Treatment of soft ground by Fibredrain and high-energy impact in highway embankment construction

S. L. LEE and G. P. KARUNARATNE
Department of Civil Engineering, National University of Singapore

The economic growth of a country depends significantly on its transportation system. The highways that form part of this system sometimes pass over compressible soft soil deposits. Two major geotechnical problems associated with the construction of road embankments on these soil deposits are stability during construction and long-term settlement. If the latter problem is not addressed adequately, excessive post-construction settlement will have serious consequences for the maintenance of road pavement, in addition to the unwarranted downdrag force (negative skin friction) and lateral pressure on the piles supporting the bridge abutments and ancillary structures. To mitigate these adverse effects, deep-seated saturated clays are pre-consolidated with Fibredrain, a vertical drain manufactured with biodegradable jute and coir fibre, and surcharge, and surface deposits of granular, clayey or peaty materials are treated by high-energy impact (HEI) application to enforce settlement and improve their bearing capacity. If post-construction secondary compression is significant it can be reduced by placing a higher surcharge than required for primary consolidation. Because of their cost-effectiveness these techniques have been and are being used in several soil improvement projects in the Southeast Asian region. Details of the treatment, the properties of and design procedure for Fibredrains, and the application of HEI are discussed.

Keywords: high-energy impact; prefabricated vertical drains; soft ground

L'expansion économique d'un pays est tributaire, dans une grande mesure, de son système de transport. Les grands axes qui font partie de ce système passent parfois sur des dépôts de sols boulants et compressibles. Les deux grands problèmes de nature géotechnique que comporte la construction de remblais de route sur ces dépôts de sol sont la stabilité au cours de la construction et le tassement à long terme. Si l'on ne se penche pas de façon adéquate sur ce dernier problème, un tassement excessif risque de se produire après la construction, ce qui comporterait des conséquences graves pour l'entretien de la chaussée, en plus d'une force d'entraînement vers le bas (frottement superficiel négatif) et de la pression latérale sur les piles de culée des ponts et structures auxiliaires. Pour pallier à ces phénomènes négatifs, on procède à la pré-consolidation d'argiles saturées profondes avec du Fibredrain, un drain vertical réalisé avec des fibres biodégradables de jute et de coir, et une surépaisseur, d'autre part des dépôts superficiels de matières granulaires, argileuses ou tourbeuses sont traités avec l'application d'un impact à haute énergie (HEI) pour assurer le tassement et renforcer leur résistance de portée. Si la compression secondaire post-construction est significative, il est possible de la réduire en appliquant une surépaisseur plus élevée que celle qui est nécessaire, pour la consolidation primaire. En raison de leur rentabilité, on a utilisé, et on utilise toujours, ces techniques dans des projets d'optimisation du terrain dans la région du sud-est asiatique. On discute également de renseignements détaillés sur le traitement, les propriétés et la procédure de conception des Fibredrains, et l'application des impacts à haute énergie.

Introduction

Construction on soft ground is a major problem in civil engineering. If the ground is untreated it is not uncommon for the construction machinery to get bogged down, and bearing capacity or slope failure may occur because of inadequate soil strength. Both short-term and long-term settlement may occur, causing severe maintenance problems for highway engineers. Differential settlement between bridges on piled foundations and approach embankments may inconvenience and endanger motorists as well as other road users. Problems arising from floods and droughts also need attention, because they can cause severe distortion in the embankment due to swelling or shrinking of expansive soils.

Excessive settlement due to the construction of embankments over soft clay is best dealt with by pre-consolidating the clay until the anticipated settlement under the design load is enforced to an acceptable level. Fibredrains, a vertical drain manufactured with biodegradable jute and coir fibres, are installed from a sand platform into the compressible clay deposit at a spacing designed to achieve the desired degree of consolidation within the project duration under the predetermined surcharge intensity. If post-construction secondary compression is significant, it can be reduced by placing a higher surcharge than required for primary consolidation.

Surface deposits with low shear strength and stiffness can be treated cost-effectively by the application of high-energy impact (HEI) to enforce settlement and improve bearing

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capacity. HEI application consists of dropping a heavy pounder, ranging normally from 10 t to 40 t, from a height usually of about 10–25 m via a platform of granular fill. Variations of this method are useful for treating surface deposits of loose granular soil by dynamic compaction (DC), soft clayey soil by dynamic replacement (DR), and peaty clay by dynamic replacement and mixing (DRM). By tamping the ground in a regular grid pattern, settlement is enforced on the underlying soft soils, leading to densification in sandy soils and reinforcement in clayey deposits. Fibredrains with surcharge and HEI application have been used separately or in combination in several soil improvement projects in the Southeast Asian region. The sequence of soil treatment is shown schematically in Fig. 1. Details of treatment, improved ground conditions and mitigation of long-term settlements are discussed in subsequent sections.

Settlement

The settlement in the subsoil under the embankment loading consists of two parts: primary and secondary compression, especially if the subsoil is composed of clay of volcanic or highly compressible peaty origin. The primary compression can be eliminated by pre-consolidating the clay with Fibredrains and surcharge before the construction of the pavement. The secondary component, which leads to post-construction settlement, should be kept within the tolerable limits of maintenance. If excessive, a greater portion of secondary compression can be eliminated by placing a surcharge higher than that required for primary consolidation settlement.

Evaluation of consolidation settlement

Figure 2 shows a typical plot of void ratio against effective pressure for a consolidating soil on a semi-logarithmic scale.

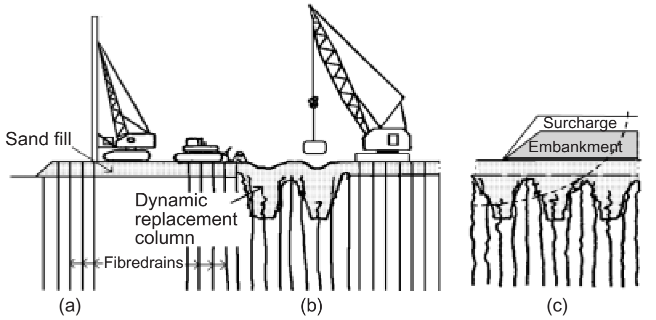

Fig. 1. Sequence of soil treatment in embankment construction: (a) PVD installation; (b) heavy tamping; (c) surcharge

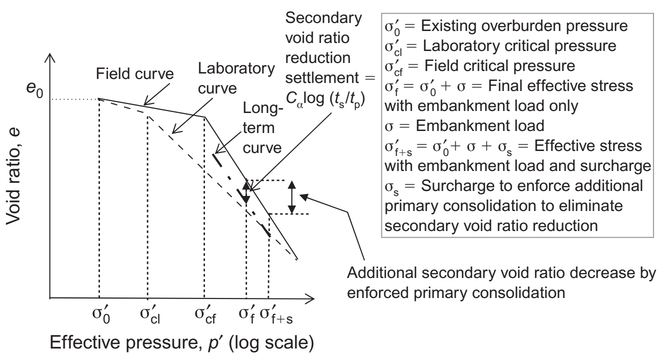

Fig. 2. Typical plot of void ratio against log of effective pressure

The existing effective overburden pressure σ'_0, the critical pressure σ'_c at which the slope of the curve changes from recompression index C_r to virgin compression index C_c, and the final effective stress σ'_f at the end of the consolidation period are defined as illustrated. The settlement that would be displayed at σ'_f as a creep phenomenon is obtained from the coefficient of secondary compression $C_\alpha = \Delta e/\log(t_s/t_p)$, where Δe is the reduction in void ratio at constant σ'_f over the time period between t_p, the end of primary consolidation, and t_s, the time at which compression is calculated.

The total settlement δT under one-dimensional compression can be evaluated in terms of void ratio reduction in the soil by

$$\delta T = \delta p + \delta s \tag{1}$$

where δp is the settlement due to primary compression involving both recompression and virgin compression, and δs is the secondary settlement under constant effective stress σ'_f after the end of primary compression. These two components are evaluated as follows.

For $\sigma'_f < \sigma'_c$:

$$\delta p = \frac{HC_r \log(\sigma'_f/\sigma'_0)}{1 + e_0} \tag{2a}$$

For $\sigma'_f > \sigma'_c$:

$$\delta p = \frac{H[C_r \log(\sigma'_c/\sigma'_0) + C_c \log(\sigma'_f/\sigma'_c)]}{1 + e_0} \tag{2b}$$

For $\sigma'_f > \sigma'_c$:

$$\delta s = \frac{HC_\alpha \log(t_s/t_p)}{1 + e_0} \tag{2c}$$

where e_0 and H are respectively the void ratio and the thickness of the soft soil layer at σ'_0.

