

Consolidation of Ground with Partially Penetrated PVDs Combined with Vacuum Preloading

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ABSTRACT

Partially penetrating prefabricated vertical drain (PVDs) with a combined surcharge and vacuum preloading are considered in this paper. The analysis results can be used in cases where the soft soil clay is too deep and not economical to be penetrated to full depth, or the surcharge preloading is too small to justify full penetration of soft clay layer or the vacuum pressure can be lost, which is caused by the permeability of the bottom of the clay for the fully penetrated drain etc.. In this model, a virtual vertical drain is assumed to reflect the real three-dimensional seepage in the soil beneath the tip of PVD instead of using the traditional one-dimensional consolidation theory, and at the same time, the well-resistance and the smear zone can be also considered. The vacuum pressure distribution is assumed to be no loss along the drain, whereas a zero pore pressure boundary condition is assumed at the interface between the bottom clay layer and the lower drainage layer. The proposed solutions are then employed to analyze a case history.

INTRODUCTION

Most of Australian coastal areas contain thick soft soils and marine deposits. Stabilization of soft formation soils by applying a surcharge load alone often takes too long. The application of prefabricated vertical drains (PVDs) and vacuum pressure can shorten the preloading period significantly by decreasing the drainage path length (Chu et al. 2004, Rujikiatkamjorn and Indraratna, 2008). However, in some case, as the soft clay layer underlies with permeable layer the vacuum pressure can be lost. Therefore, the consolidation of soft ground with partially penetrated vertical drains with vacuum preloading and surcharge preloading is required.

When PVDs partially penetrate the clay layer, the soft clay under PVD tip does not consolidate the same as the overlying stratum. The soil consolidation with partially penetrated vertical drains has already been studied by numerical methods (Ruesson, et al., 1985; Onoue, 1988a; Nakano and Okuie, 1991, Tang et al., 1999), while some empirical methods have also been proposed (Hart, et al., 1958; Zeng and

Xie, 1989). Most of studies until now adopt one dimensional theory to determine consolidation beneath the PVD. However, the seepage velocity directly beneath the drain tip should be considered as true three-dimensional rather than a one-dimensional problem. Moreover, none of the previous studies incorporate the effect of the vacuum pressure when PVDs are partially installed.

Mathematical model and solution

A single unit cell theory was employed to simulate a single drain surrounded by a soil annulus in axi-symmetric conditions (Fig. 1). The vacuum pressure was assumed to be constant, whereas a zero pore pressure boundary is assumed at the bottom of the clay layer. A virtual vertical drain was used to represent real seepage in the soil beneath the tip of the partially installed PVD.

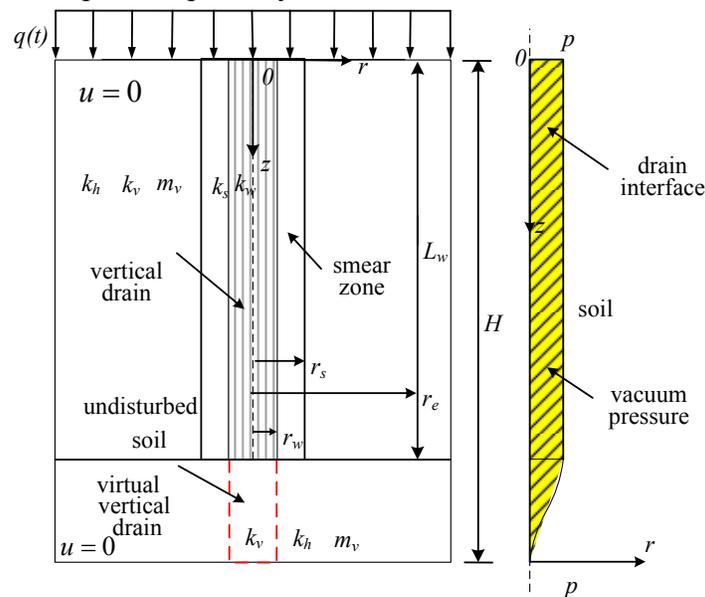


Figure 1 Analysis scheme of consolidation of clay with partially penetrated PVDs combined with vacuum pressure and surcharge preloading.

