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# Current State of Knowledge on Thermal Consolidation using Prefabricated Vertical Drains

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**ABSTRACT:** Several research works have demonstrated that subjecting normally consolidated clays to temperature less than the boiling point of water (100$^o$C) will have positive effects on its hydro-mechanical behaviour. Such effects can be exploited in improving the performance of the well-known preloading ground improvement technique that utilizes prefabricated vertical drains (PVDs). In this review paper, the applicability of a novel prefabricated vertical thermal drain (PVTD) will be presented and discussed using results of large oedometer tests and full-scale embankment tests on soft Bangkok clay. The large oedometer test results gave promising outcomes since the temperature accelerates the rate of consolidation and increases the amount of total settlement. The viability of the proposed technique was also confirmed by the full-scale embankment test results. The success of the proposed technique can be attributed to the thermally induced volume change and the increase in the hydraulic conductivity as the soil temperature increases.

Keywords: Thermal consolidation, temperature effects, Bangkok clay, prefabricated vertical drain (PVD), ground improvement.

## 1. INTRODUCTION

Construction of road embankments on top of soft normally consolidated clay deposits requires preconsolidation and strengthening of the weak compressible soils. Prefabricated Vertical Drains (PVD) are a time tested, very effective, and economical ground modification technique in such deposits (Bergado et al., 2002; Chu et al. 2004; Lorenzo et al., 2004; Chai and Carter 2011). However, the installation of prefabricated vertical drains using a mandrel causes disturbance of clay surrounding the drain, resulting in a smear zone of much lower horizontal permeability of this clay. The presence of a smear zone significantly influences the horizontal consolidation, resulting in retardation of the overall consolidation rate (Bergado et al., 1991; Hird and Moseley 2000). The long duration required to accomplish the ground improvement using PVDs is one of the disadvantages of this technique.

The aim of this paper is to review and discuss the effect of soil temperature on the performance of preloading with PVD ground improvement method using the results of large oedometer tests and full-scale embankment tests conducted by Abuel-Naga et al. (2006a) and Pothiraksanon et al. (2010), respectively.

The experimental program of the large oedometer test was designed to understand the thermo-mechanical consolidation behaviour of soft Bangkok clay using PVD, line heat source, and prefabricated vertical thermal drains (PVTD) with different arrangements. To look into the field feasibility of this innovative thermal ground improvement technique, two identical 6.0 m high full-scale test embankments for preloading were constructed over the soft Bangkok clay. A conventional PVD system was installed underneath one embankment and the novel PVTD system was utilized for the other.

In the following sections, a brief background pertaining to the thermo-mechanical behaviour of saturated clays and in particular soft Bangkok clay is presented. Then, the large oedometer thermal consolidation tests conducted by Abuel-Naga et al. (2006a), and the consolidation behaviour of the full-scale embankment test on soft Bangkok clay using PVTD that conducted by Pothiraksanon et al. (2010) are described and discussed. Finally, conclusions are drawn.

## 2. THERMO-MECHANICAL BEHAVIOR OF SATURATED CLAYS

### 2.1 Thermally induced volume change

The previous studies in the literature have conclusively shown that volumetric change of saturated fine-grained soils subjected to temperatures less than the boiling point of water (100$^o$C), depend on the stress history (Baldi et al. 1988; Towhata et al. 1993; Kuntiwattanakul et al. 1995; Del Olmo et al. 1996; Delage et al. 2000; Burghignoli et al. 2000; Laloui and Cekerevac 2003; Graham et al. 2001; Cekerevac and Laloui, 2004). In terms of volumetric strains, the normally consolidated clays contract irreversibly and non-linearly upon heating whereas the highly overconsolidated clays exhibit reversible expansion as shown in Figure 1 for soft Bangkok clay. The effect of stress history on the thermally induced volume change of different clays under different elevated temperature changes, $\Delta T$, is depicted in Figure 2. It can be seen that with increasing the over consolidation ratio, OCR, the magnitude of the thermally induced contraction volumetric strain decreases and then gradually starts to show a dilative behaviour beyond a certain OCR value.

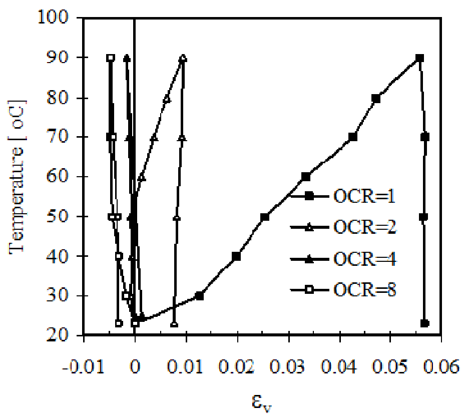


Figure 1 Soft Bangkok clay temperature volumetric strain under drained heating/cooling cycle at different OCR values, preconsolidation pressure = 200 kPa, (Abuel-Naga et al., 2006b)

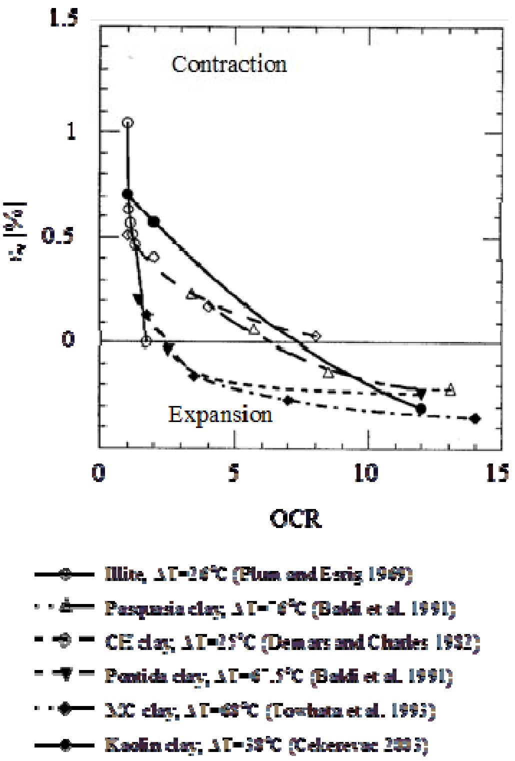


Figure 2 Effect of OCR on the temperature induced volume change for different types of clay from literature (Abuel-Naga et al., 2007c)

Many researchers attributed the thermally induce volume change behaviour to the temperature effect on the physico-chemical interactions between the clay particles which depends essentially on the clay lattice constitution, the chemical nature of the interstitial fluid, and interlayer distance (Robinet et al. 1996; Laloui and Cekerevac 2003). As the soil plasticity index, PI, could give a qualitative indication for the physico-chemical interactions of clays, PI could affect the magnitude of thermally induced volumetric strain. Figure 3 shows the relationship between the temperature induced volumetric strains of different types of normally consolidated clay under ΔT ≈ 65 to 70°C, and their PIs. The results show a reasonable linear trend between the thermally induced volume change and PI.

