Aug 11th - Aug 16th

Failure of Two High Embankments at Soft Soil Sites

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6th International Conference on
Case Histories in Geotechnical Engineering, Arlington, VA, August 11-16, 2008

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ABSTRACT

Two highway bridge approaches, about 10 and 12 m in height, near Calcutta, India constructed with mechanically stabilized earth failed recently. These embankments were founded on soft and compressible, fine-grained soils of the intertidal flats and backswamps of the Ganges delta. One of these embankments, which failed in the final stages of its construction, was constructed after foundation soils were strengthened with preloading and prefabricated vertical drain installation and the other second embankment that failed within a month of its opening for traffic was constructed on unimproved ground. Fortunately, direct collateral damage from these incidents was small. Available geotechnical data indicated that design inadequacy was the main cause of these failures. Using pre- and post- consolidation shear strengths the embankments were redesigned. Reconstruction involved PVD installation at the second site and construction of stabilizing berms at both locations. The facilities are now operational and appear to be performing satisfactorily. Details of the failures, post failure investigations and monitoring, and redesign are presented in the paper.

INTRODUCTION

Two highway embankments constructed with mechanically stabilized earth failed recently near Kolkata (Calcutta) of southern part of West Bengal state, India (Fig. 1). The sites are located within the intertidal flats and backswamps of the Hooghly, a major distributary of the Ganges (Fig. 2). One of these sites, KM 18 site, remain waterlogged throughout the year, and the other, KM 26 site, also remains waterlogged over prolonged periods. Both the embankments were retained by mechanically stabilized earth (MSE) walls constructed with compacted river sand reinforced with galvanized steel reinforcements.

Essential details of the incidents are presented first followed by outlines of the subsurface investigation and monitoring programs, the inference from these programs, redesign of the earth structures, remedial measures incorporated and subsequent reconstruction work are presented later.

Fig.1. Failures at KM 26 and KM 18site.

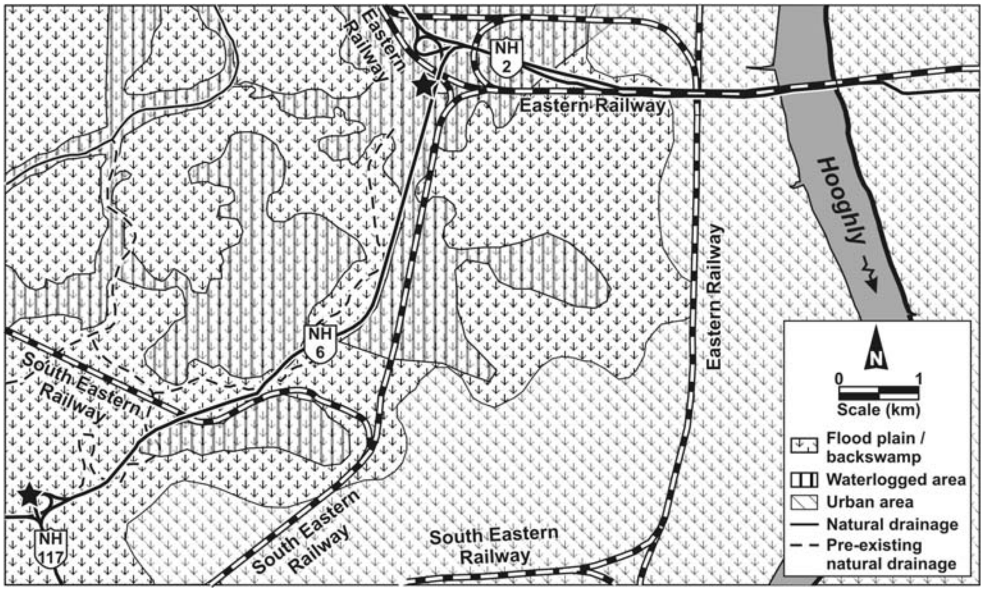

Fig. 2. Geological and geomorphological setting.

INCIDENTS

KM 26 Failure

The first event affected a highway interchange structure, under construction since July 15, 2003. The structure is retained by a Mechanically Stabilized Earth wall along the outer shoulder and with the fill slope of 2 (H):1(V) along the inner shoulder. The nearly-complete MSE wall underwent a deep seated failure in the early hours of rains received and an additional 0.4 mm of rainfall on the following day. The height of the affected embankment was between 8.9 and 9.8 m at the time of the incident. The cross section of the failed MSE wall is shown on Fig. 3. Post failure inspection indicated that the MSE wall failed due to external instability without significant internal distress.

