1-1-2012

# Non-linear analysis of soft ground consolidation at the Ballina by-pass

Xueyu Geng
*University of Wollongong*, xgeng@uow.edu.au

Buddhima Indraratna
*University of Wollongong*, indra@uow.edu.au

Cholachat Rujikiatkamjorn
*University of Wollongong*, cholacha@uow.edu.au

Richard Kelly
*Coffey Geotechnics, NSW*

# Non-linear Analysis of Soft Ground Consolidation at the Ballina By-pass

Xueyu Geng[1], Buddhima Indraratna[2], Cholachat Rujikiatkamjorn[3] and Richard Kelly[4]
[1]Research Fellow, Centre for Geomechanics & Railway Engineering; ARC Centre of Excellence for Geotechnical Science and Engineering; University of Wollongong, Wollongong City, NSW 2522, Australia; PH (+61) 2 4221 3385; FAX (+61) 2 4221 3238; email: xgeng@uow.edu.au
[2]Professor of Civil Engineering and Research director, Centre for Geomechanics and Railway Engineering; Program Leader, ARC Centre of Excellence for Geotechnical Science and Engineering; University of Wollongong, Wollongong City, NSW 2522, Australia; PH (+61) 2 4221 3046; FAX (+61) 2 4221 3238; email: indra@uow.edu.au
[3] Senior Lecturer, Centre for Geomechanics & Railway Engineering; ARC Centre of Excellence for Geotechnical Science and Engineering, University of Wollongong, Wollongong City, NSW 2522, Australia; PH (+61) 2 4221 5852; FAX (+61) 2 4221 3238; email: cholacha@uow.edu.au
[4]Principal, Coffey Geotechnics, NSW 2066, Australia email: Richard_Kelly@coffey.com.au

## ABSTRACT

This study presents a numerical assessment of consolidation under vacuum preloading with a system of vertical drains and membrane, considering the non-linear properties of the soil This membrane system, already widely used in Australia, is where an airtight membrane is placed over the drainage layer to allow a vacuum to be distributed within the sand platform, along the surface of the soil, and down the vertical drains. To date, there have only been a few fundamental investigations and field studies that have tried to assess the way in which the vacuum can propagate in soft clay, both laterally and vertically. In this model, both vertical and horizontal drainage was considered to reflect more realistic in-situ conditions. Moreover, the change of compressibility and permeability during consolidation was taken into consideration, including a possible loss of vacuum along the length of the drain. This model was initially verified for a single drain using large scale laboratory testing, and subsequently applied to a fully instrumented case study, namely the Ballina Bypass (along the Pacific Highway, NSW). The numerical solutions capturing the lateral distribution of the vacuum provided accurate predictions of the pore water pressure and associated settlement.

*Keywords:* Membrane system; vertical drain; non-linear analysis; Ballina By-pass; soft clay.

## 1 INTRODUCTION

The system of vertical drains combined with vacuum preloading is one of the most popular methods of increasing the shear strength of soil and reducing its post construction settlement by shortening the drainage path (Hansbo 1979, Atkinson and Eldred 1981, Runesson et al. 1985, Holtz et al. 1991, Hird et al., 1992, Zhu and Yin 2000, Fox et al. 2003, Walker and Indraratna 2006, Rujikiatkamjorn et al. 2009, Ghandeharioon et al. 2010).

In a membrane system, after prefabricated vertical drains (PVDs) are installed and the sand blanket is placed with horizontal perforated pipes, a membrane is laid on the top, the edges are placed in a trench and submerged under a bentonite slurry (Fig. 1) (Geng et al. 2011), and then the vacuum pumps are attached to a discharge system. One major advantage of this system is that the vacuum can be distributed within the sand platform, along the surface of the soil, and down the PVDs. However, one obvious drawback is that the efficiency of the entire system depends on maintaining an airtight system over a significant period of time. Thus far, there have only been a few fundamental investigations and field studies assessing how the vacuum propagates laterally and vertically in soft clay, and most of the existing theories still consider the coefficients of permeability and compressibility to be constant during consolidation (Davis and Raymond 1965, Basak and Madhav 1978, Xie et al. 2002, Geng et al. 2006, Cai et al. 2007). This paper presents a numerical solution for vertical drains with vacuum preloading and a membrane system, with the soil having non-linear properties. Both vertical and horizontal drainage, smear zone and the well resistance of the drain were considered in this analysis to reflect realistic in-situ conditions. The accuracy of the proposed solutions could be verified using the field measurements.

