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Strength Regain of Clay Slurry using Additives

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FINAL REPORT

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Summary

This report provides a comprehensive account of the data and observations gained from the research work carried out at the Department of Civil Engineering, Glasgow University on this 4-month contract funded by Technip UK Ltd from December 1st, 2009 to March 31st, 2010. The principal objective of the project was to determine by experiment which, if any, of a group of chemical additives were effective in accelerating the regain of strength of a marine clay following gross disturbance/remoulding by injection of water (and consequent increase in water content). The results of the test programme suggest that dosing with Calcium Hydroxide yields very promising results, and these results are presented at length. Other results are reported in the appendices and only briefly discussed herein.

Towards the end of the contract, further testing of the effect of the Calcium Hydroxide additive was undertaken at lower water contents than originally specified. In addition, some long-term tests (up to one year), which fall outside the scope of the contract, are currently being undertaken to establish whether the observed strength gain is permanent.

Literature Review

A wide-ranging search of the literature has been undertaken. Initially, this focussed on the literature relating to sedimentation and flocculation/coagulation. Subsequently, the focus shifted towards the modelling of soil strength and viscous properties and their measurement. And more recently, research relating to the improvement of soil properties and its chemistry has been reviewed. Thus, the sources encompass a spectrum of research across many disciplines: chemistry, colloidal and polymer science, oceanography, geology, geotechnical and mining engineering, amongst others.

The mechanics of sedimentation of clays and the chemistry/physics of coagulation/ flocculation are complex processes which are not well understood. By coagulation, we mean the destabilization of a colloidal suspension by addition of electrolytes, which reduce the forces of repulsion between the particles, while flocculation means the strengthening of inter-particle bonds. These are fertile areas of research and hence both natural and accelerated sedimentation of clays (and other materials, such as mine tailings and waste-water sludges) has a very extensive literature.

This review is subdivided into the following (non-exclusive) heads:

- a) Fundamental studies of settling
- b) Soil characterisation
- c) Measurement of soil properties
- d) Water and waste-water treatment
- e) Additives: Calcium hydroxide
- f) Additives: Polyvinyl alcohol
- g) Additives: Polyacrylamide
- h) Additives: Other
- i) Patents

a) Fundamental studies of settling

A substantial programme of research on the sedimentation of clays was conducted by Sills and her co-workers (e.g. Been & Sills, 1981). This work was done in the laboratory in settling columns, which were typically 100mm in diameter and up to 3m in height. Density profiles, and over time, were established using X-ray techniques. The data demonstrates, even for clays of moderate plasticity, that self-weight consolidation is only (effectively) complete after the passage of several months. The resulting sediments are, of course, very weak.

Numerical analysis of this highly non-linear problem has been undertaken by Papanicolaou & Diplas (1999), which appears to be capable of successfully modelling the results obtained by Been & Sills (1981) and others, albeit by making use of some unconventional material parameters.

Beris et al (1985) carried out a theoretical and numerical analysis of the problem of creeping motion of a sphere through a Bingham plastic: the results might be used to analyze experimental data of a falling sphere in order to determine yield shear stress.

An interesting experimental study by Gibbs (1985) demonstrates that the Stokes model, based on a spherical shape alone, is not a good model for settling velocities of clay minerals.

Gibbs encapsulates his data in simple empirical equations, such as the following for the terminal settling velocity (cm/s) of illitic minerals: $V = 0.95 D^{0.645}$, where D is the floc diameter (cm). For montmorillonite, the coefficient and exponent are 0.20 and 0.35, respectively.

The complexity of material models for clay suspensions is illustrated by, for example, the experimental work of Chafe & de Bruyn (1993) on the settling of a sphere in a very weak mixture (solids ratio of 6%). Interpretation of test data in 'semi-solid' mixtures presents similar difficulties.

b) Soil characterisation

Paul & Jobson (1991) describe the clays of the Witch Ground Basin: an area in the North Sea of some 25 000 km² situated 200 km north-east of Peterhead, and a similar distance south-east of the Shetland Islands. The upper strata were very soft (normally consolidated) silty clays of medium plasticity ($w_{ll} = 0.5$), which were only poorly recovered by borehole sampling above depths of 15m. They cite Bent (1986) who determined that the Witch Ground sediments show little variation and are dominated by illite, with subordinate chlorite and kaolinite. Smectites comprises a small proportion of the assemblage.

The characteristics of Bothkennar clay have been described by Hight et al (1992). Quoting earlier work, they report that the clay fraction is predominantly illite with kaolinite, while the silt fraction is predominantly quartz and feldspar. Again, quoting earlier work, they state that the high/very high plasticity of the soil is reduced to intermediate/high levels if the organics are removed. Further reduction in plasticity (amounting to 15%-20%) occurs if the soil is subject to air-drying.

Sharma & Bora (2003) propose that the undrained shear strength at the plastic limit should be fixed at 170 kPa and that the shear strength at the liquid limit be taken to be 1% of this value. It appears, except for highly plastic soils, that these results can be extrapolated beyond the liquid limit. From their data, and others, we suggest that the undrained shear strength of soils at $1.5w_{LL}$ is approximately 0.3kPa and falls to 0.06kPa at $2w_{LL}$.

The identification of soil strength and viscosity parameters from penetration tests depends on the assumed material model. Thus, for example, Raie & Tassoulas (2009) make use of the Herschel-Bulkley model (an advance on the Bingham model) to analyze the large scale penetration problem of torpedo anchors into marine sediments.

Finite element calculations using strain-rate dependent elastoplasticity has been used by Aubeney & Shi (2007) to analyze experimental data on the penetration of (horizontally oriented) cylinders in soft clays.

c) Measurement of soil properties

Fakher et al (1999) review the various techniques available to measure the strength of very soft clays (strength less than 0.1kPa). Test data from the literature suggest that there is considerable scatter of measured strengths at the liquid limit, although in principle the liquid limit should correspond to a specific strength value. Their own measurements of shear stress, at various strain rates, using a viscometer also reveal considerable scatter, which renders extrapolation of the data to determine the yield shear strength at zero strain rate exceedingly problematic. An interesting point of their work is the observation that it appears that there is a much clearer transition from plastic to liquid at water contents of 1.5-2 w_{LL} rather than at the Liquid Limit itself.

Chung et al (2006) explore the effect of penetration rate on soil resistance, in a study which aimed to isolate the separate effects of viscosity and consolidation, which in most earlier studies has been glossed over.

Mahajan & Budhu (2008) instrumented a fall-cone with an LVDT and high-frequency data-logger to measure the time-displacement response during its penetration. From these data, they computed the "shear viscosity". However, the results are open to various interpretations depending on the material model assumed.

Tan et al (1991) describe a novel method of measuring the yield stress of very soft soils by measuring the shear resistance to penetration of a thin plate. The range of the device appears to fall in the gap (between 10 and 100 Pa) between the viscometer and fall-cone devices.

d) Water and waste-water treatment

Coagulation and flocculation in water and waste water treatment is described in detail by Bratby (2006), while many authors (e.g., Droste, 1997) provide a more easily accessible account of the general principles. A significant conclusion which may be drawn from this body of work is that process efficiency is an empirical science, requiring the balancing of conflicting factors. However, the primary goal in wastewater treatment applications is to increase the solids ratio (defined as the ratio of weight of solids to the total weight of the suspension) to 20% or less, at which point mechanical dewatering techniques may be employed to deal with the residue. But such solids ratios are equivalent to water contents of four (4) or more; significantly higher than our starting point of (typically) twice the Liquid Limit water content. Although these works hint at an avenue of approach, and the complexity of the factors which impinge on these processes, the focus on far lower solids concentrations limits their relevance to this study.

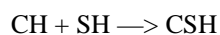
e) Additives: Calcium hydroxide

Paige-Green et al (1990) discuss the mechanism and chemistry of stabilisation reactions, and their permanence or otherwise. They refer to the well-known observation that the edges of clay platelets may be attacked by alkaline additives and the ionic balance of the platelet disrupted, which results in significant restructuring of the platelets and an overall change in the soil structure. In addition, the alumina and silica components of the clay minerals are more readily dissolved at the high pH values and may react with the calcium ions to form hydrated calcium silicates, and possibly other cementitious products (i.e., a pozzolanic reaction). The ion exchange and flocculation/aggregation reactions occur relatively quickly while the pozzolanic reactions are fairly slow (i.e., measurable in days). The pozzolanic process depends on the presence of siliceous or aluminosiliceous material which reacts with calcium hydroxide: thus, the higher the content of these materials the stronger is the pozzolanic reaction.

At the basis of the pozzolanic reaction stands a simple acid-base reaction between calcium hydroxide (Ca(OH)_2), and silicic acid (H_4SiO_4 , or Si(OH)_4). This reaction can be schematically represented as follows:



or summarized in abbreviated form as:

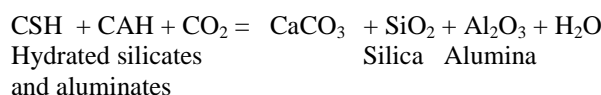


The product ($\text{CaH}_2\text{SiO}_4 \cdot 2 \text{H}_2\text{O}$) is calcium silicate hydrate but other reactions may occur too. As the density of calcium silica hydrate is lower than its constituents, a consequence of this reaction is a swelling of the reaction products (well-known in construction as a cause of spalling in concrete, viz., the alkali-aggregate reaction).

To ensure the stability of the reaction products in this process, Paige-Green et al (1990) state that sufficient CH should remain in the material in order to maintain the pH of the material at a level of at least 10 and preferably 11. Only 50ppm dissolved in the pore water is required to give this pH. However, some species of silica hydrate (e.g. C3S2H3), which may have a more significant role in strength gain, require even higher pH values (requiring 1600ppm) for stability. The implications of this that are that leaching of CH over time in a permeable material will lead to strength loss

The chemical process which leads from CH to calcium carbonate has been termed 'carbonation' for convenience. Although carbonation does promote cementation in soils, the bonds are weak compared to those produced by the pozzolanic reaction and, therefore, carbonation is best avoided.

Decomposition of hydrates follows this schematic, liberating water:



The implication here is that carbonation requires the presence of carbon dioxide, either from the atmosphere or, in this context, as a by-product of organic decay.

However, although cementation resulting from carbonation is considerably weaker than that which can be achieved by pozzolanic processes, nevertheless in absolute terms significant strengths (measured in hundreds of kPa) appear to be achievable. A note that significant quantities of salts (particularly sulphates), and reactive organic matter, may be harmful to stabilisation is pertinent.

Rajasekaran & Narasimha Rao (2002) briefly review earlier work on the lime treatment of marine clays. In their large-scale laboratory tests, they show that the compressibility of a soft plastic clay can be reduced by half by injection of a lime slurry. The improvement is essentially complete after 30-45 days. Citing their earlier work, (Rajasekaran, 1993) they claim that the cementation compounds are quite stable and the reaction is considered to be irreversible. However, they caution against applying these techniques to marine clays that contain sodium sulphate.

Janz & Johansson (2002) summarize the state-of-the-art of chemistry of binders used to increase the strength of soils. They note that the humic acids which are frequently present in organic soils inhibit the pozzolanic reactions between calcium hydroxide and pozzolanic minerals and thus render such treatments ineffective. They also note that if all the pozzolanic minerals in a soil are consumed by reaction with Ca(OH)_2 , further quantities of this additive will not provide any further strength gain.

Also of interest is the observation that pozzolanic reactions are negatively impacted by low temperature. Thus (they remark) if a test is carried out under laboratory conditions at 20C, there is a risk that that pozzolanic reactions will contribute far more to the strength gain in the test than they will in the field. Thus, laboratory test can give misleading results. In the North Sea, it is presumed that the ambient temperature will be c.4C. They also observe that mixing under laboratory conditions tends to be considerably more efficient than in the field and that this too can give rise to misleading results. In their remarks on the durability of binding agents, they note that leaching by ground water movement can have deleterious consequences, as calcium hydroxide is dissolved and calcium is liberated from the CSH gel. This may be relevant to this project, if thermally driven flows occur around the pipeline. The potential for chemical attack due to ammonium and magnesium salts, which react with hydroxide ions, should also be considered.

An early study by Andersson (1960) shows data which suggests that dosing soils (of high water content) with more than about 3% CaOH₂ yields no further increase in the Atterberg “consistency factor” (which might be interpreted as strength). Other results show that the pozzolanic strength gain is much faster and more pronounced at elevated temperatures: 7-day tests conducted at 43C and 63C yield strengths an order of magnitude greater than that obtained at 21C. However, long-term tests at 21C show a progressive increase in strength up to 6 months, to levels comparable to those attained in the 7-day tests at elevated temperatures.

Rajasekaran et al (1997) studied the reaction products of soil and calcium hydroxide using X-ray diffraction and scanning electron microscopy techniques. These confirmed the presence of calcium silicate hydrate and calcium aluminate hydrate, which bind the soil particles together. They state that these products can be in a gel state initially but then become crystalline over time. The presence of seawater does not retard the formation of these products. Addition of sodium hydroxide to the mix promotes the development of pozzolanic compounds compared to dosing with calcium hydroxide alone.

f) Additives: Polyvinyl alcohol

Ali et al (1984) measured the viscosity of mixtures of polyvinyl alcohol and Na smectite at pressures up to 1000 bar using Couette and falling-body-type viscometers. The viscosities were found to bear an exponential relationship to polymer concentration, and also increased exponentially with pressure (although the latter effect is insignificant at the pressures relevant to the current project). The soil suspensions contained only 0.5% – 3.0% clay. The test results show that beyond a certain polymer/solid ratio, the increase in viscosity slows, perhaps due to the fact that all the negative sites on the clay surface become occupied by the polymer molecules, which are adsorbed on the clay surface through hydrogen bonding. At high solids content, a modification of Arrhenius’ law proposed by Weltmann & Green (cited by Ali et al) appears to provide a better description of the increase in viscosity with solids content. This appears to be due to the fact that clay suspensions exhibit Newtonian behaviour only up to a certain solids concentration: at higher solids concentrations, plastic flow behaviour occurs.

