



Parametric Study of Large Settlement Due to Prefabricated Vertical Drain with Surcharge Preloading

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ARTICLE INFO

Article History :

Article entry : 05 – 11 – 2022
Article revised : 16 – 11 – 2022
Article received : 29 – 11 – 2022

Keywords :

Back Calculation, Consolidation Settlement, Preconsolidation, PVD, Surcharge Preloading.

IEEE Style in citing this article :

M. F. Maruf, "Parametric Study of Large Settlement Due to Prefabricated Vertical Drain with Surcharge Preloading," *U Karst*, vol. 6, no. 2, pp. 261–276, 2022, doi:<https://dx.doi.org/10.30737/ukarst.v6i2.3545>.

ABSTRACT

Prefabricated vertical drain (PVD) combined with preloading is one way to deal with soft soil conditions. The predicted settlement becomes the first information in designing the surcharge, PVD configuration and depth, and the time required to complete the consolidation. Prediction of consolidation settlement that using one-dimensional theory considerably underestimated the field settlement along the construction of the surcharge placement stage. This paper intends to evaluate the completed PVD preloading construction project. Inclinator monitoring data was evaluated to confirm whether the consolidation existed on preferred clay layers. The parametric study evaluated the root cause of the gap between prediction settlement and monitoring data. The root cause analysis continued with a parametric analysis using back calculations. The back analysis involved various C_c , C_s , and pc' that play a significant role in soil settlement. The results show that the conservative value of those parameters simulated separately could not raise the ultimate settlement into inner upper and lower bond results. However, combining those would lead to more accurate predictions that agree with the ultimate settlement. This parametric analysis result also confirms that overconfidence in picking soil parameters could lead to underestimating field settlement. Furthermore, selecting conservative parameters in consolidation settlement should avoid a big gap between prediction and field ones and put construction safe. It was decisive work to be done for further anticipation during and post-construction.

1. Introduction

Soft soil exhibits large settlement. It triggers dangerous conditions for construction if not solved in advance. When dealing with soft soil conditions, prefabricated vertical drains (PVD) combined with preloading are possibly the most extensively used in geotechnical practice. The technique accelerates consolidation by shortening the drainage channel and facilitating faster pore water flow. Preloading generates excess pore pressure that forces the pore water horizontally to the nearest PVD, taking advantage of the clay soil's higher horizontal

permeability. Inserting PVD into the subsurface creates a synthetic drainage conduit that allows pore water to be discharged toward the porous layer. Preloading and PVD, in combination, speed up the completion of consolidation. As a result, soil strength increases rapidly, and post-construction reduces post-construction settlement. The PVD method was widely used in a variety of building projects, including highway and road construction [7], bridge abutments [8], tank and container yards [9], [10], reclamation areas [10], [11], and airports [12]–[14]. PVD preloading was a leap development following wick drain as a vertical drain channel on clay [1]. The theoretical underpinnings of PVD design have been chronologically given by Hansbo [2]–[6]. Hansbo first worked on the vertical sand drain to introduce faster horizontal pore water flow. He shifted the sand drain theory into PVD when the prefabricated drain band was introduced. He also noticed that the consolidation rate should be estimated based on pore water dissipation instead of soil compression results.

PVD preloading work requires the prediction of ultimate settlement due to the construction project. The settlement prediction becomes essential to provide a framework for soft soil solutions. Terzaghi one dimensional theory is generally utilized for prediction considering the operational load [15]. The predicted settlement becomes the first information in designing the surcharge, PVD configuration and depth, and time required to complete the consolidation.

Predicting soil settlement is an art and very challenging for geotechnical engineers. Complete soil data from good-quality soil investigation work becomes very important. Soil data is very scattered from different depths. Uncertainty is almost unavoidable. Proper selection of soil parameters provides a great impact on geotechnical prediction. The large disparity between prediction and real settlement put the construction at high risk of experiencing failure. In The Rankine Lecture 50 years ago, Lambe [16] reminded geotechnical experts and engineers of the uncertainty that can arise in geotechnical prediction.

Because of the high variability of soil properties, geotechnical prediction became more erratic. He classified geotechnical prediction into five categories based on when the prediction was made and the results at the time prediction were made. Type A prediction should be made before the event, and the results remained unknown when the prediction was made. Lambe also stated that most professional literature in geotechnical prediction is of type C1, in which the prediction was made after the construction event and had the results known. The predictions evolved into autopsies to demonstrate that the model's prediction was correct.

