

Case Study on Long-term Ground Settlement of Reclamation

Project on Clay Deposits in Nansha of China

by

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ABSTRACT:

In the design of reclamation, long-term settlement is seriously concerned. This paper presents a case study on the ground settlement of a reclamation project in Nansha, China. Fourteen undisturbed soil samples were taken at different depths from this site. Both oedometer tests on fourteen specimens and triaxial hydraulic conductivity tests on nine specimens were conducted. Finite element (FE) modellings using soft soil creep (SSC) model or soft soil (SS) model for modelling clayey soils with or without creep and the calculation using a new simplified method were performed to analyze the consolidation settlement of clay deposits subjected to filled loadings. The FE simulated results and calculated results from the new simplified method were compared with the monitored data in the field. Results show that the curves of the FE modelling using SSC model and the calculation from new simplified method have the same trends as those of the measured settlements. The creep contributes to excess pore pressure dissipation during the consolidation. Furthermore, ground settlements of the reclamation are predicted with the aid of the FE and new simplified method analysis. These predictions can provide the essential information for engineers to control or reduce the settlement in future.

Keywords: ground settlement, creep, cone penetration tests, Nansha clay

1. INTRODUCTION

With the rapid development of China, many infrastructures have been constructed in coastal cities. Guangzhou is a typical coastal city locating in the Pearl River Delta area of Guangdong Province, China. Nansha is the strategic center for the future development of Guangzhou. However, serious settlement problems occurred in many projects of this city because of less concern on the consolidation settlement of thick clayey layers. In the newly built coastal apartments in Nansha, the settlements were more than 15 cm per year, which would induce the breakage of pipelines (Zhu et al. 2017).

Nansha clay is an interactive marine and terrestrial deposited soft clay. Some researchers have studied the characteristics of Nansha clay including micro-structure, compressibility behaviour, and viscous compression (Chen, Huang, and Liang 2003; Luo and Chen 2014; Liu, Wu and Wu 2010). Luo and Chen (2014) conducted a series of experimental tests such as oedometer test, shear box test, and triaxial creep test to study the creep characteristics of Nansha soils. They found that the creep compression is nonnegligible. Zhu et al. (2017) utilized the fractional derivatives to predict the one-dimensional (1-D) compression behavior including the viscosity deformation of the clayey soils in Nansha. However, most of related researches focus on the elementary behavior analysis of clayey soils in Nansha, there are very limited studies in applying the element tests into the practical applications such as the ground settlements.

The settlement problems are usually influenced by construction loadings, complex geological history, drainage conditions, and ground water variation *etc.* (Shen, Wang and Cheng 2017; Shen et al. 2014). The long-term settlement of the clayey soil usually consists of consolidation and creep settlement. Recently, Yin and Feng (2017) proposed a new simplified

method to calculate the long-term consolidation settlement of clayey soils with creep subjected to an instant loading. In history, there is a controversy on the relation of consolidation and creep: Hypothesis A regards that the creep only occurs after the consolidation of soils (Mesri and Vardhanabhuti 2005; Mesri 2009). Comparatively, Hypothesis B supports that the creep happens both during and after the consolidation stage (Bjerrum, 1967; Stolle et al., 1999; Nash and Ryde, 2001; Yin and Graham, 1989, 1994; Yin, et al., 2002; Nash and Brown, 2013; Degago, et al., 2011). Afterwards, Feng and Yin (2017) developed this new simplified method for calculating the consolidation settlement of double-layered soils with creep under the time-dependent loading. The accuracy of the simplified method was validated by the finite element modelling. However, the new simplified method was rarely applied into the field practice to calculate the long-term settlement.

In this study, a typical reclamation project in Nansha, which is Binhai Garden reclamation project, was selected to conduct the long-term settlement analysis because serious settlement problems occurred in this reclamation project. The undisturbed soil specimens were taken from the site, and multi-staged incremental oedometer tests were conducted on fourteen undisturbed clayey soils to obtain the values of compressibility parameter of the clayey soil along the depth. Afterwards, triaxial hydraulic conductivity tests were conducted on nine soil samples to gain the values of hydraulic conductivity. Then, the new simplified method, proposed by Yin and Feng (2017), and finite element modelling using soft soil creep (SSC) model and soft soil (SS) model for clayey soil were used to analysis the long-term ground settlement. In the reclamation project, the settlements of different locations were monitored from *Jan.* 2011 and the measured data are regarded as the reference to evaluate the performances of the new simplified method and finite element modelling.

2. GEOLOGICAL AND GEOTECHNICAL CONDITIONS IN NANSHA

2.1 Description of Geological Condition

Nansha area is situated in the geometric center of the Pearl River Delta, south of Guangzhou city, nearby the Shizi Ocean. Binhai Garden reclamation project lies 1 km to the south of Nansha. The plan view of Nansha area is shown in Figure 1(a). Due to the long-term river alluvium and tides retreat, organic silty clays including the silt and silty soil were deposited with some irregular silty sands. The deposits were formed during the Holocene (Wei and Wu 2011). During the sedimentary of transgression-regression cycles, the soft clay in Nansha is interactive and terrestrial deposited. The thickness of soft clay layer varies from 10 m to 30 m. This soft soil is widely distributed in Nansha areas. Many subsidence-related geologic hazards have been encountered in this region due to the poor engineering properties of soft clay. The substratum of the clayey soil is usually silty sandy layer, regarded as a drained boundary.

