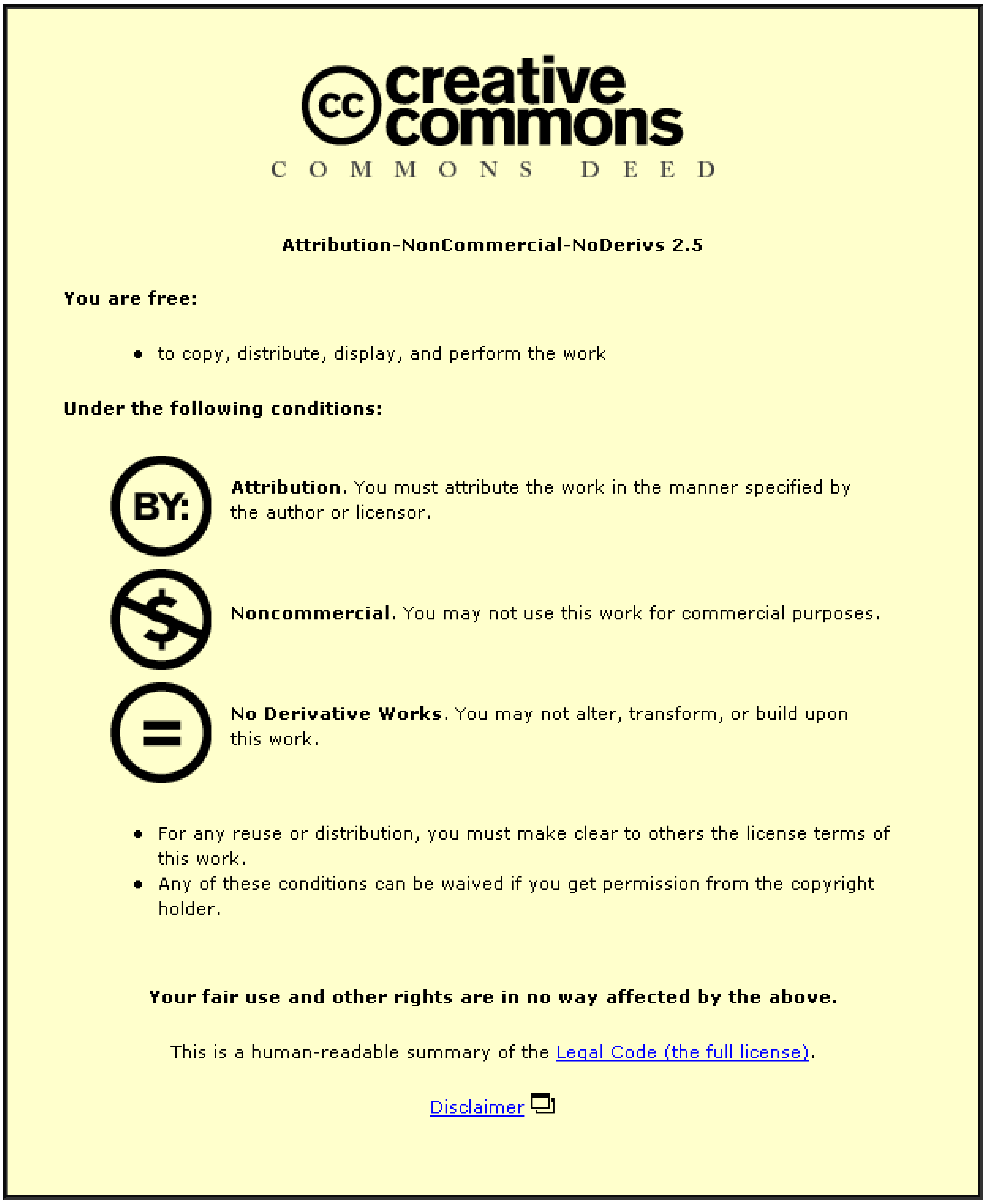

DEEP STABILISATION OF SLOPES USING LIME PILES

by

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A Doctoral Thesis in partial fulfilment of the requirements for the award of Doctor of Philosophy of the Loughborough University of Technology.

February 1995

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ABSTRACT

A comprehensive review of the literature provided much evidence of the success of lime piles in treating both soft ground and slopes. The mechanisms of stabilisation postulated by researchers is often contradictory or misleading. The use of the literature for the basis of a definitive experimental programme was not possible.

An iterative approach was adopted for the laboratory programme whereby the results from one series of tests were used in the design of the next. This resulted in a range of tests including full-scale box loading tests in which lime piles were installed in clay samples, model scale lime pile tests and soil element tests.

The stabilising mechanisms that have been established by the laboratory study are: generation of negative pore water pressure, overconsolidation of the shear zone, clay dehydration, pile strength and increased strength of stabilised clay due to lime migration. These mechanisms combine to improve any particular clay slope containing one or more shear zones.

Three field trials were conducted. A small-scale trial was carried out on a canal cutting and provided useful data regarding pore water pressure changes and installation processes. Quantitative data produced by the laboratory study, were used to design two further trials. One trial treated a 30 m stretch of failing slope using a single 'Minuteman' rig (small and lightweight plant). Quicklime was 'poured' into open holes and compacted by the drill operators. Work was complete within two weeks. The third trial, again sited on a canal cutting, was carried out using a much larger rig. One hundred and fifty 200 mm diameter piles were constructed to a depth of 3 metres within a two week period. Monitoring of pore water pressures on both sites is still occurring on a regular basis. Excavation of sections of both trials at some future date will provide additional data on stabilising mechanisms.

The research has considerably extended the understanding of the mechanisms controlling lime pile stabilisation, particularly when applied to failing slopes in British soils. Areas where further research would improve this understanding have been highlighted and in some cases work is already underway.

ACKNOWLEDGEMENTS

The project was funded jointly by four industrial partners and the Science and Engineering Research Council via the LINK Scheme in Transport, Infrastructure and Operations. This funding, and the considerable assistance of Howard Wyborn of the above organisation, is gratefully acknowledged.

The four industrial partners, Geotechnics Ltd, Cementation Piling and Foundations Ltd, British Waterways and Buxton Lime Industries, are acknowledged for their financial support. The representatives from these organisations, Len Threadgold, David Greenwood, Graham Holland and Hedley Greaves, are thanked for their support and enthusiastic input throughout the project, especially through such financially testing times.

Dr Chris Rogers is thanked for his role as supervisor to the project, his support and encouragement and for his understanding when informed of my 18 weeks maternity leave!

Thanks are also extended to the various support staff at Loughborough: to the technical staff who have assisted with laboratory and field work and to the secretarial staff who have helped with report production. In particular, Mark Harrod and John Salsbury are thanked for their numerous trips to read piezometers, and Eileen Willson who has typed reports with very little available time.

Finally, I would like to thank all those researchers in the Department who have offered help and understanding through the difficult periods that almost all research projects produce.

GLOSSARY OF TERMS

CAPITAL LETTERS

C_c	Compression Index
G_s	Specific gravity of solid particles
K_0	Coefficient of earth pressure at rest
Pc	Preconsolidation pressure, Yield stress in consolidation
T	Temperature (^oC)

LOWER CASE LETTERS

c	Unit cohesion with respect to total stresses
c'	Unit cohesion with respect to effective stresses
c_u	Undrained shear strength of saturated clay
e	Void ratio
k	Coefficient of permeability, Migration constant
q_u	Unconfined compressive strength
t	Time

GREEK LETTERS

γ	Unit weight
γ_t	Moist unit weight
Δ, δ	Change in ....
ϕ	Shear strength parameter
ϕ_u	Undrained (total stress) shear strength parameter
ϕ'	Drained (effective stress) shear strength parameter
ϕ'_r	Drained residual shear strength parameter
ω	Water content

ABBREVIATIONS

bgl	Below ground level
BH	Borehole
BS	British Standard
BSI	British Standards Institution
CBR	California Bearing Ratio
Dia	Diameter
GL	Ground level
IRS	Infra-red spectroscopy
LL	Liquid Limit
MC	Moisture content
OCR	Overconsolidation ratio
OMC	Optimum Moisture Content
PFA	Pulverised Fuel Ash
PI	Plasticity Index
PL	Plastic Limit
SEM	Scanning Electron Microscope
TEC	Total exchange capacity
UCS	Unconfined compressive strength
XRD	X-Ray diffraction

CHAPTER 1

1.0 INTRODUCTION

1.1 SLOPE INSTABILITY

Slopes have been failing for as long as slopes have existed. Slope instability only became a problem when its consequences impinged upon the lives of humans. In primitive societies the solution was simple: move away from the problem. As societies developed and dwellings became more sophisticated the consequences of slope failure became more catastrophic and this simple solution became more problematical. Even in recent times villages built on the loess terraces of Northern China may be entirely destroyed by a single slide, yet the land is valuable in terms of farming and the houses get rebuilt. In modern societies, in many instances land is in short supply, particularly now that the further development of so called 'green field' sites is being limited and controlled. It has become necessary to develop means of rendering unstable slopes stable in order for society to develop in the way it wishes.

Another aspect of slope stability for a developing society is the need to create additional slopes in order to meet its developing needs. This may be in the form of a cutting through an existing landform or the building of an embankment. Particular applications for this include the desire to create more direct transportation routes without unnecessarily steep gradients. Hence the problem of slope instability has developed from an avoidance of a natural hazard to include the design of an engineered 'structure'.

In order to design properly solutions to this problem an understanding of mechanics of slope stability had to be gained. In order for this to be achieved, an understanding of the nature of the material comprising the slope had to be reached. Soil mechanics is a relatively new science and one which is still developing. The development of slope analysis techniques began in Sweden in 1916 as a means of solving the particular problem of failing harbour walls. It was noted then that many of the failure surfaces investigated were near circular in shape. Hence, a method of analysis based on frictional considerations along a circular failure plane was developed over the following years. The concept of pore water pressure and effective stress were only introduced in 1936 by Terzaghi. A method of analysis based on the side forces acting when a slope is divided into a number of slices was developed by Bishop in 1955. In addition to mechanical and material properties this method allowed the pore water pressure in any slice to be considered. This type of model was further developed when the use of computers allowed vast numbers of computations to be made in a short space of time.

Sophisticated methods of analysis have therefore been developed. These, however are only as accurate as the soil parameters supplied to them and, despite the development of more efficient sampling and testing techniques, many uncertainties remain in predicting the stability of slopes.

Despite the problems many methods of slope stabilisation have been developed. These involve improving the mechanical or material properties of the slope, or the pore water pressure regime within that slope. Details of different improvement techniques will be discussed in Chapter 2. However, many of the existing techniques are costly in terms of materials used, the equipment required or the disruption caused whilst the works are in progress. Consider the stabilisation of a cutting slope adjacent to a major motorway. The cost of losing one or more lanes of traffic for several working weeks is enormous. There appears to be considerable benefit in the development of a slope stabilising technique that uses relatively cheap materials and equipment and allows work to progress with a minimum of disruption.

UK transportation routes are characterised by cutting and embankment slopes in highly overconsolidated clays. A recent survey of 570 km of motorways in England and Wales, conducted by Perry (1989), has found that over 17 km of embankment and 5.5 km of cutting slope, are failing. It also stated that three times that number were likely to fail in the future. The problem of time-dependent failure of slopes in overconsolidated clays occurs as a result of equilibration of pore water pressures. Excavation of overconsolidated clays, in the case of cuttings, and their recompaction, in the case of embankments, results in stress relief and considerable negative pore water pressure. These high suctions cause the embankment slopes to achieve a high degree of stability immediately after construction. It is therefore only after water ingress, and thus an increase in pore water pressure from a negative base, that problems of instability arise.

It has been shown by Crabb and Atkinson (1991) that the depths at which slip planes typically occur lie within the zone that is affected by seasonal pore water pressure variations, i.e. less than 1.5 m . The prevalence of shallow slips within 25 years of construction of motorway embankments is a result of the pore water pressure distributions and deep-seated slips are only likely to occur after a greater period of time, depending on the local water regime. While this can be considered to be a generalisation only, it is true also of the experience of the British Waterways Board.

A treatment method suitable to treat these shallow slips would therefore seem appropriate. Many techniques are used world-wide, one of those being the use of lime piles.

1.2 THE USAGE OF LIME

Lime, when intimately mixed with clay material will improve its material properties. Traditionally, this has been exploited in road building where the stiffness and strength characteristics of *in situ* material can be significantly enhanced. The Roman Appian Way is said to have been the first lime stabilised road. This usage has largely been developed overseas, although it is, more recently, becoming more popular in this country.

An example of this mixing technique for slope stability is described by Cobbe and Wrench (1990). Lime was mixed in layers with one of the Lias clays in order to improve its frictional and 'cohesive' properties, the latter via cementation reactions, and hence the stability of the slope. This is not very efficient since it requires a large amount of excavation of material which needs to be mixed with lime before compaction. It also requires space to operate and relatively large plant to carry out the work. An *in situ* technique, especially one which could be carried out using small-scale plant, is therefore required.

Deep stabilising techniques involving lime were pioneered by the Japanese in the 1970s as the increasing industrialisation and the shortage of land for development forced engineers to build on the deep soft deposits surrounding industrialised centres. The method involved the *in situ* mixing of lime (or cement) to form columns of lime stabilised material. Applications included (amongst others) settlement control underneath, and improvement of slope stability of, embankments. A similar method of forming lime columns was also developed simultaneously in Sweden.

A second technique is lime piles, which are, very basically, holes in the ground filled with lime. The method of forming the hole, the type of lime used and the filling technique are different in each country in which they have been used. It is more difficult to plot the course of the development of the usage of lime piles. It appears from the literature described in Chapter 2 that many countries have developed some system for the use of lime piles, generally with some degree of success. However, there appears to be no consistent means of construction employed to create the piles or material from which the piles are formed.

1.3 LIME PILES - A DEFINITION

For the purposes of this thesis it is necessary at this early stage to define exactly what is meant by lime pile. Firstly, it does not involve any degree of mixing of lime with clay. It is merely a hole in the ground filled with lime. The method of formation of the hole will depend on the ground conditions. One means of construction is illustrated in Figure 1.1. Further details of this technique are discussed in Chapter 2. A different technique, and the one being considered in this research project both in the laboratory and in the field, is one in which the hole is formed by augering and removing the *in situ* material prior to lime installation. The term lime is also a source of confusion. It may refer to agricultural lime (calcium carbonate), slaked lime (calcium hydroxide), in a powdered or slurry form, and quicklime (calcium oxide). The literature includes piles of each of these types of lime except agricultural lime. The project reported herein is concerned with quicklime. In summary, except when otherwise stated, lime pile refers to an augered hole filled with quicklime.

1.4 THE PROJECT

The requirement for slope stabilisation has been established along with the specific requirement for a novel cheap method of doing so. The potential for the use of lime in such a technique has also been proven. The method of lime piles appeared to satisfy the remaining criterion concerning the use of simple plant that would create minimum disruption. The aim of this project was to investigate the mechanism(s) of lime pile stabilisation and to develop design, specification and installation methods suitable for application to British clays.

Chapter 2 describes the literature associated with stabilisation techniques and the uses of lime, sufficient to provide a basis for the understanding of the lime pile treatment method. Specific literature describing the use of lime piles is presented in detail.

From the information gained in the literature a research philosophy was formulated and this is presented in Chapter 3. More specific information regarding experimental methods is described in Chapter 4. Data collected from the resulting laboratory programme are presented in Chapter 5 and discussed in detail in Chapter 6. A summary of this work appears in Rogers and Glendinning, 1994. The culmination of this discussion is the presentation of a description of the stabilising mechanisms which has resulted from the examination of the findings from the experiments.

Chapter 7 describes how the quantification of the elements of the stabilising mechanisms has been used to develop a design method. The design process involved the development of a computer model by an allied research project. This was based on a computer based slope stability analysis technique similar to those described above.

Chapter 8 describes the practical application of the technique in three field trials. The first provided supplementary data for the laboratory programme, whilst the other two were carried out in order to develop further the design and installation procedures. The preliminary results of a monitoring programme are also described.

Lastly, Chapter 9 concludes the discussion and points towards areas requiring further investigation, both in terms of providing supplementary data for this project and entirely new projects stemming from the literature review. The potential for application of lime piles to embankment clay fills is reviewed in Rogers and Glendinning, 1993.

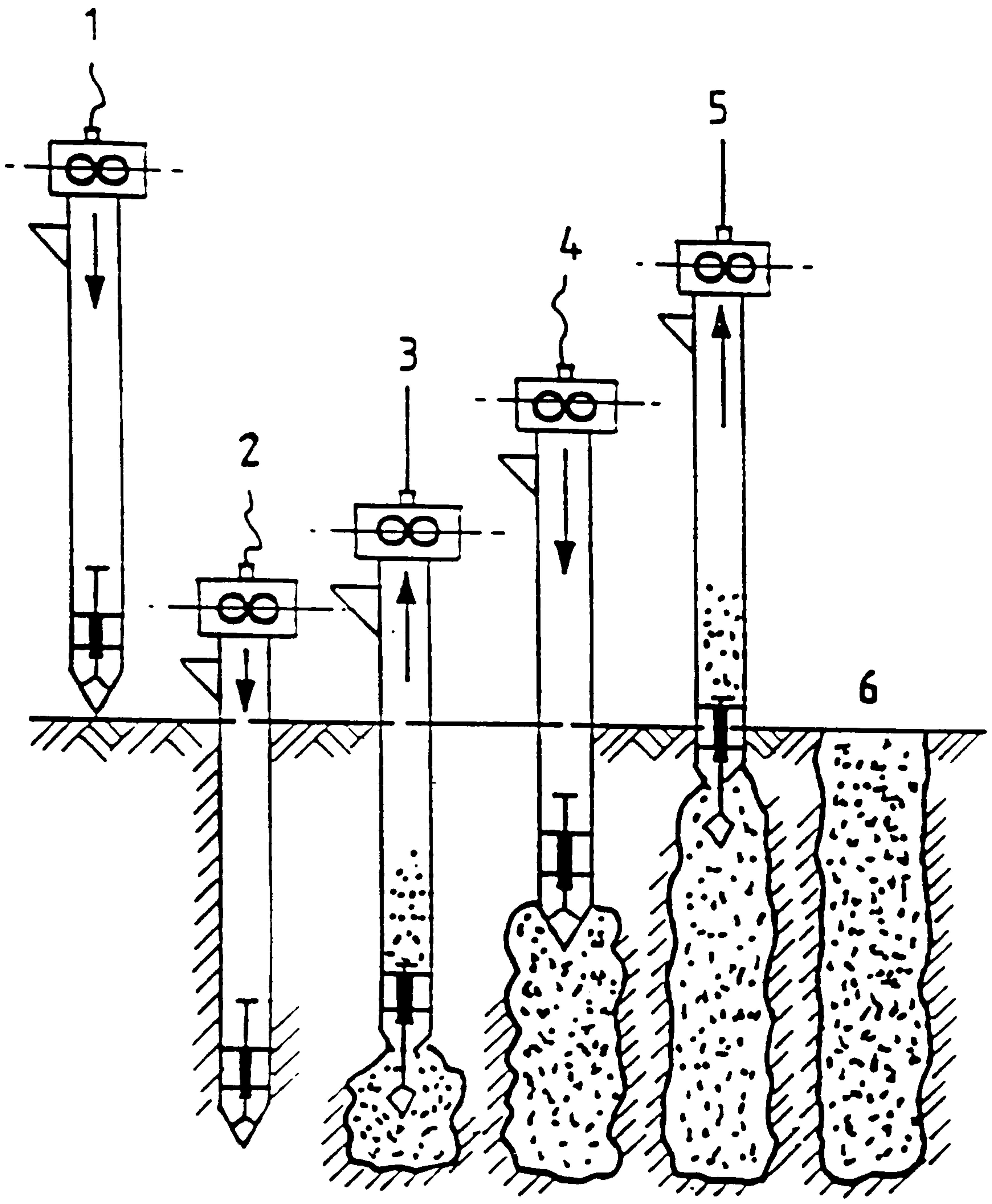


Figure 1.1 Procedure for construction of lime piles in soft soils (after Ingles and Metcalf, 1972)

CHAPTER 2

2.0 LITERATURE REVIEW

2.1 STABILISATION TECHNIQUES

It is worth firstly establishing what is meant by stabilisation. Essentially, it is the alteration of a soil's properties to meet specific engineering requirements. This requirement will govern the property needing alteration and the means by which it may be achieved.

In general, the properties to be considered include: volume stability; strength and/or stiffness; permeability and durability. More specifically, the appropriate measure of each property must be considered for each specific engineering problem. Considering strength, for example, in most forms of construction it is the bearing capacity or deformation resistance that is most commonly considered whereas for slope stability it is the shearing resistance of the soil that must be determined. Although similar design parameters might be used for different applications careful consideration must be given as to the precise parameters, and to the conditions under which they are measured, such that they are appropriate to each problem.

The means available for stabilisation broadly include chemical, thermal and mechanical processes. Again these very general titles can be broken down into specific processes, each having their specific applications. However, it is not to say that one particular process is parameter specific. Indeed, the process may be applied to alter several parameters, simultaneously. Conversely, a process used to alter one parameter may have an adverse affect on another. For instance, it may be appreciated that strength and volume stability are inter-related. An internal clay water system which is susceptible to volume change with changing moisture content may also undergo changes in the inter-particulate forces which give the clay its strength: improving one parameter hence improves the other. However, a mechanical solution to the problem of low bearing capacity may be densification by compaction. This may induce high pore water pressures which in turn, reduce strength until equilibration of these pressures occurs.

As previously stated, types of ground improvement techniques can broadly be classified into four groups. These are discussed in the following sections.

2.1.1 Mechanical Stabilisation

The density of soil is increased through the application of short-term loading. This includes surface, usually dynamic or impact, compaction by rollers and/or vibrators and deep compaction by heavy surface action or vibration at depth. The various means by which densification of deep deposits is achieved are as follows:-

i) Precompression

Precompression, or preloading, of the soil is usually effected by soil or rubble bunds placed as dead weight of appropriate mass above the stratum or strata concerned. An alternative solution is vacuum preloading by pumping from beneath a large, sealed sheet. In either case the effective stress is raised causing consolidation to a stress level typically greater than the stress that is subsequently to be applied. In saturated fine-grained soils pore water pressures will increase to facilitate the consolidation and vertical drains may need to be installed to permit their rapid dissipation. A similar result (i.e. consolidation) can be achieved by lowering the groundwater table, thus causing an increase in effective stress and hence consolidation. Structures built on such a site will settle less than on untreated ground. This method is usually accompanied by long-term drainage measures in addition to vertical drains and these are discussed in the next section.

ii) Explosion

Explosives are placed in boreholes causing loose fill to compact as a result of shock waves produced on detonation. Excess pore water pressures induced by these shock waves may take time to dissipate, as above.

iii) Dynamic Compaction

Sometimes termed heavy tamping this is achieved by the dropping of a heavy mass from a large height onto the surface causing 'dynamic compaction'. Again, excess pore water pressures may be generated and adequate time between drops must be allowed to permit dissipation. The load applications are typically applied several times at each point on a widely spaced grid pattern initially, the spacing reducing with subsequent treatment passes.

iv) Vibration

A vibrating probe inserted into the ground is used for essentially lateral compaction of granular material. This process is variously known as vibrocompaction or vibrofloatation. The volume loss caused by the compaction is compensated for by compaction of stone in layers by the poker.

v) Compaction Grouting

A viscous cement grout is injected into the ground, under controlled high pressures, thus compacting the surrounding soil.

These methods are more suitable for free-draining cohesionless soils as high induced pore water pressures could cause problems in cohesive soils with lower permeabilities. A different emphasis on the benefits of compaction exists between the two classes of soils due to different changes in micro-structure on compaction.

The inter-particle forces acting on a microstructural level are fundamental to the properties of clay soils, and these are discussed in a later section.

2.1.2 Hydraulic Modification

Pore water is dissipated via drains, which may be pumped if placed in granular soil. In fine-grained soils other inducements to water movement may be applied, such as electro-osmosis. This method of stabilisation is relevant to slope stability problems because the presence of static or flowing water may decrease stability for a number of reasons:

- reduction of shear resistance by increased water content and/or high pore water pressures;
- increase in total mass;
- seepage forces in direction of movement;
- seepage causing erosion and piping;
- changes in physico-chemical characteristics of soil by leaching; and
- increase in liquefaction potential.

Drainage of some form is the most common method of slope stabilisation. This may be in the form of a simple ditch or pattern of ditches, or a series of vertical drains filled with granular material. More recently, geosynthetics have aided the construction of such drains due to their easy handling, tensile strength and filtration properties.

As previously mentioned, pumping and/or loading offer the most common inducements to water movement. However, the method of electro-osmosis has received sporadic attention since Casagrande registered his German patent in 1935. It is worth mentioning here since it induces ion transport in the ground, a subject that shall be discussed further in later sections of this thesis.

The basic principle is to apply an electric potential to a soil to induce a movement of charged water molecules. Electrophoresis, an electrical effect at soil contact zones, may also be induced. This may improve the nature of the forces between naturally charged clay particles. However, as yet this method is largely experimental and extremely expensive as a stabilising technique.

2.1.3 Modification by inclusions and confinement

This includes reinforcement by fibres, bars meshes etc. that impart tensile strength to a constructed soil mass. Reinforcement *in situ* may also be achieved by nails and anchors. As these methods are most applicable to slope stabilisation, they shall be discussed here:

i) Ground anchors

These are tensioned structural units which transmit forces into stable rock or soil into which they are fixed by grouting. They are applied to situations where lateral, uplift, or pullout forces have to be resisted, or where confining pressures have to be generated. Although sometimes applied to unstable slopes, the method of soil nailing is perhaps more applicable to the shallow failure types anticipated as the primary application for lime piles.

ii) Soil Nailing

Soil nails are essentially rigid bars or tubes that are vibrated or driven into soil or placed into boreholes which are subsequently filled with grout. They can be used to provide stability in the sides of excavations, where they carry tension, and in slope stability, where they resist a combination of tension and shear. The application to slope stability will be summarised here.

Soil nails are intended to form a coherent structural entity which may arrest movement of an unstable slope when applied through a failure plane. This method may provide a cost-effective, long-term means of slope stabilisation by:

- providing shear resistance along the failure plane.
- providing a direct stabilising force and increasing friction on the failure plane from the normal pressure exerted on the net, and
- applying normal pressure causing consolidation of the soil, thus increasing the effective stress and increasing shear strength.

Although commonly quoted as providing shear resistance, the shear capacity of soil nails is limited and they are thus typically orientated so that they carry direct tension. It is also thought that the metal nail may provide a short circuit between the electrical potentials thought to exist between differing layers of soil deposits that have caused instability (Vender 1963, 1973 as reviewed by Hausmann, 1990).

Although these techniques have considerable potential for providing a cheap means of stabilising failed slopes, their application in the UK. has been limited, (McGown 1991). Much recent research into general design and analysis methods by Brindle and Barr (1990), Jewell and Pedley (1990), and others should see their use become more widespread.

2.1.4 Physical and Chemical Modification

This may take the form of an additive to the soil or alteration of its temperature by heating or freezing.

Heat treatment is very expensive and is no longer widely in use; freezing is occasionally used as a temporary means of stabilisation during construction. Soil treatment through heating shall briefly be discussed here due to the consideration of the heating effect from the slaking reaction of the lime piles.

Heat treatment is generally considered over 400°C. At around this temperature water is lost from within the clay lattice and a profound and permanent structural change occurs rendering the clay a stronger and more durable material. In practice such heating has been carried out as a surface treatment for road construction in New South Wales and as a deep stabilising technique through heating boreholes in the USSR and Japan.

Obviously such high temperatures are not achievable through the placement of lime piles, but there is evidence that permanent change in certain clay characteristics is brought about after much smaller temperature increases of the order of 20 to 40°C (Jefferson, 1994). Rapid heating causes pore water pressures to increase which, on dissipation, cause consolidation. The heat, in effect, causes an increased load on the clay. Cooling therefore is analogous to an unloading effect. Hence, a permanent deformation of the clay due to heating is produced in normally consolidated soils. It has also been observed that heating some clays causes coagulation and an increase in the Liquid Limit. This effect is most pronounced in clays of high activity, such as Wyoming bentonite. A limited effect has also be observed in Keuper Marl, which may be attributable to the increased dissolved salts content caused by increasing the temperature. Some swelling is also possible after heating highly active clay minerals.

Additive addition

The application of chemical additives may be carried out for a variety of reasons, which include:

- to increase strength;
- to reduce deformability;
- to provide volume stability;
- to reduce permeability;
- to reduce erodibility;
- to increase durability; and
- to control variability.

The most common types of additive include:

- Portland cement
- bitumen and tar
- lime
- chemical grouts

Cement stabilisation shall be discussed further here, lime stabilisation forming the subject of the subsequent sections of the literature review. Portland cement is perhaps the most commonly used additive. Its suitability is independent of soil type and has a wide range of applications. However, fined grained, plastic soils offer difficulty with mixing. Its engineering benefits to soil properties include:

- increased strength and stiffness;
- improved volume stability; and
- increased durability.

Although most commonly used as a surface stabiliser, cement has been used in deep stabilisation as well. Cement slurry mixed with the soil *in situ* has become a widely used method in Japan (Terasai et al, 1979). The method not only relies on the primary hydration reaction of Portland cement (when used in its dry form), but on the secondary reaction between natural pozzolans in the soil with the calcium hydroxide released by the hydrating cement. The principle of the reaction between $Ca(OH)_2$ and natural pozzolans is the basis of lime stabilisation also.

The various stabilising methods and their relative cost implications are summarised on Table 2.1 adapted from Mitchell (1981).

Table 2.1 Summary of Soil Improvement Methods (adapted from Mitchell 1981)

	Method	Principle	Most suitable soil conditions/ types	Maximum effective treatment depth	Special materials required	Special equipment required	Properties of treated material	Special advantages and limitations	Relative cost
	Blasting	Shock waves and vibrations cause displacement, with settlement to higher density	Saturated, clean sands; partly saturated sands and silts (collapsible loess) after flooding	>30m	Explosives, backfill to plug drill holes, hole casings	Setting or drilling machine	Can obtain relative densities to 70-80%, may get variable density; time-dependent gain	Rapid, inexpensive, can treat any size areas; variable properties, no improvement near surfaces, dangerous	Low
	Vibratory probe	Densification by vibration; liquefaction induced settlement under overburden	Saturated or dry clean sand	20m (Ineffective above 3-4m depth)	None	Vibratory pile driver and 750mm dia. open steel pipe	Can obtain relative densities of up to 80%. Ineffective in some sands	Rapid, simple, good under water; soft underlayers may damp vibrations, difficult to penetrate stiff overlayers, not good in partly saturated soils	Moderate
	Vibro-compaction	Densification by vibration and compaction of backfill material	Cohesionless soils with less than 20% fines	30m	Granular backfill, water supply	Vibroflot, crane, pumps	Can obtain high relative densities, good uniformity	Useful in saturated and partly saturated soils, uniformity	Moderate
	Compaction piles	Densification by displacement of pile volume and by vibration during driving	Loose sandy soils; partly saturated clayey soils; loess	>20m	Pile material (often sand or soil plus cement mixture)	Pile driver, special sand pile equipment	Can obtain high densities, good uniformity	Use in soils with fines, uniform compaction, easy to check results; slow, limited improvement in upper 1-2m	Moderate to high

Summary of Soil Improvement Methods (Continued)

	Method	Principle	Most suitable soil conditions/ types	Maximum effective treatment depth	Special materials required	Special equipment required	Properties of treated material	Special advantages and limitations	Relative cost
	Heavy tamping (Dynamic consolidation)	Repeated application of high intensity impacts at surface	Cohesionless soils, waste fills, partly saturated soils	30m ? 10m more realistically	None	Tampers of up to 200 tons, high capacity crane	Can obtain good improvement and reasonable uniformity	Simple, rapid, suitable for some soils with fines; usable above and below water; requires control, must be away from existing structures	Low
	Surcharge fills	Fill of weight in excess of that required permanently is applied to achieve a given amount of settlement in a shorter time; excess fill then removed	Normally consolidated soft clays, silts, organic deposits, completed sanitary landfills	----	Earth fill or other material for loading the site; sand or gravel for drainage blanket	Earth moving equipment; settlement markers, piezometers	Reduced water content, void ratio and compressibility; increased strength	Faster than pre-loading without surcharge, theory well developed; extra material handling; can use vertical drains to reduce time for consolidation	Moderate
	Electro-osmosis	DC current causes water flow from anode towards cathode, where it is removed	Normally consolidated silts and silty clays		Anodes (usually re-bars or aluminium), cathodes (well points or rebars)	DC power supply, wiring, metering systems	Reduced water content and compressibility, increased strength, eletro-chemical hardening	No fill loading required, can use in confined area, relatively fast; non-uniform properties between electrodes, no good in highly conductive soils	High

Summary of Soil Improvement Methods (Continued)

	Method	Principle	Most suitable soil conditions/ types	Maximum effective treatment depth	Special materials required	Special equipment required	Properties of treated material	Special advantages and limitations	Relative cost
	Pressure injected lime	Lime slurry injected to shallow depths under high pressure	Expansive clays	Unlimited, but 2-3m usual	Lime, water, surfactant	Slurry tanks, aggitators, pumps, hoses	Lime encapsulated zones formed by channels resulting from cracks, root holes, hydraulic fracture	Only effective in narrow range of soil conditions	Competitive with other solutions to expansive soil problem
	Displacement grout	Highly viscous grout acts as radial hydraulic jack when pumped in under high pressure	Soft, fine-grained soils; foundation soils with large voids or cavities	Unlimited, but a few metres is usual	Soil, cement, water	Batching equipment, high pressure pumps, hoses	Grout bulbs within compressed soil matrix	Good for correction of differential settlements, filling large voids, careful control required	Low material, high injection
	Electro-kinetic injection	Stabilizing chemicals moved into soil by electro-osmosis or colloids into pores by electro-phoresis	Saturated silts, silty clays (clean sands in case of colloid injection)	Unknown	Chemical stabilizer, colloidal void fillers	DC power supply, anodes, cathodes	Increased strength, reduced compressibility, reduced liquefaction potential	Existing soil and structures not subjected to high pressures; no good in soil with high conductivity	Expensive
	Remove and replace	Foundation soil excavated, improved by drying or admixture, and recompacted	Inorganic soils	10m (?)	Admixture, stabilizers	Excavating, mixing and compaction equipment, dewatering system	Increased strength and stiffness, reduced compressibility	Uniform, controlled foundation soil when replaced; may required large area dewatering	High

Summary of Soil Improvement Methods (Continued)

	Method	Principle	Most suitable soil conditions/ types	Maximum effective treatment depth	Special materials required	Special equipment required	Properties of treated material	Special advantages and limitations	Relative cost
	Structural fills	Structural fill distributes loads to underlying soft soils	Use over soft clays or organic soils, marsh deposits	----	Sand, gravel, fly ash, bottom ash, slag, expanded aggregate, clam shell or oyster shell, incinerator ash	Mixing and compaction equipment	Soft subgrade protected by structural load-bearing fill	High strength, good load distribution to underlying soft soils	Low to high
	Mix-in-place piles and walls	Lime, cement, or asphalt introduced through rotating auger or special in-place mixer	All soft or loose inorganic soils	> 20m	Cement, lime, asphalt, or chemical stabilizer	Drill rig, rotary cutting and mixing head, adhesive protectioning equipment	Solidified soil piles or walls of relatively high strength	Uses native soil, reduced lateral support requirements during excavation, difficult quality control	Moderate to high
	Heating	Drying at low temperatures; alteration of clays at intermediate temperatures (400-600°C); fusion at high temperatures (>1000°C)	Fine-grained soils, especially partly saturated clays and silts, loess	15m	Fuel	Fuel tanks, burners, blowers	Reduced water content, plasticity, water sensitivity; increased strength	Can obtain irreversible improvements in properties; can introduce stabilizers with hot gases	High
	Freezing	Freeze soft, wet ground to increase its strength and stiffness	All soils	Several metres	Refrigerant	Refrigeration system	Increased strength and stiffness; reduced permeability	No good in flowing ground water, temporary	High

Summary of Soil Improvement Methods (Continued)

	Method	Principle	Most suitable soil conditions/ types	Maximum effective treatment depth	Special materials required	Special equipment required	Properties of treated material	Special advantages and limitations	Relative cost
	Vibro-replacement stone and sand columns	Hole jetted into soft, fine-grained soil and backfilled with densely compacted gravel or sand	Soft clays and alluvial deposits	20m	Gravel or crushed rock backfill	Vibroflot, crane or vibrocat, water	Increased bearing capacity, reduced settlements	Faster than pre-compression, avoids dewatering required for remove and replace; limited bearing capacity	Moderate to high
	Root piles, soil nailing	Inclusions used to carry tension, shear, compression	All soils	?	Reinforcing bars, cement grout	Drilling and grouting equipment	Reinforced zone behaves as a coherent mass	In-situ reinforcement for soils that cannot be grouted or mixed-in-place with admixtures	Moderate to high

It is worth, at this stage, briefly discussing previous usage of lime as a stabiliser. Obviously, as the previous discussion indicates, it is important to remember the exact problems being addressed by the stabilisation method and the alterations required to be brought about. These may be very different from the problem being addressed by this project. This should be considered when attempting to apply any information presented to the future study of the principles and applications of lime piles.

2.2 LIME STABILISATION

2.2.1 What is meant by 'lime'?

The term 'lime' broadly relates to the calcination products of both calcitic or dolomitic limestones. Calcitic lime in either its oxide or 'quick' (CaO), or its hydrated or 'slaked' ($Ca(OH)_2$), forms are used in the UK for chemical stabilisation. Calcium carbonate ($CaC0_3$) is used solely for agricultural purposes.

i) Quicklime

This form of lime reacts rapidly with water to form slaked lime:

$CaO + H_2O \rightarrow Ca(OH)_2$

Considerable heat is generated by the reaction (65.3 kJ/mol) and considerable expansion occurs since the relative volume of quicklime to slaked lime is 1:1.99. This combination of heat generation and expansion causes handling problems and considerable care must be exercised. However, these effects are advantageous when treating heavy, wet clay soils.

ii) Slaked Lime

Due to its greater volume and mass the available lime to weight ratio is 25% higher for quicklime than for slaked lime. This makes slaked lime less economical for transportation purposes, although the problem of the

> explosive reaction with water is eradicated. It is the more suitable form for the treatment of relatively dry clay soils.

In dolomitic lime some of the calcium has been substituted by magnesium. As there is less 'available' lime, mass for mass it is less effective as a stabiliser than either of the forms described above. Hydraulic, or grey limes are also used for stabilisation and are an impure form of calcium carbonate, containing a proportion of clay. Although, again the impurities mean that there is less 'available' lime for the initial reaction, the presence of silicates and aluminates may have a beneficial effect on the long term strength.

The manufacture of the purest forms of lime is an inefficient process requiring considerable amounts of energy. For this reason, the impure forms of lime are those generally used in third world countries.

2.2.2 Clay Mineralogy and the Lime-Clay Reactions

The basis of lime stabilisation is the chemical reaction between lime and some of the constituents of clay. Hence, it can be broadly categorised as a chemical stabilising technique. However, the precise nature of the lime-clay reaction makes its definition more complex. As a chemical stabilising technique, an understanding can only be achieved if the nature of the chemicals taking part in the reactions are understood. Lime, in its various forms, has been dealt with above. The following sections will attempt to deal with the complex nature of clay itself. This shall be followed by a discussion of the clay lime reaction within this context.

It may appear that a disproportionately large amount of attention is being paid to a seemingly obscure topic, but clay mineralogy will be referred to many times in the text of this thesis and so a brief summary is appropriate at this stage. In any case, according to Mitchell (1976):

> "A knowledge of soil mineralogy is essential to a fundamental understanding of soil behaviour. Mineralogy is

> the primary factor controlling the size, shape, and physical and chemical properties of soil particles"

As the project is aimed at the development of the lime pile technique in the UK, it was felt that the mineralogy of British Clays should be concentrated upon. An introduction to mineralogy and the clay-lime reaction shall be given by a discussion of the basic minerals present in such clays. It has been shown that the success of lime stabilisation depends directly upon the percentage and type of clay minerals present in the soil. Therefore, it is this part of the soil mineralogy that shall be concentrated upon.

The term clay itself may be ambiguous. Often engineers use 'clay' to describe that part of the soil fraction smaller than 2μm. In this case, it shall be used to describe material of a particular mineral content. The discussion of the nature of this mineral content, and the effects of lime upon it, is divided hereafter into Clay Structure, Clay - Water Systems and the Clay - Water - Lime System, both in terms of lime modification and lime stabilisation.

2.2.2.1 Clay Structure

In general, the building blocks of the clay structure are of two basic types: a layer of silicon-oxygen and layer of aluminium-oxygen-hydrogen atoms. Within these layers the atoms themselves form differing structures by the way in which they bond with one another (Figure 2.1).

The atoms within the silicon-oxygen layer form a tetrahedral structure, each silicon atom being surrounded by four oxygen atoms at equidistant spaces. This provides a very stable structure, both structurally and electrostatically. Each of the tetrahedra $(SiO_4)^{4-}$ are linked by sharing three basal oxygens with adjacent tetrahedra to produce a sheet with composition $(SiO_{10})^{4-}$.

In the other layer, an aluminium ion is surrounded by six oxygen and hydrogen atoms in the form of hydroxyl groups. This structure is less stable than that of the tetrahedra due to the valence requirements of the metal ion.

When the layers are being formed, via weathering processes on the parent materials, other elements may replace both silicon and aluminium within the structure. This usually means aluminium replacement of silicon and magnesium or iron replacement of aluminium. There may even, on occasion, be no central ion present at all. When aluminium is present only two thirds of the possible positions in the atomic lattice are filled to balance the structure. This produces the dioctahedral or gibbsite structure, $Al_2(OH)_6$. However, when magnesium is present all the positions are filled to balance the structure, with this arrangement being known as the trioctahedral or brucite structure, $Mg_3(OH)_6$. The difference between the basic structural units is because of the different valencies of aluminium and magnesium. Because of the different valence requirements of the replaced ions an overall electrostatic imbalance is created, leaving the crystal with a negative charge.

The way in which the structural units are stacked and the degree of charge imbalance characterises the differing clay minerals and the way in which they behave in different environments. The structure and properties of the most relevant clay minerals will be discussed in the following sections.

(i) 1:1 Structural Group

These minerals have a two-layer unit, with one tetrahedral layer for every octahedral layer, see Figure 2.2.

They can be divided into two distinct groups: the dioctahedral kaolin group and the trioctahedral serpentine group. The kaolin group is the more commonly experienced and shall form the rest of this discussion.

Members of this group include, kaolinite, dickite, nacrite and halloysite. The main difference between them is the way in which the 1:1 layers are vertically stacked. This affects the spacing between layers and whether water is present between those layers. This discussion will centre around kaolinite as it is the more abundant in the UK. A relatively pure sample of kaolinite has also been used for a number of the laboratory experiments reported in this thesis.

Kaolinite has an inter layer spacing of 7.15 Å (O.715nm) with bonding between layers by means of weak hydrogen and Van der Waals forces between the hydroxyl groups of the octahedral sheet and the oxygen of the adjacent tetrahedral sheet. The number of layers that can be stacked together is limited, resulting in flat, hexagonal lamellar crystals.

Substitution of aluminium is rare giving kaolinite a balanced structure with no overall charge.

(ii) 2:1 Group

Minerals of this type consist of three layer units with two silicate sheets surrounding an octahedral sheet (Figure 2.2).

The tips of the tretrahedra point in the same direction, one from each silica sheet combining with one of the hydroxyl layers of the octahedral sheet. Atoms common to both become oxygen rather than hydroxyl ions. Weak bonds exist between units. Isomorphous substitution within 2:1 layers carry a net negative charge. This charge is important in the clay-water system set up and shall be discussed in the next section.

Common minerals within this group include montmorillonite and illite, both members of the Aluminian Dioctahedral Smectite Group. Montmorillonite receives the majority of its charge from substitution on octahedral sites (Amphlett, 1964) usually by magnesium. It has the general formula:

$(Al_{3.14}\ Mg_{0.08})\ (Si_{8.00})\ O_{20}\ (OH)_4\ x_{0.85}\ nH_2O.$

Where X is a monovalent interlayer cation (see next section).

Illite receives its charge from aluminium ions in the tetrahedral sites, and has the general formula:

$(Al_{4.00})\ (Si_{7.15}\ AL_{0.85})_{20}\ (OH)_4\ X_{0.85}\ nH_2O.$

2.2.2.2 Effect of Structure on Engineering Properties

The stacking of the basic structural units and extent and source of charge imbalance has a profound effect on engineering properties.

In kaolinite, the limited stacking range of the layers causes the formation of flat lamellar crystals. This results in kaolinite having an open texture. A stable structure is created by the positioning of adjacent unlike units, mutually attracted. The platy structure also leads to a relatively high shear strength, the particles having to be aligned in parallel for residual strength to be achieved

The 2:1 minerals have like groups positioned adjacent to one another, their hydroxyl groups creating inherent repulsive forces within the crystal. A cleavage plane exists between units due to weak bonds between layers. This affects the residual strength of clays, thus making them lower than for kaolinite. The values of residual strength for Kaolinite, Illite and Montmorillonite given in Mitchell (1976) are 12°, 10.2° and 4 to 10° respectively. True friction due to interparticle sliding accounts for from half to almost all the strength of most uncemented soils, see Figures 2.3, 2.4 and 2.5 showing electron micrographs of kaolinite, illite and montmorillonite respectively, (Tovey, 1971). Figure 2.6 shows sheared kaolinite: note the particle alignments (Tovey, 1971).

Apart from structure alone, perhaps a more significant contribution to the fundamental properties of clays is their interaction with water.

2.2.2.3 Clay-Water System

Due to the bond angle formed between oxygen and hydrogen, water is a polar molecule. The negatively charged end of the dipole attracts positive ions in solution, like sodium, magnesium and, more importantly, calcium, leading to ion hydration. The positive end is in turn attracted to the negative charge sites.

As previously discussed, clay particles develop a negative charge, due to ion substitution into the lattice. Additionally, broken bonds at the edges of the silica and alumina units give rise to unsatisfied negative charges. These provide the negative charge sites which attract the water molecules, with their associated ions, to the pores between clay particles. Hence, every clay crystal is surrounded by a layer of positively charged ions and water molecules (Figure 2.7).

Two opposing factors govern the dimensions of the two layers: electrostatic charge and geometry. Therefore a compromise situation is reached. Electrostatics dictate that sufficient positive charges are required to balance the negative charge on the crystal. However, because of their associated water molecules there is insufficient space available for this to happen.

Because of the concentration of positive charges created around the crystal, a further force affects the system: an osmotic pressure exists which tries to move the cations into the surrounding water environment. Hence what has become known as the diffuse double layer is set up, analogous to an atmospheric system (Figure 2.7). As different planets have different force systems creating different atmospheres so different clays have different sized double layers.

The size of the water envelope is affected by the nature of the cations it is associated with. Its stability is hence affected by the ability of the cations in solution to exchange for those already substituted in the lattice in the case of montmorillonite, or at the edges of the particle, as with kaolinite. Ions will exchange so as to make a more stable, balanced system, the greater the positive charge the greater its ability to rectify the charge imbalance. Hence ions of high positive charge will exchange for those of lower charge. The Lyotropic series has been proposed (for example see Little, 1987) placing ions in order of replacing power:

$$Na^{+} < K^{+} < Ca^{++} < Mg^{++} < Fe^{+++}$$

Other factors affecting ion exchange include temperature, pH and electrolyte concentration.

It is apparent that the degree of lattice substitution, the nature of the substituted ion and its location within the lattice will have a major effect on its ability to react with its environment.

i) Kaolinite

Due to the relatively small degree of substitution, kaolinite exhibits little attraction to hydrated ions or water, hence it is dispersed in water and exhibits only a small capacity to exchange ions. Cation exchange mainly occurs at the edges of the crystal and capacity to accommodate water within its structure is therefore not liable to alter to a large extent. Any cation exchange which does occur happens relatively rapidly due to the accessibility of the exchange sites.

ii) Illite

As the substituted ions occur within the tetrahedral layers (see Figure 2.8) they are directly attracted to the exchangeable ions, which are in turn attracted to the next layer. The cations which exist in the inter-layer spaces actually enhance an attraction between the layers so the system has stability without exchange. Again its capacity to hold water cannot be significantly altered.

iii) Montmorillonite

The substitutions occurring within the octahedral layers, Figure 2.8, create an excellent capacity to exchange ions as the attraction to the inter-layer cations is reduced by the intervening tetrahedral layers. The charge on the exchanged ion affects this attraction and hence the inter layer bond strength. It also affects the amount of water associated with each crystal. The higher the charge, the fewer ions in solution are required electostatically, leading to fewer associated water molecules. Hence montmorillonite has the ability to alter its water holding capacity. Inter-layer spacing can vary from 9.6 Å to fully dispersed.

Although montmorillonite has a greater capacity to exchange ions, it happens at a slower rate than kaolinite due to the position of the exchangeable ions in the lattice.

In all cases, ion exchange is a diffusion process and is dependent upon the mobility of the ions.

A summary of the cation exchange capacities, (CEC in Milli equivalents per 100g) for different clays are as follows:

mineral	C.E.C.
kaolinite	3-15
Illite	10-40
Montmorillonite	100-150

The clay-water interaction is manifested in soil fabric. This refers to the arrangement of particles, particle groups and pore spaces within a soil. Broad descriptions of different types of particle associations are as follows:

1. Dispersed. No face-to-face association of clay particles.
2. Aggregated. Face-to-face association of several clay particles.
3. Flocculated. Edge-to-edge or edge-to-face association of aggregates.
4. Deflocculated. No association between aggregates.

Particle associations in sediments, residual soils, and compacted clays assume a variety of forms, but most of them are related to the above configurations. A review of the relevant work appears in Mitchell, 1981.

Therefore it can be seen how the mineralogy of the clay can affect its capacity to react with lime in ion exchange. It also affects the ability of the clay to change its response to water. As this is a key factor influencing the engineering properties of a clay, it will affect the ability of the lime to change them. It also has an effect on the bonding within the clay structure itself, which must affect its engineering properties. Plasticity is a function of clay-water interaction, hence ion exchange has a marked effect on the plasticity of clays.

2.2.2.4 Clay-Water-Lime System

The reaction between lime and clay can be divided into two distinct processes: modification and stabilisation. These shall be discussed separately.

Modification

As has been previously mentioned, calcium in the lime may exchange with ions in or on the clay lattice. Observed changes and their effects on engineering properties include the following:

- Substantial reduction and stabilisation of the adsorbed water layer.
- Flocculation of the clay particles created by the attraction realised by closer proximity of the edge to face contacts, as shown in Figure 2.9.
- Increased internal friction among the agglomerates and greater aggregate shear strength.
- A textural change from a plastic clay to a friable granular material.
- Reduced plasticity.

All these improvements are short-term changes brought about by ion exchange. From the previous discussion, it is apparent how mineralogy may affect this process. Long term observations indicate a much higher degree of improvement, indeed clays of poor ion exchange capacity also exhibit such changes. It is widely accepted that the lime and the clay react chemically to form new cementitious compounds from which the clay derives its enhanced strength. The mineralogy of the original clay heavily influences this strength development.

Stabilisation

The reaction responsible for the creation of new compounds is termed a pozzolanic reaction. The silica and alumina within the clay structure react with the water and lime to form cementitious gels which crystallise to bind the structure together. The basic reaction is of the form:

Ca^{++} + OH^- + Soluble Clay Silica -> Calcium Silicate Hydrate (CSH)

Ca^{++} + OH^- + Soluble Clay Alumina -> Calcium Aluminate Hydrate (CAH)

The exact mechanism of the reaction is poorly understood. The evidence derived by previous authors has resulted in many, often conflicting, suggested reaction mechanisms. A selection of these is summarised in Table 2.2.

The ideas presented may be summarised, from modification through to stabilisation, as:

i) Small additions of lime leading to a reduction in the double layer and flocculation.
ii) Possible formation of cementicious compounds at points of contact.
iii) Continued cation exchange until a mineralogically defined limit is reached.
iv) Saturation of pore fluid with lime producing a pH of approximately 12.
iv) Alumina and silica brought into solution by high pH, the ease of solution and abundance of compounds being dictated by the mineralogy of the clay.
vi) Formation of cementicious compounds.

This is by no means conclusive, and there is no doubt that a better understanding of these mechanisms would lead to a greater ability to predict soil suitability to treatment.

Table 2.2 Summary of Suggested Reaction Mechanisms

Author	**Observation**	**Suggested Mechanism**
Newman (1987)	Flocculation	Cation exchange leading to reduction of double layer
Diamond and Kinter (1965)	Flocculation in calcium saturated clay. Changes in plasticity, shrinkage and swelling. Increase in strength.	Formation of small amounts of cementious material at points of contact
Lees et al (1982)	Clay + 2% lime addition gives sharp increase in Ø	Modification by flocculation and particle rearrangement.
Hilt and Davidson (1960)	Limitation to the increase in P.L. with addition of more lime.	Lime-fixation - crowding of cations within clay structure
Diamond and Kinter (1965)	Limitation of lime taken out of solution, Ca in same proportion as hydroxyl ions. No overall charge change. Able to wash out No heat given off.	Physical adsorbtion of lime - a single molecular layer of $Ca(OH)_2$
"	Increased adsorbtion. Not able to fully wash out.	Chemical reaction at contact points
Bell (1988)	Non-linear relationship between lime% and strength gain after 2% addition for montmorillonite; 2-4% for kaolinite	The existence of an optimum addition of lime.
Lees et al (1982)	Decreases in Ø for Kaolinite with increases for lime addition over 2%.	Optimum lime addition/ particle rearrangement dominant stabilising factor
"	No decrease in strength with larger additions of lime to montmorillonite.	Pozzolanic reactions dominant stabilising factor.
Bell (1988)	Decrease in max. dry density	Formation of cementicious compounds which form interlocking compounds.

Author	**Observation**	**Suggested Mechanism**
Brandl (1981) Mateos (1964)	Swelling potential/ pressures reduced	Formation of cementicious compounds which form interlocking compounds.
Eades and Grim (1966)	Formation of calcium aluminate hydrates/ silicate hydrates at high pH (high lime content).	High pH induces solubility of silicates and aluminates in the clay, hence reaction able to occur.
Diamond and Kinter (1965)	"	Lime adsorption leads to reaction at clay surfaces, giving rise to precipitate reaction.
Harty and Thomson (1973)	Greater improvements with highly weathered soils.	Weathering increases clay susceptibility to dissolution of silica and alumina.
Quigley and DiNaro (1978)	Greater improvements using high lime contents on weathered soil than non-weathered.	Weathering increases smectite content which reacts via a different mechanism.
Moore and Jones (1971)	Reactivity greatly reduced with high iron content.	Iron in form of positively charged aggregations reducing ion-exchange capacity.
Mateos (1964)	Lesser degree of improvement using illite and chlorite than montmorillonite.	Cation exchange capacity reduced.
Mateos (1964)	Higher increases in P.L. with increasing clay content.	Clay is the reactive soil constituent.
Diamond and Kinter (1965)	Variation in the effect on L.L clay to clay.	Type of exchangeable ion has an effect.
Bell (1988)	Kaolinite + 2% lime L.L rise; Kaolinite + <2% lime L.L. falls.	Particle surfaces modified by the action of hydroxyl ions.

Author	**Observation**	**Suggested Mechanism**
Bell (1988)	Higher compaction densities obtained in kaolinite than montmorillonite.	Expandable minerals have greater affinity for water due to shape and higher specific surface.
Lees et al (1982)	No clear optimum of moisture content-dry density compaction curve - most noticeable in montmorillonite.	Initial formation of cementicious compounds affects compactibility - montmorillonite most reactive.
Bell (1988)	Dramatic immediate increase in U.C.S. with small lime addition in montmorillonite.	Expansive clays react more quickly due to greater cation exchange capacity.
Lees et al (1982)	Cohesion and U.C.S. correlation.	Both affected by the same part of the mechanism.

Notes

1. ϕ is the angle of internal friction of the soil.
2. UCS is the unconfined compressive strength.
3. PL is the Plastic Limit of the soil.
4. LL is the Liquid Limit of the soil.

2.2.3 Engineering Implications of the Reaction

There is a vast amount of literature available on the effects of lime on a wide variety of clays from all over the world. This brief review will concentrate on British sources as this research concentrates on UK applications. The following will be considered:

A comparison of lime and cement stabilisation

The specification of soil suitability to the treatment

The changes in soil properties incurred.

In 1962 Dumbleton made an assessment of the stabilising effects of lime on British Clays. The strengths of different types of British Clays were assessed after mixing with various quantities of lime. Comparisons were made with those achieved by Portland cement. Figure 2.10 shows the relationship between additive content and unconfined compressive strength for lime and Portland cement added to London clay, for example. Generally, it was found that additions of lime up to 5% by dry weight were most effective in increasing strength. Further details of the experiments are not included here as this illustration is of the comparison between cement and lime stabilisation and not intended as an assessment of the techniques individually. It was also noted that:

"Cement usually produced greater increases in strength than lime, except perhaps with heavier clay soils" (Dumbleton 1962).

Additional work on the stabilisation of Brisith Clays in reported by many authors including Little (1987), Livesey (1988), Bell (1988), Rogers and Bruce (1990), with many publications through BACMI, the British Aggregate Construction Materials Industries.

Having been established as a viable technique in the UK, in spite of worries regarding the lower ambient temperatures here than in other countries where lime has been successfully used, the last thirty years have seen lime stabilisation become more widely used and accepted in the UK. Continuous research and practical experience have developed design criteria, testing procedures and specifications for its usage. The British Lime Association (BLA), in its Lime Stabilisation Manual (1990), describes suitable conditions conducive to stabilisation:

i) Clay properties

Clay content (or clay fraction, CV) > 10%

Plasticity index (PI) > 10%, preferably 20%

Activity $\frac{\text{Plasticity Index}}{\%\ \text{clay}} > 0.75$

Total Sulphate < 1%

ii) Other specifications

Lime addition < 2.5% by dry mass (for subgrade stabilisation)

Operations are not to be carried out below a temperature of 7°C or between September and March.

In 1993 Sherwood published a state-of-the-art review of cement and lime stabilisation, in which he discusses the physico-chemical effects of lime on clay. He describes the immediate and striking effect of the lime on plasticity and illustrates it by showing the changes wrought on London clay (Figure 2.11). The significance of this is highlighted by consideration of the soil at a moisture content of 35% and a Plastic Limit of 25%. Before lime addition the soil would have been wet of Plastic Limit, sticky and difficult to compact. With a 2% addition of lime the Plastic Limit is increased to 40%. Now the clay is 5% below Plastic limit and readily compactible. It is pointed out that chemically equivalent amounts of slaked lime and quicklime produce the same stabilising action, but that quicklime has the added benefit of drying the soil as it slakes. The effect of the pozzolanic reaction is one of strength gain with time, as illustrated previously with the example from Dumbleton (1962). Sherwood choses to illustrate the gain in strength by CBR values, (Figures 2.12).

Clay mineralogy has a profound effect upon the results of the lime clay reaction, as measured by the engineering properies of the resulting materials. Table 2.3 summarises the mineralogy of some of the more common British clays and gives examples of the sorts of changes brought about to their engineering properties. In order to keep this concise it has not been possible to include experimental detail. Therefore the appropriate source reference has been given in each case.

The review also covers the topical subject of factors mitigating against lime stabilisation, such as the presence of sulphates. These are naturally ocurring chemicals, most commonly found in most British clay formations as Gypsum or calcium sulphate. Sulphates are thought not to affect the immediate improvement in the plasticity of soils but, in the presence of excess water, they may cause problems by reacting with the hydrated cementitious materials produced by the pozzolanic reaction. A serious loss of strength occurs as a result of the expansive nature of such a reaction. Water is thought to be a key factor controlling the degree to which expansion can take place. Without a supply of excess water, sulphate cannot dissolve and therefore the reaction does nt occur. Recent problems encountered with the M40 motorway have highlighted the difficulties caused by this effect, but it is thought that with an effective site investigation and good control of water supply on site these problems can be avoided in future.

Table 2.3 British Clays: Properties, Mineralogy and Effects of Lime

Formation	LL (%)	PI (%)	**Clay Fraction < 2μm (%)**	**Average undrained shear strength (kN/m^2)**	**Effective cohesion c' (kN/m^2)**	**Effective angle of friction, φ' (degrees)**	**Residual shear strength φ'r (degrees)**	**Mineralogy**	**Effects of lime: Author, test, effects**
Palaeogene									
London clay	66-100	40-65	40-72	100-173	12-18	17-23	10.5-22	Mainly illite-mica with kaolinite and traces of chlorites, smectites and vermiculites. $CaCO_3$ barytes and gypsum also found.	Dumbleton, 1962 mix test 10% lime: increase in UCS. Also 5% reduced plasticity.
Cretaceous									
Gault clay	70-92	27-80	38-62	60	-	-	12-19	Usually illite-mica, poorly crystallysed kaolinite and expanding chlorite/vermiculite. Mixed layer structure.	Dumbleton, 1962 mix test 10% lime: increase in UCS. Also 5% reduced plasticity.
Weald clay	42-82	28-32	20-74	-	-	-	11-20	Similar to above with impurities of calcite, siderite, pyrite, gypsum, ostracod and ferrous ion	Dumbleton, 1962 mix test, 10% lime No increase in UCS in 12 weeks; affects to LL & PL.
Jurassic									
Kimmeridge clay	-	24-59	57	-	14-67	14-23	10-18	Dominantly illitic	Dumbleton, 1962 mix test; 10% lime increase in UCS
Oxford clay	-	28-50	30-70	-	0.20	21.5-28	13-17	Dominantly illitic	Dumbleton, 1962 mix test; 10% lime increase in UCS Cobbe, 1988 mix test; 2% $Ca(OH)_2$ PL and LL & PI raised 1 hour after mixing.

Table 2.3 continued...

Formation	LL (%)	PI (%)	Clay Fraction < 2µm (%)	Average undrained shear strength (kN/m^2)	Effective cohesion c' (kN/m^2)	Effective angle of friction,ϕ' (degrees)	Residual shear strength ϕ'_r (degrees)	Mineralogy	Effects of lime: Author, test, effects
Jurassic									
Upper Lias	56-68	20-39	55-65	30-150	10-17	18-25	9-13.5	Kaolinite-illite with traces of chlorite. Rare mixed layers.	Cobbe, 1988 Mix test, 2% $Ca(OH)_2$ Pl and LL raised 1 hour after mixing.
Lower Lias	56-62	32-37	50-56	-	5	27	13-16	Dominantly illitic with impurities of marcasite, pyrite, glauconite, fossils; high iron content.	Dumbleton, 1962 Mix test, 10% lime UCS raised.
Triassic									
Keuper Marl	25-60	10-35	10-50	100-150	2-80	25-42	18 - 30	>40% illite-mica 4-34% dolomite gypsum, high alkali.	Dumbleton, 1962 Mix test, 10% lime UCS raised.

Notes

1. UCS is the Unconfined Compressive strength

2.2.4 Applications

Lime stabilisation is unique due to the fact that the lime actually reacts with the clay itself, rather than the clay and the stabiliser forming a two material system. The rapid nature of the modification process also establishes its individuality. For this reason, lime stabilisation has developed its own specific applications and restrictions. For instance, lime may be applied to very heavy, wet clay soils that are unsuitable for other treatments; it may not, however, be used on granular material or soil with insufficient clay content when the treatment relies on the clay-lime reaction for its benefits. Lime stabilisation is also widely used to aid construction, rapidly rendering heavy wet clays suitable for the passage of construction traffic. It is apparent, therefore, that lime stabilisation is very versatile, with a wide range of applications.

Although used extensively overseas, particularly in the construction of road subbases, it has only relatively recently been established in the UK as an alternative to traditional subbase and capping construction. Nevertheless, the usage of lime has, at least until recently, largely been confined to the road construction industry, where it is used as a surface mixing treatment. Its usage as a deep stabilising technique has, however, occurred overseas. These applications shall be discussed in following sections.

2.3 LIME USED AS A DEEP STABILISER

The use of lime as deep stabiliser has been pioneered in several overseas countries, notably Sweden, Japan and the USA, and can be divided into three main groups: lime columns, lime slurry pressure injection and lime piles. Each shall be discussed in the following sections.

2.3.1 Lime Columns

The similarities between this technique and lime piles has resulted in considerable confusion in the literature. The term 'lime columns' refers to the creation of deep vertical columns of lime stabilised material.

The general principle of the technique is to create, *in situ,* columns of intimately mixed lime and clay. The clay-lime reaction which takes place produces columns of material of greater strength and improved (reduced) permeability than that surrounding them. Thus, despite the fact that the strength improvement is derived from the material that occurs *in situ,* a two material system is established. Its applications, therefore, are similar to those of the two material system of clay and cement, and indeed to those of stone columns that are used to strengthen soft clays.

Broms and Boman have been the principal advocates of this method, publishing many papers on column construction, design and properties, and giving many examples of their successful usage. Broms and Boman (1975) gives an introduction to the use and construction of lime columns, a complete study of design methods and applications being published by the same authors in 1983. The columns are constructed using a giant 'egg-beater' tool, shown in Figure 2.13.

Great emphasis was placed on the physical stabilising processes and the properties of clays required to promote effective stabilisation. These can be summarised as:

i) Clay Properties

Clay content > 20%

Silt plus clay content > 35%

Plasticity index > 10%

ii) Other specifications

Lime addition = 10-20% by dry weight

High temperature and pH accelerate reaction

Maximum lime particle size < 0.2mm

These may be compared to those in the BLA (1990) Lime Manual, see section 2.2.3.

With these rules observed, increases in shear strength of as much as 5 to 20 times have been recorded. Permeability within the columns was also raised by 100 to 1000 times immediately after mixing, although it is thought that the permeability will reduce in time as the stabilisation reactions progress and gel crystallisation occurs.

Applications were based on increased bearing capacities, shear strength and permeabilities. These include, essentially, foundations, with design methods being established by consideration of column strength and column-soil interaction.

Broms (1985) presents information on the use of lime columns in slope stabilisation. It was noted that high induced pore water pressures produced by traditional piling techniques were reduced by the lime's affinity for water. Design of pile installations was developed by considering the average shear strength (C_{av}) along a potential failure plane.

$$c_{av} = c_u\ (1 - a) + s_{col}\ a$$

where c_u is the initial shear strength of the soil (c_u rather than s_u is used as the author states that this value may be found by shear vane test), s_{col} is the average shear strength of the stabilised clay within the columns and a is the relative column area, the ratio of the total area of the columns and the area of the stabilised soil. An example of this application is given in Figure 2.14.

Similar work has been carried out by Holm, including an assessment with Holm et al (1983 b) where the settlement characteristics of pile groups are assessed and favourably compared with those of sand piles. Another comparison with sand piles was made by Hansbo (1987) who attempted to produce equations to aid the design of drains for soil improving techniques. The Japanese have developed the 'deep mixing method' for use with lime instead of cement (Terasai et al, 1979). Applications include slope stability, but no details are given.

The application of lime columns to British soils has been investigated by Corbet (1988), this being the only paper on the subject that considers the application for British use. Lime columns were assessed as a means of stabilising soft ground under embankments, the other forms of stabilisation being considered including drainage, vibro replacement, stone columns, geofabrics and conventional piling. Lime columns were seen as an alternative to stone columns. The properties of the clay soil to be treated are given in Table 2.4.

Table 2.4 Soil properties (after Corbet, 1988)

Property	Average Values
Natural Moisture Content	55%
Plastic Limit	22%
Liquid Limit	53%
Organic Content	3.3%
Sulphate Content	0.36%
pH	7.9
Clay Content (0.002mm)	25%
Silt Content (0.002mm - 0.06mm)	74%
Sand Content	1%
Undrained Shear Strength	10 kPa at 2m (west of railway)

Four soil samples were mixed with 7, 9, 11 and 15% lime by dry weight in a pan mixer. Samples were compacted into plastic buckets using "a light tamping", were covered in plastic and loaded to provide a surface stress of 20 kPa. Unconsolidated undrained triaxial tests were carried out on samples taken from the buckets, the results being shown in Figure 2.15. Permeability tests were conducted in a Rowe cell and some typical results are presented in Table 2.5. Samples were taken after 30 days curing, although the author states 20 to 30 days were taken to saturate each sample. Although the author notes that some samples produced no change in permeability, and indeed a few were shown to decrease. The overall trend was for permeability to increase, however.

Table 2.5 Results of permeability testing (after Corbet, 1988)

Sample Ref Permeability	Time after Mixing (Days)	Effective Pressure kPa	k (m/sec)
Untreated	N/A	50.0	5.0E-08
		100.0	5.4E-11
		200.0	2.7E-11
Sample A 7% Lime	30	50.0	5.4E-09
		100.0	1.1E-10
		200.0	8.8E-11
Sample B 9% Lime	30	50.0	3.8E-07
		100.0	1.3E-09
		200.0	2.9E-11
Sample C 11% Lime	30	50.0	1.4E-11
		100.0	3.9E-11
		200.0	1.4E-11
Sample D 15% Lime	30	50.0	7.5E-07
		100.0	2.5E-07
		200.0	5.7E-09

Columns were thought unsuitable for the application required due to the inconsistency of strength gain and the lower than expected permeability. These results may be attributable, however, to sample preparation, sample storage and unsuitability of the tests. The inconsistency of bulk density found in treated samples indicates considerable inconsistency of compaction and loading acheived by the methods described by the author. This may have lead to the inconsistency of results and poor strength gain due to lack of compaction and confinement. A more serious criticism concerns the confinement of the samples, which were sealed in a bucket with no facility for water ingress whereas in the field the material will be, by design, below the water table in a soft saturated silty clay which will be loaded by embankment construction. Thus it is unlikely that the permeability tests will indicate an accurate measure of permeability *in situ* due to the reaction of the permeating water with any excess lime and as a result of the artificially dry storage conditions, which will undoubtedly alter the lime-clay properties.

It is a pity that such a potentially interesting study was conducted in such a manner that no conclusive results have been found. The use of lime columns for UK applications should be investigated thoroughly and rigorously in future, well designed experiments and these existing data should not be allowed to prejudice judgement on UK applicability of the technique.

The literature on lime columns has been useful to give some idea of the sorts of soil properties considered necessary for their successful application, as well as an insight into the development of design criteria for their various applications. However, it is important to remember that columns are still a mix technique, the lime being designed to react only within the column itself and not migrate outwards. Indeed, Broms and Boman (1975) state that it was:

"important that the lime is thoroughly mixed with the soil since the diffusion rate of calcium ions in clay is very slow".

Migration of lime from columns was investigated by Somayazulu et al (1984) in a series of laboratory trails. Greater mixing efficiency was found to inhibit migration. Therefore, although mixing improves the properties of the soil within the column, it reduces the amount of lime migration. This may be because the permeability is reduced by the mixing action and by the cementing reaction, or that the lime is 'fixed' within the column by the rapid succession of the reaction sequence.

The migration from lime columns mixed as they would be *in situ* and using different lime contents is therefore something which merits significant further study. Some indication of migration can, perhaps be gained from the literature on lime piles and from the studies reported later in this thesis, but the data should be treated as strictly upperbound values that could occur under certain circumstances.

2.3.2 Lime Slurry Pressure Injection

This technique may provide information about the effects of lime that has not been intimately mixed with the clay. However, the technique creates a greater area of contact between lime and clay than with a pile by forcing the slurry into the natural weaknesses in the soil. It is thus difficult to compare effects by percentages of clay treated due to not knowing the precise area covered by the lime.

Lime slurry pressure injection, as the name suggests, involves the introduction of a lime slurry into the ground under pressure (Figure 2.16). First developed in the 1960's, the technique forces the slurry into the pores and fissures in the clay, causing a kind of 'mixing' effect by virtue or permeation of the slurry, although without (significant) disturbance to the inherent clay structure. The lime forms a kind of matrix, enclosing areas of untreated clay. This is thought to prevent moisture movement between these areas, thereby creating wet-dry cycle resistance in swelling clays. Applications include pavement remediation, building foundation construction and remediation, and embankment remediation, with typical injection depths of 1-2 m, 2.5-3.5 m and 3.5-15 m respectively. Injection typically takes place on a grid of 2.5 m centres using pressures of between 350 and 1400 kPa.

Blacklock and Wright (1986) review several case studies using slurry injection and describe the development of field and laboratory tests to aid with the assessment of site suitability.

In all cases, a slurry has been used, with quicklime slaked on site. This was said to improve dispersion of the hydrate particles giving fine particle size and slower settling time, thus producing a higher surface area and hence reactivity.

Site evaluation has been achieved through specific test development. A novel method of sample preparation was developed, employing traditional soil testing methods: compression and shear strength tests for low strength soils and swell tests for expansive soils. Samples are dosed with 1% lime by dry mass, as this is thought to reflect field treatment. The sample preparation described was designed to simulate the ability of the treatment to repair and stop crack growth. Unfortunately the paper fails to explain adequately how the testing programme relates to field design or how the tests simulate site conditions.

The application of the technique in India has been reported by Bhattacharya and Bhattacharya (1989) where it has been used to treat soft and expansive soils under railway lines. The same principles as previously discussed were employed, Figure 2.17 showing the injection pattern used relative to the track and sleepers. 'Before and after' soil properties are tabulated, showing variable degrees of change, Table 2.6a and 2.6b. However, this sort of testing seems inappropriate to assess a technique that only treats a relatively small, and uncontrollable, portion of the soil. Observation of the changes in soil properties will depend on testing a treated sample and extrapolation to the soil mass to deduce overall changes will be very imprecise. Testing for the reduction in water movement within a soil mass would similarly require a more sophisticated test programme. Nevertheless, on the treated sections of track it was observed that shrinkage cracking was reduced, indicating some degree of success.

The application of interest for this research would be slope stabilisation. However, a review of the technique by Rogers and Bruce (1990) noted that the potential high pore pressures induced should be considered carefully when using the technique to this end.

Table 2.6 Properties of soil before and after treatment with LSPI (after Bhattacharya and Bhattacharya, 1989)

Table 2.6a Before treatment

Stretch	Km/T.P.No.	Depth	N M C	Atterberg's Limits			Shear Strength		Grain Size		
		(m)	(%)	LL (%)	PL (%)	SL (%)	C (Kg/Cm^2)	Ø	Sand (%)	Silt (%)	Clay (%)
Barasat	8/2-3	1.5-1.95	30	61	23	11.5	0.50	4°	4	52	44
to	to	1.5-1.95	66.5	101	50	16.3	0.30	0	-	-	-
Hasnabad	9/5-6	1.5-1.95	31.8	61	22	12.0	0.40	6°	3	55	42
		1.5-1.95	128.7	164	87	17.5	0.26	0	3	59	38
Belmuri	46/21-23	1.5-2.0	34.6	71	28	18.2	0.45	8°	5	54	41
	to	2.5-2.95	28.5	70	25	17.5	0.58	4°	3	54	43
	46/22-24	1.5-1.8	37.6	74	31	21.0	0.39	0	4	50	46
		3.0-3.5	35.3	73	28	19.6	0.54	2°	2	50	48
Jougram	64/11-13 to	1 -1.40	28.9	51	23	15.5	0.45	5°	4	60	36
	64/13-15	1.5-2.0	29.2	55	26	16.7	0.51	0	3	57	40
Sibani	50/9-11	1.0-1.40	31.3	50	24	18.5	0.48	4°	5	60	35
Chandi		1.6-2.0	28.9	55	25	19.2	0.50	6°	3	58	39
Dun Dum	9/0-1	2.2-2.5	33.6	62	28	21.2	0.49	3°	3	59	38
to	to	2.2-2.5	41.4	78	32	20.5	0.30	0	4	48	48
Belgharia	9/15-17	2.2-2.8	37.0	63	26	23.4	0.40	5°	3	56	41
		2.2-2.5	35.2	60	26	22.6	0.51	4°	2	62	36
		1.63-1.93	36.6	75	31	19.5	0.48	2°	3	50	47
		1.61-1.91	48.5	82	35	20.4	0.30	0	2	47	51

Table 2.6b After treatment

Stretch	Km/T.P.No.	Depth	N M C	Atterberg's Limits			Shear Strength		Grain Size		
		(m)	(%)	LL (%)	PL (%)	SL (%)	C (Kg/Cm^2)	Ø	Sand (%)	Silt (%)	Clay (%)
Barasat	8/2-3	1.5-1.95	28.6	48	21	14.3	0.52	6°	5	57	38
to	to	1.5-1.95	52.8	78	42	18.0	0.38	2°	-	-	-
Hasnabad	9/5-6	1.5-1.95	30.5	50	20	15.6	0.52	0	3	58	39
		1.5-1.95	120.2	148	75	18.0	0.30	2°	4	60	36
Belmuri	46/21-23	1.5-2.0	31.5	53	25	20.0	0.54	2°	5	57	38
	to	2.5-2.95	26.3	59	23	19.5	0.41	5°	4	54	42
	46/22-24	1.5-1.8	34.8	61	28	21.0	0.46	2°	6	54	40
		3.0-3.5	30.2	58	27	20.5	0.50	5°	4	53	43
Jougram	64/11-13 to	1 -1.40	29.3	41	24	17.0	0.50	2°	5	60	35
	64/13-15	1.5-2.0	26.8	39	23	17.0	0.55	4°	4	55	41
Sibani	50/9-11	1.0-1.40	30.5	44	25	20.0	0.52	2°	4	63	33
Chandi		1.6-2.0	29.4	46	25	20.0	0.50	0	5	54	41
Dum Dum	9/0-1	2.2-2.5	31.5	54	26	22.0	0.54	5°	3	62	35
to	to	2.2-2.5	39.6	68	31	21.0	0.36	2°	5	54	41
Belgharia	9/15-17	2.2-2.8	40.0	53	24	20.8	0.47	0	5	52	43
		2.2-2.5	38.2	54	26	23.0	0.50	2°	3	59	38
		1.63-1.93	36.5	62	30	23.5	0.49	4°	4	55	41
		1.61-1.91	51.2	72	32	24.0	0.35	2°	4	54	42

2.3.3 Lime Piles

Lime piles are, very basically, holes in the ground filled with lime. Ingles and Metcalf (1972) show one method of lime pile construction, illustrated in Figure 2.18, in which a hollow tube is pushed into the soil to the required depth of pile and quicklime is forced into the tube under pressure as it is withdrawn. The pressure forces open the end of the tube allowing the lime to fill the cavity below. After each metre is filled, the end of the tube is closed and used to compact the lime forming the pile. The alternative method of construction is to simply auger holes to the required depth and to subsequently, or via a central stem, add quicklime and compact in layers. In some cases hydrated lime, or more commonly lime slurry, is added to the augered holes, depending on the application and perceived stabilisation mechanism.

The use of lime piles has been documented in many countries and is reviewed hereafter. Care has been taken to extract specific key points from each paper to try to acheive a degree of consistency for comparative purposes. These points are the properties, including mineralogy, of the treated clay, lime type, constuction method, and size and spacing of the piles. Where these have been omitted from the discussion the information was unavailable.

2.4 LIME PILES

Two distinct themes emerge from the literature concerning the mechanism of stabilisation and authors largely discuss one independently of the other. The first of these is the idea of pile expansion and clay dehydration. In principle, as the quicklime in the piles reacts with the water in the ground, excess water is drawn into the pile and is consumed by the reaction. As a consequence of the expansive nature of this reaction, the piles expand and cause lateral consolidation of the surrounding clay. In general, the authors who propose this mechanism are using the lime piles to improve the bearing capacity and settlement characteristics of soft ground for foundations.

The second theme concerns the migration of calcium ions from the pile into the surrounding clay and subsequent stabilisation of the clay by lime-clay reaction. For this to occur the clay must be brought into a highly alkaline state, a minimum pH of 12.4 being required for the reactions to take place. In general the authors who propose this mechanism are relying upon the lime piles to stabilise failing slopes in stiff clays or weathered shales and mudstones. These two distinct applications will be dealt with separately herein, although the experimental and field observations in some cases are relevant to both sections.

2.4.1 Lime Piles for Soft Soil Improvement

Several authors describe the use of lime piles for the improvement of soft soils, although the most notable exponents are the Japanese. Kitsugi and Azakami (1982) describe the use of their 'chemico-pile method'. This involves the formation of granulated quicklime piles constructed by driving a closed-ended casing into the ground and then forcing quicklime (using compressed air) into the cavity that is formed as the casing is withdrawn. Additives of calcium silicate and/or calcium alluminate are introduced to the quicklime, presumably to improve strength. The typical applications of the technique are quoted as improvement of settlement and bearing capacity characteristics, prevention of heave in excavations, and embankment and cutting stabilisation. The authors suggest the following as the essential stabilising processes:

1. Rapid dehydration via chemical reaction.
2. Volume expansion of the lime upon slaking by approximately 100%.
3. Improved (reduced) plasticity of the soil by clay-lime reaction.
4. Pozzolanic reactions.

An indication is given of how the quicklime content in the ground diminishes with distance from the pile, which appears to derive from an attempt to study migration through the ground. However, it would seem probable that lime being forced into a void in very soft ground under pressure would tend to displace the surrounding soil by a significant amount and yet no account is apparently taken of this fact. The authors also state that:

> "It could be considered that the bearing capacity of the soil improved by this method is mainly in the strength of the in situ lime columns."

It should be noted that throughout the literature, the terms 'lime column' and 'lime pile' have been used interchangeably and that care must be taken to discover the method of construction referred to. An average unconfined compressive strength of 431 kN/m^2 is quoted and said to be related to the confining pressure provided by the surrounding soil, although no attempt was apparently made to measure the strength under different degrees of confinement. Clearly the design of lime piles as bearing elements in isolation, rather than as a composite pile-soil system, would need to take account of the degree of lateral support from the soil. This lateral support will increase considerably with depth and thus the degree of vertical confinement of the clay close to the surface will be important in how such piles should be designed. In practice it will be the pile-soil system that resists the applied stress and an equilibrium condition will be reached.

The main stabilising processes pursued in this paper are those of dehydration and lateral consolidation caused by expansion. The following equation is derived relating water content reduction to the original water content of the ground and this may be used to assist in the estimation of pile diameter and spacing.

$$\Delta w = \frac{1 + \frac{w_0}{100}}{\gamma_t} a_s \left[h \cdot \gamma_c \left(\frac{\eta'}{100} \right) \left(1 + \frac{\varepsilon_\upsilon}{100} \right) \left(\frac{S'_r}{100} \right) \gamma_w \right] \cdot 100\%$$

where Δw = reduction in soil water content (%)

w_0 = original water content (%)

γ_t = wet density of soil *in situ* (t/m^3)

a_s = area ratio of the Chemico-pile

$a_s = \pi d^2/4S^2$

d = diameter of pile (cm)

S = pitch of piles (square arrangement) (cm)

h = equivalent of water absorption of lime column

γ_c = apparent saturated unit weight of lime column (t/m^3)

ε_υ = coefficient of volumetric expansion in slaking (%)

S'_r = degree of saturation of slaked lime (%)

γ_w = unit weight of water (t/m^3)

η' = porosity of slaked lime (%)

The equation appears to assume that water will tend to flow into the pile purely to fill the void space within it. This in turn relies on the formation method of introducing new material to the ground rather than filling an augered hole. Similar results may be achieved simply by calculating the amount of water required to produce the same water content in the pile as that in the surrounding soil. Results from the field are compared with those calculated, but appear to be only broadly comparable. It is probable that ground conditions other than water content alone affect the movement of water in the ground, and hence the achievable water content of the pile.

For ground with water contents of the orders quoted (50 to 150%) the equation calculates reasonably large reductions. However, applied to ground more typical of this country, of a water content of, say, 30%, with a pile spacing of 1m and a diameter of 0.1 m , the reduction is only 1%. Results using the equation show the necessity for very large piles at relatively small spacings. This is, in fact, what the paper advocates, but piles constructed in this manner would be relatively more expensive in the UK due to the higher cost of lime production. Nevertheless the application of this technique to soft alluvial clay soils for ground improvement in the UK could prove to be an economical solution to the considerable problems posed by such soils and the need for rapid ground improvement, which this technique would provide, would influence the decision. Indeed the requirement that the soft soil should have a significant clay content is not stated and, since the technique has been applied to loess in Russia, as described later, then all soft alluvial soils could theoretically be treated in this way.

The expansion of the piles is said to cause lateral consolidation and so 'improve cohesion' in the soil. This improvement is calculated using an estimation of the change in voids ratio brought about by the expansion, determined by multiplying the new water content by the specific gravity. This in turn is used to calculate the increase in stress, using the graph of void ratio against effective normal stress, which is multiplied by a 'strengthening factor' to reach the change in 'cohesion'. Reduction in vertical settlement is calculated by considering that part of the settlement capacity of the soil is taken up with lateral consolidation. Equations quantifying these ideas allow the determination of the length of the piles required.

The source of the lateral movements seems to be entirely attributed to the expansion of the lime. However, when the change in volume of the piles is compared with the volume of the water taken out of the soil, there is no significant net volume change (in fact there is a slight volume reduction). Nevertheless, piles of significantly increased volume in comparison with the casings used have been observed in the field and this is quoted as proof of the argument. While some expansion of the pile diameter is inherent in the assumption of no discernible net volume change, since the water from the soil becomes bound in to the pile, the values of diameter quoted exceed this expansion . This additional expansion can, however, be attributed to the fact that the driven casing will cause lateral movements of the soil and the lime applied under pressure, and subsequently compacted, will tend to expand further the cavity in ground as soft as the type mentioned. The strength increases observed in the surrounding ground can thus be attributed to water content reduction, which can be estimated using the equations of Skempton and Northey (1953).

Kitsugi and Azakami (1982) present a case study using lime piles to reduce settlements beneath a 1.5m to 3.0m high embankment. The results are presented in Table 2.7, in which it is apparent that the changes in water content are less significant than those discussed earlier in the paper. This fact, together with the observation that the water content in the piles is significantly lower than that in the surrounding soil, would indicate an incomplete reaction and incomplete water content transfer to the pile. Factors affecting this movement thus need further consideration. The relationship between change in voids ratio and water content does not appear to hold in this case and therefore the change in undrained shear strength cannot have been accurately calculated. Changes in the undrained shear strength (termed Δc) increased with depth. This could indicate that the effects recorded were due to vertical consolidation caused by reduced pore water pressures increasing the mean normal effective stress, and hence the vertical component of stress. The information presented is generally insufficient to make a full assessment of the validity of the equations or to make other suggestions about the causes of the changes occurring.

Table 2.7 **Property change after lime pile treatment (after Kitsugi and Azakami, 1982)**

Before improvement

Depth (m)	ω (%)	γ_t (g/cm^3)	G_s (g/cm^3)	e	c (t/m^2)	P_c (kg/cm^2)
5.5/6.15	122.2	1.34	2.62	3.34	1.20	0.49
7.0/7.35	88.3	1.47	2.64	2.39	1.50	0.52
10./10.8	98.3	1.42	2.68	2.75	1.90	0.59
14./14.3	85.6	1.49	2.68	2.28	2.45	0.55

After improvement

Depth (m)	ω (%)			e	$c = 1/2q_u$ (t/m^2)	P_c (kg/cm^2)
5.5/6.15	(163.0)			3.39	7.87	1.20
7.0/7.85	86.4			2.31	5.21	1.35
10./10.8	79.1			2.09	5.62	1.60
14./14.3	76.1			2.06	5.98	1.70

After improvement

Depth (m)	$\Delta\omega$ (%)			Δ_e	Δ_c (t/m^2)	ΔP_c (kg/cm^2)
5.5/6.15	-			-	(6.67)	0.71
7.0/7.35	1.9			0.08	3.71	0.83
10./10.8	19.2			0.66	3.72	1.01
14./14.3	9.1			0.22	4.52	1.15

Lime column Chemico-pile

No	d (m)	ω (%)	γ (g/cm^2)	q_u (kg/cm^2)
1	1-1.5	43.6	1.636	4.85
2	1-1.5	59.8	1.607	1.97
3	1-1.5	40.9	1.718	5.40
4	1-1.5	50.4	1.660	2.35
mean	1-1.5	48.7	1.642	3.78

Where ω : moisture content of soil (%)
γ_t : moist unit weight of soil(g/cm^3)
G_s : specific gravity of soil (g/cm^3)
e : void ratio of soil
c : conesion of soil (t/m^2)
P_c : yield stress in consolidation
q_u : unconfined compressive strength (kg/cm^2)

Several other authors describing the use of lime piles to improve bearing capacity also quote the mode of operation to be that of expansion and dehydration. Further work in Japan investigated the use of lime piles in the reduction of ground vibrations caused by traffic on soft soil. Taniguchi and Okada (1981) describe the modest success of the technique. Although the study of the effect on vibrations shall not be included here, a summary of the improvement in soil properties produced is shown in Table 2.8. 400 mm diameter piles were placed to a depth of 12 m at 0.9 m spacings on a square grid. A total of 782 piles (34 piles in 23 rows) treated a 13 to 15 m deep stratum of soft soil. It is not clear from the text where the samples for the tests were recovered from, which is unfortunate since some of the changes in soil properties quoted are considerable. It should be noted, however, that a relatively large proportion, by area, of site is subsequently taken up by the lime piles.

Table 2.8 Change of soil properties due to soft ground improvement by the lime pile method (after Taniguchi and Okada, 1981)

depth (m)	water content (%)			wet density (g/cm^3)			void ratio			dry density (g/cm^3)		
	B	A	A/B(%)	B	A	A/B(%)	B	A	A/B(%)	B	A	A/B(%)
3.1~ 3.9	94.0	64.3	68.4	1.43	1.56	108.7	2.27	1.55	68.5	0.739	0.949	128.4
5.0~ 5.8	136.0	78.8	57.9	1.34	1.52	114.0	3.39	1.97	58.0	0.566	0.851	150.4
6.0~ 6.8	104.1	74.5	71.6	1.36	1.50	110.2	2.72	1.90	70.0	0.665	0.857	128.9
8.5~ 9.3	104.4	77.5	74.2	1.41	1.50	105.8	2.66	2.01	75.4	0.691	0.842	121.9
9.5~10.3	80.0	56.2	70.3	1.46	1.63	111.8	2.16	1.45	67.2	0.812	1.046	128.8
12.2~12.5	88.6	48.1	54.3	1.48	1.71	116.1	2.28	1.25	55.0	0.782	1.157	148.0

depth (m)	liquid limit (%)			plastic limit (%)			plasticity index			unconfined compressive strength (kPa)		
	B	A	A/B(%)	B	A	A/B(%)	B	A	A/B(%)	B	A	A/B(%)
3.1~ 3.9	109.2	86.7	79.4	77.7	41.9	53.9	31.5	44.8	142.2	40.8	65.8	161.3
5.0~ 5.8	146.2	94.5	64.6	72.0	42.2	58.6	74.3	52.4	70.5	41.1	56.4	137.5
6.0~ 6.8	107.3	104.3	97.2	54.3	49.4	91.0	53.0	54.9	103.5	40.6	84.3	208.0
8.5~ 9.3	103.6	94.3	91.0	48.3	48.4	100.3	55.3	45.9	82.9	33.3	94.8	284.4
9.5~10.3	86.2	67.2	77.9	39.1	40.7	104.1	47.1	26.5	56.2	34.0	106.4	313.0
12.2~12.5	87.9	76.6	87.2	34.2	37.3	108.9	53.7	39.4	73.3	37.0	126.5	341.5

depth (m)	angle of shearing resistance (degree)			cohesion (kPa)			coefficient of consolidation (cm^2/min)			coefficient of permeability $\times 10^{-7}$ (cm/s)		
	B	A	A/B(%)	B	A	A/B(%)	B	A	A/B(%)	B	A	A/B(%)
3.1~ 3.9	2.60	2.10	80.0	17.6	29.4	166.7	—	—	—	—	—	—
5.0~ 5.8	—	—	—	—	—	—	0.034	0.049	144.1	1.10	0.60	54.5
6.0~ 6.8	2.60	2.40	92.3	23.5	43.1	188.3	—	—	—	—	—	—
8.5~ 9.3	—	—	—	—	—	—	0.031	0.042	135.5	1.81	0.42	23.2
9.5~10.3	2.90	2.80	96.6	20.6	53.9	261.9	—	—	—	—	—	—
12.2~12.5	4.00	—	—	13.7	—	—	—	—	—	—	—	—

B : before improving soft ground
A : after improving soft ground

From China Wang (1989) describes identical design methods and equations to those of the Japanese, and yet neither paper references the other. A summary of the application of lime piles in China is shown in Table 2.9.

Kado et al (1987) describe the application of the Japanese method to soft soils in Singapore. Therefore, the design and installation techniques are identical to those described earlier by Kitsugi and Azakami (1982). They also describe the application of lime piles to two construction problems encountered on the route of the Mass Rapid Transit System in Singapore. In the first of these, at Lavender Tunnel, the piles were used to limit diaphragm wall movements around a coffer dam. Piles were placed on a 1m square grid. Four weeks after placement two boreholes were sunk to investigate soil properties. Diagrams showing results from these tests and the location of the boreholes are very unclear and are not further discussed in the text. The implication is, however, that the technique was successful. The second application, at Burgis Station, has also been poorly presented.

A clearer account of the same work has been produced by Chew et al (1993). The piles were used to control retaining wall deflection during excavation. Properties of the soils encountered over the depth of excavation are shown in Table 2.10.

Table 2.9 Examples of lime pile projects in China (after Wang, 1989)

No.		1	2	3	4	5	6	7	8
Name of object		Shop and housing building	Housing building	Shop and Housing buildings	Housing buildings	Spinning workshop of a textile mill	Poaking workshop of a silk mill	Dining-room	A residential quarter
Place		Hangzhou	Hangzhou	Huzhou	Hangzhou	Shanghai	Hangzhou	Hangzhou	Hangzhou
Storeys		6	6	6	5	1	4	1-2	3-4
Soil condition		miscelleneous fill, 2.2m in thickness; siltation fill, 2.2m in thickness; loam, 1.2m; mucky soil, 29m in thickness.	plain fill, 2.2m in thickness; sandy loam, 0.9m; mucky soil, up to 32m in thickness.	miscelleneous fill, 2m in thickness; siltation fill, 2.2m; loam 2.2m; mucky soil 15m.	loam 1.6m in thickness; mucky soil 29m. There is a pool in west part.	loam 2.0m in thickness; mucky soil 2m; sandy loam 1m; mucky soil 20m in thickness.	loam 2.0m in thickness; mucky soil 20m in thickness.	miscelleneous fill, 3m in thickness; mucky soil, 20m in thickness.	miscelleneous fill, 3m in thickness; mucky soil, 20m in thickness.
Lime pile	Dia. (mm)	325	325	325	377	377	325	325	325
	Length (m)	5.5	8.0	9.0	7.0	6.0-7.0	6.0	6.0	3.0
	Spacing (m)	1.20-1.30	1.20-1.30	1.10-1.20	1.20-1.30	1.30	1.20	1.20	1.20
Completed date		1985	1984	1987			1984	1984	1984
Settlement (cm)		5.0	19.0	4.2			3:7	4.0	

Table 2.10 Typical properties of the clay treated using lime piles (after Chew et al, 1993)

	γ (kN/m^2)	G_s	ω (%)	LL (%)	PL (%)	c_u (kN/m^2)	P_c (kN/m^2)	C_c
Upper Marine clay	15	2.6	60-80	80-95	50-65	10-30	92	0.85
Lower Marine clay	15	2.6	50-60	60-80	35-50	40-70	114	0.85

Notes: P_c is the preconsolidation pressure

Piles were placed at 1.0 m centres on a square grid to a depth of 9 m, over one area, and 6 m long piles on a 1.4 m grid were used in another area. The 1.0 m grid piles began 12.0 m below ground level; the 1.4 m grid piles began 12.2 m below ground level. During excavation (approximately 12 weeks after pile placement) tests were carried out on clay at about mid-depth at the positions shown in Figure 2.19. Results from these tests are shown in Table 2.11. Samples from the piles were also taken and tested in unconfined compression. An average strength of 540 kPa was recorded. The authors note that measured shear strengths of the clay were lower than those calculated. The explanation that they offer is that the discrepancy is either due to stress release during excavation or optimistic soil parameters being used for the original calculation.

Table 2.11 Properties of soil between Lime Piles (adapted from Chew et al, 1993)

Chemo pile pitch (m)	Depth (m)	Average Vane Shear Strength (kN/m^2)		Original Soil Vane Shear Strength (kN/m^2)	Increase in Shear Strength (kN/m^2)		Average Water Content (%)		Original Water Content (%)	Decrease in Water Content (%)	
		Dia	Side		Dia	Side	Dia	Side		Dia	Side
1.0 x 1.0	GL - 15.8						50	48	70	20	22
	GL - 16.0	58	70	35	23	35					
	GL - 17.5						57	57	70	13	13
	GL - 17.7	70	80	40	30	40					
1.4 x 1.4	GL - 15.8						60	60	70	10	10
	GL - 16.0	55	60	35	20	25					

Notes:
The location of samples described as Dia and Side is given in Figure 2.19.

Tsytovich et al (1971) of Russia describe the use of lime piles to aid the compaction process of 30m of saturated loess. They state that from their (presumably significant) experience of the technique the best results were achieved in porous soils such as saturated loess. These conditions would undoubtedly enhance water movements within the soil, thus promoting a more complete reaction. Low activity limes had also been tried unsuccessfully. 250mm to 500mm diameter piles were formed using a closed-ended tube to create holes that were filled with lumps of quicklime and compacted. The piles were 5m to 8m long and were formed at 1.5m to 2.5m spacings. The authors believe that (loess) compaction was achieved by the following processes:

1. Driving of the closed-ended tube.
2. Expansion of the lime pile as a result of slaking.
3. Heat of the slaking reaction causing water content reduction through evaporation.
4. Chemical reaction between the clay and the lime penetrating the soil.

They present details of two practical trials of the use of lime piles. The first piles were constructed in groups of 12 at 2m centres within a loess soil having the properties shown in Table 2.12. Pile diameter is not recorded in this specific case, but the authors state generally that the technique creates piles having initial diameters of 250mm to 500mm.

Table 2.12 Properties of loess treated by Tsytovich et al (1971)

Pre-Treatment Properties	
Water content	24% to 30%
Plastic Limit	19% to 21%
Liquid Limit	25% to 30%
Degree of saturation	95% to 100%
Effective internal angle of friction ϕ	17 to 20°
Effective cohesion c'	4 to 5 kPa
Properties of Loess 1m from Edge of Piles	
Degree of saturation	61 to 72%
Effective internal angle of friction ϕ	17 to 23°
Effective cohesion c'	22 to 31 kPa

Steam was observed and cracks up to 40 mm wide and 480 mm long developed around each pile. Temperatures observed at the centre and edge of the piles peaked at around 4 hours after placement with maximum values of 390 and 50°C respectively. These are very similar values to those observed by the Japanese with piles placed in peat having a water content of 400% to 460% (Kitsugi and Azakami, 1982). The pile diameter was observed to increase in size by between 26% and 60%, representing a volume change of between 160% and 260%. Samples were taken 1m away from the edge of the piles and were found to have a significantly reduced degree of saturation and increased strength parameters, as shown in Table 2.12. A plate load test was carried out in the area containing the pile group, which deformed substantially less when compared with the untreated ground.

There are several questions arising from the data presented. It is unclear at what depth the tested samples were taken and, hence, if the water content reduction occurred over the entire pile length or if the reduction was a surface effect attributable to the steam. In soil with a water content close to the Liquid Limit the apparent volume expansion of the piles could have occurred at formation, the compaction process expanding the cavity. The presence of the cracks would indicate a net volume loss. The difference in strength observed could be attributable to the reduction in water content, especially in a soil of such low plasticity. As it is unclear where in the pile group the plate load took place, it is impossible to speculate on how much of the load would have been taken up by the piles themselves. The second trial demonstrated that the soil, previously too poor to compact, was improved by pile treatment by a sufficient amount to allow further compaction, 'by rams', of a site being prepared for foundations. This use of lime piles as a construction expedient is apparently novel and potentially important.

Chiu and Chin (1963) examined the expansive behaviour of quicklime piles as part of a study to assess their potential to treat Taipei Silt, in Taiwan, for foundation purposes. The experimental procedures that they adopted are unclear, although a photograph indicates that the soil was compacted into boxes and small quicklime piles were formed within the soil. An effective water supply and vertical surcharge were thought necessary for the full expansive force on the soil to be realised. The authors also thought that water content reduction was greatest for high initial water contents. This agrees with what can be surmised from the Japanese paper. The authors also mention the possibility of chemical reaction between the lime and the surrounding soil, although this aspect is not pursued in any detail in the context of soft soil improvement. It is, however, discussed by those authors describing the use of lime piles to treat failing slopes.

The expansive nature of piles has been dealt with more theoretically by Law and Chen (1993). Their paper presents a model for the prediction of the magnitude of excess pore water generation during the installation and subsequent slaking of lime (mixed with 'fly-ash') piles. The authors believe this excess pore water pressure to be the cause of lateral consolidation and hence soil improvement. Since reference is made to pile driving, it is presumed that the holes for the piles are driven and not augered. The authors state that they further develop the ideas of Holeyman et al (1983) and their model based on cylindrical cavity expansion. The principal difference is said to be the use of a non-linear pore water pressure distribution on the radial axis. Equations for excess pore water pressure are developed from consideration of increments in volumetric strain, radial and circumferential stress, and stiffness of the soil ring surrounding the pile. In considering volumetric strain, the following parameters, associated with the quicklime, are used:

- volumetric expansion under atmospheric conditions
- compressibility of quicklime
- the ratio of the compressibility of slaked lime to quick lime
- the ratio of the volume of reacted lime to the volume of the initial cavity
- the ratio of the volume of the quicklime which has reacted within the time increment to the volume of the initial cavity
- the ratio of the actual expansion to the restrained expansion

The method of quantifying these parameters is not documented, despite their inherently empirical nature.

The authors investigate the effect of different soil parameters on the magnitude of calculated pore water pressure with the following results:

- increasing Young's Modulus increases pore water pressure
- increasing Poisson's Ratio increases pore water pressure
- reducing pile spacing increases pore water pressure

Permeability does not affect the result due to the model assuming undrained conditions. This last point must be a serious flaw in the model as permeabilty will affect the rate at which water can reach the pile, hence the rate of the slaking reaction and subsequent reaction. Projecting this idea further, permeability will have an effect on the rate of pore water pressure dissipation and hence consolidation. It is unclear how much of the above exercise was purely applying numbers to the equations, or if any of the results were based on experimentation.

A case study is described where lime is mixed with fly-ash in the proportions 7:3 respectively. 200 mm diameter piles of 5 m length were placed on a 0.7 m triangular grid. Electric piezometers were placed at two depths at the centre of one triangle. The actual diameter of the piles immediately after placement was said to be 260 mm, increasing to 280 mm after a month. The highest recorded value of pore water pressure was 105 kPa, 2 hours after the piles were complete. A calculated value of 104 kPa was derived using particular quantities for the parameters describing lime expansion listed above. It is not stated from where these quantities were derived. Despite the fact that the authors state that the phenomena of drilling, placement and expansion are distinguishable in the pore water pressure response, it seems likely that the major generation of pore water pressure is due to the compaction of the quicklime. A 30% increase in diameter is recorded after compaction, with only a further 7.6% increase due to slaking. The response time of the piezometer may also mean that the value recorded 2 hours after pile completion corresponds to an effect of compaction.

The generation of high pore water pressures has not been recorded by other authors, but would have to be considered if lime piles were to be used in a situation sensitive to such pressures, like slope stability. The effect of 'fly-ash' (as opposed to quicklime only) has also not been investigated.

2.4.2 Lime Pile Stabilisation of Slopes

The general principle behind the stabilisation of slopes is quoted as being lime migration from the piles into the surrounding soil followed by reaction with the clay *in situ* to improve the strength of the clay. In order for this to occur both calcium (Ca^{2+}) ions and hydroxyl (OH^-) ions must migrate, or be transported, away from the piles into the surrounding soil, the hydroxyl ions thus creating the highly alkalinic conditions necessary for the stabilisation reactions to occur. Several American authors describe the use of lime piles in slope stabilisation, including Handy and Williams (1967), Anon (1963) and Lutenegger and Dickson (1984).

Handy and Williams illustrate their paper with a case study in which quicklime piles were successfully used to stabilise a failing fill slope where more conventional treatments had failed (Figure 2.20). 150mm diameter piles were constructed on a 1.5m grid to penetrate the shear zone situated within a perched water table. The holes were augered, and the quicklime was poured into the holes and 'watered'. The clay fill was thought to have a mineralogy of predominantly calcium montmorillonite, and had properties in the active slippage zone of $\emptyset' = 17^{o}$, $c' = 4$ kPa and a water content of 27.2%. Design methods were entirely based on the predicted migration distance of the lime and on the soil type in relation to the subsequent lime-clay reaction. After 3 months samples were taken, although the sample locations were not described, and the water content had fallen by 4%. After 1 year the unconfined compressive strength had risen relative to the 3 month strength by approximately 55%. Acid soluble aluminate and silicate were said to be indicative of a reaction having occurred 300 mm from a pile. After 2 years a soil sample taken 300mm away from a pile and 2m below the top of the pile had $\emptyset' = 21^{o}$ and $c' = 9.5$ kPa. No lime movement could be detected above the water table, thus indicating the need for water in the movement of the lime. However, it was stated that the piles were 'watered', thus providing a water supply to the region above the water table. The necessity for water, therefore, must be in the longer term if these observations are correct. Tests on the lime piles indicated that very little of the lime had reacted. A subsequent slip occurred underneath the treatment area, highlighting the need for comprehensive analysis of the unstable region prior to treatment.

Over the two year period after treatment the cracks in the houses that were founded on the slipped area closed up and no further cracking took place. This could be explained initially by the expansion of the piles due to the introduction of additional water. In the longer term, the strengthening of the lime placed through the shear zone prevented further movement. If some hydration of the piles was caused by water from the softened zone around the slip plane, improvement of properties could have been affected by dehydration and consolidation. The improved soil properties from a second programme of sampling could be explained by lateral consolidation, although the time dependence of the improvement is difficult to attribute. However, the positioning of the two programmes of sampling was not necessarily precisely identical and hence this could lead to the discrepancies in the quoted results.

The most interesting result is the measurement of acid soluble aluminates and silicates, which are indicative of the lime-clay reaction. The equation of Davidson et al (1965) used to calculate rates of movement appears to be invalid when there is a movement of water (as discussed later in this paper). However, purely considering permeability and the flow of water, a rate of ion migration of the order quoted would indicate a coefficient of permeability for the clay of $5*10^{-9}$ m/s, which seems reasonable. The anomaly appears to be that the treatment was designed on the basis of the quantity of lime required to promote a reaction and very little of the lime had reacted. As it is believed that a certain pH and concentration of lime is required before aluminates and silicates go into solution, it is questionable as to whether a reaction was occurring. It may be possible that the acid used in the test is breaking down the clay and giving these results.

Anon (1963) describes the development of the lime pile technique as a rapid and inexpensive way of stabilising road subgrades *in situ*. 230mm diameter holes were drilled to the full depth (approximately 760mm) of the fill constituting a failed embankment slope at 1.5m centres. These were then filled with hydrated lime, followed by enough water to make a thick paste. The holes were then plugged with some excavated material and 150mm of asphalt. Immediate trafficking was then permitted as this was thought to promote lime migration. Observed distress to the surface ceased within 3-4 weeks and evidence of lime in boreholes 1.5m away was found after 5 weeks. It is possible that a path for the rapid migration of lime was provided by the subbase layer. If this was the case the vibrations caused by the traffic movement would have caused an outward seepage of the lime slurry through this relatively coarse material, and this would be what was noted in the borehole.

This technique was also used to stabilise a 7.5m high, 90m long section of embankment which was moving laterally, causing the highway passing over the top to shear. 115mm diameter holes were drilled at 3m centres through the depth of the embankment and filled in the same manner as previously described. A month after treatment a levelling course was applied and no further movements were observed. No details of testing techniques nor long-term performance were presented for these two sites.

Lutenegger and Dickson (1984) reviewed the use of lime piles in the mid-west USA. They describe several case studies and offer explanations for their stabilising mechanism, but without experimental evidence. They state that, since 1963, over 17000 linear metres of lime piles have been successfully installed to stabilise landslides. They believe mineralogy to be of importance in both the success of the stabilising reactions and on the rate of diffusion of the ions, suggesting that only montmorillonitic soils are suitable for the treatment. No evidence is presented on whether the technique has been used unsuccessfully on other clays.

The design of pile spacing and diameter is considered in their paper, a large number of small diameter piles being favoured. The pile spacing is directly related to diffusion rates using the equation quoted by Davidson et al (1965), as used by Handy and Williams (1967), which states that the diffusion distance (L) is proportional to the square root of time (t), the relationship being governed by a constant (k), thus

$$L = k.\ t^{0.5}$$

They suggest that the flow of water promotes migration and they therefore advocate the installation of piles immediately preceding the rainy season. They also predict that this water flow criterion is usually satisfied in an active landslide at the most critical zone. However, Davidson et al (1965) state that the equation is only valid if the reaction rate is very much greater than the diffusion rate, and therefore if water movement was considerable the equation would no longer apply.

Lutenegger and Dickson advocate that the treatment should be applied only in the top half to one third of the slide "where it is most needed". This is in opposition to more modern ideas on slope stabilisation, which advocate treatment in the lower third, although they were probably influenced by the location of the cracking houses.

Considerable attention has been given to the use of lime piles in India, both in the field and in the laboratory. Chummar (1987) confirms their use in several different types of soil through the use of laboratory experiments. Marine and moderately expansive clays were used in model experiments, similar to those described by Chiu and Chin (1963), in which 20 mm diameter quicklime-sand piles at 200 mm spacing were used. He also describes their successful use in the field, including the stabilisation of a lateritic clay embankment, piles being placed at the likely position of the slip plane. Unfortunately there is no indication of how the laboratory trials related to the field trials, or how the pile treatment (spacing, diameter, etc) was designed.

Further field trials were described by Shanker et al (1989). They describe 300 mm diameter piles at 1.5 m spacings each forming a corner or centre of hexagonal groups. Lime only, lime and sand and lime and soil piles were all assessed for their ability to stabilise Black Cotton soil. Again, the flow of water to transport the lime was felt essential, so the piles were 'watered' for 3 weeks after installation. Also, a 50 mm diameter sand core was provided in the soil-lime pile as it was thought this would aid drainage.

Presentation of results was generally poor, but in all cases improvements were recorded in the soil surrounding the pile. These included increases in Plastic Limit, shrinkage limit and undrained shear strength, and decreases in Liquid Limit and swell characteristics. Measurements of calcium content were also made, but insufficient data were given to correlate them with other improvements.

Other recent Indian work has included studies of the effect of pile shape on its efficiency by Ayyar et al (1989) and a study of the effect of pile spacing by Venkatanarayana et al (1989). The first of these studies considered the behaviour of four types of pile in the laboratory:

1. a quicklime pile;
2. an annular lime pile with a central sand core;
3. a lime pile with occasional horizontal protrusions; and
4. a pile of slaked lime.

The quicklime pile and the pile with protrusions were also examined in a field trial.

The laboratory trials used clay wetted to allow compaction into a 560 mm diameter, 850 mm deep tank. The mineralogy of the clay used was predominantly smectite, with a Liquid Limit and Plastic Limit of 60% and 21% respectively. Augered holes were filled with quicklime and allowed to reach equilibrium with a thin layer of water on the surface. Pile sizes were calculated so as to achieve

2% by dry weight of lime. 38mm diameter tube samples were taken at fixed distances from the pile after 1, 2, 3 and 6 weeks and analysed for calcium content, shear strength and water content.

Again, presentation of results was somewhat unclear. After 6 weeks water contents had reduced for the lime pile and the pile with protrusions, more markedly in the latter case. In the case of the annular pile water contents had returned to original values and, with the slurry pile, water contents had actually risen. The greatest increase in shear strength was observed using the pile with protrusions, whereas the slurry pile actually reduced the strength of the surrounding clay after 6 weeks. The authors suggest that the efficiency of the former is due to the confinement of the clay between the protrusions. They explain the increased water content produced by the slurry pile to be due to some sort of swelling reaction caused by sulphates, thus reducing the shear strength. In all cases, it was stated that more calcium than was required for ion exchange had migrated into the surrounding soil. It is unclear, however, from where the samples were taken, and hence how far the ions had migrated and what calcium ion concentration was thought to be required for ion exchange.

Several points of interest can be drawn from this paper. Firstly, the strength gains observed could possibly be attributed to several other coinciding processes. Initially, only the quicklime piles will reduce water contents due to the slaking reaction, thus accounting for the absence of such a reduction using slurry. The differences in the pile designs may produce a different reaction rate, thus producing different water content changes at any one time: for example the pile with protrusions creates a greater area of contact between lime and clay, hence increasing the reaction rate. In addition the pile with protrusions will also hold more lime. The increased shear strength may therefore indicate simply a greater water content reduction. Conversely the increase in water content observed with the slurry pile may be due to the outward migration of water, the increased water content thus reducing the shear strength.

Another explanation could arise from the effect of lime on the clay-water interaction. The water associated with each clay particle is reduced after cation exchange has taken place, creating an excess in the system. If, at the time of testing, cation exchange was still proceeding this could account for the increased water content. It could also account for the rise in water content recorded in the case of the annular pile after the slaking reaction had ceased. However, the annular pile will contain the least amount of lime, thus perhaps only reducing water contents by an imperceivable amount in the area not immediately adjacent to the pile. The results from similar tests from the field trials appear not to be discussed.

The study of the effects of pile spacing by Venkatanarayana et al (1989) also considered clay compacted into tanks and inundated with water for an initial period after pile installation. Samples taken from between the piles were tested and results basically demonstrated that smaller spacings gave greater degrees of improvement, as one would expect. The results, however, are both insufficiently detailed and so poorly presented that definite conclusions are difficult to draw.

They present a graph (Figure 2.21) which attempts to show a comparison of the effects of the piles compared with the more traditional mix technique. Although the graph is difficult to use, by considering a particular spacing to diameter ratio, working two dimensionally, one can work out the percentage of lime contained in the piles and compare the improvements produced by the same percentage using a mix technique (e.g. for a spacing to diameter ratio of 5 the lime percentage is 5 and the improvement produced using piles corresponds to that of the mix technique). It is, however, felt inappropriate to directly compare percentages of lime between the different treatment methods and this graph should be used with caution. Using a mix technique, the modification process relies partly on pulverisation, flocculation and recompaction to achieve improvements in plasticity, strength and workability. The cementation reactions then bind small 'lumps' of (modified) clay together. In a non-mix technique, these processes do not happen. Improvements from the lime-clay reaction must be derived solely on a molecular level, the main stabilising mechanism being derived from other processes entirely.

Rao et al (1993) present the most recent work in this area from India. The authors investigate the effect of lime piles on a marine clay with Liquid Limit and Plasticity Index of 85% and 53% respectively. A 600 mm diameter, 550 mm deep tank was filled with the clay in 30 to 50 mm deep layers. A 50 mm diameter pile was formed at the centre of the tank. Piles were formed of quicklime, slaked lime and a 1:1 sand-quicklime mixture. Seawater was use to mix with the clay and 50 mm of seawater was placed over the entire surface of the tank. Various resulting phenomena were investigated and shall be discussed in turn.

1. Themocouples were placed within the piles and at various distances from the piles. Resulting readings are shown in Figure 2.22. The authors state that a temperature of 50 °C was reached in soil adjacent to the quicklime pile, although the graph appears to show only 30-35 °C at 80mm from the quicklime pile. All temperature effects were diminished to ambient within 1000 to 2000 minutes (16 to 34 hours).

2. Variation in Atterberg Limits was studied in the soil surrounding the piles. The results of this study are presented in Figure 2.23. There are several comments which must be made concerning the reliability of these results and what they actually show:

   i. The authors do not state if the measurements for 'untreated soil' were taken for soil mixed with seawater. The salt content of seawater will have a profound effect on soil properties. Lees et al (1983) studied the effects of sodium chloride on clays and found the effect to be analagous to that of lime.

   ii. The nature of the seawater is not explained and therefore may be variable between tanks (and hence types of pile material) giving inherently differing results for each type of pile.

iii. The seawater layer over the surface of the tanks provides a transport route for the lime within the piles. The lime, thus transported, will react with the clay, thus altering its properties. The quicklime pile provides the highest concentration of lime, therefore the highest degree of reaction.

These issues serve as the basis for the argument against the reliability of the remaining results.

3. Lime Migration was monitored by measurement of the concentration of the Ca^+ ion, although the authors do not state how the measurements were made, i.e. if free calcium was measured. The results are presented in Figure 2.24. Again results are prejudiced by the unknown nature of the seawater and the possible transport of calcium via the water layer. In addition pH was measured (thus monitoring the migration of hydroxyl, or OH^-, ions) and the results are also presented in Figure 2.24. It would be expected that the concentration of lime, and hence pH, will be greatest near the pile, even if the transport mechanism is via the water layer. The concentration would go up and reach saturation point as shown. It should be noted, however, that the pH in these tests does not rise to sufficiently high levels (pH $\simeq$ 12.4) for stabilisation reactions to occur.

4. The increase in strength was measured using "a cone penetrometer fabricated for this purpose", the results being shown in Figure 2.25. Again, strength gain may be derived from seawater treatment, or the lime transported by the surface water layer. Depending upon the depth at which the tests were performed, some strength gain may be derived from drying caused by saturation of the pile material and pile slaking.

5. X-Ray Diffraction Studies were carried out and diffraction patterns are presented which are intended to show the development of pozzolanic reaction products. It is difficult to contradict the evidence, except that the background scatter on the reacted sample is exaggerated when compared with the pre-treatment sample. This could indicate some degree of amplification of the pattern resulting from the treated sample.

Also, the additional substances may be due to the seawater, either directly or as a result of its reaction with the clay.

Generally, this paper may show some interesting and significant results, but is prejudiced by the lack of details about the treatment with seawater. The authors appear too enthusiastic to apply all known aspects of the lime-clay reaction to the lime pile process. This may also have led to misinterpretation of results.

A further description of the use of lime piles for slope stability is presented by Brandl (1980), who describes their use in Austria since 1968. Long-term laboratory tests and field trials have shown that long-term strength is derived from diffusion of lime from the piles. Their efficiency for any particular application is thought to be controlled by the clay mineral content, ion exchangeability and the presence of movable silica. The most suitable soils were thought to be 'loosely packed' and of low plasticity. These soils, it should be noted, promote a greater degree of water movement and hence drying also. The reasons given are that such soils have an increased natural permeability and remove the risk of rapid cementing reactions preventing further diffusion by 'sealing in' the lime.

Piles of between 80mm and 500mm in diameter at spacings of 0.5m to 3.0m (depending on soil characteristics) have been employed to give changes in $\emptyset'$ of as much as 15°. No long-term data were available, but the author doubts if the reactions can be reversed with time.

Ruenkrairergsa and Pimsarn (1982) describe the use of lime piles for slope stabilisation in which holes filled with lime and water were constructed in a loose clay shale fill embankment in Thailand. The piles were hand augered to a diameter of 150mm on a 3m grid through to the natural ground (Figure 2.26). 569 holes were drilled giving a total length of piles of 2300m. Lime and water were simultaneously poured into the holes, and the mixture was topped up four times a day over a 2 month period. Construction of the piles took place immediately prior to the rainy season as it was thought that lime migration would not occur without the flow of water.

Lateral movement of the embankment was monitored by means of aluminium rods driven through the made ground into the natural ground below. Shear strength was monitored *in situ* using an 'Iowa borehole shear device' (Handy and Fox, 1967). This also facilitated sample recovery for testing for water content and consistency limit changes. Samples were taken at 0.25, 0.50, 0.75 and 1.50m away from the pile centres and at depths of 1, 2, 3 and 4m immediately after the rainy season. Results of lateral movement monitoring are shown in Table 2.13, whilst water content and shear strength changes are shown in Table 2.14.

The authors state that lateral movement has been reduced, although the results show that movement is still occurring. The significance of the reported movements is difficult to ascertain since no 'pre-treatment' data are available. The strength changes could have occurred as a result of water content reduction. The reason for such a reduction has been attributed to improved drainage caused by the lime migration, although this observation is clearly contradictory to that of Ayyar and Ramesan (1989) reported above for slurry piles in clay. It is also possible that a difference in the climate between the years in question could be the cause. Without control data, however, this can be merely speculation. It is worth noting that a 'less wet' rainy season would also reduce movement if water were the trigger of the slip. The changes to Plasticity Indices are more interesting. Whilst the Liquid Limit has remained unchanged, the Plasticity Index has been reduced from a range of 20-30% to an average of 12%. This type of reduction in plasticity could reasonably be attributed to a lime-clay reaction.

Table 2.13 Lateral movement of a Clay Shale Slope after Lime Pile Treatment (after Ruenkrairergsa & Pimsarn, 1982)

Date	Elapsed Time, days	Distance from Observation Stakes to Reference Stakes, meters				Distance increased from Original Value, meters			
		Line 1	Line 2	Line 3	Line 4	Line 1	Line 2	Line 3	Line 4
May 20, 77	0	15.005	15.560	15.620	15.184	0	0	0	0
April 1, 78	311	15.024	15.571	15.633	15.184	0.019	0.011	0.013	0
Dec 1, 78	551	15.035	15.600	15.690	15.200	0.030	0.040	0.070	0.016
June 17, 79	748	15.040	-	15.724	15.204	0.035	-	0.104	0.020

Table 2.14 Water Content, Cohesion, and Angle of Shearing Resistance (after Ruenkrairergsa & Pimsarn, 1982)

RANGES AND AVERAGE VALUES OF WATER CONTENT, COHESION, AND ANGLE OF SHEARING RESISTANCE IN THE YEARS 1977 AND 1979

Test Depth m.	Water Content, %				Diff In Avg. w/c, %	Cohesion, kN/m²				Diff In Avg. C, kN/m²	∢ of Shearing Res., deg				Diff In Avg. Ø deg.
	1977		1979			1977		1979			1977		1979		
	Range	Avg.	Range	Avg.		Range	Avg.	Range	Avg.		Range	Avg.	Range	Avg.	
1	30.2-55.0	43.4	27.4-48.1	37.4	-6.0	0-18.3	7.1	0-17.9	7.2	0.1	22.0-44.5	33.9	29.5-48.0	39.8	5.9
2	30.5-51.2	42.6	23.5-53.8	38.3	-4.3	0-37.9	12.9	0-30.3	13.9	1.0	22.1-39.4	31.2	28.0-47.0	39.3	8.1
3	31.1-52.8	42.4	26.8-54.0	38.9	-3.5	0-35.0	11.3	4.1-27.6	15.5	4.2	20.0-40.2	32.3	31.5-46.5	38.8	6.8
4	28.5-54.5	42.5	22.3-51.8	38.0	-4.5	0-38.6	12.2	3.4-29.7	15.7	3.5	20.1-44.0	33.3	27.5-47.6	37.5	4.2

(+) means increase, (−) means decrease; 100 kN/m² = 14.5 lb/in²

Whilst this appears to be a record of the successful use of lime piles in slope stabilisation, it should be noted that the lime has been placed with a continuous supply of water over a 2 month period. This would have undoubtedly promoted lime migration through a reasonably permeable material. However, this type of operation would be extremely labour intensive and less suitable for the typically stiff, low permeability clays that occur in the UK.

Many of the authors who have addressed the problem of slope stabilisation above have considered lime migration and the subsequent reaction to be a major stabilising mechanism. Several authors take up the issue of lime migration through the soil specifically using laboratory testing.

Fohs and Kinter (1972) carried out a study using high calcium hydrated lime to treat small blocks of clay of differing mineralogy, one essentially montmorillonitic and the other kaolinitic. Free calcium ion concentrations were detected by means of acid extraction and titration with Ethylenediaminetetracetic Acid (EDTA). Values of water content were also recorded. They found that for both soils tested migration after 180 days was less than 40mm and that, even at a distance of 5mm away from the source, migration was complete after 100 days. Interestingly, they found that the water movement occurred far more rapidly, indicating the two processes to be at least only partially connected. Other factors leading to the same conclusion were that the most concentrated solution of lime contributed most lime to the soil and that increasing the amount of pore water had no effect on migration. In addition the hydrated lime must have provided sufficient water to carry the calcium ions. One piece of information left unexplained was the observation in the montmorillonite of a dry crumbly appearance to the portion nearest the lime. This could have indicated the occurrence of a reaction possibly inhibiting migration.

Lime migration into unpulverised clay was also examined by Stocker (1975). Clay samples were specifically prepared to include such unpulverised material. The investigation examines the effect of lime and cement stabilisation on the unpulverised 'lumps' within a treated soil mass. The paper posed difficulty for review as a detailed knowledge of chemical analysis techniques was required to be wholly critical. Generally, the investigation appeared to have been conducted in a systematic and rigorous manner and be well reported. It is proposed to present a summary of the paper in three parts: the properties of the soil used, the method of sample preparation and the author's conclusions.