An indication of the complexity of the physico-chemical processes at work is that three separate bonding mechanisms are mentioned via which this non-ionic polymer could be attached to the clay surface, namely, (1) ion-dipole interaction, in which metal-ligand bonding exists between exchangeable cations and the polymer, (2) bonding through a water bridge to exchangeable ions, and (3) attachment of polymer segments to the surface of the clay. From their data, the authors suggest that the first mechanism is the most probable, which would be consistent with earlier findings that PVA forms O-H-O-Si hydrogen bonding with smectite surfaces. The practical implication of the identification of the precise bonding mechanism is not apparent.

Liu & Hoffmann explored the behaviour of mixtures of clay suspensions (at concentrations of up to 1%) PVA and various surfactants.. They found that the clay minerals bind strongly with PVA and can adsorb up to five times their own weight of this additive.

g) Additives: Polyacrylamide

Proskurina et al (2006) showed that various polyacrylamides were effective accelerants of “hindered” sedimentation of a 4% bentonite solution. Yoon & Deng (2004) investigate the efficacy of various polymers, including polyacrylamides, but on suspensions containing as little as 0.2% bentonite.

O’Gorman & Kitchener (1974) investigated methods of dealing with thick slurries of Kimberlite clay, which is composed of a predominantly smectite mineral, known as saponite. Their discussion of the mechanics and chemistry of coagulation and flocculation is particularly accessible, and their observations on the limitations of theoretical models to guide the choice of coagulant/flocculant is instructive. They show that Kimberlite can be induced to coagulate by the addition of salt, which produces a very open (“house of cards”) structure due to the promotion of edge to edge contacts and edge to face contacts, rather than face to face contacts. They found, on tests carried out mainly at solids concentrations of c6%, that a combination of magnesium sulphate and an anionic polyacrylamide produced the best results, but that optimal results were obtained only in a narrow range of conditions.

h) Additives: Other

Chang et al (1997) describe a comprehensive series of tests carried out using alum on a clay suspension containing (a very high) 20% clay solids. The results confirm that pH as well as dosing levels has a significant effect on settling velocities.

Tan et al (1990), in one of a series of papers on the geotechnical aspects of soft soil deposits in Singapore, describe experiments carried out on clay slurries ($w_{ll} = 0.65$) to accelerate strength gain. Their work concentrates on slurry suspensions with water contents in the range 4-20 w_{ll} . Thus, the full range of settling modes, namely: particulate (Stokes) settling, hindered settling and self-weight consolidation is applicable. It was observed that sedimentation was accelerated if the clay was mixed with seawater rather than freshwater, but that dosing with a synthetic non-ionic polymer was even more effective. However, they also noted that there appeared to be an optimum dose beyond which any increase in polymer concentration resulted in a decrease in settling rates. Shear strength in these tests is interpreted as the “yield shear stress” which is obtained by extrapolating (to zero shear stress rate) the measurements of shear stresses (obtained from a viscometer) for a range of shear rates. The data broadly conforms to the Bingham model, which permits linear extrapolation of these data.

Tao et al (2008) report on research to accelerate the disposal of clay waste (from phosphatic mine tailings), which amount to 3Mtonnes/month in Florida. They describe the effectiveness of various flocculants to achieve solids contents up to c30% before mechanical dewatering takes place. Their results are not easy to interpret but they concluded that precise control procedures were critical for paste formation. For example, they found that an anionic flocculant had to be added to the slurry before a second (cationic) flocculant for best results. Slurry pH had significant effect on flocculation response.

i) Patents

In their patent application, Myoshi et al (1984a) state that a high water content soft soil, for example from the sea or river bed, may be improved in strength by incorporating therein an additive composed of two ingredients: (A) 5-45% by weight gypsum (calcium sulphate dihydrate) and 95-55% water-granulated fine-grained iron blast-furnace slag and (B) Portland cement. Optimum concentration of gypsum in ingredient A is between 15-35%, while the optimal concentration of Portland Cement in the combined additive is circa 1/3rd. In an example test case, a soil at a water content of 2.6 (sic) was treated with 67kg/m³ of the additive (additive/solids ratio = 20%). The plasticity of the soil is not given. This treatment produced a 7-day undrained shear strength in excess of 100 kPa after 7 days and 170 kPa after 28 days.

In another patent application, Myoshi et al (1984b) advocate treating the blast-furnace slag using sulphuric acid, thus converting part of the calcium components into gypsum, which is no longer required as a separate ingredient.

Some of the research work done on the Florida phosphatic clay slimes is protected by various patents. For example, El-Shall (1993) patents a method involving a micro-fibre matrix used in conjunction with a solution of polyacrylamide.

Soil Medium

As a model for the Witch Ground clays, we have used (on advice) samples of Bothkennar clay (with the addition of a small amount (8%) bentonite, in order to better replicate the plasticity of the Witch Ground clays: $w_{LL} \approx 50\%$). The characteristics of Bothkennar clay have been described by Hight et al (1992) as reported above. The water content at the Liquid Limit of Bothkennar clay and bentonite and mixtures of them are given in Table 1. Further experimental details are given in the Appendix (A11). Figure 1 for example, taken from Appendix A11, shows the results of a Liquid Limit determination for a mixture containing 10% bentonite.

Note that these tests were not carried out using sea-water. It may be seen that the liquid limit data for Bothkennar clay corresponds reasonably well with those reported by Hight et al - for soil which has been subjected to air-drying and removal of organics.

Prior repeated use of this material in the laboratory, which has included air-drying and subsequent grinding into fines, is thought to have depleted the organic content. The Department held a good stock of this material (approximately 150-200 kg), although much of it has now been consumed by this project. It should be noted that although the material has been homogenized by prior use, it is likely that some samples will deviate from the norm, in plasticity and/or organic content.

Table 1. Water content at the Liquid Limit of Mixtures of Bentonite and Bothkennar clay

% Bentonite	% Bothkennar Clay	% water content at LL
0	100	37
100	0	162
7	93	48
10	90	53

A supply of 3.5% salt solution (prepared by dissolving natural sea-salt in de-ionized water) has been used throughout, barring some early proving trials. Salt content has been verified indirectly using an EC meter: standard values of the conductivity of sea-water are 48mS/cm at 20°C, falling to 38mS/cm at 10°C.

Measurement of Soil Properties

Measurement of soil properties implicitly presumes that some plausible and useful material model exists in the first place. For example, the concept of an undrained shear strength in soil mechanics presumes that there is an identifiable (static) stress state at which identifiable failure occurs. Even in routine geotechnical conditions, these concepts can be hard to define. For example, it is well-known that measured strengths are strain-rate dependent and so standardized test procedures (strain-rates) must be followed to ensure that consistent results are obtained. The origins of strain-rate dependency are complex and poorly understood since they must include, amongst others, drainage (consolidation) as well as the viscoelastoplastic behaviour of the pore fluid and solid phases.

In soft soils (with high water contents), these problems are magnified by the very visible evidence of increasing strain under static loading. To characterize this behaviour as “creep” does not advance the argument very far. Thus, in this report, where it has proved convenient to characterize the material resistance to loading in terms of an undrained shear strength, some caution in interpreting these data is warranted.

The undrained shear strength of soils can be measured by various means which are well-known in the geotechnical field: e.g., the triaxial test, vane test, various cone and penetrometer tests, etc. In this report, a correlation is normally assumed in order to compute undrained shear strength from measurements of bearing capacity, allowing only for simple depth-dependent corrections (Houlsby & Martin, 2003, Salgado et al, 2004). Presentation of the bearing capacity data may permit, if desired,

better estimates to be made at some later date, taking into account other factors such as heave of the surrounding soil, boundary effects (cell wall), slumping of the cavity, etc.

Undrained strength determination explicitly assumes, of course, undrained conditions. In any free-field test (which includes all those mentioned above, bar the triaxial test) undrained conditions demand that testing be carried out at an “appropriate” loading rate, as described below. This can create considerable difficulties.

At very high water contents, these problems become more acute and generally recourse is had to various material models, of a phenomenological type, to characterize viscous behaviour. The existence of various models in the literature is an indication that none is entirely satisfactory and, moreover, it appears to be difficult to bridge the gap between viscous fluid and solid matter. From an experimental perspective, measurement of soil viscosity (as reported in the literature) also appears to be difficult, exhibiting wide scatter. Thus, in this project, no attempt has been made to measure “viscosity”, however defined. For the very weak materials tested in the early stages of this project, and which have consequently been excluded from further consideration, a simple comparative test (based on the velocity of a falling disk) provides a ready means to discriminate between mixtures of differing viscosity.

Experimental Apparatus

From the outset, it was envisaged that the testing of soil samples would be undertaken in 100mm diameter cylinders, following the example of the work of Sills’ group. However, a reappraisal of the significance of settling (or rather its lack of significance) during the early stages of the project led to the decision to eschew use of cylinders of high aspect ratio. Thus, for the most part, cylinders with a height of 150mm have been employed although some longer (300mm) cylinders have also been manufactured. The use of smaller test samples has also allowed testing to continue without having to secure fresh supplies of the soil. Depending on water content, the weight of solid material consumed per test varied from c0.5kgf to 0.9kgf.

At the outset of this study, the range of strengths (or viscosity) of the soil samples tested was expected to span several orders of magnitude (i.e., from viscous fluids to soft soils). It was realized that the proposed measurement of soil strength by means of a cone device could not have delivered satisfactory results for the weaker (viscous) mixtures. Thus, for these materials, a test was devised in-house in order to measure the viscous resistance of the mixtures to a falling weight.

The principal elements of the apparatus were a Panasonic NV DS30 video camera, capable of capturing images at 20ms intervals, and a thin weighted falling disk. The latter was constrained to fall vertically by means of a (virtually friction-free) guide frame. Interchangeable disk diameters (of 35mm, 50mm and 70mm) extended the range of mixture viscosities which could be tested. No attempt was made to identify a specific material (viscous) parameter (which entails some assumption of a material model) but rather the test was used to screen candidate additives by comparing resistances, and to assess the effects of changing additive concentrations.

The falling disk trailed a pointer which fell past a vertical scale. Following each test, the video stills were projected on to a large screen and manually inspected frame by frame. This allowed the displacement–time trace to be constructed over a distance of c100mm. Displacements were recorded to the nearest 1mm, which was sufficiently accurate to establish a smooth curve to the eye but insufficient for the purposes of repeated numerical differentiation (to compute accelerations). This problem was resolved by computing a polynomial least-squares fit of the displacement-time trace, and computing the acceleration (and velocity) traces analytically. After some experimentation, it was decided to adopt the lowest order of polynomial (namely, the 4th order) which gave satisfactory results: the results were indistinguishable from 5th order.

Although highly viscous (strong) mixtures could be tested in the same way (by reducing the disk diameter and/or increasing its mass), a more useful material parameter, namely strength, can be measured by adapting the apparatus to accommodate static loading. Although the bearing pressure of a disk is less readily interpreted in terms of shear strength than the cone device initially envisaged, useful

estimates of strength may be inferred provided that due consideration is paid to the boundary conditions and loading rate, etc. Ideally, disk penetration should be displacement controlled. From research work on spud-can foundations, Finnie(1993), quoted by Hossain et al (2005), suggests that undrained conditions are satisfied if the velocity of penetration exceeds $30C_v/D$, where C_v is the coefficient of consolidation and D is the diameter of the foundation. Assuming, $C_v = 0.05\text{mm}^2/\text{s}$ and $D = 30\text{mm}$, this yields the (slow) velocity of 3mm/minute. However, a displacement-controlled loading rig was not available for the purposes of this project. Except where the coefficient of consolidation is unusually high (e.g., for very weak mixtures, with undrained shear strengths considerably less than the strength at the Liquid Limit), it is expected that load-controlled tests can deliver useful estimates of undrained shear strength. Even for very weak mixtures, the load-controlled test will provide an effective means to screen candidate additives and additive concentrations.

Additives

A range of chemicals has been employed, mainly selected from the literature relating to waste-water treatment. Potentially useful additives, but which carry significant health risks, such as polyacrylamide, have not been employed. The additives selected for experimentation were: calcium hydroxide, sodium alginate, aluminium sulphate, aluminium potassium sulphate, potassium sulphate, chitosan and polyvinyl alcohol.

These were used singly (rather than in combination) and without modification of the resulting pH of the clay/additive mixture. Thus, for example, the pH of the 5% PVA mixture is 8.1. The selection of additive concentrations has been informed by data from the literature: generally, this results in additive concentrations of the order of 1% , but for each additive a range of concentrations (e.g., 1%, 2% & 5%) was explored. All but polyvinyl alcohol and calcium hydroxide readily dissolve in water: even at low solids concentrations, the former produces a gel while the latter produces a milky suspension. Nevertheless, this is not believed to prevent effective dispersion of the additive through the soil.

Discussion of Test Results

1. Introduction

An extensive series of tests has been conducted, beginning with benchmarking tests to verify that the experimental techniques used in this project delivered accurate results. For example, the acceleration due to gravity was computed from the observed time-displacement response of the falling disk in air. For brevity, these benchmark tests are not recorded here. During the experimental programme, a number of tests were repeated to ensure that repeatable results were obtained: these are documented as they arise.

For completeness, most of the tabulated experimental results (and the associated figures) are contained in the appendices (in Excel format), while only the main points of significance are summarized in this report. However, the results using the Calcium hydroxide additive showed much greater promise than any other and these results are described at length here.

In the following, the results are described in chronological order. Except where stated otherwise, all tests have been conducted at a water content of twice the Liquid Limit and in water containing 3.5% sea salt in solution.

