SOIL IMPROVEMENT FOR FOUNDATION TREATMENT IN TAIWAN

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A Special Lecture Delivered at the
Eighth Southeast Asian Geotechnical Conference

Reprinted from
Proceedings, Eighth Southeast Asian
Geotechnical Conference, Kuala Lumpur
11-15 March, 1985, Vol. 2
Southeast Asian Geotechnical Society
The Institute of Engineers, Malaysia

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SUMMARY

Soil Improvement techniques have been widely used to improve or to change properties of soil deposits for the purpose of strength increase, settlement control, seepage control, and reducing liquefaction potential under seismic loadings. A major part of the usuable land on the island of Taiwan consists of alluvial deposits underlain by thick layers of soft silts and clays and loose sands. These types of soils often require treatment or improvement before they can be utilized to support constructed facilities. Otherwise, very costly foundation systems may be necessary. This paper presents a general review of the soil improvement techniques employed in Taiwan. Two case records are reported, one uses prefabricated drains with preloading and the other case only utilized the conventional preloading method.

INTRODUCTION

The island of Taiwan lies about 150 km off the east coast of China mainland. It is separated from the mainland by the Taiwan Strait which has an average width of 100 km. The island is spindle-shaped, with the longitudinal axis extending roughly north-south for a length of 385 km. The maximum width is about 143 km and the total area of the island is approximately 35,960 sq.km.

Taiwan can be broadly divided into three major geologic provinces as shown in Fig. 1. These are the Central Range, including all the Tertiary sub-metamorphic and the Pre-Tertiary metamorphic complex, the Coastal Range of Neogene sediments, and the Western Foothill province composed of Neogene clastic sediments. The topography, which has been greatly influenced by geologic conditions, can also be divided into several parts. The Central Range strikes roughly parallel to the longitudinal axis of Taiwan and forms the backbone ridge of the island. It divides the island into two unequal parts, the western flank being about twice as wide as the eastern flank. The western flank declined from the Central Range westward into strips of foothills and then into broad tablelands and terraces. A wide extent of coastal plains and subsidence basins were developed on the southwest part of this foothill region. The coastal plain has a north-south length of about 240 km, and a maximum width of 45 km.

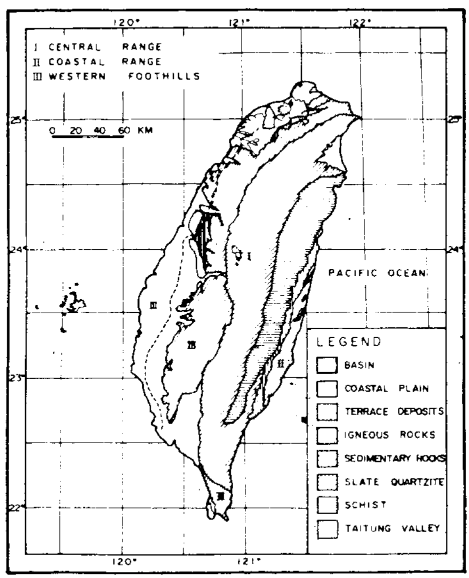

Fig. 1 : Geological Map of Taiwan

The total land area, which has elevation less than 100 m above the mean sea level, covers about 31.3% of the entire area of the island. Most of these lands are in the alluvial plains or basins, which are underlain by soft clays and loose to medium dense sands of recent deposits. More than 17 millions of people are crowded in this area which is less than one third of the whole island. In addition, Taiwan is located in the active seismic zone of Pacific Ocean and has experienced over thousand earthquakes and tremors every year. For developments in areas with these types of subsoil formations the major geotechnical problems are bearing capacity, settlement, and liquefaction. The engineers are usually faced with the choice of either adopting a very expensive and uneconomical foundations system or utilizing some techniques to improve or change the soil characteristics. The latter approach is commonly referred to as soil improvement or ground improvement. There are hundreds of methods, many of them are patented processes, available on the market today for improving properties of soils and rocks. Some of them are well developed and have sound theoretical basis, but many of them, on the other hand, are very empirical in nature. MITCHELL and KATTI (1981) have presented a well documented State-of-the-Art report on the subject of soil improvment at the Tenth International Conference on Soil Mechanics and Foundation Engineering in Stockholm. The writer in his Moderator's Report (MOH, 1982) at the Seventh Southeast Asian Geotechnical Conference has presented a general summary of the soil improvement methods which are currently in use.

In Taiwan, many soil improvement techniques have been employed in connection with design and construction of major development projects, such as oil storage farms, power plants, large industrial plants, freeway, railroad, sewage systems, etc. The most commonly used and successful soil improvement techniques and some of the more notable and large scale projects in Taiwan include:

(i) Preloading - Yung Kong Oil Terminal, Raw Water Storage Tank (CHOU et al, 1980).

(ii) Compaction sand piles for reducing liquefaction potential - Ta-Lin Thermal Power Plant, Hsin-Ta Fossil Power Plant (MOH et al, 1981), China Ship Building, etc.

