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# Stabilisation of clayey soils with high calcium fly ash and cement

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**Abstract**

The effectiveness of using high calcium fly ash and cement in stabilising fine-grained clayey soils (CL,CH) was investigated in the laboratory. Strength tests in uniaxial compression, in indirect (splitting) tension and flexure were carried out on samples to which various percentages of fly ash and cement had been added. Modulus of elasticity was determined at 90 days with different types of load application and 90-day soaked CBR values are also reported. Pavement structures incorporating subgrades improved by in situ stabilisation with fly ash and cement were analyzed for construction traffic and for operating traffic. These pavements are compared with conventional flexible pavements without improved subgrades and the results clearly show the technical benefits of stabilising clayey soils with fly ash and cement. In addition TG–SDTA and XRD tests were carried out on certain samples in order to study the hydraulic compounds, which were formed.


*Keywords:* Soil stabilisation; Stabilisation with fly ash and cement; Mechanical properties of stabilised soils; Pavement analysis

## 1. Introduction

Soil stabilisation is a technique introduced many years ago with the main purpose to render the soils capable of meeting the requirements of the specific engineering projects. In this work the possibility of stabilising fine-grained plastic soils with high clay content using high calcium fly ash with and without cement is investigated. These stabilised materials may be used as improved subgrades or capping layers or sub-bases for road or airfield pavements.

Fly ash (FA) is used in a number of countries for stabilisation of soils [1–6] or for the construction of layers of hydraulically bound aggregates [7]. It should be pointed out that both the nature of the FA and the type of soil influence the results of stabilisation very much and it is very difficult and unsafe to rely on research carried out with different soils and FAs. Therefore a detailed study was considered necessary.

High calcium fly ash is produced in large quantities in Greece (over 9 million tons per year), as well as in other countries, in electric power thermal plants using lignite as the main combustion material. The fly ashes before being distributed for use are usually homogenized and processed in order to slack all the contained free lime. In this work fly ash without the latter process is used in order to take advantage of the free lime in stabilising the fine-grained plastic soils. Cement was also used as a second additive to enhance the strength, especially at early ages. In addition, X-ray diffraction (XRD) and thermogravimetric–single differential thermal analysis (TG–SDTA) tests were carried out for the most clayey soil stabilised with 10% fly ash and 10% fly ash plus 4% cement in order to study the formation of the various hydraulic compounds.

## 2. Materials used

### 2.1. Soils

Three fine-grained clayey soils (designated Soils I, II and III) were used and their properties are given in Fig. 1 in terms of Atterberg limits, particle size distribution and soil classification.

For Soil III XRD analysis was also carried out (using CuKα radiation) and the results are presented in Fig. 2.

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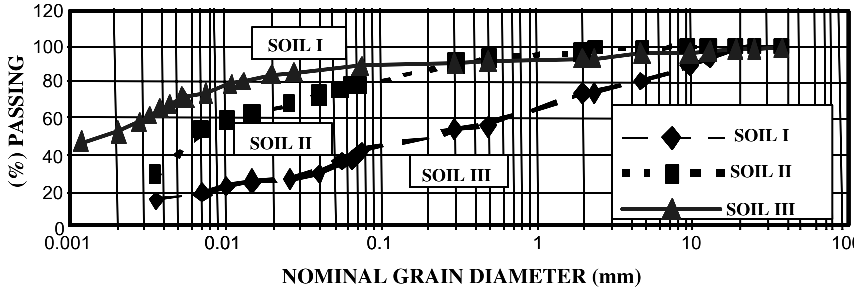


| SOIL SAMPLES | ATTERBERG LIMITS | | | AASHTO CLASSIFICATION | UNIFIED SOIL CLASSIFICATION | |
|---|---|---|---|---|---|---|
| | PL | LL | PI | | GROUP SYMBOL | GROUP NAME |
| SOIL I | 20 | 38 | 18 | A-6 | CL | LEAN CLAY |
| SOIL II | 23 | 53 | 30 | A-7-6 | CH | FAT CLAY |
| SOIL III | 18 | 43 | 25 | A-7-6 | CL | SILTY CLAY |

Fig. 1. Atterberg limits, gradation and soil classification of soil samples.

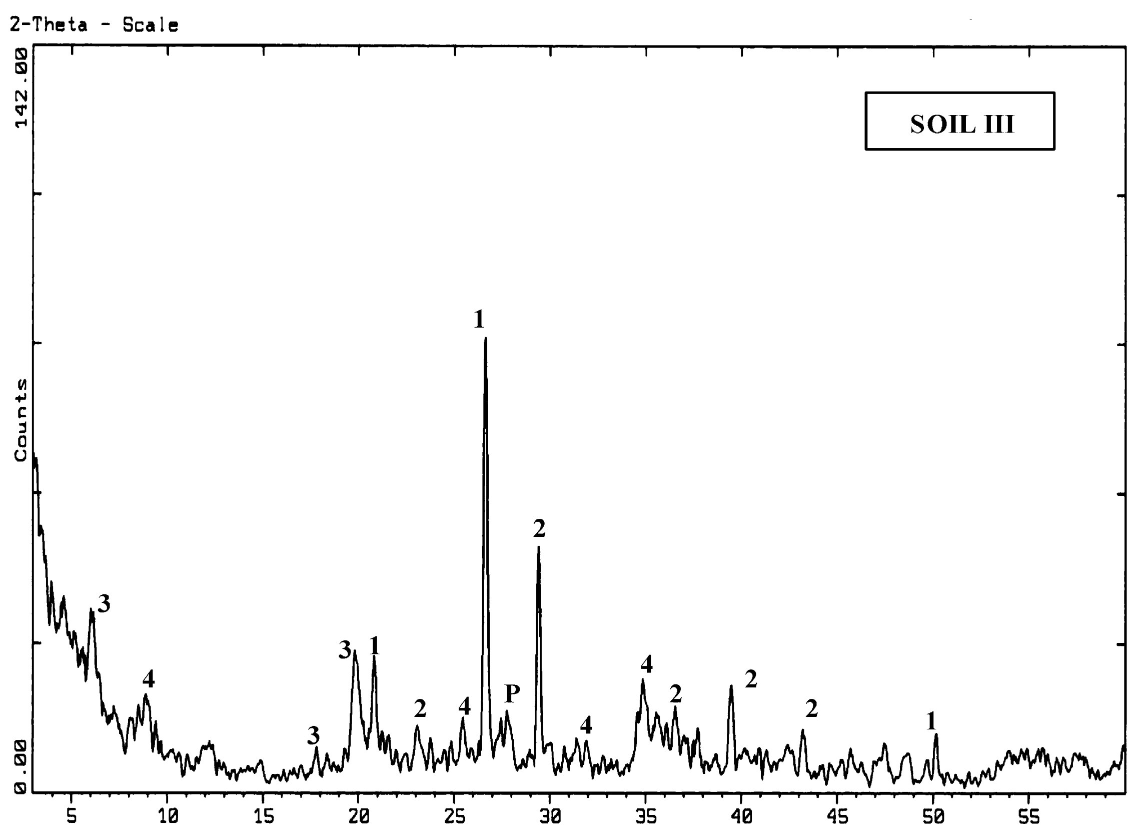


Fig. 2. XRD of Clay III sample (1: quartz, 2: $CaCO_3$ (calcite), 3: montorillonite ($Al_2O_34SiO_2 \times H_2O$), 4: muskovite ($KAl_2Si_2AlO_{10}(OH)_2$), P: plagioclase.

The main mineralogical constituents of Soil III are quartz, $CaCO_3$ (as calcite) and montmorillonite ($Al_2O_34SiO_2 \times H_2O$) with small amounts of muskovite $KAl_2Si_2AlO_{10}(OH)_2$ and plagioclase (P).

### 2.2. Fly ash

The fly ash (FA) used in the investigation was representative of a 7-day production of "Kardia" electricity power station, in Northern Greece. Two quantities were used which were collected from the power station at dates 12 months apart. Although the differences in chemical analysis (Table 1) are not considered significant, the two quantities are designated FA I and FA II. It can be seen that the free lime contents of FA are substantial 18.3% and 16.7% respectively. In Fig. 3 XRD diagram shows that the main mineralogical constituents of fly ash are: quartz, free CaO, $CaCO_3$ (calcite), $CaSO_4$ (anydrite), $Ca(OH)_2$, albite ($NaAlSi_3O_8$), and small amounts of $3CaOSiO_2$, $2CaOSiO_2$, and $3CaOAl_2O_3$.

