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## Alternative design approach for soft clay improved by PVDs

### Abstract

In this paper the design procedures for multi-stage construction based on the research know-how described by Rujikiatkamjorn and Indraratna (2009) are proposed. The length of a vertical drain, anisotropic soil permeability, and vacuum pressure are considered and a reduction in consolidation time through vacuum preloading is compared to other available methods. Design charts eliminating cumbersome iterative procedures are then developed using the equivalent drain diameter as an independent variable, to obtain the relevant drain spacing. The design examples based on the land reclamation project at the Port of Brisbane for both single and multi-stage construction are also given.

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# Alternative design approach for soft clay improved by PVDs

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## ABSTRACT

In this paper the design procedures for multi-stage construction based on the research know-how described by Rujikiatkamjorn and Indraratna (2009) are proposed. The length of a vertical drain, anisotropic soil permeability, and vacuum pressure are considered and a reduction in consolidation time through vacuum preloading is compared to other available methods. Design charts eliminating cumbersome iterative procedures are then developed using the equivalent drain diameter as an independent variable, to obtain the relevant drain spacing. The design examples based on the land reclamation project at the Port of Brisbane for both single and multi-stage construction are also given.

*Keywords:* consolidation, design charts, smear zone, vertical drains

## 1 INTRODUCTION

The construction of earth structures such as embankments and major highways over soft clay deposits having low bearing capacities, coupled with excessive settlement characteristics, requires proper planning, design and construction, and quality control (Indraratna et al. 2008). Since soft clays have low shear strength, most embankments will have low heights (2-3m) to prevent failure. The minimum required height to eliminate primary settlement and compensate for secondary consolidation is at least 3-4m. To overcome these problems, special construction techniques such as multi-staged construction and/or vacuum-preloading combined with prefabricated vertical drains (PVDs), may be considered in the design. To design a PVD system, the diameter of the influence zone ( $d_e$ ) usually has to be assumed and a few iterations are required to obtain a proper drain diameter ( $d_w$ ) (Indraratna et al. 2012).

In this paper, the design procedures for single stage and multi-stage construction based on the research know-how described by Rujikiatkamjorn and Indraratna (2009) are proposed. The length of a vertical drain, anisotropic soil permeability, and vacuum pressure are considered and a reduction in consolidation time through vacuum preloading is compared to other available methods. Design charts eliminating cumbersome iterative procedures are then developed using the equivalent drain diameter as an independent variable, to obtain the relevant drain spacing. The design examples for multi-stage construction are also given in this paper.

## 2 DEVELOPMENT OF DESIGN CHARTS

Based on Carrillo's approach (1942), the average excess pore pressure ratio considering both vertical and horizontal drainage can be expressed by:

(a) Preloading combined with vacuum application:

$$\frac{\bar{u}_t}{\Delta p} = -\frac{p_0}{\Delta p} + \left(1 + \frac{p_0}{\Delta p}\right) \sum_{m=1}^{\infty} \frac{8}{(2m+1)^2 \pi^2} \exp \left[ -\left(\frac{2m+1}{2}\right)^2 \pi^2 \frac{1}{c_{vh} L^2} + \frac{8}{\mu} \right] T_h \quad (1a)$$

(b) Vacuum application only:

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

where, the relevant dimensionless parameters are given by:  $c_{vh} = c_h/c_v = k_h/k_v$ ,  $L = l/d_e$ ,  $T_h = c_h t/d_e^2$

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

$$U_t = \left(1 - \frac{\Delta p - \bar{u}_t}{\Delta p - \bar{u}_\infty}\right) \quad (2)$$

Substituting Equation (1) into Equation (2) gives:

$$U_t = 1 - \sum_{m=1}^{\infty} \frac{8}{(2m+1)^2 \pi^2} \exp \left[ - \left( \frac{2m+1}{2} \right)^2 \pi^2 \frac{1}{c_{vh} L^2} + \frac{8}{\mu} \right] T_h \quad (3)$$

Equation (3) shows that the total degree of consolidation at any vacuum condition ( $p_0$ ) is uniquely related to the time factor ( $T_h$ ), vertical drain system configuration and the anisotropic permeability of the soil ( $\mu$ ,  $L$  and  $c_{vh}$ ).