Details of soil properties are included in Table 2.15. The high montmorillonite content should be noted. The lime used in the study was a calcitic hydrated lime and the cement was a Type I Portland cement. Precise details of sample preparation techniques are said to be given in Stocker (1968, 1972b). Soil was carefully graded to provide a maximum density lump grading with maximum lump size either 20 or 40 mm. Lime or cement contents were 0, 3 and 15 %, with curing times ranging from 30 minutes to 1 year. The soil was mixed to a pre-treatment moisture content of 12 % before addition of stabiliser and optimising water. British Standard Compaction was used at the Optimum Moisture Content appropriate to the mixture. Samples were then kept at 25 °C for the desired curing period.

The lumps were dissected in a carbon dioxide free atmosphere with 100 % humidity. The material between the lumps is referred to as the matrix. The outer 1 to 2 mm layer of the lumps, together with the matrix, was removed from any analysis as any stabiliser present may have affected the results. Specimens were then vacuum dried over saturated calcium nitrate, since this was believed to arrest the hydration reactions of cement and the lime-clay reaction. Samples were then lightly hand-ground with an agate pestle and mortar to a sufficient degree to separate sand grains. Chemical extraction procedures were said to be chosen to effect maximum dissolution of the cementitious products without affecting the unmodified soil.

Table 2.15 Soil Properties used by Stocker (1975)

Colour: mid-brown, flecked with white gypsum crystals

Particle size distribution:

BS Sieve	Per cent passing
52 mesh	100
100	98
200	91
$<5\mu$	61
$<2\mu$	52
$<0.2\mu$	24

Specific gravity: 2.71

Atterberg limits:

LL = 55 per cent
PL = 24 per cent
PI = 31 per cent

$$\text{Activity} = 0.6 \left(\frac{\text{PI}}{\% < 2\mu}\right)$$

Shrinkage limit: 12 per cent moisture content

Cation exchange capacity:

(N/20 HCl teaching)

Total exchange capacity	46 meq/100 g
Ca^{++}	36 meq/100 g
Mg^{++}	7 meq/100 g
Na^{+}	2.5 meq/100 g
K^{+}	0.5 meq/100 g
TEC of $<2\mu$ fraction	89 meq/100 g

Clay mineralogy:

Montmorillonite	approx. 30 per cent of insoluble wt
Kaolinite	approx. 13 per cent of insoluble wt
Illite	approx. 7 per cent of insoluble wt
Chlorite	approx. 2 per cent of insoluble wt

(Non-clay, almost all quartz — 48 per cent)

Specific surface areas:

Total	$250\ m^2/g$
Internal	$< 160\ m^2/g$

pH:

(1:5 soil:water, glass/calomel)
pH = 8

Soluble salts:

Predominantly ≈ 1 per cent gypsum
TDS at 40 per cent MC ≈ 30 meq/l

Organics content:

Organic carbon = 0.40 per cent (wet combustion)
Organics content ≈ 0.7 - 0.8 per cent

Optimum moisture content:

(BS compaction) OMC = 18.5 per cent

Note: meq = milliequivalent

The conclusions drawn by the author about the study as a whole are summarised as follows:

1. Lime diffused from the matrix into the lumps. The lumps within specimens with a 15% lime addition gained an average of 6% lime after 1 year; the 3% lime addition samples gained 2%.

2. Lime from the hydrating cement also diffused: a 15% cement addition produced lumps with an average lime content equivalent to the lime produced by 5% cement. All the lime expected from the addition of 3% cement had diffused.

3. 'Diffusion fronts', observed by physical differences in clay appearance, were shown not to be the limits of diffusion: 0.5% lime contents were present ahead of the fronts.

4. 0.5% lime in the lumps was sufficient to eliminate swelling from samples wetted from the cured state. 2% lime was sufficient to eliminate swelling from samples that had been air-dried.

5. 2% diffused lime increased strength ten-fold.

6. 3% diffused lime produced samples of very low permeability.

7. Diffused lime reacted with the clay at all ages, giving a product with SiO_2:Al_2O_3 molar ratio of 4.5:1. Reaction was thought to have been exclusively with the edges of the montmorillonite indicated by the slow release of potassium.

8. Identical reaction products were produced in the matrices.

9. Very little montmorillonite was consumed even when striking changes in physical properties had been produced.

96% remained unreacted in the sample that had gained in strength ten-fold.

10. No abrupt change in extent of reaction occurred at the matrix-lump interfaces, nor at the diffusion fronts.

11. Lime-clay reactions occurred in the matices of the cement-clay samples.

12. Cementation was thought to have been produced by deposition of reaction products at crystal edges. 0.5% lime effected cementation, yet it was sufficient only to provide a unit layer coverage of crystal edges.

13. The first layer coverage was thought to suppress the reaction rate by hindering lime diffusion to the reactive crystal edges. This allowed a rise in lime concentration in the adjacent pore fluid and provided the potential needed for deeper diffusion. Thus the first 0.5% lime was rapidly gained throughout the whole system.

14. The retardation of the reaction was thought to mean that a high concentration of lime was acheived everywhere. This led to a uniformity of reaction products and hence properties.

15. 'Normal' stabilisation was produced by the diffusion of lime, i.e. 6% diffused lime gave the same property changes as 6 % mixed lime.

Some general comments may also be made concerning these results in the context of the proposed project. Most significantly, the author states that it is the montmorillonite that is specifically attacked by the diffusing lime. There is no study of the effect on other minerals in isolation. Montmorillonite is rare (certainly in the quantities quoted) in British clays, so these findings cannot be directly applied. Hydrated lime is used, which has no potential for disturbing the balance of water content. It cannot be presumed that quicklime will behave in the same manner. Lastly, the largest lump size is 40 mm, meaning that, given that it is surrounded by the lime-clay matrix, 20 mm is the extent of migration. No study is made of the limits of the extent of migration or if different mechanisms operate if migration distances are not limited.

Katti and Gupta (1970) studied the migration of lime through soil by laboratory trials. Two trials were set up: quicklime piles in soil and soil piles in quicklime. The sizes of the soil pile were varied, whilst the sampling distances were altered in the case of lime piles. In both cases Black Cotton Soil was used, having a Liquid Limit of 80% and a Plastic Limit of 46%.

The density and water content for each test were varied, with soil pile sizes of 50 x 50mm, 100 x 100mm and 150 x 150mm studied. Unconfined compressive strength and shrinkage limit were used as measures of migration. Generally, it was observed that:

1. Lime had travelled to the centre of a 150mm soil specimen after 120 days.
2. Migration was aided by lower water contents and higher densities, although both in extreme proved detrimental.
3. Both low water content and high density were thought to promote capillary action.

Increases in strength of the clay were thought to be due to formation of complex cementing agents, with higher densities promoting interlocking and contact. Very high density was thought to inhibit water movement. In the case of the lime pile experiments shrinkage limit alone was studied. Results between experiments are difficult to compare due to differences in sampling distance and density of soil. However, it was clear that lime had migrated in excess of 75mm after 90 days.

Noble and Anday (1967) looked specifically at lime migration in soils in relation to lime piles used in the USA (see Handy and Williams, 1967). California Bearing Ratio (C.B.R.) moulds were used to house clay compacted to 95% maximum density. Holes of 25mm in diameter were drilled down the centre of the specimens and were filled with lime paste. The tops were plugged with clay and the samples were sealed. Migration was monitored by sampling away from the lime, leaching with acid and titrating with EDTA to determine the free calcium ion content.

It was noted that migration had reached a distance of 45mm, the limit of the experiment, in 44 days. At shorter distances lime concentration increased over 99 days. Some of the early data at certain particular distances indicated that migration had occurred, whilst later data at the same distance did not. The authors suggest that this is due to migration occurring in paths of lower density material, although no mention is made of the possibility of free calcium ions becoming unavailable due to cation exchange nor of the consistency of the measured data. In addition, no mention of the effect of soil type was made.

2.5 CONCLUSIONS

It is clear from the above discussion that the literature is very variable on the subject of lime piles. The pile construction technique varies as well as the type of lime used. Different authors measure and record different properties of both the soil prior to treatment and the soil and pile properties post-treatment. In laboratory experiments, boundary conditions vary considerably. In all papers, the level of detail is not sufficient to allow cross-analysis. It is clear that no coherent study of the stabilisation mechanisms associated with lime pile treatment has taken place and therefore no isolation of the different mechanisms has been achieved.

It is apparent that the major emphasis in the literature has been placed on the migration of lime away from the piles (at least with regard to their application to slope stability). However, no definite mechanism for migration has been established. The actual mechanism is probably a combination of ion diffusion and mass transport depending on water flow conditions. With respect to diffusion, a possible explanation for the short distances observed is that it is controlled by ion exchange. Ions may radiate away from the pile by a process of ion exchange, an equilibrium position being achieved when the potential for ion exchange is balanced by a reverse osmotic potential. The potential for ion exchange must, in turn, depend, at least to some degree, on mineralogy.

It is proposed in this thesis that the literature be discussed and summarised in conjunction with the potential experimental approaches available for appropriate research into the subject. This is done in the following chapter under the heading of research philosophy.

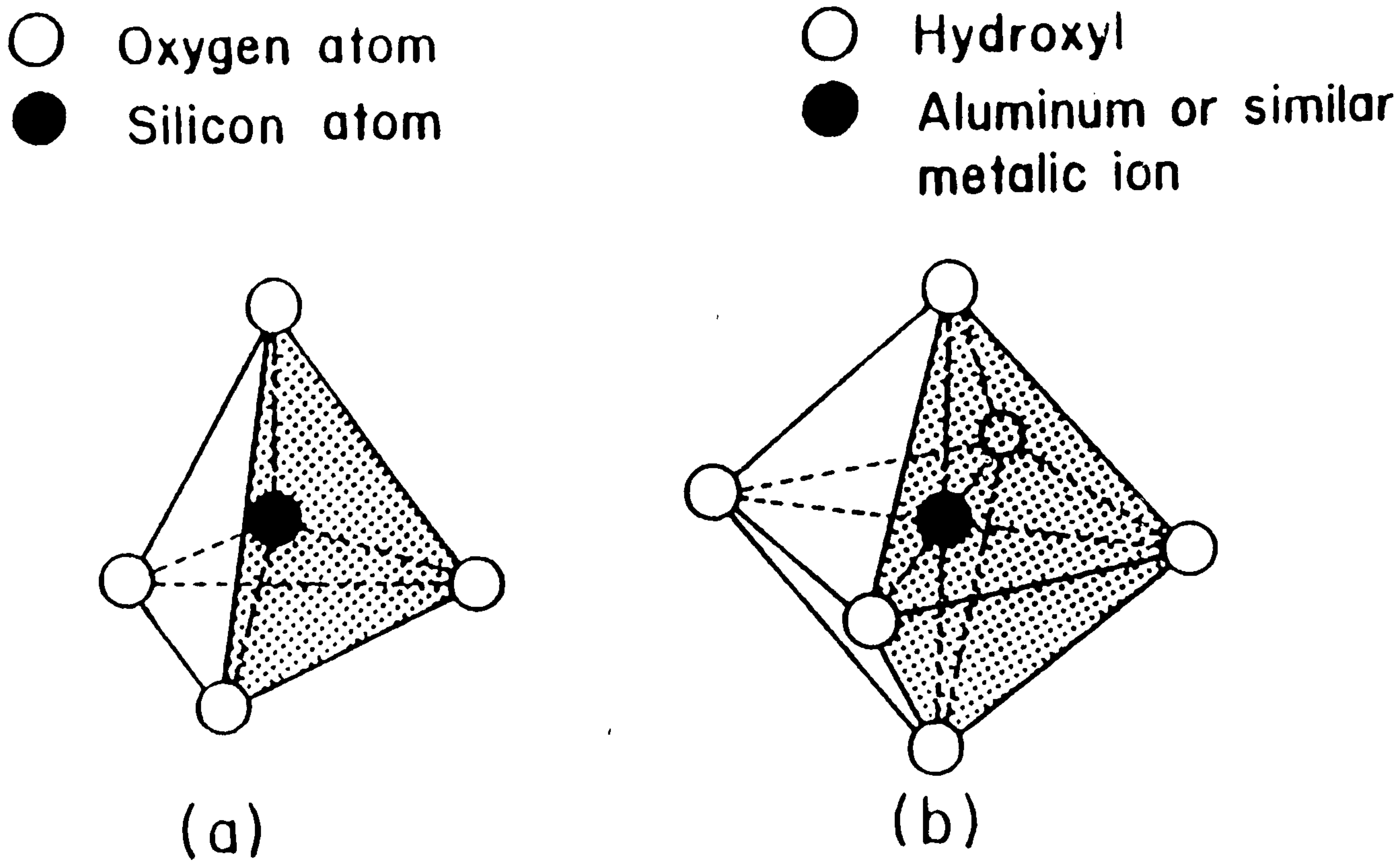


Figure 2.1 Two of the most common building blocks of soils: (a) silica tetrahedron and (b) aluminium octahedron

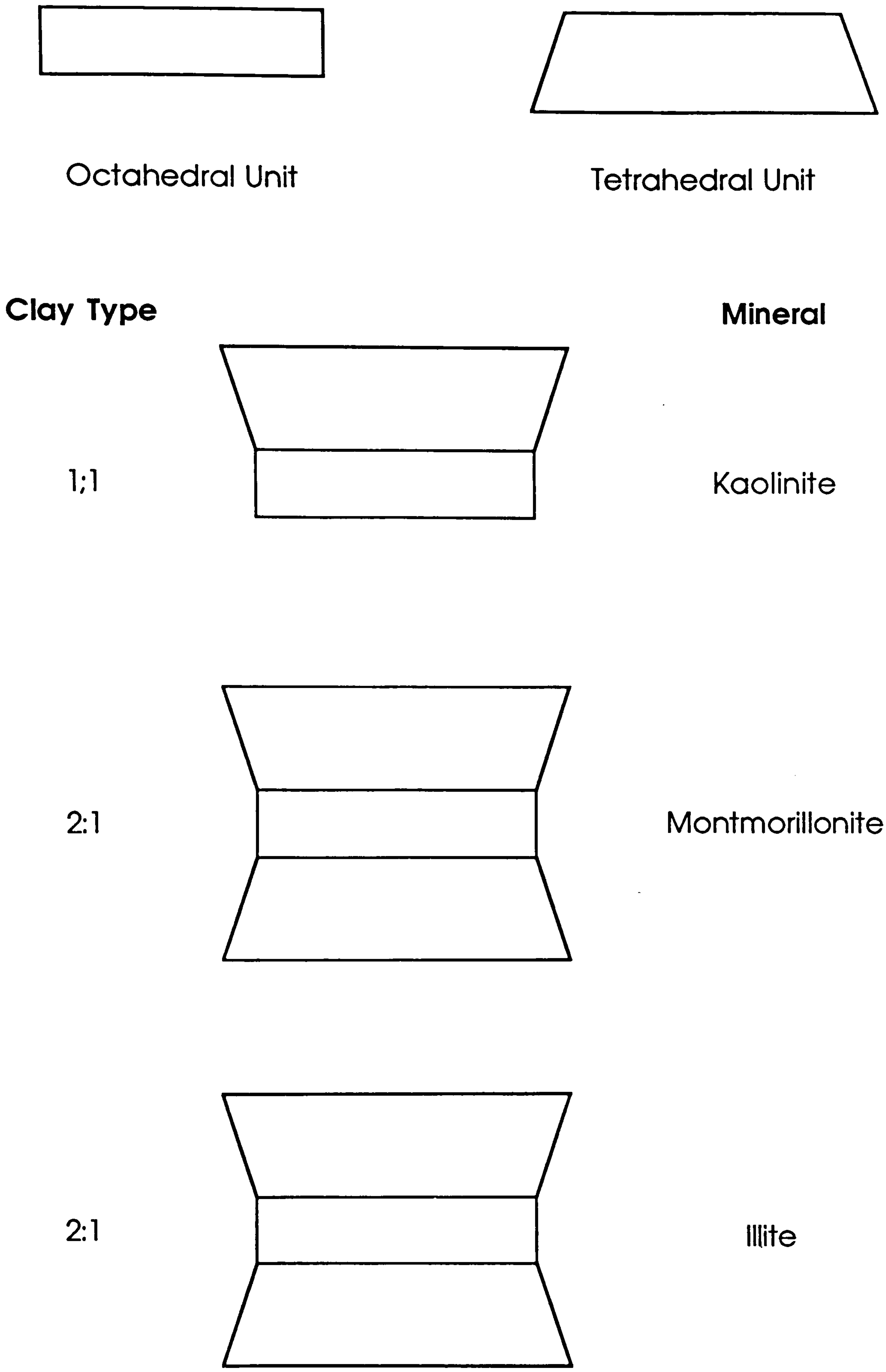


Figure 2.2 Clay Structures

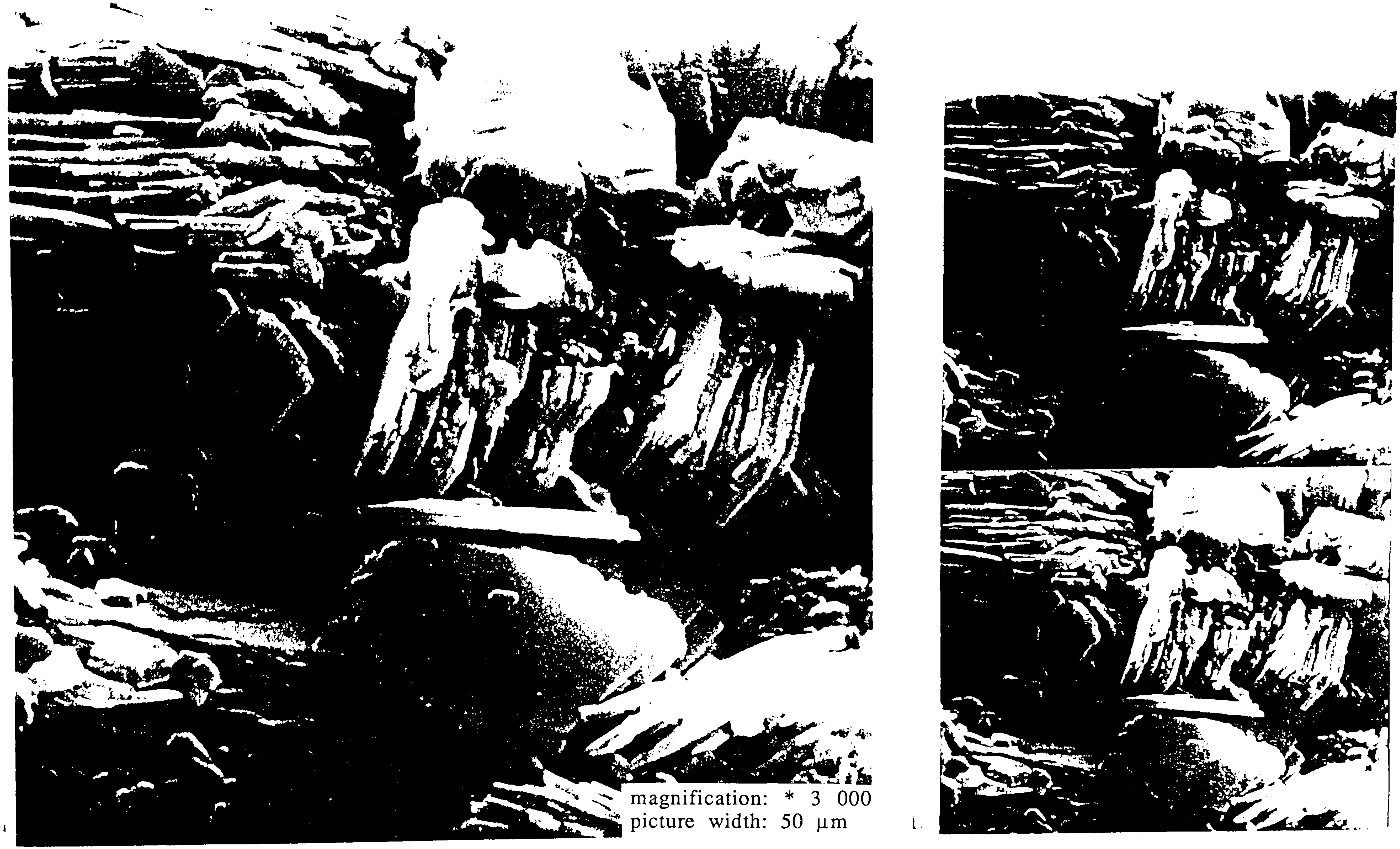


Figure 2.3 Scanning Electron Micrograph of Undisturbed Kaolin.
(After Tovey, 1971)

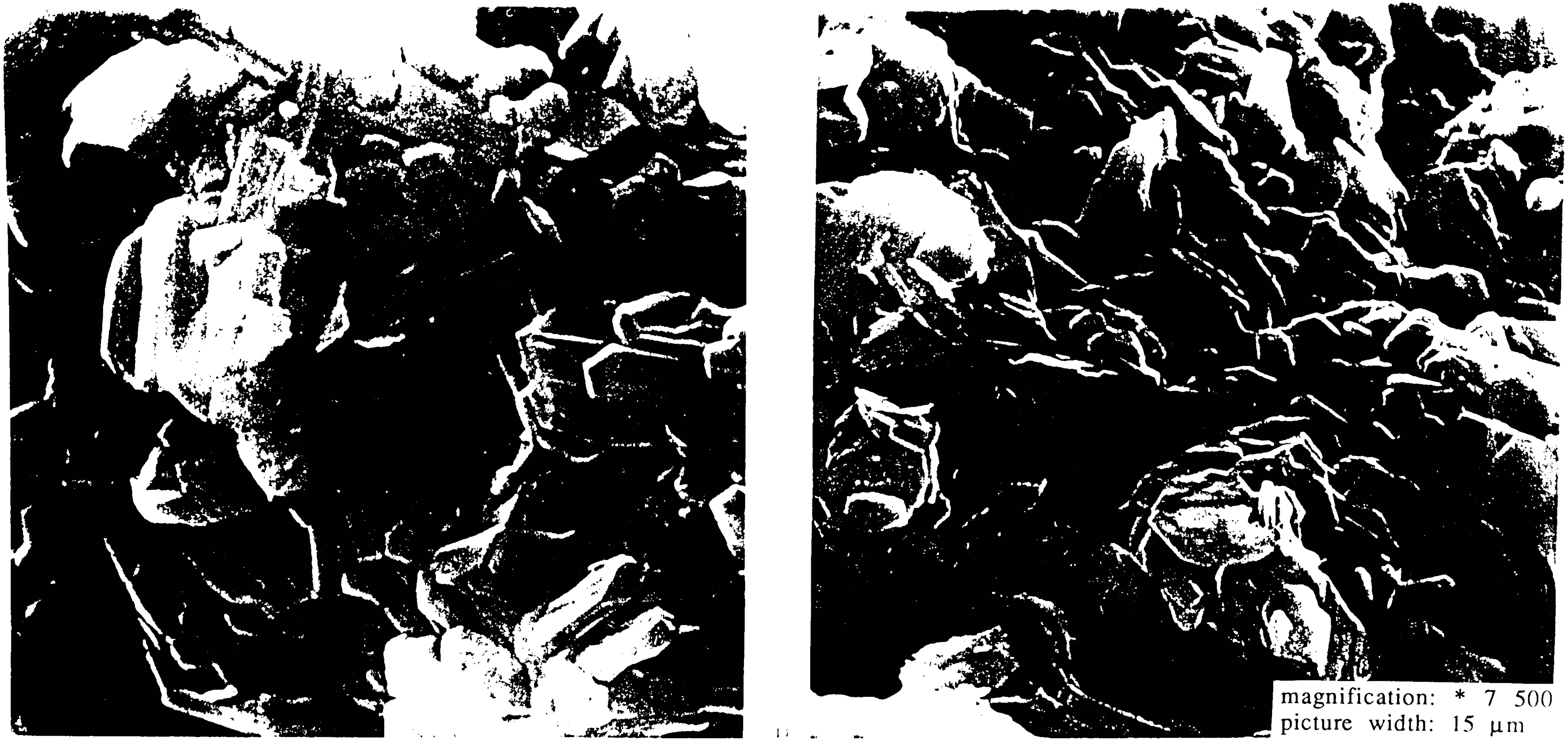


Figure 2.4 Scanning Electron Micrograph of Illite.
(After Tovey, 1971)

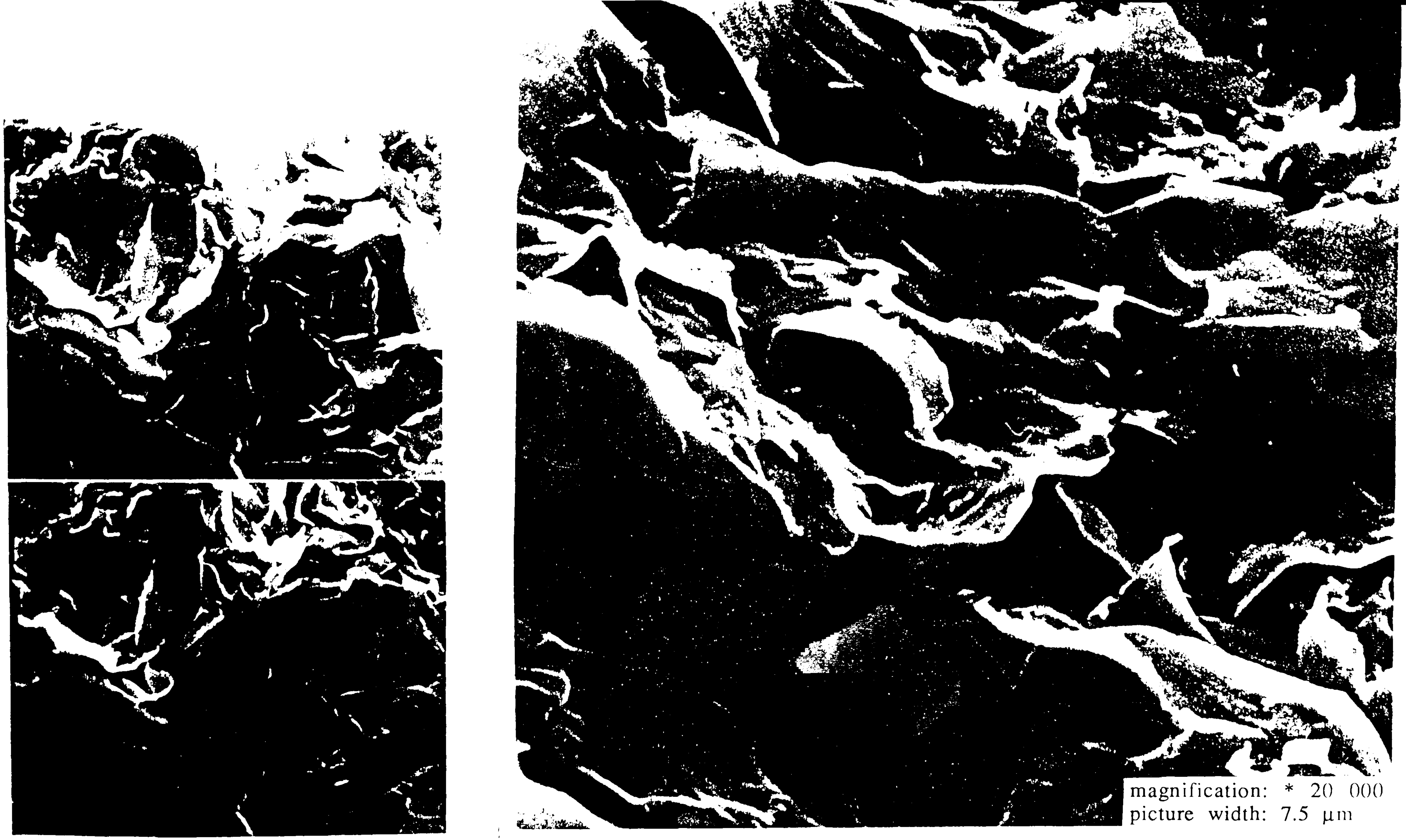


Figure 2.5 Scanning Electron Micrograph of Wyoming Montmorillonite(Bentonite) (After Tovey, 1971)

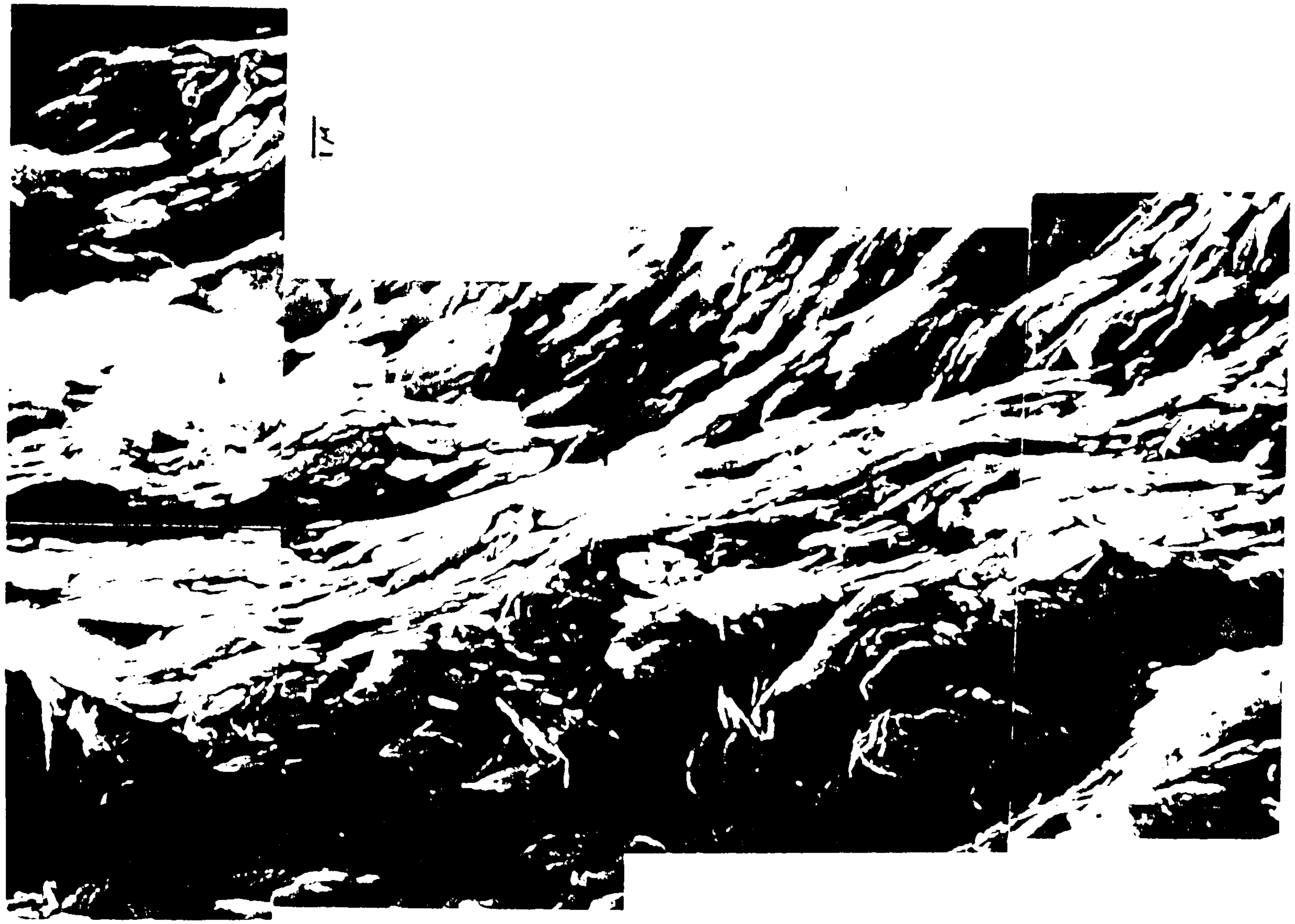


Figure 2.6 Scanning Electron Micrograph of a Failure Zone in Remoulded Kaolin. (After Tovey, 1971)

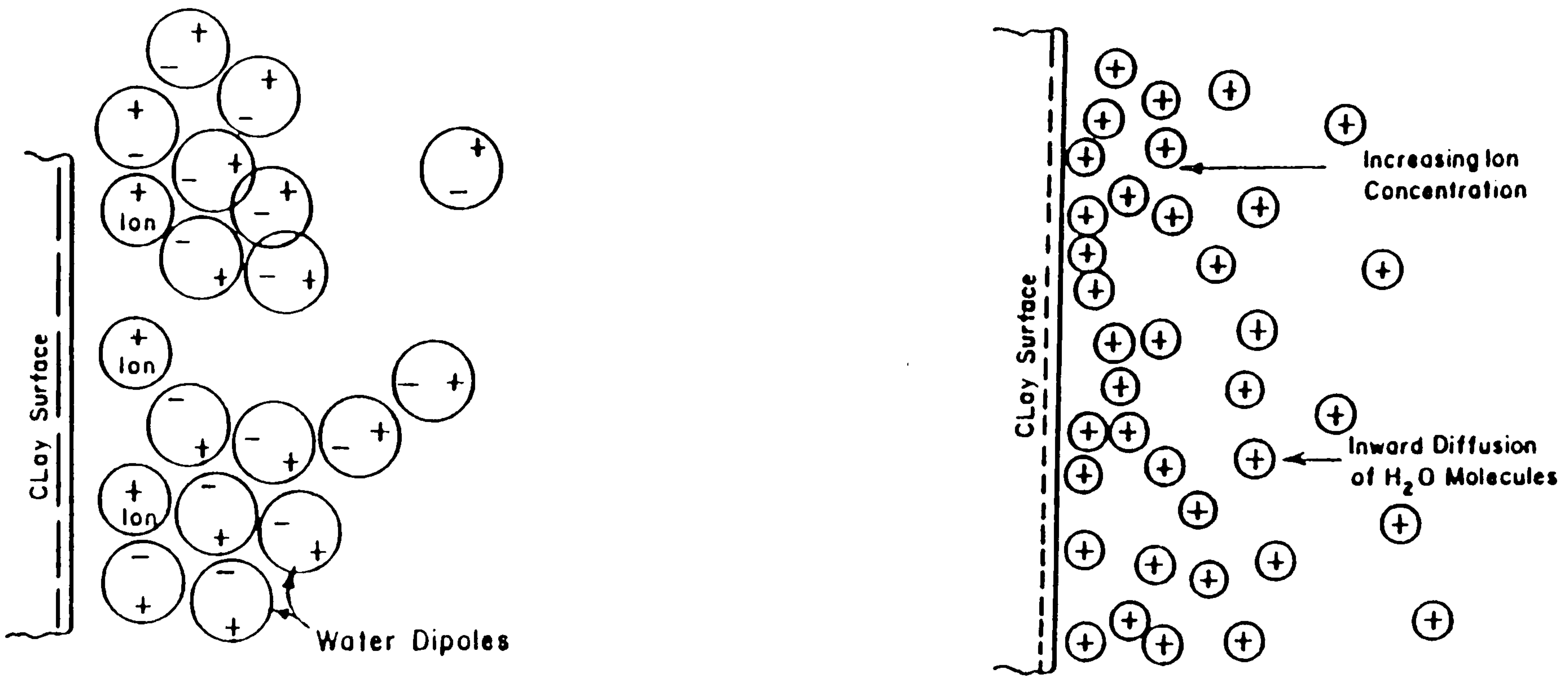


Figure 2.7 Cations and Water (a dipolar molecule) are attracted to the negatively charged clay surface to satisfy the charge potential. This results in (a) adsorbed cations and water molecules and (b) a diffused layer of cations due to their thermal activity and the infusion of water toward the clay surface because of the high electrolyte concentration (after Mitchell, 1976)

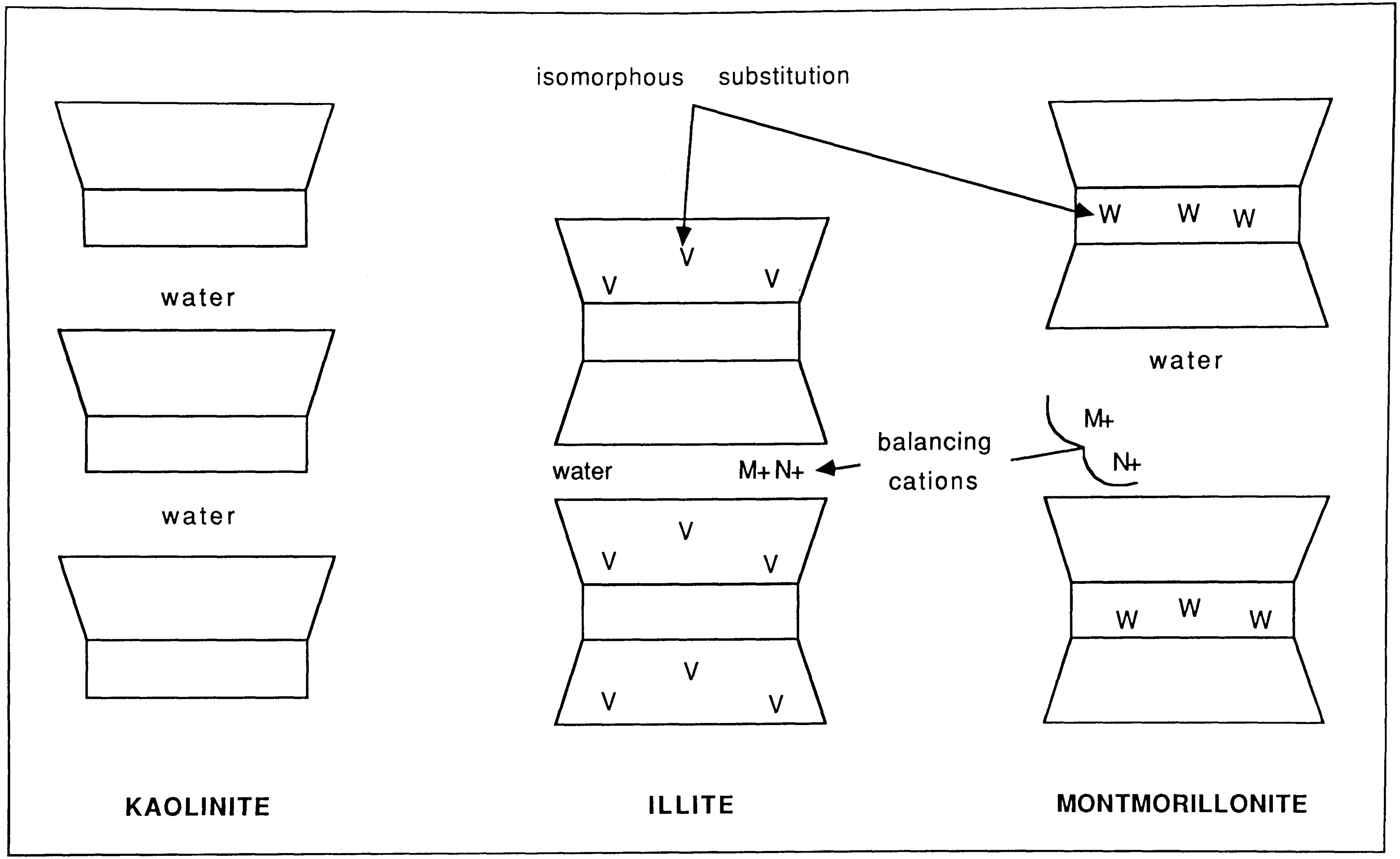
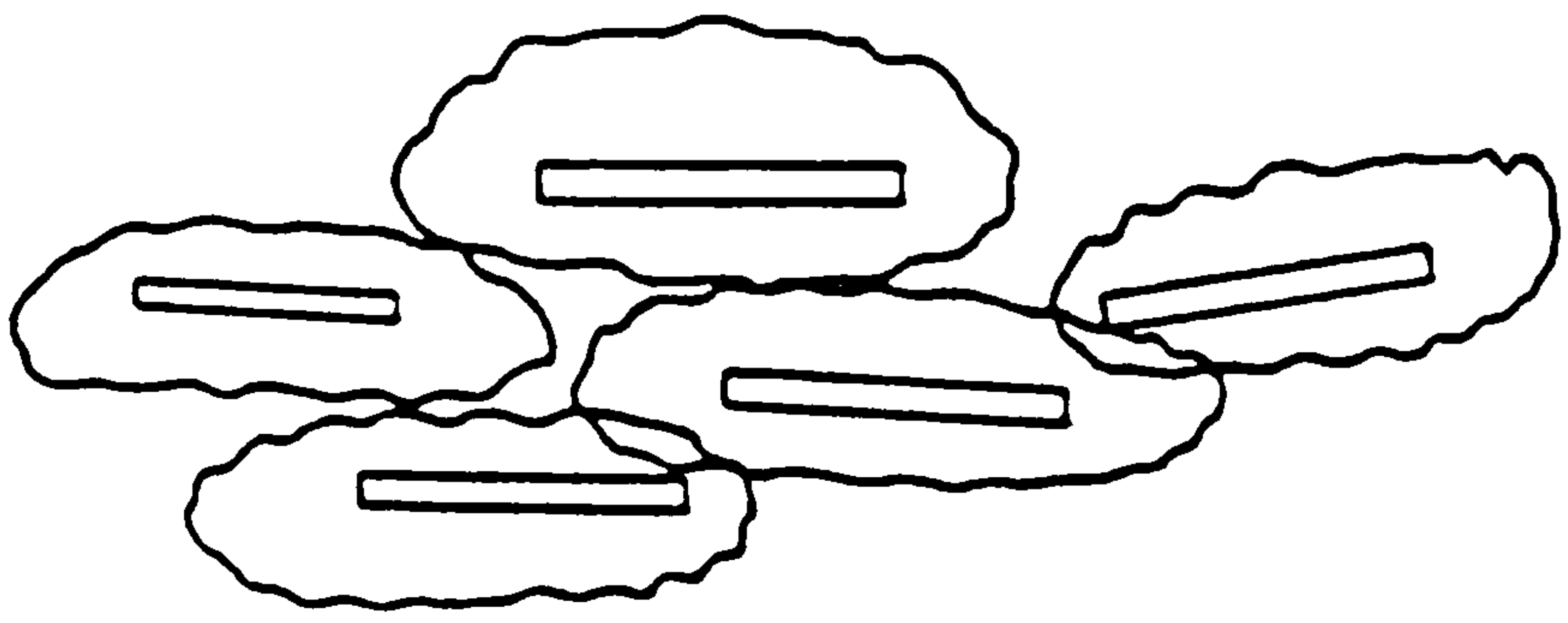
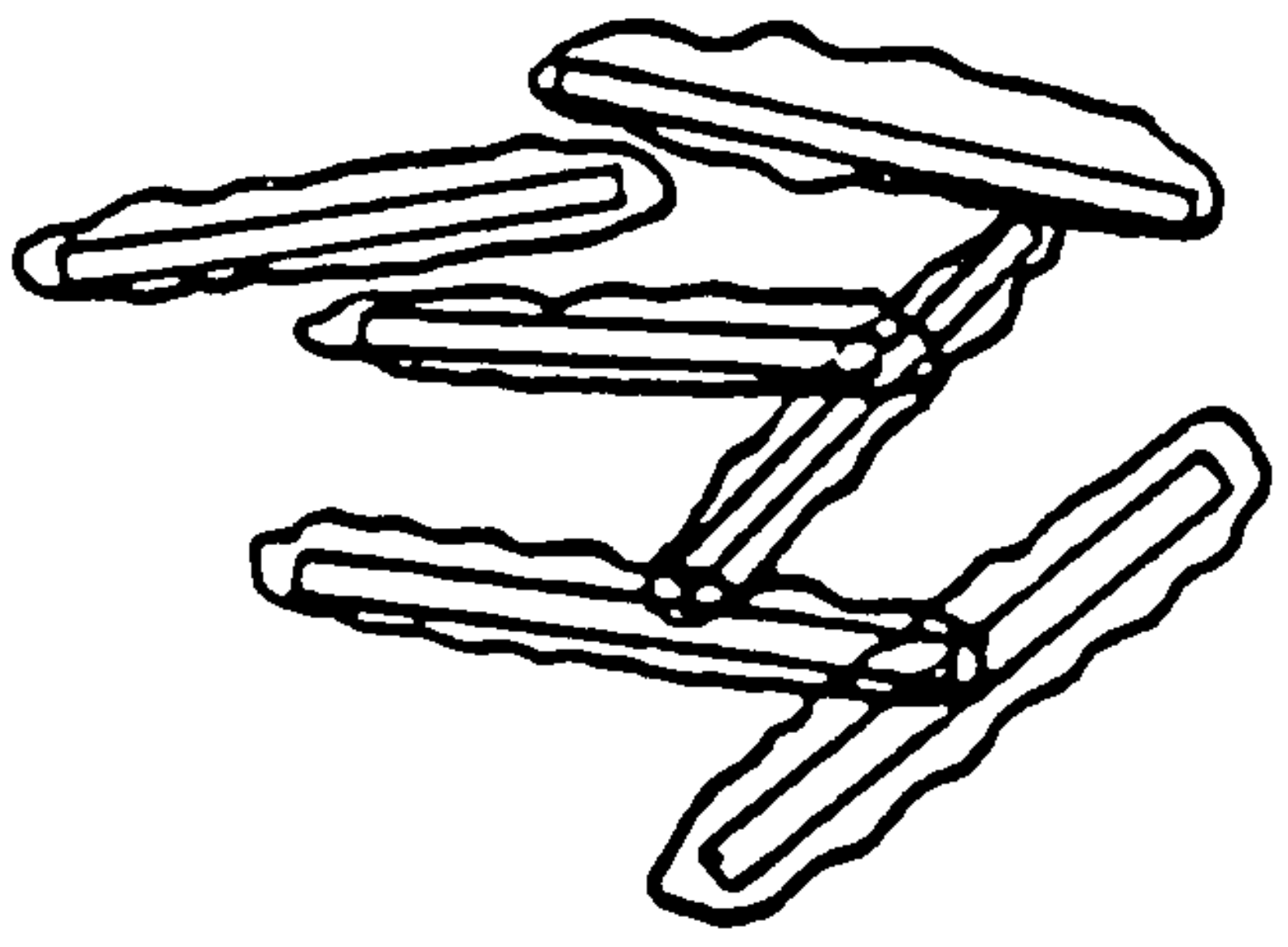


Figure 2.8 Clay-water systems



(a) Parallel arrangement of clay particles with hydrated water layers.



(b) Edge-to-face attraction induced by thin water layer which allows attractive forces to dominate.

The reason for the textural change is due to the phenomenon of cation exchange followed by flocculation and agglomeration. The above Figure (a) illustrates low strength clay soil where particles are separated by large water layers. The addition of lime (calcium) shrinks the water layer (b) allowing the plate-like particles to flocculate.

Figure 2.9 Results of Cation Exchange

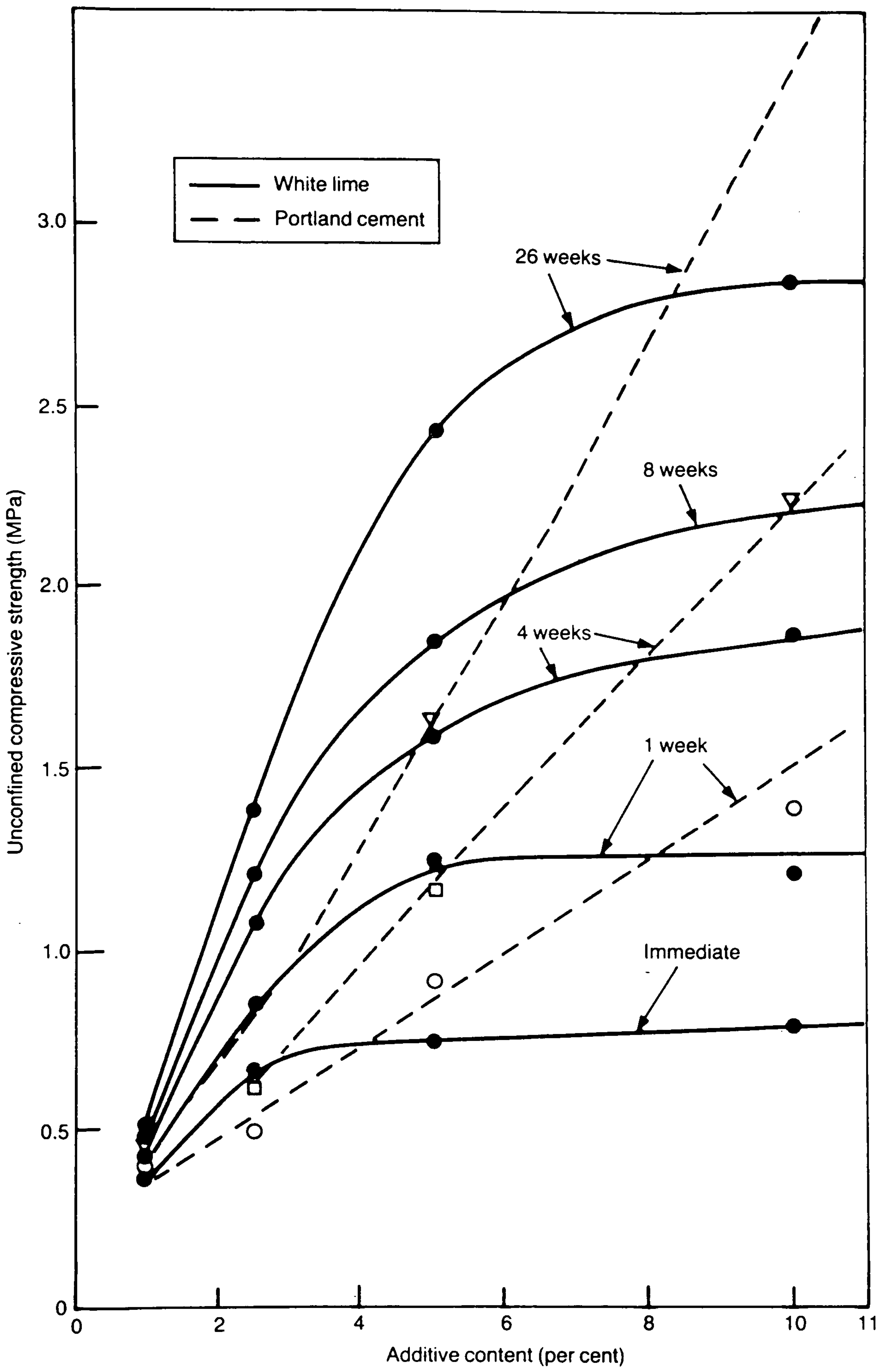


Figure 2.10 Comparison of strength gain derived from Lime and Portland Cement (after Dumbleton, 1962)

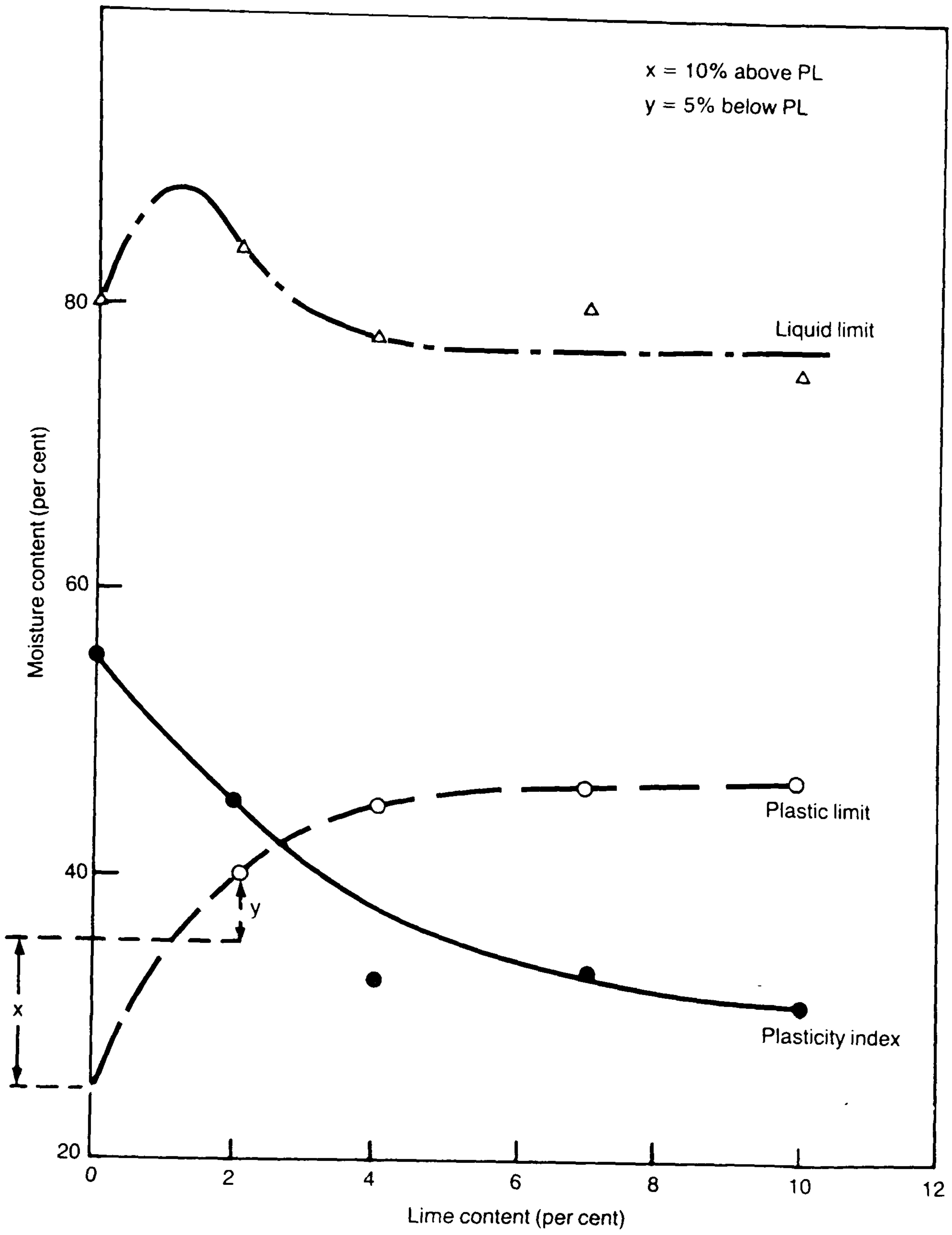


Figure 2.11 Effect of the addition of lime on the plasticity properties of London Clay (after Sherwood, 1991)

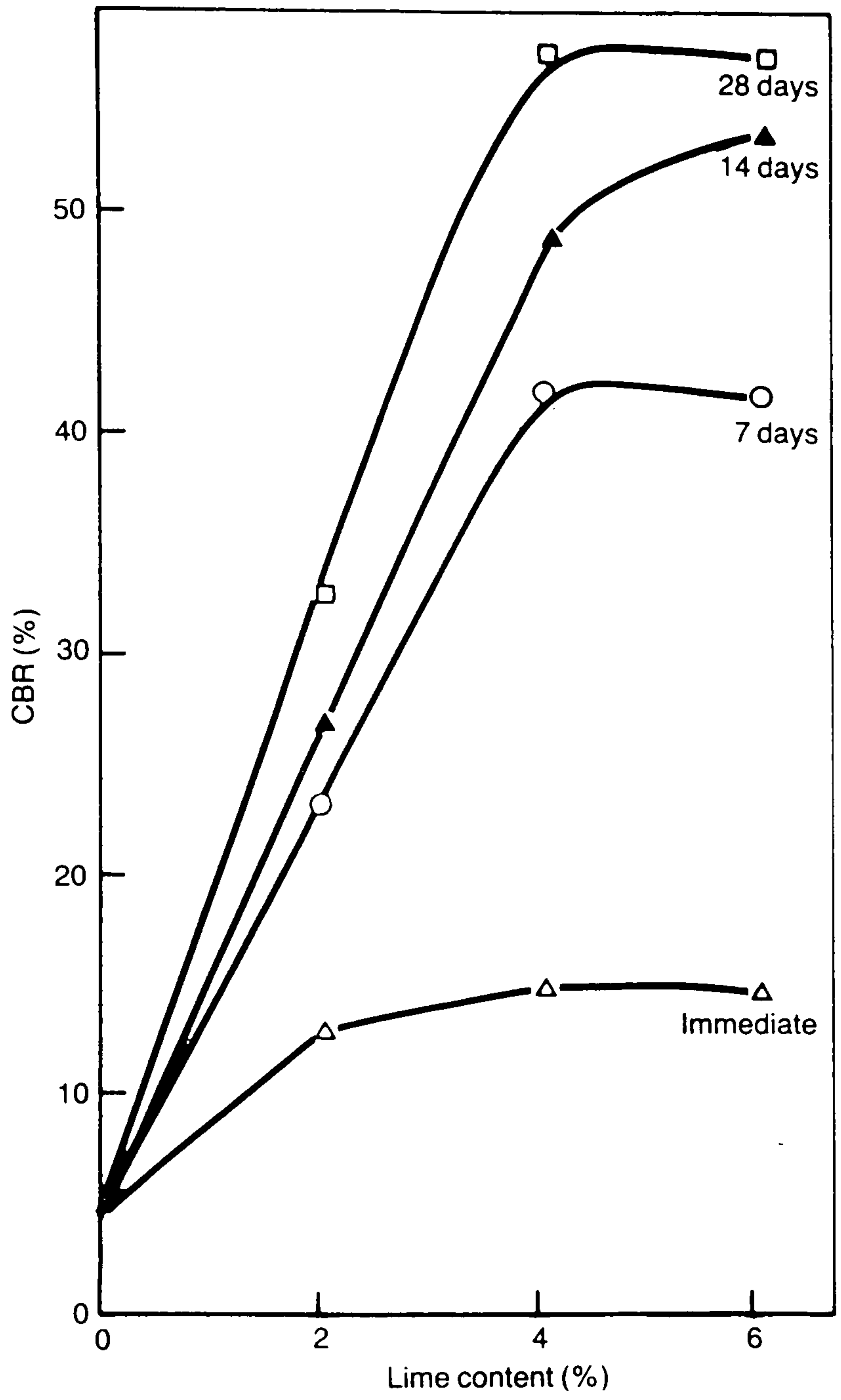


Figure 2.12 Effect of lime content and time on the CBR value of a lime stabilised soil (Littleton et al, 1988)

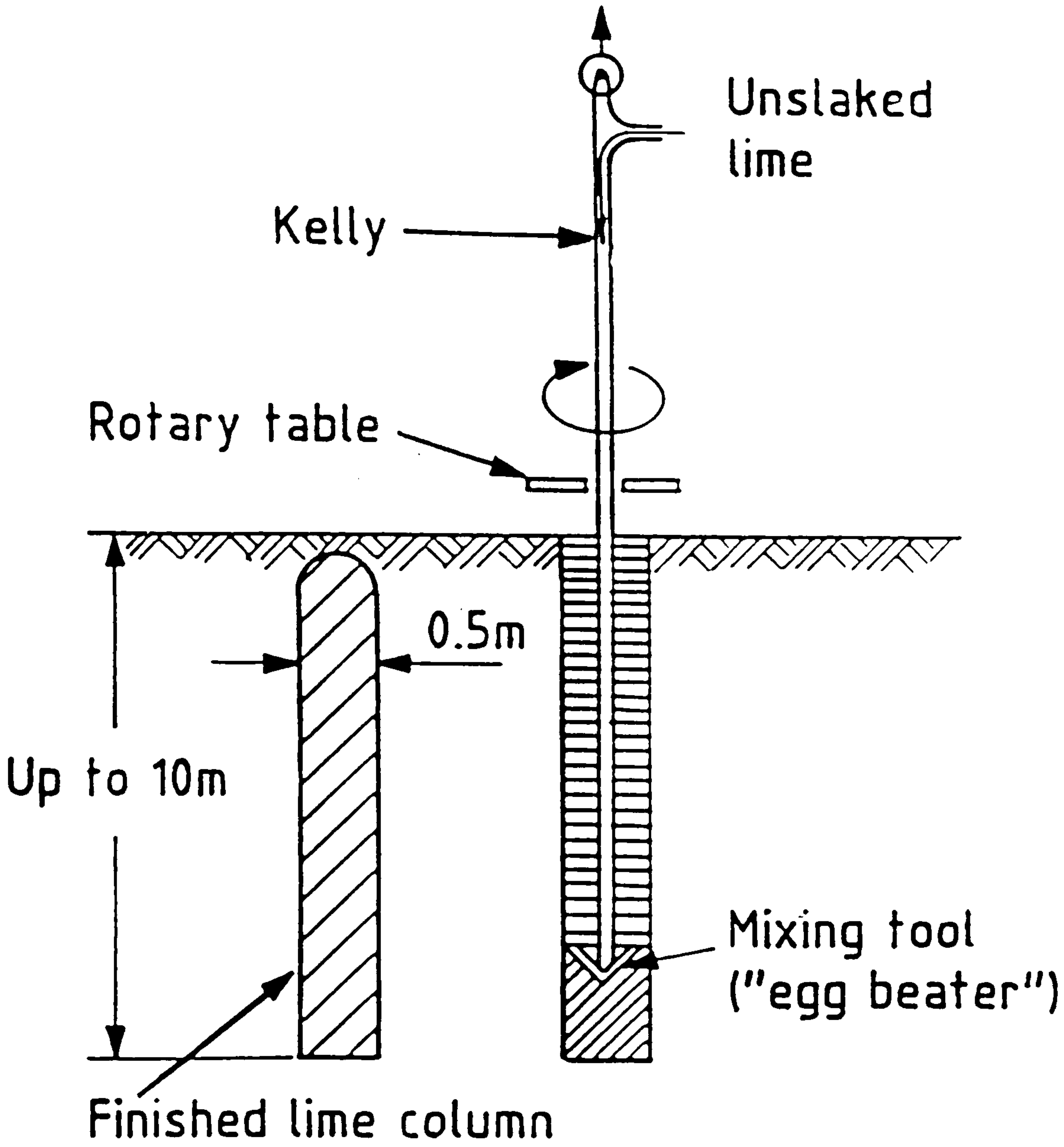


Figure 2.13 Procedure for construction of lime columns (after Broms and Boman, 1979)

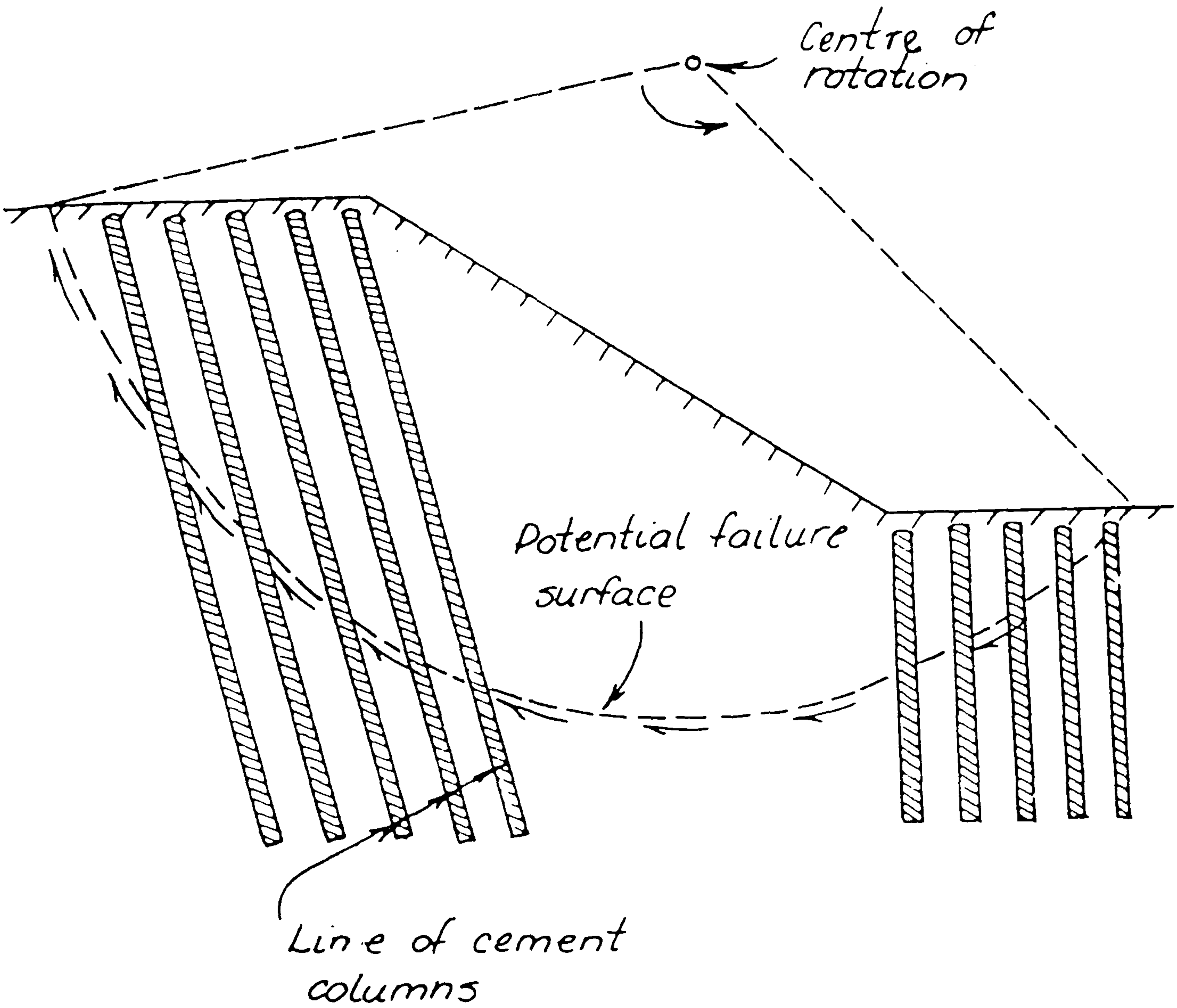


Figure 2.14 Stabilisation of slopes with lime columns (after Broms, 1985)

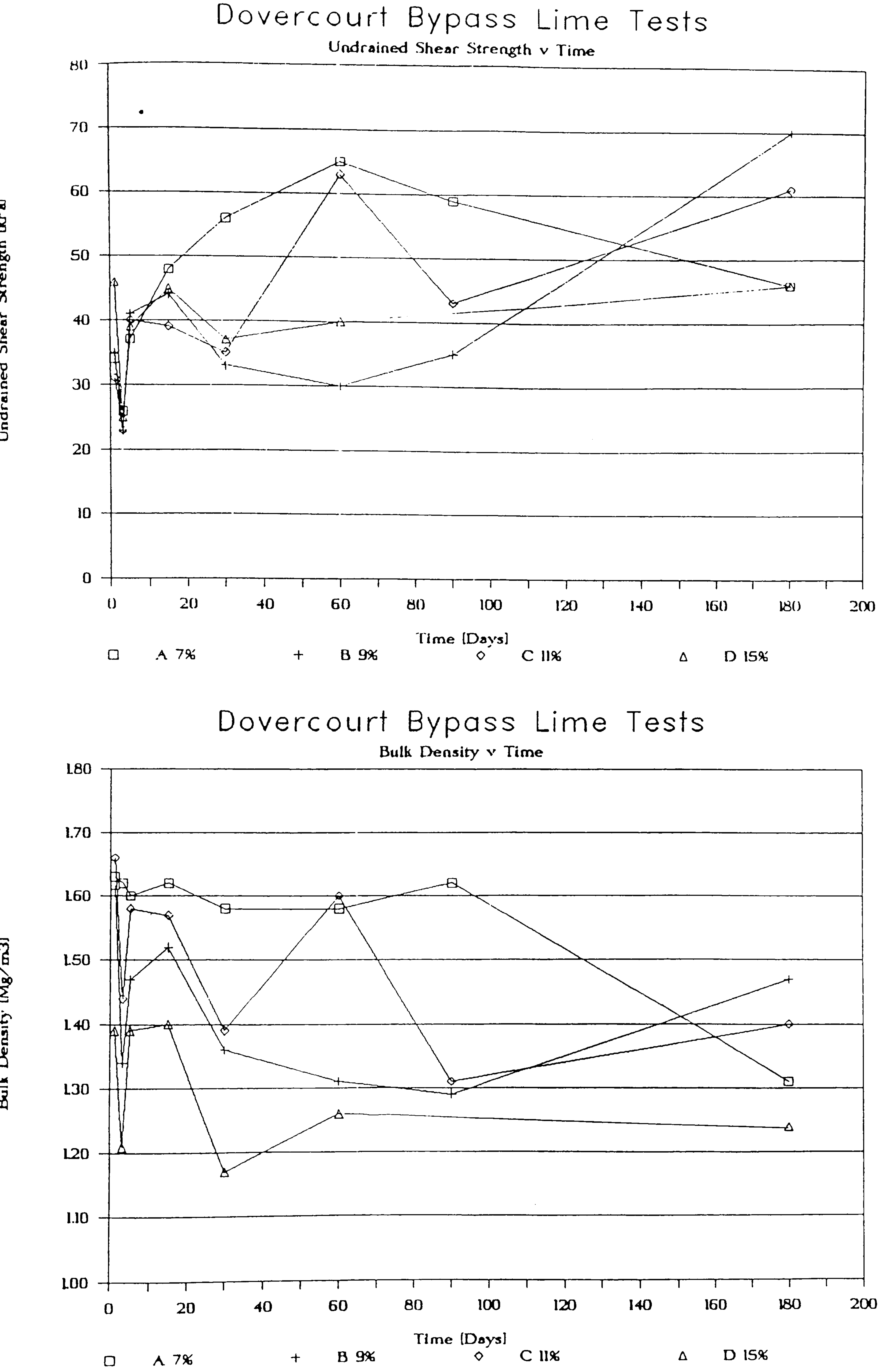


Figure 2.15 Results of testing by Corbet, 1988

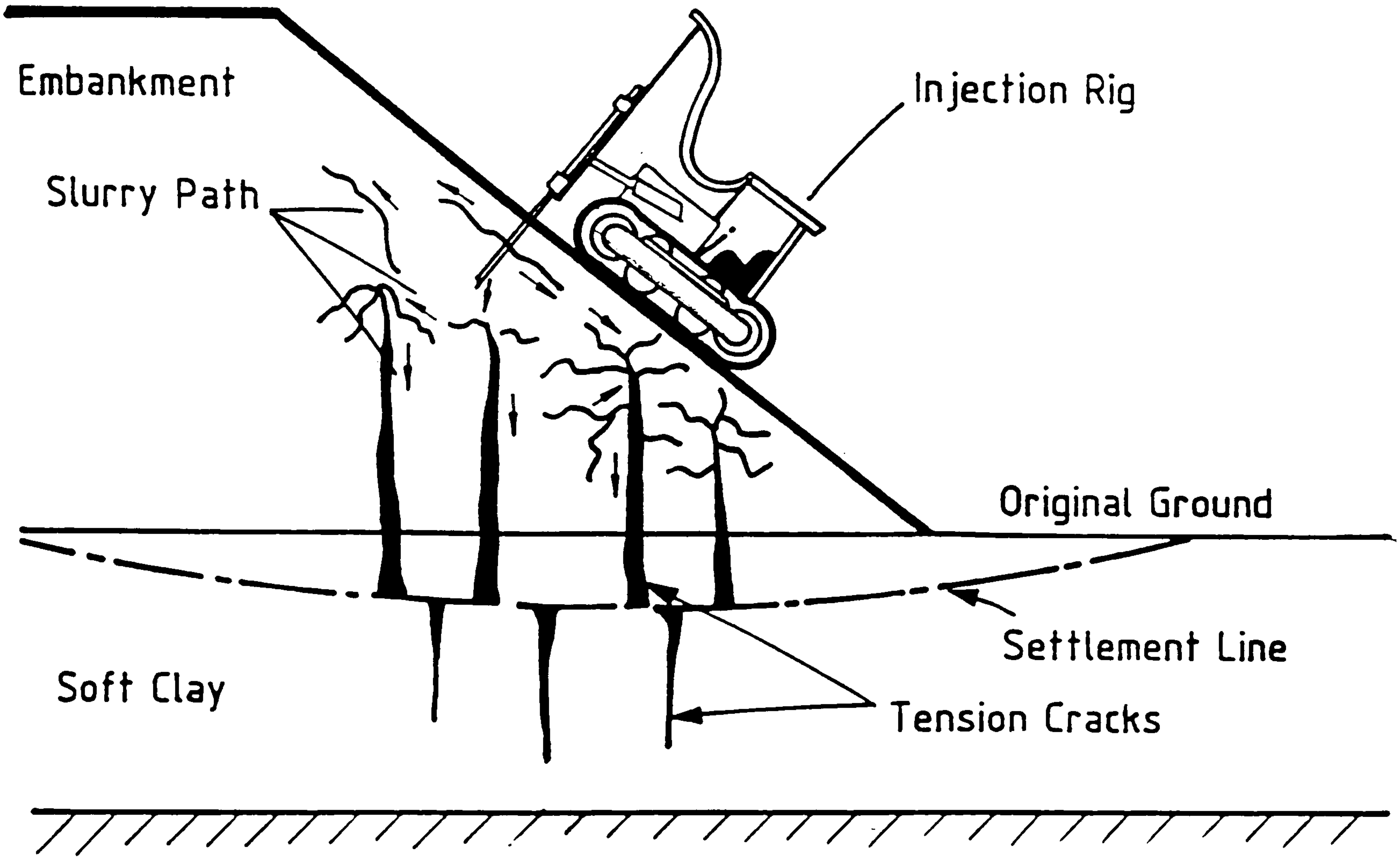
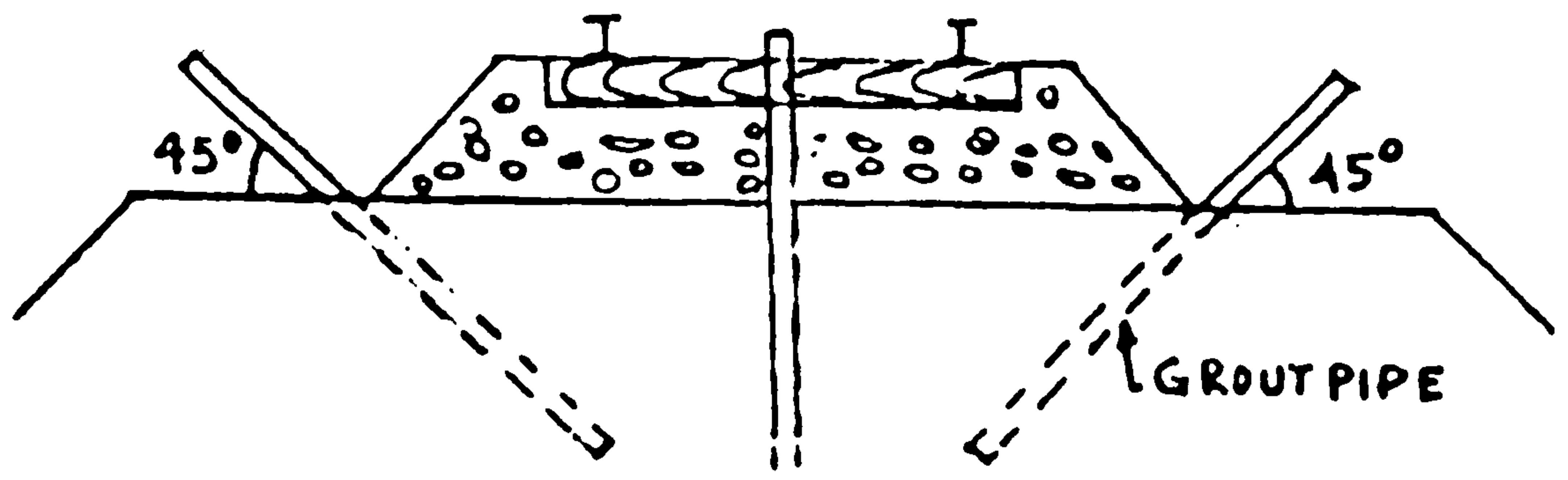


Figure 2.16 Slope Stabilisation using lime slurry pressure injection (after US National Lime Association, 1985)



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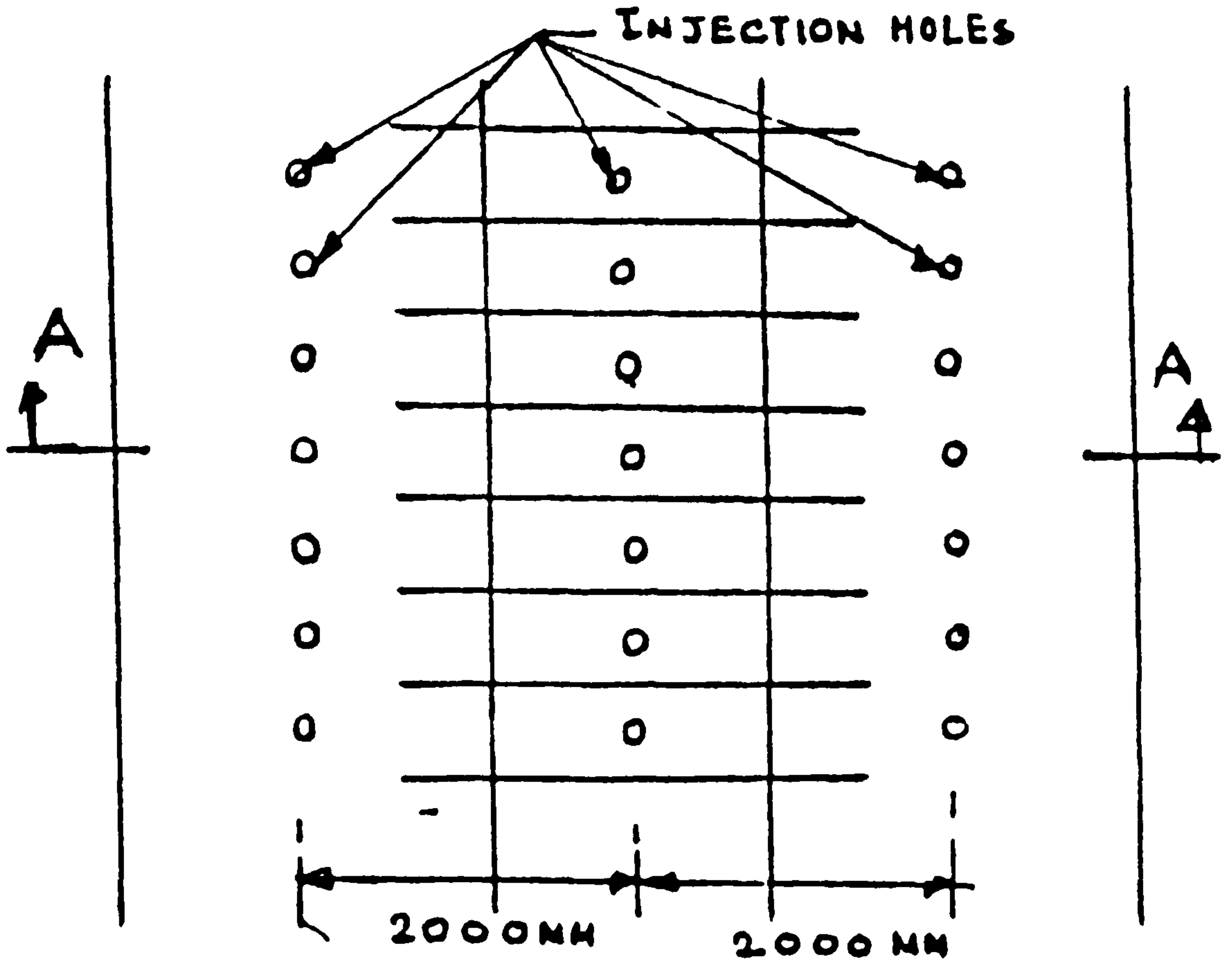


SKETCH SHOWING LAYOUT OF INJECTION HOLES.

Figure 2.17 Layout of injection holes (after Bhattacharya and Bhattacharya, 1989)

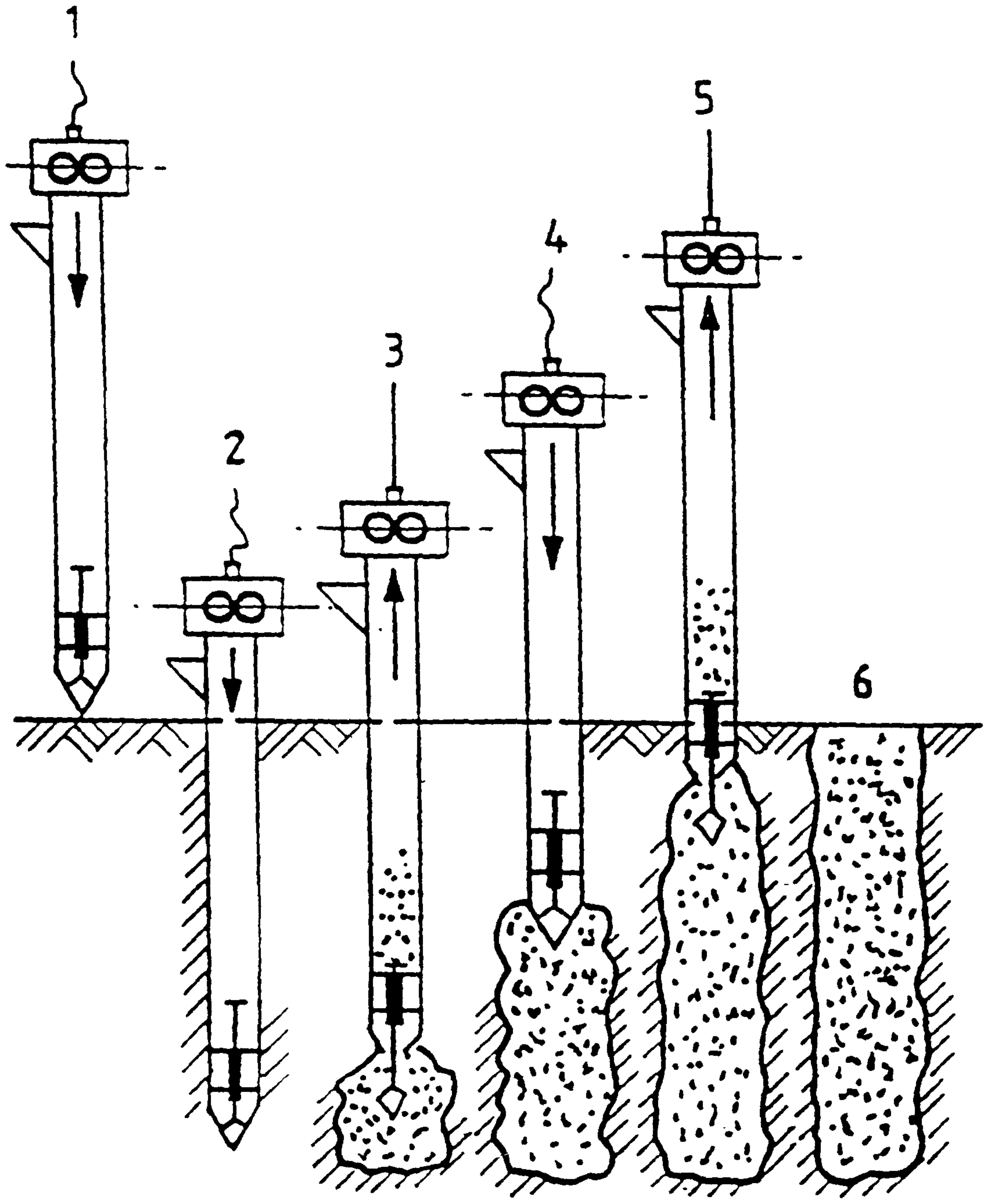


Figure 2.18 Procedure for construction of lime piles in soft soils (after Ingles and Metcalf, 1972)

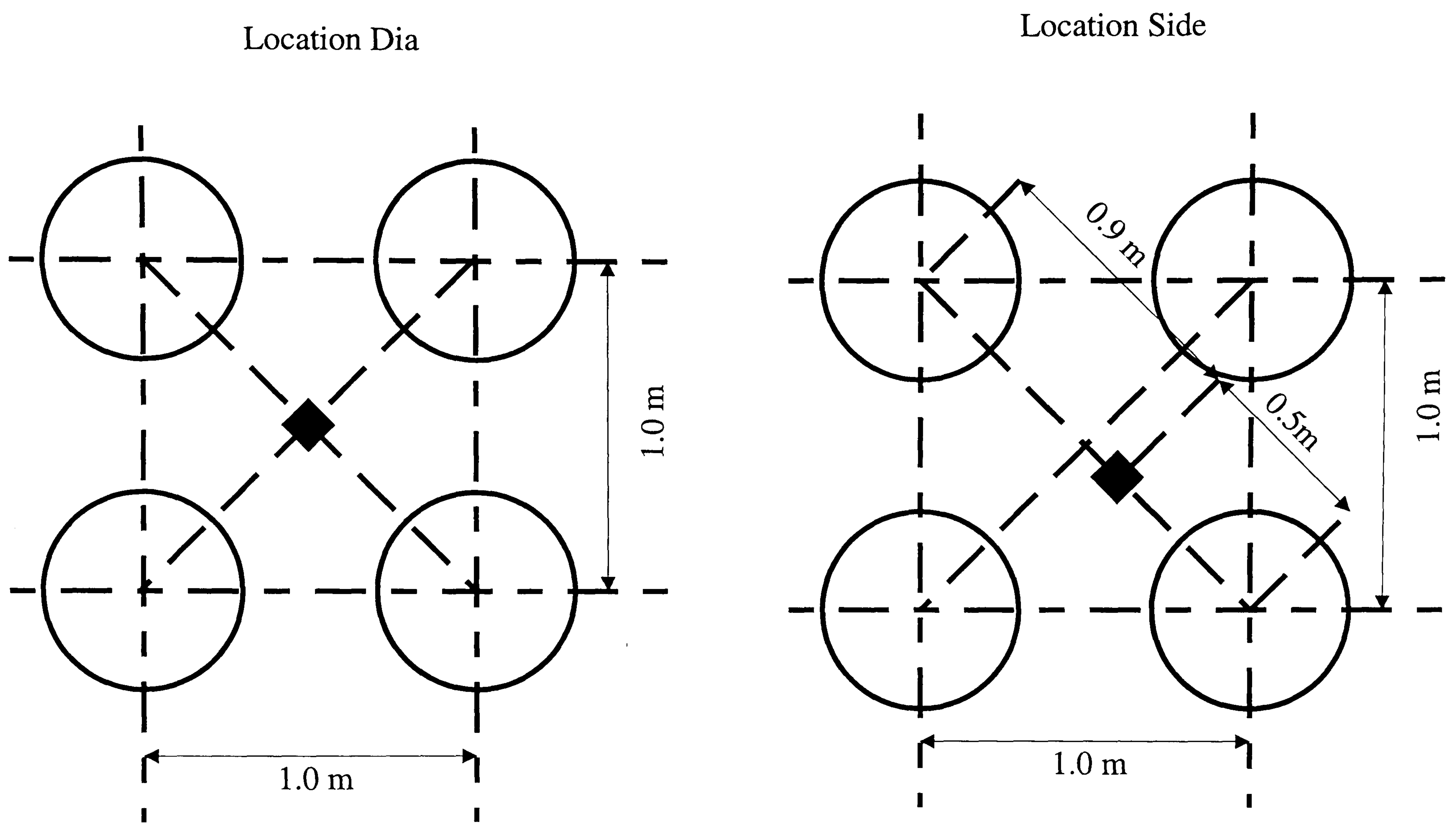


Figure 2.19 Sample locations relative to piles (Chew et al, 1993)

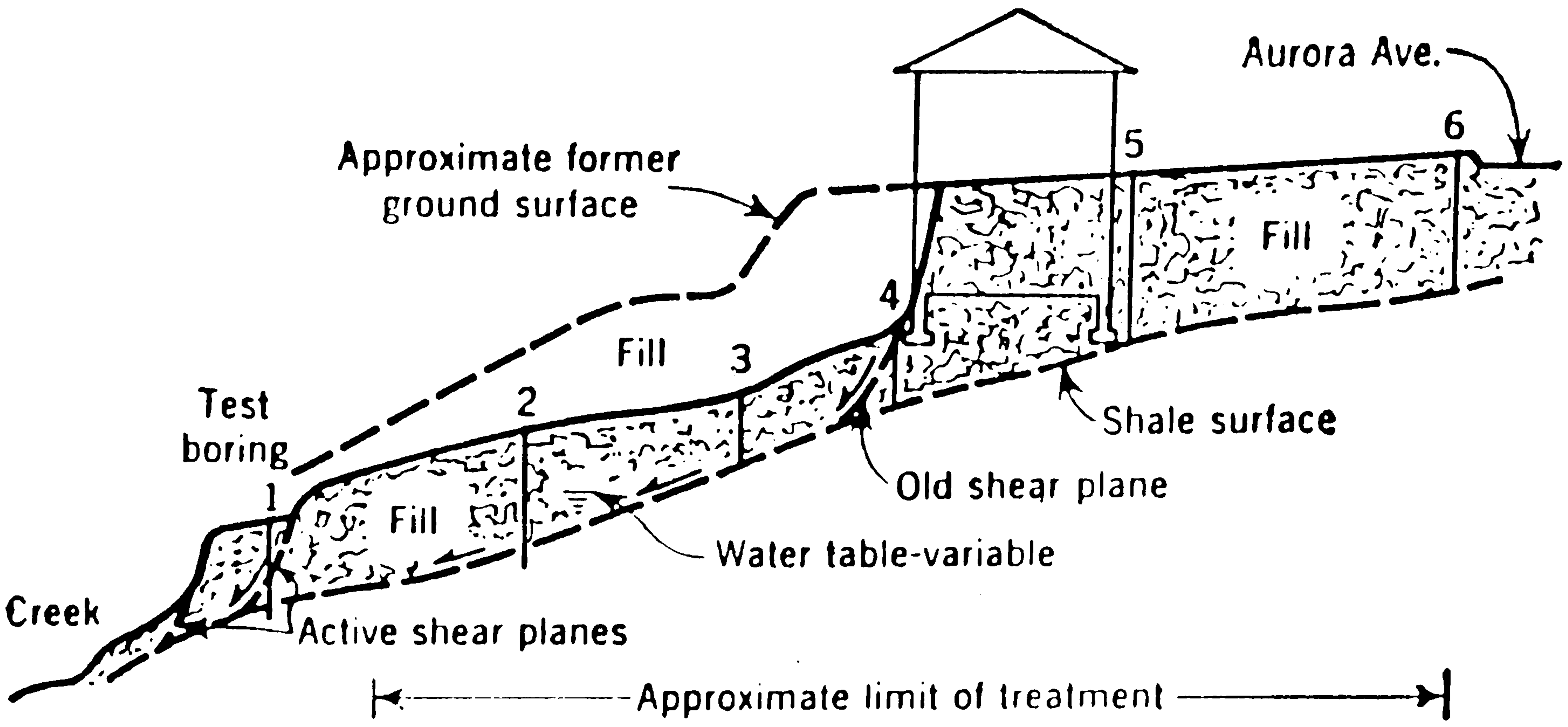
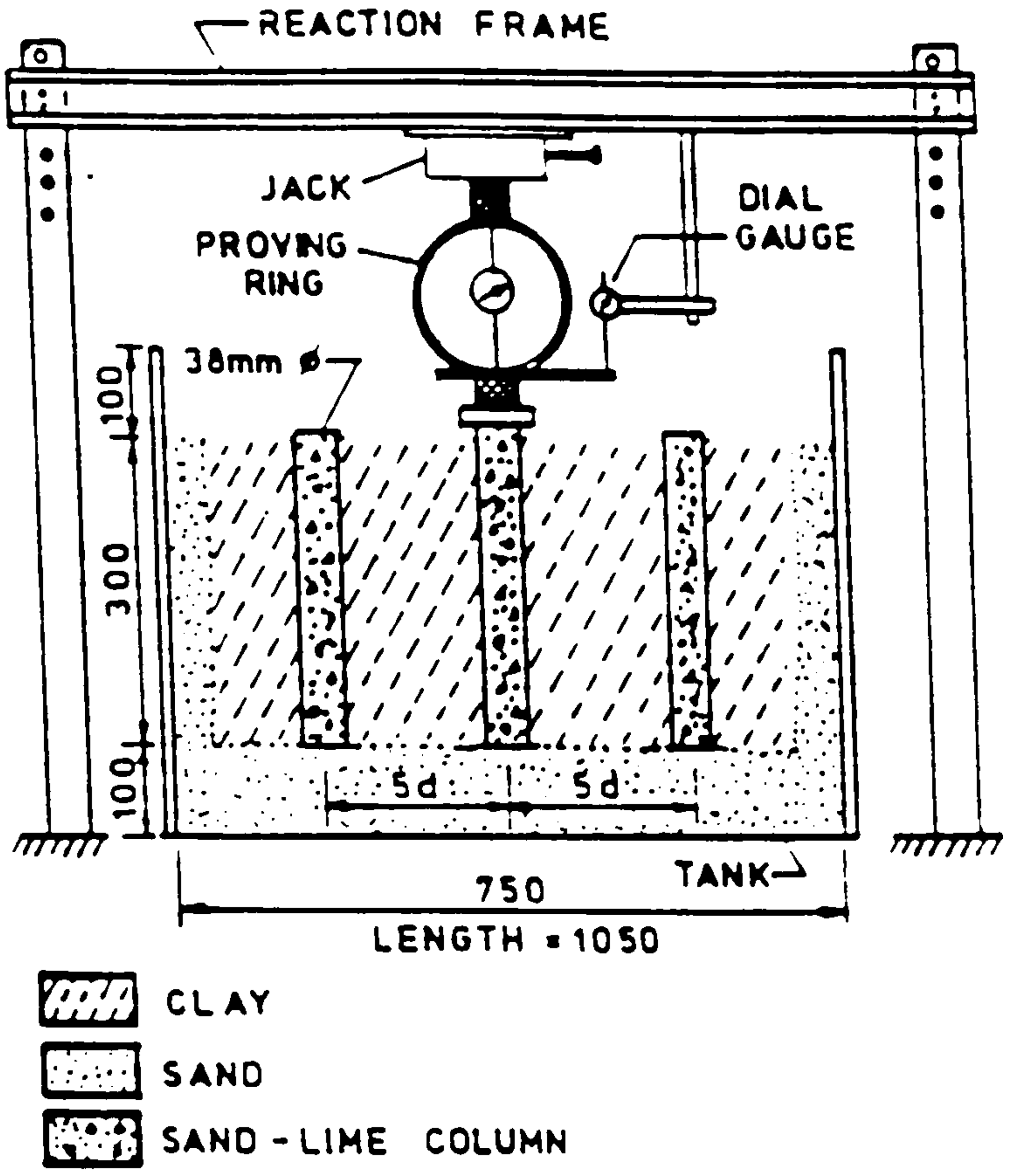
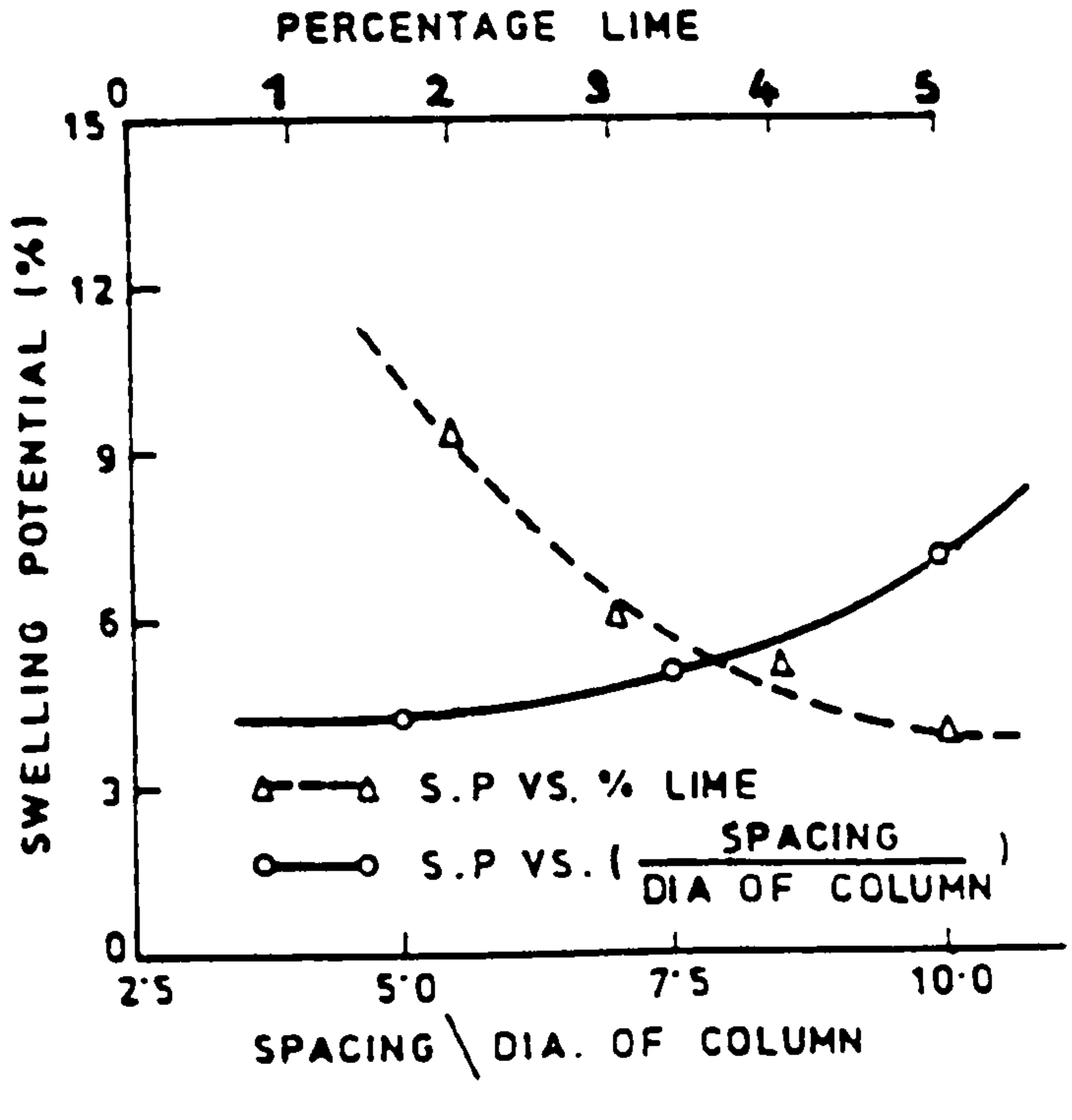


Figure 2.20 The use of lime piles in slope stabilisation in Iowa after Handy and Williams, 1976





Looking at spacin
ie a column spacin
approx. lime % =

area of cols/mix
area of tank

with sand: lime
= 38 × 300 × 3
750 × 300 × 3

From graph, swe
≡
swell pot @ spac

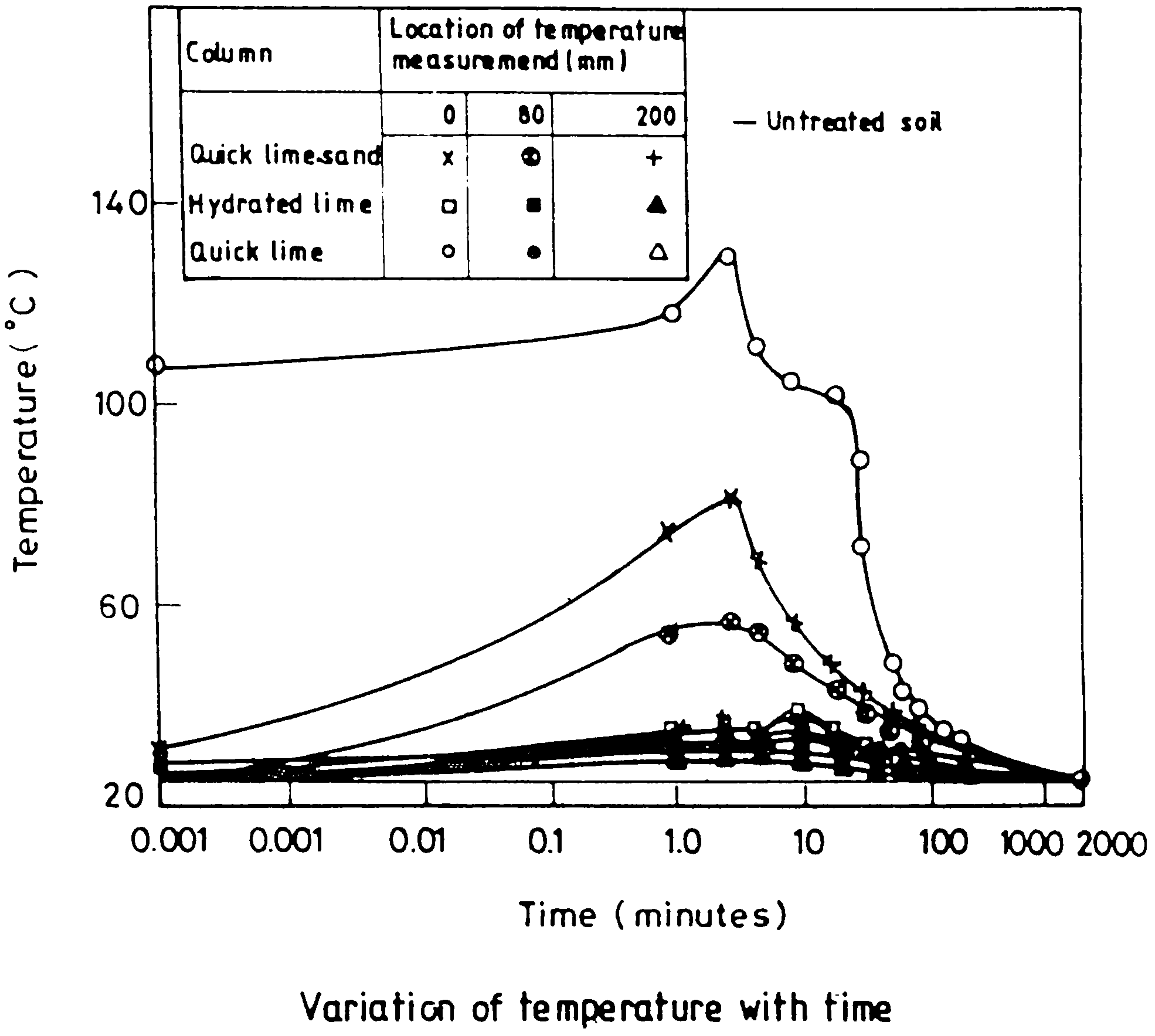
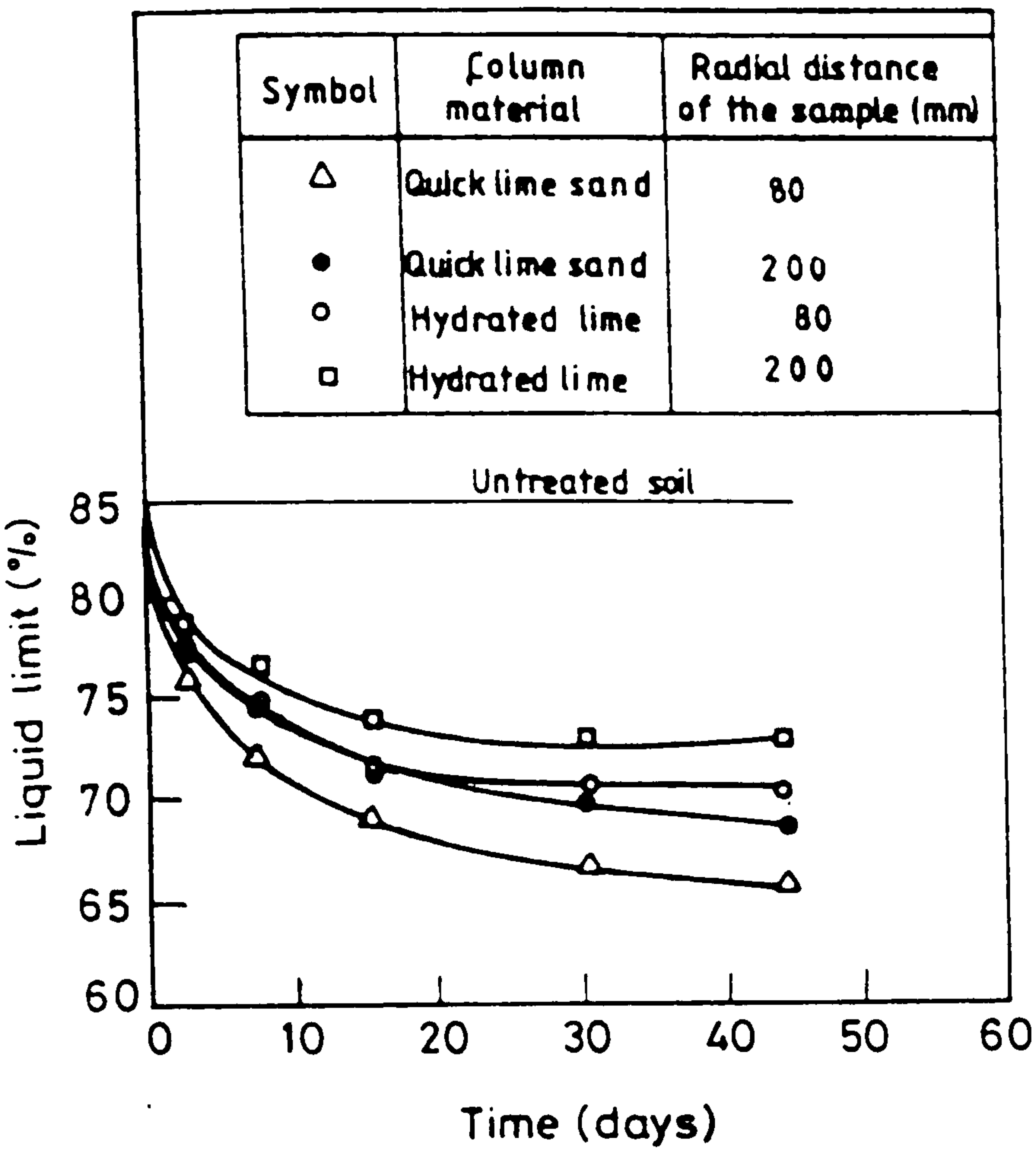
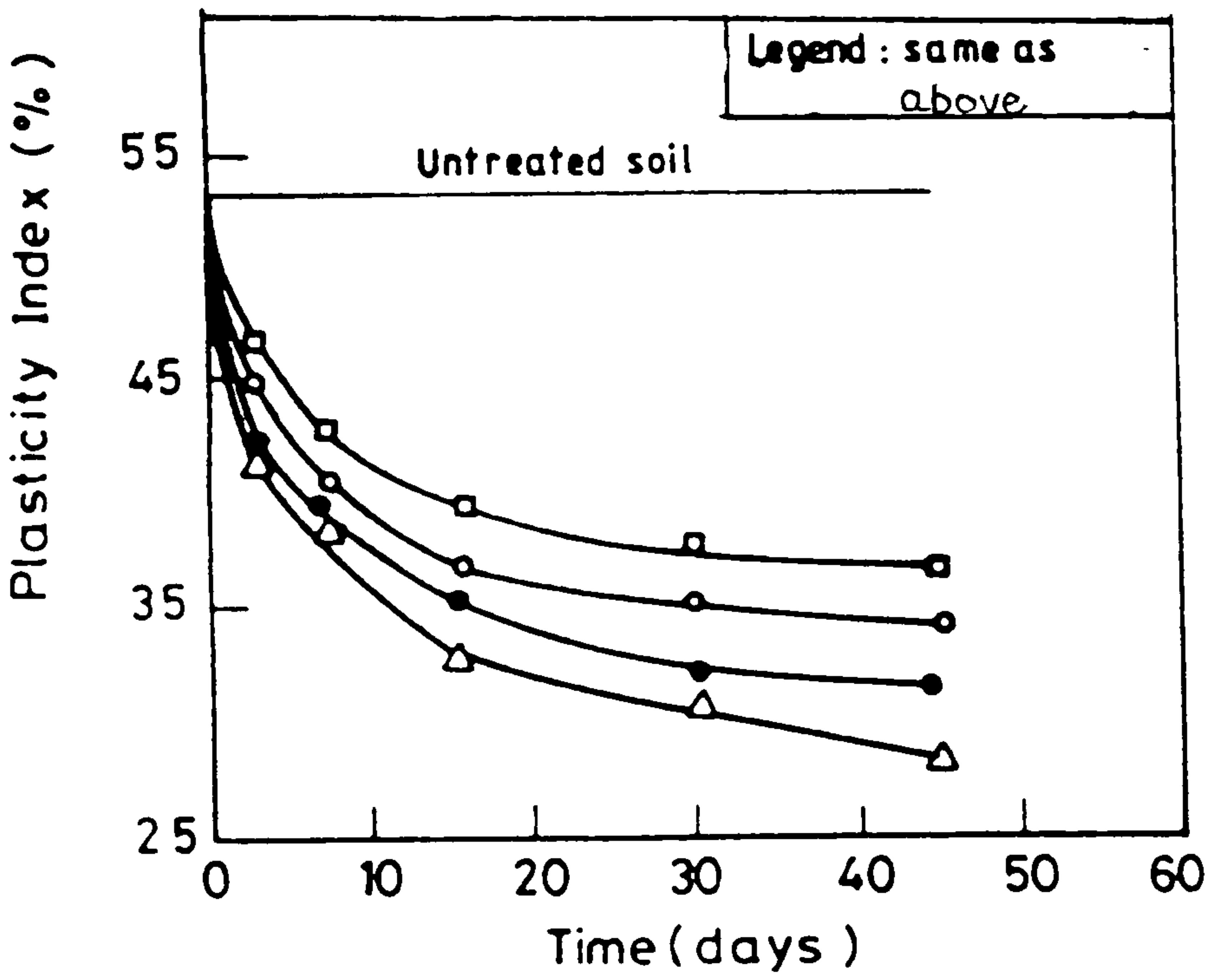


Figure 2.22 Temperature change due to lime piles (after Rao et al, 1993)

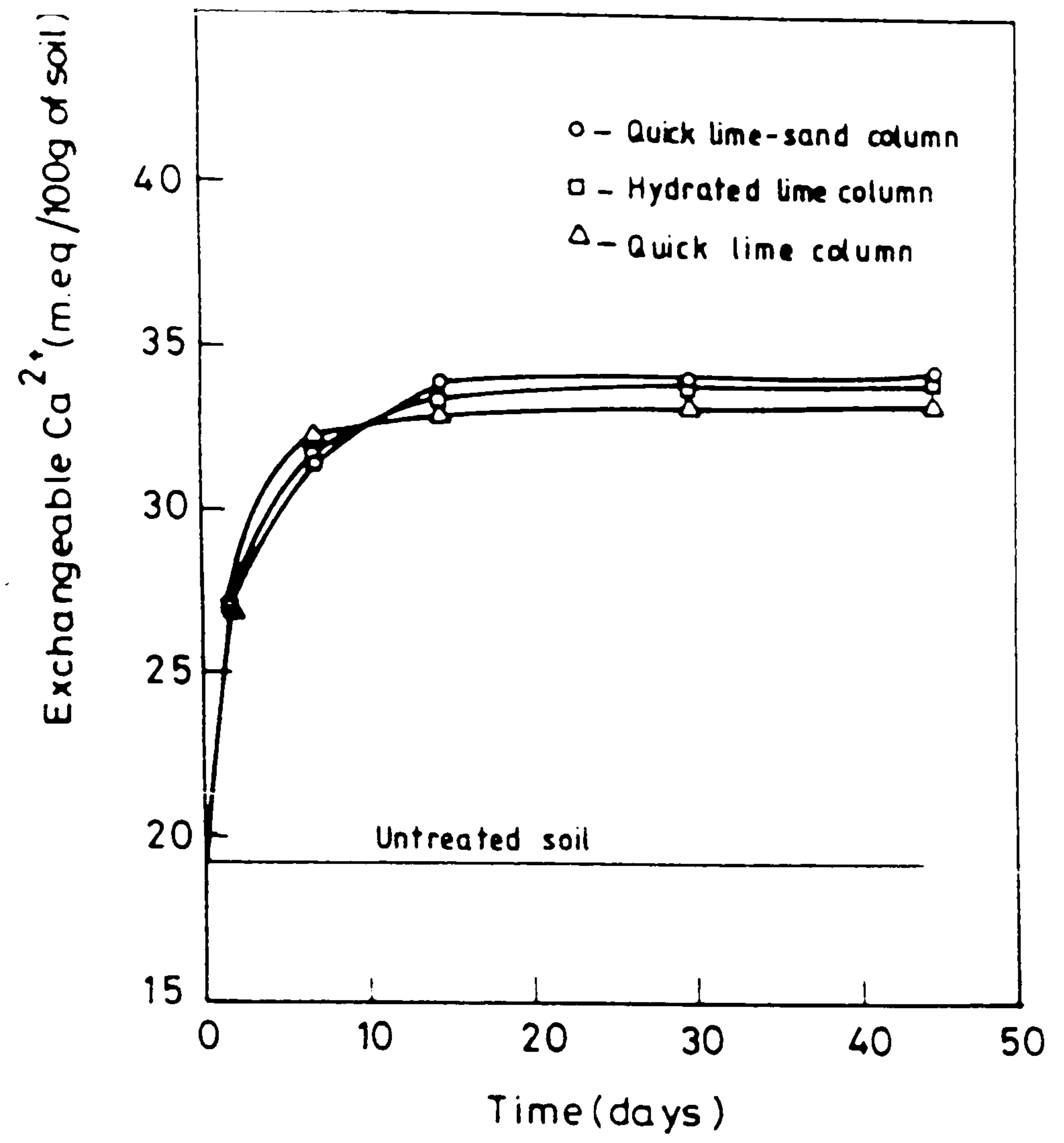


Variation of liquid limit with time

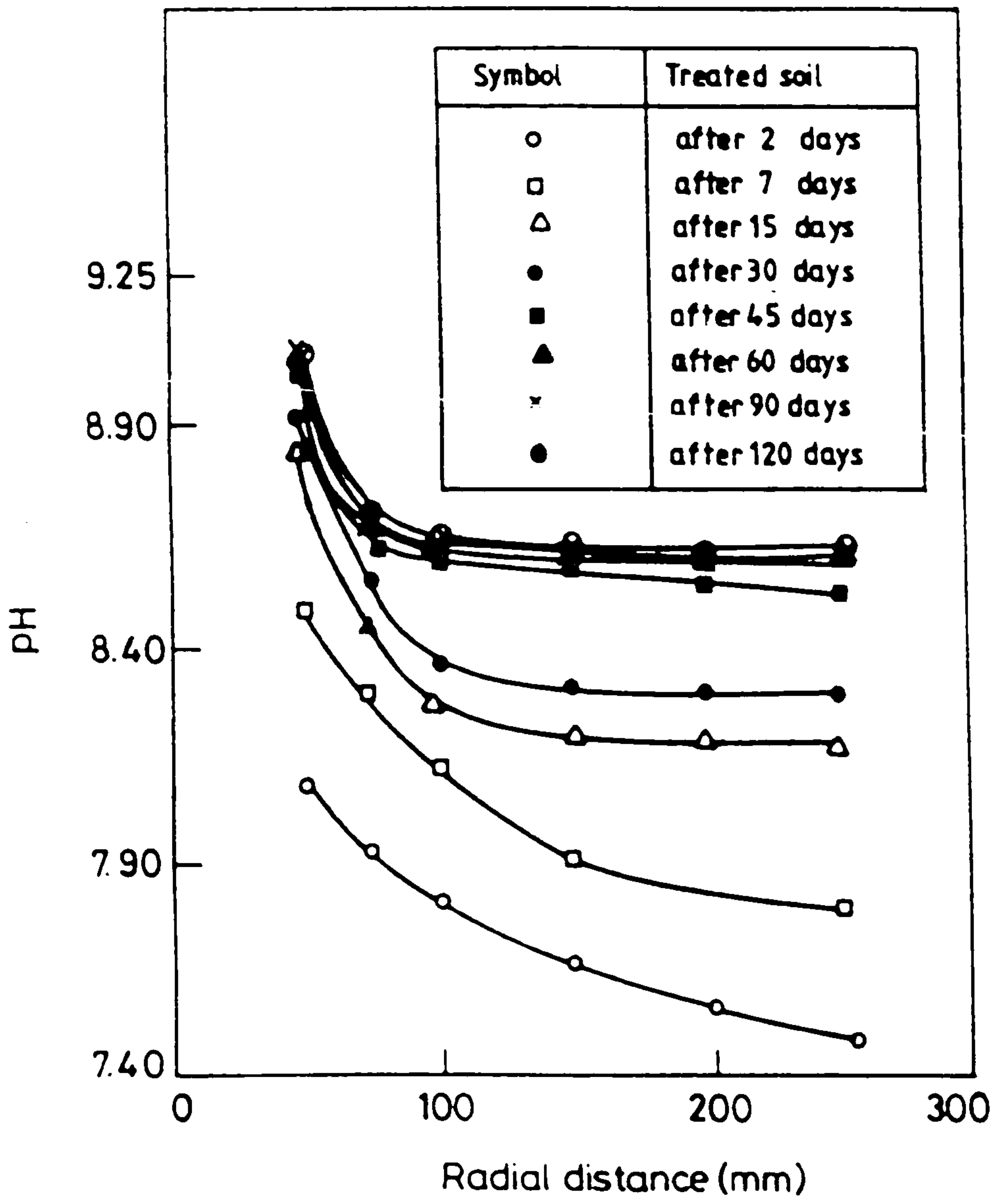


Variation of plasticity Index with time

Figure 2.23 Plasticity changes after lime pile treatment (after Rao et al, 1993)

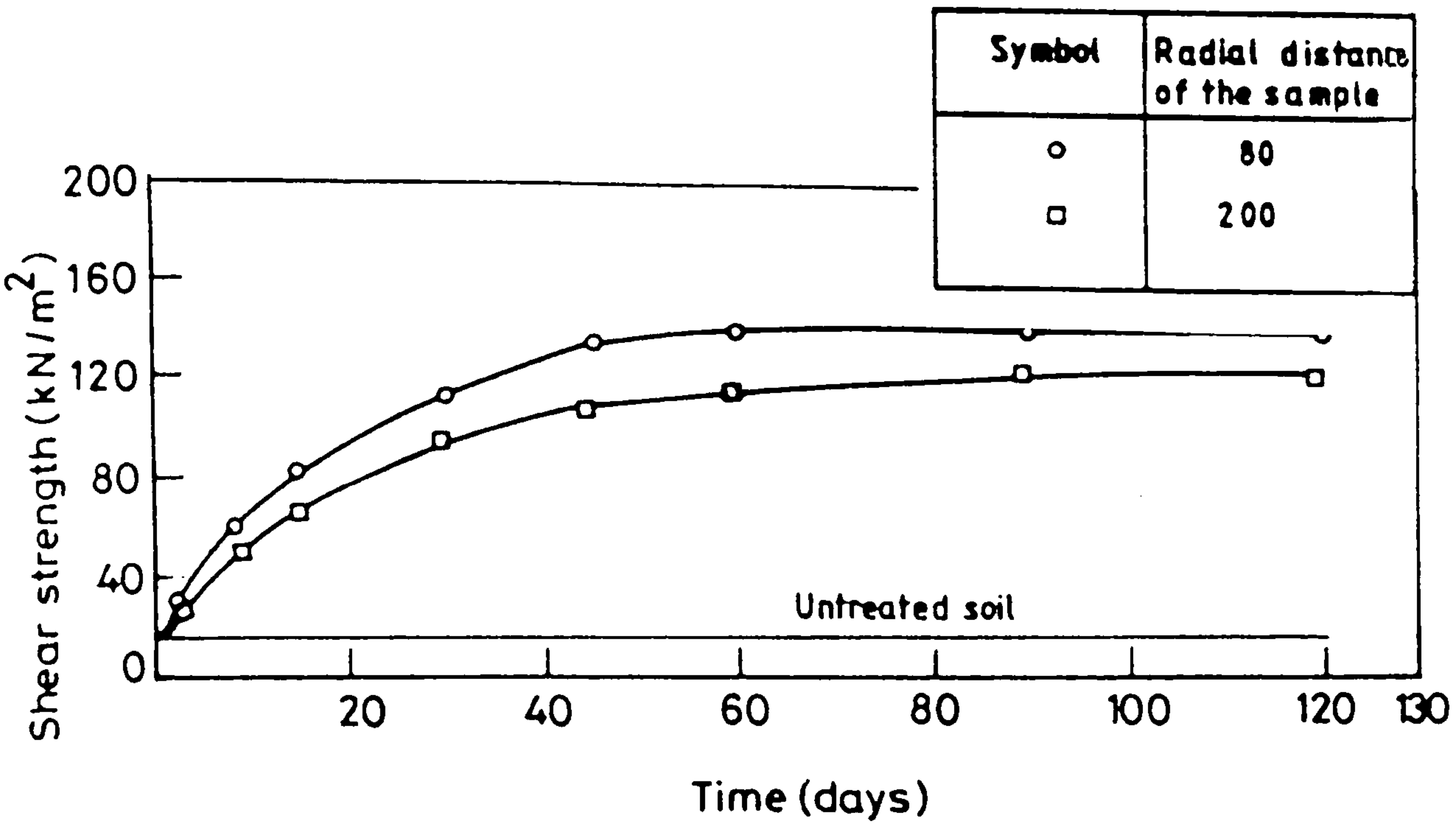


Variation of exchangeable Ca^{2+} with time.



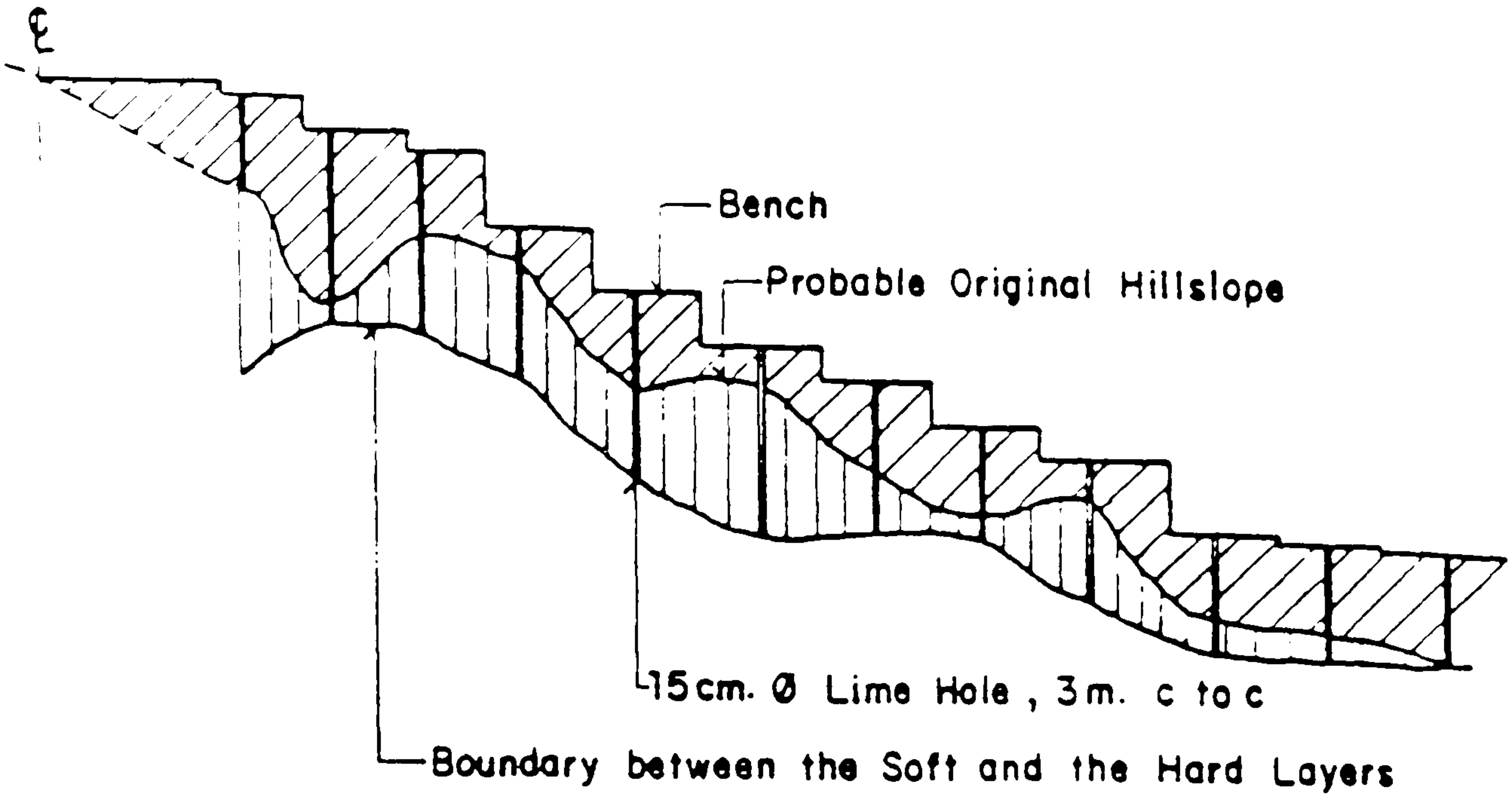
Variation of pH along the radial distance (for quick lime column setup)–long duration test.

