Failure of a railway embankment on soft soil and its restoration

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ABSTRACT

The design and construction of embankments over soft soil has always been a challenging task for engineers. Soft soils are characterized by low shear strength, high compressibility and low bearing capacity. Due to these factors, construction of railway embankment or Highways on these soils poses many challenges if they are not properly taken care while initial planning stage with proper ground modification/improvement techniques. This paper presents a case history of failure of a railway embankment, its forensic investigation and restoration strategy. Laboratory tests were performed for undisturbed samples, to find out the requisite parameters for design and modeling of embankment. The design procedures for multi-stage construction with and without incorporation of prefabricated vertical drains (PVDs), reported in literature and are followed, for analytical and numerical simulations. The embankment supported on PVDs show a much faster rate of dissipation and pore pressure.

Keywords: Embankment; soft soil; ground improvement, prefabricated vertical drains

1 INTRODUCTION

Embankment constructions are required for highways, railways etc. In many cases, these embankment lengths are in kilometers. Many times the foundation soils on which these embankments are being constructed are found to be of soft consistency and also may be organic in nature. They may undergo higher settlement and may not have sufficient bearing capacity, which may result into larger differential settlements (Hartlen and Wolski 1996). The design and construction of embankments over soft soil has always been a challenging task for engineers. Soft soils are characterized by low shear strength, high compressibility and low bearing capacity. Due to these factors, construction of railway embankment or Highways on these soils poses many challenges, if they are not properly taken care while initial planning stage with proper ground improvement techniques.

2 PROBLEM DESCRIPTION

As part of the ‘New broad gauge (BG) railway line project (about 110 km) railway embankments are being constructed. Typical cross section of the embankment is shown in Fig. 1. The site of interest is in a water logging area. The water level at the site during monsoon rises approximately to 3 – 4 m above the existing ground level (EGL). The soil reports available for the site indicate that the subsoil consists of soft soils up to about 12-13 m below EGL (Fig. 2). The construction of the embankment in this area was started during December 2010 directly on the natural soil without giving any pre-treatment.

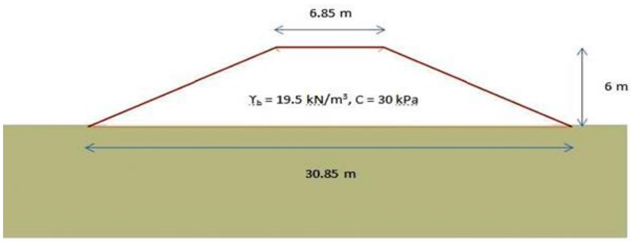

Fig. 1. Schematic diagram of the embankment

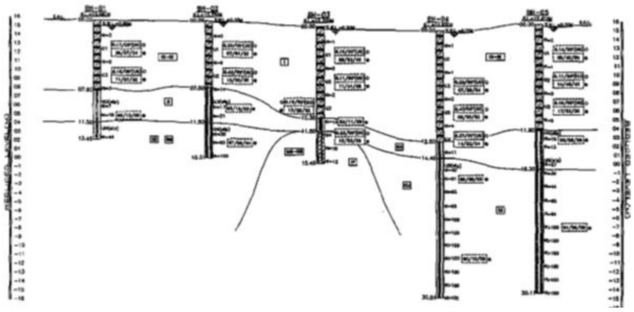

Fig. 2. Longitudinal soil profile at the site of interest

After reaching the embankment height of about 3 m during Sept/Oct 2011 (during monsoon season), first problem in that location was observed manifesting huge settlements in the order of 1- 2 m. The work was continued and reached up to 5-6 m height of embankment (about 90%) during March 2012. Subsequently, at many locations (within one km stretch), huge settlements were observed. Since then, the work related to the embankment was stopped. Huge settlements accompanied by cracks in the constructed embankments and heaving of the adjacent ground are the typical observation at that time (Fig. 3).

Fig. 3. Heaving of the ground adjacent to the embankment.

A total of five boreholes were drilled to gather subsoil information for the site, which were distributed over the area. The boreholes of approximately 150 mm diameter were drilled using water flush aided by chiseling, which were advanced to the depths ranging from 14 m to 24.5 m below ground level. The subsoil includes very soft to firm silty clay or clayey silt and fine grained silty sand with some decomposed materials near the ground surface.

Based on new soil explorations, typical cross sectional soil profile at a location is are shown in Fig. 4. Comparison of old and new reports indicates huge fill soil movement into the ground due to the embankment failure and movements. In general, the top layers of the subsoil at the site are basically composed of very soft to soft silty clay soils with decomposed wood and organic matter. The water table was mostly at the ground surface. The fill soil was placed at the site without any pre-treatment of the soft ground. It is understood that any loading on this site would result in high settlement.

