MODELLING OF A VACUUM CONSOLIDATION PROJECT IN VIETNAM

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ABSTRACT

Ground improvement technique by prefabricated vertical drain (PVD) in combination with vacuum preloading is widely used to facilitate consolidation process and reduce residual settlement. However, this technique seem hardly be estimated by both analytical method and numerical method because it has complex boundary conditions (such as vacuum pressure changing with time). Moreover, lateral displacements caused by this technique are also significant problem. Numerical modelling may be an effective design tool to estimate behavior of soft soil treated by this method, however it needs to have a proper calibration of input parameters. This paper introduces a matching scheme for selection of soil/drain properties in analytical solution and numerical modelling (axisymmetric and plane strain conditions) of a ground improvement project by using Prefabricated Vertical Drains (PVD) in combination with vacuum and surcharge preloading. In-situ monitoring data from a case history of a road construction project in Vietnam was adopted in the back-analysis. Analytical solution and axisymmetric analysis can approximate well the field data meanwhile the horizontal permeability need to be adjusted in plane strain scenario to achieve good agreement. In addition, the influence zone of the ground treatment was examined. The residual settlement was investigated to justify the long-term settlement in compliance with the design code. Moreover, the degree of consolidation of non-PVD sub-layers was also studied by means of two different approaches.

Keywords: Ground improvement; Prefacricated vertical drain; Soft clay; Stability; Vacuum consolidation.

1. Introduction

1.1. Site

Due to the rapid development of industrial zones in Vietnam, the infrastructures such as roads need to be built faster. Therefore, it requires a faster soft soil treatment method rather than traditional methods such as vertical sand drain or vertical prefabricated drain (PVD) with surcharge preloading method. Vacuum consolidation for vertical prefabricated drains in combination with surcharge preloading was proved to be the effective method because of the lower embankment surcharge height and shorter construction time than the conventional PVD preloading method (Bergado el al., 1998; Chu et al., 2000; Yan and Chu, 2005; Kelly and Wong, 2009; Rujikiatkamjornand and Indraratna, 2007, 2009, 2013; Indraratna et al., 2005, 2011, 2012; Artidteang et al., 2011; Geng et al., 2012; Chai et al., 2013a, 2013b; Vootipruex et al.,2014; Lam et al., 2015).

Bachiem road project (2017-2018) treated by the PVD vacuum consolidation technique locates at the south of Vietnam. It is 20 kilometers from the Ho Chi Minh City as shown in Fig.1. This area is in the Saigon - Dong Nai river delta (SDRD). The average thickness of the soft clay in this area is more than 20 meters. This soft clay, with low shear strength and high compressibility, may induce large settlement and stability problem for construction (Long et al., 2013).

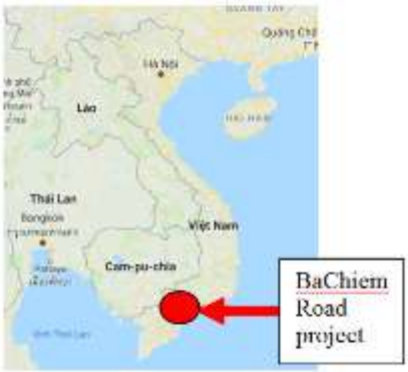

Figure1. The project location

The Bachiem road project was 2.2 km long from Bachiem bridge to Hiep Phuoc Industrial zone. Fig. 2 shows the overall layout of this project. The right hand side of this project is the Sai Gon river and there are still a number of rice fields and agricultural lands surrounding the project indicating that subs soil is very soft.

Figure 2. Project's overall layout

Fig.3 shows a typical section in this project in which numerous monitoring instrumentations were installed to examine soft ground treatment quality and possible damages to nearby houses. A selected section for numerical model is the section with the PVD's length of 15.5m as highlighted in Fig. 3. The PVD's length of this section was limited due to restriction of mandrel height under high electrical voltage lines. This section may cause differential settlements and damages to the future road. Thus, extensive monitoring measurements and ground investigations were carried out. Fig. 4 illustrates the analytical section after the construction.

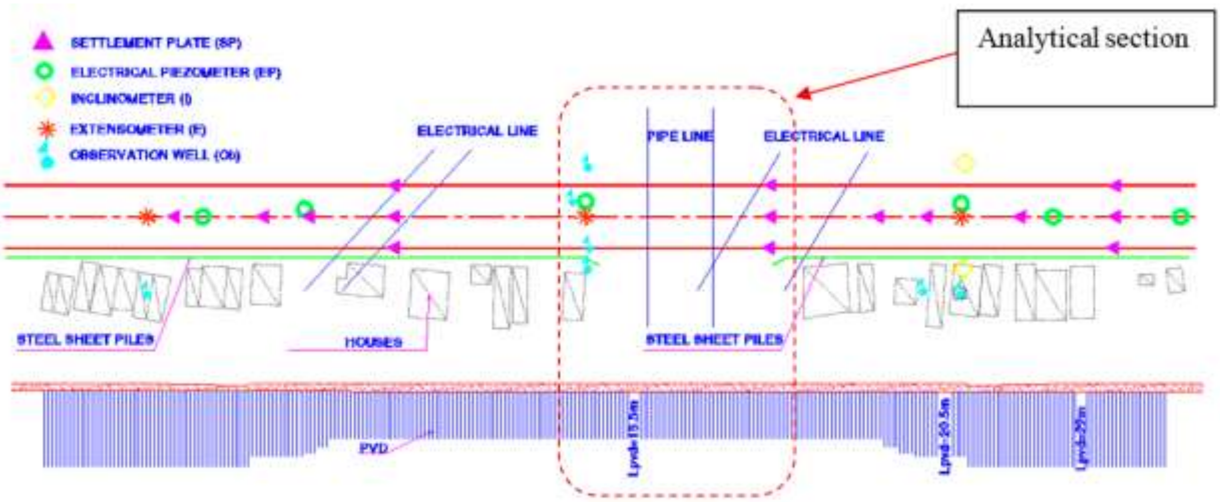

Figure 3. Layout of analytical section

Figure 4. The analytical section after construction

1.2. Soil profile

The soil profile of analysis section consists of 4 sub-soil layers:

- Top crush (0 m to 1 m): the surface soil includes: a thin layer with organic clay, blue-grey, yellow-grey, yellow-brown. This layer is mainly used for: for filling the banks, embankments, floor tiling, and rice fields.
- Layer 1a (1 m to 23 m): A very soft clay with some organic matter and thin sand layers with an average water content of 84.5%, plastic limit (PL) of 37.5%, liquid limit (LL) of 91.3%, unit weight of 14.8 $k\text{N/m}^3$ and SPT-N values from 0 to 1.
- Layer 1b (23 m to 26 m): A soft clay with some organic matters and thin sand layer with an average water content of 78.1%, PL of 36.6%, LL of 89.1%, unit weight of 1.50 $k\text{N/m}^3$, and SPT-N values from 2 to 6.
- Layer 2 (26 m to 29 m) silty or silty sand which is loose to medium dense with SPT-N values from 8-25.

Layer 1a is very soft to soft clay. Without ground treatment, the road construction would be affected due to the post-settlement. Mechanical properties of layer 1a are: *CR* (compression ratio) = 0.35, *RR* (recompression ratio) = 0.042, *OCR* (over consolidation ratio) = 1 to 2 which decreases with depth and c_v (coefficient of vertical consolidation) = 2 m^2/year (for NC stage) as illustrated in Fig.5.

