FIELD BEHAVIOR OF PVD-IMPROVED SOFT DEPOSIT NEAR JIUJIANG, CHINA

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ABSTRACT

This paper presents a case history of the performance of a full-scale test embankment constructed on very soft clay deposit in middle part of China, near Wuhan. The embankment was constructed on prefabricated vertical drain (PVD) improved subsoil. The height of the embankment is 5.3m and construction time was about one year. The thickness of soft deposit with PVD-improved was 13 m. The PVDs were installed to a depth of 8.5 m with spacing of 1.5 m in a triangular pattern. To investigate the performance of this embankment construction and the effect of PVDs, a comprehensive instrumentation program was conducted during construction. Field observation included excess pore pressure, ground settlement, and lateral displacement. Analysis of field data indicated that the ground settlements were small for a long construction and low height. This construction in general exhibited different behaviours than other projects because of his relatively long construction and low height. This project serves as a special case study and provided insights into the design and s of a long duration construction.

Keywords: Embankment construction, soft subsoil, PVD improvement, field measurement

INTRODUCTION

In order to speed up consolidation of soft subsoil and shorten the time of settlement, prefabricated vertical drain (PVD) were generally used prior to embankment construction on soft subsoil (Bergado et al. (1996), Chai et al. (1996), Hansbo (1981), Jamiolkowski et al. (1983), Miura et al. (1993), Rixner (1986) and others.). In this application, PVD reduce the drainage path and accelerate the dissipation of excess pore water pressure generated during the embankment loading. According to the specific application, key design parameters of PVDS vary, such as the equivalent diameter, the discharge capacity of the drain. Designers usually rely on simplified analytical to quantify the effect of the drains because of the uncertainty associated with PVD-improved soft subsoil under embankment loading. The best –known study on this topic was carried out by Barron (1948) under the assumptions that the subsoil is a uniform soil column with linear compressibility characteristics. Some other researchers also presented simplified solutions of consolidation problem of PVD improved soil. Beside these solution methods, the finite element method is often used to analyze the response of soft subsoil under embankment loading (Chai et al., 1993), Hird (1992)), and others, since FEM can be employed with less restrictive conditions.

In this study, a case history of the performance of a full-scale test embankment constructed on very soft clay deposit in middle part of China, near Wuhan, was investigated by settlements, lateral displacement and excess pore pressure. This construction in general exhibited different behaviours than other projects because of his relatively long construction and low height. This project serves as a special case study and provided insights into the design and s of a long duration construction.

PROJECT DESCRIPTION

The Beijing-Jiujiang-Kowloon Railway, also known as the Jingjiu Railway, is a railway in the China connecting Beijing and Kowloon. The total length of Jingjiu Railway is 2,397 kilometres, of which about 35 km was constructed on soft clay deposit. Prefabricated vertical drains were applied to

