CASE STUDY

CONSTRUCTION OF HIGH ROAD EMBANKMENT OVER THICK SOFT SILTY CLAY- A CASE STUDY

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ABSTRACT

Calicut Bypass Phase-II project included construction of 7 m high embankment over soft compressible clays up to 18 m in thickness. Approx 1.5 km length of road out of a total 5 km length consisted of abandoned paddy fields which were low lying, waterlogged and marshy in nature. On the basis of excellent results achieved in earlier phases of the by-pass construction, ground improvement work by installing pre-fabricated vertical drains (PVD) was undertaken to stabilize the embankment foundation along the weak clay areas prior to road pavement construction to avoid large post construction settlement as well as to improve the lateral stability during and after embankment construction. This project perhaps stands out as one of the highway projects completed within a very short time period of total 18 months including four month long heavy monsoon period, when embankment construction was not possible and construction of two bridges across rivers. All activities including the ground improvement had to be programmed and completed within the shortest time period possible. The paper discusses details of the extensive ground improvement and soil instrumentation carried out for the project and will illustrate how careful planning and execution can reduce the period required for highway projects involving ground improvement.

Keywords: clay, PVD, ground improvement, embankment, settlement

INTRODUCTION

Phase-II construction of Calicut by-pass was undertaken on a priority basis by the State of Kerala in 2014. The main contract was awarded to M/S ULCCS Ltd, Kozhikode, Kerala and the ground improvement work was awarded to M/S Geo-Enviro Engineers P Ltd, as specialist agency to design and execute the work.

Soil investigation carried out confirmed thick soft clay deposits below GL in three areas along the road alignment totaling approx 1.5 km in length. These areas were low lying, waterlogged and marshy in nature. Highway embankment had to be constructed up to 7 m in height above existing GL to form the proposed road alignment along these areas. On the basis of excellent results achieved in earlier phases of the by-pass construction using pre-fabricated vertical drains or PVD (R. Radhakrishnan, 2006), ground improvement using PVD had been included in the contract to stabilize the embankment foundation along the weak clay areas prior to highway embankment construction to avoid large post construction settlement as well as to improve lateral stability during its construction.

This project stands out as one of the highway projects completed possibly within the shortest time period requiring ground improvement. The entire work including embankment construction over the entire stretch and two bridges across rivers was completed within a period of 18 months with a 4 month long heavy monsoon period, well ahead of the 24 month completion stipulated by the client. All activities including ground improvement had to be programmed and completed within the shortest time period possible. By careful planning no additional surcharge load was required to shorten the consolidation period.

SUB-SOIL CONDITION

Soil investigation established soft clay existing at site was variable in thickness from about 5 m to a maximum depth of 18 m at certain locations. The soft clay was classified as highly compressible CH soil, with high plasticity and natural moisture content. Typical subsoil details from laboratory tests are included in “Table 1”. As the soft clay was weak in shear strength, it was necessary to ensure adequate embankment stability during all stages of embankment construction. Stability analyses confirmed that the 7 m high embankment will have to be constructed in three or four stages with waiting periods between stages for clay consolidation to take place under each stage of filling.

Table 1. Typical Soil Parameters

Description	Range	Average Values
Bulk Density, kN/m^3	15-19	17
Liquid Limit, %	32-60	50
Plastic limit, %	26-41	30
Natural Moisture Content, %	28-52	40
Cohesion, kN/m^2	14-22	18
Compression Index	-	0.6
Co efficient of Vertical Consolidation, m^2/year	-	1.0
Initial Void Ratio, e0	-	1.5

PVD DESIGN

Barron (1948) first proposed the general case for soil consolidation using vertical drains. Hansbo (1979) modified Barron's proposal for application with prefabricated vertical drains (PVD) and also considered soil disturbance due to PVD installation. However, as the disturbance factors are difficult to evaluate practically, only the ideal case without considering soil disturbance was employed in the PVD design for this case.

As the period available for the entire by-pass construction was very short, time available for soil consolidation and embankment construction were extremely limited. Since Kerala experiences heavy rains during the south- west monsoon period, earthwork for embankment construction was not possible during the period June to September. This monsoon period was approximately ten months after work started at site. This meant that the entire ground improvement work as well as most of embankment construction had to take place within this ten month period. In order to shorten the waiting periods for clay consolidation between construction stages as well as to keep overall completion as short as possible, the PVD spacing adopted for the project was 1.0m c/c triangular pattern.

