# Improvement of a Clay Deposit using Prefabricated Vertical Drains and Pre-loading. A Case Study

## Amélioration d'un massif d'argile à l'aide de drains verticaux préfabriqués et de pré-chargement. Une étude de cas

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ABSTRACT: Construction of a Container Terminal covering an area of of 153,000 m$^2$ is underway at Chittagong sea port in Bangladesh, situated on the right bank of the Karnaphuli river at its confluence with the Bay of Bengal. The sub-soil at the site consisted of a 4 to 6 m thick clay layer with random zones of soft to stiff clay (CL). This was underlain by a 10 m thick loose to medium dense silt/fine sand (SM) layer below which a loose clayey silt layer existed beyond 30 m depth. The unconfined compressive strength, void ratio and compression index of the soft clay zones varied in the ranges of 12~16 kPa, 0.8~1.2 and 0.2~0.3, respectively. The targeted use of the land required improvement of the sub-soil. This paper presents the design considerations, comparison of required time and cost of alternative options, effectiveness of the adopted measure and the achieved improvement of the engineering properties. Actual consolidation settlements were up to 600 mm over a period of about 30 days with PVD and pre-load. The effectiveness of the available theories of consolidation settlement under vertical and radial drainage, in the design of the ground improvement measures, has been demonstrated.

RÉSUMÉ : . La construction d'un terminal à conteneurs d'une superficie de 153.000 m$^2$ est en cours en ville portuaire de Chittagong au Bangladesh, située sur la rive droite de la rivière Karnaphuli à sa confluence avec le golfe du Bengale. Le sous-sol sur le site se composait d'une couche de 4 à 6 m d'épaisseur d'argile avec des zones aléatoires d'argile molle à raide (CL). Cela a été superposée à une épaisseur de 10 m limon / sable fin (SM) lâche à dense, une couche en dessous de laquelle une couche limon argileux lâche existait au-delà de 30 m de profondeur. La résistance à la compression, l'indice des vides et l'indice de compression des zones d'argile molle varient dans les plages de 12 à 16 kPa, 0.8 ~ 1.2 ~ 0.3 et 0.2, respectivement. L'utilisation obligatoire de ces terres requiert l'amélioration du sous-sol. Cet article présente la conception, la comparaison du temps nécessaire et le coût des options alternatives, l'efficacité de la mesure adoptée et de l'amélioration obtenue des propriétés mécaniques. Les tassements de consolidation réels ont été de 600 mm sur une période d'environ 30 jours avec PVD et pré-chargement. Dans la conception de l'amélioration des sols, l'efficacité des théories existantes de tassement de consolidation en vertu de drainage vertical et radial a été démontré.

KEYWORDS: Clay, Ground improvement, Pre-loading, PVD

## 1 INTRODUCTION

Chittagong sea port, the largest sea port of Bangladesh is situated on the right bank of the Karnaphuli river at its confluence with the Bay of Bengal. The port, that once handled mostly bulk cargo is gradually shifting its operational mode to handle increasing volume of container traffic. In this regard, Chittagong Port Authority (CPA) is implementing a project for construction of backup facilities at New Mooring behind berths 4 and 5. The site is locally known as 'NCT' (New-Mooring Container Terminal). The project area is about 153,000 m$^2$ which is planned to accommodate stacking yard for containers, passage for truck and trailer movement, tracks for Gantry Crane, electrical substation etc.

A comprehensive geotechnical investigation was carried out at the site to assess the sub-soil condition, decide on the necessity of improvement and determine relevant design parameters for the envisaged improvement methodology. The soil profile in the project area, consisted of a 4 to 6 m thick soft to medium stiff clay layer, underlain by a 10 m thick loose to medium dense silt/fine sand layer below which a loose clayey silt layer existed to more than 30 m depth.

