Preliminary Design of Prefabricated Vertical Drains-Embedded Soft Soils in the Field: An Example

Rai Bahadur Reang, Sujit Kumar Pal, and Sanjay Paul

Abstract The paper presents a rough idea of preliminary design steps of designing prefabricated vertical drains-embedded soft soils in the field. In the past years, many researchers have exercised a number of analytical approaches, experimental works in the laboratory that are very laborious and time consuming or numerical analyses on PVD-embedded soils by taking into account several parameters such as horizontal and vertical coefficients of consolidations (c_h and c_v), horizontal and vertical coefficients of permeability (k_h and k_v), Over Consolidation Ratio (OCR), pore pressure, 'n' value (ratio of effective diameter and equivalent drain diameter of PVD) or other essential parameters associated with the design. But a thorough knowledge of the procedure of the prefabricated vertical drain design cannot be perceived or acquired from those analyses since a complete design step is out of the scope of their research. They do not provide design procedures in their publications. This may cause a fresh researcher to think very deeply about the design steps or it may take them long periods to acquire a complete idea of the PVD design procedure. So, in this paper an interpretation of the PVD design is made precisely with the help of an example that will provide an aggregate idea of the design of prefabricated vertical drains-embedded soft soils in the field, and thus, the paper may be helpful to the fresh researchers in the field of PVD.

Keywords PVD · Embankment · Soft soils · Ground improvement

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1 Introduction

In practice, while soft soils are encountered before any huge-funded/costlier geotechnical engineering construction or if, as per the site requirement, the proposed area is too large to avoid, then the only way to come up with the best solution is to improve the existing ground with some low-cost agents or materials. If the construction of an embankment (in road or railway line or some other important structures) is a part of the proposed project, the best solution to improve the underlain ground is to use prefabricated vertical drains (PVDs). Once PVDs are installed into the ground, the time required for consolidation settlement of the existing field gets decreased. Consequently, the cost of the project or maintenance cost also gets reduced. However, a site engineer or designer of the project must have a thorough knowledge of how the structure is designed and completed within a minimum stipulated period of time with low cost and maintenance as far as possible.

Design charts are developed [1] analytically to determine the field spacing of PVD without requiring iterations that also consider hydraulic conductivity of the soil. Probabilistic analysis that uses finite-element modelling is a useful tool [2] to determine the consolidation of soil reinforced with PVDs. Among all properties of soil, the spatial variability of soil permeability (k) and the coefficient of volume compressibility (m_v) play the most important crucial roles in the consolidation of soils, as indicated by several researchers [3]. Well, resistance and smear effects are other influencing factors that affect the rate of consolidation [4, 5]. The parameters required to characterize the smear zone are the affected area of the disturbed soil and the ratio of the horizontal coefficient of permeability in the undistorted zone to that in the smear zone [6]. Using reconstituted soils, Onoue et al. [7] investigated the distribution of the horizontal coefficient of permeability through laboratory experiments, and suggested using a three-zone model for any variation in permeability around a drain. Indraratna and Redana [8] introduced a preliminary analytical and numerical model to capture the effect of the smear zone, but the role played by the soil structure was not properly captured. Observational techniques, such as the Asaoka method [9] or hyperbolic methods [10, 11] are widely used in practical applications (e.g. [12, 13]).

It is seen that very complicated and intricate formulae and findings are involved in many research publications, but a constitutive design method is unavailable in those studies or is out of their research scope. That is why preliminary design steps are presented in this paper with the help of an example that gives a summary of how a PVD-reinforced soft soil is designed in the field. In Fig. 1, it is desired to achieve primary consolidation plus 1 log cycle of secondary compression resulting from the highway embankment loading within 24 months (730 days) of the completion of the fill. Hence, the design steps are narrated in the next section.
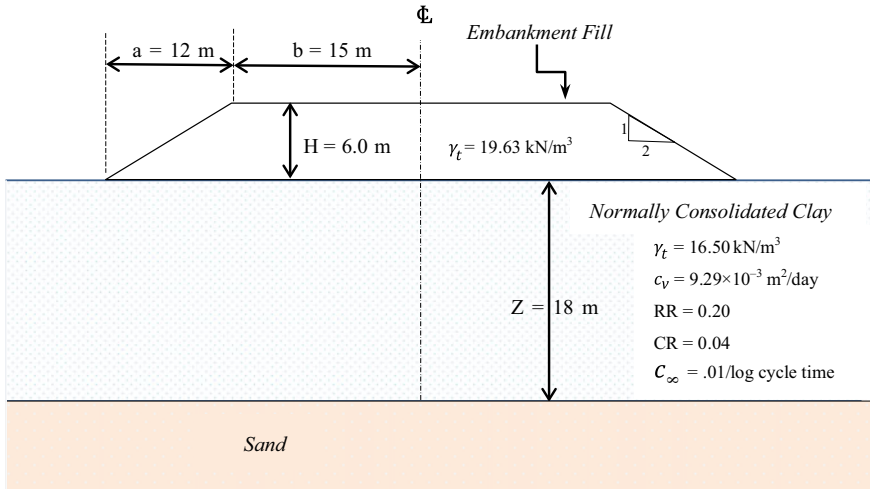
Fig. 1 An embankment over PVD-reinforced soft soil for road construction

