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Reclamation and Ground Improvement of Soft Marine Clay for Development of Offshore Terminal 4, JNPT Navi Mumbai

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Abstract. A container yard of around 100 Ha area was developed as part of the 4th terminal for Jawaharlal Nehru Port Trust (JNPT) in Navi Mumbai by means of reclamation in sea over soft, compressible marine clay. Ground improvement of the subsoil was warranted in order that the finished reclamation satisfies the stringent serviceability criteria.

Extensive offshore geotechnical investigation campaign comprised of conventional borehole sampling, laboratory tests and various in-situ field tests such as CPTu, Field Vane Shear Tests etc. Investigations revealed clay thicknesses of 4 to 22 m, with top layer showing undrained shear strengths less than 7 kPa and increasing with depth, underlain by weathered Basalt. The ground was improved using Prefabricated Vertical Drains (PVD) with preloading. Radial consolidation and cone dissipation tests were carried out to establish the coefficient of horizontal consolidation (c_h).

Stability analyses of perimeter bund, confining the marine reclamation were examined for all probable failure surfaces. The analyses showed that overfill sections beyond the permanent reclamation cope line were required along with a layer of tension geotextile in order to achieve stability during filling, to support the full design load until the cope line. Once the surcharge was removed, the overfill was cut back to the final geometry and revetment was provided along the perimeter.

Extensive geotechnical instrumentation and monitoring were conducted using multilevel magnetic extensometers, settlement gauges, inclinometer and piezometers to monitor the behaviour and performance of ground improvement. Field c_h and smear coefficients were back calculated.

Around 200 confirmatory boreholes (CBH) were carried out after completion of ground improvement to ascertain the post improvement and validate the design parameters thus eliminating the risk of post construction settlement.

Keywords: Geotechnical investigation, Ground Improvement, c_h values, PVD, Slope Stability, Surcharge, Instrumentation & monitoring, back analyses

1 Introduction

The coastal regions in and around Navi Mumbai primarily comprise of estuarine deposit of very soft marine clays which are known for their poor geotechnical properties such as high compressibility and very low bearing capacity. Lack of suitable ground improvement in this soft clay, will cause excessive settlement and adversely affect both the stability and durability of the port infrastructures built over these clays. Preloading with PVD, is one of the most commonly adopted ground improvement technique. In the ground improvement design using PVD, a reasonable estimation of both the magnitude and time of consolidation are indispensable for planning and execution of works within the project cost and time. These estimations depend on the selection and adoption of Geotechnical design parameters, especially consolidation parameters. Although, these consolidation parameters are derived from large number of in-situ field tests and sophisticated laboratory tests, it is practically difficult to accurately predict the magnitude and duration of consolidation for the given large scale of the reclamation works, especially due to smearing effects around PVD (Indraratna et al, 2003), uncertainties and variations associated with the ground condition and geotechnical parameters. Hence, a proper instrumentation and monitoring campaign is indispensable to assess the PVD performance in the field.

This paper deals with case study of 100 Ha reclamation over very soft marine clays, improved with PVD. The preloading to accelerate the consolidation settlements was designed to satisfy the future loading and stringent settlement requirements. Extensive instrumentation was carried out and the removal of the preloading was based on the observational approach from the instrumentation data.

2 Project Description

The project considered in this paper comprises of a container yard with rail & road corridor over marine clay at JNPT in Navi Mumbai. The onshore terminal area is developed by reclaiming about 90 Ha of land over the Uran Mud Flat, adjacent to the existing terminals presented in Fig. 1, to accommodate container stack yards and associated structures such as rail container depot (RCD), tank farms, port buildings, workshops, roads and utility buildings with a finished level of +7 mCD to cater to permanent surcharge loads of 50 & 30 kPa. CD implies Chart Datum and is a local co-ordinate system specific to a port. In this case 0 mCD indicates a level 2.51 m below MSL.

Fig. 1. Existing Terminals

The layout of the new terminal is shown in Fig. 2. The rectangular area is the container yard terminal measuring approximately 1000 m x 500 m and the long narrow stretch is the rail-road corridor extending to around 4 km with a width varying from 120 to 150 m. The reclamation filling was carried out through end on dumping from the existing land. Initially peripheral bunds were constructed. Between the external bunds, temporary bunds were constructed, and filling was carried out within these enclosed regions. Hydraulic filling was not an option since there was no dredged sand available within the vicinity of the site. Hence, borrow material was transported from nearby quarries and the sea was reclaimed.

3 Geotechnical Investigation Campaign

Geotechnical investigations were carried out at the project site during 2008, 2009 and 2014 by the Client. Further additional investigations were carried out by the Contractor after award of contract in 2015.

The investigation campaign comprised of 89 numbers of boreholes through conventional drilling and sampling, 63 numbers of Cone Penetration Tests (CPT) with pore pressure measurement, Cone dissipation tests and Field vane shear tests (VST). In addition to the routine laboratory tests, specialist tests such as radial consolidation and extended 1D oedometer tests for secondary compression were also conducted to obtain the consolidation parameters. Consolidation and compressibility parameters were obtained through laboratory tests, whilst the shear strength parameters were characterized from the CPT resistance and vane shear tests.

Fig. 2. The proposed new terminal layout

3.1 Strata description

The average seabed level and sea water level obtained through bathymetric survey were around 0.0 mCD and +2.5 mCD respectively. The geology of the site predominately comprises of 6 m to 20 m thick soft clay (unit 1) underlain by a thin layer of sandy gravel (unit 2) and basaltic rock (unit 3). The soft marine clay was found to be very soft, sensitive and normally to over consolidated. For the purpose of design, the entire site was divided into 10 Zones based on the use of individual sets of selected design parameters, clay layer thickness, handover periods and individual service live loadings. Table 1 presents the subsoil stratification across all zones.

Table 1. Subsoil stratification

Unit	Zone	Strata Description	Thickness (m)
`Unit 1	Zone 1	Soft Marine Clay	6 to 8
	Zone 2		8 to 10
	Zone 3		10 to 14
	Zone 4		10 to 14
	Zone 5		14 to 20
	Zone 6		16
	Zone 7		10 to 12
	Zone 8		8
	Zone 9		10 to 12
	Zone 10		14 to 16
Unit 2	NA	Sandy Gravel	0.5 to 6.5
Unit 3		Amygdaloidal Basalt	NA

4 Test Results and Design Parameters

4.1 Standard penetration tests

During the initial investigation campaign conducted by the client SPT was conducted. The SPT 'N' value varied from 0 to 4 in soft marine clay up to -12 to -14 mCD and got slightly stiffer down the depth. Marine clay was followed by a thin layer of sandy gravel layer with an average 'N' value of 50. The plot of SPT N versus depth is presented in Fig 3.

4.2 Atterberg limits

The Atterberg limits are presented in Fig 4. Liquid Limit was observed to be in the range of 70% to 110% with Plasticity Index around 30% to 80%. The Natural Moisture Content (NMC) varied from 60% to 120% and was close to the liquid limit.

