MODERATOR'S REPORT ON SOIL IMPROVEMENT

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Reprinted from

Proceedings, Seventh Southeast Asian

Geotechnical Conference, Hong Kong, 22-26 Nov., 1982

Vol. 2, pp. 325-343

Southeast Asian Geotechnical Society

Hong Kong Institution of Engineers

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INTRODUCTION

The basic concept of soil improvement-densification, cementation, reinforcement, drainage, drying and heating although were developed hundreds and thousands of years ago, they still remain valid today. Tremendous progresses have been made in recent years in the development of techniques or methods for soil improvement. Today, there are hundreds of methods, many of them are patented, available on the market for improving or rather changing properties of soils and rocks. Some of these methods or techniques are well developed and have sound theoretical basis, many of them, on the other hand, are still very empirical.

Professors James Mitchell and R.K. Katti (1981) have presented a well documented State-of-the-Art report on the subject of Soil Improvement at the Tenth International Conference on Soil Mechanics and Foundation Engineering held in Stockholm in 1981. Since then, a number of papers have appeared in various literatures reporting case histories and new development of soil improvement techniques. At this conference, there are a total of 7 papers on the subject of soil improvement. They include:

one on preloading with compaction piles,

one on sand drains,

one on grouting,

one on using lime as admixtures for stabilization,

one on lime migration piles, and

two on vibro-replacement - stone columns.

Starting from the coming Monday, 29th November, a special symposium and short course on the Improvement of Soils and Rocks will be held in Bangkok under the joint sponsorship of the Asian Institute of Technology and the Southeast Asian Geotechnical Society. A large number of papers are going to be presented and discussed.

For my report today, due to time limitation, I will only present a general summary of soil improvement methods which are currently in use and then followed with short descriptions of four case records.

SUMMARY OF SOIL IMPROVEMENT METHODS

The development of soil improvement methods can be broadly divided into the following four approaches:

(1) Improvement of soil material by applying external physical energy - Soil improvement methods under this category include precompression and in situ densification by deep compaction. These methods improve the soil quality by changing its physical properties and therefore its engineering behavior. The constituents of the soil, in terms of chemical and mineralogical composition remain essentially unchanged. The primary effects of precompression are reduction of water content and void ratio of the soil with resultant effects of reducing compressibility and increasing strength. Precompression of a soil can be induced by surcharge load in terms of placing fills or vacuum loading. Precompression of a soil can be enhanced by introducing high permeability materials into the soil such as sand drains and prefabricated drains.

The main function of deep compaction is densification of a soil in situ. After densification, the soft or loose soil deposit will have lower water content, lower void ratio, lower compressibility and higher strength. There are a number of different methods to apply the external energy for deep compaction. They include blasting, vibration, heavy tamping or so called dynamic consolidation. Compaction piles and stone columns are densification methods by introducing additional foreign materials.

(2) Improvement of soil material by applying electrical or thermal energy - Due to their electric kinetic properties, the water content of fine-grained soils can be reduced by introducing electric current to the soils. This is the so called "electro-osmosis" of soil improvement. In addition to reduction of water content, this method will also induce electro-hardening and change in the chemical properties of a soil due to movement of ions in the double layers surrounding the soil particles.

Thermal energy, either heat or freezing, can alter the soil properties. In general, application of heat will cause irreversible changes in the soil properties. On the other hand, freezing is usually used as a temporary measure for soil stabilization.

(3) Improvement of soil material by introducing admixtures - Of the

many methods of ground improvement, the use of admixtures of various types is the oldest and most wide spread. Depending upon the type of admixtures used and the method of application or adding admixtures, the resultant "improved" soil material can be quite different, some with merely changes in their physical properties and others with significant changes in their chemical and mineralogical composition. Admixtures can be applied to a soil material either by mixing or by in situ method, commonly referred as soil stabilization and grouting or injection, respectively.

(4) Improvement of soil system - The properties of a soil system can be improved or changed by using inclusions. These methods are commonly known as "reinforced earth" or "earth reinforcement" One of the most rapid development in recent years is the use of synthetic fabrics for drainage and reinforcement of soils. The subject of "geotextiles" has drawn much attention in the geotechnical engineering field. International conferences (ENPC-LCPC, 1977; IFAI, 1982) on the use of geotextiles have been organized.

FACTORS AFFECTING SELECTION OF SOIL IMPROVEMENT METHODS

There are many factors which affect the effectiveness of a soil improvement method. Not every method can be used in every case. The selection of the most suitable and effective method in any particular case can only be made after evaluation of the factors specific to the problem on hand. Some of the most important factors are:

(1) The purpose of treatment or improvement and the intended use of the treated ground.

(2) The extent of improvement, including area, depth and total volume of soil to be treated.

(3) Soil type and properties.

(4) Availability of materials such as sand, gravel or admixtures.

(5) Environmental factors.

(6) Local experience and practice.

(7) Time schedule.

(8) Cost.

Table 1 summarizes some of the essential features of the various soil improvement techniques. The general applicability of each improvement method to the various types of soils is illustrated in Fig. 1.

