Finite element modelling of marine clay deformation under reclamation fills

A. ARULRAJAH*, H. NIKRAZ† and M. W. BO‡

Connell Wagner Pty Ltd, Melbourne, Australia; †Department of Civil Engineering, Curtin University of Technology, Perth, Australia; ‡Faber Maunsell, Bradford, UK (formerly Bullen Consultants Ltd, Bradford, UK)

The ongoing Changi East Reclamation Project in Singapore consists of land reclamation and ground improvement of the foreshore for the future expansion of Changi International Airport. The deformation behaviour of marine clay under reclamation fills and surcharge was modelled by the finite element method (FEM) with Plaxis numerical modelling software. The analyses included modelling the consolidation behaviour of marine clay under reclamation fills with and without prefabricated vertical drains. Marine clay with vertical drains was modelled by both the axisymmetric unit cell and full-scale analysis methods. Marine clay that was not treated with prefabricated vertical drains was modelled by means of full-scale analysis. The numerical analysis was carried out at two case study locations: the pilot test site and the *in situ* test site. The test sites comprise vertical drain treated and untreated sub-areas that were reclaimed and preloaded under the same conditions. The result of the FEM analysis was compared with that obtained by means of observational methods. The vertical drain performance was verified for the *in situ* test site by using the value of C_h obtained from back-analysis by the Asaoka method and by FEM with Plaxis.

Keywords: field instrumentation; finite element modelling; ground improvement; land reclamation; marine clay; prefabricated vertical drains

Le Projet de Changi East qui se poursuit actuellement à Singapour consiste à récupérer des terres sur la mer et à améliorer le sol du littoral en vue de l'agrandissement futur de l'aéroport international de Changi. Le comportement de déformation de l'argile marine sous les remblais de récupération a été modélisé par la méthode d'éléments finis (FEM) avec le logiciel de modélisation numérique Plaxis. Les analyses comprenaient la modélisation du comportement de consolidation de l'argile marine sous les remblais de récupération avec et sans drains verticaux préfabriqués. L'argile marine avec drains verticaux a été modélisée à la fois par cellule unitaire axisymétrique et par des méthodes d'analyse grandeur nature. L'argile marine qui n'a pas été traitée avec des drains verticaux préfabriqués a été modélisée au moyen d'analyses de grandeur nature. Des analyses numériques ont été effectuées dans deux endroits d'étude : les sites d'essai pilote et les sites d'essai *in situ*. Les sites d'essai sont composés de sous-zones traitées et non traitées par des drains verticaux, zones qui ont été récupérées et préchargées dans les mêmes conditions. Nous avons comparé les résultats des analyses FEM avec celles obtenues au moyen de méthodes observationnelles. La performance du drain vertical a été vérifiée pour le site d'essai *in situ* en utilisant la valeur de C_h venant d'une rétro-analyse par la méthode Asaoka et par FEM avec Plaxis.

Notation

C_v coefficient of consolidation due to vertical flow
C_{vi} assumed coefficient of consolidation due to vertical flow
C'_h effective value of coefficient of consolidation due to horizontal flow
c_{ref} reference cohesion
d equivalent drain diameter
d_e equivalent diameter of cylinder of soil around drain
E_{oed} oedometer modulus
E_{ref} reference Young's modulus
G_{ref} reference shear modulus
H_0 thickness of layer
H'_{Ti} equivalent total thickness of marine clay layers
k permeability
k_e equivalent horizontal permeability of surrounding soils
k_h horizontal permeability of undisturbed soil
k_{hax} horizontal permeability of undisturbed zone in axisymmetric unit cell
k_{hpl} horizontal permeability of undisturbed zone in plane-strain unit cell
k_s horizontal permeability of soil within smear zone
k_{sax} horizontal permeability of smear zone in axisymmetric unit cell
k_v vertical permeability of undisturbed soil
l_m length of vertical drain
n_i influence ratio
q_w discharge capacity of vertical drain
R well resistance factor
r_e radius of influence zone
r_s radius of smear zone
r_w equivalent radius of vertical drain
S_t settlement at any point of time, t
S_{ult} ultimate final settlement
t time elapsed since application of surcharge
γ soil unit weight
κ^* modified swelling index
λ^* modified compression index

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ν Poisson's ratio
ϕ friction angle
ψ dilatancy angle

Introduction

Ground improvement works in the ongoing Changi East Reclamation Project in the Republic of Singapore comprise the installation of prefabricated vertical drains (PVDs) and the subsequent placement of sand surcharge to accelerate the consolidation of the underlying soft marine clay. The location of the project and the various phases of land reclamation have been explained by Bo *et al.* (2003) and Choa *et al.* (2001). In such ground improvement projects in soft marine clay, the degree of improvement attained by the clay has to be ascertained prior to surcharge removal, to confirm whether the soil has achieved the required degree of consolidation. This analysis can be predicted or back-analysed by means of finite element modelling (FEM), and subsequently the results can be compared with field settlement monitoring records. Bo *et al.* (2003) have discussed the use of vertical drains and their design methodology based on experiences in the Changi East Reclamation Project.

Two case study sites within the project were studied by the FEM method using Plaxis v. 8 numerical modelling software (Plaxis, 2002). The pilot test site consisted of vertical drains installed in sub-areas at various spacings, and an untreated control sub-area. The *in situ* test site consisted of a vertical drain sub-area and an untreated control area. The field settlement plate monitoring records were used for purposes of comparison with the FEM method at both test sites.

The results of the FEM analysis were compared with those obtained by means of observational methods at both test sites. The vertical drain performance was also verified for the *in situ* test site by using the value of C_h obtained from back-analysis by the Asaoka (1978) method, and by FEM with the numerical modelling software.

The consolidation behaviour of the marine clay and PVDs was modelled, using Plaxis v. 8, under reclamation fills with and without PVDs. Clay treated with vertical drains was modelled by both the axisymmetric unit cell and full-scale analysis methods. Clay that was not treated with PVDs was modelled by means of full-scale analysis.

Case study sites

In situ test site

The *in situ* test site comprised an area with vertical drains installed at 1·5 m square spacing and an adjacent control area with no vertical drains. The two areas were treated with the same height of surcharge preload. Instruments were installed and monitored at both areas. The instruments in the control area were installed prior to reclamation in offshore instrument platforms. These instruments were protected as the reclamation filling works commenced in the area. Instruments in the vertical drain area were installed on land at the vertical drain platform level of +4 mCD (Admiralty chart datum, where mean sea level is +1·6 mCD) just before or soon after vertical drain installation at 1·5 m square spacing. Surcharge was subsequently placed to +10 mCD. Instrumentation monitoring was carried out at both areas until 20 months after surcharge placement, which equates to a monitoring period of about 26 months. The assessment of the field instrumentation monitoring results at the *in situ* test site has been described by Bo *et al.* (1997a). Fig. 1 is a cross-sectional soil profile showing field instrument elevations at the site.

Pilot test site

The pilot test site comprised four sub-areas, three of which had vertical drains installed at various spacings. Long-duration field settlement monitoring was carried out at regular intervals at these sub-areas. The seabed elevation is about −6 mCD, and the soft marine clay in the location was up to 40 m thick. Land reclamation was first carried out to the platform level of +4 mCD. Field instruments were installed, prior to vertical drain installation, from the platform level where the drains were installed.

Following the installation of the drains, surcharge was placed hydraulically to an elevation of +7 mCD simulta-

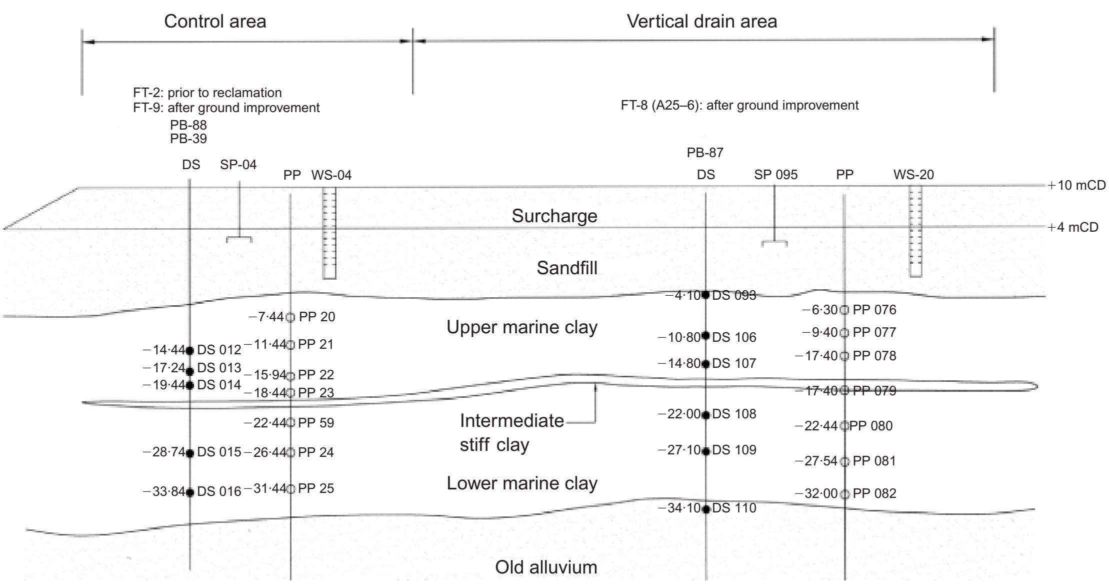

Fig. 1. Cross-sectional soil profile showing instrument elevations at *in situ* test site. SP, settlement plate; DS, deep settlement gauge; PP, pneumatic piezometer; WS, water standpipe

neously for all the sub-areas, so that the various sub-areas could be compared when subjected to the same surcharge preload. Field instrumentation monitoring was carried out for 32 months after surcharge placement, which equates to a total monitoring duration of about 42 months. The assessment of the field instrumentation monitoring results at the pilot test site has been described by Arulrajah *et al.* (2003).

