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Ground Improvement for Oil Tank Farm in Indonesia

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SYNOPSIS This case study describes the use of ground improvement to treat a highly variable site, where new oil storage tanks were constructed. Varied techniques were used comprising a combination of dynamic compaction, preload, vertical drains and replacement. Settlement data from the storage tanks during water test shows the treatment to have been successful.

INTRODUCTION

This paper presents the result of an extensive programme of ground improvement at a site in East Kalimantan, Indonesia. Pertamina, the state owned oil company retained Bechtel to design and construct a major expansion (of capacity 200,000 Barrels per day) to the existing refinery at Balikpapan on the east coast of Kalimantan. The project comprised extensive new process units, 23 new storage tanks, power and cooling water supplies and a new oil loading jetty. Total cost of the construction is in the region of $1500 million. The site had been already partially developed, during the war and there was little freedom in siting of the new plant. This necessitated construction within close proximity to existing facilities. The layout of the main tank farm is shown in Figure 1.

Dames & Moore were retained in 1980 to provide a geotechnical consultancy and accordingly a detailed soils investigation was undertaken. This comprised 268 borings and 125 Dutch cone penetration tests (DCPT) as well as in-situ vane tests, test pits, and geophysical surveys, of the above investigations 91 borings and 47 Dutch cone penetration tests were performed in the tank farm areas.

The investigation revealed highly variable conditions across the site ranging from good foundation conditions medium dense sands to deep soft normally consolidated marine clays, commonly 30 metres deep. Variability was such that even with closely spaced borings, interpolation was difficult. Figures 2, 3 and 4 illustrate three typical profiles from three different zones of the site.

Analysis of the data revealed that ground supported tanks would undergo extreme total and differential settlement, and in the worst areas of the site have inadequate factors of safety against foundation failure.

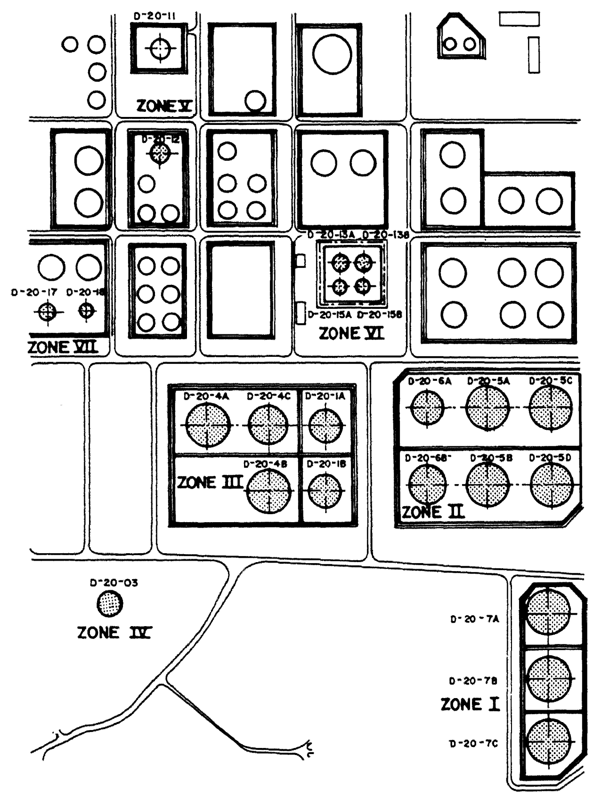


FIGURE 1. SITE LAYOUT - TANK FARM
(NEW TANKS SHADED)

Estimates of edge settlement for just one tank typically ranged from 250 to 1200 millimetres, depending upon which boring or CPT profile was assumed, which clearly demonstrated the extreme variability. It was concluded that piling or ground improvement would be required.

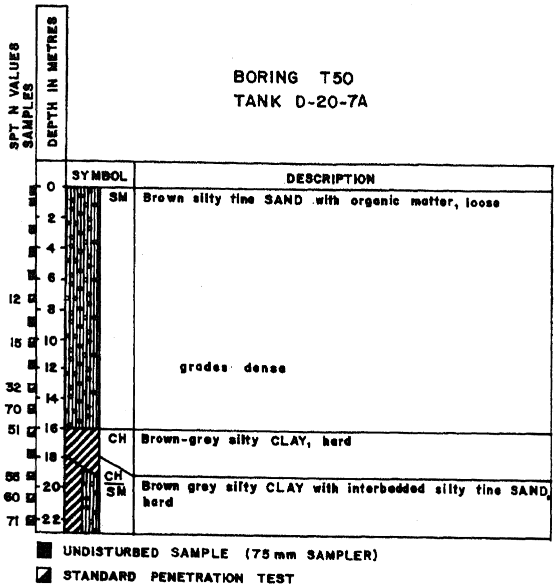


FIGURE 2. LOG OF BORING - ZONE I

The piling option whilst technically certain, had disadvantages. Obviously the cost would be very high and also severe logistic problems were forseen because 10,000 piles were already planned for the process units alone, and a large increase of this quantity was practically very difficult. Accordingly various forms of ground improvement were considered including for example:

- preloading with vertical drains;
- vibro compaction;
- dynamic compaction;
- excavation and replacement of very weak near surface deposits;

Specialist contractors were invited to propose methods of treatment and the succesful contractor Balfour Beatty Sakti Indonesia, through its subsidiary Stent-Solcompact joint venture proposed a unique solution where preload, vertical drains and dynamic compaction would be combined as necessary to suit the prevailing conditions. In predominantly sandy areas (Zones I and VI) conventional dynamic compaction was proposed. In areas of deep soft day, (Zones IV, V and VII) extensive preloading with vertical drains was selected, and in mixed areas (Zones II and III) where 11 of the tanks were located, both preloading and dynamic compaction were proposed. The following paragraphs describe these in more detail.

DESIGN AND EXECUTION OF GROUND IMPROVEMENT

The design of the treatment was conceived at tender stage by the contractor. As far as possible all the known but variable aspects of the project were accounted for including:

- Geotechnical conditions described in site investigation reports;
- Structural characteristics of the various tanks and pipe lines;
- Interaction between actual improvement works and existing structures;
- Time available for the project and required settlement target specifications.

Three principal types of treatment, with two minor variations were chosen, as described in the following paragraphs. Cognon, Liausu and Vialard (1983) give further details as to the methods employed.

Dynamic Compaction (Zone I, Zone VI)

In sandy types of soil, where calculations and experience indicated that improvement need only extend to a depth of 10 to 12 meters, Dynamic compaction was selected as the most economical method to achieve the required specifications. Two levels of compaction energy was selected to match variations in soil thickness and nature :

- high energy treatment using a total energy of 350 to 400 TM/m .
- standard energy treatment using an average energy of 120 to 160 TM/m .

The high energy dynamic compaction was used in area of greater thickness of loose soils to be treated (Zone I). The other treatment was used in Zone VI.

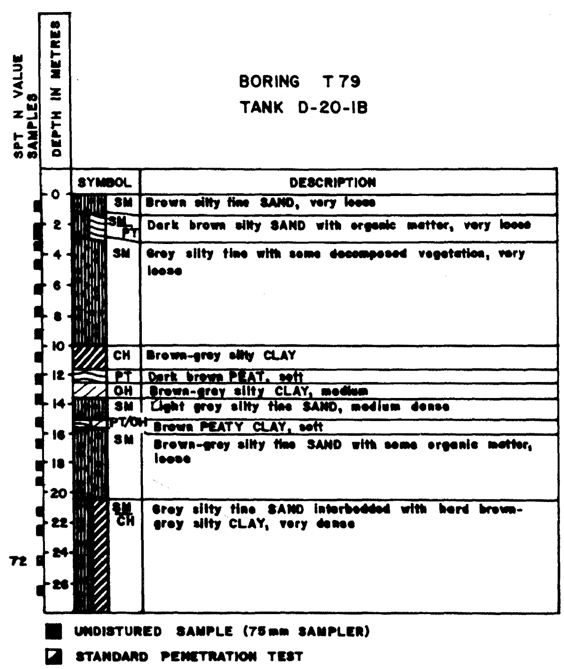


FIGURE 3. LOG OF BORING - ZONE III

Vertical Drains and Preload (Zone IV, V, VII)

This treatment was designed to improve the thick compressible clayey soils (up to about 30 metres deep) using the stressing of embankment preload and the drainage from a suitably designed network of vertical drains to accelerate the consolidation of subsoils. In order to meet the various project requirements an achievement of 85 to 95% consolidation was planned, within the treatment period. This led to a pattern of vertical drains on grids ranging from 1.2 to 2.0 metres square, using conventional drain design methods, Barron (1948) and Hansbo (1979). It was decided to use a cylindrical drain 50 millimetres in diameter of length up to 28 metres. This type of drain was preferred as it offered negligible clogging of the channel (due to BIDM fabric around the PVC pipe) with a high discharge capacity. Also the inside corrugated core allowed high strains without altering its drainage properties.

Construction of the preload was performed in stages in order to ensure safety against slip failures. Preloads up to 11 metres high were used with a longest preload period of 8 months.

