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SOFT CLAY FOUNDATION IMPROVEMENT WITH DRAINAGE AND GEO-INCLUSIONS, WITH SPECIAL REFERENCE TO THE PERFORMANCE OF EMBANKMENTS AND TRANSPORTATION SYSTEMS

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ABSTRACT: In this paper, the geotechnical aspects of soft clay improvement using prefabricated vertical drains (PVDs) with special reference to embankments will be demonstrated. The Cavity Expansion Theory is employed to predict the smear zone caused by the installation of mandrel driven vertical drains. Analytical and Numerical analyses adopting the equivalent plane strain solution are conducted to predict the excess pore pressures, lateral and vertical displacements. The advantages and limitations of vacuum application through vertical drains avoiding the need for high surcharge embankments are discussed using the proposed solutions. A few selected case histories are discussed and analyzed, including the site of the 2nd Bangkok International Airport, the coastal stretch of Muar Clay Plains in Malaysia and the Sunshine embankment, Australia. The predictions are compared with the available field data, verifying that the equivalent plane strain model can be used confidently with acceptable accuracy. Cyclic loading of PVDs is also examined in the laboratory in a manner appropriate for railway environments. It is shown that short PVDs can dissipate excess pore pressure as fast as they are built up under repeated loading conditions. Some selected on-ground experience of the first Author through the Ministry of Science and National Science Foundation during post-tsunami reconstruction efforts is described with specific reference to appropriate ground improvement requirements.

Keywords: Ground Improvement, Numerical modeling, Railway Embankments, Soft soils, Tsunami, Vertical Drains.

1. INTRODUCTION

In many countries, the majority of the population lives along the coast, which often contains soft soil deposits that require ground improvement prior to commercial or public infrastructure construction. Loose sandy soils, soft compressible clays and peaty soils are common along the coastal belt and surrounding populated areas in Sri Lanka, including the City of Colombo and its suburbs. In congested low-lying areas, the construction of rail tracks, road embankments, highways and runways necessitates the stabilization of these soft formation soils.

In this paper, the consolidation-based improvement of soft formation clays with prefabricated vertical drains (PVDs) will be described. The paper will highlight how these techniques can be applied for typical soft soil conditions with State-of-the-Art experimental and numerical models developed during the past 2 decades. The use of vacuum pressure with geomembranes is a feasible in countries such as Sri Lanka, because it is now more economical compared to the cost of high surcharge embankments that cannot be raised quickly. For instance, in order to avoid instability of soft soils,

multi-stage loading with rest periods is required causing inevitable delays. The predicted ground disturbance (smear effects) and the effects of localized soil unsaturation are compared with data obtained from large-scale radial consolidation tests and field data with extensive instrumentation. The equivalent plane strain solution can be used as a predictive tool with acceptable accuracy [1]. The theory can be extended to include cyclic loads and cyclic pore pressures as applicable for busy rail tracks.

Salient findings of research studies synthesised in an applied manner beneficial for practicing engineers within available resources should be the main aim of a cost-effective ground improvement program. While recent developments of various ground improvement schemes utilizing nanotechnology, advanced chemical, electrical and thermal methods are effective but expensive, this paper will be an attempt to highlight the cost-effective solutions for developing countries such as Sri Lanka, achieving the same effectiveness in long-term performance of infrastructure.

2. USE OF PREFABRICATED VERTICAL DRAINS

Prefabricated vertical drain (PVDs) with preloading is one of the most well known methods to improve the shear strength of soft soil and to reduce its post-construction settlement. Since permeability of soft thick clay deposit is very low, time required for preloading alone to achieve the desired settlement or shear strength can be too long [1,2]. Using PVDs, the drainage path is usually shortened from the thickness of a soil layer to half the drain spacing [4,5]. This system has been employed effectively to improve foundation soils for railway embankments, runways and highways [6].

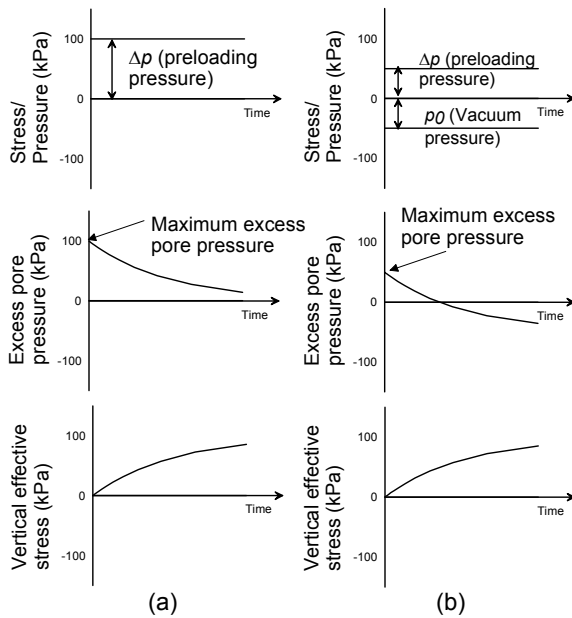


Fig. 1. Consolidation process: (a) surcharge loading (b) idealized vacuum preloading with surcharge [7]

PVDs consist of a perforated plastic core acting as the drain, protected by a geotextile sleeve (filter). The dimensions of most PVDs are in the order of 100 mm width and 4 mm thickness. The vacuum preloading method was initially proposed by Kjellman [8] in Sweden using cardboard wick drains. When a high surcharge load is required to meet the desired settlement, a combined vacuum and fill surcharge method can be considered as an alternative. Especially in very soft clays, where a high surcharge embankment cannot be built without affecting stability, the vacuum application is preferable. The PVD system can distribute the vacuum pressure to the deep subsoil, thereby increasing the consolidation rate of the reclaimed land from the sea [9,10]. The effective stress increases due to the vacuum load while the total stress remains constant (Fig. 1).

For an optimum drains and vacuum preloading system, the installation of horizontal drains in the transverse and longitudinal directions is also beneficial after placing a sand blanket [11]. All drains can then be connected to the boundary of a peripheral bentonite slurry trench typically sealed with an impermeable membrane [7]. Finally, the vacuum pumps are connected to the prefabricated discharge system attached to the slurry trenches. The suction head created by the pump accelerates the excess pore water pressure gradient in the soil towards the PVDs and the surface sand mat, as shown in Fig. 2.

