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ORIGINAL PAPER

# Improving the Stability of High Embankments Founded on Soft Marine Clay by Utilizing Prefabricated Vertical Drains and Controlling the Pace of Construction

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**Abstract**

In many cities worldwide, the development of sites underlain by thick deposits of soft marine clay has become more common to meet the demand for essential transportation infrastructure arising from the current unprecedented urban population growth. Staged construction and ground improvement techniques are two approaches frequently used in constructing high embankments over soft soils. These approaches improve stability, ensuring that the pace of construction does not cause a build-up of excess pore water pressures and an associated reduction in shear strength, which could lead to progressive failure. This study examines the use of prefabricated vertical drains (PVDs) and the pace and sequence of construction, to determine the most suitable method for constructing high embankments on soft soil deposits. The problem was modelled using a Finite Element software called PLAXIS 2D, in which the PVD is available as pre-coded drainage line elements. The modelling was carried out with a staged construction technique. The analyses considered PVD spacing ranging from 1 to 2.5 m, with PVDs extending to the crest, to the toe, or beyond the toe of the embankment. The fill layer thickness and the consolidation time following the placement of each layer were also varied to study the effect of these parameters on overall stability. The results obtained from this study show that PVDs can significantly improve the stability of embankments when combined with an appropriate consolidation time for a faster construction pace.

**Keywords** Consolidation · Marine clay · Excess pore water pressure · Prefabricated vertical drains · Settlement · Plain strain model

## Introduction

Studying the behaviour of high embankments founded on soft marine soils is a challenging area of soil mechanics, due to the complex problems posed by the weak geotechnical characteristics of the foundation soils. The low strength of marine clays significantly limits the load (and embankment height) that can be applied while maintaining adequate safety for short-term stability. In addition, the high deformability and low permeability of these soils result in excessive settlement and very slow dissipation of excess pore water pressures, which in turn significantly reduces the shear strength. Various constructive techniques are available for designing high embankments on soft soils. Altering the properties of the foundation soil or the fill layers can increase the global stability, accelerate consolidation, and reduce creep settlement. However, the most commonly applied techniques for accelerating the rate of consolidation and reducing the consolidation time are the use of prefabricated vertical drains (PVDs) [1], stone columns without encasement and granular encased columns in the foundation soils. Early work on soft Bangkok clay was carried out by Muktabhant et al. [2], Moh et al. [3], Eide [4, 5], and Bergado et al. [6]. Since saturated marine clays have very low shear strength, rapid embankment construction cannot be carried on such soils. Thus, to increase global stability, embankments are built by using staged construction, with waiting periods between the construction stages to allow time for consolidation to occur and as a result, an increase in the undrained shear strength of clay is observed. With an

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application history of more than three decades, PVDs have proven to be an efficient means of accelerating the rate of consolidation and improving the performance of the soft subsoil. In combination with the use of PVDs, staged construction has become more widespread in the construction of embankments [7].

With increasing populations, the demand for infrastructure development on soft compressible soils is constantly growing, especially in the coastal regions of many countries. Rapid developments often require the use of even the poorest of soft clays. Therefore, it is essential to stabilize existing soft clay foundations prior to and during the construction, in order to avoid excessive or differential settlement. Although a variety of soil improvement techniques are available, the application of prefabricated vertical drains is still a widely used solution [8]. Prefabricated vertical drains (PVDs) provide artificially created drainage, which is used in soft soil deposits to accelerate primary consolidation by shortening drainage paths, so that excess pore water pressures can dissipate radially rather than vertically. The main advantages of drains are that they accelerate the increase in the soil shear strength by consolidation, thus reducing differential settlement during primary consolidation, and they also reduce the depth of surcharge fill required to achieve the desired pre-compression. [9] Of the various vertical drains used to accelerate the dissipation of excess pore water pressures beneath embankments, prefabricated vertical drains are the most cost-effective. In combination with the use of surcharge fill, the radial drainage paths facilitated by PVDs stabilize the soft ground by increasing the soil shear strength and reducing post-construction differential settlement [10].

In 1940, prefabricated band-shaped drains and Kjellman cardboard wick drains were introduced in the field of ground improvement. Several other types of PVD have been developed since then, such as the Geodrain (in Sweden), Alidrain (in England), and Mebradrain (in the Netherlands). PVDs consist of a perforated plastic core functioning as a drain, and a protective sleeve of fibrous material acting as a filter around the core. Vertical drains are generally installed by using either a dynamic or a static method. In the dynamic method, a steel mandrel is driven into the ground by using either a vibrating hammer or a conventional drop hammer. In the static method, a mandrel is pushed into the soil by means of a static force. Although the dynamic method is quicker, it causes more disturbance of the surrounding soil during the installation. This results in shear strain, accompanied by an increase in the total stress and pore water pressure, in addition to the displacement of the soil surrounding the vertical drain [11].

The effectiveness of a PVD system is related to the drain characteristics, including the longitudinal permeability or discharge capacity. A decrease in discharge capacity, referred to as well resistance, retards the dissipation of pore water pressures and the associated settlement. The main causes of well resistance are deterioration of the drain filter (leading to reduction of the drain cross-section), silt intrusion into the filter (resulting in a reduction of pore space) and folding of the drain due to lateral movement [12]. The finite element method (FEM) has been generally adopted to analyze the behaviour of PVD-improved soft subsoils under embankment loading. There are three existing approaches for modelling the PVD-improved subsoil. The first method employs a one-dimensional (1D) drainage element. The second method uses a macro-element in the FEM program to consider the drainage behaviour of vertical drains. The third method is an approximate approach, which estimates an equivalent value for the vertical hydraulic conductivity of PVD-improved subsoil. With this approach, the behaviour of PVD-improved subsoil can be analyzed in a manner similar to that used for non-treated natural subsoil [13].

Hird et al. [14] developed a method for modelling the effects of PVDs in two-dimensional plane strain conditions. The accuracy of the developed method was verified using the field results of three case histories of embankments built on soft soils and improved by PVDs. They reported that a very good match was found between the obtained finite element results and the available field data. Furthermore, Rujikiatkamjorn et al. [15] presented 3D and 2D finite element models of a PVDs case study in the Tianjin Port in China. In the 3D model, the actual geometry of the PVDs and their installation pattern within the soil volume were simulated. Whereas in the 2D model, the appropriateness of the plane strain analysis approach using equivalent permeability and transformed unit cell geometry, similar to the approach adopted in this study, was examined. An acceptable agreement was found to exist between the predictions of the 2D and 3D finite element models and the available field data for the settlements, porewater pressures, and lateral displacements. Similar findings were reported by Indraratna et al. [8] and by Borges [16].

This study makes use of drainage line elements that are pre-coded in PLAXIS 2D. Hence, in the numerical modelling, drain design parameters such as the smear effect, well resistance, and discharge capacity are ignored. A drain line element acts as a macro-element which examines the vertical hydraulic conductivity of the drains by considering all the pore water pressures in the drain nodes to be equal to zero. Thus, the excess pore water pressures are dissipated via the drain element to the top of the soil layer through the difference in pore water pressure between the soil and the drain element.

The numerical models described in this paper are based on the embankment design and soil profile of a case study of an embankment in St. Stephen, New Brunswick, Canada, which failed in 2006 during the construction of a four-lane highway leading to the Canada-USA border crossing in St

Stephen. [17] In the present research, a two-dimensional finite element model of the embankment was prepared by estimating the soil properties via back analysis from the failure results obtained from the case study. After a failure similar to that observed at the site was established in the numerical model, drain line elements were added as a ground improvement technique, to study their effect on the stability of high embankments constructed over soft soil deposits. A parametric study was conducted to investigate the effect of the spacing and extent of coverage of the PVDs; the effect of different fill layer thicknesses and extent of PVD coverage with constant drain spacing; and the effect of different consolidation times for thicker fill layers, with constant drain spacing and extent of coverage of the PVDs.

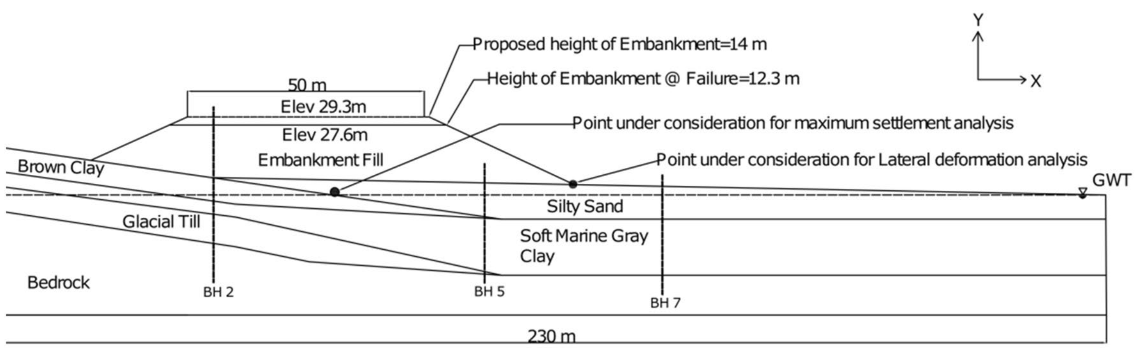
## Methodology

First the soil profile and embankment were modelled in accordance with the cross-section of the embankment from the case study (see Fig. 1). Then drainage line elements were added to model PVDs in the parametric study. This was done to examine the effects of drain spacing and the extent of coverage of the PVDs on the stability of the embankment.

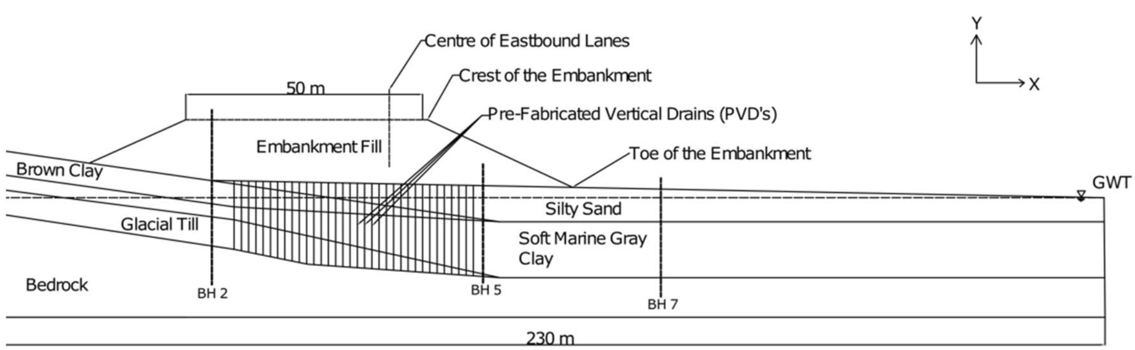
In PLAXIS 2D, when 15-node (fourth-order) soil elements are employed, each drainage line element is defined by five nodes. However, when 6-node (quadratic) soil elements are used, each drainage line element is defined by three nodes [18]. This study makes use of 15-node soil elements. Relationships between the axial force and displacement and the axial flow rate of the PVDs and hydraulic gradient of the soil profile are established in the element stiffness matrix. In the consolidation analyses, the drainage line elements are assumed to have a negligible area cross-section [18].

The drainage line elements in PLAXIS 2D are handled as seepage boundaries and are located inside the domain. At atmospheric pressure, the drains cannot work perfectly and do not permit the discharge of water leaving the domain. Hence, a prescribed head, $\varnothing*$, should be considered for drains below the water level. Thus, the conditions are as follows:

- $\phi = \phi*$; in the case of outflow
- $q.n = q_x n_x + q_y n_y + q_z n_z = 0$; in the case of suctionwhere $n_x$, $n_y$ and $n_z$ are the outward-pointing fast vector components on the boundary, and $q$ is the discharge. Here the drain itself does not generate a resistance against flow [18].




**(a)**




**(b)**

**Fig. 1** Site soil profile cross-section with boreholes (BH), showing **a** the proposed height of the embankment and the height at failure; points under consideration for numerical analysis, and **b** PVDs with a spacing of 1.5 m c/c

The drawback of using drainage line elements is that they do not simulate the exact behaviour of drains which would be used at the site, since they do not account for the well resistance, discharge capacity, smear zone effects, or clogging of the drains due to the migration of soil particles during installation. However, from work done by Wong et al. [18] in 2013, it can be concluded that the drainage line elements, although simplistic, can be used to study the effects of using drains on the stability of embankments on soft soils. However, they should not be used to design or model drains that are to be implemented in a specific project. This is because the drains modelled as line elements make an assumption that the pore pressures are equal to 0 throughout its cross section, which might not be practical, does not account for clogging and the properties of the drains. It can also be noted that by the use of these line elements, the design parameters are not modelled and hence line elements could only be used to study the effectiveness of the drains for a particular problem.

## Site Conditions and Geology

The subsurface profile and geology of the site were studied following the embankment failure. Standard penetration tests (SPTs) were conducted after the failure, immediately prior to reconstruction of the embankment. Based on the SPT corrected values for overburden pressure ($N^1{}_{60}$), a generalized soil profile, shown in Fig. 1, was developed via back analysis. SPT tests were conducted at seven boreholes on the site. The locations of boreholes 2, 5, and 7 are indicated in Fig. 1.

At borehole (BH) 2, a layer of fill 8 m thick was underlain by a very thin layer of topsoil (approximately 0.1 m), resting on a very soft gray silty lean clay layer. Under this was a layer of dense gray silty sand 0.7 m thick, underlain by a layer of till 4.6 m thick. Bedrock was encountered at about 19 m below ground level. At borehole 5, a layer of fill 3.5 m thick was underlain by 4.1 m of compact to dense brown silty sand with gravel, which rested on a very soft gray lean clay layer. Bedrock was encountered at approximately 18.5 m below ground level. At borehole 7, 3.7 m of very loose to compact brown clayey sandy silt with gravel was underlain by a thin layer (0.6 m) of soft sandy clay, resting on a layer of very soft gray clay 10.6 m thick. This was underlain by another thin layer (0.7 m) of compacted silty clayey sand with gravel, with bedrock located at a depth of 15.6 m beneath the ground level. The water content, $W_C$, of the soft clay layer ranged from 30 to 35%; with a plastic limit, $W_P$, of 22% and a liquid limit, $W_L$, of 45%. The vertical coefficient of consolidation, $C_V$, was estimated to be 0.010 cm$^2$/min, and the horizontal coefficient of consolidation, $C_H$, was estimated to be 0.012 cm$^2$/min [19].

### Soil Properties

Based on the SPT values, empirical relations were used to determine the strength parameters of the different types of soil at the site. Empirical relations [20] were used to calculate the undrained cohesion ($S_U$) of the soft gray clay layer. From the water content of 30% to 35% and the plasticity index (*PI*) of 22%, other parameters of the soft soil, such as $C_C$ and $C_S$, which determine the stiffness during loading and unloading, were obtained. The stiffness of the other soils was determined by using relations based on the average SPT values. The marine clay layer was modelled by using the soft soil (SS) model in PLAXIS 2D, and the other soils were modelled by using the hardening soil (HS) model. A detailed description of the soil models considered is provided in the following section. Table 1 summarizes the soil properties used in the numerical model, which were determined from the average

**Table 1** Summary of soil properties used in the numerical model

| Soil layers | Soil model | Unit weight (kN/m$^3$) | | Stiffness para-meters (MPa) | | | Poisson's ratio | Cohesion (kPa) | Friction angle (°) | Dilation angle (°) | Hydraulic conductivity (m/day) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | $\gamma_{dry}$ | $\gamma_{sat}$ | $E_{50}^{ref}$ | $E_{oed}^{ref}$ | $E_{ur}^{ref}$ | | | | | |
| Fill | HS | 19.5 | 22 | 50 | 40 | 150 | 0.2 | 1 | 38 | 8 | $8.65 \times 10^{-1}$ |
| Soft gray marine clay | SS | 13.8 | 16.1 | – * | | | 0.495 | 0 | 28 | 0 | $8.65 \times 10^{-5}$ |
| Brown clayey silty sand | HS | 16 | 20 | 30 | 24 | 90 | 0.2 | 5 | 29 | 0 | $8.65 \times 10^{-4}$ |
| Silty sand | HS | 16.5 | 20 | 40 | 32 | 120 | 0.2 | 0 | 31 | 1 | $8.65 \times 10^{-3}$ |
| Glacial till | HS | 17 | 20 | 40 | 32 | 120 | 0.2 | 0 | 34 | 4 | $8.65 \times 10^{-2}$ |
| Bedrock | Linear non-porous | 28 | – | $E = 6200$ | | | 0.15 | – | – | – | – |

$C_s = 0.035$ (Stiffness parameters used to model the soft marine clay layer)

*$C_c = 0.35$

SPT values, based on empirical relations. The hydraulic conductivities were determined by using estimates of $C_V$ and $C_H$ obtained from the site which is reported in the case study. The three stiffness parameters used in the hardening soil model are explained in Sects. "Soft Soil Model" and "Hardening Soil Model". The mechanical properties of the PVDs used as a ground improvement measure (with 1.5 m spacing in a triangular pattern) are shown in Table 2, as reported by Bernie et al. [19] The marine clay strength parameters for the given water content and plasticity index are estimated in accordance with Zhongkung et al. [21] and Myint et al. [22]

The empirical relations [20] which are used to model the stiffness parameters are as follows:

$$C_c = 0.009\left(W_L - 10\right); \quad I_p/74\,(\text{Approx.}) \tag{1}$$

$$C_s = I_p/370\,(\text{Approx.}), \tag{2}$$

where $W_L$ is the Liquid limit of the soil sample and $I_p$ is the Plasticity Index of the soil sample. As these parameters were available from the case study considered, these were used to model the stiffness parameters of the soft marine gray clay. On knowing the SPT values from the case study, the stiffness parameters of the remaining soil layers were calculated using the following relations [23]:

$$\text{For NC Sands}: E(\text{kPa}) = 490\left(N_{\text{corr}} + 15\right) \tag{3}$$

$$\text{For Saturated Sands}: E(\text{kPa}) = 245\left(N_{\text{corr}} + 15\right) \tag{4}$$

$$\text{For OC Sands}: E(\text{kPa}) = 735\left(N_{corr} + 24\right) \tag{5}$$

$$\text{Silty Sands}: E\,(\text{kPa}) = 300\left(N_{\text{corr}} + 6\right) \tag{6}$$

$$\begin{aligned} \text{Sand + Gravel}: E(\text{kPa}) &= 1200\,(N_{\text{corr}} + 6; \textit{for}\, N_{\text{corr}} > 15) \\ E\,(\text{kPa}) &= 490\left(N_{\text{corr}} + 15\right); N_{\text{corr}} < 15 \end{aligned} \tag{7}$$

where $E$ = Stiffness of soil, $N_{\text{corr}}$ = Corrected SPT Values.

