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SYNOPSIS Ningbo International Airport is built on the soft clay ground first in China. It has a runway 3,000m in length and 45m in width. The thickness of soft clay is greater than 32m. The calculating consolidation settlement is 0.48m, far beyond to allowable limit. The soft clay ground is improved by wick drain and surcharge precompression. The airport has been put in service since July, 1990. The process and the results of a full scale embankment test, the ground improvement and the settlement observation are described. The influence of the smear and the well resistance of the wick drain on the consolidation rate are discussed.

INSTRODUCTION

Ningbo International Airport runway is 45m wide and 3,000m long. The runway centre is 1.7m higher than the original ground level. The runway surface is rigid. In order to fly safely, it is necessary to strictly limit the subgrade settlements less than 8cm and the uneven settlements in the cross section less than 4cm during the service life.

Ningbo International Airport is located in the Ning-Feng plain in the southwest of Ningbo city. The smooth terrain is criss-crossed by rivers and canals. The first layer in the ground is 0-1.0 m clay. The substratum is 0.35m peaty in both ends of the runway. For entire runway range, the sublayer is the soft clay which thickness varies from 7m to more than 32m. The ground is composed of nine layers which indexes of the main soil layers are showed in Tab. I

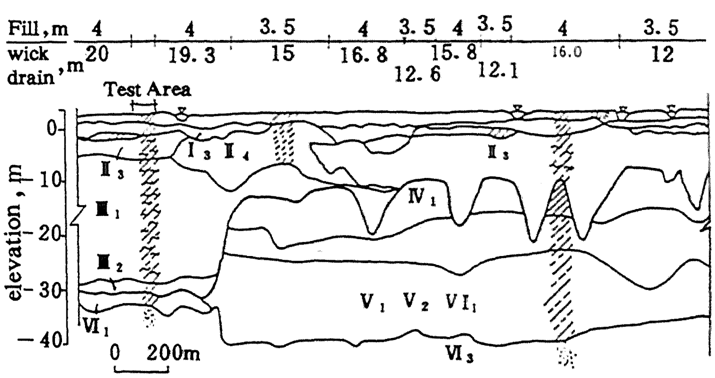

Fig. 1. Geological Section and Precompression Programme

According to calculating, the maximum consolidation settlements are greater than 48cm under the design load in the centre of the runway. And the uneven settlenemts in the cross section of the runway are greater than 11cm. To keep the design elevation of the runway during the service life the runway must be preraised. In this way, the settlements will be greater than 67cm. The rigid runway can not bear so large settlements. Therefore, the original subgrade can not meet the needs of the runway to the settlements and the ground has to be improved.

The airport is the first airfield to be built on the thick soft clay ground in China. The wick drain and the surcharge precompression are used to improve the subgrade of the project. This paper presents the process and results of the ground improvement.

TABLE I. Properties of Main Soil Strata

No.	Soil	w (%)	ρ (g/cm³)	e_0	I_p (%)	I_L	a_{1-2} (MPa⁻¹)	C_u (kPa)
I_1	clay	34.65	1.89	0.955	18.4	0.41	0.50	50
I_3	Mucky Soil	52.80	1.72	1.447	17.4	1.38	1.14	24
II_3	Muck	49.00	1.75	1.341	15.5	1.58	1.15	22
III_1	Mucky Soil	48.95	1.74	1.352	16.9	1.44	1.08	30

INDOOR TEST AND THEORETICAL ANALYSIS

The test results show that the surcharge reduces the residual deformation of soil. And the effect of the surcharge depends on the precompression load and time. When the load remains constant and the time changes, the test results are showed in Table II.

TABLE II. Effect of Preloading Time on Residual Deformation

Condition No.	Amount of Unloading, %	Preloading Time, min	Reduction of C_s
1	50	2	2/3
2	50	5	5/6

The analyses of the test results show that the deformation after unloading is consist of the suction consolidation deformation, the creep of resilience and the secondary consolidation deformation. The more the surcharge is, the greater the swelling deformation is. The longer the precompression time is, the less the residual deformation is (Zhu et al. 1991).

