CHAPTER IV
RESEARCH AND ANALYSIS

4.1 Finite Element Method of Analysis for Drainage Boundary Condition Analysis

4.1.1 The Theory of Axisymmetric Consolidation

The unit cell theory representing a single circular drain surrounded by a soil annulus in an axisymmetric condition has been used (e.g. Barron, 1948; Yoshikuni and Nakanodo, 1974; Hansbo, 1981). Most researchers accepted that under embankment loading, the single drain analysis could not provide an accurate prediction due to lateral yield and heave compared to plane strain multi-drain analysis (Indraratna, et al., 1997) although the degree of consolidation (DOC) in this model is acceptable accuracy. In this research, the method is proposed to model the behavior of soft soil improvement by vacuum combine with surcharge preloading method while the lateral displacement is concerned during consolidation.

The following assumptions are based on Hansbo solution (1981) about equal strains (ε), the variation of the permeability (k) when void ratio (e) decrease during consolidation and the volume compressibility (m_v).

(1) Soil is homogeneous and fully saturated; the Darcy's law is adopted. Depend on the purposes the outer boundary of unit cell the drainage path is occurred.

(2) 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.

(3) For the soil mass, the vacuum pressure distribution along to the drain boundary is uniform during application.

The accuracy of the FE numerical was checked against the analytical solutions of Barron (1948) and Hansbo (1981). According to Barron (1948), the degree of consolidation U for "equal-strain" consolidation is given in Eq.(4.1).

$$U = 1 - \exp\left(-\frac{8T}{\mu}\right) \tag{4.1}$$

With $$\mu = \left(\frac{n^2}{n^2-1} \right) \ln n - \frac{3n^2-1}{4n^2}; \qquad n = \frac{D_e}{D_w}$$

Where D_e and D_w 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

$$\mu = \ln\left(\frac{n}{m} \right) + \frac{k_{ho}}{k_{hs}} \ln m - \frac{3}{4}; \qquad m = \frac{D_s}{D_w}$$

For two dimensional consolidation U(%) can be expressed in terms of integrals of the excess pore water pressure over the unit cell domain as Madhav et al. 1993.

$$U = 1 - \frac{\int_y \int_x u(x,y,T)dxdy}{\int_y \int_x u_0 dxdy}; \tag{4.2}$$

Where

u(x,y,T): excess pore water pressure at any point with x, y dimension at a time factor T, u_0: the initial excess pore water pressure.

Terzaghi-Rendulic proposed differential equation for two-dimensional consolidation as

$$\frac{\partial u}{\partial T} = d_e^2 \left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} \right) \tag{4.3}$$

$$T = \frac{C_h.t}{D_e^2} \tag{4.4}$$

where

C_h: horizontal coefficient of consolidation

As the strains are small, if E' is Young's modulus for effective stress, ν' is Poisson's ratio for effective stress and the material is isotropic, Hooke's law is

$$\begin{Bmatrix} \delta\varepsilon_r \\ \delta\varepsilon_\theta \\ \delta\varepsilon_z \end{Bmatrix} = \frac{1}{E'} \begin{Bmatrix} 1 & -\nu' & -\nu' \\ -\nu' & 1 & -\nu' \\ -\nu' & -\nu' & 1 \end{Bmatrix} \begin{Bmatrix} \delta\sigma'_r \\ \delta\sigma'_r \\ \delta\sigma'_r \end{Bmatrix} \tag{4.5}$$

where:

$\{r, \theta, z\}$ is the principal axes set

$\{\varepsilon_r, \varepsilon_\theta, \varepsilon_z\}$ are the strain in radial, circumferential and vertical respectively.

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

$$C_h = \frac{(1-\nu')E'}{(1+\nu')(1-2\nu')}\frac{k_h}{\gamma_w} = \frac{k_h}{m_v \gamma_w} \quad \textbf{(4.6)}$$

Where;

k_h: horizontal hydraulic conductivity

γ_w: unit weight of water

Dummy research, for the axisymmetric unit cell the coefficient of volume compressibility (m_v) varies during consolidation; m_v calculates by expression (4.7):

$$m_v = \frac{\Delta\varepsilon_v}{\Delta\sigma'_a} \quad \textbf{(4.7)}$$

where:

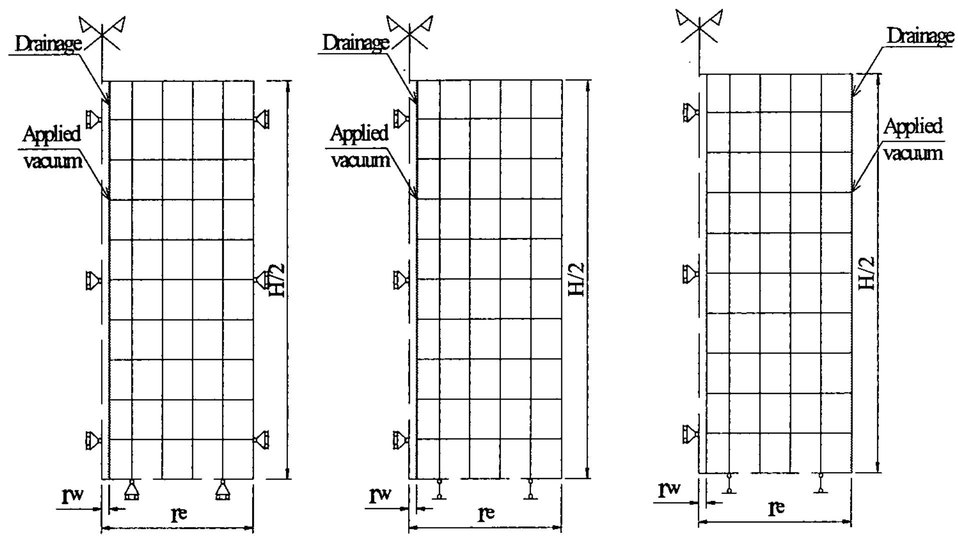
ε_v: volumetric strain of specimen

σ'_a: is the axial effective stress at time reach to degree of consolidation.

4.1.2 Solution for Axisymmetric Unit Cell Under Vacuum Pressure

An axisymmetric system of clay unit cell is used to model the behavior of soil specimen improved by vacuum preloading method.

The unit cell with size 7.5 cm x 15.0 cm in diameter (De) and height (H) respectively (H=2D), the effective Young's modulus E'=500 kN/m2, Poisson's ration ν'=0.33, λ = 0.55, κ =0.06, the horizontal and vertical hydraulic coefficient k_h=k_v=4.66E-10 m/sec and the ratio n = D_e/D_w from 10, 20 are used to analyze. D_w is equivalent diameter of vertical drain as in the **Figure 4.1**. Three cases of axisymmetric unit cells under vacuum preloading only as the **Figure 4.1** were conducted to verify the relationship about degree of consolidation in the different boundary and drainage condition.



a) Fixed boundary (FC) b) Free boundary (NC) c) Free and drainage at boundary (NB)

Figure 4.1. The boundary condition of axisymmetric cell

The first case, the outer boundary is fixed and drainage occurs at the center (FC) of unit cell. This is the conventional model to estimate the consolidation of axisymmetric unit cell, which has been conducted by several researchers before such as Indraratna (2005), Chai (2006), Tran (2007). One-dimensional consolidation theory is used for this case. As we know that, this case is very suitable for both cases the surcharge loading only and the vacuum zone is infinite. Therefore, this method should not apply well for vacuum preloading.

The second and the third case are proposed to verify the consolidation of axisymmetric unit cell under vacuum preloading in term of the lateral displacement as well as two–dimensional consolidation is concerned, the outer boundary of specimen is free or none (N) combines drainage condition at the center (NC) and the outer boundary of unit cell (NB).

The series of case studies for axisymmetric unit cell were conducted by FEM with Camclay Model, which shown in the **Table 4.1**. The cases from N1 to N12 are to check the accuracy of FEM analysis with vary ratio (n) in 10 and 20 with vacuum pressure applied of 50kPa and 100kPa respectively.

The result presents the relationship between three cases boundary and drainage condition of specimen. (FC), (NC) and (NB). The other cases (case N13 to N16) are to estimate applicable surcharge preloading during vacuum procedure.

Table 4.1. The case studies for axisymetric unit cell

N0	Case	Boundary condition	Drainage condition	Ratio D_e/D_w	Vacuum Pressure (kPa)	DOC (U%)
N1	FC-1-50	Fixed	Center	n=10	50	100%
N2	NC-1-50	Free (None)	Center			
N3	NB-1-50	Free	Boundary			
N4	FC-2-50	Fixed	Center	n=20		
N5	NC-2-50	Free	Center			
N6	NB-2-50	Free	Boundary			
N7	FC-1-100	Fixed	Center	n=10	100	100%
N8	NC-1-100	Free	Center			
N9	NB-1-100	Free	Boundary			
N10	FC-2-100	Fixed	Center	n=20		
N11	NC-2-100	Free	Center			
N12	NB-2-100	Free	Boundary			
N13	NC-2-50-1	Free	Center	n=20	50	U=70%
N14	NC-2-50-2	Free	Center			U=40%
N15	NB-2-50-1	Free	Boundary			U=70%
N16	NB-2-50-2	Free	Boundary			U=40%

Comparisons were made for the cases of fix and none fix the outer boundary with ratio n =10 and n=20. The results from the FEM were found to compare well with the analytical solutions Baron (1948). The maximum difference in *U*, for *U*>50%, was about 0.26%, occurring at *T*=0.5 for the case n=10 as shown in **Figure 4.2** and **Figure 4.3.** For the case n=20, the maximum difference in *U* was 0.17%, occurring at *T*=0.5 as shown in **Figure 4.4** and **Figure 4.5.** Conclusion the degree of consolidation in both cases fixed boundary (FC) and free boundary (NC) are almost same value and agree with Baron's theory (1948).

For the cases none outer boundary and drainage at the outer boundary (NB), degree of consolidation are almost same in both case n=10 and n =20 as during vacuum preloading. The **Figure 4.6** and **Table 4.2** show the relationship between the modeling of axisymmetric unit cell in free boundary condition, drainage at the center (NC) and at the boundary (NB) during vacuum stage. As the same DOC, the average ratio of time factor for drainage at the center and outer boundary is T_{hNC}/T_{hNB} about 6.7 and 9.7 for n=10, and n=20 respectively as showed from case N01 to N12.

