Session 12

Soil Improvement — General Report

Amélioration des Sols

J.K. MITCHELL Professor and Chairman of Civil Engineering, University of California, Berkeley, CA, USA
R.K. KATTI Professor, Indian Institute of Technology, Bombay, India

INTRODUCTION

Forty-six papers were received by the reporters for review in Session 12 on Soil Improvement. They have been separated into the six topic categories that define the scope of this Session; namely,

1. Deep compaction
2. Soil improvement by precompression
3. Injection and grouting
4. Admixture stabilization
5. Thermal stabilization
6. Soil reinforcement

This general report is confined almost exclusively to review and comment on the submitted papers. Our state-of-the-art report contains relevant background and reference information. Reference is also made to a few papers included in other sessions of this International Conference that should be of interest to engineers concerned with soil improvement methods.

DEEP COMPACTION OF COHESIONLESS SOILS

Introduction

Nine of the papers submitted to Session 12 are concerned with deep compaction of cohesionless soils, and at least one relates to and advances each of the methods reviewed in the state-of-the-art report. In addition, one paper is concerned with surface compaction using vibratory rollers, and one paper deals with the deep compaction of loess, a slightly cohesive soil type.

This latter paper, by Minkov, Evstatiev, Donchev, and Stefanov, is valuable, because it indicates how more than one method may be developed or adapted for stabilization of collapsable loess soils. Bulgarian experience over the past 20 years is summarized. The need to support structures on loess deposits of 25 to 30 m thickness is common. Heavy tamping for support of footings involves the use of concrete weights up to 7 tons and drop heights up to 10 m. The depth of loess compaction is up to 5-6 m for the highest impact energies. Soil-cement cushions and short concrete pyramid piles are also used for shallow foundation support. For the thickest loess layers deep compaction by wetting or by wetting in combination with deep blasting are reported by Minkov et al. as the only suitable method.

Blasting

Two papers present test data that enhance our understanding of in-situ deep densification by blasting. Klohn, Garga and Shukin describe a field test program for densification of loose silty sand tailings. Charges of 5 kg of TNT were placed at a depth of 4.9 m in a layer that was 6.7 m deep overlain by a 0.9 m thick access pad. The upper half of the layer contained from 40 percent to 80 percent fines, and the lower half had about 20 percent fines. In the first test three repetitions of a single charge at the same point were made. In the second test three coverages of a 41 m square grid were made. Charge spacings were at 20 m for the first two coverages and at 14 m for the third. Piezometers and settlement plates were installed to monitor the results. The first piezometer readings could be obtained 2 minutes after charge detonation. Typical pore pressure vs. time records are shown in their Fig. 4.

Several of the findings reported by Klohn et al. are noteworthy;

1. Water seepage broke out at the surface within 3 minutes after the blast and continued for an hour.
2. With the exception of two piezometers located outside the main blast area, the 12 others all showed greater excess pore pressure ratios ($\Delta u/p'$) for each successive blast. These two findings confirm that liquefaction occurred and suggest that once the initial structure had been broken down, liquefaction during subsequent blasts became easier.
3. It took from 8 to 170 hrs for excess pore pressures to dissipate, with a mean value of 24 hours. That these rather long times were required is not surprising in view of the high fines content of the tailings.
4. The average blast-induced settlement was 0.41 m. Settlement profiles are in Fig. 5 of the paper.
5. Based on the assumption of evenly distributed settlements with depth, the relative density of the sandy silt was estimated to increase from about 30 percent to about 60 percent.

The average increase in relative density deduced from settlement (30 percentage points) was significantly greater than that deduced from SPT data (15 percentage points). Such a finding is consistent with the time effects discussed in our state-of-the-art report, assuming that the SPT tests were done very soon after blasting.

Pilot, Colas des Francs, Puntous, and Queyroi present the results of a test program done to optimize the blasting parameters for compaction of a 10 m thick poorly graded sand hydraulic fill. Variables studied included quantity of charges, which varied from 33 g/m^3 to 58 g/m^3, charge spacing (4 m x 4 m and 5 m x 5 m grids were used), and charge depth. Charges were concentrated at the bottom of the layer and divided equally between the bottom and mid-depth. Each test section was 14 m x 28 m in plan. Important findings were

1. Compaction by explosives is ineffective near the ground surface, a finding consistent with previous studies.
2. If too large charges are used, the disruption of the structure may be so great as to yield a final condition no better than the initial one. Best results were obtained using 42 g/m^3 and 5 m charge spacings.
3. Division of the total charge among two levels gave improved compaction of the upper part of the layer without adversely affecting the lower part.

Pilot et al. also compacted a section using vibroflotation and obtained much better improvement than could be obtained by blasting. The cost, however, was about three times as great. Finally, in agreement with Klohn et al., Pilot et al. emphasize the need for test programs in connection with projects where blasting is to be used, in order to determine the necessary design parameters.