The equation for the excess pore water pressure dissipation in the soil stratum with PVD ($0 \leq z \leq L_w$) assuming an equal strain assumption is:

$$m_v \left(\frac{\partial \bar{u}_1}{\partial t} - \frac{dq}{dt} \right) = \begin{cases} \frac{k_s}{\gamma_w} \left(\frac{1}{r} \frac{\partial u_s}{\partial r} + \frac{\partial^2 u_s}{\partial r^2} \right) + \frac{k_v}{\gamma_w} \frac{\partial^2 \bar{u}_1}{\partial z^2}, & r_w \leq r \leq r_s \\ \frac{k_h}{\gamma_w} \left(\frac{1}{r} \frac{\partial u_n}{\partial r} + \frac{\partial^2 u_n}{\partial r^2} \right) + \frac{k_v}{\gamma_w} \frac{\partial^2 \bar{u}_1}{\partial z^2}, & r_s \leq r \leq r_e \end{cases} \quad (1)$$

where, z is the vertical coordinate, and t is the time for a given degree of consolidation r_w is the radius of the vertical drain, r_s is the radius of the smear zone, k_s is the horizontal coefficient of permeability of remolded soil, γ_w is the unit weight of water, $q(t)$ is the time-dependent surcharge preloading, m_v is the coefficient of volume compressibility of soil, $\bar{u}_1(z, r, t)$ is the average excess pore pressure of the

soil stratum improved by PVD at any depth, k_v is the vertical coefficient of permeability of the soil, r_e is the radius of influence zone, k_h is the horizontal coefficient of permeability of the soil, $u_s(r, z, t)$ is the excess pore water pressure at any point in the smear zone, and u_n is the pore pressure at any point in the zone of natural soil.

The equation for the dissipation of excess pore water pressure for the soil beneath the drain tip using a virtual drain with the same permeability of clay can be written as:

$$\frac{k_h}{\gamma_w} \left(\frac{1}{r} \frac{\partial u_2}{\partial r} + \frac{\partial^2 u_2}{\partial r^2} \right) + \frac{k_v}{\gamma_w} \frac{\partial^2 \bar{u}_2}{\partial z^2} = m_v \left(\frac{\partial \bar{u}_2}{\partial t} - \frac{dq}{dt} \right), \quad r_w \leq r \leq r_e \quad (2)$$

where, \bar{u}_2 is the average excess pore pressure of the soil stratum for the section without improvement by PVD at any depth, $u_2(r, z, t)$ is the excess pore water pressure of the soil stratum outside the area of the virtual vertical drain.

Continuity at the interface between the vertical drain and the smear zone can be determined form:

$$\frac{\partial^2 u_{w1}}{\partial z^2} = - \frac{2k_s}{r_w k_w} \frac{\partial u_{s1}}{\partial r} \Big|_{r=r_w} \quad (3a)$$

$$\frac{\partial^2 u_{w2}}{\partial z^2} = - \frac{2k_h}{r_w k_v} \frac{\partial u_2}{\partial r} \Big|_{r=r_w} \quad (3b)$$

where, $u_{w1}(r, z, t)$ is the excess pore water pressure within the real vertical drain, and k_w is the coefficient of permeability of the vertical drain. $u_{w2}(r, z, t)$ is the excess pore water pressure within the virtual vertical drain.

The average excess pore water pressure at a given depth is then given by:

$$\bar{u}_1 = \frac{1}{\pi(r_e^2 - r_w^2)} \left(\int_{r_w}^{r_s} 2\pi r u_s dr + \int_{r_s}^{r_e} 2\pi r u_n dr \right) \quad (4a)$$

$$\bar{u}_2 = \frac{1}{\pi(r_e^2 - r_w^2)} \int_{r_w}^{r_e} 2\pi r u_2 dr \quad (4b)$$

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

$$\text{Impermeable wall at } r = r_e: \frac{\partial u_n}{\partial r} = 0, \quad \frac{\partial u_2}{\partial r} = 0 \quad (5a)$$

$$\text{Continuity of pore water pressure gradient at } r = r_s: k_s \frac{\partial u_s}{\partial r} = k_h \frac{\partial u_n}{\partial r} \quad (5b)$$

$$\text{Continuity of pore water pressure at } r = r_s: u_s = u_n \quad (5c)$$

$$\text{Continuity of pore water pressure at } r = r_w \text{ implies, } u_s = u_{w1}, \quad u_2 = u_{w2} \quad (5d)$$

$$\text{At the top of the clay layer } z = 0 \text{ ensures } u_{w1} = p \text{ and } \bar{u}_1 = p \quad (5e)$$

$$\text{At the bottom of the clay layer } z = H \text{ (pervious boundary): } u_{w2} = 0, \quad \bar{u}_2 = 0 \quad (5f)$$

where, p is the vacuum pressure.

