SETTLEMENT OF EMBANKMENT ON SOFT CLAY: LARGE-SCALE EXPERIMENTAL STUDY AND NUMERICAL MODELING

Quanmei Gong[1] Key Laboratory of Road and Traffic Engineering, Tongji University, Shanghai, China
Peijun Guo Department of Civil Engineering, McMaster University, Canada
Binglong Wang Key Laboratory of Road and Traffic Engineering, Tongji University, Shanghai, China
Shunhua Zhou Key Laboratory of Road and Traffic Engineering, Tongji University, Shanghai, China

ABSTRACT
This paper studied the post-construction settlement of an embankment on soft clay improved by prefabricated plastic drainage plate. A segment of experimental embankment was constructed and instruments were installed to measure the settlement as well as the excess pore water pressure during and after the construction. The measured data were compared with the results of theoretical analysis based on single-drain theory and a finite element analysis based on an equivalent plane strain model. It was found that the finite element method could reasonably predict the settlement, while the excess pore pressure was over estimated. The single-drain theory also gave a rational final settlement, however, longer time was required for settlement to complete. The results also showed that the soil improvement method and the stage construction approach used in this study satisfy the requirements of current specifications.

RÉSUMÉ
Aucun résumé français fourni par l'auteur.

1. INTRODUCTION

The settlement, including the total settlement, the post-construction settlement and the rate of settlement, may have significant effect on the cost of maintenance, the operation condition and the safety of high-speed railways. In China, for a high speed railway subgrade, the current design guidelines require that the post-construction settlement should be less than 10 cm and the rate of post-construction settlement in the first year should be less than 3 cm/year. For the embankments constructed on soft clay, some engineering measures (e.g., soil improvement, appropriate control of construction and/or accelerating the dissipation of excess pore water pressure) have to be taken to satisfy these requirements. However, the methods used for a particular project may have significant influence on the cost of the project.

A high-speed railway from Beijing to Shanghai will be built in a few years and it will pass a wide area of soft clay. In order to verify the efficiency of different engineering measures and the applicability of various methods to estimate the total and the post construction settlements, a segment of experimental embankment was constructed. Soil improvement methods, including prefabricated plastic drainage plate combined with overload or vacuum preloading and composite foundation were applied at different sections of the embankment. Instruments were installed to measure the settlement of the subgrade soil as well as the variation of excess pore water pressure with time. The influence of other factors (e.g., construction stages and the rate of construction) on the subgrade soil settlement was also taken into account in the design of the experimental embankment. In this paper, however, only prefabricated plastic drainage plate combined with overload preloading method was discussed.

The 1D unit cell solution (Barron, 1948) of consolidation theory is commonly used to describe the consolidation process of soft clay with prefabricated plastic drainage plate. Hansbo (1981) extended Barron's theory to take into account the effect of smear and well resistance. Single-drain analysis may be accurate enough to estimate the soil behaviour along the embankment centreline, assuming that the surcharge is uniformly distributed. However, this method may not be applicable for the calculation of the settlement along the width of embankment, owing to the non-uniform distribution of loads in this direction. Moreover, the deformation of prefabricated plastic drainage plate improved subsoil is not always in 1D conditions. Hird et al (1992) developed an equivalent plane-strain analysis in which the vertical drain of a unit cell was taken into account while the effect of smearing was neglected. Chai et al (1995) and Indraratna et al. (1997 and 1999) further extended the equivalent plane strain approach to include the effect of smear and well resistance. Actually the field condition is three-dimensional (3D), but the 3D numerical calculation is time consuming, the previous researches indicate that an equivalent plane-strain model is sufficient.

This paper will compare the measured data from the experimental embankment with the analytical results from Barron's 1D unit cell solution and the numerical modelling results based on the equivalent plane strain approach via finite element method.

[1] Currently visiting scholar at McMaster University

2. BRIEF DESCRIPTION OF TEST RAILWAY SUBGRADE

The test railway subgrade is located at Kunshan, jiangshu. The thickness of soft clay is about 10m. The total length of experimental embankment is about 800m, in which 432.04 m was improved by prefabricated plastic drainage plate of 7.0~15.0 m depth and 1.2~1.8 m spacing. According to the current specification of high-speed railway subgrade, for double-lines, the top width of the subgrade is 14.2 m and the slope of the embankment is 1:1.5.

