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Ultimate Settlement of Vertical Drain Treated Ground

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Introduction

As more and more land becomes subject to urban and/or industrial development, good foundation sites are difficult to find. In recent years, an increasing need is being felt for various types of constructions in marginal, low or reclaimed lands and coastal areas which are not suitable normally for foundations, because of one or more of the following reasons : low strength, high compressibility, poor volume stability, susceptibility for liquefaction, detrimental physical and chemical changes, etc. Various ground improvement techniques aim to make poor soil strata suitable for sound foundations. They are classified (Hausmann, 1990) into the following groups : hydraulic, mechanical, physical or chemical modifications, and modification by inclusions and confinement.

Loose sands, soft clays and water fills are improved by precompression to increase their strength and minimise post-construction settlements. Large time is required to preconsolidate clays by surcharge loads, due to their low permeability. Vertical drains are installed to accelerate the rates of consolidation and decrease the time for preconsolidation of clayey soils with surcharge loads. Vertical drains create new and shorter drainage paths and orient the flow into a more permeable (horizontal) direction. The excess pore pressures generated by surcharge loads cause water to flow horizontally into the drains and vertically along the drain up into a permeable layer or blanket laid on top and down into a pervious deposit if it exists at the site. Vertical drains are very cost effective as they reduce the time required for surcharge loading, that is construction time, the thickness of the surcharge fill and other consequent problems.

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Vertical drains are classified as (i) Sand drains; (ii) Fabric encased sand drains (sandwicks); (iii) prefabricated plastic (strip) drains and (iv) natural fibre drains. Moran (Hansbo and Torstensson, 1977) proposed the use of sand drains in the late 1920s while Porter (1936) described their applicability with practical experiments. The borehole is created either by pushing a closed-end mandrel or by water jetting. To facilitate rapid construction, minimise wastage of sand and ensure continuity of the drain, the sand is prepacked in a fabric sock, as in a sandwick (Dastidar et al., 1969) and then installed in the ground as a drain. Following the development of Swedish card-board drain (Kjellman, 1948), several prefabricated strip drains have come into the market. All these drains consist of a plastic core and a filter jacket made of non-woven geotextile or synthetic paper. The plastic core serves two vital functions, viz., it supports the filter jacket and provides longitudinal flow path for drainage. The jacket separates the flow channel from the surrounding clay and prevents clogging of the drain with fines. Mohan et al. (1972) were the first to propose the use of natural fibres, viz., jute and coir. The coir fibre is woven in the form of a long strip of about 150 mm wide and 10 mm thick and rolled to give a hollow cylinder (rope drains) of diameter of about 70 mm in which there is a continuous hollow space at its centre longitudinally. Rope drains were used at Salt Lake in Calcutta (Mohan et al., 1977). Lee et al. (1989) developed a vertical drain by wrapping two layers of jute burlap over four coir strands of 3 mm to 6 mm diameter. The strip drain had a size of 80 mm to 100 mm width and 8 mm to 10 mm thickness. Venkatappa Rao et al. (1994) studied the performance and biodegradation of five different types of natural fibre drains through model tests and conclude that they are effective in consolidating even highly plastic clays as they do not clog, and biodegradation is less in saturated clays compared to that in free draining sands.

Drain Installation Effects

On Permeability

When vertical drains are installed particularly by using closed end mandrel (displacement type drains), considerable disturbance and even remoulding of the surrounding soil takes place. The effects of drain installation on soil are of two types : a reduction in the permeability and an increase in the compressibility of the soil. The former effect is studied extensively (Barron, 1948; Casagrande and Poulos, 1969; MacDonald, 1985; Aboshi and Inoue, 1986; Hansbo, 1979 and 1981, etc.). An annulus of soil, called smear zone, of size two to three times the size of the drain, gets remoulded and has a permeability which is considerably less than that of the horizontal permeability of the undisturbed clay. Onoue et al. (1991) postulate existence of two zones of disturbance (Fig. 1), a zone of extensive remoulding (Zone I) and a transition zone (Zone II) in which the permeability

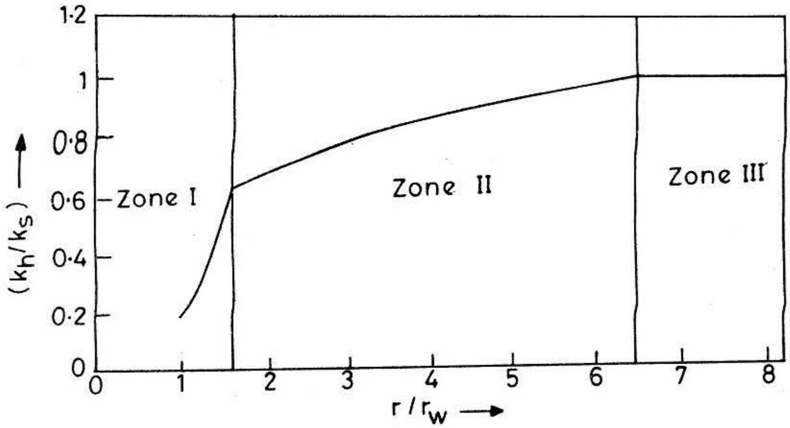

FIGURE 1 : Zones of Disturbance (from Onoue et al., 1991)

of the soil increases gradually from the remoulded value to that of the undisturbed clay (Zone III). Madhav et al. (1993) model the plastic drain treated soil as a two dimensional consolidation problem considering the soil around the drain to consist of three zones as suggested by Onoue et al. (1991).

On Compressibility and Strength

Terzaghi and Peck (1967), Schmertmann (1953), Nagaraj et al. (1991), etc., discuss the effect of disturbance on the void ratio – log effective stress relationship. Casagrande and Poulos (1969) conclude that displacement type vertical drains reduce the strength of soft and sensitive clays considerably. Akagi (1977) asserts that the installation of drains results in the consolidation of soft clays due to the large stresses induced. Madhav et al. (1995) propose a simple model to explain the destructuring of the soil structure and the inducement of pore pressure leading to larger settlement of vertical drain treated in comparison to that of the untreated case.

There appears to be no specific in situ measurements to quantify the effects of installation of displacement type drains by mandrel, on strength and compressibility/ deformational characteristics of soft soils. However, installation effects due to displacement type piles are very similar and indicate close resemblance with those due to mandrel driven drains. Figure 2 from Akagi (1989) depicts the consolidation curves for undisturbed and remoulded soils along with the strength curves for points adjacent to a driven pile. Point A corresponds to the in situ (undisturbed) soil. It moves to the left, point B and B′, towards the curve for remoulded soil with increasing disturbance, the remoulded strength being least close to the pile. With time, the points B and B′ move to C and C′ respectively, with consolidation of the soil. The paths BC and B′C′ can be seen to converge to a point at large stresses, as observed by Schmertmann (1953).