## 2 GOVERNING EQUATIONS AND BOUNDARY CONDITIONS

In order to study the loss of vacuum, the vacuum pressure along the boundary of the drain was considered to vary linearly from $p$ at the top of the drain to $\eta p$ at the bottom, where $\eta$ is a ratio between the vacuum at the top and bottom of the drain. The value of $\eta$ is between 1 and 0. If there is no loss of vacuum at the bottom of the PVDs, $\eta = 1$, and if the vacuum pressure is totally lost, then $\eta = 0$.

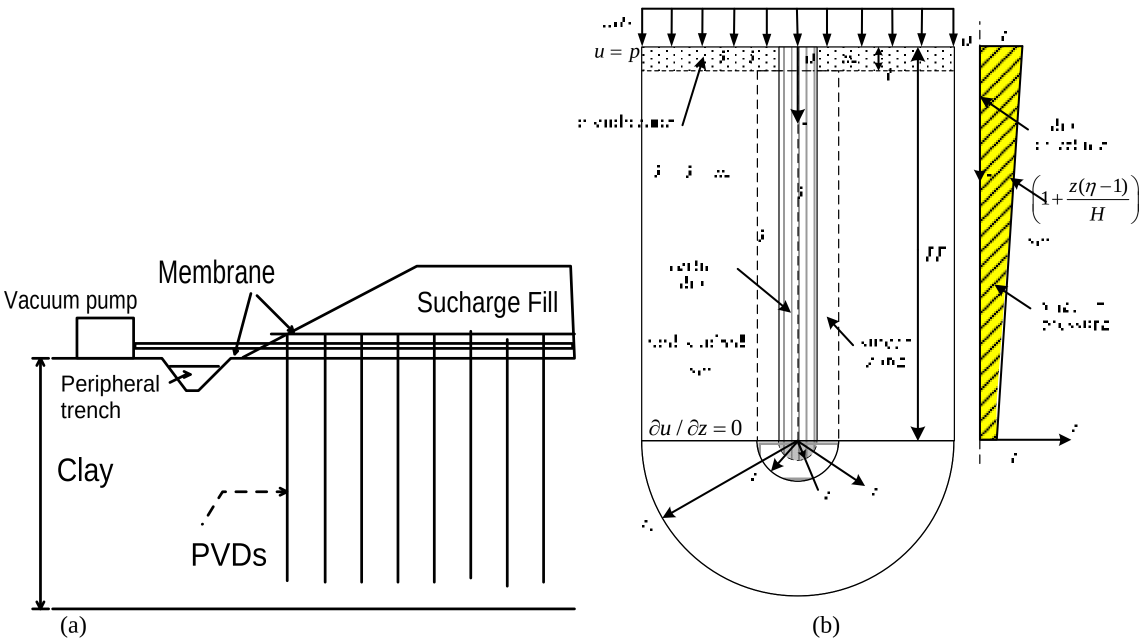


Figure 1. (a) Membrane system and (b) unit cell with vertical drain and vacuum distributed along the drain (Geng et al., 2011).

