Effects of Mandrel-Driven Sand Drains on Soft Clay

Effet du Drain en Sable Enfoncé par Mandrin sur Terrain Mauvais

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SYNCPSIS A field test was conducted to investigate effects of installation of displacement type sand drains on properties and behavior of soft clay. Driving of a closed-end mandrel causes pore pressures to rise and displaces foundation clay upwards and laterally, reducing shear strength and coefficients of consolidation considerably. Rapid and significant consolidation, however, takes place during and after sand drain installation and prior to the fill placement. While the conventional design method appears to give a reasonable value of primary settlement which occurs under the fill placed subsequently, it fails to give a satisfactory time-settlement relationship in the sand drained area as well as in the adjacent similarly loaded untreated area.

INTRODUCTION

Based on analyses of some case histories, it was pointed out by Akagi (1977a & b) that significant changes could result in shear strength and compressibility of soft clay stabilized by displacement type sand drains before fill placement. Effects of installation on properties and behavior of soft clays have not been elucidated yet.
A field test was therefore conducted driving 25 closed-end mandrel-driven sand drains, very closely spaced to intensify the effects, into a soft clay foundation which was heavily instrumented and was later sampled periodically. The test site is located on AIT campus, 40 km north of Bangkok, Thailand. The subsoil consists of 2 m thick surface crust of fairly stiff clay underlain by a 5 m thick layer of soft clay with seams of silt and fine sand, occasional decayed organics and cracks. This soft clay, at a depth of about 7 m, grades into soft sandy silt and loose silty sand which in turn are underlain by stiff clay at about 9 m depth. The soft clay between the depths of 2 and 7 m is highly plastic and slightly overconsolidated, having undrained shear strength of 10 to 30 kPa with sensitivity ranging from 3 to 8. The liquid limit is 95 and the plastic limit 30 with the average natural water content of 77%. The groundwater table was located about 1 m below the ground level.

INSTRUMENTATION AND INSTALLATION OF SAND DRAINS

Fig. 1 shows the locations of 44 control stakes for measurement of heave and lateral movements of ground surface in the western half of the test site, 18 piezometers at depths of 3, 6 and 8 m at 6 points A to F, and 3 inclinometer casings, I-1 to I-3. The piezometer is of a closed system, consisting of a porous stone tip which can be pushed into soft clay and is led to a manometer on the ground. The pore pressure was measured by adjusting a null indicator manually.
Sand drains were installed on 1.2 m spacings in square patterns and always from the center toward the outside in the order of their numbers, Nos. 1 to 25, Fig. 1. A steel pipe, 300 mm and the wall thickness of 7 mm, with an expendable bottom disc at the tip, was driven to a depth of 8 m by means of a free-falling 2.5 ton hammer. While driving was easy, a considerable difficulty was encountered to install sand columns. After the mandrel was driven to 8 m, a steel rod was lowered to the bottom, the bottom few meters of the mandrel was filled with sand and the mandrel was withdrawn 0.3 m with the rod being held stationary to ensure that the bottom disc was detached. Then the rod was replaced by a steel disc attached to another rod which was placed on the top of sand to prevent it from coming up together with the mandrel. In spite of the full weight of the hammer exerted on the disc, the sand was pushed up with the mandrel by powerful upward movements of the surrounding clay which had earlier been displaced by penetration of the mandrel. Consequently, the completed sand columns averaged only 5.70 m in length, although the mandrel was always driven to the depth of 8 m.

