Performance of PVD road embankment on soft Bangkok clay

L'Execution du draine vertical préfabriqué du remblai sur l'argille molle de Bangkok

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ABSTRACT: Performance of an 80km long road embankment (Bangkok-Chonburi New Highway) constructed on improved soft marine clay in Bangkok is evaluated during and after construction. PVDs associated with preloading were found to be effective to accelerate the consolidation settlement on soft Bangkok Clay. The data obtained from other ground improvement projects including Outer Bangkok Ring Road (Eastern Portion) and New Bangkok International Airport (under construction) are also referred for comparison.

RÉSUMÉ: La Performance d'un remblai routier de 80km de long (nouvelle autoroute Bangkok-Chonburi) construit sur un "argile marin" meuble amélioré à Bangkok est evalué pendant et après construction. PVDs associé à un pré-changemant Sést trouvé être éfficace pour accelerer le tarsement sur de l'argile meable de Bangkok avec forte teneur en eau et faible resistance au cisaillement. Les données obtenues d'autre projects d'amélioration de terrain y compris Outer Bangkok Ring Road (Partie Est) et New Bangkok International Airport (En Construction) sont aussi rapportés à titre de comparaison.

1. INTRODUCTION

The Department of Highways (DOH) in Thailand completed two major motorways construction in the greater Bangkok area recently to divert traffic, especially heavy trucks and towed vehicles, not to pass the inner city. The two motorways are the Bangkok-Chonburi New Highway (BCNH) and the Outer Bangkok Ring Road-Eastern Portion (OBRR) as shown in Fig. 1. Both motorways tend to alleviate the traffic congestion problem of Bangkok. The BCNH tends to run parallel with the Bang Na-Bang Pakong Highway (route no. 34) in the east-west direction, while the OBRR will run in a parallel direction with the route no. 1 in the north-south direction. Both motorways are embankment-type roadway with new alignments passing through the soft Bangkok Clay formation. Problems on stability and large settlement are expected to occur during roadway construction due to the limited Right of Way (ROW) and underneath soft clay. Preloading associated with Prefabricated Vertical Drains (PVDs) was employed at part of both motorways, where thick soft clay was encountered, to accelerate the primary consolidation settlement in order to reduce the post-construction settlement and maintenance cost along both motorways. There were 21 and 17 contracted sections for the total length of 80km and 52km at BCNH and OBRR, respectively. A total of 60km and 25km long road embankment was improved with PVDs along BCNH and OBRR, respectively. Construction of BCNH and OBRR was commenced in 1994 and was opened to the public in 1997 anc 1998, respectively.

Figure 1 Location plan of BCNH and OBRR (Eastern Portion)

As these two large-scale roadway projects seem to be the first introduction of PVDs in practice for the soft Bangkok Clay, a number of investigations were implemented at both roadways to evaluate the effectiveness of PVDs in accelerating the consolidation of highly plastic and soft Bangkok Clay as discussed in the following sections. The discussion in this paper mainly focuses on the BCNH where PVDs was installed at more than 75% of roadway section with large settlement encountered. A limited area at both motorways improved by using "cement column" method is not discussed in this paper.

2. SOIL PROFILE AND SOIL PROPERTIES

In general, the subsoil along project alignment can be classified into four strata as weathered crust and followed by very soft-to-soft clay, medium stiff clay, and stiff clay to the maximum borehole depth of 20m. Figs. 2 and 3 show the soil profile and natural water content, respectively, along the improved portion of BCNH. Based on the similarity of the subsoil properties, the whole project route under ground improvement portion has been divided into five zones, i.e. zone 1 to zone 5. Basic properties of very soft-to-soft clay at each zone are summarized in Table 1. In general, the soft clay thickness varied from 6m to 14m and was gradually decreased toward the north direction.

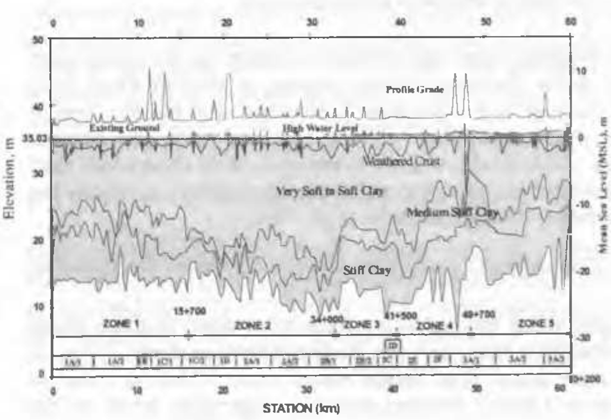

Figure 2 The soil profile within PVDs' installation area along BCNH project (Moh et. al. 1998)