To keep conformity with the earlier constructed adjacent yard, it is considered that the area will be paved with interlocking block (ILB) except the RMG (Rail Mounted Gantry) and RTG (Rubber Tyred Gantry) tracks which will be pile founded. On the basis of design requirements and geotechnical characteristics, improvement of the upper soft clay layer was considered essential to eliminate the possibility of differential settlement within the yard as well as between pile founded structures (i.e. jetty and RMG, RTG tracks) and yard. From an study of several alternatives, Prefabricated Vertical Drain (PVD) with pre-loading was adopted as the ground improvement measure for the site. Improvement measures have been completed on a part of the project area. The settlement under preloading with PVD has been monitored using settlement plates. Field and laboratory tests have also been conducted to evaluate the effectiveness of the adopted measures in terms of change of soil properties. This paper presents the geotechnical characteristics of the sub-soil in the area, the design considerations and a comparison of cost of several alternative improvement methods. From the limited data, that has so far been available, comparison of some engineering properties before and after preloading has also been made.

## 2 SITE LOCATION AND TOPOGRAPHY

The site for the container yard is in a tidal plain at a narrow strip between Chittagong hilly uplands and the Bay of Bengal. Geologically it is a recent alluvium formed by the material carried by the river Karnaphuli and its tributaries from the upper tertiary hills. Figure 1 shows the site map with grid lines. About half of the land, the eastern side (segments marked 1, 2, 3 and 4) had been used as jetty yard for more than 50 years and housed storage sheds for general cargo, road and railway tracks. The other half (western part, segments marked 1A, 2A, 3A and 4A) contained a city road, a residential area of CPA containing one/two storey building, ponds, play ground, open land, village dwellings etc. Different parts of this western side had different elevations with 1~3 m ditches. Because of earlier diverse use of the land, there was little possibility of homogeneity of the upper soil layer in the area.

## 3 SUB-SOIL CHARACTERISCTICS

A total of 67 exploratory boreholes were drilled in the area to gather information about sub-soil type and characteristics. The borehole locations were carefully decided to make them distributed over the entire area as well as to cover zones of different land use (i.e. pond, road, houses etc.). The borehole that are referred here are marked by grid points as shown in Figure 1. The boreholes, approximately 100 mm in diameter, were drilled using water flush aided by chiselling. Twelve boreholes were of 30.5 m depth and the rest were of 15.0 m depth below the existing ground level. Standard Penetration Tests (SPT). were made at 1.5 m interval. Undisturbed samples were retrieved from cohesive layers by pushing conventional 76 mm external diameter thin-walled Shelby tubes. Disturbed samples were also collected from the SPT spoon (conventional split spoon) from cohesive and cohesionless soil layers at different depths for visual-manual identification of the layers as well as for laboratory testing.

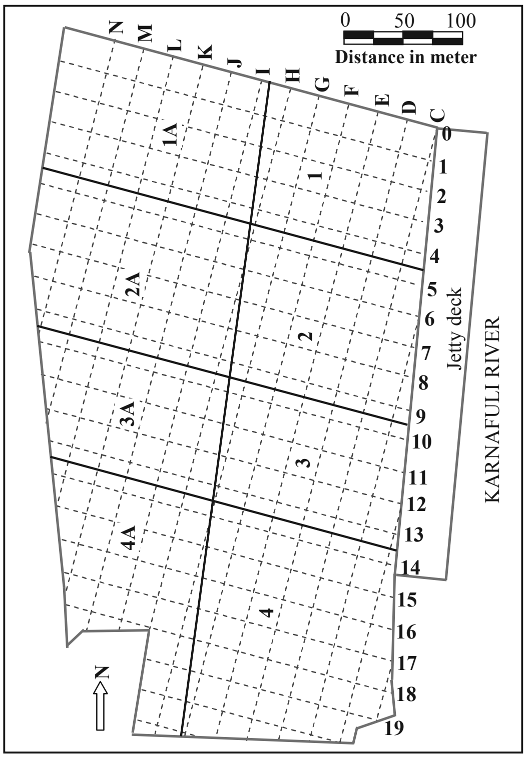


Figure 1 Site map showing grids and loading blocks.

