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Numerical Simulation and Field Measurement Validation of Road Embankment on Soft Ground Improved by Prefabricated Vertical Drains: A Comparative Study

Rufaizal Che Mamat [1,*], Azuin Ramli [1,*], Shabir Hussain Khahro [2,*] and Nur Izzi Md Yusoff [3]

[1] Centre of Green Technology for Sustainable Cities, Department of Civil Engineering, Politeknik Ungku Omar, Jalan Raja Musa Mahadi, Ipoh 31400, Perak, Malaysia
[2] College of Engineering, Prince Sultan University, Riyadh 11586, Saudi Arabia
[3] Department of Civil Engineering, Faculty of Engineering & Built Environment, Universiti Kebangsaan Malaysia (UKM), Bangi 43600, Selangor, Malaysia
* Correspondence: rufaizal.cm@gmail.com (R.C.M.); azuin.ramli@puo.edu.my (A.R.); shkhahro@psu.edu.sa (S.H.K.)

Abstract: This article provides a comparative knowledge of predicted and measured settlements of road embankments with prefabricated vertical drains (PVDs). The emphasis of this study was to investigate and back-analyse the ratio of horizontal permeability in an undisturbed area to a smeared zone, which affects the behaviour of vertical drains. Two models of PVDs in soft ground were developed by utilising a plane strain 2D numerical approach based on the equivalent permeability. Suggestions for the improvement of numerical accuracy of the soft ground beneath road embankments have been made in regards to the obtained results. The employment of the equivalent horizontal permeability in numerical analysis produced significantly similar results to those of the measured values. Furthermore, a smear effect permeability ratio of 300 produced a considerably accurate result with a model based on the equivalent horizontal permeability and measured data. Lastly, the smear effect ratio of 6 using the equivalent horizontal permeability approach was employed in order to predict the behaviour of vertical drains in the soft grounds under road embankments.

Keywords: soft ground; prefabricated vertical drains; road embankment; back-analysis; numerical analysis; field measurement

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1. Introduction

Soft grounds have presented themselves as a challenge to engineers due to their low bearing capacity, high compressibility, low shear strength, and permeability. The existence of such a challenging soil leads to massive settlements, large differential post-construction settlement, large lateral flow, and slope failure, which subsequently require a tedious revaluation process [1]. The long-established solution for soft ground improvement includes chemical admixture and sand drains. Over the past century, there has been a significant upsurge in soft-ground enhancement methods. Since geosynthetics were introduced, the rapid development of infrastructural projects on soft soils with prefabricated vertical drains (PVDs) has been observed. A substantial number of studies have reported the effects of applied PVDs to mitigate the problem of soft ground. The installation of PVDs is the most economical technique of soft ground improvement since it can accelerate the settlement of the ground foundation and enhance the safety factor of the embankment slope. This technique has recently been considered the most favourable as a result of the mentioned advantages. Several factors affecting the progress of consolidation with vertical drains were examined over the last few decades with the aim of improving the accuracy of the numerical prediction method. The main focus of investigations conducted by several researchers was on the smear effect on consolidation processes (e.g., [2–4]). If the smear effect was not considered in numerical methods, the time-settlement curve would be far below the measured settlement response [5]. The primary goal of the smear effect is the identification of the parameters employed in order to obtain the equivalent horizontal and vertical permeability. Traditionally, the ratio of horizontal permeability in the undisturbed area to the smeared zone (η) was determined based on the typical literature values. Researchers and designers have been faced with challenges when identifying such parameters, as laboratory processes and experiments prove to be intricate and complex [5,6]. The proposed η value in the literature is between 1.3 and 10, as reported by a number of researchers [7,8]. It has been reported that the best way to estimate these values is through back-analysation of the local case histories [6].

Despite this method being widely used, consideration of the solution to equivalent permeability has received little attention [7–9]. In this regard, it can lead to prediction accuracy problems. In addition, several studies have also used the back analysis method of ground settlement behaviour, but the comparison of the results obtained from the finite element method and measurements in the field is rarely confirmed [10,11]. Therefore, it is essential to use both methods at once to confirm that the smear effect parameters that were previously studied are adequate.

Recently, Nghia-Nguyen et al. found that the back-analysis method needs to be adjusted with the analytical approach to determine the permeability ratio in plane strain conditions [12]. Lin et al. used the back-analysis method to predict the settlement without considering the equivalent permeability in developing the finite element model [13]. They also emphasized and suggested that equivalent permeability and smear effect be considered for future studies. On the other hand, Hiep and Chung did not succeed in accurately predicting excess settlement using back-analysis and finite element methods because they did not consider several geotechnical parameters, including the appropriate equivalent permeability.

In principle, the consolidation of soft soil around the PVD is axisymmetric. In an effort to reduce computational time, plane strain conditions are often used for finite element analysis. For this purpose, several researchers have developed a technique for calculating the equivalence between plane strain and axisymmetric conditions [4,14]. Due to the fact that some of the previously reported studies neglected the equivalence parameter and the smear effect, consequently, all their predictions underestimated the settlement compared to the field measurement data.

This study seeks to address this issue by applying back-analysis in order to determine various smear effect parameters with equivalent permeability. In doing so, the back-analysis of numerical methods is compared with field measurements in order to evaluate the performance of a constructed embankment on soft ground. Finally, suggestions are made to increase the accuracy of the predicted soft-soil behaviour under the embankments stabilized by PVDs.

2. Case Study

2.1. Site Project Description

The site of the road embankment used in this study is located in Yan, Kedah, which connects the North-South Expressway across to the Strait of Malacca and Pulau Bunting (Figure 1). The constructed 18.2 km project is divided into two phases. The first phase connects the federal roads to the Pulau Bunting Bridge, which has a length of 14.9 km. Phase 2 connects the Gurun–Jeniang Road to the Guar Nenas road and comprises a total distance of 3.3 km. Most of these construction project sites are agricultural areas for rice cultivation. The location of the project was found to have a high groundwater level, which prompted the designers to propose three types of ground improvement techniques, viz PVDs, piles, removal and replacement. A total of 65 separate locations were determined for the installation of PVDs. This covers a total track length of about 12 km. Numerical analysis was performed on the embankment in CH 1977, as shown in Figure 2, where the ground foundation was installed with PVDs, which includes construction along 120 m.

