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Reinforced Embankments on Very Poor Foundations

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ABSTRACT

Previous research into the behaviour of geotextile-reinforced embankments on peat underlain by a firm base is extended to the case of peat underlain by a layer of very soft clay. The effects of rate of construction and the modulus of reinforcing fabric are examined. The use of geotextile reinforcement in conjunction with berms and lightweight fill is then considered. Finally, suggestions are made with regard to the reinforcement required for a number of situations involving embankments on peat underlain by very soft clay.

1. INTRODUCTION

In a recent publication,[1] the authors have examined the effect of geotextile reinforcement on the stability and deformations of embankments constructed on peat which is underlain by a firm stratum (e.g. medium to dense sand or stiff clay). It is quite common to encounter this stratigraphy; however, in regions which have been subject to 'recent' glaciation, it is also quite common for the depositional history of the deposit to have involved the sedimentation of clay, silts or lake marl prior to the formation of the peat deposit. Because the peat has low weight, the clay, silt or marl in a recent normally consolidated deposit will be very weak just below the peat, although the increase in effective stress with increasing depth below the peat will generally result in a significant increase in strength with depth.

It has generally been observed (e.g. Refs 2–5) that construction and maintenance problems, and shear failure, are far more likely to occur

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when peat is underlain by a weak layer than when it rests on a firm stratum. Thus the objective of this present paper is to investigate the potential effects of geotextile reinforcement upon the stability of embankments constructed on peat which is underlain by a soft clayey layer.

2. FOUNDATION CHARACTERISTICS

Fibrous peats are characterized by a high fibre content and water content and a low ash content. They are highly compressible and permeable, particularly during the early stages of loading. Numerous embankments have been constructed over fibrous peat deposits with varying degrees of success. However, despite this extensive empirical experience there is still no simple, accepted design procedure for determining the stability and the likelihood of excessive shear distortions for embankments constructed on peat. The reason for this is the complexity of the foundation material itself. First, at normal construction rates significant excess pore pressures will be developed. In general those excess pore pressures will be far less than would be expected under undrained conditions. Thus the behaviour of a fibrous peat foundation cannot be categorized as truly drained or undrained. Secondly, the use of the field vane test for determining the shear strength of peat is of doubtful validity. Thirdly, because of the large deformations which occur during construction, the usual assumption of small strains is not applicable for embankments on highly compressible peat deposits.

The soft cohesive layers beneath peat are typically characterized by low to negligible fibre content, a much lower water content than the fibrous peat and a high ash content. At usual construction rates, they may be considered to exhibit undrained behaviour. For cases reported in the literature (see Ref. 6), the vane shear strength in a soft stratum below peat was typically in the range from 5 kPa to 15 kPa, although shear strengths as low as 2·5–3 kPa have been reported (e.g. Ref. 3).

3. METHOD OF ANALYSIS

The results presented in this paper were obtained using the authors' plane strain, non-linear, large strain, elasto-plastic soil structure interaction analysis program (LEPSSIA). Since the method of analysis was the same as that adopted in an earlier investigation reported by Rowe and Soderman,[1] the reader is referred to that publication for details.

The critical time with regard to the stability of embankments on peat corresponds to the end of construction. At this time cohesive soils beneath the peat may be considered to exhibit undrained behaviour; however, the peat will usually have experienced significant partial dissipation of excess pore pressure. As discussed by Rowe and Soderman,[1] the excess pore pressures immediately after construction were calculated from the equation

$$\Delta u = \bar{B}\,\Delta\sigma_1 \tag{1}$$

where Δu is the excess pore pressure at a point, $\Delta\sigma_1$ is the increase in major total principal stress at that point and $\bar{B}$ is an empirical pore pressure parameter. This parameter was assumed to vary with depth and was given by

$$B = (u/u_{\max})\,\bar{B}_{\max} \tag{2}$$

where the variation in $u/u_{\max}$ is given by the limiting curve in Fig. 1.