(iii) Compaction Sand Piles with Preloading - LPG Tanks (MOH et al, 1982), China Steel Stacker - Reclaimer Area, etc.

(iv) Sand Drains with Preloading - Hsin Ta Power Plant Fuel Storage Tanks (MOH, 1982), China Steel Raw Materials Storage Yard (TSAI et al, 1981), Keelung-Naihu Section of North-South Freeway, etc.

(v) Prefabricated Drains with Preloading - Keelung River Reclamation Project.

(vi) Grouting - Underground Sewage System for Taipei City (LIN and HWANG, 1984).

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 objectives 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 sands, gravel or admixture.

(5) Environmental factors.

(6) Local experience and practice.

(7) Time schedule.

(8) Cost.

Figure 2 shows a brief flow diagram for carrying out a soil improvement project. The following sections present two case records which illustrate the approaches, identification of the problem, selection of soil improvement technique and evaluation of the effectiveness. First case is on the use of Prefabricated drains with preloading, which was the first time in use in Taiwan. The second case employs the conventional preloading method.

SOIL IMPROVEMENT WITH PREFABRICATED DRAINS

General

The Keelung River Abandoned Channel (KRAC) is a section of the old Keelung River passing through Shih-Ling District and Shi-Tse District in the suburb of Taipei City. Due to its lowlying level, the area was frequently flooded. As a part of the flood control measures in 1965, the Taiwan Water Conservation Bureau has diverted the flow of the Keelung River by straightening the river channel by abandoning the winding section at the foot of the Chi-Tan Mountain as shown in Fig. 3.

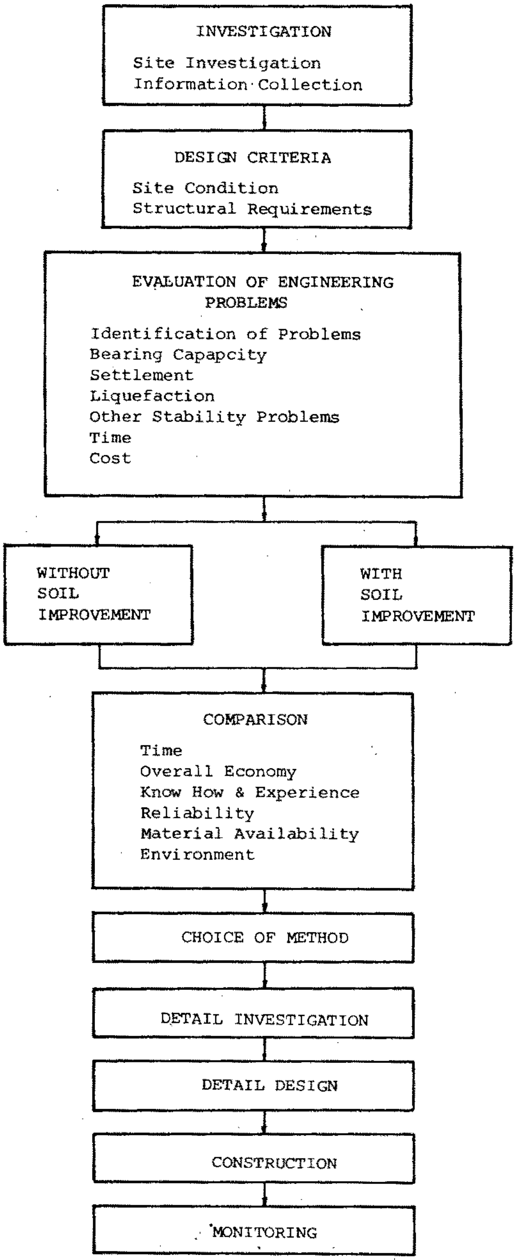

Fig. 2 : Flow Diagram for Soil Improvement Project

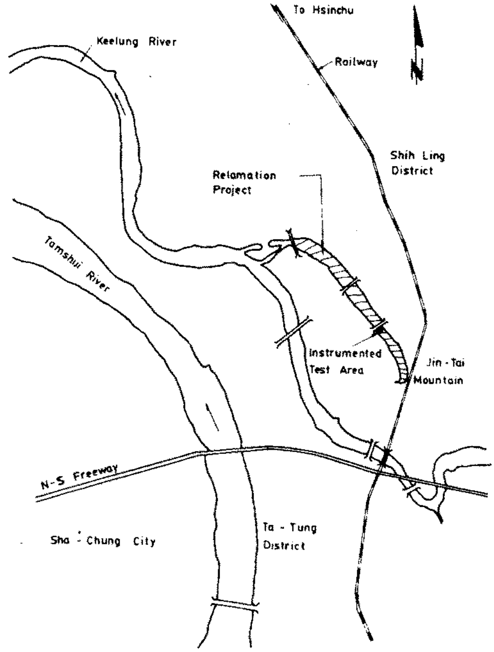

Fig. 3 : Site Location of the Keelung River Abandoned Channel Reclamation Project

The abandoned channel is about 200 m wide and 2 km long with a total area of over 30 hectares. During the past twenty years, part of the area became a dumping ground for construction wastes and household disposals and created serious environmental problems. For a number of years, the City Government was considering an effective way of reclaiming that area. In 1979, the Government decided to develop the area into a new town with public housing and associated facilities. Moh and Associates Inc. was engaged by the Public Works Department to carry out a feasibility study to evaluate the geotechnical problems which would associate with development of the area, in particular problems related to reclamation by filling of such a vast area. The study included site investigation, laboratory testing, methods of filling, material source evaluation, and design of temporary and permanent drainage facilities.