KM 18 Failure

The second incident involved an MSE wall that runs along the western edge of the northbound lanes of a highway approaching a railway overpass. A 30-year old, 9-m high earth embankment with 3H to 1V side-slope along the eastern edge of the approach carries the southbound traffic. The failure occurred immediately after midnight on February 9, 2006 about a month after the highway was opened for vehicular traffic. During the failure, a section of the newly constructed 2-lane approach vertically settled by about 3 m and laterally translated outward by about 1 m. The cross section of the failed MSE wall is shown on Fig. 4. As at KM 26 site, the MSE wall at KM 18 site appeared to have failed due to external instability without significant internal distress.

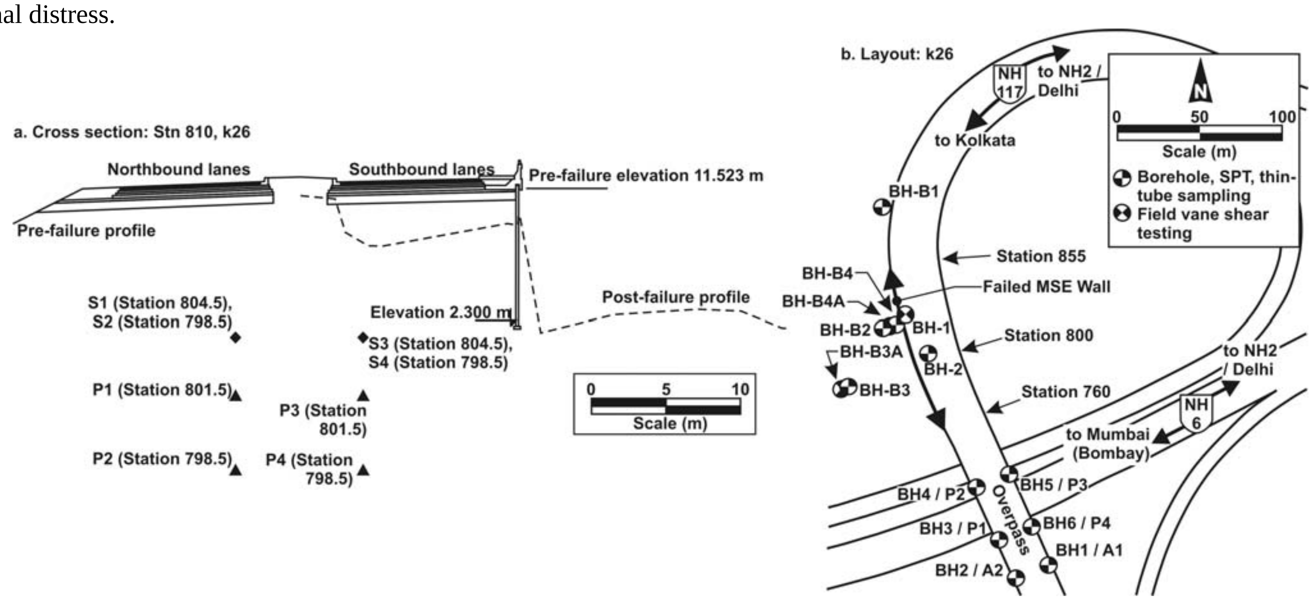

Fig. 3. Cross section and layout: KM 26 MSE wall.