Figure 3 is a schematic of these effects. The soil adjacent to the drain gets disturbed and remoulded during installation with the degree of disturbance decreasing with distance (transition zone) from the drain (Madhav et al., 1993). The void ratio – log effective stress relationships of the in situ soil and the soil in the remoulded zone are shown in Fig. 3. The *in situ* soil is characterised by a preconsolidated stress, σ'_c, and compression and swelling indices of C_{cu} and C_s respectively. Consequent to the installation of the drain by the displacement technique, the structure of the soil near the drain gets altered and pore pressures are induced due to disturbance caused. The soil after disturbance moves to point D and deforms along the remoulded curve which has a slope defined by C_{cr} with the preconsolidation effect obliterated (Schmertmann, 1953; Lo, 1972; and Nagaraj et al., 1989). Point D is to the left of point A (the original position for the in situ soil), the distance between A and D representing the pore pressure, u_d, set up during installation. After

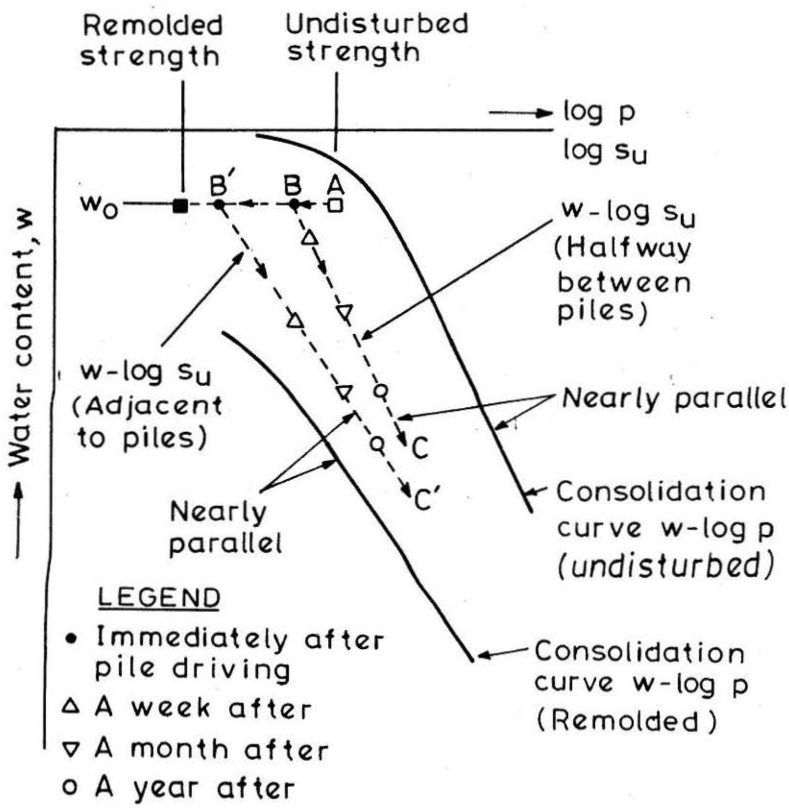

FIGURE 2 : Installation Effects on Strength (after Akagi, 1989)

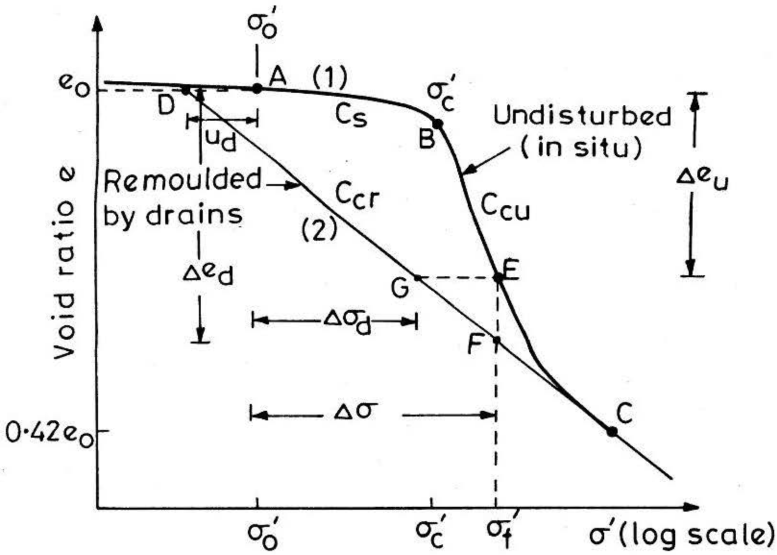

FIGURE 3 : Schematic of Disturbance Effect on Compressibility

consolidation under a preload, the positions of the points for the in situ and remoulded soils are E and F respectively. While the *in situ* soil passes through the preconsolidation stress point at B and moves to E, the remoulded soil moves directly from D to F. The void ratio – log effective stress curve for the remoulded soil is always below and/or to the left of that for the undisturbed soil. The disturbance effects due to drain installation thus cause the soil to undergo larger settlements compared to the untreated ground (Terzaghi and Peck, 1967 and Madhav et al., 1995).

The present investigation reports a model study of the installation effect on ultimate settlement of vertical drain treated ground in the laboratory. The disturbance and the consequent settlements as effected by the shoe size, over consolidation ratio and ageing, are studied. Based on a large number of case records analysed, an empirical relationship is proposed between normalised ultimate or final settlement of the drain treated ground with normalised spacing of the drains.

Material Properties and Test Procedure

Material Properties

Silty clay from the campus was used to prepare the samples for the model tests and Kalpi sand was used to prepare the prefabricated drains. For the silty clay, the liquid limit was in the range 36 to 41% while the plasticity index was about 22%. Grain size distribution tests gave sand, silt and clay in the proportions of 14, 55 and 31% respectively. Standard oedometer tests were conducted on samples prepared from a water content of 43% (slightly higher than the liquid limit). Loading was continued up to a stress level of 800 kPa with stress increment ratio of 1.0. The compression, C_c, and swelling, C_s, indices were 0.233 and 0.022 respectively. The coefficient of consolidation increased with intensity of load from 2.6×10^{-4} cm^2/s to 11.2×10^{-4} cm^2/s. The coefficient of permeability decreased with load intensity from 1.36×10^{-4} cm/s to 0.1×10^{-4} cm/s.

The clay and silt fractions were removed from the Kalpi sand. The fraction of the sand passing through 4.75 mm sieve but retained on 0.075 mm sieve was used in the preparation of prefabricated drain. Constant head permeability tests at the required unit weight of 1.72 gm/cm^3, gave an average permeability of 2.51 cm/s which was 20 to 42 times the permeability of the silty clay in the lower stress range.

