End of Construction Failure of a Slope in a Stiff Boulder Clay

Fracture à la Fin de Construction d'un Talus dans une Argile à Blocaux Rigide

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SYNOPSIS There are very few recorded case histories of end of construction failures of slopes in firm to hard boulder clays. Such a failure took place during the excavation of a temporary 60° cutting in a firm to hard boulder clay at Glenhove Reservoir. An immediate investigation of the slip revealed the presence of an inclined thin layer of silty clay some 1 m. from the base of the excavation. The interfaces between the silty clay and the boulder clay formed smooth pre-existing failure surfaces along which a wedge-type failure took place when the lateral support was removed by construction requirements. The equilibrium of the slope has been analysed in terms of total, effective and residual stresses mobilised by several modes of failure. The results of these analyses indicate that the strength of the silty clay rather than the boulder clay controls stability. In particular the assumption of the most likely failure mechanism would indicate that residual strength controls stability.

INTRODUCTION

The design of slopes of temporary excavations in stiff to hard intact clays can be made in terms of effective stresses where the shear strength of the soil can be expressed generally as $s = c' + (\sigma - u) \tan \emptyset'$. The shear strength parameters c' and $\emptyset'$ can be measured by consolidated undrained tests with porewater pressure measurements or by drained tests. In analysis the problem arises as to what the insitu porewater pressure is at the end of construction and how quickly it dissipates relative to the short life of the temporary excavation (Bishop & Bjerrum 1960). Measurement of the pore water pressures developed during excavation for small construction contracts are ruled out on the grounds of cost. They can be estimated (Skempton 1954) knowing the initial and final states of stress. The accuracy of the estimate depends on the value of the coefficient of earth pressure at rest of which there is a lack of factual knowledge and a wide scatter of estimated values in the literature. (Skempton 1961, Duncan and Dunlop 1969, Windle & Wroth 1977). For intact clays of low permeability the problem is circumvented by analysing the slopes in terms of total stresses assuming no drainage or swelling takes place during the short life of the excavation and using carefully selected values of the undrained shear strength (Chowdhury 1978). Where a failure takes place on an existing slip surface formed for example by tectonic shearing, the shear strength on the slip surface is equal to the residual value (Bjerrum 1967, Skempton 1970). Back analysis of slopes which have failed provide an excellent means of checking the validity of such currently accepted theoretical methods of analysis. Such a failure occurred during the digging of a temporary excavation in the stiff to hard boulder clay at Glenhove Reservoir. As the failure took place during excavation it is categorized as an end of construction or short term failure which would be analysed in terms of total stresses and the undrained shear strength of the intact boulder clay. The slope of the excavation as designed at 60° had an adequate factor of safety. A detailed investigation revealed that failure had taken place along the surface of a smooth pre-existing shear plane.

ENGINEERING GEOLOGY

The site is located at Glenhove near the new town of Cumbernauld, Scotland. Published geological information shows the site to be underlain by rocks of the Carboniferous Period, comprising, in ascending order, the Carboniferous Limestone Series, Millstone Grit and Coal Measures. All three rock groups are overlain by a variable thickness of glacial boulder clay. The site investigation showed between 2 m. and 3 m. of firm to stiff brown and yellow sandy gravelly clay grading into stiff dark boulder clay containing boulders up to 0.5 m. to 1 m. in diameter. At the northern end of the site bedrock was found at 4.25 m. below ground level, but towards the south the boulder clay became progressively deeper, a depth of 12 m. being recorded in the site investigation. The ground level rises some 7.5 m. from north to south.

Beneath the boulder clay are dark shales and sandstones for depths up to 2 m. From the available information it is difficult to ascertain to which group of the Carboniferous rocks they should be assigned as the minor lithology of the three groups is similar. The Airdrie One Inch Sheet 31 published by the Geological Survey of Scotland shown a fault crossing the site in an E - W direction. This fault, downthrown to the south by about 150 m., juxtaposes the Carboniferous Limestone Series to the north against both Millstone Grit and Coal Measures in the south. The reason for this is that the junction of the Millstone Grit with the overlying Coal Measures is estimated to run in an approximate N - S direction across the site. Dips across the site would appear to be quite gentle and in a direction slightly south of east.

EXCAVATION AND FAILURE

Prior to the commencement of construction the slopes of the temporary excavation were designed using the $\phi = 0$ method of analysis and the undrained shear strength of the boulder clay. A side slope of 60° to the horizontal (Approx. 2:3) was recommended. It was also recommended that surface water running down the natural slope should be prevented from entering the slope by cutting an interceptor drain at and parallel to the top of the excavation. In addition excavated material was to be placed at least 7.5 m. from the edge of the top of the excavation. A section through the as dug excavation is given in Fig. 2. Excavation commenced on 17.4.69 and was carried out in two stages, the first reducing site level to 126.5 m. and the second to 121.0 m. The excavation of the south slope was completed in the period from 23.6.69 to 6.7.69. On the 7.7.69 cracks were observed in the face of the excavation and a small failure recurred overnight between 8.7.69 and 9.7.69. A photograph of the failure taken on 10.7.69 is shown in Fig. 1.