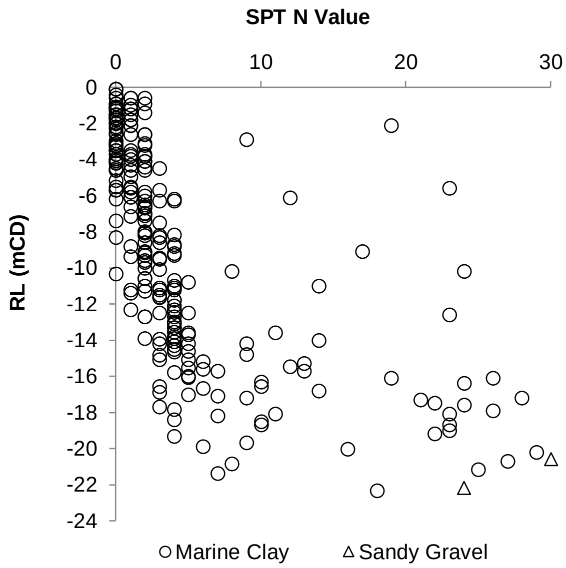

Fig. 3. Variation of SPT N with depth

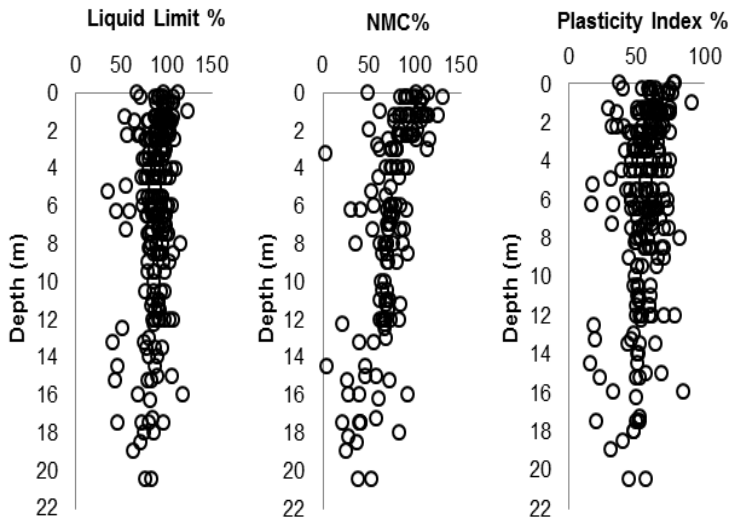

Fig. 4 Atterberg limits and natural moisture content with depth

4.3 Undrained shear strength

The undrained shear strength of the soft clay was derived from Cone Penetration tests the cone penetration resistance using equation (1).

$$C_u = (q_T - P_0)/N_{kT} \qquad (1)$$

Where $q_T = q_c + u\,(1- a)$; a is the cone correction equal to 0.75; N_{kT} is the cone correlation factor.

A number of Field Vane Shear Tests (VST) were conducted in close proximity to CPTs to estimate the correlation factor N_{kT}. N_{kT} factors were found to be higher in the shallower depth and reduced along the depth as shown in Table 2.

Table 2. Variation of N_{kT} with depth

Depth	N_{KT}
0– 5	25
5 – 6	20
> 6	15

In order to arrive at design shear strength for a particular zone or along the peripheral bund, the undrained shear strengths derived from CPTu tests and Vane shear tests were plotted as shown in Fig. 5. It was mutually agreed with the Client's consultant that the 25th percentile line would be the most appropriate design line. 25th percentile indicates that 75 % of the points are to the right side of the line and hence have strengths greater than the design strength. The Design Line (The dark black line in Fig 5) was taken as a straight line passing, as far as possible through the 25th percentile line.

4.4 Compression ratio

The compression ratio CR = ($C_c/(1+e_0)$) is plotted with depth (Fig. 6) and is observed to be in the range of 0.15 to 0.35. The top 4 to 6 m of the clay shows an average CR of 0.25. Below 6 m depth the average CR is around 0.18.

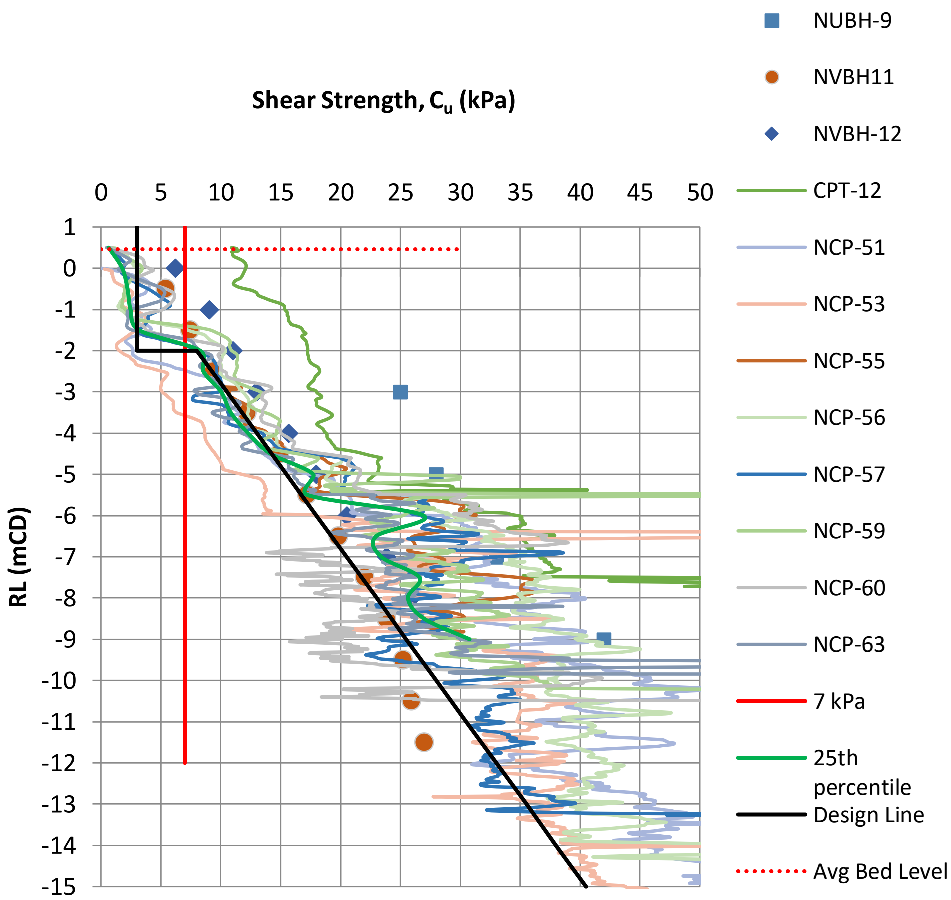

Fig. 5. Variation in undrained shear strength with depth

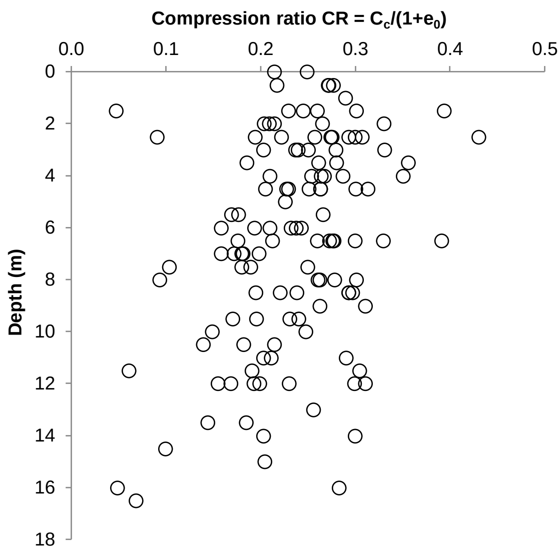

Fig. 6 Variation of compression ratio with depth

4.5 Secondary Compression

Several extended oedometer testing were carried out to determine the secondary compression index (C_α). During such tests, the consolidation loads were continued and extended up to 12 days post 24 hours after the end of primary consolidation. The measured C_α values were in the range of 0.015 to 0.035 as shown in Figure 7. Figure 8 shows the compression index ratio C_α /C_c ratio of .033 which was found to be in similar range proposed by Mesri (Mesri et al 1977).