Figure 1 presents a typical cross section of the embankment and the subsoil profile. At this cross-section, the height of the embankment is 4.5m. The installing pattern of 10.5 m long plastic drainage plates is triangular of 1.8m spacing. The soil properties are in table 1.

The construction load was applied in stages and the loading history is presented in Figure 2. The first stage was a 0.6m high sand blanket with the equivalent load being 13.8 kPa. The load in stage 2 was raised to 24.15 kPa, the loads of stages 3, 4, 5, 6, 7, 8 were 51.75 kPa, 76.13 kPa, 101.43 kPa, 108.56 kPa, 115.23 kPa and 131.1 kPa respectively. The construction of the embankment was completed in about 200 days and the train load will be applied after 250 days of the construction completion. In other words, the total period of construction and preloading is 450 days.

Various instruments, including settlement plates, inclinometers and pore pressure gauges, were installed to measure the settlement and excess pore water pressure in the subgrade soil layers. Figure 3 presents the plan view of the installation of the instruments. The measurement started at the beginning of the construction of the embankment and continues currently.

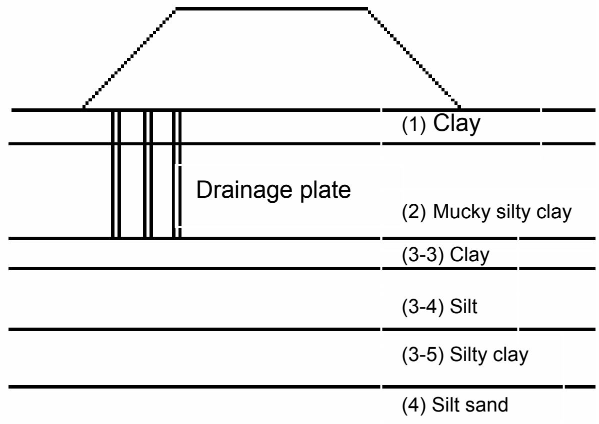


Figure 1 Cross section of the embankment and subsoil profile

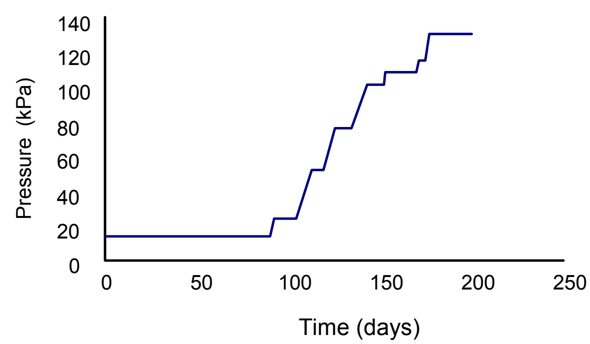
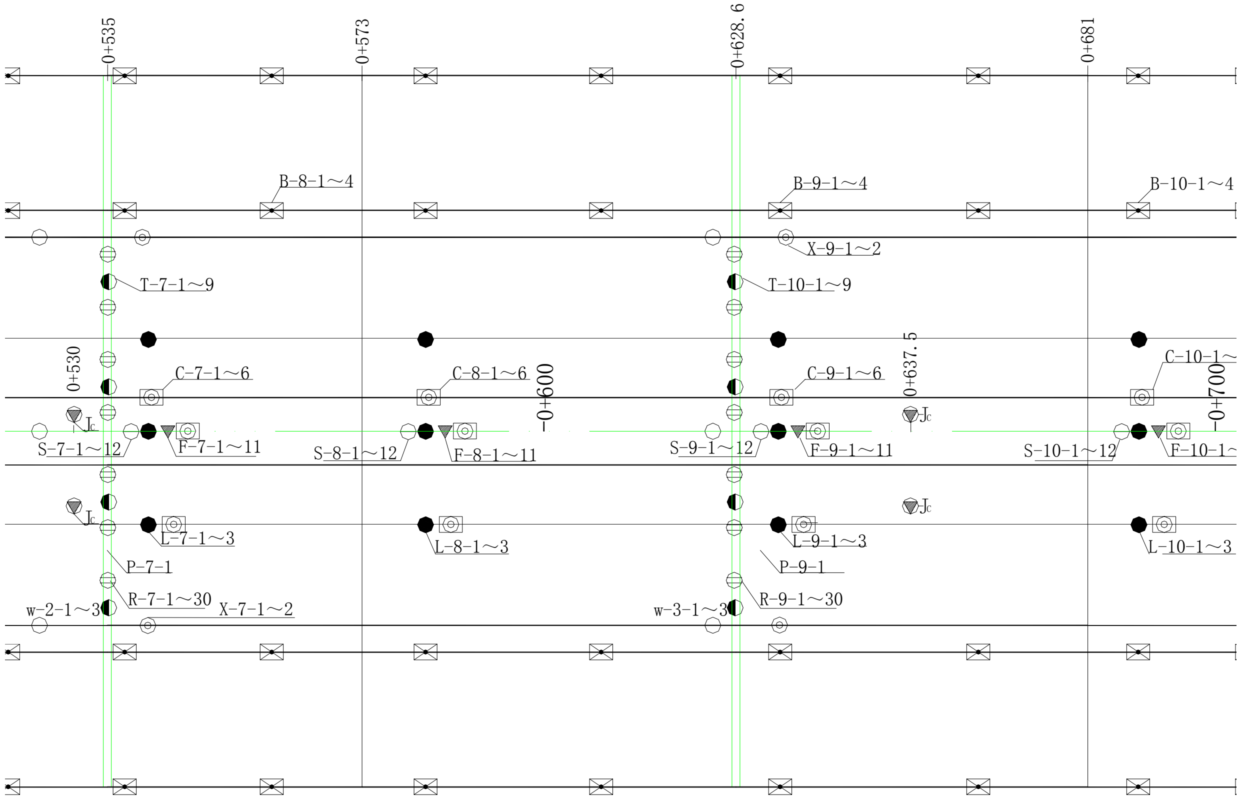