**Table 2** Summary of properties of PVDs used at the site. [16]

| Properties | Test method* | Units | Value |
|---|---|---|---|
| Weight | ASTM D1777 [24] | g/m | 75 |
| Width | – | mm | 95 |
| Thickness | ASTM D5199 [25] | mm | 3.0 |
| Mass of filter | ASTM D1777 [24] | g/m$^2$ | 140 |
| Equivalent diameter of core | – | mm | 65 |
| Discharge capacity:<br>at 10 kPa<br>at 300 kPa | ASTM D4716 [26] | m$^3$/s | $100\times10^{-6}$<br>$50\times10^{-6}$ |
| Pore size opening | ASTM D4751 [27] | mm | 0.075 |
| Permeability | ASTM D4491 [28] | cm/s | 0.02 |
| Permittivity | ASTM D4491 [28] | s$^{-1}$ | 0.3 |

*References for these standards are listed at the end of references section

## Numerical Modelling

Finite element analyses were carried out via PLAXIS 2D for the assumed plain strain problem. The model geometry was constructed from the cross-section of the soil profile shown in Figs. 1, 2. The groundwater table is assumed to be at the ground level and is assumed to remain constant in the upward sloping soil profile. Since the soil profile is not symmetric, the entire profile was modelled and analyzed. Each model consists of a construction phase modelled as a plastic calculation, and a consolidation phase modelled as a consolidation type calculation before running the analysis in PLAXIS 2D. The initial models involved modelling the fill being placed in layers with a thickness of 0.6 m, with PVD spacings of 1 m, 1.5 m, 2 m, and 2.5 m, and with PVDs extending to the crest, to the toe, or beyond the toe of the embankment. In subsequent models, fill layer thicknesses of 2 m, 3.5 m, and 4.5 m were used with a constant PVD spacing, and the time allowed for consolidation after the placement of each fill layer was set at 5 days, 10 days, or 15 days, in order to study the effect of the pace of construction on the stability of the embankment. A detailed explanation of the numerical models is given in Sect. "Development of the Finite Element Model".

### Geometry

Figure 3 shows the geometry adopted for the problem. The soil profile consists of a brown clayey silty sand layer and a silty sand layer resting on a marine clay layer 15 m thick, underlain by glacial till and bedrock. The side slope of the profile shown on the right-hand side of Fig. 3 is assumed to be 1 V:2.14H and the side slope of the profile shown on the left-hand side of Fig. 3 is assumed to be 1 V:1.43H, in accordance with the dimensions observed in the case study. The width of the crest of the embankment is 50 m, and it is assumed that the soil profile extends an additional 70 m on the right-hand side of Fig. 3, representing a total width of 150 m. Figure 3 also shows the various soil layers in the soil profile at the site. The soil profile in the numerical model is assumed to be isotropic. The marine clay is modelled in an undrained condition by the soft soil model, and the brown clayey sandy silt and other soils (except for the bedrock) are modelled in a drained condition by the hardening soil

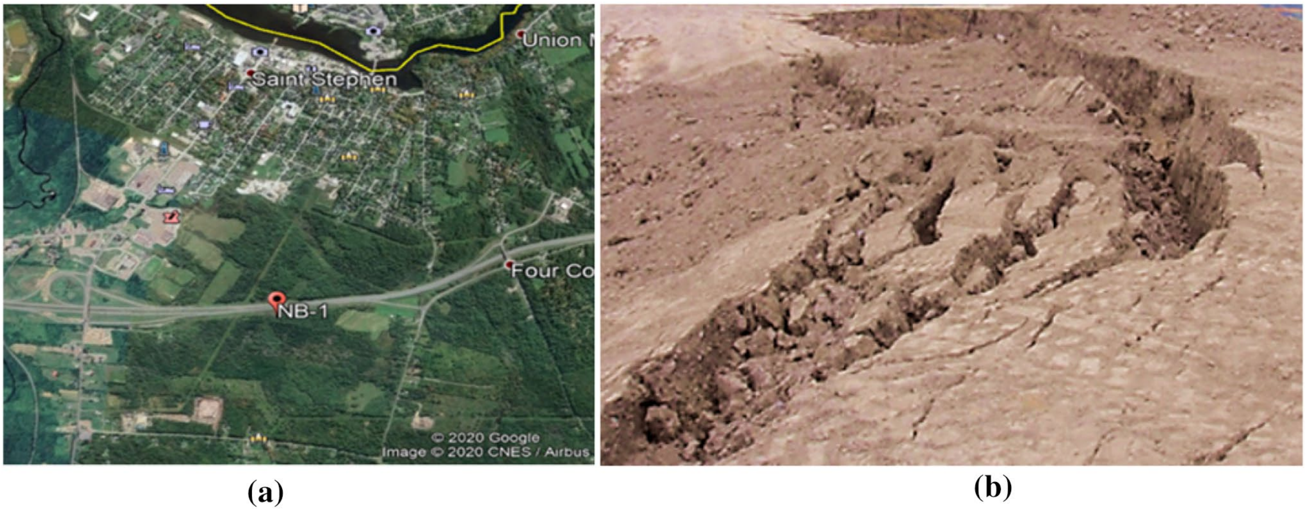



**Fig. 2** **a** Section of route 1 divided highway leading to the Canada–USA border crossing at St. Stephen, New Brunswick, and **b** embankment failure at the site (west view)

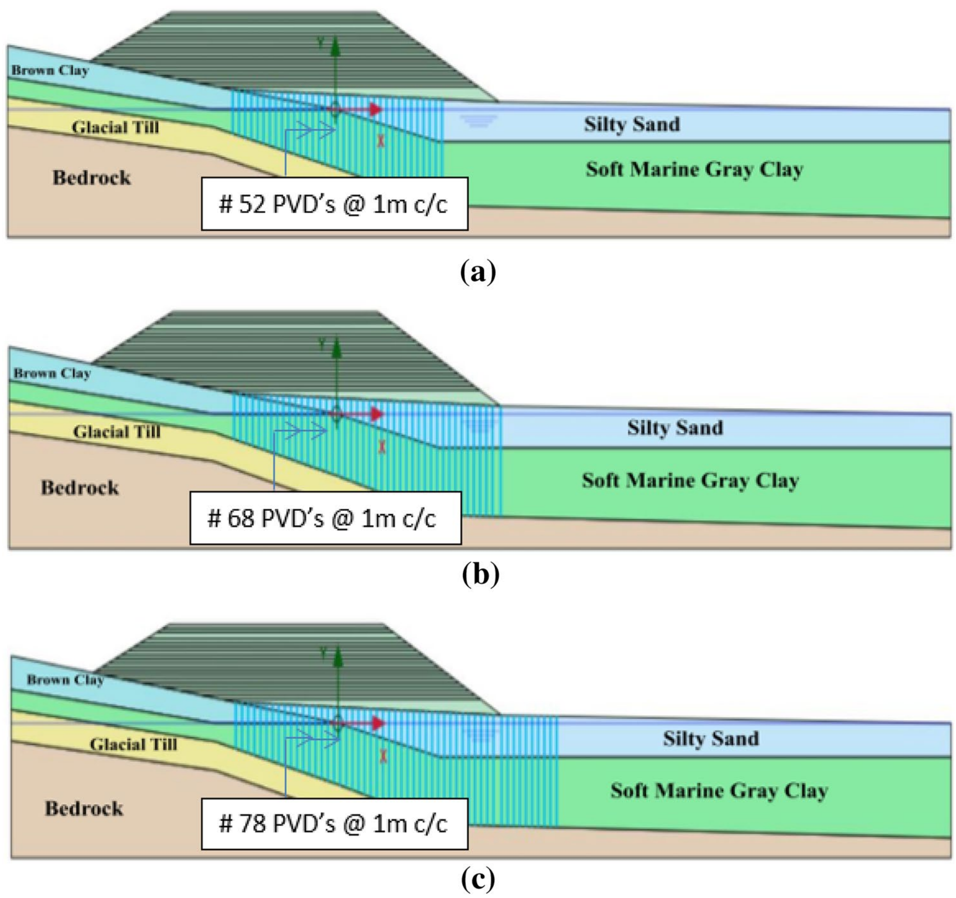



**Fig. 3** Models with PVDs extending **a** to the crest of the embankment, **b** to the toe of the embankment, and **c** beyond the toe of embankment

model. In the mesh, 15-noded triangular elements are used, with 3 degrees of freedom for each node [29]. In the numerical models, the PVDs are considered with various configurations of drain spacing and extent of coverage of the PVDs. Figure 3 illustrates three different extents of PVD coverage that are considered in this study.

## Meshing

A very fine mesh is used for all the numerical models in this study. A coarseness factor of 0.045 is used in the soil region between the drains and the drain line elements. A coarseness factor of 0.1 is used at the interface of the first fill layer and the upper soil profile in order to avoid any errors resulting from the meshing of small soil elements.

Figure 4 illustrates the different spacings of the PVDs, with PVDs extending to the crest of the embankment. With a drain spacing of 1 m center-to-center (c/c) there were 45,536 elements, with a spacing of 1.5 m c/c there were 47,846 elements, with a spacing of 2 m c/c there were 49,636 elements, and with a spacing of 2.5 m c/c there were 51,620 elements. With PVDs extending only to the crest of the embankment, for a drain spacing of 1 m, the number of elements was around 45,000; while with PVDs extending beyond the toe of the embankment, for a drain spacing of 2.5 mm, the number of elements was approximately 67,000.

In the models, it is assumed that the groundwater can drain freely in all directions except at the bottom, which is fixed, since the bedrock is non-porous. For the deformation boundary conditions, a normally fixed position is assumed for the sides of the model, the bottom is assumed to be fully fixed, and the top is assumed to be free. For the consolidation phase, a fully fixed position is assumed for all the boundaries except the top boundary, where deformation is allowed to occur freely.

## Soil Models Used in Numerical Modelling

In this study, the soil models used for numerical modelling are the soft soil model (SSM) and the hardening soil model (HSM), described below.

### Soft Soil Model

To model soft soils such as rapidly consolidated clay, peat, and clayey silts, the non-linearity of these soils must be considered. It is considered via stress-dependent stiffness. In the soft soil model, as shown in Eq. (1), the total strain ($\varepsilon_T$) is composed of an elastic component ($\varepsilon^e$) and a viscoplastic component ($\varepsilon^p$), and is based on the Mohr–Coulomb failure criterion. The modified compression index ($\lambda^*$) and the modified swelling index ($k^*$) are key model inputs, which define the compressibility of the soil in isotropic loading, and in unloading and subsequent reloading, respectively. Alternatively, if $C_c$ and $C_s$ are defined, the software can use these parameters to calculate $\lambda^*$ and $k^*$. The yield surface of the soft soil model is similar to that of the Mohr–Coulomb model, except that in the soft soil model the yield function defines an ellipse, where $M$ is the height of the ellipse, and the isotropic pre-consolidation

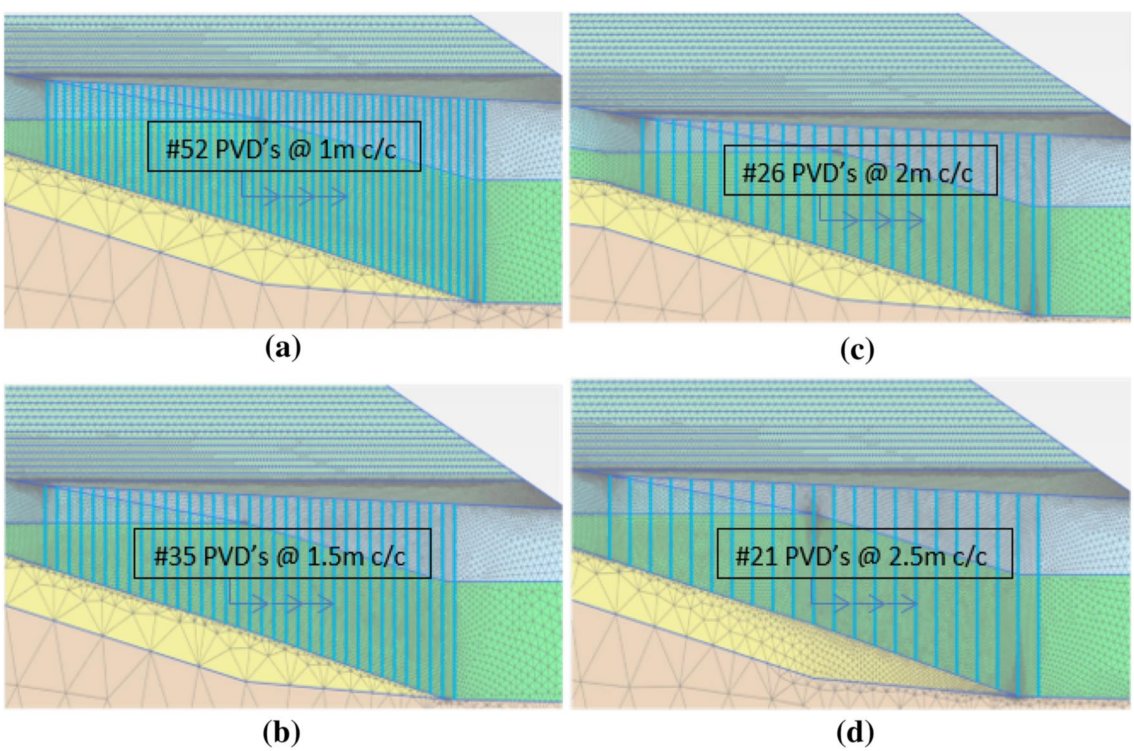



**Fig. 4** Mesh with a PVD spacing of **a** 1 m, **b** 1.5 m, **c** 2.0 m, and **d** 2.5 m

pressure, $P_p$, defines the length of the ellipse in the $q$ x $p'$ plane. Stress changes below the yield cap induce the development of elastic strains. Once the yield stress is exceeded, plastic strains are developed. The mean yield stress is updated based on the amount of accumulated plastic strain. [29] The height of the ellipse, $M$, also represents the ratio of horizontal to vertical stresses in primary loading and is, therefore, used to determine $K_o^{nc}$, where $K_o^{nc}$ represents the earth pressure at rest. The top of all the ellipses coincides with the $M$ line, and the $M$ line has a slope greater than the line defined by Mohr–Coulomb yield line. Thus, the $M$ line is referred to as the critical line. The pre-consolidation pressure decreases with compaction. It is assumed that at the initial stage there is no plastic volumetric strain, corresponding to $P_{po}$, where $P_{po}$ is the pre-consolidation pressure during the initial stage.

The total strain has components of elastic and plastic strain, as shown in Eq. (8) [30]:

$$\varepsilon_T = \varepsilon^e + \varepsilon^p, \tag{8}$$

where $\varepsilon^e$ is the elastic component, $\varepsilon^p$ is the plastic component, and $\varepsilon_T$ is the total strain.

### Hardening Soil Model

The hardening soil model accounts for soil nonlinearity via the stress-dependency of stiffness but does not account for long-term conditions such as creep and stress relaxation. The hardening soil model employs the theory of plasticity rather than the elastic perfectly plastic theory used in the Mohr–Coulomb model, and hence accounts for stress hardening of the soil. Unlike the soft soil model, which can be used to model only soft soils, the hardening soil model can be used to model different types of soil, including soft soils and granular soils. The stress-dependency of soil stiffness is defined by the power $m$, where $m = 1$ is typical for soft soils. The model requires the input of three stiffness parameters at a chosen reference pressure ($P_{ref}$): $E_{50}{}^{ref}$ (the secant modulus/triaxial modulus), $E_{oed}^{ref}$ (the oedometric modulus), and $E_{ur}^{ref}$ (the unloading–reloading modulus). The yield surface consists of a parabolic curve, and soil dilatancy and a yield cap are considered. The position of the shear hardening yield surface is determined mainly by the triaxial modulus, while the position of the yield cap (associated with compression hardening) is primarily determined by the oedometric modulus [30].