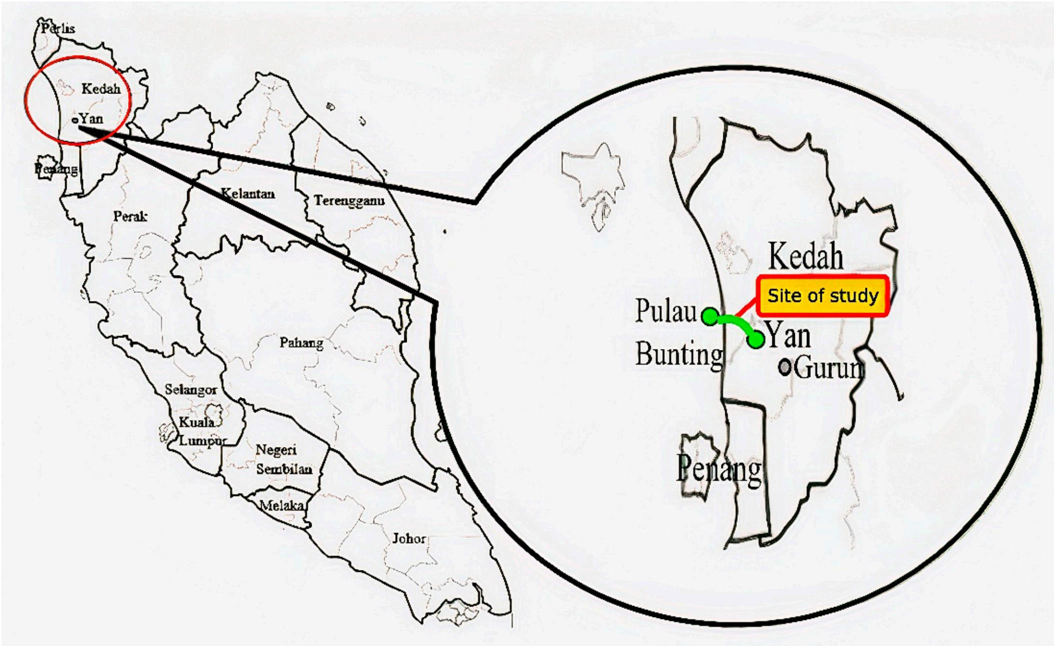


Figure 1. Location of study.

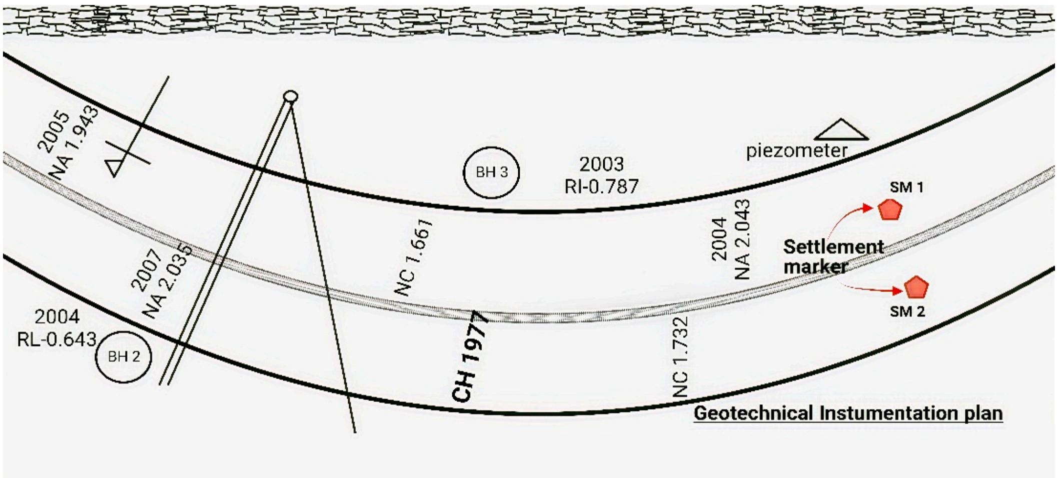


Figure 2. Instruments installation plan in CH 1977.

2.2. In Situ and Laboratory Investigation

Several in situ and laboratory investigations were conducted in order to identify the soil profile and evaluate the soil's physical and engineering properties. In order to achieve a detailed design, 37 boreholes were drilled at the proposed location using rotary-drilling techniques. In situ tests such as standard penetration tests (SPT) and shear-vane tests were carried out in 24 and 21 locations, respectively, to provide data on geotechnical properties. Through the utilisation of SPT, thick-walled sample tubes were driven into the ground to depths ranging between 150 mm and 450 mm in order to obtain undisturbed samples. During the geotechnical investigation, boreholes were prepared, and laboratory tests were carefully performed for factors such as water content, density, specific gravity, Atterberg limits, oedometer and triaxial tests.

2.3. Subsoil Profile

Deposits in the study area are categorised as soft soils consisting of five layers, which include four types of soil. These include clay silt, silty clay, soft clay, and clayey silt. The first layer under the sand cushion is clay silt with a thickness of 2 m, followed by 4 m of silty clay as the second layer. The third and fourth layers are 3m of soft clay and 9m of silty clay, respectively. The last layer is clayey silt with a thickness of about 18 m. The physical and engineering properties of soft soil deposit engineering are presented in Table 1. Based on the results of Atterberg's limit test, the second layer to the fourth layer possessed the highest plasticity. The basic properties of the subsoil, such as saturated unit weight, are between 13.6 and 15.8 kN/m^3. The subsoil permeability coefficient from the oedometer test ranged from 0.045 to 0.061 m/year. Additionally, N-values ranged from 1 to 14 on the first to fourth layers. The 2.3 m groundwater level was near the ground surface.

Table 1. Geotechnical properties.

Layer No.	Soil Strata	e_0	G_s	c_c	p_c (kPa)	k (m/year)
1	Clay silt	2.17	2.54	0.983	21	0.061
2	Silty clay	2.33	2.54	1.02	18	0.074
3	Soft clay	2.67	2.52	0.952	21	0.056
4	Silty clay	3.00	2.52	1.07	17	0.052
5	Clay silt	3.09	2.51	0.944	22	0.045

Note: e_0 = initial void ratio; G_s = specific gravity; c_c = compression index; p_c = pre-consolidation stress; k = permeability.

