Ground Improvement using Pre-loading with Prefabricated Vertical Drains

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ABSTRACT: *A container yard was constructed for handling of loaded containers at Chittagong Sea Port in Bangladesh covering an area of 60700 m^2 over a sub-soil that included a layer of soft clayey silt/silty clay at depths of 0 to 3.5 m below grade. Thicknesses of the soft stratum varied from 3 m to 7 m. Ground improvement using pre-loading with prefabricated vertical drains was undertaken to pre-consolidate the compressible sub-soils, which was followed by field monitoring. It is revealed that the classical theories can effectively be used in calculating the consolidation settlement and the time for consolidation. Predicted settlements and the consolidation time matched reasonably with the measured values. To account for smear effects, the coefficient of consolidation and the coefficient of permeability were taken as those for vertical flow. Predictions with smear diameter equal to two times the equivalent drain diameter provided an upper bound of the consolidation time while prediction without consideration for smear effects provided a lower bound of the consolidation time for the container yard project.*

KEYWORDS: Ground improvement, consolidation, vertical drain, compressible soil, container yard

SITE LOCATION: IJGCH-database.kmz (requires Google Earth)

INTRODUCTION

A container yard has recently been constructed at Chittagong Port, the largest sea port in Bangladesh, for handling loaded containers. The site is located on the bank of the Karnafully river beside the Bay of Bengal in the Indian Ocean. Figure 1 shows the location of the site along with the surrounding geological and geomorphologic features. The yard covered an area of 60700 m^2 (15 acres) and was designed to support a container load producing a contact pressure of approximately 56 kPa. The site is locally known as "Port Park". A comprehensive geotechnical investigation was carried out at the Port Park site to evaluate relevant geotechnical design parameters for the design of the container yard. The investigation revealed the presence of a soft to very soft clayey silt/silty clay layer at depths of 0 to 3.5 m below grade. The thicknesses of the soft layer varied from 3.0 m to 7.0 m. A ground improvement work was designed and carried out to pre-consolidate the soft subsoil before construction of the yard so that the settlements of the yard are minimized during the service life. This paper presents the geotechnical aspects of the design of the ground improvement method, an evaluation of the ground improvement works through field monitoring, and findings from the field monitoring regarding consolidation with pre-fabricated vertical drains.

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Figure 1. Location of the site.

SUBSURFACE CONDITIONS

The site for the container yard (Port Park area) is a tidal plain at a narrow strip between Chittagong hilly uplands and the Bay of Bengal. The surface geology of the site is mainly governed by shallow sea water and the flood plain activities of the river Karnafully and its tributaries. The subsoil includes very soft to firm silty clay or clayey silt and fine grained silty sand with some decomposed materials near the ground surface.

A total of fifteen boreholes were drilled to gather subsoil information for the site, which were distributed over the area. The approximate locations of the boreholes are shown in Figure 2. The boreholes of approximately 125 mm diameter were drilled using water flush aided by chiselling, which were advanced to the depths ranging from 14 m to 24.5 m below ground level. Disturbed and undisturbed soil samples were collected from different depths of the boreholes. A split-spoon sampler was used to obtain the disturbed samples during Standard Penetration Tests (SPT). Undisturbed samples were retrieved from cohesive layers by pushing conventional 76 mm external diameter thin-walled Shelby tubes.

The method of the geotechnical investigation was chosen based on the technology locally avaiable in Bangladesh. It is to be noted that Cone Penetration Test (CPT) equipment with the piezocone probe is not readily available in Bangladesh. Shelby tube samplers were used to collect undisturbed soil samples for this project since Osterberg or other piston samplers were not available to the drilling contractor. However, the authors have examined all shelby tube and split-spoon samples visually and carefully to identify the presence of any localized features (i.e. sand seams etc.) and the consistency of the cohesive soil. An extensive laboratory investigation was then carried out for identification of soil and for determination of geotechnical design parameters. The results of SPT were not directly used in the analysis and design.

Figure 3 shows a general subsurface condition obtained from the geotechnical investigation. Ground condition at the site was found to vary widely from borehole to borehole. Generally, the soil at the ground surface was fill materials consisting of light brown clayey silt or brown silty sand/sandy silt. The clayey silt was firm to stiff. The silty sand or sandy silt was medium dense. The fill materials extended from the ground surface and continued down to depths of 0 to 3.5 m below existing grade.

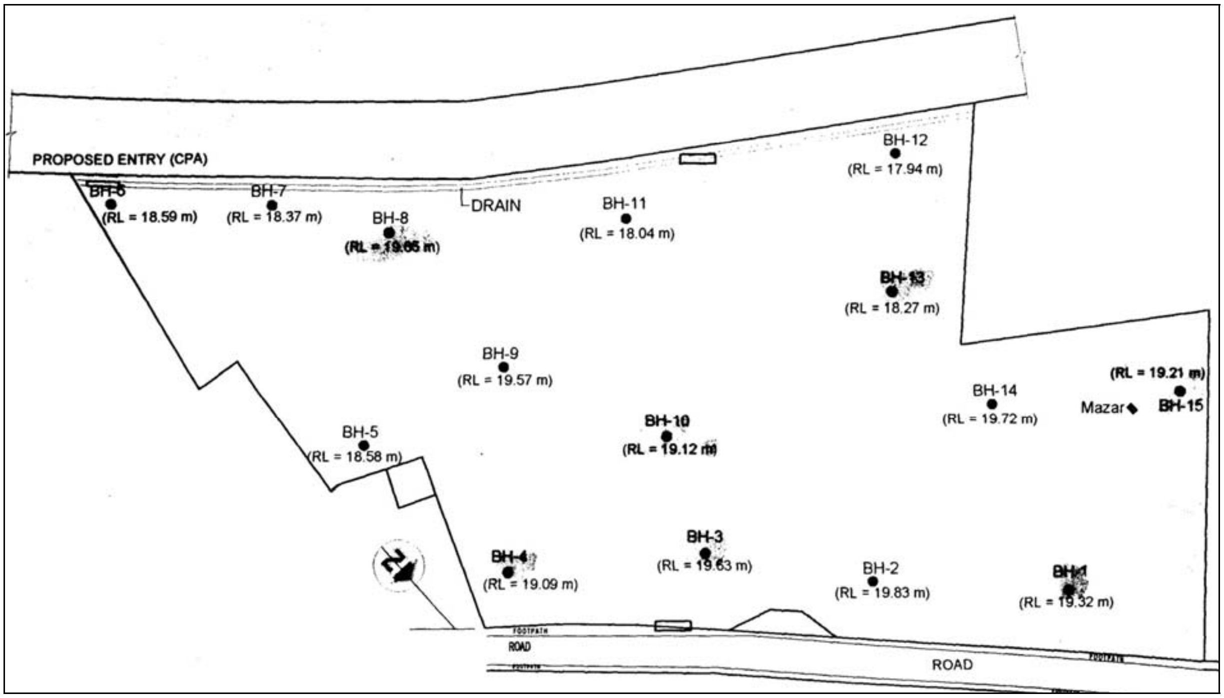

Figure 2. Approximate locations of boreholes.

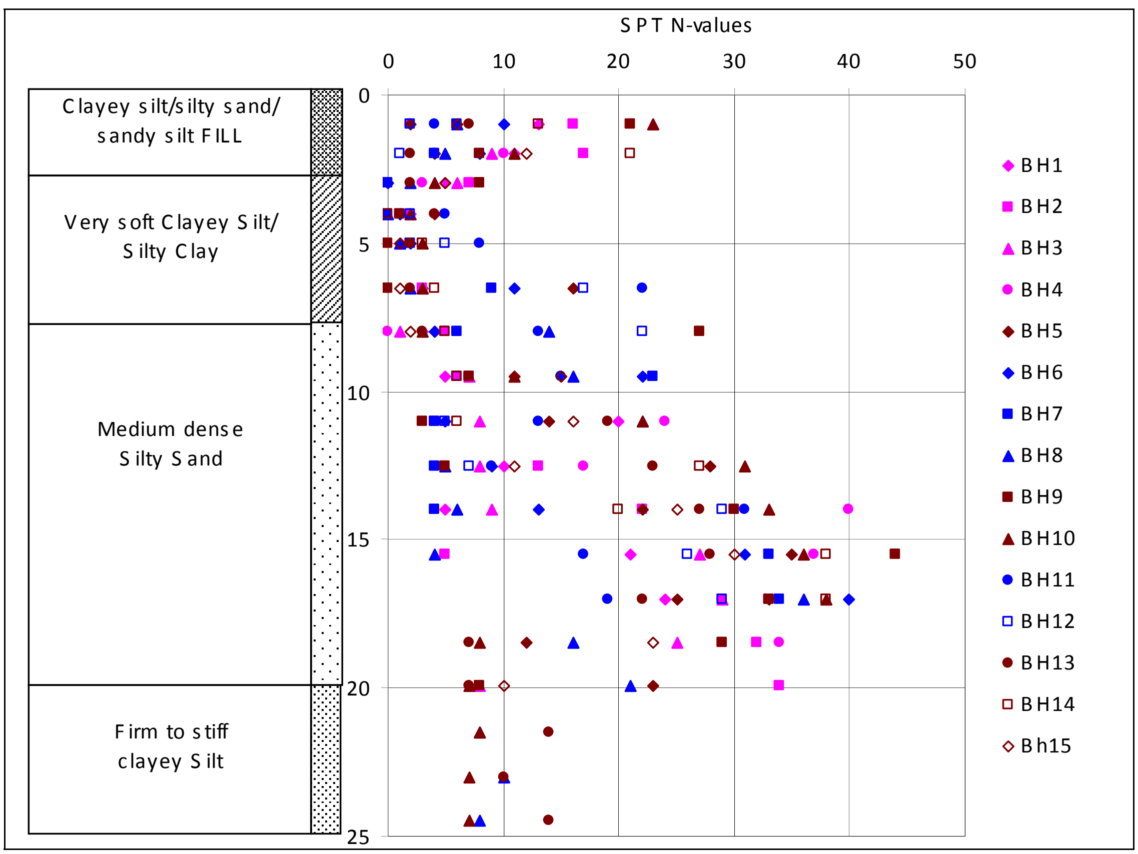

Figure 3. General ground profile along with SPT N-values.