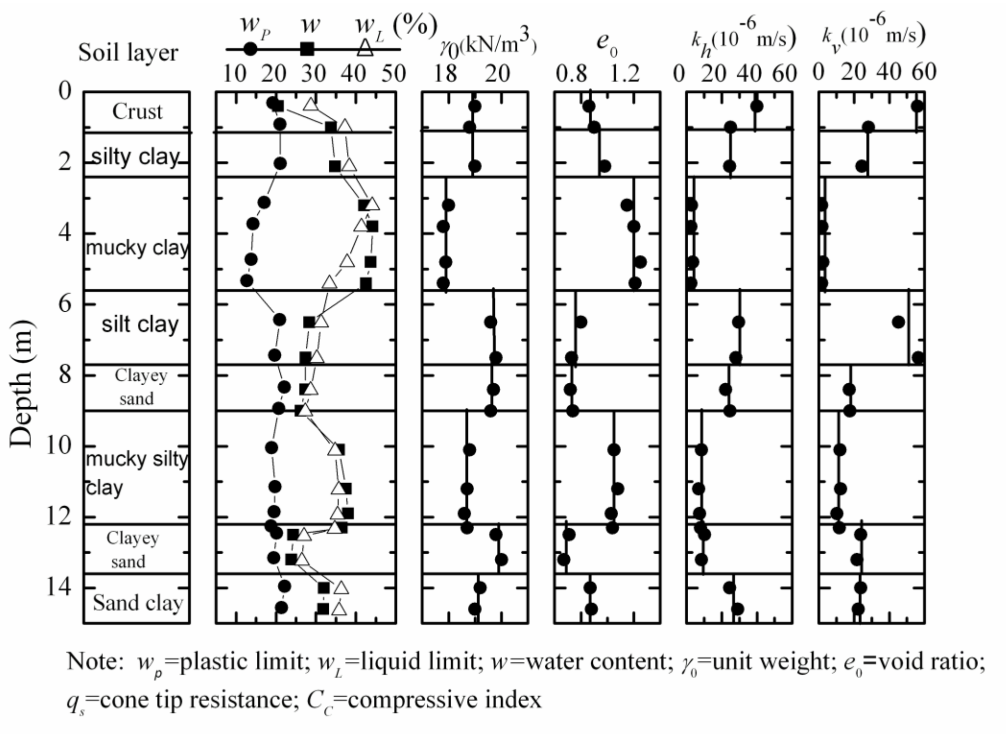


Fig. 1 Soil profile and properties at the test site

reduce postconstruction and increase the shear strength of soft soils. In order to better understand the design, construction, and performance of embankment on the soft clay deposits, some field full scale test embankment totally 200m in length were constructed and investigated. The investigated embankment construction is located middle part of China, near Wuhan in which features high groundwater tables and thick soft clay in the upper layers. The height of the embankment on PVD improved subsoil is 5.1-5.7m.

SOIL CONDITIONS

Before the construction, soil conditions at the site were explored by a series of field exploration programs. In general, the site was characterized by a thin weathered crust (TC) in the upper 1-1.5m following by a silty clay (SC1) deposit of approximately 2.5m thick. The next layer was very soft clay (MC, it is called mucky clay) with a thickness of approximately 4m for the PVD improved case. Beneath the clay, a silt clay layer (SC2) with a thickness of approximately 2m was underlain by a layer of loose clayey sand (CS1). The fifth layer is silt clay call mucky silty clay (MSC) approximately 4m thick followed by clayey sand (CS2). Beneath CS2 is a medium to stiff silty clay layer (SC3). The soil profile and soil properties of the soft deposits at the test site were shown in Fig. 2.

The soil properties along the depth were characterized by a series of laboratory tests and in situ test, such as oedometer test, triaxial tests, and unconfined compression tests. The compression index and hydraulic conductivity were determined by oedometer tests. Shear strength were obtained from consolidated undrained triaxial tests. The coefficient of compressibility, a_v(0.1-0.2), were defined by oedometer tests at stress ranges of 100-200kPa. The hydraulic conducticity were determined from laboratory oedometer tests. The observed long-term groundwater table at the test site was about 1.0m below the surface. The very soft clay layer and silty clay layer had water contents greater than their liquid limits. The soft clay exhibited medium sensitivity of about 2.0-7.0, which implied that the strength and stiffness of the soft clay would degrade once it was subjected to disturbances arising from construction activities. Both laboratory and field testing results indicated that the soil in test site feature relatively high water content, high void ratio, high compressibility and low strength.

CONSTRUCTION PROCEDURE AND FIELD MONITORING

The height of investigated embankment on PVD improved subsoil was 5.3m. Fig.2 presents the cross section and plan view of the test embankment. First, a 0.6m thick sand mat was placed on the soft ground initially with hydraulic conductivity: k>0.001m/s. Thereafter, decomposed granite was filled and compacted to a unit weight of 20kN/m^3. Figure 3 shows the loading time history for the embankment.