2.4. Construction Procedure

The construction work program of the embankment sites involved work platform preparation, installation of PVDs and the compaction of filler materials. The topsoil was cleaned and removed for the preparation of the work platform. The work platform preparation required the topsoil to be cleaned and removed. The first sand layer cushion of 0.5 m consisted of two layers of uniform sand, with the first layer of compacted sand of 0.3 m, compacted with compacting machines. Prior to the installation of PVDs, 1500 kN/m of axial stiffness was applied to the first layer of sand cushion with the use of a non-woven geotextile. They were installed with triangular patterns on a hollow steel mandrel and were driven into the ground by a stitcher attached to an excavator carrier. The outside and inner diameter of the mandrel were 127 mm and 110 mm, respectively, while the anchor shoes were discs with a diameter of about 127 mm. The spacing of PVDs were such that the centres were 1m apart, while the proposed depth was 15 m. This was aimed to assist the acceleration of the consolidation process by ensuring that groundwater discharge is effective. The second layer of 0.2 m thick sand was placed over the geotextile in order to allow the progression of a consolidation process. The 1m thickness of the embankment was constructed as non-homogenous soil was adopted as filler materials and compacted with compaction machinery. The geometry of the embankment had a crest width of 11 m with a 1V: 1H side-slope ratio.

2.5. Field Monitoring

Field monitoring of the soft ground behaviour was performed for 177 days after the embankment construction. During the process of preloading, settlement markers were utilised in order to monitor the localised ground movements. In particular, two settlement plates were installed on the left and right sides of the embankment (SM1 and SM2) with a depth of 0.5m from the subsoil surface. The settlement plate was installed before the start of the construction of the embankment and was welded into the base of the steel riser pipes. The location of the position of the instruments and the arrangement of PVDs on the cross-section of the embankment is displayed in Figure 3.

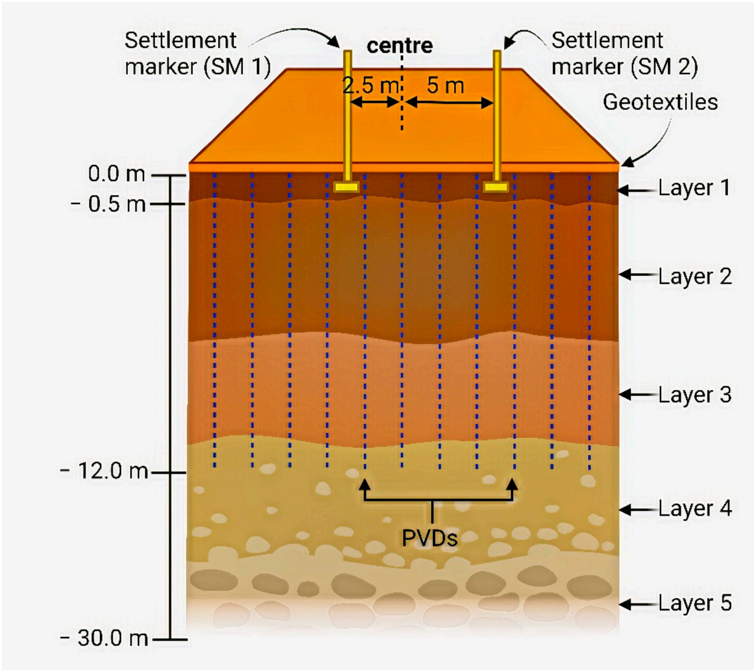


Figure 3. Ground profile and layout of field instrumentation.

2.6. Numerical Analysis

The embankment structure was modelled via a commercially available finite element code, Plaxis 2D version 8. This was employed for settlement behaviour analysis. The automatic generation of numerical solutions provided options for global and local mesh refinements. In the current study, a medium-coarse mesh arrangement and triangular plane strain model with 15 nodes were used for analysis, as presented in Figure 4. The numerical method discretisation with 318 elements and 2633 nodes was generated. The staged-construction process adopted in the numerical analysis is as follows:

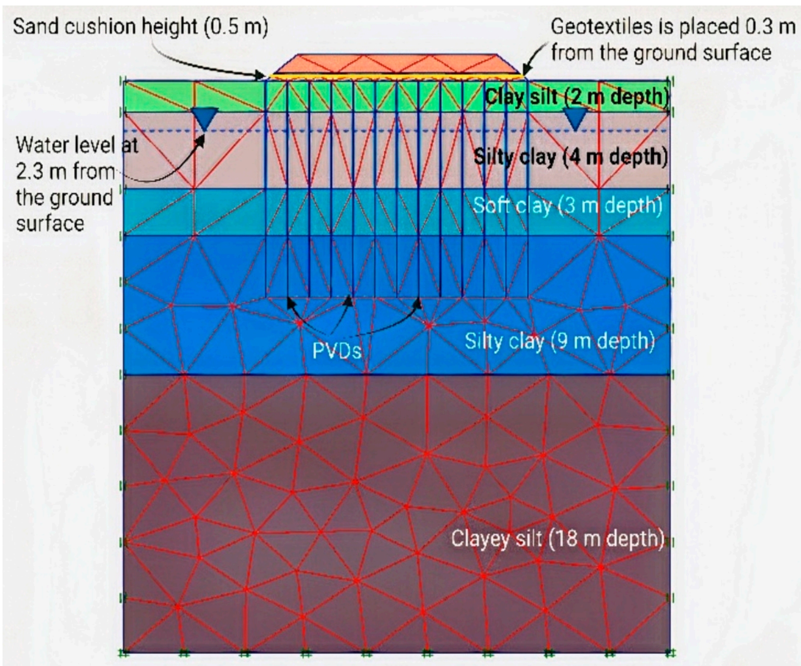


Figure 4. Finite element model with mesh structure.

1. Preparation of sand cushion as a drainage layer (7 days);
2. PVD installation (7 days);
3. Short consolidation (45 days);
4. Construction of the road embankment (7 days);
5. Long-term consolidation (about 177 days).