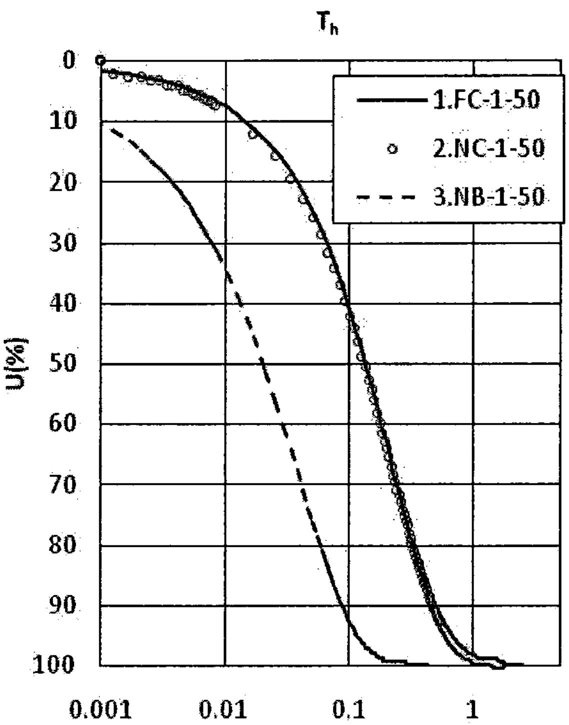


Figure 4.2. Case n=10, vacuum only; Va=50 kPa

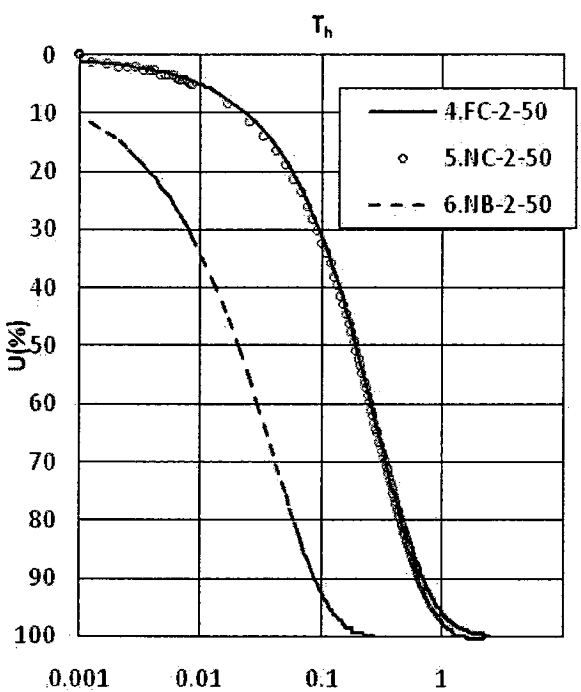


Figure 4.3. Case n=20, Vacuum only; Va=50 kPa

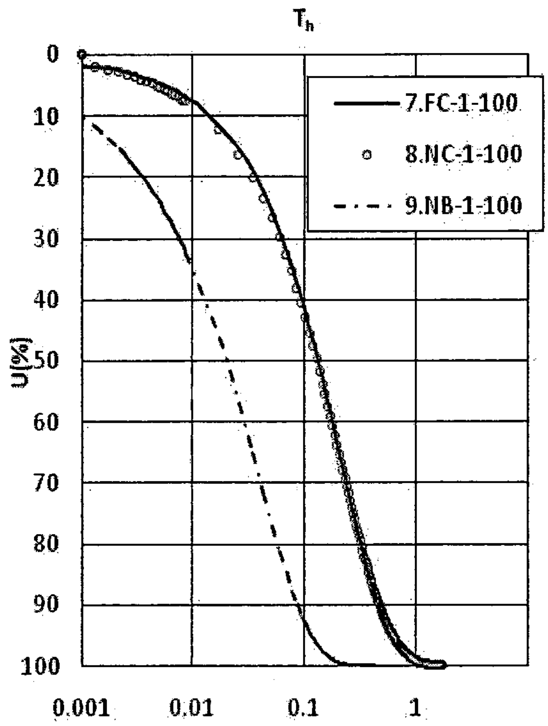


Figure 4.4. Case n=10, Vacuum only; Va=100 kPa

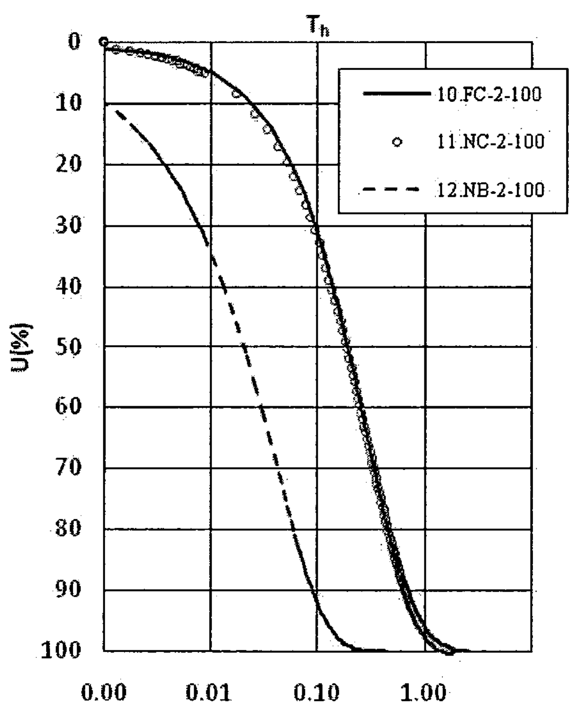


Figure 4.5. Case n=20, Vacuum only; Va=100kPa

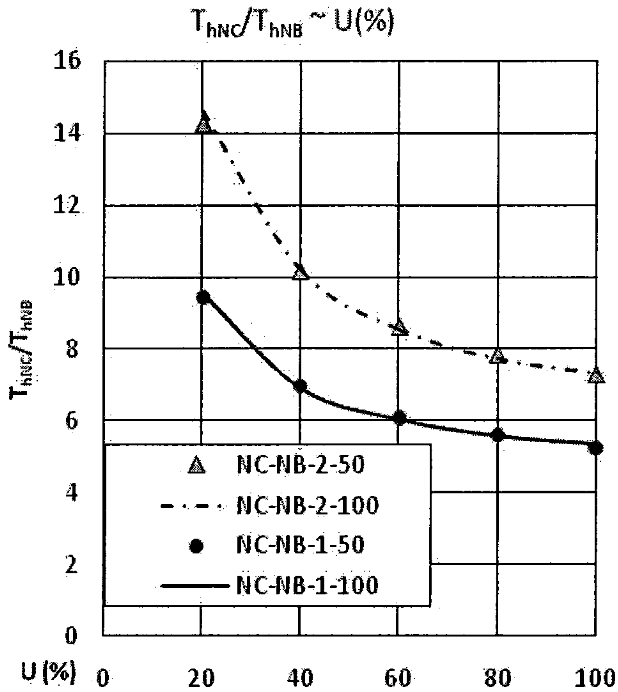


Figure 4.6. Ratio of Time factor ~U(%) for Vacuum stage only

The consolidation procedure of unit cell in (NB) case is faster than (NC) case by ratio (T_{hNC}/T_{hNB}). The different values relative to (n) ratio are shown in the **Table 4.2.**

Table 4.2. The ratio of Time factor

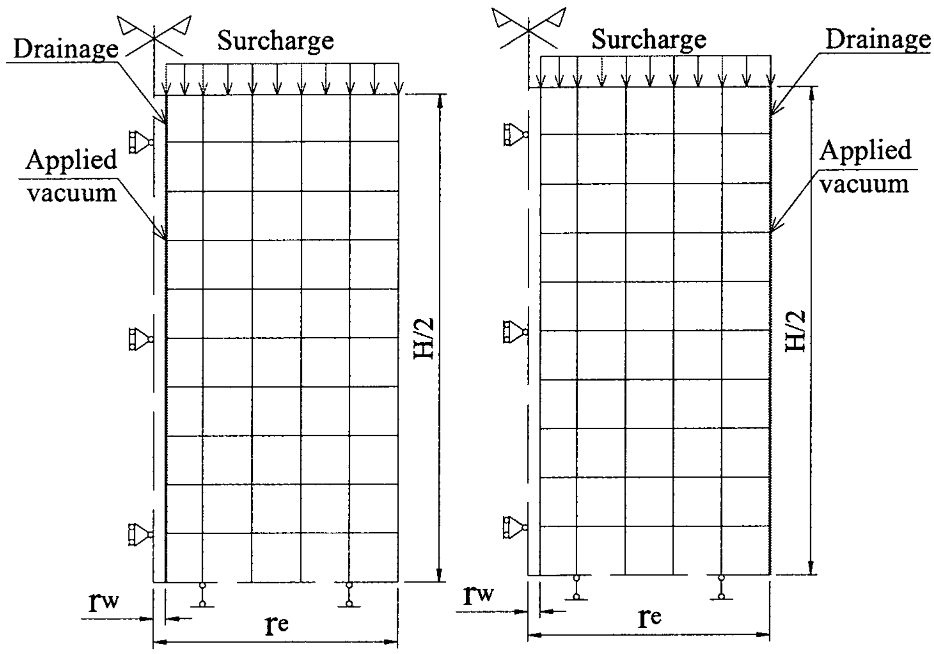
U(%)	20	40	60	80	100
n=20	14.6	10.2	8.57	7.76	7.31
n=10	9.56	6.93	6.08	5.59	5.37

The average coefficient of consolidation (C_h) in both case (NC) and (NB) during vacuum pressure only were found the same value and independent with (n) ratio, $C_{hNC} = C_{hNB} \simeq (2/3)C_{hFC}$.

4.2 Solution for Axisymmetric Unit Cell under Vacuum Combine Surcharge Loading

In this solution, the surcharge is applied after some degree of consolidation induced by vacuum pressure.

Normally, the prefabricated vertical drain (PVD) of dimensions 10 cm x 0.4 cm is installed by rectangular shape. For this research the equivalent diameter of vertical drain D_w = 5 cm and diameter of cell D_e= 100 cm were used. The example with D_e=7.5 cm and D_w=0.3875 cm, and n=20, were used in this analysis and shown in **Figure 4.7**. This unit cell also used to simulate the vacuum preloading in the laboratory test.



a)Drainage at center b) Drainage at outer boundary

Figure 4.7. Vacuum combine surcharge preloading

The surchage stages are applied in three cases of the degree of consolidation (U) reach to 40%, 70% and 100% as shown in the **Table 4.1**.

There are two stages for vacuum preloading method, the first stage is vacuum preloading only until the degree of consolidation reach to the target, and the second stage is surcharge preloading during the vacuum is maintained.

The surcharge loading estimates equal to 80% of increase vertical effective stress to prevent the failure state, for each case the loadings are 19.8 kPa, 29.6 kPa and 42.4 kPa respectively when 50 kPa vacuum pressure is applying.

The loading rate is 0.5 kPa/min for drainage at the center case (NC), the ration T_{hNC}/T_{hNB} are used from the **Table 4.3**, of 10.2; 8.16 and 7.31 as degree of consolidation 40%; 70% and 100% respectively to define the loading rate when the drainage at outer boundary (NB).

The magnitude and rate of loading determined base on the increasing of effective stress of soil mass. The results are shown as follow:

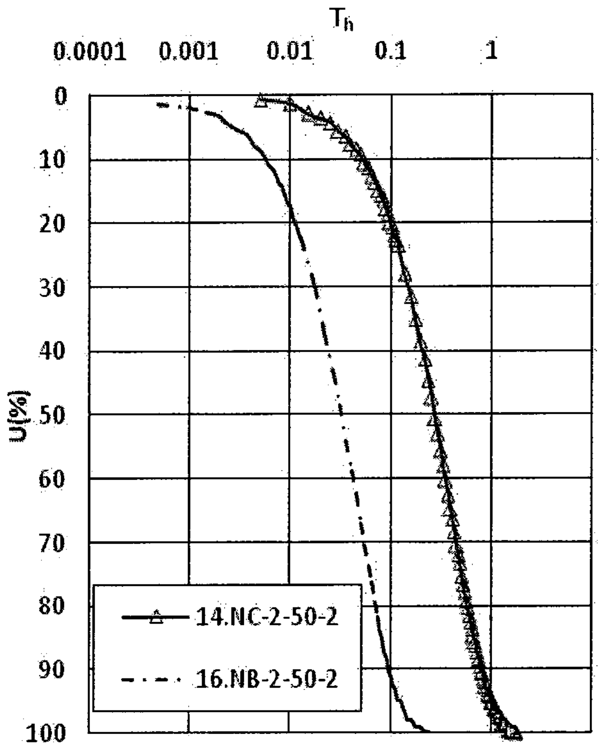


Figure 4.8. Case n=20, Vacuum - Surcharge;
Va=50 kPa, U=40%; T_{hNC}/T_{hNB}= 8.65

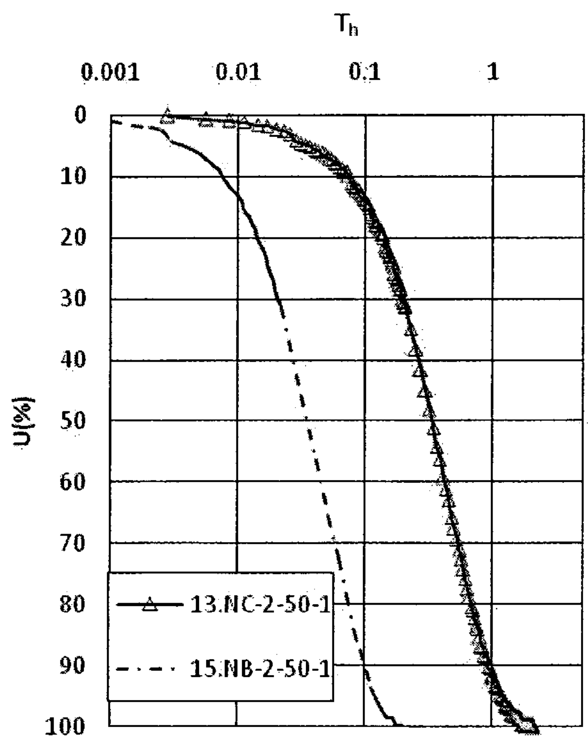


Figure 4.9. Case n=20, Vacuum - Surcharge; Va=50 kPa, U=70%; T_{hNC}/T_{hNB}= 8.71