A layer of “very soft” to “soft” cohesive silty clay/ clayey silt was encountered underneath the fill materials. The thickness of the clayey silt/silty clay varied from 3.0 m to 7.0 m and extended down to depths of 7.0 m to 8.5 m below ground surface. SPT N-values encountered in the silty clay/clayey silt layer was as low as zero. This very soft deposit was compressible and would cause excessive settlement to the container yard under service loads. The silty clay/clayey silt were also encountered scattered at different depths in few boreholes.

The soft to very soft cohesive layer was underlain by a silty sand deposit which was subsequently underlain by a layer of silt and/or silty sand. The silty sand varied widely from “loose” to “dense”, but can generally be described as “medium dense” based on SPT N-values. Ground water at the site was located at depths of 0.3 m to 3.4 m below ground level.

LABORATORY TESTS

Laboratory tests were carried out to classify the soil obtained from the boreholes and to obtain geotechnical design parameters for the design of the container yard. Table 1 shows a summary of the laboratory test program. Index property tests were conducted for classification of the soil according to the Unified Soil Classification System (USCS). Shear strengths of the cohesive soil were determined using Unconsolidated Undrained and Consolidated Undrained Triaxial Tests. One-dimensional consolidation tests were used to determine the consolidation properties. For the non-cohesive soil, grain size analysis and consolidated drained direct shear tests were conducted.

Table 1. Laboratory test program.

Borehole No.	Sample Depths (m)							
	Sp. Gr. Test	Atterberg Limit Test	Wash Sieve Analysis	Laboratory Vane Shear Test	Triaxial Compression Test		1-D Cons. Test	CD Direct Shear Test
					UU	CU		
BH-1	3.10	3.10	10.55	-	-	3.10	3.10 H&V*	-
BH-2	-	-	-	4.05, 19.15	-	-	-	-
BH-3	-	4.10	15.05	4.10	-	4.10	-	-
BH-4	6.80	6.80	-	6.80	-	6.80	6.80	-
BH-5	4.10	1.10, 4.10	-	1.10, 4.10	4.10	-	4.10	-
BH-6	-	-	12.05	-	-	-	-	-
BH-7	3.10	3.10	-	11.3	3.10	-	3.10	-
BH-8	-	3.10	10.55	3.10, 14.3	3.10	-	-	-
BH-9	4.10	4.10	13.55	4.10	4.10		4.10	12.05 to 15.50
BH-10	-	-	12.05	2.10, 20.30	2.10	-	-	
BH-11	3.10	-	-	3.10	-	3.10	3.10	-
BH-12	2.10	2.10	13.55	12.80	-	2.10	2.10 H&V	-
BH-13	5.30	2.10	-	2.10, 5.30, 18.80	-	5.30	5.30	-
BH-14	4.10	4.10	12.05	-	-	4.10	4.10 H&V	12.05 to 15.50
BH-15	-	4.10	13.55	14.10	4.10	-	-	

* H: specimen cut along horizontal direction, V: specimen cut along vertical direction.

Physical and Index Properties

Table 2 shows the results of index and physical property tests for the cohesive soil. The water contents of the samples were high, and ranged between 30% and 57%. The values of liquid limit of the samples obtained from the boreholes varied between 32 and 57, with the average value as 45. Plasticity index of the samples were between 9 and 25, with the average value as 18. The water contents were thus close to or greater than the liquid limit in most cases, justifying the low SPT N-values observed during the field tests. Specific gravity of the soil solids was found to vary between 2.71 and 2.77 for the cohesive soil. Bulk unit weight of the soil varied from 20 kN/m^3 to 23 kN/m^3.

Table 2. Summary of index properties and classification of cohesive soil samples.

Borehole No. / Sample No.	Depth (m)	Natural Water Content (%)	G_s	LL	PL	PI
BH-1 / UT-1	3.10 - 3.55	30.8 - 36.6	2.75	45	26	19
BH-3 / UT-1	34.10 - 4.55	37.4 - 44.9	-	47	28	19
BH-4 / UT-1	6.80 - 7.25	34.8 - 40.3	2.71	32	23	9
BH-5 / UT-1	1.10 - 1.55	35.8	-	56	31	25
BH-5/ UT-3	4.10 - 4.55	48.1 - 53.9	2.74	43	27	16
BH-7 / UT-1	3.10 - 3.55	45.8 - 55.8	2.75	43	28	15
BH-8 / UT-1	3.10 - 3.55	37.9 - 40.4	-	48	29	19
BH-9 / UT-1	4.10 - 4.55	42.4 - 54.1	2.74	47	27	20
BH-12 / UT-1	2.10 - 2.55	44.3 - 57.3	2.77	51	30	21
BH-13 / UT-1	2.10 - 2.55	36.5	-	41	25	16
BH-14 / UT-1	4.10 - 4.55	40.4 - 50.1	2.76	44	27	17
BH-15 / UT-1	4.10 - 4.55	37.2 - 39.5		42	27	15

Figure 4 shows the data points from the Atterberg limit tests of the cohesive soil with respect to the A-line of the Unified Soil Classification System (USCS). The data points appear to lie almost on the A-line, the boundary between clay and silt. The liquid limits for the samples were less than 50 in most cases, except for two. Thus, based on the results of the index property tests, the subsoil in the layer can generally be described as low plasticity clayey silts or silty clay (ML or CL according to USCS).

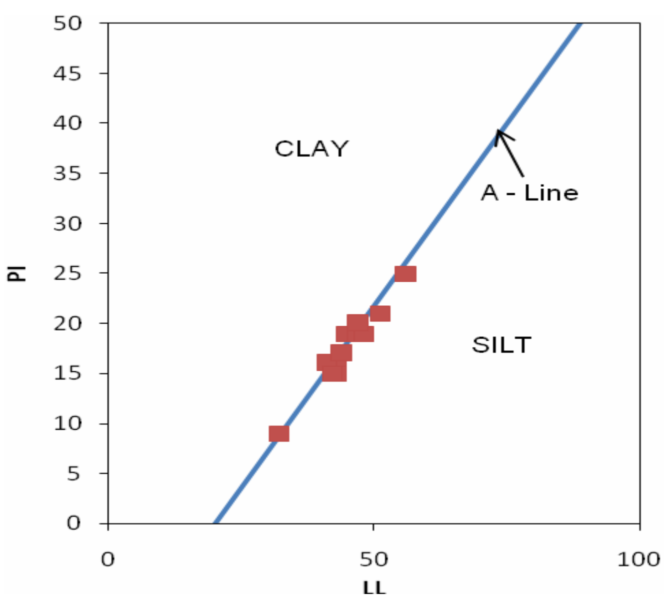

Figure 4. Data points with respect to A-line.

Undrained Shear Strengths

Laboratory vane shear tests and unconsolidated undrained (UU) triaxial tests were conducted for determination of undrained shear strength of the silty clay/clayey silt. A Pilcon Hand Vane Tester was used in the Shelby tube samplers with either a 19 mm diameter vane or a 33 mm diameter vane depending on the consistency of the samples. UU triaxial compression tests were carried out on six samples from Shelby tubes.

Undrained shear strengths obtained from vane shear and UU triaxial tests are summarized in Table 3. The undrained shear strengths, C_u, from the vane shear tests are similar to those obtained from the UU triaxial tests in Table 3, indicating that the vane shear apparatus is an effective tool for quick evaluation of the undrained strengths. Undrained shear strengths from the vane shear tests were between 9 kPa and 56 kPa, with the average value equal to 26 kPa. The average undrained shear strength from the UU Triaxial tests was 30 kPa. Water contents of the soil samples varied between 26.6% and 51.6%. The variation of the shear strengths, as obtained from the laboratory tests, is attributed to the variation of the water contents. The undrained shear strength is plotted against the water contents in Figure 5 for the vane shear and UU triaxial tests. Figure 5 reveals the decrease of shear strength with increase of water content. Based on the values of undrained shear strength, the shear strength consistency of the sub-soil can generally be described as "very soft" to "soft".

