INSIGHTS FROM CASE HISTORIES: REINFORCED EMBANKMENTS AND RETAINING WALLS

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ABSTRACT: A number of published field case histories of geosynthetic reinforced embankments and retaining walls, including the most recent cases, are reviewed in this paper. Key features of 21 case histories of reinforced embankments are investigated to address issues that include differences between the mobilized reinforcement strains and expected reinforcement strains in design, the combined use of geosynthetic reinforcement and prefabricated vertical drains, the effect of construction rates, the time dependent behaviour of reinforced embankments, and reinforced embankments over peat foundations. The conservatism of current design method is examined, and the choice of shear strength of peat foundations in design is critically reviewed. Factors contributing to low mobilization of reinforcement in embankments are identified. The review of 12 case histories of reinforced soil walls highlights the behaviour of the reinforced retaining walls with both granular and cohesive backfills. The effects of reinforcement creep and non-rigid foundations on the post construction performance of reinforced soil walls are discussed. Based on the key findings of the field case histories cited in this paper, conclusions are drawn for the behaviour of those geosynthetic reinforced soil structures and design considerations.

1. INTRODUCTION

During the past two decades, geosynthetic reinforcement and vertical drains have been used extensively in geotechnical structures, such as embankments and retaining walls. The use of the geosynthetics typically reduces the cost of construction, increases the tolerance of soil structures to ground movements, and increases the feasibility of construction of soil structures on poor foundation soils.

A number of researchers have examined the behaviour of reinforced embankments and retaining walls based on field case histories (e.g. Haliburton et al. 1980; Rowe et al. 1984a; Fowler and Edris Jr 1987; Bassett and Yeo 1988; Simac et. al. 1990; Allen et al. 1991; Delmas et al. 1992; Fishman et al. 1993; Litwinowicz et al. 1994; Rowe et al. 1995; Rowe and Mylleville 1996; Carrubba et al. 1999; Bathurst et al. 2000 and others). Finite element methods have also been used to examine the behaviour of reinforced embankments and walls under various conditions (e.g. Rowe and Soderman 1985a, 1987b; Rowe and Mylleville 1989; Hird and Kwok 1990; Karpurapu and Bathurst 1992; Li and Rowe 1999a,b and others). Prior research into the behaviour of reinforced embankments and soil walls has provided the basis for the development of design methods. The current design methods are usually based on limit equilibrium analysis which is relatively simple but can not adequately describe soil-reinforcement interaction.

Numerous case records show that the actual behaviour of reinforced embankments is significantly different from design assumptions due to simplifications made in current design methods (e.g. Duarte and Satterlee 1989; Bassett and Yeo 1988; Fritzinger 1990; Litwinowicz et al. 1994). It also has shown that the current design methods are usually conservative (Mitchell 1987; Rowe and Ho 1992; Bathurst et al. 2000). A significantly factor contributing to the conservatism may be the choice of low design strength values for both the soil and reinforcement. The conservatism also arises, in part, from uncertainties, related to (i) stress-deformation-time characteristics of soil and reinforcement, (ii) the interaction between the soil and reinforcement, (iii) the failure modes, and (iv) neglect of some conditions favourable to the stability such as the effect of partial consolidation of the foundation soils on embankment stability and the effect of wall facing on the wall stability.

Although there has been considerable research published in literature, there are still aspects of the behaviour of geosynthetic reinforced embankments and soil walls that are not fully understood. For example, relative little has been done to address issues such as the time dependent behaviour and the interactions of the different elements of reinforced soil structures. This paper reviews some of the case histories related to reinforced embankments and retaining walls and summarizes the insights gained.

2. REINFORCED EMBANKMENT OVER SOFT SOILS.

Basal geosynthetic reinforcement (including those manufactured from polyester, polyethylene and polypropylene in either textile or grid form) has been shown to allow cost-effective construction of embankments on soft clay soils (Humphrey and Holtz 1987; Fowler and Koerner 1987; Rowe 1997). The tensile force in reinforcement serves to increase of factor of safety at the end of construction. From bearing capacity considerations, the use of reinforcement increases the rigidity of the embankment and reduces the outward shear stresses on the foundation. If sufficiently reinforced, the embankment can function more like a rough rigid footing (Jewell 1988) and this increases the bearing capacity with maximum increase being predicted based on the plasticity solutions (Rowe and Soderman 1987b).

In numerous field cases, it has been reported that the failure height of reinforced embankment often exceeds the theoretical collapse height based on the classical bearing capacity theory (Prandtl 1920). This is primarily because there is usually a significant increase in the undrained strength with depth in these natural deposits and/or the very soft layer is underlain by a firm stratum at relatively shallow depth. Both factors increase the bearing capacity (Humphrey and Holtz 1986; Rowe and Soderman 1987b). Finite element analyses have shown that the use of reinforcement result in a larger and deeper failure surface than that for an unreinforced embankment. This enhances the beneficial effects of the strength increase with depth and finite depth of natural soft foundation deposits (Rowe and Soderman 1987a; Li and Rowe 2000).

Due to the low shear strength and low hydraulic conductivity of very soft to soft clayey foundation soils, embankments are often constructed in stages or at a controlled rate of construction to allow for the dissipation of excess pore pressure in the foundation soils. Consequently the strength gain of foundation soils under partial drained conditions improves the stability. The use of reinforcement also enhances the beneficial effect of strength gain in the foundation (Rowe and Li 1999; Li and Rowe 2000).

In addition to providing reinforcement, geosynthetics have also been used in the form of prefabricated vertical drains (PVDs) to speed up consolidation of thick soft deposits under embankment loadings (Hansbo et al. 1981; Jamiolkowski et al. 1983; Holtz et al. 1991). The use of geosynthetic reinforcement in combination with prefabricated vertical drains has the potential to allow construction of substantially higher embankments in considerably shorter time periods than conventional construction methods (e.g. Lockett and Mattox 1987; Schimelfenyg et al. 1990; Li and Rowe 2000). The reinforcement can also be used to eliminate the necessity of stage construction or to reduce the number of stages required in stage construction (Lockett and Mattox 1987; Li and Rowe 1999a).

2.1 Benefits due to the use of reinforcement

Case History: Mobile embankment (Lockett and Mattox 1987)

The construction of a 6.62 m high and 15.9 wide bridge approach embankment with 1v:3h slopes over weak marsh deposits in Mobile, Alabama, USA involved the use of five different geosynthetics. Six layers of high density polyethylene uniaxial geogrids (estimated from the product description to have T_{ult} = 79 kN/m and $J_{5\%}$ = 1080 kN/m) were employed to reinforce the embankment so that the full embankment height was constructed without stage construction. Vertical drains were used to accelerate the consolidation and allow construction within the time schedule. The foundation consisted of a soft silty clay layer that was underlain by silty sand and clayey sand layers. The silty clay layer had a thickness ranging from 2.74 m to 5.18 m, with reported natural water content (w_n) of 45-105%, liquid limit (LL) of 32-55% and plastic limit (PL) of 9-30%. The undrained shear strength s_u of the soil of this layer varied between 8 kPa at the top and 11 kPa near the bottom.

The primary reinforcing geogrids provided the required factor of safety in the design and maintained the desired stability during the critical time period of loading on the compressible marsh deposits. There was no lateral spreading of the embankment observed. Rapid dissipation of excess pore pressures was achieved through the short drainage paths provided by the vertical drains and sand blanket.

Key findings:

- The use of reinforcement resulted in significant savings for the project in comparison to a conventional soil improvement method.
- The use of reinforcement eliminated the need for stage construction.
- The combined use of reinforcement and vertical drains gave rise to the excellent performance of the embankment.

Case History: Embankments over soft Bangkok clay (Bergado et al. 1994; Loke et al. 1994)

Three full-scale embankments were constructed rapidly to failure on a soft clay deposit in Bangkok, Thailand that consisted of a 2 m crust and 8 m thick soft clay. Two of the embankments were reinforced with the third being unreinforced as control embankment. One of the two reinforced embankments (Embankment A) was reinforced by one layer of high strength woven geotextile with an ultimate tensile strength (T_{ult}) of 200 kN/m. The other reinforced embankment (Embankment B) was reinforced by four layers of needle-punched nonwoven geotextile with a low stiffness of which the first geotextile layer had T_{ult} = 18 kN/m (based on "in-isolation" tests) and the other three geotextile layers had T_{ult} = 8.3 kN/m (i.e. giving a total combined strength of about 43 kN/m).

The control embankment failed at a fill thickness of height 4 m, however the reinforced embankment did not fail until the thickness of the fill placement reached 6 m and 4.62 m for Embankments A and B respectively. Thus the use of reinforcement increased the failure height by 50% for Embankment A and 15.5% for Embankment B. The observed vertical deformations of the embankment and horizontal deformations of the foundation soils at the end of construction of reinforced embankments were less than those of the unreinforced embankment at about the same embankment height. In the case of Embankment A, there were no significant strains in the geotextile observed at heights below 3 m, and the geotextile strains increased to between 2% and 3.5% at a height of 4 m. A maximum strain of about 13% was recorded at failure of Embankment A (due to reinforcement failure). In Embankment B, the maximum strain of 6% was measured in the first geotextile prior to embankment failure. The maximum strain in all other layers was about 3% at failure.

Key findings:

- The tensile force mobilized after the foundation soils became plastic increased the embankment stability and substantially increased the embankment failure height.
- The reinforcement reduced the lateral deformation of the foundation soil at a given embankment height.
- The high strength and stiffness woven geotextile was more efficient than the nonwoven geotextile which had a much lower strength and stiffness.
- Based on the available evidence, it would appear that embankment A with the high strength reinforcement failed due to the reinforcement strength being reached. For embankment B there appears to have been a failure of the under-reinforced system without full mobilized of the reinforcement strength.

Case History: An access ramp embankment (Mattox and Fugua 1995)

A roadway embankment was constructed on a mangrove swamp consisting of highly organic soils where a previous unreinforced embankment had failed at a 6 m height. As a remedial measure, it was proposed that the embankment with 4 m thick granular fill and 1.5 m surcharge be constructed in two stages and be reinforced using three layers of uniaxial geogrids (T_{ult} = 79 kN/m and $J_{5\%}$ = 1080 kN/m). The embankment foundation in the mangrove swamp consisted of a 4 m thick upper layer of low plasticity sandy clay (LL = 56 % and PL = 32%) and a 13.5 m thick lower layer of higher plasticity organic clay with an average LL = 76%

and PL = 29 %. An effective stress analysis method was adopted in the design. The stability analysis, based on the conservative soil parameters back-calculated from the failed embankment, indicated that three layers of this uniaxial geogrid would be required to provide an acceptable factor of safety. During the actual construction, the embankment was constructed to a thickness ranging from 3.12 m to 3.4 m in the first stage. Subsequently, during the second stage a 1.24 m to 1.84 m thick surcharge was placed after 50 days (instead of 130 days originally proposed) due to the faster than expected pore pressure dissipation.

Key findings:

- The use of geogrid reinforcement in combination with a stage construction at a controlled rate for each stage allowed construction of an embankment over a soft deposit where an unreinforced embankment had failed previously.
- The pore pressure measured during the first stage of construction was significantly less than that the predicted in the design calculations. This is attributed to the conservatism in the selection of the consolidation coefficient.