Subsoil Conditions

Similar to the subsoil profile of the Taipei Basin, the soil profile in the KRAC area can be divided into six layers with a typical profile shown in Fig. 4.

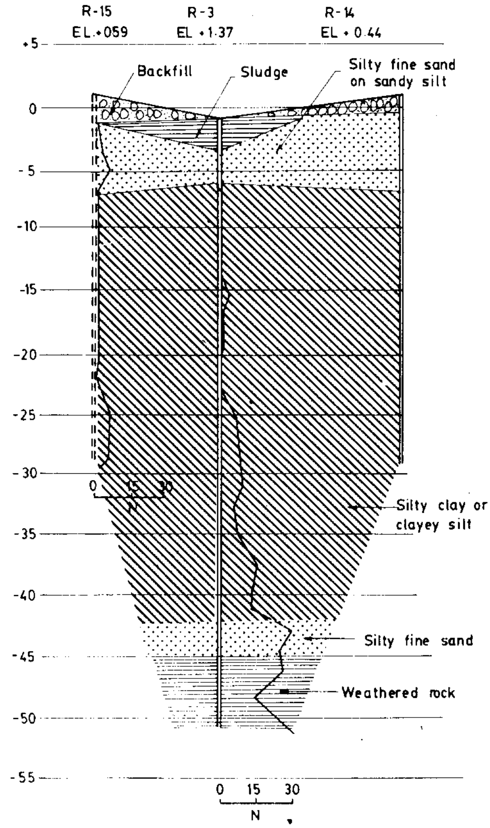

Fig. 4 : Typical Soil Profile at the Keelung River Abandoned Channel

1. Backfill

At the time of the site investigation, the two banks of the channel and a large portion of the area was covered with miscellaneous backfill consisted of construction wastes, concrete blocks, bricks, etc. The fill was rather loose and varied from 2 to 6.6 m in thickness. In areas where no fill was dumped, there was a layer of soft clay top soil of about 1 to 2 m in thickness.

2. Sludge

In the middle of the channel which was under water for the past 20 years, the river bottom was covered with a layer of sludge varying in consistency from semi-liquid to very soft and 1.5 to 4.0 m thick. The sludge was very soft, dark brown to black in color, and had moisture contents varying from 300 to 500 per cent. The solid content of the sludge was primarily of sand and silt sizes with very little clay size particles. Very little sludge was found underlying the backfills.

3. Silty Fine Sand

Immediately below the backfill or the sludge was a layer of silty fine sand varying from 2 to 10 m in thickness.

This sand was in a very loose condition underlying the sludge and became loose to medium dense at locations where backfill existed.

4. Silty Clay

Underlying the silty fine sand was a layer of soft silty clay with an average thickness of about 22 m. But in certain areas, this clay was as thick as 35 m. Due to its relatively large thickness, low strength and high compressibility, this soil stratum played the most important role in the feasibility of reclamation of the KRAC. A large number of field and laboratory tests was carried out on soil samples from this stratum to evaluate the properties of this soil layer. They are summarized as below:

(i) Moisture content - The natural moisture contents of the silty clay fell in the range of 30 to 50 per cent. In areas where there were backfills, the moisture contents tended to be lower than that in areas without backfilling. This indicated that some consolidation had occurred under the backfill load.

(ii) Shear strength - The $s_u/\bar{\sigma}_o$ ratio of this silty clay was only 0.12 to 0.20. The sensitivity value, varied from 4 to 9, with the highest value up to 15, indicated that the soil is quite sensitive.

(iii) Compressibility - The value of the compression index as determined from one dimensional consolidation tests varied from 0.2 to 0.4. The clay underlying the backfilled area was under-consolidated whilst those in the middle of the channel were slightly over-consolidated due to erosion of the overburden material caused by water flow.

(iv) Permeability - The silty clay was quite impermeable. The coefficient of permeability in the vertical direction was determined to be about 10^{-6} to 10^{-8} cm per sec and that in the horizontal direction was about 1 to 6 x 10^{-6} cm per sec. This higher permeability in the horizontal direction would play an important role in any soil improvement work.

5. Silty Fine Sand

Underlying the soft silty clay is a thin layer of silty fine sand varying from 2 to 9 m in thickness. The average SPT-N value was about 20 to 30. It varied from medium dense to very dense with N values sometimes larger than 50.

6. Weathered Rock

At El-35 to El-40, weathered rock or rock formation was encountered. The rock formation varied from sandstone to shale and the degree of weathering also varied considerable over the entire area.

