IMPROVEMENT OF SOFT BANGKOK CLAY BY USE OF PREFABRICATED VERTICAL DRAIN

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SYNOPSIS This paper reports the use of PVD and preloading for ground improvement of an 80km long new major expressway construction in Bangkok. More than 22 million linear meters of PVD was used to accelerate consolidation settlement of the very soft Bangkok Clay. Under an embankment height of 2.5 to 3.0m, maximum settlement after a 9 to 18 months waiting period reached as large as 2.60m. Construction of the ground improvement scheme was carefully controlled through the use of instrumentation monitoring. One of the most important conclusions for a successful ground improvement work using PVD is adequate and proper design of underainage system.

INTRODUCTION

Traffic congestion is always a major problem in Bangkok and the outskirts. During recent years, industrial activity and associated urbanization in the east and southeast of Bangkok have been growing rapidly. Two ports at Laem Chabang and Map Ta Pud and the future Second International Airport at Nong Ngu Hao will also increase the traffic volume as expected. In order to solve the shortage in both capacity and carriageways of regional network, the Department of Highways(DOH) decided to construct a new express highway (Bangkok-Chonburi New Highway) with fully-controlled access facility including grade separated interchanges and fly-over bridges to relieve the traffic congestion.

Almost all of this project route, except about 15km at the end, is traversing on the flat central plain of Thailand where the foundation soils consist mainly of soft marine clay up to depths of 15 to 20m. The underlying thick soft soils are expected to be the cause of severe damages to the structure as can be seen at the existing Highway No. 3 and No 34. Thus the details design has introduced Prefabricated Vertical Drain(PVD) with preloading embankment to accelerate the settlement rate during the construction period. The design's objective is to achieve at least 80% of primarily consolidation settlement within 12 months of waiting period prior to pavement construction. The designed embankment elevation is to be above the maximum water level during the 15 years of service life.

The whole highway was divided into twenty-one contract sections. Thirteen of which involve roadwork's construction and the remaining eight contract sections are interchanges. Since the depth of soft clay gradually decreases near the Chonburi area, PVD was not used on the project between Sta. 47+675 and Sta. 49+700 of section 3A/1 and after Sta. 60+000. Part of roadway at Section 1B was improved by use of cement column method due to delay in contract administration and is not discussed in this paper. The project route layout is shown in Fig. 1.

The total construction cost is 12,750 million Baht (about 500 million US dollars), of which 50% comes from the government budget and the other 50% comes from loans of Overseas Economic Cooperation Fund (OECF) of Japan. Construction of the first section of the New Highway was started in early June 1994 and the expected time to complete the last section will be in August 1998. Part of the highway after Section 1-D had been completed and opened to traffic since February 1998.

Detailed design of the Highway was carried out by a consortium led by Thai Engineering Consultants Co. (TEC). A team of consultants comprising of Katahira Engineers International (KEI), Moh and Associates Inc. (MAA), TEC, and Upham International was appointed to be the

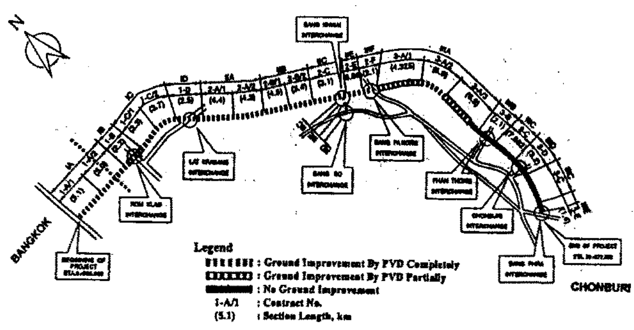

Fig. 1 The Route Plan of Bangkok – Chonburi New Highway

construction supervisor with MAA being responsible for the ground improvement and all geotechnically related work.

Due to limitation of spaces available, this paper only discusses problems encountered during construction and measures taken to ensure safety against excessive settlement and stability. More detailed discussion and evaluation of monitored data are to be presented elsewhere.

