Geotextile Applications in Ground Improvement Works

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ABSTRACT: Geotextiles have helped the civil engineering community to solve many age-old problems associated with soft ground and permitted construction in difficult grounds that in past would have been uneconomical or impractical to work on. Reinforcement geotextiles are used to stabilise soft subgrades in the construction of roads and load support platforms. Reinforcement geotextiles are also used as basal reinforcement to improve foundation stability to support embankments. This technique may be used in conjunction with prefabricated vertical drains to accelerate the rate of consolidation to control post-construction settlements. Piling is a common technique used to transfer the embankment load to firmer strata to eliminate both stability and settlement problems. Reinforcement geotextiles are also used between the pile caps to span across the soft ground. The classification of geotextiles, properties and reinforcement mechanisms are discussed. Several case studies on geotextile applications in ground improvement works are presented.

KEYWORDS: Soft ground, Geotextile, Basal reinforcement, Consolidation acceleration, Subgrade stabilisation

1. INTRODUCTION

Geotextiles are a relatively new class of construction materials in civil engineering, engineered and used since the 1950's. Through the decades the geotextile industry has grown by leaps and bounds. Its applications cover a wide spectrum that includes transportation, marine, hydraulic, environmental and geotechnical engineering. This paper is confined to the application of geotextiles to stabilise weak foundations in geotechnical engineering.

2. GEOTEXTILES

The term 'geotextile' was coined from the combination of 'geo' and 'textile'. It came about because the early history of this textile material was concerned with its use in association with soil. Geotextiles are formed from basic yarns connected to form a flexible planar material. The polymer base of the basic yarn is generally polyethylene, polypropylene or polyester; or a combination thereof. Additives are commonly used to enhance certain characteristics of the polymer used e.g. ultraviolet light inhibitors, antioxidants, thermal stabilizers, etc. The yarn properties together with the manufacturing process will generally determine the primary functions and the technical properties of the geotextile.

2.1 Classification by manufacturing process

Geotextiles may be classified according to their manufacturing process. Most of the geotextiles used today are generally either wovens or nonwovens, although a minority of others are used.

2.1.1 Woven geotextile

A woven geotextile generally consist of two sets of yarns that are interlaced and lie at right angle to each other. The yarns that run along the length of the textile are known as warp ends whilst the yarns that run across the length of the textile are weft picks. A selvedge is the longitudinal edge of a fabric that is formed during weaving to form a stable geotextile that does not fray or open up at the edges over time. The weave used to construct the selvedge may be the same, or may be different from the weave used in the body of the textile. Most selvedges are fairly narrow but they can be made wider to achieve certain technical properties, especially for better seaming efficiencies between rolls of geotextiles.

2.1.2 Knitted geotextile

Knitting is a method by which yarns are manipulated to create a textile through entanglement or knotting of yarns. The word is derived from the old English term 'cnyttan' which means 'to knot'. Thus a knitted geotextile is a geotextile that is manufactured according to the knitting production method.

2.1.3 Nonwoven geotextile

The American Society for Testing Materials (ASTM D1117-01) defines a nonwoven as follows: 'A nonwoven is a textile structure produced by the bonding or interlocking of fibres, or both, accomplished by mechanical, chemical, thermal or solvent means and combinations thereof. The term does not include paper or fabrics that are woven, knitted or tufted.' Thus a nonwoven geotextile is a geotextile that is manufactured according to the nonwoven production method.

2.1.4 Composite geotextile

A composite geotextile is a geotextile that is manufactured through the employment of more than one production method. This may be done to achieve a combination of functions and properties that may not be derived from any single production method alone.

2.2 Classification by function

Geotextiles are often classified according to the primary functions they provide in an application. It should be pointed out that some geotextiles have more than one primary functions.

2.2.1 Separation geotextile

A separation geotextile is placed in-between two dissimilar geotechnical materials with the primary function of preventing the intermixing of the two materials. The most classical example concerns the construction of road over soft subgrades. Without the use of a separation geotextile the structural aggregates will be punched into the soft subgrade until a stable intermixed support layer is created before a proper road base course can be formed. This results in costly loss of aggregates into the soft subgrade. The use of a separation geotextile can prevent the loss of aggregates into the soft subgrade and help save construction cost.