The PVD selected for the project was 100 mm wide and 3.5 mm thick with high discharge capacity as shown in "Table 2". Based on design soil parameters, predicted final primary consolidation settlement under full embankment load could vary from 600 mm to 1300 mm depending on final embankment height and clay thickness at different locations. As the maximum expected settlement for thick clay areas was high, it was important to ensure that the PVD discharge capacity in the kinked condition was adequate for the purpose.

Table 2. Technical specification of PVD Tencate HB 63

Properties	Test Standard	Unit	Value
Composite			
Discharge capacity-straight (240kPa), i=1	ASTM D4716	$10^{-6} m^3/s$	75
Discharge capacity-kinked (240kPa), i=1	ASTM D4716	$10^{-6} m^3/s$	60
Tensile strength (full width test)	ASTM D4595	N	2500
Elongation at break	ASTM D4595	%	20
Core compressive strength	ASTM D1621	kPa	475
Filter			
Grab strength (MD)	ASTM D4632	N	310
Trapezoidal tear	ASTM D4533	N	70

Puncture resistance	ASTM D4833	N	80
Apparent opening size	ASTM D4751	μm	90
Water permittivity	ASTM D4491	s^{-1}	0.75
Coefficient of permeability	ASTM D4491	10^{-4} m/s	1.80
Physical			
Nominal width	NA	Mm	100
Nominal thickness	ASTM D5199	Mm	3.50

PVD INSTALLATION

PVD was installed to the full depth of soft clay using hydraulic PVD stitchers. Hydraulic stitchers install PVD at a constant rate of penetration by static force pushing a steel mandrel without jerk or vibration during installation thus reducing disturbance to surrounding soil by the mandrel. The steel mandrel used was rectangular in cross section and had cross sectional area not exceeding 65 sq.cm as recommended, to ensure minimum disturbance to surrounding soil (FHWA,1986). A disposable anchor plate was attached to the PVD end which closed the mandrel bottom as well as anchored the installed PVD at the desired depth after penetrating the soft clay.

A heavy duty hydraulic excavator was used as base machine to power the stitcher unit for PVD installation. A photo of the PVD stitcher used is included in "Fig. 1". The maximum depth of PVD penetration was not expected to exceed 20 m. In a good working day, it was possible to install PVD at more than 400 locations in a single shift. Due to the urgency to complete the PVD installation, work continued in two shifts per day. The PVD installation depths were closely monitored with the help of soil investigation data to ensure full penetration in soft clay stratum.

Fig. 1. PVD Installation

GEOTEXTILE SEPARATOR CUM BASAL REINFORCEMENT

The areas of ground improvement were water logged and the clay layers close to ground level were very soft in nature. Initial soil filling approx. 0.7 m thick was necessary to provide a safe and stable work platform for heavy machinery to operate under such site condition.

In order to avoid mixing of embankment fill soil with the drainage filter during its placement and compaction, it was necessary to place a suitable geotextile separator between the drainage filter and the earth fill placed above it subsequently. Both non-woven and woven geotextile may be used to act as the separator. However, non-woven geotextile has low tensile strength and undergoes large strain under load, allowing large lateral embankment deflection during its construction over soft clays having poor shear strength. Woven geotextile undergoes lower strain during comparable loading and therefore is a better choice in this respect. Moreover, woven geotextile has higher tensile strength compared to non-woven and can also act as basal reinforcement for the embankment under construction. While choosing the woven geotextile, it is necessary to consider the permeability and apparent opening size to avoid clogging of the drainage filter.

Woven geotextile as shown in "Table 3" was therefore employed based on computerized embankment stability analysis to act as a separator as well as basal reinforcement for the embankment to reduce stresses induced in the soft clay particularly in the initial stages of embankment construction.

DRAINAGE FILTER

A suitable drainage filter is necessary to ensure quick drainage of pore water discharged from installed PVD in order to allow soft clay consolidation to take place without delay. As clean sand was difficult to obtain for this purpose, stone chips (aggregates) 8-10 mm in size and minimum 300 mm in thickness was employed for this purpose over the installed PVD.