SUBSOIL CONDITIONS

As interpreted from a total of 161 boreholes with 400 to 600m intervals along the improved portion of the project route, four different soil strata are identified within the depth of 25m (i.e. end of boring) below the original ground surface. The general properties of the four soil strata are summarized in Table 1.

Table 1 Simplified Soil Strata

Soil Stratum	Soil Type	Thickness m	Color	γ_t t/m^3	W_n %	W_L %	W_p %
Weathered Crust	CH	1-2	brown	1.6-1.8	40-60	60-80	25-30
Very Soft to Soft Clay	CH	6-15	dark gray	1.4-1.5	60-120	70-140	30-60
Medium to Stiff Clay	CH	2-4	gray	1.6-1.7	40-60	40-90	30-40
Stiff to Hard Clay	CH-CL	2$^+$	light brown	1.8-2.0	20-30	30-40	20

The generalized soil profile and field vane shear strength along the ground-improved portion of the New Highway is shown in Figs. 2 & 3, respectively. According to the similarity of subsoil properties, the whole soil profile along the route has been divided into five zones for ground improvement work.

PREFABRICATED VERTICAL DRAIN

The strengthening and preconsolidation of weak and compressible soils by preloading prior to construction is one of the oldest and most widely used method for soil improvement. It is particularly well suited for use with soils that undergo large volume decreases and strength increases under sustained static loads and when there is sufficient time available for the required compression to occur. For soils such as the Bangkok Clay for which compression is dominated by primary consolidation, vertical drains can provide an additional drainage path in horizontal direction to accelerate the dissipation of ground water.

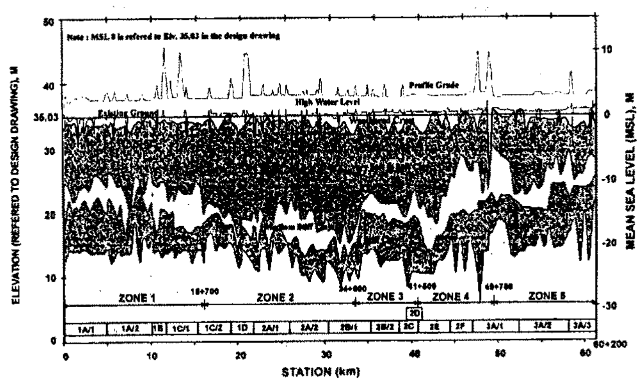

Fig2. Soil Profile Along The PVD Portion of Bangkok – Chonburi New Highway Project

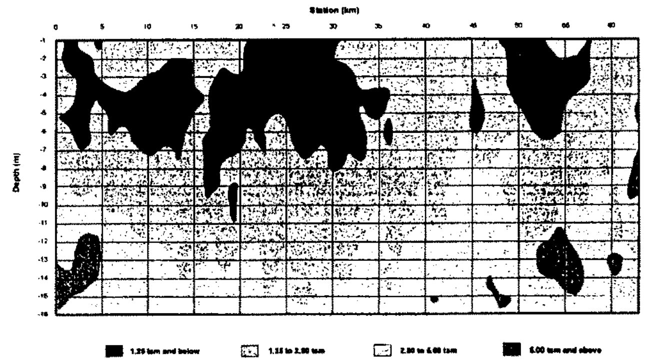

Fig. 3 Field Vane Shear Strength Profile

According to available case records, sand drains or PVD were used mostly in trial stage in Bangkok area. The Bangkok-Chonburi New Highway is the first large highway project that uses PVD to accelerate the consolidation settlement of Bangkok Clay. The performance and experience will be an important reference for future projects of similar nature.

