Excessive settlements and failure of two embankments caused by degradation of soft rockfill

Déplacements excessifs et rupture de deux remblai causés par la dégradation du remblai rocheux

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ABSTRACT: The construction of two 40m high embankments on Izmir-Cesme Motorway was completed in 1992. 20m high-reinforced earth walls supported the downstream slopes of the embankments. Highly weathered clayey schists excavated from adjacent cuts were used in the construction of both the embankments and reinforced earth walls. Continuous vertical deformations were observed on the embankments that reached to 70cm in 1998. Lateral displacements at the crest of walls were in excess of 60cm. These magnitudes of deformations led to a prefailure state in the embankments and rupture of connections of reinforcements to panels of the walls. A detailed site exploration program was undertaken to determine the causes of these deformations and local failure of the walls. The findings indicated that grain crushing and degradation of the soft rockfill caused the excessive settlements. Degradation of the fill material caused a significant reduction in the friction angle of the fill material and increased the lateral earth pressures on the reinforced earth structures.

RÉSUMÉ: La construction de deux remblais routiers, chaqu'un ayant une hauteur de 40m, situés le longue de l'autoroute Izmir-Cesme, a été fini en 1992. Les pentes des remblais en aval sont soutenues par deux murs en terre armée. Pour la construction des remblais et des murs en terre armeé, des schistes argileuses hautement modifiées par l'air et excavées dans l'endroit adjacent ont été utilisées. Des déformations verticales continues sur les remblais, qui ont atteint 70cm en 1998, ont été observées. Les déplacements lateraux à la créte des murs ont été plus de 60cm. Ces magnitudes de déformations ont causé à un état de prérupture dans les remblains ainsi qu'une rupture des connexions d'armatures des paneaux des murs. En vue de déterminer les causes de ces déformations et des ruptures locales des murs, un programme exploratoire detaillé a été exécuté sur le terrain. Les résultats obtenues ont indiqué que l'écrasement des grains et la dégradation des remblais rocheux mous ant été dans l'origine de ces déplacements excessifs. La dégradation des remblais a causé une reduction considerable dans l'angle de frottement du remblais et a augmenté les contraintes laterales du sol sur les structures en sol armé.

1 INTRODUCTION

Two sections of the Izmir-Urla-Cesme Motorway are located in a rugged terrain and are built across the steep ravines along the foothills of mountains. The construction of two embankments, D5 and D6, being 40m high were completed in 1992. Downstream slopes of the embankments are supported by 20m high-reinforced earth walls being situated as toe buttresses. Highly weathered clayey schists excavated from adjacent cuts were used in the construction of both the embankments and the reinforced earth walls. A typical cross section of the embankment D6 is shown in Figure 1. Continuous vertical deformations were observed in the embankments causing excessive cracking and depressions on the pavements. The settlements reached to 70cm in 1998 without any indication of a reduction in the rate of settlements. Measurements on the reinforced earth walls indicated lateral displacements in excess of 70cm at the crest of Tier I. These magnitudes of deformations and the mode of lateral spreading in the embankments detected by the inclinometric measurements, indicated that the embankments experience a global stability problem and reached to a prefailure state. Moreover crushing of corners of the panels, as well as rupture of connections of the metal strips to the precast panels resulting in falling down of some panels were the clear indications of internal stability problems in the walls.

It is interesting to note that the two embankments (i.e. D5 and D6) have quite similar geometry; built on the same bedrock with steep gradients; and almost identical fill materials were used in the construction of both the embankments and the reinforced earth walls. The problems faced in the two embankments were also very similar in nature.

A detailed site exploration program has been undertaken to determine the causes of the external and the internal stability problems. The investigations included borings, inclinometric measurements, sampling and laboratory testing; monitoring settlements and ground water levels; and SPT and pressuremeter tests. This paper describes findings from the investigations of the walls, probable failure mechanisms and the remedial measures considered.

