STABILISATION OF SOIL WITH CHEMICAL ADDITIVES

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1.0 Introduction: Geotechnical properties of problematic soils such as soft fine-grained and expansive soils are improved by various methods. The problematic soil is removed and replaced by a good quality material or treated using mechanical and/or chemical stabilization. Different methods can be used to improve and treat the geotechnical properties of the problematic soils (such as strength and the stiffness) by treating it *in situ*. These methods include densifying treatments (such as compaction or preloading), pore water pressure reduction techniques (such as dewatering or electro-osmosis), the bonding of soil particles (by ground freezing, grouting, and chemical stabilization), and use of reinforcing elements (such as geotextiles and stone columns) (William Powrie, 1997). The chemical stabilization of the problematic soils (soft fine-grained and expansive soils) is very important for many of the geotechnical engineering applications such as pavement structures, roadways, building foundations, channel and reservoir linings, irrigation systems, water lines, and sewer lines to avoid the damage due to the settlement of the soft soil or to the swelling action (heave) of the expansive soils.

1.1 Review of Literature

1.1.1 Lime stabilization

Several investigations were done to evaluate the soil stabilization process using lime [either CaO or Ca(OH)2] (Parkash & Sridhran 1989, Wild et al. 1993, Bell 1996, Rajasekan et al. 1997, Rajasekan & Rao 1998, Burkart et al. 1999, Qubain et al 2000, Weber 2001, Yusuf et al. 2001, Ismail 2004, and Ampera & Aydogmus 2005). Qubain et al. (2000) incorporated the benefits of sub-grade lime stabilization, for the first time, into the design of a major interstate highway pavement in Pennsylvania. The project comprised widening and complete reconstruction of 21 Km of the Pennsylvania turnpike in somerset-county. Field explorations indicated that the sub-grade is fairly homogeneous and consists primarily of medium to stiff clayey soils. To safeguard against potential softening due to rain, lime modification has been traditionally utilized as a construction expedience for highway project with clayey sub-grade. Lime improves the strength of clay by three mechanisms: hydration, flocculation, and cementation. The first and second mechanisms occur almost immediately upon introducing the lime, while the third is a prolonged effect. Qubain et al. (2000) investigated the first and second mechanisms. Laboratory tests were performed to accurately capture the immediate benefits of lime stabilization for design. Both treated and natural clayey samples were subjected to resilient modulus and California bearing ratio testing. To prevent cementation, the lime-treated specimens were not allowed to cure. Nevertheless, they showed significant increase in strength, which, when incorporated into design, reduced the pavement thickness and resulted in substantial savings. Witt (2002) mentioned (Geotechnik Seminar Weimar 2002) that Weber (2001) investigated the effect of both curing (storage) and degree of compaction on the loss loam stabilized using different additives. He obtained the best results under condition of moisture atmosphere storage. At the water storage condition, the tempering of the stabilized specimens delayed due to the changing of pH-value in the pores water. The reactivity of lime stabilized specimens was continuing under this water storage condition. He noticed that the variation of compaction degree of the stabilized specimens affected on

the behavior of the stabilized specimens and the compaction (at the highest densities) led to brittle failure behavior.

Ismail (2004) studied materials and soils derived from the Feuerletten (Keuper) and Amaltheenton (Jura) formations along the new Nuernberg-Ingolstadt railway line (Germany). His work included petrological, mineralogical studies and scanning electron microscop-analysis. Ismail (2004) treated and stabilized these materials related to road construction using lime (10%), cement (10%), and lime/cement (2.5%/7.5%). He determined consistency limits, compaction properties, and shear- and uniaxial-strength. Ismail (2004) concluded that by increasing the optimum moisture content (%) of the treated-materials (soils mixtures), the maximum dry density (g/cm3) decreased. The cohesion and the friction angle of the improved materials increased for all the treated mixtures. In case of the lime-treated materials, the cohesion decreased by curing time. For Feuerletten materials, uniaxial strength increased strongly using lime and cement together. For Amaltheenton, uniaxial strength increased strongly with cement alone. He also noticed that the loss of weight during freezing and thawing test was low and depended on the material type. Ampera & Aydogmust (2005) treated Chemnitz clayey soil (A-7-6 Group) [according to American Association of State Highway and Transportation Officials (AASHTO)] using lime (2, 4, and 6%) and cement (3, 6, and 9%). They conducted compaction-, unconfined compressive strength-, and direct shear- tests on untreated and treated specimens. They concluded that the strength of cement-treated soil was generally greater than the strength of lime-treated-soil. They also reported that lime-stabilization is (in general, more tolerant of construction delay than cement-stabilization) more suitable for the clayey soils. The relationships determined from direct shear tests were similar to those determined from unconfined compressive strength tests. Thus, the results of shear strength tests showed a similar trend to that of the unconfined compressive strength tests.