TABLE 1

SUMMARY OF SOIL IMPROVEMENT METHODS

	METHOD	MOST SUITABLE SOIL TYPE	SPECIAL ADVANTAGES AND LIMITATIONS
PRECOMPRESSION	PRELOADING FILLS	Soft clays, silts, sanitary landfills	Simple, theory well developed, uniformity; require long time
	SURCHARGING FILLS	Soft clays, silts, Sanitary landfills	Simple, faster than above; large quantity of material required
	FILLS WITH SAND DRAINS	Soft clays, silts, organic deposits	Accelerates consolidation, can sustain large settlement; cause disturbance to soil
	FILLS WITH PRE-FABRICATED DRAINS	Soft clays, silts, organic deposits	Accelerates consolidation, rapid installation, less disturbance to soil
	VACUUM LOADING	Soft clays, silts	No stability problem such as fills; process rather complicated
	ELECTRO-OSMOSIS	Saturated silts, silty clays	Relatively fast; non-uniform properties; high cost
IN SITU DEEP COMPACTION	BLASTING	Sands, silts	Rapid, inexpensive suitable for any size of area; variable properties; dangerous
	VIBRATORY PROBE	Saturated or dry clean sand	Rapid, simple; ineffective at shallow depth
	VIBRO-COMPACTION	Cohesionless soil with fines less than 20%	Can obtain high relative density and uniformity
	COMPACTION PILES	Loose sandy soils, partly saturated clays	Can obtain high relative density, uniformity; fines limitation higher
	HEAVY TAMPING	Cohesionless soils, waste fills, partly saturated soils	Simple, rapid; requires control; cause vibration

(to be continued)

TABLE 1 (continue)

INJECTION AND GROUTING	PARTICULATE GROUTING	Med to coarse sand and gravel	Low cost; hard to evaluate result
	CHEMICAL GROUTING	Med silts and coarser	Low viscosity, controllable time for setting; high cost and hard to evaluate
	PRESSURE INJECTED LIME	Expansive clays	Only effective in narrow range of soil conditions
	DISPLACEMENT GROUT	Soft fine grained soils, soils with large voids	Good for correction of differential settlement, filling voids; careful control required
	ELECTRO-KINETIC INJECTION	Saturated silts, silty clays	No good for soil with high conductivity; expensive
	GROUTING	Sands, silts, clay	In soils can't be permeation grouted
ADMIXTURES	REMOVE AND REPLACE	Inorganic soil	Uniform, controlled soil when replaced; require mixing
	STRUCTURAL FILLS	Over soft soils	High strength, good load distribution
THERMAL	MIXING-IN-PLACE PILES AND WALLS	Soft or loose inorganic soils	In situ; difficult quality control
	HEATING	Fine grained soils	Irreversible improvement; high cost
	FREEZING	All soils	Temporary; high cost
REINFORCEMENT	VIBRO-REPLACEMENT (stone and Sand column)	Soft clays and alluvial deposits	Faster than precompression, some increase in bearing capacity, increase rate of pore water pressure dissipation
	ROOT PILES, SOIL NAILING	All soils	In situ reinforcement
	STRIPS AND MEMBRANES	Cohesionless soils	Economical, increase bearing capacity, can tolerate deformation

(abstracted from MITCHELL AND KATTI, 1981)

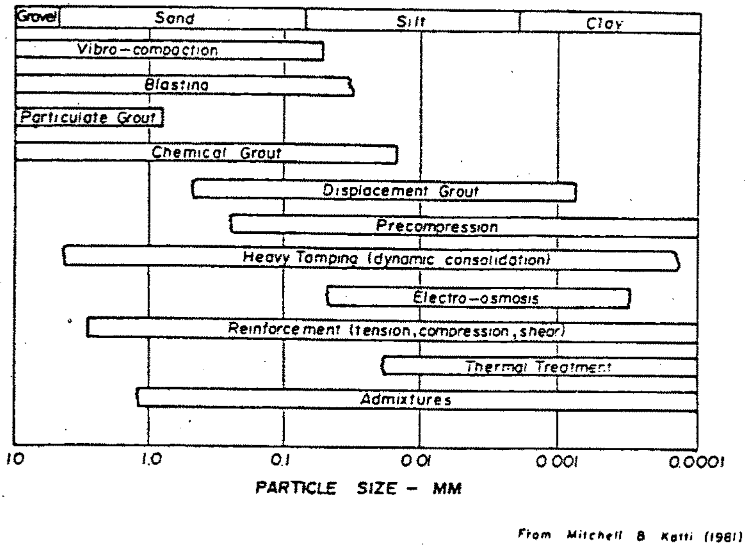

Fig. 1 Applicable Soil Grain Size Ranges for Different Improvement Methods

CASE RECORDS

Case Record 1 - Soil Improvement with Preloading

An oil tank farm (Yung Kong Oil Terminal) was constructed in the south western part of Taiwan for storage of oils at a site originally occupied by sugar cane and tomato farms. The total area of the site is about 12 hectares. The tank farm consists of eight storage tanks and other auxiliary facilities. Seven of the tanks are 60 m in diameter with 50,000 kl capacity and one tank of 35 m diameter with 10,000 kl capacity. The subsoils at the site primarily consisted of alternate layers of clayey silt or silty clay and silty sand. From the ground surface extending to the depth of about 23 m, the subsoils were relatively low in strength and high in compressibility, and they can be divided into five sublayers as illustrated in Fig. 2. Below that depth, the silty fine sand layer becomes much denser.