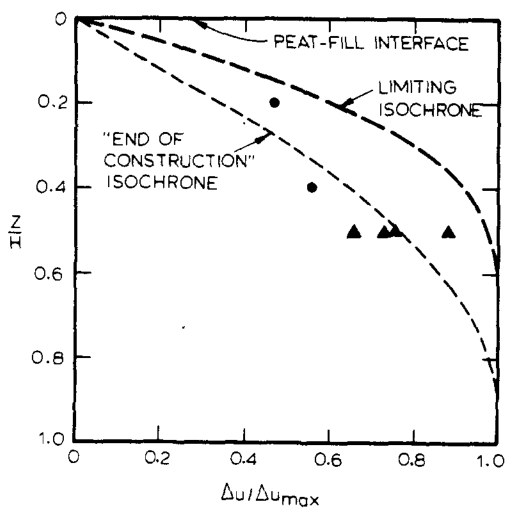

Fig. 1. Variation in excess pore pressure with depth: ●, Highway 509;[5] ▲, Bloomington Road.[9] H = drainage path; Z = depth below peat-fill interface.

Clearly, the maximum excess pore pressure $\bar{B}_{max}$ will depend on the rate of loading and the drainage conditions. The range of values of $\bar{B}_{max}$ which can be deduced from published field cases is typically 0·1–0·35,[6] although higher values have been reported (e.g. Ref. 7) where a relatively low permeability fill (till) was constructed on a shallow peat deposit underlain by relatively low permeability clay.

4. FAILURE AND COLLAPSE

Before discussing the performance of embankments of peat, it should be noted that even though an embankment may be stable there will often be significant local shear failure within the peat and any underlying soft soil. Provided that these zones where the shear strength has been reached are contained or surrounded by soil which is not plastic (i.e. where the shear stress is less than the shear strength), collapse will not occur. However, the deformation of the embankment may be significantly affected by the development of these contained plastic regions. Collapse of the embankment occurs when there is uncontained plastic flow. This may be characterized by the formation of a distinct sliding surface or by a situation where the shear deformations are such that continued addition of fill does not increase the height of the embankment unless some action is taken to stabilize the embankment. It should be emphasized that an embankment may be considered to have failed due to excessive settlement or differential settlement (which is a serviceability assessment) even though collapse, in the sense of uncontained plastic flow, has not occurred. Thus an embankment that has collapsed has failed; but an embankment which is considered to have failed due to poor performance may not have actually collapsed.

The behaviour of a number of reinforced embankments constructed on peat which is underlain by a weak layer has been reported in the literature. For example, MacFarlane and Rutka[2] reported very bad performance (failure) of a number of roads constructed on 4–6 m of peat overlaying 1·8–10 m of clay or silt. Lea and Brawner,[3] Weber[4] and Raymond[5] have reported the collapse of embankments (ranging in height from 0·5 to 2·5 m) constructed on peat (2–6 m thick) which was underlain by soft soil. Collapse in these cases usually involves formation of a crack in the embankment and a block-like slide moving on or just below the boundary between the peat and the soft soil.

5. NUMERICAL DETAILS

The finite element mesh used in these analyses involved between 1472 and 1836 constant strain triangular elements, depending on the depth of the deposit. The lateral boundaries were taken to be smooth and were located between 73 m and 78 m from the centreline of the embankment. The base beneath the soft clayey layer was assumed to be rough rigid, although the shear stresses at the base were not permitted to exceed the shear strength of the soil. The placement of fill was simulated by placing up to nine layers of fill. Up to 200 load steps were used.

6. RESULTS

Analyses were performed for low embankments (i.e. height above original ground surface of 2·5 m or less) with a crest width of 13·4 m and 2:1 side slopes resting on peat deposits with thicknesses of 3 and 5 m, which were underlain by soft clay layers 2 or 3 m thick. The ground water table was assumed at the ground surface. The granular fill was assumed to have the parameters given in Table 1. A number of analyses were also performed for embankments constructed using lightweight fill (sawdust) in conjunction with conventional fill. The properties of the sawdust, given in Table 2, are based on tests conducted on sawdust (see Ref. 6).

The peat parameters adopted (see Table 3) were selected on the basis of the review of the literature and testing described by Rowe,[6] and Rowe *et al.*[8,9] It would be considered unusual for a fibrous peat to have a combination of parameters more critical than those described in Table 3,

TABLE 1
Granular Fill Parameters

c'	0 kPa
ϕ'	32°
ψ	0°
ν'	0·35
γ	21 kN/m^3
(E'/p_a)	$100\sqrt{(\sigma'_3/p_a)}$
p_a	100 kPa