2.2 Binhai Garden Reclamation Project

In the 2000s, Binhai Garden reclamation project was constructed nearby a reclamation island of Nansha, which initially was a paddy field. Three phases were subsequently reclaimed before the construction of buildings in Nansha, (named as Phase 1, Phase 2, Phase 3), as illustrated in Figure 1(b). According to geotechnical investigations, the deposited clayey soil is around 20 m~30 m in depth, and underneath the clayey soil, there is a sandy soil layer with a thickness of 2 m~10 m. Occasional sandy soil layer with 2m in thickness is encountered within the clayey soil in Phase 2 due to the complexity of deposition process. The site did not undergo any loading before the reclamation construction. As a result, the clayey soils are in the normally consolidated state, i.e. over-consolidation ratio (OCR) equals to 1.

Table 1 lists the construction information of three phases in Binhai Garden reclamation project. During the construction, it is regarded as a ramp loading in the construction period for three phases. The unit weight of filling materials is 17.5 kN/m^3 , and the surcharge loadings are obtained from the heights of the reclamations and listed in Table 1. After the reclamation, the building construction was directly carried out without any ground treatment. Foundations of the building were designed as concrete piles with high strength, which were directly seated on the bedrock.

2.2.1 Site Investigation of Binhai Garden Reclamation Project

In order to investigate the undrained shear strength, c_u , in the site, cone penetration tests (CPTs) were performed in locations nearby three stages of Binhai Garden reclamation project in Jan. 2014. As illustrated in Figure 1(b), locations of CPTs 4~7 are nearby Point 2 in Phase 1 of Binhai Garden reclamation project, the location of CPT 3 is beside Point 8 in Phase 2, and locations of CPTs 1&2 are close to Point 13 in Phase 3.

The relationships between depth and cone resistance, q_t , for three stages of Binhai Garden reclamation project are plotted in Figure 2. Generally, there was a large value on the top of 2 m thickness, which is regarded as the crusty layer. Below that, the value of q_t increases linearly with the soil depth, corresponding to the clayey soil layer in Nansha. In the depth of 20 m~25 m, a sudden increase of the q_t indicates the existence of the sandy soil layer in this range. According to the plots of Phase 1 and Phase 3, it could be regarded as idealized double layer soils. Comparatively, there is a silty sand layer within the clay layer for Phase 2, as shown in Figure 2. Mitchell and Brandon (1998) proposed an equation of the undrained shear strength, c_u , and the cone resistance, q_t , as below:

$$c_u = \frac{q_t - \sigma'_{zp}}{N_{kt}} \quad (1)$$

where σ'_{zp} is the total overburden pressure and it is taken as the in-situ stress, q_t denotes the cone resistance in the CPT, N_{kt} represents the cone factor, and it is taken as 17 in this study. (Lunne and Eide, 1976) The values of c_u are calculated from Eq. (1). It is found that the undrained shear strengths of soft soil layer increase linearly with the depth from 2.3 kPa to 10.2 kPa.

The groundwater level was monitored by using mini CTD-Diver (model D1501), which is a reliable instrument for measuring and recording the groundwater level. The range of mini-Driver is 10 meters, with an accuracy of 0.5 cm. The mini-Drivers were placed in 3 different locations: one monitoring point nearby Phase 1, one monitoring point in Phase 2, and another location in Phase 3, respectively, as shown in Figure 1(b). It is found that the groundwater levels at the site of Phase 1 are usually 2.50 m~3.53 m below the ground surface, which are related to the tide effects. Groundwater levels at the site including Phase 2 and Phase 3 are 2.10 m~2.95 m in depth, as depicted in Figure 3.

2.2.2 Basic properties of soils in Binhai Garden Reclamation Project

Fourteen undisturbed soil samples were taken from the site of three phases in the reclamation project with different depths, as plotted in Figure 2. The soil samples were taken in Jan. 2014 and the laboratory tests including basic property tests, oedometer tests, and triaxial hydraulic conductivity tests were conducted in Feb. 2014. It should be noted that the depth of eleven undisturbed soil samples taken from site are also illustrated in Figure 2, whereas other three disturbed soil samples were taken from the Farmland and shown in Figure 4. Figure 4 shows the basic properties of fourteen Nansha clays taken from the site of Binhai Garden

reclamation project. The organic content of clayey soil is in the range of 1.7%~4.2%, and the water content, w , mostly stays in the range of 40%~70%. The density, ρ_{soil} , varies from 1.65 g/cm³ to 1.79 g/cm³, and the specific gravity, G_s , is 2.67, which is consistent with the previous studies (Liu et al. 2010). The initial void ratio, e_o , is in the range of 1.2 to 2.0. Figure 5 presents the particle size distributions of eleven *Nansha* Clays in three phases of the reclamation project with different depths. Based on the unified soil classification system, the soil type belongs to silty clay.