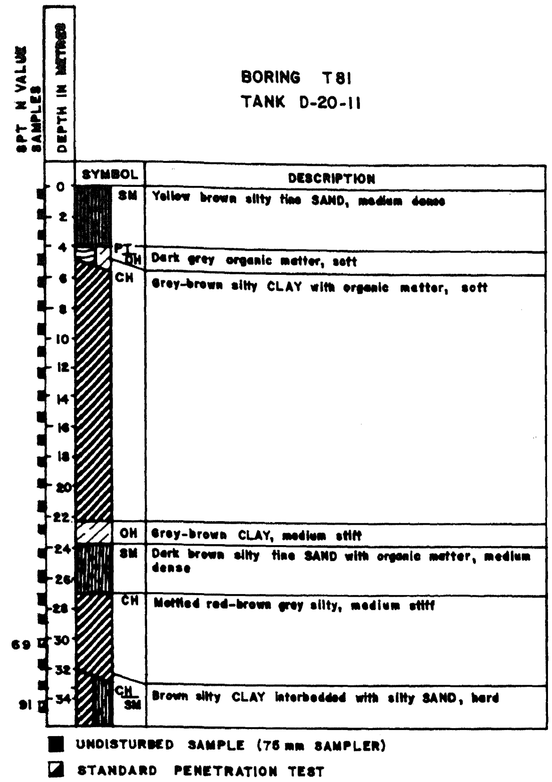


FIGURE 4. LOG OF BORING - ZONE V

Combined Methods (Zone II, III)

In these zones, complex interlayered soils were encountered, with highly variable and mixed layers of sand, and clay. Consequently, it was planned to use both described methods combined.

Preloading was necessary to enforce the consolidation of deep seated silty clay layers and the drains were necessary to accelerate the consolidation process

In addition dynamic compaction was necessary to densify the upper loose sandy layers not only to reduce the settlements but also to improve the stress distribution at depth (raft effect).

The drain grid selected was 2.0 x 2.0 metres associated with preload embankments 5.0 to 9.0 metres high, depending upon the prevailing conditions at each particular tank.

SUPPLEMENTARY TECHNIQUES

In addition to the above main treatment, various supplementay techniques were adopted in certain areas. These are briefly described below.

Vibroflotation

This method was selected for areas where dynamic compaction could not be used, (due to a vibration sensitive environment). A vibrating poker 4 metres long was lowered into the loose sand deposits and sand backfill was pushed into the resulting depression. This technique was used primarily for tank retention dykes and pipetracks where a risk from earthquake induced liquefaction of upper loose sands was considered significant. No detailed results of this treatment are presented in this paper. However significant increases in soil properties were achieved by this simple technique.

Soil Replacement

Some highly organic soils and peat were considered as "non improvable" because of their nature. Due to their location related to the future foundations (generally only the upper four metres) they were designated for excavation and replacement by more suitable soil. After this replacement operation, standard dynamic compaction would be used with greater efficiency to improve sub soil characteristics. In addition some soil replacement was performed to remove old foundations and buried debris from the tank sites.

Site Operations and Quality Control

Before starting the ground improvement works, an intensive detailed site investigation programme was undertaken to confirm the suitability of the above described treatment zoning. During the course of the work soil data were used to modify as necessary the ground improvement parameters, energy, drain grid and depth, preload height; etc or to take some additional measures such as horizontal drainage to cope with the flow of ground water out of the vertical drains. At all times, the ground improvement subcontractor worked closely with the main contractor (Bechtel) and the project geotechnical consultant (Dames & Moore). In total the ground improvement contractor conducted 50 borings, 20 pressuremeter borings and 200 Dutch cone penetration tests as part of the testing programme. According to the type of treatment used a specific quality control programme was undertaken as summerised below. Further details are available in Cognon Liasu and Vialard (1983).

Dynamic Compaction Control Testing

Quality control of the dynamic compaction included:

- o Pounder penetration tests and heave tests: to define the optimum number of blows per phase and per print;
- o Print volume survey;
- o Pore water pressure monitoring;
- o Overall enforced settlement phase by phase;
- o Soil improvement monitoring using pressurementer test, Dutch cone penetrometer tests and standard penetration tests;

Drains and Preload

The quality control programme was designed to deal with three aspects of this treatment

(i) monitoring of consolidation using:

- o piezometers (generally six hydraulic piezometers per tank)
- o settlement plates (two to four per tank)
- o settlement gauges (two to four per tank)
- o survey of preload embankment level

(ii) monitoring of soil characteristics improvement both during and after preloading using:

- o vane tests
- o pressuremeter
- o Dutch cone penetration tests

(iii) control and monitoring of embankment stability using:

- o extensometers
- o computer analysis of preload embankment stability (Conventional Bishop method was used)
- o continuous monitoring of settlement data with review by the Asaoka procedure.(1978)

DISCUSSION OF PERFORMANCE OF GROUND IMPROVEMENT

As discussed in an earlier section there were three basic types of ground improvement. We present below examples of one typical tank from each category of treatment. We emphasise that these three tanks are typical and only three are presented in this paper to keep it to a reasonable length.