Results of analysis (MAA, 1982) clearly indicate that the top 23 m of the subsoils are not adequate to support the proposed tank construction. Either the subsoils have to be improved or the tank structures have to be supported by pile foundation. The major problems of the site were high liquefaction potential of the loose sand layer, stability problem of the clayey silt and the surface soil layers, and high compressibility of the two clayey silt or silty clay strata. For a 50,000 kl capacity oil storage tank,

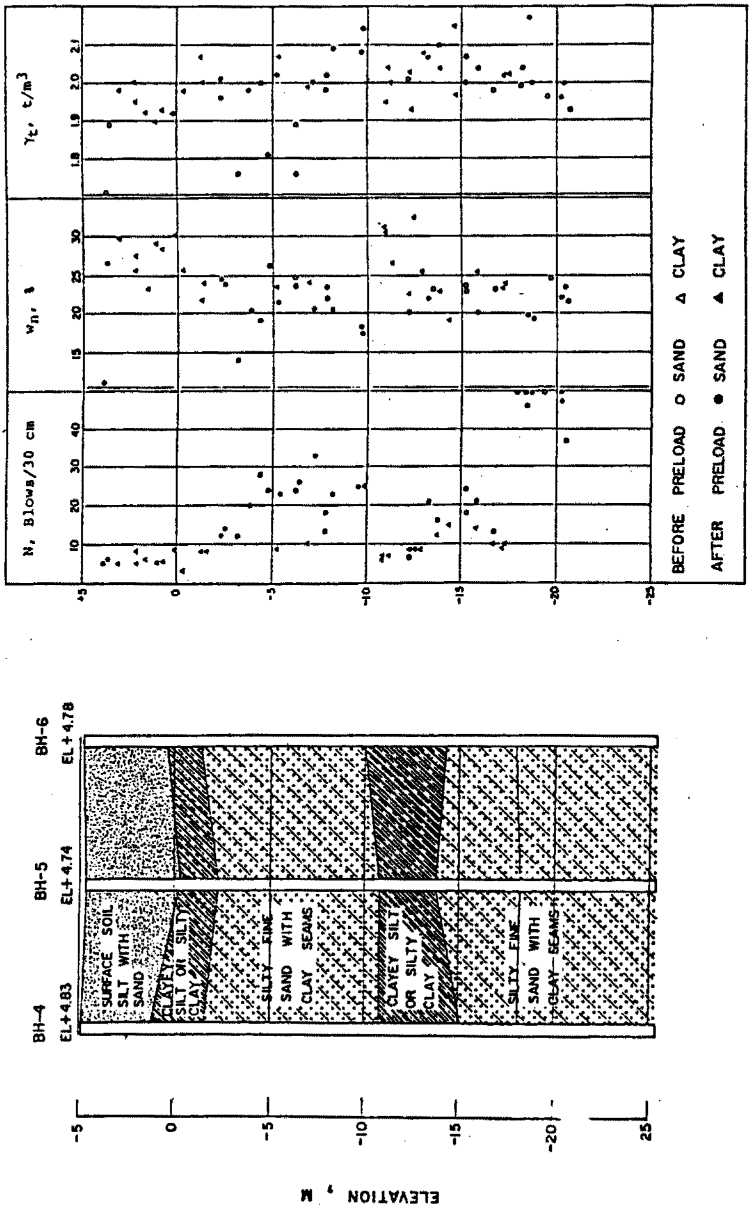

Fig. 2 Simplified Soil Profile and Soil Properties Before and After Preload - Case Record 1

it was estimated that the total settlement under center of the tank would be about 38 to 49 cm and the differential settlement between center and edge of the tank would be about 19 to 28 cm. These values of differential settlement are close to the limits suggested by various experts. In view of the large area involved in the project and the necessity of raising the ground surface level for drainage and grading purpose, it was decided that ground improvement by preloading would be the most economical measure. One of the most important technical factor which influenced the choice of improvement method was the existance of numerous thin layers or seams of permeable fine sand in the silty clay strata which would greatly facilitate dissipation of excess pore water pressure under the preload.

A simple instrumentation program was designed to monitor the effectiveness of preloading. Instrumentation installed included settlement plates and pneumatic piezometers in the center of the clay layers. The magnitude of the preload used was either equal to or slightly higher than the anticipated tank load, which was 20.64 tons per sq m and 12.47 tons per sq m for the two sizes of tanks, respectively. Figures 3 and 4 show the settlement records and piezometer readings at two of the tank sites during preloading. Several features are demonstrated in these figures. Firstly, the rate of settlement of the subsoils was quite rapid since most of the subsoils at the site were sandy soils and immediate elastic settlement contributed to a major portion of the total settlement. Secondly, the dissipation of excess pore water pressure in the clay layers under the area of Tank No. 3 was extremely rapid which showed the effectiveness of the existence of thin sand lenses within the clay strata. Thirdly, due to the large area of coverage of the preload, settlement calculations using Terzaghi's one dimensional theory yielded satisfactory prediction. However, Fig. 3 showed that the excess pore water pressure in the upper clay layer under Tank 2 preload did not dissipate although the settlement record indicated that settlement has virtually completed only few days after completion of the preload. This indicates incompatibility of pore water pressure measurement and settlement record. Unfortunately, due to inadequacy of the instrumentation program, the reasons responsible for this discrepancy could not be delineated.