Table 1 The soil properties of soft clay in BCNH project

Section (km)	w(%)	γ_t(t/m^3)	LL(%)	PL(%)
Zone 1(0+000~15+700)	60~120	1.4~1.5	70~140	30~40
Zone 2(15+700~34+700)	70~160	1.4~1.5	90~140	30~60
Zone 3(34+700~41+500)	70~120	1.4~1.5	70~120	40~60
Zone 4(41+500~49+700)	70~90	1.4~1.5	70~90	30~40
Zone 5(49+700~60+200)	70~120	1.4~1.5	70~100	30~50

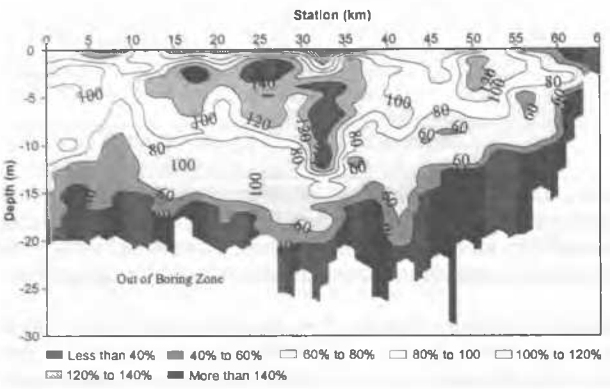

Figure 3 The natural water content profile within PVD's installation area along BCNH project

ANALYSIS AND DESIGN OF PVD EMBANKMENT

The design criteria of PVDs and preloading embankment in the BCNH are described in this section. The major objective is to reach at least 80% of primary consolidation degree before opening.

3.1 *The PVDs' Design*

PVD spacing was designed to be 1.2 m in triangular grid pattern with varied length at each contracted section, which was determined from the boring logs made in as initial phase of construction. In general, PVDs were installed for the areas encountered with more than 6m-thick soft clay but was limited to the maximum depth of 12m below the original ground to avoid the additional discharge effect from the underlying sand layer. Some parameters used in PVDs' design in the BCNH are summarized as follows:

(1) Thickness of soft clay layer = 8m~12m
(2) Total unit weight of soft clay = 1.5 t/m^3
(3) Average vane shear strength = 1.0 t/m^2
(4) Coefficient of compressibility Cc = 1.45
(5) Coefficient of horizontal consolidation c_h = 2.52 m^2/yr
(6) Coefficient of vertical consolidation c_v = 1.26 m^2/yr

3.2 *Thickness of the Sand Drainage Layer*

Sand drainage layer was designed to drain out the excess pore water during the consolidation process. A total of 45cm thick sand drainage layer was placed above the sand blanket layer, a 50cm thick sand-filled working platform over the original ground. Additional trenches traversing the side slope of the road embankment and pumping wells were also used to accelerate the groundwater dissipation (Moh et. al. 1998).

3.3 Preloading Construction & Waiting Period Determination

The preloading embankments were constructed in three stages with specified layers to reach the final height as shown in Fig. 4. The total actual final height varied from 2.55m to 3.0m as shown in Table 2. Waiting period is determined based on the increase of undrained shear strength as proposed by Stamatopoulos and Kotzias (1985). About one to three months was specified as waiting periods between the first, second and

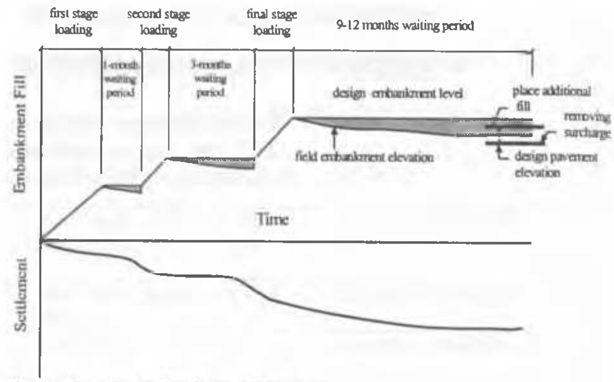

Figure 4 The preloading scheme

third stage loadings. The waiting period after the final stage loading was specified to be 9~12 months.