Using the method of Laplace transform and the inversion of Laplace transform (Durbin 1974), the solution for the excess pore water pressure u_{wi} and \bar{u}_i ($i=1,2$) could be obtained as:

$$\bar{u}_{wi}(Z, T_h) = \frac{1}{2\pi I} \int_{a-I\infty}^{a+I\infty} \hat{u}_{wi}(Z, S) e^{ST} dS \quad (i=1,2) \quad (6a)$$

$$\bar{u}_i(Z, T_h) = \frac{1}{2\pi I} \int_{a-I\infty}^{a+I\infty} \hat{u}_i(Z, S) e^{ST} dS \quad (i=1,2) \quad (6b)$$

where, $I = \sqrt{-1}$, $\hat{u}_{wi}(Z, S)$ and $\hat{u}_i(Z, S)$ ($i=1,2$) are the Laplace transforms of $u_{wi}(Z, T_h)$ and $\bar{u}_i(Z, T_h)$ ($i=1,2$), S is the Laplace transform of the dimensionless time factor $T_h = \frac{c_h \cdot t}{de^2}$, Z is the normalized parameter as $Z = \frac{z}{H}$.

The settlement of the soil is now given by:

$$s(t) = \int_0^H \varepsilon_i dz \quad (7)$$

where, ε_i ($i=1,2$) is the vertical strain of the soil stratum improved by PVD.

Verifications with finite element analysis

The predictions based on the proposed analytical solutions are compared with those obtained via numerical analysis using the commercial software ABAQUS. A total of 1400 elements (8-node bi-quadratic displacement and bilinear pore pressure) were adopted in the finite element analysis (Figures 2a and 2b). The consolidation of the unit cell with a central vertical drain was simulated based on Biot's theory (Biot 1941). A non-lateral displacement condition was created to justify the condition at the embankment centreline. The horizontal undisturbed soil permeability ($k_{h,ax}$) was obtained from 1-D consolidation tests to be 10^{-10} m/s, and the coefficient of soil compressibility (m_v) as 10^{-3} m²/kN. According to Indraratna and Redana (2000), the ratio of the undisturbed permeability to the smear zone permeability (k_h/k_s) was 4.0. The outer boundary was assumed to be impermeable, whereas the top and bottom boundaries were assumed pervious (see Figure 3b). The equivalent drain diameter (d_w) was taken to be 100mm, hence, the smear diameter (d_s) was considered to be 300mm, based on the laboratory findings described earlier by Indraratna and Redana (1997). In the following analysis, the discharge capacity (q_w) of the drain is assumed to be high enough, therefore, the well resistance can be neglected. For the analytical and numerical analysis, the following three cases were examined (a) Surcharge preloading only (80kPa); (b) Vacuum preloading only (80kPa); and (c) Vacuum (40kPa) plus surcharge preloading (40kPa)

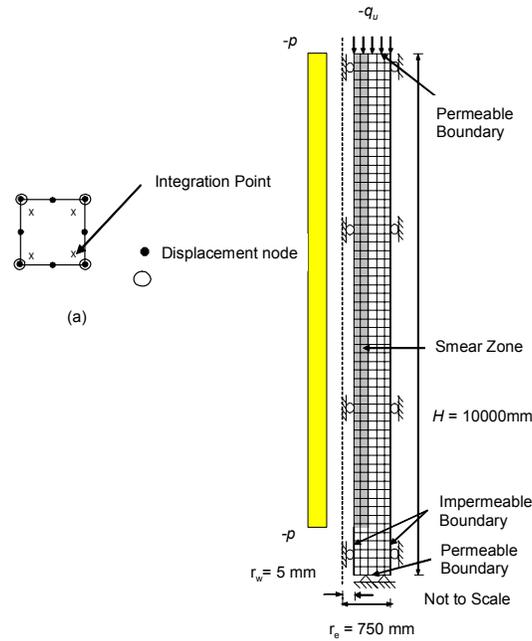


Figure 2 Finite element discretization for partially penetrating drain in unit cell, (a) Nodes and integration points for a single 8-node biquadratic displacement, bilinear pore pressure element; and (b) Mesh discretization and vacuum pressure distribution for short drain analysis.

The comparison of normalised settlement between the analytical model and numerical predictions for the above 3 cases is shown in Figure 3. For a given applied load, the normalized settlement was calculated based on the ratio of settlement divided by the ultimate settlement of the clay layer. Overall, a good agreement between the analytical and numerical techniques was obtained. It can be seen that the ultimate normalised settlements obtained for Cases (b) is always less than unity, because of the vacuum pressure loss at the bottom of the pervious boundary.