The subsurface piezometric pressure distribution in the KRAC area was not static as shown in Fig. 5. This is a common phenomenon in Taipei Basin due to the deep well pumping (MOH and OU, 1979).

solid constituents are sandy and silty size particles. Reclamation of the area would be carried out by hydraulic fill method pumping sands from the nearby Keelung River and Tamshui River. With the high water content of the hydraulic fill (water to sand ratio about 6 to 1), a large part of the sludge would flow away with the water. The remaining sludge would mix with the sand. The laboratory consolidation test results of mixtures of sand and sludge of different properties, as illustrated in Fig. 6 show that consolidation of the mixtures completed within very short period of time under a small overburden pressure. Since the thickness of the sludge over the site was relatively small, even some differential settlements which might occur were not considered as a major problem.

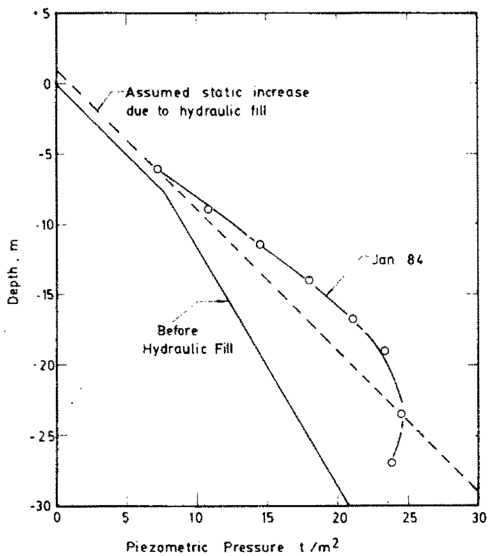

Fig. 5 : Piezometric Pressure Distribution at KRAC

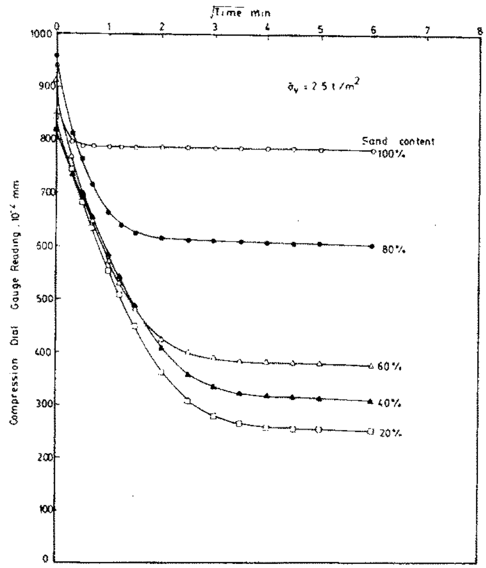

Fig. 6 : Consolidation Behavior of Sand-Sludge Mixtures

Soil Improvement Study

In order to reclaim the KRAC area for a new town development, about 2 to 4 m of fill will have to be placed over majority of the area. Two major problems which must be solved for the reclamation work were the presence of the sludge and the thick layer of low strength, high compressibilty silty clay layer. Several methods were considered and evaluated for treatment of the sludge including complete replacement by excavation, dewatering by drying, use of geotextiles and chemical stabilization. Results of laboratory tests indicate that the sludge contained only 8 to 17 per cent organic matter and the major

The soft silty clay stratum presented three problems. They were:

(i) average thickness of the clay was about 22 m with maximum up to 35 m,

(ii) the soil was extremely soft with SPT-N value varying from 0 to 2, and

(iii) the soil would undergo very large settlement under the proposed reclaimed fill load and the settlement would take a long time to complete.

In order to be able to develop this area as soon as possible, it was necessary to improve the characteristics of this soil layer by accelerating the rate of consolidation under load and increasing the strength. The use of sand drains and lime columns at this site was considered to be limited due to the deep depth of improvement required. Prefabricated band drains were considered to be more preferable for accelerating the consolidation rate of the thick soft silty clay underlying the site.

Due to variation of the thickness of the soft silty clay layer, the entire site was subdivided into three zones. Analyses were carried out to evaluate the rates of settlement under the following conditions:

(i) without any soil improvement,

(ii) using sand drains of 45 cm diameter, 15 m long at 2 m spacing, and

(iii) using 10 cm wide prefabricated drains, 30 m long at 1 m and 3 m spacings.

Figure 7 presents the estimated time-settlement curves for the average soil conditions in Zone II with different treatment methods. The average thickness of the silty clay in this zone is 32 m and the fill is 5 m. Table 1 summarizes the time required to reach different amount of residual settlements for the different conditions.