2. Additive-free clay

A series of 1-day, 3-day and 7-day viscosity tests was carried out on additive-free clay (Appendices A21, A22 & A23), using the 50mm disk. In these early tests, a 4.6% solution of sea-salt (rather than the standard 3.5%) was used, although this is not believed to have affected the results significantly. The results show that a peak velocity of 350mm/s is attained after one day, falling to 190mm/s after 3 days and 140mm/s after 7 days. This is evidence of an overall increase in viscosity of the clay over time.

Figures 2-4 are indicative of the 1-day results. Figure 2 shows the displacement trace over a period of nearly 0.4s, with data captured at 20ms intervals as described previously. A fourth-order polynomial through the data points was then determined using least-squares regression: it is this curve which is then used to generate (by analytical differentiation) the velocity and acceleration profiles which follow (Figures 3 & 4). The test was repeated (as shown) and provides evidence of very good repeatability.

As can be seen, the velocities reached are not terminal velocities (the samples were insufficiently deep to permit development of terminal velocities) and therefore these results need to be treated with caution. The distance of travel in each case is also different because the disk was lowered through the supernatant water before being allowed to drop through the underlying sediment.

Later, strength tests, (i.e., under static loading) were conducted, most recently also at $1.5W_{II}$. The original interpretation of the strength tests conducted at a water content of $2W_{II}$ in terms of undrained shear strength are now assumed to be mistaken – as demonstrated by the finding that the ultimate bearing pressures are little affected by water content. It appears that the loading rate is too slow at these water contents to prevent consolidation (i.e., drainage).

The detailed data (bearing pressure profiles, etc), obtained from these strength tests are given in Appendix A24 but the principal results, which show the bearing pressure profile with depth, are collected in Figure 5. In this figure, the depth datum is the elevation at which the disk remains stable under its own weight. This varies between approximately 10 to 25mm below the soil surface. Thus, the initial bearing pressure in all cases is either 0.4kN/m^2 (70mm disk) or 0.7kN/m^2 (50 mm disk). Clearly, if the loading was undrained, then the bearing pressures sustained by a soil with a water content of $1.5W_{II}$ should then increase with depth at faster rate until they become several times greater (perhaps 5 times greater, Sharma & Bora, 2003) than the bearing pressures in the soil with the higher ($2W_{II}$) water content. In fact, as can be seen in Figure 5, water content has very little effect. The data does however lend itself to a simple bearing capacity analysis assuming a drained response, which yields an effective friction angle of circa 20° .

Nevertheless, it will be useful to establish the undrained shear strengths of these weak soils in order to compare them with the much stronger mixtures obtained after additive dosing. As noted earlier, tests reported in the literature suggest that the undrained shear strengths of soils at $1.5W_{II}$ and $2W_{II}$ are of the order of 300Pa and 60Pa respectively.

But in the long-term, the undrained shear strength of normally consolidated soil sediments can be estimated to be

$$C_u \approx 0.3\sigma'_z$$

which becomes:

$$C_u \approx 2z$$

where C_u is expressed in terms of Pascals and z is the depth, in mm. Thus, an upper bound for the undrained shear strength in the test cells used in this project cannot be much greater than 200Pa (i.e., 0.2kPa) at the base of the cell and half this value at mid-depth. Data from Been & Sills (1981) suggests that these strengths can not be achieved before the passage of several months.

For completeness, we now consider the rates of consolidation observed in some of these tests and compare them with the rates predicted by theory. According to McNamee & Gibson (1960) and Zhang

et al (2009), the time (t_{50}) taken for 50% consolidation to take place under a vertically loaded disk of radius r is approximately:

$$t_{50} = 0.2r^2 / C$$

where C is the “coefficient of consolidation”. The value of the numerical coefficient (here taken to be 0.2) depends on the boundary conditions and embedment etc. (Compare $t_{50} = 0.2H^2/C$ in one dimensional consolidation theory, where H is the drainage path length). Observations of tests conducted at $1.5*W_{ll}$ and $2*W_{ll}$ using a 50mm diameter disk reveal that t_{50} is approximately 21s (Figure 6) and hence, $C \approx 6 \text{ mm}^2/\text{s}$. The settlement measurements were captured manually at 15s, 30s, 60s and 120s, and give rise to some experimental scatter as shown in Figure 6. A linear relationship between settlement and the square root of time, at early times, is observed but water content appears to have no significant effect on the rate of consolidation.

The computed value of the coefficient of consolidation seems to be extraordinarily high: for example, the US Navy Manual (1971) suggests that an upper bound* for remoulded soils with $W_{ll}=0.5$ is $0.04 \text{ mm}^2/\text{s}$, and $0.16 \text{ mm}^2/\text{s}$ for undisturbed normally consolidated soils. However, the present test conditions are unusual both in terms of the very high water content and the stress state (i.e., at a state of plastic flow), and therefore classical rate of consolidation theory may be of limited applicability.

(*Appendix A0 contains some calculations which support this upper bound value)

3. Aluminium Sulphate (“alum”)

The earliest set of tests on additives was carried out using aluminium sulphate (“alum”) as an additive at concentrations of between 1% and 5% (and over durations of 1-,3- and 7-days). The results showed that the optimal additive concentration was circa 2-3%. These tests generated significant quantities of gas over the first few days, believed to be carbon dioxide, but which then was largely dissipated over the next few days. As a consequence, some difficulties were experienced in assuring repeatable test results at 3 days. At that stage in the project, the measurable increase in viscosity of the mixture was noteworthy, but in practical terms the strength increase (estimated to be of the order of 100Pa) is inconsequential. This series of tests was therefore terminated. (The experimental data for this set of tests is available in hard copy and will be made available separately with the hard-copy version of this report.

4. Aluminium Potassium Sulphate

The results (see Appendices A41, A42 for full details) obtained using aluminium potassium sulphate were the first in which loss of strength over time was observed. Moreover, the strength loss was, essentially, total. This unexpected observation prompted us to repeat the entire series of tests in order to verify that the phenomenon was indeed real and not attributable to laboratory error.

Tests conducted at concentrations of 1.25%, 2.5% and 5% (see Figure 7) showed that an optimal dose of 2.5% additive produced the greatest strength gain in the 1-day tests, yielding an undrained shear strength at mid-depth of $c0.3 \text{ kN/m}^2$. (Given the probable deviation from undrained conditions in such weak sediments, the measured bearing capacity data shown in the Figure should be given greater weight than estimates of undrained shearing strength). The mechanisms which give rise to an optimal dose concentration of $c2\%$ are not known.

The repeatability of the test results at 1.25% concentration (shown in Appendix A41) was poor and indeed among the worst encountered in any test. Nevertheless, the disparity between the test results was not sufficient to mask the significantly larger effect of changes in additive concentration. By contrast, repeatability of the tests conducted at 2.5% additive concentration was good (shown in Appendix A42).

Remarkably, in the 3-day tests, all the strength gain was lost and the mixture became as weak as the original additive-free clay. The explanation for this phenomenon is unknown: it suggests that the (weak) short-term bonding is neutralized by a slow reaction which changes the chemistry of the materials. This series of tests was terminated.

5. Chitosan

Chitosan is a natural product used in water-treatment as a flocculant, amongst other uses. One-day tests were conducted at 0.5%, 1% and 2% concentrations (see Appendix A51 for full details) and showed that the additive had virtually no effect on the viscosity of the mixture. This result is apparent by inspection of Figure 8 which shows the velocity trace for these tests. A repeated test at 0.5% concentration (shown in Appendix A51) demonstrated very good reproducibility. Tests using this additive were discontinued, given that the one-day tests had failed to indicate any significant gain in strength.

6. Sodium Alginate

The chemical compound sodium alginate is the sodium salt of alginic acid and, like chitosan, derived from natural products. It is used commercially as a flocculant in water treatment and also as a thickener in the food industry, amongst other uses. One-day tests were conducted at 0.5%, 1% and 2% concentration (see Appendix A61 for full details) with very similar (null) results to that obtained using chitosan. This is illustrated by the velocity trace which is reproduced in Figure 9. One of the tests (at 0.5% concentration) was repeated and, again, yielded very good reproducibility (shown in Appendix A61). However, on the evidence that there was no significant effect of the additive on viscosity after one day, it was decided to discontinue testing this additive.

7. Potassium Sulphate

One-day tests were carried out using potassium sulphate as an additive in concentrations of 1.25%, 2.5% and 5% (see Appendix A71 for full details). These tests, illustrated by Fig. 10, showed that increasing the additive content increased the strength of the mixture, although the gain in strength between 2.5% and 5% was relatively small, which suggests that higher doses would yield little further strength gain. Repeatability, at a concentration of 1.25%, was good (shown in Appendix A71). But the gain in strength due to the additive was considered to be far too small to justify further exploration in longer-term tests: the bearing capacity barely reached 1.5kN/m^2 , which is only twice the bearing capacity of the additive-free soil. Consequently, these tests were also terminated.

8. Polyvinyl alcohol

Promising results were obtained in 1-day tests using this additive and consequently further longer-term tests, as well as investigations into the influence of mixing techniques and other factors, were conducted. The relevant data and figures are given in full in the appendices (Appendix A81 - 89), and only a few key figures are reproduced here for convenience.

Figure 11 shows the results for one-day tests using 1.25, 2.5% and 5% additive concentration, together with evidence of the very good repeatability of these tests. Increasing the additive concentration increases the measured strength. At 5% concentration, the bearing capacity increases to about 5kN/m^2 , which suggests a corresponding undrained shear strength of $c0.5\text{ kN/m}^2$. (For these strengths, the assumption of undrained conditions is tenable).

Interestingly, the rate of strength gain appears to increase with increasing additive concentration, but no attempt was made to test whether this holds true at even higher concentrations. However, in hindsight, the restricted supply of water in these tests has probably increased these strengths.

Concerns regarding the influence of the mode of mixing was assuaged by repeating the 5% tests and comparing the results obtained by mixing in (a) one layer, (b) four layers. There was some effect as can be seen in Appendix A82 but this is small compared to the effect of additive concentration. Also shown in that appendix is the very good repeatability of the results for this type of mixing too.

Three-day tests (Figure 12 and Appendix A83) at 5% concentration showed a further substantial gain in strength, with bearing capacity increasing by about 80%. Once again, the results were found to be repeatable to a high degree. But little further gain in strength was observed in the 7-day tests (Appendix A84).

Suspicion that the PVA-dosed soils were absorbing water from their surroundings suggested that the strengths measured hitherto might be higher than they would be if there was a copious supply of water available, as would be the case near the surface of the ocean floor (but not necessarily at greater depths in the soil where inflow would be inhibited by the soil itself). To test this, in the subsequent tests, samples were stored under (salt) water following additive dosing. These tests are labelled as “saturated” in some figures in the appendices (“submerged” is the more apposite description). Filter paper was affixed to the top of the open cylinder to prevent loss of soil into the bath: in later tests, a small slit was cut into the paper to ensure that the samples remained flooded at all times.

Unlike the other additives tested to date, PVA fails to dissolve readily in water and accordingly some experiments were conducted using (vigorous) machine mixing: of the PVA/water mixture and also the PVA/water/soil mixture.

The results of these experiments, for 1-day tests, can be seen in Appendices A85-86. The vigorous machine mixing (of the PVA/clay mixture) resulted in the unexpected separation of a very weak suspension from a very strong “sediment” - whose strength exceeded the (then) capacity of the apparatus ($c4\text{kN/m}^2$). Although of theoretical interest, this mixing is far removed from the mixing likely to occur in practice and this phenomenon has not been explored further.

Three-day tests, using 5% PVA, in which a more thorough mixing of the PVA-additive in the soil was achieved by machine mixing of the PVA/water mixture prior to its subsequent mixing into the soil in four layers are shown in Appendix A87. The data shows that at least a 50% increase in strength is achievable by this more thorough mixing regime. As before, a significant gain in strength (approximately 70% in this case) is observed in the period between 1 day and 3 days. Also shown in Appendix 8.7 are some tests conducted at far higher (soil) water contents (namely, $4W_{LL}$ and $6W_{LL}$). Although very low strengths are recorded, these are many times greater than the strength of additive-free soil at this water content.

Longer-term (7 day) tests are shown in Figure 13 (and Appendix A88) again at a PVA concentration of 5%. Unexpectedly, the 7-day strength collapses to a level less than that attained after one day. The reasons for this are unknown but it is suspected that dilution of the PVA gel, as water is drawn into the sample, may be to blame. The 7-day test was repeated to ensure that this was not an experimental error: the results (shown in Appendix A88) conclusively demonstrates that the strength loss is genuine. Nevertheless, a long-term test (of 30 days) was carried out to verify these findings and to establish whether or not further strength loss would take place. The data (shown in Appendix A89) shows that any further strength loss beyond 7 days is minimal.

Although the initial results obtained using the PVA additive at 5% concentration were extremely promising, it appears that the mixture weakens progressively in the presence of free water. However, significant residual strength remained after 30 days, corresponding to a bearing capacity of $c7\text{kN/m}^2$ (undrained shear strength of $c0.7\text{kN/m}^2$). A drawback of this additive is that it does not dissolve readily in water but instead forms a gel which may have repercussions for mixing efficiency in practice. For these reasons, this programme of tests was terminated in favour of the Calcium hydroxide testing programme, which is described in the following section.

9. Calcium Hydroxide

The full results for the first series of tests using the Calcium Hydroxide additive are given in Appendices A91 – A95. For example, Figure 14 shows results for 1-day tests at additive concentrations of 1.25%, 2.5% and 5%. A considerable strengthening of the soil takes place, if not quite to the same degree as that which takes place in the PVA-dosed clays. Interestingly, it appears that a greater additive concentration than about 2 or 3% is less effective: the strength obtained using a 5% additive concentration is at least 30% less than the strength attained with a 2.5% concentration. Thus, it appears that there is an optimal dosing rate – in regard to the 1-day strength – of about 2 or 3%.

The repeated test (at 1.25% concentration), shown in Appendix A91, provides a good level of agreement, and the discrepancies between the two data sets is relatively small compared with the effects of changing the dose levels.