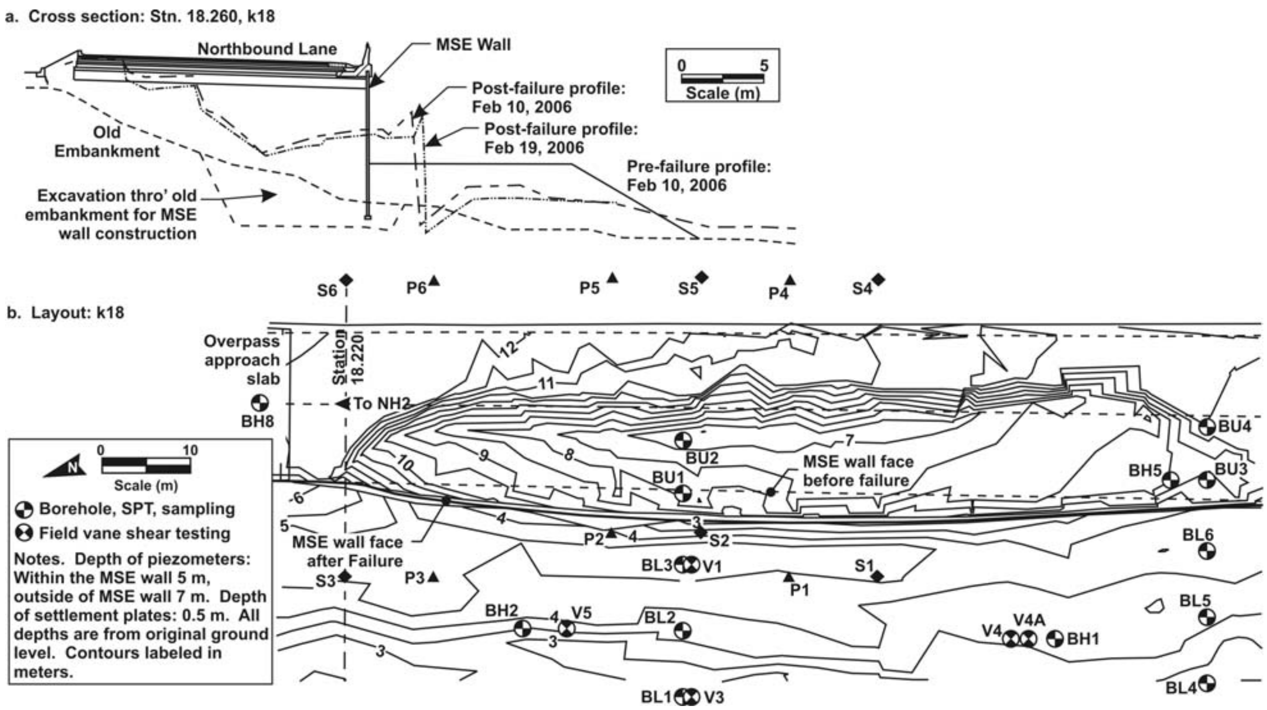

Fig. 4. Cross section of the KM 18 MSE wall.

The failed MSE wall was constructed for four laning of a pre-existing two-lane, undivided highway that runs on the 30-year old embankment with 3(H):1(V) side slopes. The construction of the old embankment also triggered several slope failures. One such failure occurred in 1966, when the embankment reached its full height 10.7 m. The instability was assessed to be due to a deep-seated circular slip that day lighted just beyond the toe. During this failure, the maximum embankment height within the stable stretch was 6.7 m. As remedial measures, the failed embankment was removed, the highway elevation was lowered and the embankment was reconstructed along with a 2.1-m high stabilizing berm along the edges of the embankment. Another slope failure occurred later immediately to the south of the railway tracks. Since poor subsurface conditions did not allow construction of an embankment of required height at the location of this failure, the earth embankment at the location of this failure was replaced by structural spans added to the bridge structure for carrying the overpass. Details on the original geotechnical investigation and design of the highway in this area can be found in Gangopadhyay and Das (1969).

Incidentally, in the recent constructions for four laning project, the north approach to the railway overpass at KM 18 site was of similar details as those of the failed MSE wall. The subsurface conditions at the north approach site and the location of KM 18 failure were also comparable. However, unlike the failed MSE wall, the north approach was constructed on ground improved by PVD installation followed by preloading and the MSE wall along the north approach has remained stable since its construction.

SUBSURFACE INVESTIGATION AND MONITERING

KM 26 Site

Pre failure subsurface investigation basically consisted of drilling of six boreholes for foundation design for the overpass structure and other boreholes were drilled in the post-failure investigation. The field work in these investigations included conducting standard penetration tests (SPTs), extraction of thin-tube samples, and field vane shear testing (VST). The laboratory tests included unconsolidated undrained (UU) triaxial and one dimensional incrementally loaded consolidation tests on selected thin tube samples, and tests for grain size distribution, natural moisture content, liquid limit and plastic limit.

Subsurface investigations at KM 26 indicate that the site is underlain by 15-m thick, grey, silty clay of Holocene age over stiff, yellow-brown, stiff silty clay of Pleistocene age containing calcareous nodules and silt and sand interbeds. The top 10 m of the Holocene soil was soft and contained organics, and the lower 5 m of this unit was firm. Groundwater was within 1.0 to 1.5 m of the original ground surface at the time of post-failure investigation. The soil samples were classified as CL according to ASTM D2487 (ASTM 2007). The undrained shear strength, s_u, of KM 26 site soils from UU tests and VSTs are plotted in Fig. 5a against the effective vertical stress, σ'_v. All raw undrained shear strengths from VST measurements were corrected in this study according to Bjerrum (1974).