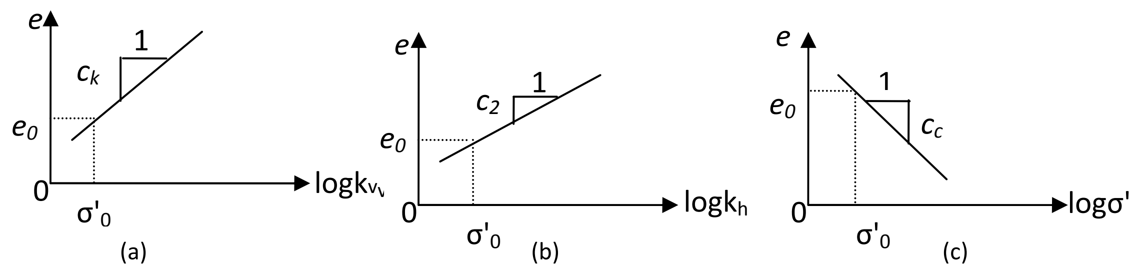


Figure 2. Constitutive relationships of compressibility and permeability of the soil layer: (a) hydraulic conductivity in the vertical direction; (b) hydraulic conductivity in the horizontal direction; (c) compressibility of the soil

The governing equations are:

$$\frac{\partial \varepsilon_v}{\partial t} = \frac{1}{1+e_0}\frac{\partial e}{\partial t} \tag{1}$$

$$-\frac{k_s}{\gamma_w}\left(\frac{1}{r}\frac{\partial u_s}{\partial r} + \frac{\partial^2 u_s}{\partial r^2}\right) - \frac{k_v}{\gamma_w}\frac{\partial^2 \overline{u}}{\partial z^2} = \frac{\partial \varepsilon_v}{\partial t} \qquad r_w \le r \le r_s \tag{2}$$

$$-\frac{k_h}{\gamma_w}\left(\frac{1}{r}\frac{\partial u_n}{\partial r} + \frac{\partial^2 u_n}{\partial r^2}\right) - \frac{k_v}{\gamma_w}\frac{\partial^2 \overline{u}}{\partial z^2} = \frac{\partial \varepsilon_v}{\partial t} \qquad r_s \le r \le r_e \tag{3}$$

$$\frac{\partial^2 u_w}{\partial z^2} = -\frac{2k_s}{r_w k_w}\frac{\partial u_s}{\partial r}\bigg|_{r=r_w} \tag{4}$$

$$\bar{u} = \frac{1}{\pi(r_e^2 - r_w^2)}\left(\int_{r_w}^{r_s} 2\pi r u_s dr + \int_{r_s}^{r_e} 2\pi r u_n dr\right) \tag{5}$$

The change in void ratio with permeability and effective stress (Fig. 2) can be expressed by:

$$e - e_0 = c_c \log(\sigma_0^{'} / \sigma^{'}) \tag{6a}$$

$$e - e_0 = c_k \log(k_v / k_{v0}) \tag{6b}$$

$$e - e_0 = c_2 \log(k_h / k_{h0}) \tag{6c}$$

The boundary conditions for both the radial and vertical directions are as follows:

$$r = r_e: \ \frac{\partial u_n}{\partial r} = 0, \ \frac{\partial \bar{u}}{\partial r} = 0 \tag{7a}$$

$$r = r_s: \ k_s \frac{\partial u_s}{\partial r} = k_h \frac{\partial u_n}{\partial r} \tag{7b}$$

$$r = r_s: \ u_s = u_n \tag{7c}$$

$$r = r_w: \ u_s = u_w, \ \bar{u} = u_w \tag{7d}$$

$$z = 0: \ u_w = p, \ \bar{u} = p \tag{7e}$$

$$z = H: \ \frac{\partial u_w}{\partial z} = \frac{\eta - 1}{H} p, \ \frac{\partial \bar{u}}{\partial z} = 0 \tag{7f}$$

The initial condition is:

At $t = 0$, $\bar{u} = q_0$ (7g)

where $r_s$ is the radius of the smear zone, $r_e$ is the radius of the influence zone, $r$ is the radial coordinate, $z$ is the vertical coordinate, $t$ is the time, $\varepsilon_v$ is the vertical strain, $e$ is the void ratio, $e_0$ is the initial void ratio, $c_c$ is the compression index, $c_k$ is the vertical hydraulic conductivity index, $c_2$ is the horizontal hydraulic conductivity index, $k_h$ is the horizontal coefficient of permeability of the soil, $k_v$ is the vertical coefficient of permeability of the soil, $k_s$ is the coefficient of permeability in the smear zone, $k_w$ is the coefficient of permeability of the vertical drain, $\bar{u}$ is the average pore pressure, $u_s$ is the pore pressure at any point in the smear zone, $u_n$ is the pore pressure at any point in the natural soil zone, $u_w$ is the excess pore water pressure within the vertical drain, $q$ is the time-dependent surcharge preloading, $q_0$ is the initial value of preloading, $L_w$ is the thickness of the sand, $H$ is the thickness of the whole layer (both sand blanket and clay), and $p$ is the vacuum pressure.

Equations (2) and (3) are highly non-linear, considering coupling between the pore fluid phase and soil skeleton and hence do not have a general solution with the boundary conditions mentioned above. Therefore, a finite element method was used here..

## 3 APPLICATION TO A CASE STUDY

In order to reduce traffic congestion at Ballina, Australia, the Pacific Highway linking Sydney to Brisbane was constructed. This by-pass route crosses a flood plain that consists of highly compressible and saturated marine clays up to 20m thick. Before construction began, a vacuum assisted surcharge load in conjunction with PVDs was used to shorten consolidation time and stabilise the deeper layers of subsoil. A trial embankment was built at the southern approach to Emigrant Creek, north of Ballina, to be used for a test period of 128 days, and then 34 mm diameter circular drains were installed in a square pattern, at 1.0m intervals. The locations of the field instruments, including the surface settlement plates, inclinometers and piezometers, are shown in Figure 4. The embankment was divided into 2 sections, i.e. Section A without vacuum pressure, and Section B with vacuum pressure. A 70 kPa (suction) vacuum was then applied at the drain interface. The bottom layer of soft clay at each settlement plate and the geotechnical parameters of the four layers of sub-soil obtained from standard oedometer tests are listed in Tables 1 and 2, respectively.

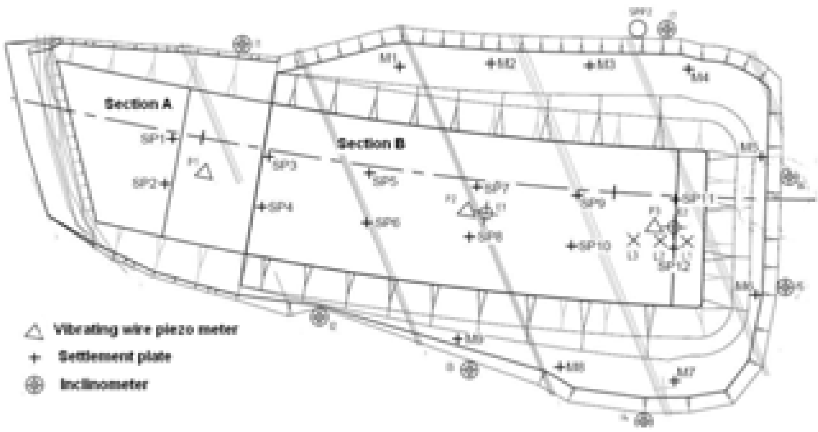


Figure 3. Layout of the instruments for the test embankment at Ballina Bypass.