This first set of results was obtained without storing (“submerging”) the samples in a bath of sea-water. Given the test experience with the PVA additive, further tests were conducted to assess the effects of submergence: the results are shown in Appendix A92 for an additive concentration of 1.25%. The effect, in these 1-day tests, is, because of the different sample depths, less than it might appear at first sight. Scaling of the depth axis to the same dimension reduces the apparent discrepancy. Further, when experimental deviations are taken into account, it would be unwise to draw strong conclusions from the disparity between these two sets of results.

All subsequent tests were carried out under submerged conditions and (in this series) using a 2.5% dose: Figure 15 shows results for 3-day tests which reveal that the strength after 3 days is practically double the strength at 1 day. Repeatability tests demonstrate very good agreement. The soil continues to strengthen through 7 days (see Appendix A94) reaching values comparable to the peak strengths achieved by PVA-dosed soil. In terms of undrained shear strength, this may be estimated to be approximately 1kPa. Finally, in this series of tests, the 30-day results are shown in Figure 16. The very substantial gain in strength over this period may be adducible to hydration of the sample, which (it was thought) might be a consequence of the filter-paper impeding water flow, or a hardening-induced reduction in permeability of the sample itself. In any event, in subsequent tests, a small slit was made in the filter paper to rule out external flow constriction.

A later series of tests was conducted at 30 days to explore the effect of additive concentration in the medium term. Tests were carried out at concentrations of 1%, 2% (also repeated) and 3%. To accommodate the higher strength of the mixture, a smaller (15mm diameter) disk was employed. For completeness, the full results are given in Appendix A941 and in particular, the repeated test (at 2% concentration) which demonstrates very good repeatability is of note.

The principal results from all the tests carried out on the soil at a water content of $2W_{ll}$ are shown in Figure 17. Here, the estimated undrained shear strength at sample mid-height is employed as the key variable. These estimated values, for the reasons described earlier, should be treated with some caution. However strength is a preferred parameter here because bearing pressure is, for a buried disk, dependent on disk diameter and this varies considerably between the test series.

Quadratic trend-lines have been drawn through the 1-day and 30-day tests to facilitate interpretation but should not be given too great a weight: for example, the data does not support a reduction in strength at 30 days for additive concentrations greater than 4%.

The results show that, over a period of 30 days, an additive concentration of circa 2% is needed to achieve the maximum gain in strength but that a higher dose appears to confer no further improvement. Moreover, it is clear from these results that the soil strengthens over time at all concentrations: the magnitude of any further strengthening awaits the outcome of longer term tests (in progress) for additive concentrations of 1-3%. The strength after 30 days is significantly higher than the remoulded strength at the Liquid Limit, and thus some forty (40) times higher than that in the additive-free soil.

Tests conducted at a water content of $1.5W_{LL}$

Following submission of the interim report in early March, a new series of tests using the Calcium Hydroxide additive was performed. The essential difference between these tests and those that had been undertaken before was that the water content in these tests was reduced to $1.5 W_{LL}$. A significant consequence was that mixing of the soil/additive/water mixture became more difficult.

(Note: Some 1-day and 3-day tests were carried out at $3W_{LL}$: these data are currently available only in hard copy but will be made available on request).

Although the optimal dose at $2W_{LL}$ was established to be circa 2-3% (at least for one-day tests) it was unclear whether this would prevail in general.

Full results for the 1-day and 3-day tests at $1.5 W_{LL}$ are shown in Appendix C1 and C2, respectively, for additive doses of 1.25%, 2.5% and 5%. The one-day tests (see Figure 18) again show that a weaker result is obtained with a 5% dose compared with a 2.5% dose, which indicates that there is an optimal dose at circa 2%. The peak undrained shear strength, for the 2.5% concentration, is estimated to be circa 0.5kPa. (As usual, the computed results for undrained shear strength from bearing capacity should be treated with caution, for weaker mixtures at least).

After 3 days, the peak strength in these samples increases substantially to circa 1.6kPa (refer to Figure 18 and Appendix C2), but is overtaken by the strength gain in the 1.25% samples, whose strength apparently increases to 2.7kPa (approximately). No duplicate tests had been conducted to verify this particular result, which subsequent testing suggests is anomalously high. (A possible source for the discrepancy may be variations between different samples of the soil.)

Given that the results for the 3-day, 1.25% additive concentration tests appeared to be anomalous a further series of tests (tabulated in Appendices B1, B2 and B3) were conducted. It should be noted that Appendix B1 is simply a re-calculation of the results of Appendix C2, based on calculation of the net bearing capacity: the differences are not large.

Appendix B2 and B3 together contain the results of two series of tests carried out at additive concentrations of 0.5% to 2% and 3% to 5%, respectively. The principal results are summarized in Table 2 which shows representative values of the bearing capacity and undrained shear strength of the mixture at mid-depth.

Table 2 Three-day strength of clay slurry at $1.5W_{LL}$ dosed with Calcium Hydroxide

% $\text{Ca}(\text{OH})_2$	Bearing capacity kPa	Shear strength kPa
0.5	6	0.5
1	10	1.1
1.25*	20	2.7
1.5	12	1.6
2	20	2.7
2.5*	13	1.6
3	10	1.0
4	9	1.0
5	10	1.1
5*	11	1.2

*These data are taken from Appendix C.

On this evidence, it is tempting to discard the data obtained for the 1.25% case and consequently conclude that it appears that an optimum calcium hydroxide dose remains at circa 2% at this lower water content. At the least, it is possible to conclude from Table 2 that an undrained shear strength of at least 1kPa is achievable after 3 days following a dose of (at least) 1% Calcium Hydroxide. However, at three days, a high concentration of the additive appears to confer no advantage.

The 7-day results are tabulated in Appendix D1, and are summarized in Table 3. As in Table 2, the quoted values refer to representative values of the bearing capacity and undrained shear strength of the mixture at mid-depth. In the case of the 4% additive concentration, the load capacity of the apparatus (at that time) was exceeded at a disk penetration of one-third depth, so the values given are obtained by extrapolation.

Table 3 Seven-day strength of clay slurry at $1.5W_{LL}$ dosed with Calcium Hydroxide

% Ca(OH) ₂	Bearing capacity kPa	Shear strength kPa
1	11	1.4
2	11	1.5
2(dupl.)	30	3.7
3	24	2.9
4*	42	5.0

*extrapolated values

A significant point is that the repeatability of the tests conducted at 2% concentration is poor, although this should be seen in the context of an order of magnitude gain of strength over the additive-free soil. The poor repeatability may be further evidence of soil sample variability, and hence an average of the two measured strengths (i.e., 2.6kPa) would provide a better estimate. It should also be noted that the increase in strength over 7 days appears to be much higher for the high dosage (3% and 4%) samples than for the low-dosage samples, and as a consequence the optimum dose appears to have shifted to a higher concentration.

The final series of tests was conducted at 40 (sic) days to explore the effect of additive concentration in the medium term. Tests were carried out at concentrations of 1%, 2% (also repeated), 3% and 4%. To accommodate the anticipated higher strength of the mixture, a smaller (15mm diameter) disk was employed. For completeness, the full results are given in Appendix D2 and in particular, the repeated test (at 2% concentration) which demonstrates very good repeatability is of note.

The principal results (undrained shear strength at sample mid-height) from all the tests carried out on the soil at a water content of $1.5 W_{ll}$ are shown in Figure 20. Quadratic trend-lines have been drawn through some of the data sets to facilitate interpretation. On this figure, it is clear that the 3-day test result at 1.25% concentration (remarked on earlier) is indeed anomalous and this has been excluded.

The results show that strength increases with time at all additive concentrations. Further, the apparently “optimal dose” observed at early times is not evident in the medium term. Thus, for example at 40 days, the strength appears to be broadly proportional to additive dose, up to a concentration of 3%. However, it is not clear whether there is any further significant strength gain beyond that concentration.

The strengths are typically 2-4 times higher than those for the correspondingly dosed soils at the higher water content ($2W_{ll}$). This ratio is much less than the strength ratio between additive-free soils: in other words, the soil strengths have converged.

Close examination of the undrained shear strength (depth) profiles reveal that the computed undrained shear strengths are not constant with depth. In part, this may reflect errors in the assumed correlation between bearing capacity and shear strength. However, the pronounced drop in strength towards the surface of samples is open to interpretation.

Conclusions

The results of this experimental study has clarified the path which should be followed to reach the ultimate goals of the project. Early focus on the acceleration of particulate settling (at very high water contents) has given way to the promotion of cementitious bonding, at rather lower water contents. The effectiveness of a number of additives commonly employed in waste-water treatment facilities, and in other fields, has been investigated and found wanting. Of these, polyvinyl alcohol appeared to offer promise but its effectiveness proved to be short-lived.

Treatment of soils using calcium hydroxide has a long history and in this context (i.e., to strengthen soils with very high water contents) seems to offer a very satisfactory solution, albeit that test results to date have still only been undertaken up to 40 days. The principal conclusions of this series of tests are:

The strength of slurries (with water contents $< 2 \cdot W_{ll}$) can be increased by over an order of magnitude by dosing with 2% Calcium hydroxide: this dosage realizes strengths in excess of the (remoulded) strength of soils at the Liquid Limit (i.e., 2kPa). Strengths increase continually over time (up to 40 days), but quantification of any subsequent strength increase awaits further tests. For concentrations less than 2%, it is tentatively suggested that strength is proportional to additive concentration. The use of additive concentrations higher than 3% appear to offer little further strength gain.

Longer-term tests (up to one year) will establish whether the strength gains using the calcium hydroxide additive are permanent, although if these gains are due to pozzolanic reactions, the evidence of the literature suggests that this will be so. Finally, it may be worthwhile to conduct control experiments under refrigeration.

The literature points to certain factors, such as the presence of sulphates and organic material, which tend to inhibit Calcium Hydroxide soil-strengthening, and these topics also deserve further consideration.

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Appendix A0

Coefficient of consolidation: an upper-bound estimate

The coefficient of consolidation is linked to the compressibility and permeability of the material via the equation:

$$C = k / m_v \gamma_w$$

where the coefficient of compressibility (m_v), in one-dimensional consolidation, is defined as follows:

$$m_v = \frac{de}{d\sigma'_z} \frac{1}{1+e}$$

Measurement of both permeability (k) and coefficient of compressibility (m_v) is exceedingly difficult in very soft soils. Although both increase rapidly with increasing water content, their ratio (and thus C) is customarily assumed to remain constant, for moderate changes in water content.

Measurements of permeability (e.g., Pane & Schiffman, 1997, Al-Tabbaa & Wood, 1987, Mesri & Olson, 1971, Berilgen et al, 2006, etc.) suggest that kaolinite and illitic soils have permeabilities of the order of 10nm/s and 0.05nm/s, respectively at a void ratio of 2. These permeabilities are increased by a factor of about three (3) if the void ratio increases to 2.7. (These void ratios correspond to water contents of $1.5*W_{ll}$ and $2*W_{ll}$, respectively, in the present context).

The normal consolidation line, assuming linearity on a semi-logarithmic scale, can be expressed in the form:

$$e = e_0 - \lambda \log_e \sigma'_z$$

where the gradient λ is linked to the compression index C_c by the equation: $C_c = 2.3\lambda$.

Thus, using the well-known Skempton (1944) correlation:

$$\lambda = 0.4(W_{ll} - 0.1)$$

we obtain a value of λ of the order of 0.16.

Differentiating the void ratio / effective stress equation, we obtain:

$$\frac{de}{d\sigma'_z} = \frac{-\lambda}{\sigma'_z}$$

Now, ignoring the negative sign; noting that the applied bearing pressures in very soft soils are of the order of 1kN/m^2 , and substituting through the above equations, we find that:

$$m_v \approx 0.05 \text{ m}^2 / \text{kN}$$

Finally, taking $\gamma_w = 10 \text{ kN/m}^3$, and $k = 10 \text{ nm/s}$, we obtain:

$$C \approx 0.02 \text{ mm}^2/\text{s}$$

which compares well with the upper-bound value quoted in the Report.