Askarov, Bobylev, and Margotiev discuss both a blasting method and a compaction pile method that are used for improvement of loess deposits. Both are reported to be economical. A combination presoaking and blasting method somewhat different than that described in our state-of-the-art report has been used that involves underwater explosions in water-filled trenches. Charges totalling 1 kg/m^2 are placed 0.5 m above the trench bottom and 1.5 to 2 m below the water surface. An area in the range of 300 to 700 m^2 is treated at one time. Compaction leading to final densities of 1700 to 1800 kg/m^3 extends to a depth of 7 to 8 m.

Although not included among the papers for Session 12, the contribution by Charlie, Mansouri, and Ries to Session 1 should contain results useful to soil densification by blasting. A method is presented to predict blast-induced porewater pressure changes utilizing Biots' theory of propagation of elastic waves in a fluid-saturated porous medium. Computed and measured porewater pressure changes are compared.

Vibrocompaction and Compaction Piles

The storm surge barrier being constructed for closure of the Oosterschelde estuary in the Netherlands requires that 66 piers, each of foundation area 25 x 50 m, be supported on a sand foundation. Densification of this sand is necessary because of its initially loose condition. Davis, Nelissen, and Pladet describe the compaction vessel, "Mytilus," which has been assembled to carry out this compaction. Four coupled vibrators, which can penetrate the bottom to a depth of 15 m, spaced at 6.5 m center-to-center are supported. The vibrating tubes, which are 42.5 or 47.5 m long, have a tip diameter of 2.1 m inclusive of 12 radial fins. A vibrator on top provides a 1200 kN dynamic force at 25 Hz, producing a 4 mm amplitude at the bottom. Water and air jets are used to aid penetration. The dead weight of a complete vibrating tube is 400 kN. Compaction is in 1 m lifts as the probes are withdrawn. Cone penetration resistance data are presented by Davis et al. to illustrate the effects on compaction of (1) extraction velocity, (2) vibration time at each level, (3) surcharge, and (4) horizontal distance between sets of compaction points. The influence of each of these on the amount of improvement obtained is as would be anticipated. An equivalent relative density of 70-75 percent can be achieved, provided the silt content of the sand is less than about 12 percent.

A case history of the use of compaction piles to reduce the liquefaction potential of the upper 6 m of a loose fine sand is presented by Moh, Ou, Woo, and Yu. Compaction sand piles of 450 mm diameter (700 mm after installation), 7.5 m long, and spaced at 1.8 m in a triangular pattern were chosen after a test program that included also spacings of 1.6 and 2.0 m. Compaction piles were selected rather than vibroflotation, because the gradation of the sand was at the boundary for effective densification by vibroflotation. Densification was evaluated using the SPT. Relative densities greater than 65 percent (Gibbs and Holtz criteria) were obtained in all cases, and 92 percent of the results indicated a relative density greater than 75 percent. Piezometers indicated, somewhat surprisingly, that it took three to four weeks to dissipate the pore pressures generated during installation of the piles. Results obtained are illustrated graphically in the paper.

The test program described by Faraco provided a useful comparison of deep densification methods. At a site in the Canary Islands volcanic ash ("picón") was dumped hydraulically to provide support for dockyard facilities with imposed pressures of 400 kPa. A coarse fraction (particles larger than 75 mm) made up of cemented ash, accounted for 25 percent of the material. Of the remainder, 50 percent was gravel and sand, and less than 4 percent was fines. The methods of compaction tested were vibrocompaction, Terraprobe, compaction piles, and blasting. The tests were done in an area where the fill thickness was 8-10 m. Blasting gave poor results and was not analyzed in the paper.

A relative density of 80 percent (Gibbs and Holtz criteria) after compaction was desired as a minimum. The following maximum spacings were determined to satisfy this criterion by the different methods. Also listed are the times required per probe.

Method	Max. spacing (m)	Time/probe for treatment to 8-10 m depth (min.)
Vibroflotation (Held-Francki)	2.5	45-60
Vibroflotation (Keller)	3.0	45-60
Terraprobe	2.0	15-30
Compaction Piles	1.5	90

Terraprobe and vibroflotation were the final choices for use on this project. Faraco notes also that all of the methods resulted in soil loosening in the upper 2 m.

That deep densification methods are ineffective in the upper two or three meters has been the case generally. Surface compaction, often using vibratory rollers, is usually used following deep compaction. Schwartz, Yates, and Tromp report on a test program to determine the best means for densifying a collapsing sand using a vibratory roller. The top 5 m of the aeolian Kalahari sand is collapsing. The loose structure is metastable because of iron oxide and kaolinite clay bridges between particles. A 15,500 kg dead load vibratory roller (ABG 185) was used.

Schwartz et al. compared test strips compacted after no prior surface treatment, after removal of vegetation, after ripping to a depth of 800 mm, and after ripping and wetting. In no case did rolling give a density increase at depths greater than 1.1 m. Densification was obtained only in those cases where the surface was ripped prior to compaction.

Heavy Tamping

Three contributions to Session 12 are on ground improvement by heavy tamping. Two are concerned with case histories. The third, by Jessberger and Beine, deals with the mechanics of the process and the development of relationships useful for selection of weight size, weight mass, and height of fall to achieve densification to a desired depth. Special laboratory apparatus has been developed which can be used to measure dynamic stresses in the soil after impact, thus enabling determination of fall heights and falling weight masses required to develop a failure condition in the soil.