2.2.3 Settlement Problems in Reclamation Project

As shown in Figure 1(b), different locations on the ground in the site were monitored with 15 settlement plates from Jan. 2011. The settlement plate was placed on the surface of the clayey soil layer. The monitoring points of Phase 1 are Points 1~5, the monitoring points for Phase 2 are Points 6~10. Points of 11~15 are in the areas of Phase 3. Thirteen locations of buildings, which are adhered to the wall, windows of the buildings, were also monitored from Jan. 2011. It was found that the settlements of the buildings were less than 1.7 mm per year. Comparatively, the monitoring ground settlements were in the range of 88 mm~220 mm per year.

The large differential settlement between the building and the ground leads to the pipeline ruptures and infrastructure damages in Phase 1 and Phase 3 as well as cracks of the building wall in Phase 2, as illustrated in Figure 6. The settlement problems are so serious that people's normal life is severely affected, and the prediction of the long-term differential settlements is extremely concerned for the repair. Currently, the ground settlement is monitored and collected every three months and the corresponding results will be presented in the following parts.

3 Experiments on Nansha Clay

The multi-staged incremental oedometer and triaxial hydraulic conductivity tests were performed in the soil laboratory to investigate soil properties including compressibility and hydraulic conductivity. There are some disturbances of the soil during the transportation and sampling. The silicone oil was spread in the thin-wall tube before the sampling. After the sampling, upward or downward movements of the piston are absolutely prevented. During the sampling, the central core part of the tube was selected to prepare the soil sample and the manually cutting was carefully performed to minimize the influences of the samples. Therefore, the sample disturbances during the transportation and specimen preparation can be neglected. Furthermore, results from previous results of the clayey soil, and silty sand soil were also utilized and compared with test results in this part.

For the multi-staged incremental oedometer test, the diameter of the specimen is 70 mm, and the thickness is 19 mm (Yin 1999; Cheng and Yin 2005; Feng et al. 2017). Top and bottom of the specimen were attached with filter paper. The silicone grease was utilized to minimize the possible friction (Sridharan and Gurtug 2004). The loading was applied with Wykeham Farrance conventional oedometers, following the procedures described in British Standard (1990). The loading sequence is listed as: (loading) 10 kPa→25 kPa→50 kPa→100 kPa→200 kPa→(unloading) 100 kPa→50 kPa→25 kPa→(reloading) 50 kPa→100 kPa→200 kPa→(loading) 400 kPa. In the loading stage, the loadings were maintained for 1 day while the loadings were kept for more than 1 hour during the unloading and reloading stages, as suggested by Yin and Tong (2011). The temperature was kept at $20 \pm 0.5^{\circ}\text{C}$ in the soil laboratory to minimize the temperature effect.

For the triaxial hydraulic conductivity test, the hydraulic consolidation cell was adopted with

the diameter of 70mm and the height of 140 mm. The effective confining pressure was controlled, and the void ratio can be calculated from the measured water volume during the consolidation stage. The detailed procedures followed BS 1377-6 (1990). The rate of flow Q was measured in the triaxial permeability test within 4 hours for each effective confining pressure. The vertical hydraulic conductivity (k_v in unit of m/s) can be calculated as:

$$k_v = \frac{1.63QL}{A\{(p_1 - p_2) - p_c\}} \times R_t \times 10^{-4} \quad (2)$$

where L is the height of soil specimen (in mm), A is the cross area of the soil sample (in mm^2), $(p_1 - p_2)$ is the difference of the bottom and top pressure in the test, p_c is the loss of the pressure in the triaxial cell, which is calibrated as 1 kPa, R_t is the correction factor of temperature on the viscosity of water, and it is taken as 1 when the temperature is 20 °C.

3.1 Compressibility of Nansha Clay

Taking one soil specimen in Phase 1 as an example, the relationships of vertical strain and time (in log scale) in loading, unloading, and reloading stages are shown in Figure 7. Figure 8 presents the curves of $e - \log(\sigma'_z)$ (void ratio versus log vertical effective stress), obtained from conventional oedometer test, for all undisturbed samples from different depths.

From the experimental results of samples at different depths in Phase 1 and Phase 2, it is found that initial void ratio, e_o , in the middle part of the clayey soil layer is larger than that at the top and the bottom. This is because of the top and bottom drainage boundary of consolidation and sedimentary process of clayey soils. However, since the period after completion of the filling is very short for Phase 3, the initial void ratio, e_o , in this Phase decreases with the depth, which is mainly related to the sedimentary effect. In addition, the

soil specimens taken from the farmland nearby Binhai Garden were also tested and their results are regarded as the reference of the initial stress-strain state at the site. Therefore, based on the results of three phases and farmland, it is deduced that the void ratio, e , in the middle of soil layer could be regarded as the initial void ratio.