The cement used was Portland cement II 35 produced according to Greek Standards equivalent to CEN II B-M/32.5.

Table 1
Chemical analysis of Kardia's fly ash samples

| Oxide | Percentage | |
|---|---|---|
| | Fly ash I | Fly ash II |
| $SiO_2$ | 19.9 | 25.24 |
| $Fe_2O_3$ | 5.72 | 5.08 |
| MgO | 3.65 | 4.29 |
| CaO | 48.97 | 44.77 |
| $Na_2O$ | 0.6 | 0.28 |
| $K_2O$ | 0.45 | 0.69 |
| $Al_2O_3$ | 9.26 | 10.20 |
| $SO_3$ | 7.25 | 6.49 |
| Loss of ignition | 3.01 | 2.65 |
| Free Cao | 18.31 | 16.73 |

### 2.3. Other stabilising agents—secondary investigation

In an attempt to estimate the stabilising effect of the free lime content of the fly ash some additional mixes were prepared with commercially available chemically pure powdered hydrated lime ($Ca(OH)_2$).

## 3. Mixes investigated and types of tests carried out

The quantities of fly ash used were 5%, 10% and 20% by mass of the dry soil, and of cement (combined with fly ash) 2% and 4%. Some additional mixes were prepared with only cement for comparative purposes although it was anticipated that the results would be poor due to lack of homogeneity, which is inevitable when mixing plastic fine-grained soils with cement.

The quantity of hydrated lime selected in the secondary investigation was designed to be approximately equal to the quantity of the free lime contained in the

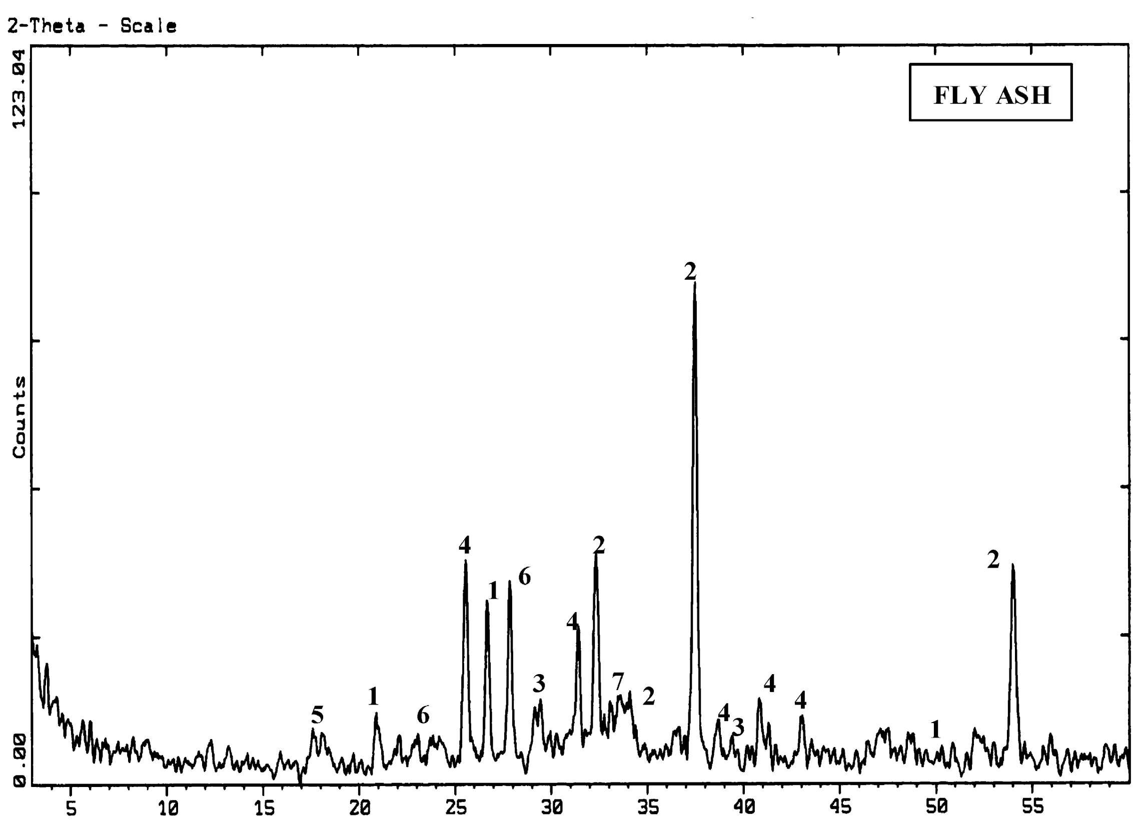


Fig. 3. XRD of fly ash sample (1: quartz, 2: free CaO, 3: $CaCO_3$ (calcite), 4: $CaSO_4$ (anydrite), 5: $Ca(OH)_2$, 6: albite ($NaAlSiO_3$), 7: $3CaOSiO_2$/$2CaOSiO_2$.

FA mixes of the main investigation in an attempt to estimate the hydraulic and pozzolanic effect of FA. It is believed that since the soils examined and the FA have similar grading characteristics, the mechanical effect of mixing of the soils with FA would be minimal.

## 4. Material preparation for tests

The oven-dry soils were initially mixed with the predetermined quantity of FA in a dry state and subsequently mixed with the water so that the mix acquired the intended moisture content. Initial mixing was carried out in a laboratory mixer for at least 2 min and the mix was subsequently put into plastic bags, where the mixing was continued by shaking and overturning the bag for at least 5 min. Finally, the air was squeezed out by hand and the bags were sealed and stored in the curing room ($20 \pm 1$ °C, $96 \pm 2$% RH) for 20–24 h. Before use the material was remixed again in the plastic bag by hand shaking, overturning and squeezing the bag. It was observed that the mix prepared with this method contained no appreciable number of lumps larger than 5 mm. When mixes combining FA and cement were prepared the necessary quantity of cement was added to the mix during this operation.

## 5. Preparation of specimens

All specimens were prepared with the static compaction method (BS 1924, Test 10) at the optimum moisture content and maximum density determined by the standard compaction test (BS 1924 Test 3). The specimens were demoulded 1 min after completion of the compaction, were wrapped with thin plastic film and were stored in the curing room until testing at 7, 28 and 90 days. The specimens examined for the effect of water immersion were cured as the other specimens (wrapped in plastic sheets in the curing room) for 83 days and then were put into water containers stored in the curing room until testing at 90 days.

## 6. Testing methods

Cylindrical specimens ($\varnothing = 50$ mm, $H = 100$ mm) were used for unconfined compressive and indirect tensile (splitting) strength tests. For modulus of elasticity determinations larger cylindrical specimens ($\varnothing = 70$ mm, $H = 140$ mm) were used so that the demountable strain measuring equipment could be accommodated. The compressive and indirect tensile strengths were determined on a simple constant speed cross-head moving machine at a speed of 1 mm/s. Bending tests and uniaxial tensile were also carried out on larger specimens (100 mm × 100 mm × 500 mm) which were compacted by the vibrating hammer method (BS 1924 Test 5). The flexural strengths are only given in this report.

The modulus of elasticity was determined on a servo-hydraulic testing machine with the following types of loading:

(a) Constant rate of stress increase of 0.2 $N/mm^2/s$ up to 1/3 of the compressive strength.
(b) Monotonically stress increase so that it reached 1/3 of the compressive strength in 0.3 s.
(c) Sinusoidal stress variation between 0.1 and 1/3 of the strength at a frequency of 1.67 Hz.
(d) Constant rate of deformation of 1 mm/min up to failure.

The above tests gave the opportunity to access the influence of the type of loading on the value of the modulus of elasticity. Type "a" loading is within the range of standard testing of cementitious materials. Type "b" and "c" loading simulate the stress rate imposed by moving vehicles on pavement subgrades [8] and type "d" loading was used in an effort to obtain the complete stress–strain relationship at a deformation rate similar to that imposed by traffic [8].

## 7. Presentation and discussion of the results

### 7.1. Atterberg limits

All the materials (Soils I, II and III) became non-plastic 24 h after mixing with 5%, 10%, 20% FA by mass. In addition, all materials became friable 24 h after the initial mixing with FA and the lumps were minimised after remixing.

