Improvement of soft clays using vacuum-assisted consolidation method

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Improvement of Soft Clays Using Vacuum-assisted Consolidation Method

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Abstract

In this study, the analytical modeling of one-dimensional consolidation by vertical drains with vacuum preloading considering both vertical and horizontal drainage is presented. In this method, the total degree of consolidation based on excess pore water pressure dissipation is related to the time factor (T_h), drain configuration and anisotropic soil permeability. The analytical predictions are compared with the observed data from consolidation testing. Three different test series were conducted in a large-scale consolidation apparatus designed and installed at the University of Wollongong. It is shown that the authors' analytical model can accurately predict the laboratory behavior. Finally, the analysis of a selected case history employing the authors' model demonstrates the applicability of this approach for typical field conditions.

Introduction

Vacuum preloading method was first introduced by Kjellman (1952) to improve the strength of soft soil and to reduce the surcharge height. An increase in the effective stress in the soil mass is readily gained by the application of vacuum pressure (Qian et al. 1992), which can result in rapid consolidation while eliminating the need for a high surcharge fill. Recently, a system of vertical drains enhanced by vacuum preloading has been successfully applied in land reclamation projects and soft clay embankments (Indraratna et al. 2004, Chu et al. 2000). Mohamedelhassan and Shang (2002) discussed the application of vacuum pressure and its benefits, but without any prefabricated vertical drains (PVDs). An analytical solution for radial consolidation incorporating vacuum effect was introduced by Indraratna et al. (2005). In this paper, the authors present mathematical solutions to vacuum-assisted consolidation with both vertical and horizontal drainage including the smear effect and well resistance attributed to prefabricated vertical drains. The analytical predictions are compared with large-scale experimental results. The analysis of a selected case history employing the authors' solution is also presented.

Theoretical Formulations for Vertical and Radial Consolidation Incorporating Vacuum Preloading

In this section, the analytical solution of unit cell for vacuum consolidation considering both vertical and horizontal drainage is presented. Figure 1a illustrates the drainage condition for horizontal drainage. The average excess pore pressure ratio ($\bar{u}_{v,t}/u_0$) at a given time t proposed by Mohamedelhassan and Shang (2002) is:

$$\frac{\bar{u}_{v,t}}{u_0} = -\frac{p_0}{u_0} + \left(1+\frac{p_0}{u_0}\right)\sum_{m=1}^{\infty}\frac{8}{(2m+1)^2\pi^2}\exp\left(-\left(\frac{2m+1}{2}\right)^2\pi^2\frac{c_v t}{l^2}\right) \quad (1)$$

where, $\bar{u}_{v,t}$ = average excess pore pressure, u_0 = initial excess pore pressure, p_0 = applied vacuum pressure, m = integer, c_v = coefficient of consolidation for vertical drainage, l = soil thickness and t = time.

For radial consolidation analysis, the assumptions, the boundary conditions, and the initial conditions are similar to the solution proposed earlier by Indraratna et al. (2005) as shown in Fig. 1b. The average excess pore pressure ratio due to radial consolidation ($\bar{u}_{h,t}/u_0$) at a given time t is:

$$\frac{\bar{u}_{h,t}}{u_0} = \left(1+\frac{p_0}{u_0}\right)\exp\left(\frac{-8c_h t}{\mu d_e^2}\right) - \frac{p_0}{u_0} \quad (2a)$$

In the above equation,

$$\mu \approx \left[\ln\left(\frac{n}{s}\right) + \frac{k_h}{k_s}\ln(s) - \frac{3}{4} + \pi \frac{2k_h}{3q_w} l^2 \right] \quad (2b)$$

$$n = d_e / d_w \quad (2c)$$

$$s = d_s / d_w \quad (2d)$$

where, d_s = the diameter of the smear zone, d_e = the diameter of soil cylinder dewatered by a drain, d_w = the equivalent diameter of the drain, k_s = horizontal soil permeability in the smear zone and q_w = drain discharge capacity. For the above equation, the vacuum pressure ratio (VPR) for vacuum combined surcharge preloading can be introduced by the value of p_0/u_0 (i.e. applied vacuum pressure/initial excess pore pressure). For fully saturated clay, the value of u_0 is equal to the preloading pressure.