TABLE 2
Lightweight Fill Parameters

c'	0 kPa
ϕ'	32°
ψ	0°
ν'	0·05
γ	10·2 kN/m^3
E'	850 kPa

TABLE 3
Peat Parameters

c'	1·8 kPa
ϕ'	27°
ψ'	0°
ν'	0·15
K'_0	0·176
G_s	1·5
e_0	9
γ	10·3 kN/m^3
E' ($0 \leq \sigma'_v \leq 20$ kPa)	85 kPa
E' ($20 < \sigma'_v \leq 40$ kPa)	110 kPa
E' ($40 < \sigma'_v \leq 60$ kPa)	140 kPa
E' ($\sigma'_v > 60$ kPa)	225 kPa

although this possibility cannot be excluded. (Clearly, if there is any doubt for a particular peat, this doubt can be resolved by performing appropriate tests.)

Analyses were conducted for the clay layer beneath the peat having an undrained shear strength increasing with depth (2·5 kPa to 5 kPa and 2·5 kPa to 7·5 kPa) as well as for homogeneous layers having undrained strengths of 5, 7·5, 10 and 15 kPa. The parameters assumed for the clay are given in Table 4.

For the analyses performed using geotextile reinforcement, it was also assumed that:

(a) the geotextile is located at, or close to, the surface of the peat;
(b) the geotextile–soil interface friction angle exceeds 25°; and
(c) there is no competent root mat.

TABLE 4
Clay Parameters Used When Underlying Peat

c_u	Variable (2·5, 5, 7·5, 10, 15 kPa)
ϕ_u	0°
ψ	0°
ν_u	0·48
K_0'	0·5
G_s	2·7
e_0	2·5
γ	14·6 kN/m^3
E_u	1500 kPa

Key to symbols in Tables 1–4:

c'	cohesion intercept
ϕ'	angle of internal friction
ψ'	dilatancy angle
ν'	Poisson's ratio
γ	unit weight
E	Young's modulus
σ_3' (σ_3)	minor principal effective stress
σ_v'	vertical effective stress
p_a	atmospheric pressure
K_0'	coefficient of earth pressure at rest
G_s	specific gravity
e_0	initial void ratio

Clearly, the presence of a competent root mat will improve embankment performance.

In the following discussion, the height of the embankment (h) is the height above original ground level. The thickness of the fill is the sum of the embankment height and the settlement.

The basic factors influencing the performance of embankments constructed on peat will be illustrated with reference to typical results observed for the case of a 5-m-deep peat deposit underlain by 3 m of very soft clay with an undrained shear strength of 7·5 kPa. Results for other cases are summarized in Tables 5 and 6. Unless otherwise noted, the pore pressure parameter $\bar{B}_{max}$ in the peat will be taken to be 0·34 and a granular fill is assumed. The same general trends were observed for other depths of peat and clay, and for other strengths of the underlying clay.

Figures 2 to 6 show the plastic region and velocity fields obtained for a number of cases considered in this investigation. The 'plastic region'

TABLE 5

Geotextile Modulus Values for Embankments on Fibrous Peat (see Text for Limitations)

Peat thickness (m)	Underlying strata	Strength of underlying strata c_u (kPa)	Maximum height of fill above original ground level (m)			
			1·0	1·5	2·0	2·5
3	2 m Clay	15	NRR[a]	NRR	500 (5%)[b]	1000 (6%)
		10	NRR	NRR	500 (6%)	1000 (6·5%)
3	2 m Clay	7·5	NRR	150 (10%)	500 (8%)	1000 (7%)
		5	NRR	150 (14%)–500 (8%)[c]	2000 (4%)	PF[d]
3	2 m Clay	2·5→5	150 (10%)	2000 (5·5%)	PF	PF
5	3 m Clay	15	NRR	150 (14%)–500 (6·5%)	500 (9·5%)–1000 (5·5%)	1000 (6·5%)–2000 (4%)
		10	150 (5%)	500 (8·5%)–1000 (7·5%)	2000 (5·5%)	PF
5	3m Clay	7·5	500 (5%)	1000 (8·5%)–2000 (5%)	PF	PF

[a] NRR = no reinforcing geotextile required.

[b] 500 (5%) = A geotextile with modulus E_f = 500 kN/m is recommended. Under the assumed conditions a maximum geotextile strain of approximately 5% is anticipated.

[c] 150 (14%)–500 (8%) = For the assumed conditions the embankment could be constructed using E_f = 150 kN/m but the expected strain of 14% is large. If conditions are likely to be as assumed, a higher modulus geotextile is recommended. E_f = 500 kN/m would give 8% strain under the assumed conditions.

