

SIMULATION OF VACUUM CONSOLIDATION ON SOFT GROUND BY TRIAXIAL TEST AND ITS APPLICATION

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Abstract

The prediction behavior of improved soft soil using vacuum preloading method should be concerned significantly not only in the laboratory but also in the field. It aims at avoiding the risks of instability of the embankment surcharging during construction. The simulation vacuum preloading method using tri-axial apparatus is proposed to predict the behavior of soft soil improvement in the laboratory and the increasing of undrained shear strength of soil specimen at any degree of consolidation. The simulation has been carried out at Hokkaido University, while the undrained shear strengths, at 40%, 70% and 100% of vacuum consolidation are at 43.2kPa, 53.8kPa and 62.49kPa, respectively by tri-axial apparatus. This modeling is the effective method to determine the increasing rate of strength of the ground corresponding to loading rate during consolidation process, and to restrict using field test during construction, the cause of damage of airtight sheet membrane, and lossing of vacuum pressure. A case study of vacuum consolidation on very soft clay at Nakhorn Sri Thammarat Airport project is presented.

Keywords: Ground improvement, Soft soil, Tri-axial test, Vacuum consolidation

Introduction

The vacuum preloading consolidation method furthermore has been become the popular effective method to improve soft soil introduced by Kjellman (1952) [1] in early 1952 and enhanced in recent years with the merging of new materials and technologies. The prediction behavior of improved soft soil by vacuum preloading method should be concerned significantly in the laboratory and the field to predict and avoid the risks of instability of the embankment during construction.

The modeling vacuum method to improve soft soil in the laboratory has been performed by Indaratna 2008 [2] using the large-scale apparatus and follows one-dimensional consolidation theory (Tezaghi). The results obtained from this modeling get somewhat evaluated the behavior of soft soil reinforced by vacuum preloading method in the laboratory. However using the large specimen 45cm x 90 in diameter and height respectively in the large-scale apparatus, the tested time was more than one month.

In addition, the horizontal deformation (ϵ_r) during tested time, which is the typical deformation of soft soil improvement by vacuum, and the increasing of shear strength of soil specimen, could not be measured. So far, the controlling surcharge processing during vacuum construction has not been discussed sufficiently.

The new method is proposed using tri-axial apparatus to simulate the comprehensive behavior of soft soil improved by vacuum preloading method in the laboratory and to support the engineering task quickly and make this method become familiar in the future. Stability of embankment during construction should be concerned to ensure that the project can perform well with safety.

Solution for Axisymmetric Unit Cell Under Vacuum Pressure

The axisymmetric cell soil has been used for long time to analyze consolidation of soft ground using prefabricated drains [3] and vacuum preloading method. For this solution, the assumptions were used as follow as:

- Soil mass subjected vacuum pressure follows axisymmetric consolidation.
- The vacuum pressure distributed along to the specimen is uniform.
- Under vacuum pressure only, soil mass is subjected isotropic stress state, then the coefficient of horizontal earth pressure (K) equals to one; meanwhile under the surcharge only the value can calculate from Equation (1).

$$K = 1 - \sin \varphi \quad (1)$$

Where

φ – is the friction angle of soil

Hansbo's solution (1981) [4] with assumes as: equal strains (ϵ), variation of the permeability (k), decreasing of void ratio (e) and volume compressibility (m_v) during soft soil consolidation. This solution based on assumptions as follow:

- Soil is homogeneous and fully saturated; the Darcy's law is adopted.
- The permeability of soil is assumed constant during consolidation
- Soil strain is uniform at the boundary of the cell. The small strain theory is valid. Therefore, Hooke's law should be applied for calculation.
- For the soil mass, the vacuum pressure distribution along to the drain boundary is uniform during application.

According to Barron (1948) [5], the degree of consolidation U for "equal-strain" consolidation is given in Equation (2).

$$U = 1 - \exp\left(-\frac{8T}{\mu}\right) \quad (2)$$

With

$$\mu = \left(\frac{n^2}{n^2 - 1}\right) \ln n - \frac{3n^2 - 1}{4n^2} \quad (3)$$

$$n = \frac{de}{dw} \quad (4)$$

Where

de and dw are diameters of unit cell and equivalent of vertical drain respectively.

Hansbo (1981) introduced a circular smear zone (of diameter d_s) in the solution, which resulted in a modified expression for μ :

$$\mu = \ln\left(\frac{n}{m}\right) + \frac{k_{ho}}{k_{hs}} \ln m - \frac{3}{4}; \quad m = \frac{d_s}{d_w}$$

$$T = \frac{C_h \cdot t}{D_e^2} \quad (5)$$

where

C_h – horizontal coefficient of consolidation.

The coefficient of consolidation C_h in the horizontal direction for axisymmetric plane strain deformation is showed as:

$$C_h = \frac{(1-\nu')E'}{(1+\nu')(1-2\nu')\gamma_w} \frac{k_h}{m_v\gamma_w} = \frac{k_h}{m_v\gamma_w} \quad (6)$$

where

k_h – horizontal hydraulic conductivity

γ_w - unit weight of water

Dummy research, the coefficient of volume compressibility (m_v) varies during consolidation and m_v value can be calculated by expression:

$$m_v = \frac{\Delta\varepsilon_v}{\Delta\sigma'_a} \quad (7)$$

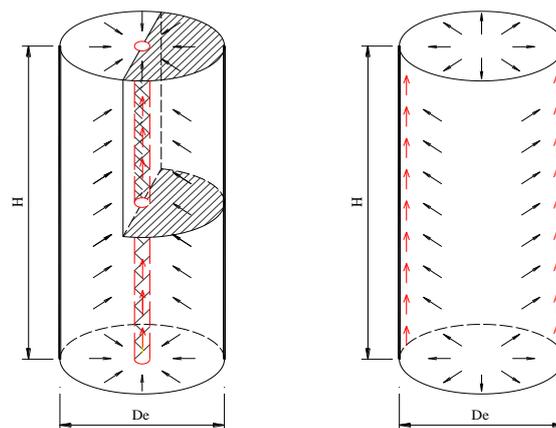
where:

ε_v - volumetric strain of specimen

σ'_a - is the axial effective stress at time reached to degree of consolidation.