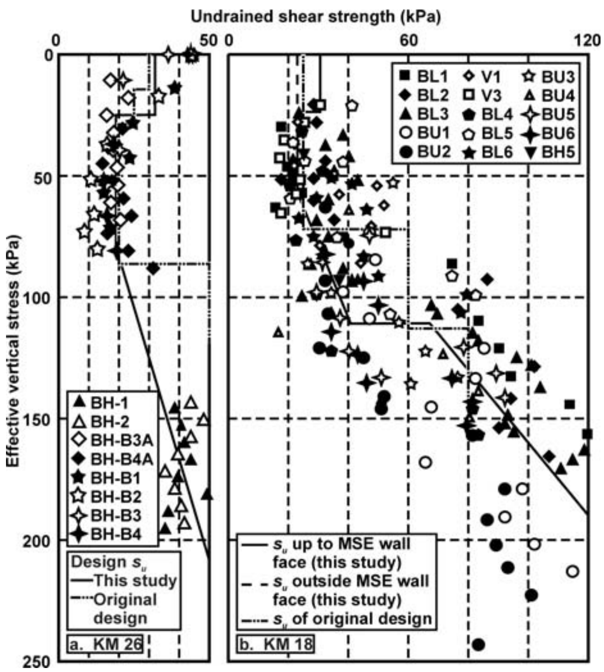

Fig. 5. Undrained shear strengths.

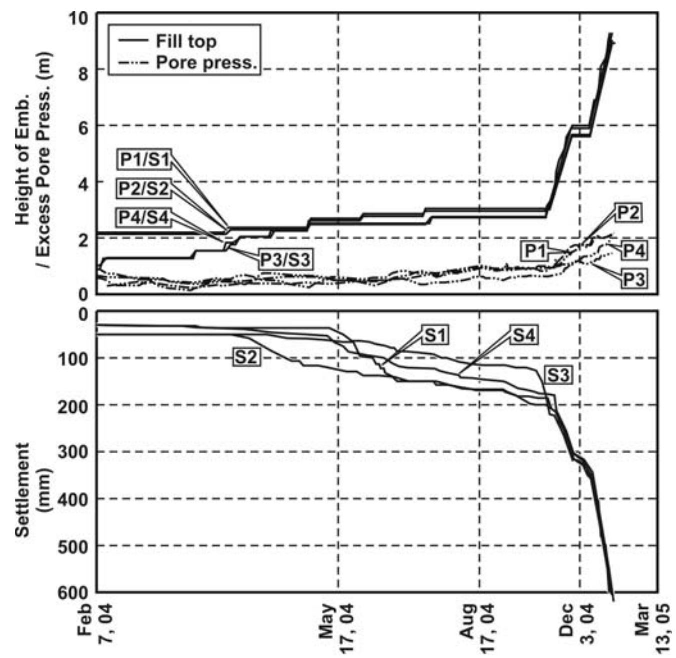

Fig. 6. KM 26 site monitoring data.

One dimensional consolidation test data from KM 26 site indicated that at the time of failure the consolidation process under the stresses imposed by 6.7 m high embankment was complete. This result was used for preparing Fig. 5a. The sensitivity of the soil layers were between 4 and 7.

The embankment and the MSE wall at KM 26 site were constructed after installing PVDs to depths between 11.5 and 15.5 m in square grids with 1.5 m spacing. The PVD treated zone covered the entire embankment footprint and extended to 4 m outside of the MSE wall face. To monitor the settlements and pore water pressure development during fill placement, four settlement plates S1, S2, S3 and S4 and four stand pipe piezometers P1, P2, P3 and P4 were installed near Station 800 (Fig. 3). These monitoring data indicate that the rapid construction rate after October 31, 2004 lead to a rapid pore water pressure development and accelerated settlement rate (Fig. 6).

KM 18 Site

The available geotechnical data at the design stage originated from two boreholes and all other boreholes were drilled after the failure. The field work in these investigations included conducting standard penetration tests (SPTs), extraction of thin-tube samples, and field vane shear testing (VST). The laboratory tests included unconsolidated undrained (UU) consolidation tests of selected thin tube samples, and tests for grain size distribution, natural moisture content, liquid limit and plastic limit.

Data from these investigations indicate that the site is underlain by a sequence of Holocene silty clay, over stiff silty clay of Pleistocene age. The upper 5 to 8 m of the Holocene unit was firm and over-consolidated with pre-consolidation pressures of up to 200 kPa underlain by a 3 to 5-m thick soft, compressible, normally to lightly over-consolidated layer containing organics and peat inclusions. The deepest part of the Holocene soil was firm and normally consolidated. The Pleistocene unit classified as silty clay with sand or sandy silt partings was stiff and over-consolidated with pre-consolidation pressures of up to 300 kPa. Groundwater was within 1.0 to 1.5 m of the original ground surface at the time of post-failure investigation. The soil samples classified as CL according to ASTM D2487 (ASTM 2006). The values of s_u for KM 18 site soils from UU tests and VSTs are plotted in Fig. 5b. For plotting these data it was assumed that the deposits were completely consolidated under the stresses due to the old embankment, and that the degree of consolidation due to the newly constructed MSE wall that failed, the embankment behind the MSE walls and the stabilizing berm was 50 % over the top 2.5 m thickness of the native foundation soils. The consolidation of the deeper native soil layers because of the new embankment was neglected. The sensitivity of the soil layers were between 4 and 7.