Jessberger and Beine show that the dynamic stress at the soil surface, $\sigma_{o,dyn}$ can be related to the base area of the weight, A, the mass of the weight, m, and the height of fall, h according to

$$\sigma_{o,dyn} = \alpha \cdot \frac{m}{A} \sqrt{2 \cdot g \cdot h} \qquad (1)$$

where α is a proportionality constant, and g is the acceleration of gravity. To obtain densification at depth z will require developing a dynamic stress $\sigma_{z,dyn}$ at that depth to break down the soil structure. Two equations are given for estimation of $\sigma_{z,dyn}$

$$\sigma_{z,dyn} = \alpha \cdot \frac{m}{A} \sqrt{2 \cdot g \cdot h} \left(1 - \frac{z}{\sqrt{z^2 + r^2}}\right)^{\nu}$$

$$7 < \nu < 15 \qquad (2)$$

$$\sigma_{z,dyn} = \alpha \cdot \frac{m}{A} \sqrt{2 \cdot g \cdot h} \left(1 + \frac{z}{r} \tan \theta_o\right)^{-2}$$

$$15° < \theta_o < 25° \qquad (3)$$

where r is the radius of the loaded area. In principle, then, the constant α can be determined by laboratory test, and the value of $\sigma_{z,dyn}$ could be estimated so as to create a failure condition. This would require knowledge of the soil strength envelope, the initial in-situ stress condition, and the proportion of $\sigma_{z,dyn}$ that is transmitted to the pore water. On a qualitative basis the equations establish the functional forms relating A, h and m for a given z. It will be of interest to relate these results to the empirically derived relationship that the effective depth of improvement, D, can be given by

$$D = 1/2 \sqrt{Wh} \qquad (4)$$

where W is the falling weight.

Hanzawa describes a case of underwater dynamic consolidation at a site in the Arabian Gulf where it was necessary to densify a 5 m-thick layer of loose sand overlying a medium dense fine sand. The silt content of the loose sand was 20 to 40 percent. A square tamper 2.4 x 2.4 m in plan weighing 32 tons was dropped 10 to 12 m. This energy was sufficient to improve the properties of the full 5 m of loose sand, as may be seen from Hanzawa's Fig. 5. The shear strength of undisturbed samples was increased by at least 100 percent, and the resistance to liquefaction was increased by 40 percent. The stress-strain curves showed a change from brittle behavior before tamping to plastic behavior afterwards. A decrease in water content of about 5 percentage points at the center of the layer was measured.

Charles, Burford, and Watts provide specific details about dynamic consolidation used for ground improvement at 5 sites. Materials treated included boulder clay; refuse fills composed mainly of clayey sand and ash, with fragments of brick, glass, metals, etc.; soft alluvial soil; and domestic refuse. Graphs are included which show settlement and pore pressures as a function of time following compaction. Charles et al. conclude that dynamic consolidation can be better than surcharge loading for old refuse fills. One reason for this is that the high impact loads can crush buried containers. They conclude also that widespread use of dynamic consolidation on soft alluvial soils is unlikely.

SOIL IMPROVEMENT BY PRECOMPRESSION

Ten papers assigned to Session 12 fall into the category of soil improvement by precompression. Of these, seven are case histories. An eleventh paper, by Dingazov and Germanov, has been included in Session 12 for review; however, it is not concerned directly with soil improvement. In their paper Dingazov and Germanov present theoretically derived isochrones of pore pressure in the clay cores of high dams at the end of construction and after rapid filling of the reservoir. They take into account soil creep and degree of saturation in these solutions.

Preloading Analysis

The paper by Aboshi, Matsuda, and Okuda is relevant to the design of surcharge fills for preloading without vertical drains. By means of a special consolidometer consisting of five oedometers connected in series it is shown that the consolidation at different distances from a drainage boundary in a layer undergoing one-dimensional compression is not uniform. The strain and residual pore pressure at the end of primary consolidation, as defined in the usual way, vary with z/H, as may be seen in their Fig. 3. Data are presented to illustrate the rebound at some points within a layer, and continued consolidation at others, after removal of a surcharge loading used to accelerate the settlement to be expected under a permanent loading. Aboshi et al. show also that the greater the average degree of consolidation reached under a surcharge loading, the lower will be the rate of secondary compression or creep settlement when it subsequently reappears. Excellent agreement between observed and predicted settlements for a field case is shown. Unfortunately, details of how the prediction was made are not presented.

Preloading has been used successfully by Colleselli, Mazzucato, and Previatello for improvement of interstratified clay, silt, and sand deposits in the lower Po Valley. They report that the results of standard laboratory compression tests can be used to calculate settlements in good agreement with observed values. The Skempton-Bjerrum method is used to estimate the settlement. Predictions of settlement rates using laboratory derived values of consolidation coefficient and the Terzaghi theory did not give good results. Predictions were improved if field-measured values of c_v deduced from the behavior of test embankments were used.