For soft soils (where $m = 1$) [31], the stiffness modulus can alternatively be entered in terms of the compression index ($C_c$) which is related to the stiffness moduli as shown in Eqs. (9), (10), and (11) [30], where $\nu$ is Poisson's ratio, $e_o$ is the initial void ratio, and $P_{ref}$ is the reference pressure.

$$E_{oed}^{ref} = 2.3\left(1 + e_o\right)\frac{P_{ref}}{C_c} \tag{9}$$

$$E_{ur}^{ref} = 2.3\left(1 + e_o\right)\frac{(1+\nu)(1-2\nu)P_{ref}}{(1-\nu)C_c} \tag{10}$$

$$E_{50}^{ref} = 1.25E_{oed}^{ref}, \tag{11}$$

where $E_{50}^{ref}$ $E_{50}{}^{ref}$ is the stiffness modulus for primary loading in the drained triaxial shear test, $E_{ur}^{ref}$ is the stiffness modulus for unloading and reloading in drained triaxial shear test, and $E_{oed}^{ref}$ is the stiffness modulus for primary loading in the oedometer test. In the present study, it is assumed that $E_{oed}^{ref}$ = 0.8 $E_{50}^{ref}$.

For a more detailed explanation of the soil models, please refer to the PLAXIS material models manual [31].

## Development of the Finite Element Model and Objectives of the Parametric Study

### Development of the Finite Element Model

The construction sequence of the embankment built at St. Stephens is modelled as a staged construction procedure, which is analyzed in PLAXIS 2D. The construction of the embankment was begun in 2005 and preceded until failure of the embankment in July 2006. In 2005, the embankment was constructed to a height of 5 m in 90 days. Construction then stopped for the winter, which allowed the embankment to consolidate for 270 days. Construction of the embankment resumed on June 11, 2006, and continued until July 4, 2006, when the embankment failed at a height of 12.3 m. In this study, the soil parameters of the underlying marine clay layer were varied in the model, until a deep-seated circular slip failure was replicated at an embankment height of 12.3 m, as was observed at the St. Stephens site.

- For the control case, first the soil profile was modelled, and then the embankment construction sequence was modelled using a staged construction technique with a fill layer thickness of 0.6 m. This case was done primarily to calibrate the model for further parametric studies. A consolidation period of 15 days was allowed after the placement of each fill layer until the embankment reached a height of 5 m. To avoid errors due to meshing of smaller elements, the first fill layer was assumed to be 2.4 m for all the models. For the subsequent construction, the consolidation period allowed after the placement of each fill layer was 1 day until the embankment reached a height of 12.3 m, which was when failure occurred. The failure was replicated as observed at the site. In the control case, no drains were used, since the purpose of the control case was primarily to calibrate the model in accordance with the failure that was observed at the site.

- For the case with a normal pace of construction, the thickness of the fill layer was set to 0.6 m. Following the control case, this was done to study the effects of using PVDs on the overall stability of embankments. The soil profile was modified with PVDs modelled via drainage line elements in PLAXIS 2D, with variations in the drain spacing and the extent of PVD coverage. Drain spacings of 1 m, 1.5 m, 2 m, and 2.5 m were used, with PVDs extending to the crest, to the toe, or 14 m beyond the toe of the embankment. The distance of 14 m beyond the toe of the embankment was chosen, because the distance between the crest and the toe of the embankment was also roughly equal to 14 m. The consolidation period following the placement of each fill layer was considered to be 1 day for the purpose of this study.
- For the case with a fast pace of construction, fill layer thicknesses of 2 m, 3.5 m, and 4.5 m were used, with a constant drain spacing of 1.5 m. This was done to study the effects of construction sequence when used in conjunction with PVD's on the overall stability of the embankment. The PVDs extended to the crest, to the toe, or to 14 m beyond the toe of the embankment. A drain spacing of 1.5 m was selected, because findings from the case with a normal pace of construction indicated that in terms of economy and safety, this spacing was the most suitable (see Sect. "Case with Normal Pace of Construction"). The consolidation time following the placement of each fill layer was 1 day, for the purpose of this study. This case was designed to investigate the effect of different fill layer thicknesses combined with the use of PVDs, as well as effect of varying the extent of PVD coverage.
- For the case with construction allowing more time for consolidation, fill layer thicknesses of 2 m, 3.5 m, and 4.5 m were used, with a constant drain spacing of 1.5 m. This case was done to primarily study the effects of consolidation period followed by a faster construction pace along with the use of PVD's on the overall stability of the embankment. In this case, the PVDs extended to the toe of the embankment. This extent of PVD coverage was selected, because findings from the case with a fast pace of construction indicated that in terms of economy and safety, this extent of coverage was the most suitable (see Sect. "Case with Fast Pace of Construction"). Consolidation times of 5 days, 10 days, and 15 days were used. This case was designed to investigate the effect of different consolidation times on the stability of embankments constructed over soft soils, with a constant drain spacing and constant extent of coverage by the PVDs.

### Objectives of the Parametric Study

A parametric study was conducted which varied the drain spacing, the extent of the area covered by the PVDs, the fill layer thickness, and the consolidation time. There are few reports in the literature showing the effects of these parameters, although these are some of the essential factors that need to be prioritised to optimise safety, costs, and construction time, in the design of embankments constructed over soft soils. The three cases considered in the parametric study can be summarized as follows:

- Case with normal pace of construction: Fill layer thickness of 0.6 m; drain spacings of 1.0 m, 1.5 m, 2.0 m, and 2.5 m; PVDs extending to the crest, to the toe, and beyond the toe of the embankment; and consolidation time of 1 day.
- Case with fast pace of construction: Fill layer thicknesses of 2.0 m, 3.5 m, and 4.5 m; constant drain spacing of 1.5 m; PVDs extending to the crest, to the toe, and beyond the toe of the embankment; and consolidation time of 1 day.
- Case with construction allowing more time for consolidation: Fill layer thicknesses of 2.0 m, 3.5 m, and 4.5 m; constant drain spacing of 1.5 m; PVDs extending to the toe of the embankment; and consolidation times of 5 days, 10 days, and 15 days.

## Results and Discussion

This paper focuses primarily on the embankment design parameters of factor of safety (FS), settlement, the excess pore water pressures generated, and the extent of lateral and vertical deformation. A crucial aspect of embankment design is global stability, to prevent failure and excessive deformation. Thus, the results obtained from the models in the parametric study are used to study the effects of using PVDs, the drain spacing, the extent of the area covered by the PVDs, and the pace of construction on the stability of high embankments built on soft soils. Because it was necessary to calibrate the numerical models to replicate the failure observed in the field, the control model calibrated from the construction sequence used in the St. Stephens case study is considered first, as the control case. Gravity loading is used for the initial calculation in all the numerical models, because the layers of soil are not horizontal. Each construction phase, involving the placement of a fill layer, is assumed to have duration of 1 day as a plastic calculation phase. The installation of PVDs is assumed to have duration of 2 days as a plastic calculation phase. The results are discussed for the control case, the case with a normal pace of construction, the case with a fast pace of construction, and the case allowing more time for consolidation.

## Control Case

From Fig. 5a, it can be seen that with a consolidation period of one day following the placement of each fill layer, the factor of safety (FS) decreases as the embankment height increases. The results are presented for embankment heights of 7.2 m, 9.6 m, and 12.3 m above ground level. The embankment height of 12.3 m is the height immediately before failure. The FS at an embankment height of 7.2 m is 1.681, and the FS immediately before failure is 1.034. Thus, the stability of the embankment decreased as the height of the embankment increased, and it was not possible to complete construction of the embankment to the full design height, due to the loss of effective strength in the foundation soil.

Figure 5b shows the change in settlement of the embankment as the embankment height increases. All settlements were calculated beneath the embankment where maximum settlement was estimated as shown in Fig. 1. At an embankment height of 7.2 m, the settlement is 267.3 mm, and immediately before failure the settlement is 427.1 mm. Settlement is one of the most important parameters to be considered in the design of embankments. The greater the settlement during the construction period, the more strength is gained by the soft soil during the consolidation process, since faster settlement in the initial period indicates a rapid dissipation of excess pore water pressures. In the control case, there was a 9-month break in construction during the winter. The subsequent construction phase that began in the spring of 2006 was accelerated, with a consolidation period of only one day following the placement of each fill layer. This resulted in insufficient settlement and a reduced gain in soil strength, with little dissipation of excess pore water pressures. This eventually led to failure of the embankment at a height of 12.3 m, due to shear strength failure. Thus, the consolidation time plays a key role in determining the overall stability of embankments constructed over soft soils.

Figure 5c shows that there is a decrease in the percentage of excess pore water pressure during consolidation. This parameter represents the increase in excess pore water pressure in comparison to the hydrostatic pore water pressure at a reference point which was 6.675 m beneath the centre of the embankment. This point is considered to be around the mid-point of the marine clay layer where there was the maximum concentration of excess pore pressures. The hydrostatic pressure at this point was 65.48 kPa, and the increase in excess pore water pressure at the

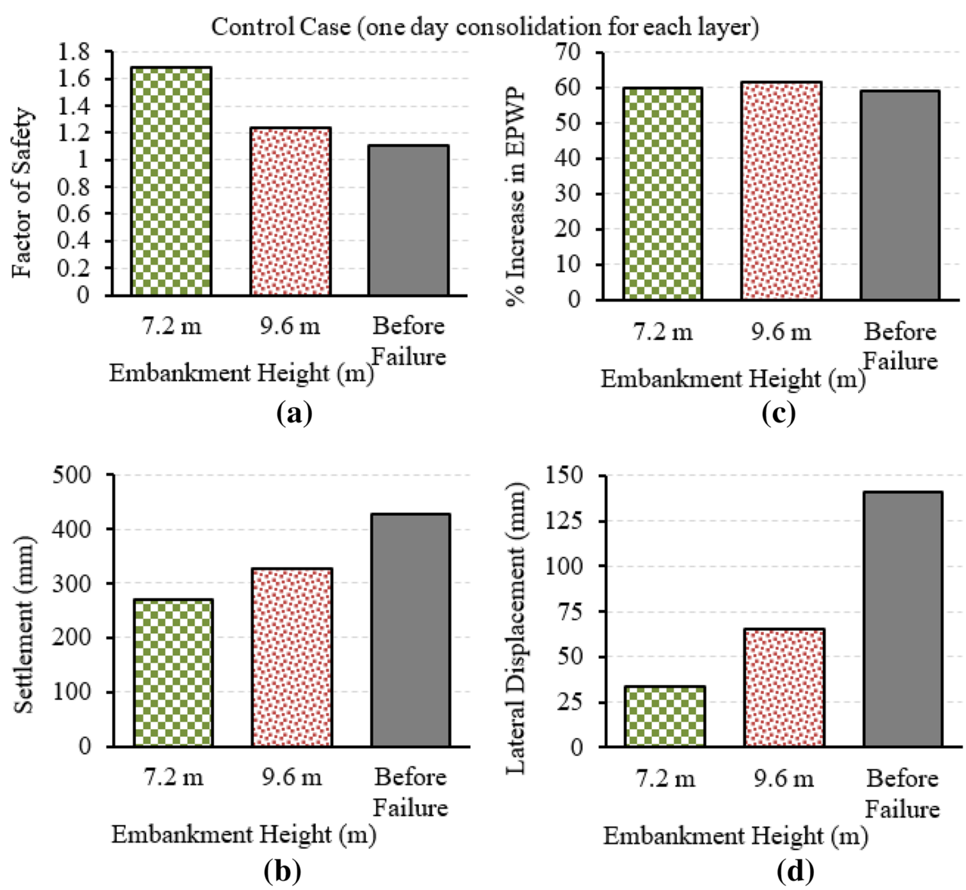



**Fig. 5** **a** Factor of safety, **b** settlement values, **c** % Increase in EPWP and **d** lateral displacement, in the control case, at embankment heights of 7.2 m, 9.6 m, and 12.3 m (immediately before failure)

embankment height under consideration is expressed as a percentage. For the control case model, when the embankment was around 7.2 m, the increase in excess pore pressure is 62.6%, and just before failure it is 59.3%. Since the consolidation period between construction stages was modelled for only one day, there is little dissipation of the excess pore pressure. Hence, there is minimal variation in the excess pore pressure during the consolidation period. Since there is an accumulation of excess pore pressure, stability is reduced significantly, as the effective strength decreases. Hence, the time allowed for consolidation must be increased to prevent the development of excess pore water pressure.

Figure 5d shows that lateral displacement increases as the height of the embankment increases. All the lateral displacements were calculated at the toe of the embankment as shown in Fig. 1. It was estimated that one of the causes of failure at the study site was the lateral deformation of the toe of the embankment. Hence, this is also an important parameter to be considered in the design of embankments. During the construction and consolidation phases, deformation of the toe of the embankment occurs due to the high deformability of the soft soils, which means that this factor must always be taken into consideration in the design of such embankments. Because the effects of lateral deformation have not been extensively investigated in the literature, this paper attempts to relate the lateral deformation to the stability of the embankment.

## Case with Normal Pace of Construction

This case studies the effect of PVD spacing and the extent of the area covered by the PVDs on the stability of the embankment, for the particular problem considered in this paper. Since the focus is primarily on the spacing and extent of coverage of the PVDs, this case does not include the situation with no drains. The situation with no drains is considered in the following case. This case examines the effect of drain spacing on the stability of the embankment when the PVD coverage extends to the crest, to the toe, or beyond the toe of the embankment. The consolidation time following the placement of each fill layer is assumed to have duration of 1 day.

### Factor of Safety

Figure 6 shows that the factor of safety increases as the area covered by the drains increases, and decreases as the spacing between the drains increases. When the area covered by the drains extends to the crest of the embankment, with a drain spacing of 1 m there is a maximum FS of 1.897, and with a drain spacing of 2.5 m the embankment

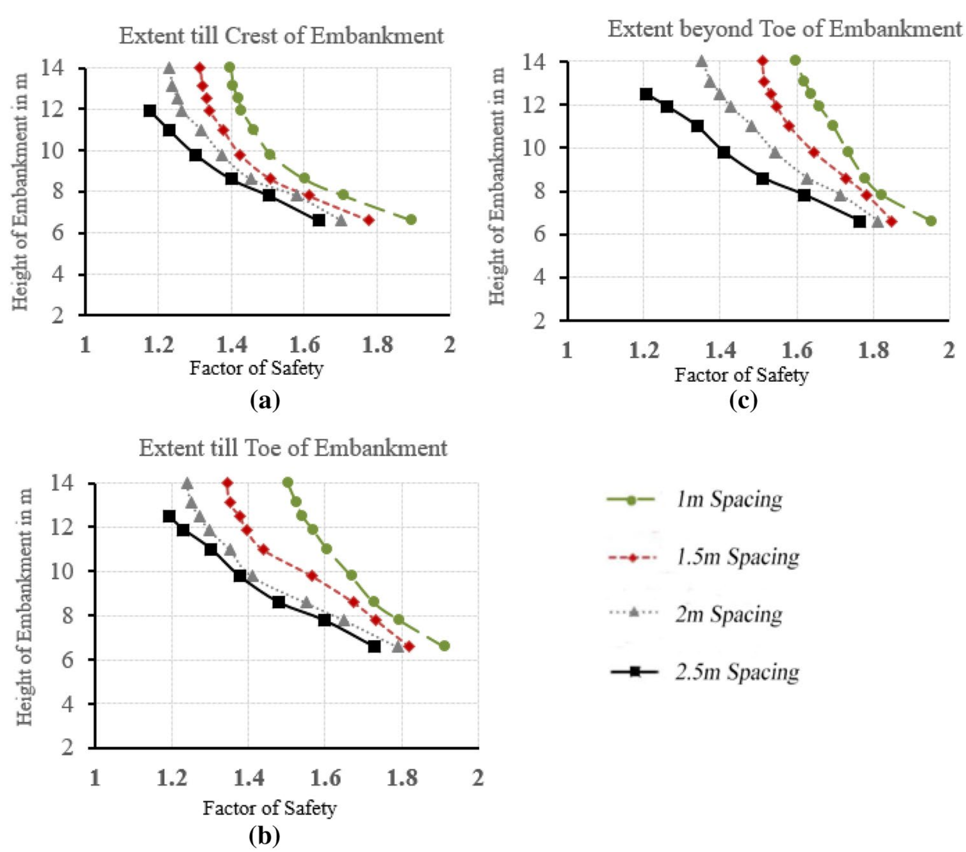



**Fig. 6** Factor of safety results for normal pace of construction, with 0.6 m fill layer thickness and PVDs extending to **a** the crest, **b** to the toe, and **c** beyond the toe of the embankment

is found to fail at a height of 11.9 m. When the area covered by the drains extends to or beyond the toe of the embankment, with a drain spacing of 2.5 m the embankment is found to fail at a height of 12.8 m. When the area covered by the drains extends to the toe of the embankment, a maximum FS of 1.953 is found for a drain spacing of 1 m. Thus, extending the area covered by the drains beyond the toe of the embankment with a drain spacing of 1 m yields the highest factor of safety, but may not be the most economical solution. Because failure occurs with a drain spacing of 2.5 m, it can be seen that a drain spacing of 1.5 m is the most suitable for the problem under consideration.

