Material State	Elastic Properties (kPa)
Base	
In compression, E_{bc}	6.89×10^6
In tension, E_{bt}	1.38×10^6
Subgrade	
In compression, E_{sc}	6.89×10^4

Calculation of α is done with

$$\alpha_1 = 0.64,$$
$$\alpha_2 = 1.20 \text{ (frictionless interface)},$$
$$\alpha_3 = 1.60 \text{ (subgrade carries no tension), and}$$
$$\alpha_1 \cdot \alpha_2 \cdot \alpha_3 = 0.24 \times 1.20 \times 1.60 = 1.23.$$

Calculation of k is done, using a crack propagation criterion, with

$$k = 2.36 \text{ (interpolation from Table 3)},$$
$$\sigma_M(\alpha_1\alpha_2\alpha_3) = kT,$$
$$\sigma_M = kT/\alpha_1\alpha_2\alpha_3 = \text{allowable edge or interior tensile stress to be used in the chart shown in Figure 7, and}$$
$$\sigma_M = [2.36(0.5 \times 1980)]/1.23 = 1900 \text{ kPa } (276 \text{ lbf/in}^2)$$

and, using a crack initiation criterion, with

$$k = 0.73, \text{ and}$$
$$\sigma_M = [0.73(0.5 \times 1980)]/1.23 = 593 \text{ kPa } (86 \text{ lbf/in}^2).$$

Resulting thicknesses are summarized below (1 mm = 0.039 in):

Thickness	Stress Used for Design: Edge Stress (mm)	Interior Stress (mm)
Suggested method (crack propagation)	457	380
Suggested method (crack initiation)	1140	812
50 percent flexural strength criterion	710	558

In this table thicknesses obtained by using the procedure suggested by Mitchell, Dzwilewski, and Monismith (2) are also shown. Note that these values lie between those obtained by using the procedure reported here.

SUMMARY

In this paper a design procedure permitting incorporation of crack propagation in the thickness-selecting process for cement-stabilized bases has been briefly outlined. This procedure makes use of a fatigue criterion that is based on a theory of fracture originally proposed by Griffith (4).

The data presented indicate that crack initiation and propagation rate are determined by base and subgrade properties, interface friction, and load magnitude. The simplified design procedure presented here incorporates these variables.

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Abridgment

Clay Mineral Weathering Controls on Lime and Cement Stabilization of Southwestern Ontario Clay Borrow

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This paper describes the significant role of clay mineral weathering in the upper 3 m of a clay crust in the compaction and stabilization characteristics of borrow materials extracted from different levels of the crust. The main mineralogical control is an increase in smectite (swelling clay) content toward the surface that is caused by oxidation weathering of unstable chlorites in the original soil.

MATERIALS AND METHODS

The soil samples were taken from a 2.5-m trench dug

Figure 1. Compaction-moisture relationships.