Vertical Drains

Akagi describes a field test in which 25 closely spaced (1.2 m on centers) closed-end, mandrel-driven sand drains were installed. The purpose was to determine the effects of installing displacement drains on the properties of soft clay. As would be expected, drain installation in this way induced substantial excess pore-water pressures, caused ground heave and lateral displacement, and was followed by consolidation. Dissipation of excess pore pressures was rapid. Lateral and vertical coefficients of consolidation were reduced to a fraction of their initial values.

The prediction of the time-settlement behavior for untreated treated ground adjacent to the test section was poor. Akagi concluded, as a result, that our inability to determine vital parameters for prediction of the time-settlement relationship for an embankment on an untreated foundation is a more serious problem than the accurate definition of complexities brought in by sand drain installation.

Solutions for clay consolidation using vertical prefabricated drains are developed by Hansbo. The effects of smear and well resistance are taken into account. The results of these analyses, in the form of easily applied equations, are given in Hansbo's previous article in Ground Engineering (Vol. 12, No. 5, 1979) and summarized in our state-of-the-art report. Among the main findings are that:

1. Well resistance can adversely affect the discharge capacity of small diameter sand drains and long, prefabricated band-shaped drains.
2. The equivalent diameter of a band-shaped drain can be defined by

$$d = \frac{2(b + t)}{\pi} \tag{5}$$

   where b is drain width and t is its thickness.
3. The filter should be fine enough to prevent clogging of the core.
4. A prefabricated drain should have a high discharge capacity.

Geodrains were used to consolidate the soft, lacustrine deposits up to 14 m thick for the foundations of the 43 m high Lornex tailings dam in British Columbia, Canada. Burke and Smucha report that spacings of 2.4 m were used under the main embankment, with 4.6 m spacings under upstream and downstream berms. The embankment placement rate was controlled by the rate of dissipation of excess pore water pressures. An interesting feature of the instrumentation on the project was use of a weir near the downstream toe to measure water coming out of the foundation. To facilitate installation of the drains, a working pad consisting of a 0.9 m thick sand layer over a geotextile membrane was used.

Chalmers and Harris present a case history of a heavy 6-storey steel frame building with a raft foundation placed on weak, cohesive soils improved by preloading and vertical drainage with Sandwicks. Sandwicks are seamless woven polypropylene stockings 65 mm in diamter, pneumatically filled on site with washed and graded sand. The preloading program and building performance were monitored with piezometers, settlement indicators, and inclinometers. Measured and predicted settlements agreed well. Pore pressure contours and settlement profiles are presented in the paper.

Chalmers and Harris found that a simple analytical treatment for the Sandwick design spacing yielded good results. Disturbance caused by the displacement method of drain installation did not appear to cause additional settlement. The use of a conventional pile foundation for this structure at this site would have doubled the cost.

Geodrains and surcharge loadings were also used to preconsolidate the marine clay underlying the second runway, taxiways, and turnoffs of the new Changi International Airport in Singapore, as described by Choa, Karunartne, Ramaswarmy, Vijiaratnam, and Lee. An area of 275,000 m^2 was treated to an average depth of 18.5 m using a 4 m sand surcharge. Design Geodrain spacings were 3.2 m in a square grid in areas where the clay thickness was 5 to 15 m, and 2.5 m where the clay thickness was greater than 15 m. Pore pressure vs. time and settlement vs. time data are presented.

Choa et al found that the two most reliable indicators of the degree of improvement were settlement records at various depths and precompression pressure values obtained from careful consolidation tests on undisturbed samples. A "stagnation" of pore water pressures in the treated area at 3 to 6 m above the areal groundwater table was found. It was attributed to a back pressure effect from adjacent untreated areas and change in the coefficients of consolidation and compressibility in the region of the pre-consolidation pressure.

An existing airport runway in Turku, Finland, was lengthened 560 m over a 4 to 14 m thick layer of soft clay. Geodrains were also used at this site, as described by Rathmayer and Leminen. Spacings were either 1.0 m on centers or 1.3 m on centers. The factors leading to the choice of this method of soil improvement are listed, and the design method used is summarized. Predicted and observed time-settlement curves agreed very well as shown by the authors' Fig. 8. Rathmayer and Leminen report somewhat anamolous pore pressure results, with excess pore pressures remaining at the end of the consolidation period. The exact reason for this has not been determined.

Tsai, Lee and Chao evaluated soil replacement, drainage, preloading, vibro-compaction, electro-osmosis, and chemical stabilization before selecting sand drains and compacted sand piles for use at a previously reclaimed marshland site to be developed as a raw material storage yard. A 5 to 8 m thick layer of soft ground was called upon to carry up to 350 kPa. Compacted sand piles were used in stacker-reclaimer areas of more restrictive allowable settlements. The sand drains used in the material storage yard were 400 mm in diameter and spaced at 2.1 or 3.4 m. The Barron and Terzaghi theories were used for design. Results presented by Tsai et al indicate excellent agreement between estimated and observed time-settlement curves (their Figs. 3 and 5). Agreement between predicted and measured pore pressure vs. time curves was only fair, as shown in their Fig. 3.