DYNAMIC COMPACTION AS PRIMARY TREATMENT (TANK D-20-7A)

Tank D-20-7A was investigated in the site investigation stage (June 1980 to January 1981) by three borings. The subcontractor performed a supplementary investigation of three DCPT and two borings after contract award. A typical boring is shown in figure 2.

Based on the results of investigations (and a subsurface radar survey which showed numerous buried metal objects) it was decided to excavate and backfill the upper 2.5 to 4.0 metres to remove shallow peat deposits and buried objects. This was performed in two phases in the period September to March 1982.

Dynamic consolidation was then performed, comprising a total of 10 passes plus one ironing pass on a 10 metre square grid pattern. Average energy used was 400 tonne metre per square metre, and an enforced settlement of 370 millimetres was measured. Control testing at the site comprised pressure meter, tests, DCPT and borings with SPT. Figure 5 illustrates the improvement achieved by the treatment.

COMBINED PRELOAD AND DYNAMIC COMPACTION (TANK D-20-1B)

Tank D-20-1B was investigated by one boring and one DCPT at the initial site investigation stage. A log of the boring is shown on figure 3 which shows the variably layered sequence of the strata in this part of the site. The ground improvement subcontractor performed the supplemental investigation in August 1981 which comprised one boring and four DCPTs. This additional data indicated the need for the combined design with a preload 9.0 metres height and 16 metres long prefabricated drains on a 2.0 metre rectangular grid.

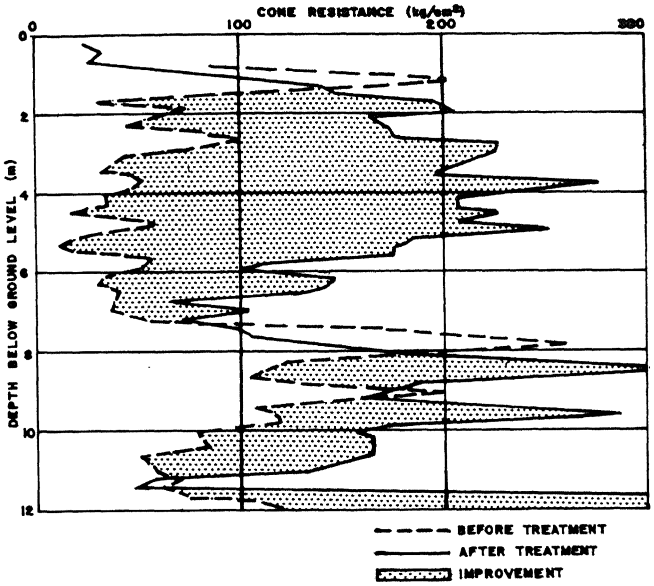


FIGURE 5. COMPARISON OF DCPT RESULTS
ZONE I – TANK D-20-7A

As in the case of the tank previously described there were numerous buried objects under the tank foundation. Consequently it was decided to excavate and replace the upper two metres across the tank. This was performed after preloading and before the Dynamic compaction.

The vertical drains were installed in February 1982 and at that time the instrumentation was also installed. This comprised two settlement gauges, four settlement plates and two piezometers. The preload was built in three stages to a height of 9.0 metres during the period March to July 1982 and a settlement of 660 to 700 millimetres was recorded by the instrumentation as shown on Figure 6. The settlement data indicated that consolidation of the clay layers was in excess of 90 percent at the end of the preload period. Following removal of the preload, the tank area was excavated and backfilled and four DCPTs were performed as intermediate tests.

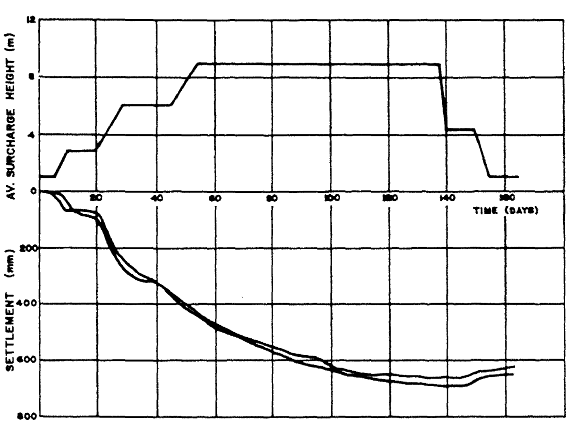


FIGURE 6. SETTLEMENT OF PRELOAD
TANK D-20-1B

Dynamic compaction was then performed in September 1982 comprising five passes plus one ironing pass. Post treatment testing comprised four DCPTs and four pressuremeter borings. Figure 7 shows the improvement obtained by the preload stage and Figure 8 shows further improvement during the subsequent dynamic compaction phase of the treatment.

