GEOTEXTILE REINFORCEMENT OF A DEEP MUSKEG DEPOSIT

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ABSTRACT

The construction of 4 to 5 m high embankments across a muskeg deposit varying from 4 to 6 m in thickness provided an opportunity to undertake a field evaluation of geotextile reinforcement as well as a general evaluation of embankment performance on muskegs. Performance evaluation was aided through porewater pressure, lateral deformation, settlement, and geotextile deformation monitoring, along with post construction evaluation of settlement. The results of this study provided interesting findings which have since been implemented by Alberta Transportation and Utilities. Perhaps, one of the more important findings was that geotextiles are not absolutely necessary for construction of stable embankments on muskegs. This reinforces past and present experiences which utilize a stage construction approach for embankment construction.

Notwithstanding the findings of this study, geotextiles are regarded as beneficial for stabilization of failures, enhancing construction trafficability on waterlogged sites, and reinforcing sites where the soils underlying the muskegs are very weak.

INTRODUCTION

Alberta Transportation and Utilities began utilizing geotextiles around 1977 as reinforcement for muskegs and soft ground where stability problems were perceived at the design stage or when such problems, occurring during the construction and maintenance phases, were perceived to be best solved using this material type.

The urge to utilize geotextiles was also prompted by the increasing popularity of this product and the ever increasing sales pressures from suppliers and manufacturers to try innovative materials and techniques in highway construction. In this initial phase, the Department's use of geotextiles was generally one of experimentation with a new product with the overall objective to ascertain its usefulness.

Two notable projects where geotextiles were used with some degree of success was SR 651:02 and SR 918:02 both located in Northern Alberta. These projects were reported by Diyaljee and Comchi (1985) and Diyaljee, et al (1986). On both projects, geotextile was utilized during the construction phase when ground instability problems occurred. Both non-woven and woven geotextiles were used, the former on the SR 651:02 project and the latter on the SR 918:02 project.

In 1984–1985, there was a gradual but positive approach to utilize woven geotextiles for reinforcement to make use of their tensile strength and stiffness characteristics. Also, it was observed that prices of both types of geotextiles were becoming comparable and in some instances woven geotextiles could be obtained cheaper than non-woven types.

In 1986, it was felt that there was a need for the Department to determine requirements for geotextiles in the light of frequent requests for its use on construction projects. Some of the questions to be answered were as follows:

1. Was geotextile really required for reinforcement of embankments on muskeg especially when, in the past, construction was accomplished without the use of this material?

2. What was the difference in performance between non-woven and woven geotextiles when used in muskeg construction?

3. What minimum material characteristics were required for use on muskegs? It was felt that a cheap, low strength, low modulus woven fabric would be sufficient since most embankments constructed in the traditional manner performed satisfactorily and problems only occurred when the material underlying the muskeg was very weak.

An attempt was made to answer some of these questions during the twinning of Hwy. 16 through the Town of Entwistle. In this project, the proposed route was to traverse a deep muskeg deposit which had been crossed on the existing roadway in the conventional manner of "floating" the embankment directly on the muskeg. Although the same technique could have been utilized it was decided that this stretch was useful for undertaking some field research.

In addition, the opportunity was also taken to assess whether the time for stage loading could be defined since this was generally arbitrarily chosen. The need to determine the minimum waiting period was thought to be important in light of the present trend of fast-track construction.

The purpose of this paper is to present the findings of a field study undertaken to assess the effects of geotextiles on embankment performance, and the performance of embankments on muskegs, in general. The assessment is aided through evaluation of pore pressure monitoring and evaluation, settlement monitoring, geotextile instrumentation, slope inclinometer monitoring, and engineering judgement based on visual observations.

SITE LOCATION AND DESCRIPTION

The site is situated along the Yellowhead Highway (Hwy 16) and located between East of the Town of Entwistle and the Pembina River Bridge approximately 108 kilometres west of the City of Edmonton. The Yellowhead Highway is a major east-west link to the four Western Provinces and is used extensively for recreational as well as commercial travel. The existing Hwy 16 through Entwistle was a 2-lane highway with a finished surface width of 12.8 m. Four laning this section of highway was undertaken as part of the Yellowhead Highway twinning from the Saskatchewan Border to Jasper. Prior to 1986 about 125 kilometres of this 560 kilometre route was already twinned.

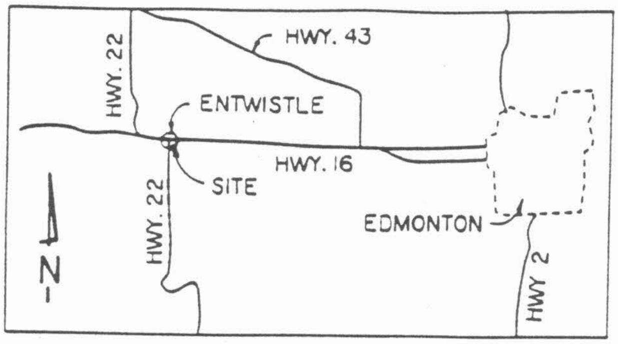

FIG. 1. Location of Site

Figure 2 shows the muskeg section which extends from STA 32 + 200 to STA 32 + 520 on the westbound lane and between STA 3 + 160 and STA 3 + 500 on the north-west service road. The muskeg deposit runs in a north-south direction and crosses the existing highway (referred to as the eastbound lane) constructed in 1962.

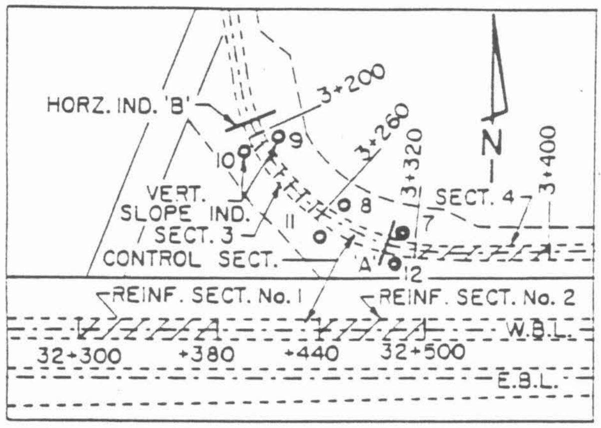

FIG. 2. Plan showing details of Test Sections

Ground topography along the westbound lane and service road was generally flat with drainage towards the service road. The westbound lane was bare of vegetation and contained only very short grasses. Presumably, this area was cleared during the construction of the eastbound lane since the fence line demarcated the area from thick vegetation through which the service road alignment was established.