A lot of publications have reported PVD preloading prediction and performance [7], [9], [10], [17]–[21], among others. Most of them are classified under type C1, in which

prediction and performance are closely related. [18], [20], [21] are involved in Type A prediction. [18] prediction overestimated measured settlement, while [20], [21] underestimated the field condition. Utilizing numerical modeling, [21] used complicated relations and more detailed soil parameters to generate the better prediction. The articles above have discussed more on prediction and performance of PVD instead of investigating the root cause of the gap between the prediction and the field results. The parametric study should be useful for such gap evaluation root cause.

This paper intends to evaluate the completed PVD preloading construction project in which the designed settlement considerably underestimated the final observational settlement. Terzhagi one dimensional consolidation theory was adopted for final settlement prediction. Hansbo classical theory was used to conduct the parametric study. The results were then utilized to evaluate the root cause of the large discrepancy between prediction and real settlement. The analysis results should provide critical information useful for consolidation settlement design. The results should also provide a framework for soil parameter selection for construction settlement due to consolidation.

2. Research Method

The PVD preloading work was initiated with an estimation of the final consolidation settlement. The consolidation settlement prediction was assessed by Geotechnical Design Standard SNI-85602017 and Construction and Building Manual Pd T-13-2004-A, both of which employ Hansbo's theory [3]. Settlement plates were installed to monitor field settlement during preloading, and an inclinometer was used for lateral soil movement monitoring. Furthermore, the Asaoka observational procedure evaluated the field monitoring settlement [22]. The Asaoka results would be used to measure prediction ones. The inclinometer monitoring data and parametric study, along with Hansbo's theory, were then conducted to investigate the gap's root cause.

The prediction of consolidation settlement conformed with the procedure described in the following sections:

2.1. Consolidation Settlement

For normally consolidation soils, the consolidation settlement can be estimated using the well-known Terzaghi equation [15] as follows:

$$S_c = \frac{C_c \times H}{1 + e_o} \log \left(\frac{\sigma'_0 + \Delta\sigma'_0}{\sigma'_0} \right)$$

Where S_c is consolidation settlement, C_c is compression index, H is the thickness of the soil layer undergoing the consolidation process, e_o is an initial void ratio, and $\Delta\sigma'$ is effective stress increment.

For overconsolidated soils, the equation becomes:

$$S_c = \frac{C_s \times H}{1 + e_o} \log \left(\frac{\sigma'_0 + \Delta\sigma'_0}{\sigma'_0} \right)$$

Consolidation Settlement is as follows:

$$S_c = \frac{C_s \times H}{1 + e_o} \log \frac{p'_c}{\sigma'_0} + \frac{C_c \times H}{1 + e_o} \log \left(\frac{\sigma'_0 + \Delta\sigma'_0}{p'_c} \right)$$

2.2 Vertical Consolidation Rate

Terzaghi also proposed a method for determining the time required to complete the consolidation process and the degree of consolidation achieved. The following equation can be used to calculate the time factor T_v during the consolidation process:

$$T_v = \frac{C_v t}{h_{de}^2}$$

Where C_v is the coefficient of vertical consolidation, t is time, and h_{de}^2 is the longest drainage path in the vertical direction.

The average degree of vertical consolidation process U_v can be estimated as follows:

$$T_v = \frac{\pi}{4} \left(\frac{U_v}{100} \right)^2 ; U_v \leq 52.6\%$$

$$T_v = 1.781 - 0.933 \log_{10}(1 - U_v) ; U_v \geq 52.6\%$$

2.3 Radial Consolidation Rate

Hansbo [3] proposed a simplified vertical drain dimension as a full circle. The equivalent diameter of vertical drain d_w can be calculated as follows:

$$d_w = \frac{2(a + b)}{\pi}$$

A and b are the width and thickness of the vertical drain, respectively. This project utilized

The average degree of consolidation in radial direction U_r during PVD – surcharge preloading can be evaluated using the Hansbo equation as follows:

$$U_r = 1 - \exp \left(\frac{-8T_r}{F(n)} \right)$$

Where T_r is the time factor for radial consolidation, $T_r = C_h t / d_e^2$; d_e is the diameter of the drainage zone bordering a PVD; d_e equal to $=1.13S$ or $=1.05S$ for square and triangular

PVD installation pattern sequentially; S is the distance between PVD; C_h is coefficient of consolidation in the horizontal direction. Spacing factor $F(n)=[\ln(n)-0.75+n^{-2}]/[n^2/n^2-1]$; $n = d_e/d_w$; $d_w = (a+b)/2$ is equivalent diameter of PVD in which a and b is PVD thickness and width respectively.

