate, due to the oxidation of sulfides, and that the earthworks program suited this development.

The addition of lime along with the mineralogy also provided the environment for the formation of ettringite (see Prof S Jefferis comments at the ICE),

As with the Saxmundum bypass no comments were made as to the number of soaked CBR swell tests conducted during the stabilisation works

It was concluded from this M40 investigation, that the CBR swell tests conducted were unsatisfactory in replicating either, the degree of swell, or the site conditions experienced for the lime stabilised material in the field.

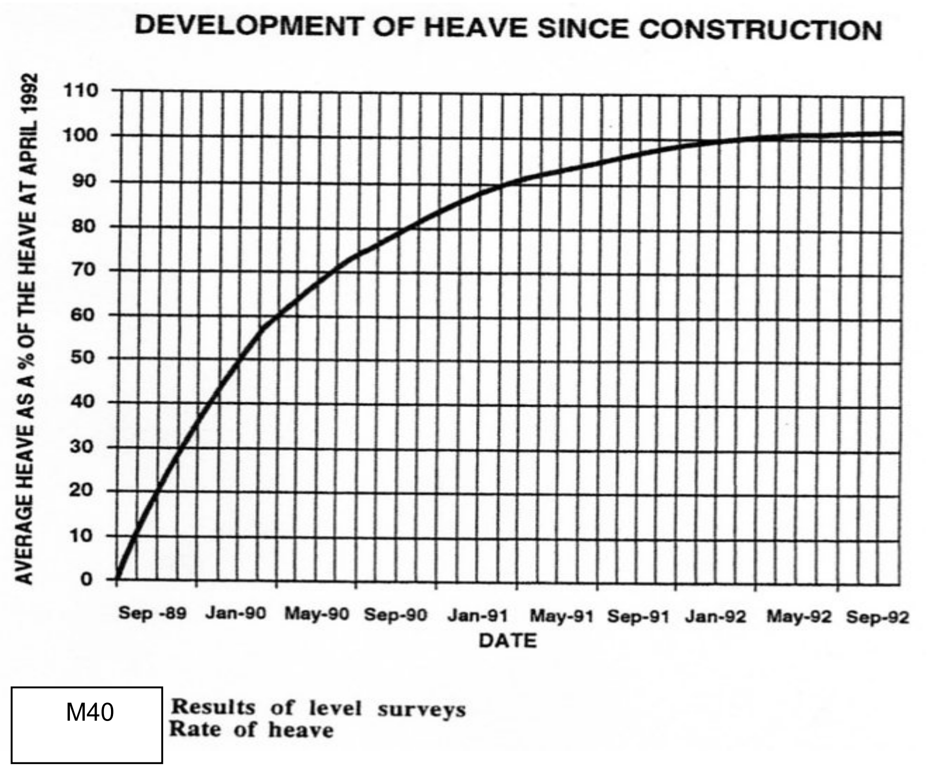


Figure 5. M40 chart showing the average heave versus time graph

Considering the amount of heave recorded on site and the relatively short time that it took the heave to develop (i.e. 60% heave within 6 months), it does ask questions as to why this could not be reproduced in the laboratory. It also does seem to question whether or not the soaked CBR procedure is allowing the water to penetrate the specimen, or maybe could it even be self sealing in some way?

Snedker quoted: “The most significant test was carried out on a sample of field material mixed with lime, compacted into a cylindrical mould and cured.

The specimen was removed from the mould and immersed in water.

Within minutes it began to disintegrate and within hours had collapsed completely”.

Snedker has also suggested that if swell tests are required, that due consideration be given to carrying out tests on cylindrical specimens which have been prepared to reproduce the stabilisation process.

Snedker continued to suggest that curing at 20°C should allow the formation of ettringite if the environment permits. The specimens should be removed from their moulds, cured and then be placed upright in a water bath at 20°C. Swell can then be observed and if required, the temperature can be cycled below 15°C to observe the effects. It is believed that below 15°C thaumasite is formed instead of ettringite (both being products of sulphate attack).

The investigation ultimately attributed the expansion and subsequent failure of the capping layer to the presence of sulfides within the materials which were not detected using the chemical testing employed as standard at the time of construction (the test being for Total Sulfate). Detailed laboratory testing showed the formation of ettringite in material with only 0.37% laboratory measured total (acid soluble) sulfate

After the M40 experience, it was considered that stabilisation was still fairly new in the UK and that the specifications and guidelines needed improving.

It was therefore recommended by the HA that future contracts considering stabilisation needed to conduct a desk study into the potential risks of sulfate development, along with sufficient testing at the ground investigation stage to establish the total sulfur content, total sulfate content and the mineralogy of the soil.

In 1989 a paper was published by *Thomas, Kettle and Morton; "The oxidation of pyrite in cement stabilised shale" Journal of engineering geology, London*.

This paper looked at the engineering implications of pyrite oxidation for cement stabilised minestone (csm). It compared three different curing regimes:

1: specimens sealed in polythene at 30°C (taken as the bench mark for the other curing regimes)

2: specimens sealed in polythene for 7 days then totally immersed at 30°C

3: specimens sealed in polythene for 7 days then placed on a porous plate with capillary access and exposed to an environment at 100% relative humidity at 30°C

The specimens were later monitored for volume stability and strength over a 410 day period. It was found that due probably to the strength of the specimens that the effects of the geochemical mechanisms were undoubtedly hidden by the greater effect of the physical mechanisms. A 100 day monitoring period was therefore chosen as the benchmark.

M D A Thomas (1986): "The performance of the cement stabilised minestone suggests that the rate of expansion/water uptake is controlled by grading, plasticity, sulfate content and mineralogy, each of these elements resulting in a different expansion rate".

The results of the long-term expansion of csm specimens over a 100 day period showed little difference in change in strength between using curing regime 2 or curing regime 3. Whichever regime was adopted, the loss/gain in strength pattern was very similar for both.

When comparing the rate of change in sulfur mineralogy, there seemed to be a similar rate of sulfate production using either curing regime 2 or 3.

There seemed to be a good correlation between the two curing regimes (2 & 3), for the rate of pyrite oxidation and the original pyrite content of the raw minestone. The gradient of the regression lines showed that approximately 3% of the original pyrite is oxidised in any 100-day exposure.

There was a slight difference in the sulfate produced in the specimens, as curing method 2 showed a slightly lower sulfate level than regime 3, and this was thought to be due to the leaching of water soluble sulfates in the totally immersed specimens. This was borne out when the host water was analysed.

Conclusions from the Thomas, Kettle and Morton paper were that sulfide minerals in minestone are capable of oxidising, if sufficient water is made available.

That total sulfate contents **and** total pyrite contents need to be analysed, therefore giving an indication into the total potential sulfate available after oxidation.

A degree of expansion can be tolerated, provided that the csm does not incur a reduction in strength, and therefore if the long term strength remains stable, or increases, the csm can be considered to be durable.

P Sherwood (1993),*State of the Art Review ”Soil stabilisation with Cement and Lime”* showed that there are benefical properties to be found from stabilising some materials with both lime & cement. This was due to some materials (such as silt), having a low plasticity therefore not being suitable for lime only stabilisation, and at the same time being too plastic for cement only stabilisation, but when modified with lime first and then stabilised with cement were found to perform well, even giving enhanced results.

It is interesting to note that the author when reviewing the laboratory assessment of suitability for stabilisation, suggests that “As a preferred alternative to determining the sulfate content, immersion tests on compressive strength or CBR specimens may be used to ascertain whether any constituents, such as sulfates, are present in harmful concentrations”. This assumption has also been suggested by other documents such as TRL505, where it is suggested that “the currently used CBR swell test is a prime indicator to the suitability of selected cohesive fills for achieving satisfactory lime stabilisation”. TRL505 also adds that “the CBR swell test indicates when the lime and any sulfates or sulfides in the fill material react together to produce highly expansive products such as ettringite or thaumasite”.