FIGURES

- Figure 1. Liquid Limit determination for a mixture of Bothkennar Clay & bentonite mixed in the ratio of 90:10.
- Figure 2 Displacement-time response of 50mm diameter disk falling from rest in additive-free soil at $2*W_{ll}$ after sedimentation over 24 hours. (From Appendix A21)
- Figure 3 Velocity-time response of 50mm diameter disk falling from rest in additive-free soil at $2*W_{ll}$ after sedimentation over 24 hours. (From Appendix A21)
- Figure 4 Acceleration -time response of 50mm diameter disk falling from rest in additive-free soil at $2*W_{ll}$ after sedimentation over 24 hours. (From Appendix A21)
- Figure 5 Bearing pressure (capacity) with increasing depth in additive free soil (From Appendix A24)
- Figure 6 Settlement-time trace of 50 mm disk in additive-free soil, loaded at various depths. (For $w = 1.5W_{ll}$ and $w = 2W_{ll}$)
- Figure 7 Effect on bearing capacity of dosing with Aluminium Potassium Sulphate (1-day test, 50mm disk, $w=2*W_{ll}$ From Appendix A41)
- Figure 8 Effect on velocity trace of dosing with Chitosan (1-day test, 50mm disk, $w=2*W_{ll}$ From Appendix A51)
- Figure 9 Effect on velocity trace of dosing with Sodium Alginate (1-day test, 50mm disk, $w=2*W_{ll}$ From Appendix A61)
- Figure 10 Effect on bearing capacity of dosing with Potassium Sulphate (1-day test, 50mm disk, $w=2*W_{ll}$ From Appendix A71)
- Figure 11 Effect on bearing capacity of dosing with Polyvinyl Alcohol (1-day test, 50mm disk, $w=2*W_{ll}$ From Appendix A81)
- Figure 12 Effect of time on bearing capacity after dosing with 5% Polyvinyl alcohol (1-day and 3-day tests, 50mm disk, $w=2*W_{ll}$ From Appendix A83)
- Figure 13 Effect on time and sample submergence on bearing capacity after dosing with 5% Polyvinyl Alcohol (3-day and 7-day tests, 50mm disk, $w=2*W_{ll}$ From Appendix A88)
- Figure 14 Effect on bearing capacity after dosing with Calcium Hydroxide (1-day tests, 50mm disk, $w=2*W_{ll}$ From Appendix A91)

- Figure 15 Effect of time on bearing capacity after dosing with 2.5% Calcium Hydroxide (submerged conditions)
(1-day and 3-day tests, 50mm disk, $w=2*W_{ll}$ From Appendix A93)
- Figure 16 Effect of time on bearing capacity after dosing with 2.5% Calcium Hydroxide (submerged conditions)
(1-30 day tests, 50mm disk, $w=2*W_{ll}$ From Appendix A95)
- Figure 17 Effect of time and Calcium Hydroxide concentration on computed undrained shear strength of soil at mid-height
($w = 2*W_{ll}$. Disk radii: 15-50mm)
- Figure 18 Effect on bearing capacity of dosing with Calcium hydroxide (submerged conditions)
(1 day tests, 35mm disk, $w=1.5*W_{ll}$ From Appendix C1)
- Figure 19 Effect on bearing capacity of dosing with Calcium hydroxide (submerged conditions)
(3 day tests, 35mm disk, $w=1.5*W_{ll}$ From Appendix C2)
- Figure 20 Effect of time and Calcium Hydroxide concentration on computed undrained shear strength of soil at mid-height
($w = 1.5W_{ll}$. Disk radii: 15-50mm)

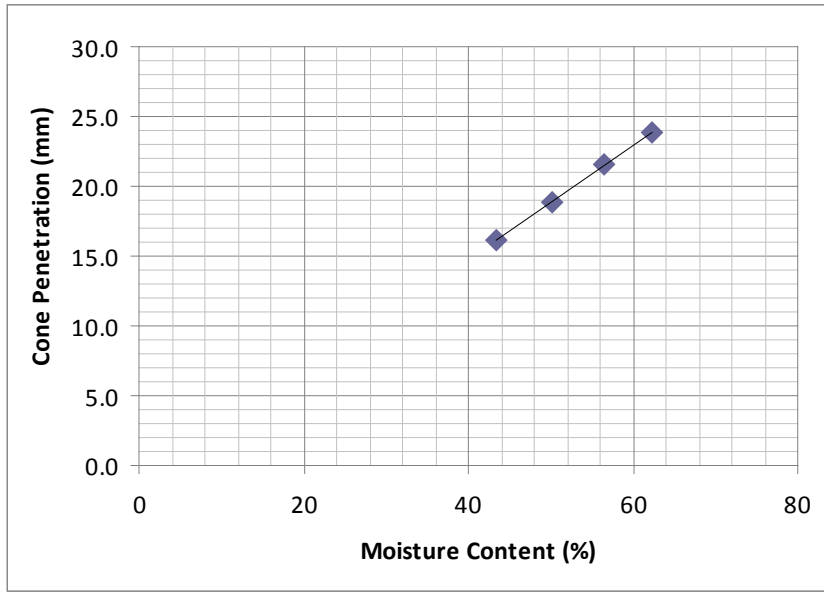


Figure 1. Liquid Limit determination for a mixture of Bothkennar Clay & bentonite mixed in the ratio of 90:10.

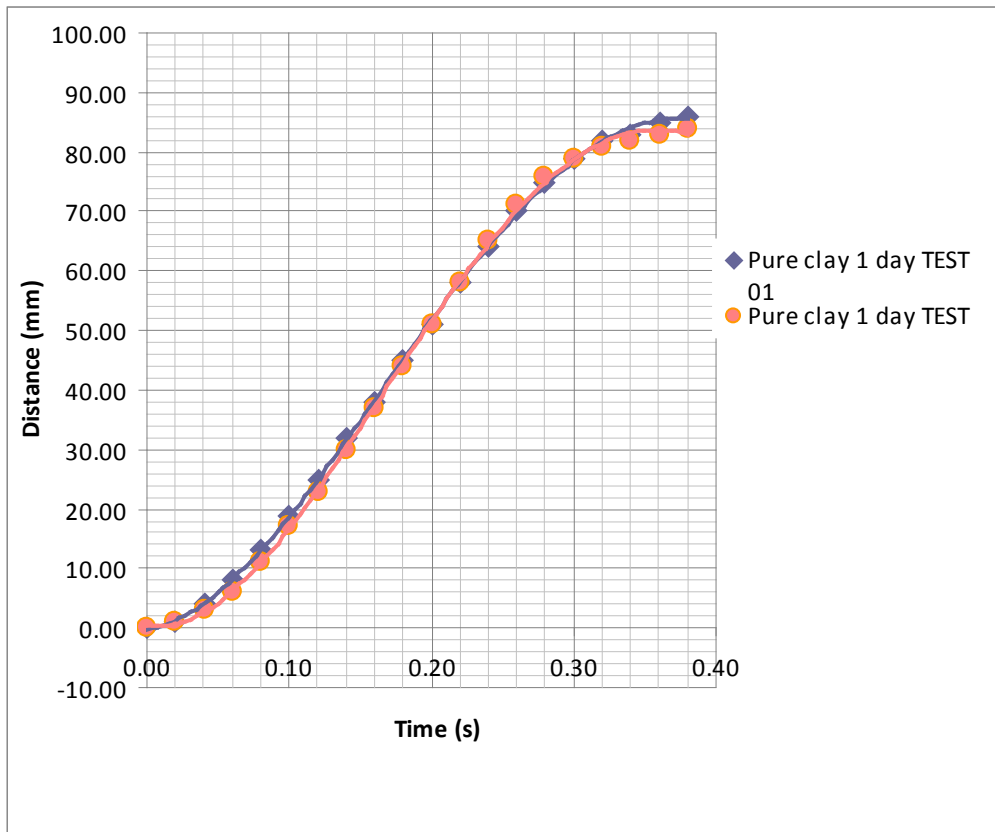


Figure 2 Displacement-time response of 50mm diameter disk falling from rest in additive-free soil at $2 \cdot W_{II}$ after sedimentation over 24 hours. (From Appendix A21)

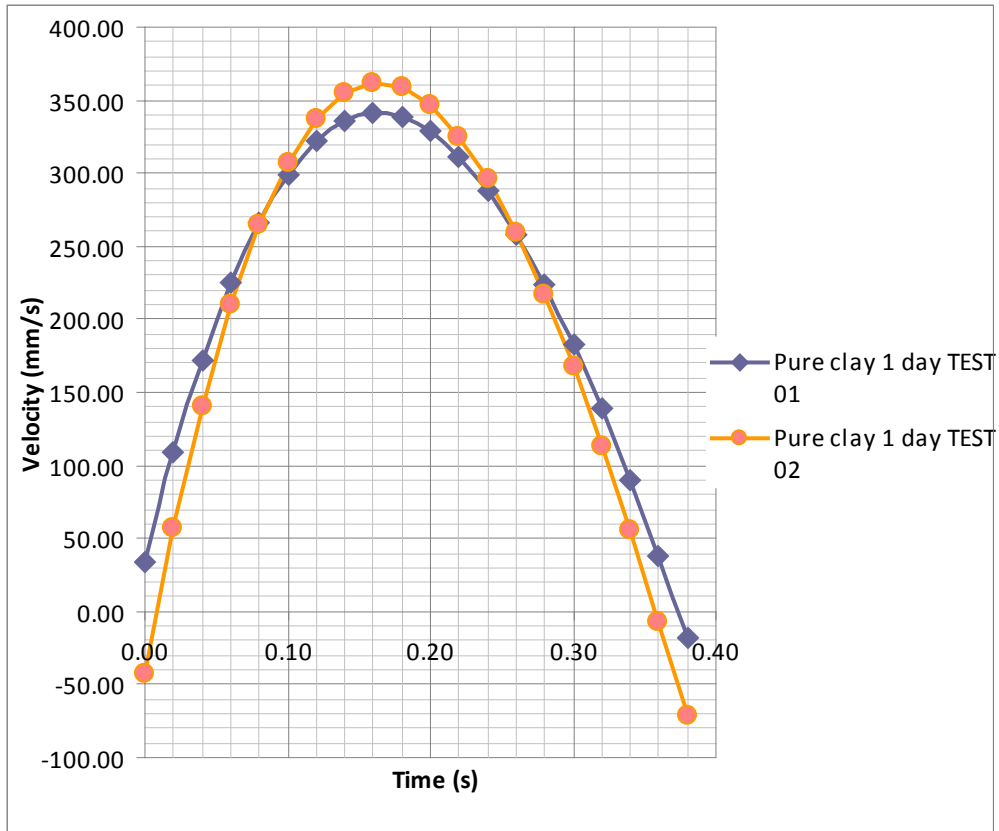


Figure 3 Velocity-time response of 50mm diameter disk falling from rest in additive-free soil at $2 \cdot W_{II}$ after sedimentation over 24 hours. (From Appendix A21)

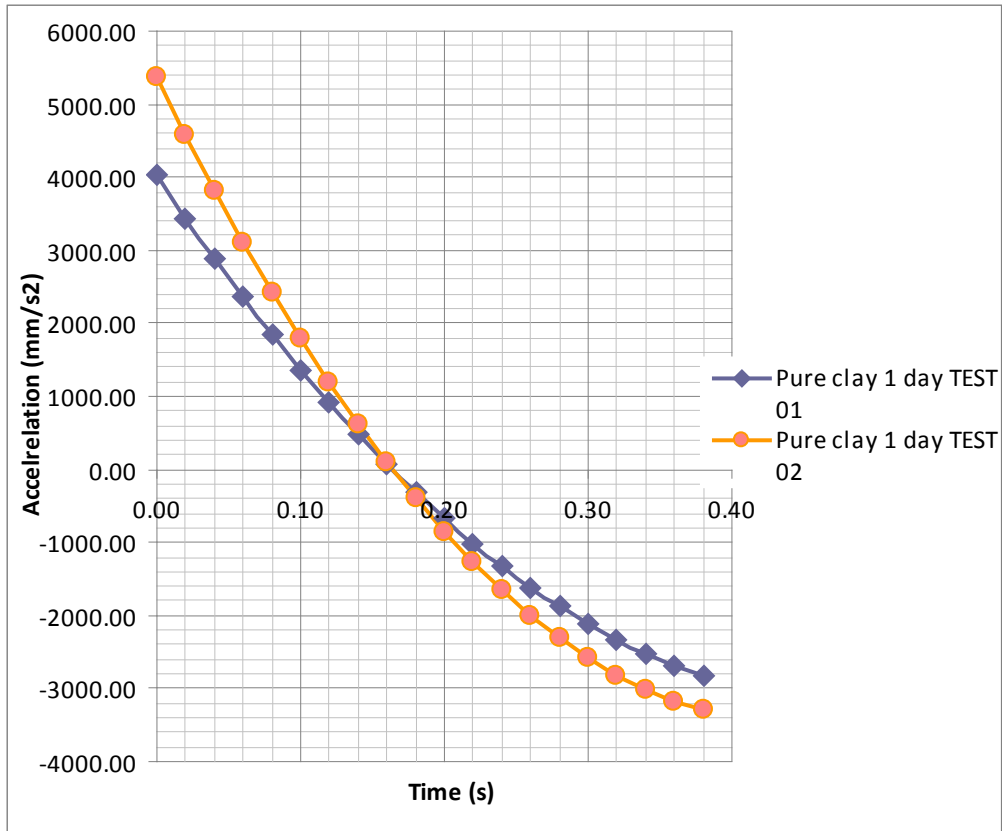


Figure 4 Acceleration -time response of 50mm diameter disk falling from rest in additive-free soil at $2 \cdot W_{II}$ after sedimentation over 24 hours. (From Appendix A21)

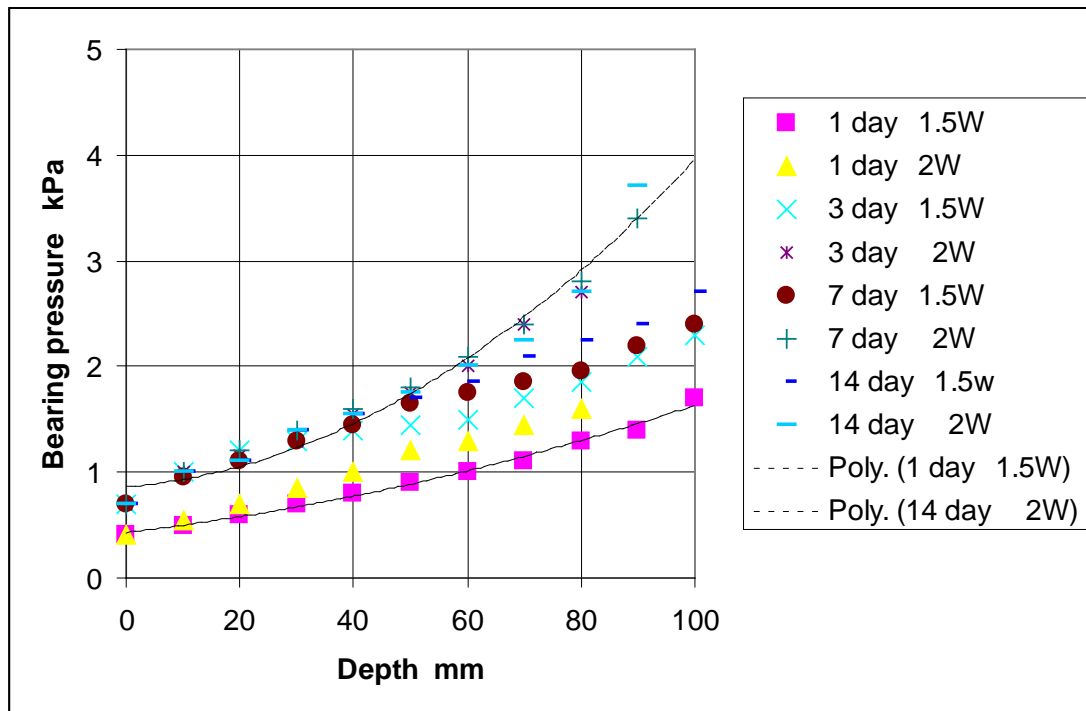


Figure 5 Bearing pressure (capacity) with increasing depth in additive free soil (From Appendix A24)