It can be observed that there is nearly no apparent vertical pre-consolidation pressure, σ_{zp} , in these curves, which illustrates the tested soil is in the normally consolidated state. This feature is consistent with the geotechnical investigation about the farmland. This confirms that we can take the soil weight as the pre-consolidation pressure, σ_{zp} , to determine the over-burden loading history. In other words, the value of OCR is 1. Bjerrum (1954) discussed the apparent pre-consolidation pressure is related to the plasticity index and the leaching of the clayey soils. Lunne et al. (2006) emphasized the influences of sample disturbance and sampling equipment on the soil parameters including the pre-consolidation pressures. Therefore, more advanced geotechnical test on Nansha clay is worth being conducted in future. C_r represents the slope of unloading-reloading lines in the curves of $e - \log \sigma'_z$, and C_c denote the slope of linear portion of $e - \log \sigma'_z$ at normally consolidated state. The corresponding values for parameters C_r and C_c can be obtained and plotted in Figure 9. It is found that C_r ranges from 0.0414 to 0.0734, and C_c varies from 0.122 to 0.423. These values are very close to those of Hong Kong Marine Deposits (HKMD) (Zhu, Yin and Graham 2001; Yin and Feng 2017; Feng and Yin 2017; Feng et al. 2017).

3.2 Viscosity of Nansha Clay

It has been demonstrated, from the results of multi-staged incremental oedometer test, that the viscous behavior of Nansha clayey soil is also an important feature (Luo and Chen 2014). As

shown in Figure 7, the creep behavior is also observed, and creep coefficient, C_{ae} , is defined in this study to describe the viscous behavior:

$$C_{ae} = \frac{-\Delta e}{\Delta \log(t)} \quad (3)$$

where Δe is the change of void ratio, $\Delta \log(t)$ represents the change of time in log-scale. Figure 10 presents the evolution of creep coefficient versus σ'_z for three stages of Nansha clay with varying depths. It is found that values of creep coefficient, C_{ae} , increase linearly with the vertical loading within 100 kPa, while values of creep coefficient remain constant when the loading is larger than 200 kPa. This is because that the undisturbed soil specimens from the field are mostly below the depth of 10m, and the added fill loading is also gradually transferred to the soil skeleton structure in the process of sedimentary. Previous test results are also used to compare with the measured creep coefficient, and the data are in good consistency with the tested result in Phase 2 and Phase 3. In order to minimize the errors, the average value of creep coefficient is adopted: for Phase 1, the value of creep coefficient, is taken as 0.01414, and the creep coefficients for Phase 2 and Phase 3 are 0.00935 and 0.01047, respectively.

3.3 Hydraulic Conductivity of Nansha Clay

The hydraulic conductivity is an important parameter in many geotechnical settlement problems (Nagaraj and Miura 2001). Triaxial testing was conducted on nine soil specimens to obtain the hydraulic conductivity value of Nansha clay, and results of the calculated values from Eq. (2) with different void ratios are plotted in Figure 11.

Based on triaxial hydraulic conductivity tests, there is a nonlinear relationship between the permeability of Nansha clayey soils and void ratio (Tavenas, et al. 1983), expressed as:

$$k_v = k_{v0} \exp[(e - e_0)/C_k] \quad (4)$$

where k_{v0} is the initial hydraulic conductivity corresponding to the initial void ratio, e_0 , e is the void ratio and C_k is a constant to describe the relationship of hydraulic conductivity and void ratio. The initial void ratio, e_0 , is taken as the value in the middle depth of each phase, as mentioned above, and the corresponding hydraulic conductivity values are obtained as 3.004×10^{-9} m/s, 1.95×10^{-9} m/s, and 1.82×10^{-9} m/s for Phase 1, Phase 2, and Phase 3, respectively, as presented in Figure 11. Values of C_k are obtained as 0.4432, 0.935, 0.3788 for Phase 1, Phase 2, and Phase 3, respectively. It should be noted that only the values of vertical permeability were measured in the triaxial hydraulic conductivity tests because there is no horizontal drainage in the site of Binhai Garden. Generally, the horizontal permeability is 2 times of the vertical permeability (Zhu, Yin and Graham, 2001).

3.4 Parameters of Sandy Soil in Nansha

There are relatively few studies on the sandy soil in Nansha because settlement problems are mainly related to the clayey soil. Zhang and Huang (2012) conducted a series of laboratory tests of the silty sand in Guangzhou, and found that the permeability is in the range from 4.13×10^{-7} m/s to 3.38×10^{-6} m/s, and the Elastic Modulus, E' , ranges from 3.85 MPa to 9.83 MPa. Therefore, the permeability is taken as 1.73×10^{-1} m/day and Elastic Modulus, E' , is 4.0 MPa to describe the sandy soil layer.