Discussion

It is evident from above field cases that the use of basal geosynthetic reinforcement provided an increase in embankment stability and reduced embankment deformations. The benefits arising from the use of geosynthetic reinforcement include the improvement of the embankment behaviour, cost savings, an increase in the feasibility of embankment construction, and the elimination of stage construction in some cases. In the last case, the designers appear to have been over cautious and the embankment likely could have been safely constructed to the design height without stage construction.

2.2 The comparison of field reinforcement force and strain with design values

Case History: Reach A test levee (Fowler and Edris Jr 1987; Duarte and Satterlee 1989)

A high strength woven geotextile reinforced levee test section was built to raise the existing hurricane levee in Plaquemine Parish, Louisiana, USA. The centre of the new levee was raised by 3.8 m. The soil profile involved low plasticity organic silt near ground surface overlying highly plastic soft clays with thin silt and sand lenses with moisture contents varying from 25% to 285%. The consistency of most of these soils varied from medium to soft. The undrained shear strength s_u of the foundation soil ranged from 7.2 kPa at the ground surface to 21.5 kPa at an 18 m depth.

Slope stability analyses that were performed to establish the required geotextile strength for a factor of safety of 1.3 against both circular and non-circular failure indicated a reinforcement force of 210 kN/m was necessary. A high strength woven geotextile product having an ultimate strength T_{ult} = 664 kN/m and a stiffness, J, at 5% strain of about 5950 kN/m was chosen for the basal reinforcement.

Strain gages were attached to the geotextile to monitor the deformation of the reinforcement over a two-year period. Settlement plates, piezometers and inclinometers were also used to monitor the performance of the embankment. The measured maximum horizontal movements at the end of construction below the slope of the levee and the maximum settlement was about 25 cm and 30 cm respectively. The observed maximum strain in the geotextile was 2% at the end of construction and this increased to 3.5% at 400 days after the construction. This magnitude of strain (3.5%) corresponded to a tensile force of 57.6 kN/m based on the load - strain curve of the geotextile recovered from the field. In comparison, the required reinforcement force was predicted to be 117 kN/m based a wedge method analysis for a factor of safety of one. Hence, the expected tensile force in the geotextile reinforcement was 2.03 times larger than the measured value and the design value of 210 kN/m is 3.65 higher than the measured value. This indicated that the reinforced levee had performed better than had been predicted in the limit equilibrium analyses.

Key findings:

- The range of geotextile strains and loads measured in the test section were significantly less than the values predicted during design.
- The design methods and the stability analyses were conservative.
- More accurate and economical designs could be made provided that the mechanics of the composite section can be better understood.
- The geosynthetic reinforcing technique resulted in a cost savings about one-half to

two-thirds the cost of conventional construction methods.
- The size of the levee and amount of land required was reduced due to the use of the high strength geotextile.

Case History: The Westminster levee (Hadj-Hamou and Bakeer 1991)

This levee test section 106.7 m in length and 3.05 m in height was located on the west bank of the Mississippi River in the town of Marrero in Jefferson Parish, Louisiana, USA. Two layers of uniaxial, extruded HDPE geogrids (T_{ult} = 79 kN/m and $J_{5\%}$ = 1080 kN/m) were used as reinforcement. The top 4.6 m of the foundation consisted of extremely soft to very soft clays with organics, underlain by very soft to soft clays to a depth of approximately 17 m, underlain by layers of medium soft clay and stiff to very stiff overconsolidated clay to 26 m. The natural water content varied from 40% to 350% in the organic clay and from 40% to 80% in the soft clay. The undrained shear strength was about 7.2 kN/m for the first 6.1 m and then increased with depth at a rate of 1.12 kPa/m thereafter. The geogrid strain anticipated in design calculations was in the order of 2% for the bottom layer and 1.8% in the top layer. However, the maximum strains recorded after construction were 0.57% and 1.7% in the top and bottom geogrid layers respectively.

Key findings:
- The maximum mobilized geosynthetics force was only 73% of the allowable design strength (34.3 kN/m).
- The use of geosynthetics allowed for considerable saving of marsh land and fill materials.

Case History: A Japanese embankment (Hashizume et al. 2000)

A 13.2 m high reinforced embankment was constructed over a 15 m thick soft ground in a 8 month period, in Shimanto-belt region, Japan. The embankment design required a reinforcement force of 450 kN/m which was provided by a geosynthetic reinforcement product with ultimate strength of 700 kN/m at 10 percent failure strain. The observed tension in the reinforcement (about 60 kN/m) was only 13% of the design value. The maximum horizontal displacement and settlement were 25 cm and 80 cm respectively at the end of construction.

Key findings:
- The mobilized tension of the geosynthetic at the end of construction was significantly less than that expected in the design.

Discussion

Current design methods for reinforced embankments are usually based on limit equilibrium analyses (Jewell 1982; Ingold 1982; Milligan and La Rochelle 1984; Fowler and Koerner 1987; Leshchinsky 1987; Mylleville and Rowe 1988; Holtz et al. 1997 and others). Other design approaches include the use of plasticity solutions (Rowe and Soderman 1987a; Jewell 1988) and design charts (e.g. Hird 1986; Low et al. 1990 and others). For embankments on soft cohesive deposits, the foundation soils are commonly assumed to respond in an undrained manner during embankment construction and the critical time with respect to stability is typically considered to be at the end of construction.

In general, reinforced embankments are designed considering the four principal failure mechanisms (i.e. bearing capacity failure, lateral sliding of embankments over reinforcement, and a limit equilibrium type failure associated with either breakage or pull-out of reinforcement). The last failure mechanism governs the design in many cases. Frequently, the conventional slip circle method for slope stability analysis is modified to include the restoring moment of reinforcement tensile force in addition due to the restoring moments provided by soils (e.g. Jewell 1982; Holtz et al. 1997).

Due to the simplifications and assumptions made in design the field behaviour of reinforced embankment can be significantly different from that anticipated in the design as shown in the above cases. In some cases, the mobilized reinforcement force was only a small fraction of that predicted in design. With a better understanding of the behaviour of geosynthetic-reinforced embankments and the conservatism in design assumptions and analyses, current design methods could be improved to achieve more accurate and economic design. Factors contributing to conservative design will be discussed in more detail in a later section.

2.3 The combined use of prefabricated vertical drains

Case History: A flood protection embankment (Lau & Cowland 2000)

A 10 km long 4 m high embankment was constructed on a very soft foundation on the Hong Kong side of the Shenzhen River in China as a part of the flood protection project. The foundation soils consisted of 6 - 12 m river mud and alluvial clay deposits. The river mud was very soft to soft silty clay with occasional shell fragments and sand lenses, with thickness varying from 6 m to 10 m and void ratio as high as 3. The alluvial clay (with PI = 49%) was a firm sandy to gravely clay with thickness ranging from 1 m to 9 m.

Field vane tests indicated a strength varying from 7.5 to 16 kPa for the 12 m thick soft soils. Based on the undrained analysis for short-term stability, a woven geotextile with a characteristic strength of 200 kN/m was employed to increase the short-term stability, however, this alone was not sufficient to achieve the required factor safety. Therefore, the prefabricated vertical drains with 1.5 m triangular spacing were installed to achieve rapid strength gain by shortening the consolidation period. The embankment was constructed in three stages and the last stage started after the required 75% consolidation was achieved. The undrained shear strength measured by cone penetration tests indicated the strength gain as predicted was sufficient to achieve the required factor safety at the end of the final construction.

The piezometer monitoring indicated significant dissipation of excess pore during the construction. The inclinometer results indicated that the maximum cumulative horizontal displacement of 83 mm occurred at about 2 m below the ground surface below the side berm. The settlement near the centre of the embankment stabilized at about 0.65 m in about five months after the end of construction.

Key findings:

- Neither reinforcement nor PVDs alone would be sufficient for the embankment construction to design height. The combined effects of both reinforcement and vertical drains increased the short-term stability and made it feasible to construct this embankment with the required factor of safety.
- Monitoring of the dissipation of excess pore pressure allowed for control of the rate of embankment construction.
- The use of PVDs and control of the construction rate effectively reduced the excess pore pressure during construction and accelerated the dissipation after construction.
- The rate of strength gain of the soft clays due to partial consolidation arising from the presence of the prefabricated vertical drains was rapid and significant.
- The measured maximum horizontal displacement in the soft soil was relatively small due to the combined use of reinforcement and PVDs.
- The use of geosynthetic reinforcement proved to be cost-effective.

Case History: A fabric reinforced dike (Schimelfenyg et al.1990)

A 5 m high reinforced dike was constructed to contain contaminated sediments that were dredged from the New Bedford harbor bottom at the New Bedford superfund site in Massachusetts. The project site lies on a large flat, partially submerged seaboard lowland and the foundation consisted of sand and organic clay layers. The organic clay with thickness varied from 1.2 m to 5.2 m had average water content of 105%, liquid limit of 106% and plasticity index of 73%. The undrained shear strength of the clay soils from field vane tests increased with depth and ranged from approximately 1.2 kPa to 11.5 kPa. A single layer of high strength polyester geotextile with an ultimate tensile strength about 880 kN/m and 200 kN/m in the warp and fill directions, receptively and the secant modulus at 5% elongation of about 8800 kN/m and 2000 kN/m in each of the principal directions was used. A vertical drain system with 1.5 m spacings was also used to shorten the consolidation time.

The dike was constructed in two stages. The undrained stability analysis indicated a factor of safety 0.6 without reinforcement and 1.6 with reinforcement having a mobilized tensile force of 350 kN/m (i.e. 5% tensile strain). Strains of 0.6 to 2.2% were observed during phase I construction (the first 1.2 m lift) followed by -0.1 to 0.5% during installation of the vertical drains, 0.5 to 3.5% during phase I consolidation, 0 to 0.8 % during phase II construction (a 3.8 m lift) and 0.1 – 1.5% during phase II consolidation period. The rates of strain increase were typically very gradual. The maximum mobilized strain of 7% exceeded the design value. Significant mud waves were observed during phase I construction. The total settlement was in the 0.9 m to 1.2 m range, which was less than the estimated value 1.5 m to 1.8 m for primary consolidation during design in ten month.

Key findings:

- During Phase I (i.e. before installation of the PVDs) significant reinforcement strain and mud waves were observed; however, very little increase in reinforcement strain was observed during the phase II construction (after the PVDs had been installed). This difference can be attributed to the shear strength gain achieved due to pore pressure dissipation facilitated by the installation of PVDs.
- The use of PVDs decreased the foundation heave and horizontal shear deformations during construction.
- The differential consolidation settlement gave rise to a significantly increase in reinforcement strain during consolidation.
- A relatively high embankment could be achieved over a foundation soil having extremely low undrained shear strength by using both reinforcement and PVDs. However, the mobilized reinforcement strain can be large due to combined plastic and consolidation deformations due to relatively high embankment. This observation is consistent with what would be expected based on FEM analyses by Li and Rowe (2001a).

Discussion

The field cases examined in this section show that the use of geosynthetic reinforcement in combination with prefabricated vertical drains has the potential to allow the cost-effective construction of substantially higher embankments in considerably shorter time periods than conventional construction methods. This is because the synergistic effects of the tensile reinforcement and the strength gain of the foundation soil due to partial consolidation by PVDs is greater than either effect alone and the use of reinforcement enhances the beneficial effects of the partial consolidation on embankment stability (Rowe and Li 1999; Li and Rowe 1999b). Both the PVDs and reinforcement served to reduce lateral deformations, heave and vertical shear settlement. PVDs also reduced reinforcement strains relative to those without PVDs.