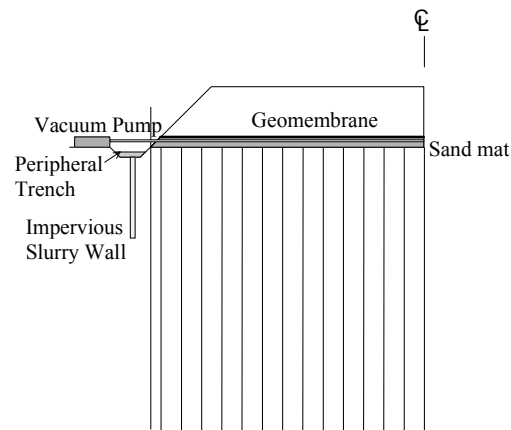


Fig. 2. PVDs and Vacuum preloading system [7]

3. FACTORS AFFECTING VERTICAL DRAINS PERFORMANCE

3.1. Smear Zone

Smear zone is the disturbed region that occurs when installing a PVD by a rigid mandrel. This causes a significant reduction in soil permeability around the drain, which in turn reduces the rate of consolidation. Based on laboratory tests conducted using a large-scale consolidometer at University of Wollongong, the smear zone can be quantified based either on the variation of permeability or the water content of the soil surrounding the drains [12,13]. Fig. 3 shows the variation of the ratio of the horizontal to vertical permeability (k_h/k_v) at different consolidation pressures along the radial distance, using a large-scale consolidometer. The water content variation with the radial distance is shown in Fig. 4. As expected, both the k_h/k_v ratio and the water content considerably decrease towards the drain. The permeability ratio between the undisturbed and the disturbed smear zone (k_h/k'_h) is approximately 1.5, and the extent of smear zone (r_s) is 4-5 times the radius of the vertical

drain (r_w). However, this ratio can vary depending on the installation method and the soil conditions.

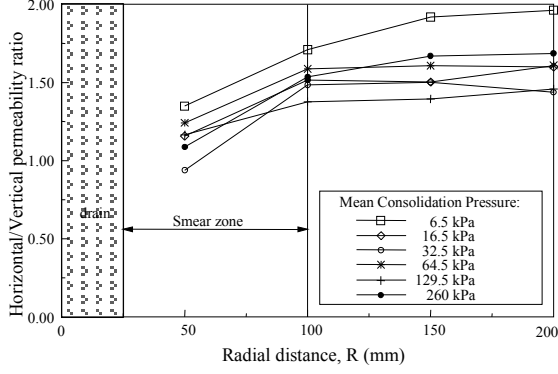


Fig. 3. Ratio of k_h/k_v along based on the large-scale consolidation tests (after [12])

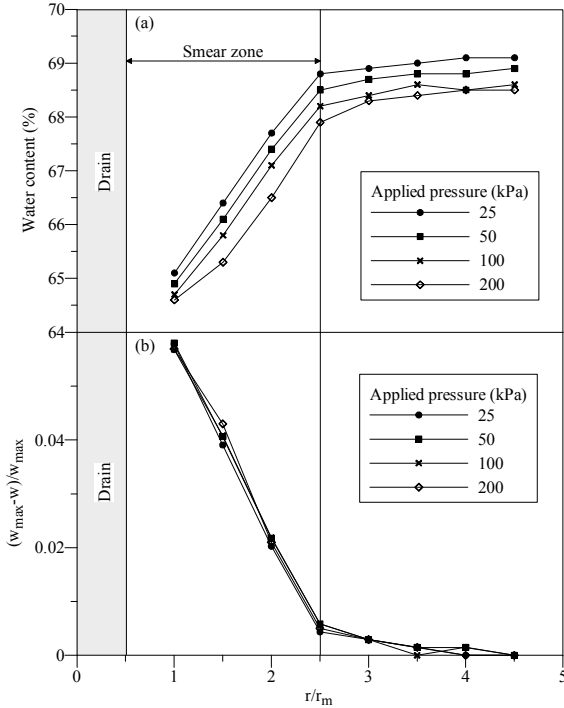


Fig. 4. (a) water content, and (b) normalized water content reduction with radial distance at a depth of 0.5 m (after [13])

Apart from the laboratory measurements, Sathanathan [14] employed the cylindrical cavity expansion theory to estimate the extent of the “smear zone”, caused by mandrel installed drains. The modified Cam-clay model (MCC) was incorporated as explained in detail elsewhere by Collins & Yu [15] and Cao et al. [16]. Only a summary is given below. For soil obeying the MCC model, the yielding criterion is:

$$\eta = M \sqrt{\left(\frac{p'_c}{p'} \right) - 1} \quad (1)$$

where, p'_c : the stress representing the reference size of yield locus, p' = mean effective stress, M = slope of the critical state line and η = stress ratio. Stress ratio at any point can be determined as follows:

$$\ln \left(1 - \frac{(a^2 - a_0^2)}{r^2} \right) = - \frac{2(1+\nu)}{3\sqrt{3}(1-2\nu)} \frac{\kappa}{\nu} \eta - 2\sqrt{3} \frac{\kappa\Lambda}{\nu M} f(M, \eta, OCR) \quad (2)$$

where,

$$f(M, \eta, OCR) = \frac{1}{2} \ln \left[\frac{(M+\eta)(1-\sqrt{OCR-1})}{(M-\eta)(1+\sqrt{OCR-1})} \right] - \tan^{-1} \left(\frac{\eta}{M} \right) + \tan^{-1} (\sqrt{OCR-1}) \quad (3)$$

where, a = radius of the cavity, a_0 = initial radius of the cavity, ν = Poisson's ratio, κ = slope of the overconsolidation line, ν = specific volume, OCR = over consolidation ratio and $\Lambda = 1 - \kappa/\lambda$ (λ is the slope of the normal consolidation line). The corresponding mean effective stress, in terms of deviatoric stress, total stress and excess pore pressure, can be expressed by the following expressions:

$$p' = p'_0 \left[\frac{OCR}{1 + (\eta/M)^2} \right]^\Lambda \quad (4)$$

$$q = \eta p' \quad (5)$$

$$p = \sigma_{rp} - \frac{q}{\sqrt{3}} + \frac{2}{\sqrt{3}} \int_r^{r_p} \frac{q}{r} dr \quad (6)$$

