ORIGINAL PAPER

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 fourth 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 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 m–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 (PVDs) with preloading. Radial consolidation and cone dissipation tests were carried out to establish the coefficient of horizontal consolidation (c_h).

The marine reclamation is confined on all sides by a perimeter bund with revetment, Stability analyses carried out for this bund showed that it was necessary to have overfill beyond the reclamation cope line along with tension geotextiles so as to achieve the prescribed factors of safety against slope failure. After removal of the preloading, 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.

Introduction

The Indian coast is laden with small and big ports catering to voluminous amount of trade. The coastal region around Mumbai houses the Jawaharlal Nehru Port Trust (JNPT) on the Navi Mumbai side near Uran. This area comprises very soft marine clays which are highly compressible and possess a very low bearing capacity. Three terminals already exist here which were built on this soft clay, the first terminal being built way back in the late 1980s.

Ground improvement becomes imperative in this area for the construction of container terminal, so as to handle large loads from containers. The most commonly adopted technique is the use of prefabricated vertical drains (PVDs) with preloading. This paper deals with case study of 100 Ha reclamation over very soft marine clays for the fourth terminal at JNPT, improved with PVD. The preloading to

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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.

Project Description

The project considered in this paper comprises a container yard with rail and road corridor over marine clay at JNPT in Navi Mumbai. The onshore terminal area is developed by reclaiming about 100 Ha of land over the Uran Mud Flat, adjacent to the existing terminals presented in Fig. 1. This terminal accommodates container stack yards and associated structures such as rail container depot (RCD), tank farms, port buildings, workshops, roads and utility buildings. The finished level is + 7 mCD and caters to permanent surcharge loads of 50 and 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.

The layout of the new terminal is shown in Fig. 2. The rectangular area is the container yard terminal measuring approximately 1000 m × 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.

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 89 numbers of boreholes through conventional drilling and sampling, 63 numbers of cone penetration tests (CPTs) with pore pressure measurement, cone dissipation tests and field vane shear tests (VSTs). 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. Manohar et al. [1] have given a brief description of the investigation.

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 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 sub-soil stratification across all zones.

Fig. 1 Existing terminals

Fig. 2 The proposed new terminal layout

Table 1 Subsoil stratification

Unit	Zone	Strata description	Thickness (m)
Unit 1	Zone 1	Soft marine clay	6–8
	Zone 2		8–10
	Zone 3		10–14
	Zone 4		10–14
	Zone 5		14–20
	Zone 6		16
	Zone 7		10–12
	Zone 8		8
	Zone 9		10–12
	Zone 10		14–16
Unit 2	NA	Sandy gravel	0.5–6.5
Unit 3		Amygdaloidal basalt	NA

Test Results and Design Parameters

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.

Atterberg Limits

The Atterberg limits are presented in Fig. 4. Liquid limit was observed to be in the range of 70%–110% with plasticity index around 30%–80%. The natural moisture content (NMC) varied from 60 to 120% and was close to the liquid limit.

Undrained Shear Strength

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

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

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

A number of field vane shear tests (VSTs) were conducted in close proximity to CPTs to estimate the correlation factor N_{kT}. N_{kT} factors were found to be higher in
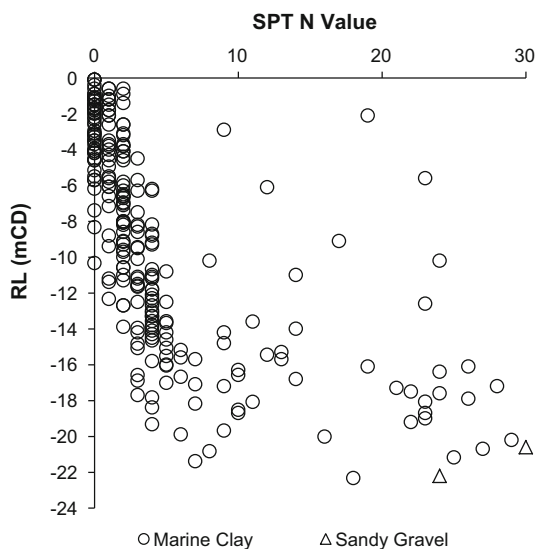
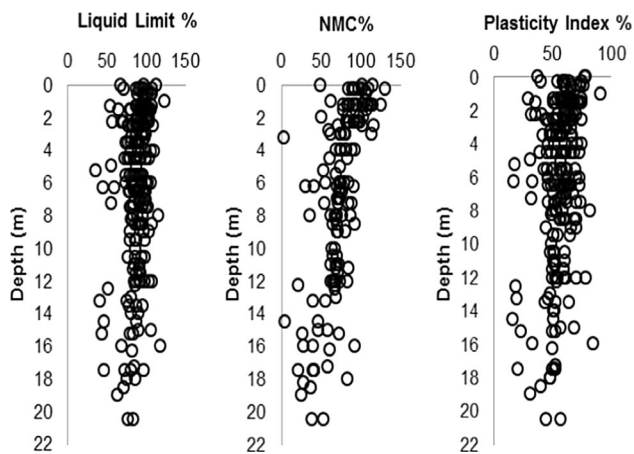**SPT N Value**

Fig. 3 Variation of SPT N with depth

Fig. 4 Atterberg limits and natural moisture content with depth

the shallower depth and reduced along the depth as shown in Table 2.

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
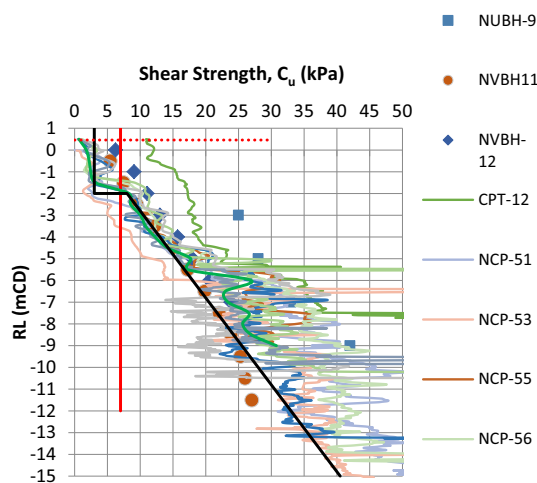

Fig. 5 Variation in undrained shear strength with depth

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.

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 0.15–0.35. The top 4 m–6 m of the clay shows an average CR of 0.25. Below 6 m depth, the average CR is around 0.18.

