INTERACTION OF EMBANKMENT AND FOUNDATION ON CLAY

INTERACTION ENTRE REMBLAI ET ASSIETTE DANS L'ARGILE

ВЗАИМОВЛИЯНИЕ НАСЫПИ И ФУИДАМЕНТА НА ГЛИНЕ

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SYNOPSIS. The foundation performance of an embankment on a slightly overconsolidated clay is presented along with theoretical reasoning for the observed behaviour. The interaction of the embankment material and the foundation clay is shown to cause a redistribution of stresses, and a completely different behaviour to the normal one-dimensional analysis based only on the vertical stress. Analysis in terms of undrained and drained soil parameters shows this redistribution should occur.

The method of selection of the foundation soil properties is briefly outlined. A finite element programme which can deal with non-linear elastic elements is used to perform the theoretical analysis. Thus the interaction relationship is due to the continuum nature of the real problem rather than the normal one-dimensional treatment which is more common in present day engineering practice.

RESUME. On présente les performances de l"assiette d'un remblai sur de l'argile légèrement surconsolidée, ainsi que le raisonnement théorique qui explique le comportement observé. On montre que l'interaction entre le materiau du remblai et l'argile de l'assiette cause une redistribution des contraintes, et un comportement entièrement différent de celui predit par l'analyse unidimensionnelle normale qui est basée seulement sur les contraintes verticales. Une analyse tenant compte des paramètres du sol avec et sans drainage montre que cette redistribution devait avoir lieu.

On résume brièvement la méthode de sélection des propriétés du sol formant l'assiette. On se sert d'un programme à éléments à élasticité non lineaire, pour accomplir l'analyse theorique. Ainsi la relation d'interaction est due à la nature continue du problème réel plutôt qu'au traitement unidimentionel normal plus commun dans la pratique des ingénieurs de nos jours.

INTRODUCTION

The Kars bridge approach fill was built upon a cemented Leda clay foundation which was loaded so as it acted as a slightly overconsolidated foundation soil. The instrumentation of the foundation soil has been previously presented by Eden (1960) and a diagramatic layout is shown in Fig. 1. The embankment was built in two stages because of stability considerations (Eden and Poorooshasb, 1968). Stage 1 was placed in November 1959. Stage 2 was placed 20 months later. Eden and Poorooshasb (1968) have presented the observational records up to July 1967. The field data was kindly provided to the writer by Mr. Eden. An analysis of the undrained performance has been described by Raymond (1973). Herein discussion will be concentrated on the drained settlement behaviour only. Space does not permit a more extensive discussion and furthermore it is mainly the total settlements which are of major importance once stability has been assured.

CONSOLIDATION TESTING

Oedometer tests were performed to determine the soil's consolidation characteristics. Standard increment loading tests with direct permeability measurements were conducted on samples in which the drainage was both across and along the natural strata (i.e. determination of the vertical and horizontal coefficient of permeability) as recommended by Raymond and Azzouz (1969). In addition small load increment tests were performed in order to better define the preconsolidation pressure. Typical test results are given in Table 1 and Fig. 2. Fig. 2 shows the preconsolidation pressure results (Schmertmann's, 1955, minimum, probable and maximum) obtained from the same samples whose values are given in Table 1 with drainage across the strata. Also shown are the results obtained by Eden and Poorooshasb (1968). The tests performed under the writer's direction were sampled with a Swedish piston sampler and in agreement with the work of Raymond et al (1971) show a higher preconsolidation pressure, and therefore less apparent sample disturbance, than those obtained by Eden and Poorooshasb (1968) who used unlined steel tube samplers.

For design purposes the settlements were calculated using the small increment test results and the preconsolidation pressures shown on Fig. 2.