Selection of consolidation parameters

The accuracy of settlement computation depends on the evaluation of an accurate stress distribution and the estimation of realistic field properties of the subsoil. Most consolidation properties determined in the laboratory are subject to sampling disturbance, which causes changes in the soil properties such that the values of C_c and σ'_c are usually underestimated (Hansbo, 1987). The laboratory curve of e against $\log p'$ curve also tends to move towards the void ratio axis, reducing the values of σ'_c and C_c. The value of C_r may also change.

Work done on carefully taken block samples and piston samples by Holtz *et al.* (1986) indicated that the laboratory determined critical pressure, σ'_{cl}, and the actual field value, σ'_{cf}, may vary considerably.

In a highway project, because of its wide coverage over many different types of soil, a vast amount of soil data will be collected. They should be statistically analysed, compared with other available data on similar soils, and the most probable parameters evaluated and adopted. Trends in attaching significance of the natural water content of a clay soil to the soil properties are shown in Fig. 3(a) for the variation of C_c and in Fig. 3(b) for $C_\alpha/(1 + e_0)$ against natural water content (Mesri and Rokshar, 1974).

For low embankment heights the settlement in the primary range will be confined mainly to the recompression zone. Settlement will also occur at these heights at a much faster rate owing to the larger C_v (coefficient of consolidation with vertical flow) in the clay in the recompression zone. For higher embankments and in thicker deposits of underlying

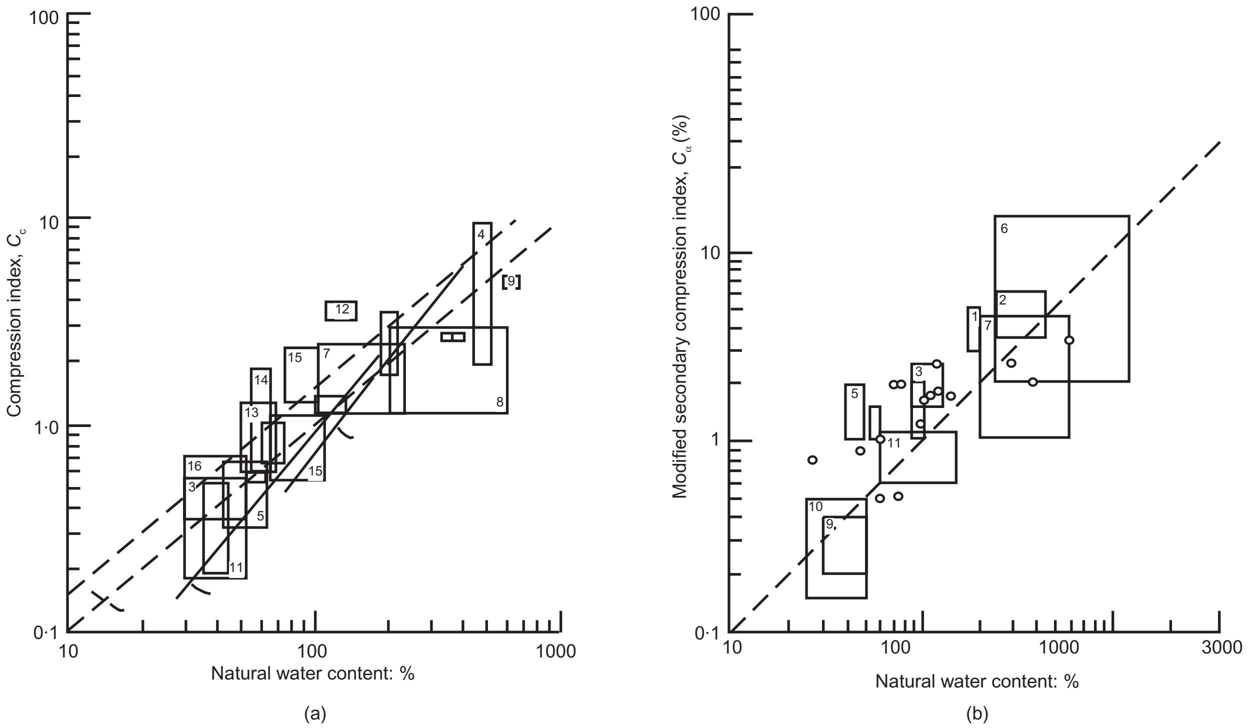

Fig. 3. (a) Variation of C_c with natural water content: 1, amorphous and fibrous peat; 2, calcareous organic peat; 3, soft to medium clay; 4, Mexico City clay; 5, Victoria marine clay; 6, organic silty clay; 7, peaty organic silt; 8, Canadian muskeg; 9, Kingston Marsh deposit; 10, soft varved clay; 11, stratified silty clay and sand; 12, Bangkok marine clay; 13, Whangamarino clay; 14, marine silty clay; 15, sensitive marine clay; 16, Rutherford clay. (b) Variation of $C_\alpha/(1 + e_0)$ with natural water content: 1, Whangamarino clay; 2, Mexico City clay; 3, calcareous organic silt; 4, Leda clay; 5, Norwegian plastic clay; 6, amorphous and fibrous peat; 7, Canadian muskeg; 8, organic marine deposits; 9, Boston blue clay; 10, Chicago blue clay; 11, organic silty clay; ○, organic silt etc. (after Mesri and Rokshar, 1974)

clay the total settlement could be excessively high, necessitating improvement of soft ground.

Properties of Fibredrain

Most prefabricated vertical drains (PVDs) are manufactured with a polymeric core and a geotextile filter sleeve. Fibredrain, a PVD developed at the National University of Singapore, consists of four axial coir strands enveloped within a filter comprising two layers of jute burlap (Lee *et al.*, 1995). Jute in raw form, after preliminary treatment, is spun into yarns and woven into burlap, which is used for the two filter layers. Coconut coir used in the core is spun into strands after preliminary processing. Figs 4 and 5 show respectively a bale of Fibredrains delivered on site and a schematic view of Fibredrain. It has shown satisfactory performance in many projects in Southeast Asia and Japan involving the treatment of soft marine and fluvial clays, as well as peaty clays, and in some cases in conjunction with surface densification using the application of high-energy impact (HEI).

From the environmental point of view, the energy consumption of jute and coir products in the production, transformation and utilisation/disposal stages is much lower than that of synthetic products (De, 1994). The raw materials for the production of Fibredrain are natural fibres extracted from jute plants and ripened coconuts, both of which are renewable resources. Jute plants and coconut trees grow in soil with nutrients such as nitrogen, phosphorus and trace elements, as well as water, sunlight and carbon dioxide. No air pollutants or water pollutants are generated during this process. The production of jute fibres and coconut coir is labour intensive, requiring land and manpower as the main inputs. The solid waste generated during the production and utilisation/disposal stages is basically organic and biodegradable, and in some instances is reusable as building materials. When used in stabilising soft subsoil, Fibredrain decomposes biologically after the consolidation process. The decomposed products are organic wastes with traces of methane and carbon dioxide, which are non-polluting. It is a green product, environmentally friendly and ecologically harmonious. Some of the laboratory tests conducted to examine the properties of Fibredrain are reported in the following; further details can be found in Karunaratne *et al.* (1999).

Fig. 4. Bale of Fibredrains and anchor shoes

Tensile strength

Figure 6 shows the tensile strength of Fibredrain in air-dry conditions, under conditions of soaking in water for 4 days,

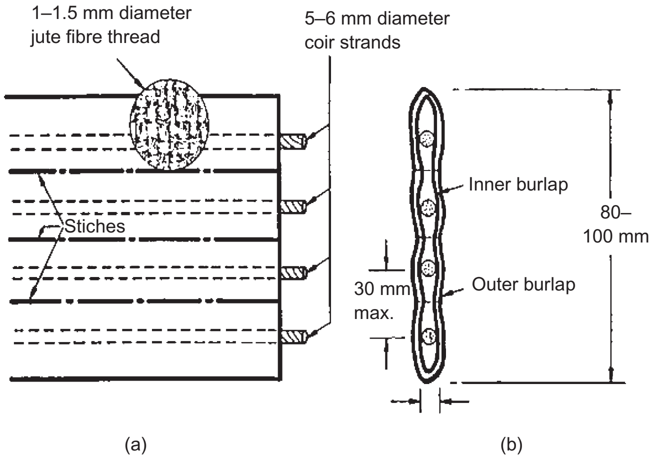

Fig. 5. Schematic view of Fibredrain: (a) elevation; (b) cross-section

after exposure for 50 days outdoors, and after consolidation in Ariake clay for 126 days (Miura *et al.*, 1995b). The tensile strength ranged from 8·96 kN (916 kg), 8·45 kN (860 kg), and 7·56 kN (771 kg) to 2·04 kN (208 kg) for the respective cases. The strength deterioration with time in the Ariake clay shows the biodegradability of the fibres. It should be pointed out that the deterioration in strength does not imply a reduction in drainage capacity, as observed in the Pantai Mutiara reclamation project in Jakarta, Indonesia (see below), where Fibredrains performed satisfactorily for more than two years (Lee *et al.*, 1988). Because of the high tensile strength (6·8 kN with 8·7% strain at rupture), robustness and flexibility of the core and the jute filter layers, Fibredrain can withstand installation stresses, and high-energy tamping while retaining its drainage functions unimpaired. As the mandrel is driven into the ground during the installation process, soils intrude into the mandrel through the interface between the mandrel and the anchor shoe, and develop significant tensile forces in the PVD as the mandrel is withdrawn. In recent field measurements of installation stresses on plastic PVDs, a tensile force of 1·4 kN on the filter was recorded at an installation depth of 30 m (Karunaratne *et al.*, 2002). It is imperative that the PVD should possess sufficient tensile strength to withstand these tensile forces.