The overall degree of consolidation soil due to a combination of vertical and radial flow can then be expressed as follows [11], [23]:

$$U_{vr} = 1 - (1 - U_v)(1 - U_{vr})$$

The consolidation settlement at time t can be determined as follows:

$$S_t = S_c U_{vr}$$

2.4 Project Requirement

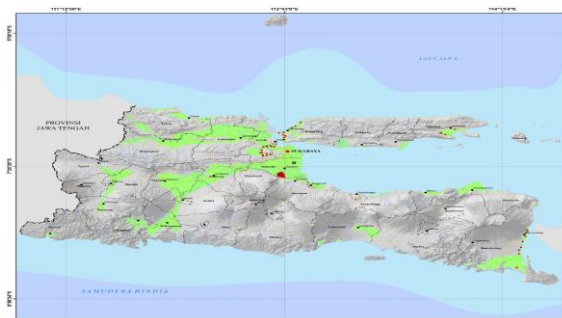
The PVD preloading design process requires acceptance criteria for achieving target settlement in PVD preloading treatment. The following are the criteria:

1. The settlement caused by the consolidation process on alluvium formation will be occurred due to overburden pressure as follows:
 - a. Soil filling to achieve final elevation +2 m above the existing level
 - b. 10 kPa pressure load is adopted as the operation load
2. The duration of the consolidation process is 4 months
3. Target settlement shall be achieved with allowable residual settlement below 30 mm.

2.5 Field Condition

a. Soil Profile

The PVD preloading project was situated in the reclamation area on East Java's northwest coast. According to a geological map published by the Geological Research and Development Center in 1992, the site's primary geological formation is alluvium (Qa). Alluvial sediment spreads across East Java, particularly in the lowlands along major rivers. As shown in **Figure 1**, sediments form soft soil deposits in those areas.