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 h after the end of primary consolidation. The measured C_α values were in the range of 0.015–0.035 as shown in Fig. 7. Figure 8 shows the compression index ratio C_α /C_c ratio of 0.033 which was found to be in similar range proposed by Mesri et al. [2].

Table 2 Variation of N_{kT} with depth

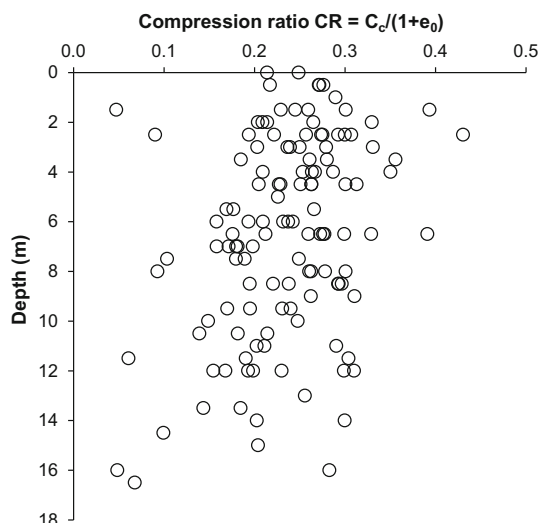| Depth	N_{KT}
0–5	25
5–6	20
> 6	15

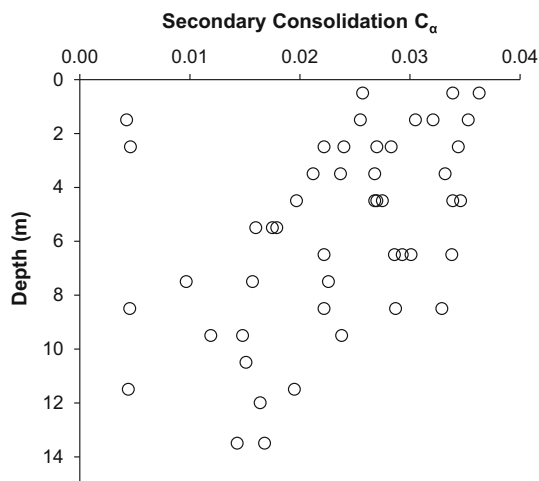**Fig. 6** Variation of compression ratio with depth

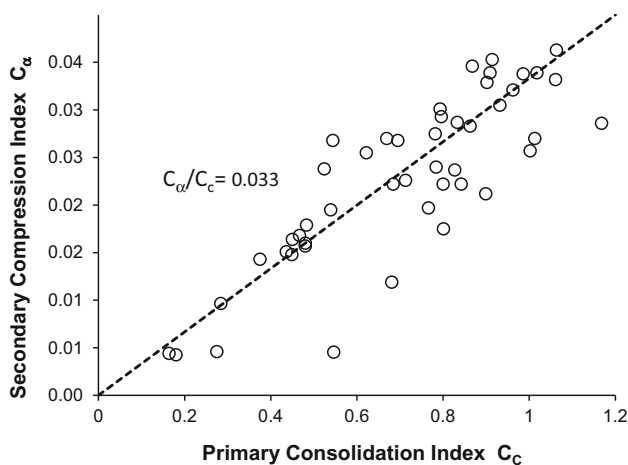**Fig. 7** Variation of secondary compression index with depth

Fig. 8 Compression index ratio C_α / C_c

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 m^2/year–2 m^2/year. Similarly, c_h value measured using radial consolidation tests was in the range of 1.9 m^2/year–4.25 m^2 /year, yielding a c_h / c_v ratio of 2.1–2.5. The variations of these parameters along the depth are shown in Fig. 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 [3]). The in situ C_h values were estimated to be in the range of 3.4 m^2/year–4.86 m^2/year and found to be in the similar range of values noted in radial consolidation tests.

Preconsolidation 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 2–5 in the top portion, which might presumably have resulted from the ageing and desiccation process. The OCR was found to be reducing with the depth as shown in Fig. 10.
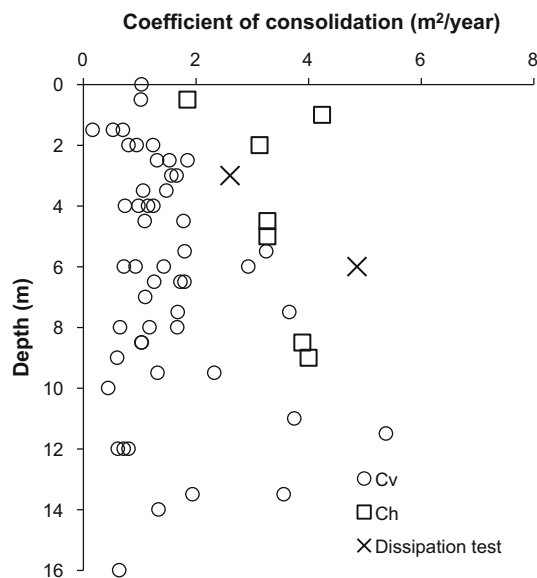

Fig. 9 Variation of coefficient of consolidation c_v and c_h with depth
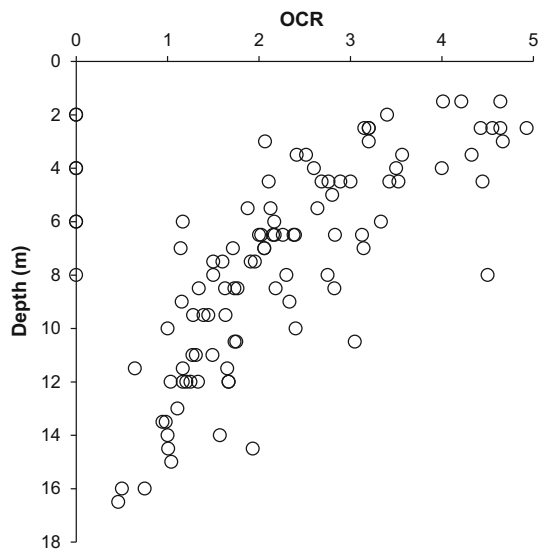
Fig. 10 Variation of OCR with depth

Organic Content

The organic content in the clay was between 5 and 10% as depicted in Fig. 11.