Considering that the water is drained in both horizontal and vertical directions (Fig. 1c), the overall average excess pore pressure $\overline{u_t}$ is calculated based on Carrillo's approach (1942), which can be expressed by:

$$\frac{\overline{u}_t - \overline{u}_{t=\infty}}{\overline{u}_{t=0} - \overline{u}_{t=\infty}} = \left(\frac{\overline{u}_{h,t} - \overline{u}_{h,t=\infty}}{\overline{u}_{h,t=0} - \overline{u}_{h,t=\infty}}\right)\left(\frac{\overline{u}_{v,t} - \overline{u}_{v,t=\infty}}{\overline{u}_{v,t=0} - \overline{u}_{v,t=\infty}}\right) \quad (3)$$

Substituting Equations (1) and (2) into Equation (3), the overall average excess pore pressure ratio can be expressed by:

(a) Preloading combined with vacuum application:

$$\frac{\overline{u_t}}{u_0} = -\frac{p_0}{u_0} + \left(1 + \frac{p_0}{u_0}\right)\sum_{m=1}^{\infty}\frac{8}{(2m+1)^2\pi^2}\exp\left(-\left[\left(\frac{2m+1}{2}\right)^2\pi^2\frac{1}{c_{vh}L^2} + \frac{8}{\mu}\right]T_h\right) \quad (4a)$$

(b) Vacuum application only

$$\overline{u_t} = -p_0 + p_0\sum_{m=1}^{\infty}\frac{8}{(2m+1)^2\pi^2}\exp\left(-\left[\left(\frac{2m+1}{2}\right)^2\pi^2\frac{1}{c_{vh}L^2} + \frac{8}{\mu}\right]T_h\right) \quad (4b)$$

where, the relevant dimensionless parameters are given by:

$$c_{vh} = c_h/c_v = k_h/k_v \quad (4c)$$

$$L = l/d_e \quad (4d)$$

$$T_h = c_h t / d_e^2 \quad (4e)$$

The overall average degree of consolidation with time (U_t) can now be evaluated conveniently by:

$$U_t = \left(1 - \frac{u_0 - \overline{u_t}}{u_0 - \overline{\overline{u_\infty}}}\right) \quad (5)$$

where, u_0 and $\overline{u_\infty}$ are the average excess pore pressure when t = 0 and $t \to \infty$, respectively.

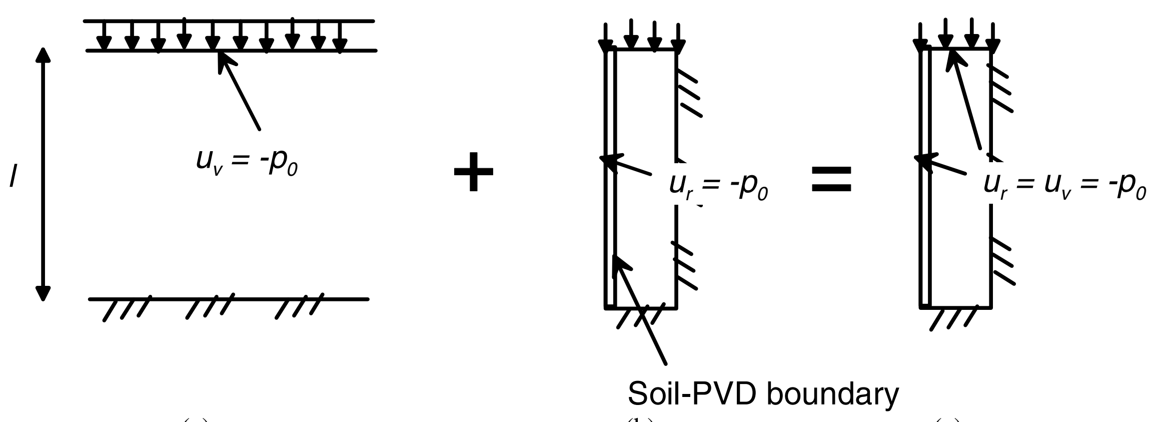


Figure 1. Drainage conditions (a) horizontal drainage, (b) vertical drainage and (c) vertical and horizontal drainage