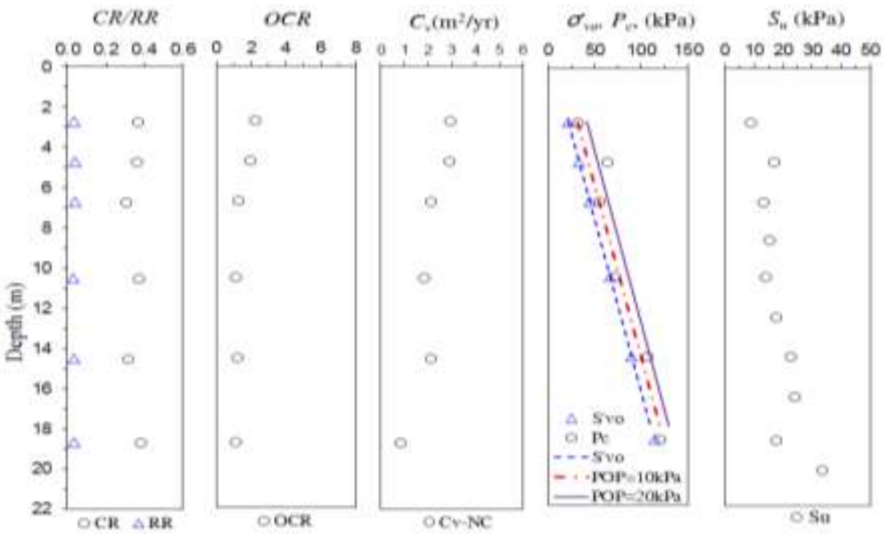

Figure 5. Soil mechanical parameters of layer 1a

Permeability value of layer 1a is a crucial factor for numerical analyses. In fact, the permeabilities change with pressure and void ratio as shown in Fig. 6 and Fig. 7. These figures were derived from odometer tests at different depths from the analytical section. The input permeability value for numerical modeling was simplified as a constant value. Therefore, the lab permeability of $k_v = 1\times10^{-5}$ (m/day) is used as vertical permeability.

For most natural deposits, the horizontal permeability k_h is greater than that in vertical direction, and the ratio (k_h/k_v) varies from 1 to 15 (Jamiolkowski et al. 1983). The horizontal permeability, therefore, was assumed as $k_h = 2k_v = 2\times10^{-5}$ (m/day). Initial void ratio was simplified as a constant input value with $e_o = 2$ for layer 1a as shown in Fig.7.

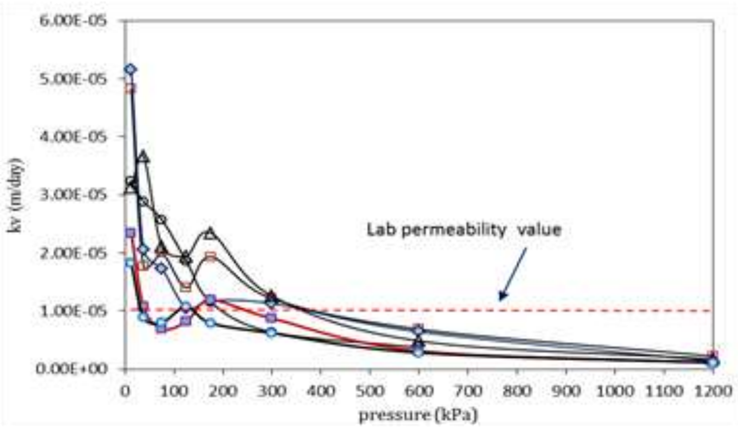

Figure 6. Permeability with pressure changes

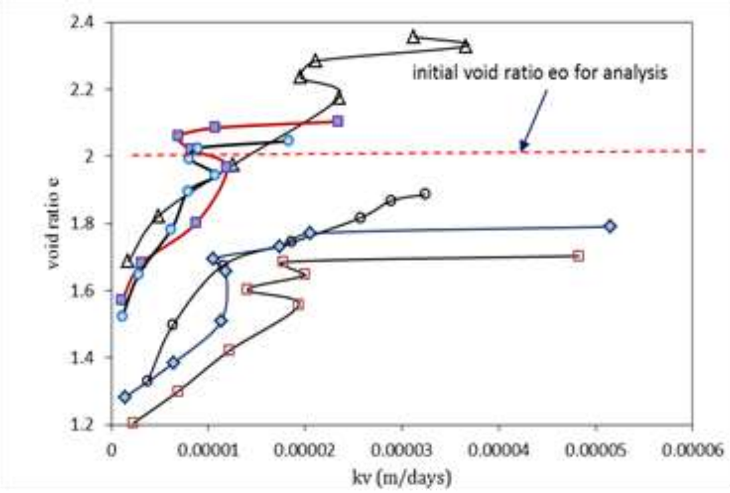

Figure 7. Permeability with void ratio changes

1.3. PVD properties

PVDs were installed in a triangle pattern with spacing of $S = 1$(m). Thus, the equivalent influential diameter is $D_e = 1.05S = 1.05$(m) (Baron. 1974). The PVDs can be simplified into circular shape with equivalent drain's dimension of $d_w = \frac{2(a+b)}{\pi} = 0.066$(m) (Rixner et al 1986). The mandrel dimension was 0.06m×0.12m ($w\times l$). Thus, the equivalent mandrel dimension in circular shape were $d_m = 2\sqrt{\frac{wl}{\pi}} = 0.096$ (m). Moreover, the diameter of smear zone is $d_s = \frac{(5\div6)}{2}d_m = 0.24\div0.29$(m)

(Rixner et al 1986). The value of $d_s = 0.25$(m) was adopted in this study. Plus, well resistant with very high permeability of the drain $k_w = 100$(m/day) was not considered in this model.

2. Methodology

Matching scheme is proposed in order to derive proper properties of soil/drain used in design stage. It can be done by following 3 steps:

1- Generating consolidation problem of PVD by the analytical solution (Jie Han, 2015) with initial soil/drain properties. It is noted that the analytical solution by Hansbo (1981) modified by Jie Han (2015) is used for PVD design of surcharge preloading technique. It may also be appropriately applied for time-dependent loading and vacuum consolidation technique.

2- Simulating this consolidation problem in a single unit cell under axisymmetric condition with similar initial soil/drain properties. Permeability ratio of undisturbed clay to smear clay (k_h / k_s) is calibrated by the settlement of numerical model, the analytical solution, and the actual site data. The appropriate soil/drain properties will be adopted in subsequent 2D numerical model.

3- Modeling complete consolidation case under plane strain (2D) condition. It is to verify both vertical and horizontal displacements with field data.

3. Results and discussion

3.1. Generating analytical solution

A simplified method by Jie Han (2015) was employed to account for the loading steps. This method converted the earlier degree of consolidation at the time t_i and pressure p_i to the degree of consolidation U_j under new pressure p_j with an equivalent time $t_j = t_i + \Delta t$ in which Δt is an additional time calculated using the total time. Further, vacuum pressures are approximately considered as a loading which can be added with the surcharge loading. Initial soil properties taken from Fig. 5 are summarized in Table 1. In addition, the horizontal consolidation coefficient (c_h) was assumed to be double the vertical value. In other words, the c_h value of 4 m^2/year is adopted. It is noted that the layer top-crush and silty sand layer (layer 2) were neglected in this analytical solution due to their low deformation.

Moreover, the actual loading sequence of the analytical section presented in Fig. 8 was carried out with two-step loadings. It was simplified as a single-ramp loading. In addition, vacuum pumps gained the maximum vacuum pressure of -60 kPa for only 5 days.