The geotechnical parameters of Unit 1 are derived and presented in Table 3

Design Criteria

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
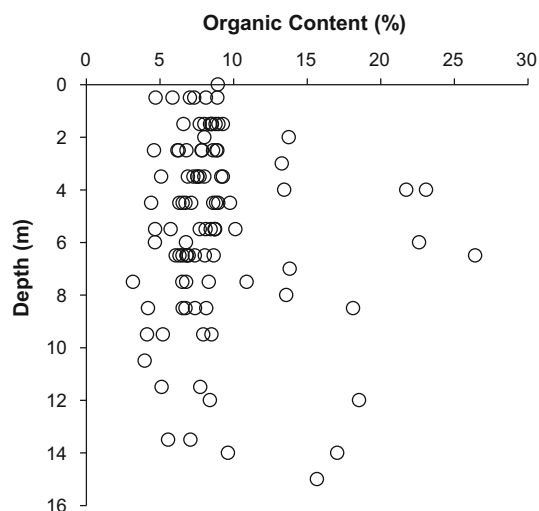

Fig. 11 Variation of organic content with depth

reclamation area had to be designed to withstand the operational loads given in Table 4 and satisfy the settlement criteria given in Table 5.

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

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 are as given in Table 6.

Ground Improvement using PVD

As is observed from the previous sections, the subsoil comprised very soft clay over the top 4 m–6 m. Beyond this depth, the shear strength increases (refer to Figure 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 specifications of the PVD used are mentioned in Table 7

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 8.

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:

- Stage 1 filling to + 5.5 mCD, above the highest high water tide by end on dumping.
- Install PVD, drainage blanket layer, PVD, geotextiles and instrumentation on stage 1 platform.
- 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.
- 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

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

Table 4 Terminal area loads

Sr. no	Area description	Design load (kPa)	
		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

Table 6 Factors of safety requirement

Condition	FOS
Temporary	1.3
Permanent static	1.5
Permanent seismic	1.1

in shear strength and then raise to final design surcharge level.

- 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.

- After reaching final surcharge level, wait until 95% consolidation is achieved.
- After completion of ground improvement surcharging, the surcharge is removed to + 6.0 mCD. At the edges, the overfill is cut back to the final geometry.

Table 7 Specifications of PVD

Sr. no	Property	Unit	Value
1	Material (core)	–	Polypropylene or HDPE
2	Material (filter)	–	PP or PET non-woven
3	Tensile strength	kN/m	Minimum 2
4	Elongation at 1.0 kN	%	< 10
5	Drain width	mm	100 nominal
6	Thickness	mm	Minimum 3

Table 8 Shear strength variation with depth

Area	From RL (mCD)	To RL(mCD)	Shear strength C_u (kPa)
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 3430—4160	0	−4	7.5 + 1.5 kPa/m
	-4	−8	13.5 + 3.0 kPa/m

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). The fill material was displacing the mud. In order to determine the actual thickness of replacement and the shear strength of the soft clay, confirmatory boreholes were conducted. Jeevan Reddy et al. [4] have described the formation of mudwave in soft clay.

Figure 13, which presents the top of marine clay before and after stage 1 filling, as seen from the confirmatory boreholes, shows that 2.5 m–3.5 m of the soft clay had been displaced and was replaced by the murrum fill.

Ground Improvement Design

A triangular pattern of PVD was adopted at 1.0- and 1.2-m spacing. The design was carried out as per IS 15284 Part 2 [5]. The preloading requirements for the various design loads mentioned in Table 4 worked out to those given in Table 9.

The preloading up to + 10.0 m CD to + 12mCD was built up in three stage lifts with the waiting period that was allowed between stages 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

Fig. 12 Formation of mudwave ahead of the fill

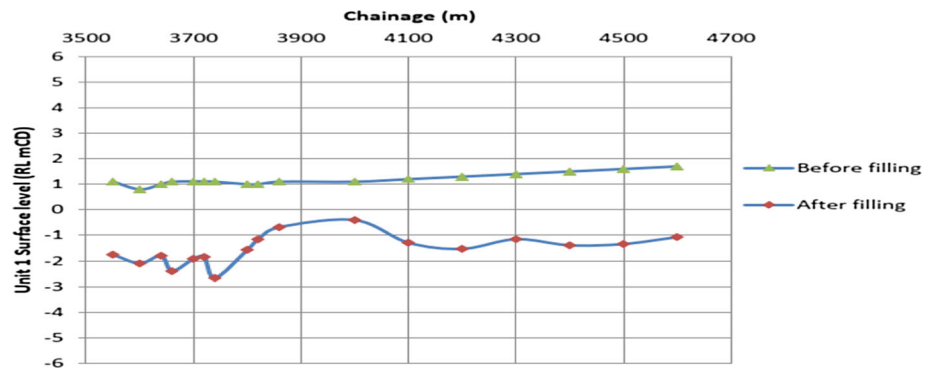**Fig. 13** Top of marine clay: before and after filling

Table 9 Design preloading levels

Area	Preloading level (mCD)
Container yard	+ 12.5
Rail area access road and gate	+ 10.5
Buildings and car park	+ 10.0

settlement and the absolute value of the total primary consolidation settlement varied across the site. The time varied from 75 to 120 days for 1.0-m spacing of PVD, and settlements varied from 1.0 m to 2.5 m.

Stability of Edge Slopes

The virgin shear strength of the soft clay as presented in Fig. 5 or Table 8 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 recognized software Slope/W by Geoslope Intl. [6]

Slope/W has been used in the market since 1977. Initially, it was developed by Professor D.G. Fredlund at the University of Saskatchewan. Slope/W effectively analyses both simple and complex problems for a variety of slip surface shapes, pore water pressure conditions, soil properties, analysis methods and loading conditions. The Morgenstern–Price method has been used to calculate factors of safety. Circular slips have been considered.

At every stage of construction, the FOS of 1.3 had to be maintained as mentioned in Table 6. Since the first 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 10.

Table 11 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.

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

$$\frac{\Delta C_u}{\Delta \sigma'_v} = \left[0.11 + 0.37 \cdot \frac{\text{PI}(\%)}{100}\right] \times U \qquad (2)$$

where ΔC_u is the increase in undrained shear strength and $\Delta \sigma'_v$ is the increase in effective stress. For PI value of 65%, this increase works out to be 0.35U, where U is the percentage consolidation.

