Failure of Railway Embankment

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SYNOPSIS: This article deals with the failure of a railway embankment in North Peloponese. The geothechical investigation that followed aimed to specify the causes of the failure. The results of the above investigation are presented, an evaluation of the causes of failure is discussed and the remedial measures which have been taken are thoroughly described.

INTRODUCTION

In the north part of Peloponese (Greece), 20 Km after the city of Corinthos, a road rail juction was constructed. The juction was consisted of the railway embankment and a concrete fly - over The reasons for this structure were first to eliminate further collisions and second to reduce the distance by 3Km. The rail embankment project begun in 1983 and by October 1985 was completed, having max height 18m and length 350 m. Theborrow pit area, situated one mile north to the rail line, consisted of two horizontal layers, one with well graded clay-gravel and one with red clay. Consequently it was necessary to mix the material from the two layers into one,in place. The excavation was carried out by bulldozer and then a shovel tyred tractor loaded the material directly in trucks,passing under a metal screen to separate the boulders. The fill was hauled to the embankment´s site, spread by a bulldozer D7 in layers 50 cm thick along the axis and subsequently compacted with a vibratory half tyred roller, to the 90-95 percent Mod AASHTO density.
Next summer (1986) on the top of the embamkment, longitudinal cracks appeared, along the axis, having length 2-3m and width varying from mm to 3cm (Fig 1). After this occurence and before the lining of the rail steel bars, Public Works Research Laboratory-Soil Division, was asked by the inspector supervising the works, to proceed a geotechnical investigation, in order to identify the causes of the failure and propose measures to restore the embankment.

THE GEOLOGY OF THE NEAR BY AREA

The surrounding the project area is mainly consisted of brackish to lacustrine deposits,containing clay and clayey gravel, yellowish to white Marls, intercalations and lenses of loose or dense conglomerates, coarse sandstone and marly limestones, having Pliocenic age.
The embankment was lodged on a conglomerate lens as it was identified after visual inspection and two shafts made as deep as possible, by a mechanical digger. No borehole logging investigation was done.

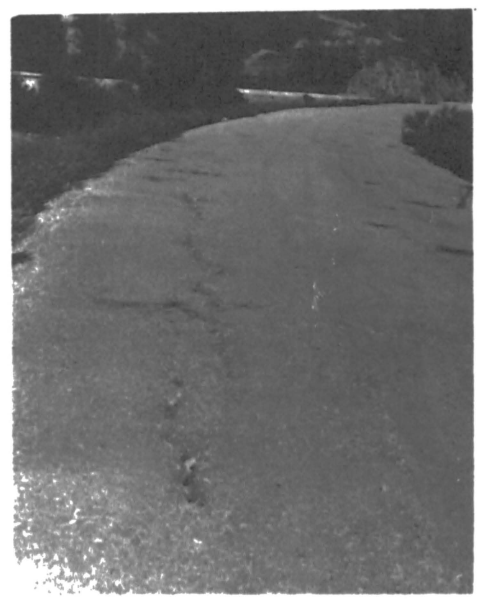

Fig. 1. Longitudinal cracks on the embankment.

LABORATORY INVESTIGATION

A laboratory testing program was initiated to identify the problem and design remedial measures. Soil samples were obtained from the embankment and tested for :
classification
linear shrinkage
swell potential
mineralogy by x-ray diffraction
shear strength tests

Classification Test

Particle size, liquid limit, plastic limit, plasticity index, were determined in accordance with ASTM test procedure. The coarse fraction passing US 3/8 sieve ranges between 32-66 percent. The fine material passing US 200 sieve varies between 21-92 percent.The liquid limit ranges between 32-43 percent. The plastcity index varies between 18-27 percent. The colloidal content (minus 2 micron) varies between 16-34 percent. So the soil samples were identified as GC,SC,CL material, by the Unified Soil Classification System (AUSCS).

Linear Shrinkage

Eight bar linear shrinkage tests were carried out according to BS.1377. For this semi spherical soil bars were prepared in moulds, out of the liquid limit test. The obtained values were :

10.3	14.2	11.4	12.4
10.7	12.3	8.5	14.3

According to Altmeyer (1955) and Chen (1975) all the above values (> 8) are in the critical stage of potential volume change.

Swell Potential

Load expansion swell tests were performed on re - moulded soil samples,taken out of Proctor tests. The moisture contents were the optimum and above optimum. Half inch thick samples were placed between air dry porous stones of a 2.5 inch diameter consolidometer. The samples were subjected to loaded and Expanded test.

Loaded tests : Initial dial reading was recorded after applying a small load of 0.0703 Kg / cm² (1 psi). The specimen was saturated under zero swelling condition by applying small increments of load, until the full swelling pressure was developed.