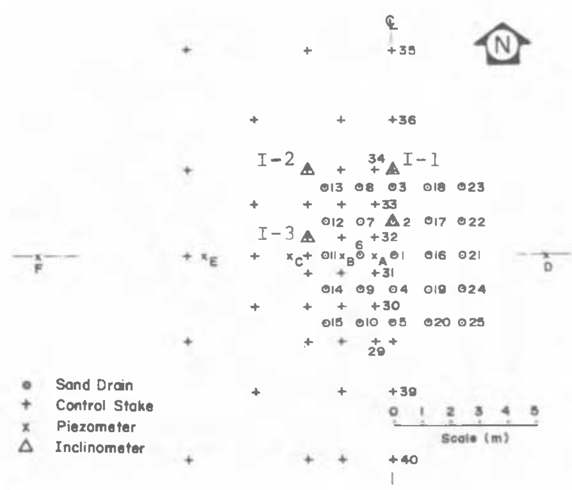


Fig. 1 Plan of Test Site

PORE PRESSURES DURING AND AFTER INSTALLATION

Fig. 2 summarizes the excess pore pressures measured at point A(A-1, A-2 and A-3 at depths of 3, 6 and 8 m, respectively) and ground heaves observed near the N-S centerline(stakes Nos.29 to 34). Excess pore pressures measured by these piezometers continued to rise until the second row of sand drains, Nos.6 to 10, was completed. Except F series which indicated nominal changes, all the piezometers similarly detected rapid changes showing cummulative increase of pressures as sand drains were driven in their vicinity and rapid dissipation as the driving took place away from their locations.
Observed excess pore pressures at greater depths were always greater in magnitude and dissipated at greater rates. The maximum excess pore pressure, Δu, is considered as something indicative of the stress induced by driving and is found comparable in magnitude to the computed effective overburden pressure, p', e.g., as follows:

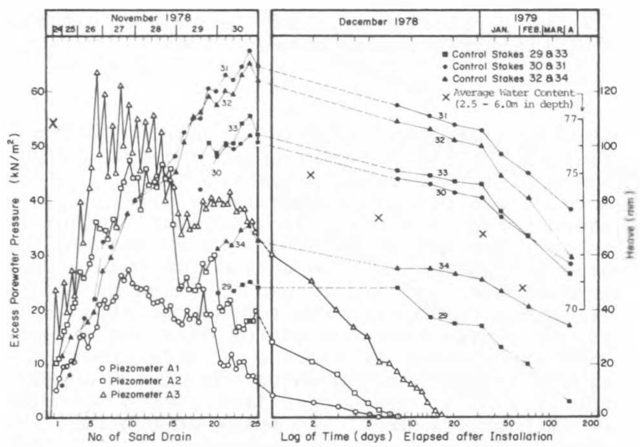


Fig.2 Excess Pore Pressures and Ground Heaves During and After Sand Drain Installation

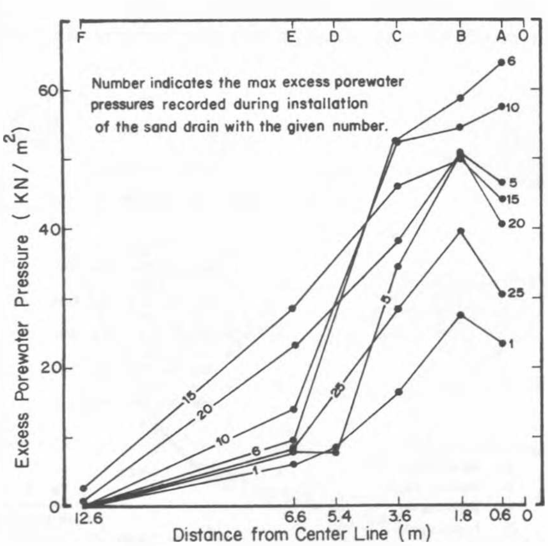


Fig.3 Excess Pore Pressures at 8.0 m Depth Along E-W Centerline

Piezometer	Depth(m)	p'(kPa)	Δu(kPa)	Δu/p'
A-1	3.0	29.8	26.3	0.88
A-2	6.0	45.8	47.5	1.04
A-3	8.0	58.4	63.8	1.09

Fig.3 shows excess pore pressures recorded by piezometers at a depth of 8 m immediately after the driving of sand drains whose numbers are indicated therein. Each curve generally shows a rapid decrease in pressures with the distance from the sand drain being driven. Unfortunately piezometer A-3 recorded lower values than B-3 in most cases. Manual adjustment of a null indicator apparently limited the accuracy of piezometer readings which changed very rapidly near the point of driving. The actual values of Δu were probably somewhat greater than those given in the above. Dissipation of excess pore pressures remaining after installation of all the sand drains was so rapid the pressure of 34.6 kPa at A-3 disappeared completely in 17 days, Fig. 2.