The effectiveness of the preloading was further checked by soil boring and testing. It was found that after preloading, the N value increased, water content decreased and the total unit weight increased for both sand and clay as illustrated in Fig. 2. The undrained shear strength of the upper clay layer increased from 1.6 to 3.7 tons per sq m therefore, the stability of the soil was greatly improved. Values of the coefficient of volume compressibility of the clay layers decreased by two-third to three-fourth. The estimated residual settlements of the large tanks under water test were only 10 to 12 cm under the center and 5 to 7 cm along the edges.

Case Record 2 - Improvement of Foundation Soil with Sand Drains and Surcharge Fill

A total of three fuel oil storage tanks was proposed to be constructed at a new fossil power plant at Hsin-Ta-Kong in the southern part of Taiwan.

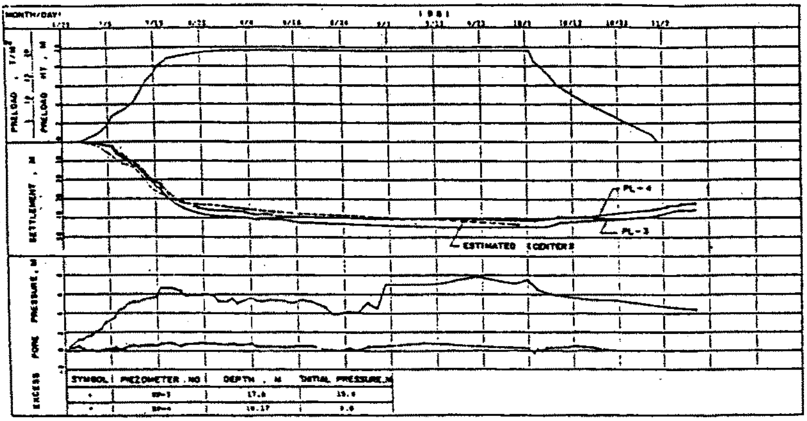

Fig. 3 Settlement and Pore Pressure Variation During Preload for Tank No. 2 - Case Record 1

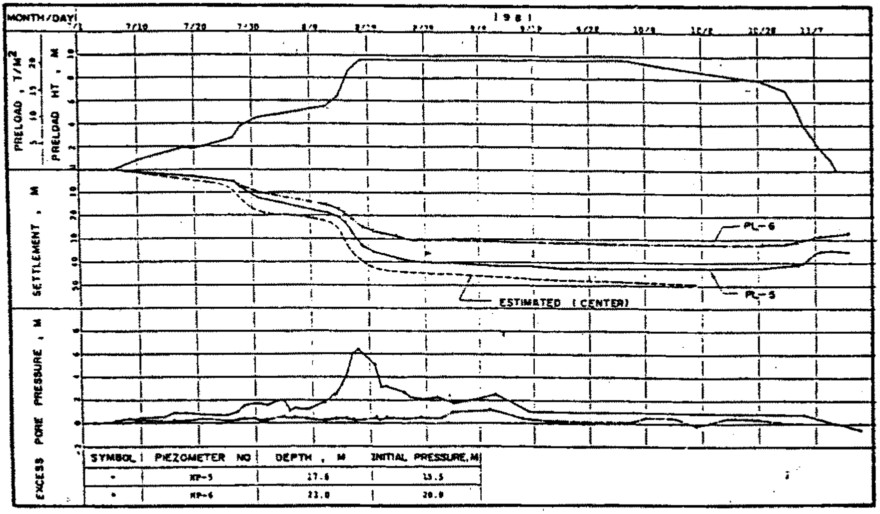

Fig. 4 Settlement and Pore Pressure Variation During Preload for Tank No. 3 - Case Record 1

The tanks are 70 m in diameter and 13 m high with a storage capacity of 50,000 kl. Each tank occupies a site area of about 6,900 sq m. The western and northern boundaries of the site are near seashore with the closest distance of only about 60 m.

The subsoils at the site consisted of alternative layers of silty sand and silty clay. From the ground surface downward, the subsoil strata could be approximately divided into 5 layers as shown in Fig. 5.

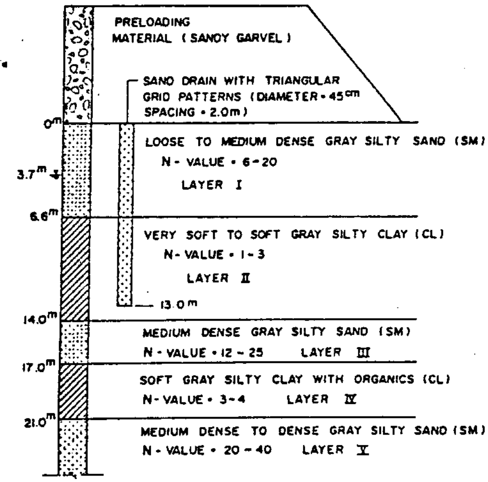

Fig. 5 Simplified Subsoil Profile - Case Record 2

They are: (1) From surface to 6.5 m, loose to medium dense silty sand. A part of this layer is hydraulic fill placed to raise the ground level of the entire power plant site, (2) Very soft to soft silty clay containing some sand lenses with N values of only 1 to 3, about 7.5 m thick, (3) Medium dense silty sand of about 3 m in thinkness, (4) Soft silty clay containing organic material, about 4 m thick, (5) Medium dense to dense silty sand containing seams of silty clay extending from a depth of 21 m downward.