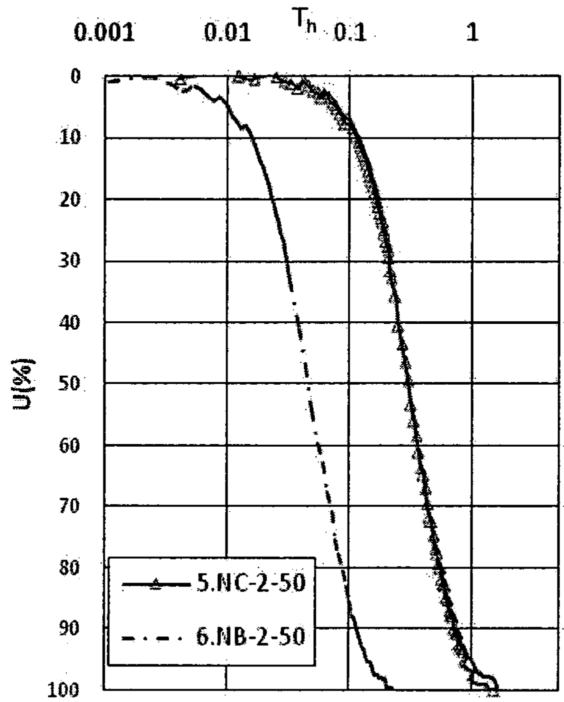


Figure 4.10. Case n=20, Vacuum - Surcharge; Va=50kPa, U=100%

From the **Figure 4.11** and **Table 4.3** the average of ratio of time factor are equal to 8.71 & 8.65 for vacuum pressure are 100 kPa and 50 kPa respectively, when U=100% the ratio of time factor is nearly same value 7.4

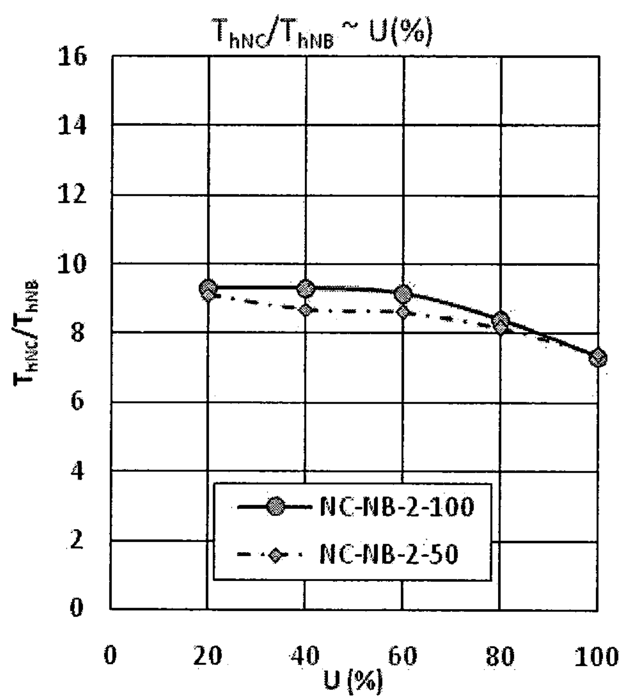


Figure 4.11. Ratio of Time factor ~U(%) for Vacuum - Surcharge preloading

Table 4.3. The ratio of time factor for n=20, Vacuum combine surchage

U(%)	**20**	**40**	**60**	**80**	**100**
100kPa	9.33	9.31	9.16	8.41	7.32
50kPa	9.10	8.70	8.63	8.17	7.41

In the **Figure 4.12**, degree of consolidation at 70%, the maximum different in volumetric strain is 1.75% occuring at 420 min for case 40% is almost the same in the **Figure 4.13.**

From the analyzing above, the final volumetric strain in case outer boundary is almost same value as that in drainage at the center as surcharge applied at 40%; 70% and 100%. Under vacuum 50 kPa and surcharge was applied as degree of consolidation is 100%.

The **Figure 4.14** showed the deformation of unit cell (volumetric strain ε_v) during time of vacuum and vacuum combine surcharge are same in two cases (NC) and (NB) after adjustment with the ratio of time factor T_{hNC}/T_{hNB}.

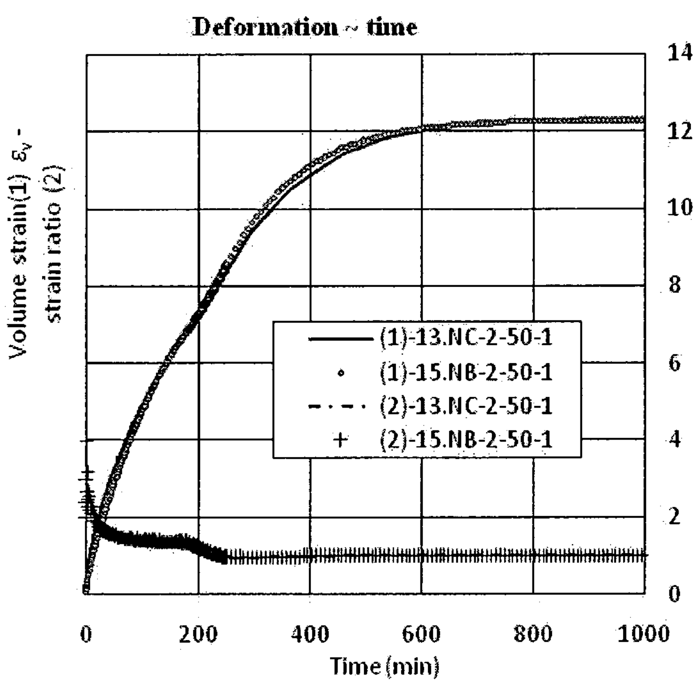


Figure 4.12. Case n=20, Vacuum - Surcharge;

Va=50 kPa, U=70%; T_{hNC}/T_{hNB}=8.71

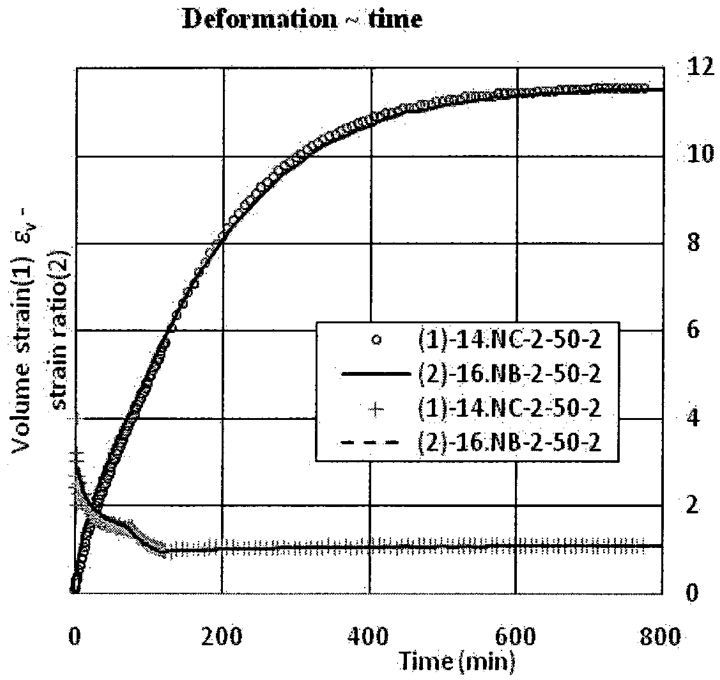


Figure 4.13. Case n=20, Vacuum - Surcharge; Va=50 kPa, U=40%; T_{hNC}/T_{hNB}= 8.65

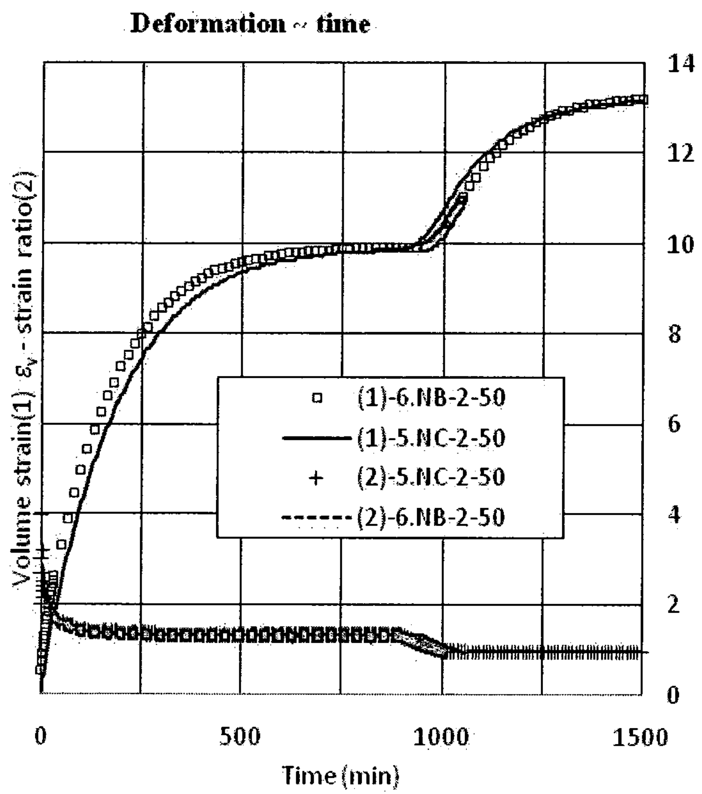


Figure 4.14. Case n=20, Vacuum - Surcharge; Va=50 kPa, U=100%; T_{hNC}/T_{hNB}= 7.31

The strain ration $\varepsilon_r/\varepsilon_a$ are the same in both cases and vary from 4 to 1.3 during vacuum stage and reduce to 0.98 during the surcharge was apply. These results

The strain ration $\varepsilon_r/\varepsilon_a$ are the same in both cases and vary from 4 to 1.3 during vacuum stage and reduce to 0.98 during the surcharge was apply. These results are good agreement with vacuum preloading theory; the lateral deformation of embankment is inward during applying vacuum pressure. These results also agree with the solution above. To make this solution more effectively, the laboratory test by Tri-axial apparatus should be carried out to support this research.

4.3 Simulation in Laboratory Test

4.3.1 Soil Specimen

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

The specimens of clay 75mm x 150mm in diameter and height, respectively were used for this research, which remolded in the laboratory from the commercial Kasaoka clay powder. The specimens are suitable to control consolidation time under vacuum condition by tri-axial apparatus.

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 $K_o = 0.5$. The physical properties of soil are listed in the **Table 4.4**.

Table 4.4. Kasaoka 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, Gs	2.67
Initial void ratio, e_o	1.17

The permeability coefficient (k) and compression index (Cc) obtained from the standard odometer test result, were shown in **Figure 4.15.**

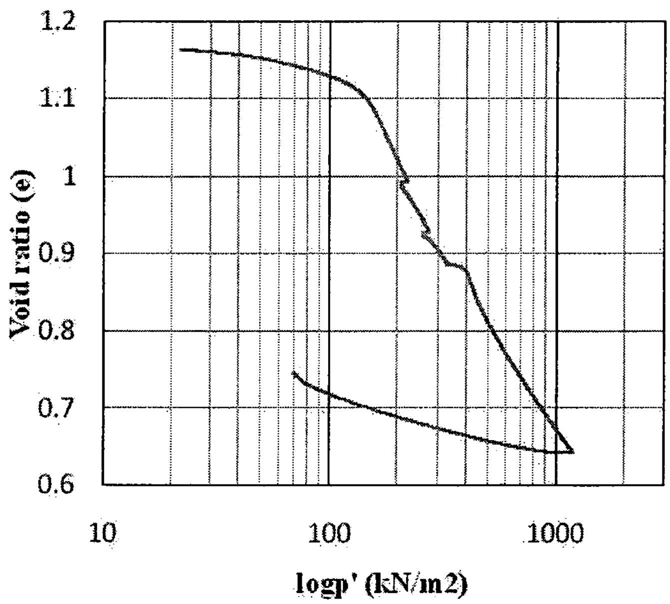


Figure 4.15. Void ratio ~log(p') graph

Series conventional tri-axial test were carried out to verify the failure line (K_f) and relationship between the coefficient K and the ratio s_u/σ'_v, that shown in **Figure 4.16** as well as to predict the shear strength.

