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# Geotextile Strain in a Full Scale Reinforced Test Embankment

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## ABSTRACT

*A geotextile reinforced test embankment was constructed on a soft organic clayey silt deposit at Sackville, New Brunswick, Canada in September/October 1989. A relatively high-strength polyester woven geotextile (ultimate strength of 216 kN m$^{-1}$) was used as reinforcement. The reinforcement was instrumented with a number of electrical resistance, electro-mechanical and mechanical gauges. The details of this instrumentation and field performance of the geotextile reinforcement during the construction of this test embankment are described in this paper.*

*The field data indicated that the strain in the geotextile was comparatively small (typically less than about 0·7%) up to an embankment thickness of 3·4 m. The strain increased to a maximum of about 2% when the embankment thickness was increased above 4·1 m, suggesting the initiation of movement (or yielding) of the foundation soil. A large increase of strain was evident for thicknesses exceeding 5·7 m and the available evidence indicates that the soil approached failure at a fill thickness of about 5·7 m. The strain increased to over 8·5% when the embankment was first constructed to 8·2 m thickness and then failed as the soil continued to deform at constant fill thickness and the geotextile strain increased until failure (inferred tearing) of the geotextile occurred. After the embankment failed at a thickness of 8.2 m, the addition of more fill did not result in any* *increase in the net embankment height of 6·6 m, reached just prior to failure of the geotextile. However, although the embankment exhibited signs of distress and large deformations, there was no classical rotation collapse at 8·2 m fill thickness.*

## 1 INTRODUCTION

A test embankment was constructed on a soft compressible organic clayey silt deposit at Sackville, New Brunswick, Canada in September/October 1989. This embankment was reinforced with a single layer of relatively high-strength polyester woven geotextile with an ultimate tensile strength of 216 kN $m^{-1}$. The geotextile was instrumented with a number of electrical, electromechanical and mechanical gauges.

The site investigation indicated that there was a root mat underlain by organic clayey silt whose strength increased with depth. Details of the site conditions, the soil profile, the layout of the embankment, and the details of instrumentation (which included piezometers, settlement plates, augers, heave plates and inclinometers, and a total pressure cell) have been described by Rowe *et al.* (1995). A typical borehole log together with the field-vane shear strength profile and 50 $cm^2$ cone data is given in Fig. 1. The geometry, instrumentation, and summary of the sequence of construction of the reinforced section are shown in Fig. 2.

The observed pore pressure response, the variation in both the vertical and horizontal displacements of selected points on the ground surface as well as in the foundation soil, and the horizontal displacement profile along the vertically placed inclinometers have been described by Rowe *et al.* (1995). In summary, the embankment exhibited essentially elastic behaviour to a height of about 2·4 m and relatively small horizontal and vertical deformations to an embankment thickness of 3·4 m. The deformations of the embankment increased rapidly as the embankment approached a thickness of 5·7 m and the available field evidence suggests that the shear strength of the soil was exceeded at this load level; however, the embankment did not collapse. Additional fill was placed until the embankment failed at a fill thickness of about 8·2 m. Thus, based on conventional geotechnical monitoring, it would appear that the reinforcement increased the thickness of fill that could be added prior to collapse, although the field data also indicated that the maximum net gain in height for the embankment was 6·6 m. This raises the question as to how the geotextile performed during construction. Thus, this present paper provides details concerning the instrumentation and performance of the geotextile.