2.2.2 Filtration geotextile

A filtration geotextile is applied in subsoil drainage applications to allow passage of fluids from a soil while preventing the uncontrolled passage of soil particles. Some examples of the use of filtration geotextiles include their use as filter layers in the construction of subsoil drains and construction of revetments and dykes in marine and hydraulic engineering. Filtration geotextile is also a component in the manufacture of prefabricated vertical drains (PVDs) that are commonly used for deep subsurface drainage for consolidation acceleration of soft clay layers.

2.2.3 Reinforcement geotextile

A reinforcement geotextile is primarily to resist stresses and to a lesser degree to contain deformations in geotechnical structures. As such the engineering properties i.e. tensile modulus, geotextile/soil interaction, etc. are important in determining its effectiveness as a reinforcement geotextile. Examples in ground improvement works include its use to improve bearing capacities of soft subgrades and to improve stability of embankments constructed over weak foundations.

2.2.4 Others

Other functional classifications include protection geotextile and surficial erosion control geotextile which is seldom seen in ground improvement works. Containment geotextile is sometimes used in geotextile contained stone columns. Subgrade stabilisation geotextile is a multi-functional geotextile used for subgrade bearing capacity improvement in roads and load support platforms. They combine the separation, filtration and reinforcement functions to maximise the effectiveness of the geotextile in that application.

2.3 Properties related to geotextiles

The technical properties of geotextiles are often grouped under the following categories; physical, hydraulic, mechanical, interface and durability properties. A selection of these properties is presented by manufacturers in datasheets to reflect the engineering capabilities for design and applications, and allow engineers to select the right product.

2.3.1 Physical properties

Typical physical properties include unit mass, thickness, width and length. Properties like unit mass and thickness generally are not used for design purposes but allow a quick gross check at site to determine if a delivered product is possibly of the wrong grade. Width and length of a geotextile roll allows the installer to plan for his installation works. Roll width and weight allows the transporter and workers to plan out the best handling methodology.

2.3.2 Hydraulic properties

The most common hydraulic properties listed in datasheets include the opening size and the flow capacity normal to the plane of geotextile. They are related to design criteria for both separation and filtration function checks. The opening size of a geotextile is generally characterised as O_{90} or O_{95} and may sometimes be referred to as the apparent opening size (AOS) or the equivalent opening size (EOS). The test standards most commonly adopted for testing the opening size of a geotextile are ASTM D4751 and ISO 12956. The flow capacity normal to the plane of the geotextile may be measured in terms of flow rate, Darcy's permeability or permittivity. The test standards most commonly adopted for testing the flow capacity normal to the plane of the geotextile are ASTM D4491 and ISO 11058.

2.3.3 Mechanical properties

The mechanical properties typically listed in datasheets can be categorised as either index properties or performance properties. The most universal mechanical index property is the geotextile CBR puncture resistance which is used to index the robustness of the geotextile against installation damage. The test standard for the geotextile CBR puncture resistance is ISO 12236. In America it is designated as ASTM D6241.

For reinforced soil designs distinction is made between the structure service life span and the reinforcement design strength service duration. You may have a long term structure but the reinforcement functional duration may be long term or short term. For example, road pavements may be required to last 20 to 50 years, but the geotextile used to reinforce the base course is only put in tension for a short period of time when the vehicle wheel load passes over a particular area. Although the loading is repeatedly applied when new vehicle wheel load passes through, the induced tension happens over seconds before it relaxes again. Such is the example of a long term engineering structure where the reinforcement application is effected over short term reinforcement functional durations.

Short term tensile strength and stiffness properties of the reinforcement geotextiles govern the design of reinforced soil applications with short term functional durations. The test standards most commonly adopted for short term tensile strength and stiffness of a geotextile are ASTM D4595 and ISO 10319. Figure 1 shows the typical short term tensile strength versus elongation profile for woven and nonwoven geotextiles. Woven geotextiles have yarns generally straight in the machine and cross directions resulting in high tensile modulus in both directions and therefore are well suited as reinforcement geotextiles.

