# CHAPTER II

# LITERATURE REVIEW

## 2.1 Soil Improvement

### *2.1.1 Soft Clay*

The design and construction of infrastructure facilities in the soft soil of coastal regions is necessary due to extensive urbanization and industrialization and quite often in many lowland areas.

It is necessary to determine the compressibility, shear strength and permeability characteristics of clays to find practical solutions to the geotechnical problems encountered in soft clay.

Clay is regarded as very soft stage if the unconfined compressive strength ($s_u = q_u/2$) is less than 25kPa and as soft stage with strength is presented in the range of 25 to 50kPa (Terzaghi & Peck 1967). The natural soft clay's permeability is increasingly realized in solving many geotechnical problems.

All soft soil improvement techniques seek to improve the soft soil characteristics that match the desired results of a project, such as an increase in density and shear strength to prevent problems of stability, the reduction on soil compressibility, influencing permeability to reduce and control ground water flow or to increase the rate of consolidation, or to improve soil homogeneity.

It is very important to improve unfavorable ground for the particular requirements that poses a challenge to geotechnical engineers. So many researchers focus on soft soil improvement. At various stages, different methods have been published Kjellman (1952), (Mitchell 1970, 1981, Hansbo (1987), Broms 1979, 1987, Kamon 1991, Kamon & Bergado 1991, Bell 1993, and others). The recent book by Bergado et al. (1996) provides extensive details of the application of many soft soil improvement techniques.

**The various techniques are to:**

- Reduce the settlement of embankment or structure;
- Improve the shear strength of soil and hence increase the bearing capacity of foundations;

- Increase the factor of safety against possible slope failure of embankments and earth dams;
- Reduce the shrinkage and swelling of soil during operation stage.

### *2.1.2 The Basic Compression Theory to Improve Soft Soil*

The stress increase of soil caused by the construction of foundations or other loads compresses the soil layers for improvement. The general shape of the deformation of the specimen against time for a give load increment is shown in **Figure 2.1**.

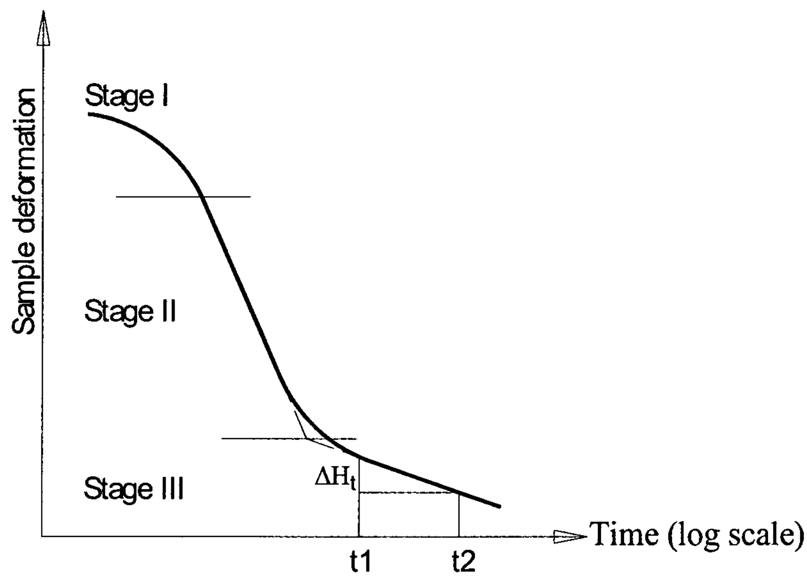



Figure 2.1. The Sharp of settlement of soil clay

From the plot, it can be observed that there are three distinct stages, which may be described as follows:

- **Stage I:** Initial compression ($S_i$), which is mostly caused by preloading, by the elastic deformation of dry soil and of moist and saturated soils, without any change in the moisture content. Initial compression calculations are generally based on equations derived from the elasticity theory.
- **Stage II:** Primary consolidation ($S_c$) occurs during the excess pore water pressure, is gradually transferred into effective stress because of the pore water dissipation.
- **Stage III:** Secondary consolidation ($S_s$) appears after the excess pore water pressure dissipates completely, deformation of the specimen takes

place because of the plastic readjustment of soil fabric. It follows the primary consolidation settlement under a constant effective stress.

The total settlement of ground *S(t)* at time *t* can be represented as follow:

$$S_t = S_i + S_c + S_s \qquad 2.1)$$

### *2.1.3 The Fundamentals of Consolidation*

When a saturated soil layer is subjected to a stress increase (under loading), the pore water pressure (PWP) immediately increases to gain the loading with soil skeleton. In sand that has high permeability; the drainage caused by the increasing in the pore water pressure is completed immediately. Pore water drainage is accompanied by a reduction in the volume of soil mass, resulting in settlement. Because of rapid drainage of the pore water in sand, elastic settlement and consolidation take place simultaneously. For cohesive soil (clay), which has low hydraulic conductivity, the consolidation settlement is time dependent.

Consider a clay layer of thickness $H_t$ located below the groundwater level and between two highly permeable sand layers as shown in **Figure 2.2.**

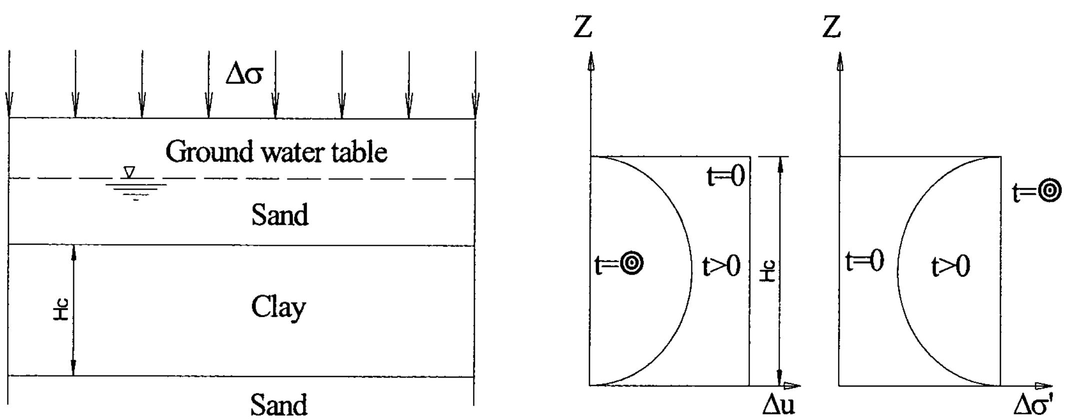



Figure 2.2. Principal of consolidation

If an increasing surcharge of $\Delta\sigma$ is applied at the ground surface over a giant area, the pore water pressure in the clay layer will increase. For a surcharge of *infinite extent*, the immediate increase of the pore water pressure $\Delta u$, *at all depths* of the clay layer will be equal to the increase of the total stress $\Delta\sigma$.

Thus, immediately after the application of the surcharge

$$\Delta u = \Delta\sigma$$

Since the total stress is equal to the sum of the effective stress ($\Delta\sigma'$)and the pore water pressure ($\Delta u$), at all depths of the clay layer the increase of effective stress due to the surcharge (immediately after application) will be equal to zero (i.e., $\Delta\sigma' = 0$, where $\Delta\sigma'$ is the increase of effective stress).

In other words, at time $t = 0$, the entire stress increase at all depths of the clay is taken by the pore water pressure and none by the soil skeleton. It must be pointed out that, for loads applied over a limited area, it may not be true that the increase of the pore water pressure is equal to the increase of vertical stress at any depth at time $t = 0$.