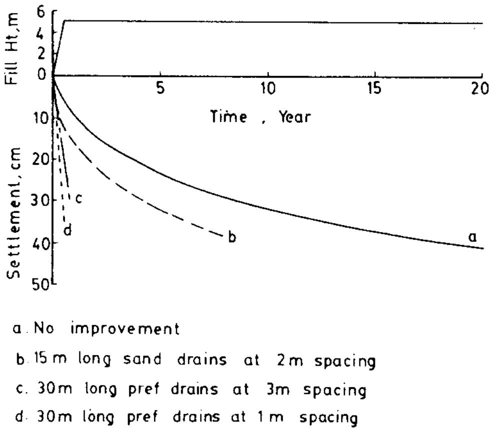

Fig. 7 : Estimated Time-Settlement Curves in Middle of Channel of Zone II

Design and Construction of Soil Improvement

The basic criterion for the design of the soil improvement scheme was that the average residual settlemnt under the proposed fill of 2 to 4 m at the time when the reclaimed land is delivered to the Government, (i.e. up to El + 3 m) should not be more than 10 cm 6 months after completion of installation of the drains. On this basis, prefabricated drains at 1.5 m and 2.0 m spacing were adopted. According to the new town development plan, approximately 5.2 per cent of the reclaimed land will be used for parks and 0.4 per cent for parking. In these areas, less stringent requirements for residual settlement could be tolerated.

There are many types and makes of prefabricated drains available on the market, such as Alidrain and Geodrain from Sweden, Colbond from the Netherlands, Bando drains and PVC drains from Japan. Each make has its special characteristics and each claims that they are the best. Since the total quantity of drain materials involved in the project was very large, in excess of 1.3 million linear meters and since it is necessary to drive the drains to a depth of 30 m, strict specifications and prequalification requirements were prepared. One of the emphasis was placed on performance records of the drains in deep soft soils. Some of the drains were reported to behave unsatisfactorily under pressure due to collapse of the filter layer or the corrugation of the core. Laboratory tests were carried out on a number of different types of drains. The final specifications for material's requirements are listed in Table 2. The selection of the drain material was made by open tender of prequalified drain makes. The Ret-Ser Engineering Agency is the main contractor responsible for the installation, and Moh and Assocaites serves as a consultant on the project for design and construction monitoring.

In the construction operation, the hydraulic fill was placed first to the desired elevation with a small surcharge. Prefabricated drains were then installed. The excess fill was then removed to fill in adjacent grounds after placement of drainage trenches.

Instrumentation Monitoring and Performance

In order to evaluate the effectiveness of the prefabricated drains for accelerating the consolidation settlement of the subsoils under the fill load, an area in Zone II was selected for observation and control. Two different drain spacings, i.e. 1.25 m and 1.5 m, were

Table 1 Estimated Time Required For Various Residual Settlement (Zone II)

Fill Height 5.0 m		Time Required, years			
		No Soil Improvement	15 m long Sand Drains at 2 m spacing	30 m long Pref. Drains at 3 m spacing	30 m long Pref. Drains at 1 m spacing
Residual Settlement	10 cm	12.5	6.5	0.96	0.41
	15 cm	8.5	4.2	0.72	-
	20 cm	5.9	2.7	0.59	-

Table 2 Properties of Different Batches of Prefabricated Drains

Properties	Specifciation Requirements	Batch				
		B2	B8	B10	B13	B17
Tensile Strength, kg/10 cm width	>70					
Elongation at 70 kg/10 cm, %	<15					
Permeability of Filter Layer cm/sec	1×10^{-3} to 2.5×10^{-5}	2.9×10^{-5}	3.1×10^{-5}	5.8×10^{-5}	13.4×10^{-5}	14.1×10^{-5}
Permeability of Drains, cm/sec Longitudinal	$>10^{-1}$ under 3 kg/cm^2 Pressure	13.6	7.8	8.2	6.1	12.2
Permeability of Drains in Soil Mass	$>10^{-2}$ under 3 kg/cm^2 Pressure	20.4	6.6	6.7	5.4	-
Drain Size, cm						
thickness	-	0.320	0.272	0.305	0.293	0.316
width	8.12	9.55	9.60	9.60	9.60	9.60
filter thickness	-	0.019	0.019	0.018	0.021	0.021
filter width	-	12.5	14.34	14.56	15.18	13.28

used. Instrumentations including surface settlement plates, deep settlement points, and pneumatic and hydraulic piezometers were installed as shown in Fig. 8. The pneumatic piezometer is a closed system and gives instantaneous response of changes in the pore pressure values. At the instrumentation control area, additional borings, samplings, Dutch cone penetration soundings were carried out immediately after the completion of the fill operation. Similar tests will be performed at the end of the six months period after installation of all the drains to evaluate the efficiency and performance of the drains.

In the instrumented area, placement of the fill was started in early November 1983 and reached the design elevation plus surcharge in about 45 days. The fill with surcharge was left to consolidate for about 8.5 months before installation of the drains. Figure 9 illustrates the settlement and pore pressure monitoring data obtained to date.

From the figure, it can be observed that the installation of prefabricated drains has greatly accelerated the rate of consolidation of the underlying soils. Within a four months time, the degree of consolidation increased from about 20 per cent which took almost 8.5 months to 60 - 80 per cent. More detailed analyses of the results will be presented after the completion of the project.