Although assigned to Session 4 of this Conference, the paper by Trautwein, Olson, and Thomas is relevant to precompression with vertical drains. They describe improved equipment for measurement of radial consolidation properties of clays and present simple curve fitting techniques useful to practicing engineers.

Electro-Osmosis

Casagrande, Wade, Wakely, and Loughney describe two successful applications of electro-osmosis for stabilization of sensitive silts, thus enabling excavation on steeper slopes than would have been possible otherwise: 2.5 to 1 instead of 10 to 1 in one case, and 2.5 to 1 instead of 5 to 1 in the other. Diagrams and photographs in the paper illustrate the project layouts and conduct of the work. Both projects were in British Columbia, Canada. At the Canadian Pacific Railroad project, the spacing between anodes and cathodes was 3.0 m; at the Revelstoke Dam excavation, it was 4.6 m, and voltages were 150 and 160 volts DC, respectively. At the Revelstoke Dam the anodes and cathodes penetrated the full thickness of the silt stratum, a depth of 12 to 61 m. Such a great treatment depth may be unprecedented.

INJECTION AND GROUTING

Four papers assigned to Session 12 are on some aspect of injection and grouting for ground improvement. Joshi, Natt, and Wright describe a field application and laboratory research on lime and lime-flyash slurry injection. A three stage injection of a 13 m high unstable railroad embankment and its foundation had the objective of penetrating shear planes, tension cracks, and voids. The embankment and its foundation have performed well in the three years since treatment, as reflected by an allowable train speed of 70 km/hr after treatment as compared to 32 km/hr before.

The results of Joshi et al's laboratory tests on soft, high water content pastes of bentonite and kaolinite indicated that lime deposited in seams by slurry injection not only reacts with the soil adjacent to the seam, but also with the clay some distance away. Although the visible reaction extended into the paste a distance of only 16 mm after 18 months, chemical analysis showed that calcium had migrated 75 mm into the clay. It was concluded that soft clays can be stabilized by lime injection.

Coumoulos and Koryalos describe in detail a grouting technique for use in uncompacted, man-made debris fills, composed mainly of stones, timber, concrete, and garbage. A fluid cement grout containing a small amount of bentonite is used to penetrate and fill voids. Grout hole spacings of 2.5 m or less are required. The procedure can be used both under damaged structures and prior to construction of new buildings.

The electrochemical hardening of highly plastic marine clay containing a high content of soluble salts was studied by Katti, Dongarwar and Patwardhan. The test program included injection of NaCl solution at the anode and the incorporation of $CaCl_2$ piles in the samples. It was found that the zones of hardening around the electrodes could be increased by reversing polarity.

An existing structure founded on loose, silty, fine sands was to be used for an application requiring operation of die forging machines.

To stabilize the sand against excessive settlement and vibration under the dynamic loads, a program of compaction and silicate chemical grouting was proposed. Woods and Partos describe how laboratory resonant column tests were used to determine that the grouted sand would have a required shear wave velocity representative of adequate shear modulus. Crosshole shear wave velocity measurements were made to evaluate the effects of grouting in the field. The crosshole test was a sensitive and accurate means for evaluating the effects of grouting on soil stiffness. It was possible to exceed the minimum required shear wave velocity by the grouting procedure used, and Woods and Partos report that the forging equipment performed satisfactorily. Relevant data and results are contained on figures in their paper.

Although not a part of this session, the paper by Aleksandrovsky et al in Session 3 is of interest because it deals with jet grouting. The technique used in the USSR differs from that in Japan in that lower water pressures and higher discharge rates are used.

ADMIXTURE STABILIZATION

Introduction

In our state-of-the-art report, stabilization using admixtures and by ion exchange was reviewed according to several topics: principles of admixture stabilization, properties of treated soils, new stabilizer materials, structural fills, and deep mixing methods. The eleven papers contributed to Session 12 that are about soil improvement using admixtures can be reviewed within this same framework.

Principles of Admixture Stabilization

An extensive investigation, involving tests on 19 red tropical soils, was made by Queiroz de Carvalho to determine factors responsible for effective lime stabilization. Lime reactivity, defined as the difference between the unconfined compressive strength of the lime-treated soil after 28 days curing and the unconfined compressive strength of the untreated soil, was used as the measure of effectiveness. Lime reactivity did not correlate with usual classification properties, e.g., Atterberg limits, clay content, density. It was found that by far the best single correlation was with the amorphous silica content. If the alumina content was included as a second parameter, then the correlation was further improved. Thus, the importance of amorphous materials to lime-soil reactions is clearly shown, at least for the early stages of the cure period. Queiroz de Carvalho presents a sound explanation for why this should be so.

A similar finding is reported by Brandl. His Fig. 11 shows a linear increase in compressive strength 270 days after lime treatment with percentage of what he terms "semimovable" silica in the untreated soil. While it has been recognized for some time that lime stabilization is of greatest effectiveness in highly plastic soils, it may be that the significant factor is that it is these soils that contain the greatest amounts of amorphous constituents.