Table 3. Undrained shear strengths from laboratory vane shear and UU triaxial tests.

Borehole No.	Depth (m)	Vane shear		UU Triaxial		Consistency
		Water cont. (%)	C_u, kPa	Water cont. (%)	C_u, kPa	
BH-2	4.05 - 4.50	44.9	21			Soft
BH-2	19.15 - 19.50	36.6	32			Soft
BH-3	34.10 - 4.55	37.4	28			Soft
BH-4	6.80 - 7.25	44.9	12			Very Soft
BH-5	1.10 - 1.55	35.8	52			Firm
BH-5	4.10 - 4.55	48.1	16	48.8 - 51.6	15	Very Soft
BH-7	11.3 - 11.75	38.3	15			Very Soft
BH-7	3.10 - 3.55			45.8	7	Very Soft
BH-8	3.10 - 3.55	39.3	30	40.4	30	Soft
BH-8	14.3 - 14.75	37.6	19			Very Soft
BH-9	4.10 - 4.55	51.1	9	43.2 - 43.5	13 - 18	Very Soft
BH-10	2.10 - 2.55	34.9	56	34.1 - 35.1	99 - 102	Firm to stiff
BH-10	20.30 - 20.75	38.7	36			Soft
BH-11	3.10 - 3.55	31.8	17			Very Soft
BH-12	12.80 - 13.25	50	14			Very Soft
BH-13	2.10 - 2.55	36.5	20			Soft
BH-13	5.30 - 5.75	36.6	18			Very Soft
BH-13	18.80 - 19.25	26.6	50			Firm
BH-15	4.10 - 4.55	39.4	22	37.2 - 39.5	21 - 27	Soft

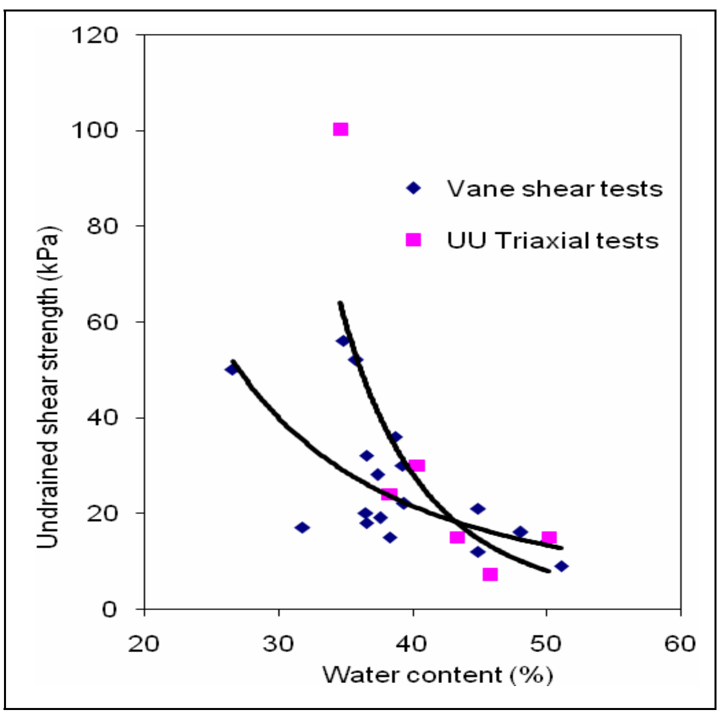

Figure 5. Undrained shear strength versus water content.

Compressibility and Permeability Parameters

Nine Shelby tube samples collected from the silty clay or clayey silt layer were selected for determination of the compressibility and permeability properties using one-dimensional consolidation tests. Coefficient of permeability of the samples was determined indirectly from the results of one-dimensional consolidation tests as (Terzaghi, 1943):

$$k = C_v \gamma_w m_v \quad (1)$$

where C_v is the coefficient of consolidation, γ_w is the unit weight of water and m_v is the coefficient of volume compressibility.

Consolidation tests were carried out on samples of 63.5 mm diameter and 25 mm height using a stress increment ratio of 1 (i.e., a load ratio of 2). Test specimens were cut along horizontal and vertical directions from the Shelby tube samples to determine horizontal and vertical consolidation properties, respectively, using traditional one-dimensional consolidation tests. Nine specimens were prepared along vertical direction and three specimens were prepared along horizontal direction and then tested in the consolidation cells.

A typical void ratio (*e*) versus effective vertical stress (*p*) plot and a plot of the coefficient of consolidation (C_v) against the effective vertical stress from the consolidation tests are presented in Figure 6. Compression index (C_c) and swelling index (C_r) were determined from the slopes of the loading and unloading portions, respectively, of the *e*-log*p* curves. Table 4 presents a summary of the results from the consolidation tests.

The values of C_c from twelve tests were between 0.17 and 0.45 with the average value equal to 0.3. The recompression index, C_r, was calculated to be between 0.05 and 0.07. The initial void ratios (e_0) of these samples varied from 1.04 to 1.62 with the average of 1.28. Pre-consolidation pressures calculated using the Cassagrande method were found to range between 30 kPa and 50 kPa, which are less than ground stresses expected under the container load of 56 kPa.

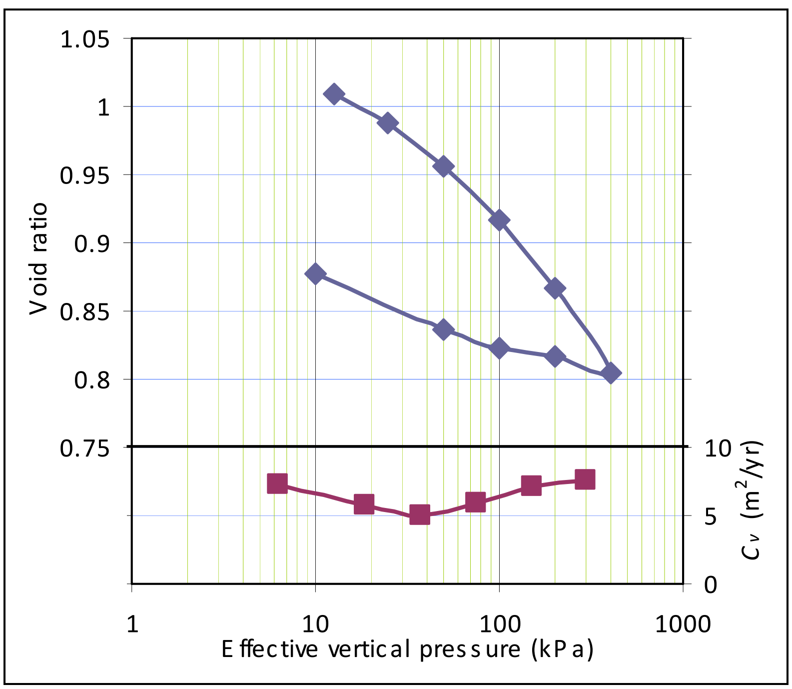

Figure 6. Typical void ratio and coefficient of consolidation versus effective stress.

The coefficients of consolidation from the tests were found to vary widely from the consolidation tests. The coefficient of vertical consolidation (C_v) from nine specimens varied between 2 m^2/year and 21 m^2/year. The coefficient of horizontal consolidation (C_h) from the three specimens ranged from 12 m^2/year to 70 m^2/year. The ratios of the horizontal to vertical coefficient of consolidation, C_h/C_v, were between 1.2 and 5.0.

Table 4. Summary of one-dimensional consolidation test results.

Borehole / Depth	W (%)	e_0	C_c	C_v (m^2/yr)	C_h (m^2/yr)	k_v (m/year)	K_h (m/year)
BH-1 / UT-1 3.10 - 3.55 m	36.5	1.04	0.22	13-59	13-70	0.038 to 0.694	0.054 to 1.513
BH-4 / UT-1 6.80 – 7.25 m	39.6	1.10	0.17	18-56	-	0.076 to 0.0631	-
BH-5 / UT-3 4.10 - 4.55 m	53.9	1.48	0.44	4-8	-	0.0191 to 0.0192	-
BH-7 / UT-1 3.10 - 3.55 m	55.8	1.62	0.44	7-12	-	0.024 to 0.264	-
BH-9 / UT-1 4.10 - 4.55 m	54.1	1.48	0.37	2-4	-	0.012 to 0.158	-
BH-11 / UT-1 3.10 – 3.55 m	33.9	1.04	0.19	23-76	-	0.060 to 1.167	-
BH-12 / UT-1 2.10 - 2.55 m	57.3, 49.4	1.53, 1.38	0.4, 0.40	4-6	12-19	0.016 to 0.180	0.047 to 0.002
BH-13 / UT-2 5.30 – 5.75 m	41.1	1.13	0.29	5-8	-	0.017 to 0.164	-
BH-14 / UT-1 4.10 - 4.55 m	45.8, 40.4	1.27, 1.20	0.31, 0.27	6-21	31-57	0.019 to 1.010	0.066 to 2.302

The variations of the coefficient of vertical consolidation with the average effective stress for each of the samples are plotted in Figure 7. As seen in the figure, the magnitudes of the C_v were high at the low stress levels that corresponded to the recompression range (where the stresses were less than the pre-consolidation pressure). These high C_v may be associated with the low volume compressibility, m_v of the soil in the recompression range. In the compression range (where stress was greater than the pre-consolidation pressure), C_v values were generally less associated with the high coefficient of volume compressibility. The point of abrupt decrease in C_v in Figure 7 corresponds to the pre-consolidation pressure for the samples. The pre-consolidation pressure on the basis of this reduction in value of C_v ranges from 20 kPa to 75 kPa (Figure 7), which is similar to that obtained from the Cassagrande method. Since the pre-consolidation pressures were less than the ground stresses under the container yard, the coefficient of consolidation in the compression range was used for the design of the yard.