The alignment of the service road crossed willows and woody vegetation as well as tall swamp grasses. The site was waterlogged and could only be traversed by using hipwaders in contrast to the westbound lane which was firm and easily traversed by foot. The firm surface of the westbound route might have been due to earth spillage during the eastbound lane construction.

SUBSOIL CHARACTERISTICS

General

The geotechnical investigation consisted of determining the depth of muskeg along the two routes, nature of soils underlying the muskegs, and subsoil characteristics along the existing eastbound lane. The latter was judged to be the best guide in determining the settlement characteristics of the muskeg along the proposed routes.

Six test holes were drilled using a B-61 rig equipped with hollow stem augers. Two of these test holes were done, one each, within the eastbound lane and service roads. Testing on the service road was limited since the land was not cleared at the time of the investigation.

Westbound Lane and Service Road

The subsurface stratigraphy encountered in both the westbound and service road alignments consisted of muskeg varying in thickness from 4 to 6 m, overlying silty clay 2 to 3 m thick varying from soft to stiff with depth. Below the clay, hard sandstone was encountered which was penetrated 0.6 m before hole termination.

The muskeg was fibrous and woody. Moisture contents ranged from 620 to 712%. Shear strengths, using a 50 mm x 100mm vane, varied from 5 to 10 kPa with some values smaller than 5 kPa. These values are within the range reported by Anderson (1962).

Oedometer testing on 75 mm diameter samples gave an initial void ratio of 6.56 and coefficient of compressibility of 3.

The silty clay ranged in moisture content from 35 to 44% with a liquidity index of 0.5 on average. Shear strength from unconfined compression testing varied from 10 to 25 kPa in the zone immediately below the muskeg to 78 kPa above the sandstone.

Eastbound Lane

The subsoil stratigraphy in the eastbound lane was determined from a test hole at sta 32+410 done on the shoulder of the roadway. Here, the muskeg was found to be 3 m thick under an embankment fill of 5.6 m. Moisture content of the compressed muskeg varied from 292% at the fill/muskeg interface to 358% at a depth of 7.6 m reducing to 291% at 8.10 m and 114% at 8.50 m. Shear strength of the muskeg in unconfined compression varied from 27 to 88 kPa without any distinct trend in relation to moisture content e.g., strength increasing with decreasing moisture content.

Settlement of the muskeg was determined over the 24 year period after construction by comparing the subsoil profile at STA 32+413, 30 metre left of median centreline, where the virgin muskeg thickness was found to be 6.25 m, with the profile at STA 32 + 410. From this comparison, the settlement under the eastbound lane was determined to be about 50% of the original muskeg thickness.

DESIGN CONSIDERATIONS

Design cross-sections for the westbound lane and service road are shown in Figures 3 and 4. These sections illustrate geotextile reinforcement and locations of piezometers. The proposed gradeline across the muskeg sections indicated 5 m average fill on the westbound lane and 4 m average fill on the service road. These fill heights were considered acceptable using a stage loading sequence since the eastbound lane was constructed on the same deposit under a similar loading and no failure was reported during or after construction.

The construction method to be employed was to "float" the embankment directly on the muskeg using 2.5 m maximum fill height for the initial stage with subsequent stages not exceeding 2 m. In general, a 4 week shutdown period was stipulated after stage 1 and any subsequent stage. However, it was known that this period could be varied depending on pore pressure response.

The entire embankment was to be constructed with clay fill to be obtained from cut areas and borrow sources adjacent to the site. A granular drainage layer between the muskeg and fill, contemplated during initial design, was subsequently eliminated. The clay fill was essentially a CI-CL clay.

TEST SECTIONS

The muskeg stretch along each of the two roadways was divided into six test sections varying from 60 m to 80 m in length per section. Four of these sections were geotextile reinforced consisting of two reinforced sections along each roadway. The remaining two sections were non-reinforced and located between reinforced sections. The layout of the test sections is shown on Figure 2.

The geotextiles planned to be used for reinforcement were P500X, P600X, HP1000 and HP1200, all woven Mirafi products. For competitive bidding, generic specifications were written. This resulted in the GTF 500 and GTF 800, both from Exxon, and P500X and P600X, both from Mirafi, being accepted to satisfy the specification requirements. The characteristics of these materials are summarized in Table 1.

Table 1: Geotextile Properties

Properties	**P500X**	**P600X**	**GTF 500**	**GTF 800**
Polymer Structure	Polypropylene Woven		Polypropylene Stichbonded woven	
Tensile Strength[1] (kN/m)	26/15	35/28	77/70	103/89
Tensile Elongation[2] (°/o)	20/20	20/15	20/15	20/20
Mullen Burst (MPa)	2.8	4.2	10.3	10.3
E.O.S. (mm)	0.54	0.54	0.14	0.14

[1] Warp/Fill
[2] Warp/Fill
Tensile Strength determined from 100 mm x 200 mm sample, 75 mm gauge length, 25 mm x 50 mm long grip.

INSTRUMENTATION

Piezometers

Petur type piezometers were installed in both the service road and westbound lane in the muskeg as well as the underlying soft silty clay. Ten piezometers were installed in the westbound lane between STA 32 + 340 and STA 32 + 460 and 8 were installed along the service road between STA 3 + 212 and STA 3 + 297. The locations and depths of the piezometers are shown on the plan drawing, Figure 2 and on the cross-sections Figures 3 and 4. The purpose of the piezometers was to monitor the pore water pressure response as the fill was placed for the following purposes.

1. To determine how the porewater pressure responded as the construction progressed.

2. To ensure that embankment construction did not proceed if excess porewater pressure exceeded a value of 0.3 to 0.4 considered to be critical for embankment stability. This would allow the experience oriented shutdown period of 4 to 6 weeks between loading stages to be verified.

Settlement Plates

Settlement plates to monitor settlement of the subsoils were installed along both the westbound lane and service road. Eight plates were installed, four on each roadway.

Settlement Profilers

Two horizontal slope inclinometers (settlement profilers) were installed along the service road at locations shown on Figure 2. The purpose of the profilers was to determine the settlement profile across the roadway as embankment fill progressed. No profilers were installed in the westbound lane since the median construction was perceived to result in this equipment being damaged at an early stage. Also, for the system used, both ends of the profiler had to be open and this would not be possible between the westbound lane and eastbound lane.