Simulation Vacuum Preloading in Laboratory Test

Tri-axial apparatus can clearly evaluate the failure mechanism as well as the capacity of increasing shear strength of soil in the laboratory. Under vacuum pressure condition alone, the soil mass at any depth is subjected the isotropic stress status ($K=1$). With flexible functions of the tri-axial machine, the isotropic condition of the soil mass can generate the same vacuum condition by controlling the lateral earth pressure (K). During vacuum condition, the surcharge loading can be generating as axial force by the loading rod at top of the machine. Only radial drainage is induced during consolidation by the filter paper which covers around the boundary of specimen and overlaps the porous discs at both ends of specimen acted as the drainage layer. For the large-scale oedometer apparatus, the drainage is established at the center of specimen. The Finite Element Method (FEM) has been used to define the different between the drainage path conditions of specimen, and defines the correction factor for this simulation. This relationship has been reported by Duong T.N et.al (2011) [6]. The boundary conditions were illustrated in the Figure 1.



a) Drainage at center b) Drainage at outer boundary

Figure 1. The modeling of axisymmetric cell in FEM

Soil Specimen

The specimens of clay 75mm x 150mm in diameter and height respectively were used for this research. The specimens is suitable to control consolidation time under vacuum condition by tri-axial apparatus.

The serial tests were performed by tri-axial apparatus in the laboratory at Hokkaido University to simulate the behavior and control the instability of Akasaoka clay improved by vacuum preloading method.

The specimens were pre-consolidated under a pressure of 100kPa and OCR=1.25. The effective stresses in vertical and horizontal directions were 80kPa and 40kPa, respectively. Under pre-consolidated condition, the coefficient of horizontal earth pressure at rest is $K_0 = 0.5$. The physical properties of soil are listed in the Table 1.

Table 1. Akasaoka Clay's Properties

Soil Properties	Values
Unit weight (kN/m ³)	17.5
Water content, w (%)	46.5
Liquid limit, WL (%)	62
Plastic limit, WP (%)	27.5
Plasticity index, PI (%)	34.5
Specific gravity, G _s	2.67
Initial void ratio e ₀	1.17

The permeability coefficient (k) and compression index (Cc) obtained from the standard odometer test result were shown in Figure 2.

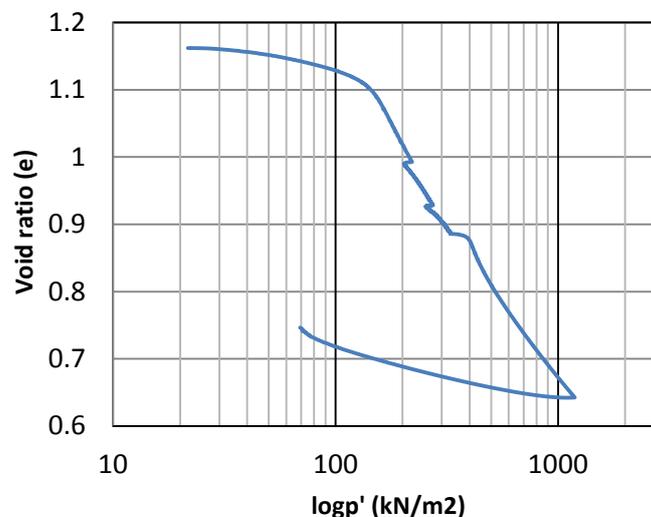


Figure 2 . Void ratio ~log(p') graph

The conventional tri-axial tests were also conducted to verify the failure line (K_f) and the relationship between the coefficient of the horizontal earth pressure (K) and the ratio s_u / σ'_v as shown in Figure 3. The shear strength of soil can be predicted based on this relationship.

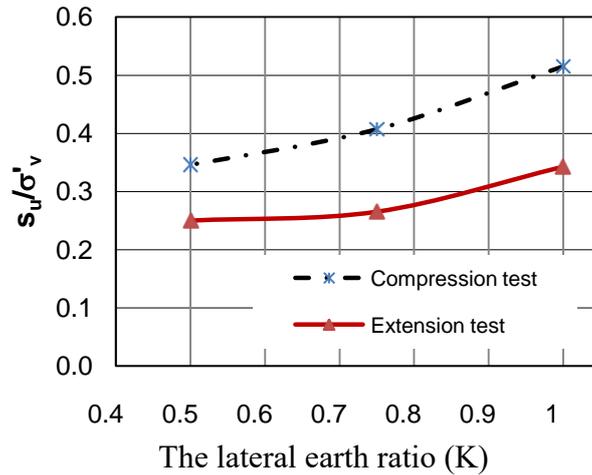


Figure 3. Relationship shear strength, effective stress and earth ratio

Test Procedure to Control the Instability of Soil Specimen Under Vacuum Preloading

It is very important to define the increasing of soil capacity gradually under vacuum preloading method in the laboratory to avoid any risk in cases applied surcharge over the field soil capacity.

Simulation behavior of soft soil improvement by vacuum and surcharge loading can be carried into five stages as: loading step to saturate soil specimen, generating the vacuum pressure condition, applying vacuum, applying surcharge loading and undrained shearing stage.

The loading and recompression step, the specimen is saturated fully with B value more than 0.98 to reach to the initial pre-compression stress. The effective vertical and horizontal stresses in soil specimen gradually reach to 80kPa and 40kPa, respectively after saturation 24 hours.