Fig. 9 demonstrates a comparison of surface settlements by the analytical solution and the field data with the permeability ratio of $k_h / k_s = 3$. The analytical results are in good agreement with the field measurements. Therefore, the initial soil properties and the permeability ratio of $k_h / k_s = 3$ are then applied for axisymmetric numerical modeling of the subsequent step.

Table 1

Initial soil properties for the analytical model

Layer	Depth (m)	RC	RR	p_c (kPa)
1a +PVD	1-3	0.35	0.042	31.6
1a+PVD	3-7	0.35	0.042	49
1a+PVD	7-11	0.35	0.042	72.7
1a+PVD	11-15	0.35	0.042	95.4
1a	15-19	0.35	0.042	118.6
1a	19-23	0.35	0.042	141.8
1b	23-26	0.23	0.023	156.3

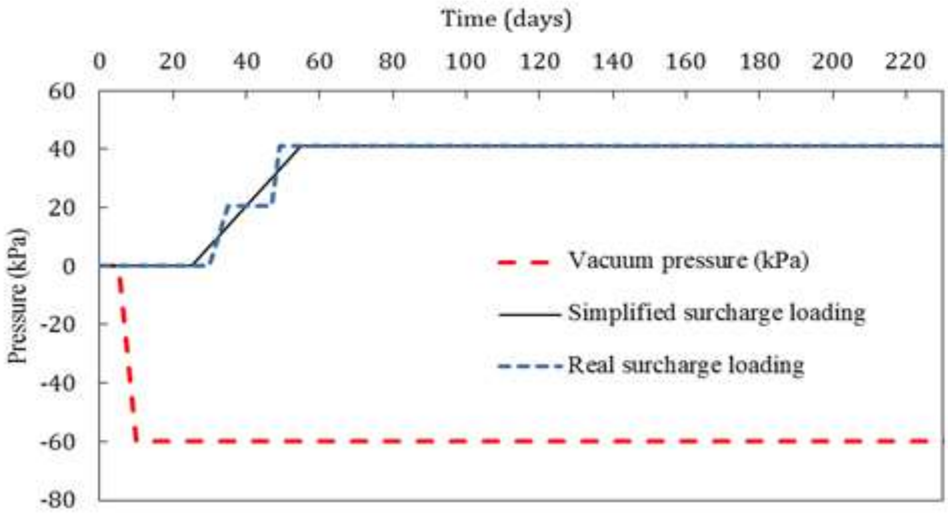

Figure 8. Loading sequence

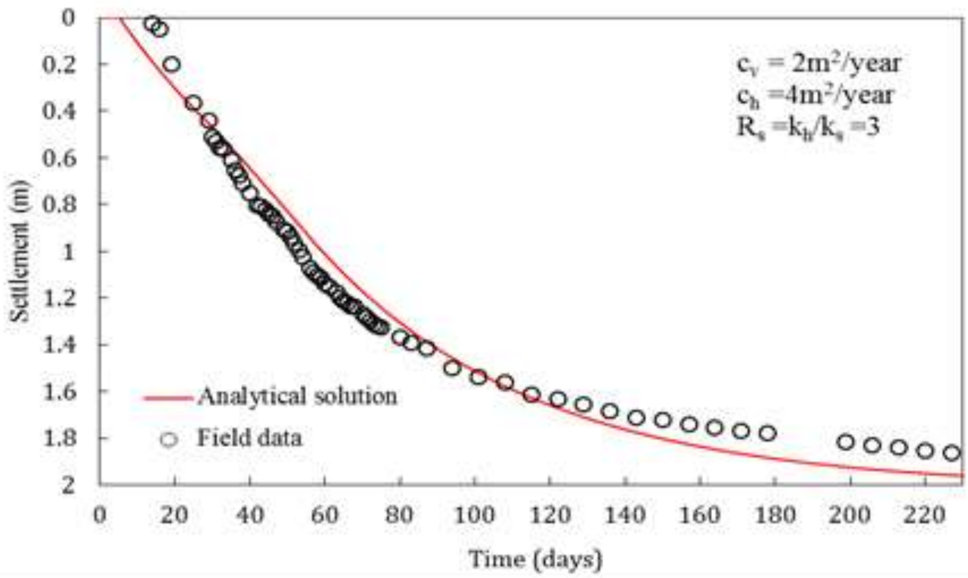

Figure 9. Comparison of surface settlement between the analytical solution and field data

3.2. Numerical modelling for axisymmetric condition

ABAQUS numerical software is utilized for modelling the consolidation problem of axisymmetric numerical modeling. The model is shown in Fig. 10 with a unit cell section. Total height of the model is 29m with 4 soil layers. The length of PVD is 15m. The top crush and layer 2 were also modelled in this model with the thickness 1m and 3m, respectively. The vacuum pressure and the surcharge load were applied at the top boundary. Vacuum pressure was assigned as the negative pore water pressure at a node which could later increase the effective stress and induce the settlement. ABAQUS software is capable of modelling vacuum pressure in cooperating with the advanced soil models such as modified Camclay model and Morh-coulomb model. The soft soil model based on the modified Camclay model is widely applied for PVD consolidation problems of soft soil (Rujikiatkamjornand and Indraratna. 2008., Lin and Chang. 2009). The soft soil properties for the modified Camclay model are listed in Table 2. In specific, the in-situ past pressure (p_c) in layer 1a has been approximated by 6 separated layers. It is because the actual past pressures (in Fig. 5) are about 10kPa - 20 kPa larger than the effective pressures, whereas, the past pressure can be only defined as constant value along the depth of a layer in the model. Moreover, layer 2 is silty sand layer which was simplified in the modified Camclay model with compressive indices of $\kappa = 0.011$ and $\lambda = 0.11$ and high permeability of $k_h = 1$ (m/day).