Wagner, Harmse, Stone, and Ellis describe the chemical treatment of a dispersive soil reservoir and embankment that had failed on first filling. The paper by McDonald, Stone, and Ingles, included in Session 6 of the Conference, also deals with dispersive soils and their stabilization. Together these two papers present an excellent overview of the practical treatment of dams and reservoirs subject to dispersive soil problems. It has been recognized for some years that calcium is an effective stabilizer for dispersive soils, and lime has been used in a number of cases. However, McDonald et al discuss the role of bicarbonate in dispersive soil behavior and how calcium can be removed as a result of carbonation if lime is used. As gypsum is not subject to this problem, and it is lower in cost, it was used successfully by Wagener et al for repair of the failed embankment and reservoir bottom and for treatment of the reservoir water.

Another paper from Session 6 that is of interest within the context of admixture stabilization is that by Kastman and Huibregtse. It is concerned with methods for detoxifying soil and includes consideration of the introduction of chemicals to oxidize, reduce, precipitate, neutralize, or polymerize the hazardous constituents.

Properties of Treated Soils

The paper by Brandl cited above contains a large number of test results for four soils treated with lime, Portland cement, and chemicals. The soils ranged from silty sand to clay. The beneficial effects of cement and lime on mechanical properties are shown as functions of additive content and curing time and are consistent with observations by other investigators. Brandl concludes that the direct measurement of a mechanical property continues to be the best means for evaluation of a stabilizer's effectiveness. Also significant is the finding that almost all mechanical properties were adversely influenced by addition of sodium chloride or calcium chloride as stabilizers.

Evans and Bell present the results of laboratory investigations and describe field applications of phosphoric acid and lime stabilization of loess soil. Both materials, when added in relatively small amounts (less than 5 percent), rendered the soil non-dispersive and increased strength. The field applications included erosion control, fill stabilization, slope stabilization, and stabilized cut-off trenches. Very little on the subject of phosphoric acid stabilization of soils has appeared for almost 20 years. The data presented by Evans and Bell, therefore, is of interest in assessing the present potential for this form of treatment. It is significant, however, that economics, handling hazards, and mixing considerations all favored the use of lime for the purposes discussed in the paper.

Hammond studied the autogenous healing of lime and cement-stabilized soils. By autogenous healing is meant that regain in strength and stiffness that can result following fracture or flawing of a cemented material. Some 1,400 specimens were tested in this study. The most important factors controlling the amount of strength regain were found to be the initial curing period, length of cure after disturbance, curing conditions, and the type and amount of stabilizer.

Healing is favord by short initial curing periods (<14 days), long curing periods after flaws are induced, humid curing, and lime as the stabilizer instead of cement. Although these results might be anticipated qualitatively, a particular value of the paper is establishing quantitatively the magnitudes of the effects.

New Stabilizer Materials

The need for stabilization of soft clays at high water contents is addressed by Matsuo and Kamon. The addition of polyvalent cations was studied as a means for property improvement. Aluminum chloride and iron powder were selected as sources of Al^{3+} and Fe^{3+}. Substantial increases in strength were obtained. It is hypothesized that the aluminum aggregates the clay particles as a result of ion exchange. Oxidation of the iron powder in the soil releases Fe^{3+} which binds particles together by physical adsorption. Because oxidation is a necessary part of the combined reaction, the method is considered by Matsuo and Kamon as applicable only for shallow soil stabilization.

Gaspar reports on chemical soil treatments using chemicals of the type sometimes referred to as compaction aids. One of these is Reynolds Road Packer which has a pH of 1. The other reported on is CBV^{77}, a similar material but with pH = 0. Previous reports on the effectiveness of materials such as these have been mixed; the present case appears to be no exception. Under proper conditions these chemical treatments may be economical in regions short of coarse-grained material.

Structural Fills

Lychko, Pevzner, Belenky, Ilyina, and Zagovora describe the use of open hearth slags and carbonate sludges in foundation engineering. Special procedures are used during construction of fills from slags to prevent their swelling and decomposition. To stabilize and improve the properties of the sludges, a treatment using a non-uniform direct electric field has been developed. Evidently crossing electrical gradients are applied that result in accelerating the dewatering process by a factor of two. Intermittent application of voltage allows a 30 percent decrease in power consumption. The water content of carbonate sludges can be reduced to 7 to 10 percent by this treatment; whereas, with usual drainage techniques a reduction only to 30 to 32 percent could be achieved. The material is suitable for use in structural fills following treatment. The presentation of further details of this innovative technique by Lychko et al is encouraged.

Deep Mixing Methods

Holm, Bredenberg, and Broms report further on the development of lime columns as foundations for light structures in Sweden. Strength values as a function of time are presented for four sites. Among the tests used for strength determination are unconfined compression and fall cone in the laboratory, and the pressuremeter, weight sounding device, lime column penetrometer, and earth screw auger in the field. It is proposed that the settlement of an area reinforced by lime columns can be estimated based on the assumption that the stiffness corresponds to the sum of stiffnesses of columns and intervening unstabilized soil. The time-settlement relationship can be calculated, according to Holm et al, assuming that the lime columns function as vertical drains.