Source: *Soft Soil Distribution Map, Wardoyo (2019) [24]*

Figure 1. Soft Soil Distribution Map

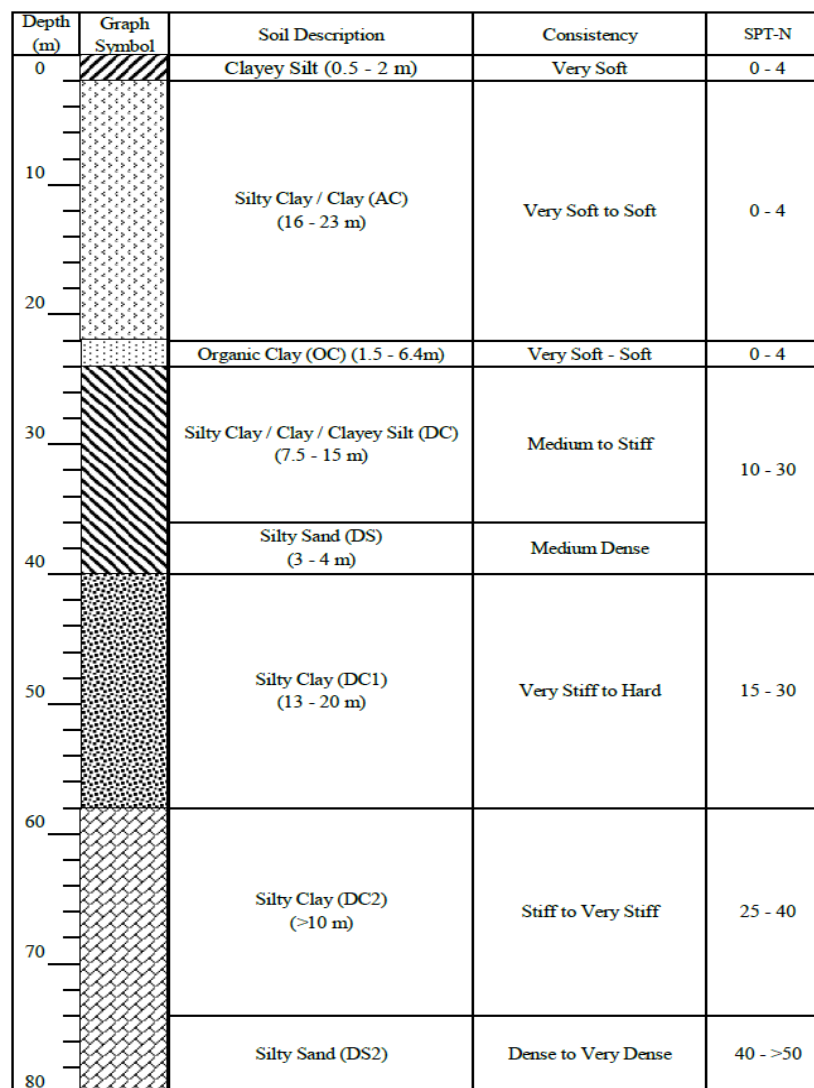


Figure 2. Distribution of Soft Soil Area in East Java

Soil investigation was carried out by boring to a depth of 50 - 80 m to collect undisturbed samples. **Figure 2** depicts the site's typical soil stratigraphy. It confirms the presence of soft alluvial deposits up to a depth of 20 - 23 m. The thickness of the first layer was between 0.5 and 2 meters, and the SPT N-values ranged from 0 to 4. The soil was relatively firm, so it was assumed to be reclamation fills that had not been properly compacted. The fills were made up of Clayey Silt and Silty Clay, with traces of fine sand and gravel. Alluvial clay (AC) was overlaid by the fill layer and was consistently determined in all boreholes and cone penetration tests across the site. The thickness ranged from 16 to 23 m. The soil layer was composed of silty clay with high plasticity and clay with traces of fine-grained sand and shell fragments throughout the deposits. Gray, dark gray, and blackish gray were the colors. AC was very soft to soft and provided no resistance to the standard penetration test (SPT). The organic matter found at the bottom of the recent alluvium was identified as organic clay (OC). The thickness of OC varied greatly, ranging from 1.5 to 6.4m.

Underneath the organic clay has discovered a subsoil that was considerably more rigid. As shown in **Figure 2**, the SPT N-values increased significantly compared to the overlying layer AC. This layer, designated as DC, was inferred to be older alluvial than AC and is known geologically as Diluvium. The SPT N-values indicated that the consistency of the older alluvial deposits increased with depth. In general, the thickness ranged from 13 to 20 M. Lens sand was present in some areas, as indicated by CPT test results, with a thickness of 1 - 3 m.

Deposits below DC were identified as silty clay and clay with a stiffer consistency, which was relatively more homogeneous than the DC subsoil. DC1 was assigned to the subsoil. The colors are dark grey, gray, bluish gray, and yellowish gray. For SPT N-values ranging from 15 to 30, the consistency ranged from stiff to very stiff, with a tendency to increase with depth. Drilling in 75-meter boreholes confirmed the thickness of DC1 at 13 and 20 meters. The lowest part of the boreholes investigated a very stiff silty clay layer with SPT N-values ranging from 25 to 40. On top is very stiff to hard silty clay, and the bottom is dense to very dense silty sand.

b. Engineering Properties

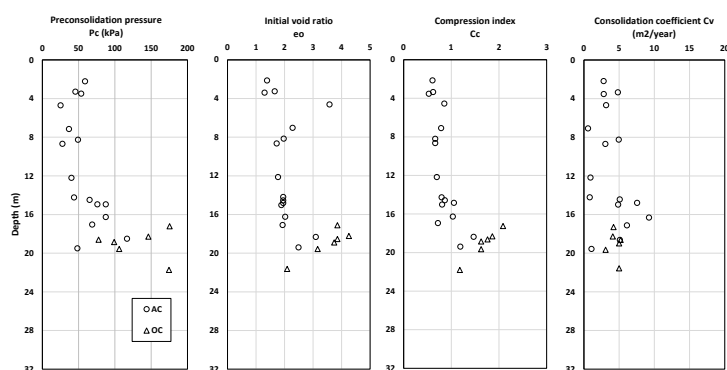
The engineering properties were obtained from the alluvium sediments AC and OC. Only those subsoils could be collected using a routine sampling technique. Because the subsoil was very stiff to hard, no undisturbed samples were collected from beneath layers. The fill layer beneath the ground surface yielded no samples. The properties of the fill layer were estimated using a common correlation with SPT N-values. **Tables 1** and **2** summarize the index properties and mechanical properties.

