A Study of a Breach in an Earthen Embankment Caused by Uplift Pressures

Étude d'une brèche causée par des sous-pressions dans un talus de terre

by Arthur MARSLAND, M.Sc. (Building Research Station, Watford, Herts. England)

Summary

Investigations made at the site of a large breach in a river embankment which occured under flood conditions are described. The significant features of the breach were the rapidity with which it developed and the massive nature of the debris washed out into the marsh. Preliminary soil surveys indicated that the bank may have failed as a result of instability caused by uplift pressures in an underlying sandy gravel layer.

After the breach had been repaired the variations of the pore pressure in this gravel stratum were measured at intervals over a period of about two years so as to cover as wide a range of tidal and ground-water conditions as possible. These results were extrapolated to give an estimate of the pore pressures just prior to collapse of the bank. Stability analysis using these pore pressures helped to confirm the suggestion that the breach was caused by high water pressures in the underlying gravel.

Sommaire

L'objet de ce rapport est de rendre compte des résultats des études faites à l'emplacement d'une brèche survenue dans le talus d'une rivière lors d'une inondation. Les caractéristiques de cette rupture sont la rapidité avec laquelle elle s'est développée et la nature massive des débris alluviaux déposés dans le marais. Un examen préliminaire du sol a montré que la rupture du talus pouvait être attribuée à l'instabilité causée par des sous-pressions dans les couches sous-jacentes de gravier sableux. Après réparation de la brèche, on a mesuré les variations de pression interstitielle dans cette couche de gravier pour une période de deux ans afin de couvrir une gamme aussi étendue que possible de conditions de marées et de niveau de la nappe. On a extrapolé ces résultats pour déterminer les pressions interstitielles avant la rupture du talus. L'analyse de stabilité basée sur ces pressions interstitielles a contribué à confirmer l'idée que la brèche avait été causée par de hautes pressions de l'eau dans le gravier sous-jacent.

Introduction

On the night of January 31 - February 1, 1953 extensive breaching occurred in the sea and river defences of Eastern England. One of the worst breaches in the Thames Valley occurred on the Eastern bank of the River Darent downstream of Dartford Lock on the Northern outskirts of Dartford, Kent (see CROWTHER, 1953). The rapid formation of the breach and the presence of a layer of gravel at a shallow depth suggested that failure may been due to uplift pressures which had developed in the gravel layer at high water. This paper describes the investigation carried out to study the cause of the failure.

Description of the failure

The first warning of danger was a report by the Keeper at Dartford Lock that water was coming over the bank at many points. Shortly after this the bank suddenly gave way and the marsch was rapidly flooded to a depth of 5-6 ft. By the next morning the breach was 90 ft. wide and 6-8 ft. below marsh level and the gravel was exposed at the sides and bottom of the breach. The most unusual feature of this particular failure was the fact that large lumps of clayey peat with a maximum size of 6 ft. cube were carried as far as a 1,000 ft. into the marsh (see Fig. 1). Away from the breach only superficial erosion of the weak top soil occurred, and the bank was completely undamaged at many of the lower spots.

Site investigations and soil properties

In the locality of the breach site the Darent Valley is covered with a thin layer of alluvium composed mainly of peat overlying a stratum of gravel about 10 ft. thick, which rests upon chalk. The gravel is exposed on low hills about 500 ft. to the West and 2,000 ft. to the East of the river as shown in Fig. 2. In the area of the breach the river channel is very close to the Western side of its flood plain and in consequence the gravel is likely to be nearer the surface here than elsewhere.

To ascertain the sequence of the soil stratum in more detail borings were made in the area of the breach and at intervals along the East bank of the Darent as far as the River Thames. The levels of the ground surface and the top of the sandy gravel are given against each bore hole in Figs. 2 and 3. It will be seen that the surface level of the marsh was lowest near the breach site (+ 2·5 to + 3·5 O.D.N.) and that the top surface of the gravel stratum falls from about -3·0 O.D.N. at the breach site to -20 O.D.N. at the mouth of the Darent. The thickness of peat and peaty clay above the gravel was thus only about 6 ft. at the breach site and gradually increased on both sides.