Fig. 1. INITIAL FAILURE

A section through the slope immediately prior to failure is given in Fig. 2. In this figure a layer of silty clay dipping from south to north at 4° is assumed. In subsequent weeks a continuous regressive type failure took place, accompanied with the formation of large tension cracks at the top of the slope. A physical examination of the failure revealed a thin layer of silty clay some 1 m. above the base of the excavation. This layer had a maximum thickness of 260 mm. and had smooth surfaces where it made contact with the boulder clay. These pre-existing failure surfaces sloped downwards from south to north at an angle varying from 2½° to 5°. Failure took the form of a triangular wedge of boulder clay sliding along the smooth surface of the silty clay when the lateral support was removed by construction operations. In places the thin layer of silty clay had been squeezed out by the unstable overlying boulder clay moving over the stable boulder clay in the base of the excavation. Detailed photographs of the failure surface are given in Fig. 3 and 4. A close up photograph of the silty clay and its surface is shown in Fig. 5.

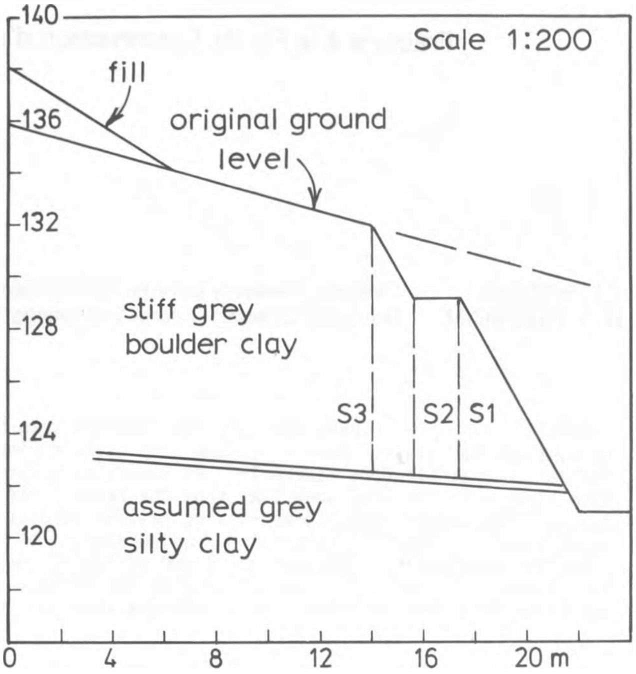


Fig. 2 SECTION THROUGH FAILURE

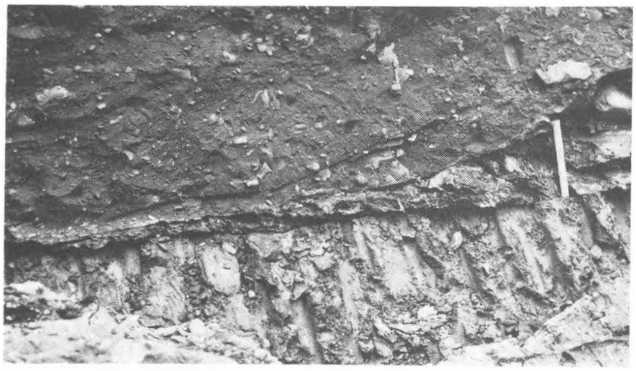


Fig. 3. FAILURE SURFACES

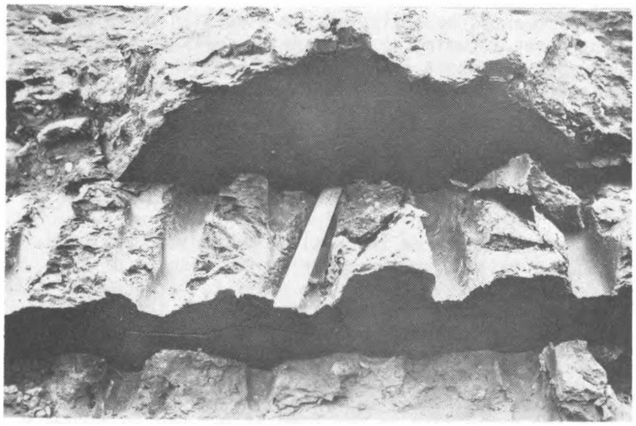


Fig. 4. FAILURE SURFACES.