In practice most design charts for vertical drains employ dimensionless horizontal time factor - consolidation curves ( $T_h$  vs.  $U_h$ ) to obtain the drain spacing ( $S$ ) as a function of  $n$  (i.e., Barron 1948). Usually, a number of iterations have to be performed to obtain required parameters such as  $n$ . As the availability of the size of PVDs is limited by the manufacturer, Rujikiatkamjorn and Indraratna (2007) established the appropriate design charts using the equivalent drain diameter ( $d_w$ ) as a known variable, in order to determine the drain spacing ( $d_e$  or  $S$ ). Therefore:

Rearranging Equation (1) gives:

$$\gamma = - \frac{8T'_h}{\ln \left( \frac{1-U_t}{u^*} \right)} \quad (4)$$

where,

$$u^* = \sum_{m=1}^{\infty} \frac{8}{(2m+1)^2 \pi^2} \exp \left[ - \left( \frac{2m+1}{2} \right)^2 \pi^2 T_v \right] \quad (5)$$

$$T_v = c_v t / l^2 \quad (6)$$

$$T'_h = c_h t / d_e^2, \text{ and} \quad (7)$$

$$\gamma = n^2 [\ln n + \xi - 0.75] \quad (8a)$$

$$\xi = \left( \frac{k_h}{k_s} - 1 \right) \ln(s) \quad (8b)$$

Figure 1 shows the relationships between  $T_v$  and  $u^*$  as represented by Equation (5). Figure 2 illustrates the contour plot of  $\xi$  (Equation 8) when the values of  $k_h/k_s$  and  $s$  are in the range of 1-8 and 2-8, respectively (Sathananthan et al. 2008). Employing a linear regression analysis (with  $R^2 > 0.99$ ),  $n$  can be arbitrarily expressed by:

$$n = \exp(\alpha \ln \gamma + \beta); \text{ where,} \quad (9a)$$

$$\alpha = 0.3938 - 9.505 \times 10^{-4} \xi^{1.5} + 0.03714 \xi^{0.5} \quad (9b)$$

$$\beta = 0.4203 + 1.456 \times 10^{-3} \xi^2 - 0.5233 \xi^{0.5} \quad (9c)$$

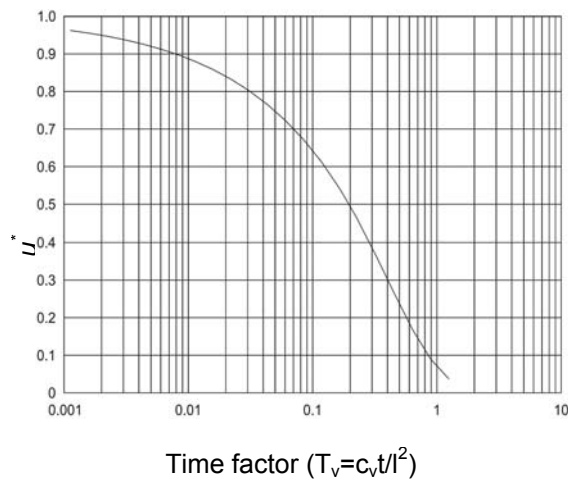


Figure 1. Relationship between  $T_v$  and  $u^*$  (Rujikiatkamjorn and Indraratna, 2007)

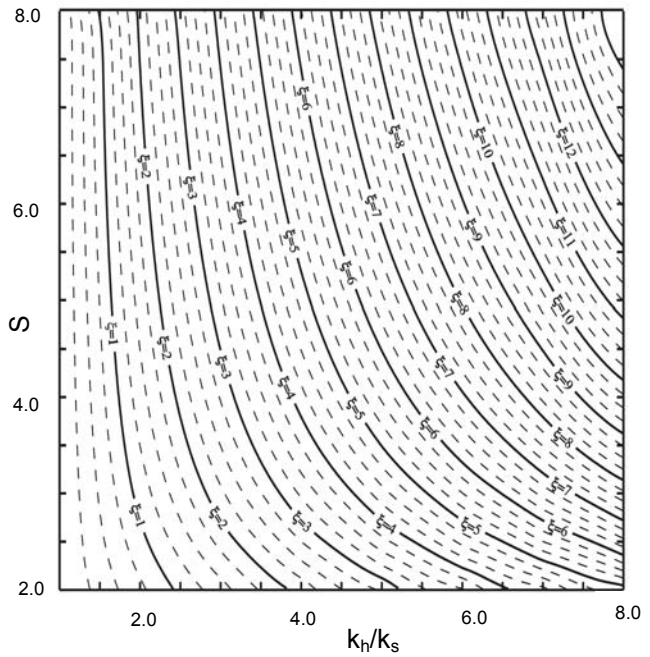


Figure 2. Contour plot of  $\xi$  based on Equation (8b) (Rujikiatkamjorn and Indraratna, 2007)