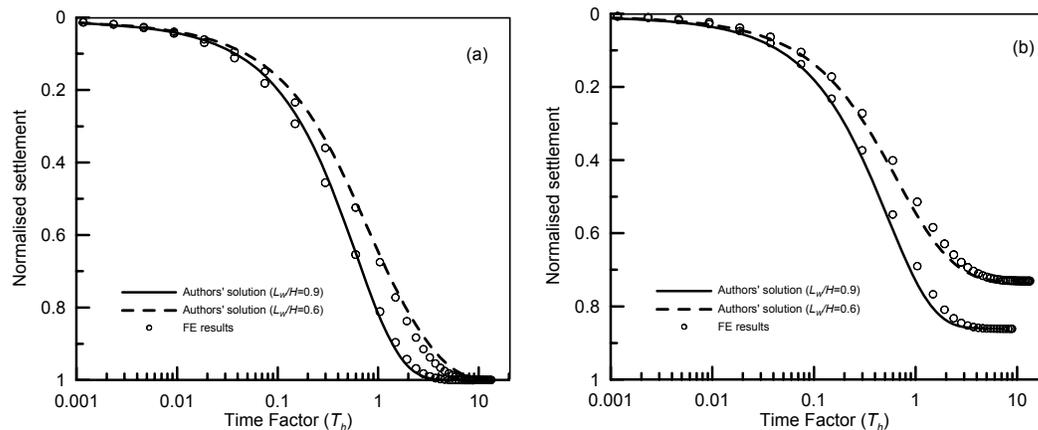


Figure 3 Comparison of model prediction between Authors method and finite element analysis: (a) Surcharge only; (b) vacuum only.

Case Study

Tianjin port is situated on the western side of the Bohai Gulf in China. The soils in and under the reclaimed land for the port had to be improved to enhance their capacity so as to safely and with limited settlements sustain heavy loading from the stacking yard. PVDs with vacuum preloading combined with surcharge preloading were used to accelerate the time dependent settlement of the underlying soft soil strata. To prevent losing vacuum pressure partially penetrating PVDs were used in the pilot test area as fine sand was underlying the soft clay layer. A typical soil profile of the area consists of a 4 m thick hydraulic fill underlain by 15 m of silty clay, followed by 7 m of clayey sandy loams, which were in turn overlain by fine sand. The hydraulic fill was primarily dredged silty clay with clayey silt laminations. The particle size distribution, frequency, and thickness of the clayey silt laminations depended largely on the distance from the discharge points of the hydraulic fill. Since the discharge points were mainly on the southern side of the reclamation area and the outlet points in the northern side, the particle size distribution was finer in the northern half of the site. The soil properties determined before improvement (Fig. 4) were reported by Choa (1990). The soil and drain parameters used in the analysis adopted from Choa (1990) are tabulated in Table 1.

Table 1. Vertical drain parameters and soil properties

Spacing	1.0 m (square)
Length of vertical drain	18 m
Dimension of drain	100×4 mm ²
Discharge capacity, q_w	100 mm ³ /year (per drain)
Dimension of mandrel	120×50 mm ²
k_h / k_w	1.05×10^{-6}
k_h / k_v	1.0
k_h / k_s	4.0
c_v	$1.589 \times 10^{-7} \text{ m}^2 / \text{s}$
c_h	$2.514 \times 10^{-7} \text{ m}^2 / \text{s}$