A section through the bank and marsh layers adjacent to the breach is shown in Fig. 4 together with values of density, moisture content and the results of unconfined compression tests made on undisturbed samples taken from the bank during normal tide conditions. Spot levels taken at intervals of 100 ft. along the top of the bank varied between 15·0 and 16·5 O.D.N. while at either side of the breach levels of 15·5 O.D.N. were recorded. The bank was composed of a firm clay and shrinkage cracking was limited to the top 12 in.

In order to make an estimate of the stability of the bank under flood conditions it was necessary to have information on the effective stress properties of the soil types present. Of particular interest were the soft organic clays, peats and the silty sand which occurred as a transition layer 2-3 in. thick between the peat and the sandy gravel. Equilibrium shear box tests on undisturbed samples of the silty sand gave a Φd of only 30° as compared with an assumed value of at least 40° for the underlying well graded Thames ballast. (See BISHOP, 1948). Except under abnormal water pressure

Fig. 1 Photograph of outwash debris on marsh at rear of breach.
Photo des alluvions et de débris sur le marais, derrière la brèche.

conditions the weakest soil at this site is the normally-consolidated organic clay which is sandwiched between the layers of peat. Shear box tests on this clay gave a $\Phi d = 24°$ and $\Phi\ cu = 19°$, while undrained triaxials on 1 ½ in. diam. samples gave effective stress parameters $\Phi' = 22°$ and $c' = 0{\cdot}9$ lbs./sq. in. Remoulding reduced the shear strength to about a quarter of its original value.

The lightly consolidated peat had a natural moisture content of 250-450 per cent. Most of the undisturbed 1 ½ in. diam. samples were consolidated in a triaxial cell under an all round pressure for 1-2 days after which they were strained at a constant rate under either "drained" or undrained conditions. Pore pressures developed during shear were measured in all the undrained triaxial tests. The maximum deviator stress $(\sigma_1-\sigma_3)$ occurred at between 10-15 per cent strain after about 25 minutes while in the drained tests it occurred at about 15-20 per cent strain in 4 hours. Slow shear box tests in which failure was reached in about 10 hours were also made on undisturbed samples of peat previously consolidated for 3 days. The average values of c' and Φ' obtained from all these triaxial and shear box tests on peat after applying all necessary sheath and drain corrections, are given below.

Drained triaxial tests $\Phi' = 21°\text{-}24°$ $c' = 0 - 0{\cdot}8$ lbs./sq. inch.

Drained shear box tests $\Phi' = 33°$

Undrained triaxial tests $\Phi' = 24°$ $c' = 2{\cdot}0$ lbs./sq. in.

The values of c' and Φ' given by the various types of test differ considerably. A contributory cause of this variation may be that the time of test used in the drained triaxials was insufficient to allow full drainage to take place. Some of the samples which were left overnight under the maximum deviator stress gave strengths from 6-27 per cent higher when they were strained further. On the other hand another drained triaxial test on a sample consolidated for 6 days and then strained to failure over 3 days gave a Mohrs envelope in agreement with the average values of c' and Φ' given above for the drained triaxials. This sample was drained at the bottom and the pore-pressure was measured at the top and allowance was made for the small residual pore-pressure measured at failure. The high values of Φ' obtained for peat in the slow drained shear box tests may be due to the small thickness of the samples and the fibrous texture of the peat. Positive pore pressure developed during shear in the undrained triaxial tests caused the samples to be overconsolidated at failure and this resulted in values of c' higher than those given by the drained tests.

Measurement of the pore pressure changes caused by the tide

At the time of making the general site investigations in March 1953 temporary apparatus was used to make preliminary measurements of the changes in pore water pressure in the top of the gravel which were produced by the tide (see pt. J, Fig. 5). These measurements indicated that the pore water pressures in the gravel varied with the tide and a more extensive set of piezometers were installed to cover future high tides. Both vertical standpipes using ½ in. diam. polythene tube and twin tube piezometers of the type described by Penman (1956) were used. By the time the piezometers had been installed the bank had been rebuilt and the remains