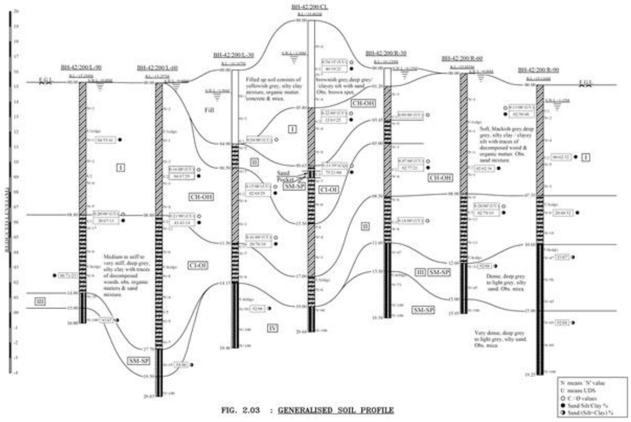

Fig. 4. Typical cross sectional soil profile after failure of the embankment.

3 SOIL PROFILE

Some of the laboratory investigation results mentioned in the soil testing report provided by the testing agency indicated relatively high cohesion values as against the expected shear strength corresponding to the SPT N (standard penetration test number) values. It has been noted that the reported in-situ N-values are significantly low and are in sharp contrast with the estimated shear strength parameters. Hence, it was suggested to conduct static cone penetration test (SCPT) at various locations of the study area. Based on the this, SCPTs were carried out at 16 locations for a total distance of 168 m. The maximum depth of SCPT was 19.2 m from the existing ground level (EGL). Based on SCPT results, average values of shear strength parameters, cohesion and angle of internal friction of soil were reported for four distinct zones below the EGL at all 16 locations of SCPT. A general trend for all the 16 SCPT sites is that the soil strata has sufficient capacity for an average depth greater than 15 m from EGL.

Various laboratory tests (grain size distribution, vane shear test, unconfined compression test, permeability and consolidation) were conducted, on the undisturbed samples obtained during the explorations as per the relevant Indian standard (IS) codes (Phalgun 2015).

Based on various soil investigation reports and results obtained from the undisturbed samples, a typical soil profile was evolved as shown in Fig. 4. Typical properties considered for different layers are shown in Table 1.

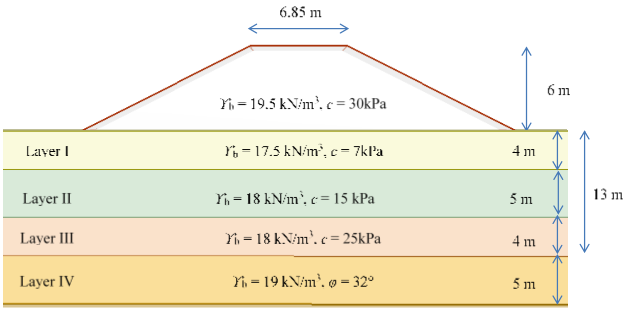

Fig. 4. Soil Profile considered for the analysis.

Table 1. Material properties used for embankment problem analysis.

Stratum	Fill	Layer I	Layer II	Layer III	Layer IV
Bulk Density (kN/m^3)	19.5	17.5	18	18	19
Specific Gravity	2.45	2.2	2.34	-	-
Cohesion (kPa)	30	7	15	25	0
Angle of Internal Friction, φ °	-	-	-	-	32-35
Compression Index, C_c	-	1.079	0.365	0.664	-
Coefficient of Consolidation, C_v (cm^2/sec)		0.000838	0.000576	0.001369	
permeability (cm/sec), k_v	-	8.76x10^{-9}	1.57x10^{-8}	5.05x10^{-8}	-
Initial void ratio, e_o	-	2.719	1.126	1.702	-

4 ANALYSIS OF EMBANKMENT STABILITY

Embankment problem has been analyzed using the soil profile data considered along the embankment stretch. Four different in-site soil types along with fill soil were considered as shown in Table. 1. Analysis was carried using SETTLE3D software (Rocscience 2014). For the construction of 6 m high embankment, in the modeling layered stage construction, was considered. The primary consolidation settlement of 0.505 m, 0.755 m, and 0.951 m after 3.14 m, 4.64 m, and 6 m embankment heights were observed. After full height of embankment constructed, a surcharge of 24 kPa has been applied to consider ballast material, sleepers and rails load (RDSO 2005). Fig. 5 shows the total displacement contours obtained after full height embankment.