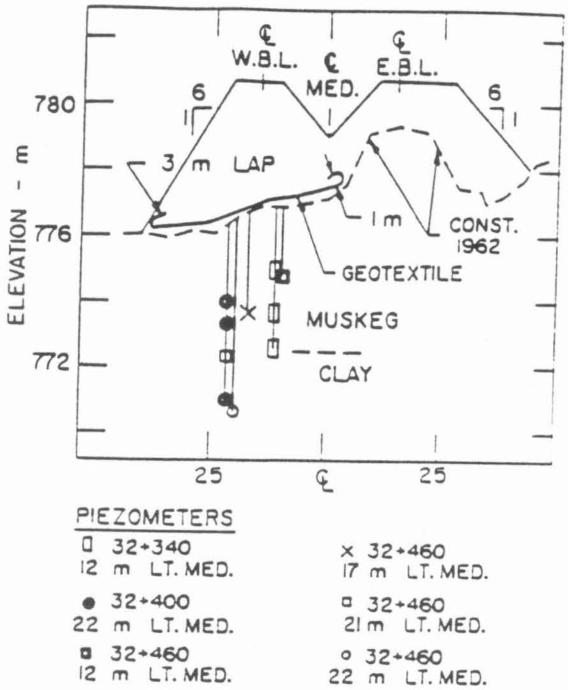

FIG. 3. Piezometer Locations Westbound Lane

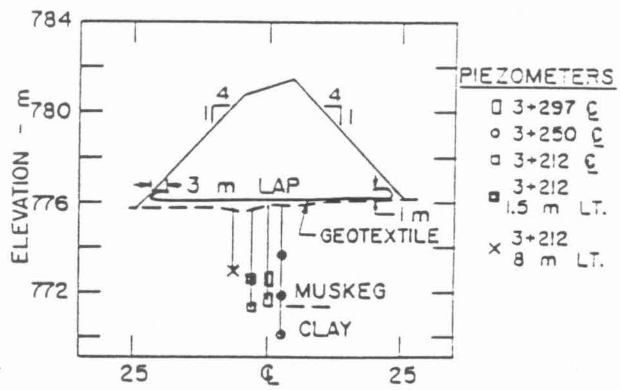

FIG. 4. Piezometer Locations Service Road

Slope Inclinometers

Six slope inclinometers were installed along the service road at locations shown in Figure 2. The inclinometers were anchored in the sandstone underlying the muskeg and silty clay. The inclinometers were used primarily to monitor horizontal spreading of the embankment perceived to occur during fill placement and to ascertain whether geotextile reinforcement minimized lateral spreading. As shown, only two inclinometers were located in reinforced areas with the remainder in non-reinforced areas. No slope inclinometers were placed in the westbound lane.

Geotextiles

As mentioned previously, four reinforced sections were planned. The location of these sections are shown on Figure 2. GTF 800 and GTF 500 from Exxon were placed in reinforced sections 1 and 2, respectively, while P500X and P600X from Mirafi were placed in reinforced sections 3 and 4, respectively. Also planned, was the instrumentation of the geotextiles placed in each of the sections. The purpose of the instrumentation was to measure strains during fill construction so that performance of the geotextiles with differing strength and stiffness characteristics could be compared.

Instrumentation of all geotextiles as contemplated could not be accomplished. Only the geotextiles on the westbound lane were instrumented. Geotextile instrumentation in the service road was not attempted since the geotextiles were placed directly on the cut vegetation precluding proper installation of gauges. A soil pad rising above water level would have been desirable prior to the placement of the geotextiles to provide a relatively plane surface for instrumentation.

Bison strain gauges utilized by others (Rowe, et al 1984) were used for instrumentation. The gauge size used was 10 cms in diameter and the layout of the instrumentation is shown in Figure 5. In reinforced section 1, Figure 2, four gauges per set were used to obtain two readings of geotextile deformation transverse to the highway alignment and two readings parallel to the alignment. In reinforced section 2, six gauges were used in a different orientation and with a different installation technique.

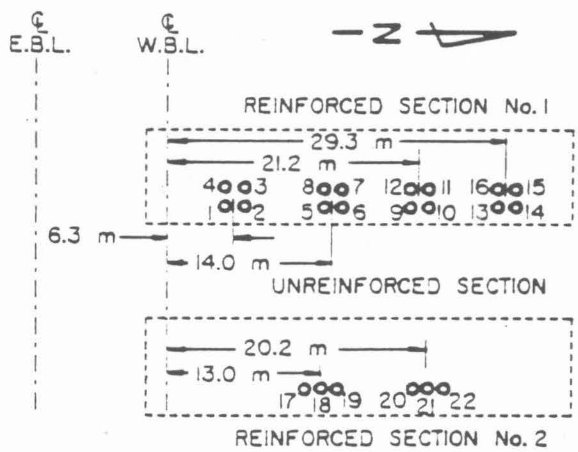

FIG. 5. Layout of Bison Gauges

The geotextile instrumentation, field monitoring and results reporting were undertaken by Rick Chalaturnyk and Gerry Cyre of the Department of Civil Engineering, University of Alberta. For complete details on the instrumentation, including recommendations for proper installation, the reader is referred to an unpublished report by the University of Alberta (1987) for Alberta Transportation and Utilities. This report can be obtained through Alberta Transportation and Utilities, Geotechnical Services Section.

CONSTRUCTION

Embankment fill construction through the muskeg sections began on August 1, 1986 on the westbound lane and on August 6, on the service road. Prior to any fill placement laying of the geotextiles was coordinated for instrumentation installation. Gauges were purchased in June 12, 1986 and received on July 27, 1986 with scheduled field installation on July 31. The August 1 scheduled fill placement on the westbound lane was only confirmed at 8 p.m. on July 30, 1988. This type of last minute scheduling of work by Contractors is typical on road construction projects.

Since the time frame was insufficient modified gauge installation techniques were used. In reinforced section 1 the gauges were attached with nylon bolts while in reinforced section 2 the gauges were attached by contact cement and alligator strap. The gauges were protected from damage during fill placement by placing a thick non-woven geotextile, folded over several times, at each gauge location followed by the placement of sand and gravel over top the non-woven geotextile. The geotextiles were placed transverse to the alignment with overlaps of 1 m between strips as shown in Figure 6.