In general, the sub-soil at the site is found to consist of a layer of 'soft to medium stiff silty clay' extending from the ground surface to about 4 to 6 m depth. This layer is underlain by a 10 m thick 'loose to medium dense fine sand/silt' layer. A 'clayey silt' layer is encountered below the fine sand/silt layer which extend beyond the maximum depth of investigation (i.e., about 30 m from surface). Thus, up to a depth of 30 m the sub-soil at the site is idealized to have three distinct layers (top silty clay layer, intermediate fine sand/silt layer and bottom clayey silt layer). In a small number of boreholes medium dense sand was encountered near the ground surface instead of the clay layer, which was probably a fill during past use of the land. Figure 2 presents the field SPT-N values at different depths for the explored borehole locations and the stratigraphy. In the top silty clay layer, field SPT-N values (not corrected for overburden) ranged between 2 to 10. SPT-N values greater than 10 in the top layer are for locations where pockets of sand/silt exist. In the second layer (fine sand/silt), field SPT-N values ranged between 5 and 44. In the bottom layer, which is 'clayey silt', the field SPT-N values ranged between 5 and 27 except in one borehole where a 'sandy silt' layer existed.

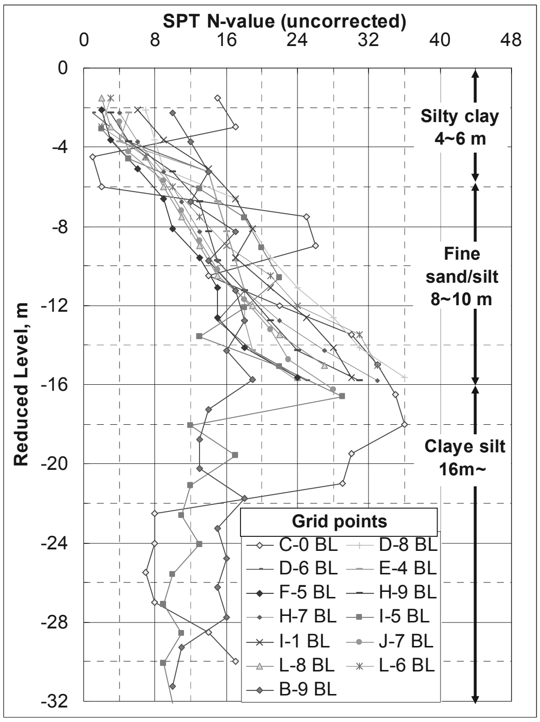


Figure 2 General ground profile and variation of SPT with depth.

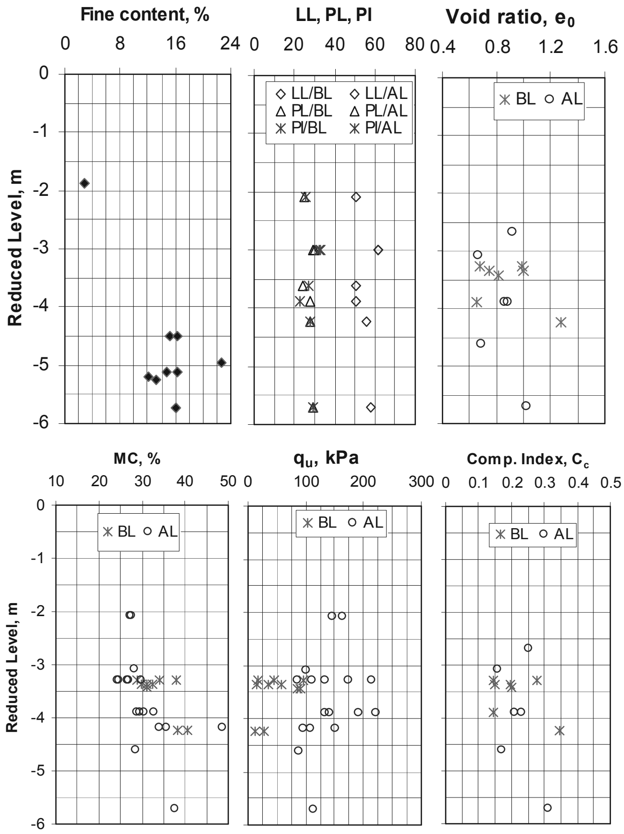


Figure 3 Variation of index, strength and deformation properties of the upper clay deposit with RL (BL=Before loading, AL=After loading).