Table 3. Woven Geotextile Specification

Description		Test Standard	Value
Mass, kg/m^2		ASTM D5261	0.160
Tensile Strength, kN/m	Warp	ASTM D4565	50
	Weft		40
Elongation at break, %	Warp	ASTM D4565	25
Grab Tensile Strength, N		ASTM D4632	1400
Trapezoidal Tearing Strength, N		ASTM D4533	800
Water Permeability, $l/m^2/s$		ASTM D4491	37
Index Puncture Resistance, N		ASTM D4833	425
Apparent Opening Size, µm		ASTM D4751	130

EMBANKMENT CONSTRUCTION

As the underlying soft clay was weak in shear strength, it was necessary to consider lateral embankment stability during all stages of its construction. Embankment loading had to be controlled to ensure that shear failure in underlying soft clay does not happen at any stage of embankment filling. Stability computations established the maximum fill levels possible in each stage of embankment loading as well as minimum waiting periods required in between loading stages for the soft clay to gain sufficient strength before the next stage of embankment filling takes place. Minimum Factor of safety considered for each stage of loading was 1.2 and 1.1 under static and seismic condition respectively. Under full embankment load the FS required was 1.3 and 1.1 for static and seismic conditions respectively. Based on this, the embankment construction had to be carried out in 4 stages where soft clay was thick and embankment height maximum. Please refer "Fig.2(c)". The woven geotextile basal reinforcement also helped to improve embankment stability. The required minimum waiting period between stages was 4 weeks.

GEOTECHNICAL INSTRUMENTATION

Platform settlement gauges and Casagrande Piezometers were installed at selected locations to monitor ground settlement and dissipation of pore water pressure respectively during clay consolidation. Settlement gauges were installed approx. 0.5 m below ground level and Piezometers were installed within boreholes at depths approximately middle of the soft clay thickness and 1m above the bottom of the soft clay layer. Both instruments were installed as per recommended procedure and in groups for comparison of records at the required locations. The instruments were monitored throughout the embankment construction and clay consolidation for a period of more than 12 months. Typical observed settlement and Piezometer excess pore pressure at one location (Settlement Gauge S_2 and Piezometers P3 and P4), where maximum soft clay depth was 18 m below GL and embankment height was 7 m, are included in “Fig.2(a)”and “Fig.2(b)”. P3 & P4 were installed at mid depth and at 1.0 m above bottom of the soft clay layer, respectively.

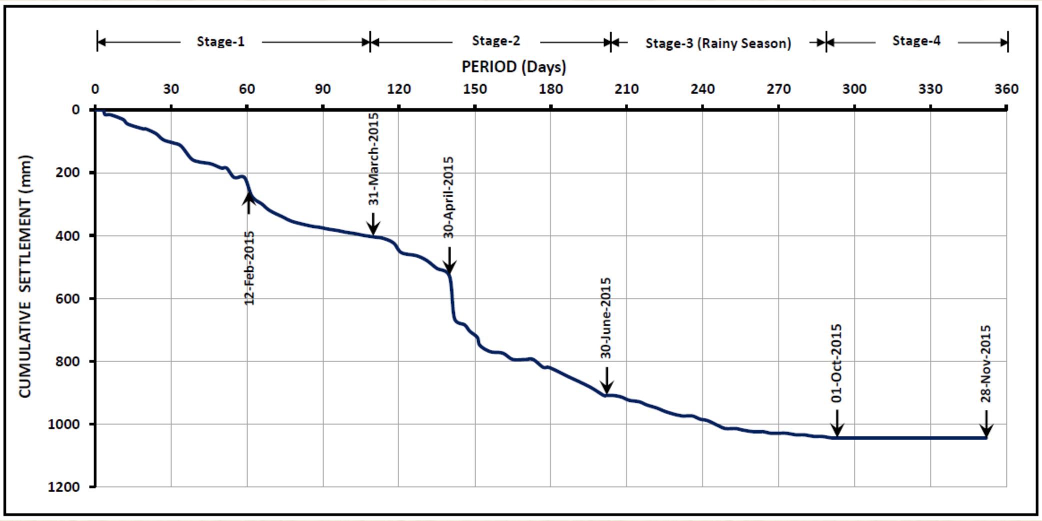

Fig. 2(a). Observed Settlement at Ch 2+820 (S_2)