Figure 2 shows the layout plan and vertical drain spacing of the sub-areas at the site, and Fig. 3 shows a profile of the field instrumentation.

Theory

In the modelling of vertical drains in Bangkok clay by Lin *et al.* (2000), the interface element was used with the same soil property as the adjacent soil except for its permeability. Furthermore, the conversion scheme for well resistance was achieved by using interface elements. Well resistance was automatically considered in interface element for axisymmetric and plane-strain unit cells by the equivalent discharge capacity of interface elements to that of vertical drains.

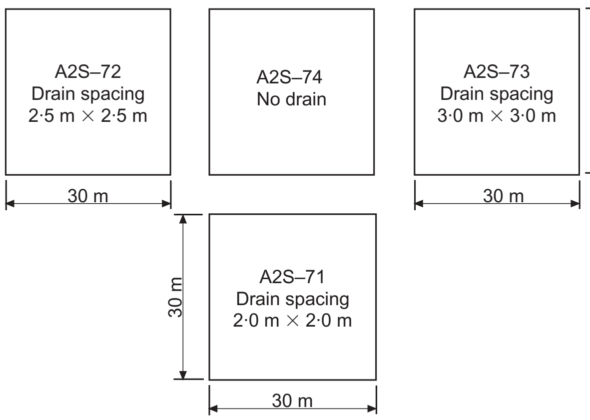

Fig. 2. Layout plan and vertical drain spacing of sub-areas at pilot test site

For the axisymmetric unit cell analysis of vertical drains in this study, the authors applied the method first proposed by Lin *et al.* (2000) in considering the smear effect by using the equivalent horizontal permeability of the surrounding soil. However, the method was modified to incorporate the marine clay multi-layers present at Changi.

The conversion scheme from axisymmetric to plane-strain conditions as proposed by Lin *et al.* (2000) was used for the full-scale analysis. For modelling PVDs in the full-scale analysis method in this study, the authors used the drain element of Plaxis v. 8.

It is necessary to consider the smear effect for the consolidation rate of vertical drain treated ground with finite permeability. This effect occurs because of the installation of vertical drains, which disturbs the soil surrounding the mandrel. The resulting smear zone depends on the shape of the mandrel and the anchor rod, and on the method of installation. Bergado *et al.* (1992) has verified the diameter of the smear zone to be twice the equivalent cross-sectional area of the mandrel for soft Bangkok clay.

Because a PVD has a limited discharge capacity, the effect of well resistance varies with the permeability of the surrounding soils, the discharge capacity, and the length of the vertical drain drainage path. Consequently the well resistance may affect the distribution of excess pore water pressure with depth and distance from the vertical drain during consolidation. The contribution of well resistance is minimal for such long lengths of vertical drains, and can be ignored in the numerical modelling analyses. Lin *et al.* (2000) state that previous analysis of field performance of vertical drains in soft clay deposits has indicated that well resistance is negligible when the well resistance factor R, as defined in the following equation, is greater than 5:

$$R = \frac{q_\mathrm{w}}{k_\mathrm{h} l_\mathrm{m}^2} \tag{1}$$

where q_w is the discharge capacity of the vertical drain, k_h is the horizontal permeability of the undisturbed soil, and l_m is the length of the vertical drain.

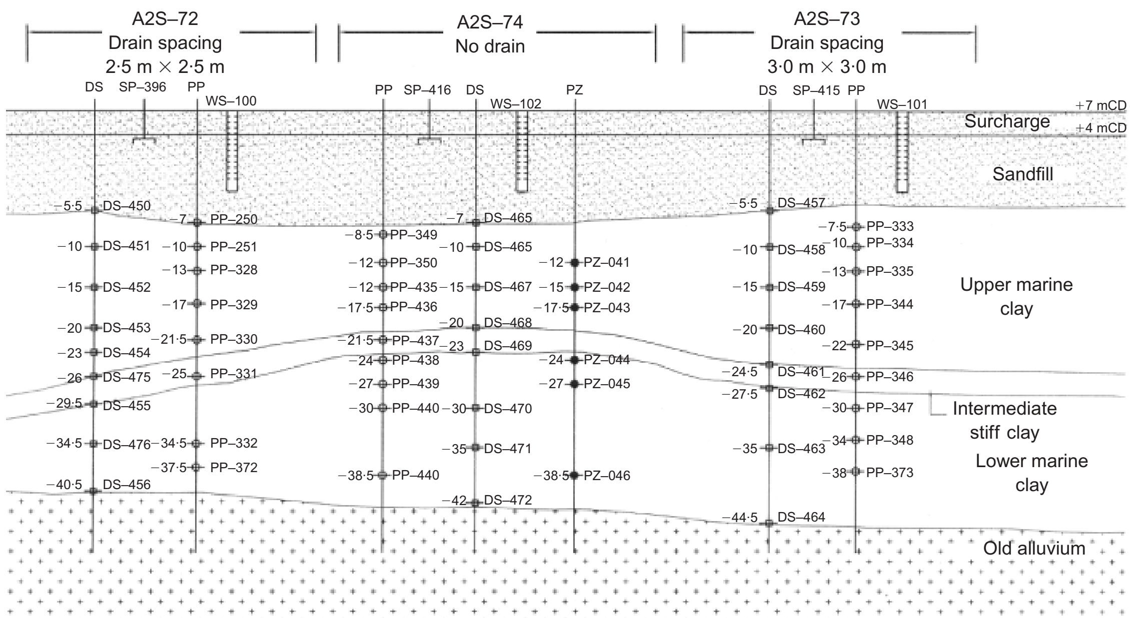

Fig. 3. Cross-sectional soil profile showing instrument elevations at pilot test site. SP, settlement plate; DS, deep settlement gauge; PP, pneumatic piezometer; WS, water standpipe

Axisymmetric unit cell analysis of PVDs

The vertical drains installed in the test sites were modelled in an axisymmetric unit cell analysis with Plaxis v. 8. The smear effect was considered by using the equivalent horizontal permeability of the surrounding soils, k_e (Lin *et al.*, 2000), which is defined as

$$k_e = \frac{k_h \ln(r_e/r_w)}{\ln(r_e/r_s) + (k_h/k_s)\ln(r_s/r_w)} \quad (2)$$

where r_e is the radius of the influence zone, r_w is the equivalent radius of the vertical drain, r_s is the smear effect radius, k_h is the horizontal permeability of the undisturbed soil, and k_s is the horizontal permeability of the soil within the smear zone.

The equivalent horizontal permeability of the surrounding soil was taken as twice that of the equivalent vertical permeability. Fig. 4 shows schematically the conversion of the axisymmetric unit cell from undisturbed marine clay with smear zone to that of the equivalent horizontal permeability of surrounding soils. Table 1 indicates the soil data parameters used for the FEM of vertical drains by the axisymmetric unit cell analysis.

Full-scale analysis of PVDs

The drain element available in the Plaxis v. 8 finite element program was used to model the PVDs for the vertical drain area at the *in situ* test site by the full-scale analysis method. This method uses the open consolidation boundary condition at which the excess pore water pressure is set to zero during the consolidation process in all nodes that belong to a drain.

The 6-node triangular element was adopted in the analysis. This provides second-order interpolation functions for displacement, and its stiffness matrix is evaluated by numerical integration using three integration points. In modelling the ground improvement, the following conditions were considered.

(*a*) Consolidation analysis was performed under 2-D plane-strain conditions.
(*b*) Marine clay layers were simulated by using the soft-soil model. This is based on stress-dependent stiffness, and allows for time-dependent behaviour. Failure behaviour is according to the Mohr–Coulomb criterion.
(*c*) The sandfill layer was simulated by using the Mohr–Coulomb model.