Figure 2 Construction loading history

Table 1. Soil properties

Soil Layer	Depth (m)	Water content w (%)	Density (kN/m^3)	Coefficient of consolidation (cm^2/s×10^{-3})		Compression modulus E_s (MPa)	Coefficient of secondary consolidation	Compression index C_c	Swelling index C_s
				C_v	C_h				
(1)	0.0-3.0	31.9	19.2	2.09	1.45	4.61	0.0064	0.25	0.03
(2)	3.0-10.5	43.5	17.9	0.82	5.16	2.55	0.0086	0.30	0.04
(3)-3	10.5-12.2	24.5	20.3	9.46	5.47	9.39	0.0019	0.14	0.06
(3)-4	12.2-15.4	35.5	18.7	11.38	12.86	10.01	—	0.20	0.02
(3)-5	15.4-21.0	36.4	18.7	7.82	9.31	6.08	0.0047	0.33	0.02
(4)	21.0-	—	—	—	—	28.25	—	—	—



observation border pile; settlement plate; observation pile on surface; earth pressure cell
flexible sensor; pore-water pressure gauge; telescoping tube settlement gauge;
clinometer; static sounding; profile settlement gauge; water level gauge

Figure 3 Plan view of the installation of instruments

3. SINGLE-DRAIN THEORY METHOD TO EVALUATE THE SETTLEMENT

Given a foundation subjected to a uniform ally distributed pressure *p*, the total settlement of the foundation is expressed as

$$S = S_d + S_c + S_s \qquad [1]$$

where s_d is immediate settlement, s_c is primary consolidation settlement and s_s is the secondary consolidation settlement.

The immediate settlement s_d can be estimated based on elastic theory, i.e.

$$S_d = \frac{PB\left(1-\nu^2\right)}{E} \cdot F \qquad [2]$$

where *E* is elastic modulus, *F* is settlement coefficient, *B* is the width of load area and ν is Poisson's ratio.