The C_v in the compression range was generally constant in most cases of the consolidation tests conducted (Figure 7). The C_v increased moderately with the increase of stress for few samples. However, the increase of the C_v with the increase of the stress was not considered in the design. The average value of the coefficients in the compression range was used for the design of the container yard. However, the extreme high values (i.e. 56 m^2/ year or 70 m^2/year obtained for the samples from Boreholes 4 and 11) and the extreme low values (i.e. 1.5 m^2/year for the sample from Borehole 9) were neglected for calculation of the average C_v. The design value of the coefficient of vertical consolidation was thus estimated to be 7.5 m^2/year.

Figure 8 plots the coefficient of horizontal consolidation against the effective stresses for the three samples. Horizontal consolidation coefficients were also high in the recompression zone, decreased at the pressures of 25 kPa to 50 kPa, and then gradually increased in the compression range of pressures. C_h in the compression zone varied from 12.5 m^2/year to 30 m^2/year. The average of the C_h in the compression range (neglecting the upper or lower extreme values) was 15.5 m^2/year, which was used in the design of the yard.

Depending on the stress ranges, the values of coefficient of vertical permeability (k_v) of the samples varied from 0.012 m/year to 1.009 m/year and the coefficient of horizontal permeability (k_h) of three samples varied from 0.047 m/year to 2.302 m/year. Figure 9 shows the coefficients of vertical and horizontal permeability plotted for the range of average effective stresses between 10 kPa and 300 kPa. The coefficients of permeability were very high initially in the re-compression zone (for stress < 25kPa) and decreased almost linearly with the increase of the effective stresses in the compression zone. However, a constant value for each of the coefficients was considered reasonable for design purpose. The coefficient of vertical permeability and the coefficient of horizontal permeability were found to range from 0.032 m/year to 0.063 m/year and from 0.047 m/year to 0.095 m/year, respectively, within the range of design stresses (i.e. 60 to 100 kPa). The averages of the ranges were taken as the design values for the coefficients. The coefficients of vertical and horizontal permeability were thus estimated to be 0.047 m/year and 0.073 m/year, respectively, in the design.

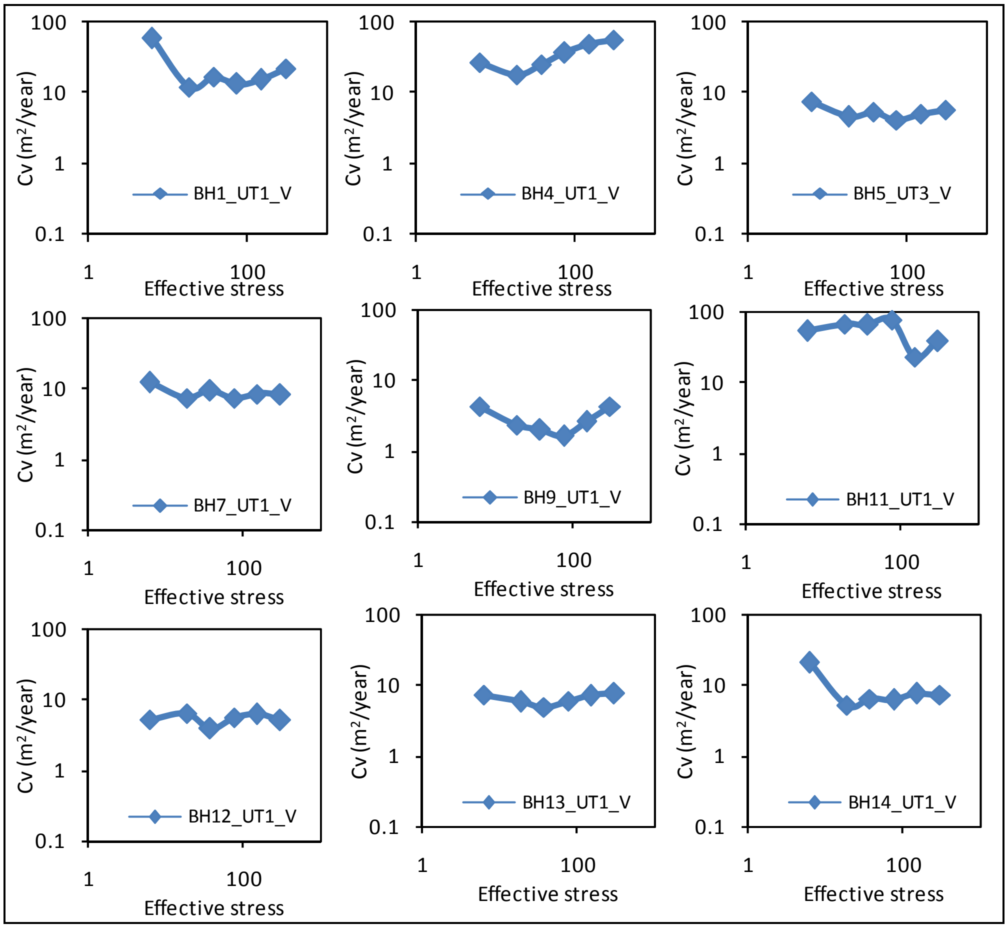

Figure 7. Coefficients of vertical consolidation against effective stresses.

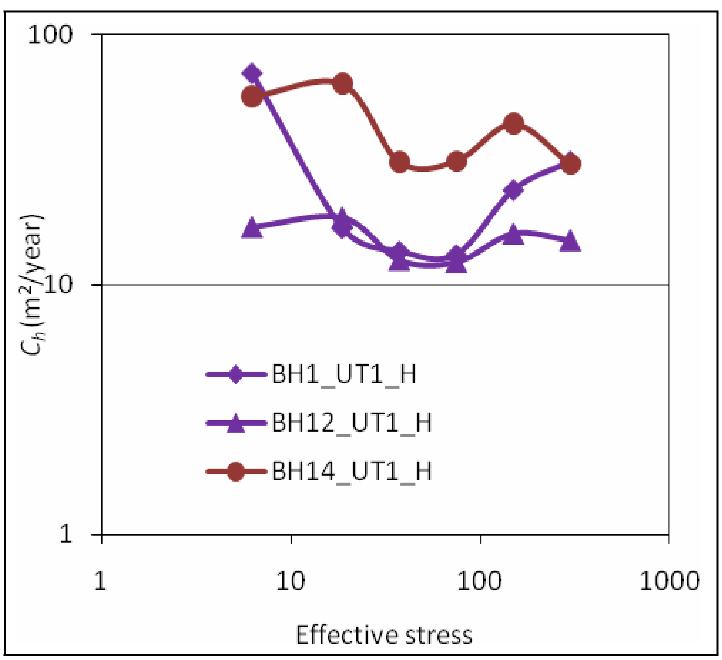

Figure 8. Coefficients of horizontal consolidation versus effective stresses.

CONSOLIDATION SETTLEMENTS

The classical one-dimensional consolidation theory of Terzaghi (1943) was used for calculation of the consolidation settlements due to full design load (i.e. 56 kPa) and the time for consolidation. The one-dimensional consolidation theory expected to work reasonably for a thin layer of compressible soil relative to the loaded area. As discussed earlier, the thicknesses of the soft soil at the site were thin (3 m to 7 m) compared to the area of container yard (60700 m^2).

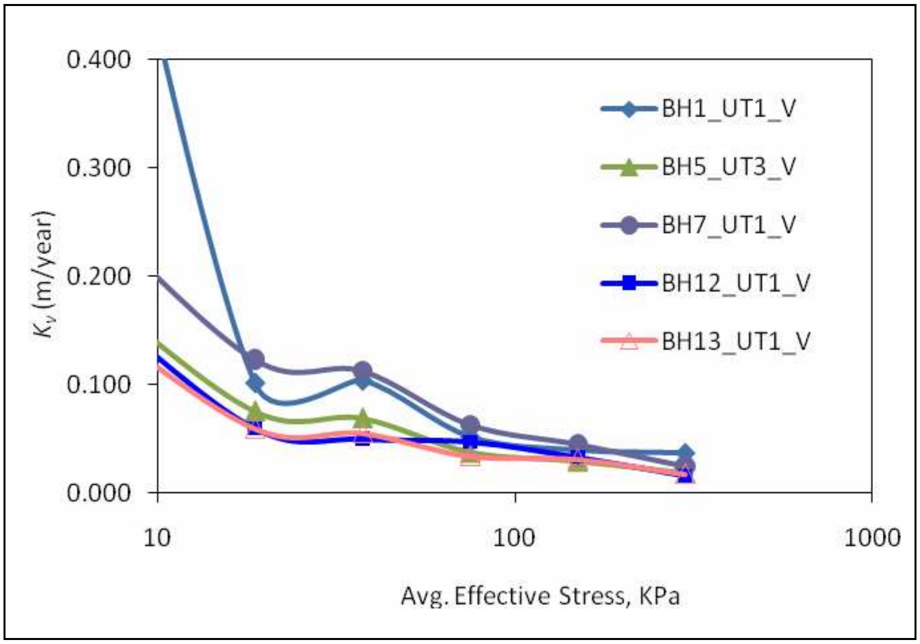

Figure 9. Coefficients of permeability from laboratory tests: (a) Vertical.