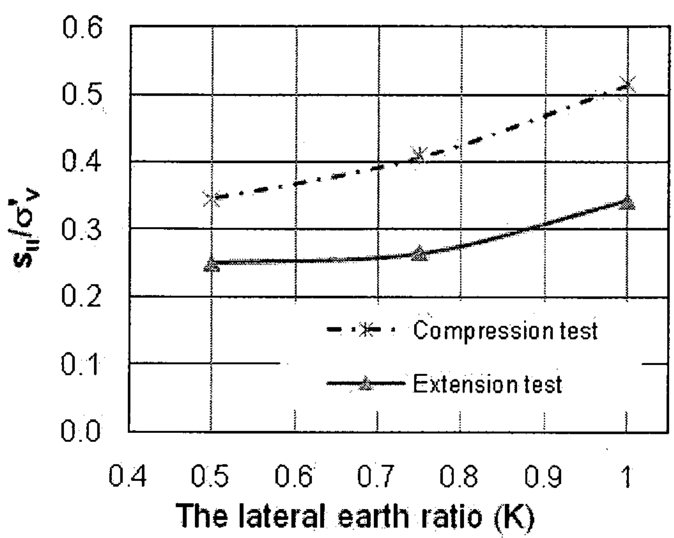


Figure 4.16. Predict shear strength by effective stress and earth ratio

4.3.2 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 vacuum pressure is simulated by applying the effective stress target with the lateral earth pressure ratio equal to one (K=1), the soil specimen is subjected the same condition under vacuum pressure as the period research, the behavior of soil mass under surcharge and vacuum pressure is shown in the **Figure 4.17.**

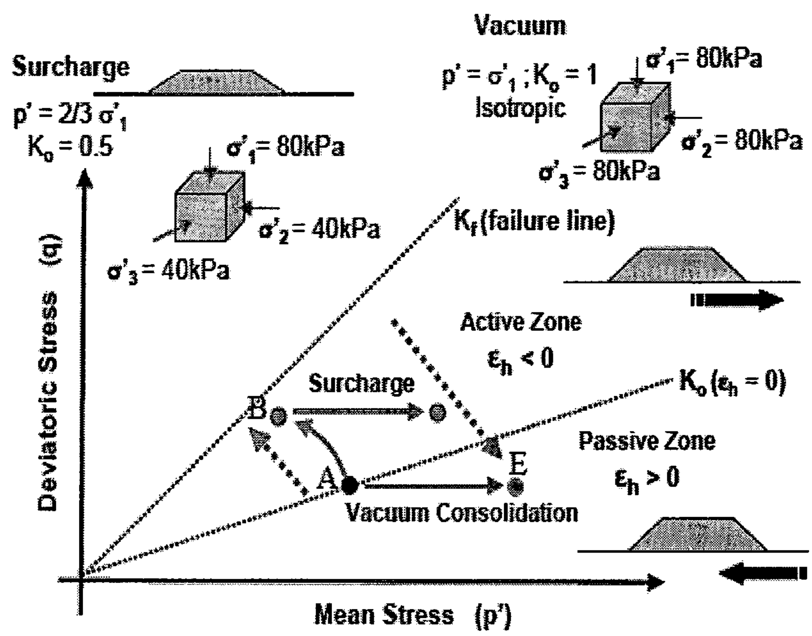


Figure 4.17. Behavior of soil mass under vacuum and surcharge preloading

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 PWPs was raised up to 250kPa and 300kPa, respectively as shown in **Figure**

4.18, vacuum pressure supplying step occurred in the first stage for 155minutes.

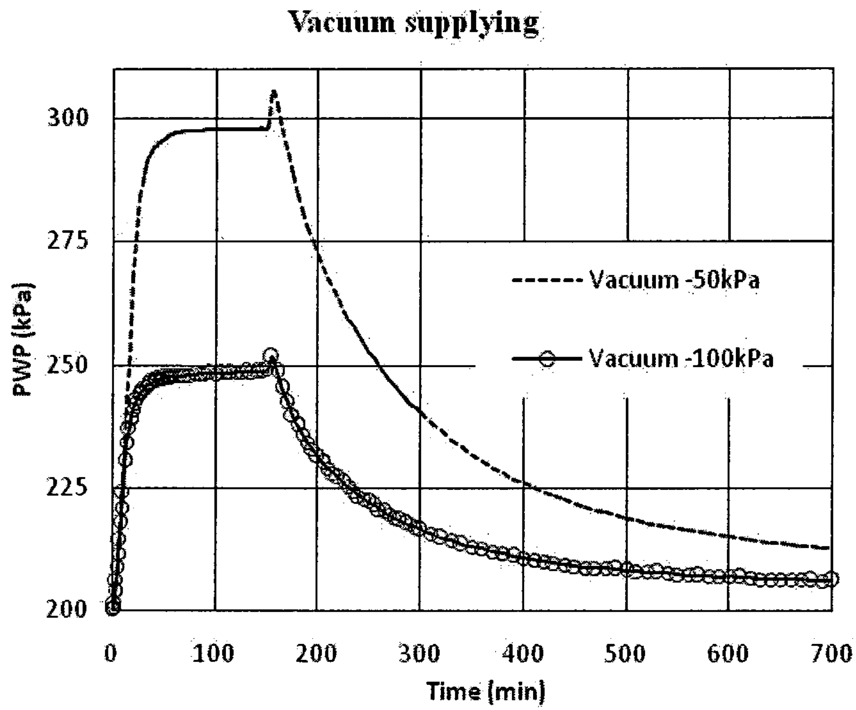


Figure 4.18. Excess pore water pressure in vacuum condition

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.

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 the **Figure 4.19,** K value can estimated as follows:

$$K = \frac{\sigma'_3 + \sigma'_{va}}{\sigma'_1 + \sigma'_{va}} \tag{4.8}$$

During drained progress stage, the consolidation has been occurred due to the excess pore water is dissipated, the effective stress of soil as well as the shear strength will be increase, this behavior is suitable with the vacuum mechanism.

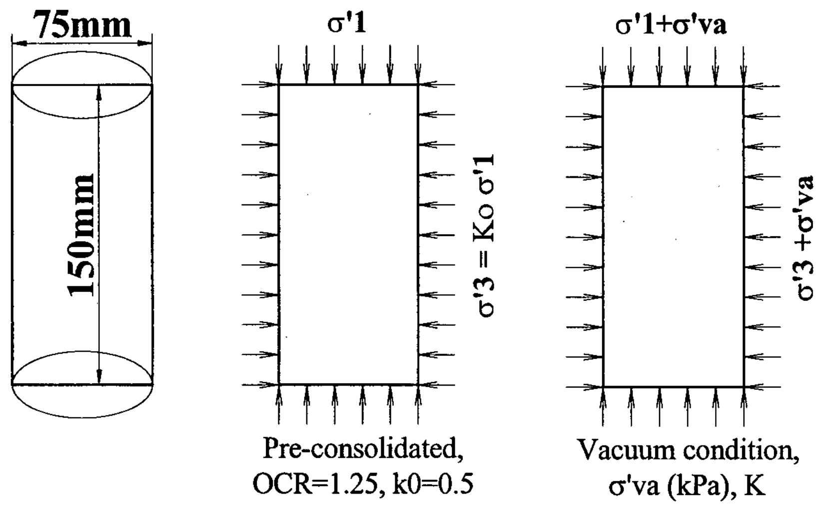


Figure 4.19. Soil specimen under Vacuum condition

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 4.5 and 4.6.**

Table 4.5. Parameter in vacuum proceed by tri-axial apparatus (U=10%)

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) (Undrained)	290	200	250	80	40	0.50
Vacuum applied	290	200	200	130	90	0.692
Vacuum supplying (100kPa)(Undrained)	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 4.6. Parameter analysis in vacuum proceed by tri-axial apparatus (U=100%, 70%, 40% and 20%)

Recompression s_u=32.4kPa			Vacuum pressure	Target effective stress (Vacuum loading)				U%
σ_1	σ_3	K	σ_{va}	σ'_1	σ'_3	K_{va}	s_u/σ'_v	
				180	140	0.78	0.42	100
80	40	0.5	**100**	143	103	0.72	0.40	70
				116	76	0.66	0.39	40
				130	90	0.692	0.40	100
80	40	0.5	**50**	112	72	0.64	0.38	70
				98	58	0.59	0.37	40

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 predicted by empirical method proposed by Tanaka:

$$s_u = (s_u / \sigma'_v) * 0.8 * \Delta p \tag{4.9}$$

where;

s_u/σ'_v: can be determined from Figure 4.16

Δp: the loading is applied (vacuum pressure or surcharge loading).

The undrained shear strength of soil specimen from lab test result is analysis to check the capacity of soil improved.

4.3.3 Test Results and Analysis

The simulation result was shown in the **Table 4.7**. The surcharges were applied at degree of consolidation of 100%, 70%, and 40% loading.

at degree of consolidation of 100%, 70%, and 40% loading.

Table 4.7. The data of vacuum procedure simulation to control instability of soil

Vacuum pressure	Target effective stress (Apply loading)					U%	Prediction	Test
σ_{va}	$q=2*Su$ $=\sigma'_1-\sigma'_3$	σ'_1	σ'_3	K_{va}	Su/σ'_v		Su	Su
	121.6	261.6	140	0.535	0.358	100	89.08	89.66
100	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
	82.4	172.4	90	0.522	0.355	100	58.19	62.49
50	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

These results agree with behavior of soil improvement by vacuum preloading theory. The **Figure 4.20** and **Figure 4.21** are shown the relationship between case (NB) in FEM and lab test, the vacuum pressure of 50 kPa was applied only and combination to surcharge respectively. In the both cases the DOC in 100% were induced, the largest different of U is 3.5% between FEM and lab test occurring at T_h=0.3.

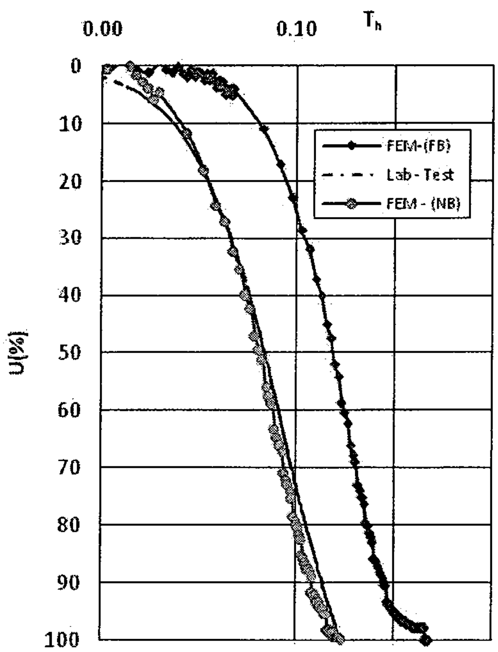


Figure 4.20. Case n=20, Vacuum only; Va=50 kPa, U=100%

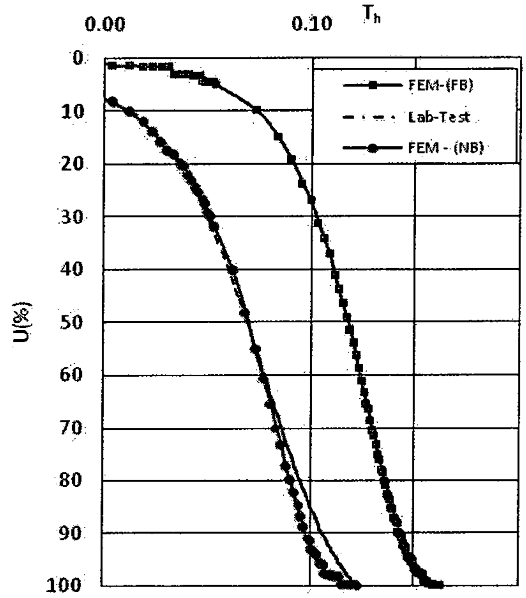


Figure 4.21. Case n=20, Vacuum - Surcharge; Va=50kPa, U=100%

The final deformation of specimen was nearly same as at end of vacuum stage and vacuum combined with surcharge as shown in **Figure 4.22.** The largest different in volumetric strain 0.3% occurred at 350 min as DOC at 95%.

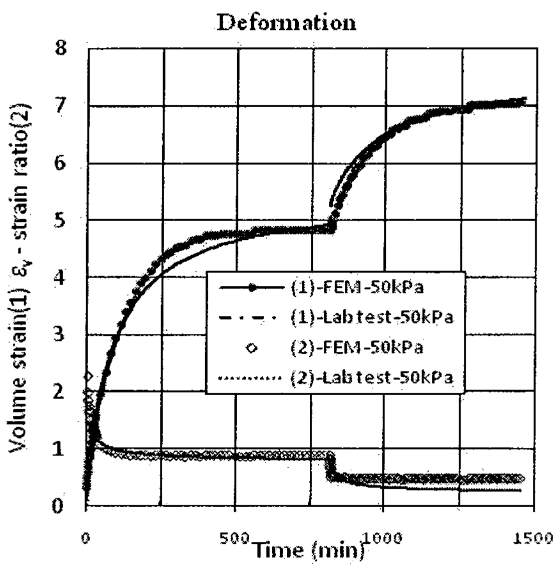


Figure 4.22. Case n=20, Vacuum - Surcharge; Va=50kPa, U=100%

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.