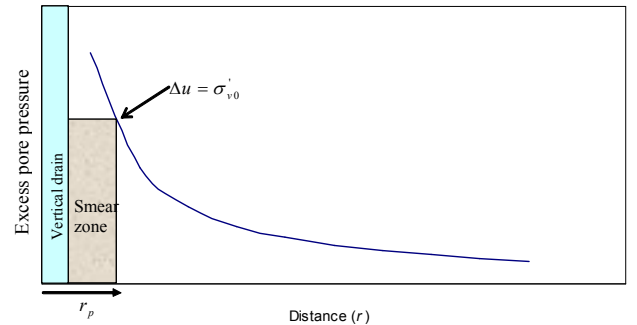


Fig. 5. Smear zone estimation by the Cavity Expansion Theory

Employing Equations (4)-(6), the excess pore pressure due to mandrel driving (Δu) can be determined by:

$$\Delta u = (p - p_0) - (p' - p'_0) \quad (7)$$

where, p_0 = initial total mean stress. The extent of the smear zone can be defined by the region in which the excess pore pressure tends to exceed the initial overburden pressure (σ'_{v0}) (Fig. 5). In this region surrounding the drains ($r < r_p$), the soil properties including anisotropy are altered severely.

3.2. Drain Unsaturation

Unsaturation of soil adjacent to the drain can occur as a result of mandrel withdrawal (air gap) and the relatively dry condition of PVDs. Indraratna et al. [17] described the retardation of pore pressure dissipation due to drain unsaturation in large-scale laboratory testing through a series of models, at the drain-soil interface.

The FEM programme ABAQUS incorporating the modified Cam-clay theory [18] was used. The soil moisture characteristic curve (SMCC) including the effect of drain unsaturation was captured by a thin elastic layer of drain elements. The equivalent permeability coefficients using Indraratna and Redana [19] method and the apparent past maximum pressure are listed in Table 1.

Table 1. Modified Cam-clay parameters [17]

Soil Model Parameters	Values
λ	0.15
κ	0.05
Critical state void ratio, e_{cs}	1.55
Critical state line slope, M	1.1
Permeability in undisturbed zone, k_{hp} (m/s)	9.1×10^{-11}
Poisson's ratio, ν	0.25
Permeability in smear zone, k'_{hp} (m/s)	3.6×10^{-11}

The following 3 models were considered:

Model 1 –Linear vacuum pressure distribution along the drain length with a fully saturated drain. The soil behaviour is based on the modified Cam-clay parameters (Table 1).

Model 2 –With the application of linearly varying vacuum pressure, a layer of unsaturated elements is simulated at the PVD boundary ($E = 1000$ kPa, $\nu = 0.25$).

Model 3 – Conditions are similar to Model 2, but the time-dependent variation of vacuum pressure (vacuum removal and reloading) is simulated.

Fig. 6 shows the surface settlement predicted from the above described models. The predictions show that the assumption of unsaturated soil layer at the drain-soil boundary with time dependent vacuum pressure variation (Model 3) is reasonable. Full saturation represented by Model 1 over-predicts the settlement.

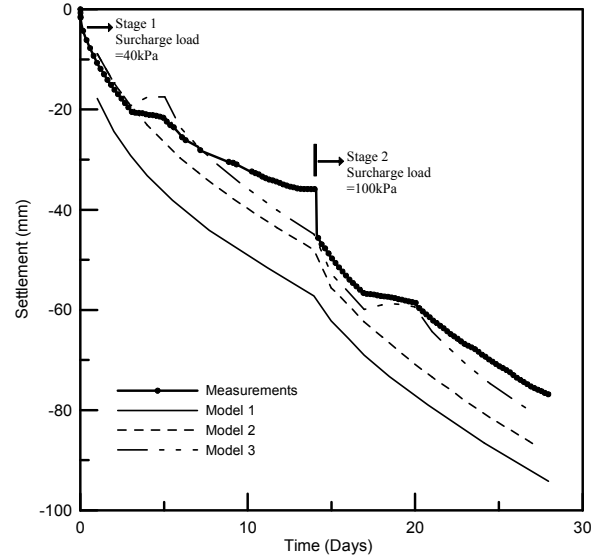


Fig. 6. Predicted and measured surface settlement [17]

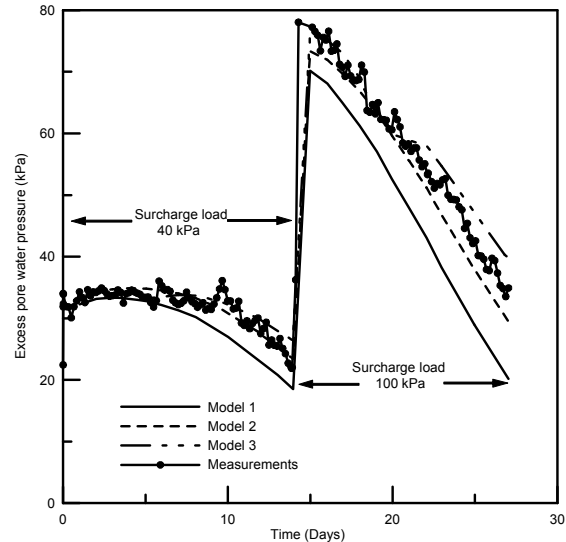


Fig. 7. Predicted and measured excess pore water pressure from the consolidometer [17]

The predicted and measured excess pore water pressures are presented in Fig. 7. Models 2 and 3 agree well with the laboratory observations. Model 3 gives the highest pore pressures, suggesting that the unsaturated soil-drain boundary causes a delay in the dissipation of excess pore water pressure. In view of both settlements (Fig. 6) and excess pore pressures (Fig. 7), Model 3 provides the most accurate predictions in comparison with the

laboratory measurements. There is no doubt that the probable existence of drain unsaturation at least at the start of consolidation is an important aspect that should be captured in numerical modeling.

3.3. Discharge Capacity

The discharge capacity is one of the most important parameters that controls the performance of PVDs. The discharge capacity depends primarily on the following factors: (i) the area of the drain; (ii) the effect of lateral earth pressure; (iii) possible folding, bending and crimping of the drain and (iv) infiltration of fine particles into the drain filter, as shown in Fig. 8.