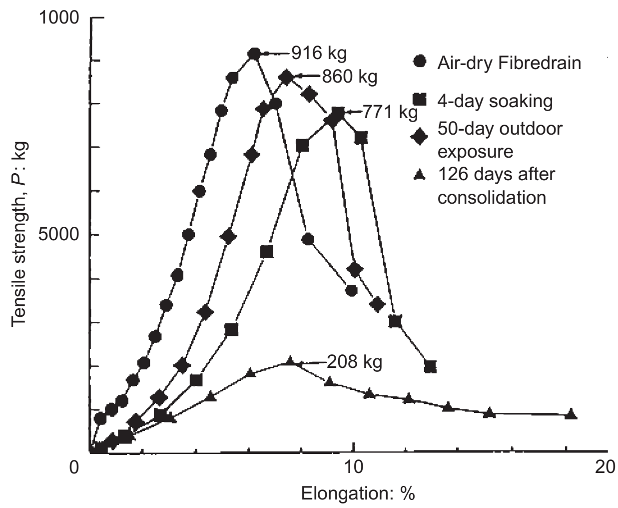

Fig. 6. Tensile test and biodegradability of Fibredrains (after Miura *et al.*, 1995b)

Filter permeability and clogging potential

The large apparent opening size (AOS) of Fibredrains, in the range 200–600 μm, serves to tap pervious layers and lenses in clay deposits. The double filter layer, which has a smaller effective AOS, intercepts the clay slurry generated immediately after installation of the Fibredrain in the clay as the mandrel is withdrawn. The cross-plane permeability of the filter was measured under a constant head across a stack of four identical burlaps fastened across an open end of a circular cylinder (Lee *et al.*, 1994). The cross-plane filter permeability determined in this way for clean water was better than 10^{-5} m/s. This is equivalent to the hydraulic conductivity of fine sand, which helps in tapping natural drainage layers such as sand and silt seams, lenses and other pervious paths. Clay slurry with water content from 65% to 600% was passed through the four burlap layers mentioned above under a pressure head of 0·5 m of water. About 2 l of marine clay slurry at a water content of 260–600% flowed out within 15 min. For Singapore marine clay ($w_L = 70\%$) at a water content of 65%, the passage of water was slower but clear from the beginning.

This scenario is similar to the field installation stage of PVDs as the mandrel is withdrawn, when disturbed slurry clay under high water pressure tends to rush through the filter into the core. The four burlap filter layers were removed carefully and cleaned in an ultrasonic agitator to extract the clay embedded in each burlap layer separately. Gravimetric analysis showed that soil particles were retained on the outermost burlap layer and the immediately adjacent inner layer, but no particles were detected in the third or fourth burlap layers (Fig. 7(a)). The water passing through the four burlap layers was examined and found to be similarly free of any soil particles (Lee *et al.*, 1994). The

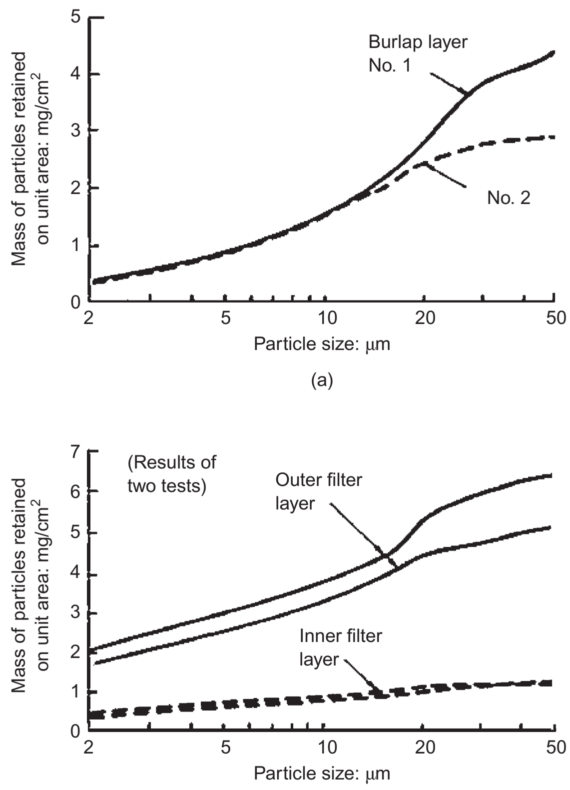

Fig. 7. Distribution of particles retained on filter layers: (a) filter test; (b) axial permeability test

retention of soil particles on the two burlap layers under axial permeability testing with marine clay at 65% water content is shown in Fig. 7(b).

This study shows that clay near the liquid limit will not enter the drain core during the installation process or the consolidation process, but will instead be retained by the two burlap layers. It is evident from this series of tests that the AOS of the burlap filter layers need not be too small to prevent clay intrusion into the core, but a larger AOS of the order of 200–600 μm is beneficial in tapping pervious seams and lenses in clay deposits.

Kinking

Miura *et al.* (1995a) compared the effective coefficient of consolidation, C_h, of Fibredrain (FD) and a plastic drain identified as PD in Ariake clay ($w_L = 86–97\%$, $I_p = 48–54\%$, $w_N = 105–34\%$, $G_s = 2{\cdot}59–2{\cdot}62$) by installing them separately in consolidation cells 500 mm in diameter and 500 mm high. On back-analysis, at consolidation pressures of 98 kPa and 294 kPa, the effective C_h was found to be 1·35 m^2/yr and 2·19 m^2/yr with PD, and 9·2 m^2/yr and 10·7 m^2/yr with Fibredrain respectively. Figs 8 and 9 show the deformed shapes of the two drains after carefully removing the clay around the drains at the end of consolidation (Miura *et al.*, 1995b; Aboshi *et al.*, 2001).

The PD has deformed significantly, resulting in kinking and decreased axial drainage capacity. The deformation of Fibredrain, on the other hand, was confined largely to an increase in the thickness and longitudinal compression of the drain without kinking. Axial compression in the Fibredrain is manifested largely as an increase in cross-sectional area, mainly because of the unwinding characteristics of jute fibres in both the filter layer and the core. The resulting increase in cross-sectional area and the decrease in filter opening size enhance the unclogged water flow into the drain. The above experimental results show that the effective C_h of the same clay under the same consolidation pressures is a function of the performance of different PVDs. Aboshi *et al.* (2001) reported the consolidation of Hiroshima clay (w_L = 49%, w_P = 21%, C_C = 0·36, P_C = 300 kPa) at 15 m depth and clay from Kojima Bay, Japan (w_L = 64%, w_P = 22%, C_C = 0·53, P_C = 78 kPa) in 300 mm diameter laboratory consolidation cells with Fibredrain and PD. Pore pressure measurements in the Fibredrain test showed zero excess pore pressure at the end of primary consolidation, whereas the PD test showed 5 m of water head remaining at respective axial strains of 24% and 19%. The measurement of axial permeability at the end of the test was reported to be 7×10^{-9} m/s for Fibredrain and zero for PD, illustrating the effect of kinking. Examination of the drain materials after the tests showed that Fibredrain had increased its thickness without kinking, whereas the PD had developed three to four kinks, which completely curtailed the axial flow capacity, as shown in Fig. 9.