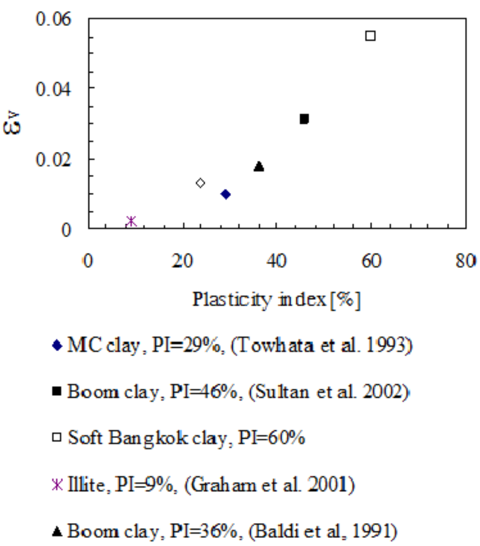


Figure 3 Relationship between plasticity index of soil and temperature induced volumetric strain of different normally consolidated clays where ΔT≈65-70°C, and $T_o$≈20-25°C, (Abuel-Naga et al., 2007c)

## 2.2 Effects of temperature on preconsolidation pressure and normal compression line

The previous experimental results reported by Eriksson (1989), Boudali et al. (1994), and Moritz (1995) show a decrease in the preconsolidation pressure with increasing temperature up to 20 to 25°C. However, above this temperature level (20 to 25°C) the preconsolidation pressure is approximately constant and independent of temperature as shown in Figure 4. Furthermore, earlier experimental results by Hueckel and Baldi (1990) and Robinet et al. (1996) on reconstituted Pontida silty clay and Boom clay, respectively, show that subjecting the normally consolidated clay to heating/cooling cycle induced an apparent overconsolidation state at constant plastic strain condition. Consequently, further loading under elastic stiffness condition is required to reach again the yielding mode. Along the same line, the preconsolidation pressure of soft Bangkok clay shows temperature independency during the heating phase from 25°C to 90°C, as shown in Figure 5. Furthermore, soft Bangkok clay shows apparent overconsolidation state after subjected to a heating/cooling phase as shown in Figure 6. The previous research works also show that the compression line moves to the left as the soil temperature increases, with similar slope, causing reduction in the elastic domain at constant plastic strain (Campanella and Mitchell 1968; Hueckel and Baldi 1990; Graham et al. 2001; Laloui and Cekerevac 2003). Similar behaviour was observed for soft Bangkok clay as shown in Figure 5.

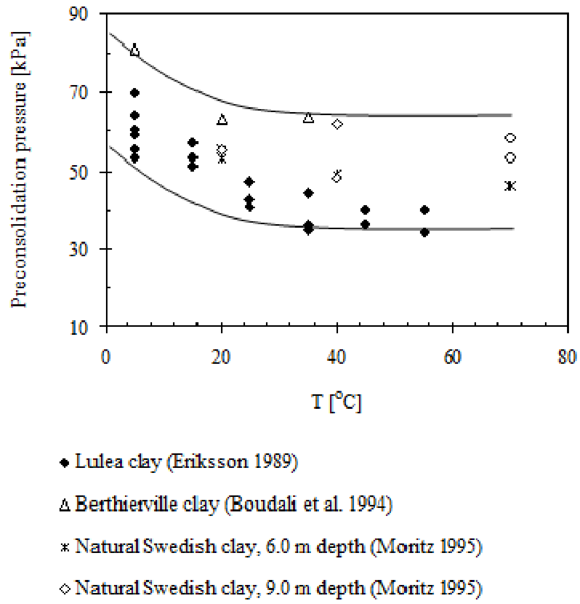


Figure 4 Temperature effect on preconsolidation pressure (Abuel-Naga et al., 2007c)

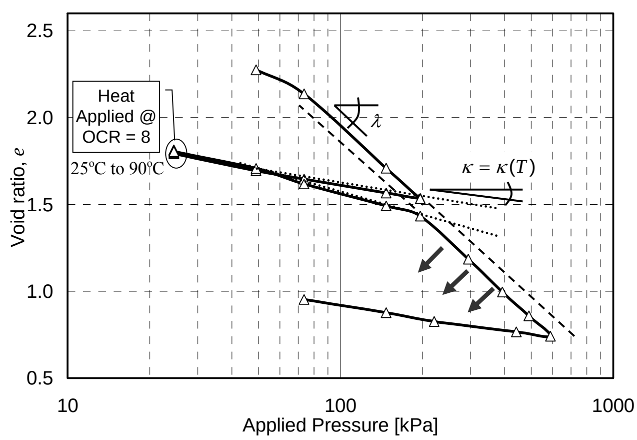


Figure 5 Consolidation curve of soft Bangkok clay at different temperatures (Trani et al., 2010)

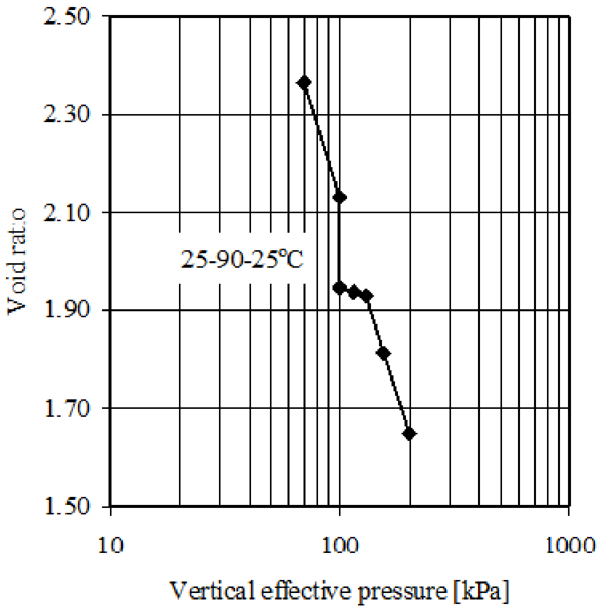


Figure 6 Temperature induced overconsolidation state of normally consolidated soft Bangkok clay after drained heating/cooling cycle (Abuel-Naga et al., 2006b)

