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STUDY OF ENGINEERING BEHAVIOUR OF EXPANSIVE CLAYS DUE TO ADDITION OF LIME

Sukumar Joshi
Aryan Institute of Engineering & Technology, Bhubaneswar

ABSTRACT: Adjustment of expansive clays with synthetic substances has been found to decrease expanding. This paper presents impact of lime on Free Swell Index (FSI), expand potential, enlarging pressure, combination attributes, compaction qualities and strength conduct. Lime content was shifted as 0%, 2%, 4% and 6%. Expand potential and enlarging pressure diminished with expansion in lime content, yet simply up to 4% lime. Expansion of lime past 4% brought about expanded swell potential and enlarging pressure. Solidification qualities improved with expanding lime content as reflected in pressure record, coefficient of union and optional union. Compaction and strength qualities worked on up to 4% lime content.

1. INTRODUCTION

Problems with Expansive Soils

Expansive soils swell on imbibition of water during monsoon, reduce in density and become slushy. But in dry seasons, they shrink because of evaporation of water, and become hard due to increase in density (Mckeen 1988). The alternate swelling and shrinkage causes distress to civil engineering structures built on these soils are severely damaged (Chen 1988, Nelson & Miller 1992). The annual cost of damage to buildings, structures and roads caused by expansive soils is estimated at £150 million in the UK, $1000 million in the USA, and many billions of pounds world-wide (Gourley *et al.* 1993).

Stabilisation Techniques

Various innovative foundation techniques have been devised for ameliorating problems with expansive soils, the chief of which are sand cushion, Cohesive Non-Swelling (CNS) layer, belled piers (Chen 1988) and Granular Pile-Anchor (GPA) foundation (Phanikumar 1997). Chemical stabilization, in which chemicals are added to expansive clays for reducing heave, also met with success. Lime has been found to be the most effective and economical of all additives. Addition of lime to expansive soils reduces swell potential and increases workability and strength.

Lime columns or lime-soil columns were also tried to stabilize expansive clays *in situ* (Rao 1984). It was reported that diffusion of lime is effective up to a radial distance of about 3 times the diameter of lime-soil column. Flocculation, cation exchange and pozzolonic action are responsible for the amelioration effects in lime-soil mixes. The lime content required to improve the physical properties of soils, which is the lime fixation point, varies between 3 to 10% depending on the soil. Lime-soil blends were reported to have developed substantial tensile strength also. This paper presents an experimental study on effect of lime on characteristics of a remoulded expansive soil including swell potential, swelling pressure, secondary consolidation, compaction and strength.

2. EXPERIMENTAL PROGRAMME

Test Materials and Variables

The expansive soil used in the test programme was collected from a depth of 1.2 m below the ground level. It had a Free Swell Index (FSI) of 165%. The index properties of the soil are shown in Table 1. Unslaked lime was used as the additive.

Table 1: Index Properties of Expansive Soil

Specific gravity	2.72
Liquid limit (%) Plastic limit Plasticity index	100 27 73
Gravel (%) (>6.20–4.75 mm)	0
Sand (%) (4.75–0.075 mm)	7
Silt (%) (0.075–0.002 mm)	24
Clay (%) (<0.002 mm)	69
Free Swell Index (FSI)	250
USCS classification	CH

In the tests done for the evaluation of swelling properties, the initial water content (w_i %) and the dry unit weight ($?_{di}$, kN/m^3)

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of the soil were kept constant at 0% and 13 kN/m^3 respectively. An initial water content of 0% was chosen to obtain measurable values of swell potential and swelling pressure. Lime content (by dry weight of expansive soil) was varied as 0%, 2%, 4% and 6%. Two samples were tested for the evaluation of each parameter, and the average reported.

Tests Performed, Sample Preparation and Test Procedures

Free Swell Index (FSI) Test

Soil fraction passing 425 µm sieve was oven-dried at a temperature of 105°C. Ten grams of the oven-dried soil were poured separately into two graduated cylindrical glass jars of 100 ml capacity, one containing distilled water and the other kerosene. After allowing the jars to stand for 24 hours, the final volumes of the soil in the jars were noted. FSI was calculated (Holtz & Gibbs 1956) as,

$$\text{FSI} = [(V_s - V_k)/V_k] \times 100 \qquad (1)$$

where V_k = Volume of soil in kerosene, and
V_s = Volume of soil in water.

Two samples were tested and the average FSI reported.