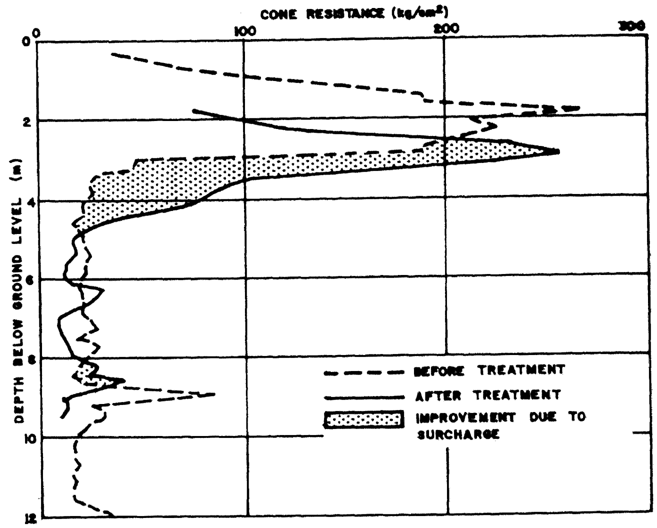


FIGURE 7. COMPARISON OF DCPT RESULTS
ZONE III – TANK D-20-1B

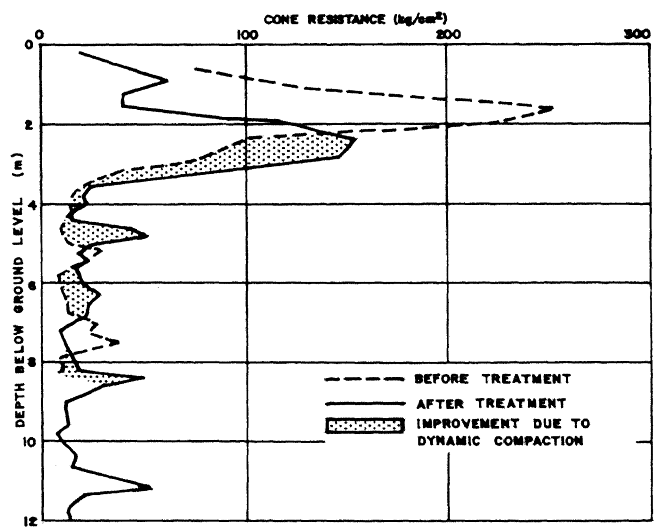


FIGURE 8. COMPARISON OF DCPT RESULTS
ZONE III – TANK D-20-1B

PRELOAD ONLY SOLUTION (TANK D-20-11)

The original site investigation performed comprised two borings. A log of one boring is shown on figure 4. This was supplemented by additional investigations in September 1981 comprising one further boring, vane tests, and three DCPT's. The investigations showed that very deep soft clays were present with shear strengths ranging from 15 to 30 KN/m^2.

Consolidation test data showed very low values of the coefficient of consolidation (Cv) and consequently a vertical drain grid spacing of 1.2 metres was selected, with drain lengths of 26 metres. A preload height of 11.0 metres was chosen in order to overconsolidate the soft clays. The vertical drains were installed in March 1982. Instrumentation comprising two settlement plates, four settlement gauges, four piezometers and two extensometers were installed in April 1982. The preload was constructed in seven stages during the period May to December 1982 to a final height of 11.0 metres. At the final height the settlement gauges registered settlements ranging from 2.13 to 2.62 metres, as shown on Figure 9. Some of the pore pressure data obtained during the preloading period is presented on Figure 10. Post treatment testing was performed in January 1983 which comprised three DCPT's and three pressuremeter borings. Figure 11 shows a comparison of before and after DCPT results, showing a small but measurable improvement in cone resistance. Vane test data taken during the preload period also showed improvement.

Based upon the actual preload height and the estimated loading during the water test, it was desirable that the soils achieved at least 80 per cent consolidation during the preload period.

Although large settlements occurred during the preload, this did not necessarily indicate that the soils have achieved the required degree of consolidation. A portion of the observed preload settlement was believed to be due to undrained settlement and lateral yielding of the soft clay. In particular a very large and rapid settlement of the preload occurred when the second lift of the preload was added. This relatively large settlement of 550 millimetres in few days occurred with only minor dissipation of pore water pressures. This observed settlement as incompatible with the measured dissipation of pore water pressure if purely consolidation settlement was occurring and was presumed therefore to be caused by undrained settlement.

Inspection of the time settlement records by the Asaoka construction indicated that the degree of consolidation achieved during preloading was around 80%. However, in this situation this method did not provide an accurate assessment of the degree of consolidation owing to the sensitivity of the plotting procedures required.

In order to make an accurate assessment of the degree of consolidation the response of soil pore water pressure by means of piezometers was studied. The assessment of these data indicated that when the preload was removed the soils had achieved approximately a 65 to 70 per cent degree of consolidation.