### 3 INFLUENCE OF DRAIN LENGTH AND ANISOTROPY

Figure 3 shows the comparison of the degree of consolidation when  $L$  varies between 1 and 10 for  $c_{vh} = 1$ . It is evident that when  $L$  increases the radial consolidation becomes important. If  $L$  is more than 10 (i.e.  $l \geq 10d_e$ ), the vertical consolidation can be ignored. Figure 4 shows the comparison of the degree of consolidation when  $c_{vh}$  is from 1 to 10 for  $L = 1$ . As  $c_{vh} = c_h/c_v$  decreases, the influence of vertical consolidation becomes significant, as expected. From the analysis it shows that for very long vertical drains ( $l \geq 10d_e$ ), the effect of vertical consolidation is insignificant, and the anisotropic soil permeability plays a significant role in controlling consolidation.

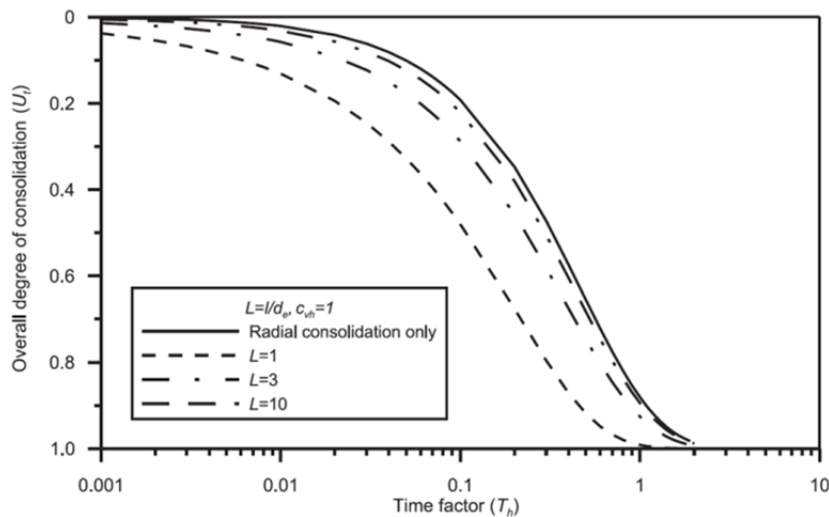


Figure 3. Effect of drain length on the degree of consolidation (Rujikiatkamjorn and Indraratna, 2007)

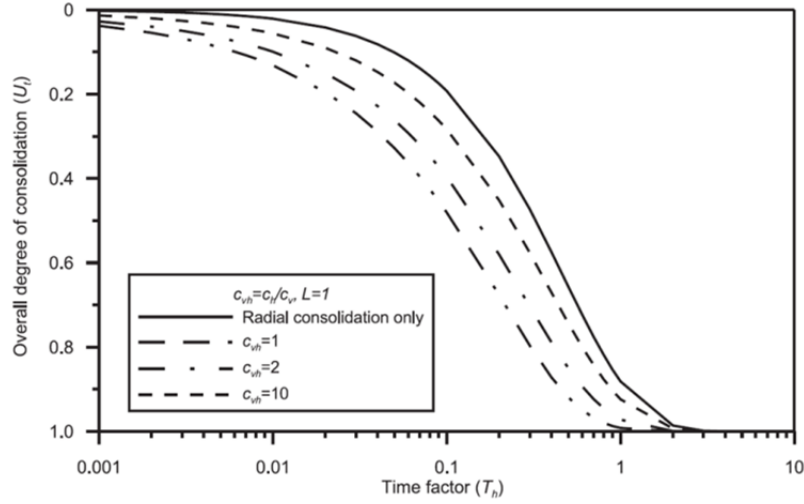


Figure 4. Effect of  $c_{vh}$  on the degree of consolidation (Rujikiatkamjorn and Indraratna, 2007)