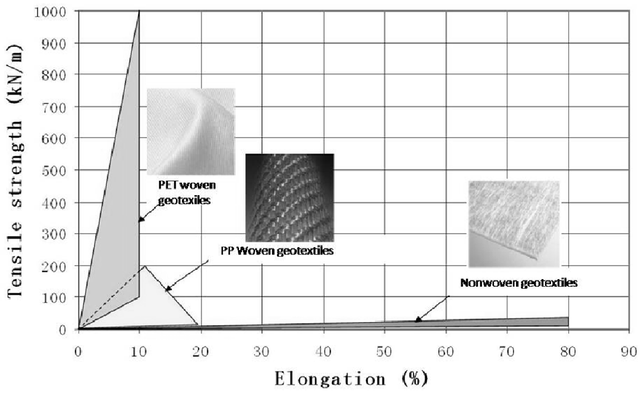


Figure 1 Typical tensile strength versus elongation for woven and nonwoven geotextiles

For the design of reinforced soil applications with long term reinforcement functional durations, time dependent tensile strength and elongation behavior or creep is also important. These performance properties for long term reinforcement design will be discussed in more detail in Section 3.3 of this paper.

2.3.4 Interface properties

The interface properties generally refer to frictional properties between the geotextile and another sheet material (including geotextile, geomembrane, geogrid, etc.) or between the geotextile and soil. For reinforcement geotextiles interface properties are particularly important for design. These properties are discussed in more detail in Section 3.4 of this paper.

2.3.5 Durability properties

Geotextile applications have a successful track record of over 50 years worldwide. Except for a relatively short period of exposure during installation, they remain in a buried or covered condition over their service lives. During the initial years durability of geotextiles under buried or covered conditions was a concern and a major topic of discussion. Today, with the proven track record of over 50 years the concern has mostly disappeared and the topic of discussion is mostly focussed on special aggressive ground conditions and adoption of relevant tests for specification works.

However, resistance against UV degradation during exposure remains a concern. There are two principal types of accelerated weathering equipment i.e. Xenon arc equipment and fluorescent UV equipment. The test standards most commonly adopted for testing UV degradation of geotextiles are ASTM D4355 which uses the

Xenon arc equipment and EN 12224 which uses the fluorescent UV equipment. The ASTM D4355 test method using simulated UV radiation with irradiance of 0.35 W/m^2 at 340 nm has a test cycle of 500 hours and translates to 52 MJ/m^2 of UV radiant exposure. The EN 12224 test method using UVA-340 lamp simulated UV radiation has a test cycle of 430 hours and translates to 50 MJ/m^2 of UV radiant exposure. The results are presented as a residual tensile strength in percentage terms at the end of the test cycle. A residual tensile strength percentage of 70% is typically specified, which equates to an equivalent outdoor natural weathering of about 2 to 6 months, depending on various factors that determine the amount of radiation that reaches the Earth's surface.

3. GEOTEXTILE REINFORCMENT MECHANISMS AND LOAD TRANSFER

3.1 Lateral restraint reinforcement mechanism

When load is applied on the ground, deformation will occur. This is a result of soil movement to mobilise shear resistance to support the load applied. On soft ground the sideway movement of soil can be significant. The loading from an embankment has a vertical as well as a horizontal component. The lateral earth pressure of the embankment fill exerts an outward shear stress on the foundation, which will contribute to the lowering of the bearing capacity of the foundation (Jewell, 1996).

A summary of reinforcement mechanics of an embankment on soft ground is shown in Figure 2. By placing a reinforcement layer between the soft ground and the embankment fill, bearing capacity can be improved in two ways. Firstly, the reinforcement may resist the outward shear stress caused by the embankment fill lateral pressure. Secondly, the reinforcement may reverse the interface shear stress to act inwards, thereby further increasing the bearing capacity of the foundation.

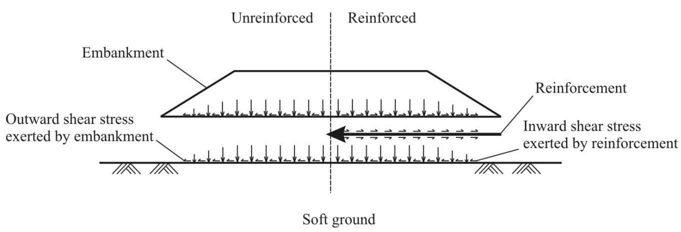


Figure 2 Lateral restraint reinforcement mechanism

3.2 Tensioned membrane reinforcement mechanism

When the foundation deformation response is non-uniform a different mechanism will develop. A summary of reinforcement mechanics of a differentially deforming foundation subject to vertical loading is shown in Figure 3. By placing a reinforcement layer spanning the differentially deforming foundation, the reinforcement will act as a tensioned membrane to support load.