### 7.2. Compaction tests

The results of compaction tests (standard compaction BS 1924) are given in Fig. 4a and b for Soil I in terms of initial moisture content of the mix i.e. calculated from the amount of added water (Fig. 4a) and of the measured moisture content (Fig. 4b) from samples taken from the compacted specimen. Similar results were obtained for the other soils. There is a difference about 2 percentile units between the two moisture contents, which is primarily attributed to the chemical combination of the water mainly with the free lime in the FA since minimal evaporation could take place as every precaution was taken to avoid drying during the procedures of mixing compacting and storing. It can be seen that—as in the case of lime stabilisation—the maximum dry density is decreased and the optimum moisture content is increased as the fly ash content is increased (from 5% to 20%). It should be pointed out that the high

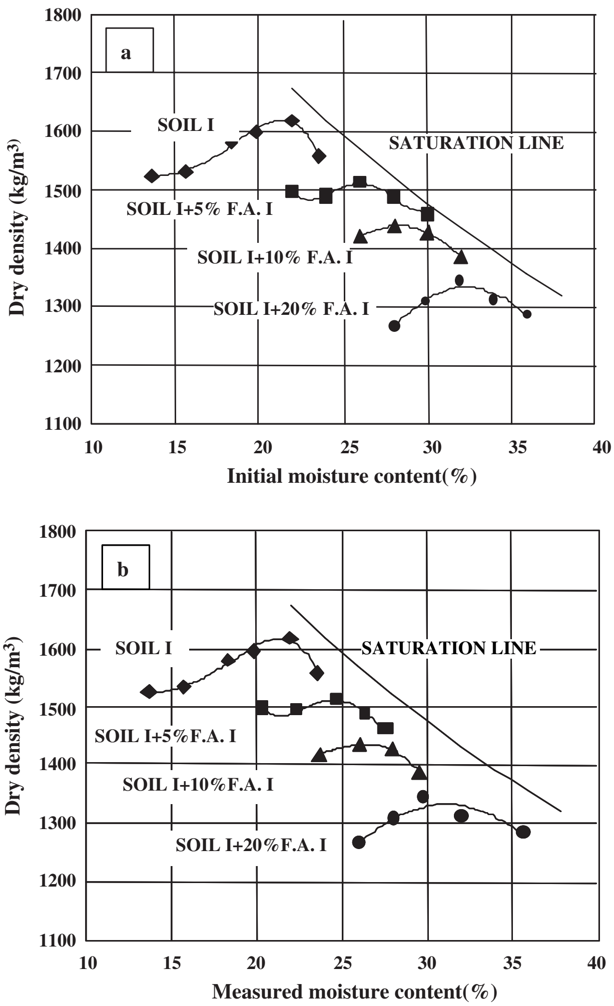


Fig. 4. Standard compaction test for Clay I: (a) initial moisture and (b) measured moisture.

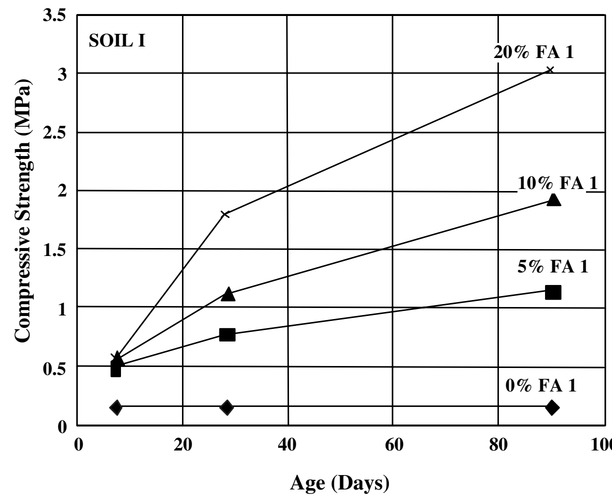


Fig. 5. Effect of fly ash addition on uniaxial compressive strength—Clay I.

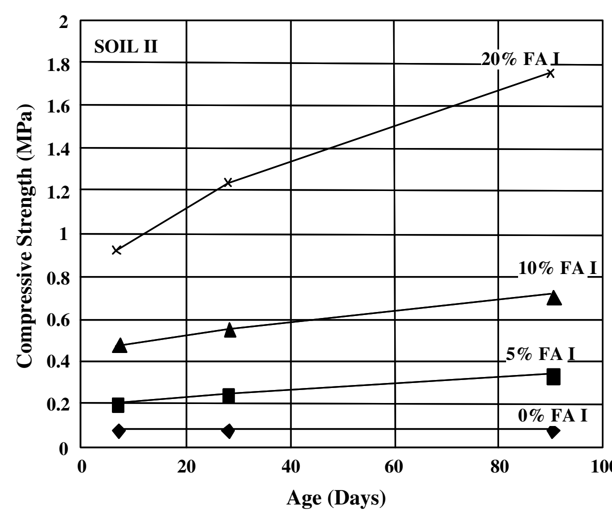


Fig. 6. Effect of fly ash addition on uniaxial compressive strength—Clay II.

optimum moisture contents may be difficult and costly to achieve during dry seasons. On the other hand during wet seasons the high water demand of FA will facilitate compaction.

### 7.3. Strength tests

Figs. 5–7 show the development of the unconfined compressive strength in relation to curing time for Soils I, II and III respectively. It can be seen that considerably higher compressive strengths are obtained with Soil I than with Soils II and III.

The effect of hydrated lime on the strength gain of the mixes is shown in Fig. 8. Since the strength values obtained with lime are considerably lower than the values obtained with FA (the strengths in Fig. 8 with $Ca(OH)_2$ contents 0.92%, 1.83% and 3.66% are compared with the strengths for 5%, 10% and 20% FA contents in Soil I respectively in Fig. 5) it is inferred that the effect of fly ash on strength is not due to its free lime content alone but also to hydraulic and pozzolanic reactions. The effect of combining 2% and 4% cement with fly ash is shown in Figs. 9–11 for Soils I, II and Soil III respectively. The early strengths are higher for cement. It should be noted that the 90-day strength of both Soil I, and Soil II are higher when 20% of fly ash is used than with the combination of 10% fly ash and 2% or 4% cement. However, in the case of Soil III the effect of combining cement with FA is more pronounced since the 90-day strengths are increased by two to six times. It is evident therefore, that the soil type greatly influences the results.

The use of high percentages of fly ash is, in certain cases, more effective than the combination of FA and cement but the problems associated with the use of large quantities of fly ash would need to be addressed. Some

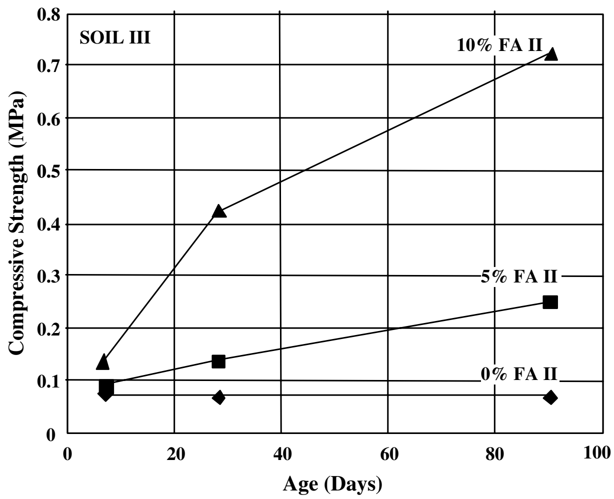


Fig. 7. Effect of fly ash addition on uniaxial compressive strength—Clay III.

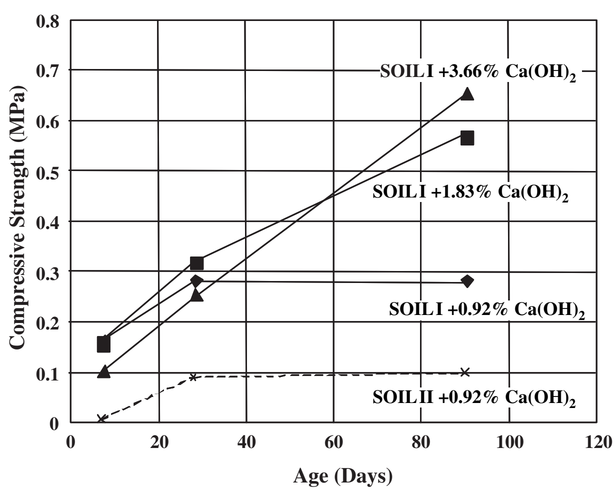


Fig. 8. Effect of $Ca(OH)_2$ content on uniaxial compressive strength—Clays I and II.

of these problems may be: transport costs, practical problems of spreading and mixing these large quantities of FA and increased water demand. On the other hand there are soils (such as Soil I or III), which may be satisfactorily stabilised with small percentages of FA and cement.