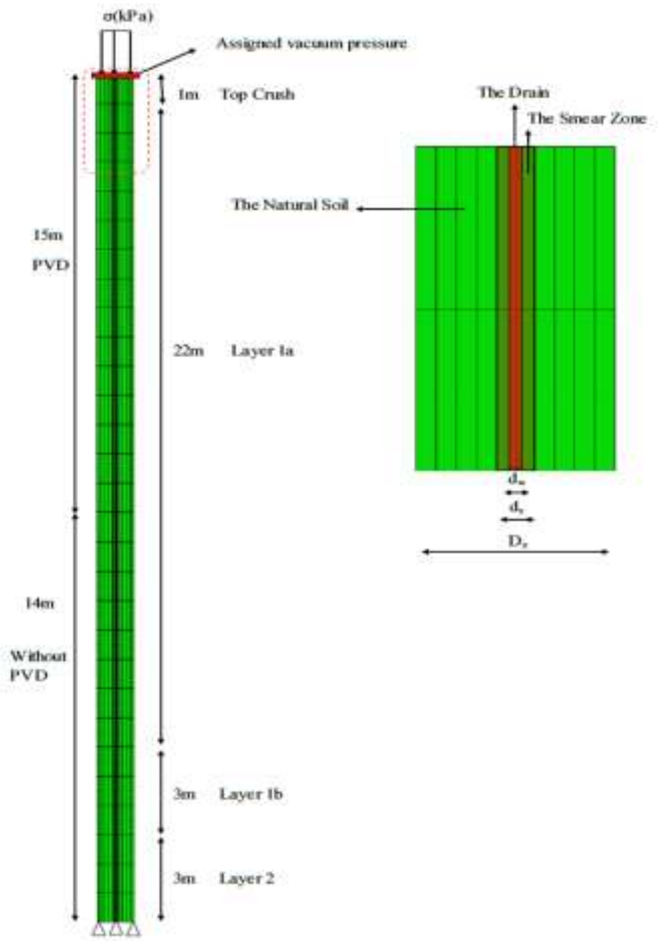

Figure 10. Axisymmetric numerical model

Table 2

soil properties for Modified Camclay model

Layer	Depth (m)	κ	λ	p_c (kPa)	M	k_h (m/day)
1a+PVD	1-3	0.054	0.459	31.6	0.8	2×10^{-5}
1a+PVD	3-7	0.054	0.459	49	0.8	2×10^{-5}
1a+PVD	7-11	0.054	0.459	72.7	0.8	2×10^{-5}
1a+PVD	11-15	0.054	0.459	95.4	0.8	2×10^{-5}
1a	15-19	0.054	0.459	118.6	0.8	2×10^{-5}
1a	19-23	0.054	0.459	141.8	0.8	2×10^{-5}
1b	23-26	0.03	0.303	156.3	1	1×10^{-5}
2	26-29	0.011	0.11	168.3	1	1

Numerical settlement results of the axisymmetric model given in Fig .11 show excellent agreements in not only surface settlement but also in all deep settlements. It implies that the numerical and analytical solution could have the consistent results if soil/drain properties were appropriately calibrated.

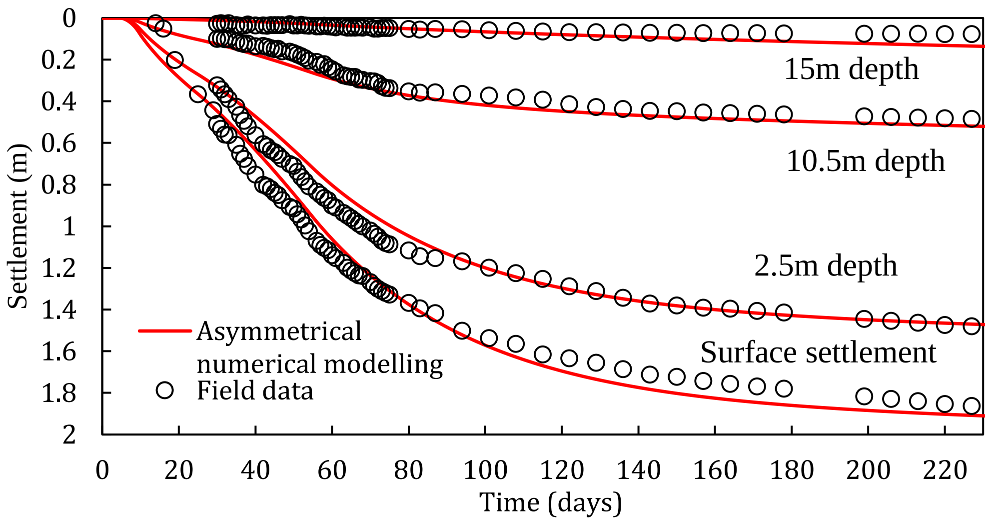

Figure 11. Comparison of settlements by the Asymmetrical numerical modeling and field data

3.3. Plane strain (2D) numerical modeling

The soil/drain properties above are utilized for 2D numerical analysis. Meshing is given in Fig.12 with 647 8-node elements of CPE8RP and CPE8R. Soft soil layer 1a with PVD was simplified as a single layer with equivalent vertical permeability (Chai et al., 2001). The equivalent vertical permeability k_{ev} is an essential factor by Chai's approach is presented in simple equations which considering the PVD's influenced factor μ and undisturbed vertical, horizontal permeability (k_v, k_h):

$$k_{ev} = \left(1 + \frac{2.5l^2}{\mu D_e^2}\frac{k_h}{k_v}\right)k_v \qquad (1)$$

where: l is the drainage length of PVD improved zone, μ the influence factor of PVD geometry as expressed by Hansbo (1981).

$$\mu = \ln\left(\frac{n}{s}\right) + \frac{k_h}{k_s}\ln(s) - \frac{3}{4} + \frac{\pi 2l^2 k_h}{3q_w} \qquad (2)$$

where: q_w is the discharge capacity. This value has been extensively studied as a very large value about $q_w = 100m^3/year$, while the horizontal permeability is very small value. Therefore the last term in equation (2) can be neglected. $n = \frac{D_e}{d_w}$, and $s = \frac{d_s}{d_w}$.

Nevertheless, the ratio of permeability of $R_s = \frac{k_h}{k_s} = 3$ used to derive the equivalent vertical permeability leads to faster predicted settlement than field data measurements. Therefore, it is proposed a higher of ratio of permeability of $R_s = \frac{k_h}{k_s} = 8$ with corresponding $k_{ev} = 1.62\times10^{-3}$(m/day). As a result, the settlement results of the 2D numerical analysis are matched with the field data as shown in Fig.13. The ratio of permeability of $R_s = \frac{k_h}{k_s} = 8$ was also proposed by Lam et al. (2015) when modeling vacuum consolidation of PVD in second international Bangkok airport.

