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Design and construction control of a large embankment with reinforced earth walls

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ABSTRACT : A case history is briefly outlined on the observational method for a high embankment with three stage reinforced earth walls, the overall height being 38m. The finite element method has been employed to predict the stress–deformation behavior of the complex structure. The parameters needed for the analysis have been estimated on the basis of the back analysis for an actual halfway performance. The finite element prediction based on the parameters back–analysed has been used to make a final decision on the cross section of the structure and also for the construction control.

1 INTRODUCTION

This paper is concerned with the observational procedure for a large scale embankment with reinforced earth walls, incorporating specifically the back analysis technique. In recent years, a greater number of embankments and/or cut–slopes with reinforced earth walls have been designed and constructed in Japan and their dimensions are gradually increasing. Conventional design procedures for these structures are, however, based only on the limit equilibrium methods which do not take into account deformation behavior. It is therefore quite difficult for designers not to have certain misgivings as to whether or not the conventional procedure is sufficient when the size of the particular structure exceeds a certain limit. The case study reported herein is a typical example of such structures.

In the observational procedure, the reinforced earth walls are assumed to be made of an orthotropic elastic material, whose elastic constants are determined from the back analysis. The embankment is assumed to behave like the nonlinear elastic material proposed by Duncan, et al., its properties being obtained from triaxial compression tests. Displacement measurements are used to perform the back analysis as the construction proceeds. Then, the results of the analysis are used to adjust the design as well as to set up a method of construction control.

2 DESCRIPTION OF THE PROBLEM

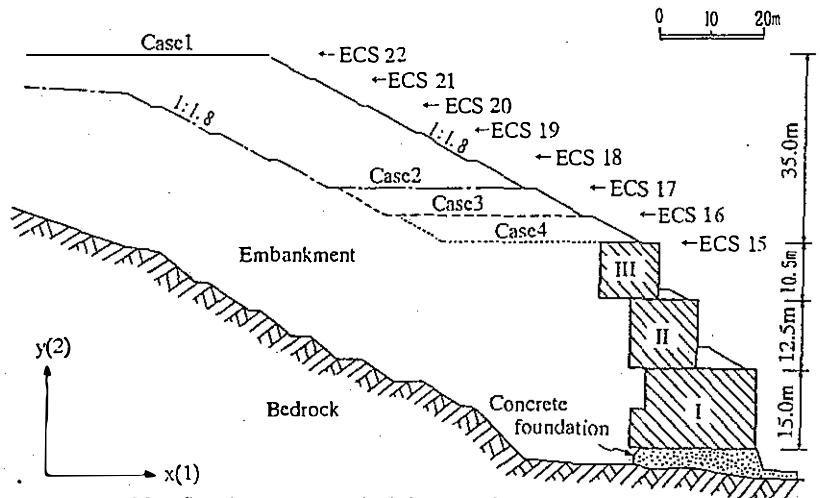
The project involved a high embankment construction flanked on one side by a complex of three stage reinforced earth walls having a maximum overall height of 38m as shown in Fig.–1. The height of the embankment is 35m above the top of the reinforced earth walls and the inclination of the slope is approximately 1:1.8. However, early in the planning stage there were four potential cross sections, i.e. Cases 1 ~ 4 as shown in Fig.–1. It was thus decided that the final cross section would be determined through the predictive stress–deformation analysis(FEM) for the four sections by using the necessary parameters obtained from the back analysis for an actual halfway performance.

The conventional design method ensured the stability for each of the four Cases both in static and seismic conditions. Moreover, stability analyses using the Janbu method have shown that under the following two conditions a) and b), the complex structure satisfies an allowable safety factor of 1.2:

a) Assumed slip lines are almost horizontal and do not intersect any strips within the reinforced earth blocks.

b) An increase in strength is not expected for the reinforced earth walls.