Partial consolidation of the foundation soil during embankment construction is significant so that the reinforcement strain is reduced compared to the case without PVDs. Li and Rowe (2000, 2001a) have shown that for foundation soils with PVDs installed the average consolidation at the end of embankment construction ranges between 20% and 45% depending on construction rates and spacings. A design procedure that considers the combined effect PVDs and reinforcement was proposed by Li and Rowe (2000a).

2.4 Factors contributing to low mobilization reinforcement strain

Since typical designs are based on a "factor of safety" of 1.3 (or greater), it follows that the designs essentially are based on limit equilibrium of the design reinforcement force and soil with an undrained shear strength 25% less than that expected in the field. Thus the design force is not a prediction of the force expected in the field. An estimate of the force expected to be mobilized would be that calculated using the expected strength (ie assuming factor of safety of unity). This will be less than the design value (which used FS > 1) and hence the force mobilized in the filed can be expected to be less than design value for all cases where the mobilized strength is similar to the expected strength. However, as will be illustrated below the mobilized reinforcement force is often less than that based on the "expected" strength and a factor of safety of unity.

Case History: Levee test sections (Varuso et al. 1999)

Geosynthetic reinforced levee test sections were constructed by the New Orleans District of the U.S. Army Corps of Engineers to derive a new design methodology that would adequately account for the gain in shear strength of soft foundation materials due to consolidation during and shortly after construction. The test section, 274.5 in length, was located south of the Mississippi River in Louisiana, USA. The 25 m thick foundation deposits consisted of soft to medium clays with silt lenses and organics underlain by soft to very soft clays with relatively high water content, which were underlain by medium massive clays with some silts. Three test sections 91.5 m in length were reinforced using one layer of a geotextile with a design force at 5% strain $T_{5\%}$ = 85 kN/m for Section 1, one layer of uniaxial geogrid with $T_{5\%}$ = 85 kN/m for Section 2, and two layers of geogrids consisting of the upper layer with $T_{5\%}$ = 57 kN/m and the lower layer with $T_{5\%}$ = 17.5 kN/m for Section 3.

The levee was design based on a wedge method analysis assuming soil strengths derived from unconfined undrained (UU) tests and the force in the geosynthetics at 5% strain with a factor of safety of 1.0. The levee was constructed at an average rate of 1 m/month. The maximum reinforcement strain mobilized was 2.63%, 2.15% and 1.97% at the end of construction and 3.6%, 2.6% and 3% about 400 days after construction for Sections 1, 2 and 3 respectively. The maximum horizontal movement observed at 2.7 m depth was less than 50 mm and was well within the range expected for lateral movements beneath such levee on a soft foundation. The estimated force in the reinforcement was about 48% of the design value (i.e. 84 kN/m). The observed reinforcement strain and horizontal displacement were consistent with the levee embankment being stable even though it was designed for a factor of safety of one based on an undrained analysis and UU shear strength results. The soil samples from the post construction borings were tested in the laboratory and the resulted showed that the undrained shear strength of soil in the upper three meter soil layer increased by 135% compared to the initial strength. In the subsequent strata, the increase ranged from 50% to 67%.

Key findings:

- The stable performance of the reinforced levee that was designed for a factor of safety of one indicates that the design method or/and parameters used were conservative. One factor may be that the unconfined undrained test typically underestimates the undrained shear strength (due to disturbance).
- The pore pressure, settlement and post construction boring test data indicated a very rapid and substantial increase in shear strength due to consolidation of the soft foundation soils.
- The gain in shear strength due to partial consolidation during construction increased the levee stability and reduced the foundation shear deformations. Consequently the partial consolidation during construction may also have contributed to the mobilized reinforcement strain being significantly lower than the design value of 5%.
- The increase of embankment stability due to the strength gain by partial consolidation was effectively achieved by controlling the construction rate to allow excess pore pressure dissipation.

Case History: Wilmington harbor dike (Fritzinger 1990)

The dike was found on 7.5 m to 30.5 m thick, weak and highly compressible silts and clays underlain by 1.5 m to 6.0 m of dense Pleistocene sands and gravels in Wilmington, Delaware. Three major zones of fine grain foundation soils had a thickness of 6 m, 6 m and 18 m, undrained shear strength of about 4.8 kPa, 7.2 kPa and 9.4 kPa respectively. Zone 1 was of particular concern due to its initial very low undrained shear strength. The design adopted a wide-bermed embankment with a high strength geotextile as tensile reinforcement and vertical drains (with 3-m triangular pattern) to strengthen the foundation soil. The reinforcement was a woven polyester geotextile with a stiffness J = 3300 kN/m and a tensile strength of T_{ult} = 260 kN/m.

The embankment construction was performed in two stages of which the first stage involved placing 3 m thick fill with a 180 m width and 12.5H:1V side slopes. The second stage embankment consisted of a 3 m high dike with 3H:1V side slopes and a 3.7 m top width. Recorded strain in geotextile warp and fill directions during construction of the stage 1 and 2 of the embankment construction were much less than the 5 percent assumed in design. Measured dike settlements had values that closely agree with design assumptions. Inclinometer readings revealed negligible lateral movement in the embankment or foundation.

Key findings:

- Significantly consolidation occurred during construction due to the use of vertical drains
- The mobilized reinforcement strain was well below design value of 5%.

Case History: A trial embankment over Malaysian Muar clay (Chai and Bergado 1993)

A geogrid reinforced trial embankment was constructed on soft Muar clay with vertical band drains in the foundation. To achieve a design final height of 6 m after the end of consolidation, the embankment was constructed to a thickness of 8.5 m at the end of construction. Two layers of geogrid reinforcement with a tensile strength of 110 kN/m at the peak strain of 11.2% were laid at the leveled ground surface in a 0.5 m thick sand blanket with 0.15 m vertical spacing to obtain a minimum factor of safety of 1.3 at the end of construction. The soil profile consisted of a 2 m weathered crust (with s_u = 10 - 20 kPa and w_n = 60%), a 5 m very soft silty clay layer (with s_u = 10 - 16 kPa and w_n = 80-105%) and a 10 m thick layer of soft clay (with s_u = 18 - 30 kPa and w_n = 50 - 100%). Below the clay layers there was about 0.6 m of peat overlying a thick deposit of medium dense to dense clayey silty sand. The construction was finished in three stages within a 400 day period. The settlement at the end of construction was 1.5 m due to significant consolidation. The lateral horizontal displacements of 350 mm at the toe and 450 mm in the foundation soil 5 m below the ground were observed at the end of construction. The estimated maximum tension force and reinforcement strain in each of geogrid reinforcement was 13 kN/m and 2% respectively.

Key findings:

- Significantly consolidation settlement was developed at the end of construction due to the use of PVDs and stage construction.
- The strength gain of the foundation soils due to the use PVDs and stage construction significantly increased the embankment stability. Consequently, the reinforcement was not mobilized to the extent expected at design height.

Discussion

The case histories examined above have shown that the current design methods are conservative since the field performance of the reinforced embankments under working conditions were better than anticipated. The observed reinforcement strain and (deduced force) were usually less than the design values for a required factor of safety or the values predicted for equilibrium assuming the undrained strength of the soil has been fully mobilized. This can be attributed to three primary factors. Firstly, current design methods conservatively assume undrained conditions for the foundation soils during embankment construction. However in reality, significant partial consolidation can occur at typical rates of construction. This may occur when the soil is overconsolidated during early stages of loading and is especially evident when vertical drains are used to enhance the drainage conditions (Leroueil et al. 1978; Rowe et al. 1995; Li and Rowe 1999a; Leroueil and Rowe 2001). The consequent beneficial effect of the partial consolidation on the stability is more significant for reinforced embankments than for unreinforced embankments (Li and Rowe 1999a).

The second conservatism arises from the selection of the undrained shear strength of the foundation soils. Due to the uncertainty associated with the in-situ operational shear strength of foundation soils, the design strength is often selected conservatively based on the in-situ and laboratory tests. The third reason resulting in a reinforcement strain lower than the design value is that the embankment is usually designed with a required global factor of safety greater than one in a conventional design method or the factored strength of the foundation soil used in a limit states design method. Under working conditions, the mobilized reinforcement strain and force should be lower than the design values unless the shear strength of the foundation has been overestimated in the design.

Finite element analyses (Rowe and Li 1999; Li and Rowe 2000) have demonstrated that the magnitude of reinforcement strain at working conditions typically range between 1% and 3%, which is consistent with many field observation and substantially lower than the typical design strain of 5%.

2.5 The Effect of construction rate and stage construction.

Case History: Hubrey Road embankment (Rowe & Mylleville 1996)

An extruded, biaxial PP geogrid with T_{ult}=19.2 kN/m and $J_{5\%}$ = 280 kN/m was used as the basal reinforcement for construction of a 1.3 m and 1.7 m thick embankment. The foundation consisted of 1.8 to 1.9 m of soft to firm black fibrous peat, underlain by 2.2 – 2.4 m of very soft organic silt with numerous shells overlying 0.4-0.6 m of soft fine organic silt. A fine to medium sand and firm sandy silts with some clay were encountered below the organic silt. The black fibrous peat had the water content of the organic between 250 and 700% and organic content ranging between 76 and 90%. The very soft organic silt had the water content ranging between 250 and 480% and organic content ranging between 13 and 34%.

Stage 1 construction consisted of placing the geogrid over the root mat on the soft organic deposit and then placing 1.5 m of granular fill on the top of the geogrid. At the end of construction, a maximum settlement of 0.49 m and an average settlement of 0.28 were observed. After 11 month consolidation (at the end of Stage 1 construction), the maximum settlement was 1.5 m and an average of 1.1 m. Peak strains were reached in the first 10 days after fill placement with measured values between 0.25% and 1.75%. Rowe and Soderman (1985b) discuss the importance of considering the magnitude of excess pore pressures generated during the construction of embankments on peat and recommend that construction rate be controlled such that B_{max} (i.e. maximum excess pore pressure over vertical applied embankment load) is less than or equal to 0.34. Therefore there was no evidence of problems in the vicinity of one section where the measured B_{max} had values of between 0.34 and 0.38, which were of the same order as the maximum recommended by Rowe and Soderman (1985b). However, at another section the construction rate was so fast that the generated excess pore pressure gave rise to B_{max} = 0.7 (i.e. significantly higher the recommended value). Consequently, a large rotation failure occurred at this section of the embankment.

Key findings:

- The control of the rate of construction was import to maintain the short-term stability for embankments over peat foundations.
- It was recommended that the construction rate should be slow enough to ensure that the pore pressure parameter B_{max} remained below 0.34. However, this recommendation was not followed at one section and a failure occurred when the excess pore pressure parameter B_{max} was 0.7 (>0.34).
- The geogrid reinforcement was most beneficial during the placement fill since some large tension cracks developed in the root mat just ahead of the advancing fill.

Case History: Grassy Sound highway embankment (Volk et al. 1994)

This 2 km long highway embankment was built over a root mat underlying by a 6 - 7 m thick very soft organic marine clay. The foundation was installed with vertical drains with 0.76 m spacing in a triangular pattern. The embankment was constructed to a height ranging from 2.8 to 4.7 m in 4 stages. The very soft clay had undrained shear strength of 1.4 kPa at the top to 4.8 kPa at the bottom and index properties of w_n = 90-400%, LL = 90-200%, PL = 30-80% and PI=50-130%. Beneath the very soft organic clay stratum was stiff sandy silty clay/clayey silt with thickness varying from 0.8 to 1.5 m. Two layers of high strength polyester geotextile with (T_{ult} = 438 kN/m, J = 3500 kN/m) and (T_{ult} = 730 kN/m, J = 5840 kN/m) were used for the upper and lower layer respectively.