One Dimensional Swell-Consolidation Tests

One-dimensional swell-consolidation tests by free swell method (ASTM 1996, D4546) were performed on oven-dry soil passing 4.75 mm sieve. Lime was thoroughly mixed with the oven-dry soil sample and compacted in the consolidation ring (diameter 60 mm and height 20 mm) to the predetermined dry unit weight. The samples were compacted in four layers, each layer having a thickness of 5 mm to ensure a uniform dry unit weight. A filter paper and a porous stone were placed at each end of the sample. This unit was placed in the oedometer and the loading pad positioned centrally on the top porousstone. This assembly was mounted on the loading frame and a surcharge of 5 kPa was placed on the loading hanger after setting the dial guage reading to zero. The sample was allowed free swell by inundation. After the sample reached equilibrium heave, it was subjected to consolidation till placement void ratio was reached. A plot was made between void ratio (e) shown as the ordinate and the applied pressure plotted on logarithmic scale (log p), shown as the abscissa. The maximum load intensity applied was 640 kPa, which was kept on the specimens for 48 hours to allow secondary consolidation. Swell potential was obtained as the ratio of the increase in thickness (? H) to the original thickness of the specimen (H), expressed as a percent. The pressure on the *e-log p* plot, corresponding to the initial void ratio of the specimen was recorded as the swelling pressure.

Proctor Compaction Tests

The standard Proctor compaction test (ASTM 2000, D698a) was performed on soil-lime blends to study the Maximum Dry Density (MDD) and the Optimum Moisture Content (OMC) of the blends. Each blend was allowed a homogenization time of 20 minutes before compaction.

Unconfined Compression Tests

The test was conducted according to ASTM 2000, D2166 on cylindrical soil samples obtained from the Proctor compaction mould in which the blends were compacted at the respective MDD and OMC. The blends were allowed a homogenization time of 20 minutes. The failure stress or peak compressive stress was recorded. If failure occurred before 20% strain, the corresponding stress was recorded as the failure stress. However, if failure exceeded 20% strain, the stress corresponding to 20% strain was recorded as failure stress.

3. DISCUSSION OF TEST RESULTS

Effect of Lime on Plasticity Index

Figure 1 shows the effect of lime on Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PI) of the expansive soil. LL decreased and PL increased with increasing lime content resulting in decreased PI. Flocculation takes place when lime is mixed with clays. As the concentration of lime is increased, there is a reduction in clay content and a corresponding increase in the percentage of coarse particles. This results in a reduction of PI. At a lime content of 4%, a maximum reduction in PI of 36% was obtained. When the lime content was increased beyond 4%, there was no further change in PI.

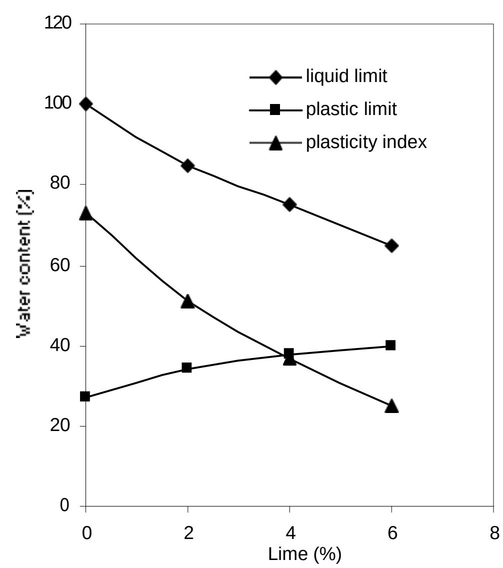

Fig. 1: Effect of Lime on LL, PL and PI

Effect of Lime on Swelling Behaviour

Free Swell Index (FSI) decreased with increasing lime content (Table 2). Surface activity decreases with flocculation, reducing FSI. FSI indicates potential for swell which is a result of