Test Procedures

Model tests (Fig. 4) were conducted in cylindrical metal tanks of diameter 27 cm and height 65 cm. A valve was provided at the bottom of the tank for drainage. Soil samples were prepared by sedimentation and

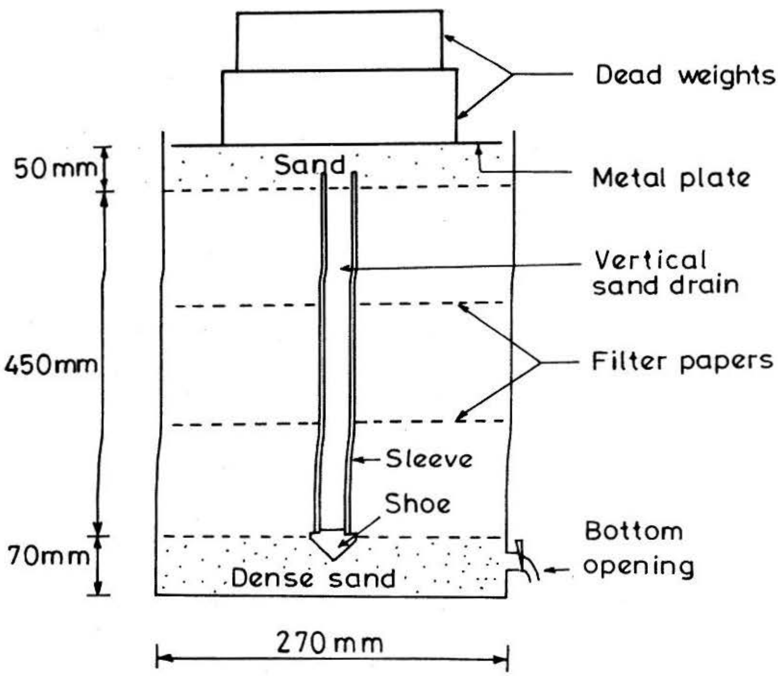

FIGURE 4 : Experimental Set-up

consolidation using the technique described in detail by McManus and Kulhawy (1993). The reconsolidation of the reconstituted soil produced uniform and nearly identical soil deposits in the laboratory. The inside of the tank was coated with a lubricant to minimise side friction. 70 mm thick sand layer was placed at the bottom of the tank. A filter paper was placed on its top and slurry of the soil prepared poured. Locally available silty clay (Kanpur silt) was air dried, powdered and sieved through 2 mm size sieve to remove the coarse fraction. Required quantity of the soil was mixed with sufficient amount of water and stored for 24 hours to attain uniformity. In the present investigation, slurry was prepared at an initial water content of $43.1\% \pm 0.2\%$ which is slightly higher than the liquid limit of the soil. The slurry was poured in three layers of 15 cm thick. After pouring each layer, filter paper cut in the form of sectors was placed on its top before pouring in the next layer. The inner layers of the filter paper reduced the initial consolidation time without increasing the strength of the deposit. The sample was left standing under its own weight for 24 hours after which 50 mm thick saturated sand layer was placed on top of the slurry. A steel plate with a diameter of 26.5 cm rests on the sand layer. Loads were applied to the steel plate to consolidate the soil deposit.

The slurry was consolidated to stresses of 3.25 kPa and 6.5 kPa. These stresses were achieved in stages, e.g. 0 to 1.625 kPa, 1.625 kPa to 3.25 kPa, etc. Three dial gauges placed 120° with each other on top of the plate measured the consolidation settlements. Each stress increment was maintained till the changes in the dial gauge readings were less than a division (0.01 mm) in an hour. The next increment of stress was applied and the procedure repeated. To study the effect of ageing, the load was sustained for a further period of 10 days after the primary consolidation was complete. Tests on untreated soil were conducted up to a stress level of 26.0 kPa through a stress increment ratio of one.

Tests with Vertical Drains

Three types of samples, (i) consolidated to a stress level of 3.25 kPa; (ii) consolidated to a stress level of 6.5 kPa; and (iii) consolidated to stress level of 6.5 kPa and aged for 10 days, were prepared for testing after drain installation. On each of these samples, three types of tests were done as follows : (a) prefabricated sand drain, also know as sandwick, of diameter 25 mm, installed with a shoe of base diameter of 30 mm; (b) prefabricated sand drain of diameter 25 mm, installed by a shoe of base diameter 40 mm; and (c) prefabricated sand drain of size 30 mm × 9 mm, installed by a rectangular tapered shoe of base size 48 mm × 15 mm. The prefabricated drains were made by pouring dry sand in to a sock of filter fabric. The drain was saturated with water before installation. The drain was placed on the shoe and forced down with a centrally placed steel rod of diameter 8 mm in to the sedimented and consolidated sample. After driving the drain to the fill depth, the steel rod was withdrawn slowly leaving the shoe inside.

Monitoring of Tests

Miniature cone penetration tests were done and water contents determined after each test, without or with drain, at radial distance of 2.5, 7.5 and 12.0 cm from the centre and depth of 0.0, 6.5, 26.0, 32.0 and 37.0 cm from the top. The miniature cone penetrometer consists of a cone 20 mm in diameter and an apex angle of 60°, driving arrangement and load indicator. It is pushed into the soil gradually and the force applied is noted from the load indicator from which the penetration resistance is calculated. After dismantling the test set up, samples were collected from different locations of the sample and their water contents determined in the standard way.

Results and Discussion

In one of the tests, a continuous load test was carried out on a sample prepared from the slurry without installing a drain, to compare the results with those from the oedometer test. Observed settlements were on an average 84.3% of those estimated based on the oedometer results. The height to diameter ratio of the sample in the oedometer was 0.333 while it was 1.667 in the model test. Hence the differences could be due to side resistance in the model tests even though the inside of the tank was coated by a lubricant. The values of the coefficients of consolidation in the vertical direction for different stress increments in the model tank tests calculated by Taylor's and the Asaoka's (Asaoka, 1978) methods compare (Table 1) well with those from the oedometer test. The void ratio – effective stress relations (Fig. 5) for

TABLE 1 Comparison of C_{vz} Values

Stress Level at Mid Depth (kPa)	Coefficient of Consolidation (cm^2/s)		
	Oedometer Test (Estimated)	Model Test (Untreated Sample)	
		Square Root Plot	*Asaoka Method*
5.223	2.520×10^{-4}	2.445×10^{-4}	2.492×10^{-4}
6.848	2.570×10^{-4}	2.534×10^{-4}	2.603×10^{-4}
9.285	2.620×10^{-4}	2.555×10^{-4}	2.493×10^{-4}
14.160	2.800×10^{-4}	2.628×10^{-4}	2.685×10^{-4}
23.910	3.180×10^{-4}	2.997×10^{-4}	3.012×10^{-4}

