The observed behaviour of a geotextile-reinforced embankment constructed on peat

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The design, instrumentation, and field performance of two instrumented sections of a geotextile-reinforced embankment are described. The 1–1.5 m high embankment was constructed in three stages on top of a peat deposit that extended to depths of up to 7.6 m. The peat was highly compressible with average water contents of 445% and 785% at the two instrumented sections.

A polypropylene, monofilament woven fabric (Permealiner 1195) was used to reinforce the embankment over the less compressible section of the deposit. At stage I, 1.37 m of fill resulted in settlements of approximately 1 m but only 1% transverse geotextile strain of 21%. Stage II construction involved bringing the embankment to final grade, with a total fill and pavement thickness of 4.2 m. Settlements at stage III were relatively small. A strong, twisted, slit film, polypropylene woven fabric (Geolon 1250) was used to reinforce the more compressible section of the deposit. At stage I, 2.74 m of fill resulted in settlements of approximately 3.1 m and transverse geotextile strains of 3%. At stage II, a total of 6.1 m of fill had been added resulting in settlements of approximately 4.6 m. At stage III, the embankment was brought to grade by reducing the fill thickness and constructing the pavement. The final fill and pavement thickness was 5.7 m. It is concluded that the use of a single layer of even a very strong geotextile was insufficient to prevent large shear deformations in these deep, compressible, peat deposits. The procedures used in the design represented the state-of-the-art at that time; however, they did not provide a good indication of how the embankment would perform in the field. It is recommended that simplified limit equilibrium design procedures should be viewed with considerable caution when designing geotextile-reinforced embankments on peat.

Keywords: embankment, muskeg, peat, geotextile, settlement, pore pressures, field observation, instrumentation, soil reinforcement.

L’étude, l’instrumentation et le comportement de deux sections instrumentées d’un remblai renforcé par géotextile sont décrits. Le remblai de 1 à 1.5 m de haut a été construit en trois étapes à la surface d’un dépôt de tourbe d’une épaisseur allant jusqu’à 7,6 m. La tourbe était très compressible avec des teneurs en eau de 445% et 785% aux deux sections instrumentées.

Une toile de monofilament de polypropylène tissé (Permealiner 1195) a été utilisée pour renforcer le remblai au dessus de la section la moins compressible du dépôt. Durant l’étape I, 1,37 m de remblai ont produit des tassements de 1 m environ, mais seulement 1% de déformation transversale dans le géotextile. Durant l’étape II, un total de 3,87 m de remblai avait été ajouté avec des tassements totaux d’environ 3 m et des déformations transversales du géotextile de 21%. La construction de l’étape III impliquait la construction du remblai jusqu’à son élévation finale, avec une épaisseur totale de remblai et de chaussée de 4,2 m. Les tassements lors de l’étape III ont été relativement petits.

Une toile forte en bandes de polypropylène torsadées tissées (Geolon 1250) a été utilisée pour renforcer la section la plus compressible du dépôt. Durant l’étape I, 2,74 m de remblai ont produit environ 3,1 m de tassement et des déformations transversales du géotextile de 3%. Après l’étape II, un remblai total de 6,1 m avait été ajouté, résultant en des tassements de 4,6 m environ. Lors de l’étape III, le remblai était amené à sa hauteur finale en réduisant l’épaisseur de remblai et en construisant la chaussée. L’épaisseur finale de remblai et de chaussée était de 5,7 m.

On en conclut que l’utilisation d’une seule nappe de géotextile, même très fort, est insuffisante pour empêcher les grandes déformations dans de tels dépôts de tourbe épais et compressibles. Les procédures utilisées dans ce projet représentent l’état des connaissances du moment, mais elles ne fournissent pas d’indication valable sur le comportement probable du remblai. Il est recommandé de considérer les méthodes simplifiées de calcul d’équilibre limite avec beaucoup de prudence lorsqu’on étudie des remblais renforcés de géotextiles sur tourbe.

Mots-clés: remblai, tourbe, géotextile, tassement, pression interstitielle, observation de chantier, instrumentation, renforcement des sols.