This latter assumption is contrary to Terashi and Tanaka, who state that the permeability of columns made using the deep mixing method in soft clay is very low, and therefore, the columns cannot be considered as drains. Kawasaki, Niina, Saitoh, Suzuki, and Honjyo state that the permeability of improved soil is lowered with increased cement content. It appears, therefore, that clarification be made of the conditions under which cement and lime columns of soft clay can act as vertical drains.

Terashi and Tanaka examined the behavior of column groups made by deep mixing using large model tests and finite element analyses. They point out that the column behaves much like a pile of low strength material. Step-by-step methods for estimation of bearing capacity and settlement are presented. There are similarities with analyses of stone and sand column foundations.

Kawasaki et al describe the features of a deep mixing method using a cement slurry, give detailed information on the properties of improved soils, and present some examples of construction. Their paper provides an up-to-date report on the status of the method.

THERMAL STABILIZATION

Only one paper in Session 12 relates to thermal stabilization. Knutsson reports on the shear strength of frozen soil. He evaluated the effect of freezing conditions on the strength of a sand, a silt, and a clay as measured by direct simple shear. It was found that samples frozen unidirectionally and sheared in a direction perpendicular to the heat flow were weaker than samples frozen isotropically. This result can be explained in terms of the orientation of hexagonal platy ice crystals relative to the shear directions.

SOIL REINFORCEMENT

Introduction

As noted in our state-of-the-art report, the subject of soil reinforcement has been studied intensively, and numerous important advances have been made over the past several years. The eight papers on earth reinforcement contributed to this session contain further significant contributions.

Stone and Sand Columns

Stone column installation techniques presently used in India utilize commonly available piling equipment, as discussed by Datye and Nagaraji. A mixture of stone and sand in a ratio of 1:0.2

to 0.5 is rammed into pre-bored holes as illustrated in Fig. 1 of their paper. They indicate that better columns can be obtained at less cost by these means than by utilization of vibrocompaction equipment. A design approach is presented by Datye and Nagaraji that is based on the results of a load test on a single column. It is considered, in accord with previous studies, that the columns derive support from the surrounding clay as a result of resistance to cylindrical cavity expansion.

The paper by Hartikarnen in Session 1 of this Conference also deals with stone columns. A case history is described in which stone columns were used to support a grain silo. The columns were required to transmit shear and dissipate excess pore water pressure in the event of an earthquake.

Soil Nailing

Schlosser, Louis, Kernoa, and Eckmann describe an experimental study of the behavior under shearing of a silty clay of low plasticity reinforced by bars perpendicular to the shear plane. Relevant bar and soil displacements and forces were measured to enable evaluation of both the interactions between the materials and their theoretical interpretation.

The results obtained by Schlosser et al indicate that shearing transversely to the bar reinforcements in a nailed soil results in the development of flexural stress in the bars. The shear carried by the reinforcements can be interpreted as mobilization of an apparent cohesion in the nailed soil. The maximum value of the cohesion is developed when the lateral load resistance of the soil adjacent to the nail is reached.

The results of large scale field tests (6 m high cuts), laboratory tests, and soil mechanics analyses have been used by Gässler and Gudehus both to develop an understanding of soil nailing reinforcement mechanisms and as a basis for preparation of design charts. Figures illustrating horizontal displacements of nailed slopes and horizontal earth pressures due to soil weight and surface surcharge loadings are presented. Horizontal displacements are only a very small percentage of the wall height, and horizontal earth pressures, which are well below the Coulomb value,can be assumed uniformly distributed.

Gässler and Gudehus conclude that stability calculations can be based on a two-body translation as shown in their Fig. 11. Axial forces in the nails are accounted for directly; whereas, transverse shears of the type studied by Schlosser et al apparently are not. An indirect accounting for this shear may be through a fictitious traction included by Gässler and Gudehus to close their force polygons.

The lateral support system described by Bang et al (see our state-of-the-art report, Fig. 46) is very similar in construction and detail to the soil nailing system studied by Gässler and Gudehus. The paper by Shen, Bang, DeNatale, and Mitchell in Session 1 of this Conference describes field measurements and analytical predictions for a 12 m excavation using their system.

Tensile Reinforcements and Reinforced Earth

Jewell and Jones point out that the applicability of reinforced earth construction could be greatly extended if cohesive soils and waste materials could be used as backfill materials. The results of laboratory tests and a limit analysis are presented to show that both the short and long term strength of cohesive soil can be increased by reinforcement.

Because cohesive soils and mine waste are more corrosive than the cohesionless soils used for most reinforced earth structures, plastics and glass, which are highly resistant to chemical and biological attack, are proposed as reinforcing materials. Examples of mine waste reinforced structures reinforced with plastic grids and glass fiber strips are presented by Jewell and Jones. The authors state that a grid is a particularly suitable form of reinforcement. No data are presented to indicate the effectiveness of grids relative to the other reinforcement forms.