The parameters used in the design of soil improvement work with PVD on this project are summarized as below:

Embankment fill thickness : 2.7m
Counterweight berm thickness : 1.8m
Thickness of soft clay layer : 8-12m
Total unit weight of soft clay : 1.55t/m^3
Average vane shear strength : 1.0t/m^2
Coefficient of compressibility, C_c : 1.45
Coefficient of horizontal consolidation, c_h : 8.0×10^{-4} cm^2/sec
Coefficient of vertical consolidation, c_v : 4.0×10^{-4} cm^2/sec
Time for 70% degree of consolidation : 5 months
Time for 95% degree of consolidation : 9.5 months
PVD length : 10-14m
PVD spacing : 1.2m triangular grid
Thickness of sand drainage layer : 50 cm

The typical cross section of the New Highway with PVD system is shown in Fig. 4. Due to the existence of under-pressure of groundwater in the stiff clay and the

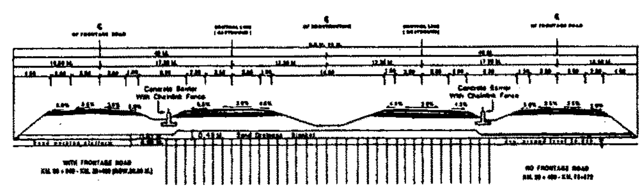

Fig. 4 Typical Cross – Section of Roadway

underlying sand layer, PVD were installed in the very soft to soft clay layer with the installed length less than 12 m during the construction to eliminate the discharge effect. As concluded by Moh and Woo (1987), sand drains penetrating to the stiff clay and underlying sand layer will cause additional settlement in the soft clay by providing paths of water flow. The actual PVD installed length varied from 7m to 12m depending upon the subsoil condition. Four PVD brands including Mebra MD7007, Colbond CX-1000, Amerdrain 607 and Flodrain FD4-457A with a total amount of 23.5million linear meter had been installed in this project..

During PVD installation, samples were collected and tested according to the specification. The procedure of quality control during PVD installation is illustrated in Fig. 5. The required test items with related criteria and averaged test results of the four brands are summarized in Table 2.

Table 2 Average Test Results of PVD Material

Brand	Test Results					
	Apparent Opening Size (AOS) µm	Tensile Strength kN	Trapezoidal Tear Strength kN	Puncture Strength kN	Burst Strength kPa	Discharge Capacity, 7 Days 2.0 ksc @ HG=1, m^3/yr
Colbond CX-100, (51 Tests)	75	0.85	0.17	0.27	1,332	1,440
Mebra MD-7007, (89 Tests)	75	0.92	0.18	0.28	1,237	2,180
Flordrain FD4-457, (2 Tests)	75	0.81	0.23	0.29	1,321	1,228
Amedrain 607, (5 Tests)	75	1.29	0.49	0.37	1,382	1,527
Accepted Value	<90	>0.35	>0.10	>0.20	>900	>500

CONSTRUCTION PROCEDURE

Since most of the project alignment run through shrimp farms or agricultural fields, 50cm thick sand was first placed after clearing, grubbing or scarifying the existing surface to provide a firm working platform for heavy equipment. Sand blanket was then constructed above the working platform to drain out the excess groundwater due to the installation of PVD and pre-compression. The designed thickness of sand drainage blanket is 45cm under the main preloading embankment and 20cm under the counterweight berm with 1% gradient to both sides.In general, preloading embankment was constructed in 3 stages with one to three month(s) waiting period between first, second and third stage loading to reach final designed height of 2.55 to 3.0m. After completion of the final stage loading, the conditions for the termination of waiting period is specified as:

(1) Nine months after completion of surcharge if measured in-situ settlement is greater than 80% of the total calculated settlement; or