Figure 2.24 Migration due to lime piles (after Rao et al, 1993)

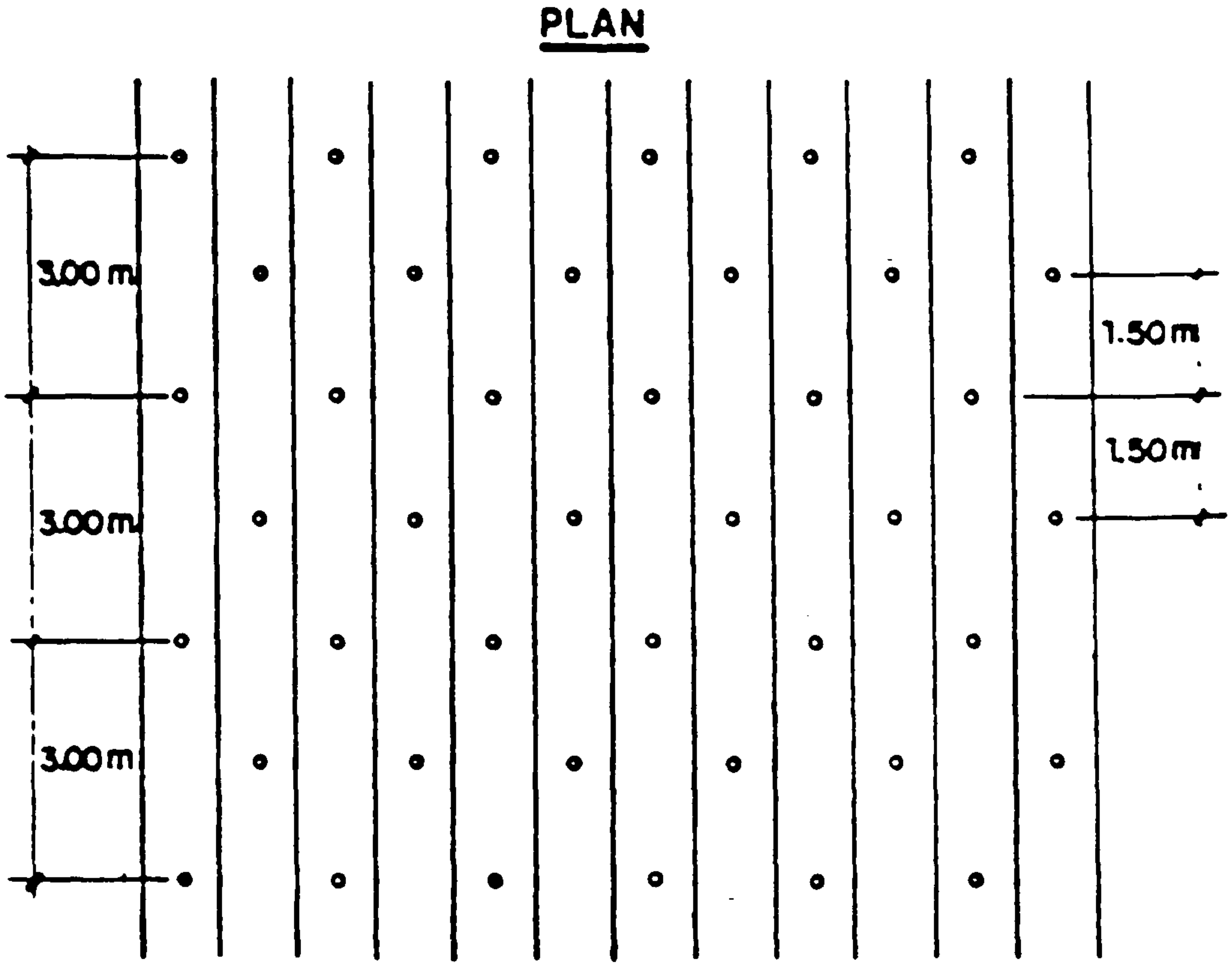


Variation of strength withtime (for time column setup)-long duration test.

Figure 2.25 Strength changes due to lime piles (after Rao et al, 1993)



Cross section of clay shale embankment at km. 115 + 500 on the Chiangmai – Chiangdao Fang National Highway



Deep hole lime stabilization for unstable fill

Figure 2.26 Application of lime piles to a clay shale embankment (after Ruenkrairergsa and Pimsarn, 1982)

CHAPTER 3

3.0 RESEARCH PHILOSOPHY

3.1 SUMMARY AND DISCUSSION OF THE LITERATURE

Lime piles have been successfully used worldwide for the treatment of clay soils *in situ.* The technique has not, however, apparently been used in the UK although similar problems are being faced here as those to which the technique has been applied. The reasons for this lack of application of an apparently successful technique are not immediately clear but may lie in the lack of knowledge of overseas construction projects and the lack of understanding of how lime piles work.

Lime piles have been used for two distinct purposes. The first application is in the treatment of soft soils to improve their bearing capacity, and in this case uses relatively large diameter quicklime piles at relatively close spacings. The result is a significant reduction in the water content of the soil, densification of the soil and concomitant increases in strength and stiffness. The second application is in the stabilisation of failing slopes. In this case both quicklime piles and lime slurry piles have been used, with the intention of causing ion migration and subsequent lime-clay reactions to occur in the soil mass. In both cases much work has been done, but the experimental evidence for their operation is often poorly presented and requires careful interpretation if it is to be used for practical purposes. A summary of the postulated stabilisation mechanisms is presented hereafter and is discussed from the point of view of fundamental soil mechanics.

It is clear from the literature that several possible stabilising mechanisms exist. These can be summarised as follows:

3.1.1 Lateral Consolidation

Lateral consolidation is only claimed in the case of quicklime piles. As the quicklime draws in water from the surrounding ground it reacts or slakes to form hydrated lime. Slaked lime has a lower density than quicklime and hence expansion occurs. This expansion is said to cause lateral consolidation of the ground surrounding the pile. However considerations of stoichiometry of the hydration reaction indicate that there is a small net volume loss and thus the idea of physical expansion cannot be valid if the water causing the hydration reaction is drawn from the soil and the soil is a readily compressible medium. Densification of the surrounding soil due to water loss does occur in a closed system, however, and thus a form of consolidation could be said to occur as a result of seepage forces caused by the (high) negative pore water pressures generated by the addition of strongly hydrophilic quicklime to the ground.

In the cases where water was added to a quicklime pile, for example by surface inundation, then lateral consolidation caused by physical expansion is possible. In soft soils the physical displacement of the soil by a driven tube followed by compaction of the lime in the piles can similarly cause physical expansion and thus lateral consolidation. Thus the effect of soil densification can occur by several means, but the mechanisms differ and the term lateral consolidation is, perhaps, confusing. The successful use of lime piles in loess indicates that the densification sought can be achieved without a significant clay content, this type of improvement thus being applicable, for example, to alluvium in general.

3.1.2 Water Content Reduction

The slaking reaction "uses up" some water from the soil surrounding the piles, and, being highly exothermic, it also produces a significant amount of heat. These two processes in combination are thought by some authors to cause a significant water content reduction. The volume of water used up in the slaking reaction is simple to calculate, the calculation showing that piles of significant diameter at relatively small spacings, as are used in the soft soil treatment of the far east, are necessary to generate large water content reductions. Large piles and a source of rapidly available water (as in soils with a high permeability and/or a high water content) will create the conditions for significant steam generation and water loss from the soil as a result of it boiling off. Once more the presence of clay is not necessary, and since it would reduce the permeability of the soil it would prove a hindrance to stabilisation by this mechanism.

3.1.3 Clay-Lime Reaction

It is widely reported that lime migrates from the piles and reacts with, and hence stabilises, the ground surrounding the lime piles. In order for this to occur both calcium ions and hydroxyl ions must migrate through the clay. Saturation of the system by calcium ions is necessary for maximum cation exchange on the available exchange sites of the clay minerals and hydroxyl ions produce the highly alkalinic ($pH > 12.4$) conditions that are required for dissolution of silica and alumina prior to the formation of calcium silicate hydrate and calcium aluminate hydrate, the gels that crystallise to cause stabilisation. Laboratory experiments have shown that the migration in clays is restricted to a relatively small distance, as would be expected from consideration of the operation of clay barriers. Water transport of ions from quicklime piles in intact clay, for example in clays having a high water content, does not appear to be a valid mechanism, and indeed it is suggested that capillarity provides the mechanism such that drier clays are better in this respect.

In cases where lime slurry has been used to create the piles, migration over significant distances is reported, presumably as a result of hydraulic transport. 'Watering' of quicklime piles has also been suggested in order to improve hydraulic transport of ions. The measurements of ion migration are, however, suspect in many cases and in some cases migration is claimed without measurements to support the claim. The self-limiting nature of the reactions, in which stabilisation of the soil adjacent to a pile inhibits further migration, must also be considered in the migration process.

3.1.4 Cracking

This discussion assumes that the clay remains intact and compresses or swells as a result of changes in stress conditions. Tsytovich et al (1971) noted the development of radial cracks around each pile placed into loess. This suggests a significantly different response of this material at least to the effects of drying and pile expansion. Such cracking could have significant implications on the migration of calcium ions (especially if the cracks were to become subsequently filled with water), pore water pressure and water content reduction. In terms of slope stability, crack development could also have a significant effect which should not be entirely ignored. Observation of crack development in any laboratory programme must play an important role.

These mechanisms form the primary sources of stabilisation according to the literature, although other potential stabilisation mechanisms derive from considerations of the mechanics of the process. It is noted above that the addition of quicklime to a soil results in negative pore water pressures which draw in water to the piles to react. In soft, wet and relatively permeable soils the pore water pressures will be high only in the immediate vicinity of the pile and for a short time. In drier clays of low permeability, however, the suctions will be far higher and will last far longer and, since the suctions increase the mean normal effective stress, the soil will become stronger and more stable. This could be of considerable advantage to failing slopes, if only to provide time for other stabilisation mechanisms to come into operation.

The literature has thus provided much evidence of the success of lime piles in treating both soft ground to improve its bearing capacity and slopes to improve their stability. A significant proportion of the literature on the subject is contradictory or misleading. Thus although the technique is of value, the need for a comprehensive programme of research, both in the laboratory to isolate the stabilisation mechanisms operating and in the field to prove the techniques, has been clearly established. This project aims to investigate the work from the various countries in which the technique has been used and the stabilisation mechanisms that have been postulated. It will establish and quantify the true stabilisation mechanisms and use these data to develop a design method for use in this country. This will be used in two full-scale field trials.

3.2 EXPERIMENTAL APPROACH

As a result of the apparently contradictory experimental evidence presented in the literature it was considered essential that each potential stabilisation mechanism be examined individually in appropriately designed laboratory tests. There were insufficient experimental or practical data upon which to base such a programme of research, however. An initial experiment was thus designed and implemented and an iterative approach subsequently adopted whereby the results from one series of tests were used in the design of the next.

An assessment was made of the above stabilising mechanisms and the first experiment was designed to investigate what were considered to be the key operations affecting the viability of the process as a whole. These factors were:

1. the movement of calcium ions in a soil, and
2. the subsequent lime-clay reaction.

The success of previous laboratory experiments carried out at Loughborough, notably those reported by Loh (1990) and Goh (1991) led to a similar approach being adopted in this first experiment. This experimental technique essentially involved scale model simulation of lime pile construction in clay compacted into 0.4 m cubic boxes. Thus a mechanism had been isolated for study and an experimental approach dependent on the results of this work developed.

As data were collected and experience gained by observation about the way in which clay behaved in response to lime piles, a greater understanding was reached about the way in which lime piles affect the ground. Additional stabilisation mechanisms therefore could be postulated and appropriate experiments designed for their investigation. As further understanding was reached it was felt that larger scale models of lime piles in clay could be designed and implemented to facilitate maximum data collection. These were then designed as a means of studying simultaneously the various modes of operation that had previously been isolated. This approach was found to be more productive than studying the combined mechanism at the start of the project as a means of isolating individual mechanisms subsequently.

An important influence on the overall approach was the design and execution of two full-scale field trials. Both sites were chosen by the British Waterways Board, one of the project sponsors. Both sites had failures in cutting slopes, one in London Clay and one in Lower Lias Clay, and presented some problems in terms of navigation to waterways traffic. It was decided in the early stages of the project that these should be designed once a full understanding of the stabilisation mechanisms had been reached via the laboratory programme. This necessarily meant that they would not be undertaken until the latter part of the project. Nevertheless, an investigation of the sites had to be undertaken and monitoring instrumentation installed. Samples were integrated into the laboratory study to facilitate design.

A concurrent project, developing a computer model for slope stabilising techniques, was used in the design process. Results from the laboratory programme were converted into a form suitable to allow analysis, and therefore design.

Hence a philosophy was adopted which would allow the research to be driven by observation and to develop in the critical areas. This prevented the study being unduly affected by the preconceived ideas of previous researchers. A series of experiments was developed, the results from which allowed the successful development of several large-scale laboratory models. Whilst the laboratory programme was underway, the computer model was being developed so that it was in a suitable form to be used for design at the time of the field trials. The laboratory programme and the computer model could then be concurrently developed to be mutually beneficial in terms of data transfer. For example the model could highlight critical areas for investigation, and the information supplied from the laboratory increased its ability to do so. The results from the laboratory programme as a whole and the experience gained from installing the model piles produced the information required for the computer model to be used for design of the field trials. The field trials consequently served a dual role: a crystallisation of the ideas from the laboratory and the gaining of practical experience of installation and monitoring. Their planning and successful execution thus represented a culmination of the experience and ideas gained over the course of the research.

CHAPTER 4

4.0 EXPERIMENTAL METHODS

As described above, an iterative approach to the experimental programme was adopted. It has not been possible to write the following section entirely in chronological order to show precisely the order in which it developed. The Chapter has been organised in such a way as to group similar experiments together, showing the development from one to the next. Sometimes this has meant that some details of the conclusions have had to be included to explain the aims of subsequent experiments. However, the main body of results and conclusions appear in following Chapters.

4.1 GENERAL METHODS

Where lime is referred to, calcium oxide Grade SG 60, as defined by BLI (1993) has been used unless otherwise stated. Its reactivity has been tested according to BS 6463 (BSI, 1987). To ensure reactivity has been maintained all lime has been heated to in excess of 450° C overnight before use in the laboratory. All lime has been supplied by Buxton Lime Industries. Their Chemical Safety Data Sheet has been referred to for handling and storage, see Appendix 1.

The principal clay used for the study has been English china clay Grade 50 supplied by Watts, Blake, Bearne and Co. Ltd., Newton Abbott. Other clays used have been dug from the sites of the field trials and are broadly a lower Lias clay and a London clay. Wyoming bentonite, supplied by Steetley Minerals Limited of Middlesborough, has also been used. Details of general engineering properties examined for each of these clays are given in Section 4.4.1. Mineralogical analyses have also been conducted, see Section 4.4.2.

The presence of lime has been detected by means of an acid-base indicator, phenolphthalein, which undergoes a colour change from clear to pink in the pH range 9 to 10. Although there are some problems with this method which are discussed in 4.4.2, it has been used previously for this purpose (TRB, 1987). Where calcium ion content has been specifically tested for, the method outlined in Appendix 2 has been used.

Where possible British Standard methods have been adopted. These are referred to specifically in the appropriate section. The Atterberg Limit test has been used extensively to detect changes in plasticity wrought by lime. When lime is introduced to the clay-water system by means of migration (i.e. calcium ions moving through solution from a concentrated source) those changes are necessarily very small. It is recognised that there are limitations in the accuracy of such a subjective test and results have been treated with due caution.

Where experiments refer to shear box testing the standard direct shear box was used. In some cases an automatic shear box is referred to. This box was computer controlled and allowed automatic rapid reversal of the box between passes. Details of this box may be found in Rouiaguia (1990).

The Pulverised Fuel Ash used was supplied by Pozzament. Chemical details are given in Appendix 3.

Mercury manometers have been used to measure pore water pressure. These have been tested with a static head of water of 0.5 and 1.0 m, with no discernible error in reading being detected in either case. The maximum readable pressure was plus or minus 160 mm of mercury. This has proved insufficient in some cases, so estimations have been made in these instances. For absolute values of pressure the head of water on the side of the gauge not connected to the box should have been subtracted. This has been ignored as it was the change in pressure that was of interest rather than absolute values. Due to the effect of parallax the readings are only accurate to plus or minus 2 mm mercury (0.3 kPa).

Temperature in the box tests has been measured by means of 'k-type' thermocouples and recorded using a data logging device. Their accuracy was tested by placing each one in a water bath and comparing the result to that recorded by a standard mercury thermometer. For any individual thermocouple, an accuracy of within plus or minus 1°C was recorded. However, within an 8 hour trial at constant temperature, the variability of the reading on any one thermocouple could be as much as 0.5°C. Therefore changes in temperature within this range are subject to considerable inaccuracy. The response time between each thermocouple was not discernibly different.

4.2 BOX TESTS

4.2.1 Small Box

The first trial was conducted in a 0.4m cubic box. It consisted of a steel frame with 18 mm thick wooden side panels, lined with polythene sheeting and filled with lower Lias clay. The box is illustrated in Figure 4.2. The sectional arrangement was very similar to one section of the large box arrangements, Figure 4.3. The clay was compacted at a Liquidity Index of 0.5 in 50 mm layers using a manual 'sheepsfoot' compactor (Figure 4.1). The aim of compaction was to minimise, and in practice to virtually eliminate, air voids. A 50 mm diameter hand auger was used to form the void into which powdered quicklime was compacted in layers, thus creating a central lime pile. The pile was placed down to within 25 mm of the base of the box. Two manometers were placed 50 mm from the base of the box and 170 mm from the edge of the pile, on opposing box faces. A reservoir of saturated Leighton Buzzard sand was placed around the end of the tubes to ensure full saturation and hence water continuity from the clay to the manometer. Geotextile was wrapped around the tube ends to avoid ingress of clay. Once the box was filled, the clay was covered in polythene and an 18 mm thick wooden board and a static stress of 40 kPa was applied to the surface of the clay.

Subsequent pore water pressure changes were monitored. The pile was created once cessation of change indicated that equilibrium had been achieved.

After curing periods of 0, 7, 15 and 22 days small samples of the clay surrounding the pile (avoiding areas that had previously been disturbed) were taken at 0, 50, 125 and 175 mm from the pile respectively using a 50 mm diameter hand auger. The resulting voids were subsequently filled with wax. Additional samples were taken at 50 and 125 mm after 22 days, 125 and 175 mm after 29 days and 50, 125 and 175 mm after 57 days. These samples were tested for changes in Liquid and Plastic Limit and free calcium content. Visible changes such as crack size and position and texture of the clay were also noted. After a curing period of 20 weeks calcium migration was examined using the acid base indicator painted onto the surface of the clay and along the excavated surface of the cracks.

Two similar box arrangements were filled with English china clay at a Liquidity Index of 0.5 and used to study the effect on pore water pressure of mixing Pulverised Fuel Ash (PFA) with the lime. Two boxes were filled: in one a quicklime pile was created in the same manner as outlined above and in the other the same method was used to create a pile consisting of quicklime and PFA mixed in the ratio 10:1 respectively. Pore water pressures were monitored as above at distances very much closer to the piles of 25 mm. Migration was monitored by means of painting indicator onto the clay surface. A static head of 13 kPa was used in each case.

The idea was further developed to allow more control of the normal stress and to allow the study of different initial conditions.

4.2.2 Large Boxes

Following the success of the above test a 1.0m by 1.0m by 0.5m deep steel box was fabricated with the facility of a 'water-bag' arrangement within its lid to supply an even, maintainable pressure to its contents (Figure 4.3). Water was supplied from the mains along a pipeline into the water bag. The pressure was regulated using a Norgren R43-201-NNLD valve and monitored using a Budenberg Standard testing gauge. A vent to atmosphere was also provided.

The box was divided internally into four equal compartments so that different pile types could be studied simultaneously. The box was initially filled with English china clay at a moisture content of 48% (equivalent to a Liquidity Index of 0.5) in layers using hand compaction, as in the previous model, and allowed to consolidate under a normal stress of one bar. Manometers and 'k' type thermocouples were placed during the compaction process. Piles were formed as before with diameters of 100 and 150 mm at two different degrees of lime compaction, a plastic sheet being spread over the clay surface to prevent lime scatter. Moisture contents of the excavated clay were taken. To achieve good compaction the 100 mm diameter piles were compacted in four layers using 10 blows of a 2.5 kg compaction hammer, 20 blows being applied to the layers for the 150 mm diameter pile. Both 100mm and 150mm piles were also tested without compaction. A 40 mm thick clay 'cap' was placed on the top of each pile.

After the piles were formed the normal stress was reapplied and the system left intact whilst pore water pressure changes and temperature fluctuations were monitored. Changes in pore water pressure and temperature were measured both very close to (50 mm), and somewhat removed from (200 mm), the edge of the pile, as illustrated in Figure 4.3 (i.e. Arrangement 1). Measurements were taken until a state of equilibrium had been established, i.e. cessation of changes in the readings. This defined the end of the test. At this stage water contents of the soil 50 mm and 200 mm from each pile were taken and migration was monitored using the indicator. The individual piles were then excavated and U38 mm samples tested to destruction to obtain a value for pile strength. This was achieved using a U100 sample tube provided with a cutting edge, placed over the head of the pile and driven by a heavy falling hammer. Clay surrounding the pile was dug out to remove the tube. The clay surrounding each pile was then carefully cleaned away to expose the lime. Sample recovery was very poor due to the relatively brittle nature of the pile material. For this reason only the 100 mm diameter piles could be sampled as the larger piles would require the U100 tube to be driven through the lime itself.

Two other arrangements were investigated (Arrangement 2 and Arrangement 3). The details of these arrangements are shown in Figure 4.4. Due to some difficulties experienced with the consolidation of the clay in the first experiment, a 100 mm deep layer of coarse Leighton Buzzard sand was laid in the base of the boxes and only the top manometer positions used. A granular grade of quicklime (Fine 6, BLI (1993)) was used to correspond with the type of lime to be used in the field trials. Variation in curing periods and sampling positions between the different box arrangements will be made evident as the results are discussed.

4.3 TUBE TESTS

4.3.1 U100

In addition to box model tests, the ideas of Noble and Anday (1967) were used to study migration and the strength changes occurring around a pile. A set of tests was carried out on samples of English china clay of different water contents and on one lower Lias clay sample. For these tests 100mm diameter samples of the clays, at a known water content, were created by compaction, using British Standard light compaction to BS 1377: Part 4 (BSI, 1990), into Proctor moulds with subsequent extrusion into plastic tubing. The water contents used are given in Table 4.1. In the case of the English china clay sample at a water content of 55 % clay was placed directly into the plastic tubes and voids expelled by means of vibration. An undisturbed sample of lower Lias clay from one of the field trial sites was taken during the site investigation (detailed in Chapter 8) and was also used in this study. Piles were created using a 12 mm rod driven to create a central hole into which quicklime was poured. The quicklime was compacted using a small rod, and the samples were sealed in plastic. Control samples were set up, using talc as the pile material. Samples were tested after 0, 7, 21 and 56 days curing at a constant temperature of 20°C plus or minus 2°C.

Table 4.1 U100 set-up conditions

Clay Type	Water Content (%)	Pile Type
Undisturbed lower Lias clay	~22 ~25	Talc Lime
Remoulded lower Lias clay	~42 ~42	Talc Lime
Remoulded English china clay	35 35 35 45 45 45 55 55 55	Talc Lime None Talc Lime None Talc Lime None

Migration of hydroxyl (OH^-) ions was monitored by means of application of phenolphthalein, and strength change by cutting U38 samples from the clay surrounding the pile and testing using undrained triaxial compression tests (in accordance with BS 1377: Part 8 (BSI, 1990)). The water content of the clay surrounding the pile was also found.

A prediction of the final water content was made using the following method.

1. The amount of water used in the slaking reaction was calculated using considerations of the stoichiometry of the reaction.

2. The expanded volume of the pile was calculated using the assumption that, if fully slaked, quicklime will double in volume.

3. The amount of water used to bring the pile material to the same moisture content as the surrounding clay was calculated.

4. The water leaving the clay into the pile was subtracted from the initial amount of water and the final moisture content calculated.

A prediction of the final strength of the clay due to the reduction in moisture content was also calculated using the method of Skempton and Northey (1953).

4.3.2 Perspex Tubes

A smaller-scale version of the above experiment was carried out specifically to investigate migration rates. This set of experiments examined the effects of water content, lime type and clay type on the rate and distance of ion migration. Perspex tubes, having an internal diameter of 30 mm, were filled with clays at different water content. Water content was specified using the Liquidity Index to ensure that the amount of 'free' water was the same for the different clay types, for comparative purposes. The clay was first compacted at the desired water content into a 100 mm diameter plastic tube (as above) to achieve minimum air voids. The perspex tubes, which were manufactured with a cutting edge at one end, were then pushed into the clay and excavated, thus filling the tubes. The effect of a 'shear plane' was examined by the subsequent extrusion of half the sample into another tube, shearing across by hand, then extrusion of the remaining portion into the tube. Thus a sample with a single 'shear plane' was created. Reconstituted samples of china, London and lower Lias clays were principally used for this study. In addition, mixtures of china and lower Lias clays, at a single Liquidity Index, in the proportions 1:1, 1:3 and 3:1 were also examined.

A 6 mm diameter quicklime pile was placed at the centre of each tube, using a copper tube to first create the void and compacting quicklime or powdered slaked lime using a 5 mm diameter steel rod. Lime slurry was supplied using a syringe. A summary of the clay and lime combinations examined is given in Table 4.2.

Table 4.2 Perspex tube set-up conditions

Mineralogy	Water Content	Clay Set-up	Liquidity Index	Lime Type
Lower Lias clay	42	Normal	0.5	Quick
	30	Normal	0	Quick
	36	Normal	0.25	Quick
	44	Normal	0.6	Quick
	42	Slips	0.5	Quick
	42	Slips	0.5	Slurry
	30	Slips	0	Slurry
	30	Normal	0	Slurry
	42	Normal	0.5	$Ca(OH)_2$
English china clay	48	Normal	0.5	Quick
	50	Normal	0.6	Quick
	42	Normal	0.25	Quick
	36	Normal	0	Quick
Wyoming bentonite	45	Normal	0	Quick
mixed English china : lower Lias clay				
1 : 1	42	Normal	0.39	Quick
3 : 1	42	Normal	0.39	Quick
1 : 3	42	Normal	0.39	Quick

The time taken for the ions to reach the edge of the tube (13 mm) was recorded. This was achieved by integrating the acid-base indicator into the clay which changed colour as the alkaline hydroxyl ions passed through it. Sufficient indicator was added to create a standard solution in the water added to the clay to achieve the desired moisture content. Small samples were tested with sufficient lime to achieve a lime content of 0.5 % by dry weight to ensure a colour change was visible.

It is recognised that the use of an indicator will detect the presence of OH^- ions, not Ca^{2+} ions. The opposing ionic nature of these ions, however, will mean that their movements are related, if not identical. To detect Ca^{2+} directly poses a considerable problem. Firstly, it is very difficult, if not impossible, to ensure that a sample is free from the original placed quicklime due to the close proximity of the pile necessitated by the scale of migration. Secondly if the calcium ions react with the clay itself, a simple test for free calcium ions will give misleading results since only a proportion of the ions that have migrated will be recorded. More complex tests for additional calcium ions on exchange sites and/or evidence of pozzolanic reactions within the clay mineral would be necessary. Even then the variability in 'background' levels of calcium would create difficulties with interpretation of results. The beauty of the indicator is its simplicity. It is also noted that the indicator has been used both integrated into the clay and painted onto its surface. Experimental evidence suggests no perceptible difference in results.

After a colour change had been observed, samples were extruded and dissected. Observation was made of the size of the pile, cracking, and areas affected by migration. Some of the samples were tested for changes in Plasticity Index and Mineralogical changes (see 4.5.2). The effect of the indicator on the plasticity of the clays was tested and found to be imperceptible.

4.4 SHEAR BOX TESTS

4.4.1 Mini Piles

This investigation was designed to research the effect of treatment using lime piles on the strength of a shear failure plane. The 100mm x 100mm square shear box was used with the direct shear apparatus both to create the shear plane and measure shear strength, both before and after treatment with a lime pile. The procedures outlined for shear testing in BS 1377: Part 7 (BSI, 1990) were followed where appropriate.

A normal stress of 100 kPa was used throughout using English china clay at a water content around the Plastic Limit (36 %). Both normally and overconsolidated samples were tested, an overconsolidation ratio of 3 being used. Normally consolidated lower Lias clay samples were also tested using a water content around its Plastic Limit of 30 %. Measurements were firstly taken of the residual strength before treatment. Five rapid passes of the box took the sample to close to residual strength. Readings were taken on the subsequent pass where drained conditions were maintained throughout through the use of the appropriate shearing rate.

Samples were then transferred to 100 mm x 100 mm x 30 mm moulds , manufactured from 3 mm thick steel, for curing. A top and bottom plate was provided to contain the sample and cover the lime piles. Care had to be taken during movement of the sample to ensure that K_o conditions were maintained. Normal stress conditions were maintained during curing by means of a loading apparatus, consisting of a steel frame and loading rod. Similar equipment to that used for the perspex tubes was used to create a central pile passing through the shear plane. Quicklime was used in all cases except in the case of a control experiment which looked at the effects of talc. Water contents of the samples were found by testing the material extracted for pile placement.

The mould and sample were placed in a double polythene bag together with a wet paper towel to maintain a humid environment for curing. Several samples were stacked together in one loading frame and left to consolidate for 7, 14, 28 and 56 days. Samples were removed from the loading frame and pushed from the mould into the shear box to re-test after curing.

After final shearing of the treated sample, it was split along its shear plane and examined for calcium migration using phenolphthalein. The diameter of the migration circle was measured and the samples photographed. The sample was then cut in half vertically through the lime pile to check the extent of migration, as shown in Figure 4.5.

4.4.2 Shear Plane Consolidation

The experimental work outlined above provides information on the changes in certain properties of the clay, but little information on how the material forming the shear plane itself will react to those changes. The idea of pore water pressure reduction caused by quicklime slaking in lime piles was first raised by Rogers and Bruce (1990). However, its effects were then only considered to be temporary: an immediate effect which would be beneficial before the longer-term clay stabilisation reactions took over. Its consequences were only considered over the time in which it was present, i.e. while the reaction was taking place. It is now thought that the longer-term effect on any shear failures present within a slope could be more crucial. If the reduction in pore water pressure is considered as an increase in effective stress on the shear plane, it will result in consolidation of the remoulded zone associated with the shear failure. This consolidation might be sufficient to raise the shear strength in this area above the fully-softened critical state or residual value, whichever is applicable. This rise in shear strength might in turn be sufficient to prevent further failure. The rise in shear strength due to overconsolidation of the shear plane has been studied, using a 100 mm shear box in accordance with BS 1377: Part 7 (BSI, 1990) where appropriate.

The aim of this work was thus to observe the effect of overconsolidation on the shear strength of a pre-formed shear plane. The direct shear box apparatus was used for its ability to create samples with shear planes within them at known, and variable, vertical effective stresses. In some cases the automatic shear box was used to allow more samples to be examined simultaneously. These results have been treated separately.

An initial assessment was made by consolidating the samples to a relatively high normal effective stress (150 kPa) and creating the shear plane using uni-directional shear. This allowed peak and residual shear strengths at this stress to be found. Samples were then subjected to a further vertical stress of 20 kPa (i.e. 170 kPa in total), the incremental value being shown to reflect the pore water pressure reduction in the field (from the small-scale field trial described in Chapter 8). This additional stress was supplied when the sample was at the centre of its travel. After the period of consolidation samples were allowed to swell under a vertical stress of 150 kPa once more and the new value of strength was determined by re-shearing the sample. Little change was observed under these high levels of stress, as would be experienced in deep-seated slip planes.

It was considered that at such a high normal effective stress, a very well defined shear plane would be developed with a high degree of particle orientation. In such a case slight overconsolidation would have little beneficial effect. In the case of shallow slips the shear plane definition will be poorer, the degree of particle orientation will be lower, and the effect of a 20 kPa reduction in pore water pressure could be proportionately greater. In order to achieve realistic results for shallow slips in overconsolidated clays, two approaches were considered:

1. Consolidate to a low normal stress, shear, overconsolidate and shear again; or

2. Consolidate to a high normal stress, allow to swell under a low normal stress, shear under a low normal stress, overconsolidate and shear again.

Both approaches have been attempted, with the anticipation that the latter case would better simulate an ill-defined shear plane which would be more susceptible to re-alignment of particles by overconsolidation. An initial study was carried out using a consolidation pressure and increment of overconsolidation pressure of 150 and 20 kPa respectively, while swelling and shearing took place at normal effective stresses of 30 and 7 kPa. As this approach proved successful, a further, more comprehensive programme of investigation followed. The whole programme (including the initial study) is summarised in Table 4.3.

Table 4.3 Shear box programme

Effective Normal Stress at Consolidation (kPa)	Effective Normal Stress when Shearing (kPa)	OCR at Shear
30	30	1
75	75	1
150	7.5	20
150	15	10
150	30	5
150	75	2
150	150	1
300	15	20
300	30	10
300	75	4
300	300	1
400	20	20
400	40	10
400	400	1

The effect of relaxation of the apparatus has been examined by the application of stress increments to samples at the centre of travel and observing stress readings over a five day period, whilst the box remained stationary. A selection of stress combinations have been tested in both the direct and automatic shear boxes in order to assess repeatability.

4.5 LIME-CLAY TESTING

A selection of clays was tested using standard geotechnical tests and mineralogical analysis as a means of characterisation. The same clays were reacted with lime and re-tested. This would not only allow the observation of the effect of lime on geotechnical properties but also to try to reconcile these changes with those occurring mineralogically. Mineralogical tests were conducted on samples that had had lime migrated through them in order to compare with the changes imparted on lime-clay mix samples. If mineralogical change could be shown to be an indication of strength change this could be used to assess the likely strength change of the clay surrounding a pile.

4.5.1 Engineering Testing

4.5.1.1 Standard geotechnical tests

The following standard tests were carried out on samples of English china clay, lower Lias clay (from one of the field trial sites) and Wyoming bentonite. At the time of the study the site of the second field trial had not been chosen and hence was not integrated into the programme. Atterberg Limits of clay from this site have since been found.

Particle Size Distribution, Atterberg Limits, Specific Gravity, total sulphate content and organic content tests were carried out according to BS 1377 (BSI, 1990) and the Total Lime Consumption Test was carried out to BS 1924 (BSI, 1990).

4.5.1.2 Lime-clay mix tests

These were carried out on the above clays to assess susceptibility to lime treatment. The procedure was as follows:

Quicklime was tested to BS 6463 (BSI, 1987) for available lime content (i.e. the proportion, by mass, of calcium available to participate in a chemical reaction). A sufficient mass of lime was used to provide 2% available lime above the initial lime consumption value for each clay. Initial lime consumption tests BS 1924 (BSI, 1990) were therefore performed on each clay and 2% added to the resulting values. In these cases 4% lime was used for each clay.

Water was added to provide sufficient to satisfy the lime slaking reaction plus sufficient to achieve a water content equivalent to a Liquidity Index of 0.1. In reality, only a relatively small proportion of the water added to the mixture took part in the slaking reaction. In the case of English china clay with 4% lime the amount of water added to satisfy the slaking reaction was 0.03% of the total amount added.

BS 1924 (BSI, 1990) was followed in selecting the mixing period. 30 minutes in a rotary mixer was found sufficient to achieve a homogeneous mix whilst preventing excessive drying.

Mellowing took place in sealed plastic bags with damp cloths placed between layers to maintain humidity. The mellowing period was found by setting up trial samples and testing for changes in Atterberg Limits after 8, 24, 36 and 72 hours. The mellowing period was marked by the levelling out of these changes. 72 hours was used in each case.

The materials were then compacted using British Standard heavy compaction (BSI, 1990) and samples were extruded immediately on compaction to form U38 samples (three per mould). These were sealed and cured for 0, 7, 14, 28 and 90 days.

Quick undrained triaxial tests were conducted at cell pressures of 50, 100 and 200 kPa to BS 1377: Part 8 (BSI, 1990), with a loading rate of 1.4 mm per minute. Drained tests were also carried out on samples of lower Lias and English china clays at cell pressures of 0, 400 and 600 kPa using a loading rate of 0.04 mm per minute.

4.5.2 Mineralogical Testing

4.5.2.1 Infrared spectroscopy (IRS) and X-Ray Diffraction (XRD)

IRS is a rapid and economical means of analysing the structure of clays. It may be applied to both crystalline and amorphous material, making it suitable for the analysis of the development of clay-lime reaction products. Details of the technique are reported by Russell (1987). Spectra are produced which are individual to a clay type. Various parts of the spectra may be attributed to different structural features. XRD produces analogous spectra but is only suitable for crystalline material. IRS and XRD have been used in an attempt to plot the breakdown of particular structural features after curing with lime and to trace the establishment of new materials.

Mineral samples were pre-ground in isopropyl alcohol to optimise particle size for IRS and XRD. For IRS analysis 1 mg of each sample was incorporated into Potassium Bromide pressed discs, 12 mm in diameter. IRS spectra were recorded over a wave range of 400 to 4000 cm^{-1} in a Perkin Elmer 580 B Infrared Spectrometer both before and after heating for 6 hours at 150 ^oC. Ground samples were also analysed using XRD. Samples from the test tube tests have been examined in this way in order to establish if a lime-clay reaction has occurred. Lime-clay mix samples were also analysed in this manner in order to compare them with the test tube samples and to reconcile the mineralogical changes with the concurrent geotechnical changes. English china clay and Wyoming bentonite samples were subsequently re-analysed using a Nicolet Magna-IR Fourier Transform IRS so that traces could be more readily compared. The samples tested are shown in Table 4.4.

Table 4.4 Samples tested for mineralogy

Clay Type	Sample Type	% lime	Water Content (%)	Curing Period (days)
English china clay	perspex tube	-	48	< 240
	mixed	4	46	7
	undrained triaxial	4	46	14
	sample	4	46	28
	No lime	-	46	-
lower Lias clay	perspex tube	-	42	< 240
	mixed	4	26	7
	undrained triaxial	4	24	14
	sample	4	26	28
	No lime	-	28	-
Wyoming bentonite	perspex tube	-	45	< 240
	mixed	4	73	7
	undrained triaxial	4	72	14
	sample	4	69	28
	No lime	-	76	-

4.5.2.2 Scanning electron microscopy and microanalysis

Scanning electron microscopy (S.E.M.) has also been used to analyse clay fabric and the effects of lime treatment upon it. Additionally, the machine has been used to give elemental breakdowns of particular particles highlighted on the microscope screen. (see from McHardy and Birnie, 1987). This was also used as it was thought that it may have been useful in plotting the chemical breakdown of clay minerals and the development of reaction products. Results were inconclusive and hence have not been presented. Although potentially of interest it was felt that inclusion of these results would have unduly concentrated on a relatively minor part of the study.

All results are presented in the following Chapter, with sufficient discussion to make logical sense of the data being presented. A more detailed discussion will appear in Chapter 6.

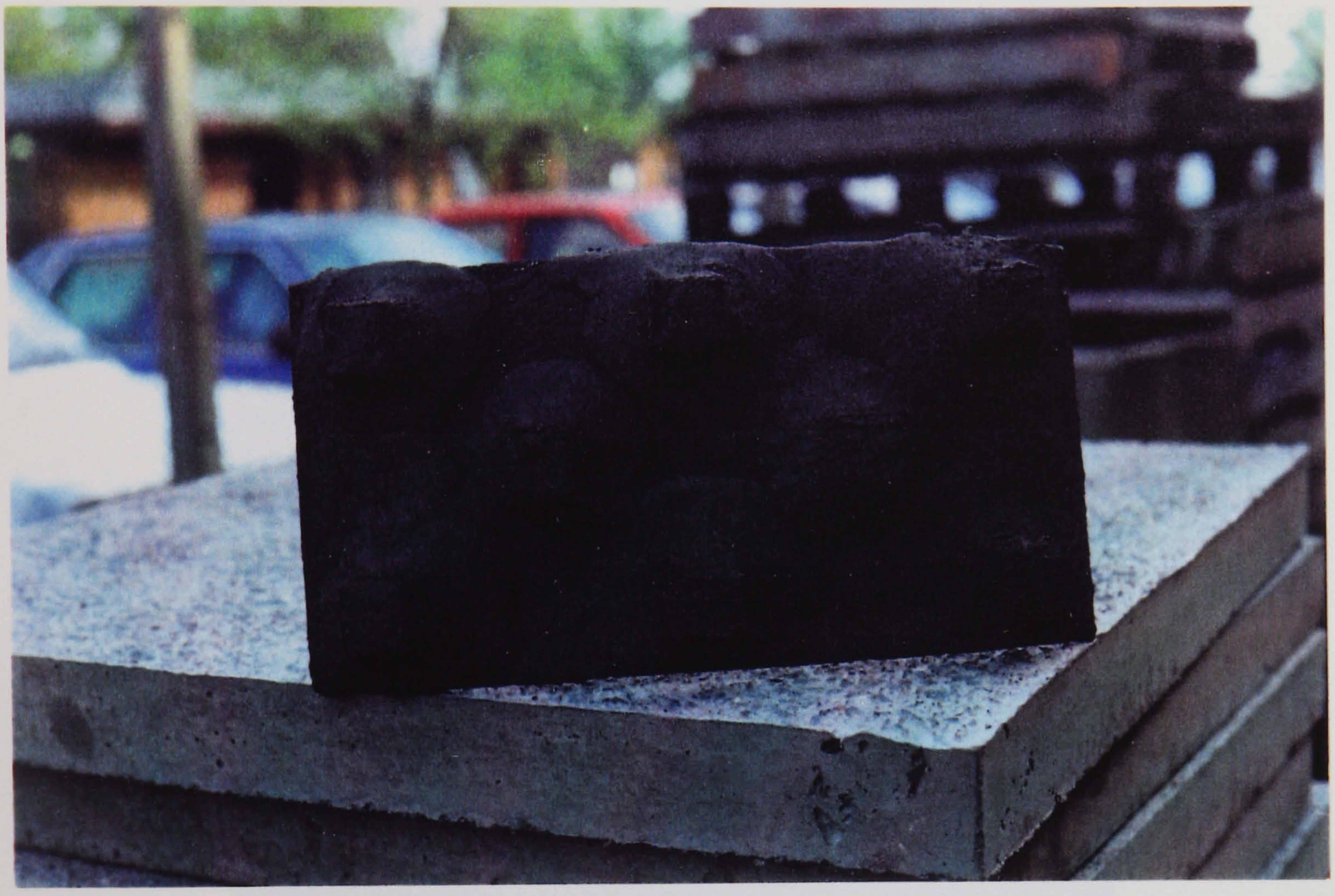


Figure 4.1 Base of the 'sheepsfoot compactor'

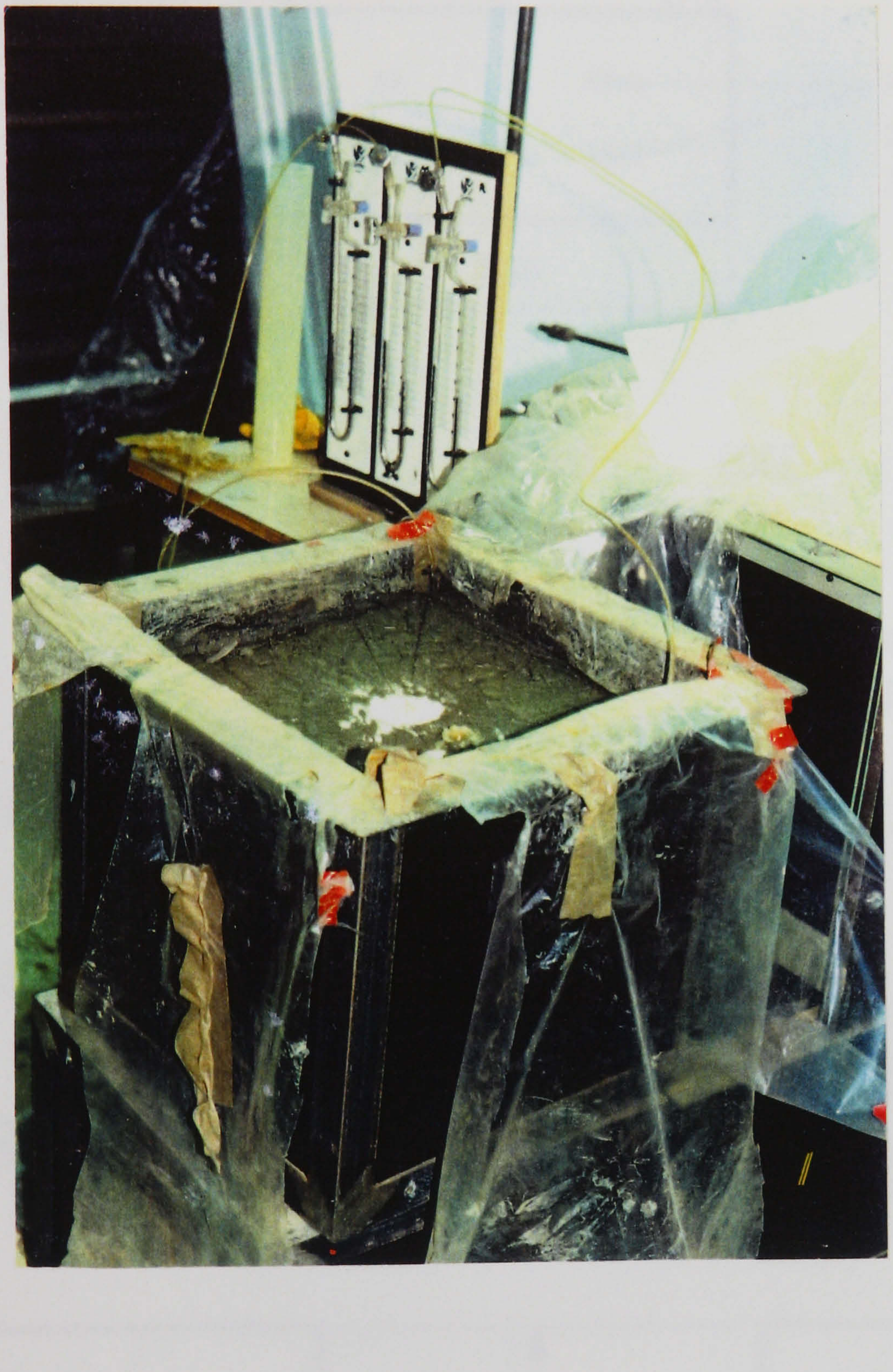


Figure 4.2 Small box laboratory set-up

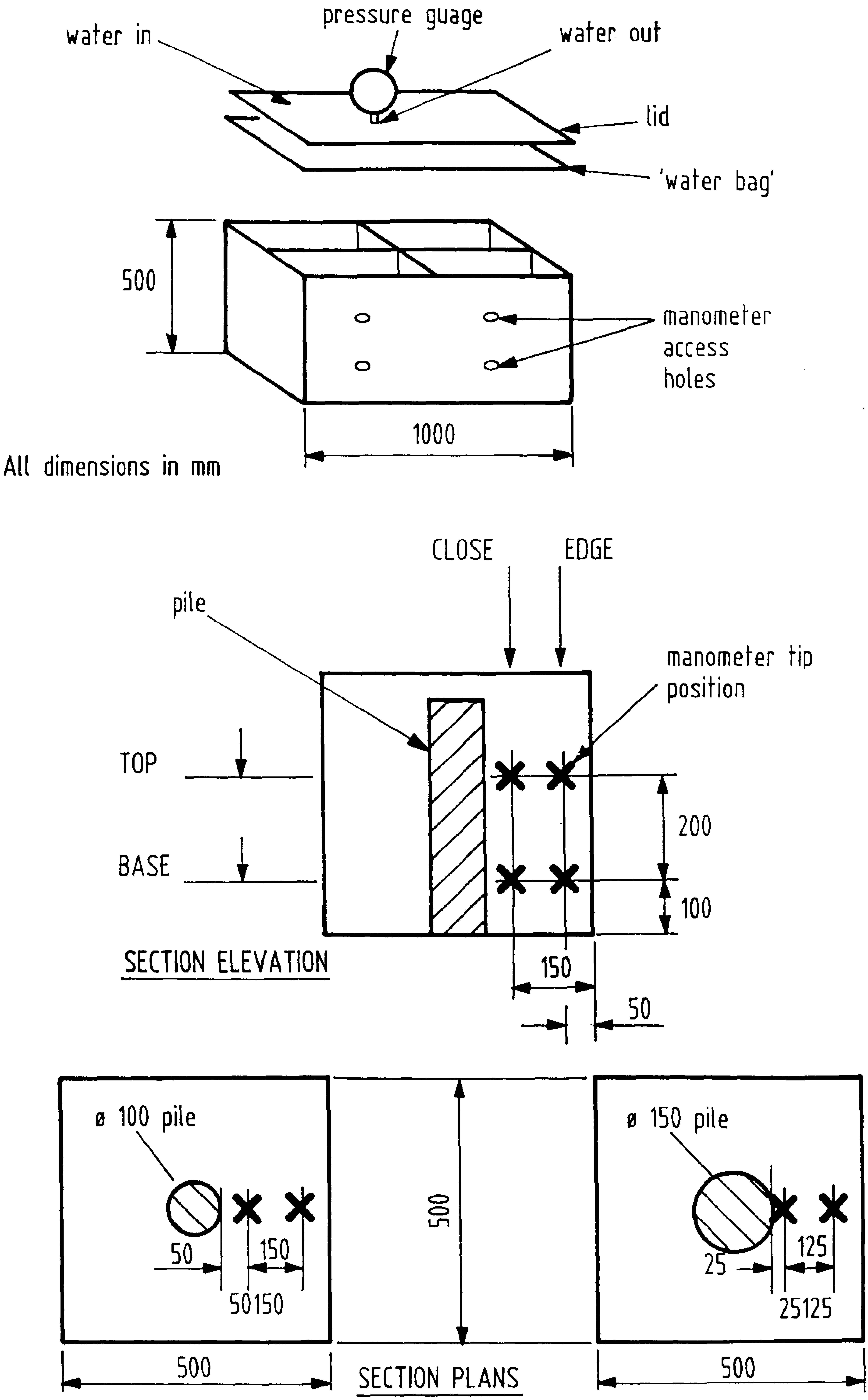


Figure 4.3 Large box laboratory set-up and details of Arrangement 1

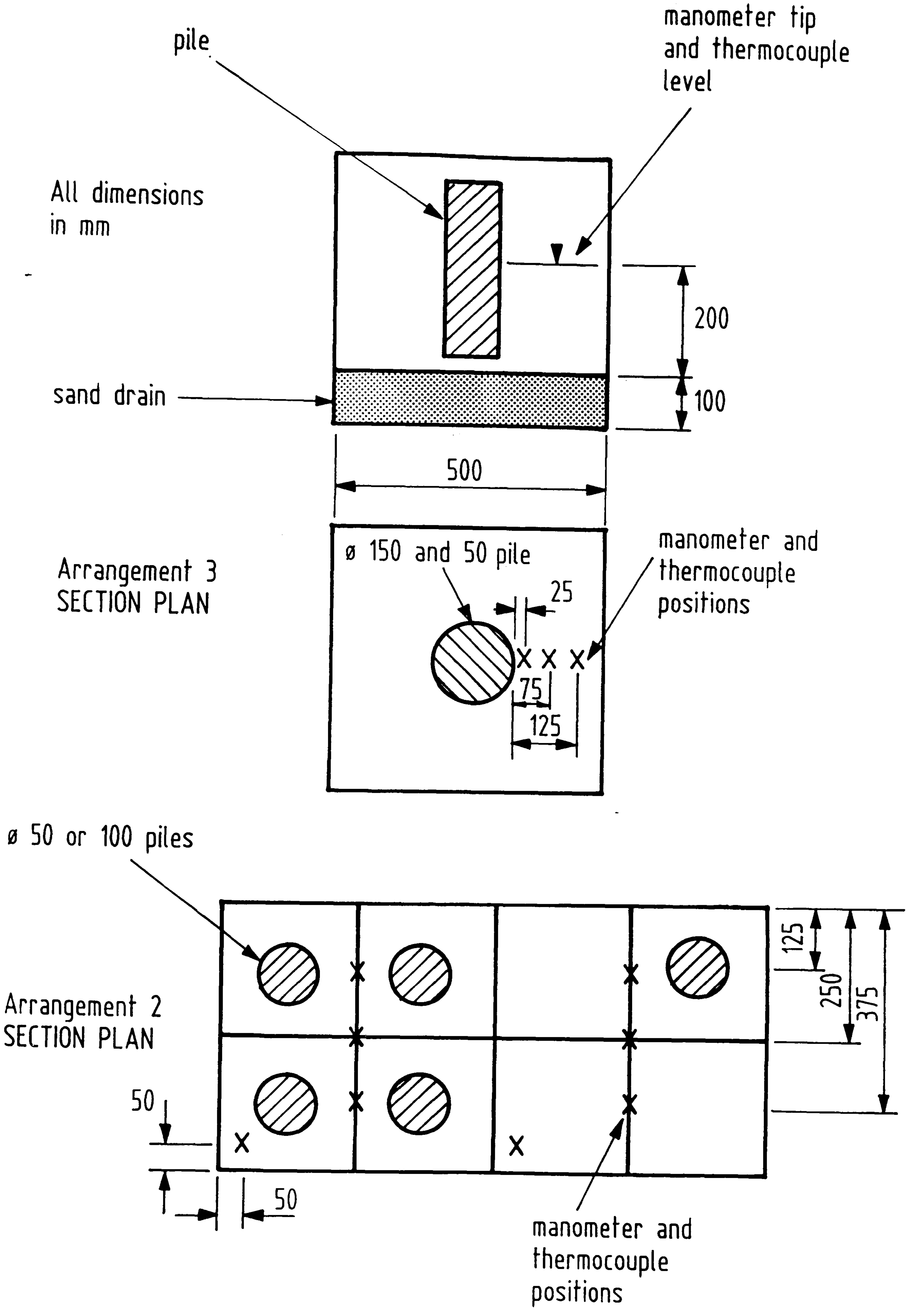


Figure 4.4 Details of large box laboratory set-ups 2 and 3.

Figure 4.5 Migration in the shear box sample

CHAPTER 5

5.0 RESULTS

In Chapter 3 the conclusions drawn from the literature were discussed. A summary of the possible modes of operation was compiled and the experimental approach aimed at their investigation described. Chapter 4 described the resulting programme. This Chapter aims to present the results of this experimental programme by discussing each mode of operation in turn and including data pertinent to that subject from each experiment. New modes of operation are also included. In presenting the results of the experiments general observations made have also been included to assist the understanding of the data collected.

The modes of operation to be discussed are migration, clay property changes, changes in water content and pore water pressure and mineralogical changes. A full discussion of these results will appear in Chapter 6.

5.1 MIGRATION

Observations of migration were made in the box tests, perspex tubes, U100 samples and shear box tests. It should be noted (with reference to 4.3.2) that this was achieved using phenolphthalein and hence the actual ion being detected in each case was the OH^- ion. The results reported herein were factual at the time of reporting. Recent literature searches have shown that the method of testing used may be inappropriate for describing the likely migration of calcium ions, whist remaining satisfactory for hydroxyl ions. This is discussed further in Chapter 6.

5.1.1 Box Tests

20 mm of migration was observed in the first small box with no enhancement along the cracks. 10 to 40 mm was observed in all large box tests, with variability of plus or minus 5 mm. No correlation in the variability with pile size or level of compaction could be made since the results appear to be consistent between tests. The results from the large boxes are summarised in Table 5.1.

5.1.2 Tube Tests

5.1.2.1 Perspex tubes

The results will be sub-divided into the different factors affecting migration rates. A full discussion of these effects will be included in Chapter 6.

5.1.2.1.1 Migration vs. Water Content

Figure 5.1 shows a graph of migration time against Liquidity Index for English china, London and lower Lias clays. This clearly shows how increasing water content reduces migration time.

Handy and Williams (1967) quote an equation from Davidson et al (1965) to describe migration as follows:

$$l = kt^{1/2}$$

where l = distance in inches

t = time in days

k = migration constant (inches/root days)

However, Davidson (1965) himself states that it is only valid for situations where there is no transport of ions by a flow of water.

Table 5.1 Migration in large boxes

Box Description	Pile Description	Water Content before Pile Placement (%)		Duration (days)	Distance (mm)	Comments
Arrangement 1 - one pile per box	Ø 150 compacted	average 37	36.9	70	35-40	no enhanced migration along cracks
	Ø 150		37.3	70	35-40	
	Ø 100		36.9	70	35	
	Ø 100 compacted		37.2	70	30	
Arrangement 2 - 2 boxes, single piles 2 boxes, four piles	1 x Ø 50	average 41 NB: Water contents taken before normal stress applied	40.9	13	10	enhanced migration along cracks
	1 x Ø 100		41.2		10	
	4 x Ø 50		*		15	
	4 x Ø 100		42		15	
Arrangement 3 - one pile per box	Ø 150	average before piles 36.5	35.6	25	30	no enhanced migration along cracks
	Ø 150		38.2		30	
	Ø 50		35.8		30	
	Ø 50		36.5		30	

*water content not taken due to mercury in box

Figure 5.2 shows the relationship between water content and k (the migration constant) for the clays. This demonstrates a direct relationship between the constant and water content. The graph could be used to predict the value of k, but other conditions probably contribute to the rate of migration outside the perspex tube environment, making this exercise academic at this stage.

5.1.2.1.2 Migration vs. Mineralogy

Figure 5.1, showing the relationship between migration rate and Liquidity Index, also demonstrates the differing responses of English china, London and lower Lias clays.

The value of k for the mixtures of lower Lias and English china clay were found to be intermediate to the values of the pure clays for that particular Liquidity Index (Figure 5.3).

5.1.2.1.3 Effect of Lime Type

Table 5.2 shows the migration times for different lime types using lower Lias clay at Liquidity Indices of 0 and 0.5. From the limited data on the wetter sample it would appear that using slurry instead of quicklime has no effect on migration rate, but slaked lime delays it substantially.

5.1.2.1.4 Slip planes

From results of tests on lower Lias clay at a Liquidity Index of 0.5, the presence of a slip plane does not alter the rate of migration. However, the colour change was observed to initiate along the slip plane.

5.1.2.1.4 Other observations

Several observations have been made from logging extruded samples, and these are recorded in Appendix 4.

Table 5.2 Lower Lias clay migration times

Water Content %	Clay Set-up	Lime Type	Date Set-up	Time to 1st colour change (days)	Comments
42 42	Normal Normal	Quick Quick	22/10/91 2/12/91	10) 15)	Only sections show migration
30	Normal	Quick	4/11/91	55	Centre of tube only
36	Normal	Quick	4/12/91	21	Below line of circumferential crack
44	Normal	Quick	4/2/92	11	
42	Slips	Slurry	29/1/92	13	
42	Slips	Quick	29/1/92	13	Along slip
42	Normal	Slurry	29/1/92	13	
42	Normal	$Ca(OH)_2$	29/1/92	none observed	
30	Slips	Slurry	29/1/92	50	
30	Normal	Slurry	29/1/92	53	

These include:

- cracks often mark the boundary between migration occurring or not occurring:
- migration often corresponds to pile swelling;
- migration is sometimes inhibited at the ends of tubes where drying is more likely to occur;
- lower Lias clay samples often only show migration over part of the sample for the whole period of the test;
- English china clay samples show more evenly distributed migration.

5.1.2.2 U100 tubes

Very limited observable migration occurred, even along cracks and fissures, in undisturbed samples of lower Lias clay. Where migration could be perceived, the distance over which migration occurred was never more than 1-2 mm. Migration only occurred in 10% of the total length of the pile measured.

Significant migration did however occur in the reconstituted lower Lias clay samples. 83% of the total migration that took place after eight weeks (18 mm) occurred in the 1 to 3 week period. After 8 weeks the clay through which migration had occurred was visibly lighter in colour and stiffer to the touch than the remaining clay.

Migration distances in English china clay (reconstituted to different water contents) after 7 days were 8, 17 and 15 mm for water contents of 35%, 45% and 55% respectively; after 21 days a further 9 mm of migration was observed in the 35% sample whilst only a further 5 mm was observed in the 45% and 55% samples, Figure 5.4. It was observed that, on application of the indicator, a darker colour change occurred nearer the centre of the migration circle.

Values of the migration constant for the reconstituted samples of lower Lias and English china clay were plotted onto the graph of Liquidity Index against k for the perspex tube experiments, as shown in Figure 5.5. It can be observed that results were generally in agreement.

The migration distance after 3 weeks for the reconstituted sample of English china clay at a water content of 35% may be compared with that achieved in the kaolinitic clay sample tested by Fohs and Kinter (1972). For a sample having a water content of 24% they achieved approximately 19 mm migration in 4 weeks, whereas this experiment achieved 19 mm in 3 weeks at a water content of 35%. Fohs and Kinter, however, show there to be only a very small percentage of lime present at this distance and time.

Again, it must be concluded that increased water content increases migration rates.

5.1.3 Shear Box Tests

As data were available at constant water content, over a variable time period, k (the migration constant) was determined from a graph of the square root of time against migration distance, Figure 5.6. The values of the migration constant for the normally consolidated and overconsolidated sample are 0.18 and 0.17 respectively. In all cases except the normally consolidated sample at 28 days the migration was constant in all directions. However, in that particular sample the migration distance was furthest along the line of the shear plane, as shown in Figure 5.7.

5.1.4 Conclusions

Migration rate is proportional to water content, and is greater in English china clay than lower Lias clay at the same Liquidity Index. However, in both clays the migration rate shows a similar relationship to change in Liquidity Index. Mixtures of the two clays give rates intermediate to those of the two pure clays.

This shows that, for these two particular clays, migration rates are directly proportional to water content, but exact rates are individual to clay type. This means that other factors than water content alone influence migration, although they may be in turn affected by water content. Increased migration rate may be related to permeability, English china clay being more permeable than lower Lias clay. Alternatively the difference in migration rates may be caused by ion exchange, as this is a diffusion process and related to water movement. However, in the absence of further data no firm conclusion may be drawn, since it may be a combination of both or other, additional, factors.

It must also be noted that these values may only be valid in one particular environment. Figure 5.5 shows the graph of migration constant against Liquidity Index with values calculated for the shear box and U100 experiments. While many of the values from the U100 tests are in agreement with those from the perspex tubes, those from the shear box are significantly reduced.

Migration does not occur evenly over the whole tube at once, but begins in small patches that spread. Discrete, very well defined slip planes seem not to promote greater migration rates, but do coincide with the first signs of migration. This probably means that migration occurs along preferential paths, be they areas of less well compacted material or pre-formed slip planes. This may indicate a link between permeability and migration rate.

Discontinuities in migration paths often occur at cracks, evidence of migration being present on one side of a crack and not on another. Migration, if non-uniform, often seems also to coincide with the swelling of the pile material. Both these facts may be an indication of the requirement for movement of water. The crack acts as a barrier to movement and the pile swelling shows that movement of water has taken place.

Initial results also seem to show that only the slaked lime shows a drop in migration rate from that achieved with quicklime. This may be because, unlike quicklime and lime slurry, no significant potential for water movement is produced.

What is not immediately apparent is why some areas of the lower Lias clay show no signs of migration at all, while others have completely changed. It is possible that there is insufficient water left after the slaking of quicklime in the relatively unaffected areas to produce sufficient continuity of movement. A comparison could be made between these observations and results using slaked lime where no drying occurs.

5.2 CLAY PROPERTY CHANGES

5.2.1 Lime-Clay Mix Tests

The results from a series of 'slowly sheared' triaxial tests conducted on lower Lias and English china clays mixed with lime conducted by Lee (1992) are summarised in Table 5.3. The aim was to carry out drained triaxial compression tests, although the absence of pore water pressure measurement yields uncertainty concerning pore suction dissipation. Nevertheless the improvement with time for different clays is evident. For further details of this work refer to Rogers and Lee (1994). The changes in Atterberg Limits with time are shown in Figures 5.8 and 5.9.

The results from undrained triaxial compression tests (carried out on Wyoming bentonite in addition to the above clays) are summarised in Table 5.3. The results of initial lime consumption, or Eades and Grim tests, conducted on these clays are given in Appendix 5 .

Table 5.3 Results of undrained triaxial tests on lime-clay mixes (after Lee, 1993)

Soil Type	Lime Content %	Curing Period (days)					
		0	7	14	28	56	90
		Undrained shear strength, c_u (KN/m^2), Øu (degrees)					
lower Lias clay	0	54,9	-	-	-	-	-
	Water content (%)	*28*					
	4	60,33	-	324,17	304,27	-	415,30
	Water content (%)	*26*		*24*	*24*	-	*26*
Wyoming bentonite	0	29,3					
	Water content (%)	*76*					
	4	71,3	104,7	128,9	53,20	-	168,7
	Water content (%)	*73*	*72*	*72*	*67*	-	*67*
English china clay	4	11,4	23,6	26,6	40,6	-	98,11
	Water content (%)	46	46	46	46	-	46
		Drained strength of lime-clay mixes c' (kPa), Ø' (degrees)					
English china clay	1		125,48		140,51	200,50	
	3		125,52		120,64	240,60	
	5		275,60		300,61	310,63	
lower Lias clay	0				7,18		
	4		1025,43		950,55	1500,58	
	6		1175,56		1225,57	1400,62	
	8		1450,43		1800,55	3500,48	

5.2.2 Box Tests

It was hoped that by taking samples at different distances from the pile in the box tests at one time an idea of the rate of migration and the change in concentration of Ca^{2+} ions with time would be found. It was also hoped to relate these results to the changes in water content and Plastic Limit. It would have been ideal additionally to measure changes in Liquid Limit, but insufficient material was available from one sample to perform this test and larger samples would have been overly disruptive to the test as a whole. However, there was no discernible change in the Plastic Limit, over the period of the test and the calcium ion concentrations showed no definite trend. The problems associated with testing for the Ca^{2+} ion have been discussed in 4.4.2 and this type of testing was not pursued in the other box tests.

A summary of results is given in Table 5.4.

Table 5.4 Small box investigation: clay property changes

Distance from pile (mm)	Time (Days)	P.L. (%)	Water Content (%)	Acid Soluble Ca^{++} % (dry weight)	Water Soluble Ca^{++} % (dry weight)
1st Pile					
-	0	30	42	5.80	0.34
50	7	32	38	5.80	0.36
75	15	32	38	5.80	0.36
75	22	32	38	5.80	0.42
125	22	30	40	5.80	0.36
175	22	30	37	5.80	0.40
175	29	31	39	5.70	0.36
125	29	30	39	5.50	0.36
125	29	30	39	5.60	0.36
50	57	30	38	5.80	0.36
125	57	32	37	5.80	0.36
125	57	29	39	5.80	0.36
175	57	29	39	5.70	0.36
2nd Pile					
50	7	28	36	5.80	0.37
75	14	28	36	6.10	0.40
125	14	28	36	5.10	0.37

5.2.3 Tube Tests

5.2.3.1 U100 Tubes

No change in Atterberg limits was detected over an eight week period for the undisturbed samples of lower Lias clay. Undrained shear strengths showed no obvious increase over this period either, although the presence of fissures within the sample created variation in the results.

Little observable undrained shear strength increase occurred in the reconstituted sample of lower Lias clay during the first three weeks after the piles had been installed. Failure of the triaxial test specimens occurred essentially by barrelling/squashing at 20% strain and no distinct peak in the stress-strain curves was detected. However, the behaviour of the migration zone was noticeably different: slight differential movement was observed, accompanied by surface cracking.

After a curing period of 8 weeks, a distinct shear plane was developed, and peak undrained shear strength was achieved. Strength development is shown in Figure 5.10.

No measurable strength gain had occurred in the reconstituted English china clay after 7 days, which was consistent with the lack of any changes in plasticity after 7 days. All samples with lime piles failed in a differential manner, the migration zone tending not to compress as much as the rest of the sample. All samples with talc piles failed by barrelling alone.

After 21 days all samples with talc piles showed no strength increase and failed by barrelling whereas all samples with lime piles showed increases in strength of between 15 and 20 kN/m^2. All of these latter samples developed distinct shear planes and a failure pattern of:

Table 5.5 Results of U100 tests on china clay

Curing Period (weeks)	Migration (mm)	Strength Change (KN/m^2)	Pile Water Content (%)	PI (%)	Clay Water Content (%)
	Original water content of 35%				
0	-	-	-	21	35
1	8	3	34	19	35
3	20	10	34	19	35
8	25	39	55	20	33
	Original water content of 45%				
0	-	-	-	21	45
1	17	0	43	20	45
3	22	8	41	20	42
8	28	8	72	17	42
	Original water content of 55%				
0	-	-	-	21	55
1	15	3	45	21	51
3	20	5	48	11	50
8	22	9	78	15	50

i) barrelling

ii) differential compression

iii) shear plane development

iv) peak strength development

Changes in strength, water content and Atterberg Limits are shown in Table 5.5. It is notable that there is little measurable change in Atterberg Limits except in the sample with an initial water content of 55 %. Strength change seems entirely attributable to change in water content in every other case. This is illustrated in Figures 5.11 to 5.13 and in Table 5.6, which favourably compares calculated strength with actual measured strength. The calculated strength was found using the method of Skempton and Northey (1953). The higher degree of migration in the wettest sample may have led to the presence of some reacted material in that sample.

Table 5.6 Strength changes related to water content change in U100 samples

Clay type	Water content (%)		c_u (kPa)		
	initial	**final**	**initial**	**final**	**calculated**
lower Lias	42	40	25.5	36.5	36.9
Engish china clay	35	33	85	124	126.8
	45	43	26	34	37.9
	55	51	18	27	28.4

5.2.3.2 Perspex tubes

Table 5.7 shows recorded changes in water content and Plastic Limit for several samples tested in the 30 mm diameter perspex tubes. These tests were carried out as an experiment to discover what sort of data could be collected from the perspex tube tests and have not been planned sufficiently well to retrieve very comprehensive, and therefore, useful data.

However, it can be seen that in all cases the Plastic Limit has risen and the water content fallen. In fact, the water content has fallen to 80% of the original values in the case of the lower Lias clay samples and 70% of the original values in the mixed clay samples. It must be noted, however, that the mixed samples have had a longer curing period.

Table 5.7 Perspex tube tests - changes to water content, lime content and Plastic Limit

Sample	Age at test (since colour change) days	Original water content %	Original plastic limit %	Original % Ca++ by dry weight (water soluble)	Final water content %	Final plastic limit %	Final % Ca++ by dry weight (water soluble)
lower Lias clay	49(34)	42	30	0.17	35	38	0.16
lower Lias clay (colour change)	78(23)	30	30	0.17	23	34	0.14
lower Lias clay (no colour change)	78(23)	30	30	0.17	21	33	0.18
lower Lias clay	61(40)	36	30	0.17	29	37	-
English china: lower Lias clay 3 : 1	59(55)	42	33	-	28	37	-
English china: lower Lias clay 1 : 3	55(48)	42	32	-	29	36	-
English china: lower Lias clay 1 : 1	56(47)	42	33	-	29	37	-

5.2.4 Shear Box Tests

5.2.4.1 Mini piles

As outlined in 4.4.1 this series of tests involved the creation of 100 mm square samples, through the use of the direct shear box, containing single shear planes. 6 mm diameter lime piles were created to pass perpendicularly through the shear plane and the samples re-sheared after varying curing periods. The aim was to find the improvement in residual shear strength wrought by the lime migrating through the clay.

Table 5.8 shows the results of the tests on both the overconsolidated and normally consolidated English china clay. Both samples were sheared under a normal effective stress of 100kN/m^2. One sample had been overconsolidated to an overconsolidation ratio of 3. The residual shear strength before the lime pile was constructed for the overconsolidated soil varied between 28.0 kN/m^2 and 31.5 kN/m^2, and for the normally consolidated soil between 32.5 and 37.5 kN/m^2.

The graphs in Appendix 6 show the shear strength against displacement plots for all samples tested for the various curing periods. All were considered to reach residual strength after 12mm displacement on the fourth reversal. However, this may have not been correct and form a source of error.

After construction of the lime pile and curing all samples reached a new peak strength after a displacement of approximately 4 mm. Both the normally consolidated and overconsolidated soil samples reached a peak strength after 28 days. Insufficient data have been collected to perceive any pattern of strength gain with time.

Once again the residual strength of all samples was considered to be reached after 12 mm displacement. Again, insufficient data are available to show a clear pattern of increase or decrease in residual strength with time. However, the residual strength was (marginally) lower than the original residual strength before the lime pile for the normally consolidated sample after 7 days and the overconsolidated sample after 14 days.

Table 5.8 Shear box set up conditions and results

Overconsolidated English china clay average water content before lime pile = 34.5%					
	Shear Strength kN/m^2				
Curing Period (days)	Before Lime Residual Strength	After Lime Maximum	After Lime Residual	Change in Residual Strength	Migration (mm) Radial Distance from Centre of Pile
7	31.5	42.0	29.3	-2.2	19
14	25.0	41.5	32.8	+7.8	24
28	28.0	47.0	44.5	+16.5	26
56	-	40.7	35.5	(+7.5)	33
Normally consolidated English china clay average water content before lime pile = 37%					
7	34.0	44.7	40.5	+6.5	17
14	37.5	43.0	35.5	-2.0	22
28	36.0	50.1	40.0	+4.0	29
56	32.5				

It is unclear from the data available whether the migration of lime from the pile has caused a significant change in the samples' residual strength. Any apparent increase in strength may have resulted purely from the 'doweling' effect of a central pile. This is explored further in Chapter 6. As water enters the pile the lime will slake and form a coherent mass of cemented material forming a small area of high shear strength.

5.2.4.2 Shear plane consolidation

The aim of this work was to observe the effect of overconsolidation on the shear strength of a pre-formed shear plane. This was in order to simulate the effect of (temporarily) reducing pore water pressure. The direct shear box apparatus was used to create samples containing shear planes formed under various effective normal stresses and at different levels of overconsolidation. This was to simulate different depths of slip surface in material of differing stress history. Once the sample had approached its residual shear strength shearing was temporarily halted, and an overconsolidation stress of 20 kPa was both applied and removed before shearing was resumed. The effect on residual strength was observed and compared with that obtained from stoppage alone. Both china and lower Lias clays were used for this study.

The residual strength parameters for English china clay were determined using effective normal stresses of 150, 300 and 400 kPa. Shear strengths of 39, 69 and 80 kPa were derived respectively, thus giving a residual friction angle (ϕ_r') value of 11°. Over the course of the investigation a wide range of normal stresses have been applied, the failure envelope for this stress range being illustrated in Figure 5.14. This illustrates the curvature at low normal stresses experienced by Anayi et al (1988).

Figure 5.15 illustrates a complete plot of shear stress against distance travelled for the final pass of the box, showing the effect of stoppage and overconsolidation. The results of stoppage alone produced a small, but rapid, immediate reduction in stress after half an hour followed by a much slower relaxation if the sample was left overnight. If shearing was resumed after half an hour the stress rapidly resumed its initial value, whereas if the sample was left overnight, a small rapid increase in stress over and above that of the original value was generally observed before steady state was resumed. These results for a sample overconsolidated to 300 kPa and allowed to swell back to 15 kPa (giving an overconsolidation ratio of 20) are presented in detail in Figure 5.16 and illustrate this trend.

The effects of overconsolidation are similar to those described above for cessation of shear, although more pronounced. One important difference shown in Figure 5.16, however, is that the baseline shear stress rises after overconsolidation and requires considerable further shearing before it falls to the original value.

Taking 300 kPa again as the effective normal stress applied to the sample during the consolidation process, a similar pattern of results is obtained using an overconsolidation ratio of 10. These results are illustrated in Figures 5.17 and 5.18. The results for a normally consolidated sample under an effective normal stress of 300 kPa are illustrated in Figure 5.19.

Taking the overconsolidation ratio of 10 results from samples consolidated under effective normal stresses 150 and 400 kPa are shown in Figures 5.20 and 5.21. The sample consolidated under an effective normal stress of 300 kPa has been shown in Figure 5.18. The effects of overconsolidation were proportionally the most pronounced in the samples sheared at the highest normal effective stress with little effect using an effective normal stress of 15 kPa. This is contrary to what was expected as samples sheared under the lowest normal effective stress would have the most disrupted shear planes. Also, given a particular initial normal effective stress, the greater the degree of overconsolidation, the greater the effect of additional overconsolidation excepted when considering a normally consolidated sample tested at a high normal effective stress. This result is again difficult to explain as such a sample was found to have a smooth easily separable shear surface. These effects are discussed more fully in Chapter 6. Additionally, there appears to be no detectable trend in base-line reading with effective normal stress in the samples tested. Only the sample sheared under an effective normal stress of 30 kPa shows an increase in base-line reading. The base-line of the sample sheared under an effective normal stress of 15 kPa remained constant and the that of the sample sheared under an effective normal stress of 40 kPa actually reduced.

Measurement of vertical movements have generally been unreliable due to problems with instrumentation. Those available are illustrated in Figures 5.22 to 5.24 and show very small compressions associated with consolidation for stoppage and much larger compressions (although still small in absolute terms), followed by a small dilation once shearing is resumed for overconsolidation of the shear plane. The relevance of this with regard to the shear stress will be discussed in Chapter 6.

A summary of the results obtained is given in Table 5.9, in which the improvements have been quantified in terms of increases in shear strength. Improvements have been measured from the value of shear strength recorded immediately before stoppage to the maximum value of shear strength obtained on resumption of shear. The difference between the lowest value recorded whilst the box was stopped and the maximum value was not used as this was felt to be a function of the relaxation of the box itself. Figure 5.25 illustrates the fluctuations produced in shear strength merely by loading a sample in the shear box. In some cases a steady increase in shear strength was recorded towards the end of the travel of the box, as illustrated in Figure 5.26. Again, any value recorded here was ignored as it was also felt to be a function of the box. At the end of travel, the size of the sample is significantly reduced and hence for the same normal load an ever increasing normal stress is induced.

The dashed entries in Table 5.9 signify that no sample was tested with the implied combination of normal stresses. The results summarised in Table 5.9 have also been illustrated in Figure 5.27 as a plot of percentage increase in residual strength against normal stress at shear and in Figure 5.28 as a revised strength envelope for each overconsolidation ratio. The value taken as residual strength in the calculation was the value of shear stress recorded immediately prior to stoppage. This, therefore gives a measure of the percentage change in shear strength actually experienced by the sample at the time of overconsolidation, rather than making a comparison with a more globally interpreted value of residual strength.

The residual strength envelope for lower Lias clay is illustrated in Figure 5.29. Similar relaxation of shear stress was experienced on stoppage of the box as with the English china clay. Resumption of shear after stoppage alone produced a gradual resumption of the pre-stoppage value. Overconsolidation produced a gradual increase in shear strength to above the value obtained previously in the case of overconsolidated samples, with no change observable in the normally consolidated samples. The slight gradual increase in the shear strength has been ignored and attributed to end of travel effects discussed earlier. A typical plot of shear stress against distance for overconsolidated samples is shown in Figure 5.30 with associated vertical movements in Figure 5.31. Similar plots for normally consolidated samples are shown in Figures 5.32 and 5.33. A summary of the combinations of shear stress and overconsolidation ratio studied is given in Table 5.9 along with the increases in residual shear strength produced by overconsolidation. Again, a full discussion of these results, including the issue of repeatability, will appear in Chapter 6.

Table 5.9 Shear plane knitting: Maximum increase in residual shear strength (kPa)

Normal stress at shear (kPa)	Normally consolidated sample	OCR = 4	OCR = 10	OCR = 20
English china clay				
7.5	-	-	-	0.61
15	-	-	0.78	0.96
20	-	-	-	1.01
30	0.50	-	0.48	-
40	-	-	0.92	-
75	1.13	0.58	-	-
150	-	-	-	-
300	1.51	-	-	-
400	-	-	-	-
lower Lias clay				
15	-	-	-	1.47
20	-	-	-	-
30	-	-	3.22	2.18
40	-	-	4.69	-
150	0.36	-	-	-
300	2.02	-	-	-
400	1.74	-	-	-

5.3 CHANGE IN WATER CONTENT

5.3.1 U100 Tubes

The results of the calculation of water content change and the measured water content are shown in Table 5.10. These can be seen to compare favourably. These measurements were taken adjacent to the pile, whereas the average values for the U38 samples taken as a whole were lower and are given in Table 5.10. This was particularly pronounced in the English china clay sample at a water content of 55 %, which also visibly produced the most condensation on its plastic covering.

These water contents were used to calculate strength changes, as discussed previously, using the method of Skempton and Northey (1953). Table 5.6 illustrates that a favourable comparison can be made between these predictions and the measured values.

Table 5.10 U100 water content changes

Clay Type	Initial Water Content %	Calculated Water Content due to slaking alone (%)	Calculated Water Content due to slaking and expansion (%)	Measured Water Content (%)
lower Lias clay	42	41.5	41	41
English china clay	35 45 55	34.5 44.5 54.5	34 43 53	34 43 53

5.3.2 Box Tests

In the initial trial in the small box samples were taken at varying distances from the pile at varying curing periods. These were tested for a number of properties, including water content. This information has been presented in Table 5.4.

A normal stress was applied to the large boxes via a water-bag arrangement which prevented an on-going programme of sampling as described above for the small box. Samples were therefore taken after a single curing period at different distances radiating from the pile and tested for water content. Table 5.11 summarises the results from all three large box arrangements, including curing period and the distance from the pile the sample was taken.

Table 5.11 Water content changes in large box arrangements

Arrangement	∅ Pile	ω start (centre)	Δ w % reduction		Comments
			50mm from pile	200mm from pile	
1	150C	36.9	4.8	4.7	measurements taken 70 days after pile placement
	150	37.3	5.6	4.2	
	100	36.9	4.9	3.0	
	100C	37.2	4.7	5.0	
3	150	35.6	-	0.7	measurements taken 25 days after pile placement
			1.2	+6.4	
			+3.1	+4.0	
			-	0.9	
3	150	38.2	2.5	-	absolute values are confusing as some are higher than original values
			1.6	2.3	
			+6.6	4.8	
			3.5	+6.5	
3	50	35.8	1.6	6.8	scatter is considerable values 50 and 200 mm from the pile are shown for illustration
			2.2	2.8	
			3.6	+0.2	
			+5.9	-	
3	50	36.5	0.8	+2.5	
			+8.6	0.4	
			2.0	+3.7	
			-	-	

Table 5.11 Continued

Arrangement	Ø Pile	ω start (centre)	Δ w % reduction			Comments
			row (1)	row (2)	row (3)	
2	100	40.9	5.6	5.9	6.6	pile placed in corner of box; readings taken after a curing period of 3 days
			4.1	5.3	5.9	
			4.5	5.0	-	
			4.0	-	7.4	
			3.0	5.6	5.3	
				5.0		
				3.4		
	50	41.2	3.9	4.2	5.0	not linear ∴ all pile material not reacted?
			3.6	3.9	4.3	
			3.6	3.7	3.8	
			3.8	4.2	3.6	
			4.2	4.4	4.0	
			3.8	4.8	4.6	
				-		
				+ 1.3		
			Position			datum water content may be too high as taken prior to application of normal stress all readings taken after 13 days curing
	100	42.0	(1)	7.9		
			(2)	8.4		
			(3)	8.2		
			(4)	9.0		
			(5)	8.5 - centre		
			(6)	8.0		
			(7)	8.8		
			(8)	8.4		
			(9)	8.7		

Notes:
Dashed entries refer to values which imply more than a 10% increase in water content

For arrangement 2 refer to Figure 5.35 for sampling positions. Where row numbers are referred to, readings are recorded in order of distance from the pile, ie the reading closest to the pile first etc.

For Arrangement 1 Table 5.11 shows the changes in water content both adjacent to the pile (50 mm away) and at a distance of 200 mm after a curing period of 10 weeks. Generally, from the limited data, it can be seen that the compacted piles produce a more extensive change in water content, i.e. the effect penetrates further across the box, whereas the uncompacted piles reduce the water content to a greater degree adjacent to the pile. There seems to be no discernible difference between the different size of pile.

The cracking in the clay surrounding the piles is illustrated in Figure 5.34. It can be seen that the larger, compacted pile produced the greatest degree of cracking, with the trend being that greater pile diameter or greatest compaction of the pile produces more cracking. This might be expected since the volume of lime is greater due to both effects. The localised effect of these cracks on water content was not recorded.

The water content values for Arrangement 2 have been used to create contours of water content surrounding the piles and are shown in Figure 5.35. It may be seen that the single piles produce a change in water content which radiates away from the pile producing a marginally greater and more penetrative change. The pile group appears to have produced a relatively even water content between the piles.

Cracking appears most marked in the group of 100 mm diameter piles where cracks form the boundary of a square block of material between the piles, as shown in Figure 5.36. This may account for the even distribution of water content between this boundary. The depths of the cracks for this group of piles have been plotted along sections between the piles and are shown in Figure 5.37.

The pattern of cracks around each pile is shown in Figure 5.38. Again, the larger piles produce larger, more penetrating cracking. Contours of water content have also been plotted for each box in Arrangement 3 and are shown in Figure 5.39. These show clearly the radial effect of water content reduction but indicate little difference between the effect of the larger piles and the smaller ones.

5.4 PILE STRENGTH

5.4.1 Box Tests

A summary of measured pile strengths for all box arrangements is given in Table 5.12.

Table 5.12 Pile strengths

Box Number	Pile Description (Diameter mm)	Water content (%)	Curing Period (days)	Strength (kPa)
1	∅ 100	36.9	70	450-500
2	∅ 100 single	41.2*	13	250
	∅ 50 single	40.9*	13	240
	∅ 100 x 4 group	42.0*	13	420
3	∅ 150	35.6	25	400
	∅ 150	38.2	25	430
	∅ 50	35.8	25	200
	∅ 50	36.5	25	290

*water contents taken prior to applying normal stress. All other values refer to samples taken after application of normal stress and cessation of change in pore water pressure.

5.4.2 Lime-PFA Mix Tests

A great deal of difficulty was experienced in forming 38 mm diameter triaxial samples of a consistent size and water content for lime-PFA mixes. These shortcomings are discussed in Chapter 6 along with suggested improvements in the test method.

5.5 PORE WATER PRESSURE CHANGES

5.5.1 Small Box

Manometer readings for the lime pile installed into the small box are shown in Figure 5.40. These show a slight increase in pore water pressure immediately after creating the pile, probably due to the compaction of the clay surrounding it caused as the pile was created by the excavation and compaction process. Thereafter the pressures drop, indicating water being drawn into the lime for slaking. However, after two weeks the pressures begin to rise again. A similar observation was recorded for a similar trial using Lias clay by Rogers and Bruce (1990).

One possible explanation for this unsustained reduction in pore water pressure was thought to be the level of reactivity of the lime itself. The lime used had been stored for a considerable length of time and it is possible that a significant proportion could have already slaked to calcium hydroxide. A quick investigation into the heat produced on the addition of water confirmed this suspicion.

To measure the scale of these effects the original pile was replaced by one containing fresh lime. Similar observations and tests were carried out on the remaining available clay. The results (Figure 5.40) confirmed the effect of fresh lime on the change in pore water pressure. Once again there was an initial rise due to pile installation, followed by a reduction of 12 kPa below the original datum. The pore water pressure thereafter fell further to reach -13 kPa 130 days after the fresh lime was installed.

5.5.2 Large Boxes

Arrangement 1 was the first to be studied and hence, with hindsight, some desirable data were not collected. The drawback with this experiment is that it takes a disproportionate amount of time to set up and, once the piles are placed, the clay cannot be re-used to supply further data. Nevertheless valuable information was gathered and the results from each separate box are presented in Figures 5.41 to 5.44. It can be seen that the results from the box containing the 150 mm diameter uncompacted pile (Figure 5.42) are dissimilar to the remaining arrangements and have been subsequently treated with caution and ignored in the description of trends. It also can be seen that the changes wrought by lime pile placement are less pronounced at the furthest distance from the pile, with similar behaviour shown at the upper and lower positions. The pore water pressures at the edge of the box were lower than those at the centre at the time of pile placement, due to the reduction in applied effective normal stress via the water bag at this position (as discussed in Chapter 6). The pore water pressures at the central positions reached values within the expected 3 kPa of one another (from consideration of the static head difference between the points of 300 mm), when considering fully saturated conditions.

Differences between pile types can be studied using Figures 5.45 to 5.48. These show the changes in pore water pressure occurring in boxes containing three different types of pile after their placement. No data were recorded for the period of pile placement: no effect has therefore had to be assumed. At the top of the box all three piles appear to respond in a similar manner. At the base of the box, the 100 mm diameter uncompacted pile produced the greatest response and the 150 mm diameter compacted pile produced the least response. The smaller, uncompacted pile responded to surface stress with excess pore water pressure at the base of the box. However, the larger, compacted pile appeared to respond to the same increase in surface stress by a reduction in pore water pressure.

These results will be discussed in detail in Chapter 6, although it is difficult with this limited amount of data to produce any conclusive results about the effect of pile size and compaction on pore water pressure.

Arrangement 2 constituted a final year undergraduate project. The student did not posses the knowledge gained from Arrangement 1 since the work was carried out simultaneously and whilst an attempt was made to improve experimental procedures, further errors, of a different nature, occurred. The primary reason for this was the need to allow the student freedom to undertake the work in his own way. The most significant of the failings of this work was the omission of a datum value for pore water pressure, all results being presented from one hour after pile placement. Hence, absolute values of the changes in pore water pressure cannot be calculated. These results cannot, therefore, be directly compared with those previously gathered. However, they may nevertheless be used in a qualitative manner to compare the results gained within the confines of this particular set up.

Results for the individual boxes are presented in Figures 5.49 to 5.52. These show changes in pore water pressure from one hour after pile placement. The readings from manometers placed in the corner of the boxes have been ignored as it was felt that they were not being fully stressed via the water bag for reasons explained earlier.

The single 100 mm diameter pile (Figure 5.49) showed no increase in pore water pressure due to pile placement at 75 mm and 125 mm from the pile from one to two hours, whilst at 230 mm an increase of approximately 0.5 kPa occurred over the same period. At 75 mm a maximum reduction in pore water pressure of 31.9 kPa was recorded after 26 hours; the other manometers recorded their lowest values after 3 days. After 2 days the manometers at 125 mm and 230 mm recorded pore water pressures of a similar value.

The single 50 mm diameter pile (Figure 5.50) produced reductions in pore water pressure which appeared as a 'double trough'. Here the closer to the pile, the faster the response to the change in pore water pressure. The value of pore water pressure recorded at 255 mm from the pile is greater after 2 hours than one hour, possibly implying that the pore water is still increasing at this point. The values recorded by the remaining manometers reduced between these times. A greater magnitude of reduction appears to be achieved at the furthest distance from the pile, but without datum values at the time of pile installation this cannot be taken as definitely the case. In all cases the reductions in pore water pressure produced were smaller than those produced by the larger (100 mm diameter) pile.

In the arrangement of four 50 mm piles (Figure 5.51), the manometer at the centre of the group gave a maximum reduction in pore water pressure of 14.6 kPa after 3 hours. This change is of similar magnitude to that obtained by the manometer at the same distance from a single pile of the same size except that the change occurred with a faster response time. The manometer reading that recorded the greatest effect from the combination of piles (centre, left) produced such a large negative pore water pressure that mercury was 'sucked' into the box in less than 24 hours which deemed the manometer inoperable for the remainder of the test. Similarly, the manometers in the box containing the four 100 mm diameter piles produced reductions in pore water pressure at the centre of the group similar to the value obtained at the same distance from the single 100 mm pile. Also the manometer at the centre left position was made inoperable and hence produced no further results.

The preparation of Arrangement 3 necessitated the removal and replacement of normal stress. It was observed that significant fluctuations in the pore water pressure readings occurred. Therefore, unloading and re-loading of the clay at the time of pile placement would also cause these problems and impair the results. The load-unload cycle was carefully studied prior to placing the piles in Arrangement 3, with the results presented in Figures 5.53 to 5.56. It may be seen that the response of the clay to unloading is to swell and hence produce reductions in pore water pressure of the order of 10 kPa. Reloading induces high positive pressures of the order of 20 kPa which subsequently dissipate. It was thought that this may be attributable to the localised system around the sand cell at the end of the manometer tube. When the system was unloaded there was a tendency for water from the sand cell to drain into the clay. Due to the permeability of the clay, little actual drainage took place. When the load was replaced, water from the clay was forced into the sand cell causing a positive pressure which would only dissipate as the excess water drained back into the clay.

Figures 5.53 to 5.56 also show that there is little difference in the pore water pressures generated across the horizontal section of the box, or in their response to lime pile treatment. Figure 5.57 shows the good agreement in results between the two boxes containing the 150 mm diameter piles. Figure 5.58 shows not such good agreement in the 50 mm diameter piles but on re-examination of the results from 50 mm (1) diameter pile, it seems likely these data should be viewed with some scepticism due to the erratic way in which the system may (or may not) reduce in pore water pressure on unloading. For these reasons the manometer readings 25 mm from the 150 mm (1) and 50 mm (2) piles in have been used to illustrate the remainder of the analysis.

Figure 5.59 shows good agreement in measurements prior to pile placement and similar subsequent response. However, on increasing the normal stress to 75 kPa the smaller pile produced the larger increase in pore water pressure. On further increase to 100 kPa, the smaller pile was the only one to respond.

The resulting pore water pressure changes presented are the actual measured readings at each point in time. The magnitude of the reductions caused by the piles have been calculated from the difference in the response pre and post pile placement to the addition of normal stress. This is discussed further in Chapter 6.

Overall, the maximum net reductions in pore water pressure produced by the piles under discussion were 21.5 kPa for the 150 mm diameter pile and 17 kPa for the 50 mm diameter piles. This accounts for the response of the clay alone to the load-unload cycle.

Within 200 hours of pile placement, the smaller pile had resumed its original pore water pressure, whilst the larger pile was 2.5 kPa below the original. However, on removal of the load, the boxes containing the 50 mm diameter piles produced reductions in pore water pressure of a similar order to that observed previously. The boxes containing the larger piles remained at the same value of pore water pressure even after a significant period of time. This could indicate that the tubing became blocked after the final loading stage.

The problems associated with these experiments are evident and are discussed further in Chapter 6, along with suggested improvements.

5.5.3 The Effect of PFA

In all cases measurements have been taken 25 mm from the edge of the pile. In the box with the lime only pile (box 1, Figure 5.60) manometers 3 and 4 have been ignored due to excessive ingress of air into the tube. Similarly, manometer 5 in the box containing the lime-PFA pile (box 2, Figure 5.60) has also been ignored.

The results from all manometers are shown in Figures 5.61 and 5.62. This clearly shows a reduction in the pore water pressure response caused by the addition of PFA, i.e. the reduction in pore water pressure produced by the piles is less pronounced.

5.6 TEMPERATURE CHANGES

5.6.1 Box tests

The results from all four boxes in Arrangement 1 are illustrated in Figures 5.63 to 5.66. A summary of key points arising from the graphs is given in Table 5.13. From the limited data it may be stated that all thermocouples registered a rise in temperature within one hour of pile placement with the most pronounced peaks of change occurring adjacent to the piles. The compacted piles produced a greater increase in temperature adjacent to the pile whereas the uncompacted piles produced a temperature rise which penetrated the horizontal section to a greater extent and over a longer duration.

The larger piles produced a higher temperature rise of longer duration, with a greater difference in the duration of the temperature rise across the box, when compared with the smaller pile.

Results from each individual box in Arrangement 2 are shown in Figures 5.67 to 5.70 with salient points included in Table 5.13. It may be seen that in general the temperature rise decreases with distance from the pile and that the larger piles gave greater rises in temperature. If a comparison is made between these results and the previous arrangement it may be seen that, for the 100 mm diameter pile and at similar distances from the pile, a lower rise in temperature is recorded with a longer time to peak. This may be attributable to the water content of the clay and will be discussed in Chapter 6.

Table 5.13 Temperature changes in large box arrangements

∅ Pile	Arrangement Number (C signifies compacted)	Distance (mm)	Max ΔT (°C)	time to maximum (hours)	time to ambient (hours)
150	3	25	20.3, 1.1	3.5	90
	1 (C)	50	15.5	7	80
	1	50	11	5.5	100
	3	125	10.8, 6.4	8, 6.5	90, 80
	1	200	8.5	10	75
	1	200	5.5	9	95
100	1 (C)	50	10	7.5	78
	1	50	9	6.5	100
	2	75	6.5	6	72
	2	125	4.5	14	72
	1 (C)	200	2	10	75
	1	200	4.5	9	95
	2	230	3.8	5	72
50	3	25	3.8	9·5	82
	3	75	1.6	9	63
	2	100	2.3	8	72
	3	125	2, 0.8	9, 11	79
	2	150	2.5	16	72

Note

Dual entries relate to more than one box with the same size pile.

The results from Arrangement 3 are shown in Figures 5.71 to 5.74 with salient features in Table 5.13. The results for the 150 mm diameter pile are comparable with those obtained from the compacted pile in Arrangement 1 where clay water contents were similar. Table 5.13 combines the results of peak temperature (temperature and time taken to achieve that temperature) for both tests. The value for 75 mm in Arrangement 3 (Figure 5.74) appeared rather high and on excavation of the clay in the box this thermocouple was found to be at the same location as the 25 mm thermocouple. Obviously disturbance during compaction and pile placement disturbs the precise positioning of the instrumentation.

The smaller diameter pile produced a much smaller rise in temperature with no clearly defined peak. The time taken for that rise in temperature to be recorded was much longer, although the duration of the elevated temperature was comparable. There was still a noticeable difference in the temperature rise with distance from the pile.

If a comparison is made between the 50 mm diameter piles in the previous box at similar distances from the pile, similar rises in temperature have been recorded although the time to peak is shorter. This may be attributable to increased water content and will be discussed in Chapter 6.

5.6.2 Perspex Tubes

An observation made during the perspex tube experiments was that a noticeable temperature rise was felt in the wetter English china clay samples, with no such observation being made in the remaining samples. This would indicate that the wetter, more porous samples allowed the slaking reaction to proceed more quickly and hence produce the rapid rise in temperature.

5.7 MINERALOGICAL CHANGES

Samples from the mix test and perspex tube programmes were tested using Infra-red spectroscopy and X-Ray diffraction methods to establish the mineral content of the clays used in the experimental programme and to study the changes wrought by the lime-clay reaction. Comparison between the results from samples reacted with intimately mixed lime were compared with those reacted with migrating lime.

5.7.1 Mix Tests

Figure 5.75 shows part of the Infra-red spectra for English china clay alone and after reaction with 1, 3 and 5% lime. The traces have been overlain to show the region of hydroxyl stretching from 4000-3000 cm^{-1} when tested at room temperature for each condition. English china clay alone shows a pattern of absorption typical of well crystalline kaolinite; two strong, sharp bands near 3697 and 3620 cm^{-1} together with two intermediate, weaker sharp bands near 3650 cm^{-1}. It also shows a broad, medium intensity band near 3450 cm^{-1} due to adsorbed water which increases in intensity as the lime level increases from 1 to 5 %. After heating to 150 ^{0}C (Figure 5.75) to remove adsorbed water the lime-treated samples still show enhanced adsorption near 3450 cm^{-1} compared to the untreated clay. The residual hydroxyl absorption may be due to an amorphous phase formed by reaction of the English china clay and the lime. The 3 and 5 % traces are identical perhaps suggesting that an end point may have been reached in the reaction, a maximum amount of amorphous phase has been produced by the reaction with 3% lime.

Another interesting aspect of this clay-lime system is the development of the calcium carbonate formation with increasing lime content. Figure 5.76 shows the spectra between 300 and 1800 cm^{-1} for English china clay alone and after reaction with 1, 3 and 5% lime off-set on the vertical scale for purposes of clarity. 1 % lime shows the presence of absorption bands at 1432 and 878cm^{-1} (Figure 5.76) which increase in intensity in the 3 % lime sample. The two bands are due to the presence of calcite. When the lime content is increased to 5 % the calcite peaks are still present but the development of two new bands at 1476 and 860 cm^{-1} indicates the presence of aragonite, a less stable crystalline form of calcium carbonate.

Infra-red spectra of the hydroxyl stretching region of Wyoming bentonite run at room temperature and 150 ^{0}C (Figure 5.77) show the presence of a strong, broad band near 3630 cm^{-1} typical of dioctahedral smectites, together with a medium intensity broad band near 3430 cm^{-1} due to the presence of adsorbed interlayer water in the swelling clay. As the time of the reaction increases from 7 to 28 days with 4 % lime the absorption band due to water also increases. After heating to 150 ^{0}C there is still residual absorption near 3430 cm^{-1} which increases with the reaction time and may again be due to the formation of an amorphous phase.

The calcium carbonate development is somewhat more variable than in the English china clay system. After 7 and 14 days both calcite (1421 and 879 cm^{-1}) and aragonite (1479 and 859 cm^{-1}) are present (Figure 5.78). Aragonite is dominant after 28 days.

5.7.2 Perspex Tube Tests

The major mineral components of lower Lias from the perspex tube test programme were: disordered kaolinite, illite, quartz and calcite with some gypsum. After treatment with lime the Gypsum component was much reduced and there was some indication of an amorphous phase being present. This conclusion was based on the appearance of broad absorption in the hydroxyl stretching region close to 3300 cm^{-1}. These observations were backed up by the XRD spectra, see Figures 5.79 and 5.80

The English china clay used in the perspex tube tests was found to be a reasonably pure mineral consisting of well-crystallised kaolinite with only a little quartz. After treatment with lime some modification of the kaolinite had occurred. Again there was evidence of the presence of an amorphous phase based on a broad absorption near 3300 cm^{-1}. The amount of this phase was comparatively significant and probably reflects the susceptibility of kaolinite to dissolution in strong alkali (see areas for further investigation). Some calcite was identified, presumably derived from the lime component, see Figures 5.81 and 5.82

Wyoming bentonite was found to be another relatively pure sample with only a small amount of quartz and an amorphous phase of silica (XRD suggests cristobilite). After treatment with lime there was again some evidence of an amorphous phase: a broad band near 3300 cm^{-1} was visible after heating the disc to 150^{0}C. The heating process, designed to remove adsorbed water, showed an apparent increase in water content of the treated sample. It was suggested that this may have been due to the occurrence of ion exchange. However, XRD analysis showed the treated sample still to be entirely sodium saturated. Again, the presence of lime in the treated sample was indicated by the presence of calcite, as shown in Figures 5.83 and 5.84

5.8 CONCLUSIONS

The experimental programme has provided a great deal of information regarding the various stabilising mechanisms associated with the quicklime pile treatment of failing slopes. This has been achieved through a variety of experimental procedures from the large box modelling to the small perspex tubes.

Because of the complex nature of the stabilising mechanism as a whole it has not been possible to carry out experiments to completely define each contributory factor. Rather it is felt a considerable achievement to have isolated the stabilising mechanisms and gone some way to study each one. Further discussion of the results in Chapter 6 will explore the significance of the data to a greater degree and lead the way to demonstrating how the data obtained may be used in the design of lime pile installations (discussed in Chapter 7).

It is felt overall that in attempting to study all of the contributing stabilising mechanisms, a completely definitive experimental procedure was never developed. Compromises had to be made in order to meet realistic time constraints. Nevertheless it is felt that this has encouraged ingenuity and has produced a well rounded engineering study.