[d] PF = potential failure for the assumed conditions: do not construct.

TABLE 6

Geotextile Modulus Values for Embankments Constructed on Fibrous Peat Using Lightweight Fill (see Text for Limitations)

Peat thickness (m)	*Underlying strata*	*Strength of underlying strata* c_u *(kPa)*	*Maximum height of fill above original ground level (m)*			
			1·0	*1·5*	*2·0*	*2·5*
5	3 m Clay	15	NRR[a] LF ≃ 1 m	NRR LF[b] = 1·5 m	150 (10%)–500 (6%)[c] LF = 1·9 m	500 (9%) LF = 2·1 m
5	3 m Clay	7·5	NRR LF ≃ 1 m	150 (10%) LF = 1·5 m	500 (6%) LF = 1·9 m	1000 (9%) LF = 2·5 m
5	3 m Clay	2·5→7·5	500 (5%) LF = 1·2 m	1000 (5%) LF = 1·6 m	2000 (4%) LF = 2·2 m	2000 (6%) LF = 2·5 m

[a] NRR = no reinforcing geotextile required.

[b] LF = 1·5 m ≡ thickness of lightweight fill used = 1·5 m. For any particular application, the thickness of lightweight fill should be selected to be approximately equal to the end of construction settlement.

[c] 150 (10%)–500 (6%) = For the assumed conditions, the embankment could be constructed using E_f = 150 kN/m but the expected strain would be 10%. A higher modulus fabric with E_f = 500 kN/m would give better performance with an expected strain of 6%.

(indicated by the cross-hatching) is defined as the region of the soil (fill, peat or clay) where the shear strength has been reached and irrecoverable plastic straining is occurring. The 'velocity field' indicates direction and relative magnitude of the soil deformations and hence can be used to identify the failure mechanism. In each figure, the deformed position of the interface between the fill and peat, and the peat and clay, has to be shown to indicate the relative magnitudes and distribution of the deformations for each case.

The excess pore pressures developed in the peat can have a significant effect on shear deformations and stability of embankments on peat overlying soft soil. Figure 2(a) and (b) shows the calculated plastic zone and deformations expected if an unreinforced embankment was constructed sufficiently slowly such that no excess pore pressures developed in the peat (i.e. $\bar{B}_{max} = 0$) but undrained conditions still prevail in the clay for embankments 1·5 m and 2 m above original ground level. In both cases, the plastic region is quite extensive; however, it is still 'contained' (i.e. surrounded) by soil which has not failed, and hence despite the large deformations collapse has not occurred.

These results suggest that for the assumed condition (in particular $\bar{B}_{max} = 0$) an embankment could be constructed to 2 m above original ground level. Nevertheless, even under these 'ideal' conditions for the peat, a comparison with results obtained for a similar embankment and peat deposit underlain by a firm base (Fig. 3) indicates that the presence of the very soft clay layer significantly increases the shear deformations and the corresponding settlement. For example, with a 2-m-high embankment the end of construction settlement of 4·55 m given in Fig. 2(b) for peat underlain by clay is much greater than the settlement of 2·6 m obtained when the same peat is underlain by a firm base (Fig. 3(b)).

Recognizing that excess pore pressures will invariably develop in the peat, an analysis was performed for the more typical case where $\bar{B}_{max} = 0{\cdot}34$. Figure 2(c) and (d) shows the plastic region and the corresponding velocity field when the embankment collapses at a height of about 0·9 m. The corresponding collapse height for peat underlain by a firm base was 1·4–1·5 m.

Introducing a geotextile improves the embankment stability and Figure 4(a) shows that for the assumed conditions an embankment could be constructed to 1·5 m if a geotextile with modulus $E_f = 2000$ kN/m were placed at, or near, the interface between the peat and the fill. It should be noted that this figure shows the deformation and plastic region at the end