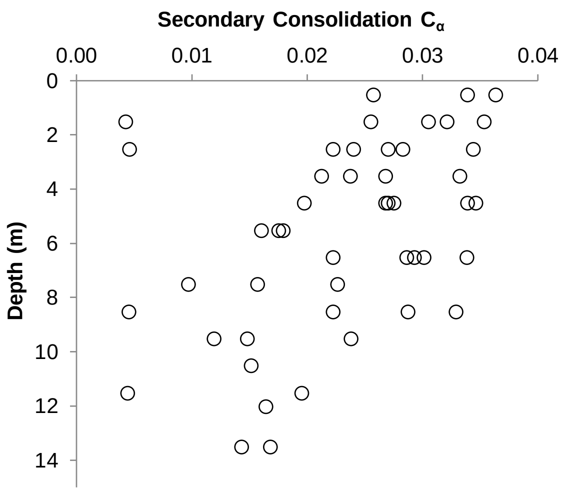

Fig. 7. Variation of secondary compression index with depth

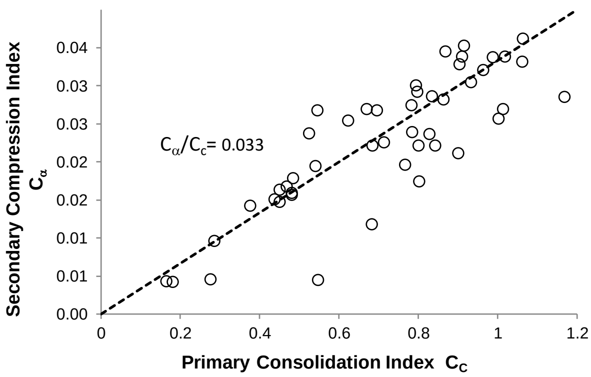

Fig. 8. Compression index ratio C_α / C_c

4.6 Coefficient of consolidation (c_v and c_h)

Conventional 1D oedometer consolidation tests were carried out for different ranges of effective stress on the retrieved clay samples. As the estimated preloading for the reclamation was in the range of 100 to 200 kPa, the coefficient of vertical consolidation (c_v) corresponding to this pressure range was found to be around 0.8 to 2 m^2/year. Similarly, c_h value measured using radial consolidation tests was in the range of 1.9 to 4.25 m^2 /year, yielding a c_h / c_v ratio of 2.1 to 2.5. The variations of these parameters along the depth are shown in Figure 9.Few cone dissipation tests were also conducted at certain depths by interrupting the cone penetration and monitoring the excess pore pressure dissipation with time. C_h values were calculated from the modified time factor derived using the strain path solution method (Teh and Houlsby, 1991). The in-situ C_h values were estimated to be in the range of 3.4 to 4.86 m^2/year and found to be in the similar range of values noted in radial consolidation tests.

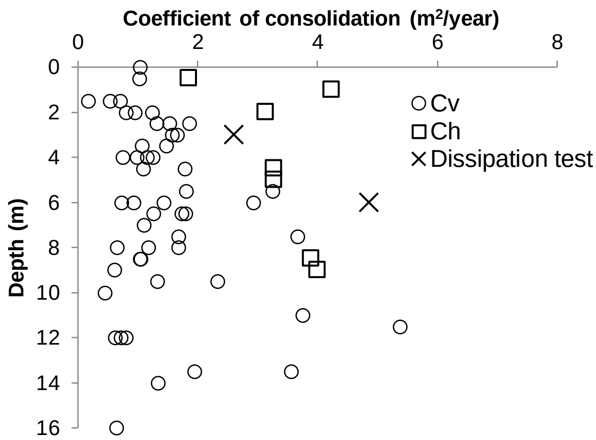

Fig. 9. Variation of coefficient of consolidation c_v and c_h with depth

4.7 Pre-consolidation Pressure and OCR

The preconsolidation pressure obtained from the one dimensional consolidation tests, clearly showed that clay was over consolidated with OCR in the range of 2 to 5 in the top portion, which might presumably resulted from the aging and desiccation process. The OCR was found to be reducing with the depth as shown in Figure 10.

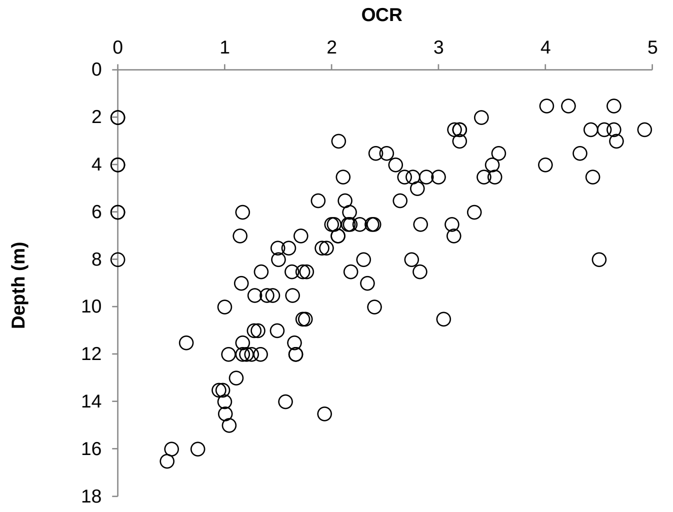

Fig. 10 Variation of OCR with depth

4.8 Organic Content

The Organic content in the clay was between 5 to 10% as depicted in Fig 11.

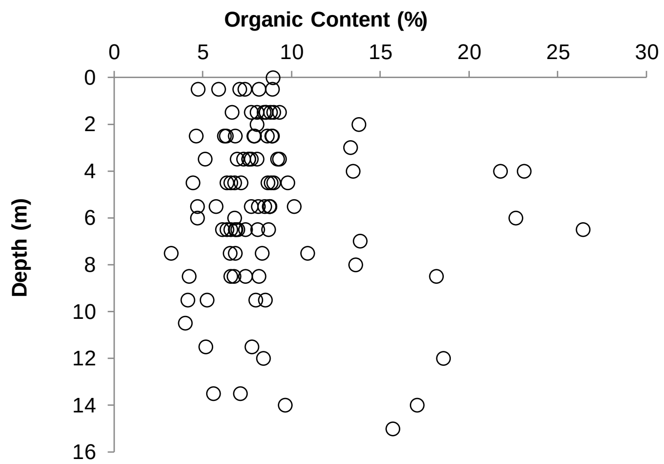

Fig. 11. Variation of organic content with depth

The Geotechnical parameters of Unit 1 is derived and presented in Table 3

Table 3. Geotechnical parameters for Unit 1

Parameters	Values
Density (kN/m^3)	15
Thickness (m)	3.7 – 22.5
Plasticity Index I_p	55 – 70
Compression Ratio CR	0.19 – 0.3
Coefficient of consolidation c_v (m^2/yr)	0.8 – 1.7
Coefficient of radial consolidation c_h (m^2/yr)	1.9 – 5.2
c_h/c_v	2.1 – 2.5
Secondary compression Index C_α	0.016 – 0.034
C_α/C_c	0.023 – 0.055
OCR	1.5 – 3.4

5 Design Criteria

5.1 Confined reclamation areas

The 100 Ha area had to be reclaimed using borrow material, since no dredged sand was available within a radius of 100 km from site, to a finished level of +7.0 mCD. The reclamation area had to be designed to withstand the operational loads given in Table 4 and satisfy the settlement criteria given in Table 5.