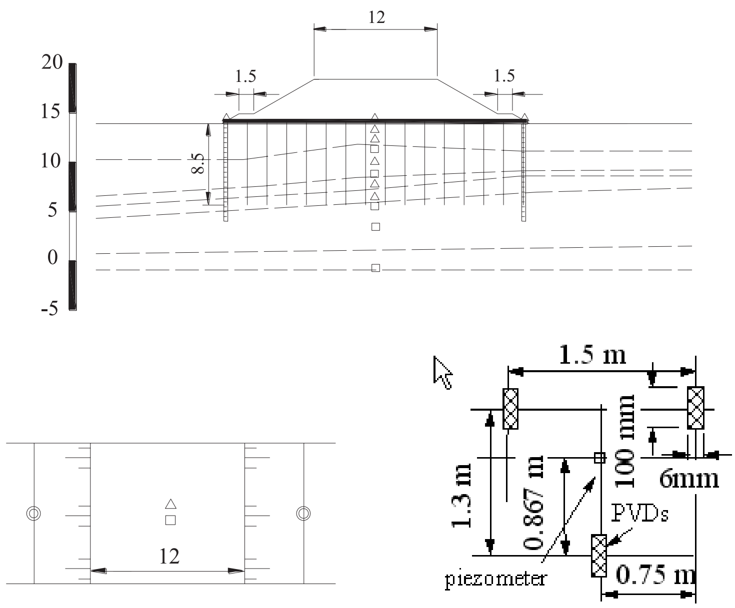


Fig. 2 Cross-section and plan view of embankment, field instrumentation and arrangement of PVDs in field

Table 1 Size and physical properties of PVDs used in this field project.

Thickness (mm)	Width (mm)	Unit weight (g/m)	Q_w* (m^3/yr)	Material	
				Filter	Core
4	100	100	1500	Nonwoven Polyolefine	polyethylene

* Provided by manufacturer.

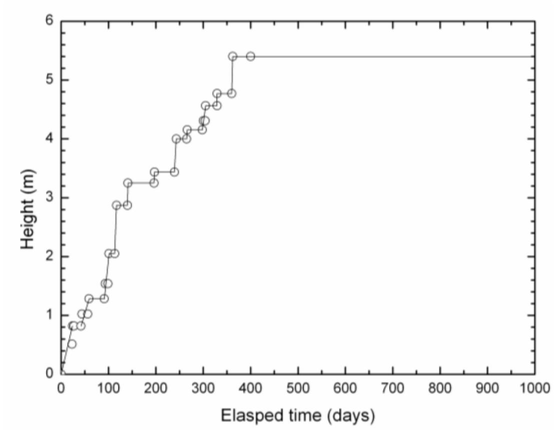


Fig. 3 Load time history of the embankment

Table 1 lists the properties of PVDs used in the test embankment, which were installed in a triangular pattern with a spacing of 1.5-8.5m deep. The discharge capacity, Qw, provided by manufacturer is greater than 1500m3/year.

To monitor the performance of this PVD improved subsoil, a comprehensive field instrumentation program was conducted (Qiao, et al 1997, Gu, 1999). The observed performance included the surface settlement, lateral displacement of subsoil at the middle point of the embankment side slope, and pore pressure variation by piezometers at different depths. The used instruments are inclinometers, settlement indicators, and piezometers. Inclinometers were placed at the toe of the embankment where excessive lateral movement is of some concern. Settlement plates are installed immediately after installing the vertical drains, measuring the elevation of the top of the reference rod. Surface settlement points measure vertical displacement with depth, for ample, along an embankment centerline. Settlement is evaluated periodically until the surcharge embankment is completed; Piezometers were installed at the bottom of the sand blanket, at various intermediate depths within the compressible layer. Figure 2 shows the layout of the field instrumentation.

SETTLEMENTS

The development of ground settlements at center and toes of the embankment are shown in Fig.4. The ground settlements almost increased linearly with time during the first 100 construction days when the height of embankment is lower than 2.8m. The maximum settlement in 1 day was about 9mm.

However, the maximum settlement in 1 day was approximately 14mm when the height of embankment was 2.8m and 4.8m. This phenomenon may be attributable to the following factors: i) reduction of consolidation yield stress of crust; ii) a localized failure occurs. The maximum ground settlement was 290 mm at the end of embankment construction with elapsed time of about one year. The ground settlement increased approximately 100 mm after 240 days.