2 Design Steps of the PVD-Embedded Soil

2.1 Design Assumptions

Here, some assumptions are made in the design of PVD. These are: (i) Embankment and surcharge loadings occur instantaneously for the purpose of settlement calculations. (ii) Stability of the embankment (i.e., staged loading, toe berms etc.) is considered in a separate analysis, so not presented here.

2.2 Design Methodology

The steps are discussed in the followings: (A) Evaluate the effects of the proposed embankment—(i) Calculate effective stress increment under centerline due to embankment, (ii) Develop stress history and stress change profile, (iii) Predict total settlement due to embankment, (iv) Consider time rate of consolidation; (B) Evaluate the required surcharge—(i) Estimate the required height of surcharge, (ii) Predict primary consolidation due to embankment and surcharge, (iii) Calculate required $\bar{U}_h$, (iv) Check for drain spacing; (C) Other design aspects—(i) Soil disturbance, (ii) Drain resistance, (iii) Drainage blanket, etc.

(A)   Evaluation of the effects of the proposed embankment—

(i)   Calculation of effective stress increment under centerline due to embankment:

Here, the increment of effective stress due to embankment under its centerline is calculated using the method proposed by Osterberg [14], as given below.

$$\sigma_z = I \cdot q \tag{1}$$

$$\Delta\sigma_v = 2 \times I \times p \tag{2}$$

where

$$I = \frac{1}{\pi}\left[\left(\frac{a+b}{a}\right)(\alpha_1 + \alpha_2) - \frac{b}{a}\alpha_2\right] \text{ and } p = H\gamma_t$$

Results of different parameters (z, a/z, b/z, I and $\Delta\sigma_v$) are shown along with the soil profile sketch in Fig. 2.

(ii) Development of stress history and stress change profile:

The next step is to acquire the knowledge and idea about the soil profile through a past history or any other sources (Fig. 3). Stress history of the soil profile is necessary to get the information about the ground soil whether it is an overconsolidated and normally consolidated one. A graph is shown in Fig. 4 so that it helps understand how much does stress profile important in the design of an embankment and PVD.

(iii) Predict total settlement due to embankment:

(a) Initial undrained settlement: does not affect the PVDs, so $p_i = 0$.
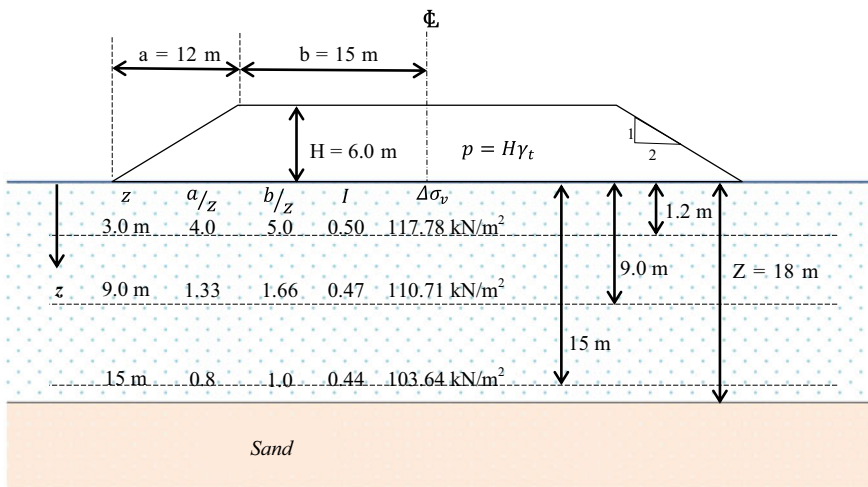(b) Primary consolidation: The formula for primary consolidation is given as

