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Performance of Highway Embankments Constructed Over Sri Lankan Peaty Soils

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Abstract: The construction of the Southern Expressway in Sri Lanka involved extensive ground improvement work as many parts of the Expressway traverses through flood plains and marshy ground consisting of very soft peat, organic soils, and clays. Depending on the ground conditions, various ground improvement methods including remove and replacement, preloading, preloading with vertical drains, dynamic compaction and vacuum consolidation were applied to improve the soft soil to build the embankments with heights varying from 2m to 12m. In this project, embankments of about 4 km in length were constructed by improving the peaty soil basically by the application of the heavy tamping method. The length of the embankments that were built by improving the peaty soil by vacuum assisted surcharging is around 2.5 km. The details of the field instrumentation program and field monitoring program to assess the soft ground improvement are presented. The performance of the ground improvement was evaluated in terms of the degree of consolidation, improvement of the physical and engineering properties, increase in preconsolidation pressure and gain in shear strength of the peaty soil. The results of the post construction surface settlement monitoring of the expressway carried out up to date reconfirm that the ground improvement work was very successful and the expected residual settlements are well below the allowable limit of the contract.

Keywords: Peat, consolidation, embankment, settlement, stability

1. Introduction

The Southern Highway is Sri Lanka's first E Class highway that links the Sri Lankan capital Colombo with Matara, a major city in the south of the island. The length of 96 km section from Colombo to Galle had been completed and opened to traffic in November 2011. Many parts of the highway traverse through flood plains and marshy ground consisting of very soft peat, organic soils, and clays. Especially, in the major flood plain of Welipenna river, Bentota river and Gingaga river areas, thick peat and organic clay deposits were found. These problematic soils have low shear strength, high compressibility and low bearing capacity, and therefore it needs to be improved to avoid excessive settlement and prevent stability failure during expressway embankment construction. Also, these peaty soils posses high creep settlements and therefore it is necessary to improve the soft ground to control the post construction settlements [1]. Many ground improvement methods have been used on soft soil to improve its bearing capacity and minimize the anticipated settlement in this section of Southern Expressway Project. Ground improvement methods such as surcharging, surcharging with pre-fabricated vertical drains, rock replacement, heavy tamping, vacuum consolidation and piled embankment have been used to improve the soft soil in order to control the post construction settlements and to ensure the stability of the highway embankment.

This paper presents the ground improvement methods applied in a section of the Southern Expressway between Ch.0+000 km to Ch.66+160 km to improve the peaty soil, with some background information on the design methodology. In the first 34 km of the highway, about 50% of the area is covered by soft ground and from 34 km to 66 km, the distance covered by the soft ground is around 12 km. In this project, embankments of about 4 km in length were constructed by improving the peaty soil basically by the application of heavy tamping method. The length of the embankments that were built by improving the peaty soil by vacuum assisted surcharging was around 2.5 km. The problems encountered during ground improvement work and embankment construction and the solutions given for the same are highlighted and discussed. The details of the laboratory and field investigations carried out before and after ground improvement, field instrumentation program and field monitoring program that was carried out during and after construction of highway embankment to assess the soft ground improvement are presented.

2. Details of the Subsoil Profile

Many geotechnical investigations have been carried out since the inception of the project in order to assess the condition of the soft ground. At the preliminary stage, to provide information to bidders and to facilitate initial designs, boreholes were carried out at 500 m intervals.

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After commencement, boreholes were carried out at about every 50 m interval in order to provide the necessary information for the detailed design.

Site investigation consisted of bore holes with Standard Penetration Test (SPT), hand augering, Cone Penetration Test with pore pressure measurement (CPTu) as in-situ testing and a series of laboratory testing such as index property tests, unconsolidated undrained triaxial compression tests and conventional consolidation tests.

The investigation identified that the soft ground area of the highway mainly consisted of peat, organic clay, alluvial clay and loose sand deposits. The distribution of soft soil deposits along the highway trace from Kottawa to Kurundugahahetekma is shown in Fig. 1.

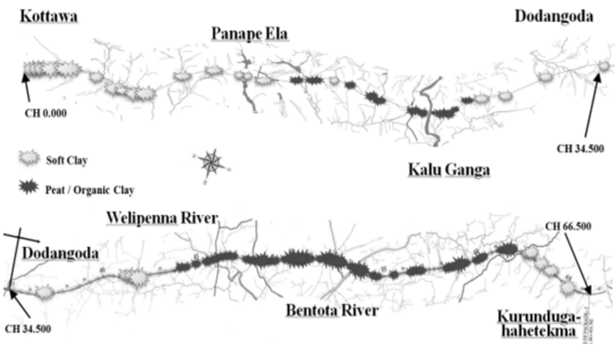

Fig. 1 Distribution of soft ground areas

Silty clay and silty sand were found as top soil in most of the lowland areas up to a depth of 1.5 m to 3.0 m. This was followed by sand to lateritic soil and the thickness of the layers varied from 1 m to 5 m.