The site was only instrumented before MSE wall reconstruction. Earlier there was no instrumentation at KM 18 site. However, a brief chronology of initial construction is as follows. Since the site is permanently waterlogged, a 1-m high earth embankment was constructed in mid January 2004 about 1-m to the west of the MSE wall alignment for dewatering the area for construction. Dewatering began in late January 2004, after which the site was stripped to a depth of about 500 mm. A 500-mm thick compacted sand pad was placed on the stripped surface. Where the base of the MSE wall was to be at an elevation lower than the original ground level, the original ground surface was excavated for accommodating a 500-mm thick compacted sand pad underneath the base of the MSE wall. Sand filling commenced in early February 2004 and MSE wall construction began by the end of February 2004. The MSE wall and the reinforced sand embankment behind it were about 8 m high by mid June 2004 and 9.25 m by early February 2005. There was virtually no earthwork between mid June 2004 and mid January 2005. Earthwork began in end November 2005 and the embankment construction was complete in early December 2005. The paving work was completed by mid January 2006 and the stretch was opened for vehicular traffic by mid January 2006.

Data from the instruments installed after failure for monitoring the reconstruction activities, presented in Figure 7, indicate that the settlements were continuing to develop and pore water pressures were still dissipating after about 4.5 months of preload placement (completed in the first week of January 2007) and PVD installation (completed in the third week of October 2006).

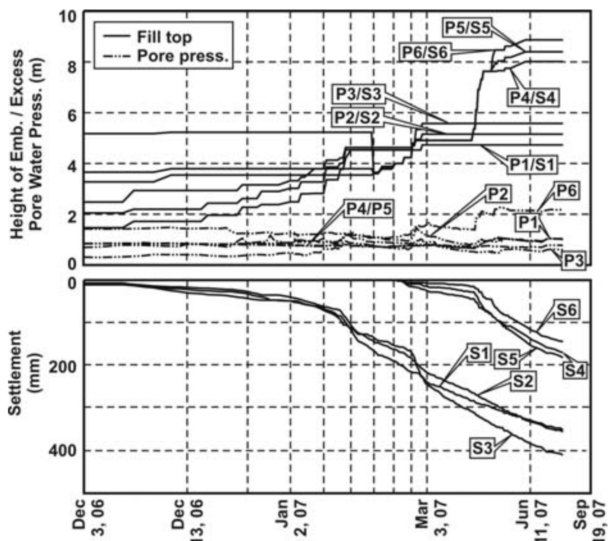

Fig. 7. KM 18 site monitoring data.

INITIAL EXTERNAL STABILITY ASSESSMENT

The failure patterns at KM 26 and KM 18 sites are indicative of deep seated failure due to external instability of foundation soils. Consequently, internal designs of the MSE walls are not examined in this paper. Design of the embankments was to be according to the Indian Roads Congress (IRC) document IRC: 75 (IRC 1979). This document calls for a limit equilibrium factor of safety against external failure of 1.25 and an allowable settlement of up to 600 mm. Brief accounts on limit equilibrium stability analysis of KM 26 and KM 18 embankments are provided in the following subsections.

KM 26 Embankment

As indicated earlier, construction at KM 26 site began after installation of PVDs (Colbonddrain® CX1000) to 15 m depths on an average in square grid pattern at 1.5 m centers. A mandrel of diamond-shaped cross section with diagonals nominally measuring 50 mm and 120 mm was used in PVD installation.

Measurements from KM 26 site indicate that the undrained shear strength at this location can be expressed as a function of effective vertical stress shown with a solid line in Fig. 5a. Since, the undrained shear strength is a function of effective stress under which the deposit was in equilibrium, for estimating the shear strength at the time of failure the state of consolidation at that time needs to be assessed.

At KM 26 site, the average degree of consolidation was obtained using $c_{vh} = 0.02\ \mathrm{m^2/day}$ (c_{vh}: coefficient of consolidation for flow in the horizontal direction) and $k_h = 1\times10^{-10}\ \mathrm{m/sec}$ (k_h: horizontal hydraulic conductivity). These estimates are from the one dimensional consolidation tests performed in the laboratory and the assumptions that the ratio of vertical to horizontal coefficients of consolidation of 2 and smear zone diameter to be 2.5 times the equivalent mandrel diameter applicable for massive deposits (Hansbo 2004). The results indicate that the average degree of consolidation for the foundation soils at the time of failure for the vertical pressure for embankment height of 3.5 m was 100 % and that for the stage above 3.5 m that was constructed relatively rapidly between October 31, 2004 and the time of failure was about 50 %.