By 1995 the DTp had issued *HA74/95* which clearly set out a recipe for the design and construction of lime stabilised capping. (As well as chemical checks for total sulfur contents, and total sulfate contents, it also recommended the use of the CBR test to monitor swell).

Over the next 10 years stabilisation was carried out on various contracts fairly successfully without any major problems being published. Different binders were now being used to achieve high strength/stiffness layers using fine grained materials.

Cement stabilisation of granular materials had been fairly widely used since the 1950's and the DoT were now using lime & cement as well as lime or cement for the stabilisation of capping materials.

As mentioned above, BAA had developed a specification using lime to modify the material (i.e. make the material friable, reducing the plasticity, and therefore creating a better matrix for the cement to cover the particles). This dual process seemed to be opening doors to accommodate many more types of soils. *HA74/95* now acknowledged this and contracts such as the construction of the A27 at Patching were quick to use both lime only, and lime and cement stabilisation as a replacement for importing a stone capping layer. *(British Lime Association: BLI stabilisation project report No.4)*

At the same time, engineers were now looking to use other combinations of binders to modify what would be unsuitable materials, into suitable general fill.

The New A13 – Rainham Bypass was the first DoT contract (1995) to use lime and pulverised fuel ash (PFA) for improving an unsuitable silt deposit. On its own the silt did not have a large enough clay fraction to create the pozzolanic reaction normally expected in stabilisation, but with PFA and lime the silt could now be dried out and stabilised. It is worth noting from this paper that the total sulfate, sulfur & organic contents were well above the recommended guidelines for stabilisation. However 100,000m3 of previously unsuitable silt was treated and the contract was deemed a success. *(A Nettle; Treatment of Silt using Lime and PFA to form embankment fill for the new A13; Thomas Telford; Lime stabilisation 1996).*

In 1995 B C J Chaddock conducted trials in the pavement test facility at Crowthorne to investigate "*The structural performance of stabilised Soil in Road foundations" Later 1996: TRL 248*. This report concluded that stabilised sub-bases provided foundations with similar stiffness but improved resistance to deformation by construction traffic, even when laid thinner than traditional Type 1 sub-base. It also recommended that on the basis of the performance of lime and cement treated soil in the pavement test facility, that lime and cement treated materials should be included as sub-base in the Department of Transport Standards.

By the year 2000 there had been another review of HA74 which reiterated the need for sufficient checks for potential sulfate content from un-oxidised sulfide compounds.

The latest review updated a few classification categories and gave some easy-to- follow flow charts for the derivation of classes of material for stabilised capping.

This advice note acknowledged lime improvement, but did not highlight the potential risks involved with this improvement when adding lime to a soil.

HA 74/00 also suggests using the CBR for monitoring the swell, and states that if the 7 day CBR value tested to BS1924:1990: Part 2 is on average >15% with no individual <8% then this material is suitable for stabilisation.
As discovered by Snedker and Temporal the CBR swell tests conducted in the M40 investigation did not show up any heave on poorly hydrated material or material with high sulfate contents, it seems odd therefore that this HA document can refer to Snedker's report and not acknowledge the shortcomings of this test.

If, for example, a CBR specimen had on average 5mm heave it would be acceptable for use, as long as no individual specimen had >10mm. In practice it would be interesting to know if any specimen with 5mm heave can get a 15% CBR value. It is also worth noting that these 28 day swell specimens in HA74/00 were not required to be tested for CBR value as part of the pass/failure criteria, it was only the 7 day specimens (i.e. 3 days air cure + 4 days[3+1] soak).

Hypothetically, using this method of testing, a material with very little binder in a fairly dry condition may achieve on average a 7 day CBR value of >15%, with a separate specimen achieving a 28 day swell of less than 5mm, which would therefore make the material suitable as a capping material, in accordance with HA74/00.

Biczysko.S.(2003)

In 2001/2002 the A428 Crick bypass was constructed using a hydraulically bound stabilised material as a foundation layer, instead of the historical standard approach of using a stone capping and a stone sub-base layer. The works were conducted outside the standard SHW using new design control techniques instead of the previously used soaked CBR approach.

Volumetric stability tests were conducted ahead of the works using 100mm diameter specimens, compacted in the moisture condition apparatus (MCV test), extruded and soaked un-confined for 90 days in aerated water at 20^oC +/- 2^oC. If the specimens displayed a volumetric swell of greater than 2.5% they were deemed to have failed.

A minimum binder content was chosen(not less than 8% by mass) for the works, along with various other control methods such as a permeability test(not less than 1×10^{-9} m/s) , standard density tests (>90% compaction compared to MCV specimens, compacted to refusal), as well as in-situ stiffness tests to show strength gain (> 50mpa after 48 hours using a light dynamic plate test).

To date there have been no recorded areas of failure within the foundation layer.

Ground Engineering Magazine (Dec 2004)

As previously mentioned there has been a recent failure on another DoT contract (A10 Waddesmill Bypass), and sometimes history seems to repeat itself. Based on the information released to date there are more than one or two similarities between the two largest stabilisation road failures in UK history.

At present the failure on the A10 contract has so far been attributed (as with the M40), to the expansion and subsequent failure of the capping layer due to the presence of sulfides within the materials which were not detected using the chemical testing employed as standard at the time of construction.

It must be noted that at the time of writing no official conclusion has been drawn.

The question must be asked that if notice had been taken of HA74/00 and the material at the A10 Waddesmill bypass had been deemed suitable for stabilisation (as each step is clearly set out in HA74/00, giving a red light or a green light to each stage of the lab investigation), why did the lab trial or the site work not show any signs of failure? If there is an investigation being conducted after the failure on the A10, is the CBR test expected to pick up on this potential problem? And as highlighted by Snedker on the M40 investigation the question must be asked as whether or not any unconfined soaking tests were conducted.

These questions have been asked to the Highways Agency and, as of yet, there has been no official reply.

D N Little & B Herbert Tremti Sept 2005

A lot of research has been conducted into establishing thermodynamic modelling, therefore creating a phase diagram which can be used to establish thresholds where the formation of ettringite begins. This technique has been used in the USA on some major highways and airports.

As the paper points out, each material and binder combination to be considered has a unique mineral and chemical activity. It was suggested that the use of geochemical models could be used to simulate physical and chemical processes in subsurface systems, and that the precipitation-dissolution of ettringite in soils could be predicted using thermodynamics.

At the Tremti conference, Prof D Little said that he thought that this thermodynamic phase modelling approach would only be suitable for assessing UK soils on the larger scale contracts due to the high variability of the geological strata likely to be encountered within a single contract, and the respective costs involved.

Pyrite oxidation of Lime Stabilised materials; S Jefferis ICE –Jan 05

During the evening proceedings Prof S Jefferis (Surrey University) presented a brief paper raising his concerns that the CBR test may not allow enough water through the specimen to enable the oxidation of pyrites in the sample.

It was the opinion of Prof S Jefferis that the mechanics of swell are as follows:

"For sulphidic soils (initially low in sulphate) swell requires a period of damp but unsaturated soil to allow atmospheric air in to oxidise the sulphide to sulphate (the late spring/summer season), and then a wet near saturated soil for ettringite/ thaumasite growth. This means that heave generally is over in one season".

TRL Report 505 (2001)

In 2001 the TRL Report 505 "Swell test requirements for lime stabilised materials" (MacNeil and Steele, 2001) was issued to validate or revise the swell limits given in HA74/95 (subsequently superseded by HA74/00 and now superseded by HA74/07). The report set out to ensure that only "suitable" cohesive fills are permitted for lime stabilisation within the MCHW. TRL 505 states that the CBR swell test is currently the primary test procedure for determining the suitability of selected cohesive fills for achieving satisfactory lime stabilisation. Furthermore, TRL 505 states that the CBR swell test can indicate the potential for excessive swell of the material in situ, under stabilised conditions, even if this swelling is the result of lime and any sulfides or sulfates in the material reacting together to produce highly expansive products such as ettringite or thaumasite.

