STABILITY ANALYSES FOR RECLAMATION BUND ON MARINE CLAY

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ABSTRACT:

A container yard is being developed near Mumbai by creating around 90 Ha of land through reclamation in sea over soft, compressible marine clay. The thickness of the marine clay varies from 4 to 18 m over the reclamation area underlain by Basaltic rock. The strength of the soft clay varies from very soft to stiff clay, and susceptible to large settlements and shear failure. This stratum cannot support the reclamation fill by its own, hence Ground improvement is essential to improve the strength and stiffness properties so that the finished reclamation is able to satisfy the serviceability criteria. Stability analyses of perimeter bund confining the marine reclamation were examined for all probable surfaces including circular, non-circular and sliding failures as per BS 6031. The analyses show that overfill sections beyond the permanent reclamation cope line are 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 is removed, the overfill is cut back to the final geometry and revetment is provided along the perimeter bund.

KEYWORDS: Marine Clay, Ground Improvement, PVD, Surcharge, Slope Stability.

1 INTRODUCTION

In any marine reclamation project, construction of a confinement bund around the area of reclamation is the first activity. The filling is carried out within this confined area either hydraulically, if marine sand is available, or by end on dumping if filling is material from nearby borrow sites. The stability of the slopes of this reclamation bund is crucial as the slightest failure can be destructive in terms of material loss into the sea, loss of time thus resulting in financial losses. Time becomes all the more critical in private projects because the Liquidity damages levied for delay in completion is huge. In this context the reclamation bund slopes need to be carefully analyzed for stability, prior to filling, during filling and post construction.

The project considered in this paper comprises of a proposed container yard with rail & road corridor over marine clay near Mumbai 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, i.e. 0 mCD = -2.51MSL. The ground had to be improved using PVD and preloading until 95% consolidation is achieved.

The section described in this paper pertains to the rail/road portion. The requirement is that the permanent surcharge load of 30 kPa is expected to act within the rail/road portion and traffic load of 10 kPa to act all the way to the cope line, i.e. to the edge of the finished geometry. Hence additional filling beyond the cope line was warranted with ground improvement. This guaranteed that the Factors of Safety as described in section 3.2 are achieved.

The current paper briefly describes the stability analyses for edge slopes of the reclamation during the construction at different stages of loading with fill materials. The edge structure is also analyzed for the permanent edge slopes for long-term serviceability.

2 GEOTECHNICAL DATA

2.1 Geotechnical profile, Parameters-Virgin State

A detailed review of the geotechnical investigation data was carried out to identify the variations in the geology across the footprint of the reclamation areas. The sub soil profile over the entire site comprises marine clay with thickness varying from 4 to 18 m followed by basaltic rock.

The shear strength profile, parameters and corresponding design line for the selected sections are presented in Figure-1 and the corresponding design shear strength parameters for marine clay are presented in Table 1. It is seen that the clay is extremely soft close to the sea bed (+1 mCD) and increases gradually with depth.

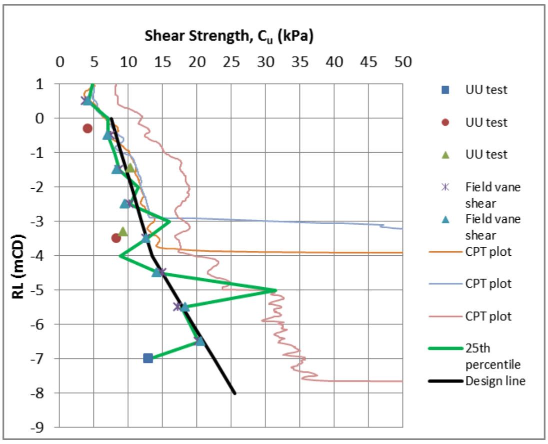

Fig. 1 Plot of Shear Strength versus RL

Table 1 Geotechnical design parameters (Virgin state)

Unit weight [kN/m^3]	Shear Strength Parameters, c_u [kPa]
15	7.5 + 1.5 kPa/m from 0.0 to -4.0 mCD 13.5 + 3.0 kPa/m below -4.0 mCD

2.2 Geotechnical profile, Parameters-After Stage 1 filling

The filling proceeded with land based filling until the fill reached a level of +5.5 mCD above the maximum water level from which level PVD were installed for ground improvement. This was termed as stage 1 filling. The filling indicated that the top 2 to 3 m of the soft clay was getting displaced thus resulting in a mudwave. This displaced mud was getting replaced by the fill material. Confirmatory boreholes were conducted to witness the actual thickness of replacement and the shear strength of the soft clay.

Figure 2 presents the top of marine clay before and after stage 1 filling, as evidenced from the confirmatory boreholes.