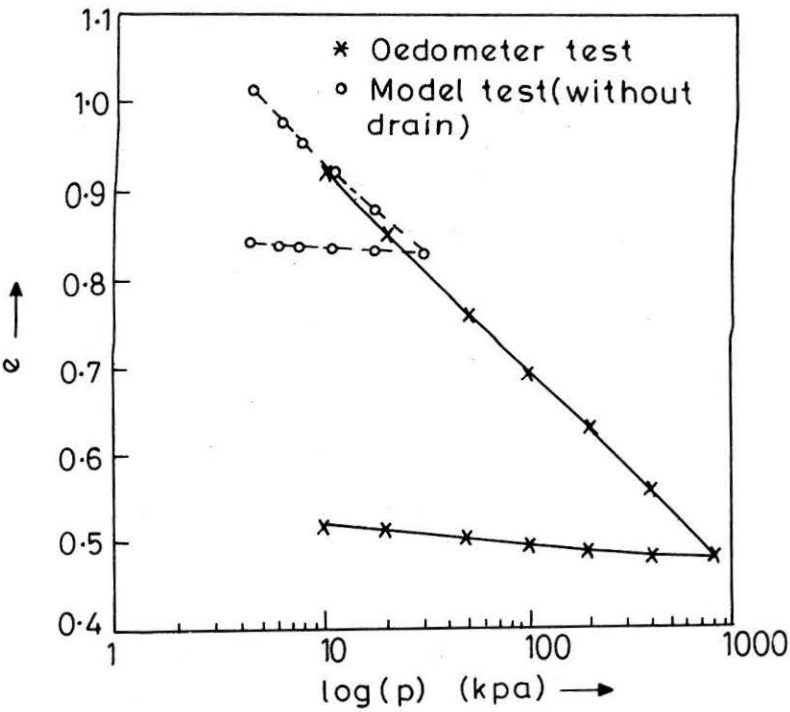

FIGURE 5 : e – log σ′ Relationship from Oedometer and Model Tests

the oedometer and model tank tests are close to each other, thus justifying the quality of testing.

Effect of Disturbance on Settlements

Experimental time – settlement curves for the cases, viz., OCR equal to 1.75 and 2.5 and OCR equal to 2.5 with ageing for the load increments : (i) 0.0 to 1.625 kPa (reloading), (ii) 6.5 to 13.0 kPa (virgin loading) and (iii) 13.0 to 26.0 kPa (virgin loading), are presented in Figs. 6 to 8 respectively. The sample was unloaded to install the vertical drains and hence the stress increment 0.0 to 1.625 kPa is a reloading one. Expectedly, the untreated sample exhibits very little settlement (0.07 to 0.088 cm) while the samples into which vertical drains were installed show considerably larger settlements (0.5 to 0.6 cm) for the same increment. Obviously the larger settlements are due to the reconsolidation of the disturbed or remoulded soil around the drains. Subsequently the samples were loaded in the virgin loading range. Even in this range, the settlements for the cases with the drains, are more and are mobilised faster than for the untreated case. The rates of consolidation are to be expected to be faster for thr drain treated cases. The ultimate settlements are clearly achieved. The settlements are incomplete in the case of the untreated samples.

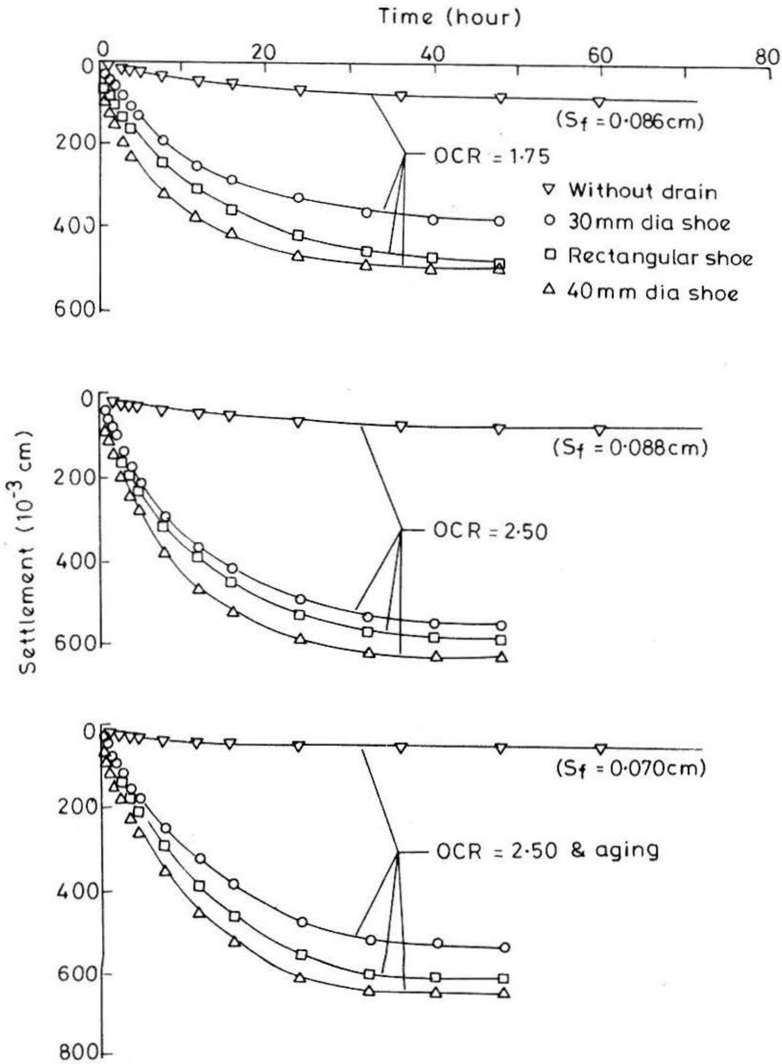

FIGURE 6 : Time – Settlement Curves for Stress Increment of 0.1 to 1.625 kPa

To compare ultimate settlements, the time – settlement data is replotted in the hyperbolic (Swamy and Rao, 1980; Sridharan et al., 1987) and the Asaoka's plots. The ultimate settlement values from these two methods are nearly the same. Tables 2 to 4 compare the ultimate settlements for the three cases studied and for all the stress increments. In all the cases, the settlement values obtained for the tests on drain treated samples are considerable more than those for the untreated sample for the stress increments in the reloading