For the profile of peak undrained shear strength indicated with the solid line on Fig. 5a, the results of limit equilibrium stability computations for the MSE wall configuration at failure indicate that the structure was marginally stable at the time of failure according to the Generalized Limit Equilibrium (GLE) procedure (Chugh 1986) (Fig. 8). Software package XSTABL Version 5.1 (Interactive Software Designs, Inc. 1994) was used in all stability analysis of this study.

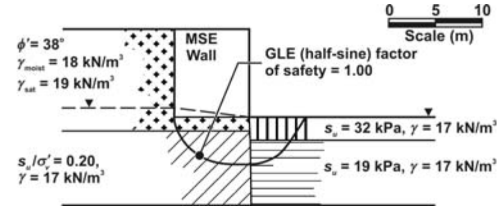

Fig. 8. Undrained stability of KM 26 embankment at failure.

In comparison, the minimum factor of safety against circular slip in the original design was under static loads was 1.15 in the slope stability analysis. However, in the original design the maximum considered embankment height including pavement structure was 9 m against the actual maximum height of 9.8 m and a total unit weight of 15 kN/m^3 (in stead of 18 to 19 kN/m^3 representative of the compacted river sand used to construct the embankment) was assumed for the embankment body because of the initial plan of constructing the embankment partly with fly ash. Most importantly, the strength parameters used in the original design, shown on Fig. 5a for comparison, were based on the assumption that when the embankment construction reached a height of 9 m, the average degree of consolidation for the soil layers underneath would be 90 % under the surcharge imposed by an embankment of 7.1 m height resulting in an undrained shear strength of 50 kPa minimum. Such a degree of consolidation was not achieved because of rapid construction rate. Moreover, even if 90% consolidation was to be achieved, available test data from the site indicate that the undrained shear strength would only have increased to about 30 kPa, an estimate smaller than the value assumed in the original design. Thus the bases of the original design were, in general, unconservative. Even with these inputs, the overall stability requirement of IRC (1979) of a limit equilibrium factor of safety of 1.25 was not fulfilled.

KM 18 Embankment

Data from KM 18 site indicate that (a) the undrained shear strengths at this location decreases westward and (b) the undrained shear strengths can be expressed as functions of effective vertical stress as shown with a solid and a dashed line on Fig. 5b.

External undrained stability of the configuration of the MSE wall assessed using the simplified Bishop method (Bishop 1955). The input parameters used in the analysis were based on the assumptions that (a) the soil layers underneath the MSE wall were fully consolidated under the vertical stress imposed by the old embankment, (b) that the degree of consolidation due to the MSE wall and the newly constructed embankment was 50 %, and (c) the undrained shear strength profiles can be approximated with the solid and dashed lines of Fig. 5b. The strength parameters for the old embankment were back figured from stability analysis to match the observed instance of deep seated rotational failure during the construction of the embankment in the sixties described earlier. In the stability assessment of the old embankment, strength parameters for foundation soil were assumed in accordance with the dashed line of Fig. 5b. The results of external stability assessment for the MSE wall at failure indicate that the MSE wall and the embankment behind it were indeed marginally stable at the time of failure (Fig. 9).

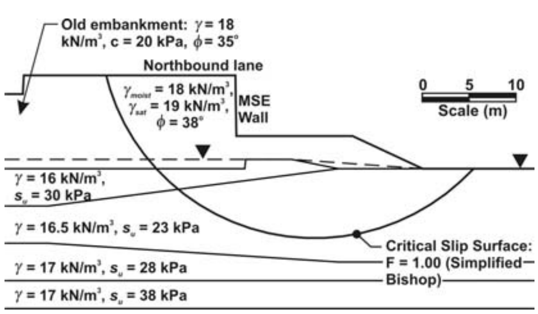

Fig. 9. Undrained stability of KM 18 embankment at failure.

In comparison, the minimum factor of safety against circular slip in the original design was under static loading was 1.42 in the slope stability assessment. However, in the original design the maximum considered embankment height including pavement structure was 8.4 m against the actual maximum height of 10.5 m and a total unit weight of 15 kN/m^3 was assumed for the embankment body because of the initial plan of constructing the embankment partly with fly ash. Most importantly, the undrained shear strengths used in the original design, shown on Figure 5b for comparison, were over a large portion of the depth range of interest greater than those inferred from laboratory and in-situ test data. Thus the bases of the original design were, in general, unconservative.

LESSIONS LEARNT

Undrained Shear Strength

The main factor contributing to the failures at KM 26 and KM 18 sites appears to be the difficulty of estimating the post consolidation undrained shear strength of the soft foundation soils. Examination of $s_u/\sigma'_v - OCR$ data from KM26 and KM 18 sites were found to be in reasonable agreement with the SHANSEP framework (Ladd et al., 1977) and relationship between s_u/σ'_v and OCR could be approximated by $s_u/\sigma'_v = 0.25 \times (OCR)^{0.76}$ ($r^2 = 0.80$).