After application of the surcharge (i.e., at time $t > 0$), the water in the void spaces of the clay layer will be squeezed out and will flow toward both the highly permeable sand layers, thereby reducing the excess pore water pressure. This, in turn, will increase the effective stress by an equal amount, since $\Delta\sigma' + \Delta u = \Delta\sigma$.

Thus at time $t > 0$,

$$\Delta\sigma' > 0 \qquad \text{and} \qquad \Delta u < \Delta\sigma$$

Following is a summary of the variation of $\Delta\sigma$, $\Delta u$, and $\Delta\sigma'$at various times. **Figure 2.2** shown the general nature of the distribution of $\Delta u$ and $\Delta\sigma'$ with depth.

This gradual process of increase in effective stress in the clay layer due to the surcharge will result in a settlement that is time-dependent, and is referred to as the process of consolidation.

Table 2.1. Mechanism of consolidation of soil

| **Time, t** | **Total stress increase, $\Delta\sigma$** | **Excess pore water pressure, $\Delta u$** | **Effective stress increase, $\Delta\sigma'$** |
|---|---|---|---|
| 0 | $\Delta\sigma$ | $\Delta\sigma$ | 0 |
| >0 | $\Delta\sigma$ | $<\Delta\sigma$ | >0 |
| $\infty$ | $\Delta\sigma$ | 0 | $\Delta\sigma$ |

### *2.1.4 The Primary Consolidation (Terzaghi theory)*

The one-dimensional primary consolidation settlement (caused by an additional load) of the clay layer, having a thickness $H_c$ calculates as:

$$S_c = \frac{\Delta e}{1+e_o} H_c; \qquad (2.2)$$

$$\frac{\Delta e}{1+e_o} = \varepsilon_v \qquad (2.3)$$

Where:

$S_c$: primary consolidation settlement

$\Delta_e$: total change of void ratio caused by additional load applied

$e_0$: initial void ratio of the clay layer before application of the load

$\varepsilon_v$: vertical strain.

For normally consolidated clay, the field e-logσ' curve is be as shown in Figure 2.3a (Das, 2004). If $\sigma'_o$ is the initial average effective overburden stress on the clay layer and $\Delta\sigma'$ is the average effective increase in pressure on the clay layer caused by the added load, the change in void ratio caused by the load is

$$\Delta e = C_c \log \frac{\sigma'_o + \Delta\sigma'}{\sigma'_o} \qquad (2.4)$$

Where:

$C_c$: the compression index, or can be defined as the slope of the e-log σ' graph as shown in **Figure 2.3a**, combining Eq. (2.2) and Eq. (2.4) yields.

$$S_c = \frac{C_c H_c}{1+e_0} \log \frac{\sigma'_o + \Delta\sigma'}{\sigma'_o} \qquad (2.5)$$

For over-consolidated clay, the field $e$-log $\sigma'$ curve is as shown in **Figure 2.3b**. In this case, depending on the value of $\Delta\sigma'$, two condition may arise.

First, if $\sigma'0 + \Delta\sigma' < \sigma'c$ then

$$\Delta e = C_s \log \frac{\sigma'_o + \Delta\sigma'}{\sigma'_o} \qquad (2.6)$$

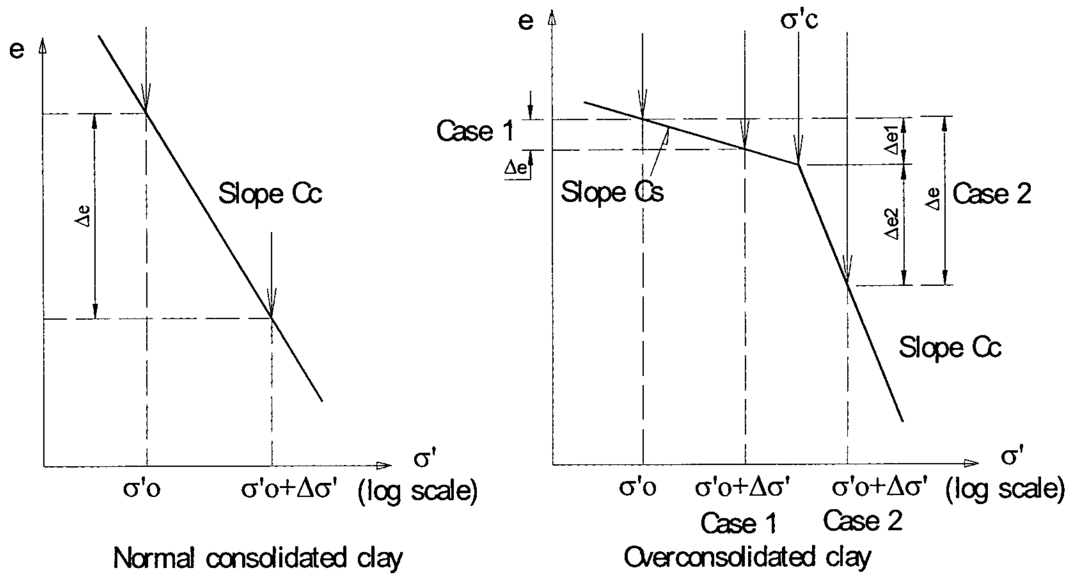



Figure 2.3. One-dimensional settlement calculation (a) is for Eq. 2.5); (b) is for Eq. (2.7) and (2.9) (after Das, 2004)

Where:

$C_s$ is the swelling index. In most case, the value of the swelling index is 0.20 to 0.25 of the compression index (Das, 2004). Combining Eq. (2.2) and Eq.(2.6) gives

$$S_c = \frac{C_s H_c}{1+e_0} \log \frac{\sigma'_o + \Delta\sigma'}{\sigma'_o} \tag{2.7}$$

Second, if $\sigma'_c < \sigma'_0 + \Delta\sigma'$ then

$$\Delta e = \Delta e_1 + \Delta e_2 = C_s \log \frac{\sigma'_c}{\sigma'_o} + C_c \log \frac{\sigma'_o + \Delta\sigma'}{\sigma'_c} \tag{2.8}$$

Now combine Eq.2.2 and Eq. 2.8 gives

$$S_c = \frac{C_s H_c}{1+e_0} \log \frac{\sigma'_c}{\sigma'_o} + \frac{C_c H_c}{1+e_0} \log \frac{\sigma'_o + \Delta\sigma'}{\sigma'_c} \tag{2.9}$$

## 2.2 Compact Vacuum Consolidation Method

### *2.2.1 Introduction on Vacuum Method*

There are so many methods to treat soft soil; they can be classified in several ways such as mechanical stabilization, hydraulic modification, chemical stabilization and inclusion of confinement materials such as geosynthetics into the soil. Nowadays the vacuum preloading method becomes the most popular method to improve soft soil. A technique using atmospheric pressure as a temporary loading is principally the compact vacuum consolidation method. It is concerned as an effective method of improving soft soil conditions, which was introduced by Kjellman (1952); this method has been successfully used for soil improvement in a number of countries (Holtz (1975); Chen and Bao (1983); Bergado et al. (1998); Chu et al. (2000); Indraratna et al. (2005)). With the merging of new material and new technologies, this method has been further improved in recent years and was applied several projects in Asian countries, which is shown in the *Table 2.2*

