Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering

doi:10.3233/978-1-61499-656-9-1861

Failure of a highway embankment on soft clay foundation – A case study with remedial measures

L'echec d'un remblai de route sur la fondation d'argile douce – Une etude de cas avec les mesures reparatrices

RKM Bhandari* & Pinaki Roychowdhury
LEA Associates South Asia Private Limited
B-1/E-27, II[nd] Floor, Mohan Cooperative Industrial Estate, Mathura Road, New Delhi-110044, India
e-mail: rkmbhandari@yahoo.co.in

ABSTRACT

The paper presents a case history of a highway embankment which failed while being constructed over soft to very soft marine clays improved with band drains and preloading. The failure occurred at a height greater than the height determined from the bearing capacity considerations. The paper examines the reasons and nature of failure and briefly describes the remedial measures that were proposed for implementation during reconstruction of the embankment. The measures suggested included provision of basal reinforcement, construction of wide berms and providing band drains at closer spacing to achieve early strength grains. In addition, staged construction was recommended besides checking out performance of the embankment through a comprehensive instrumentation programme. Critical considerations involved in the design of prefabricated vertical band drains are presented and discussed in relation to the project site conditions.

RÉSUMÉ

Le papier présente une histoire de cas d'un remblai de route qui a échoué pendant qu'étant construit par-dessus doux aux argiles marines très douces a amélioré avec les égouts de bande et preloading. L'échec est arrivé à une hauteur plus grande que la hauteur a déterminé des considérations de capacité de maintien. Le papier examine les raisons et la nature d'échec et décrit brièvement les mesures réparatrices qui a été proposée pour l'implémentation pendant la reconstruction du remblai. Les mesures ont suggéré ont inclus la provision de renforcement basal, la construction de berms large et de fournir les égouts de bande à espaçant plus près pour atteindre les grains de force premiers. En plus, la construction montée a été recommandée en plus payant la note l'exécution du remblai par une émission d'instrumentation complète. Les considérations critiques ont impliqué dans la conception d'égouts de bande verticaux préfabriqués sont présenté et est discuté par rapport aux conditions de site de projet.

1 INTRODUCTION

Under the infrastructure development programme launched by Govt. of India, M/s National Highway Authority of India (NHAI) planned to construct a 10.5 km stretch of highway linking the Vishakhapatnam (Vizag) Port with the National Highway NH5 near the Ayyappa Temple on the outskirts of the Vizag town. The Eastern and Western Coastal belts of India are known to accommodate soft to very soft highly compressible clays and the Vizag site located on the eastern coast appeared to be no exception. However, for construction of the Vizag connectivity road project, ground improvement of the very weak foundation clays formed an essential feature which was carried out with the twin objectives of minimizing the post-construction settlements as well as the long term maintenance costs.

2 THE PROJECT

The project stretch was divided into the following three sections based on the foundation type:

(i) Main Flyover: This structure with a length of 2.495 km (ch. 0.138 km to 2.663 km) was planned to bridge over a large number of railway lines and road crossings falling in the corridor at this section. The flyover was supported on 1.2m diameter bored cast-in-situ piles with their tips placed in hard rock with a socket length of 1.5m (1.25 D, D being the pile diameter). The hard rock below each foundation was located based on the field geo-electrical studies of the stretch, Rao et.al. (2003).

(ii) The Ground Improvement Zone (G.I. Zone): This is a two lane roadway stretch of nearly 4.142 km length (ch. 2.633 km to 6.775 km). Here, the embankment was supported on ground improved by band drains and preloading with the provision that the given treatment must achieve 90% degree of consolidation in 90 days with band drains installed in triangular pattern at 2.0m c/c spacing.

(iii) The Four Laning Portion: This covers nearly 3.6 km long stretch between ch. 6.775 km to 10.366 km. The ground in this zone was improved by removing the weak soils to their limited depth and filling back the excavations with compacted granular material, except for a limited stretch of 425m which was underlain by soft clays between ch. 6.775 km to 7.200 km. This latter section was improved by employing the band drains having the same characteristics as mentioned in (ii) above.

Of the three sections, design and construction of embankments in the GI zone presented a major challenge to the geotechnical engineer due to presence of very soft to soft clays which extended to considerable depths. The following sections deal with analysis and design of embankments in this particular zone with special reference to the failure of embankment experienced in the 170m long Section A within km 4.250 to km 4.420.