Table 1. Thickness of the soft clay.

| Settlement plate | SP1 | SP3 | SP5 | SP7 | SP9 | SP11 |
|---|---|---|---|---|---|---|
| Thickness (m) | 6.7 | 9.7 | 11.7 | 14.7 | 17.7 | 24.7 |

Table 2.Soil parameters at SP11.

| Depth (m) | Soil Type | $k_{h0}\times10^{-10}$ (m/s) | $k_{v0}$ (m/s) | $c_c$ | $e_0$ | $\sigma_0'$ (kPa) | $q_u$ (kPa) | $p$ (kPa) | $c_{h0}\times10^{-8}$ (m/s) | $\gamma$ (kN/m$^3$) |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.0-3.3 | Clay | 15 | $k_{h0}/1.5$ | 1.41 | 3.73 | 31 | 72 | 0 | 1.36 | 14.5 |
| 3.3-4.8 | Silty clay | 7.2 | $k_{h0}/1.5$ | 0.6 | 1.15 | 37 | 153 | -70 | 2.2 | 14.9 |
| 4.8-8.3 | Silty clay | 5.88 | $k_{h0}/1.5$ | 1.13 | 2.16 | 50 | 153 | -70 | 1.89 | 15.3 |
| 8.3-24.7 | Clay | 4.6 | $k_{h0}/1.5$ | 1.15 | 1.94 | 79 | 153 | -70 | 2.14 | 15.5 |

Note: $q_u$ is the surcharge loading.

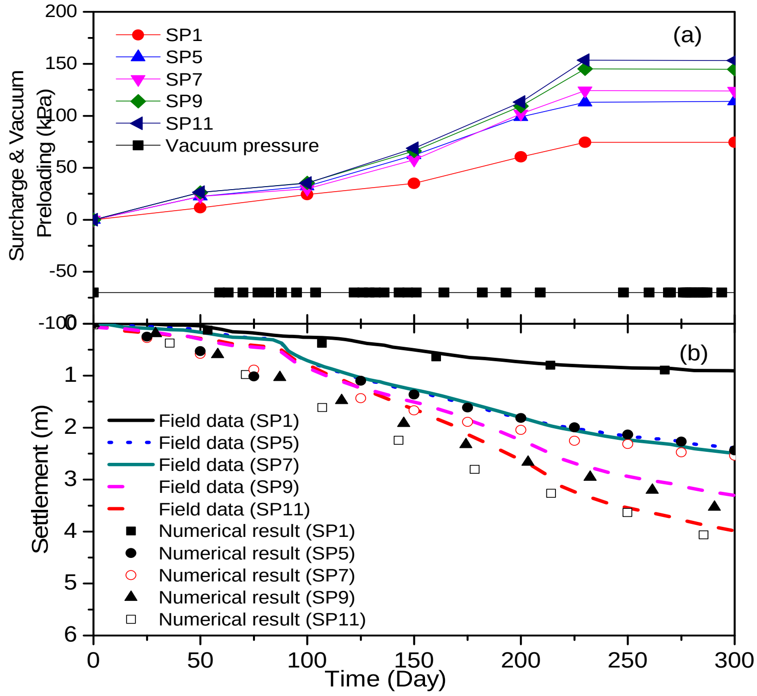


Figure 4. (a) History of applied surcharge loading and vacuum pressure; (b) Settlement comparison between field data and the numerical solutions.

Figure 4 compares the field data with the numerical results. During construction the settlement matched the field data quite well, but settlement increased when the thickness of the soils and amount of surcharge preloading increased. Figure 5 shows that after the embankment had been constructed, the dissipation of excess pore water pressure was slightly faster than the data measured in the field. In fact, during the early stages of construction, the surcharging preloading had a considerable influence on the dissipation of excess pore water, and this is especially so when construction takes a long time.

Figure 6 shows the difference between the degree of consolidation based on the dissipation of excess pore water pressure ( $U_p = \int_0^H \int_{r_w}^{r_e} r(q-u)drdz / \int_0^H \int_{r_w}^{r_e} rq_u drdz$ ) and the settlement ($U_s = \int_0^H \int_{r_e}^{r_w} r\varepsilon drdz / \int_0^H \int_{r_e}^{r_w} r\varepsilon_f drdz$), with a different construction time factor $T_{vc}$, respectively. For the same time factor ($T_v = c_{v0}t / H^2$, $c_{v0} = k_{v0}(1+e_0)\sigma_0^{'} \ln 10 / \gamma_w / c_c$), $U_p$ is always less than $U_s$, which is similar to the one dimensional non-linear consolidation theory obtained by Cai et al. (2007), Geng et al. (2006) and Xie et al. (2002). This also shows that the settlement based on $U_s$ occurred at a slightly higher rate than the settlement based on $U_p$. The difference between the linear and non-linear solutions increased with an increase in the rate of the construction time factor $T_{vc}$. For the degree of consolidation defined by the settlement ($U_s$), the non-linear model under constant load was the same as the linear model. However, for a given time factor ($T_v$) the degree of consolidation defined by the excess pore water pressure ($U_p$), the non-linear model predicted that $U_p$ under a constant load would be smaller than the linear solution.