In the flood plains of Panape, Kalu Ganga, Welipenna and Bentota river areas sub soil consisting of mainly peat, organic clay, very soft inorganic clay and silt layers was found. The total average thickness of the compressible layer was in the range of 4 m to 11 m. In some areas loose silty sand layers were present under the above compressible layers. In the valley areas between hillocks, instead of cohesive inorganic clays, very loose to loose silt and sand were found ranging from 0.5 m to 4m thickness. The details of the Geotechnical properties of the subsoil have been given in [2] and [3].

3. Design of Soft Ground Improvement

Soft ground improvement design had to be carried out in order to control the settlements and to ensure the stability of the highway embankment as required in the technical specification. According to the technical specification, the embankment had to be designed and constructed by improving the soft ground in order to control the continued settlement to 15cm at the road center after a period of 3 years following the acceptance of the paving. In addition, the maximum residual differential settlement had to be not more than 0.3% change in grade over longitudinally within three years after construction. In order to achieve the above criteria, most or all of the primary settlement and some of the secondary settlement that would have occurred under the final embankment height alone were forced to take place by improving the soft ground.

The soft ground was improved mainly by using the following methods based on the subsoil conditions. Soft clay of shallow thickness was improved by placing a surcharge load. Shallow peat and organic clay deposits were removed and replaced with rock in order to support the embankments. The subsoil with relatively thick soft clay layers were improved by installing vertical drains and placing a surcharge load. The embankments on the relatively thick peat and organic deposits were constructed by improving the ground by heavy tamping method and the vacuum consolidation method from sections 0.0 km to 34.5 km and from 34.5 km to 66.5 km respectively.

In rock replacement method, all compressible layers of the sub soil were removed and replaced with rock, completely eliminating the settlements. In the ground improvement method of application of surcharge load with or without vertical drains, future settlement of the highway embankment was controlled as required in the contract by designing of an appropriate surcharge load. Most or all of the primary settlement and some of the secondary settlement that would have occurred under the final embankment height alone were forced to take place under the surcharge load. In addition, it was expected that the soil beneath the embankment would become over consolidated or stiffer due to the surcharging of ground. The aim of applying the surcharge was to eliminate 100% of primary consolidation settlement and enough secondary settlement such that the residual settlement is within acceptable performance limits. The residual settlement for a given length of time after construction was estimated as the remaining secondary settlement that occurs during the required time after the eliminated equivalent time of secondary compression has elapsed. In the design of surcharge, it was expected to have 1.1 over consolidation ration (OCR) for inorganic clays and 1.2 to 1.3 OCR for peat and organic clays in order to reduce the secondary settlements during the operation period.

The additional treatments were done after improving the ground in the bridge and under pass approaches in order to smooth the transferring of expected differential settlements between the approach embankment and the bridge deck. After the ground improvement, the approach area within the improved peat layer was replaced with rock within the approach embankment areas in order to eliminate the secondary settlements. The approach embankments on thick peat layers were constructed with special geogrid arrangements in order to have a smooth gradient between the approach embankment and the bridge deck.

4. Embankment Construction on Peaty Soil

Embankments over peaty deposits in the Southern Expressway between Ch. 0.000 km to Ch 34.500 km were constructed by improving the peaty soil using the heavy tamping method whereas vacuum consolidation technique was applied to improve the peaty soil in the section between Ch.34.500 km to 66.500 km. This part presents the details of the heavy tamping method and the vacuum consolidation techniques applied in the project.

4.1 Heavy Tamping Method

Heavy tamping method resulted in a quick decrease in void ratio of the peaty soil and instantaneous settlement of ground under impact. Heavy tamping method was designed to enforce the settlements that would be caused by the construction of earth embankment on soft ground by applying impact energy. Different energy levels had to be imparted by considering the anticipated settlement of the compressible layer under the respective designed embankment heights. In the estimation of settlements, all primary consolidation settlements and secondary settlements at the end of 3 years after construction were considered. The estimated settlement of peaty soil layers of different thickness under various embankment heights is shown in Fig. 2. In the calculation of settlement, the values of 0.428, 0.0428, 0.05, and 20 kPa were used for the parameters $c_c/(1+e_0)$, $c_r/((1+e_0)$, c_α, and P_c respectively where c_c is the compression index, e_0 is the initial void ratio, c_r is the recompression index, c_α is the coefficient of secondary consolidation and P_c is the preconsolidation pressure.