Table 4. Terminal area loads

Sr. No.	Area Description	Design Load (kN/m^2)	
		Static condition	Seismic Condition
1	Container Yard	50	25
2	Rail Area Access Road and Gate	30	15
3	Buildings and Car Park	20	10

Table 5. Residual Settlement Criteria

	Maximum total ground settlement (mm)		
	After 2 years	After 5 years	After 20 years
Overall Residual settlements	100	150	300

The maximum allowable differential settlement was 1V:200H measured between any two points on the site, no more than 8 feet apart.

5.2 Reclamation Edge and Revetments

The reclamation edges had to be analysed for stability of their slopes during construction and the permanent condition. The Factors of Safety to be achieved were as given in Table 6.

Table 6. Factors of safety requirement

Condition	FOS
Temporary	1.3
Permanent Static	1.5
Permanent Seismic	1.1

5.3 Ground Improvement using PVD

As is observed from the previous sections, the subsoil comprised of very soft clay over the top 4 to 6 m. Beyond this depth the shear strength increases (Ref. Fig. 5). The compressibility characteristics also indicate high long term settlements. In order to satisfy the design load requirements and the long term settlement criteria it was imperative that the ground be improved.

Ground improvement using PVD and preloading was adopted considering the tight time schedule, vast area to be improved and expertise and resources available, although other methods of ground improvement were discussed.

The entire area was divided into ten zones, as described earlier in this paper. Design parameters for strength and compressibility were derived for each zone and different stretches along the reclamation edges. Typical design parameters for one zone and one edge are shown in Table 7.

Table 7. Shear Strength variation with depth

Area	From RL (mCD)	To RL(mCD)	Shear Strength C_u (kN/m^2)
Zone 3	1	-4	5 kPa to -1.5 mCD
			7 + 2 kPa/m from -1.5 to -4.0 mCD
	-4	-12	12 + 2.8 kPa/m below -4.0 mCD
Edge along CH	0	-4	7.5 + 1.5 kPa/m
3430 - 4160	-4	-8	13.5 + 3.0 kPa/m

5.4 Construction methodology

It was decided that the PVD would be installed from a surface above the Highest Water Level. Hence the construction methodology adopted was as under:

1. Stage 1 filling to +5.5 mCD, above the Highest High Water Tide by end on dumping.
2. Install PVD, drainage blanket layer, PVD, geotextiles and instrumentation on stage 1 platform.
3. Within sections, which are sufficiently set back from the edge of stage 1 filling platform, commence stage 2 filling to 7 m/8 m CD and stage 3 filling to final design surcharge level.
4. For edge zones, wait for specified period for the required gain in shear strength in unit 1 and then raise to stage 2 filling of 7 m CD. Again, wait for required gain in shear strength and then raise to final design surcharge level.
5. The fill above +5.5 mCD was compacted in layers to 95% MDD. The fill above +8.0 was loose fill and not compacted since it would act as surcharge and would be removed after completion of design consolidation settlements.
6. After reaching final surcharge level, wait until 95% consolidation is achieved.
7. After completion of ground improvement surcharging, the surcharge is removed to +6.0 mCD. At the edges, the over fill is cut back to the final geometry.

5.5 Mudwave formation

As the filling of stage 1 to +5.5 mCD progressed it was observed that the top very soft clay was getting displaced due to the filling process, thus resulting in a mudwave (Fig 12). This displaced mud was getting replaced by the fill material. Confirmatory boreholes were conducted to witness the actual thickness of replacement and determine the shear strength of the soft clay.

Fig. 12. Formation of mudwave ahead of the fill

Fig.13 presents the top of marine clay before and after stage 1 filling, as evidenced from the confirmatory boreholes. From the figure it is clear that 2.5 to 3.5 m of the soft clay had displaced and was replaced by the moorum fill.

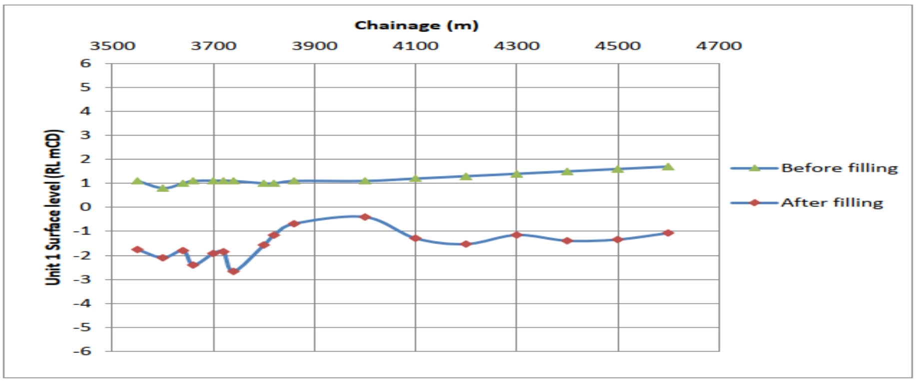

Fig. 13. Top of marine clay: Before and after filling

5.6 Ground improvement design

The PVD were installed in triangular pattern at 1.0 and 1.2m spacing which was followed by the preparation of filter blankets. The design was carried out as per IS 15284 Part 2 (2004). The preloading requirements for the various design loads mentioned in Table 4 worked out to those given in Table 8

Table 8. Design preloading level

Area	Preloading Level (mCD)
Container yard	+12.5
Rail Area Access Road and Gate	+10.5
Buildings and Car Park	+10.0

The preloading up to +10.0 m CD to +12mCD was built up in three stage lifts with the waiting period allowed between each stage to allow for consolidation, thereby enhancing the compressibility of underlying soft clay and stabilizing the reclamation slopes for the next uplift. Based on the thickness of clay layer and the design load requirement, the time for achieving 95% consolidation settlement and the absolute value of the total primary consolidation settlement varied across the site. The time varied from 75 days to 120 days for 1.0 m spacing of PVD and settlements varied from 1.0 m to 2.5 m.

5.7 Stability of edge slopes

The virgin shear strength of the soft clay as presented in Fig 5 or Table 7 indicates that the subsoil is too soft that the filling had to be carried out in stages, otherwise the edge slope would be continuously failing. Hence ground improvement was required and filling was carried out in stages based on slope stability analyses using the industry software Slope/W by Geoslope Intl.

At every stage of construction, the FOS of 1.3 had to be maintained as mentioned in Table 6. Since the 1st stage of filling up to +5.5 mCD was carried out without ground improvement, maintaining this FOS was not possible and hence resulted in mudwave. However, the mudwave offered some resistance and helped in achieving a 1 in 3 slope at the edge.