The soil specimen is subjected under vacuum condition same as the studied period. The behavior of soil mass under surcharge loading and vacuum pressure loading is shown in Figure 4. The generating vacuum pressure condition stage is simulated by applying the effective stress target with the lateral earth pressure ratio equal to one ($K=1$).

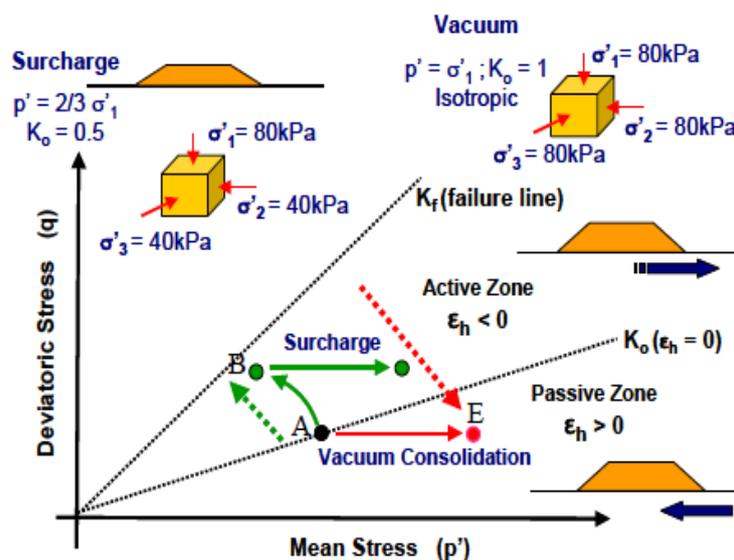


Figure 4. Behavior of soil mass under vacuum and surcharge preloading

During the stage of generating vacuum pressure condition, the drainage vale is closed and then excess pore water pressure (PWP) has been increasing up to the desired vacuum pressure. In this research, vacuum pressures are designed at 50kPa and 100kPa to correspond to the depths of specimens. The PWP was raised up to 250kPa and 300kPa, respectively as shown in Figure 5. The vacuum pressure supplying step occurred in the first stage for 155minutes.

The vacuum pressure is applied by opening the valve as shown in the second stage, PWP has been reduced while the effective stress is gradually increased until the PWP completely dissipated. Finally, the vertical effective stress target at 130kPa and 180kPa were generated.

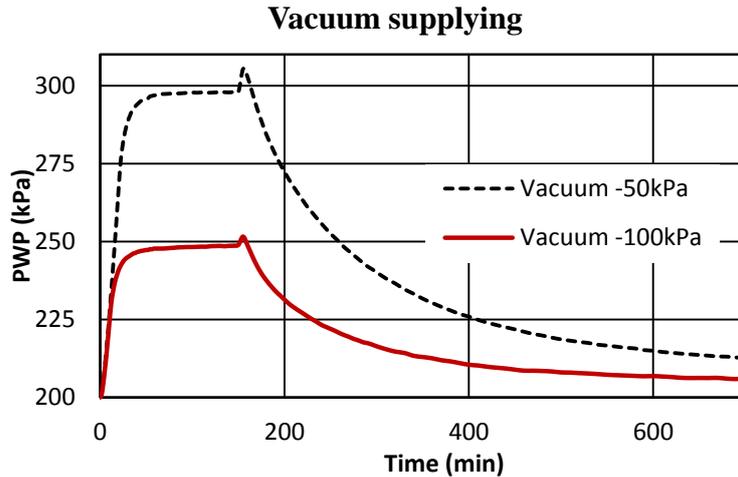


Figure 5. Excess pore water pressure in vacuum condition

The coefficient of horizontal earth pressure (K) is the ratio of the effective horizontal earth pressure due to the confinement from the surrounding soil mass to the vertical effective stress. From Figure 6, The coefficient of horizontal earth pressure K can be estimated as follow:

$$K = \frac{\sigma'_{3+ \sigma'_{va}}}{\sigma'_{1+ \sigma'_{va}}} \quad (8)$$

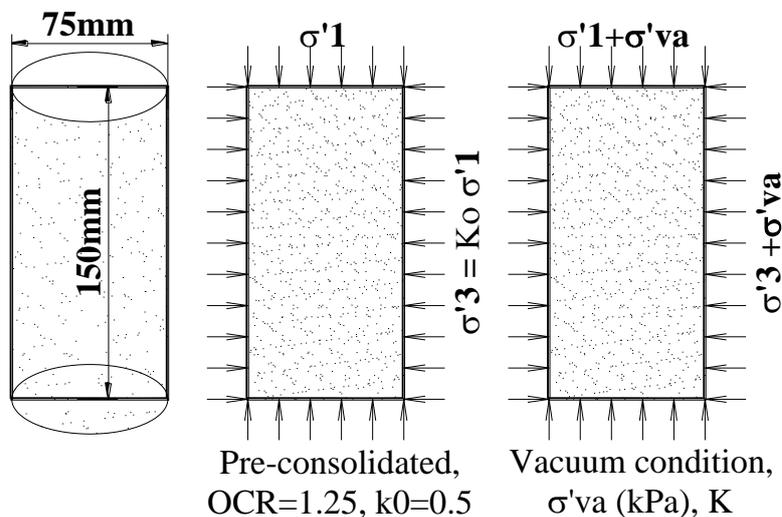


Figure 6. Soil specimen under Vacuum condition

During drained progress stage, the consolidation has been occurred due to the dissipation of excess pore water pressure. The effective stress of soil as well as the shear strength has been increasing, and this behavior matches to the vacuum mechanism (Indraratna 2005) [7]. When the effective stress increases to the target and the excess pore water dissipates completely the soil is fully consolidated. However, data from laboratory test and FEM showed that the end of primary consolidation (EOP) was gained when the excess pore water pressure was dissipates about 95%. The parameter in vacuum proceed by tri-axial apparatus is shown in Table 2 and Table 3.