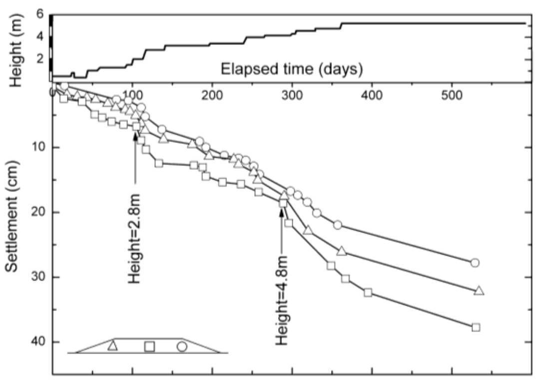


Fig. 4 Development of the ground settlements (Data from Qiao et al 1997)

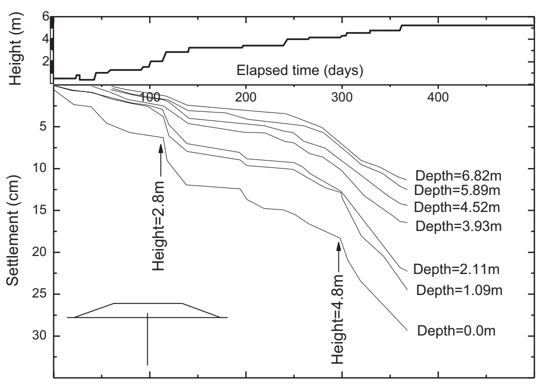


Fig. 5 Settlements at different depth

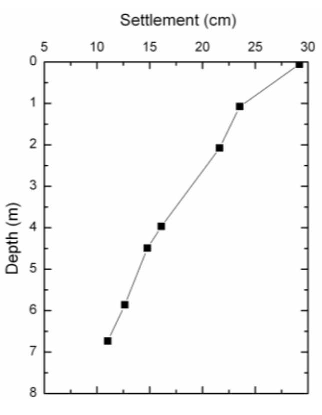


Fig. 6 Settlement vary with depth at the end of embankment construction

Figure 5 presents the development of settlement at different depth. All the settlements at different depth almost increased linearly with time during the first 100 construction days when the height of embankment is lower than 2.8m. The settlement at depth=1.09m and depth=2.11m showed development patterns similar to that observed at surface when the height of embankment was 2.8m and 4.8m. Figure 6 shows that settlement varies with depth at the end of embankment construction. It can be seen that the settlement at different depth almost decreased linearly with depth.

LATERAL DISPLACEMENT

The measured lateral displacements with depth at embankment toes are summarized in Fig. 7. The lateral displacements at ground surface are almost zero. It can be clearly seen that the depth with maximum lateral displacement decreased when the height of embankment increased. The maximum lateral displacement of left side is about 70mm, while the value of right side is about 55mm at the end embankment construction. The place where maximum lateral displacement occurred is approximately 6m blow ground surface. The difference of measured lateral displacement of the two shoulders of embankment may be due to the non-uniformity of the in situ soils (see Fig. 2).

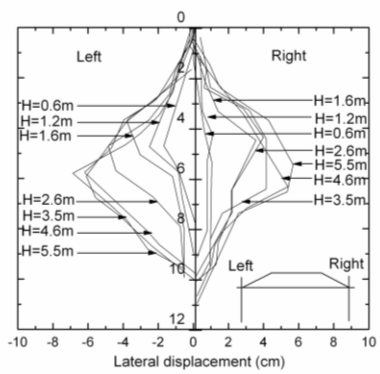
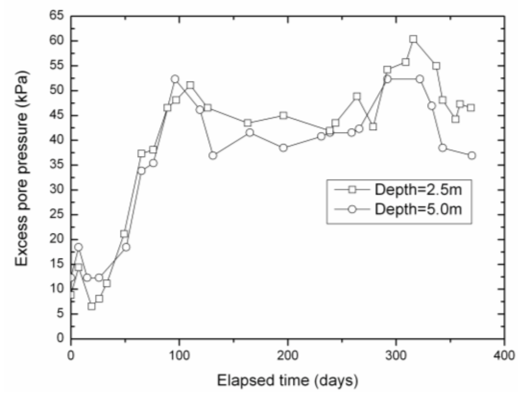