## 2.3 Effects of temperature on shear strength

Graham et al. (2001) investigated the undrained shear strength of normally consolidated remolded Illitic clay specimens at different temperature levels (28, 65, 100°C). The specimens were subjected to drained heating before shearing under undrained condition at elevated temperature. The results showed that the shear strength increased as the temperature increased as shown in Figure 7. Similar observation was reported by Kuntiwattanakul et al. (1995) for reconstituted MC clay specimens tested at room temperature (20°C) and elevated temperature of 90°C as shown in Figure 8. Abuel-Naga et al. (2006b; 2007b) investigated experimentally the effect of temperature on the undrained triaxial compression shear strength behaviour of normally consolidated soft Bangkok clay specimens at different temperature levels and histories. Temperature histories relate to specimens being subjected to heating/cooling cycles before conducting shear testing. The test results indicated that the undrained shear strength and secant modulus of the normally consolidated clay increases as the soil temperature increases or after subjecting to a temperature history as shown in Figure 9. Abuel-Naga et al. (2007c; 2009b) proposed a robust constitutive model for predicting the temperature effects on saturated clays. The model predictions for the different types of clay are shown in Figures 7, 8 and 10. Reasonable agreement can be observed between the test results and the model predictions.

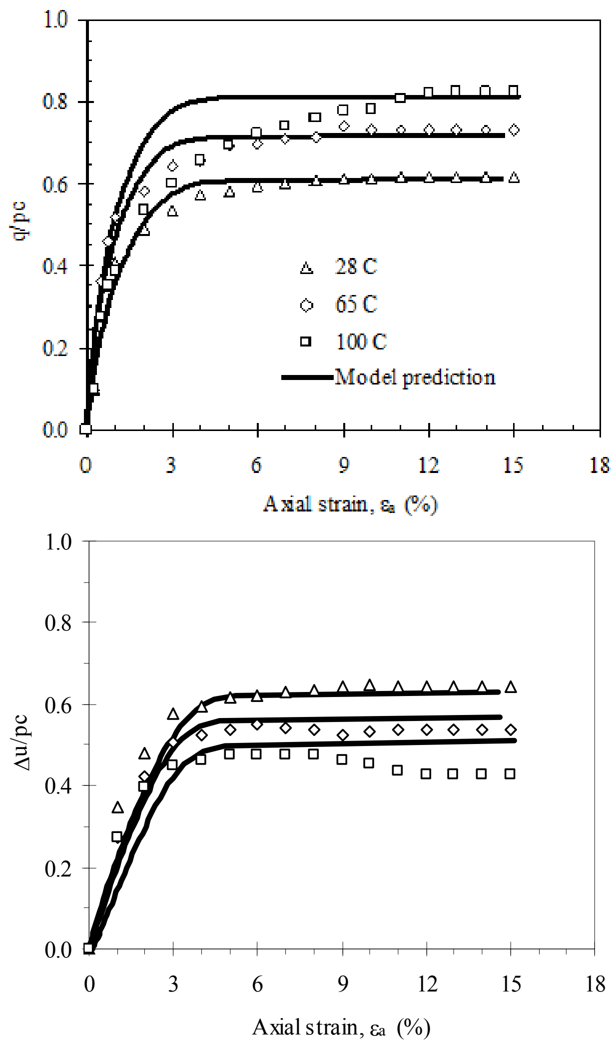


Figure 7 Comparison between the proposed model prediction (Abuel-Naga et al., 2009b) and the compression triaxial test results of normally consolidated MC clay specimen at different temperature levels, $p_c$=1500 kPa, (Graham et al. 2001)

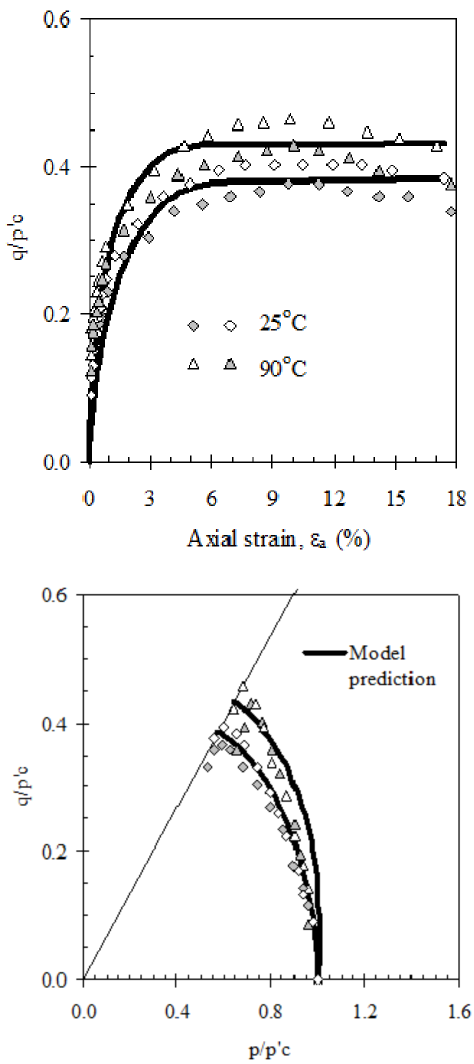


Figure 8 Comparison between the proposed model prediction (Abuel-Naga et al., 2009b) and the compression triaxial test results of normally consolidated MC clay specimen at different temperature levels, $p_c$=196 kPa, (Kuntiwattanakul et al. 1995)

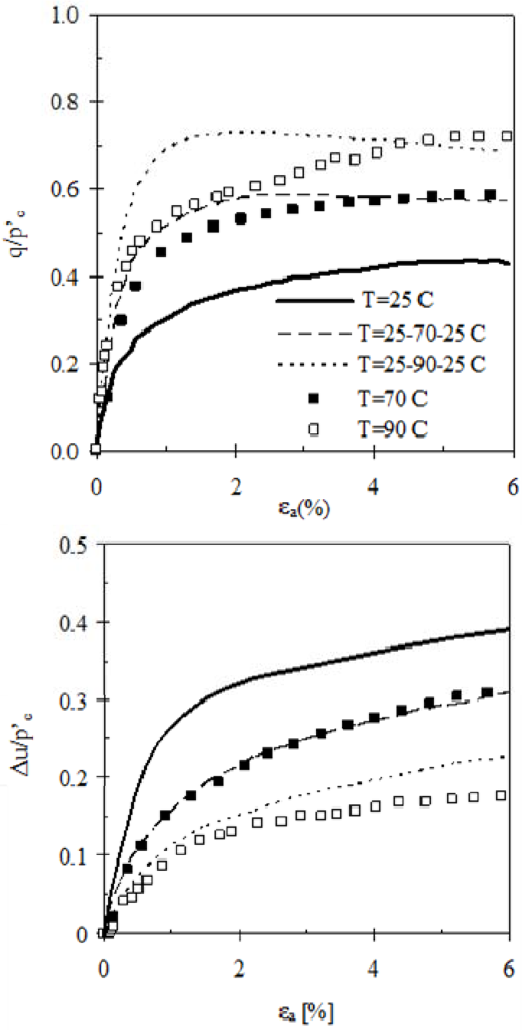


Figure 9 Undrained triaxial compression test results of normally consolidated soil tested at different temperatures or after subjecting to different heating/cooling cycles (Abuel-Naga et al., 2006)