1Retired.
Introduction

Soft, highly compressible organic deposits are widely distributed in Canada. The construction of embankments or roadways across these deposits requires either the excavation and replacement of the organic material with fill or the placement of the fill on top of the soft foundation. Economic and construction considerations often preclude excavation and replacement. However, the alternative of construction on top of soft peat has many associated difficulties (e.g. MacFarlane and Rutka 1959; Raymond 1969; Samson and La Rochelle 1972).

In the design of roads or embankments on soft foundations, it is becoming more and more common to make use of geotextiles. It is generally accepted that these materials will act as separators and filters if placed between the organic and fill materials. Furthermore, it is often suggested that the geotextile will act as reinforcement, reducing tensile strains and lateral and vertical deformations and increasing embankment stability, thereby allowing more cost-effective embankment designs. On the basis of these arguments, a portion of a road embankment built on a soft organic deposit was constructed using geotextiles as reinforcement. This embankment was designed by the Ministry of Transportation and Communications using the technology available in the early 1980’s (e.g. Haliburton 1981). Two sections of the embankment were instrumented so as to provide data regarding the effectiveness of the geotextile. Construction commenced in the summer of 1981 and proceeded in three stages with completion in the summer of 1982.

In this paper, the design of the embankment will be described together with the basic engineering properties of the peat and the results of the field measurements. In a subsequent paper (Rowe et al. 1984), the engineering properties required for a detailed analysis of the embankment behaviour will be presented and the results from the subsequent calculations compared with the field data. The implications of this field case history for future design with geotextiles will then be discussed.

Site description and soil profile

The instrumented embankment forms part of an extension to Bloomington Road (Regional Road 40; Regional Municipality of York) and is located between Leslie Street and Highway 404 near Aurora, Ontario, Canada. The general area is hummocky with numerous poorly drained organic deposits. The road traverses one of these deposits, which has a maximum thickness of 7.6 m and a length of almost 300 m.

An initial investigation by the Ministry indicated that the depth of peat varies considerably along the proposed road centre line as shown in Fig. 1. A detailed field investigation involving 10 boreholes was carried out at, or near, the instrumented sections A and B at chainages 1 + 982 and 2 + 119 respectively. After construction of stage I, an additional set of 5 boreholes at locations other than the instrumented sections was placed by Warnock Hersey (1981), under contract to the Ministry.

The muskeg cover classification (MacFarlane 1969) at the site ranges from ADE at the shallow sections, to BEH at the instrumented sections. The water table is at or above the original ground surface.

The nature of the peat in this deposit varies considerably. In general, the upper 1 m may be classified as predominantly amorphous containing woody, fine fibres held in a woody, coarse, fibrous framework (category 6—MacFarlane 1969). In terms of the modified Von Post classification (Landava and Pheeney 1980) it may be classed as \( H_{6-8} B_{2-3} F_{1-2} R_{0-1} W_{0-1} \).

The underlying peat ranges in classification from category 2 (nonwoody, fine, fibrous peat) to category 15 (woody mesh of fibres enclosing amorphous—granular peat containing fine fibres); however, it is predominantly a fine, fibrous peat with some woody particles (category 11) having a modified Von Post classification \( H_{5-7} B_{3-5} F_{1-2} R_{1-2} W_{1} \).

At the two sections investigated by the Ministry, the muskeg was underlain by silty sand and sand of variable depth as shown in Fig. 2. As indicated by Figs. 1 and 2, the location of the peat—sand interface varied considerably both transversely and longitudinally. Subsequent drilling by Warnock Hersey (1981) indicated the presence of some isolated pockets of organic clay \( (w_h = 213\%, w_L = 267\%, w_p = 172\%) \) and organic silt \( (w_h = 30\%, w_L = 42\%, w_p = 34\%) \) between the peat and the sand. Where present, this material was typically 1.3 m thick except for a thickness of 4 m at chainage 1 + 988 (i.e. 6 m from instrumented section A). These isolated pockets of clay/silt were not evident from the borings conducted prior to construction and were not considered in the design. The embankment performance at chainage 1 + 988 was, however, visibly worse than that at the instrumented section A (chainage 1 + 982).