The beneficial effect of combining the two stabilising agents can also be estimated by comparing (Fig. 11) the strengths of Soil III stabilised with 2% or 4% cement with those obtained when 5% or 10% FA is combined with 2% or 4% cement. This beneficial effect is attributed to the transformation of the soil due to FA that allows better distribution of cement and increases its effectiveness. Figs. 12 and 13 give the splitting tensile strength development of Soils I, II and III for various percentages of fly ash or fly ash and cement. As in the case of

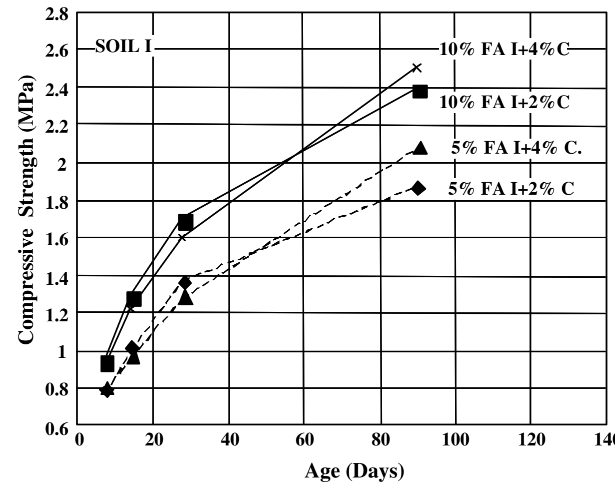


Fig. 9. Effect of combination of fly ash and cement addition on uniaxial compressive strength—Clay I.

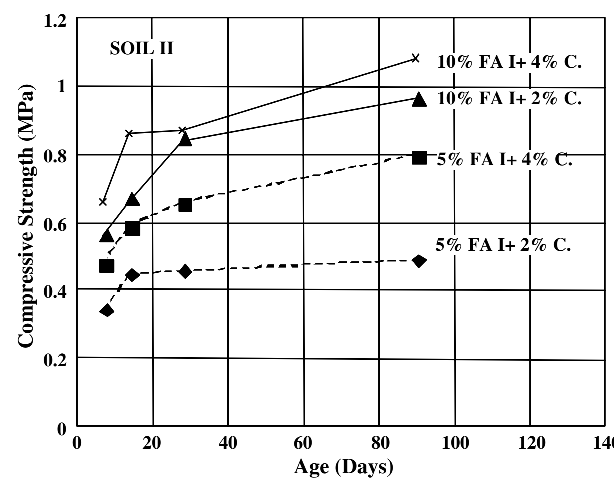


Fig. 10. Effect of combination of fly ash and cement addition on uniaxial compressive strength—Clay II.

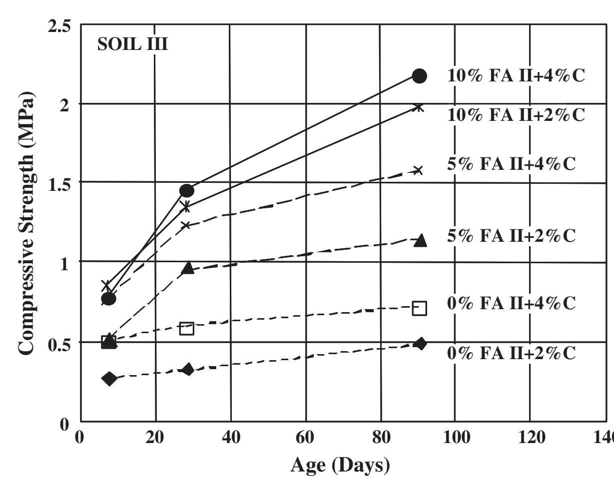


Fig. 11. Effect of combination of fly ash and cement addition on uniaxial compressive strength—Clay III.

compressive strength, fly ash reacts more favourably with Soil I than with Soil II or Soil III and the effect of combining cement with FA is significant. The relationship between of splitting tensile strength and uniaxial compressive strength is shown in Fig. 14.

Fig. 15 shows the development of flexural strength with age for Soil III with 10% FA and 10% FA ash combined with 4% cement.

### 7.4. Effect of water immersion

The effect of 7-day water immersion on the 90-day compressive strength is shown in Table 2. It can be seen that the ratio of the strength after 7-day immersion to that of normal curing is higher than 0.8 except for the case of Soil II with 5% and 10%, where the ratio is 0.47 and 0.76 respectively.

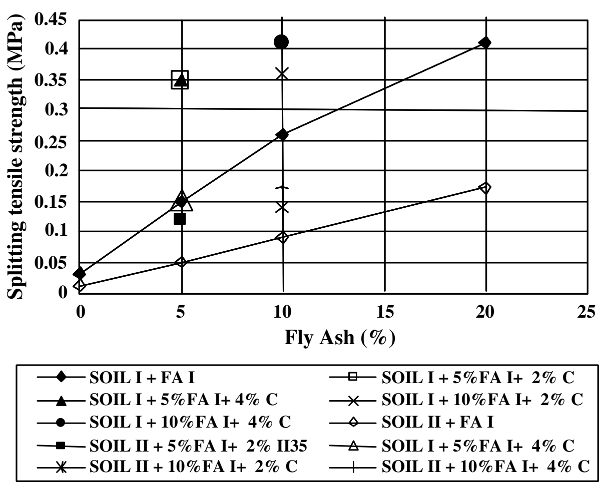


Fig. 12. Effect of combination of fly ash and cement addition on tensile (splitting) strength—Clays I and II.

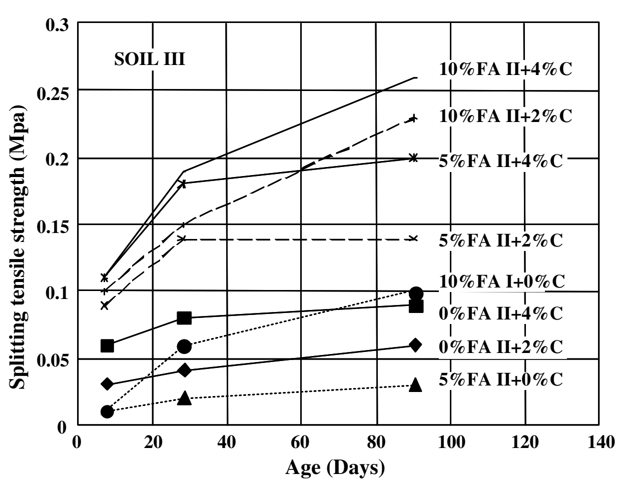


Fig. 13. Effect of combination of fly ash and cement addition on tensile (splitting) strength—Clay III stabilised with cement and or fly ash.

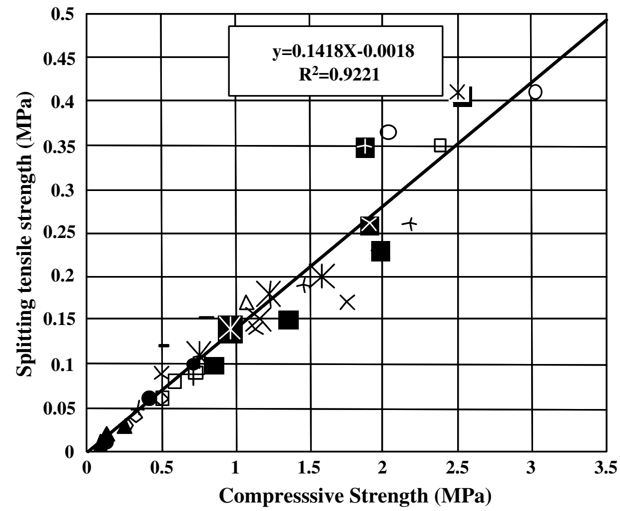


Fig. 14. Relationship between compressive and splitting tensile strength of Clays I, II and III.