Table 1. Summary of index properties

		AC	OC
Unit Weight	(kN/m ³)	13.5 - 15	12 - 13
Specific Gravity		2.45 - 2.65	2.1 - 2.3
Water Content	(%)	75 - 110	125 - 200
Atterberg Limits			
Liquid Limit	(%)	75 - 110	110 - 150
Plastic Limit	(%)	30 - 35	35 - 40
Plasticity Index	(%)	40 - 70	80 - 100
Grain Size Distribution			
Gravel	(%)	0	0
Sand,	(%)	< 10	< 10
Silt,	(%)	40 - 60	20 - 30
Clay,	(%)	50 - 60	60 - 70

Source: Soil Investigation Results

The index properties for AC and OC are shown in **Table 1**. Specific gravities range from 2.45 to 2.65, typical for non-organic soil. The fact that the water contents are generally close to their liquid limits indicates that the AC is soft and compressible. Even for OC, the water content occasionally exceeds the liquid limit. It implies that OC is very soft and occasionally close to fluid. Plotting the liquid and plastic limits of AC and OC into the plasticity chart reveals that they fall into the fat clay (CH) category.



Source: Soil Investigation Results

Figure 3. Consolidation Properties Distribution with Depth

The consolidation parameters can be evaluated in **Figure 3**. For AC, it is clear that all of the consolidation stresses (p_c') are close to or slightly larger than the effective overburden stress (σ_0'). While the p_c' value of OC is rather slightly over its respective σ_0' value. It indicates that AC is normally applied to lightly overconsolidated soil. Meanwhile, OC soils are a little overconsolidated. **Table 2** summarizes the common range of AC and OC consolidation properties.

Table 2. Consolidation properties

	AC	OC
Preconsolidation Pressure, p_c' (kPa)	$\sigma_0' + 0$ to 30	$\sigma_0' + 20$ to 100
Initial Void Ratio, e_0	1.2 - 3	2 - 4
Compression Index, C_c	0.6 - 1.5	1.2 - 2.1
Coefficient of Consolidation, c_v (m ² /year)	3 - 5	3 - 5

Source: Soil Investigation Results

Permeability is another important property of soil that plays an important role in the consolidation process. **Table 3** summarizes the permeability data obtained from both laboratory and field testing. It demonstrates that both methods produce similar results. The permeability of all clayey subsoils ranges from low to very low. The sandy soil, on the other hand, has a medium permeability. These properties will impact the time required to achieve full consolidation.

Table 3. Soil permeability from both laboratory and field tests

Zone	Permeability Laboratory Tests	Permeability Field Tests	Category of Permeability
	(cm/sec)	(cm/sec)	
AC	$2 \times 10^{-5} - 7 \times 10^{-5}$	$1 \times 10^{-6} - 3 \times 10^{-5}$	Low to Very Low
OC	8×10^{-5}	2×10^{-5}	Low
DC1	$2 \times 10^{-6} - 8 \times 10^{-5}$	$5 \times 10^{-7} - 2 \times 10^{-4}$	Low to Very Low
DS1	-	2×10^{-3}	Medium
DC2	6×10^{-6}	$3 \times 10^{-7} - 2 \times 10^{-5}$	Low to Very Low

Source: Soil Investigation Results

3. Results and Discussions

3.1 Settlement Prediction

a. Settlement Due to Operational Load

The operational load consists of a 2 m fill embankment and an operational pressure load of 10 kPa. The compacted soil embankment will have a density of 17 kN/m³. The total applied load during operation is $17 \times 2 \text{ m} + 10 \text{ kPa} = 44 \text{ kPa}$. **Table 4** depicts calculating consolidation settlement due to the planned applied load during operation.