TRL 505 reviewed alternative limits and test methodologies for assessing the swell of stabilised soils within the experimental works and concluded that the only alternative was CEN 1997 – ‘Unbound and hydraulically bound mixtures for roads, specification for lime treated mixtures for road construction and civil engineering’. This standard includes limits for volumetric swelling.

Table 1). TRL 505 goes on to state that the volumetric expansion values equate to linear swells of 6.35mm (Volumetric Swelling S_V - 5%) and 12.7mm (S_V - 10%) for samples compacted into standard BS 1377 CBR moulds (127mm high).

Considering the above information that (in their opinion) there is only one alternative to the CBR test, it is surprising to see that the prEN test was not used as a comparison, and an assumption that the mathematical Linear expansion of 6.35mm has been taken as a direct equivalent to the 5% volumetric expansion used in the prEN test. The likelihood of a confined specimen such as the CBR test giving the same expansion as a totally unconfined specimen is difficult to accept (see example below). Simply calculating the surface area and permeability of the soil would suggest that water may be more readily available when using the un-confined test, instead of the confined CBR test.

It would seem logical therefore that if a report was required, to validate or revise the swell checks, then a comparison should have been made between the CBR test and at least one other test, especially as it was deemed to be the only other currently acceptable alternative.

Table 1. Swell test limits as defined in CEN 1997

CEN 1997

Determination of volumetric swelling (Sv).

The potential for swelling of the mixture shall be assessed as below by the volumetric swelling (Sv) test on cylinders subjected to total unconfined immersion in water at elevated temperatures

If Sv ≤ 5%, the soil/material is suitable for treatment.

If Sv ≥ 10%, the soil/material is unsuitable for treatment.

If 5% < Sv ≤ 10%, the soil/material warrants further study.

Example based on the assumptions made in TRL505:

If a typical soil being tested had a permeability of 1×10^{-7} m/s (high for a clay), at this rate in 3 days the water would have permeated 25.9mm into the material.

Therefore using the alternative prEN test the 50mm diameter x 50mm high specimen would be totally saturated in 3 days.

However with the BS 1924:CBR specimen, after the first three days, the water would only have been able to ingress through the base (also bear in mind that the CBR is 127mm high), and that the specimen is not totally submerged in the first 72hours.

Assuming that the stabilised material within the CBR mould did not adhere to the sides of the mould in any way, and that there were no frictional forces to overcome (between the mould and the specimen) therefore allowing the specimen to lift up vertically unhindered, and also that the saturated material had expanded 5% of the saturated depth (i.e. 26mm), we would still only expect to see up to 1.3mm of heave, this is well below the 5mm threshold.

At this permeability the specimen would have to have been in soak for at least 9 days before it was saturated. The above example was only looking at 3days in soak.

If the permeability of more highly plastic material (potentially more expansive), with an even slower permeability (typically < 1 x 10^{-8} m/s), was used, then this could reduce the water ingress by a further factor of ten, needing even longer to soak than that used in TRL 505.

There is also the thought that the stabilised soil being tested would have been mixed and that possibly due to the potential re-organisation of the clay platelets it may be found that the natural direction of expansion which the clay wants to take, may be being constrained by the sides of the moulds.

The TRL 505 report adopted a testing programme based on seven cohesive soils taken from around the UK, namely:

- Mercia Mudstone
- Oxford Clay
- Lower Lias Clay
- London Clay
- Gault Clay
- Weald Clay
- Kimmeridge Clay

These materials exhibited a wide range of physical and mechanical properties. Total sulfate content ranged from <0.04 to 1.9 % SO_3, and total potential sulfates (TPS) from <0.10 to 4.75 % SO_3. None of the 62 specimens tested had a swell value in excess of 10 mm, the upper limit stated in HA74/07, and only one specimen had a swell in excess of the upper 5 mm average swell value. The report found that the clay types mentioned previously would be appropriate for treatment with lime and/or cement, even with TPS of up to 5.7 % (% SO_4), as they were all "suitable" based on the CBR swell test limits. This is despite the fact that HA guidance suggests that the upper limiting value of TPS should not exceed 1% and warns that there is evidence that, for some materials, values as low as 0.25% may cause swelling (*Sherwood, 1992*). TRL 505 concluded that "The currently specified laboratory tests for lime stabilisation of capping materials, including the CBR swell test procedure, are effective performance indicators for mix design and long term durability". TRL505 recommended that "the CBR swell test procedure be retained in the UK specification in its current form". TRL505 also suggested that "the swell limits specified in HA74/00 (and now HA74/07) for lime treatment are considered to be appropriate for the typical range of British clays stabilised in the trial, and should not be altered"!

BS EN 14227-11 (2006) provides maximum linear swell values for use as acceptability criteria of soaked CBR specimens. The average swell should not exceed 5 mm, with no individual linear swell value exceeding 10 mm. The material is then placed in one of three categories given in Table 2.

The volumetric swelling (G_v) determined using BS EN 13286-49 (2004), should not exceed 5%. However, if it is between 5% and 10%, then its use maybe supported by additional testing. Equating the linear swell to a volumetric expansion (as in TRL 505), then the criteria specified in BS EN 14227-11 is more rigorous than that of CEN 1997; however, it does not outright exclude materials that do not meet this criteria.

Table 2. Linear swelling categories taken from BS EN 14227-11 (2006)

Average maximum swelling of the specimens (mm)	Maximum swelling of any individual specimen (mm)	Classes
5	10	LS_5
3	6	LS_3
1	2	LS_1

Chronological Summary:

In 1962: Sherwood undertook a regime of testing which managed to create an environment where specimens would show expansion if subjected to a high sulphate solution. At the same time the specimens would not collapse if the sulphates were removed from this solution. The aim in the laboratory, is surely to try and replicate the conditions in the field. The unconfined soaking conditions did seem to create this environment, but it must be noted that the specimens were allowed to cure for sufficient time to gain some strength prior to immersion, and were conducted with fairly high binder contents.

1986: DTp-SHW dropped the sulphate content and used the un-confined immersion test.

1986: DTp-SHW introduced a specification for lime stabilisation to create a capping layer (No soaked CBR tests were included).

1988: Failure of lime stabilised capping layer on the Saxmundham bypass.(No soaked tests).

1990: Failure of lime stabilised capping layer on the M40 Motorway, (No soaked tests).

1991: Investigation into the failure of lime stabilised capping layer on the M40 Motorway, (The soaked CBR tests did not replicate the heave in the field, 1No unconfined specimen disintegrated).

1993: Sherwood review recommended soaked CBR tests.

1995: HA74/95 released recommended soaked CBR tests.

2000: HA74/00 reviewed and still recommended soaked CBR tests at 28days but for swell measurement only.

2001: A428 Crick bypass successfully constructed a hydraulically bound foundation layer using unconfined soaked MCV specimens to record the volumetric stability of the material.

2001: TRL505 reported that the soaked CBR test and the current HA swell limits are good indicators to assess a materials suitability for stabilisation. The CBR swell test did not fail any of the 62 test specimens, even those with up to 4.75% sulphate content.

2003: A10 wadesmill bypass was conducted using lime stabilisation as a capping material in accordance with the new HA74/00 guidance. The contract failed that winter.

2004: GE magazine reported that due to the A10 failure the HA were to review the HA74/00.

1.91 Conclusions of Literature Review

Considering the methodology adopted in the CBR test, along with experiences previously found, it can be seen that swell due to oxidation may be very difficult to achieve with the current test procedure under the current curing conditions.

As found in the Thomas, Kettle & Morton report oxidation of pyrite did occur in the fully saturated specimens (regime 2) and in the partially saturated specimens (regime 3), even though there did not seem to be much difference between the two regimes. This trial was, however, conducted on stabilised colliery shale, with fairly high strengths being achieved.
Would the two curing regimes have given different results if they had been conducted on a more cohesive type of material?