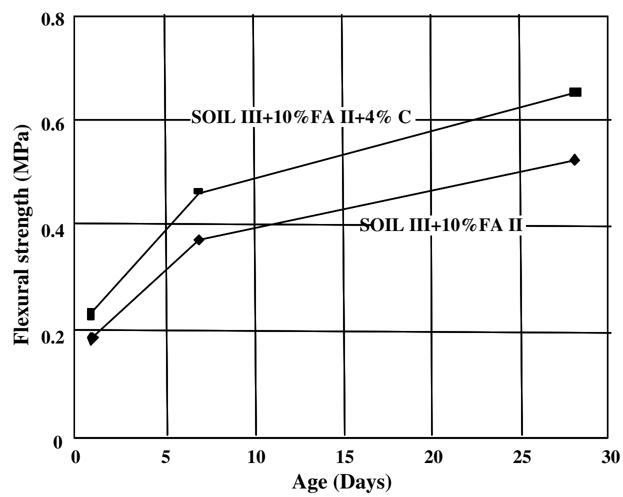


Fig. 15. Effect of combination of fly ash and cement addition on flexural strength—Clay III stabilised with cement and or fly ash.

### 7.5. Modulus of elasticity

Figs. 16 and 17 show typical stress–strain curves up to 1/3 of the ultimate load for loading types a, b and c for Soil I stabilised with 20% FA at an age of 90 days. The stress–stain curve is essentially linear up to 1/3 of the strength, a fact that has been observed elsewhere for lime or cement stabilised fine-grained materials [9–11]. It can be seen (Fig. 16) that the effect of the loading rate (loading type a and b) or the number of loading cycles (Fig. 17 the first and the 100th loading cycle) does not create any appreciable difference in the stress–strain curve. Fig. 18 shows a complete stress–strain curve in

Table 2
The effect of 7-day water immersion on 90-day compressive strength

| Soil samples | Fly ash (%) | Cement (%) | Compressive strength after immersion (MPa) | Percentage of standard cured compressive strength (%) |
|---|---|---|---|---|
| Clay I | 5 | 0 | 0.95 | 81.9 |
| | 5 | 2 | 1.65 | 88.2 |
| | 5 | 4 | 2.07 | 86.6 |
| | 10 | 0 | 1.79 | 93.7 |
| | 10 | 2 | 1.77 | 85.5 |
| | 10 | 4 | 2.35 | 94.0 |
| | 20 | 0 | 2.79 | 92.1 |
| Clay II | 5 | 0 | 0.16 | 47.1 |
| | 5 | 2 | 0.39 | 79.5 |
| | 5 | 4 | 0.66 | 82.5 |
| | 10 | 0 | 0.54 | 76.01 |
| | 10 | 2 | 0.80 | 83.33 |
| | 10 | 4 | 0.99 | 91.67 |
| | 20 | 0 | 1.42 | 83.0 |

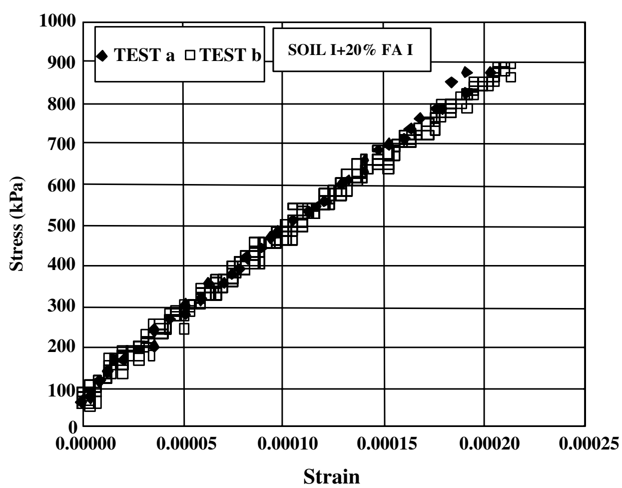


Fig. 16. Typical stress strain diagram for constant rate stress application. Clay I with 20% fly ash I.

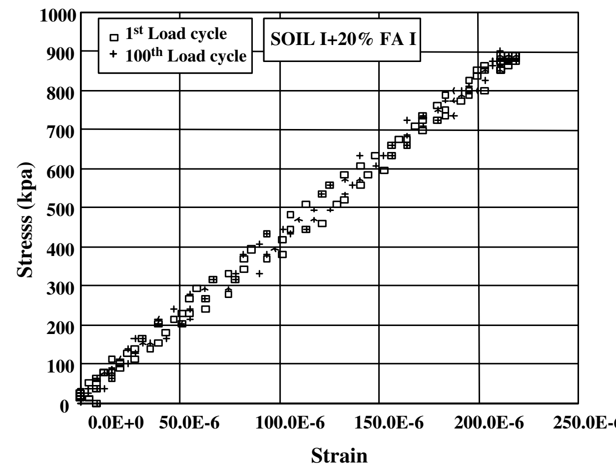


Fig. 17. Typical stress strain diagram for sinusoidal loading for the first and hundredth load application. Clay I with 20% fly ash I.

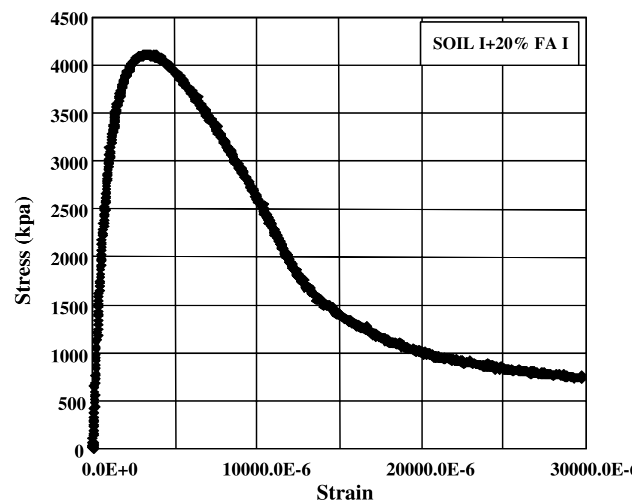


Fig. 18. Typical stress strain diagram for constant rate of straining. Clay I with 20% fly ash I.

compression (Soil I + 20% FA) obtained by applying a constant rate of straining of $333\times10^{-6}$ $s^{-1}$. In general, the three types of loading examined have little effect on the stress–strain relationship and this is reflected in the 90-day modulus of elasticity values in Figs. 19 and 20. It is noted that the values of modulus of elasticity are high and as in the case of strength the modulus of elasticity of Soil I is higher than that of Soil II.

### 7.6. CBR tests

Fig. 21 shows the 90-day old (24-h soaked) CBR values in relation to soil type and fly ash content. It can be seen that as in the case of strength, much-improved CBR values are obtained in case of Soil I while in case of Soils II and III the increases are not as high. It should be noted however, that the 15% minimum CBR value usually required by many specifications is by far attained by the three soils with only 5% fly ash. The relation of CBR vs compressive strength is shown in Fig. 22. It can be seen that, for the three fine-grained soils examined, a linear relationship between CBR and strength exists although this applies strictly to the soils examined. Similar relationships have been found for cement-stabilised soils by Maclean as referred to in Ref. [11].

### 7.7. XRD investigation

In Figs. 23 and 24, representative X-ray diagrams are presented for 7-day and 6-month old specimens stabi-

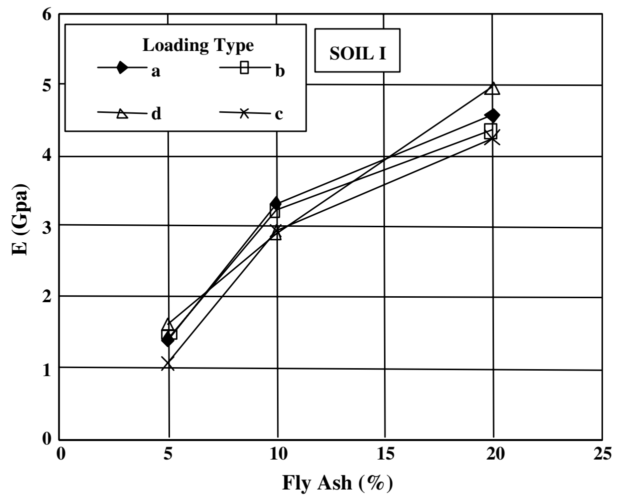


Fig. 19. Effect of loading type on the modulus of elasticity for various percentages of fly ash. Clay I.