The main foundation problems at this site for supporting the fuel storage tanks were the low bearing capacity, therefore stability problem of the two silty clay layers and the high compressibility of the soft silty clays. The estimated total settlement under proposed tank load was well over 1.0 m. Due to the existance of high permeability sand layers and sand seams in the clay strata, the designers decided to use the combination of sand drains and surcharge fill to improve the subsoil properties. A total of 1,746 number of sand drains of 45 cm diameter and 12.5 in length at 2 m spacing was installed at the site of each tank. The surcharge fill consisted of sandy gravel and was placed in about 0.5 m lifts up to a total height of 8.3 m which was equivalent to a load of 16.2 tons per sq m. This load is about 20 per cent higher than the design full tank load. Moh and Associates was engaged by the constructor, the Ret-Ser Engineering Agency to control and to monitor the preloading work (MAA 1981a). Instrumentations consisting of settlement points and piezometers were installed after the installation of sand drains and prior to the preloading.

The sand drains installed were of displacement type which have caused considerable disturbance to the subsoils and generated high excess pore water pressure in the silty clay. Due to the low stability of the clay strata, the surcharge fill had to be placed slowly in small lifts. The rate of preloading was controlled by stability analysis on the basis of measured pore water pressures and estimated variation of shear strength of the soil. Figure 6 shows the variation of factor of safety against height of preload.

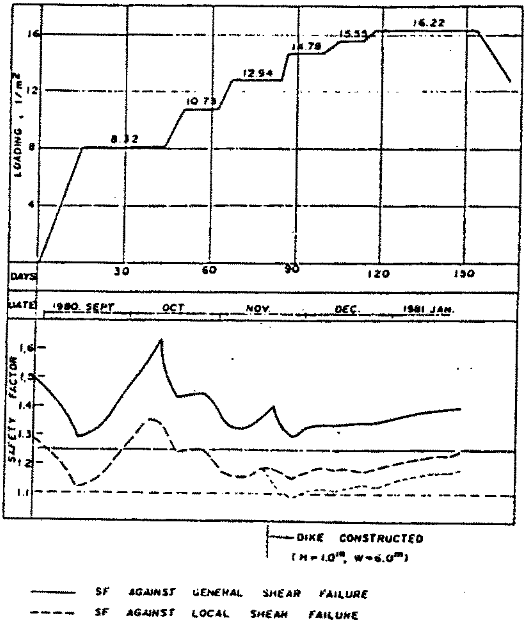

Fig. 6 Variation of Factor of Safety During Preloading - Case Record 2

Figures 7 and 8 present the results of settlement and pore water pressure measurements during preloading. The results indicate that the top sand layer (Layer I) was highly permeable and majority of settlement, both elastic and consolidation settlemnets in this layer occurred very rapidly. For the first clay layer (Layer II) which was the main layer for improvement, approximately 80 per cent of the total consolidation settlement had occurred during the period of preload. About 80 to 85 per cent of the excess pore pressure developed due to surcharge load had dissipated and the undrained shear strength increased from 1.9 to 4.0 tons per sq m. Dissipation of the excess pore water pressure in Layer IV was not as rapid as that in the upper clay layer although the thickness of the latter was larger. This tended to indicate that the sand drains played an effective role in accelerating consolidation of the soil. For the Layer V even though the soil was a medium dense to dense silty sand, considerable amount of settlement has occurred which amounted to about 40 per cent of the total settlement under preload. About two-thirds of the settlement was immediate elastic compression and one-third was consolidation settlement. The latter was most likely contributed by the silty clay seams in the sand layer.

By using soil parameters determined on samples taken after the preload period, the residual settlements of full tank were estimated to be about

15 cm under the center and 9 cm under the edge. The actual recorded settlement during water test was about 4 cm under the edge.

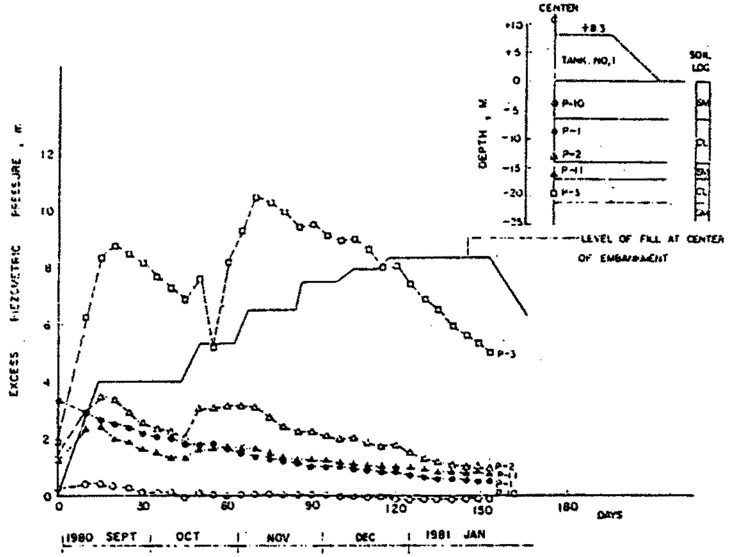

Fig. 7 Variation of Piezometric Level with Time in the Center of Preloaded Area - Case Record 2