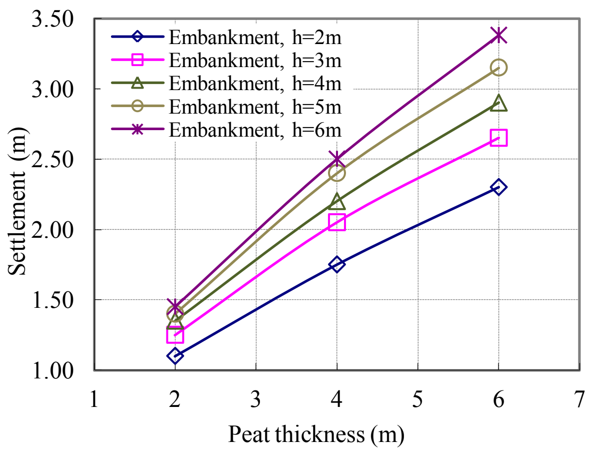

Fig. 2 Predicted settlement in peaty clay due to different embankment heights.

The energy level required to enforce the designed settlement was estimated using the graph shown in Fig. 3. First, the soft soil, which was to be consolidated, was overlain by a working platform of lateritic soil to facilitate the movement of machinery. Then, a strong type fibre drain (band drain) was installed by a machine in soft subsoil in a square pattern with a spacing of 1 m. The required energy was applied to the soil by dropping a large weight on the ground surface repeatedly in phases on a grid pattern over the entire full base width of the embankment using multiple passes. A high energy level was applied in 5 phases whereas only 4 phases were used to apply a low energy requirement.

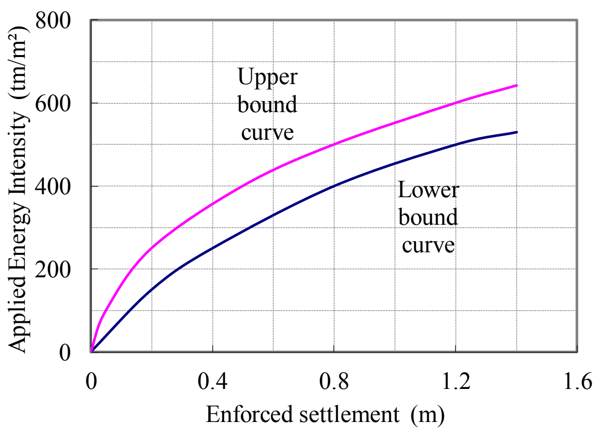

Fig. 3 Relationship between the enforced settlement and applied energy [4].

The energy intensity per phase was gradually increased from 15% - 20% of the total energy intensity to 30% - 40% at the last phase prior to the ironing phase. The spacing of the prints of the pounder dropped on the square grid was estimated as 2H to 2.5H, where H is the thickness of the compressible layer. As the ground strength improved the spacing was reduced in the subsequent phases. In the ironing phase, a smaller drop height was used at very close spacing for removing surface unevenness and to compact the soil at shallow depths.

During tamping, once the depth of the crater formed by the drop of pounder exceeded the height of the pounder, the crater was back filled and leveled with soil. The dimension of the crater was recorded in order to calculate the volume of soil introduced. The above process was continued in all phases of tamping operation. Using the crater fill volumes, the enforced settlements were calculated and if the enforced settlement was less than the required then another phase of tamping was introduced until the required settlement was achieved.

The weight of the pounder was 15 tons and it was built by stacking and bolting a series of 25 mm thick mild steel plates, 2 m by 2 m in plan area. A 60 ton capacity mobile crane that was equipped with an automatic lifting and release mechanism of the pounder was used in the tamping operation. The height of drop is governed by the pounder dimension, crane capacity and boom configuration and hence, was found to be a maximum 8 m in this operation. The depth of improvement generally depends on the total amount of energy applied to the soil, which is a function of the weight of the tamper and the drop height as shown in the following equation as reported by [5].

$$D = n\sqrt{(WH)} \qquad (1)$$

In the above equation, D is the depth of soft/loose soil to be improved, W is the weight of the tamper (pounder) in tons, H is the height of drop in m and n is a constant ranging from 0.3 to 0.6 and for the peaty soils found in the site it was taken as 0.35. According to the above data, the practically possible improvement depth that could be achieved in the present operation was about 3.5 m to 4 m. However, according to the investigation, it was observed that the soft compressible layer thickness was higher than above in most of the locations and therefore the underneath deeper soft layers were not improved properly by heavy tamping. These underneath deeper soft layers were improved after the heavy tamping operation by keeping a surcharge load for a sufficient period of time. Fig. 4 illustrates the major steps in the heavy tamping ground improvement adopted in the project.

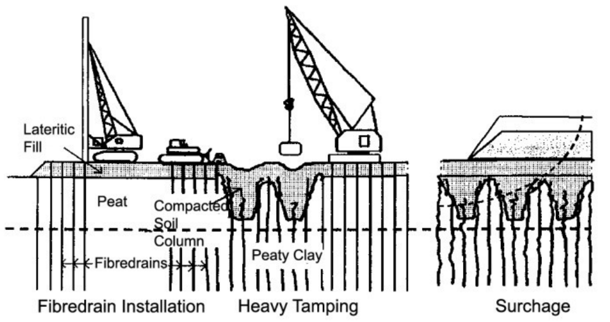

Fig. 4 Major steps in heavy tamping ground improvement.