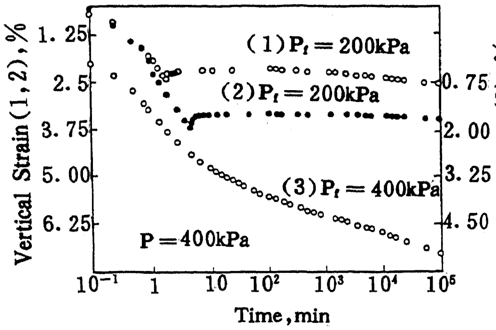

Fig. 2 Relationship of Vertical Strain and Time

As stated above, the residual deformation after unloading depends on not only the percent consolidation before unloading but also the amount of surcharge. The effective stress area ratio, R, is able to reflect the influence of the factors. R is defined as the ratio of the effective stress area under the permanent load to the effective stress area before unloading in the vertical direction. For one-dimensional compression condition under the large scale load, the simple formula is as

$$R = P_t / (\bar{U} \times P) \quad (1)$$

where P_t and P are correspondingly the permanent load and the fill; $\bar{U}$ is the average percent consolidation of the ground.

The residual deformation of soil depends on R. The larger the R is, the larger the residual deformation is. While R is less than a threshold the soil will have only the swelling deformation during some time, as shown in Fig. 3. So, the controlling for R can obtain the goal of controlling residual deformation.

Theoretical analysis and test results show that if R is less than 0.75 the residual settlement will be very small during 20 years for Ningbo International Airport runway ground (Pan et. al 1991).

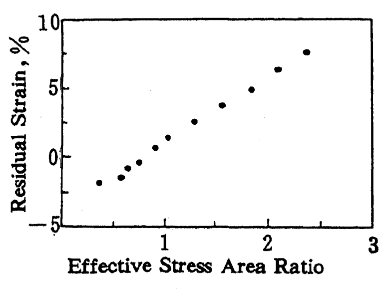

Fig. 3. Relationship of Effective Stress Area Ratio and Residual Strain

FULL SCALE EMBANKMENT TEST AND RESULTS ANALYSIS

In order to decide the detail programme of the ground treatment a full scale embankment with 108m long and 78m wide is built. The ground is improved by the surcharge precompression and the wick drain with diameter 7cm, length 20m and distance 1.4m between wick drains. The test began in October, 1986, and ended in December, 1987. The fill is consist of the sand mat with the thickness, h, 0.35m and the density, ρ, 1.53g/cm^3; the roller compation slag fill with h = 1.22m and ρ = 2.0g/cm^3; the semi-roller compation slag fill with h = 1.30m and ρ = 1.80g/cm^3 and the slag fill with h = 1.25m and ρ = 1.55g/cm^3. The practical surcharge is 1.60m thick and is approximately 36.6 percent of the fill.

To monitor the behavior of the test embankment, some observation measurements, such as the settlement plates, the telescoping tube settlement gauges, the deep settlement gauges, the piezometer tips and the border piles, are fixed up, as shown in Fig. 4. Fig. 5 expresses the relationship of the settlement and the time.

The settlement results show that the ground produces small swelling after unloading. Soon afterwords the deformation becomes gradually stable. The pore water pressure decreases with unloading and becomes a suction pressure. After that the suction pressure dissipates gradually. So, the ground will not take place the consolidation settlement under the design load. Tab. 3 expresses the calculating values from observation results.

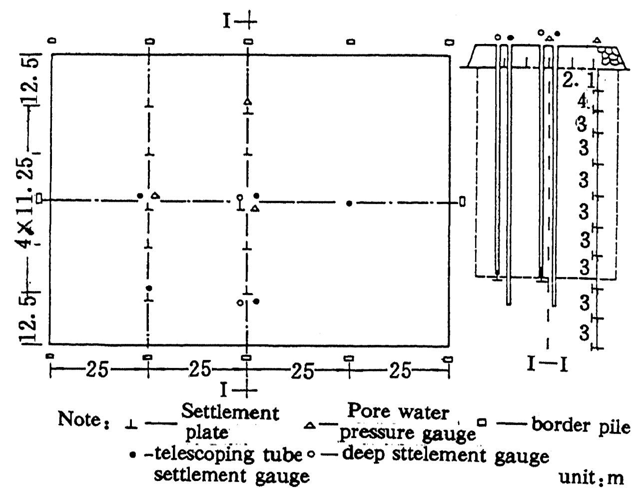

Fig. 4. Instrumentation Plan of Test Embankment

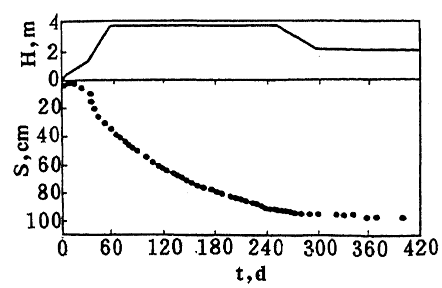

Fig. 5. Relationship of Settlement and Time

TABLE Ⅲ. Calculating Results from Test

Location in Runway	S_∞ (m)	S_t (m)	B (1/d)	$\bar{U}$ (%)	R
Centre	1.110	1.110	0.0070	81.4	0.78
Side	0.952	0.913	0.0076	82.0	0.70