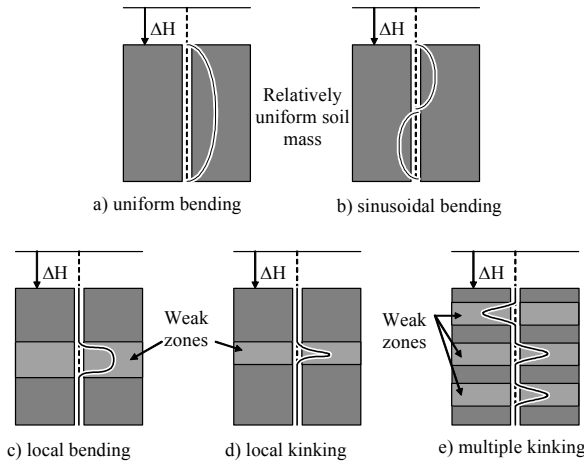


Fig. 8. Deformation modes of PVDs (after [20])

4. EQUIVALENT PLANE STRAIN THEORY FOR SOFT SOIL IMPROVED BY PVDS

Plane strain multi-drain analysis can be used to predict soft soil behavior based on an appropriate conversion procedure. Indraratna and Redana [19, 21] and Indraratna et al. [22] converted the vertical drain system into an equivalent parallel drain wall by adjusting the coefficient of permeability of the soil, and by assuming a plane strain cell width of $2B$. The half width of the drain b_w and half width of the smear zone b_s may be equaled to their axisymmetric radii r_w and r_s , respectively, implying $b_w = r_w$ and $b_s = r_s$.

Making the magnitudes of R and B also to be the same, Indraratna and Redana [19] presented a relationship between k_{hp} and k'_h . The influence of smear effect can be modelled by the ratio of the smear zone permeability to the undisturbed permeability, as follows:

$$\frac{k'_{hp}}{k_{hp}} = \frac{\beta}{\frac{k_{hp}}{k_h} \left[\ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k'_h}\right) \ln(s) - 0.75 \right] - \alpha} \quad (8)$$

If smear and well resistance effects are ignored in the above expression, then the simplified ratio of plane strain to axisymmetric permeability is readily obtained, as also proposed earlier by Hird et al. [23], as follows:

$$\frac{k'_{hp}}{k_h} = \frac{0.67}{[\ln(n) - 0.75]} \quad (9)$$

For vacuum preloading, the equivalent vacuum pressures in plane strain and axisymmetric are the same.

5. APPLICATION OF NUMERICAL MODELLING IN PRACTICE AND FIELD OBSERVATION

Indraratna et al. [11,24], Indraratna & Redana [19] and Indraratna et al. [22] made an attempt to analyze the performance of 3 embankments constructed on soft clay, one built until failure on the Muar plain, Malaysia, one stabilized with PVDs only and the other stabilized with both PVDs and vacuum preloading, Thailand. At these sites, the soft clay is mostly of marine, lagoonal or deltaic origin characterized by high compressibility, very low permeability and low shear strength. Therefore, the ground improvement techniques are necessary to prevent excessive and differential settlements in the field.

5.1. Performance of Test Embankment built to Failure on Muar Clay

The sub-soil profiles and corresponding properties are shown in Fig. 9. The subsoil consists of a topmost weathered crust (1.5-2 m depth) underlain by soft to very soft clay layer up to 20 m deep. Underneath the soft clay layer, dense silty sand layer can be found at 20-24 m depth. The unit weight of soil is between 15-17 kN/m³. The undrained shear strength was minimum at a depth of 3 m ($C_u \cong 8$ kPa), increasing linearly with depth. Extensive laboratory tests were also conducted, including oedometer, Unconsolidated Undrained (UU) and Consolidated Undrained (CU) triaxial tests.

The failure of the embankment and foundation was initiated by a “quasi slip circle” type of rotational failure at a critical embankment height of 5.5 m, with a tension crack propagating vertically through

the crust and the fill (Fig. 10). Indraratna et al. [24] analysed the performance of the embankment using 2D finite element analysis incorporating two different constitutive soil models, namely, the hyperbolic stress-strain behavior using the finite element code ISBILD [26] and the Modified Cam-clay theory using the finite element program CRISP [27].

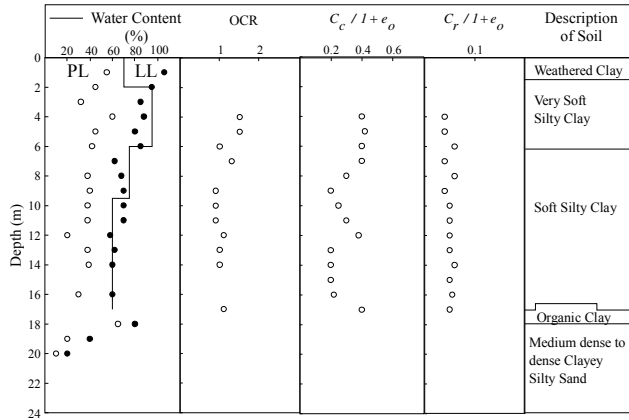


Fig. 9. In-situ soil properties at the site of Muar plain [25]

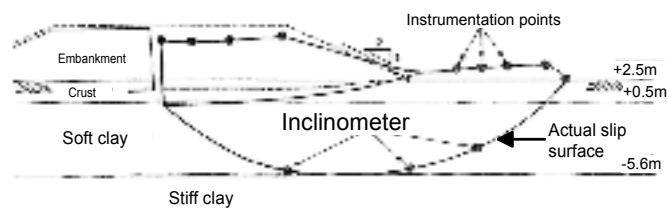


Fig. 10. Failure pattern of embankment and foundation (modified after [28])

The modes of analysis could be divided into two types: undrained and coupled consolidation. For the undrained condition, excess pore pressures do not have adequate time to dissipate and they build up during loading while the volumetric strain is zero. For the coupled consolidation analysis, the excess pore pressures are generated simultaneously with drainage and volume change (positive or negative). Total stress may also change during loading.