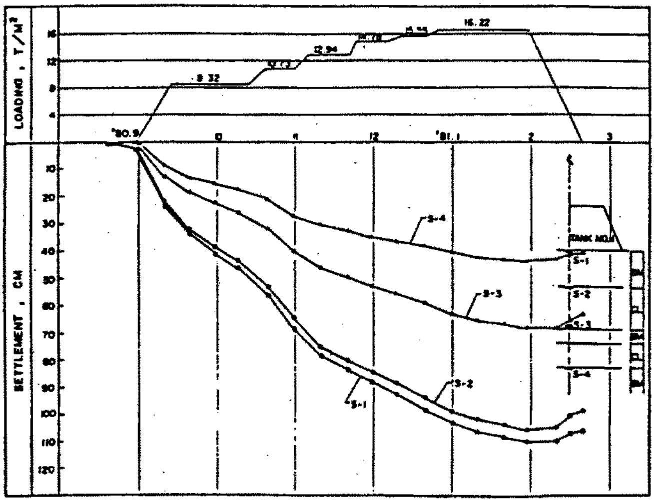

Fig. 8 Settlement Results at Center of Tank Area During Preload - Case Record 2

Case Record 3 - Soil Improvement with Sand Compaction Piles and Preloading

Two refrigerated LPG storage tanks, each 40 m in diameter and 25,000 kl in capacity were constructed on a reclaimed land. The subsoils consisted mainly silty sand or fine sand with thin layers of sandy silt. The top 10 to 15 m of the sandy soils was quite loose and has low relative density. Moreover, the variation of the subsoil deposit over the entire site appeared to be quite erratic. Typical soil profiles with average SPT values are shown in Fig. 9.

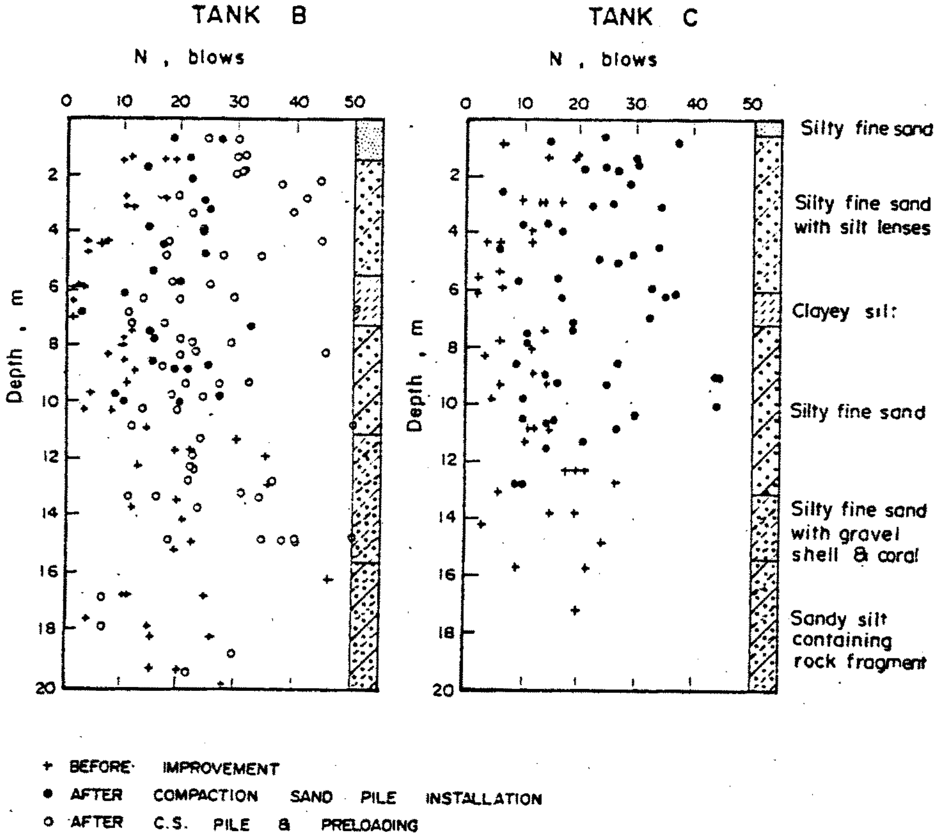

Fig. 9 Variation of N-value Before and After Soil Improvement - Case Record 3

As described by Moh, Woo and Yu (1982) in a paper presented at this conference, the major problems at the site were high liquefaction potential, low bearing capacity and high differential settlement. Comparisons were made between the use of pile foundation and mat foundation with soil improvement. The principal criterion of soil improvement at this site was to increase the relative density of the fine sand deposit within the top 10 m depth to a minimum value of 75 percent which would give a factor of safety of 1.17 against liquefaction under a seismic acceleration of 0.2 g. Analyses indicate that at this relative density, the bearing capacity of the subsoil would be sufficient to support a mat foundation and the magnitude of differential settlement would also decrease to allowable limit. For the purpose of achieveing better control of the settlement problem, a combined soil improvement scheme using compaction sand piles and preloading was adopted. The compaction sand pile system consisted of 40 cm diameter sand piles, 10 m long and spaced at 2.5 m centers in a triangular pattern. Preloading was schieved by placing sand fill over the tank area.