4.2 Effect of the Fibre drain Installation for the Tamping Energy

The in situ permeability of peat is relatively higher than that of ordinary clays and therefore it was assumed that quick dissipation of excess pore water pressure would occur within the peat during the tamping operation. However, it was noted that the process of pore water pressure dissipation was rather slow and therefore no further densification can be achieved by imparting additional energy to the soils. This could have been due to the rapid reduction of permeability of the peaty soil as a result of damage to the structure of the soil, presence of substantial amount of clay content and long drainage paths. Therefore, it was noted that the energy level required to achieve the designed enforced settlement was very high as the excess pore pressure was not allowed to dissipate as expected. Having experienced the above difficulty, it was decided to install special strong type fibre drains in the peaty soil prior to the tamping in order to eliminate the development of high excess pore water pressure during the heavy tamping operation. Fig. 5 shows the applied tamping energy level in order to achieve the designed enforced settlement with and without fibre drain installation in similar ground conditions. As shown in Fig. 5, the energy that was required to achieve the designed enforced settlement in fibre drain installed peaty ground was significantly lower than the energy required without fibre drains. This clearly shows that part of the applied energy was wasted in compressing the 'incompressible' water. Therefore, provision of a strong band drain such as fibre drains to withstand heavy tamping impact is useful in reducing the required tamping energy level as described in [4].

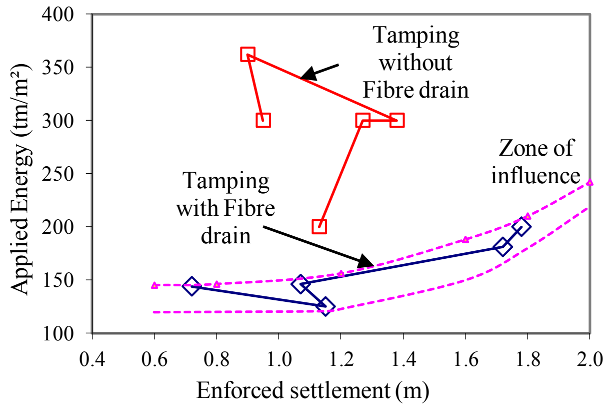

Fig. 5 Comparison of Applied energy vs. Enforced settlement with and without fibre drains.

4.3 Vacuum Consolidation Method

The vacuum consolidation was carried out using the "Compact Vacuum Consolidation" (CVC) patent by Maruyama Industry, Japan. A brief description of the method adopted at the site is given below and the schematic construction is shown in Fig. 6.

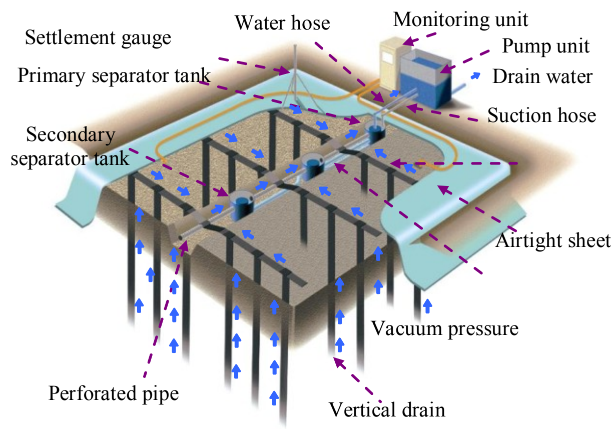

Fig. 6 Schematic construction of compact vacuum consolidation.

In the application of vacuum consolidation method, about a 1.0 m to 1.5 m thick fill was constructed on the original ground surface to form a working platform for the band drain installation machine. Band drains were installed by a machine up to a designed depth from the original ground surface in a square pattern with a spacing of 1 m. Thereafter, flexible horizontal drains (300 mm wide and 4 mm thick) were laid on top of the fill with a horizontal spacing of 1 m and then connected to the vertical band drains in order to ensure adequate horizontal drainage capacity. Subsequently, the tank system was installed and connected to the designed pipe systems. Small ditches were excavated perpendicular to the horizontal drains at 20 m intervals and filled with aggregates after placing perforated pipes. Instrumentation such as settlement plates, displacement stakes, electrical piezometers and differential settlement gauges were also installed at the designed depths. After installation of vertical drains, horizontal drains, perforated pipes and separator tanks, the surface of the treatment area was covered by a protection sheet. Thereafter, an air tight sheet was laid on top and the periphery trench system was constructed to provide air tightness and the necessary anchorage at the boundary of the treatment area. Vacuum pressure was then applied using a vacuum pumping system patented by Maruyama Industry Co. Ltd, Japan by connecting the suction and water hoses to the vacuum pump. After confirming that there were no leaks through the air tight sheet, filling was commenced.