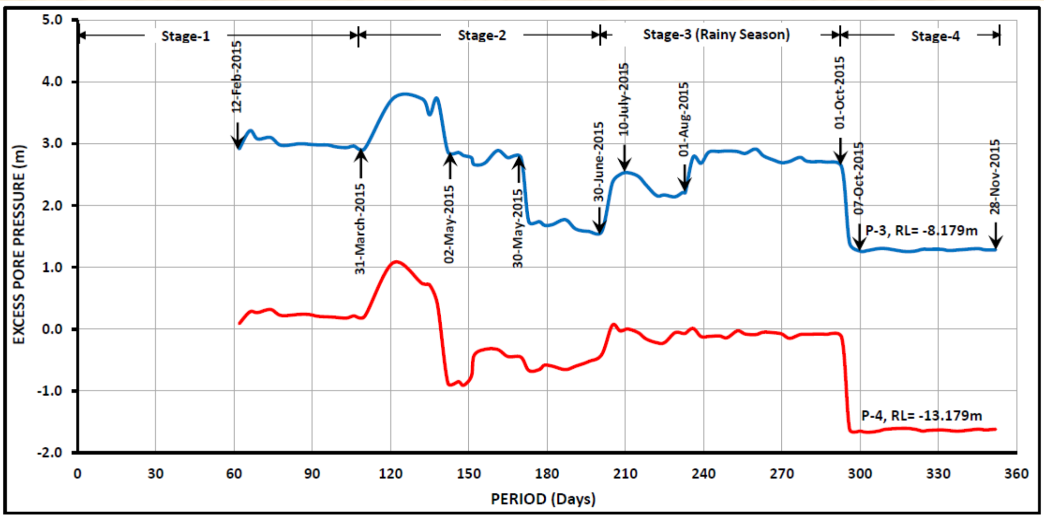

Fig. 2(b). Piezometer Excess Pore Pressure

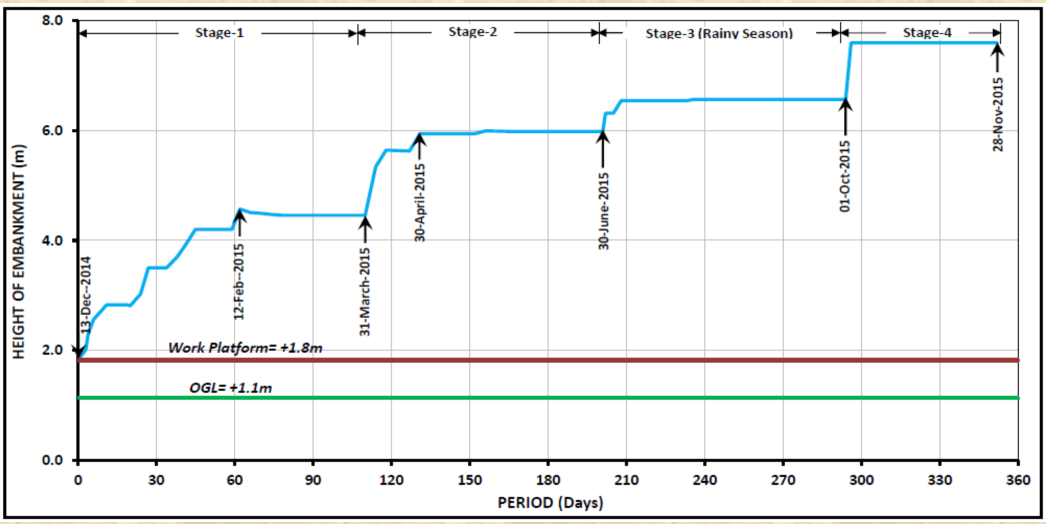

Fig. 2(c). Embankment Fill Height

Comparing with the embankment loading at the same location (Fig.2(c)), it may be clearly observed that both settlement gauge and Piezometers at this location responded in general, as expected in accordance with the embankment loading. "Table 4" below shows the settlement observed under different loading stages. The maximum settlement recorded at this location was 1044 mm under the completed embankment. The degree of consolidation achieved under each stage of embankment loading is included in "Table 4". It is significant to note that no settlement was observed under the final stage of loading as by end of stage-3 already 100% of total primary settlement had taken place.