In order to have a strict control of the quality of the drain material, the specifications required that a full series of laboratory tests be carried out on random samples taken for every 100,000 linear meters of the drain material to be used. Table 2 shows the results of 5 of the batches. The properties appeared to have a wide variation although they all met the specification requirements. This points out the importance on quality control of the drain material.

SOIL IMPROVEMENT BY PRELOADING

General

A tank terminal was constructed on a flood plain in the southwestern part of Taiwan for storage of oils. The site was originally occupied by sugarcane and tomato farms. The total area of

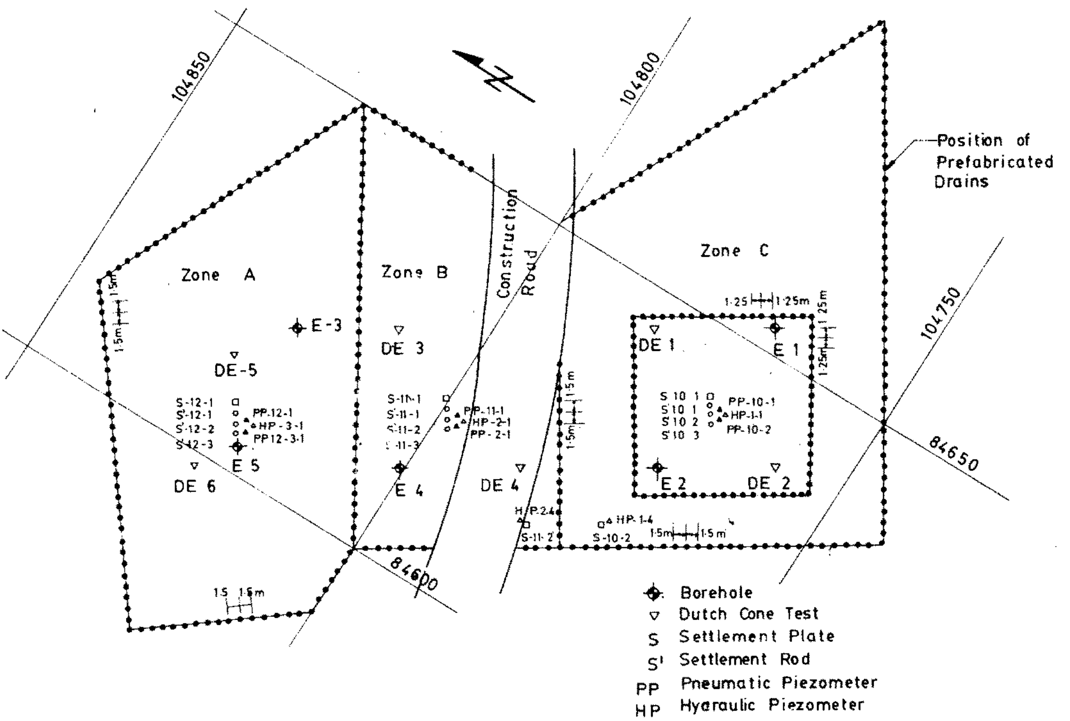

Fig. 8 : Location Plan for Instrumentations

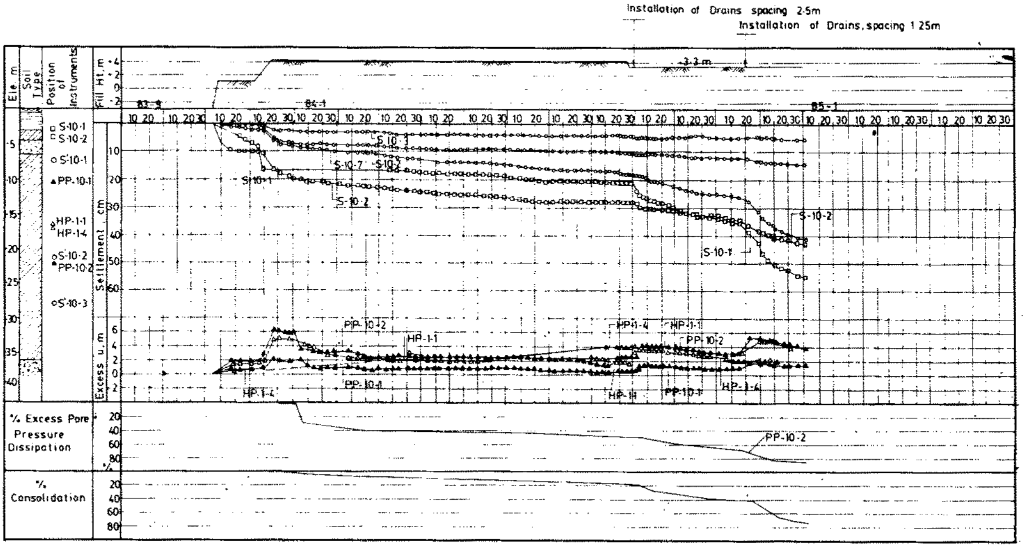

Fig. 9 : Settlement and Pore Pressure Records in Test Area C

the site is about 12 hectares. The tank farm consists of 8 storage tanks and other auxiliary facilities. Seven of the tanks are 61 m in diameter, 18.3 m high with a storage capacity of 50,000 kl each and one tank is smaller in size, 36.6 m in diameter and 12.2 m high with 10,000 kl capacity. The layout of the tanks is shown in Fig. 10.