It was expected to apply the surcharge by means of a vacuum pressure of 70kPa to compensate the primary consolidation settlements and to minimize the secondary settlements that can take place in the proposed highway embankment. However, in many areas the applied vacuum pressure was less than the designed value and therefore the above designed surcharge was applied by means of both vacuum pressure and embankment fills. The designed load was kept until the expected settlement completed.

5. Field Monitoring Program

An extensive monitoring program was carried out to understand the field behavior of the foundation soil due to the different ground improvement methods. The improvement of the soft ground was monitored through the measurement of settlement and the excess pore water pressure during the construction period. Settlement plates were installed at the top of the soft layer or on top of the pioneer layer and piezometers were installed at the middle of the soft layer. The settlement stakes were installed near the toe of the embankments to check the stability during the construction. In addition to the above, in the areas improved by vacuum consolidation, a vacuum pressure monitoring unit was used to measure the vacuum pressure at the pump and under the air tight sheet. Also, a water discharge meter was used to measure the rate and the total discharged water flow due to the vacuum operation. An automatic data acquisition unit was connected with the piezometer, vacuum pressure monitoring unit and water discharge meter to keep continuous records. The locations of the instrumentation and the field monitoring data for the embankment constructed using the vacuum consolidation method are shown in Fig. 7 and Fig. 8 respectively.

Installed instruments were monitored at selected time intervals to investigate the performance of the intended ground improvement process and the stability of the embankments. The filling rates and the vacuum pressure were adjusted when the stability of the embankment was seen to be at risk, based on the field monitoring data. During the first two weeks of the vacuum operation, the vertical settlement and lateral movement devices were monitored twice a day and the frequency of monitoring was increased with the progress of the embankment construction work. After construction of the embankment up to the required level, monitoring frequency was reduced to once a week. But daily records of the vacuum pressure at pump and under the sheet as well as piezometer readings were taken from the beginning to the completion date of the treatment.

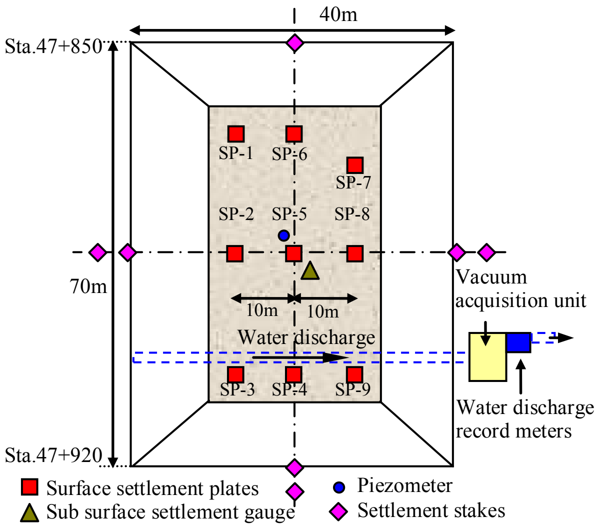

Fig. 7 Plan of the embankment and instrumentation layout.

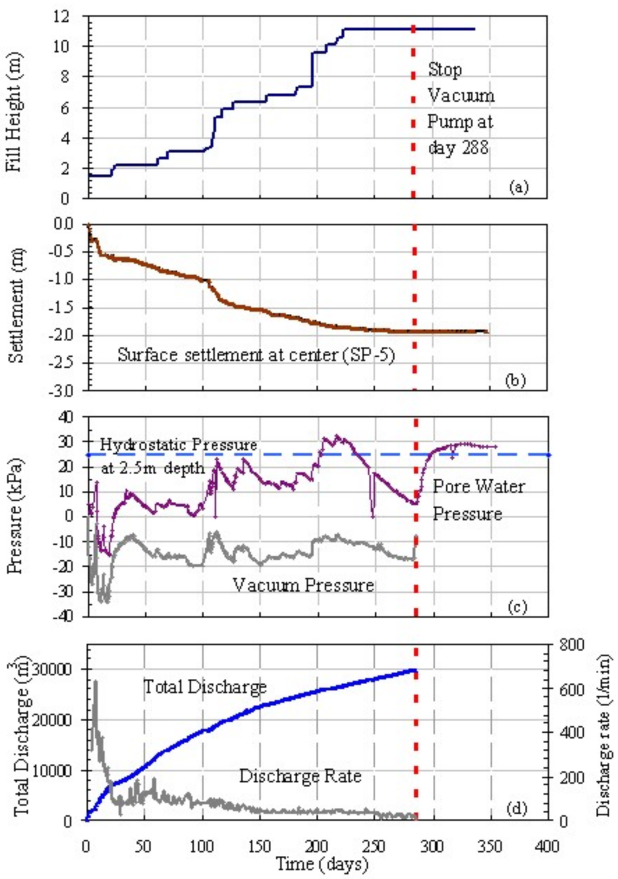

Fig. 8 Field monitoring data.