Extensive laboratory tests have been conducted on samples of top silty clay layer (BRTC, BUET Report, 2009) and some of the results are presented in Figure 3. The layer may be characterized as having LL=50~60, PL=20~30, PI =20~30, NMC=20~35%. According to Unified Soil Classification System (USCS), the soil in this layer is mostly plastic-silty clay of low plasticity (CL), though a few samples were found to be clay of high plasticity (CH). On the USCS chart, the data points lie just above A-line. The dry unit weight varied in the range of 13~15 kN/m$^3$ and the range of void ratio was 0.80~1.20. The variation of these properties can be seen from Figure 3. The layer also has some organic content is about 1.4~4.0%. The values of the coefficient of consolidation in the vertical direction, $C_v$ were mostly within 3.1 to 25.2 m$^2$/year and at some location as low as 0.79 m$^2$/year.

## 4 DESIGN OF GROUND IMPROVEMENT METHOD

It was decided by CPA, that the project will be carried out by local contractors. Therefore, capacity, experience, equipments etc. of local contractors were to be considered in the design of the yard. Furthermore, ground improvement was to be completed for the entire project site within one year. Hence, the area was divided into four blocks and time for improvement for each block was 3 months. An area adjacent to the north boundary of the site was earlier developed for similar purpose, by a foreign contractor, where dynamic temping was used for ground improvement and interlocking block pavement was made. To keep similarity with the earlier part, interlocking block pavement was decided for this yard too.

The presence of very soft to medium stiff silty clay at various locations within the site indicated strong possibility of substantial total and differential settlement unless effective measures for improvement of sub-soil are undertaken before the construction of pavement for the Container Yard. Therefore, effective measures for improvement of sub-soil before the construction of pavement were considered essential in order to avoid/minimize future problems.

The necessity and extent of the ground improvement measures are judged with an objective to reduce the differential settlement and maintenance operations considering the maximum load from stacking of containers on the entire area (i.e. p= 52 kN/m$^2$). It should be understood that a solution, for which there will be no future settlement, will lead to high cost and time for completion and thus may not be practical. The load on the RTG tracks from the gantry is estimated to be 77.5 kN/m$^2$. The extent of improvement and design of pavement system at the site is targeted to keep maintenance option with minimal disruption. For RTG and RMG tracks and other facilities, suitable deep/shallow foundations will be considered so that they do not undergo relative settlement with respect to jetty top.

Five alternatives, that appeared to be feasible for local contractors, were assessed. These are- (i) preloading (ii) sand drain with surcharge (iii) PVD with surcharge and (iv) dynamic temping and (v) soft pocket identification, removal and compacted backfilling. Table 1 presents the comparison of cost and completion time for different methods. Both time and cost depends to some extent on the number of equipments mobilized and source of material, particularly the surcharge (max. 5 m of soil considered). Considering the capacity of local contractors minimal engagement of equipments and dredge sand from the Karnaphuli river were considered. Though dynamic temping/compaction appeared to be very prospective in terms of time and cost, it posed the risk of damaging the adjacent yard and structures. Finally, PVD with surcharge was adopted as the ground improvement measures, mainly because of reduced time in PVD driving compared to sand drain installation, though PVD is an imported material. Also this method was considered advantageous over other methods in bringing the clay layer to a state where differential settlement potential will be reduced as it will automatically take care of soft zones and bring the soft and stiff zones to closer soil properties in terms of deformation and strength.

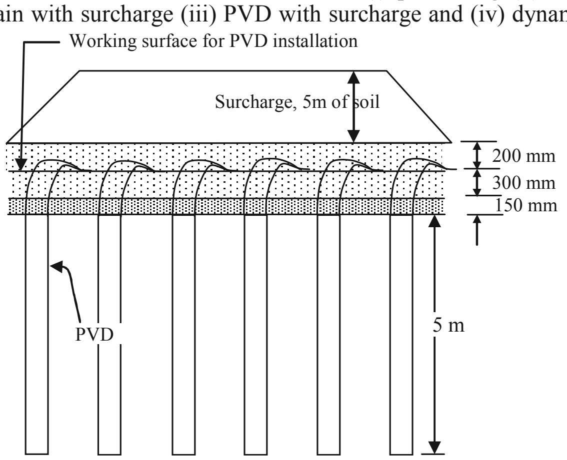


Figure 4 Details of the ground improvement work.