3 SUBSURFACE CONDITIONS

For preparation of the Detailed Project Report (DPR), the subsoil information was originally collected through boreholes performed using wash boring. Application of this technique to the Vizag soil was questionable since it resulted in washing out of fines from the clayey soils thereby seriously geopardising assessment of their realistic properties. In addition, the road alignment was shifted by 300 to 400m from its original location. If the existing DPR data were to be extrapolated to the new location, such an application would have resulted into fairly uncertain and irreliable predictions.

Table 1 : Geotechnical Parameters for Embankment Design

Soil Type	Avg. Thick.	LL %	PL %	PI %	W_n %	e_0	Specific Gravity	C_u t/m^2	ϕ deg	γ_b t/m^3	P_c t/m^2	C_c	C_r	C_v m^2/year	m_v cm^2/kg	K_v cm/sec	K_h/k_v
Very soft to soft marine clay	10	76	37	39	66	1.17	2.68	0.65	0	1.70	6	0.45	0.045	7.00	0.0896	$3x10^{-7}$	2.5
Stiff to very stiff clay	3	77	35	42	66.7	1.10	2.70	13.0	6	2.0	7	0.43	0.043	5.62	0.0800	0.49 x 10^{-7}	
Weathered Rock	Khondalite, Average N > 60																

To be able to carryout design and construction works with respect to the actual ground conditions, the contractor carried out 6 number boreholes using rotary drilling spread over the 4.142 km stretch of the G.I. Zone.

These boreholes were extended into the rock below the clayey strata with the objective of establishing thickness and depth of the soft and stiff clays and also to obtain information on the soil parameters for checking out the tender provisions with regard to the band drain provision stated above in Para 2(ii). Static cone penetration tests, though originally planned by the contractor, were later deleted on cost considerations.

Based on this preconstruction investigation, the soil parameters (refer Table-1) concerning the analysis and design of embankment in the failure zone were developed from the nearest borehole (BH5) located some 100m away from edge of the zone. It was observed that undrained shear strength of the soft to very soft clays extending to an average depth of 10m was of the order of 6.5 kPa followed by 3.0m thick stiff to very stiff clay which, in turn, was underlain by weathered and hard rock respectively.

4 PVD DESIGN CONSIDERATIONS

The finished height of embankment above the original ground level was 4.1m for the G.I. zone. The estimated settlement, using the traditional one dimensional consolidation analysis, was computed to be 0.87m. Consequently, 5.0m height of embankment was required to be placed above the original ground level which led to an expected total settlement of 1.0m (S_f). Laboratory consolidation tests yielded a weighted average coefficient of vertical consolidation c_v = 7 m^2/yr for the soft clay layers. The time rate settlement analysis indicated a time period in excess of 30 years corresponding to 90% degree of consolidation of the clay subsoils. To achieve the desired degree of consolidation within the project duration, prefabricated vertical Band Drain (PVD) were used based on the following considerations:

4.1 *Diameter of PVD (d_e)*

The equivalent diameter, d_e, of the drain was determined in accordance with Rixners et al. (1986) expression employing the width (105mm) and thickness (3.5 mm) of the proposed drains which yielded, d_e = 54mm.

4.2 *Smear Effect*

The installation of band drain causes substantial remoulding of soil in its vicinity resulting in reduced permeability of the soil in the disturbed region. Hansbo (1979) recommended a simple way of considering this effect in design by using a reduced value of the equivalent diameter. Accordingly, a reduced equivalent band drain diameter, d_r = 50mm was adopted as suggested by Hansbo.

4.3 *Well Resistance*

Whether the proposed band drains would have sufficient well resistance to retard the flow of water through the grooved core channels was examined by considering magnitude of the ratio q_w/k_c as purported by Hansbo (1979). Here q_w represents the discharge potential of the drains and k_c represents the soil permeability. For the Vizag drains, the manufacturer determined a value of $q_w = 40x10^{-6}$ m^3/sec (1260 m^3/yr) corresponding to vertical pressure of 200 kPa and a hydraulic gradient of 0.5.