1.1.2 Materials and Methods

Lime can be used either to modify some of the physical properties and thereby improve the quality of soil or to transform the soil into a stabilized mass, which increases its strength and durability. The amount of lime additive will depend upon either the soil to be modified or stabilized. Generally, lime is suitable for clay soils with PI ≥ 20% and > 35% passing the No.200 sieve (0.074 mm). Lime stabilization is applied in road construction to improve sub-base and sub-grades, for railroads and airports construction, for embankments, for soil exchange in unstable slopes, for backfill, for bridge abutments and retaining walls, for canal linings, for improvement of soil beneath foundation slabs, and for lime piles (Anon, 1985 & 1990). Lime stabilization includes the use of burned lime products, quicklime and hydrated lime (oxides and hydroxides, respectively), or lime by-products (codel) (TTN, 1998).

In the present study, hydrated lime (according to DIN 1060) was used. The chemical

composition of the hydrated lime illustrated in table 2.5. *Hydrated lime* is calcium hydroxide, Ca (OH)2. It is produced by reacting quicklime (CaO) with sufficient water to form a white powder. This process is referred to as *slaking*.

High calcium quicklime + water   Hydrated lime + Heat

CaO   + H2O   Ca(OH)2 + Heat
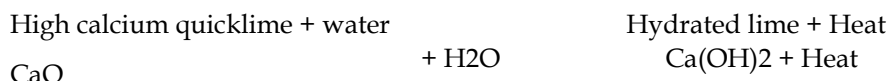
Hydrated lime is used in most of the lime stabilization applications. Quicklime represents approximately 10% of the lime used in lime stabilization process. Other forms of lime sometimes used in lime stabilization applications are dehydrated dolomitic lime, monohydrated dolomitic lime, and dolomitic quicklime (TTN, 1998). Calcium oxide (quick lime) may be more effective in some cases, however the quick lime will corrosively attack equipment and may cause severe skin burns to personnel (Muntohar & Hantoro, 2000). The Addition of the hydrated lime Ca(OH)2, *in situ* or in laboratory, is either as slurry formed by the slaking of quicklime, or as dry form (dry powder).

Table (2.1) General recipe of lime- and cement-stabilization modified after the German standard (Leaflet for soil stabilization and soil improvement; soil stabilization using binding agents, 1997). X = suitable (X) = conditional suitable - = unsuitable

Soil type			Binding agent	
			Quicklime and hydrated lime after DIN (10 60)	Cement after DIN (11 64)
Coarse grain size soil			-	X
Mixed grain size soil			(X)	X
Fine grain size soil	silt	Low plasticity	X	X
		Medium P.	X	(X)
		High P.	X	-
	clay	Low plasticity	X	(X)
		Medium P.	X	-
		High P.	X	-
Organic soil	silt		X	(X)
	clay		X	-

2.0 Fly ash

2.1 Background

Fly ash is a by-product (waste material) of burring coal at electric power plants. It is a fine residue composed of unburned particles that solidifies while suspended in exhaust gases. Fly ash is carried off in stack gases from a boiler unit, and is collected by mechanical methods or electrostatic precipitators. Fly ash is composed of fine spherical silt size particles in the range of 0.074 to 0.005 mm (Ferguson, 1993). Fly ash collected using electrostatic precipitators usually has finer particles than fly ash collected using mechanical precipitators. Fly ash is one of the most useful and versatile industrial by-products (Collins & Ciesielski, 1992).

When geotechnical Engineers are faced with problematic soils (such as clayey or expansive soils), the engineering properties of those soils may need to be improved to make them suitable for construction. Waste materials such as fly ash or pozzolanic materials [pozolanic materials "pozzolans" are a source of silica and alumina with high surface area (Choquette el al., 1987)] have been used for soil improvement. Recent investigations reported that fly ash is a potential material to be utilized for soil improvement (Muntohar & Hantoro, 2000).