Bearing capacity analyses indicated that the embankment fill should be placed in lifts not more than 1.4 m thick for each stage. Stability analyses indicated that T_{req} = 335 kN/m was required for a factor of safety of 1.3 at the end of construction. Using two layers of geotextile, the available tensile force at 5% was 292 kN/m for the lower geotextile and 175 kN/m for the upper geotextile (i.e. ΣT = 467 kN/m $\geq T_{req}$ and hence one might infer a design strain of 3.6% assuming a linear response and both layers straining equally. The observed total settlement ranged from 1.8 to 2.7 m and the maximum horizontal displacement ranged from 0.4 to 0.5 m. The geotextile strains after the end of primary consolidation were 4.6% for lower layer and 2.2% for upper layer. Assuming a linear geotextile force-strain curve this would correspond to a mobilized force of 346 kN/m, which is marginally above that predicted for a factor of safety of 1.3. Nevertheless, the mobilized strains were both below the design strain of 5%. The difference in strain between the upper and lower layers suggests some “bending” in the embankment. No mud-waves or cracking of the embankment was observed.

Key findings:

- Four stage construction was used so that the soft organic foundation could gain sufficient strength to permit construction to the design height without exceeding the design reinforcement strain.
- The actual strain developed in the high-strength geotextile was slightly less than the

design strain but the mobilized force after consolidation was greater than predicted in the design.
- The vertical settlement was less than the predicted settlement.
- The monitoring of reinforcement strain was combined with stage construction to ensure the reinforcement strain did not exceed the allowable strain.
- The strain was not uniformly distributed between the two layers, with the lower layer experiencing more than twice the strain of the upper layer.

Case History: Roadway embankment (Shimel & Gertje 1997)

A 4.3 m high reinforced embankment was constructed as a test embankment for the construction of a roadway in Tacoma, Washington. The embankment stability and construction schedule were primary concerns. The foundation consisted of a relatively thin layer of sandy fill material underlain by 9 to 15 m of very soft to soft, clayey silt with LL = 32-74%, PI = 3-36% and s_u = 10-19 kPa. Due to the thick deposits of soft soils, the embankment could not be constructed to the required grade in one stage. Therefore, staged construction was adopted and combined with the use of high-strength geotextile reinforcement to allow for a shorter construction period. Two layers of high strength, woven polyester geotextiles with a tensile strength of 200 kN/m were placed at the base of the test embankment.

The maximum settlement at the end of construction was 0.7 m (0.5% of the height) and the maximum measured lateral movement in the foundation was about 5.8 cm. The embankment performed well during construction except for an incident where there was an excessive rate of fill placement. In this case, at the time of the incident, a rapid increase in pore pressures and noticeable lateral movement were observed.

Key findings:
- The use of high strength reinforcement in combination with the staged construction made it feasible to construct the embankment over the soft foundation within the project schedule.
- The rate of embankment construction was controlled to maintain stability with pore pressure and horizontal displacement monitoring being used to control the rate of fill placement.

Discussion

Due to the low undrained shear strength of very soft soils, it is often essential to allow the dissipation of excess pore pressure during embankment construction. In many cases the cohesive deposits overlie relatively permeable deposits. Hence with the natural two-way drainage conditions, the partial consolidation can be achieved during embankment construction by controlling the rate of fill placement. It has shown in a number of unreinforced embankment field cases (Leroueil et al. 1978; Crooks et al. 1984) and reinforced embankment cases cited above that construction pore pressure can be significantly lower than the applied embankment pressure due to the partial consolidation that occurs during embankment construction. Finite element analyses (Rowe and Li 1999; Li and Rowe 2001a) show that the beneficial effects of consolidation is enhanced by the use of reinforcement and the partial consolidation during embankment construction can contribute to increasing the height at which a reinforced embankment would fail. Therefore, the control of the construction rate to allow the dissipation of excess pore pressures during embankment construction is a useful approach for ensuring embankment stability over soft foundations.

2.6 Reinforced embankment constructed over rate sensitive soils

Case History: Sackville embankment (Rowe et al. 1995, Rowe & Hinchberger 1998)

A fully instrumented embankment with both reinforced and unreinforced sections was constructed over a soft compressible clayey silt deposit in Sackville, New Brunswick, Canada. The foundation soils below the root mat consisted a number of sub-layers that had an average vane strength of approximately 22 - 26 kPa for the first few meters and this increased to about 40 kPa at the 10 m depth. The natural water content ranged from 40 % to 70%, the liquid limit from 42% to 76% and the plastic limit from 15% to 23%. The liquidity index exceeded the unity at depths from 1 m to 6 m. The reinforcement was a polyester woven geotextile with T_{ult} = 216 kN/m at 13% failure strain and J_{sec} = 1466 kN/m at 5% strain.

The field monitoring indicated that the embankment behaved elastically up to about 2.4 m fill thickness and that there were significant plastic deformations in the soil during the construction of the embankment from 5 to 5.7 m. The unreinforced embankment failure height was about 6.1 and the reinforced embankment failure height was 8.2 m. Rowe et al. (1995) described the failure of the Sackville test embankment as a viscous type of failure. During the construction, for fill thicknesses greater than 2.4 m, the recorded reinforcement strain and embankment deformations increased significantly with time after each embankment lift was placed. It has demonstrated that the time-dependent deformations of the embankment, which occurred during stoppages in construction, could be modelled using an elasto-viscoplastic constitutive model.

Key findings:

- The use of reinforcement increased the failure height and reduced the viscoplastic deformations of the foundation soils at a given fill thickness (relative to what would be expected for an unreinforced embankment).
- The deformations and excess pore pressures increased after the end of construction due to the undrained creep of the rate sensitive foundation soil.
- The failure height of the unreinforced embankment was lower than the predicted.
- The rate of loading had an effect on the apparent undrained shear strength.
- The vane shear strength over estimated the strength available during construction due to the strain rate effects on the undrained shear strength of this rate sensitive clay.
- The critical time for stability may not be during construction, but at some time following construction.
- There was no sudden or brittle failure but rather a quite ductile failure even when the reinforcement appeared to have failed. This is attributed, in part, to the ability of the soil to carry overstress but then creep to failure.
- Special care is required when constructing embankments over rate-sensitive soils.

Discussion

It is well known that natural soft cohesive soil deposits exhibit significant time dependent behaviour and the undrained shear strength of natural soft clays is strain rate dependent (Casagrande and Wilson 1951; Perloff and Osterberg 1963; Bjerrum 1972; Graham et al. 1983; Leroueil and Marques 1996; Sheahan et al. 1996). Due to the viscoplastic nature of some soft clayey foundations, embankments often experience significant post construction creep deformations or even failure when excess pore pressures increase or remain at a nearly constant level following the completion of construction (Crooks et al. 1984; Kabbaj et al. 1988; Rowe et al. 1996). The rate sensitive characteristics of soft foundation soils have significant influence on the behaviour of reinforced embankments (Rowe and Hinchberger 1998; Li 2000). The tension mobilized in reinforcement at the end of construction may be significantly different from that assumed in convention design approaches (Rowe and Li 2000).

Rowe and Hinchberger (1998) attributed the time dependent behaviour of Sackville embankment to the viscoplastic behaviour of the sensitive foundation soils. The finite element analyses indicate that the reinforcement strain can increase significantly after the end of construction due to the creep of foundation soil (Li 2000). It has been shown that the undrained shear strength based on field vane tests may significantly overestimate the operational undrained strength of the rate sensitive soils (Rowe and Hinchberger 1998; Rowe and Li 2000). The measured undrained shear strength should be corrected for the design of embankments over rate-sensitive soils. A correction factor allowing for the strain rate effects on the operational strength of foundation soils was proposed by Li and Rowe (2000).

2.7 The increase of reinforcement strain after construction

Case History: A reinforced embankment (Litwinowicz et al. 1994)

A 2.8 m high embankment was constructed on 4 - 10 m deep very soft to soft marine clay underlain by sandy deposits, in Australia. The natural water contents ranged between 40 and 120%; the liquidity indices were between 1.5 and 2.5; the undrained shear strength was 5 -12 kPa with linear increase with depth. One of the two embankment sections was reinforced with an HDPE geogrid having T_{ult} =

200 kN/m and J_{sec} = 3500 kN/m at 3% strain. The other section was reinforced using a polyester geogrid with T_{ult} = 200 kN/m and J_{sec} = 2100 kN/m at 3%. The design reinforcement force T_{req} was 60 kN/m at assumed tensile strain of about 3% for a factor of safety of 1.3.

The maximum strain of the polyester geogrid and HDPE geogrid reinforcement at the end of construction was reported to be 1.65% and 0.4% respectively. The corresponding mobilized force was reported to be 26 kN/m the geogrid and 28 kN/m for the geotextile. These measured values were essentially half or less of the design value. The maximum strain of in HDPE geogrid increase by 100% after the end of construction and the strain of PET geotextile increase slightly.

Key findings:

- The increase of HPDE geogrid strain at constant embankment load was mainly attributed to creep of the geogrid. However the magnitude of increase (i.e. 0.5%) was significantly lower than the predicted value 2.5% by the isochronous creep curves. This could be due to the fact that the creep of the reinforcement can be limited by the soils when the embankment is under working conditions (Li and Rowe 2001b).
- Both sections showed a slight increase in reinforcement strain and force due to foundation deformations during consolidation.
- The mobilized forces for both embankment sections with the same thickness were similar, but significantly lower than the design force.
- The partial consolidation during construction and the fill strength contributed to the low level of strain mobilization in both types of reinforcement.

Case History: A trial embankment (Bassett & Yeo 1988)

A 7 m high trial embankment reinforced with a uniaxial HDPE geogrid (T_{ult} = 79 kN/m and $J_{5\%}$ = 1080 kN/m) was constructed across a 4.5 m thickness of soft clay /peat/soft clay installed with vertical drains. At the end of construction, the maximum geogrid strain and force were about 2% and 14 kN/m, respectively. Data obtained from reinforcement load cells showed the loads developed rapidly during construction (T_{max} = 14.0 kN/m) and continued to increase with time for the monitored 400 days after construction (T_{max} = 16 kN/m). This finding was contrary to the design assumption that the maximum reinforcement force would occur at the end of construction and that thereafter stress relaxation would result in a slowly decreasing load. The strain increased from 2% to 3% (i.e. by 50%) and the load in the geogrid at the same position increased by 10%. Even though the reinforcement strain and force increased after construction, the mobilized values were low compared to the design strain of 5% and design force of 25 kN/m to 30 kN/m.

Key findings:

- The increase of reinforcement strain and force after the end of construction was contrary to the design assumptions.

Discussion

It has been shown that the reinforcement strain does not always reach the maximum value at the end of construction and that the reinforcement strain increased after the end of construction in a number field cases. In the cases of Reach A test embankment (Fowler & Edris Jr 1987) and New Bedford dike (Schimelfenyg et al. 1990), the increase in reinforcement strain was mainly attributed to the effect of consolidation deformations of the foundation on the reinforcement deformation. The significant increase in strain in the high strength polyester reinforcement used for the Sackville embankment (Rowe et al. 1995) at a constant fill thickness was caused by creep of the foundation soil. In addition to the effects of consolidation and creep of foundation soils, the viscous nature of geosynthetics can also contribute the increase of reinforcement strain after construction under constant embankment load illustrated by the above field cases. In these two cases the trend of increasing of reinforcement strain and force observed are consistent with the findings from FEM analyses (Li and Rowe 2001b). In summary, the increase strain can be associated with the creep of reinforcement, creep of the foundation, and consolidation (e.g. the differential consolidation settlements). When arising from foundation deformation this increase in strain will also correspond to an increase in reinforcement force.