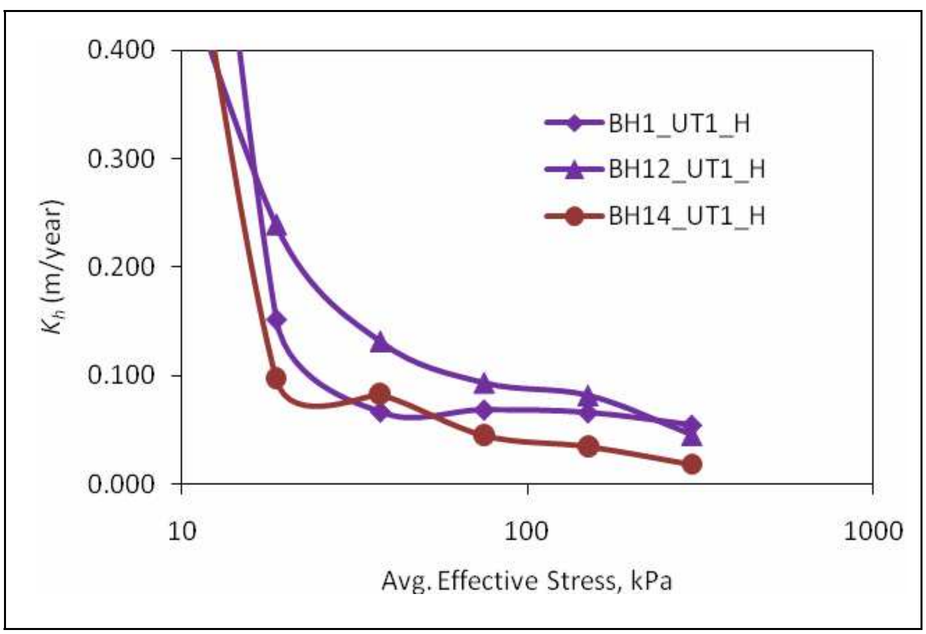

Figure 9. Coefficients of permeability from laboratory tests: (b) Horizontal.

One dimensional consolidation settlement according the classical theory is given by (Terzaghi, 1943):

$$S_c = \frac{C_c}{1+e_0} H \log \frac{p_0' + \Delta p}{p_0'} \tag{2}$$

where, S_c = the consolidation settlement, e_o = initial void ratio; H = thickness of layer; Δp = increase in total vertical stress at the centre of layer; p_o' = effective vertical stress at the centre of layer.

The time for consolidation settlement (t) is given by (Terzaghi, 1943):

$$t = \frac{T_v H^2}{c_v} \tag{3}$$

where, T_v = time factor, H = length of drainage path, c_v = coefficient of consolidation for vertical flow.

The maximum settlements due to the design load of the container yard were calculated to be 450 mm for 7 m thick layer and 200 mm for 3 m thick layer of compressible soil. These settlements are too high from serviceability consideration of the yard. It was therefore considered necessary to pre-consolidate the soil before construction of the yard. Use of pre-loading for pre-consolidation was first considered due to the simplicity of the method and the suitability for implementation using local technology. However, the time required for the consolidation using pre-loading was a major concern in the design of the pre-consolidation. With the consolidation coefficient estimated from the laboratory tests, (i.e. C_v = 7.5 m^2/year), the time required for 90% consolidation was estimated to range from 1 year to 5.5 years for 3 m and 7 m layers of soft soil, respectively. Preloading with vertical drains (sand drains or Prefabricated Vertical Drains) was therefore considered for the design of the yard to accelerate the consolidation process. A brief description of the design of the ground improvement method using preloading with vertical drains is outlined below.

GROUND IMPROVEMENT

Design Assumptions

Vertical drains are used to allow drainage in the horizontal direction over a much shorter drainage path so that consolidation can take place in a shorter period of time. The theory of consolidation by radial drainage and by combined radial and vertical drainage is well documented in the literature (Barron, 1948; Hansbo, 1960). The effects of vertical drain on the consolidation are generally analyzed using an idealized model shown in Figure 10. In this model, the vertical drain is idealized as an equivalent circular drain. An annular zone, called a smear zone, is considered in the soil surrounding the drain to account for the disturbance caused by the installation of the drain. The permeability of the smear zone in the vicinity of the drain is reduced compared to the native soil due to installation disturbance. Several methods are available to account for the smear effects in the design i.e., Yoshikuni and Nakanodo (1974), Hansbo (1981), Xie (1987) and others.

Hansbo (1979) and Holtz et al. (1987) presented the conventional design procedures for vertical drains. For an ideal case of radial drainage, an expression for the average degree of consolidation, U_h, at a certain depth, z is presented as:

$$U_h = 1 - \exp\left(-\frac{8C_h t}{\mu D^2}\right) \tag{4}$$

where,

$$\mu = \ln\left(\frac{D}{d_s}\right) + \frac{k_h}{k_s}\ln\left(\frac{d_s}{d_w}\right) - \frac{3}{4} + \pi z(2l - z)\frac{k_h}{q_w} \tag{5}$$

Here, C_h is the coefficient of horizontal consolidation, t is the time of consolidation, D is the equivalent diameter of the soil cylinder dewatered by a drain, d_w is the equivalent drain diameter, d_s is the diameter of the smear zone, k_h is coefficient of horizontal permeability of the undisturbed soil, k_s is the permeability of the smeared soil, q_w is the discharge capacity of the drain, and l is the maximum discharge length of drain. The equivalent diameter, D depends on the pattern of drain installation. For centre to centre spacing of S between drains, the equivalent diameter is given by D = 1.05S for triangular pattern and D = 1.13S for square pattern of drain installation. The equivalent drain diameter for the drains with rectangular cross-csection is given by (Hansbo, 1979):

$$d_w = \frac{2(a+b)}{\pi} \tag{6}$$

where, a = drain width, b = drain thickness

From Equation (4), the time for consolidation can be expressed as:

$$t = \frac{\mu D^2}{8C_h}\ln\left(\frac{1}{1-U_h}\right) \tag{7}$$

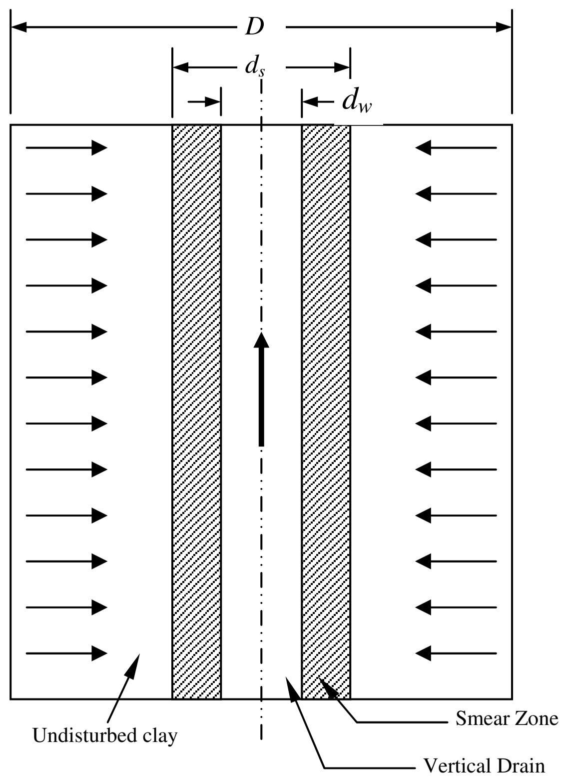

Figure 10. Idealization of consolidation using vertical drain.

The method of Hansbo (1979, 1981) is widely used in the design of consolidation with vertical drains. Xiao (2001) presented an evaluation of the Hansbo method using elasto-plastic finite element analysis and reported that the method provided a good estimation of the degree of consolidation. The Hansbo method was used in the design of vertical drains for the container yard project at Chittagong port. Parameters for the native soil were used as those determined from the laboratory investigations, discussed earlier. However, assumptions were made regarding the smear zone and the smear effects for use in the equations (Equations 5 and 7).