The primary consolidation settlement s_c consists of two components

$$S_c = \overline{U}_1 S_{1\infty} + \overline{U}_2 S_{2\infty} \qquad [3]$$

in which $\overline{U}_1 S_{1\infty}$ and $\overline{U}_2 S_{2\infty}$ denote the settlements of the improved zone and the underlayer of improved zone, respectively. The final primary consolidation settlement S_∞ is calculated by dividing the soil into sub-layers and calculating the sum of the settlement of each individual layer. The average consolidation degree of the improved zone $\overline{U}_1$ is given in Eq. [4]

$$\overline{U}_1 = 1 - \frac{8}{\pi^2} e^{-\beta_{rz} t} \qquad [4]$$

in which

$$\beta_{rz} = \beta_r + \beta_z$$

$$\beta_r = \frac{8 c_h}{(F + J + \pi G) de^2}$$

$$\beta_z = \frac{\pi^2 c_v}{4H^2}$$

$$F = \ln(n) - \frac{3}{4}$$

$$J = \ln(s)(\frac{k_h}{k_s} - 1)$$

$$G = (\frac{k_h}{k_w})(\frac{H}{d_w})^2$$

$$n = \frac{d_e}{d_w}$$

$$s = \frac{d_s}{d_w}$$

in which, β_{rz} , β_r , β_z are consolidation factors; F is a function related to n; J is smear factor, it is a function related to smear ratio and reduction of permeability; G is a factor of well resistance ; k_w , k_h , k_s are permeability coefficients of well material, undisturbed zone and smear zone respectively; c_v , c_h are subsoil consolidation coefficients in vertical and horizontal direction; H is the length of the longest drainage path, under one-way drainage condition it is the thickness of clay layer, under two-way condition it is half of the thickness. d_w , d_s are the diameters of well and smear zone respectively, d_e is equivalent diameter.

The average consolidation degree of the soil blow the prefabricated plastic drainage plate improved zone is calculated according to Terzaghi's consolidation theory. If the load is time-dependent, the consolidation degree may be determined by Eq.[5].

$$\overline{U}'_t = \sum_{i=1}^{n} \frac{\dot{q}_i}{\sum \Delta p}\left[\left(T_m - T_{m-1}\right) - \frac{\alpha}{\beta} e^{-\beta t}\left(e^{\beta T_m} - e^{\beta T_{m-1}}\right)\right] \quad [5]$$

in which, *n* is the number of stages, $\dot{q}_i$ is loading rate of i[th] stage, $\Sigma\Delta p$ is the accumulated load, T_{m-1} and T_m are the beginning and ending time of the i[th] stage and α and β are two factors.

The secondary consolidation settlement S_s is calculated by

$$S_s = \sum_{i=1}^{n} \frac{c_{\alpha i}}{1+e_{1i}} \lg(\frac{t_2}{t_1}) h_i \quad [6]$$

where $c_{\alpha i}$ is the coefficient of secondary consolidation, h_i is the thickness of clay layer at the beginning of secondary consolidation, t_1 is the ending time of primary consolidation, and t_2 is the time at which the secondary compression starts.

According to Equation [2], the immediate settlement of the test railway subgrade is 9.2 cm. The primary consolidation settlement is in the range of 68.1 to 77.1 cm, depending on whether the *e~lgp* curve or the compression modulus is used in the calculation. The settlement induced by secondary consolidation is approximately 6.1 cm. Consequently, the total settlement ranges from 83.4 to 92.4 cm.

4. NUMERICAL METHOD TO EVALUATE THE SETTLEMENT

The consolidation of soil improved by a single prefabricated plastic drainage plate is an axially symmetric problem. However, the settlement of a foundation on a clay layer improved with many prefabricated plastic drainage plates cannot be considered as an axially symmetric problem. Indraratna et al. (1994, 1997 and 1999) proposed an equivalent, plane strain model, in which axisymmetric permeability outside and inside the smear zone was converted into equivalent plane strain parameters. The assumption made by Indraratna et al. was that the average degree of consolidation for both the axisymmetric and the equivalent plane strain conditions should be identical at a given stress level. By assuming that the radius of the axisymmetric influence zone of a single drain (R) could be replaced by half-width (B) in plane strain, the relation between the coefficients of permeability outside the smear zone k_{hp} and inside the smear zone k'_{hp} may be expressed as

$$k_{hp} = \frac{k_h\left[\alpha + \beta \frac{k_{hp}}{k'_{hp}} + \theta\left(2lz - z^2\right)\right]}{\left[\ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k'_h}\right)\ln(s) - 0.75 + \pi\left(2lz - z^2\right)\frac{k_h}{q_w}\right]} \quad [7]$$