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 × 1.05 × 1.05 × 1.00 = 1.6

and partial factor for pull out = 1.5

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

Table 10 Design parameters of reclamation fill

Description	Unit weight (kN/m³)	Shear strength parameters C_u (kPa)	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 11 Shear strength required at top of marine clay at one particular edge

Fill levels	Shear strength c_u (kPa)
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

Slope Stability Analyses

Temporary Construction stage

Slope stability analyses were carried out for every stage of construction in order to achieve the required FOS. Circular and non-circular slip surfaces using the Morgenstern–Price method were used to calculate the FOS. Figures 14 and 15 present the critical circular and non-circular slip surfaces for the final stage, i.e. stage 3.

The analyses considered the increased shear strength resulting from partial consolidation from the previous stage. Table 5 shows the FOS values for the different stages of filling. In order to avoid unrealistic failure mechanisms, care is taken to ensure that the middle segment line is longer than the two end projection segment lines, in the sliding block failure mechanism.

The FOS values for different stages of filling for circular and non-circular surfaces are depicted in Table 12. Since the minimum FOS of 1.3 is achieved in stage 3 filling, this stage dictated the geometry of the temporary filling. This resulted in 12-m overfilling beyond the cope line and the adoption of tension geotextiles.

Permanent Condition

The preloading was removed after 95% consolidation was achieved and the final level of + 7 mCD was maintained. The overfill beyond the cope line, which was necessary to achieve the required FOS for the stage 3 filling, was cut back. The perimeter bund was checked for the long-term static and seismic cases using Slope/W. A surcharge loading of 30 kPa for main reclamation and 10 kPa for service road were considered for static case. The critical slip surfaces for the static and seismic cases are presented in Figs. 16 and 17, respectively. Table 13 presents the FOS values for both cases which are greater than 1.5 and 1.1, respectively.

For performing the seismic analysis, IITK-GSDMA guidelines [7] for seismic zone 4 were used. IS 1893 [8] is

Fig. 14 Model of slope stability for + 10.5 mCD level fill (circular failure) —construction stage
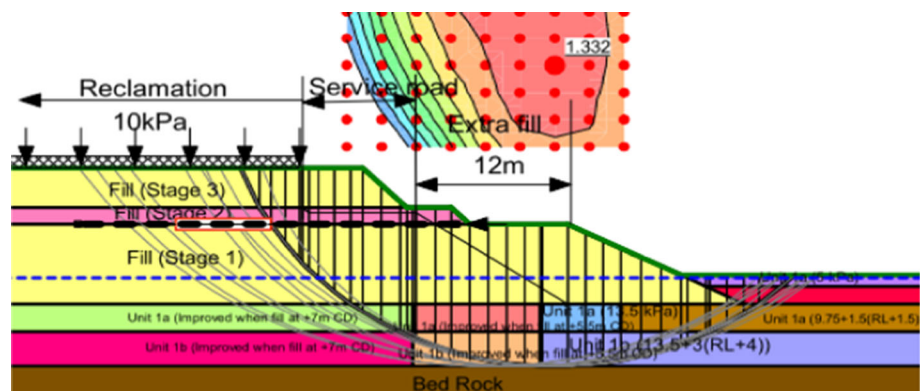
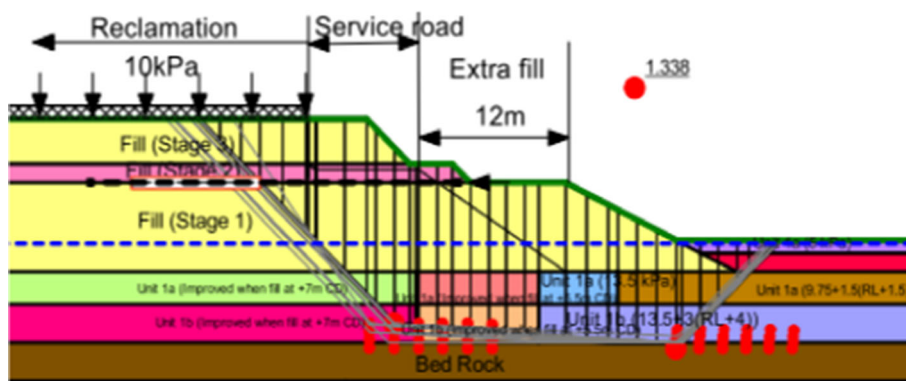**Fig. 15** Model of slope stability for + 10.5 mCD level fill (sliding failure)—construction stage

Table 12 FOS values for different stages of fill levels

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

Fig. 16 Slope stability model for permanent static case
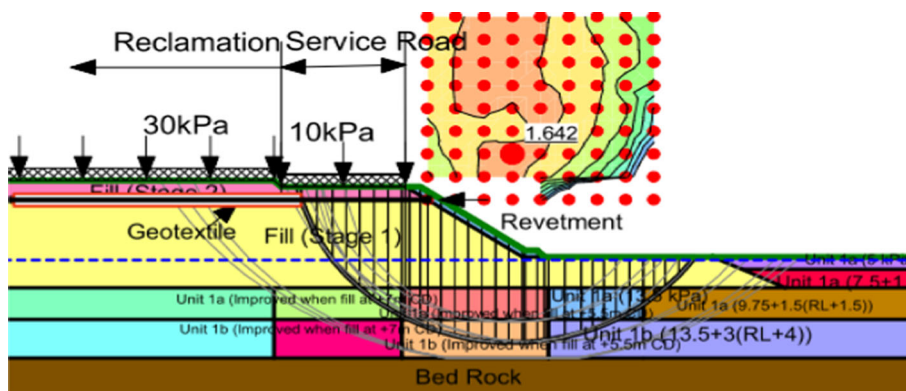
also referred to. The horizontal (k_h) and vertical (k_v) seismic coefficients were derived as given below:

$k_h = 1/3 \times Z \times I \times S$ and $k_v = 0$, where Z = 0.24, I = 1.5 and S = 1.2 and surcharge loading is reduced by 50%.

The geometry for the three stages of loading, as arrived from the slope stability analyses, is presented in Fig. 18.