Table 2.2: The effective of vacuum-preloading method

| Project | Tanjin New Harbour, China | Northeast New Railway, Japan | Factory in Lianyungang City, China |
|---|---|---|---|
| Soil type | Silty clay | Peat and silt | Marine clay |
| Water content (%) | 55 | 580-860 | 69-85 |
| Void ratio | 1.4 | - | 1.62 – 2.36 |
| Natural bulk density (g/cc) | 1.73 | 1.05 | 1.54-1.61 |
| Initial shear strength (kPa) | 16.7-20.6 | 3.4-5.9 | 5.7-19.6 |
| Area of involvement ($m^2$) | 1250 | 1950 | 4000 |
| Thickness (m) | 16 | 13 | 10 |
| Degree of vacuum (mm Hg) | 600 | 700 | 650 |
| Increase in shear strength (kPa) | 131-190 | 186-190 | 170-440 |
| Increase in bearing capacity (%) | 300 | 200-300 | 250 |
| Estimated settlement (mm) | 811 | 2040 | 1000 |
| Measure settlement (mm) | 565 | 1490 | 700 |
| Reduction in settlement (%) | 69.5 | 73 | 70 |

To achieve best results, this method should combine with prefabricated vertical drains (PVDs), which are installed before vacuum pressure applying. The airtight plastic sheet is buried in the surrounding separation walls. Water and air in soft soil can be drawn out from the vertical drains through the system of perforated pipes by a pump (see **Figure 2.4**). With new technique from Maruyama Ltd,Co, the water and air are pumped out by two separately trends, hence the vacuum pressure can be maintained during construction procedure at high pressure and the surcharge can be applied above the airtight sheet immediately. When pore water and air are pumped out of the ground, a water and air are pumped out of the ground, a difference in pressure is formed at the separation surface, which induces compression of the clay.

A common vacuum pressure from 60 to 80 kPa is usually used in design step; however, a higher vacuum pressure of up to 90 kPa may be achieved sometimes at the field. When the surcharge loading more than 80 kPa is required, a combined vacuum and fill surcharge should be applied for a good result.

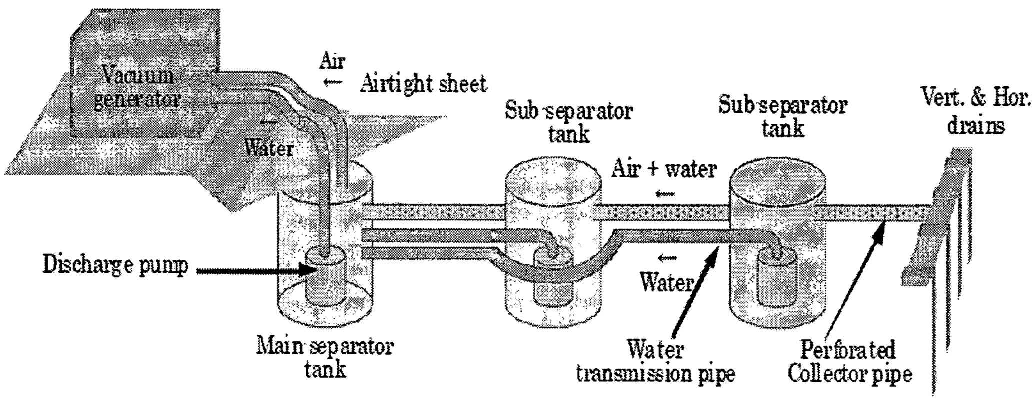



Figure 2.4. Air water separation vacuum pump system (et al. 2004)

For treatment of very soft ground, the time of vacuum preloading method is faster than that of surcharge preloading method, because the 80 kPa vacuum pressure can be applied instantly, without causing instability problem. The vacuum preloading method is also cheaper than the surcharge preloading method for an equivalent load (Chu et al. 2000).

Yan and Chu (2003) showed that the cost of soil improvement by vacuum preloading reduces over one-third that by conventional surcharge alone. The effectiveness of this system relies on: (a) reliability (airtight) of membrane, (b) effectiveness of the seal between the ground surface and the membrane edges, and (c) soil conditions and the location of ground water level (Cognon et al., 1994).

The mechanisms of vacuum preloading and conventional and innovative techniques relate to equipment, materials, monitoring, analysis and numerical simulation that are discussed in the followings.

The increase the effective stress tends towards isotropic with the compressive lateral deformations (**Figure 2.5**). Since there no shear failure, preloading can be applied at rapid rate. The method is based on the idea of applying vacuum the way of reducing the pore water pressure in the soil the effective stress is increased without changing the total stress.

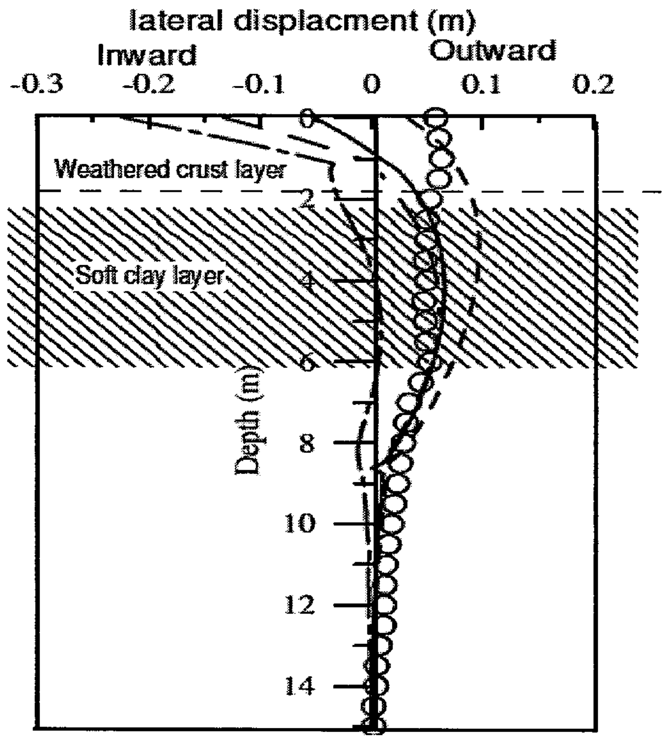



Figure 2.5. The lateral movement of soil due to by vacuum preloading

### *2.2.2 Mechanisms of Vacuum Preloading Method*

The mechanisms vacuum preloading method is shown as the **Figure 2.6** and **Figure 2.7**. In saturated soils, the total stress (σ) at any point within the soil mass is the combination of the effective stress (σ') and the pore water pressure (u) (Terzaghi, 1943).

Thus, the total stress at any point within the soil mass can be written as Eq(2.10):

$$\sigma' = \sigma - (+u_{\Delta p}) \tag{2.10}$$

Under the surcharge loading only, the effective stress is gained by the dissipation of positive excess pore water pressure after the load application. In contrast, the effective stress is increased by the applied negative pore pressure ($-u_{vac}$) under the vacuum condition. Equation (1) can be rewritten based on the vacuum and fill preloading as Eq.(2.11):

$$\sigma' = \sigma - (+u_{\Delta p}) - (-u_{vac}) \tag{2.11}$$

It can be seen that the effective stress increases by negative suction, thereby, reducing the risk of shear failure. The performance of this system depends on the vacuum condition under the airtight membrane (Indraratna et al. 2004). The intensive pore pressure measurement under membrane and inside the PVD with depth should be performed to verify the reliability of the vacuum system.