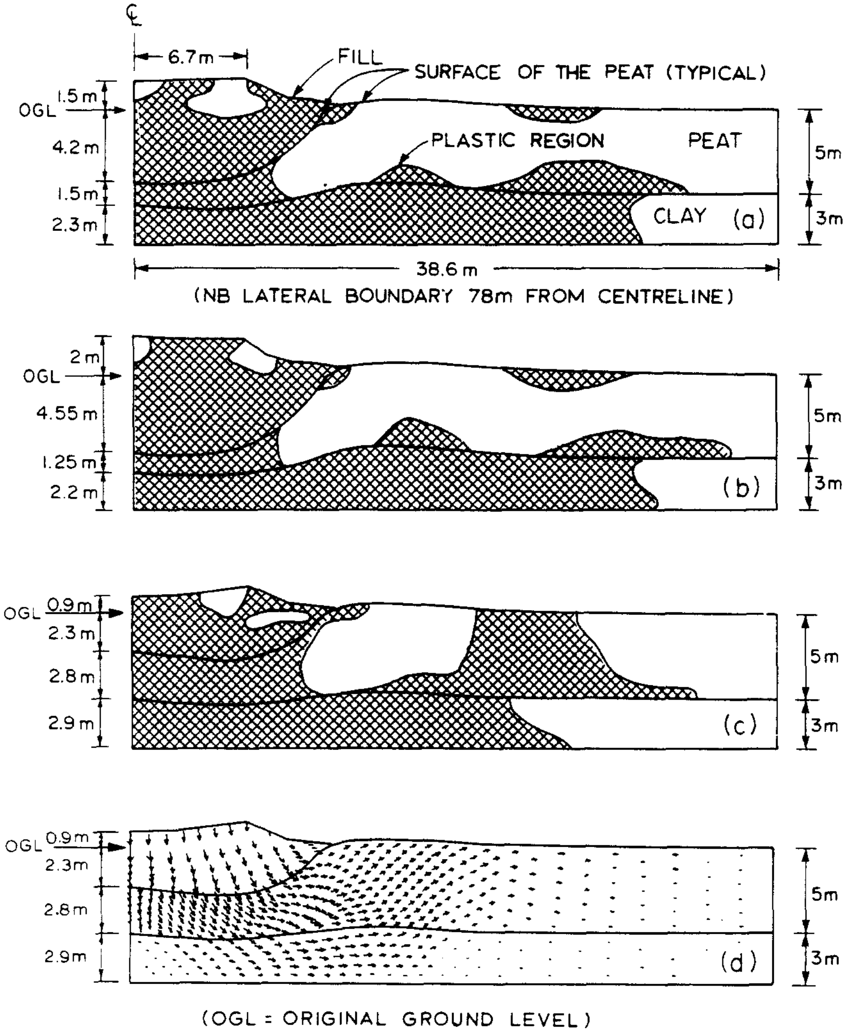

Fig. 2. Unreinforced embankment. Peat underlain by very soft clay: (a) $\bar{B} = 0$, $h = 1{\cdot}5\,\mathrm{m}$; (b) $\bar{B} = 0$, $h = 2{\cdot}0\,\mathrm{m}$; (c) $\bar{B}_{max} = 0{\cdot}34$, $h < 1\mathrm{m}$, plastic region at collapse; (d) $\bar{B}_{max} = 0{\cdot}34$, $h < 1\mathrm{m}$, velocity field at collapse.

of construction and represents the most critical time with regard to stability. However, as express pore pressures dissipate, additional settlement will occur and it will be necessary to add more fill to bring the embankment back to grade.