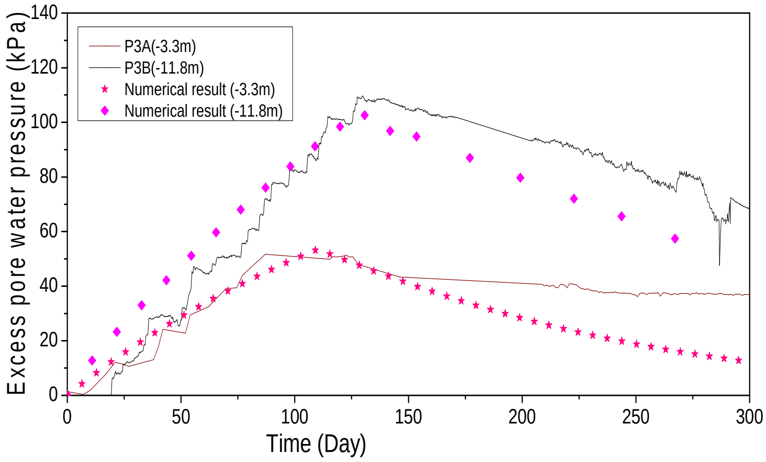


Figure 5. Comparison of excess pore water pressure between field data.

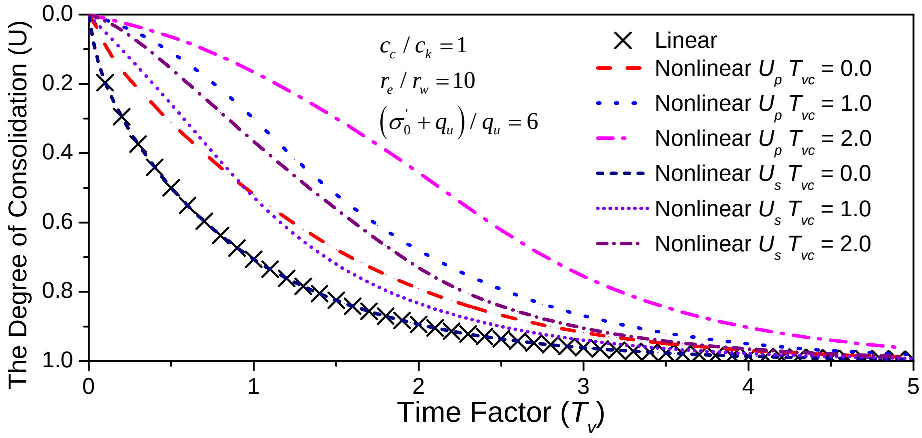


Figure 6. Different degree of consolidation between $U_p$ and $U_s$ with different construction time factor $T_{vc}$ under ramp loading

## 4 CONCLUSION

This study presents vertical drains with vacuum preloading as a solution for a membrane system that considered the non-linear property of soft clay. Both vertical and horizontal drainage were included in this analysis to reflect realistic in-situ conditions. The general solutions of pore water pressure, settlement, and the degree of consolidation were derived based on the FEM. Considering the non-linear characteristics of the soil, the degree of consolidation defined by pore water pressure ($U_p$) was quite different from the degree of consolidation defined by settlement ($U_s$). It was further found that $U_p$ was less than $U_s$, which is true of typical soft soils.