(2) Twelve months after completion of surcharge

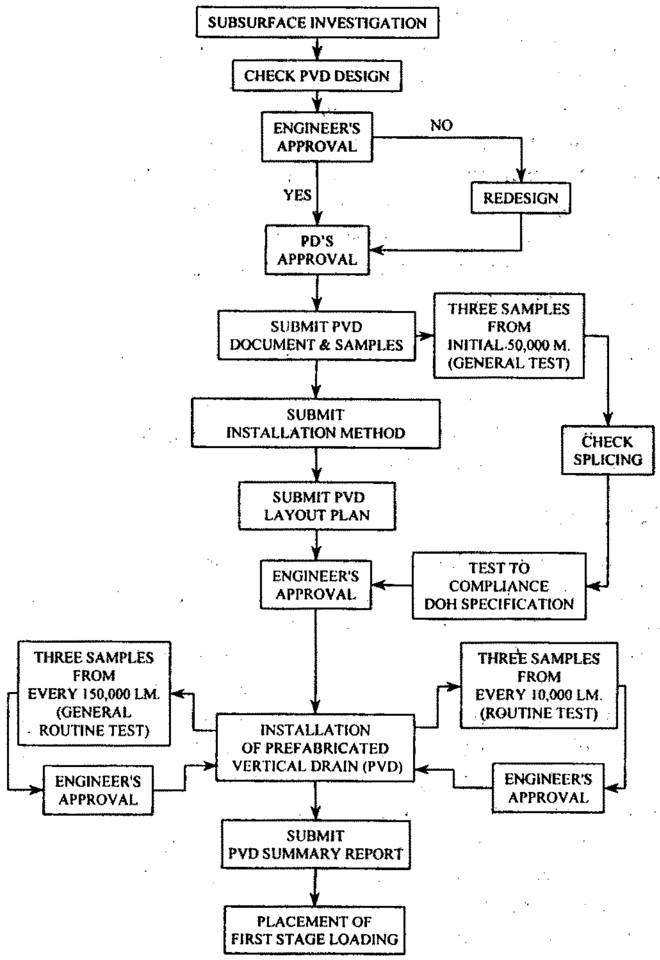

Fig. 5 Flow Chart of Quality Control in PVD Installation

In addition to the above criteria, a maximum monthly settlement rate of 2cm was decided as an additional requirement to reduce maintenance cost and to ensure embankment safety after evaluation of field settlement data.

INSTRUMENTATION AND MONITORING

Stability and settlement control of embankment on soft clay deposits are the main considerations in the design of this project. Geotechnical instrumentation scheme for ground improvement work is employed to ensure safe and economical construction of the embankment. During construction of the Bangkok-Chonburi New Highway, instrumentation has played an important role in the safety control. There are six types of instruments including surface settlement plates, deep settlement plates, pneumatic piezometer, alignment stakes, inclinometers and observation wells in the instrumentation program. However, focus of data interpretation was made mainly on vertical/horizontal deformation and pore pressure changes.

Due to limit of budget, each contract section (about three to five kilometer long) had only one control station which had five to seven surface settlement plates, one deep settlement plate, nine piezometers, four alignment stakes, three inclinometers and one observation well. A typical instrumentation profile is shown in Fig. 6.

In general, monitoring was carried out before and after each change in loading and seven days thereafter until the beginning of asphalt pavement construction. However, the frequency is related to construction activity, to the rate at which the readings are changing, and to the requirements of data interpretation.

PERFORMANCE OF GROUND IMPROVEMENT

Evaluation of the ground improvement performance was made primarily on the basis of settlement data and changes in soil properties.

Settlement

Settlement values at sections of Zone 2 (15+700km to 34+000km) have the highest value among others as corresponding to the subsoil condition. Settlement of more than 250cm were recorded at sections 2A/1, 2A/2 and 2B/1. The observed maximum settlement profiles under the main embankment and counterweight berm with related fill height and soft clay thickness along the entire project route is shown in Fig. 7. Figures 8 and 9 show typical results of settlement and lateral movement of the foundation soil. The maximum settlement occurred at or near the center of embankment. However, as the settlement data are taken from the surface settlement plates, the large lateral movement should have some contribution to the results.

Since the field settlement data is much higher than designed total settlement, Asaoka's method(1978) was adopted to predict the ultimate consolidation settlement by use of monitoring data in order to determine the 80% degree of consolidation for the consideration of surcharge removing. A minimum one-month interval of settlement data has been selected for all sections in predication of the final settlement. Table 3 shows the comparison of designed total settlement, observed maximum settlement

