Investigation and design of a fly ash road embankment in India by CPT

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ABSTRACT: A sub-soil investigation was carried out for the proposed construction of a 4 km pond ash road embankment from Kalindi Colony to Kalindi Kunj in New Delhi, India. The proposed alignment runs along the bank of the river Yamuna and there is a barrage towards the downstream side slope of the river. The flood back water along with sewage water gets accumulated and remains in water logged condition almost for 6 months in a year along the proposed site. This has created soft condition of the sub soil. In-order to evaluate the sub soil condition, field tests were carried out by Standard Penetration Test (SPT) and Cone Penetration Test (CPT). Soil samples were collected and their geotechnical characteristics were investigated. This paper presents the details of the sub-soil strata in the proposed alignment, analysis and interpretation of the field and laboratory tests, settlement analysis and stability analysis of the pond fly ash embankment. Based on the laboratory and field results, the design of pond ash embankment with and without berm was also carried out in sudden draw down and steady seepage condition with seismic factor.

1 INTRODUCTION

The Public Works Department, State Government of Delhi has taken up construction of the Kalindi bye pass from Kalindi colony, Ring road to Kalindi Kunj, Delhi, India. This road alignment takes off from the ring road near Kilokri and passes through Khizrabad, Okhla and Jasola villages and would be linked to National Highway (NH)- 2. This alternative road is expected to decongest the NH-2. The total length of the road alignment is about 3.6 km. During monsoon season, the flood waters from the Yamuna river and sewage from the adjoining residential colonies form ponds in the proposed road alignment. The majority of these ponds dry up during summer. After the detailed survey and marking the lay out at the site, it was found that certain stretches of the proposed road were having soft slushy soil of varying thickness. Hence, detailed sub soil investigations were taken up and obtained the data on sub-surface conditions at the site so as to evaluate the soil parameters, which would help to determine the design and stability of the proposed road embankment. This paper discusses the results of the sub-soil investigation, settlement analysis, and design and stability analysis of the designed fly ash embankment. Construction of the fly ash embankment was in progress in 2009, as shown in the Fig. 1

Fig. 1 Typical cross section of fly ash embankment construction is in progress

2. SUBSOIL INVESTIGATION

2.1 Standard penetration test (SPT)

For carrying out subsoil investigations, bore holes were drilled at different locations along the road alignment up to hard strata. The sub soil investigations were carried out in accordance with the guidelines given in BIS 1892. After studying the available information and conducting field reconnaissance of the existing site conditions, it was decided to drive six bore holes in the 3.6 km stretch which was under investigation. The depth of bore holes was varied from 9 m to 15 m. As the road alignment passes through the water logged area, pond fly ash filling was carried out in such areas to create a working platform for the sub soil investigation work. This working platform was later incorporated as a part of the road embankment. Initially bore holes of 150 mm diameter were drilled using hand auger and were then advanced using power driven hydraulic drilling machine. Seamless flush jointed casing of 150 mm internal diameter was used to prevent collapsing of the borehole. Representative samples of subsoil were collected at regular intervals. Standard penetration tests (SPT) were conducted at various depths inside the bore holes as per BIS 2131. The number of blows required for the 30 cm penetration of the split spoon sampler was recorded as SPT 'N' values. The SPT N values were corrected as per the standard procedure. The presence of soft slushy subsoil and very low SPT values indicated that sandy subsoil is in a loose state and may experience considerable settlement due to construction of road embankment. This led the designers to consider adopting suitable ground improvement techniques. However adoption of such technique would have increased the project cost considerably and created problems during implementation due to densely populated areas adjoining the stretch. Pond fly ash, being a light weight material, was an obvious choice for embankment construction in this project. However it was also decided to undertake further studies on sub soil using Cone Penetration Test (CPT) equipment to realistically assess the in-situ sub soil strength. Typical subsoil investigation results by SPT (N values corrected) are presented in Table 1.