## 5 ACKNOWLEDGEMENTS

The authors acknowledge the support of the Australian Research Council (ARC), Road and Traffic Authority (RTA), Queensland Department of Transport and Main Roads (QTMR), Coffey Geotechnics and Douglas Partners Pty Ltd.

## REFERENCES

Atkinson, M.S. and Eldred, P. (1981). "Consolidation of soil using vertical drains."Geotechnique, 31(1), 33-43.

Basak, P., and Madhav, M. R. (1978). "Analytical Solutions of Sand Drain Problem." Journal of Geotechnical engineering, ASCE, 104(GT1), 129-135.

Cai, Y. Q., Geng, X. Y., and Xu, C. J. (2007). "Solution of One-dimensional Finite-strain Consolidation of Soil with Variable Compressibility under Cyclic Loadings." Computers and Geotechnics, 34, 31-40.

Davis, R. E., and Raymond, G. P. (1965). "A Non-linear of Consolidation." Geotechnique, 15(2), 161-173.

Fox, P. J., Di Nicola, M. and Quigley,D. W. (2003)."Piecewise-linear model for large strain radial consolidation."J. Geotech.Geoenviron. Eng., 129(10), 940–950.

Geng, X. Y., Indraratna, B. and Rujikiatkamjorn, C. (2011). "Analytical solutions for a single vertical drain with vacuum and time-dependent surcharge preloading in membrane and membraneless systems."Int. J.Geomech., ASCE, 2012, 12(1): 27-42.

Walker, R. and Indraratna, B. (2006). "Vertical Drain Consolidation with Parabolic Distribution of Permeability in Smear Zone."J. of Geotechnical & Geoenvironmental Engineering, ASCE, 132(7), 937-941

Geng, X. Y., Cai, Y. Q., and Xu, C. J.(2006). " Non-linear Consolidation Analysis of Soil with Variable Compressibility and Permeability under Cyclic Loadings." Int. J. Numer. Anal. Meth. Geomech, 30, 803-821.

Geng, X. Y. (2008). "Non-linear consolidation of soil with vertical and horizontal drainage under time-dependent loading". Proceeding of 2008 International Conference on Advanced Computer Theory and Engineering, 800-804.

Ghandeharioon, A., Indraratna, B., and Rujikiatkamjorn, C. (2010). "Analysis of soil disturbance associated with mandrel-driven prefabricated vertical drains using an elliptical cavity expansion theory."Int. J. Geomech.ASCE, 10(2), 53-64.

Hansbo, S. (1979). "Consolidation of clay by band-shaped pre-fabricated drains."Ground Eng., 12(5), 16-25.

Holtz, R. D., Jamiolkowski, M., Lancellotta, R. and Pedroni, S. (1991). "Prefabricated vertical drains: design and performance", CIRIA ground engineering report: ground improvement. UK: Butterworth-Heinemann Ltd, 131.

Hird C. C., Pyrah I.C., and Russel D. (1992). "Finite element modeling of vertical drains beneath embankments on soft ground."Geotechnique, 42(3), 499-511.

Runesson K, Hansbo S. andWiberg N. E. (1985)."The efficiency of partially penetrating vertical drains."Geotechnique. 35(4), 511–516.

Walker, R. and Indraratna, B. (2006). "Vertical drain consolidation with parabolic distribution of permeability in smear

Rujikiatkamjorn, C., Indraratna, B., and Chu, J. (2009). "2D and 3D Numerical Modeling of Combined Surcharge and Vacuum Preloading with Vertical Drains." International Journal of Geomechanics, ASCE, 8(2), 144-156.

Xie, K. H., Xie, X. Y., and Jiang, W. (2002). "A Study on One-dimensional Nonlinear consolidation of double-layered Soil." Computers and Geotechnics, 29, 151-168.

Zhu, G., and Yin, J. H. (2001). "Consolidation of Soil with Vertical and Horizontal Drainage under Ramp Load." Geotechnique, 51(4), 361-367.