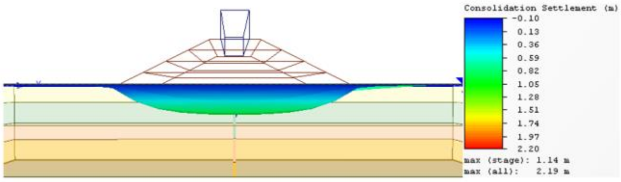

Fig. 5. Total Settlement after full height embankment construction.

From the analysis it is clearly evident that without proper ground treatment, huge localized deformations can lead to embankment failures which are already evident from the field.

5 RESTORATION OF EMBANKMENT

It is evident from the above discussions that there is a strong need for ground improvement scheme to develop the entire problematic area of approximately 1000 m × 200 m. For ground improvement scheme, the following three options could be considered: 1. Prefabricated vertical drains (PVDs) or sand piles with preloading; 2. Stone columns; and 3. PVDs with dynamic replacement columns (hybrid) (Sahu *et al*. 2013). However, owing to the time constraint of the project and unavailability of stone aggregate nearby to the site, PVD (option 1 as mentioned earlier) has been considered as the feasible ground improvement measure by adopting stage-wise construction of embankment to allow sufficient time between each stage.

Wick drains or PVDs are driven into the soil at regular spacings to speed up consolidation. Pore water flows horizontally towards the drains and then water flows freely up the drains to the surface. These speeds up consolidation in two ways: i) horizontal permeability is usually higher than vertical permeability so horizontal flow is faster, ii) the lengths of the flow paths are significantly shortened (Hausmann 1990).

Design of PVDs ground improvement scheme involve: settlement evaluations, fixing the PVDs configuration, designing the stage wise construction scheme and stability evaluations at different stages of the construction. The designer proposed PVDs installation for 13.25 m length below the ground surface at a spacing of 0.8 m center to center in triangular pattern. PVD design calculations which were done according to the equations proposed by Hansbo (1981) are satisfactory for the parameters assumed (k_h/k_v = 1.5 and d_s/d_w = 2). The design procedure of staged embankments with PVDs is followed according to (Sinha et.al., 2008).

As the construction of the embankment will be in stages and height of the first stage of embankment will be evaluated based on in-situ undrained shear strength. Each subsequent height of embankment (subsequent stage) construction is carried out after completion of either 90% primary consolidation of the previous stage loading. Due to this consolidation the strength of the in-situ soil will be enhanced which will be considered for the subsequent stage loading. Thus, the height of embankment of the subsequent stages depends upon the gain in undrained strength of subsoil. Based on the time required to gain strength with the construction of the 0.2 m layer, the waiting period is determined for each subsequent layers. The waiting period depends on soil parameters such as subsoil thickness, C_h/C_v ratio and different PVD factors like smear, drain spacing and well resistance, pattern of laying of PVD, etc. (Sinha et.al., 2008). Table 2 shows the properties of PVDs considered for calculations and numerical analysis. Figure 6 shows the PVD ground improvement scheme implemented at site including a 150 mm course sand levelling course and a non-woven geotextile layer below the embankment.

Table 2. Prefabricated vertical drain (PVD) properties.

General Properties	
Cross-section Type	Strip
Width of the drain, B	0.1 m
Thickness of the drain, t	0.004 m
Spacing, D	1.0 m
Coefficient of Vertical Consolidation, C_v	0.0061 m^2/day
Drainage Pattern	Triangular
Drain Length	13
Smear Zone	
Ratio of smear zone to diameter of drain, S	2
Ratio of horizontal permeability in undisturbed zone to permeability in smear zone, kh/ks	3

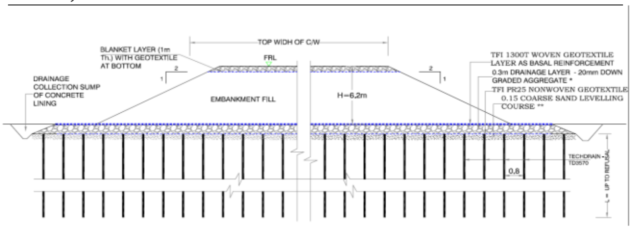

Fig. 6. PVD ground improvement scheme implemented at site.

6 CONCLUDING REMARKS

This paper presented a case study on failure of a railway embankment constructed on soft soil, without paying attention to its general behavior. This resulted in significant delay in the project construction time and also involved huge additional expenditure. This forensic study highlights the importance of proper site investigation and soil properties determination to evaluate the in-situ soil behavior due to the loading conditions. With the properly evaluated soil parameter the numerical model could provide the estimation of anticipated settlements which could be utilized for proper planning of the project. The study also provides a good example of effective performance of prefabricated vertical drains with staged construction scheme as an effective strategic technique to restore the distressed/failed embankment.

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