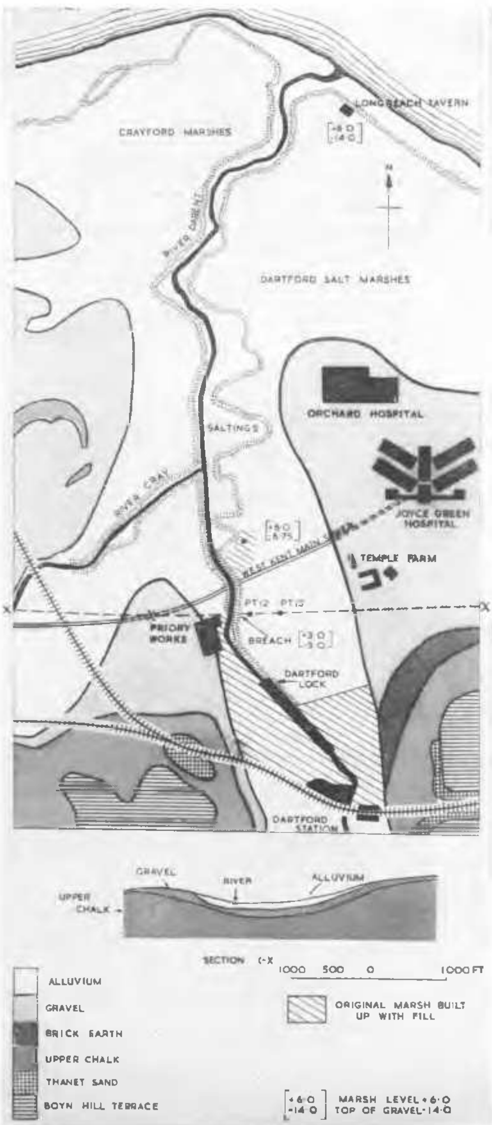


Fig. 2 Geological map and section across valley.
Carte géologique et section de la vallée.

of the scour hole on the river side of the bank had become covered with a thick layer of mud. Before choosing a suitable section for detailed study measurements were first made on a number of piezometers installed at intervals along the bank. These gave substantially the same results which indicated that any local changes in the ground water flow produced by the breaching had been largely restored. The main pore pressure section was chosen on the line of one of the points used in these preliminary measurements.

Detailed measurements were made of the water pressure changes which occurred during tides of various heights throughout the seasons during 1953, 1954 and 1955. The results of typical observations made during this period are plotted in Fig. 5. The changes in the water pressure in the gravel were only of the order of 2-3 ft. of water which represented about 25 per cent of the tidal range. Measurements taken at different distances from the bank (including points 12 and 13 Fig. 2) showed that at high water the pressure decreases with distance from the bank for some way and then starts to increase again as the general water table in the river basin (Fig. 2 (b)) becomes the predominating factor. At low water the flow of ground water is towards the river. In general the results showed that higher initial water pressures in the gravel at low water gave rise to greater water pressures at high tide. The decrease in slope of the plot of water pressure against tide level, as high tide was approached, was also less when the initial ground water pressure was high. There was some indication that the maximum water pressure depended to some extent on the length of time that the tide remained near its peak but this could not be definitely established.

These results can be extrapolated to give an estimate of the pore water pressures which existed just prior to the collapse of the bank during the flood on the 1st February 1953. On that occasion shallow overtopping flooded the marsh behind the bank to a limited extent, which reduced the storage capacity of the marsh soils and may have caused a more rapid increase in the pore pressure than would normally occur.

Stability analyses

Approximate stability analyses of the type shown in Fig. 6 have been made for the bank subjected to different tidal conditions. The shear strengths of the silty sand and peat were estimated from the slow drained shear box tests assuming the pore pressures to be the same as those extrapolated from the measurements made in the top of the sandy gravel. Because of the difficulty of estimating the actual pore pressure changes in the clay within the bank alternative values of strength based on unconfined compression tests or on fully drained shear tests using a steady seepage condition have been used in some of the analyses. It has also been assumed that the full shear strength of all the soils was mobilised simultaneously.

In view of the above remarks and also the difficulty of deciding what tension cracks, if any, to allow, it is obviously not possible to obtain precise figures to represent the stability of the bank. It is, however, considered that the results given below provide a reasonable guide as to the factor of safety of the bank under the various conditions. The factor of safety is defined throughout as the ratio of the measured to the mobilised shear strength.