In the full-scale analysis finite element model, the permeability for an axisymmetric radial flow was converted to that of a plain strain flow with smear effect. In the FEM analysis, pore water flow in the plain strain unit cell is considered as 2-D plane-strain flow. The conversion from radial flow of an axisymmetric unit cell to 2-D plane flow of continuous drainage wall systems of plane-strain unit cell can be carried out by the method of Lin *et al.* (2000). The equivalent permeability of the marine clay with consideration for smear effect can be calculated by converting the axisymmetric unit cell to that of a plane-strain unit cell (Lin *et al.*, 2000) as follows

$$k_{hpl} = \frac{k_{hax}\pi}{6\left[\ln(n_i/s) + (k_{hax}/k_{sax})\ln(s) - 0{\cdot}75\right]} \quad (3)$$

where k_{hpl} is the horizontal permeability of the undisturbed zone in the plane-strain unit cell; k_{hax} is the horizontal permeability of the undisturbed zone in the axisymmetric unit cell; k_{sax} is the horizontal permeability of the smear zone in the axisymmetric unit cell; n_i is the influence ratio, given by r_e/r_w; and s is the smear ratio, given by r_s/r_w.

The equivalent horizontal permeability of the marine clay after applying the conversion from axisymmetric flow to plane-strain flow with smear effect consideration was used in the finite element analysis of the vertical drains for the full-scale analysis.

Table 2 indicates the soil data parameters used for the FEM of PVDs by full-scale analysis. Fig. 5 shows the deformed mesh, and Fig. 6 shows the extreme vertical displacements by the full-scale analysis for the vertical drain area at the *in situ* test site, 20 months after surcharge placement.

Full-scale analysis of untreated control embankments

In the full-scale numerical modelling of the untreated control embankments of the two sites, the 6-node triangular

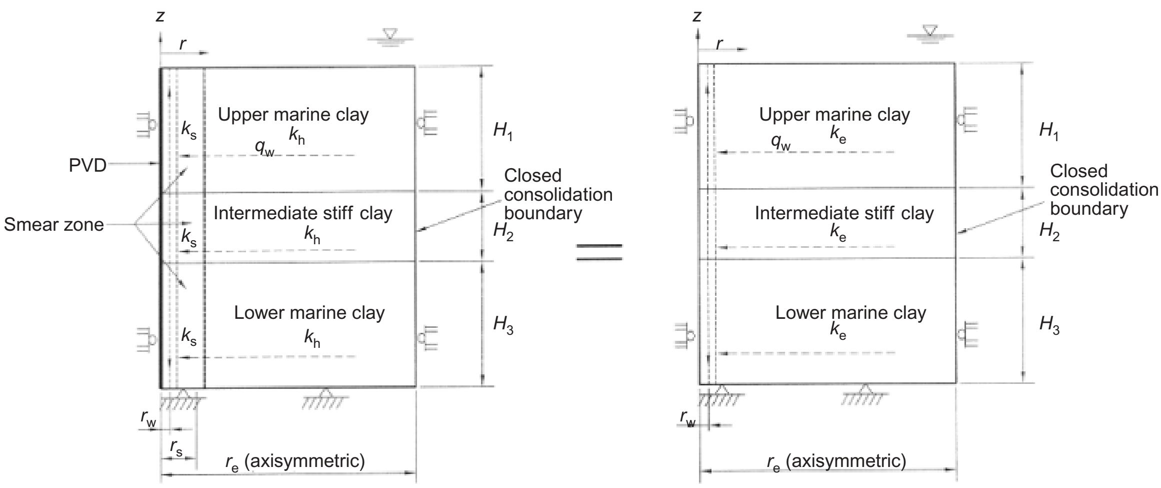

Fig. 4. Conversion of axisymmetric unit cell from undisturbed marine clay with smear zone to equivalent horizontal permeability of surrounding soils

Table 1. Soil parameters for axisymmetric unit cell analysis of PVD

Mohr–Coulomb	Reclamation sandfill		
Type	Drained		
γ_{unsat}: kN/m^3	17·00		
γ_{sat}: kN/m^3	20·00		
k_h: m/day	1·000		
k_v: m/day	1·000		
E_{ref}: kN/m^2	13 000·000		
ν	0·300		
G_{ref}: kN/m^2	5000·000		
E_{oed}: kN/m^2	17 500·000		
c_{ref}: kN/m^2	1·00		
ϕ: deg	31·00		
ψ: deg	0·00		
Soft soil	**Upper marine clay**	**Intermediate stiff clay**	**Lower marine clay**
Type	Undrained	Undrained	Undrained
γ_{unsat}: kN/m^3	15·00	15·00	15·00
γ_{sat}: kN/m^3	15·50	15·50	16·00
k_e: m/day	$2{\cdot}66 \times 10^{-5}$	$6{\cdot}25 \times 10^{-5}$	$2{\cdot}81 \times 10^{-5}$
k_v: m/day	$1{\cdot}33 \times 10^{-5}$	$3{\cdot}13 \times 10^{-5}$	$1{\cdot}41 \times 10^{-5}$
λ^*	0·150	0·060	0·170
κ^*	0·018	0·011	0·025
c: kN/m^2	1·00	1·00	1·00
ϕ: deg	27·00	32·00	27·00
ψ: deg	0·00	0·00	0·00
ν_{ur}	0·150	0·150	0·150
K_0^{nc}	0·55	0·47	0·55

Table 2. Soil parameters for full-scale analysis of PVD

Mohr–Coulomb	Reclamation sandfill		
Type	Drained		
γ_{unsat}: kN/m^3	17·00		
γ_{sat}: kN/m^3	20·00		
k_h: m/day	1·000		
k_v: m/day	1·000		
E_{ref}: kN/m^2	13 000·000		
ν	0·300		
G_{ref}: kN/m^2	5000·000		
E_{oed}: kN/m^2	17 500·000		
c_{ref}: kN/m^2	1·00		
ϕ: deg	31·00		
ψ: deg	0·00		
Soft soil	**Upper marine clay**	**Intermediate stiff clay**	**Lower marine clay**
Type	Undrained	Undrained	Undrained
γ_{unsat}: kN/m^3	15·00	15·00	15·00
γ_{sat}: kN/m^3	15·50	15·50	16·00
k_{hpl}: m/day	$4{\cdot}67 \times 10^{-6}$	$1{\cdot}10 \times 10^{-5}$	$4{\cdot}95 \times 10^{-6}$
k_v: m/day	$2{\cdot}34 \times 10^{-6}$	$5{\cdot}50 \times 10^{-6}$	$2{\cdot}48 \times 10^{-6}$
λ^*	0·150	0·060	0·170
κ^*	0·018	0·011	0·025
c: kN/m^2	1·00	1·00	1·00
ϕ: deg	27·00	32·00	27·00
ψ: deg	0·00	0·00	0·00
ν_{ur}	0·150	0·150	0·150
K_0^{nc}	0·55	0·47	0·55

element was adopted in the analysis. The horizontal permeability of the undisturbed soil, k_h, was taken as twice the vertical permeability of the undisturbed soil, k_v, based on the properties of Singapore marine clay

$$k_h = 2k_v \tag{4}$$

Table 3 indicates the soil data parameters used. These were obtained from laboratory tests on marine clay (Bo *et al.*, 2003). Fig. 7 shows the deformed mesh, and Fig. 8 shows the extreme vertical displacements by the full-scale analysis for the untreated control area at the *in situ* test site, 20 months after surcharge placement.

Comparison of FEM results

In situ test site

Table 4 and Fig. 9 show the comparison of the actual field settlement and the results of the Plaxis v. 8 numerical modelling method for the *in situ* test site. Fig. 10 illustrates

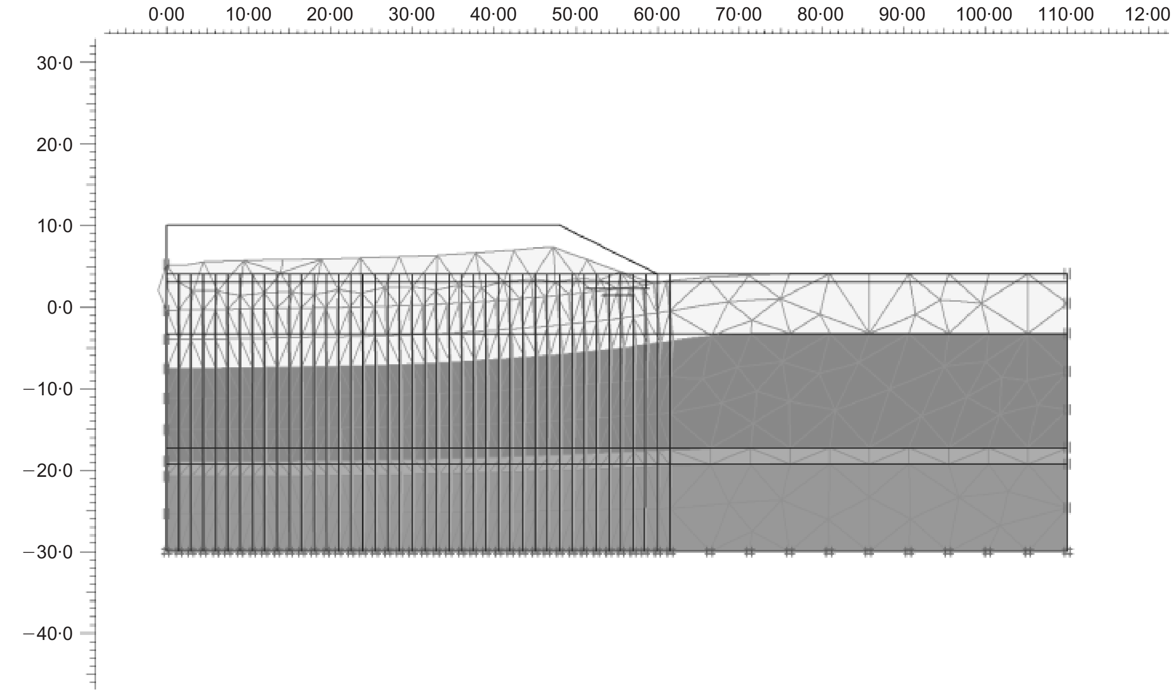

Fig. 5. Deformed mesh by full-scale analysis of vertical drain area (1·5 m × 1·5 m) at *in situ* test site, 20 months after surcharge placement