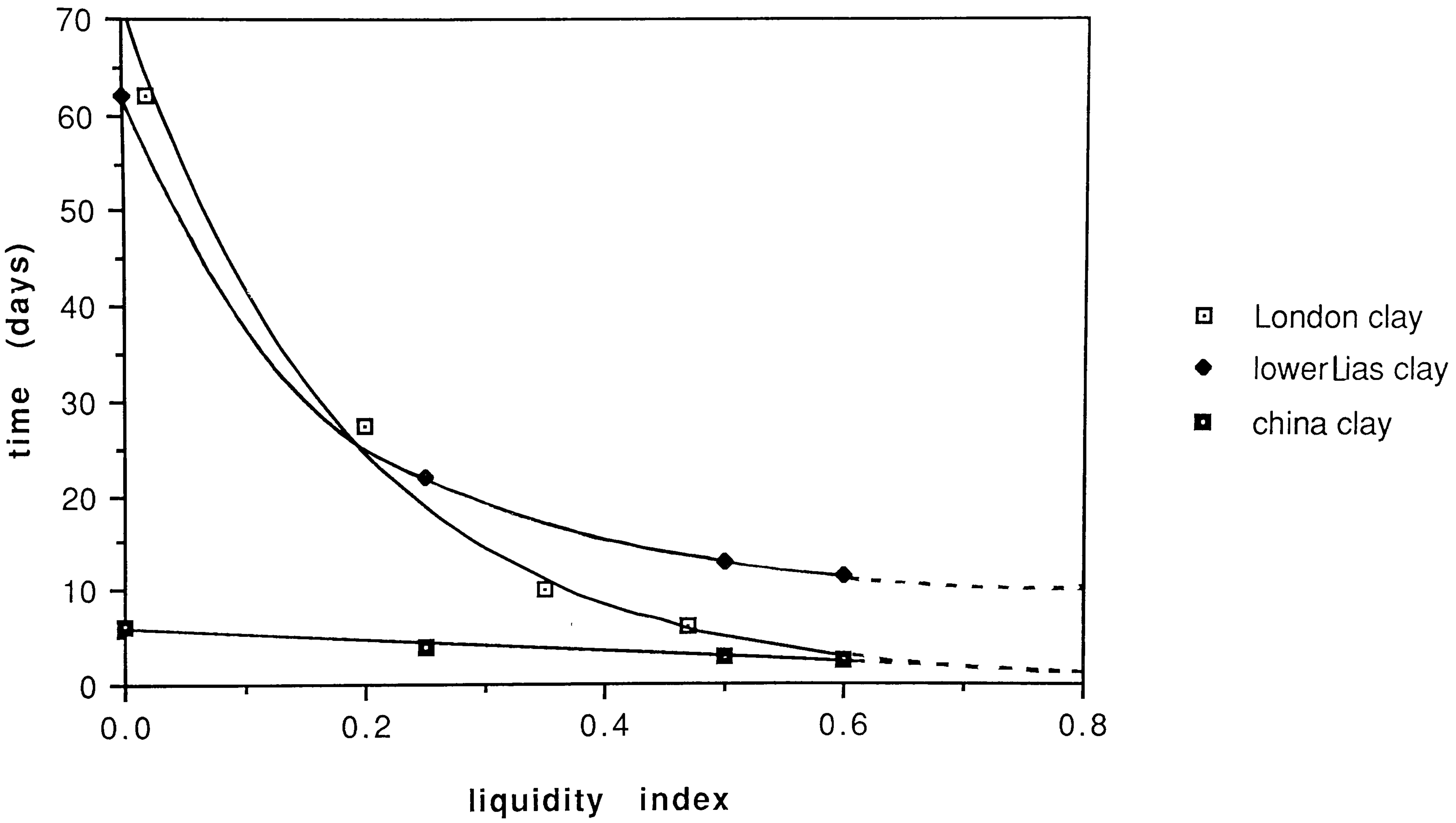


Figure 5.1 Perspex tube tests - migration time against Liquidity Index

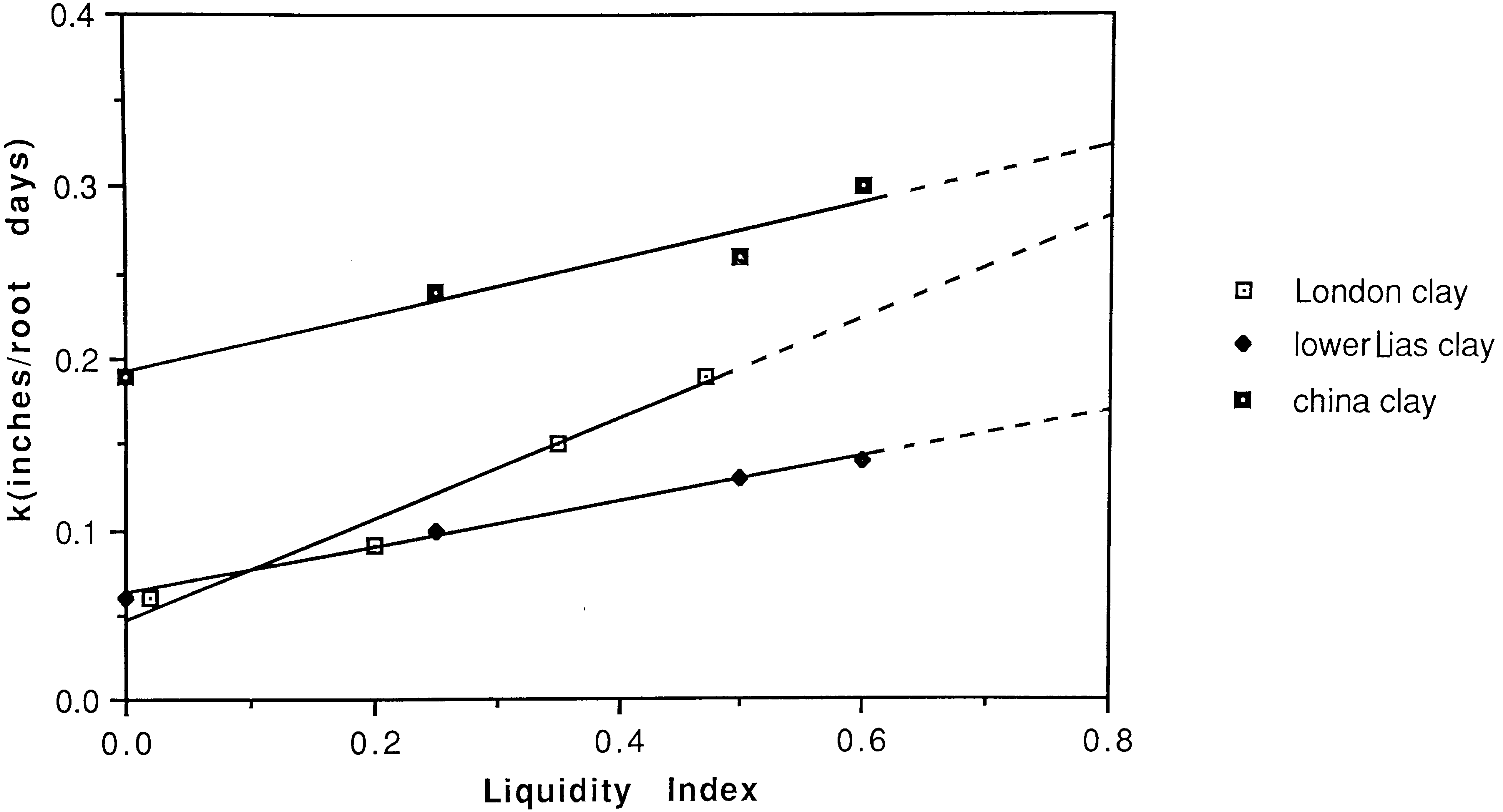


Figure 5.2 Perspex tube tests -
migration constant k against Liquidity Index

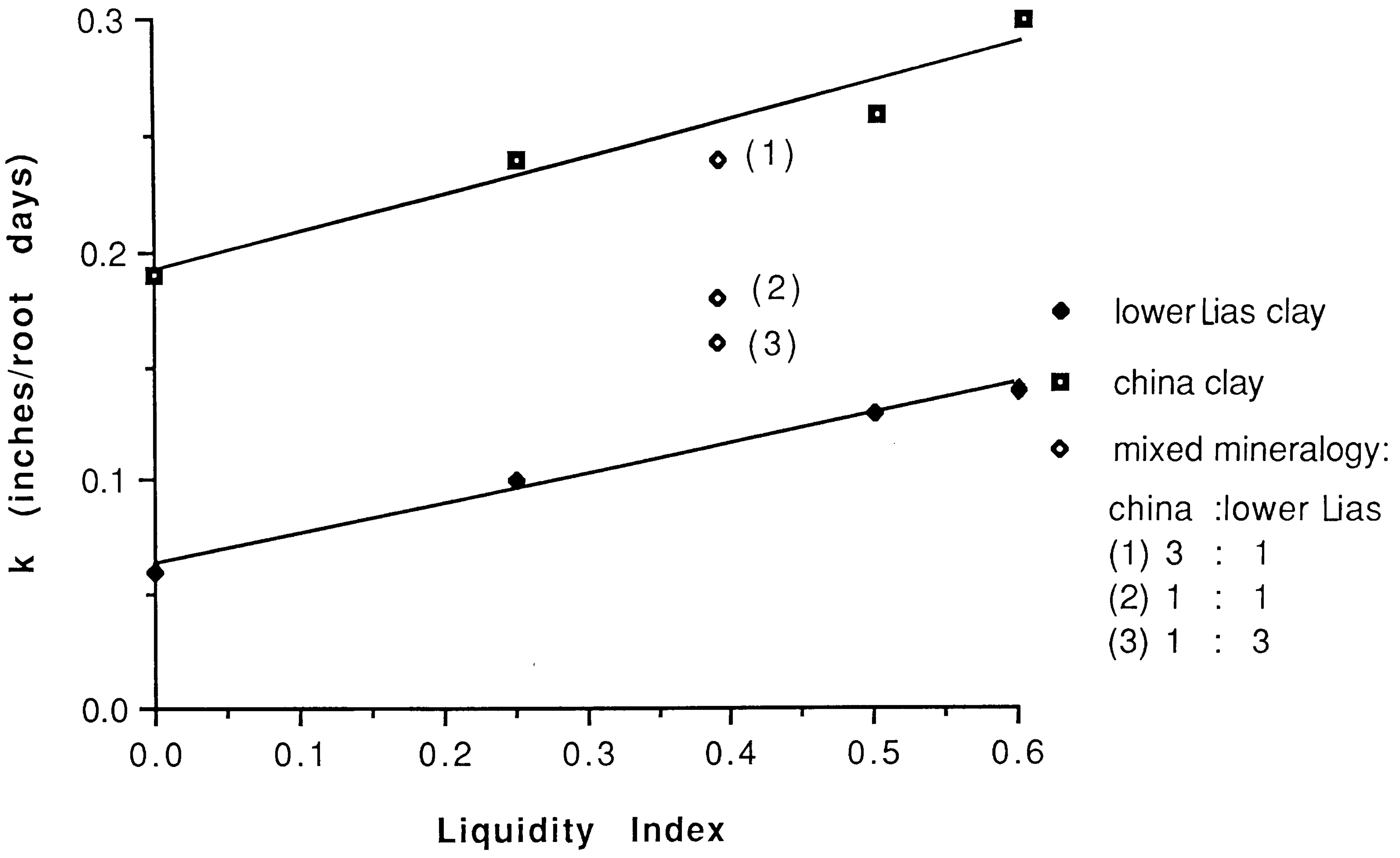


Figure 5.3 Perspex tube tests - effect of mineralogy

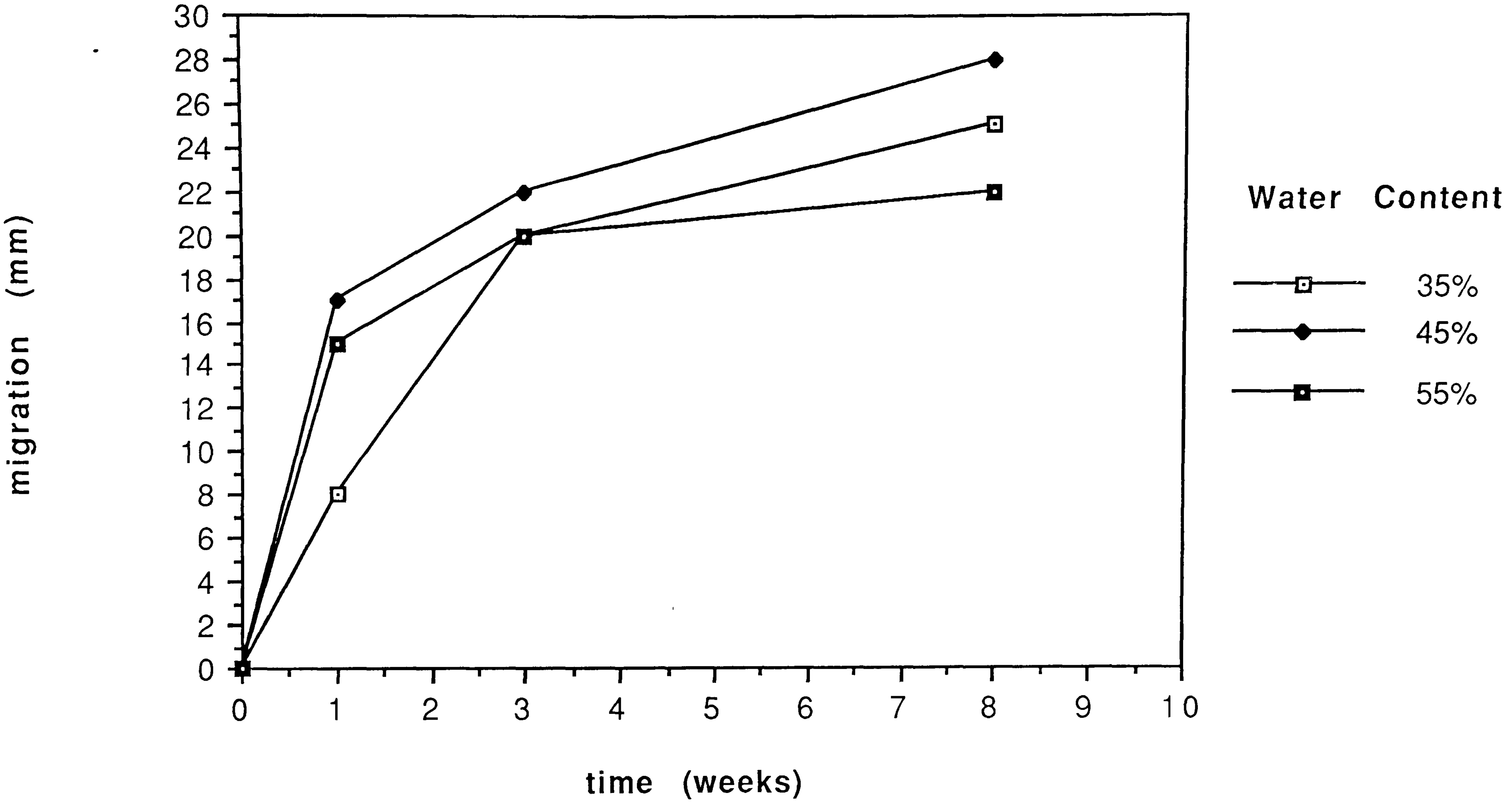


Figure 5.4 U100 tests on English china clay - migration distance against time

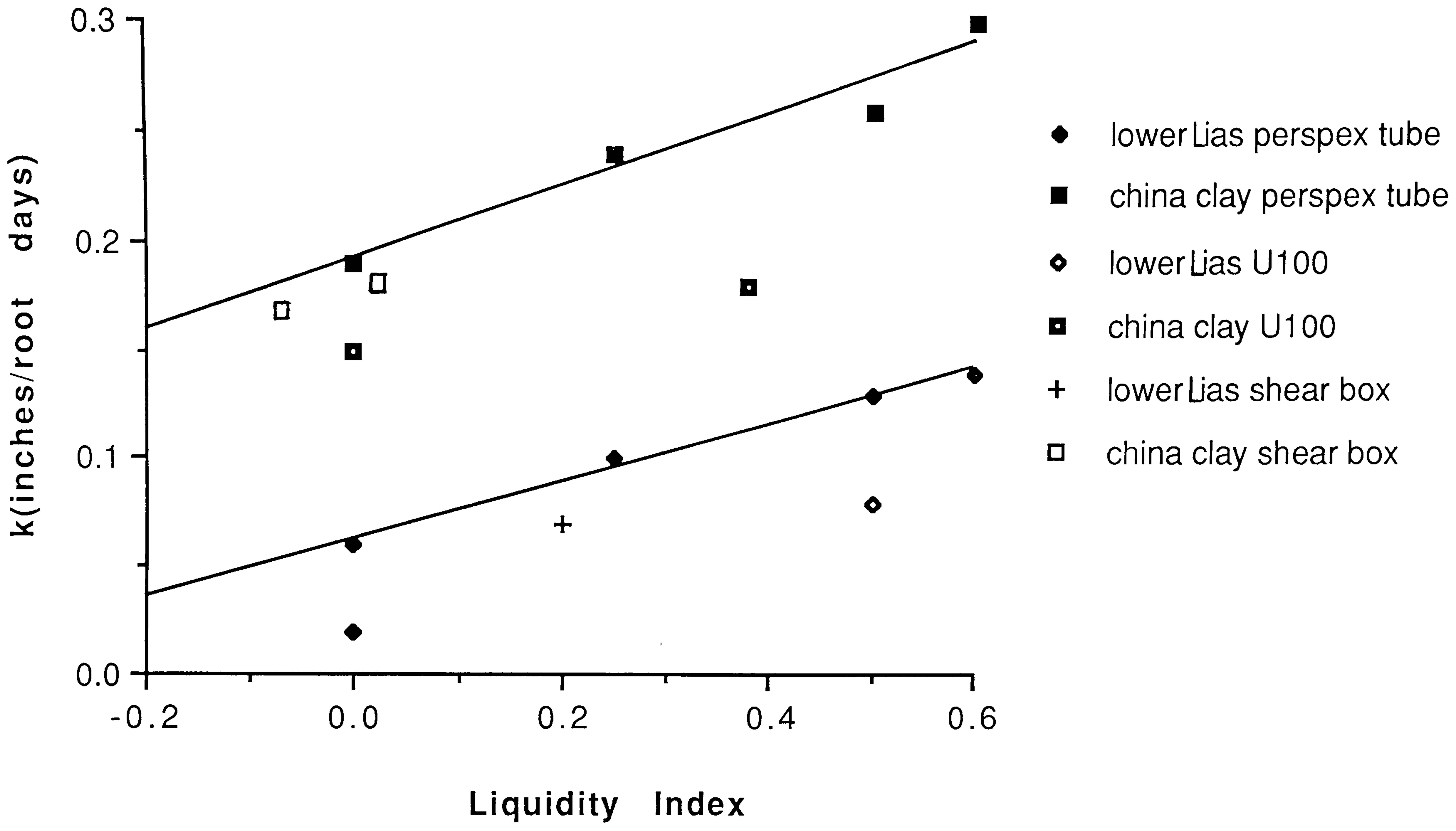


Figure 5.5 Comparison of migration in U100 and perspex tube tests

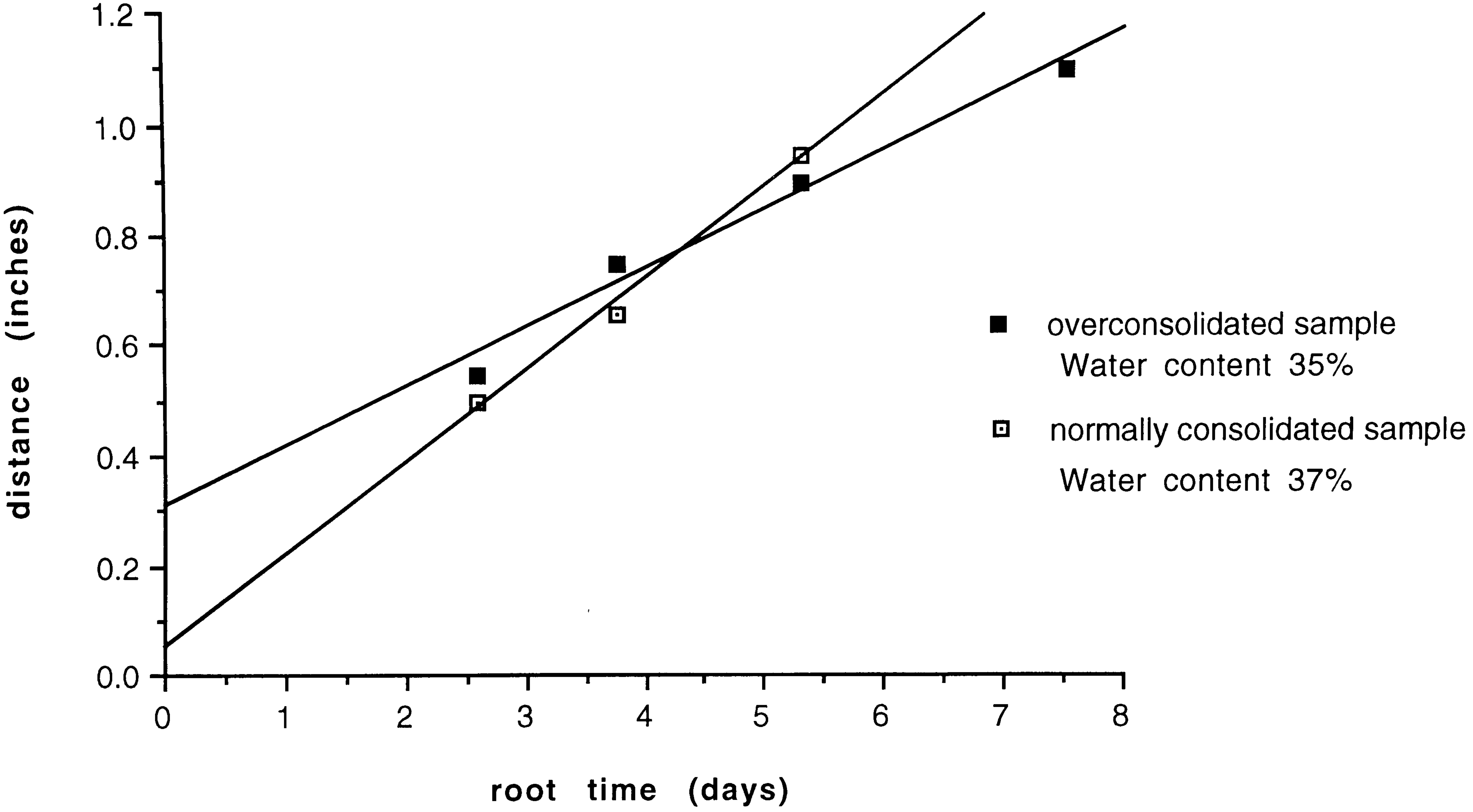


Figure 5.6 Migration rates in shear box samples

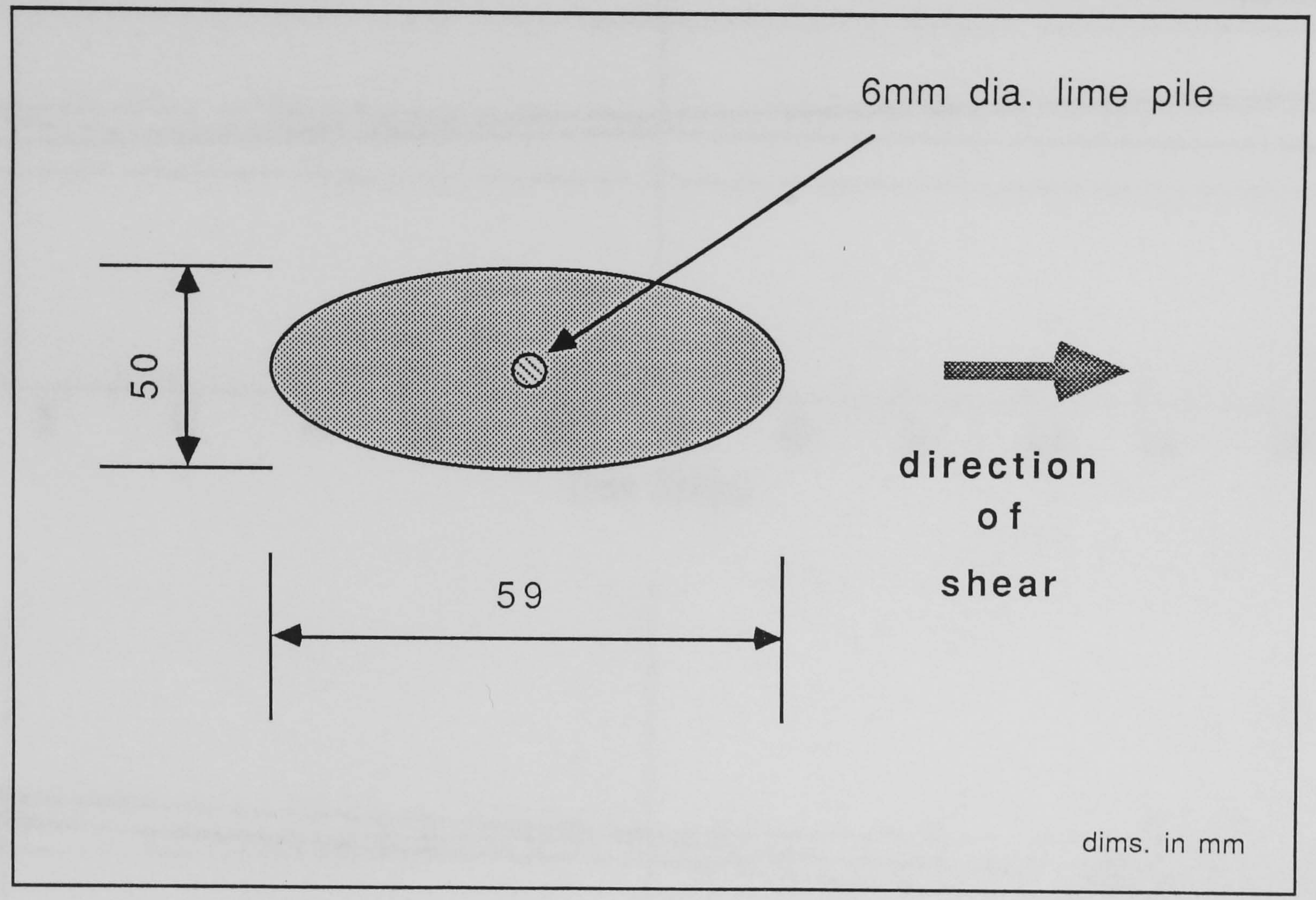


Figure 5.7 Differential migration in shear box samples

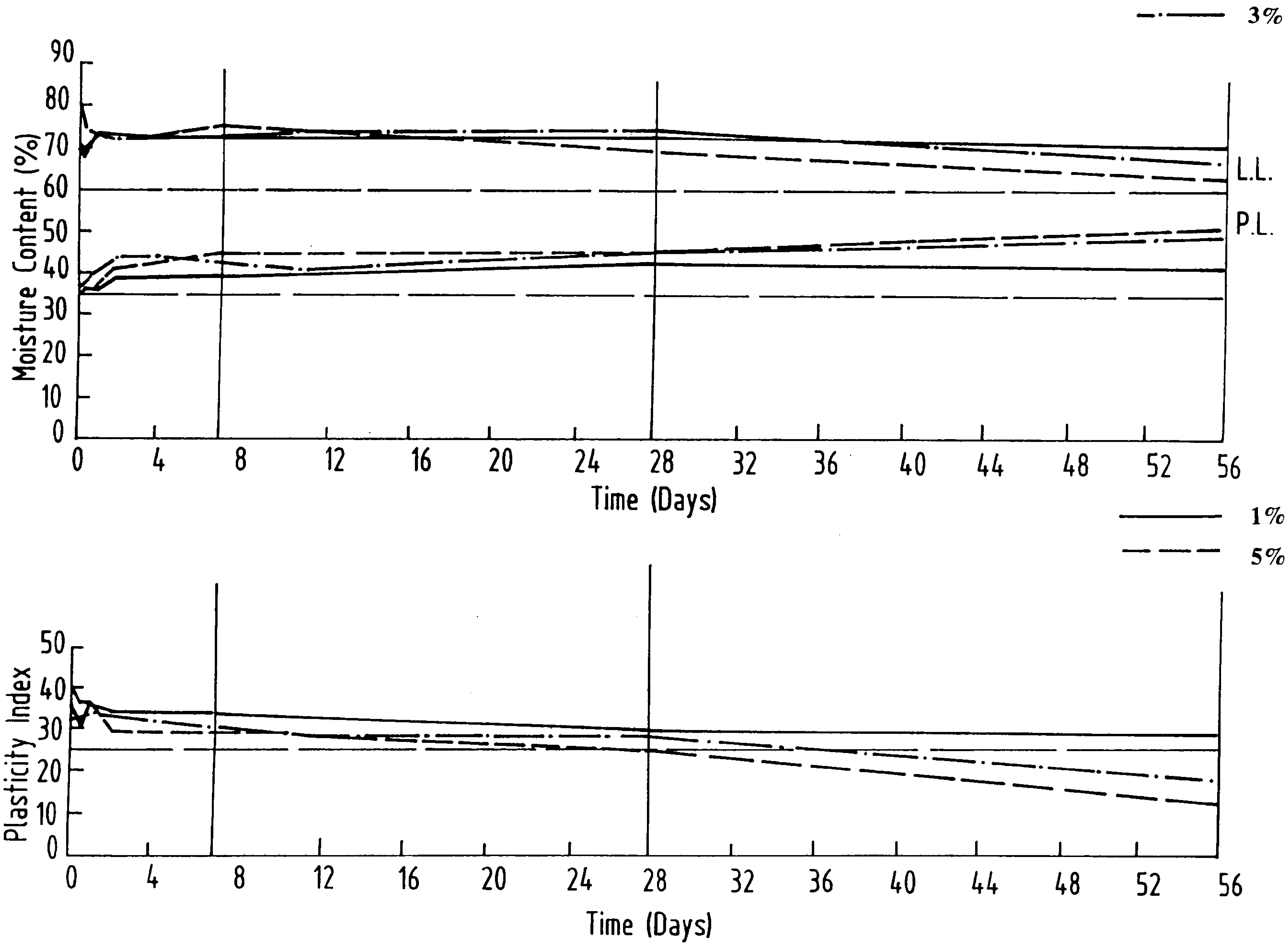


Figure 5.8 Atterberg Limits, English china clay plus 1, 3 and 5% quicklime

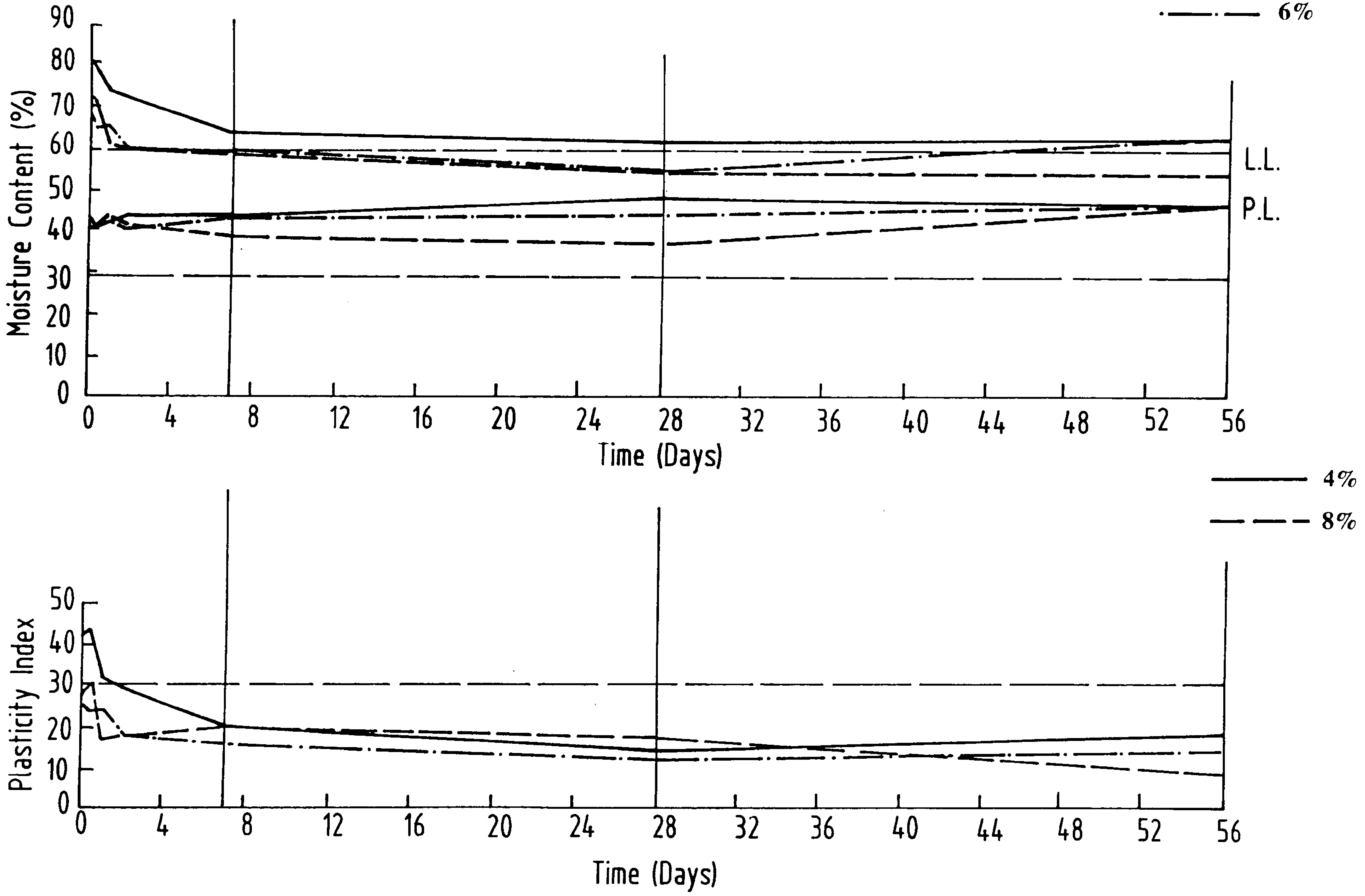


Figure 5.9 **Atterberg Limits, lower Lias clay plus 4, 6 and 8% quicklime**

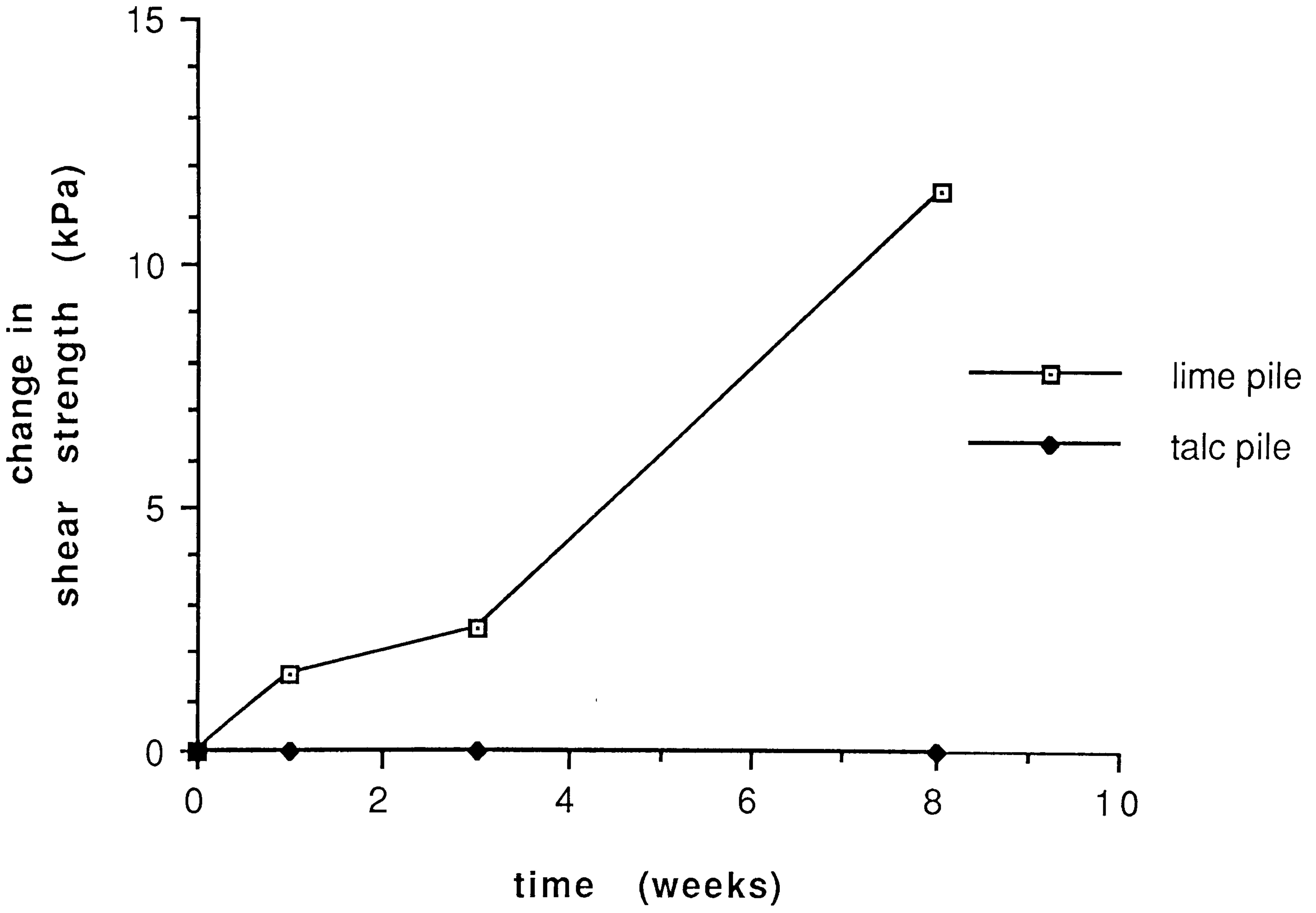


Figure 5.10 Strength development in U100 samples of lower Lias clay

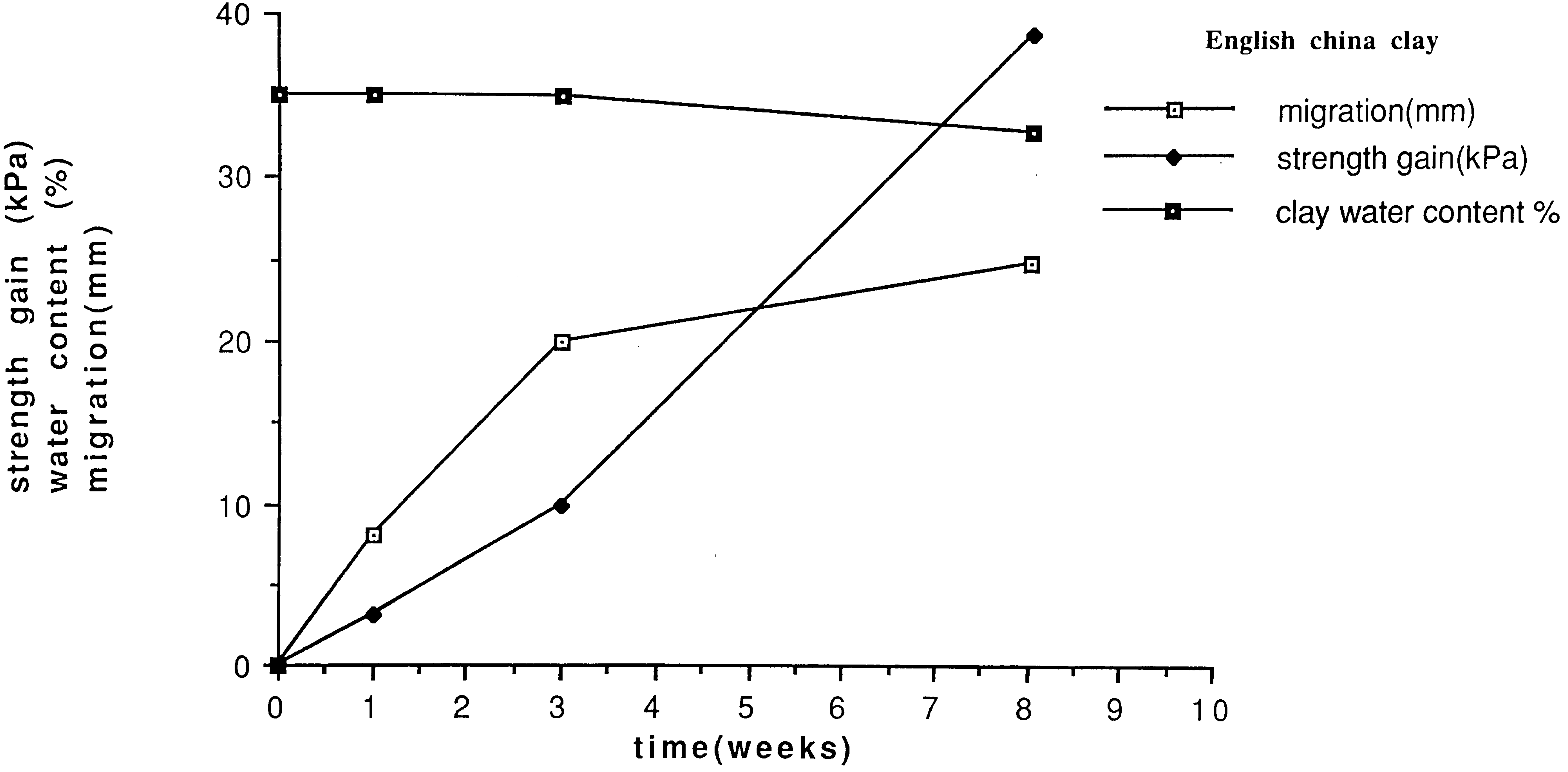


Figure 5.11 U100 tests - changes in water content, strength and migration - 35% sample

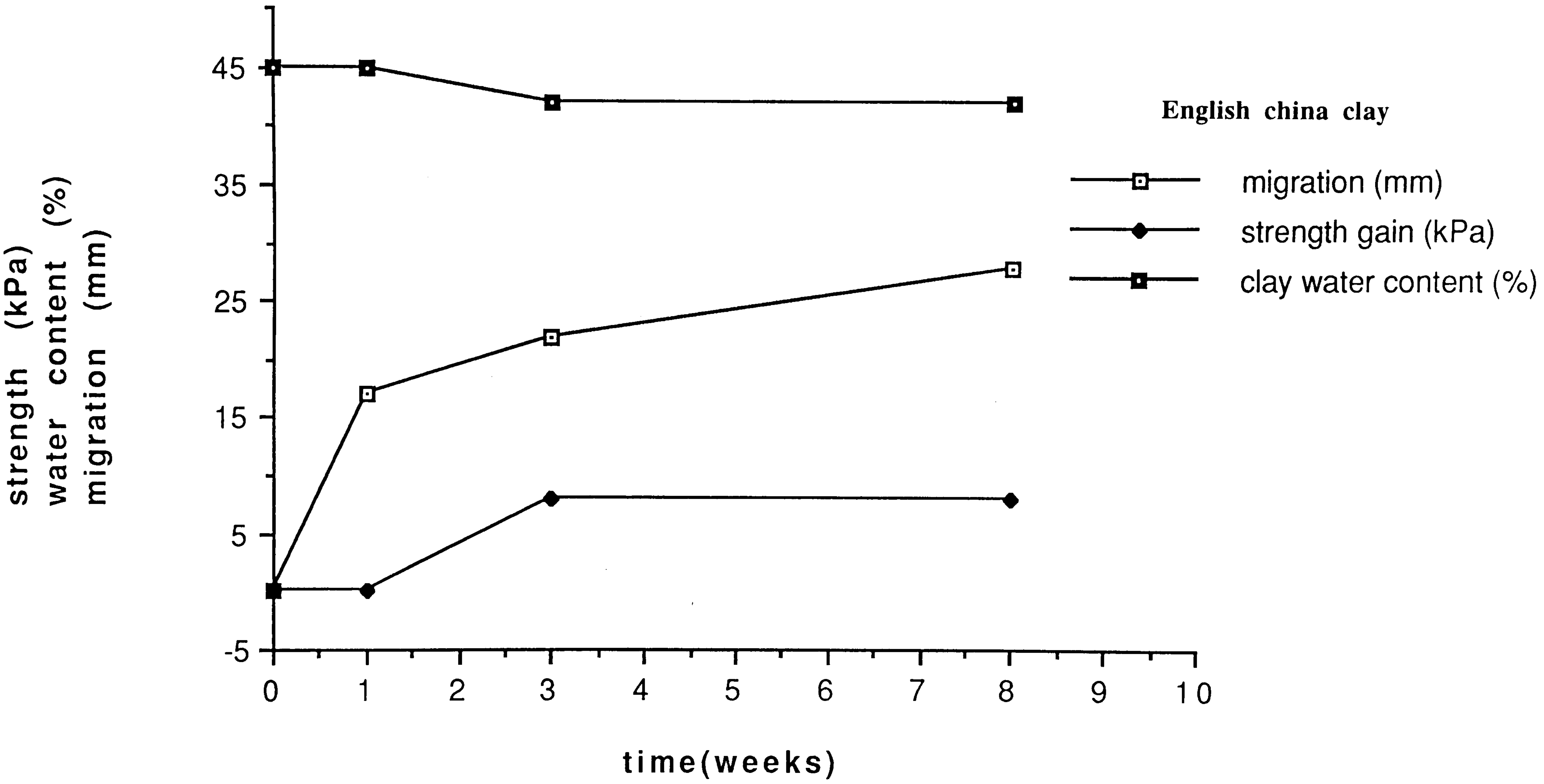


Figure 5.12 U100 tests - changes in water content, strength and migration - 45% sample

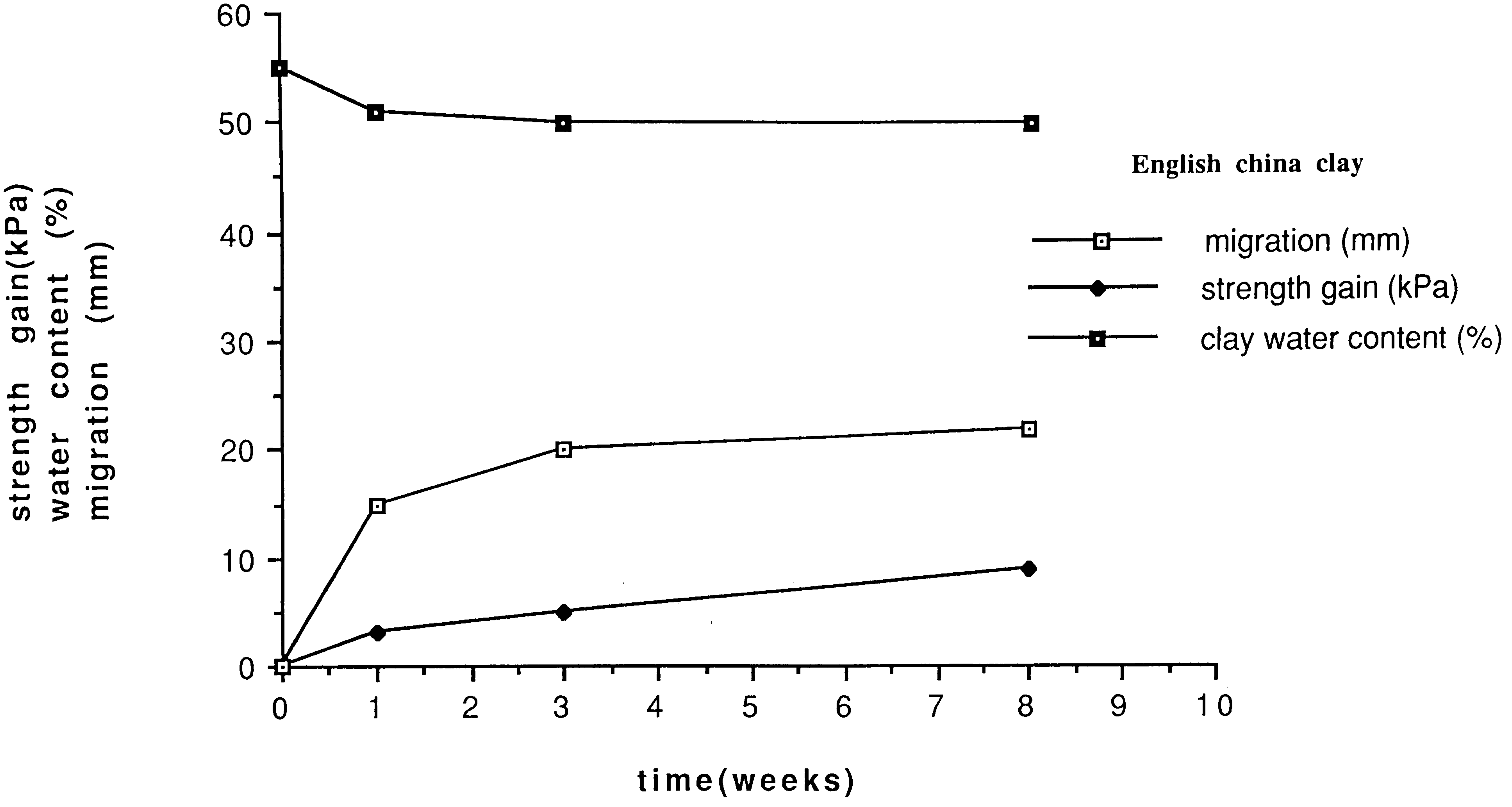


Figure 5.13 U100 tests - changes in water content, strength and migration - 55% sample

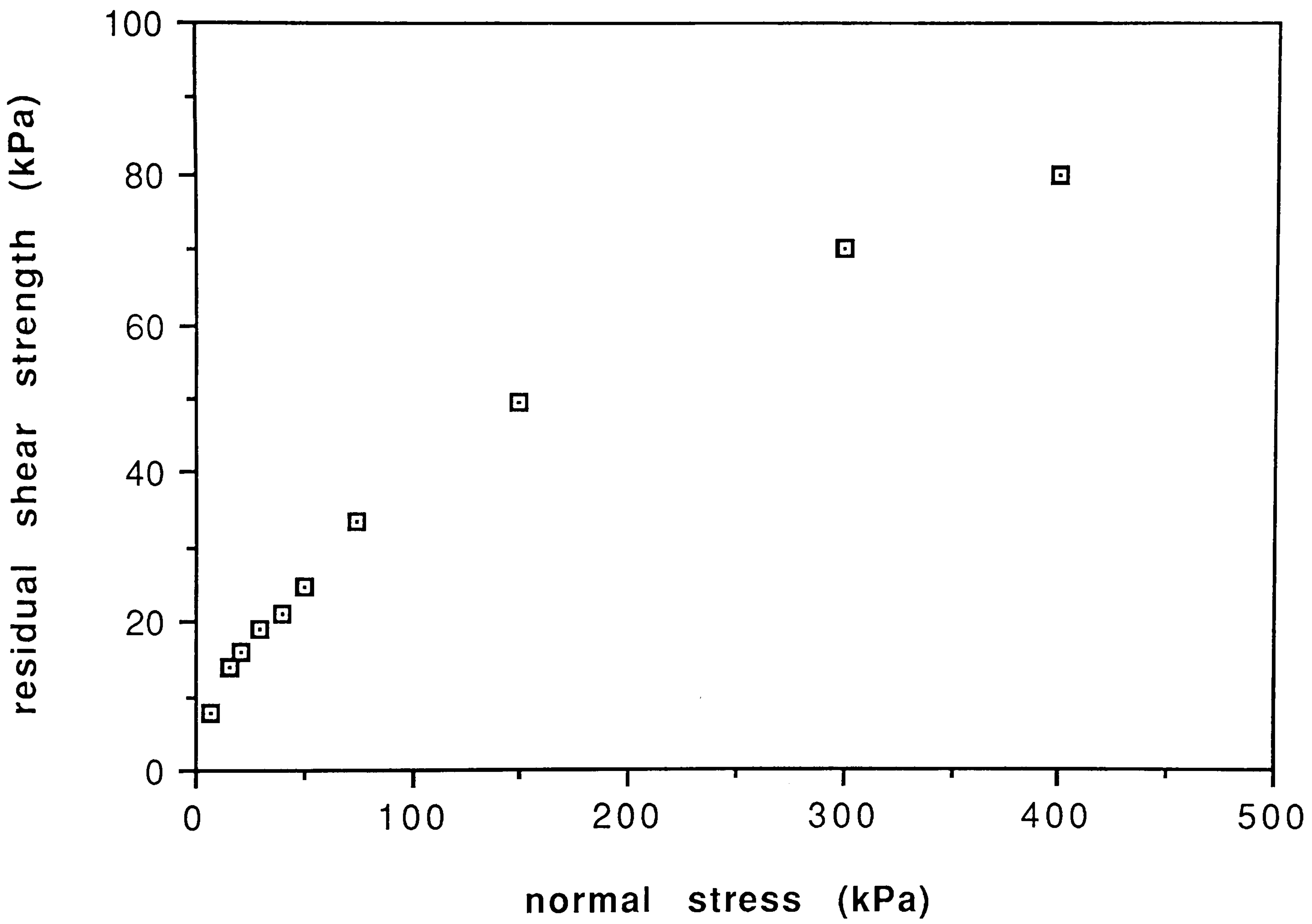


Figure 5.14 China clay residual strength envelope

Figure 5.15 Effective consolidation pressure 300 kPa, OCR 20 - complete plot of final travel showing stoppage and overconsolidation

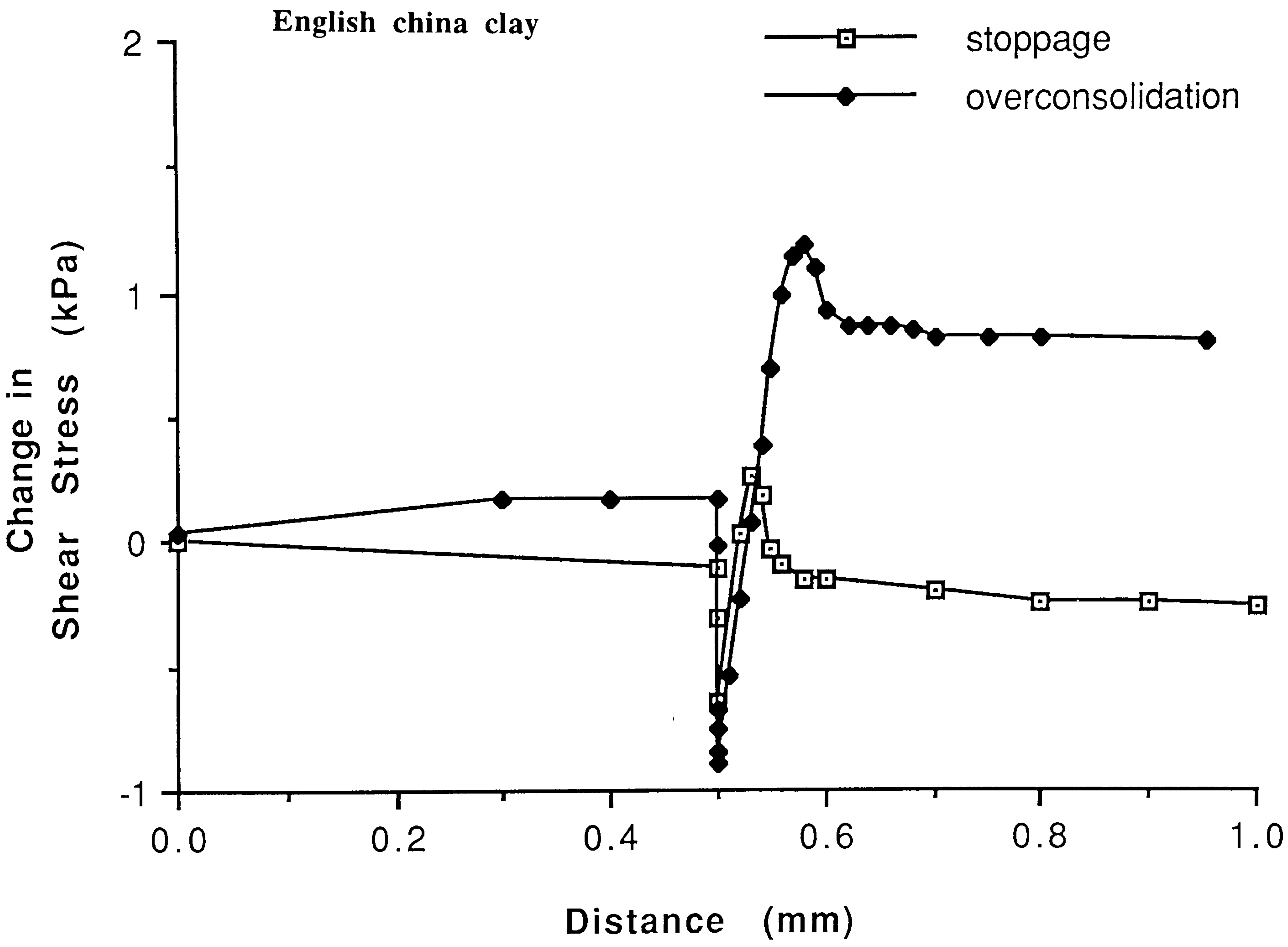


Figure 5.16 Effective consolidation pressure 300 kPa, OCR 20 - effects of stoppage and overconsolidation on residual strength

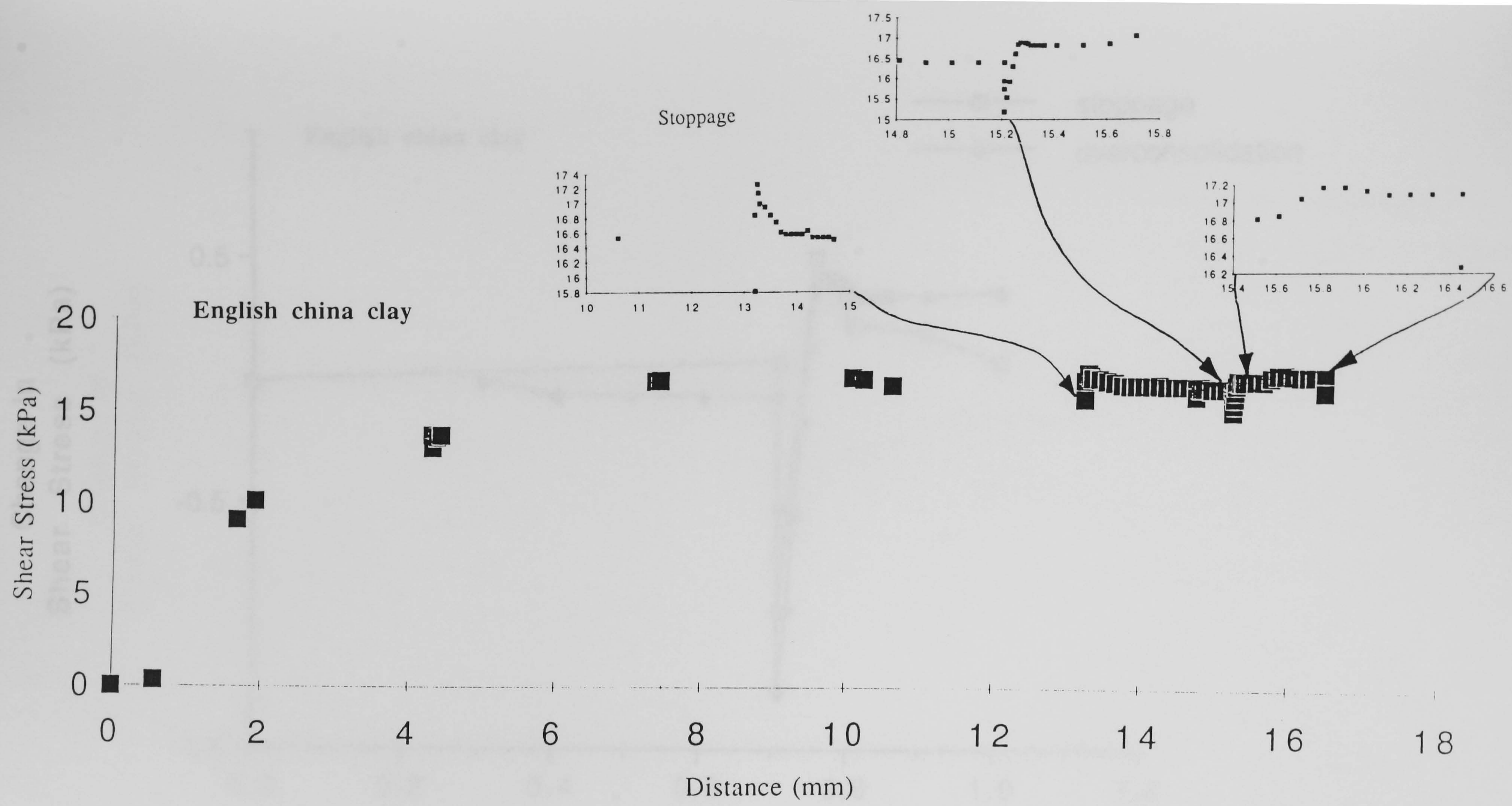


Figure 5.17 Effective consolidation pressure 300 kPa, OCR 10- complete plot of final travel showing stoppage and overconsolidation

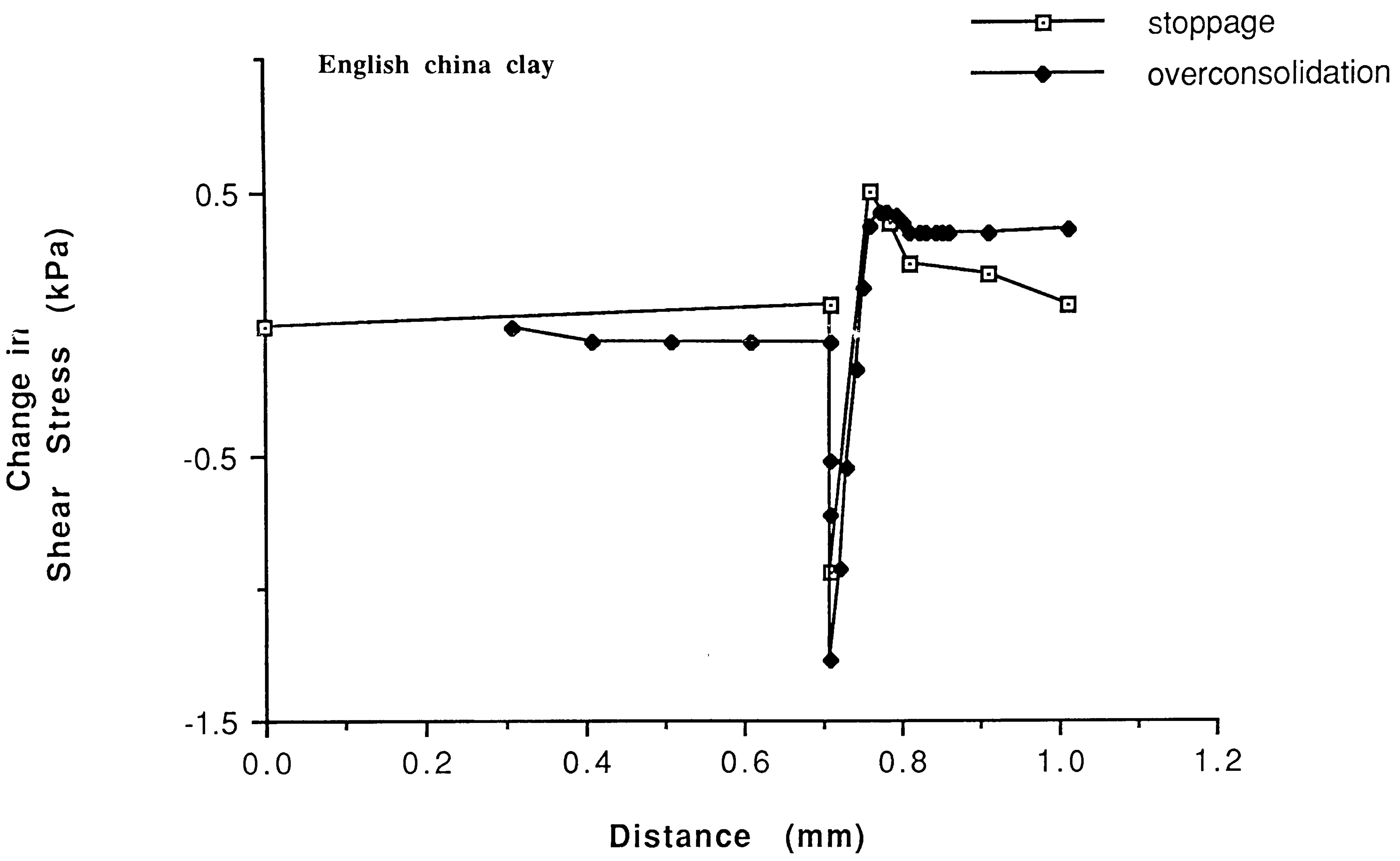


Figure 5.18 Effective consolidation pressure 300 kPa, OCR 10 - effects of stoppage and overconsolidation on residual strength

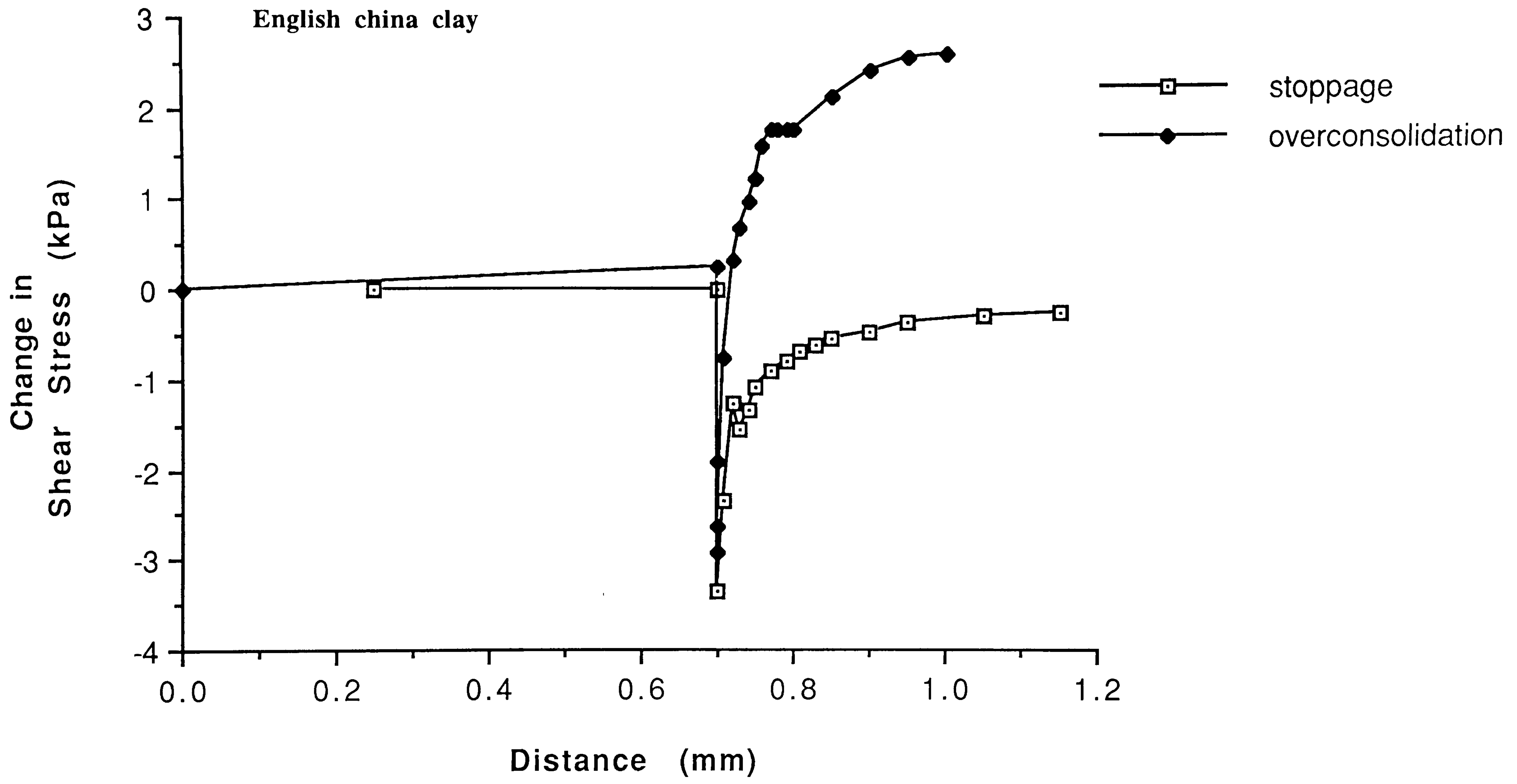


Figure 5.19 Effective consolidation pressure 300 kPa, OCR 1 - effects of stoppage and overconsolidation on residual strength

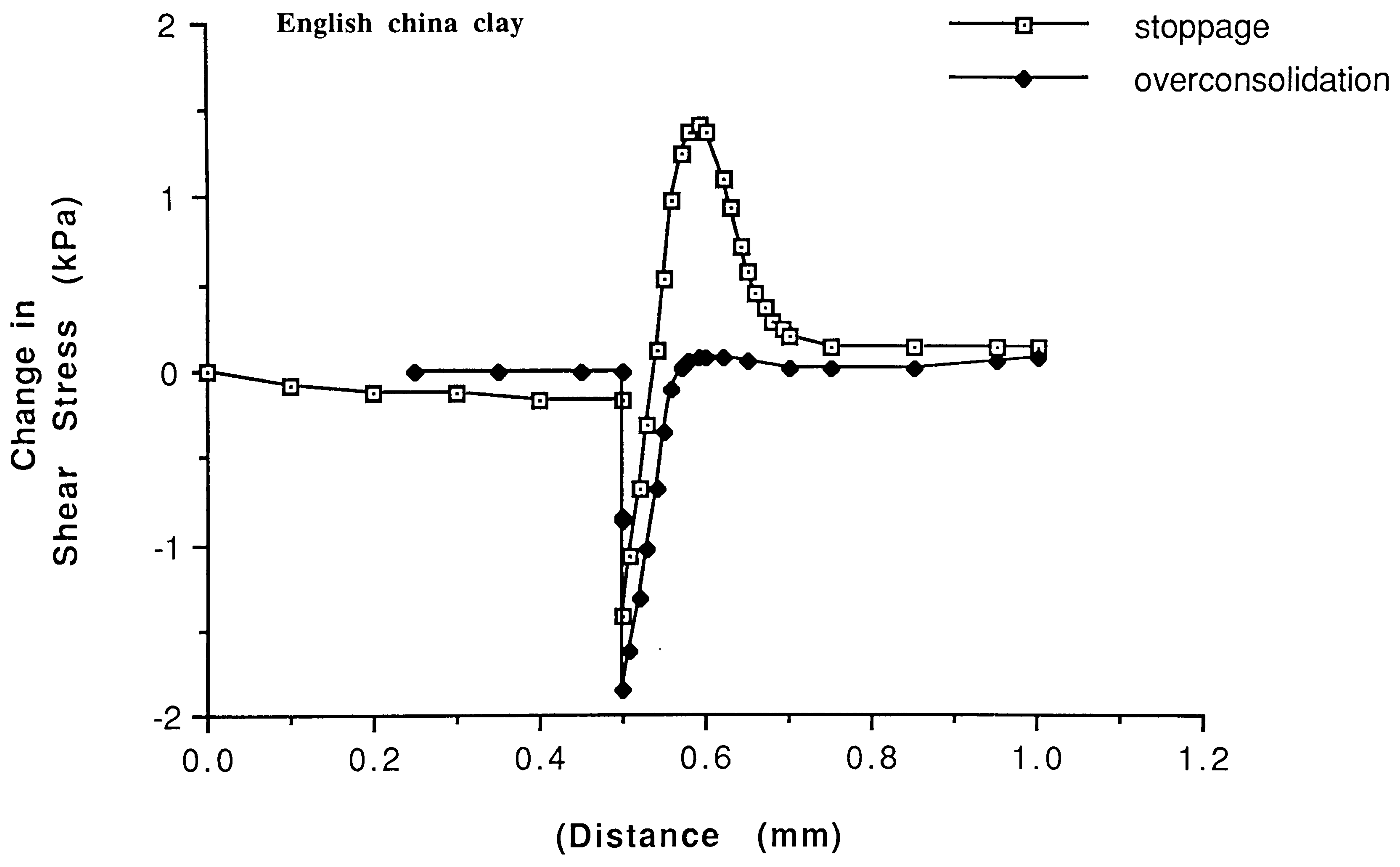


Figure 5.20 Effective consolidation pressure 150 kPa, OCR 10 - effects of stoppage and overconsolidation on residual strength

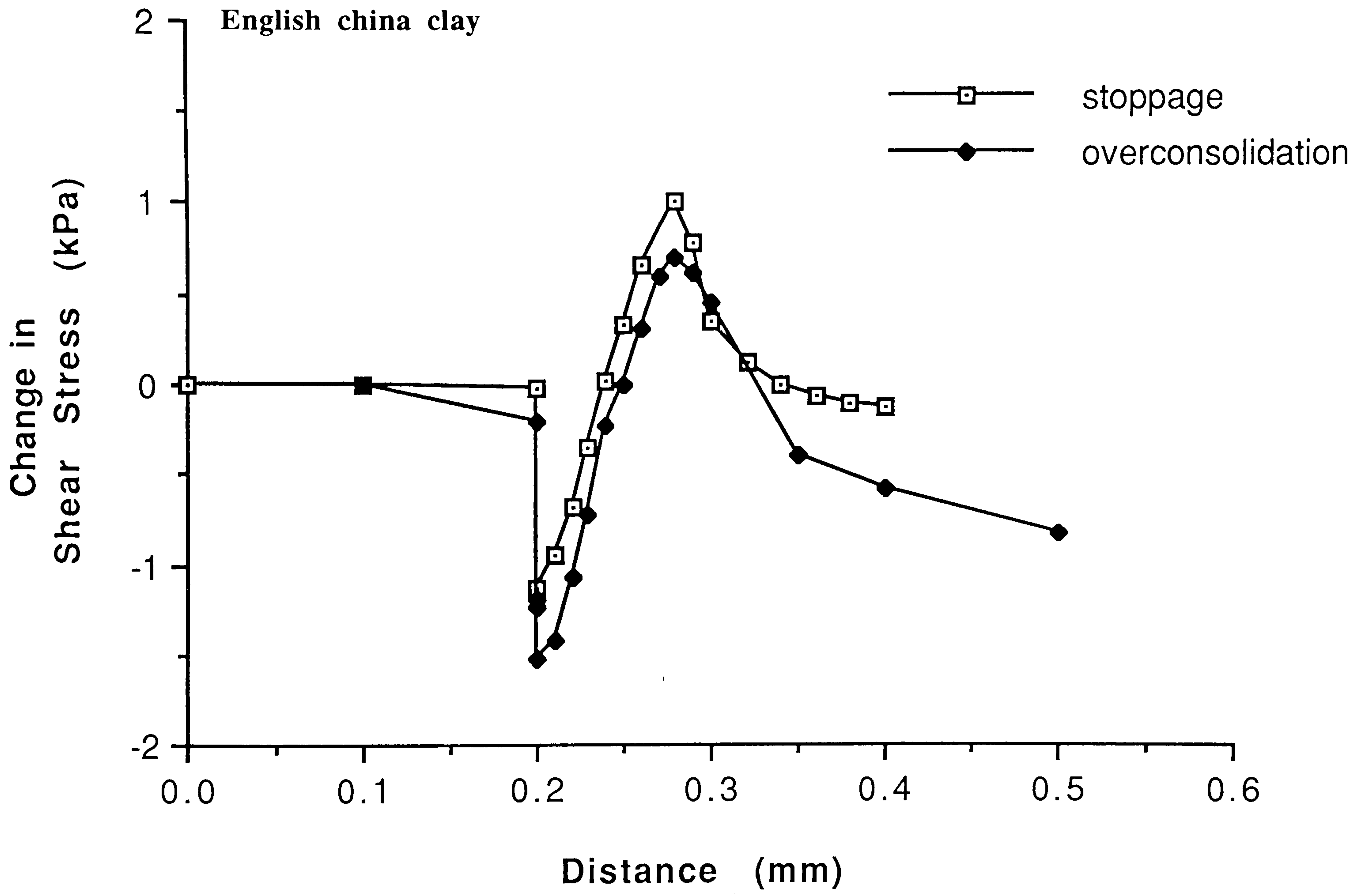


Figure 5.21 Effective consolidation pressure 400 kPa, OCR 10 - effects of stoppage and overconsolidation on residual strength

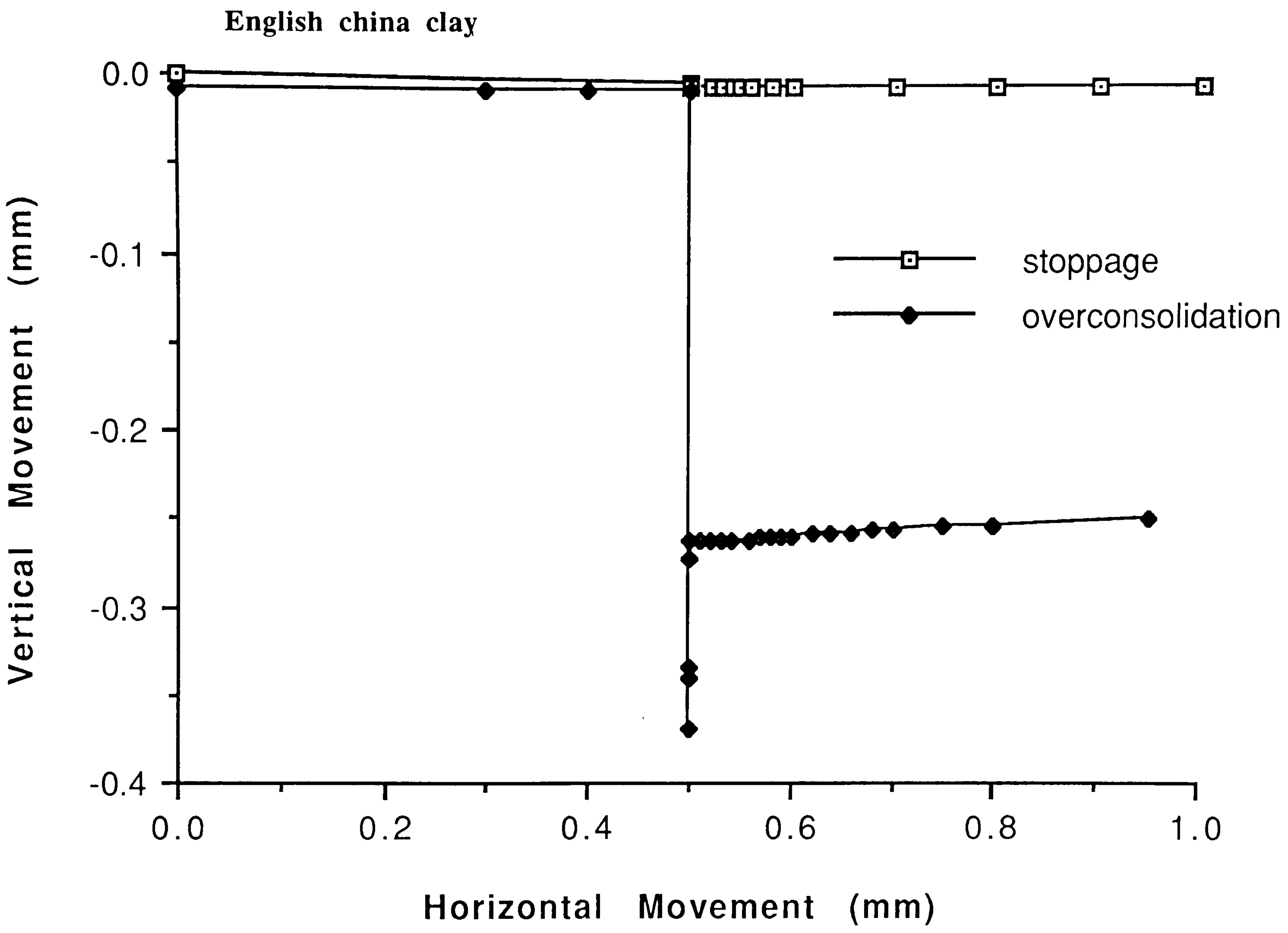


Figure 5.22 Effective consolidation pressure 300 kPa, OCR 20 - effects of stoppage and overconsolidation on vertical movement

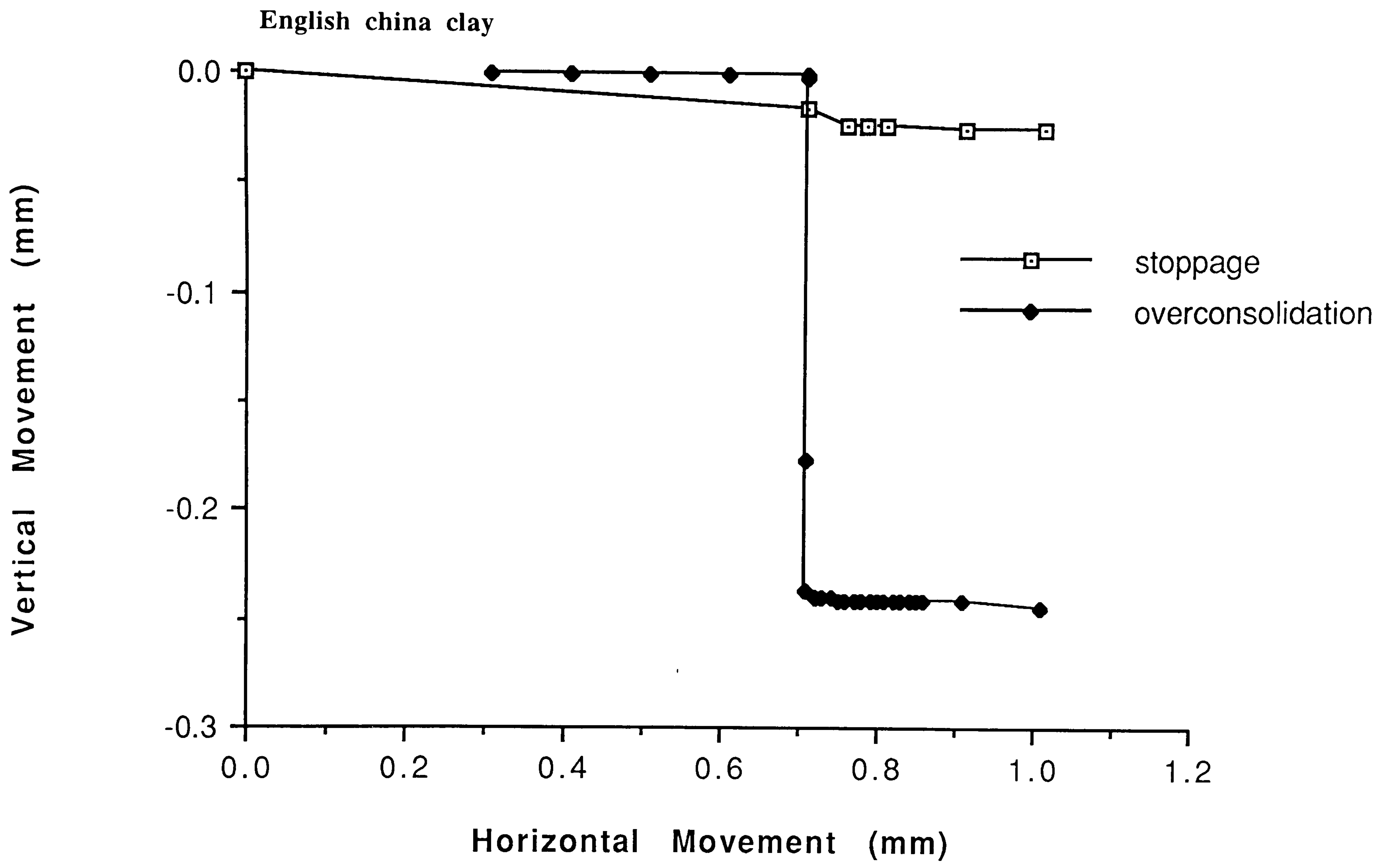


Figure 5.23 Effective consolidation pressure 300 kPa, OCR 10 - effects of stoppage and overconsolidation on vertical movement

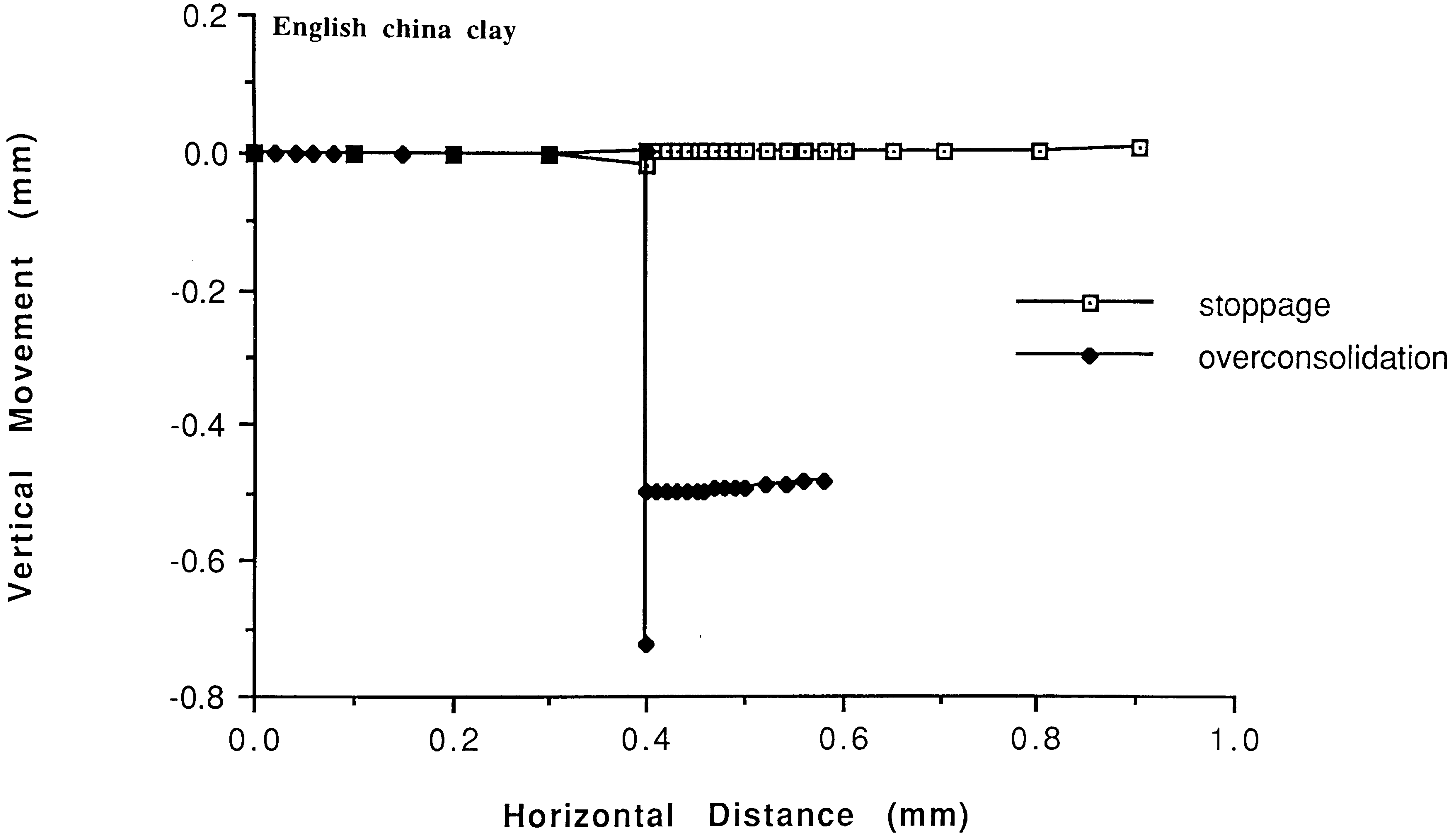


Figure 5.24 Effective consolidation pressure 150 kPa, OCR 10 - effects of stoppage and overconsolidation on vertical movement

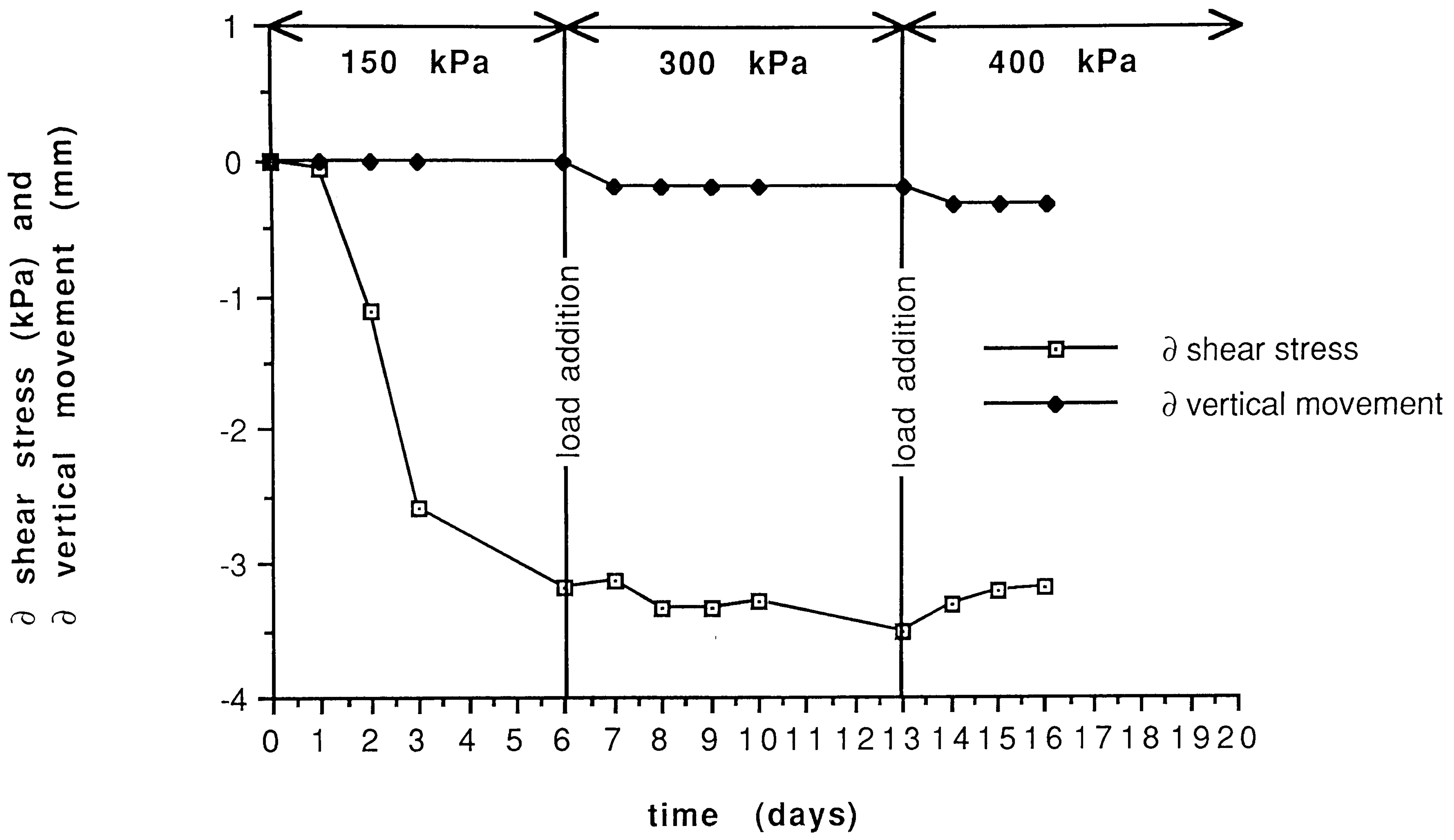


Figure 5.25 Shear box relaxation

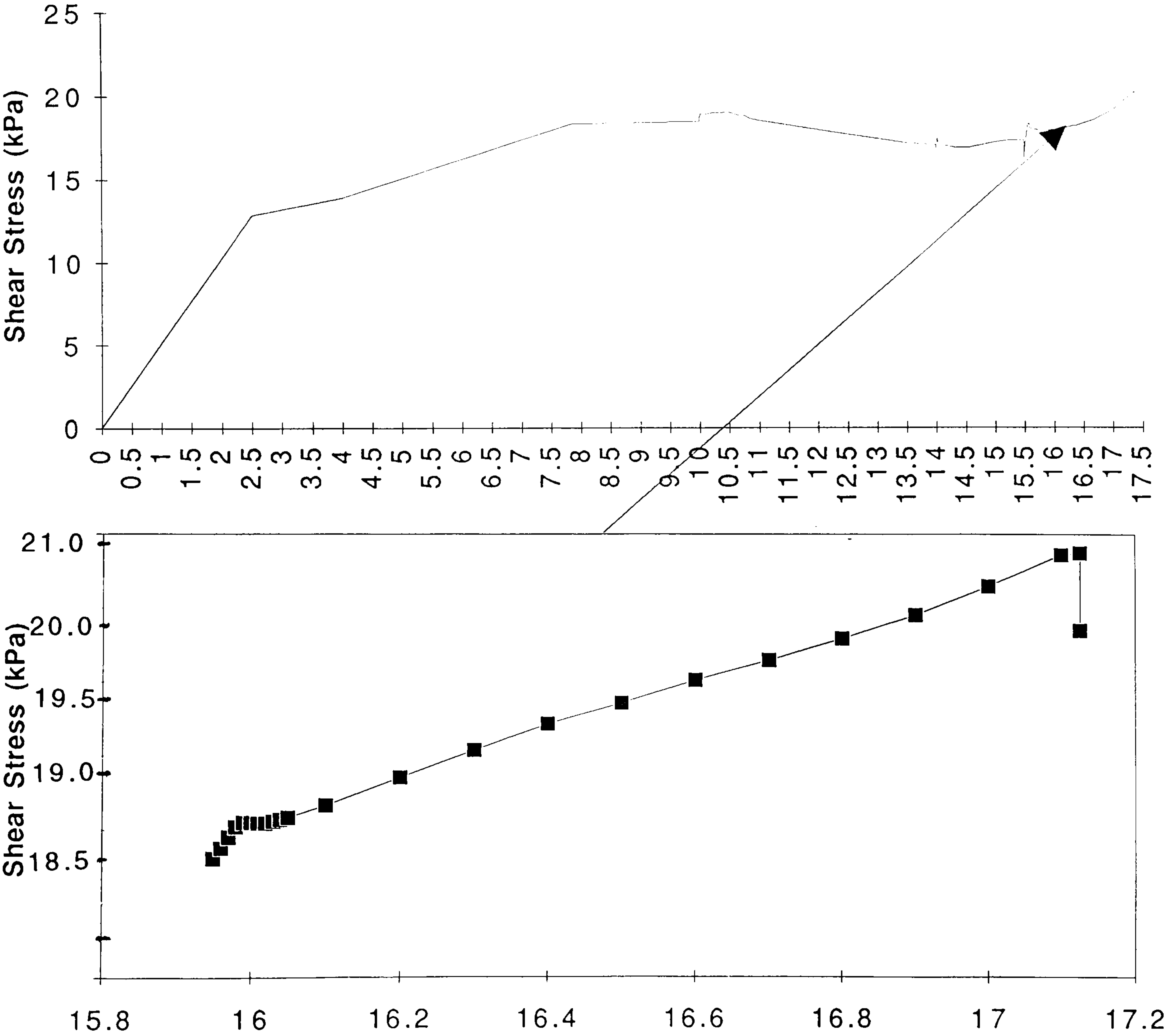


Figure 5.26 Shear box end of travel effects

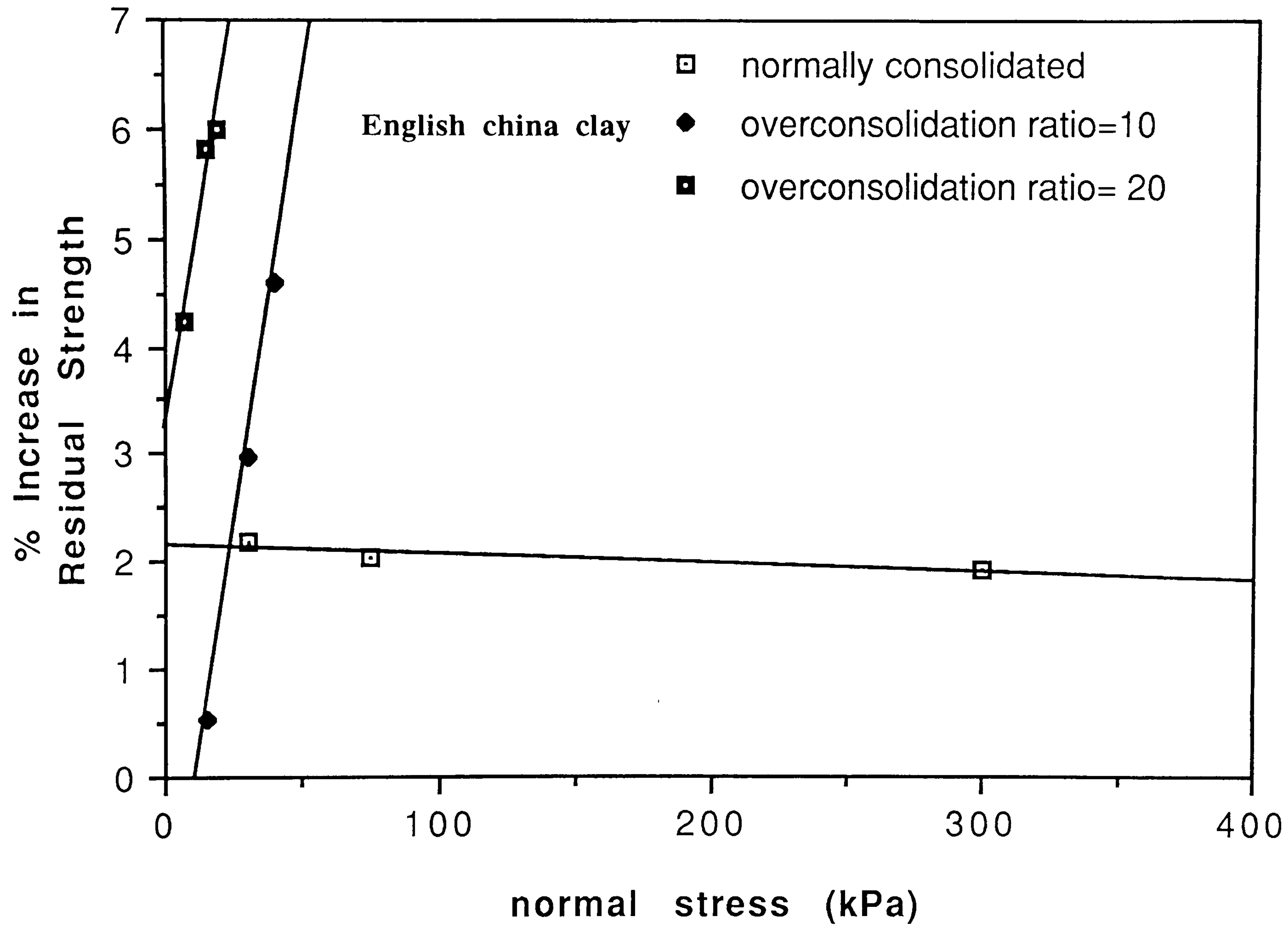
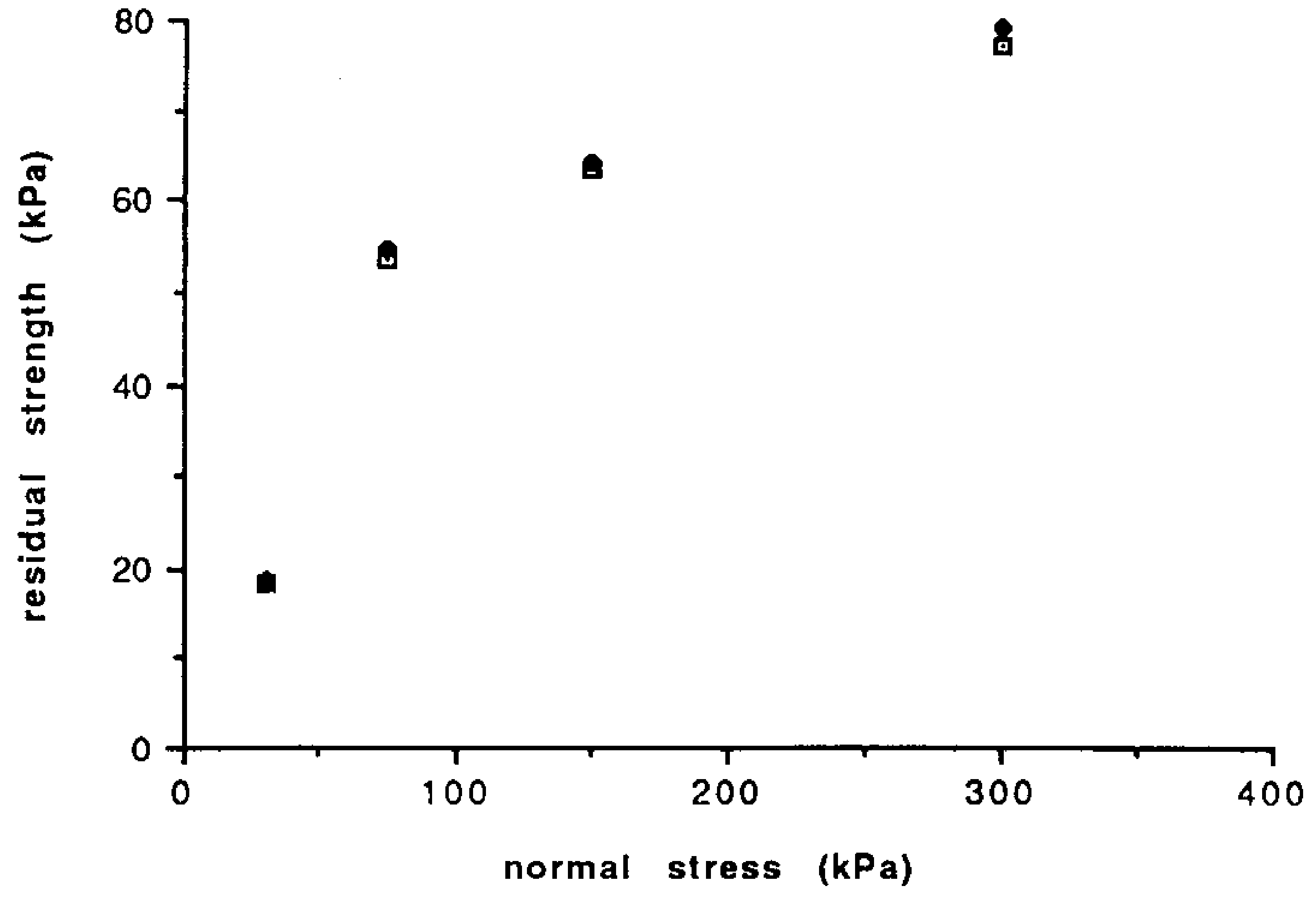
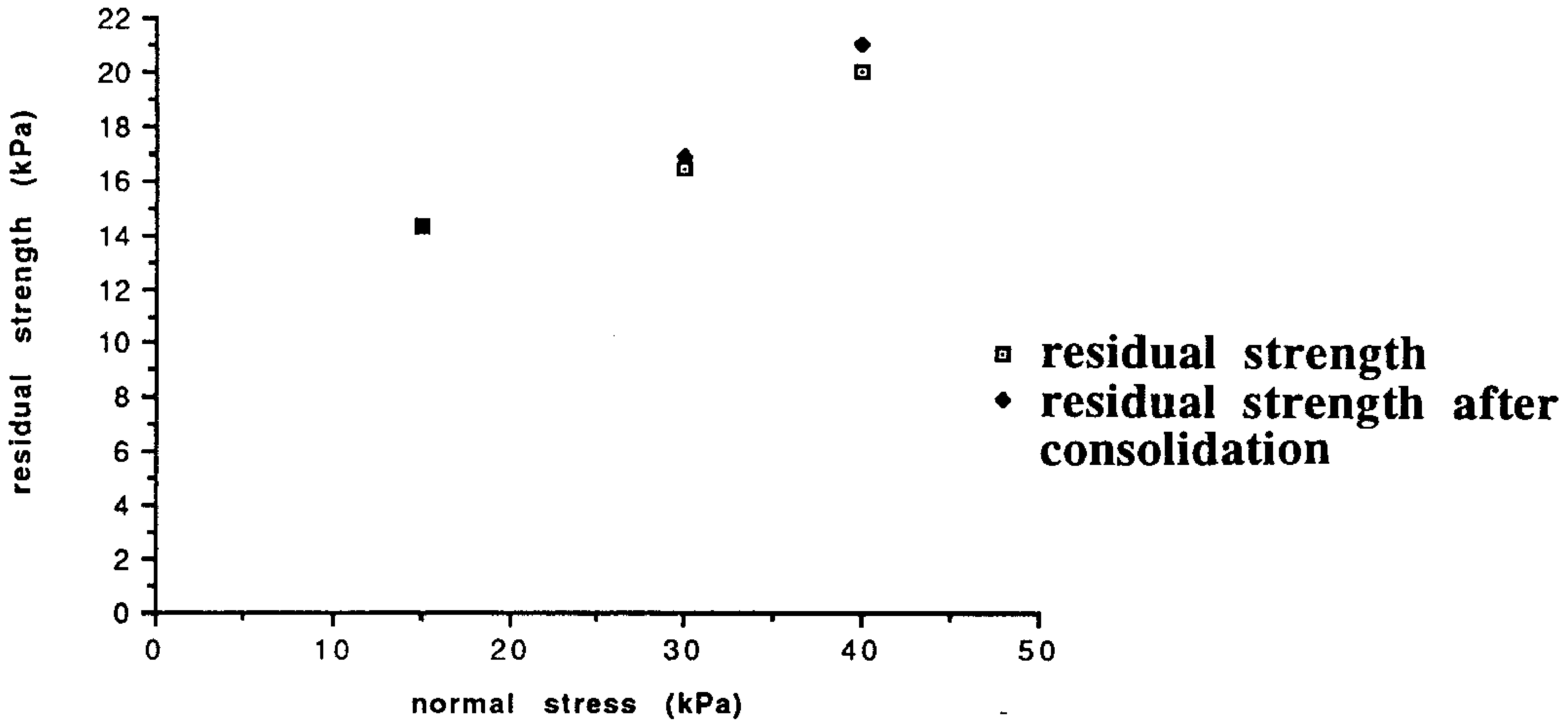


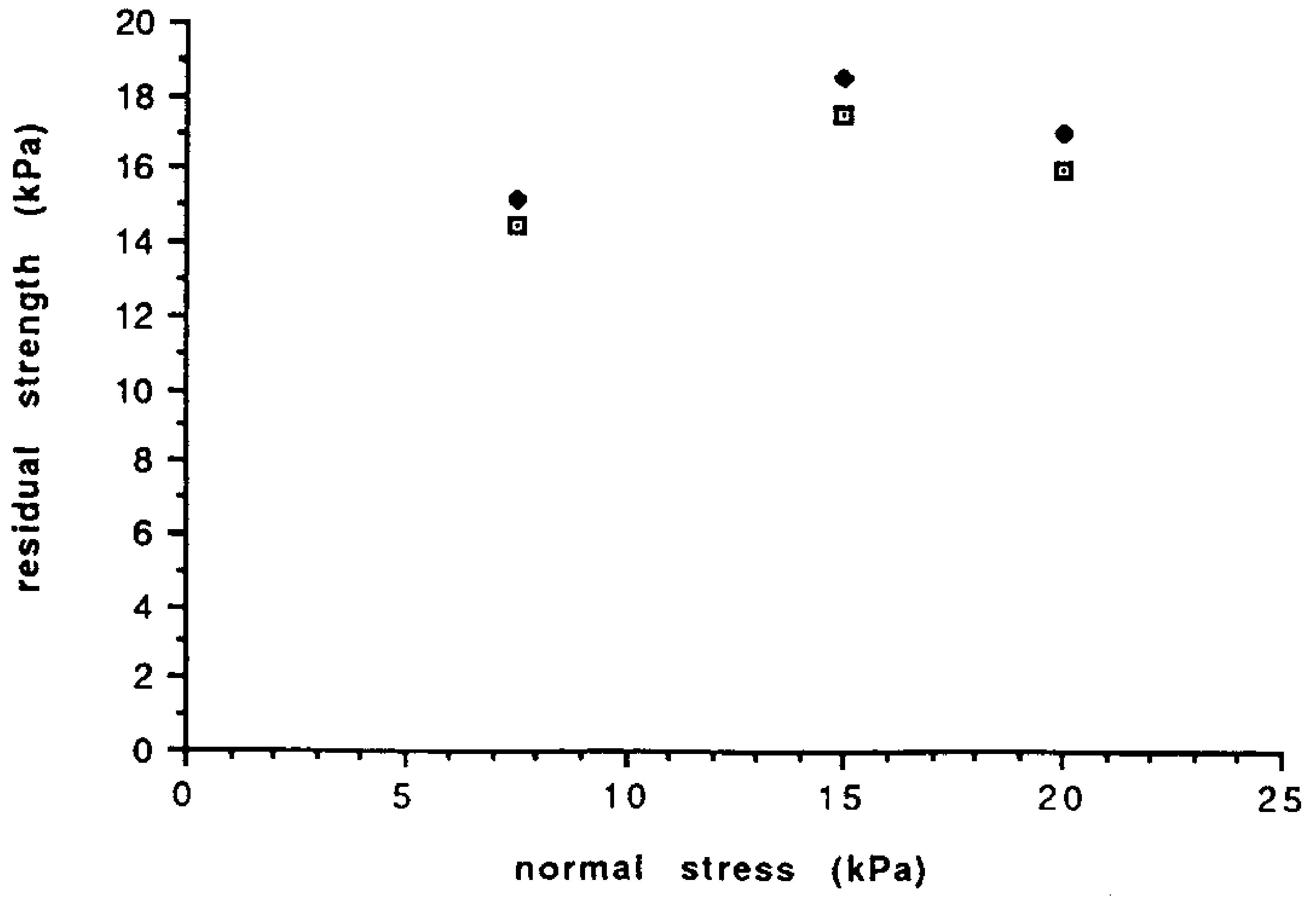
Figure 5.27 Percentage increases in residual strength



Normally Consolidated Samples



Overconsolidation Ratio = 10



Overconsolidation Ratio = 20

Figure 5.28 Revised residual strength envelopes for English china clay

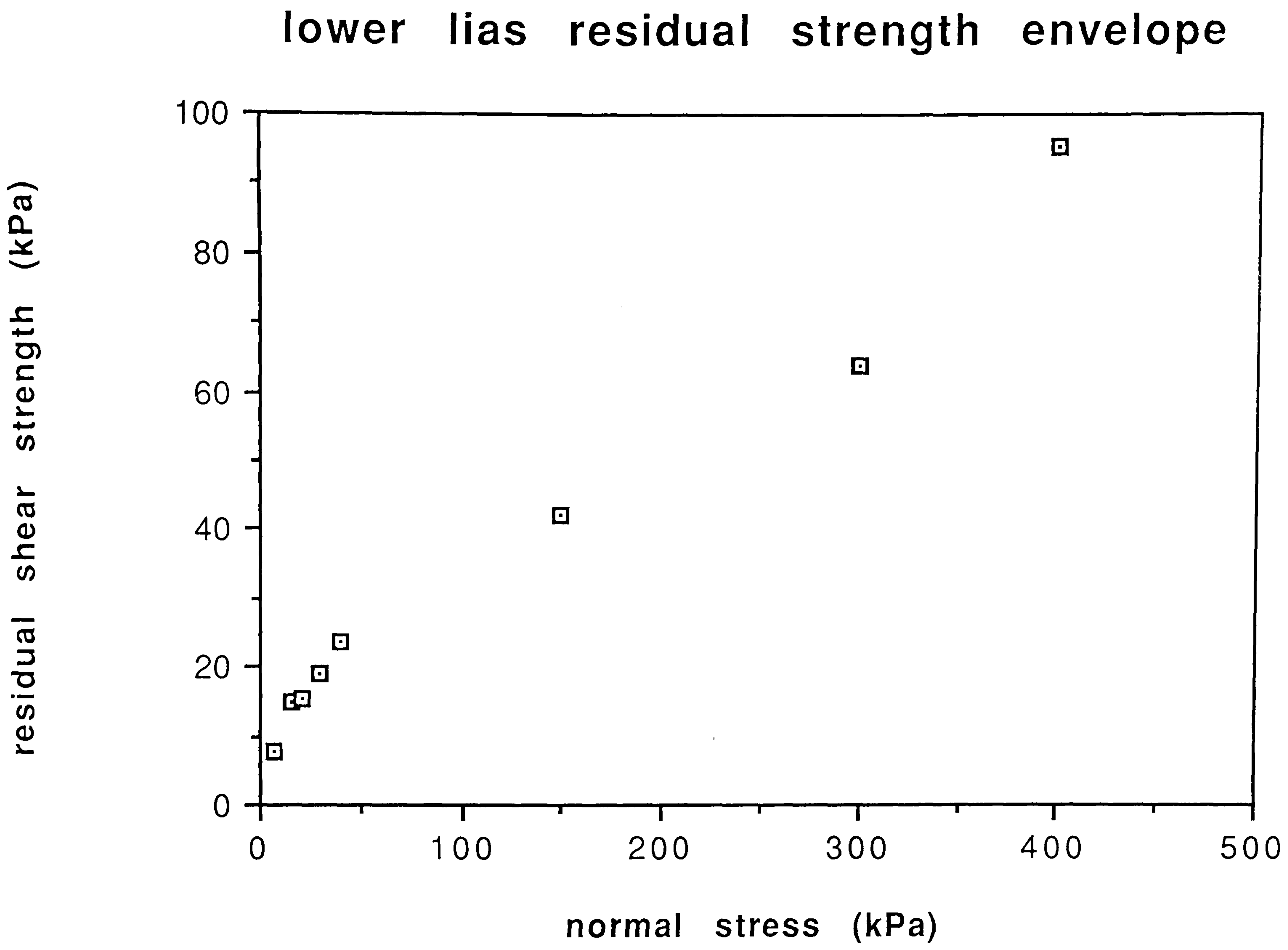


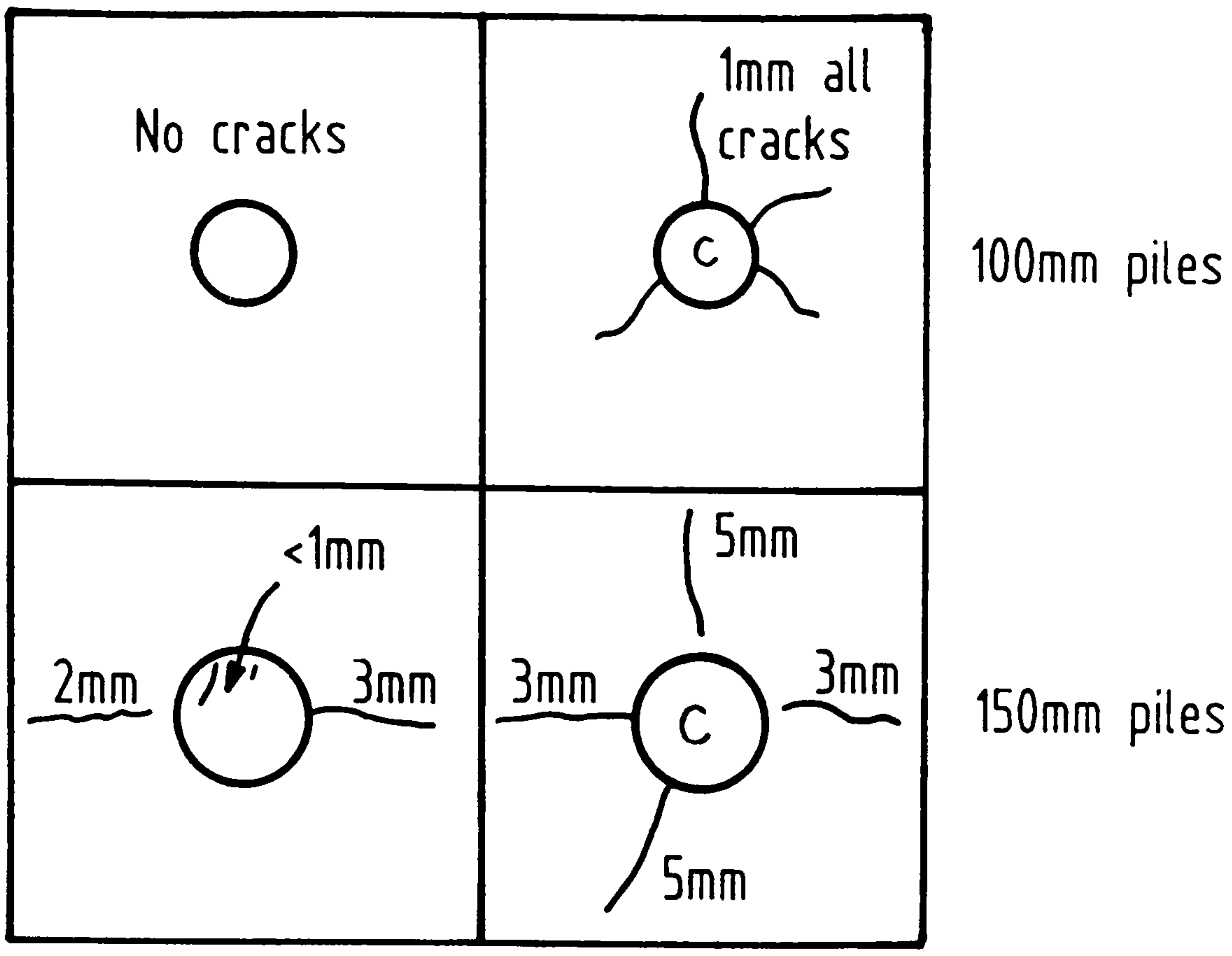
Figure 5.29 Lower Lias residual strength envelope

Figure 5.30 Lower Lias -
Effective consolidation pressure 400 kPa,
OCR 10 - effects of stoppage and
overconsolidation on residual strength

Figure 5.31 Lower Lias -
Effective consolidation pressure 400 kPa,
OCR 10 - effects of stoppage and
overconsolidation on vertical movement

Figure 5.32 Lower Lias -
Effective consolidation pressure 400 kPa,
OCR 1 - effects of stoppage and
overconsolidation on residual strength

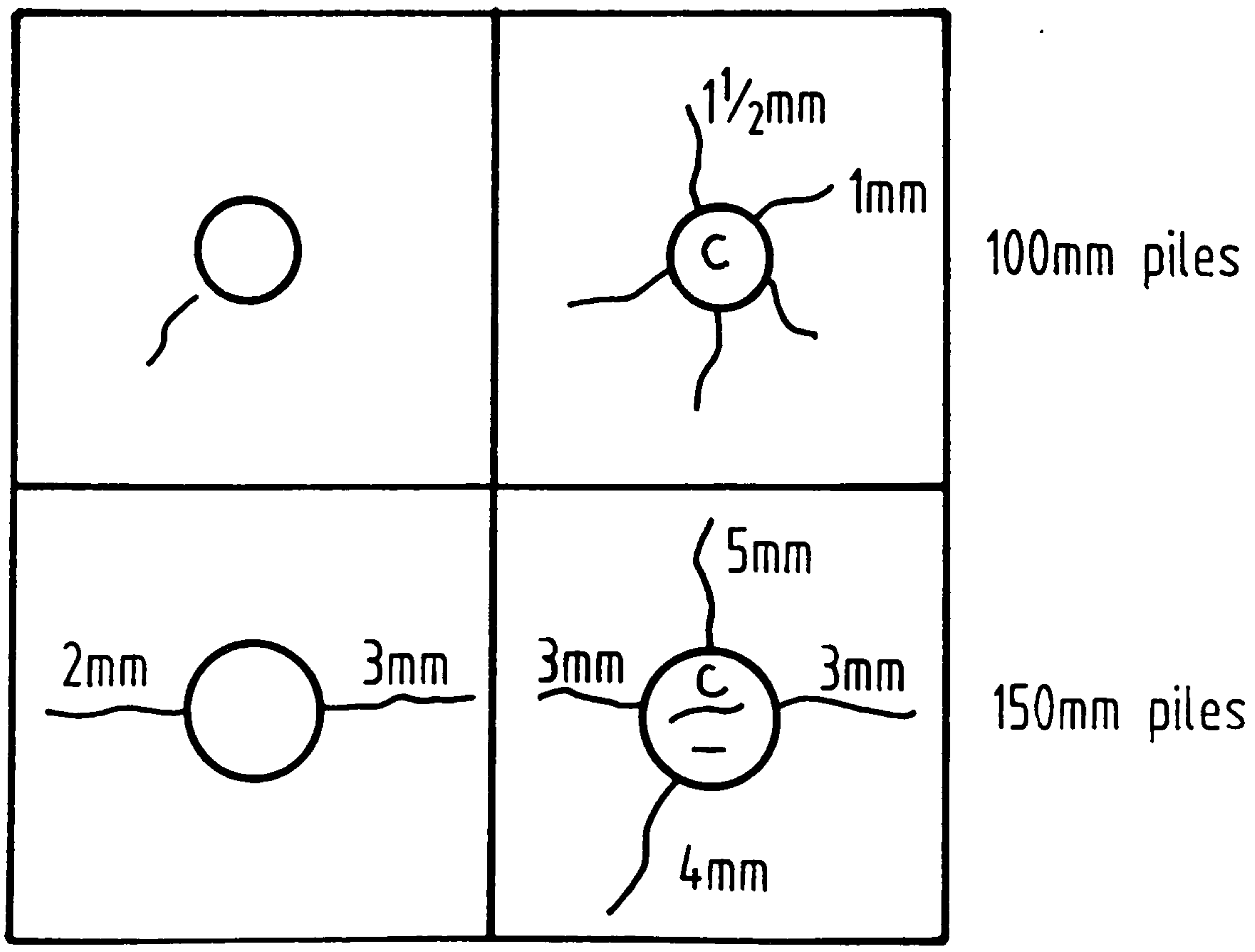
Figure 5.33 Lower Lias -
Effective consolidation pressure 400 kPa,
OCR 1 - effects of stoppage and
overconsolidation on vertical movement



Cracks at 1 WEEK

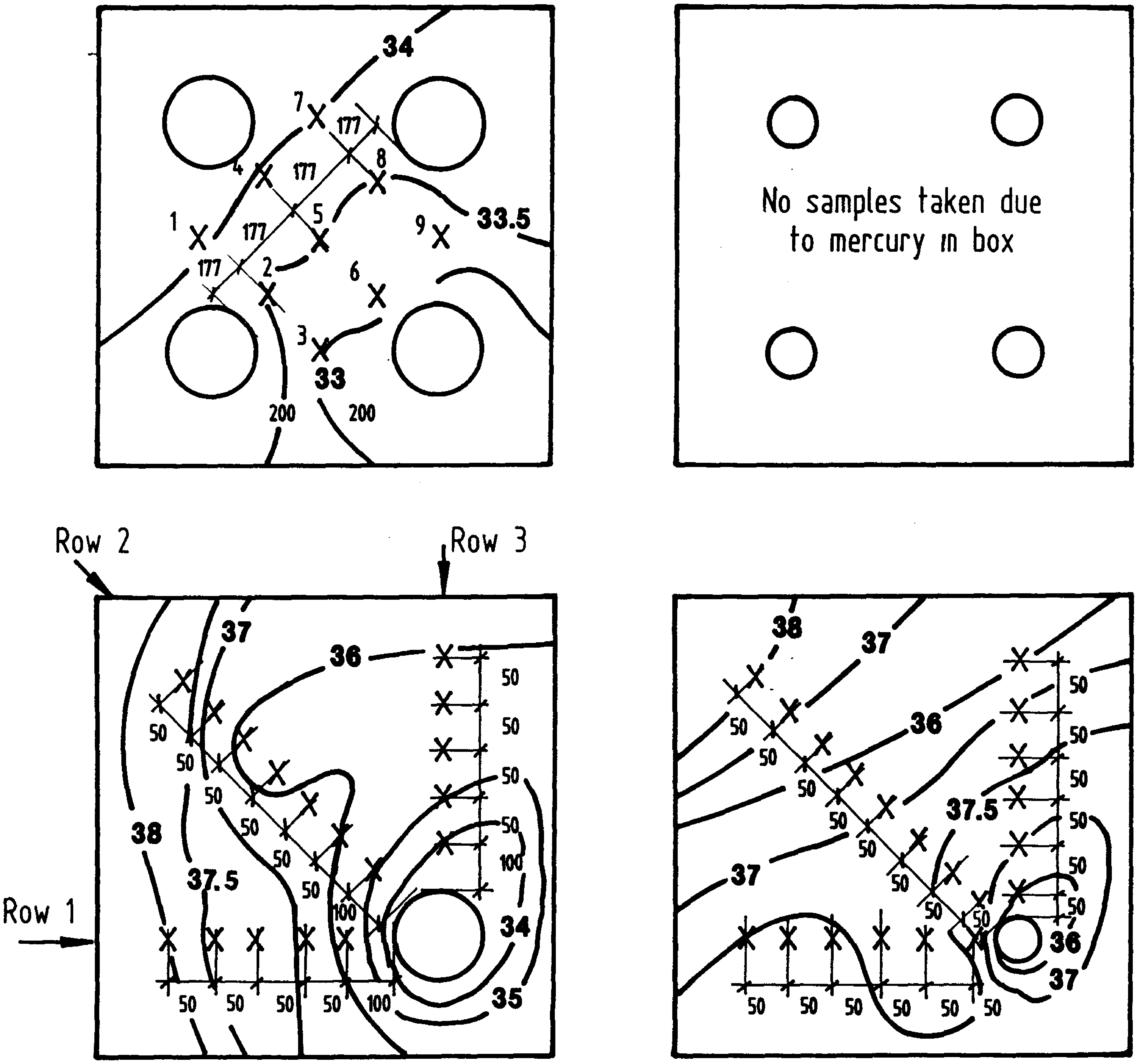
Notes

measurements denote crack width
crack lengths drawn to scale relative to box
'C' signifies compacted piles



Cracks at 6 WEEKS

Figure 5.34 Arrangement 1 - Cracking pattern



100mm piles

50mm piles

Notes

Numbers in bold denote water content

Figure 5.35 Arrangement 2 - Water content contours

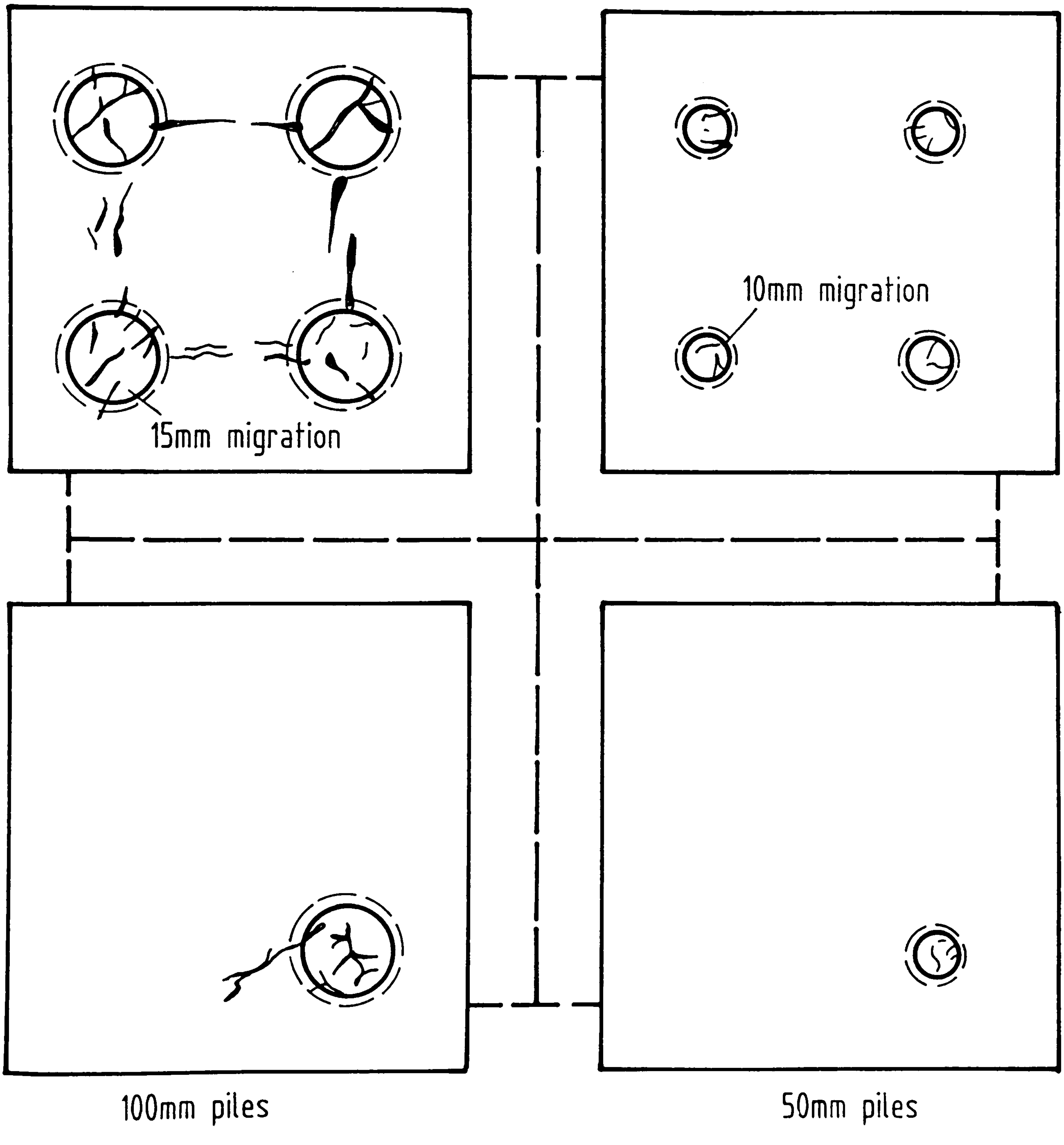


Figure 5.36 Arrangement 2 - Cracking pattern

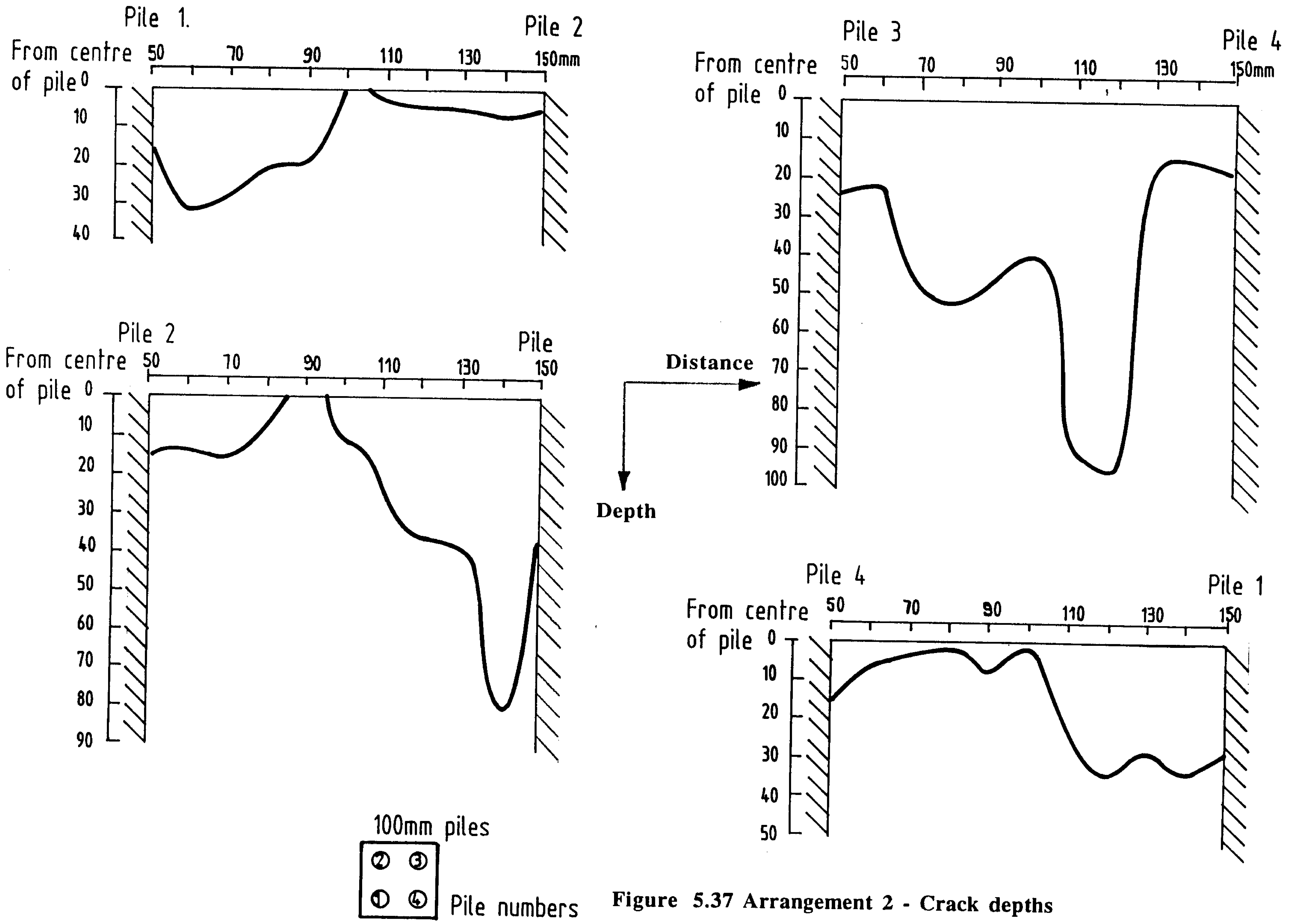


Figure 5.37 Arrangement 2 - Crack depths

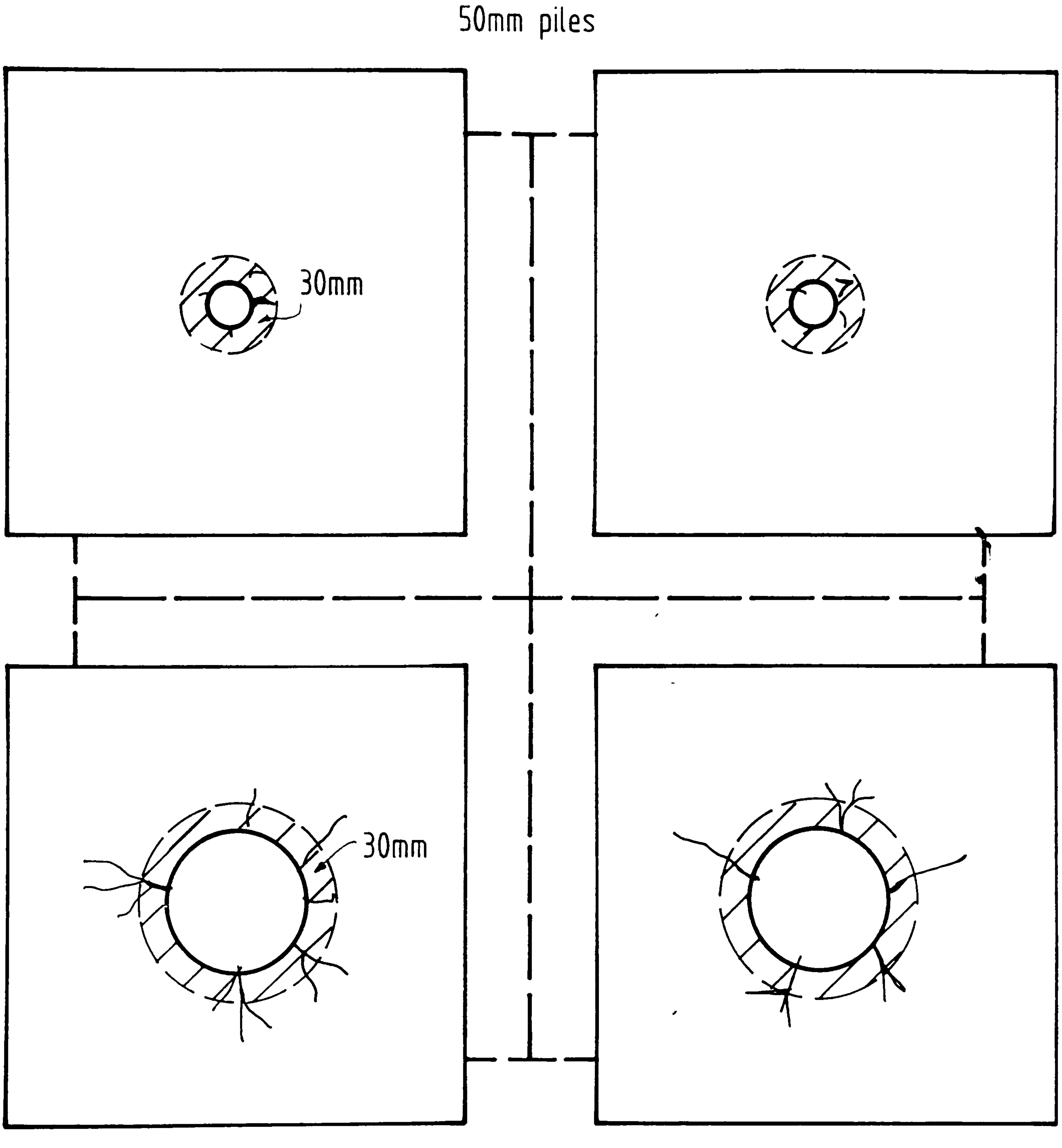
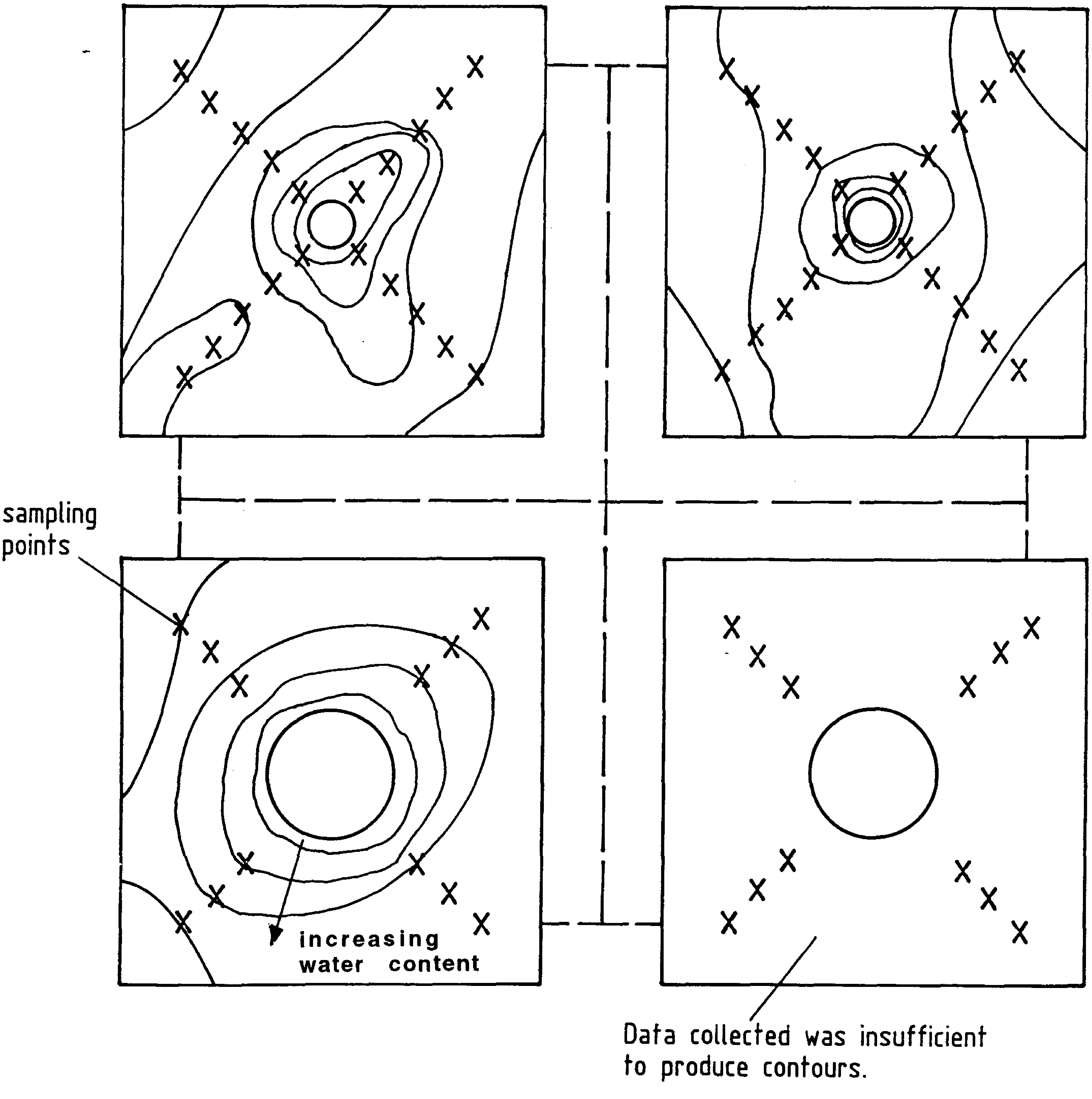