Table 1 Typical SPT values along the proposed embankment

<table>
<tr><th>Depth (m)</th><th>Type of Test</th><th>BIS Soil Classification</th><th>Plasticity properties</th><th>Gravel (%)</th><th>Sand (%)</th><th>Silt (%)</th><th>Clay (%)</th><th>SPT ‘N’ Value</th></tr>
<tr><td>3.0</td><td>SPT</td><td rowspan="3">Loose greyish sandy silt (SP)</td><td>Non Plastic</td><td>8</td><td>81</td><td>11</td><td>0</td><td>2</td></tr>
<tr><td>5.0</td><td>SPT</td><td></td><td></td><td></td><td></td><td></td><td>3</td></tr>
<tr><td>7.0</td><td>SPT</td><td></td><td></td><td></td><td></td><td></td><td>5</td></tr>
<tr><td>8.5</td><td>SPT</td><td rowspan="3">Loose to medium dense fine sand (SP)</td><td>Non Plastic</td><td>0</td><td>79</td><td>21</td><td>0</td><td>4</td></tr>
<tr><td>9.5</td><td>SPT</td><td></td><td></td><td></td><td></td><td></td><td>9</td></tr>
<tr><td>13.0</td><td>SPT</td><td>Non Plastic</td><td>0</td><td>92</td><td>8</td><td>0</td><td>17</td></tr>
<tr><td colspan="9">Bore hole terminated</td></tr>
</table>

2.1.1 Discussion of SPT result

Starting from chainage 400 m (near Kalindi) to 2800 m, the subsoil property is not much varying. At the proposed road location, no rock outcrop is seen. There exits about 1 to 2 m thick loose to medium dense fill material at the top. Below this layer, very soft loose grayish silty sand (slushy soil) exists for a thickness of about 1.5 to 3 m. Beneath this layer, loose to medium dense grayish sandy soil with mica particles is found up to a depth of about 9 to 10 m. The subsoil below this depth is quite dense and has concretions of nodules of varying sizes made up of impure calcium carbonate (kankar). SPT N values below 10 m were observed to be more than 20. The water table was found at a depth of 1m below the existing ground level from chainage 0 m to 800 m and almost at the ground level from chainage 800 m to 3600 m. The SPT N values for very soft slushy soil were very low (less than 5). The natural moisture content in slushy subsoil was found to be in the range of 60 to 70 %. There was no organic matter present in the subsoil. Clay content in the subsoil was also found to be negligible. The subsoil up to 10 m depth is predominantly fine grained sand.

2.2 Cone Penetration Test (CPT)

Cone Penetration Test was carried out at different locations along the proposed embankment as per BIS 4968 (Part 3). Cone penetration resistance is measured by mechanical cone penetration test equipment. Cone resistance versus depth is plotted in Fig. 2.

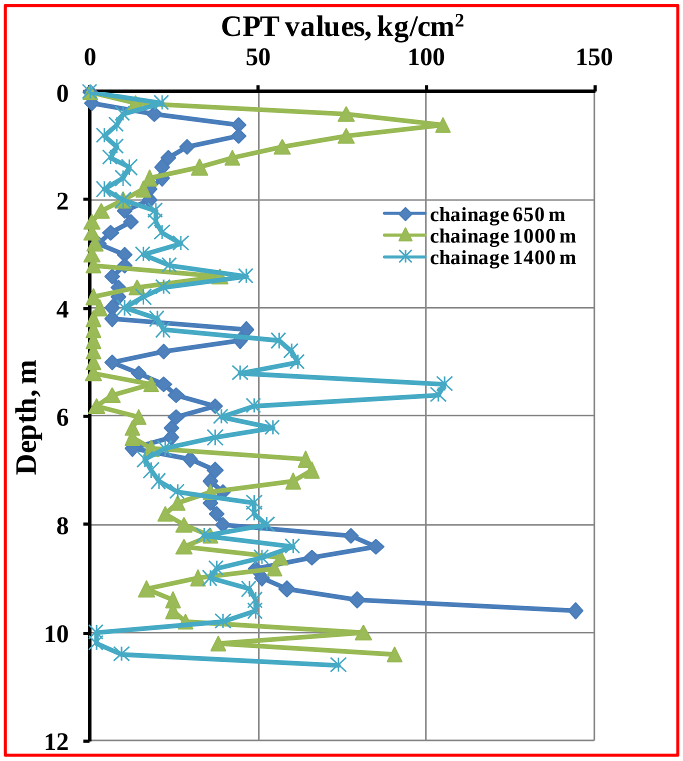

Fig. 2 Depth versus Cone resistance (CPT Test) (Note: 1 kg/cm^2 = 98 kPa)

2.2.1 Discussion of CPT result

From the Fig, 2, it was observed that the values of cone resistance from CPT is around 1 kg/cm^2 up to 6 m depth, 20 kg/cm^2 from 6 to 8m depth and more than 20 kg/cm^2 beyond 10 m depth**.**

3. SETTLEMENT CALCULATION FOR THE EMBANKMENT

As already pointed out in the previous section, the sub-soil in the proposed alignment consists of sandy soil up to a depth of about 15 m. For granular soils (which include sand) the allowable pressure which may be applied on the sub-soil is governed by considerations of settlement, rather than that of the shear strength of soil. For this reason, accurate prediction of the settlement of structures founded on granular soils is of considerable practical importance. Keeping this view, the 'Schmertmann (1978) method' based on cone penetration test (CPT) was adopted for settlement calculations.