Narain, Saran, and Talwar describe laboratory model tests that were designed to study the behavior of reinforced earth walls in which failure was by reinforcement slippage rather than by rupture. Well-instrumented model walls 1.5 m high were used. The variation in tension along reinforcements, the locus of maximum tension behind the facing, and measured horizontal stresses as a function of depth are shown in their Figs. 2, 3, and 6, respectively. The experimental relationships found by Narain et al agree surprisingly well with those presently used for reinforced earth wall design as summarized in our state-of-the-art report.

Brown and Poulos present the results of a limited parameter study of the effect of buried horizontal reinforcements on bearing capacity. The finite element solution used assumes full displacement compatability; i.e., no slip along the soil-reinforcement interface until full mobilization of friction. Constant resistance is assumed thereafter. The results show that the reinforcement spreads the footing load and causes a wider and deeper mobilization of soil strength. The stresses carried by the reinforcement increase with depth of layer. Brown and Poulos conclude that their results give reasonable agreement with published model test results. The results of analyses to illustrate the reinforcing effect of reinforcements in a soil bridging a soft inclusion or void at depth are presented.

McGown, Andrawes, Mashhour, and Myles determined the influence of horizontal and inclined layers of non-woven geotextiles in plane strain model sand embankments 0.9 m high. The embankments were constructed on soft, sponge rubber foundations. The stiffening effect of the reinforcement as regards horizontal deformations and settlements was demonstrated.

The authors did a special test aimed at taking into account the fact that tensile reinforcement is most effective if oriented in the direction of maximum extensional strains. In attempting to set up the appropriate orientation of fabric layers (their Fig. 4), the embankment construction sequence was changed. As a result, extensional strain directions were also changed, so the new fabric orientation was inappropriate. Thus, the influences of construction effects on strain field orientations should be considered when considering tensile reinforcement directions. McGown et al obtained good agreement between measured behavior and that predicted by a finite element analysis. It is important that the construction sequence be correctly modelled in the analysis.

In their paper, Cartier, Long, Pouget, Barqillat, and Cudennec discuss both urban refuse and used tires in civil engineering. Construction over municipal and industrial waste fills is facilitated by preloading under surcharge fills to minimize later settlements. Biochemical degradation of materials should be considered, as well as compression by usual means.

Old tires can be used as reinforcing inclusions. Cartier et al illustrate ways in which tires may be arranged and interconnected to form tensile reinforcements (their Tables 1 and 2 and Fig. 4). Assemblages of whole tires, treads, or sidewalls are possible. It may be noted that an earth structure reinforced with tire side walls interconnected using bent reinforcing bars was constructed recently in California for a highway slope stabilization.

CONCLUSIONS

The papers included in Session 12 provide significant extension to the state-of-knowledge and state-of-practice in the area of soil improvement. There are several results considered of special importance in the opinion of the authors, and these are listed below in conclusion of our general report.

Deep Compaction

1. Valuable new insights into soil densification by blasting are provided by the field test results of Klohn et al and Pilot et al. In addition, the need for field test programs prior to large scale blasting projects is shown.

2. The results of the comparative study of deep densification methods by Faraco can be used to support the generalization that different methods will have different effectiveness at different sites. They also confirm that methods of deep densification are generally ineffective in the first one or two meters below the ground surface.

3. The theoretical basis for design of heavy tamping, as reflected by the interrelationships between mass of the falling weight, its cross section, its height of drop, and the depth of ground improvement, has been extended and improved by Jessberger and Beine.

4. The special usefulness of heavy tamping for compaction of refuse fills is illustrated by Charles et al.

Precompression

1. The test results of Aboshi et al support previously stated, but not confirmed, concepts of consolidation under surcharge fills and behavior after surcharge removal.

2. Available theory for prediction of settlement rates for precompression using vertical drains gives good results (Chalmers and Harris, Choa et al). Prediction of pore pressure dissipation rates is not as satisfactory.

Injection and Grouting

1. There is now evidence that soft clays can be improved by lime slurry injection (Joshi et al).

2. Refuse fills can be stabilized by grouting (Coumoulos and Koryalos).

3. Evaluation of grouted soil properties by the resonant column test in the laboratory and crosshole shear wave velocity measurements in the field can be excellent means for determination of grouting effectiveness for ground stiffening purposes (Woods and Partos).

Admixture Stabilization

1. Amorphous silica is the most important soil constituent needed to insure lime reactivity and successful hardening of lime-treated soils (Queiroz de Carvalho and Brandl).

2. Gypsum may be a more effective and more economical admixture for treatment of dispersive clays than lime (Wagener et al).

3. Quantitative information on factors influencing antogenous healing of lime and cement treated soils is now available (Hammond).

4. Clay stabilization using the multivalent cation system Al^{3+} plus Fe^{3+} may be possible (Matsuo and Kamon).

5. It is not clear whether lime columns act as drains (Holm et al vs. Terashi and Tanaka).

Soil Reinforcement

1. Analysis procedures and design methods for soil nailing are now becoming available (Schlosser et al and Gässler and Gudehus).

2. Construction sequence can influence optimum orientation of tensile reinforcement (McGown et al).

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