Figure 5.38 Arrangement 3 - Cracking pattern



Note: contours depict a change of 1%

Figure 5.39 Arrangement 3 - Water content contours

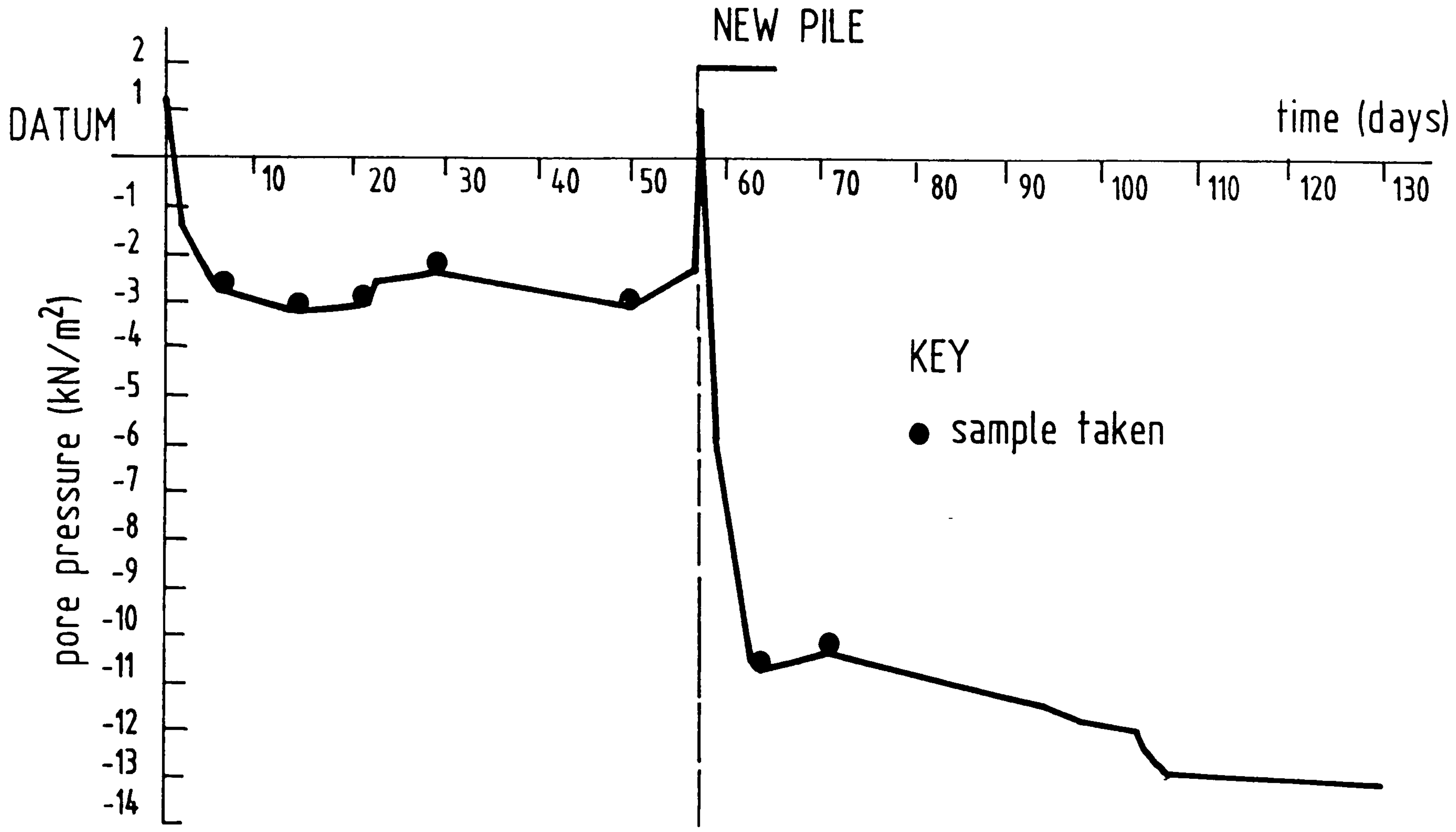
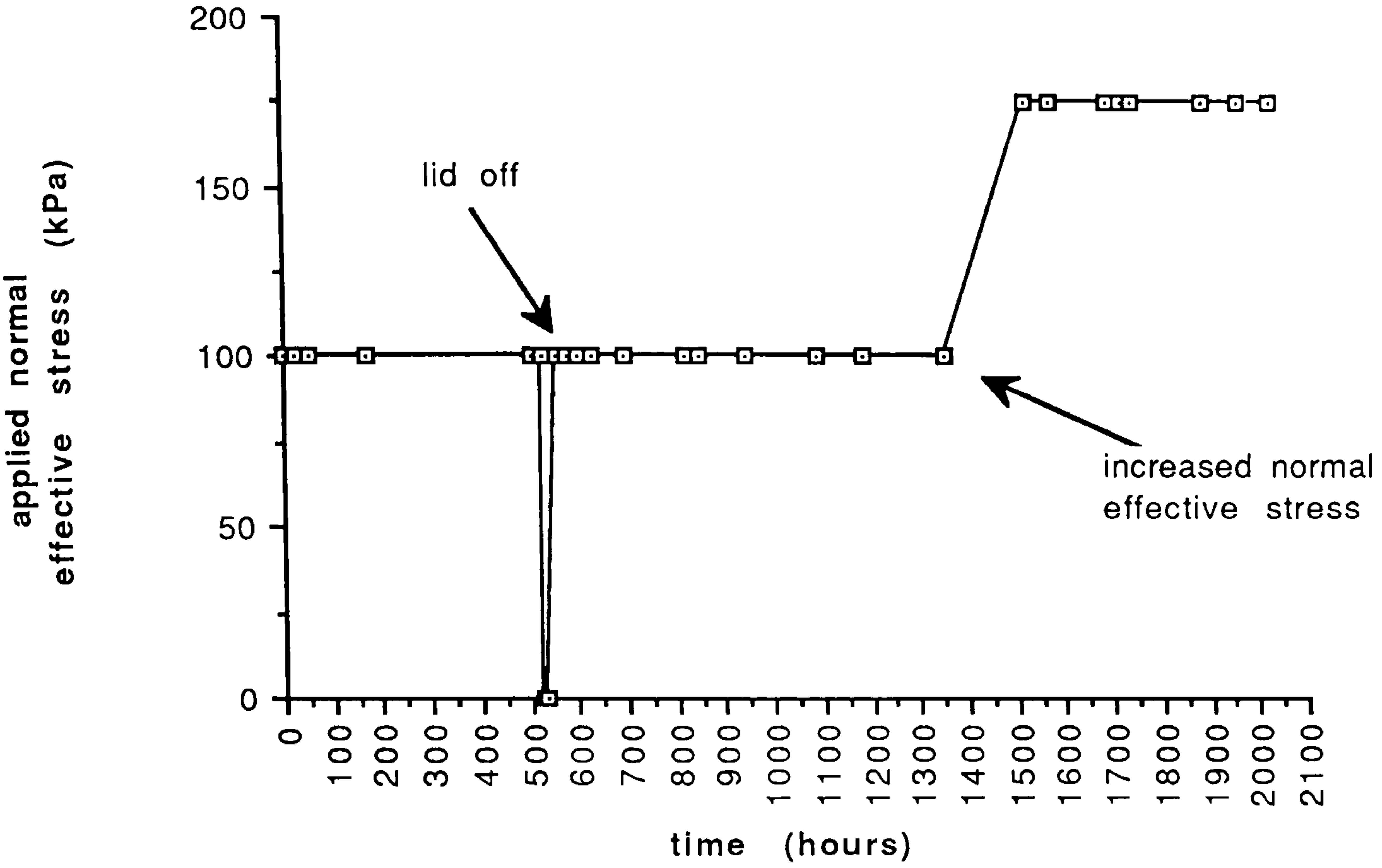


Figure 5.40 Small box - pore water pressure changes

Arrangement 1- normal effective stress



Arrangement 1 - 150 mm Diameter Compacted Pile

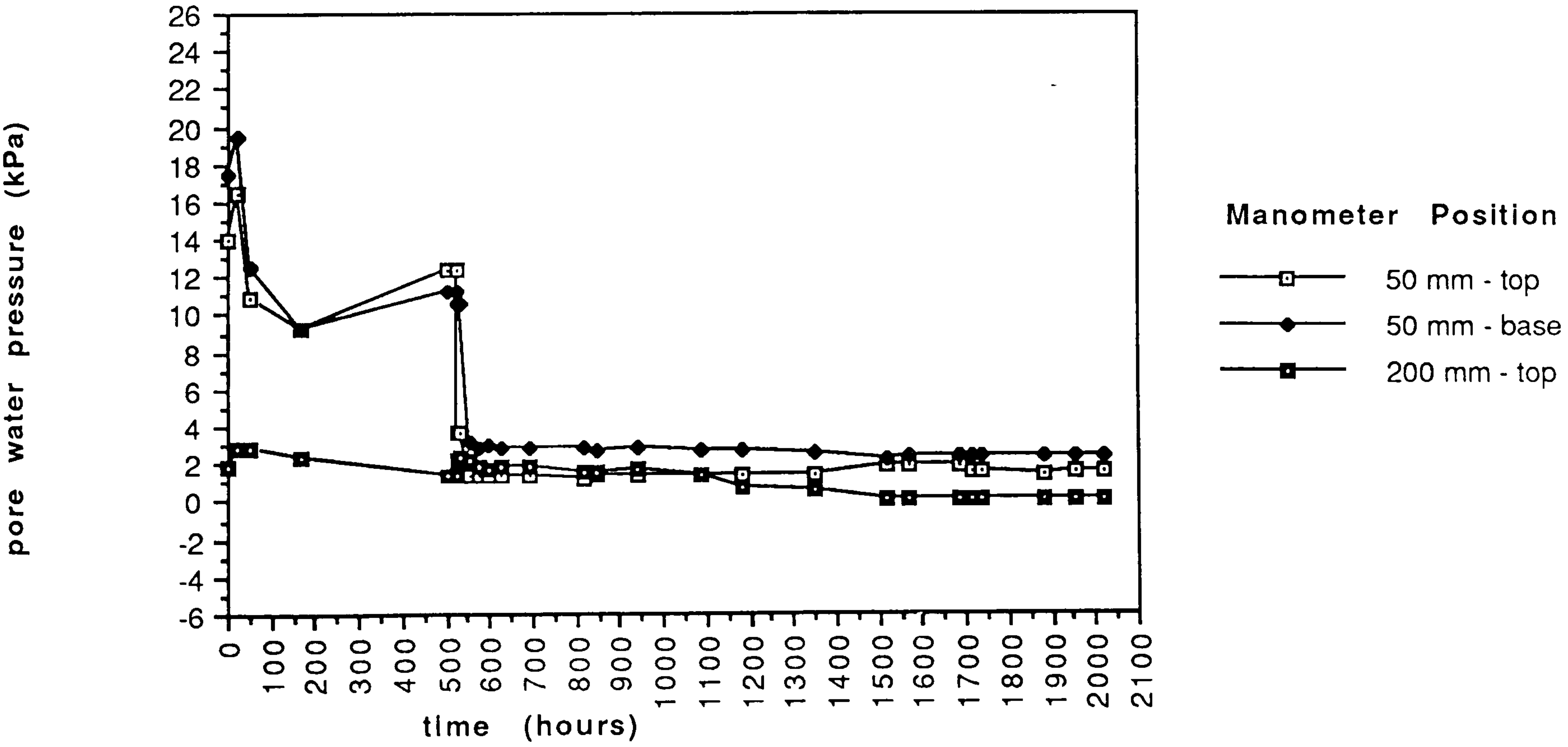
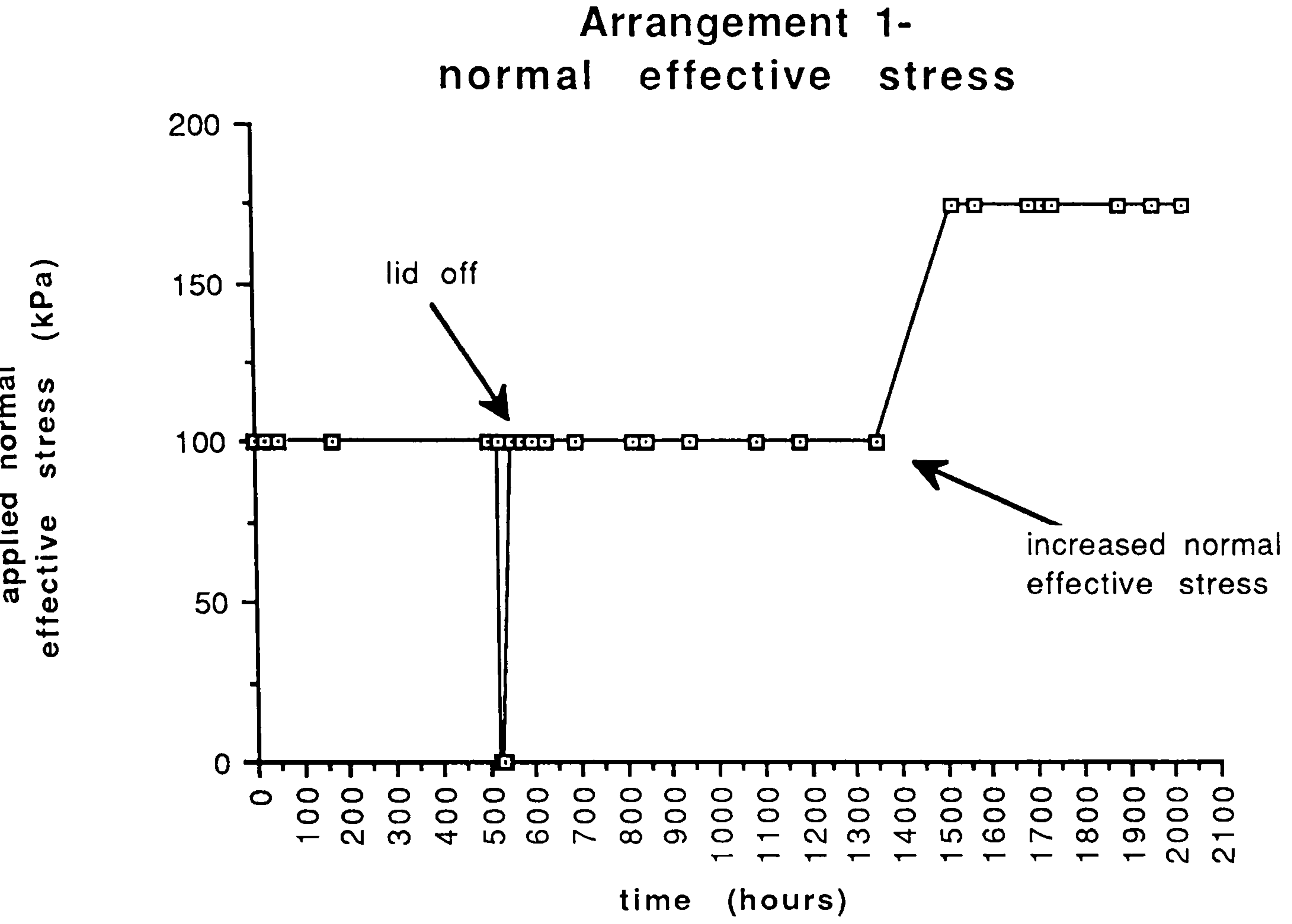


Figure 5.41 Arrangement 1 - 150 mm diameter compacted pile - post-pile pore water pressures



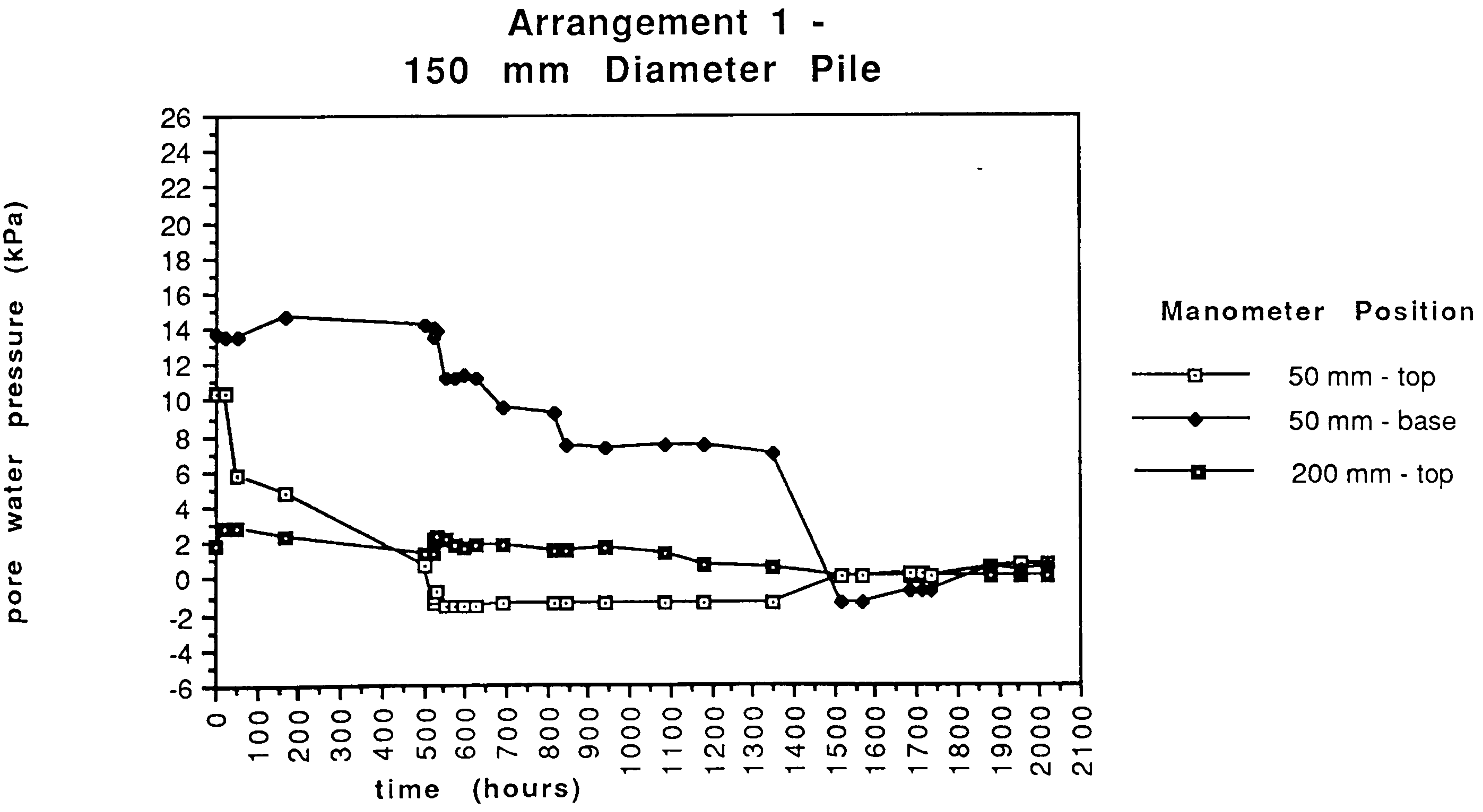
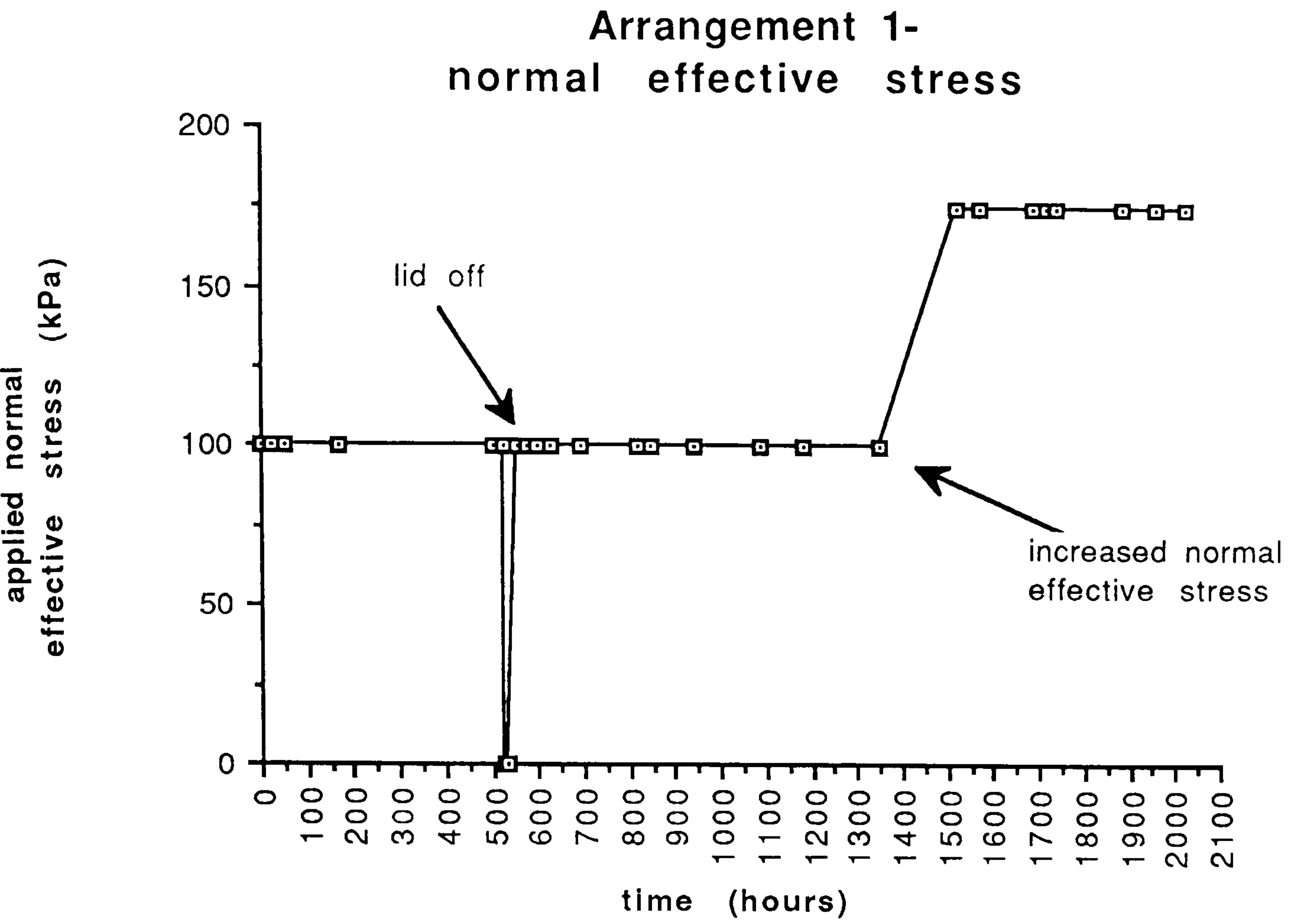


Figure 5.42 Arrangement 1 -
150 mm diameter pile -
post-pile pore water pressures



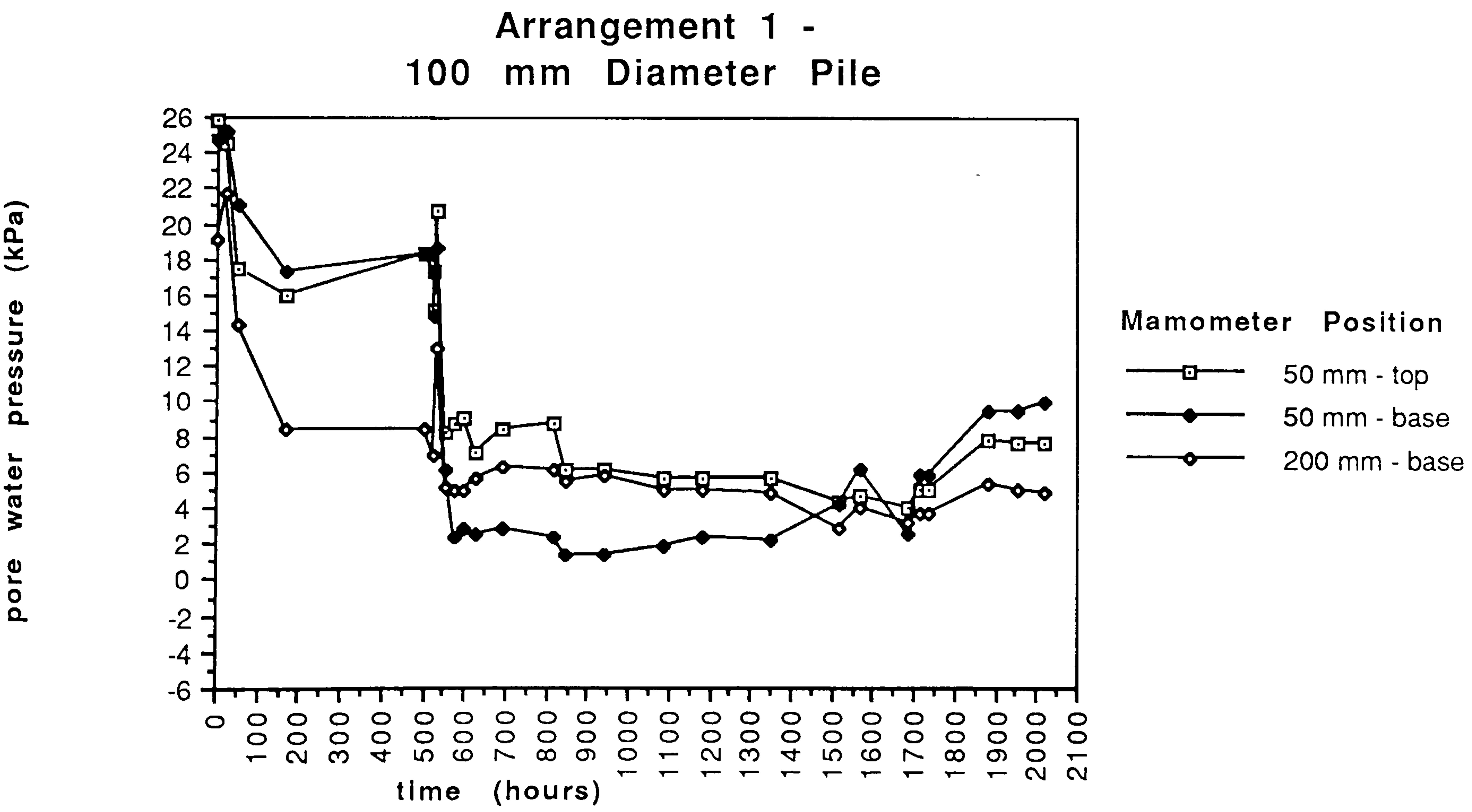
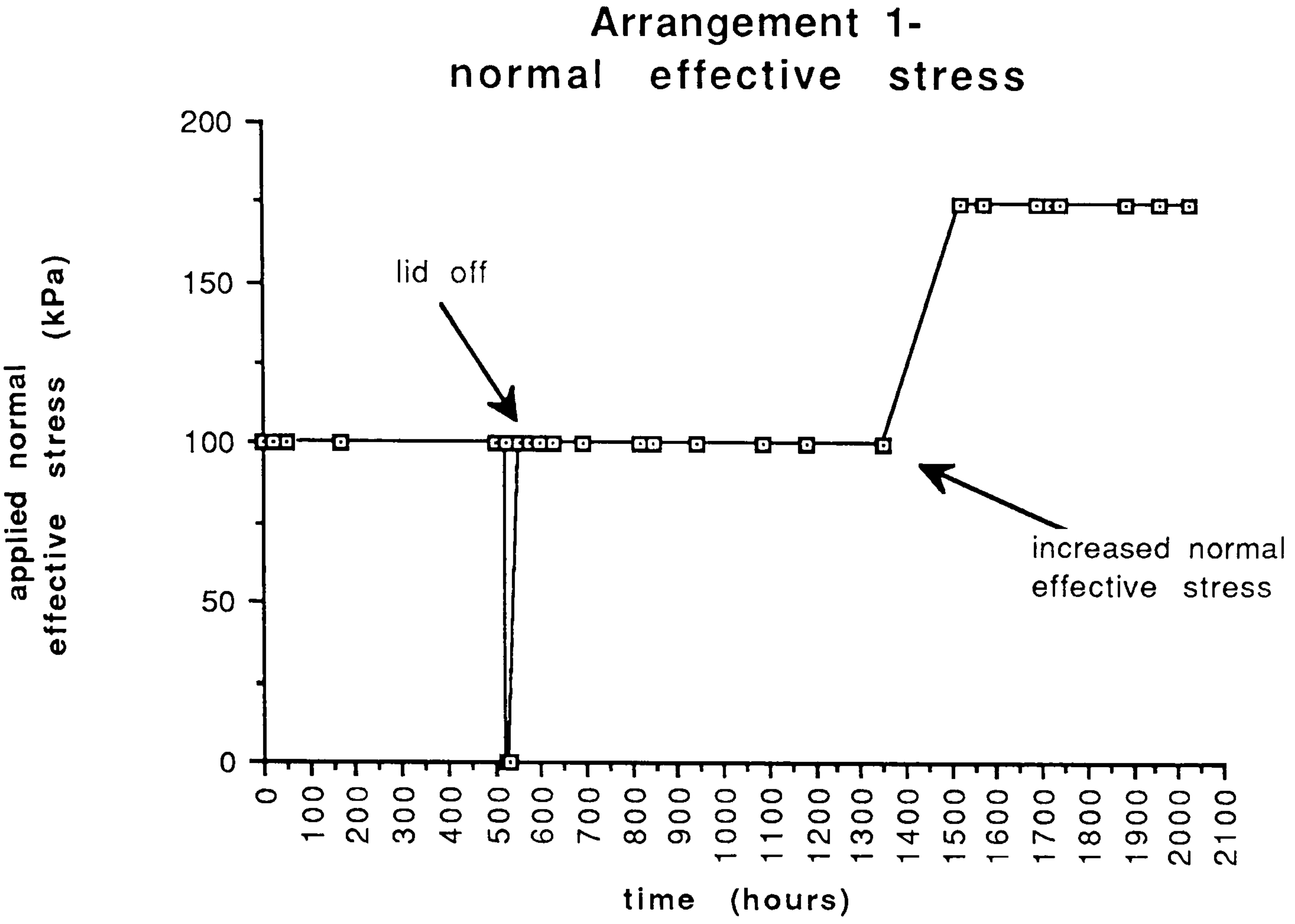


Figure 5.43 Arrangement 1 -
100 mm diameter pile -
post-pile pore water pressures



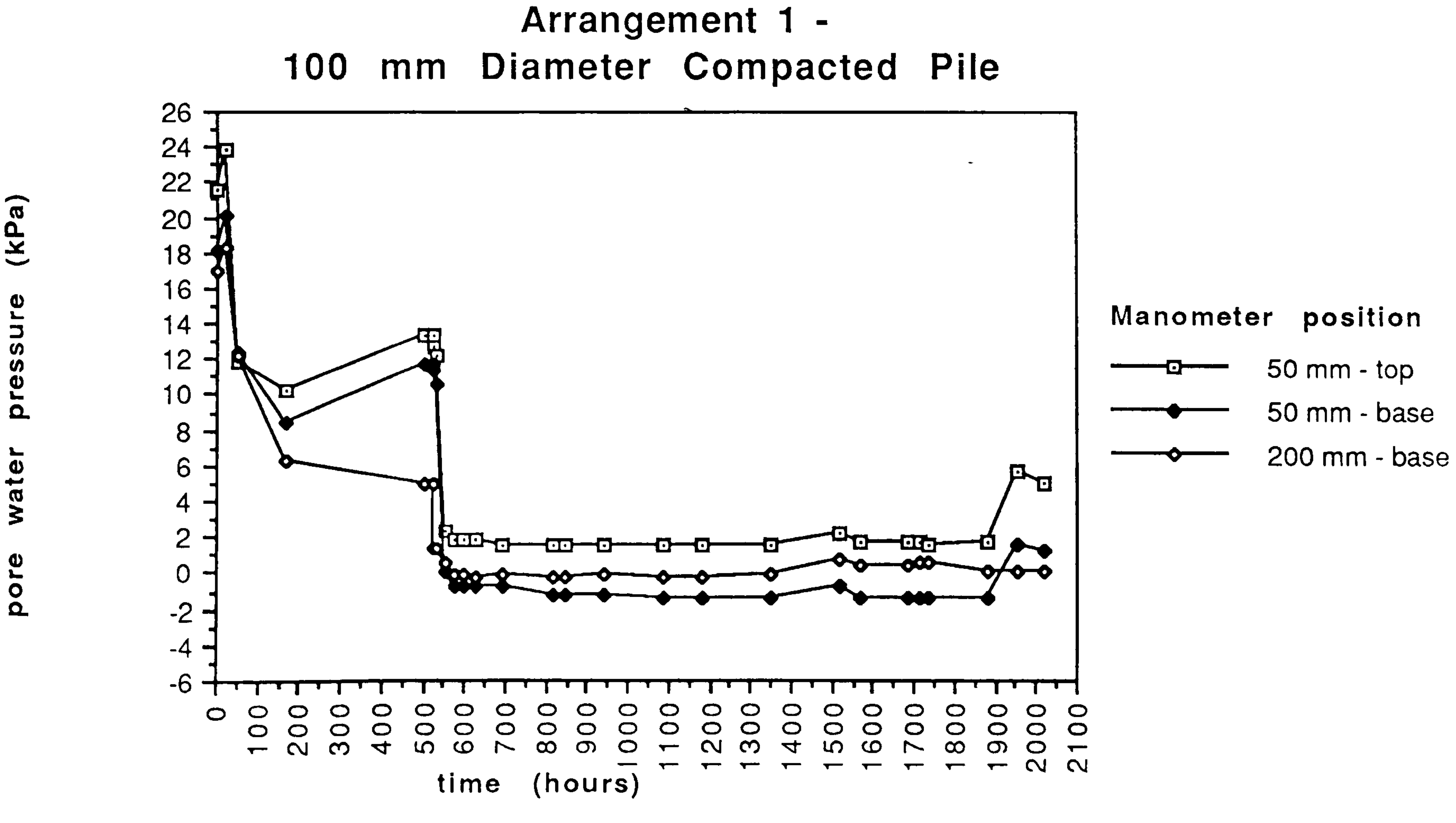


Figure 5.44 Arrangement 1 -
100 mm diameter compacted pile -
post-pile pore water pressures

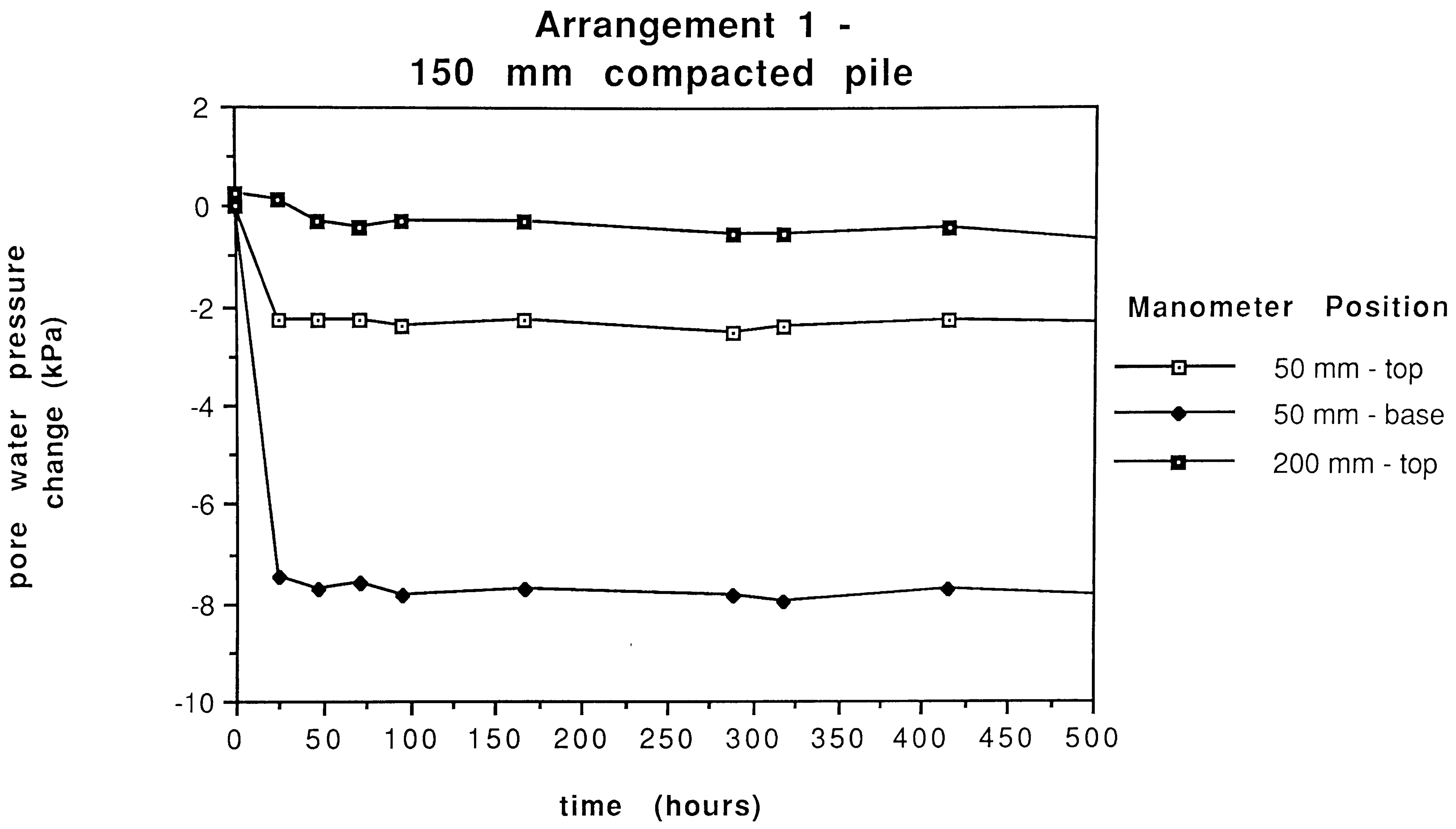


Figure 5.45 Arrangement 1 -
150 mm diameter compacted pile -
post-pile pore water pressures

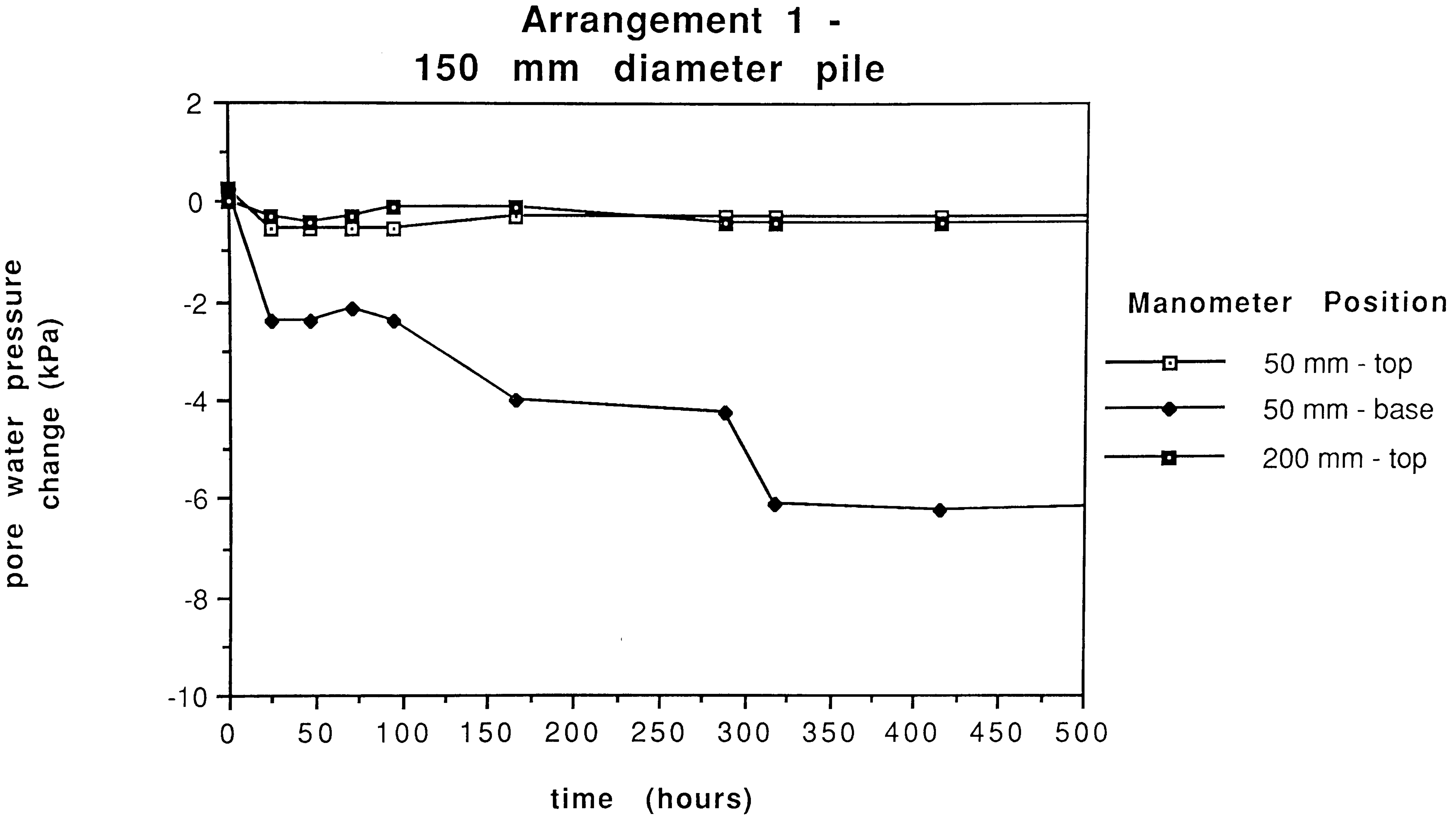


Figure 5.46 Arrangement 1 -
150 mm diameter pile -
post-pile pore water pressure changes

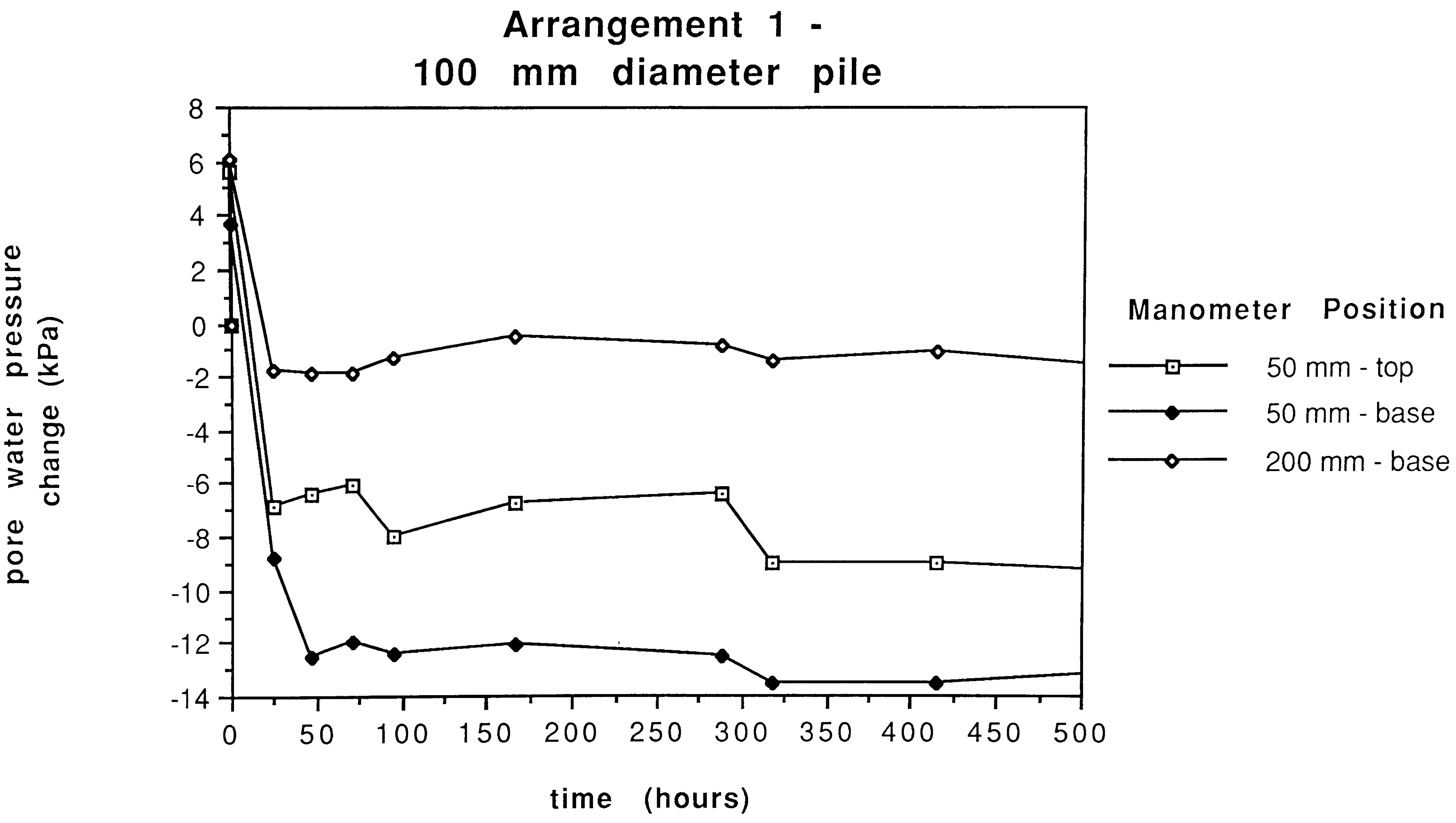


Figure 5.47 Arrangement 1 -
100 mm diameter pile -
post-pile pore water pressure changes

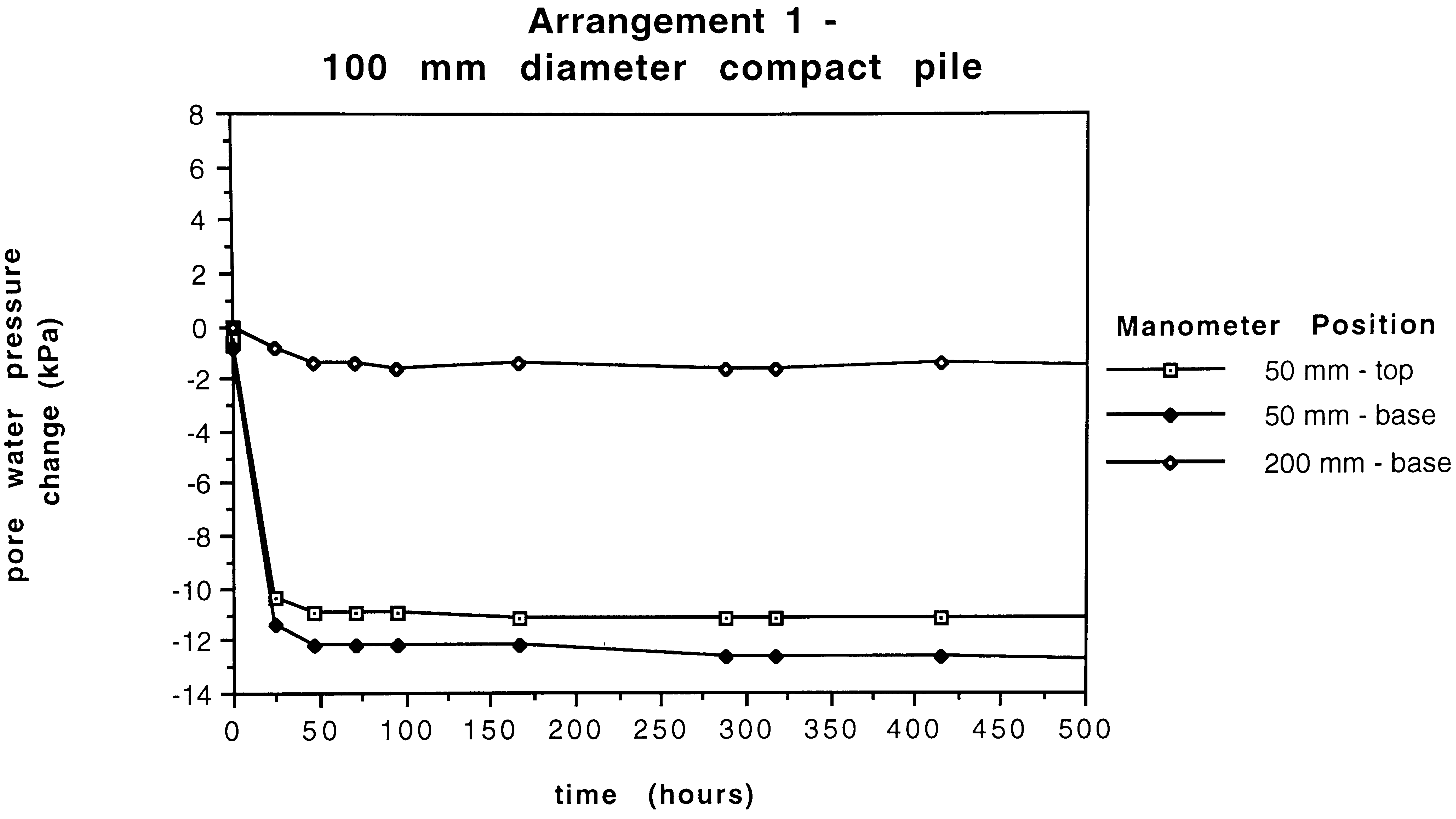


Figure 5.48 Arrangement 1 -
100 mm diameter compacted pile -
post-pile pore water pressure changes

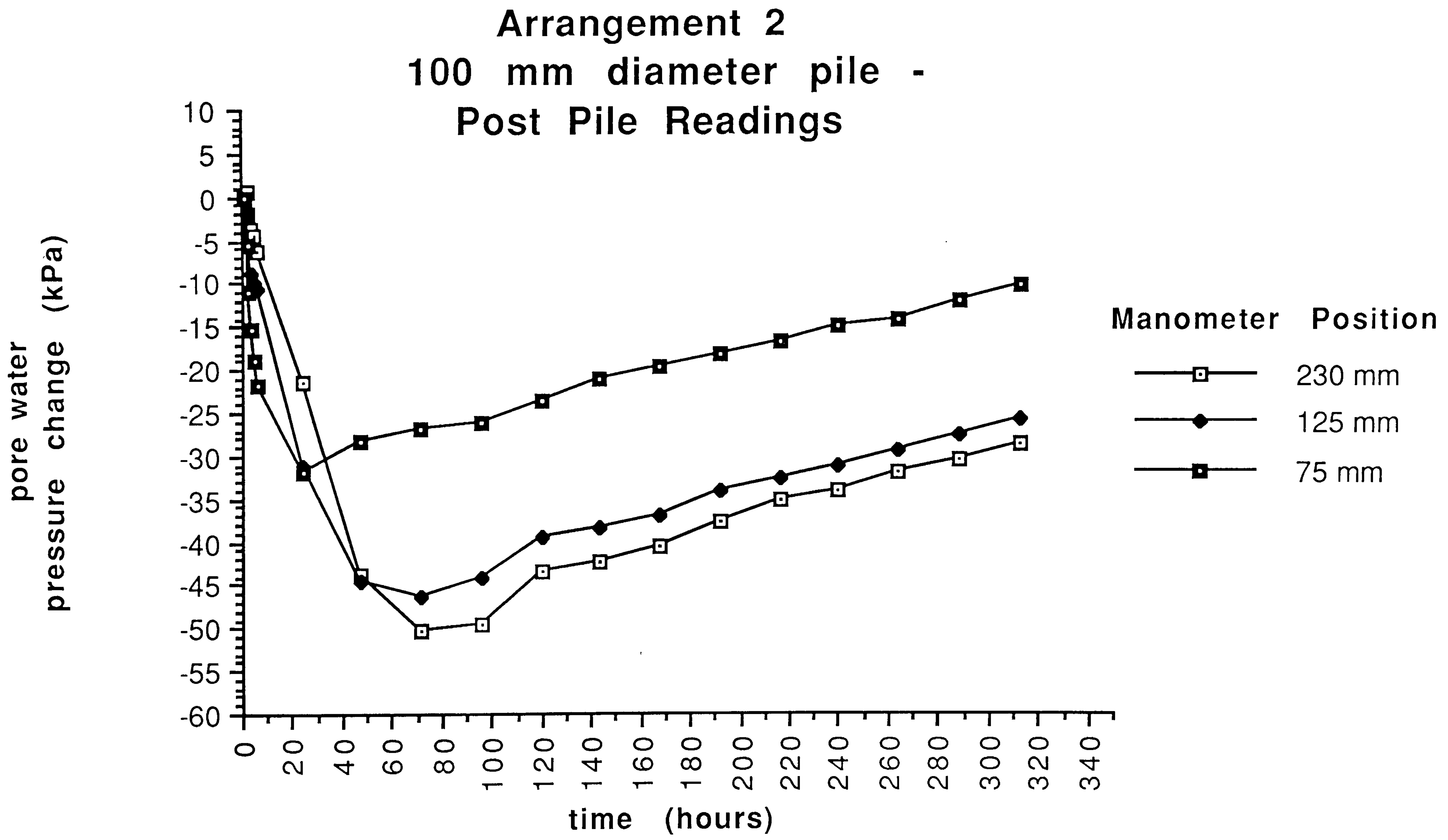


Figure 5.49 Arrangement 2 -
100 mm diameter pile -
post-pile pore water pressures

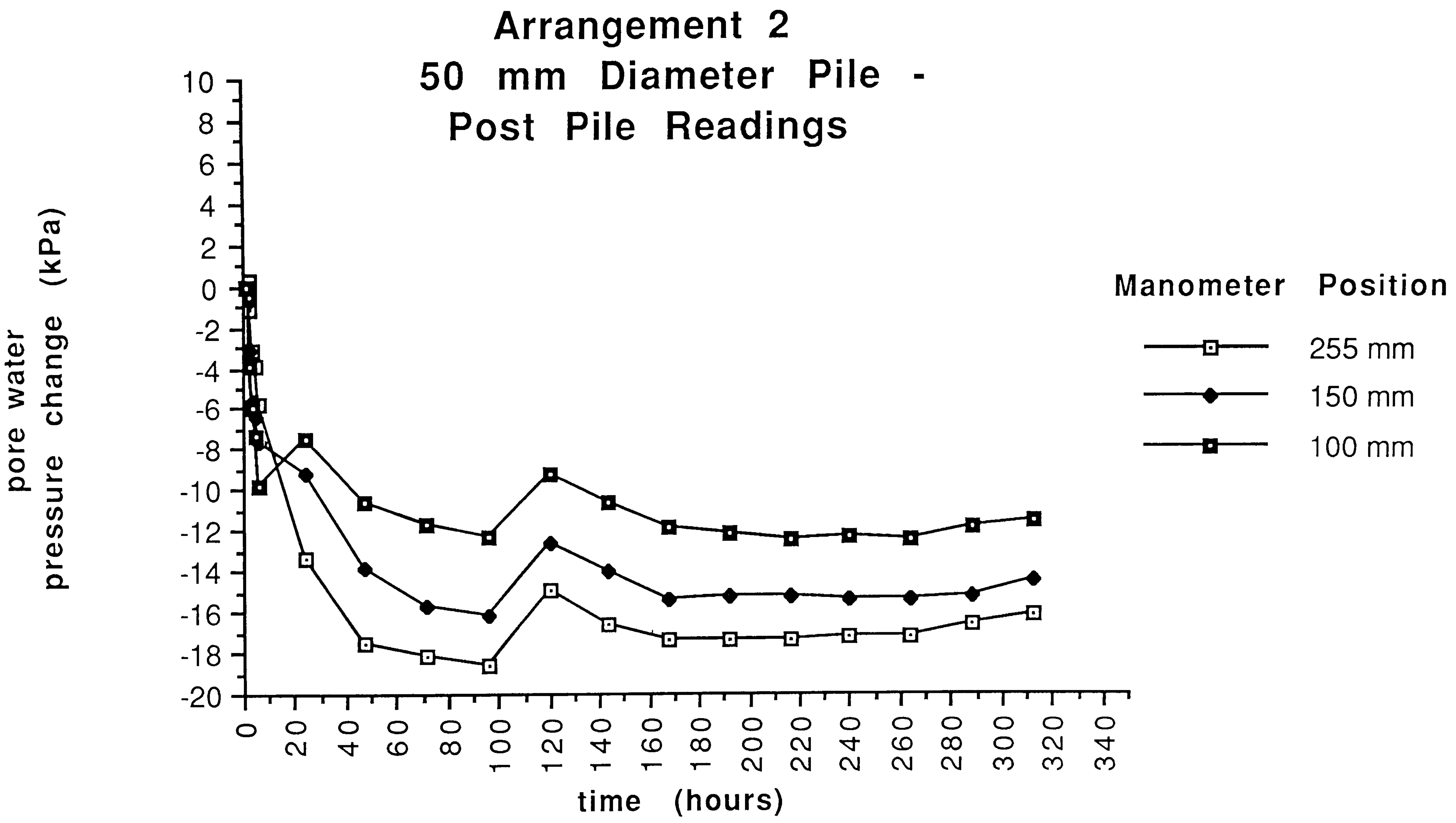


Figure 5.50 Arrangement 2 -
50 mm diameter pile -
post-pile pore water pressures

Figure 5.51 Arrangement 2 -
50 mm diameter pile group-
post-pile pore water pressures

Figure 5.52 Arrangement 2 -
100 mm diameter pile group-
post-pile pore water pressures

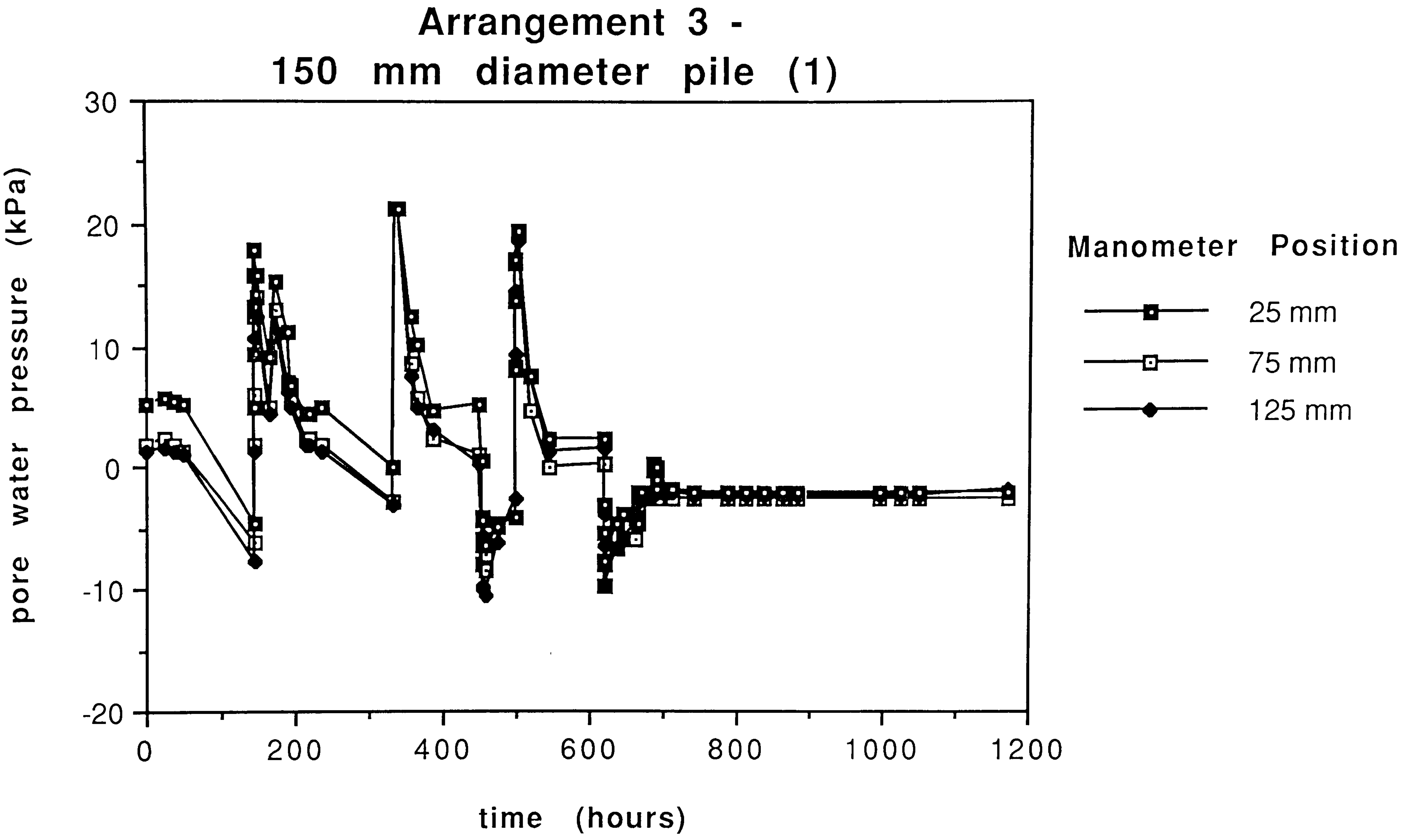


Figure 5.53 Arrangement 3 -
150 mm diameter pile (1)-
pore water pressures

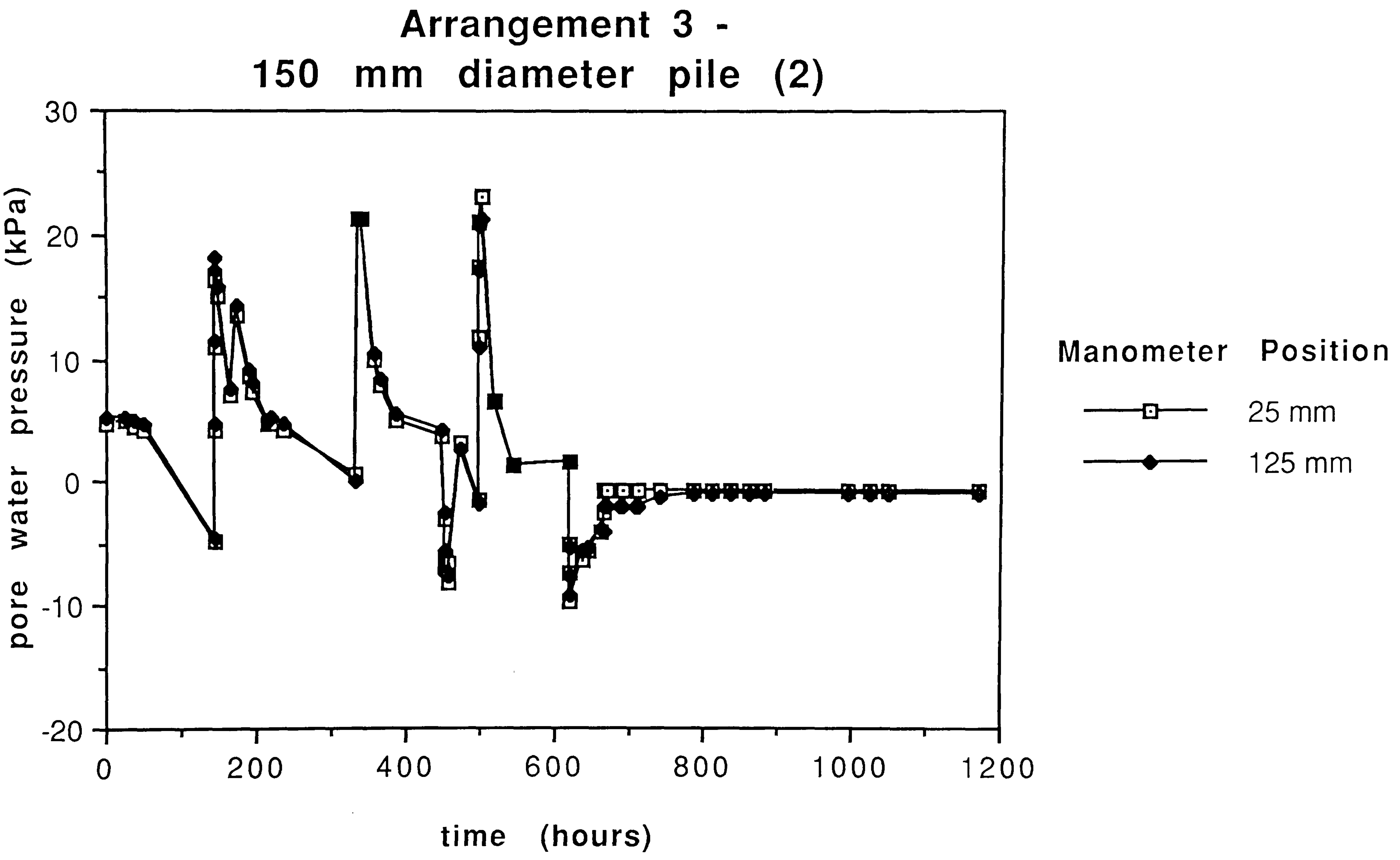


Figure 5.54 Arrangement 3 -
150 mm diameter pile (2)-
pore water pressures

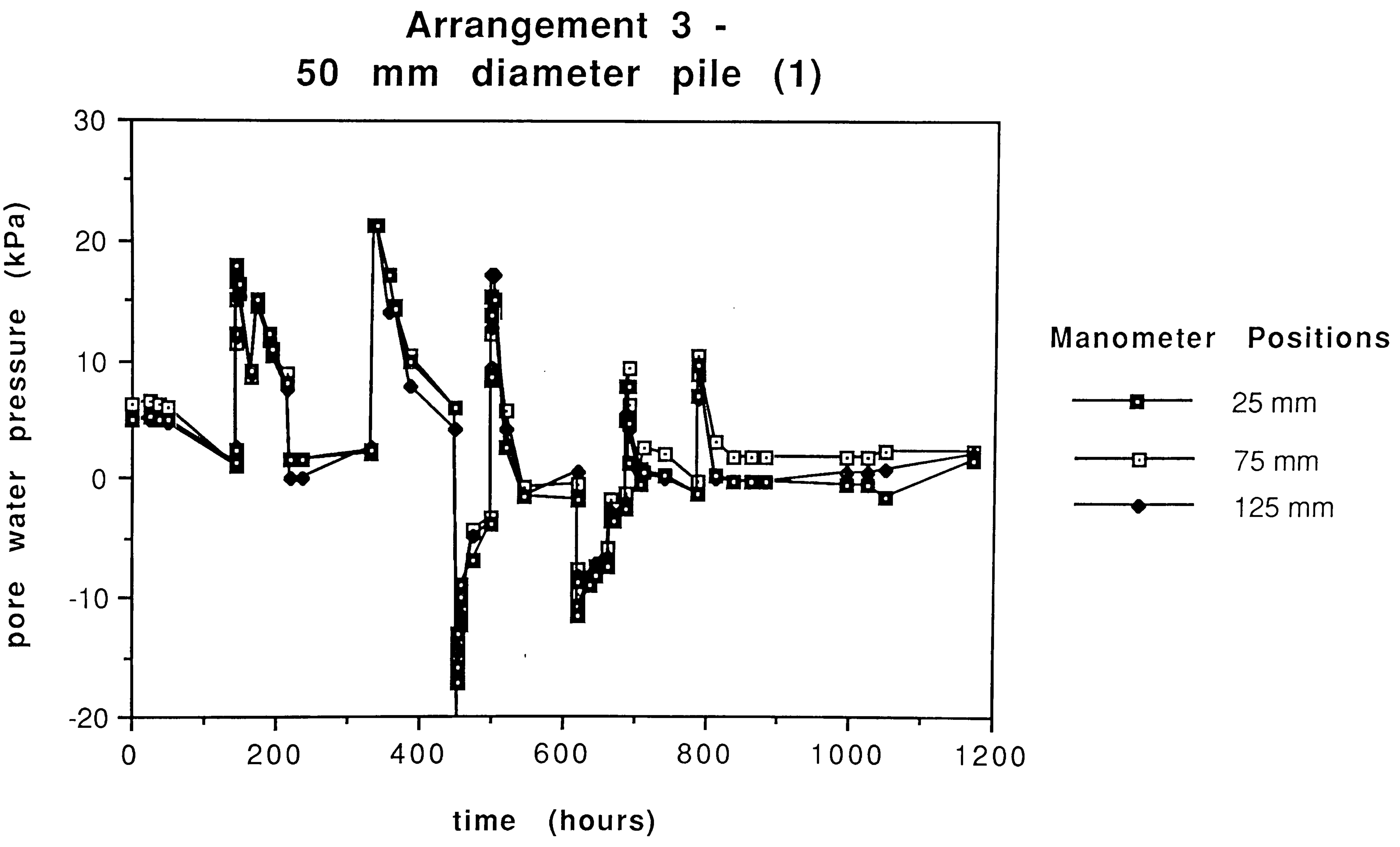


Figure 5.55 Arrangement 3 -
50 mm diameter pile (1)-
pore water pressures

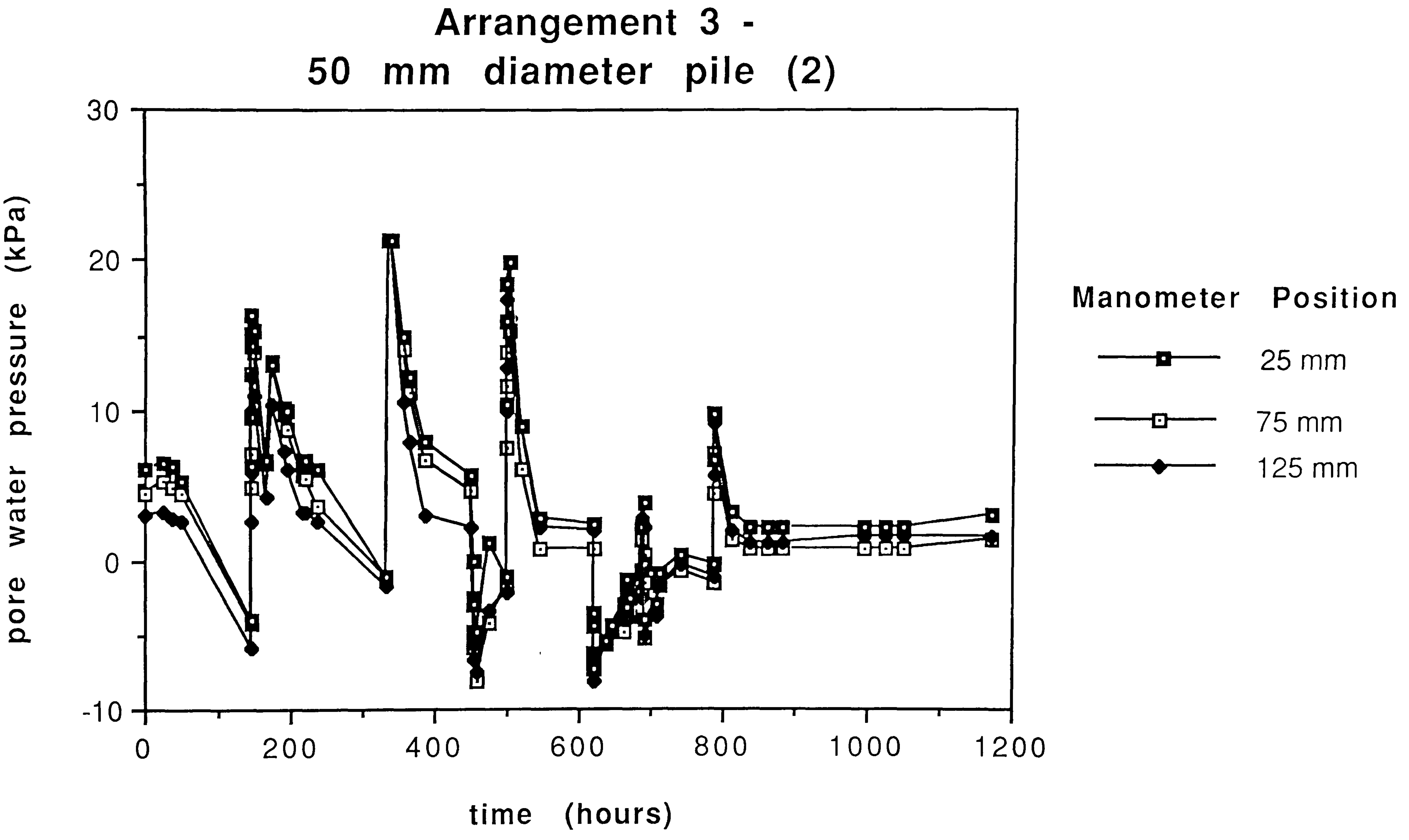


Figure 5.56 Arrangement 3 -
50 mm diameter pile (2)-
pore water pressures

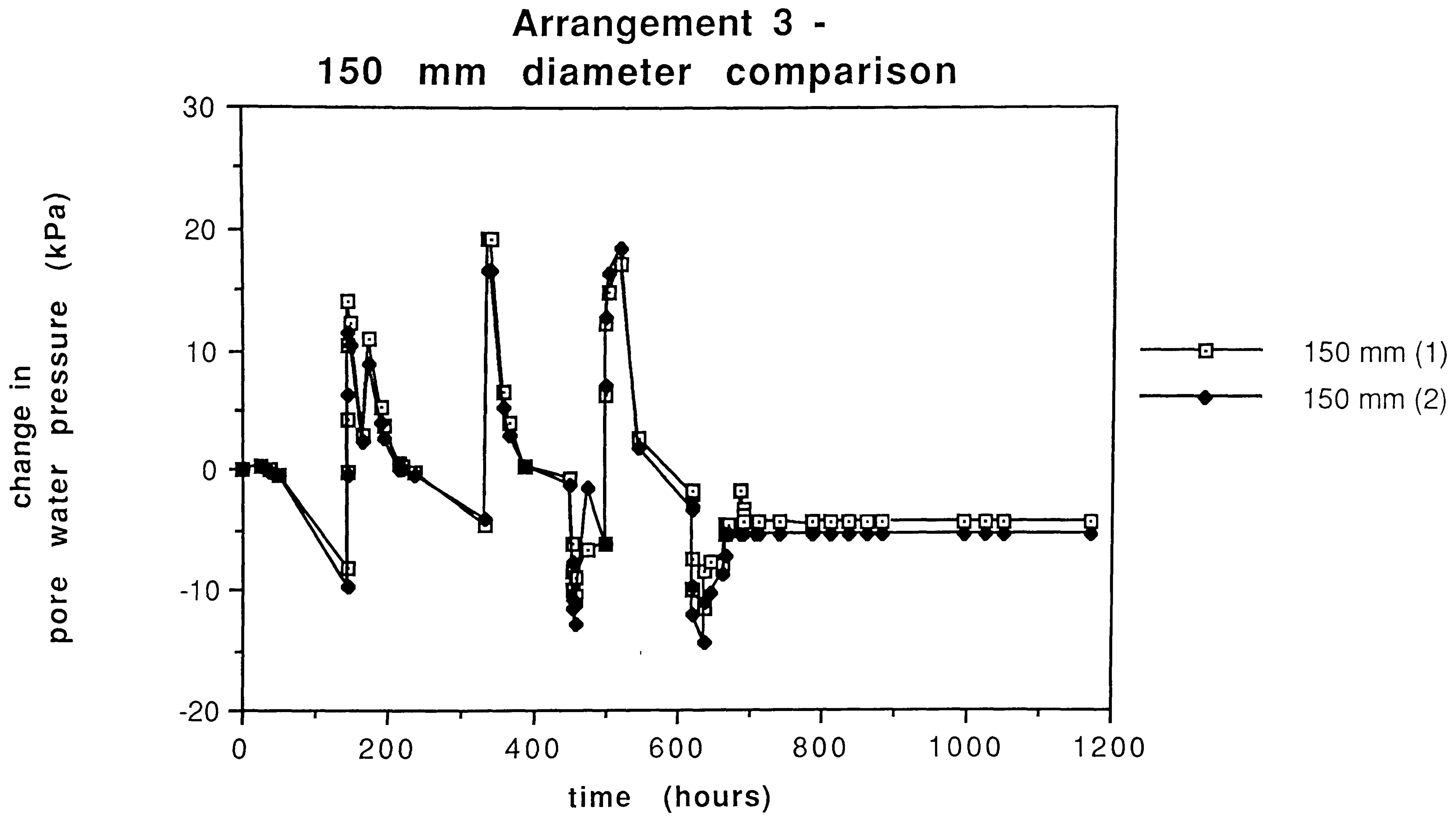


Figure 5.57 Arrangement 3 -
150 mm diameter piles -
comparison of pore water pressures

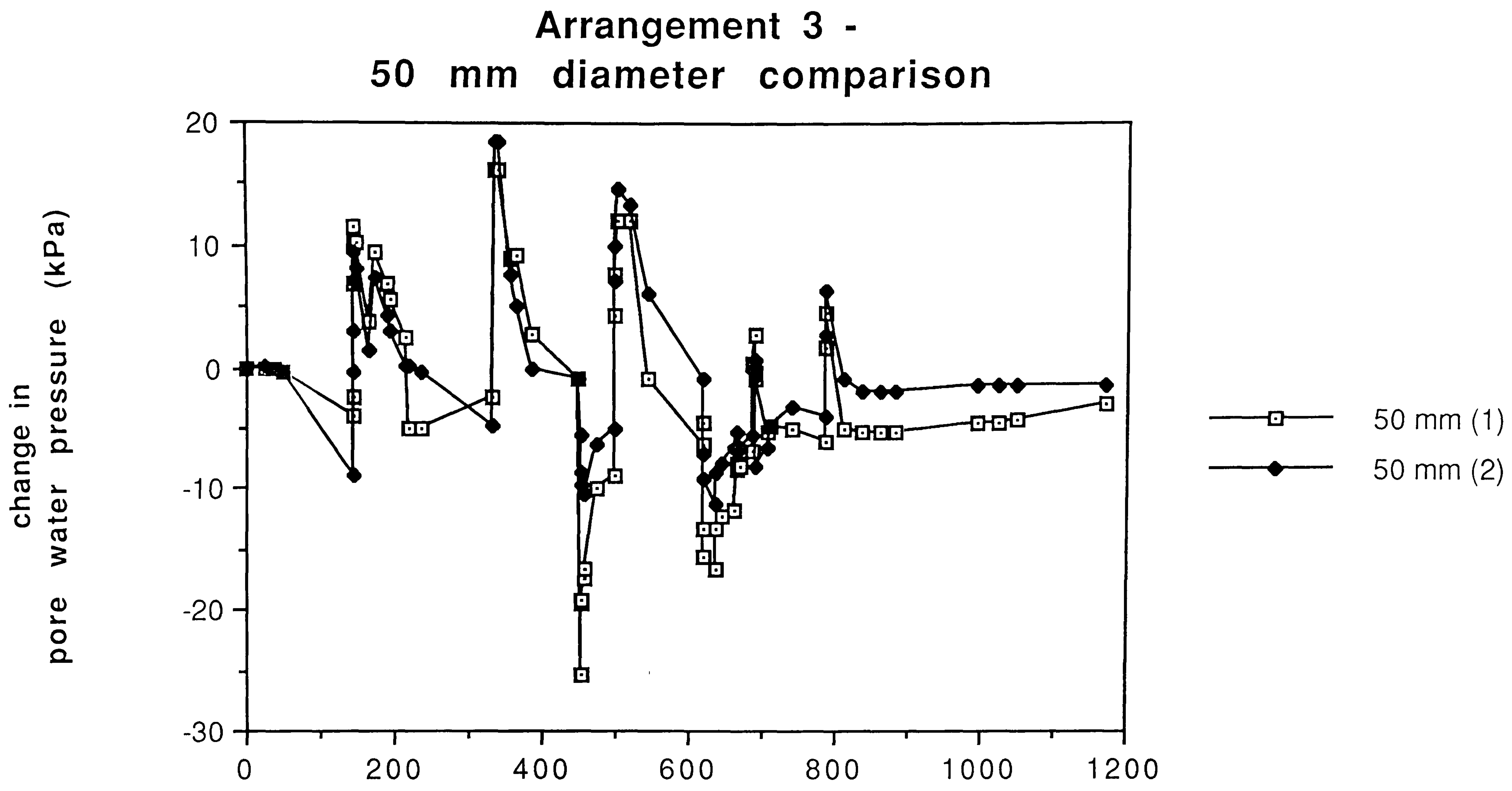


Figure 5.58 Arrangement 3 -
50 mm diameter piles -
comparison of pore water pressures

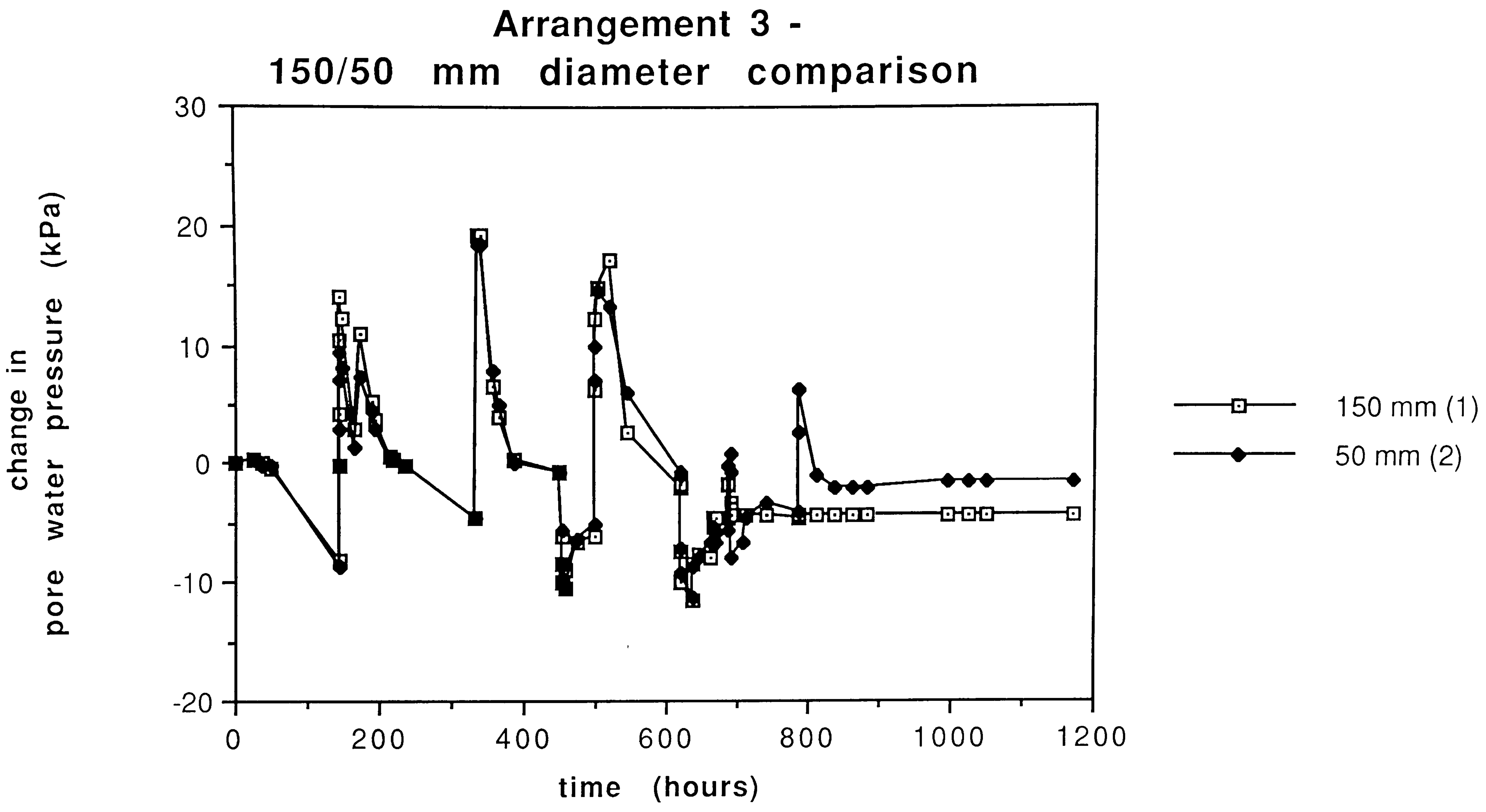


Figure 5.59 Arrangement 3 -
50 mm diameter pile (2) and 150 mm diameter pile (1) - comparison of pore water pressures

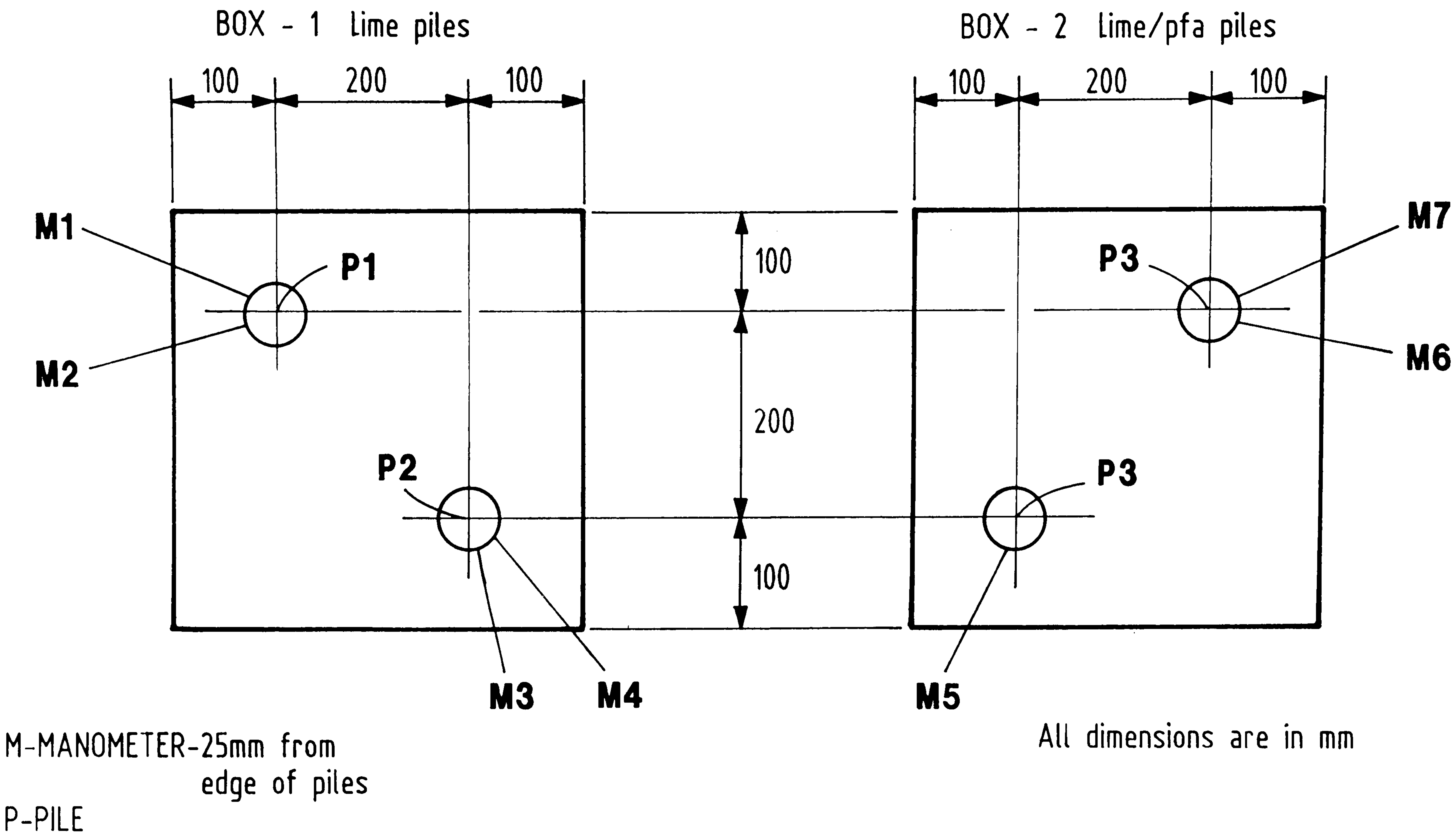
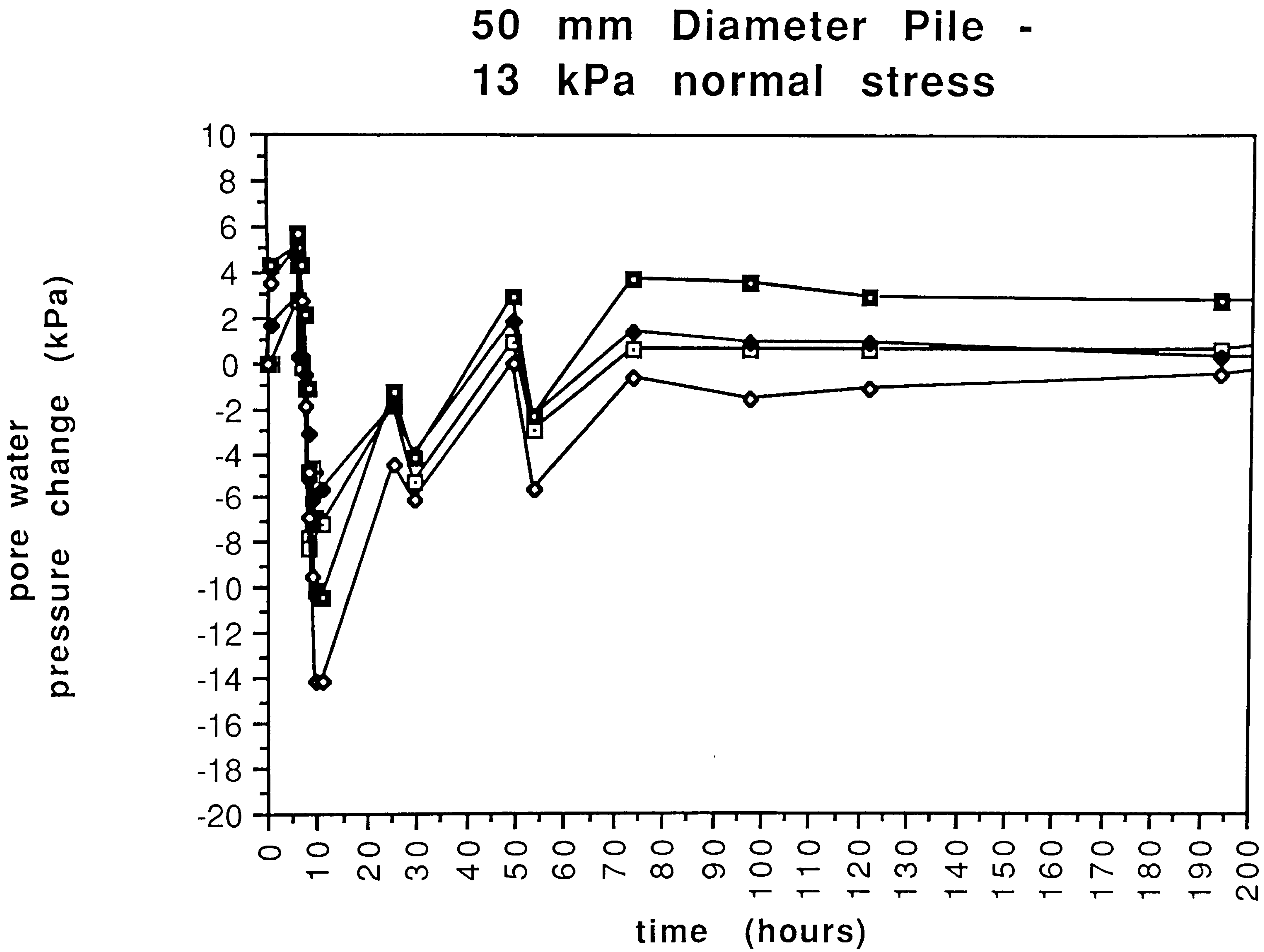
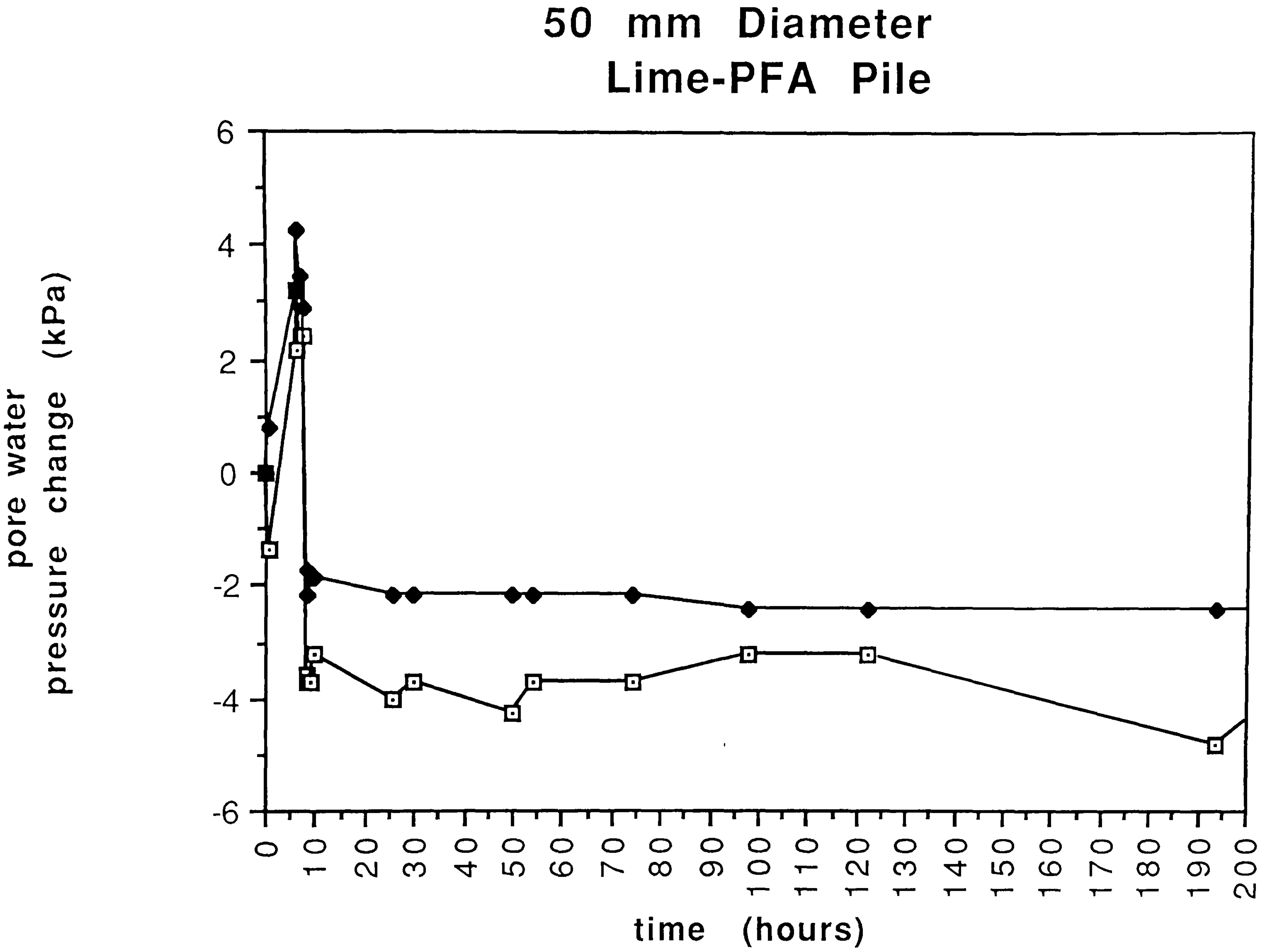


Figure 5.60 Lime-PFA pile tests - pile and sampling positions



Note: all manometers placed
25 mm from the pile

Figure 5.61 50 mm diameter lime pile -
pore water pressure changes



Note: both manometers placed 25 mm from the pile

Figure 5.62 50 mm diameter lime-PFA pile - pore water pressure changes

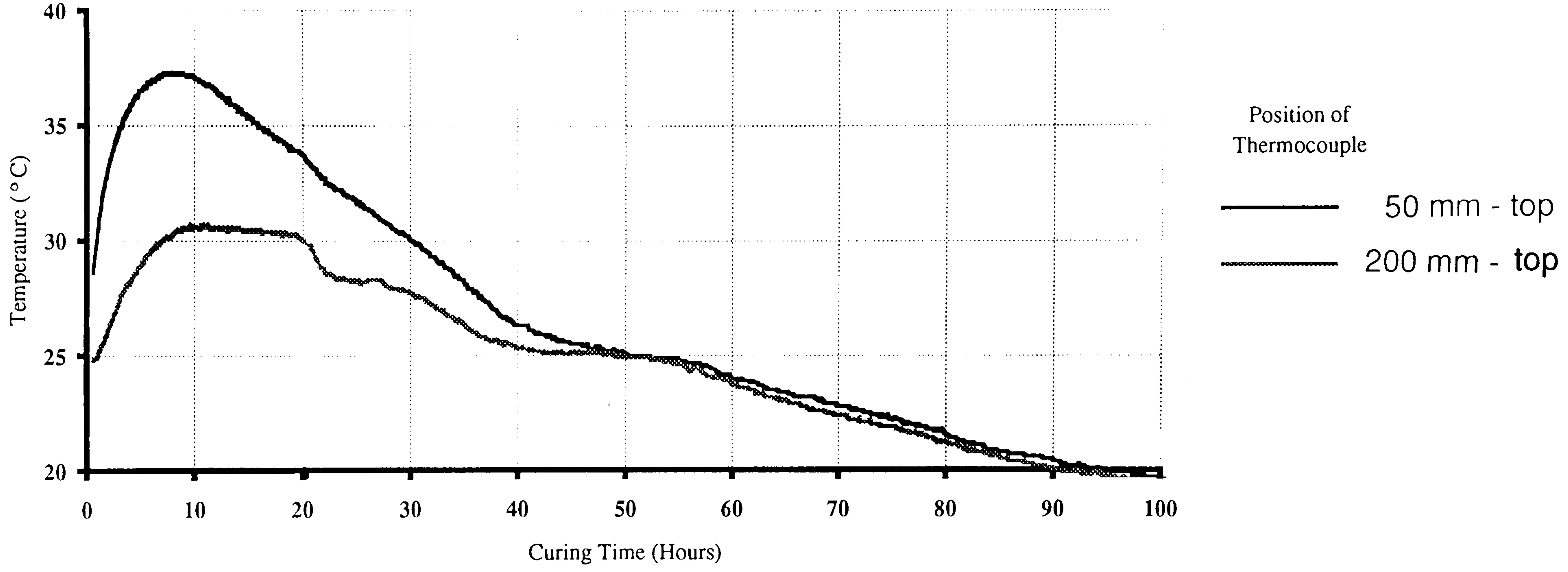


Figure 5.63 Arrangement 1 -
150 mm diameter compacted pile -
post-pile temperatures

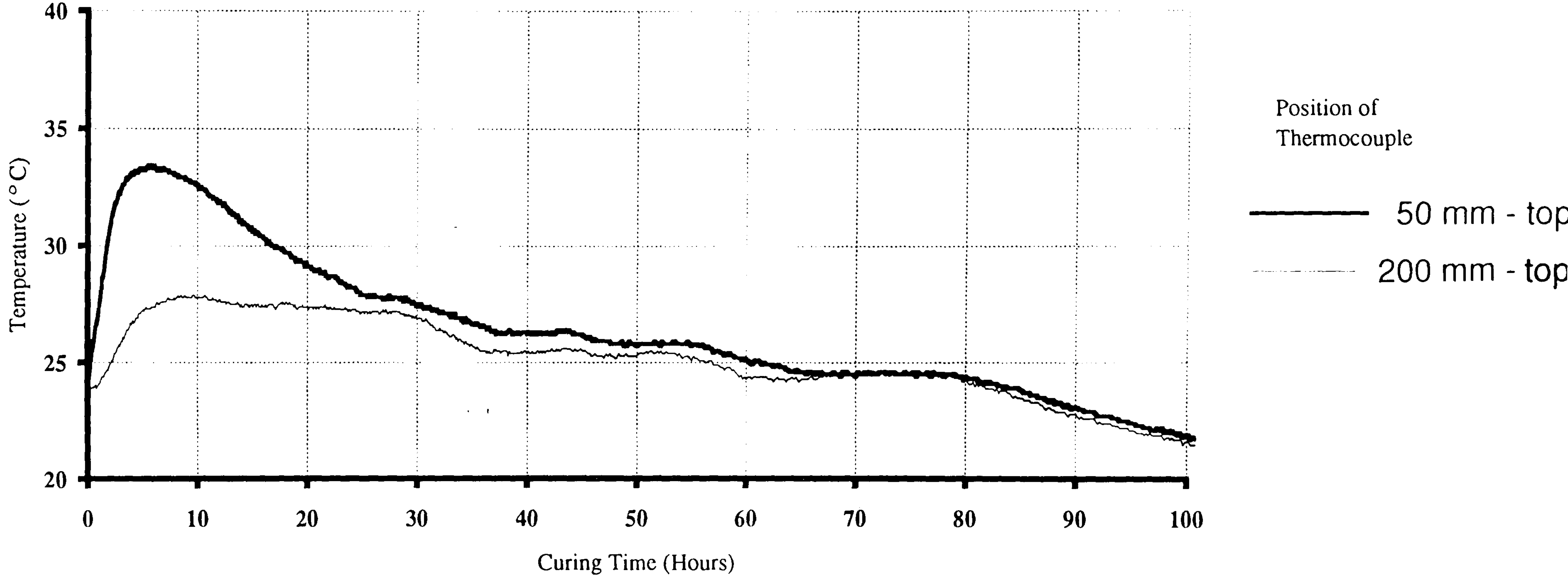


Figure 5.64 Arrangement 1 -
150 mm diameter pile -
post-pile temperatures

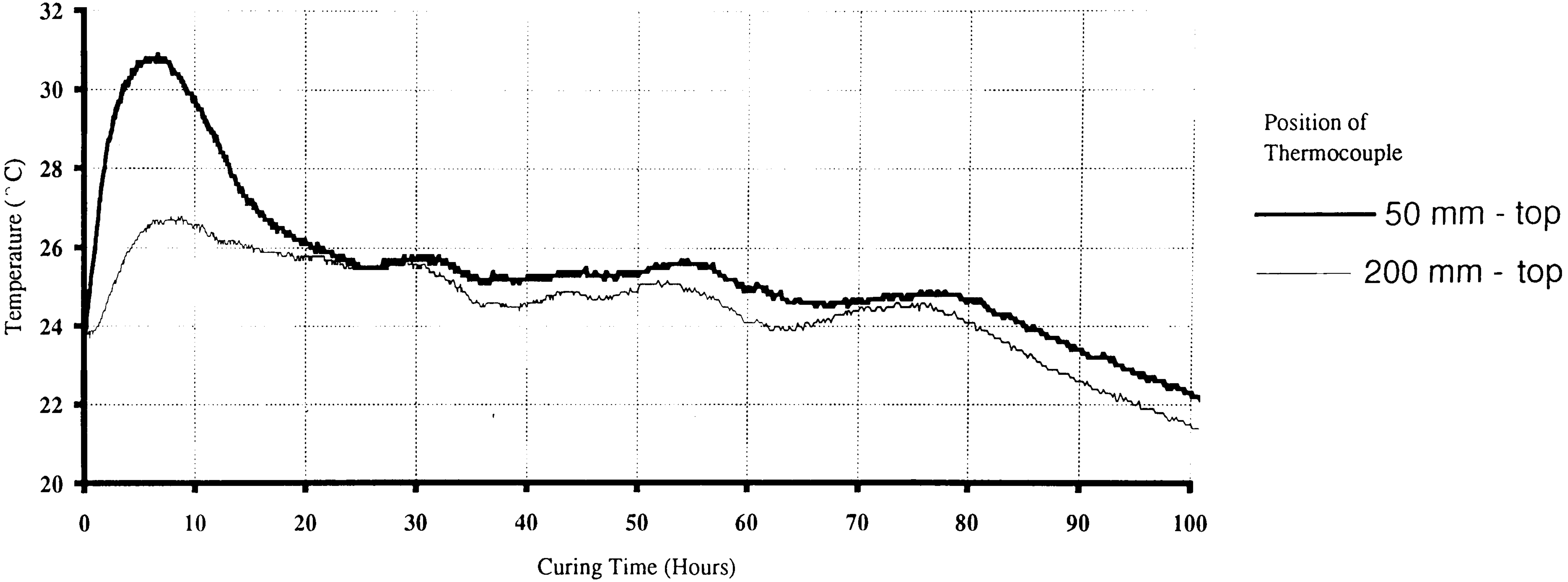


Figure 5.65 Arrangement 1 -
100 mm diameter pile -
post-pile temperatures

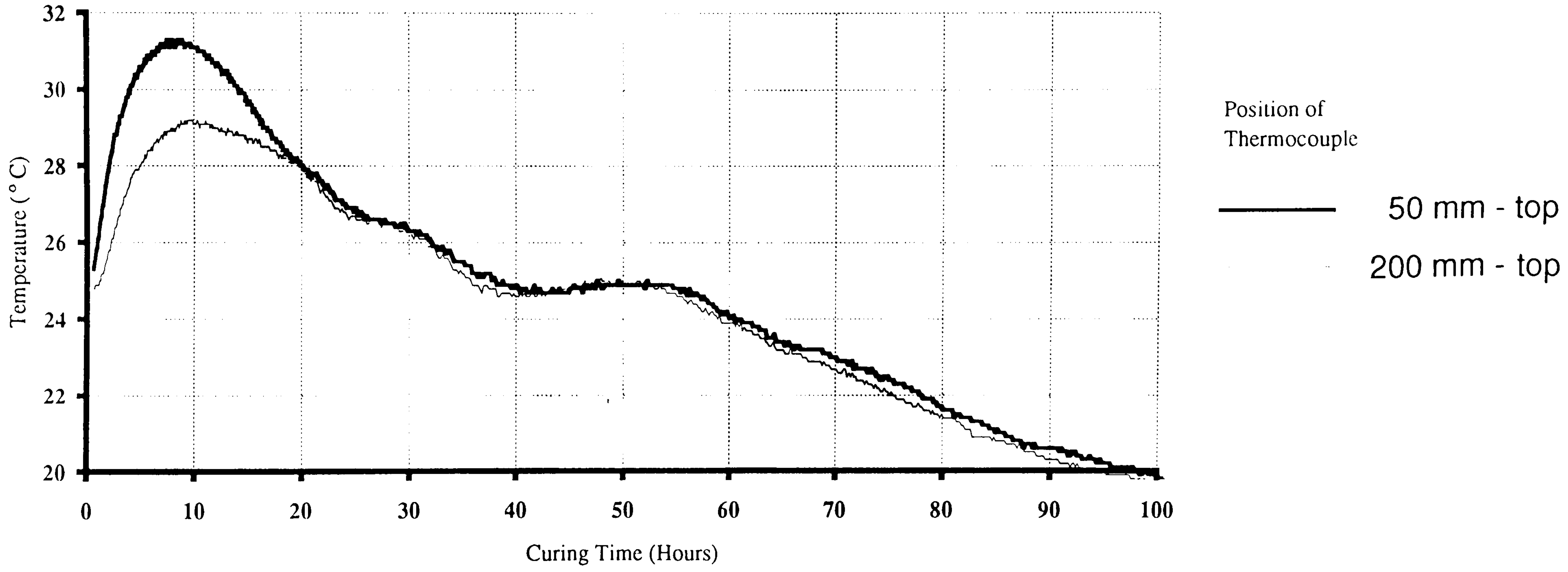


Figure 5.66 **Arrangement 1 -**
100 mm diameter compacted pile -
post-pile temperatures

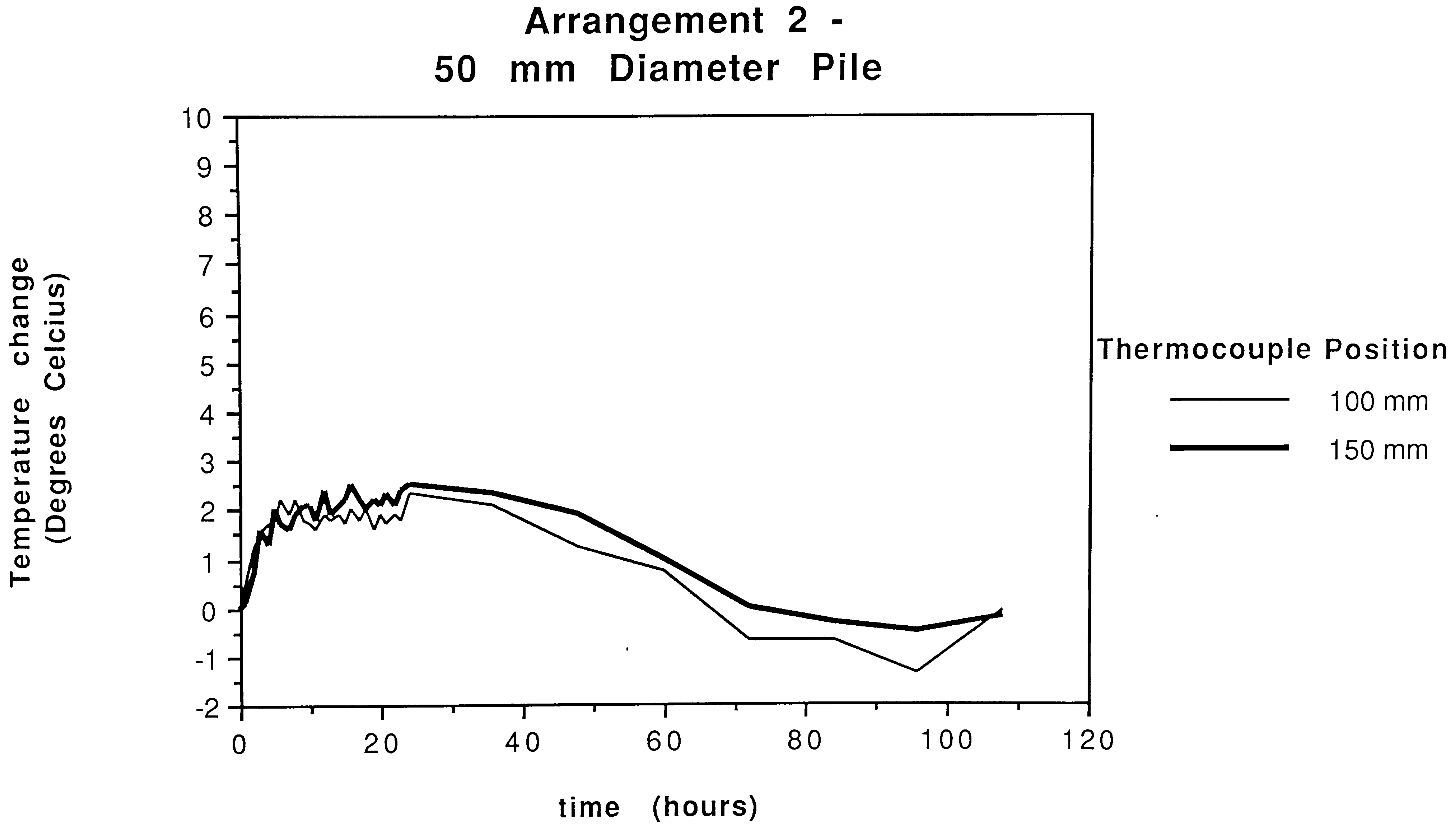


Figure 5.67 Arrangement 2 -
100 mm diameter pile -
post-pile temperature changes

Figure 5.68 Arrangement 2 -
50 mm diameter pile -
post-pile temperature changes

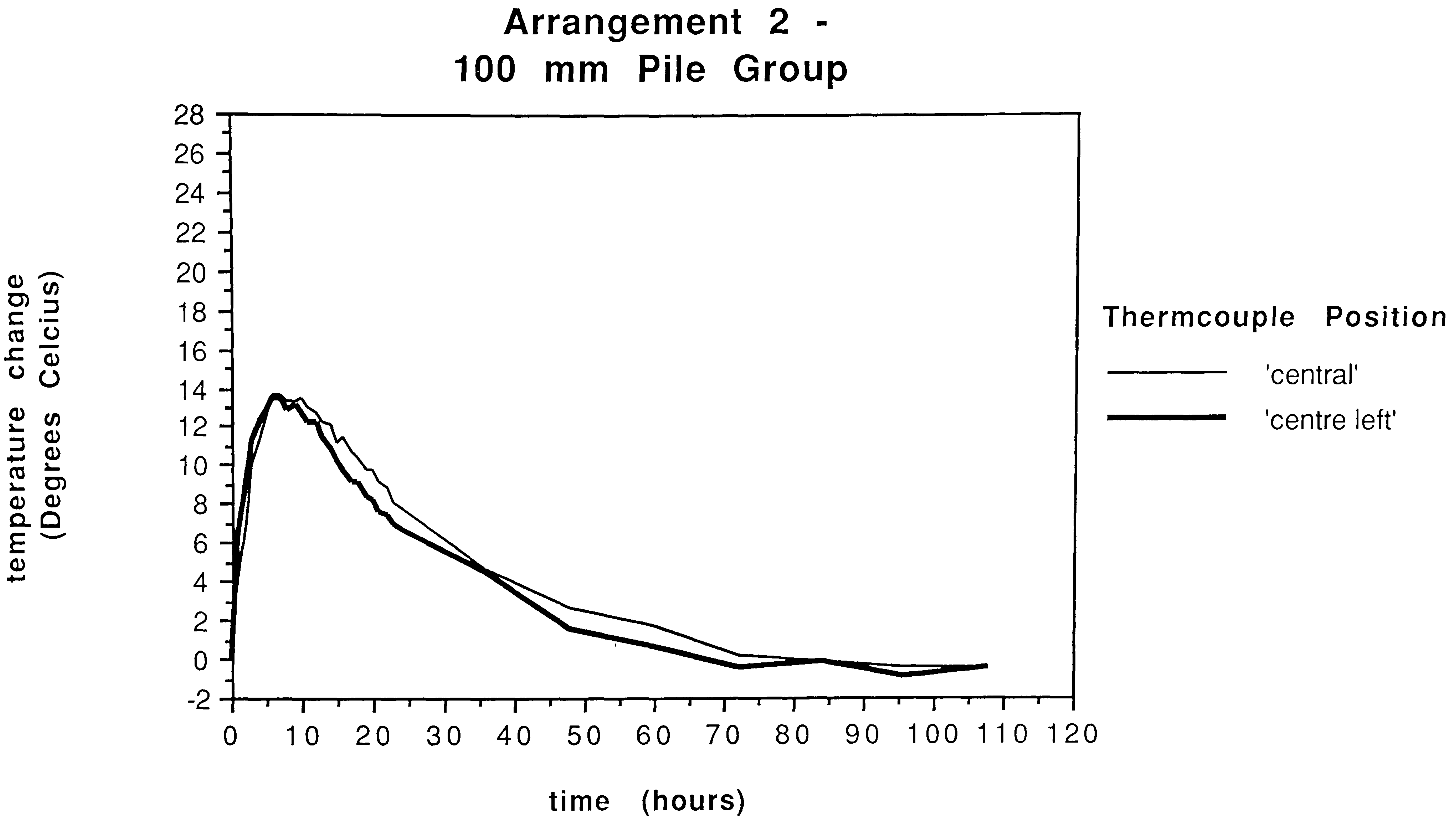


Figure 5.69 Arrangement 2 -
50 mm diameter pile group-
post-pile temperature changes

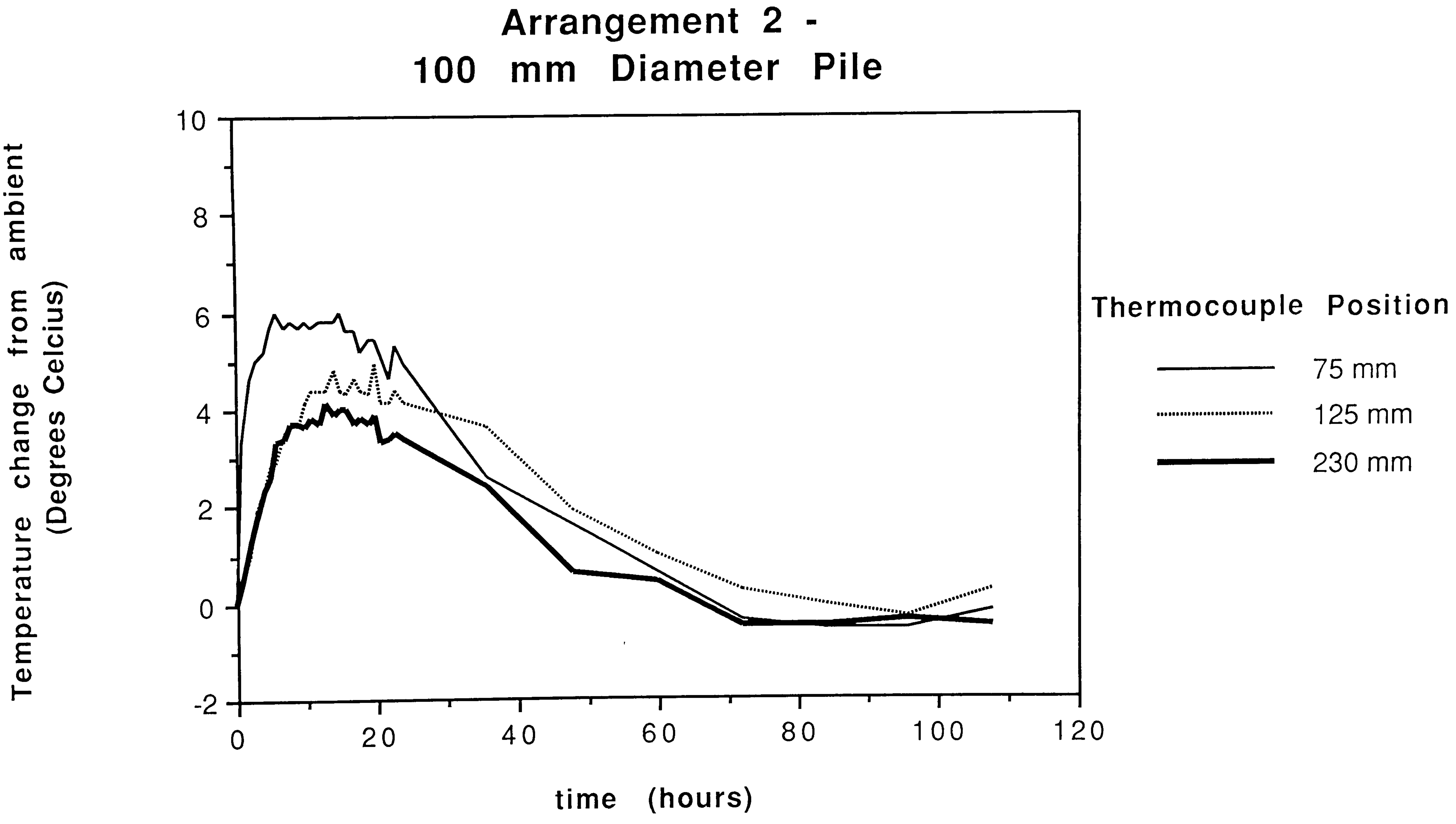


Figure 5.70 Arrangement 2 -
100 mm diameter pile group-
post-pile temperature changes

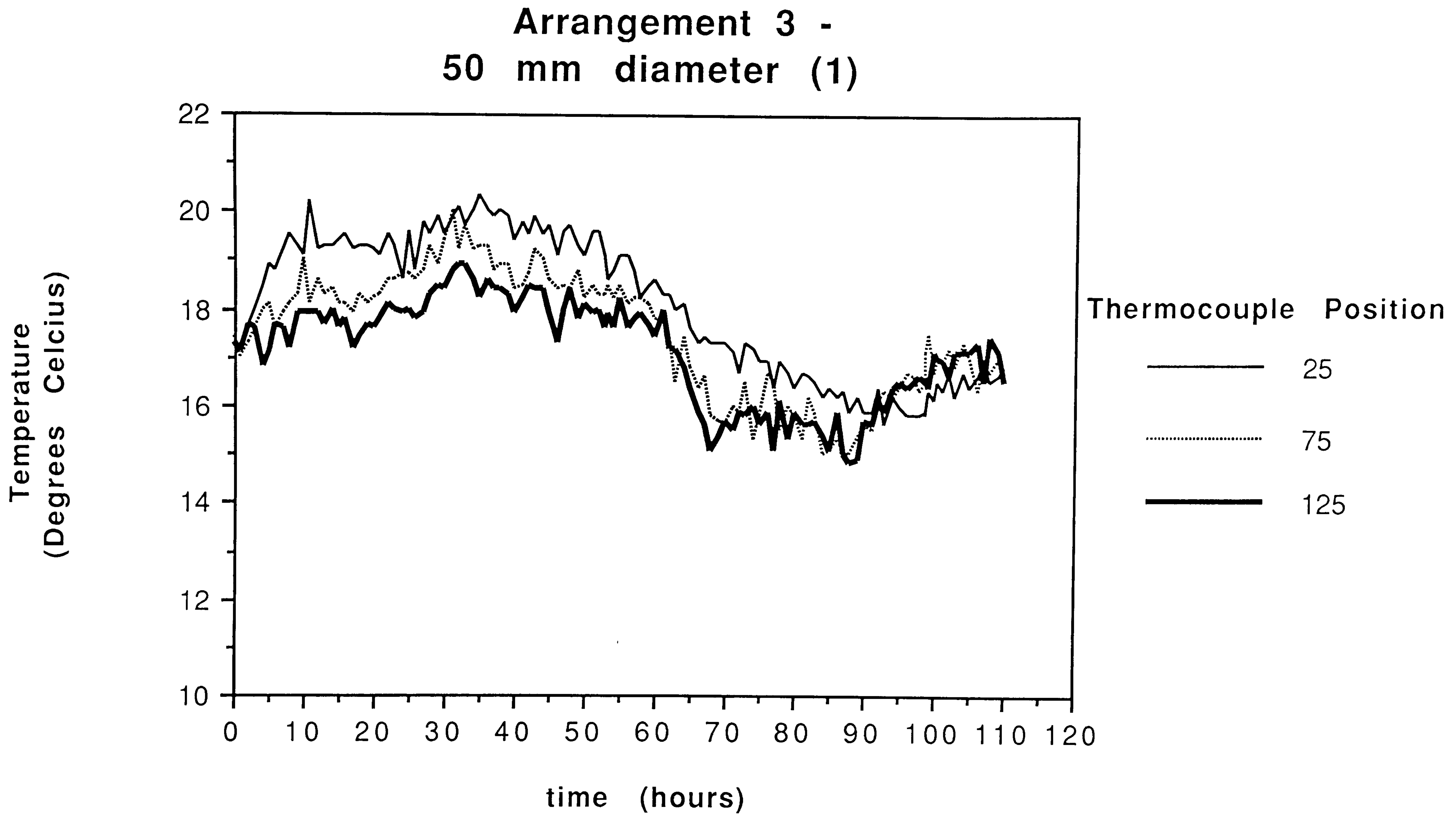


Figure 5.71 Arrangement 3 -
150 mm diameter pile (1)-
temperature changes

Figure 5.72 Arrangement 3 -
150 mm diameter pile (2)-
temperature changes

Figure 5.73 Arrangement 3 -
50 mm diameter pile (1)-
temperature changes

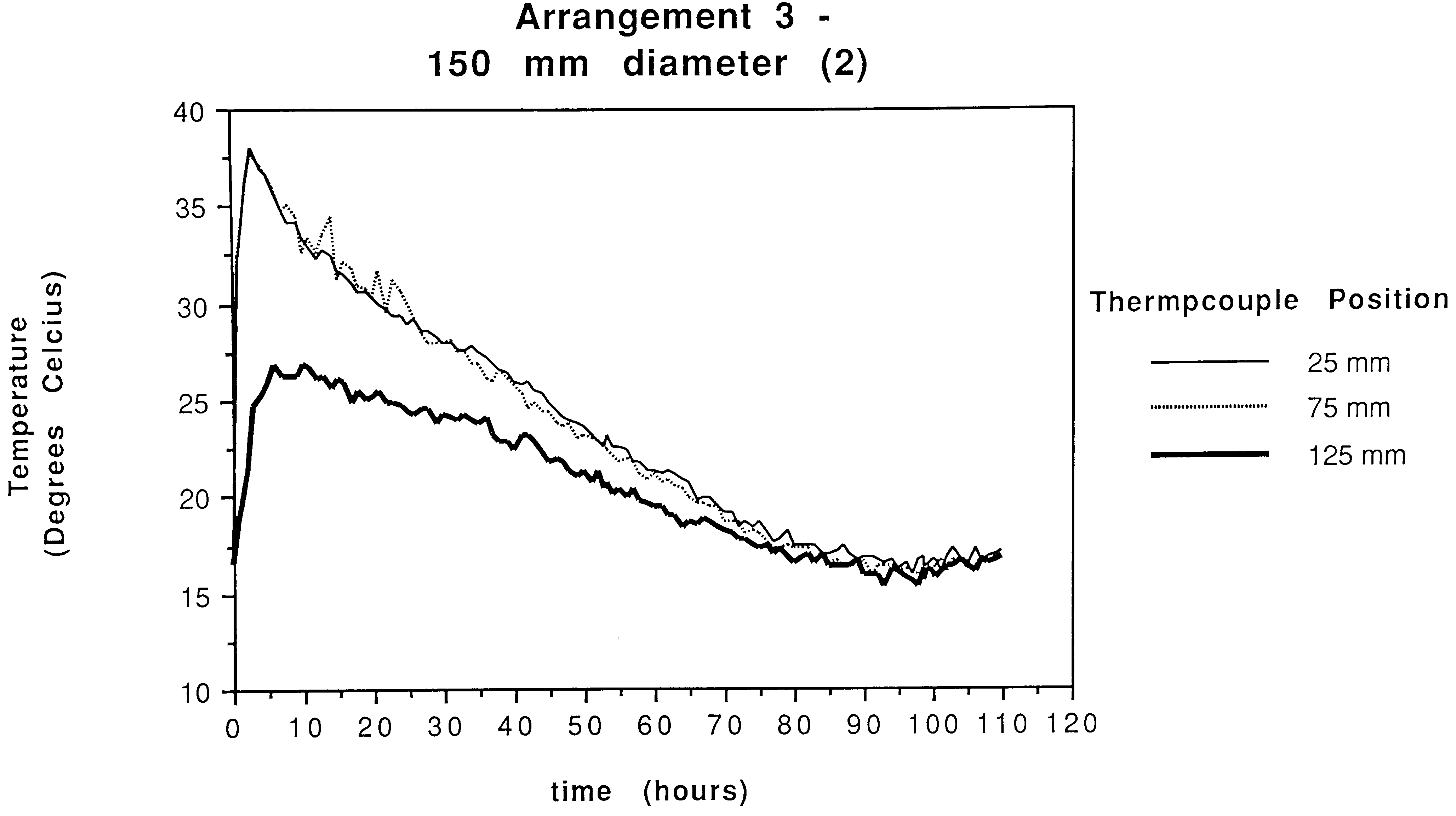


Figure 5.74 Arrangement 3 -
50 mm diameter pile (2)-
temperature changes

Infra-red spectrograph between 3100 and 4000 cm^{-1} of china clay after mixing with 1, 3 and 5% quicklime

Figure 5.75

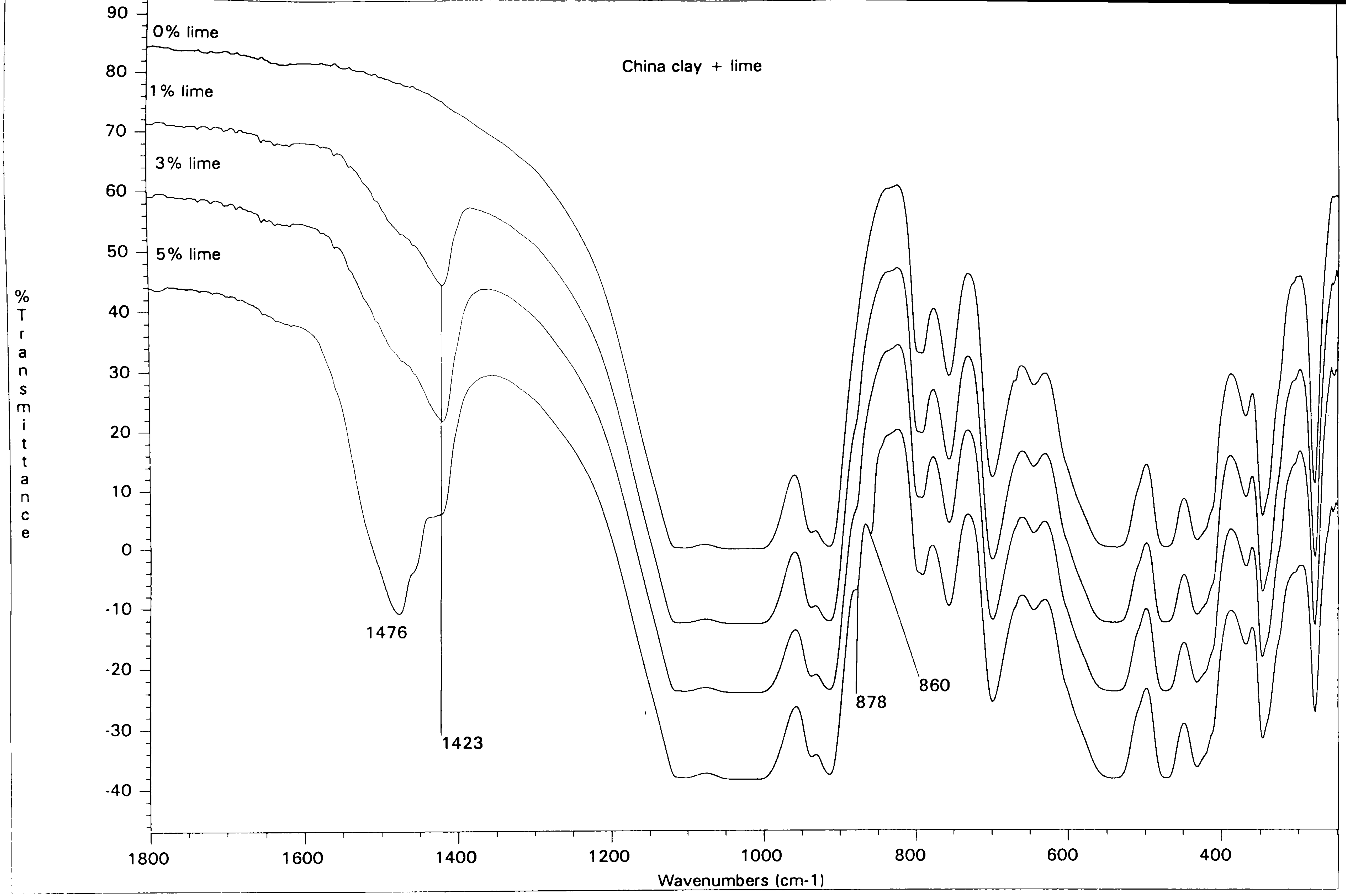


Figure 5.76 Infra-red spectrograph between 400 and 1800 cm^{-1} of china clay after mixing with 1, 3 and 5% quicklime

Infra-red spectrograph between 3100 and 4000 cm^{-1} of Wyoming bentonite after mixing with 1, 3 and 5% quicklime

Figure 5.77

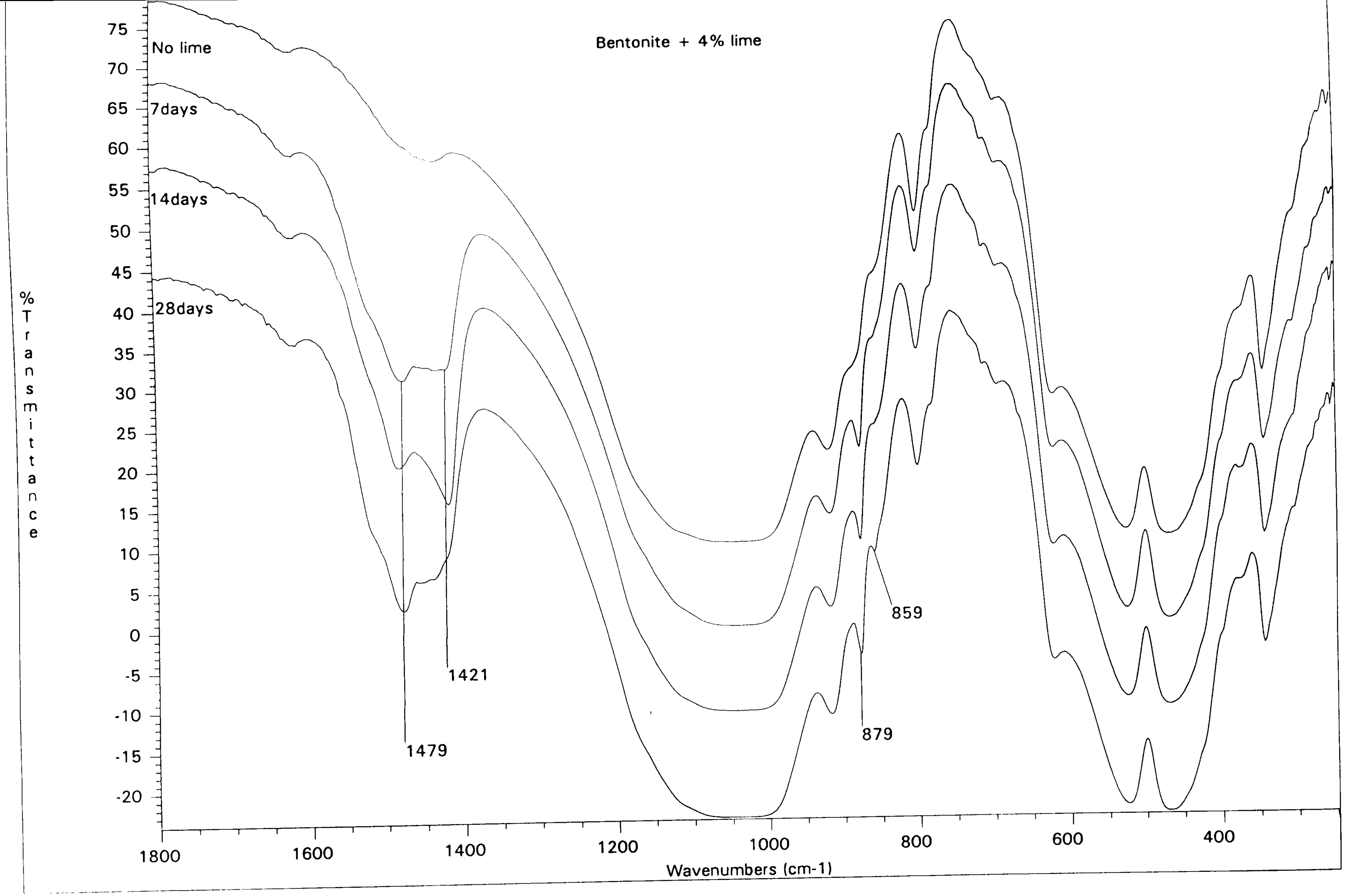
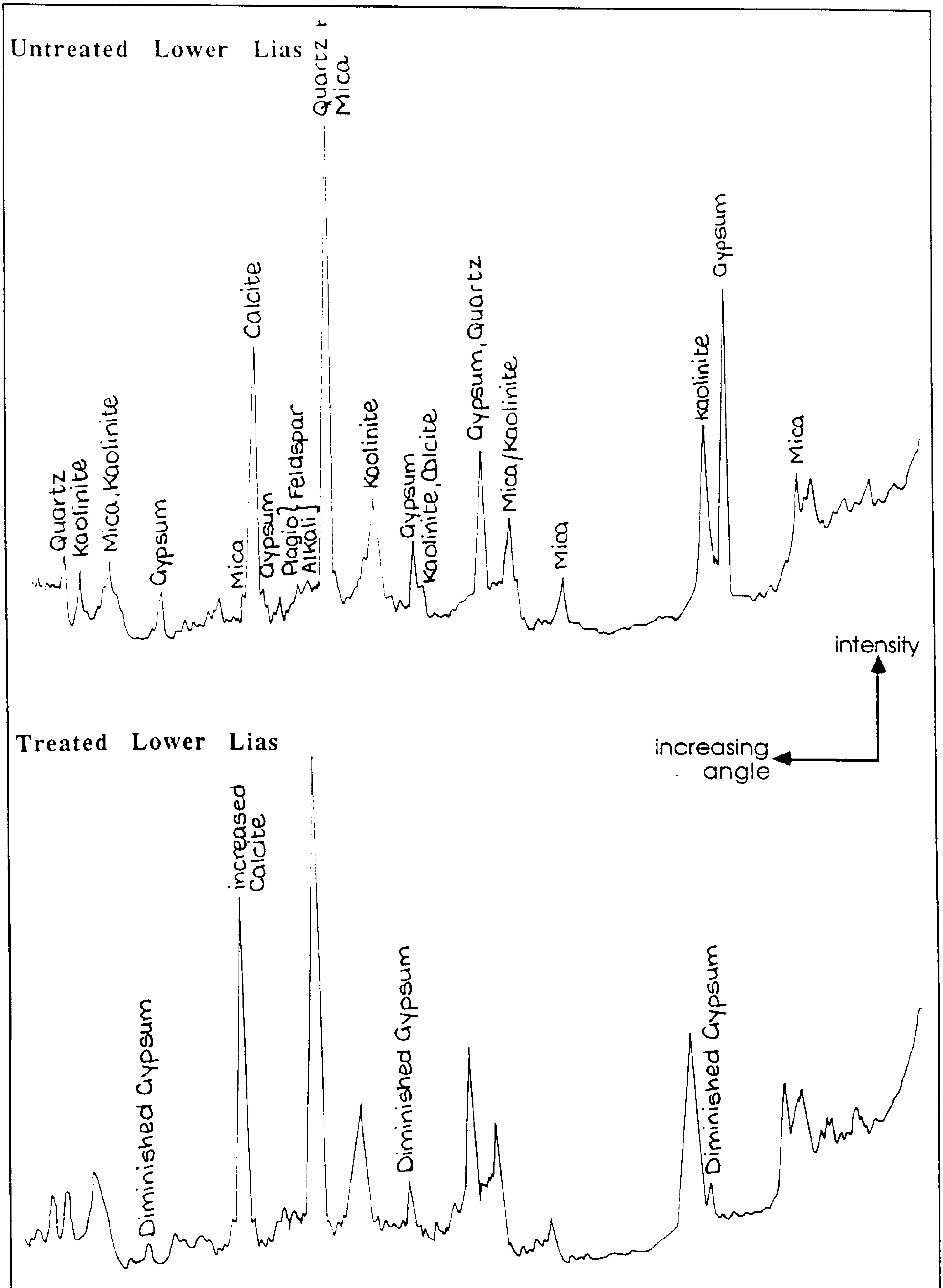


Figure 5.78 Infra-red spectrograph between 400 and 1800 cm^{-1} of Wyoming bentonite after mixing with 1, 3 and 5% quicklime

Figure 5.79 Infra-red spectrograph between 400 and 4000 cm^{-1} of lower Lias clay with and without

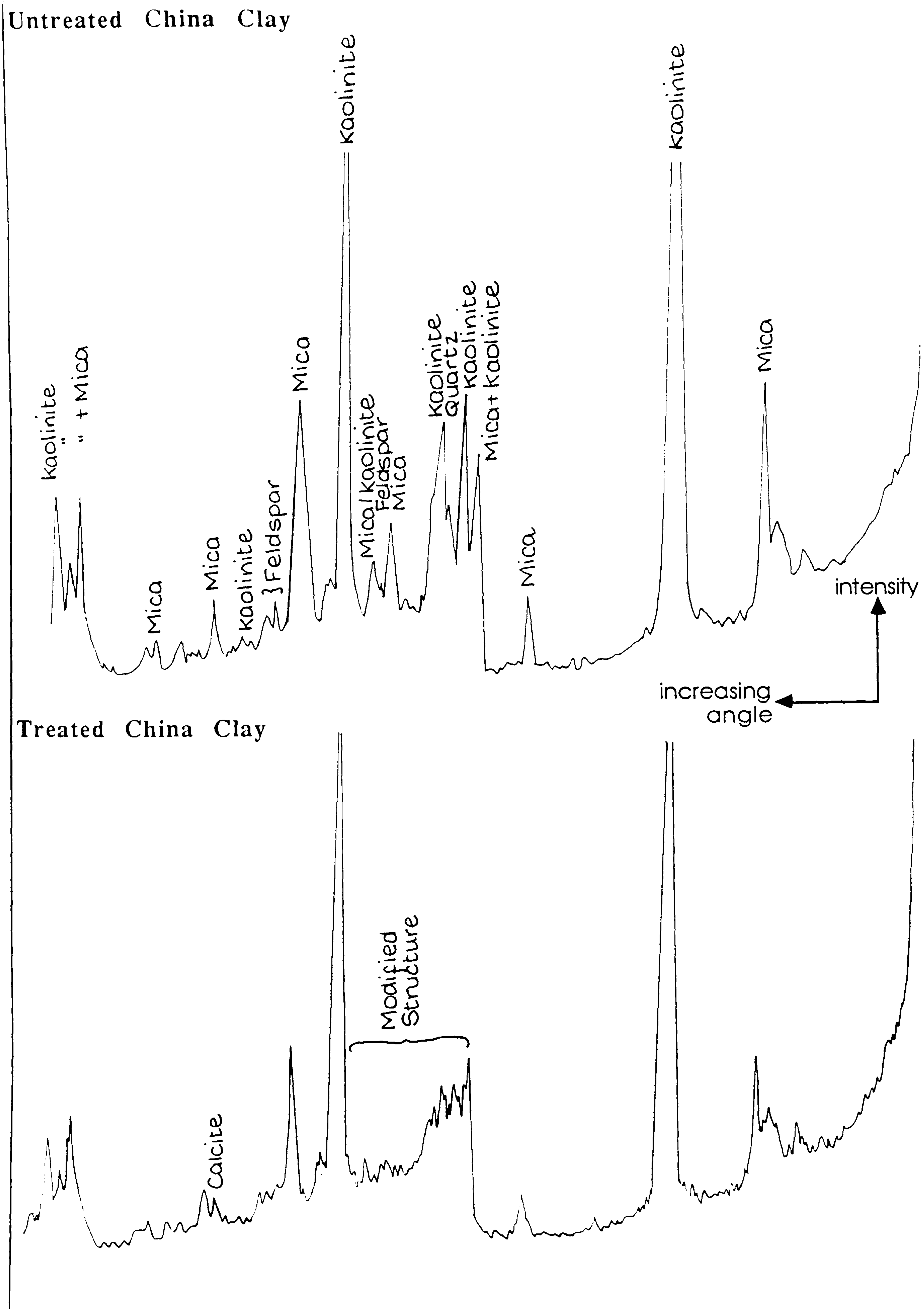


Untreated Lower Lias

Treated Lower Lias

Figure 5.80 X-Ray diffraction of Lower Lias clay with and without quicklime migration.