2.8 Embankments over peat

Case History: Bloomington Road embankment (Rowe et al. 1984a, b)

A 1 – 1.5 m high geotextile-reinforced embankment (with 5.7 m fill thickness) was constructed in three stages over a 1 year period on a highly compressible peat deposit in Aurora, Ontario, Canada. The thickness of the peat in this deposit varied between 5 m and 7.6 m. The average water content of the peat was 445% and 785% for at the two investigated Sections A and B and the peat deposit was underlain by silty sand and sand. Section A was reinforced by a woven geotextile with T_{ult} = 41 kN/m and Section B was reinforced with another woven geotextile with T_{ult} = 178 kN/m. The maximum vertical settlement observed was 3.2 m and 4.6 m and the maximum reinforcement strain measured was 21% and 5% for Sections A and B respectively.

Key findings:

- The high modulus geotextile used in Section B reduced lateral movements.
- As the extent of local plasticity in the foundation increased, the reinforcement played an increasingly more important role in reducing the shear deformations and increasing stability.
- Even a very high modulus geotextile did not prevent large consolidation settlements of this embankment on peat.
- In the analysis of embankment performance, the use of effective deformation and strength parameters (combined with pore pressures) provided the best agreement between calculated and observed behaviour.

Case History: An experimental highway embankment (Matichard et al. 1994)

A highway by-pass in France involved the construction of a 5.8 m high experimental embankment on a peat bog over a period of 20 days. The foundation consisted of a 1.5 - 3 m peat layer overlaying a 1 - 2 m thick clay layer and 1 – 1.8 m clayey gravel layer underlain by the bedrock. The water content ranging between 150% and 319% for the peat and between 44% and 86% for the clay, which had an undrained shear strength 40 - 60 kPa. Two layers of geotextile reinforcement with T_{ult} greater than 120 kN/m were used to prevent a rotational failure that had been predicted to occur at the design height if unreinforced. It was assumed that a significant fraction of the peat located under the embankment would be displaced laterally during the construction of the embankment.

The maximum settlement was 0.4 m at the end of construction and this increased to 0.6 m due to consolidation during the first 100 days after construction. The maximum horizontal displacement at the base of embankment was 50 mm and this increased to 82 mm after 100 days. The maximum strain in the upper geotextile was 0.7% at the end of construction and 0.8% after 100 days; the maximum strain in the lower geotextile was 1.3% at the end of construction and 1.7% after 100 days.

Key findings:

- The primary goal of removing the peat by punching was not achieved. This may be attributed to the increase of effective stress in the peat due to the relative rapid dissipation of the excess pore pressure.
- The construction settlement consisted about 67% of the total measured settlement.
- The use of geotextile reinforcement allowed the construction of the embankment to the final design grade without failure.
- The maximum tensile force calculated from the average strain was about 20 kN/m in the lower geotextile; this was only about one third of the design value.
- There was a significant (factor of two) difference in strain between the upper and lower reinforcement layer.

Case History: A road embankment (Oikawa 1996 et al.)

A road embankment was built on a peat deposit with a thickness varying from 1.2 to 11.3 m. The typical cross section of the embankment and foundation consisted 6 m high embankment with 12 m width, 4.4 m – 7.3 m fibrous and amorphous peat layers, 1.7 - 3 m clayey silt layer, 1.5 m sandy silt and gravel layer. The fibrous peat had a natural water content w_n = 400% and the paper reports an average unconfined compressive strength s_u = 11 kPa although the validity of this strength for a fibrous peat is subject to question. The amorphous peat had w_n = 260% and s_u = 20 kPa. The underlying clayey silt had w_n = 90%, s_u = 28 kPa and sandy silt w_n = 35% and a reported s_u = 127 kPa (also subject to question if this is sandy silty). The stability analyses indicated that the embankment could only be constructed to 1.8 m without reinforcement. At the design height of 6 m, the use of reinforcement increased a factor of safety of from 0.46 to the required 1.15 immediately after the construction. Five layers of geogrid reinforcement were used in a conjunction with a two-stage construction sequence.

One layer of geogrid with a tensile strength of 17.7 kN/m, was placed directly on the ground and four layers of geogrid, each with T_{ult} = 108 kN/m, were installed at a 60 cm vertical spacing above 1 m thick sand working platform. The Stage 1 construction consisted of the placement of 1.8 m fill (i.e. equivalent to the predicted unreinforced embankment failure height) in about a one month period. This was achieved with two layers of geogrids reinforcement in the embankment and one layer on the ground. There was no evidence of rotational failures, tension cracks or large surface waves after placing the first 1.8 m fill placement. At the end of construction Stage 1 the maximum measured outward horizontal displacement was about 2-3 cm at the interface of fibrous-amorphous peat 3 m below the ground surface and this hardly changed during consolidation. Only 5 cm of heave was observed in Stage 1.

Stage 2 construction took about one month and began ten months after completion of Stage 1. It involved construction of the embankment to the design height of 6 m with two additional reinforcement layers. During the second stage of construction about one month, the maximum horizontal displacements of 6-8 cm at a depth of 3 m and very small heave (10cm) at the toe were observed. The measured settlements below the centreline and below the shoulders were almost same during both stages. The reinforced embankment performed like a rigid footing with relatively uniform settlement. The settlement at the end of construction was about 175 cm, which was 29% of the embankment height. Rapid settlements were observed during the fill placing periods in both construction stages and the magnitude of the rapid settlements amounted to more than 70% of the ultimate settlement. The embankment was safely constructed to the design height of 6 m in two stages.

Key findings:

- The construction of a relatively high embankment on peat deposits was made possible by reinforcing the embankment using multi-layers of geosynthetics.
- The reinforcement layers resulted in the rigid-footing-like behaviour of the embankment over a relatively thick peat deposits.
- The most of the consolidation settlement occurred during construction due to the rapid dissipation of excess pore pressure in the peat layers.
- The combined effects of reinforcement and rapid consolidation of peat resulted in relatively small shear deformations of the foundation subsoil during and after the fill placement.

Case History: A highway embankment (Kerr et al. 2001)

A two-lane highway embankment with a total fill thickness in the order of five meters including surcharge was constructed over a peat foundation paralleling to the existing two-lane Highway 69 north of Port Severn in Ontario, Canada. The stratigraphy of the foundation soils consisted 2.5 m saturated peat overlying 2.5 to 5.5 m of soft to firm clay overlying bedrock. The peat was dark brown to black with moisture contents varying from 92% to 581% with an average of 320% and the undrained shear strength was estimated as 5 kPa. The soft to firm clay had moisture contents decreased with depth from 10% to 60% and undrained shear strength of 5 kPa in the upper portion and 10 kPa in the lower portion. A uniaxial HDPE geogrid with design strength of 61.3 kPa and a secant stiffness of 21.9 kPa at 2% was used as reinforcement. The embankment was designed with a minimum factor of safety of 1.4. The embankment was stable during construction and showed no evidence of distress at this time. A tension crack was observed approximately 10 months after completion of construction as a result of differential consolidation settlement. The maximum total settlement of 1.02 m was observed after 19 months. However, almost half of this total settlement was developed during construction. The construction had no serious effect on the adjacent existing embankment during the monitoring period.

Key findings:

- Using geosynthetic reinforcement, the embankment was safely constructed over peat deposits without influencing the adjacent highway embankment.
- The use of reinforcement eliminated the need for removing peat and reduced the construction time in half.
- The geosynthetic reinforced embankment mitigated a number of environmental concerns by reducing the effect on wet land, avoiding dredging and disposal of foundation soils and avoiding contamination of groundwater that may have occurred if the existing foundation soil had been undisturbed.

Discussion

Peats typically have a high natural water content (50 to 2000%), high void ratio (5 to 15 but may be up to 25), and are high compressibility. The hydraulic conductivity of peat is typically high at high void ratios but reduces significantly as the peat compresses. Due to the fact that the porous nature of peats usually allows some dissipation in excess pore pressure during construction at typical rates of construction, the undrained bearing capacity type of failure in peat foundation is relatively rare. The failures of embankments over peats are usually caused by excessive shear deformations of embankments rather than definitely sliding surface. It has been shown that the rate of construction and dissipation of pore pressures is critical to maintain the embankment stability (Rowe and Mylleville 1996). Since the initial effective stress within peat is very low, if loaded too quickly, the only shear strength available to carry the load is a small apparent cohesion (due to the tensile strength of the fibres in the peat) and the tensile strength of any root mat. Effective stress analyses, rather than total stress analyses based on the "undrained shear strength", are applicable to embankments over peat foundations and give good predictions (Rowe et al. 1984b).

2.9 Summary

Numerous embankments have been successfully constructed on soft foundations using geosynthetics. The field cases summarized in this Section demonstrate the effectiveness of geosynthetic reinforcement and key findings from the use of geotextile and geogrid reinforcement and prefabricated vertical drains in embankment construction over soft foundations. The foundation soils were typically characterized by a high water content and either a fine grained composition or a high organic content. Geosynthetic reinforcement and/or PVDs allowed construction on these difficult foundation soils within prescribed construction schedules and performance criteria.

Reinforcement increased the factor of safety against rotational failure and served to maintain the structural integrity of the embankments. The bearing capacity of the foundation soil was also increased due to use of reinforcement. The reinforced embankment failure height is usually greater than that calculated based on classical bearing capacity theory using the Prandtl solution for a strip footing on a deep homogeneous clay layer (i.e. $H_f = 5.14\ s_u$) due to the typical presence of either a significant increase in undrained strength with depth in foundation soils and/or a firm stratum at relatively shallow depth beneath the soft layer. The instrumented results show that the use of high tensile strength, high modulus geosynthetics effectively reduced the movements at ground level, minimized lateral spreading, and reduced differential consolidation settlements.

To reach a design grade it is sometimes necessary to combine reinforcement with a staged construction method so that the foundation soil can have sufficient strength gain for the final embankment load. It has been shown that the effect of strength gain of foundation soils due to partial consolidation is enhanced by the use of geosynthetic reinforcement. It has also shown that the performance of reinforced embankments is sensitive to construction rates. At typical construction rates partial consolidation can be significant. This is especially true when PVDs are installed or when the foundation includes fibrous peat layer. The consequent strength gain in the foundation can effectively reduce reinforcement strain developed at the end of construction. Therefore, field monitoring is essential to controlling the rate of fill placement during construction so that excess pore pressures are allowed to dissipate sufficiently during construction.

The magnitude of the mobilized reinforcement strain and force have been found to be significantly less than the values used or expected with current design methods in numerous cases where the embankments were constructed over conventional soft soils. This is mainly attributed to the conservative assumption of undrained conditions used in most current design methods combined with conservative assumptions associated with soil strength.