Table 2. Parameter in Vacuum Proceed by Tri-Axial Apparatus (U=100%)

Step	CP	BP	PWP	σ'_v	σ'_h	(K)
	(kPa)					
Step loading	220	200	200	20	20	1
Recompression	240	200	200	80	40	0.50
Vacuum supplying (50kPa) (Undrain)	290	200	250	80	40	0.50
Vacuum applied	290	200	200	130	90	0.692
Vacuum supplying (100kPa)(Undrain)	340	200	300	80	40	0.50
Vacuum applied	340	200	200	180	140	0.778

Where:

CP - Cell pressure

BP- Back pressure

PWP- Pore water pressure

σ'_v - Vertical effective stress target

σ'_h - Horizontal effective stress target

K – Coefficient of horizontal earth pressure

Table 3. Parameter Analysis in Vacuum Proceed (U=100%, 70%, 40% and 20%)

Recompression Su=22.4kPa		Vacuum Pressure	Target Effective Stress (Vacuum Loading)				U%	Prediction Su	
σ_1	σ_3	K	σ_{va}	σ'_1	σ'_3	K_{va}	Su/σ'_v	Su	
				180	140	0.78	0.42	100	60.80
80	40	0.5	100	143	103	0.72	0.40	70	46.09
				116	76	0.66	0.39	40	35.94
				98	58	0.59	0.37	20	29.17
				130	90	0.692	0.40	100	41.20
80	40	0.5	50	112	72	0.64	0.38	70	34.24
				98	58	0.59	0.37	40	29.17
				89	49	0.55	0.36	20	25.78

Note: All units in kPa

The shearing steps were carried out to define the undrained shear strength of improved soft soil. Depend on the goals; this step could be performed at the time after vacuum loading completely or at the degree of consolidation (DOC) gradually increasing to 40%, 70%, 100% combined with surcharge. The data was analyzed to verify the capacity of improved soil and to control the preloading surcharge at the site to avoid the soil failure or

instability of over surcharge to its capacity.

The capacity of soil can be predicted by empirical method proposed by Tanaka:

$$s_u = (s_u / \sigma'_v) * 0.8 * \Delta p \tag{9}$$

Where

s_u / σ'_v – can be determined from Figure 2

Δp - the loading (vacuum pressure or surcharge loading)

Test Results and Analysis

The simulation result was shown in the Table 4. The surcharges were applied at degree of consolidation of 100%, 70%, and 40% loading.

Table 4. The Data of Vacuum Procedure Simulation to Control Instability of Soil

Vacuum Pressure	Target Effective Stress (Apply Loading)					U%	Prediction	Test
	σ_{va}	$q=2 * S_u = \sigma'_1 - \sigma'_3$	σ'_1	σ'_3	K_{va}		S_u / σ'_v	S_u
100	121.6	261.6	140	0.535	0.358	100	89.08	89.66
	92.176	232.2	140	0.603	0.375	70	82.65	79.36
	71.872	211.9	140	0.661	0.389	40	78.21	75.57
50	82.4	172.4	90	0.522	0.355	100	58.19	62.49
	68.488	158.5	90	0.568	0.366	70	55.15	53.8
	58.336	148.3	90	0.607	0.376	40	45.93	43.2

Note: All units in kPa

The tested undrained shear strength was found to agree well to the predicted S_u as shown in the Figure 7. The maximum different of shear strength was about 6% and 4% for vacuum pressure at 50kPa and 100kPa, respectively. However, the specimen still stable during surcharge was applied.

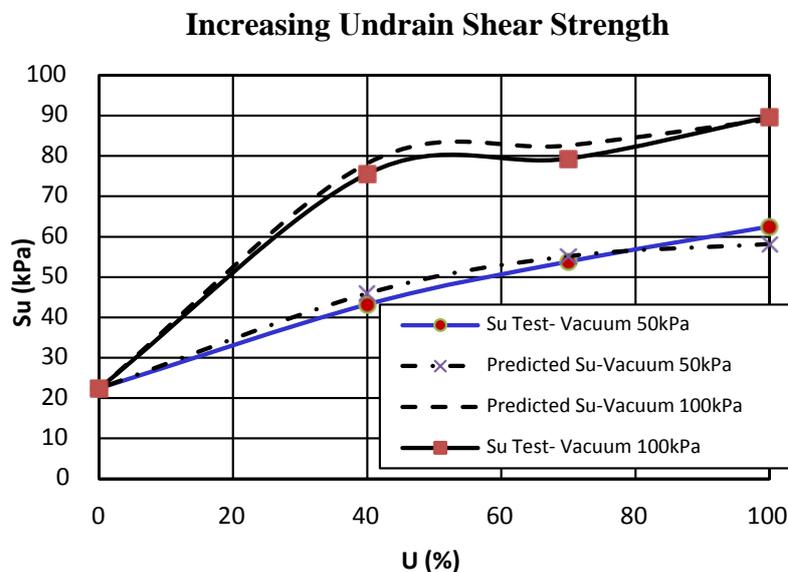


Figure 7. Increasing un-drain shear strength

The final deformation of specimen was nearly same as at end of vacuum stage and vacuum combined surcharge as shown in Figure 8. The largest different in volumetric strain 0.3% occurred at 350min as DOC at 95%. The volumetric strain of 4.92% and 7.12% were found in both cases FEM and laboratory test. The strain ratio ($\varepsilon_r/\varepsilon_v$) illustrated the inward lateral deformation of specimen subjected isotropic stress. This ratio was nearly one during only applied vacuum pressure and was reduced if surcharge was established. These results agree with behavior of soil improvement by vacuum preloading theory.