The tested undrained shear strength was found to agree well to the predicted s_u as shown in the **Figure 4.23**.

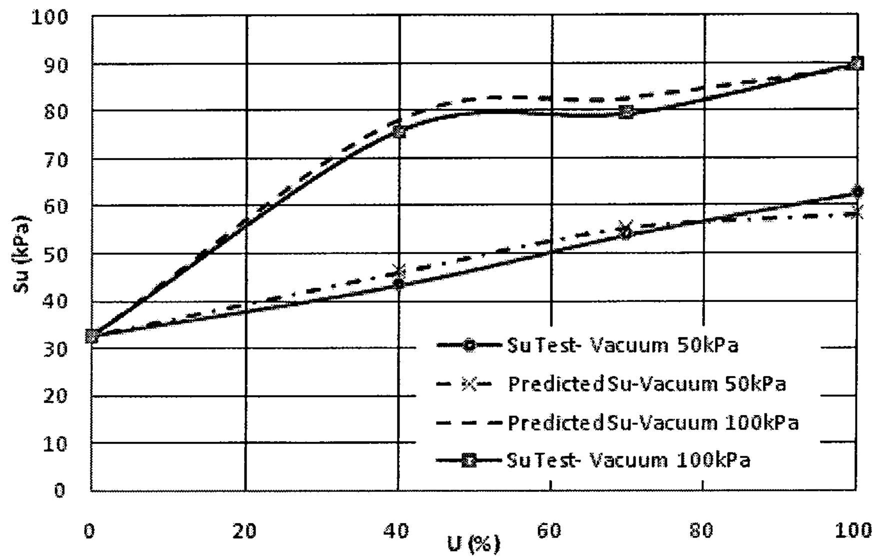


Figure 4.23. Increasing undrained shear strength

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. The stress path was shown in the **Figure 4.24**. There were three stages during vacuum preloading simulate in the laboratory as: pre-consolidation stage (AB), vacuum application stage (BC) and surcharge loading combination stage (CDEF).

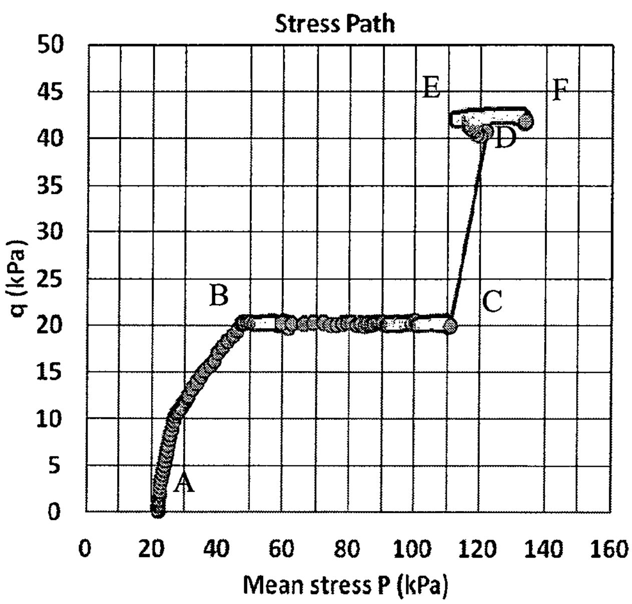


Figure 4.24. Stress path

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 is closed to the failure line, then moves to point F. The behavior of soil specimen under vacuum preloading method simulated by Tri-axial apparatus is matched the before studies as shown in Figure 4.17.

The increasing of undrained shear strength of soil specimen after applied vacuum preloading at some degree of consolidation of 40%, 70% and 100% were shown in **Figure 4.25.** 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.

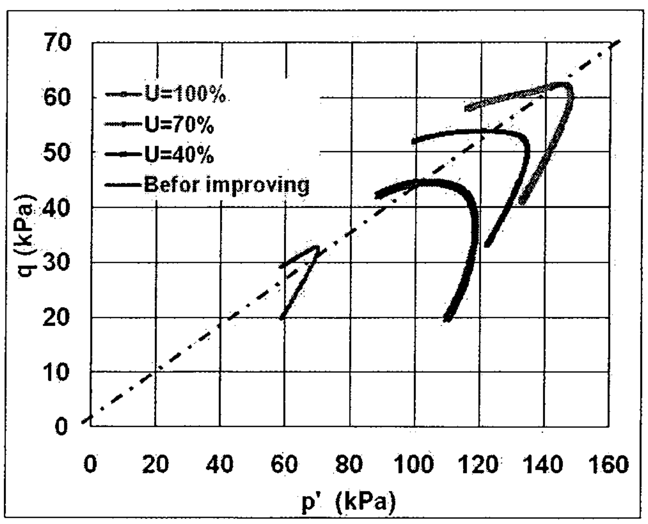


Figure 4.25. Increasing undrained shear strength

4.4 Summary Conclusions

The study has been developed based on the combination of finite element analysis and the results of laboratory experiments to simulate a new appropriate method. This method can be widely applied for soft ground improvement by vacuum preloading method. The results from the study can be summarized as follows:

4.4 Summary Conclusions

The study has been developed based on the combination of finite element analysis and the results of laboratory experiments to simulate a new appropriate method. This method can be widely applied for soft ground improvement by vacuum preloading method. The results from the study can be summarized as follows:

1) The axisymmetric unit cell used to model the behavior of soil treatment by vacuum preloading as none boundary conditions are considered, the behavior of soil is more close to real soil state in the field.

2) Two cases of drainages boundary showed that time for consolidation in cases outside drainage of unit cell is faster than at the center by T_{hNC}/T_{hNB} ratio. However, the deformations of the specimens in all cases are in same shape and value with the same applied condition.

3) Results of experiments by tri-axial apparatus entirely agree with FEM model. It is suggesting that the theories are given full compliance, highly compelling to predict the behavior of soil improvement by vacuum preloading method.

4.5 Case study of Vacuum Consolidation at Nakhorn Sri Thammarat Air port.

4.5.1 Site Description

The project was separated into 8 zones of soft soil treatment area which is shown in **Figure 4.26**. The area of each zone is between 3000-4000 m^2, 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 1.00 m-1.50 m thick as a working platform.

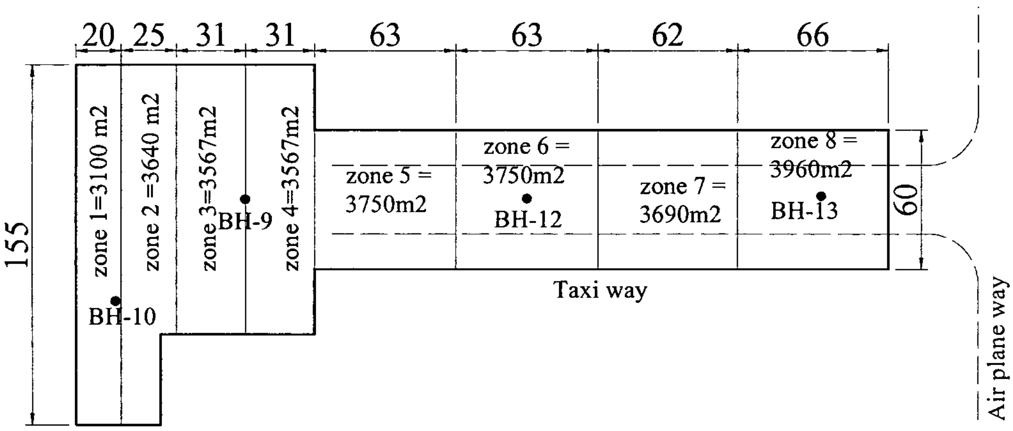


Figure 4.26. The CVM is applied

The purpose of this part is to assess the performance of vacuum consolidation method for very soft soil treatment of 30,000 m^2 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 cross section of embankment is designed in 4.0m high in sand over the sand mat is used to replace mud on the original ground as shown as **Figure 4.27**.

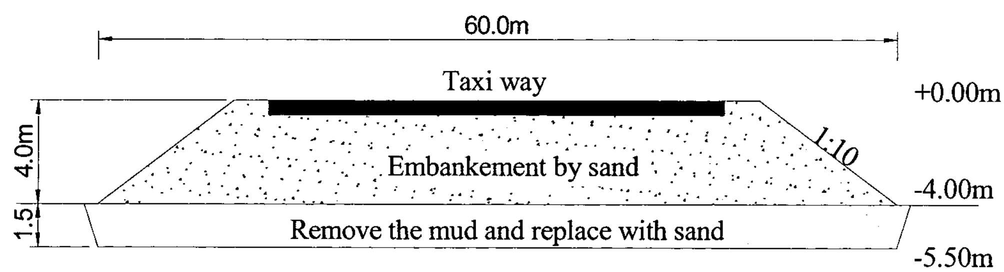


Figure 4.27. The cross section of the Taxiway

The construction area was located on the low land and marshy areas. The high embankment (4.00-4.50 m) was design over the low land and marshy areas.

4.5.2 Soil Conditions

The soil investigations were carried out on beginning of the project. The field investigation for soft soil comprised boring, sampling, 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). The field work undertaken during field investigation shown in **Table 4.8.**

Table 4.8. Summary of field investigations in soft soil

Type of investigation	Quantity (Nos)
Boreholes	12
SPT Test	98
Vane shear strength	48
CPTu Test	12

From the soil profile as shown in **Figure 4.28,**

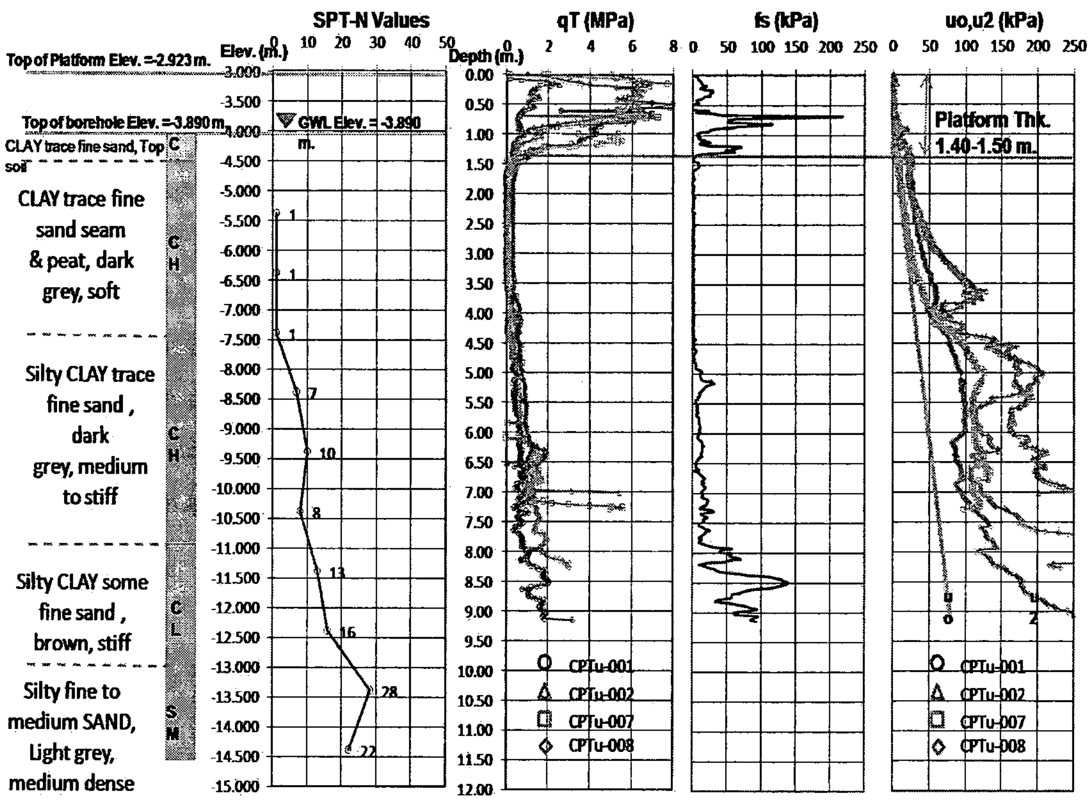


Figure 4.28. Soil profile of the project

The soft soil layer distributed to 5.5m depth from the surface with high content of silt and clay (more than 80% of fine particle). Therefore, 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. According to the laboratory test, the soil properties at depth 2.00-3.00m from bore hole elevator (depth 3.00-4.00m from platform elevator), are e_o = 2.492, Cc=0.946, Cr = 0.271, and at depth 4.00-5.00m from bore hole elevator (depth 5.00-6.00m from the platform elevator) are e_o=0.788, Cc=0.248, Cr=0.139.