Table 1. Typical Consolidation Test Results

	Standard Increment						Small Increment	
	Drainage Across Strata (ie vertical)				Along Strata (ie horizontal)			
Depth	e_f	k_f	C_r	C_c	e_f	k_f	C_r	C_c
2.00	0.958	3.5	0.015	0.16	0.912	24.2	0.0100	0.282
4.00	1.305		0.020	0.32	1.31	8.6	0.0041	0.921
5.00	1.37	3.1	0.008	0.69	1.43	6.5	0.0031	0.929
6.00	1.66	1.0	0.022	1.10	1.58	4.1	0.0126	1.227
7.00	1.74		0.15	1.21	1.77	5.8	0.0057	1.333
8.00	1.62	5.2		1.12	1.58	7.8	0.0133	1.369
9.00	1.86	4.0	0.018	0.95	1.97	9.1	0.0058	2.014
10.00	1.96	4.3	0.040	1.32	1.93	6.5	0.0121	2.611
11.00	1.72	3.8	0.010	0.78	1.74	5.6	0.0099	1.561
12.00	1.89	6.2	0.010	1.47	1.94	6.5	0.0100	2.064
14.00	1.54		0.022	1.02	1.66	3.8	0.0073	2.479
m		mm/mo.				mm/mo.		

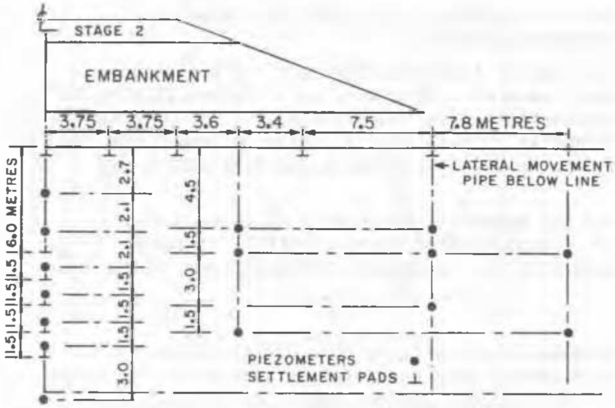

FIG. 1. INSTRUMENTATION BELOW EMBANKMENT

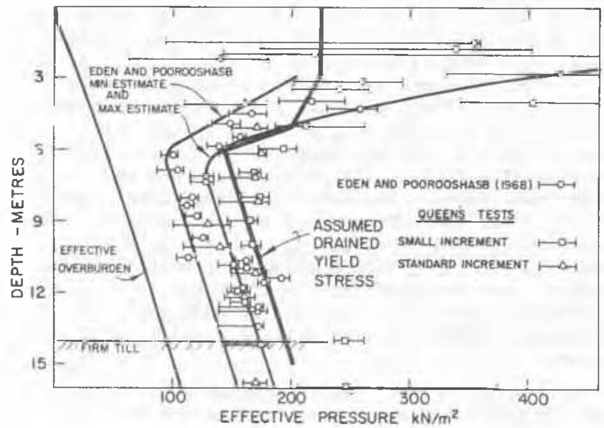

FIG. 2. OEDOMETER PRECONSOLIDATION PRESSURE RESULTS

DRAINED TRIAXIAL TEST RESULTS

All the drained triaxial tests were performed with a constant cell pressure and at a constant rate of strain. The specimens were first consolidated at different selected cell pressures measured above a back pressure datum. The back pressure was used to ensure the specimens remained fully saturated. Figure 3 shows typical results for strains up to about 3% obtained at a cell pressure equal to the overburden pressure. Except for the heavily overconsolidated specimens taken from the crust material the soil first increased rapidly in axial stress, and then increased slowly as the strain increased. Even at strains of 30% the lightly overconsolidated specimens showed no signs of failure. It may also be seen from Fig. 3 for the lightly overconsolidated specimens that the initial rapid loading is accompanied by a volumetric strain which is approximately directly proportional to the axial strain. This means that in this range Poisson's ratio from this type of test is constant. Figure 4 shows a series of tests performed on the crust soil at different confining pressures. Defining yield in the same manner as Mitchell (1970) then the axial stress difference and strain at yield increases and then decreases as the confining pressure increases. Based on a large series of tests the drained soil properties were selected as given in Table 2.

For the samples tested at confining pressures between 1 to 1/4 of the preconsolidation pressure the stress difference at yield when added to the confining pressure approximately equalled the preconsolidation pressure. Thus the yield stress differences $(\sigma_y - \sigma_o)$ shown in Table 2 were taken as the difference between the preconsolidation pressure and the overburden pressure for the deeper layer and somewhat less above 5 metres depth. The assumed drained yield stress (σ_y) is shown on Fig. 2.