Samples of peat were obtained with 50 mm diameter and 150 mm diameter Shelby tubes. The properties of the peat are summarized in Table 1. Figure 3 shows the variation in void ratio \( (e_0) \) and compression index \( (C_v) \) with depth at the instrumented sections. Figure 4 shows the variation in vane shear strength with depth obtained using the standard MTC vane (197 mm × 67 mm).

West of the organic deposit (see Fig. 1), the road alignment required an extensive cut in sand. A typical grading curve for the sand is shown in Fig. 5. When it was compacted as embankment fill, in-situ density measurement indicated that this material had an average bulk unit weight of 20.4 kN/m\(^3\) at a moisture content of 14%. In well-compacted areas (e.g. near the centre of the embankment), the average unit weight was 21 kN/m\(^3\). The loose fill at the edges had unit weights as low as
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**Fig. 1.** Centre line profile at Bloomington Road.

**Fig. 2.** Design profile and instrumentation at stations A and B.

18.9 kN/m$^3$. At a unit weight of 20 kN/m$^3$, the fill had a friction angle of 32°.

**Design**

The following construction methods were considered during the design stage: (1) complete excavation and backfilling; (2) partial excavation and displacement; (3) displacement by surcharging; (4) direct construction on top of the organic deposit using lightweight fill; and (5) direct construction on top of the organic deposit using the local granular fill.

Complete excavation and backfilling was considered to be impractical at this location because of the depth of the peat. Methods employing displacement were rejected due to the unfavourable geometry of the underlying firm deposits. Finally, lightweight fill was not locally available and had to be rejected because of cost. Thus it was decided to construct the embankment directly on top of the peat using the local sand fill (see Fig. 5). However, the adoption of this construction technique raised concern regarding short-term stability and long-term settlements.

Ripley and Leonoff (1961) observed that for embankments where failures had occurred, the measured vane strengths were much higher than the calculated undrained strengths. Landva (1980) also indicated that the available strength of peat may be much lower than the apparent strength measured by field vane tests. The authors' experience also suggests that the field vane strength almost always overestimates the available
Table 1. Properties of peat at Bloomington Road

<table>
<thead>
<tr>
<th>Property</th>
<th>Section A Chainage 1 + 982 (m)</th>
<th>Section B Chainage 2 + 119 (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No. of tests Range Mean value Standard deviation</td>
<td>No. of tests Range Mean value Standard deviation</td>
</tr>
<tr>
<td>Unit weight (kN/m³)</td>
<td>10 9.6–11.5 10.4 0.7</td>
<td>5 9.2–10.0 9.6 0.4</td>
</tr>
<tr>
<td>Water content¹ (%)</td>
<td>12 223–690 445 180</td>
<td>7 490–1040 785 230</td>
</tr>
<tr>
<td>Organic content (%)</td>
<td>7 17–80 50 26</td>
<td>6 8.3–18.5 13 4</td>
</tr>
<tr>
<td>Initial void ratio, eₒ</td>
<td>7 2.7–16.9 9.9 4.9</td>
<td>6 3.8–10 6.8 2.4</td>
</tr>
<tr>
<td>Compression index, Cₑ</td>
<td>7 1–9 4.8 2.7</td>
<td>5 1.38–1.56 1.47 0.09</td>
</tr>
<tr>
<td>Specific gravity of solids, Gₛ</td>
<td>6 1.5–2.05 1.7 0.2</td>
<td>8 7–25 18 5</td>
</tr>
<tr>
<td>Vane shear strength² (kPa)</td>
<td>41 6–38 17 8</td>
<td>16 1.2–7.3 3.1 1.6</td>
</tr>
<tr>
<td>“Sensitivity”, Sᵣ</td>
<td>39 1.5–4 2.6 0.9</td>
<td>5 216–324 268 46</td>
</tr>
<tr>
<td>Liquid limit⁵ (%)</td>
<td>5 138–255 212 52</td>
<td>5 38–78 56 17</td>
</tr>
<tr>
<td>Plastic limit (%)</td>
<td>5 38–78 56 17</td>
<td>5 38–78 56 17</td>
</tr>
<tr>
<td>Plasticity index (%)</td>
<td>5 38–78 56 17</td>
<td>5 38–78 56 17</td>
</tr>
</tbody>
</table>

¹Percent of dry weight. These values represent a lower limit since some water is invariably lost during sampling.