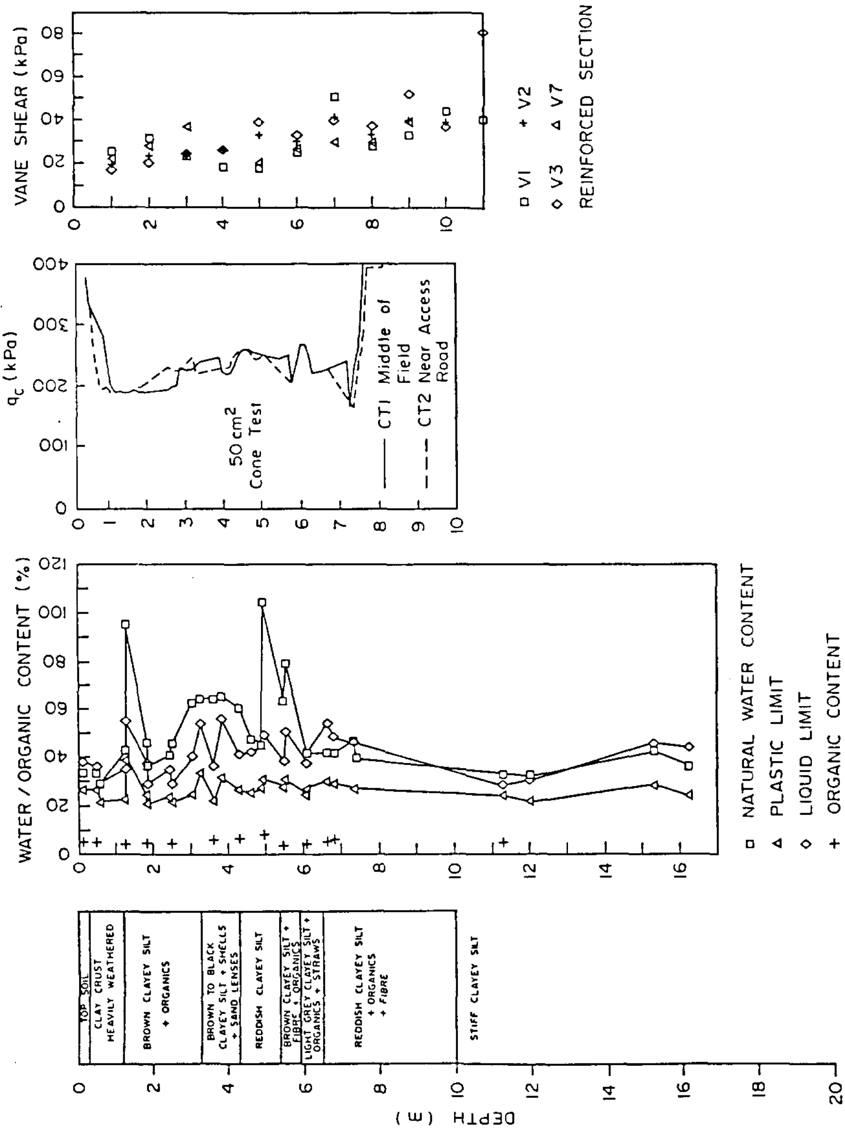



**Fig. 1.** Foundation soil profile (after Rowe *et al.*, 1995).

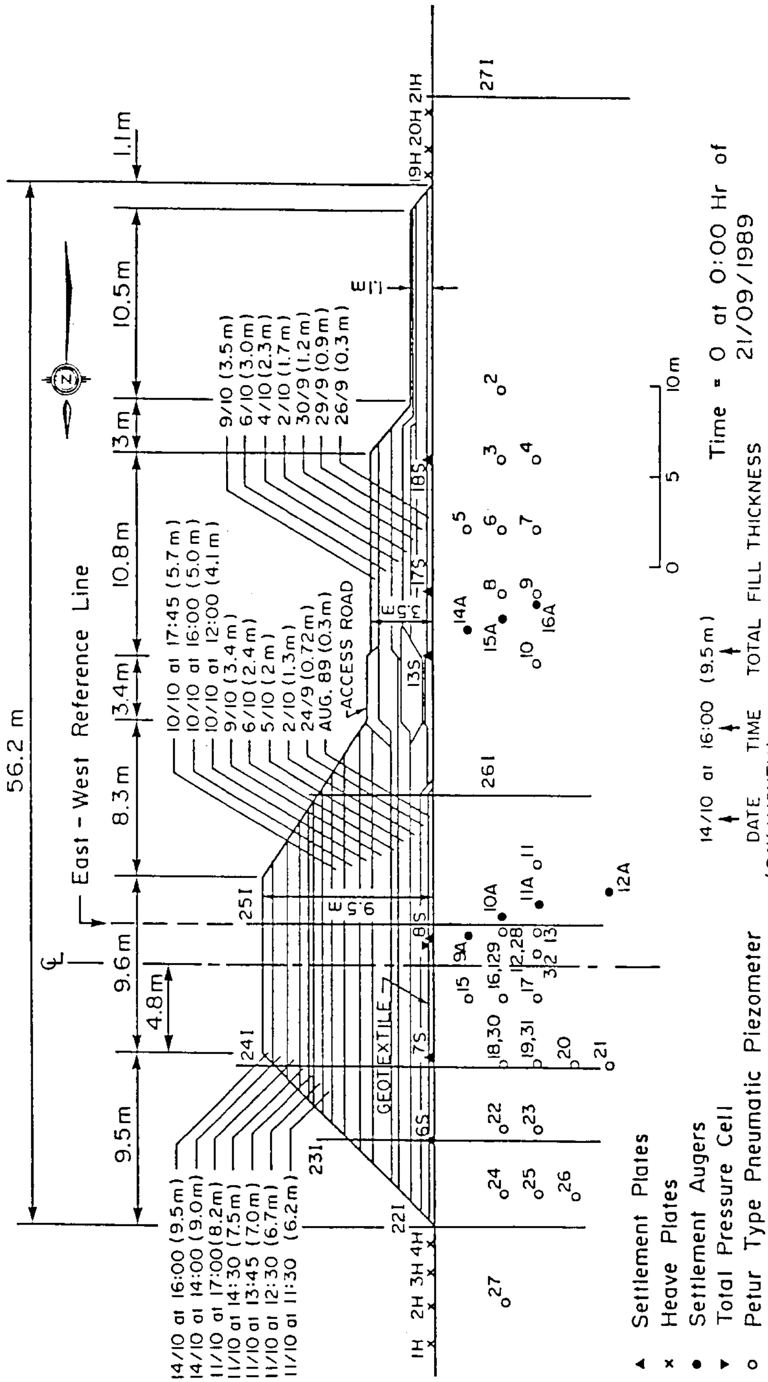



**Fig. 2.** Instrumentation layout and construction sequence of reinforced embankment (after Rowe *et al.*, 1995).

## 2 INSTRUMENTATION OF THE GEOTEXTILE

Geotextile strains were monitored both in the transverse and longitudinal directions. A total of 34 electrical resistance, seven electromechanical, and seven mechanical strain gauges (described below) were installed on the geotextile to measure the strain in the transverse direction. The strain in the longitudinal direction was monitored with four electrical resistance gauges installed at different locations.

The electrical resistance gauges were 100 mm long (manufactured by Micro-Measurements Division, Measurements Group Inc., Type EP-08-40CBY-120) and were installed on to the geotextile under controlled conditions in a storage shed. These gauges are easily damaged and despite considerable care, five of the original 38 were damaged during transport from the storage shed and placement of the geotextile in the field. The location of the electrical resistance gauges, together with the electromechanical (ring type) and mechanical strain gauges, are shown in Fig. 3 (the gauges that were damaged have been omitted from this figure).

The electromechanical gauge consisted of a thin metal ring fastened to the geotextile, at two diametrically opposite places, by means of fastening plates welded to the ring and small bolts and nuts (see Fig. 4). The ring end of the fastening plate was narrow and had a 90° angle projection symmetrical to its centre line to provide a gap between the geotextile and the ring. The strain induced in the ring caused by the deformation of the geotextile was measured with two electrical resistance strain gauges (30 mm long strain gauges, manufactured by Showa Measuring Instruments Ltd — Type N11-FA-30-120-11) installed diametrically opposite to each other on the outer surface of the ring. The electrical strain gauges fixed on the ring were calibrated against the displacement between the support ends of the ring, and hence the strain in the geotextile, to allow an inference of the strain in the geotextile to be made from the observed strain in the strain gauges on the ring. The ring was calibrated by inducing known displacements between the supports using a micrometer arrangement (see Fig. 5) and then recording the corresponding readings of the electrical strain gauges. The ring gauges were protected with metal cover pans in the field.

The mechanical gauges monitored the physical movement (and relative movement) of seven points on the geotextile. Each reference point consisted of a 10 mm diameter bolt passed through the weave of the geotextile and fastened to two stainless steel plates ($80 \times 80 \times 1$ mm thick) on either face of the geotextile (see Fig. 6). A 3 mm thick, needle-punched nonwoven geotextile was placed between the steel plate and the woven geotextile, to provide a 'cushion' and protect the woven geotextile from indentation by the steel plate when the fastening bolt was tightened. A