4 FINITE ELEMENT MODELLING AND NEW SIMPLIFIED METHOD PREDICTION FOR THE RECLAMATION PROJECT

4.1 Finite Element Modelling Description

The Soft Soil Creep (SSC) model and Soft Soil (SS) model in Plaxis (2015 version) were

adopted to simulate the clayey soils in the Binhai Garden reclamation project. SSC model is widely used to predict the consolidation settlement including creep (Vermeer and Neher, 1999; Nash and Brown, 2013), whereas SS model is used to simulate the consolidation settlement of clayey soils but neglecting the creep behavior. The modified compression index, λ^* , is determined from the compression curve of $\varepsilon - \ln p'$ when the soil is in the normal consolidated state; The modified swelling index, κ^* , is obtained from the curve of $\varepsilon - \ln p'$ when the soil is at the unloading-reloading stage; The modified creep index, μ^* , is determined from the curve of $\varepsilon - \ln t$ after the dissipation of excess porewater pressure ($t_o=1$ day by default). Details could be referred to Plaxis Manual (Brinkgreve, Kumarswamy, Swolfs, 2015). Recently, Zou et al. (2018) presented a novel elasto-viscoplastic formulation based on the Nishihara model to account the time-dependent settlement and stress relaxation of clayey soils. Zhu et al. (2019) described the relationship of inter-particle bonding/debonding and the creep degradation. The sandy soil in the profile of three phases was simulated by the Mohr-Coulomb (MC) model. Fill construction and consolidation settlement of the reclamation project have been simulated separately. The two-dimensional plane strain with 15-node triangular elements was selected in the finite element modelling. The groundwater variation is also simulated based on the monitored data, as shown in Figure 3. In the FE modelling, the implicit algorithm with Newton-Raphson's method was utilized to solve the partial differential equations for the couple of excess porewater pressure dissipation and creep deformation.

Figure 12 shows the profile of simulation models for three phases of the Binhai Garden reclamation project, and the profile is mainly based on the results from CPTs. It can be observed that Phase 1 and Phase 3 are typical double layered soil profile, and Phase 2 is a multi-layered soil profile. The initial pre-consolidation state of the clayey soil was considered

by setting $OCR = 1$, the horizontal deformation was fixed to be zero for the lateral boundaries, and both vertical and horizontal deformations were fixed to be zero at the bottom boundary. The soil layers were set to be drained condition both at the surface and at the bottom. The global mesh was pre-set to be fine to make sure the mesh was dense enough for obtaining accurate results. The parameter values of SSC model for the soil layer of each phase are listed in Table 2. For the SS model, all the parameter values are in consistent with those in Table 2 but no creep parameter. It should be noted that the crusty layer was not considered in the finite element simulation since the actual site before the fill construction was used as the farmland. The construction period was considered in the finite element simulations by setting the stage of adding fill as the consolidation type, where the loading is assumed to increase linearly within the construction period.

4.2 The New Simplified Method for Multi-layer Soils with Creep

Degago et al. (2011) reviewed the difference of Hypothesis A and Hypothesis B and summarized the experimental data from previous literature to access whether the creep occurs during the consolidation phase, and they proved that the measured time-dependent deformation of clays generally exhibits a good agreement with Hypothesis B. Yin and Feng (2017) proposed and validated a new simplified method to calculate the consolidation settlement of clayey soils with creep subjected to an instant loading. Feng and Yin (2017) extended the new simplified Hypothesis B method for calculating the consolidation settlement of multi-layer soils with creep, expressed as:

$$S_{totalB} = \sum_{i=1}^n S_{primary}^i + \sum_{i=1}^n S_{creep}^i = U_a \sum_{i=1}^n S_{fi} + \sum_{i=1}^n [\alpha S_{creep,fi} + (1-\alpha) S_{secondary}^i] \quad (5)$$

$$= U_a \sum_{i=1}^n \varepsilon_{fi} H_i + \sum_{i=1}^n \{ [\alpha \varepsilon_{creep,fi} + (1-\alpha) \varepsilon_{secondary}^i] H_i \} \quad \text{for } t \geq 1 \text{ day}$$

where $\sum_{i=1}^n S_{primary}^i$ is the total “primary consolidation” settlement of multi-layer soils, $\sum_{i=1}^n S_{creep}^i$

366 is the creep settlement of multi-layer soils, $\sum_{i=1}^n S_{f,i}$ is the summation of final “primary
 367 consolidation” settlement of multi-layer soils, $\sum_{i=1}^n S_{creep,f,i}$ is the total creep settlement of
 368 multi-layer soils calculated from the instant loading path, $\sum_{i=1}^n S_{\text{secondary"},i}$ is the total creep
 369 settlement of multi-layer soils calculated from the delayed loading path, as illustrated in
 370 Figure 13; The instant loading path represents path of soil elements close to the drainage
 371 boundary, whose stress-strain state occurs instantly, the influence of excess porewater
 372 pressure could be neglected, whereas the delayed loading path refers to the path of soil
 373 elements far away from the drainage boundary, whose stress-strain state is delayed due to the
 374 dissipation of excess porewater pressure; U_a is the average degree of consolidation for
 375 double soil layers. α is a parameter to describe the de-coupling phenomenon of
 376 consolidation and creep, which is simplified as a constant by Yin and Feng (2017). In this
 377 study, it is suggested to take $\alpha = U_a$ for the thick soil layer because the de-coupling
 378 phenomenon of consolidation and creep is affected by the average degree of consolidation of
 379 the soil layer. For multi-layer soils, each stratum is influenced by the action of the other in the
 380 interface (Zhu and Yin 1999, 2005). The average degree of consolidation, U_a , for
 381 double-layered soils is determined