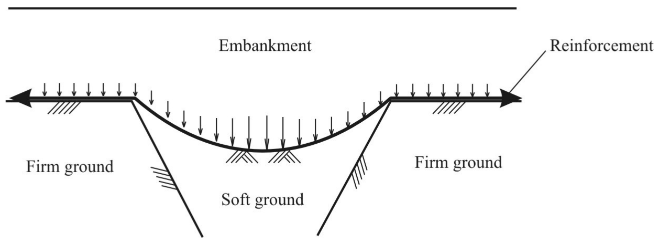


Figure 3 Tensioned membrane reinforcement mechanism

Examples of such applications include geotextiles spanning pile caps and voids or subsidence prone ground. When a sludge pond needs to be capped over a geotextile layer is usually placed over the sludge before placing capping fill material. The initial access is done through the advancing of finger berms spaced at specific distances apart. The ground underneath the finger berms will settle while the ground in-between the finger berms will heave. Tensioned membrane effect is brought into action both underneath the finger berms as well as the restrain in-between the finger berms.

3.3 Time dependent geotextile load-strain characteristics

Fundamental to evaluating the performance of reinforced soil foundations, the geotextile is required to carry tensile load, at defined strains, over the design life (Lawson, 1995). The methodology used to assess the tensile load capability over time for geotextile reinforcements is shown in Figure 3.

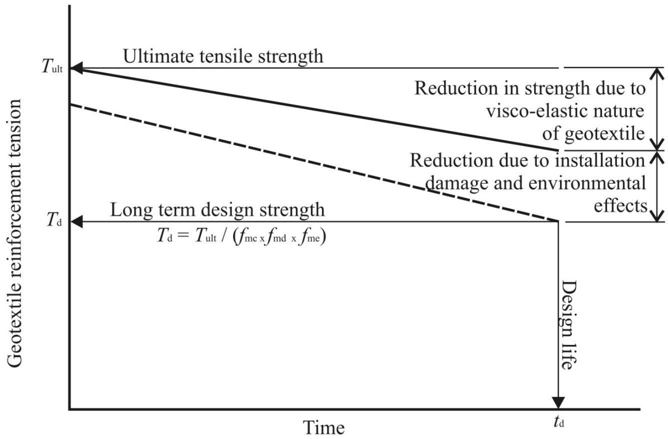


Figure 3 Methodology used to assess tensile load capability over time for geotextile reinforcements (Yee, 2005)

Two fundamental characteristics act to reduce the load carrying capability over time. These are a reduction in strength due to visco-elastic nature of polymeric geotextiles and a reduction in strength due to installation damage and environmental effects. The magnitudes of these reductions depend on the type of geotextile used, the environment in which it is installed, and the time over which the geotextile is required to carry the tensile load. Relevant partial factors of safety are applied to account for creep, installation damage and environmental effects to derive at the allowable design strength for the geotextile, given in Eq. (1) as follows:

$$T_{\mathrm{d}} = \frac{T_{\mathrm{ult}}}{f_{\mathrm{mc}} f_{\mathrm{md}} f_{\mathrm{me}}} \qquad (1)$$

where, T_d, is the allowable design strength of the reinforcement geotextile at the specified design life; T_{ult}, is the short term ultimate tensile strength of the reinforcement geotextile; f_{mc}, is the partial factor relating to creep rupture over the required design life of the reinforcement geotextile; f_{md}, is the partial factor relating to installation damage of the reinforcement geotextile; and f_{me}, is the partial factor relating to environmental effects on the reinforcement geotextile.

The methodology used to assess the tensile load-strain behaviour over time for geotextile reinforcements is shown in Figure 4. Polymeric geotextiles undergo creep or increase in strain under constant load over time due to their visco-elastic nature. This change in strain over different time periods is normally presented in terms of isochronous creep curves. These curves enable the determination of reinforcement strain over any design life and can be divided into an initial (elastic) strain component and a creep (visco-elastic) strain component.