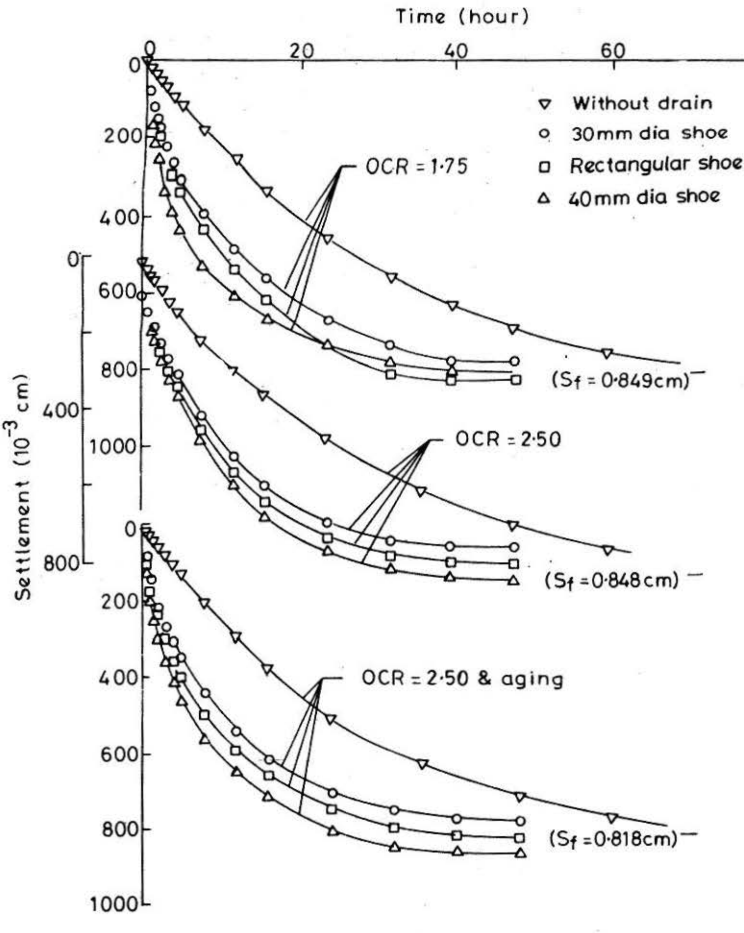

FIGURE 7 : Time – Settlement Curves for Stress Increment of 6.5 to 13.0 kPa

range. For the smaller shoe size (diameter of 30 mm), the settlements for the untreated samples for the stress increments in the virgin loading range are slightly greater than those for the drain treated samples if the OCR of the sample is 1.75. For higher OCR values and if the sample was aged, the settlements of the drain treated ground are more than those for the untreated one. For the stress increment of 13.0 to 26.0 kPa, the settlement values for the untreated samples are 1.086, 0.942 and 0.902 cm respectively for samples with OCR equal to 1.75 and 2.5 and aged sample with OCR of 2.5. The corresponding settlements for the drain treated ground range respectively from 0.969 to 1.077 cm, 0.992 to 1.122 cm and 0.98 to 1.132 cm. It should be noted that the ultimate settlement ratio (Table 5) defined as the ratio of drain treated ground to that of untreated one, increases with increasing shoe size

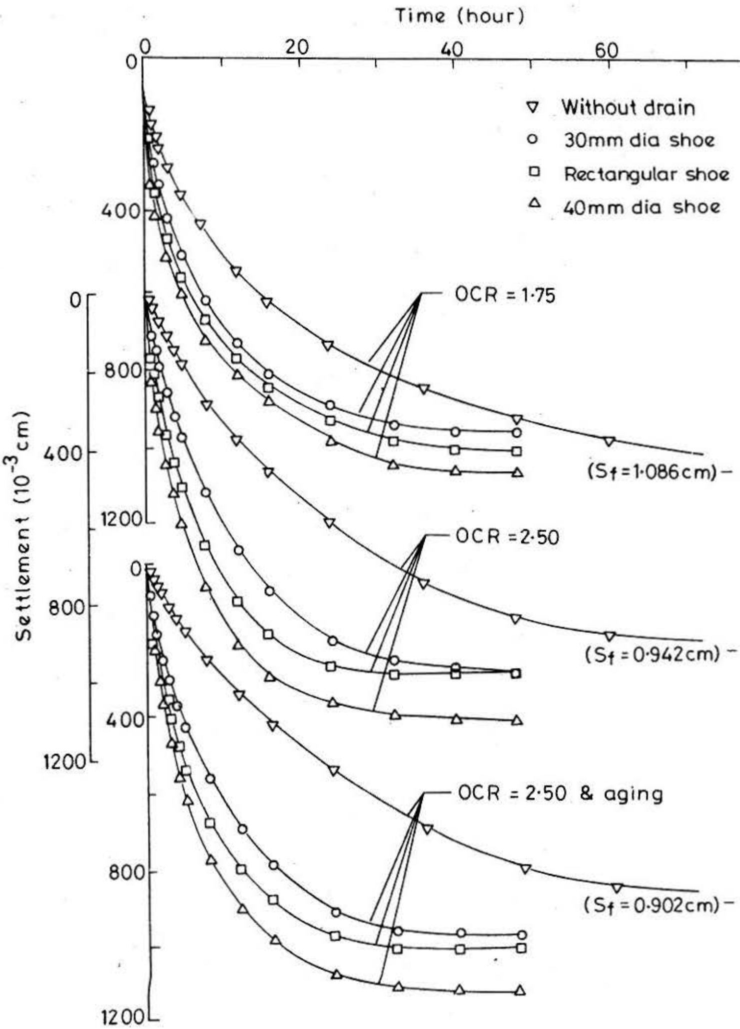

FIGURE 8 : Time – Settlement Curves for Stress Increment of 13.0 to 26.0 kPa

(area) as the disturbance would be larger if the mandrel or shoe size is larger.

A comparison of the water contents (Table 6) confirms the above observations. The initial water contents lie in a close range 42.2% to 43.3%. The final water contents decrease with increasing shoe size and are the least for the conical shoe of size 40 mm (base area of 12.57 cm^2) thus implying larger settlements have taken place. For example, for the case the sample with OCR of 2.5, the initial water content was close to 43.0 in all cases, but

TABLE 2 Settlements for OCR Equal to 1.75

Stress Increment (kPa)		Ultimate Settlement (cm)			
From	To	Untreated Sample	30 mm dia. Shoe, Base Area 7.069 cm^2	Rectangular Shoe, Base Area 7.2 cm^2	40 mm dia. Shoe, Base Area 12.566 cm^2
0.0	1.625 *	0.086	0.387	0.490	0.497
1.625	3.25 *	0.151	0.667	0.571	0.560
3.25	6.5 *	0.668	0.560	0.626	0.690
6.5	13.0	0.849	0.796	0.842	0.818
13.0	26.0	1.086	0.969	1.015	1.077

* Reloading

TABLE 3 Settlements for OCR Equal to 2.5

Stress Increment (kPa)		Ultimate Settlement (cm)			
From	To	Untreated Sample	30 mm dia. Shoe, Base Area 7.069 cm^2	Rectangular Shoe, Base Area 7.2 cm^2	40 mm dia. Shoe, Base Area 12.566 cm^2
0.0	1.625 *	0.088	0.563	0.603	0.647
1.625	3.25 *	0.060	0.474	0.452	0.582
3.25	6.5 *	0.165	0.625	0.676	0.647
6.5	13.0	0.848	0.776	0.819	0.863
13.0	26.0	0.942	0.992	0.991	1.122

* Reloading

TABLE 4 Settlements for OCR Equal to 2.5 and Aging

Stress Increment (kPa)		Ultimate Settlement (cm)			
From	To	Untreated Sample	30 mm dia. Shoe, Base Area 7.069 cm^2	Rectangular Shoe, Base Area 7.2 cm^2	40 mm dia. Shoe, Base Area 12.566 cm^2
0.0	1.625 *	0.070	0.550	0.620	0.660
1.625	3.25 *	0.050	0.480	0.525	0.580
3.25	6.5 *	0.151	0.610	0.630	0.655
6.5	13.0	0.818	0.785	0.832	0.872
13.0	26.0	0.902	0.980	1.010	1.132