$$382 \quad U_a(T, T_c) = \begin{cases} \frac{T_c}{T} - \sum_{n=1}^{\infty} \frac{c_n}{\lambda_n^4 T_c} [1 - \exp(-\lambda_n^2 T)] & T \leq T_c \\ 1 - \sum_{n=1}^{\infty} \frac{c_n}{\lambda_n^4 T_c} [1 - \exp(-\lambda_n^2 T_c)] \times \exp[-\lambda_n^2 (T - T_c)] & T > T_c \end{cases} \quad (6)$$

383 where T and T_c are the normalized time factor and normalized construction time factor,
 384 respectively, $T = \frac{c_{v1} c_{v2} t}{(H_1 \sqrt{c_{v2}} + H_2 \sqrt{c_{v1}})^2}$, $T_c = \frac{c_{v1} c_{v2} t_c}{(H_1 \sqrt{c_{v2}} + H_2 \sqrt{c_{v1}})^2}$, λ_n is the root of the equation
 385 $\sin \theta + p \sin(q\theta) = 0$ for both top and bottom drained condition (*condition1*) and equation

386 $\cos\theta - p\cos(q\theta) = 0$ for one side drained condition (*condition2*). Values of c_n are determined
 387 by:

$$388 \quad c_n = \begin{cases} \frac{2[m_{v1}H_1\xi\sin(\lambda_n\xi) + m_{v2}H_2\omega\sin(\lambda_n\omega)]^2}{\omega^2\xi^2(m_{v1}H_1 + m_{v2}H_2)[m_{v1}H_1\xi\sin^2(\lambda_n\xi) + m_{v2}H_2\omega\sin^2(\lambda_n\omega)]} & \text{for condition1} \\ \frac{2[m_{v1}H_1\xi\cos(\lambda_n\xi)]^2}{\omega^2(m_{v1}H_1 + m_{v2}H_2)[m_{v1}H_1\xi\cos^2(\lambda_n\xi) + m_{v2}H_2\omega\sin^2(\lambda_n\omega)]} & \text{for condition2} \end{cases} \quad (7)$$

389 where

$$390 \quad \begin{aligned} p &= \frac{\sqrt{k_2m_{v2}} - \sqrt{k_1m_{v1}}}{\sqrt{k_2m_{v2}} + \sqrt{k_1m_{v1}}} \\ q &= \frac{H_1\sqrt{c_{v2}} - H_2\sqrt{c_{v1}}}{H_1\sqrt{c_{v2}} + H_2\sqrt{c_{v1}}} \\ \omega &= \frac{(1+q)}{2} \\ \xi &= \frac{(1-q)}{2} \end{aligned}$$

391 Details can be found in Zhu and Yin (1999, 2005). This method is utilized in this part to
 392 analysis the settlement of the reclamation project. The values of parameters used in the new
 393 simplified method are listed in Table 3. In this reclamation project, the top and bottom of the
 394 double soil layers are drained boundary, and *condition 1* is used for Binhai Garden project.
 395 Fill construction periods of Binhai Garden project last one year for *Phase 1*, two years for
 396 *Phase 2* and four years for *Phase 3*, respectively. In order to compare with the FE simulations,
 397 three different construction periods are calculated and compared for each phase in Binhai
 398 Garden.

399

400 In the calculation, double soil layers are divided into sub-layers with 0.5 m thick to determine
 401 the initial effective stress state and final effective stress state. For the j -th sub-clay layer, the
 402 final strain in the consolidation can be calculated as:

$$403 \quad \varepsilon_{fj} = \frac{C_e}{(1+e_o)} \log\left(\frac{\sigma'_{zp,j}}{\sigma'_{zo,j}}\right) + \frac{C_c}{(1+e_o)} \log\left(\frac{\sigma'_{zf,j}}{\sigma'_{zp,j}}\right) \quad (8)$$

404 where $\sigma'_{zo,j}$ is the initial effective stress, considering OCR is 1 in this project, the initial

effective stress state is at point 3 $(\sigma'_{zp}, \varepsilon'_{zp})$, $\sigma'_{zf,j}$ is the final effective stress, and final effective stress state after the stress increment is at point 4 $(\sigma'_{z4}, \varepsilon'_{z4})$ in Figure 13. For the sand layer, the “primary” consolidation settlement is directly calculated from: $S_f = \frac{\Delta\sigma'_z}{E'} H$, where H is the thickness of sand layer. Values of S_f , m_v and c_v for Nansha clay and sand soil layers, and factors for double soil layers, p and q , are calculated and listed in Table 4. The average degree of consolidation, U_a , can be determined with the help of interpolation method by using the solution charts for different loading periods. In this part, the calculation equation is written in a MATLAB program to calculate the average degree of consolidation. The creep compression is calculated by using Eq. (8) since the final effective stress state is in a normal consolidation state

$$\begin{aligned} \varepsilon_{creep,f} &= \frac{C_{ae}}{1+e_o} \log\left(\frac{t_0+t_e}{t_0}\right) \\ t_e &= t - t_c/2 - t_o \\ \varepsilon_{secondary} &= \frac{C_{ae}}{1+e_o} \log\left(\frac{t}{t_{EOP}}\right) \text{ for } t \geq t_{EOP} \end{aligned} \quad (9)$$