6. Assessment of the Soft Ground Improvement

The performance of the ground improvement was evaluated in terms of the degree of consolidation, improvement of the physical and engineering properties, increase in preconsolidation pressure and gain in shear strength of the peaty soil.

6.1 Estimation of degree of Consolidation

The ground improvement achieved was investigated by calculating the degree of consolidation using the observed field settlements before termination of the vacuum operation and the removal of surcharge. The degree of consolidation is calculated as the ratio of the current settlement to the expected ultimate primary settlement. In the present work, ultimate primary settlement and the degree of consolidation were estimated by means of the Asaoka [6] and hyperbolic (Tan et al. [7]) methods using the measured field settlement data. The graphical plot of the Asaoka method based on the observed settlement under the center of embankment (SP 5) at vacuum consolidated ground improvement section from Ch.47+850 km to Ch. 47+920 km is shown in Fig. 9.

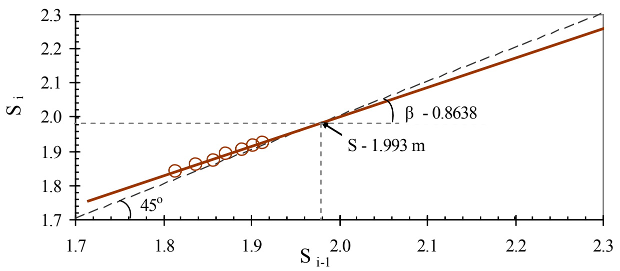

Fig. 9 Graphical plot of Asaoka method.

It is seen that at the end of CVC treatment the achieved degree of consolidation is around 97%. The degree of consolidation was also calculated based on the pore water pressure measurements (PWP), and laboratory consolidation testing of peaty samples after the treatment program. The comparison of the degree of consolidation for each method is shown in Table 1.

Table 1 Estimation of the degree of consolidation

Location	Degree of Consolidation		
	Asaoka Method	Laboratory Data	PWP
Ch. 45+380 – Ch. 45+430	97.83%	83.10% 73.87%	79.46%
Ch. 47+850 – Ch. 47+920	97.10%	100.00% 100.00%	100.00%
Ch. 52+950 – Ch. 53+000	97.57%	80.21% 90.91%	100.00%
Ch. 53+660 – Ch. 53+730	96.65%	96.70% 83.62%	68.71%

If the degree of consolidation from the PWP measurement is assumed to be accurate, Asaoka Method accurately estimates the degree of consolidation in treatment areas 47 + 850 to 47 + 920 and 52 + 950 to 53 + 00 whereas Asaoka method over predicts the degree of consolidation in treatment areas 45 + 380 to 45 + 430 and 53 + 660 to 53 + 730. However, in treatment area 53 + 660 to 53 + 730 the degree of consolidation from the laboratory test results agree very well with the same estimated from the Asaoka method. Therefore, based on this investigation it can be concluded that the degree of consolidation estimated from the Asaoka method is reasonably accurate.

In order to assess the secondary settlements, for each monitoring point, the long-term settlement was predicted by extrapolating the secondary settlement rate over a period of 3 years. Predictions were made by preparing a plot of displacement against log (time) for each settlement plate, with the best-fit line through the data extended to define the likely settlement after 3 years. The surcharge was removed only after confirming the residual settlement by considering both the primary and secondary consolidation settlements as described above.

6.2 Improvement of Physical and Mechanical Properties of Peat

Site investigation was carried out to assess the actual ground improvement in the areas improved by the heavy tamping and vacuum consolidation method just before the removal of surcharge. Investigation was carried out in the improved as well as the adjacent unimproved area in order to assess the ground improvement. Site investigation comprised of advancing of bore holes with Standard Penetration Test (SPT), collection of undisturbed soil samples, performing of Field Vane Shear Test, Cone Penetration Test (CPT) and performing of laboratory tests. The improvement of physical and mechanical properties of peat due to heavy tamping improvement is described in [3]. The observed subsoil in one of the CVC improved areas deduced from the borehole investigation is shown in Figure 10. In the same figure recorded SPT values are also plotted along the depth.

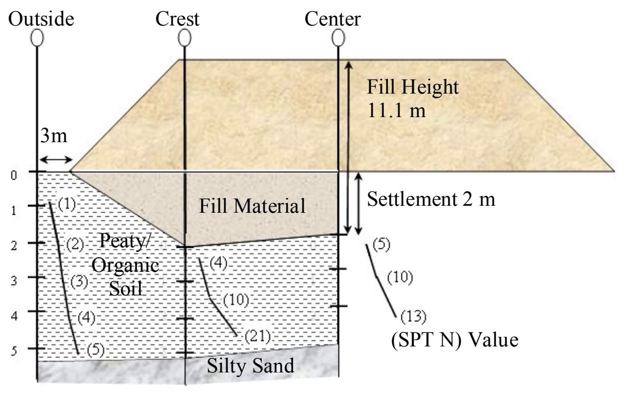

Fig. 10 Subsoil profile of improved and unimproved area.