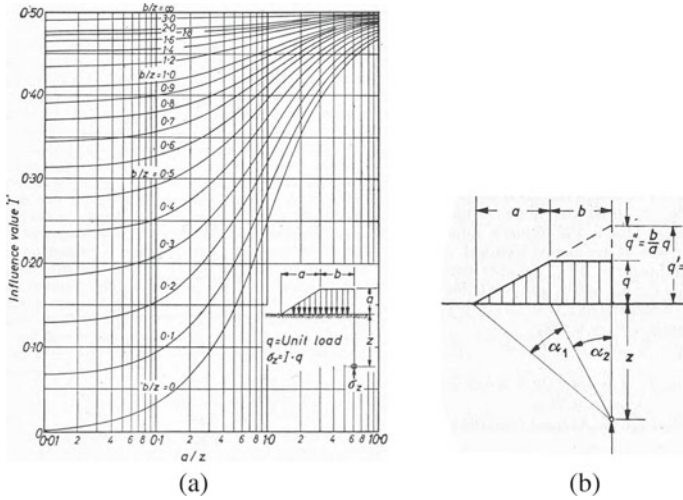**Fig. 2** Effective stress under the centerline of the embankment

Fig. 3 Influence chart for vertical stress due to embankment loading (After [14])
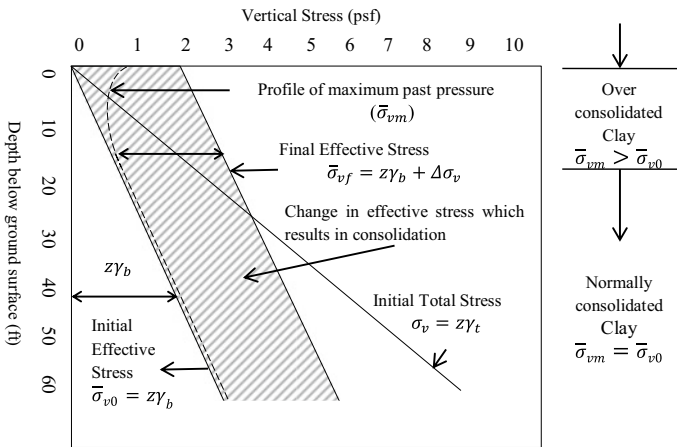

Fig. 4 Stress profile of a soil [15]

$$p_p = RR.H.log\left[\frac{\bar{\sigma}_{vm}}{\bar{\sigma}_{v0}}\right] + CR.H.log\left[\frac{\bar{\sigma}_{vf}}{\bar{\sigma}_{vm}}\right] \qquad (3)$$

The values of primary consolidations (p_p) are determined as per Eq. (3) and the results of other parameters are shown in Table 1.

Therefore, total amount of primary consolidation is $(p_p) = 0.65 + 0.55 + 0.37 = 1.57$ m.

Table 1 Stress results with respect to depth

Depth interval	$\bar{\sigma}_{v0}$ (kN/m²)	$\bar{\sigma}_{vm}$ (kN/m²)	$\bar{\sigma}_{vf}$ (kN/m²)	H_i (m)	CR	RR	p_p (m)
0–6	20.59	47.88	140.29	6	0.20	0.04	0.65
6–12	61.29	61.29	174.28	6	0.20	0.04	0.55
12–18	101.98	101.98	208.28	6	0.20	0.04	0.37

(c) Secondary consolidation: The formula for secondary settlement is given as

$$p_s = H.c_\infty \log\left[\frac{t_f}{t_p}\right] = (18)(0.01)\log\left[\frac{10t_p}{t_p}\right] = 0.18\,\text{m}.$$

Therefore, total consolidation settlement is given as

$$p_c = p_i + p_p + p_s = 0 + 1.57 + 0.18 = 1.75\,\text{m}.$$

(iv) Time rate of settlement:

The general equation for consolidation considering both horizontal and radial direction is given as

$$\bar{U} = 1 - \left(1 - \bar{U}_v\right)\left(1 - \bar{U}_h\right)$$

Here, two-way vertical drainage is considered, so, $\bar{U}_h = 0$, $\therefore \bar{U} = \bar{U}_v$
Now, considering $\bar{U}_v = 90\%$, (T = 0.843)

$$\therefore\ t = \frac{TH^2}{C_v} = \frac{(0.843)\left(\frac{18}{2}\right)^2}{9.29 \times 10^{-3}}\ 7350\ \text{days} \gg 730\ \text{days}\ (2\ \text{yrs.})$$

Therefore, there is a need to consider other options.