Values of t_{EOP} are also determined to be the time when the $U_a = 98\%$ for double soil layers.

5. INTERPRETATION OF THE MONITORING SETTLEMENT AND PREDICTION OF THE FINITE ELEMENT SIMULATION AND NEW SIMPLIFIED METHOD

The mechanism of the settlement problems at the site is analyzed. The finite element modellings using the SSC model and SS model and new simplified method calculation were compared with the measured settlements for three phases of Binhai Garden reclamation project. The comparison of finite element modellings with the SSC model and the SS model could directly illustrate the viscosity effect (Yin et al. 2011). Since there is no initial data recorded before the construction of three phases of Binhai Garden reclamation project, the settlement after Jan. 2011 was specially investigated in this study. The settlements in all the

finite element simulations are reset with an offset in *Jan.* 2011, then, the numerical simulation results are compared with the measured ground settlements after *Jan.* 2011.

5.1 Settlement Problem Analysis in Binhai Garden Reclamation Project

Figure 14 illustrates the differential settlement mechanism of the reclamation project. After the construction of the fill and buildings, large consolidation settlement occurred in the clayey soil layer due to the surcharge of added fills, while ignorable settlement was observed in the buildings because the foundations of building are concrete piles with high strength ($E_c=32$ GPa), which were directly inserted into rock bed. The permeability of concrete piles in the geotechnical design report is 2×10^{-12} m/s, which is too small to be neglected. The difference between the ground settlement and building settlement leads to serious settlement problems, as described in three phases of the reclamation project (see Figure 6). Therefore, the ground settlement due to the surcharge loading is regarded as the major factor that induces the settlement problems in the reclamation project.

5.2 Comparison of the Measured and Predicted Settlements for Binhai Garden Reclamation Project

5.2.1 For Phase 1 of Binhai Garden Reclamation Project

According to the geotechnical investigations, fill construction of Phase 1 was started from *Jan.* 2000 and completed in *Jan.* 2001, and the settlement was monitored from *Jan.* 2011. The monitored data, the results from finite element simulations and the calculation results from the new simplified method are plotted in Figure 15.

It is found that the settlement of Phase 1 is close to the end of consolidation after ten years' consolidation and the creep settlement would contribute the main settlement in future.

Comparing the monitored settlement data of Point 2, which is nearby the location of the calculated soil layers, with the finite element simulations, it can be observed that monitored data agree well with the results from finite element modelling with the SSC model as well as the results from the new simplified method. From the curves of FE simulation and new simplified method, it is estimated that the ground settlement of Phase 1 will be within 0.07 m in the coming ten years (from 2019 *Jan.*). Comparatively, the finite element modelling with the SS model gradually underestimates the settlement from *Jan.* 2011.

The viscosity of marine clay has a significant influence on the dissipation of excess pore water pressure (Yin et al. 2011). Figure 16 presents the dissipation of excess pore water pressure along depth in the finite element modelling. It is found that there is an obvious difference of the excess pore water pressures in the finite element analysis with the SSC model and the SS model in *Jan.* 2002, which is related to the viscosity effect. *In the undrained condition, the creep would induce an increase of excess pore water pressure, which could therefore reduce the effective stress on the soil skeleton when the total earth pressure is constant (Yin et al. 1994; Yin and Zhu 1999).* In the fully coupled finite element modelling, the difference of excess pore water pressure between the analysis with the SSC model and the SS model decreases from *Jan.* 2011 to *Jan.* 2021, which indicates that the compensated elastic deformation is a gradually displaying process in the subsequent time of Phase 1. Therefore, the creep behaviour has a large influence on the total settlement in the prediction.

5.2.2 Phase 2 of Binhai Garden Reclamation Project

Soil layer of Phase 2 in Binhai Garden reclamation project is a multi-layered condition. The actual fill construction lasted for 2 years (from 2004 to 2006). The beginning of 2004 is selected as the initial time for Phase 2 and Figure 17 shows the curves of settlement-log(time)

from finite element simulations and new simplified method calculation for soil profile of Phase 2. The monitored settlement data of Point 8 (it is close to the soil profile, as shown in Figure 1(b)) are also plotted in Figure 17.