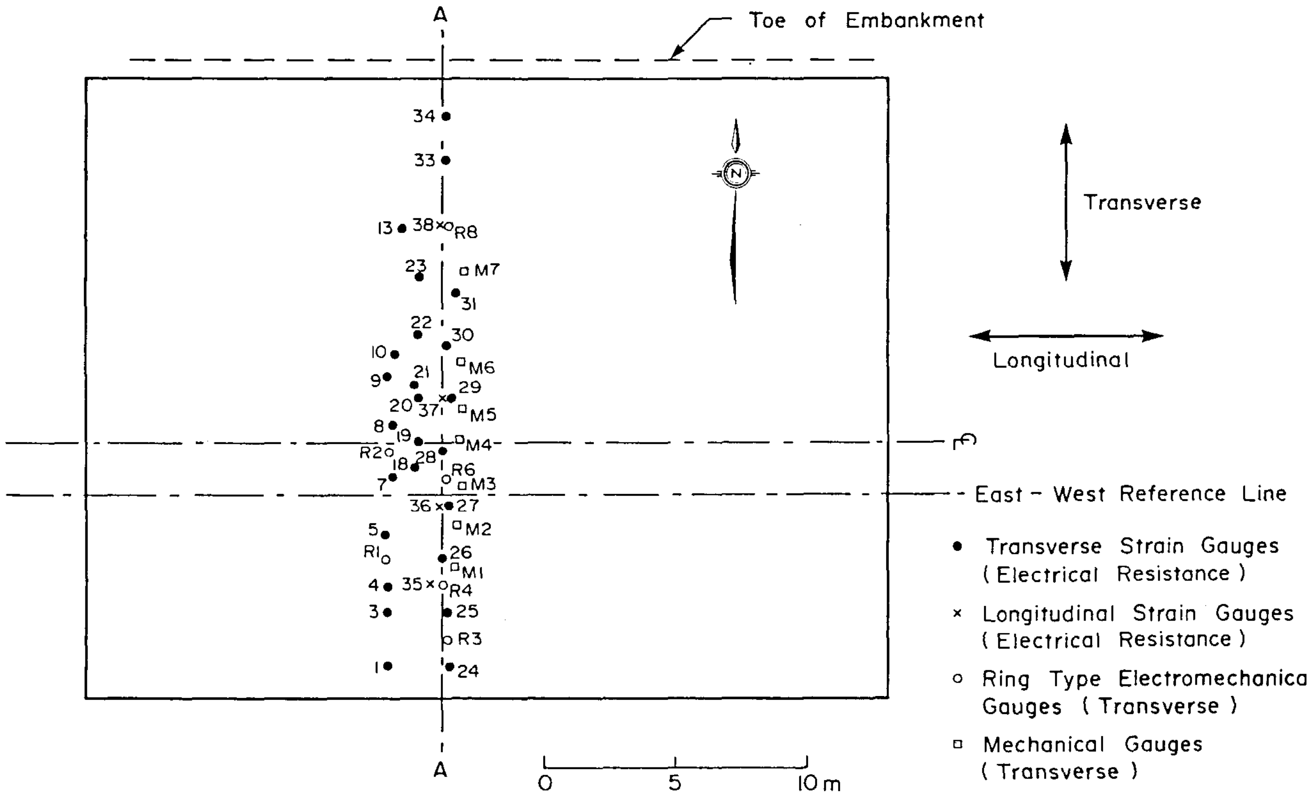



**Fig. 3.** Plan of geotextile reinforcement and layout of strain gauges.

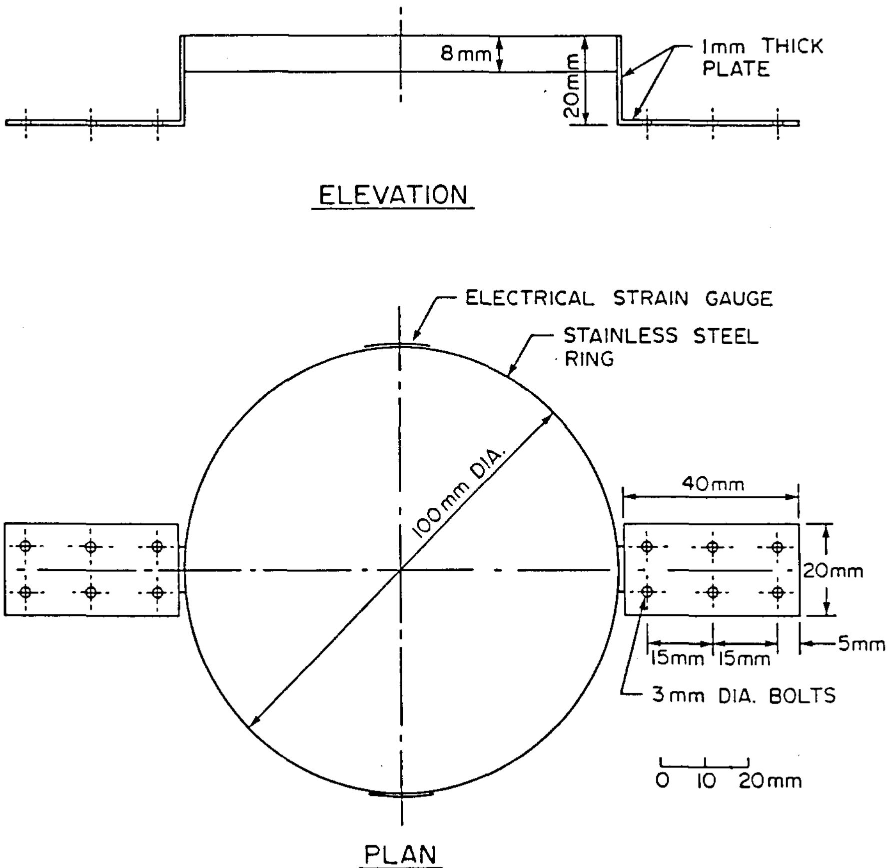



**Fig. 4.** Details of electromechanical strain gauge.

piano wire was securely fastened to the reference point and directed outside the embankment in the transverse direction. The other end of the wire was connected to a spring which was in turn connected to a fixed reference point outside the embankment. A small but consistent tension was applied to the wire with the use of this spring. The portion of the wire within the embankment was protected with a 20 mm diameter PVC tubing. A reference bead was mounted on the piano wire outside the toe of the embankment. Each reference point had a separate wire and a separate protective tubing. The physical displacement of the reference bead (i.e. the physical displacement of the reference point) was monitored with respect to the fixed reference point, well away from the embankment, during the entire construction and monitoring period, and the corresponding strains were inferred based on the initial length between the reference points. The absolute movement of the reference point was also monitored.

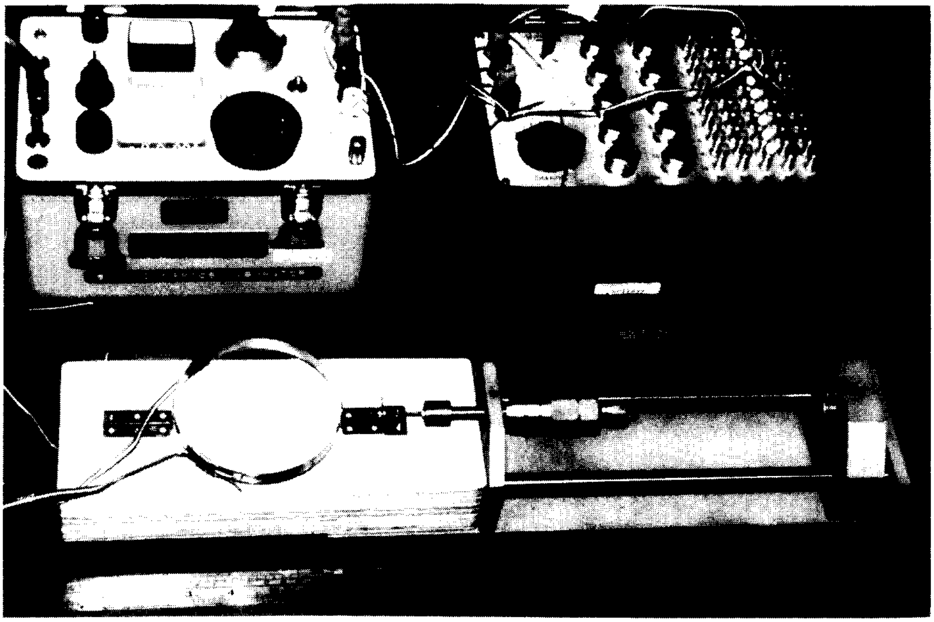


**Fig. 5.** Arrangement used for the calibration of electromechanical ring gauges.

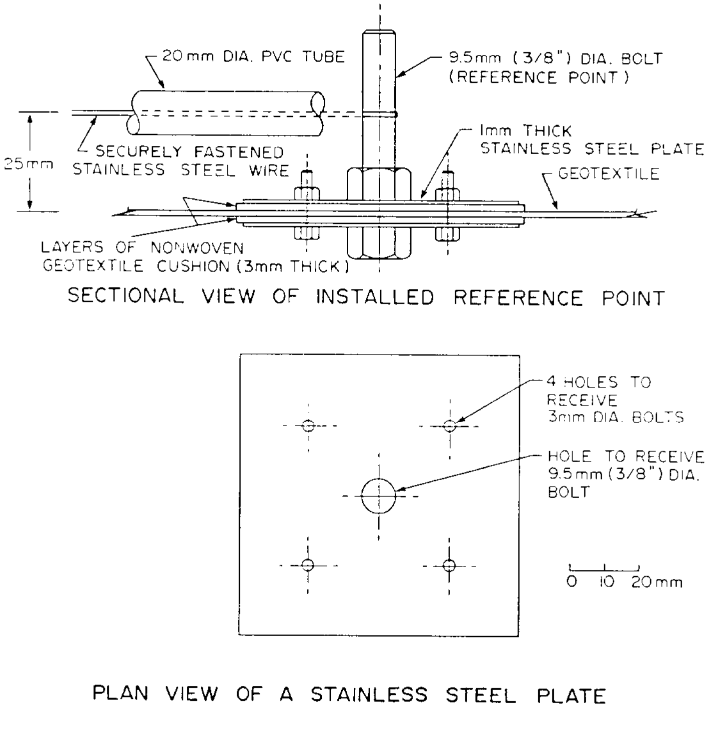



**Fig. 6.** Details of mechanical gauge reference point.

It is important to note that in the three types of gauges each provide an indication of strain over different gauge lengths. The electrical resistance gauges measure strain at a local point with a gauge length of 100 mm. The electromechanical gauges measure strain over a length of about 180 mm. The mechanical gauges monitor strain over a gauge length of between 1·1 m and 3·3 m, depending on which pair of monitoring points is being considered, and represent an average strain over the length being monitored.

## 3 COMMENTS ON INSTALLATION OF STRAIN GAUGE AND CONSTRUCTION CONSIDERATIONS

The reinforcement used on this project was Nicolon style 68300 multi-filament polyester woven geotextile with a unit weight of 631 $g\,m^{-2}$. The geotextile was factory sewn and delivered to a storage area close to the site. A typical tensile force–elongation plot obtained from a wide strip tensile test (Draft CGSB Standard, 1986) performed on a 200 mm wide sample of this geotextile is shown in Fig. 7. The average tensile strength properties of this geotextile, determined from five tensile tests, are summarized in Table 1. The elastic modulus reported in this table is the slope of the linear (offset) portion of the tensile force–elongation plot.

The electrical resistance strain gauges were installed on the geotextile, allowing only short lead wires and protected with Dow Corning 3145 RTV adhesive/sealant, under dry conditions in the storage area. All the precautions suggested by Sluimer and Risseew (1982) and Schimelfenyg *et al.* (1990) were followed during the installation process. The fastening plates for fixing the electromechanical ring gauges were also bolted to the geotextile prior to moving the geotextile to the site. The geotextile was transported to the site and placed on a 0·3–0·5 m thick granular 'working platform' which served to provide a level surface. The mechanical and electromechanical ring gauges were installed after the geotextile was placed in position at the site. Long lead wire electrical connections for the

**TABLE 1**
Stress–Strain Characteristics of the Geotextile

| | |
|---|---|
| Tensile strength | 216 $kN\,m^{-1}$ |
| Failure strain | 13% |
| Elastic modulus | 1 920 $kN\,m^{-1}$ |
| Initial modulus | 257 $kN\,m^{-1}$ |
| Secant modulus (0–5% strain) | 1 466 $kN\,m^{-1}$ |
| Secant modulus (0–10% strain) | 1 678 $kN\,m^{-1}$ |

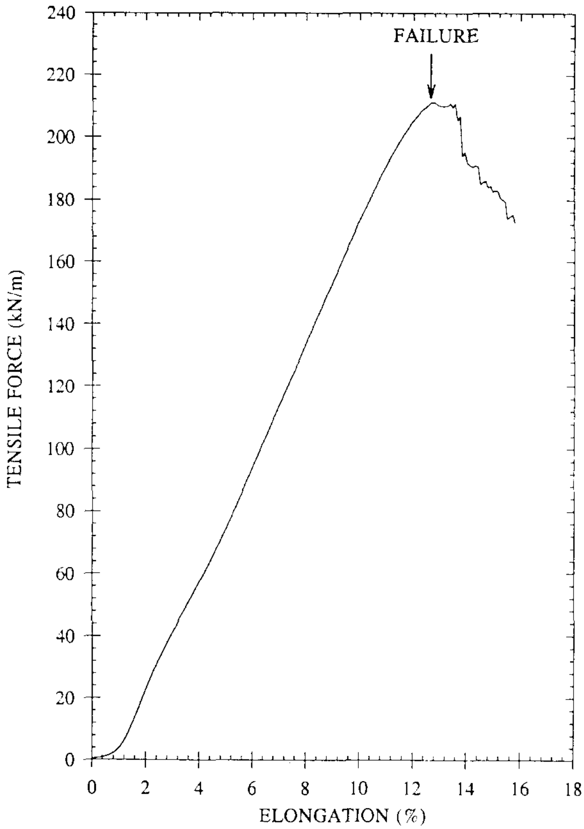



**Fig. 7.** A typical tensile force vs elongation plot.

electrical and electromechanical gauges were installed at the site by field soldering. Protective measures such as waterproofing of all the soldering connections with shrink tubing and applying waterproof coatings, encasing the lead cables of the mechanical gauges within plastic tubes, and the covering of electromechanical ring gauges with heavy duty metal cover pans were undertaken in a systematic manner to prolong the life of the gauges.