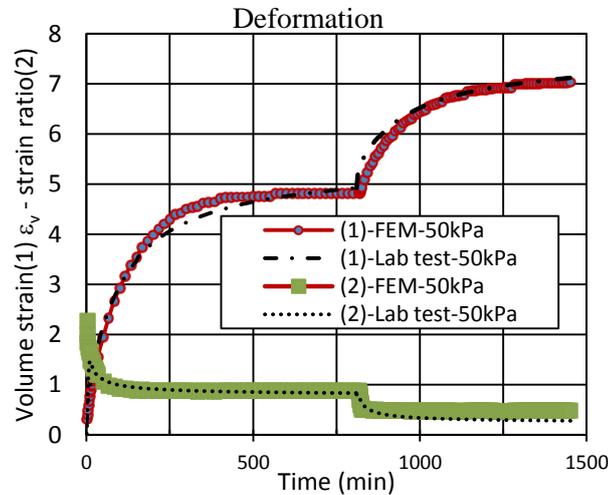


Figure 8. Vacuum combine surcharge; $V_a=50\text{kPa}$, $U=100\%$

The stress path was shown in the Figure 9. There were three stages during vacuum preloading simulated in the laboratory as: pre-consolidation stage (AB), vacuum application stage (BC) and surcharge loading combination stage (CDEF).

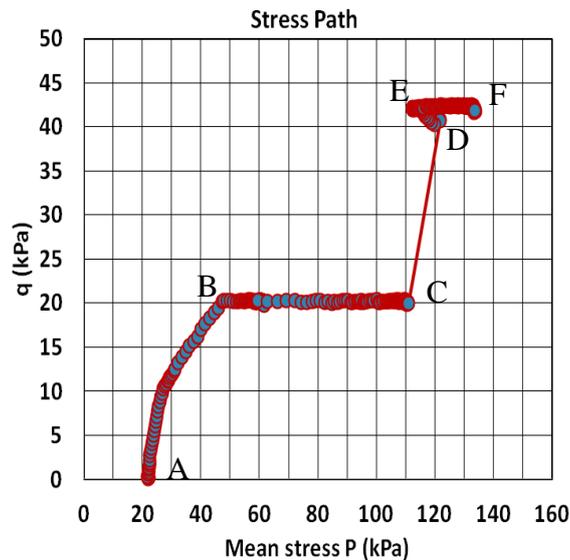


Figure 9. Stress path of soil under vacuum preloading simulation

Under vacuum condition, the stress path of soil moved from B to C, which was far from the failure line. Applying surcharge at point B may lead the stress point to the failure line and cause failure the embankment. At point C when the consolidation at 100% under vacuum pressure surcharge can be applied safety without any risk of embankment. However, the time of construction is longer than apply surcharge at some degree of

consolidation. When surcharge is applied on CD line, the stress path has been changed from D to E and closed to the failure line, then moves to point F.

The increasing of undrained shear strength of soil specimen before improvement and after applied vacuum preloading at some degree of consolidation of 40%, 70%, and 100% were shown in Figure 10.

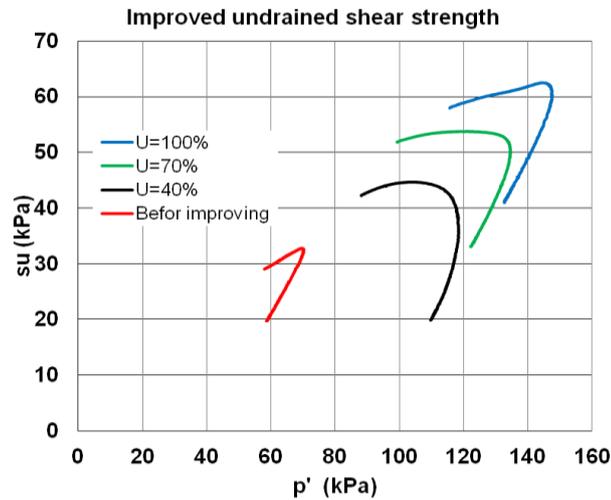


Figure 10. Increasing undrained shear strength

This stage was carried out at end of each test, the failure test were conducted to define the shear strength in undrained condition. The undrained shear strengths defined at 40%, 70% and 100% for vacuum consolidation of 43.2kPa, 53.8kPa and 62.49kPa, respectively. These results agree the approach proposed by Tanaka. The surcharge of each time applied about 10kPa was used for this analysis compare to 0.5m height of embankment.

Case Study of Vacuum Consolidation at Nakhorn Sri Thammarat Airport

The taxiway and apron area of the Nakhorn Sri Thammarat airport at south of Thailand was constructed on 5.0 meter thick of very soft dark gray clay in the swamp area. The project was separated into 8 zones of soft treatment area as shown in Figure 11.