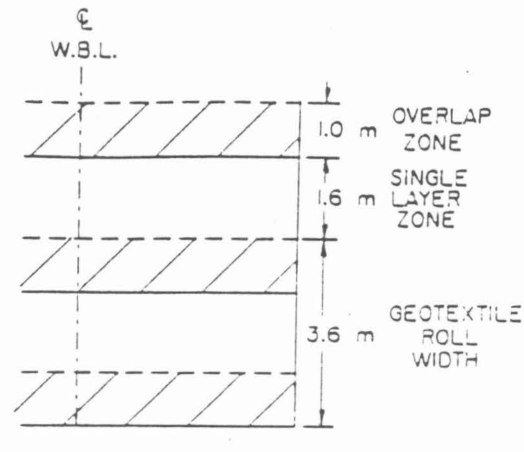

FIG. 6. Overlapping of Geotextile

Embankment fill was placed by bottom dumping from motor scrapers and pushing onto the muskeg with a D6 tractor. No equipment was allowed to traffic directly on the muskeg or geotextile. Compaction of the fill was afforded by sheepsfoot type compactors.

Construction of the embankment fill was monitored full time by a Geotechnical Technologist stationed on site. This technologist was responsible for monitoring all piezometers, installing and reading settlement plates, installing settlement profilers and observing embankment behaviour as construction progressed.

Embankment construction to design subgrade elevation was completed around September 6, 1986 on both roadways giving a construction period of about 1 month. Following this, an asphaltic concrete pavement with granular base was placed on the westbound lane. This construction was completed in October 1986. The reason for the quick paving of this roadway was to allow traffic accommodation for an interchange construction consisting of two overpasses. The service road was gravel surfaced since this roadway was not to accommodate main highway traffic. In September 1987 an asphaltic overlay was placed on the westbound lane to remove some settlement which had resulted in a distinct dip in the roadway surface.

RESULTS OF MONITORING PROGRAMME

Piezometers

The pore pressure results obtained from the piezometer monitoring are shown on Figures 7 through 12. The porewater pressure values have been reported in terms of Ru which has been calculated by dividing the pore pressure readings by the height of fill and density of material above the tip. No adjustments were made for material density and a value of 20 kN/m^3 was used throughout for both muskeg and embankment fill. This approach was found to be reasonably quick for field evaluation to ascertain that the excess pressure parameter did not exceed about 0.4. This concept was used throughout construction for controlling the rate of fill placement. Using this approach no shutdowns were necessary although in a few instances the excess pressure parameter exceeded 0.4. The time lag in many instances shown on the Figures is due to construction sequencing by the Contractor. On this project the Contractor was shown that cooperation for instrumentation placement and monitoring would be beneficial to him in terms of a shorter time to construct on the muskeg.