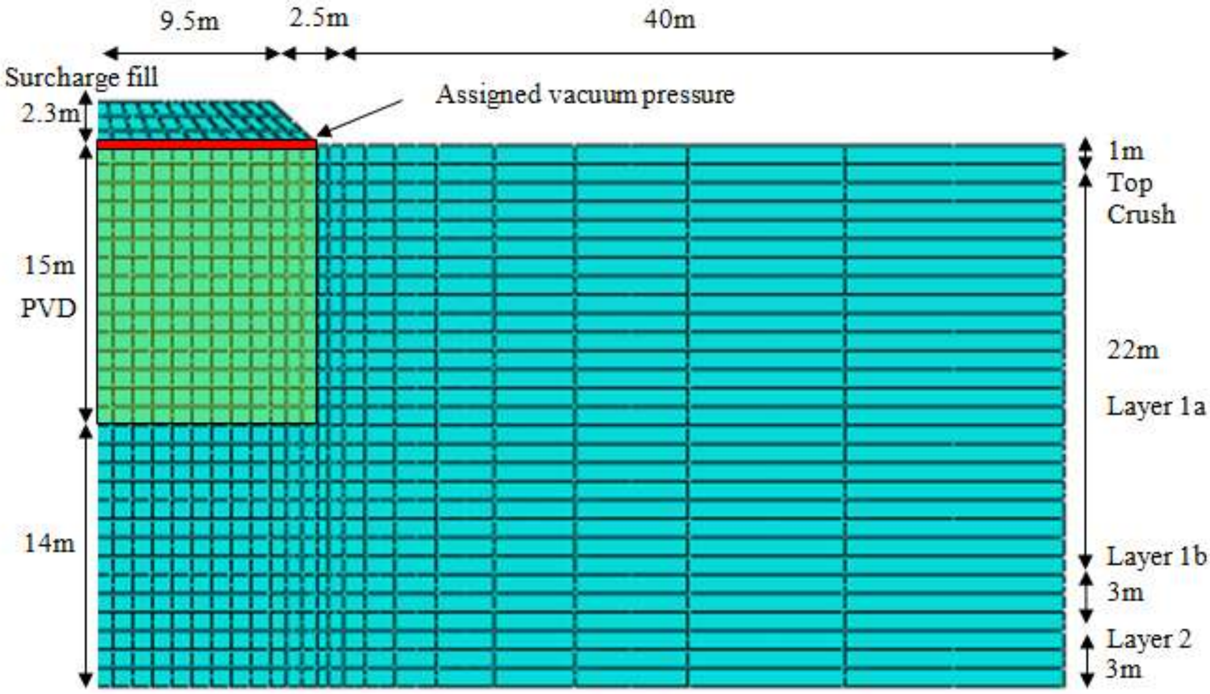

Figure 12. 2D plane strain numerical model

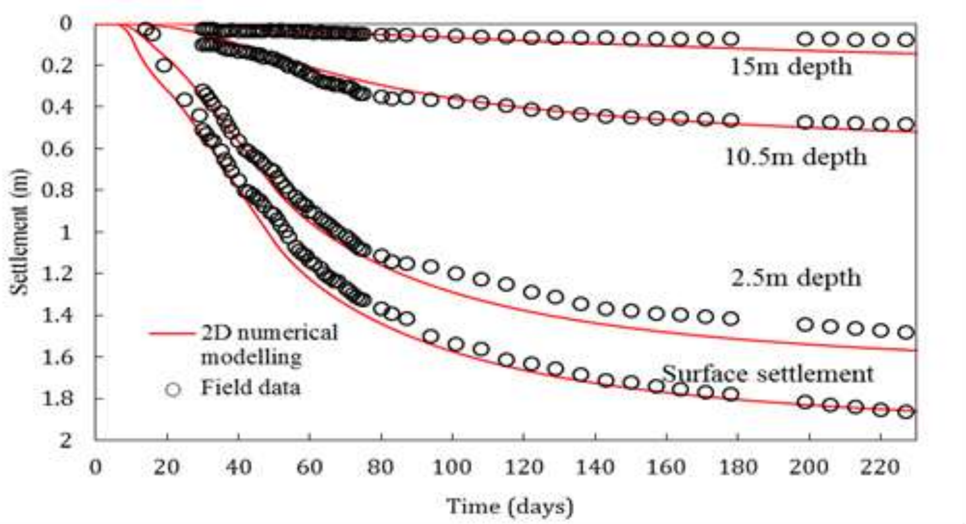

Figure 13. Comparison of settlements by the 2D numerical modeling and field data

3.4. Influence zone

Influence zone can be determined by means of soil deformation during the treatment. As shown in Fig. 14, the maximum lateral displacement is 0.5 m inward the embankment due to vacuum force. This movement significantly decreased to 0.1m at 7.5m away from the slope toe; and it was negligible at 10m away from the embankment's toe. Thus, it can be concluded that in this soil treatment the safe boundary may extend to the length of at least 10m from the embankment's toe. Moreover, an existing structure may be severely damaged if it locates within 7.5m from the surcharge toe.

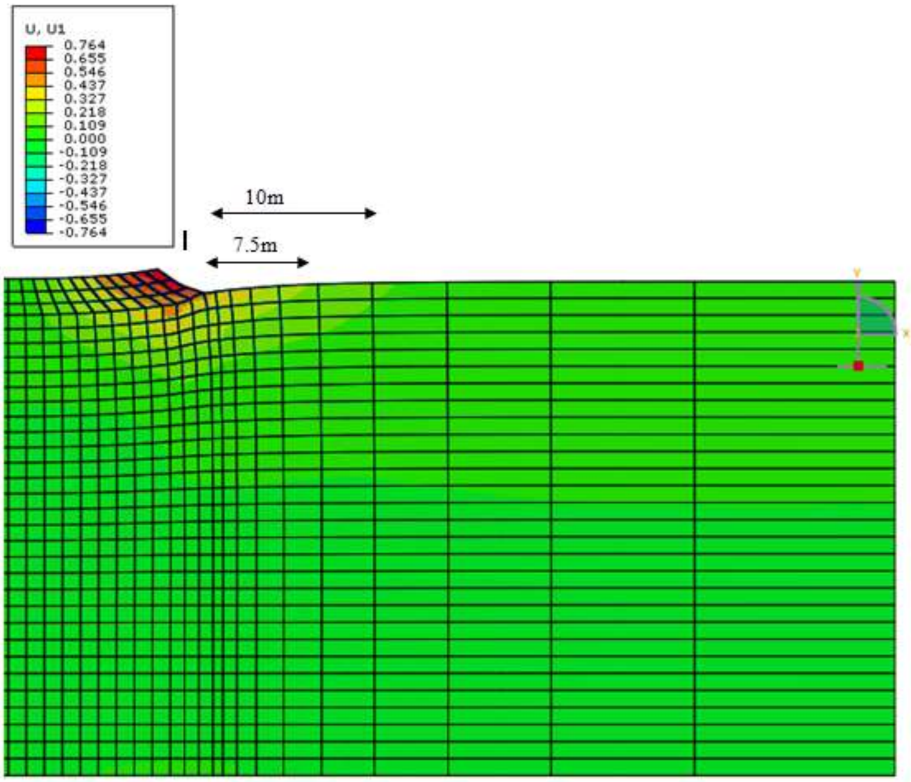

Figure 14. Lateral displacement by 2D numerical modelling

3.5. Evaluating degree of consolidation of non-PVD zone

The most challenging task is the evaluation of the untreated zone because it can have a very low dissipation of the excess pore pressures and it may cause large settlement. By means of numerical modeling, this paper presents two simple approaches including: (1)evaluation through the excess pore water pressures dissipation and (2) evaluation through the effective vertical pressure changes.

A-The first approach

Pore pressure in the axisymmetric model given in Fig.15 shows that excess pore pressure of 26 kPa still remains in the untreated zone while the treated zone is completely consolidated with the excess pore pressure of -60 kPa (equal to the vacuum pressure). The total magnitude of excess pore water pressure of 100kPa is the summation of the maximum surcharge load (40 kPa) and the maximum vacuum pressure (-60 kPa). Therefore, the degree of consolidation at the final consolidation stage (240 days) is calculated by the following equation:

$$U_{Asym} = \frac{u_{surcharge} - u_t}{u_{total}} = \frac{40-26}{100} 100\% = 14\% \quad (3)$$

where: U_{Asym} is the degree of consolidation by the axisymmetric model, $u_{surcharge}$ is the maximum excess pore pressure induced by maximum surcharge loading, u_{total} is the total of excess pore pressure, u_t is the excess pore pressure at the final consolidation stage (240 days).

The excess pore pressure at the final consolidation stage in the 2D model is shown Fig.16. The excess pore pressure of untreated zone only 6.849 kPa which is lower than that of axisymmetric model. The reason may come from the difference in the surcharge stress distribution in 2D model and in the axisymmetric model. Fig.17 presents the vertical stress distribution due to an embankment by analytical solution. It can be seen from Fig.17 that the vertical stress reduces when the depth increases under an embankment. The middle of the untreated zone has a depth of 22m. The influence value at this depth is $I = 0.25$.

The maximum stress distribution at that depth is

$$\Delta\sigma_z = 2I\sigma = 2 \times 0.25 \times 40 = 20\,(\text{kPa}) \quad (4)$$

The maximum stress at the middle of untreated zone is also the maximum excess pore pressure by surcharge loading.

$$u_{surcharge} = \Delta\sigma_z = 20\,(\text{kPa}) \quad (5)$$

The total of excess pore pressure is summation of the vacuum pressure and the excess pore pressure by the surcharge loading.

$$u_{total} = u_{va} + u_{surcharge} = \left|-60\right| + 20 = 80\,(\text{kPa}) \quad (6)$$

The degree of consolidation at the final consolidation stage is determined by equation:

$$U_{2D} = \frac{u_{surcharge} - u_t}{u_{total}} = \frac{20 - 6.8}{80} 100\% = 16.5\% \quad (7)$$

where U_{2D} is the degree of consolidation by the 2D model.