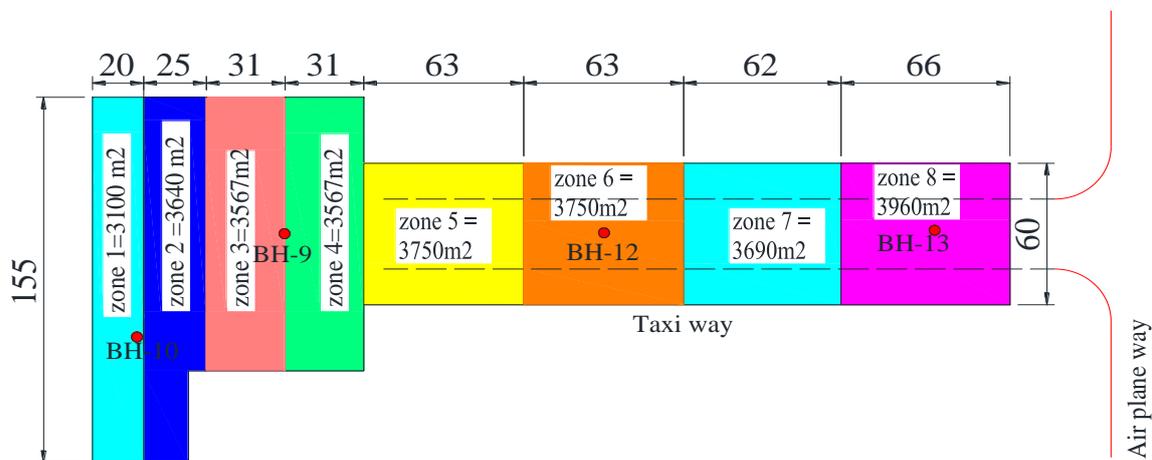


Figure 11. The layout of project for soft soil treatment

The area of each zone is between 3000-4000m² which is matched to the capacity of the vacuum pumping unit and to remain the vacuum pressure more than 70kPa under airtight sheet during construction. The filled sand has been placed on the swamp area of about 1.00m-1.50m thick as a working platform.

The purpose of this separated part is to assess the performance of vacuum consolidation method for very soft soil treatment of 30,000m² of Apron and Taxiway from field instrumentation work. Using field monitoring data for monitoring of accelerating the rate of consolidation and increasing of over consolidation ratio (OCR) could reduce the long term settlement during service period. The construction area was located on the low land and marshy areas. The high embankment (4.00-4.50m) was designed over this low land and marshy areas by preloading method.

Site and Ground Conditions

The soil investigations were carried out at beginning of the project. The field investigation for soft soil comprised boring, sampling and in-situ testing. The boring consists of augering, wash boring with in-situ test included Standard Penetration test (SPT), Vane shear test and cone penetration test (CPTu).

From the soil profile as shown in Figure 12, the soft soil layer is distributed to 5.5m depth from the surface with high content of silt and clay (more than 80% of fine particle). Hence, the depth for soft soil treatment shall be 5.50m from platform. Moreover, the possibility of leakage of vacuum pressure at the tip of the band drain is also low.

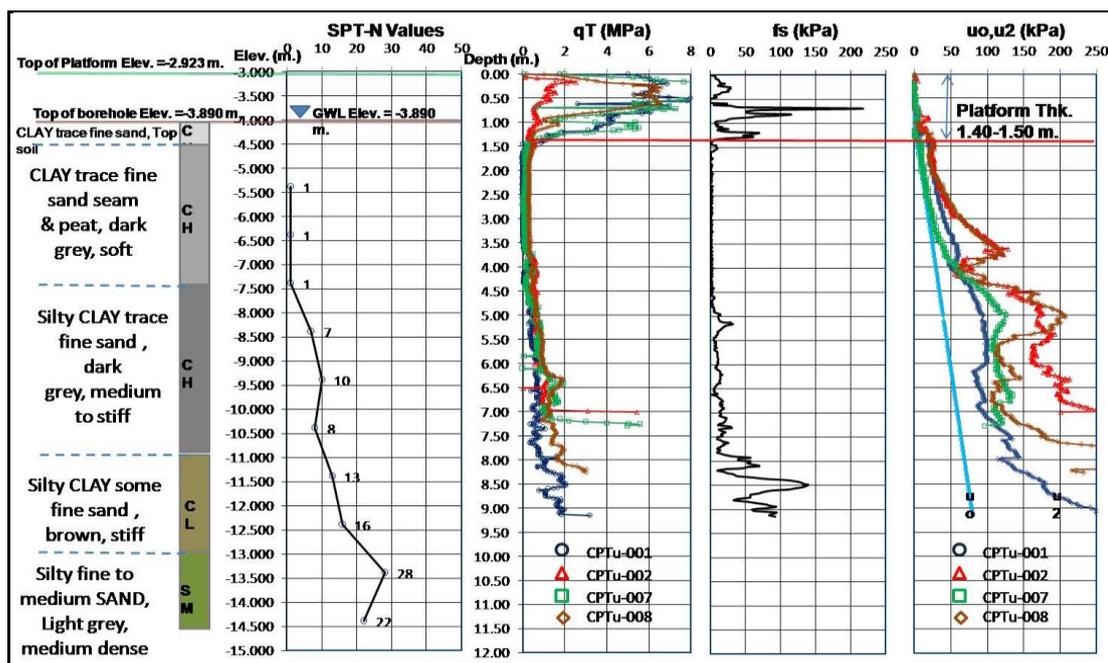


Figure 12. Soil profile of the project

According to the laboratory test, the soil properties at depth 2.00-3.00m from bore hole elevation (depth 3.00-4.00m from platform elevation), are $e_o = 2.492$, $C_c = 0.946$, $CR = 0.271$, and at depth 4.00-5.00m from bore hole elevation (depth 5.00-6.00 from the platform elevation) are $e_o = 0.788$, $C_c = 0.248$, $CR = 0.139$.

The low compressibility parameter of the medium to stiff clay is encountered at depth 4.00-5.00m. Therefore the settlement at this layer is less significant compare to the soft clay between 0.00-4.00m depth. However the deep settlement gauges was installed to check the settlement. At depth 3.75m to 4.25m, the sand lens is encountered with the sand particle content more than 50%.

It would be good to install the standpipe piezometer at the edge of treatment area in the sand lens and can compare the rate of decrease of water pressure with the other depth.

Field Monitoring Works

The field monitoring work has been beneficial in evaluating soil behavior under real field conditions, as well as assessing the performance of new materials and the methods used in the design and construction of geotechnical tasks. The stage of construction (embankment filling) was applied for this project. The embankment was constructed on the soft soil with the rate of filling be governed by the increase in soil strength due to consolidation process and requires close monitoring and communication between design engineer, constructor and supervising engineers. Geotechnical instrument scheme for ground improvement work was designed to ensure safe and economical construction of embankment.