Reinforced embankments constructed on rate sensitive soils require special care. For such foundation soils, the most critical time with respect to embankment stability may be at some time after the end of construction rather than during construction. Thus, the reinforcement strain and force can increase substantially with time due to the creep of the foundation soils. A contributing factor to the time dependent behaviour of reinforced embankments is the viscous behaviour of geosynthetics especially those made of high density polyethylene and polypropylene. In these cases, the reinforcement strain was seen to increases with time due to creep of geosynthetics under constant embankment load. There has paucity of data showing the stress relaxation of reinforcement after construction. This is considered to be because the post construction horizontal deformations of foundation soils resulting from creep and consolidation deformations give rise to an increase in reinforcement force that can offset the stress relaxation in reinforcement (Li and Rowe 2001b).

3. REINFORCED SOIL WALLS

During the past thirty years the design of reinforced soil structures and their behaviour during the design life have been extensively investigated through both theoretical and experimental studies. The typical reinforced soil wall structures have four types of facing units, namely, full height panel, incremental panel, modular, and wrapped-around facing. The failure modes for reinforced soil retaining walls are widely categorized in the literature as either internal failure modes or external failure modes. The internal failure modes include typically involve either breakage or pullout of the reinforcement. The external failure modes include: (a) sliding of the reinforced soil mass; (b) bearing capacity failure; (c) overturning of the wall; and (d) general slip surface failure.

Limit equilibrium methods are used extensively for the design of reinforced soil walls. Classical soil mechanics methods have been found satisfactory for the analysis of the external stability of reinforced soil structures. Internal stability considerations require that reinforcement be able to carry the tensile forces transferred to them by the soil without rupture. Also, there must be sufficient bond between the reinforcement and the soil in the resisting zone for the reinforcement to develop the required force without pull out. The basis for the internal design is to evaluate the required spacing and lengths of reinforcements so as to satisfy these conditions. The conventional Tie-back Wedge method of limit equilibrium analysis is widely used to design geosynthetic-reinforced soil walls (especially in North America). An addition internal design consideration concerns the durability of the reinforcement. Reinforcement deterioration can result from corrosion, creep, and chemical and biological attack of the different reinforcement materials.

Numerous field cases have shown that the conventional design is conservative. This may be due to the uncertainties associated with the time-dependent properties of geosynthetic, complexity of the behaviour of the reinforced soils walls due to the interactions between different components, and the uncertainties associated with construction. The behaviour of reinforced walls with granular backfills has been extensively investigated. However, the behaviour of reinforced walls with cohesive backfills is not yet fully understood. Similarly, little work has been done on walls constructed over foundations that are not very stiff. Creep of geosynthetic reinforcement contributes to the complexity of time dependent behaviour of reinforced soil walls. A number of field cases will be reviewed in following sections to highlight some factors relating to these issues.

3.1 Soil walls with granular backfills

Case History: Algonquin full scale test wall (Simac et. al. 1990)

A full scale 6.1 m high by 15 m wide geogrid reinforced soil test wall was constructed at a stone quarry in Algonquin, Illinois, USA using a continuous filament polyester geogrid and 200 mm high incremental facing units. The geogrid reinforcement had an ultimate tensile strength of 39 kN/m at failure strain 15% and a long-term allowable design load of 15.5 kN/m. Eight layers with a length of 4.3 m were used with an average spacing of 0.75 m between layers. The reinforcement was connected to the facing system through friction between courses. The test wall was constructed with very low factor of safety (i.e. long term internal factor of safety less than 1.1 without surcharge loading) based on limit equilibrium calculations using a tied-back wedge model. An inclined 2.1 m high surcharge was placed on the wall 104 days after construction to mobilize the reinforcement tension while the system was monitored.

The inclinometer data obtained during construction indicated a relatively uniform rotation in the soil mass with the maximum lateral movement at the top of the wall of 37 and 20 mm at the front and back of the reinforced soil respectively. The deduced lateral strain in soil mass was between 0.5% and 1%. The measurements of the strain profiles of the reinforcement layers under the inclined surcharge indicated that the locations of the maximum reinforcement force were consistent with the theoretical Rankine failure plane. The maximum reinforcement strain under the applied surcharge was 0.95% (corresponding to a tensile force of 9.2 kN/m) in the 2 m high geogrid layer, which did not change significantly after the placement of surcharge. During the 15 month monitoring program, there was no significant time dependent reinforcement strain observed in the geogrid. The earth pressure measurement indicated a relatively uniform bearing pressure distribution at the base of the wall.

Key findings:

- The instrumentation data was found to confirm the applicability of a design methodology using limit equilibrium analysis and the tied-back wedge model.
- The polyester geogrid reinforcement did not experience significantly time dependent deformations.
- The modular facing system and friction facing connection performed well.
- Strains associated with wall construction were sufficient to develop the Rankine active state of stress and the Rankine stress distribution appeared to be most appropriate for the magnitude and location of maximum reinforcement force.

Case History: Highbury avenue wall (Bathurst 1991)

An instrumented soil retaining wall 125 meters in length was constructed to a maximum height of 7.1 m in London, Ontario, Canada. The facing consisted of full height reinforced concrete panels propped during placement and compaction of the granular fill. The retained backfill comprised a coarse sand fill and was reinforced by using layers of uniaxial polyethylene geogrid. The movements of inclinometers attached directly to the back of the panels and at 1.5 m from the back of the panels suggested that the volume of soil with significant deformation was reasonably well defined by the potential Rankine failure plane originating at the toe of the wall and propagating up through the retained soil. The wall deformations increased with time after the prop release during a 631 day monitoring period. The maximum post construction out-of-alignment movement of the wall was about 1.2% of the height of the panels. The maximum recorded reinforcement strain was 3.5% and high connection strain was observed. The high connection strains were due to the downward relative movement of the retained soil with respect to the relatively rigid facing.

Key findings:

- The rigid panel facing gave rise to high connection strains in the reinforcement.
- The magnitude of lateral wall movements appeared to have been more sensitive to the quality of construction rather than panel height.
- The strain in geogrids at locations along the potential Rankine failure plane increased with time during post construction due to the creep of the reinforcement.
- Creep of reinforcement contributed to the time dependent deformation of the wall.

Case History: An instrumented geogrid soil reinforced wall (Fishman et al. 1993)

A 4.72 m uniaxial extruded HDPE geogrid (T_{ult} = 79 kN/m; $J_{5\%}$ = 1080 kN/m; T_{allow} = 19.3 kN/m) was used to reinforce a wall with a full-height concrete facing in Tucson, Arizona, USA. The 3.66 m long geogrid layers were mechanically connected the concrete facing panels at various elevations. It was assumed in design that no lateral earth pressure would be transferred to the wall face since it should be transferred to geogrids as tension. However, the lateral earth pressure against wall facing was measured to be 12 kPa near the bottom of the wall, 2.5 kPa near the mid-height and about 5 kPa near the top.

Tensile strains in the geogrids ranged between 0.3% and 0.8% with corresponding forces of 3.3 to 8.7 kN/m. These mobilized forces were significantly less than the allowable design strength of 19.3 kN/m for this particular geogrid reinforcement. The predicted reinforcement strains based on the design equation were higher than the measured values with closer agreement for the upper layers than the lower layers. There was evidence to suggest that there had been relative movement between soil and geogrid at locations close to the wall face.

Key findings:

- The design method was conservative as indicated by the lower than expected reinforcement force mobilized in field.
- A full-height rigid concrete facing may attract lateral earth pressure and partially inhibit load transfer between the soil and extensible reinforcement.

Case History: RMC reinforced walls (Bathurst 1999; Bathurst et al. 2000)

A long-term research program involving the construction of over 10 reinforced full-scale geosynthetic reinforced soil walls is in progress at the Royal Military College, Kingston, Ontario, Canada (RMC). All walls are constructed on a rigid (concrete) foundation. Six of these walls reported by Bathurst (1999) and Bathurst et al. (2000) include one 3 m high full-height panel wall, one 3 m high incremental panel wall, one 3.6 m wrapped-face wall and three 3.6 m high segmental retaining walls. Different layers of weak biaxial polypropylene geogrids with different strength and stiffness were used. The full-height and incremental panel walls had 4 layers of reinforcement with a tensile strength of 12 kN/m; the wrapped-face and the first segmental wall had 6 layers of reinforcement with a strength of 14 kPa; the second segmental wall had 6 layers of reinforcement with a strength of 7 kN/m; and the third segmental wall had 4 layer of reinforcement the same as that for the first. All walls were applied with a surcharge by a loading facility to stress levels well exceeding working load conditions.

The monitoring program included the monitoring the reinforcement strain, wall deflection, reinforcement connection load, and horizontal load acting on the toe and vertical pressure at the base of the wall. In general, the deformation of the system increased with the magnitude of the surcharge and the time-dependent deformations due to the reinforcement increased with a constant surcharge. The failure planes observed for the full and incremental panel walls were consistent with the Coulomb wedge prediction. For the full panel wall, the largest strains were observed at the reinforcement connection behind the wall and the surcharge capacity was higher than that of the incremental panel wall due to the effect of the rigidity of the wall facing. The restrained toe attracted a significantly portion of the lateral earth force acting on the back of the full-height panel wall.

During the construction and surcharge of the first segmental wall, creep deformations were observed during each load increment and the maximum reinforcement strain was observed at the connections at the end of construction. Significant time dependent reinforcement strains were observed during the periods with a constant surcharge load for the second modular wall. Under the highest surcharge load, the distribution of the peak strain in reinforcement layers for all three segmental walls indicated a log-spiral failure plane, which can, in practice, be approximated by a simple Coulomb failure plane. For the segmental walls the amount of wall deflection at the end of construction at the top of the facing column was between 2 and 4% of the height of the wall. The wrapped-face wall had the higher horizontal movement than other walls and the reinforcement strain as high as four times the maximum reinforcement strain for the comparable first modular block wall was recorded at the end of construction.

Key findings:

- The connection loads were largest for the wall with a modular block facing.
- The toe of the wall facing carries a significant portion of the horizontal forces from the backfill acting on the wall facing. This load capacity (which is neglected in design) contributes to the conservatism of current design methods.
- Due to soil down drag forces, the vertical load acting at the toe is much greater than the sum of the block weights.
- Current Coulomb earth pressure theory overestimates reinforcement and connection loads.
- High quality data from large-scale, well-instrumented geosynthetic reinforced soil walls is required to guide the development of

rational methods of analysis and design of these structures.

- The strain in the PP geogrid reinforcement under a constant load increased with time due to creep.
- A relatively rigid wall facing reduces the reinforcement strains that would otherwise develop in a wrapped-face wall.

3.2 Soil walls with cohesive backfills

Case History: A geogrid reinforced retaining wall (Burwash and Frost 1991)

A 9 m high retaining wall reinforced with a uniaxial extruded HDPE geogrid (T_{ult} = 79 kN/m and $J_{5\%}$ = 1080 kN/m) was constructed in Calgary, Alberta, Canada in the spring of 1984. Foundation soils consisted of a deep deposit of very stiff low plastic clay till with the groundwater table well below the ground surface. The backfill, consisting of low plastic clay till (25% sand, 50% silt and 25% clay), was compacted to minimum of 95% standard Proctor dry density. The lateral earth pressures based on Rankine theory were calculated in the design by treating the backfill as friction material with an internal friction angle of 30^0. The wall was reinforced by up to 10 layers of geogrid with lengths up to 6.8 m. The length of geogrid (L) to height of wall (H) ratio (L/H) was at least 0.7. A factor of safety of 1.5 was applied to the long-term design load to give the allowable strength used in the internal stability calculations to establish the spacing of the reinforcement layers. Conventional stability analyses showed that the factor of safety exceeded 1.5 for global or deep-seated failure and 2.9 for the overturning.