²Determined from 1D consolidation tests on 49 mm diameter × 12.5 mm nominal height samples using nominal applied pressures of 3, 6, 9, 12, 18, 24, 36, 48, 72, 96, 144 kPa with load increments applied every 24 h.

³Determined using the standard MTC vane (197 mm × 67 mm). See Landva (1980) for a discussion of the validity of vane strength.

⁴Sᵣ = undisturbed/remoulded vane strength.

⁵Could only be determined for the more amorphous peat samples.

Fig. 3. Variation in peat compressibility characteristics.

strength of peats and that the use of the remoulded vane strength in a conventional ϕ = 0 stability analysis provides a better indication of short-term stability. (This procedure can be questioned on several grounds and should be regarded as an empirical approach.)

Assuming negligible geometry changes under "undrained" conditions and also assuming that the embankment was constructed to the maximum design height of 1.5 m (γ = 20.4 kN/m³) in one stage, the stability immediately after construction was estimated. Using remoulded shear strengths of 6.5 kPa at station A and 5.8 kPa at station B, a ϕ = 0 circular arc stability analysis yields unacceptable factors of safety of 1.15 and 1.05 at stations A and B respectively.

Fig. 4. Variation in vane strength with depth at (a) station A and (b) station B.

It was postulated that the stability of the embankment with respect to a rotational type failure could be increased by the use of a reinforcing geotextile placed near the peat–fill interface. The geotextile was primarily selected on the basis of a circular arc stability analysis,
in which the following additional assumptions (based on Haliburton 1981) were made: (a) The geotextile is placed with the stronger direction (usually the warp) transverse to the fill alignment. (b) The tensile forces required to maintain stability can be mobilized at relatively small strains (i.e. the deformations will not be "excessive"). (c) The tensile force in the fabric will act tangentially to the failure arc. The additional restoring moment along the predetermined critical circle is then obtained by multiplying the circle radius by the tensile force in the fabric. Thus, to increase the factor of safety to 1.3, a tensile force of approximately 20 kN/m and 39 kN/m would be required at stations A and B respectively. It should be emphasized that these calculations are based on assumptions considered suitable in 1981. More recent research (e.g. Rowe et al. 1984; Rowe and Soderman 1984; Rowe, 1984) casts doubt on the validity of some of these assumptions.

The requirement that the stabilizing tensile force be developed at relatively small strain implies that the geotextile selected for reinforcement should have a high tensile strength under relatively small elongation (i.e. high tensile modulus). For this reason, together with reasons of economy and availability, two strong woven fabrics were selected for the evaluation of the effect of fabrics on the stability and performance of the proposed embankment. One of the fabrics, with the trade name Geolon 1250, was a plain weave, twisted, polypropylene, slit film material, produced by Nicolon Corp. The other material, Permealiner 1195, distributed by Synflex Industries, was a calendered polypropylene monofilament, with plain weave. The physical and

<table>
<thead>
<tr>
<th>Properties</th>
<th>Permealiner 1195</th>
<th>Geolon 1250</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass (g/m²)</td>
<td>225</td>
<td>730</td>
</tr>
<tr>
<td>Thickness (mm)</td>
<td>0.4</td>
<td>2.26</td>
</tr>
<tr>
<td>Mullen bursting pressure (MPa)</td>
<td>3.5</td>
<td>11</td>
</tr>
<tr>
<td>Equivalent opening size (μm)</td>
<td>212</td>
<td>425</td>
</tr>
<tr>
<td>Breaking force in grab test¹ (kN)</td>
<td>1.78</td>
<td>5.56</td>
</tr>
<tr>
<td>Estimated ultimate strength from grab test² (kN/m)</td>
<td>42</td>
<td>133</td>
</tr>
<tr>
<td>Ultimate tensile strength³ (kN/m)</td>
<td>41</td>
<td>178</td>
</tr>
<tr>
<td>Elongation at rupture in grab test (%)</td>
<td>30</td>
<td>18</td>
</tr>
<tr>
<td>Ultimate tensile strain³ (%)</td>
<td>37</td>
<td>24</td>
</tr>
</tbody>
</table>

¹In the warp direction.
²Estimated ultimate strength = 24 × grab strength (see text).
³Determined from wide strip test on 500 mm wide × 100 mm gauge length samples, warp direction, strain rate 2%/min.
⁴Used at station A, chainage 1 + 982, see Fig. 1.
⁵Used at station B, chainage 2 + 119, see Fig. 1.
mechanical properties of the selected geotextiles are given in Table 2.