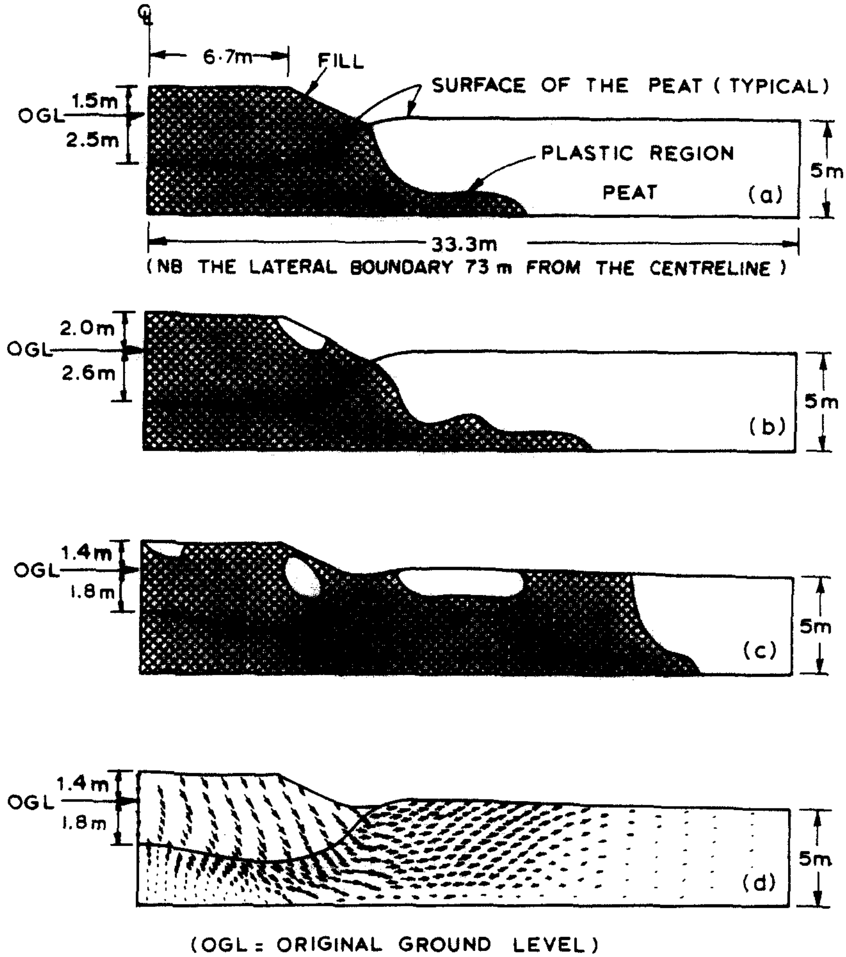

Fig. 3. Unreinforced embankment. Peat underlain by a firm base: (a) $\bar{B} = 0$, $h = 1{\cdot}5$ m; (b) $\bar{B} = 0$, $h = 2{\cdot}0$ m; (c) $\bar{B}_{max} = 0{\cdot}34$, $h \sim 1{\cdot}5$ m, plastic region at collapse; (d) $\bar{B}_{max} \approx 0{\cdot}34$, $h \sim 1{\cdot}5$ m, velocity field at collapse.

There is a limit to how much even a very high modulus fabric can achieve, as illustrated by Figure 4(b) and (c), which shows that at normal construction rates (i.e. $\bar{B}_{max} = 0{\cdot}34$) a 2000 kN/m geotextile would not permit construction of the embankment to a height of 2 m. It is particularly important to note that the stiff geotextile does significantly modify the failure mechanism (compare Figures 2(d) and 4(c)) by almost eliminating lateral deformations of the embankment, and in this case

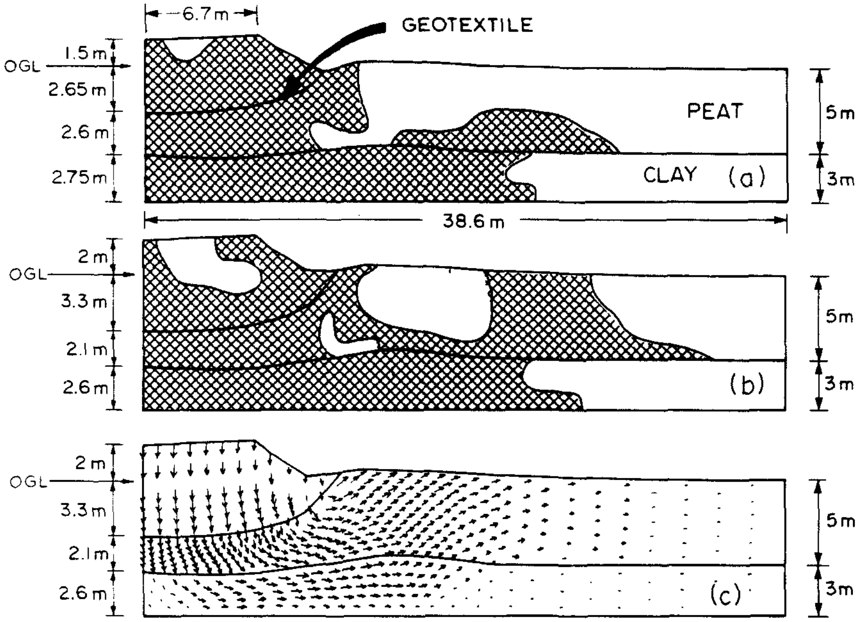

Fig. 4. Geotextile-reinforced embankment. Peat underlain by very soft clay (E_f = 2000 kN/m, $\bar{B}_{max}$ = 0·34): (a) h = 1·5 m; (b) $h \sim 2$ m, plastic region at collapse; (c) $h \sim 2$ m, velocity field at collapse.

collapse is governed by lateral squeeze primarily within the soft clay layer. The performance of this embankment could not be improved by using an even stiffer geotextile and a different design and/or construction procedure would be required to achieve a 2 m grade under the assumed conditions.