Back analysis of the pore pressure data indicates that the soft clay had a horizontal coefficient of consolidation of about 0.4 m^2/year, an exceptionally low value, but one which was in agreement with the laboratory test rests.

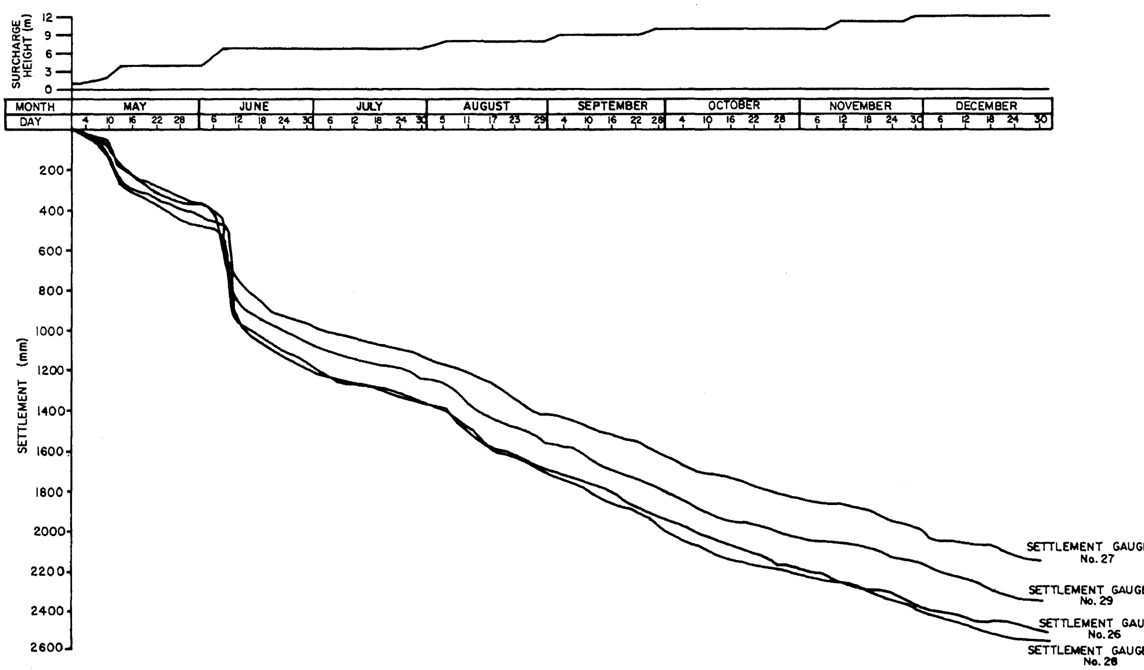


FIGURE 9. SETTLEMENT OF PRELOAD TANK D-20-11

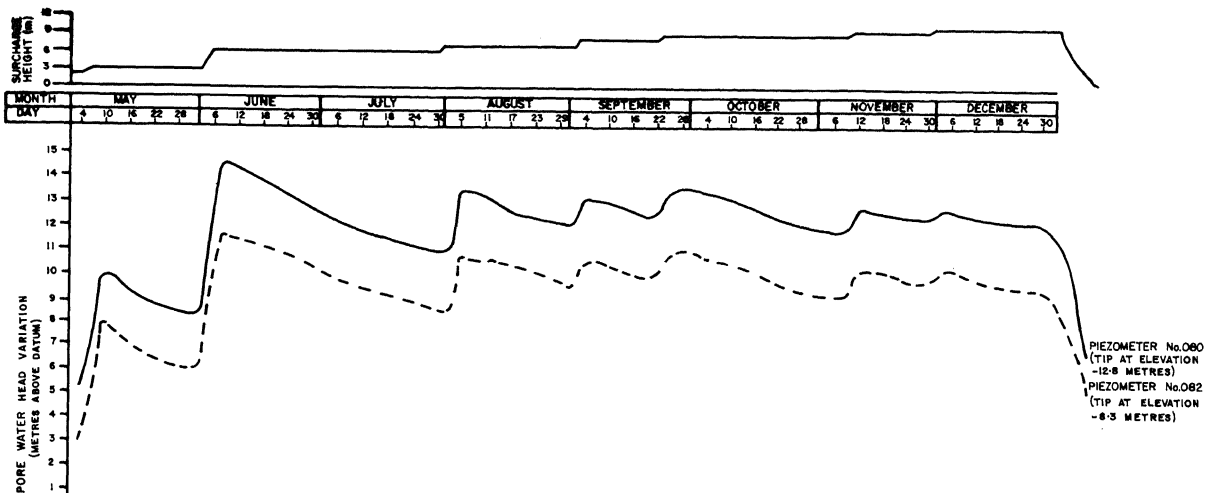


FIGURE 10. TYPICAL PORE PRESSURE DATA TANK D-20-11

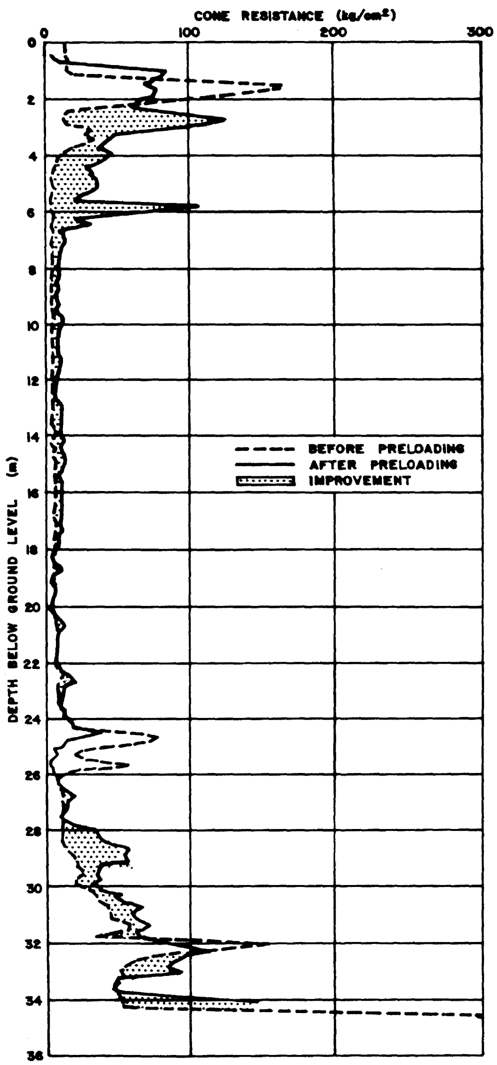


FIGURE 11. COMPARISON OF DCPT RESULTS ZONE V – TANK D-20-11

TANK SETTLEMENTS

Following the ground improvement discussed in the previous section, all tanks were subjected to carefuly monitored full height water test. Depending upon the performance during the ground improvement phase of work, the duration of the water test was varied, with the slowest water test being for the tanks on deep soft clay.

Water test settlements for all tanks were broadly in line with expectations as illustrated in the three tanks used as example in this paper. Figure 12 presents a summary of all water test settlement data at the time of preparing this paper. Settlements were monitored by surveying eight locations around the tank shell, and also across two orthogonal diameters by hydraulic settlement profiling tubes (Soil Instruments Ltd type).

TANK D-20-7A

Original estimates for tank D-20-7A (16 metres high, 50 metres diameter) without any treatment were as follows:

Centre : 400 mm

Edge : 200 mm to 250 mm