Two major safety criteria were specified during the loading stages: (1) the maximum settlement rate of 2 cm/month and (2) the maximum ratio of lateral movement to vertical settlement of 0.33. The final waiting period at some road sections had been extended for about one more year to satisfy the removing criteria due to excessive settlement and lateral movement. Additional fills were also required before reaching the final sub-grade elevation of +2.70 m. Total construction time of this road varied from 2 to 3 years, depending on the embankment performance.

4. PERFORMANCE OF THE PVD EMBANKMENT

4.1 *Surface Settlement*

Most preloading embankments of the BCNH were constructed on soft marine clay and many existing fishponds or canals were underneath or nearby along the project alignment. Large settlement was expected during the preloading period. The embankment performance was mainly monitored through instrumentation installed at control station of each contracted section. The vertical and horizontal movements are the major concern during embankment construction. In general, the subsoil conditions of zones 3, 4 and 5 are comparatively better than those of zones 1 and 2. Zone 2 is the most critical section where most of the embankments are situated on very thick layer of very soft-to-soft clay of 15-20 meters with low shear strength (0.5 t/m^2) and high water content (160%). It is, therefore, not surprising that the embankment within zone 2 area performed with the largest settlement and lateral movement accordingly.

The design and observed maximum settlement under the main embankment as well as the observed maximum lateral movement at control station (a comprehensive instrumentation area) of each zone are summarized in Table 2. In general, embankment has settled substantially during construction as expected. The variation of settlement is according to the variation of soft clay thickness.

A maximum of 200cm to 275cm settlement was recorded in zone 2, which is not completely resulted from the consolidation process and the involvement of lateral movement to the large settlement will be further discussed. Moderate settlement of 100cm to 180cm was recorded at other areas. If we look at the typical settlement trough as shown in Fig. 5, it is interesting to

Table 2 Observed maximum surface settlements and lateral movement

Zone	Actual preloading (m)	Design settlement (cm)	Observed max. settlement (cm)	Observed max. lateral movement (cm)
1	2.7~3.0	124~185	126~185	9.48~22.7
2	2.85~3.0	152~185	169~275	15.1~58.6
3	2.85	100~165	134~186	33.3~34.2
4	2.55~2.85	95~135	107~182	13.1~34.4
5	2.55~2.7	114~214	76~181	13.5~27.8

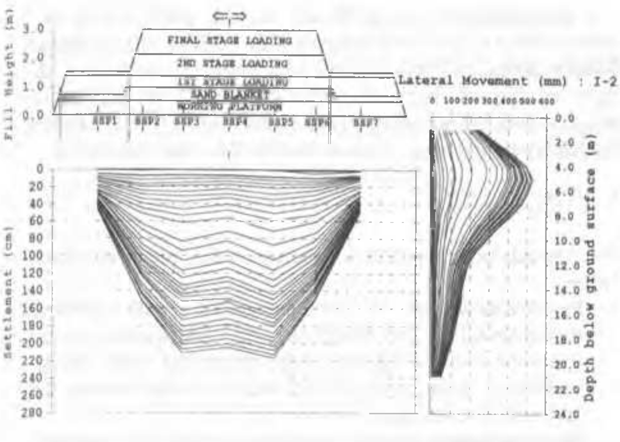

Figure 5 Settlement trough at Section 2A/2 (Sta. 28+200)

notice that the maximum settlement does not occur at the center of road embankment as per the theoretical calculation. The large lateral movement occurred at both sides may be the major reason for such settlement configuration.

In predicting the total settlement to determine the degree of primary consolidation, Asaoka's method is used on the basis of the measured time-settlement data (Asaoka, 1978). The observed maximum settlement at most sections has exceeded the estimated value in the design. This is probably, in one hand, due to the uncertainties involved in the prediction of total settlement and in the other hand, higher preload than the design because of leveling process during construction.

Except for Sections 2A/1, 2A/2 and 2B/1, the rate of settlement decreased continuously to a value of 2 cm/month for about one year prior to the preloading removal to sub-grade level at most sections. Therefore, surcharge-removing criteria of 2 cm/month seems to be a realistic criteria to be achieved. However, it took 14 to 17 months for Section 2A/1 and 2B/1 to remove the surcharge fill prior to the pavement construction. The relatively long waiting period could be due to the inadequacy of the thickness of the sand drainage layer.

Based on the limited survey data from DOH in August 1999, post-construction settlement varied from 30cm to 70cm were recorded. The continuation of large settlement even after removal of surcharge implies that excess pore pressure did not dissipate fully during preloading stage. The large difference of settlement between PVDs and non-PVD portions, which was inevitable, caused the sand blanket to be sheared off at the intersection of berm and main embankment and induced cracks due to differential settlement as observed at several sections. The insufficient preloading height at some areas where the embankment was lower than the sub-grade level at end of waiting period might also cause the increase of post-construction settlement.