### Settlement

Figure 7 shows that as the extent of the area covered by the drains increases, settlement within a given time increases, but not by a significant amount. When the area covered by the drains extends only to the crest of the embankment, settlement is less than when the drains extend beyond the toe of the embankment. This essentially means that final settlement

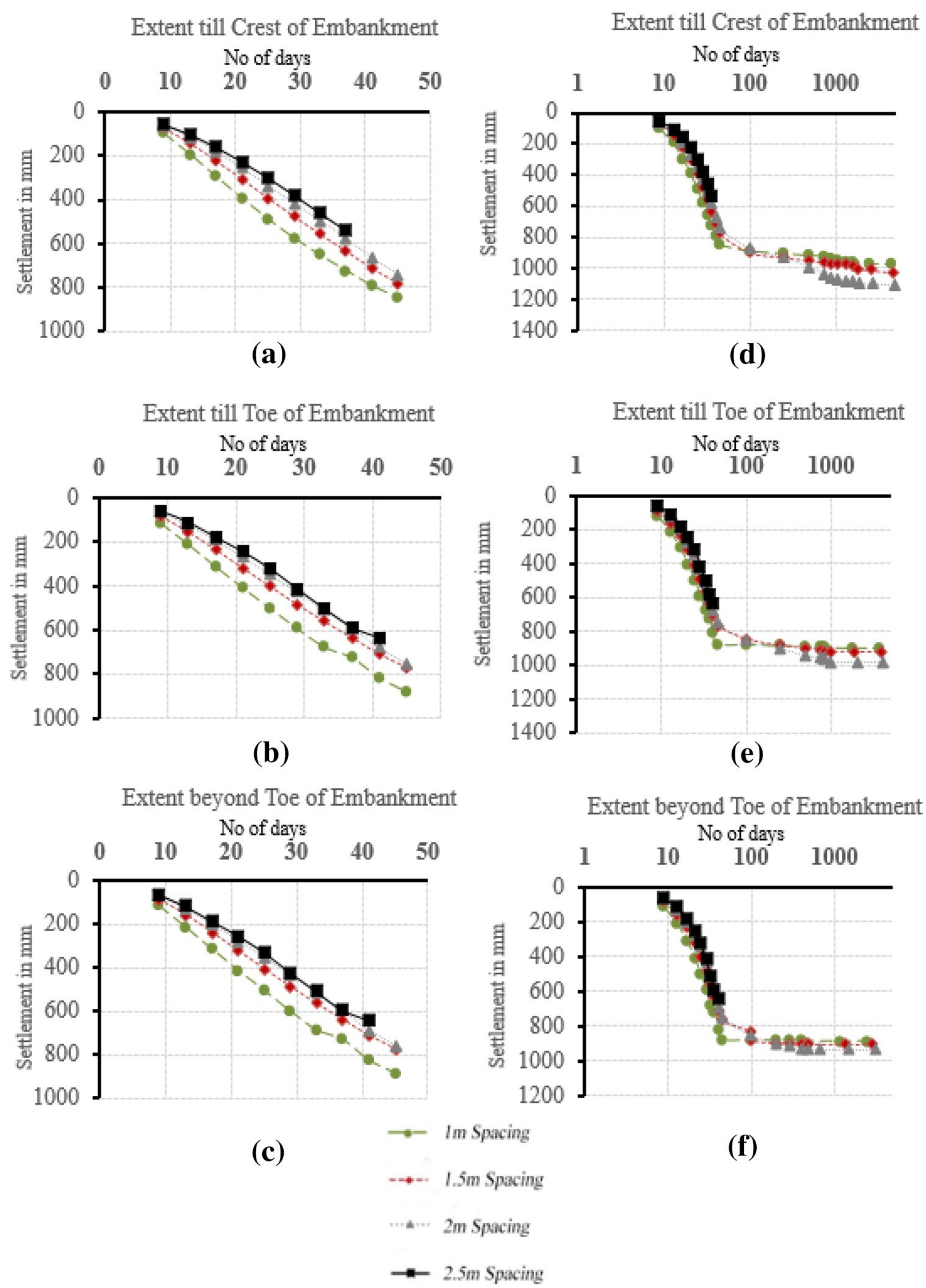



**Fig. 7** Settlement results for normal pace of construction, with 0.6 m fill layer thickness and PVDs extending to **a**, **d** the crest, **b**, **e** to the toe, and **c**, **f** beyond the toe of the embankment

occurs more quickly when the drains extend to or beyond the toe of the embankment than when the drains extend only to the crest. This is due to the fact that when the drains extend beyond the toe of the embankment, there is a faster rate of consolidation and hence greater settlement.

Figure 7 also shows that for a 40-day period, the settlement ranges from 800 mm for a drain spacing of 1 m, to 600 mm for a drain spacing of 2.5 m. It can also be seen that as the spacing of the drains increases, the rate of initial settlement decreases. This means that as the drain spacing increases from 1 m to 2.5 m, the dissipation of excess pore water pressure decreases and hence the settlement is reduced, causing the soil to lack effective strength, which eventually leads to failure. The models are also analysed for 95%, 98%, and 100% consolidation, as shown in the graphs on Fig. 7d–f. It can be seen that when the area covered by the drains is expanded from the crest to the toe of the embankment, it takes less time to reach 95% consolidation, since many more drains are available to help dissipate the excess power water pressures and consolidate the soil. With a drain spacing of 2.5 m, failure occurs after 37 days, at a settlement value of 540 mm, when the PVDs extend only to the crest of the embankment. For the same drain spacing, with PVDs extending beyond the toe of the embankment, failure occurs after 45 days, at a settlement value of 640 mm.

## Excess Pore Water Pressure

Since this case primarily focuses on studying the effect of PVD spacing on the stability of the embankment, Figs. 8, 9 compare the effect of different drain spacings. Figure 8 shows excess pore water pressure distributions at an embankment height of 7 m (mid-height) and Fig. 9 shows excess pore water pressure distributions immediately prior to failure. It can be clearly seen from the two figures that as the drain spacing increases, the dissipation of excess pore water pressure decreases. The slower dissipation of excess pore water pressures significantly reduces the effective strength of the soil, which increases effective stresses and eventually leads to failure. The maximum excess pore water pressure at mid-height is around 140 kPa and the maximum excess pore water pressure immediately prior to failure is around 260 kPa.

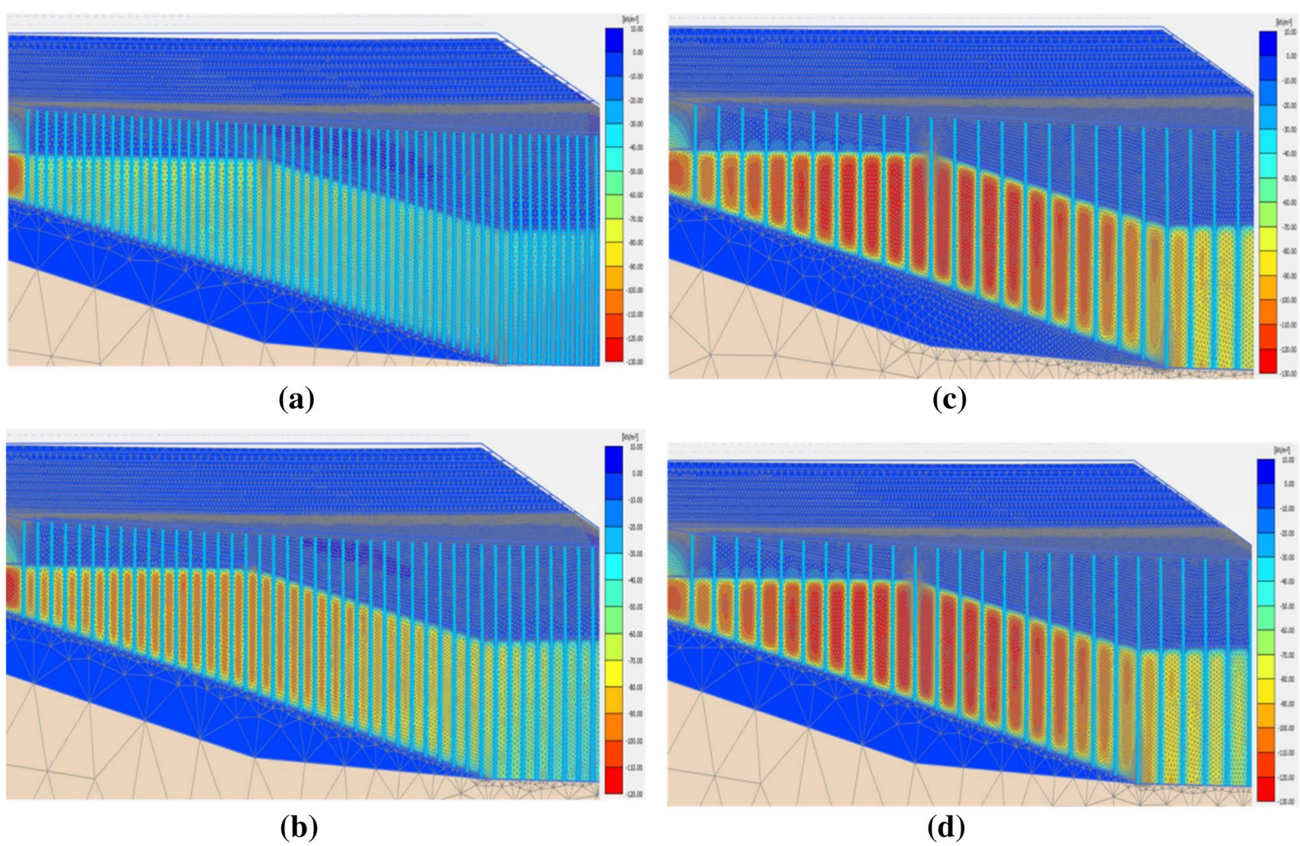


**Fig. 8** For an embankment height of 7 m, distribution of excess pore water pressures with drain spacing of **a** 1 m, **b** 1.5 m, **c** 2 m, and **d** 2.5 m

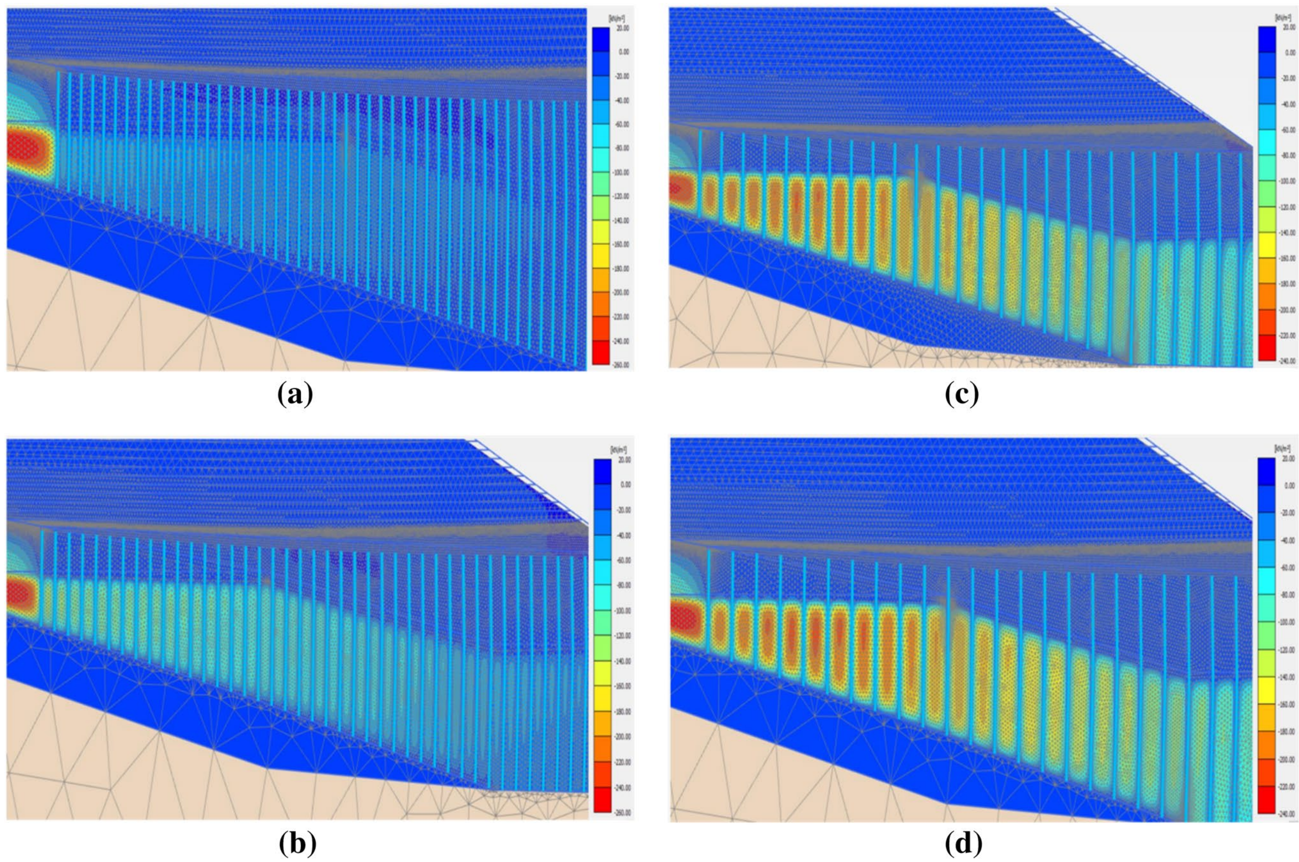


**Fig. 9** For embankment height immediately prior to failure, distribution of excess pore water pressures with drain spacing of **a** 1 m, **b** 1.5 m, **c** 2 m, and **d** 2.5 m

### Lateral and Vertical Deformation

Figure 10a–c shows that as the extent of the area covered by the PVDs expands from the crest to beyond the toe of the embankment, the lateral displacement decreases. With a larger area covered by the PVDs, the dissipation of excess pore water pressure increases, resulting in greater vertical deformation due to increased settlement, rather than lateral deformation. It can also be seen that as the drain spacing increases, the lateral deformation increases. This is due to a reduced dissipation of excess pore water pressures as the drain spacing is increased, leading to less settlement and more lateral deformation.

Figure 10d–f shows that the vertical deformation increases as the extent of the area covered by the drains increases. However, there is no significant difference in the result when the PVD coverage is expanded from the toe to beyond the toe of the embankment. It can also be seen that when the PVDs extend only to the crest of the embankment, the vertical deformation is above ground level (GL), where the $x$-axis represents ground level. Vertical deformation above the ground level is due to heaving of the soil, resulting from loss of effective strength of the foundation soil. Thus, to strengthen the foundation soil, it is recommended that the drains should extend to the toe of the embankment, at least for the problem under consideration.

## Case with Fast Pace of Construction

For the particular problem considered in this paper, this case studies the effect of the extent of the area covered by PVDs and the effect of thicker fill layers on the stability of the embankment. Since the focus is primarily on the extent of PVD coverage, this case includes a model with no drains, for the purpose of comparison. The extent of the area covered by PVDs is considered with a constant drain spacing of 1.5 m. This drain spacing was selected based on the results presented in Sect. "Case with Normal Pace of Construction". In this case, fill layer thicknesses of 2 m, 3.5 m, and 4.5 m are considered. The consolidation time following the placement of each fill layer is assumed to be 1 day.