The conventional tests for determining the strength of geotextiles (the "grab test"—CGSB Standards 4-GP-2; method 9.2) involves applying a tensile force along the centre 25.4 mm section of a 100 mm wide strip of fabric. Thus the resulting breaking force cannot be interpreted as being the property of the fabric along a width of 25.4 mm, since the adjacent width contributes to the measured strength. For any stability analysis, the tensile strength and the modulus must be expressed in terms of a value per unit linear measure. Since it is usually the grab
strength that is measured, a factor is needed to convert the grab tensile strength to a unit width. As a result of correlation studies, it was suggested that 60% of the ultimate grab tensile strength of these woven fabrics be used for the tensioned width of 25.4 mm (Rowe and Ho 1984; L. Murch, personal communication). Thus the estimated ultimate tensile strength (in kN/m) is approximately 24 times the grab breaking strength (in kN). The estimated ultimate strength determined in this manner is compared with the ultimate tensile strength determined from a wide strip test in Table 2. For the Permealiner 1195, the agreement is excellent. For the Geolon 1250 the estimated value is conservative. (The reason for the discrepancy is discussed by Rowe and Ho 1984).

Based on the foregoing calculations, the forces required to maintain a factor of safety of 1.3 against rotational failure represent approximately 0.5 and 0.3 of
the estimated ultimate tensile strength of the Permea-
liner 1195 (at station A) and Geolon 1250 (at station B)
respectively.
To further ensure the stability of the embankment, a
stage construction sequence was specified. To reduce
long-term settlements, a surcharge was applied at the
deeper section of the deposit.

Instrumentation
To allow monitoring of the construction procedure
and to observe the geotextile performance, field
instruments were installed at two of the deepest sections
of the swamp. The locations chosen for instrumentation
are shown in Fig. 1 and the instrumentation layout is
given in Fig. 2.
A total of six brass Geonor piezometers was installed
beneath the centre of the embankment and beneath the
crest of the slope to determine pore water pressures at a
depth of 1.5 and 3.0 m below ground level. Piezometer
leads were directed out from beneath the embankment to
enable readings by means of Bourdon gauges.
Vertical movements of the ground were measured by
running a pressure transducer through water-filled
37.5 mm diameter PVC tubing. Three tubes were
installed at each instrumented section. To provide
duplication in measurements, two tubes were initially
installed, one at the original ground surface and one at
the top of the working platform. However, at the end of
stage I, it was feared that the ground movements would
threaten the function of the lower tube and an additional
tube was installed at the top of the fill just prior to
construction of stage II. Hand and power auger borings
were carried out at various stages of construction to
confirm the settlement readings.
Elongations in the geotextile were monitored both in
the transverse and longitudinal directions at the centre
line, and in the transverse direction at a point 6 m from
the centre line (i.e., beneath the shoulder). Two
independent sets of gauges were installed, one operated
hydraulically and the other by electric current (see Fig.
6). The hydraulic gauges (Barsvary et al. 1982)
consisted of a water-filled cylindrical body with an
activation piston. The body was clamped to the
geotextile at a point 200 mm distant from the actuating
point for the piston arm. Elongation in the geotextile
caused withdrawal of the piston arm and resulted in a
lowering of the water level in a sight tube remote from
the embankment but connected to the cylinder. A second
measure of elongation was accomplished by means of
Bison strain gauges. This system consists of two metal
rings, 101 mm diameter × 9.5 mm thick; each ring is
wrapped with wire, encased in plastic, and the leads are
directed out from under the embankment. By introduc-
ing a current in the coils of one ring and measuring the
effect on the other, the distance between the coils is
determined. Since the coils are independently affixed to
the geotextile at an original spacing of about 200 mm,
the distance between the coils is directly related to the
elongation of the geotextiles.
Lateral displacement of the foundation subsoil was
measured with vertical inclinometers that were installed
at the toe of the embankment as shown in Fig. 2. The
88 mm diameter aluminum inclinometer casings con-
sisted of 1.5 mm long sections joined by riveted
Fig. 11. Field observations at station A: chainage 1+982.
machined couplings. The inclinometers were installed immediately after completion of the working platform by augering into the lower sand deposit and placing the tubes into the open boreholes.