Lightweight fill, such as sawdust, may be used to increase embankment stability; however, to avoid problems due to aerobic deterioration of the sawdust, it is essential that the top of the sawdust be at, or below, the water table at the end of the construction and that the surface of the sawdust be below the seasonal minimum height of the water table after dissipation of excess pore pressures and bringing the embankment to grade. Thus the amount of sawdust which can be used is limited by the anticipated settlements. Experience has shown that the use of lightweight fill alone may not be sufficient to ensure stability and that other stabilizing measures such as berms (or geotextiles) may be necessary. This problem is illustrated in Figure 5(a) and (b) which shows that an embankment

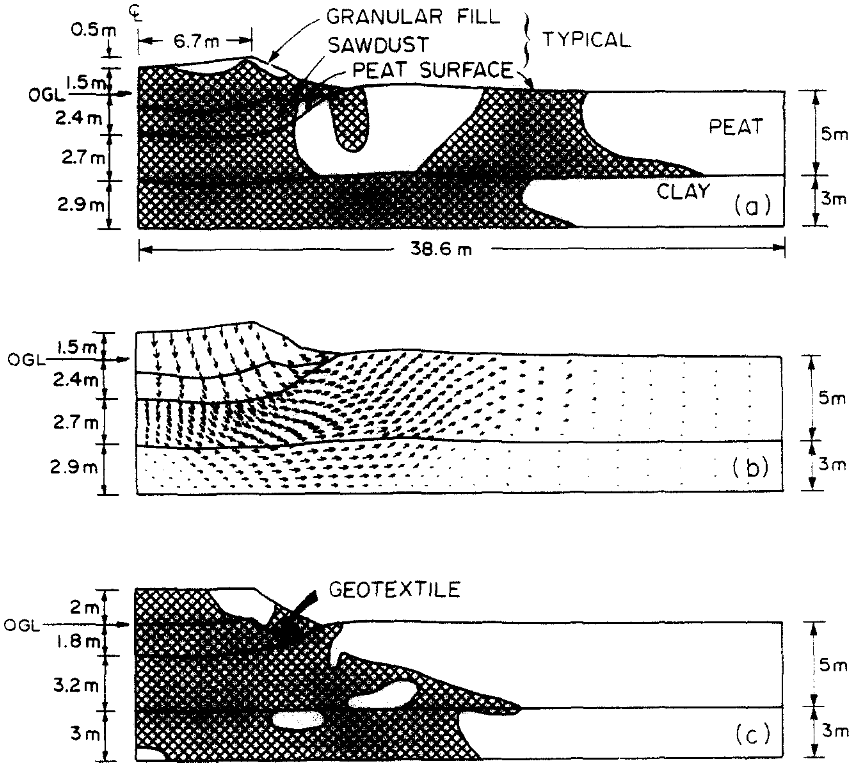

Fig. 5. Embankment constructed using 2 m of lightweight fill (sawdust). Peat underlain by very soft clay ($\bar{B}_{max}$ = 0·34): (a) $h<2$ m, no geotextile, plastic region at collapse; (b) $h<2$ m, no geotextile, velocity field at collapse; (c) h = 2 m, geotextile reinforced, E_f = 500 kN/m.

could not be constructed to 2 m using 2 m of lightweight fill. Again the sloping crest in Fig. 5(a) and (b) is an indication that it was not possible to construct the embankment to grade. The use of a 500 kN/m modulus geotextile beneath the 2 m (initial compacted thickness) of sawdust significantly improved the performance of the embankment and permitted construction to a height of 2 m as shown in Fig. 5(c). This figure also shows that the optimal amount of sawdust was used. After placement of the 2 m of granular fill above the sawdust it had compressed to 1·8 m and the top of the sawdust at the end of construction was coincident with the water table. Similar analyses showed that the embankment could be con-