The fixture of standard boundary conditions was applied along the bottom and sides of the model structure. The bottom layer of soil beneath the embankment is clayey silt and is stiff. Therefore, the horizontal and vertical deformations at the bottom were set to zero (u_x = u_y = 0). The horizontal component of the displacement was fixed (u_x = 0) on the vertical boundary far from the embankment. A sensitivity study was performed to establish the length of the left and right boundary, and 16 m was ultimately chosen. A boundary condition of zero pore pressure was given to the top of the soft clay layer, making it permeable. The PVD models were made using the drain elements encoded into the Plaxis 2D software. The excessive pore pressure was set to zero in the drain element to perform consolidation analysis.

2.7. Modelling an Equivalent Plane Strain

The methods used for modelling soft grounds treated by PVD in the plane-strain-analysis have been reported in 4 categories. The first category, as Hird et al. and Chai et al. have summarised, is the degree of horizontal consolidation in the plane strain of a unit cell, which has been aligned with the degree of radial consolidation in the axisymmetric model [4,14]. The 1-D drainage element applied in the plane strain analysis may reveal that excessive pore pressure in PVD is equal to zero (u = 0). The second category, which is the effect of the smear zone, is negligible, as the PVD discharge capacity was assumed to be infinite when the quadrilateral macro-element was adopted in numerical methods [15,16]. The third category is an unrealistic method in which the model is developed without improvement through an equivalent vertical permeability [14]. In numerical analyses where PVD was modelled by solid elements of equivalent width and equivalent permeability to soft grounds in the plane strain, a fourth category is frequently applied [11]. In the following study, the fourth category was adopted to develop two numerical models. Model 1 is characterised by an equivalent horizontal permeability, while Model 2 is based on the combination of the equivalent horizontal and vertical permeabilities. Previous studies reveal that the consolidation around PVDs is axisymmetric [17]. However, most numerical analysis methods are based on plane strain. Thus, the equivalent permeability of soft ground needs to be converted from the axisymmetric unit cells to the plane strain in order to allow for numerical analysis [17,18]. Hird et al. proposed the following model, in which the equivalent horizontal permeability's strain is converted from axisymmetric to plane strain before numerical simulation was performed [4]. The model is as follows:

$$k_{hpl} = \frac{2}{3} \cdot \frac{B^2}{{D_e}^2} \cdot \frac{1}{\mu} . k_h, \tag{1}$$

In this equation, B is the width of the plane strain unit cell, D_e is the diameter of the unit cell, μ is a factor of the PVD geometry, and k_h is the horizontal permeability of the subsoil. Hansbo [2] indicated that the value of μ can be obtained with the following equation:

$$\mu = \ln(n/s) + (k_h/k_s)\ln s - 0.75, \tag{2}$$

In this equation, n is the ratio of the diameter of the unit cell and drainage, s is the ratio of the diameter of the smear zone and drainage, k_s is the horizontal permeability in the smeared zone, and d_w is the diameter of the smeared zone and the vertical drain, respectively. The diameter unit cell, D_e, for PVDs installed with the triangular pattern is calculated via Equation (4). Rixner et al. [19] expressed the equivalent drain diameter (d_w) using the following equation:

$$d_w = (w + t)/2, \tag{3}$$

$$D_e = 1.05d, \tag{4}$$

such that w and t represent the width and thickness of the PVD, respectively, and d is the spacing of the PVDs. The diameter of the smear zone, d_s, is calculated by the equations proposed by Hird and Moseley [20]:

$$d_s = 1.6d_w, \tag{5}$$

Furthermore, Chai et al. [14] expressed the equivalent vertical permeability (k_{ve}) equation as follows:

$$k_{ve} = \left(1 + \frac{2.5l^2}{\mu D_e{}^2}\frac{k_h}{k_v}\right)k_v, \tag{6}$$

where l and k_v are the drainage length and the vertical permeability of the subsoil, respectively.

The parameters for PVD are listed in Table 2. In order to fit the measured data, the smear effect permeability ratio was varied, and the equivalent horizontal and vertical permeability values were back-calculated and are presented in Tables 3 and 4, respectively. As Hansbo [21] has suggested, when k_h is equal to k_v, then the η (k_h/k_s) values are close to unity.

Table 2. Parameters of equivalent horizontal and vertical permeability.

Parameters	Symbol	Value
The width of the unit cell plane strain, m	B	0.75
The equivalent drainage diameter, m	d_w	0.052
Smear zone diameter	d_s	0.176
Horizontal permeability ratio of undisturbed soil and smear, (k_h/k_s)	η	3
The ratio of the diameter of the smear zone and drainage, (d_s/d_w)	s	3.38
Unit cell diameter, m	D_e	1.58
The ratio of the diameter of the unit cell and drainage, (D_e/d_w)	n	30.3

Table 3. An equivalent horizontal permeability in m/day.

$\eta = k_h/k_s$	Clay Silt (Layer 1)	Silty Clay (Layer 2)	Soft Clay (Layer 3)	Silty Clay (Layer 4)	Clay Silt (Layer 5)
1	3.38×10^{-6}	3.43×10^{-6}	2.80×10^{-6}	2.88×10^{-6}	2.80×10^{-6}
3	1.62×10^{-6}	1.64×10^{-6}	1.34×10^{-6}	1.38×10^{-6}	1.34×10^{-6}
6	9.59×10^{-7}	9.73×10^{-7}	7.92×10^{-7}	8.17×10^{-7}	7.95×10^{-7}
9	6.75×10^{-7}	6.85×10^{-7}	5.60×10^{-7}	5.75×10^{-7}	5.60×10^{-7}
10	6.08×10^{-7}	6.17×10^{-7}	5.04×10^{-7}	5.18×10^{-7}	5.04×10^{-7}
20	2.70×10^{-7}	2.74×10^{-7}	2.24×10^{-7}	2.30×10^{-7}	2.24×10^{-7}
30	2.16×10^{-7}	2.19×10^{-7}	1.79×10^{-7}	1.84×10^{-7}	1.79×10^{-7}
40	1.62×10^{-7}	1.64×10^{-7}	1.34×10^{-7}	1.38×10^{-7}	1.34×10^{-7}
100	6.62×10^{-8}	6.71×10^{-8}	5.50×10^{-8}	5.64×10^{-8}	5.49×10^{-8}
200	3.38×10^{-8}	3.43×10^{-8}	2.80×10^{-8}	2.88×10^{-8}	2.80×10^{-8}
300	2.16×10^{-8}	2.19×10^{-8}	1.79×10^{-8}	1.84×10^{-8}	1.79×10^{-8}

Table 4. An equivalent vertical permeability in m/day.