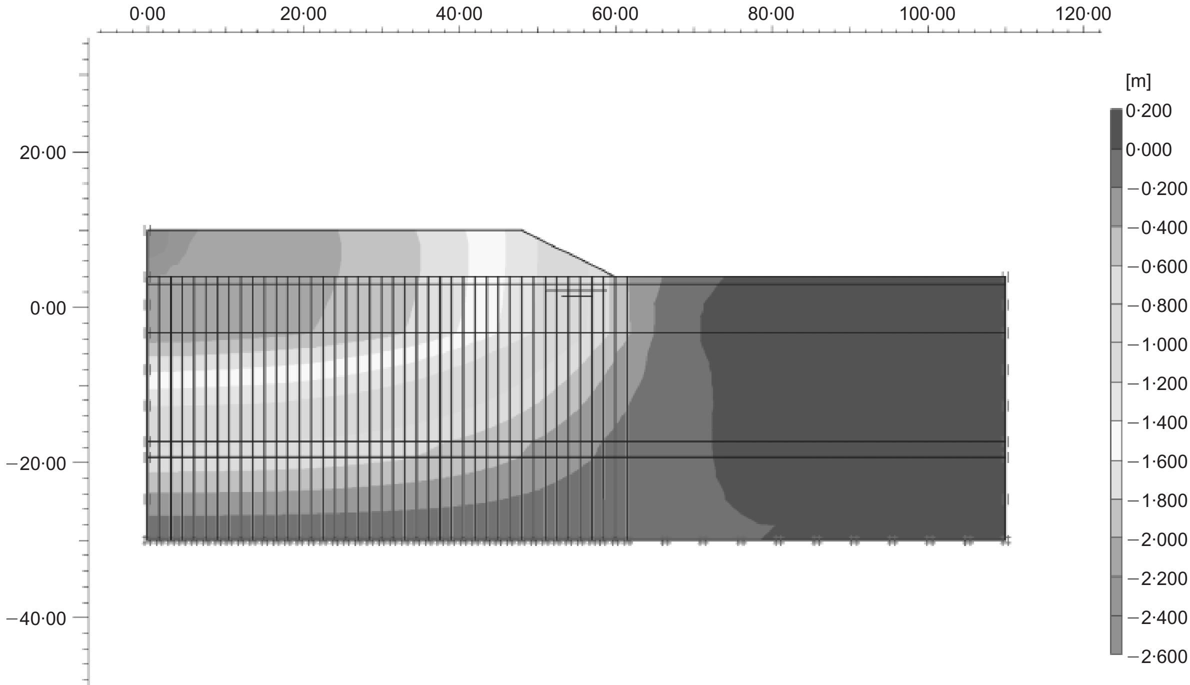

Fig. 6. Vertical displacement by full-scale analysis of vertical drain area (1·5 m × 1·5 m) at *in situ* test site, 20 months after surcharge placement

the comparison between the ultimate settlement by FEM with the actual field settlement at the site. Excellent agreement was obtained, both for the embankment with vertical drains and for the control embankment.

The matching technique used in the finite element analysis of the vertical drains was based on that used previously in the modelling of Bangkok clays with PVDs, but was modified to incorporate the marine clay multi-layers present at Changi. The axisymmetric unit cell and the full-scale analysis of vertical drains were found to be in excellent agreement both with each other and with the actual field settlement results at the pilot test site.

The axisymmetric unit cell analysis result was found to be settling at a slightly slower rate than the full-scale analysis and actual field settlement after 360 days. As evident for the vertical drain area in Table 4 and Fig. 9, there is a difference of only 0·144 m of settlement between the actual field settlement (2·404 m) and the axisymmetric unit cell FEM analysis (2·260 m) after a surcharge period of 20 months. The axisymmetric unit cell analysis provides a slightly lower settlement than that of the full-scale analysis.

The full-scale analysis with the use of the drain element, on the other hand, was found to match the actual field settlement very well until the 630-day period.

Table 3. Soil parameters for full-scale analysis of untreated control embankment

Mohr–Coulomb	Reclamation sandfill		
Type	Drained		
γ_{unsat}: kN/m^3	17·00		
γ_{sat}: kN/m^3	20·00		
k_h: m/day	1·000		
k_v: m/day	1·000		
E_{ref}: kN/m^2	13 000·000		
ν	0·300		
G_{ref}: kN/m^2	5000·000		
E_{oed}: kN/m^2	17 500·000		
c_{ref}: kN/m^2	1·00		
ϕ: deg	31·00		
ψ: deg	0·00		
Soft soil	Upper marine clay	Intermediate stiff clay	Lower marine clay
Type	Undrained	Undrained	Undrained
γ_{unsat}: kN/m^3	15·00	15·00	15·00
γ_{sat}: kN/m^3	15·50	15·50	16·00
k_h: m/day	$3{\cdot}67 \times 10^{-5}$	$8{\cdot}64 \times 10^{-5}$	$3{\cdot}89 \times 10^{-5}$
k_v: m/day	$1{\cdot}84 \times 10^{-5}$	$4{\cdot}32 \times 10^{-5}$	$1{\cdot}95 \times 10^{-5}$
λ^*	0·150	0·060	0·170
κ^*	0·018	0·011	0·025
c: kN/m^2	1·00	1·00	1·00
ϕ: deg	27·00	32·00	27·00
ψ: deg	0·00	0·00	0·00
ν_{ur}	0·150	0·150	0·150
K_0^{nc}	0·55	0·47	0·55

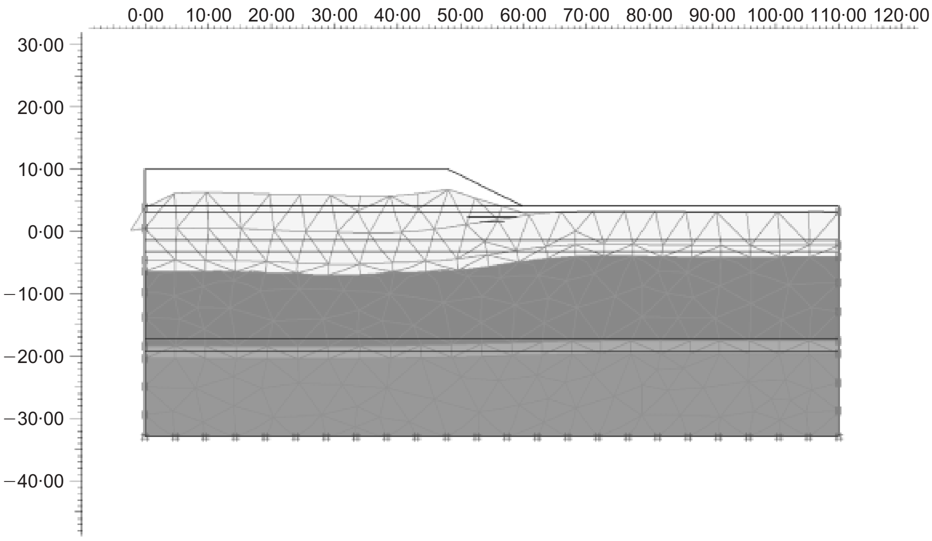

Fig. 7. Deformed mesh by full-scale analysis of control area (no drains) at *in situ* test site, 20 months after surcharge placement

As is evident for the vertical drain area in Table 4 and Fig. 9, there is a difference of only 0·084 m of settlement between the actual field settlement (2·404 m) and the full-scale FEM analysis (2·320 m) after a surcharge period of 20 months. In the authors' analysis of the full-scale embankment with vertical drains for the *in situ* test site, the drain element was successfully utilised instead of the interface element previously used for modelling Bangkok clay.