The degree of consolidations by both numerical models are in good agreement. The untreated zone had a little increase in the degree of consolidation (14%-16.5%).

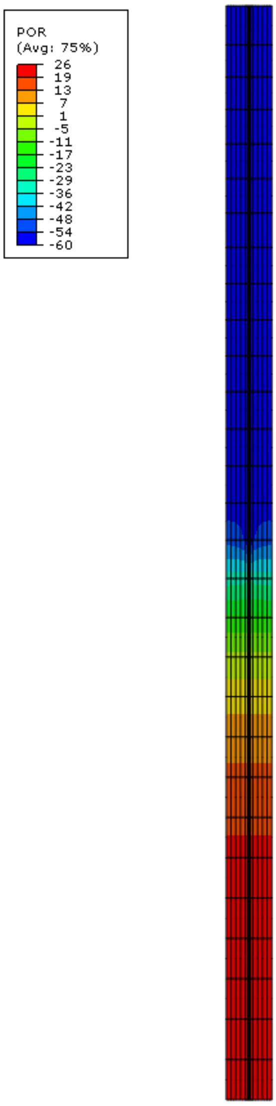

Figure 15. Excess pore pressure at the final consolidation stage by axisymmetric modelling

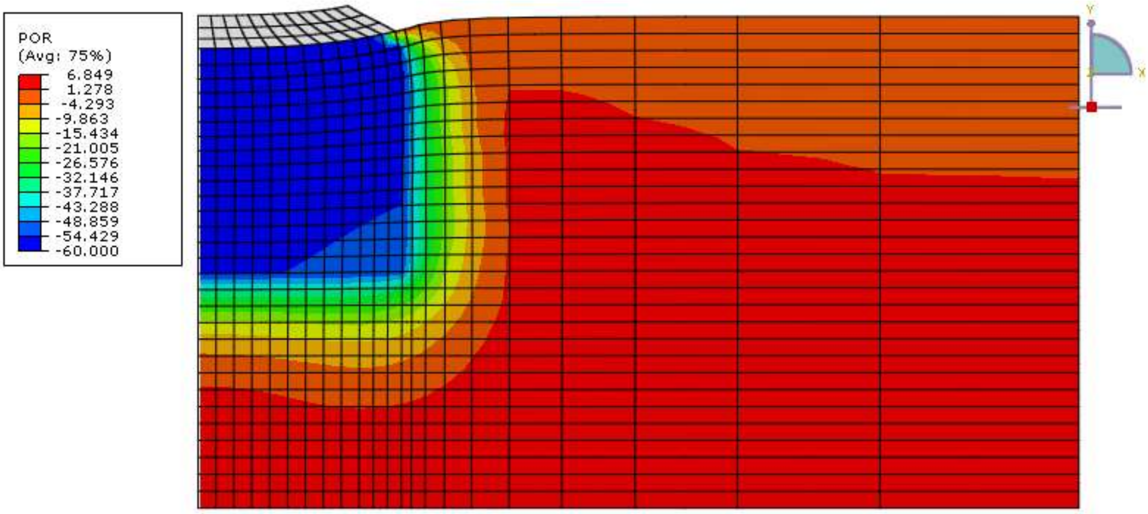

Figure16. Excess pore pressure at the final consolidation stage by 2D modelling

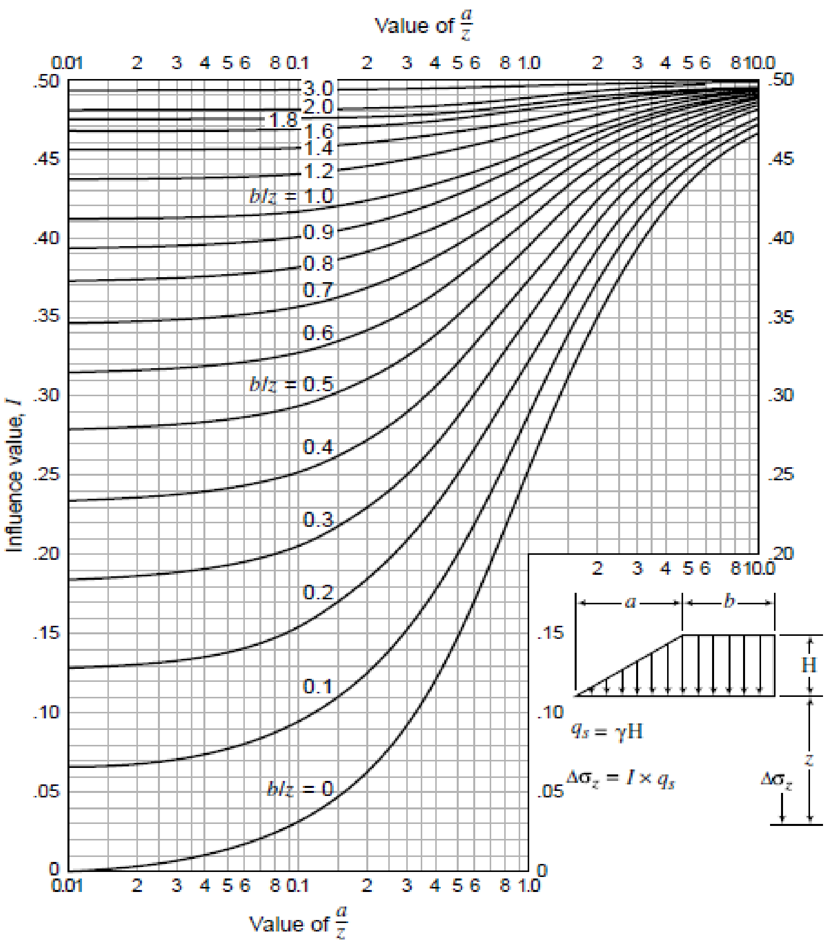

Figure 17. Stress distribution under an embankment load

B-The second approach

The effective vertical pressure at the initial stress state is illustrated in Fig.18. The vertical stress linearly increases along the depth due to the self weight of soil to the maximum value of -168.2 (kPa). While, Fig. 19 shows the stress increase at the final consolidation stage. The untreated zone shows not much increase in term of the effective vertical pressure while the treated zone shows significant improvement (the tresses change at least of 70 kPa).

The stress level in the middle of the untreated zone are about -112 to -126 (kPa) at the initial state, while they are about -125 to 140 (kPa) at the final consolidation stage (240 days) (Fig.18 and Fig.19). The average vertical effective stress increase is $\Delta\sigma'_t = 13.5$ (kPa), while the total vertical effective stress increase (when the excess pore pressure are completely dissipated at the middle of the untreated zone) are $\sigma'_{total} = u_{total} = 80$ (kPa). Therefore, the degree of consolidation is determined by equation:

$$U = \frac{\Delta\sigma'_t}{\sigma'_{total}} = \frac{13.5}{80}100\% = 16.8\% \qquad (8)$$

The degrees of consolidation by the first and second approaches are almost matched (16.5 and 16.8 %, respectively) indicating that the untreated zone has very low improvement with the average stress increase of 13.5 kPa.