The geometric parameters involved in Equation [7] are

$$\alpha = \frac{2}{3} - \frac{2b_s}{B}\left(1 - \frac{b_s}{B} + \frac{b_s^2}{3B^2}\right)$$

$$\beta = \frac{1}{B^2}\left(b_s - b_w\right)^2 + \frac{b_s}{3B^3}\left(3b_w^2 - b_s^2\right)$$

$$\theta = \frac{2k_{hp}^2}{k'_{hp} B q_z}\left(1 - \frac{b_w}{B}\right)$$

$n = R/r_w$

where

R = the radius of the influence zone of the drain;
r_w = (a+ b)/4 is the radius of the drain with *a* and *b* being the width and thickness of prefabricated plastic plate respectively;
s = r_s/r_w, with r_s being the radius of the smear. r_s can be evaluated as 3~4 times of r_w;
z = the depth of the drain under consideration;
q_w = the discharge capacity of the drain;
q_z, b_s, b_w are the equivalent plane strain discharge capacity of the drain and the width of the smear and the width of the drain respectively; $b_w = r_w$; $b_s = r_s$; $q_z = \frac{2}{\pi_B} q_w$.

If both the smear and well resistance are ignored, then the simplified ratio of plane strain to axisymmetric horizontal permeability k_h becomes

$$\frac{k_{hp}}{k_h} = \frac{0.67}{\left[\ln(n) - 0.75\right]} \quad [8]$$

If the effect of well resistance is ignored, the permeability in the smear zone can be isolated by neglecting the final terms of the denominator and numerator, the Eq. [7] can be replaced by

$$\frac{k'_{hp}}{k_{hp}} = \frac{\beta}{\frac{k_{hp}}{k_h}\left[\ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k'_h}\right)\ln(s) - 0.75\right] - \alpha} \quad [9]$$

Where k_{hp} and k'_{hp} are the permeability of the undisturbed horizontal and the corresponding smear zone, respectively.

Table 2 summarizes the equivalent coefficients of permeability under plane strain conditions. It should be noted that the horizontal and vertical coefficients of permeability are the same in the smear zone.

Table 2 Coefficients of permeability of undisturbed and smear zones (10^{-12}m/s)

Soil Layer	k_h	k_z	k_{hp}	k_{zp}	k'_{hp}
1	4000	520	99.1	129	2.57
2	1440	680	357	168	9.26
3-3	1120	630	277	156	7.2
3-4	2320	1700	575	421	14.9
3-5	1680	1240	416	307	10.8

The numerical analysis was carried out using FLAC, a commercial software based on explicit finite difference method. The modified cam-clay model was chosen as the soil models and the model parameters for each soil layer are presented in Table 3.

Figure 4 presents the discretized finite difference mesh used in numerical simulations. Owing to the symmetry of the geometry, only a half of the embankment-subgrade system is taken into account in the mesh. A finer mesh was employed in the zone with prefabricated plastic drainage plates to capture the large pore pressure gradient in this region. The range of the model is 100 m from the embankment centerline and 52.2 m in depth, which is the thickness of compression layer according to theoretical analysis. The width of the analyzed domain was four times of the half of bottom width of the embankment, which is considered large enough to substantially reduce the boundary effects. The upper boundary was set to drain boundary with zero excess pore water pressure. Since the variation of excess pore pressure and the deformation of the subgrade soil are of the major interest, the embankment was replaced by a series of equivalent vertical pressure applied on the upper boundary.

Table 3 modified cam-clay model parameters used in Numerical Analysis

k	λ	M	P_c (kPa)	P_1 (kPa)	ν_λ	e	n	c (kPa)	φ (^{0})	bulk modulus (mPa)	shear modulus (mPa)
0.022	0.109	0.94	204.	100	1.786	0.89	0.471	14.0	15.5	9.225	5.532
0.032	0.130	1.12	86.5	100	1.562	1.22	0.5495	3.7	18.9	5.1	3.06
0.015	0.061	1.13	253.	100	1.974	0.69	0.4083	4.0	21.6	18.78	11.27
0.022	0.087	1.06	290	100	1.974	0.98	0.495	3.0	23.2	20.03	12.01
0.023	0.09	1.07	344	100	1.675	0.99	0.4975	14.0	23.6	12.17	7.296
0.01	0.04	1.244	452	100	1.924		0.4975	0	40	56.5	33.9