Table 2 Change in peat thickness, water content and void ratio due to CVC improvement

Location	Peat Layer Thickness (m) Initial	Final	Water Content (%) Initial	Final	Void Ratio Initial	Final	% Change in Thickness of Peat	% Change of Water Content	% Change of Void Ratio
Ch.45+380-Ch.45+430	7.70	2.30	378.3 406.8	163.6 137.3	6.83 10.11	3.916 3.805	70.13	56.75 66.25	42.66 62.36
Ch.47+850-Ch.47+920	4.70	2.00	370 398	141.0 168.0	5.54 5.58	1.810 1.940	57.45	61.89 57.79	67.33 65.23
Ch.52+950-Ch.53+000	5.85	2.85	378.3 471.2	175.7 105.8	4.10 9.35	1.900 2.580	51.28	53.56 77.55	53.66 72.41
Ch.53+660-Ch.53+730	5.25	2.76	111.6 122.9	86.6 79.7	3.06 2.66	1.810 2.000	47.43	22.40 35.15	40.85 24.81

The observed change in the peaty soil layer thickness, the changes in water content and void ratio due to CVC ground improvement are given in Table 2. Accordingly, the initial thickness of the peat layer has been reduced by 50%-60% after ground improvement. The above reduction reasonably agreed with the percentage change of water content and void ratio values obtained from peaty soil collected from the improved and unimproved areas. The summary of laboratory test results from the peat samples collected from the improved and unimproved area of Ch.47+850 to Ch.47+920 is given in Table 3. Consolidation tests revealed the significant reduction in the compression index which is proportional to primary consolidation settlement. The compression index of the peat layer has reduced from a range of 2.65 to 2.13, to as low as 0.90 as a result of the ground improvement. The average reduced value is about 1.65.

Table 3 Summary of laboratory test results

Location	Depth (m)	W_n %	e_o	c_c	c_α	c_u (kPa)
Improved Area – BH1	3.0	227	4.21	1.80	0.055	85
	3.5	266	4.50	1.95	0.061	120
Improved Area – BH2, 5	3.5	141	1.81	0.90	0.037	55
	4.5	168	1.94	1.94	0.048	70
Unimproved Area – BH3	2.5	370	5.54	2.13	0.110	22
	3.5	398	5.58	2.65	0.120	33

The reduction of secondary compression is very important as the secondary compression phenomenon is dominant in the peaty soil. The results of long term consolidation tests carried out in the improved and unimproved peaty samples are shown in Table 3. It reveals that the coefficient of secondary consolidation has reduced from a range of 0.10 to 0.13 to a range of 0.03 to 0.06. Subsequently the ratio of C_α / C_c has decreased from 0.050 to 0.029 due to ground improvement. The estimation carried out based on the above information assures that the residual settlement would be less than 150 mm by the end of 3 years after construction as required in the contract [2].

6.3 Increase in Preconsolidation Pressure and Undrained Cohesion

It is expected that the subsoil behaves in an over consolidated state during the service life of the structure after the completion of ground improvement. This can be verified by the pre-consolidation pressure determined through the 1-D consolidation test. Consolidation tests carried out from section Ch.47+850 to Ch.47+920 indicated that the preconsolidation pressure of the peaty soil found under the embankment has increased from 28 kPa – 38 kPa range to 160 kPa – 180 kPa range after ground improvement as shown in Figure 11. The expected load induced on the peaty layer due to the proposed embankment is around 145 kN/m^2. Therefore, the subsoil will behave under the over consolidated state with an Over Consolidation Ratio (OCR) of 1.2 to 1.3 during the service life of the highway and hence will only give rise to very small settlements in the future.

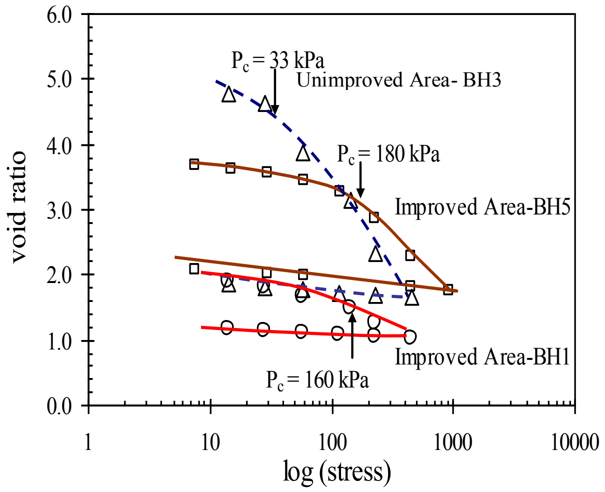

Fig. 11 Consolidation test results.