Fig.–2 shows the critical slip surface and associated minimum factor of safety from the stability analysis. In this way, the current structure if based on the conventional design procedures, will show



I, II and III : The first, second and third stages of the reinforced earth walls
ECS : Embankment construction stage

Fig.–1 Cross section of the high embankment

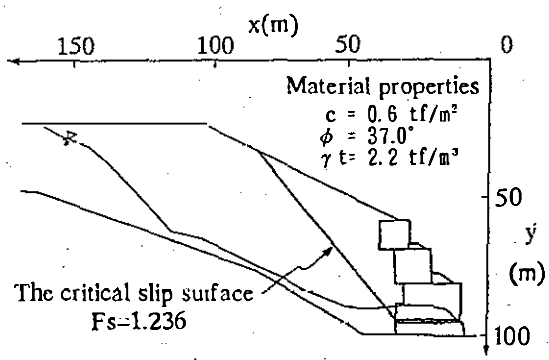


Fig.–2 Result of the stability analysis under conditions a) and b)

no signs of instability. But, as mentioned before, the authors do not believe that the limit equilibrium methods which do not take into account the deformation behavior, provide sufficient information for extremely large-scale structures. Therefore, through back analysis, the observational procedure had been introduced to select a final cross section and to be used in the construction control.

3 DESIGN PARAMETERS BASED ON BACK ANALYSIS

One very important aspect of the F.E. deformation analysis is how the reinforced earth wall, a complex of skins, strips and soils between strips, can be modeled in numerical analysis. The most realistic approach is to consider that the reinforced earth wall consists of three different materials. Frictional behavior between soils and strips or skins should be consequently taken into consideration with interface elements. We could not take this approach however due to time restrictions. Thus we have decided to idealize the reinforced earth wall as an orthotropic linear elastic body. The stress–strain matrix for the orthotropic elastic material(Zienkiewicz, Cheung 1967) is given as

$$[D]=\frac{E_2}{(1+\nu_1)A}\begin{vmatrix} n(1-n\nu_2^2) & n\nu_2(1+\nu_1) & 0 \\ n\nu_2(1+\nu_1) & (1-\nu_1^2) & 0 \\ 0 & 0 & m(1+\nu_1)A \end{vmatrix} \qquad (1)$$

$$(n = E_1/E_2,\ m = G_2/E_2,\ A = 1 - \nu_1 - 2n\nu_2^2)$$

where E_1 and E_2 = elastic moduli, G_2 = shear modulus, ν_1 and ν_2 = Poisson's ratios where subscripts 1 and 2 stand for the horizontal and vertical direction, respectively.

On the other hand, the embankment soil behind the reinforced earth walls has been modeled on the Duncan and Chang's nonlinear elastic constitutive law (Duncan, Chang 1970). The tangential modulus for this model is as follows:

$$Et = K\,Pa\left(\frac{\sigma_3}{Pa}\right)^{n'}\left\{1-\frac{Rf\,(1-\sin\phi)\,(\sigma_1-\sigma_3)}{2\,c\cos\phi+2\,\sigma_3\sin\phi}\right\}^2 \qquad (2)$$

Where c, ϕ, Rf, K, n' = material constants, and Pa = the atmospheric pressure.

How to determine the parameters appearing in the equations above is also a very important aspect. In principle, the material constants in Eq.(2) may be determined from triaxial compression tests, in reality though such tests cannot easily be conducted on coarse-grained embankment materials. Furthermore, no experimental procedure is available for determining the parameters governing the orthotropic elastic material.

Giving due consideration to what has just been described above, we have decided to procure the parameter values in terms of the back analysis. The fundamental concept of the back analysis is to minimize the objective function defined by the square sum of the differences between observed and predicted displacements with the help of some nonlinear optimization technique. Originally we intended to back analyse all the parameters in Eqs.(1) and (2). But preliminary investigations had revealed that it was quite difficult because of the excessive number of parameters to be back analysed. Consequently it had been decided that only the parameters in Eq.(1) were to be estimated through the back analysis. As for the parameters in Eq.(2), we employed the results of laboratory triaxial shear tests conducted on specimens, 10cm in diameter and prepared from a modeled material having a grain size distribution curve parallel to that of the prototype material. The values for the parameters so obtained are possibly close to but no longer real. Nonetheless, the parameters for Eq.(1) should be identified and this would presumably negate the error which may arise from the discrepancy. As a consequence, we have estimated that such an approach would have no serious defects in toto.