Fig. 5. SILTY CLAY FAILURE SURFACE

LABORATORY TESTS

On the day after the failure undisturbed block samples of the boulder clay and the silty clay were taken along with samples containing the actual shear surface. Classification tests were carried out on disturbed samples. Triaxial and shear box tests were carried out on hand trimmed samples to determine the shear strength parameters in terms of total, effective and residual stresses. Shear box tests were carried out on the actual shear surface to determine the residual shear strength of the silty clay. The results of these tests are given in Table 1. Particle size distribution curves were determined for some of the 38 mm diameter triaxial specimens and the results are shown in Fig. 6.

Soil	W_L	W_p	W	γ	C_u	C'	ϕ'	C'_r	ϕ'_r
	%	%	%	kNm^{-3}	kPa	kPa	o	kPa	o
Silty Clay	56	22	18	20.3	120	22	18	0	15
Boulder Clay	31	14	10	21.8	385	29	25	-	-

TABLE 1. SOIL PROPERTIES

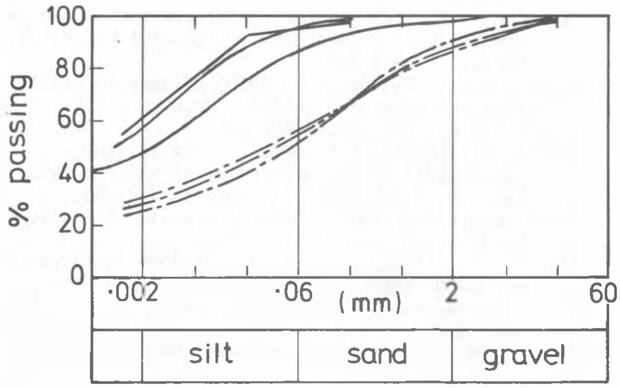
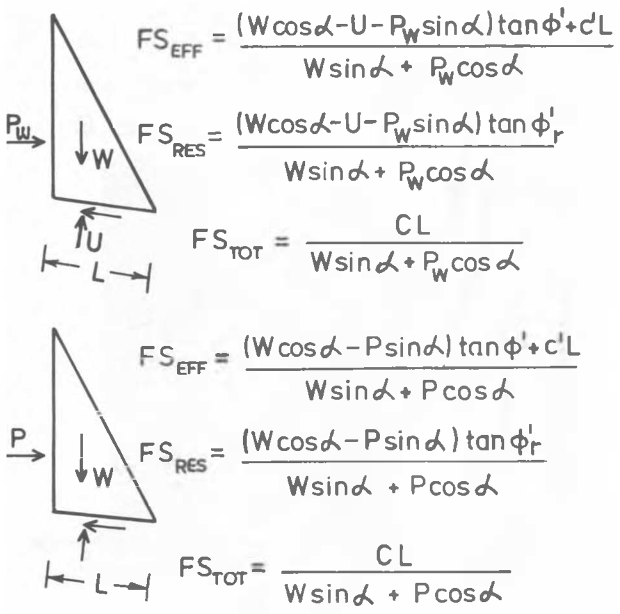


Fig. 6. GRADING CURVES SILTY CLAY (——) AND BOULDER CLAY (- —)

STABILITY ANALYSIS

Three sections S1, S2, S3 as shown in Fig. 1 were analysed for angles of slope of the silty clay layer of 3, 4 and 5 degrees. Two failure models were analysed. In the first a vertical tension crack extending from the surface to the layer of silty clay was assumed containing water varying in depth from 0 to the full depth of the crack. The factors of safety of the slope in terms of total, effective and residual stresses using the shear strength parameters of the silty clay as given in Table 1 and the methods of analysis as given in Fig. 7. were determined. Section S3 was found to give the lowest factors of safety for the analyses in terms of total and effective stresses. As the factors of safety are not too sensitive to the small variations of one degree only those for section S3 and a slope of 4° are given in Table 11 and Fig. 8. Section S1 was found to give the lowest factors of safety in terms of residual strength and these are given in Table 11 and Fig. 8. The second failure model consisted of a simple earth pressure model with zero pore water pressures. The factors of safety were obtained as before for various values of the coefficient of active earth pressure varying from 0.05 to 1.5, and the results are given in Table 11 and Fig. 9.