Consolidation test results related to some other sections also indicated that the pre-consolidation pressure of the peaty soil found under the embankment has increased as shown in Table 4. Table 4 also shows the expected load induced on the peaty layer due to the proposed embankment and the subsoil over consolidation ratio. According to the data in Table 4, the sub soil will behave under the over consolidated state with an Over Consolidation Ratio (OCR) of 0.98 to 1.33. It should be noted here that even though the applied vacuum pressure and the fill surcharge load is adequate to yield an OCR value in the range of 1.2 to 1.3, sometimes the calculated OCR is less than that the anticipated value. This might be due to the inaccurate Pc value obtained from the consolidation test as a result of sample disturbance.

Table 4 Increase in preconsolidation pressure and undrained cohesion

Location	Expected Load (kPa)	P_c (kPa)	OCR	C_u (kPa)	$\Delta c_u / \Delta \sigma'_v$
Ch.45+380-Ch.45+430	160.0	180	1.13	79.0	0.49
		160	1.00	57.0	0.36
Ch.47+850-Ch.47+920	145.0	200	1.37	55.0	0.36
		180	1.25	70.0	0.45
Ch.52+950-Ch.53+000	152.5	150	0.98	41.5	0.27
		170	1.11	38.2	0.25
Ch.53+660-Ch.53+730	150.0	170	1.13	54.0	0.36
		147	0.98	50.5	0.34

The SPT and Field Vane Shear results indicate that the strength has improved in the compressible layer due to the ground improvement and as a result the status of the compressible layer has been changed from very soft to medium stiff state. The strength gained due to ground improvement was investigated by calculating the ratio between the increments of undrained shear strength of peaty soil and the effective stress ($\Delta c_u / \Delta \sigma'_v$). The undrained cohesion of the peaty soil was determined from unconsolidated un-drained triaxial tests and the preconsolidation pressure was obtained from oedometer tests on undisturbed soil samples. The ratio between the increment of undrained shear strength of peaty soil and the effective stress after the treatment program was obtained to be 0.25 to 0.49.

7. Observed Settlement After Pavement Construction

The surface settlement of the highway embankment constructed over the improved soft ground was monitored by installing the settlement markers at 50 m intervals after construction of the road pavement. Initially, for about a 6 month period, before the road was opened to traffic, surface settlement was monitored at both the center and the edge of the embankment. The observed settlements were in the range of 0 mm to 5 mm in most of the ground improved sections except at very few locations where high embankments were constructed over thick peat deposits by improving the vacuum consolidation method. The observed surface settlement in those areas was around 10 mm to 20 mm at end of six months after the construction of the pavement. After the highway was opened to traffic in November 2011, settlement monitoring was carried out only along the edge of the highway embankment considering safety reasons. The observed total surface settlement up to September 2012, ten months after opening to traffic, is shown in Figure 12. The observed settlement was less than 5 mm in most of the sections and in only two locations the settlement exceeded 20 mm. The maximum observed settlement was 35 mm and the settlement prediction using the monitoring data indicates that the estimated residual settlement is less than 15 mm at the end of 3 years after the handing over of the project.

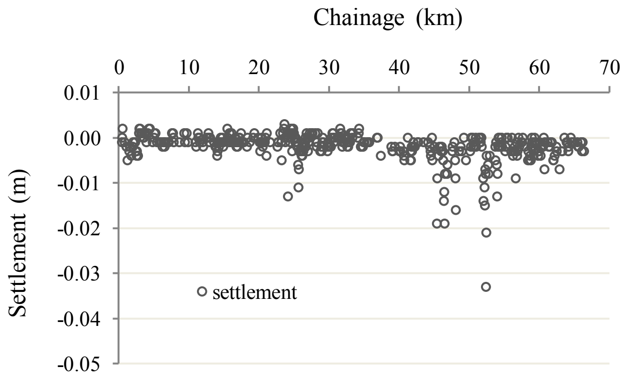

Fig. 12 Results of the surface settlement monitoring.

8. Conclusion

This paper presents successful application of ground improvement work carried out in the construction of Southern Highway project in Sri Lanka. Ground improvement methods such as heavy tamping method and vacuum consolidation techniques were applied to construct the high embankments over thick peaty deposits. In both methods, a surcharge load had been applied to over consolidate the peaty soil. Field monitoring data obtained during the construction period indicates that the primary consolidation settlement due to final load of the highway embankment has already been completed and the secondary settlement had been reduced to control the residual settlement within acceptable performance limits. Investigations carried out at the site show that both physical and mechanical properties of the peat have improved significantly and the peaty soil will behave in an over consolidated state with a ratio of 1.2 to 1.3 during the service life of the highway. The results of the post construction surface settlement monitoring of the expressway carried out up to date reconfirm that the ground improvement work was successful and the expected residual settlements are well below the allowable limit in the contract.

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