$\eta = k_h/k_s$	Clay Silt (Layer 1)	Silty Clay (Layer 2)	Soft Clay (Layer 3)	Silty Clay (Layer 4)	Clay Silt (Layer 5)
1	3.46×10^{-4}	3.51×10^{-4}	2.87×10^{-4}	2.94×10^{-4}	2.87×10^{-4}
3	2.09×10^{-4}	2.12×10^{-4}	1.73×10^{-4}	1.78×10^{-4}	1.73×10^{-4}
6	1.34×10^{-4}	1.36×10^{-4}	1.11×10^{-4}	1.14×10^{-4}	1.11×10^{-4}
9	1.01×10^{-4}	1.02×10^{-4}	8.36×10^{-5}	8.58×10^{-5}	8.36×10^{-5}
10	9.34×10^{-5}	9.48×10^{-5}	7.75×10^{-5}	7.95×10^{-5}	7.75×10^{-5}
20	5.68×10^{-5}	5.77×10^{-5}	4.71×10^{-5}	4.84×10^{-5}	4.71×10^{-5}
30	4.32×10^{-5}	4.39×10^{-5}	3.59×10^{-5}	3.68×10^{-5}	3.59×10^{-5}
40	3.61×10^{-5}	3.67×10^{-5}	3.00×10^{-5}	3.08×10^{-5}	3.00×10^{-5}
100	2.28×10^{-5}	2.31×10^{-5}	1.89×10^{-5}	1.94×10^{-5}	1.89×10^{-5}
200	1.82×10^{-5}	1.85×10^{-5}	1.51×10^{-5}	1.55×10^{-5}	1.51×10^{-5}
300	1.66×10^{-5}	1.69×10^{-5}	1.38×10^{-5}	1.42×10^{-5}	1.38×10^{-5}

2.8. Modelling Parameters Properties

A number of constitutive models that can represent the behaviour of geomaterials were observed in numerical solutions. In this study, two constitutive modelling schemes have been adopted in order to ensure that the stress-strain behaviour of the soil can be effectively represented. The filler material of the embankments and sand cushions was modelled using the Mohr–Coulomb (MC) model, while the soft ground was modelled by the soft soil (SS) model. In recent years, both schemes of this model have been widely adopted for numerical analysis by researchers investigating embankments with PVDs [10,22,23]. The MC model requires five parameters: cohesion (c), friction angle (φ), Poisson ratio (ν), dilatancy angle (ψ) and elastic modulus (E). In contrast, the SS model is defined by two parameters, i.e., the modified swelling index (κ*) and the modified compression index (λ*). The modelling parameters employed in the numerical analysis are summarised in Table 5.

Table 5. Material properties.

Materials	Type	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	ν	E_{ref} (kN/m^2)	c_{ref} (kN/m^2)	ϕ (°)	ψ (°)	Λ *	κ *
Fill material	D	16.5	18.0	0.3	15,000	10	20	0	-	-
Sand cushion	D	17.0	18.5	0.3	20,000	0	30	0	-	-
Clay silt	UD	13.1	13.6	0.15	-	75	0	0	0.10	0.02
Silty clay	UD	13.3	13.8	0.15	-	43	0	0	0.12	0.03
Soft clay	UD	13.6	14.0	0.15	-	43	0	0	0.11	0.03
Silty clay	UD	14.0	14.5	0.15	-	11	0	0	0.13	0.03
Clay silt	UD	14.5	15.8	0.15	-	10	0	0	0.14	0.03

* γ_{unsat} = unsaturated unit weight; γ_{sat} = saturated unit weight; E_{ref} = reference modulus; c_{ref} = reference cohesion.

2.9. Methodology of Back-Analysis

It was observed that the performance of the embankment stabilised by PVDs was influenced by several factors, such as ground properties, smear effect, and properties of the sand cushion [24,25]. In order to back-calculate the PVD behaviour in the field, subsoil properties and analysis methods should be verified first. As previously mentioned, the smear effect permeability ratio (η) has been proposed with a wide range of values. In back-analysis, the effect of the drain will be overestimated when a value lower than the actual value of the subsoil permeability is employed. Thus, one of the embankment construction sections at the case study location was applied in order to calibrate the smear effect parameters in the numerical procedure. Subsequently, the performance data of the numerical methods of drainage of the fixed soil parameters and PVD were compared and evaluated. As previously discussed, the smear effect is an uncertain factor in drainage behaviour. Since these factors are independent of one another, a set of different values of η is listed in Tables 3 and 4. This was initially determined based on the literature. Subsequently, the equivalent horizontal and vertical permeability were calculated for the development of numerical models.

2.10. Model Performance Evaluation Indices

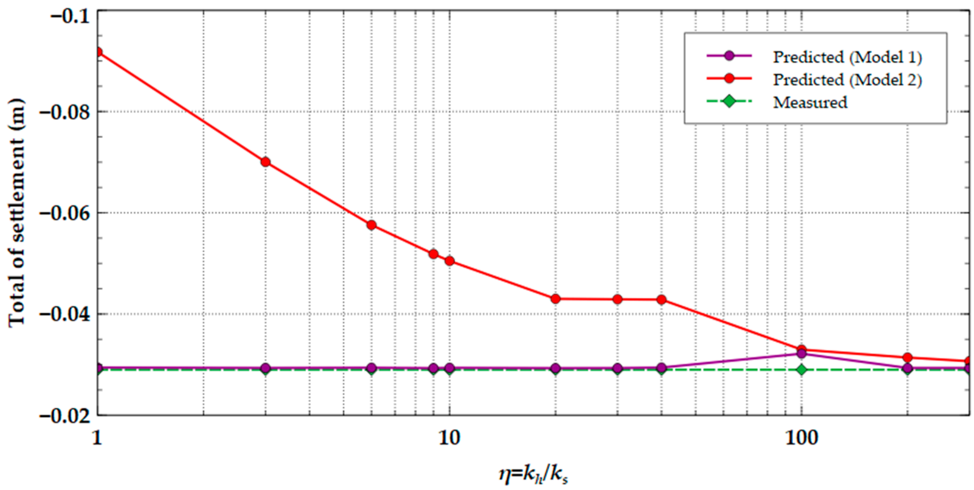
In this study, output data from the numerical analysis were compared to data measured using the consolidation time. Numerical methods have been used to predict the behaviour of embankments located on soft grounds. In order to predict the accuracy of the developed models, the correlation coefficient (R^2) and the average percentage difference ($PD_{average}$) were employed. The value of R^2 was derived from the graph of the equation, where the linear-square fit line fitted the least The $PD_{average}$ was performed on two settlement data, which were predicted and measured based on the following equation:

$$PD_{average} = \left(\frac{\frac{|V_1 - V_2|}{(V_1+V_2)/2} \times 100\%}{n} \right), \quad (7)$$