From Tab. Ⅲ it can be known that R of the subgrade is 0.78

and little greater than 0. 75. So, the ground will still take place a small amount of settlement during the service life. According to calculating, the ground before unloading has produced 108% of the settlements under the design load which the preraising of the runway is considered. Therefore, there are not the consolidation settlements, S_c, after unloading. But there still are a part of the secondary consolidation settlement. The observation results of Duhu dam sand drain ground show that the secondary consolidation settlements during 20 years is about 6% of total settelements, S_∞. So, even though the decrease of the coefficient of secondary consolidation caused owing to the surcharge precompression is not considered, the secondary consolidation settlements in the centre and in the side of the runway during the service life of 20 years has correspondingly only 5. 03 cm and 4. 66cm. Therefore the settlements and the non-uniform settlements can meet the demands.

GROUND IMPROVEMENT PROGRAMME AND CONTROLMENT OF UNLOADING

Under the basis of the test study it is decided that the surcharge preloading and the wick drains are used to improve the soft clay ground. And the effective stress area ratio is used as a main standard of the design and the construction.

On Ground Improvement Scheme

From the consolidation factor, B_0, calculated from the test results, the consolidation fator, B_1, while the distance between the wick drains is changed can be calculated. According to the geological condition the lengths of the wick drain in the different location of the runway are decided. In accordance with the settlements and $\bar{U}$ calculated R is obtained. If the R is less than 0. 78, the programme is considered to be suitable. Tab. Ⅲ shows 3 kinds of schemes. Combining with the time limit for the project and the economical analysis of the programme, the practical ground improvement scheme of the surcharge preloading and wick drains with 7cm in diameter and 1. 5m in distance between wick drains is showed in Fig. 1. The amount of wick drains for the first phase of the project is 158, 900.

ABLE Ⅳ. Fill Programme for the Second Phase of the Project

Fill (kPa)	S_c (m)	P_{0f} (kPa)	$\bar{U}$ (%)	P_f (kPa)	R
85	1. 46	30. 9	95	63. 2	0. 78
90	1. 52	30. 9	90	62. 8	0. 78
95	1. 58	30. 9	85	62. 2	0. 77

On Controlment of Unloading

In order to monitor the ground behavior 59 settlement plates are placed in the fill for the first phase of the project. The exponential function method is used to calculate the final settlements, the immediate settlements, the consolidation factor and the average percent consolidation from the observation results. The fill, P, the original design load, P_{0f}, and the design load considering practical settlements, P_f, can be calculated. So, R can be gained from formula 1. If R is less than 0. 78, the surcharge is permitted unloading. Owing to the inhomogeneity of soil, the ground in some locations produces more settlements than that calculated. For these locations, the fill designed is not enough. So, it is essential that the second fill is loaded on time. Through the precompression of more than 7 months, the surcharge is unloaded. Tab. V describes a part of the result of the ground improvement.

TABLE V. Results of Ground Improvement for the First Phase of the Project

No.	Settlements (m)	$\bar{U}$ (%)	P_f (kPa)	P (kPa)	R
1	1. 307	75. 7	63. 3	110. 3	1. 76
2	0. 725	85. 8	41. 4	64. 4	0. 74
3	1. 156	72. 6	58. 4	104. 1	0. 77
4	0. 946	77. 7	47. 8	78. 5	0. 78

ON WELL RESISTANCE AND SMEAR EFFECT

The radial average percent consolidation of the sand drain ground is given by Hansbo (1979) as follows

$$U_h = 1 - e^{-8T_h/F} \quad (2)$$

$$F = F_n + F_s + F_r \quad (3)$$

where $T_h = C_h t/d_e^2$. C_h is the coefficient of consolidation in the horizontal diresttion. d_e is the equivalent drain diameter. F is the factor which expresses the additive effect due to the spacing of the drain, F_n; smear effect, F_s; and well resistance, F_r. For typical values of the spacing ratio, n ($n = d_e/d_w$, d_w is the diameter of sand drain), of 20 or more, the spacing factor, F_n, can be simplified to

$$F_n = \log n - \frac{3}{4} \quad (4)$$

$$F_s = (\frac{k_h}{k_s} - 1)\log \frac{d_s}{d_w} \quad (5)$$

$$F_r = \pi z(2L - z)\frac{k_h}{q_w} \quad (6)$$

Where k_h and k_s are correspondingly the permeability coefficient of the original soil and the remolded soil in the horizontal direction; L is the maximum drain distance in the vertical direction; z is the distance from the drainage end of the drain; q_w is the discharge capacity of the drain at hydrulic gradient of 1.