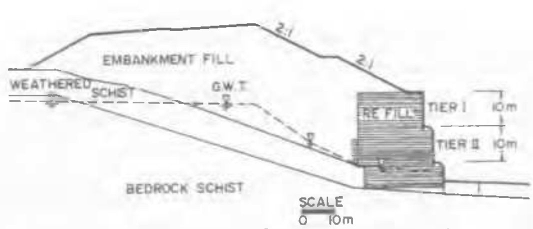

Figure 1. The typical cross section of the Embankment D6

2 DEFORMATIONS IN THE EMBANKMENTS

The results of topographic measurements on the embankments as well as on the reinforced earth walls are as follows:

- Embankment D5: Measurements were started in March 1995. Settlements reached to 50cm on the right shoulder of the pavement in December 1997. Settlements on the centerline and on the left shoulders were on the order of 22cm and 8cm respectively. Tier I of the toe buttress settled 65cm, and lateral movements at the crest of the reinforced earth wall reached to 72cm.
- Embankment D6: Measurements were started in April 1993. Settlements recorded on the right shoulder of

the pavement was 74cm; on Tier I 28cm, and on Tier II 11cm. The lateral movement on the crest of the reinforced wall was on the order of 68cm.

Vertical and lateral deformations measured at the crest of Tier I of Embankment D6 are shown in Figure 2. Another 44cm of settlement and lateral deformations would be added to S_V and S_H in Figure 2. These deformations occurred from 1992 to 1993 during which topographic measurements are not available. It is noted that both the settlements and the translational deformations continue to increase without any trend of reduction in the post construction rate of deformations for a period of 5 years. The magnitude of the deformations in both directions is in the same order of magnitude; and the rate of deformations in vertical and lateral directions is almost the same (i.e. 75mm/year).

In general the magnitude of the deformations was proportional to height of the embankment and highest settlements took place at the right shoulder of the embankment where the thickness of the fill is maximum.

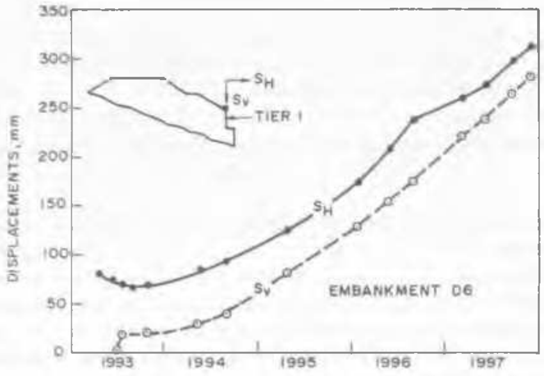

Figure 2. Vertical and lateral deformations measured at the crest of Tier I of Embankment D6

The inclinometric measurements taken between August 1997 and April 1998 (i.e. 9 months) are shown in Figure 3.

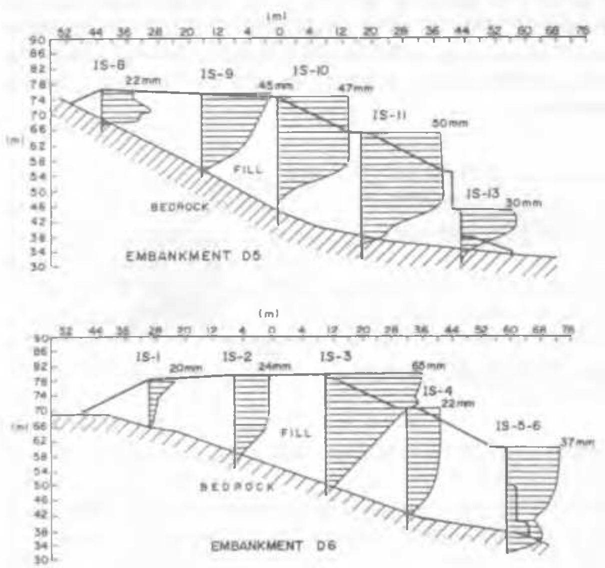

Figure 3. The inclinometrric measurements taken between August 1997 and April 1998

The magnitude of the lateral deformations varied over a range from 20mm to 65mm being proportional to the height of the embankment. The movements are creep type in the upstream side of the embankment; and more or less slide type in the downstream part where the reinforced earth walls are situated. This mode of deformations may suggest that slide and translational type movements occur around the reinforced earth walls possibly due to lack of adequate internal and external stability; and rest of the embankment follows this movement as creep deformations. The inclinometric deformations show that movements start at the embankment fill-base rock boundary indicating a plane of weakness at the base of the embankment. This is possibly due to the presence of ground water close to the bedrock surface and its capillary affects as will be discussed later.