Trigger for External Stability

It is apparent from the data and results presented in the previous section that the failures at KM 26 and KM 18 appears to be due to the use of inappropriately high shear strength, incorrect cross sectional geometry and unit weight of embankment material in the initial stability assessments. Both the embankments were externally marginally stable at the time of failure.

It is evident from available instrumentation records that the embankment at KM 26 site was undergoing rapid settlements over approximately 2.5 months prior to failure. The construction rate was not controlled to allow settlement rates to decelerate. As deformations increased the strain within the foundation soils beyond those at which the peak undrained shear strengths are mobilized. This eventually led to failure.

While the situation at KM 18 site also appears to be similar, no direct evidence of accelerated settlement rate immediately before failure is available because of the absence of instrumentation and monitoring of initial construction activities continuing through the early operational phase of the MSE wall and embankment. However, available surveying records from the site at and immediately after indicates that the event was a progressive failure possibly triggered because of ongoing deformations of sensitive foundation soils due to inadequate factor of safety against external instability.

Information obtained during post failure investigations and review earlier records from both sites before failure indicate that both the failures could be avoided by undertaking adequate geotechnical investigation at the design stage and appropriate coordination between the design office and construction activities.

RECONSTRUCTION

Design of Remedial Measures at KM 26 Site

Reconstruction work at the KM 26 failure site included construction of a two-stepped, 5.2-m high stabilizing berm along the outer face of the MSE wall followed by the reconstruction of the MSE wall and the highway embankment. The overall berm width measured outward from the face of the reconstructed MSE wall to the toe of the lower bench was 31.4-m.

The minimum factor of safety under static loads against overall rotational failure for the MSE wall, embankment and stabilizing berm at the end of construction is estimated at 1.20 (Fig. 10).

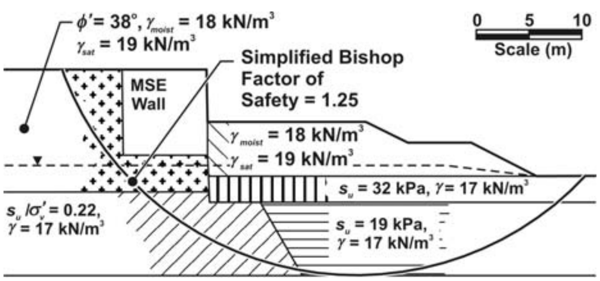

Fig. 10. Undrained stability of remodeled KM 26 embankment.

Since berm construction was scheduled before MSE wall reconstruction, the shear strengths for the stability assessment were based on the assumption that the foundation soils within the PVD treated zone extending to a distance of 4 m outward from the MSE wall face would consolidate under the weight of the berm by the time the MSE wall reconstruction was complete. As such, the factor of safety meets the recommendation of IRC (1979). Furthermore, as the foundation soils underneath the berm consolidate, the factor of safety is expected to reach a value of 1.35 after about 5 years of berm construction.

Reconstruction Activity at KM 26 Site

The earthwork for berm construction was complete by mid April 2005. The MSE wall and highway embankment construction was complete by mid June 2005. The pavements construction was complete by July 2005 and the reconstructed highway interchange was reopened for vehicular traffic in November 2005. Since then the earth structure appears to be performing satisfactorily.

Design of Remedial Measures at KM 18 Site

The remedial measures at the KM 18 site included installation of PVDs to a 13-m depth, construction of a two-stepped stabilizing berm along the outer face of the failed MSE wall, and reconstruction of the MSE wall and the highway embankment. For a configuration shown in Figure 11, the minimum factor of safety against overall rotational failure for the MSE wall, embankment and stabilizing berm at the end of construction was estimated at 1.22. For these assessments it was assumed that the consolidation of soils within the PVD-treated zone would be 75 % complete at end of construction and the effect of consolidation outside the PVD-treated zone was neglected. Although the computed factor of safety at end of construction is slightly smaller than the recommendation of IRC (1979), in longer term the factor of safety is expected to reach 1.38 upon completion of consolidation of soils within the PVD-treated zone.

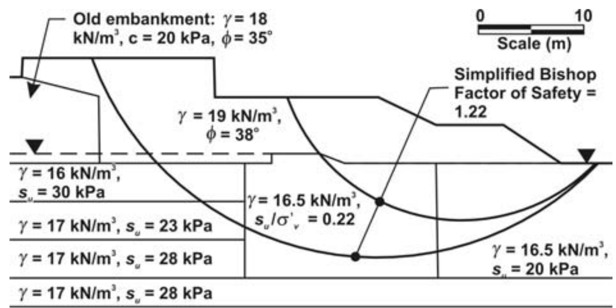

Fig. 11. Undrained stability of remodeledKM 18 embankment.