It was felt that there is therefore a need to review the current durability testing procedures in order to evaluate how effective they are at predicting adverse in-situ performance. This is important as the MCHW does not currently specify upper limits of sulfates or sulfides, relying solely on laboratory testing to determine suitability. The background to this includes failures on a few stabilisation contracts, such as the A10 Wadesmill Bypass, Saxmundham bypass, and the M40 motorway contracts, combined with the introduction of IAN73 (HD25) Design Guidance for Road Pavement Foundations, and a review of HA74/00 (now superseded by HA74/07) - Treatment of Fill and Capping Materials using either lime or cement or both.

Current concerns from some specialised consultants and contractors working within the industry indicate that the California Bearing Ratio (CBR) swell test (BS 1924-2:1990), and herein referred to as the CBR swell test, is not an effective test for the assessment of a materials behaviour under site conditions. The pass criterion for the CBR swell test may deem a particular material suitable for use, yet, when subjected to site conditions, it may behave differently and cause a potential construction failure. Conversely, the European accelerated swelling test (BS EN 13286-49: 2004) is relatively poorly understood in relation to UK practice.

Britpave issued guidance on the stabilisation of sulfate bearing soils (Britpave, 2005) to assist practitioners. This included risk mitigation measures, such as the use of ground granulated blast furnace slag, which may inhibit expansive sulfate reactions. However, unless stabilisation specifications rely solely on limiting values for chemical constituents, a reliable laboratory testing procedure to determine volumetric stability is essential.

Each of the following variables have been cited in the previous literature review as factors which may individually or in combination promote/inhibit the formation of ettringite and/or thaumasite. The same factors may furthermore affect the softening and/or swelling characteristics of stabilised fine-grained soils. These factors are listed in Table 3.

Table 3. Variables which may have an effect on the swelling characteristics of a soil.

Chemistry	**Classification**	**Workability (site and laboratory)**
Total sulfate content Total sulfur content Water soluble sulfate content Surrounding migratory conditions Organic matter content pH	Plasticity index Activity index Particle size distribution Moisture content (and or MCV)	Initial lime consumption - value Binder content Mellowing Degree of pulverisation Degree of compaction Density gradient Curing time

Mineralogy		Curing temperature Confining conditions Permeability Strength / stiffness

It has been suggested by industry practioners that the conditions for oxidation of pyrite and/or the development of swell achieved in the field are not mimicked using the current laboratory CBR swell test (BS1924:1990 method). This results in laboratory swells smaller than those found in situ, due to inhibited formation of expansive minerals (ettringite for example). This has therefore been chosen as one of the key areas of research required.

In summary, the use of the current BS 1924-2 1990 CBR swell test and associated swell limits for predicting in situ behaviour may not be fit for purpose, since failures of stabilised soils have been reported, particularly on the M40 contract, where significant heave was observed in a material which was considered suitable using the CBR swell test. In addition, there seems to be a clear difference between the terms “Softening” & “Swell” as both can occur during immersion of specimens. For the purpose of this work they are defined as:

- **Softening** refers to the process by which a material loses strength/stiffness and softens, possibly due to **design or workmanship** limitations , and;

- **Swell or swelling** refers to the process by which a material expands, possibly due to insufficient **chemical durability**.

2. Phase 2 - Classification testing

2.1 Materials

Four sources of material were selected for inclusion within this laboratory study. They were selected to represent materials that are common candidates for soil stabilisation within the UK.

The first two sources were selected to allow an initial comparison between low and high plasticity clay:

- Glacial Till: Brown Silty SAND with some Clay, Chalk & Gravel. Report reference: LPM
- Weathered London Clay: Orange-Brown mottled grey silty CLAY. Report reference: HPM

The second two sources were selected to allow a comparison between relative levels of TPS:

- Oxford Clay: High Total Potential Sulfate/Sulfur Material. Report reference: H-TPS
- Oxford Clay: Low Total Potential Sulfate/Sulfhur Material. Report reference: L-TPS

Classification testing was undertaken to determine physical and chemical characteristics. These are summarised in Table 4. Chemical analysis of sulfide/sulfate content was conducted in accordance with TRL report 447. The remaining material property tests were carried out in accordance with the relevant British Standard. Activity index is defined as the ratio of plasticity index to the percentage by mass of soil particles less than 2 microns. Initial Lime Consumption Value, Plasticity Index and pH Value, determined in accordance with BS 1924-2: 1990, BS 1377-2: 1990, and BS 1377-3: 1990 respectively.

Table 4. Classification test and chemical analysis

Report Reference	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index (%)	Clay Fraction (%)	Activity Index	Organic Matter Content (%)	Sulfate Content (%SO_4)	Water Soluble Sulfate (g/l as SO_4)	Total Sulfur (% as S)	pH Value	Initial Lime Consumption Value (%)
LPM	19	35	16	12	0.87	1.10	0.04	0.04	0.02	6.7	3.5
HPM	34	89	55	53	1.03	0.90	0.06	0.06	0.03	6.1	3.4
H-TPS	26	63	37	42	0.88	0.98	4.55	1.60	1.78	7.5	4.2
L-TPS	21	56	34	45	0.76	0.66	0.29	0.33	0.11	7.3	3.3

Oxford Clays from the south and east of England are known to contain 5 – 15% sulfides (reduced form of sulfur. The chemical testing indicates that the majority of sulfur is in the form of sulfate (rather than sulfides). Variation in levels of sulfate in a clay is dependant to a greater degree on the local ground conditions rather than clay type. High concentrations can build up at the base of the weathering zone, consequently resulting in the upper layers having much reduced sulfate concentrations due to the effect of mobile ground water.

The water content and moisture condition value (MCV) were determined in accordance with BS EN 1097-5: 1999 and BS EN 13286-46: 2003, respectively. The results are presented in table 5.

Table 5: Moisture content and MCV value

Material Reference	Moisture Content (%)	Moisture Condition Value (MCV)
LPM	23	4.0
HPM	30	10.2
H-TPS	28	11.4
L-TPS	23	13.2

3. Phase 3 – Experimental Laboratory Testing Stage 1

3.1 Test Procedure - Trial Mixture Design

3.1.1 Introduction

The trial mixtures for Phase 3 Stage 1 were formulated to investigate the effect that design and workmanship may have on the durability of stabilised fine grained soils. Glacial Till and London Clay, with a low sulfate content, were used to minimise the effect that sulfate reactions will have on the results of the durability testing, as the onus is on how binder content, degree of pulverisation and compactive effort affect the durability of the specimens measured using the two key test procedures selected for comparison (as concluded in the literature review), outlined in Sections:

1.8.2.1: European Accelerated swelling test: BS EN13286-49: 2004.
1.8.2.2: Soaked California Bearing Ratio test: BS 1924: 1990: Part 2.

3.1.2 Trial Mixtures

Mix 1: Low Plasticity Material (LPM), with a low Total Potential Sulfate content (L-TPS).
This material was purposely mixed with low binder content, with a poor degree of pulverisation and a low compactive effort, in order to create a lime treated material which might fail the CBR swell test.

Mix 2: Low Plasticity Material (LPM), with a low Total Potential Sulfate content (L-TPS).
This material was mixed with sufficient lime to enable stabilisation to occur (i.e. at its Initial Lime Consumption Value), with a good degree of pulverisation and a good compactive effort, in order to create a lime treated material which might pass the CBR swell test.

Mix 3: Low Plasticity Material (LPM), with a low Total Potential Sulfate content (L-TPS).

This material was mixed with sufficient lime and cement to enable stabilisation to occur, with a good degree of pulverisation and a good compactive effort, in order to create a mixture which might pass the CBR swell test.

Mix 4: High Plasticity Material (HPM), with a low Total Potential Sulfate content (L-TPS).

This material was purposely mixed with low binder content, with a poor degree of pulverisation and a low compactive effort, in order to create a lime treated material which might fail the CBR swell test.

Mix 5: High Plasticity Material (HPM), with a low Total Potential Sulfate content (L-TPS).

This material was mixed with sufficient lime to enable full stabilisation to occur (i.e. at its Initial Lime Consumption Value), with a good degree of pulverisation and a good compactive effort in, order to create a lime treated material which might pass the CBR swell test.