Instrumentation and Monitoring

In order to know the settlement behaviour of the soft clay, whether the prescribed primary consolidation is over and the deformation of the side bunds, it becomes imperative to carry out instrumentation and monitor them at regular intervals. 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. [9]), uncertainties and variations associated with the ground condition and

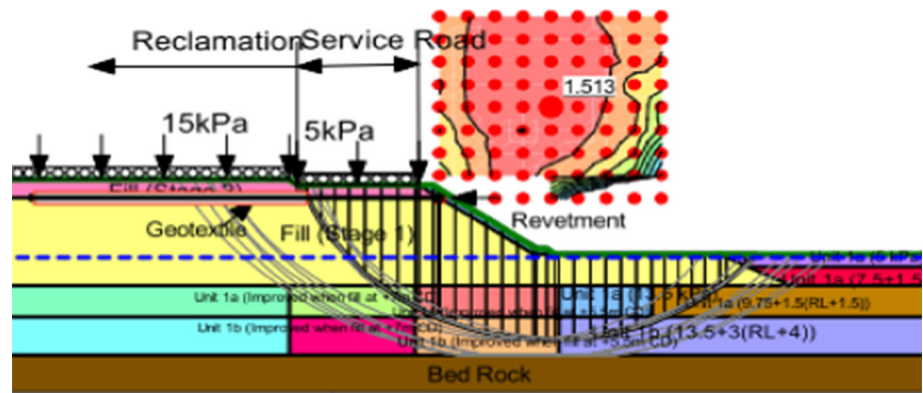**Fig. 17** Slope stability model for permanent seismic case

Table 13 FOS values for permanent static and seismic cases

Stages	Fill levels	Critical FOS	
1	+ 7.0 mCD static	Circular	1.64
		Sliding shallow	2.20
		Sliding deep	1.86
2	+ 7.0 mCD seismic	Circular	1.51
		Sliding shallow	1.97
		Sliding deep	1.68
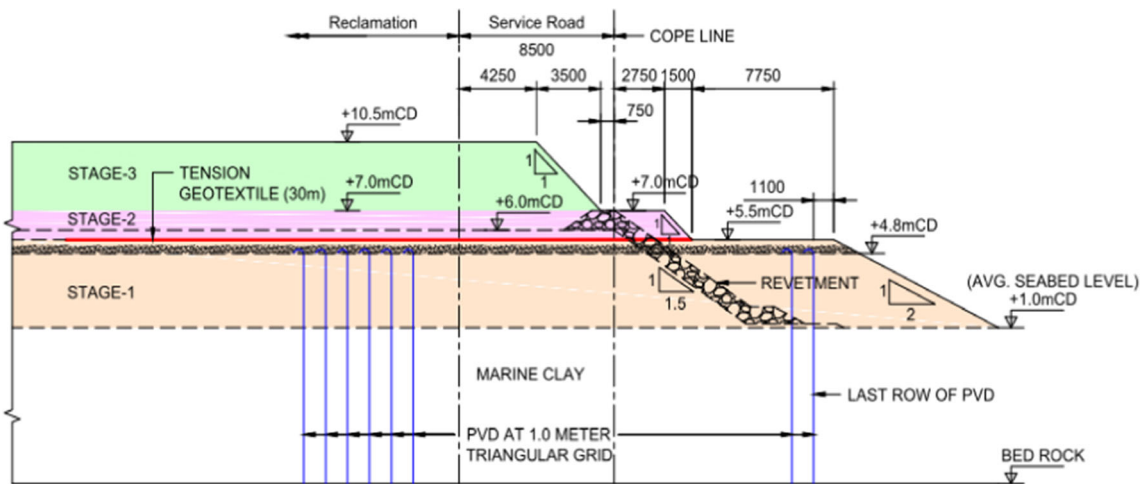
Fig. 18 Geometry arrived from slope stability analyses

geotechnical parameters. Large number of instruments were installed within the site as presented in Fig. 19. The instrumentation comprised open standpipe, vibrating wire piezometers, surface settlement markers, deep settlement markers, inclinometers and magnetic extensometers. Some of the instruments are presented in Figs. 20 through 22. Clusters were installed in grids of 25 m × 25 m, and a total of around 1000 numbers if instruments were installed. A 100 m × 100 m size grid comprised 25 settlement markers and other instruments and boreholes.

Inclinometers were provided at every 100-m spacing along the reclamation boundary to monitor the lateral movement and stability during preloading. Ashok et al. [10] have described about instrumentation.

Surcharge Removal

Various methods are available for the back-analysis of instrumentation data. Sridharan et al. [11] have suggested rectangular hyperbola method for predicting the magnitude
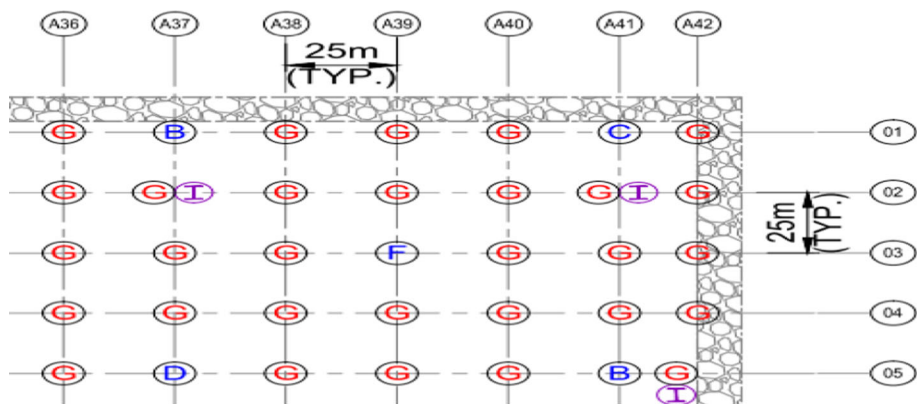
Fig. 19 Instrumentation layout plan

Fig. 20 Pore water measurements by vibrating wire piezometer

of primary compression from tests. 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 [12] was adopted using field measurement data from the instrumentation. The settlement data from settlement markers and magnetic extensometers were used in the Asaoka plots.