* Reloading

TABLE 5 Comparison of Ultimate Settlement Ratios

OCR Aging	Stress Increment (kPa) (Virgin Loading)	Settlement Ratio			
		Untreated Sample	30 mm dia. Shoe, Base Area 7.069 cm^2	Rectangular Shoe, Base Area 7.2 cm^2	40 mm dia. Shoe, Base Area 12.566 cm^2
1.75	6.5 to 13.0	1.0	0.938	0.992	0.963
	13.0 to 26.0	1.0	0.892	0.934	0.992
2.5	6.5 to 13.0	1.0	0.915	0.966	1.018
	13.0 to 26.0	1.0	1.053	1.052	1.191
2.5 & Aging	6.5 to 13.0	1.0	0.960	1.017	1.066
	13.0 to 26.0	1.0	1.086	1.120	1.253

TABLE 6 Comparison of Water Contents

Shoe Type	OCE = 1.75			OCR = 2.5			OCR = 2.5 & Aging		
	A	B	C	A	B	C	A	B	C
*	42.2	31.8	26.38	43.3	31.1	28.18	43.1	30.3	29.70
1	43.2	30.9	28.47	43.1	29.7	31.09	43.2	29.7	31.25
2	43.0	30.5	29.07	43.0	29.6	31.16	42.9	29.4	31.40
3	43.3	30.3	29.53	42.9	29.3	31.70	42.9	28.8	32.87

* Without Drain
1 Conical Shoe of 30 mm Base Diameter
2 Rextangular Shoe of Base Size 48 mm × 15 mm
3 Conical Shoe of 40 mm Base Diameter
A Initial Average Water Content
B Final Average Water Content
C Percentage Reduction in Water Content

the final water contents were 31.1%, 29.7%, 29.6% and 29.3% for the drains with shoes of size 30 mm dia., rectangular base and 40 mm dia. respectively. The final water contents decrease with increasing shoe size signifying increasing effect of disturbance and reconsolidation. Thus the over all settlement of the drain treated ground would be larger than that of the untreated one.

Miniature cone penetration tests were carried out at different depths and different radial distances from the centre, in each model after the final unloading. Typical cone penetration resistance (q_c) versus depth curves are presented in Fig. 9 for the untreated soil and for the sample in which a vertical drain was installed with 30 mm dia. shoe. In both cases, the OCR of the sample was 2.5. The q_c values for the untreated sample can be seen to vary from about 70 kPa at the top to about 90 kPa at a depth of 40 cm. There was no marked variation in q_c values with distance from the centre as the sample was expected to be radially uniform. For the sample with the vertical drain, the q_c values increase from about 80 kPa at the top to nearly 110 kPa at 40 cm depth. Obviously the drains were effective in consolidating the soil. The q_c values are significantly higher than those for the untreated case. Also, a clear trend in the variation of q_c with radial distance from the centre can be noted. Soil near the drain gets fully consolidated and consequently the q_c values there are higher. The kinks which correspond to higher q_c values at depths of 13 cm and 27 cm are due to the filter papers

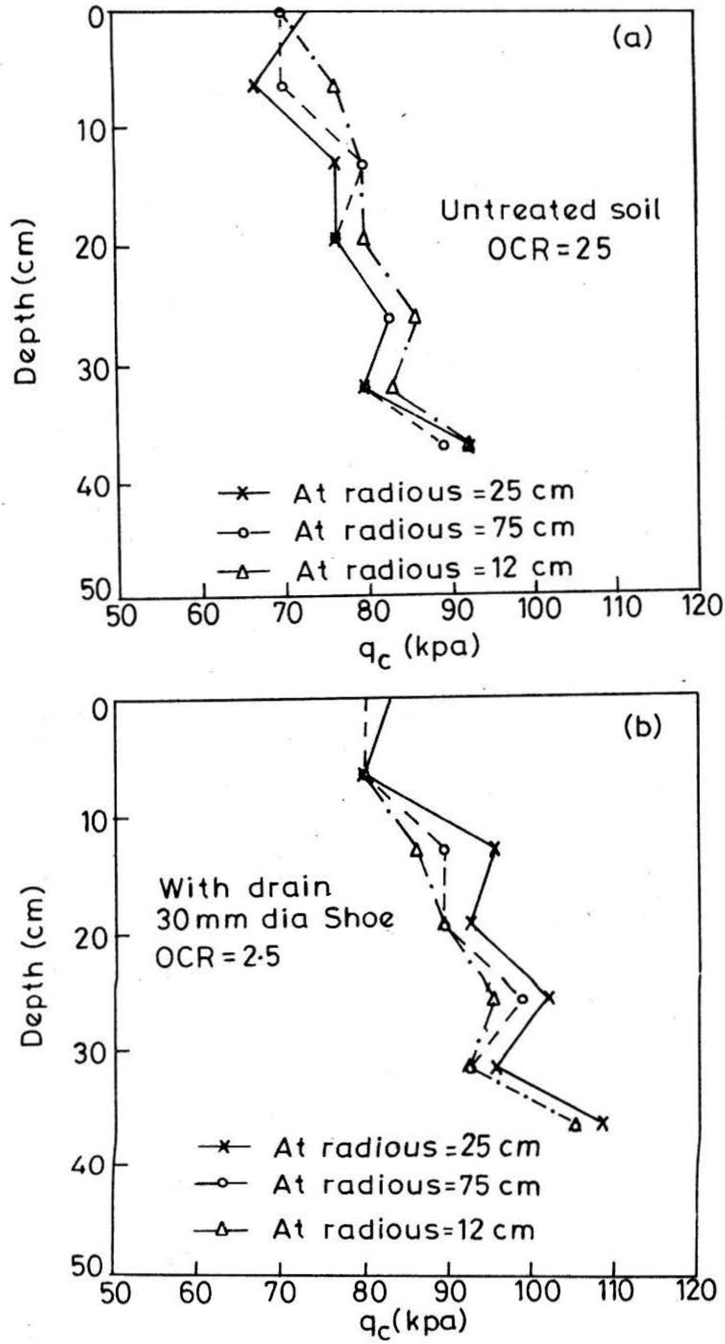

FIGURE 9 : Variation of q_c with Depth and Radial Distance from Centre (a) Untreated Soil; (b) Drain Treated Ground with 30 mm Dia. Shoe and OCR = 2.5

placed at those locations to speed up consolidation of the sample in the beginning, i.e., before the drains were installed. Even for such a small depth of the sample, the nearly linear increase in q_c with depth is remarkable and indicates a normally consolidated soil profile.

In Situ Results and Correlation

Case records, some of them spanning a long period of 14 years or more, of settlement of vertical drain treated ground are available comparing them with those of untreated ground. While primary consolidation settlements would be nearly or almost completed for drain treated ground, the untreated ground settlements are for incomplete primary consolidation and have to be extrapolated to estimate the final or ultimate settlements. Both the methods, viz., Asaoka (1973) and the hyperbolic, have been used to extrapolate the untreated and drain treated ground settlements. In most of the cases, both these methods gave nearly the same final or ultimate settlement values. Because of the larger time span over which the settlements of untreated ground are extrapolated, their final values could be somewhat approximate. However, in almost all the cases, the final settlements of drain treated ground were larger than those of the untreated ones.