Fly ash is generated in huge quantities (more than 65 million metric tons per year in the USA) as a by-product of burning coal at electric power plants (Ferguson, 1993). The potential for using fly ash in soil stabilization has increased significantly in many countries (for example in Wisconsin, USA) due to the increased availability and the introduction of new environmental regulation (NR 538, Wisconsin Administrative Code) that encourage the use of fly ash in geotechnical applications since it is environmentally safe (Şenol et al., 2002).

2.2 Classification and chemical reactions of fly ash:

Fly ash is classified into two classes: F and C. Class F fly ash (non-self-cementing fly ash) is produced from burning anthracite and bituminous coals and contains small amount of lime (CaO) to produce cementitious products. An activator such as Portland cement or lime must be added. This fly ash (pozzolans) has siliceous and aluminous material, which itself possesses little or no cementitious value but it reacts chemically (in the presence of moisture) with lime at ordinary temperature to form cementitious compounds (Chu et al., 1993).

Class C fly ash (self-cementing fly ash) is produced from lignite and sub-bituminous coals (low-sulfur subbituminous coals), and usually contains significant amount of lime (Cockrell & Leonard, 1970). This type (class C) is self-cementing because it contains a high percent of calcium oxide (CaO) ranging from 20 to 30%.

2.2.1 Source and composition: The fly ash used in the present study is from a local electric power plant at Lippendorf, South of Leipzig city, Saxony, Germany. Physical and chemical properties of the Lippendorf fly ash are summarized in Table 2.2 along with physical properties of class C and class F fly ashes. The fly ash has a powdery texture. It is light grey in colour, which indicates high calcium oxide content (Meyers et al., 1976). The types of coal of this fly ash are lignite and subbituminous. Table (2.2) Physical properties, chemical composition and classification of fly ashes

Fly ash	Classification after ASTM Standard C 618	Gs g/cm³	Specific surface area m²/g	Percent Finer (%)	Moisture content (%)	LOI (%)	Lime CaO (%)	Other oxides (SiO₂ + Al₂O₃ + Fe₂O₃) (%)	Sulfur Trioxide (%)
Type 1	Class C**	-	-	-	3	6	24.3*	50	5
Type 2	Class F**	-	-	-	3	6	8.7*	70	5
Lippendorf	Off-spec.	2.8	1.1	81.5	0.11	0.22	38.3	46.00	9.1

The chemical composition of fly ash is one of the most important indicators of material quality for various applications. Detailed chemical composition of Lippendorf fly ash is summarized in Table 2.3, along with typical chemical composition of class C and F fly ashes. Table (2.3) Chemical composition of Lippendorf fly ash and typical chemical composition of both class C and F fly ashes.

Chemical elements	Lippendorf fly ash (%)	Typical class C (%)	Typical class F (%)
SiO_2	32.20	39.9	54.9
Al_2O_3	11.20	16.7	25.8
Fe_2O_3	2.60	5.8	6.9
CaO	38.3	24.3	8.7
MgO	4.10	4.6	1.8
SO_3	9.10	3.3	0.6
LOI	0.22	6	6

2.2.2 Index- and compaction-properties

The specific gravity and the specific surface area of Lippendorf fly ash is showed in Table 2.2. The specific gravity of Lippendorf fly ash (2.8 g/cm3) is high relative to the typical values of the fly ashes (specific gravity typically range from 2.11 to 2.71) (Chu & Kao, 1993). In general, as mentioned before, Fly ash collected using electrostatic precipitators, like Lippendorf fly ash usually has finer particles than fly ash collected using mechanical precipitators. Figure 2.1 illustrates the particle size distribution of Lippendorf fly ash (according to DIN 18 123). Grain size analysis of fly ash was carried out through combination of dry sieving- and sedimentation-analysis. Sedimentation-analysis was conducted by granulometer CILAS 920 using the fine fraction < 400 μm (resulted from the dry sieving) in suspension in an appropriate (Isopropanol) liquid.