The parameters for the reclamation fill material used in the slope stability analyses are given in Table 9.

Table 9. Design parameters of reclamation fill

Description	Unit weight (kN/m^3)	Shear strength parameters C_u (kN/m^2)	Φ (degrees)
Stage 1 (below +5.5 mCD)	17	5	30
Stage 2 (from 5.5 mCD to +7.0 mCD)	19	5	32
Stage 3 (above +7.0 mCD)	17	5	30

Table 10 shows the shear strength requirement at the top of the marine clay so that the required FOS is achieved for every stage of filling including the preloading for one typical edge.

Table 10. Shear strength required at top of marine clay at one particular edge

Fill Levels	Shear strength c_u (kN/m^2)
Seabed to +5.5 mCD	13.5
From +5.5 mCD to +7.0 mCD	26.0
+7.0 mCD to preloading top	34.0
Permanent condition at +10.5 mCD (after 95% consolidation)	56.0

The increase in shear strength due to increase in effective stress for every stage of filling was calculated by Skempton's equation

$$\frac{\Delta C_u}{\Delta \sigma_v'} = [0.11 + 0.37\ \frac{PI(\%)}{100}\ ] \text{ x U} \qquad (2)$$

For PI value of 65% this increase works out to 0.35U.

5.8 Tension geotextiles

In order to achieve the required FOS, it was required to provide a single or two layers of tension geotextiles, one at +5.5 m CD and the other, if required, at +6.5 mCD. It was observed that these geotextiles were required only for the temporary construction stage and were not required for the permanent stage.

The Partial Factor for Tension capacity was derived as:

PF = PF for creep x PF for installation damage x PF for Environmental effect x PF for material = 1.45 x 1.05 x 1.05 x 1.00 = 1.6

and Partial Factor for Pull out = 1.5

Geotextiles with Ultimate tension capacities of 600 to 1000 kN/m were used at various edge stretches depending on the subsoil condition and preloading levels.

5.9 Slope Stability Analyses

5.9.1 Temporary construction stage

Slope stability analyses were carried out for every stage of construction so as to achieve the required FOS. The Morgenstern-Price method was used to calculate the FOS. Both circular and non-circular slip surfaces were considered. Critical Circular and Sliding block slip surfaces are checked and presented in Figures 14 & 15 for the final stage, i.e. stage 3. After every stage the increased shear strength of the marine clay, due to the partial consolidation resulting from previous stage was considered in the analyses. The FOS values for the corresponding stages of fill levels are shown in Table 5. In selecting the sliding block surface, care is taken to ensure that the middle segment line is longer than the two end projection segment lines. This is to avoid unrealistic failure mechanisms.

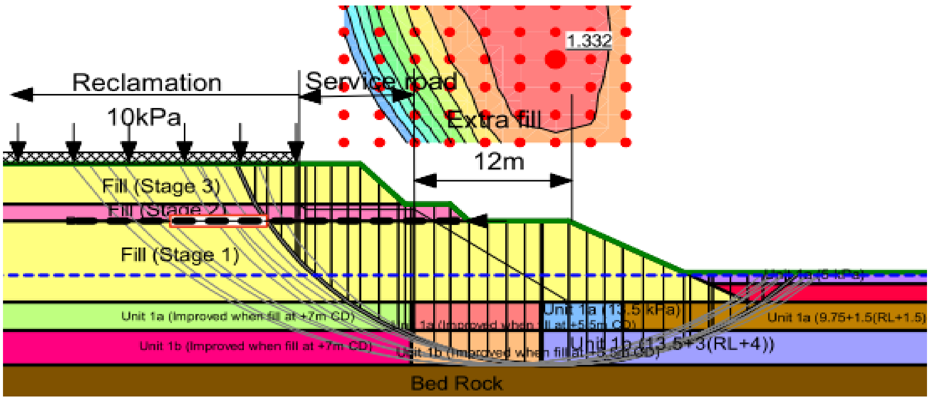

Fig. 14. Slope stability model for +10.5 mCD level fill (Circular failure) – Construction stage

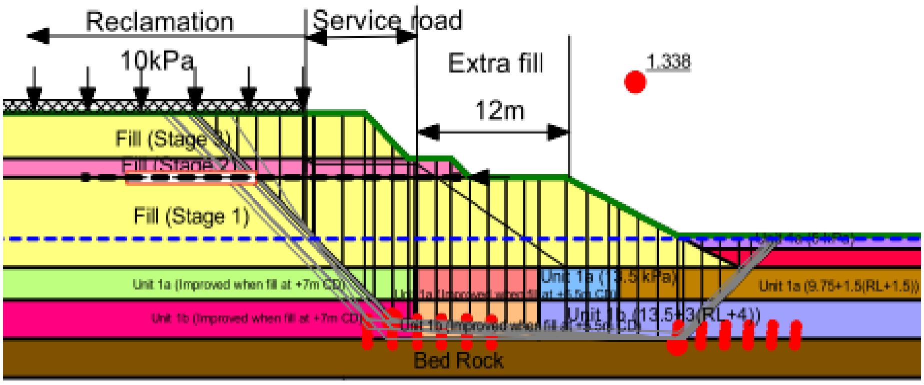

Fig. 15. Slope stability model for +10.5 mCD level fill (Sliding failure) – Construction stage

Table 11 shows the FOS values for different stages of fill levels for circular and non-circular surfaces. From Table 11 it is clear, that the minimum FOS of 1.3 is achieved in stage 3 filling. Hence this stage dictated the geometry of the temporary filling which necessitated 12 m overfilling beyond the cope line and the necessity of Geotextile.

Table 11. FOS values for different stages of fill levels

Stages	Fill Levels	Critical FOS	
1	Sea bed level to +5.5mCD	Circular	1.43
		Sliding Shallow	1.96
		Sliding Deep	1.64
2	+5.5 mCD to +7.0 mCD	Circular	1.53
		Sliding Shallow	2.26
		Sliding Deep	1.77
3	+7.0 mCD to +10.5 mCD	Circular	1.33
		Sliding Shallow	1.59
		Sliding Deep	1.33

5.9.2 Permanent Condition

After the achievement of 95% consolidation, the surcharge was removed to achieve a final level of +7 mCD & the over fill was cut back to the required geometry. The perimeter bund was checked for the long term static and seismic cases using Slope/W. For this case a surcharge loading of 30 kPa for main reclamation and 10 kPa for Service Road are considered for static case. Figures 16 and 17 present the critical slip surfaces for the static and seismic cases respectively. Table 12 presents the FOS values for both cases which are greater than 1.5 and 1.1 respectively.

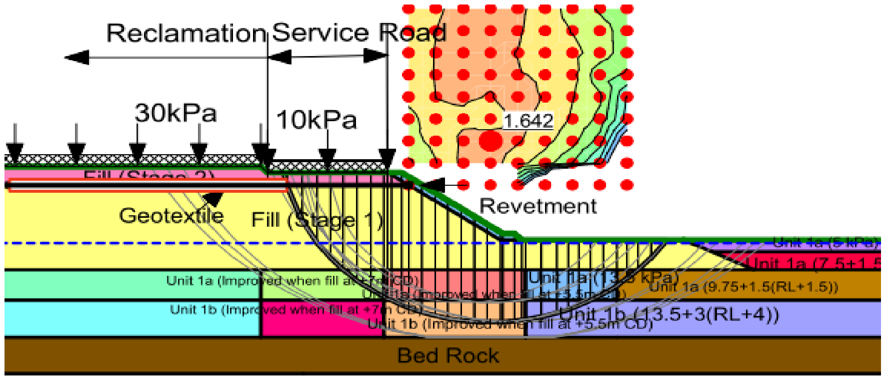

Fig. 16. Slope stability model for permanent static case

For the seismic case, the horizontal (k_h) and vertical (k_v) seismic coefficients considered in the analysis are derived based on IITK-GSDMA guidelines for seismic zone 4. These are derived as under:

k_h = 1/3 x Z x I x S and k_v = 0; Where Z = 0.24, I = 1.5 and S = 1.2 and surcharge loading is reduced by 50%.