For the purpose of evaluating the effectiveness of compaction sand piles, Standard Penetration Tests and soil sampling were carried out after installation of the compaction piles and preload. Results of the SPT shown in Fig. 9 indicate that not only the N values of the soil within the depth where compaction sand piles were installed have increased,substantial densification was also achieved at deeper depths. Test results in the figure also indicate that placement of preload further increased the relative density of the soils at shallow depths. On the basis of the relative density values of the soil after improvement, the factor of safety against liquefaction increased from values less than 1.0 to more than 2.0. Factors of safety for mat foundation against bearing capacity failure were increased significantly to values above 3.0.

Due to variation of the subsoil conditions at the site, particularly that the subsoils on the wharf side (i.e. north side) appeared to be more compressible, considerable amount of differential settlement was expected. After the soil improvement with preloading and compaction sand piles, the amount of differential settlement was greatly reduced to tolerable limits , Fig. 10.

Case Record 4 - Soil Improvement for Flyover Approaches Using Prefabricated Drains and Surcharge Fill

A flyover for an expressway was constructed in Singapore on a turnkey basis. The Ret-Ser Engineering Agency was the constructor and the Moh and Associates was the designer responsible for both structural and geotechnical design. Due to gradient requirement, fill embankments were required to be constructed at both approaches. The required embankment height at the north approach was 2.9 to 3.3 m and that at the south approach varied from 3.3 to 4.0 m. The client, the Public Works Department, has set the settlement criterian: (1) the settlement of the approach embankments must reach 100 per cent primary consolidation and a significant portion of secondary compression should be completed at the end of construction, and (2) residual settlement after completion of

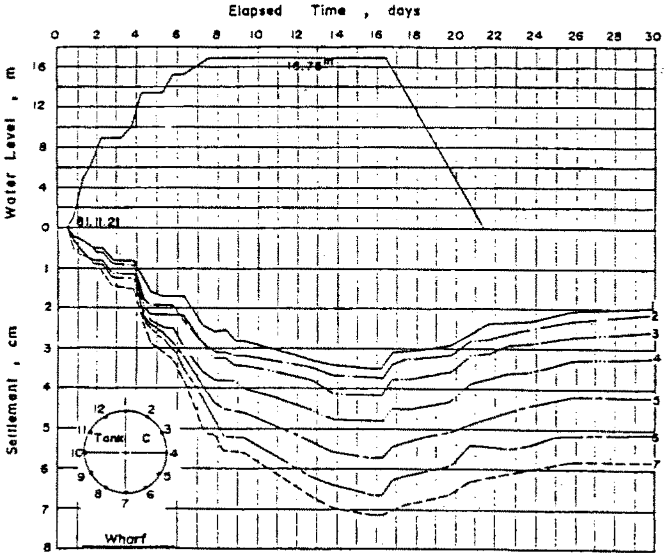

Fig. 10 Settlement Records of Tank C During Water Test - Case Record 3

construction shall not exceed 50 mm per year.

The subsoil conditions at the site are very poor. They consisted of a layer of silty clay top soil about 1.5 to 2.9 m thick, followed by a layer of very soft organic peaty clay and marine clay with water content over 100 per cent which was equal to or larger than the liquid limit of the soil. The peaty clay varied in thickness from 3 to 8 m at the north end and about 11 m at the south approach. This clay was underlain by a 1 to 4 m thick layer of loose silty sand and then followed by medium dense sand, very stiff clay, and very dense silty sand.

There were two major problems in designing the approaches. They were stability of the embankment fills and large settlement of the embankments. The estimated value of total settlement at the north approach was about 1.3 to 1.5 m. After careful evaluation of the soil properties and site conditions, it was decided that the subsoils sould be improved by using a combination of prefabricated drains and surcharge fill. At the north approach, prefabricated drains at specings of 1.1 m and 1.3 m were installed to a depth of about 10 m, i.e. to the top of the silty sand layer. A surcharge load of 0.7 to 1.6 m of fill in excess of the required embankment height was placed in stages. Instrumentations including settlement plates, piezometers and inclinometers were installed for monitoring the soil improvement work.

Effective stress stability analysis was carried out to control the rate of filling by considering the height of fill, height of side berms and magnitude of excess pore water pressure. A minimum factor of safety of 1.3 was considered as acceptable for stability. The results are summarized in Fig. 11 (MAAS, 1982).