$\therefore\ T = \frac{tc_v}{H^2} = \frac{(730)\times 9.29 \times 10^{-3}}{9^2} = 0.083$
$\therefore\ \bar{U}_v = 32\%$

Therefore, there is a need for surcharge weight since the design requirement is to achieve primary consolidation plus 1 cycle of secondary compression within time, $(t < 10t_p)$.

(B) Evaluation of the required surcharge —

(i) Estimation of the required height of surcharge:

$$\bar{U} = 1 - \left(1 - \bar{U}_v\right)\left(1 - \bar{U}_h\right)$$
$$= 1 - (1 - 0.32)(1 - 0.85),\ [\text{assuming},\ \bar{U}_h = 0.85]$$
$$= 0.90$$

Again, we know that

$\bar{U} = \frac{p_c}{p_{cf}}$ or, $p_{cf} = 1.94$, [Given, $\bar{U} = 0.90$ and $p_c = 1.75$].

Now, the equation for final settlement is given as

$$p_{cf} = RR.H.\log\left[\frac{\bar{\sigma}_{vm}}{\bar{\sigma}_{v0}}\right] + CR.H.\log\left[\frac{\bar{\sigma}_{vf}}{\bar{\sigma}_{vm}}\right] + CR.H.\log\left[\frac{\bar{\sigma}_{vs}}{\bar{\sigma}_{vf}}\right] \tag{4}$$

Or, $1.94 = 1.57 + CR.H.\log\left[\frac{\bar{\sigma}_{vs}}{\bar{\sigma}_{vf}}\right]$

Or, $\left[\frac{\bar{\sigma}_{vs}}{\bar{\sigma}_{vf}}\right] = 1.27$, [where, $\bar{\sigma}_{vs}$ is the effective vertical stress due to surcharge]

Therefore, the minimum surcharge height should be $(1.27-1)(6) = 1.62$ m. Let us assume the height of the extra surcharge is 2.5 m (Fig. 5).

(ii) Prediction of primary consolidation due to embankment and surcharge: The formula for final consolidation is given as (Table 2).
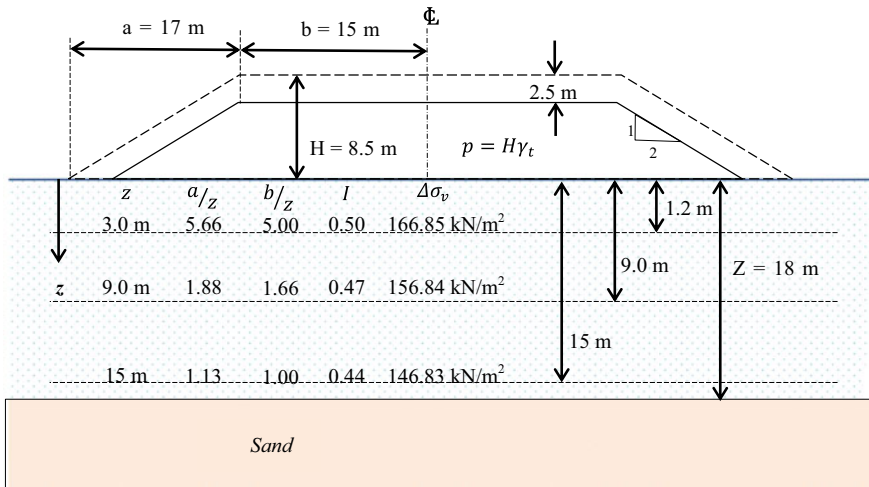

Fig. 5 Effective stress (embankment loading + extra surcharge of 2.5 m high) under the centerline of the embankment