**Sequence of construction**

The first stage of construction began in July of 1981. After clearing the site (see Fig. 7), a sand working platform was placed to facilitate laying of the geotextile. As a result of the swamp surface irregularities, and in order to avoid punching failure caused by construction traffic, the working platform varied in thickness from 0.3 to 1.0 m with an average of 0.6 m. The geotextiles were then laid transverse to the road alignment, overlapping adjacent strips by 0.7 m. The first lift of fill on the geotextile was 0.3 m thick. Anchorage of the geotextile was achieved by folding back the extra 3 m length of fabric over the first lift, and placing a second lift over the fabric near the toes (see Fig. 8). These toe fills were compacted prior to filling the middle portion of the embankment. Subsequent lifts were added by the same outward—inward construction sequence to achieve the design earth grade.

The stage II construction began after the excess pore pressure, due to initial construction, had dissipated. Earth fill was brought up to the subgrade level. In areas where the deposit was deepest, earth fill surcharge was placed up to a level of 0.3 m above final grade in the centre 8 m of the embankment (see Fig. 9).

The third and final stage of construction consisted of removing sufficient earth fill to allow for the construction of a 0.6 m thick crushed aggregate base and asphaltic pavement. Additional earth was placed in the area of the shoulders and side slopes to establish the design geometry. The completed roadway is shown in Fig. 10.

**Field observations**

The construction sequence, as well as instrumentation readings for settlement, geotextile elongation, and excess pore water pressure versus time, is shown in Figs. 11 and 13 for instrument stations A and B respectively. Cross sections of the embankment and settlement profiles are shown for key points in the construction sequence in Figs. 12 and 14 for stations A and B respectively.

Because of the markedly different behaviour at stations A and B, they will be discussed separately.

**Station A**

During stage I construction, 1.37 m of fill was added at the centre line. Immediate settlements were about 50% of the total height of fill added. Prior to stage II
Fig. 13. Field observations at station B: chainage 2+119.
During stage I construction, the settlements had increased to approximately 75% of the original thickness of fill placed. Dissipation of excess pore pressure occurred within 60 days of construction. The rate of settlement at 90 days after construction was in the order of 0.1 m/month. It is interesting to note that elongations of 5% in the geotextile parallel to the centre line were about 4% higher than those measured transverse to the centre line (1%), implying that plane strain conditions did not apply. No evidence of mud waves or tension cracking was observed during this stage of construction. Borings in September 1981 confirmed the location of the geotextile and fill-peat interface indicated by the profilometers.

During stage II construction, an additional 2.5 m of fill (as measured at centre line) was placed, which increased the net fill height by only 1.1 m. Subsequent settlements over the next 6 months reduced the net fill height to 0.8 m. Geotextile elongations at centre line were measured to be as high as 21% in the transverse direction and 8% in a direction parallel to the centre line. The measured elongations transverse to the centre line beneath the shoulder were slightly negative, indicating that the geotextile was subjected to slight compression forces at this point. Transverse elongations indicate that the geotextile strain was not uniform in the transverse direction.

Some difficulty was encountered with the piezometers in stage II. The pore pressure readings at 1.5 m below the centre line never dissipated. At 3 m below the centre line, the pore pressures appeared to dissipate very quickly but inexplicably rose again in January 1982. The pore pressure readings below the shoulder dissipated quickly.

"Mud waves" and tension cracks in the root mat immediately north of the embankment were first observed during stage II construction. Longitudinal tension cracks were also observed within the embankment fill outside the limits reinforced by the geotextile (see Figs. 15 and 16). Borings in June, 1982 indicated that the peat beneath the embankment had consolidated considerably, since it was no longer possible to turn the field vanes. The average moisture content in the peat had decreased to about 120%.