Reconstruction Activity at KM 18 Site

PVDs (Colbonddrain® CX1000) were installed in October 2006 at 1.2 m spacing in square grid pattern using a mandrel of diamond-shaped cross section with diagonals nominally measuring 50 mm and 120 mm was used in PVD installation. The PVD treatment covered an area of width of 19 m (measured outward from the face of the failed MSE wall) north of station 18.300 and 17 m south of station 18.300. The stabilizing berm was constructed between October 2006 and February 2007. The upper bench of the stabilizing berm was 20 m wide and reached an elevation 3 m below the finished road level. The maximum overall berm width measured from the base of the reconstructed MSE wall to the toe of the lower bench was 37 m. Three standpipe piezometers and three settlement gages were installed through the stabilizing berm as discussed earlier. After the removal of the failed embankment to an elevation approximately 5.75 m above the original ground surface, reconstruction work for the MSE wall and the highway embankment was taken up in February 2007. Three standpipe piezometers and three settlement gages were installed within the footprint of mechanically stabilized earth wall after the removal of the failed embankment. These installations were extended with the increasing elevation of the top of the constructed embankment. Drilling, sampling and Standard Penetration Testing were carried out near the end of April 2007 to check whether the gain in undrained shear strength due to accelerated consolidation of soft soils within the PVD-treated zone was as assumed in redesign. These data indicate that the undrained shear strengths increased by between 25 % and 100 % following accelerated consolidation of the soils within the PVD-treated zone and for the most part exceeded those assumed in the overall stability assessment for embankment redesign. As expected, the increases were most significant within the softer zones and relatively minor within firm to stiff layers.

Embankment reconstruction above 8 m height was allowed from May 2007 after these data were reviewed and fill placement and paving work was completed by the beginning of June 2007. The reconstructed highway embankment was reopened for vehicular traffic by mid-2007. The earth structure appears to be performing satisfactorily since the reopening of the highway. All the monitoring instruments at this site were in serviceable conditions throughout the reconstruction work and the initial operational phase of the structure. Monitoring data from these installations during the reconstruction work and initial operational phase of the highway have been presented earlier.

CONCLUSIONS

Case histories pertaining to highway embankment failure at two soft soil sites have been presented. Both the sites are located within the floodplains and backswamps of river Hooghly, a distributary of the Ganges underlain by soft silty clay and clayey silt. One of these sites, KM 18, remains waterlogged throughout the year, and the other, KM 26, also remains waterlogged over prolonged periods. The MSE wall and the associated embankment at KM 26 site was constructed on ground improved by PVD installation followed by preload placement, while construction at KM 18 site was undertaken after dewatering the work site over unimproved ground.

Geotechnical investigation and analytical work undertaken after the failures indicate that the KM 26 and KM 18 failure was caused by the following:

- Rapid construction rate that did not allow consolidation of foundation soils needed for development of undrained shear strengths assumed in the initial design, and
- Underestimation of driving force because of the use of smaller embankment heights and material unit weight in the overall stability assessment of the initial design.
- The post-failure investigations at the KM 18 site indicated, on the other hand, that KM 18 failure was primarily caused by
    - Overestimation of undrained shear strength, and
    - Underestimation of driving force because of the use of smaller embankment heights and material unit weight in the overall stability assessment of the initial design.

Both these incidents could be avoided by undertaking adequate geotechnical investigation at the design stage and appropriate coordination between the design office and construction activities.

Extensive geotechnical investigations were completed at both failure sites after failure to investigate the causes of failures. Review of these data and available pre failure information indicate that the SHANSEP approach provides a reasonable guidance for estimating the undrained soil strengths of the soft foundation soils at these sites.

The reconstruction at KM 26 site basically involved construction of a stabilizing berm along the outer face of the MSE wall. The reconstruction at KM 18 site involved installation of PVDs along the outer margins of the MSE wall and preloading of the PVD-treated area and undertaking MSE wall and highway embankment reconstruction following adequate consolidation and strengthening of the soft soils within the PVD-treated zone. Reconstruction activities at KM 26 and KM 18 sites spanned approximately six months. Both these structures have been carrying vehicular traffic over several months and appear to be performing satisfactorily.

ACKNOWLEDGEMENTS

The authors would like to thank Professors C. Subbarao, S.P. Dasgupta and D.P. Ghosh of IIT Kharagpur, and Mr. C.C. Bhattacharya, former Director General (Road Development), Ministry of Road Transport and Highways, Government of India for reviewing parts of the work reported here. Thanks are also due to Mr. H. Fries Mr. P. Minhas, and Lt. Col. G.G.K. Nair of the project implementation team for technical help.

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