## 2.3 Effect of temperature on hydraulic conductivity

Many researchers studied the effect of temperature on the coefficient of hydraulic conductivity (Towhata et al., 1993; Habibagahi 1977; Morinl and Silva 1984; Houston and Lin 1987; Burghignoli et al. 1995; Delage et al., 2000). In general, all of these studies have reported that the hydraulic conductivity of soil increased with increasing the temperature. However, some of these studies used the indirect method which employs the coefficient of consolidation measurements obtained from isothermal consolidation tests performed at various temperatures (Habibagahi 1977; Towhata et al. 1993). On the other hand, some of these studies used the direct methods such as the constant head method (Morinl and Silva 1984; Delage et al. 2000). In fact, as Terzaghi theory has a series of assumptions which do not properly fit the actual behaviour of natural clays the indirect method should not be used for determining the hydraulic conductivity (Tavenas et al. 1983). Delage et al. (2000) observed that hydraulic conductivity values estimated by the indirect method is higher than the hydraulic conductivity estimated using direct method by factor of about four in the case of Boom clay.

The effect of temperature up to 90°C on hydraulic conductivity of soft Bangkok clay was investigated by Abuel-Naga (2006b) using flexible wall permeameter.

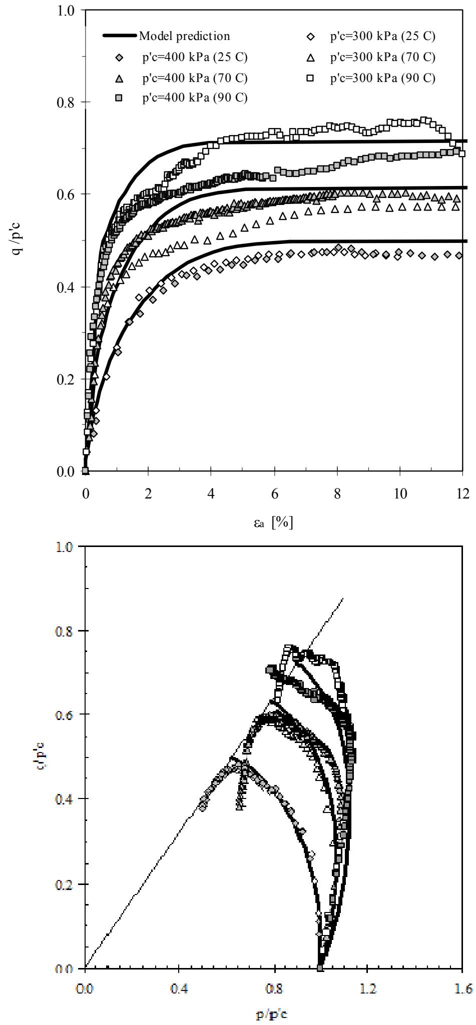


Figure 10 Comparison between the proposed model prediction (Abuel-Naga et al., 2009b) and the compression triaxial test results of normally consolidated soft Bangkok clay specimen at different temperature levels (Abuel-Naga et al. 2007b)

The results indicated that as the soil temperature increased, the hydraulic conductivity also increased as shown in Figure 11. This behaviour was attributed to the thermal evolution of the pore soil liquid viscosity.

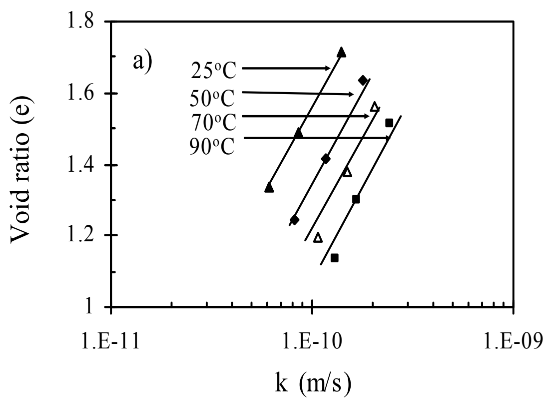


Figure 11 Effect of temperature on hydraulic conductivity of soft Bangkok clay (Abuel-Naga 2006b)

### 3.1 Large Oedometer Test (Abuel-Naga et al., 2006a)

Abuel-Naga et al. (2006a) investigated the effect of soil temperature on the performance of PVD using a large oedometer apparatus as shown in Figure 12. Soft Bangkok clay samples obtained from 3.0 to 4.0 m depth have been used in these studies. The height and inner diameter of the odemoeter cell was 200 mm. Dead load was used to apply the vertical stress. The soil temperature was raised using a line heat source either attached to the PVD point (PVTD) or installed independently between the PVD points. The scaled-down PVDs were created by disassembling, cutting, and reassembling the full-size drains. The core was cut to 20 mm in width and about 200 mm in length. The PVTD was created by using two scaled-down PVD cores fitted back to back with a flexible wire heater (2 mm in diameter) placed in the grooves as shown in Figure 13a. The separate line heat source was created by wrapping a flexible wire heater around a metal plate 20 mm wide and 200 mm long as shown in Figure 13b. For both types of line heat source, a thermocouple (K-type) was placed at the mid-height of the line heat source with direct contact with the surrounding soil. This thermocouple was used for both temperature measurements and the feedback signal for the thermo-controller unit.

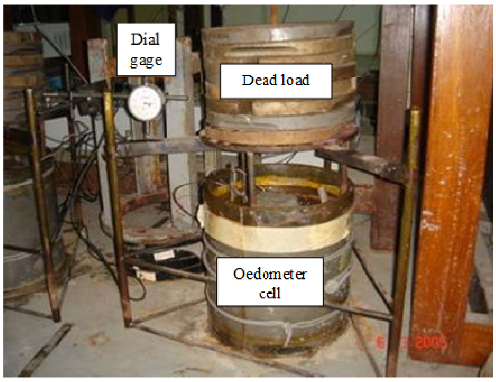


Figure 12 Large oedometer apparatus (Abuel-Naga et al., 2006a)



Figure 13 a) PVTD configuration; b) Line heat source configuration (Abuel-Naga et al., 2006a)

Figure 14 shows different arrangements of PVD, PVTD, and line heat source that were investigated under thermo-mechanical consolidation, with simultaneous increases of soil temperature (from 25 to 90$^{o}$C) and effective stress (from 0.0 to 30 kPa). Reference testing was also conducted where only the vertical effective stress was increased from 0.0 to 30 kPa while the soil temperature was not changed. The settlement induced by mechanical (reference test) and thermo-mechanical path was measured with time.