Overall stability of the embankments is evaluated by back analysis along non-circular failure surfaces. In determining the failure surface, the major cracks on the pavements and the inclinometric measurements were considered. Janbu's (1956) method with parallel interslice forces was used in the analysis. The results of back analysis revealed that an operative residual friction angle of $\phi'_r = 19^0$ for the embankment fill. This magnitude of frictional resistance has been found not comparable with the design friction angle of $\phi' = 36^0$.

3 CHARACTERISTICS OF THE FILL MATERIAL

Highly weathered clayey schists excavated from the nearby cuts were used in the construction of both the embankments and the reinforced earth fills. Typical grain sized distribution of the fill material is shown in Figure 4. According to the specifications of the project, minimum of 90% modified proctor maximum dry density is required; and this compaction level was achieved in every stage of the construction with only few exceptions.

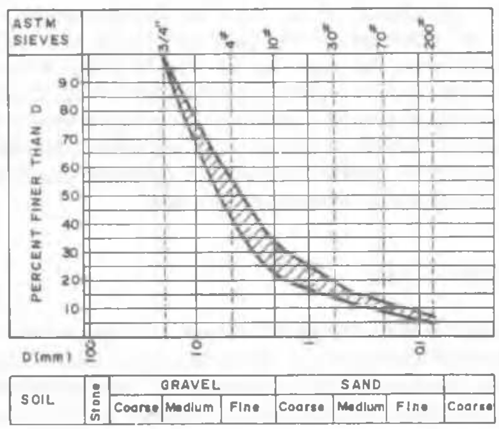

Figure 4. Typical grain size distribution of fill material

There are laboratory test including sieve analysis of the fill materials carried out during placement of the fill. Recent investigations have been undertaken 5 years after the construction and gradation of the fill material were determined on the samples recovered by continuous dry coring. During this period the embankments experienced excessive deformations. The recent data allowed comparison of the gradation of the fill material during placement and after the construction.

The histograms given in Figure 5 show that a significant particle crushing and degradation have taken place in the fill material resulting in increased percentages of fines (silt and clay); and reduction in the coarse fraction (sand and gravel). This is a well-known phenomenon referred as " evaluation of weak and weathered rock when used as embankment fill" in the literature (Vaughan, 1994; Lade et al 1996). When weak rock is used as fill material in high embankments, the material consists of primarily angular coarse gravel and cobble size grains possessing large pores and porous fabric, during the placement of the fill. Although some particle crushing takes place during compaction, the process is not finalized unless heavy over compaction efforts are utilized during the construction. Subsequently high pressures due to self-weight of the embankment act at the particle contacts giving rise to particle crushing and grinding. Eventually crushed materials fill the voids and significant volume reduction takes place in the embankment. Presence of water accelerates this degradation process. The ground water levels shown in Figure 1 in Embankment D6 were detected during recent investigations in 1997. It is noted that the ground water levels are close to the base rock surface and rose into the embankment at the center line of the embankment. Presence of water level in the embankment as well as the capillary rise of ground water through the highly weathered upper layers of the bedrock aggravated the degradation of the fill material at lower elevations where high overburden pressures act.

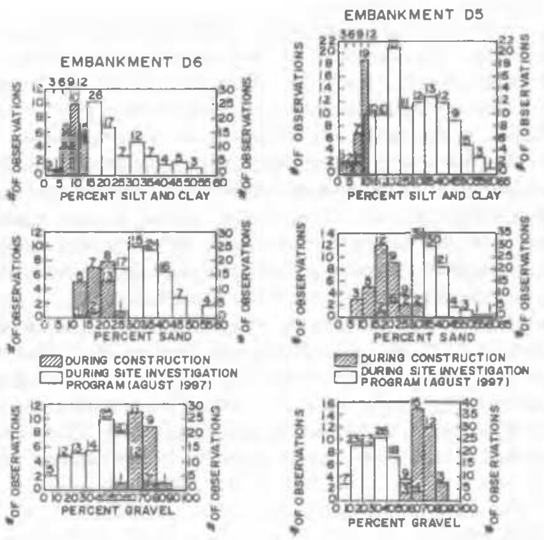

Figure 5. Histograms illustrating the change in the grain size of the fill material

The characteristics of the fill obtained from recent investigations are shown in Figure 6. Average properties of the fill are as follows: Fine content (<0.074mm) varies over a range from 5% to 55%, however in the majority of the samples fine content is in the range 15% to 35%. In general the material is non-plastic; in some samples liquid limits LL= 30-35% and plasticity indices PI=10-15 are noted. Gravel size particles (i.e. >4.76mm) comprised 30% to 60% of the fill material. Standard penetration resistances are on the order of N_{45} = 20-30 blow counts/ft, indicating that presently the fill is in a medium dense to dense state.