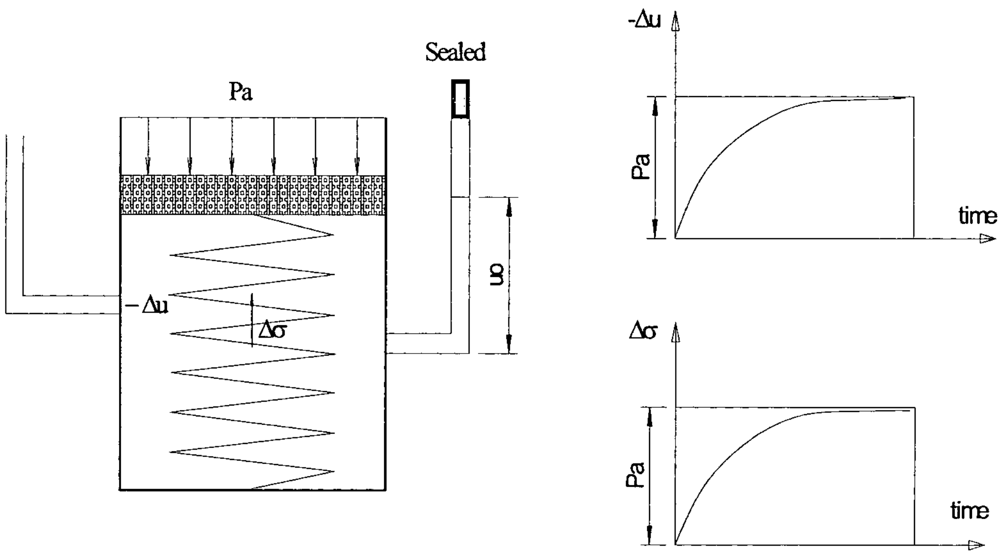



Figure 2.6. The mechanisms vacuum preloading method

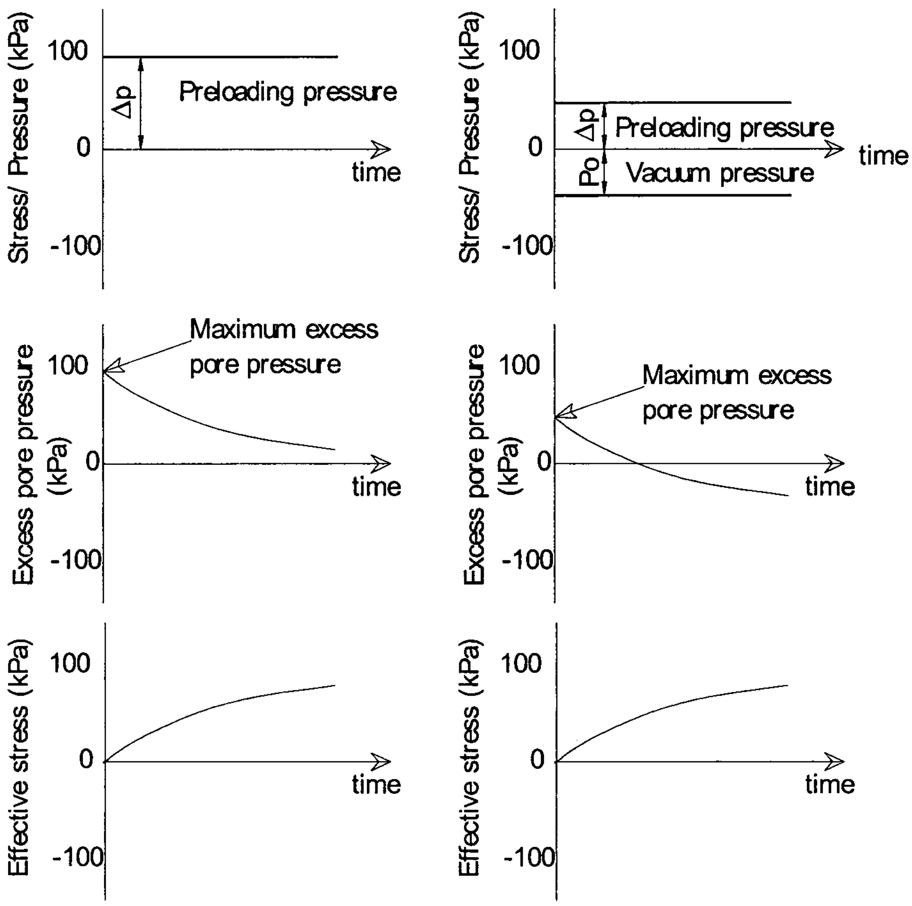
In terms of stress path distributions, the stress state can be described in tri-axial space with mean effective stress p′ and the deviator stress q, defined as Eq.(2.12) and Eq.(2.13) respectively:

$$q = \frac{(\sigma_1' - \sigma_3')}{2}; \tag{2.12}$$

$$p' = \frac{(\sigma_1' + \sigma_3')}{2} \tag{2.13}$$

Where:

$\sigma'_1$, $\sigma'_3$: are the principal normal stresses.




(a) Preloading; (b) Combine preloading and vacuum (Indraratna et al. 2004)

Figure 2.7. The consolidation process

In **Figure 2.8** different stress paths are described. Starting from an in situ stress state at point A, the curve ABC describes the case of conventional preloading.

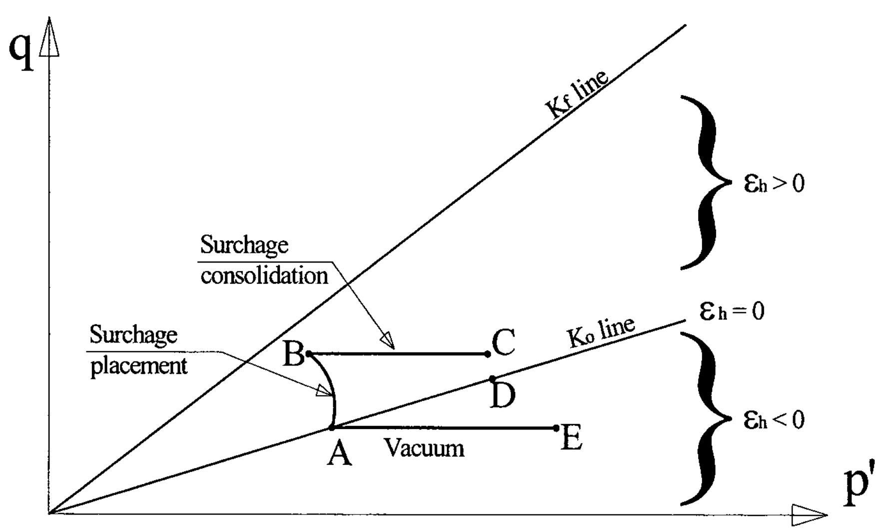



Figure 2.8.The stress path (p'~q)

When the surcharge is applied, it follows curve AB with a possible failure if point B would cross the failure $K_f$ line. Consolidation will take place from B to C in the area of $\varepsilon_h > 0$ above the $K_o$-line and hence, outward lateral deformation will occur.

Line AD corresponds to oedometric consolidation. As for vacuum induced preloading, the stress path follows the line AE. This is because during vacuum consolidation, the soil is under isotropic conditions and thus the principal normal stresses are equal. It can be seen that the entire stress path is under the $K_o$-line with the field of $\varepsilon_h < 0$, and hence, under horizontal compression or inward lateral displacement respectively.

## 2.3 Recent Studies on Vacuum Consolidation Method (CVM)

Wangthong (2008) studied the ground improvement of soft clay using PVDs with and without vacuum in field and in the laboratory by large scale consolidometer. The specimens were reconstituted with the surcharge of 50kPa until 90% degree of consolidation as predicted by Asaoka (1978) method. The rate of settlement of soil improvement with vacuum pressure is higher than PVD only due to the higher flow rate of water to the PVD channel with vacuum pressure. However, the final settlement of both two methods is the same value. After consolidation, the variation of shear strength in the consolidometer is higher than the shear strength for PVD with vacuum. Water content was measured shown higher percent decrease of water content for PVD with vacuum. However, the settlement rate increased with corresponding increased in the coefficient of horizontal consolidation $C_h$.