It is generally very difficult to quantify the extent of smear zone (i.e. d_s) and the smear effects on the soil properties. Several studies were conducted for determination of the smear zone and the smear effects for consolidation with vertical drains. Hansbo (1981 and 1997) estimated the diameter of smear zone, d_s as 1.5 to 3 times the diameter of the drain, d_w. Bergado et al. (1991) proposed to assume smear diameter as 2 times the diameter of the drain. However, Indraratna and Redna (1998), Bo et al. (2000) and Xiao (2001) indicated that the smear zone diameter can be as high 4 to 8 times the diameter of the drain. The upper bound value of Hansbo (1981), i.e. $d_s = 3\ d_w$ was chosen to examine the smear effect in the design of the container yard.

Smear effects can significantly reduce the permeability and the coefficient of consolidation. The effect on the coefficient of permeability is generally considered as the reduction ratio with respect to the coefficient of horizontal permeability, k_h/k_s. Researchers suggested using a value of the reduction ratio in the range of 2 to 6 (Hansbo, 1981; Onoue, 1992; Indraratna and Redna, 1998; Hird and Moseley, 2000). Hansbo (1997) proposed to use the coefficient of permeability for the smear zone, k_s as same as the coefficient of vertical permeability, k_v. Following Hansbo (1981), the value of $k_s = k_v$ was used for the smear effect in the design for the container yard project.

Balasubramaniam et al. (1995) and Chu et al. (2002) determined C_h including smear effects based on back-calculation of field settlement data. They reported that C_h value can be less than the C_v due to the smear effect. However, $C_h = C_v$ was assumed for the design of the ground improvement method for this project.

The last term in Equation 5 account for the drainage congestion (well resistance) for the case when the drain does not have sufficient discharge capacity. However, the vertical drain commonly used has sufficient discharge capacity and thus the term can normally be neglected (Chu et al., 2004; Rajikiatkamjorn and Indraratna, 2010). Discharge capacity of the vertical drain with respect to the horizontal hydraulic conductivity of the soil for the Container Yard Project at Chittagong Port also revealed that the contribution of well resistance in Equation 5 was negligible.

The presence of thin drainage seams or layers within the silt/clay formation, if any, may accelerate significantly the consolidation process (Gibson and Shefford, 1968; Abid and Pyrah, 1990). The presence of such drainage layer was however not considered during design in order to obtain the upper bound value of the time for consolidation. A settlement monitoring program was then considered to observe consolidation with time so that the effects of acceleration (or deceleration) of consolidation can be incorporated during the construction (surcharge can be removed whenever the consolidation is completed).

Design of Vertical Drains

Soil improvement works for the Port Park Area of Chittagong Port was designed based on the information on geotechnical profiles and the results from laboratory tests. Maximum thickness of the soft soil layer (i.e., 7 m) was conservatively considered for the design. Vertical drains were designed to install down to the depth of approximately 9 m below the ground level to cover the full depth of the soft clay layer.

Table 5 shows three different options of vertical drains initially examined. Time for 90% consolidation with 200 mm diameter sand drains at 1.5 m center-to-center (c/c) spacing in square pattern was calculated to be 80 days, which is significantly less than the estimated period of consolidation without vertical drains (1 year to 5.5 years, as described earlier). The times for 90% consolidation with prefabricated vertical drains (PVDs) at 1.0 m and 1.5 m spacings were calculated to be 48 days and 125 days, respectively. Approximate costs for each of the options are also shown in Table 5. Based on the comparison, PVDs (width = 100 mm, t = 4 mm) at 1.0 m c/c in a square pattern was chosen for implementation by the project owner (Chittagong Port Authority) due to the lowest consolidation time.

Table 6 shows the physical, mechanical, and hydraulic properties of the pre-fabricated vertical drain specified in the design, based on the PVDs available in the South Asian market. Major design requirements for the PVDs are discharge capacity, strength, and the apparent opening size (AOS). Discharge capacity of the drain should be large enough to ensure efficiency of the drain and to avoid drainage congestion. Xie (1987) established that drainage congestion in vertical drain can be ignored if the following condition is met.

$$\frac{\pi}{4}\frac{k_h}{q_w}l^2 \le 0.1 \tag{8}$$

For the Port Park Container Yard project at Chittagong Port, the required discharge capacity was calculated to be $q_w > 28.0$ m^3/year using Inequality (8) and $k_h = 0.073$ m/year and $l = 7.0$ m. The discharge capacity specified in Table 6, based on the commonly available PVDS, is much greater (100 times) than the required value.

Apparent opening size (AOS) of the filter should be large enough to provide sufficient permeability, yet small enough to prevent the fine particles of the soil from entering the filter and the drain. The permeability of the filter is generally expected to be larger than the permeability of the surrounding soil by at least one order of magnitude from the consideration of clogging effects (Chu et al., 2004). Permeability for the PVD was specified to be 6310 m/year, based on the information of most PVDs available in South Asia. This value is almost 105 times greater than the permeability of the soil (k_h = 0.073 m/year).

Table 5. Approximate cost and consolidation time for three options of soil improvement.

Option	Consolidation Time (days)	Approximate Cost (Taka/ft^2)
(1) 200 mm diameter sand drain @ 1.5 m c/c in a square pattern	80	50
(2) PVDs (width = 100 mm, t = 4 mm) @ 1.0 m c/c in a square pattern	48	65
(3) PVDs (width = 100 mm, t = 4 mm) @ 1.5 m c/c in a square pattern	124	30

Table 6. Physical, mechanical and hydraulic Properties of PVD.

Properties	Quantifier	Specified Value
Drains		
Weight per unit length	Minimum	70 gm/m
Width	Minimum	100 mm
Thickness	Minimum	4 mm
Water discharge capacity	Minimum	2840 m^3/year
Core		
Tensile strength	Minimum	750 N
Filter Jacket		
Apparent opening size (AOS)	Maximum	90 μm
Grab tensile strength	Minimum	400 N
Elongation at break	Minimum	50 %
Puncture resistance	Minimum	130 N
Burst strength	Minimum	800 kPa
Permeability	Minimum	6310 m/year

To prevent the penetration of fine particles, commonly used design criteria are (Carroll, 1983):

$$O_{95} \le (2\text{-}3)\, D_{85} \quad (9)$$

and

$$O_{50} \le (10\text{-}12)\, D_{50} \quad (10)$$

where O_{95} is the AOS of filter, O_{50} is the size which is larger than 50% of the fabric pores, and D_{85} and D_{50} are the sizes for 85% and 50% of passing of soil particles by weight. Based on field observations of PVDs, a more relaxed criterion for O_{95} was found adequate for Singapore and Bangkok clay (Chu et al., 2004; Bergado et al., 1993). For the silty clay or clayey silt encountered at the site of the container yard at the Chittagong Port, the PVDs design criteria of O_{95} < 90 μm were found satisfactory.

PVDs should have adequate strength to sustain the tensile load applied during installation. Therefore the strength of the core, the filter, and the entire drain are generally specified during design of PVDs. Kremer et al. (1983) suggested that a drain must withstand at least 0.5 kN of tensile force along the longitudinal direction without exceeding 10% in elongation. Tensile strength of the core was specified to be 0.75 kN in the current project. Filter jacket was required to have minimum grab tensile strength, puncture resistance, and burst strength as 0.4 kN, 0.13kN, and 800 kPa, respectively.

Surcharge load required for pre-consolidation was estimated to be equivalent to the design load of 56 kPa. Considering unit weight of sand fill as 18 kN/m^3, required height of the surcharge fill was approximately estimated to be 3.0 m.

METHOD OF CONSTRUCTION

The construction of the ground improvement work involved preparation of the existing ground, placement of local sand to raise ground level where required, placement of a drainage blanket of coarse sand, installation of Prefabricated Vertical Drains (PVDs), and then pre-loading. Considering the large area (60700 m^2) of the container yard, the ground improvement work was accomplished in three segments with each segment consisting of approximately 20200 m^2.

An approximately 150 mm thick local sand layer was first placed over the leveled ground after stripping of topsoil and unsuitable materials in order to attain the required grade for the container yard. The local sand had Fineness Modulus (FM) greater than 1.0 and fines content (material passing #200 sieve) less than 3%. The layer of the local sand was compacted using vibratory rollers to obtain a relative density of approximately 85%. A drainage blanket consisting of coarse sand (fineness modulus greater than 2.2) was then placed over the local sand to facilitate draining of water to be collected by the PVDs. Thickness of the drainage blanket layer was approximately 450 mm, which was designed to compensate the settlement expected due to the consolidation. The lower 250 mm of the drainage blanket was placed before installation of the PVDs to provide a working platform for PVD installation. The remainder of the drainage layer was placed after installation of PVDs to allow the drains to discharge into the sand layer. Surface of the sand blanket was adequately compacted using vibratory rollers and then leveled. The degree of compaction of the sand layer was such that the relative density of the compacted sand is at least 85%.

PVD were installed using a mandrel that provided minimum subsoil disturbances. A hollow mandrel or sleeve was advanced through the subsoil using vibratory, constant load, or constant rate of advance methods. The mandrel combined with the anchor had a maximum projected cross-sectional area of 70 cm^2. The anchor was used to remain in place at the bottom of the PVD when the mandrel was removed after installation. As mentioned earlier, the PVD was installed to the depth of 9 m below ground level to cover the full depth of the soft soil.