GROUND MOVEMENTS DURING AND AFTER INSTALLATION

As shown in Fig.2, the ground surface continued to rise during installation. Control stake No. 31 located 0.6 m from point A recorded a maximum of 135 mm after sand drain No.24 was installed. After all the sand drains were driven, heaved ground subsided steadily at much slower rates than those of pore pressure dissipation. Based on the records of 10 heave stakes near the N-S centerline, Fig.4 illustrates profiles of the ground surface during and after installation. In spite of the fact that no overload was applied, it is evident the once heaved ground settled slowly but consistently. The average water contents between 2.5 and 6.0 m in depth were determined at various times after installation and shown in Fig.2. The soft clay must have under-

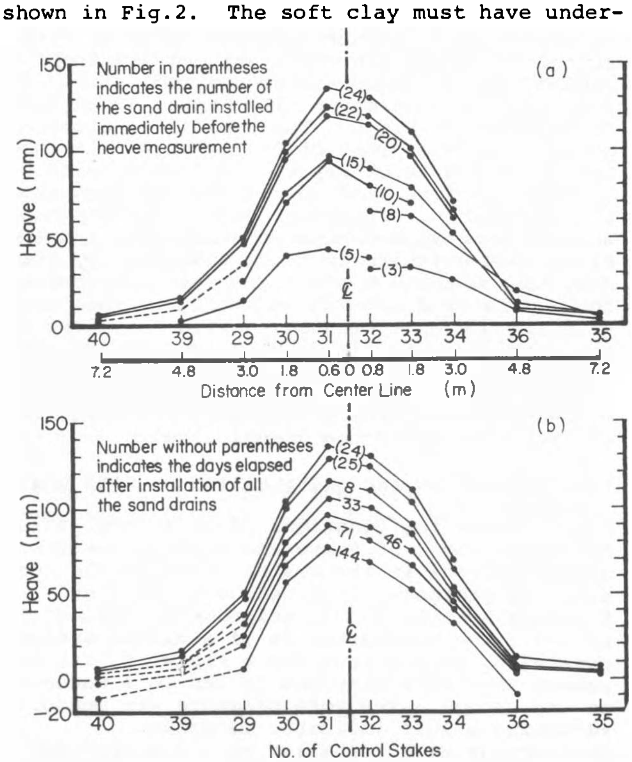


Fig.4 Heaves of Ground Surface Along N-S Centerline

gone significant consolidation as total stresses decreased and effective stresses increased. Lateral movements of heave stakes were observed and were found considerably smaller than heaves. Inclinometer data were taken periodically to determine lateral displacements with depth. Fig.5 gives two typical results (a) N-S components of movements of plastic casing I-1 and (b) E-W components of I-3. The foundation soil was displaced continuously during installation, but rebounded noticeably after installation.
Based on heaves and lateral movements observed in the western half of the site, it was concluded that when all the sand drains were driven, the total volume of the foundation soil displaced amounted roughly to that of sand poured, 11.0 m^3.