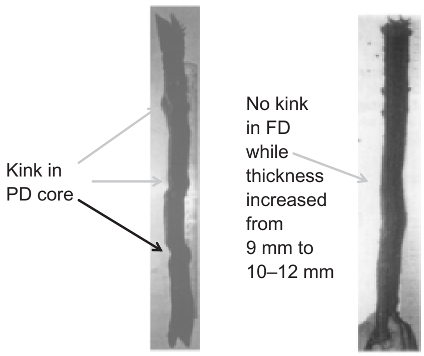

Fig. 8. Thickness of PD and FD after consolidation (after Aboshi *et al.*, 2001)

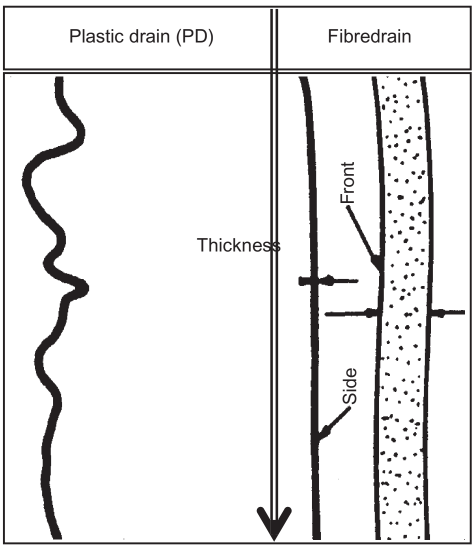

Fig. 9. Deformed shape of PD and Fibredrain (after Miura *et al.*, 1995b)

Axial discharge capacity

To evaluate the discharge capacity of Fibredrain a 200 mm long specimen was installed at the centre of a 100 mm diameter cylinder of Singapore marine clay (w_N = 65%, w_L = 70%, w_P = 40%). A coarse sand layer, 10 mm thick, was provided at both ends of the clay cylinder, which was enclosed within a large-diameter triaxial membrane. The discharge capacity was measured by connecting the drainage ends to a constant-head tank. The lateral pressure was increased in stages, and the volume change in the clay cylinder was measured, followed by determination of the discharge capacity under each pressure (Karunaratne *et al.*, 1999). Figure 10(a) shows the mean discharge capacity of Fibredrain under several hydraulic gradients plotted against argument of lateral pressure.

Discharge flow capacity

Required discharge capacity, Q_r

The average degree of consolidation $\bar{U}$ of a clay deposit under surcharge accelerated by PVD installation can be determined by (Hansbo, 1981)

$$\bar{U} = 1 - e^{-8C_h t/D^2 \mu} \tag{3}$$

where C_h is the effective coefficient of consolidation with horizontal flow, D is the influence diameter of a vertical drain, t is time, $\mu \approx \ln(D/d)^{-\frac{3}{4}}$, and d is the equivalent drain diameter $= (a + b)/2$, where a and b are the width and the thickness of the PVD cross-section respectively (Welsh, 1983).

The average degree of consolidation $\bar{U}$ may be taken as the ratio of current settlement s to the final settlement s_f: that is,

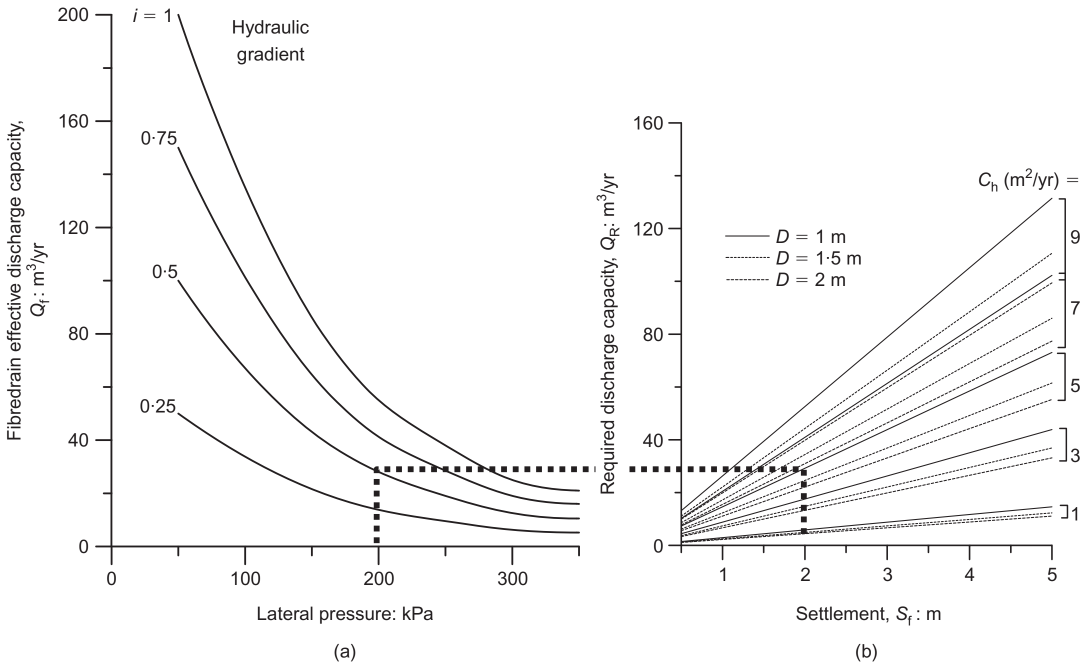

Fig. 10. Design charts for Fibredrain: (a) effective discharge capacity; (b) required discharge capacity

$$\bar{U} = \frac{s}{s_f} \tag{4}$$

Assuming that all parameters in equation (3) are relatively constant, differentiation of equation (4) with respect to time in view of equation (3) gives the rate of settlement as

$$\frac{\partial s}{\partial t} = s_f \frac{8c_h}{D^2\mu} e^{-(8c_h t/D^2\mu)} \tag{5}$$

The maximum rate of settlement occurs at the early stage of consolidation; taking $t = 0$, equation (5) yields

$$\left(\frac{\partial s}{\partial t}\right)_{t=0} = \dot{s}_0 = \frac{8s_f C_h}{D^2\mu} \tag{6}$$

where $\dot{s}_0$ is the initial ground settlement rate. If $A = \pi D^2/4$ denotes the influence area of a PVD, then the required discharge flow rate Q_R is estimated by

$$Q_R = A\dot{s}_0 = \frac{2\pi s_f C_h}{\mu} \tag{7}$$

Figure 10(b) shows the variation of Q_R with s_f for various values of C_h and D/d, based on equation (7) and assuming $d = 0{\cdot}055$ m. It can be seen that the larger the final settlement s_f, which depends on the thickness and compressibility of the clay deposit, and C_h, which is characterised by the compressibility and the hydraulic conductivity of the clay as well as the performance of the PVD, the larger is the required Q_R. It should be noted that reducing D leads to an increase of Q_R.

Effective discharge capacity of PVD

The effective discharge capacity of a PVD can be determined by field tests, or can be deduced from laboratory tests. Depending on how close the simulation of the laboratory test is to field conditions, the effective discharge capacity Q in the field should be deduced from the laboratory values Q' by

$$Q = \frac{Q'}{R} \tag{8}$$

where R denotes a reduction factor suggested by Koerner (1997) in the form

$$R = F_i F_d F_c F_b F_k \tag{9}$$

Koerner (1997) suggested that Q' be determined from short-term laboratory tests in accordance with ASTM D4716 (1991). In equation (9) F_i represents the reduction factor due to intrusion of the filter into the core space ($F_i = 1{\cdot}5$–$2{\cdot}5$); F_d is due to creep deformation of the core ($F_d = 1{\cdot}0$–$2{\cdot}5$); F_c refers to chemical clogging of the filter and core space ($F_c = 1{\cdot}0$–$1{\cdot}2$); F_b is due to biological clogging of the filter and core space ($F_b = 1{\cdot}0$–$1{\cdot}2$); and F_k is due to kinking of the PVD under axial deformation caused by the compression of the clay layer ($F_k = 1{\cdot}0$–$4{\cdot}0$).

Referring to equation (9), kinking is not a problem for Fibredrain, and hence F_k for Fibredrain is close to 1·0. For short duration projects F_c is low, or close to 1·0 (Koerner, 1997). Mlynarek (1998) and Mlynarek and Rollin (1995) have reported that biological clogging is not a problem if the apparent opening size (AOS) is large. The AOS of Fibredrain is in the range 200–600 μm when measured individually: hence F_b for Fibredrain is 1·0. The reduction factors due to deformation of the core and intrusion of the filter into the core space, $F_i \times F_d$, due to lateral pressure, have been taken into account in the testing procedure.

Design of soil improvement

In the design of soil improvement work using Fibredrain, S_f can be estimated from the results obtained from soil investigation and design loads. The value of Q_r can be obtained from Fig. 10(b) as a function of S_f for various values of influence diameter D and the effective C_h and compared with the discharge flow capacity given in Fig. 10(a), which depends on the lateral pressure corresponding to the depth of installation and the design load and surcharge. The hydraulic gradient set up along the PVD may be estimated by the intensity of the design load and surcharge in terms of metres of water divided by the length of the flow path along the drain.