The low compressibility parameter of the medium to stiff clay is presented at depth 4.00-5.00m; hence, the settlement at this layer is less significant compare to the soft clay between 0.00-4.00m. However, the deep settlement gauges installed to check the settlement. At depth 3.75m to 4.25m, the sand lens located with the sand particle content more than 50%. It would be good to install the standpipe piezometer the edge of treatment area the depth of sand lens and compare the rate of decrease of water pressure with the other depth. The index and engineering properties of soft soil at depth 0.0-4.0m from existing ground that found as a soft to very soft clay (CH) are shown in Table 4.9.

Table 4.9. Summary Index and Engineering Properties of soil at depth 0.00-4.00m

No	Index and Engineering properties	Values
1	Moisture contents (%)	75-90
2	Void ratio, e	2.38-2.49
3	Specific gravity, Gs	2.62-2.66
4	Bulk unit weight (g/cm^3)	1.42-1.51
5	Percent of saturation (%), Sr	94-98
6	Compression Index, C_c	0.932-0.945
7	OCR	1.10
8	Coefficient of consolidation, C_v (m^2/year)	3.2
9	Undrained shear strength from UU Test (kPa)	10-14
10	Undrained shear strength from Vane Shear Test (kPa)	15-20

to 23 kPa with depth. The soil from surface to 6.0 m depth is very soft clay, with high compressive and high water content.

4.5.3 Construction and Field Monitoring Work of Vacuum Preloading Method

The field monitoring work has been beneficial in evaluating soil behavior work 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, construction 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 as follow as:

(1) The settlement plates and displacement stakes installed at designed locations for the monitoring purpose before doing any activities.

(2) Placing and spreading the embankment (1.00-1.50 m thick) on the original very soft ground surface were carried out to provide a suitable working platform.

(3) The ARPAS drain KD-100 has been used for the band drain material. The band drain was installed up to 4.50-6.00 m depth below working platform to medium clay layer. The band drain was installed in square pattern with 1.00 m spacing.

(4) The piezometers and differential settlement gauges were installed with appropriate depth and locations.

(5) The perforated pipes were installed in the interval 20-30 m in the stabilized area.

(6) The primary and secondary separate tanks was installed and connected to the all of the pipe line systems.

(7) The horizontal drains were laid in perpendicular to the perforated pipes and passing under the pipes for accelerating the water flow to the pipes.

(8) The non woven geo-textile (protection sheets) and airtight sheets was placed to protect the vacuum systems from leakage.

(9) Excavation of peripheral trenches (1.50m. depth) and laying of the airtight sheets up to the trench and backfilling with the embankment material were carried out to provide the anchorage length for airtight sheets.

(10) The main tank was connected to vacuum pipes as well as the water and air pipes. The vacuum pumps was capable of generating -100kPa pressure.

(11) Start vacuum pumping operation. During 1st week of operation, the daily checking to investigate and repair for the leakage points on the airtight sheets.

(12) Preloading by embankment filling will be performed layer by layer.

(13) Loading embankment layer 2, 3 and 4; during loading the pore water pressure and settlement as well lateral movement will be measure to control the stability of the embankment; Check the settlement; if it is reach 90% degree of consolidation, the vacuum pump will stop;

There are six types of instruments including surface settlement plates, sub surface settlement gauges, electric type piezometer, inclinometer, PVC Automatic Acquisition Unit and water discharge record meters in the instrumentation program. The types and arrangement of instrumentation are shown in **Figure 4.30**.

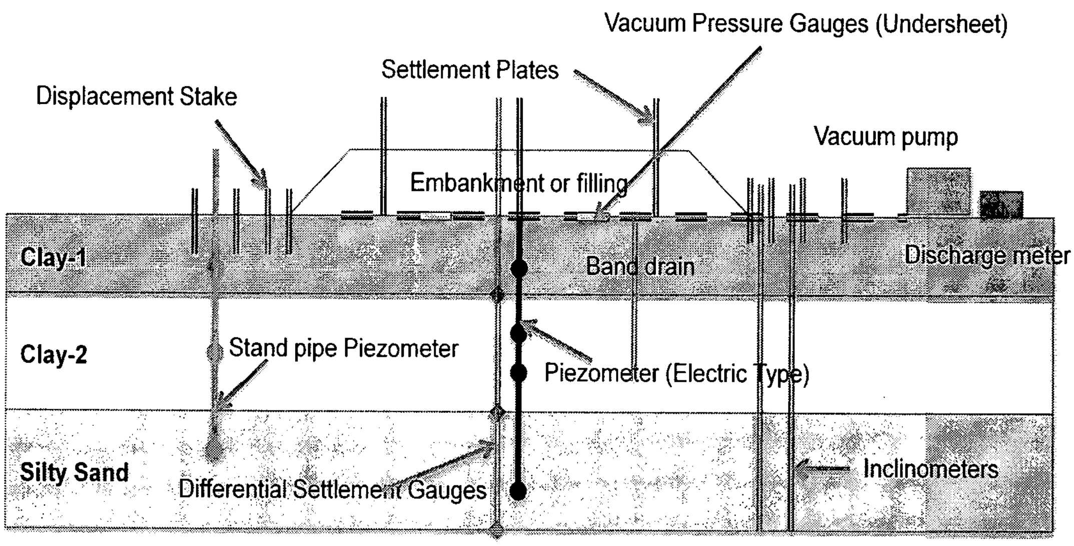


Figure 4.30. Arrangement of instrumentations

List of functions and frequency for different types of instrumentation are shown in **Table 4.10**

Table 4.10. The lists and nos of field instrumentation works

GI-Zone	Piezometer	Settlement plates	Sub-surface settlement	Inclinometer
1	1	8	3	2
2	1	5		
3	2	4	2	
4		3		
5	2	6	2	
6	1	3	2	
7	1	3	2	
8	1	3	2	
Total	**9**	**35**	**13**	**2**

The Schematic of CVC method is shown as Figure 4.30

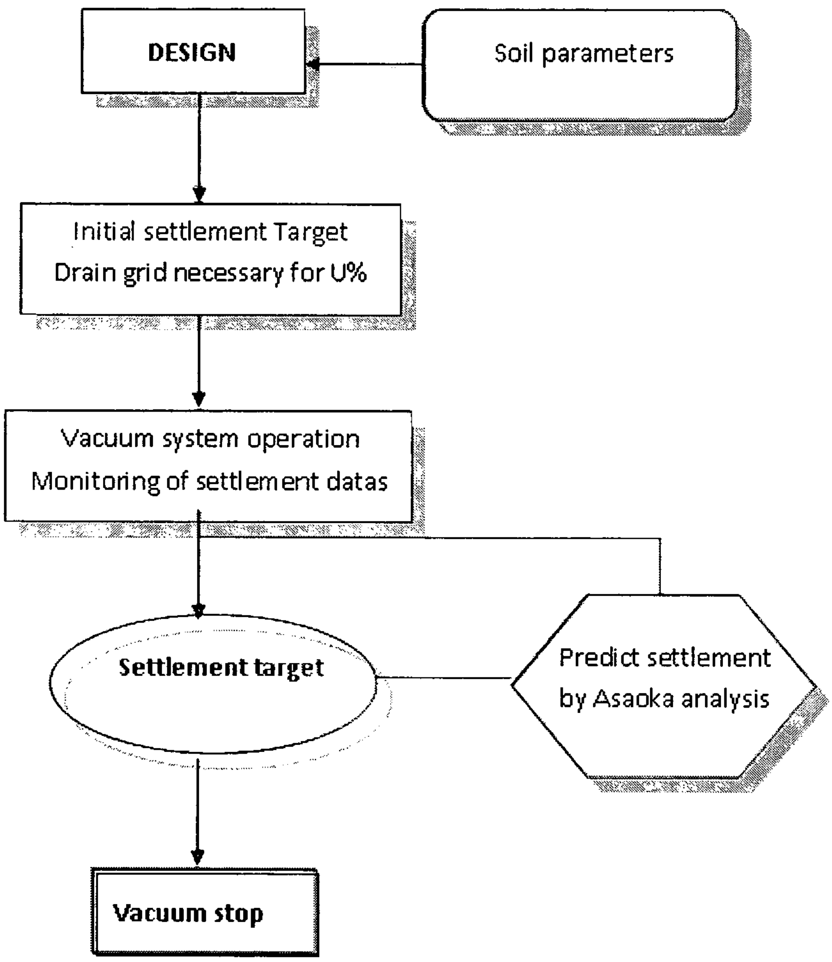


Figure 4.31. The schematic of CVC method

4.5.4 Behavior and Performance of Vacuum Consolidation Method

During construction by CVM the behavior and performance of embankment is conducted by field data before and after vacuum preloading. As analysis the important of the data in the **Chapter 2** to estimate the behavior of soft soil improvement by vacuum preloading method, during treatment the soft soil we have to measure the data versus time by the instruments at the field.

The preloading consists of three steps shown in Figure 4.32. 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, the sand fill were applied with eight layers for 55 days, with the thickness of each layer about 35 cm. 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.

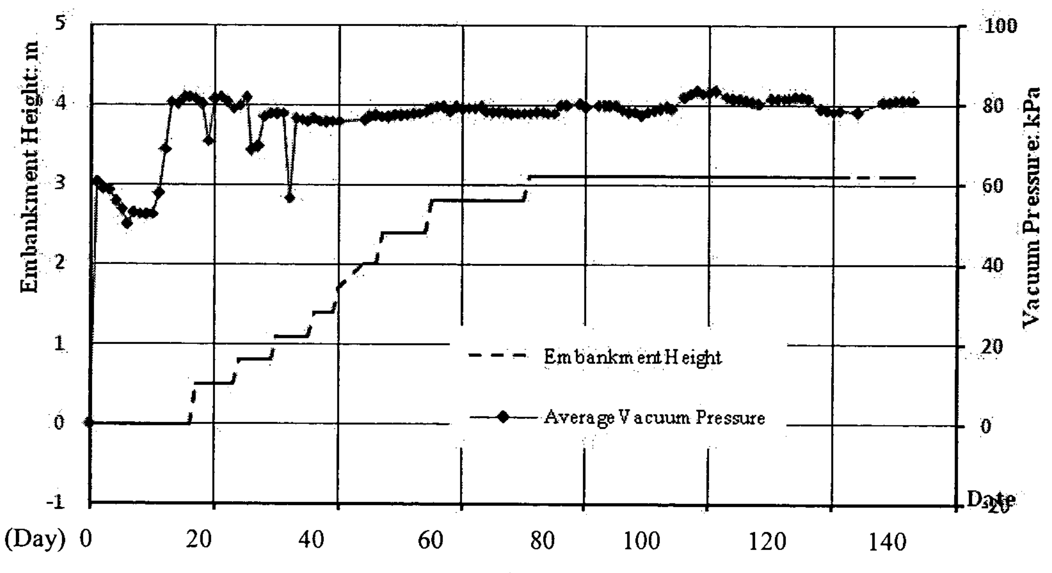


Figure 4.32. The vacuum preloading stages

The construction procedure is completed when the recorded final settlement reached the required value by Asaoka's method (1978) as shown in **Figure 4.33** 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.