Assuming that tension cracks did not exist then under low tide conditions the factor of safety against the possibility of a landward slip occurring was estimated to be about 1·5. At higher tide levels the pore pressures in the gravel increase and the bank becomes less stable. Thus, for a water pressure distribution in the gravel 1 ft. lower than that estimated for the 1953 storm surge (extrapolated from the November 1953 curves in Fig. 5), the corresponding factor of safety was about 1·3. With this water pressure distribution in the gravel (line (a), Fig. 6) the weight of the clayey peat above the gravel in the marsh is almost balanced by the water pressure and the ratio of the uplift pressures to the weight of the overlying marsh layer is :

$$\frac{\gamma_w H}{\gamma D} = \frac{62{\cdot}5 \times 6{\cdot}5}{72 \times 6} = 0{\cdot}9$$

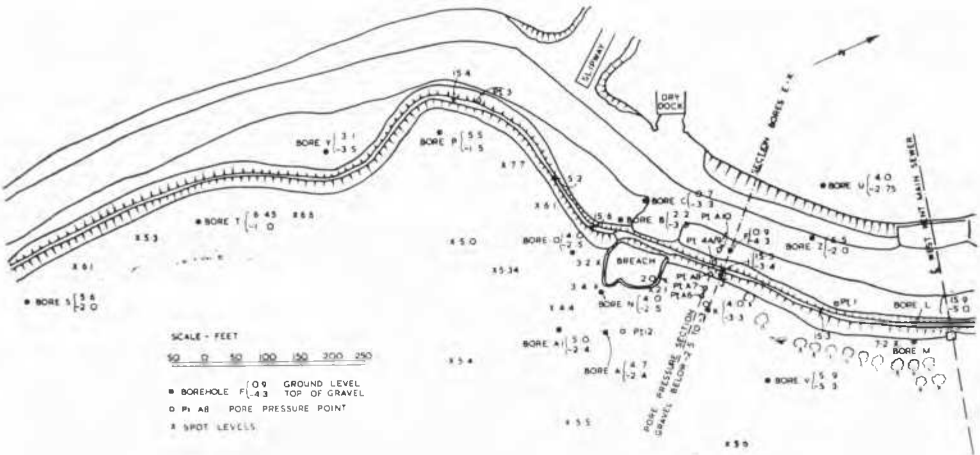


Fig. 3 Position of boreholes and pore pressure points.
Position des forages et des points de pression interstitielle.

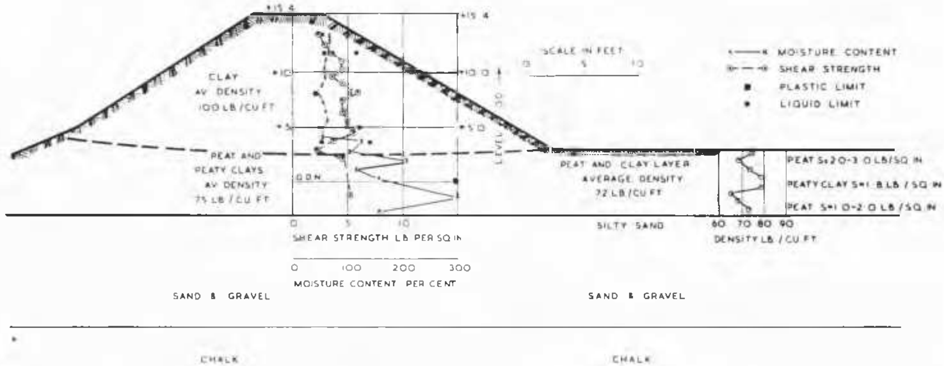


Fig. 4 Section through bank showing soil data.
Section du talus montrant les propriétés du sol.

where γ_w = density of water;