#### 4 DESIGN CONSIDERATIONS FOR STAGED EMBANKMENT CONSTRUCTION

The procedures for constructing a staged embankment are as follows:

1. For a given slope and width, the maximum surcharge ( $q_{max}$ ) may be determined by Bishop's limit state theory based on an undrained shear strength analysis (Ladd, 1991). The factor of safety for embankment slope stability should be more than 1.3.
2. The required surcharge load ( $q_{req}$ ) to eliminate primary consolidation due to permanent loading ( $q_f$ ) and to compensate for secondary consolidation during the life time of the permanent structure, can be calculated by: (Ladd, 1991)

$$q_{req} = \sigma'_i 10^{\left( \frac{\left[ C_r \log \left( \frac{p_c}{\sigma'_i} \right) + C_c \log \left( \frac{q_f}{p_c} \right) \right] + C_{\alpha}(1+e_0) \log \left( \frac{t_s}{t_p} \right)}{0.9 C_c} \right)} - \sigma'_i \quad (10)$$

3. If  $q_{max} > q_{req}$ , the single stage construction may be carried out. If the application of vacuum pressure ( $p_0$ ) is available and  $q_{max} > q_{req} - p_0$ , single stage construction can also be executed. If  $q_{max} < q_{req}$  or  $q_{max} < q_{req} - p_0$ , then multi-stage construction is recommended and the design procedures are given below.
4. For the first stage of construction, a maximum surcharge pre-loading to prevent embankment instability ( $q_{max}$ ) can be applied. For a given period of time ( $t$ ), drain spacing can be calculated using the design steps for a single stage loading given in the previous section. The average degree of consolidation at the end of the first stage ( $\bar{U}$ ) should be approximately 70%, as consolidation occurs faster at the beginning.
5. An increase in the average shear strength at the end of the first stage of construction can be determined by the Stress History and Normalised Soil Engineering Properties method, SHANSEP, (Ladd and Foott, 1974). An increase in the undrained shear strength can be estimated as follows:

$$\left( \frac{S_u}{\sigma'_{v0}} \right)_{OC} = \left( \frac{S_u}{\sigma'_{v0}} \right)_{NC} OCR^m \quad (11)$$

For soft Bangkok clay:

$$\left( \frac{S_u}{\sigma'_{v0}} \right)_{NC} = 0.22 \quad (12a)$$

$$m = 0.8 \quad (12b)$$

Substituting Equation (12) into Equation (11), Equation (11) becomes:

$$\left(\frac{S_u}{\sigma'_{v0}}\right)_{OC} = 0.22 OCR^{0.8} \quad (13)$$

The shear strength ( $S_u$ ) at a given effective vertical stress ( $\sigma'$ ) is evaluated by the following equations:

$$\frac{S_u}{S_{ui}} = \left(\frac{\sigma'}{\sigma'_i}\right)^{0.2} \quad \sigma'_i \leq \sigma' \leq p'_c \quad (14)$$

$$\frac{S_u}{S_{ui}} = 0.22 \frac{(\sigma' - p'_c)}{S_{ui}} + \left(\frac{\sigma'}{\sigma'_i}\right)^{0.2} \quad \sigma'_i \leq \sigma' \leq \sigma'_i + q_{req} \quad (15)$$

An increase in the effective vertical stress ( $\Delta\sigma'_v$ ) due to embankment loading can be determined based on elastic solution (Poulos and Davis, 1974), which can be expressed by:

$$\Delta\sigma'_v = q_{max} I_q \bar{U} \quad (16)$$

It can be seen that the stress increments vary from place to place under the embankment. Therefore, the influence factor ( $I_q$ ) would be adopted based on the location below the embankment (Fig. 5). The soil under embankment loading should be divided into 3 zones at least, to determine the stress increment due to consolidation. The factor of influence can be determined by:

$$I_q = \frac{1}{\pi} \left( \beta + \frac{x\alpha}{a} - \frac{y}{R_1^2} (x - b) \right) \quad (17)$$

6. The factor of safety for embankment stability of the second construction stage can be calculated using the initial shear strength plus the shear strength increased during the first stage of consolidation. If the safety factor is less than 1.3 for the required surcharge load  $q_{req}$ , Steps 4-5 should be repeated for additional stage loading.