The full-scale analysis of the untreated control area was also found to be in excellent agreement with the actual field settlement after the final monitoring period of 785 days. As is evident for the control area in Table 4 and Fig. 9, there is a difference of only 0·019 m of settlement between the actual field settlement (0·706 m) and the full-scale FEM analysis (0·687 m) for the untreated control area after a surcharge period of 20 months.

Table 5 indicates the comparison of the Asaoka (1978), hyperbolic (Tan, 1995), piezometer and FEM methods at the *in situ* test site. As can be seen, the ultimate settlement obtained by FEM is lower than that predicted by the Asaoka and hyperbolic methods for the vertical drain area (1·5 m × 1·5 m). The degree of consolidation obtained by FEM for the vertical drain area (1·5 m × 1·5 m) is subsequently slightly higher than that obtained by the Asaoka, hyperbolic and piezometer methods. For the vertical drain area, a degree of

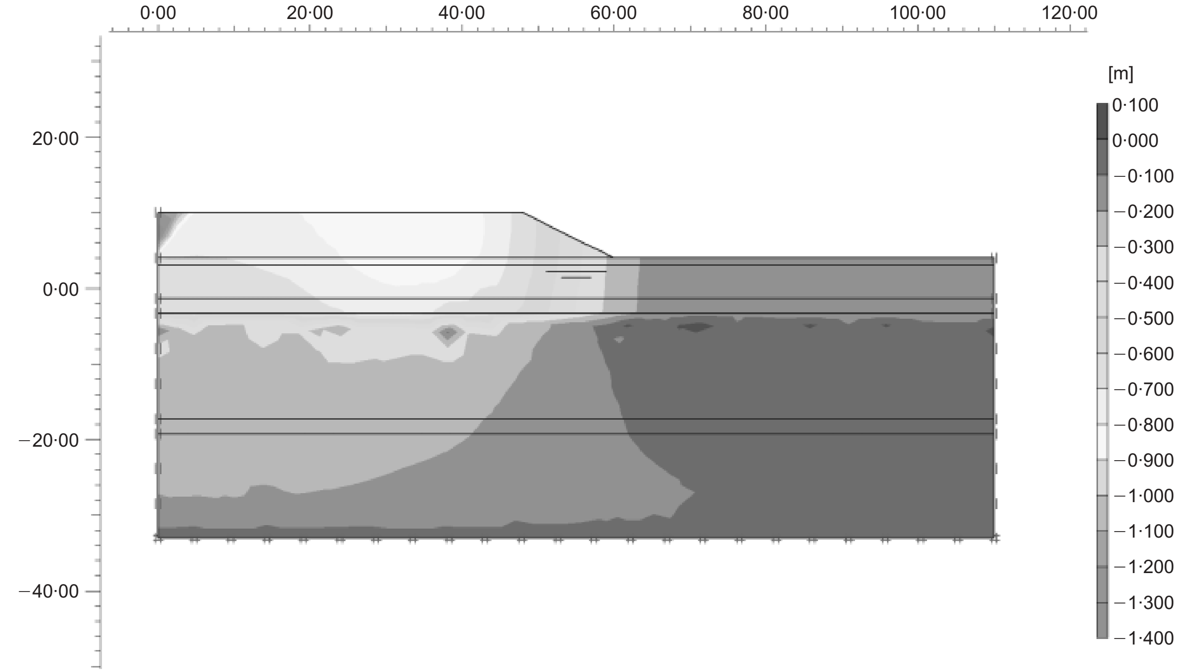

Fig. 8. Vertical displacement by full-scale analysis of control area (no drains) at *in situ* test site, 20 months after surcharge placement

Table 4. Comparison between FEM results and actual field settlement at in situ *test site 20 months after surcharge*

Sub-area	Field settlement to date; m	Full-scale FEM analysis: m	Axisymmetric unit cell FEM analysis: m
Vertical drain: 1·5 m × 1·5 m	2·404	2·320	2·260
Control: no drain	0·706	0·687	–

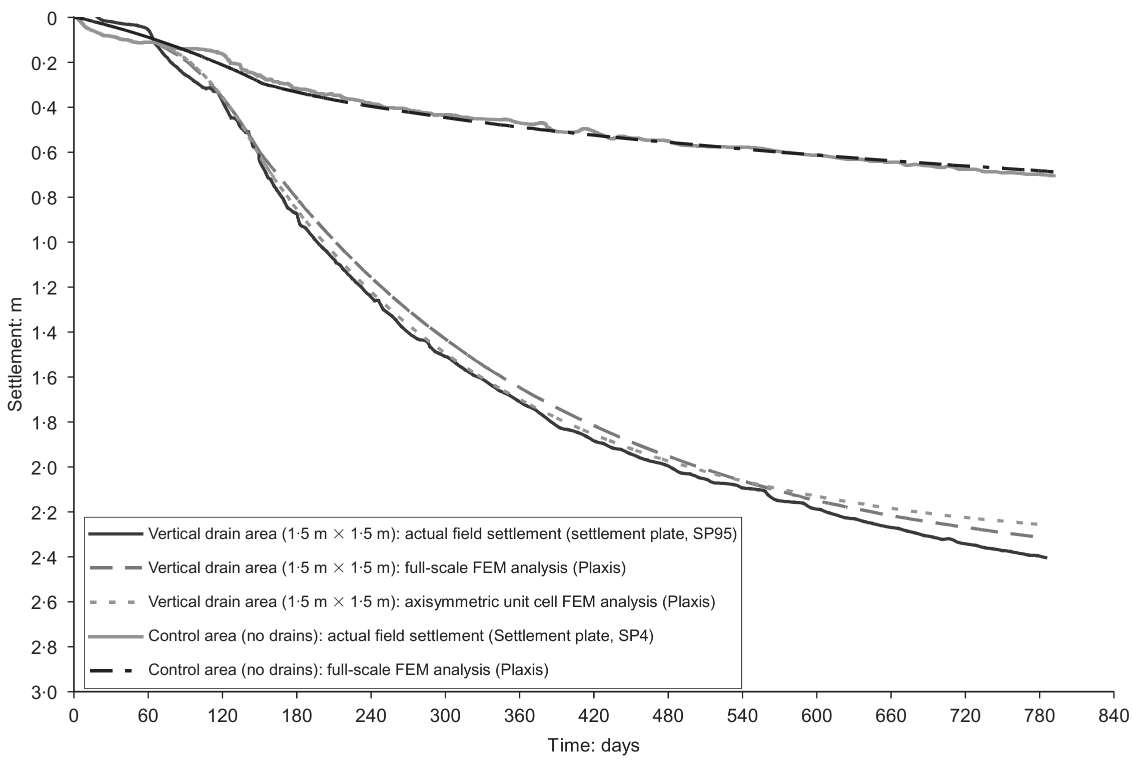

Fig. 9. Comparison between FEM results and actual field settlement at *in situ* test site, 20 months after surcharge placement

consolidation of 87·8% was obtained from the FEM method as compared with 80·1% from the Asaoka method, 80·0% from the hyperbolic method and 80·0% from the piezometer method.

The degree of consolidation obtained by FEM for the untreated control area was also found to be slightly higher than that obtained by the piezometer method. For the control area, a degree of consolidation of 27·9% was obtained from the FEM method as compared with 20·0% from the piezometer method.

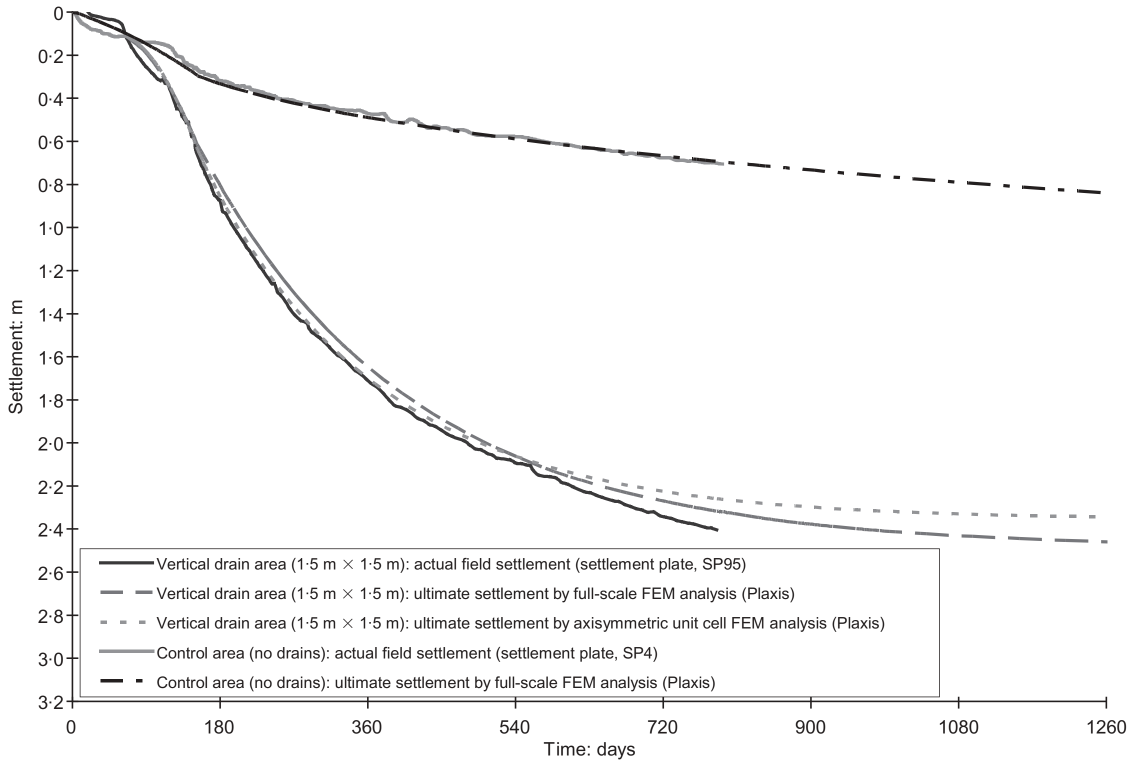

Fig. 10. Comparison between ultimate settlement by FEM and actual field settlement at *in situ* test site

