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An investigation into embankment failure along a section of a major highway

Une recherche sur l'échec de remblai le long d'une section d'une route importante

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ABSTRACT
The construction of Bhanga-Bhatiapara section of Dhaka-Khulna road under the Southwest Road Network Development Project started in January, 2002. While carrying out the filling of the embankment near approach of a bridge, a slip failure in the embankment occurred. The available bore log reports of four boreholes drilled in July 2001 near the failure location revealed the existence of very soft silty clay layers of thickness varying between 5 m and 8 m. Embankment failure was initiated when the fill height was gradually raised to about 6 m. For a fill height of approximately 6 m, slope stability analyses of the failed embankment section were conducted using Simplified Bishop Method and the method proposed by Low. This paper presents an analysis of general soil condition at the site, possible reasons that lead to failure of embankment and necessary remediation that may be undertaken for safe construction of the approach embankment.

RÉSUMÉ
La construction de la section de Bhanga-Bhatiapara de la route de Dhaka-khulna sous le projet de développement de voirie de sud-ouest a commencé en janvier, 2002. Tout en effectuant le remplissage de l'approche proche de remblai d'un pont, un échec de glissade dans le remblai s'est produit. Les rapports disponibles de notation d'alésage de quatre forages forés dedans juillet 2001 près de l'endroit d'échec ont indiqué l'existence des couches silty très douces d'argile d'épaisseur changeant l'échec de remblai entre 5 m et 8 m. a été lancé quand la taille de suffisance a été graduellement augmentée à environ 6 m. Pour une taille de suffisance d'approximativement 6 m, des analyses de stabilité de pente de la section échouée de remblai ont été conduites en utilisant l'évêque simplifié Method et la méthode proposée par Low. Cet article présente une analyse d'état général de sol à l'emplacement, les raisons possibles qui mènent à l'échec du remblai et de la remédiation nécessaire qui peut être entrepris pour la construction sûre du remblai d'approche.

1 INTRODUCTION

The construction of Bhanga-Bhatiapara section of Dhaka-Khulna road under Southwest Road Network Development Project (SRNDP) Contract No. 3 has been awarded to a Consortium of local and foreign contractors. While carrying out the filling of the embankment near approach of a bridge at chainage 5+540 km, a slip failure in the embankment occurred. The contractor for SRNDP requested Department of Civil Engineering, BUET to provide expert opinions about the causes of failure of embankment at a bridge approach at chainage 5+540 km along Bhanga-Bhatiapara section of Dhaka-Khulna road and suggest appropriate remedial measures for safe construction. A BUET team of Consultants looked into the matter. The Consultants visited the site on July, 2002 to assess the overall condition of the site where embankment failure took place.

This paper presents an analysis of general soil condition at the site, possible reasons that lead to failure of embankment and necessary remediation that may be undertaken for safe construction of the approach embankment.

2 GENERAL SOIL CONDITIONS AT THE PROJECT SITE

The available bore log reports (September 2001) of the four boreholes drilled at chainage 5+525 km of Bhanga-Bhatiapara section have been reviewed in order to identify the approximate depth of soft clay layers at this location of the embankment site. The depth of these four boreholes varied from 38 m to 45 m.

Thickness of soft clay layer at chainage 5+525 km varies between 5 m and 8 m. For the soft clayey layers in this section, liquid limit, plastic limit and plasticity index vary from 33 to 68, 21 to 31 and 9 to 37, respectively. Percent clay in cohesive layers and percent sand in the non-plastic sandy layers have been found to vary from 11% to 20% and 62% to 90%, respectively.