Following the dynamic comaction treatment, further estimates of settlement during the water test were made using the post treatment insitu test data. The results of those calculations were as follows:

From Dutch cone data using Meyerhof (1965)

Centre : 130 to 270 mm

Edge : 80 to 160 mm

From Pressuremetre data using Menard (1965)

Centre : 170 to 260 mm

Edge : 100 to 160 mm

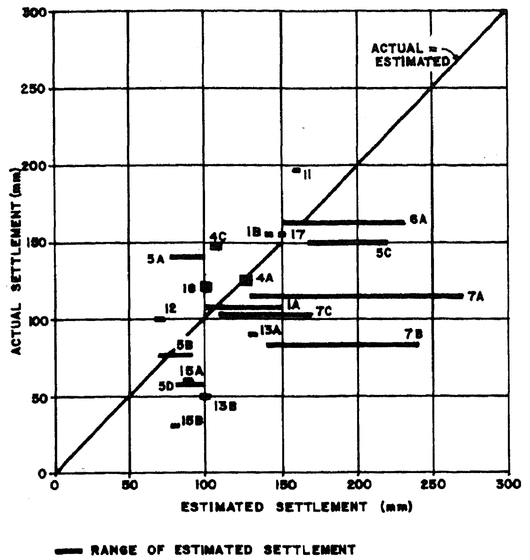


FIGURE 12. TANK CENTRE SETTLEMENT DURING WATER TEST AFTER GROUND IMPROVEMENT (DATA TO HAND NOV. 1983)

These settlements may be compared to the actual settlements which were measured during the water test as follows:

Centre : 115 mm

Edge : 58 to 81 mm

These data were measured during the water test to 16 metres height after a period of 18 days. Measurements showed that during the last few days at full height there was negligible further settlement. The actual settlements recorded approximate to the lower end of the range predicted using the Dutch cone data. This observation was broadly repeated for all the seven tanks treated by dynamic compaction alone. (see figure 12) Experience elsewhere has also indicated that predictions of settlement after dynamic compaction are often too high. This is believed to be due to the overconsolidation effect which the treatment has on granular materials, particularly in the upper layers whereas the methods of settlement estimation using Dutch cone are based on what are usually normally consolidated sand deposits.