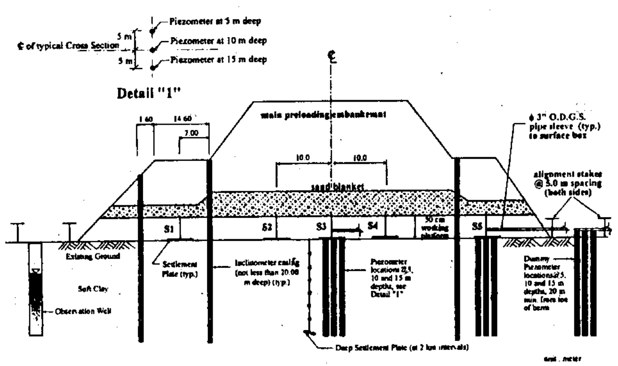

Fig. 6 Typical Layout of Instrumentation

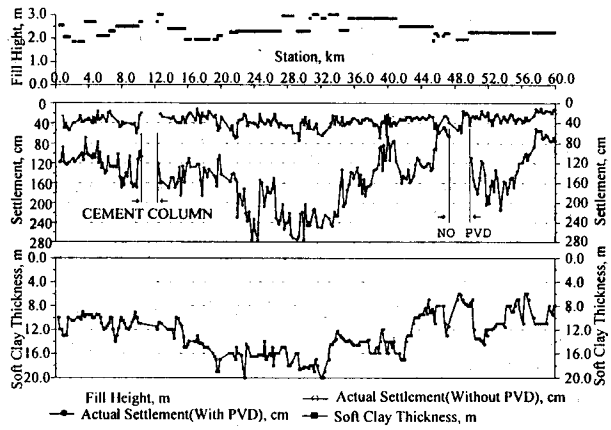

Fig. 7 Settlement Profile with Fill Hight and Soft Clay Thickness (Up To Sept, '97)

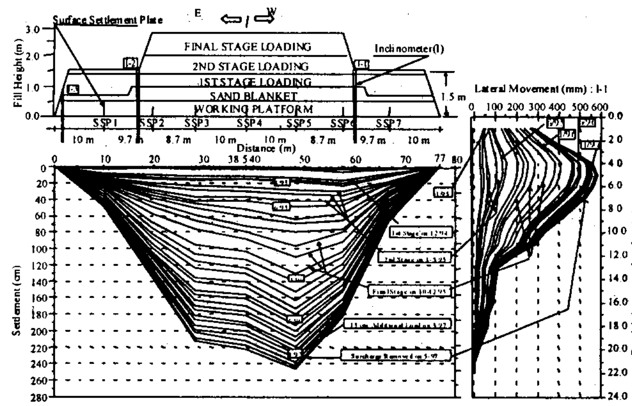

Fig. 8 Observed Settlement and Lateral Movement at Section 2 – B/1, Sta. 32+510

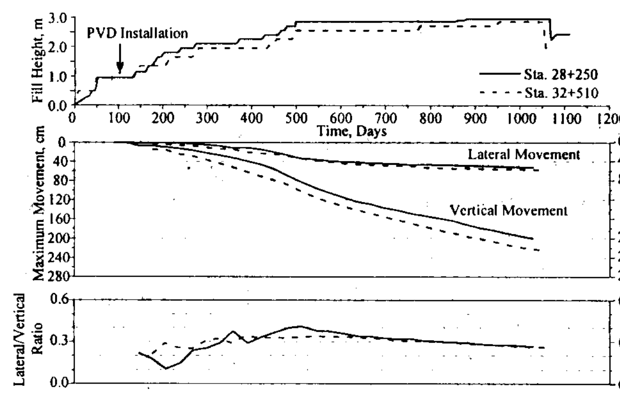

Fig. 9 Observed vertical and LATERAL Movement Versus Time with Fill Height at Sta. 28+250 and Sta. 32+510

and the predicted total settlement by Asaoka's method. It appears that the actual settlement at most sections are much larger than the designed settlement. Of course, the additional fill due to leveling process and longer waiting period have also contributed to the higher settlement.