Expanded test : Initial dial reading was recorded after applying a load of 0.0703 Kg/cm². The specimen was saturated and expanded, until the expansion was completed under full swell conditions The results of swell potential are illustrated on table I.

TABLE I. Range of Engineering Parameters

W.C. %	D.Density KN/m³	Swell Press. Kg/cm²	Swell %
11.5	19.0	2.25	-
14.3	18.6	-	7.0
14.0	18.7	-	7.5
15.8	17.9	2.38	-
16.0	17.9	1.80	-
16.1	18.0	-	3.5

X - Ray Diffraction test

The mineralogical composition of the fine particles was determined by x-ray diffraction analysis, as described by Brown (1980). The sample was allowed to air dry, powdered, saturated with glycerin, heated to 550°C to decompose carbonates and tested by Cuka radiation. The following minerals were identified by their characteristic x-ray peaks :

Quartz	30 %
Calcite	30 %
Montmorillonite	15 %
Illite	15 %
Chlorite	10 %

Shear Strength Tests

The material of the embakment contained a large proportion of coarse grained fraction (gravel and cobbles), which made the use of conventional triaxial and direct shear box testing apparatuses extremely difficult or even impossible. In order to overcome this problem and model the actual field conditions, as closely as possible, it was decided that the large shear box apparatus,(300mm square by 160 mm thick), mainly used for testing aggregates, offered a great advantage.

Two series of tests were performed on prepared specimens with different water content and densities (γ_b = 18.5 KN/m³, γ_b = 20.5 KN/m³). Field densities and water content were previously determined in situ at different points and depths. The results from these shear tests are shown in figure 2.

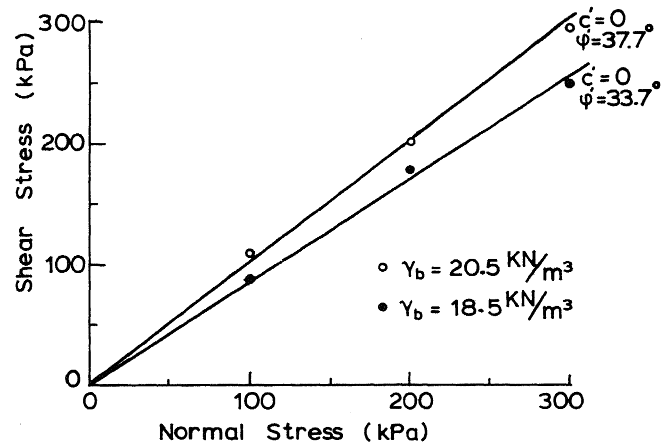

Fig. 2. Shear Strength Envelopes of fill material

SLOPE STABILITY ANALYSES

Slope stability analyses were performed on a typical cross section of the embankment, using as shear stress parameters the results of the large shear box tests. In the analyses we considered circular failure surfaces. The slope stability analyses were carried out for sixty-four possible centers of rotation and various slip circles.Different radius for each center were used.

Factors of safety in excess of 3.0 were computed, indicating adequate factor of safety against slope stability failure.

CAUSES OF FAILURE

The above mentioned investigation showed that there was no problem of slope stability failure because :

i. No signs of slope slides were observed and

ii. Slope stability analyses carried out using as shear strength caracteristics the results of shear box tests on prepared soil samples, indicated adequate factors of safety (> 3). The densities and water content of the samples were previously determined in situ, by the sand-cone replacement method.

From the linear shrinkage, the x-ray diffraction, the swelling percent and the swelling pressure tests, was revealed that the fine portion of the fill material contained swelling clay minerals. In this area there are remarkable sequences of wet and dry seasons (Fig.3). After the impregna - tion of the embankment during the wet season, intense shrinkage of the material followed, during the dry period, caused the appearance of cracks (Fig.1). In order to avoid the rainwater percolation,through the cracks and fissures,which would cause the material to swell and slake, the following remedial measures have been taken.

Fig. 3. Average Rainfall of Decade 1970-80

REMEDIAL WORKS

The top three meters of fill material were removed by a bulldozer D8 and carried away by trucks loaded by a tyred shovel tractor. For the next layer, the first 25cm were removed and kept in place. The other 25cm were scarified by a grader ripper and mixed with five percent lime (Fig.4).