Table 4. Total consolidation settlement due to operational load

Top (m)	Bottom (m)	e_0	C_c	C_s	σ_0' (kPa)	p_c' (kPa)	S_c (m)
0	2				27		
2	4	1.5	0.5	0.1	31.5	50	0.088
4	14	2	0.6	0.12	58.5	50	0.596
14	17	2	0.7	0.14	87.75	75	0.162
17	22	3.9	0.8	0.16	105.75	100	0.139
22	24.95	3	1.6	0.32	124.38	160	0.052
						$S_c =$	1.037

Source: Calculation Results

The operational load might generate a final consolidation settlement of approximately 1.037 m. This final settlement turned into a target for the PVD surcharge loading design.

b. Settlement Due to Surcharge Load

After simulating several scenarios with varying surcharge preloading heights, a surcharge height of 3.5 m was selected. The density of the soil surcharge was set conservatively at 13 kN/m³. **Table 5** illustrates the calculation procedure for surcharge consolidation settlement. This surcharge was equal $13 \times 2 \text{ m} = 45.5 \text{ kPa}$ loading and might cause 1.064 m settlement.

Table 5. Total Consolidation Settlement Due to Surcharge Loading

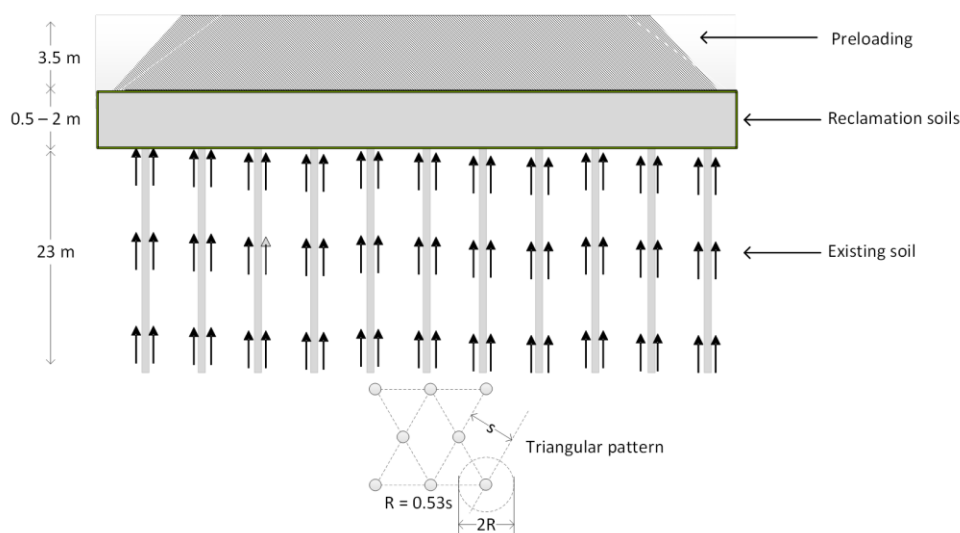
Top (m)	Bottom (m)	e_0	C_c	C_s	σ_0' (kPa)	p_c' (kPa)	S_c (m)	
0	2				27			
2	4	1.5	0.5	0.1	31.5	50	0.091	
4	14	2	0.6	0.12	58.5	50	0.609	
14	17	2	0.7	0.14	87.75	75	0.165	
17	22	3.9	0.8	0.16	105.75	100	0.143	
22	24.95	3	1.6	0.32	124.38	140	0.057	
							$S_c =$	1.064

Source: Calculation Results

3.2 PVD – Preloading Design

After preloading, the acceptance criteria required a residual settlement of 30 mm. To meet the requirement, the targeted settlement should be achieved by PVD preloading work was $1.037 - 0.03 = 1.007$ m.

Figure 4 presents the PVD preloading setup. A triangle PVD pattern was used to achieve the desired result. The PVD was installed at 1.3 m apart, resulting in a drainage zone d_e of 1.365 m. calculated the equivalent diameter of vertical drain d_w to be 0.066 m using 100 mm x 3.8 mm PVD dimension. C_h was estimated as $2C_v$ using data from Table 2.