Indraratna (2008) carried out a system of PVDs with surcharge load to accelerate consolidation by shortening the drainage path. The study is proposed based on radial soil permeability and changing of vacuum pressure. The prediction of smear zone and effects of drain unsaturated compared with laboratory data from large-scale radial consolidation tests. In this method, the vacuum creates a suction head that increases the effective stress. The effectiveness of vertical drains on cyclic loading was also performed in laboratory. The research show that vertical drains can dissipate the built up excess pore pressure under repeated loading, and that short drains can be sufficient in certain cases better than driving the drains to install the entire depth of soft clay deposits. The effects of soil disturbance and vacuum pressure can affect soil consolidation considerably, which means that these aspects need to be modeled correctly in any numerical approaches.

For estimating of the behavior of soil clay improved by vacuum preloading the laboratory test on the large – scale radial drainage consolidometer has been carried out by Indraratna (2008). However, the specimen with the size 45 cm in diameter and 90 cm high could not be feasible to obtain undisturbed sample of this size, and the clay mixed with water for several days to be ensure saturation before making the samples.

## 2.4 The Techniques of Vacuum Preloading Method

The common vacuum system is shown as **Figure 2.9**. For the vacuum preloading technique, the PVDs and the horizontal pipes are used for the distribution of vacuum pressure and dissipation of pore water.

The horizontal pipes and the top ends of PVDs are buried in a sand blanket made of course sand, which transmits the vacuum to PVDs. Corrugated flexible pipes (50 to 100 mm in diameter) are normally used as horizontal pipes. These pipes are perforated and wrapped with a permeable fabric textile to act as a filter to prevent sand get in to the pipes. The horizontal pipes are connected to the main vacuum distribution pipes.

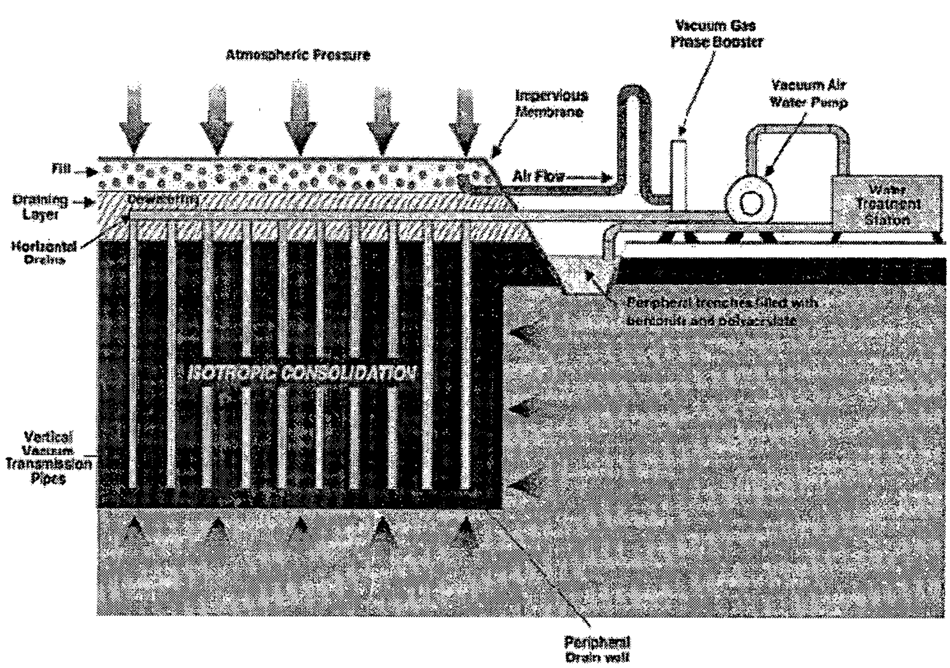



Figure 2.9. Vacuum system (after Masse et al., 2001)

Three layers of thin membranes are often required to seal the area which improved by vacuum. The membranes are buried into a trench at the four boundaries of the area. For this reason the entire soil improvement area often, need to be subdivided into small areas to facilitate the installation of the membranes and capacity of vacuum unit. Then Vacuum pressure is applied continuously for during of preloading stage.

## 2.5 Degree of Consolidation (DOC)

The degree of consolidation (DOC) is usually used as one of the criteria for assessing the effectiveness of soil improvement. It is also used as a design specification in a soil improvement contract. DOC is normally calculated using settlement data, as the ratio of current settlement to ultimate settlement. However, for a soil improvement project, the ultimate settlement (final settlement) is unknown and has to be predicted and several methods are available for estimating the ultimate settlement. On the other hand, the pore water pressure monitored during construction stage is also used to assess the degree of consolidation.

### *2.5.1 Calculate Degree of Consolidation by The Settlement (Terzaghi).*

The rate of consolidation is defined as the rate of dissipation of the excess pore water pressure in the soil after loading applying. It is important in order to estimate the degree of consolidation, U. The consolidation settlement at an arbitrary time, $S_{ult}$, is given as

$$S_{ult} = U . S_t \tag{2.14}$$

Where

$S_{ult}$: the final settlement at end of consolidation (ultimate settlement)

$S_c$ : the settlement at time (t)

Follow Terzaghi, If the applied load is constant, the percentage of primary consolidation U is related to a dimensionless time factor $T_v$ as follows Eq.(2.15):

$$U = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M^2} \exp(-M^2 T_v); \qquad Tv = \frac{C_v t}{H_d^2} \tag{2.15}$$

Where $M = \frac{\pi}{2}(2m+1)$

Therefore $U = 2\sqrt{\frac{Tv}{\pi}}$ when U < 0.52 (52%, $T_v$ = 0.213)

And $U = 1 - \frac{8}{\pi^2}\exp(-\frac{\pi^2}{4}T_v)$ when U > 0.52

where:

$C_v$: coefficient of consolidation for vertical drainage,

$$C_v = \frac{k_v}{m_v \gamma_w} \qquad (m^2/sec)$$

$k_v$: coefficient of vertical permeability, (m/s)

$m_v$: coefficient of volume change = $\Delta e/\Delta\sigma_v$

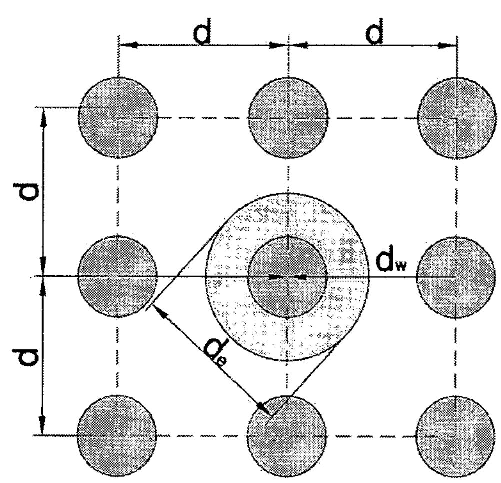
$\gamma_w$: unit weight of water ($kN/m^3$)

*t: time (year)*

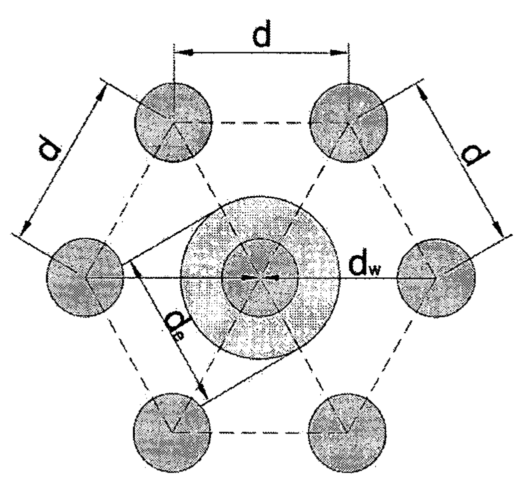
$H_d$: height of drainage path, where $H_d$ = H/2 for double drainage because pore water pressure can move downward and upward to escape. $H_d$ = H for single drainage case. (m)