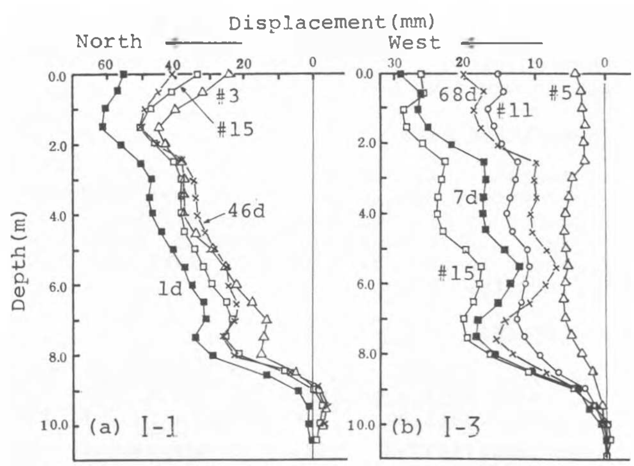
CHECK BORINGS AND LABORATORY TESTING

Check borings were made at the center of 4 sand drains in the corners of the sand drained area, 2 days, 1 week, 1 month and 2 months after installation. Undisturbed samples were taken and tested for strength and compressibility characteristics. As has been demonstrated earlier by Akagi (1977a & b), a unique relationship exists between water content and undrained shear strength (1/2 of unconfined compressive strength), and the manner in which the once-reduced strength of the soft clay increases as consolidation proceeds may be related to its compressibility, as shown in Fig.6. The data given are the average values obtained between 2.5 and 6.0 m in depth. The water content data are identical with those given in Fig.2.
While compressibility was relatively unaffected by disturbance due to driving, coefficients of consolidation were reduced considerably, Fig.7. Both the vertical and horizontal coefficients of consolidation, c_v and c_h, in Fig.7 were determined from standard oedometer tests on undisturbed samples taken prior to sand drain installation. Since c_v' and c_h' obtained from samples taken after installation fall within a fairly narrow range, Fig.7 shows the mean values of 3 determinations made on samples taken 2 days, 1 week and 1 month after installation. c_{vr} is the value obtained from a remolded sample. While all the results in Fig.7 were obtained from the depth of 4.5 m, the results from the depths of 2.5 and 7.0 m show a similar trend, i.e., c_v and c_h are roughly of the same magnitude and the effect of disturbance on the coefficients is considerable, apparently increasing with depth as shown below:

Depth (m)	Mean c_v'/c_v	Mean c_h'/c_h
2.5	0.58	0.63
4.5	0.38	0.36
7.0	0.24	0.12

SETTLEMENT DUE TO FILL PLACEMENT

About 8.5 months after sand drain installation, 2-m high embankments with 12x12 m base and 6x6 m top were constructed over the sand drained area (Area S) and an adjacent untreated area (Area N). Fig.8 shows the time vs. surface settlement data taken at the center of each area for the first 100 days, together with results computed by Terzaghi's and Barron's conventional procedures using oedometer test results. It appears apparent the settlement in Area S was not accelerated significantly enough to justify the sand drain installation in comparison with that observed in Area N. The difference between curves S and N after about Day 50 remained constant through the end of observation, Day 164, being approximately 40 mm. The heave remaining at the center of



Note: "#15" indicates lateral displacements measured immediately after sand drain No.15 was installed, and "46d" indicates displacements 46 days after all the sand drains were installed.

Fig.5 Lateral Displacements Measured by Inclinometers

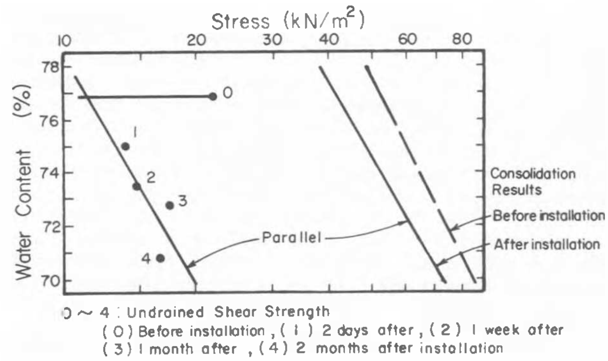


Fig.6 Water Content and Undrained Shear Strength Before and After Sand Drain Installation

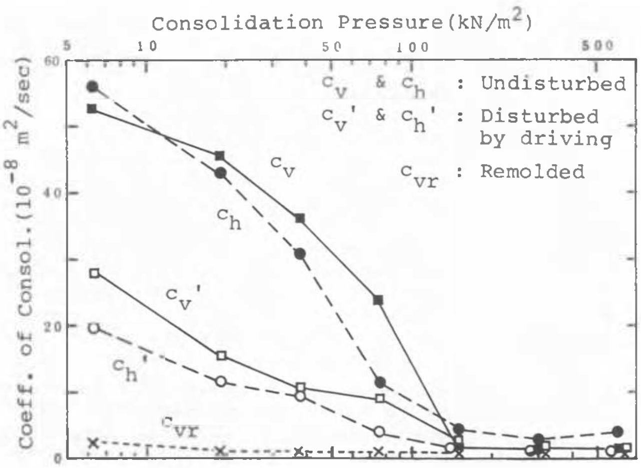


Fig.7 Effect of Disturbance on Coefficients of Consolidation at 4.5 m Depth

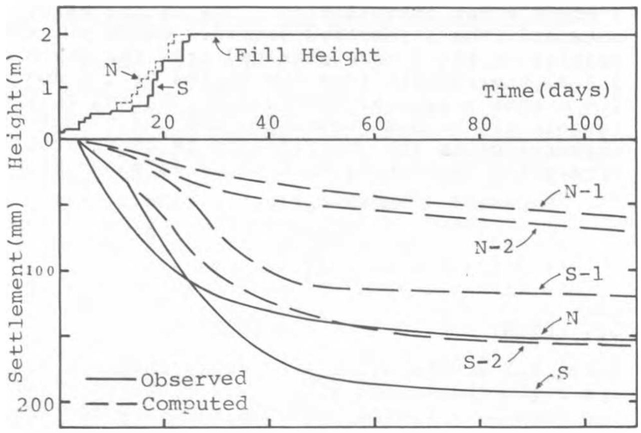


Fig.8 Time vs. Settlement Relations in Sand Drained Area(S) and Untreated Area(N)