The average settlements in OBRR project can be divided into two portions based on the settlement results as presented in Table 3. The portions (0~41+550.00km) with less soft clay thickness encountered average settlement of 370mm only where PVDs were installed at partial sections as well. The remaining portions (41+550km ~65+325.00km) with full PVD installation had average settlement of 920mm. In general, settlement in OBRR is less the observed in BCNH mainly due to the less soft clay thickness.

4.2 *Lateral Movement*

As shown in Table 2, the maximum lateral movement of 52.88cm and 58.5cm occurred at Sections 2A/2 and 2B/1 of zone 2, respectively. The maximum lateral movement at other sections varied from 10cm to 35cm. Lateral deformation at Section 2A/1 is unexpectedly low compared to Sections 2A/2 and 2B/1 under similar sub-soil conditions. In general, the maximum lateral movement occurred at 3.0m~5.0m depth below the original ground surface depending on the soft clay thickness (10.0m~15.0m). Therefore, location of maximum lateral movement is around one-third of the soft clay thickness.

Shear induced displacement (or plastic deformation), particularly beneath the side slopes of the embankment, results in an expulsion of soft clay underneath and increases the amount of total measured settlement on the base of the embankment. In particular, this phenomenon may become serious in low-lying areas having thick soft clay and subsoil consists of high water content. As mentioned before, the lateral movement has some contribution to the total field settlement data. In respect of settlement and lateral movement areas, the proportion of lateral movement to settlement at each zone is summarized in Table 4. The lateral movement area was calculated from the plot of maximum lateral movement vs. depth multiplied by 2 (for both sides). The high ratio of 12.09 % and 11.22 % at Sections 2A/2 and 2B/1, respectively, may possibly prove that there is a substantial contribution of lateral movement to vertical settlement at these sections. In other sections, a low to moderate ratio of lateral to vertical movement areas indicates low or negligible contribution of lateral movement.

Table 3 The average settlement under ground improvement area along OBRR (Eastern Portion) project (Suzuki 2000)

Contracted Sections	Station, km	Surface settlement, mm
1A/1 ~ 2-C/2	0 ~ 41+550	220 ~ 420
3-A ~ 3G	41+550 ~ 65+325	800 ~ 1,570

Table 4 The average ratio of lateral to vertical movement at each zone

zone	settlement area Av (m^2)	lateral movement area Ah (m^2)	Av/Ah (%)
1	55.61[1] (42.80~72.68)[2]	2.52 (1.80~3.50)	4.68 (3.14~6.27)
2	80.88 (57.46~101.10)	6.23 (2.56~11.34)	7.35 (3.03~12.09)
3	63.42 (54.75~68.80)	4.62 (3.80~5.16)	7.41 (5.70~9.42)
4	49.09 (42.00~61.00)	3.64 (2.50~5.72)	7.14 (5.95~9.38)
5	59.01 (30.56~79.95)	2.82 (1.52~3.94)	4.80 (4.51~4.97)

1 – the average value; 2 – the min. and max. value at each zone

4.3 *The Change of Soil Properties*

The comparison of natural water content and undrained shear strength before and after ground improvement at control station of Sections 1A/2 and 3A/1 are summarized in Figs. 6. The soil data was obtained from the bored samples in 1994 (before PVD installation) and 1998 (one year after open to traffic). A significant decrease of natural water content and increase of undrained shear strength have been observed from the upper 10 meters of soft clay layers after the ground improvement for about 1 year. Additional field vane shear tests were also performed at same location in 1999, two years after the highway open to traffic. The undrained shear strength did not change very much as comparing with the data in 1998. The change of soil properties tends to indicate the effectiveness of the PVDs with preloading to improve the soft Bangkok Clay.

4.4 *Coefficient of Horizontal Consolidation (c_h)*

The average back-calculated coefficients of horizontal consolidation (c_h) under different loading stages in each zone are summarized in Table 5. The c_h values were calculated from time-settlement data and Asaoka's approach of the radial consolidation (Asaoka, 1978) with the following formula:

$$c_h = \frac{(1-\beta)\times D_e^2 \times F}{8\times\beta\times\Delta t} \quad (1)$$