Fig. 21 Lateral deformation measurements by inclinometer
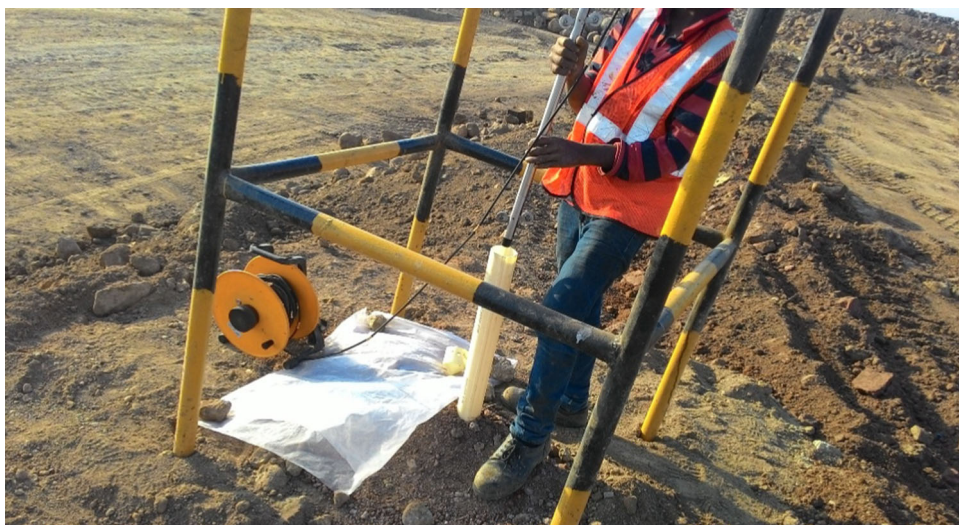
Fig. 22 Settlement measurements by magnetic extensometer

A typical Asaoka plot from one of the settlement gauges is presented in Fig. 23.

Figure 24 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.

Back-Analyses

In order to verify and back-calculate the various design parameters, back-analyses were performed using the data from the instrumentation. The prediction based on multi-level magnetic extensometers is presented in Fig. 25, and those from settlement gauges are presented in Fig. 26. Each magnetometer had 5–7 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.

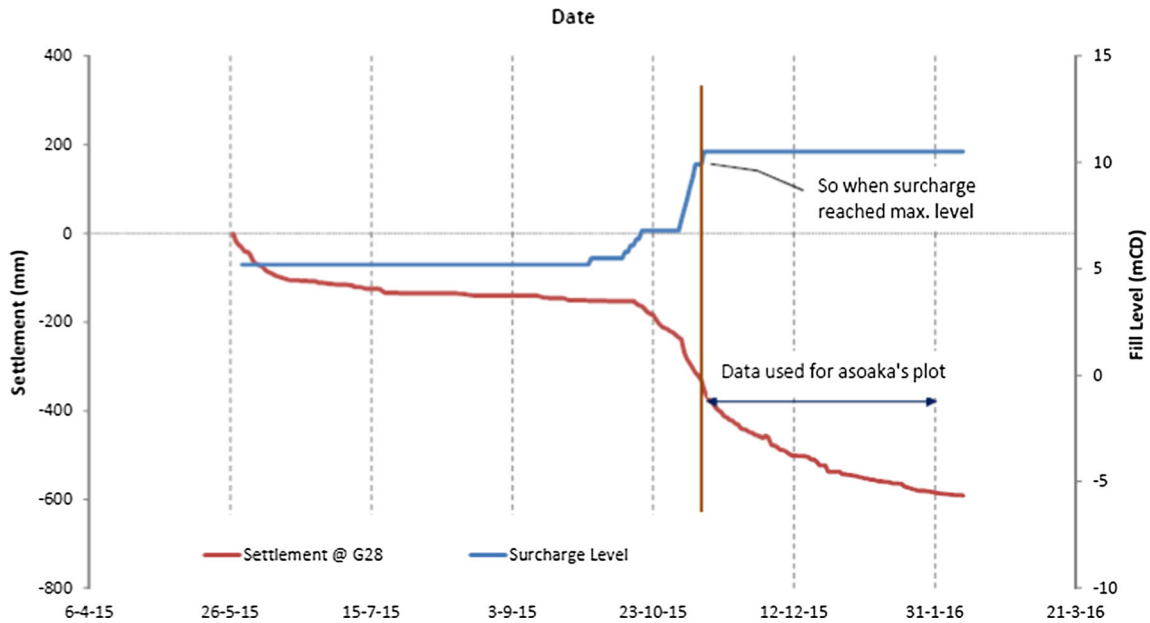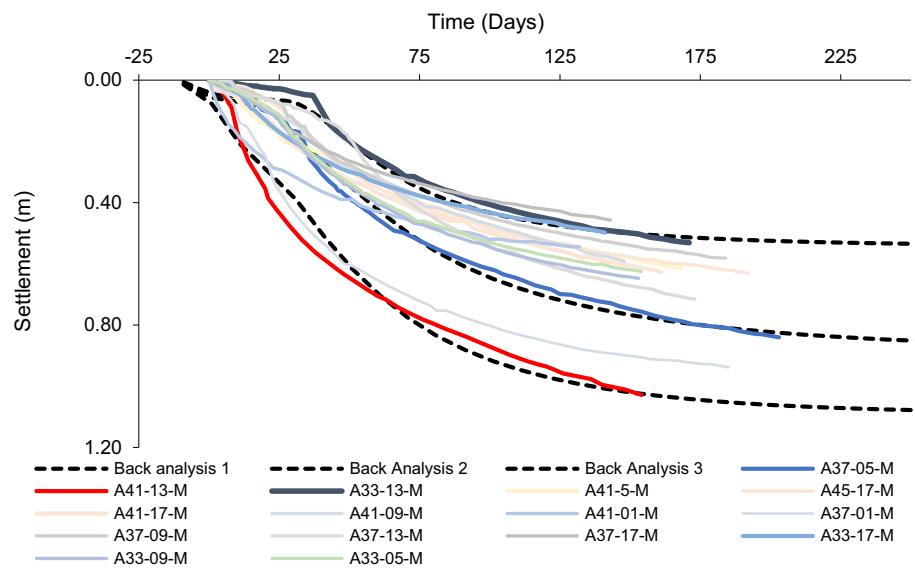**Fig. 23** Typical Asaoka plot for a settlement gauge
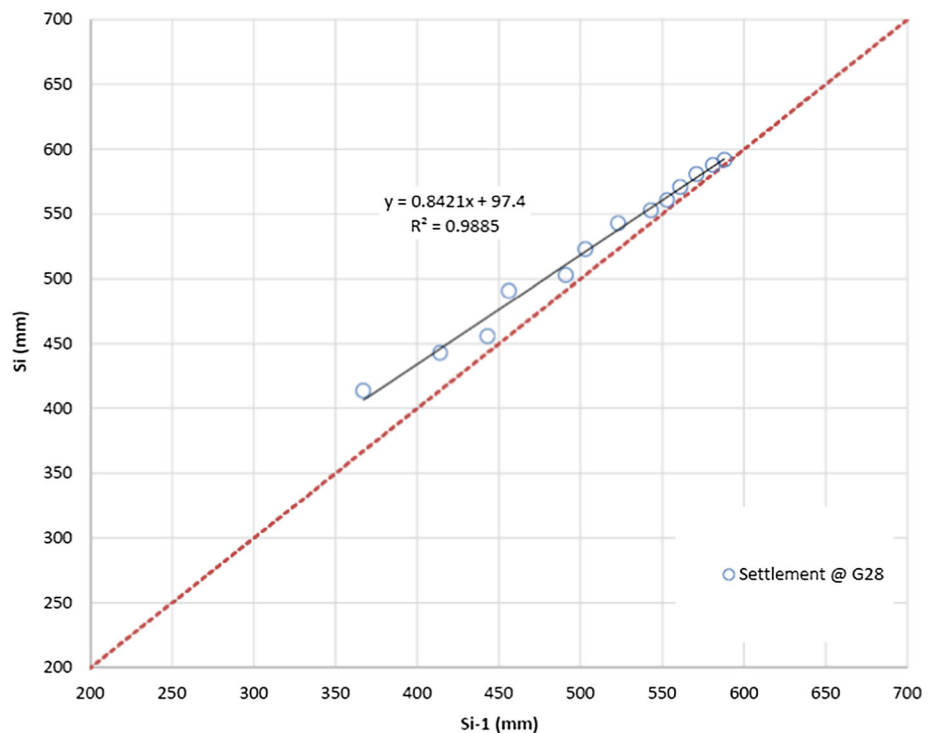
Fig. 24 Plot of settlement versus time and raising of the reclamation fill