It has been reported that variations in hydraulic gradient do not significantly influence the discharge potential of the drain, Holtz et al. (1991). His this conclusion was derived from a number of band drains that were tested with variable hydraulic gradient and lateral pressure. Thus, the given discharge capacity can be treated to be independent of the hydraulic gradient. Further, the 200 kPa pressure represented the actual (field) vertical pressure to which the Vizag drains were in reality subjected to due to the combined loading comprising the embankment load as well as the soil overburden. Thus, the given q_w of 1260 m^3/yr was believed to represent in good measure the actual discharge potential of the drains for the Vizag site conditions.

The parameter k_c for the Vizag soils was determined to possess a weighted average value of $6.05x10^{-7}$ cm/sec.

With the known q_w and k_c, a value of 6610 m^2 was computed for the ratio q_w/k_c. On the basis of his studies, Hansbo (1979) made the observation that for a soil treated with band drains, the well resistance cannot usually be ignored when the ratio q_w/k_c is less than 3000 m^2. Comparing this value with that determined above for the Vizag soil (i.e. q_w/k_c = 6610 m^2), it is seen that the well resistance of the Vizag band drains should not be of any concern.

This observation was further confirmed by comparing the actual discharge potential of 1260 m^3/year of the Vizag drains with the discharge capacities considered adequate in practice. Holtz et al. (1991) observed that as long as q_w is greater than 100 to 150 m^3/year, there should be no significant increase in the consolidation time implying thereby that the well resistance of the Vizag drains is not a phenomenon to reckon with.

4.4 *In-Situ Discharge Capacity*

The flow rate for the Vizag band drain was estimated using the following relationship (Lee et al., 2000) assuming that the largest flow in a PVD occurs at the maximum rate of ground settlement at time t = 0:

$$Q_r = 2\pi \cdot S_f \cdot C_h / \mu \qquad (1)$$

where Q_r = the required flow rate under the field conditions; S_f = final settlement under the maximum applied load; C_h = effective coefficient of consolidation with horizontal flow.

Various parameters involved in equation (1) were evaluated as follows:

Average Coefficient C_h : c_h/c_v = 2.5, Avg. C_v = 6.31 m^2/yr considering both the clay layers. This, in turn, yielded C_h = 15.8 m^2/year.

Computed Final Settlement using conventional consolidation theory, $S_f = 1.0m$

$$\mu = \ln\left(D/d_r\right) - 0.75 \tag{2}$$

where D represents the influence diameter of band drain = 2.1m for 2.0m triangular spacing of drains and d_r = 50mm. Using equation (2), $\mu = 2.988$.

Inputting the above known parameters in equation (1), the actual discharge Q_r obtained from a drain in critical early stages of the drainage process is expected to be of the order of 33.2 m^3/year or 1.05 x 10^{-6} m^3/sec. This being significantly less than the potential capacity of the drain as per the manufacturer's specifications (1260 m^3/yr), it was believed that the selected Band Drain (FD 747W) can be expected to perform satisfactorily under the Vizag soil conditions.

5 EMBANKMENT CONSTRUCTION

The band drains were installed through the 730mm thick initial embankment (IE) to project 450mm into the 600mm thick sand blanket (SB) overlying the IE. The combined thickness of 1330mm ensured adequate projection of sand blanket above the original ground surface notwithstanding the large ground settlement. To permit escape of the porewater coming out of the band drains, a layer of compacted crushed gravel (6mm down) was provided on the exposed surface of the blanket along the side slopes, see Figure 3.

The height of 1.33m was greater than the safe height of 1.25m computed based on the bearing capacity consideration. The latter was computed utilizing (i) minimum undrained shear strength of 6.5 kPa (ii) a factor of safety of 1.5 and (iii) bearing capacity factor Nc of 5.7 as recommended in the Indian Roads Congress Guidelines, IRC 75-1979. However, though the height provided was greater than the safe height, no sign of distress was noticed anywhere and the embankment continued to be built. Accordingly, 3.33m height of the embankment above the OGL (original ground level) was built in 170m long stretch (Section A) between km 4.250 to km 4.420 and also in a 65m long stretch (Section B) between km 4.030 to km 4.095. In yet another part representing the super-elevated portion of 420m length (Section C) between km 5.425 to km 5.845 located westward some 1000m away from Section A, the embankment height reached was 2.83m over the OGL.