Figure 5.81 Infra-red spectrograph between 400 and 4000 cm^{-1} of china clay with and without quicklime migration



Untreated China Clay

Treated China Clay

Figure 5.82 X-Ray diffraction of china clay with and without lime migration.

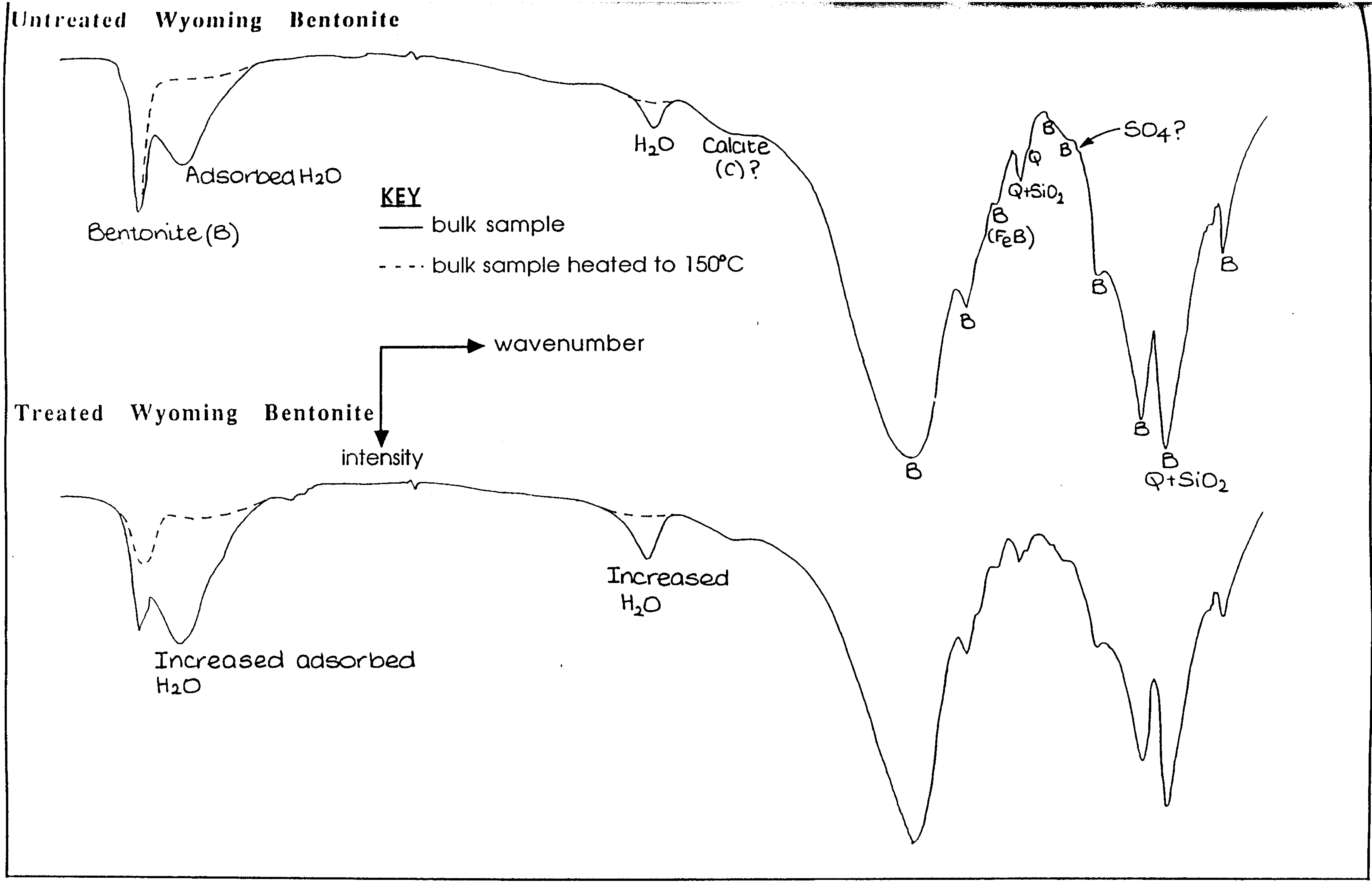
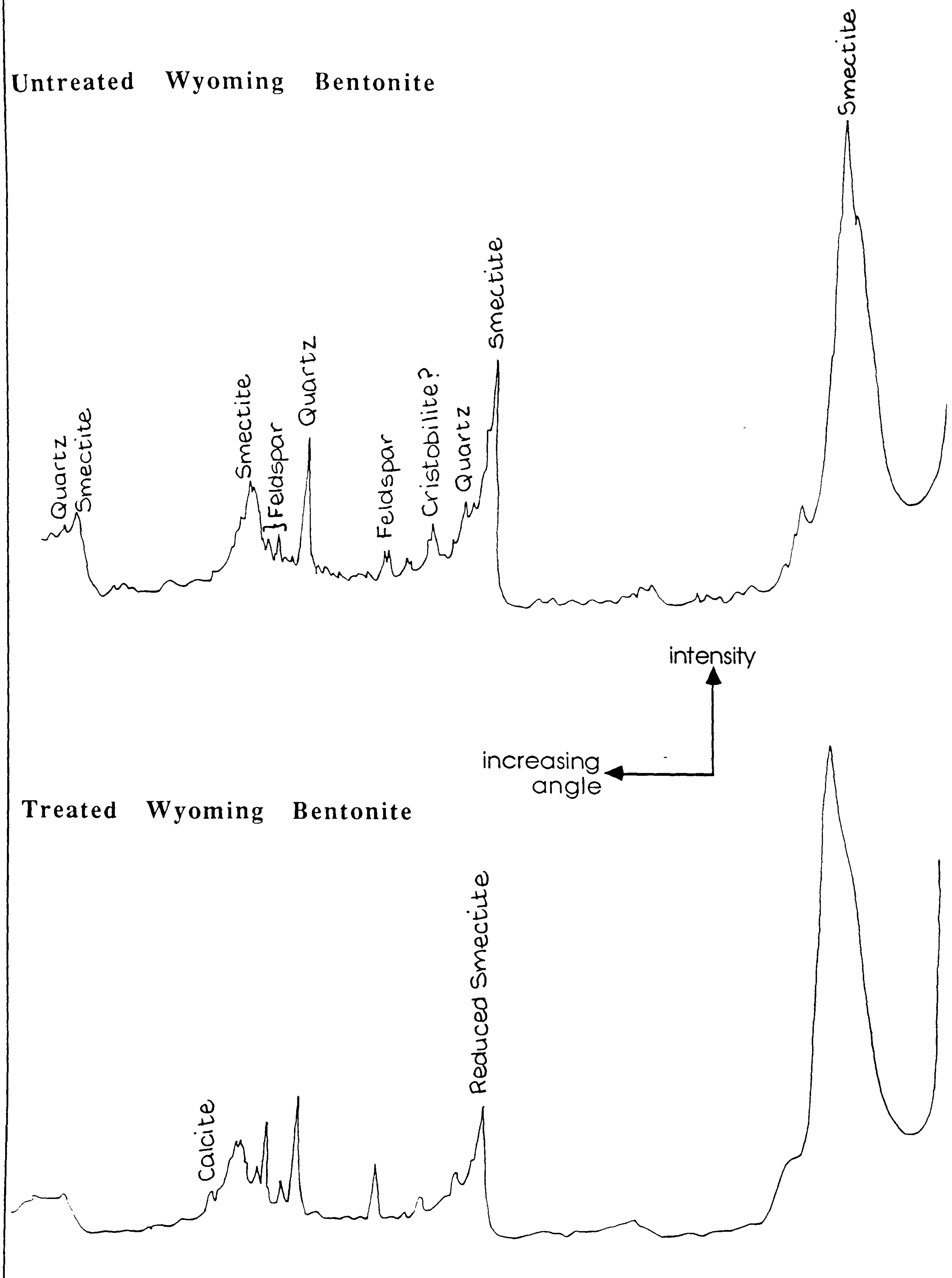


Figure 5.83 Infra-red spectrograph between 400 and 4000 cm^{-1} of Wyoming bentonite with and without quicklime migration



Untreated Wyoming Bentonite

Treated Wyoming Bentonite

Figure 5.84 X-Ray diffraction of Wyoming bentonite with and without quicklime migration.

CHAPTER 6

6.0 DISCUSSION OF RESULTS

In Chapter 5 the results from the laboratory investigation have been presented in such a manner as to illustrate the evidence to support the postulated stabilisation mechanisms. The data were presented under the sub-headings of those stabilisation mechanisms rather than presenting results for experiments in turn. The discussion will also take this form, each section culminating in a conclusion as to the likely contribution of that mechanism to the overall stabilisation process both in terms of magnitude and duration of the effect. This information will then be carried forward to Chapter 7 when design of full-scale installations will be discussed. The data required for this need necessarily be in a mechanism by mechanism form: hence the reasoning behind the presentation.

6.1 MIGRATION

The issue of lime migration remains not entirely quantified. Whilst some factors that influence the rate have been investigated in the laboratory, two essential influences in the field have necessarily been omitted: soil fabric and water movement. In order to fill perspex tubes, U100 tubes and boxes the clay has been reconstituted and compacted, leaving it without its original structure, wet of natural conditions and without the capacity for further ingress of water. Only the undisturbed U100 sample maintained the original soil structure (and even then this was only a single sample and as such might have been unrepresentitive of the structure of the soil as a whole), although, as with the compacted samples, it remained without the capacity for water movement. The attempt to create samples with regions of enhanced permeability and to generate water movement was fraught with difficulties, the most significant being that permeability could not be increased sufficiently to produce measurable flows of water without introducing another material, such as sand, into the system. This clearly defeated the object of the investigation.

It is planned to excavate selectively some of the piles used in the field trials at least one year after installation and examine the migration distances in an attempt to assess the above influences. Until then the discussion will be limited to the factors investigated in the laboratory and postulations of the effect of field conditions.

6.1.1 Water Content

As demonstrated previously, there appears to be a direct relationship between migration rate and water content. The most rapid transportation of ions is through solution and is a diffusion process. This means that ions move from a region of high chemical concentration (i.e. the pile) to a region of low concentration in a random manner.

It therefore seems logical that increasing water content increases migration rate as, not only is there more water present for the transportation of ions as water content is increased, but also the clay becomes more porous. Within a fixed volume, water is present where there once was clay. However, it must be remembered that it is a clay-water system that is being examined in which, unlike with an inert material where additional water will simply increase the space between the solid particles, there exists an interaction between the water and the clay mineral. (This is the reason for expressing the results in terms of Liquidity Index.) The mineralogy of the clay therefore influences the migration rate of the calcium ions also.

6.1.2 Mineralogy

Particle size and shape will undoubtedly affect the porosity of the clay and hence the migration rate. The relatively large platy particle of the English china clay has provided a material of relatively high porosity and hence produced the fastest migration rates. However, perhaps the better illustration of the effect of mineralogy is the result from the London clay sample, Figure 5.1. At low water contents the migration rate is comparable to that of the lower Lias clay, but as water is added migration rates increase to approach those of the English china clay. This is thought to be attributable to the interaction of the clay and the water. At a low water content the water present is largely inherently bonded into the clay structure and therefore cannot assist in the transport of the ions through pore space. As water is added, the clay structure becomes saturated, and hence additional water enters the pore space and aids ion transportation. This effect is most pronounced in the more plastic London clay. The lower Lias and the English china clay have lower plasticity (and indeed have identical values of Plasticity Index), so less water is inherently bonded into the clay structure and particle size and shape are the more influential factors. Particle shape and the amount of water inherently bonded into the clay structure are dictated by mineralogy. As these two factors influence migration rate, mineralogy must influence migration rate.

When mixtures of lower Lias and English china clay were used in the perspex tube tests and the values of the factor k at a particular Liquidity Index were compared with those of the parent clays, they appeared at intermediate values and in proportion to the amount of each clay present. It should be noted that although this effect is clearly demonstrated for these particular clays, it may be different for more plastic, swelling clays where the minerals themselves may interact.

In all cases it must be remembered that the factor k has been calculated for a specific distance and that this distance is less than the limiting distance for migration found in other laboratory experiments. As there is a reaction between the lime and the clay there are several other mineralogical influences on migration rate and the limiting distance. As the calcium ions migrate through the clay they will react and continuously reduce the concentration gradient, hence reducing the migration rate. The reaction will also change the nature of the material that the ions are migrating through and hence also influence the migration rate. It is known that mineralogy strongly influences this reaction and thus mineralogy has a further influence on migration rates.

Because of the problem of the migration being limited, as far as distance is concerned it has been difficult to compare values of k for different experiments as it was not possible to time exactly when a particular distance was reached in the majority of tests since sample disturbance would have resulted. Most of this discussion is therefore limited to the results from the perspex tube tests.

6.1.3 Slip Planes

It has been stated that increased porosity increases migration rates. In the perspex tube tests, pre-formed 'slip planes' did not increase migration rates above those produced by samples which did not contain them, although they were the sites for the initiation of the colour change and this must have had some small effect. This almost certainly means that the porosity of the material is increased by the formation of the shear plane to form the 'easiest route' for migration.

In one of the English china clay samples from the mini pile shear box tests increased migration in the direction of shear was observed. This must be attributed to particle orientation and, to a limiting degree, increased porosity. If that shear plane were also to be an area of increased water content this would enhance migration rates further. In all cases a discrete shear 'plane' has been formed. A shear 'zone', containing a region of relatively wet, remoulded material would almost certainly contain an increased porosity and hence migration rate.

In the field, within a shallow slip which will have relatively low values of effective normal stress, multiple, ill-defined shear zones will form. This region will consist of disturbed, not fully orientated particles, within a material of higher water content and porosity than that surrounding it. In this situation the degree of enhanced migration rate will be greater. The material is much more akin to the remoulded material referred to above than the reorientated material formed on a discrete shear plane.

6.1.4 Cracking

It was observed in the perspex tubes that cracks formed the boundary to some regions of migration. This may mean that continuity of water flow is not maintained across such a barrier even in a lateral sense (meaning migration is confined to the material between two radial cracks, with no migration between other, similar cracks) and so prevents migration.

In the large boxes, where radial cracks of several millimetres in width were formed, very little enhanced migration was observed along any of these cracks.

In both the perspex tube and the large box tests there was no facility for water ingress. However, if these cracks were subsequently to form a route for water to ingress the pile from an external source, further enhanced migration along these cracks may be found. The region of water within the crack would provide an ideal environment for diffusion of lime from the exposed pile.

6.1.5 Field Considerations

It must be appreciated that the samples of clay set up in the laboratory are by no means representative of what may be found in the field. In order to create a homogeneous material, samples have been prepared at relatively high water contents which minimise air voids. Thus, in effect, clay barriers have been created which, in other circumstances, may be expected to reduce the movement of chemicals in the soil. It has been shown that areas of increased porosity in a sample increase migration rates so long as water continuity is maintained. This may be provided by the natural fabric in the ground. The attempt, in the U100 experiment, to examine migration in an undisturbed sample was unsuccessful as the water content was very low and there was no facility to simulate groundwater.

It has been suggested by Handy (1994) that the radial cracks formed by the initial reaction fill with water, causing further expansion of the pile into the cracks, and enhance migration rates over a distance of up to 0.5 m. He has also experienced similar migration rates to those discussed above. Excavation of the field trials at a later date should provide further data on the subject.

6.1.6 Conclusions

Several conclusions may be drawn about the influences on migration of hydroxyl ions in clay soil. The most pronounced effects are that migration rate is proportional to water content and that increased migration rate is related to porosity.

Within the perspex tube samples, migration does not occur evenly over the whole sample, but begins in small patches that spread. Discrete slip planes seem not to promote detectably greater migration rates but do coincide with the first signs of migration. These first signs are not, however, seen earlier than in samples which do not contain slip planes. This probably means that migration occurs along preferential paths, be they areas of less well compacted material or pre-formed slip planes with orientated particles.

Discontinuities in migration paths often occur at cracks, migration being present on one side of a radial crack over the entire length of the tube and not present at all on the another side. Migration, if non-uniform, often seems also to coincide with the swelling of the pile material. Both of these facts may be an indication of the requirement for movement of water. The crack acts as a barrier to movement and the pile swelling shows that movement of water has taken place.

Results also seem to show that while lime slurry provides lime migration rates similar to that of quicklime, the slaked lime produces very much reduced rates of migration. This may be because, unlike quicklime and slurry, no significant potential for water movement is produced.

What is not immediately apparent is why some areas of the lower Lias clay samples show no signs of migration at all, while others show a complete colour change due to the indicator. It is possible that there is insufficient residual water after the slaking of quicklime in these areas to produce sufficient continuity of movement.

This type of test could be further used to study influences on migration rates. It could be adapted to study such factors as permeability, pH, temperature, clay content, water flow, pile expansion and mineralogy.

Literature alluding to the inappropriate use of pH indicator to provide an indication of the presence of calcium ions (i.e. that the assuption, used elsewhere, that hydroxyl and calcium ions migrate at the same rate) was discovered whilst writing this thesis. The discussion has been left unaltered as it reports accurately the results found using this method. However, Quigley et al (1990), show that whilst both the hydroxyl and calcium ions have diffusion constants of a similar order, there is evidence of calcium ions 1.2 m away from the base of a landfill site within 15 years of placement of the fill. Hydroxyl ions were not present at this distance. Quigley (1994) suggested that this is because the migration of hydroxyl ions is attenuated by the clay to a far greater extent than that of the calcium ions. If calcium ions do migrate over this order of distance, after a period of 10-15 years an entire area treated using lime piles would show an increased ion content if sampled. Whilst stabilisation would only occur to the extent already discussed, modification could take place over the entire treated zone. This is because stabilisation would require the elevated pH produced by the hydroxyl ions, whereas modification would occur as a result of ion exchange alone. With experiments of the time-scale carried out, very little modification would have taken place, so therefore could not have easily been observed. Further research at Loughborough is investigating the issue of ion migration and clay modification.

6.2 LIME-CLAY PROPERTY CHANGES

The reaction of interest to this project is the reaction between lime migrating through the soil and the clay minerals present within that soil. Several attempts were made to measure this in a literal sense: to produce a movement of calcium ions through a soil of known strength and to re-assess that strength after varying curing periods. Results from these experiments will form the first part of this discussion. As these experiments eventually proved unsuccessful (at least for the purpose for which they were designed) another, more indirect, method of strength assessment had to be found.

An assessment was made of the strength gains wrought by lime on several clays when intimately mixed. Accompanying mineralogical changes were also studied. In a parallel study mineralogical changes were assessed for samples which had been subjected to a migration of calcium ions (supplied by lime). Plasticity changes were also measured where possible. The results from these experiments have been documented in Chapter 5. This discussion will compare the mineralogical and plasticity changes observed in the migrated samples with those of the mixed samples and by inference conclude the likely strength changes being found in the migrated samples by assuming strength change is proportional to mineralogical change. This will form the second half of the discussion.

6.2.1 Direct Methods

The first direct method attempted required initial measurement of the shear properties of a clay using the direct shear device. Small lime piles were placed at the centre of the sample passing through the shear plane formed during strength measurement. Hence a high chemical gradient, and thus the capacity for calcium movement, was created. It was predicted that calcium ions would migrate outwards from the pile and react with the clay, so cementing the shear surface together. The results showed no particular trend with increasing curing time and the size of the zone of migration meant that it was difficult to differentiate between the additional strength achieved by the lime pile alone and that from the reacted clay. If the shear strength of the pile material is taken to be 0.4 MPa and the area of the pile $8.0*10^{-5}$ m^2 then the additional strength provided by the pile may be assessed by a simple proportioning of strengths to areas:

$$\Delta\tau = (\tau_p - \tau_c) * a_p/a_c$$

where $\Delta\tau$ = increase in shear strength

τ_p = shear strength of pile

τ_c = shear strength of clay

a_p = pile cross-sectional area

a_c = clay cross-sectional area

Table 6.1 Shear box samples - calculated strength of the migrated zone

Overconsolidated clay during period (days)	Residual strength before (kPa)	Residual strength after (kPa)	Strength due to pile (kPa)	Migrated zone area (m^2)	Migrated zone strength (kPa)
7	31.5	29.3	-	-	-
14	25.0	32.8	3.00	0.00040	128
28	28.0	44.5	2.98	0.00080	180
56	-	35.5			
Normally consolidated clay					
1	32.5	37.1	2.94	0.00009	200
7	34.0	40.5	2.93	0.00030	174
14	37.5	35.5	-	-	-
28	36.0	40.0	2.91	0.00066	18
56	32.5	45.1	2.94	-	149

Table 6.1 shows these calculated strengths due to the pile. It is also possible to calculate a strength for the reacted clay material by attributing the remaining strength increase to the clay in the migration zone. This is also shown in Table 6.1. Although again there seems to be no trend developed with time, especially with the limited data, it does seem that there has been some increase in strength due to the lime-clay reaction with an average strength of 170 kPa being recorded (ignoring the suspiciously low value of 18 kPa). Despite observing no trend of increased strength with time within the time period of the test it might be expected that the strength due to the migrated zone would increase further with

time. Part of the problem in measuring the strength of the material was its relatively small area in comparison with the rest of the sample. An attempt was made to develop this experiment so that larger samples could be formed.

The previous experiment relied on the chemical gradient produced when lime was placed as a pile to cause the movement of calcium ions through the clay. As the calcium moves through the clay it will react, and consequently reduce that chemical gradient, until a state of equilibrium is produced and a limiting condition is reached. This explains the short distances of migration observed. There will also be a thermal gradient as the lime slakes, although its effect in piles of this scale was thought to be negligible. In practice, when larger piles are used this thermal gradient will be greater and aid the outward movement of water and hence ions. A hydraulic gradient will exist in the short term as water is drawn into the pile, but this will oppose outward movement of calcium ions. It was thought that inducing a hydraulic gradient across a sample would enhance movement of calcium ions, at least in one direction.

The shear box curing device, described earlier, was developed as it was thought that shear box samples, with their increased permeability along a shear plane, would produce the best results. Several difficulties were encountered with the device itself, not least the problem of creating a square seal around the top of the sample. Considerable losses of water occurred, especially at high hydraulic gradients. Samples left in the device in excess of 90 days showed no sign of increase migration rates. No throughput of water was noted. It was concluded that the combined effect of loss in head and lack of enhanced permeability due to discrete shear plane formation, meant that this experiment was unlikely to yield beneficial results and further techniques were sought.

As discussed previously, the aim the U100 experiments was to measure the strength change that took place when lime was allowed to migrate through clay. The sorts of distances possible, however, were not appreciated at the outset. In reality, very little of the 38 mm diameter triaxial test samples had lime present within them and all of the strength change could be attributed to water content reduction. For clay in the remoulded state, the method of Skempton and Northey (1953) for calculating strength change due to water content reduction proved accurate and reliable.

Observations made of the English china clay sample at a water content of 55% did indicate the presence of reacted material around the pile. When the indicator was applied it formed a 'marbled' pattern with circles of material remaining white in colour and veins of intense pink in between. The circles remaining unchanged could represent 'flocs' of reacted material in which the free calcium and hydroxyl ions that had migrated through these areas had become bound in by the stabilisation reaction. However, this change was not accompanied by a significant strength change, although sampling position and size meant that only a very small proportion of the sample contained material from the migrated zone and therefore its enhanced strength may not have been detectable.

The results from this series of experiments remain inconclusive with insufficient material being available for testing. The U100 tests have indicated the sorts of strength change possible from water content reduction in a remoulded clay. The shear box tests may indicate a change in strength in a zone of reacted material although the results are inconsistent and many more tests would have been required to reduce the scatter and provide confidence in the general trend of increase strength. As stated above a more indirect method of measurement was found.

6.2.2 Indirect Method

The strength development of English china clay, lower Lias clay and Wyoming bentonite has been assessed by mixing these clays (at a water content determined by considerations of plasticity) with quicklime at the 'Eades and Grim' (lime fixation point) value for that particular clay. Strength has been determined using the quick undrained triaxial compression test after varying curing periods. The strengths, expressed in terms of $\emptyset_u$ and c_u are shown in Table 5.3. These are now illustrated in graphical form in Figure 6.1. Clearly these graphs must be read together as the two parameters are used to describe a single line. Thus a high value of c_u is commonly associated with a low $\emptyset_u$, and vice versa, and this should be taken into account when the trends in data are being considered.

It has been chosen to illustrate the mineralogical changes by Infra-red Spectroscopy (IRS) as this is the only method which clearly shows the possible development of an amorphous phase. The samples retained after triaxial testing have been tested using IRS, the results having been illustrated in Figures 5.75 to 5.78. It should be noted that the curing periods used to define the samples refer to the times at which they were tested in triaxial tests. Mineralogical tests took place when all samples were at least one year old. It is interesting that a development in mineralogical properties can still be seen. This indicates that the reaction was halted on drying of the samples, a result that could be expected from consideration of the reaction mechanisms.

By comparison of the strength changes with mineralogical changes it may broadly be concluded that strength development coincides with the development of an amorphous phase. The differences in calcite formation are of interest, but have not been used as an indication of strength for the purposes of this study. Further analysis would be required in order to ascertain the contribution of such formation to the strength of the material. This form of analysis is entirely qualitative. Quantitative analysis was outside the scope of this project but is possible using IRS techniques and would form a valuable part of any further study.

mineralogical and strength changes

Clay Type	Sample Type	% Lime	ω %	Curing Period (days)	Strength $\varnothing°_u, c_u$ KN/m^2	Mineralogical Comment
English china clay	Triaxial	0	46	-	too wet to test	Adsorbed water increases with curing period
		4	46	7	6°, 23	
			46	14	6°, 26	
			46	28	6°, 40	
				90*	11°, 98	Not tested
	Perspex tube	-	48	> year	-	Adsorbed water more pronounced than 28 day sample
lower Lias clay	Triaxial	0	28	-	9°, 54	Little observable development between 7 and 28 days
		4	26	7	/	
			24	14	17°, 324	
			26	28	27°, 304	
			24	90	30°, 415	Not tested
	Perspex tube	-	42	> year	/	More pronounced adsorbed water than 28 day sample
Wyoming bentonite	Triaxial	0	76	-	3°, 29	Little observable development between 7 and 28 days
		4	73	7	7°, 104	
			72	14	9°, 128	
			69	28	20°, 53	
			69	90	7°, 168	Not tested
	Perpex Tube	-	45	< year	-	Little change

* note: these are the curing periods to triaxial test, period to mineralogical test all approximately one year

The samples tested from the perspex tube tests were also at least one year old when tested for mineral content. Table 6.2 attempts to compare the results from the perspex tube samples with those from the mix tests. It may be seen that, in the case of English china clay and lower Lias clay, the amount of amorphous phase present in the perspex tube samples corresponds to the level between 28 and 90 days curing in the mixed samples. In the case of the Wyoming bentonite very little development of an amorphous phase was present in the perspex tube sample. This may be because the Wyoming bentonite sample had a much lower water content than the mixed samples when compared with the English china clay and lower Lias clay samples.

An alternative explanation is that the result may point to the fact that the reaction takes a different form in different clay minerals. The English china clay has been shown to be an almost pure sample of kaolinite, whilst the lower Lias clay is a mixture of illite and disordered kaolinite. The lime-clay reaction is thought to occur at the edges of crystals in kaolinite, exactly the point at which the lime is supplied by migration. The largely montmorillonitic Wyoming bentonite reacts by a different mechanism, and thus at a point where migrating lime may not be supplied to the key regions of the crystal lattice. This is however contrary to the findings of Stocker (1968) and the thoughts of Handy and Williams (1967). Although this would make an interesting area for further study it lies without the scope of this project.

Further tests were carried out on samples of English china clay with different lime contents after 90 days curing (prior to triaxial testing). It is noteworthy that the perspex tube sample developed an amorphous phase to a degree that lies somewhere between that of the mixed samples using 3% and 5% lime. The 'Eades and Grim' value was found to be around 4% lime and represents the amount of lime required to satisfy the ion exchange reaction in the clay. This may indicate that samples with lime migrating through them react as if only sufficient lime to satisfy ion exchange is supplied. This seems reasonable as once ion exchange has occurred, any excess lime is then free to move elsewhere in the clay water system.

The perspex tube (migrated) samples seem to have reacted in a similar manner to that of the mixed samples, with the possible exception of the Wyoming bentonite. If the level of amorphous phase material is directly correlated with strength, the perspex tube samples have attained a strength somewhere between that of the 28 and 90 day mix test samples at a lime content at the 'Eades and Grim', or lime fixation, value. It may therefore be reasonable to assume a strength based on these results for the purposes of design. A 28 day strength of mixed material at a lime content of the 'Eades and Grim' value would seem appropriate. This is lower than that typically found by the shear box tests, but possibly more appropriate since the shear box results were inconsistent and the method used in the calculation somewhat contrived. The use of an equivalent mixed sample strength has the advantage of design data being readily obtainable via traditional testing methods and the method has some scientific basis. Further study into the development of strength versus mineralogical change is, however, felt advisable for a full understanding to be achieved.

6.2.3 Conclusions

A direct method for the measurement of the strength of migrated samples has not been found. A method has been found for readily obtaining an appropriate strength to assign to the migration zone for the purposes of design. As the migration zone is relatively small in comparison with the pile, it is felt that sufficient reliance may be placed on such a strength without concern for the factor of safety. However, only a very small amount of data has been collected concerning the mineralogical changes that occur as strength increases due to the lime clay reaction. This would constitute a study in its own right and make a valuable contribution to the understanding of this complex reaction.

6.3 PORE WATER PRESSURE CHANGES

Pore water pressure change in the laboratory has been solely examined in the box studies by means of mercury manometers. This means of measurement was chosen for its simplicity and relatively low cost, but was not without problems.

In order to achieve a representative measurement, a continuous path of water must exist from the point of measurement to the mercury in the u-tube. This means that the sand cell in the clay needs to be fully saturated at all times and at a constant pressure, even when the pore water pressure in the clay is fluctuating. As discussed previously, this appears not to be the case, so leaving interpretation of the results problematic.

A detailed study of the effect of removing and replacing the effective normal stress to the surface of the clay in the boxes was carried out in Arrangement 3. This showed that high positive pore water pressures occurred immediately on replacement of the stress. Reasons for this have previously been discussed. It has been assumed that a similar response would have occurred after pile placement were it not for the reduction in pore water pressure produced by the slaking lime. The actual value placed on this reduction has been calculated by subtracting that after the pile was placed to that prior to pile placement. An example of the relative magnitude of response is illustrated in Figure 6.2. All quantitative data for Arrangement 3 have been calculated in this way.

Water continuity also requires the connecting tubing to be entirely free of air. With fluctuating pressure, from large positive to large negative values, if there is any residual air in the system it will inevitably find its way into the tubing, thus reducing the accuracy of the results. De-airing may be achieved sporadically by disconnecting the tubing from the manometer and forcing de-aired water downwards, eventually into the sand cell. Without a double tubing system, where water can be forced around a complete loop, this can only ever be partially effective. It also has the problem of temporarily increasing the local pressure in the sand cell.

A further problem experienced was the accuracy with which the sand cells can be spatially placed. Great care was taken to ensure precise placement, but on excavation after completion, some manometers were found to be up to 50 mm from the intended position. This must occur due to movements produced by the action of compacting clay over the sand cells with subsequent non-homogeneous application of pressure.

The interpretation of a steady state of pore water pressure was also difficult. Fluctuations occur due to the reasons outlined above and due to fluctuations in the mains pressure supplying the water bag, leading to discontinuous application of normal stress. In Arrangement 1 of the large boxes it was found that the manometers at the edges of the box did not respond in such a pronounced manner as those nearer the centre. A probable explanation was that their positioning meant that the clay in this area did not experience the full portion of the supplied normal stress. When the lid was removed after consolidation, a band of clay 50 mm wide was left uncompressed around the edge of the box due to the size of the air bag below the lid. Although stress would be expected to equilibrate with depth, a residual effect still appears to have occurred. The effect could have been increased due to friction, or adhesion, at the box wall. Manometers placed 25 and 50 mm from the edge of the box fell beneath this zone. This problem was also experienced in some instances in Arrangement 2 but was overcome in Arrangement 3, where manometers were placed at least 50 mm from the edge of the box and no significant differences between the edge and more central manometers were experienced. The manometers placed near the edge of the first small box arrangement also gave well defined responses where a static load had been supplied by means of dead weights, indicating that the edge effect did not occur in this test.

If further investigations of this kind are to be carried out in future other means of investigating pore water pressure should be investigated more fully. However, despite these criticisms, many of the features affecting the change in pore water pressure created by lime piles have been investigated successfully and much useful data has been collected. These shall be discussed in the following sections.

The problem of data collection has also been discussed previously. This has meant that, whilst the results from Arrangements 1, 2 and 3 have been compared (with salient points presented in Table 6.3), Arrangement 2 has been used in a qualitative sense only. The results shall be discussed in terms of the factors affecting change in pore water pressure.

6.3.1 Pile Size

The results from Arrangement 3, presented in Table 6.3, clearly show that the larger piles produce a larger reduction in pore water pressure. The results from Arrangement 2 also support this argument. It is recognised that results from Arrangement 1 disagree, but lack any information about the response of the clay alone to an application of normal stress and thus are not directly comparable. This is considered to have resulted in the reductions recorded being generally less pronounced than those in Arrangement 3. If the responses of the boxes differed in terms of application in normal stress this could account for the discrepancy between these boxes as far as pile size is concerned.

Table 6.3 Pore water pressure (pwp) reduction around lime piles

pile Ø (C signifies compacted pile)	distance to manometer (mm)	σ_n (kPa)	Decrease in pwp (to nearest .5kPa)	Comments
150	50	100	0.5	top of box
150	50	100	2.5	base of box, reduces to -6 after 315 hours
150C	50	100	2.5	top of box
150C	50	100	7.5	base of box
100C	50	100	10.5	top
100C	50	100	13.5	base
100	50	100	7.0	top, initial increase
100	50	100	90	base, increase first, decrease to -12.5 after 48 hours
150	25	50	21.5	top
150	75	50	20.0	top
150	125	50	19.5	top
50	25	50	13.0	top
50	75	50	11.5	top
50	125	50	11.0	top
50	25	50	17.0	top
50	75	50	13.0	top
50	125	50	13.0	top

The response of the 50 mm diameter pile in Arrangement 2 showed a 'double trough' effect similar to the temperature effect in the 50 mm pile in Arrangement 3. Conclusions are difficult to draw, but these results may illustrate a two stage reaction in the pile material in which the outer pile material reacts, producing a reduction in pore water pressure, and expands, thus increasing the pressure. The expansion breaks the outer layer of reacted material, allowing further reaction and further reduction in pore water pressure. This type of reaction sequence has been suggested by Chiu and Chin (1963). If this is the case it is unclear why all the piles have not shown such a response, particularly those with a larger diameter. However, the variability in the manometer readings are such that any slight changes may be overlooked.

6.3.2 Pile Compaction

Results from Arrangement 1 suggest that pile compaction increases pore water pressure response, but without the knowledge of the response of the clay alone to application of normal stress this cannot be definitive, despite its logic.

6.3.3 Pile Groups

It cannot be concluded that, at the same distance from a pile, a group of piles produces a greater reduction in pore water pressure than a single pile. Again, quantitative analysis is difficult but results appear to suggest that the response is not greater at the centre of the pile group studied in Arrangement 2 than the summation of the effects from the individual piles. Penetration of the effect has been difficult to assess. It may be that this effect is peculiar to the centre of equally spaced piles. The manometers not placed equidistant from piles have not produced any data due to mercury ingress into the box. Results from Arrangements 1 and 3 appear to show a slight reduction in pore water pressure spatially across the box. This difference is not of sufficient magnitude to either be outside the inaccuracies of the manometers or to extrapolate to greater distances.

6.3.4 Clay Type

Few data are available about the influence of clay type as English china clay was used in all arrangements. However, the small box of the initial experiment was filled with lower Lias clay with a 50 mm pile. This may be compared with the English china clay box using a similar pile used in the PFA study. Both clays were placed at the same Liquidity Index. The results appeared to show a similar magnitude of both positive response due to pile placement and subsequent reduction in pore water pressures. The English china clay appeared to show some sort of multiple trough response, with an overall long-term increase in pore water pressure, whilst the lower Lias clay remained approximately at the same level of pore water pressure over the duration of the test. It would be unwise to draw many conclusions from this, other than making the seemingly obvious statement that different clays will respond differently, if for no other reason than the coefficient of permeability will be different, and hence response time will be different.

6.3.5 Effect of PFA

Figures 5.61 and 5.62 clearly show how the addition of PFA inhibits the production of the negative pore water pressure response. The positive pressures produced by the pile placement appear however, not to be affected. Careful consideration must be given in deciding whether the enhanced strength produced by the PFA can outweigh the effects produced by the reduction in pore water pressure.

6.3.6 Field Considerations

From the conclusions drawn in the preceding sections, several observations may be made about the effect of field conditions. Piles of 200 mm are likely to be installed, increasing the effect on pore water pressure as increasing size increases the magnitude of pore water pressure reduction. Spacings of up to 2 m are the least which may be practical. It is unknown what the effects are at such distances. The readings taken in the laboratory suggest that there will be a reduction in pore water pressure at such distances. However, should drying occur only locally around the pile (rather than the gradually diminishing effect with increasing distance from the pile) then pore water pressure effects will not be transmitted further than that local region around each pile.

The major effect will be produced by the clay into which the piles are placed. It will almost certainly be much drier than that in the experiments. This should increase the magnitude of the reduction in pore water pressure as the *in situ* material will be of a lower water content than those used in the laboratory. Even if the material into which the piles are placed is fill, it will not be as homogeneous as in the laboratory, containing voids or fissures. This may have the effect of creating a discontinuous response over the length of the pile. Having said this, it is primarily the effect on the shear zone which is of most interest. This area will be reworked to some extent and of higher water content than the main body of material. It is, therefore, not unreasonable to transfer the results gained in the laboratory to the design of field installations.

6.3.7 Conclusions

Increasing pile size and compaction increases the magnitude of reduction in pore water pressure. The relationship is not linear and therefore an optimum value may be reached in both cases. The reductions produced in the laboratory from a single pile in relatively wet clay are of the order of 20 kPa. There is some spatial variation in the values achieved, but insufficient data are available to quantify this phenomenon. Clay type affects the response, particularly the duration of the response. Increasing the number of piles in a particular area should increase the response for a set distance from the piles, provided that the distance is sufficiently close to the piles to experience an effect. Field conditions will alter the response of the clay and, as with all geotechnical projects, site specific judgements must be applied.

6.4 SHEAR PLANE CONSOLIDATION

As discussed previously, the stabilising effect of the reduction in pore water pressure is not permanent, lasting for different periods of time depending on the groundwater and surface water conditions. However, if the reduction in pore water pressure is considered as an increase in effective stress, the effects of this are of a permanent nature. It has been postulated that an increased effective stress will consolidate the shear zone present within a failing slope, thus realigning the particles within that zone and improving its strength to above that of residual, halting the slide.

The direct shear box has been used in order to model this phenomenon in the laboratory. It was recognised that shear planes formed in normally consolidated samples at high normal effective stresses would not model the field situation of a shallow slip formed in a heavily overconsolidated material. It was felt that the field situation would produce a shear *zone* of partially aligned particles rather than a discrete shear *plane* with well aligned particles. The samples used for the study were, therefore firstly overconsolidated (to a range of values of overconsolidation ratio) before shearing at a relatively low value of effective normal stress. This was thought to model the field situation more satisfactorily.

The results presented in Chapter 5 show clearly two important findings. Firstly, given a value of effective normal stress used to initially consolidate the sample, increasing the degree of overconsolidation increases the improvement in the value of residual strength produced. Initially, this may seem contrary to the idea that the more disrupted the shear plane, the greater the 'knitting' of particles produced by overconsolidation, and the greater the rise in residual strength. Overconsolidation produces a greater alignment of particles and hence a better defined shear plane. However, for a given effective stress, to achieve a greater degree of overconsolidation the sample will necessarily have been sheared at a lower effective stress. Very low values of effective stress cause an ill-defined shear plane to form and hence overconsolidation has a greater effect, as expected. This result suggests that the shallower the situation of the shear zone, the greater the effect of overconsolidation on its residual strength. In practice, for slips of depths up to 5 m an increase in the effective cohesion of 1 kPa may be applied.

The second observation, and one which is illustrated in Figure 5.27 is contrary to the above argument. The graph shows that, given a value of overconsolidation ratio, increasing the effective normal stress at shear increases the effect. It also shows that, given the normal stress at shear (i.e. the depth of the shear zone) increasing overconsolidation, increases the effect. Results for the lower Lias clay are presented in Figure 6.3 and show similar trends, if not quite so clearly. In addition to this, if the results from the tests using an overconsolidation ratio of 20 (Figures 6.4, 5.16 and 6.5), are compared with those produced using a ratio of 10 (Figures 5.18, 5.20 and 5.21) then it may be seen that not only is the percentage change in residual strength greater for the higher ratio, but the effect is not so diminished as shearing is continued. For the ratio of 10 in all but the test using a normal stress of 30 kPa, the increase in stress is rapidly reduced to the pre-consolidation value. In all of the tests with a ratio of 20, a significant increase in shear stress is sustained for the remainder of the distance of the test.

Normally consolidated samples of both china and lower Lias clays appear to produce the same percentage increase for all values of normal stress. However, as discussed previously, particularly in the case of the lower Lias clay, the increase observed is a gradual one and may be attributable to end of travel effects. These will almost certainly increase with higher normal stresses. These results, therefore, have been omitted from the remainder of the discussion.

When comparing the results from the two different clays, it may be seen from Figure 6.6 that the lower Lias produced a greater percentage rise in residual strength for a given effective normal stress and overconsolidation ratio than the English china clay. This may be attributable to differences in particle size and shape and the nature of the shear zone being formed in each case. The lower Lias does not show the rapid immediate dropping off of a portion of the gain that was typical of the results from the English china clay. This again may be attributable to particle size, shape and orientation. The English china clay has been shown to be very pure and contain only one type of particle. Those plate-like particles become aligned upon shearing and, once aligned, any degree of overconsolidation will not promote additional interlocking. The lower Lias clay contains different sizes and shaped minerals which would be more difficult to align and hence the degree of interlocking produced by overconsolidation would be greater. Unfortunately, due to problems with equipment combined with very small changes, much of the data concerning vertical movements has been unreliable. However, those which have been retrieved show that larger effects are coincident with increased dilation on shearing. Larger amounts of dilation are experienced with the lower Lias clay samples than with the English china clay samples. An increase in 'cohesion' (as used in slope stability analysis) of as much as 5 kPa may be possible from lower Lias clay treatment, although additional work in verifying this result should firstly be carried out.

Approximately the same 2% increase occured for all normally consolidated samples was again observed. This value is small and would be ignored in practice. Due to its size and consistency, it is possibly a further indication that the effect may be due to the apparatus in this case.

6.4.2 Conclusions

The existence of an effect is not disputable, although explaining its physical basis is difficult, given the seemingly contradictory evidence. Further, rigorous testing is required in order to firmly establish these results. Several tests have been repeated on both shear boxes with a reasonable degree of repeatability, Figures 6.7 and 6.8. Most importantly, the pattern of the resulting traces were similar. This reinforces the idea that the effect is due to the clay rather than the operations of the shear box. The level of repeatability was reduced at very low effective stresses. This is thought to be attributable to the increased variability of the shear plane produced at these stresses. Further repeat tests, however, would be advisable to confirm the exact values of increased residual strength.

A scanning electron microscope study of the particles consisting the shear zone would aid this study as the effect does appear to be dependent on the type of shear plane formed, and therefore on clay mineralogy.

It is also thought that the size and nature of the shear plane does not make the best possible representation of the field conditions. The larger, 300 mm shear box would provide a larger area over which shearing could take place and allow edge effects to be less influential. The discrete shear plane formed within the direct shear box, even at low normal stresses, is thought to be less suseptible to the effects of overconsolidation. In the field, where a shear 'zone' of reworked material, several millimetres thich may exist, the effect may be magnified.

For the purposes of design, an increase in the value of effective cohesion of 1 kPa has been used in order to design the field trials. In future, this may be used as a conservative estimate until further testing has been carried out, although it may be assumed that a greater effect will be realised especially in highly overconsolidated clays.

6.5 WATER CONTENT

The study of the changes in water content has been conducted in two parts: the U100 study and the large box arrangements. The former principally investigated the effect *of* the change in water content on strength, while the latter investigated the effects of initial conditions *on* the change in water content. Thus, a prediction could be made of the change in water content produced by any pile arrangement in any clay and the likely effects.

In both these cases it must be stressed that the piles have been placed within a closed system with no facility for water ingress, so long-term results are not possible to state. Again, data from field trials will be sought at a future date.

6.5.1 Improved Strength

The U100 study showed that by reducing water content, undrained shear strength was improved. The equations of Skempton and Northey (1953) were used to predict those strength changes and were found to correlate very well with those found in practice. It should be stressed that the equations were developed for clays at the critical state and that the clay compacted into the tubes would represent this state very well. The use of the equations for the calculation of strength changes experienced in undisturbed material would not be so appropriate. In clays with water-filled fissures, the water for the reaction may be supplied through this route and not affect the clay mass as a whole. In the longer term it is not clear how long this effect would be sustained, given a supply of water. It is anticipated that the reworked region of the shear zone will be most affected, which in terms of slope stability is the critical zone and therefore of most benefit. This material is also most likely to be reworked, and as such at its critical state.

6.5.2 Pile Geometry

Water content measurements from the preliminary Arrangement 1 tests were only taken at two distances from the pile and were therefore inconclusive as far as pile size was concerned. Also, the relatively small difference between the two pile diameters chosen may have led to any differences in measurements being imperceptible. Therefore, the effect of pile geometry was difficult to assess reliably.

The greater number of samples taken from the boxes of Arrangements 2 and 3 allowed the changes in water content to be plotted as contours around the piles. The results clearly show that the larger pile produced the larger reduction in water content around the pile. The relationship, while related to pile size is, however, not proportional to the volume of the pile material and therefore may indicate that not all of the quicklime in the piles has reacted and/or that the piles have not become fully saturated.

It should be noted that the measurements taken are, however, to a degree unreliable. This is because the original water contents were taken at the corners of the boxes where the piles were placed to prevent over-sampling of the clay affecting results. It was not possible to sample at the individual points in the clay surrounding the piles both before and after pile placement. It was shown in the results from Arrangement 3 that the water contents at the edges of the boxes are significantly higher that those at the centre, and this effect might have occurred in Arrangement 2. This may be attributable to the reduced stress and lack of drainage facility at the edge of the boxes.

These high initial values of water content in Arrangement 2 may lead to a larger calculated change than was actually the case. Having made this point, the changes are of a similar order to those found at similar distances in Arrangement 1. The greater problem arises with the interpretation of the results from Arrangement 3. The original water contents were assessed at the centre of the boxes and were therefore comparatively low. The water contents taken after pile placement appear to suggest a radial reduction around the pile but many of the water contents measured towards the edges of the boxes were higher than the original value at the centre. It is likely that a radial reduction is produced, given the results from Arrangement 2, but absolute values are not possible to be deduced without knowing the initial values at all points across the box. If this test were to be repeated, one box would be left without a pile placed to act as a control for water content sampling.

6.5.3 Pile Compaction

Data are only available from Arrangement 1 and are therefore limited. They appear to suggest that compaction reduces the amount of water content reduction, at least adjacent to the pile. This may because compaction reduces the amount of water the pile can absorb.

6.5.4 Pile Groups

It can be seen from comparison of the results from Arrangement 2, between the group of 100 mm diameter piles and the single 100 mm pile, that increasing the number of piles within a given area will increase the amount of water content reduction. Obviously, this is provided that the point of interest is within the zone of influence of more than one pile, i.e. that the spacing is sufficiently close.

6.5.5 Field Considerations

As mentioned previously, the structure of the clay will have an influence on the amount and effect of water content reduction and on the duration of the effect. In addition there is some evidence to suggest that the original water content of the clay will affect the magnitude of the reduction. The results from the U100 tests suggest that the higher the initial water content, the greater the magnitude of the reduction. This is also inherently true in the way water content reduction has been calculated. If a large proportion of the reduction is attributed to water absorbed by the pile (rather than reacted) then the larger the hydraulic gradient between the pile and the clay the larger the amount of water that will be absorbed by the pile, and thus the larger the decrease in water content. The relatively dry conditions probably to be encountered in the field will mean that water content reduction is necessarily small. The readings taken in the laboratory suggest that the reduction in water content gradually diminishes with increasing radial distance from the pile. However, it is possible than in drier conditions drying will occur only locally around the pile, thus completely desaturating a very small annular zone rather than the effect being transmitted further into the soil around each pile.

If the shear zone is of a wetter material than the clay surrounding, it thus will mean that this critical zone will be preferentially treated and that the effect occurs at significant radial distances from the pile.

6.5.6 Conclusions

Increasing pile size increases the magnitude of water content reduction produced. This effect is not linear with pile volume and therefore a limiting pile diameter will be reached whereby further volume will not have a significant effect. Pile compaction may reduce the effect adjacent to the pile by reducing the piles' ability to absorb water. The zone of influence appears to be exceeded by the dimensions of the box, although precise dimensions are not attributable due to the difficulties explained above with measured initial water content values. If piles are placed in groups where their spacing is less than the individual zones of influence, an increased reduction in water content will be produced at any particular distance from the pile. Therefore reduced pile spacing increases the effect of water content reduction. Consideration of field influences is complicated and difficult to assess. The likely (relatively low) water content of natural material is such than any improvement due to drying is potentially small, at least in the short term and may be limited to a very small annular zone. The possibility of subsequent water ingress, perhaps via the tension cracks that form in such structured materials, means that this effect perhaps should be ignored for design purposes. An enhanced effect to the shear zone is considered to be of benefit and, although difficult to quantify unless good quality measurements are made from the shear zone, should be allowed for in design.

6.6 TEMPERATURE CHANGE

Temperature change itself is not seen as a mode of stabilisation, although it has been studied in a rigorous fashion in the boxes and by observation in the perspex tubes for several reasons.

Many authors have observed elevated temperatures on pile placement, and Tsytovich et al (1971) and Kitsugi and Azakami (1983) have suggested it as a mode of water content reduction. Other authors have noted that elevated temperatures increase the rate and effectiveness of the lime-clay reaction, for example Raj et al (1989). Raised temperatures will change the chemical environment in which the lime is diffusing and have an influence on the properties of the clay mass itself, as reported by Jefferson (1994). The following influences on temperature change have been studied.

6.6.1 Pile Geometry and Spacing

The maximum recorded temperature rises for each pile arrangement at a particular distance have been combined in Figure 6.9. This clearly shows the inverse square pattern of temperature distribution with respect to distance from the pile. It also shows that the larger the pile diameter, the larger the rise in temperature at a given point. For example, at a distance of 50mm from the pile the rise in temperature approximately follows a proportional relationship with pile diameter. There are insufficient data to verify such a relationship, although for practical purposes, a 200 mm diameter pile would produce a curve above that of the 150 mm pile as it is clear that the larger the diameter the larger the rise in temperature. This also means that the larger the pile diameter, the greater the distance over which heating occurs.

Generally, the time taken to reach maximum temperature increases with distance from the pile and the larger the pile, the shorter the time taken to maximum temperature for any one distance. The duration of the temperature rise appears proportional to pile size and distance from the pile. Compaction of the pile seems marginally to reduce the duration of the temperature rise. This may be because, as slaking occurs, expansion causes densification of the pile material to such an extent that further reaction within the pile is prevented or possibly greatly slowed down. This phenomenon would be more pronounced the more compact the pile.

These statements have been made on the basis of data obtained from any single box arrangement where clay conditions are similar. Comparison of results from the different arrangements, especially in the case of Arrangement 2, has been treated with caution as differences in water content may make a substantial difference to the results. However, Figure 6.9 does appear to suggest that reasonably good agreement exists between the data.

With groups of piles the heating effect at any distance between the piles is greater than for a single pile, but there are insufficient data to conclude that temperatures are simply additive. Undoubtedly, however, the time taken to reach the maximum temperature is markedly reduced.

From the limited data available, pile compaction also appears to increase the maximum temperature, although with little effect on the rate of temperature rise.

If it is considered that temperature rise is an indication of the rate of the slaking reaction, it may be concluded that the larger and more compact the pile, the greater the extent of the reaction. This seems logical as the larger the circumference of the pile the greater the amount of lime in contact with the clay. Also, compaction increases the amount of lime in any given area, hence increasing the amount of lime slaking at any one time.

6.6.2 Clay Mineralogy and Water Content

Due to problems of material availability, all boxes were created from English china clay. An attempt was made to study the effect of water content, but due to the material being consolidated under similar normal stresses, all clay was at a similar water content at the time the piles were placed. (This should perhaps have been obvious with hind-sight). However, observations made during the creation of the perspex tube samples indicated that the wetter and more porous the clay the greater the rise in temperature felt. Again, if temperature is used as a measure of the rate and extent of reaction, this means that the wetter more porous clays allow the reaction to proceed more rapidly. This must be due to the relative availability of water for reaction.

Additional data on this subject have been gained from a parallel study, in which similar tests on undisturbed samples of London clay were carried out (Rogers et al, 1994). 50 mm diameter quicklime piles placed in heavily overconsolidated samples of London clay having a water content of the order of 33% produced a maximum rise in temperature of 2.5 ^{o}C at a distance of 25 mm from the pile after 12 hours. Water content is obviously a key factor influencing temperature rise. At a similar distance from a 50 mm pile placed in English china clay the maximum temperature rise 25 mm from the pile was 3.2 ^{o}C after 9.5 hours. The water content of the clay was of the order of 36%. Unfortunately these rises in temperature are low enough to be significantly affected by the fluctuation of the thermocouples. However, they are of a similar order for clays of similar water content. The delay in reaction may be considered longer for the London clay and may be attributable to clay mineralogy indicated by Plasticity Indices and structure. It is interesting that in both cases the rise in temperature shows a distinct 'double peak'. This, as discussed earlier, may be attributable to the successive expansion and reaction of the pile or the fluctuation in reading of the thermocouple. This may explain why it is not seen on a greater scale with larger piles.

6.7 PILE MATERIAL

All of the previous discussion has been based on the properties of, and changes occurring in, the clay surrounding the piles. This section will concentrate on the undoubtedly important area of the pile material itself.

6.7.1 Expansion

The evidence for the effects of pile expansion appear contradictory, but an attempt has been made to explain the phenomena occurring. It was felt that, in very soft materials the method of placement may have caused a large degree of the observed lateral compression of the material, rather than being solely attributable to pile expansion. In such materials, the piles are formed by inserting a casing and introducing quicklime through the base by compressed air. The insertion of the causing will cause the material to compress and, in a sufficiently soft material, the pressure of the lime will cause additional compression. Laboratory data on pile expansion have been gathered on an observational basis.

Firstly, the actual amount of expansion occurring must be considered in the large boxes. The pile diameters measured are listed in Table 6.4. It is worth noting that, despite the difference in the curing times, both were in excess of the duration of the changes in temperature occurring, indicating the end of the reaction. These diameters are far short of the diameters required if the material were to reach double its initial volume, as predicted by consideration of the chemical reaction and densities of the different chemicals. As quicklime is a particulate material, some of the expected expansion will cause a filling of the void space within the volume of the pile itself, causing densification of the hydrated lime thus formed. Hence, not all the expansion is observed as an increase in volume. This may account, in part, for why the observed diameter was not as great as that predicted. It also follows that the more compact the pile the greater the effect of expansion on the overall volume of the pile.

Table 6.4 Pile expansion in large box arrangements

Arrangement	Initial pile ∅ (mm)	Water content (%)	Final pile ∅ (mm)	Increased volume (%)
2	50 (single)	40.9	50	zero
	50 (group)	*	50	zero
3	50	35.8	50	zero
2	100 (single)	41.2	105	10
	100 (group)	42.0	110	21
3	150	35.6	170	28

* water content not taken due to mercury in box

Pile diameters measured in the perspex tubes also reinforce this argument (see extrusion logs, Appendix 4). The diameters are also not always consistent over the length of the pile, so expansion is not necessarily a homogeneous process. This would be expected as the clay was not entirely homogeneous and the pile will have expanded in areas of lower density as these offer less resistance. Hence, in the situations provided, the expansion of the pile actually achieved in practice is only a small proportion of what may be expected from chemical considerations. The expansion perceived in the test tubes may also be attributable to lateral compression of the material as the pile is compacted. The greatest apparent expansion was recorded for the wettest English china clay samples, which are most susceptible to compression. In other clays the expansion was noted at the base of the pile, also an area susceptible to lateral compression.

Secondly, for lateral consolidation to occur as a result of expansion, densification of the surrounding clay must occur as a result of the expulsion, or extraction, of water due to the stress provided by the expanding pile. For expansion to occur, water must flow into the pile and react. It has already been shown that the volume of water required to satisfy such a reaction is greater than the additional volume provided by the slaked lime. As the lime in the pile is particulate, and there is a hydraulic gradient between the pile and the surrounding soil, water will also tend to flow into the pile under a hydraulic gradient as well as a chemical one. Thus, there is an overall volume loss in the clay-lime system as a whole and no means to provide expulsion of water. However, the clay has undoubtedly lost water to the piles and therefore densification of the clay around the piles must have occurred. It has been shown that the change in water content around each pile is radial in form in the relatively wet samples tested herein (not in undisturbed London clay samples tested by Rogers et al, 1994, as discussed later) and hence such densification may easily have been mistakenly attributed to lateral consolidation in the past. Such densification, and concomitant strength gain, is most efficient in soils at its critical state (Skempton and Northey (1953)). Soils most representative of this behaviour are normally consolidated, wet clays, typical of the marine clays experienced by those authors expounding lateral consolidation (most notably Kitsugi and Azakami (1982)). It has also been noted from the perspex tube tests that the wetter the clay, the more the degree of expansion. This will increase the amount of water extracted from the clay and hence the densification.

The final conflicting piece of evidence is the production of radial cracks around the piles after slaking has occurred. Superficially these may be attributed to expansion, especially as the larger piles produce larger cracks and larger piles would be predicted to expand to a greater degree. However, if the radial nature of the drying of the clay is again considered these cracks may be more satisfactorily explained. Consider the soil surrounding the pile as a ring of clay drying and the drying effect is far greater at the inner circumference of the ring than progressively into the soil. The drying will cause contraction, which is similarly most pronounced at the inner circumference of the ring. Eventually that contraction will cause the clay to split at the point of contact with the lime and the crack will proceed progressively into the soil. This scenario would cause the pattern of cracking that was observed. It may also be appreciated that the larger pile, and hence the ring of soil, the greater the effect of contraction and the greater the degree of cracking.

In the experiments reported by Rogers et al (1994), it was found the degree of radial cracking was most pronounced in a heavily overconsolidated London clay as opposed to a reworked fill material. The relatively low permeability and water content of the samples was also thought to be significant. The demand of the pile for water was high and the potential for supply low due to these factors. The inherent strength of the clay structure was high, with little of no confinement provided to intact 300 mm cubic block samples. Expansion therefore caused radial cracking which penetrated to the edge of the sample. One additional observation made was that lime from the pile 'fell' into the cracks. If these cracks were filled with water subsequently a transport route for calcium ions has been produced.

Hence, it may be shown that expansion does not occur to the extent predicted, densification of the soil may be attributed to drying and cracking is due to the differential nature of this drying.

6.7.2 Pile Strength

Initial results suggest that it was reasonable to assume a pile strength of 500 kPa, equivalent to that of a weak concrete. This strength was assumed to develop within two weeks of placement. The lower strengths observed from testing piles from the large box arrangements were largely from the smaller piles where problems of sampling were most acute and sample damage prior to testing probably accounts for the low observed strengths. The Japanese data from Kitsugi and Azakami (1982) suggested average unconfined compression strengths of 430 kPa, alluding to a typical undrained shear strength of around 200 kPa. The value of 500 kPa was used to design the field trials, but future installations should be preceded by further testing of the pile material.

The effect of PFA addition has been difficult to assess by the means chosen. Problems were experienced at all stages of testing. Initially, sample preparation attempted to simulate the slow ingress of water experienced by the piles in the ground but, in reality, resulted in samples of inconsistent water content, causing inherent differences in strength. Longitudinal expansion of the piles reduced the strength of the material as the piles were effectively unconfined. The problem of preparing 38 mm samples from larger, 100 mm diameter samples, also meant that an insufficient number of samples were produced to created full strength envelopes. This created a further source of unreliability in the results.

Generally, this set of experiments was unsuccessful in producing data about the effect of relatively small PFA additions to the lime. It did, however, provide useful experience in designing a further set of experiments for future study. The importance of this work on PFA within the context of the project is debatable, merely producing a refinement to the general aims and objectives of the project as a whole and potentially invalidating other stabilisation mechanisms, such as pore water pressure reduction. The field trials have been designed and installed on the basis of lime alone and therefore PFA addition really falls under the remit of further research and will be included in the relevant section in Chapter 9.

6.8 SUMMARY

The migration of calcium from the piles is small in terms of distance but has a profound effect on the clay it reaches. Further testing is required in order to obtain quantitative data for the strength of this material, although an indirect means of assessment has been developed. The effect of water content, clay mineralogy and pile geometry on that strength also needs to be assessed. Evidence of increased migration rates when water content and particle orientation are increased suggests that the method will preferentially treat material in the shear zone.

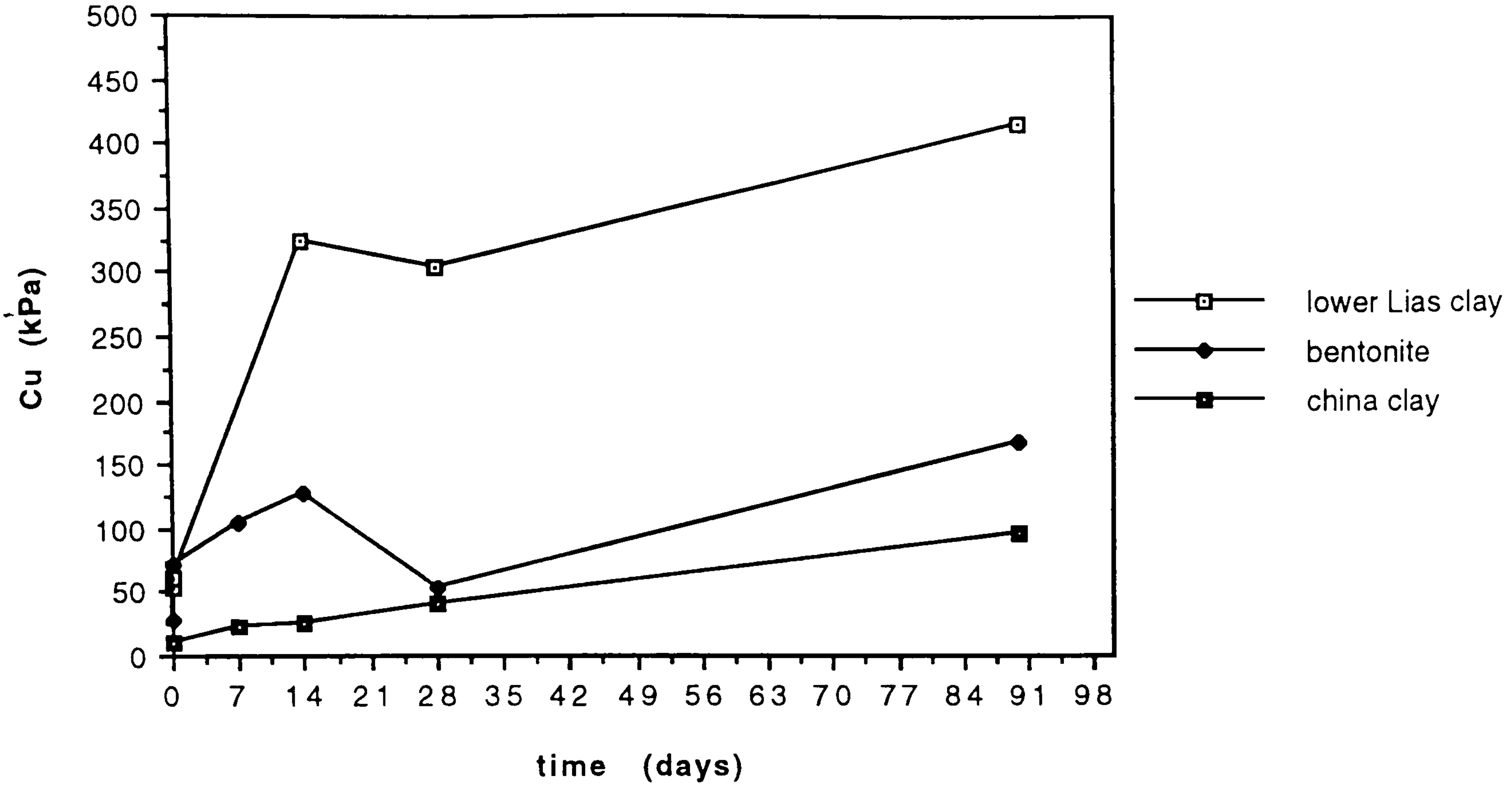
Pore water pressure changes have been difficult to assess in the laboratory, but indicate reductions in pore water pressure in the order of 20 kPa. Indications are that in drier field conditions this effect would be magnified. Pile size, spacing and compaction all affect the magnitude of the reduction. The reduction in pore water pressure causes an overconsolidation of the shear zone, having a beneficial effect on shear strength in that region.

The water content of the clay surrounding the piles is reduced by slaking and absorption effects. Again the higher water content of the shear zone means that this region is treated preferentially. Increased compaction of the pile material reduces this effect.

Pile expansion in stiff, relatively dry materials is very limited. The problem of cracking in these conditions is potentially detrimental (although might facilitate ion migration) and the factors affecting crack development require further investigation.

Evidence from the literature and initial testing of the pile material from the box model tests indicate a cohesion value of between 200 and 400 kPa for the pile material. Further investigation of this parameter is required so that greater reliance may be placed on its value. Enhancements to this strength are also a subject for further investigation.

Each of the above mechanisms contribute to an overall stabilising mechanism. Data concerning the effect of pile dimensions and clay properties have been combined with quantitative data about the magnitude and duration of each effect and used to develop a design method. This development and the resulting design method are discussed in Chapter 7.


Data from undrained triaxial tests using 4% lime - Cu changes

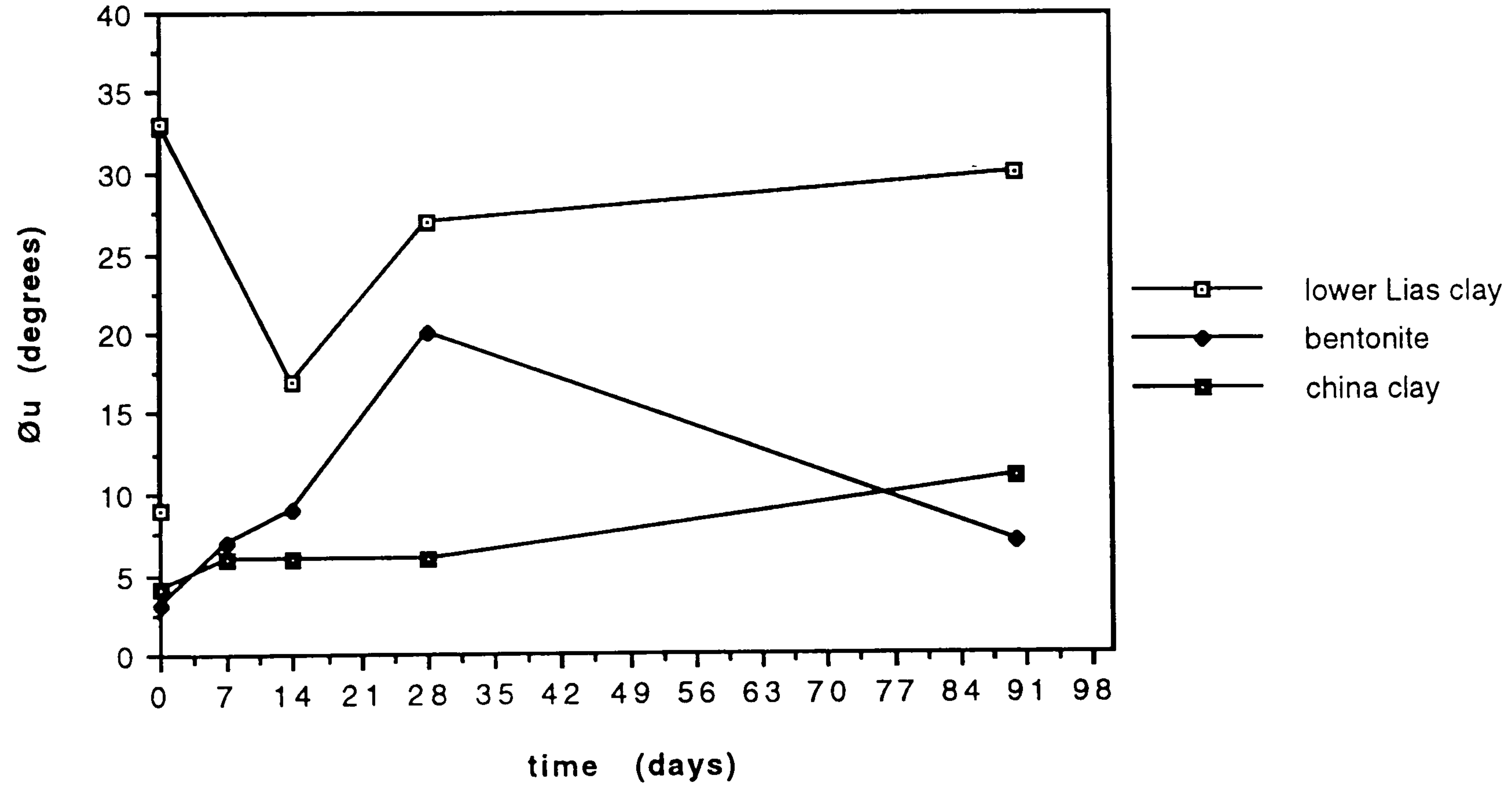

Data from undrained triaxial tests using 4% lime - Øu changes

Figure 6.1 Undrained strength development of china clay mixed with quicklime

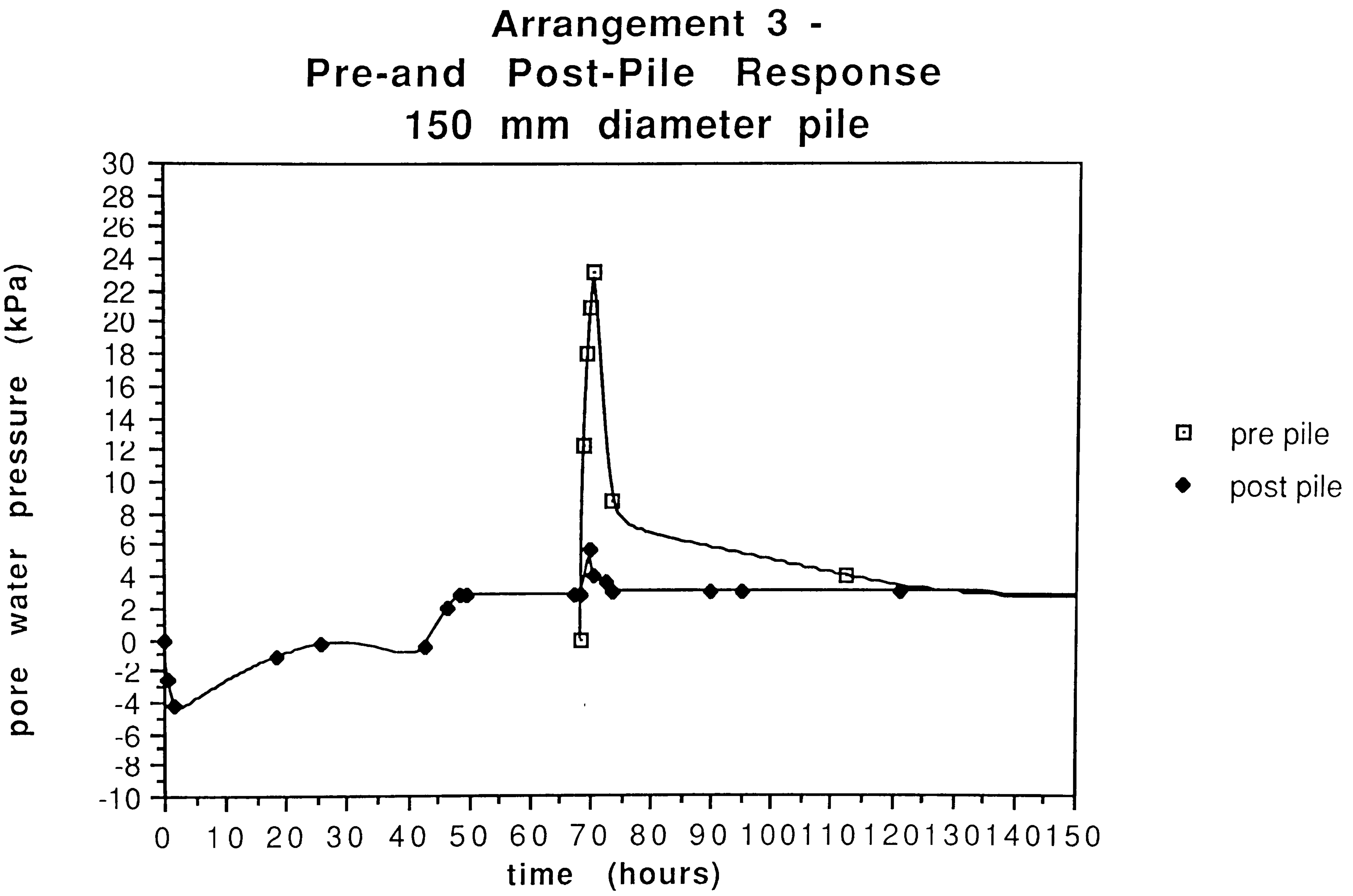


Figure 6.2 **Arrangement 3 -**
comparison of pore water pressure response
pre- and post pile placement

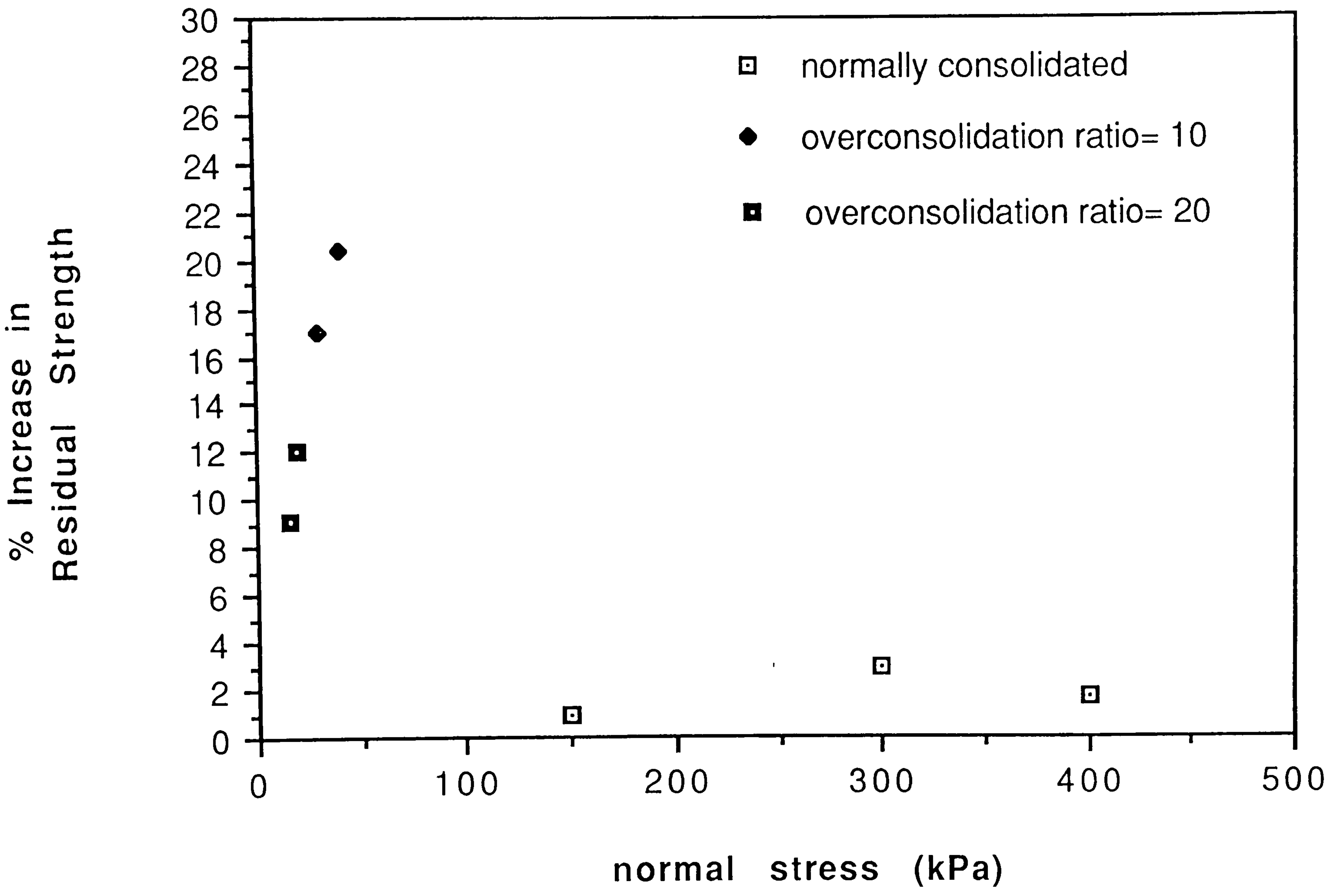


Figure 6.3 Lower Lias clay - percentage increase in residual strength after overconsolidation

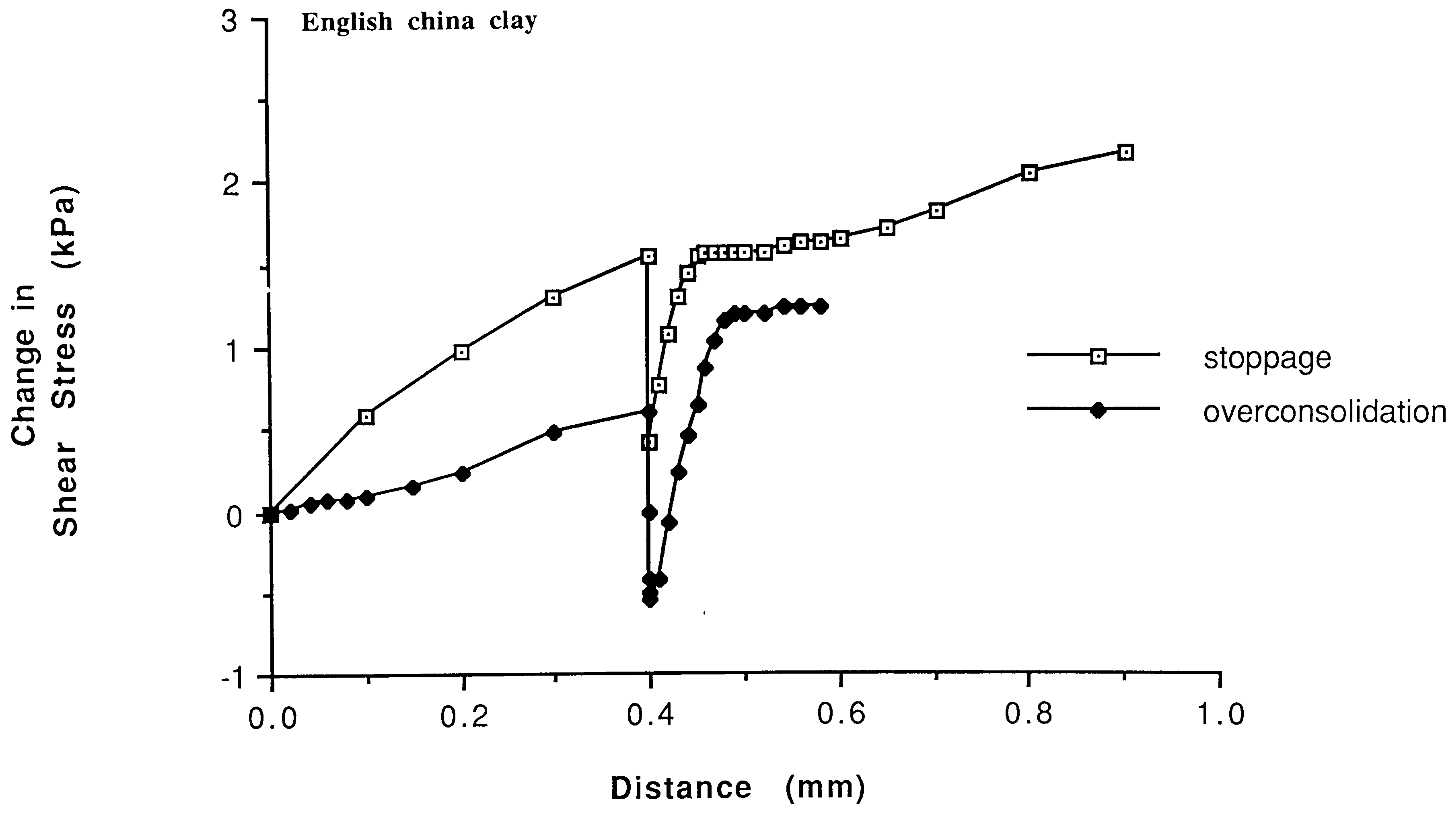


Figure 6.4 Effective consolidation pressure 150 kPa, OCR 20 - effects of stoppage and overconsolidation on residual strength

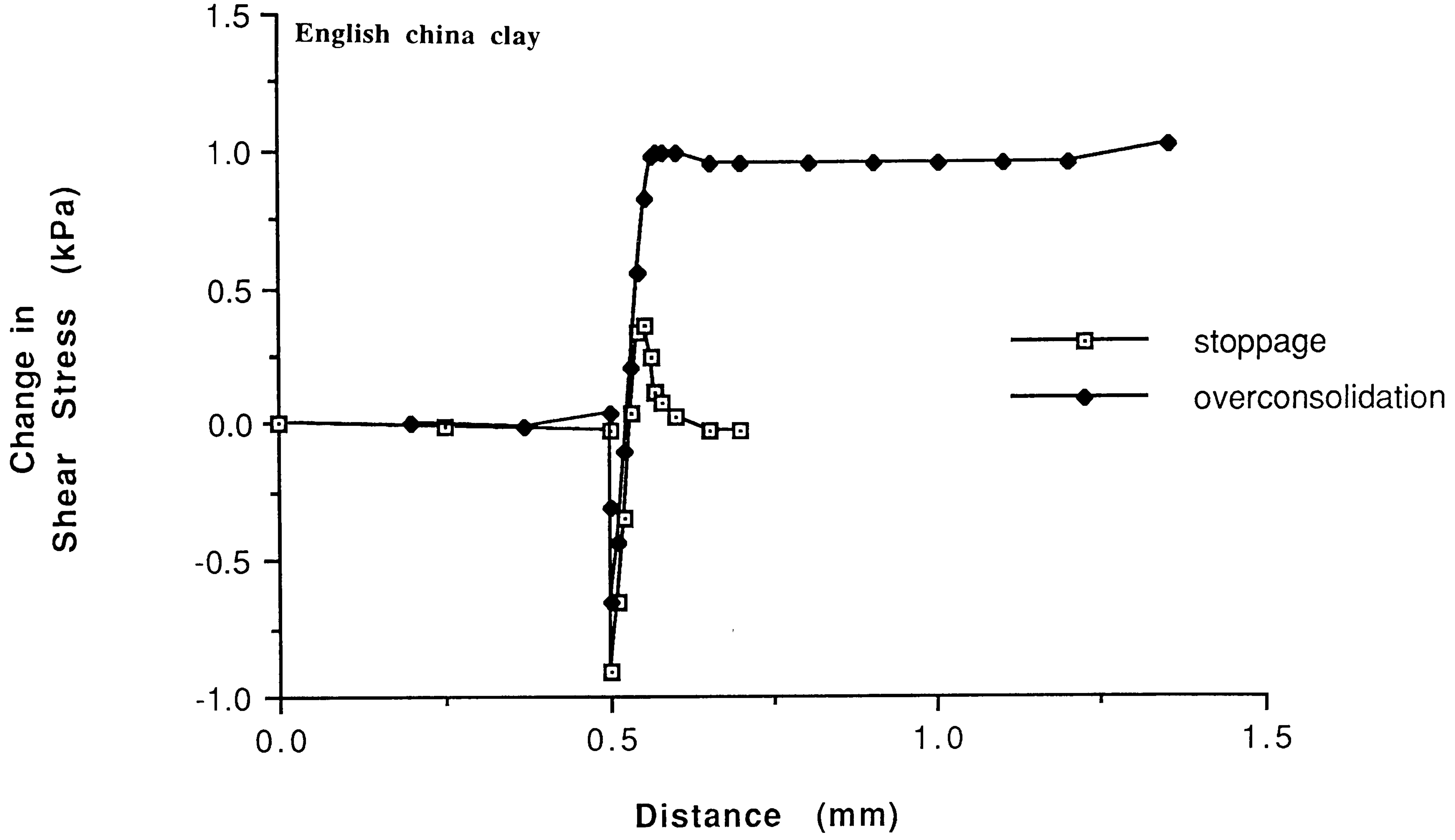
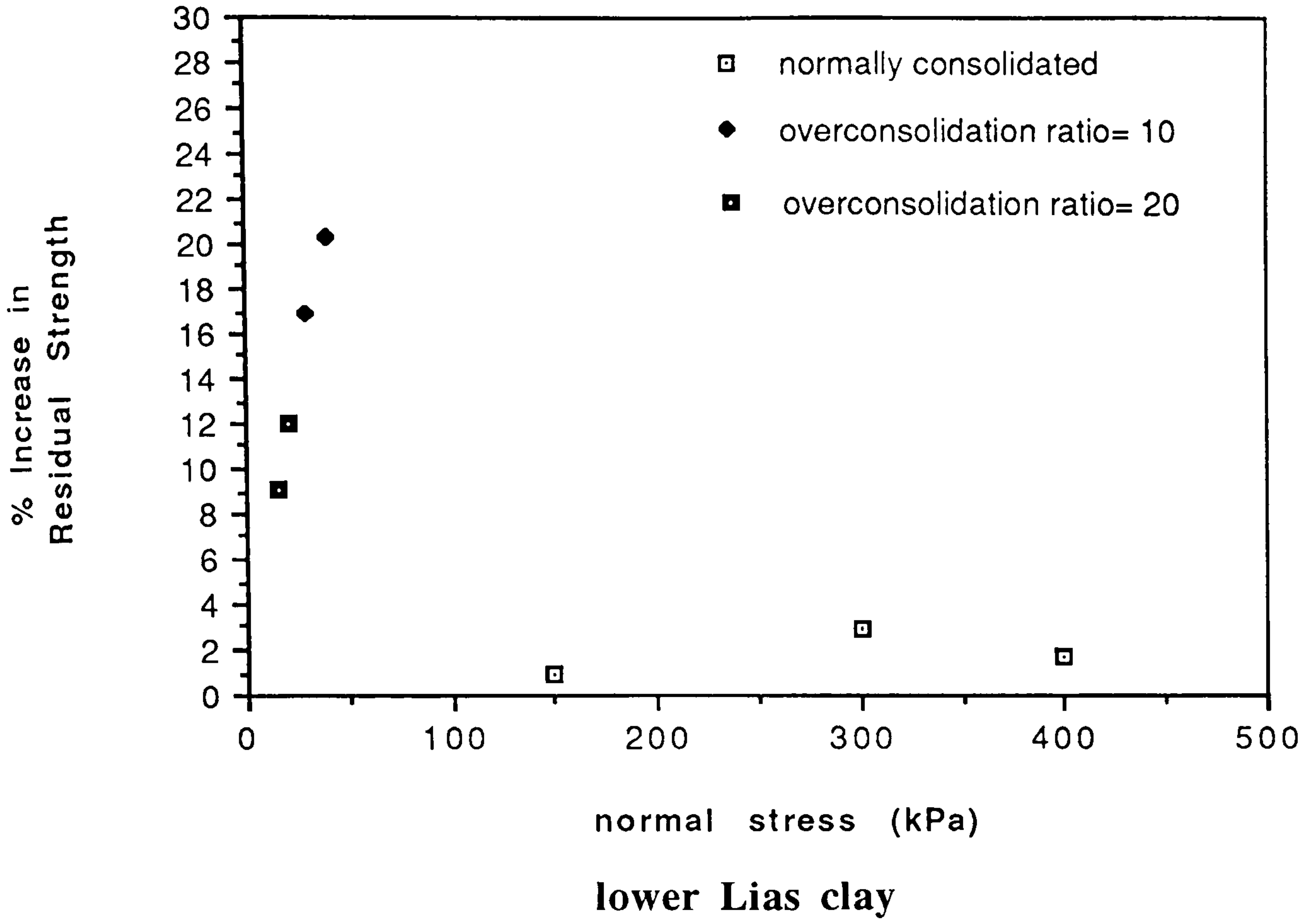


Figure 6.5 Effective consolidation pressure 400 kPa, OCR 20 - effects of stoppage and overconsolidation on residual strength



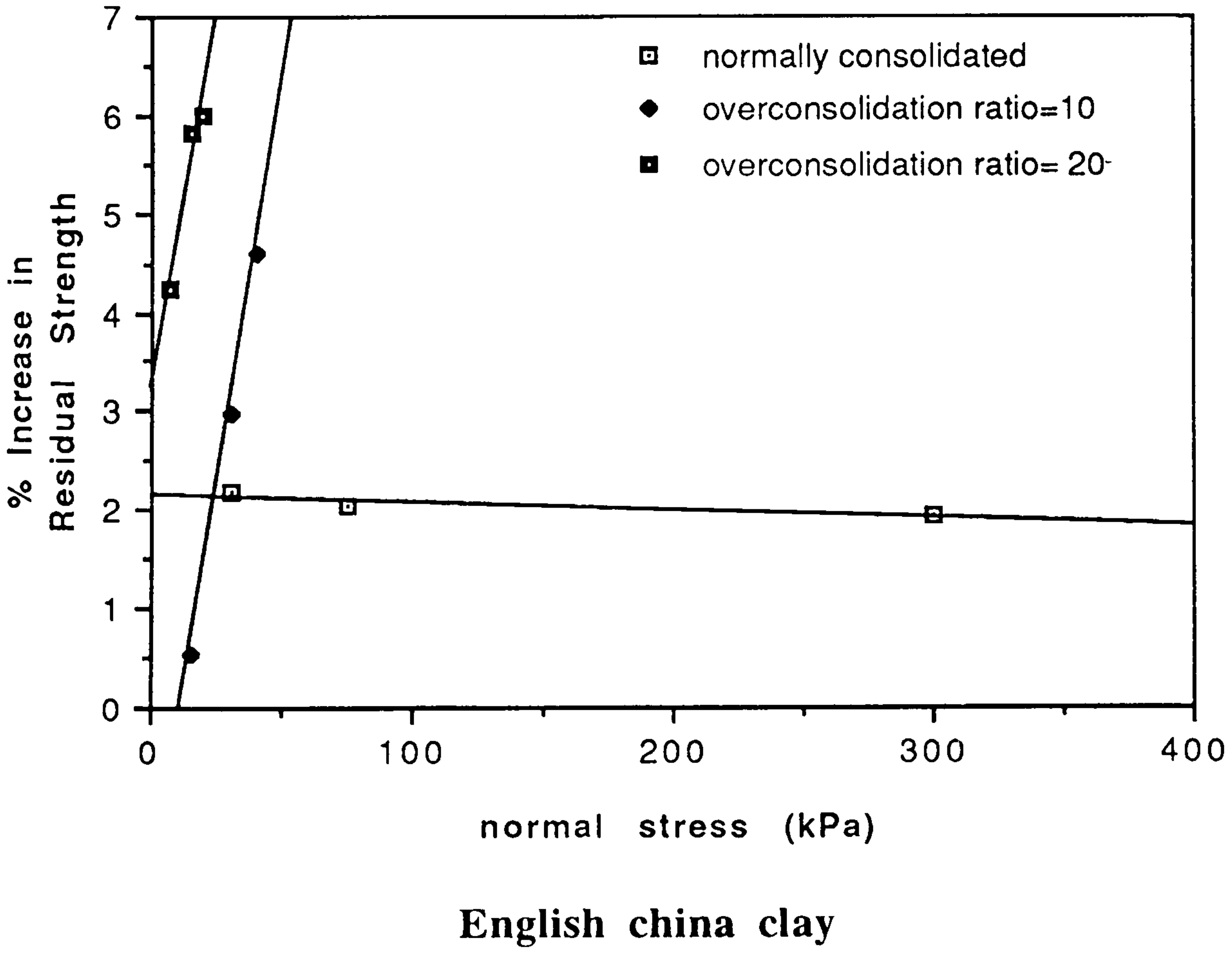
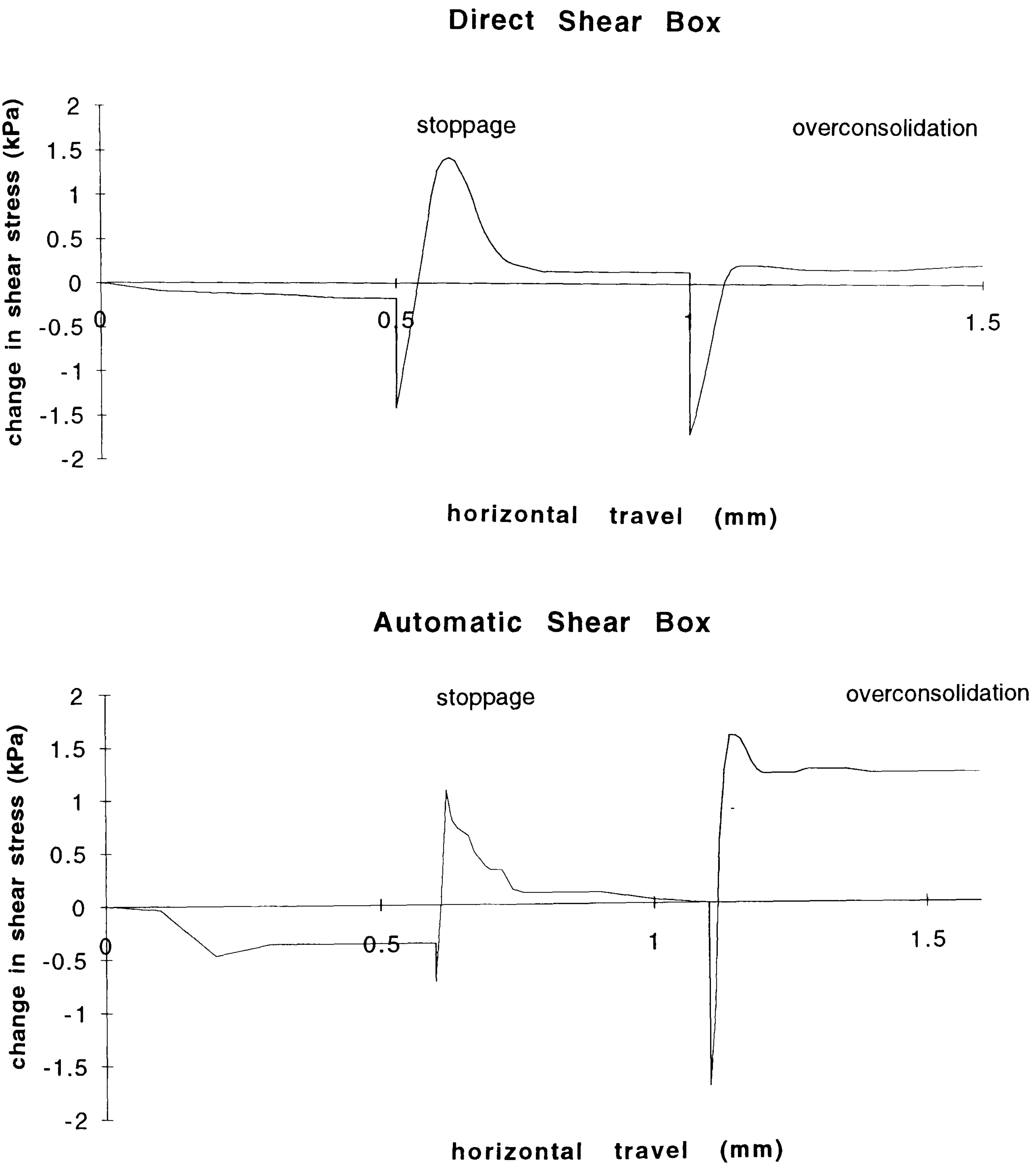


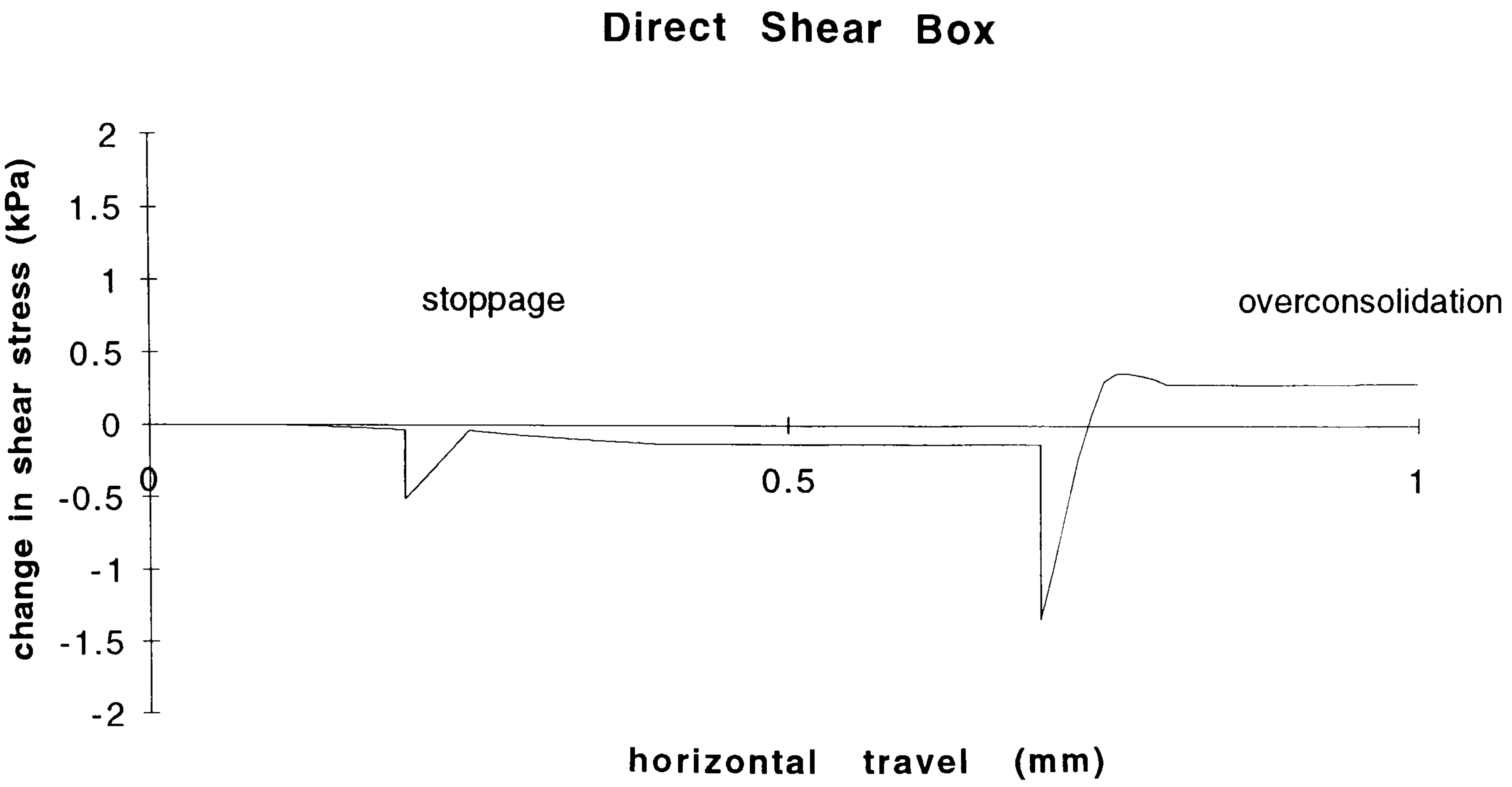
Figure 6.6 Lower Lias and china clay - percentage increase in residual strength after overconsolidation



Direct Shear Box

Automatic Shear Box

Figure 6.7 Effective consolidation pressure 150 kPa, OCR 10 - plot showing stoppage and overconsolidation in both direct and automatic shear boxes



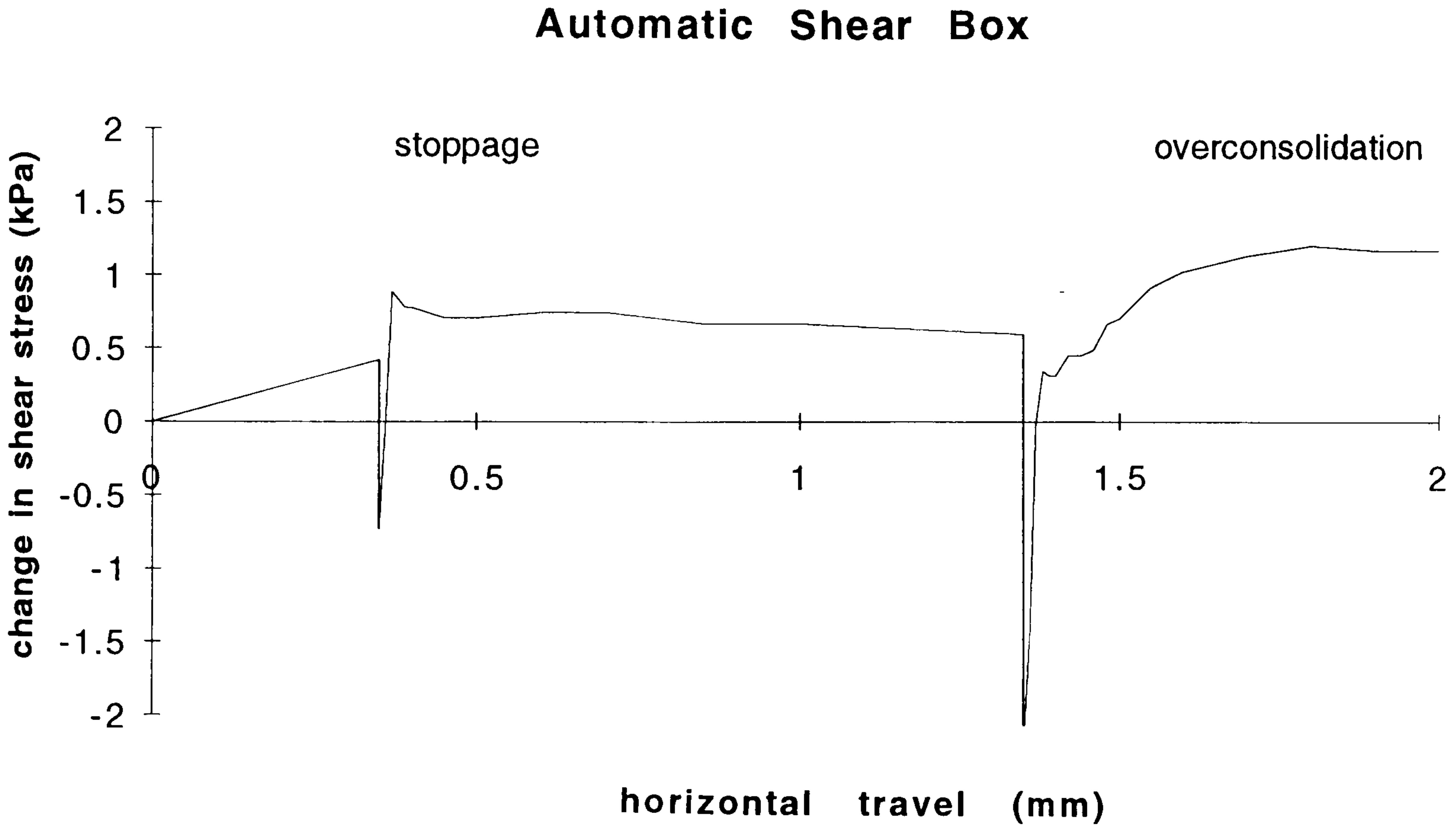


Figure 6.8 Effective consolidation pressure 300 kPa, OCR 10 - plot showing stoppage and overconsolidation in both direct and automatic shear boxes

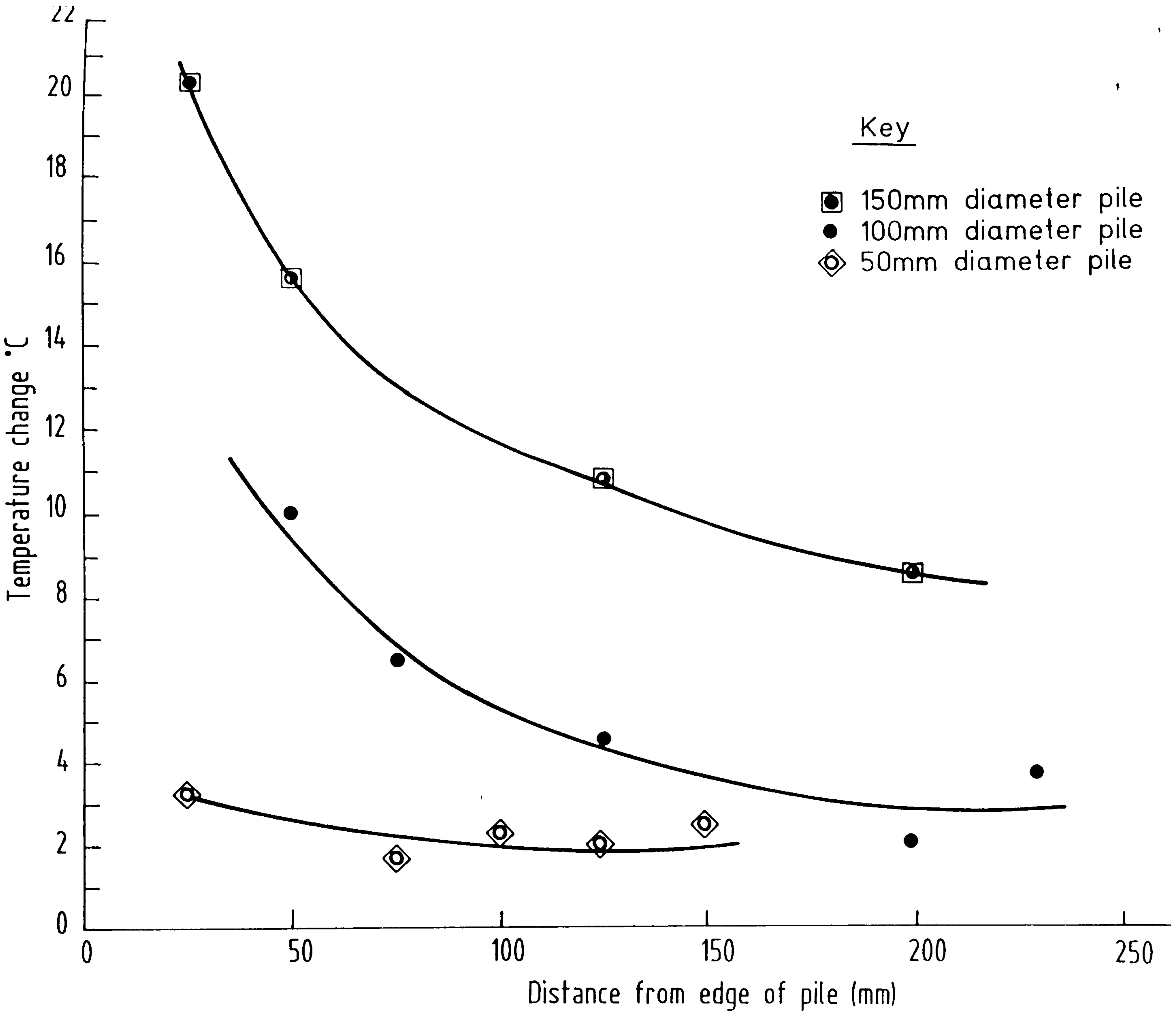


Figure 6.9 Variation of maximum temperature increase with distance from the pile in large box tests using English china clay

CHAPTER 7

7.0 DESIGN OF LIME PILES

Chapter 6 discussed the results from the laboratory investigation in terms of the stabilising processes. These results were also discussed in the context of field conditions. This chapter will present a philosophy for the design of full-scale lime pile installations and use the above information to suggest suitable values for each stabilising mechanism that can be incorporated into such a design.

7.1 DESIGN PHILOSOPHY

The slope failures that have been concentrated upon in this project have been the shallow failures that are causing concern to the owners of transportation routes. The results are also applicable to deep failures. 95% of the failures recorded in the recent survey by Perry (1989) were less than 2m deep. Therefore, in the case of shallow failures, the effective normal stresses acting on the failure plane are small, typically in the region of 10 to 40 kPa. The implication is that a significant change in the effective angle of internal friction (ϕ') of the shear zone would be needed to create stability by frictional means since the resistance is proportional to the effective normal stress.

However, when analysing such slopes it is evident that small changes in the effective cohesion (c') will dramatically alter the factor of safety of the slope. It is argued by many that c' effects in soils are artificial and c' provides a poor basis for analysis. However, if the increase in strength were to be considered as a 'cementation' rather than a 'cohesion', then reliance on such changes becomes more acceptable. In the case of lime piles this additional c' is relatively large and derives from different sources, the effects of which are superimposed to generate a considerable improvement in safety factor. For this reason it is not necessary in most cases to form lime piles throughout the failed area of the slope, but for example to construct the piles to a regular grid pattern over, say, one third of the height of the slope. A parallel research project considering the analysis of failed and treated slopes is being undertaken by Geotechnics Ltd. This is examining the results of differential treatment with the aim of optimisation.