Making allowances for circumstances, we performed the back analysis by using the monitored displacements immediately after the second stage of the reinforced earth walls' construction. Fig.-3 shows the F.E. mesh used for the back analysis. Horizontal displacements obtained from an inclinometer attached to the surface of the skin plates were utilized in the back analysis as the observed displacements specified earlier. In Fig.-3 the dotted marks indicate the points at which the displacements were given to the analysis.

As stated before, the parameters to be back analysed herein were those for the orthotropic elastic material. The parameters for the embankment soil were known; these were as follows:

K = 1096, n' = 0.26, c = 10.1tf/m^2, ϕ= 35.5°, Rf = 0.77

Poisson's ratio was assumed to be 0.2. Foundation concrete and rock were both assumed to be linearly elastic, the modulus values of which were E = 1.0x10^{6}tf/m^2 and E = 1.4x10^{6}tf/m^2, respectively. Poisson's ratios were assumed to be 0.3 for both materials. The unit weight of the embankment soil was γ_t = 2.3tf/m^3.

The Nelder and Mead's simplex method (Kowalik, Osborne 1968) was employed as an optimization technique. The back analysis was performed with four different initial values for the independent variables, i.e. with four different initial simplexes. Table-1 lists the results; the computer used was a SONY NWS-3460. In the Table, U denotes minimum values for the objective function and IN the iteration number in the simplex method. Fig.-4 compares the back analysed horizontal displacements with the monitored ones.

As a result of this back analysis, the parameters for No.1 in Table-1 have been employed as the optimal solution because the value of U in No.1 is the smallest of the four, even though the differences do not vary from one to the other considerably.

4 DESIGN AND CONSTRUCTION CONTROL

Stress-deformation analyses were made predictively for each of the four Cases according to the parameters obtained. The objectives of the analyses were first to make a final decision on the cross section out of the four possibilities, and then to lay down the criterion for construction control. Fig.-5 gives the predicted horizontal displacements

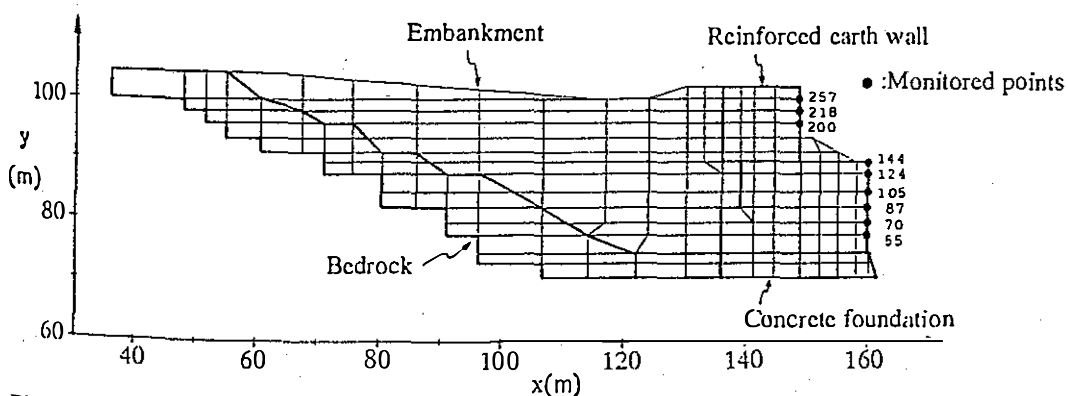


Fig.-3 Finite element mesh for the back analysis

Table-1 Summary of the back analysed parameters

	No. 1		No. 2		No. 3		No. 4	
	Initial	Back-analysed	Initial	Back-analysed	Initial	Back-analysed	Initial	Back-analysed
n	0.1	0.0473	1.0	0.0283	0.01	0.550	1.0	0.0382
m	0.01	0.0351	0.4	0.0283	0.01	0.0801	1.0	0.0323
E_2 (tf/m^2)	10000.0	25534.0	20000.0	28057.5	100000.0	27497.4	1000.0	25225.6
ν_1	0.1	0.231	0.3	0.00702	0.01	0.0222	0.1	0.0692
ν_2	0.1	0.0000248	0.3	0.109	0.01	0.000115	0.1	0.0564
U		0.000731		0.000739		0.000750		0.000734
IN		126		131		141		252
CPU time		389min		470min		445min		783min