3.1 Settlement calculation using Schmertmann & Hartman (1978) method

The Schmertmann and Hartman (1978) method utilising an empirical relationship between sub-surface investigation data and soil properties to approximate the pattern for vertical strains in the stressed soil zone, offers a procedure to calculate settlement resulting from the combined effect of volume distortion and compression in sand deposits. The Schmertmann and Hartman (1978) gives the following equation for calculating settlement (δ):

$$\delta = C_1 C_2 \Delta p \sum \left(\frac{I_z}{E} \right) \Delta z \qquad (1)$$

Where
Δp = increase in effective overburden pressure at ground level
Δz = thickness of layer under consideration
C_1 = depth embedment factor
C_2 = Correction factor for creep
I_z = Strain influence factor
E = Young's Modulus

Obtaining exact value of E through testing soil samples is very difficult and hence correlations have been developed to relate this soil modulus to CPT resistance, q_c. The approximate values of E for different soil types are given below Table 2:

Table 2. Young's modulus (E) of soil

Soil type	Approximate value of E in terms of q_c
Sand silt mixture	$1.5q_c$
Fine to medium coarse sands	$2q_c$ to $3q_c$ (depending on density and compactness)
Sand-gravel mixtures	$4q_c$

Correction factor C_1 is applied to compensate for the effect of foundation depth (or embedment), where

$$C_1 = 1 - 0.5\left(\frac{\sigma_0'}{q_n} \right) \qquad (2)$$

Where σ_0' = effective overburden pressure at foundation level and q_n = net foundation pressure

The correction factor C_2 for creep is given by

$$C_2 = 1 + 0.2\log_{10}\left(\frac{t}{0.1} \right) \qquad (3)$$

where t is the time in years at which the settlement is calculated.

3.2 Typical settlement calculations at chainage 1600 m

Calculation of overburden pressure (Δp) at ground level due to embankment

Considering the height of the embankment, bulk density of the fill materials and assuming the water table assumed to be at ground level, the total overburden pressure (Δp) due to embankment and pavement would be about 9.3 t/m^2 (91 kPa).

Calculation of depth embedment factor C_1

Correction factor C_1 is applied to compensate for the effect of foundation depth (or embedment). In the present case, the embankment would be located above ground level and there will not be any removal of earth. Hence C_1 = 1.0

Calculation of correction factor for creep C_2

The correction factor C_2 for creep is given by $C_2 = 1 + 0.2\log_{10}\left(\frac{t}{0.1} \right)$ where t is the time in years at which the settlement is calculated. Assuming the time period for completing the construction to be 1 year, C_2= 1.20

Calculation of strain influence factor I_z

I_z = Maximum strain influence factor = $0.5 + 0.1\sqrt{\frac{\Delta p}{\sigma_z}}$

Δp = Net overburden pressure = 9.3 t/m^2 (91 kPa)

σ_z = Vertical pressure at depth B/2 below embankment (prior to construction of embankment)

Substituting these values, σ_z = 45 t/m^2, so I_Z = 0.546

The maximum value of I_z would be adopted for settlement calculation.

Calculation of E

The subsoil is sandy soil. So E = 2q_c. Usually the CPT data is divided into several layers representing similar sub-soil characteristics. At chainage 1+600, the CPT results are more or less similar and there is not much difference in q_c values with depth. Hence from the CPT data, the average value of q_c at chainage 1+600 = 16 kg/cm^2 = 160 t/m^2

So E = 2 x 160 = 320 t/m^2

Calculation of Δz

From the field data it is observed at chainage 1+600, the height of fly ash embankment is about 3.2 m and depth of sandy sub-soil strata is about 7.6 m. Below the sandy sub-soil hard stratum is encountered. This hard stratum is not expected to undergo any settlement and settlements would be confined to the sandy soil strata of 7.6 m thickness. Hence Δz = 7.6 m

Substituting the values of Δp, Δz, C_1, C_2, I_z, and E in equation (1),

δ = 0.1447 m $\Rightarrow$ About 14.5 cm

In this manner settlement was computed at different chainages using CPT data. The computed settlement values generally vary from about 7 to 14.5 cm. The maximum settlement would be occurring from chainage 1600 m to 2000 m and it would be about 14.5 cm. However because of the high permeability of sandy soils, most of the settlement will occur during the course of embankment construction itself. After the end of construction, therefore only minor settlements due to creep are likely to occur.