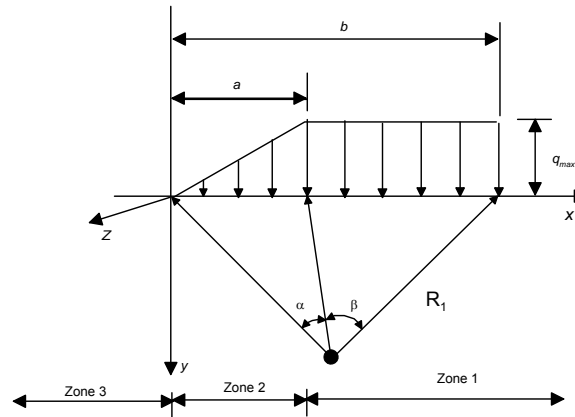


Figure 5. Diagram showing the location and parameters for calculating the factor of influence

## 5 DESIGN EXAMPLE FOR STAGED CONSTRUCTION WITH SURCHARGE ONLY

A 40m wide embankment with a side slope of 2:1 (H:V) was constructed on this site. The permanent load ( $q_f$ ) and design life ( $t_s$ ) time are assumed to be 60 kPa and 30 years, respectively. The drain is 10m long x 100mm diameter x 4mm thick. The values of  $k_r/k_s$  and  $d_s/d_w$  for this case study are assumed to be 2 and 6, respectively. PVDs are installed in a square pattern. The ground water table is assumed to be at the surface and one way drainage is assumed in consolidation without PVDs.

**Step 1.** Maximum surcharge ( $q_{max}$ ) with a 5 kPa machinery live load can be determined using slope stability analysis (Fig. 6). For a safety factor of 1.6,  $q_{max}$  is 45 kPa (2.5m height of surcharge).

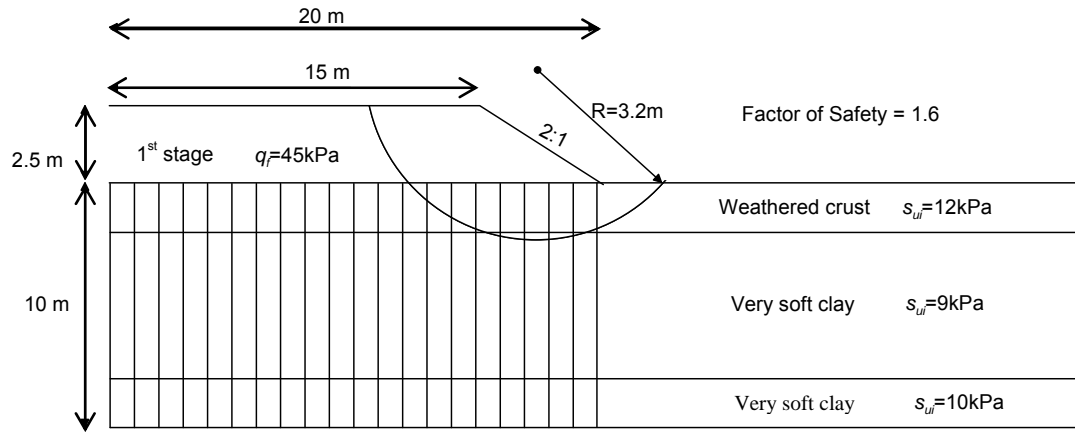


Figure 6. Slope stability analysis for the first stage embankment loading

**Step 2.** The required surcharge load ( $q_{req}$ ) to eliminate primary consolidation due to permanent loading ( $q_f$ ) and compensate for secondary consolidation during the lifetime of this permanent structure can be determined by Equation (10). Therefore, the required surcharge load ( $q_{req}$ ) is 70 kPa (4m surcharge load). The expected settlement is 0.82m.