The design process, therefore, is one of optimisation. An initial design was necessary in order to begin such a process. Consideration of the laboratory results and the limitations of plant and cost had to be made. However, the initial stage was to decide what parameters to choose as the variables in the design process. These were pile diameter, pile depth, pile spacing, pile compaction and, as mentioned above, dimensions of a zone of treatment.

The depth of treatment would necessarily be site specific and would be required to allow the piles to pass through any existing failure plane and further potential failure planes. For this reason, it was decided that the piles should be created up to surface (minus a 500 mm clay plug to prevent vertical swelling and stop ingress of rain water) rather than attempt to target any perceived critical zone.

In terms of the strength of the pile, the larger the diameter the greater the stabilising benefit. In terms of the effect of pore water pressure, dehydration and temperature this has also been found to be the case to a certain degree but it was felt that an optimum diameter would be achieved over which little further benefit could be attained. In terms of plant, size of auger available is a limitation and ease of operation must also be a consideration on a site that is necessarily unstable. A notional diameter of 200 mm was chosen but the ability to alter this dimension had to be incorporated into the design procedure to allow for different site conditions.

When considering pile spacing, obviously the closer the piles the greater the stabilising benefit. The effect of each pile at any distance from the pile will be magnified as the piles become closer to one another. In terms of strength, the greatest effect would occur if the whole slope were to be entirely replaced by lime.

Therefore it was necessary to choose a spacing that is practically obtainable, yet remains sufficiently small to ensure suitably consistent performance. A spacing of 1.5 m was chosen.

Compaction has been shown to have beneficial effects in terms of pore water pressure reduction, but had a negative effect on dehydration. Again, practical considerations are of great importance as it is not clear how a high level of compaction could possibly be achieved. It was decided to compact by hand tamping at intervals that were worked out on site so that the installation process was not significantly slowed down.

It was decided from experience of past slope stabilisation works that the optimum treatment is spatially within the lower third of a slope, and this was the area initially targeted. Subsequent analysis confirmed this argument.

So, having decided which material parameters to target in design and which physical variables were critical in the process, it was necessary to evaluate the stabilising mechanisms provided by the lime piles in these terms.

7.2 STABILISING MECHANISMS

In order to determine parameters for design it was important again to examine the stabilisation mechanisms in turn.

7.2.1 Pore Water Pressure

The first point to note was that quicklime piles generate relatively high negative pore water pressures in the overconsolidated clay cuttings and embankments that are typical of the UK. These suctions immediately increase the stability of the slope, in the same way that considerable stability exists due to the high stress relief suctions that are generated when such slopes are created. If the suctions reach the same level as those initially present (e.g.

see Crabb and Atkinson, 1991 and Figure 7.1) then it might be expected that the life of the slope would be considerably extended. In any case the suctions will greatly increase stability in the short term and allow the long-term improvements to take effect, whether derived from the lime piles alone or from a combination of lime piles and other stabilisation techniques (e.g. drainage). In this regard it is considered that any temporary increases in pore water pressure caused by drilling will be rapidly removed and replaced by the negative pore water pressures, and thus will not compromise further the short-term stability of the slope.

The actual value of reduction is difficult to assess. The various influences on its magnitude have been discussed in Chapter 6, a not insignificant factor being the unquantifiable effect of field conditions. The almost certainly drier conditions will increase any effect observed in the relatively wet conditions of the laboratory. The reductions encountered in single piles of 150 mm diameter were of the order of 20 kPa at a distance of 175 mm from the pile. It was noted that increased diameter and grouping the piles will magnify this effect. However, at a spacing of 1.5 m the maximum distance over which the effect must act in the case of the notional 200 mm diameter piles is 860 mm (equivalent to half the maximum distance between two piles, assuming a square arrangement of piles). There will undoubtedly be some sort of radial diminishing of the effect around the piles which has not been possible to quantify from the laboratory results. It is also notable that the effect will also occur to a diminishing extent outside the confines of the treated zone, i.e. away from the edges of the outermost piles. For this reason again, some form of compromise situation had to be reached and it has been decided to apply a reduction of 20 kPa to the entire treated zone, recognising that this may merely be a representation and may not be realistic of what the ground is actually experiencing. However, results from the initial field trial recorded reductions of a similar order at a distance of 700 mm away from a 150 mm diameter pile.

7.2.2 Lime-Clay Reaction

The improvement in stability caused by migration of the calcium ions and stabilisation of the clay under highly alkaline conditions has been shown to be restricted to an annular zone of 20-30 mm from the edge of the pile, at least in the remoulded clay used in the laboratory. The improvement in the strength of the clay caused by this reaction is difficult to measure, due to the limited nature of the zone. However, it has been found that the strength changes occurring when lime is mixed with clay are accompanied by changes in plasticity. The strength of a sample that has lime migrating through it is very difficult to determine, whilst the Plastic Limit is relatively simple to find. As a first estimation of the strength in the annular zone it is proposed to scale the strength found from mix tests by the proportion of the mix test Plastic Limit change achieved by the migrated sample. For instance, a lower Lias clay sample from the perspex tube tests after 28 days curing (from the time the lime had reached the edge of the tube) achieved a 4% increase in Plastic Limit. A sample mixed with 4% quicklime, after the same curing period, achieved a 19% change. This is a ratio of 1:5 which could be used in design to scale down the increase in strength parameters achieved by the mixed sample fivefold.

This has the advantage of being a very simple method which could in the future be carried out for pile installation design using conventional testing methods. Testing has been carried out on several different clays, looking at strength changes occurring after mixing with different percentages of lime. Perspex tube samples of similar clays have already been tested and could be readily repeated using the same or different clays for comparison with published data. An appropriate range of scale factors could be developed.

The disadvantage of this method is its crudeness, it not being clear if such a direct correlation between strength and plasticity changes is appropriate. The difference in lime content may also be a problem. It is difficult to ascertain the exact lime content produced by migration due to the ongoing reaction with the clay or whether it is sufficient to produce stabilising reactions at all.

This was the method used in the design of the field trials as it used the only available data at that point in time. To try to reach a clearer understanding, the mineralogical changes occurring in the mixed and migrated samples were compared and a suitable value to assign to clay strength was developed in this way. A far less contrived and readily obtainable value was derived. This was the strength achieved in an undrained triaxial test after mixing the clay with a lime addition equivalent to the Eades and Grim (initial lime consumption) level and curing for 28 days.

The dimension of the annular zone may be estimated from the box tests. The small box with a 50 mm pile gave 15 mm, the large box with 100-150 mm piles gave 25-30 mm and the U100 tubes with 12 mm diameter piles gave 18 mm. As these tests were carried out in clay at a relatively high water content (LI = 0.5) it may be reasonable to expect that 30 mm is the maximum likely migration distance in the drier field conditions yet where greater time is available and while no hydraulic transport occurs. It is likely that the higher water content associated with a shear zone will promote lime migration and hence the most critical zone in terms of slope stability will be targeted by the treatment. It is also possible that the distance over which migration occurs will be increased to up to 300 mm in the field if water-filled cracks promote hydraulic transport of calcium ions. Indeed, far greater distances of migration might be achieved if the findings of others are shown to be correct (e.g. Handy and Williams (1967) although the mechanisms promoting this migration of both calcium and hydroxyl ions still remain unclear. Observations will be made in order to examine this phenomenon when sections of the field trials are excavated after a period of at least twelve months subsequent to pile placement.

This improvement was applied to the slope as a whole by the consideration of the ratio of the area affected to the area of clay that is not affected, and thus the improvement when (considerably) factored down was relatively small, although it might not be insignificant for the reasons expounded above.

As briefly mentioned in Chapters 5 and 6, evidence has come to light which could mean that the entire treated zone is modified by calcium ion exchange after a period of some 10-15 years. Research is currently being undertaken in order to substantiate these ideas. However, if this is the case it should be relatively simple to find the enhanced strength of the clay material by mixing clay from any site with a lime addition equivalent to the Initial Lime Consumption ('Eades and Grim') value. The precise area over which this strength applies will be more difficult to deduce and will vary with time. Research is underway in order that this may be assessed using considerations of permeability and clay structure. At the most simplistic level, a best and worst case scenario may be applied where none of the clay is modified immediately after pile placement and all of the site is modified at a specified time later, say 15 years.

7.2.3 Dehydration

The dehydration of the clay is potentially significant also, since the process of dehydration is accompanied by effective overconsolidation of the clay. The increase in strength caused by dehydration can be estimated, as illustrated above, although it is not the increase in strength of the intact clay that is of most importance here, but the effect on the shear zone. The lime pile will act as a desiccant within the slope and will thus differentially remove water from the clay since the availability of water will vary. Thus the shear zone, which typically has a higher water content following dilation during shear, will have more available water and thus will lose water more readily, and hence rapidly, after lime pile installation. For this reason the gain in strength in the zone of least strength will be proportionally greatest, while the areas away from the shear zone, through which secondary or subsequent slippage might occur, will also be strengthened.

The degree of improvement in strength clearly depends upon the volume of water removed from the clay, and thus the volume of quicklime added to the slope and the degree of expansion occurring. This in turn is dependent on the pattern, spacing, diameter and depth of the piles. Current studies are underway investigating the effect of confinement on the degree of expansion. Initial results suggest that at increasing effective normal stresses expansion is reduced. Hence the depth of the point of interest may also be of consideration, although at the low effective normal stresses associated with the treatment of shallow slopes the degree of confinement is relatively low and hence expansion should be comparable to that observed in the laboratory. The reduction in water content has been averaged over the plan area of the slope for design purposes, with a global increase in associated strength gain, in the initial work. As the design process would have proportioned a radial effect by a ratio of affected to non-affected areas (as with migration) it was felt that this averaging would have made little difference to the design.

It is hoped that further work, by subsequent excavation of the field trial treated areas, will enable a better idea of differential desiccation such that the design parameters used for checking the stability of the critical shear surface can be refined based on shear zone dehydration. One potential problem, however, is the differential dehydration that occurs in the plane of cross-section, causing rapid water loss and tension cracking adjacent to the piles while the clay away from the piles remains relatively unaffected. If this phenomenon, caused by the relatively low permeability of the clay in comparison with lime's high demand for water (and noticed in the larger box tests in clays of high water content), is repeated in the forthcoming field trials then the potential exists for ingress of surface water through the cracks. The clay will be rehydrated and the beneficial effects of drying will be removed. However, this may be off-set by the enhanced migration produced by the facility for hydraulic transport provided by the cracks. One method of counteracting this potential problem would be to auger the holes to beyond the shear plane, and through any potential underlying shear planes, and to fill the holes with quicklime to a point, say, 0.5m above the shear plane. The remainder of the holes would then be plugged with excavated clay and/or a bentonite seal.

7.2.4 Pile Strength

The strength of the pile itself is relatively simple to incorporate into the design process, since a typical strength (0.5 MN/m^2 was determined for the lime piles from the laboratory tests) can be applied to the slope using the ratio of areas of the pile and the surrounding clay. The design process carried out for the full-scale field trials has emphasised the importance of this parameter in governing the long-term stability of a slope stabilised by this technique. Work is now being conducted to investigate further the strength of quicklime after slaking and the factors affecting its development and ultimate value. Work will also be carried out to investigate the effect of PFA addition on the strength of the pile.

In order to rely upon this strength, the construction operations must be subject to adequate controls to ensure appropriate compaction, and thus density, requirements are met uniformly throughout the pile. A factor could be applied to the strength to account for any variability in this respect.

7.2.5 Shear Plane Consolidation

The strength increase caused by the overconsolidation of the shear plane can be estimated from the data presented above, but in practice would be determined from shear box (or other shear) tests in the laboratory. The degree of overconsolidation of the shear plane, i.e. the increase in mean normal effective stress, is dependent upon the negative pore water pressure generated. It would appear reasonable from the laboratory work, however, to assume that a mean normal effective stress increment of 20kPa will occur and it is suggested that this value be used in tests that replicate the stress history of the clay concerned. Thus for clay strata from which undisturbed samples cannot be taken, remoulded samples should be consolidated to the preconsolidation pressure, allowed to swell to recreate the overconsolidation ratio *in situ* and then temporarily overconsolidated using the 20kPa increment in the manner described above. The strength increase, either as a direct increase or interpreted as an increase in ϕ' above the residual value, can then be used for design.

7.4 STABILITY ANALYSIS

The results of the laboratory testing programme were used for the slip analysis and in the design of the pile installation. These were carried out by Lina Lucas of Geotechnics Limited as part of the parallel project and will be published in her MSc thesis. Essentially the analysis adapted existing slope stability techniques for both circular and non-circular analysis.

The failed slope was analysed, in the traditional manner, using material parameters found from site investigation and laboratory testing, and where appropriate, the position of the slip scarp. The factor of safety was assumed to be just below 1 if the slope was found to be still moving, or just above 1 if no further movement could be detected. A desired factor of safety was decided upon and the treatment adjusted so that it could be met. In the case of the canals treated in the field trials a factor of 1.1 was deemed appropriate due to the low risk factors involved.

Results from the laboratory programme have been converted into suitable parameters for use in the programmes. Improved strength parameters were apportioned in accordance with the proportion of the area treated. In other words, strength parameters for the pile material were assigned to the diameters of the pile only, improved clay strength (from the lime-clay reaction) only to the annular migration zones around each pile. From then the analysis concentrated upon finding the optimum region on the slips cross-section, for treatment, Figure 7.2. As previously stated, pile size was largely dictated by available equipment, so that the analysis was used to fine-tune the pattern of piles so that the desired factor of safety could be achieved.

7.5 TIME DEPENDENCY

When applying these parameters to the design process it should remembered that the improvements are time-dependent, although the short-term improvement caused by the negative pore water pressure (suction) effects should counteract any instability problems that would occur prior to the longer-term improvement occurring. Another important aspect of this work is the strain compatibility of the treated and untreated clay, and of the pile itself, when subjected to shear forces. The extreme example of strain incompatibility is the case of slopes that have been stabilised using conventional augered piles at wide spacings and that have continued to move as a result of plastic flow of the clay between, and around, the piles. Lime treated clay is well known for its progressively developed brittleness as the stabilisation reactions progress (e.g. Rogers and Bruce (1990), and Rogers and Lee (1994)), a graphic example of the change in stress-strain properties being given in Figure 7.3. The stress-strain curve of the hydrated, solidified lime forming the pile is similarly brittle, and thus the progressively increasing loss of ductility as the centre of the pile is approached from the main body of the clay should be accounted for in design.

7.6 CONCLUDING REMARKS

The design process, as outlined above, has been applied to two field trial sites which will be discussed in Chapter 8. In addition to the considerations discussed above of plant suitable for site specific conditions and soil conditions, restrictions of time and labour also had to be reflected in the designs. Excepting this, the design process was worked through in the same manner as that which would be used on any future installation. The results of that design process and its practical outcome is the subject of Chapter 8.

For the design of further installations, further data are required in order that specific site conditions may be incorporated into the design. The effect of general parameters such as clay water content and mineralogy, pile geometry and the position of the shear zone must all be quantifiable in terms of all the stabilising mechanisms. The gathering of these data will require the combination of laboratory testing, to tackle specific parameters, and the rigorous monitoring of field installations to gain more detailed information about the effect of field conditions. It would be of great benefit therefore if the first full-scale usage of this technique were to be monitored carefully. Thus future installations could be designed with confidence.

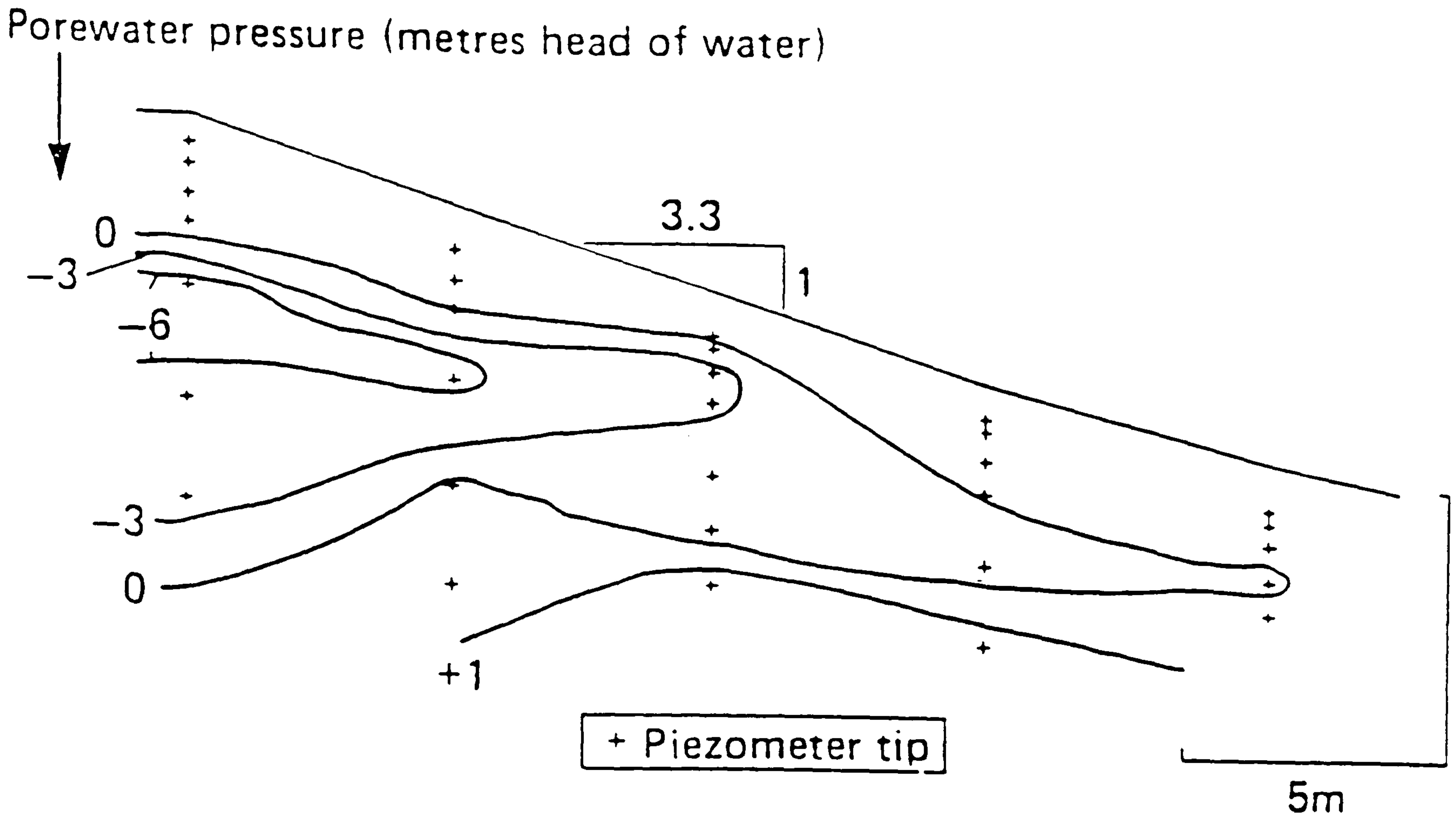


Figure 7.1 Typical distribution of pore water pressure with depth in an embankment monitored by the TRL near Writham
(after Crabb and Atkinson)

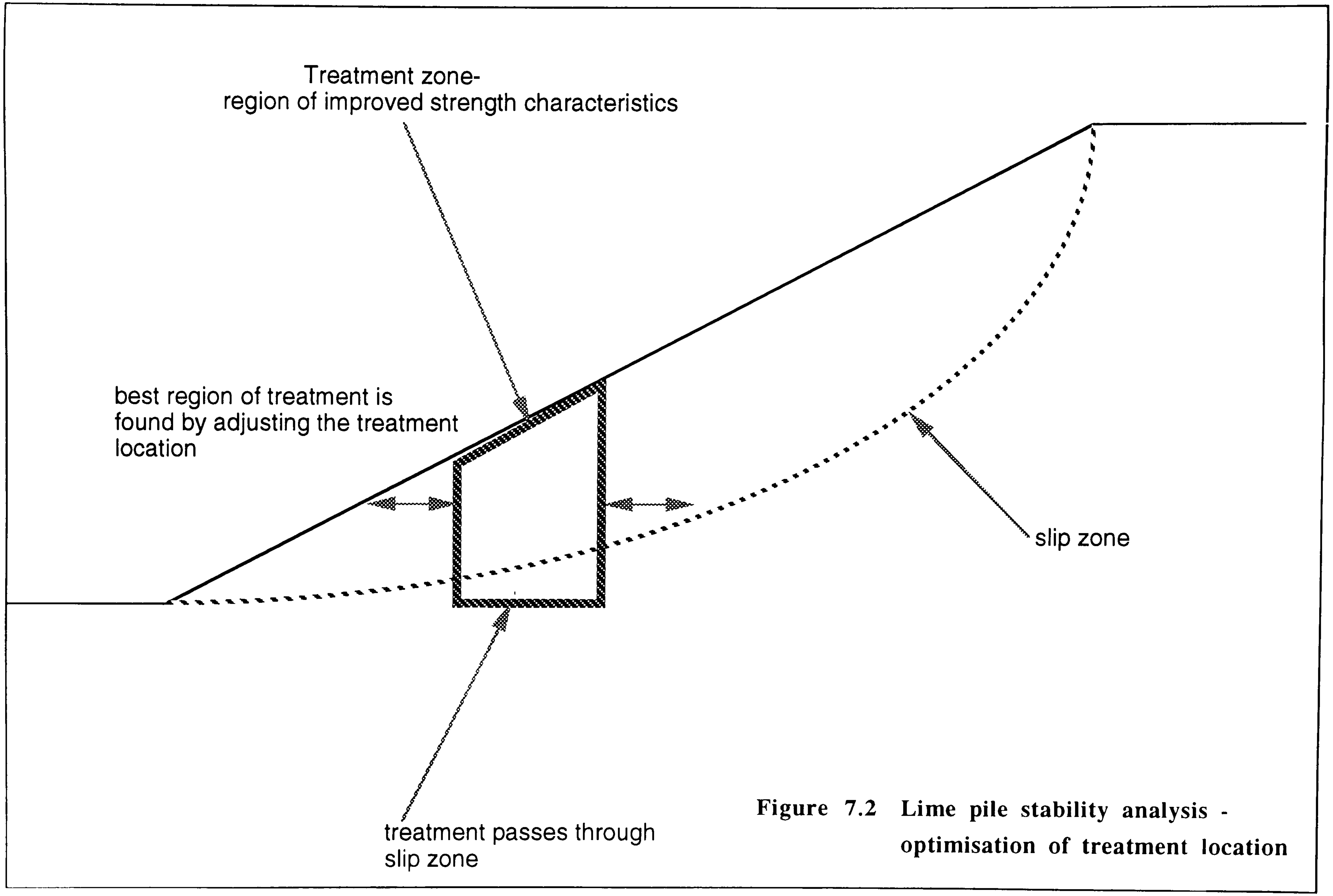


Figure 7.2 Lime pile stability analysis - optimisation of treatment location

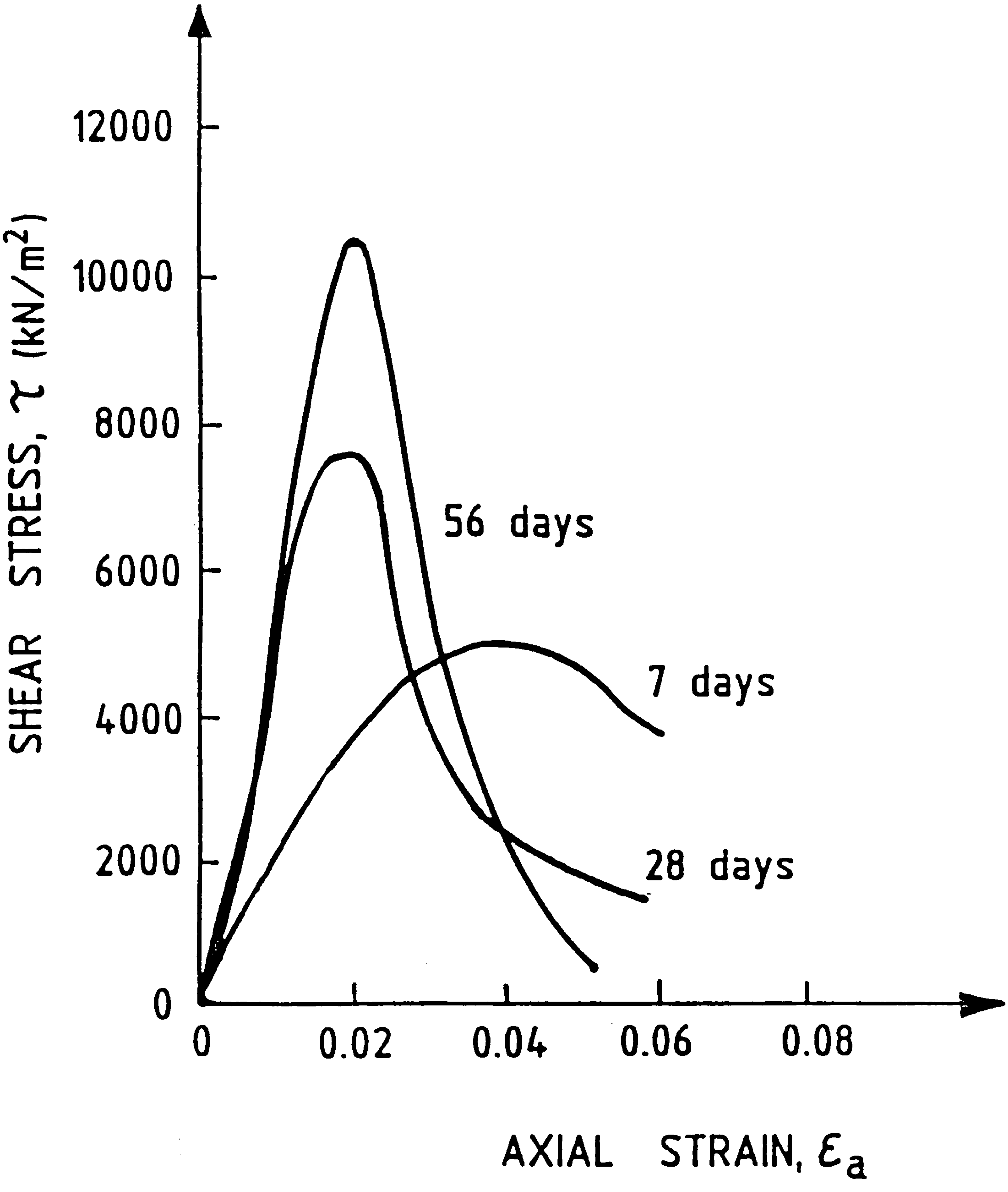


Figure 7.3 Shear stress against axial strain for undrained triaxial compression tests on lower Lias clay with 4% lime and zero confining pressure
(after Rogers and Lee, 1994)

CHAPTER 8

8.0 FIELD TRIALS

The aim of the field trials was to confirm the data obtained from the laboratory investigation for design and to gain information not available from the laboratory. Practically this was achieved through the development of the design process and practice of full scale installation techniques. Limited information could be gained about the effect of the piles on ground conditions *in situ* during the installation process and the period immediately afterwards through the monitoring of pore water pressure changes. Additional information will be gained, specifically concerning the effect of field conditions on the remaining stabilising mechanisms, by excavating areas of both trials at a later date.

8.1 FENNY COMPTON

8.1.1 Location

The site lies approximately twelve kilometres north of Banbury where the A423(T) crosses the South Oxford Canal and the Didcot to Chester railway line. The site has the approximate Grid Reference SP 438 522.

To the east of the A423 the canal and the railway line run parallel to one another, the canal approximately 20m to the north. An area of the canal bank 350m east of the road and adjacent to the railway has failed in two sections (see Figure 8.1 and 8.2).

The canal in this area used to be in a tunnel but a cutting, approximately 1 km in length, was created around the turn of the century. Some previous stabilisation works have been carried out to the north bank, accompanied by the stripping of 2m of fill from the top of the south bank.

The area around the slippage zone is notably free of the dense vegetation characterising the remainder of the south bank. There is evidence of an attempt to redress this by the planting of a number of young trees. During the summer months the immediate area of the slips becomes overgrown with seasonal vegetation highlighting it from its surrounds.

8.1.2 Geology

Published geological information, from Geological Sheet 201 and Geological Memoirs 201 entitled "Geology of the country around Banbury and Edge Hill", indicates the area to be underlain by lower Lias clay of the Jurassic age. No drift deposits are indicated within the vicinity of the site. This area of lower Lias clay is said to consist of an alternation of fine-grained argillaceous limestones with grey calcite mudstones and shales.

Previous available site investigations have largely confirmed this description.

8.1.3 Previous Investigations

In April and May 1987 Exploration Associates (Warwick) Limited undertook a site investigation on behalf of Warwickshire County Council. The findings of the investigation were recorded by Warwickshire County Council (1987). The purpose of the work was to investigate distress in the bridges over the railway line and design foundations for their replacement. Hence the exploratory work was concentrated in the area immediately adjacent to these structures.

The investigation comprised the drilling of thirteen light cable percussion boreholes, eight being extended by rotary boring techniques. Seven standpipe piezometers and three inclinometers were installed within particular boreholes. A programme of laboratory testing was carried out on samples taken from these boreholes.

All of the boreholes revealed made ground or topsoil overlying lower Lias deposits. The shallow lower Lias deposits were said to consist of stiff to very stiff dark grey silty fissured clays tending to very weak siltstone with depth. Evidence of selenite crystals was also found within the clay.

The water content values were observed to reduce with depth, the upper deposits averaging 27% while the lower deposits reduced to 16%. Undrained shear strength values increased with depth in the range 96 kN/m^2 to 290 kN/m^2. Atterberg limits indicate the lower Lias to be of intermediate to high plasticity with average values of Liquid Limit and Plastic Limit of 54% and 30% respectively. Consolidated undrained triaxial and drained shear box tests recorded effective stress parameters in the range c'=0-9 kN/m^2 and Ø'=27-34^o. (The values of Liquid Limit, Plastic Limit and water content found for the lower Lias clay on the south bank were 54%, 30% and 30% respectively).

Undrained shear strength values for the siltstone ranged from 275 kN/m^2 to 2200 kN/m^2.

Soluble sulphate tests on the lower Lias clay revealed values ranging between 0.12% and 1.08% with tests on a 2:1 water soil extract (on those samples above 0.5%) giving concentrations in the range 1.92g/l to 4.58g/l.

In June of the same year another site investigation was carried out by B.B. Drilling Limited for British Waterways Board with results recorded by British Waterways Board (1987). The investigation was designed to provide design parameters for remedial works to the north bank referred to earlier. Twelve boreholes were sunk at various positions in the north bank and associated laboratory testing was performed. A similar sequence of strata to that of the previous investigation was described, giving comparable test results.

Using these results, several sections have been drawn in order to predict the possible details of geology below the slips, as shown in Figure 8.3.

8.1.4 Site Investigation

The purpose of this investigation was to provide information for the design and monitoring of the first field trial.

Trial pits were dug by hand into both slipped areas in the locations shown in Figure 8.4 in order to establish the possible depth of the shear zones and provide samples of the encountered material for laboratory testing. The trial pit details are given in Appendix 7, which show that the shear zone of relatively soft, wet clay approximately 30 mm thick occurred at a depth of 3m. The shear zone was sufficiently soft to allow a spade to slide through it whilst the clay immediately below was so stiff that further excavation was prevented and that above was very difficult to excavate by hand.

Three hydraulic piezometers, numbered 1, 2 and 3, were installed into the smaller slip at the locations shown on Figure 8.5. They were installed in the centre of approximately 1 m long sand cells at depths of 2.3, 2.9 and 2.6 m respectively. Installation at these depths was intended to permit interception of the shear zone. All installations were achieved using a 'Minuteman' rig to monitor specifically the effects of a small trial section of lime piles, described below.

8.1.5 Small-Scale Trial

Results of the laboratory programme available at the time of the design of this trial concerning the mechanisms of ion migration indicated that particular site conditions which proved impossible to recreate in the laboratory may have a significant bearing on the success of the process. These included water content, water flow and soil fabric. The idea of retrieving a large undisturbed sample from the site of the trial was investigated but proved impractical. A small field trial to study the effect of the above processes to complement the laboratory programme therefore seemed sensible at this stage. An investigation was made into field methods of measuring calcium levels before the laboratory programme indicated that this would not yield useful data.

The piezometers were installed and read and de-aired on a regular basis for a six month period prior to the installation of the piles. These indicated that the water level in the slope is approximately the same as that of the canal.

Seven holes were drilled at the positions shown in Figure 8.5, again using a 'Minuteman' Rig. Holes 1 to 5 were drilled down until the perceivably harder layer, as found in the trial pits, was reached (approximately 3 lengths of the drill). This meant that the actual holes were only 2.5m deep due to the spoil irretrievably left in the base of the holes. It was felt, from the evidence provided by the trial pits, that this was sufficient to penetrate the slip plane. However, holes 6 and 7 were drilled down to 4 lengths to ensure an entirely open hole down to the stiffer material.

Several problems were experienced with the lime. The presence of fines meant that the material was difficult to handle and even larger sized pieces easily broke down into dust. An attempt was made to fill holes 1 and 2 by pouring the lime down the holes and then compacting. Due to the fines and the lime's fierce affinity for water the holes were bridged at a relatively shallow depth. The bridged lime cemented and formed a plug too hard to be pushed out. Subsequent holes were filled by pouring lime through a tube inserted into the hole and slowly withdrawn. Holes 1 and 2 were augered out by hand and filled in the same way. This requirement of masking the hole from the lime may need to be considered when developing ways of installing full-scale lime piles, although evidence from the full-scale trials conducted indicated that larger diameter holes did not cause this problem.

Hole 7 was not filled for some five days after the start of the trial due to the presence of a blockage in the hole. This had to be augered out using a 100mm diameter hand auger and, due to the restriction on depth that could be achieved with this device, a depth of more than 2m could not be reached.

It was noteworthy that no water was encountered in any of the holes except 7, which had stood empty for 4 days. Perceivably softer material was encountered in hole 5 at a depth of 2m. Each hole was capped with 150 mm of Bentonite placed as dry powder.

Pore water pressures were monitored closely during the period of the trial and on a regular basis thereafter. Figures 8.6 to 8.9 show clearly the response of the piezometers to the installation of the piles. It can be seen that the initial response was an increased pore water pressure, with a subsequent drop in piezometers 2 and 3. As number 3 was the piezometer in most close proximity to the piles it was the most likely to show an immediate response. Also, there was a delay in the installation of pile 7 which may have affected the response of piezometer 2.

The response of piezometer 3 is also most easily explained. A combination of the drilling process, compaction of the lime and possible swelling caused an increase in pore water pressure of approximately 12 kPa. This was followed by a fall in pressure caused by the slaking reaction by 24 kPa to 12kPa below the original baseline readings. On completion of the reaction pore water pressures returned to those experienced prior to pile installation. This was primarily due to its position below the water table and the highly fissured nature of the surrounding clay. It may also be possible that the shear zone could have caused water ingress from the canal. Piezometer 2 at first appeared to respond similarly, only in a less pronounced manner. However, the subsequent increase in pressure followed by a fall is not experienced to the same extent in piezometer 3. It is possible the reaction has been delayed for some reason in the piles surrounding piezometer 2. The rise in pore water pressure is similar in magnitude to that previously experienced in 3. It may be possible that, as Chiu and Chin (1963) suggest, the immediate reaction causes a 'skin' of reacted material which prevents further reaction until swelling causes this skin to crack. This, however, does not explain why such a sequence of events did not happen in 3.

In retrospect, it is perhaps unfortunate that the area chosen for the trial was so close to the canal, where the slip plane passes beneath the level of the canal. This was originally chosen as no means of design was available and it is generally considered most effective to treat the lower part of a slip. Also, it was felt that the relatively high water table would promote ion migration. Unfortunately, the choice of trial location meant that pore water pressure changes are very greatly influenced by the level of the canal. With the emphasis of stabilising now thought to be governed by pore water pressure changes and pile strength, the positioning of piezometers was altered in further trials with this in mind.

8.1.6 Full-Scale Trial

The major restriction on further works has been the lack of monitoring possible. This was largely due to a shortage of funds. However, funds became available from British Waterways Board to install equipment into the 'large' slip and installation took place in early 1993. Two slip indicators were installed to a depth of 3.4 m (S1) and 3.2 m (S2), as shown in Figure 8.10. Three hydraulic piezometers, installed to a depths of 4 m (P1), 3 m (P2) and 2 m (P3), were also installed at the locations shown in Figure 8.10.

Access to the site was very limited. There was no tow path on the side of the slipped region and the waterway was required to remain open at all times. The situation was further complicated by the close proximity of the railway running parallel to the site on the southern bank. After consideration of these access problems it was decided to carry out the full-scale trial using the Minuteman Rig. The installation was designed on the basis of the limitations to the size of the drill and the depth of the mudstone (at which point drilling was forced to cease). Due to financial constraints, the contractor having pledged a certain total sum to be donated to the project, two weeks drilling could be undertaken using two operators. A timed trial of the equipment indicated that 75 holes of the desired depth could be completed in this time.

Three rows of 150 mm diameter lime piles, each containing 25 piles, were constructed at 1.2 m centres in March 1994, Figure 8.11. Drilling was to begin around a piezometer in order to retrieve the most data. The spacing was designed using the process referred to in Chapter 7. Lime was placed using a similar method to that used in the small trial. Automation was not considered necessary due to the slow nature of the drilling process using this equipment.

8.1.7 Results

In practical terms the trial was very successful. The process, although slow, achieved steady progress and the 75 holes were complete within 8 working days. The drilling process is illustrated in Figure 8.12. The non-automation of the filling of the holes with lime did not pose significant practical problems or time restrictions. However, in a commercial environment drilling and installation of lime might take place over extended periods The safety of operators handling quicklime may then need to be better addressed than the basic, although adequate, precautions taken herein. The filling and compaction processes are illustrated in Figure 8.13.

The lime was compacted after each 50 kg bag had been emptied into the hole, using a wooden stake and, on average, 2 bags were used in each hole.

Spoil from the drilling process was used to create a platform for the next hole and to form the clay plug at the top of each pile. Insufficient material was used for the plug (100- 200 mm) and within days of placement the piles had expanded upwards out of the ground, in some cases as much as 300 mm, as shown in Figure 8.14. This poses the problem of a lack of confinement at the top of the pile, thus forming a pile of potentially reduced strength at least in its upper portion.

Piezometer readings were taken on a twice daily basis by the drill operators. Unfortunately, due to a misunderstanding over the detailed operation of the monitoring equipment, all of the data from this period were lost. Further readings were subsequently taken but appear to indicate that inappropriate operation of the monitoring equipment had caused the system to be almost entirely full of air. It also appeared possible that the system may never have been fully de-aired, since it was not appreciated that the de-airing system adopted for the small-scale trial might not be sufficient to de-air this larger system. Only the very lowest values of pore water pressure previously recorded relative to the read-out cabinet corresponded to water levels below ground level. Even then a water level above canal level is indicated, which does not correspond to the data collected from the small-scale trial and thus renders them suspect (although not necessarily wrong). All piezometer data have been plottted and are shown in Appendix 8

Further regular readings are currently being taken, in conjunction with a more rigorous de-airing technique, in an attempt to gain further information. Eventually, a steady state will again be reached. This may be compared with that at the time immediately after pile placement. Thus, the pore water pressure response due to pile placement may be inferred.

The slip indicators were read immediately before the trial took place and indicated that movement had taken place only in S1 at a depth of 1.68 m from ground surface. This would correspond to a shear zone above the stiff mudstone and is therefore in agreement with evidence from the trial pits. Recent readings have not indicated further movement in the remaining slip indicator confirming the surface observations that no further movement of the slip has occurred. This is in contrast with several other areas of the cutting slope where new, and apparently continual, movement is taking place.

8.1.8 Conclusions

The process practically was very successful, indicating the potential of the use of relatively small scale plant. This will allow the treatment of very inaccessible sites using the process. This would be of particular benefit to densely vegetated sites where the use of larger equipment would prove difficult or would be prohibited due to environmental considerations. It may also prove useful in the treatment of cutting slopes adjacent to major communications links where lane closure would be very costly. Further sites, such as the one treated here, where access is limited to small water going vessels, are also possible. The system therefore has potential as a low-cost means of treating traditionally 'problem' sites.

Compaction of the lime in practice could prove difficult to ensure, if only in terms of persuading operators to carry out the operation regularly when target production is being attempted. This procedure would have to be stressed in any future operating instructions. The placement of an inadequate clay plug was also a problem that was exacerbated by the operators tending to overfill the holes. This again should be stressed in the operating instructions. These two factors are critical for, as previously discussed, an inefficiency in these areas may lead to piles of reduced strength and the long term reliability of the technique appears to rely heavily on this factor.

8.2 IVER

8.2.1 Introduction

This site, having a Grid Reference of TQ 042 806, is situated on the Slough Arm of the Grand Union Canal, adjacent to the M25, at Iver near Uxbridge. The failure is in a London clay cutting, see Figures 8.15 and 8.16. It is, at present, causing some navigational problems.

8.2.2 Previous Investigation

In August 1978 Nicholls, Colton and Partners undertook a site investigation in the immediate vicinity of the road bridge after it began to fail. This was reported by Nicholls, Colton and Partners (1978). They concluded that this was due to the failure of the cutting on the southern bank (the opposite side to the current failure). This was thought to be due to the age of the cutting in relation to the time-dependent reduction in shear strength of London clay. It was probable that failure in this locality (rather than any other) was aggravated by the consistently higher water table. Some geotechnical data are available from boreholes drilled at this time. Following the construction of the M25, the northern embankment was reinstated by Wimpey after using it as a route for construction traffic. It is thought by British Waterways Board staff that this reinstatement covered over a small stream (Figure 8.15). The failure of the northern bank occurred soon afterwards (Figure 8.16). It is possible that the alteration of groundwater conditions caused by the burial of the stream could have aggravated an already potentially hazardous situation.

8.2.3 Site Investigation

A site investigation was carried out by Geotechnics Limited in October 1993 and is described in their report, Geotechnics Ltd. (1993). Six 150 mm diameter boreholes were drilled by cable percussion tool techniques to depths varying between 4.35 m and 6.40 m below ground level. The positions of these in relation to the existing slip are shown in Figure 8.17. Hydraulic piezometers were installed in boreholes 1, 2, 3, and 4 at depths of 2.2, 3.5, 5.0 and 2.3 m below ground level respectively. Two piezometers were installed at depths of 3.2 and 5.6 m in borehole 6 in order to investigate above and below a potential slip. The precise location of the slip plane could not be located, although stability analysis indicated a depth of 3.5 metres at this location. All piezometers were placed with 1 m long sand cells. Inclinometers were also installed into boreholes 3, 4 and 5 to a depth of 6 m below ground level. Full details of borehole records, sampling programme and the subsequent laboratory testing programme are available from Geotechnics Limited, Geotechnics Ltd. (1993).

The results of the laboratory testing programme were used for the slip analysis and in the design of the pile installation. These were carried out by Lina Lucas of Geotechnics Limited as part of the parallel project and will be published in her MSc thesis.

Piezometers were read and de-aired on a regular basis, the results of which will be presented later. The inclinometers have been monitored by staff of Geotechnics Limited with results held in the above report.

8.2.4 Pile Installation

Again, the process proved successful with 150 lime piles constructed in 7 working days. Piles were placed in the positions shown in Figure 8.18 using a very much larger drilling rig than the one used at Fenny Compton. An area of very wet material was encountered, as shown, with water standing in the boreholes almost to ground surface. An additional row of boreholes was placed in this vicinity as shown. It is notable that this very wet region coincides with the position of the buried stream referred to earlier.

Boreholes were drilled to depths of between 3.25 and 3.75 m and took an average of four 50 kg bags of lime per hole. Problems were encountered filling holes that contained standing water: either the holes collapsed and little lime could be placed, or the reaction of the lime with the water sprayed both into the air. In some instances lime was sprayed in a fountain to a height exceeding that of the mast of the drill. Site safety in these instances will become an issue if this technique is to be applied commercially.

Spoil material was once again used for the plug at the top of each pile, but it was ensured that 0.5 m was provided every time by close site supervision. This has been sufficient to prevent vertical expansion.

Due to the drilling operation the ground surface has been left in a very rough condition. The way in which lime is supplied to the holes means that there is some spillage. Rotovating the surface after piling would mean that the surface would be levelled and the excess lime could react with the material *in situ*, creating a 'capping layer'.

8.2.5 Results

Piezometer readings are again the major source of data from this trial. Readings were successfully taken during the period of the trial, although some data from the period immediately afterwards were lost due to battery failure. It was considered possible that some of the readings prior to the trial indicated that the system may not have been fully de-aired since some of the readings represent water levels above ground level. (All readings were expected to be negative with respect to the readout point due to the relative height of the tips in comparison to the readout point. Air in the system would mean that a negative (relative) pore water pressure is not measured to be as negative as when the system is entirely full of water. Hence a possible reason for the seemingly high water table.) However, the evidence for the presence of a covered stream could equally mean that the readings are accurate. Also, when a more rigorous de-airing technique was applied little reduction in consecutive readings occurred, as shown in Figures 8.19 to 8.23. It would appear, therefore, that fears of error were unfounded. For the purposes of analysis the readings shall be taken as accurate, especially when considering changes in pore water pressure.

Piezometers 1 and 2 occupy somewhat remote positions from the piles, their recorded values of pore water pressure being shown on Figures 8.19 and 8.20. Also shown is the day of commencement of drilling and the day of commencement of drilling in the vicinity of that piezometer. It may be seen that, taking the pore water pressure at the commencement of drilling in the vicinity of the pile as a datum, pore water pressure in both piezometers reduce by a maximum of 10 kPa in both cases.

Piezometer 3 occupies a position on the site with relatively dry conditions. Its response is shown in Figure 8.21. Using the same datum for pore water pressure a maximum reduction of 27 kPa is recorded. (It should be noted, however, that such a datum represents a water level some 1.8 m above ground level. If a datum of ground level is taken this represents a reduction in pore water pressure of 10 kPa. Nevertheless a greater reduction in piezometer 3 was expected from the site conditions and -27 kPa is consistent with this thinking.)

Piezometers 4 and 6a and 6b (6a is 2.4 m and 6b 5.6 m below ground level) are positioned in the wetter end of the site. Their responses are shown in Figures 8.22 and 8.23. If a datum for pore water pressure is taken as described above, using the pore water pressure recorded immediately before drilling in the vicinity of the piezometer, then the magnitudes of pore water pressure reduction are 15 kPa, 8 kPa and 17 kPa respectively. (If, again as above, in the cases of 4 and 6b, where the pore water pressures at datum were above ground level, a datum of ground level is taken, then the reductions are 12 and 5 kPa respectively.) The reductions, using both methods of choosing the datum value, are summarised in Table 8.1.

Table 8.1 Summary of pore water pressure changes

Piezometer	Maximum immediate reduction (kPa)	
	Datum 1 before drilling in vicinity of piezometer	Datum 2 Water level @ GL
1	-10	-10*
2	-10	-10*
3	-27	-10
4	-15	-12
6a	-8	-8*
6b	-17	-5

* These piezometers recorded water levels below ground level using Datum 1 so have not been adjusted

It is possible that artesian pore water pressures exist, particularly in the vicinity of the piezometers placed in borehole 6 as they are positioned in the area of the site where the stream was thought to have been buried. The relative magnitudes of reduction calculated using the first method of datum definition is consistent with the relative magnitude of water content of the soil in the vicinity of the piezometer. Piezometer 3 was in the drier part of the site and experiences the greatest reduction in pore water pressure. This value of datum also produces reductions in pore water pressure that are, in general, greater in the vicinity of the piles, and less pronounced in piezometers 1 and 2, positioned furthest away from the piles. Piezometer 4 was positioned in the wetter part of the site and therefore the maximum immediate reduction in pore water pressure of 15 kPa as opposed to the 27 kPa reduction in piezometer 3 could be expected. Piezometer 6 was positioned in the wettest part of the site so the immediate reduction of 8 kPa could also be expected in 6a. Deeper below ground level in 6b, where surface water effects were less influential, the reduction was increased to 17 kPa.

As at Fenny Compton, pore water pressures will continue to be taken in order to build up a clearer picture of steady state conditions.

Inclinometers have shown very little movement either before or after the trial. It seems likely that failure took place immediately after reinstatement with little subsequent movement taking place. Nevertheless, the inclinometers will continue to be monitored on a regular basis.

8.2.6 Conclusions

The process has once again proved successful in terms of installation procedures, the larger equipment producing a rapid production of piles (Figure 8.24). Site safety, however, will have to be addressed in order to allow the process to be commercially viable, especially in wet conditions.

The larger plug has prevented vertical expansion of the pile.

Further pore water pressure data are required to completely validate results. It seems likely that the idea that a stream was covered during reinstatement works was correct.

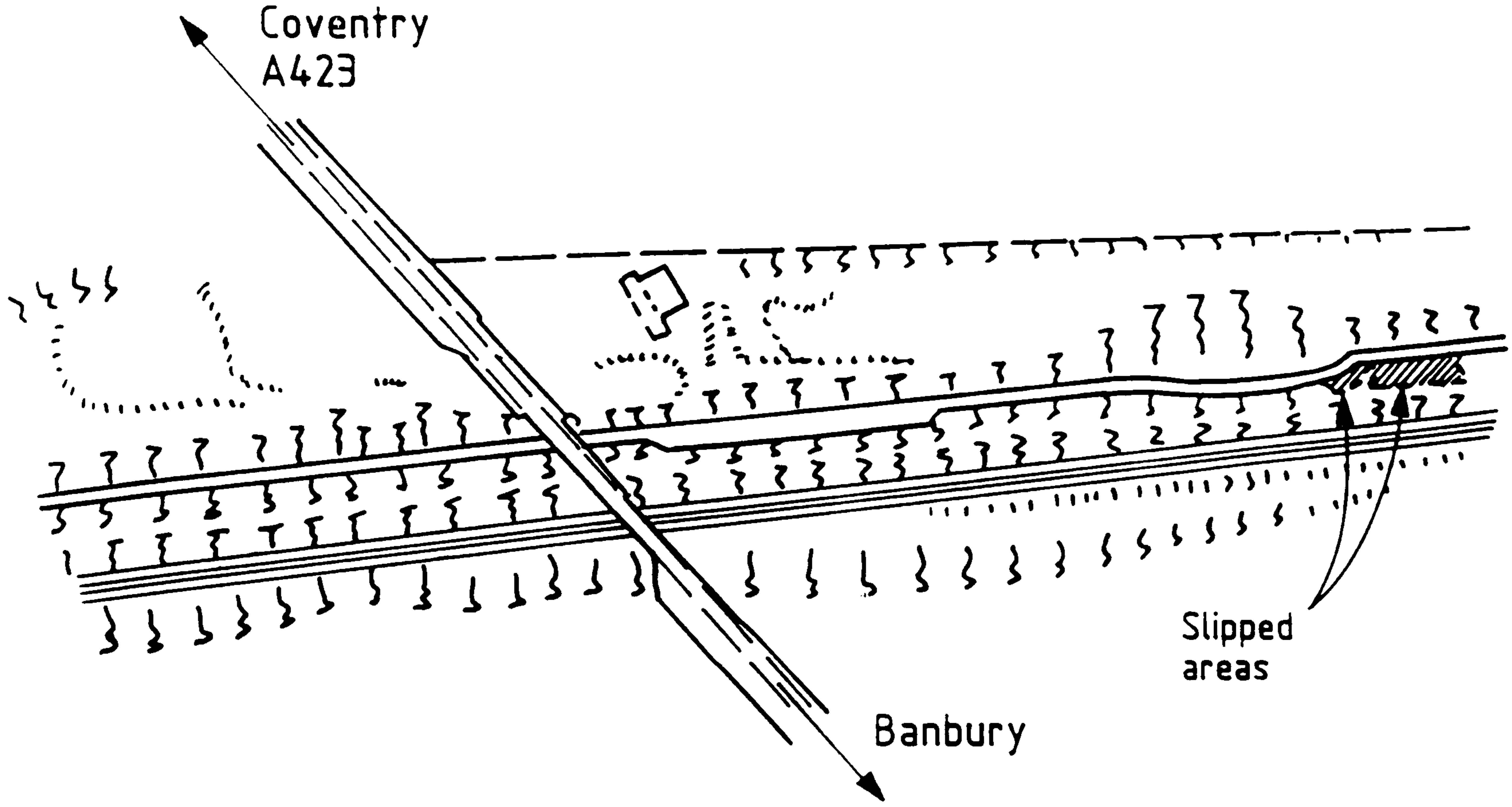
The rotavation of the surface will not only create a level surface but will potentially provide a lime stabilised capping layer.

8.3 CONCLUSIONS

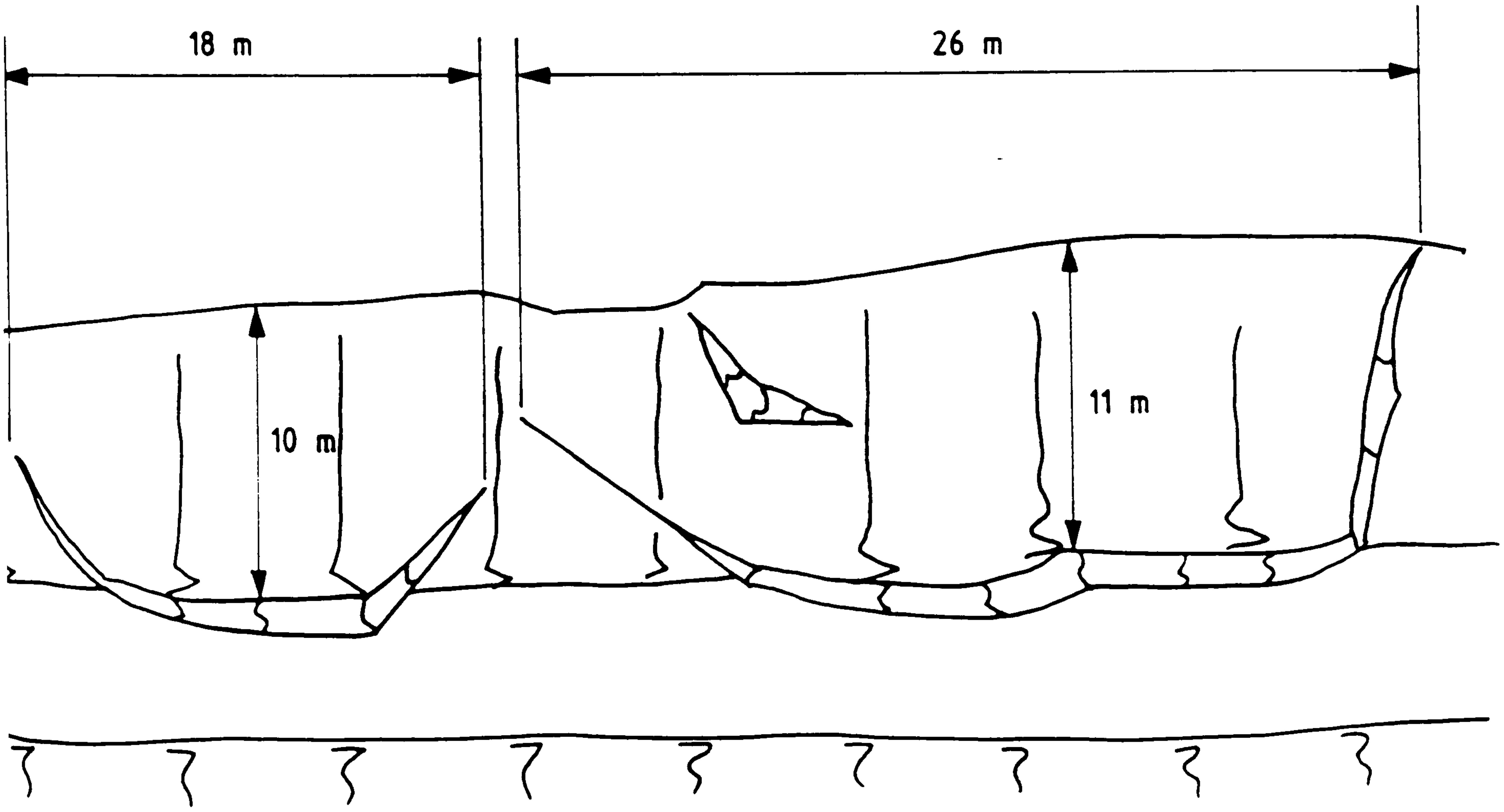
In addition to the conclusions drawn from the individual trials, several major points may be drawn. Of particular note is the fact that this process may be carried out with small plant and two operators to produce a relatively rapid result. The second trial at Fenny Compton, although unsuccessful in terms of gathering definitive pore water pressure data, has led to this conclusion. As previously stated, this means that the technique may be of particular benefit in treating sites with access problems.

When used commercially care must be taken in several areas, including safety in handling lime, lime compaction and the provision of an adequate plug.

The aim of the field trials was to gain practical experience in the design and installation of lime piles and gain some information about their effect on a failing slope. In this respect it is felt that it has been achieved. At some future date, it will be beneficial to gain additional data through the excavation of specific areas of piles to look at such phenomena as lime movement, cracking, pile expansion, pile strength properties, soil water content and properties of the soil in the migration zone. This will allow a full assessment of the effect of site conditions in general and in site-specific terms. It will undoubtedly enhance the knowledge gained concerning stabilising mechanisms and allow a greater reliance on the existing data.



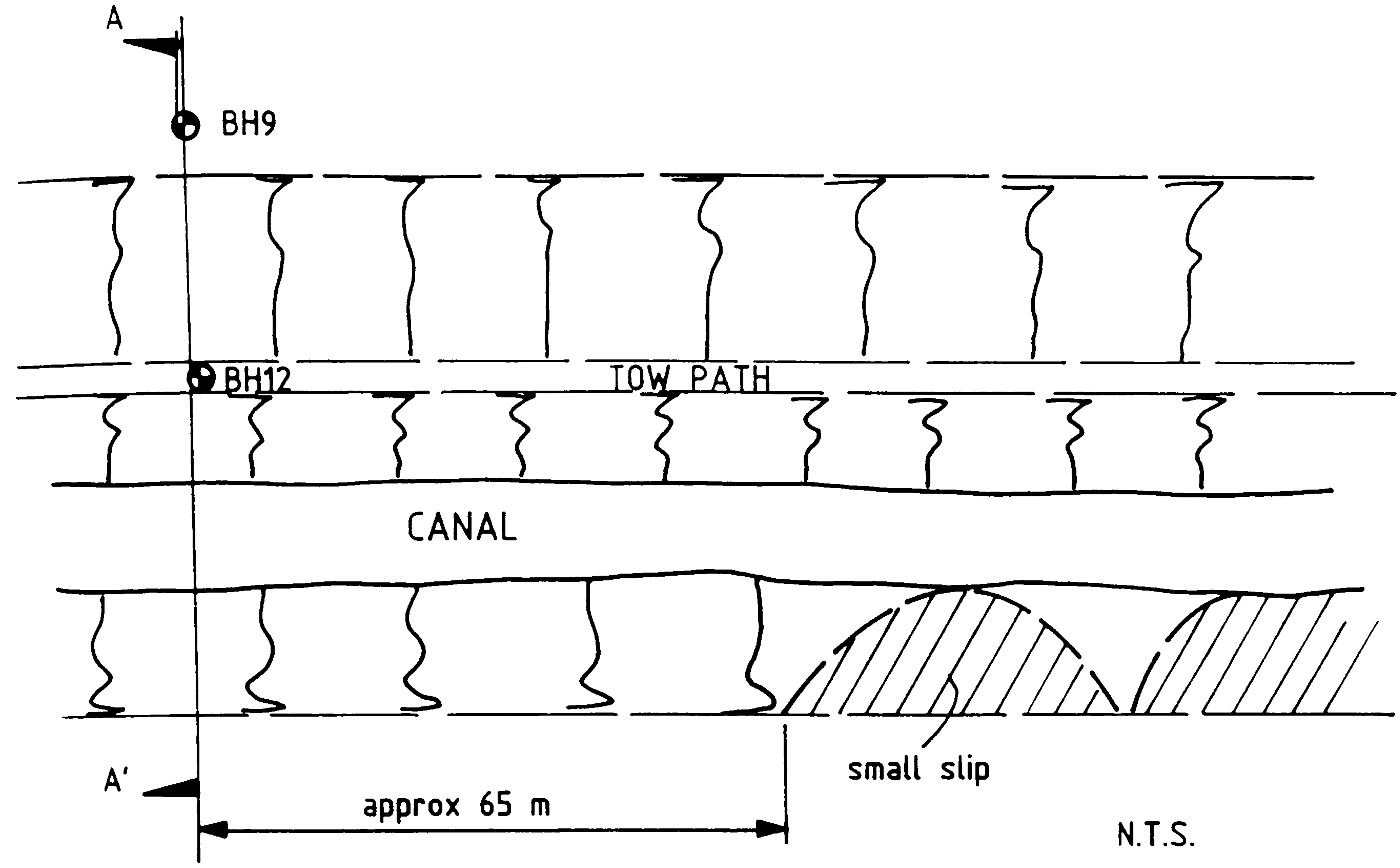
Location Plan - NTS



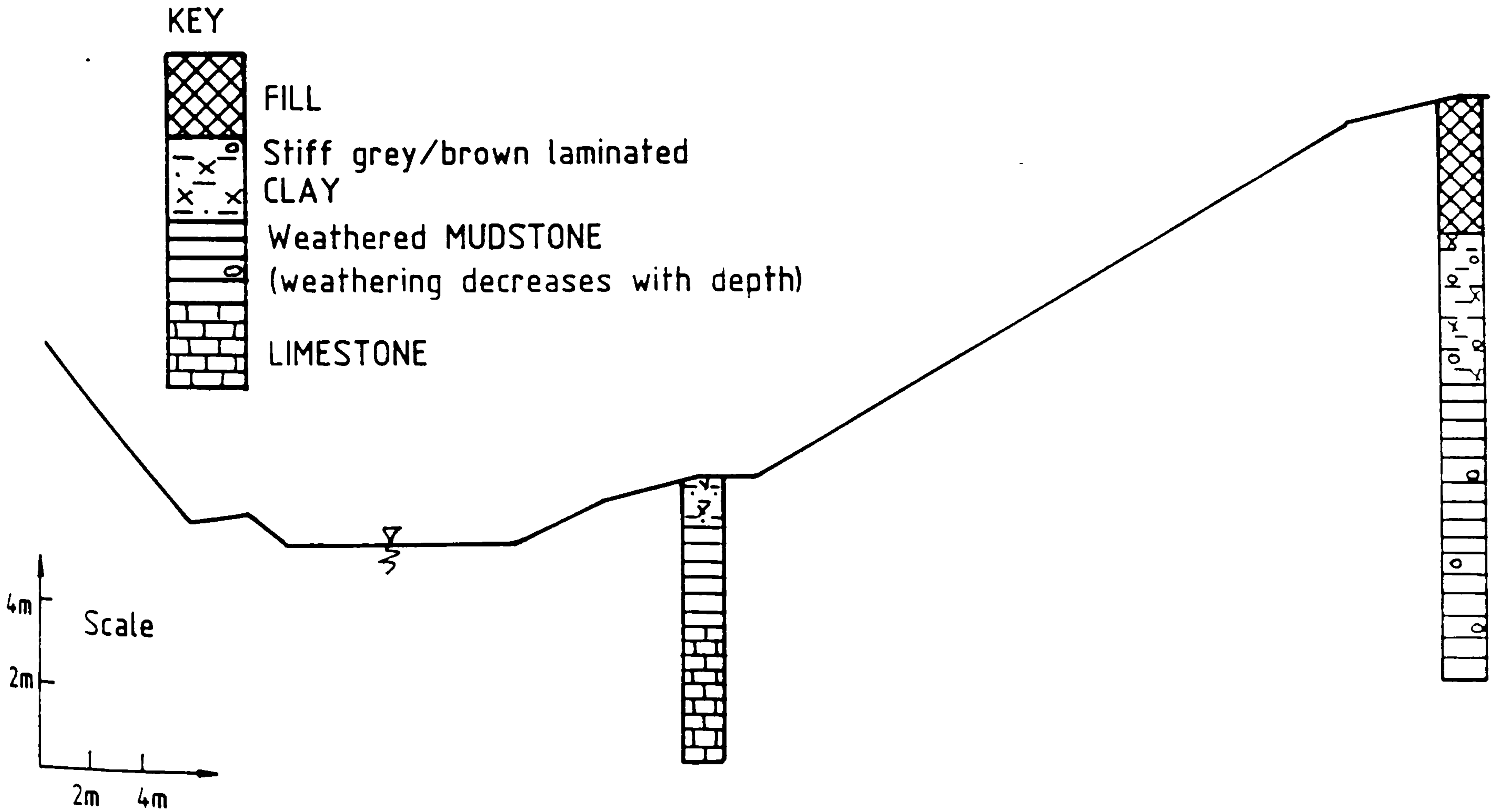
Slip Areas - NTS

Figure 8.1 Fenny Compton Location Plan

Figure 8.2 Fenny Compton - 'Large' and 'Small' slips



SKETCH SECTION LOCATION

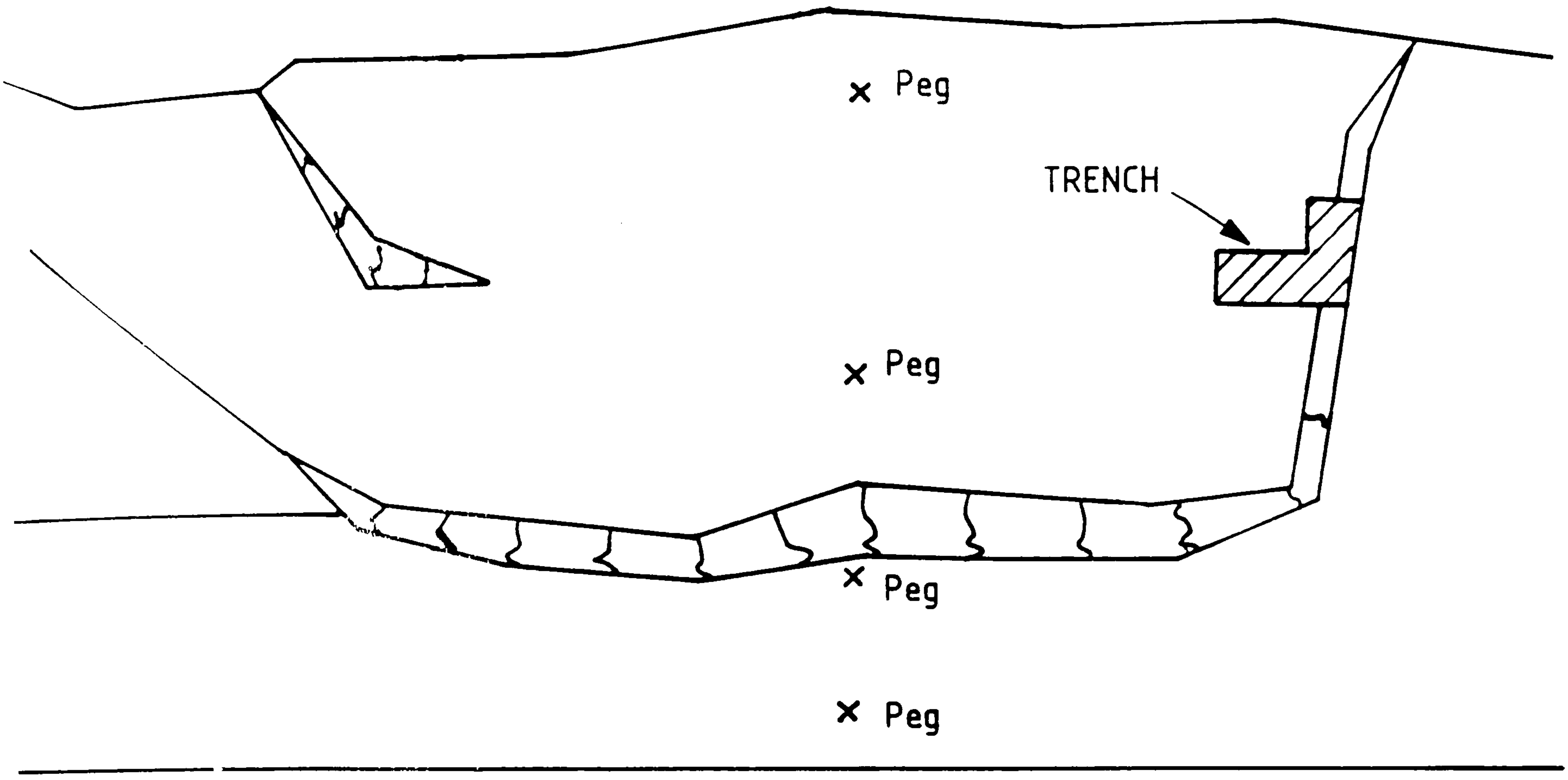


SECTION A-A

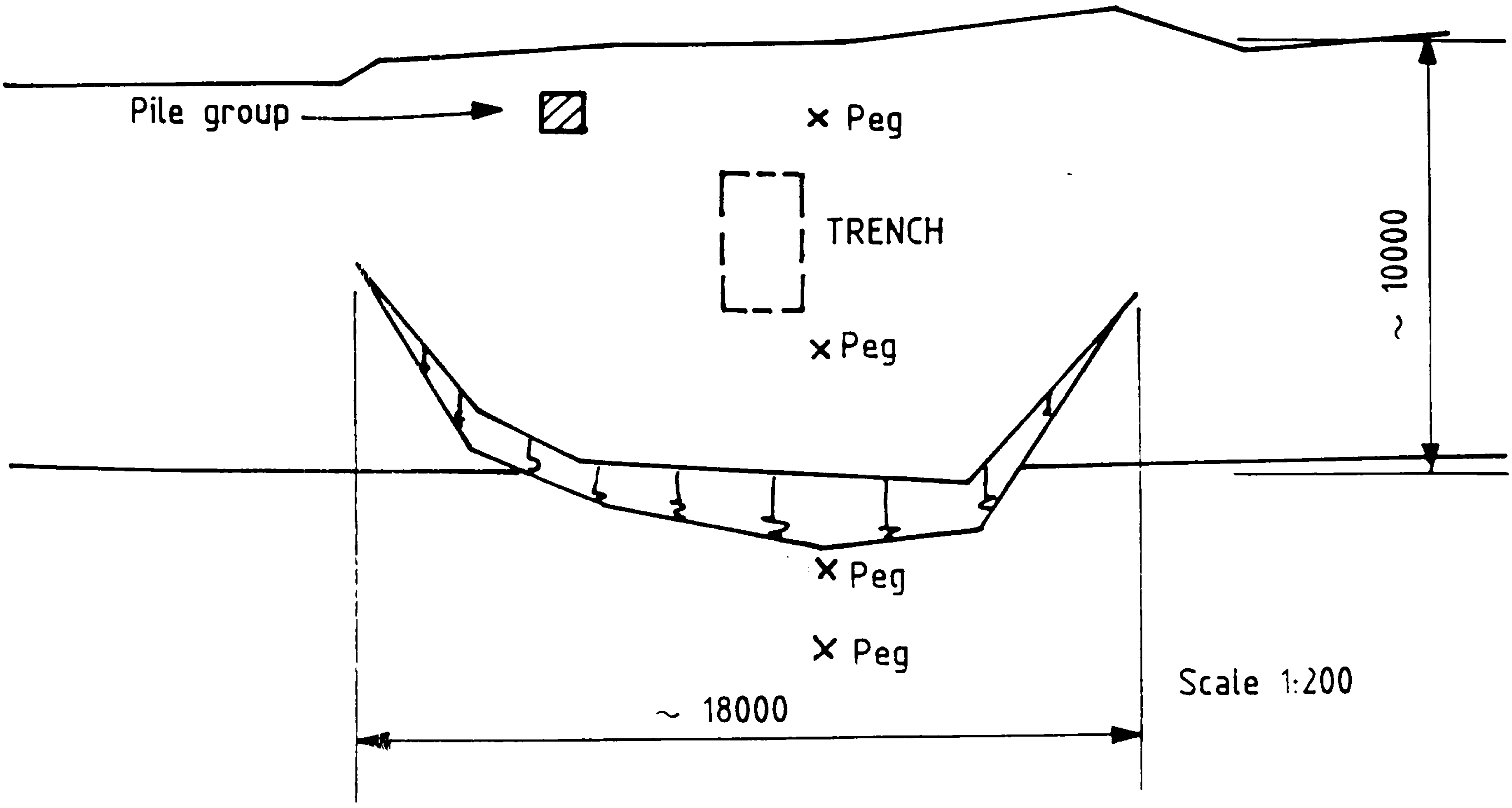
NOTE:

Section drawn from:
"Site Investigation of Fenny Compton Cutting on the South Oxford Canal for British Waterways Board" (1987)

Figure 8.3 Fenny Compton - Section through canal banks



PLAN - Large Slip Scale 1:200



All dimensions in mm

PLAN - small slip

Figure 8.4 Fenny Compton - Site investigation layout

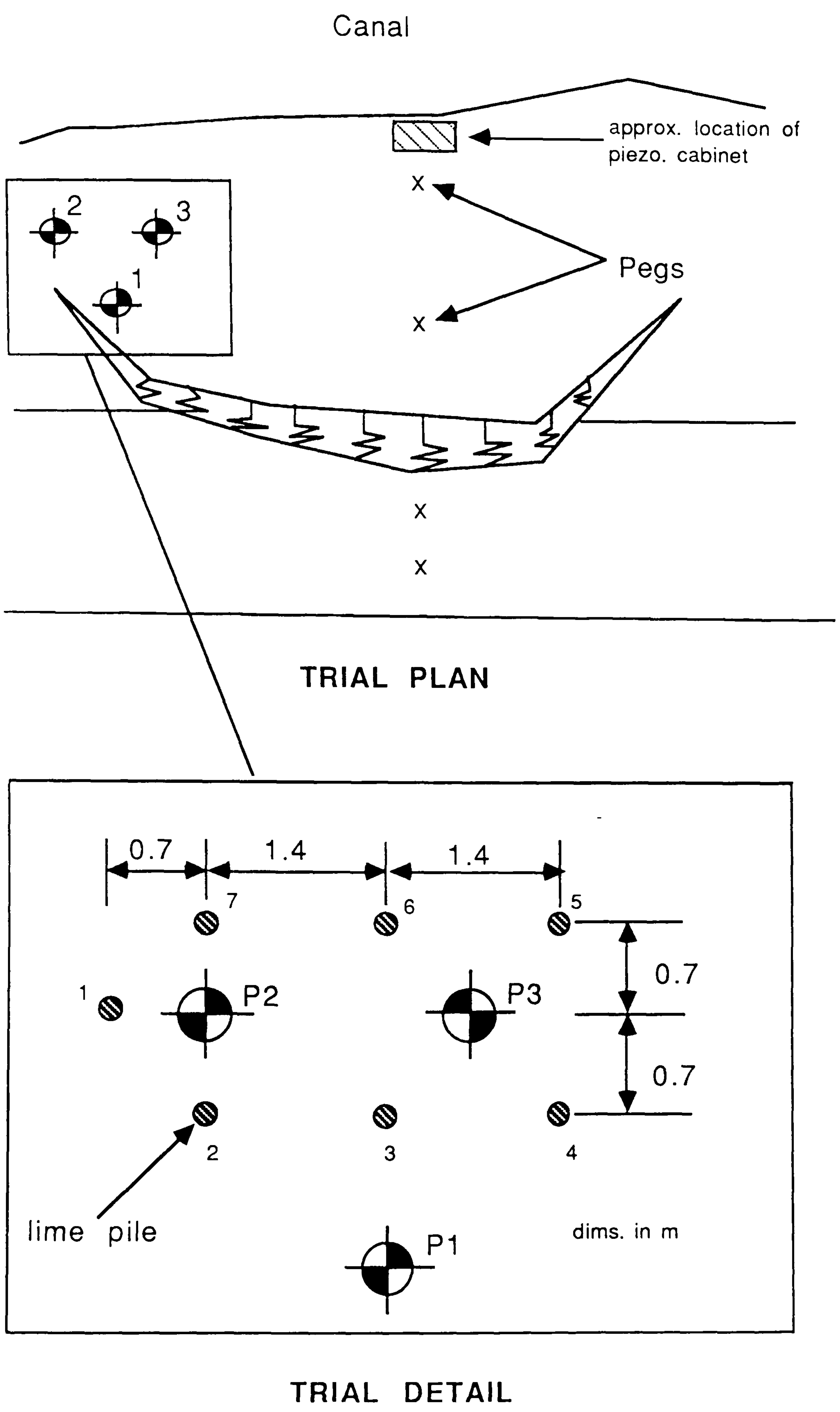


Figure 8.5 Fenny Compton field trial plan

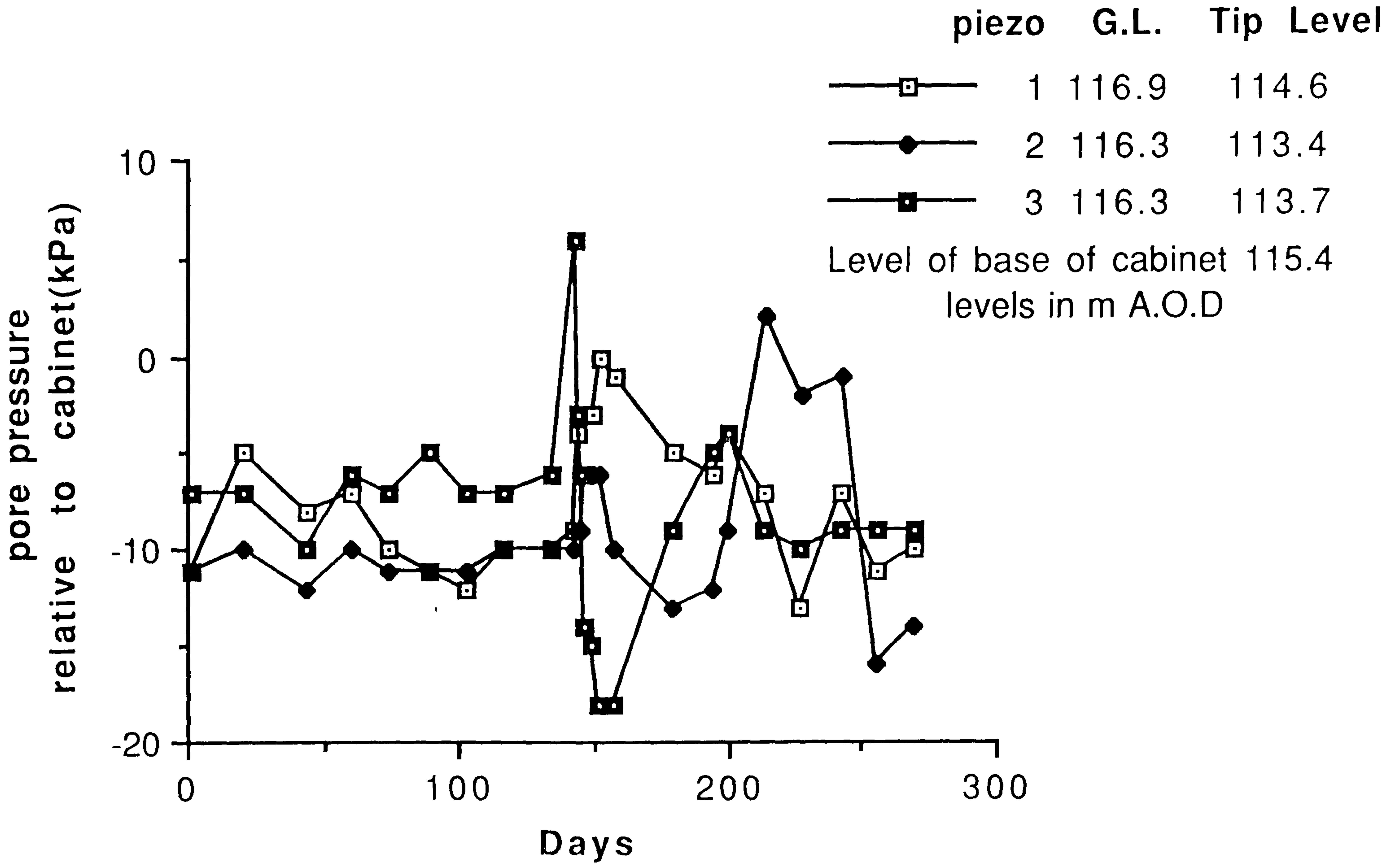


Figure 8.6 Fenny Compton piezometer readings

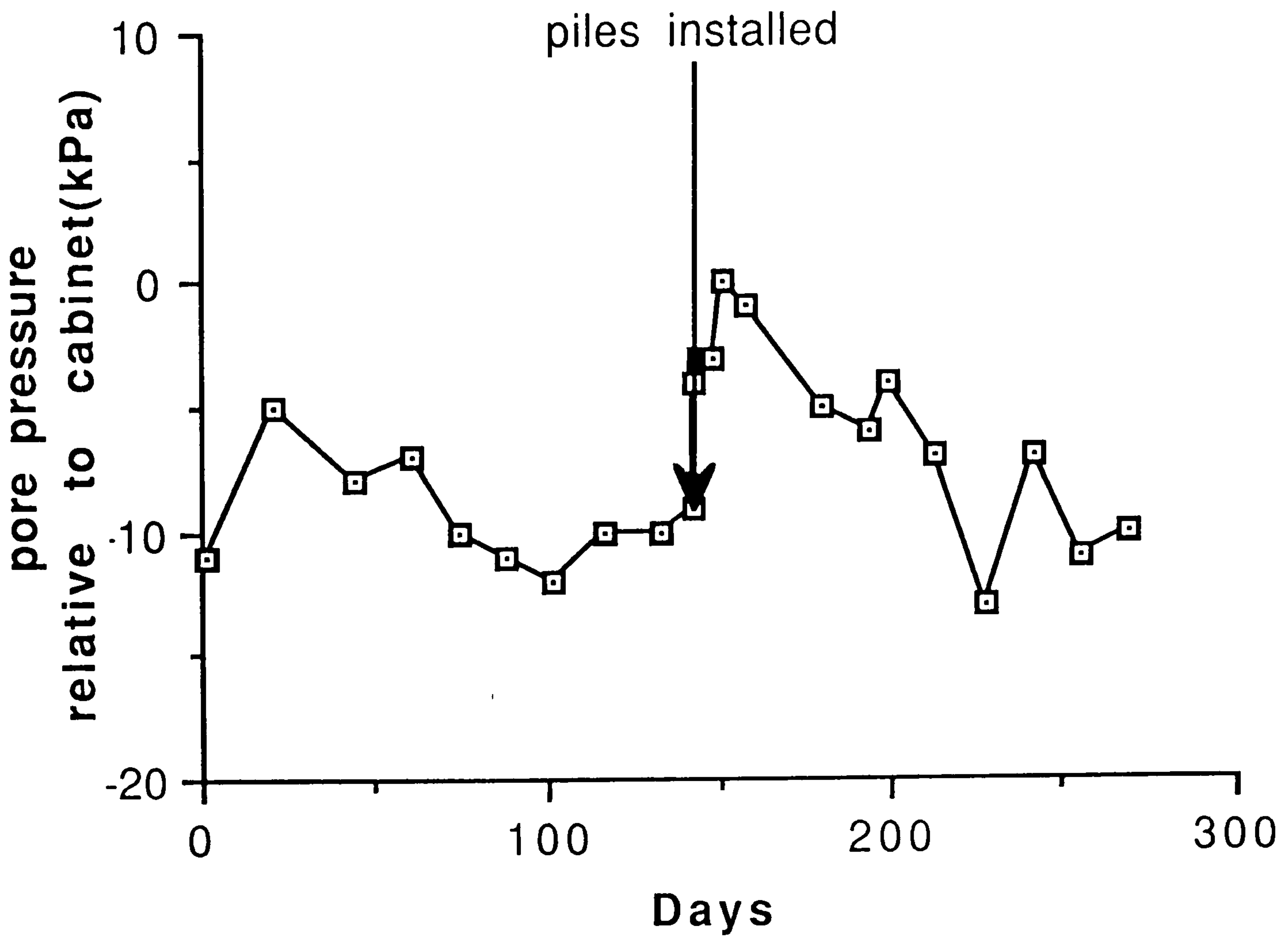


Figure 8.7 Fenny Compton readings from piezometer 1

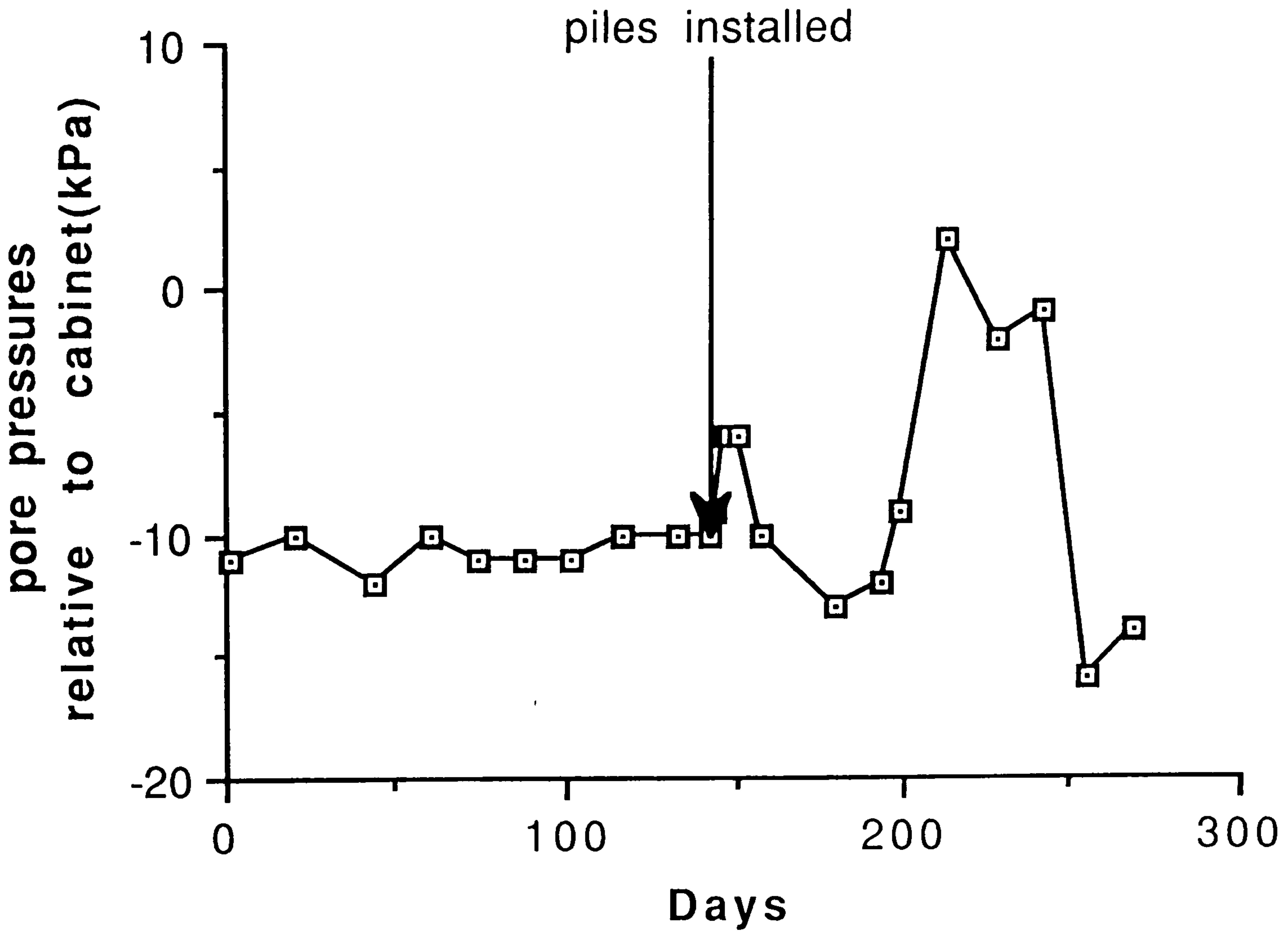


Figure 8.8 Fenny Compton readings from piezometer 2

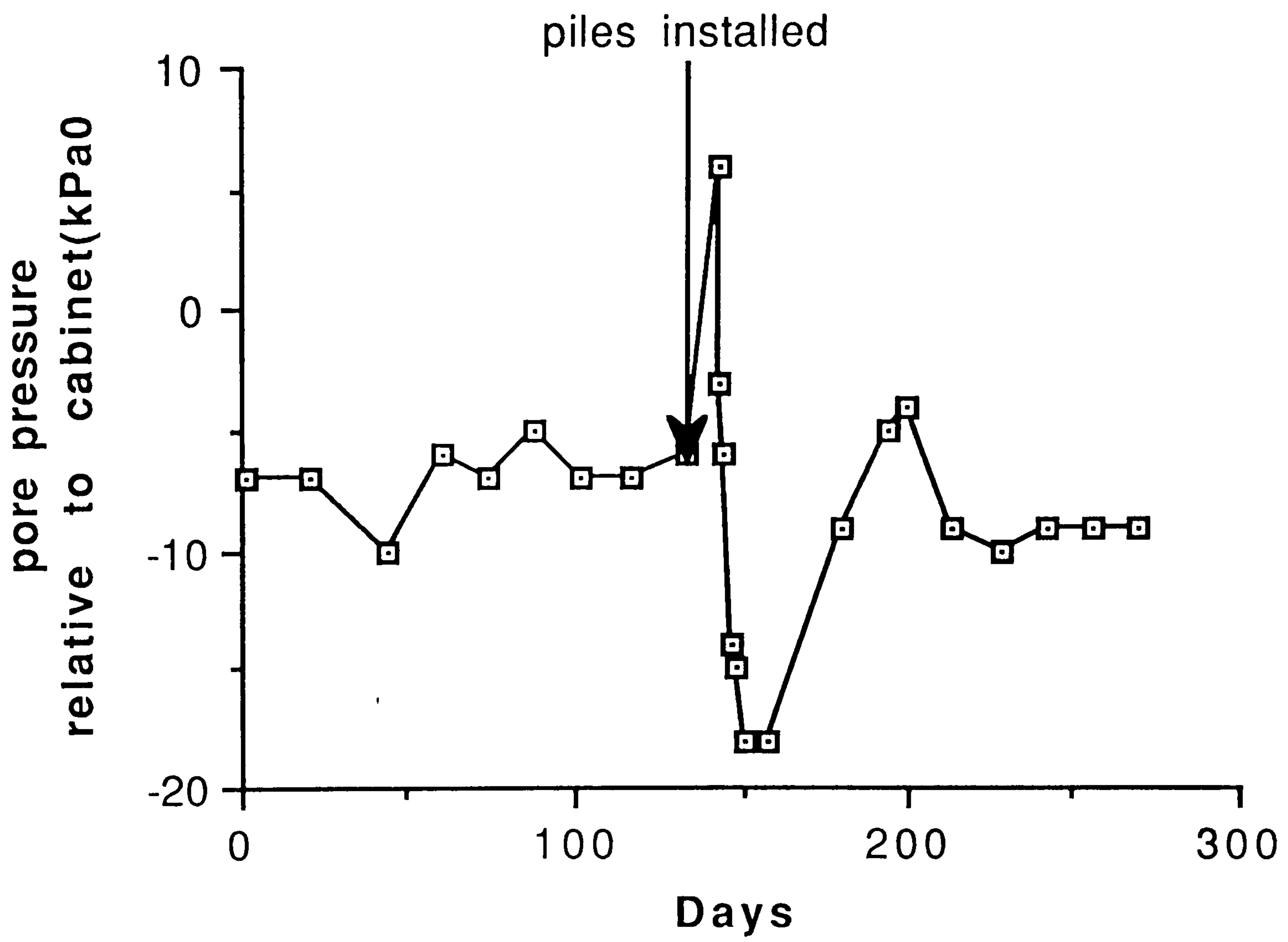


Figure 8.9 Fenny Compton readings from piezometer 3

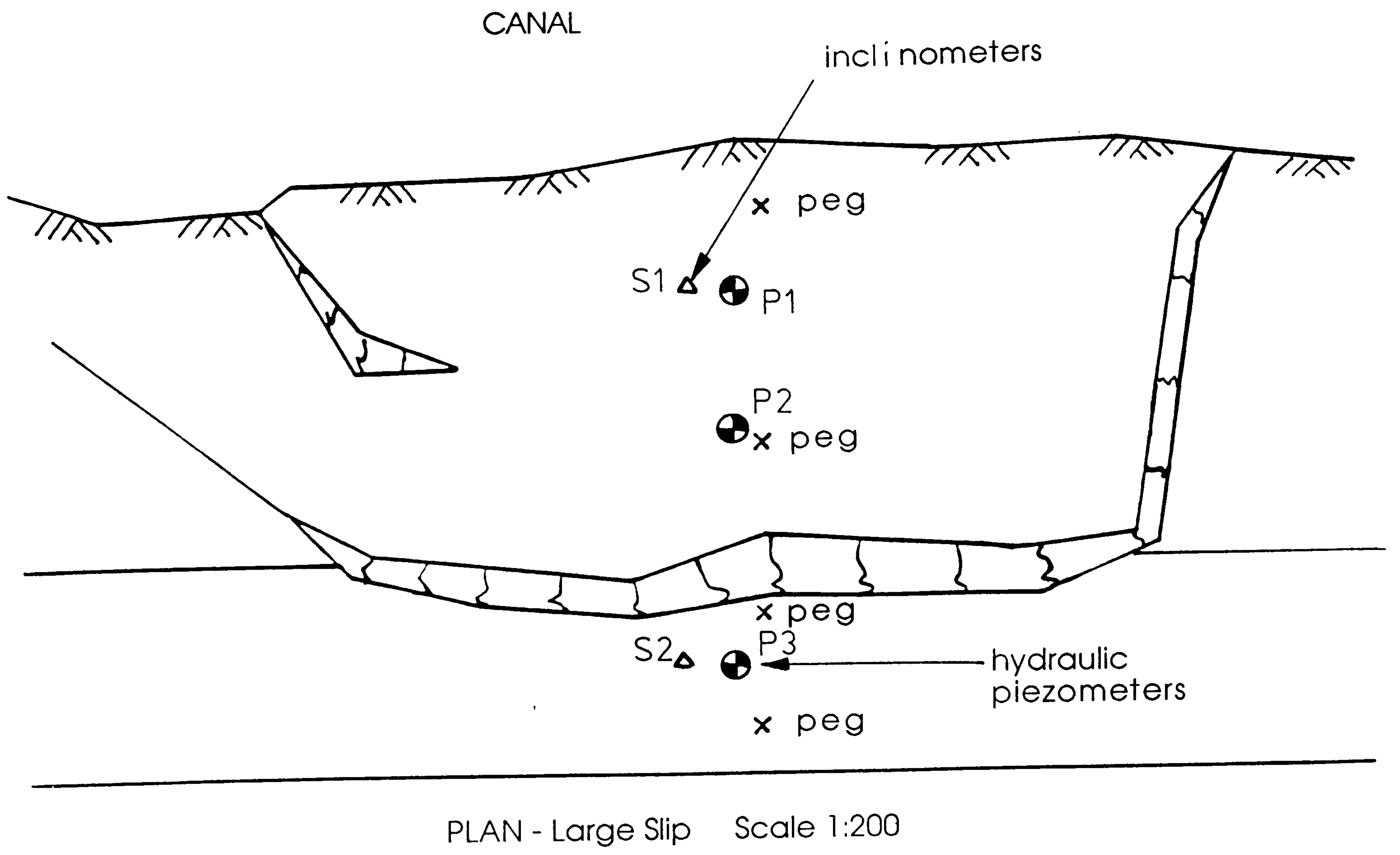


Figure 8.10 Fenny Compton large slip monitoring locations

Figure 8.11 Setting out rows of lime piles at Fenny Compton

Figure 8.12 Pile installation using a 'Minute-Man' rig

Figure 8.13 The lime filling and

Figure 8.14 Vertical lime pile expansion

Figure 8.15 Iver site location plan

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Figure 8.16 Iver - location of tension cracks

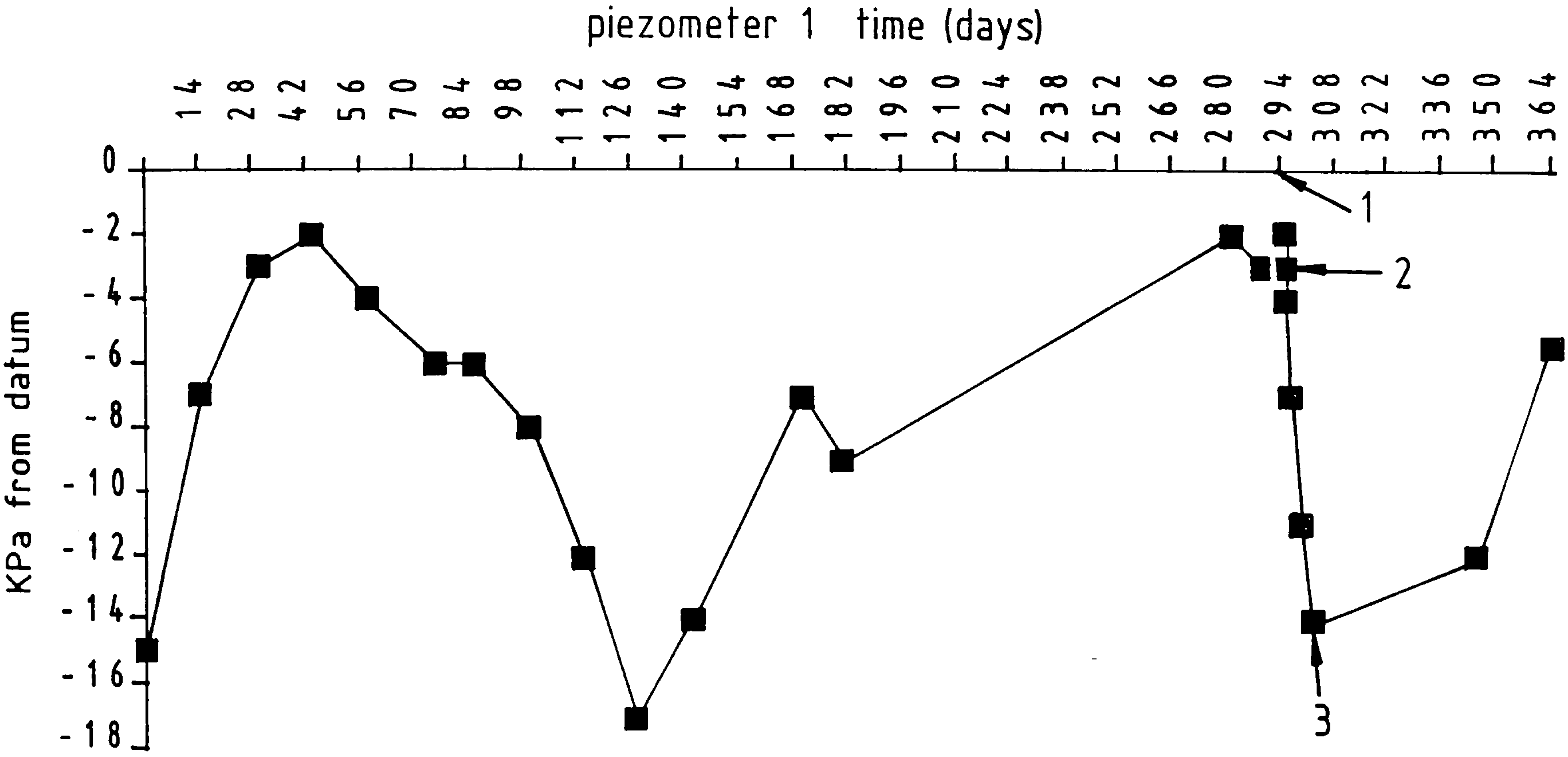
Figure 8.17 Iver - site investigation location plan

Notes A double ⊕ indicates a piezometer and inclinometer location

Dates relate to te particular piles drilled in one day

⊙ indicates holes containing standing water

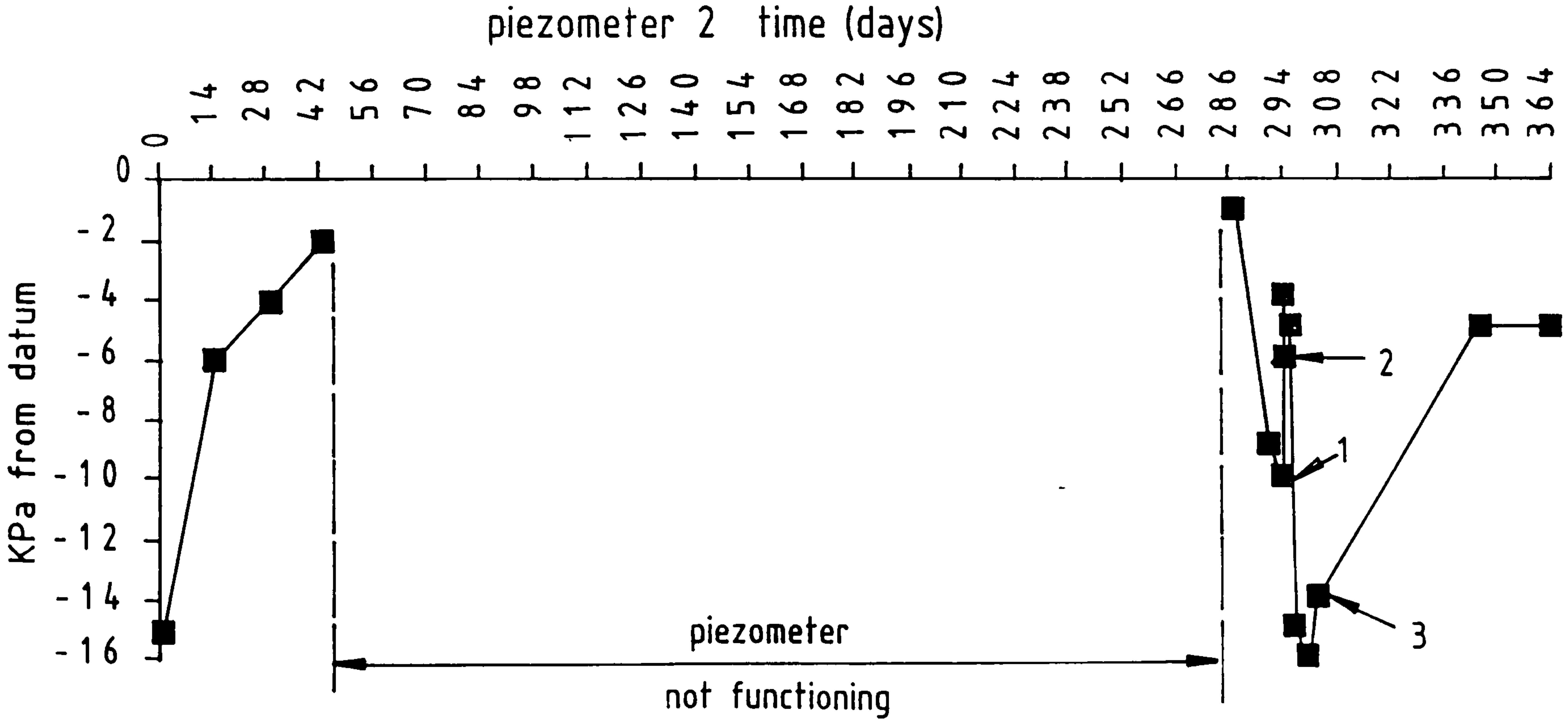
Figure 8.18 Iver - pile location plan



Notes

1) Drilling begun
2) Drilling begun in region of piezo
3) 1st de-airing

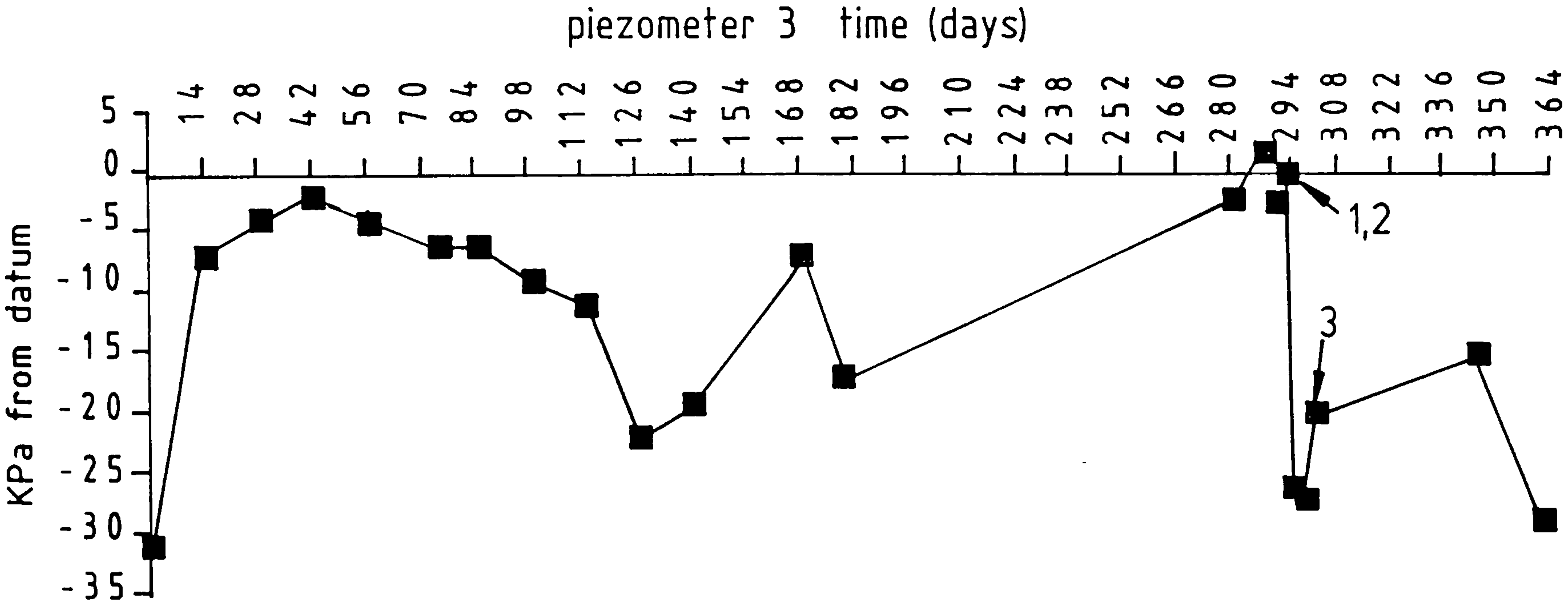
Figure 8.19 Iver - readings from piezometer 1



Notes

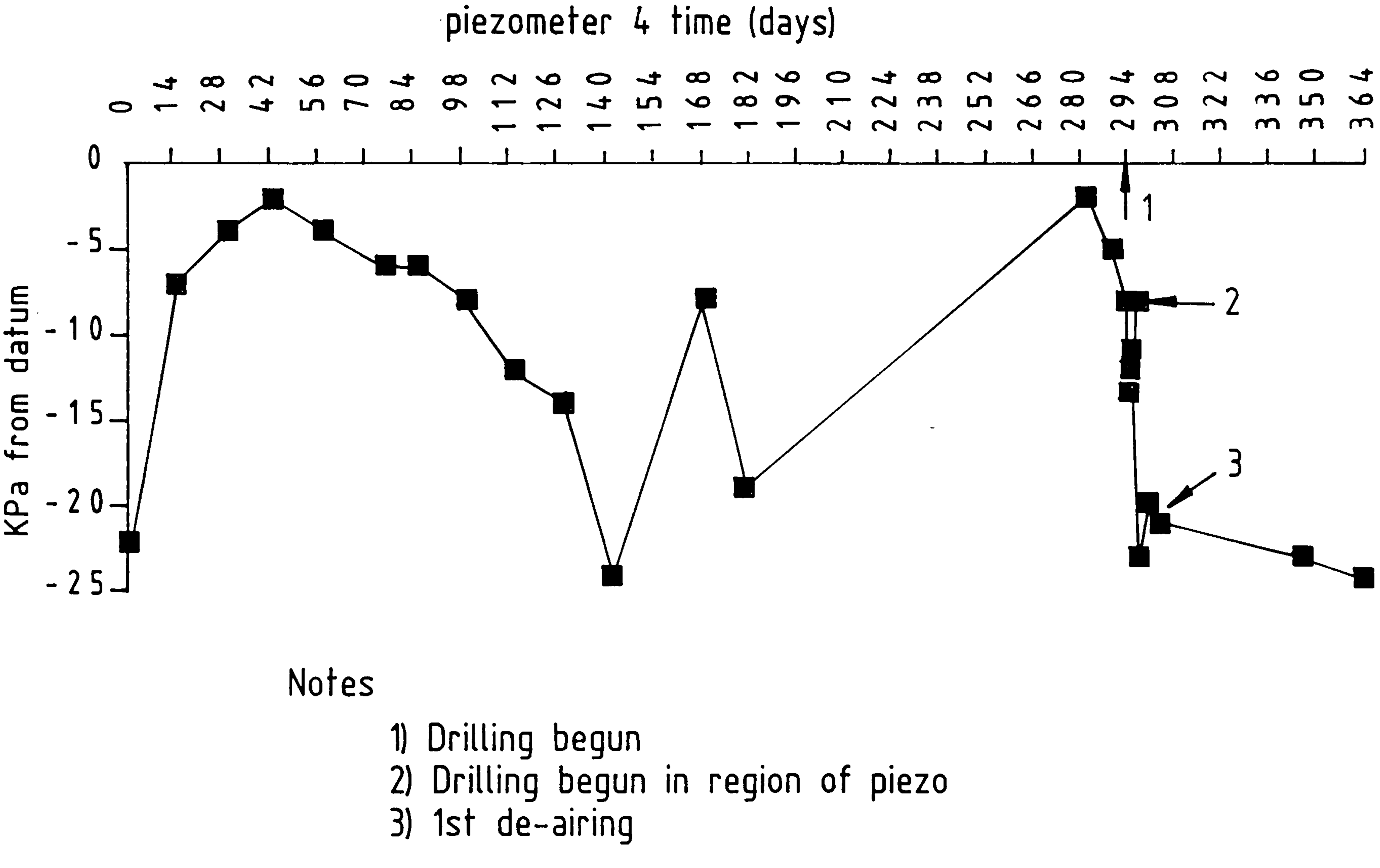
1) Drilling begun
2) Drilling begun in region of piezo
3) 1st de-airing

Figure 8.20 Iver - readings from piezometer 2



Notes
1) Drilling begun
2) Drilling begun in region of piezo
3) 1st de-airing

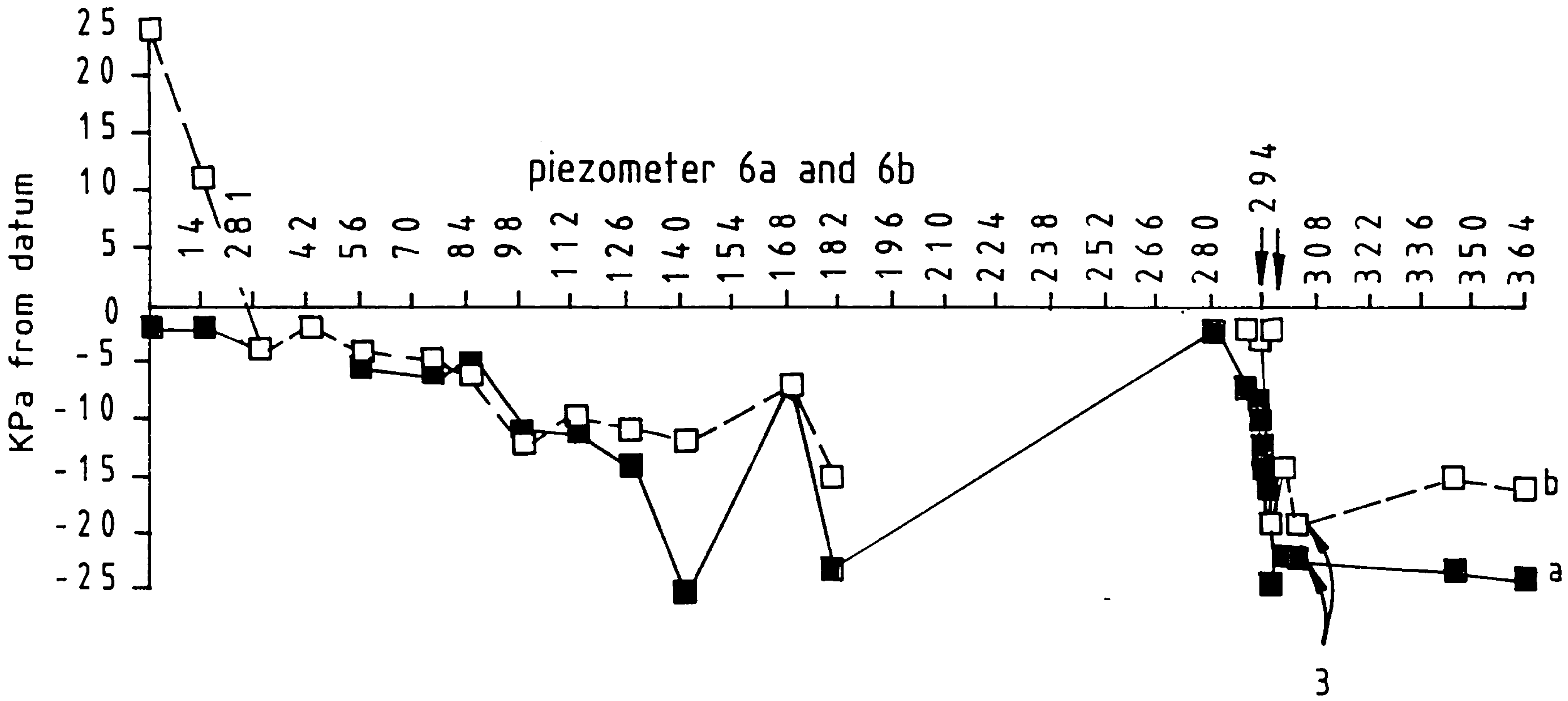
Figure 8.21 Iver - readings from piezometer 3



Notes

1) Drilling begun
2) Drilling begun in region of piezo
3) 1st de-airing

Figure 8.22 Iver - readings from piezometer 4



Notes

1) Drilling begun
2) Drilling begun in region of piezo
3) 1st de-airing

6a 2.4m bgl
6b 5.6m bgl

Key

□ — b
■ — a

Figure 8.23 Iver - readings from piezometer 6

Figure 8.24 Pile installation at Iver

CHAPTER 9

9.0 CONCLUDING DISCUSSION

9.1 REACTION MECHANISMS

A comprehensive review of the relevant literature was undertaken, almost certainly providing the first major review of the literature pertinent to the subject of lime piles. The literature has provided much evidence of the success of lime piles in treating both soft ground to improve its bearing capacity and slopes to improve their stability. The mechanisms of stabilisation postulated by authors from several different countries were in some cases incorrect, and a significant proportion of the literature on the subject was contradictory or misleading. Thus the need for a comprehensive programme of research was clearly established.

It was clear from the literature that many possible stabilising mechanisms exist. These were investigated in a comprehensive laboratory programme, consisting of both model lime pile tests and small-scale element testing. The results from this programme enabled the list of stabilising mechanisms to be refined and each component verified and, in most cases, quantified. A brief discussion of the stabilising mechanisms follows.

9.1.1. Lateral Consolidation

Lateral consolidation was only claimed in the literature in the case of quicklime piles. Theory suggested that as the quicklime draws in water from the surrounding ground it reacts, or slakes, to form hydrated lime. Slaked lime has a lower density than quicklime and hence expansion occurs. This expansion was said to cause lateral consolidation of the ground surrounding the pile. However considerations of stoichiometry of the hydration reaction and absorption, by the pile, of water into its void space, indicated that there was a small net volume loss from the soil and thus the idea of physical consolidation cannot be valid. Measurements of piles used in the laboratory model tests indicate that the physical expansion of the pile does not occur to the extent predicted from the relative densities of quick and hydrated lime. Radial cracking around piles also suggests a volume loss from the soil-water medium. This suggests that the resistance of the soil, degree of confinement and available water may have an influence on the degree of expansion possible. This is an area requiring further specific laboratory investigation.

In the case of very soft materials the perceived expansion may have been attributable to the displacement method of forming the cavity and compaction caused by lime placement under high pressure.

9.1.2 Water Content Reduction

The slaking reaction "uses up" some water from the soil surrounding the piles, and, being highly exothermic, it also produces a significant amount of heat. These two processes in combination were thought by some authors to cause a significant water content reduction. The volume of water used up in the slaking reaction alone has been calculated and shown to be very small in comparison with the volume of water held in the pore space between soil particles. A more significant amount of water is lost from the *in situ* soil as the pile absorbs water into its pore space. The amount of water lost is therefore proportional to the water content of the surrounding soil. Large piles and a source of rapidly available water (as in soils with a high permeability and/or a high water content) will create the conditions for significant water loss from the soil. The large box models have shown the radius of influence to be up to 150 mm, although these results are derived from piles placed in material of relatively high water content and permeability. The presence of cracking could present a problem, especially in dry conditions where there is insufficient water to satisfy the slaking reaction and the permeability of the material prevents water from entering from a significant distance from the pile. The presence of clay is not necessary for this stabilising mechanism to operate, and since it would reduce the permeability of the soil it would prove a hindrance to stabilisation by this mechanism.

9.1.3 Clay-Lime Reaction

It was widely reported that lime migrates from the piles and reacts with, and hence stabilises, the ground surrounding the lime piles. Laboratory experiments have shown that the migration of both calcium and hydroxyl ions in conjunction in clays is restricted to a relatively small distance, as would be expected from consideration of the operation of clay barriers. 20 to 30 mm is the limit on the distance of ion migration as predicted from the laboratory experiments. A recent literature survey in the field of contaminant migration from land-fill sites has indicated that, whilst the hydroxyl ions are restricted to distances of this order, calcium ions, over a period of more that ten years, may migrate up to 2 m by diffusion alone. This means that, whilst stabilisation is restricted to a 20-30 mm annular zone surrounding the pile, modification may occur over the entire treated area using piles placed at sufficiently close spacings. This idea is one that is also under investigation in the laboratory.

Consideration of mineralogical changes has indicated a strength improvement in the stabilised zone equivalent to the strength achieved by mixing quicklime with clay at a percentage equivalent to the initial lime consumption value and curing for 28 days.

In some cases in the literature lime slurry has been used to create the piles and migration over significant distances is reported, presumably as a result of hydraulic transport. 'Watering' of quicklime piles has also been suggested in order to improve hydraulic transport of ions. This aspect of this mechanism is also an area requiring further laboratory experimentation, although it must be recognised that this technique will negate any benefit gained from the above dehydration mechanism.

These mechanisms form the primary sources of stabilisation according to the literature, although other potential stabilisation mechanisms derived from considerations of the mechanics of the process. These are summarised in the following paragraphs.

9.1.4 Reduction in Pore Water Pressure

The addition of quicklime to a soil results in negative pore water pressures which draw in water to the piles to react. In soft, wet and relatively permeable soils the pore water pressures will be high only in the immediate vicinity of the pile and for a short time. This is the case with the laboratory experiments where reductions of the order of 20 kPa were reported. In drier clays of low permeability, however, the suctions will be far higher if water continuity is maintained and will last far longer. This is an aspect yet to be studied in detail and would benefit the design of field installations.

9.1.5 Consolidation of the Shear Zone.

Since the suctions described above increase the mean normal effective stress, the soil will become stronger and more stable, particularly in the remoulded region of the shear zone where significant consolidation would be expected to occur. Experiments have investigated this phenomenon for two different types of clay using the direct shear box. Further experimentation on different types of clays would be beneficial. Also, the method of experimentation did not easily allow the study of the effect of overconsolidation on a shear zone, rather than discrete shear plane. If a method of sample preparation could be developed a series of experiments of this nature would add valuable data.

The effect of additives to the quicklime, such as PFA, on pile strength is also under investigation. The enhanced benefit in terms of strength is being compared to the effect upon pore water pressure reduction, water content reduction and ion migration.

9.1.6 Strength of the Pile Material

A third benefit, alluded to by one author, is the strength of the piles themselves supplying shear resistance along a shear plane in failing slopes. This would be beneficial as long as pile durability and strain compatibility, both during strength gain and after the pile has fully strengthened, are accounted for. Analysis carried out for the design of the field trials indicated the importance of the strength of the lime for the long-term stability of the treated slope. A thorough investigation of the strength of slaked lime (and the factors controlling that strength) is therefore necessary in order for the strength parameters used to be relied upon. This is an area currently under investigation in the laboratory.

9.2 FIELD TRIALS

A computer based design method has been developed by a concurrent MPhil project and has been successfully used to design two full-scale field trials. Their successful completion has highlighted the different methods of installation available for this technique. Perhaps the most important finding in this respect is the suitability of the relatively small 'Minuteman' rig for lime pile construction, whilst still maintaining a reasonable installation rate. This has implications for the use of the method for what would otherwise be relatively inaccessible sites, such as those adjacent to canals like Fenny Compton and sites adjacent to other (particularly busy) transportation routes. This may mean considerable cost savings in relation to the time spent in the closure of those routes if this technique were to be adopted instead of a more traditional method. Environmentally sensitive sites could also be treated as the small-scale plant has a minimal disruptive effect on the vegetation covering a site.

It is felt that it would be beneficial to excavate areas of both trials in order to investigate the physical effect that the piles have had on the soil surrounding them. It is also felt that rigorous monitoring of any future installations would also add to the data available on pore water pressure changes associated with lime piles placed in field conditions.

9.3 FURTHER STUDY

Overall the project has proved very successful. Despite the areas highlighted for future study it is felt that a thorough investigation of the stabilising mechanisms has been made with additional data being supplied by field trials. Publications derived from the research are Rogers and Glendinning (1993) and Rogers and Glendinning (1994). An area considered worthy of detailed research is the second application for lime piles discovered in the literature.

It has been shown that lime piles have been used in the treatment of soft soils. This is an area not explored in the context of British soils. A similar study to the one above would provide data on the mechanisms operating in this particular type of soil condition. Again full-scale field trials would be necessary to practice the technique. It is seen as suitable for such applications as the treatment *in situ* of leaking canal bases, inadequate highway subgrades and, possibly, tailings lagoons. It is hoped that such a project will supply valuable additional information to the field of ground improvement.

The aim of the field trials was to gain practical experience in the design and installation of lime piles and gain some information about their effect on a failing slope. In this respect it is felt that it has been achieved. At some future date, it will be beneficial to gain additional data through the excavation of specific areas of piles to look at such phenomena as lime movement, cracking, pile expansion, pile strength properties, soil water content and properties of the soil in the migration zone. This will allow a full assessment of the effect of site conditions in general and in site-specific terms. It will undoubtedly enhance the knowledge gained concerning stabilising mechanisms and allow a greater reliance on the existing data.

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APPENDIX 1

QUICKLIME SAFETY DATA

BUXTON LIME INDUSTRIES

CHEMICAL SAFETY DATA SHEET

1. IDENTIFICATION OF THE SUBSTANCE AND COMPANY

Product name:	Quicklime
Other names:	Burnt lime
CAS No:	1305-78-8

Address/Tel:
Buxton Lime Industries
Tunstead Quarry
Wormhill
Buxton
Derbyshire SK17 8TG

0298 768444

In an emergency: Dial 999

For specialist advice
(transport emergency) 0298 27500

2. COMPOSITION/INFORMATION ON INGREDIENTS

Calcium Oxide CaO > 90%. Small quantities of calcium carbonate, magnesia and trace elements.

Hazardous Ingredient - calcium oxide.

3. HAZARD IDENTIFICATION

Irritating to eyes and skin. Risk of serious damage to eyes. May cause burns in the presence of moisture.

4. FIRST AID MEASURES

Skin Contact - An irritant; may cause burns in presence of moisture. Remove contaminated clothing. Wash immediately with plenty of water.
Eye Contact - Causes very painful irritation and may cause serious damage to eyes unless immediate treatment is given. SPEED IS ESSENTIAL. Remove particles with cotton wool bud, irrigate with eyewash or clean water for at least 10 minutes. Obtain medical attention immediately.
Inhalation - Irritating to the respiratory tract. May cause inflammation of respiratory tract. Remove from exposure and keep warm and at rest. Irrigate nose and throat with water for at least 20 minutes.
Ingestion - Causes corrosion and damage to gastrointestinal tract. Do not induce vomiting, wash out mouth with water and give copious quantities of water to drink.

Further medical treatment

Symptomatic, if necessary. No known delayed effects. Prolonged or repeated contact with skin may result in more severe irritation or dermatitis. Prolonged repeated inhalation of high dust concentrations may cause ulceration and perforation of the nasal septum and pneumonitis.

It is advisable to ensure that eyewash facilities are readily available where Quicklime may be handled.