Recently, during July, 2002 five holes were drilled at chainage 5+540 km along Bhanga-Bhatiapara section after failure of the embankment took place. In three holes (Boreholes VS-01, VS-02 and VS-03) in situ vane shear tests were carried out in clayey and silty soils up to depth of 10 m below the existing ground level. Field vane shear tests were performed at an interval of 1 m depth. In these holes, below 10 m depth, Standard Penetration Tests (SPT) were also conducted in sandy soils up to a depth of 15 m. SPTs were also carried out at an interval of 1m depth. SPTs have been performed using auto-trip hammer. Undisturbed samples using 100 mm diameter open-drive tube sampler were also collected from clay and silt strata from boreholes VS-01, VS-02 and VS-03. In the other two boreholes (Boreholes SPT-01 and SPT-02), SPT was carried out up to depth of 15 m at an interval of 1 m depth. Three tube samples were also collected from these two boreholes. It can be seen from these boreholes that at chainage 5+540 km the thickness of fill and soft clay layer vary from 4.8 m to 6.2 m and 2.9 m to 4.8 m, respectively.

2.1 *Field Vane Shear Strength and N-values from Standard Penetration Test (SPT)*

The logs of the holes VS-01, VS-02 and VS-03, show in situ vane shear strength in the undisturbed and remoulded state of the fill and soft sub-soil along with the description of soil and depth of strata encountered. The logs also show the N-values obtained from SPT carried in the sandy soils encountered below the soft sub-soil consisting of grey silty clay. It can be summarized that undisturbed field vane shear strength of the soft sub-soil consisting of grey silty clay ranged between 22 kPa and 42 kPa. It should be mentioned that these strengths should be corrected by a suitable correction factor such as suggested by Bjerrum (1972) and Aas et al. (1986).

On the basis of the values of undrained shear strength the samples obtained from boreholes VS-01, VS-02, VS-03, SPT-01 and SPT-02, the fill materials are, in general, firm and firm to stiff in consistency while the underlying sub-soils are, in general, soft and soft to firm in consistency.

N-values obtained from SPT in the sandy layers (below the soft sub-soil) up to depth of 15 m in the boreholes VS-01, VS-02 and VS-03 varied between 11 and 16. N-values for the fill material obtained from boreholes SPT-01 and SPT-02 varied widely between 2 and 11. For the soft clay below the fill, in borehole SPT-01, N-values obtained from SPT varied between 0 and 2. However, N-values in borehole SPT-02 were found to vary between 2 and 4. N-values obtained from SPT in the sandy layers (below the soft clay) of boreholes SPT-01 and SPT-02 ranged between 9 and 20 up to depth of 15 m.

2.2 *Laboratory Vane Shear Strength of Undisturbed Soil Samples*

At BUET Geotechnical Engineering Laboratory, the Pilcon Hand Vane Tester has been used to determine shear strength of the fill material and the clay sub-soil from undisturbed clay samples. Tests were carried out on 4 inch diameter tubes in which undisturbed soil samples were encased. Depending on the consistency of the samples, either 19 mm diameter vane or 33 mm diameter vane was used. The laboratory vane shear strength of samples of the soft clay sub-soil range between 13 kPa and 19 kPa. Based on the values of undrained shear strength, the clay sub-soil can be considered as very soft in consistency. Water contents of the soft clay samples have been found to vary between 30.5 % and 46.8 %.

2.3 *Index, Compressibility and Permeability Properties*

Index property tests for the identification and classification of sub-soil, including moisture content, liquid limit and plastic limit test, and particle size analysis test were carried out at the Geotechnical Engineering Laboratory of BUET. Besides one-dimensional consolidation tests were also carried out to evaluate compressibility and permeability properties of the soft clay sub-soil.

Liquid limit, plastic limit and plasticity index of a soft silty clay sample (sample UD-08, depth: 7.30 m to 7.75 m) obtained from borehole VS-03 have been found to be 49, 25 and 24, respectively. From the particle size distribution curve, percent clay (< 0.002 mm) is 26, percent silt (0.002 mm to 0.06 mm) is 66 and percent sand (0.06 mm to 2 mm) is 8, which were determined using MIT Texural Classification System. Using the results of index property tests, soil sample obtained from boreholeVS-03 has been classified according to Unified Soil Classification System (USCS) as outlined in ASTM D2487 (ASTM, 1989). The cohesive sample is typically clay of low to medium plasticity (USCS Symbol is CL).