Subsoil Conditions

The subsoils at the site primarily consisted of alternative 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. Below that depth, the silty fine sand layer became much denser as revealed by the SPT-N values. The subsoils down to a depth of 30 m can be approximately divided into 5 sublayers as shown in Fig. 11. Also shown in the figure are the SPT-N values, natural moisture contents, total unit weights and average strength properties of the various sublayers.

Goetechnical Analysis

In designing foundation for oil storage tanks, the main geotechnical problems are:

(i) bearing capacity of the subsoils,

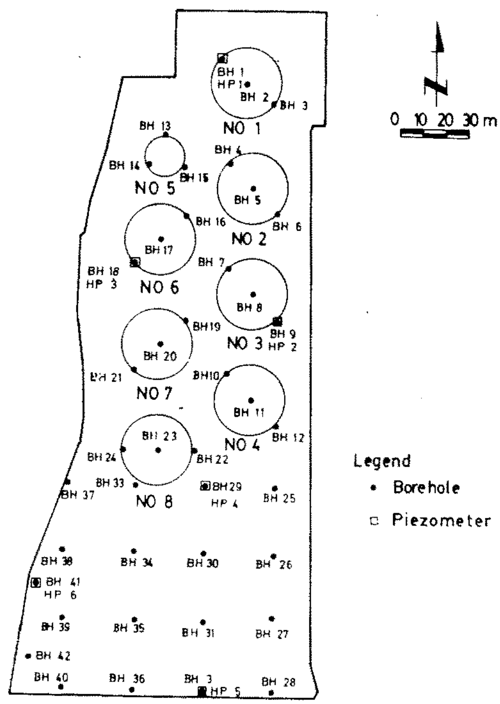

Fig. 10 : Site Plan of Yung Kong Oil Tank Terminal

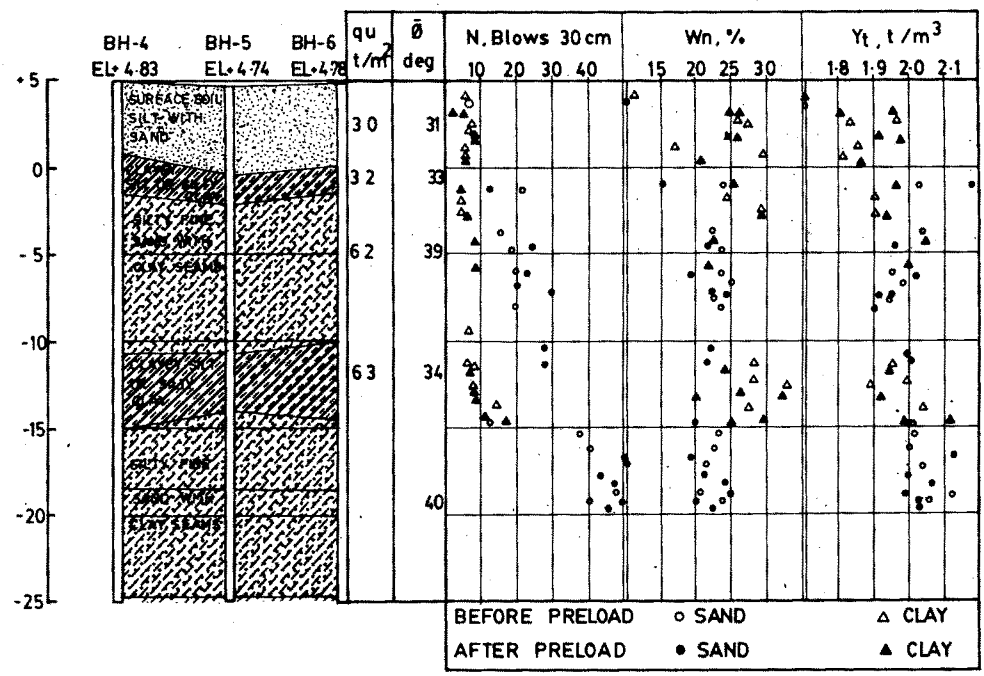

Fig. 11 : Simplified Soil Profile and Soil Properties Before and After Preloading — Yung Kong Oil Tank Terminal Site

(ii) liquefaction potential, and

(iii) foundation settlement.