Soil parameters used for the Modified Cam-clay model (MCC) in CRISP program are shown in Table 2, which also summarizes the values of the bulk modulus (K_w), and the coefficients of horizontal and vertical permeabilities (k_h and k_v). A summary of soil parameters applicable for undrained and drained analyses is given in Table 3. For the embankment surcharge ($E = 5100$ kPa, $\nu = 0.3$ and $\gamma = 20.5$ kN/m³), the shear strength

parameters ($c' = 19$ kPa and $\phi' = 26^\circ$), were obtained from drained triaxial tests.

Table 2. Modified Cam-clay Soil parameters used in CRISP (Source: [24])

Depth (m)	κ	λ	$K_w \times 10^4$ (cm/s)	$k_h \times 10^{-9}$ (m/s)	$k_v \times 10^{-9}$ (m/s)
0-2.0	0.05	0.13	4.4	1.5	0.8
2.0-8.5	0.05	0.13	1.1	1.5	0.8
8.5-18	0.08	0.11	22.7	1.1	0.6
18-22	0.10	0.10	26.6	1.1	0.6

Table 3. Soil parameters for hyperbolic stress strain model for ISBILD (Source: [24])

Depth (m)	K	c_u (kPa)	K_{ur}	c' (kPa)	ϕ' (degree)	γ (kN/m ³)
0-2.5	350	15.4	438	8	6.5	16.5
2.5-8.5	280	13.4	350	22	13.5	15.5
8.5-18.5	354	19.5	443	16	17.0	15.5
18.5-2.5	401	25.9	502	14	21.5	16.0

Note: K and K_{ur} are modulus number and unloading-reloading modulus number used to evaluate compression and recompression behavior of soil, respectively.

At this site, various instruments were installed including piezometers, inclinometers and settlement plates (Fig. 11). Excess pore pressure variations beneath the embankment, lateral and vertical displacements and mobilized shear stress contour at failure were obtained from the two finite element analyses. Some of the results obtained are described below.

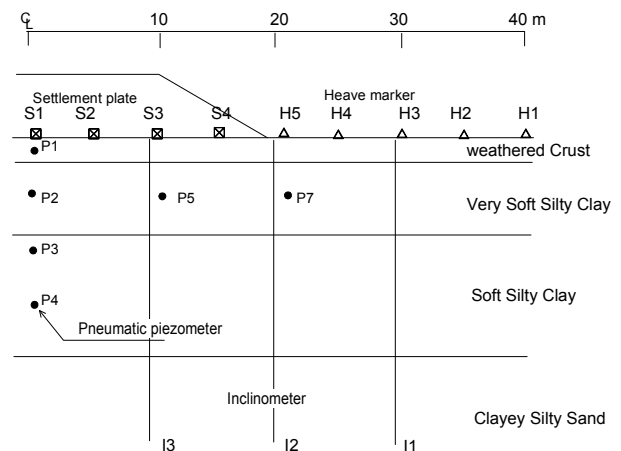
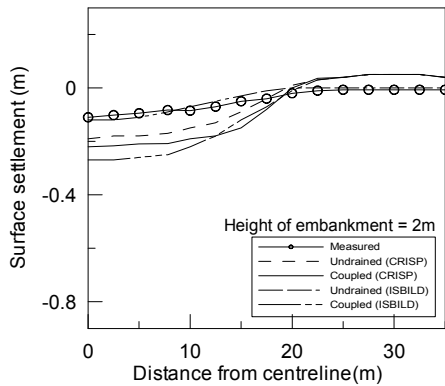


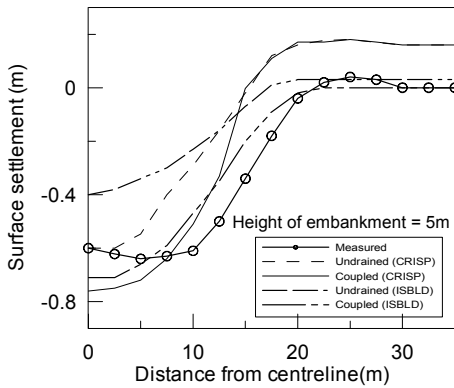
Fig. 11. Vertical cross-section of Muar embankment indicating the location of instruments (modified after [29])

The predicted and measured surface settlements for various fill heights (2 and 5 m) are compared in Fig. 12. For the predictions at the initial fill height (2 m), the undrained prediction by ISBILD agrees well with the field value, except for the area near the

centerline of the embankment, whereas the other predictions generally overestimate the vertical settlement. When the fill height is more than 2 m, the maximum measured vertical settlement is observed at a lateral distance 8-10 m away from the centerline, rather than at the centerline. At the failure height (Fig. 12b) the undrained analysis using the hyperbolic stress-strain model gives a better agreement with the field measurements.



(a)



(b)

Fig. 12. Surface settlement profiles for 2 and 5m fill heights (data from [24])

Figure 13 shows the variation of lateral displacements for the location I3 for both the MCC and the hyperbolic stress-strain models at the critical embankment height (5.5 m). As expected, the maximum lateral displacement occurs at a depth of 5 m below the ground surface in the upper very soft clay layer. The predictions from the modified Cam-clay agree with the measured results in the upper soft clay layer, whereas they overpredict the field behavior at greater depths.

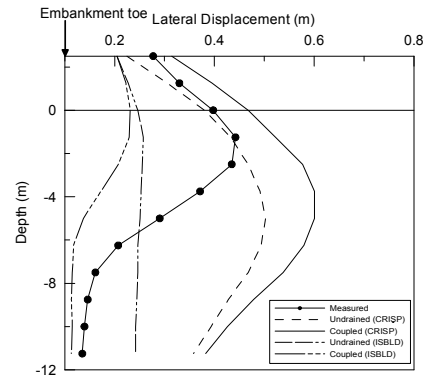


Fig. 13. Lateral displacement profile at failure (modified after [24])

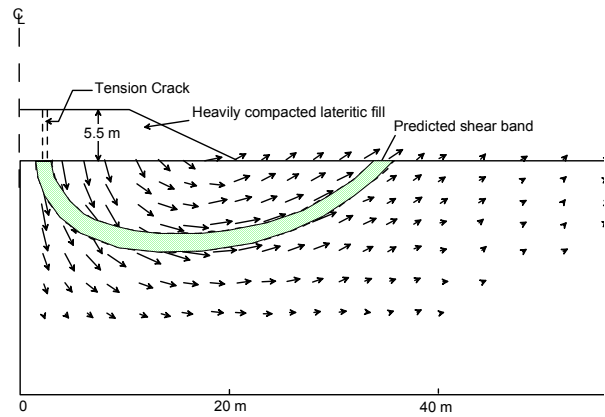


Fig. 14. Maximum incremental displacement vectors at failure (modified after [24])

The zones of yielding and potential failure surface are determined based on the boundaries of yielded zone and maximum displacement vectors using the coupled consolidation (CRISP) (Figs. 14 and 15). Each contour indicates the current field height. The yielded zone can be found close to the bottom of the soft clay layer and subsequently progresses towards the centerline of the embankment. The actual failure surface is located within the predicted shear band.