Table 2. Properties Selected From Drained Tests

Depth Metres	Percentage Strain at Values of $\frac{\sigma-\sigma_o}{\sigma_y-\sigma_o}$ 1/4	1/2	3/4	7/8	15/16	1	$(\sigma_y-\sigma_o)$ kN/m^2	ν
0	0.306	0.695	1.25	1.60	1.72	2.50	225	0.20
3	0.306	0.695	1.25	1.60	1.72	2.50	205	0.20
5	0.245	0.560	1.00	1.29	1.40	2.00	165	0.15
6	0.171	0.390	0.70	0.90	0.98	1.40	100	0.09
14	0.122	0,278	0.50	0.64	0.70	1.00	100	0.06
Post Yield $\frac{\Delta(\sigma-\sigma_o)}{\Delta\varepsilon} = 0.05 \frac{(\sigma_y-\sigma_o)}{\varepsilon_y}$ below 5 metres								

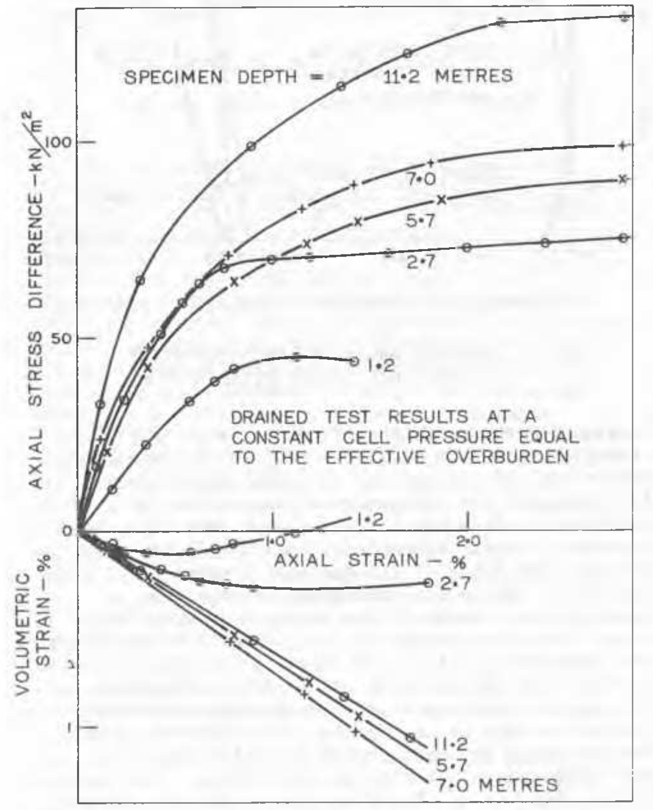

FIG. 3. DRAINED TESTS AT σ_3 = OVERBURDEN PRESSURE

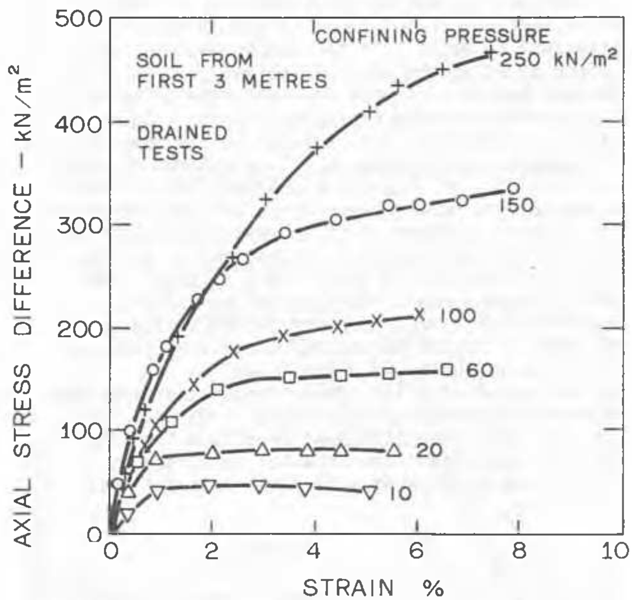

FIG. 4. DRAINED TESTS ON CRUST SOIL

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DRAINED ANALYSIS

The drained soil properties as tabulated in Table 2 were used in a finite multi-linear elastic element programme (Hollingshead and Raymond, 1971). The yield or failure of the elements was based on the vertical stress generated by the embankment loading and was equated equal to the yield stress shown on Fig. 2 less the overburden pressure. Unfortunately no tests were run on the embankment material which was a granular fill. Based on other tests on granular material the drained modulus of elasticity would be of the order of 150 MN/m^2 per metre of depth. Nevertheless analyses were performed using three different values of fill modulus.