To accommodate large deformations, the lead wires of the electrical and electromechanical gauges were taken along zig-zag paths, in addition to providing extra lengths of wire on a zig-zag form closer to the gauges. The lead wires of the electrical and electromechanical gauges were directed to two data collection stations, one located close to inclinometer 23I and the other close to the inclinometer 26I (see Fig. 2), and passed through 100 mm diameter PVC pipes to the top of the embankment. These pipes were extended with the use of PVC couplings as the construction progressed.

A 0·4 m thick 'upper layer' of granular fill was carefully placed over the geotextile without allowing passage of either the trucks or the bulldozer directly on the geotextile. To avoid damage to the strain gauges, spreading of this granular fill directly over the gauges was performed manually for the first lift of fill.

The electrical and electromechanical gauges were monitored with Vishay strain indicator boxes. To facilitate fast reading, the lead wires of the strain gauges were connected through 'switch and balance units' (10 channels/strain gauges per unit) to the strain indicator boxes (one strain indicator box per switch and balance unit). Initial readings of all the strain gauges were recorded before placement of any fill over the geotextile.

## 4 GEOTEXTILE STRAIN DEVELOPMENT WITH TIME

The geotextile strain measurements will be examined in groups according to the distance of the group of gauges from the toe of the embankment. For the convenience of interpreting the data, the entire instrumented width of geotextile in the transverse direction (i.e. north–south direction) has been divided into several narrow bands parallel to the toe (or longitudinal axis) of the embankment (see Fig. 3 for location of gauges). Typical variations of geotextile strain as measured from different strain gauges with time are shown in Figs 8–12. The layout of strain gauges on the geotextile reinforcement is shown in Fig. 3. For ease of comparison

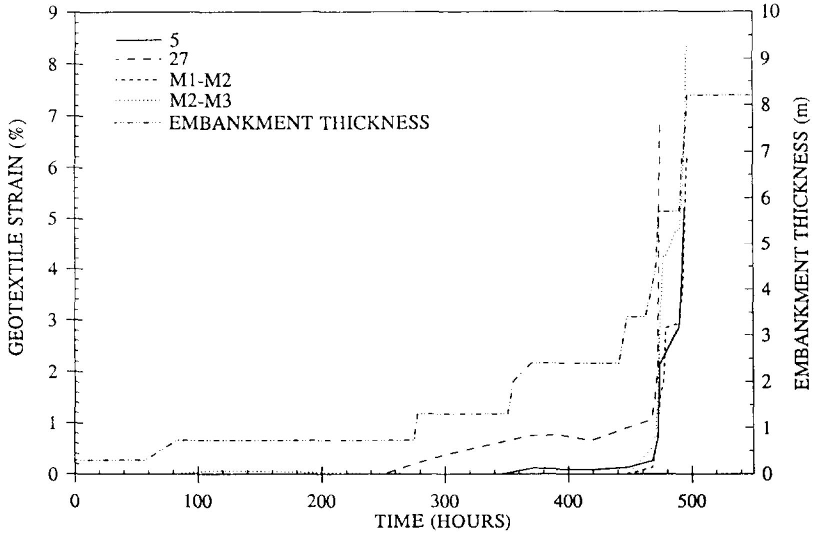



**Fig. 8.** Variation of geotextile strain with time for gauges 5, 27, M1–M2 and M2–M3.

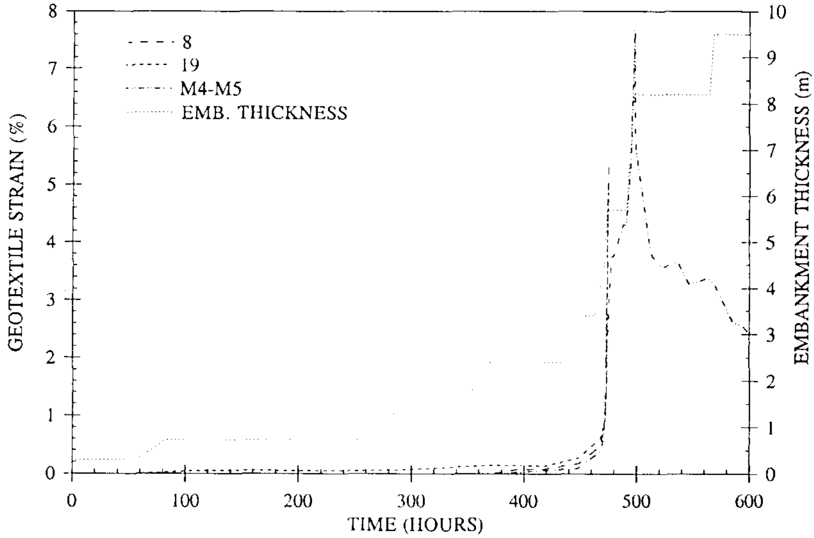
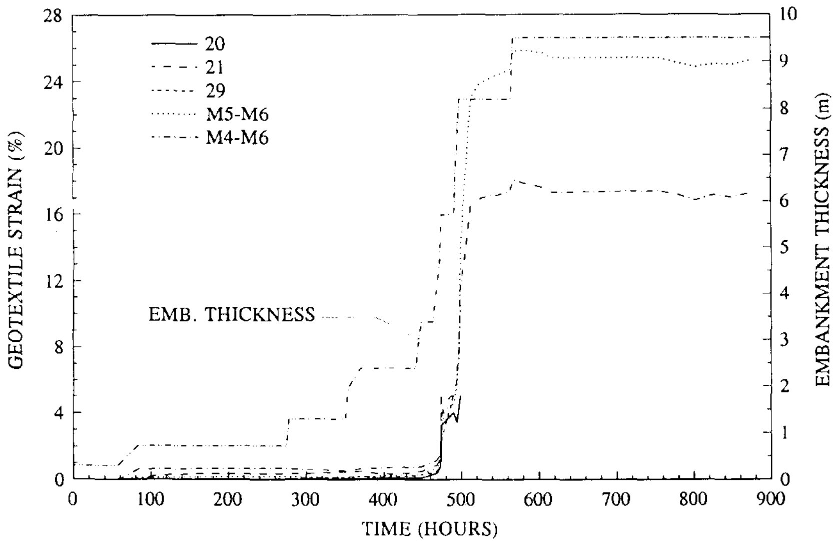



**Fig. 9.** Variation of geotextile strain with time for gauges 8, 19 and M4–M5.

with the construction sequence, the variation of the embankment fill thickness with time is also superimposed on these figures. Time (in hours) is measured from a nominal datum at 0 h on 21 September 1989. After placement of the geotextile on the 0·3 m working mat, placement of the overlying 0·4 m thick granular layer was completed at 83 h. As is evident from Figs 8–12, the strains either remained constant or experienced a small drop between 83–275 h (when the embankment was at a constant thickness of 0·7 m). This is considered to be due to self-adjustment of the geotextile eliminating the initial slackness and any small wrinkles induced during installation. During early stages of construction (up to about 3·4 m thickness) the foundation soil underwent some consolidation, as evidenced by the dissipation of pore pressures (Rowe *et al.*, 1995). It is hypothesized that this consolidation may have contributed to the slight decrease of strain at 0·7 m thickness.