4. DESIGN AND STABILITY ANALYSIS OF THE FLY ASH EMBANKMENT

Considering the results obtained from the field and laboratory investigation of sub soil samples and embankment fly ash fill materials, design of high embankment (5m) was carried out. Fly ash embankment was designed with soil cover of 2m thickness and with or without a berm of 3m width. The side cover soil has a plasticity index (PI) in the range of 5 – 9 %. The side slopes of the embankment were kept 1(V):2(H). The river side of the embankment slope is protected by providing stone pitching.

It is observed that the fly ash embankment is subjected to both sudden draw down and steady seepage conditions under the highest flood level with seismic effect. Sudden draw down conditions develop when flood waters recede at a very fast pace as may happen during opening of the barrage gates during flooding.

Stability analysis was carried out using computer software. It is observed that Factor of Safety is critical (FoS = 1) under sudden draw down with side cover of thickness 2m. To increase the Factor of Safety (FoS = 1.29) an additional 3m berm was provided on either side of the embankment. The results of stability analysis are tabulated in Table 3.

Table – 3 Results of the stability analysis of 5m high embankment

Embankment side slope conditions considered for analysis	Bishop's Method	Petterson's Method
Sudden draw down condition – 2 m side soil cover	1.00	1.01
Sudden draw down condition – Berm of 3 m thickness + 2 m side soil cover	1.29	1.28
Sudden draw down condition – Berm of 3 m thickness + 2 m side soil cover	1.57	1.55

5. CONCLUSIONS

A subsoil investigation was carried out for the proposed construction of 5m high embankment in order to evaluate the sub soil condition. Stability analysis was also carried out to investigate the stability of designed embankment.

- The sub soil stratum in the proposed alignment of Kalindi Bye pass predominantly consists of 'Poorly graded fine sand (SP)'. Further at shallow depths (up to about 3 m), this material was observed to be in a loose state especially in between the chainage 800 m to 2200 m. However this layer is devoid of clayey soil.
- Sand layer between 3 m to 15 m depth was found to be loose to medium dense. Sub-soil below 15 m depth was found to be in a densely compacted state having 'N' values in excess of 20. The results of CPT tests also confirm this finding.
- Berm of width equal to 3 m and height 5 m may be provided by the river side of the embankment from chainage 2150 m to 3000 m. This is essential to prevent any failure of embankment side slope during to sudden draw down conditions.
- The settlement of the embankment in the total reach from chainage 00 to 3600 m was computed to be between 7 to 14.5 cm. The maximum settlement of 14.5 cm is expected to occur in the reach between 1600 to 2000 m. The total construction period of the project is expected to be about two years. As the sub-soil undergoing settlement is mainly sandy type, the settlement would occur during the construction phase itself.

6. ACKNOWLEDGEMENTS

Authors express their deep sense of gratitude to Public Works Department, Government of Delhi for sponsoring the study. Authors like to acknowledge the help of GTE divisional staff members as Sh. Kanwar Singh, P.S. Prasad, Sh. S.C. Saha and other laboratory assistants during field and laboratory investigations. This paper has been published with kind permission of Director, CRRI New Delhi.

7. REFERENCES

1. CRRI Report. 2006. Geotechnical investigation and design of embankment using fly ash for Kalindi bye pass.

2. Schmertmann, J.H. and Hartman, J.P. 1978. Improved strain influence factor diagram, *Journal of geotechnical engineering division*, ASCE, Vol. 104 (GT8).
3. Sharma,V. 1885. Geotechnical investigation for new Yamuna Bridge connecting Okhla with NOIDA, *Journal of Indian Roads Congress*, Vol. 46 part II, pp 452-489.
4. BIS code 1892, 1979. Code of practice for subsurface investigation for foundations, published by *Bureau of Indian Standard*.
5. BIS code 2131, 1981. Method for Standard Penetration Test for soils, published by *Bureau of Indian Standard*.
6. BIS code 4968 Part 3, 1976. Method for subsurface sounding for soils, Static Cone Penetration Test, published by *Bureau of Indian Standard*.