3.4 Geotextile/soil bond

Reinforced soil is a composite material. To be able to behave as a composite material, the reinforcement must bond with the adjacent soil. Bond can be developed either through friction and/or adhesion between geotextile and soil. The shear resistance developed through interaction between soil and geotextile can be assessed by performing direct shear and pullout tests under a range of overburden pressures. In the analysis of the reinforced soil structure, when the assigned slip plane intersects a tensile element, the tensile resistance that can be mobilized is the lower of the rupture strength of the reinforcement and the pullout resistance of the reinforcement in soil. The pullout resistance of the reinforcement from soil is given by Eq. (2) below:

$$T_{po} = \frac{2\alpha_{po} \tan \phi_s \, \gamma_s \, z \, L_{po}}{f_{po}} \qquad (2)$$

where, T_{po}, is the pullout resistance of the reinforcement from soil; α_{po}, is the coefficient of pullout resistance of the reinforcement from soil; ϕ_s, is the angle of internal friction of soil; γ_s, is the unit weight of soil; z, is the soil overburden height above the reinforcement layer; L_{po}, is the embedment length of the reinforcement resisting pullout from soil; and f_{po}, is the partial factor relating to pullout resistance of the reinforcement from soil. Sometimes the most critical failure mechanism may involve soil/geotextile interface sliding. The resistance along the interface of soil and reinforcement is given by Eq. (3) below:

$$\sigma_{sg} = \frac{\alpha_{sg} \tan \phi_s \, \gamma_s \, z \, L_{sg}}{f_{sg}} \qquad (3)$$

where, σ_{sg}, is the sliding resistance along the interface of reinforcement and soil; α_{sg}, is the coefficient of soil/geotextile interface sliding resistance; γ_s, is the unit weight of soil; z, is the height of overburden soil above the reinforcement layer; L_{sg}, is length of the sliding surface along the interface of soil and reinforcement; and f_{sg}, is the partial factor relating to soil/geotextile interface sliding resistance. Table 1 shows the soil interaction coefficients of geotextile reinforcement products recommended for design by Koutsourais et al (1998).

Table 1 Test results and recommended design soil interaction coefficients (adapted from Koutsourais et al, 1998)

Condition	Tested α_{sg}	Tested α_{po}	Recommended for α_{sg} and α_{po}
Woven PET geotextile/sand	1.0		0.9
Woven PET geotextile/clay	0.71-0.93	0.82-0.91	0.7
Woven PP geotextile/sand	0.9		0.9
Woven PP geotextile/clay	0.58-0.64	0.66-0.71	0.6

* PET = polyester, PP = polypropylene

3.5 Geotextile joints and load transfer efficiency

The decision to use an overlap or sewn seam is based on a few factors. The first factor is the weakness of the ground upon which the geotextile is placed with respect to the potential for mud-waving during the backfill operation. The second factor is the material and labour costs of deploying individual geotextile panels with extra material required for the overlap versus the material and labour costs of sewing and installation. Lastly, it also depends on the feasibility of deploying individual geotextile panels in poor access and/or climatic conditions. Typically sewing tends to be a more economical option over overlapping when ground CBR is 1 or weaker. Sewing may be mandatory when ground CBR is less than 0.5.

The types of commonly used on-site sewn seams are the prayer seam, "J" seam and butterfly seam (see Figure 5). The prayer seam is the easiest to make and is commonly used for required seam strengths of 40 kN/m and below. The "J" and butterfly seams are more difficult to make and are commonly used to develop higher seam strengths.

Two types of stitches are used. The single thread chain stitch (type 101) is simpler but the stitch runs the risk of unravelling. For required seam strengths of more than 25 kN/m or when seaming heavier, higher strength geotextiles, the double thread chain stitch (type 401), which does not unravel easily, is generally used. Sewing thread is commonly available in Kevlar, nylon, polyester and polypropylene. Typically, polyester sewing thread is used for seaming higher strength geotextiles with cross-roll direction strengths of 50 kN/m or more. Table 2 provides guidance for developing seam strengths.