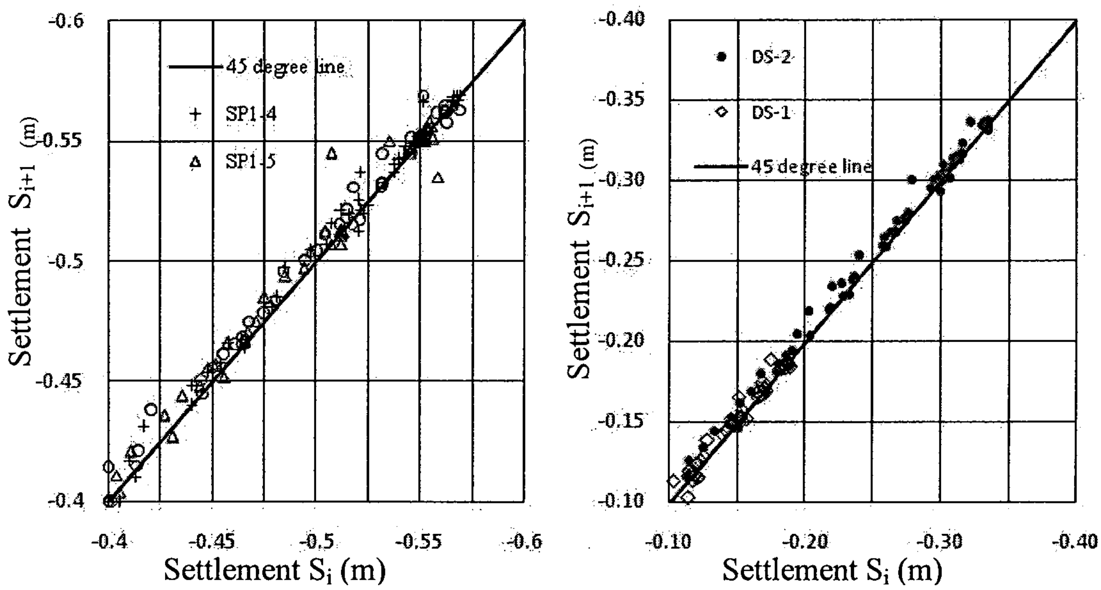


Figure 4.33. The final settlement-Zone GI-1 by Asaoka's method

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 Eq (4.10):

$$S_n = \beta_o + \beta_1 . S_{n-1} \qquad (4.10)$$

Where:

S_1, S_2, ..., S_n are settlements 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 with the expression:

$$S_{lut} = \beta_o/(1-\beta_1) \qquad (4.11)$$

These settlements are presented in **Figure 4.34 and Figure 4.35**, 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.

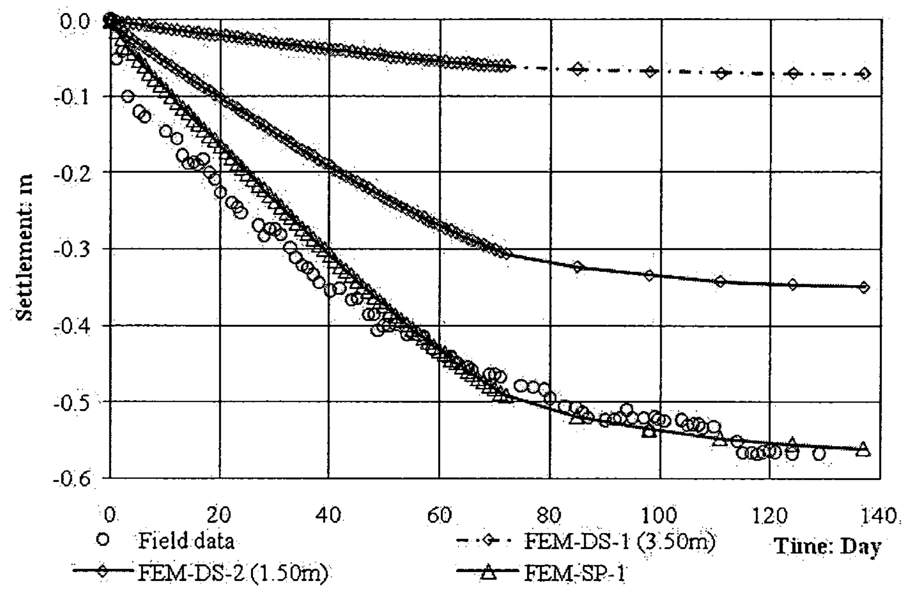


Figure 4.34. The settlement at zone GI-1

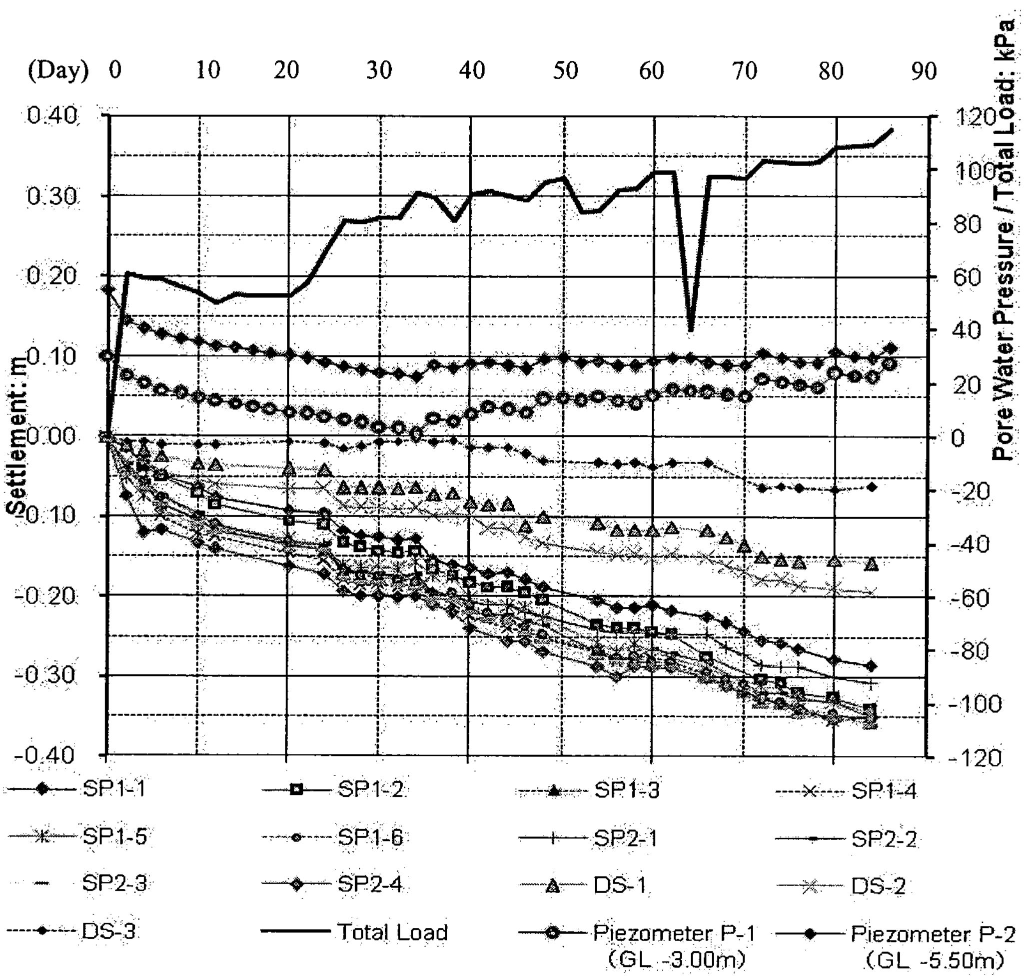


Figure 4.35. Field monitoring Zone 1&2

The excess pore water pressure during construction was shown in the **Figure 4.36 and Figure 4.37**.

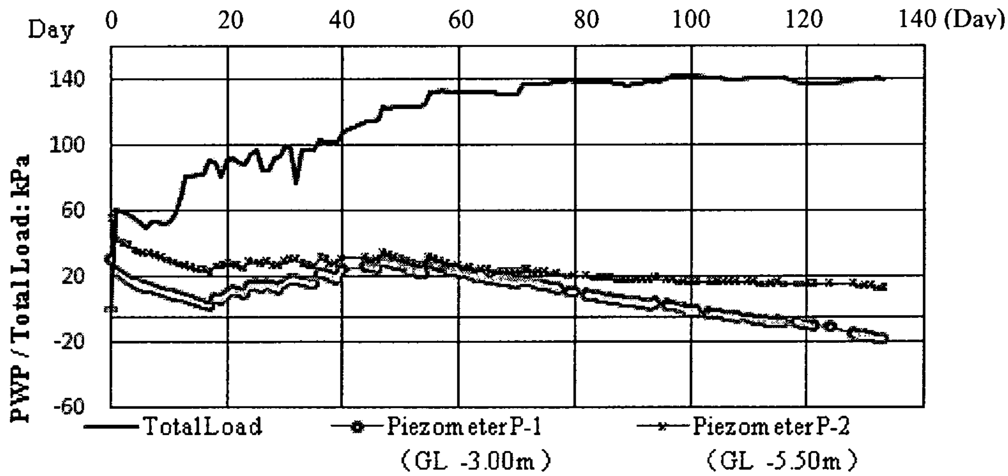


Figure 4.36. Pore water pressure from piezometer data

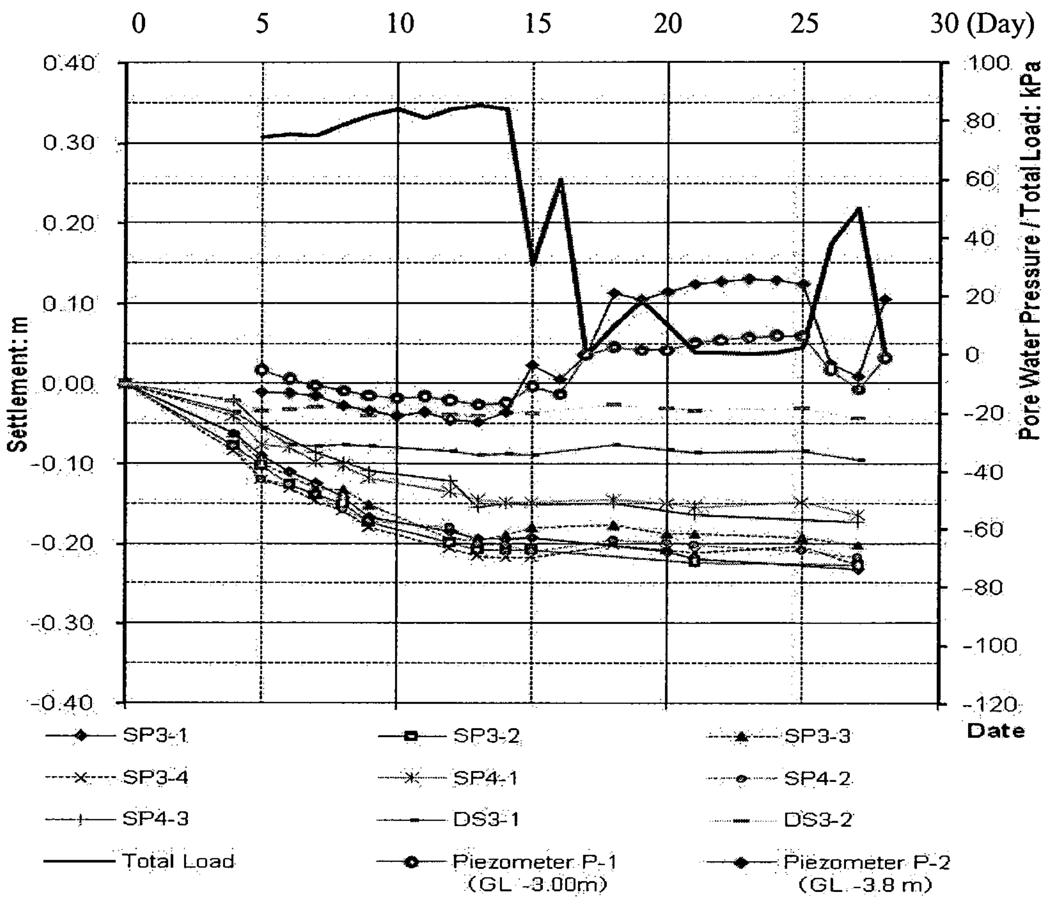


Figure 4.37. Problem during surcharge applying in Zone 3,4

After vacuum pressure apply the excess pore water pressure occurred immediately in negative value gained the vacuum pressure, if the surcharge was fill the excess pore water pressure will be increased during apply surcharge fill stage, then it will be dissipated gradually. However damage of airtight sheet occurred during earthwork construction, the pressure is lose and the pore water pressure raised up, the soft soil is swelled.

The pore water pressure change is shown in the **Figure 4.38**, during surcharge loading the excess pore water pressure increases in first 30 days and 60 days, during the third stage, the pore water pressure reduced and lead to the suction line due to excess pore water dissipated.