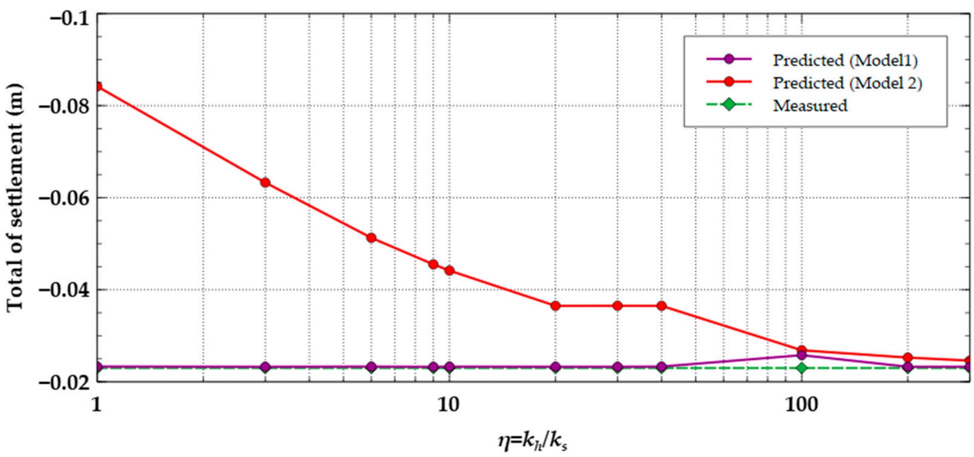
where V_1 and V_2 are the predicted and measured data, respectively, and n is the number of data in a series of datasets. The higher the R^2 values and the lower the $PD_{average}$, the higher the accuracy of the predictions of the developed models.

3. Results and Discussion

The purpose of the soft ground embankment analysis was to verify the model parameters of the smear effect using the numerical method. In back-analysing drain behaviour, only parameters related to drainage behaviour were adjusted in order to fit the measured data. In order to fit the measured data, the permeability in the smear zone varied, and the values were back-calculated. The soil permeability values in Tables 3 and 4 were determined from Equations (1) and (6) for the development of numerical simulation models. The simulation results were compared with the measured data in Figure 5 in order to evaluate the smear effect parameter's performance. According to Figure 5a, 0.084 m of settlement was observed after 177 days of consolidation in model 2 with η = 1, while adopting η = 300 may result in a reduction of 0.061 m in consolidation settlement after the same elapsed time, thereby indicating that variations in the smear zone properties can affect the consolidation rate. Thus, it is clear that the η is a key factor in estimating the total of settlements in PVD analysis.



(**a**)



(**b**)

Figure 5. Performance of ground settlement between predicted and measured for 177 days consolidated: (**a**) SM1; (**b**) SM2.

As can be seen from Figure 5a,b, Model 1′s prediction and measurement approximately equalled those of the $\eta = 1$ to $\eta = 300$ cases, thereby providing the best fit to the field-measurement comparisons of Model 2. On the other hand, the graph of Model 1 indicates that the total settlement was consistent with the different values of the smear effect parameters. In contrast to Model 2, the graph shows the existence of a smear effect parameter value of 20, which is a significant decrease. The values were once again relatively consistent until a η value of 40 was reached. The predicted values of total settlement at the η values of 100 were quite similar in both models. Interestingly, these predicted models are found to have a relatively same total of settlement at $\eta = 300$ as the measured values. Furthermore, it must be noted that the predictions made by both values of 300 were relatively close to the measured value. Therefore, with the existence of such alignments in values, it can be concluded that the models have similar disturbance rates. Hence, the magnitude of smear characteristics is also relatively the same as those measured in the smear zones.

In order to assess the performance of numerical model prediction, the R^2 and $PD_{average}$ statistical parameters were employed. As shown in Figure 6, the R^2 was generated from the graph, where the line y = x, in addition to the datasets of the total settlement, were depicted. This was to display a reasonable correlation between predicted and measured values. It is evident from this figure that Model 1′s R^2 exceeds 0.99 at two different locations. As expected, the R^2 values in Model 2 increased with the permeability ratio. However, the η from 20 to 40 has an R^2 of 0.97 and increases after $\eta = 100$. The analysis indicates that η at 300 for the models was found to have a similar and accurate prediction of $R^2 = 0.99$.

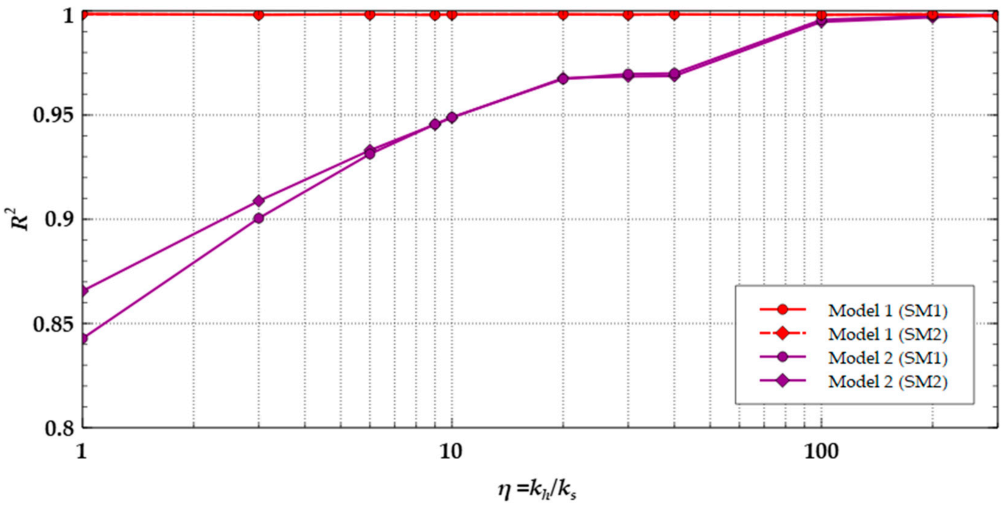


Figure 6. Model performance with R^2.