### *2.5.1.1 Effective diameter of drain (de)*

When installing drain board, triangle arrangement (Barron, 1948) which is highly economic and square arrangement (Kjellman, 1948) which is easy to apply in practice are used generally. Arrangement method of drain board is shown in **Figure 2.10**




(a) Regular square pattern
($d_e = 1.128 \times d$)




(b) Regular triangle pattern
($d_e = 1.050 \times d$)

Figure 2.10. Arrangement method of drain board

In analysis regarding the ground where drain board installed, it is calculated after conversion to effective diameter of drain ($d_e$) due to the complexity in calculation

*2.5.1.2 Equivalent diameter of PVD*

Since the cross-section of PVD has a shape of plate, it is hard to apply to drainage theory. Therefore, based on equal surface concept, equivalent diameter is calculated, which shown in *Table 2.3*

Table 2.3. Equivalent diameter ($d_W$) of PVD

| Researchers | Proposed $d_W$ | Application |
|---|---|---|
| Hansbo(1979) | $d_w = \frac{2(a+b)}{\pi}$ | In general, the equivalent diameter is calculated to 5.9~6.6cm, considering safety 5cm is applied |
| Jansen and den Hoedt(1983) | $d_w = \frac{(a+b)}{2}$ | |
| Fellenius and Castonguay(1985) | $d_w = (1.5\sim3.0)x\frac{2(a+b)}{\pi}$ | |
| Suits et al.(1986) | 38 ~ 64 mm | |
| Rixner et al.(1986), Hansbo(1983) | $d_w = \frac{(a+b)}{\pi}$ | |

*2.5.1.3 Concept for application of $C_h$*

**a) Smear effects**

• The sub-soils are disturbed during installation of vertical drain depending on the soil's sensitivity and macro fabric. This is so-called "Smear effects"

• The effects of soil disturbance (smear effects) have to be applied in the analysis by assuming an annulus of smear zone of clay layer around the drain.

• Within this annulus of diameter (smear zone), the remolded and disturbed soils has a coefficient of permeability which is lower than the coefficient of permeability for horizontal direction of the undisturbed clay

**b) Diameter of smear zone**

$$d_s \cong 2.0 d_m (d_s / d_m \cong 2.0)$$ Bergado et.al (1991)

Where:

$d_s$: Diameter of smear zone

$d_m$: Conversion diameter of mandrel

**c) Permeability of smear zone**

$$k_s \cong 1.0k_v (k_s / k_v \cong 1.0)$$ Hansbo(1987), Bergado et.al (1991)

Where:

$k_s$: Permeability of smear zone

$k_v$: Permeability of vertical direction for clay

**d) Coefficient of consolidation of horizontal direction**

$$c_h \cong 1.5c_v (c_h / c_v \cong 1.5)$$

$$c_h / c_v \cong k_h / k_v$$ Bergado et.al (1991)

$$k_h / k_v = k_h / k_s = 1.5$$

Where:

$k_h$: Permeability of horizontal direction for clay

$k_s$: Permeability of smear zone

$k_v$: Permeability of vertical direction for clay

$C_h$: Coefficient of consolidation of horizontal direction

$C_v$: Coefficient of consolidation of vertical direction

### *2.5.2 Yoshikuni 's Method (1979)*

Since Yoshikuni's equation is derived from without smear effects, the analysis of degree of consolidation in case of considering the smear effects has to be carried out based on that the coefficient of consolidation of horizontal direction ($C_h$) is equal to that of vertical direction ($C_v$). Because the permeability of smear zone ($k_s$) is equal to that of vertical direction ($k_v$) for undisturbed clay layer, the degree of consolidation is defined as the Eq. (2.16).

$$U_h = 1 - \exp\left(\frac{-8T_h}{F(n) + 0.8L}\right) \tag{2.16}$$

Where: $F(n) = \frac{n^2}{n^2 - 1}\log n - \frac{3n^2 - 1}{4n^2}$; $L = \frac{32}{\pi^2} \times \frac{K_c}{K_w} \times \left(\frac{H}{d_w}\right)^2$

$T_h$: Time factor

n: Ratio of spacing ; $n = d_e/d_w$

$d_e$: Effective diameter of drain *(m)*

$d_w$: Conversion diameter of drain material *(m)*

$k_W$: Permeability of vertical drain *(cm/sec)*

$k_c$: Permeability of clay *(cm/sec)*

$H_d$: height of drainage path, where $H_d$ = H/2 for double drainage because pore water pressure can move downward and upward to escape. $H_d$ = H for single drainage case. *(m)*

### *2.5.3 Hansbo's Method (1981)*

Hansbo's equation is derived from with smear effects, following conditions of parameters has to be applied.

• Coefficient of consolidation of horizontal direction is 1.5 times larger than that of vertical direction ($C_h/C_v$= 1.5)

• Therefore, permeability of horizontal direction is 1.5 times larger than that of smear zone ($k_h/k_v$= 1.5)

• Diameter of smear zone is 2.0 times larger than that of mandrel ($d_s/d_m$= 2.0). DOC defined as the Eq.(2.17).

$$U_{hz} = 1 - \exp[-\frac{8T_h}{F}] \tag{2.17}$$

Where

$$F = F(n) + F(s) + F(w)$$
$$= [\ln(\frac{d_e}{d_w}) - 0.75] + [(\frac{k_h}{k_s} - 1)\ln(\frac{d_s}{d_w})] + [\pi z(2H_d - z)\frac{k_h}{q_w}]$$

$T_h$: Time factor

$d_e$: Effective diameter of drain *(m)*

$d_w$: Conversion diameter of drain material*(m)*

$k_h$: permeability of horizontal direction for clay (cm/sec)

$k_s$: Permeability of smear zone (cm/sec)

z: depth of soft clay *(m)*

$q_w$ : Discharge Capacity of drain ($cm^3$/sec)

### *2.5.4 Car-rillo Theory to Calculate DOC*

Car-rillo's theoretical solution (1942) is used to combine the vertical and radial drainage effects to estimate the degree of consolidation as the Eq.(2.18);

$$U_{vr} = 1 - (1-U_v)(1-U_r) \quad (2.18)$$

where:

$U_v$,$U_r$: the average degree of consolidation due to vertical and radial drainage, respectively.

This equation used to apply in vacuum preloading method which presents three- dimensional consolidation solutions and based upon the assumption that the total stress remain constant, so that the rate of change of excess pore pressure is equal to the rate of change of volume at all points in the soil. The formulas for degree of consolidation are shown in the *Table 2.4.*

Table 2.4. The period method to define the DOC

| Proponent | Formula | Apply |
|---|---|---|
| Terzaghi | $U = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M^2} \exp(-M^2 T_v)$ | - Surcharge only |
| Yoshikuni (1979) | $U_h = 1 - \exp(\frac{-8T_h}{F(n) + 0.8L})$ | - Without smear zone<br>- Drain |
| Hansbo (1981) | $U_h = 1 - \exp(\frac{-8T_h}{F})$ | - Concern Smear zone<br>- Drain |
| Car-rillo (1942) | $U_{vr} = 1 - (1-U_v)(1-U_r)$ | -Horizontal consolidation<br>-Vertical consolidation |

### *2.5.5 Asaoka (Empirical Method)*

The method of Asaoka (1978) for prediction of settlement magnitude and Hansbo (1979) for prediction of settlement rate will be combined together to analyze the field observation data of CVC. The measured settlements will be then compared with the predictions. The comparison of settlement behavior CVC is plotted with time.