Table 5. Comparison of settlement assessed by Asaoka, hyperbolic, piezometer and FEM methods at in situ *test site 20 months after surcharge*

Sub-area	Comparison	Asaoka	Hyperbolic	Piezometer	FEM
Vertical drain: 1·5 m × 1·5 m	Ultimate settlement: m	3·000	3·005	–	2·640
	Settlement to date: m	2·404	2·404	–	2·320
	Degree of consolidation: %	80·1	80·0	80·0	87·8
Control: no drain	Ultimate settlement: m	–	–	–	2·465
	Settlement to date: m	0·706	0·706	–	0·687
	Degree of consolidation: %	–	–	20·0	27·9

Pilot test site

Table 6 and Fig. 11 show the comparison of the actual field settlement and the Plaxis v. 8 numerical modelling method for the pilot test site. Fig. 12 shows the comparison between the ultimate settlement by FEM and the actual field settlement at the pilot test site. Excellent agreement was obtained between the FEM analysis and the actual field settlement for both the embankment with vertical drains and the control embankment.

The axisymmetric unit cell analysis result was found to be in good agreement with the actual field settlement for sub-areas A2S-71 (2·0 m × 2·0 m) and A2S-73 (3·0 m × 3·0 m). The axisymmetric unit cell analysis result was found to be settling at a slightly slower rate than the actual field settlement for sub-area A2S-72 (2·5 m × 2·5 m).

As is evident in Table 6 and Fig. 11, there is a difference of only 0·021 m between the actual field settlement (1·687 m) and the axisymmetric unit cell FEM analysis (1·666 m) for sub-area A2S-71 (2·0 m × 2·0 m) after a surcharge period of 32 months. There is a difference of only 0·002 m between the actual field settlement (1·264 m) and the axisymmetric unit cell FEM analysis (1·262 m) for sub-area A2S-72 (2·5 m × 2·5 m). There is a difference of only 0·012 m between the actual field settlement (0·948 m) and the axisymmetric unit cell FEM analysis (0·960 m) for sub-area A2S-73 (3·0 m × 3·0 m).

The full-scale analysis of the untreated control embankment of sub-area A2S-74 (no drains) was also found to be in excellent agreement with the actual field settlement. The settlements were found to be in very close agreement after

Table 6. Comparison of settlement between FEM results and actual field settlement at pilot test site 32 months after surcharge (41·9 months of monitoring)

Sub-area	Field settlement to date: m	Full-scale FEM analysis: m	Axisymmetric unit cell FEM analysis: m
A2S–71 2·0 × 2·0 m	1·687	–	1·666
A2S–72 2·5 × 2·5 m	1·264	–	1·262
A2S–73 3·0 × 3·0 m	0·948	–	0·960
A2S–74 No drain	0·503	0·435	–

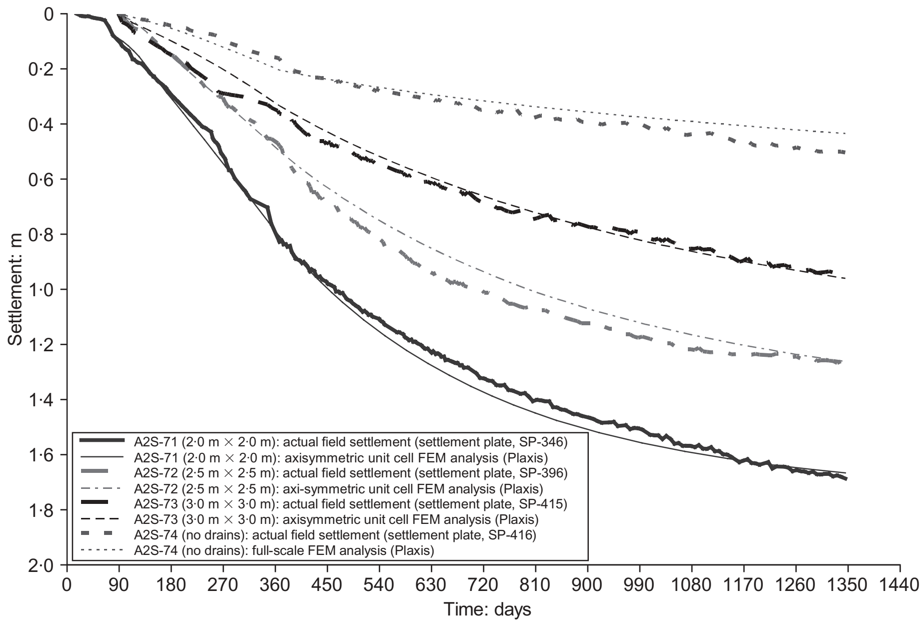

Fig. 11. Comparison between FEM results and actual field settlement at pilot test site, 32 months after surcharge placement

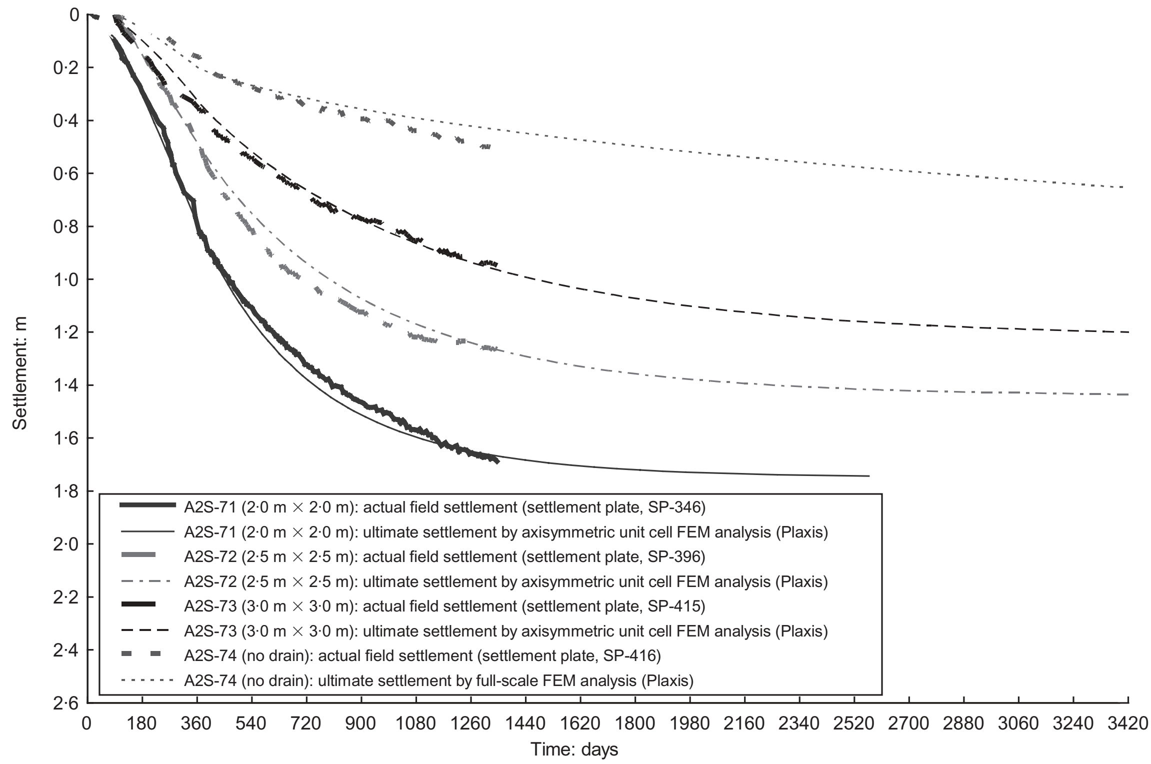

Fig. 12. Comparison between ultimate settlement by FEM and actual field settlement at pilot test site

the surcharge period of 32 months (monitoring period 41·9 months). As is evident for sub-area A2S-74 (no drains) in Table 6 and Fig. 11, there is a difference of only 0·068 m between the actual field settlement (0·503 m) and the full-scale FEM analysis (0·435 m) for the untreated sub-area A2S-74 after a surcharge period of 32 months (monitoring period 41·9 months).