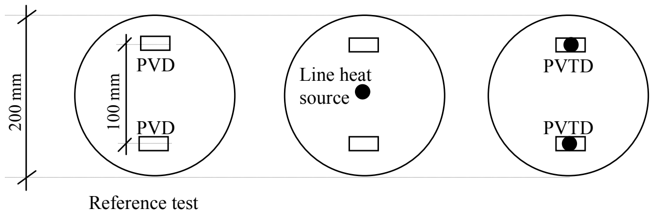


Figure 14 Thermo-mechanical consolidation test configurations (Abuel-Naga et al., 2006a)

## 3. THERMAL CONSOLIDATION TECHNIQUE

Based on the above discussion for the temperature effects on the hydro-mechanical behaviour of clays it can be concluded that increasing the temperature of saturated clays to less than the boiling point of water (100$^{o}$C), affects positively its engineering properties (permeability, compressibility, and shear strength). This conclusion triggered the development of an innovative thermal ground improvement technique that was tested on soft Bangkok clay using large oedometer apparatus by Abuel-Naga et al. (2006a). Furthermore, the feasibility of this technique was also assessed using full-scale embankment tests by Pothiraksanon et al. (2010). Table 1 shows the physical properties of soft Bangkok clay. XRD analysis shows the mineralogical composition of soft Bangkok clay consists of Smectites (Montmorillonites and Illites) ranging from 54 to 71% with Kaolinites (28 to 36%) and micas (Ohtsubo et al. 2000). The experimental program and the obtained results of Abuel-Naga et al. (2006a) and Pothiraksanon et al. (2010) will be presented and discussed in the following sections.

Table 1 Physical properties of soft Bangkok Clay (Abuel-Naga et al., 2006b)

| | |
|---|---|
| Liquid limit (%) | 103 |
| Plasticity index | 60 |
| Water content (%) | 90-95 |
| Liquidity index | 0.62 |
| Grain Size Distribution | |
| Clay (%) | 69 |
| Silt (%) | 28 |
| Sand (%) | 3 |
| Total unit weight (kN/m$^3$) | 14.3 |
| Dry unit weight (kN/m$^3$) | 7.73 |
| Specific gravity | 2.68 |
| Specific surface area (m$^2$/g) | 237 |

The results in Figure 15 show that raising the soil temperature increases significantly the consolidation rate. Therefore, the thermal consolidation method can be considered a promising approach since it enhanced the performance of preloading with PVD by reducing the consolidation time. The results in Figure 15 also indicate that using PVTD is preferable than the separate line heat source since the thermo-PVD induced higher consolidation rate. The advantage of the PVTD can be attributed to the coincidence of the drainage point and the smear zone at the centre of the maximum temperature zone. Consequently, significant reduction of the smear effect can be achieved in this case since raising the temperature increases the hydraulic conductivity of soils as shown in Figure 11.

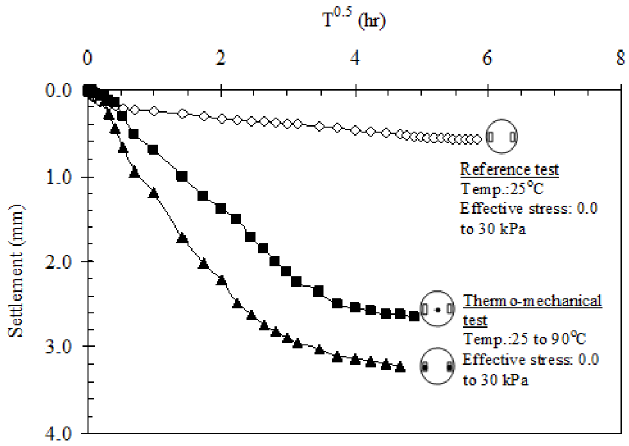


Figure 15 Thermo-mechanical consolidation test results of undisturbed specimen (Abuel-Naga et al., 2006a)

### 3.2 Full-Scale Embankment Test (Pothiraksanon et al., 2010)

The site of the embankment tests is located inside the campus of the Asian Institute of Technology (AIT), which is 42 km north of Bangkok, Thailand. AIT is located within the Central Plain of Thailand which contains the deltaic-marine deposit of soft clay widely known as "soft Bangkok clay". The typical stratigraphy at the location of AIT consists of an upper weathered crust of dark brown clay from the ground surface to about 2.0 m depth. This layer is underlain by soft, highly compressible, gray clay with fissures, silt seams and fine sand lenses down to about 8.0 to 9.0 m depth. Below the soft clay layer lies about 6.0 m of stiff clay. Then, dense to very dense sand and gravel layers alternate with stiff to hard clay layers starting at 14.0 m to about 400 m depth. The ground water table fluctuates with the season but is close to an average value of 2.0 m below the ground level.

#### 3.2.1 Embankment test construction

Two identical 6.0 m high full-scale embankments were constructed at the AIT site where the distance between them is 60.0 m. Figure 16 shows the general layout of the constructed embankment. The dimensions of the embankments were 11 m x 11 m at the bottom and 3 m x 3 m at the top. The embankment fill material consisted of compacted Ayuttaya sand. Geogrid and geotextile were used for increasing the slope of embankment. Sand gabion blocks were also used along the upper part of embankments as shown in Figure 17. A conventional PVD system was installed underneath the first embankment whereas a novel prefabricated vertical thermo-drain (PVTD) system was utilized for the second one.

Nine PVD/PVTDs 8.0 m deep were installed beneath the embankments on a square grid of 1.0 m spacing. A commercial PVD with 100 mm × 4.3 mm cross-section was utilized in this study. The PVTD unit consists of a U-tube made of cross-linked polyethylene plastic (PEX) attached to a conventional PVD unit as shown in Figure 18. Preheated water at about 70 to 90°C is circulated through the attached U-tube to raise the soil temperature underneath the embankment. A solar panel system was used to heat the circulated water from ambient temperature (25°C) to 72°C then an electrical heater was utilized to raise its temperature from 72°C to 90°C. A special water pump able to work at elevated temperatures was used to circulate the hot water through the PVTD system.