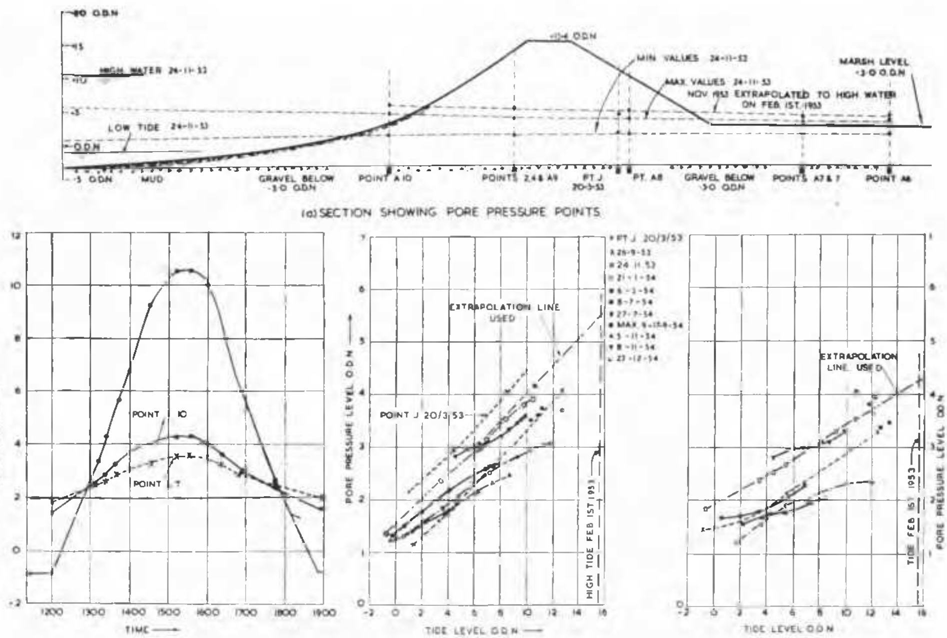
γ = average saturated density of peat and clay layers;

H = head of water under peat (water pressure in the gravel) in ft.;

D = depth of peat and clay above the gravel in ft.

As the tide level rises further the water pressures in the gravel increase until a condition is reached when the marsh at the toe of the bank (to the right of AB in Fig. 6) begins to heave. This means that the passive resistance provided by the marsh at the toe of the bank is reduced to zero and in addition a small uplift force begins to act on the bank across the plane AB. This condition is attained by the pore pressure distribution extrapolated for a 15·5 O.D.N. tide from the results obtained in November 1953 in Fig. 5, and shown as line (b) in Fig. 6, when $\frac{\gamma_w H}{\gamma D} = 1{\cdot}09$. Water pressures slightly greater than were necessary to lift the layer of peat and clay could occur at this site because of the limited extent of the low area.

Typical values of the estimated factor of safety for a tide level at the top of the bank using various assumptions are



(b) TYPICAL TIDE AND PRESSURE RESULTS 24-11-53

(c) POREWATER PRESSURE MEASURED AT POINT A9 PLOTTED AGAINST TIDE LEVEL OVER VARIOUS TIDES.

(d) POREWATER PRESSURE MEASURED AT POINT A7 PLOTTED AGAINST TIDE LEVEL OVER VARIOUS TIDES.

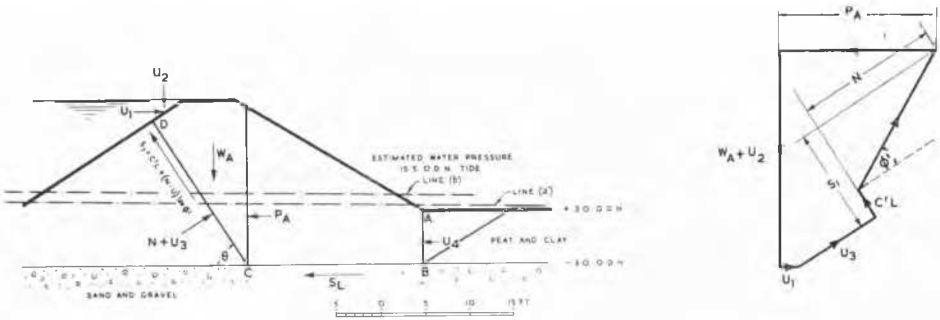
Fig. 5 Pore pressure measurements.
Mesures des pressions interstitielles.

given in the table. In calculating these values of F no allowance has been made for tension cracks. For this reason the factor of safety (F) may be about 20 per cent too high where a $\Phi = 0$ shear strength condition has been assumed for the bank material. In the analyses using $\Phi' = 24°$ for the clay in the bank the effect of including a tension crack was small because of the low normal effective stress across the upper part of the shear plane. It will thus be seen that the factor of safety for overall stability under the extreme flood conditions prior to the formation of the breach was near unity.