The difference in behaviour between stages I and II should be noted. Most of the settlement in stage I appears to have been due to compression of the underlying peat and the geotextile experienced negli-
ble transverse strain. However, the lateral movement and "mud waves" observed at stage II would suggest that a large component of the settlement at this stage was due to shear deformation which, in turn, resulted in the development of large fabric strains.

Stage III construction brought the total thickness of fill and pavement to 4.2 m and resulted in 0.1 m of additional immediate settlement. Settlements in the 7 months following paving amounted to an additional 0.1 m.

Station B

At the centre of the embankment, 2.74 m of fill was added in stage I. The immediate settlements amounted to 2.4 m (i.e., almost 90% of the total height of fill placed). In the months following construction, additional settlements of about 0.7 m occurred and the roadway was almost completely submerged as indicated in section 2 of Fig. 14 and as shown in Fig. 17.

The transverse strains in the geotextile at the centre line averaged approximately 3% during July and August 1981 (a peak value of 4.4% was recorded during construction). These strains gradually decreased to a negligible amount in September 1981. Similar trends were observed for the transverse strains at the shoulder. The longitudinal strains at the centre line were compressive with an average of −1% and a peak of −2%. The elongation gauges at this location malfunctioned after September 1981 and the increase in strains noted towards the end of September should be viewed with some skepticism.

Rapid dissipation of pore pressure occurred at the upper piezometers during and after loading. The deeper piezometer showed rapid pore pressure dissipation during construction but then remained relatively constant at an excess head of approximately 1.25 m for 2 months. The pore pressures dissipated to 0.6 m during the third month. It may be speculated that the rapid dissipation and consolidation that occurred in the upper portion of the peat deposit was accompanied by a decrease in permeability and, thus, the dissipation of pore pressure from the lower portion of the deposit was inhibited.

In stage II construction, the centre of the embankment was loaded with an additional 3.4 m of fill. Immediate settlements due to this 3.4 m of fill amounted to 1.5 m. The height of the embankment, above average original ground surface, was 1.5 m immediately after construction and, as a result of settlement, decreased by only 0.03 m in the 6 months following construction. This would suggest that the preceding settlement was largely the result of shear deformations.

Evidence of "mud waves" and tension cracks in the root mat on both sides of the embankment was apparent during this stage of construction. Borings carried out in June, 1982 indicated that the peat had shear strengths in excess of 100 kPa, as measured by field vane tests, suggesting considerable consolidation had occurred in the 6 months following stage II construction. This was further confirmed by field moisture contents, which had decreased to an average of 150%.

In stage III construction, about 1.0 m of fill was removed from the centre of the embankment. Additional fill was added to the shoulders and side-slopes, and the road was completed by the construction of the pavement...
structure. At the end of stage III, the total thickness of fill plus pavement was 5.67 m. In the 7 months following construction, additional settlement of about 0.2 m occurred.

**General comments on instrumentation**

At stage I, the profilometers provided a very consistent set of settlement readings. The location of the geotextile was checked by hand augering at a number of locations. The depth to the geotextile, peat-fill interface and to the bottom of the swamp were also confirmed by means of a number of boreholes placed at the end of stage I. It is considered that the settlements at this stage are accurate to better than 0.3 m and possibly to better than 0.15 m.

The large deformations associated with stage II of the construction caused a failure of the profilometers at the peat–fill interface. The positions of the interface shown in Figs. 12 and 14 were deduced from the results for the profilometers immediately below the geotextile by adjusting for the distance between the two profilometers at the end of stage I. (This correction neglects any compression of the fill between the geotextile and the peat, which may have occurred during stage II.) Boreholes were placed to confirm the location of the peat–fill interface. To avoid possible damage to the instruments, these boreholes were located 3 and 6 m from the instrumented sections at stations A and B respectively. The peat–fill interface located in these boreholes was (in three of the four cases) within 0.6 m of that inferred from the profilometer readings. Both the surface and depth of swamp varied with position and since the boreholes were not at quite the same location as the profilometers, some caution must be exercised in comparing the two results. Nevertheless, the borings confirm the general trends and magnitudes of the settlements indicated by the profilometers. Thus, although the writers do not have quite the same confidence in the settlement readings at stage II as at stage I, it is believed that these readings are accurate to better than 0.6 m and are probably accurate to 0.3 m.