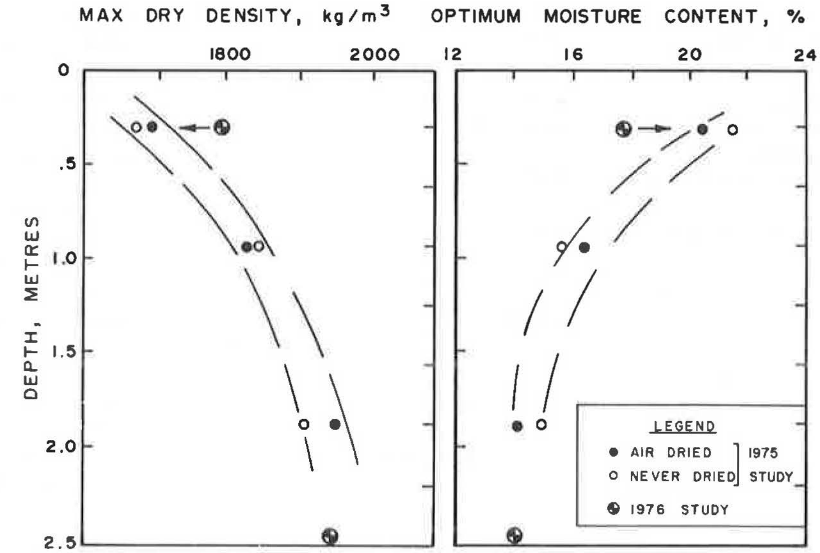
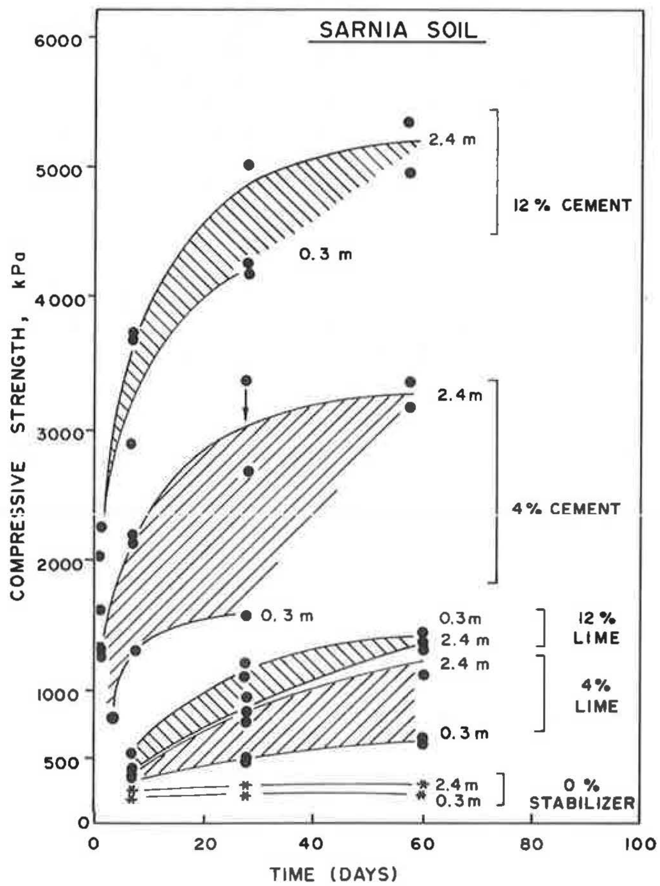


Figure 2. Unconfined compressive strengths of Sarnia soils stabilized with 0, 4, and 12 percent hydrated lime and cement at 0.3-m and 2.4-m depths.



into clay deposits of the St. Clair clay plain about 6.5 km southeast of Sarnia, Ontario. Source rocks for the sediments, which are believed to be Wisconsin age waterlain tills, were primarily Devonian carbonates and shales.

The soil studied was a very stiff, brown, desiccated, fissured, clayey silt containing scattered pebbles and sand throughout. With depth the soil became grey, less fissured, and softer.

The mineralogical compositions of the soil from 0.3- and 2.4-m depths and certain index tests are summarized in the table below.

Item	At 0.3-m Depth	At 2.4-m Depth
Chlorite, %	~6	~7
Illite, %	~19	~17
Smectite, %	~8	~2
Carbonate, %	41	~51
Quartz and feldspar, %	~26	~23
Liquid limit, %	52	25
Plastic limit, %	23	14
<2 μm, %	55	35
Activity, P.I./[(% < 2 μm) - 9]	0.63	0.42
pH	7.5	7.6
Swell potential, %	2→6	<1

The most significant aspect of the mineralogy is the increase in smectite from 2 percent at depth to 8 percent near surface. Extensive X-ray diffraction studies (1, 2, 3) have shown conclusively that the smectite is produced by oxidation weathering of original soil chlorite, a process that is most intense at surface. Carbonate leaching from the surface soils (41 percent at 0.3 m compared to 51 percent at 2.4 m) is significant. At other sites negligible amounts of carbonate may be found to depths of 30-40 cm (1).