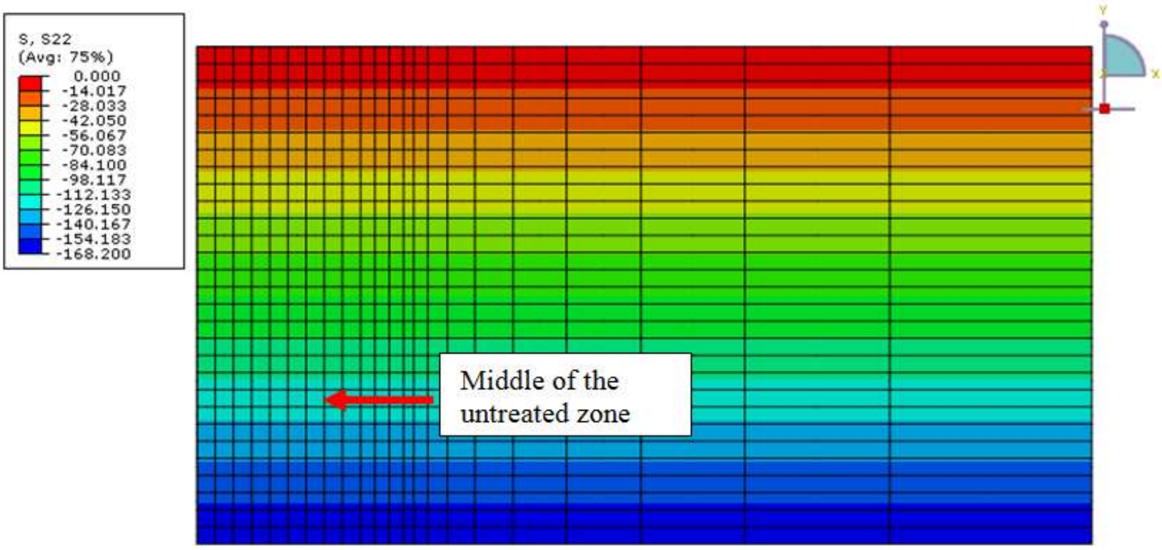

Figure 18. the initial state of vertical effective stress

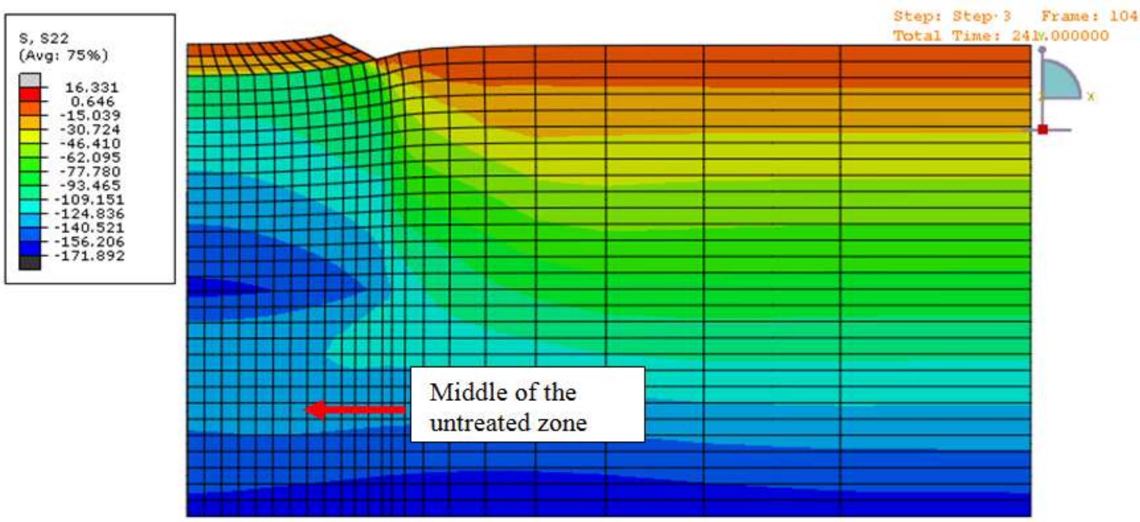

Figure 19. the vertical effective stress at final consolidation stage

3.6. Field verification

Field tests by static cone penetrated test (CPTu) carried out before and after treatment were shown in Fig.20. The treated zone shows greatly improvement in term of the net cone resistance (0.65 MPa). with almost double the initial value (0.35 MPa) while, the untreated zone has a little of increase in cone resistance. That can be explained from the shear strength increase by SHANSEP technique (Ladd 1977).

$$\left(\frac{c_u}{\sigma_v}\right)_{OC} = \left(\frac{c_u}{\sigma_v}\right)_{NC} OCR^m \qquad (9)$$

where $\left(\frac{c_u}{\sigma_v}\right)_{NC} = 0.25$ for silty clay, $m = 0.8$, $OCR = 1.2$ is the over consolidation ratio of the middle of untreated zone.

Substituting the above values to equation (9) yields

$$\left(\frac{c_u}{\sigma_v}\right) = 0.25 \times 1.2^{0.8} = 0.29 \qquad (10)$$

Substituting the vertical effective stress increase $\Delta\sigma'_v$ the equation (10) can be rewritten in the form of shear strength increase Δc_u.

$$\left(\frac{\Delta c_u}{\Delta \sigma_v}\right) = 0.29 \qquad (11)$$

Based on the plane strain analysis, the average vertical effective stress increase by the numerical method of $\Delta\sigma'_v = 13.5$ (kPa), subsequently the increase in shear strength is:

$$\Delta c_u = 0.29 \times 13.5 = 3.9 \text{ (kPa)}. \qquad (12)$$

The shear strength increase was very low ($\Delta c_u = 3.9$ kPa) therefore, it can not make changes in the net cone resistance of CPT tests. The results of field tests and numerical modellings proved the untreated zone could hardly to be improved by consolidation technique.

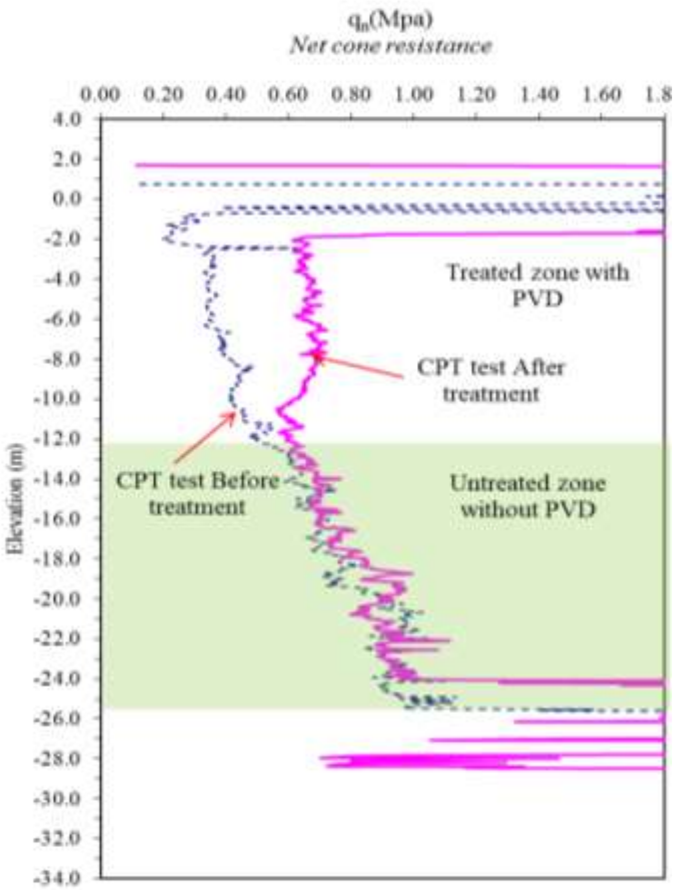

Figure 20. CPTu tests before and after treatment

3.7. Evaluation of residual settlement

Residual settlement is the serviceability settlement when the road is under vehicle loading. The low degree of consolidation of the non-PVD layer may cause residual settlements if increased stresses by operational loading is larger than the past pressures by surcharge loading. The residual settlements are evaluated by numerical modeling with whole process of load removal and operation load. The load removal is the vacuum pressure of -60 kPa and the operation load is the vehicle load of 15 kPa. The total time of residual settlement is 10 years after completing construction. The loading sequences are given in Fig. 21.