Four one-dimensional consolidation tests were carried out at BUET Geotechnical Engineering Laboratory on two undisturbed tube samples (sample UD-07 of borehole VS-02 and sample UD-08 of borehole UD-08) obtained from Ch. 5+540 km at depths of about 7 m to 8 m. From each sample two specimens, one sliced horizontal (vertical sample) and the other sliced vertical (horizontal sample), were prepared to perform consolidation tests. Compressibility and permeability properties of four samples were determined from incremental loading by one-dimensional consolidation tests.

Time-deformation curves have been plotted for each pressure increment and from these plots times corresponding to 90% consolidation, i.e., t_{90} were determined using Taylor's Curve Fitting Method (Das, 1993). Coefficients of consolidation for vertical and horizontal flow (c_v and c_h), and coefficient of permeability in vertical and horizontal direction (k_v and k_h) were calculated for each stress increment.

The values of C_c of the two vertical samples are 0.38 and 0.46 while the values of C_c of the two horizontal samples are 0.57 and 0.39. The initial void ratio (e_0) of the four samples was found to vary between 1.18 and 1.70. Depending on the stress range, the values of coefficient of consolidation for vertical flow (c_v) and coefficient of vertical permeability (k_v) of the two soil samples have been found to be in the range of 2.49 to 13.34 m^2/year and 1.62 x 10^{-10} to 7.02 x 10^{-9} m/sec, respectively. Depending on the stress range, the values of coefficient of consolidation for horizontal flow (c_h) and coefficient of horizontal permeability (k_h) of the two soil samples have been found vary from 4.01 to 30.87 m^2/year and 2.36 x 10^{-10} to 2.59 x 10^{-8} m/sec, respectively.

More detail description of laboratory tests can be found in BRTC (2002).

3 STABILITY ANALYSES OF THE EMBANKMENT ALONG BHANGA-BHATIAPARA SECTION AT FAILURE LOCATION

Assessment of probable causes of failure of earth embankments usually require an assessment of geological and geotechnical soil conditions at the site and application of limit equilibrium analyses in the form of slope stability analysis. This is a conventional soil mechanics stability problem. Pre-existing slip planes within the soil, or lenses and bends of cracker material can have a significant effect on slope stability. Stability analyses were carried out to evaluate the factor of safety against sliding and bearing capacity failure of the embankment. The stability of these surfaces was evaluated for short term undrained condition. Short term undrained behaviour is expected until excess pore pressures have dissipated. The program used for stability analysis was PCSTABL5M. In the present case only Bishop Method has been followed because of its effectiveness and suitability for the type of data that were available for analysis.

Stability analyses of embankment have also been carried out using the method proposed by Low (1989). Embankments constructed on soft clay foundations typically have potential failure mode in the form of an approximately circular slip surface extending into the soft foundation. An infinite number of slip circles all tangential to a given trial limiting tangent can be drawn. Among all these possible slip circles passing through the soft foundation, the critical circle has the lowest factor of safety.

The method proposed by Low (1989) is simple and convenient semi-analytical procedure to calculate the factor of safety of embankments constructed on soft clay. Stability numbers N_1 and N_2 are developed for the normalized foundation strength and normalized embankment strength, respectively. The factor of safety (in terms of overall moment equilibrium) has been computed as the sum of two components, each equal to the product of the respective stability number and the corresponding normalized strength. This method assumes short-term undrained response of the soft clay foundation. It can deal with cases where the undrained shear strength of the soft clay varies with depth.

The embankment shown in **Fig. 1** is characterized by its height (H), the slope angle (β), the cohesion (C_m) the angle of internal friction (ϕ_m) and the unit weight (γ). The subscript m stands for embankment. The foundation material is characterized by its undrained shear strength (C_u). The corresponding angle of internal friction ϕ_u has been assumed to be zero. The undrained shear strength (C_u) of the foundation can vary with depth. The "trial limiting tangent" shown in **Fig. 1** denotes a horizontal line at the depth D below the top of the clay foundation to which potential slip circles are tangential. In all the stability analyses, the values of ΔC_T and D_c have been taken as zero. The equation for factor of safety in **Fig. 1** has been derived using a number of approximations (Low, 1989). It has been assumed in all the stability analyses that the computed factor of safety at any depth within the soft soil layer should be at least 1.2 to ensure adequate safety of the whole soft soil layer against bearing capacity failure. Chowdhury (1978) reports the following acceptable values of factor of safety (F.S) for all situations: For 100 % probability of failure, F.S = 0.8; for 50 % probability of failure, F.S = 1.0; for 10 % probability of failure, F.S = 1.2. Thus in the present analyses any slope with factor of safety less than 1.2 has been considered unsafe with potential for failure.