The strains in the geotextile were comparatively small ($< 1{\cdot}3\%$), up until 448 h when the embankment reached 3·4 m thickness. The strain increased rapidly (in some cases to above 5%) when the embankment thickness was increased from 3·4 to 5·7 m between 448 and 475 h. Many of the electrical resistance strain gauges became defective when the embankment was constructed from about 5 to 5·7 m (between 472 and 475 h). Large horizontal and vertical displacements in the inclinometers and settlement monitoring points were observed during this construction phase, as reported by Rowe *et al.* (1995). It was not possible to monitor most of the electrical resistance strain gauges after 5·7 m thickness, prob-



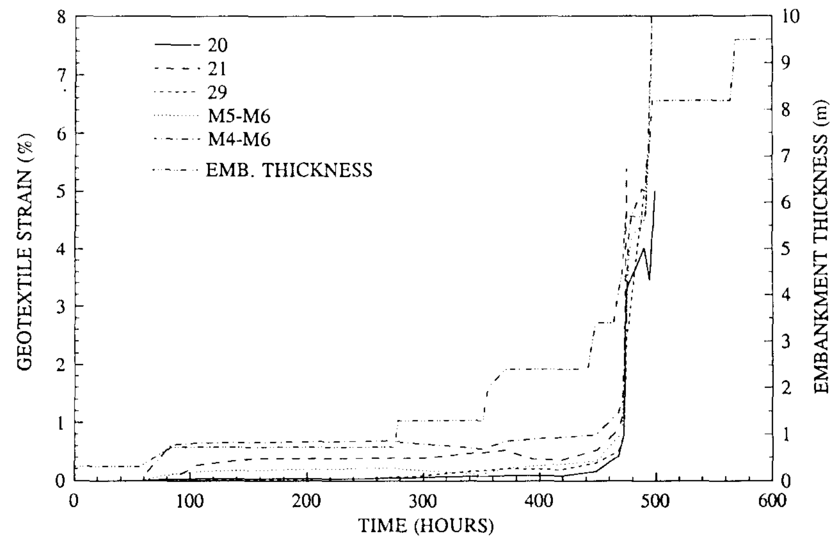




**Fig. 10.** (a), (b) Variation of geotextile strain with time for gauges 20, 21, 29, M5–M6 and M4–M6.

ably due to the damage caused to the cables by excessive movements. The development of strain at a number of key locations along the reinforcement is discussed below.

Figure 8 shows the strain–time response of gauges, 5, 27, M1–M2 and M2–M3 (see Fig. 3). Electrical resistance gauges 5 and 27 indicated small strains ($<0{\cdot}5\%$) during the construction of the embankment up to 3·4 m thickness. The mechanical gauge M1–M2 indicated some development of

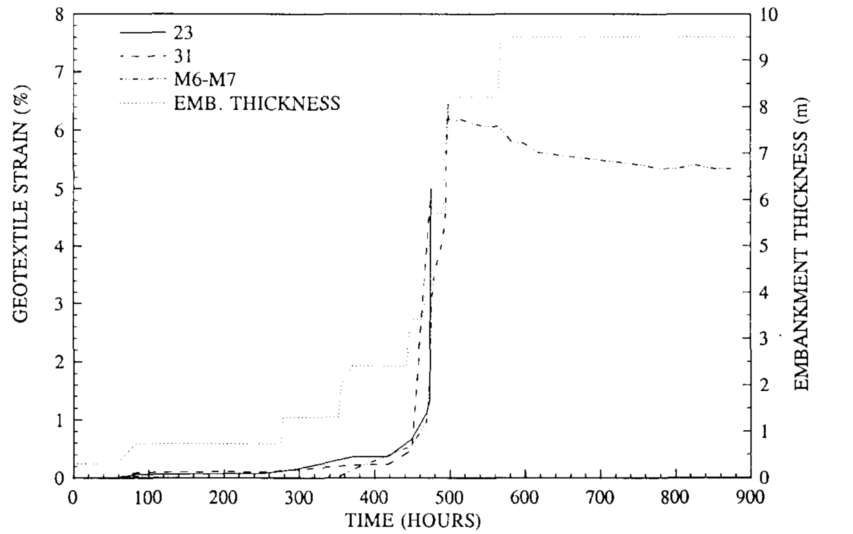



**Fig. 11.** Variation of geotextile strain with time for gauges 23, 31 and M6–M7.

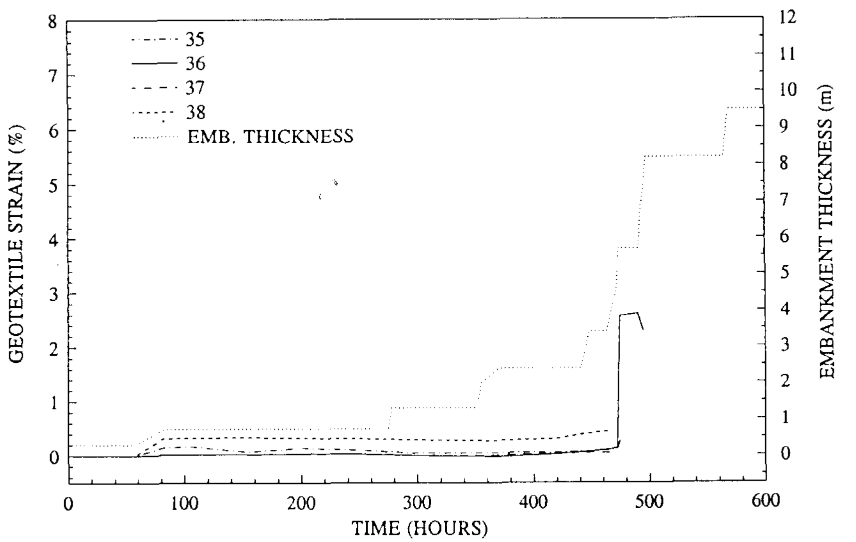



**Fig. 12.** Variation of geotextile strain with time for longitudinal gauges 35–38.

strain (up to 1%) during this period, much of which may have been due to taking up slack in the geotextile. In contrast, the strain in mechanical gauge M2–M3 was small and consistent with the electrical gauges. Between 448 and 463 h, as the embankment was maintained at a thickness of 3·4 m, there was a gradual increase in strain at all gauges. During this period there was negligible change in pore pressure, but the embankment did deform and this is reflected in the geotextile strain. As the embankment was constructed from 3·4 to 5·7 m, there was a rapid increase in strain at all gauges. At 490 h (5·7 m), the strain was about 3% based on electrical resistance gauge 5 and mechanical gauge M1–M2. Moving further north, electrical resistance gauge 27 gave a strain of about 6·8% before ceasing to function, while mechanical gauge M2–M3 gave a strain of about 4·8%. Thus, all gauges indicated significant mobilization of the geotextile when the embankment reached 5·7 m — at this fill-thickness the toe of the embankment had moved horizontally a distance in excess of 1·7 m. When the embankment was constructed to 8·2 m thickness, the strain indicated by mechanical gauges M1–M2 and M2–M3 increased to about 6·2% and 8·4%, respectively.

Figure 9 shows the strain–time response of gauges 8, 19 and M4–M5 located between 13·6 and 14·2 m from the toe of embankment. Electrical resistance gauges 8 and 19 and mechanical gauge M4–M5 indicated strains of less than about 1% until 471 h (4·5 m embankment thickness). Both electrical gauges indicated a rapid increase in strain to about 5% when the embankment thickness was increased to 5·7 m but then ceased to function. The mechanical gauge M4–M5 responded more slowly but nevertheless showed a significant increase from about 0·7% strain at 4·5 m thickness to about 4·3% at 5·7 m (490 h). Further rapid increase in strain (from 4·3 to about 7·7%) was observed in this gauge between 490 and 497 h, during which period the fill thickness had been increased from 5·7 to 8·2 m. A rapid decrease of strain was recorded in this gauge afterwards; this is considered to be due to the movement of the monitoring point M5, caused by the yielding of the geotextile between M5 and M6, as discussed later in this paper. It should be noted that the decreasing trend continued even when additional fill was placed between 564 and 568 h to increase the thickness from 8·2 to 9·5 m.

All the gauges in the region between 11·8 and 12·6 m from the toe of embankment (i.e. gauges 20, 21, 29 and M5–M6 — see Fig. 3) indicated strains of less than about 1·4% up until 472 h (i.e. 5 m thickness) followed by a rapid increase of strain (see Fig. 10). The strain in gauge 21 increased rapidly from about 1·2 to 5·4% between 472 and 475 h when the fill thickness was increased from 5 to 5·7 m and this gauge ceased to function after 475 h. Gauge 29 indicated a rapid increase of strain from about 0·8 to 2·4% during the same period (i.e. between 472 and 475 h) followed by a continuous increase in strain (up to about 5%) as the embankment was maintained at 5·7 m thickness up until 490 h; gauge 29 ceased to function after 490 h. Gauge 20 indicated a rapid increase of strain from about 0·7 to 3·3% between 472 and 475 h during which period the embankment thickness was increased from 5 to 5·7 m. This gauge also indicated an increase of strain from 3·3 to 4% between 475 and 490 h when the fill thickness was constant at 5·7 m. It indicated a drop of strain from 4 to 3·5% between 490 and 495 h (i.e. during the construction from 5·7 to 7·5 m), a rapid increase of strain to about 5% when the fill thickness was increased to 8·2 m (497 h) and it could not be monitored after 499 h (8·2 m thickness). This drop of strain may be attributed to the yielding of the geotextile in the nearby region, as discussed later in this paper.