$$q_w = k_w A_w = k_w \pi d_w^2/4 \quad (7)$$

Where k_w is the permeability coefficient of sand; d_s is the diameter of the disturbed zone. Jamiolkowski et al. (1981) have proposed that the diameter of disturbed zone is related to the diameter of the mandrel, d_m, as $d_s = (2.5 \text{ to } 3)d_m$. Hansbo (1987) has recommended the relationship as $d_s = 2d_m$.

Formula 2 can be rewritten as

$$U_h = 1 - e^{-Bt} \quad (8)$$

$$B = 8C_h/(d_e^2 F) \quad (9)$$

According to above formulas, calculating the consolidation of the wick drain ground, the results are shown in Fig. 6-Fig. 10.

Fig. 6 and Fig. 7 express the results that the smear effect is not considered, $F_s = 0$. Fig. 6 expresses that U_h is dispered over the depth for the different k_w before unloading (t=220. 5d). Fig. 7 shows the relationship of k_w and the ratio of consolidation factors, B_i/B_r, where B_i is the factor of the ideal sand drain ground and B_r is the factor for $F_s = 0$. It can be found from Fig. 6 that the well resistance holds up the consolidation of soil. The smaller the k_w is, the greater the well resistance effect is. The well resistance effect will be very clear as k_w is less than 10^{-2} cm/s. As Fig. 6 shows, the difference of the average percent consolidation of the ground is 28. 5 percent while k_w are 2.0×10^{-2}cm/s and 2.0×10^{-3}cm/s. Thus, to decrease the well resistance effect it is necessary that the coefficient of permeability of sand is required more than 10^{-2} cm/s in engineering.

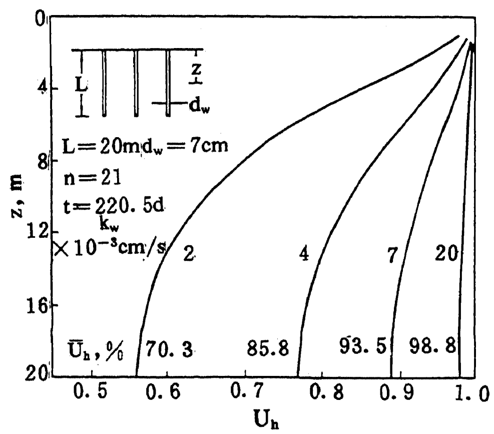

Fig. 6 Relationship of U_h and depth for different k_w

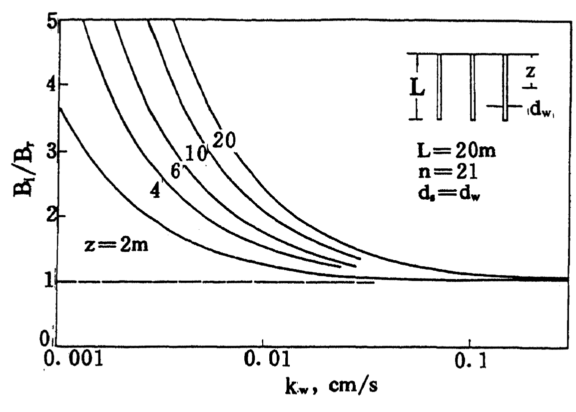

Fig. 7 Relationship of k_w and B_i/B_r

Fig. 8 gives the relationship of the ratio of the permeability coefficients, k_h/k_s, and the ratio of the consolidation factors, B_i/B_s, where B_s is the consolidation factor that the well resistance is not considered, $F_r = 0$. From Fig. 8 it can be seen that the smear effect will increase with the increasing of the disturbed zone and the increasing of k_h/k_s. But the smear effect does not change with the increaseing of depth. Analysising Fig. 8 it can be found that if the coefficient of permeability in the smear zone is 1/10 of the permeability coefficient of in the undisturbed zone, the ratio of the consolidation factors will be 5. 3 while the diameter of the smear zone is 3 times that of the sand drain. Thus it can be seen that the smear effect must not be ignored.