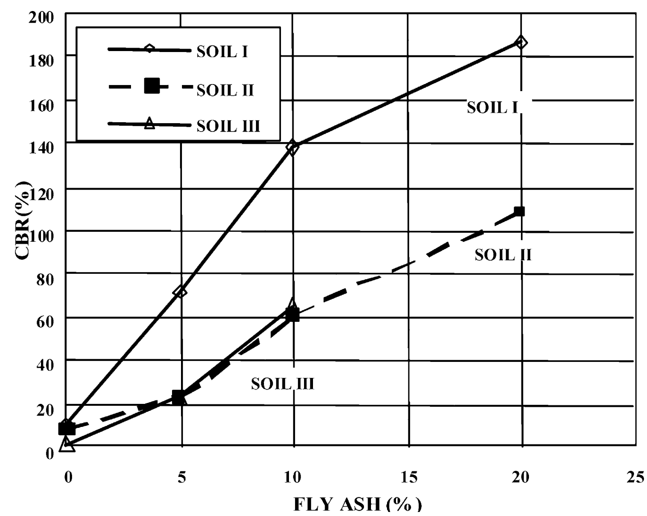


Fig. 21. Variation of Californian Bearing Ratio for Clays I, II and III stabilised with fly ash.

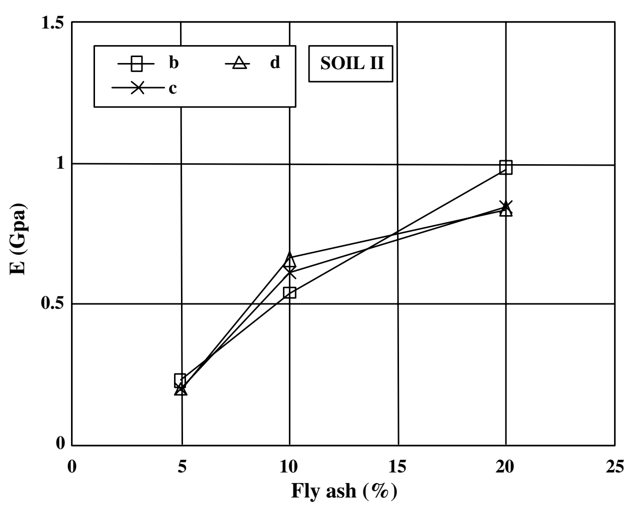


Fig. 20. Effect of loading type on the modulus of elasticity for various percentages of fly ash. Clay II.

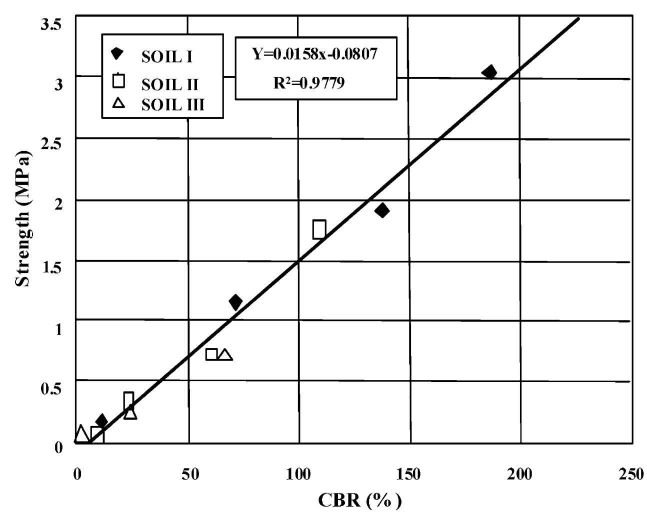


Fig. 22. Relation between CBR and strength in uniaxial compression.

lised with 10% FA (designated FA-7d and FA-6m) and 10% FA plus 4% cement (designated FAC-7d and FAC-6m). The possible hydraulic compounds that have appeared are products of the cement or fly ash hydration, these include e.g. various types of tobermorites, calcium aluminium silicate hydrates like gismodine ($CaAl_2Si_2O_8 \cdot 4H_2O$), portlandite ($Ca(OH)_2$), etc. as well as products resulted of the reaction mainly of the $SiO_2$ contained in clay with the portlandite released during the hydration of fly ash and cement. In Fig. 23 it is observed that after curing for 7 days the Soil III sample containing 4% cement appears to have increased amounts of $Ca(OH)_2$, tobermorites 11 and 14 Å ($Ca_5(Si_6O_{18}H_2) \cdot 4H_2O$) (27.8°, 2$\theta$) and riverseidite. This is attributed to the effect of the addition of cement which has enhanced the effect of fly ash, offering easy production of hydraulic compounds, like tobermorites especially at the early ages. A comparison between XRD diagrams of samples cured for 7 days and 6 months (Fig. 24) shows that there is a remarkable increase of the compounds corresponding to tobermorite as well as the decrease of $SiO_2$ which has reacted with $Ca(OH)_2$.

### 7.8. TG–DTG–SDTA study

The thermal profile of the samples studied, the weight loss at the various stages of thermal reactions as well as the 1st derivative of the weight loss are presented in Figs. 25 and 26. In Table 3 the temperature changes concerning the various compounds are shown. It can be observed that due to the percentage of clay the various

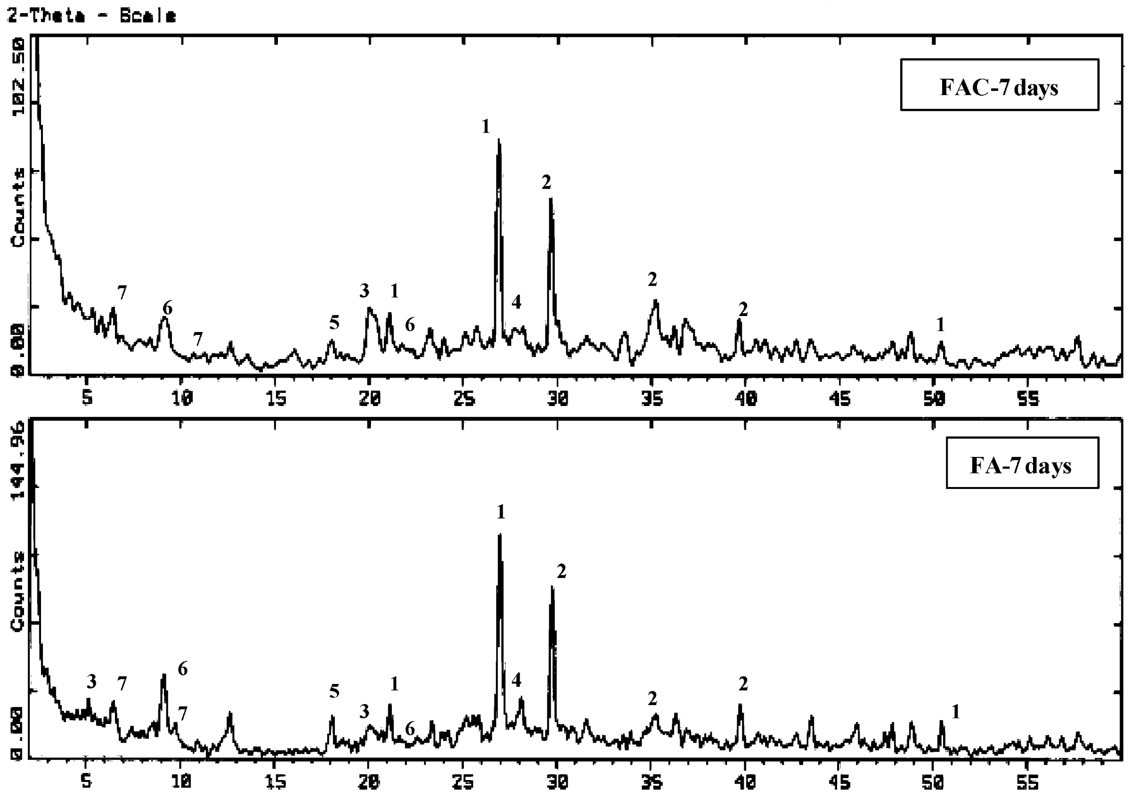


Fig. 23. XRD-diagram of FAC-7d and FA-7d samples (1: $SiO_2$ quartz, 2: $CaCO_3$, 3: $Al_2O_34SiO_2 \times H_2O$, 4: tobermorite, 5: $Ca(OH)_4$, 6: ettringite, 7: riverseidite).