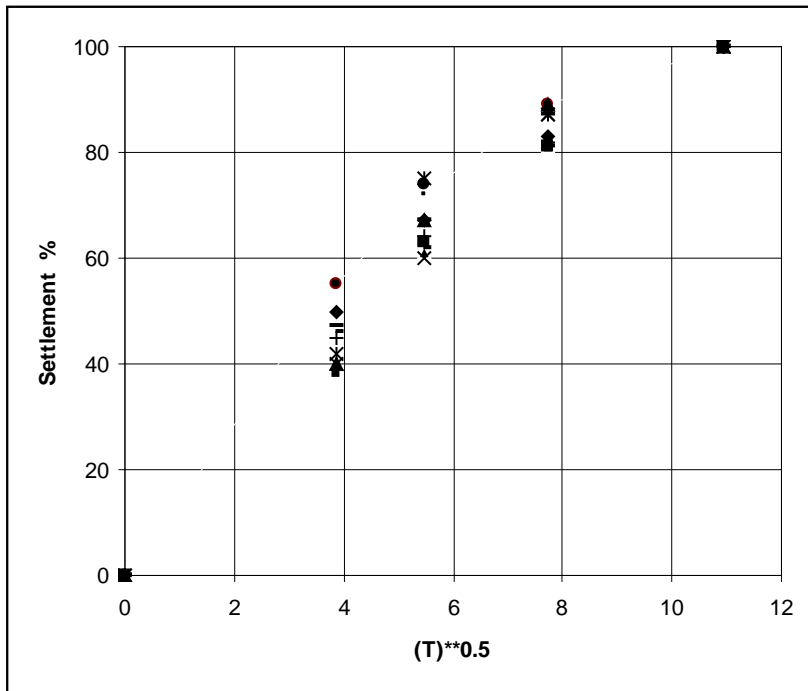


Figure 6 Settlement-time trace of 50 mm disk in additive-free soil, loaded at various depths. (For $w = 1.5W_{II}$ and $w = 2W_{II}$)

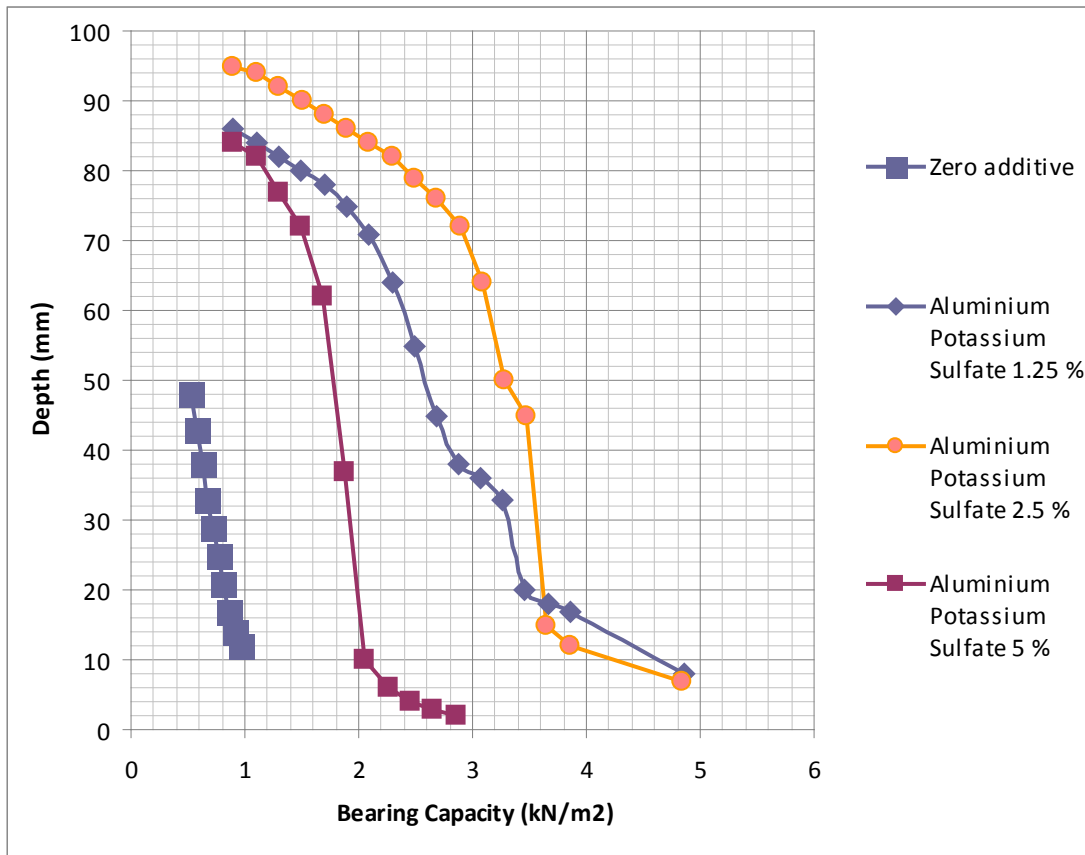


Figure 7 Effect on bearing capacity of dosing with Aluminium Potassium Sulphate (1-day test, 50mm disk, $w=2*W_{II}$ From Appendix A41)

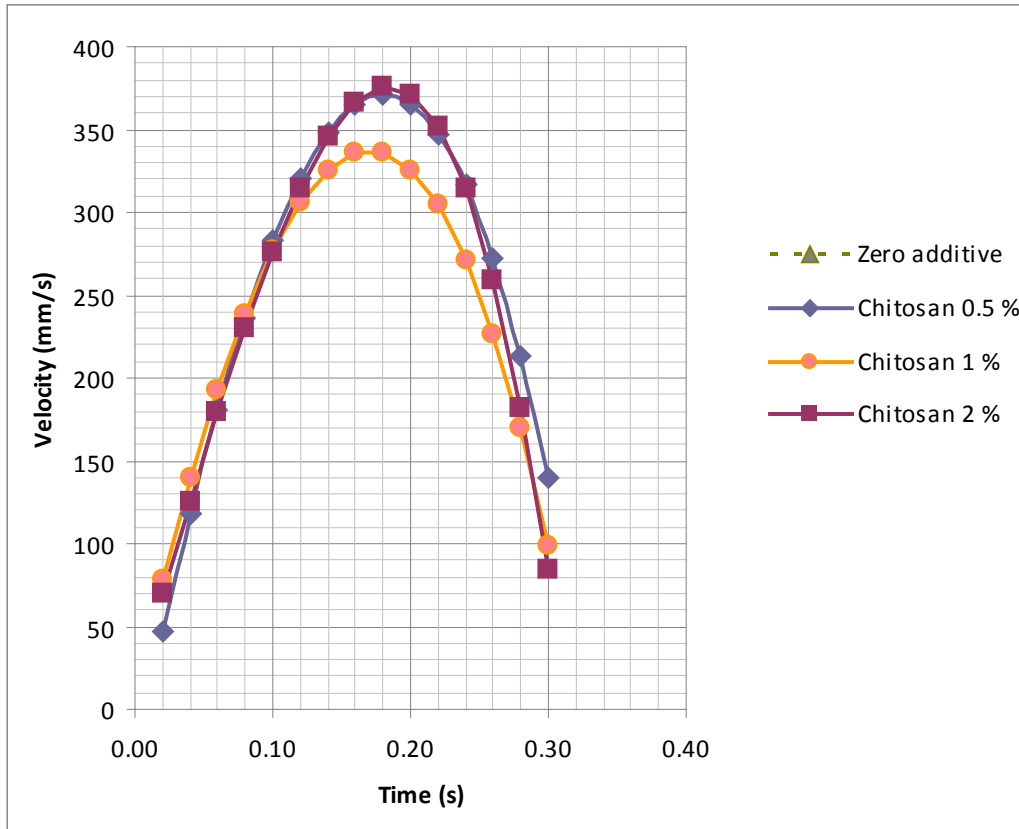


Figure 8 Effect on velocity trace of dosing with Chitosan (1-day test, 50mm disk, $w=2*W_{II}$ From Appendix A51)

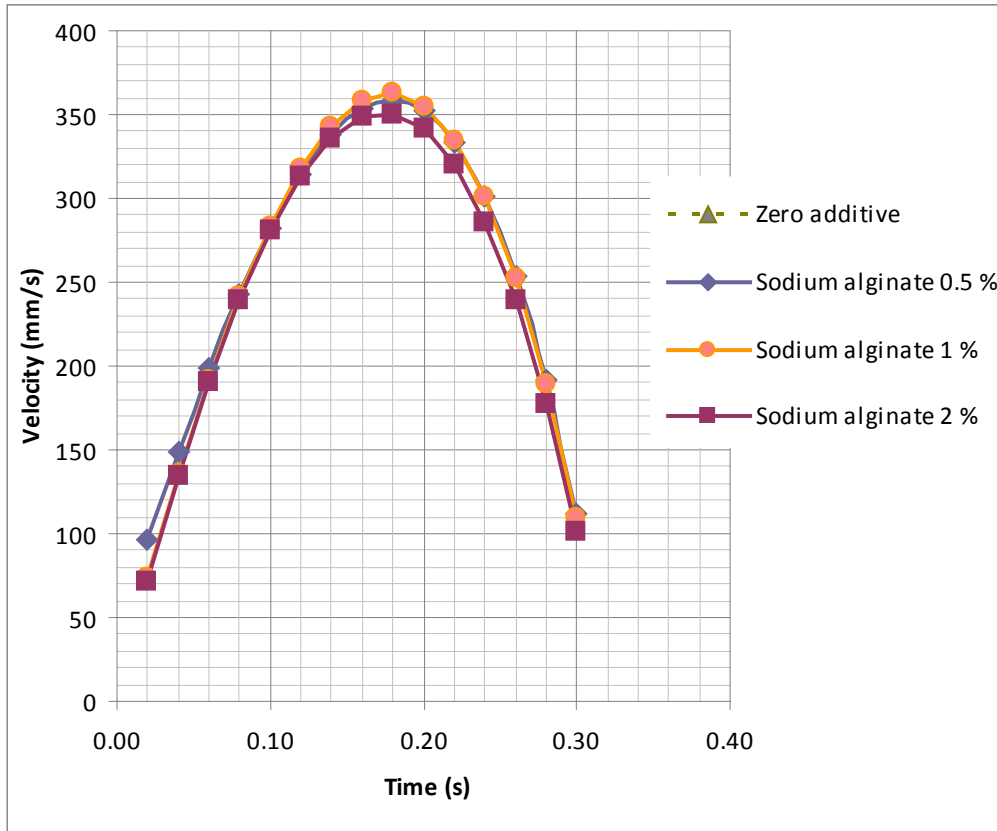


Figure 9 Effect on velocity trace of dosing with Sodium Alginate (1-day test, 50mm disk, $w=2*W_{II}$ From Appendix A61)

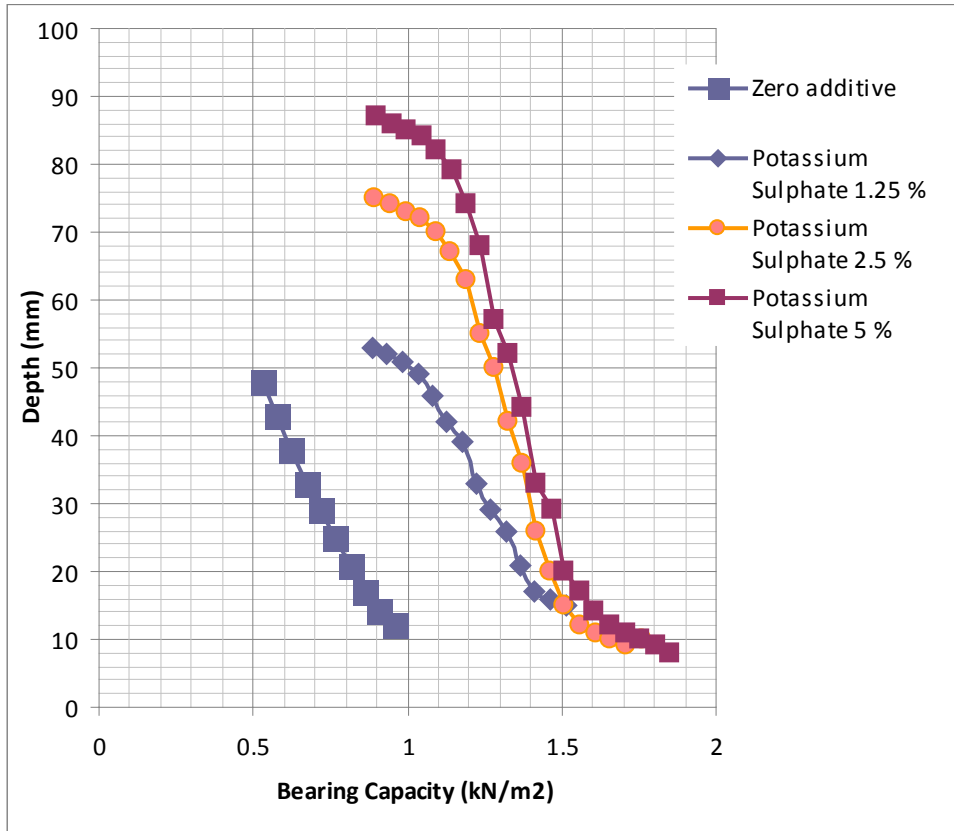


Figure 10 Effect on bearing capacity of dosing with Potassium Sulphate (1-day test, 50mm disk, $w=2*W_{II}$ From Appendix A71)