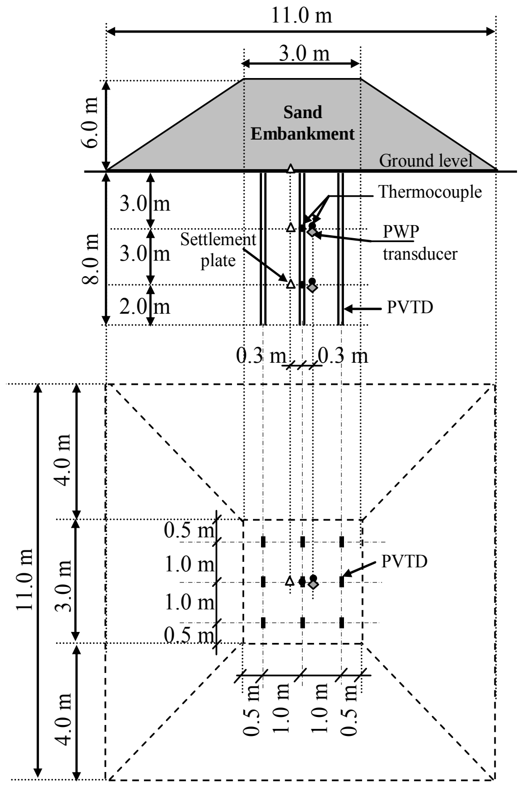


Figure 16 Layout of full-scale PVTD embankment (Pothiraksanon et al., 2010)

Figure 17 Use of geogrid, geotextile, and sand gabion blocks for increasing the slope of embankment (Pothiraksanon et al., 2010)

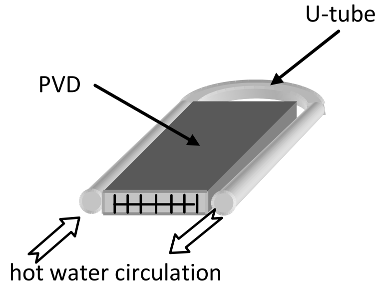


Figure 18 PVTD configuration (Pothiraksanon et al., 2010)

The monitoring system of PVTD embankment consisted of settlement plates installed 0.3 m away from the central PVTD point at three different depths (0.0, 3.0, 6.0 m) as shown in Figure 16. On the other side of the central PVTD two pairs of thermo-couples were installed at 3.0 and 6.0 m depth. Each thermocouple pair consisted of one thermocouple attached to the outer side of the U-tube whereas the second was located 0.3 m apart from the PVTD unit as shown in Figure 16. Furthermore, two pore water pressure (pwp) transducers were installed 0.3 m away from the central PVTD point at two different depths (3.0, 6.0 m) as shown in Figure 16. The monitoring system of the PVD embankment was similar to the PVTD embankment except for the thermo-couples, as one thermo-couple was installed only 0.5 m away from the central PVD point at 3.0 m depth. Moreover, two additional pore water pressure transducers were installed 50.0 m away from both embankments at two different depths (3.0, 6.0 m) to record reference pore water pressure at the site. The testing program of the PVD and PVTD embankments included; the embankment building stage, the mechanical consolidation stage for the PVD embankment and thermo-mechanical consolidation stage for the PVTD embankment. During these stages, temperature, pore pressures, and settlement readings were collected at different time intervals. The thermo-mechanical consolidation stage involves circulation of hot water (70 to 90°C) through the PVTD system. The heating stage lasted for 110 days until the primary consolidation was completed. Then, the whole system was left to cool down for about 90 days.

### 3.2.2 Heat transfer

The temperature at the PVD embankment test was approximately 25±1oC during the test period. This observation confirms that the heat radius of influence of the PVTD embankment is less than the distance between the two embankments (60.0 m). Figure 19 shows the PVTD embankment temperature history at 3.0 m depth during the testing period. The first curve (solid line) shows the interface temperature between the PVTD point and the surrounding soil whereas the second curve (dashed line) shows the soil temperature 0.3 m away from the central PVTD as shown in Figure 16. The observed temperature fluctuation was due to a technical problem that occurred in the electrical heating system during the test period. Abuel-Naga et al. (2009a) used these results to numerically back-calculate the thermal conductivity of soft Bangkok clay. The numerically obtained thermal conductivity value (1.3 W/m°C) was very close to the laboratory determined values by Abuel-Naga et al. (2008, 2009a).

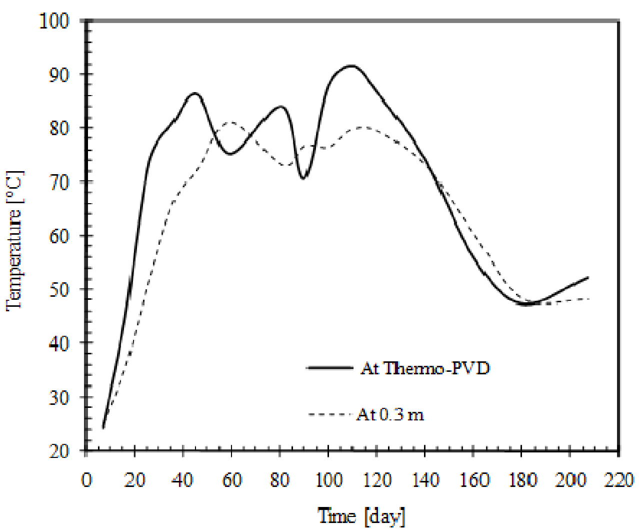


Figure 19 Typical temperature history of PVTD embankment at 3.0 m depth (Pothiraksanon et al., 2010)

### 3.2.3 Consolidation results

Figure 20 shows the initial and final stresses under the centre line of the test embankment and the clay maximum past pressure as well. The stress condition indicated that the soft clay layer located between 2.0 to 7.0 m depth is normally consolidated. The settlement results of both embankments are plotted in Figure 21. In general, the PVTD embankment yields more settlement. The analysis of settlement data of the layer from 0.0 to 6.0 m depth is illustrated in Figure 22. The change in the thickness of this layer (ΔH) was calculated using the measurements of the settlement plate at surface, $S_o$, and 6.0 m depth, $S_6$, ( $\Delta H = S_o - S_6$) as shown in Figure 22a. The excess pore water pressure measurements at 3.0 m depth were used to determine the end of the primary consolidation stage as shown in Figure 22b. The PVTD embankment shows higher excess pore water pressure than the PVD embankment due to thermally induced pore water pressure and volume change (Abuel-Naga et al. 2007a). The thermally induced pore water pressure is generated due to the difference in the thermal expansion coefficient of water and the soil solids. The thermal expansion coefficient of water ($\alpha_w$= 1.7 x $10^{-4}$ $^oC^{-1}$) is approximately 15 times larger than thermal expansion coefficient of solids ($\alpha_s$= 1.0 x $10^{-5}$ $^oC^{-1}$).