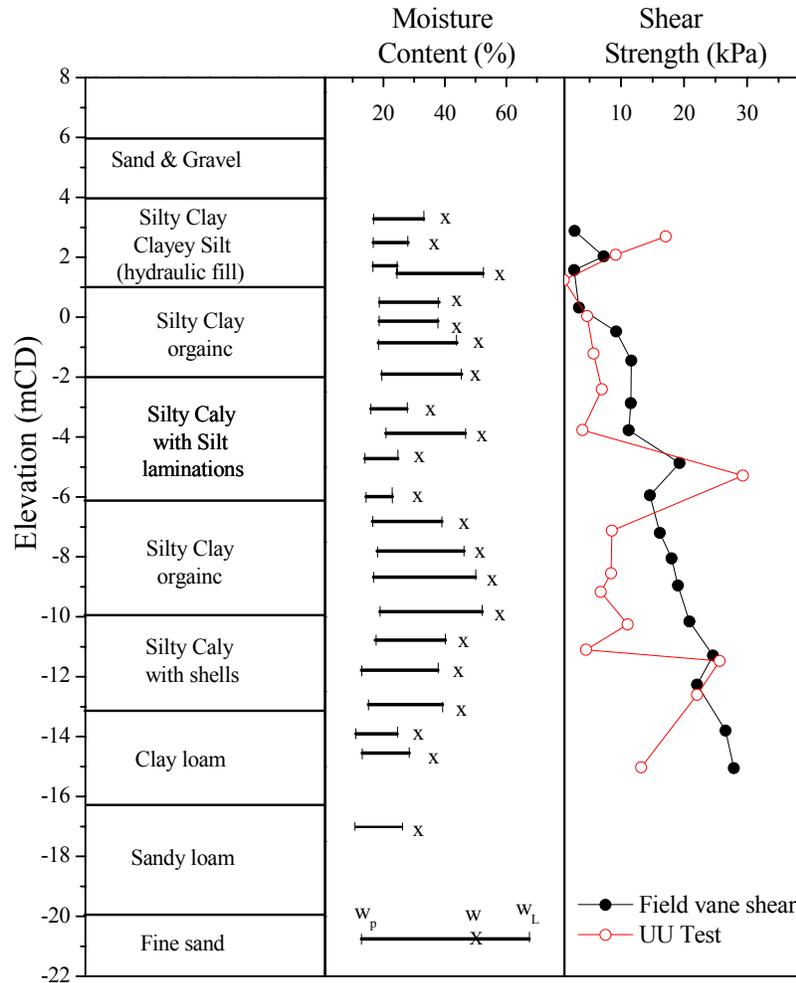


Figure 4 Soil properties before improvement.

All the PVDs were 18m long while the clay layer was 24 m thick (i.e., $\rho = 0.75$). The required preloading intensity was 107 kPa, while the maximum preloading which could be obtained from vacuum suction was expected to vary between 80 and 90 kPa. It was proposed that a combined vacuum and surcharge fill would be needed to achieve the desired preloading pressure for 95% degree of consolidation after about 6 months (Figs. 5a and 5b). The progress of the ground improvement was monitored through extensive instrumentation, sampling, as well as field and laboratory testing. A time-dependent vacuum was applied based on the field measurements (Fig. 5b). The settlement predictions based on Eq. 7 agreed with the field measurements (Fig. 5c). When a PVD is longer but does not totally penetrate the entire clay (i.e. $\rho = 0.9$), it would cause more settlement. However, if it were to approach the layer of fine sand there would be vacuum loss from the bottom pervious boundary which will create a zero vacuum head at the bottom end of the PVD. In this case, the analysis assumed a triangular distribution of suction down the drain (Indraratna et al. 2004, 2005); hence, the resulting settlement would be smaller.