The design of spacing to achieve a certain degree of primary consolidation, say 80% under surcharge, which is equivalent to 100% primary consolidation under design load, within a prescribed time is determined, as usual, by means of equation (3) using the influence diameter D and effective C_h determined from Fig. 10b. For example, under a lateral pressure of 200 kPa, $i = 0{\cdot}5$ and a settlement S_f equal to 2 m, the effective C_h is $5\ \mathrm{m^2/yr}$ with an influence diameter of 1·5 m. The time of consolidation by equation (3) corresponding to $\overline{U} = 80\%$, $S_f = 2$ m, $C_h = 5\ \mathrm{m^2/yr}$ and $D = 1{\cdot}5$ m is $t = 84$ days. It should be emphasised that the time of consolidation cannot be reduced by decreasing D because this leads to larger Q_R, which will be greater than the effective discharge capacity of the drain under the lateral pressure and hydraulic gradient. In most cases the effective C_h is smaller than the C_h of the soil determined in the laboratory (Rowe, 1968; Head, 1985). If the latter is smaller, it should be used in equation (3) to determine the time of consolidation.

Back-analysis of field settlement–time data obtained in Singapore marine clay using a plastic PVD (Choa *et al.*, 1979) showed effective C_h values in the range $2–10\ \mathrm{m^2/yr}$. Figs 10(a) and 10(b) show that for $i = 0{\cdot}5$ and a lateral pressure of 200 kPa, the values of effective C_h corresponding to settlements of 3, 2 and 1 m are 3, 5 and $7\ \mathrm{m^2/yr}$ respectively.

High-energy impact

Because of the large extent of highways over a variety of site conditions, varying subsoil material may be encountered, ranging from cohesive to granular soil, including organic peat in waterlogged ground, and waste material from mine areas and landfills. The deep-seated clay soil can be treated with Fibredrains and surcharge. For surface deposits, the application of high-energy impact (HEI) has been successfully employed in several projects in this region (Choa *et al.*, 1979; Lee *et al.*, 1989) and elsewhere (Lukas, 1986).

When improving in situ soil deposits by HEI, one of the physical limitations usually met is the depth. With standard HEI techniques it is generally not economical or practical to reach depths exceeding 5–7 m below the ground surface for clay or 8–10 m below the ground surface for sand. When the type of soil to be improved varies, it is customary to change the improvement technique. With HEI application the method and equipment remain essentially the same, but allowance is made in the application of the energy level and intensity.

HEI application consists of dropping a heavy pounder ranging from 10 to 40 t from a height, usually about 10–25 m, via a platform of granular fill. To treat loose granular deposits dynamic compaction (DC) was used effectively for more than three decades (Menard and Broise, 1975; Ramaswamy *et al.*, 1979). In the case of soft clay it is found to be ineffective when used alone on saturated soft clay as the pore pressure dissipation is governed only by the permeability of the clay (Choa *et al.*, 1979). For partly saturated cohesive soil it is found to be quite effective (Ramaswamy *et al.*, 1979). When the clay is soft and saturated, sand columns can be formed with HEI by systematically punching sand into the clay. This method, known as dynamic replacement (DR), has been used successfully even in a gap-graded sandy soil (Lee *et al.*, 1984). An adaptation of DR for peaty clay (Lee *et al.*, 1984) produced sand columns and sand claquage within the peat when increased pounding energy was applied directly on the sand columns. This technique, described as dynamic replacement and mixing (DRM), has been able to eliminate overall settlement, including a large percentage of the long-term secondary components.

The principle of successful implementation of HEI application lies in recognising the most appropriate energy levels to be used. The earlier approach of using empiricism based on field operation is complemented by a more rational approach of determining the enforced settlement, which is a function of the imposed stress from the structure to be built upon the soil and the soil modulus or compressibility. Lo *et al.* (1990) showed that there is a unique relationship between the saturation intensity applied through HEI and the enforced settlement. Fig. 11 shows the enforced settlement upon application of HEI energy in various projects worldwide. It can be seen that for every site (depending upon the soil characteristics and geometric configuration, involving depth, water table and stratification) there is a saturated intensity I_s beyond which further HEI application produces no tangible results in terms of settlement. Fig. 12 illustrates the operational parameters in HEI application, namely the

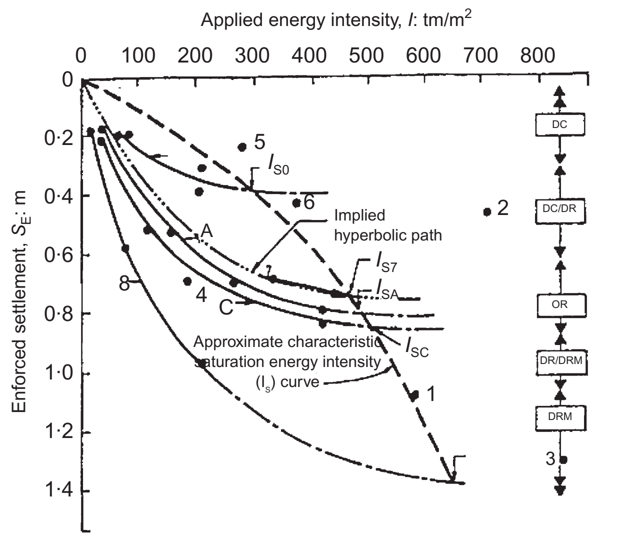

Fig. 11. Energy intensity against enforced settlement in HEI application with applicable DC, DR and DRM treatment regions (after Lo *et al.*, 1990): 1, Bishan; 2, Jurong; 3, Tauernautobahn, Ennstal; 4, Paya Lebar; 5, Corby; 6, Cadiz; 7, Jacksonville; A, B, Johannesburg; C, Nice; D, Changi

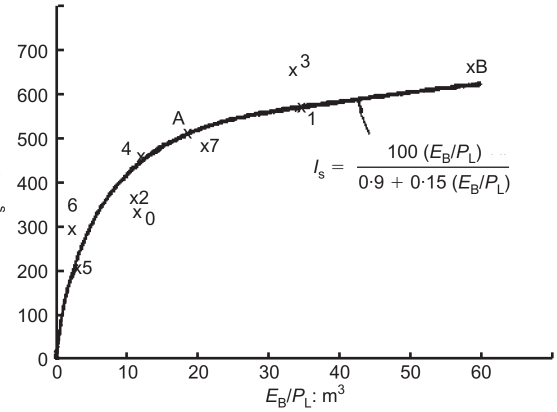

Fig. 12. Saturated energy intensity against E_B/P_L in HEI application (after Lo *et al.*, 1990)

pounder weight, the drop height (which together give the energy per blow, E_B), and the limit pressure P_L of the soil, plotted against the saturated intensity. The spacing, the number of drops at one location and the number of phases used are the other operational parameters in a given project that determine the cost of treatment.

For soft clay deposits exceeding about 5–7 m in depth, long-term consolidation settlement can be accelerated by Fibredrains and surcharge. In the approach of this scheme, shown in Fig. 1, in which Fibredrains are installed at predetermined spacing, a granular mat 1–2 m thick is placed on the ground and HEI is applied at an appropriate energy level to improve the surface deposits. The embankment is constructed, according to the specifications, by increasing the height progressively, based on the newly improved strength of the soil. If the degree of consolidation at time t under an embankment fill load is U_t, then the prevailing clay strength is given by

$$S_u = \frac{S_{u0} + \eta\sigma U_t \mu}{100} \tag{1}$$

where S_{u0} is the original undrained shear strength of the clay soil, σ is the average stress due to the fill load over the depth, η is the relevant stress distribution coefficient, and μ is the ratio of undrained shear strength to effective stress of normally consolidated clay. The stability of the fill should be checked based on the improved strength of the clay before raising the fill height further.

Method of construction

Following the decision to install Fibredrains and surcharge to accelerate consolidation of the underlying soft clay, and subsequently use HEI application for surface as well as fill compaction, the suitability of Fibredrains to withstand HEI was checked in several projects; otherwise the drain would be ruptured and its continuity impaired. As shown in Fig. 1, a working platform for the installation machines should be made with a suitable material available with fines less than 10% for a minimum of 50 cm in thickness. The drains should terminate in this sandy platform.

For HEI application, if required, a similar working platform is adequate except that the thickness should be increased to about 1–1·5 m. Based on the anticipated settlement under the embankment and the modulus of the subsoil, the energy level to be used will be determined. With the progress of HEI more sand will be added and finally the surcharge fill is placed in accordance with the drain design. The construction of the embankment follows afterwards. A suitable material should be selected; highly swelling minerals should be avoided in order to prevent excessive absorption of water within the embankment during wet weather and shrinkage during dry weather, which might result in distress in the pavement. In the event that the use of such materials is unavoidable, then they should be shielded from fluctuations of the water table and from flooding.

Field-monitored case studies

Hiroshima, Japan

Yoshida *et al.* (1995) reported the installation of Fibredrains in Hiroshima under the jurisdiction of the Hiroshima Prefecture government. The site was underlain by about 15–18 m of Hiroshima Clay (w_N = 90%, C_c = 1·30, e_0 = 2·40, γ_t = 15 kN/m^3, q_u = 120 kN/m^2). Fibredrains were installed at a spacing of 1·1 m and sand drains at 2·5 m spacing for comparative assessment. A considerable excess pore pressure remained in the clay at the end of 8 months in the sand drain area (Aboshi, 1999).