Table 3 Comparison of Designed, Esimated and Actual Settlement

SECTION	STATION	WAITING PERIOD AFTER FINAL STAGE		DESIGN SETTLEMENT M.	OBSERVED MAXIMUM SETTLEMENT CM.	ESTIMATED TOTAL SETTLEMENT CM.
		PLANNED	ACTUAL			
		MONTH	MONTH			
1-A/1	0+000-5+100	9 - 12	under construction	1.24 - 1.5	122.50	160.0
1-A/2	5+100-10+100		12 - 14		168.00	178.0
1-B	10+100-12+400		under construction	1.4 - 1.6	106.70	130.0
1-C/1	12+400-15+700		10 - 13	1.52 - 1.85	185.00	190.0
1-C/2	15+700-19+600		12 - 14		173.70	210.0
1-D	19+600-22+000		9 - 11	1.52 - 1.69	169.20	190.0
2-A/1	22+000-26+400		14 - 15	1.70 - 1.83	275.40	285.0
2-A/2	26+400-30+700		15 - 17		272.70	290.0
2-B/1	30+700-35+200		15 - 17	1.55 - 1.65	260.00	300.0
2-B/2	35+200-38+600		12 - 15		186.10	230.0
2-C	38+600-41+500		9 - 13	1.00 - 1.17	134.20	160.0
2-D	0+000-1+000		12 - 13	1.18 - 1.26	166.10	170.0
2-E	41+500-45+450 (MIAN ROAD		9 - 13	0.95 - 1.35	160.20	165.0
	1+000-2+880 (ACCESS ROAD)				182.30	195.0
2-F	45+450-47+675		10 - 11	0.96 - 1.06	106.70	110.0
3-A/1	47+675-52+000		12 - 13	1.14 - 2.14	181.50	210.0
3-A/2	52+000-57+600		10 - 11		201.50	215.0
3-A/3	57+600-64+500		9		76.30	80.0

Lateral movement

In the design stage, the calculated total settlement consisted of consolidation settlement only, without considering the effect of shear strains caused by lateral displacement. Ladd (1991) has reported that the ratio of maximum lateral movement to maximum settlement for embankment fill on normally consolidated clay might be as high as 0.9 ± 0.2 in initial stage of loading and drops to 0.16 ± 0.07 when consolidation settlement predominates. Large lateral movement usually signifies potential shear failure. In this project, a value of 0.33 was adoped as the criteria for stability control. However, special attention was given to the control of construction rate when the ratio reached 0.25. It is interesting to note for sections 2-A/2 and 2-B/1 where the soft clay thickness is largest and the site is immediately surrounded by shrimp farms and water khlongs, this ratio was highest and reduced to about 0.2 during the final waiting period of the final stage load as shown in Fig. 10.

Soil Properties

In general, settlement should cause densification of soft soils. During preloading, the water content, void ratio and coefficient of premeability should decrease and the undrained shear strength, modulus of compressibility and penetration should increase.

Comparison of natural water content before and after ground improvement according to soil data obtained from additional soil investigation during final waiting period are illustrated in Fig. 11. It can be seen that the water content has decreased significantly within the soft clay strata. However, where the original water content of soft clay was less than 80%, the difference was not so large. For estimation purpose, Stamatopoulos & Kotzias (1985) has proposed the following equation to calculate the decrease of water content:

$$w_n = -(w_n + 1/G) \times (\delta / H) \qquad \ldots\ldots(1)$$

where w_n = natural water content,

G = specific gravity of soils,

δ =settlement, H=thickness of compressible soils

The change in water content determined from the above equation is also compared with the direct measurements as shown in Fig. 11. In consistent with decrease in water content, undrained shear strengh increased substantially due to effect of ground improvement. Stamatopoulos & Kotzias (1985) also proposed the following equation to calculate the change in undrained shear strength:

$$\Delta s_u = (G/0.434 \times C_c) \times s_u \times \Delta w_n \qquad \ldots\ldots(2)$$

where Δw_n =change in water content,

G=specific gravity of soils

s_u=undrained shear strength,

C_c=compression index

Comparison of undrained shear strength obtained from field vane shear tests and calculation from the above equation is illustrated in Fig. 12.