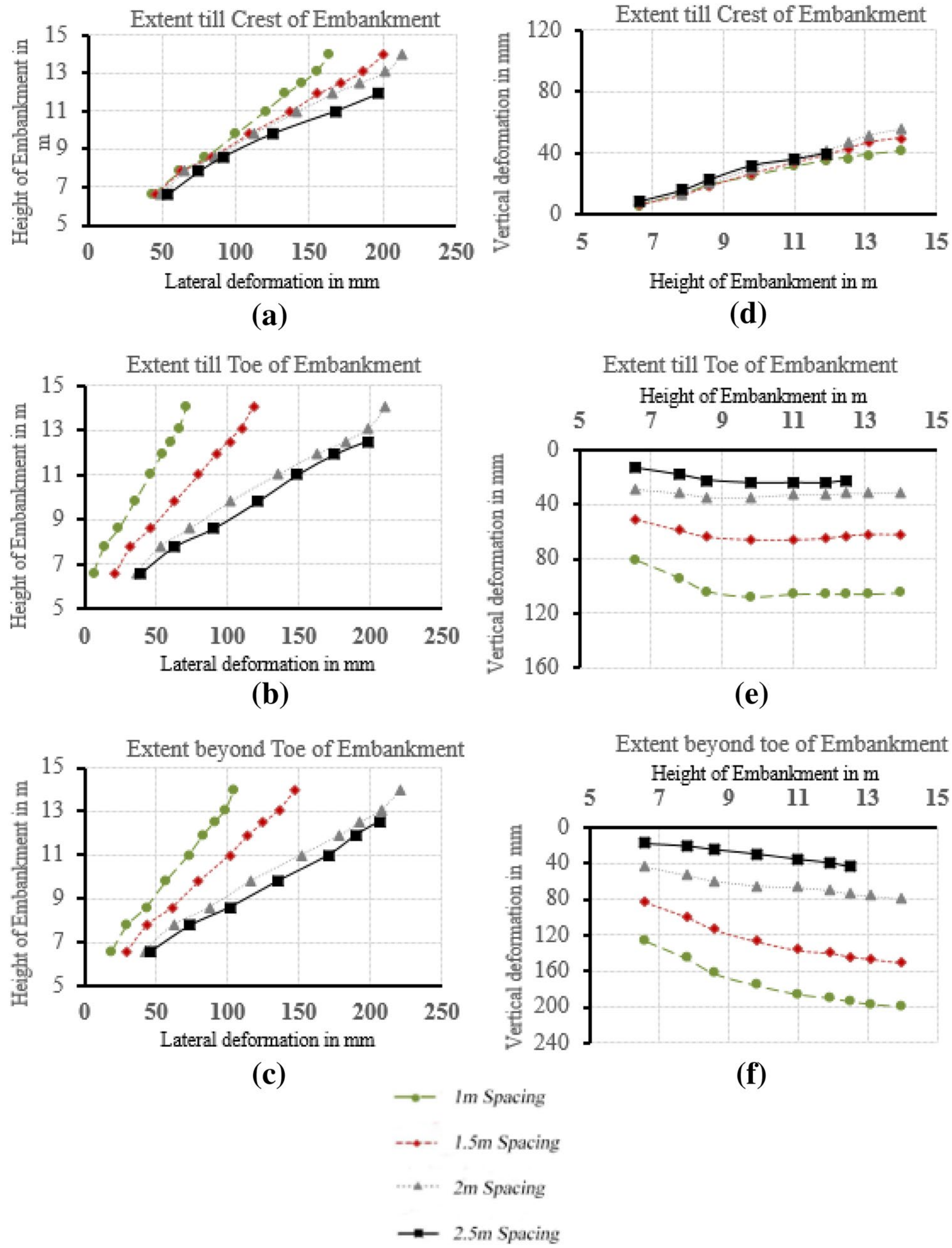



**Fig. 10** Lateral (**a**–**c**) and vertical (**d**–**f**) deformation for normal pace of construction, with 0.6 m fill layer thickness and PVDs extending to the crest, to the toe, and beyond the toe of the embankment

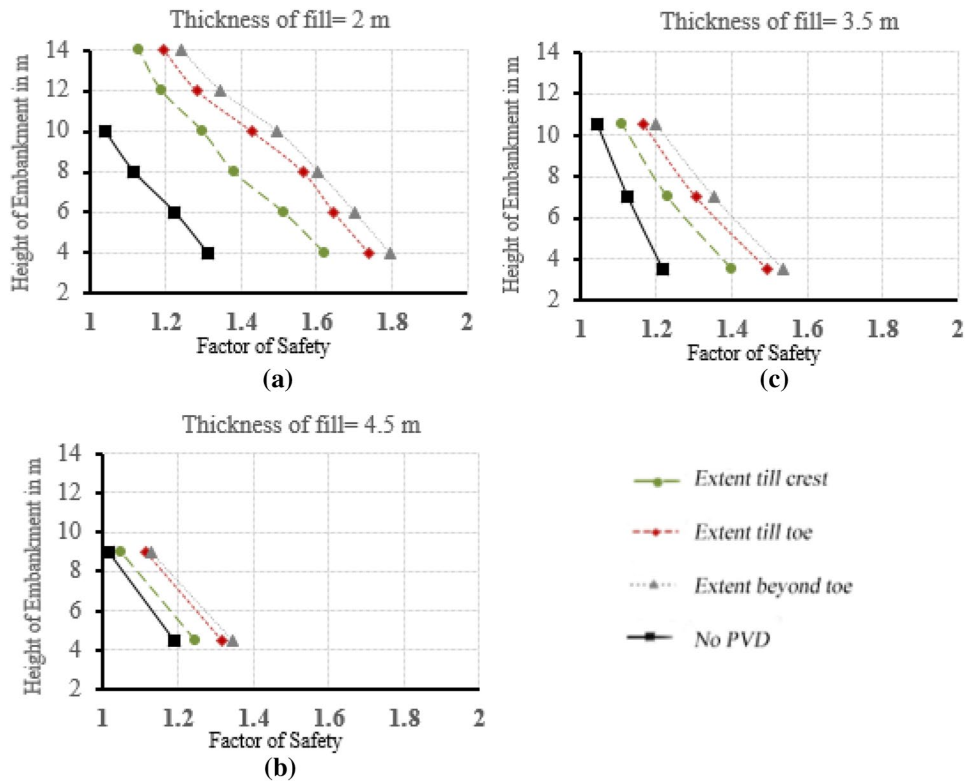
## Factor of Safety

Figure 11 shows that as the extent of the area covered by drains increases, the factor of safety increases. However, there is no significant difference in the result when the PVD coverage is expanded from the toe to beyond the toe of the embankment. The FS decreases as the fill layer thickness increases, due to greater effective stresses acting on the soil over a shorter time when thicker fill layers are used. When drains extend beyond the toe of the embankment, there is a maximum factor of safety of 1.794 for a fill layer thickness of 2 m; however, with fill layer thicknesses of 3.5 m and 4.5, the embankment fails at heights of 10.5 m and 9 m, respectively. Thus, for the problem under consideration, a fill layer thickness of 2 m, with PVDs extending to the toe of the embankment is considered the most suitable in terms of the safety and cost of the project. It can also be seen the use of drains results in significant improvement in the factor of safety. In the model with no PVDs, failure occurs at embankment heights of 10.5 m and 9 m, for fill layer thicknesses of 2 m and 4.5 m, respectively.

## Settlement

Figure 12 shows that as the extent of the area covered by the PVDs increases, the rate of initial settlement increases.

Fig. 11 Factor of safety results for fast pace of construction, with 1.5 m drain spacing and fill layer thicknesses of **a** 2 m, **b** 3.5 m, and **c** 4.5 m




However, the settlement curves are similar when PVDs extend to the toe of the embankment, and when they extend beyond the toe. A greater number of drains cause the settlement to increase, due to more rapid dissipation of excess pore water pressures. It can also be seen that as the fill layer thickness increases, the settlement increases, because of a more rapid rate of application of effective stresses. However, this sudden increase in settlement over a short period of time gives rise to excess pore water pressures, thus reducing the effective strength of the soil, which leads to failure of the foundation soil.

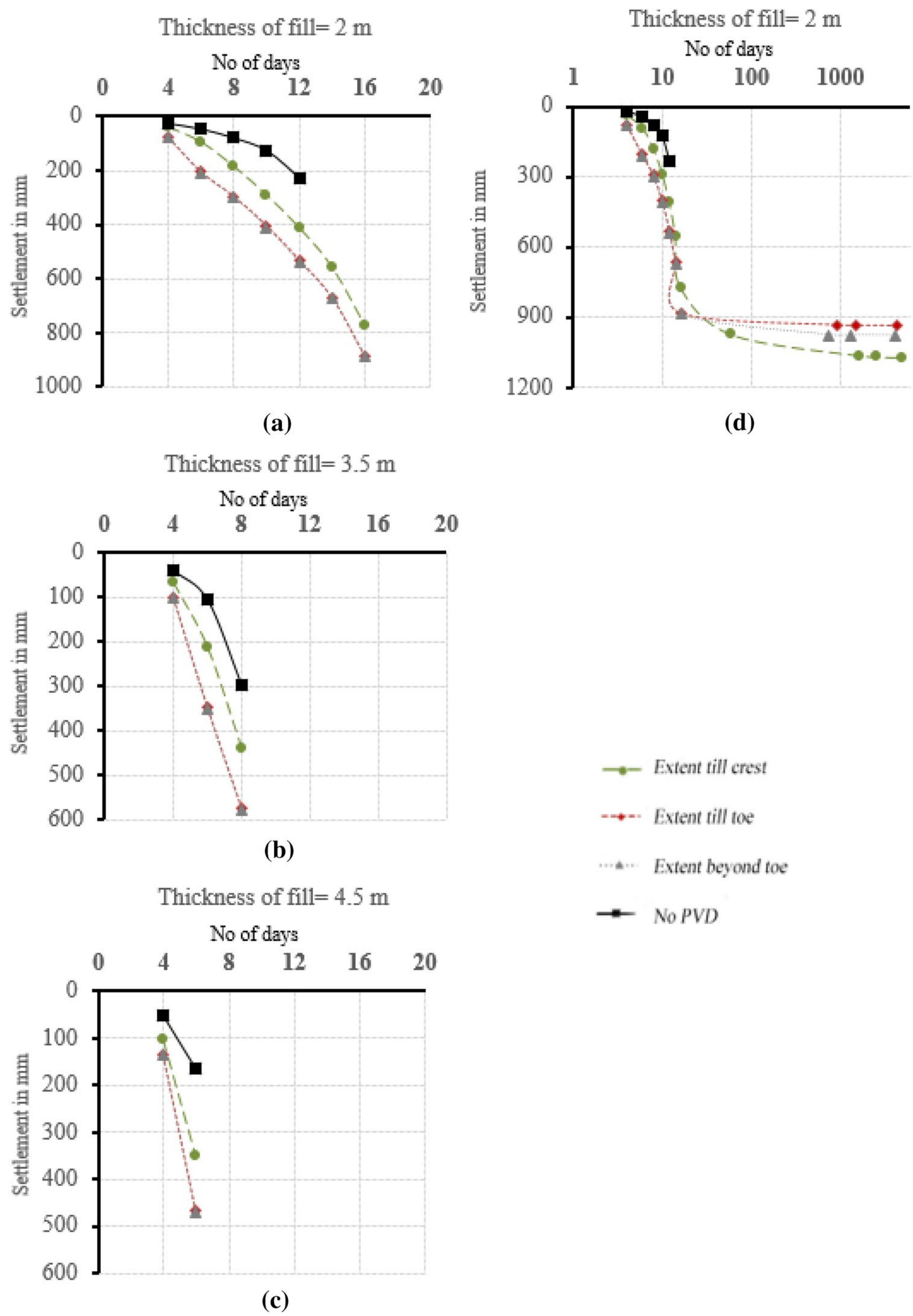
For a fill layer thickness of 3.5 m (Fig. 12b), in days 8–9, the settlement ranges from 440 mm with PVDs extending to the crest, to 580 mm with PVDs extending to the toe of the embankment. With a fill layer thickness of 4.5 m (Fig. 12 (c)); in days 6 to 7, the settlement ranges from 350 mm with PVDs extending to the crest, to 470 mm with PVDs extending to the toe of the embankment. For the model where no drains were used, the final settlement ranges from 230 mm with a fill layer thickness of 2 m, to 166 mm with a fill layer thickness of 4.5 m. The PVDs thus help to dissipate excess pore water pressures, resulting in gradual settlement over time. The settlement results for 95%, 98%, and 100% consolidation are shown in Fig. 12d. It can be clearly seen that with a fill layer thickness of 2 m, 95% consolidation is achieved in the shortest time with PVDs extending to the toe or beyond the toe of the embankment.

### Excess Pore Water Pressure

Figures 13, 14 show excess pore water pressure distributions for an embankment height of 7 m (mid-height) and the final height, respectively. A fill layer thickness of 2 m is used. The excess pore water pressures range from about 130 kPa at mid-height to 250 kPa at the final embankment height. It can be seen that the dissipation of excess pore water pressure increases as the extent of the area covered by PVDs increases. For the model with no PVDs, there is an accumulation of excess pore water pressures directly beneath the embankment, which significantly reduces the effective strength of the soil in that zone.

Figure 15 shows excess pore water pressure distributions for a fill layer thickness of 3.5 m. It can be seen that with thicker fill layers, the dissipation of excess pore water pressure is reduced, because thicker fill layers mean a shorter construction time, with less time for consolidation of the soil. Thus, with PVDs extending only to the crest of the embankment, failure occurs before the proposed embankment height is reached, with an effective vertical stress of around 260 kPa. However, increasing the area covered by the PVDs results in greater dissipation of excess pore water pressures. Figure 16 shows excess pore water pressure distributions for a fill layer thickness of 4.5 m. The results follow the same pattern as seen in Fig. 15. However, the dissipation of excess pore water pressure is further reduced, and with PVDs extending

**Fig. 12** Settlement results for fast pace of construction, with 1.5 m drain spacing and fill layer thicknesses of **a**, **d** 2 m, **b** 3.5 m, and **c** 4.5 m




only to the crest of the embankment, the embankment fails at a height even lower than is the case with a fill layer thickness of 3.5 m. As shown in Figs. 15, 16, with no PVDs, excess pore water pressures are concentrated beneath the embankment in the marine clay layer, thus significantly reducing the effective strength of this layer.

## Lateral and Vertical Deformation

Figure 17 shows that the lateral deformation increases as the fill layer thickness increases. This is because there is less dissipation of excess pore water pressures when there is a significantly faster rate of application of effective stresses. This

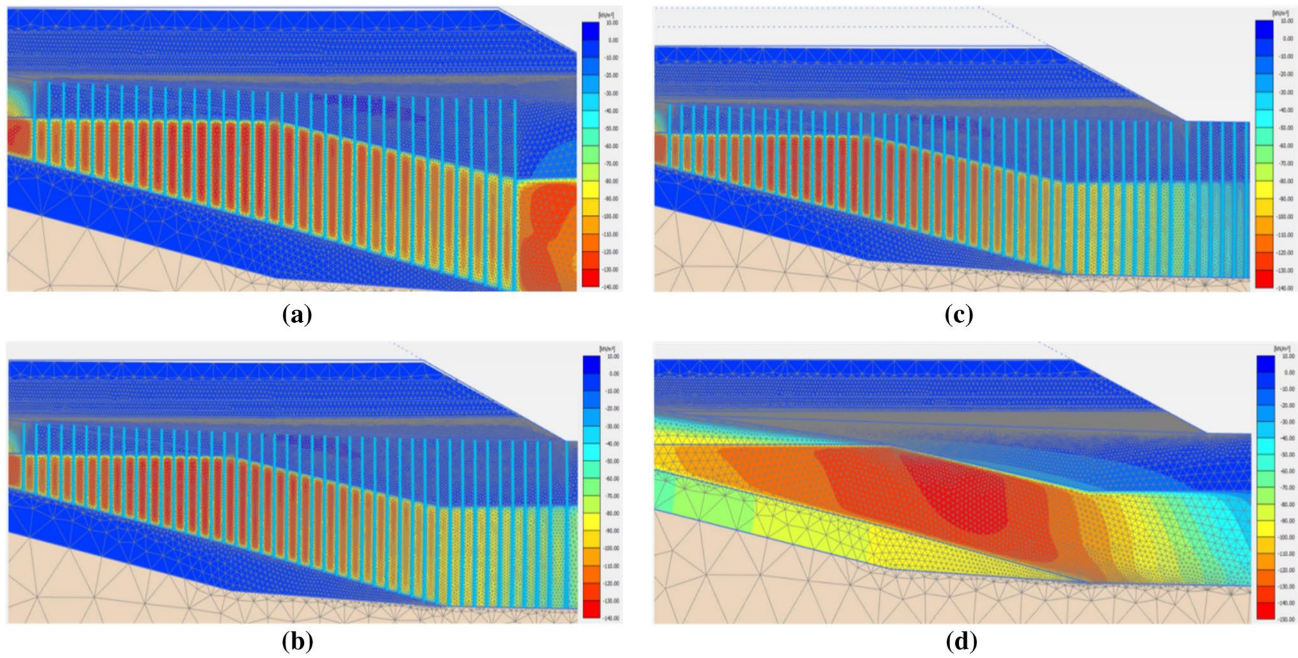


**Fig. 13** For an embankment height of 7 m and fill layers 2.0 m thick, distribution of excess pore water pressures with PVDs extending **a** to the crest, **b** to the toe, and **c** beyond the toe of the embankment; and **d** with no PVDs