Mix 6: High Plasticity Material (HPM), with a low Total Potential Sulfate content (L-TPS).

This material was mixed with sufficient lime and cement to enable full stabilisation to occur, with a good degree of pulverisation and a good compactive effort, in order to create a lime and cement treated material which might pass the CBR swell test.

3.1.3 Additional Considerations

Materials were sourced and classified in accordance with the guidance given in HA74/07. For each of the above mixes, specimens were prepared across a range of water contents (i.e. MCV range between 8 and 12).

The standard BS EN 13286-49: 2004 European accelerated swelling test, as required when working to the new Series 800 (MCHW1, 2007) specification for soil lime/cement treated materials, was conducted, in which the curing of the specimens prior to immersion is 1.5 – 2 times the workability period of the cement. However, questions have been raised regarding the short curing/workability time of the BS EN 13286-49: 2004 tests when cement treated materials are being used, and, therefore, an extra curing regime was put in place. This resulted in an identical set of specimens being cured for a longer period (i.e. 72 hours @ 20 ^{o}C) prior to being totally immersed and tested. This curing regime is herein referred to as ‘non standard’ in relation to the European accelerated swelling test.

3.2 Results of Phase 3, Stage 1 Laboratory Testing

The results of the Phase 3 Stage 1 laboratory testing are presented in Table 6 to table 8

Table 6. Preliminary testing of Phase 3, Stage 1 mixes

Mix Reference	Lime Content (%)*	Cement Content (%)*	Water Content (%) **	Material Passing 5mm (%)	MCV
LPM 1A	1.5	-	23	52	8.4
LPM 1B	1.5	-	21	52	10.1
LPM 1C	1.5	-	20	52	11.9
LPM 2A	3.5	-	24	55	8.9
LPM 2B	3.5	-	22	55	10.7
LPM 2C	3.5	-	21	55	12.4
LPM 3A	1.5	3.0	24	63	8.2
LPM 3B	1.5	3.0	22	63	10.1
LPM 3C	1.5	3.0	21	63	11.8
HPM 4A	1.5	-	33	20	7.8
HPM 4B	1.5	-	30	20	10.3
HPM 4C	1.5	-	28	20	12.4
HPM 5A	3.5	-	34	35	8.7
HPM 5B	3.5	-	32	35	10.5
HPM 5C	3.5	-	30	35	12.8
HPM 6A	1.5	3.0	33	60	8.5
HPM 6B	1.5	3.0	32	60	10.3
HPM 6C	1.5	3.0	29	60	12.2

Note Table 6, 7 & 8: DD is the dry density in Mg/m^3, WC is the water content in %. * lime & cement contents were calculated as a percentage of the dry mass of soil. ** Water content, Percentage passing the 5mm BS test sieve, and MCV were taken at the time of compaction. s = standard test method, ns = non-standard test method. All of the above CBR specimens were prepared, made, cured and tested in accordance with BS 1924-2:1990 method 4.5 (using a 2.5kg rammer).

Table 7: 7 day and 28 day CBR results for Phase 3, Stage 1 mixes

Mix Reference	**Average 7 day CBR**			**28 day CBR (%)**			**WC (%)**	**DD (Mg/m^3)**
	% CBR	**WC (%)**	**DD (Mg/m^3)**	**Top**	**Bottom**	**Average**		
LPM 1A	16	25	1.57	6	14	10	26	1.56
LPM 1B	18	24	1.60	7	16	12	24	1.61
LPM 1C	22	22	1.65	6	14	10	24	1.62
LPM 2A	17	26	1.56	25	26	26	25	1.57
LPM 2B	18	23	1.61	28	29	29	25	1.59
LPM 2C	24	23	1.64	37	36	37	24	1.63
LPM 3A	35	24	1.58	43	46	45	24	1.59
LPM 3B	60	23	1.63	65	85	75	24	1.63
LPM 3C	70	22	1.64	95	90	90	22	1.63
HPM 4A	13	35	1.38	6	12	9	35	1.36
HPM 4B	14	32	1.42	5	10	7	33	1.41
HPM 4C	15	32	1.43	5	9	7	33	1.42
HPM 5A	16	36	1.35	11	18	14	35	1.36
HPM 5B	18	34	1.38	12	19	15	35	1.37
HPM 5C	20	32	1.40	12	20	16	33	1.40
HPM 6A	26	33	1.40	27	45	35	34	1.39
HPM 6B	28	31	1.40	26	40	35	32	1.41
HPM 6C	35	30	1.44	35	50	45	30	1.45

Table 8: Swell and expansion measurements for Phase 3, Stage 1 trial mixes

Mix Reference	28day CBR Swell (mm)	Volumetric Expansion G_v (%) s	Expansion G_v (%) ns
LPM 1A	1.7	5.6	-
LPM 1B	2.6	6.2	-
LPM 1C	2.4	6.3	-
LPM 2A	0.1	1.2	-
LPM 2B	0.5	1.6	-
LPM 2C	0.5	1.5	-
LPM 3A	0.0	2.6	0.3
LPM 3B	0.0	2.9	0.1
LPM 3C	0.0	2.2	0.5
HPM 4A	3.9	11.4	-
HPM 4B	3.2	10.9	-
HPM 4C	3.7	12.0	-
HPM 5A	2.9	3.4	-
HPM 5B	2.7	4.4	-
HPM 5C	2.8	4.7	-
HPM 6A	0.8	5.7	1.8
HPM 6B	1.0	6.4	2.2
HPM 6C	0.7	5.8	2.0

3.3 Discussion of Phase 3, Stage 1 Laboratory Testing

Mix 1 (LPM 1 – L-TPS) results show that the 7 day CBR swell test has passed the CBR requirement of >15% (achieving 16-22%); however after 28 days soaking the CBR decreased (achieving 10-12%), but was still greater than the minimum of 8%. More importantly, this implies that stabilisation had not occurred. The 28 day CBR swell results were all <5 mm. The European accelerated swelling test, however, gave approximately 6% expansion. Together with the fall in CBR value, this would suggest that the material requires further investigation or additional lime.

Mix 2 (LPM 1 – L-TPS) results show that the 7 day CBR swell test has passed the CBR requirement of >15% (achieving 17-24%). After 28 days soaking, the CBR values increased further (achieving 26-37%), and recorded <1 mm swell. The European accelerated expansion test also showed very little expansion. (1-2%), which suggests that all tests, including CBR increase, deemed the material to be suitable for stabilisation.

Mix 3 (LPM 1 – L-TPS) results show that the 7day CBR swell test has passed the CBR requirement of >15% (achieving 35-70%). After 28 days soaking, the CBR values increased further (achieving 45-90%) and recorded <1 mm swell. The European accelerated expansion test also showed very little expansion (<3%), which in practice would suggest that all criteria deemed the material to be suitable for stabilisation.

In addition, when the specimens subjected to the non-standard Volumetric Expansion test were cured for 3 days prior to soaking (similar to the CBR), the volumetric expansion reduced. (Please note that in the EN accelerated swelling test, the standard curing period for cement treated material is 1.5 to 2 times the workability period of the binder, i.e. only 3-4 hours, before the specimen is placed in soak).

Mix 4 (HPM 1 – L-TPS) results show that the 7 day CBR swell test just fell short of the CBR requirement of >15%, achieving 13-15%. However, after 28 days soaking the CBR reduced to 7-9%. More importantly, this implies that stabilisation had not occurred. The 28 day CBR swell results showed <5 mm. However, the European accelerated swell test showed approximately 11% expansion, which confirms the CBR results indicating that the material used is unsuitable for treatment.

Mix 5 (HPM 1 – L-TPS) results show that the 7day CBR swell test has passed the CBR requirement of >15% (achieving 16-20%). After 28 days soaking, the CBR values decreased to 14-16%, and recorded <5 mm swell. The European accelerated swelling test also showed 3-5% expansion, which suggests that both tests deemed the material to be suitable for stabilisation, even though the strength of the material reduced after 28 days soaking.