From the graph in Figure 7, it can be observed that the highest R^2 is 0.9985 at $\eta = 1$ at the right of the embankment, while at $\eta = 300$, the R^2 value of 0.9974 is at its lowest and is at the left of the embankment. Furthermore, the values of η at 6, 10 and 40 had a similar R^2 value of 0.9983 at two different locations. This clearly indicates that the coefficients of these three parameter values are of high reliability and efficiency. Further analysis revealed that $\eta = 1$ for model 2 had the highest $PD_{average}$ of about 90 to 100%, as shown in Figure 8. Nonetheless, the lowest $PD_{average}$ for model 2 was found at $\eta = 200$ at about 5%. On the other hand, the best prediction performance was obtained in model 1, with a $PD_{average}$ of about 1 to 10%. In Figure 9, η is at 6, and 200 had the lowest $PD_{average}$ ranging between 0.75 and 0.82%, respectively. The $PD_{average}$ approaching 0% indicates that the predicted value of the total settlements is approximately similar to the measured values.

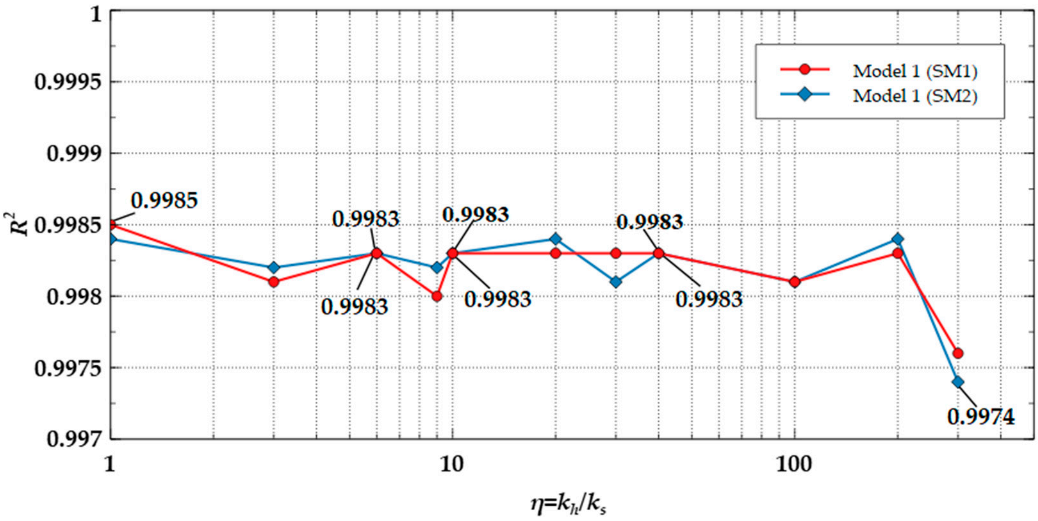


Figure 7. R^2 for Model 1 with different locations.

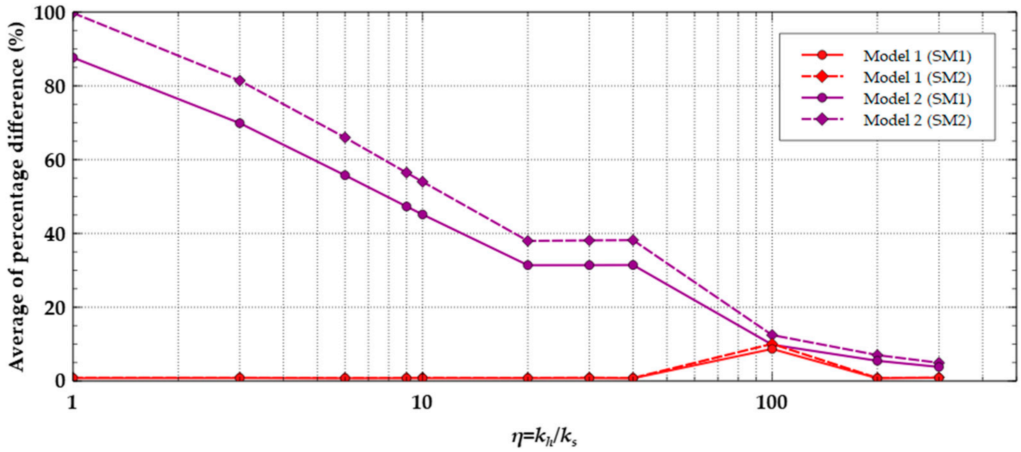


Figure 8. Model performance with the $PD_{average}$.

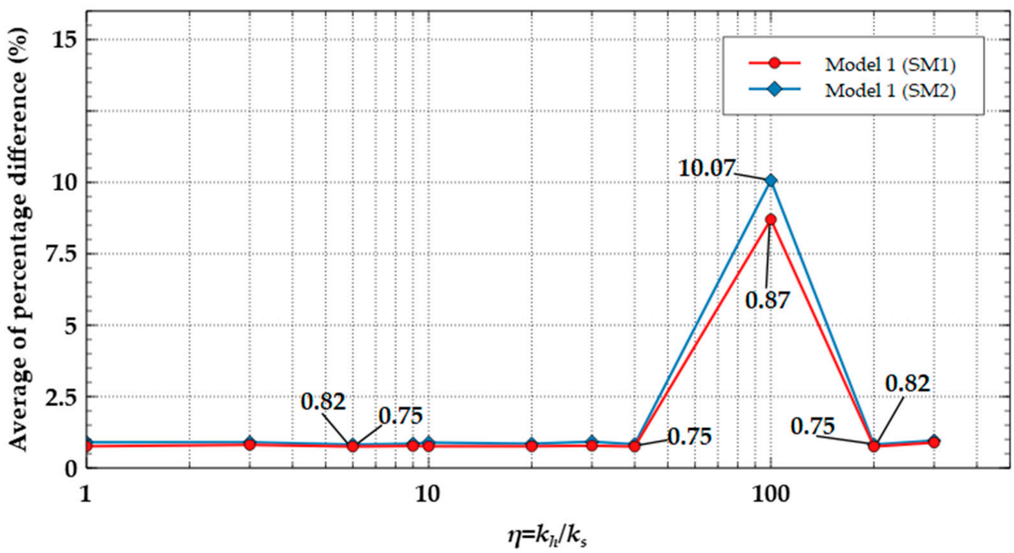


Figure 9. $PD_{average}$ for Model 1 with different locations.