The wall performed satisfactorily for 16 months when signs of settlement were first observed in the fill behind the wall. Conditions gradually deteriorated and over the next 22 months settlement of the backfill approached 0.9 m in one area. The top of the retaining wall rotated outward about the toe and a deflection of 310 mm was recorded with a slope indicator over a 17 month period, which was 3.4% of the height of the wall or 2^0 rotation about the toe. The average moisture content of the upper 3 m clay backfill had increased from 10.5% at placement to 16.5-17.5%, which was 1.5% to 3.0% above standard Proctor optimum.

Key findings:

- The saturation of the clay backfill and consequent loss of the soil strength has caused distress of the retaining wall.
- The unexpected large settlement of the wall was a result of the collapse of the clay backfill due to water infiltration.
- Possible poor compaction may have contributed to the excessive deformations of the wall.
- The provision of surface drainages is important for cohesive backfills since the saturation occurred by ponding of surface runoff near the face of the wall in this case.

Case History: A reinforced soil wall (Itoh et al. 1994)

A full scale 7.5 m high by 15 m wide geogrid reinforced soil wall was constructed of cohesive soil with the purpose to evaluate the applicability using cohesive soil. The backfill soil consisted of 23.3% sand, 53.3% silt and 23.4% clay with w_{opt} = 25.4%, LL = 54.6% and PL = 26.8% and was reinforced by 11 layers of geogrid (T_{ult} = 79 kN/m and $J_{5\%}$ = 1080 kN/m) in combination of 6 layers of nonwoven geotextile as horizontal drainage layers. The facing was a geogrid wrap-around with sand bags. A layer of 1 m surcharge was placed on the backfill upon the completion of the reinforced wall.

The performance of the wall was highly time dependent. The maximum geogrid strain was about 0.6% at the end of construction and increased to 3.1% over a 5-month period after the end of construction. This magnitude of strain was reported to correspond to 40 kN/m tensile force, which was higher than the design value of 31.4 kN/m. The locations of maximum strains in all geogrid layers were observed close to the facing. The deformation of the facing gradually increased during the post-construction period from 120 mm at the top 15 days after the end of construction to 380 mm 5 month after the end of construction. Five months after the construction, the settlement in the backfill was 58 cm at 3.5 m from the wall face and 88 cm at the wall face. The pore water pressure measurements showed that the pore water pressure immediately after the construction was negative and increased to about 29 kPa near the centre five months later due to heavy rainfall. The backfill behind the assumed two-part wedge slip surface experienced significant deformations and there was considerable differential settlement between the facing and the backfill.

Key findings:

- A reinforced soil wall can be constructed safely using a cohesive soil although the deformation may be greater than expected.
- The performance of the reinforced cohesive soil wall exhibited significant of time dependent behaviour. Both the variation of pore pressure in the backfill due to the rainfall and creep of the reinforcement may result in the time dependent deformations of the wall.
- The two-part wedge slip surface commonly assumed in design for reinforced soil walls may not be suitable for walls with cohesive backfill as indicated by post construction deformations of this wall.

Case History: Two Denver test walls (Wu 1991)

Two 3 m high and 1.2 m wide geosynthetic reinforced walls, one with a cohesive backfill and the other with a granular backfill, were constructed at University of Colorado in Denver, USA. Both walls were constructed with an timber facing, reinforced by 12 layers of a nonwoven heat-bonded polypropylene geotextile. The reinforcement sheet was nailed to the wall face. The granular backfill soil was subrounded Ottawa silica sand and the cohesive backfill was a slightly silty, fine to coarse and clay mixture with LL = 37%, P=19% and w_n=19.3%. The walls were allowed to creep for a period of 100 hours; thereafter, the surcharge pressure was increased until a failure condition developed or until the capacity of loading mechanism was reached.

For the granular backfill wall the wall experienced relatively large deformations when the surcharge pressure was increased from 186 to 207 kPa; the maximum wall displacement was 198 mm at the mid height of the wall. For the cohesive backfill wall, the largest deformation of the wall occurred as the surcharge pressure was increased from 207 to 227 kPa; the maximum wall movement was 241 mm at the mid height of the wall. The measured reinforcement strains indicated that the creep strain of the geotextile in granular backfill during a 36 hour period was insignificant and the creep strain of the geotextile in the clay backfill during a 100 hours period was significant at a surcharge pressure of 103 kPa. The maximum strains were about 6.0% in the 0.15H high layer under the surcharge of 186 kPa and 5.5% in the 0.88H high layer under the surcharge of 227 kPa layer for the granular backfill wall and cohesive backfill wall respectively.

Key findings:

- The failure surcharge pressure was far greater than that indicated by current design methods for both walls.
- Well-controlled full-scale tests are useful for verification of analytical models and validation of design methods.

Case History: Barren river plaza retaining wall (Leonards et al. 1994)

A geogrid reinforced retaining wall with heights varying between 3 and 6 m was constructed in Glasgow, Kentucky, USA as a part of the development of a shopping centre. The wall had Keystone block facing and a compacted clay backfill with a slope about 1V:2H. Prior to the development, the foundation at the site consisted of a of 7.5 – 10 m thick layer of a silty clay to clayey silt with some sand overlying bedrock. The silty clay soils were classified as highly plasticity with liquid limits in the range of 50-65%, plastic limits of 25-35 and natural water content slightly below the plastic limits. Although, the initial design of the wall was based an assumed granular backfill, the on-site fine materials were used for the backfill because no granular materials was available on site. Shortly after construction distress was observed on the backfill slope. This was associated with large wall deformations which were found to be related to heavy rainfall and the swelling nature of the backfill. At one section the relative displacement of the wall was about 0.28 m with essentially all of the movement in the bottom 0.6 m. The deterioration of the wall with time eventually resulted in the collapse of a 21 m section.

Key findings:

- The collapse of one section of the wall was a result the omission of the upper layer of geogrid reinforcement.
- The backfill behind the geogrids did not meet compaction specifications due to poor compaction control. As result, the backfill was compacted dry of optimum and this

resulted in a large loss in strength during the periods of heavy rainfall.

- The large outward movement of the wall in other sections was caused by the misplacement of geogrid layers due to the sloping bedrock.
- Had a proper site investigation been completed, the properties and variability of soils, ground water and surface runoff conditions would have been identified,
- A geogrid reinforced was recommended without realizing that the space needed for construction was not available.
- Care must be taken with compaction to control the water content and density to avoid the poor postconstruction performance of the wall.

Discussion

The use of the cohesive soil as wall backfill during the construction of reinforced soil walls is feasible provided that special care is taken. The cases cited above have shown that the post construction behaviour of reinforced soil walls are very much dependent on the compaction quality of cohesive backfills especially for the backfill soil with swelling characteristics (which should be compacted so as to minimize/prevent swelling after periods of heavy rainfall). Poor compaction can give rise to large post construction deformations due to the infiltration of water. The saturation of the clay backfill and consequent loss of the soil strength can cause the failure of retaining walls. To reduce the infiltration of rainwater during rainfalls periods, it is essential to install the surface drainage facilities that can eliminate ponding and promote rapid runoff of the rainwater.

3.3 Soil walls with creep susceptible reinforcement

Case History: Seattle wall (Allen et al. 1991)

A 12.6 m high geotextile reinforced wall with a wrap-around facing was constructed in Seattle, Washington to retain a surcharge fill more than 5 m in height for a future bridge abutment. The foundation soil consisted of 6 m thick of dense granular materials overlying 0 – 15 m thick of soft silty clays and clayey silts, which was underlain by very dense glacial deposits. Four types of geotextiles with a constant vertical spacing of 0.38 m were used at different height with the higher stiffness for the upper portion and lower stiffness for the lower portion of the wall. The design was based on a conventional tie-back analysis and the maximum reinforcement strain was predicted to be in the order of 2.5% to 3.5% at the final load applied.

The wall facing experienced a maximum of about 150 mm horizontal movement at the mid-height and 90 mm at the top of the wall 9.5 months after construction. The post construction deflection of the wall face was small near the bottom of the wall but increased to about 30 mm near the top of the wall. Significant settlements were also observed and these were likely due to the consolidation of the foundation materials. The settlement was greatest at the wall face and least near the middle of the reinforced section. This was consistent with the measured magnitude of vertical stress at the wall base. The maximum reinforcement strain measured by strain gages was 0.5% and based on the extensometers the strains ranged from 0.7% to 1%. The observed strains were significantly lower than the values expected in design. The surcharge resulted in relatively small increases in strain (i.e. less than 0.05%) in the lower reinforcement layers and relatively greater increases (i.e. 0.1% to 0.2%) in the upper layers. The measured horizontal strains in soil were in the order of 1% to 2% and maximum soil strain at the wall face under the surcharge was greater than 7%. The difference between reinforcement and soil strains suggested that some slippage occurred between the soil and reinforcement. The strain measurements indicated that there was creep of the geosynthetics after construction; however, the magnitude of the creep strain was less than the creep strain predicted based on in-isolation creep tests.

Key findings:

- The observed reinforcement strains indicated the design method was conservative for this relatively high soil wall.
- The measured field reinforcement creep rates were lower than predicted based on in-isolation creep data.
- The higher the reinforcement layer, the greater the mobilized force, the greater the creep strain after construction, and the larger the post construction deformation of the wall face.
- The creep of reinforcement did not affect the vertical stresses at the base of the wall.

- The locations of the peak strain in the geotextile layers followed a curved surface.

Case History: A tilt-up panel wall (Knight and Valsangkar 1993)

A 6.1 m high and 389.5 long wall was constructed in Fredericton, New Brunswick, Canada using a uniaxial HDPE geogrid reinforcement and individual precast waffled concrete tilt-up panels. The primary geogrid soil reinforcement was mechanically attached to the panel at a vertical spacing of 1.22 m. Additional unattached geogrid reinforcement was placed between attached geogrid layers to increase internal stability. Based upon design calculations, a total of 8 layers of medium strength (120 year design strength of 16 kN/m) uniaxial HDPE geogrids, and one layer of low strength (120 year design strength of 8 kN/m) HDPE geogrid were required. The maximum reinforcement strain observed was less than 0.5% at the end of construction and increased to 1.4% during a 14 month monitoring period. The wall panel, which was initially constructed with 2% slope towards the backfill, was near vertical 14 months after construction.

From the observed lateral wall pressures, during the construction period, the lateral wall pressure was well below the predicted Rankine values and relatively constant with depth; however during the monitoring period, the lateral wall pressure at the mid-height of the wall gradually increased to values equal to or greater than the predicted values, while the pressures remained very low at the base of the wall.

Key findings:

- The increase of the reinforcement strain was in part caused by geosynthetic creep.
- Lateral wall pressures acting on the wall facing were initially very low during and following the construction period due to the transfer of load to reinforcement. However, the lateral pressures near the mid height of the wall increased considerably with time some time after the construction.
- The stress relaxation of the reinforcement may contribute the increase in lateral earth pressure acting on the wall with time.

Case History: Two reinforced walls (Carrubba et al. 1999, 2000)

Two 4 m high reinforced soil walls were constructed in Italy near the town of Vicenza. One was reinforced using three layers of high density polyethylene (HDPE) geogrids with a tensile strength of 45 kN/m and the other was reinforced using three layers of polypropylene (PP) geogrids with a tensile strength of 20 kN/m. After the completion of the walls, both walls were surcharge up to failure with a 3.5 m thick layer of surcharge backfill in three stages. Subsequently the development of reinforcement strain had been monitored for about two years. Two different failure mechanisms were identified: pullout failure in the HDPE geogrid reinforced wall, and tensile failure of the reinforcement in the PP reinforced wall.