Fig. 25 Back-analyses:
magnetic extensometer data

In performing the back-analyses, adjustments to the compression ratio (CR) and the horizontal coefficient of consolidation (C_h) were carried out so as to match the theoretical predicted curve. It was observed that horizontal coefficient of consolidation C_h of around 3.75 m²/year–4.0 m²/year with C_h / C_v ratio around 2.3 fitted the curve. The smearing permeability ratio and smear diameter ratio were estimated to be in the range of 3–4 and 2.5, respectively.

C_h Back-Calculation from Piezometer Data

Hansbo's [13] equation can be rewritten as under:

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

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

where Δu_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). Equation 3 can be rewritten as,

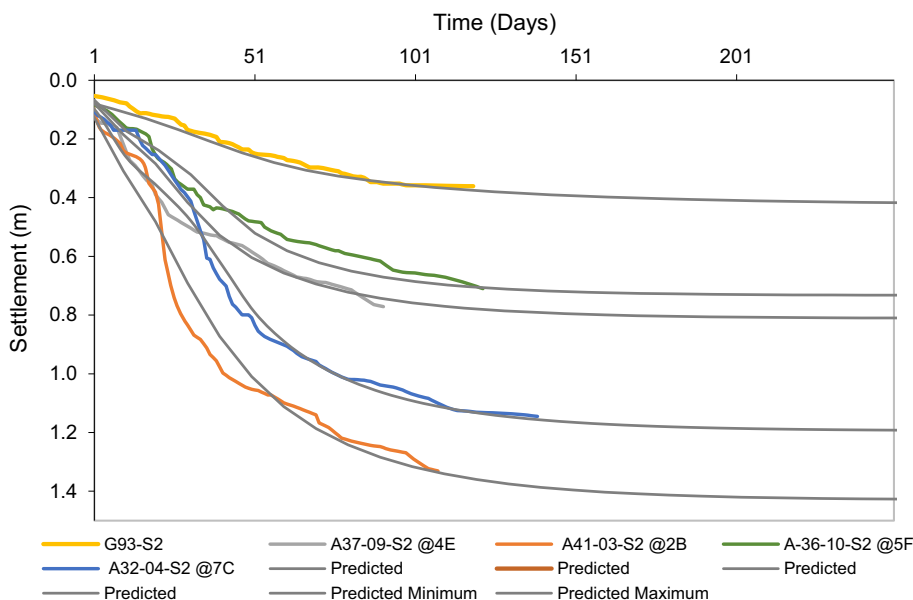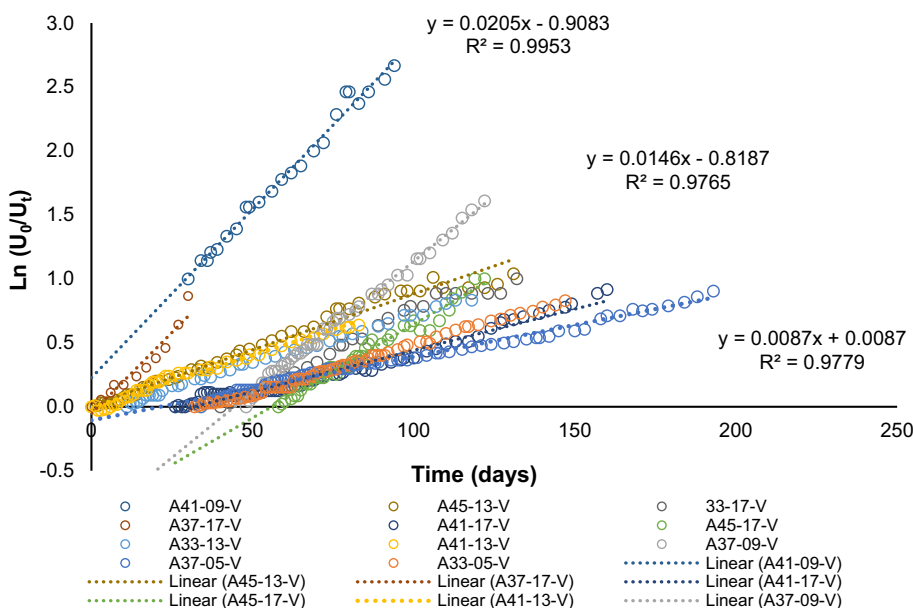**Fig. 26** Back-analyses: settlement gauge data

Fig. 27 Estimation of C_h from piezometer data

$$\ln = \frac{\Delta u_0}{\Delta u_t} \alpha t \tag{5}$$

It can be seen that Eq. (5) is linear with time, with α being the slope of the line. C_h can now be back-calculated directly from the piezometer data.