PROBLEMS DURING EMBANKMENT CONSTRUCTION

Inadequate Underdainage

The original design of the embankment construction with preloading and PVD incorporated only a sand blanket for underdrainage. The design thickness of the sand blanket was 45cm under the main preloading embankment and 20cm under the counterweight berm with 1.0% gradient to both sides. There were no PVD designed under the

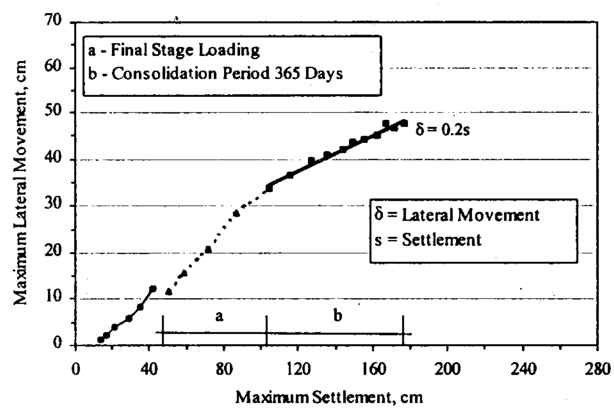

Fig. 10 Vertical Settlement vs Lateral movement Section 2-A/2

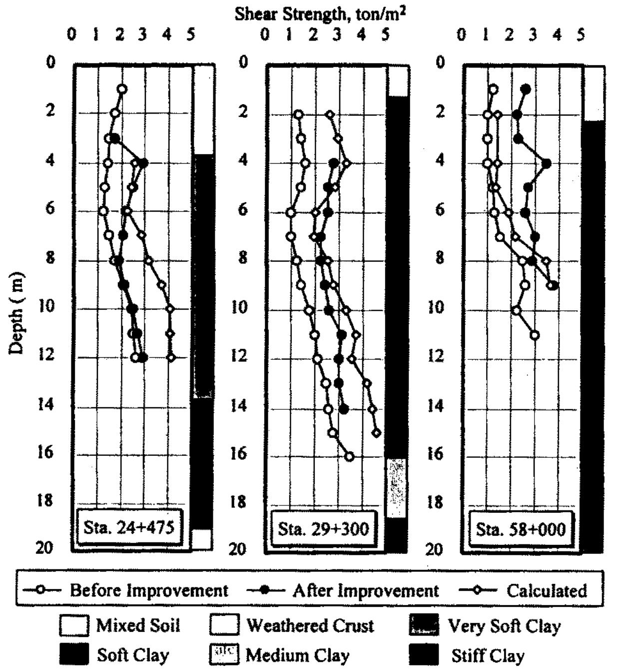

Fig. 11 Comparison of Water Content Before and After ground Improvement at Sta. 24+475, 29+300 and 42+750

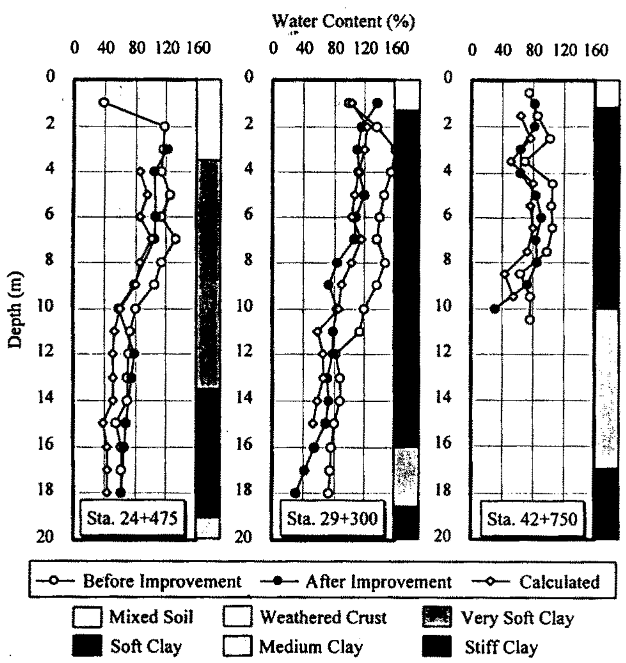

Fig. 12 Comparison of Undrained Shear Strength Before and After Ground Improvement at Sta. 24+475, 29+300 and 58+000