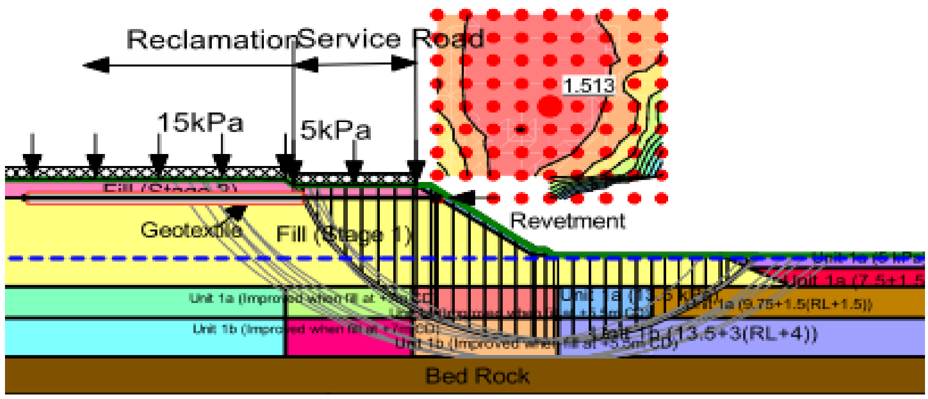

Fig. 17. Slope stability model for permanent seismic case

Table 12. FOS values for permanent static and seismic case

Stages	Fill Levels	Critical FOS	
1	+7.0 mCD	Circular	1.64
	Static	Sliding Shallow	2.20
		Sliding Deep	1.86
2	+7.0 mCD	Circular	1.51
	Seismic	Sliding Shallow	1.97
		Sliding Deep	1.68

Fig.18 shows the geometry that is arrived at from the slope stability analyses, for the 3 stages of loading with the overfill and 1 layer of geotextile.

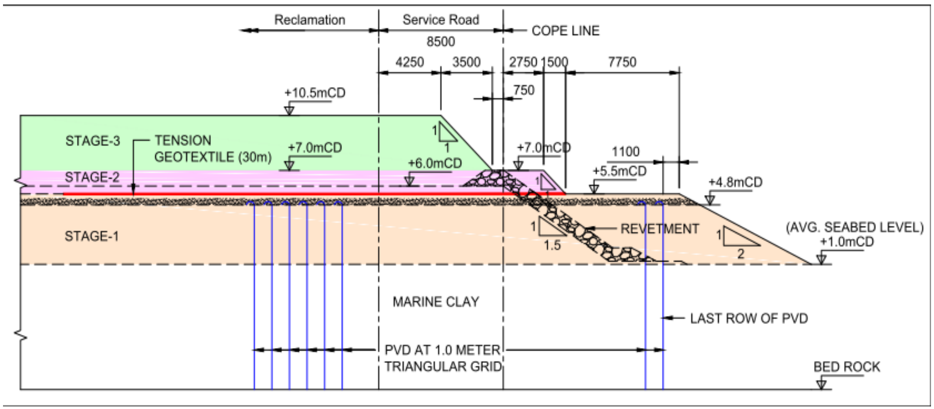

Fig. 18. Geometry arrived from slope stability analyses

6 Instrumentation and Monitoring

Extensive instrumentation and monitoring were carried out all over the reclamation zone as shown in the snapshot presented in Figure 19. Instrumentation included surface settlement markers, deep settlement markers, magnetic extensometers, open standpipe, vibrating wire piezometers, and inclinometers. Instrument clusters were installed in regular grid patterns of size 25mx25m as shown in Figure 19 over the entire site to closely monitor the process of consolidation. Totally around 1000 numbers of instrument grids were proposed. The instruments were arranged in such a way that at every 100m x 100m size of ground improvement, there were at least 25 numbers of settlement markers/plates, 5 numbers of boreholes, 4 numbers of vibrating wire piezometers. Inclinometers were provided at every 100m spacing along the reclamation boundary to monitor the lateral movement and stability of the preload. In the figure G indicates Settlement markers on top of surcharge and at stage 1 level. Similarly, other points like A B C D E and F indicate different combinations of instruments along with settlement markers as in G, and boreholes.

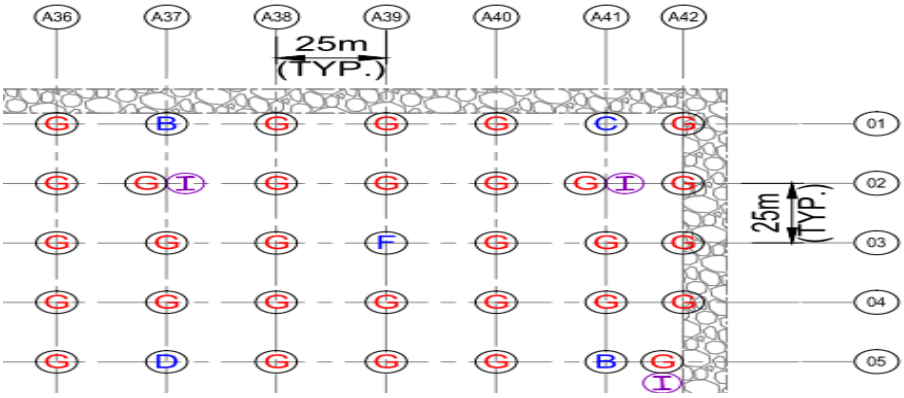

Fig. 19. Instrumentation Layout Plan

6.1 Surcharge removal

The time for removal of surcharge was slated as completion of 95% consolidation. To ascertain whether 95% completion has been achieved, the graphical method suggested by Asaoka (1978) was adopted using field measurement data from the instrumentation. The settlement data from settlement markers and magnetic extensometers were used in the Asaoka plots. A typical Asaoka plot from one of the settlement gauges is presented in Fig. 20.

Fig. 21 shows a plot of the settlement with time and raising of the fill with time. It is to be noted that the data used for Asaoka plots started from a time when the fill reached its maximum preload level.