The mechanical gauge M5–M6 indicated a rapid increase of strain (from about 1·4 to 2·5%) between 472 and 475 h during which period the embankment was constructed from 5 to 5·7 m thickness. Although the fill thickness was constant at 5·7 m between 475 and 490 h the deformations continued and were accompanied by an increase of strain up to about 4·6%. The strain in this gauge increased from 4·6 to about 8·5% between 490 and 497 h (i.e. during the construction of the embankment from 5·7 to 8·2 m thickness). A large increase of strain (from about 8·5 to 13·9%, see Fig. 10(a)) was recorded in this gauge between 497 and 498 h followed by a very large increase of strain (from 13·9 to 23% between 498 and 512 h) while the thickness remained constant at 8·2 m; it would appear that the geotextile tore/yielded at about 497–498 h.

The strain–time responses of gauges 23, 31 and M6–M7 in the region between 8·15 and 9·5 m from the toe of embankment are shown in Fig. 11. All these gauges indicated small strains (< 0·7%) until 488 h (i.e. 3·4 m fill thickness) followed by a rapid increase in strain to about 5% between 488 and 475 h (i.e. when the fill thickness was increased from 3·4 to 5·7 m). Neither of the electrical resistance gauges 23 or 31 could be monitored after 475 h (5·7 m thickness). The strain in the mechanical gauge continued to increase from 2·4 to 3·9% between 475 and 490 h when the fill thickness was constant at 5·7 m. This gauge indicated a rapid increase in strain (from 3·9 to 6·4%) between 490 and 497 h when the fill thickness was increased from 5·7 to 8·2 m. A significant drop in the strain was observed in this gauge between 497 and 498 h which was followed by a continuous decease in strain during the brief stoppage of construction (at the reinforced embankment section), between 497 and 564 h (8·2 m thickness), as well as afterwards (including the period of further addition of fill to increase the thickness from 8·2 to 9·5 m between 564 and 568 h). However, it indicated a relatively constant strain of about 5·3% after 755 h. It was noted earlier that the strain readings in mechanical gauge M4–M5 decreased rapidly after the embankment was constructed to 8·2 m thickness (i.e. after 497 h, see Fig. 9). A very large increase of strain, from about 8·5 to above 23% (much higher than the 13% failure strain observed during laboratory tensile tests, see Table 1), was recorded in gauge M5–M6 between 497 and 512 h (see Fig. 10). The responses of gauges M5–M6, M4–M5 and M6–M7 are all consistent with the interpretation that the geotextile tore/yielded in the region between M5 and M6 (i.e. near M5). This tear is consistent with the magnitude of strain observed between M5 and M6 (relative to the failure strain in the geotextile). The reduction in strain in the geotextile on either side of M5–M6 (i.e. M4–M5 and M6–M7) is also consistent with the expected form of stress redistribution if the geotextile tears. Note that the inferred circular type (primary) failure surface passes through M4 (or near M4, see Fig. 13) and it is likely that there is a (shear) failure zone surrounding this failure surface which could extend between M3 and M5. It was reported that this reinforced embankment failed at a thickness of about 8·2 m and the failure was of plastic or visco-plastic type (see Rowe *et al.*, 1995). Since it was possible to increase the fill thickness up to 9·5 m thickness without any abrupt failure (i.e. dramatic collapse) of the embankment, it appears that even with tearing of the geotextile close to M5 (i.e. at a distance of about 13 m from the toe), there was a stress redistribution between the soil and geotextile, although the displacement increased considerably and it was not possible to subsequently increase the net height of the embankment above the 6·6 m achieved just prior to the failure of the geotextile.

Figure 12 shows the strain–time responses of the longitudinal gauges. Gauges 35, 37 and 38 all indicated low strain ($< 0{\cdot}5\%$) until they ceased to function at about 475, 472 and 468 h (i.e. 5·7 m, 5 m and 4·1 m thickness), respectively; the failure of the gauge was attributed to damage to the wires as a result of the large transverse movements. The small longitudinal strains suggest that plane strain conditions are approximated in the instrumented mid-zone (i.e. the middle 4 m portion) of the 25 m long reinforced embankment, during its construction up to 5·7 m thickness. Gauge 36 also indicated very low strain ($< 0{\cdot}15\%$) until 475 h (i.e. 5·7 m thickness) but showed an increase to about 2·6% afterwards. However, it was not clear whether this apparent increase was due to longitudinal strain in the geotextile or was due to damage to the electrical connections and/or cable, caused by large transverse movements.

## 5 COMMENTS ON THE VARIATION OF GEOTEXTILE STRAIN WITH EMBANKMENT THICKNESS

A typical variation of geotextile strain (in the transverse direction, as measured from various gauges) with embankment thickness is presented in Fig. 14. This figure indicates the strain, obtained from different types of gauges (i.e. electrical, mechanical and/or electromechanical ring gauges) installed in the narrow band of geotextile between 16·6 and 17·6 m from

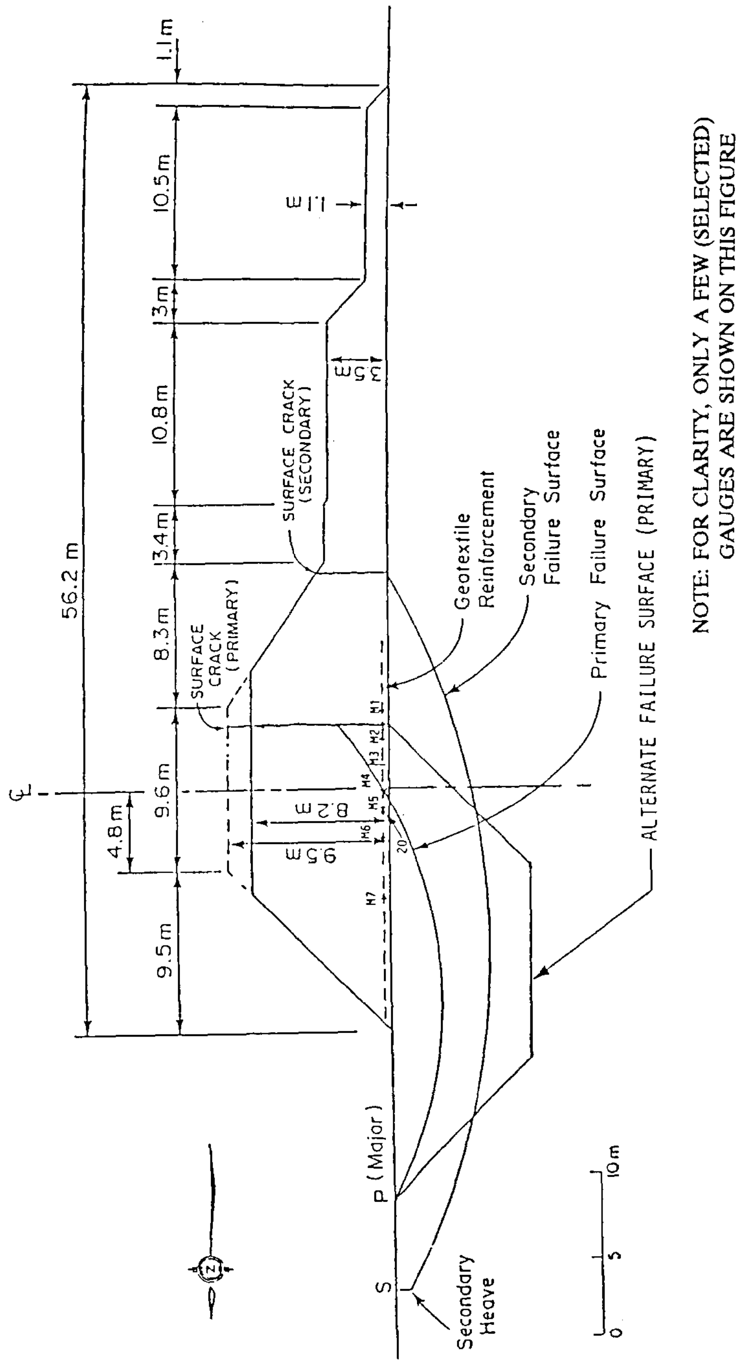



**Fig. 13.** Inferred failure surfaces of the reinforced embankment and the location of mechanical strain gauges.

the toe, vs the embankment thickness. The mean (i.e. the average) as well as the lower and upper limits of the strain readings are presented separately so that the range of measured geotextile strain at different embankment thicknesses could be interpreted. Most of the electrical resistance and electromechanical ring gauges could not be monitored during the later stages of construction (especially after 5·7 m thickness), as discussed previously, and the limited available data after this stage are presented either as an average, minimum or maximum, depending on the amount of data and the magnitude of the data from the electromechanical and mechanical gauges relative to the electrical gauges prior to failure of the latter.