For bearing capacity analyses, the potential of local failure, general shear failure and plastic flow must be considered. Results of analyses of the proposed tanks, (MAA, Inc. 1982; YU et al 1984) showed that the factor of safety against local failure in the second layer of clayey silt soil (from top) under some of the tanks was close to 1.0 and the factor of safety against plastic flow was less than 2.0. These results clearly indicated that the top 23 m of the subsoils were 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 of the clayey silt and the surface soil layers, and high compressibility of the two clayey silt or clay strata. For a 50,000 kl capacity oil storage tank, 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 (DE BEER, 1969; GREENWOOD, 1974). 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 existence 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. It was estimated that 90 per cent of the consolidation settlement would occur only two months after the preloading.

Preloading and Its Effects

One of the feature which is worthy to mention is the economical and effective utilization of the preloading material. In view of material availability, loss of material due to handling (i.e. loading and unloading) and the need for good base material for tank foundations as well as roads in the terminal area, well graded crushed stone was selected as the preloading, a proper sequence of loading and unloading was planned so that only the necessary amount of the crushed stone required for the final construction with minimum wastage was procured and transported to the site. 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. For the larger tanks, the 20.6 tons per sq m preloading required a circular truncated cone of crushed stones with base diameter of 80.2 m, top diameter of 61.0 m and height of 29.6 m.

A simple instrumentation program was designed to monitor the effectiveness of the preloading. Instrumentations installed included settlement plates and pneumatic piezometers in the center of the clay layers. Figures 12 and 13 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 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. The differences between the predicted settlements and the observed values were in the order of 10 per cent.

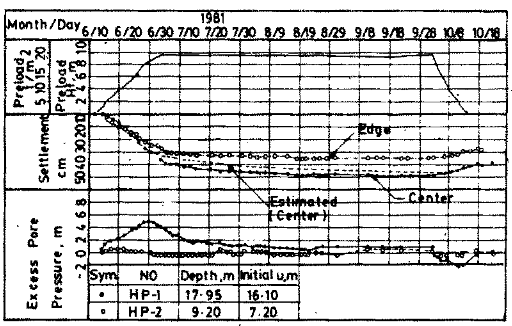

Fig. 12: Settlement and Pore Pressure Variations During Preload for Tank No. 1

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 the sand and clay as illustrated in Fig. 11. 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 compressibilty of the clay layers decreased by two-third to three-fourth.

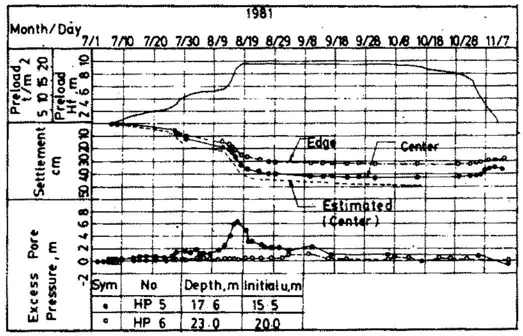

Fig. 13: Settlement and Pore Pressure Variations During Preload for Tank No. 3

Water Test

It is a common practice for oil storage tanks to have a water load test before the oil is actually stored. Besides checking the integrity of the tank structure for possible cracks and leakage, water test could also be considered as a form of preloading which would reduce the residual settlement of the tank under actual fuel load. CHOU et al (1980) has reported a case using water testing as preloading for a 65,000 tons raw water tank in Taiwan. At the present site, water tests were carried out after the tanks were constructed. Water was pumped into the tank at a rate of 0.7 tons per sq m per day up to a maximum load of 18 tons per sq m. The full load of water was left in the tank for a period of 30 days. A total of 12 settlement observation points were installed around the tank shell for monitoring the settlement of the tank. Figure 14 shows the settlement record for Tanks No. 1 and No. 3. Results of theoretical analyses indicated that the estimated residual settlements of the large tanks under the water test load were 11 to 14 cm in the center and 6 to 8 cm around the edges. These values were very close to the observed ones as illustrated in the figure.

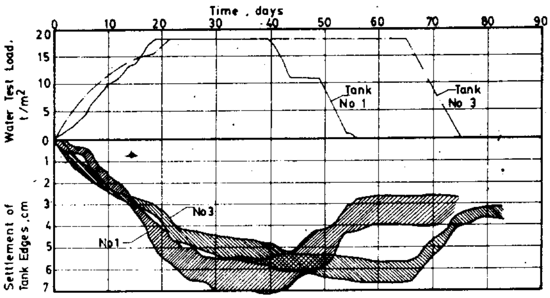

Fig. 14 : Settlement of Tank Shell During Water Test

ACKNOWLEDGEMENTS

The writer wishes to acknowledge the assistance given by the following organizations and individuals who have helped in collecting information for this paper or rendered assistance during the execution of the projects: Mr Benjamin Chi Chief Engineer, and engineers of Ret-Ser Engineering Agency at the site of the Keelung River Reclamation Project; Director and Deputy Director of the Engineering Maintanence Bureau of the Taipei Public Works Department; Chief Engineer, Deputy Chief Engineer, Project Engineer of the China Petroleum Corporation and Engineers of the BES Engineering Corp. at the Yung Kong Oil Terminal Project site. Special thanks are due to the writer's colleagues, particularly Mr K. Yu and Dr C.D. Ou.

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