An activity of 0.63 at 0.3-m depth compared to one of 0.42 at 2.4-m depth reflects the increased amount of smectite at surface. Similarly, swell potentials (4) as high as 6 percent on surface soils were less than 1 percent at depth. For an extensive discussion of the geology and mineralogy, the reader is referred to Quigley and Ogunbadejo (1).

All soil samples were air dried to water contents of 8-10 percent, hand crushed, sieved to < 4.75-mm mesh, and stored in sealed containers. Completely air drying the samples results in irrecoverable collapse of a large portion of the smectite in these soils (5) and was carefully avoided in this series of tests. Harvard miniature compaction was carried out by using a spring-loaded tamper that delivered a foot pressure of 1170 kPa at a rate of 20 tamps/layer in a five-layer sequence.

Specimens were prepared for strength and swell testing by static compaction between two pistons using the optimum moisture and resultant density values obtained by the previous kneading compaction. Amounts of water and stabilizer are expressed as weight percentages of the dry soil. All samples were wrapped in plastic, waxed, and cured at 21° C in a humid room for the time desired. Unconfined compression tests were run at a strain rate of 0.5 percent/min.

The cement used was a type I portland (C_3S = 54.6 percent, C_2S = 16.5 percent, C_3A = 7.8 percent, C_4AF = 9.2 percent), and the hydrated lime was a commercially available grade with a calcium to magnesium weight ratio of 2:1.

RESULTS AND DISCUSSION

The major results of the study are summarized in Figures 1 and 2. In Figure 1, the compaction-moisture characteristics versus depth are plotted for nonstabilized soils. It is very clear from the plots that the maximum dry density increases from about 1700 kg/m^3 at 0.3-m depth to about 1950 kg/m^3 at depth. The optimum moisture contents correspondingly decrease from about 19 or 20 percent at 0.3-m depth to 14 or 15 percent at depth. These changes are attributed directly to the 8 percent smectite at 0.3-m depth compared to 1 or 2 percent at 2.4-m depth and follow well-established trends identified in the literature (6). Check tests done by standard Proctor (not plotted) yielded densities almost identical to the Harvard miniature densities as long as the samples were never completely air dried.

A summary plot of the unconfined compressive strength of the compacted soils, with and without various percentages of cement and hydrated-lime stabilizers, is given in Figure 2. It is immediately obvious that cement is a more effective stabilizer than the hydrated lime and that the strength gain is also much more rapid for cement than for lime.

It is also obvious that, for both the lime and cement, the smectite-bearing surface soils are generally less effectively stabilized (weaker) than the deeper, smectite-free, less weathered soils. Tests at other stabilizer contents showed that the optimum content of lime was about 4 or 5 percent; little additional strength gain was achieved at higher levels, even though very long-term increases might be possible.

The nonstabilized samples yielded unconfined compressive strengths of about 250 kPa, and the smectite-rich soils were slightly weaker than the nonsmectitic soils. Four percent lime produced a strength of 600 kPa (only a 2.5-times increase) for the 0.3-m sample compared to a fivefold increase for the 2.4-m sample. At 12 percent lime, the curves actually reversed themselves. The 0.3-m smectite-rich soil was stronger at 60 d than the 2.4-m smectite-deficient soil.

The interaction of lime and cement with the various mineral constituents of soils has been described by many authors (6, 7, 8, 9, 10, 11, 12, 13), so the results of the stabilization study are essentially predictable once the soil mineralogy is known and understood. The main contribution of this paper is to show how weathering may influence the clay mineralogy within a shallow borrow pit and how it may markedly affect compaction and stabilization. Special tests on the <2-μm soil fraction showed the smectite clay minerals from 0.3 m to be surprisingly amenable to lime stabilization with 4 percent lime and 4 percent cement and to yield unconfined strengths of 1200 and 1400 kPa respectively. Swell tests run on a variety of stabilized soils showed that as little as 2 percent of both lime and cement completely eliminated swelling of the smectitic soils from 0.3 m, provided a day or two of curing was allowed.