The variation in strain with thickness (see Fig. 14, for example) was similar both before and after the brief stoppage of construction at 5·7 m thickness (i.e. between 475 and 490 h). This continuation of the same trend suggests that the sequence of construction employed at the site did not significantly influence the overall behaviour of the reinforced embankment. Significant increases of the slope of the strain vs thickness plots were observed between 3·4 and 5 m (e.g. at about 4·1 m thickness for the gauges shown in Fig. 14), indicating that the embankment apparently started to move (i.e. the foundation yielded) when the thickness was about 3·4 m. Additional strain data are given in Figs 15–20. The geotextile strains were comparatively small (the average was less than about 0·72%) up to an embankment thickness of 3·4 m. The average value of strain

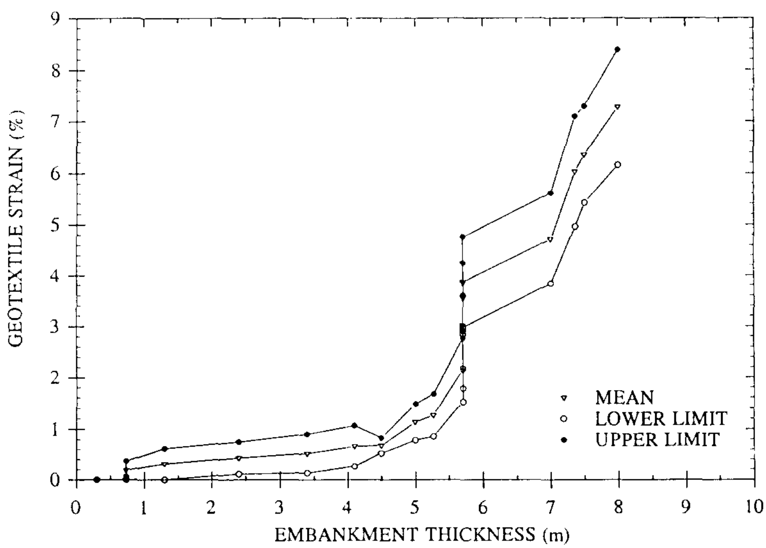



**Fig. 14.** Variation of geotextile strain with embankment thickness for gauges between 16·6 and 17·6 m from embankment toe.

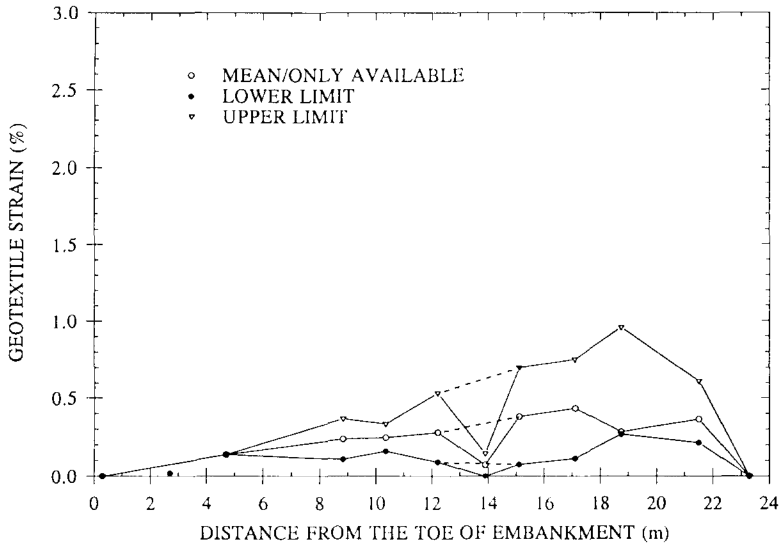



**Fig. 15.** Geotextile strain distribution at embankment thickness = 2·4 m (372 h).

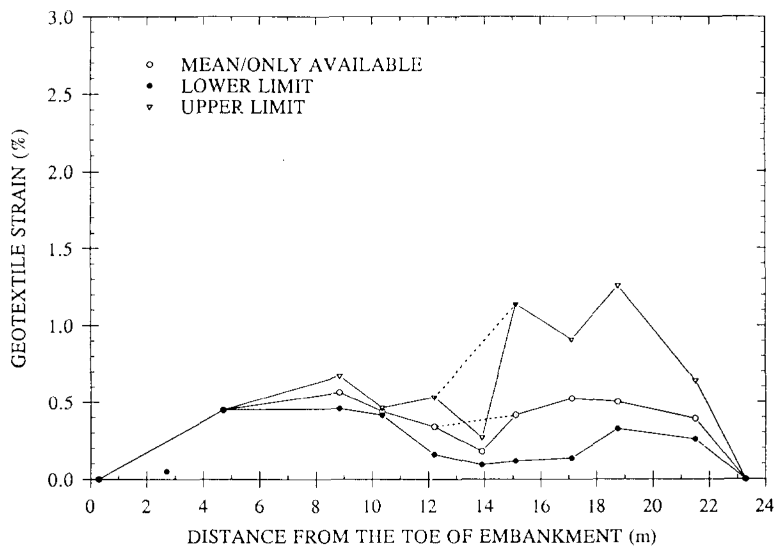



**Fig. 16.** Geotextile strain distribution at embankment thickness = 3·4 m (448 h).

increased to about 1%, 2% and 3% when the embankment thickness was increased to 4·1 m (at 468 h), 5·0 m (at 472 h) and 5·7 m (at 475 h), respectively. It would appear that the geotextile did not contribute significantly to the stability of the embankment up to about 4·1 m thickness, but that this contribution increased gradually above 4·1 m thickness. The large

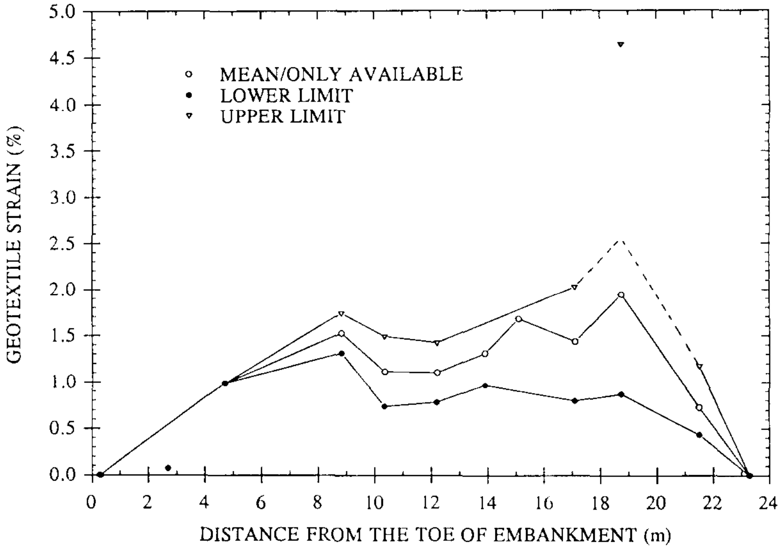



**Fig. 17.** Geotextile strain distribution at embankment thickness = 5·0 m (472 h).

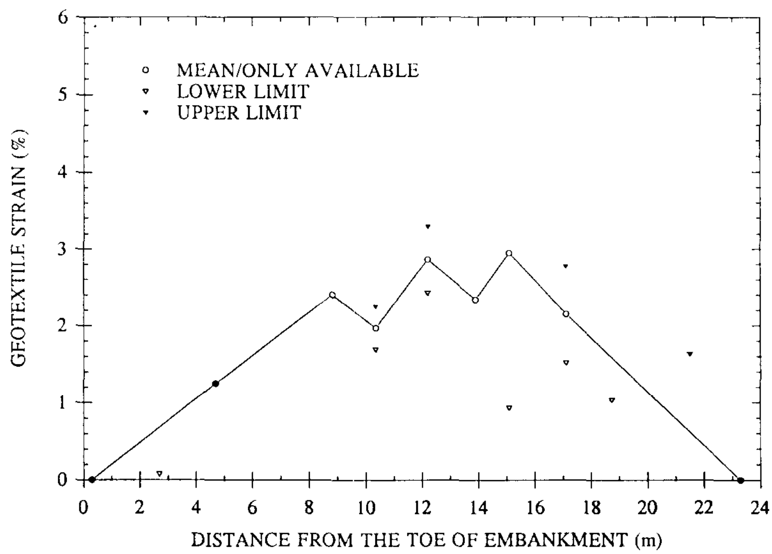



**Fig. 18.** Geotextile strain distribution at embankment thickness = 5·7 m (475 h).

increase of the average value of strain, from about 3% to 5%, at 5·7 m thickness suggests that the soil approached failure at about 5·7 m thickness. The excess pore pressure and both vertical and horizontal displacement responses also indicated that the soil approached failure at about 5·7 m thickness (see Rowe *et al.*, 1995). The strain increased to over 8·5%