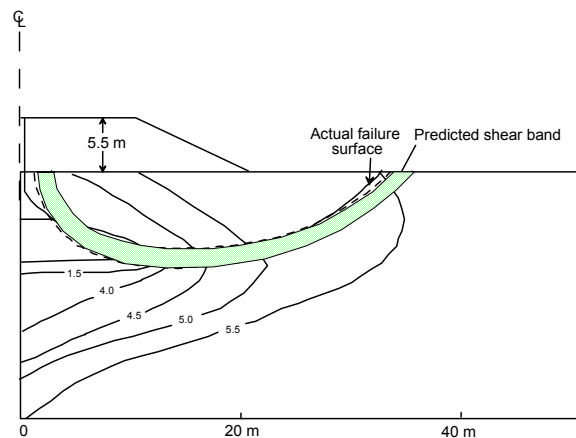


Fig. 15. Boundary zones approaching critical state with increasing fill thickness (modified after [24])

5.2. Performance of Test Embankment Stabilized with Vertical Drains on Soft Clay, Thailand
 Indraratna & Redana [19] and Indraratna et al. [22] analyzed the performance of soft ground improvement by vertical drains at the Second Bangkok International Airport (SBIA), Thailand. PVDs with and without vacuum preloading were employed beneath 2 embankments, namely, TS1 and TV2, respectively. PVDs were installed 12 m deep in a triangular pattern with 1.0 m spacing. The constant values of k_h/k_s and d_s/d_w were assumed to be 2 and 6, respectively. For the plane strain FEM simulation, the equivalent permeability inside and outside the smear zone and vacuum pressure were determined using Equations (17)-(19). The well resistance was neglected [19]. The finite element mesh discretization contained 8-node bi-quadratic displacement elements with bilinear pore pressure shape functions (Fig. 16).

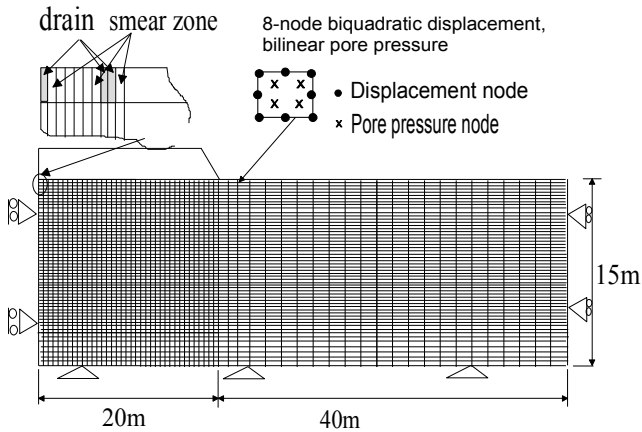


Fig.16. FE mesh of embankment for plane strain analysis at 2nd Bangkok Int. Airport (after [22])

The predictions of ground settlement at the embankment centerline are shown in Fig. 17. The time required to achieve the desired settlement can be reduced from 200 days to 120 days, if the vacuum pressure is applied together with a surcharge load. This is because, the embankment construction together with vacuum pressure application would not involve several construction stages as in the case of surcharge alone [24]. Figure 18 shows the time-dependent excess pore water pressure. As expected, the vacuum loading generates negative excess pore pressures, thereby minimizing the risk of any shear failure.

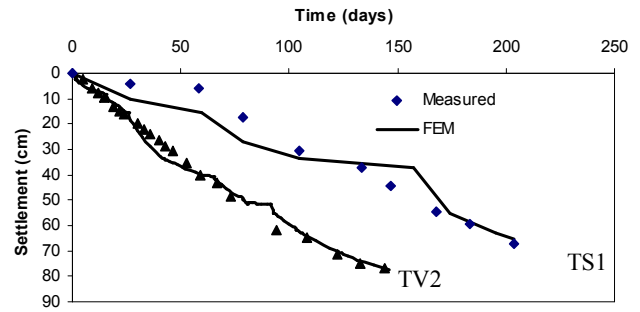


Fig.17. Surface settlement at the centre-line for embankments (after [19,22])

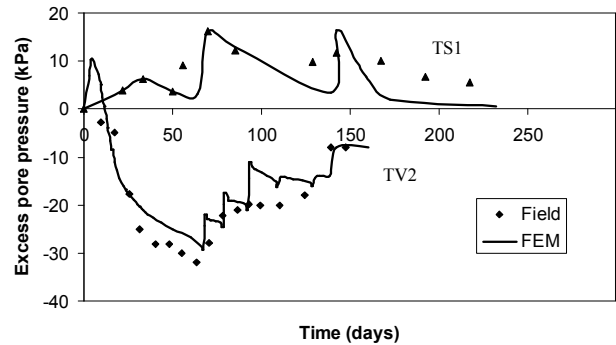


Fig. 18. Excess pore pressure predictions (after [19,22])

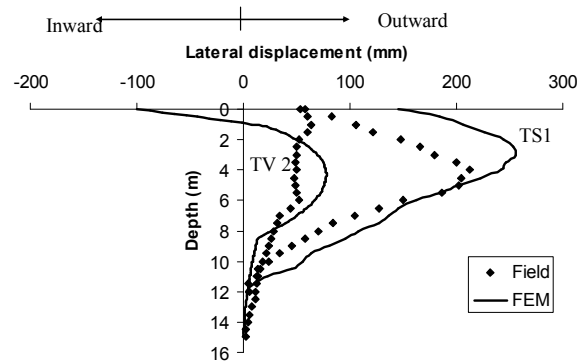


Fig. 19. Lateral displacements at the end of construction (after [19,22])

The comparisons between predicted and measured lateral movement at the end of construction for embankments TS1 and TV2 are illustrated in Fig. 19. The plane strain FEM model provides a good prediction of the lateral displacement beneath the embankment TS1. For the embankment TV2, the predicted lateral movement agrees with the field data at a depth below 4 m, whereas the discrepancies between the predicted and measured results occur mainly at the weathered surface crust (about 0-2 m depth). Comparison between the cases of with and without vacuum pressure indicates that

vacuum preloading significantly causes an inward lateral movement of soil towards the embankment centerline. In Fig.19, the stiffness of the compacted crust at TV2 is not properly modeled in the FEM analysis, hence the significant deviation from the field data observed close to the surface.