Figure 2(b) shows the excess pore pressure registered during the period of observation. Due to some delay, Piezometers were installed for the project only two months after Stage-1 embankment construction commenced. In general, the excess pore pressures registered are as expected. However, the pore pressure dissipation appears to be slower initially but was much quicker under the later stages of embankment construction. It may be noted that both the Piezometers- at mid clay layer and 1m above bottom of clay layer- responded similarly to fill level changes. Much lower excess pressures were registered at the lower piezometer (P-4) than the one at mid layer (P-3). Excess pore pressure computations showed some negative excess pressure at times. However, it may be concluded that they may not really be negative pore pressures in soil but could rather be due to observed ground water levels in reference wells established some distance away. The sudden excess pore pressure dissipation noticed at the end of stage-3 is hard to explain and requires further study of the results.

Table 4. Observed Settlement and Degree of Consolidation under stages of loading

Loading Stages	Fill Height, m	Observed Settlement, mm	Degree of Consolidation, %
Stage-1	3.4	404	39
Stage-2	1.5	505	87
Stage-3	0.6	135	100
Stage-4	1.0	Nil	100

DISCUSSION

Figure 3 shows the Degree of consolidation ($\bar{U}$) against the time period for consolidation under the four different stages of embankment construction at Settlement gauge location S_2 as shown in "Fig. 2(a). The same figure also shows the predicted degree of consolidation against period 'ideal case' proposed by Hansbo (1979) when similar embankment is constructed in a single stage. As is noticed the required $\bar{U}$=90% is achieved much earlier in the case of the ideal one stage construction. However sub-soil conditions dictate the construction of the embankment in a number of stages, four in this case. While planning the total construction period, it is therefore important to consider the stages of embankment construction.

Secondly, it may be noted that there was no settlement observed under the final 1m filling under stage-4 construction (Fig.2(a)). A larger final stage loading might have caused further clay consolidation and settlement delaying the embankment construction. It is clearly seen that in cases where embankment construction period is limited, speed of embankment construction should be maximized under earlier stages of loading. The success of this project may be attributed to the fact that most embankment filling took place before the heavy monsoon period giving ample time for full clay consolidation to take place during the monsoon period.

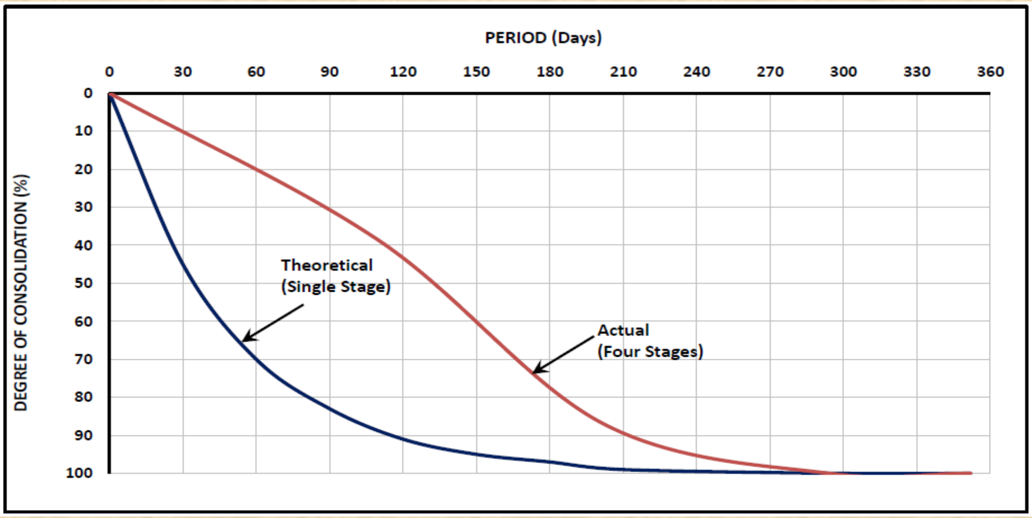

Fig. 3. Comparison of Degree of Consolidation

CONCLUSIONS

The project clearly demonstrates the importance of PVD in reducing the time period required considerably for high embankment construction over thick soft compressible clays. By careful planning, design and execution of ground improvement work using PVD and controlling embankment construction, high embankments can be constructed safely and within shorter construction periods.

Suitable Geotextile material can be incorporated in embankment construction for safety as well as to speed up construction. A woven geotextile was used in this case for the dual purpose- as a separator as well as a basal reinforcement.

Geotechnical instrumentation is an integral part of ground improvement work for purposes of control as well as to ensure that desired results have been achieved. Careful installation and monitoring and good knowledge of geotechnical engineering to interpret results are essential for the success of such works.

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