It is also often used as a design specification (Chu and Yan, 2005). The degree of consolidation is normally calculated as the ratio of the current settlement to the ultimate settlement. However, for a soil improvement project, the ultimate settlement is unknown and has to be predicted.

### *2.5.6 Degree of Consolidation Based on Pore Water Pressure*

Once the pore water pressures at different depths are measured during preloading, the initial and final pore water pressure distributions with depth can be plotted (Chu et al. 2000). For generality, a combined fill surcharge and vacuum load case is considered. The typical pore water pressure distribution profiles for a combined loading case are shown schematically in Figure 2.10. Using this profile, the average DOC, $U_{avg}$, can be calculated as Eq.(2.21)

$$U_{avg} = 1 - \frac{\int [u_t(z) - u_s(z)]dz}{\int [u_0(z) - u_s(z)]dz} \tag{2.21}$$

Where

$$u_s(z) = \gamma_w z - s \text{ (kPa)}$$

$u_0(z)$: initial pore water pressure at depth $z$;

$u_t(z)$: pore water pressure at depth $z$ at time $t$;

$u_s(z)$: suction line, $z$ =depth;

$\gamma_w$: unit weight of water;

s: suction applied,

The integral in the numerator in Eq. (2.211) is the area between the curve $u_t(z)$ and the suction line $u_s(z)$, and the integral in the denominator the area between the curve $u_0$(z) and the suction line $u_s$(z).

The method shown in Eq. (2.21) has the following advantages over the method using settlement data:

- The DOC calculated using Eq.(2.21) relies only on field pore water pressure data, whereas when calculating the DOC using settlement data, the ultimate settlement has to be predicted;
- Not only the final DOC, but also the DOC at any time can be calculated using Eq.(1), as $u_t(z)$ represents the pore water pressure at any time, $t$;
- And for consolidation involving multiple layers, Eq. (2.21) can be applied to any single layer to calculate the DOC achieved in a particular layer. In this case, the upper and lower limits of the integrals in Eq. (2.21) are set to be the top and bottom of that soil layer. However, it is

not easy to calculate the DOC for each layer for multilayer soils using settlement, as the settlement of each layer may not be monitored directly and the ultimate settlement of each layer has to be predicted, too.

The schematic illustration of pore water pressure distributions versus depth under vacuum preloading is shown as in **Figure 2.11**.

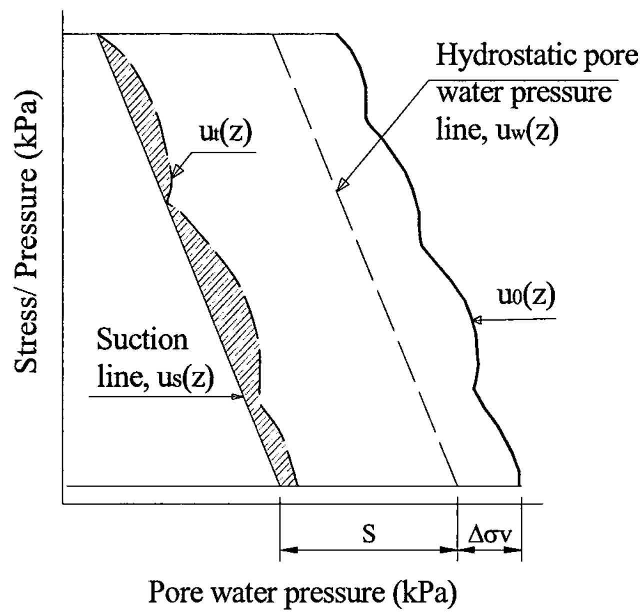



Figure 2.11. Pore water pressure distributions under vacuum preloading

## 2.6 Laboratory Test

The Tri-axial Shear Test is one of the most reliable methods available for determining the shear strength parameters. It is widely used for both research and conventional testing. The test is considered for the following reason:

- It provides information on the stress-strain behavior of the soil that the direct shear test does not.
- It provides more uniform stress conditions than the direct shear test does with its stress concentration along the failure plane.
- It provides more flexibility in term of loading path.

Three standard types of tri-axial test are generally conducted:

- Consolidation – Drained test or drained test (CD test)
- Consolidation – Undrained ed test (CU test)

- Unconsolidation – Undrained ed test (UU test)

In this researching the CU test will be used to estimate the behavior of soft soil before improvement as well after improvement. To model the soil improvement in the lab, the proceeding of vacuum improvement by tri-axial apparatus simulated by decrease the backpressure after saturating the specimen. However the PVD cannot install inside the specimen therefore this simulation will be conducted by another manner, which discusses in next chapter.

## 2.7 Field Test Results

To estimate the behavior of soil improvement, beside some test in the laboratory during improvement we need observe the effective of the method by the instruments. Normally the information need to measure such as:

- The bearing capacity of the ground before and after improvement. These parameters we can measure by CPT and SPT test at the field.
- The deformation of the embankment includes the vertical settlement and lateral deformation.
- The stability of the embankment during improvement. This task can control by inclinometers and observe the deformation of the ground in time.
- Pore water pressure in the soil
- The discharge of water squeezes out the soil ground.

## 2.8 Numerical Method

For estimation the behavior of Vacuum Consolidation method by FEM, so many research success in this field. Such as: The three-dimensional (3D) and two dimensional (2D) numerical analysis of a case study of a combined vacuum and surcharge preloading project for a storage yard at Tianjin Port, China, Buddhima Indraratna (2008). At this site, a vacuum pressure of 80 kPa and a fill surcharge of 50 kPa were applied on top of the 20 m-thick soft soil layer through prefabricated vertical drains PVD to achieve the desired settlements and to avoid embankment instability.

In 3D analysis, the actual shape of PVDs and their installation pattern with the in situ soil parameters were simulated. In contrast, the validity of 2D plane strain analysis using equivalent permeability and transformed unit cell geometry was examined. In both cases, the vacuum pressure along the drain length was assumed to be constant as substantiated by the field observations. The finite-element code, ABAQUS, using the modified Cam-clay model was used in the numerical analysis.

The predictions of settlement, pore-water pressure, and lateral displacement were compared with the available field data, and an acceptable agreement was achieved for both 2D and 3D numerical analyses.

It is found that both 3D and equivalent 2D analyses give similar consolidation responses at the vertical cross section where the lateral strain along the longitudinal axis is zero. The influence of vacuum may extend more than 10 m from the embankment toe, where the lateral movement should be monitored carefully during the consolidation period to avoid any damage to adjacent structures.

Hird et al. (1995), Chai et al. (1995), and Indraratna et al. (2005) introduced an *equivalent* two-dimensional (2D) plane strain approach to predict the soft clay behavior improved by the vertical drain system (**Fig. 2.12**).