**Step 3.** It can be seen that  $q_{max} < q_{req}$ , therefore, a multi-stage construction is required. For the first stage the selected height of the embankment based on the stability analysis, is 2.5m (45 kPa). The time required to approach a 70% degree of consolidation is three months. The drain spacing for square pattern installation is determined using the procedures for a single stage loading given in a previous section. The values used for  $k_h/k_s$  and  $d_s/d_w$  for this case study are assumed to be 2 and 6. Using figure 3 value of  $\zeta$  can be obtained as 1.75. Using that value and equations 3-9 a spacing of 1.2m is obtained. Development of these charts are described in detail in section 2. The associated settlements and stress increments at the centreline can be determined by (Table 1):

Table 1: Calculated settlement and stress increment at the centerline

| Soil layer | $R_u$ | $I_q$ | $\Delta\sigma'_v$<br>(kPa)<br>(Eq. 16) | $\rho$<br>(m)      |
|------------|-------|-------|--|--------------------|
| 1          | 0.33  | 0.9   | 27.14                                  | 0.04               |
| 2          | 0.32  | 0.7   | 21.42                                  | 0.25               |
| 3          | 0.30  | 0.6   | 18.90                                  | 0.03               |
|            |       |       |  | Total $\rho=0.32m$ |

**Step 4.** The new soil shear strength after consolidation in stage 1 can be calculated using Equation (14) (Table 2).

Table 2: Shear strength development after stage 1 construction

| Soil layer | $S_{ui}$<br>(kPa) | $p'_c$<br>(kPa) | $\Delta\sigma'_v$<br>(kPa)<br>(Eq. 11) | $\sigma'_{vi}$ at the<br>middle of soil<br>layer<br>(kPa) | $\sigma'_{vf}$ at the<br>middle of soil<br>layer<br>(kPa) | $S_{ui}$ (after<br>consolidation)<br>(kPa)<br>(Eq. 14) |
|------------|-------------------|-----------------|--|---|---|--|
| 1 (zone 1) | 12                | 58              | 27.14                                  | 6   | 33.14   | 16.9   |
| 2 (zone 1) | 9                 | 45              | 21.42                                  | 32  | 53.42   | 11.85  |
| 3 (zone 1) | 10                | 70              | 18.90                                  | 47  | 65.90   | 10.7   |
| 1 (zone 2) | 12                | 58              | 15.08                                  | 6   | 21.08   | 15.4   |
| 2 (zone 2) | 9                 | 45              | 13.77                                  | 32  | 45.77   | 9.67   |
| 3 (zone 2) | 10                | 70              | 12.60                                  | 47  | 59.60   | 10.5   |



**Note:** For Zone 3 (outside the improvement area), soil shear strength is assumed to be the same as the initial soil shear strength.

**Step 5.** Using the shear strength estimated above, the safety factor obtained for the second stage of construction from Bishop's method is equal to 1.5 (Fig. 7). Therefore, no further staged construction is required.

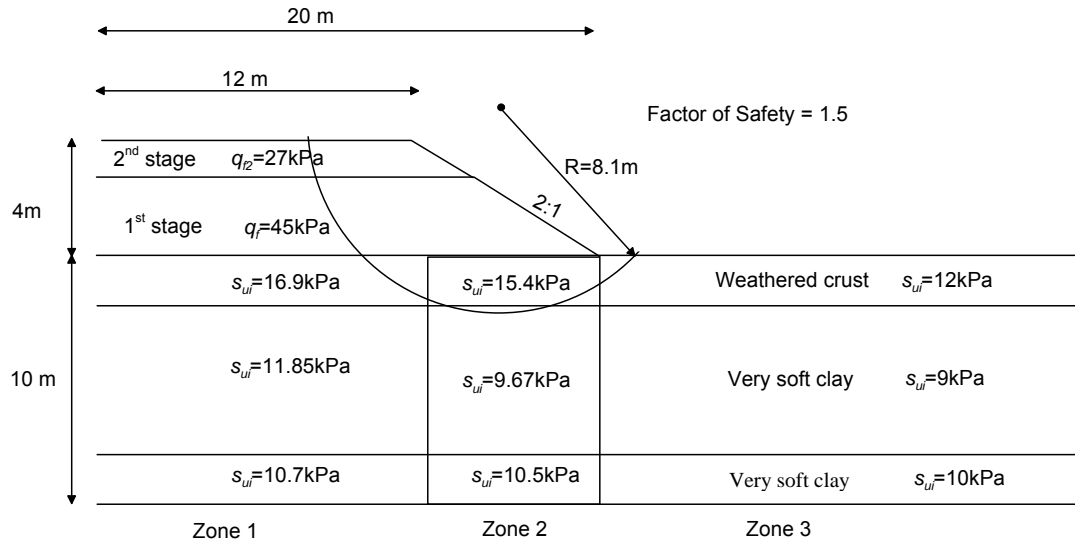


Figure 7. Slope stability analysis for the second stage loading

**Step 6.** The time required for an expected settlement of 0.82m is 5 months. The multi-stage construction and predicted surface settlement are summarised below (Fig. 8).