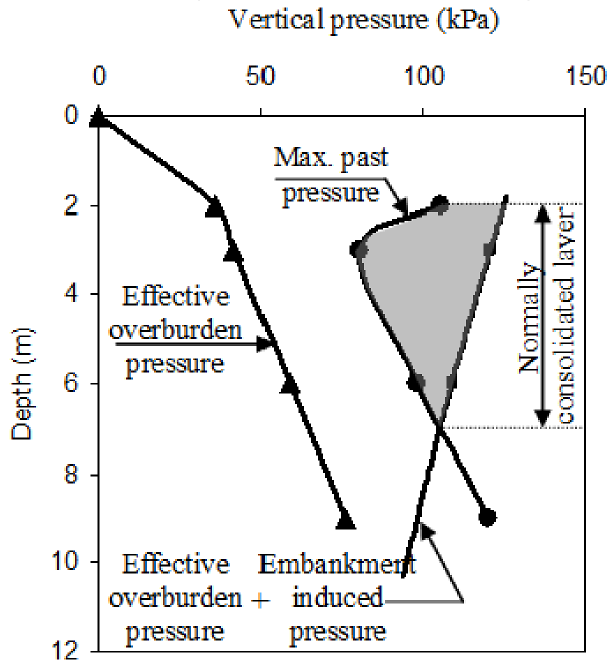


Figure 20 Stress condition under the centre line of the test embankment (Pothiraksanon et al., 2010)

The difference in ΔH between the two embankments is attributed to the thermal consolidation at the PVTD embankment, which shows irreversible behaviour upon soil cooling. As the heating/cooling process of normally consolidated clay changes it to a lightly over consolidated clay as shown in Figure 6 (Abuel-Naga et al. 2006b), the clay underneath the PVTD embankment can carry extra load with less settlement. The results in Figure 22a also show that the consolidation rate of the PVTD embankment is higher than the PVD embankment. The amount of consolidation generated by the PVD embankment at the end of its primary consolidation stage (after 80 days) can be achieved after 25 days for the PVTD embankment. This behaviour can be explained in light of the increase in hydraulic conductivity at elevated temperatures as the result of the thermal evolution of water viscosity (Abuel-Naga et al. 2006b).

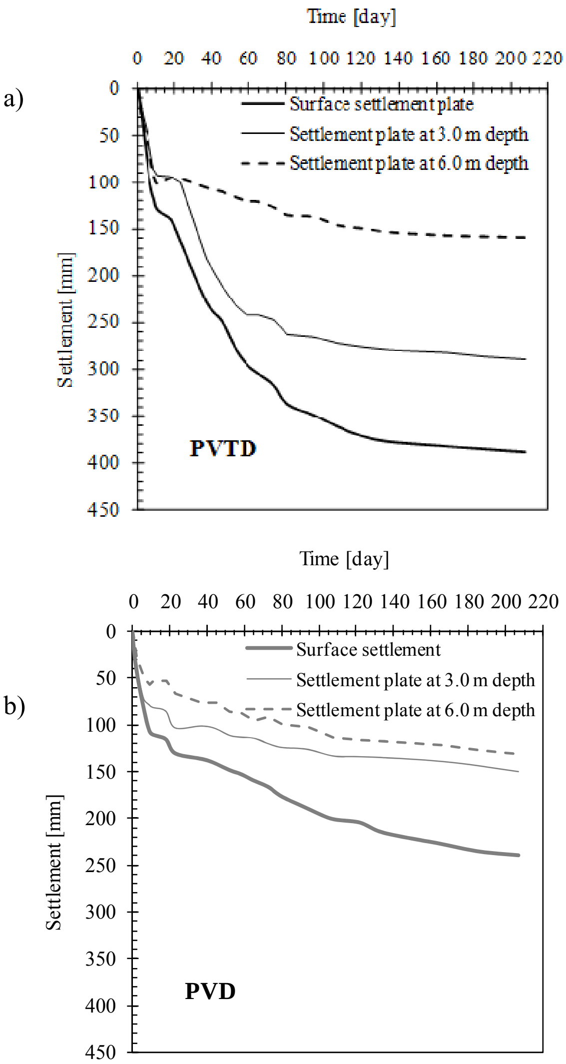


Figure 21 Settlement results of PVD and PVTD embankment (Pothiraksanon et al., 2010)

## 4. ANALYTICAL SIMULATION FOR CONSOLIDATION BEHAVIOR OF FULL-SCALE EMBANKMENT TEST

In this section the consolidation behaviour of soil underneath PVD/PVTD embankment shown in Figure 22a will be analytically simulated in order to investigate the temperature effect on the soil model parameters. The total consolidation of the soil ($\Delta H$) underneath the embankment has two components $\Delta H_u$ and $\Delta H_d$ which represents the elastic component that occurs initially under undrained condition, and the consolidation components that combined with drainage behaviour, respectively. The elastic component $\Delta H_u$ can be estimated as follows:

$$\Delta H_u = qBI\left(\frac{1-\nu^2}{E_u}\right) \tag{1}$$

where q = loading pressure (120 t/m2); B = the width of the embankment (3.0 m); I = an influence factor considering a rough rigid base (0.5); υ = Poisson's ratio (0.5), and Eu= undrained Young's modulus (Eu= 300 t/m2 for soft Bangkok clay). The component ΔHd due to the mechanical and thermal load can be determined using Abuel-Naga et al. (2007c) model.

The general form for the average degree of consolidation of soil cylinder containing a central drain can be expressed as follows (Hansbo 1979):

$$t = \left[\frac{D_e^2}{8c_h}\right]\left[F(n) + F_s + F_r\right] ln\left(\frac{1}{1-U_h}\right) \tag{2}$$

where $D_e$ = the diameter of the equivalent soil cylinder; ch = the horizontal coefficient of consolidation; t = the time after an instantaneous increase of the total vertical stress; F(n) = the factor that expresses the additive effect due to the spacing of drains. $F_s$ and $F_r$ are the smear effect and well resistance, respectively. Hansbo (1979) gave the following expressions for the spacing, well resistance, and smear factors, respectively:

$$F(n) = ln\left(\frac{D_e}{d_w}\right) - \frac{3}{4} \tag{3}$$

$$F_r = \pi z(L-z)\left(\frac{k_h}{q_w}\right) \tag{4}$$

$$F_s = \left(\frac{k_h}{k_s} - 1\right) ln\left(\frac{d_s}{d_w}\right) \tag{5}$$

$$F = F(n) + F_r + F_s \tag{6}$$

where $d_s$ = diameter of the smeared zone; $d_w$= equivalent diameter of the drain; L = length of the drain when opened at one end only; z = vertical distance from opened end of drain; $k_s$ = permeability of the disturbed zone in the horizontal direction; $k_h$ = horizontal hydraulic permeability; and $q_w$ = discharge capacity of the drain. As discharge capacities of most PVDs available in the market are relatively high, the well resistance effect can be ignored in most practical cases (Yeung 1997; Chu et al. 2004; Rujikiatkamjorn and Indraratna 2007).