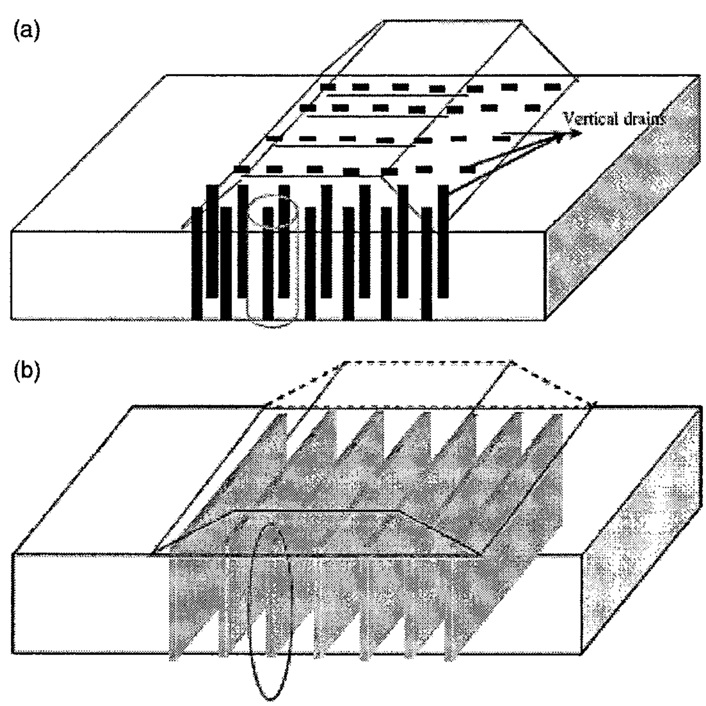



Figure 2.12. PVDs configuration
(a) 3D condition (square pattern); (b) Equivalent plane strain condition

The embankment loading is considered a strip load. This method can be conveniently simulated as a multi-drain system in numerical finite-element modeling (FEM) modeling. Discrepancies between 2D predictions and observations, especially in terms of excess pore pressure and lateral displacements, are often noted (Cheung et al. 1991).

Since the last decade, improved and user-friendly three-dimensional (3D) finite element codes have emerged as a powerful tool capable of capturing ground response details that cannot be analyzed using traditional 2D (plane strain) finite-element software (Small and Zhang 1991).

### *2.8.1 Three-Dimensional Finite-Element Analysis*

For 3D analysis, a single row of drains with influence zones has been considered, however a smear zone could not be concerned (Cheung et al. 1991; Borges 2004). This study demonstrates that a 3D analysis should be considered for embankments where the 2D plane strain condition may not be appropriate due to the nature of embankment geometry among the other reasons

### *2.8.2 Two-Dimensional Plane Strain Finite-Element Analysis*

To focus on the radial consolidation problem using a plane strain finite-element analysis, the appropriate equivalence between the plane strain and true axisymmetric analysis has to conduct to obtain realistic predictions. Various conversion procedures have been proposed earlier (e.g., Shinsha et al. 1982; Hird et al. 1992; Bergado and Long 1994; Chai et al. 2001; Indraratna et al. 2005). Cheung et al. (1991) employed the conversion procedure, which assumes that the settlement response at 50% degree of consolidation is the same for both 2D and axisymmetric (3D) conditions (Shinsha 1991).

However, significant differences of the excess pore pressure predictions were found between these two cases. In this study, the conversion method proposed by Indraratna et al. (2005) is adopted for the 2D plane strain analysis. In this approach, not only is the entire degree of consolidation response for the equivalent 2D approach the same as that of the 3D analysis, but also the smear zone was explicitly modeled. Even though this equivalent method may increase the number of elements significantly in the FEM mesh, hence the computational time, the method still provides an acceptable accuracy for multi drain analysis (Indraratna et al. 2004). Indraratna et al. (2005) have further refined details of the permeability conversion for the equivalent plane strain condition to consider the vacuum consolidation. A summary of the conversion from the axisymmetric to the equivalent plane strain model is presented below, for the benefit of the readers.

To obtain the same consolidation as the axisymmetric condition, the corresponding ratio of the smear zone permeability to the undisturbed zone permeability in plane strain analysis ($k_{s,ps}/k_{h,ps}$)can be obtained by (Indraratna et al. 2005) as shown in Eq.(2.22)

$$\frac{k_{s.ps}}{k_{h.ps}}=\frac{\beta}{\frac{k_{h.ps}}{k_{h.ax}}\left[\ln\left(\frac{n}{s}\right)+\frac{k_{h.ax}}{k_{s.ax}}\ln(s)-\frac{3}{4}\right]-\alpha} \tag{2.22}$$

Where:

$$\beta=\frac{2(s-1)}{n^2(n-1)}\left[n(n-s-1)+\frac{1}{3}(s^2+s+1)\right]$$

$$\alpha=\frac{2}{3}\frac{(n-s)^3}{n^2(n-1)};\qquad s=\frac{d_s}{d_w};\qquad n=\frac{d_e}{d_w}$$

where

$k_{s,\mathrm{ax}}$ and $k_{h,\mathrm{ax}}$: horizontal soil permeability in the smear zone and in the undisturbed zone, respectively,

In the axisymmetric configuration;

$d_e$: diameter of soil cylinder dewatered by a drain;

$d_s$: diameter of the smear zone;

$d_w$: equivalent diameter of the drain,

By ignoring both smear and well resistance effects, the simplified ratio of equivalent plane strain to axisymmetric permeability in the undisturbed zone can be attained as Eq.(2.23)

$$\frac{k_{h.ps}}{k_{h.ax}}=\frac{0.67\frac{(n-1)^2}{n^2}}{\left[\ln(n)-\frac{34}{}\right]} \tag{2.23}$$

### *2.8.3 One - D Finite-Element Analysis*

Considering that most finite element codes used in practice do not include special drainage element, a simple approximate method for modeling the effect of PVD has been proposed by Chai and Miura (1997). The PVD increases the mass permeability in the vertical direction. Consequently, it is possible to establish a value of the vertical permeability which approximately represents the combined vertical permeability of the natural subsoil and the radial permeability towards the PVD. This equivalent vertical permeability *($K_{ve}$)* is derived based on equal average degree of consolidation together with several assumptions:

The deformation mode of PVD improved subsoil is close to one-dimensional. Thus, one-dimensional consolidation theory can be used to represent the consolidation in the vertical direction and the unit cell theory of Hansbo (1979) for radial consolidation is applicable.

The total degree of consolidation is the combination of vertical and radial consolidation by using the relationship proposed by Scott (1963).

In order to obtain a one-dimensional expression for the equivalent vertical permeability, an approximate equation for consolidation in vertical direction is proposed as follows:

$$U_v = 1 - \exp(-3.54)\ T_v$$

Where:

$T_v$: the dimensionless time factor.

$$T_v = \frac{C_v t}{H^2} \qquad\qquad C_v = \frac{K_{ve}}{m_v \gamma_w}\ (\mathrm{m^2/sec})$$

Where:

$C_v$: vertical consolidation coefficient.

t: time

H: length of vertical drain

$m_v$: coefficient of volume change $m_v = \Delta e / \Delta \sigma_v$

The equivalent vertical permeability, $K_{ve}$, can be expressed as:

$$K_{ve} = \left(1 + \frac{2.26.L^2}{F.D_e^2} \cdot \frac{K_h}{K_v}\right).K_v$$

Where

$$F = \ln\left(\frac{D_e}{d_w}\right) + \left(\frac{K_h}{K_s} - 1\right)\ln\left(\frac{d_s}{d_w}\right) - \frac{3}{4} + \frac{\pi 2L^2 K_h}{3q_w}$$

$D_e$: the equivalent diameter of a unit PVD influence zone,

$d_s$: the equivalent diameter of the disturbed zone,

$d_w$: the equivalent diameter of PVD,

$K_h$ and $K_s$: the undisturbed and disturbed horizontal permeability of the surrounding soil, respectively,

$q_w$: the discharge capacity of PVD.

The effects of smear and well resistance have been incorporated in the derivation of the equivalent vertical permeability.

The horizontal deformation $\varepsilon_r$, which is the typical deformation of soft soil improvement by vacuum, could not be measured. So far, the controlling surcharge processing during vacuum construction has not been discussed sufficiently