Mix 6 (HPM 1 – L-TPS) results show that the 7day CBR swell test has passed the CBR requirement of >15% (achieving 26-35%). After 28 days soaking, the CBR values increased further (achieving 35-45%), and recorded <1 mm swell. The European accelerated expansion test showed >5% expansion. This suggests that the CBR test ‘passed’ the material; however, the European accelerated expansion test indicated the material required further investigation.

3.4 Summary

The results show that soaked CBR values and swell test results of the CBR specimens tested are dependent on specimen quality in relation to binder content, pulverisation and compactive effort. The soaked CBR test did not appear to be sensitive to variability associated with factors such as density and pulverisation. It is also inferred that the increased durability of a material associated with high CBR values and low percentage swell is the result of the development of the cementitious matrix within the specimen. This is demonstrated by the reduction in swell measurements on specimens subjected to an extended period of air curing prior to immersion. The results show that in terms of high CBR and low swell both the low and high plasticity clays are the most durable when treated with lime and cement compared to the other mixes, although a greater addition of lime may also have the same effect. The high plasticity clay has been shown to be more susceptible to volumetric swell than the low plasticity clay. This is expected since it is likely to have greater potential for water absorption due to the intercalation of water between the clay particles.

In general, the soaked CBR value tested at the top of the specimen is lower than that tested at the bottom. It is suggested that the confinement at the bottom inhibits the further absorption of water after the initial void spaces have been filled. As the water moves up the specimen, probably by capillary action, the absorption of water through intercalation between the clay particles is greatest at the unconfined end of the specimen, resulting in lower CBR values at the top than at the bottom.

The results demonstrate that high plasticity clays are more likely to present problems in terms of their stabilisation than low plasticity clays. In relation to the use of the soaked CBR and European swell test for assessing the suitability of a material for stabilisation, further investigation would be required to draw any further conclusions regarding the effect that the degree of pulverisation and compactive effort has on the results. The promotion of the cementitious reactions by greater homogenisation of lime and cement (higher degree of pulverisation) will promote strength development in the specimen and hence lead to an increase in durability. Up to a point, increased addition of lime and cement produces a more durable material. The curing conditions used in terms of curing and soaking times and the relative timing of the test will also affect the results, as they inevitably affect the physico-chemical processes associated with hydraulic reactions and those of reactive guest species such as sulfide and sulfate compounds.

It was also noted that on extruding the CBR specimens after testing, some considerable force was required to overcome the specimen’s adhesion/frictional resistance against the internal surface of the CBR mould. This was, therefore, investigated further.

Figure 6. Extrusion of a specimen from the CBR mould

Figure 6), the load required to initially overcome the adhesion / frictional forces holding the CBR specimen in place was typically in the region of 16 kN – 20 kN. Given that the typical surface area of a CBR mould is approximately = 0.06 m^2, a pressure of typically 300 kN/m^2 is necessary to overcome this initial adhesion. Once the initial adhesion strength had been overcome, a constant applied force of between 6 – 10 kN was required to enable further extrusion of the specimen.

4. Phase 3 – Experimental Laboratory Testing Stage 2

4.1 Introduction

The aim of the Phase 3, Stage 2 testing was to test a material that had relatively high total potential sulfate. This material could potentially fail the CBR swell test with regard to the current swell limits set out in HA74/07.

After in-depth laboratory trials (not reported here), a material was found which had the potential to fail both the CBR swell test and the European accelerated swelling test. The material chosen was a sample of Oxford Clay taken from a site in the Milton Keynes area. The samples were taken from differing depths giving clearly different TPS values. It was decided, therefore, to use this material for all of the Phase 3, Stage 2 laboratory testing. After classification testing (Section 3), the Oxford Clay was separated into two different groups - High Total Potential Sulfate material (H-TPS), and Low Total Potential Sulfate material (L-TPS). All the specimens were tested using both the CBR swell test and the European swell test as in Phase 3, Stage 1. In addition, mixtures designed to give specimens with relatively high strengths were cured to a non-standard method, and then tested for loss of strength on immersion. The loss of strength on immersion specimens were manufactured by MCV apparatus and compacted to refusal.

4.2 Loss of Strength on Immersion Test Procedure

The Specification for Highway Works, Volume 1, (MCHW 1) Series 800, Clause 880.4 (2007 amendment) requires hydraulically bound materials (HBMs) to be checked for loss of strength on immersion. This test is a strength test conducted on HBM specimens (1:1 height to diameter) after 14 days air cure at 20±2 °C followed by 14 days soaking at 20±2 °C. These specimens are compared with identical specimens air cured for 28 days at 20±2 °C used as controls. At the end of the curing period, the compressive strength (BS EN 13286-41: 2003) of the soaked specimens must achieve at least 80% of that achieved by the unsoaked specimens to be deemed durable.

4.3 Results of Phase 3, Stage 2 Laboratory Testing

The results of the Phase 3, Stage 2 laboratory testing are presented in Tables 9 to 13. Figures 7 to 10 show specimens which have been subjected to the CBR and European accelerated swelling test.

Table 9. Stage 2 designs and initial mixture properties

Mix Reference	Lime Content (%)*	Cement Content (%)*	Material Passing 5 mm (%)**	Moisture Condition Value (MCV) **	Water Content (%) **
H-TPS 7A	1.5	-	16	11.5	26
H-TPS 7B	1.5	-	16	10.5	31
H-TPS 7C	1.5	-	16	8.6	34
H-TPS 8A	3.5	-	30	13.6	28
H-TPS 8B	3.5	-	30	10.0	32
H-TPS 8C	3.5	-	30	12.0	33
H-TPS 9A	1.5	3.0	51	13.0	27
H-TPS 9B	1.5	3.0	51	10.7	30
H-TPS 9C	1.5	3.0	51	9.4	30
L-TPS 10A	1.5	-	28	13.2	22
L-TPS 10B	1.5	-	28	11.0	25
L- TPS 10C	1.5	-	28	9.7	27
L-TPS 11A	3.5	-	37	13.5	24
L-TPS 11B	3.5	-	37	10.8	26
L-TPS 11C	3.5	-	37	8.5	29
L-TPS 12A	1.5	3.0	41	11.6	24
L-TPS 12B	1.5	3.0	41	9.8	26
L-TPS 12C	1.5	3.0	41	8.0	28

Key: H-TPS, clay with high Total Potential Sulfate content. L-TPS, clay with low Total Potential Sulfate content. *lime & cement contents were calculated as a percentage of the dry mass of soil.
** Water content, percentage passing the 5mm BS test sieve, and MCV were taken at the time of compaction.

Table 10. 7 day CBR values including water content (WC) and dry density (DD) data

Mix Reference	7 day CBR (%)			
	Top	Bottom	WC (%)	DD (Mg/m^3)
H-TPS 7A	8	28	30	1.50
H-TPS 7B	3.0	10	33	1.43
H-TPS 7C	1.5	5.0	36	1.38
H-TPS 8A	9.0	23	32	1.44
H-TPS 8B	4.5	18	39	1.36
H-TPS 8C	5.5	14	35	1.40
H-TPS 9A	17	55	32	1.45
H-TPS 9B	13	29	31	1.45
H-TPS 9C	9.0	21	34	1.40
L-TPS 10A	27	40	24	1.62
L-TPS 10B	18	27	26	1.60
L-TPS 10C	8.5	11	27	1.54
L-TPS 11A	30	35	29	1.51
L-TPS 11B	30	30	28	1.53
L-TPS 11C	21	20	30	1.48
L-TPS 12A	85	65	26	1.55
L-TPS 12B	45	60	27	1.55
L-TPS 12C	35	40	28	1.50

Table 11. 28 day CBR values including water content (WC) and dry density (DD) data