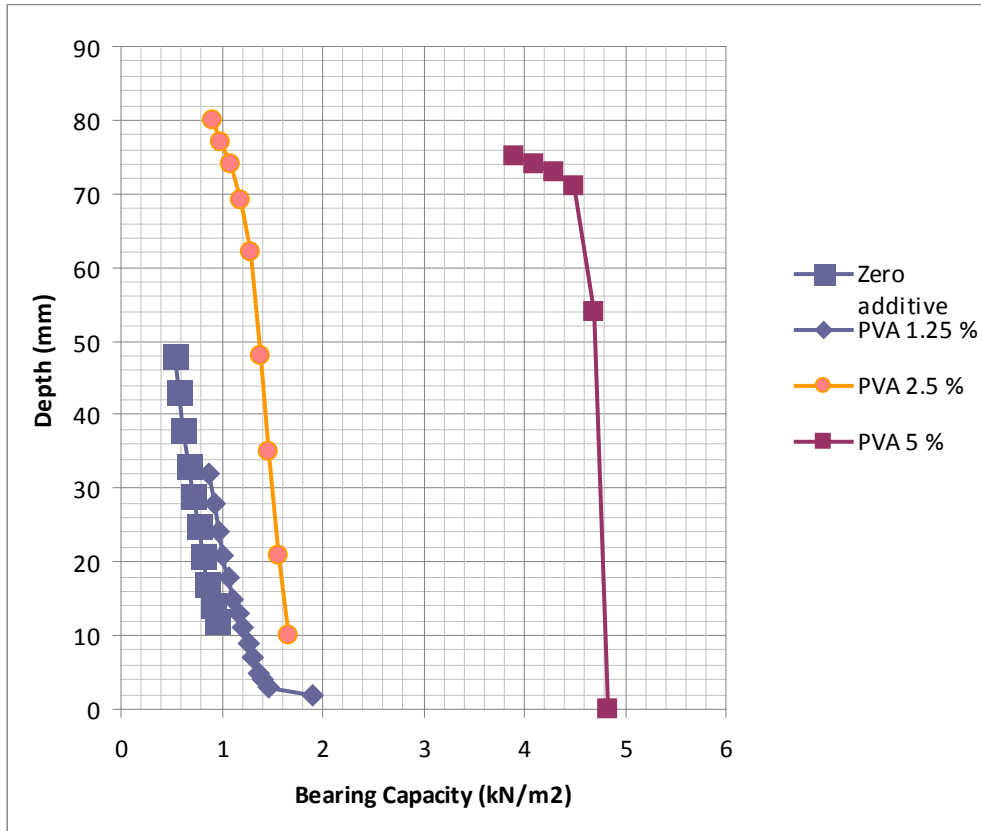


Figure 11 Effect on bearing capacity of dosing with Polyvinyl Alcohol (1-day test, 50mm disk, $w=2*W_{II}$ From Appendix A81)

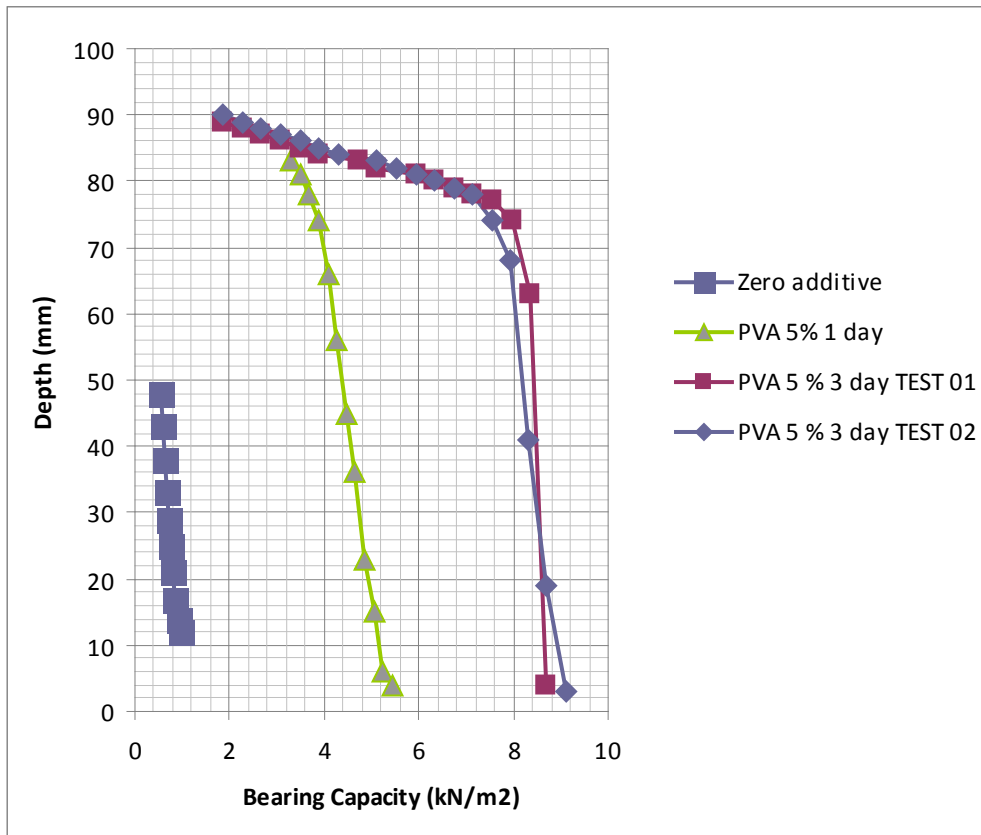


Figure 12 Effect of time on bearing capacity after dosing with 5% Polyvinyl alcohol
 (1-day and 3-day tests, 50mm disk, $w=2*W_{II}$ From Appendix A83)

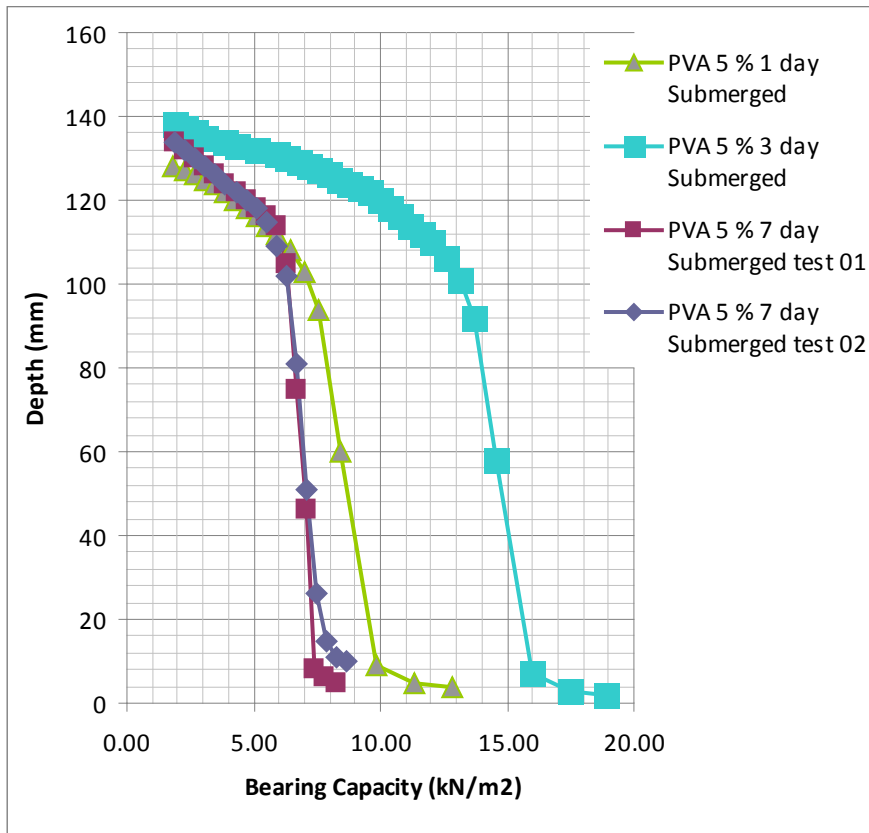


Figure 13 Effect on time and sample submergence on bearing capacity after dosing with 5% Polyvinyl Alcohol (3-day and 7-day tests, 50mm disk, $w=2*W_{II}$ From Appendix A88)

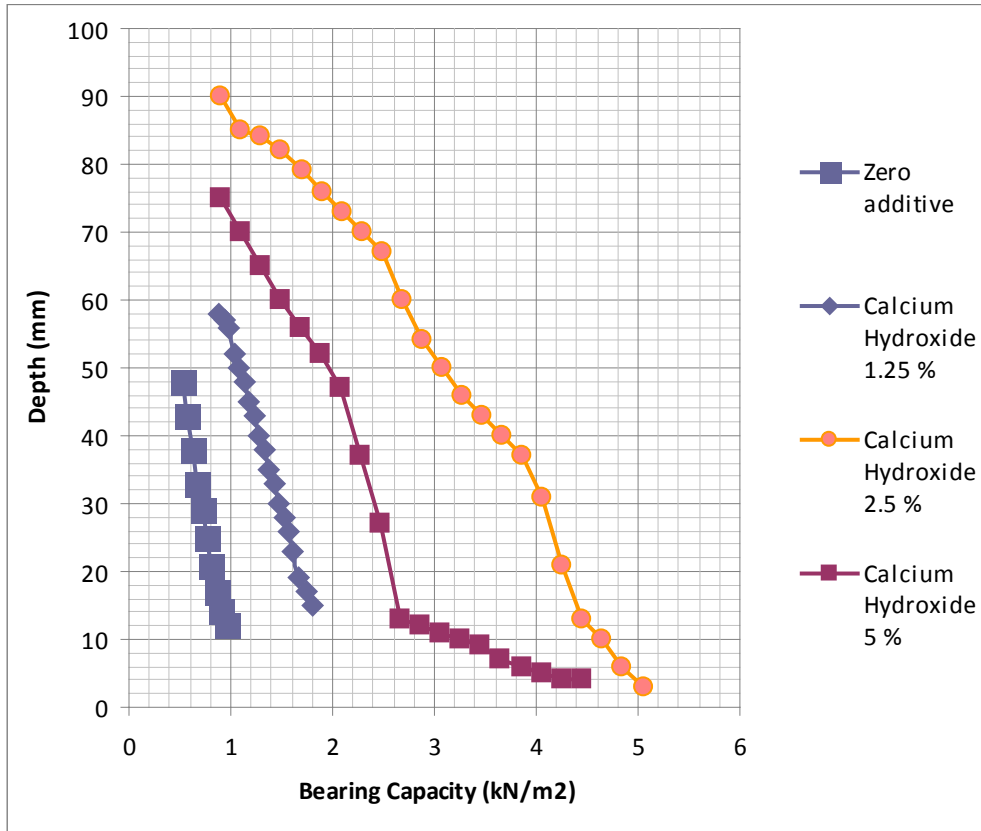


Figure 14 Effect on bearing capacity after dosing with Calcium Hydroxide (1-day tests, 50mm disk, $w=2*W_{II}$ From Appendix A91)

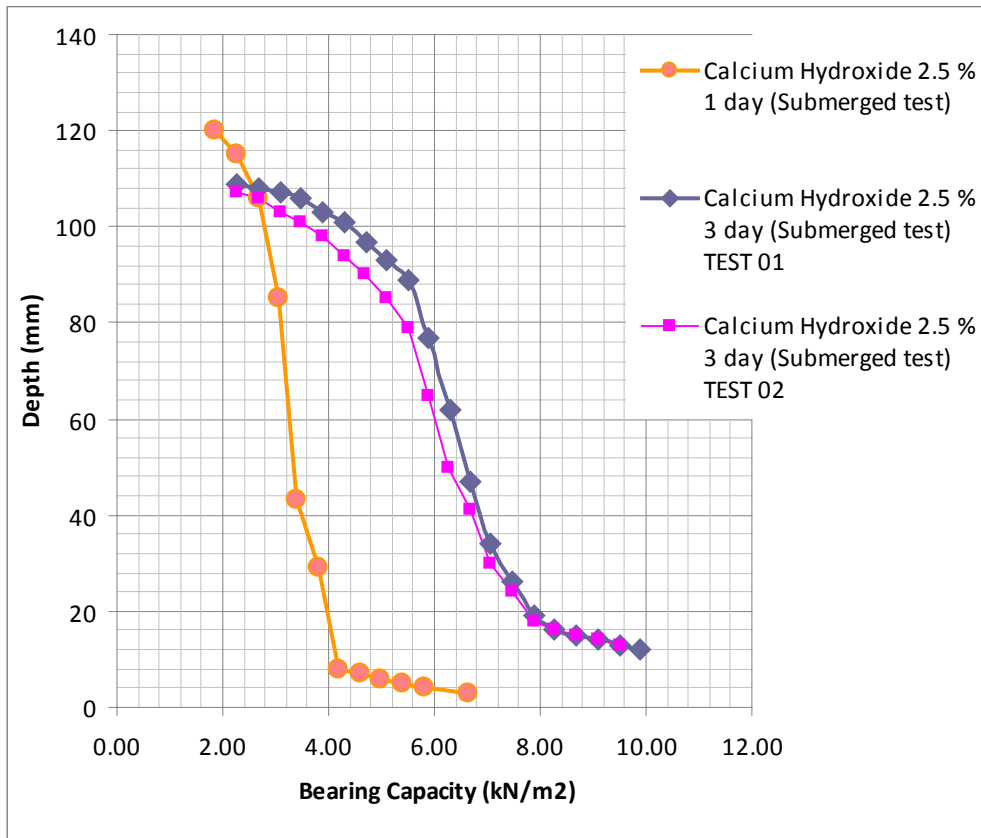


Figure 15 Effect of time on bearing capacity after dosing with 2.5% Calcium Hydroxide (submerged conditions)
 (1-day and 3-day tests, 50mm disk, $w=2*W_{II}$ From Appendix A93)