Figure 13 shows the monitoring results of piezometers installed in the clay in the Fibredrain area at El. −9·2 m, −12·5 m and −19 m, where the clay occurs between El. −8·5 m and −25 m. Piezometers were also installed at El. −10·6 m, −13·6 m, −16·6 m, −19·6 m and −22·6 m within the Fibredrain core to investigate the well resistance. Within the same 8-month period, the piezometers in the Fibredrain core showed only an excess pore pressure not exceeding 10 kPa, indicating that well resistance was insignificant and the axial flow capacity was more than adequate.

Treatment of ex-mining land Near Kuala Lumpur, Malaysia

The use of vertical drains and surcharge in conjunction with HEI application for the treatment of ex-mining land for a housing project (Lee *et al.*, 1989) is briefly presented. A safe bearing pressure of 60 kN/m^2 with a factor of safety of 2·5 was necessary for this project. The site, near Kuala Lumpur, is composed of mine tailing soils consisting of loose sandy deposits to very soft clayey soils. Soil profiles from typical borehole log show that N values are less than 2 blows/30 cm in clays and between 2 and 10 blows/30 cm in sands.

The construction commenced with placing a working platform with selected fill materials containing fines less than 10%. To treat the deep-seated clay, varying from 9 m to 21 m, Fibredrains were first installed at 1·5 m square spacing, followed by the application of HEI from a 15 t pounder falling freely from heights of between 10 m and 25 m. With 6 to 12 blows per pass for three passes, over a 6 m × 6 m grid together with a fourth 'ironing pass', energy of 225–250 t m/m^2 for DC was imparted in the sandy areas and 270–335 t m/m^2 for DR in the clayey deposits. A surcharge fill of 4·5 m was placed; the resulting settlement was monitored, and was found to taper off in 60–70 days. Fig. 14(a) shows the load–time–settlement curve for the surcharge as observed in three settlement plates. Fibredrains and surcharge combined with HEI enforced an anticipated total settlement of the order of 1 m. Fig. 14b illustrates the improvement shown by CPT before and after the HEI application. Plate load tests carried out subsequently showed negligible settlement.

This case record shows that using vertical drains, surcharge and HEI application, a safe bearing capacity adequate for five-storey residential houses can be easily achieved. For highway embankment construction this treatment method can be easily adopted.

Peaty soil treatment in Singapore

A series of field trials was conducted for evaluating the feasibility of stabilising the peaty clay and fluvial clay at the Bishan Depot site of the Singapore Mass Rapid Transit System (Lee *et al.*, 1984). Techniques involving surcharge only, vertical drains and surcharge, and the dynamic replacement and mixing (DRM) method of HEI application were investigated. In Zone B of the site, designated for DRM, some 7·8 m of peaty clay was underlain by 5·6 m of fluvial clay. Fibredrains were installed at 2·2 m square spacing from a 1 m thick sand blanket to improve the deep-seated fluvial clay. Afterwards, HEI application was used in treating the upper layer of peaty clay.

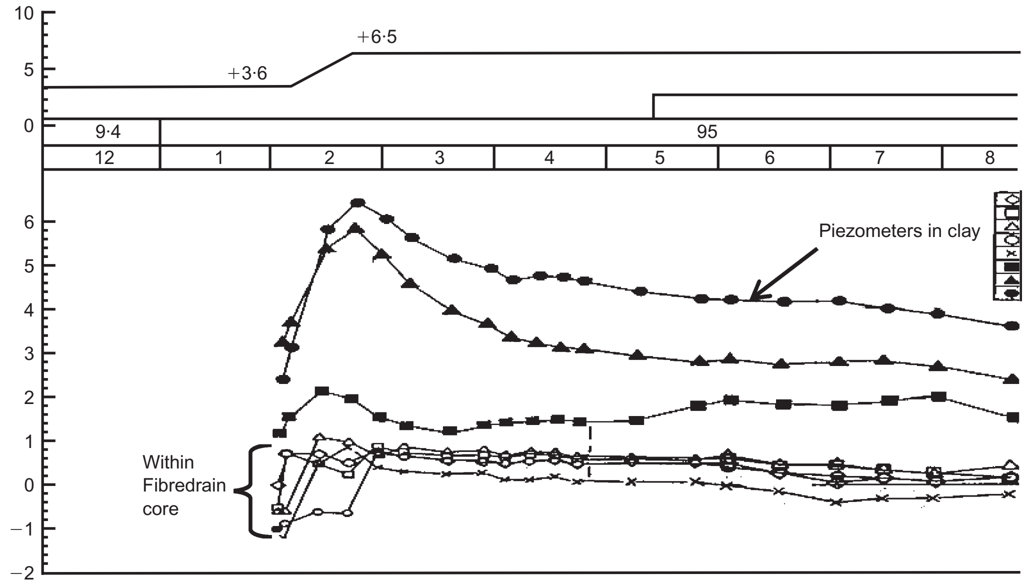

Fig. 13. Piezometric variations within Fibredrain and in the surrounding clay: Hiroshima, Japan (Yoshida *et al.*, 1995)

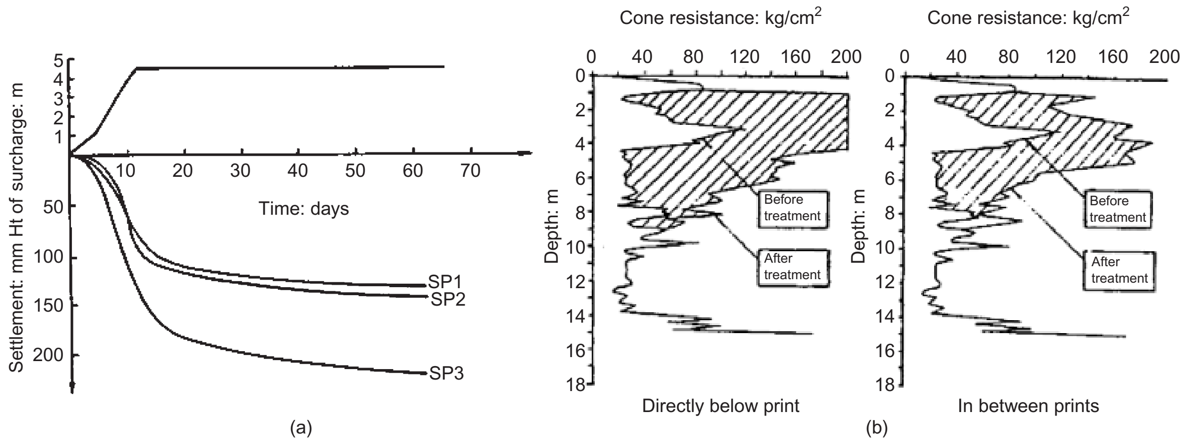

Fig. 14. (a) Load–settlement–time behaviour of ex-mining site; (b) CPT results before and after HEI application (after Lee *et al.*, 1989)

The HEI application consisted in dropping a 15 t pounder from a height of 15–20 m. Four passes were completed, on an 8 m × 8 m grid system, the last two passes being in between the previous print locations. The first two passes resulted in the formation of sand columns about 4 m long, tapering from an average diameter of 4 m at the surface to 2 m at the base. The third and fourth passes formed sand columns at intermediate locations and compacted the peaty clay in a three-dimensional confinement effected by the vertical sand columns and horizontal sand blanket at the surface. The fifth pass was the DRM phase, involving a 15 t pounder dropped from a height of 20 m with 35 blows per print. This action resulted in widespread replacement and mixing of the in situ peaty clay with sand charges.

The ground, after treatment, was effectively transformed into a stiff sand raft with pockets of peaty sand 3·2 m thick, followed by 4·5 m of sandy peat. The properties of the peaty soil, such as natural water content $w_N = 137\%$, undrained shear strength $C_u = 14\ \text{kN/m}^2$ and modulus $E = 1{\cdot}6\ \text{MN/m}^2$, were improved to $w = 115\%$, $C_u = 17\ \text{kN/m}^2$ and $E = 3{\cdot}2\ \text{MN/m}^2$ in the sandy peat area, while the stiff sand raft had a modulus of $E = 13{\cdot}7\ \text{MN/m}^2$.

A surcharge equivalent to 3·7 m of well-rolled clayey sand fill was placed subsequently for verification. Fig. 15 shows the observed settlement, along with the predicted settlement based on the improved properties. Comparison with other treatment methods carried out in the project can also be seen in Fig. 15 for the first 6 months. The settlement in site B began to peter out compared with the drains and surcharge method in Zone A and surcharge only in Zone C. Thus it is quite evident that DRM largely reduced the compressibility of the peaty clay, and drains and surcharge installed before DRM effectively accelerated consolidation settlement in the underlying fluvial clay. The result was a sand raft of high bearing capacity with negligible long-term settlement under an embankment load of about 3·7 m.