The instrumentations were installed gradually with the vacuum construction technique. The settlement plates and displacement stakes were installed at designed locations for the monitoring purpose before doing any activities. Placing and spreading the embankment (1.00-1.50m thick) on the original very soft ground surface were carried out to provide a suitable working platform.

There were six types of instruments including surface settlement points, sub surface settlement gauges, electric type piezometer, inclinometer, PVC Automatic Acquisition Unit and water discharge record-meters. The types and arrangement of instrumentation are shown in Figure 13.

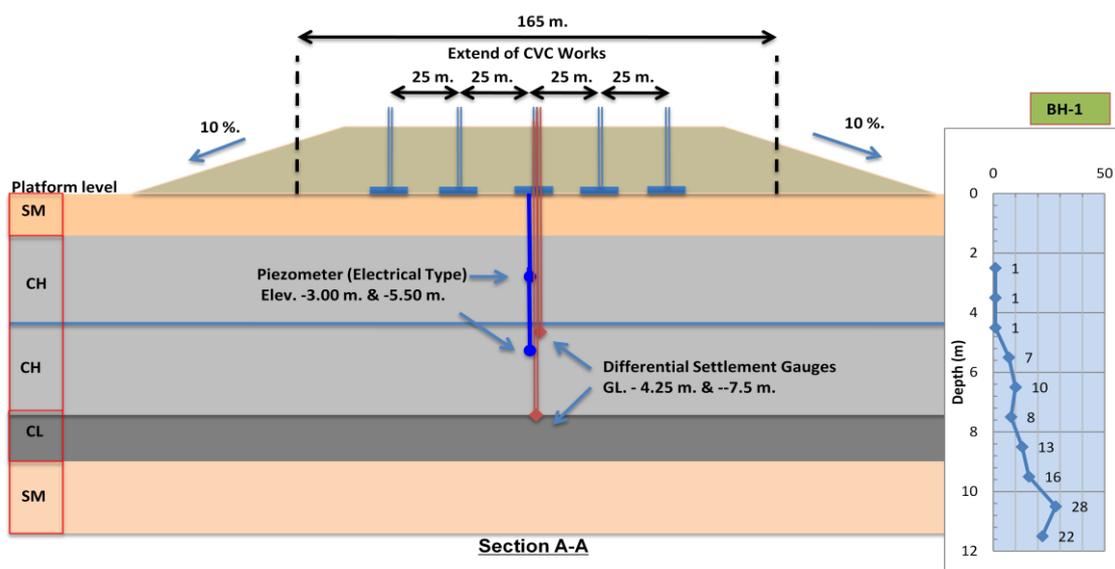


Figure 13. Arrangement of instrumentations

The ARPAS drain KD-100 has been used for the band drain material. The band drain was installed up to 4.50-6.00m depth below working platform to medium clay layer. The band drain installed in square pattern with 1.00m spacing. The vacuum pumps was capable of generating -100kPa pressure.

Data and Analysis

The preloading consists three steps as shown in Figure 14. The first stage only vacuum pressure was applied for two weeks to check leakage of airtight sheets and create the first consolidation degree of soft ground. The second stage was conducted after very soft soil gained some degree of consolidation. The sand fill were applied 3.2m height with eight layers for 55 days, with the thickness of each layer about 35cm. The last stage of loading took place during 65 days. Under this stage the soil was subjected the combination of vacuum pressure and embankment loading. The vacuum is maintained at high pressure of 80kPa during whole construction procedure, the stability of embankment is controlled well, the lateral movement is not presented at the field observation.

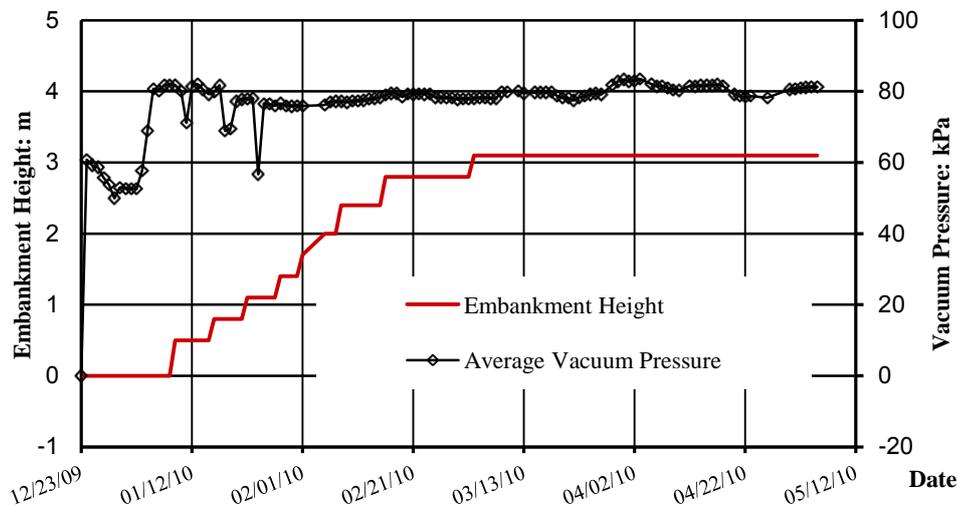


Figure 14. Preloading procedure

The construction procedure is completed when the recorded final settlement reached the required value by Asaoka's method (1978) as shown in Figure 15 to control the settlement of improved embankment. The ultimate settlements were 0.55m, 0.35m and 0.10m at the ground surface, 1.5m depth and 3.5m depth, respectively.