Fig. (2.1) Particle size distribution of Lippendorf fly ash

Fig. (2.2) Compaction curve of Lippendorf fly ash (compacted immediately after adding water)
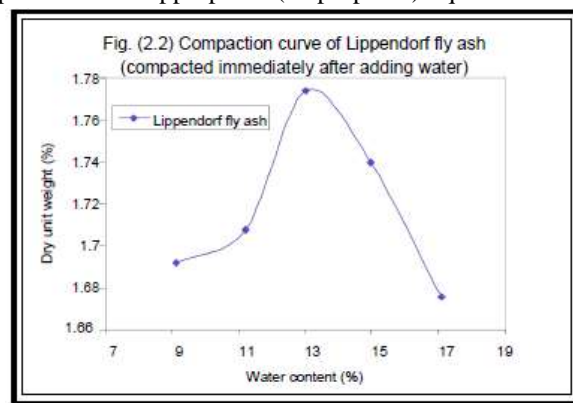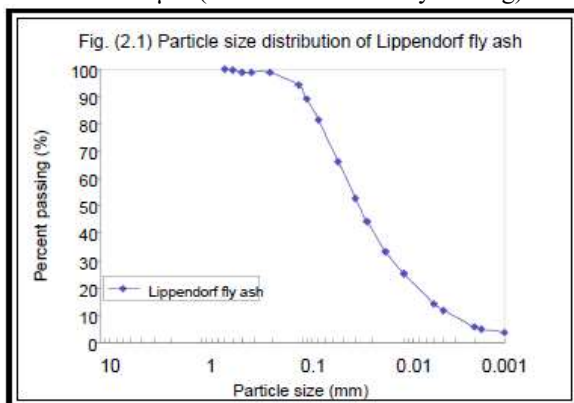
3.0 Chemical and Mineralogical Analysis

3.1 Chemical analysis: X-ray fluorescence spectrometry (XRF) X-ray fluorescence analysis of both natural soils and chemical additives was investigated using XRF-technique (type of the instrument is SRS 3000 Siemens). Chemical compositions of the three different studied natural soils, fly ash, and the hydrated lime are summarized in Table 2.5.

3.2 Mineralogical Analysis: X-ray powder diffraction (XRD) X-ray powder diffraction technique with Cu Kα radiation (type of instrument is Siemens D 5000 diffractometer with a generator operating at 40KV, 30mA and with Ni-Filter) was used to determine the mineralogical composition of the natural soils and the chemical additives. Soil mineralogy provides the basis for understanding the basic mechanisms of chemical stabilization. It also helps to identify types of clay minerals in the studied soils in order to determine the ability of the soils to expand. The presence of some clay minerals, like Table (2.5) Chemical composition of both natural soils and chemical additives (Hydrated lime and Fly ash) high-activity smecite clays as montmorillonite in the soil is a good indication of the swell potential. These clays cause problems of excessive expansive characteristics, which

lead to much damage to the structures built in and on them (Nicholson, et al., 1994). The mineral composition of natural tertiary clay, organic silt, and weathered soil is shown in Appendixes (1b, 2, and 3, respectively) and in Table 2.6.

Chemical Elements	Tertiary Clay (%)	Weathered soil (%)	Organic Silt (%)	Hydrated lime (%)	Fly ash (%)
SiO2	54.9	52.1	41.70	0.503	32.20
Al2O3	18.50	21.3	11.80	0.279	11.20
Fe2O3	5.73	6.26	4.24	0.225	2.60
Σ =	79.13	79.66	57.74	1.007	46.00
SiO2/Al2O3	2.97	2.45	3.53	1.802	2.88
CaO	2.41	1.34	18.70	73.4	38.3
MgO	1.86	1.65	1.79	0.556	4.10
Na2O	0.140	0.179	0.36	0.0321	0.160
K2O	2.98	3.66	2.60	0.0737	0.267
MnO	0.0261	0.0132	0.105	0.0359	0.113
TiO2	0.974	1.78	0.711	0.0185	0.99
P2O5	0.116	0.0919	0.444	0.0288	0.0758
SO3	4.23	1.92	4.64	0.191	9.10
LOI	6	6.8	12.8	24.6	0.22

high-activity smecite clays as montmorillonite in the soil is a good indication of the swell potential. These clays cause problems of excessive expansive characteristics, which lead to much damage to the structures built in and on them (Nicholson, et al., 1994). The mineral composition of natural tertiary clay, organic silt, and weathered soil is shown in Appendixes (1b, 2, and 3, respectively) and in Table 2.6.