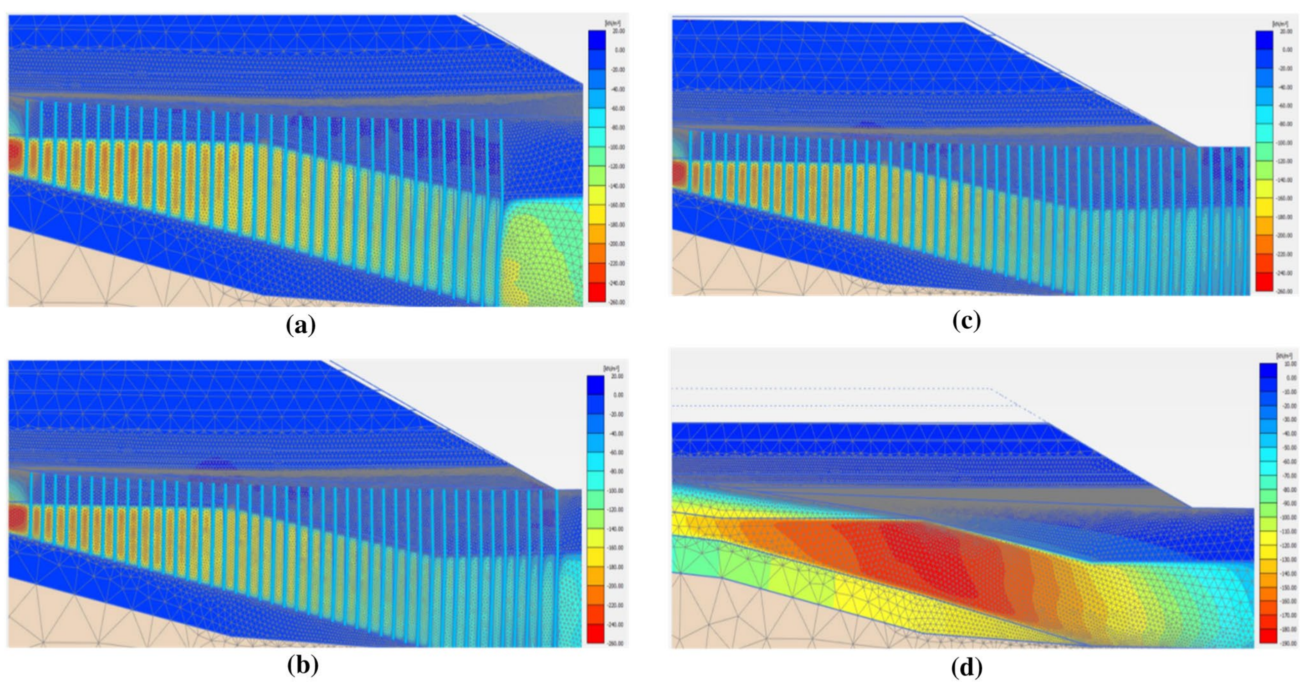


**Fig. 14** For embankment at final height and fill layers 2.0 m thick, distribution of excess pore water pressures with PVDs extending **a** to the crest, **b** to the toe, and **c** beyond the toe of the embankment; and **d** with no PVDs

results in a slower rate of consolidation, leading to increased lateral deformation. It can be seen that for a given embankment height, the lateral deformation is greater for fill layer thicknesses of 3.5 m and 4.5 than for a fill layer thickness of 2 m. It should be noted that with fill layers 3.5 m and 4.5 m thick, the embankment fails at heights of 10.5 m and 9 m, respectively. It can also be seen that as the area covered by PVDs increases, the lateral deformation decreases. This is

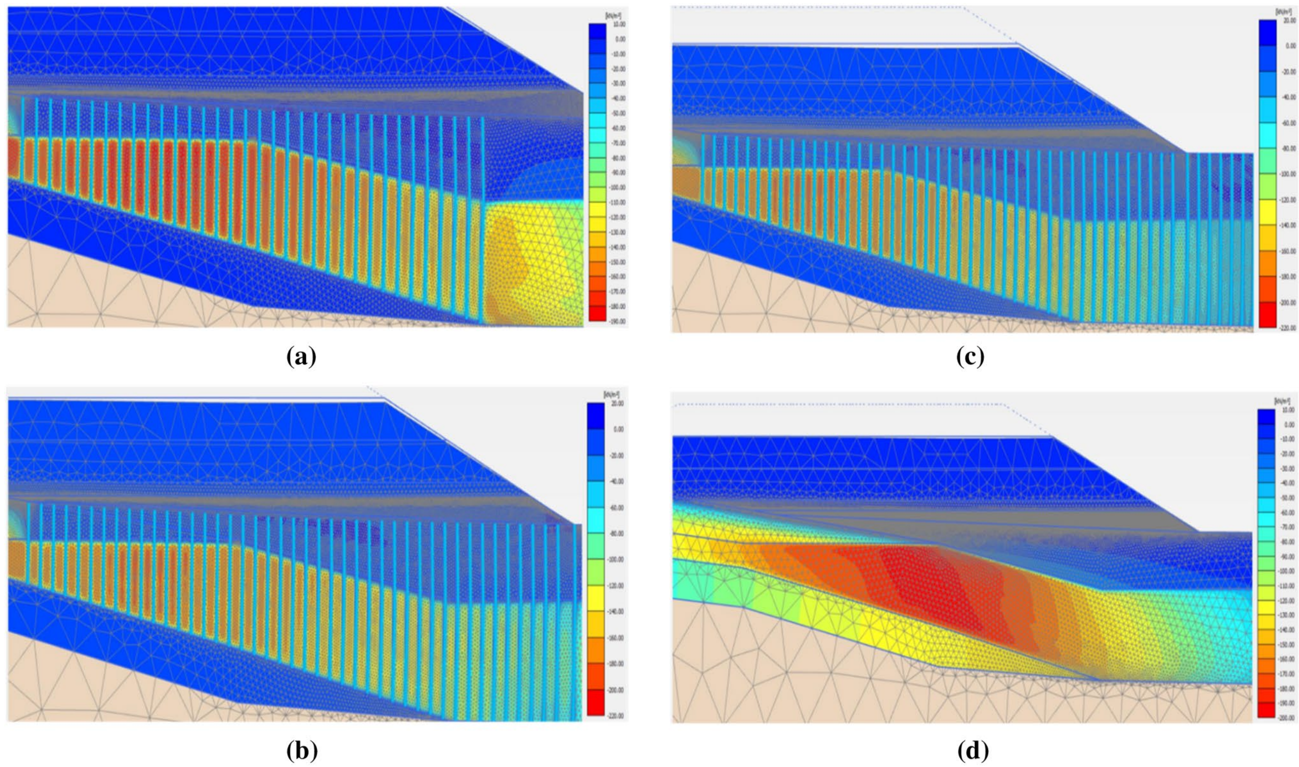



**Fig. 15** For fill layers 3.5 m thick, distribution of excess pore water pressures with PVDs extending **a** to the crest, **b** to the toe, and **c** beyond the toe of the embankment; and **d** with no PVDs

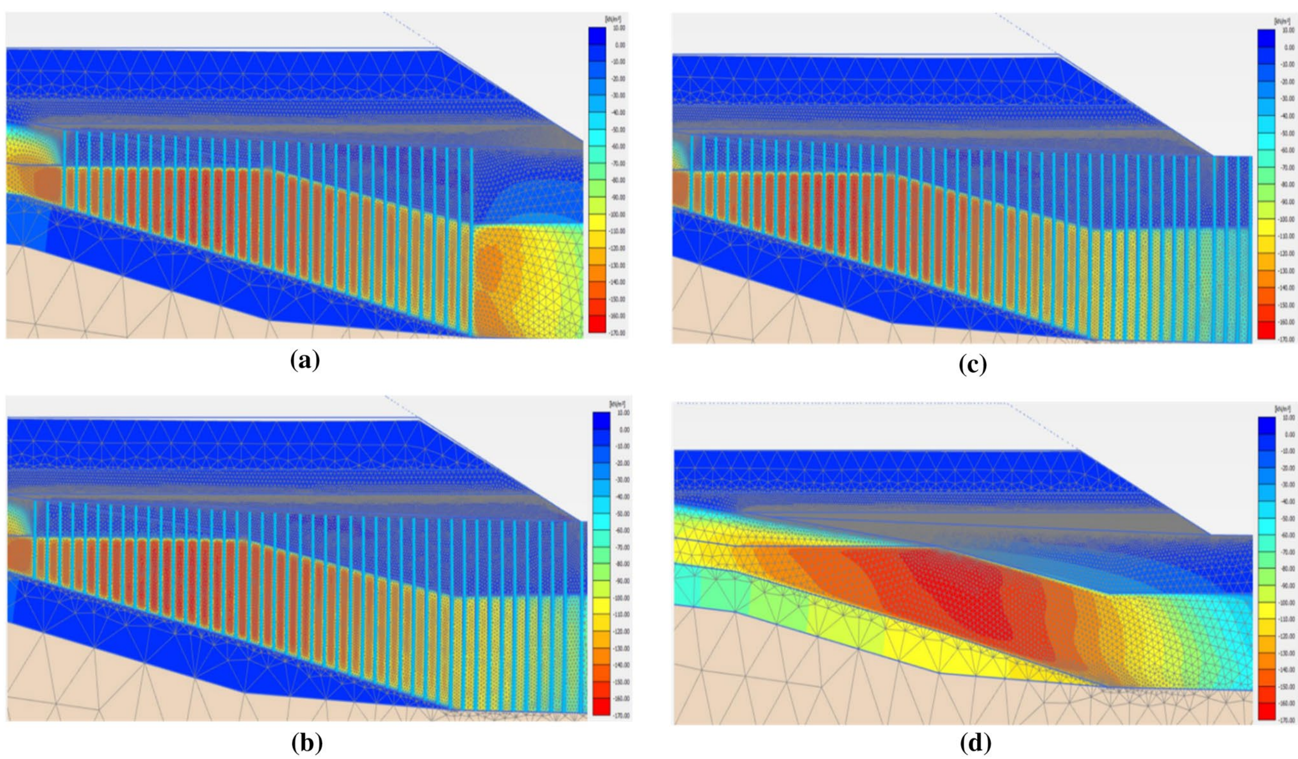



**Fig. 16** For fill layers 4.5 m thick, distribution of excess pore water pressure with PVDs extending **a** to the crest, **b** to the toe, and **c** beyond the toe of the embankment; and **d** with no PVDs

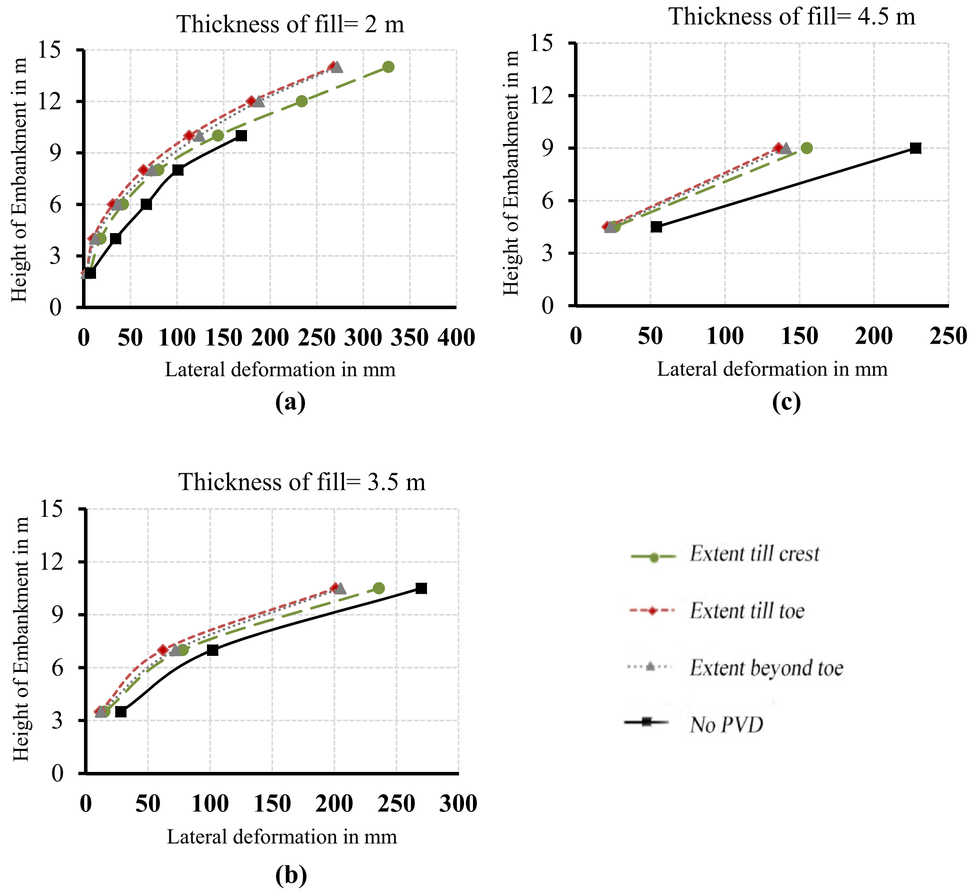



**Fig. 17** Lateral deformation results for fast pace of construction, with 1.5 m drain spacing and fill layer thicknesses of **a** 2 m, **b** 3.5 m, and **c** 4.5 m

due to more rapid dissipation of excess pore water pressures, leading to a higher rate of consolidation, which results in less lateral deformation and greater vertical deformation.

Figure 18 shows that as the fill layer thickness increases, the vertical deformation also increases. Here the *x*-axis represents the ground level. From Fig. 18a–c, when PVDs extend only to the crest of the embankment, or when there are no PVDs, there is heaving of the soil, with vertical deformations above the ground level. This is because the slow rate of consolidation results in a rapid loss of effective strength, causing the soil to heave above the ground level.

From Fig. 18d–f it is observed that when PVDs extend to or beyond the toe of the embankment, there is an increased rate of consolidation, with the result that vertical deformations remain below the ground level. It can be seen that when a larger area is covered by PVDs, greater vertical deformation results, due to increased consolidation.

## Case with Construction Allowing More Time for Consolidation

For the particular problem considered in this paper, this case studies the effect of increasing the consolidation time and the effect of thicker fill layers on the stability of the embankment. Since the focus is primarily on the pace of construction, in this case the use of PVDs is kept constant, with PVDs extending to the toe of the embankment (based on the findings in Sect. "Case with Fast Pace of Construction") and with a drain spacing of 1.5 m (based on the findings in Sect. "Case with Normal Pace of Construction"). In this case, fill layer thicknesses of 2 m, 3.5 m, and 4.5 m are considered, and consolidation times of 5 days, 10 days, and 15 days are used following the placement of each fill layer.

## Factor of Safety

Figure 19 shows that as the consolidation time increases, the factor of safety increases. This is because excess pore water pressures can dissipate rapidly due to the presence of drains, and the longer consolidation times permit increased consolidation. With greater consolidation, the soft soils gain strength due to the increase in effective stresses and decrease in pore pressures, and the stability of the embankment therefore increases. As shown in Fig. 19a, at an embankment height of 14 m, the highest factor of safety obtained is 1.769, for fill layers 2 m thick, consolidated for 15 days after the placement of each layer; and the lowest factor of safety obtained is 1.442, for fill

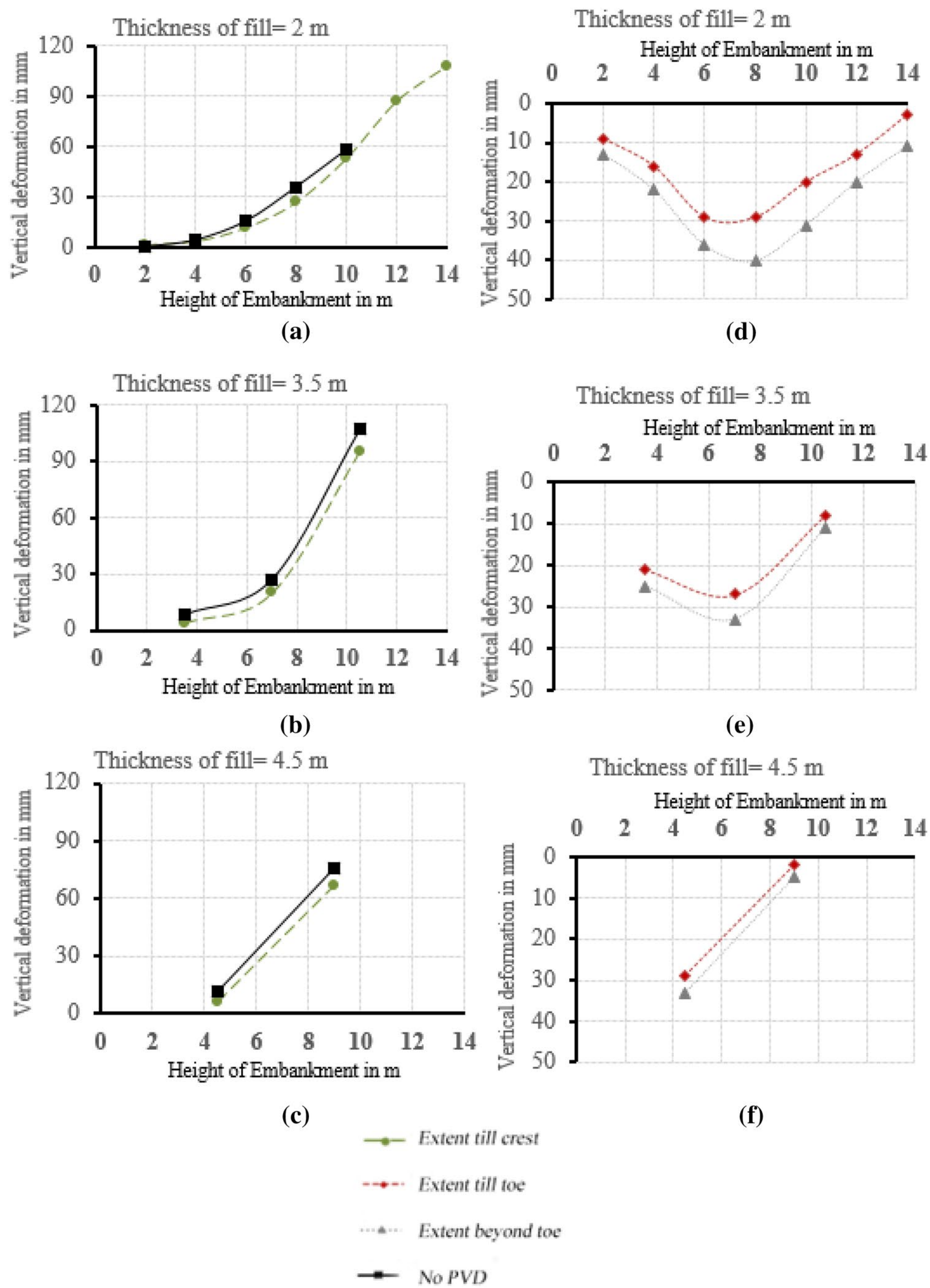



**Fig. 18** Vertical deformation results for fast pace of construction, with 1.5 m drain spacing and fill layer thicknesses of 2 m (**a**, **d**), 3.5 m (**b**, **e**), and 4.5 m (**c**, **f**)

layers 4.5 m thick, consolidated for 5 days (Fig. 19c) after the placement of each layer. Thus, the embankment stability increases significantly with an increase in consolidation time. However, when the costs and total duration of the construction are also taken into consideration, it may be concluded that consolidation periods of 5–10 days, with fill layer thicknesses of 2–3 m, may provide the most suitable design solution for this particular problem.