The Ska-Edeby test field in Sweden, is one of the most well studied sand drain treated ground. 3 to 5 m thick post glacial slightly organic clay overlies 4 to 10 m thick glacial clay. The liquidity index of the clays is considerably more than 1.0 and sensitivity ranges between 10 and 20. Sand drains 18 cm in diameter have been installed at 0.9 m, 1.5 m and 2.2 m spacing. Holtz and Broms (1972) report settlements under a fill of 1.5 m after 14 years to be 54 cm for the untreated ground, 61 cm for drains spaced at 2.2 m, 66 to 81 cm for drains spaced 1.5 m and 56 cm for drains spaced at 0.9 m.

Hansbo and Tortensson (1977) compare settlements from geodrain treated ground with those of untreated area. 7 to 9 m of highly plastic clay (w_n = 50 to 100%, S_u = 10 kPa and $C_v = 0.5 \times 10^{-8}$ m^2/s) is underlain by silt and sand. The clay is highly overconsolidated to a depth of 5.0 m and the settlements are of secondary nature. The final settlements of the untreated and of test areas with drains at 1.6 m and 0.8 m are respectively 54 cm, 70 cm and 94 cm.

Embankments 10 to 12 m high are built (Ladd et al., 1972) on a soft silty clay deposit 12 to 14 m thick (w_l = 35%, PI = 15% and w_n = 50%) and with liquidity index ranging between 1.5 and 2.5 and sensitivity in the range 10 to 15. Non-displacement type vertical sand drains 0.305 m in diameter are installed at spacings of 4.95 m, 3.85 m and 2.75 m. The ultimate settlement of untreated ground ranges between 0.81 and 0.9 m, while those with the drains are 0.78 m, 0.96 m and 0.96 to 1.29 m respectively for drain spacings of 4.95 m, 3.85 m and 2.75 m.

Results from fully instrumented pilot tests for the second runway of Changi airport, Singapore, built on soft marine clay up to depths of 40 m, are reported by Chao et al. (1979). Both sand drains and flexible drains have been used. Geodrains installed at spacings of 3.2 m, 2.6 m and 2.1 m produce final settlements of 3.2 m, 2.6 m and 2.1 m respectively compared to final settlements of 68 to 75 cm of untreated ground. Chao et al. (1987) report that relatively large spacings are necessary as the soft clay gets remoulded due to the installation of displacement type drains at close spacings.

Singh and Hattab (1979) report a study on effect of installation methods and spacing on sand drain efficiency in terms of the coefficient of consolidation. Sand drains were installed by open and closed mandrels, auger and jetted mandrels. They rank the methods of installation for efficiency as auger, jetted, closed and open mandrels for closer spacings and closed mandrel, jetted mandrel, auger and open mandrel at larger spacings. From their reported data on coefficients of compressibility, it has been found that the sand drain treated ground settles more than the treated one for spacing of sand drain diameter ratios of 6 and 10. Settlements of drain treated ground for a spacing ratio of 3 was less than the untreated one, as the sand drain acted as a stiff reinforcing element much like a stone column and carried a higher proportion of applied stress than the original soil. Sand drains installed by open mandrel lead to the largest settlements.

A site underlain by 25 m thick soft to firm clay with liquidity index of 1.16 and sensitivity of 4, was treated with plastic board drains at 1.2 m spacing (Crawford et al., 1992). A 12.4 m high test fill settled by 3.3 m while adjacent test fills of 6 m and 11.4 m on untreated ground settled by 0.56 m and 0.61 m respectively in 400 days and were expected to settle by 0.56 m and 0.96 m after 25 years. In both the cases, the measurements indicate that primary consolidation was complete and settlements were of secondary nature.

The variation of settlement ratio, μ, ($= S_d/S_{unt}$ – Settlement of the drain treated ground to that of the untreated one, with the spacing ratio, S/d), based on measured or estimated final settlements from the case records presented in the preceding paragraphs, is depicted in Fig. 10. A large variety of drain types (sand drains, plastic board drains, etc.) have been used in the cases reported. The settlement ratio decreases from about 1.7 at a spacing ratio of 2.1 to nearly 1.0 for spacings greater than 12 times the diameter of the drain. The results of Ladd et al. (1972) for jetted drains and Crawford et al. (1992) for plastic drains (points could not be shown in Fig. 10 as they fall much beyond the plot) exhibit much larger settlement ratios. A curve

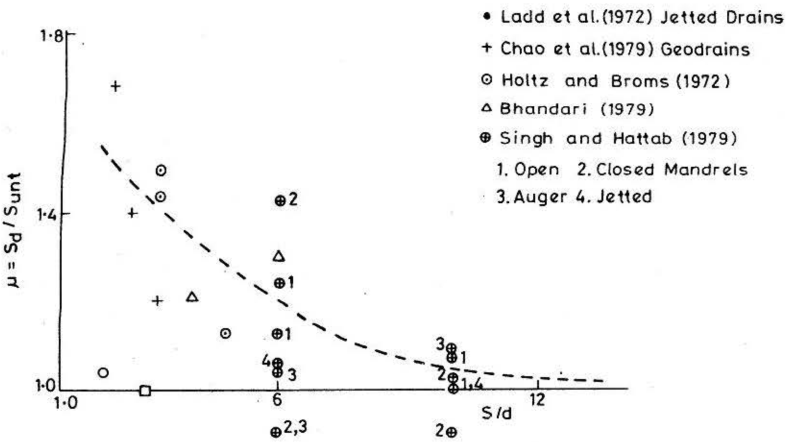

FIGURE 5 : e – log σ′ Relationship from Oedometer and Model Tests

averaging the points indicating 40% and 20% more settlements at spacing ratios of 3.0 and 6.0 respectively may be used for an approximate estimation of additional settlements due to drain installation induced disturbance or remoulding of the soil.

Conclusions

Ground treatment with vertical drains is often resorted to for accelerating consolidation of soft soils under preloads. The installation of drains by mandrel in particular disturbs and remoulds the soil. The effect of disturbance or remoulding (called smear) is accounted for in many theories which predict the rate of consolidation of drain treated ground. The additional settlements caused by the remoulding due to drain installation effects, are analysed through a simple mechanistic model, model studies in the laboratory and an empirical study of measured and/or estimated final settlements of untreated and drain treated grounds. The former study confirms that the settlement of drain treated ground is much more than that of the untreated one, the ratio increasing with the degree of disturbance quantified by pore pressure induced during installation. Based on the empirical study, a design curve is presented to estimate the settlements of drain treated ground in terms of the spacing of the drains.

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References

ABOSHI, H. and INOUE, T. (1986), "Prediction of Consolidation Settlement of Clay Layers Especially in the Case of Soil Stabilisation by Vertical Drains", *Proc. IEM – JSSMFE Joint Symp. on Geotechnical Problems*, pp.31-40.