5.3. Effect of Soil Viscosity on Excess Pore

Pressure Dissipation and Lateral Deformation

It has been noted in some case studies that in spite of using PVDs, excess pore water pressures do not always dissipate as expected. This is often attributed to filter clogging, excessive reduction of the lateral permeability of the soil surrounding the drains, damage to piezometer tips etc. However, recent numerical analysis suggests that very high lateral strains and associated stress redistributions (e.g. substantial heave at the embankment toe) can also contribute to the retarded rate of pore pressure dissipation. Some examples are shown in Figs. 20 and 21.

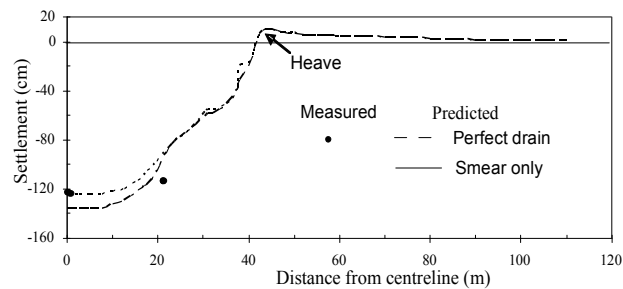


Fig. 20. Surface settlement profile after 400 days ([19,30])

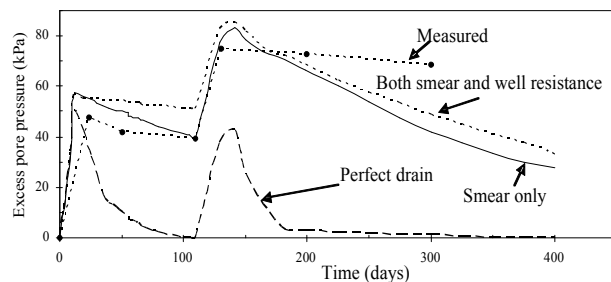


Fig. 21. Excess pore water pressure variation at piezometer location, P6 (after [19,30])

In order to investigate the effect of secondary compression (creep) during the consolidation, the performance of a full-scale test embankment constructed on soft clay with PVDs at the Sunshine Motorway, Queensland, Australia was analyzed [31]. The data was provided by the Queensland Department of Main Roads, as a part of Australian Research Council collaboration (32). Subsoil layer at this site composes of very soft, highly compressible, saturated organic marine clays of

high sensitivity. An instrumented trial embankment was constructed with three different ground improvement schemes (i.e. Section A: PVDs @ 1m spacing, Section B: No PVDs and Section C: PVDs @ 2m spacing). In this study, only Section C is analysed using the numerical analysis (PLAXIS) incorporating Soft Soil Creep (SSC) and Modified Cam-Clay (MCC) models [33, 34].

The adopted parameters of 3 subsoil layers obtained from the consolidation tests are listed in Table 1. The modified creep index (μ^*) was determined using oedometer test at the end of primary consolidation. The compression parameters (λ^* and κ^*) for both SSC and MCC models are assumed to be the same.

Table 4 Soil elastoplastic parameters used in the numerical analysis

Depth (m)	0.0-2.5	2.5-5.5	5.5-11.0
Soil type	Silty clay	Soft silty clay	Silty clay
M (assumed)	0.98	1.20	1.18
λ^*	0.27	0.48	0.26
κ^*	0.027	0.048	0.026
ν	0.3	0.3	0.3
e_0	1.85	3.10	1.75
μ^*	0.012	0.02	0.012
k_h ($\times 10^{-9}$ m/day)	14.65	8.17	6.31
γ_s (kN/m^3)	16.4	13.7	15.9

Figure 22 shows the comparison of the surface settlement for MCC and SSC models with the field measurement. Numerical results from both models generally agree well with the field measurements. Figure 23 compares the excess pore pressures between the field and numerical data. The predictions obtained from the SSC model are better than the MCC model after 50 days. The phenomenon of undissipated excess pore pressure during the ongoing settlement can be captured by the SSC model. This is because, at a given mean effective stress, the viscous nature of clay causes additional soil compression and the excess pore pressure for a certain period of time.

Figure 24 presents the comparisons of the lateral displacement at the embankment toe. The SSC model gives better predictions, whereas the MCC model over-predicts the lateral yield. Undoubtedly,

the inclusion of creep model provides a better prediction of the excess pore pressure and lateral displacement.

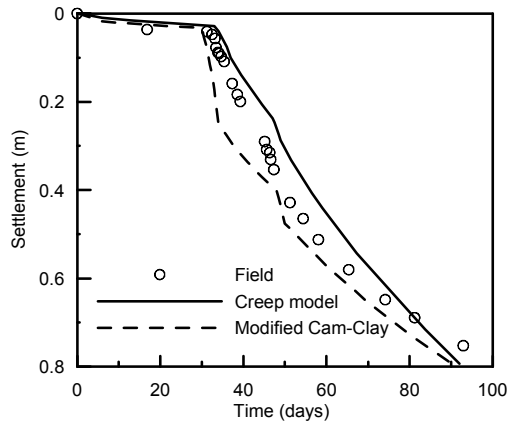


Fig.22. Surface settlements at the embankment centerline

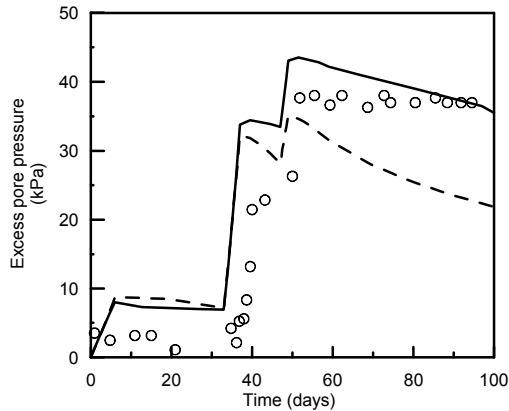


Fig. 23. Variation of excess pore water pressure at 3m deep below the surface and 1.0m away from centerline