Substituting Equations (1) and (4) into Equation (5) gives:

$$U_t = 1 - \sum_{m=1}^{\infty} \frac{8}{(2m+1)^2 \pi^2} \exp\left(-\left[\left(\frac{2m+1}{2} \right)^2 \pi^2 \frac{1}{c_{vh} L^2} + \frac{8}{\mu} \right] T_h \right) \tag{6}$$

Equation (6) shows that the total degree of consolidation based on excess pore pressure dissipation at any vacuum condition (p_0) is uniquely related to the time factor (T_h), vertical drain system configuration and anisotropic permeability of the soil (μ , L and c_{vh}).

Laboratory Setup and Discussions

Laboratory Setup and Procedures

The schematic diagram of the large-scale radial drainage consolidation cell is shown in Fig. 2. Its main body consists of two half sections made of stainless steel (450 mm in internal diameter and 950 mm in height). In order to eliminate the friction along the cell boundary, a 1.5 mm thick ultra-smooth Teflon sheet was inserted around the periphery. As a result, the height and diameter (h/d) ratio of the cell can be much higher than a conventional oedometer, enabling the appropriate testing of a mandrel-driven, prefabricated vertical drain (PVD). The loading system is equipped with an air jack compressor system via a rigid piston. Water and air tightness of piston is achieved using an O-ring system on its periphery, which is lubricated with grease to reduce friction. In the loading calibration, over 97% loading was transferred to soil sample. A displacement transducer connected to a data logger is located at the top of the piston. A vacuum pump capable of applying a suction upto -100 kPa is used above the PVD. The cell is also equipped with a specially designed steel hoist and guider from which a PVD can be inserted vertically along the central axis of the cell.

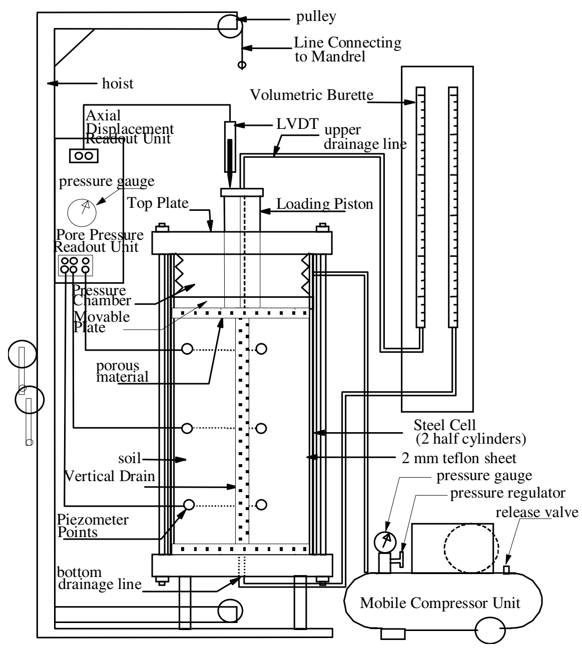


Figure 2. Large-scale consolidation apparatus (after Indraratna and Redana, 1998)

Reconstituted alluvial clay from Moruya (New South Wales) was used in the consolidation cell. The clay size particles (<2 μm) accounted for about 40% - 50% of the specimen, and particles smaller than silt size (<6 mm) constituted about 60% of the specimen. Selected geotechnical properties of sample are shown in Table 1.

Three different series of tests were conducted in the large-scale consolidation chamber built at University of Wollongong. The first series of tests (Test SP) followed the conventional method where a typical surcharge load (30 kPa) was applied with no vacuum pressure. For the second series (Test VP), only a vacuum pressure of 40 kPa was applied without any surcharge load. For the third series (Test SV), a vacuum pressure of 40 kPa together with a 30 kPa surcharge load was applied to further accelerate consolidation. The main stages of the test procedure included: (1) reconstituted clay preparation, (2) drain installation and (3) measurement of settlements and excess pore pressures. In the first stage, following the recommendation of Burland (1990), the clay specimen was mixed thoroughly by a mechanical mixer with a water content slightly greater than the liquid limit. The surface settlement was measured by a linear voltage differential transformer placed at the top of the piston. A total of 6 calibrated diaphragm-type piezometers were installed at 3 different depths (i.e., top, middle and bottom of the cell) to measure the pore-water pressures at various points. The fully saturated piezometer tips were kept in position using thin rigid stainless steel tubes. The LVDT and pore pressure transducers were connected to a common data logger.