From the Figure 2 it is clear that 2.5 to 3.5 m of the soft clay had displaced and was replaced by the murrum fill.

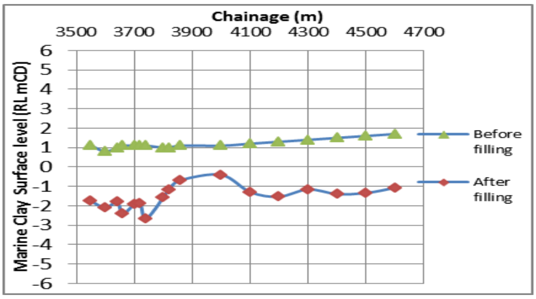

Fig. 2 Top of marine clay: Before and after filling

Figure 3 presents the design shear strength after the replacement.

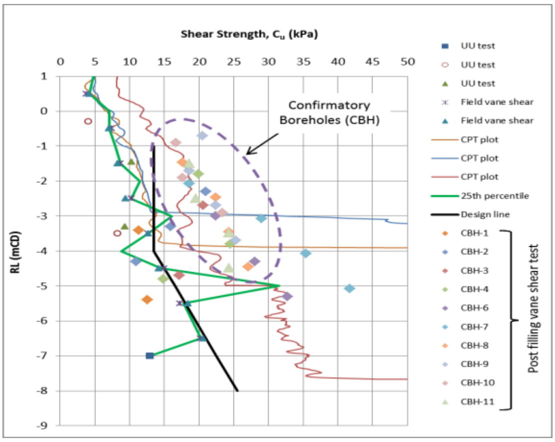

Fig. 3 Design Shear Strength Profile considered in analysis of edge slope

It is seen that the shear strength of soft clay, 2 to 3 m below the fill are higher than the virgin shear strength. Beyond this depth the shear strength is close to the virgin shear strength. In the analysis a replacement of the top 2.5 m during the filling process is considered and shear strength to a level of -4 mCD is considered a constant 13.5 kPa. Below -4 mCD the shear strength follows the same trend as given in Figure 1or Table1.

2.3 Geotechnical parameters for Reclamation fill

Table 2 Geotechnical design parameters for Reclamation Fill

Description	Unit weight [kN/m^3]	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 2 presents the design parameters for the reclamation fill that are used for analyses.

3 GROUND IMPROVEMENT AND RECLAMATION

3.1 Construction methodology and Sequence

The ground improvement scheme for the reclamation comprises of the installation of pre-fabricated vertical drains (PVD) from stage 1 spaced at 1.0 m c/c in a triangular grid pattern and the application of a surcharge pre-loading to expedite consolidation and generate strength gain.

Table 3 Shear strength required at top of Marine clay

Sl No	Fill Levels	c_u [kPa]
1	Seabed to +5.5 mCD	13.5
2	From +5.5 mCD to +7.0 mCD	26
3	+7.0 mCD to surcharge top	34
Permanent Condition (After 95% consolidation)		
4	At +10.5 mCD	56

The filling is carried out in stages so as to achieve the shear strength required to raise to the next level. The stages and the required gain in shear strength are presented in Table 3. In order to confirm that the required shear strength is achieved, confirmatory vane shear tests were carried out at every stage after the design waiting period.

3.2 Design Requirements

The required factors of safety for edge stability are tabulated below. It may be noted that the FOS during temporary stage is as high as 1.3 mainly to avoid excessive deformation resulting in bending of PVD.

Table 4 Factors of Safety Requirement

Condition	FOS
Temporary	1.3
Permanent static & seismic	1.5 & 1.1

3.3 Tension Geotextile

It was proposed to use 1 layer of tension geotextile at +5.5 mCD, so as to satisfy the FOS requirements in the temporary and permanent stages. In the stability analysis the following parameters for the Geotextile reinforcement are adopted based on the data sheet given by the suppliers.

Partial Factor (PF) for Tensile Capacity = 1.6

This is derived as = 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 and Partial Factor for Pull out = 1.5

The Geotextile reinforcement at +5.5 mCD level has a length of 30m and an ultimate tensile capacity of 600 kN/m.

4 SLOPE STABILITY ANALYSIS

Static slope stability analyses were carried out using the industry recognized software Slope/W licensed by Geoslope Inc. (version 2012). The Morgenstern-Price Method was used to calculate factors of safety reported in this section. Both circular and non-circular slips were considered.

4.1 Temporary condition (Construction Stage)

A traffic load of 10 kPa was assumed on the top of each stage loading as per CIRIA C580.