Fig. 4. Soil-Lime Mixing by Grader

To obtain uniform mixture a disc harrow pulled by a tractor was used in eight passes. After the first pass, it was noticed that an amount of li - me was taken away by the wind. So it was decided to stop the mixing and wet the layer up to optimum. Dry unit weight of the material plus lime was 19 KN/m^3,optimum moisture content 13 percent. For this, six water tanks per layer were used and then the mixing by the disc harrow ought to be continued. Afterwards each layer was turned over by a grader Cat 14. For this, each layer was separated longitudinally in two strips, starting from the middle towards the edges. In order to succeed good mixing, thirty five passes of grader were necessary per layer. During the mixing several plasticity index tests were carried out in our project laboratory,until the material had achieved a non-plastic index.
Each layer was compacted by a roller Galion(14T) and one vibrating roller Hamm-Asdag (35T). In order to achieve hundred percent Mod AASHTO density, fifteen passes per layer were necessary. The ASTM sand-cone method was used to test the compaction of each layer. The rest of the fill, up to the top, was substituted with selected crushed gravel, having dry unit weight 22-23 KN/m^3, optimum moisture content 6-7 percent, and fine material passing US N200 sieve 5-7 percent. The material was hauled by trucks, spread by a grader to layers 30cm thick, wetted to optimum, scarified for better mixture with water and compacted by six passes of the vibrating roller, to hundred percent of Mod AASHTO density (Fig.5).

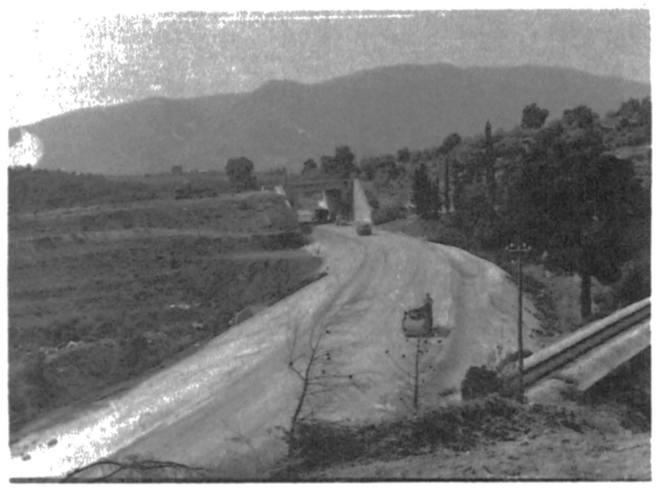

Fig. 5. Final Leveling of Embankment with Selected Crushed Material

After the final level was reached,the full length of the embankment was protected from water percolation and moisture changes. For this, the surface was covered by a membrane of hot petrol emulsion (3.5 lit per m^2) and then an asphalt membrane was applied by a man-carried springler (7 Kg per m^2). Finally because the whole stucture was too dark in colour, a thin layer of crushed stone (0.0-5.0 mm) was spread and stuck on top of the asphalt membrane.The above remedial measures are outlined in figure 6.
The works were completed by December 1986. Until August 1987 no complementary works have been carried out, that is placing ballast and rail-steel bars, in order to attend the behaviour of the embankment. No cracks or any other signs of failure have been noticed since then.

CONCLUSIONS

The absence of investigation for expansive soils during the design of embankments might have unfavorable consequences concerning the behaviour of the project.

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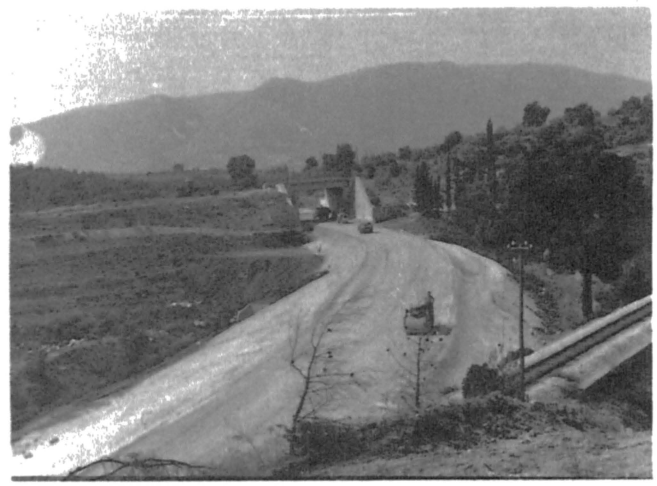

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The existence of expansive soils in relation to the peculiar climatological conditions in Greece (certain dry months are followed by periods of heavy rain and vice versa) Have as result to activate the destructive action of the clay minerals.
A proper geotechnical investigation should contain all the relevant laboratory tests in order to identify the existence of swelling soils.

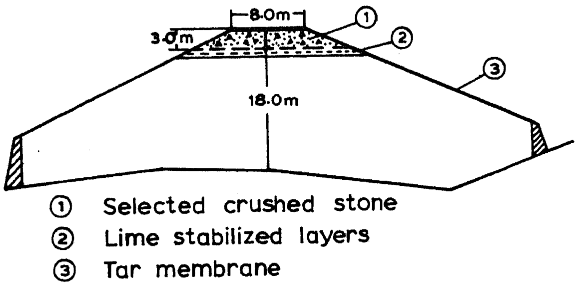

Fig. 6. Cross Section of Embankment after Remedial Works.

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