5. FIRE FIGHTING MEASURES

Non combustible and inhibits the spread of flame. No special fire fighting procedure or explosion hazard is identified. Substance reacts violently with water and generates heat. Risk of igniting combustible materials when wetted.

6. ACCIDENTAL RELEASE MEASURES

Spillages. Contain spillage and keep dry if possible. Use vacuum suction unit, or shovel into bags (using appropriate protective clothing - see Section 8). Cover or enclose area if possible to avoid unnecessary dust hazard. Avoid contamination of drains and watercourses. Spillage into watercourses must be alerted to the National Rivers Authority or other appropriate regulatory body.

7. HANDLING AND STORAGE

7.1. Handling. Avoid contact with skin and eyes. Avoid inhalation of high concentration of dust.

7.2 Storage. Should be stored in a cool dry environment free from draughts. Bulk storage should be in a purpose-built silo. Product in bags should be stored in draught-free brick or concrete building. Quicklime must not be allowed to come into contact with water as it generates intense heat, nor should it be stored on a flammable structure or with flammable materials.

8. EXPOSURE CONTROL/PERSONAL PROTECTION

8.1 Wear suitable gloves, overalls and eye/face protection. Wear suitable respiratory protection equipment if exposure to atmospheric dust levels above the occupational exposure limit is likely

8.2 Occupational Exposure Limit:
$2\ mg/m^3$ (8 hr TWA)

8.3 Handling systems should preferably be enclosed, or suitable ventilation installed to maintain atmospheric dust below OEL.

8.4 Rubber, leather or fabric/composite gloves provide suitable hand protection. Long sleeved overalls, close fitting at openings. Wide vision full goggles with anti-mist for eye protection. Use approved dust respirators, or air-stream helmet.

9. PHYSICAL AND CHEMICAL PROPERTIES

Form - solid of varying sizes from large lumps to fine powder
Colour - white or off-white
Odour - faint "earthy" odour
pH - 12.4 (aqueous solution approx 2 g/litre as $Ca(OH)_2$)
Solubility in water - 1.33 g/litre at 10° (reacts with water to form Calcium Hydroxide)
Vapour pressure - 0 at 20°C
Specific Gravity - 3.4
Melting point: - 2570°C

10. STABILITY AND REACTIVITY

10.1 Stable

10.2 Conditions/Materials to avoid:

Minimise exposure to air to avoid degradation. Reacts vigorously with strong acids. Attacks aluminium, lead and brass in the presence of moisture. Reacts violently with moisture, generating heat.

10.3 Hazardous decomposition products:

None.

11. TOXICOLOGICAL INFORMATION

Inhalation - High concentrations of dust are irritant to the respiratory tract. Gross inhalation may cause inflammation, ulceration, perforation of nasal septum and pneumonitis.

Skin Contact - Irritant in the presence of moisture. May cause burns.

Eye Contact - Very painful irritant, may cause burns. Risk of severe and permanent damage to eyes.

Ingestion - May cause corrosion damage to the gastrointestinal tract.

Long term exposure - Prolonged and repeated skin contact may cause dermatitis.

12. ECOLOGICAL INFORMATION

12.1 Mobility
Sparingly soluble in water (as hydroxide) to form alkaline solution. Low mobility in most ground conditions.

12.2 Persistence and degradation
Non bio-degradable - reacts with moisture to form calcium hydroxide, and reacts with atmospheric and dissolved carbon dioxide to form calcium carbonate (chalk).

12.3 Toxicity
Harmful to aquatic organisms in high concentrations (generally greater than 100 mg/l).

12.4 Effects on Effluent Treatment
High concentrations (> 100 mg/l) may have a sterilising effect in sewage works. Product is extensively used in treatment of acid wastes and sewage sludges.

13. Disposal Considerations

Disposal should be in accordance with current local and national legislation. Quicklime can normally be disposed only to licensed waste facilities.

14. Transport Information

Not classified as hazardous for transport.

15. Regulatory Information

15.1 Classification, Packaging and Labelling

Dangerous Substances Regulations 1984.
Statutory Instrument 1244.
Classification for conveyance - none.
Classification for supply - Irritant.

Risk phrases
- Irritating to skin
- Risk of serious damage to eyes

Safety phrases
- Do not breathe dust
- Wear suitable gloves and eye/face protection
- In case of contact with eyes, rinse immediately with water and seek medical advice.
- Keep out of reach of children

15.2 Occupation Exposure Limits 1987 - HSE Guidance Note EH40/87. OEL $2\ mg/m^3$ (8 hr TWA)

15.3 Data Sheet prepared in accordance with Directive 91/155/EEC.

16. OTHER INFORMATION

Date of issue - July 1993

APPENDIX 2

CALCIUM ION ANALYSIS

CALCIUM TESTING

Method

Samples were first dried in a vacuum.

(a) Water Soluble Calcium

- i) 10g of dried, ground sample plus 100ml of Reverse Osmosis (R.O.) water: mix thoroughly for ~ 45 minutes.
- ii) Centrifuge mixture for ~ 10 minutes.
- iii) Pour off supernatant into conical flask and wash 'slug' with R.O. water.
- iv) Make solution up to 250ml with R.O. water; mix.
- v) Take 25ml of solution into a beaker, add 2 drops of KOH, sufficient to create an alkaline solution.
- vi) Add indicator tablet/sachet.
- vii) Titrate with E.D.T.A. until the solution just turns completely mauve/blue.
- viii) Repeat several times, take average result.

(b) Acid Soluble Calcium

- i) Repeat steps (i) - (iv) except adding ~ 10ml (until efforvescence ceases) of conc. HCl at stage (i) instead of 10ml of the water.
- v) Repeat final stages with 1ml of solution and 4-5 drops of 8N KOH.

Methods of testing for the free and locked in calcium:

FREE CALCIUM.

EDTA Tritations

A masking agent is required to eliminate effects of trace metals. This may account for some of the 'rogue' results achieved on previous tests, in which no masking agent was used, although the results may also be due to not filtering the leachates obtained, the tiny clay particles affecting individual titrations.

Atomic Adsorption

A more sensitive and reliable method than above.

Method:

- treat with Nitrous Oxide and Acetylene flame (brakes down $CaSO_4$)
- make a solution of 1-10ppm
- acidify with high grade HCl to pH<2 (approx. 1ml acid to 100ml leachate)
- the use of clean apparatus is essential, preferably that which has been soaked in acid beforehand.

TOTAL CALCIUM

A fusion technique is advised using one of the following methods:

- fuse 0.2g dry clay with 2g of Tetraflux (Lithium tetraborate) in a platinum crucible.
- use atomic adsorption to detect Calcium

or

- fuse with sodium carbonate in a silica crucible
- leach with 10% Nitric Acid and filter

This method may cause Silica to come out of solution which must be filtered off.

REACTION TYPE/PRODUCTS

A range of possible analysis techniques are described as follows:

Particle Size Analysis

Clays pose a problem as the results depend on the method of initial treatment. It is necessary to treat the clay in such a way as to reflect the conditions under which it is being studied. Non-spherical particles are also difficult. The pore sizes and shapes may also be studied; both are dry techniques.

APPENDIX 3

DETAILS OF PULVERISED FUEL ASH

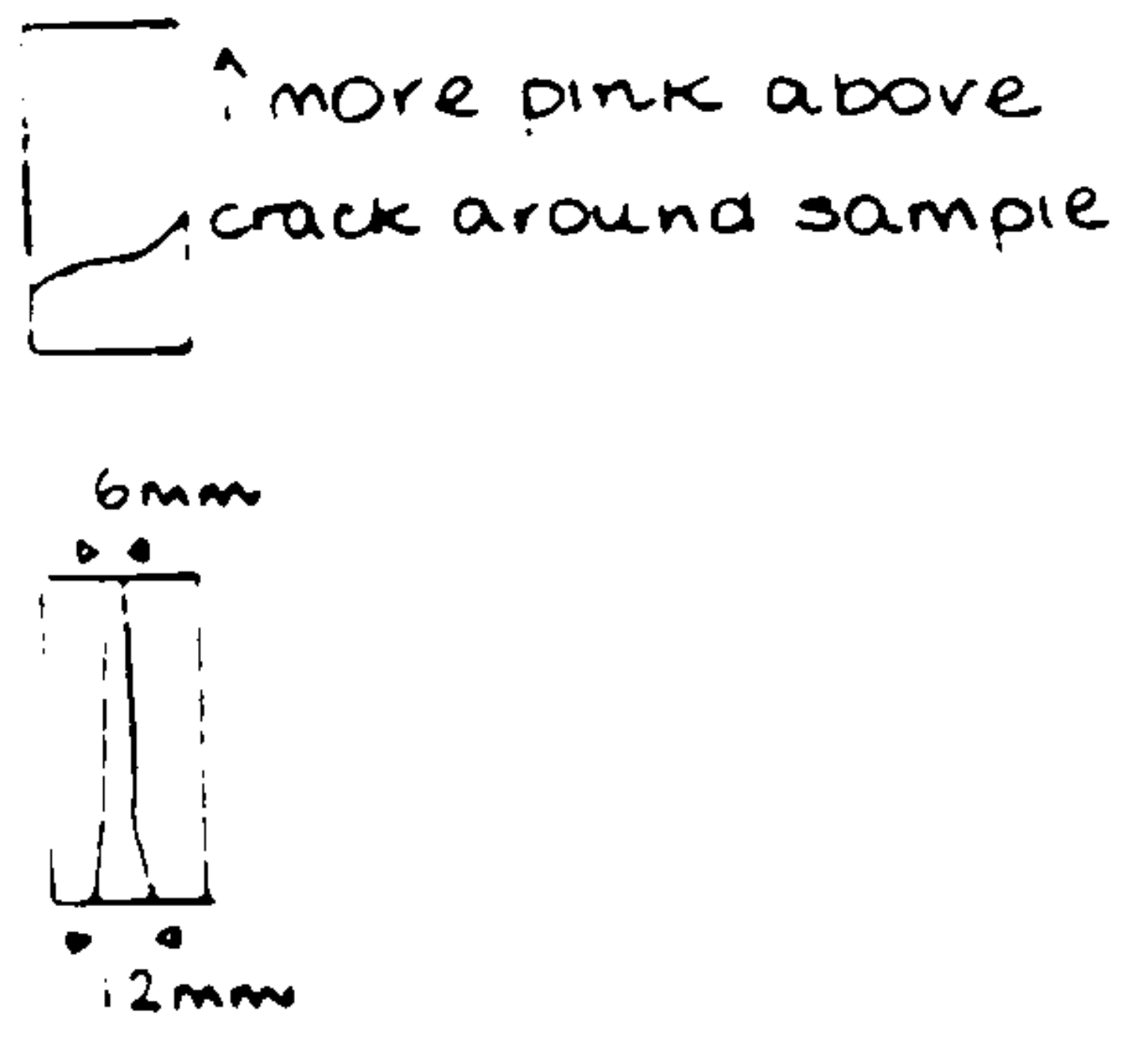
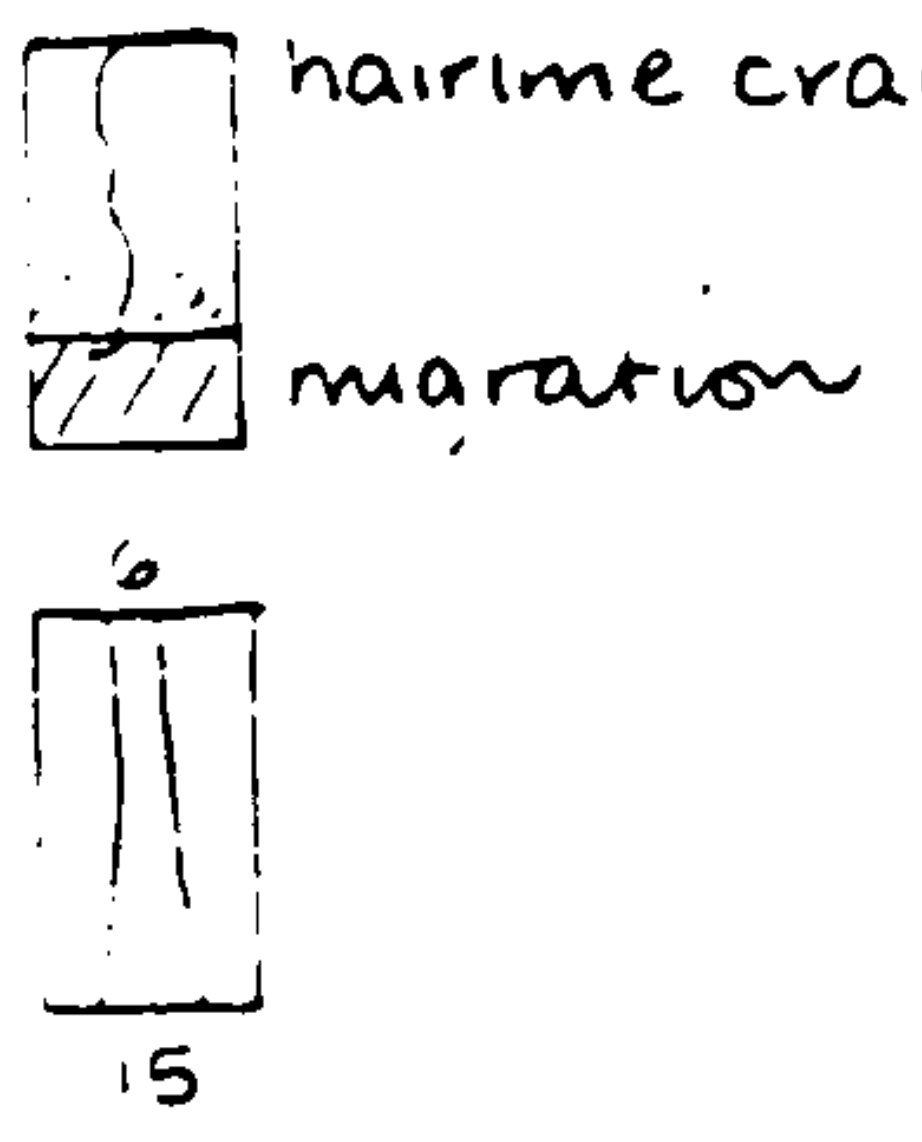
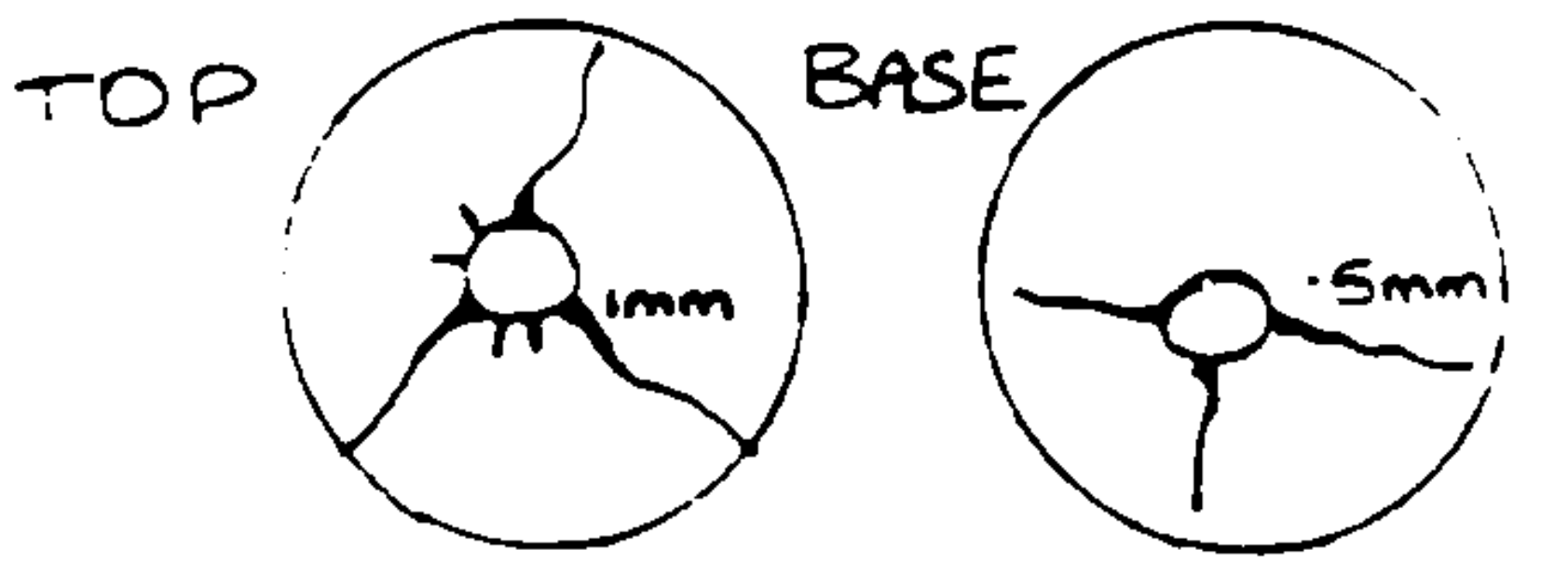
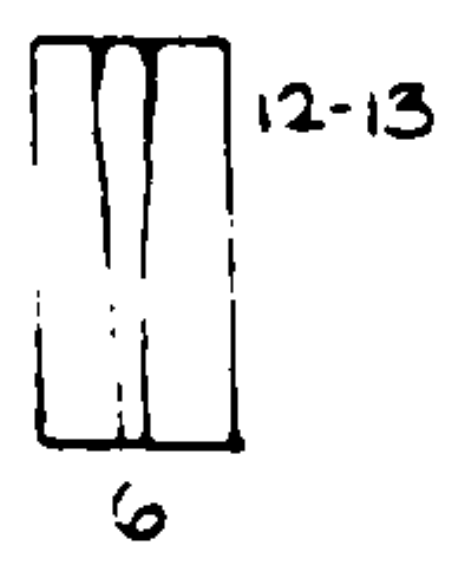
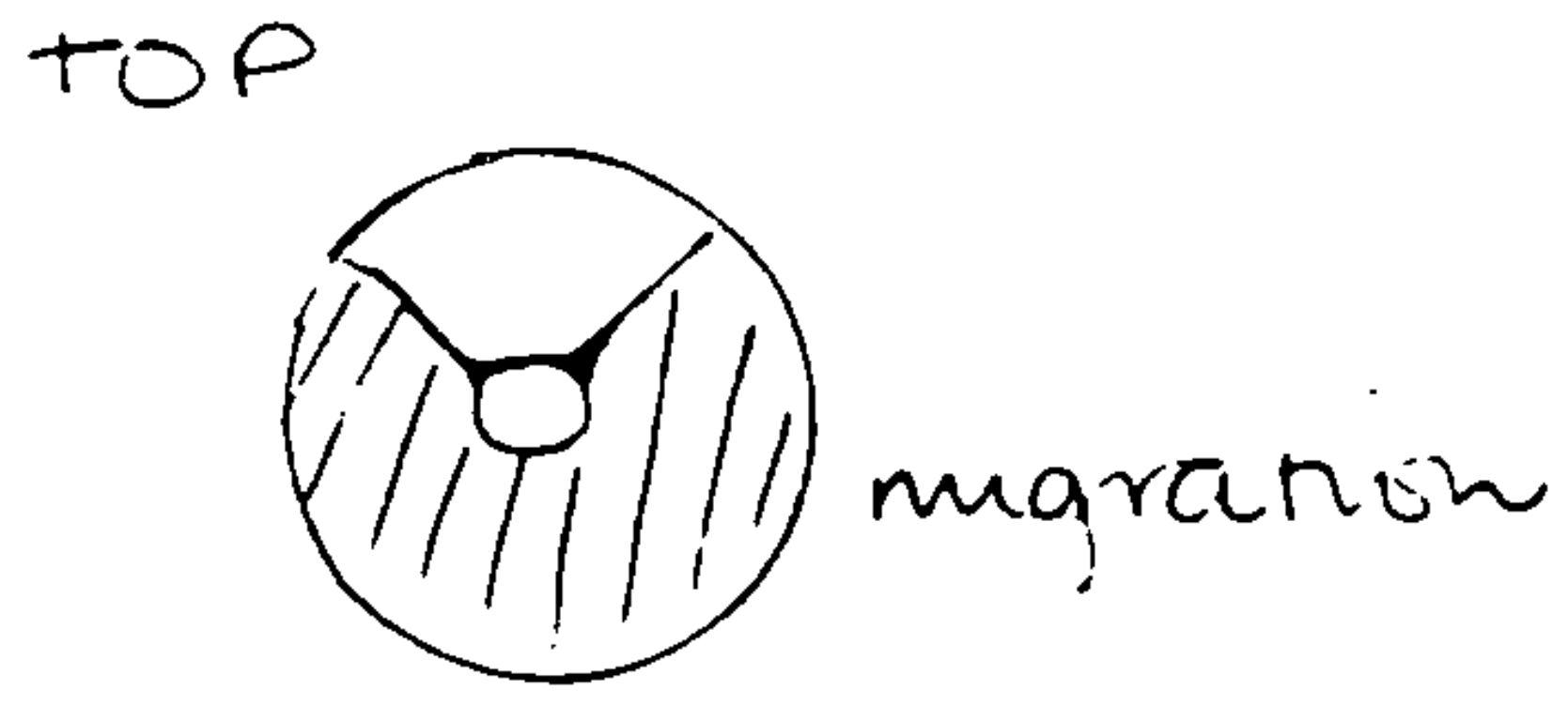
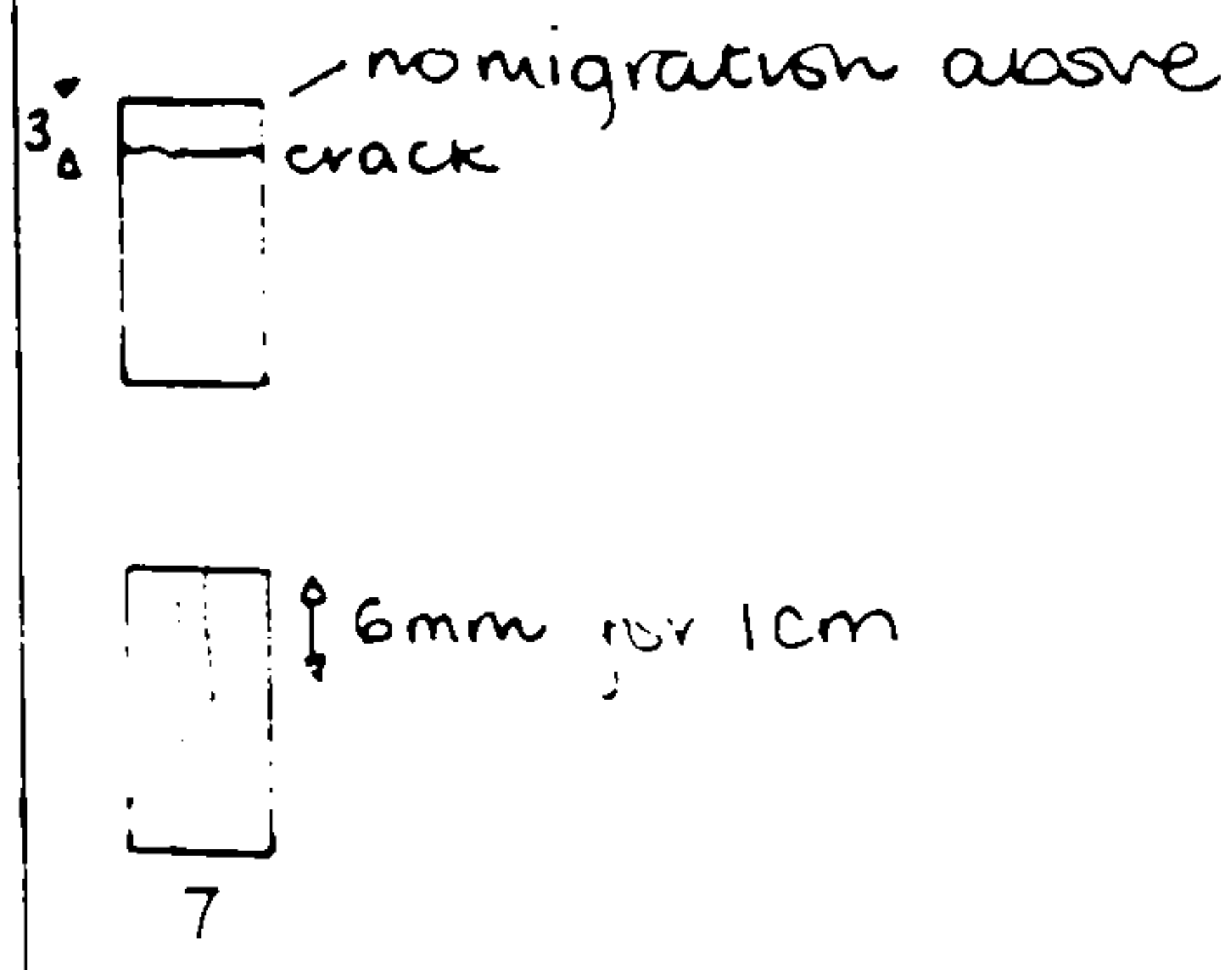
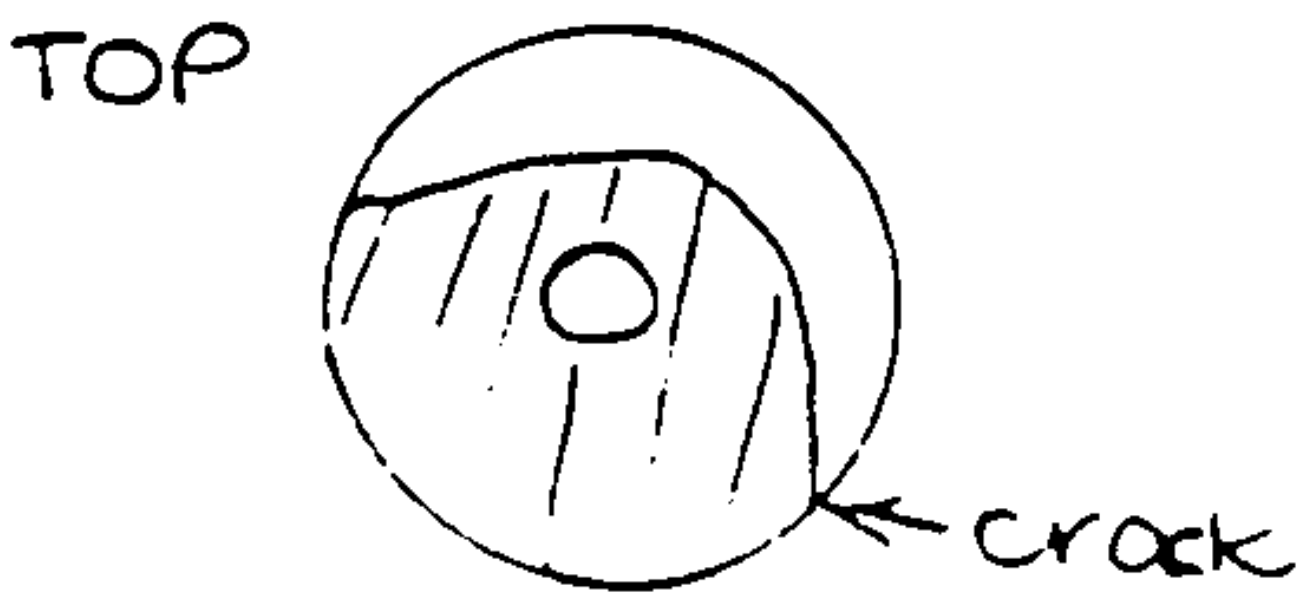
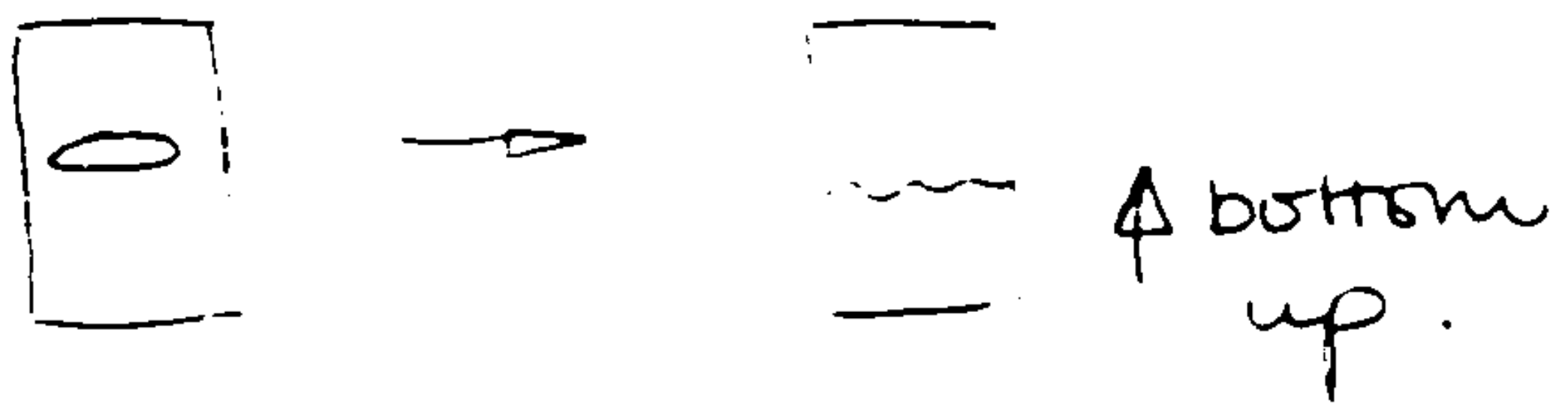
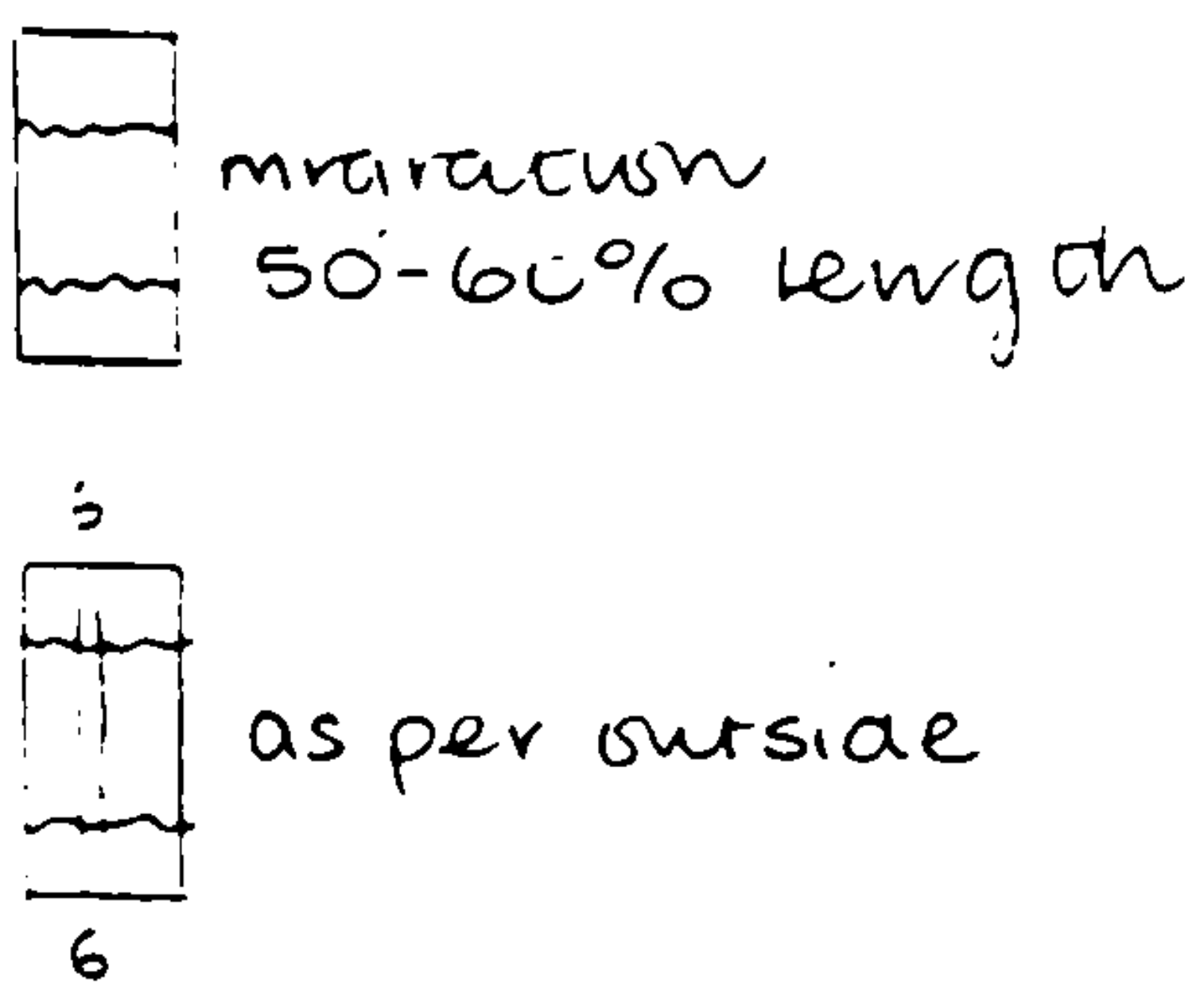
Chemical composition of PFA

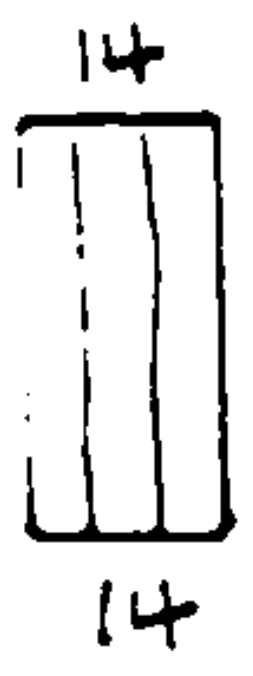
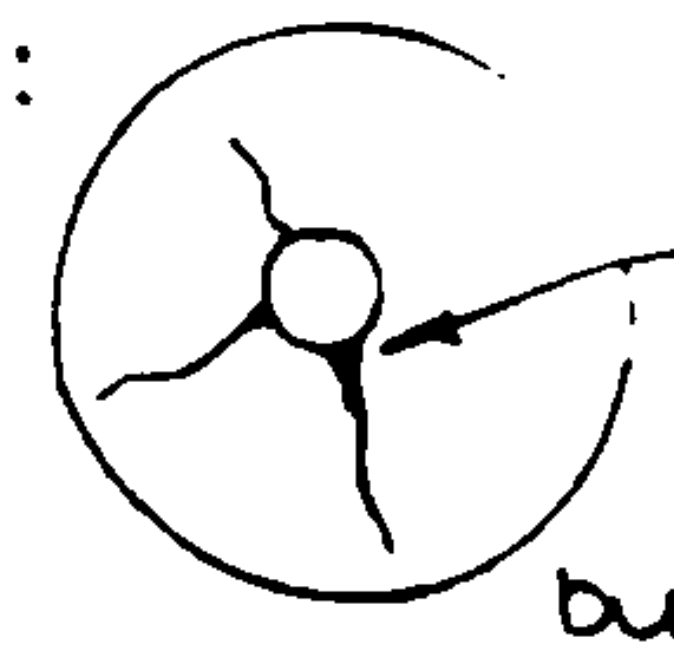
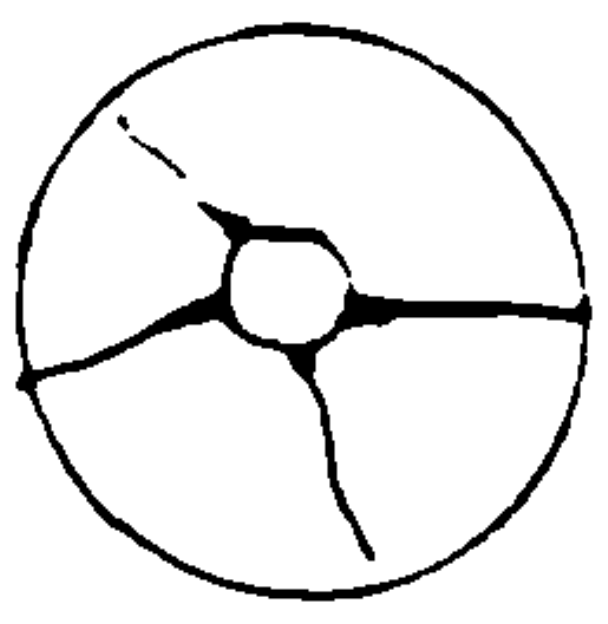
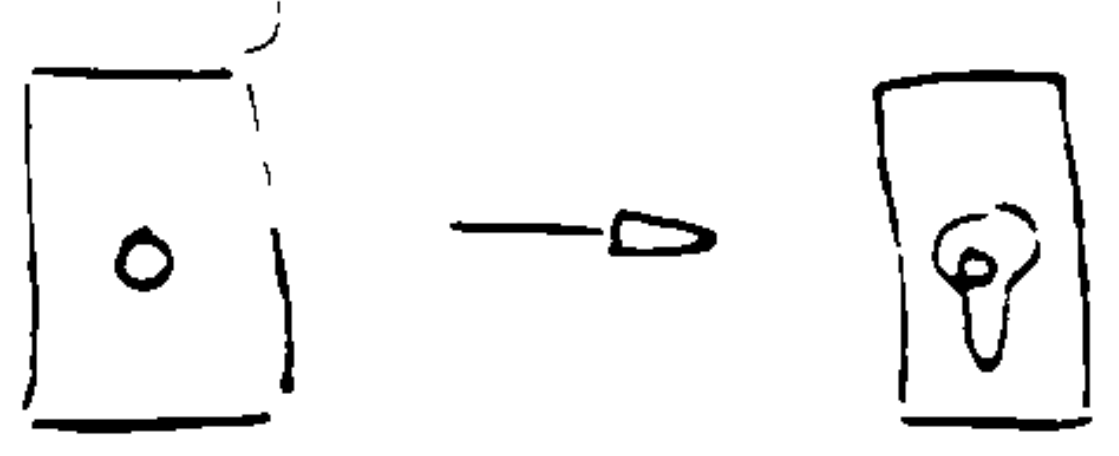
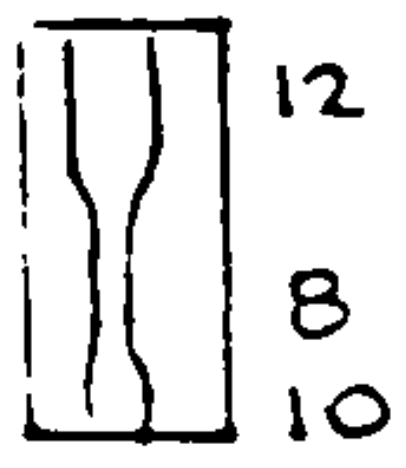
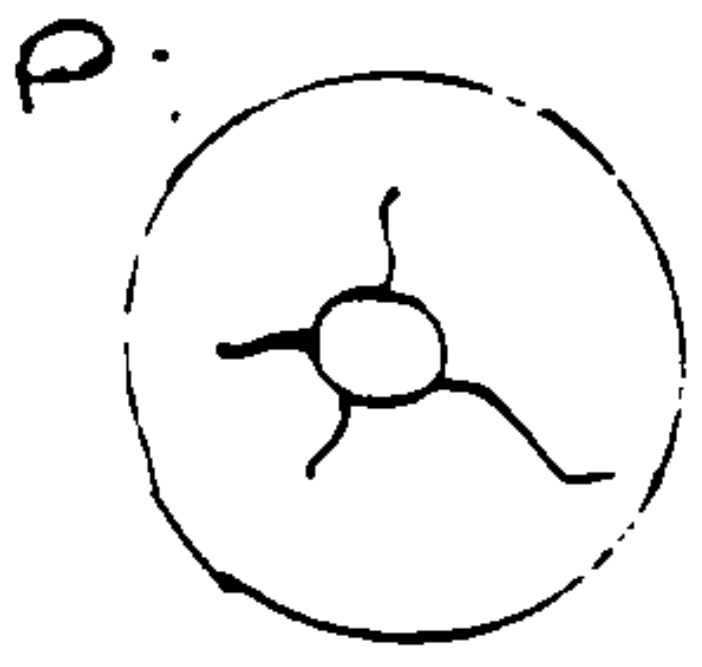
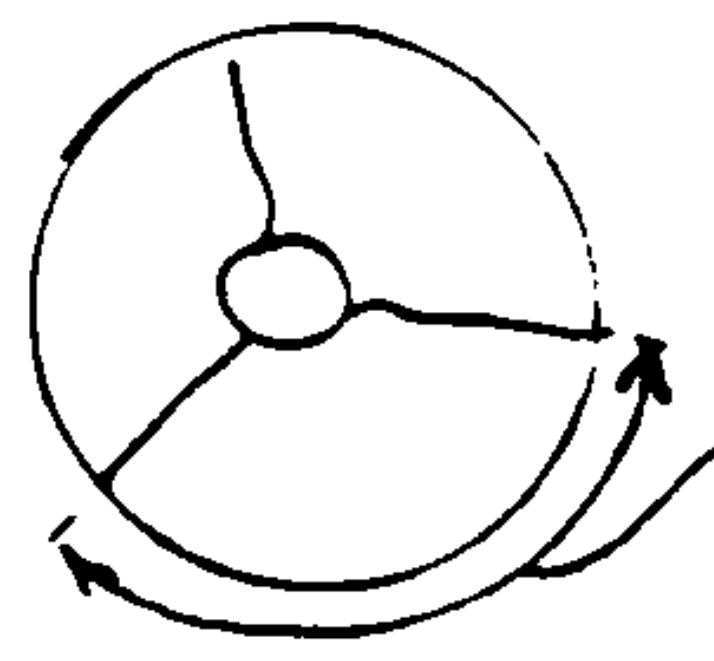
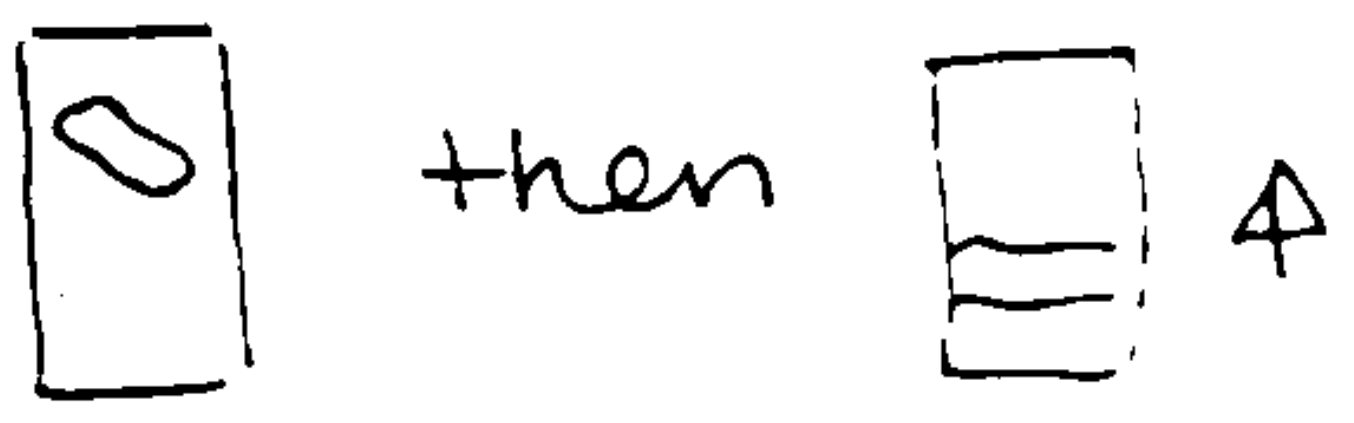
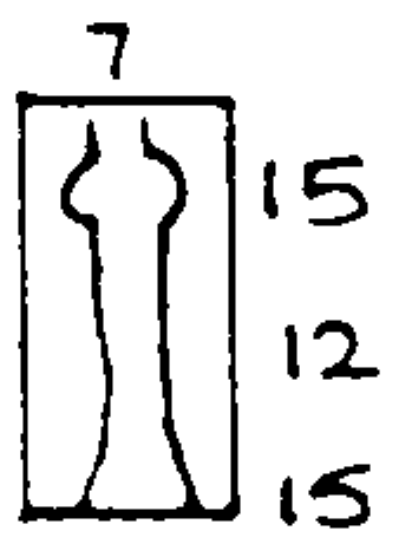
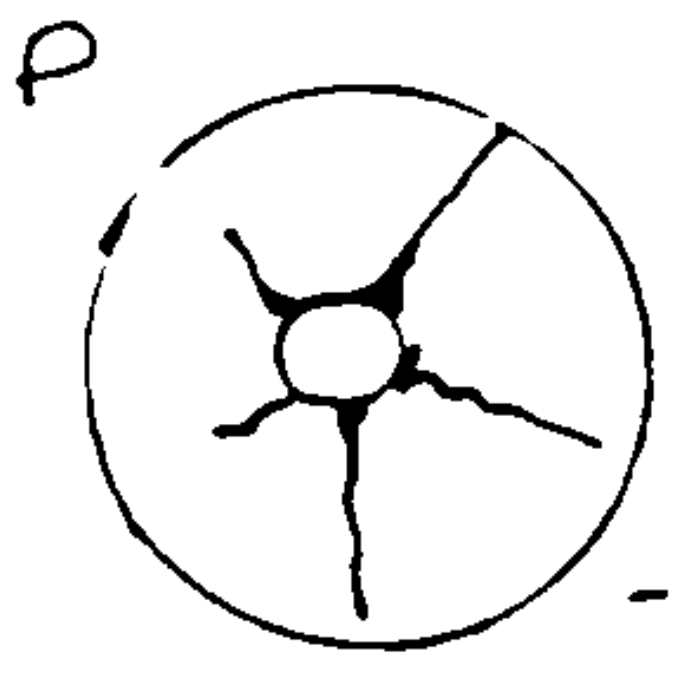
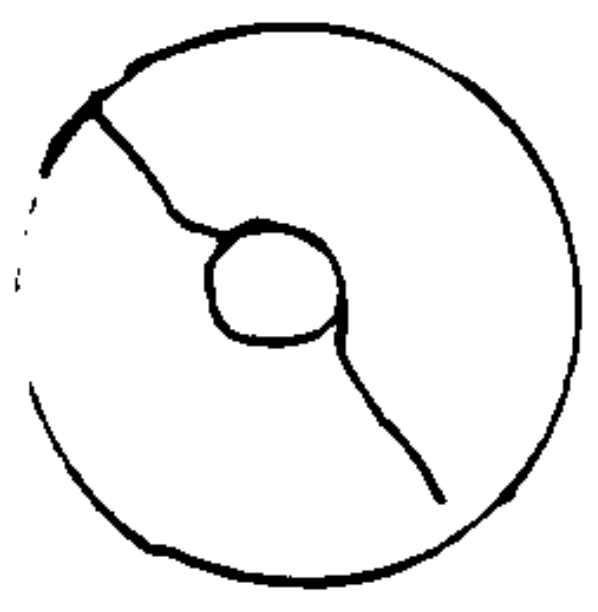
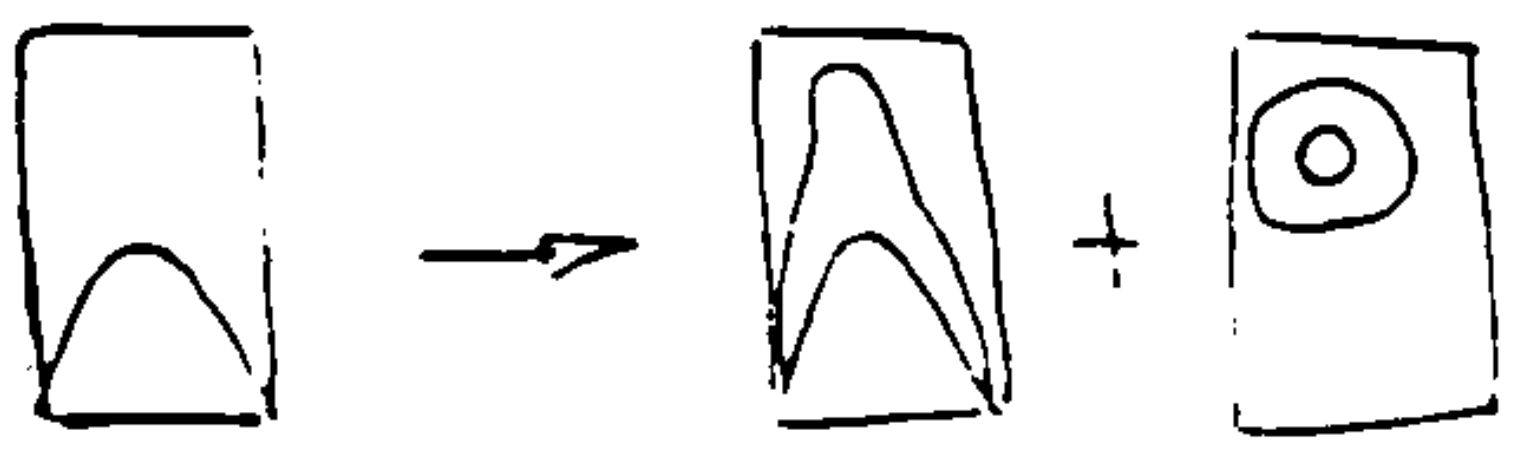
CHEMICAL DESCRIPTION	MAXIMUM WEIGHT (%)	MINIMUM WEIGHT (%)	TYPICAL WEIGHT (%)
Silicon (as SiO_2)	52	48	50
Aluminium (as Al_2O_3)	32	24	28
Iron (as Fe_2O_3)	15	7	10.5
Calcium (as CaO)	5.3	1.8	2.3
Magnesium (as MgO)	2.1	1.2	1.6
Potassium (as K_2O)*	4.5	2.3	3.6
Sodium (as Na_2O)	1.8	0.8	1.2
Titanium (as TiO_2)	1.1	0.9	1.0
Sulphur (as SO_3)	1.3	0.48	0.7
Chloride (as Cl)	0.15	0.05	0.08

* Acid soluble

APPENDIX 4

PERSPEX TUBE EXTRUSION LOGS

Sample	Longitudinal	Lateral	Comments
as moisture content 42% set up: 2/12/91 ogged 0/1/92		migration through sample	even distribution of tiny air bubbles (fractions of mm) very fine crystals present - no visible effects
as moisture content 42% set up. 22/10/91		internal migration reflects external 	migration coincides with pile swelling no visible difference for migration along cracks.
as moisture content: 42% set up: 22/10/91		migration as per surface. 	ditto
as moisture content 36% set up: 4/12/91 ogged: 3/2/92			less migration where no pile swelling migration progression: 
as moisture content 30% set up: 4/11/91 ogged: 0/1/92			no cracking no pile expansion no visible differences in porosity

Sample	Longitudinal	lateral	Comments
Lias: China 1 : 3 Moisture Content: 42% set up: 6/12/91 logged: 3/2/92	 cracks penetrate length from face  creation or swelling?	TOP:  1.2mm pile raised by ~1mm  BASE: 	migration:  clay v. dry - desiccated pile/clay interface  bark like clay crumbles.
Lias: China 3 : 1 Moisture Content 42% set up: 10/12/91 logged 3/2/92	 cracks penetrate length from face no migration 	TOP:  raised pile BASE:  no migration between cracks.	migration:  drier where no reaction less 'bark' effect.
Lias: China 1 : 1 Moisture Content 42% set up: 9/12/91 logged: 3/2/92	 cracks penetrate 	TOP  1mm cracks -raised pile 1mm BASE  hairline crack.	migration: 

APPENDIX 5

INITIAL LIME CONSUMPTION TEST RESULTS

DATA SHEET

Project : LIME
Project No : 92-7386

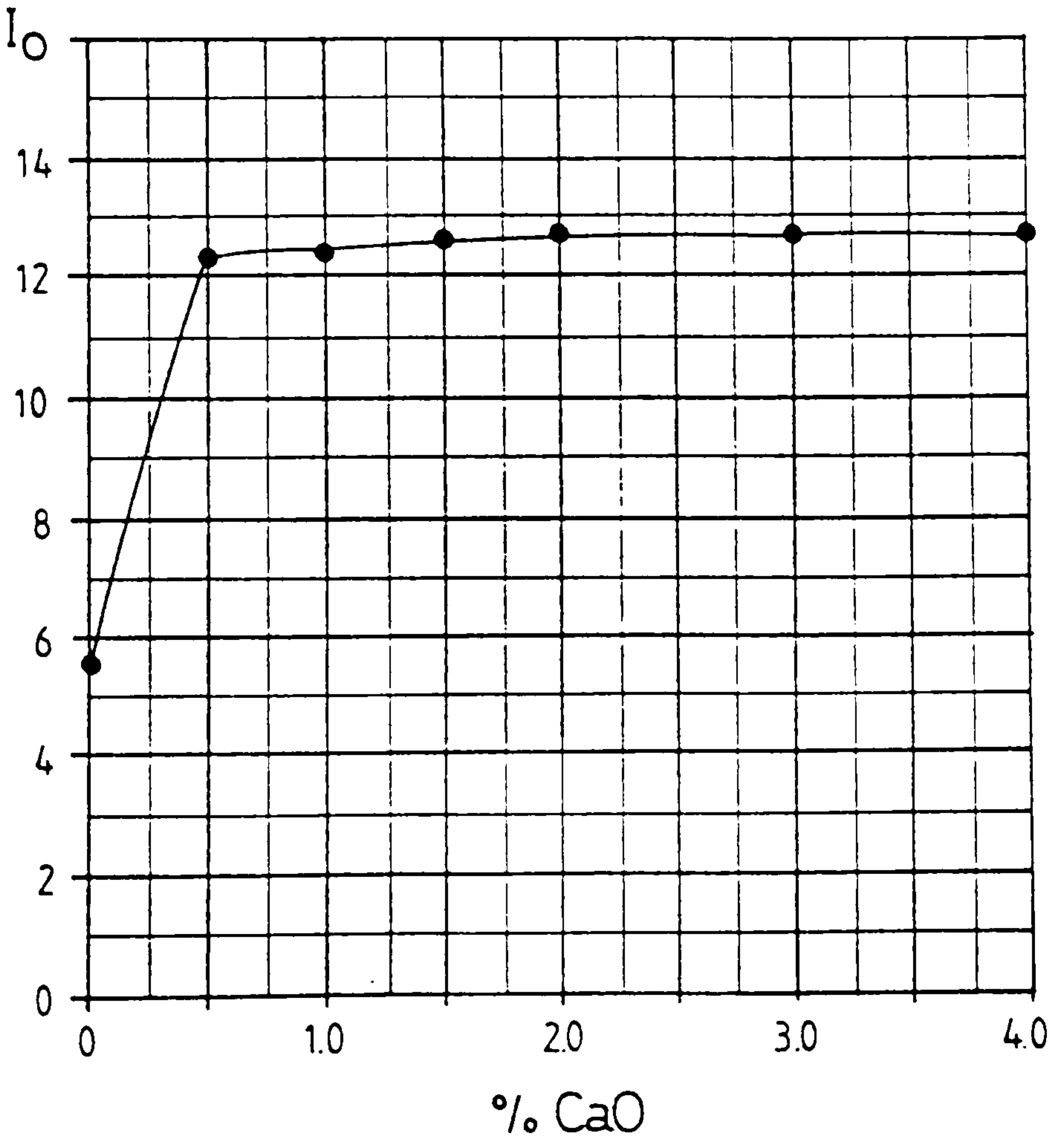
Batch N°.

EADES & GRIM TEST

ENGLISH CHINA CLAY

Sample S92 27608

Bottle N°.		0	1	2	3	4	5	6		
% CaO		0	0.5	1.0	1.5	2.0	3.0	4.0		
g CaO		0	0.1	0.2	0.3	0.4	0.6	0.8		
pH	I	5.44	12.17	12.48	12.61	12.64	12.68	12.69		
	II									
Mean pH										



geotechnics

DATA SHEET

Project : LIME
Project No : 92-7386

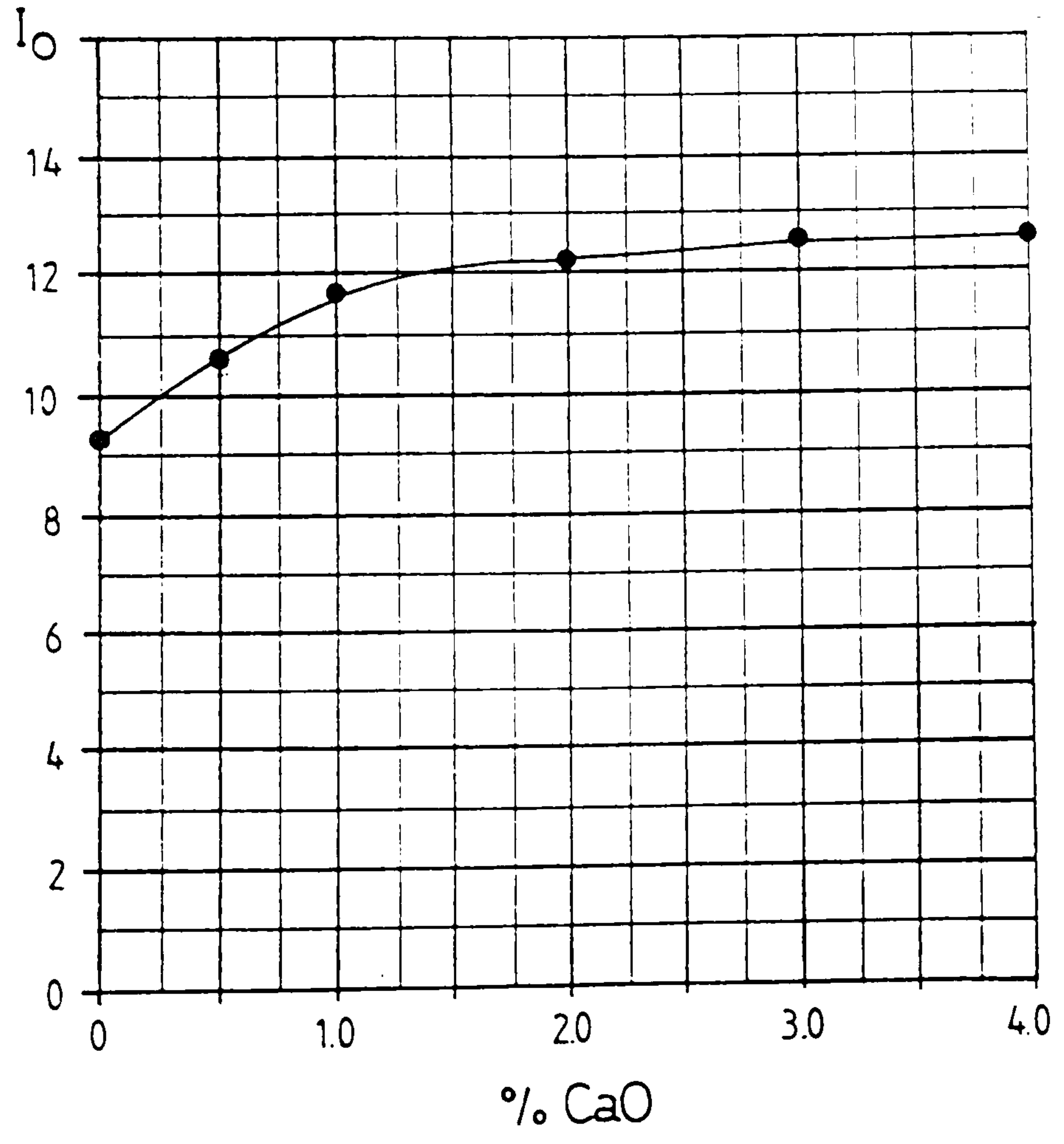
Batch Nº.

EADES & GRIM TEST

WYOMING BENTONITE

Sample S92 27607

Bottle Nº.		0	1	2	3	4	5	6		
% CaO		0	0.5	1.0	2.0	3.0	4.0	6.0		
g CaO		0	0.05	0.1	0.2	0.3	0.4	0.6		
pH	I	9.15	10.64	11.76	12.07	12.53	12.64	12.79		
	II	10.10	10.10	12.12	12.42	12.70	12.80	12.93		
Mean pH										



geotechnics

DATA SHEET

Project : LIME
Project No : 92 - 7386

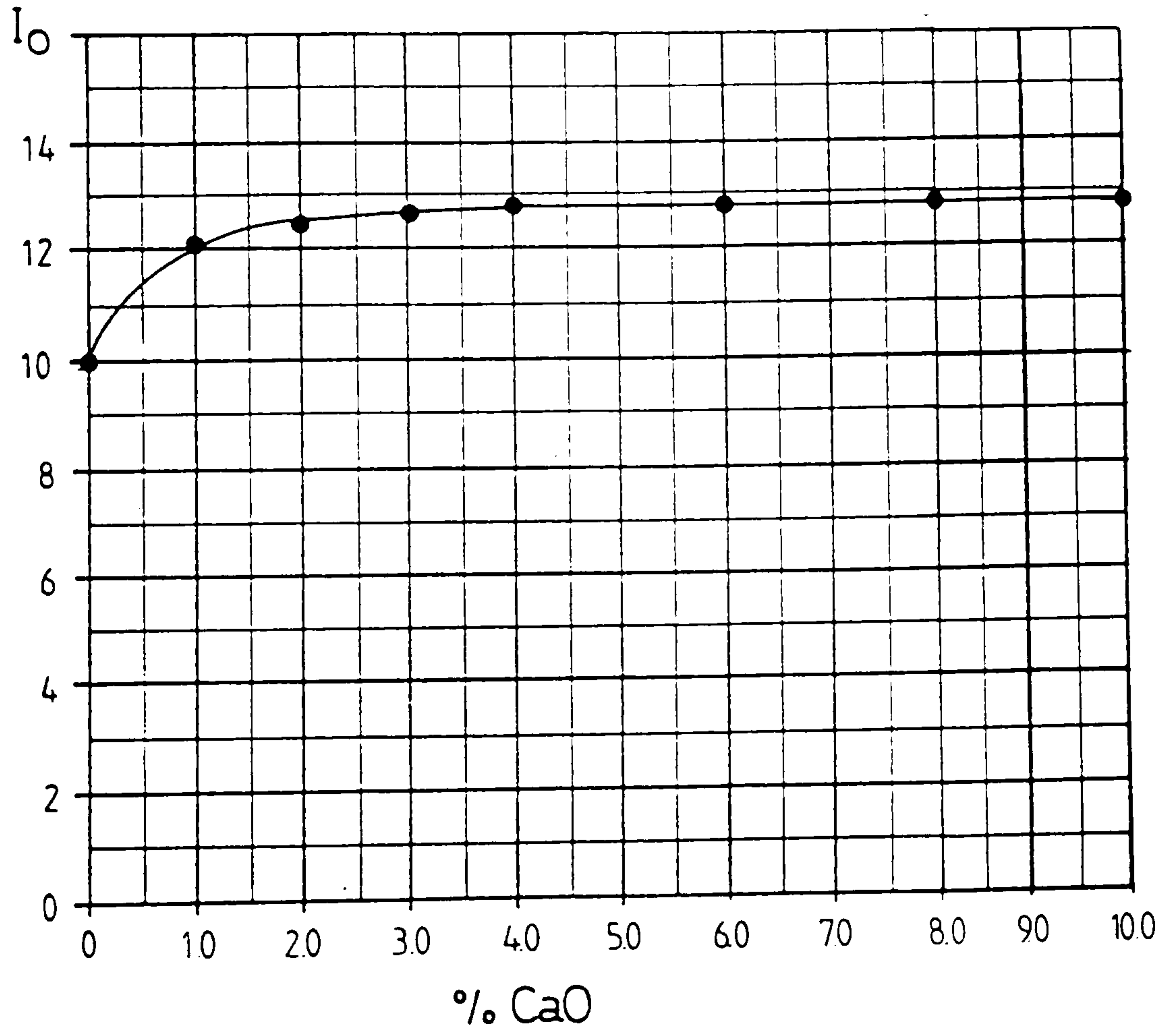
Batch Nº.

EADES & GRIM TEST

WYOMING BENTONITE

Sample Nº. S92 27607

Bottle Nº.		0	1	2	3	4	5	6	7	
%CaO		0	1.0	2.0	3.0	4.0	6.0	8.0	10.0	
g CaO		0	0.1	0.2	0.3	0.4	0.6	0.8	1.0	
pH	I	10.10	12.12	12.42	12.70	12.80	12.93	12.92	12.93	
	II									
Mean pH										



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DATA SHEET

Project : LIME
Project No : 92-7386

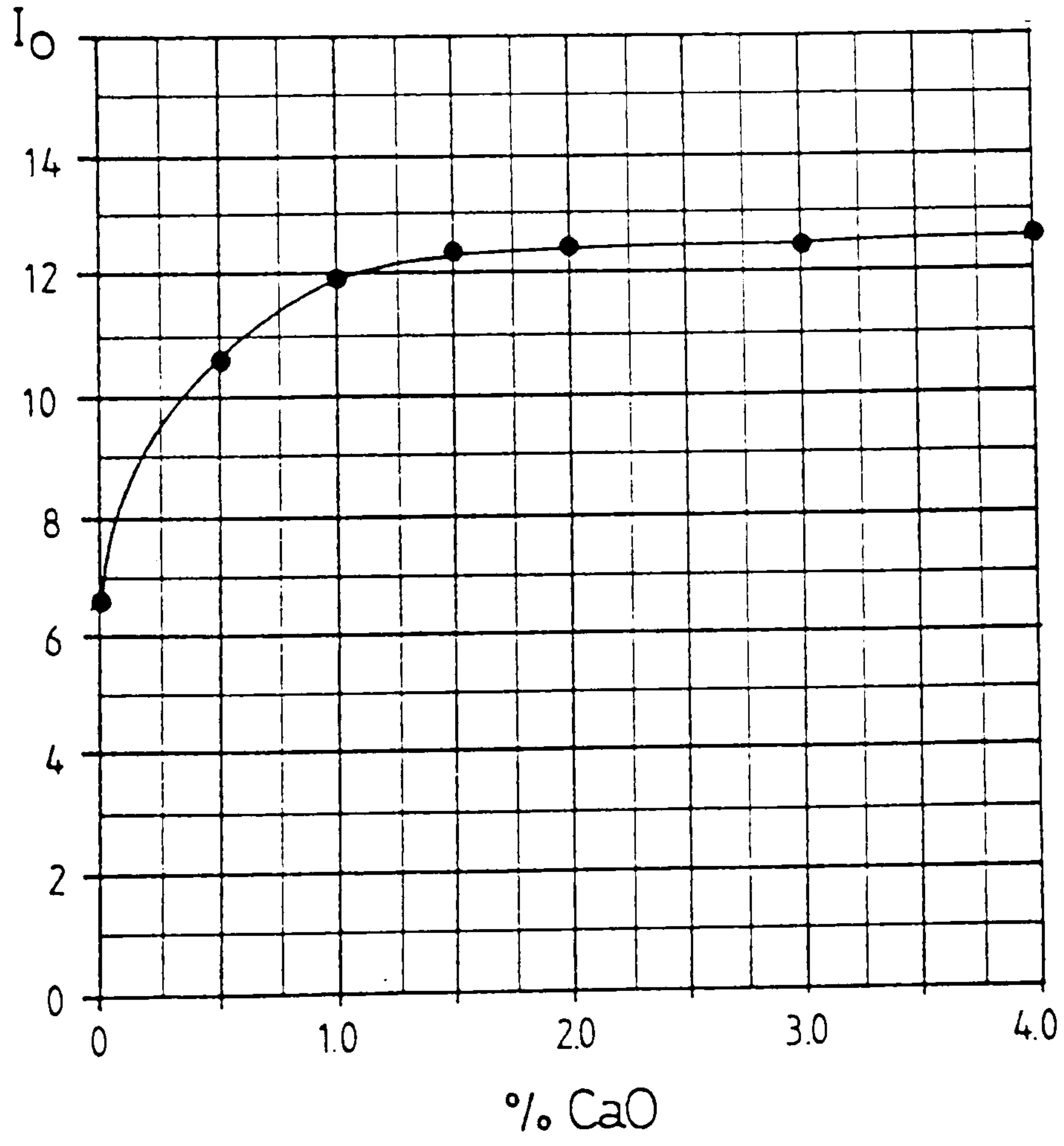
Batch Nº.

EADES & GRIM TEST

LOWER LIAS CLAY

Sample S92 26687

Bottle N°		0	1	2	3	4	5	6		
% CaO		0	0.5	1.0	1.5	2.0	3.0	4.0		
g CaO		0	0.1	0.2	0.3	0.4	0.6	0.8		
pH	I	6.66	10.66	11.80	12.36	12.42	12.57	12.59		
	II									
Mean pH										



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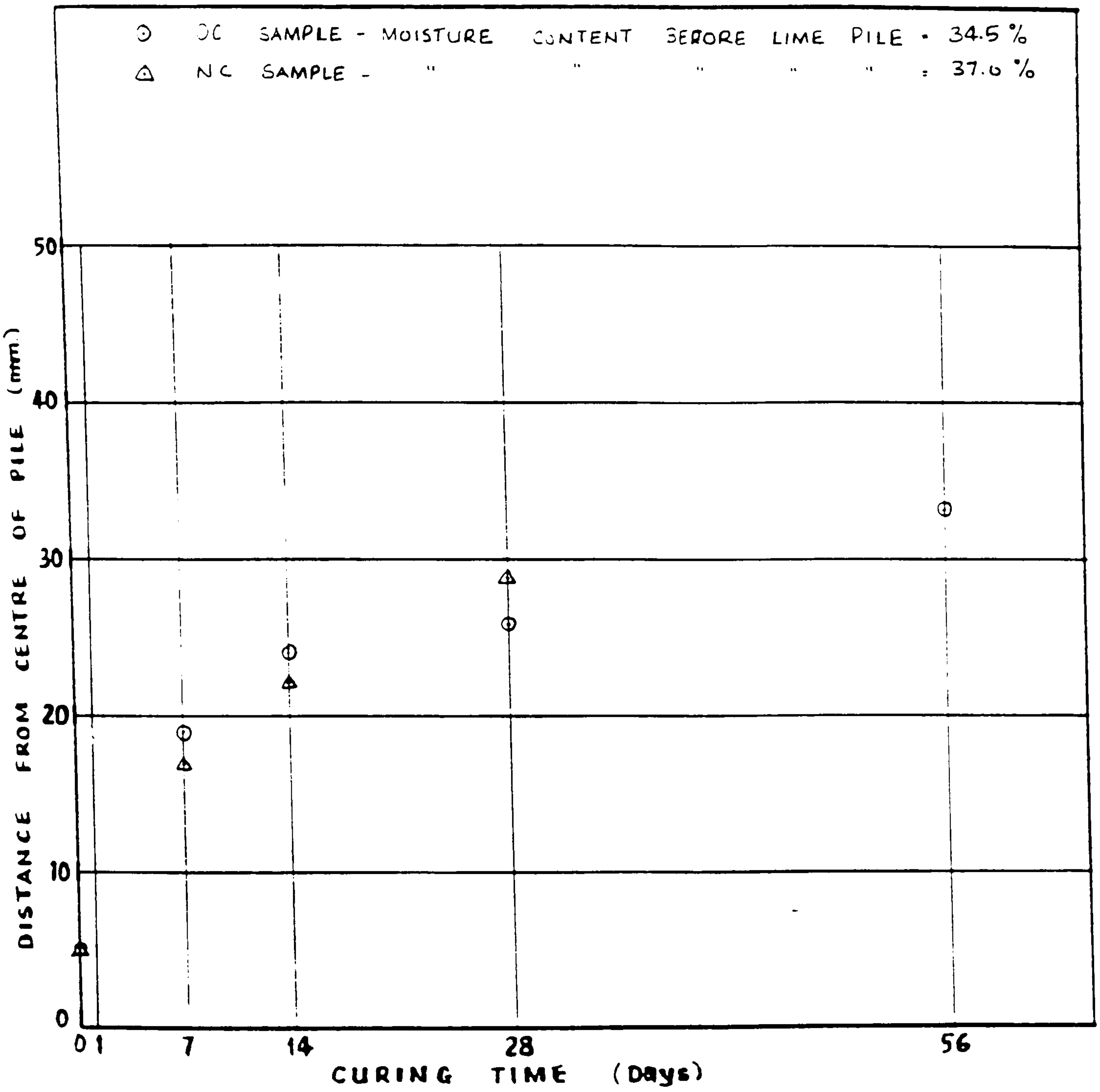
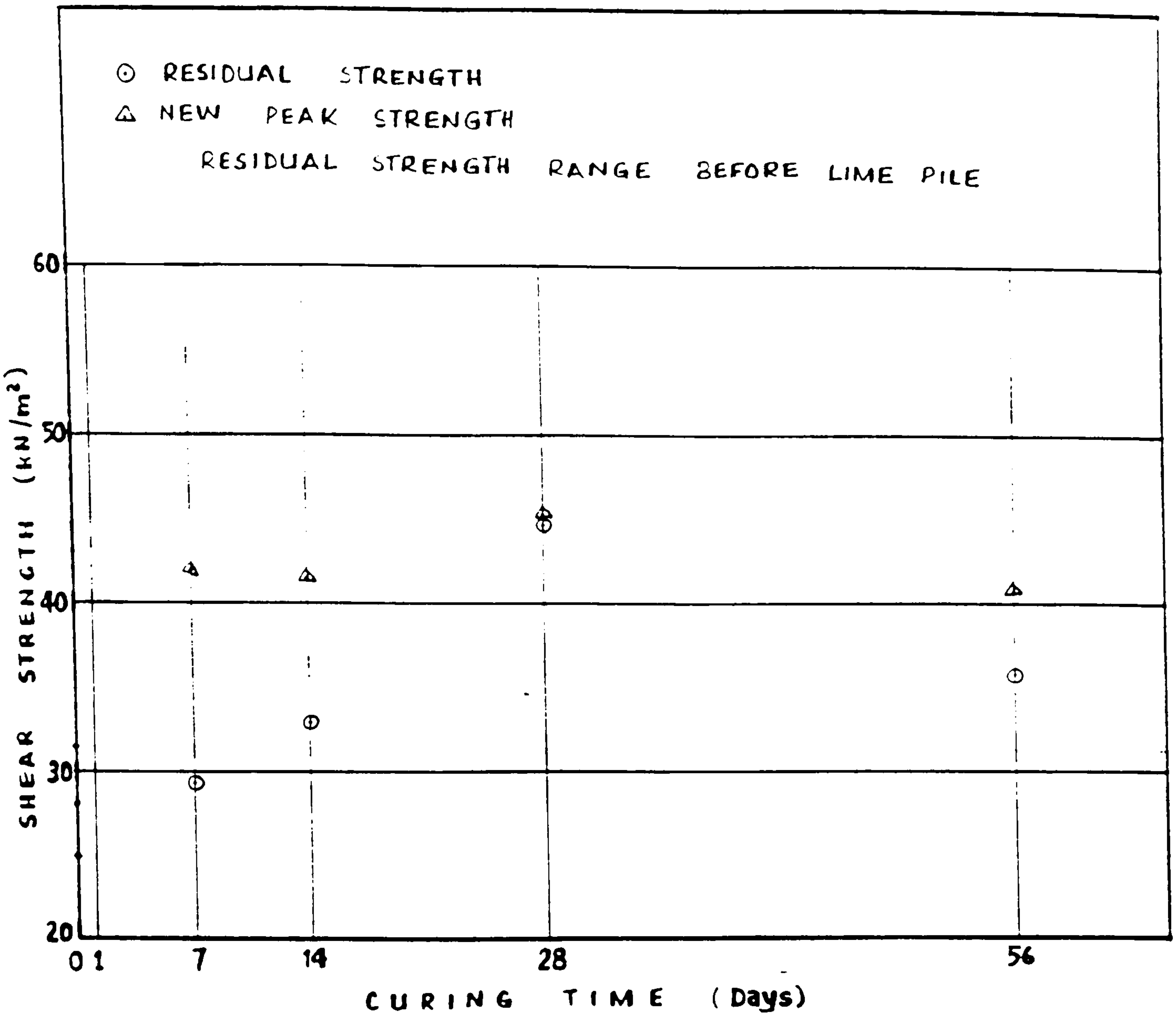
APPENDIX 6

MINI-PILE TEST RESULTS

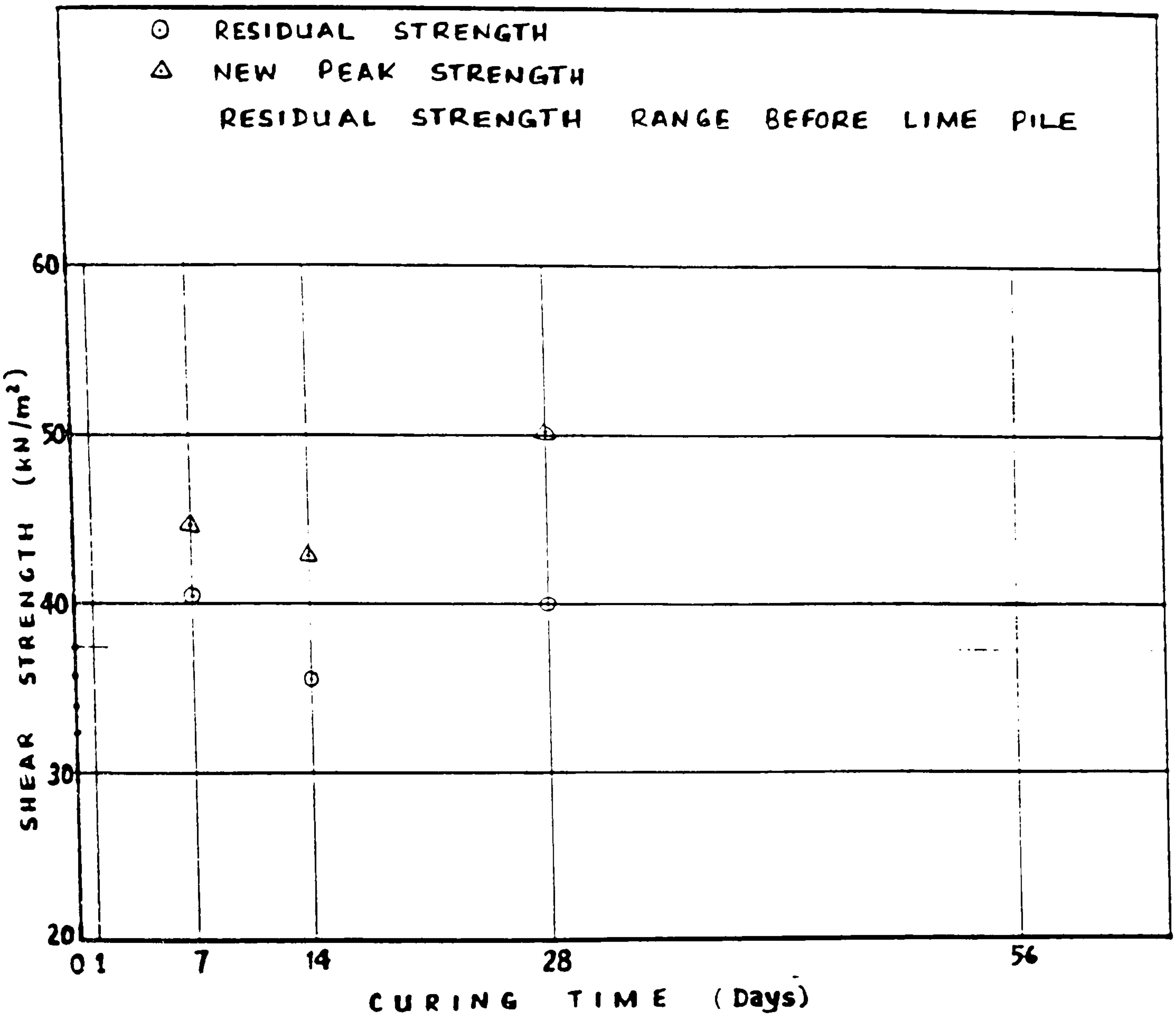


MIGRATION OF LIME RADIALLY FROM 10mm Ø LIME PILE

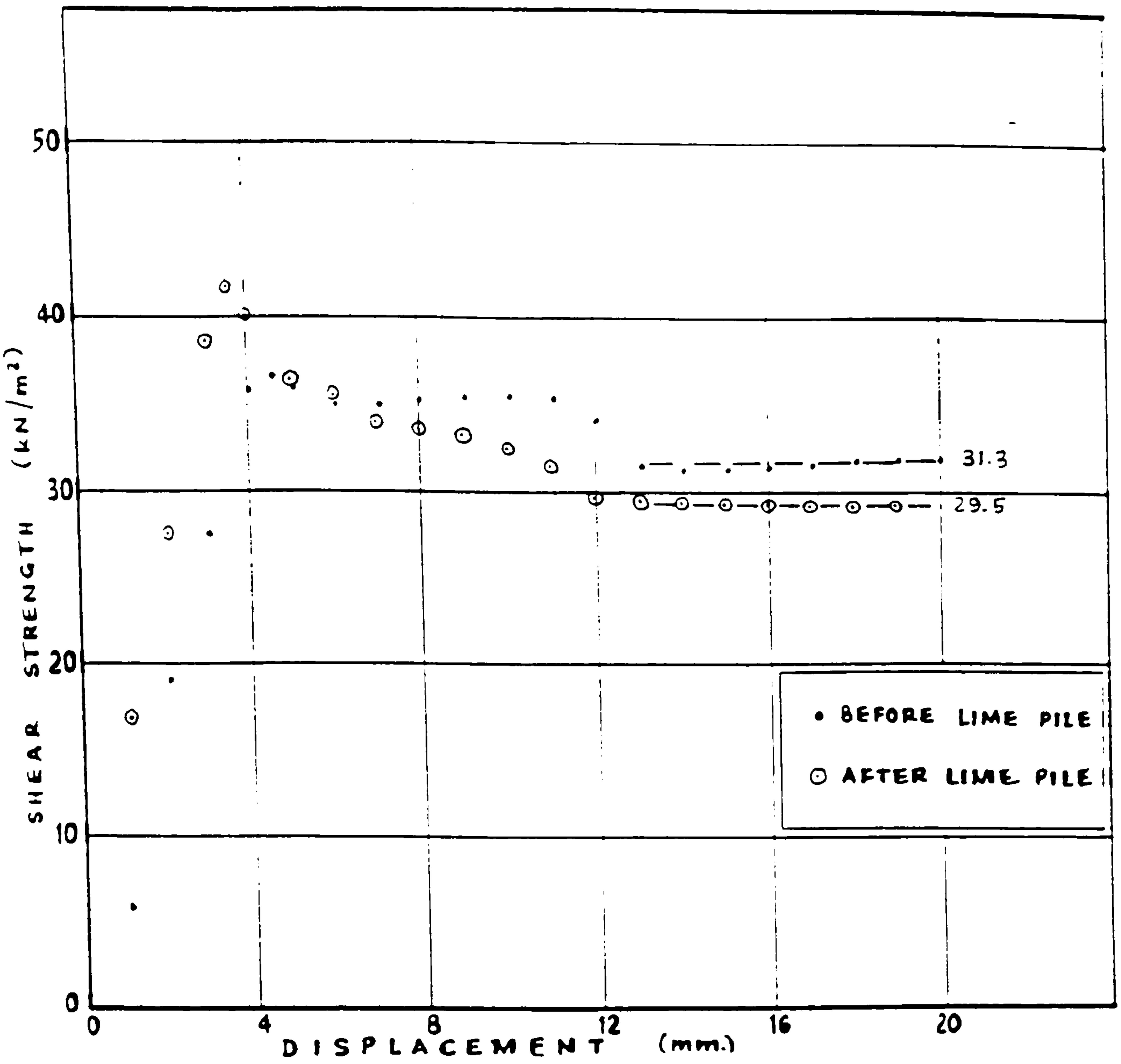


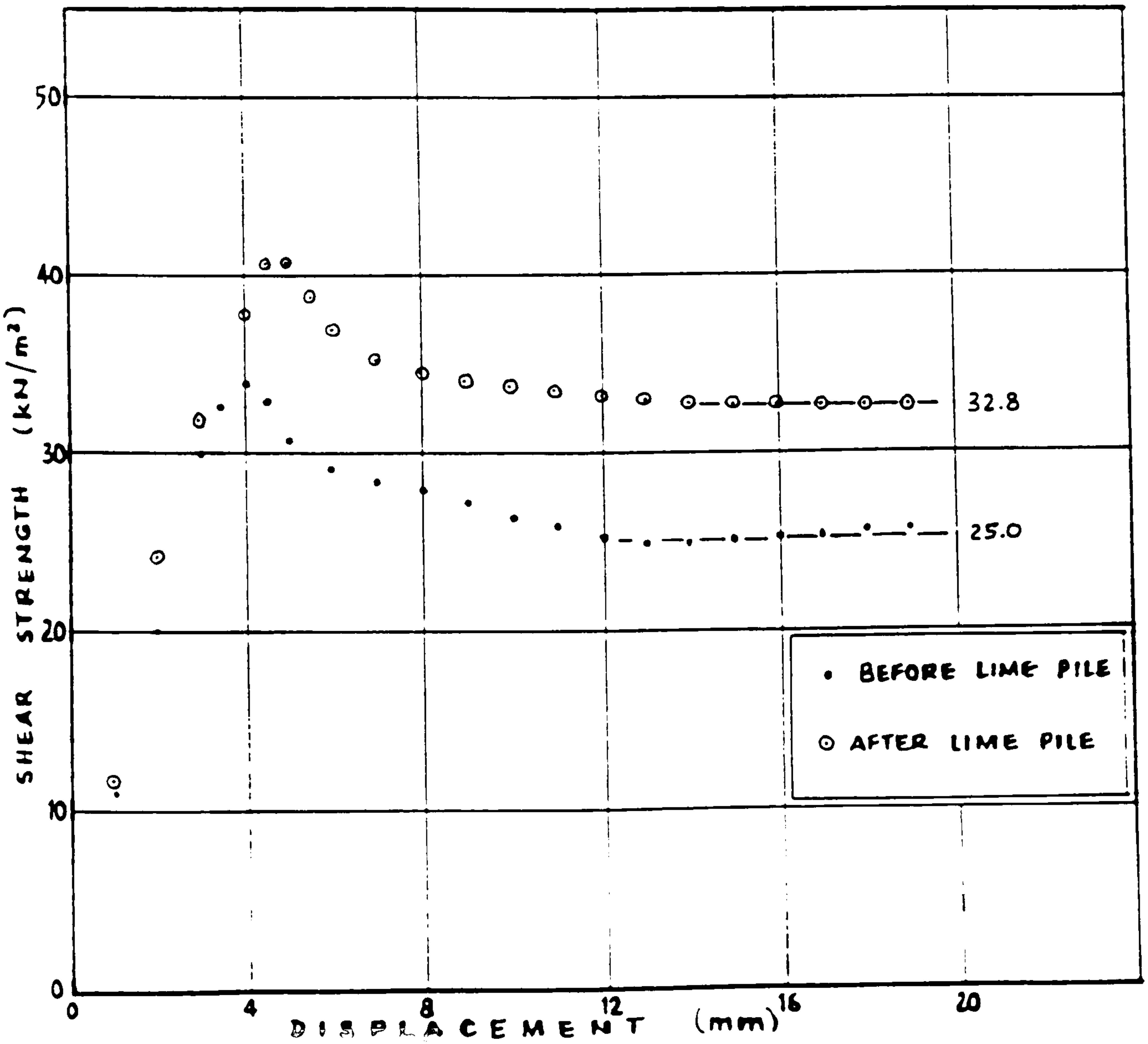
RESIDUAL AND NEW PEAK STRENGTH AFTER CURING WITH LIME PILE

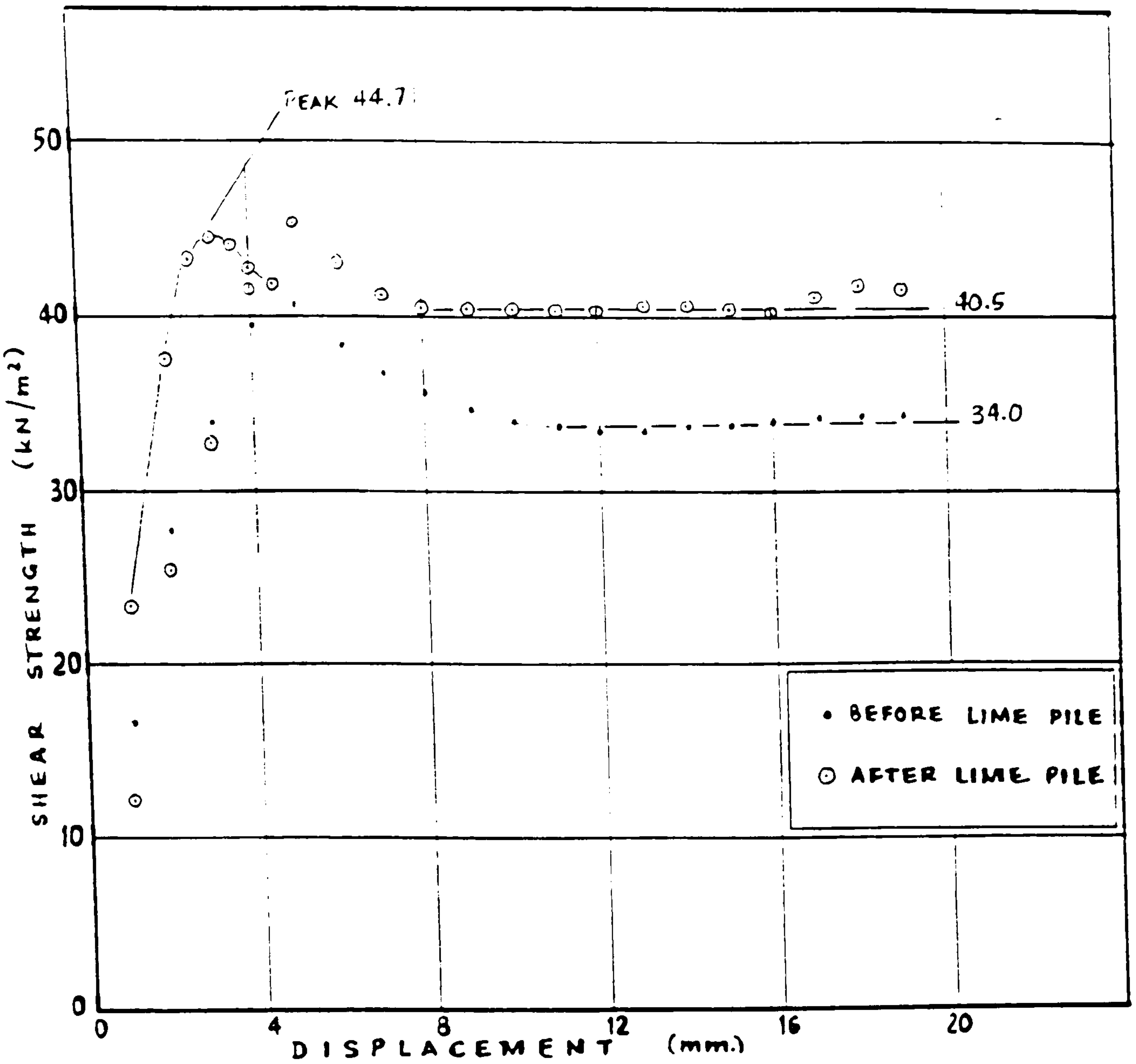
CHINA CLAY (OC) w = 34.5 %

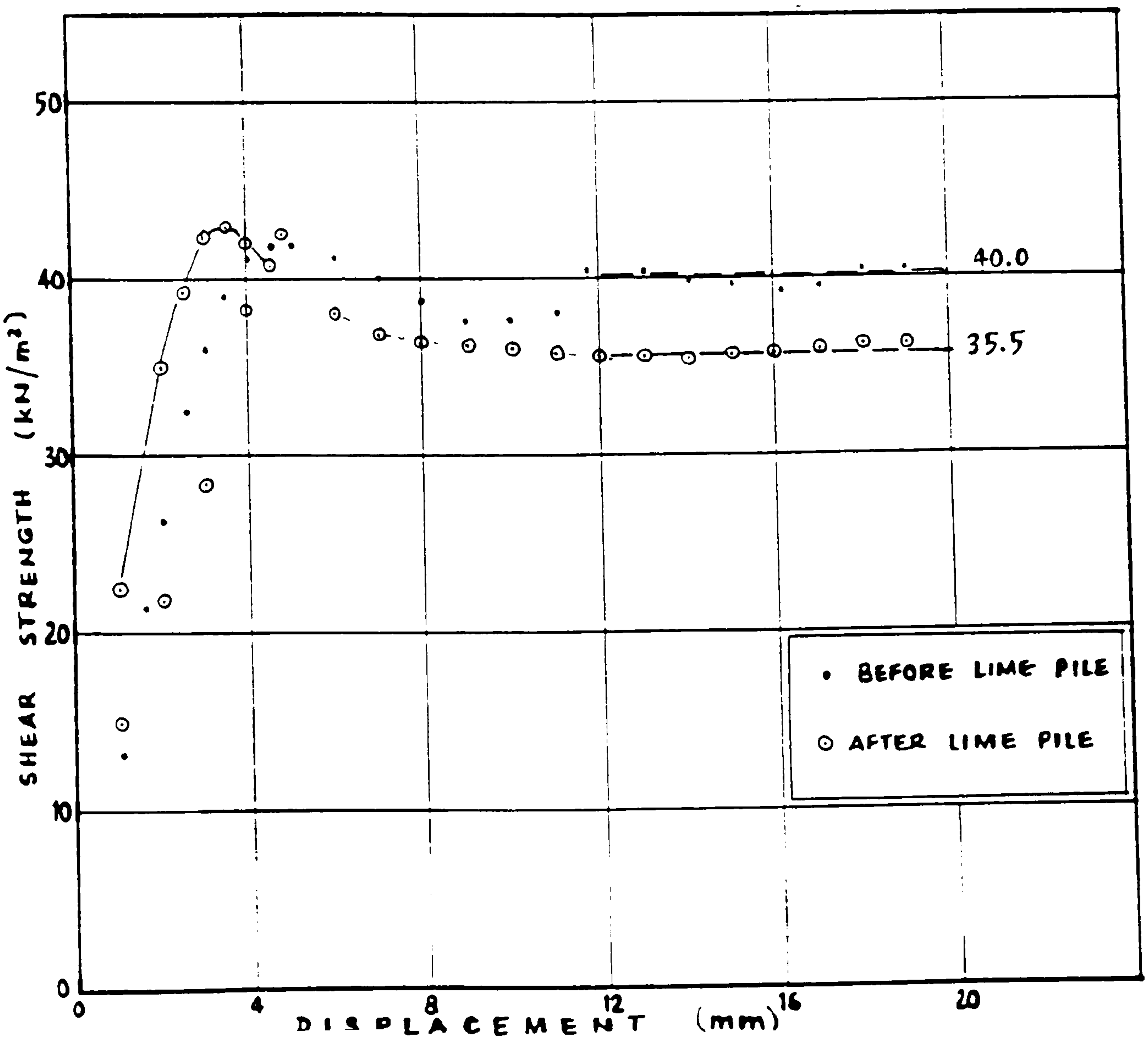


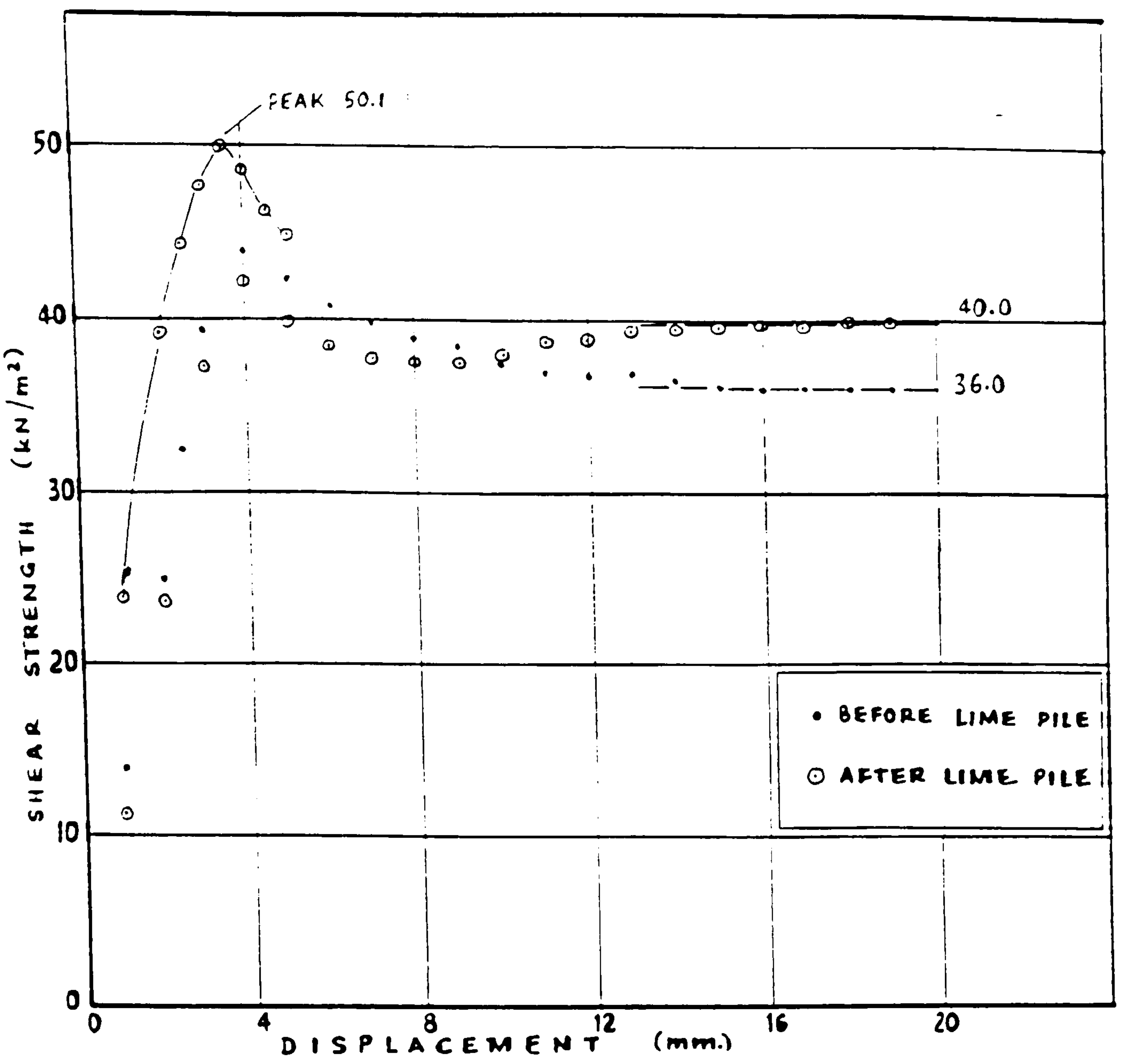
RESIDUAL AND NEW PEAK STRENGTH
AFTER CURING WITH LIME PILE
CHINA CLAY (NC) w = 37.0 %

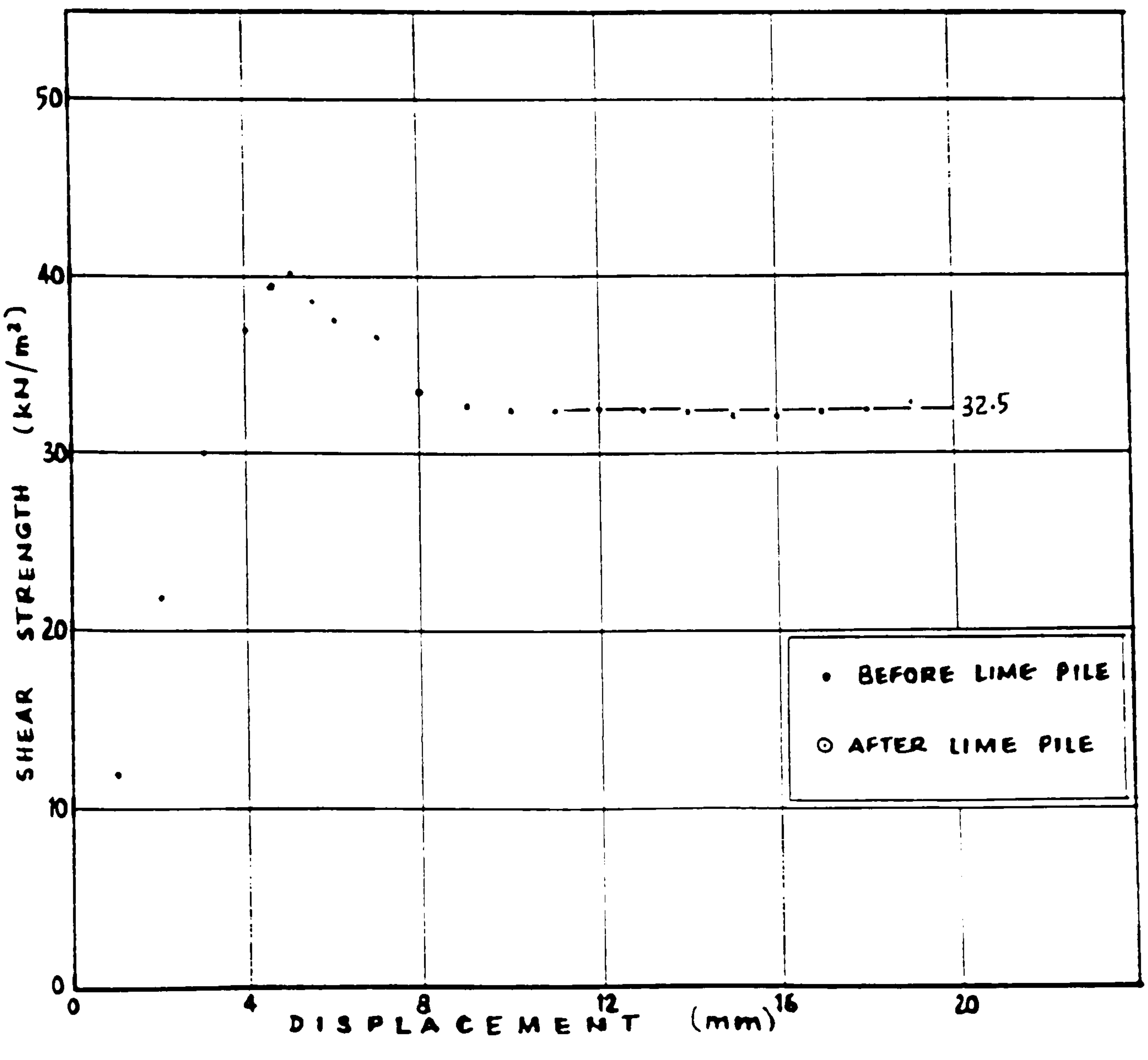

SHEAR STRENGTH (kN/m²)
0
10
20
30
40
50
0
4
8
12
16
20
DISPLACEMENT (mm.)
31.3
29.5
• BEFORE LIME PILE
⊙ AFTER LIME PILE


SHEAR STRENGTH (kN/m²)
0
10
20
30
40
50
0
4
8
12
16
20
DISPLACEMENT (mm)
32.8
25.0
• BEFORE LIME PILE
⊙ AFTER LIME PILE


PEAK 44.7
40.5
34.0
SHEAR STRENGTH (kN/m²)
50
40
30
20
10
0
0
4
8
12
16
20
DISPLACEMENT (mm.)
• BEFORE LIME PILE
⊙ AFTER LIME PILE


40.0
35.5
SHEAR STRENGTH (kN/m²)
50
40
30
20
10
0
0
4
8
12
16
20
DISPLACEMENT (mm)
• BEFORE LIME PILE
⊙ AFTER LIME PILE

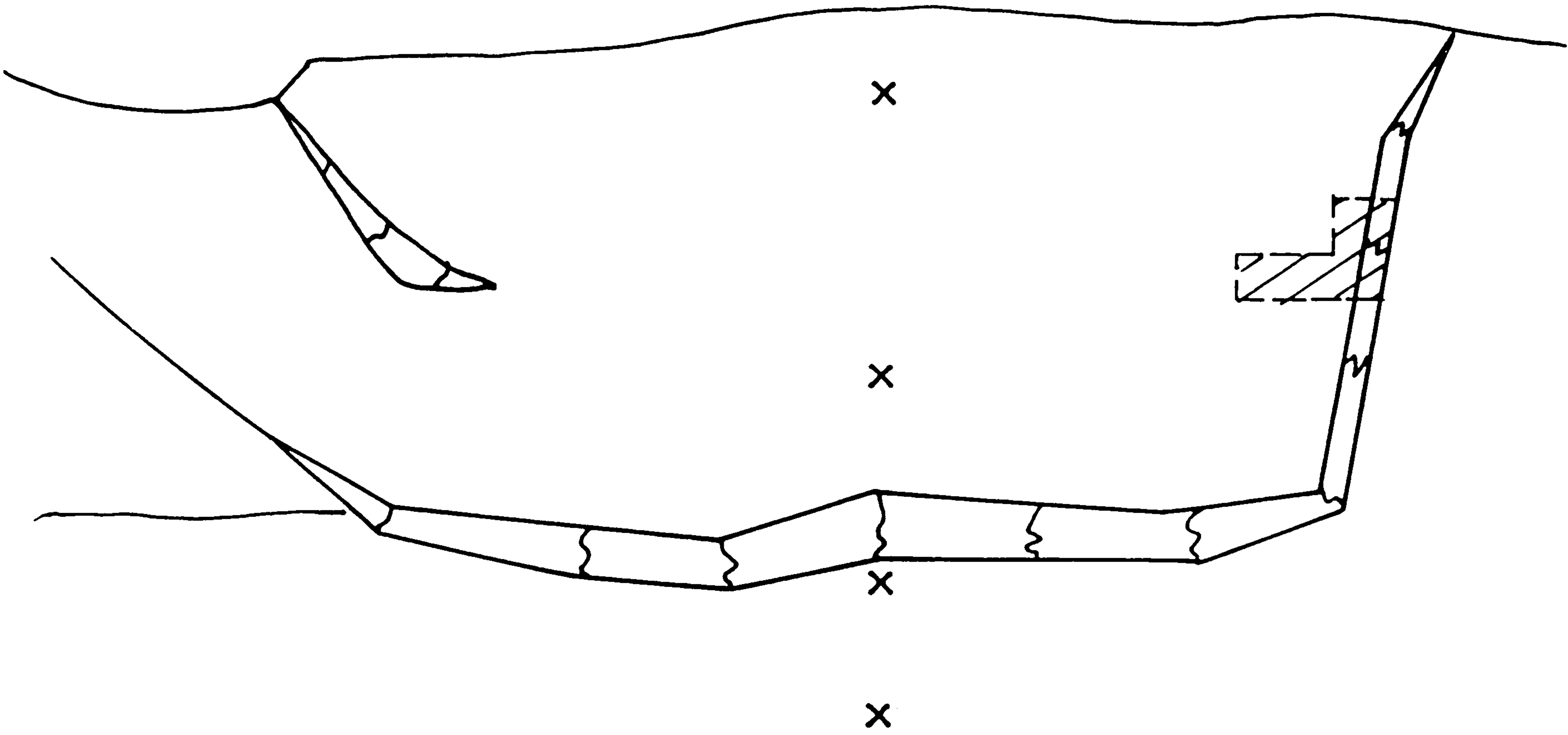

PEAK 50.1
40.0
36.0
• BEFORE LIME PILE
⊙ AFTER LIME PILE
SHEAR STRENGTH (kN/m²)
0
10
20
30
40
50
DISPLACEMENT (mm.)
0
4
8
12
16
20

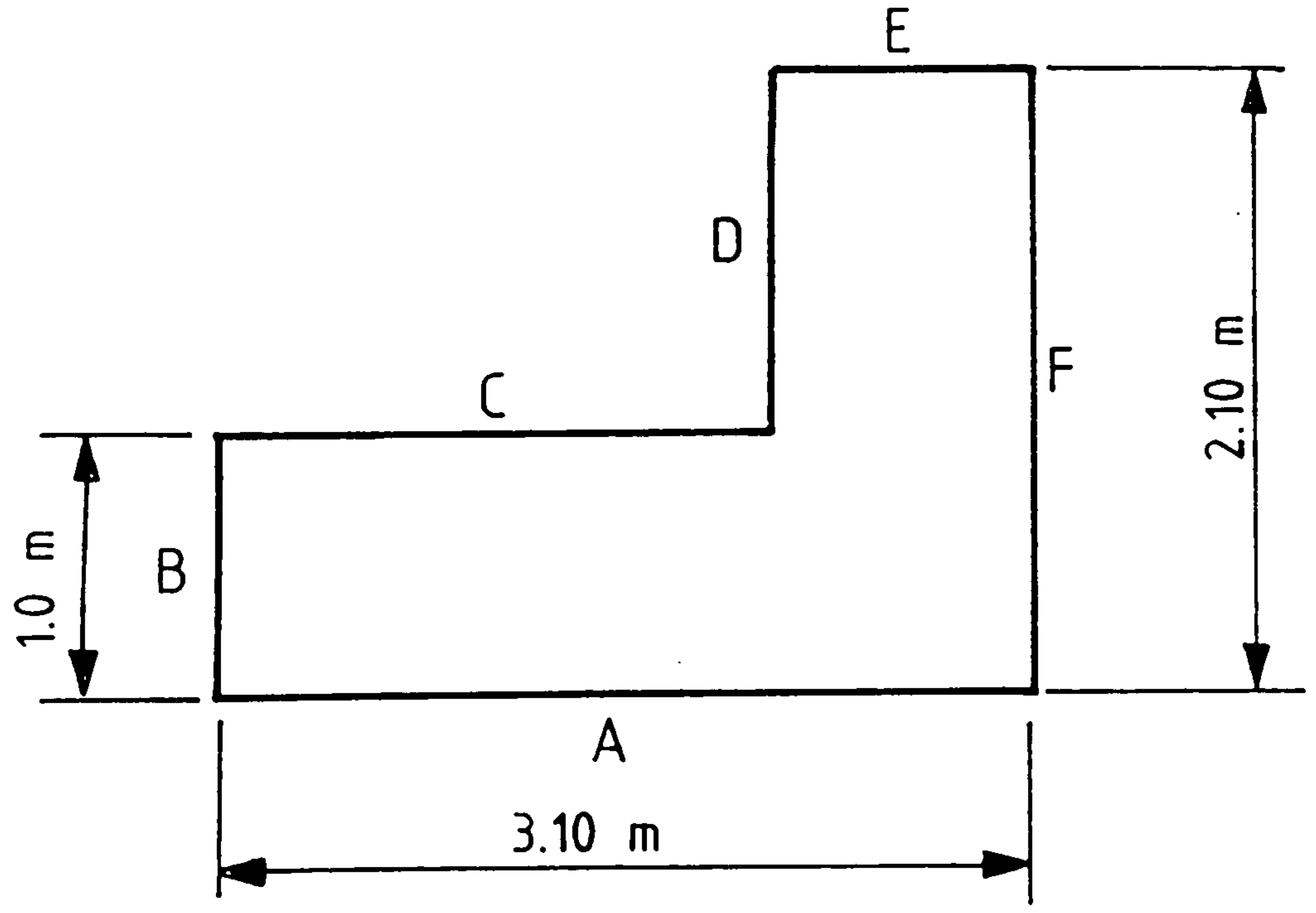

32.5
• BEFORE LIME PILE
⊙ AFTER LIME PILE
SHEAR STRENGTH (kN/m²)
0
10
20
30
40
50
DISPLACEMENT (mm)
0
4
8
12
16
20

APPENDIX 7

FENNY COMPTON TRIAL PIT DETAILS

Fenny Compton Large Slip Trial Pit





Fenny Compton - Small Slip Pit

25.9.91
Pit started approximately as per plan.

Notes:

~ 1m bands of gypsum anhydrite crystals within firm grey clay with occasional bands of red/brown staining.

Clay as above continues down to 3m becoming firm /stiff - stiff

1.4m -	fossil (belemnite) found
2.4m -	other fossils found
3m -	very soft patch perpendicular to cut on up hill face but very stiff at base.

Large Slip Pit

Refer to figures below.

1. Brown/grey topsoil/fill, containing numerous roots up to ϕ 2 cm. Clay pipe piece also found.

2. Firm to stiff thinly laminated very silty clay with orange/brown oxidation bands up to 5cm thick and spots of fine white powdery crystals (gypsum?). Oxidation bands contain numerous pebble sized iron-stone nodules.

3. Firm-stiff becoming stiff brown mottled grey silty clay with pockets of clear needle-like crystals.

4. Stiff grey mottled brown silty clay with thin laminations and clear crystals within the bedding planes. The clay often shows signs of oxidation accompanying the crystals. Occasional fossils.

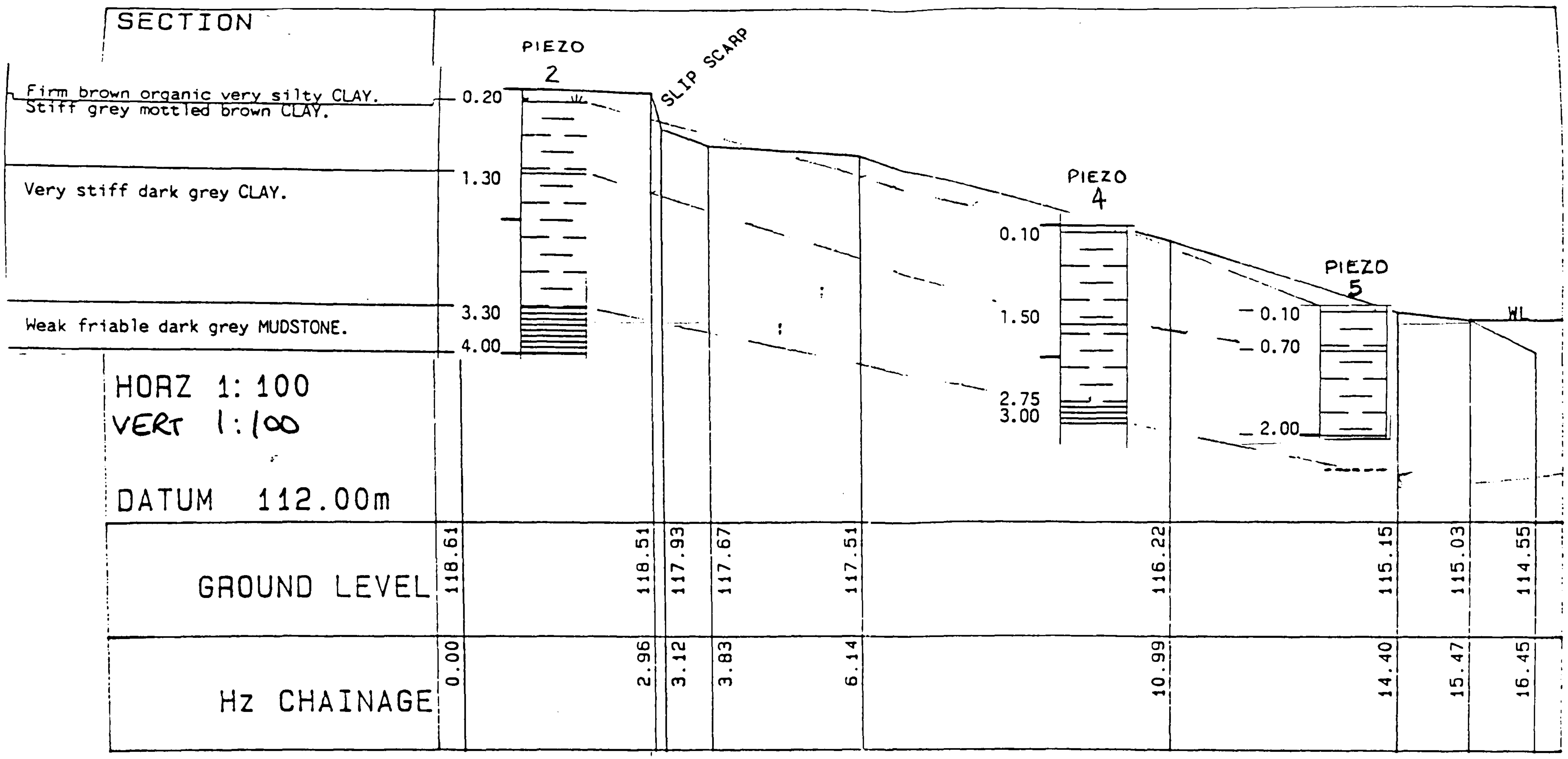
5. Hard grey very silty clay becoming weak grey mudstone in excess of 400mm thick.

NOTES: U100 taken at 1.2m in firm grey clay.
U100 failed at 1.7m due to clay being too hard.

Trial Pit Faces

APPENDIX 8

FENNY COMPTON FULL-SCALE TRIAL PIEZOMETER DETAILS



SECTION THROUGH PIEZO'S

Notes

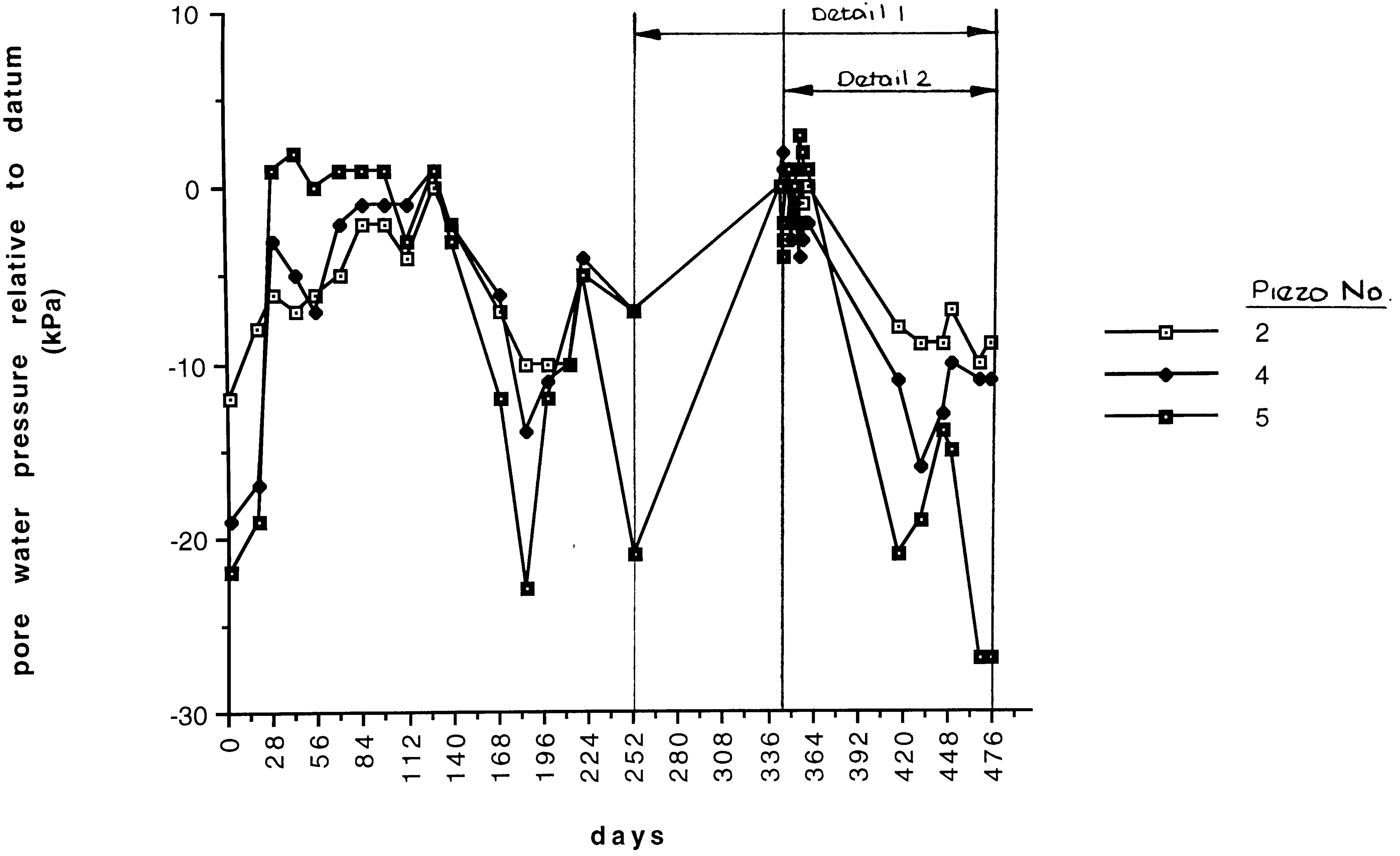
All tips installed at top of mudstone

Readout unit positioned directly above piezo 4

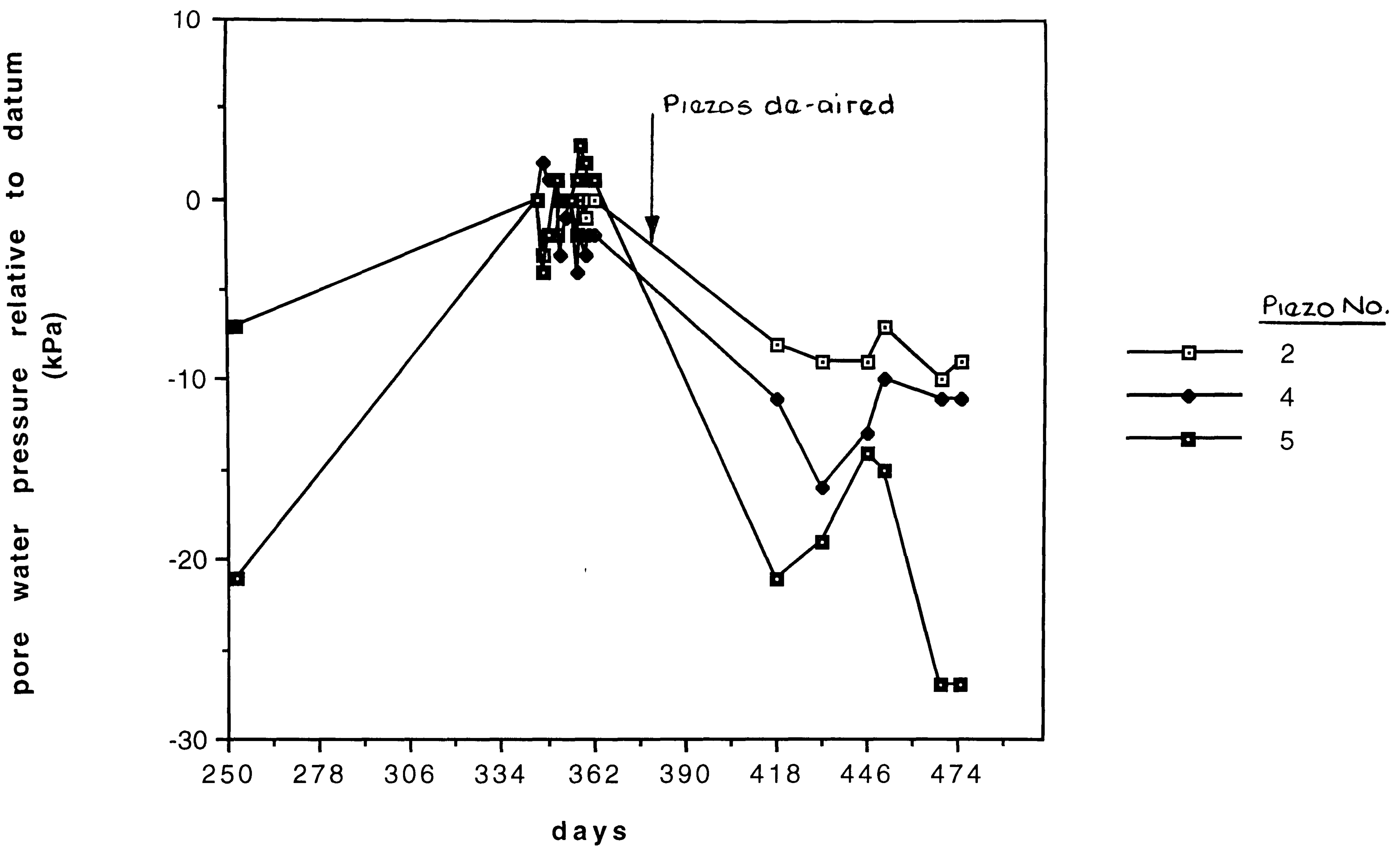
Datum level 116·70m AOD

Tip levels:
piezo 2 115·31m AOD
4 113·79m
5 113·15m

Fenny Compton Full-Scale Trial
Piezometer Data



Detail 1 of Fenny Compton Piezometer Data



Detail 2 of Fenny Compton Piezometer Data