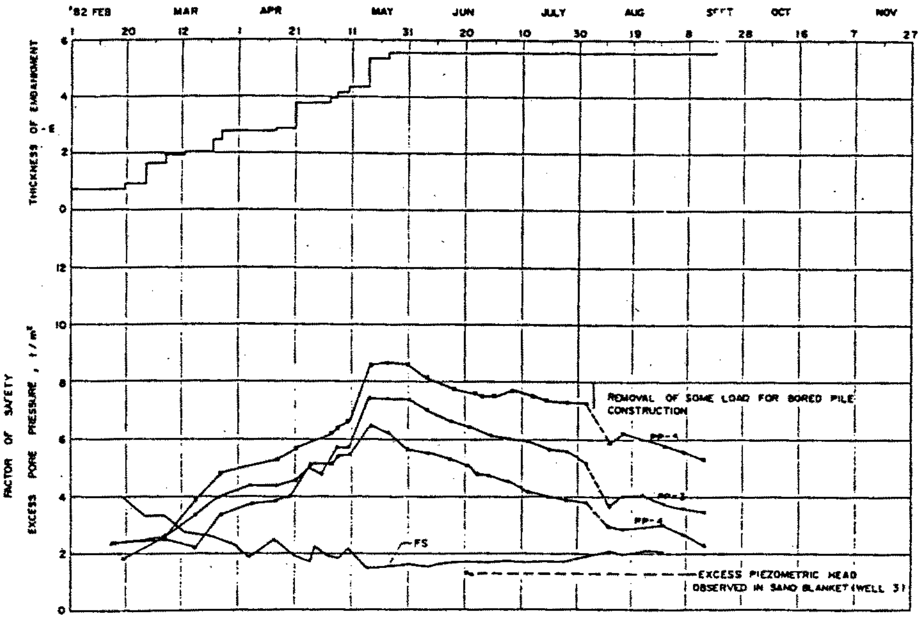

Fig. 11 Variation of Excess Pore Pressure and Factor of Safety with Embankment Loading and Time (North Embankment CH.1680 - CH. 1710) - Case Record 4

Figure 12 shows the measured settlement as compare with predicted values. There were five components which contributed to the total measured settlement. They were: (1) primary consolidation of the soft clay, (2) immediate settlement of the sand below the clay, (3) shear deformation of the soft clay, (4) secondary compression of the soft clay, and (5) consolidation of the initial working platform backfill. Evaluation of the settlement records indicated that the actual values of both the coefficient of volume compressibility m_v and coefficient of horizontal consolidation c_h were larger than those values determined from laboratory test results. The actual c_h was more than twice the value derived from lab test. This was a significent factor in determining the rate of settlement and demonstrate the effectiveness of the vertical drains. Significant contribution to vertical settlement was observed due to shear deformation of the soft clay, Fig. 12. This component of the total settlement occurred shortly after the placement of fill and ceased to increase soon after completion of fill.

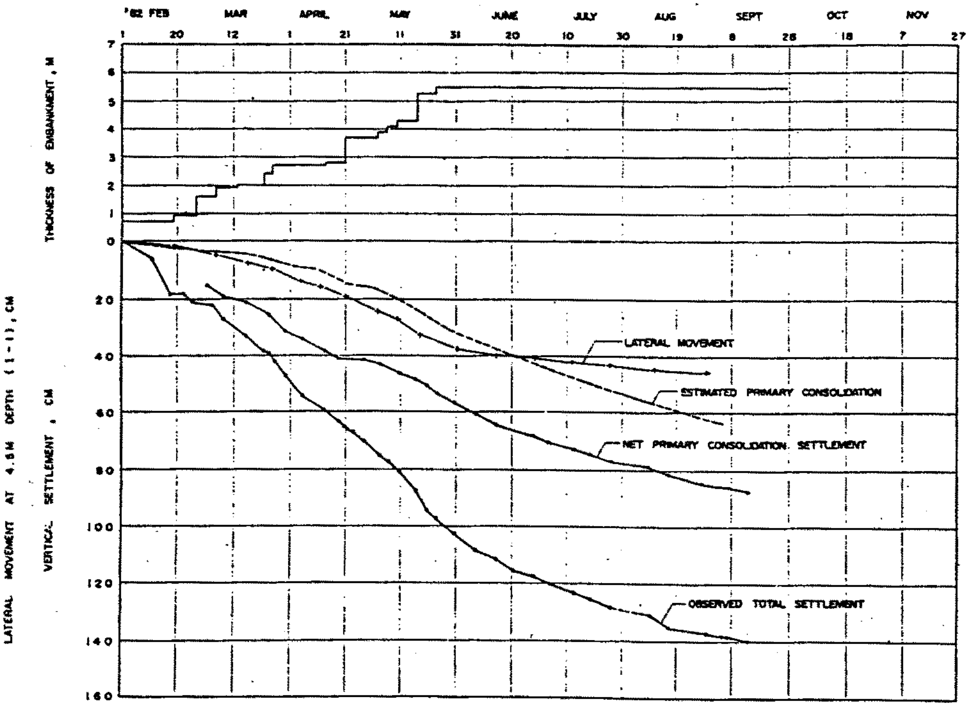

Fig. 12 Settlement and Lateral Movement at North Embankment (CH.1680 - CH. 1710) - Case Record 4

SUGGESTIONS FOR DISCUSSION

The above report described four case records using four different types of soil improvement methods. The problems at each site were different and the subsoil conditions were different. The reasons for choosing a particular method at a particular site were briefly discussed and the effectiveness of the soil improvements was described. However, in the evaluation and interpretation of results, a number of discrepancies were noticed. From case histories described in the literature as well as in the case records presented above, the following five topics are suggested for discussion.

A. Methods for Reliable Assessment of Critical Design Parameters, k_h, c_h, etc.

B. Methods for Assessment or Evaluation of Results of Soil Improvement.

C. Displacement Method vs Non-displacement Method for Installation of Vertical Drains.

D. Use of Dynamic Consolidation on Cohesive Soils - Effect of Surcharge and Drainage, Mixing of Granular Fills, Lateral Squeezing Pore Pressure Dissipation.

E. Soil Improvement Using Vertical Drains - Rate of Consolidation Estimated from Vertical Settlement Not Compatible with That from Pore Pressure Dissipation.

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