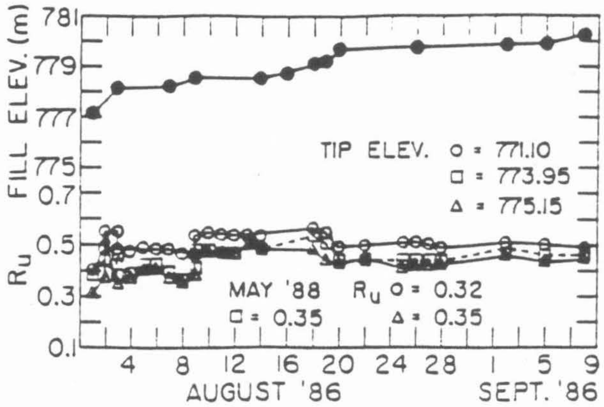

FIG. 7. Porewater Pressure Response Sta 32 + 460

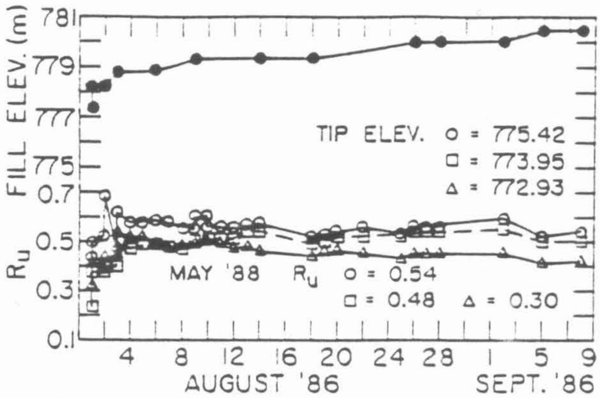

FIG. 8. Porewater Pressure Response Sta 32 + 340

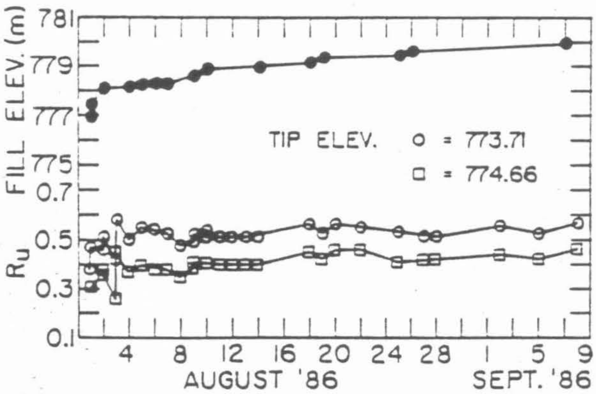

FIG. 9. Porewater Pressure Response Sta 32 + 400

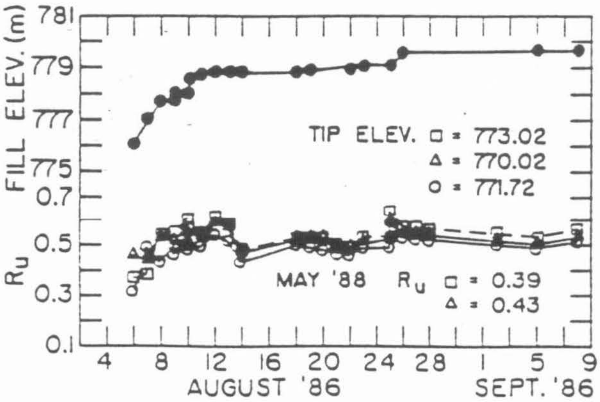

FIG. 10. Porewater Pressure Response Sta 3 + 250

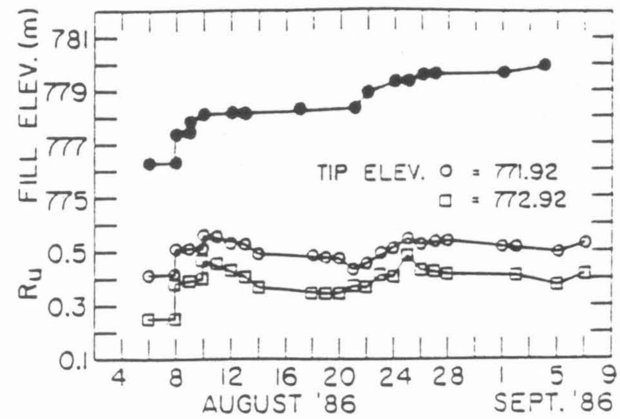

FIG. 11. Porewater Pressure Response Sta 3 + 297

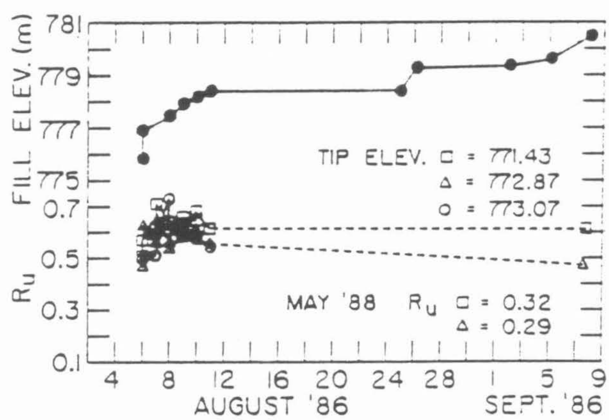

FIG. 12. Porewater Pressure Response Sta 3 + 212

As shown on the Figures, the piezometers response was generally good and followed the trend that is generally anticipated i.e., porewater pressure rising with increasing loads and levelling off with no further addition of load. At no stage during construction did the porewater pressures revert to the original.

Despite pore pressures not reducing to initial values, the excess pressures were generally not critical and continued fill placement did not result in any instability problems.

Also shown on some of the Figures are pore pressure responses after about 21 months following construction. Some of the Figures show that the pore pressures have reverted to original conditions. However, it was not possible to determine exactly how long after fill construction this situation was realized due to the infrequent readings taken after construction.

Based on the information gathered from this monitoring the following deductions were made:

1. The period between stages of loading could be shortened to two weeks instead of four weeks.

2. Embankment performance on muskegs could be enhanced by loading the muskeg for at least 1 year, or where feasible 2 years, to ensure that pore pressure dissipation and consolidation would be completed prior to construction of an asphalt pavement structure.

Settlements

Results of settlement monitoring are shown on Figures 13 and 14. All results are for settlement on the service road. Settlement plates on the westbound lane were damaged quite frequently during fill placement leading to erratic readings. Most plates broke at the connection between the riser and base plate. The base plates were made of plywood which was considered unsuitable for muskeg instrumentation. A base constructed of steel to which the initial riser could be welded may have given better performance.

The settlement curves shown on Figure 13 are for two out of the four plates installed on the service road. The two other two plates were destroyed during construction. Settlement attained varied from 0.8 to 1.2 m with the smaller settlement occurring under the smaller load, as might have been anticipated, and shorter time period of monitoring. The larger settlement was recorded up to the completion of embankment construction and under a larger loading.

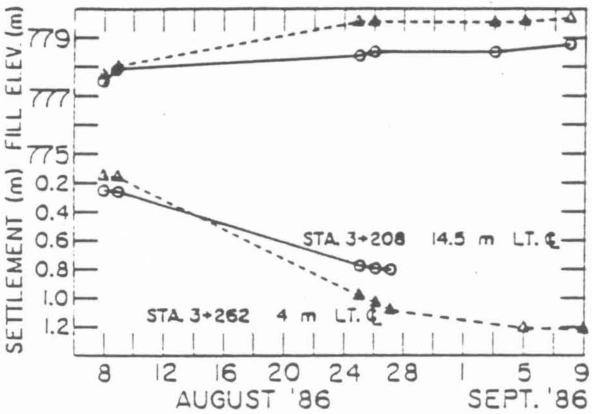

FIG. 13. Settlement Plate Records

The settlement profilers shown in Figure 14 portray the settlement for a 2 month period from the start of fill construction. Further readings were impossible since the ends of the profilers sunk below original ground as settlement progressed.

A comparison of settlements during construction with those determined through post construction drilling will be discussed subsequently.

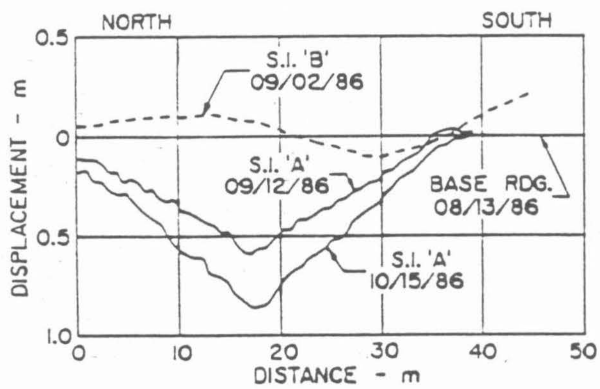

FIG. 14. Settlement Profile from Horizontal Slope Inclinometers

Lateral Deformation

Lateral deformations occurring during embankment construction are summarized on Figures 15 and 16 for four of the six inclinometers installed. The remaining two inclinometers were destroyed during construction.

The results do not show any distinct influence of the geotextile on lateral deformation. This might have resulted since the geotextiles on the service road were placed with severe wrinkles. The deformation results, however, do show the lateral spreading that occurs with muskeg loading. These results are confirmed by visual observations of field personnel who reported some lateral movement of the embankment in the northerly direction. All slope inclinometers are now non-functional having sheared at the points of movement which were within the muskeg or at the interface of the muskeg and underlying silty clay soil.

Geotextile Reinforcement

Of the 16 Bison gauges installed on reinforced section 1 only 6 gauges were found to provide meaningful results. These were gauge pairs 15 and 16, and 3 and 4, which measured strains transverse to the highway alignment and gauge pair 4 and 1 which measured strains parallel to the highway alignment.

The load-time curves for these gauges are shown on Figure 17 for the GTF 800 geotextile only. Gauges on the GTF 500 geotextile, reinforced section 2, did not function when read initially. The geotextile loads were calculated using the load-strain curve for the GTF 800 geotextile shown in Figure 18. This data was obtained from tests by the manufacturer.