The remaining 200 mm layer of coarse sand was placed over the finished surface after installation of PVD, which was compacted and leveled for placement of settlement measuring gauges. Thirty settlement measuring gauges were placed to measure the rate and magnitude of the settlements. Figure 11 shows a schematic of settlement gauge along with the approximate locations of the points of settlement measurements. A settlement gauge includes a base plate and a stand pipe. The base plate of the gauge was placed on top of the leveled granular layer, while the elevation of the top of the stand pipe was monitored (using a Surveyor's level) to obtain the ground settlements.

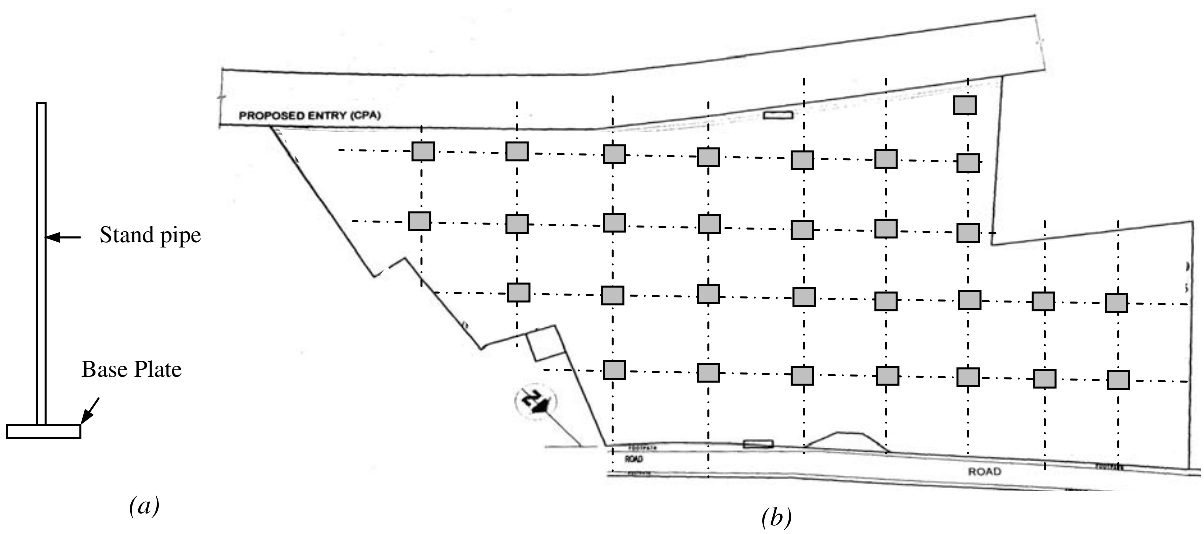

Figure 11. Settlement monitoring program: a) Settlement gauge, b) Points of settlement measurement (schematic).

A surcharge load consisting of 3.0 m high fill of sand was placed over the drainage layer for pre-loading. Surcharge material was placed in two layers of approximately equal thickness. Total area was divided into three segments for placement of surcharge and settlement monitoring as well as for PVD installation so that material from one segment could be reused in the other segment when consolidation in the first segment was completed (established through monitoring time-settlement responses). The sides of the surcharge load were kept vertical along the boundary of the area using sand bags or brick stacks (Figure 12). Figure 12 show a profile detailing schematically the ground improvement work.

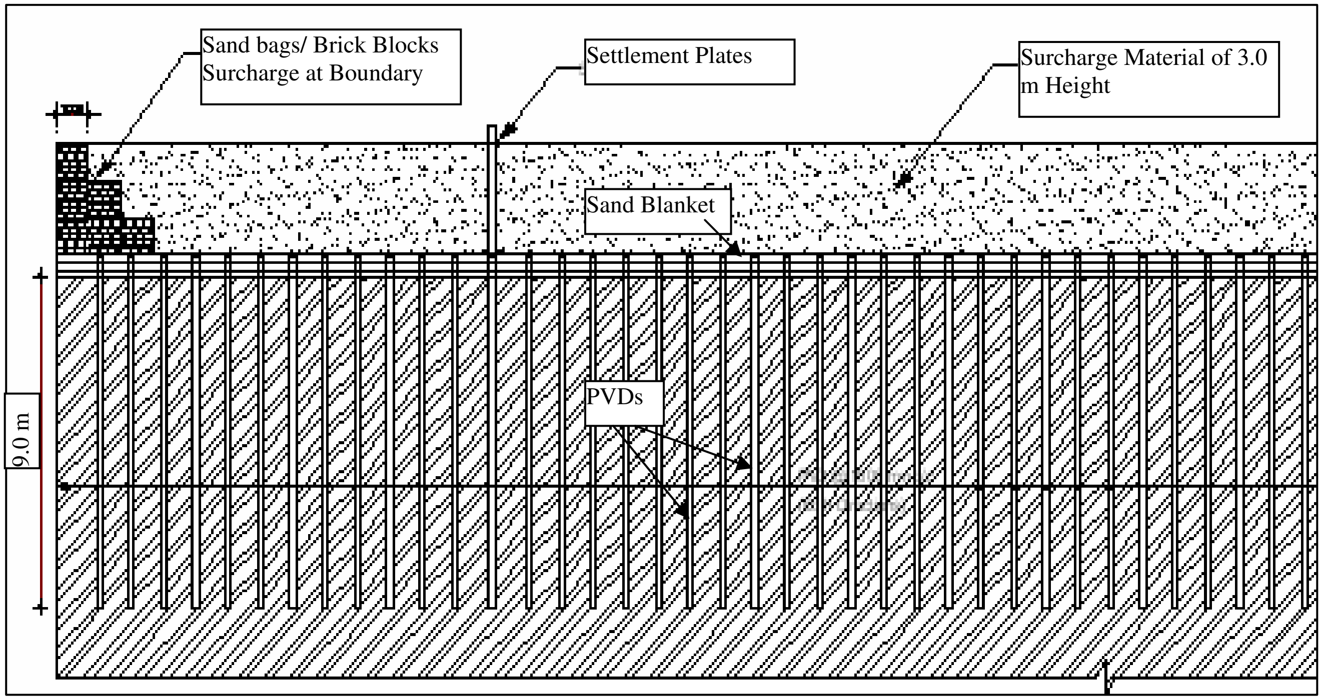

Figure 12. Detail of the ground improvement works (schematic).

MONITORING OF SETTLEMENTS

Ground settlements were monitored using the settlement gauges during consolidation under surcharge pre-load to validate the design assumptions and to ensure pre-consolidation before construction of the container yard. Surveyor's leveling apparatus were used to monitor the movement of the tips of the standpipes of the settlement gauge. Settlement monitoring started immediately after placement of the surcharge to the full height (i.e. 3.0 m) and continued until consolidation was completed. It generally took 10 to 12 days to place the surcharge materials to the full height within a segment of the whole area. Immediately after placing surcharge to the full height within a segment (in 10 to 12 days), the ground settlements were first measured using the settlement gauges within that segment. The first measured settlement for each gauge is termed herein as the "initial settlement". The same procedures of surcharge placement and settlement monitoring were then followed for the rest of the area. As discussed earlier, the whole area was divided into three approximately equal segments for placement of surcharge materials.

Figure 13 shows the results of settlement monitoring at different locations within the area of pre-consolidation. The figure shows initial settlement of 80 mm to 300 mm due to the placement of 3.0 m high surcharge before measurements started. The maximum settlements measured during the monitoring period varied from 220 mm to 415 mm, which are very close to the settlements estimated during design. As discussed earlier, the maximum settlements were estimated to range from 200 mm to 450 mm during design. Thus, the one-dimensional consolidation theory appeared to reasonably estimate the settlements for the 60700 m^2 of loaded area overlying 3.0 m to 7.0 m thick layer of compressible soil. Figure 13 revealed that the settlements were almost completed after 30 to 52 days of preloading. Time of consolidation was estimated to be 48 days during design with the PVDs at 1.0 m centre to centre spacing (Table 5). Thus the estimated time reasonably matched with the observed consolidation period.

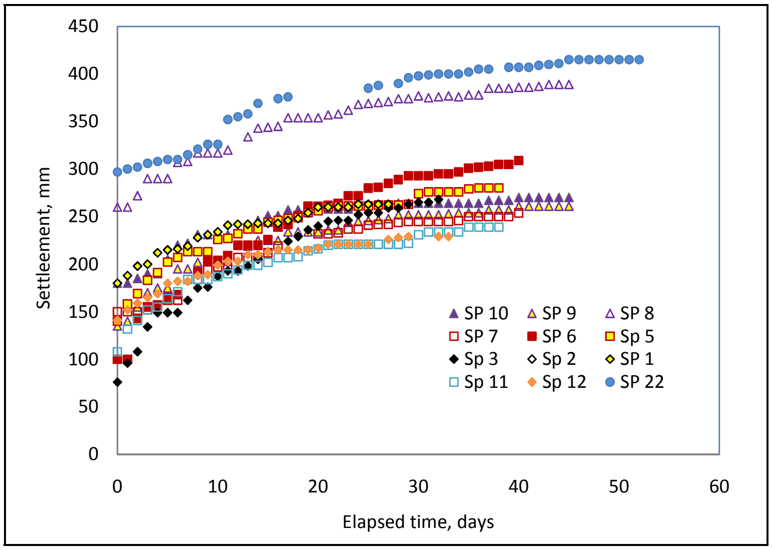

Figure 13. Ground settlements with time.