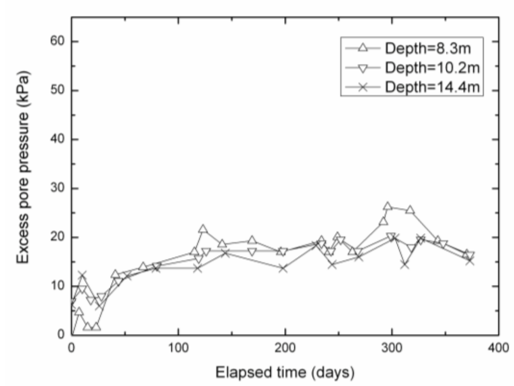
Fig.7 Measured lateral displacement

PORE WATER PRESSURE

Figure 8 shows the observed excess pore pressures during embankment construction. As shown in Fig 8a, the measured pore pressure at a depth of 2.5m and 5.0m increased quickly when the height of embankment increased to 2.8m. Then, the excess pore pressure dissipated with progressing of consolidation. However, there were little excess pore pressure dissipations during the consolidation after that. The pore pressure increased again when the height of embankment was 4.8m till the end of construction, and then decreased with time. The pore pressures at a depth of 8.3m, 10.2m and 14.4m exhibited different patterns with those observed at 2.5m and 5.0m. The piezometers showed no excess pore pressure dissipation during the consolidation period till the end of construction. One of the reasons is that the depth of installed PVDs is 8.5m. The subsoil below 8.5m has not been improved, and remained low coefficient of consolidation. The other reason may be that piezometers might be installed in the smear zone. If piezometers were installed in the smear zone, higher excess pore pressure would be measured.



(a)



(b)

Fig. 8 Pore pressures at different depth under the centre of the embankment test section

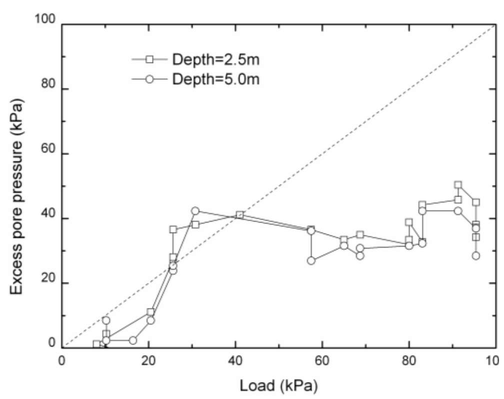


Fig. 9 Pore pressure measured under the centre of the embankment test section as a function of the embankment load

Figure 9 shows that the observed excess pore pressures during embankment construction, which are presented as a function of the applied stress of the embankment at the ground surface. As shown in Fig. 9, the measured excess pore pressures did not agree with the tendency found by Travenas and Leroueil (1980), that is, two phases of pore pressure response during construction can be identified: the first, during which the increases in pore pressure are low; the second, during which the increase in pore pressures is approximately equal to the increase in total stress. The increases of excess pore pressure in the early stages of loading were larger than the increase in total stress. This phenomenon may be due to the reduction of consolidation yield stress of crust layer. In this site, there is a weathered top crust layer, which has much higher strength and stiffness. The stiff top crust may tend to distribute the loading from embankment over a larger area and reduce the level of stress to the underlying of softer subsoils. Moreover, top crust layer has a very high overconsolidation ration and has a relative high coefficient of consolidation. However, installing PVDs will disturb the top crust layer, reduce its consolidation yield stress, and reduce the coefficient of consolidation. Hence, the increase of excess pore pressure becomes larger in the shallower depth at the early stage of embankment construction.

CONCLUSIONS

The following conclusions are based on the field measured:

1. The ground settlements almost increased linearly with time during the first 100 construction days when the height of embankment is lower than 2.8m. The settlements at shallower depth increased quickly when the height of embankment was 2.8m and 4.8m.
2. The lateral displacements at ground surface are almost zero. The maximum lateral displacement is about 70mm with a depth of 6m.
3. The measured pore pressure at a depth of 2.5m and 5.0m increased quickly when the height of embankment increased to 2.8m, and then decreased. After that, there was almost no excess pore pressure dissipation during the consolidation period till the end of construction. The pore pressure at deeper depths exhibited no dissipation during embankment construction.

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