Table 2: Summary of Test Results

Lime (%)	*Liquid limit (%)*	*Plastic limit (%)*	*Plasticity index*	*FSI (%)*	*S (%)*	p_s (kPa)	*MDD* (kN/m^3)	*OMC (%)*	*UCS (MPa)*	C_c (cm^2/kg)	$c_v \times 10^{-3}$ (cm^2/sec)	C_α
0	100	27	73	250	26.7	330	13.6	34	0.19	0.5	0.32	0.013
2	85	34	51	220	13.70	210				0.64	3.86	
4	75	38	37	180	3.0	160	13.8	29	0.20	0.17	2.63	0.00
6	65	40	25	200			13.7	28	0.18			

chemical activity at the colloidal level. When lime content was increased beyond 4%, FSI increased. FSI reduced by 27% at 4% lime content (see Table 2). Figure 2 shows the rate of heave for varying lime contents (0%, 2%, 4% and 6%). The rate of heave is presented in plots showing time (minutes) on X-axis and heave (mm) on Y-axis. The untreated expansive clay (0% lime) attained a heave of 5.4 mm (at a swell potential of 27%) in three days. The curve indicated increase in heave even after 3 days (4320 minutes). However, heave decreased upon addition of lime. Moreover, final heave was obtained within the first 60 minutes of inundation. When lime content was 2% and 4%, heave (mm) was respectively 2.8 mm and 0.6 mm, indicating a percent reduction of 50% and 95% (see Figure 2). Heave increased at 6% lime content. Swell potential decreased with increasing lime content (see Table 2). The reduction in swell potential (%) was significant at a lime content of 4%. However, swell potential increased when the lime content was increased to 6%. Swelling pressure also decreased with increasing lime content. At 4% lime, swelling pressure was reduced by 52% (see Table 2).

Swelling pressure could not be measured at a lime content of 6% as the sample could not be compressed even at higher compressive loads. This can be attributed to the development of cementitious products.

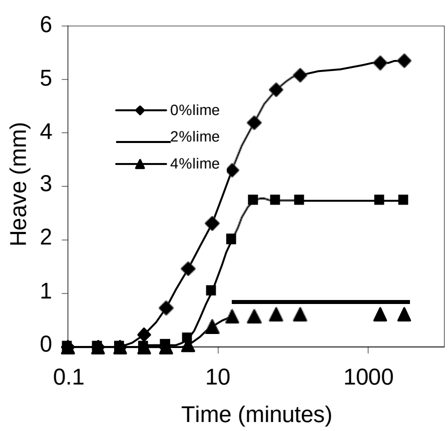

Fig. 2: Effect of Lime on Rate of Heave

Effect of Lime on e-log *p* Curves

Figure 3 shows the e-log *p* curves of clay-lime blends for varying lime content. Equilibrium void ratio at the end of swelling decreased, indicating that swelling decreased with increasing lime content. Swelling pressure also decreased with increasing lime content. Compression index (C_c) increased up to certain lime content and thereafter decreased with increasing lime content. The values of C_c were 0.5, 0.64 and 0.16 for the lime contents of 0%, 2% and 4% (see Table 2). At low lime contents, change in void ratio under the applied stress would be large because of flocculation and consequent increase in void space of the sample. Hence, C_c increased up to 2% lime content, and as lime content increased from 2% to 4%, there was a significant reduction in C_c. Coefficient of consolidation (c_v) also increased with increasing lime content up to 2% and decreased when the lime content was increased to 4% (see Table 2). The data shown pertain to a compressive stress within 160 kPa to 320 kPa. At 2% lime content flocculation would result in increased permeability of the blend. Compressibility also increased as indicated by increased compression index. The combined effect of increase in permeability and compressibility resulted in an increased C_v. At higher lime contents cementitious products would reduce compression index and permeability, decreasing C_v.

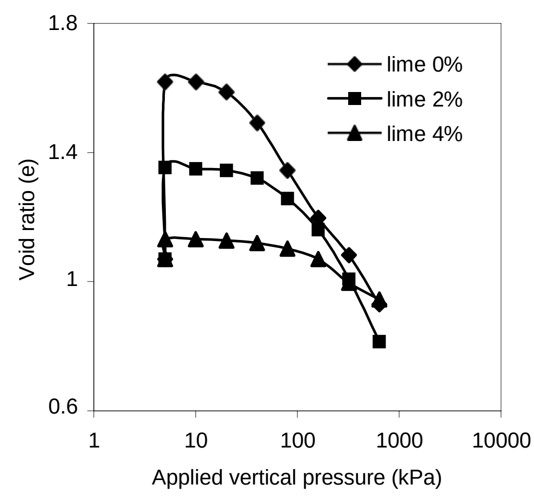

Fig. 3: Effect of Lime on e-log *p* Curves

Secondary consolidation of lime-clay blends was studied under a compressive stress of 640 kPa. Casagrande's log time-dial reading curves were plotted for the samples. Figure 4 shows the secondary consolidation behaviour of unblended expansive clay (0% lime) and clay blended with 4% lime. Secondary compression is the compression beyond the point of intersection of the two linear portions of the curve. Secondary compression of untreated expansive clay was more than that of the clay blended with 4% lime. Table 2 shows the values of secondary compression index, C_α, which is the ratio of change in void ratio Δe between times t_1 and t_2 and the logarithm of time interval $\Delta\log(t_1 - t_2)$ in Figure 4. C_α of unblended expansive clay was more (indicating higher secondary compression) than that of clay blended with 4% lime.