Key findings:

- The reinforcement strain was dependent on both surcharge stress levels and geogrid creep properties.
- The reinforcement creep became evident as surcharge and time increased.
- The creep strain rate linearly increased with the level of total strain.
- The maximum strain rate recorded was 1.25% per year for PP geogrid (from a total strain of 4%), which was higher than that in HDPE geogrid (i.e. 0.2% per year from a total strain of 1.5%).
- Time dependent increase in reinforcement strain was most significant at locations on the failure surface.
- The upper reinforcement experienced more creep than the lower reinforcement.

Discussion

Above cases have shown that the viscous behaviour of geosynthetic reinforcement will significant affect the post construction performance of the reinforced soil walls. The creep and stress-relaxation of the geosynthetic reinforcement may give rise of the transfer load from the geosynthetics to the wall facing, which cause increase in lateral pressure against to the wall. Consequently, it gives rise to the time-dependent deformations of the wall. Therefore, it is important to choose an appropriate long-term strength of the geosynthetic reinforcement. Since the creep strain increases with the stress level (i.e. the higher the tensile load, the more post construction creep reinforcement strain), creep of the reinforcement strain may be significantly reduced at working stress levels.

3.4 Reinforced soil walls constructed over foundations with soft soils

Case History: A Geotextile reinforced soil wall with concrete facing (Nakajima et al. 1996)

A 8 m high reinforced test wall with a concrete block facing and 0.5 m crushed stone surcharge was constructed over a foundation that consisted of 2.5 m granular fill material overlaying 1.6 m thick Kanto-loam and 1.3 m clay layer underlain by sand layers. The wall was reinforced by using 11 layers of 6 m long geotextile. The predicted safety factor was approximately 1 based on the design manual used.

The maximum horizontal wall displacement at about the mid-height of the wall at the end of construction was approximately 65 mm plus approximately 20 mm horizontal displacement that arose from the foundation movement. This increased to approximately 95 mm during a 161 day period after the end of construction. The settlement at the base of the facing was 60 mm at EOC and increased to about 78 mm 160 days after EOC. The wall movement virtually stopped after 160 days

The distribution of strain in each layer suggests that it was maximized in the section close to the reinforcement joints with the facing. The maximum strain value of the reinforcement was about 1% at EOC which was about 9 kN/m when converted into a tensile force. This was much smaller than the design strength of 29.4 kN/m, in construction stage was completed without any safety problems.

The vertical pressure measurements indicated that in the distant area from the facing, the vertical pressure was equal to or smaller than the pressure due to the weight of the fill and that underneath the facing, the value far exceeded the pressure due to the self weight of the concrete blocks.

Key findings:

- The rigid facing with incremental concrete blocks gave rise to the maximum horizontal movement at the mid height of the wall.
- The increase in horizontal wall movements with foundation settlement during post construction indicated the horizontal wall movements were associated to the foundation movements during consolidation under the wall load.
- The maximum reinforcement strain was relatively low and occurred directly behind the wall where the reinforcement was mechanically connected to the concrete facing blocks. Under these stable conditions, the maximum reinforcement strains expected based on a wedge analysis was masked by the high connection reinforcement strains.
- The relatively rigid facing gave rise to the maximum vertical stress at the base of the concrete facing.

Discussion

There have not been many cases published in literature for reinforced retaining walls constructed over soft foundations. Based on the one case cited above it appears that settlement of the foundation soil has an effect on the post construction lateral wall deformations. The consolidation and potential viscous behaviour of relatively soft foundation soils seems to increase the complexity of soil-reinforcement interactions in reinforced soil walls on such foundations. The post construction performance of reinforced walls will be dependent on the time dependent deformations of the foundation soil. Future research is needed to investigate the effect of soft foundations on the behaviour of reinforced soil walls.

3.5 Summary

Twelve case histories of geosynthetic-reinforced soil walls have been reviewed. The performance of these walls, which had varying backfill, reinforcement, facing, instrumentation and construction sequences, has been highlighted and key findings have been summarized.

The deformations of the reinforced walls with granular backfills and mobilized strains in reinforcement under working conditions based on current design methods are relatively low; however, the movement can be significant enough to create the active state of stresses in soil mass. Once the back fill soil reaches the active state, the reinforcement provides most of the tensile resistance to maintain stability of the structure. The failure plane or the locations of maximum reinforcement strains can be reasonably described by the Rankine or Coulomb theory that are used in common design methods. The current design methods are often conservative as indicated by the measured low level of reinforcement force mobilized in field and the failure surcharge pressures consistently greater than those predicted by design methods. The conservatism arises from several factors including the uncertainty regarding soil and reinforcement properties, and not fully accounting for the interactions between the backfill soil, reinforcement, facing, and foundation soils. The full scale experimental data showed that the toe of the wall facing carries a significant portion of the horizontal forces from the backfill acting on the wall facing (Bathurst et al. 2000). This horizontal load capacity contributes one source of conservatism in current design methods.

It has been shown that the surcharge capacity of the reinforced wall is affected by the rigidity of the facing. The distribution of maximum reinforcement force can also be affected by the type of facing system (e.g. a rigid facing appears to result in a high reinforcement force in the reinforcement behind the facing unit). A relatively rigid wall facing reduces the reinforcement strains that would otherwise develop in a wrapped-face wall. The full-height rigid panel facing tends to give rise to high connection strains in the reinforcement at the upper layers; relatively flexible modular facing can accommodate wall deflection and the connection strains are usually much less than the strains along the potential failure plane. A full-height rigid concrete facing may also attract lateral earth pressure and reduce the load transfer of earth pressure to reinforcement. The connection load appears to be the function of the rigidity of the facing system.

The tied-back wedge model worked reasonably well for walls with granular backfill materials. However, the observed magnitude and pattern of deformations of the walls with cohesive backfill can be significantly different from that for granular backfill. The time-dependent deformations of clayey backfill walls tend to be much larger than those of granular backfill walls. A reinforced soil wall constructed with a cohesive backfill can have a higher safety factor at the end of construction (where there are negative pore pressures within the unsaturated backfill soil) than subsequently. The negative pore pressures can decrease significantly with infiltration of water and may become positive if the backfill saturates (e.g. due to heavy rainfalls and poor drainage conditions). The saturation of the clay backfill gives rise to the loss in soil strength and consequently causes the distress of the retaining wall. Thus, high quality compaction and appropriate provision of drainage are essential to prevent the post construction deformations. This is especially true when the backfill clay has swell characteristics.

The post construction deformations of reinforced walls can also be a result of the viscous behaviour of geosynthetics. The field cases have shown that, the greater the mobilized force in reinforcement (especially at locations along the potential failure plane), the greater creep strains after construction and the larger the post construction deformations. The time dependent behaviour of reinforcement has not been found to significantly affect the vertical stresses at the base of the wall. However, the creep and stress-relaxation of reinforcement after construction can increase the lateral pressures acting on a rigid wall face. The polyester geogrid reinforcement is less creep susceptible than high density polyethylene or polypropylene reinforcement; therefore, the stiff polyester reinforcement will be favourable in reducing the time dependent wall deformations.

For walls constructed over soft-firm foundation deposits, wall movements associated with the foundation shear and consolidation can be significant.

4 CONCLUSION

This examination of the case histories also raises a number of issues regarding the reporting of case histories themselves. In many cases it appears that the shear strength of the foundation may have been underestimated due to the reliance on tests (e.g. unconfined undrained) that may not give a good measurement of the available undrained shear strength. Also when estimating the shear strength it is important to use the most likely strength profile when comparing the expected behaviour with and without reinforcement.

There are circumstances where the use of undrained shear strength parameters must really be questioned. For example fibrous peats and sandy silts (with negligible clay) are unlikely to be well characterized by undrained shear strength s_u. Reporting of all the reviewed soils data (including natural water content, liquid limit and plastic limit, and sensitivity) allows the reader to assess the consistency of data.

When reporting field data it is important to recognize that there are situations in very soft soils where inclinometer data may be misleading (very soft soils can potentially flow around the inclinometer unless considerable car is taken with the installation). Also it is important to indicate the method of reinforcement strain measurement and acknowledge that load stiffening due to the use of strain gages can lead to an underestimate the strain actually occurring in the reinforcement.

The field performance of reinforced embankments and soil walls summarized in this paper has provided insights regarding the behaviour of these structures both during construction and following construction. For embankments constructed over soft foundations, the behaviour of embankments is mainly dependent on the time dependent behaviour of both geosynthetic reinforcement and foundation soils. The partial consolidation enhances the function of geosynthetic reinforcement to reduce deformations and increase stability; while the creep of foundations can cause a significant post construction increase in reinforcement strains. The use of prefabricated vertical drains, stage construction, and the control of construction rates can significantly increase the embankment stability due to the partial consolidation of the foundation soil during construction. Following construction of geosynthetic reinforced embankments, the reinforcement strain may increase due to the creep of the geosynthetic itself. However, the magnitude of reinforcement creep in the embankment may be lower than that expected from creep test data.

For reinforced soil walls, it has shown that the field performance depends on the type of reinforcement, facing, backfill and foundation soils. The current design methods can reasonably predict the behaviour of the reinforced walls with granular backfills at the end of construction. However, the long term performance of geosynthetic reinforced soil walls warrants more consideration. Likewise, walls with cohesive backfill and/or built on non-rigid foundation soils require more study.

Based the review of a number of case histories, the following conclusions can be drawn:

- Geosynthetic reinforcement and prefabricated vertical drains (PVDs) can substantial increase the stability of embankments constructed over soft foundations.
- The construction of geosynthetic reinforced embankments is cost-effective compared to conventional construction methods.
- PVDs accelerate the consolidation of the foundation soil and increase the embankment stability due to the significant strength gain associated with consolidation during construction.
- Slow construction or stage construction can significantly increase embankment stability when embankments are constructed over soft foundations.
- Embankments can be safely constructed over peat soils using reinforcement in combination with appropriate construction rates.
- The reinforcement strain observed in the field under working conditions are usually lower than the design values and indeed typically lower than would be predicted assuming limit equilibrium with expected undrained shear strength. This suggests the current design methods are conservative.
- The low level of mobilization of reinforcement can be attributed to the low shear strength adopted for design, partial consolidation of foundation soils during construction and the working stress conditions.
- Even though reinforcement strain may be low at the end of construction, the reinforcement strain can be increase significantly when the foundation soils are rate sensitive and when the reinforcement is creep susceptible.
- The most critical time with respect to stability may occur at some time after construction when embankments are constructed over rate sensitive foundation soils.
- For reinforced soil walls with granular backfills, the observed reinforcement strains are usually lower than the expected values.
- The Rankine or Coulomb theory can reasonably predict the locations of the maximum reinforcement strain with granular backfill, however, they overestimate the magnitude.
- The connection load of the modular wall may be higher than for a full-height panel wall as a result of the effect of the facing rigidity.
- The behaviour of the reinforced wall with cohesive backfills is very sensitive to the

compaction quality and changes in the moisture content due to infiltration of water.
- The creep of reinforcement after construction occurs when creep susceptible geosynthetics are used.

ACKNOWLEDGEMENTS

The work reported in this paper was funded by the Natural Sciences and Engineering Research Council of Canada.

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