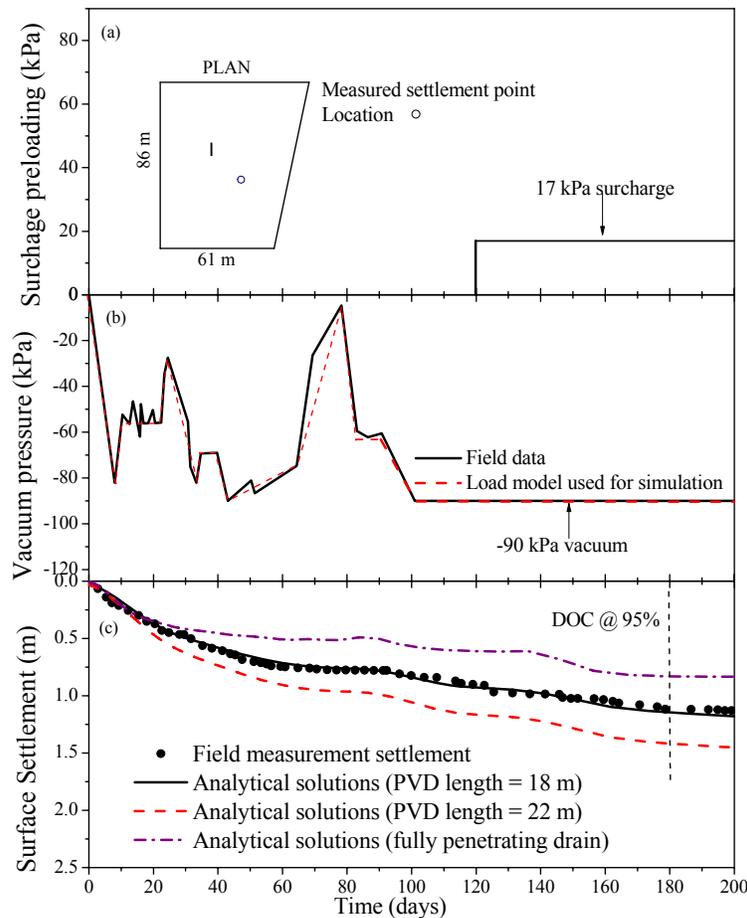


Figure 5 Settlement due to vacuum preloading combined with surcharge preloading.

CONCLUSION

An analytical solution for partially penetrating vertical drains for vacuum preloading combined with surcharge preloading was presented. In this model, a virtual vertical is assumed to reflect the real three-dimensional seepage in the soil beneath the tip of PVD instead of using the traditional one-dimensional consolidation theory, while at the same time, the well-resistance and the smear zone can also be considered. The vacuum pressure distribution is assumed to be “no loss” along the drain, whereas a “zero pore pressure” boundary condition is assumed at the interface between the bottom clay layer and the lower drainage layer. The present results compared with the finite element predictions. A case history taken from Tianjin Port, China, was discussed and analyzed using the analytical solutions proposed in this study, capturing the time-dependent vacuum pressure variation and the surcharge preloading history based on the field data. Very accurate predictions were obtained when the authors’ proposed analytical solutions (models) were compared with field data, due to the appropriate consideration of the vacuum distribution along the drain length and the realistic flow conditions beneath the drain tips. If the PVDs fully

penetrate towards the bottom of the soft clay layer and reach the underlying pervious sand layer, then the resulting settlement at any given time would become smaller due to the loss of vacuum.

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REFERENCES

- Barron, R.A., (1948). "Consolidation of fine-grained soils by drain wells." *Transactions of the ASCE*, 113: 718–742.
- Carrillo, N., (1942). "Simple two and three-dimensional cases in the theory of consolidation of soils." *Journal of Math. Phys.*, 21: 1-5.
- Choa, V., Wong, K.S., and Low, B.K., (1990). "New airport at Chek Lap Kok, geotechnical review and assessment ." *Consulting Report to Maunsell Pte Ltd.* Singapore.
- Chu, J., and Yan, S.W., (2005). "Application of vacuum preloading method in soil improvement project." *Case Histories Book, Edited by Indraratna, B. and Chu, J., Elsevier, London.* Vol. 3: 91-118.
- Durbin, F., (1974). "Numerical Inversion of Laplace Transform: An Efficient Improvement to Dubner and Abita's Method." *The computer Journal*, 17(9): 371-376.
- Hart E.G., Kondner R.L., Boyer W.C., (1958). "Analysis for partially penetrating sand drains." *Journal of Soil Mechanics and Foundation Division, Proceedings of ASCE*: 1812-1—1812-15.
- Indraratna, B., Bamunawita, C., and Khabbaz, H., (2004). "Numerical modeling of vacuum preloading and field applications." *Canadian Geotechnical Journal*, 41: 1098-1110.
- Indraratna, B., Rujikiatkamjorn C., and Sathananthan, I., (2005). "Analytical and numerical solutions for a single vertical drain including the effects of vacuum preloading." *Canadian Geotechnical Journal*, 42: 994-1014.
- Indraratna, B., and Rujikiatkamjorn C., (2008). "Effects of partially penetrating prefabricated vertical drains and loading patterns on vacuum consolidation." *In K. R. Reddy, M. V. Khire & A. N. Alshawabkeh (Eds.), GeoCongress* (pp. 596-603). USA: ASCE.
- Onoue, A., (1988). "Consolidation of multilayered anisotropic soils by vertical drains with well resistance." *Soils and Foundations*, 28(3): 75-90.
- Runesson, K., Hansbo S., Wiberg N.E., (1985). "The efficiency of partially penetrating vertical drains." *Geotechnique*, 35(4): 511-516.
- Tang, X.W., Onitsuka K., (1998). "Consolidation of ground with partially penetrated vertical drains." *Geotechnical Engineering Journal*, 29(2): 209-231.
- Zeng, G.X., Xie, K.H., (1989). "New development of the vertical drain theories." *Proc. 12th ICSMFE, (2), Rio de Janeiro*: 1435-1438.