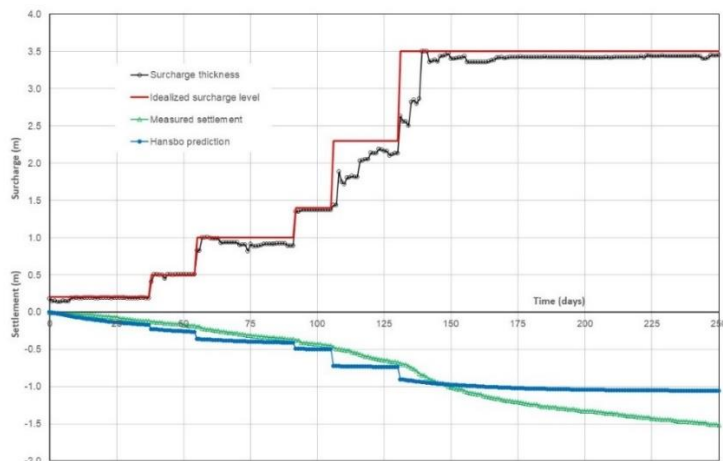
Source: Design Results

Figure 4. PVD Preloading Setting

Applying those parameters' values into an Equation with t equal to 4 months would result in a degree of consolidation of 98%. It means the settlement completed was $98\% \times 1.064 = 1.040$ m. The targeted settlement, according to the criteria, has already been surpassed.

3.4 Monitoring Data

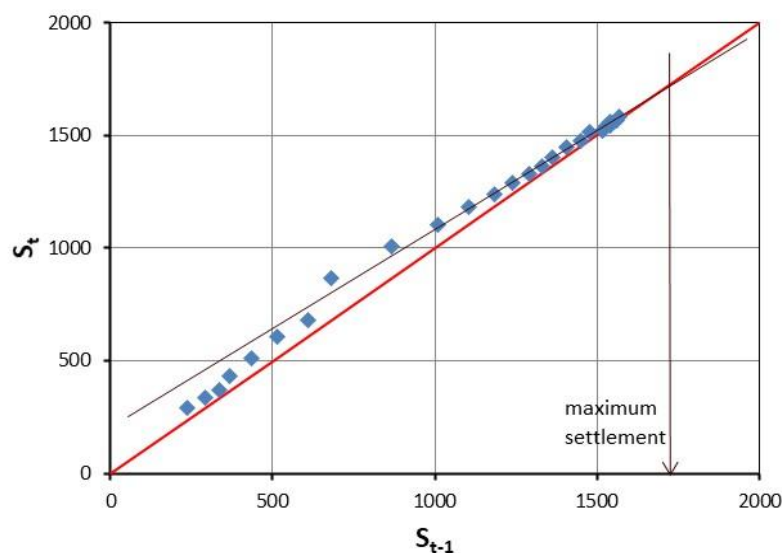
Field stage construction was not the same as PVD design ones. The PVD contractor required approximately 4 months for field preparation, PVD and instrument installation, and embankment preloading stage construction. The field settlement was recorded from the settlement plates installed.



Source: Calculation Results

Figure 5. Hansbo Prediction Compares with Measured Data

Figure 5 compares the measured settlement to Hansbo's prediction. It was calculated using Equation Settlement at every state and was determined following the procedure outlined in Section 2.1 – 2.3 and in line with the practical work described in [25]. The preloading project took a long time, and surcharge preloading required several stages to achieve the embankment target elevation. For calculation purposes, idealized instantaneous embankment elevation was proposed at every stage. As previously estimated, Hansbo's settlement reached 97.7% after four months of 3.5 m surcharge. The curve became flat and asymptotic, showing that the settlement has approached the maximum predicted settlement. However, the settlement plate data has not yet reached an asymptotic state. 4 months surcharge preloading developed 1.51 m settlement.



Source: Calculation Results

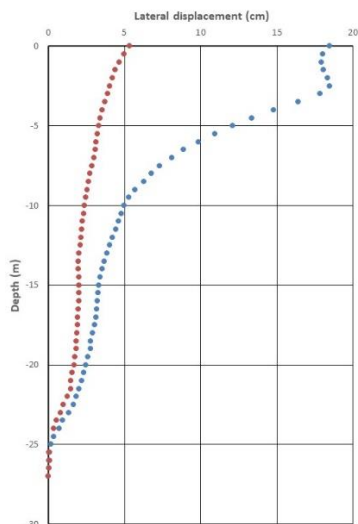
Figure 6. Maximum settlement using the Asaoka method

While using the Asaoka method [22], the final settlement was approximately 1.710 m (**Figure 6**). The average degree of consolidation at the end of preloading was $1.51/1.71 = 88.3\%$. It was much beyond the predicted degree of consolidation. The final settlement was about 160% larger than Terzaghi's one-dimensional settlement results. Theoretical prediction grossly underestimated the final settlement due to surcharge loading.