According to the finite element simulations, the ground settlement after January 2011 is still in the consolidation stage. Figure 17(b) compares the finite element simulations and new simplified method predictions with the monitoring settlement of Point 8 (2011 *Jan.* to 2018 *Dec.*). The settlements nearby Point 8 in Phase 2 in coming ten years can reach 0.14 m, and it is observed that the settlement of Phase 2 is still in the consolidation stage, as illustrated in Figure 17(a). Thus, it is necessary to adopt the ground improvement techniques to avoid more serious settlement problems.

The response of excess pore water pressure after the construction, as shown in Figure 18, demonstrates that it is in the main consolidation stage from January 2011 to January 2021. The creep compression induces more than 9 kPa in excess pore water pressure (the difference of pore water pressure between simulations with SSC model and those with SS model) in *Jan.* 2011. After ten years, the largest difference is within 6 kPa. As a result, the compensated elastic deformation in the fully coupled analysis is also displayed in the consolidation stage of Phase 2.

5.2.3 Phase 3 of Binhai Garden Reclamation Project

Figure 19 displays the simulated results from finite element modelling and new simplified method calculation as well as the monitored settlements of Point 13 in Phase 3. In the field, the fill construction of Phase 3 began in *Jan.* 2005, and finished in *Jan.* 2009. In other words, the construction period is 4 years.

502

503 Because the time of monitoring settlement is very close to the completion of fill construction,
504 the ground settlement from *Jan. 2011* to *Jan. 2021* is in the early stage of consolidation for
505 clayey soil layers. The predicted settlement in coming ten years will be as large as 0.36 m. To
506 solve the settlement problems, repair works are inevitable to be conducted. As a comparison,
507 the predicted ground settlement with the SS model is very close to that with the SSC model
508 (as illustrated in Figure 19b) because it is mainly in the consolidation stage for Phase 3.

509

510 Again, the response of excess pore water pressure is analyzed in the finite element modelling
511 along depth in Figure 20. It is seen that the difference of excess pore water pressures between
512 the finite element modelling with the SSC model and that with the SS model increases from
513 *Jan. 2011* to *Jan. 2021*, which is reduced by the viscous compression in the fully coupled
514 analysis. This also demonstrates that the importance of measuring the excess pore water
515 pressure at the site because there is only a small settlement difference between simulated
516 results with the SSC model and those with the SS model. It should be noted that all the
517 compensated excess pore water pressure by viscous behavior will gradually dissipate in the
518 subsequent consolidation. In other words, the viscosity effect has the small influence on the
519 excess pore water pressure in the early stage of consolidation. The influence on excess pore
520 water pressure gradually vanishes with time but displays on the settlement (*e.g.* Phase 1).

521

522 Based on the analysis and discussion of the results in the above sections, it is demonstrated
523 that the creep behavior of clayey soils in Nansha has the unneglectable influences on the
524 settlement as well as dissipation of excess porewater pressure. The new simplified method,
525 which considers the creep settlement during and after the consolidation, could provide the
526 close predictions of settlement compared with the monitored data in the sites. However, there

are some limitations in this study: (1) the horizontal drainage conditions are not fully discussed in this reclamation project because prefabricated vertical drains (PVDs) were not utilized in the soft ground improvement; (2) the nonlinear compressibility of the soft soil was not considered whereas it commonly exists in the field conditions (Zhu and Yin, 2012; Feng and Yin, 2019); (3) the influences of creep on the dissipation of excess porewater pressure are not fully considered and discussed.

6. CONCLUSIONS

In this study, the settlement problems in three phases of a reclamation project in Nansha are analyzed. The cone penetration tests (CPTs) were conducted nearby the sit of the reclamation project. Fourteen undisturbed soil specimens were taken from different depths in the field at the same site and tested in soil laboratory. Then, the finite element (FE) analysis and new simplified method were performed and results are compared with the measured settlements for three phases of the reclamation project. The FE modelling and new simplified method calculated results are used to predict the future ground settlements of the reclamation. Based on the results and discussions, findings and conclusions are presented as follows:

- (a) The serious settlement problems were observed in Binhai Garden reclamation project, and the settlement problems are directly related to the consolidation settlements of the soft soil layer in site.
- (b) Based on the results of multi-staged incremental oedometer test, the compressibility of clayey soils in Nansha is highly nonlinear and the viscous behaviour is nonnegligible.
- (c) The results of triaxial hydraulic conductivity tests illustrated that the hydraulic conductivity of the undisturbed clayey soils decreases nonlinearly with the void ratio.
- (d) The monitored ground settlements in the field are compared with the results from finite element modelling and calculated results from new simplified method. It is found that the

552 results from the finite element analysis using the soft soil creep (SSC) model agree well
553 with the monitored settlement data for three phases in the reclamation project.

554 (e) By comparing with the finite element simulations with the SSC model and simulations
555 with the soft soil model, it is found that the soil viscosity has an obvious influence on the
556 excess pore water pressure dissipation but little effect on settlements in the early
557 consolidation stage (Phase 3 of the reclamation project), and the soil viscosity gradually
558 transfers the creep influence from the excess pore water pressure to the settlement during
559 the subsequent consolidation (Phase 1 and Phase 2 of the reclamation project).

560

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