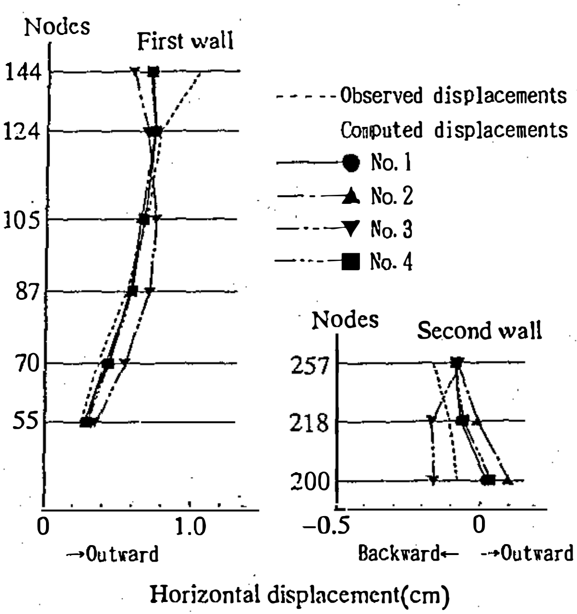


Fig.-4 Comparison between observed and back analysed horizontal displacements

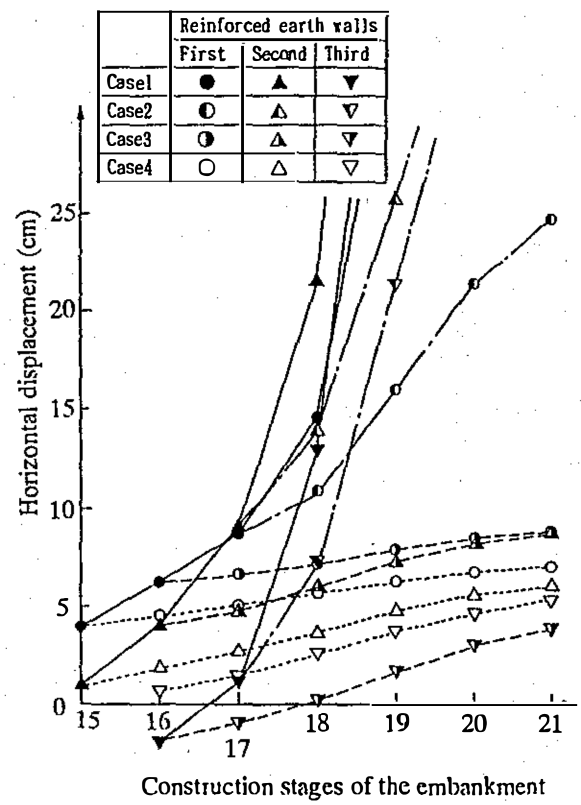


Fig.-5 Displacements at top corners of the reinforced earth walls

with progress of the construction at the top corner of each reinforced earth wall. Figs.-6~9 show the deformation behavior and the spread of failure zones in terms of the Mohr-Coulomb failure criterion. As seen from Fig.-5, extremely rapid displacement growth has taken place in Cases 1 and 2 after the eighteenth stage of the embankment construction sequence. Accordingly we could not draw the displacements with an appropriately reduced scale at the final stages for Cases 1 and 2. The state of deformation and the failure zones have thus been drawn at the twentieth stage, not at the final one, as in Figs.-6 and 7.

Case 4 of the cross section was ultimately chosen for this project through an integrated judgment of the displacements and failure zones. The following criterion was then set up as a construction control:

"After the fifteenth stage of construction, if the monitored displacements exceed the predicted ones for Case 4 shown in Fig.-5, then work will be abruptly halted and some countermeasure will be devised."

The construction work was in fact performed without violating this criterion at all.