(1) 150 MN/m^2 per metre depth of fill
(2) 15 MN/m^2 per metre depth of fill
(3) 0 MN/m^2 per metre depth of fill

An additional prediction was performed assuming the vertical stress increase equal to the pore pressure obtained from the undrained analysis and coefficient A equal to, first, 1.0 and, second, 0.5. A straight one-dimensional type settlement analysis was then calculated similar to that suggested by Skempton and Bjerrum (1957). This resulted in much larger settlements away from the centreline than would be obtained from the use of the increase in vertical stress only since outside the loaded area the major principal stress is close to horizontal.

LONG TERM MOVEMENTS

The drained or long term movements were calculated using the assumptions outlined in the last section. Figure 5 shows the various predicted settlements under the centreline of the embankment. Figure 5 also gives the observed movements to Sept. 1971 and the projected movements at the projected zero pore water pressure assuming the present rate of dissipation is maintained. The projected settlements below the centreline are closest to the predicted results assuming the embankment to have no stiffness and the foundation acts as a continuum. The Skempton and Bjerrum (1957) approach using pore pressure coefficient $A = 0.5$ and the stress distribution from an undrained analysis is the next closest result but tends to give greater settlements at depth. If the common engineering approach of using the vertical stress from an undrained analysis is used the over prediction on the centreline becomes exceedingly great. Such an analysis is obtained below the centreline when the pore pressure coefficient $A = 1$. At first it might be assumed that the continuum approach should lead to greater settlements since there will be increased shear strains, however, this assumption ignores the stress redistribution which theoretically, and in the author's opinion in actual fact, occurs. The effect of stress redistribution can be further demonstrated by calculating the one-dimensional settlement of a soil having the properties given by Table 2 and subjected to a load equal to the vertical height of fill. Under these conditions the estimated settlement would be about 1701 mm. This is greater than the settlement predicted from the oedometer tests since the triaxial test results include shear settlement within the triaxial test.

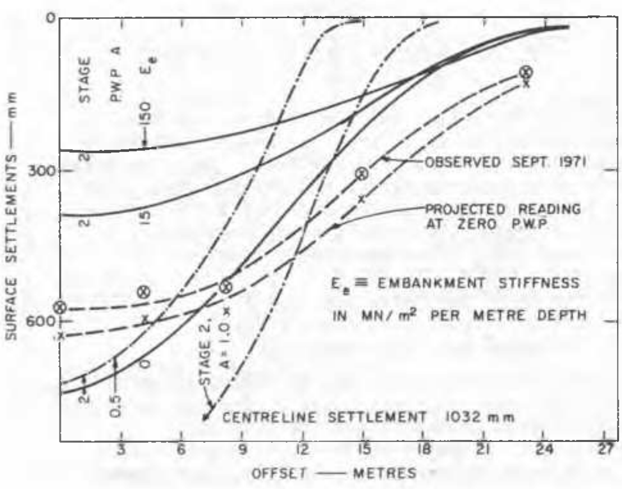

FIG. 5. TOTAL SURFACE SETTLEMENTS

The theoretical stress redistribution is illustrated for Stage 1 in Fig. 6 where a few of the resulting stresses below the centreline are shown. Similar trends occur for the Stage 2 analysis. Considerably smaller vertical stresses result from a drained analysis than from the undrained analysis resulting in smaller stress magnitudes above the preconsolidation or yield stress and thus less total settlements. In addition there are exceedingly large differences in the resulting theoretical lateral stresses.