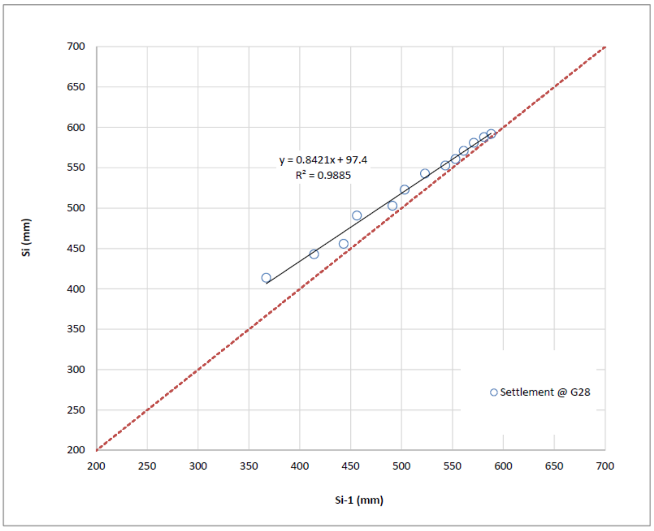

Fig. 20. Typical Asaoka plot for a settlement gauge

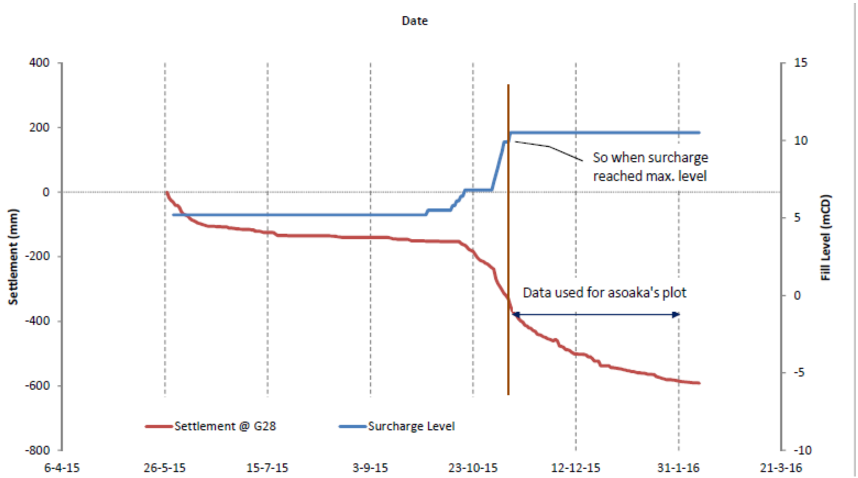

Fig. 21 Plot of settlement versus time and raising of the reclamation fill

7 Back Analyses

Rigorous back analyses were performed based on the observed field data to theoretically back calculate various design parameters such as coefficient of consolidation, smear coefficients, OCR and compression ratio of the clay. Fig 22 and Fig 23 show the back analyses prediction based on the multilevel Magnetic extensometers and settlement gauges respectively. Each magnetometer had 5 to 7 magnets installed over depth. It may be noted that the settlement plotted are for the topmost magnet on the top of the marine clay which represents the maximum settlement.

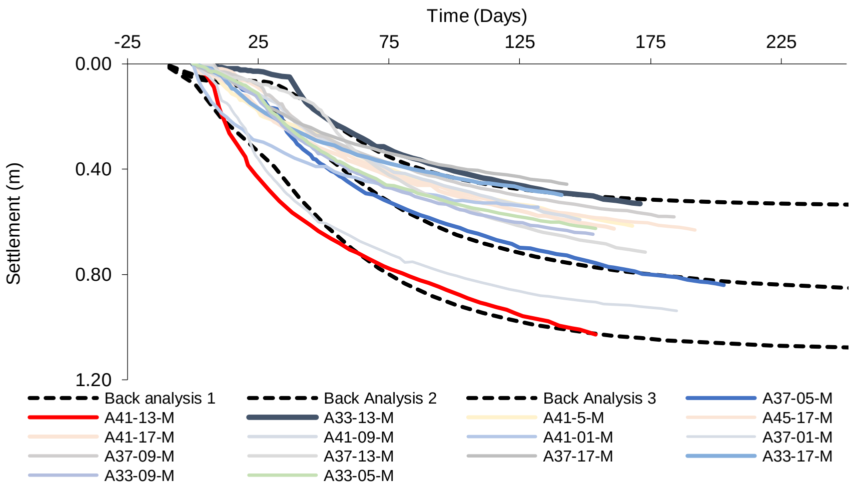

Fig. 22. Back analyses based on Magnetic extensometer data

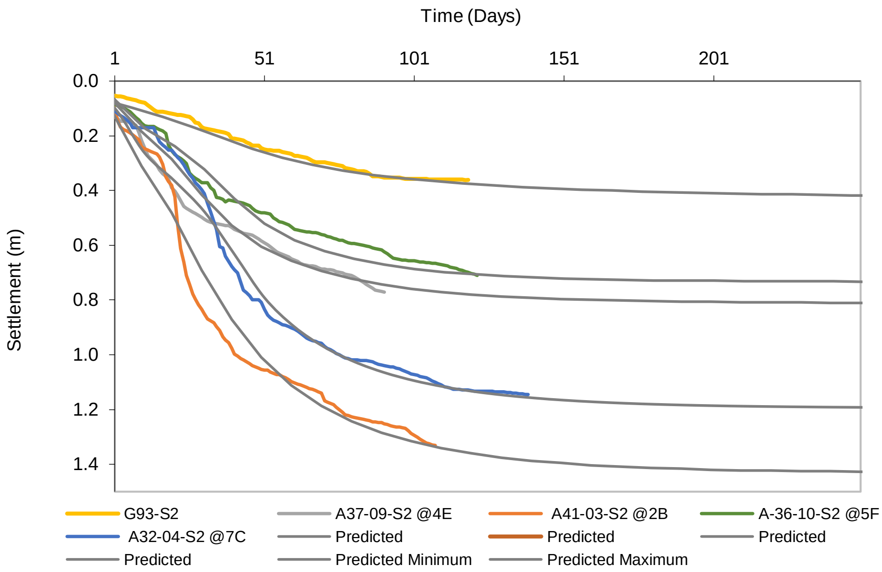

Fig. 23. Back analyses based on settlement gauge data

The back analyses calculations considered the multiple stage ramp preloading and the effect of submergence in surcharge due to settlement. The Compression Ratio (CR) and the horizontal coefficient of consolidation (C_h) were adjusted in the back calculation to match the theoretical predicted curve with various settlement data observed in the field, both horizontally and vertically as shown in Fig. 22. The horizontal coefficient of consolidation C_h which gave similar settlement trend as observed in the field was around 3.75- 4.0 m^2/year with C_h / C_v ratio around 2.3. Similarly, smearing permeability ratio was also estimated to be in the range of 3 to 4 with smear diameter ratio of 2.5.

7.1 Back calculation of C_h from piezometer data

The following relationship can be derived from the equation proposed by Hansbo (1979).

$$1- \frac{\Delta u_t}{\Delta u_0} = 1- \exp(-\alpha t) \tag{3}$$

$$\alpha = \frac{8C_h}{D_e F} \tag{4}$$

where u_0 is the excess pore pressure observed at time t=0; Δu_t is the excess pore pressure at time t; D_e is the effective diameter of a unit cell of drain; and F is a resistance factor for the effects of spacing, smear, and well resistance.

F = ln(n)-0.75+ ln(s)(k_h/k_s-1). The equation 3 can be rewritten as,

$$\ln \frac{\Delta u_0}{\Delta u_t} = \alpha t \quad (5)$$

Hence, equation (5) shows a linear relationship with time, with α being the slope of the line, from which C_h can be back calculated directly from the piezometer data.