Table 2 Stress (including surcharge) results with depth

Depth interval	$\bar{\sigma}_{v0}$ (kN/m^2)	$\bar{\sigma}_{vm}$ (kN/m^2)	$\bar{\sigma}_{vs} = \bar{\sigma}_{v0} + \Delta\bar{\sigma}_v$ (kN/m^2)	H_i (m)	CR	RR	p_{cf} (m)
0–6	20.59	47.88	188.17	6	0.20	0.04	0.81
6–12	61.29	61.29	225.51	6	0.20	0.04	0.68
12–18	101.98	101.98	259.51	6	0.20	0.04	0.49

$$p_{cf} = RR.H \cdot \log\left[\frac{\bar{\sigma}_{vm}}{\bar{\sigma}_{v0}}\right] + CR.H \cdot \log\left[\frac{\bar{\sigma}_{vs}}{\bar{\sigma}_{vm}}\right] \tag{5}$$

Therefore, the total value of settlement is $(p_{cf}) = 0.81 + 0.68 + 0.49 = 1.98$ m. Check,

$$\bar{U} = \frac{p_c}{p_{cf}} = \frac{1.75}{1.98} = 0.88, \text{ (OK).}$$

(iii)   Calculation of the required $\bar{U}_h$:

The general equation for consolidation considering both horizontal and radial direction is given as

$$\bar{U} = 1 - \left(1 - \bar{U}_v\right)\left(1 - \bar{U}_h\right)$$
$$\text{Or, } \bar{U}_h = 1 - \frac{(1-\bar{U})}{(1-\bar{U}_v)} = 1 - \frac{(1-0.89)}{(1-0.32)} = 0.84$$

(iv)   Check the approximate drain spacing:

$$t = \frac{D^2}{8 \times c_h} \times \left[ln\left(\frac{D}{d_w}\right) - \frac{3}{4}\right] \times ln\left(\frac{1}{1 - \bar{U}_h}\right)$$

where
$D =$ Effective diameter of area covered by each drain
$d_w =$ Equivalent diameter of each drain
$\bar{U}_h =$ Degree of consolidation settlement
Assuming,
$d_w = 0.05$ m, $c_h = c_v = 9.29 \times 10^{-3}$ m²/day, and, t $= 730$ days, $\bar{U}_h = 0.84$

$$t = \frac{D^2}{8 \times 9.29 \times 10^{-3}} \times \left[ln\left(\frac{D}{0.05}\right) - \frac{3}{4}\right] \times ln\left(\frac{1}{1 - 0.84}\right)$$
$$\text{Or, } t = \frac{D^2}{8 \times 9.29 \times 10^{-3}} \times \left[ln\left(\frac{D}{0.05}\right) - \frac{3}{4}\right] \times ln\left(\frac{1}{1 - 0.84}\right)$$
$$\text{Or, } t = 24.658 \times D^2\left[ln\left(\frac{D}{0.05}\right) - \frac{3}{4}\right] \tag{6}$$

The result of 'D' (Effective diameter of the area covered by each drain) is obtained from this Eq. (6) by adopting the trial and error method, and a calculation using this method is shown in Table 3.

(C)   Other design aspects—

(i)   Soil disturbance:

Table 3 Results of 'D' obtained by trial and error method

D (metre)	t (days)
3.00	$742 > 730$
2.75	$607 < 730$
2.90	$686 \approx 730$

Due to the effects of soil disturbance on the PVD in the field, the value assumed $c_h = c_v$ may not be true always. So, more detailed analyses should be performed in the final design of the PVD-reinforced soil.

(ii) Drain resistance:

In this calculation, drain resistance was not considered due to the relatively short length of the drain and two-way drainage. But, in the final design, it should be calculated very strictly and included.

(iii) Drainage blanket:

In the final design, the effects of the drainage blanket also need to be included.

3 Conclusions

In the past decades, many research have been carried out on PVD-improved ground taking into account several techniques such as numerical or analytical approaches or experimental works, but a comprehensive design method is still remaining out of their research scope, i.e., they do not discuss the design steps in their publications. So, a new researcher or field engineer, at PVD related works/projects, may face some difficulty to run/accomplish the project smoothly if he cannot make out the fundamental design procedures/steps of the PVD design. So, the researchers/planners need a thorough understanding of the subject. This paper explains an example that gives an idea of the preliminary design procedure, and hence, this paper may help them form/strike out a suitable and appropriate method for PVD design.

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