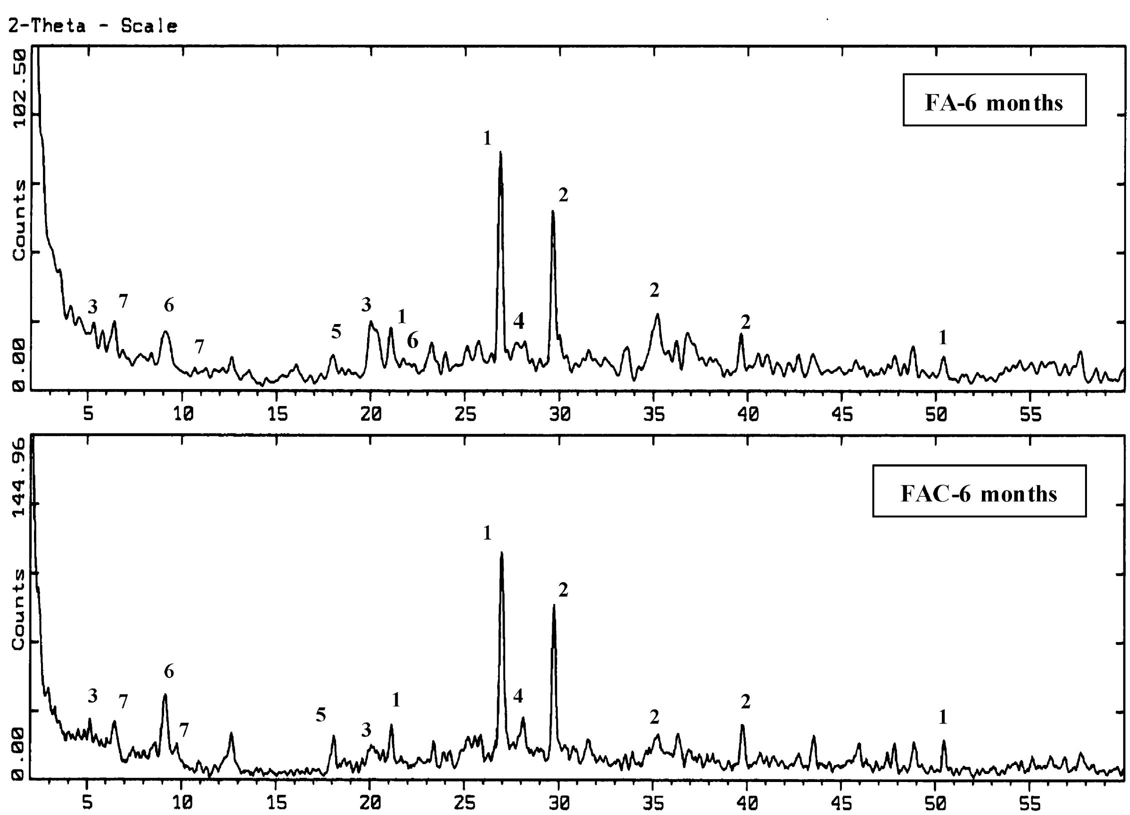


Fig. 24. XRD diagram of FAC-6m and FA-6m samples (1: $SiO_2$ quartz, 2: $CaCO_3$, 3: $Al_2O_34SiO_2 \times H_2O$, 4: tobermorite, 5: $Ca(OH)_4$, 6: ettringite, 7: riverseidite).

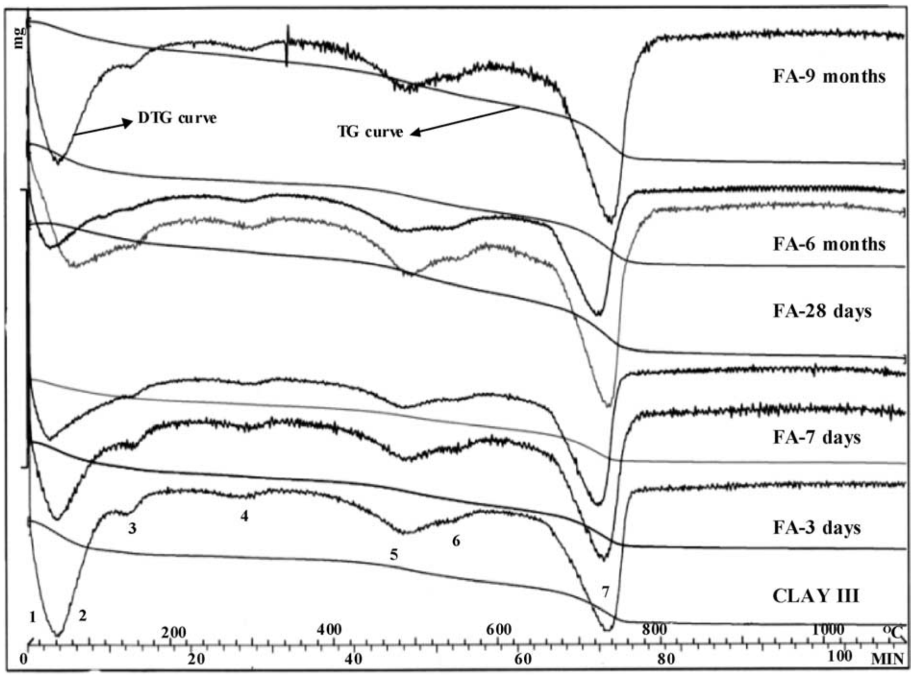


Fig. 25. TG–DTG curves for the mixture of Clay III and fly ash (1: sorbed moisture, 2: dehydration of tobermorite, 3: ettringite, 4: 1st dehydroxylation of montmorillonite, 5: 2nd dehydroxylation of montmorillonite, 6: $Ca(OH)_2$, 7: main dehydroxylation of montmorillonite).

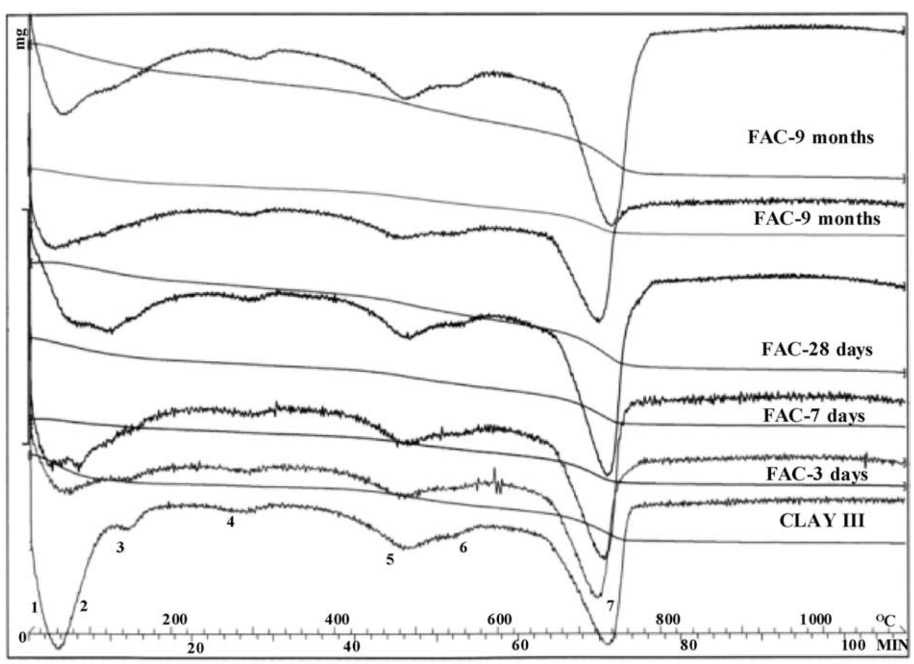


Fig. 26. TG–DTG curves for the mixture of Clay III fly ash and cement (1: sorbed moisture, 2: dehydration of tobermorite, 3: ettringite, 4: 1st dehydroxylation of montmorillonite, 5: 2nd dehydroxylation of montmorillonite, 6: $Ca(OH)_2$, 7: main dehydroxylation of montmorillonite).