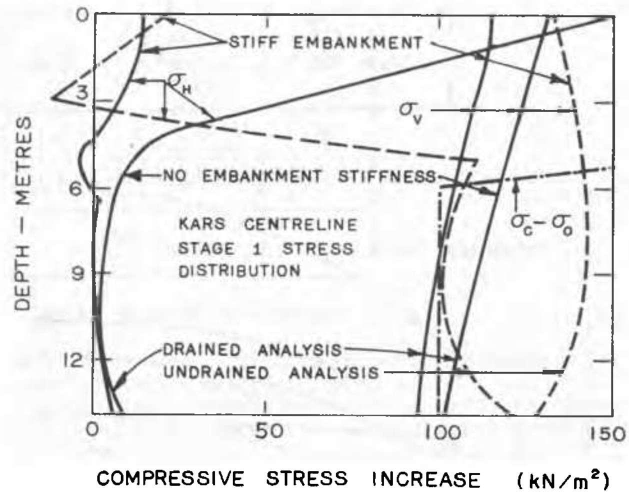

FIG. 6. DRAINED AND UNDRAINED CENTRELINE THEORETICAL STRESS DISTRIBUTION

Considering the surface settlements shown in Fig. 7 it may be seen that similar trends exist between the centreline and the start of the embankment slope. As the toe of the embankment is approached the predicted surface settlements become less than those observed. In the theoretical analysis it was assumed that the soil had constant properties at any one depth. Below the embankment it might be expected that there will be larger increases in all round stress than below the toe. From the results of the laboratory tests on the crust soil (Fig. 4) it is obvious that the strength and stiffness resulting from the stress changes will be greater below the centreline than below the toe. The selected values for the crust at least should vary with the horizontal dimension from the centreline. The major problem however is the selection and interpretation of suitable laboratory or in situ tests to obtain the soil's properties.

The lateral movements predicted by the continuum approach were small with a maximum value of 7 mm. movement toward the centre at the surface. The smallness of the movements agrees with Eden and Poorooshasb's (1968) observation that the lateral movements were of a small magnitude and definitely less than 13 mm. Obviously the one-dimensional method cannot be used to estimate lateral movements.