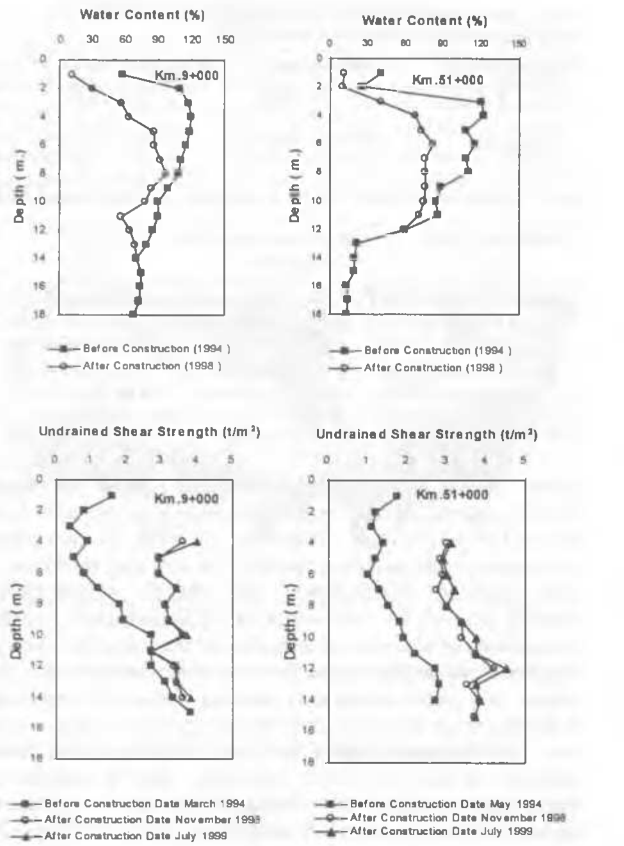

Figure 6 The change of natural water content and undrained shear strength before and after road embankment construction in BCNH project

Table 5 Back-calculated horizontal coefficient of consolidation, c_h

Zone	Loading stage	Fill height, m	c_h, (m^2/year)
1	1st stage loading	1.35	2.20
	2nd stage loading	2.10	1.20
	3rd stage loading	2.78	0.80
2	1st stage loading	1.35	2.50
	2nd stage loading	2.01	1.52
	3rd stage loading	2.82	0.86
3	1st stage loading	1.35	2.40
	2nd stage loading	2.10	2.00
	3rd stage loading	2.80	0.87
4	1st stage loading	1.35	2.80
	2nd stage loading	2.03	1.50
	3rd stage loading	2.65	0.87
5	1st stage loading	1.35	2.67
	2nd stage loading	2.00	1.40
	3rd stage loading	2.70	0.87

Note: c_h value of 2.52 m^2/year was used in the design.

Where D_e = equivalent diameter of a unit drain influence zone; ß = slope of the settlement-time plot; Δt = time interval in settlement-time plot; F = factor considering the effect of on smear, spacing and well resistance and can be obtained from settlement plot.

The c_h in here presents the average values of whole soil depth and has a tendency of decreasing value as loading increases. The c_h under first stage loading is close to the one used in the design or 2.52 m^2/year and decreases to 1.5 and 0.8 m^2/year for the second and final stages loading, respectively. The low value of c_h indicates that the subsoil has undergone substantial amount of consolidation as well as the prolonged waiting period.

Since settlement data at various depth of soft clay layer are not available in BCNH project, the c_h values obtained from only surface settlement plates might be more or less deviated from the most probable value. Based on the performance of test embankment (1.2m PVD spacing with 4.2m surcharge height) at Second Bangkok New International Airport constructed in the same vicinity, the back-calculated c_h at depth 6m and 8m is 0.9 m^2/year and 0.8 m^2/year, respectively (ADG 1995). Therefore the back-calculated c_h value in the BCNH looks reasonable.

5. CONCLUSION

The conclusions from the discussion in this paper are drawn as below:

1. Performance of PVDs associated with preloading embankment on soft Bangkok Clay is satisfactory as can be observed from settlement data associated with change of undrained shear strength and natural water content before and after construction.
2. Field settlement at some sections in the BCNH is about 1.5 times higher than the design value due to the longer waiting period and higher surcharge load. In general, settlement in the OBRR under PVDs portion is less than the data in the BCNH.
3. Maximum settlement occurred at both sides of road embankment in zone 2 with the involvement of large lateral movement.
4. The average back-calculated coefficient of horizontal consolidation (c_h) could be drawn as:
   - 2.5 m^2/year under 1st stage loading
   - 1.5 m^2/year under 2nd stage loading
   - 0.8 m^2/year under 3rd stage loading
5. The total construction time and post-construction settlement could be reduced if the PVD embankment is designed properly (ex. sufficient preloading height and drainage system, etc.).

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