According to the field instrumentation records, the evaluation criteria for stop vacuum operation was assessed by considering the degree of consolidation and rate of settlement. The settlements at certain time intervals were described by Equation (10):

$$S_n = \beta_0 + \beta_1 \cdot S_{n-1} \quad (10)$$

Where S_1, S_2, \dots, S_n are settlement observations. S_n denotes the settlement at time t_n . The time interval $\Delta t = (t_n - t_{n-1})$ is constant. The first order approximation should represent a straight line on a $(S_n$ vs $S_{n-1})$ -co-ordinate.

The values of β_0 and β_1 are given by the intercept of the fitted straight line with the S_n - axis and the slope. The ultimate primary settlement can be calculated by equation expression in Equation (11):

$$S_{lut} = \beta_0 / (1 - \beta_1) \quad (11)$$

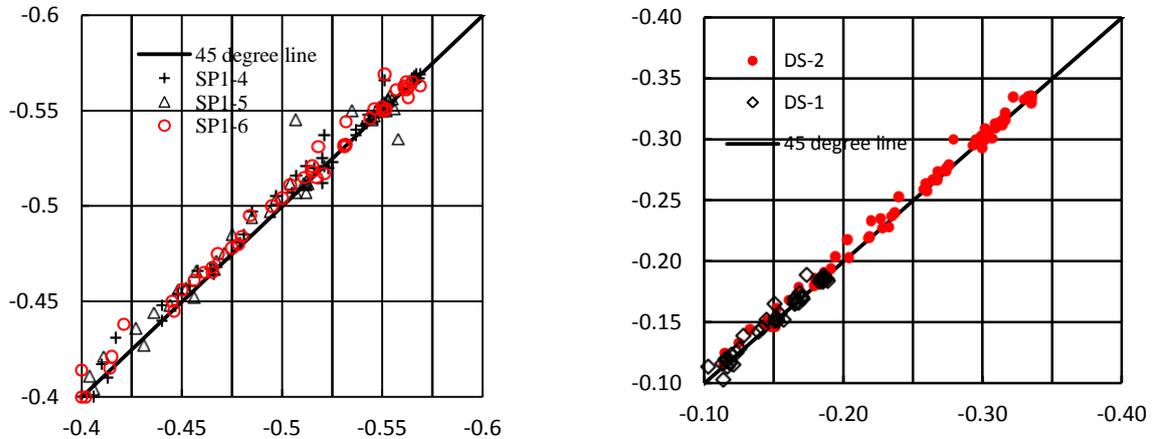


Figure 15. The settlement-Zone GI-1 by Asaoka's method

These settlements are presented in Figure 16, which compared well to finite element method (FEM). The FEM prediction was followed the proposed method for tri-axial test. The different settlement just occurred in the first two weeks, after that they were almost same value, and the final settlement of 0.56m agreed with Asaoka's method and FEM approach.

The average of final settlement was found about 0.57m after 135 days with the degree of consolidation more than 90% and the rate of settlement almost less than 1mm per day.

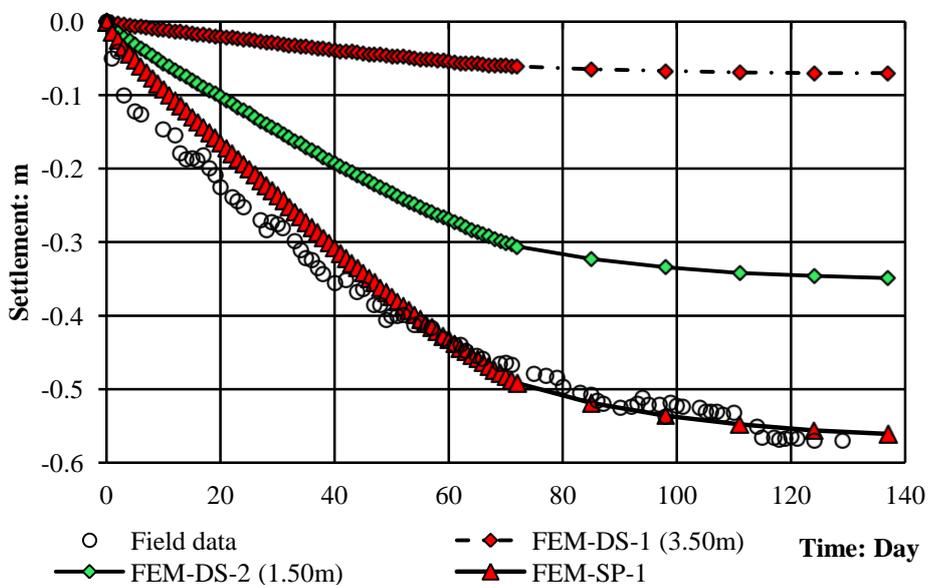


Figure 16. The final settlement-Zone GI-1

Conclusions

This research proposed a method to control the risk of very soft soil improvement by vacuum preloading method by tri-axial apparatus. The result can be applied at the site to support earth works to avoid any instability of embankment.

The modeling by Tri-axial apparatus is the mean to determine the rate increasing strength of the treated ground corresponding to loading rate during consolidation process. The increasing undrained shear strength of soil specimen can be defined at any degree of consolidation. The undrained shear strengths defined at 40%, 70% and 100% of vacuum consolidation are 43.2kPa, 53.8kPa and 62.49kPa respectively.

Case study of vacuum consolidation at Nakorn Sri Thammarat airport is performed well to control the unstable of the loading embankment. The primary consolidation settlement is gained of about 0.57m, with the degree of consolidation more than 90% after only 135days vacuum operation. The period construction accelerated significantly compare to the conventional method. The 1mm/day rate of settlement was measured at last 12 days before stop vacuum operation. The vacuum pressure of 70 to 85kPa of vacuum pressure were generated the overconsolidated state of soil. The improved embankment is safety during construction with carefully proposed procedure.

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