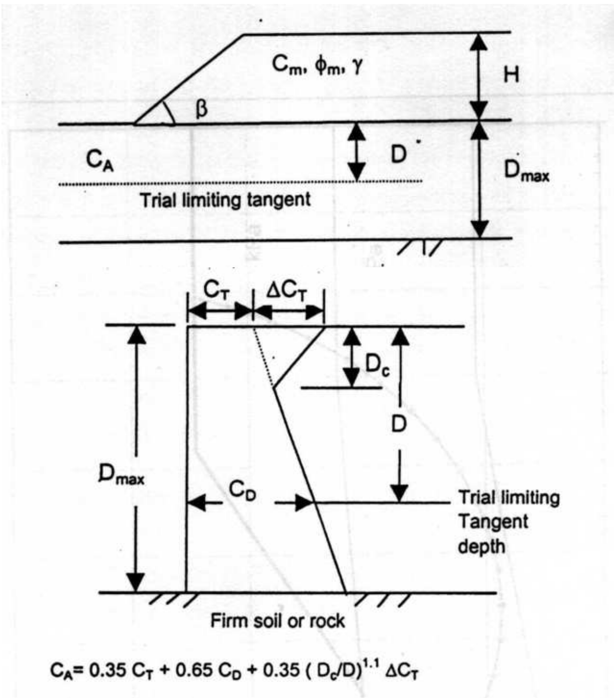

Figure 1 Notations for calculating FS for embankment on weak foundation (after Low, 1989)

4 PROBABLE CAUSES FOR EMBANKMENT FAILURE

It is understood that the embankment failure was initiated when the fill height was gradually raised to about 6 m. Based on the geotechnical data of the previous investigations and with a fill height of approximately 6 m, slope stability analyses of the failed embankment section with side slopes 1:2 (vertical : horizontal) were performed. In the analysis the strength of fill and clay sub-soil were taken as 30 kPa and 10 kPa, respectively. The factor of safety obtained by Modified Bishop Method is 0.674.

Although soil strength values are idealized but this indicates that the soft sub-soil did not have the capacity to support the overlying fill soil. As a result the slope failed by sliding. Another slope stability analysis for the same section of the embankment was carried out using the method proposed by Low (1989). It deserves mentioning that in this method the soft soil layer is considered to be uniform and extends up to a great depth. The following geotechnical properties are assumed to represent the embankment section within the Bhanga-Bhatiapara section.

<u>Properties of Embankment Fill</u>
Height of embankment = 6 m
Undrained shear strength of fill, $C_m = 30\ kN/m^2$
Undrained angle of internal friction, $\phi_m = 0°$
Unit weight of fill, $\gamma_m = 18\ kN/m^3$
<u>Properties of Soft Soil Layer</u>
Maximum thickness = 7 m
Undrained shear strength , $C_D = 10\ kN/m^2$
Undrained angle of internal friction, $\phi_D = 0°$

It has been found that the value of factor of safety is 0.79 for a soft clay layer thickness of 5 m and the factor of safety reduces to 0.72 for a clay layer thickness of 7 m. This clearly indicates that the strength of the clay layer was not adequate to support the overlying fill soil even with a slope of 1:2 (vertical : horizontal). As such failure of the embankment slope took place.

In order to investigate the effect of fill and sub-soil shear strength on stability of the same embankment section another two analyses were carried out. In one analysis the strength of fill and clay sub-soil were taken as 55 kPa and 15 kPa, respectively while in the other analysis the strength of fill and clay sub-soil were taken as 55 kPa and 20 kPa, respectively. In the former case the factor of safety obtained is 1.05 and for the later it is found to be 1.25. This suggests that if the clay layer had an undrained shear strength of greater than 20 kPa, failure of the embankment section would not have occurred when the filling height was limited to 6 m only. From the above stability analyses it is evident that failure of the embankment can be attributed to the existence of a very soft sub-soil layer over which the fill was placed. The underlying soft clay is incapable of supporting the full embankment height (9 m) without improving the ground conditions.