berm sections. The inadequacy of proper underdrainage was noticed and discussed at the beginning of the construction supervision review. However, due to budget constraint at the time, modification of the design was delayed. Problem of inadequate underdrainage was revealed by high excess porewater pressure, extremely slow rate of dissipation as monitored after the second stage of loading, change order of design by installing additional trenches and pumping wells to facilitate the out flow of water due to compression of the soft clay under embankment loading was undertaken. Figure 13 shows schematic design of the additional drainage facilities. Immediately after construction of these measures, large amount of water flowed out, the effect was obvious. However, the efficiency of this remedial measures was not as good as if the original design had the proper underdrainage facilities incorporated. Several minor failures did occur in sections 2-B/1 before installation of the additional drainage measures.

Another problem of poor drainage was caused by the covering of side slopes of preloading embankment with top soils for preventing erosion of the sandy slope during rainy season. Only after the contractors were advised to remove top soils from the lower part of the side slope to clear the drainage outlets, slippage of slopes was stopped.

Cracking of Embankment

Two types of cracking of embankment were observed:

(1) Cracking between the main embankment and berm – Since the design did not place any PVD underneath the counterweight berm, relatively much smaller settlement occurred under the berm as comparing to the main embankment with PVD, Fig 7, cracks developed along the boundaries between the PVD improved and the non-PVD part due to the large differential settlement. In fact, the lack of PVD under the berm section actually impedes the efficiency of PVD for accelerating consolidation of the soft clay under the main embankment due to longer flow path of water from the PVD to drainage outlet. The different thickness of the sand drainage blanket also contributed to the problem of inefficiency. In a number of locations, the sand blanket was sheared off between the PVD and non-PVD sections.

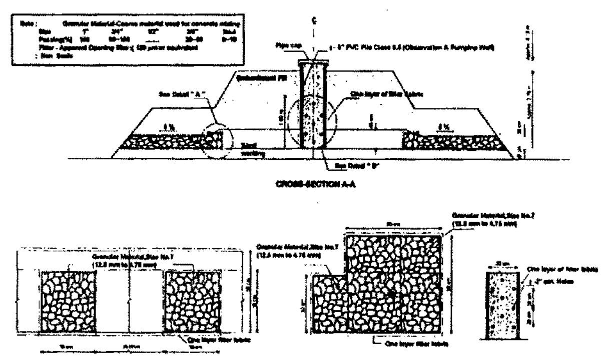

Fig. 13 Typical Cross Section of Additional Trench

(2) Cracking along the centerline of main embankment – occurrence of cracking along centerline of the main embankment in general is a warning to potential stability failure. Actions such as reducing preloading height, extending counterweight berm and improving underdrainage for relieving excess porewater pressure were taken to increase the stability.

CONCLUSIONS

Based on observations of the embankment performance during construction of the 80km long Bangkok-Chonburi New Highway and analysis of the results, the following conclusions are made:

1. The use of PVD and surcharge to accelerate settlement rate of Bangkok Clay is satisfactory as can be observed from the settlement data. Lateral movement may have some contribution to the large vertical deformation in thick soft clays.
2. The water content of the very soft clay has been greatly reduced and the undrained shear strength increased due to the use of PVD and preloading.
3. The design thickness of sand blanket has been proved to be inadequate. Sand blanket had been sheared off at the intersection of main embankment and berm as settlement occurred. The inefficient drainage facility is the main cause for some embankment failure.
4. Monitoring data can be a very important indication of embankment safety. Lateral to vertical movement ratio can be useful if a proper value is selected. For the project reported, a ratio of 0.25 to 0.33 has been adopted which appeared to be an effective control criterion.
5. Cracking induced by differential settlement and insufficient stability is the most common problems for embankment constructed on soft clay.
6. Efficiency of PVD with surcharge preloading could be greatly improved if proper underdrainage system was incorporated.

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