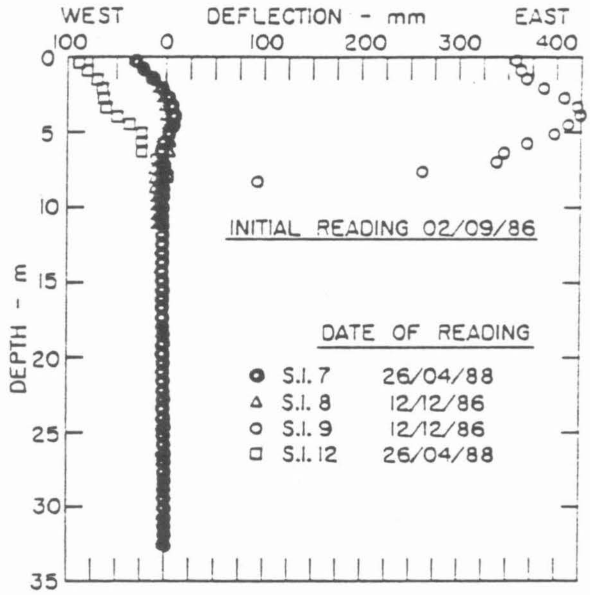

FIG. 15. Slope Inclinometer Readings East-West Direction

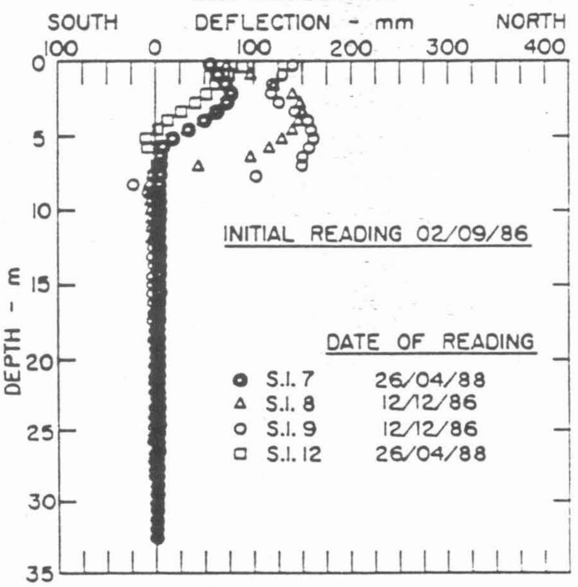

FIG. 16. Slope Inclinometer Readings North-South Direction

An important feature of Figure 17 is the rapid development of load within the geotextile. The maximum load was generally attained near the end of construction after which the geotextile loads remained relatively constant or decreased with time. This would indicate that the geotextile was contributing more to the short term stability rather than the long term stability.

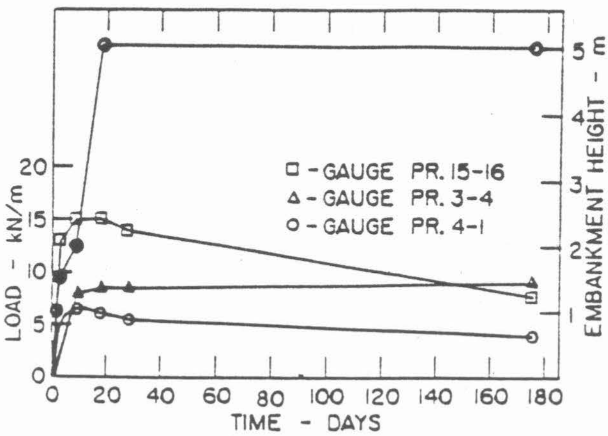

FIG. 17. Load-Time Curves
GTF 800 Geotextile

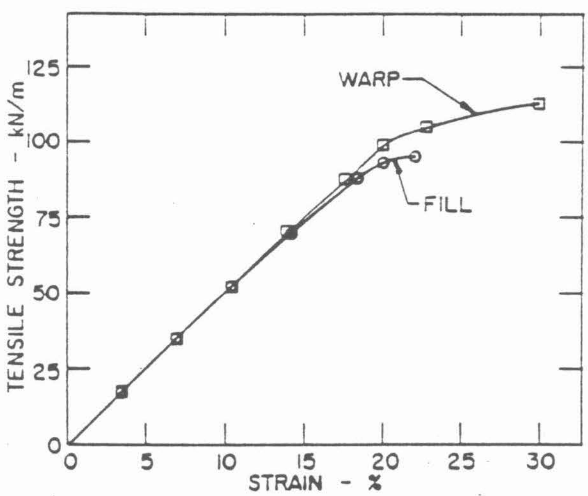

FIG. 18. Load-Strain Curve
GTF 800 Geotextile

Another aspect of Figure 17 is the variation of load with time. For gauge pair 15 and 16 the load decreased from 15 kN/m to 7.8 kN/m after 160 days. This decrease in load is likely due to the consolidation and strength gain of the muskeg. This observation confirms the findings of Rowe (1984) who reported that the geotextile effect was most significant during and just after the embankment construction and that there was a decrease in load following the embankment construction. Also of significance is the other curves on Figure 17 which show no appreciable decrease in loading. For these curves the gauges are situated nearer to the median centreline and are believed to be on ground that was initially stiffer caused by displacement of earth towards the westbound lane during the eastbound lane construction.

The actual loads attained by the geotextile are felt to be a minimum and may have been influenced by the overlapping of the geotextile producing non-uniform stiffness in the longitudinal direction. A factor of 1.63, representing the ratio of the width of a single layer and overlap zone, to the width of a single layer zone, can be utilized to adjust the loads on Figure 17. Using this factor, the maximum load for gauge pair 15 and 16 might be 24 kN/m. This modified value is considerably smaller than that which would be obtained using the sliding wedge theory (lateral embankment sliding) to determine loading transverse to the alignment.

POST CONSTRUCTION INVESTIGATION AND ANALYSIS

In May - June 1988, a drilling investigation of the westbound lane and service road was undertaken to study the settlement characteristics of the muskeg deposit through (a) comparing the drilled thicknesses with the thicknesses before construction and (b) comparing the monitored settlement with settlement derived from (a).