4.0 Test procedures

4.1 Unconfined compressive strength test

Unconfined compressive strength tests were conducted according to DIN 18 136. A photograph of a soil specimen subjected to unconfined compression is shown in Figure 2.10. Unconfined compressive strength for natural soils, for untreated compacted, and for treated stabilized specimens is determined by using computerized triaxial instrument without application of the cell pressure ($\sigma 3$ = zero). The dimensions of the tested specimens (for natural specimens) are 120 mm height and 95 mm diameter and the dimensions of untreated compacted and treated stabilized specimens are 120 mm height and 100 mm diameter.

The maximal vertical strain according to DIN 18 136 is equal to 20% from the maximal height of the tested specimen, so that, the maximal vertical strain = 20 / 100 * 120 mm = 24 mm.
Unconfined compressive strength (qu–value) of the tested specimens is either at the failure of the specimen or at the maximal vertical strain (ε) equal to 20% of the original height of the soil specimen (DIN 18 136). The speed of deformation (strain rate), according to DIN 18 136, is at least equal to 1% of the maximal height of the tested sample = 1/100 * 120 mm = 1.2 mm/min. In the present study, the strain rate for both undisturbed natural soils, untreated compacted, and for treated stabilized specimens was equal to 0.2% of the maximal height of the tested specimens. The strain rate is 120 mm * 0.2 /100 = 0.24 mm/min, according to DIN 18 136 for the cemented and stabilized specimens.

Fig. (2.9) Temperature-humidity chamber.

Fig. (2.10) Computerized triaxial cell to measure the unconfined compressive strength (σ 3 = zero).

After compaction the specimens were extruded, sealed in polyethylene paper, and stored in ≥ 98% relative humidity at 40°C ± 2 for 7 days curing (for soil-lime mixtures) in computerized temperature-humidity chamber (Fig. 2.9). For soil-fly ash and soil-lime/fly ash mixtures, the specimens are stored in ≥ 98% relative humidity and at 25°C ± 2 for 7 days curing. After 7 days curing period, all treated stabilized specimens were tested in unconfined compression at strain rate of 0.24 mm/min. The general relationship between unconfined compressive strength and the quality of the sub-grade soils used in pavement applications (Das, 1994) is as follow:

Qu-values		Quality of sub-grade	
25 – 50	kpa	soft	sub-grade
50 – 100	kpa	Medium	sub-grade
100 – 200	kpa	stiff	sub-grade
200 – 380	kpa	very stiff	sub-grade
› 380	kpa	hard	sub-grade

4.2 CBR test

CBR test was conducted according to TPBF-StB part B 7.1, 1988 for natural soils compacted with natural water content, for untreated compacted specimens with optimum water content, and for all treated stabilized specimens at the optimum water contents using standard proctor effort (Fig. 2.11) and computerized CBR-instrument (Fig. 2.12). The dimensions of the tested specimens are 125 mm height (H) and 150 mm diameter (Ø). The test was conducted with annular surcharge mass of 5 Kg. A natural soil specimen (tertiary clay) undergoing a CBR-test is shown in Figure 2.12.

CBR = P/Ps * 100 (%)
Where: P is plunger-load in N/mm² for tested soil.
Ps is plunger-load in N/mm² for standard soil (see Fig. 2.13).

Fig. (2.11) Standard proctor instruments.Fig. (2.12) Computerized CBR-instrument, CBR-test conducted on tertiary clay specimen.

For soil-lime mixtures, the specimens were left in the mold, sealed using polyethylene paper, and left to cure for 7 days at 40°C ± 2 and ≥ 98% relative humidity prior to testing. For soil-fly ash and soil-lime/fly ash mixtures, the specimens were left in the mold, sealed using polyethylene paper, and cured for 7 days (at 25°C ± 2 and ≥ 98% relative humidity) prior to testing.

The general relationship between CBR-values and the quality of the sub-grade soils used in pavement applications (Bowles, 1992) is as follow:

CBR - values		Quality of sub-grade	
0– 3	%	very poor	sub-grade
3– 7	%	poor to fair	sub-grade
7– 20	%	fair	sub-grade
20 – 50	%	good	sub-grade
› 50	%	excellent	sub-grade

The sub-grades having (0 – 7%) CBR–values are very poor and poor to fair. They are considered as unstable sub-grades and need to be stabilized, especially, in terms of pavement applications.

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