Figure 27 depicts that the slopes of the various lines vary from 0.008 to 0.02. Using these values through back-analysis, the C_h values work out to be 1.8 m²/year–4.7 m²/year. From Figs. 22 and 23, the smearing diameter ratio and permeability ratio are back-analysed as 2.5 and 4, respectively. Slope less than 0.008, resulting in C_h of less than 2.0 m²/year, was ignored since the dissipation of

excess pore water pressure was observed to be improper in some piezometers due to probable clogging of filter.

Back-Calculation of C_h from Magnetic extensometer

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

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

$$\ln \frac{S_t}{S_f - S_t} = \alpha t \tag{7}$$

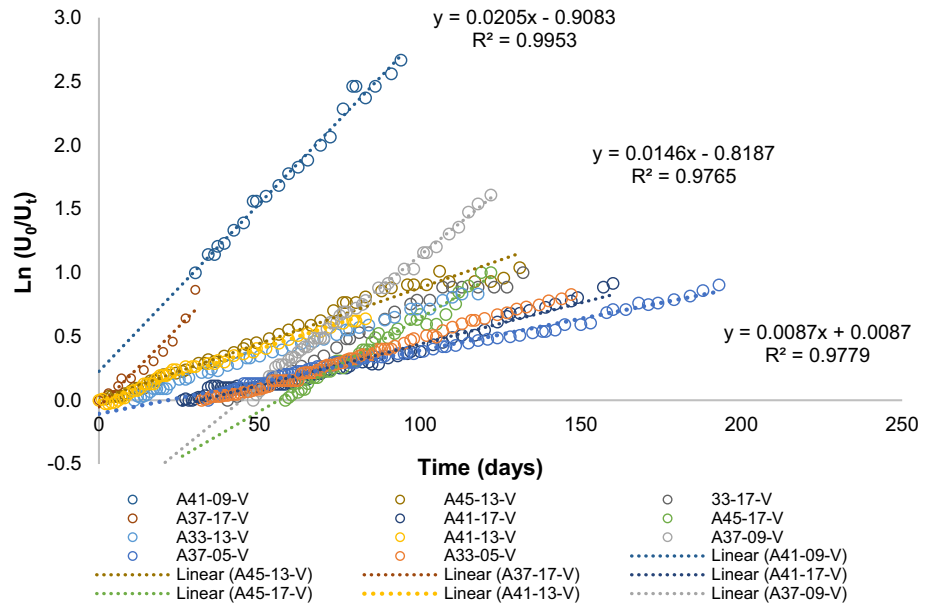**Fig. 28** Estimation of C_h from magnetic extensometer data
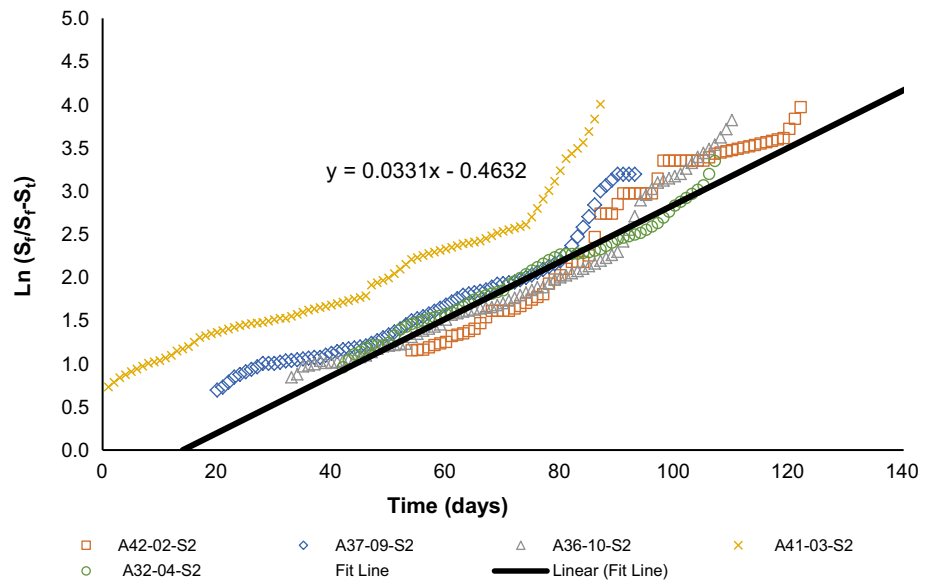

Fig. 29 C_h estimation from settlement gauges

Table 14 Summary of estimated C_h values

Sr. no	Method	Estimated C_h values (m²/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

where S_f is the final settlement and S_t is the settlement at any time t. Equation 7 is linear and can be used to estimate the slope α from the field settlement data.

From Fig. 28, the value of the slope of the straight line, α = 0.0185. From this, the C_h is back-calculated as 3.7 m²/year for smearing diameter ratio of 2.5 and permeability ratio of 3.

Back-Calculation of C_h from Settlement gauges

α was back-calculated from the data from settlement gauges installed at + 5.5 mCD as 0.033, and the resulting C_h value of about 4.7 m^2/yr was obtained (Fig. 29).

C_h value as evaluated from these 3 methods varied from 2 to 5.2 and is presented in Table 14.

Conclusion

This paper presents the ground improvement carried out at JNPT Terminal 4 for a reclamation of around 100 Ha. The subsoil comprised 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, and it was observed that the derived design parameters were in line with the observed back-calculated values.

Some learnings from the work are listed below:

(1) Adequate number of instruments are necessary to get a fairly good idea of the soil behaviour.
(2) Many instruments would get damaged during the construction or some may stop working. Hence, provision should be made for such eventualities.
(3) Vibrating wire piezometers may not measure increase in pore pressures during the loading process. One should be prepared for this.
(4) It is good practice to rely data from settlement gauges and mechanical instruments and then compare the results with non mechanical type instruments.
(5) Provision for elaborate instrumentation during the tendering process is very critical. This could help in 'visualizing' the actual behaviour and eventually gaining confidence in the futuristic behaviour of the reclamation.
(6) Smear plays a very important role in accelerating consolidation settlements. It was observed that the smear zone was larger in deeper clay deposits. Evaluation of smear zone is a difficult task. However, a reasonable evaluation would help in evaluating a reasonably accurate time for primary consolidation settlements.
(7) If evaluation of design parameters is fairly accurate and the design is robust, then the value and time for settlements would be lesser than design estimates.

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.

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