This project started in 1989 and was completed in 1991. In Fig.-10 prediction and performance in the horizontal displacements are compared from the fifteenth stage of construction up to the twenty-first stage, that is just after the end of construction. The discrepancy is conspicuous between predicted and

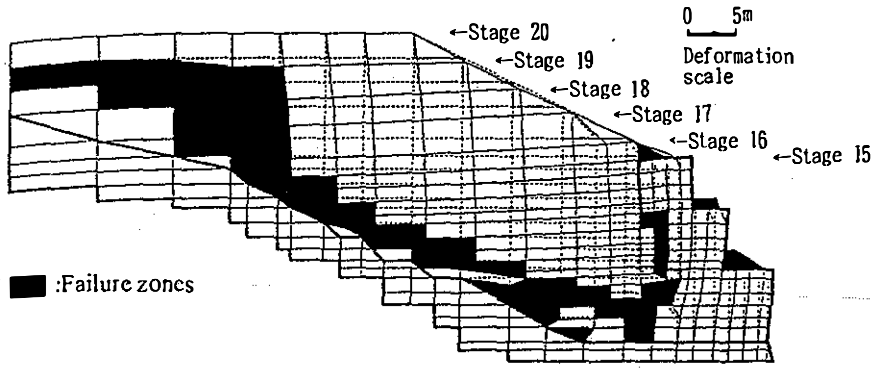


Fig.-6 Deformation behavior and spread of failure zones (Case1)

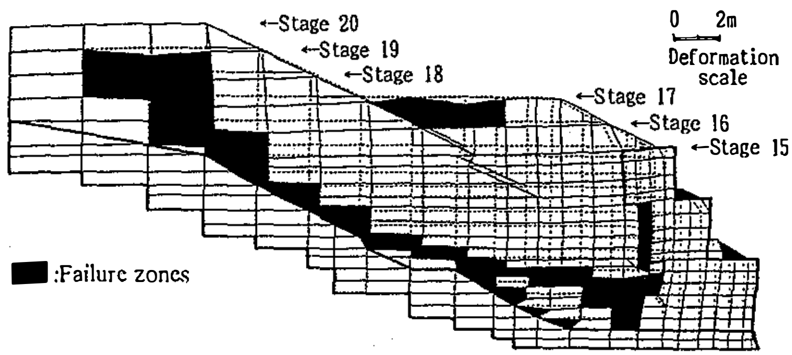


Fig.-7 Deformation behavior and spread of failure zones (Case2)

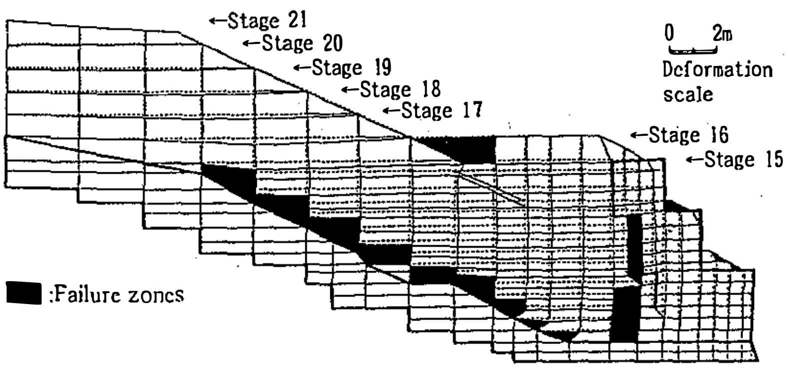


Fig.-8 Deformation behavior and spread of failure zones (Case3)

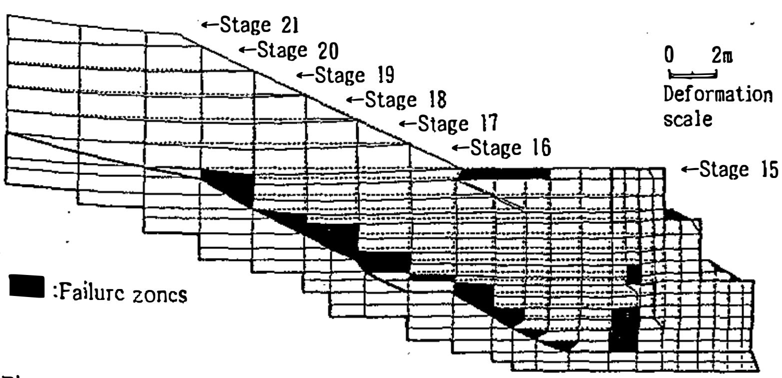


Fig.-9 Deformation behavior and spread of failure zones (Case4)