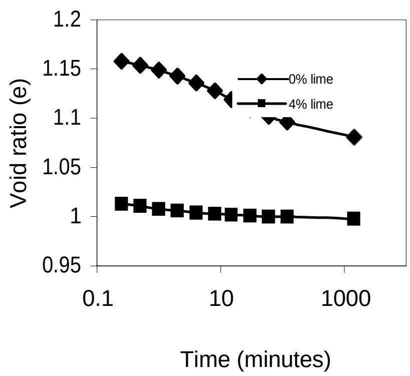

Fig. 4: Effect of Lime on Secondary Consolidation

Effect of Lime on Compaction Characteristics

Figure 5 shows compaction curves for varying lime contents (0%, 4% and 6%). Maximum Dry Density (MDD) increased and Optimum Moisture Content (OMC) decreased with increasing lime content. Flocculation results in particles rolling over themselves more easily during compaction. Therefore, clay-lime blends attain to higher densities than unblended clay. MDD was 13.6 kN/m^3, 13.8 kN/m^3 and 13.7 kN/m^3 and OMC was 34%, 29% and 28% for respective lime contents of 0%, 4% and 6%.

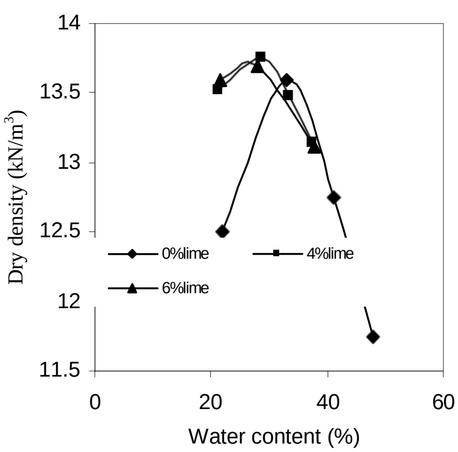

Fig. 5: Effect of Lime on Compaction Characteristics

Effect of Lime on Strength Behaviour

Figure 6 shows the stress-strain behavior of clay-lime blends. Failure stress and failure strain increased up to a lime content of 4% and thereafter decreased. Failure strain of unblended clay was 8%, which increased to 11.5% when lime content was increased to 4%. Failure strain decreased to 6% at a lime content of 6%. Table 2 summarizes the test results.

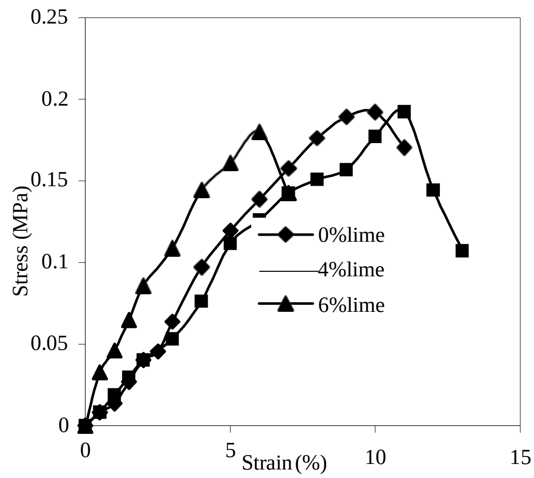

Fig. 6: Effect of Lime on Stress-Strain Behaviour

4. CONCLUSION

A lime content of 4% resulted in the maximum reduction in plasticity index. FSI decreased from 250% to 125% when the lime content was increased from 0% to 4%. Heave of 2.8 mm and 0.6 mm was observed at lime contents of 2% and 4%. Swelling pressure reduced by 52% at a lime content of 4%. Compression index was 0.5, 0.64 and 0.16 for the respective lime contents of 0%, 2% and 4%. While C_v increased, secondary compression decreased at a lime content of 4%.

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