The settlement performance with η = 6 is reported in Figure 10, and it is observed that the analysis resembles the settlement curve well. As previously mentioned, the location distance of the left and right corners from the centre of the embankment were 2.5 and 5 m, respectively. According to Figure 10, 0.03 m of the settlement was observed after 177 days of consolidation on the left of the embankment, while on the right of the embankment, there was a reduction of 0.007 m in the consolidation settlement after the same elapsed time, indicating that the location differences could influence the consolidation rate. In addition, the settlement profile displays that a high total of settlement occurs in the location near the centre of the embankment and decreases at the embankment toe. This finding supports previous research, in which the amount of settlement at the centre of the embankment gradually decreases towards the embankment toe [26]. On the other hand, the process of consolidation settlement can be seen in a rapid manner in the initial stages of the embankment filling, which displays significant differences in the magnitude of settlement. This may be attributed to the compaction energy imposed on the top layer of the soft ground during sand filling, which results in high settlement rates.

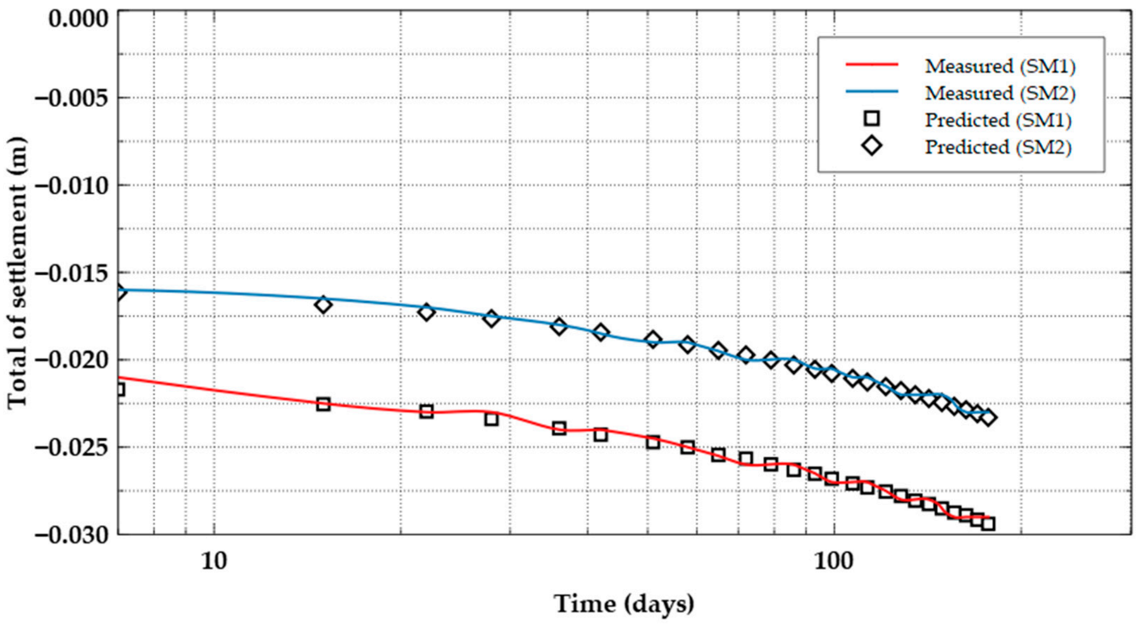


Figure 10. Settlement behaviour performance between two locations.

The comparative results in this study indicate that back-analysis is valuable in decisive parameters related to drainage behaviour. According to the literature, laboratory tests have limitations in determining the permeability values in smear zones. Therefore, it is recommended to back-evaluate the embankment stabilised with PVDs for the existing case history. There are several recommendations for predicting the behaviour improvement of vertical drainage on soft ground. They are as follows:

1. Evaluate the smear effect parameters via existing case history by verification from predicted and measured comparison values;
2. Determine the permeability in the smear zone proposed by Hird et al. as in Equation (1) using the value of η with 6;
3. Simulate the behaviour of vertical drains improved on soft ground by numerical analysis to predict the settlement.

4. Conclusions

The current study proposes a wide range of smear effect parameters for utilisation in practical design, while no comprehensive method has been proposed thus far in order to estimate the properties of smear zones accurately. Therefore, several factors that influence the behaviour of the vertical drains were investigated by numerical methods as well as back-analysis of field performance data. Based on the results of this study, the following conclusions were derived:

1. The permeability in the smear zone is one of the main factors affecting vertical drain behaviour. Accordingly, Equation (1) should be utilized in numerical analysis;
2. The smear effect parameter has a significant effect on drain rate consolidation. A value of $\eta = 6$ was suggested for accurate and reliable predictions in order to determine the equivalent horizontal permeability of embankment settlements despite the fact that the combination of equivalent horizontal and vertical permeability is not significant in numerical analysis; $\eta = 300$ is still recommended in this case. However, all numerical models developed in this study were found to have a total of settlement quite similar to when $\eta = 100$ was employed;
3. According to the performance analysis of the developed numerical model, the total settlement was approximately similar to the increasing η values for the model with equivalent horizontal permeability. In contrast, the model developed with the combination of equivalent horizontal and vertical permeability revealed that increasing the η values tends to reduce the magnitude of predicted settlements;
4. Location observations affect magnitude settlement. In the identification of the settlement profile, it was observed that the total of the settlement was higher when located near the centre than when near the embankment toe;
5. The finite element software was observed to be very useful in predicting the settlement of embankment stabilized with PVDs. The settlement simulated by Plaxis 2D produced considerably accurate representations of settlement during the consolidation process.

A methodology was also recommended in order to predict the behaviour of PVD-improved subsoils. The first step was to evaluate the η by back-analysis based on an existing case history. Next, Equation (1) with $\eta = 6$ was determined. Lastly, the behaviour of soft soil stabilized with PVDs could be predicted using Plaxis 2D.

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