Reclamation on seabed clay at Pantai Mutiara, Jakarta, Indonesia

Pantai Mutiara, a 90 ha residential-cum-recreational development, was constructed on land that had been reclaimed at the waterfront in the Pluit area of Jakarta. The project site, located immediately north of metropolitan Jakarta, extends about 1500 m from the old shoreline and spans a width of 650–750 m, as shown in Fig. 16.

Part of the existing site was reclaimed for approximately 400 m from the original shoreline during 1979 to 1981. The

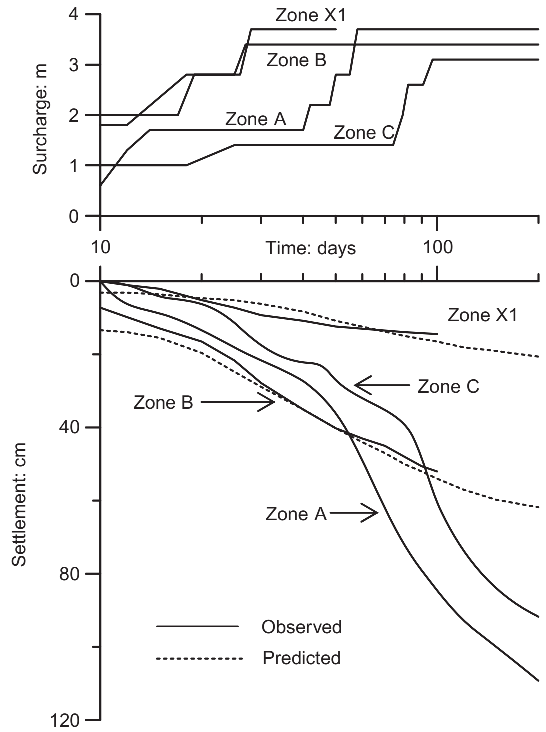

Fig. 15. Surface settlement of various treated areas under surcharge at Bishan Depot, Singapore

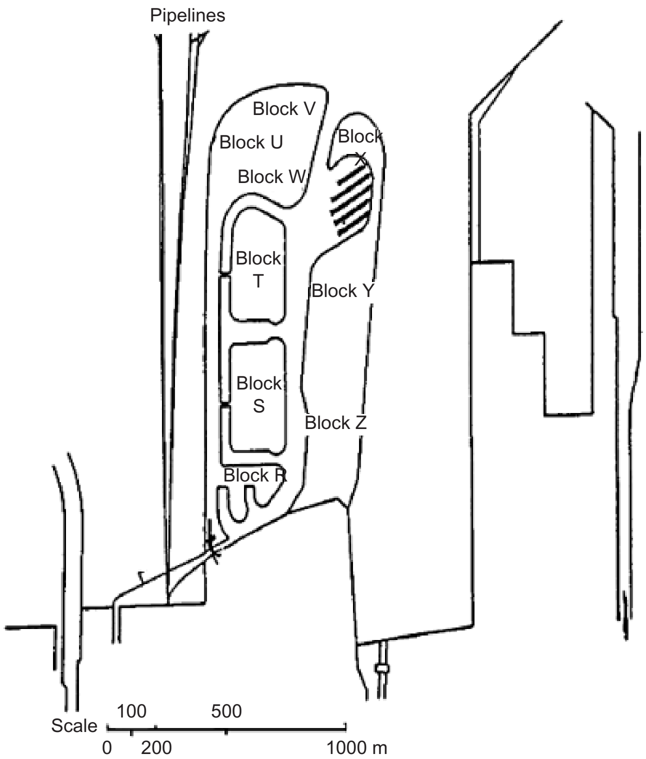

Fig. 16. Site plan of Pantai Mutiara reclamation in Jakarta, Indonesia

area beyond this was reclaimed in three phases to a proposed formation level of El. +2·6m. Phase IA of the reclamation works consisted in filling an area designated as 'Block R' (Fig. 16) from the end of 1986 to 1988, where the seabed varied from El. 0 m to El. −1·2 m. Phase IB, consisting of Block S and Block Z, where the seabed varied from El. −1·5 to El. −2·5 m, was completed in 1994 using dredged clay. Phase II, comprising Blocks T, U, V, W, X and Y, started in August 1994 and was completed in 1997. In each phase, the sequence of reclamation work involved construction of perimeter dykes, placement of fill, soil improvement using vertical drains and surcharge, dredging of internal canals and marina, and construction of canal/marina revetment. Temporary dykes were also constructed to allow phase development of the project.

A typical soil profile across the north–south direction of the proposed development is shown in Fig. 17. The seabed at the site varies from El. 0 m to El. −5 m and the proposed formation level is El. +2·6 m. The site is underlain by a layer, 16–18 m thick, of very soft to medium stiff silty clay followed by interbedded layers of silts and sands of dense to very dense consistency and very stiff silty clays. The upper 12 m of the deposit are highly plastic silts and organic clay with $w_N = 70–150\%$, $w_L = 60–130$, $I_P = 30–70$, void ratio $e = 1·5–3·5$ and $C_c = 0·7–1·2$. Between El. −12m and El. −18m the soft to medium stiff silty clay is relatively less compressible, with $w_N = 50–90\%$, void ratio e $\approx$ 1·7, and $C_c = 0·5$. The SPT N value is 0 in the very soft silty clay, 2–15 blows/30 cm in the soft to medium stiff silty clay layer, and more than 50 blows/30 cm in the dense to very dense sand/silt strata. The cone resistance rarely exceeds 1 MPa until 12 m depth. Thereafter, it increases to about 2–3 MPa between 12 m and 18 m, and exceeds 3 MPa below 18 m depth. The vane shear strengths increased from 2 kPa near the seabed to about 14 kPa at 10 m depth. Between 12 m and 18 m depths the shear strength values vary from 20 to 40 kPa, and below 18 m depth the soil was too stiff for the vane apparatus. The consolidation tests confirmed that the clayey soils are mostly normally consolidated, with a C_u/p' ratio of about 0·22–0·25.

Fibredrains for consolidation

Settlement of the filled areas will occur owing to the consolidation of the underlying soft sediments. To provide construction stability during fill placement, and to minimise post-construction settlement to an acceptable magnitude, a system of vertical drains with preloading was adopted. Initially, plastic vertical drains were installed, but because of a devaluation of the local currency they were replaced by locally manufactured Fibredrains at the request of the developer, as the material costs were lower. More than 350 000 m of Fibredrains (80 mm × 10 mm) were installed in a 2 m square grid over an area of about 4 ha in Block R to a depth of about 16–18 m. Because suitable materials were not

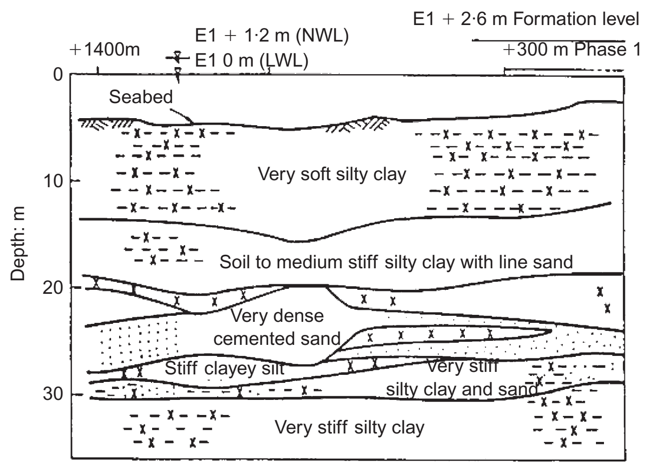

Fig. 17. Soil profile at Pantai Mutiara reclamation in Jakarta, Indonesia

available, fill placement was interrupted, and the reclamation works stretched to more than 15 months. Following Lee *et al.* (1988) the back-calculated C_h for this soft clay was obtained as 3·8 m^2/yr using equation (3).

For Block R, at a fill level of +2·6 m, the observed settlement was 1·2 m. Using the design chart for Fibredrains in Fig. 10(b), for the back-calculated $C_h = 3{\cdot}8\ \mathrm{m^2/yr}$, an equivalent influence diameter $D = 2{\cdot}25$ m, and final settlement of 1·4 m, of which 0·2 m is due to oversurcharge to account for minor superimposed load and secondary settlement, the required discharge capacity Q_R is 15 m^3/yr. The lateral pressure within the 16 m depth of seabed clay ranged from about 25 kPa to 75 kPa, using a lateral earth pressure coefficient of 0·5. Fig. 10(a) shows that, within this range of lateral pressure, Fibredrain possesses more discharge capacity than needed. After 12–15 months of drain installation the fill level was raised by 1 m. The ground settlement accelerated at all depths as expected, and the effective presence of the drain was observed. In situ vane tests indicated $C_u = 20\ \mathrm{kN/m^2}$ within the mid-depth of clay, corresponding to an average vertical effective stress p' of 80 kN/m^2. Based on $C_u/p' = 0{\cdot}25$, nearly 90–95% consolidation of clay under the applied load is complete.