Table 7 shows the comparison of the ultimate settlement and degree of consolidation assessed by using the Asaoka, hyperbolic, piezometer and FEM methods at the pilot test site. As can be seen, the ultimate settlement obtained by FEM is lower than that predicted by the Asaoka and hyperbolic prediction methods for sub-area A2S-71 (2·0 m × 2·0 m). The degree of consolidation obtained by FEM for sub-area A2S-71 is subsequently slightly higher than that obtained by the Asaoka, hyperbolic and piezometer methods. For sub-area A2S-71, a degree of consolidation of 95·6% was obtained from the FEM method as compared with 91·8% from the Asaoka method, 93·7% from the hyperbolic method and 86·2% from the piezometer method.

The ultimate settlement obtained by FEM was found to be higher than that predicted by the Asaoka and hyperbolic

Table 7. Comparison of settlement and degree of consolidation assessed by Asaoka, hyperbolic, piezometer and FEM methods at pilot test site 32 months after surcharge (41·9 months of monitoring)

Sub-area	Comparison	Asaoka	Hyperbolic	Piezometer	FEM
A2S–71 2·0 m × 2·0 m	Ultimate settlement: m	1·838	1·801	–	1·743
	Settlement to date: m	1·687	1·687	–	1·666
	Degree of consolidation: %	91·8	93·7	86·2	95·6
A2S–72 2·5 m × 2·5 m	Ultimate settlement: m	1·412	1·408	–	1·436
	Settlement to date: m	1·264	1·264	–	1·262
	Degree of consolidation: %	89·5	89·8	82·5	87·9
A2S–73 3·0 m × 3·0 m	Ultimate settlement: m	1·200	1·169	–	1·217
	Settlement to date: m	0·948	0·948	–	0·960
	Degree of consolidation: %	79·0	81·1	73·1	78·9
A2S–74 No drain	Ultimate settlement: m	–	–	–	1·281
	Settlement to date: m	0·503	0·503	–	0·435
	Degree of consolidation: %	–	–	37·0	33·9

prediction methods for sub-areas A2S-72 (2·5 m × 2·5 m) and A2S-73 (3·0 m × 3·0 m). The degree of consolidation obtained by FEM for sub-areas A2S-72 and A2S-73 was subsequently slightly lower than that obtained by the Asaoka, hyperbolic and piezometer methods. For sub-area A2S-72, a degree of consolidation of 87·9% was obtained from the FEM method as compared with 89·5% from the Asaoka method, 89·8% from the hyperbolic method and 82·5% from the piezometer method. For sub-area A2S-73, a degree of consolidation of 78·9% was obtained from FEM as compared with 79·0% from the Asaoka method, 81·1% from the hyperbolic method and 73·1% from the piezometer method.

The degree of consolidation obtained by FEM for the untreated sub-area A2S-74 (no drains) was also found to be slightly lower than that obtained by the piezometer method. A degree of consolidation of 33·9% was obtained from the FEM method as compared with 37·0% from the piezometer method.

In conclusion, it can be said that reasonable agreement was obtained between the FEM analysis and the actual field settlements for both the vertical drain treated embankments and the untreated control embankments, at both sites. The axisymmetric unit cell and the full-scale analysis of vertical drains were found to be in excellent agreement with each other and with the actual field settlement results.

Performance verification of PVDs

The vertical drain performance was verified for the *in situ* test site by using C_h obtained from back-analysis by the Asaoka method and by FEM with Plaxis v. 8.

Back-analysis using C_h from Asaoka method

Conventional calculations by applying the Barron (1948), Hansbo (1979) and Yoshikuni and Nakanodo (1974) theories with well resistance and smear effect were compared with the actual field performance. The conventional vertical drain design with a C_h of 0·78 m^2/year obtained from back-analysis by applying the Asaoka method was generated. This was compared with the actual field performance. It was found that the curve of the calculated time rate of settlement with a C_h of 0·78 m^2/year was similar to the field curve, as shown in Fig. 13, up to a surcharge period of 12 months, after which the field settlement curve slowed down.

Based on the settlement plate monitoring results (SP-095), a settlement of 0·691 m was recorded during filling operations from the vertical drain platform level (+4 mCD) to the surcharge level (+10 mCD). This settlement was incorporated in the comparison of degree of consolidation between field and back-analysis for the vertical drain area.

Proposed modified Asaoka equation

Settlement at any point of time, S_t, can be calculated as a fraction of the final settlement S_{ult} from the following Asaoka equation (Hausmann, 1990):

$$\frac{S_t}{S_{ult}} = 1 - \frac{8}{\pi^2}\exp\left[-\left(\frac{8C'_h}{d_e^2\alpha} + \frac{\pi^2 C_v}{4H_0^2}\right)t\right] \tag{5}$$

where

$$\alpha = \frac{n^2 \ln n}{n^2 - 1} - \frac{3n^2 - 1}{4n^2} \tag{6}$$

$$n = \frac{d_e}{d} \tag{7}$$

where d_e is the equivalent diameter of the cylinder of soil around the drain (= 1·128s for a square grid), C_v is the coefficient of consolidation due to vertical flow, C'_h is the effective value of the coefficient of consolidation due to horizontal flow, H_0 is the thickness of the layer, t is the elapsed time since the application of surcharge, and d is the equivalent drain diameter.

However, the above equation is suitable for a single layer of clay only. The authors propose that the equation be modified to allow for the analysis of multiple layers of marine clay (in this case upper marine clay, intermediate clay and lower marine clay) by considering the equivalent thickness of the marine clay.

As marine clay consists of several layers (in this case three), the authors propose that the equivalent thickness of the marine clay has to be calculated to enable the values of the equivalent thickness, equivalent drainage and assumed coefficient of vertical consolidation to be input to the proposed modified Asaoka equation. The equations used for computation of equivalent thickness of marine clay as proposed by Choa and Wong (1992) are as follows.

Equivalent thickness of layer 1, H'_1

$$H'_1 = H_1\left(\frac{C_{vi}}{C_{v1}}\right)^{0.5} \tag{8}$$

where C_{vi} is an initial assumed value.

Total equivalent thickness of all layers, H'_{Ti}:

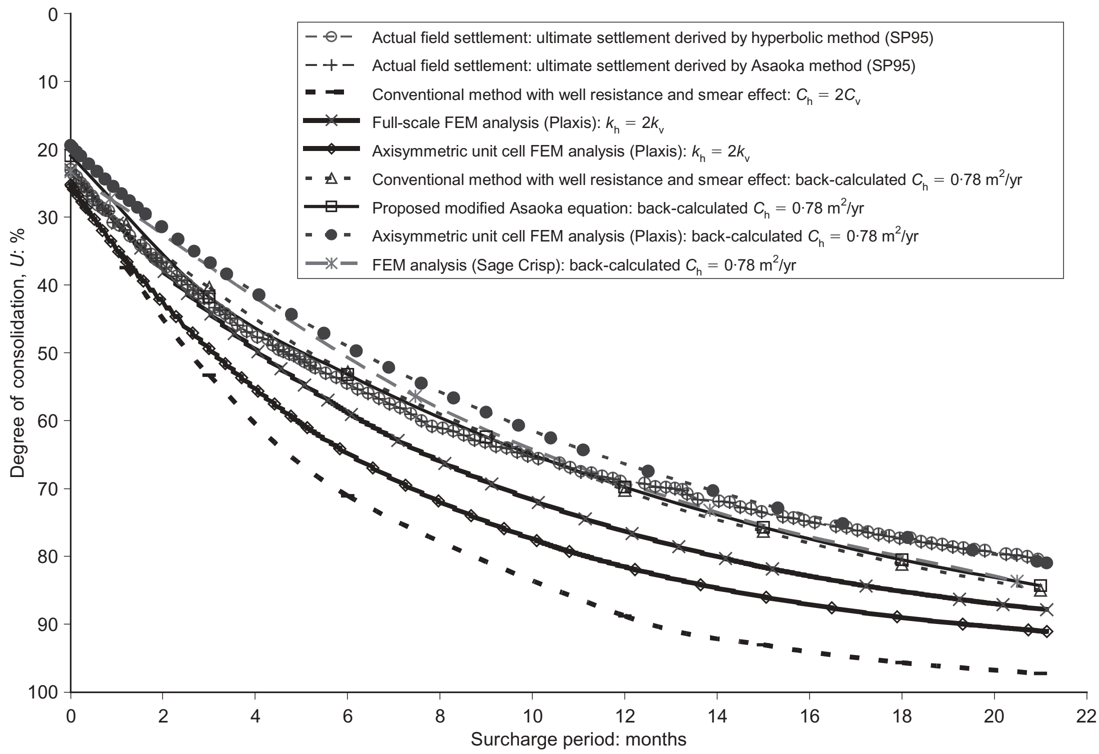

Fig. 13. Vertical drain performance verification comparison of degree of consolidation by various methods, 20 months after surcharge placement

$$H'_{\mathrm{Ti}} = H'_1 + H'_2 + H'_3 + \ldots + H'_n \tag{9}$$

Equivalent drainage thickness, H_{dri}:

$$H_{\mathrm{dri}} = \frac{H'_{\mathrm{i}}}{2} \tag{10}$$

The authors suggest that the equivalent coefficient of consolidation due to vertical flow, C_{vi}, and equivalent thickness of the layers, H'_{Ti}, be incorporated into the Asaoka equation. The modified equation is defined as

$$\frac{S_t}{S_{\mathrm{ult}}} = 1 - \frac{8}{\pi^2}\exp\left[-\left(\frac{8C'_{\mathrm{h}}}{d_{\mathrm{e}}^2\alpha} + \frac{\pi C_{\mathrm{vi}}}{4H'^2_{\mathrm{Ti}}}\right)t\right] \tag{11}$$