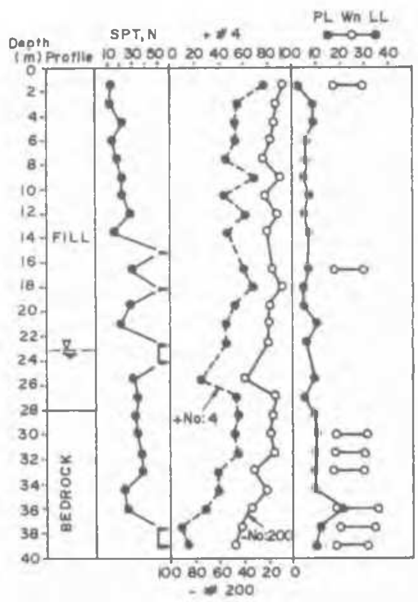

Figure 6. The characteristics of fill material obtained from recent investigations

The results of drained direct shear tests on representative compacted specimens prepared from sampled recovered from the embankment fill by dry coring revealed friction angle of ϕ' = 28-31^0 (8 tests).

The bedrock in the region consists of 6m to 12m thick highly to extremely weathered clayey schists and graphitic schists; underlain by relatively intact rock. There are large blocks (up to 1.0m nominal diameter) of intact rock in the upper zone. This zone has the characteristics of a residual soil with N_{45} = 33-44 and percent fines %F=30-50.

4 DISCUSSION OF FAILURE MECHANISM

The two reinforced earth walls being 20m high supporting high embankments above the walls have undergone continuous excessive settlements and translational deformations. The damages included intolerable depressions and cracking on the pavements, bulging and tilting in faces of the walls, crushing in the corners of the precast panels, and finally rapture of panel-strip connections and falling down of several panels which are interpreted as the signs of forthcoming collapse of the embankments. The observations and the results of the analyses are insufficient to identify a unique failure mode that led to the collapses and movements. There are several deficiencies in the project concerning selection of fill material, design and construction of the embankment which all contributed to the failure.

In the particular project the responsibility for the internal and the external stability of the reinforced earth structures are separated. Reinforced earth walls were designed assuming friction angle of ϕ' = 36^0 for both the wall backfill and the embankment fill and only internal stability was considered in the design. The designer's specifications required that the fill shall not contain material which will loose its frictional characteristics, material sensitive to water. Apparently these specifications were not followed. A more realistic friction angle for the compacted fill which has experienced grain crushing and degradation would be ϕ' = 30^0 as revealed by the shear box tests on the laboratory compacted samples. Consequently active K_a and at rest K_0 earth pressure coefficients are higher than what has been assumed in the design producing a significant increase in the design tension in the reinforcing strips. The ruptures at the panel-strip connections are due to under design of the strips in tension. Moreover higher magnitudes of external earth pressures acted on the reinforced earth structures possibly resulting in unforeseen tilting and translational movements of the reinforced earth walls.

As suggested by Lee et al (1994) another failure mechanism may arise from translational movement of the reinforced soil mass placed on slopping terrain. Such movement of the reinforced wall would have a down slope component that would reverse the direction of the shear stresses at the back of the reinforced soil mass as illustrated in Figure 7. This would reduce the vertical stress acting on the reinforcing strip around the rear end of the reinforced soil mass. Thus a reduction in the mobilized friction between fill material and reinforcing strips would trigger pullout or wedge failures. This may result in further movements and bulging of the wall face and an increase in the tensile stresses on the strips. Differential settlement between the reinforcement and the facing panels may also contribute to overstressing the strips.