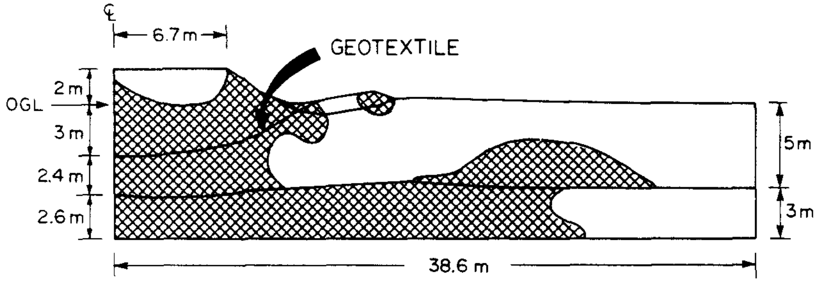

Fig. 6. Geotextile-reinforced embankment constructed with a 5 m wide × 1 m high berm. Peat underlain by very soft clay ($\bar{B}_{max}$ = 0·34, h = 2 m, E_f = 2000 kN/m).

structed to 2·5 m by using 2·2 m of lightweight fill in conjunction with a 2000 kN/m modulus geotextile.

An alternative to the use of lightweight fill would be the use of berms. Analyses showed that, for the assumed conditions, an embankment could be constructed to a height of 2 m by first constructing 16 m wide × 1 m thick berms. A smaller berm could be used in conjunction with a geotextile and Fig. 6 shows that an embankment could be constructed to a 2 m grade by using a 5 m wide × 1 m thick berm in conjunction with a 2000 kN/m geotextile located at, or near, the interface between the fill and peat. An examination of Fig. 6 also suggests that the use of a wider berm would be highly desirable.

Table 5 summarizes the cases where the analyses indicated that an embankment could be constructed to the height, h, above the original ground level for 3 and 5 m of peat underlain by 2 or 3 m of very soft clay assuming the foundation and fill parameters given in Tables 1, 3 and 4 and a value of $\bar{B}_{max}$ in the peat equal to 0·34.

The modulus of the geotextile required to provide stability under the assumed conditions together with the expected maximum fabric strain under these conditions is given in Table 5. The fabric to be used should have a secant modulus (over the strain range from zero to the expected strain) greater than or equal to that given. For a number of cases, a range of modulus values is given. The analysis would indicate that the embankment could be constructed using a geotextile within the range specified. However, if the conditions are as bad as assumed, the embankment would be very close to failure using geotextiles at the low end of the range and for these cases it would be far better to adopt a fabric with a modulus in the upper end of the specified range. If there is a competent root mat and/or the anticipated value of $\bar{B}_{max}$ is less than 0·25, then a fabric with a modulus in the lower end of the range may be adequate.

The results given in Table 5 assume the embankment is constructed of conventional fill. If lightweight fill is available, then its use should be seriously considered. Table 6 gives the required geotextile properties and lightweight fill (sawdust) thickness for a 5 m thick peat deposit underlain by 3 m of soft clay. The thickness of sawdust given in Table 6 corresponds to the compacted thickness and was selected to ensure that the sawdust would be submerged (assuming a water table at the surface) at the end of construction. For peat deposits with depths or compressibility slightly different from that assumed, similar results would be obtained by using the same geotextile as suggested in Table 6 provided that the thickness of the sawdust was adjusted so that it is approximately equal to the anticipated settlement.

The numbers given in parentheses in Tables 5 and 6 represent the expected geotextile strains for the given geotextile for the worst assumed conditions. The expected force in the geotextile can be deduced from the strain and the modulus. Since the geotextile strains are sensitive to construction sequence, strains greater than or less than those indicated in Tables 5 and 6 may be anticipated under some circumstances and any geotextile selected should have an adequate factor of safety against failure of the geotextile itself. Thus a geotextile should be selected which (a) has an appropriate modulus, and (b) has an adequate factor of safety against failure of the geotextile.

The height above original ground level, h, used in the design should be the maximum height of the embankment and includes any surcharge which may be applied.

7. CONCLUSIONS

The finite element analyses performed in this study have indicated that geotextile reinforcement may be an effective method of improving the performance of embankments constructed over peat deposits which are underlain by a layer of soft clay. The stabilizing effect of the geotextile was seen to increase as the geotextile modulus increased; however, the results also indicate that situations may occur where the use of even a very stiff geotextile may not be sufficient to ensure stability of low embankments constructed from granular fill. All other things being equal, the use of a geotextile in conjunction with lightweight fill appears to be the most satisfactory means of improving the performance of embankments on these very poor foundations.

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