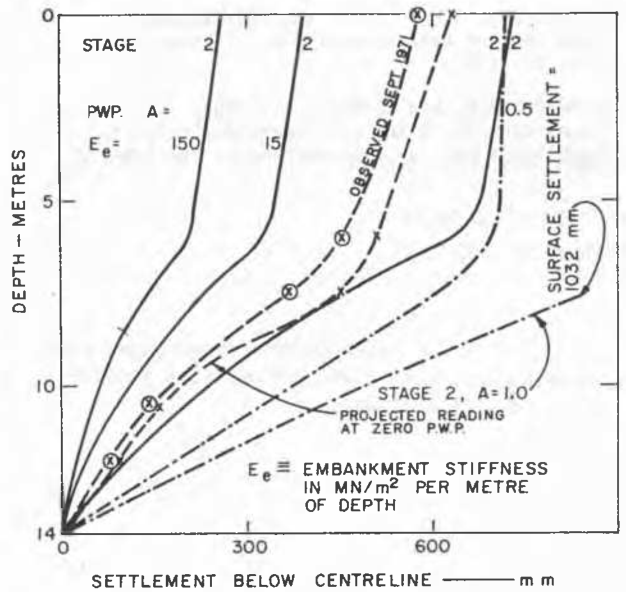

FIG. 7. TOTAL CENTRELINE SETTLEMENTS

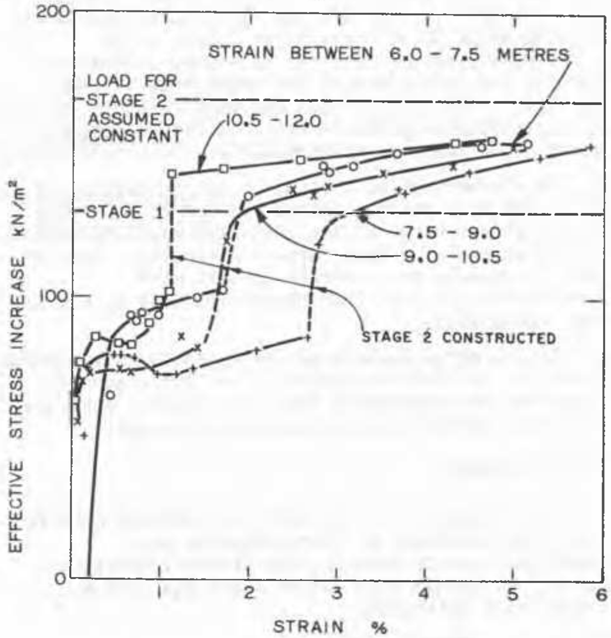

FIG. 8. CENTRELINE STRESS-STRAIN RELATIONSHIP

In the case record discussed the embankment was constructed in two stages. In interpreting the field results in terms of the Skempton and Bjerrum (1957) method of analysis several questions of interpretation of analysis arise. The most obvious is that of a settlement analysis using $A \neq 1$ in which case there is an instantaneous change in the effective vertical stress on application of the load which is disregarded. Figure 8 shows a plot of the observed strains calculated from the different in centreline auger movements against the estimated effective vertical stress increase obtained by assuming the total vertical stress remains constant and equal to the weight of fill above the centreline. The instantaneous change in vertical stress on application of a load is obvious from Fig. 8 and throws into question the Skempton and Bjerrum (1957) approach of calculating long term settlements. If the effective stress increases as shown in Fig. 8 then the preconsolidation pressure is about 75 kN/m^2 greater than the overburden pressure rather than 100 kN/m^2 as assumed from Fig. 2. However the instantaneous increase during Stage 2 construction would raise the preconsolidation pressures to values greater than 100 kN/m^2 above the overburden pressure if the stress-strain observations after Stage 2 was built are projected back to zero strain. Obviously the stress redistribution which occurs if the results shown in Fig. 6 are meaningful, must be a partical explanation.

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Further supporting evidence of the stress redistribution effect is obtained by plotting the compression (or strain) occurring below the centreline at different depths against time. This is shown in Fig. 9 and it may be seen that the compression has occurred almost independently of the pore pressures observed. Interestingly the effects of the Stage 2 construction are indistinguishable in the results shown in Fig. 9.

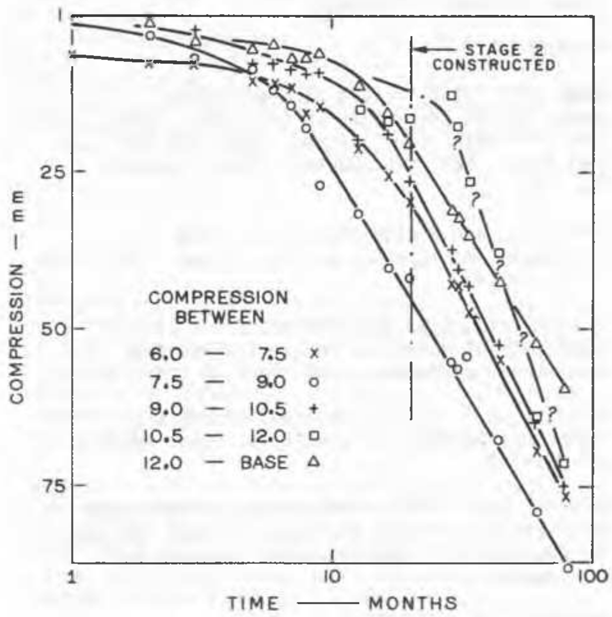

FIG. 9. CENTRELINE RATES OF COMPRESSION

CONCLUSIONS

1. The drained settlement performance and drained soil properties of a Leda clay foundation soil are presented. The drained soil properties were related

to the yield stress as defined by Mitchell (1970) and selected from the drained laboratory results by running tests at cell pressures equal to the overburden pressure plus the estimated increase in lateral pressure given by the centreline drained analysis. The yield stress was equated equal to the preconsolidation pressure obtained from oedometer tests minus the overburden pressure.

2. The drained analysis assuming the foundation to be a continuum and the embankment had no stiffness resulted in centreline settlements closest to the projected end of primary observed results. Outside the loaded area the observed results were considerably greater than those predicted by any of the used analyses.

3. Stress redistribution was believed to have a major effect on the observed results. If the arguments presented are reasonable then considerable evidence of stress redistribution has been presented.

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