Since, from e~log(p) curves, most of the samples of the upper clay layer was found to be normally consolidated, the total consolidation settlement under the working loads (52 kPa) without improvement was calculated using $S_c=\sum\Delta e.H$, $\Delta e=C_c/(1+e_0)\log(\Delta p+p'_0)/p'_0$ and $p'_0=\sum\gamma' H$ where, $e_0$= initial void ratio, $p'_0$ = effective past maximum overburden pressure, $\gamma'$=effective unit weight of soil, H=thickness of the compressible layer. The estimated settlement for different borehole locations varied from about 140 mm to 570 mm. This variation is due to difference in $e_0$, $C_c$ and layer thickness. In these estimations, $\Delta p$ is calculated as $\Delta p = \sigma_0[1-\{1+(r/z)^2\}^{-1.5}]$ where $\sigma_0$=Intensity of stress applied on the surface, r = radius of the loaded area, $\Delta p$=increase in stress at depth z from the centre of the loaded area. This expression for $\Delta p$ is obtained by integration of Boussinesque's equation that gives the stress at a point within a semi-infinite, homogeneous, isotropic, weightless, elastic half-space for a point load on the surface (Bowels,1988). Estimated time to achieve this consolidation (Uav≈ 99%) varied from about 50 days to more than 700 days for different borehole locations. The time was determined using Terzaghi's one dimensional consolidation theory with double drainage and constant initial pore pressure distribution using the equations (Das, 1983):

$$U_{av} = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M^2} e^{-M^2 T_v} \ , \ T_v = \frac{C_v t}{H^2} \text{ and } M = \frac{\pi}{2}(2m+1)$$

It was intended to apply a surcharge with PVD such that a maximum of 25 mm of total settlement remains to occur in future under the working loads expecting a differential settlement of not more than 12 mm. Estimation of required time to achieve this level of consolidation was made considering both vertical and radial drainage (Carillo,1942) as $U=1-(1-U_v)(1-U_r)$ where $U_v$ and $U_r$ are the average degree of consolidation respectively for vertical and radial drainage. The average degree of consolidation for radial drainage was calculated using the following as

$$U_r = 1 - e^{\left(-\frac{8T_r}{m}\right)} \text{ where } T_r = \frac{C_{vr} t}{d_e^2}$$

$$m = \frac{n^2}{n^2 - S^2}\ln\left(\frac{n}{S}\right) - \frac{3}{4} + \frac{S^2}{4n^2} + \frac{k_h}{k_s}\left(\frac{n^2 - S^2}{n^2}\right)\ln(S)$$

$$n = \frac{d_e}{d_w} \text{ and } S = \frac{d_s}{d_w}$$

The equivalent diameter of PVD was calculated following Hansbo (1979) as

$$d_w = \frac{2(b+t)}{\pi}$$

where, b is the width and t is the thickness of PVD. Considering smear effect S was chosen between 1.0 and 1.2. The effective diameter of soil column around the PVD was taken as $d_e$=1.06s, s=PVD spacing in triangular pattern.

For all the calculations horizontal permeability is taken as the same as the vertical permeability. From the calculations it appeared that for 5 m surcharge and PVD the target settlement would occur within 10 to 50 days in different locations.

Table 1 Comparison of estimated cost and completion time for different ground improvement methods.

| Method | Time (month) | Cost (million USD) | Comments |
|---|---|---|---|
| Preloading | 36 | 1.96 | Reliable<br>Better assessment of improvement<br>Long time required for improvement |
| Sand drain with surcharge | 20 | 3.29 | Relatively less reliable<br>Installation of drains takes long time |
| PVD with surcharge | 14 | 3.24 | Reliable<br>Good control of field operation |
| Dynamic compaction | 11 | 2.04 | Vibration may damage adjacent facilities<br>High noise pollution<br>Assessment of improvement needs lot of field tests |
| Soft pocket identification, removal and improvement, compacted backfilling | 12 | 7.25 | Relatively less reliable<br>Assessment of improvement is difficult<br>Highly dependent on field monitoring and control |

The PVD used were of 100 mm width, 3 mm thick placed 1m c/c in triangular pattern. Other properties - Drain: Water discharge capacity $90\times10^{-6}$ $m^3/s$, and $60\times10^{-6}$ $m^3/s$ respectively at 10 and 350 kPa (i=0.5); Core: Tensile strength 700 N; Filter jacket: Apparent Opening size (AOS) 90 µm, Grab tensile strength 400 N, Elongation at break 50%, Puncture resistance 130 N, Burst strength 800 kN/m$^2$, Permeability $2\times10^{-4}$ m/s. Details of the ground improvement work is shown in Figure 4.