At the same time samples of the subsoils were to be taken for moisture content, strength, and compressibility determinations to also allow evaluation by comparison, changes induced by embankment loading. Since this investigation was completed only a few of the subsoil samples have been tested. Hence, only the settlement characteristics of the muskeg determined through drilling will be addressed.

The subsoil investigation consisted of drilling 14 test holes with a B-61 auger rig equipped with solid stem augers. Test holes were drilled along and transverse to the alignment and at locations as close as possible to holes done prior to construction. All test holes penetrated the underlying silty clay soil but did not reach the sandstone.

From the boring results, cross-sectional and longitudinal soil profiles were obtained along both roadways. Figures 19 and 20 show the longitudinal profiles for the westbound lane and service road, respectively, while Figures 21 through 25 show the cross-sectional profiles.

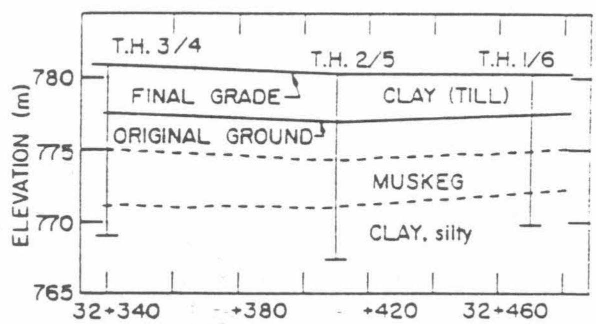

FIG. 19. Longitudinal Profile along
Westbound Lane

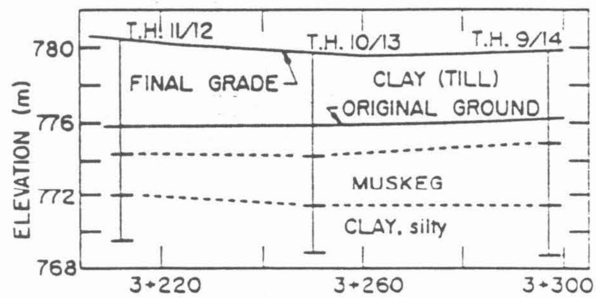

FIG. 20. Longitudinal Profile along
Service Road

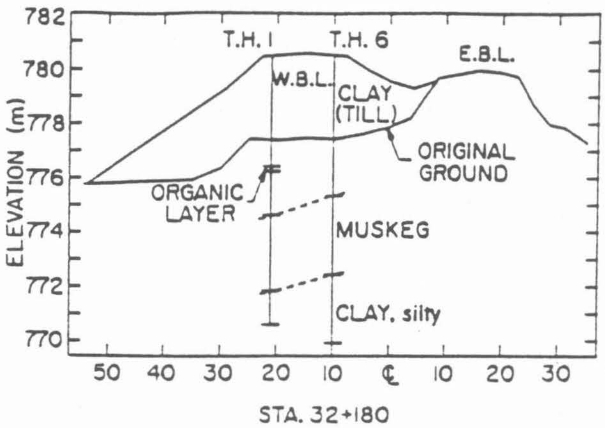

FIG. 21. Cross-sectional Soil Profile Westbound Lane

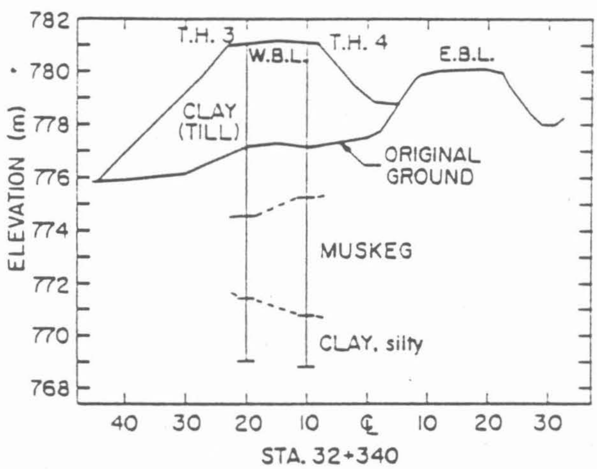

FIG. 22. Cross-sectional Soil Profile Westbound Lane

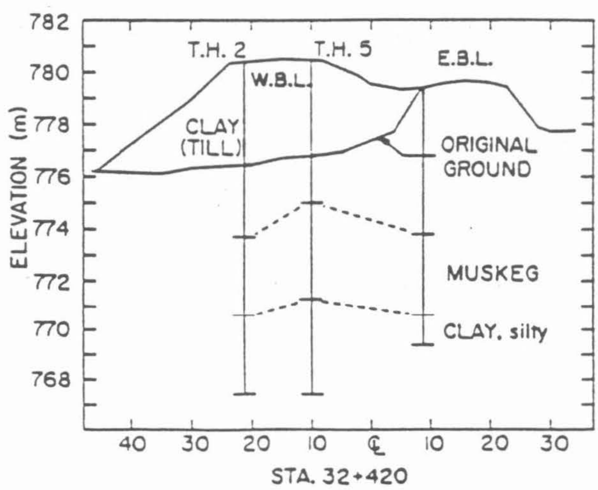

FIG. 23. Cross-sectional Soil Profile Westbound Lane

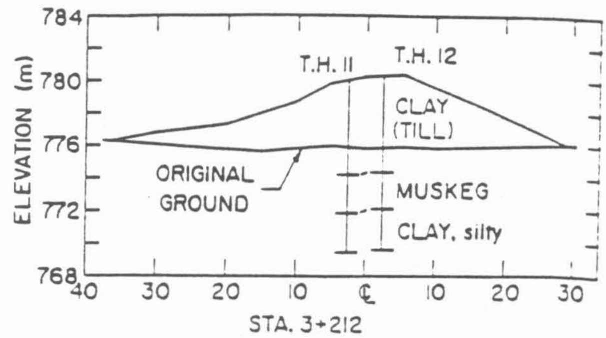

FIG. 24. Cross-sectional Soil Profile Service Road

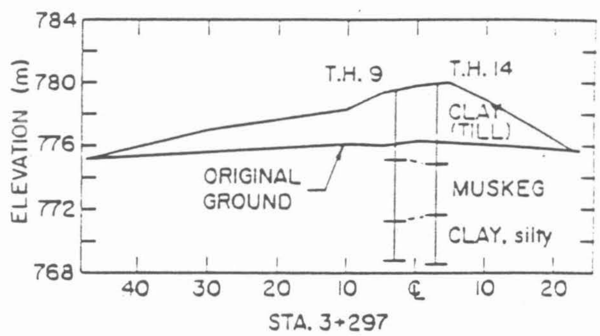

FIG. 25. Cross-sectional Soil Profile Service Road

Westbound Lane Settlement

From the cross-sectional profiles, Figures 21 to 23, the settlement has been determined at a distance of 10 m left of median centreline to vary from 29% at Sta 32 + 340 to 42% at Sta 32 + 470. At 20 m left of median centreline the settlement varied from 47 to 49%. The smaller settlement closer to the median confirms the observation of smaller geotextile loading in this area due to stiffer subsoil conditions influenced by the eastbound lane construction.