Settlement plates were not used because of concern regarding the inconvenience it would have caused during construction. In retrospect, the inconvenience could probably be justified, at least for settlement plates near the shoulder. The use of settlement plates in conjunction with the profilometers would have reduced the need, and consequently the cost, of the boreholes required to confirm the settlements.

The hydraulic strain gauges performed very well considering the quantity of fill and the magnitude of the deformations that were encountered. The results from these gauges were generally confirmed by the Bison strain gauges. Some difficulties were experienced with the readout device(s) for the Bison gauges and hence these gauges did not perform as reliably as the hydraulic gauges.

The Geonor piezometers provided some indication of the pore pressure development and dissipation during construction. However, the deformations were such that their locations could not be defined after installation. Some difficulty was also experienced owing to gas collecting in the piezometers.

Measuring lateral deformation in peat is a difficult undertaking. These difficulties arise because of the large deformation gradient in peat together with the fact that the relatively stiff inclinometer casing may not deform to the same extent as the surrounding peat. The inclinometers at Bloomington Road all failed during the first stage of construction and no useful conclusions could be reached on the basis of the limited data that was obtained.

**Road performance**

Bloomington Road was completed in June 1982 (see Fig. 10). Since completion, settlement of the roadway embankment has continued to occur. In general, differ-
ential settlements in the transverse direction are minor, while significant differential settlement has occurred in the longitudinal direction. These differential movements have resulted in depressions occurring across the roadway in areas adjacent to the deeper portions of the swamp. It is speculated that these differential settlements could have been reduced if the stage II surcharge had been extended over the entire length of the swamp rather than confined to the two deeper areas. Remedial measures are being contemplated.

Conclusions

The design, instrumentation, and field performance of two instrumented sections of a geotextile-reinforced embankment constructed on peat have been described. The peat was highly compressible with average water contents of 445% and 785% at the two sections. The peat depth varied considerably with position (both longitudinally and laterally), having a maximum depth of 7.6 m. A moderate strength (Permealiner 1195) and a high strength (Geolon 1250) geotextile were used to reinforce different sections of the embankment. The embankment was constructed in three stages over the period of 1 year.

Large settlements were observed at both instrumented sections, with a maximum fill thickness of 5.7 m and a maximum settlement of 4.7 m being recorded. A significant component of the settlement appears to have been due to shear distortion, particularly during stage II construction. Tension cracks were observed in the fill (outside of the reinforced area) and in the peat during stage II construction. Large lateral movements were evident from “mud waves” and tree movements adjacent to the embankment.

The geotextile at the edge of the embankment appeared to be unstressed and hence folding back of the fabric was unnecessary. Unfortunately, the devices used for measuring the geotextile strains failed prior to completion of the embankment and so the maximum fabric strains may not have been measured. However, these devices did show that: (a) small geotextile strains were developed despite large settlements in stage I of construction, when the deformation was predominantly due to compression of the peat; (b) large fabric strains (in excess of 21% for the Permealiner 1195) were developed in stage II of construction when large shear deformations were apparent; (c) the transverse fabric strain varied appreciably with position; and (d) the longitudinal fabric strains were significant. At one section compressive strains of up to 2% were observed. At the other section tensile strains of 8% were recorded. These longitudinal strains reflect the irregular depth of the peat.

The role played by the geotextile reinforcement will be discussed in a subsequent article (Rowe et al. 1984) where the results of a detailed analysis of this embankment behaviour will be compared with the field data and the implications of this case history for future design will be discussed. Nevertheless, it is apparent from the field data presented here that the use of a single layer of even a very strong geotextile did not prevent large shear deformations. Thus, although the procedures used in the design represented the state-of-the-art at that time, they did not provide a good indication of how the embankment would perform in the field and it is recommended that simplified limit equilibrium design procedures should be viewed with considerable caution when designing geotextile-reinforced embankments on peat.
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