Reasons that the various portions of the embankment with considerable heights were stable against the computed safe height of 1.25m could be attributed to the following:

(i) Dissipation of pore pressure through the band drains allowing gain in strength to occur through the intervening period (approx. 10 weeks) that elapsed after installation of band drains.
(ii) Significant increase in undrained strength with depth in the Vizag deposits, which actually was observed during fresh phase of geotechnical investigations undertaken subsequent to failure.
(iii) Presence of a stiff to very stiff clay stratum at depth of about 0.5 times the base width of the embankment (34.0m) which contributed significantly in increasing the bearing capacity of the soil.

For constructing the remaining height of embankment in section A, some 30 truck loads of the fill material were deposited on its 170m long stretch in stockpiles of approximately 1.8m height spaced nearly 1.5m c/c placed across the entire width of the road crest. The dumping was completed around the afternoon of April 23, 2003 and during inspection the same evening, a longitudinal crack running precisely along center line of the embankment in Section A was observed, see Figure 1.

It must be noted that no such dumping was done on Section B and Section C.

Figure-1. Distressed Embankment Section at km. 4+260 as on 24.04.2003 (AN), Vizag Port Road Connectivity Project

6 OBSERVATIONS AND ANALYSIS

Figure 2 shows that left side of the embankment suffered damage although the material was stockpiled on either side of the centerline. A week after development of the crack, the left side of the embankment was found to have settled 500mm relative to the right side which suffered an average settlement of around 100mm and the latter became apparent only when the damaged section was opened for repairs several months after the failure incident.

However, there was no cracking observed at any time on the right side. Absence of this cracking appeared to be due to the following reasons:

(i) Presence of haul road well compacted under action of construction traffic in close proximity to the right toe served as berm which possibly prevented a rotational slip.
(ii) In initial stages of construction of the haul road, the fill material sank into the soft marine clay repeatedly which every time was compensated by adding granular fill till the situation stabilized. This process thus may have created a berm of hard crust capable of preventing development of a failure surface.
(iii) Further, there was no water body on right side of the embankment as was seen on the left side about 60m or so away from the left toe.
(iv) Progressive inspections revealed that, with stockpiles in place, further embankment movements ceased around 7 days after initiation of cracks. Subsequent to this, maximum crack width measured was 750mm and maximum depth around 1900 mm.
(v) Horizontal and Vertical measurements made on the pillar (refer Figure 2) indicated its horizontal movement of about 80 cm from its original location and that it also suffered a heave of 20 cm. Also, the outer edge of the drain exhibited a crack of 25 to 50mm width.

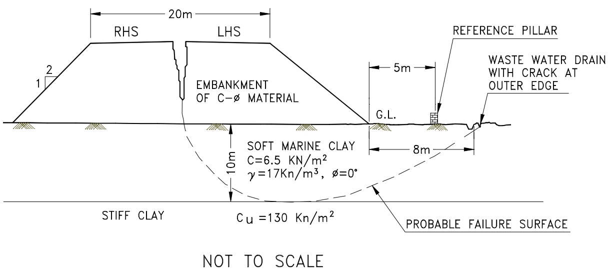

Figure-2. Failure Pattern of Embankment, Vizag Port Road Project

Taking cognizance of the significant strength differential that existed between the soft marine clay and the very stiff clay stratum and also the observed horizontal and vertical displacement of the two structures mentioned above, a plausible configuration of the failure surface drawn tangential to the stiff clay layer in Figure 2 points out to the probable deep seated rotational failure which engulfed the embankment and the soft clay down to the top of the very stiff clay layer to emerge beyond the left toe of the embankment.

7 SUGGESTED REMEDIAL MEASURES AND CONCLUSIONS

A comparison of the cracked section A loaded relatively quickly with other sections B and C where no fill was deposited, conclusively established overstressing of the soft clay stratum by temporary stockpiled fill material as the main cause of failure of embankment. Though severity of ground movement could be suspected to lead to some folding of drains, Miura et al. (1998) observe that folding of drain does not influence discharge capacity significantly as it does not change significantly both the length and the cross sectional area of the drainage path. Effectiveness of the discharge capacity of the existing band drain was accordingly considered to remain uninhibited. However, to achieve early strength gains, which, in turn, will discourage initiation of rotational failure, fresh installation of band drains at centroid of each triangle was recommended. This resulted in a reduced band drain spacing of 1.15m centers invoking a hexagonal pattern in those sections where band drains already existed at 2.0m centers. In virgin sections of the proposed highway, a triangular spacing of 1.15m was adopted.