## Settlement

Figure 20 shows that as the consolidation time following the placement of each fill layer increases, the settlement becomes more gradual. Figure 20a shows that after 40 days, for fill layers with a thickness of 2 m, settlement is around 380 mm for consolidation times of 15 days, and is approximately 780 mm for consolidation times of 5 days. From

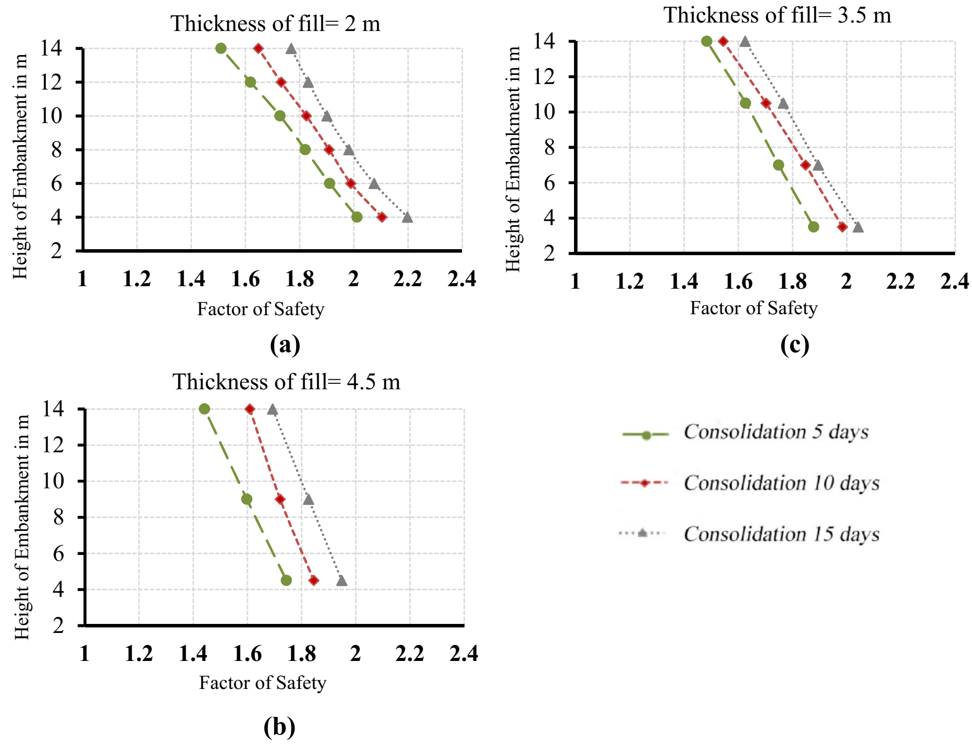



**Fig. 19** Factor of safety results with 5, 10, and 15 days allowed for consolidation, with 1.5 m drain spacing, PVDs extending to toe of embankment, and **a** 2 m, **b** 3.5 m, and **c** 4.5 m fill layers

Fig. 20, it can be seen that the rate of settlement increases sharply with an increase in fill layer thickness, resulting in a dramatic loss of effective strength in the foundation soil. From Fig. 20a–c, it can be seen that after 40 days, for consolidation times of 15 days, the settlement is around 380 mm for fill layers 2 m thick around 610 mm for fill layers 3.5 m thick and around 760 mm for fill layers 4.5 m thick With longer consolidation times, the settlement becomes more gradual, and greater strength is gained by saturated soft soils such as marine clays. For the particular problem under consideration, consolidation times of 10–15 days are recommended.

From Fig. 20d–f, it can be seen that with consolidation times of 15 days and a fill layer thickness of 2 m, 95% consolidation is achieved in around 1019 days, with a settlement of about 846 mm. In contrast, with consolidation times of 5 days and a fill layer thickness of 4.5 m, 95% consolidation is achieved in around 869 days, with a settlement of about 952 mm.

### Excess Pore Water Pressure

Figure 21 shows excess pore water pressure distributions for a fill layer thickness of 2 m. It can be seen that as the consolidation time increases, the dissipation of excess pore water pressures increases, due to greater consolidation. The excess pore water pressure ranges from 260 kPa for a consolidation period of 5 days to 110 kPa for a consolidation period of 15 days. Figure 22 shows excess pore water pressure distributions for a fill layer thickness of 3.5 m, where the excess pore water pressure ranges from 260 kPa for a consolidation period of 5 days to 170 kPa for a consolidation period of 15 days. Figure 23 shows excess pore water pressure distributions for a fill layer thickness of 4.5 m, where the excess pore water pressure ranges from 260 kPa for a consolidation period of 5 days to 190 kPa for a consolidation period of 15 days. Thus, for a consolidation period of 15 days, upon completion of construction of the embankment, the excess pore water pressure ranges from around 110 kPa for fill layers 2 m thick to 190 kPa for fill layers 4.5 m thick, due to the more rapid increase in effective stresses with the addition of thicker fill layers.

### Lateral and Vertical Deformation

Figure 24a–c shows that as the fill layer thickness increases, the lateral deformation increases. However, as the time allowed for consolidation increases, the lateral deformation increases considerably over long periods of time. For a fill layer thickness of 2 m, the lateral deformation increases in a linear elastic manner up to an embankment height of 6 m, and then increases in a linear plastic manner up to an embankment height of 14 m. For a fill layer thickness of 3.5 m, the lateral deformation increases in a linear elastic manner almost throughout, and for a fill layer thickness of 4.5 m the increase is completely linear elastic. This is because the embankment height of 14 m is reached in only 3 or 4 construction phases, for fill layers with a thickness of 4.5 m and 3.5 m, respectively. Thus, with thicker fill layers,

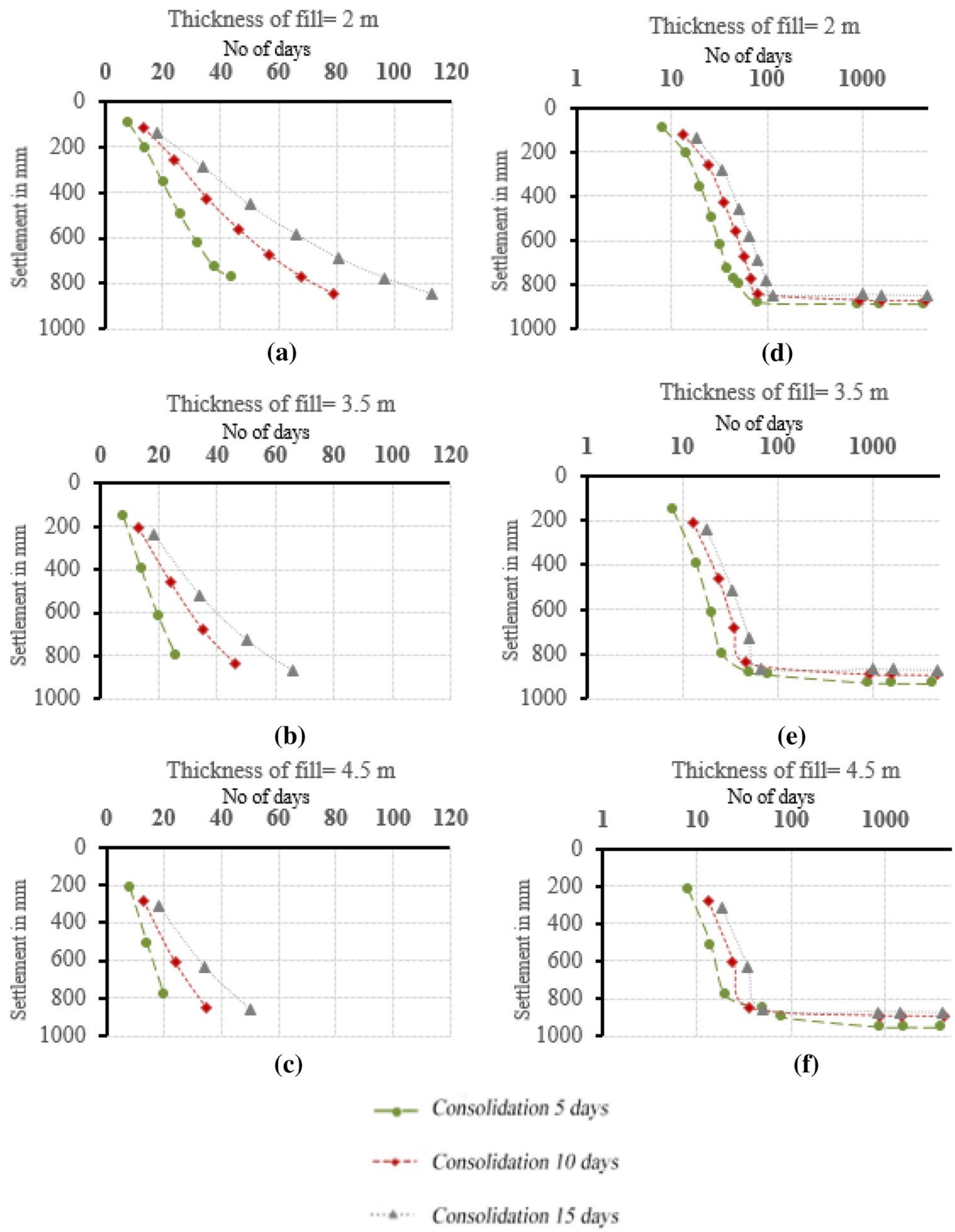



**Fig. 20** Settlement results with 5, 10, and 15 days allowed for consolidation, with 1.5 m drain spacing, PVDs extending to toe of embankment, and 2 m (**a**, **d**), 3.5 m (**b**, **e**), and 4.5 m (**c**, **f**) fill layers

there are fewer points on the graph to interpolate, which results in a more linear curve in the graphical representation.

Figure 24d–f shows that vertical deformation increases as the consolidation time and the fill layer thickness increases. Here the $x$-axis represents the ground level. As the time allowed for consolidation increases, there is greater consolidation, resulting in more settlement, with the dissipation of excess pore water pressures. Thus, increased consolidation times result in increased settlement and vertical deformation. As the fill layer thickness increases, there is a sharp increase in the settlement, which reduces the effective strength of the soil. However, increased consolidation times allow the resulting excess pore water pressures to dissipate, and hence failure of the embankment does not occur even with greater fill layer thicknesses. Thus, it is evident that increasing the consolidation time significantly improves the stability of the embankment.

## Conclusions

From the parametric study presented in this paper, the following conclusions can be arrived at:

- The use of ground improvement techniques such as prefabricated vertical drains improves embankment stability significantly, while reducing the duration of construction and project costs.

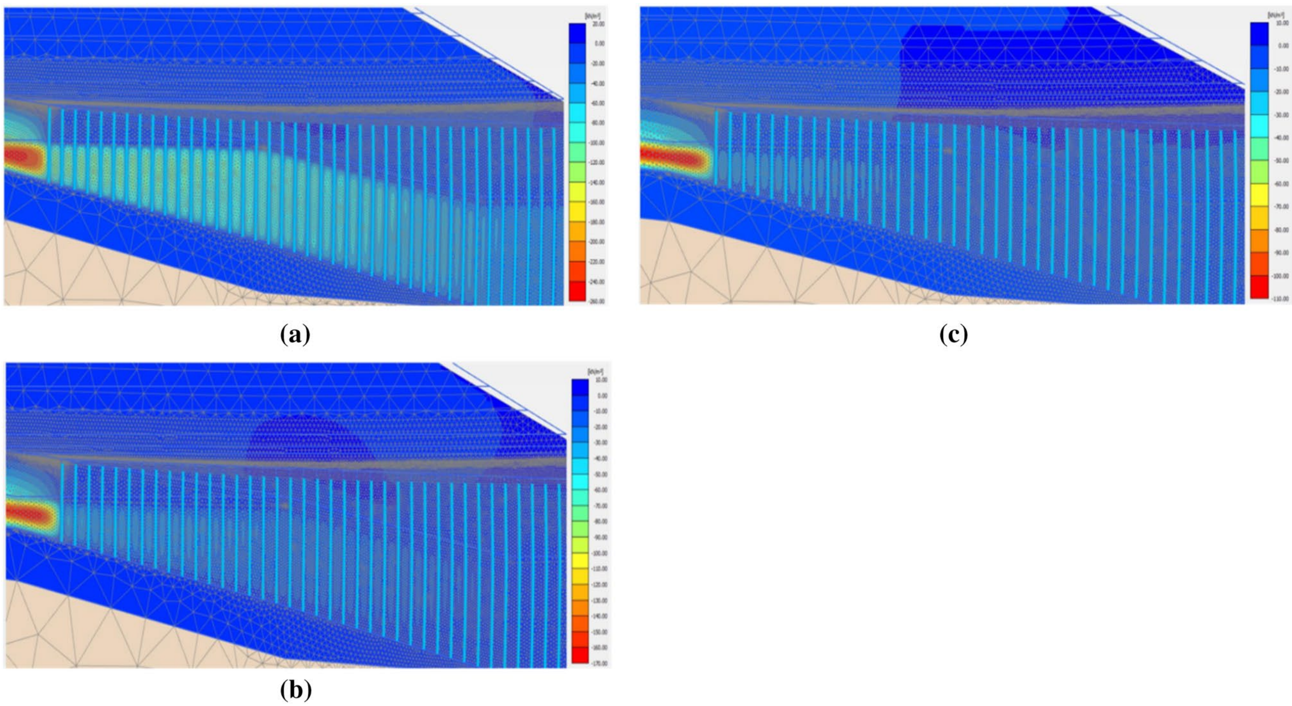


**Fig. 21** For a fill layer thickness of 2 m, distribution of excess pore water pressures with consolidation times of **a** 5 days, **b** 10 days, and **c** 15 days