Comparing the settlements at various locations at Sta 32 + 410, it is noted that for the virgin muskeg (20 m left of median centreline) the settlement is 94% of the settlement determined under the eastbound lane from the initial geotechnical investigation. At other locations, values varied from 94 to 98% of the eastbound land settlement. For all practical purposes, therefore, we can consider that primary settlement and perhaps some secondary settlement (to be discussed later) have been completed along the westbound lane.

Service Road

From the cross-sectional profiles, Figures 24 and 25, the settlement has been determined to vary from 21 to 29% at Sta 3 + 297 to 38 and 40% at Sta 3 + 212. The generally smaller settlements obtained in comparison to the westbound lane are influenced by the smaller embankment height and to some extent by the lateral spreading of the muskeg. The monitored settlement represents 50 to 75% of the settlement obtained using the empirical correlation chart, Figure 26.

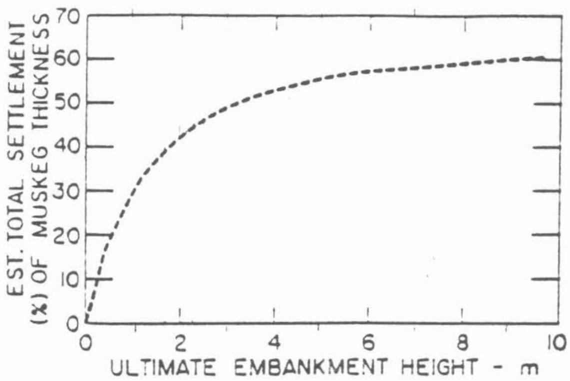

FIG. 26. Empirical Settlement Chart used by Alberta Transportation and Utilities

General

It will be assumed that the settlement attained under the service road after 21 months represents the total primary consolidation settlement. This assumption is not unrealistic since the settlement of the virgin muskeg under the westbound lane, over the same time period, represented 94 to 98% of the 24 year settlement of the eastbound lane. In relation to the empirical correlation chart, Figure 26, the settlement would represent about 86% of the predicted settlement.

In comparing the settlements obtained from the settlement plates and profilers with the total settlement of the service road it can be determined that the majority of these settlements was achieved within 30 to 70 days from the start of fill construction. A similar time period for primary consolidation can be associated with the westbound lane. These results demonstrate the well known behaviour of muskeg i.e., primary settlement occurs rapidly under loading.

Since the westbound lane had to be repaired with a 100 mm asphaltic concrete overlay one year following pavement construction, a longer time period prior to pavement construction may be more desirable. It may therefore be appropriate to allow at least a one year period following construction to remove most of the undesirable settlement prior to any asphaltic concrete surfacing.

In consideration of secondary settlement it has been shown that this can represent about 5 to 10% of the total settlement (Samson and La Rochelle, 1972). Assuming that all primary settlement was completed during construction or shortly thereafter, the settlement that occurred about 1 year after construction can be taken to represent secondary consolidation of the muskeg. From the settlement characteristics one can deduce that secondary settlement would vary from 2 to 6% of the total settlement. Hence, a further 50 to 150 mm of settlement is expected to occur over the next 23 years.

SUMMARY AND CONCLUSIONS

A field research program was incorporated into a highway construction project to investigate the effects of geotextile reinforcement of embankments on muskeg, as well as the general evaluation of embankment performance on muskegs. The following conclusions were reached based on results obtained from instrumentation monitoring, post construction drilling, and visual observations.

1. Geotextiles are not absolutely required for construction of embankments on muskegs.
2. The use of geotextiles appear to improve the short term stability of the embankment.
3. There were no distinct differences in lateral deformation between reinforced and non-reinforced sections.
4. The use of geotextiles did not appear to result in any perceptible reduction in vertical settlement.
5. Geotextile instrumentation with Bison gauges require proper installation to obtain any meaningful results.
6. A low strength woven geotextile can be used as reinforcement for muskegs using the stage loading technique of construction. It appears, however, based on visual observations that a high strength fabric would be beneficial for very weak areas to minimize lateral spreading of the embankment.
7. The majority of primary consolidation of the muskeg occurred within one to two months following fill construction.
8. Complete dissipation of excess porewater pressure was not achieved at all locations although it appeared that over the 21 month period hydrostatic conditions prevailed in some areas.
9. With instrumentation and monitoring, embankment construction was undertaken in half the time that would have been required according to the contract. Excess porewater pressures rarely attained values that were critical.
10. Secondary consolidation is expected to result in a 50 to 150 mm settlement of the embankments within the next 23 years.
11. Settlement monitoring using settlement plates requires sturdy equipment to ensure resistance against damage. The use of horizontal slope inclinometers, although providing a complete cross-sectional settlement profile, may not be suitable for monitoring settlements of muskeg on account of the large deformations involved.
12. Settlements obtained from actual drilling were, in general, smaller than those obtained using the empirical correlation chart.
13. Construction of 4 to 5 m high embankment fills can be undertaken successfully over 4 to 6 m deep muskeg deposits utilizing a stage construction approach.
14. Construction of embankment fills at least a year ahead of pavement construction would appear to result in removal of all primary settlement and some secondary settlement.

PRACTICAL APPLICATION

Some of the findings of this research have since been incorporated into design and construction practices and are being utilized on the remainder of Hwy 16 twinning projects. In the majority of cases the following have been implemented:

1. The use of stage construction with a 2 week interval between loading stages.
2. The construction of embankment fill at least 1 year in advance of asphaltic concrete pavement construction.
3. The selective use of geotextiles to aid construction e.g., for trafficability on waterlogged sites, reinforcement in weak subsoil areas and in the correction of failures.
4. The use of woven polypropylene geotextile with a 30 kN/m wide tensile strip strength for reinforcement when required in problematic areas.

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