The C_{h} of 0·78 m^2/year obtained from back-analysis by the Asaoka method was used in the equation. It was found that the calculated curve for settlement rate with a C_{h} of 0·78 m^2/year is also similar to the field curve, as shown in Fig. 13, up to a surcharge period of 12 months, after which the field settlement curve slows down. The proposed modified Asaoka equation ties in very well with the back-analysis results by the conventional method, and therefore can be used in future instead of back-analysis using the conventional calculations.

Conventional design of PVDs with back-calculated C_{h}

The conventional design method for vertical drains was carried out with consideration for well resistance and smear effect, and using a C_{h} to C_{v} ratio of 2. The predicted settlement rate was found to be much more rapid than the actual field settlement. Similar findings have been reported by Bo *et al.* (1997b) and Chun *et al.* (1997).

The same design method was also carried out using the back-calculated C_{h} of 0·78 m^2/yr. It can be seen in Fig. 13 that the resulting calculated settlement rate is similar to the field curve up to a surcharge period of 12 months, after which the field settlement curve slows down.

Finite element modelling of PVDs

Finite element modelling of the vertical drains was carried out using Plaxis v. 8 with both axisymmetric unit cell analysis and full-scale embankment analysis, by means of the conventional modelling method using $k_{\mathrm{h}} = 2k_{\mathrm{v}}$ for Singapore marine clay. It is evident in Fig. 13 that the consolidation rate by the FEM method is faster than the actual field settlement.

Finite element modelling was also carried out using the back-calculated C_{h} = 0·78 m^2/yr by axisymmetric unit cell analysis. The calculated settlement rate was found to be similar to the field curve if the back-calculated C_{h} = 0·78 m^2/yr was used in the FEM analysis, as illustrated in Fig. 13. Similar findings have been reported previously by Bo *et al.* (1997b) and Balasubramaniam *et al.* (1995) with the use of the Sage-Crisp 2D (Crisp, 1995) FEM program.

Table 8 and Fig. 13 indicate the vertical drain performance verification comparison of settlement and degree of consolidation by various methods at the centroid location of the vertical drain points. All curves in Fig. 13 start from an average degree of consolidation of more than 20%, as this amount of consolidation has taken place as a result of sand filling to the surcharge level.

Findings and discussion on performance verification of PVDs

Table 8 and Fig. 13 indicate that the field consolidation rates are in good agreement with the newly proposed modified Asaoka method and the conventional method using the back-calculated C_{h} = 0·78 m^2/yr. The actual field measurement is slightly slower by only about 5%. The

Table 8. Comparison of vertical drain performance verification by settlement and degree of consolidation by various methods at 20 months after surcharge

Method employed (PVD spacing of 1·5 m square)	Ultimate settlement: m	Settlement to date: m	Degree of consolidation, *U*: %
Actual field settlement: hyperbolic method	3·005	2·404	80·0
Actual field settlement: Asaoka method	3·000	2·404	80·1
Conventional method: well resistance and smear effect	3·005	2·923	97·3
Full-scale FEM analysis (Plaxis): $k_h = 2k_v$	2·640	2·320	87·8
Unit cell FEM analysis (Plaxis): $k_h = 2k_v$	2·480	2·260	91·1
Conventional method: back-calculated $C_h = 0{\cdot}78$ m^2/year	3·000	2·553	85·1
Proposed modified Asaoka eqn: back-calculated $C_h = 0{\cdot}78$ m^2/year	3·000	2·530	84·3
Unit cell FEM analysis (Plaxis): back-calculated $C_h = 0{\cdot}78$ m^2/year	2·454	1·987	80·9
FEM analysis (Sage Crisp): back-calculated $C_h = 0{\cdot}78$ m^2/yr	2·963	2·489	84·0

modified Asaoka equation is found to be in good agreement with the conventional method using the back-calculated C_h. It is therefore proposed that the modified equation can be used in similar multi-layer schemes of marine clay in future instead of using the conventional method with back-calculated C_h.

It was found from back-analysis of the field instrumentation monitoring results that the actual field coefficient of consolidation due to horizontal flow is only 0·78 m^2/year. However, field measurements of coefficient of consolidation due to horizontal flow measured, before reclamation, by the various *in situ* test equipments are much higher than the back-analysed and design (C_h) values. It is therefore recommended, based on these findings, that the assumed coefficient of consolidation due to horizontal flow in the design stage should not be more than 1·5 times the coefficient of consolidation due to vertical flow when thick layers of homogeneous clay are present.

The field curve is noted to be slowing down after one year of surcharging. This may indicate a reduction in the permeability of the vertical drain filter due to clogging. It may also be due to a reduction of permeability in the surrounding soil caused by void ratio changes in the later stages of consolidation.

The main factor accounting for the lower C_h values back-calculated from field settlement measurements is the smear effect incurred by insertion of the mandrel during the installation of vertical drains. For soft marine clay this effect can be significant, as the spacing of the drains is normally 1·5 m. Bo *et al.* (1998) have reported that the permeability of soil in the smear zone could be reduced by one order of magnitude or to the k_h of the remoulded clay. The smear zone was found to be four to five times the equivalent diameter of the vertical drain. When drains are installed at close spacing, the back-calculated C_h values will generally be greatly influenced by this smear zone.

C_h values, measured from *in situ* tests prior to reclamation, are C_h values based on *in situ* overburden pressures. These C_h values would reduce with any increase in additional load. This can be seen in the reduction of C_h values in the laboratory with each load increment. Therefore the use of C_h values from *in situ* tests prior to reclamation may not be so conservative, because this C_h value accounts for the existing overburden pressure, and would be reduced with the increments in fill load. Also, the boundary conditions for *in situ* tests and for field conditions with vertical drains are different. Hence average C_h values surrounding the effective area of the vertical drain could be different from *in situ* tests, which have a smaller effective flow area.

Vertical drains installed in the project are performing to improve the soil drainage system, but their performance is slightly slower than that predicted. An exact superimposed time rate of settlement curves between field and prediction is extremely difficult to obtain, because there are various natural variations that cannot be modelled. It would therefore be more effective to design the vertical drain, especially where thick layers of homogeneous clay exist, with a lower specified degree of consolidation but a higher surcharge load (higher additional load) in order to gain the equivalent stress gain within a shorter duration when vertical drains are fully performing.

Conclusions

Reasonable agreement was obtained between the FEM analysis and the actual field settlements for both the vertical drain treated embankments and the untreated control embankments, at both the *in situ* test site and the pilot test site. The axisymmetric unit cell and the full-scale analysis of vertical drains were found to be in excellent agreement with each other and with the actual field settlement results.

The matching techniques used in the finite element analysis of the vertical drains was based on that used previously in the modelling of Bangkok clays with PVDs. However, the modelling of the Singapore marine clay treated with vertical drains was modified to incorporate the marine clay multi-layers present in the Singapore marine clay at Changi. The modelling technique used by the authors is found to provide similar excellent agreements in their use for the modelling of Singapore marine clay with vertical drains.

For the performance verification of PVDs, the field time rate of consolidation is found to be in good agreement with the newly proposed modified Asaoka method and the conventional method using the back-calculated C_h = 0·78 m^2/yr. The actual field measurement is slightly slower than the proposed modified Asaoka and the conventional method using the back-calculated C_h = 0·78 m^2/year, by only about 5%. The newly proposed modified Asaoka equation is found to be in good agreement with the conventional method using the back-calculated C_h. It is therefore proposed that the modified equation can be used in similar multi-layer schemes of marine clay in future instead of using the conventional method with back-calculated C_h.

The field curve is noted to be slowing down after one year of surcharge. This may be due to a reduction in the permeability of the vertical drain filter caused by clogging, or to a reduction of the permeability of the surrounding soil caused by void ratio changes in the later stages of consolidation.

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