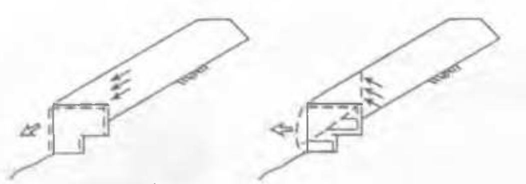

Figure 7. Illustration of effect of translational movement on reinforced earth wall (after Lee et al, 1994)

Lee et al (1994) also point out that two part composite wedge analysis proposed by Smith and Wroth (1978) reveal higher tensile forces on the reinforcing elements as compared to the widely used coherent gravity design method, particularly in toe buttress walls supporting embankments situated on sloping terrains. Therefore in the reinforced earth walls under consideration the reinforcing strips may be under designed with respect to the tension forces when the coherent gravity method is used.

The inclinometric data shown in Figure 3 indicates that the embankments suffer from overall stability problems. The probable failure surfaces starts somewhere on the right lane, extends down to base rock and reach to the reinforced earth region. The presence of the groundwater either in the embankment or close to the bedrock surface gives raises to wetting of the embankment fill material. Wetting occurs when the fill is in direct contact with groundwater; or through capillary rise of the groundwater that is situated at shallow depths in the bedrock. The overburden pressures are the highest at the bottom of the fill where wetting process is also in action. These factors aggravate the particle crushing and degradation mechanism, resulting in deterioration in the frictional properties of the fill material.

A global stability problem through the weakest zone at the bottom of the embankment may also be considered as a failure mechanism. In this case extremely high magnitudes of lateral pressures would act on the reinforced earth walls leading to a external stability problem and failure of the walls. Unfortunately present data do not allow to clearly stating whether the sliding mass of the embankment caused failure of the reinforced earth walls, or translational movement of the reinforced earth walls due to inadequate internal and external stability, triggered the global stability problem.

It is also worth to mention that in embankments crossing the steep ravines there is always a possibility that some spring water that may appear following the rainy seasons would seep into the embankment from slopes of the valley. To avoid seepage of groundwater into the embankment as well as to eliminate capillary effects, provision of a drainage blanket enclosing the entire embankment is a common design practice (Lee et al 1994). This blanket was missing in the presented case.

5 REMEDIAL WORKS

It was decided that a design solution is needed to address the internal stability problems of the existing structures and improve the global stability of the entire embankments. New reinforced earth structures placed to support the damaged reinforced earth walls comprised the remedial design scheme. The available land limited the width of the new structures to 10m. Thus the required height of the new walls was found to be on the order of 15-18m according to the slope stability analyses.

In order to minimize the settlements under the new embankments, upper few meters of the foundation soil was replaced by a rockfill. In the construction of the new reinforced earth structures crushed limestone, which is nill insensitive to water and has practically grain crushing potential, was used as backfill material. The design scheme for the Embankment D6 is shown in Figure 8. The construction of the new walls was completed in 1999, and the two embankments performed satisfactorily since then.

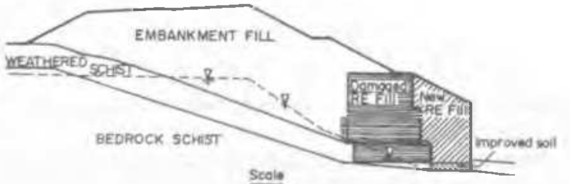

Figure 8. The remedial design scheme for the Embankment D6

6 CONCLUSIONS

Findings from the recent investigations revealed that the two embankments suffered from global stability problems. Moreover the reinforced earth walls supporting the downstream slopes embankments experienced internal and external stability problems. Excessive settlements, lateral deformations and break in the panel-strip connection indicated that the embankments reached to a failure state.

It is found that the main reason for the failure of the embankments is the degradation and crushing of the fill material used in the construction of the embankments and the reinforced earth walls. The degradation of the material result in lower friction angles than the frictional resistance presumed in the design stage, and subsequent increase in the lateral earth pressures and a reduction in the mobilized friction in the reinforcements. Presence of water leak to the embankments aggravates the process of the degradation. Lack of a drainage blanket enclosing the entire embankment, and no consideration given to the composite wedge analysis for a reinforced earth structure supporting and embankment on a sloping terrain were the other design deficiencies contributed to the failure of the embankments.

It is found that construction of a new reinforced earth wall in front of the damaged ones effectively improve the global stability of the embankment as well as internal and external stability of the existing reinforced earth walls. This solution was found more economical and efficient as compared to rigid retaining structures, which would attract excessive lateral loads.

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