Table 1. Soil properties of reconstituted clay sample

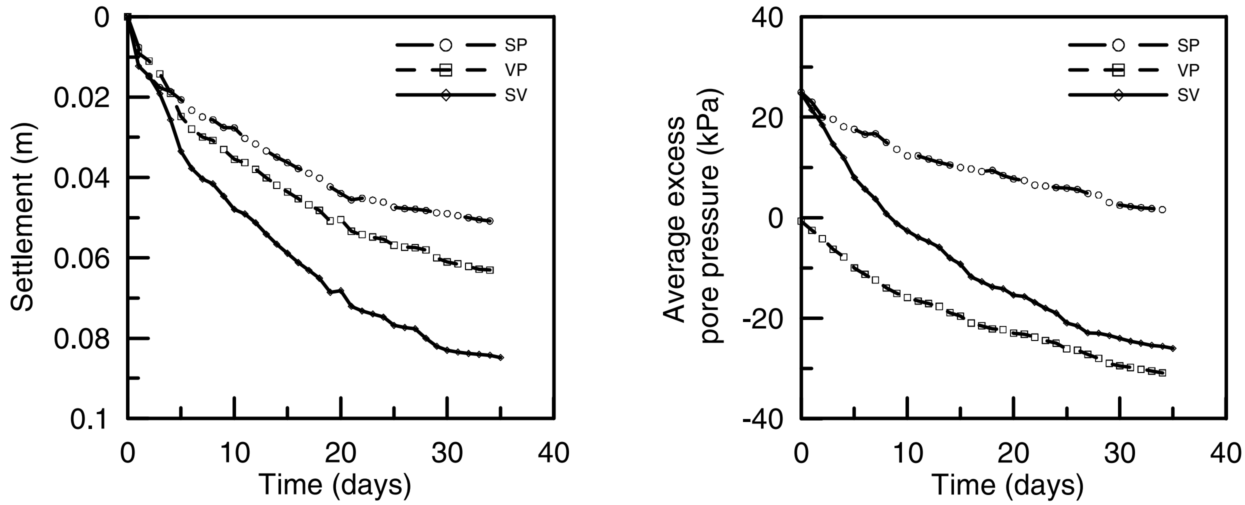
Soil properties	Reconstituted clay
Water content (%)	45
Plastic limit, w_p (%)	17
Initial void ratio (e_0)	1.14
Plasticity Index, PI (%)	28
Unit weight, γ_t, (kN/m3)	17

Within the large-scale consolidometer cell, the clay was placed and compacted in layers in the apparatus. An initial preconsolidation pressure of 20 kPa was applied for 7 days before the installation of the vertical drain (i.e. p_c' = 20kPa). In the second stage, a 100 mm × 3 mm band drain, which equals to 32.5 mm of equivalent drain radius (r_w), was then installed vertically using a steel mandrel. After the drain installation, the mandrel was withdrawn by the hoist system, and subsequently, an initial precompression stress of 20 kPa was immediately applied for all tests (i.e. p_c'=20kPa). For Test SP, to simulate surcharge consolidation, the large clay sample was further subjected to additional surcharge loading in increments of 30 kPa (u_0). In the case of Test VP (vacuum preloading), the large clay sample was loaded with an application of vacuum pressure of 40 kPa only (p_0). In the case of Test SV (surcharge and vacuum preloading), the large clay sample was loaded axially in increments of 30 kPa (u_0), together with a vacuum pressure of 40 kPa (p_0). The duration of each test was approximately 35 days. The corresponding excess pore pressure and settlement behaviors were recorded and plotted.