The details of this special study on the fines are beyond the scope of this abridgment, but we believe the work to be highly significant with respect to lime stabilization, especially if proper trace additives can be found and further research is planned on this subject.

CONCLUSIONS

The following conclusions may be drawn from the research described in this paper.

1. Oxidation weathering has altered inactive soil chlorites to active soil smectite in the near-surface soils of southwestern Ontario and has increased their activity from 0.4 to greater than 1.
2. The surface soils, which contain from 8 to 15 percent smectite, compact to a maximum dry density of up to 300 kg/m^3 less than the deeper soils that contain only 2 or 3 percent smectite.
3. The surface soils are more difficult to stabilize with hydrated lime and cement than the deeper soils because of their smectite contents. The optimum lime content is about 5 percent.
4. Cement is a more efficient stabilizer than lime for the whole soil and gives unconfined compression strengths two to three times those of lime at the 5 percent stabilizer level.
5. The soil fines (< 2 μm), however, are just as effectively stabilized by 4 percent lime as by 4 percent cement and yield nearly identical unconfined compression strengths. Because the clay fraction is so amenable to lime stabilization, further work is recommended, especially on the use of trace additives.

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Concerning Pressure-Grouted Soil Anchors

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Until recently, anchored sheet piling walls were almost exclusively provided with long horizontal anchors that had anchor walls, slabs, deadmen, or pile clusters at their ends. These were anchored in the passive zone of the soil wedge behind the classical rupture wedge of soil. A relatively new method in foundation engineering for back-tying of excavation walls is the pressure-grouted soil anchorage. This new kind of soil anchorage system has now become an important element in current foundation-engineering practice. It has gained increased significance and popularity, and its use continues to increase. This paper describes some of the basic principles involved in the tie-back anchorage or wall-anchor-soil system, reviews a basic type of soil anchor, and presents a method for stability analysis of a pressure-grouted soil anchorage system.

Anchored sheet piling walls have been constructed, almost without exception, with long horizontal anchors that have anchor walls, slabs, deadmen, or pile clusters at their ends. These were anchored in the classical Coulomb's passive zone of the soil wedge behind the classical rupture wedge of soil.

The pressure-grouted soil anchorage—a relatively new method in foundation engineering for back-tying of excavation walls—has become an important element in current foundation-engineering practice. This paper describes some of the basic principles involved in the tie-back anchorage system, reviews a basic type of soil anchor, and presents a method for stability analysis of a pressure-grouted soil anchorage system.

PRESSURE-GROUTED SOIL ANCHORS

Definition

The pressure-grouted soil anchor or tie-back is a special and important substructure anchoring element. It may be a steel rod, or a steel cable, or a multistrand of high-tension steel wires. Such an anchor is designed to be installed either horizontally or at an inclination to the horizontal. Its purpose is to anchor, in one or several tiers, various temporary or permanent earth-retaining and foundation structures to resist lateral, vertical, inclined, and hydrostatic uplift forces.

A pressure-grouted soil anchor works in tension. The integrally performing wall, anchor, and soil form the so-called wall-anchor-soil system, frequently referred to as the tie-back system.

Uses of Soil Anchors

Pressure-grouted soil anchors are used as both temporary and permanent support systems for sheeted excavations in sandy as well as clayey soils (1, 2, 3, 4, 5, 6, 7, 8, 9, 10). Tie-backs also stabilize river and canal banks, shore-walls, and earth slopes. In addition, they are used at construction sites where there is a lack of space between the building and the property lines. They are also used to transmit to the ground tensile forces from guy wires of suspension bridges and tentlike (or stressed-cable) structures whose roofs are supported by a stressed-cable network and to transmit to the ground hydrostatic uplift forces acting on the bases of submerged foundations.

Types of Soil Anchors

The multitude of pressure-grouted soil anchors on the market and in the state of development prohibit a complete listing and description of them here. Almost every foundation-engineering firm in the business of soil anchorage has its own trademarked or patented soil anchor de-