Further study of the settlement data showed that the tank performance was well within its allowable tolerances and tha the inservice settlements due to average loading would be very small.

TANK D-20-1B

Original estimates for tank D-20-1B (15 metres high, 41 metres diameter) without any treatment were as follows:

Centre : 1750 mm

Edge : 800 to 1600 mm

Following the preload and dynamic consolidation treatment further estimates of settlement for the water test were made. These calculations were based partly upon the post treatment insitu test data but mainly upon the actual preload performance. The results of those calculations were as follows.

Centre : 140 to 160 mm

Edge : 80 to 100 mm

These estimates may be compared to the actual settlements which were measured during the water test as follows:

Centre : 155 mm

Edge : 46 to 69 mm

As may be expected the estimates of settlement for the 11 tanks with combined preload and dynamic compaction were generally more accurate than the previous example of dynamic compaction alone. Tank 1B is typical of the results obtained for this group of tanks.(see figure 12) It is interesting to note that the ratio of edge to centre settlement was almost invariably less (usually less than 0.4) for these tanks than the DC only tanks. (usually about 0.6). This is believed to be due to the deeper compressible layer causing the settlement.

Further study of the settlement data showed that the tanks in this group were well within within their allowable settlement tolerances and that in service settlement would be very small.

TANK D-20-11

Original estimates for tank D-20-11 (13 metres high, 27 metres diameter) without any treatment were as follows, assuming the tank could be loaded slowly enough to present failure.

Centre : 1500 mm

Edge : 1000 to 1100 mm

(Based on the performance of the preload it would appear that even these figures were optimistic).

Following the preload programme and the review of pore pressure data discussed in the previous section, estimates were made for the settlement of tank 11 during water test. It was estimated that the recompression and further virgin consolidation settlement expected to occur during the four week water test would be:

Centre : 160 to 270 mm

Edge : 100 to 160 mm

Moreover, the possibility of further irreversible undrained lateral yielding of the soils was considered in light of the experience gained during the preload programme. Assuming instantaneous tank loading to full height it was calculated using Foott and Ladd (1981) that this effect could account for as much as a further 200 mm at the tank edge.

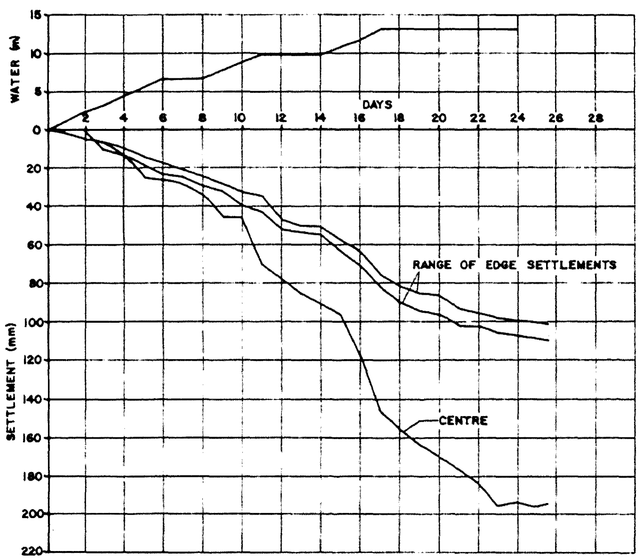
Accordingly the water test was planned to allow a gradual loading of the tank with intermittant holding periods to observe settlement trends.
Measured settlement during the water test was as follows:

Centre : 195 mm

Edge : 101 to 109 mm

The water test settlement curve is shown in fig. 13.

The data indicated that there was no significant undrained yielding of the soils and the results compare well to expected values for consolidation. The settlement data showed that the tank was within its settlement tolerances and calculations of further settlement during the life of the tank under average product loading indicated in only small further settlements. Consequently the tank was put into service.



TANK D-20-11
FIGURE 13. SETTLEMENT DURING WATER TEST

CONCLUSIONS

This case study demonstrates quite clearly that even with highly variable and unfavourable soil conditions, ground improvement can be effectively used to treat large foundations. A novel combination of various types of ground improvement have successfully been combined together to produce a cost effective foundation design which was accomplished within the overall project schedule. These results presented in this paper are an example of the results for the project. Figure 12 shows tank water test settlements compared to estimated values for those tanks tested at the time of preparing this paper. All tanks were subject to one of the forms of ground improvement discussed. Overall capital savings in foundation construction costs of several million dollars have been effected with negligible increased risk to the user.

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