Seawall construction on soft clay

In Phase II, with the reclamation of about 17–20 ha of land with hydraulically dredged seabed clay from the vicinity of the site, in order to increase the undrained shear strength of the underlying sea bed clay to about 22 kN/m^2 at El. −6 m at the periphery of the reclamation so as to facilitate seawall construction without instability, an estimated degree of consolidation of 90% was needed. With an average C_h of about 9 m^2/yr, a drain spacing of 1·4 m was adopted to achieve this degree of consolidation in 3 months with an appropriate surcharge (Lee *et al.*, 1988).

In this reclamation, excavation for drainage channels in an area where Fibredrain was installed about eight years earlier revealed no trace of its existence, which verified the biodegradability and hence the ecological friendliness of Fibredrain.

A typical section of the western breakwater was at a seabed level of El. −4·5 m. The dyke was designed to retain fill materials inside the reclamation areas and at the same time protect the reclaimed land from wave forces. Prior to construction of the dykes, bamboo clusters consisting of seven bamboo poles, 8 m long and 80–100 mm in diameter, were inserted vertically into the soft seabed at 1·5 m spacing with the aid of a floater guide until the tops of the clusters were at the seabed. These bamboo clusters have two Fibredrains wrapped around the centre pole of the cluster (Fig. 18). The bamboo is tied together by jute ropes and bamboo pins. A bamboo raft comprising seven layers of bamboo mat was placed on top of the clusters with the aid of a crane mounted on a pontoon. Each bamboo pole has a nominal 1 t tensile strength.

Sand was then spread over the rafts to a height of 0·30 m. For temporary dykes, sandbags were placed on top of this reinforced raft–sand layer. For permanent dykes, the raft–sand layer was covered with a geotextile (Polyfelt TS700) before continuing the build-up with graded quarry stone. Two rock mounds comprising 200 kg quarry stone were constructed gradually with a slope of 1:1·5. The next stage consisted of placing the geotextile on the centre portion between the two rock mounds, after which sand was placed up to El. −1·0 m. After allowing a period of time for the underlying soft clay to gain strength from dissipation of excess pore pressure through Fibredrains wrapped in the bamboo clusters, quarry stones of 15–50 kg were put in place.

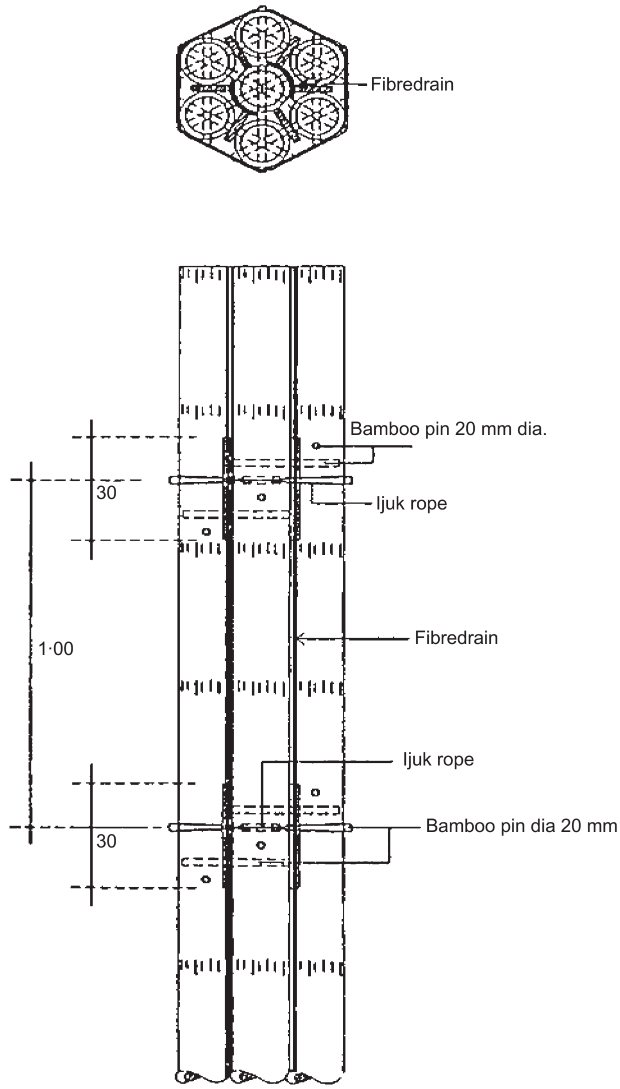

Fig. 18. Bamboo cluster with Fibredrains

To provide protection against wave forces, a 2 m thick layer of precast tetrapods weighing 1·6 t each was placed on the external sides of the dyke up to a crest height of El. +4·0 m. There was a time lapse between each stage to allow the underlying soft clay to gain strength.

During construction of the dyke the bamboo raft acts as a tension element, increasing the dyke's stability. The bamboo clusters increase the length of a potential rupture surface during the fill operation, and the Fibredrain accelerates the gain in strength of the underlying soft clay. Stability analysis was carried out for each stage, and the minimum factor of safety was 1·10 for a deep-seated failure. The factor of safety will increase gradually with time owing to consolidation of the underlying clay layers. The stability of the dyke with the reclamation fill behind it was also analysed, and the minimum factor of safety was found to be about 1·20 at the end of construction.

During the initial stages of construction the bamboo clusters supported the dyke and, with the raft, prevent the loss of sand and quarry stones in the seawall into the soft clay below. At the later stages, the weight of the dyke will exceed the bearing capacities of the bamboo clusters. The latter will subside, and the surrounding clay will commence the consolidation process, with a consequent increase in shear strength. From previous measurements of completed dykes, the settlement is estimated to be of the order of 1·5–2·0 m.

The incorporation of bamboo rafts and bamboo clusters with Fibredrains can be utilised in the construction of highway embankments over shallow water underlain by soft ground.

Concluding remarks

A considerable part of the cost incurred in the construction of highway embankments on soft ground arises from the need to pre-treat the soft soil in the foundation. If proper soil treatment is not carried out, protracted and costly road maintenance cannot be avoided. The most economical method of dealing with embankment settlement on soft ground is to eliminate the primary compression and a large portion of the secondary compression. The most cost-effective method for achieving this objective is to consolidate the deep-seated clay with Fibredrains and surcharge and to densify the surface deposits with HEI application, if required.

Fibredrains are proven to withstand installation and HEI tamping stresses with the following performance characteristics in the consolidation of soft soil.

(*a*) The effective coefficient of consolidation C_h is a function of the hydraulic conductivity and compressibility of the soil as well as of the field performance of the PVD.

(*b*) PVD field behaviour is greatly influenced by the deformation of the core and the filter, and the clogging and kinking of the drain.

(*c*) In field monitoring, the slope of the settlement–time curve in the early stage of consolidation multiplied by the influence area per drain gives the effective discharge capacity of the PVD–soil system under the surcharge.

(*d*) For Fibredrains, the effective discharge capacity in the field is given in Fig. 10(a) as a function of lateral pressure. The required discharge capacity is shown in Fig. 10(b) as a function of the final settlement S_f for various values of the influence diameter D and the effective C_h. With a given lateral pressure and final settlement S_f the values of D and C_h can be determined in Fig. 10. With these values of D and C_h, the consolidation time can be calculated from equation (3).

(*e*) The consolidation time cannot be reduced by decreasing D because this leads to an increase in Q_r.

(*f*) It should be emphasised that Fibredrain continues to function even at an axial strain of 24% (Aboshi *et al.*, 2001). In soil improvement projects where the PVD is observed to be underperforming, the effective discharge capacity of the PVD should be examined, with particular reference to the effect of kinking under axial strain caused by settlement and clogging. The damage caused by installation stresses may be important for PVD with lower tensile strength. For Fibredrain the effect of these factors is not significant, as discussed earlier.

(*g*) Fibredrain is environmentally friendly in the production and transformation processes, and biodegradable in the utilisation phase. Hence it is ecologically harmonious.

(*h*) In highway construction Fibredrains can be installed for the consolidation of soft subsoil accompanied by HEI application for embankment construction, if required, because their high tensile strength can withstand HEI.

(*i*) Where embankment construction over shallow water underlain by soft ground is required, bamboo raft and clusters with embedded Fibredrains can be effectively used, ensuring rapid consolidation with satisfactory foundation and slope stability.

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Discussion contributions on this paper should reach the editor by 1 April 2008