5 CONDITION OF EMBANKMENT SECTION AFTER FAILURE

In order to assess the ground conditions after the slope failure field vane shear tests and SPTs were carried out at five locations. The test locations and test results are discussed in detail in Articles 2.1 to 2.3 of this paper. Based on above geotechnical investigations and survey data provided by the Contractor, pier locations, ground profile and profile of soil layers along centreline of the bridge and embankment are shown in **Fig. 2**. This figure indicates that a portion of the soft clay layer on the Bhatiapara side is overlain by fill and this portion of clay layer will undergo consolidation and thereby strength of this layer will increase with time. While the soft clay layer near the middle pier and toe of the embankment is unlikely to be consolidated since there is very little surcharge over this portion of clay layer.

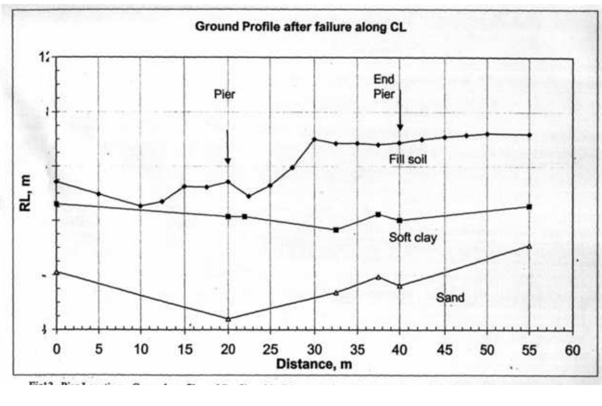

Figure 2 Pier locations with profile of soil layers along the centerline of bridge and embankment after failure (after BRTC, 2002)

Recent geotecnical investigations carried out after the failure show that undrained shear strength of the soft clay layer is slightly more than that assumed in the stability calculations described in previous article. Field vane shear test results indicate that the corrected undrained shear strength (using Bjerrum's correction factor of 0.8) of the soft clay layer can be taken as between 15 kPa and 30 kPa within the portion over which a fill of about 6 m height of soil exist.

Stability analyses were performed using the existing soil profile. In all analyses fill strength has been taken as 55 kPa. Values of factor of safeties were found to be 1.527, 1.814 and 2.186 for undrained shear strength values of 10 kPa, 15 kPa and 20 kPa of soft clay layer respectively. Here the most critical circle is for factor of safety of 1.81. Therefore, under the existing fill height (approximately 6 m) the slopes may be considered stable. But any further loading will reduce the factor of safely of the slope and may reinitiate sliding.

6 STABILITY OF EMBANKMENT SECTION AFTER PLACEMENT OF FULL HEIGHT OF FILL

Stability analyses of the embankment were performed with an expected full fill height of 9 m. In all analyses fill strength has been taken as 55 kPa. Initially, the soft clay layer has been assumed to be homogeneous and of uniform strength. Values of factor of safeties were found to be 0.897, 1.017 and 1.66 for undrained shear strength values of 10 kPa, 15 kPa nad 20 kPa of soft clay layer respectively. These results show that the minimum undrained shear strength of the soft clay layer should be about 20 kPa for stability of the embankment. It is highly unlikely that the shear strength of the clay layer near middle pier and toe of the embankment can be increased beyond 10 kPa because no surcharge can be placed over this portion. Although the strength of the clay layer beyond the bridge opening can be increased by preloading due to fill above it.

Considering above conditions, stability analyses of the embankment were performed with an expected full fill height of 9 m. In all analyses fill strength has been taken as 55 kPa. The clay layer has been divided into two zones as shown in **Fig 3**. In one area where preloading is not possible the undrained shear strength of the clay has been assumed to remain at 10 kPa. While in the other area where preloading will occur, shear strengths of 15 kPa, 20 kPa and 30 kPa have been assumed. Values of factor of safeties were found to be 0.971, 1.062 and 1.243 for undrained shear strength values of 15 kPa, 20 kPa and 30 kPa of soft clay layer respectively in the preloded zone.