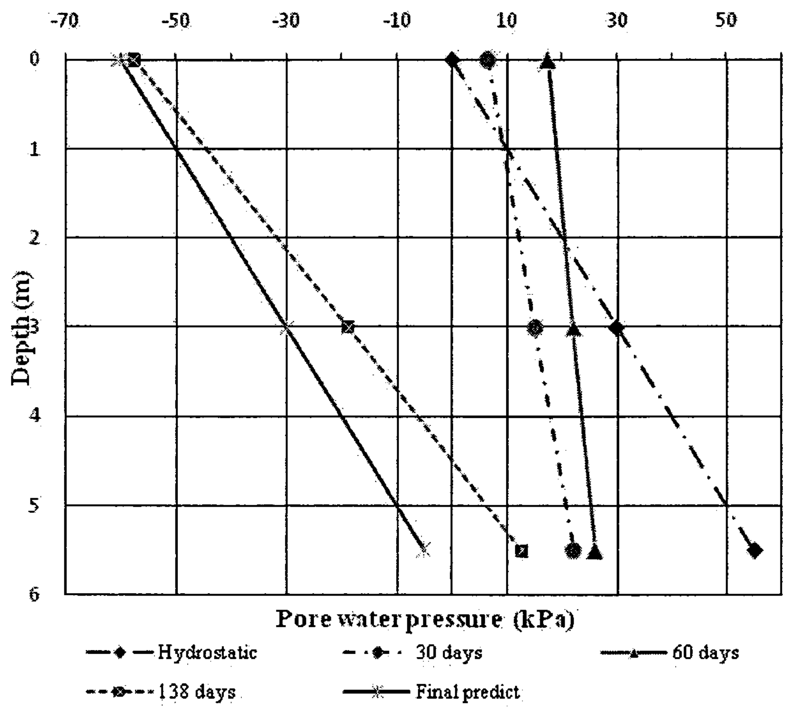


Figure 4.38. Dissipation of excess pore water pressure

The average DOC, U_{avg}, can be calculated as Eq(3):

$$U_{avg} = 1 - \frac{\int [u_t(z) - u_s(z)]dz}{\int [u_0(z) - u_s(z)]dz}; \quad (4.12)$$

where: $$u_s(z) = \gamma_w z - s \text{ (kPa)} \quad (4.13)$$

$u_0(z$, $u_t(z)$, $u_s(z$), γ_w , s- initial pore water pressure at depth z; pore water pressure at depth z at time t; suction line; unit weight of water; and suction applied.

The Degree of Consolidation predicted from distribution of excess pore water pressure indicated that it reduced with depth of improvement by vacuum. In Figure 4.39, the DOC values was more than 95% and 80% at 3m and 5.5m depth, respectively, therefore the result agrees with prediction.

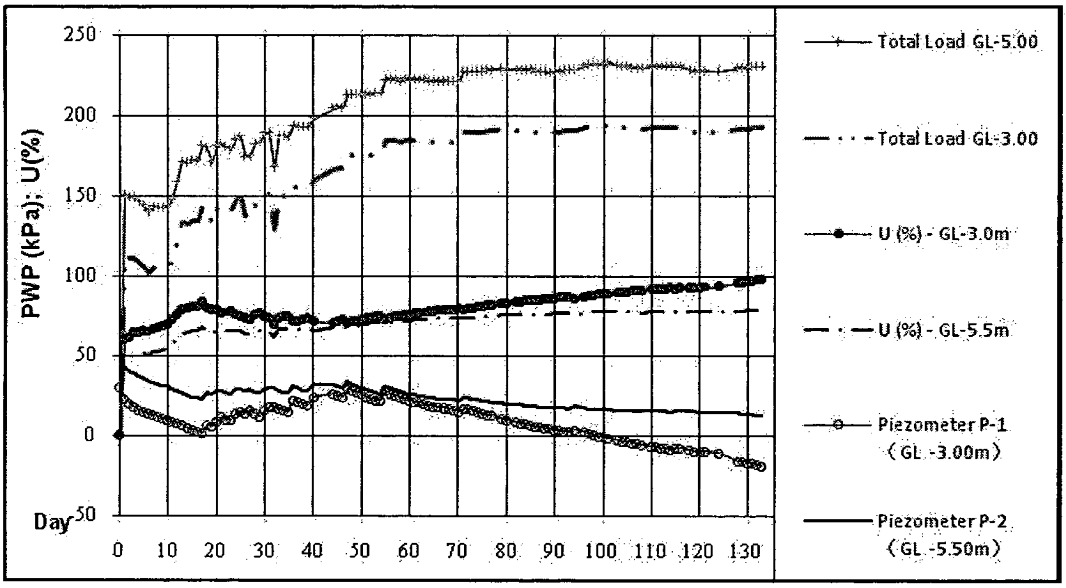


Figure 4.39. Predict DOC by dissipation of the excess pore water pressure (EPWP)

The measurement of water discharge was automatically carried out as shown in Figure 4.40. The drained volume constant is marked to stop the vacuum pressure.

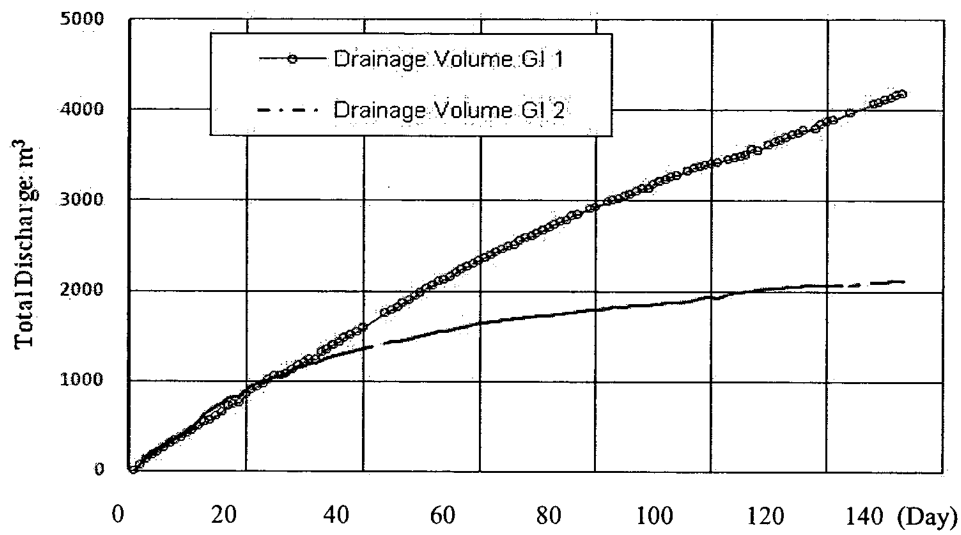


Figure 4.40. The discharge of zone GI-1 & GI-2

The stages of embankment filling at each GI-Zone 1-8 are shown in **Table 4.11.** From the plotted, huge delay on vacuum operation on Zone 3,4,5 and 6 due to the repaired of damage of airtight sheet during earthwork construction and delay of filling works.

Table 4.11. The stage of embankment filling period and vacuum operation

GI-Zone	Date of start of vacuum operation	Date of start filling embankment	Date of finished embankment	Date of stop vacuum operation	Period time (day)	Height of embankment
1	23 Dec 2009	8 Jan 2010	3 May 2010	7 May 2010	135	3.10
2	23 Dec 2009	8 Jan 2010	3 May 2010	7 May 2010	135	3.10
3	9 Jan 2010	20 Jan 2010	4 July 2010	28 July 2010	**200**	2.80
4	9 Jan 2010	20 Jan 2010	4 July 2010	28 July 2010	**200**	2.80
5	30 Dec 2009	17 Mar 2010	29 Jun 2010	28 July 2010	**210**	2.80
6	30 Dec 2009	17 Mar 2010	29 Jun 2010	28 July 2010	**210**	2.80
7	13 Feb 2010	3 Mar 2010	10 Apr 2010	11 July 2010	148	2.40
8	13 Feb 2010	3 Mar 2010	30 Apr 2010	11 July 2010	148	2.10

The summary of settlement, degree of consolidation, rate of settlement and OCR value for GI Zone 1-8 are shown in Table 4.12.

Table 4.12. Degree of consolidation and rate of settlement

GI-Zone	Avg Sett	DOC	Rate of settlement	OCR value	
	(m)	(%)	(mm/day)	Before	After
1	0.55	95.10	0.63	1.1	1.25
2	0.57	95.80	1.00	1.1	1.25
3	0.53	N.A	0.50	1.1	1.20
4	0.53	N.A	0.58	1.1	1.20
5	0.52	N.A	0.63	1.1	1.20
6	0.55	N.A	0.22	1.1	1.20
7	0.58	93.90	1.00	1.1	1.30
8	0.55	91.67	0.90	1.1	1.50

During the earthwork construction some settlement plates and depth settlement gaugers were damaged, the data could not be used as a reference in monitoring records. The average of final settlement was found about 0.55m after 135 days with the degree of consolidation at more than 90% and the rate of settlement almost less than 1mm per day.

After reach the target of degree of consolidation, the vaccum operations finished, applying of vacuum pressure act as pre-loading pressure in sub-soil. The sub-soil was a over consolidated state, the OCR were evaluated as the actual loading (dead load and live load) which could be applied in the future.

The **Figure 4.39** is proposed to illustrate real excess pore water pressure of soft soil under vacuum consolidation preloading method.

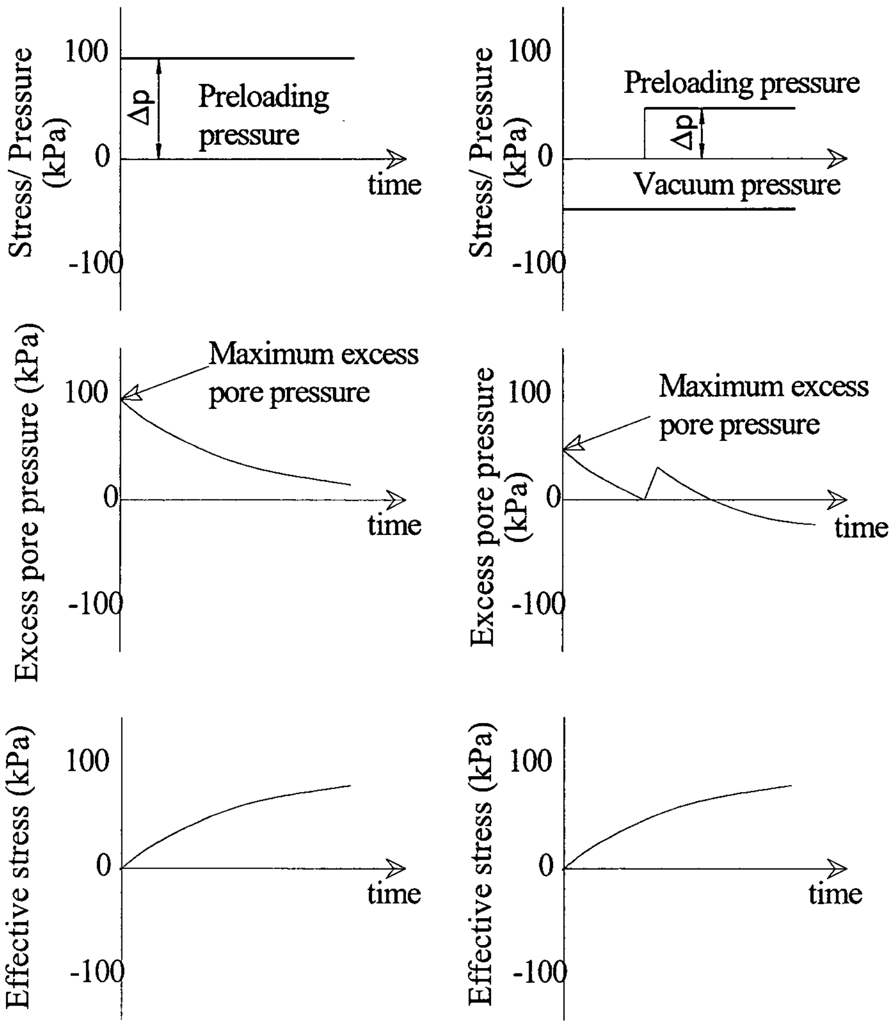


Figure 4.41. The real excess pore water pressure

4.5.5 Summary Conclusions

Based on the field monitoring records and the analysis of soil improvement by vacuum consolidation method for Apron and Taxi way of Nakorn Srithammarat Airport, It can be concluded as follows:

(1) The primary consolidation settlement of soft to very soft clay found in range 0.5-0.65m, with the degree of consolidation more than 90% after only 135days vacuum operation. The period construction accelarated sigfinicantly compare to the conventional method. The 1mm/day rate of settlement was measured at last 12 days before stop vacuum operation.

(2) The degree of consolidation evaluated from dissipation of excess pore water were varied with depth, it was reduced with the depth of soil improvement, due to the distribution of vacuum pressure varies along to the vertical drain as well as the compact of deformation of it during deposit of embankment.

(3) The 70 to 85kPa of vacuum pressure that was generated the overconsolidated state of soil with the minimum OCR values equal to 1.20. The embankment improved by surcharge and vacuum pressure can be compensated the actual loading in the future.