Staged-construction procedure of embankment building would necessarily be followed based on bearing capacity consideration of soil, taking due account of the strength gain at end of each stage. The design accomplished indicated a two stage construction.

A high strength high modulus woven polyester geotextile with an allowable tensile strength of 180 KN/m was recommended to be interposed at the interface between the embankment and the top of the sand blanket. Placement of the geotextile at such a level was done with a view to avoid formation of the mandrel holes in the geotextile due to installation of band drains through it.

In view of the possibility of water logging taking place during monsoons of the low lying lands on either side of the embankment, provision of a 10m wide and 1.33m thick berm commensurating with top level of the sand blanket, was also made. Due to the low height of embankment at the berms, the band drains under them were spaced at 2.0m c/c without any geotextile.

To be able to undertake designs for all the above substantive measures with respect to the actual subsoil conditions, an elaborate geotechnical investigation through a reputed soil contractor covering full stretch of the G.I. Zone was initiated. Such an investigation incorporated also the field static cone and the vane shear tests. Continuous monitoring through instrumentation was also recommended.

The damaged section was dug down to a level till the triangular pattern of previously installed band drains became clearly visible over the entire failure stretch. This required some 20 cm excavation down below to the top of the sand blanket besides first removing the 2.0m height of embankment built earlier over the sand blanket. The exposed part of drains was lapped wherever so required. The installation of drains at centroid of each triangle was then undertaken. Subsequent to this, the sand blanket was made up to the required grade of RL = 2.85 m and this was followed by spreading the high strength geotextile in strips running transversely to the length of the embankment. Ends of the geotextile were firmly secured before taking up the embankment construction over it. The layered system followed for reconstruction of the embankment is depicted in Figure 3.

Recording of the instrument data is currently under progress. The outcome of the remedial measures is intended to be analysed and presented later when the response data monitored from both the loading stages become available.

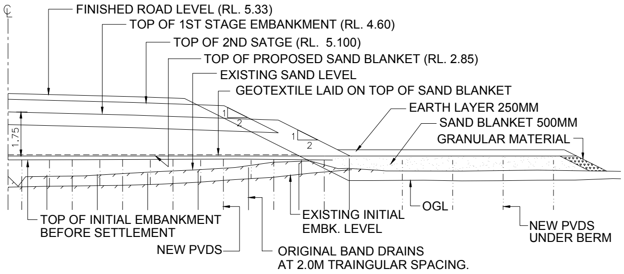

Figure-3. Corrected Embankment Profile with remedial measures after failure – Typical Section at chainage 4+300 km.

ACKNOWLEDGMENTS

The authors express their wholehearted appreciation to M/s National Highways Authority of India for sanctioning implementation of the remedial measures. Cooperation extended by the site staff at various stages of the work reported herein is gratefully acknowledged. Special thanks are due to B.P. Kala for typing the manuscript and P. Das for drafting the figures.

REFERENCES

Hansbo, S. July 1979. Consolidation of Clay by band shaped prefabricated drains. Ground Engineering,

Holtz, R.D; Jamiolkowski, M.B and Pedroni, R. 1991. Prefabricated Vertical Drains: Design and Performance. CIRIA Report.

Lee, S.L., Karunaratne, G.P. & Aziz, M.A. March 2002. Design and Performance of fiberdrain in soil Improvement Projects. 4th International Conference on Ground Improvement Techniques.

Miura, N; Chai, J and Toyota, K. 1998. Investigation on some factors affecting discharge capacity of prefabricated vertical drain. 6th International Conference on Geosynthetics. P.845-848.

Rao, V.V., Parkasarao, B.S and Sarma, A.S.R. December, 2003. Subsurface Investigations for Foundations to Port Connectivity Project in Vishakhapatnam: A Case Study, IGC Roorkee, India, P.621-25.

Rixner, J.J et al. 1986. Prefabricated vertical drains. FHWA RD – 86/168, Washington, D.C., USA.