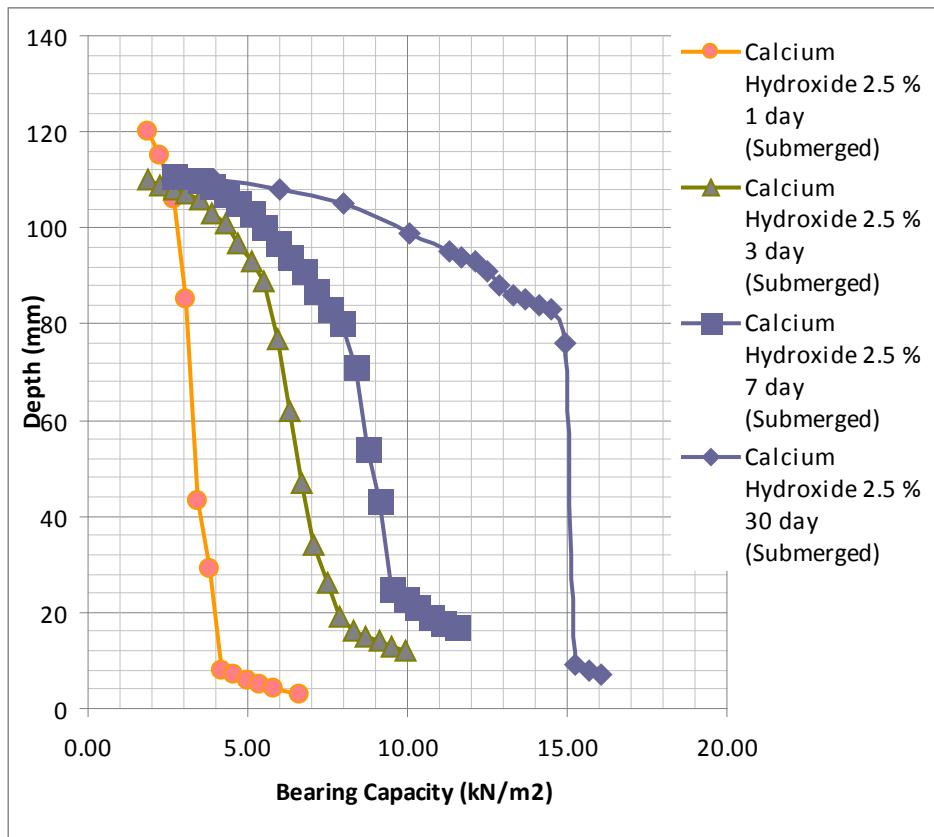


Figure 16 Effect of time on bearing capacity after dosing with 2.5% Calcium Hydroxide (submerged conditions)
 (1-30 day tests, 50mm disk, $w=2*W_{II}$ From Appendix A95)

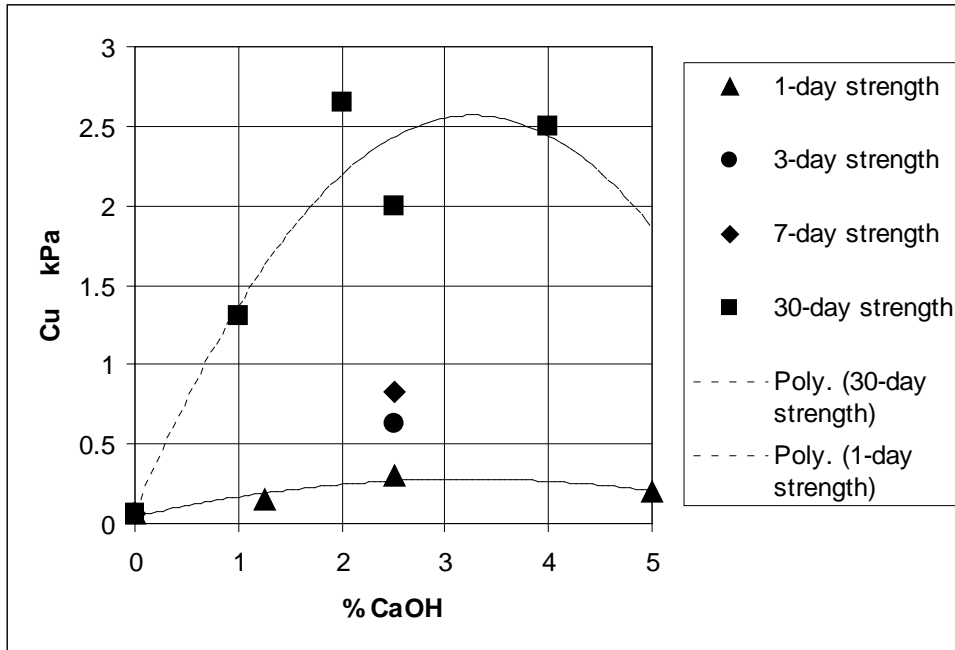


Figure 17 Effect of time and Calcium Hydroxide concentration on computed undrained shear strength of soil at mid-height ($w = 2 * W_{II}$. Disk radii: 15-50mm)

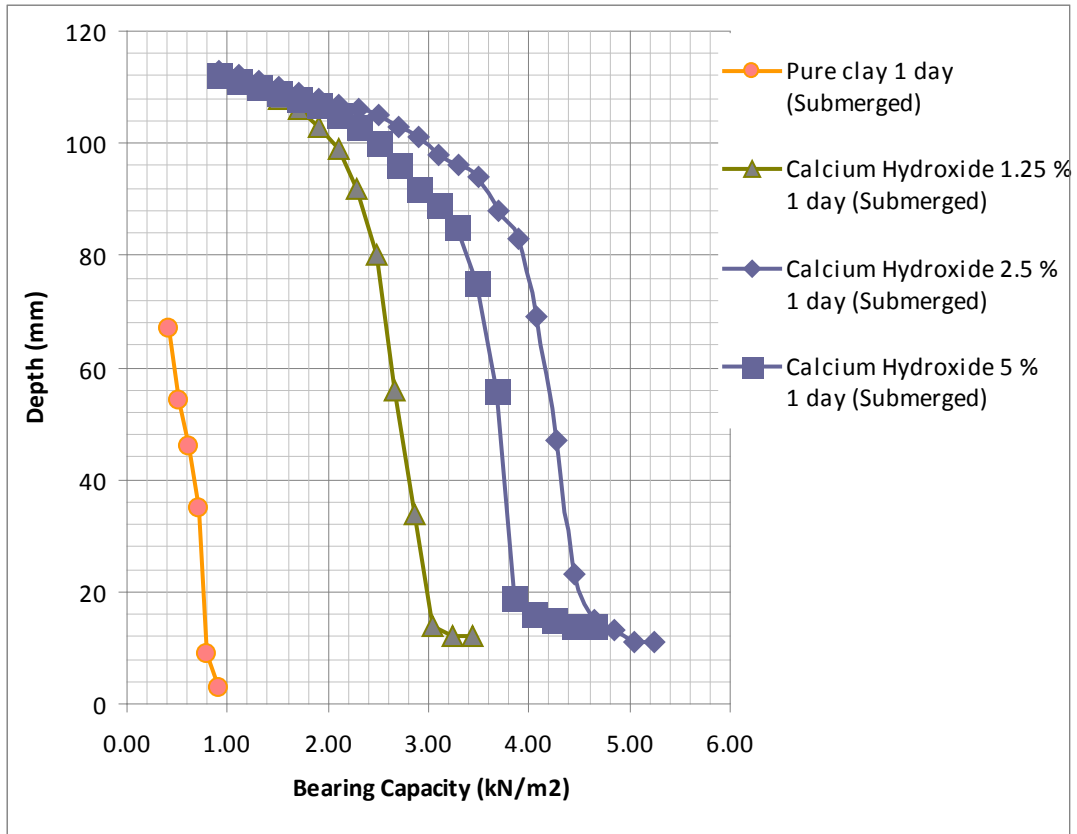


Figure 18 Effect on bearing capacity of dosing with Calcium hydroxide (submerged conditions) (1 day tests, 35mm disk, $w=1.5*W_{ll}$ From Appendix C1)

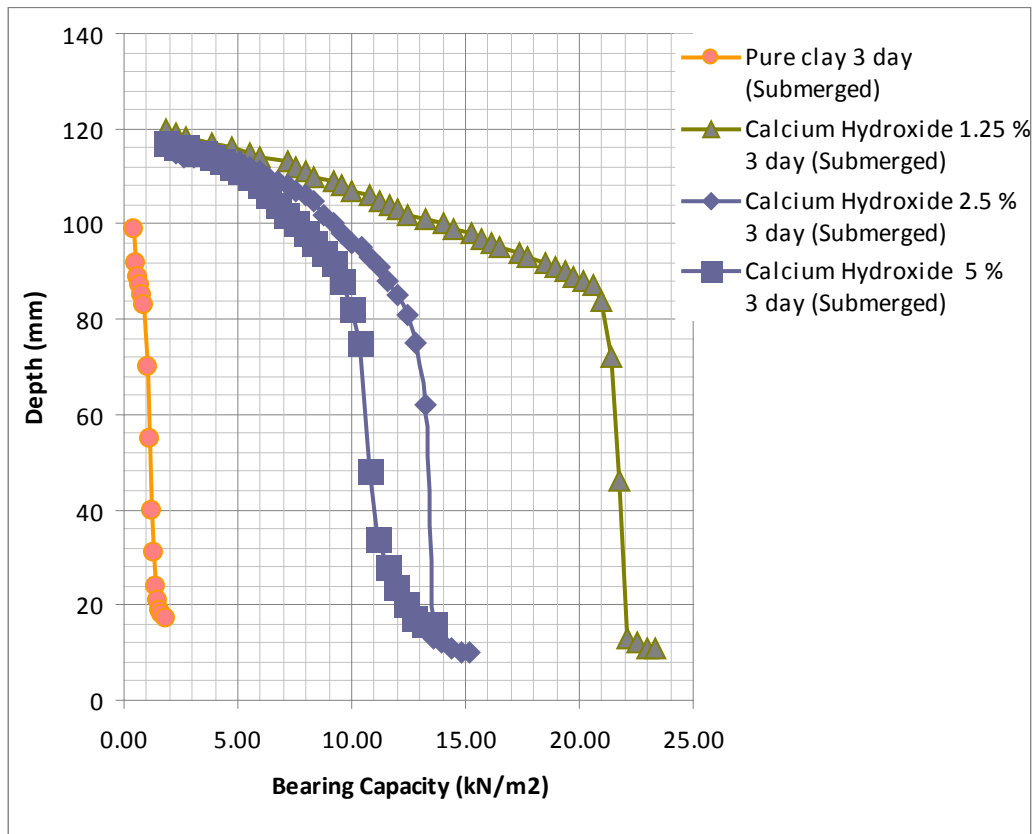


Figure 19 Effect on bearing capacity of dosing with Calcium hydroxide (submerged conditions) (3 day tests, 35mm disk, $w=1.5 \cdot W_{ll}$ From Appendix C2)

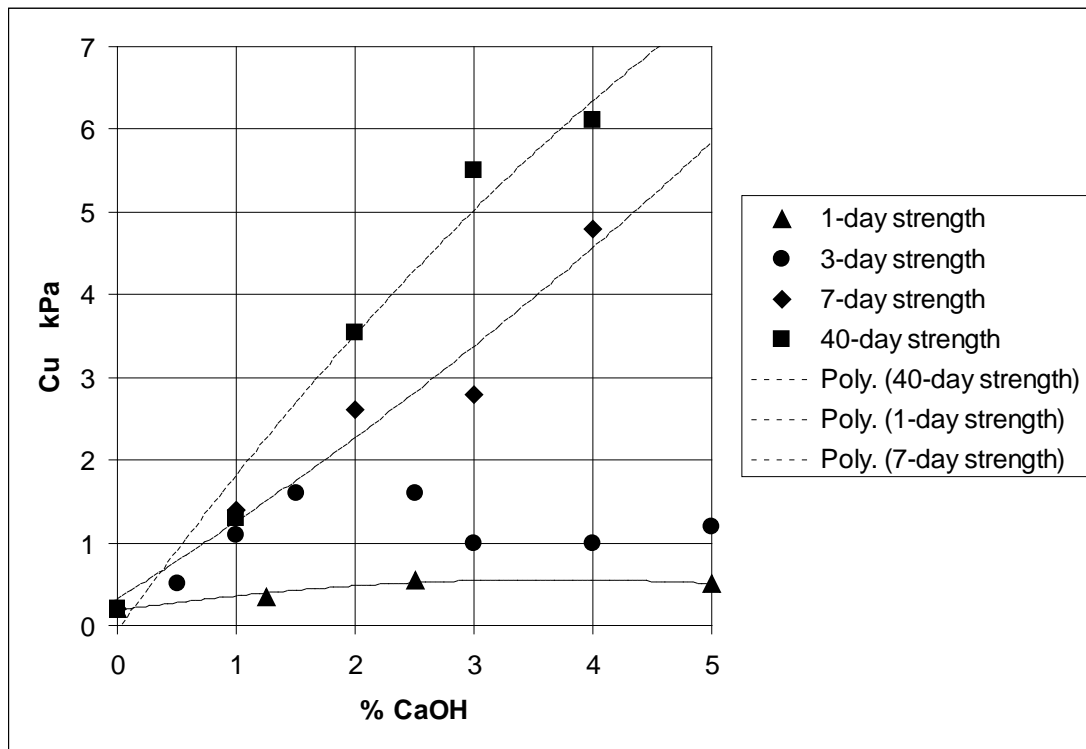


Figure 20 Effect of time and Calcium Hydroxide concentration on computed undrained shear strength of soil at mid-height ($w = 1.5W_{ll}$, Disk radii: 15-50mm)