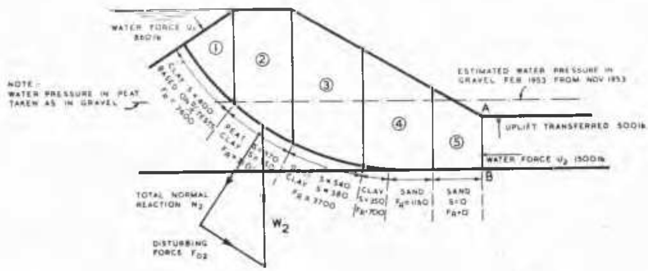
Typical values of estimated factor of safety under flood conditions

Type of analysis	*Water pressures used*	*Strengths used*			*Estimated F*
		Foundation clay	*Peat*	*Bank * clay*	
Circle	Water level in clay and peat taken as same as that in underlying gravel	$\Phi' = 24°$	$\Phi' = 33°$	$\Phi' = 24°$	1·0
		$\Phi' = 24°$	$\Phi' = 33°$, $c' = 2$ lbs./in.^{2}	$\Phi' = 24°$	1·0
		$\Phi' = 24°$	$\Phi' = 33°$	$\Phi = 33°$, $s = 400$ lbs./ft.^{2}	1·25
Wedge	Steady seepage in bank	$\Phi' = 24°$	$\Phi' = 33°$	$\Phi' = 24°$	0·7
	Peat, silty sand and foundation clay same as in underlying gravel	$\Phi' = 22°$, $c = 0·9$ lbs./in.^{2}	$\Phi' = 33°$	$\Phi' = 22°$, $c' = 0·9$ lbs./in.^{2}	0·9
		$\Phi = 24°$	$\Phi_d = 33°$	$\Phi = 0$, $s = 400$ lbs./ft.^{2}	1·1

* Clay in part of bank through which shear plane passes is wetted every tide and the strength is much lower than in centre of bank.



(a) WEDGE ANALYSIS – FACTOR OF SAFETY (F) NEGLECTING PASSIVE RESISTANCE OF TOE = $\frac{S_1 \sec\theta + S_L}{(W_A+U_2)\tan\theta + U_1 - U_4}$



(b) CIRCLE ANALYSIS. FACTOR OF SAFETY = $\frac{\Sigma F_R}{\Sigma F_D + U_1 - U_2}$ = 1·25

Fig. 6 Analyses.
Analyses.

Conclusions

The evidence provided in this paper suggests that the rapid development of the large breach which occurred at Dartford during an extraordinarily high tide in 1953 was due to high water pressures in the underlying stratum of sandy gravel. This conclusion is based on the following facts :

(1) This was the only major breach along the River Darent and it occurred at a point where the levels of the ground surface and the gravel layer, together with the general water table profile, gave the worst combination of conditions for this type of failure.

(2) The large size of the outwash debris make it unlikely that failure occurred as a result of overtopping erosion especially since some of the larger lumps were composed of clayey peat from below marsh level. This was substantiated by the fact that even though shallow overtopping occurred over all the banks in the area the surface erosion was only slight because of the relatively firm condition of the clay in the bank.

(3) Stability analyses, in which estimates of the water pressure in the sandy gravel based on field measurements were used, indicated that a failure due to uplift was possible.

This investigation shows the importance of considering the effect of uplift pressures on the overall stability of flood defence banks in all cases where it is known that there are pervious layers with access to the river or sea. Because of the many local factors which may influence ground water flow it is necessary to measure the water pressures in the pervious strata over as wide a range of weather and tide conditions as possible. Stability analyses using this data can then be made to give an approximate idea of the degree of stability likely under flood conditions. It is, however, necessary to take into consideration any factors which may change the pattern of ground water flow as for example an improved connection between the previous layers and the river caused by erosion.

References

[1] Crowther, G. C. (1953). Damage to the Kent Coastline and Restoration Works, *Proc. Conference on North Sea Floods*, Inst. Civil Engrs., Dec. 1953, p. 145.

[2] Bishop, A. W. (1948). A large shear box for testing sands and gravels, *Proc. 2nd Int. Conference of Soil Mechanics*, vol. 1, pp. 207-211.

[3] Penman, A. D. M. (1956). A Field Piezometer Apparatus, *Géotechnique*, vol. VI, No. 2, June 1956, p. 57.

[4] Snell, E. L. (1953). Damage to Essex Coastline and Restoration Works, *Proc. Conference on North Sea Floods*, Inst. Civil Engrs., Dec. 1953, Fig. 42 opposite, p. 165.