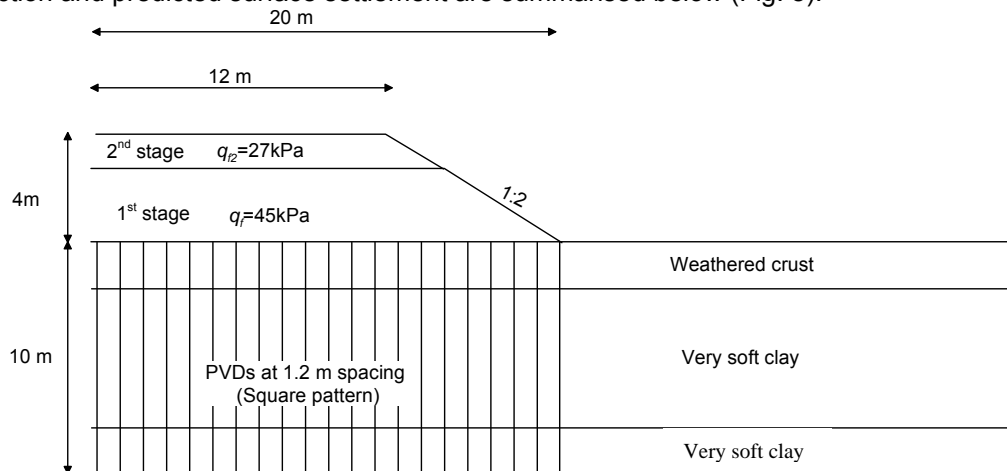


Figure 8. Design cross section of embankment with multistage loading

## 6 CONCLUSION

A system of vertical drains combined with vacuum preloading is an effective method for accelerating soil consolidation. In this chapter suitable design charts for vertical drains that considered both vertical and horizontal drainage, were developed. As a result the conventional and often cumbersome trial and error methods used to estimate the appropriate parameters could be avoided. Once the equivalent drain diameter  $d_w$  and other relevant parameters are known, the influence zone diameter  $d_e$  can be readily obtained without any further iterations or interpolations. A tentative design procedure for single stage and multi-stage embankment construction has been developed to consider the benefits from PVDs and vacuum pressure. Staged construction requires controlled rates of loading to increase the shear strength and stabilise the embankment because of partial consolidation. The time required for multi-staged construction is usually longer than for single stage construction. The spacing of PVDs significantly affects the degree of consolidation at the end of construction. The application of vacuum

pressure can substantially decrease the required height of the embankment and, therefore, the need for staged construction may be eliminated. During construction, field monitoring is required in order to check actual behaviour against design expectations.

## 7 ACKNOWLEDGEMENT

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## NOTATIONS

|             |   |  |
|-------------|---|--|
| $a$         | = | Width of PVDs  |
| $b$         | = | Thickness of PVDs                                    |
| $c_v$       | = | Coefficient of consolidation for horizontal drainage |
| $c_h$       | = | Coefficient of consolidation for vertical drainage   |
| $C_c$       | = | Compression index                                    |
| $C_r$       | = | Recompression index                                  |
| $C_\alpha$  | = | Coefficient of secondary consolidation               |
| $de$        | = | Diameter of influence zone (m)                       |
| $d_s$       | = | Smear zone diameter                                  |
| $d_w$       | = | Equivalent diameter of vertical drain (m)            |
| $k_h / k_s$ | = | Smear zone permeability ratio                        |
| $l$         | = | Length of PVDs                                       |
| $p'_c$      | = | Effective preconsolidation pressure                  |
| $q_f$       | = | Permanent structure loading                          |
| $s_{ui}$    | = | Initial undrained shear strength                     |
| $t_s$       | = | Design life time for the permanent structure         |
| $U_t$       | = | Average degree of consolidation with time            |
| $\phi'$     | = | Shear strength of fill material                      |
| $\gamma_t$  | = | Total unit weight of subsoil and fill material       |
| $\sigma'_i$ | = | Initial effective stress of each sub-soil layer      |

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