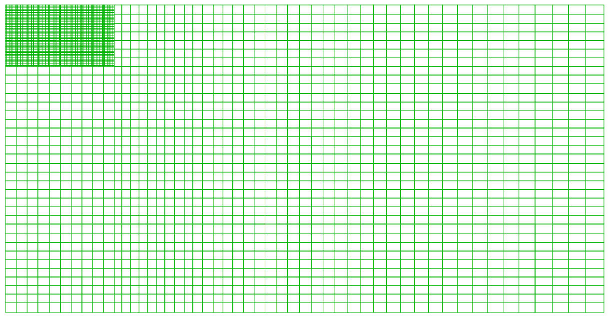


Figure 4 Finite-difference mesh in analysis

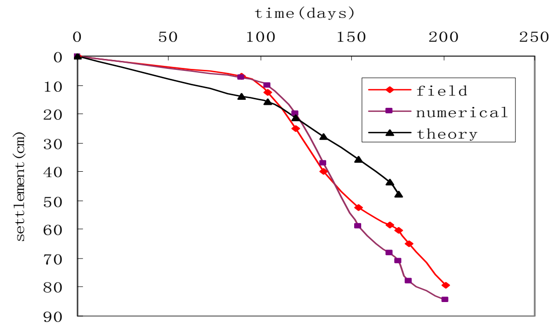
5. COMPARISON OF NUMERICAL RESULTS AND MEASURED DATA

Figure 5 compares the calculated settlements based on the single-drain theory and the equivalent plane strain numerical modeling. One observes from Figure 5a that the ground surface settlements predicted by the single-drain theory are larger than the measured values at early stages of construction. With the increase of time, the theoretical settlements become smaller than measured data. This observation indicates that the immediate and the consolidation settlements may be over and under estimated, respectively, by the theory. In addition, the neglect of the effect of shear stress in the theory may also induce the under estimation of the settlement. On the other hand, the results from equivalent plane strain numerical modelling are generally in good agreement with measure data.

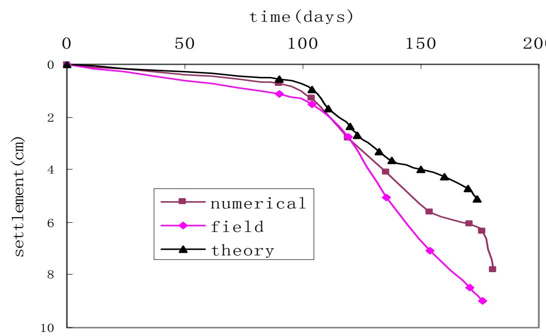
Figure 5(b) presents the comparison of the settlements at the top of the underlayer of the improved zone. In this case, the settlements from the single-drain theory are consistently smaller than the measured ones, owing to the fact that the estimated immediate settlement from Equation [2] becomes very small. The results in Figures 5a and b reveal that the single-drain theory tends to under estimate the settlement of soil improved by prefabricated plastic drainage plates.

Figure 6 compares the predicated and measured lateral displacements at the toe of the embankment slope. Obviously, the single-drain theory cannot give the lateral displacement occurred in soil. Even though the numerical results have the same trends as the measured data, the magnitude of the lateral displacements, however, are very much different: the calculated values are almost 2.5 times of the measured ones. The significant discrepancy in lateral displacement may be attributed to the geogrid installed in the sand blanket. One may expect some improvement of the calculated lateral displacement at the toe if the geogrid is taken into account in numerical simulations.

The variation of excess pore water pressure is another important factor in the prediction of settlement. Figure 7 presents the comparison of excess pore pressure obtained from numerical modelling and field measurement. The calculated excess pore pressure is almost twice of the field measurement. Several factors may be responsible for the discrepancy between the calculated and measured data. For example, in numerical modelling, the soil is assumed fully saturated while the in-situ soils are usually partially saturated. Improper soil parameters used in numerical modelling may also contribute to this discrepancy.