For Ningbo International Airport test zone the diameter of the mandrel, d_m, is 10. 5cm, and the permeability coefficient of clay in the horizontal direction, k_h, is 9.88×10^{-8} cm/s. Supposing $d_s = 2d_m$, the average percent consolidation of the ideal sand drain ground calculated before unloading is 99. 65 percent. But the average percent consolidtion of the practical sand drain ground calculated from the observation results is only 81. 4 percent. So, the decrease amount of the average percent consolidation owing to the well resistance and the smear effect ec al (the nonidealization of the sand drain ground) is 18. 25 percent. Thus it can be seen that the effect of the nonidealization of the sand drain to the consolidation rate is very large in Ningbo International Airport test zone

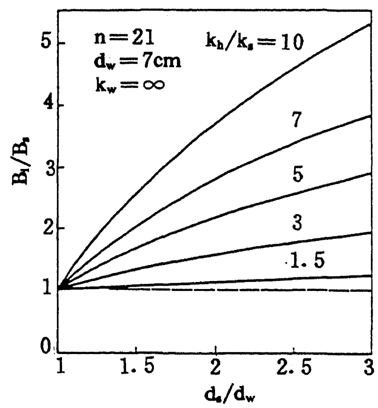

Fig. 8 Relationship of k_h/k_s and B_i/B_s

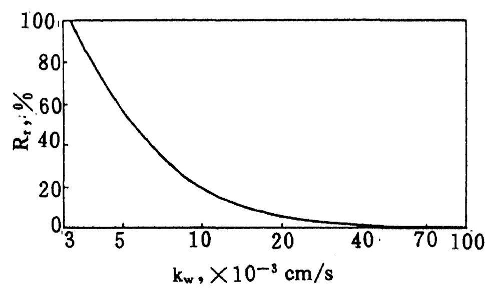

Fig. 9 Relationship of R_r and k_w

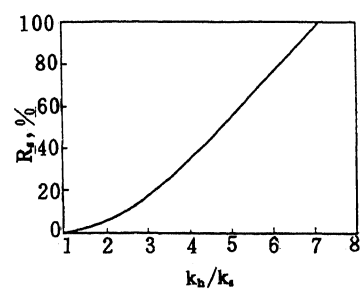

Fig. 10 Relationship of R_s and k_h/k_s

Fig. 9 shows the relationship of the proportion that well resistance makes up the nonidealiztion of the sand drain ground, R_r, and the coefficient of the permeability of sand, k_w. Fig. 10

gives out the relationship of the proportion that smear effect makes up the nonidealization, R_s, and the ratio of the coefficients of the permeability, k_h/k_s. From Fig. 9 it can be seen that the proportion of the well resistance to the nonidealiztion will rapidly decrease, but the proportion of the smear effect to the nonidealization will increase fast as the permeability coefficient of the sand increases. With the increasing of the disturbed extent of soil, the proportion of the smear effect to the nonidealization will increase.

For the test zone ground, the k_w is about 7×10^{-3}cm/s to 2.1×10^{-2}cm/s. It can be seen that the well resistance accounts for 6% to 35% of the total nonidealization. And the smear effect makes up more than 65% of the nonidealization. Thus the smear effect is a main factor leading to the nonidealization of the wick drain ground in the test zone.

SUMMARY

While the permeabikity coefficient of sand is great than 10^{-2} cm/s the main factor leading to the nonidealization of the sand drain ground is the smear effect. So studying and developing the new machine and equipment and the new method setting up the sand drain which have less smear effect will have practical significant.

Ningbo International Airport project has been in service since July, 1990. According to monitor, the settlement of the ground remains stable. The project obtains good economical benifit. The surcharge precompression has been also successfully applied in the ground improvement of Wenzhou International Airport and Shenzhen International Airport since 1988 in China.

ACKNOWLEDGEMENT

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REFERENCE

Hansbo, S. (1979), "Consolidation of Clay by Bandshaped Prefabricated Drains," Ground Eng'g., Vol. 12, No. 5, pp. 16-25.

Hansbo, S. (1987), "Design Aspects of Vertical Drains and Lime Column Installations", Proc. 9th Southeast Asian Geot. Conf., Bangkok, Thailand, Vol. 2, pp. 8-1 to 8-12.

Pan, Q. Y., Zhu, X. R. and Xie, K. H. (1991), "Some Aspects of Surcharge Precompression on Ground with Sand Drains", Chinese Journal of Geot. Engin., Vol. 13, No. 2, 1-12.

Zhu, X. R. and Pan, Q. Y. (1991), "The Deformation of Subsoil after Surcharge Unloading", J. of Zhejiang University, Vol. 25, No. 2, pp. 246-256.