Mix Reference	28 day CBR (%)		28 day WC (%)			28 day DD
	Top	Bottom	Top	Bottom	Av	DD (Mg/m^3)
H-TPS 7A	2.5	7.5	48	33	40	1.34
H-TPS 7B	2.0	8.0	51	35	43	1.27
H-TPS 7C	1.5	6.5	50	37	44	1.26
H-TPS 8A	5.5	25	49	35	42	1.31
H-TPS 8B	5.0	35	54	34	44	1.29
H-TPS 8C	5.5	29	53	36	44	1.28
H-TPS 9A	19	60	42	32	37	1.38
H-TPS 9B	4.5	40	48	33	40	1.32
H-TPS 9C	4.0	30	51	36	43	1.23
L-TPS 10A	19	55	26	23	25	1.61
L-TPS 10B	22	40	27	26	26	1.58
L-TPS 10C	17	29	28	26	27	1.54
L-TPS 11A	50	60	29	27	28	1.53
L-TPS 11B	55	55	29	26	27	1.53
L-TPS 11C	35	40	32	29	30	1.48
L-TPS 12A	65	130	27	24	25	1.58
L-TPS 12B	60	90	27	25	26	1.56
L-TPS 12C	30	80	28	28	28	1.51

Table 12. CBR swell and European accelerated swelling test data

Mix Reference	CBR swell (mm)	Volumetric Expansion G_v (%) standard test method	Volumetric Expansion G_v (%) non standard test method
H-TPS 7A	21.4	46	X
H-TPS 7B	16.5	41	X
H-TPS 7C	15.9	30	X
H-TPS 8A	12.7	49	X
H-TPS 8B	10.8	44	X
H-TPS 8C	8.9	43	X
H-TPS 9A	9.5	64	47
H-TPS 9B	9.6	62	46
H-TPS 9C	7.7	63	46
L-TPS 10A	0.7	0.0	X
L-TPS 10B	0.9	0.0	X
L- TPS 10C	1.4	-0.9	X
L-TPS 11A	0.6	0.3	X
L-TPS 11B	1.2	0.8	X
L-TPS 11C	0.5	-0.4	X
L-TPS 12A	0.3	3.2	-0.1
L-TPS 12B	1.2	2.6	0.2
L-TPS 12C	0.7	0.8	0.4

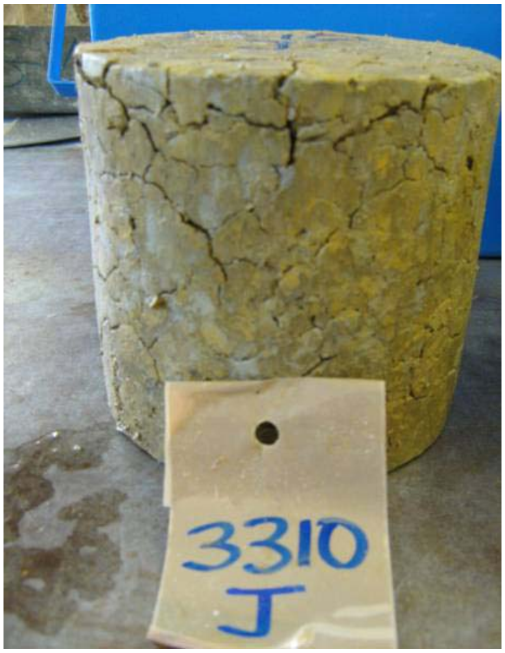
Note: X indicates not tested

Table 13. Results of immersion testing in accordance with Series 800 (MCHW1)

Material Reference	Specimen Details	Initial WC (%)	MCV	Final WC (%)	Dry Density (Mg/m^3)	Unconfined Compressive Strength (N/mm^2)	Volumetric Expansion (%)	Loss of Strength on Immersion (%)
H-TPS-9	Unsoaked specimen	27.1	13.0	30.5	1.48	0.88	21	100
	Soaked specimen	27.1	13.0	45.5	Disintegrated			
L-TPS-12	Unsoaked specimen	29.7	10.7	26.0	1.59	1.15	-0.3	32
	Soaked specimen	29.7	10.7	26.1	1.59	0.78		

Figure 7a: Un-soaked MCV Specimen (H-TPS-9) prior to strength testing

Figure 7b: Soaked MCV Specimen (H-TPS-9) prior to strength testing



It can be seen from figure 7b that the mixture H-TPS-9 exhibited significant swelling and spalling during the soaking phase of the loss of strength on immersion test. The control specimen in comparison is still in good condition. The change in colour of the immersed specimen suggests that the mineralogical composition has also changed.

Figure 8a: **7 day CBR specimen, mix H-TPS-7A**

Figure 8a: **7 day CBR specimen, mix H-TPS-7A**

Figure 8a & 8b show specimens that have undergone 7 and 28 day CBR swell testing. It is evident in both specimens that they have swelled significantly in the direction in which they are unconfined.

Figure 9. L-TPS-12B material with 1.5% lime and 3% (14 days air curing and 14 days soaking), MCV specimens, loss of strength = 32%, 0% volume change:

Figure 10. L-TPS-10B material with 1.5% lime, European accelerated swelling test specimens, 0% volume change:

Figure 9 and Figure 10 show specimens manufactured with low TPS material and subjected to the loss of strength on immersion test and the European accelerated swell test. No significant degradation of the specimens is evident, suggesting that both tests can be used for the evaluation of materials susceptible to deleterious sulfate reactions.

4.4 Discussion of Phase 3, Stage 2 Laboratory Testing

As expected, based on the results from Phase 3 Stage 1, the CBR values obtained at the top of the specimens after soaking were lower than those at the bottom. This is likely to be due to the confinement of the specimen as described in Section 3.4. Increased water content was also measured at the top compared to the bottom. This was more significant for the high sulfate material (H-TPS) across the range of binders tested. Average water contents of the H-TPS specimens increased during the 28 day CBR swell test along with evidence of volumetric expansion. No significant increase was noted in any of the L-TPS specimens. A possible explanation for this would be expansive sulfate minerals breaking the bonds in the cementitious matrix resulting in an increase in permeability.

Significant swell was observed for all specimens manufactured using H-TPS, with the greatest CBR swell reported for H-TPS 7 (1.5% Lime). All the specimens manufactured using this material failed the criteria for durability in both the CBR swell test and European accelerated swelling test. Conversely, all specimens manufactured with L-TPS passed both criteria. As with the Phase 3, Stage 1 testing, all three swell tests gave differing degrees of failure, for example H-TPS-9 CBR specimens resulted in swells of <10 mm, whereas the European accelerated swelling test specimens showed >60% expansion. These results would suggest that the material requires further study by the criteria of the soaked CBR swell test but would be unsuitable for use under the European accelerated swelling test. The cemented H-TPS specimens subjected to the standard European test had less time than the specimens produced with lime only to cure prior to being immersed, which may explain the difference in expansion (i.e. 20% difference). The addition of further calcium aluminate in the form of cement may also promote additional reactions which form deleterious sulfate minerals such as gypsum and ettringite, both of which are associated with disruptive volumetric changes. Further work would be required to investigate the chemistry associated with volumetric stability of stabilised soils.

The ‘loss of strength on immersion specimens’ (H-TPS-9) lost all of their strength and gave a volumetric expansion of 21%. The reduced expansion relative to specimens undergoing the European volumetric expansion regime (21% expansion for the former compared to 46% for the latter) could be attributed to the 14 day air curing period for the former before going into soak, or to the size and density of the smaller, less dense European accelerated swelling test specimens. Increased curing times for materials treated with hydraulic binder(s) are associated with development of the cementitious matrix and associated improvement in mechanical properties. The smaller specimens used in the European accelerated swelling test would result in greater water penetration in relation to its size, compared to specimens undergoing the loss of strength on immersion test, resulting in a greater percentage increase in expansion.

The L-TPS-12 specimens passed the durability criteria of the CBR swell and the European accelerated swelling test; however, the material failed the loss of strength on immersion test, achieving only 70% of the unsoaked strength. It was noted in the laboratory that, visually, the material seemed to have softened, although the results had not shown any expansion.

5. Conclusions

The principle aim of a durability test is to assess a material's performance over its designed life. In relation to the volumetric stability of stabilised soils, the durability tests investigated must predict whether or not a particular material is suitable for this purpose. This is given by the pass/fail criteria of the test.