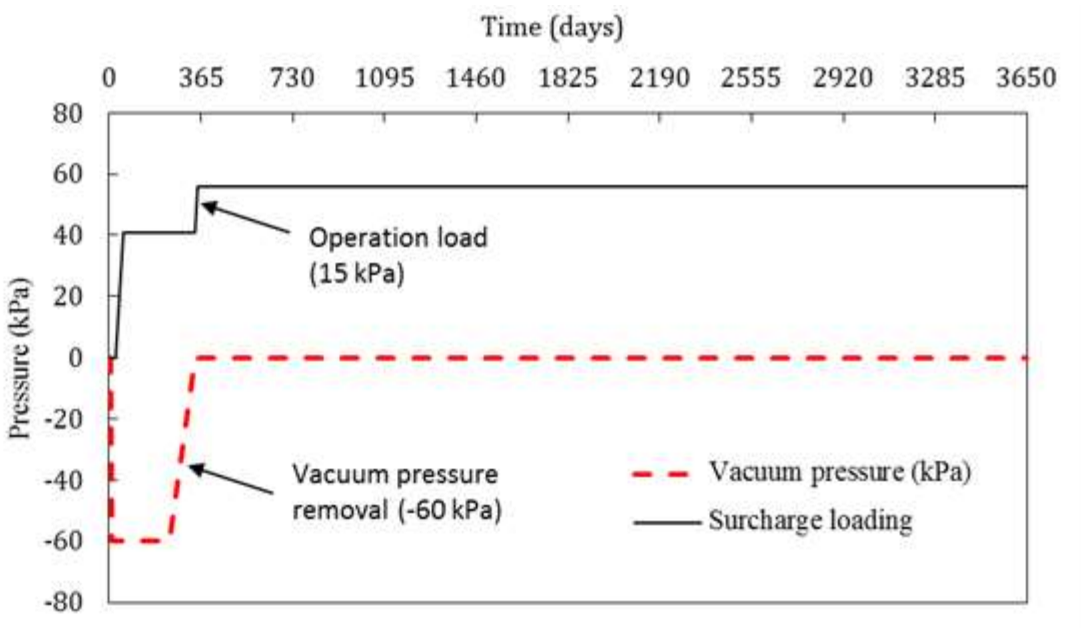

Figure 21. Loading sequences of load removal and operation load

The estimated residual settlement is shown Fig. 22 in which no further settlement is observed. It can be attributed to: (a) operation surcharge (15kPa) is significantly smaller than the vacuum load (-60kPa) and (b) stress increase of 7.5 kPa due to the operation load is lower than the past stress increase of 13.5 kPa during the ground treatment. Thus, the residual settlement is only secondary settlement.

Moreover, the settlement due to the creep effect can be estimated analytically by

$$S_s = C_\alpha . H . \log \frac{t_p}{t_o} \tag{13}$$

where the secondary coefficient $C_\alpha = 1.1 \times 10^{-2}$ is determined through odometer tests, $H = 24$ (m) is the total thickness of compresive layer, $t_p = 15$ (years) is the design time, $t_o = 1$ (year) is the completed consolidation time. Therefore, the residual settlement can be calculated as

$$S_s = C_\alpha . H . \log \frac{t_p}{t_o} = 1.1 \times 10^{-2} \times 24 \times \log \frac{15}{1} = 0.3 \text{(m)} \tag{14}$$

These residual settlement for 15 years is satisfactory required residual settlement of 0.4m for 15 years according to Vietnam's road standard.

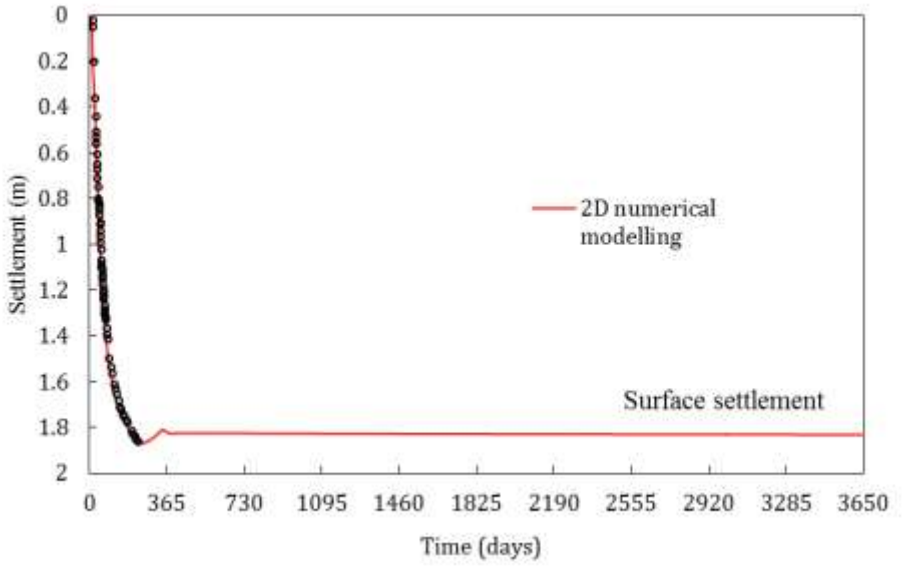

Figure 22. Residual settlement

4. Conclusions and recommendations

Several conclusions can be drawn from the Bachiem road project:

1- This paper introduces a matching scheme for selecting the soil/drain properties for numerical modeling. There are excellent consistent in soil/drain properties of the analytical solution and the axisymmetric modeling, while the 2D modeling has to adjust the equivalent vertical drain with direct relation to the permeability ratio of $R_s = \frac{k_h}{k_s}$

2- The analytical solution and the axisymmetric modeling have the permeability ratio of $R_s = \frac{k_h}{k_s} = 3$, while the 2D modeling has the permeability ratio of $R_s = \frac{k_h}{k_s} = 8$. The analysis results in term of settlement of these models are in good agreement with the field data.

3- The lateral displacement of vacuum consolidation technique investigated by 2D modeling indicated that the safe boundary for existing constructions would be 10m from the embankment's toe. The effectiveness of soil treatment can be verified by the CPTu test. The cone resistance increased significantly within the PVD boundary, meanwhile,

4- Numerical results of untreated zone are then verified by the cone penetrated tests. The field test results are consistent with numerical results which prove the effectiveness of the numerical methods in solving the complicated consolidation problems by PVD vacuum technique.

Load removal of vacuum pressure and the operation load of vehicle loading can be modeled by numerical simulation to investigate the residual settlement. However, it need more advance model which is capable of simulation of creep effect. The residual settlement of analysis section is satisfied the required settlement by relevant Vietnam's standard■

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