$$FS_{EFF} = \frac{(W\cos\alpha - U - P_W\sin\alpha)\tan\phi' + c'L}{W\sin\alpha + P_W\cos\alpha}$$

$$FS_{RES} = \frac{(W\cos\alpha - U - P_W\sin\alpha)\tan\phi'_r}{W\sin\alpha + P_W\cos\alpha}$$

$$FS_{TOT} = \frac{CL}{W\sin\alpha + P_W\cos\alpha}$$

$$FS_{EFF} = \frac{(W\cos\alpha - P\sin\alpha)\tan\phi' + c'L}{W\sin\alpha + P\cos\alpha}$$

$$FS_{RES} = \frac{(W\cos\alpha - P\sin\alpha)\tan\phi'_r}{W\sin\alpha + P\cos\alpha}$$

$$FS_{TOT} = \frac{CL}{W\sin\alpha + P\cos\alpha}$$

Fig. 7 METHODS OF ANALYSIS

DISCUSSION

The results of the analysis of both failure models in terms of total and effective stresses indicate that section S3 is the most critical, whereas the analysis in terms of residual stresses indicate that section S1 is the most critical. The actual initial failure on site as can be seen in Fig. 1 took the form depicted by section S1 and not that of S3. This means that the analyses in terms of residual stresses conforms to the real mode of failure whereas those in terms of total and effective strengths do not.

Model	Section S3 Ht. of Water m.	F.S. Tot.	F.S. Eff.	Section S1 HT. of Water m.	F.S. Res.
Tension Crack Plus Water	0.0	18	8	0.0	4
	2.3	12	5	1.8	2
	4.7	6	2	3.5	0.8
	7.0	3	1	5.3	0.3
	9.3	2	0.6	7.0	0.2

Earth Pressure No Water	Coeff K	S3 F.S. Tot.	F.S. Eff.	S1 F.S. Res.
	0.05	9	4	
	0.15	5	2	1
	0.25	3	1.4	0.5
	0.50	1.7	0.7	0.3
	1.0	0.9	0.4	0.1
	1.5	0.6	0.2	0.1

Table 11 FACTORS OF SAFETY FOR SHEAR PLANE SLOPE OF 4°

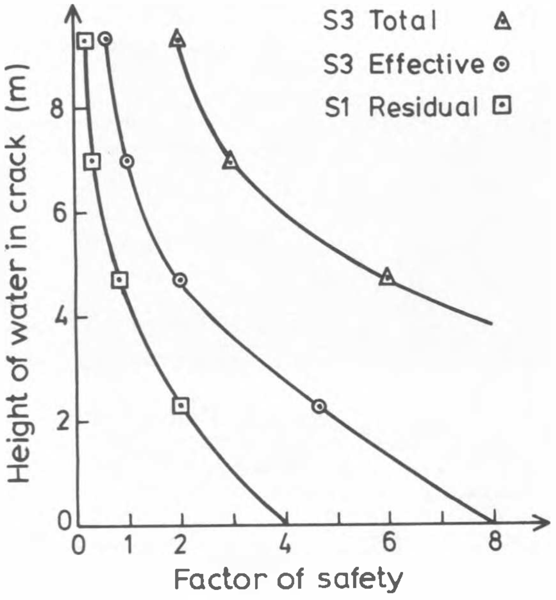


Fig. 8 FACTOR OF SAFETY VERSUS HEIGHT OF WATER IN TENSION CRACK.

In addition the factors of safety in terms of total and effective strengths with the exception of those corresponding to unrealistic depths of water in the tension crack and high coefficients of earth pressure, are greater than unity. The factors of safety in terms of residual strengths are mostly less than unity. A coefficient of active earth pressure of 0.15 (or 4.2 m. of water in a tension crack) give factors of safety of unity in terms of residual strengths.

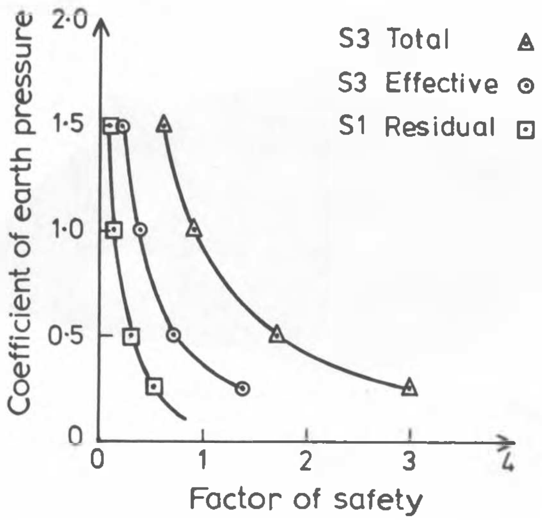


Fig. 9. FACTOR OF SAFETY VERSUS COEFFICIENT OF EARTH PRESSURE.

CONCLUSION

A realistic analysis of the failure indicates that the residual shear strength as mobilised on the pre-existing smooth surfaces of the silty clay controls the stability of the slope.

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