Laboratory Results and Discussions

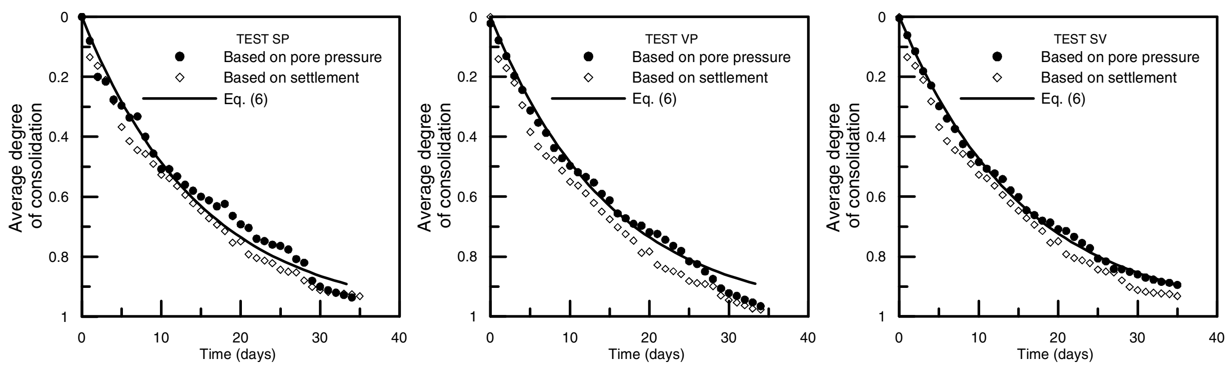
In this section, the consolidation behaviour of soft clay in the large-scale consolidometer under combined vacuum and surcharge preloading was analysed and compared to the proposed analytical solution presented in the previous section. Figures 3a and 3b illustrate the measured average excess pore pressures and surface settlements. The excess pore pressures for each location were determined by deducting the hydrostatic pore water pressure from the total pore pressure at the measurement location. It is clearly shown that the combination of vacuum and surcharge pressure maximises the settlement, and this settlement depends on the magnitude of the applied surcharge and vacuum. In Figure 3b, positive excess pore pressure is observed when the surcharge pressure is employed (i.e., Tests SP and SV). As expected, the measured excess pore pressures become negative for the tests with the vacuum application (Tests VP and SV). The measured ultimate negative average excess pore pressure is approximately 80% of the applied vacuum pressure due to vacuum loss along the drain length. This is in accordance with previous laboratory observations made by Indraratna et al. (2004).

Figure 4 shows the predicted and measured values of the degree of consolidation. The measured degree of consolidation was calculated based on the settlement and the average excess pore pressure. It can be seen that the degree of consolidation based on strain was slightly greater than that based on the excess pore pressure. The proposed Equation (6) was employed to predict theconsolidation behavior based on excess pore pressure dissipation. Since the smear effect is more significant than the well resistance for relatively short drains, it has been assumed on the basis of previous data (Indraratna and Redana, 1998) that the smear zone is approximately three times the equivalent drain diameter, k_h/k_s = 1.5 and the well resistance is neglected. Based on the standard oedometer tests conducted by the authors, c_h of 2.8×10^{-3} m^2/day was used in the analysis. The predictions (Equation 6) agree well with the measured results based on excess pore pressure, while they slightly underestimate the degree of consolidation based on strain. This is because, the prediction of the degree of consolidation (Equation 6) is derived from excess pore pressure dissipation with the assumption of constant soil compressibility.



(a) (b)

Figure 3. Comparison between measured results obtained from large-scale consolidation tests (a) Settlements and (b) Excess pore pressures



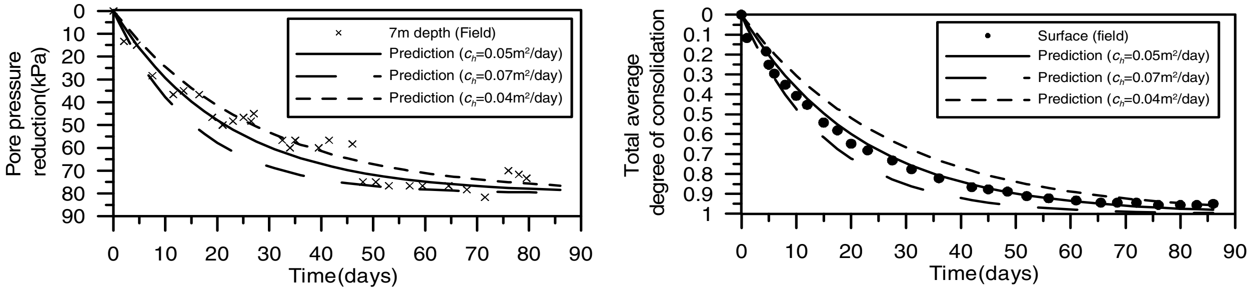
(a) (b) (c)