Table 2 Guidance for developing seam strengths (TenCate Geosynthetics, 2013)

Required CD seam strength (kN/m)	Geotextile CD tensile strength (kN/m)	Seam Type P/J/BF	Stitch Lines (single/double)
18	35	P/J	single
35	53 – 70	J/BF	double
53	70 – 105	J/BF	double
70	105 – 140	J/BF	double
88	175 – 220	J/BF	double
105	210 – 263	J/BF	double
123	245 – 306	J/BF	double

where CD = cross direction, P = prayer seam, J = "J" seam, BF = butterfly seam

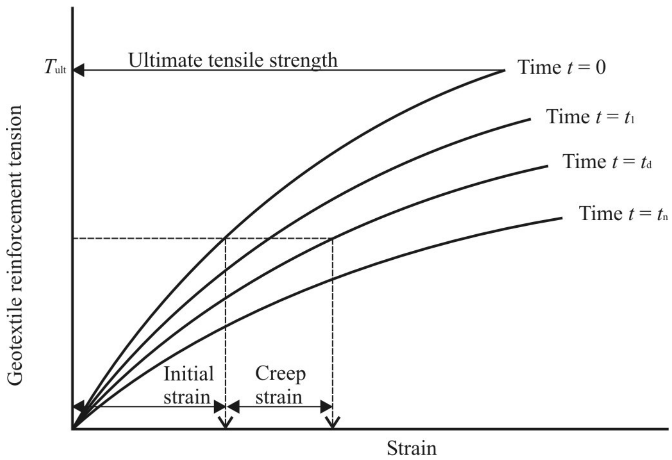


Figure 4 Methodology used to assess tensile strain capability over time for geotextile reinforcements

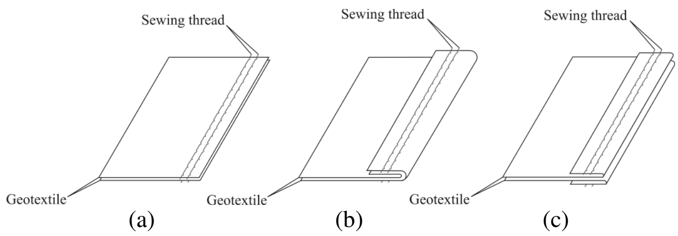


Figure 5 Types of commonly used on-site sewn seams (a) prayer seam (b) "J" seam (c) butterfly seam

4. SOFT GROUND ISSUES AND IMPROVEMENT WORKS

The focus of this paper is on saturated soft clay and silt layers typically formed during recent geological times by alluvial deposition in a lacustrine, riverine or marine environment. They normally occur around coastlines, floodplains and old inland lakes.

The soil is either normally consolidated or slightly overconsolidated with bulk densities from 14 to 16 kN/m^3 and undrained shear strengths from 10 to 30 kN/m^2. The ground water level is typically close to ground level and may even be tidal influenced. Sometimes a top layer of recent fill or a desiccated crust may exist on top of the soft ground formation. The geotechnical problems associated with such grounds include having low subgrade bearing capacities for roads and load support platforms; inability to support high embankments and the experiencing of large and delayed settlements under the influence of loading imposed by the embankment fill.

4.1 Road subgrade improvement

Traffic and other loadings require a pavement or load supporting platform structure for one or more of the following reasons: for load distribution to reduce load intensity on the subgrade, to provide an all weather traffic support surface, and to provide riding comfort. Research works have shown the following:

- The separation function of a subgrade stabilisation geotextile prevents the loss of aggregates into the soft subgrade (see Figure 6) and an effective subgrade stabilisation geotextile requires high permeability to allow rapid dissipation of excess pore pressures built up during transient wheel load passes.
- Due to rut depth limitations in design only the lateral restraint reinforcement mechanism is realised (see Figure 7a) while the tensioned membrane reinforcement mechanism is not (see Figure 7b); and the initial tensile stiffness modulus (typically at 2% elongations) of subgrade stabilisation geotextile is the key performance property for the reinforcement effectiveness.
- Good interaction property between the subgrade stabilisation geotextile and the aggregates at the interface is essential for effective lateral restraint of the aggregates.

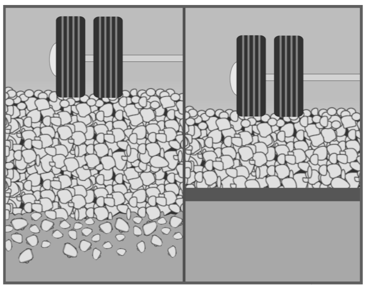
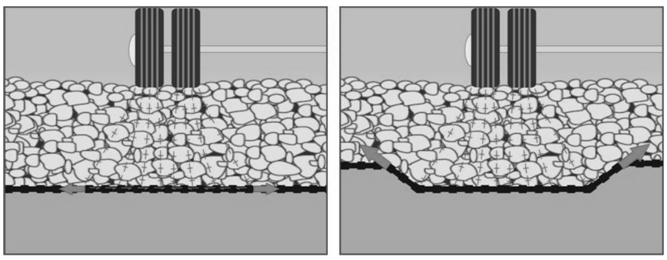