(a) Ground surface



(b) Underlayer of improved zone

Figure 5 Comparisons of embankment center settlement by different methods

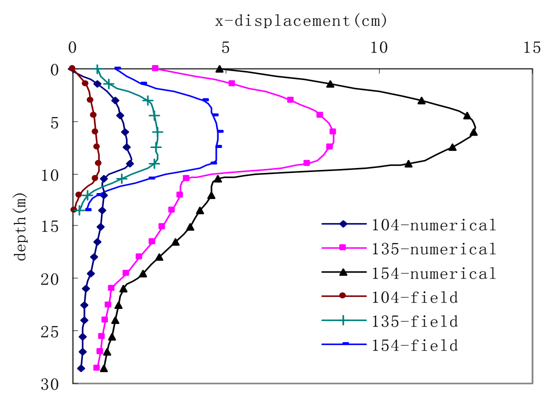


Figure 6 Lateral displacement of slope toe

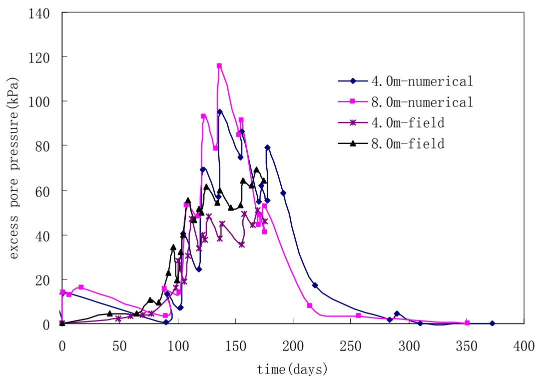


Figure 7 Excess pore pressure at embankment center

In terms of the time required for excess pore water pressure to dissipate completely, for the zone improved with prefabricated plastic drainage plates, close results are obtained from the single-drain theory and the numerical modelling based on the equivalent plane strain approach; see Table 4. However, for soils blow the improved zone, the equivalent plane strain approach predicts less time for complete dissipation of excess pore pressure.

Table 4 Time of excess pore pressure dissipation (months)

	Theoretical	Numerical
Improvement zone	15	13.3
Under layer	>50	40

Table 5 Comparison of settlement (cm)

	Theoretical	Numerical	Field measurement
Final settlement	92.4	91.0	87.0
Post construction settlement	8.9	1.85	
Post construction settlement rate/first year	1.4	1.48	

Table 5 is the comparison of the final settlement, post-construction settlement as well as the rate of settlement in the first year of post-construction. Since the *in-situ* measurement of settlement is currently on going, the final settlement herein is extrapolated using Asaoka method based on available data. One finds that the final settlements obtained from different methods are very close. However, the post-construction settlements estimated from the single-drain theory and numerical modelling are quite different. This may be attributed to the effect of the secondary consolidation that was not taken into account in the numerical modelling reported in this paper. It is also noted that both methods predict almost the same rate of settlement in the first year of post-construction.

6. CONCLUDING REMARKS

This paper presents the preliminary results of a large-scale experimental study on the settlement of embankment on soft clays improved with different methods. The single-drain theory and the equivalent plane strain finite difference method were used to estimate settlements and excess pore pressures when prefabricated plastic drainage plates were used to accelerate consolidation and to reduce the post-construction settlements of a high-speed railway subgrade soil. Based on this preliminary study, the following remarks are made:

1) The predicted final settlement from both the single-drain theory and the equivalent plane strain approach is close to that extrapolated from the available measured data. However, different results were obtained for the variation of the settlement and the dissipation of excess pore pressure with time. The numerical method based on the modified cam-clay model incorporated in two-dimensional explicit finite difference program FLAC provided comparable settlements with the measured data. However, it tends to over predict the excess pore pressure, particularly in the zone between two plastic drainage plates.

2) The lateral displacements at the toe of the embankment obtained from numerical simulations are larger than the field measurement, most likely due to the influence of the geogrid installed in the sand blanket.

3) According to the results from both modelling and f measurement, when prefabricated plastic drainage plates are used to accelerate the process of consolidation, the post-construction settlement for the experimental embankment is less than 10 cm and the rate of settlement in the first year of post-construction is less than 2 cm/year. The requirements in the current design guidelines are satisfied.

7. ACKNOWLEDGEMENTS

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