3.5 Parametric Study

The excessive settlement that developed during PVD surcharge preloading exceeded the theoretical prediction. As previously stated, Lambe [16] had already highlighted the difficulty of geotechnical prediction before the construction event. A thorough investigation should be carried out to determine the root cause of this large settlement.

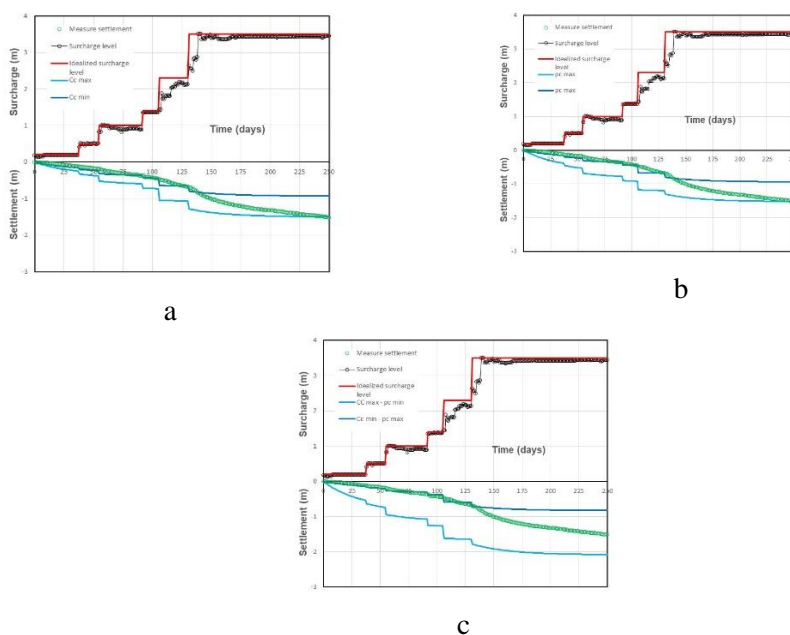
Due to data availability, the design only predicted settlement on the AC and OC layers at 2 - 25 depths. Both the topmost surface layer and the bottom layers were ignored. **Figure 5** presents inclinometer data collected during preloading. It shows that lateral displacement occurred only up to a depth of 25 m. Furthermore, no lateral deformation requires no effective stress increment in that depth below 25 m in lateral and vertical directions. As a result, vertical deformation should also be very small and can be neglected. Taking only 25 m depth for settlement prediction was proven acceptable.



Source: Measurement results

Figure 7. Lateral Displacement Monitoring Results

Further investigation was carried out by conducting a parametric study following the Equation. The parametric study simulated various C_c , C_s , and p_c using **Table 2** and **Figure 3** as boundary values. For simulation, C_s equal to $0.1C_c$ was selected [26].



Source: Measurement results

Figure 8. Parametric study results (a) for C_c -max; (b) for p_c -max'; and (c) combination of C_c -max, C_s , and p_c -max'.

The simulation results can be evaluated in **Figure 7**. **Figure 7a** and **7b** clearly shows that choosing C_c -max, C_s , and p_c -max' and utilizing them separately developed a final settlement up to 1.5 m. The field's final settlement was yet beyond the calculation settlement. The gap between calculation and observational settlement was about 20 cm. **Figure 7c** presents

simulation results when a combination of p_{c-max} ' and C_{c-max} are selected. The figure shows that the calculation result exceeded the observational settlement result. It means that for this project, conservative values of parameters avoid parameters and provide a better prediction.

The previous parametric study results clarify that taking a risk on parameter selection yields a large underestimation of final consolidation settlement. Conservative selection slightly overestimates the settlement. However, it creates a prudent design that ensures the construction in the future time.

4. Conclusion

A thorough examination of large settlements during PVD surcharge preloading was carried out. For this project, the mean value of consolidation parameters generates an underestimation of the final consolidation settlement prediction compared with field monitoring data. Furthermore, it puts the construction project at higher risk. Back calculation with different parameters reveals that a combination of conservative values of C_{c-max} , C_s , and p_{c-max} ' achieve good agreement on final settlement prediction matching with the observational settlement. This parametric study results show that conservative selection of consolidation parameters provide prudent design to ensure construction safety. In the future, utilizing statistical procedure might be applied to guide the parameter selection when a number of soil data are available.

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