Figure 14 presents the observed settlement expressed in terms of the degree of consolidation. Predictions of the degree of consolidation with and without considerations of smear zone are also compared in the figure. Figure 14 shows that the prediction without consideration for smear effects provides a lower bound of the consolidation time with respect to the measured t-U relation. The calculation with consideration for the smear effect provided an upper bound of the consolidation time. Diameter of the smear zone was assumed as two times the equivalent drain diameter (i.e. $d_s = 2d_w$) for calculation with smear effects presented in Figure 14. Coefficient of consolidation and the coefficient of permeability with the smear effect were taken as the coefficients of vertical consolidation and vertical permeability, respectively (i.e. $C_s = C_v$ and $k_s = k_v$). Thus, the prediction with $d_s = 2d_w$, was found to provide an upper bound consolidation time. The assumptions of $C_s = C_v$ and $k_s = k_v$ were found reasonable for the container yard project at Chittagong Port. Monitoring of the settlements confirmed the presence of smear effects, leading to the measured consolidation time being greater than the prediction without consideration for smear effects.

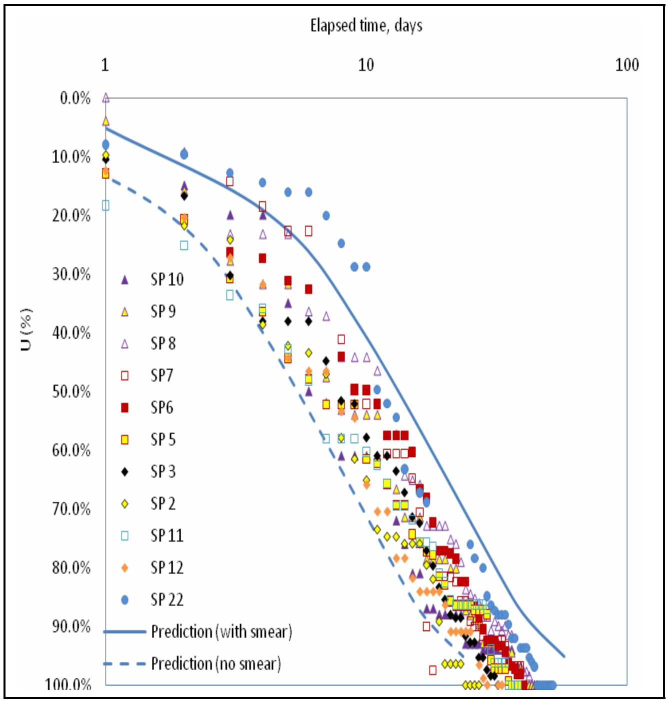

Figure 14. Measured and predicted Degree of Consolidation.

CONCLUSION

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

- A detailed laboratory investigation is useful for determining the geotechnical design parameters for analysis of consolidation with prefabricated vertical drains.

- Based on the laboratory investigations, design values of C_v, C_h, k_v and k_h were 7.5 m^2/year, 15.5 m^2/year, 0.047 m/year and 0.073 m/year, respectively. These corresponded to C_h/C_v value of 2.07 and k_h/k_v value of 1.53. The coefficient of compressibility, C_c from the laboratory tests ranged from 0.17 to 0.45.

- Classical theories of consolidation with the parameters from laboratory tests resulted in estimates of the ground settlements and the consolidation time that were similar to those observed during field monitoring. The one dimensional consolidation theory was found reasonable in estimating the settlements for the 60700 m^2 area overlying 3.0 m to 7.0 m thick compressible soil. The Hansbo theory of radial drainage successfully estimated the time of consolidation.

- The Hansbo theory without consideration for smear effects provided lower bound of the consolidation periods while estimation with smear diameter two times the equivalent drain diameter provided upper bound of the consolidation periods.

- To account for smear effects, the assumptions for the coefficient of horizontal consolidation and the coefficient of horizontal permeability as those for vertical flow (i.e. $C_h = C_v$ and $k_s = k_v$) was found satisfactory for the container yard project.

- The effect of drainage congestion can generally be neglected in most prefabricated vertical drain with sufficient discharge capacity.

- Installation of the vertical drains reduced pre-consolidation time significantly (from about 1 to 5 years without vertical drain to about 50 days with PVDs).

REFERENCES

Abid, M.M., and Pyrah, I.C. (1990), "Consolidation behavior of finely laminated clays". Computers and Geotechnics, 10 (4), 307-323.

Balasubramanian, A.S., Beegado, D.I., Long, P.V., and Thayalan (1995). "Experiences with sand drains and prefabricated vertical drains in ground improvement of soft clays". Seminar on Eng. for Coastal Development, Singapore, s29–s40.

Barron, R. A. (1948). "Consolidation of Fine-Grained Soils by Drain Wells". Transactions, ASCE, 113, 718-754.

Bergado, D.T., Assakami, H., Alfaro, M., and Balasubramaniam, A.S. (1991). "Smear effects of vertical drains on soft Bangkok clay". Journal of Geotechnical Engineering 117 (10), 1509–1529.

Bergado, D.T., Alfaro, M.C., and Balasubramaniam, A.S. (1993). "Improvement of soft Bangkok clay using vertical drains". Geotextiles and Geomembranes 12 (7), 615–663.

Bo, M.W., Bawajee, R., Chu, J., and Choa, V. (2000). "Investigation of smear zone around vertical drain". Proceedings of the Third International Conference on Ground Improvement Techniques, Singapore, 109–114.

Carroll, R.G. (1983). "Geotextile filter criteria". Transportation Research Record, 916, 46–53.

Chu, J., Bo, M.W., Chang, M.F., and Choa, V. (2002). "The consolidation and permeability properties of Singapore marine clay". Journal of Geotechnical & Geoenvironmental Engineering 128 (9), 724–732.

Chu, J., Bo, M.W., and Choa, V. (2004). "Practical Considerations for using Vertical Drains in Soil Improvement Projects". Geotextiles and Geomembranes, 22, 101-117.

Gibson, R.E., and Shefford (1968). "The efficiency of horizontal drainage layers for accelerating consolidation of clay embankment". Geotechnique, 18, 327-335.

Hansbo, S. (1960). "Consolidation of Clay with Special Reference to Influence of Vertical Sand Drains". Proceedings of Swedish Geotechnical Institute, No. 18.

Hansbo, S. (1979). "Consolidation of Clay by Band-Shaped Prefabricated Drains". Ground Eng., 12(5), 16-25.

Hansbo, S. (1981). "Consolidation of fine-grained soils by prefabricated drains". Proceedings of the 10th International Conference on Soil Mechanics, Stockholm, 3,677–682.

Hansbo, S. (1997). "Aspects of vertical drain design: Darcian or non-Darcian flow". Geotechnique 47, 983–992.

Hird, C.C., and Moseley, V.J. (2000). "Model study of seepage in smear zones around vertical drains in layered soil". Geotechnique 50 (1), 89–97.

Holtz, R.D., Jamiolkowski, M., Lancellotta, R., and Pedroni, S. (1987). "Performance of Prefabricated Band-Shaped Drains". Construction Industry Research and Information Association (CIRIA), London, England; Report RPS 364.

Indraratna, B., and Redna, I.W. (1998). "Laboratory determination of smear zone due to vertical drain installation". Journal of Geotechnical Engineering, ASCE 124 (2), 180–184.

Kremer, R.R.H.J., Oostven, J.P., Van Weele, A.F., Dejager, W.F.J., and Meyvogel, I.J. (1983). "The quality of vertical drainage". Proceedings of the Eighth European Conference on Soil Mechanics and Foundation Engineering, Helsinki, 2, 721–726 .

Onoue, A. (1992). "Precompression and vertical drain designs". Workshop on Applied Ground Improvement Techniques, Southeast Asian Geotechnical Society, AIT, pp.1–78.

Rujikiatkamjorn, C., and Indraratna, B. (2010). "Radial consolidation modelling incorporating the effects of a smear zone for multi layer soil with downdrag caused by mandrel action". Canadian Geotechnical J. 47, 1024-1035.

Terzaghi, K. (1943). "Theoretical Soil Mechanics". John Wiley and Sons, New York

Xiao, D.P. (2001). "Consolidation of soft clay using vertical drains". Ph.D. Thesis, Nanyang Technological University, Singapore.

Xie, K.H. (1987). "Consolidation theories and optimisation design for vertical drains". Ph.D. Thesis, Zhejiang University, China.

Yoshikuni, H., and Nakanodo, H. (1974). "Consolidation of fine-grained soils by drain well with filter permeability". Soils and Foundations 14 (2), 35–46.