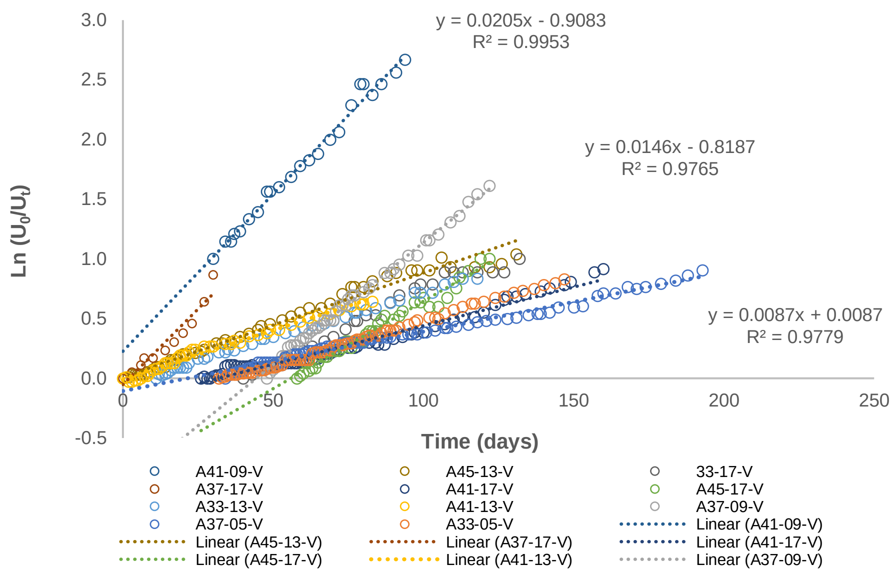

Fig. 24. Estimation of C_h from Piezometer data

As shown in Figure 24, for the slope of the line (α) of around 0.008 to 0.02, the back analysed values of C_h were in the range of 1.8 m^2/yr to 4.7 m^2/yr including the smearing diameter ratio and permeability ratio of 2.5 and 4 as estimated from the back analyses results (Figure 22 & 23). The C_h values found lower than 2.0 m^2/yr (α ~ 0.008) were ignored, as the dissipation of excess pore pressure was observed to be improper in those piezometers due to clogging of the filter.

7.2 Back calculation of C_h from Magnetic extensometer

Equation (3) can be rewritten in terms of settlement as Equation (6) and can be rearranged as Equation (7)

$$\frac{S_t}{S_f} = 1 - \exp(-\alpha t) \quad (6)$$

$$\ln \frac{S_t}{S_f - S_t} = \alpha t \quad (7)$$

where S_f is the final settlement and S_t is the settlement at any time t. The linear relationship of this function can be used to estimate the slope α from the field settlement data

As shown in the Figure 25, for the value of α = 0.0185 estimated from the settlement data, the back calculated C_h was around 3.7 m^2/yr for the adjusted smearing diameter ratio and permeability ratio of 2.5 and 3 respectively

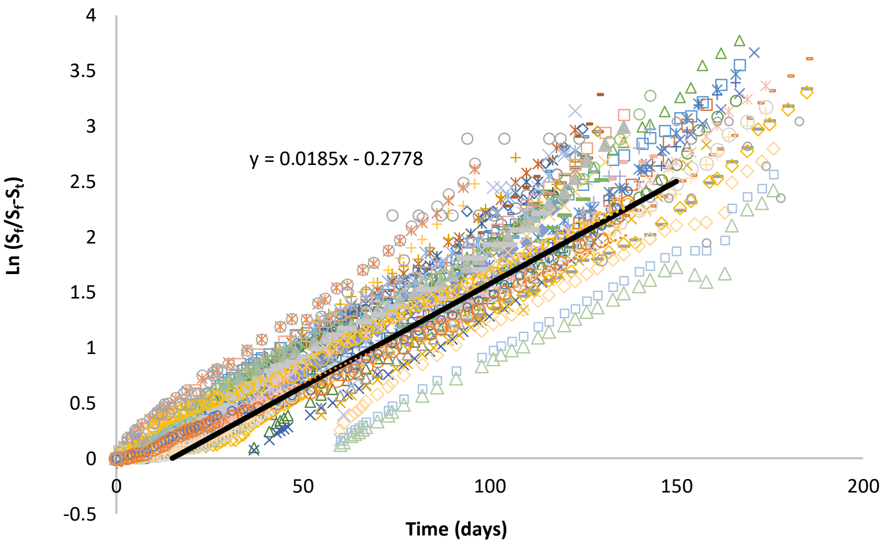

Fig. 25. Estimation of C_h from Magnetic Extensometer data

7.3 Back calculation of C_h from settlement gauges

The back calculation carried out based on the settlement gauges data installed at +5.5 mCD, yielded α around 0.033 and corresponding C_h value of about 4.7 m^2/yr as shown in Figure 26. Based on various methods as discussed above, the C_h values were estimated to be around 2 to 5.2 m^2/yr as shown in Table 13.

Table 13. Summary of estimated C_h values

Sr. No.	Method	Estimated C_h values (m^2/year)
1	Radial consolidation tests in laboratory	2 – 4.2
2	Field cone Dissipation tests	2.5 – 5.2
3	Back analysis based on field settlement data	3.75 – 4.0
4	Back analysis from α value	3.42 – 4.7

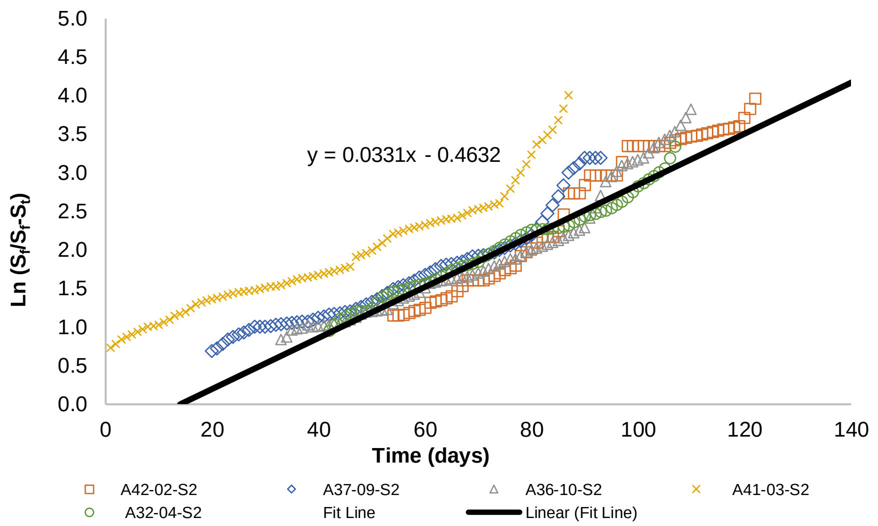

Fig. 26. Estimation of C_h from settlement gauges

8 Conclusion

This paper presents the ground improvement carried out at JNPT Terminal 4 for a reclamation of around 100 Ha. The subsoil comprised of very soft to soft clay with thicknesses varying from 4 to 22 m. To cater to the high loading and stringent settlement requirements, ground improvement using PVD with preloading was carried out successfully, with elaborate instrumentation and monitoring. Surcharge removal was based on Asaoka's graphical method. Based on the instrumentation reading back analyses for determining the consolidation parameters of the clay were also carried out.

From the contractor and owner's perspective the project was very well executed, to the satisfaction of the owner. The port is operational now in full swing.

Acknowledgement

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