## 5 ASSESSMENT OF GROUND IMPROVEMENT

Monitoring of settlement has been made using settlement plates placed at 25 m grid as shown in Figure 1. After preloading exploratory boreholes were made at selected locations with field SPT and laboratory tests were conducted on collected undisturbed samples.

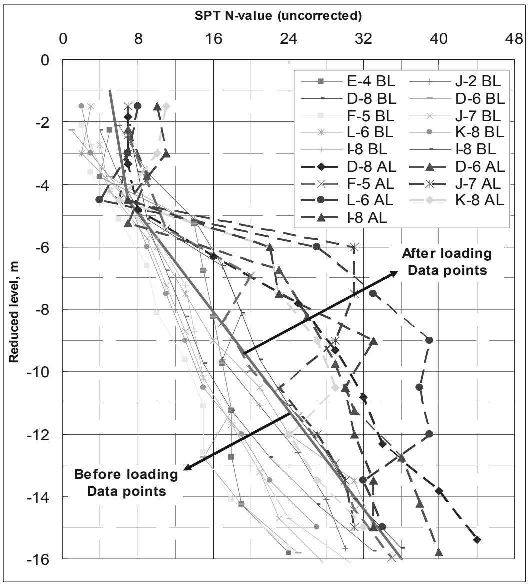


Figure 5. Variation of SPT-N value before and after loading.

Figure 5. compares the Field SPT-N values at several spots before and after preloading. It can be observed that in the upper silty clay layer the SPT-N values has become twice or more up to about 3m depth. At about 4~5m, which is the boundary between the clay and sand/silt layer the SPT-N values have not changed. The field SPT-N values are found to increase significantly in the 'fine sand/silt' layer up to about 12 m.

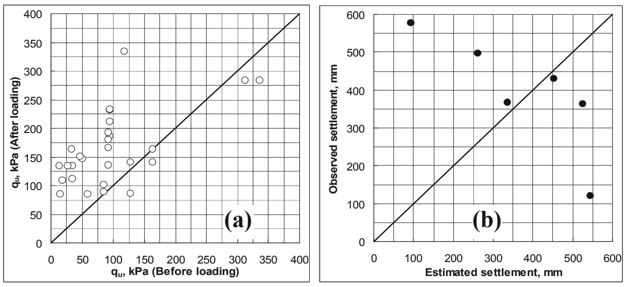


Figure 6 Comparison of (a) unconfined compressive strength and (b) observed and estimated settlement .

The unconfined compressive strength of the upper clay layer before and after preloading can be seen on Figure 6(a) for different locations and depths. In general the unconfined compression strength has increased at most of the spots. However, the magnitude of increase is not same. A few data points lie below the 45 degree line, that apparently shows to have reduction in strength but quite unlikely. The reason for these discrepancies may be the variation of non-plastic silt content in the layer.

In Figure 6(b) the recorded settlements are plotted against the estimated settlement for some of the grid points. Out of the six locations (for which estimates were made) four appear to match reasonably well. For one location the observed settlement is about five times the estimated value, which may be due to presence of localized sand lenses. On the other hand for another location the observed settlement is one-fifth of the estimated value, which may be due to clogging or disturbance of the clay during drain installation. Conclusive comments regarding the variation may be made when data from the remaining project work become available.

## 6 CONCLUSIONS

The following conclusions can be made based on the design and field monitoring of the ground improvement work:

1) Due to the application of the surcharge with PVD the consolidation settlement could be achieved within the stipulated time.

2) Both the SPT-N value and unconfined compressive strength were found to increase satisfactorily due to application of preload with PVD.

3) The available theories of 1-D consolidation and combined vertical and radial consolidation used in the design of ground improvement for the project site using PVD and preload appeared to have been fairly applicable. Predicted and observed settlement matched reasonably.

## 7 ACKNOWLEDGEMENTS

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