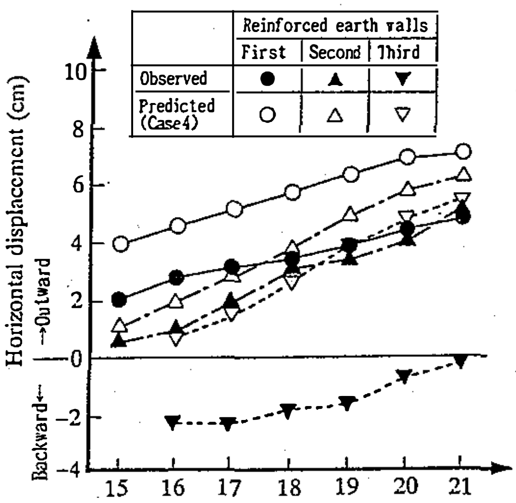


Fig.–10 Comparison of prediction and performance in horizontal displacements

observed displacements at the top corner of the third stage of the reinforced earth walls in Fig.–10. This is because, during the construction period for the sixteenth stage, some unexpected loading was made on the level ground surface behind the third stage of the walls. The reason for the loading is omitted here. It should, however, be noted that after removal of the load, i.e. after the sixteenth stage, the transition of observed displacements bear a similar shape to those of our predictions.

Fig.–11 shows the most recent data monitored in February 1992 together with the predicted data which are obviously irrelevant to elapsed time.

5 CONCLUDING REMARKS

The observational procedure has been used for the design and construction control of a large embankment supported by three stage reinforced earth walls. The back analysis was carried out for an actual halfway performance to identify the material properties used in predicting stress-deformation behavior. The selection for the final cross section of the embankment was made on the basis of the finite element analysis with properties back-calculated from field measurements. The result of the finite element analysis was also used for setting up a criterion for construction control. The construction work proceeded smoothly and without hesitation owing to the infallibility of the criterion. The case study described in this paper has conclusively demonstrated that it is quite effective to follow an appropriate observational method based on the back analysis for a large scale structure.

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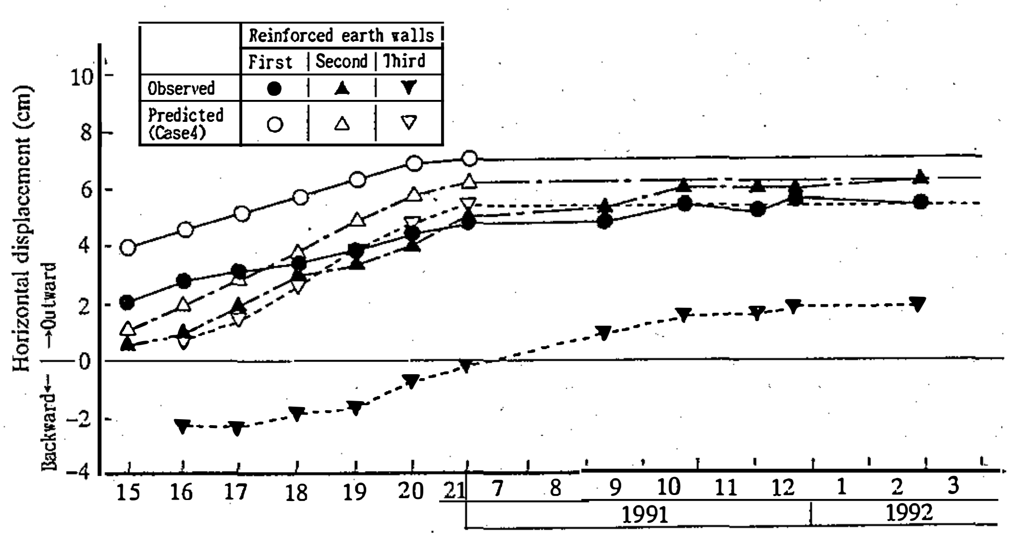

Fig.–11 Comparison of prediction and performance in horizontal displacements (The most recent data monitored in February 1992)