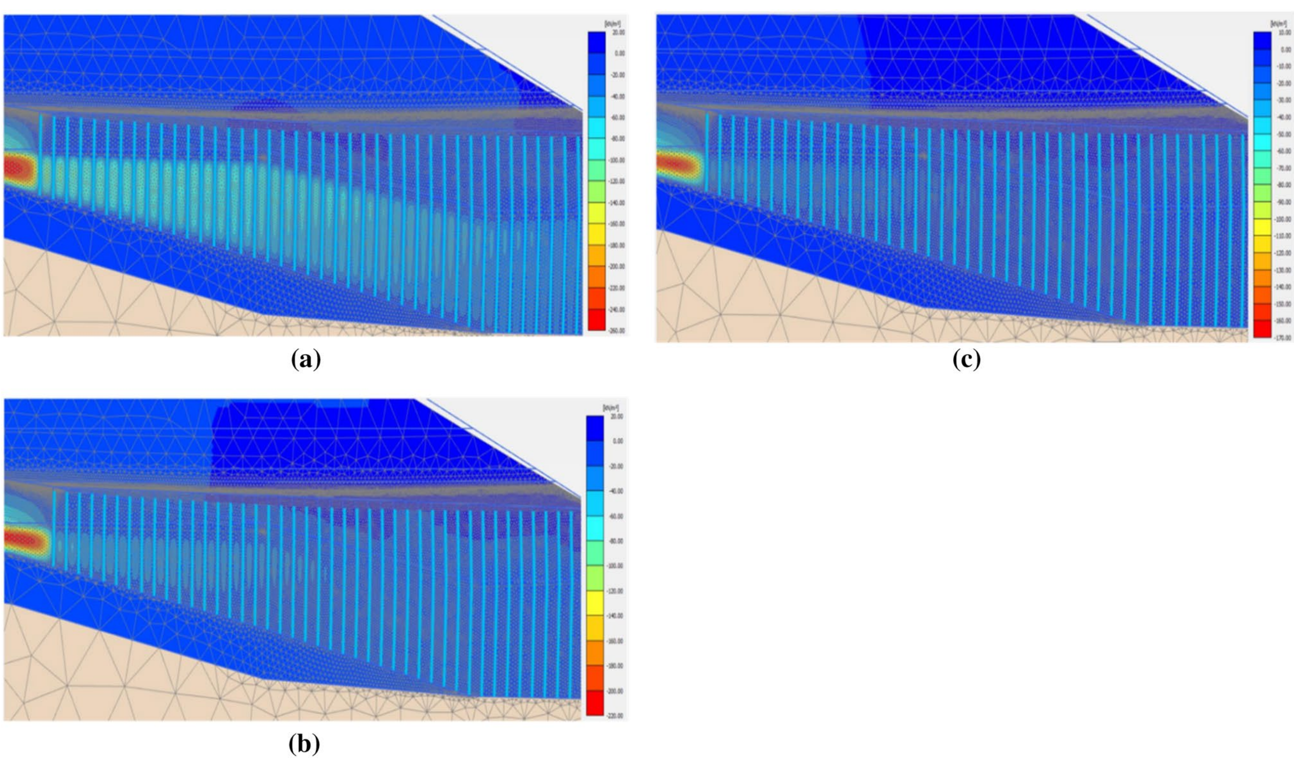


**Fig. 22** For a fill layer thickness of 3.5 m, distribution of excess pore water pressures with consolidation times of **a** 5 days, **b** 10 days, and **c** 15 days

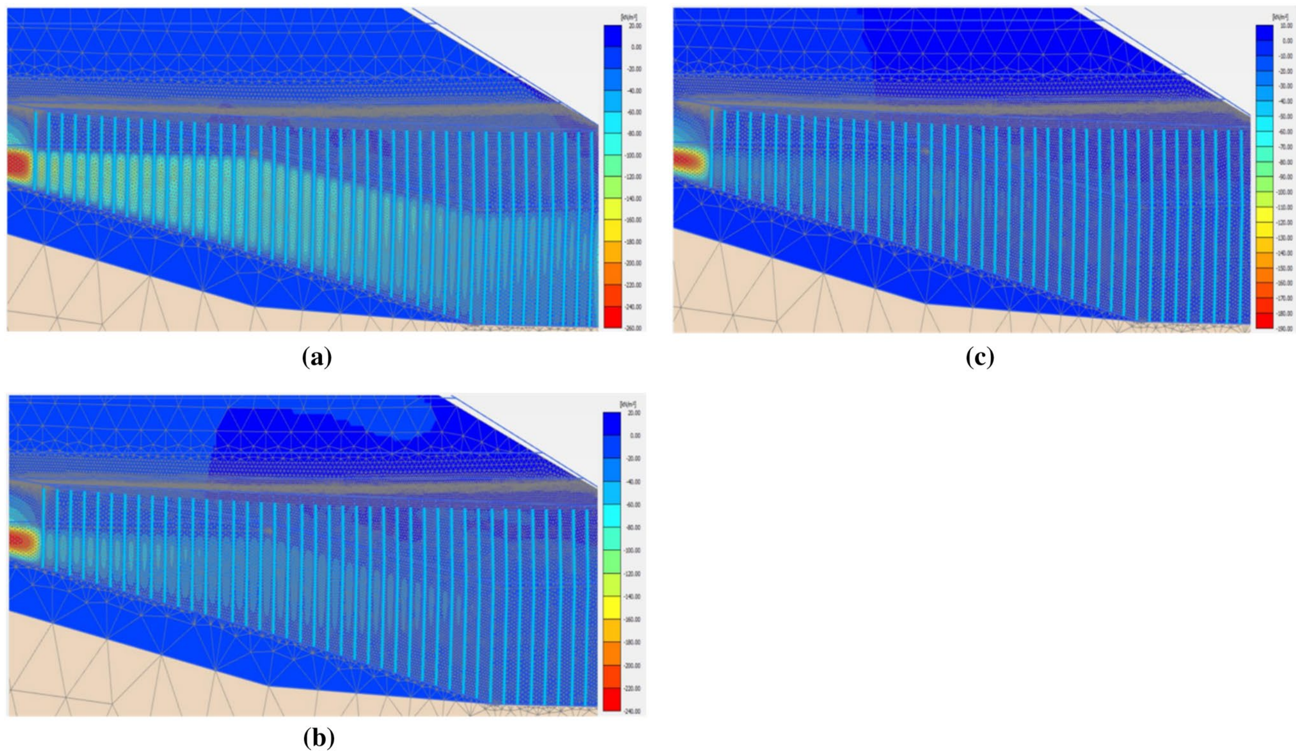



**Fig. 23** For a fill layer thickness of 4.5 m, distribution of excess pore water pressures with consolidation times of **a** 5 days, **b** 10 days, and **c** 15 days

- Drain spacing is one of the key parameters that influence the embankment stability. It could be concluded that a drain spacing of 1.5 m c/c provides a safe, economical solution that is the most suitable for the problem under consideration. A rough estimate of 1.5 m spacing could also be considered for various projects aiming to implement the use of drains for embankments founded on soft saturated soil deposits.
- Another important parameter apparent from this study is the extent of the area covered by the drains. The soil profile region where excess pore water pressures are concentrated must first be estimated or modelled without the use of drains. Based on this, the drains must be constructed so that they extend to the toe or beyond the toe of the embankment, depending on the stability and cost requirements of the project. For the problem under consideration, the extent of coverage found to be most suitable was for the drains to extend to the toe of the embankment.
- The pace of construction was studied by using thicker fill layers to reduce the construction time. It can be concluded that the embankment can be constructed by placing compact fill in layers 2–2.5 m thick. This enables the construction to proceed at a faster pace but reduces the stability of the embankment. Thus, there is a trade-off between stability and reducing the construction time and cost of the project.
- The pace of construction was also studied by combining the use of drains with varying consolidation times. It was found that even with a fill layer thickness of 4.5 m, a consolidation time of 5 days following the placement of each fill layer resulted in a relatively stable embankment, in comparison to the situation with a consolidation time of only 1 day, which caused the embankment to fail. Hence, allowing sufficient time for consolidation is one of the most important parameters influencing the pace of construction, in order to stabilise embankments founded on soft soil deposits. However, to minimise construction costs, using fill layers 2–2.5 m thick and allowing a consolidation time of 10–15 days following the placement of each fill layer would be the optimal design solution for the problem under consideration.

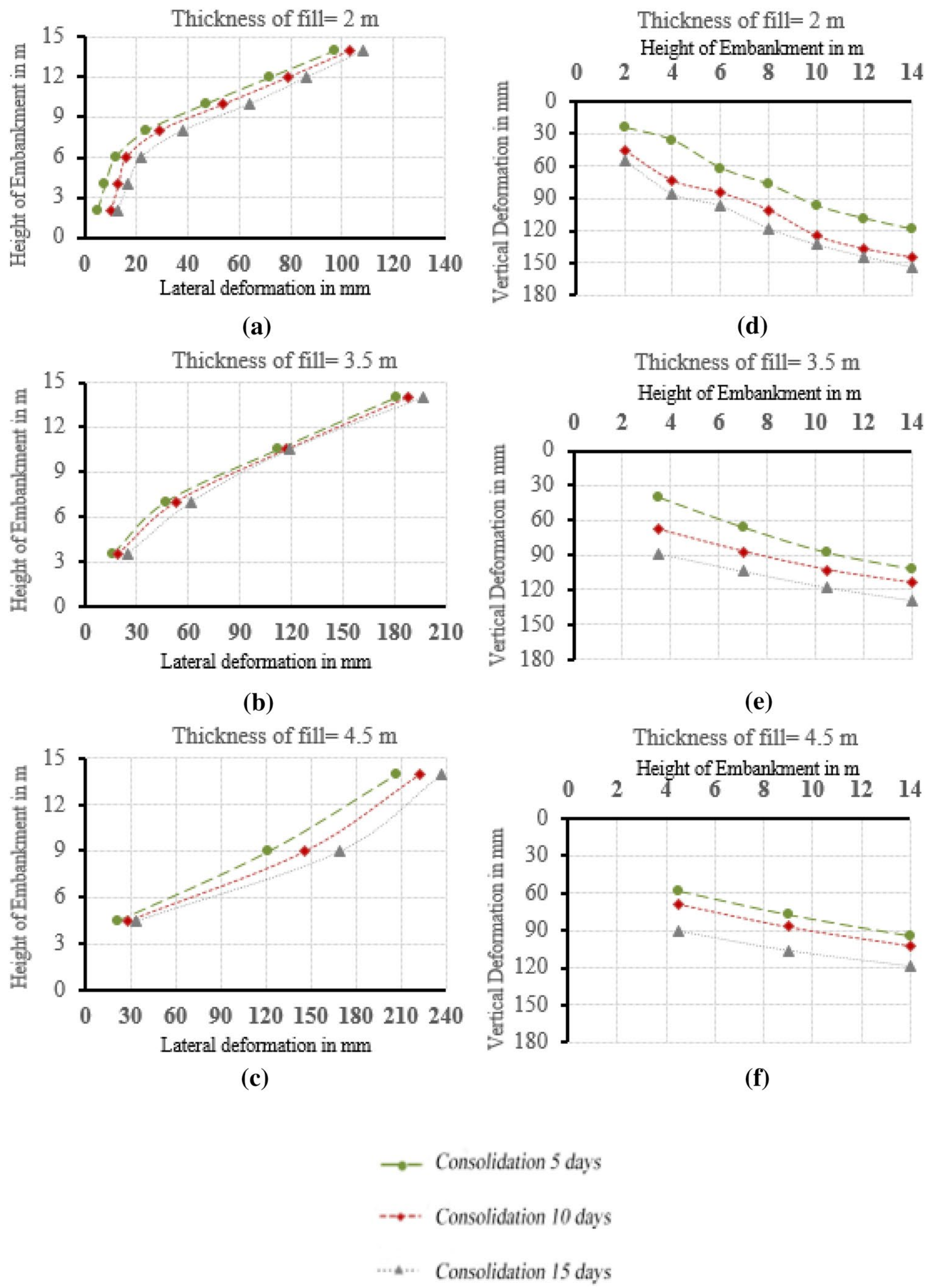



**Fig. 24** Lateral (**a**–**c**) and vertical deformation (**d**–**f**) results with 5, 10, and 15 days allowed for consolidation, with 1.5 m drain spacing and PVDs extending to toe of embankment

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**Author contributions** The authors confirm contributing to the paper as follows, Conceptualization: HN. Data curation: RB. Analysis and interpretation of results: RB and HN. Draft manuscript preparation: RB. HN reviewed the results and approved the final version of the manuscript. RB and HN addressed all the reviewers' comments.

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**Availability of data and materials** All data, models, and code generated or used during the study appear in the submitted article.

## Declarations

**Conflict of interest** The authors declare that they have no known competing financial interests that could have appeared to influence the work reported in this paper.

## References


1. Borges JL (2004) Three-dimensional analysis of embankments on soft soils incorporating vertical drains by finite element method. Comput Geotech 31(2004):665–676
2. Muktabhant C, Teerawong P, Tengamnuay V (1967) Engineering properties of Bangkok subsoils, Proc. of Southeast Asian Conference on Soil Engineering, Bangkok, pp1–7
3. Moh ZC, Nelson JD, Brand EW (1969) Strength and deformation behaviour of Bangkok clay, Proc. of 7th Int. Conf. on Soil Mechanics and Foundation Engineering, Mexico, pp287–295
4. Eide O (1968) Geotechnical engineering problems with soft Bangkok clay on the Nakhon Sawan highway project, Pub. No 78, Norwegian Geotechnical Institute, Norway
5. Eide O (1977) Exploration, sampling and in-situ testing of soft clay in the Bangkok area, Proc. of International Symposium on Soft Clay, Thailand, pp122–137
6. Bergado DT, Anderson LR, Miura N, Balasubramaniam AS (1996) Soft Ground Improvement, American Society of Civil Engineers, 427 pages
7. Ma L, Shen S-L, Luo C-Y, Ye-Shuang Xu (2011) Field evaluation on the strength increase of marine clay under staged construction of embankment. Mar Georesour Geotechnol 29(4):317–332
8. Indraratna B, Sathananthan I, Rujikiatkamjorn C, Balasubramaniam AS (2005) Analytical and numerical modeling of soft soil stabilized by prefabricated vertical drains incorporating vacuum preloading. Int J Geomech 5(2) ISSN 1532-3641/2005/2-114-124
9. Iyathurai S (2005) Modelling of vertical drains with smear installed in soft clay, Doctor of Philosophy thesis, School of Civil, Mining and Environmental Engineering, University of Wollongong, 2005. https://ro.uow.edu.au/theses/1924
10. Indraratna B, Zhong R, Rujikiatkamjorn C (2016) An analytical model of PVD-assisted soft ground consolidation. Adv Transp Geotech. https://doi.org/10.1016/j.proeng.2016.06.162
11. Indraratna B, Bamunawita C, Redana I, McIntosh G (2003) Modelling of prefabricated vertical drains in soft clay and evaluation of their effectiveness in practice. http://ro.uow.edu.au/engpapers/394
12. Lin DG, Chang KT (2009) Three-dimensional numerical modelling of soft ground improved by prefabricated vertical drains. Geosynth Int 16(5):339–353. https://doi.org/10.1680/gein.2009.16.5.339
13. Shen S-L, Chai J-C, Hong Z-S, Cai F-X (2005) Analysis of field performance of embankments on soft clay deposit with and without PVD improvement. Geotext Geomembr 23(2005):463–485
14. Hird CC, Pyrah IC, Russell D, Cinicioglu F (1995) Modeling the effect of vertical drains in two-dimensional finite element analyses of embankments on soft ground. Can Geotech J 32:795–807
15. Rujikiatkamjorn C, Indraratna B, Chu J (2007) 2D and 3D Numerical Modeling of Combined Surcharge and Vacuum Preloading with Vertical Drains. Int J Geomech 8(2)
16. Borges JL (2004) Three-dimensional analysis of embankments on soft soils incorporating vertical drains by finite element method. Comput Geotech 31:665–676
17. Bernie Mills and Jared McGinn (2010) Design, construction, and performance of a highway embankment failure repaired with tire-derived aggregate. Transportation Research Record: Journal of the Transportation Research Board, No. 2170, Transportation Research Board of the National Academies, Washington, 2010, pp. 90–99
18. Wong YKJ (2013) Comparison of drainage line elements in PLAXIS 2D and 3D applied in consolidating marine clay deposits. SEIC 2013, IP 6-63
19. Mills B, El Naggar H, Valsangkar A (2015) North American overview and a Canadian perspective on the use of tire-derived aggregate in highway embankment construction. Chapter 22, Ground Improvement Case Histories, Volume 2, Elsevier Ltd.
20. Manual on estimating soil properties for foundation design (1990), prepared by Cornell University, Ithaca, New York, EPRI EL-6800
21. Ouyang Z, Mayne PW (2018) Effective friction angle of clays and silts from piezocone penetration test. Can Geotech J 55:1230–1247
22. Bo MW, Arulrajah A, Sukmak P, Horpibulsuk S (2015) Mineralogy and geotechnical properties of Singapore marine clay at Changi. Jpn Geotech Soc Soils Foundations 55(3):600–613
23. Bowles JE (1988) Foundation analysis and design. 4th Edition, McGraw Hill
24. ASTM D1777-96(2019), Standard test method for thickness of textile materials, ASTM International, West Conshohocken, 2019, www.astm.org
25. ASTM D5199-12(2019), Standard test method for measuring the nominal thickness of geosynthetics, ASTM International, West Conshohocken, 2019, www.astm.org
26. ASTM D4716/D4716M-20, Standard test method for determining the (in-plane) flow rate per unit width and hydraulic transmissivity of a geosynthetic using a constant head, ASTM International, West Conshohocken, 2020, www.astm.org
27. ASTM D4751-20b, Standard Test methods for determining apparent opening size of a geotextile, ASTM International, West Conshohocken, PA, 2020, www.astm.org
28. ASTM D4491 / D4491M-21, Standard Test Methods for Water Permeability of Geotextiles by Permittivity, ASTM International, West Conshohocken, 2021, www.astm.org
29. Gaber M, Kasa A, Abdul-Rahman N, Alsharef J (2019) Simulation of sequential construction of embankments on reinforced soft clay foundation. Indian Geotech J 49(2):224–231
30. Sexton BG, McCabe BA (2013) Numerical modelling of the improvements to primary and creep settlements offered by granular columns. Acta Geotech 8:447–464
31. PLAXIS material models 2019