AKAGI, T. (1977), "Effect of Mandrel-driven Sand Drains on Strength", *Proc. 9th ICO SMFE*, Tokyo, Vol.1, pp.3-6.

AKAGI, T. (1989), "Settlement and Strength of Soft Ground Stabilised by Vertical Drains", *In the Art and Science of Geotech. Engg.*, E.J. Cordings et al. (eds.), Prentice Hall, Englewood, Cliffs., pp.596-612.

ASAOKA, A. (1978), "Observational Procedure for Settlement Prediction", *Soil and Foundation*, Vol.18, No.4, pp.87-101.

BARRON, R.A. (1948), "Consolidation of Fine Grained Soil by Drain Wells", *Trans. Am. Soc. of Civil Engrs.*, Vol.113, Paper 2346, pp.718-754.

CASAGRANDE, L. and POULOS, S. (1969), "On the Effectiveness of Sand Drains", *Can. Geotech. J.*, Vol.6, No.3, pp.287-326.

CHAO, V., KARUNARATNE, G.P., RAMASAMY, S.D., VIJIARATNAM, A. and LEE, S.L. (1979), "Consolidation of Changi Marine Clay of Singapore using Flexible Drains", *Proc. 7th ECSMFE*, Brighton, Vol.3, pp.23-36.

CHAO, V., KARUNARATNE, G.P., RAMASAMY, S.D., VIJIARATNAM, A. and LEE, S.L. (1981), "Drain Performance in Changi Marine Clay", *Proc. 10th ICSMFE*, Stockholm, Vol.3, pp.623-626.

CRAWFORD, C.B., FANNIN, R.J., DE BOER, L.J. and KERN, C.B. (1992), "Experiences with Prefabricated Vertical (wick) Drains at Vernon, B.C.", *Can. Geotech. J.*, Vol.29, pp.67-79.

DASTIDAR, A.G., GUPTA, S. and GHOSH, T.K. (1969), "Application of Sandwicks in Housing Projects", *7th ICOSMFE*, Mexico, Vol.2, pp.59-64.

HANSBO, S. (1979), "Consolidation of Clay by Band Shaped Prefabricated Drains", *Ground Engrg.*, July, pp.16-25.

HANSBÓ, S. (1981), "Consolidation of Fine Grained Soils by Prefabricated Drains", *10th ICOSMFE*, Stockholm, Vol.3, pp.677-682.

HANSBO, S. and TORTENSSON, B.A. (1977), "Geodrain and Other Vertical Drain Behaviour", *9th ICOSMFE*, Tokyo, pp.533-540.

HAUSMANN, M.R. (1990), *Engineering Principles of Ground Modification*, McGraw Hill Publ. Co., New York.

HOLTZ, R.D. and BROMS, B.B. (1972), "Long Term Loading Tests at Ska-Edeby, Sweden", *Proc. Spec. Conf. on Perf. of Earth and Earth-supported Structures*, ASCE, Purdue, Vol.1.1, pp.435-464.

KJELLMAN, W. (1948), "Accelerating Consolidation of Fine Grained Soils by Means of Cardboard Wicks", *Proc. 2nd ICOSMFE*, Rotterdam, Vol.2, pp.302-305.

LADD, C.C. (1972), "Performance of Embankments with Sand Drains on Sensitive Clay", *Proc. Spec. Conf. on Perf. of Earth and Earth-supported Structures*, ASCE, Vol.1, pp.211-242.

LEE, S.L., KARUNARATNE, G.P., DAS GUPTA, S.D. and RAMASAMY, S.D. (1989), "A Vertical Drain Made of Natural Fibre for Soil Improvement", *Int. Workshop on Geotextiles*, Bangalore.

LO, K.Y. (1972), "The Influence of Mechanical Disturbance on the Consolidation of Clays", *Proc. 3rd SEACSMFE*, Hong Kong, pp.223-232.

MACDONALD, P. (1985), "Settlement of Fills on Soft Clay with Vertical Drain", *Proc. 11th ICOSMFE*, San Francisco, pp.2213-2216.

MADHAV, M.R., MIURA, N. and PARK, Y-M. (1995), "Settlement of Vertical Drain Treated Ground", *Int. Symp. on Settlement and Consolidation of Clayey Soils*, Ed. by H. Yoshikuni and O. Kusakabe, pp.695-700.

MADHAV, M.R., PARK, Y-M. and MIURA, N. (1993), "Modelling and Study of Smear Zones around Band Shaped Drains", *Soils and Foundations*, Vol.33, No.4, pp.135-147.

MCMANUS, K.J. and KULHAWY, F.H. (1993), "Preparation of Large-size Laboratory Deposits of Cohesive Soils", *Geotech. Testing J.*, Vol.16, No.3, pp.372-382.

MOHAN, D., JAIN, G.S. and SHARMA, D. (1972), *A Method of Strengthening Soil*, Indian Patent No. 134557.

MOHAN, D., JAIN, G.S., SEN GUPTA, D.P. and SHARMA, D. (1977), "Consolidation of Ground by Vertical Rope Drains", *IGJ*, Vol.7, pp.106-115.

NAGARAJ, T.S., SRINIVASA MURTHY, B.R., VATSALA, A. and JOSHI, R.C. (1990), "Analysis of Compressibility of Sensitive Soils", *J. of Geotech. Engrg.*, ASCE, Vol.116, No,1, pp.105-118.

ONOUE, A., TING, N.H., GERMAINE, M. and WHITMAN, R.U. (1991), "Permeability of Disturbed Zone around Vertical Drains", *Proc. Geotech. Engrg. Congress*, ASCE, Vo,.2, pp.870-890.

PORTER, O.J. (1936), "Studies of Fill Construction over Mudflats including a Description of Experimental Construction using Vertical Sand Drains to Hasten Stabilisation", *1st ICOSMFE*, Cambridge, Vol.1, pp.229-235.

SCHMERTMANN, J.H. (1953), "Estimation of True Consolidation Behaviour of Clay from Laboratory Test Results", *Trans. ASCE*, Vol.118, pp.311-325.

SINGH, G. and HATTAB, T.N. (1979), "A Laboratory Study of Efficiency of Sand Drains in Relation to Methods of Installation and Spacing", *Geotechnique*, Vol.29, No,4, pp.395-422.

SRIDHARAN, A., MURTY, N.S. and PRAKASH, K. (1987), "Rectangular Hyperbola Method of Consolidation Analysis", *Geotechnique*, Vol.37, No.3, pp.355-368.

SWAMY, K.K. and NARASIMHA RAO, S. (1980), "The Prediction of Settlements and Heave in Clays}, *CGJ*, Vol.17, pp.623-631.

TERZAGHI, K. and PECK, R.B. (1967), *Soil Mechanics in Engineering Practice*, 2nd ed., John Wiley and Sons, New York.

VENKATAPPA RAO, G., HARIPRASADA RAO, V. and BALAN, K. (1994), "Performance of a Natural Fibre Strip Drain – A Model Study", *Proc. IGC*, pp.291-294.