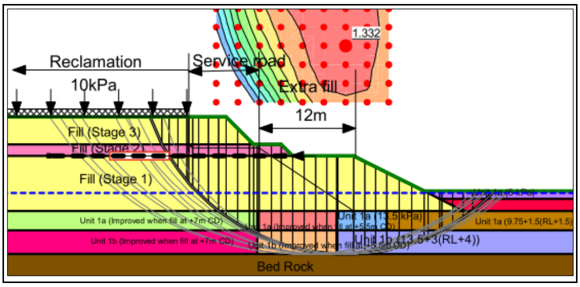

Fig.4 Slope stability model for +10.5 mCD level fill (Circular failure)

Critical Circular and Sliding block slip surfaces are checked and presented in Figures 4 & 5 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.

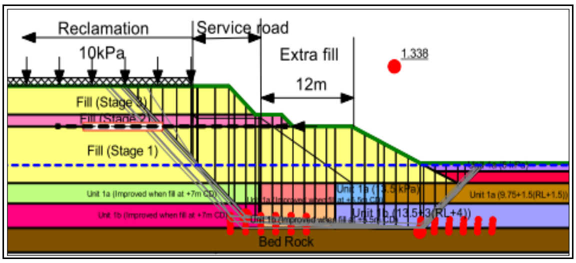

Fig.5 Slope stability model for +10.5 mCD level fill (Sliding failure)

From Table 5 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.

Table 5 FOS values for different stages of fill levels

Stages	Fill Levels	Critical FOS Circular/Non-circular	
1	Bed level to +5.5 mCD	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	+7.0mCD to +10.5mCD	Circular	1.33
		Sliding Shallow	1.59
		Sliding Deep	1.33

4.2 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 is 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.

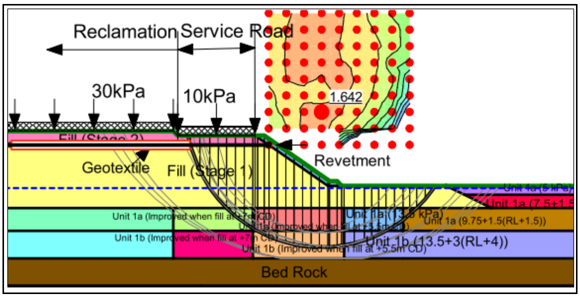

Fig.6 Slope stability model for permanent static case

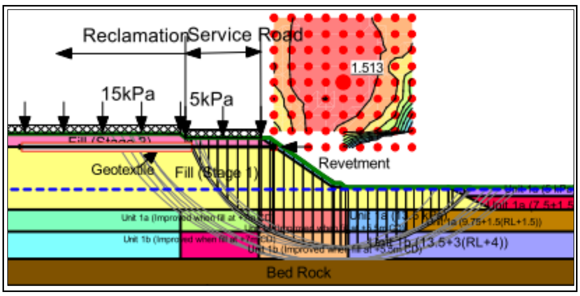

Fig.7 Slope stability model for permanent seismic case

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%.

Figures 6 and 7 present the critical slip surfaces for the static and seismic cases respectively. Table 6 presents the FOS values for both cases which are greater than 1.5 and 1.1 respectively.

Table 6 FOS values for permanent static &seismic case

Sl. No	Fill Levels	Critical FOS Circular/Non-circular	
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

Figure 8 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.

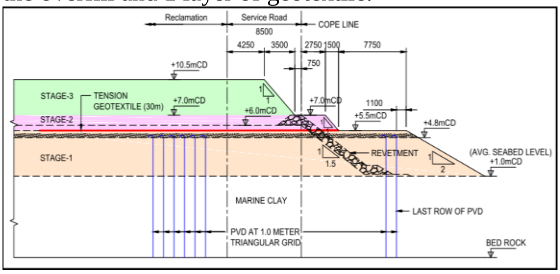

Fig.8 Geometry arrived from slope stability analyses

5 SUMMARY AND CONCLUSIONS

This paper presents the slope stability analyses of reclamation bund on soft clay improved with PVD and surcharge loading. The analyses show that the filling has to be carried out in stages so as to achieve the required FOS of 1.3 in the temporary stage. An over fill of 12m beyond the cope line & a layer of geotextile were necessitated in order to support a surcharge load to +10.5 mCD corresponding to a design load of 30 kPa. At every stage of construction the theoretical gain in shear strength was confirmed with confirmatory vane shear tests before raising to the next stage. This paper demonstrates the importance of carrying out detailed slope stability analyses for marine reclamation.

References

BS: 6031 (2009), *Code of practice for earth works*, British standards

IITK-GSDMA (2007), *Guidelines for Seismic Design of Earth Dams and Embankments*.

IS: 1893 (Part 1) (2002), *Criteria for Earthquake Resistant Design of Structures*, Bureau of Indian Standards, New Delhi, India

Stability modelling with Slope/W Manual. *Geo-Slope International Ltd*., December 2012.