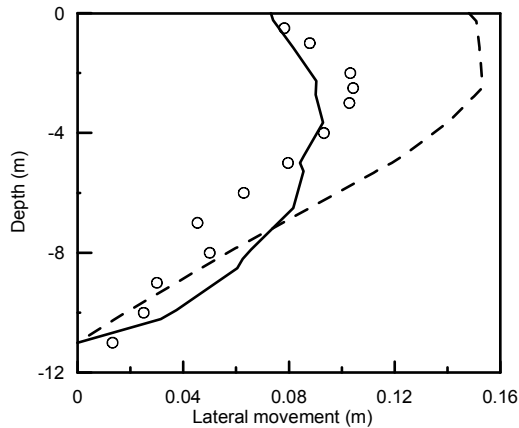


Fig. 24. Predicted and measured lateral displacements at the embankment toe

6. BEHAVIOUR OF SHORT VERTICAL DRAINS SUBJECTED TO CYCLIC TRAIN LOADS

Soft soil deposits can sustain high excess pore water pressures during both static and cyclic (repeated) loading. The performance of prefabricated vertical drains (PVD) for dissipating cyclic pore water pressures is discussed. For very low permeable

soils, the increase in pore pressures will decrease the effective load bearing capacity of the formation. Even if the rail tracks are well built structurally, undrained failures can impede the train speeds apart from the operational delays. Under circumstances of high excess pore water pressures, clay pumping may clog the ballast causing poor drainage.

As described earlier, PVDs speed up consolidation and curtail lateral movements. The stability of rail tracks and highways built on soft saturated clays is often controlled by the magnitude of lateral strains, even though a gain in shear strength and load bearing capacity can be increased during consolidation. If excessive settlement of deep estuarine deposits cannot be tolerated in new railway tracks, continuous ballast packing may be required. However, the rate of settlement can still be controlled by optimising the drain spacing, drain length and the drain installation pattern. In this way, while the settlements are acceptable, the reduction in lateral strains and gain in shear strength of the soil beneath the track improve its stability significantly.

6.1. Laboratory testing

A large-scale triaxial test (300 mm diameter and 600 mm high) was used to study the effect of cyclic load on the radial drainage and consolidation by PVDs (Fig. 22). The excess pore water pressure was monitored via miniature pore pressure transducers, saturated under deaired water with vacuum pressure. A remoulded estuarine clay was tested. Most soft clays will have natural water contents approximately 75% and a Plasticity Index above 35%. The very soft undisturbed clays in Northern Queensland have typical c_u values less than 7-8 kPa.



Fig. 22. Large-scale triaxial apparatus (a) Large-scale triaxial rig, (b) soil specimen

The tests were conducted at frequencies of 5-7 Hz, typically simulating average train speeds of 60-100 km/h typical of 25-30 tonnes/axle train loads. Fig. 23 indicates that the maximum excess pore water pressure beside the PVD during the cyclic load application (T4) is significantly less compared to that near the cell boundary (T3). Also as expected, the dissipation of excess pore pressures close to the outer cell boundary (e.g. T1 and T3) is slower than that of T4 and T2 closer to the PVD. The test results reveal that PVDs reduce the generated maximum excess pore pressure effectively even under cyclic loading.

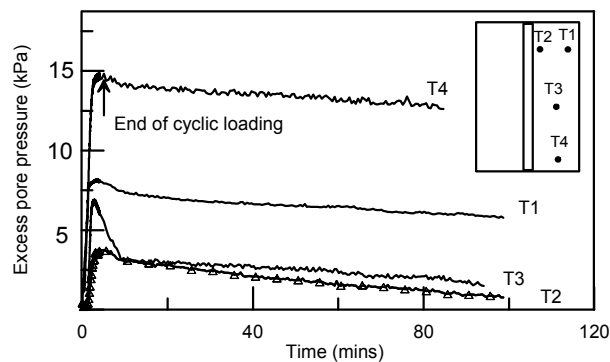


Fig. 23. Measured excess pore pressure Dissipation at various locations from the PVD

7. CONCLUSIONS

The use of prefabricated vertical drains, their properties and associated merits and demerits have been discussed. The extent of smear zone can be predicted by the cavity expansion theory and validated by large scale laboratory tests based on the variation of water content and lateral permeability of the soil. It is found that soil unsaturation at the vertical drain boundary due to mandrel driving could delay the excess pore pressure dissipation during the early stages of the consolidation process. The behavior of soft clay under the influence of PVD and vacuum application was described on the basis of selected case histories where both field measurements and predictions were available.

A conversion procedure based on the transformation of permeability and vacuum pressure was introduced to establish the relationship between the axisymmetric (3D) and equivalent plane strain (2D) conditions. The plane strain solution was applied for case history analysis, proving its reliability in predicting the soil behavior. Field behavior as well as model predictions confirm that the expected

efficiency of the vertical drains also depends on the magnitude and distribution of the vacuum pressure.

An accurate prediction of lateral displacement depends on the careful assessment of soil properties including the overconsolidated surface crust. This compacted layer is relatively stiff, and therefore it resists ‘inward’ movement of the soil upon vacuum application. Clearly, the modified Cam-clay model is inappropriate for modeling the behavior of a weathered and compacted crust. To improve the accuracy of the prediction, the creep nature needs to be included to simulate the delayed excess pore pressure dissipation during consolidation. An analysis of the case histories showed that the vacuum application via PVD substantially decreases lateral displacement, thereby reducing potential shear failure during rapid embankment construction.

There is no doubt that a system of vacuum-assisted consolidation via PVD is a practical approach for accelerating the radial consolidation. Such a system eliminates the need for placing a high surcharge load, as long as air leaks in the field can be prevented using effective membranes. Accurate modeling of vacuum preloading requires laboratory and field studies to investigate the exact nature of vacuum pressure distribution within a given soil formation and PVD system. In addition, a resilient system is required to prevent air leaks that can reduce the desirable negative pressure (suction) with time.

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