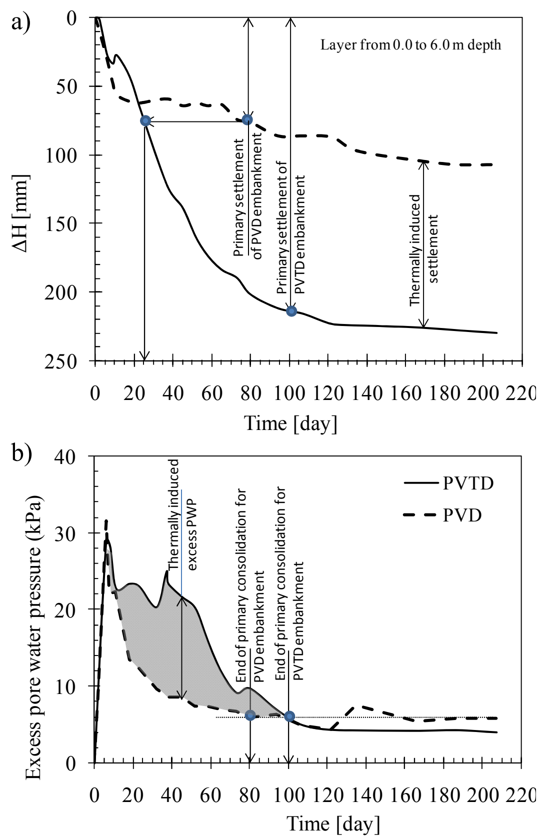


Figure 22 PVD and PVTD embankment results, a) consolidation of the layer from 0.0 to 6.0 m depth, b) excess pore water pressure at 3.0 m depth (Pothiraksanon et al., 2010)

Therefore, $F_r$ will not be considered in this study. Bergado et al. (1992) suggested that $d_s/d_w$= 2.5. Rixner et al. (1986) showed that $d_w$ can be obtained as follows:

$$dw = (a+b)/2 \tag{7}$$

where a and b = the thickness and width of the band-shaped drain, respectively. Hansbo (1979) analytical model (Eqs. 2 to 7) was used to back analyse the PVD embankments results presented in Figure 22a as explained by Bergado et al. (1992). The back-analysis showed that the values of $c_h$ and $k_h/k_s$ ratio for PVD embankment are 5.5 $m^2$/year and 9.0, respectively. The model prediction using these values is shown in Figure 23. In PVTD case, the effect of temperature on of $c_h$ and $k_h/k_s$ can be estimated initially using the laboratory experimental results shown in Figure 11. The results of heat transfer in Figure 19 indicated that within the smear zone ($d_s$=2.5$d_w$) the soil temperature is about 85$^o$C. Therefore, the increase of the horizontal hydraulic conductivity of the smear zone due to the imposed thermal load can be estimated based on the results in Fig. 11 as follows:

$$(k_s)^{PVTD} = 2.8\,(k_s)^{PVD} \tag{8}$$

The temperature effect on horizontal hydraulic conductivity in the area between PVTD points can be estimated based on the average temperature in this area. The heat transfer numerical simulation results that conducted by Abuel-Naga et al. (2007c) shows that the average temperature in this area is about 60$^o$C. Using the results in Figure 11, it can be estimated that:

$$(k_h)^{PVTD} = 1.8\,(k_h)^{PVD} \tag{9}$$

Consequently the values of ch and $k_h/k_s$ ratio for PVTD embankment can be estimated as follows:

$$c_h^{PVTD} = 1.8\,c_h^{PVD} = 9.9\ m^2/year \tag{10}$$

$$(k_h/k_s)^{PVTD} = (k_h/k_s)^{PVD}\,(1.8/2.8) = 5.78 \tag{11}$$

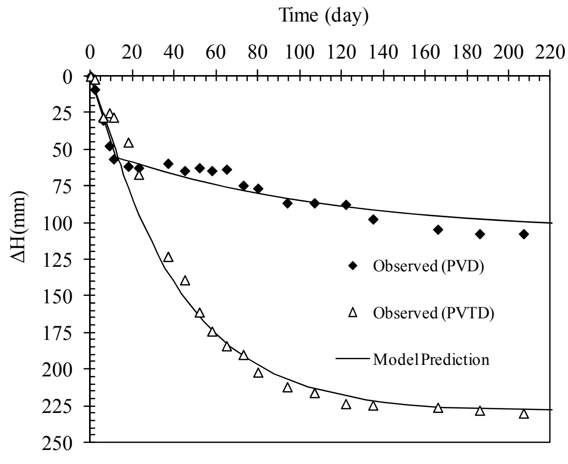


Figure 23 Comparison between the measured PVD and PVTD embankment consolidation and the model results

The design parameters for both embankments are listed in Table 2. The model prediction using the PVTD design parameters is shown in Figure 23. It fits well with the field measurements. These results indicated that PVTD system offers a solution to partially remove the smear effect and consequently accelerate the consolidation process using PVD system. Moreover, PVTD system also has another two important advantages regarding the irreversible contraction volume change and the improvement of shear behaviour upon subjecting the soft soils (normally consolidated clays) to heating/cooling cycle as shown in Figure 9.

Table 2 Design parameters

| | PVD embankment | PVTD embankment |
|---|---|---|
| $d_w$ (m) | 0.052 | |
| PVD spacing (m) | 1.0 (square grid) | |
| $d_s$ (m) | 0.13 | |
| $c_h$ ($m^2$/year) | 5.5 | 9.9 |
| $k_h/k_s$ | 9 | 5.78 |

## 5. CONCLUSION

Based on the laboratory and field results, the PVTD system shows the following advantages over the conventional PVD system:

- The PVTD system yields more settlement due to the thermal consolidation of normally consolidated clays.
- The thermal consolidation is irreversible upon cooling of the clays.
- The PVTD system shows a higher rate of consolidation. This behavior can be attributed to the increase of the soil hydraulic conductivity at elevated temperatures which can reduce the detrimental effects of the smear zone.
- The PVTD system changes the normally consolidated clays to lightly overconsolidated clays. Subsequently, the soil can carry extra load with less settlement.
- Using a solar heating technique with the PVTD system is a cost effective method to heat the ground.

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