From the above results, it appears that factor of safety of finished slope of the embankment is marginal even with an undrained shear strength of 30 kPa of the clay layer in the preloaded zone. From the field vane shear tests the undrained shear strength of the soft clay layer have been found to vary between 15 kPa and 30 kPa. Therefore, it will not be possible to place 9 m of fill at a time, i.e., placement of additional 3 m of fill over existing 6 m fill. However, the fill can be placed after allowing sufficient time to consolidate (about 70 percent) the soft clay layer in order to gain adequate shear strength.

From laboratory consolidation tests performed undisturbed samples from the soft clay layer an average c_v-value of 4 m^2/yr can be considered. With this c_v-value and a length of drainage path of 4 m, time required for 70% consolidation will be approximately 19 months. This estimated time is based on the assumption that the clay layer is homogeneous. However, the materials in the soft ground are usually heterogeneous. The sedimentation environment of the materials are changed frequently during the formation of soft ground and soils with a variety of grading, including sandy materials, must be present in the soft ground. Therefore, in situ c_v-value will be much higher than that obtained in the laboratory and as such the actual time required for 70% consolidation of the soft clay layer will be less than19 months.

The additional 3 m fill can therefore be placed in three stages. At each stage a fill thickness of 1 m should be retained for about 5 months before placement of next stage of fill. This would require a total waiting time of about 15 months. Considering construction schedule for the project, this may be too long a waiting time. Although installation of sand drains or wick drains may expedite the consolidation process. AS the portion of the soft clay near the bridge pier and embankment toe cannot be preloaded some uncertainty remains about overall effective ness of preloading. Therefore, improving soil by preloading is not recommended instead it will be better to add an additional bridge span on the Bhatiapara side of the approach embankment.

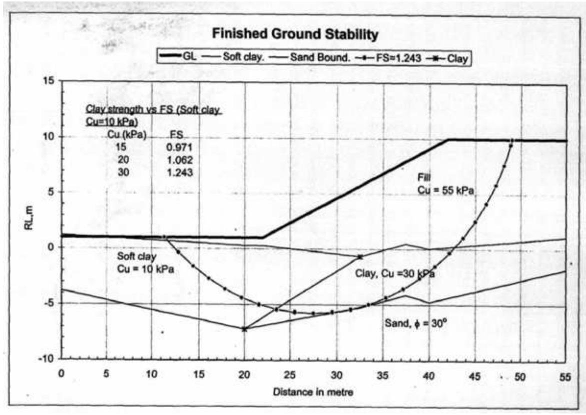

Figure 3 Stability analysis of existing ground for different values of undrained shear strength of the soft clay layer

7 CONCLUDING REMARKS

An investigation into the causes of failure of embankment along Bhanga-Bhatiapara section of Southwest Road Network Development Project (Contract No. 3) at 5+540 km was carried out. Using available soil reports, additional soil investigations and survey data, a series of stability analyses were conducted for the failed section of the embankment. The investigation reveals that a very soft clay layer exists below the embankment fill. Stability analyses reveal that failure of the embankment is attributed to the existence of this very soft sub-soil layer over which the fill has been placed.

Stability calculations also show that it will not be possible to place 9 m of fill at a time, i.e., placement of additional 3 m of fill over existing 6 m fill. However, the fill can be placed after allowing sufficient time to consolidate the soft clay layer in order to gain adequate shear strength. This may be achieved by placing the additional 3 m fill in three stages. Each stage of fill of thickness 1 m should be retained for about 5 months before placement of next stage of fill. This will require approximately 15 months time. As this is time consuming and also some uncertainty remains about the overall effectiveness of preloading due to the fact that portion of soft clay near the bridge pier and embankment toe cannot be preloaded, this method is not recommended. Instead it is suggested that an additional bridge span towards the Bhatiapara side may be added to shift the loaded portion of the embankment to somewhat better ground.

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