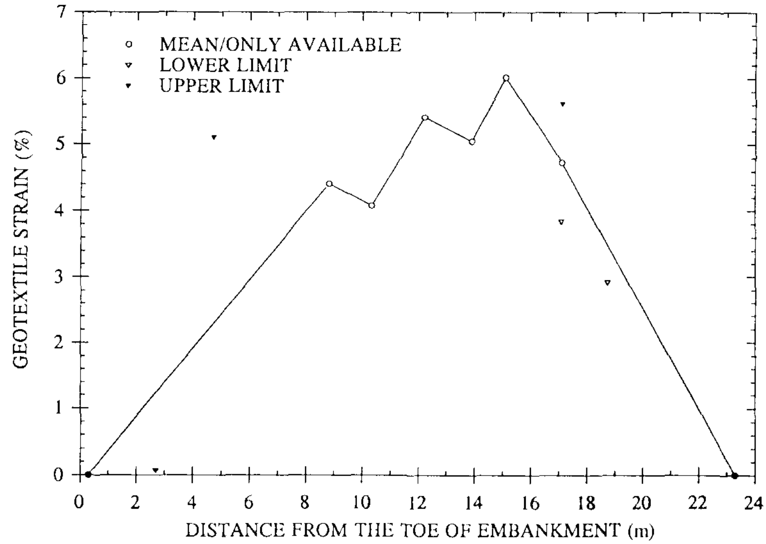



**Fig. 19.** Geotextile strain distribution at embankment thickness = 7·0 m (493 h).

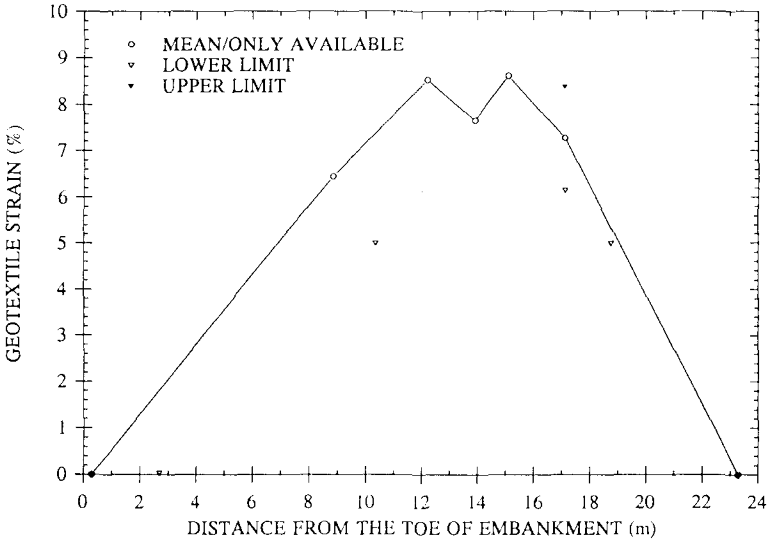



**Fig. 20.** Geotextile strain distribution at embankment thickness = 8·2 m (497 h).

when the embankment was raised to 8·2 m thickness. It is apparent that the role of the geotextile in providing stability to the embankment increased significantly after 5·7 m thickness. Rowe *et al.* (1995) suggested that the construction of the embankment above 5·7 m thickness was possible only due to the influence of the geotextile and the strain data also show some evidence in support of this conclusion.

## 6 GEOTEXTILE STRAIN DISTRIBUTION

The variations of geotextile strain along the transverse (i.e. north–south) direction of geotextile reinforcement (i.e. the strain distribution profile) at different stages of construction are presented in Figs 15–20. The average and both upper and lower limits of the strains, inferred from the strain vs thickness data, are plotted separately to facilitate interpretation of the range of strain at each location along the (north–south) centre line of the geotextile at different stages of construction. It should be noted that the strain distribution shown in Fig. 20 was when the embankment thickness was increased to 8·2 m (i.e. 497 h) and the maximum strain in the geotextile increased rapidly to about 13·9% in about 0·8 h while the thickness was constant at 8·2 m (see Fig. 10(a) also).

Figures 15 and 16 clearly indicate that the strains were less than about 1·3% when the embankment was constructed up to 3·4 m thickness. At 5 m thickness, the largest strain (of about 4·7%) was observed about 18·75 m from the embankment toe (see Fig. 17). This value comes from the strain readings of rings 1 and 4 placed in the region 18–19·5 m from the toe. The trend of a sharp increase of strain from about 2% (at about 17·1 m from toe) to about 4·7% (at about 18·75 m from toe) and the sharp drop to about 1% (at about 21·5 m from toe), appears erroneous and it is the authors' opinion that the largest strain of 4·7% is not realistic. The inferred largest strain (assessed by extrapolating the trends in the neighbouring regions) is expected to be about 3% (see Fig. 17).

A sharp drop of strain in the neighbourhood of 13·9 m (from the toe) is observed in the strain profiles, particularly during the early stages of construction (i.e. up to 3·4 m embankment thickness, see Figs 15 and 16), indicating a clear abrupt deviation from the trend exhibited in the neighbouring regions. These drops were due to the comparatively small strains observed in gauges 8, 19 and M4–M5 placed between 13·6 and 14·2 m from the toe during the early stages of construction. It is suspected that the geotextile in this region was subjected to some local pretension strain of about 0·1–0·4% (the range estimated, from Figs 15 and 16, as the difference between the expected strain for continuation of the same trend as the surrounding regions and the obtained strain readings), resulting in a zero shift, and was the cause of the lower strain readings in these gauges. It should be noted that this zero shift is small compared to the strains obtained during the later stages of construction (say above 5 m thickness) and does not significantly affect the strain data of later stages of construction.

The strain profiles indicate that the maximum strain occurred between about 17 and 19 m from the toe when the embankment thickness was at or below 3·4 m (see Figs 15 and 16). The location of maximum strain along the geotextile shifted towards the centre line of the embankment to between about 12 and 15 m when it was raised above 5·7 m thickness (see Figs 18–20).

## 7 COMMENTS ON THE PERFORMANCE OF INSTRUMENTATION

The geotextile strain measurements proved to be very successful. Of the original 38 electrical gauges, five were lost during transport and placement of the geotextile. Given the weight and bulky nature of the geotextile, the nature of the gauges and the harsh environment, this is a low failure rate. Out of the remaining 33 gauges, 29 of them continued to function until an advanced state of failure (i.e. when the gauge or the cable was damaged due to large deformation). Five of the ring gauges also functioned reasonably well until an advanced stage of failure. The mechanical gauges performed very well and provided useful data during the entire construction and monitoring period. Five of the mechanical gauges continued to function until the embankment failed.

## 8 SUMMARY AND CONCLUSIONS

A relatively high strength ($216\,kN\,m^{-1}$) polyester woven geotextile was used as a reinforcement in a test embankment constructed to failure on soft organic clayey silt at Sackville, New Brunswick, Canada. The geotextile was instrumented with a number of electrical, electromechanical and mechanical gauges. Details regarding the installation of these gauges, their responses during embankment construction and their performance have been presented in this paper.

The strains were comparatively small (typically less than about 0·72%) up to an embankment thickness of about 3·4 m. The strain increased to about 1, 2 and 3% when the embankment thickness was increased to 4·1, 5·0 and 5·7 m, respectively, suggesting significant movement (or yielding) of the foundation soil during the construction of the embankment above 4·1 m. A large increase of strain from about 3 to 5% was evident at 5·7 m thickness, suggesting that the soil approached failure at about 5·7 m thickness. However, the embankment did not fail and additional fill could still be placed to achieve increased height above the surrounding area. The maximum strain occurred between about 17 and 19 m from the toe, when the embankment thickness was below 3·4 m, but shifted towards the centre line of the embankment to between about 12 and 15 m when it was raised above 5·7 m thickness.

This field investigation indicates that the contribution of the geotextile to the stability of the embankment was not significant up to about 4·1 m thickness, but that its contribution increased gradually after about 5 m thickness. The strain increased to about 8·5% when the embankment was first raised to 8·2 m thickness and then continued to increase with time until failure of the geotextile appeared to have occurred while the embankment remained at 8·2 m. It was apparent that the role of the geotextile in providing stability to the embankment increased substantially after 5·7 m thickness. Rowe *et al.* (1995) concluded that the construction of the embankment above 5·7 m thickness was possible only due to the influence of the geotextile, and the reinforced embankment failed at a thickness of about 8·2 m (i.e. at a net height of about 6·6 m). The strain data presented in this paper also provide evidence in support of this conclusion. The general response of this reinforced embankment followed the sequence predicted by Rowe and Soderman (1987*a*). However, the level of 'undrained creep' of the foundation and its consequent effect on geotextile strain is greater than has been previously recorded in the literature for reinforced embankments. This aspect of reinforced embankment behaviour requires additional study.

## ACKNOWLEDGEMENTS

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