Area S was estimated to be 54 mm when the filling operation started. Since curve S shows the settlement measured from the heaved surface, the net settlement in Area S was in fact always slightly less than that in Area N.
The relation between observed settlements and logarithm of time using the data given in Fig.8 suggests primary settlements of 146 and 188 mm in Areas N and S, respectively. Hence the net settlement in Area S was 188 - 54 = 134 mm.
The routine settlement analysis on the basis of oedometer test results obtained prior to sand drain installation gives a primary settlement of 130 mm under a 2-m embankment at the center of each area. The settlement analysis for Area S disregards sizable stresses developed during driving which must have altered the initial effective stresses considerably. Considering also such complexities as higher compressibility of the clay due to disturbance, stress concentration on sand columns and presence of a surface crust, it is rather surprising to find that the observed net settlement of 134 mm and the computed value of 130 mm are in good agreement.

As has often been noted(e.g., Akagi 1979), settlement took place much faster than had been predicted. The analysis assumes double drainage for 2 layers, 0-8 m and 8-12 m in depth, each having different consolidation characteristics based on oedometer test results. The disagreement in Area N in particular is appalling between curve N and curve N-1 which is the result of computation on the basis of Terzaghi's 1-dimensional theory. 3-dimensional consideration, curve N-2, does not help much as long as one assumes that $c_v \approx c_h$. If the 1-dimensional analysis is adhered to, it would require a c_v value almost 30 times as great as the laboratory value to obtain a time-settlement curve which would give a reasonable fit to the observed. It is highly likely that the horizontal permeability of a small element of clay tested in an oedometer is far less than that of the clay mass containing silt and sand seams, occasional decayed organics and cracks.

In Area S also, comparison shows poor agreement. If computation is based on the data available prior to construction as is normally done, curve S-1 may be obtained. If it is based on the data after sand drain installation, curve S-2 results, which however is not available under normal circumstances. In addition to the fact that both S-1 and S-2 ignore the heave existing and complex stress conditions at the start of filling operations, the coefficients of consolidation used were determined from laboratory data. The analysis for S-2 employed values which were 1/3 to 1/5 of those used in the analysis for S-1. Our present inability to determine vital parameters for prediction of the time-settlement relationship of an embankment on an untreated foundation seems to be a more serious problem than difficulties with estimating factors required to define complexities brought in by installation of mandrel-driven sand drains.

CONCLUSIONS

The results of this field test program are limited due to the unfortunate fact that a) only 25 sand drains were driven, b) lengths of sand drains attained were shorter than had been intended, c) a surface crust overlies the soft clay and d) the clay is slightly overconsolidated. Nevertheless, it may be concluded that:

1) When displacement type sand drains are installed on close spacings, stresses comparable to effective overburden pressures are induced in soft clay by mandrel driving, and are manifested by development of large excess pore pressures, ground heaves and lateral displacements and subsequent consolidation phenomena.

2) After installation excess pore pressure dissipates rapidly, while ground heave subsides and lateral displacement rebounds gradually as water content decreases and once-reduced strength increases significantly in a relatively short time.

3) Vertical and horizontal coefficients of consolidation could be reduced to a mere fraction as a result of mandrel driving. Disturbance appears to increase with depth.

4) In addition to the need to further elucidate complexities brought in by installation of mandrel-driven sand drains, it is felt essential to improve our capability, which is often totally unsatisfactory, to predict the time-settlement relation of a structure on a natural soft clay foundation. Without ameliorating it, it is not even possible to determine whether sand drains are indeed required.

ACKNOWLEDGEMENTS

Appreciation is due to M. Zubair, M.M.R. Mukul and Pornthep Asanittong, all former graduate students at AIT for their enthusiastic participation in this field test program, which was financed in part by a special grant from Toyo University, Japan.

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