The CBR swell test will generally have a variation in CBR value between the top and bottom of the test specimen. A specimen could fail the criterion one end and pass at the other. BS 1924-2: 1990 states that the top and the bottom of the specimen should be tested. The recently issued HA 74/07 (2007) recommends the use of BS EN 13286-47: 2004. This is a linear swelling test for measuring the swell and strength of a specimen in which the bottom of the specimen only is inverted, soaked and subjected to the CBR swell test.

The newly adopted CBR swell test is potentially more conservative (in relation to current guidance) than the test it replaces.

It was noted that during the CBR swell test, water did not distribute equally throughout the specimen, and displayed a very clear moisture content difference between the top and the bottom of the specimen. This could be attributed to a number of factors such as the density of the specimen and the air voids it contains. However, it is suggested that the main reason for the difference is the material's confinement within the mould, and/or the adhesion/friction between the specimen and the mould. As the air voids are filled, the confinement of the specimen by the mould prevents it from absorbing further water. Therefore expansion and hence water absorption is reduced. This is evident in the specimens undergoing the 28 day CBR swell test, since the water content at the top of the specimens was greater than at the bottom, due to the top of the specimen being less confined and free to expand. It is therefore recommended that the use of the new swell test procedure in BS EN 13286-47: 2004, as set out in HA74/07, is used instead of the older BS 1924-2 1990 test method.

Specimen preparation methodology will affect the results of the testing as shown in Phase 3 Stage 1 of the laboratory programme. This should be undertaken in a manner that will reflect the likely in situ design specification in terms of MCV, degree of pulverisation and dry density, so that the durability test results are meaningful and can be related to the durability of the material in situ.

Both the CBR swell test and European accelerated swelling test failed the high TPS material while passing the low TPS material. Non-standard curing of the lime and cement treated material gave a reduction in the observed volumetric expansion compared to the standard curing regime. This is likely to be due to the greater development of the cementitious matrix prior to immersion. The loss of strength on immersion test failed both the high and low TPS materials, suggesting that the >80% pass/fail criterion is possibly too severe (note that French practice adopts a >60% pass/fail criterion for fine-grained mixtures).

Clearly high TPS contents infer that the material would be problematic with respect to stabilisation and this is well documented in the literature. The TPS content of the H-TPS and L-TPS materials was 4.55% and 0.29%, respectively. The literature suggests that TPS levels as low as 0.3% can lead to the formation of detrimental sulfate minerals under favourable environmental conditions. The curing time prior to test generally results in an improvement in performance. Further work is being undertaken in order to determine how appropriate the current test methodologies are in relation to predicting the behaviour of stabilised soils containing significant TPS. This is being undertaken as part of a separate SW and NTEC MPhil study. Test samples were taken from all of the Oxford Clay specimens after curing, these are to be tested for mineralogy using the Xray Deffraction analysis, and will be reported separately. The causes/reasons for the volumetric expansion such as ettringite formation and/or high montmorillonite content are outside the scope of this paper.

The research findings indicate that the CBR swell test procedure (recommended by HA74/00) seemed to pass a material as being suitable for stabilisation, whereas the European accelerated swelling test would sometimes fail the same material. When comparing the pass/fail criteria from the three durability test methods, all three resulted in differing recommendations: The test most likely to pass a material as being suitable was the CBR swell test, with the loss of strength on immersion test, with a limit of > 80%, being the most difficult test criterion to satisfy.

Interim conclusions suggest that the CBR swell test as set out in BS 1924: 1990 is not sufficiently rigorous to be used as the sole determinant for the durability (specifically volumetric stability) of stabilised soils, Out of the 12 mixtures tested, the CBR swell test alone would have passed 9 of the mixtures as suitable for use. The European accelerated swelling test alone, however, would have permitted just one of the mixtures to be used. There is therefore a clear case that one test may pass a specimen whereas a different test may fail the same specimen. This is therefore an area that needs further investigation in order to give confidence to the user as to which test is preferred.

Unanswered questions regarding the EN13286-49 Swell test:

Due to the size of the specimens being prepared (50mm), the maximum particle size in the specimen has to be <6.3mm, therefore possibly being unrepresentative of material being stabilised?

The curing time of 1.5 to 2 times the workability of the material seems very conservative?

Example:

If, a clay material is modified with lime, and then mixed with cement prior to compaction and trimming. According to the standard it must go into soak within 2 hours from the addition of cement.

This seems to be a very short curing period before immersion?

Has the specimen had a chance after only 2 hours curing period, to gain sufficient strength before trying to overcome the naturally expansive nature of the material?

Would machines be allowed on this material on site, at such an early age?

When testing soil cement treated materials, there is a choice between the loss of strength/stiffness on immersion test, or the prEN13286-49 volumetric swell test.
As mentioned above the volumetric swell test does seem fairly onerous for these types of materials as they have only got up to 2 hours hydration/strength development before being immersed, compared with the 14 days air cured alternative test.

Note: It is important that the engineer considering the use of stabilised materials must realise that these durability tests must be conducted as complimentary tests to the chemical analysis and not instead of the chemical analysis (i.e. total potential sulfate contents, etc), when making an assessment of a soil's suitability for stabilisation.

Until further investigation suggests otherwise it is recommended that:

A: The CBR swell test conducted in accordance with B.S.1924: 1990: Part 2 is withdrawn, and replaced with the new B.S.EN 13286-47: 2004 CBR Linear Swell. It is also recommended that either a liner or releasing agent be used to reduce internal friction between the specimen and the sides of the mould.

B: When considering a soils suitability for use as a Capping material (foundation class 1) then both the new B.S.EN 13286-47: 2004 CBR Linear Swell test & B.S.EN 13286-49: 2004 Accelerated Swelling test be considered for use.

C: When considering a soil's suitability for use as a sub-base (foundation class 2 or greater) material then both the new B.S.EN 13286-49: 2004 Accelerated Swelling test & the Loss of strength on immersion (Volume stability) test in MCHW Vol 1: series 800: clause 880 be considered for use.

Note: Further work outside the scope of this research is currently being conducted to develop a modified swell test using the MCV apparatus as potentially a more practical size of test to replicate the performance of the in-situ material, as it can accommodate a large particle size, be compacted in a single layer, is cylindrical and can be extruded and tested in its unconfined state.

It is recommended that further investigation is required to develop appropriate guidance, along with a comparison between the HA74/07 swell test method and the MCHW series 800 volumetric expansion method. Consideration could also be given to comparing the swell limits set out in BS EN 14227-11. One dimensional and volumetric categories may be a more useful approach, although at present linear swell category 1 may not necessarily equate to the volumetric expansion category 1. (see Table 2).

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Appendix A Material Suitability Criteria

Material Suitability Criteria as set out in HA74/07

Material Property	Defined and tested in accordance with:	Class 7E/9D	Class 7I/9E
Plastic Limit	BS 1377:1990:Part2	✓	✓
Liquid Limit	BS 1377:1990:Part2	✓	✓
Plasticity Index	BS 1377:1990:Part2	✓	✓
Particle Size Distribution	BS 1377:1990:Part2	✓	✓
Organic Matter	BS 1377:1990:Part3	✓	✓
Total sulfate content	BS 1377:1990:Part2	✓	✓
Total sulphur content	BS 1047:1990:Part2	✓	✓
Initial Lime Consumption	BS 1924:1990:Part2	✓	X
California Bearing Ratio	BS 1924:1990:Part2	✓	✓
Swelling	BS 1924:1990:Part2	✓	✓
Moisture Condition Value (for stabilised material)	BS 1924:1990:Part2	✓	✓
Moisture Content (for stabilised material)	BS 1924:1990:Part2	X	X
Optimum Moisture Content (for stabilised material; 2.5kg test)	BS 1924:1990:Part2	✓	✓
Degree of Pulverisation (>30% for Lime treated materials & >60% for Cement treated materials)	BS 1924:1990:Part2	✓	✓

✓ = test required under current guidelines.

X = test not required under current guidelines.