thermal peaks, which are attributed to the hydraulic products formed, are almost overlapped by the peaks for clay minerals. For this reason a quantitative calculation of tobermorite 14 Å was performed at the temperature range 60–140 °C, taking into account the corresponding amount of clay constituents in this range. The weight loss corresponding the dehydroxylation of tobermorite 14 Å in the temperature range 60–140 °C is shown in Table 4. From this table it may be concluded that a continuous formation of tobermorite takes place

Table 3
SDTA peaks

| Temperature range (°C) | Corresponding compound |
|---|---|
| 0–60 | Sorbed moisture |
| 60–140 | Tobermorite 14, 11 Å |
| 140–200 | Ettringite |
| 200–300 | Montmorillonite (1st dehydroxylation) |
| 400–490 | Montmorillonite (2nd dehydroxylation) |
| 490–510 | $Ca(OH)_2$ |
| 510–600 | $SiO_2$ |
| 600–780 | Main dehydroxylation of montmorillonite |
| 780–900 | $CaCO_3$ decomposition |

Table 4
Tobermorite 14 Å dehydration

| Sample | Weight loss (%) |
|---|---|
| FA[a]-3 days | 0.71 |
| FA-7 days | 0.77 |
| FA-28 days | 0.82 |
| FA-6 months | 1.10 |
| FA-9 months | 1.12 |
| FAC[b]-3 days | 0.78 |
| FAC-7 days | 0.80 |
| FAC-28 days | 0.92 |
| FAC-6 months | 1.05 |
| FAC-9 months | 1.07 |

[a] Fly ash.
[b] Fly ash and cement.

up to a 6-month hydration. The tobermorite formation rate seems to be lower after the 6 months curing period. The phenomenon is more intense in the samples containing cement mainly at the early ages. This means that the cement addition offers the benefit of better setting and hardening behaviour at the early ages.

### 7.9. Implications in pavement design

Stabilised layers of fine-grained soils are prone to restrained movement cracking caused by drying shrinkage and thermal contraction. It is therefore believed imperative that stabilised layers of this type receive adequate curing immediately after completion of compaction. On the other hand, construction traffic may cause also severe cracking. This can happen if traffic is allowed when the material is hardened enough to acquire a substantial percentage of its modulus of elasticity but is not strong enough to withstand the stresses imposed by the traffic. In order to take best advantage of the high values of modulus of elasticity and consequently reduce the required pavement thickness, it is recommended that a 200 mm thick protective overlay of unbound well graded crushed granular material is placed immediately after compaction of the stabilised layer. This will ensure efficient curing of the stabilised

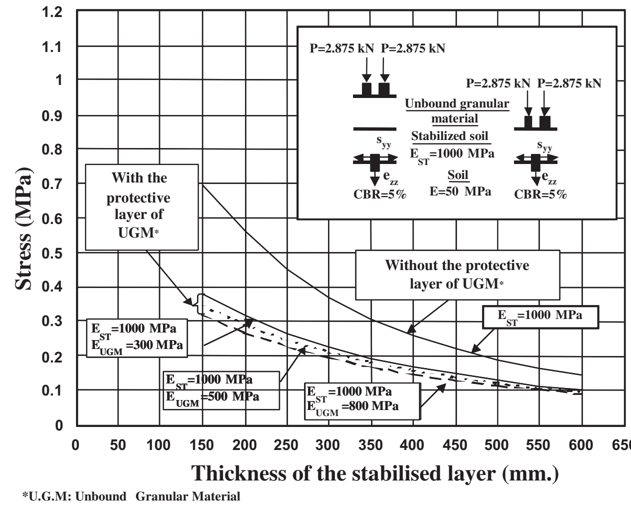


Fig. 27. Stress $\sigma_{YY}$ vs thickness of stabilised layer.

layer and simultaneously reduce any stresses induced by site traffic (Fig. 27). Since the strength development of this type of stabilised material is slow, the construction of the unbound layer will not cause any damage to the "green" stabilised layer if the construction is carried out immediately after compaction of the stabilised layer. The unbound granular material should also eliminate any danger of reflection cracking. Fig. 27 shows the flexural stresses developed at the bottom of the stabilised layer due to single axle load of 115 kN on double tires. The beneficial effect of the protective layer of unbound granular material is evident. If an allowable flexural tensile stress of 0.2 MPa for a limited number of load repetitions (construction traffic) is assumed it can be seen that the required thickness of the stabilised layer is approximately 480 mm no immediate covering of unbound granular layer is used compared with 350 mm when a 200 mm thick overlay is used.

Fig. 28 shows two equivalent pavements designed so that they have the same subgrade strain $\varepsilon_{zz}$ (same deformation) and the same horizontal strain $\varepsilon_{rr}$ at the bottom of the asphalt layer (same fatigue behaviour). Pavement A consists of 100 mm thick asphalt layer, 350 mm thick unbound well graded crushed stone layer and a stabilised layer 350 mm thick on a subgrade of CBR = 3%. Pavement B is a conventional flexible pavement consisting of an asphalt layer 260 mm thick, an unbound granular material 400 mm thick and an improved 500 mm thick course of imported material constructed over the same subgrade of CBR 3%. The analysis of both pavements was made taking into consideration the modulus of elasticity/stiffness values shown in the figures. It can be seen that a considerable saving in asphalt thickness for pavement A illustrating the beneficial effect of placing an in situ stabilised layer on top of a poor subgrade.

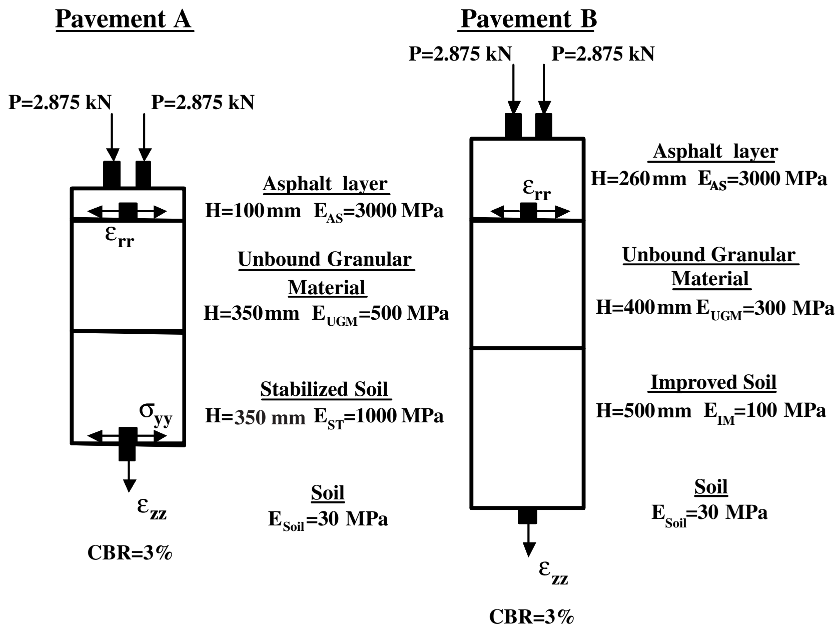


Fig. 28. Equivalent pavements.

## 8. Conclusions

This work shows that the potential benefit of stabilising clayey soils with high calcium fly ash but this depends on the type of soil, the amount of stabilising agent and the age.

The study of the formation of the hydraulic products during the curing of clay containing as a stabilising agent high calcium fly ash shows that a significant amount of tobermorite is formed leading to a denser and more stable structure of the samples. A further addition of cement provides better setting and hardening and the combination of these two binders can increase the early as well the final strength of the stabilised material. The free CaO of fly ash reacts with the clay constituents ($SiO_2$ and the other aluminium silicates) leading to the formation of tobermorites and calcium aluminium silicate hydrates as well.

The mechanical properties such as strength (compressive, tensile and flexural), modulus of elasticity and CBR are considerably enhanced. If suitable measures are taken in order to avoid or minimise cracking of the stabilised layer and maintain the high modulus values, substantial reductions of the total pavement thickness, and in particular of the asphalt course, may be achieved.

However, the above pavement analysis is based on laboratory values and in situ tests are needed in order to find more realistic values of the mechanical properties of the stabilised layer.

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