Figure 4. Comparison between predicted and measured results obtained from large-scale consolidation tests (a) Test SP, (b) Test VP and (c) Test SV

Application to Case History

The Location of Port of Huanghua is about 90 km to the east of Cangchou in Hebei Province, China (Gao, 2004). The soil consists of a very soft hydraulic fill (taken from the harbour basin) 4m in thickness, underlain by more than 16m of soft, thick clayey soil. Based on an in-situ investigation in a pilot area, the consolidation characteristics of the soil can be summarised as follows: the undrained shear strength varies from 1.6 to 6.0 kPa, and the horizontal coefficient of consolidation (c_h) ranges from 3×10^{-2} to 7×10^{-2} m^2/day. It can be observed that the values of c_h are very high at a depth of 4.30-7.60m and 9.40-12.10m ($c_h \approx 7\times10^{-2}\,\mathrm{m}^2/\mathrm{day}$). Preloading using surcharge fill alone to obtain the expected settlement cannot be applied in this area because the top soil layer has a very low undrained shear strength. Therefore, vacuum preloading was considered to be the most appropriate method. PVDs 18.5m long were installed in a square pattern at a spacing of 1.5m. Subsequently, a water collection system with a perforated pipe system was installed after placing a sand layer. Airtight seals were provided by a membrane liner submerged in a trench at the border of the improved area. A vacuum pressure up to 80kPa was applied for approximately 3 months.

The observations (i.e. pore pressure reduction and degree of consolidation) are compared with the analytical results in Fig. 5, based on Equations (4)-(6). Upper and lower bounds of c_h (0.07 and 0.04 $\mathrm{m}^2/\mathrm{day}$) were used in the analysis. The prediction of degree of consolidation based on the average value of $c_h = 0.05\,\mathrm{m}^2/\mathrm{day}$ is also plotted. The values of $s = 3$ and $k_h / k_s = 2$ were applied in the analysis (Saye, 2003). The observed pore pressure reductions at 0.75m away from the centreline and at 12m depth also match reasonably well for average value of $c_h = 0.05\,\mathrm{m}^2/\mathrm{day}$ (Fig. 5a). It can be seen that the prediction of the total degree of consolidation using an average value of c_h ($0.05\,\mathrm{m}^2$/day) slightly underestimates the field measurements (Fig. 5b).



(a) (b)

Figure 5. Comparison between measured and predicted results (a) pore water pressure reduction and (b) total average degree of consolidation (Huanghua Port project, China, Gao, 2004)

Conclusions

Analytical modeling for consolidation considering both vertical and horizontal drainage has been proposed for vertical drains system incorporating vacuum preloading. The performance of a prefabricated vertical drain incorporating vacuum preloading was studied in the laboratory test, employing a large-scale consolidometer. Compared to the conventional method, the vacuum pressure application not simply acts as an additional surcharge load but also increases the excess pore pressure gradient towards the PVD to accelerate the consolidation process. In the case of vacuum application, the excess pore pressure becomes negative at the end of consolidation, whereas with the surcharge pressure, the excess pore pressure is always positive. The degree of consolidation based on measured settlement and measured average excess pore pressure is compared to the proposed solution by the authors. It is found that the authors' model agrees well with the measured average excess pore pressure rather than with the measured strain. This is attributed to the degree of consolidation that is based on the excess pore pressure dissipation rather than strains as formulated in the model. Subsequently, for a case history, the predictions made by the proposed analytical solution for vacuum assisted consolidation was compared with the actual field data. The model was found to be reliable in predicting the field behavior.

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