Figure 6 Illustration of separation function



(a) (b)

Figure 7 Base course reinforcement mechanisms (a) lateral restraint (b) tensioned membrane

Roads are designed according to serviceability limiting criteria in the form of acceptable rut depths under the influence of repeated wheel loads passing by. Unpaved roads can generally tolerate relatively large rut depths. The limiting rut depth adopted for unpaved road design is typically between 50 to 100 mm. The Giroud-Han (G-H) design method provides a design tool to determine the thicknesses of unreinforced and geotextile reinforced aggregate bases for unpaved roads over soft subgrade.

Asphalt paved roads or flexible pavements on the other hand tolerate relatively smaller rut depths. This is because the asphalt surfacing layer will be subject to cracking if the rut depths are large. Flexible pavements are generally designed with limiting rut depth of 15 to 20 mm and can be accomplished using different design methods. The American Association of State Highway and Transportation Officials (AASHTO) design method is the most widely used and accepted design method in the United States. The focus of the analysis is to determine the required structural number (SN) value for the project to support the anticipated level of traffic. The AASHTO design method can account for the contribution of the reinforcement geotextile inclusion to the SN value of the pavement.

4.2 Load support platforms and sludge pond cappings

Load support platforms are designed according to stability limiting criteria in the form of localised bearing capacity and global stability failures. The load usually comes from construction equipment like excavators, tractors, piling equipment, PVD installation equipment, lifting cranes, etc. Sludge pond capping is an extreme form of load support platform whereby the subgrade is so soft that fill material will sink in on its self weight. Establishing an initial support platform to allow vehicles and machinery for normal earthworks to proceed can be challenging and requires a combination involving the use of reinforcement geotextile and special earthwork construction methodology (see Figure 8). The reinforcement geotextile is usually placed as one large prefabricated piece and anchored at the pond edges using stabilisation berms. Finger fills are then advanced in a controlled manner from one bank to the opposite bank before these finger berms are widened systematically to cover the whole surface area. It should be pointed out that this technique only helps to provide a stabilized load support platform and further foundation improvement works is necessary before the ground can support structures.

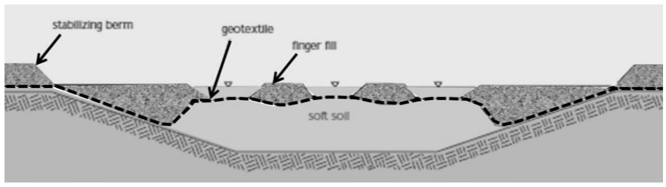


Figure 8 Pond capping with reinforcement geotextile

4.3 Basal reinforced embankments with PVDs and surcharge

Reinforcement geotextile can be used at the base of the embankment to provide adequate stability in soft ground foundations. Design for reinforcement geotextile is normally done using a slip circle analysis design program using the limit equilibrium method. The program can be used to determine the tensile strength of the geotextile reinforcement to achieve a target factor of safety against failure. The reinforcement geotextile is laid with the main strength direction as a continuous layer across the embankment to resist the potential slip failure. However, the application of basal reinforcement in general does not improve settlement. Typically, road pavements and railway tracks may be constructed above embankments. The settlement design criterion is usually based on a certain tolerable differential settlement for the structure supported by the embankment; the more sensitive the structure the smaller the tolerable differential settlement should be. Differential settlement will be minimal if the embankment post construction settlement is kept to a minimal. With soft clay foundations, consolidation settlement will continue for decades after the completion of the embankment construction. This is because the trapped excess pore pressures in the soft clays take a long time to dissipate due to low permeability of the clay and a long dissipation path involved.

Obviously this post construction settlement will contribute to large differential settlements taking place along the road pavement or railway track constructed on top of the embankment over its design life. When PVDs are installed, the dissipation path changes from vertical to horizontal.

Typically, PVD installed spacing range between 1 to 2 m apart. This results in a significantly reduced dissipation path. In addition, horizontal permeability in soft clays can be a few times higher than vertical permeability, often due to presence of sand layers or lenses. Surcharge is often used in combination with PVD installation to achieve near 100% consolidation settlement within a short period of 3 to 12 months, depending on design requirements.

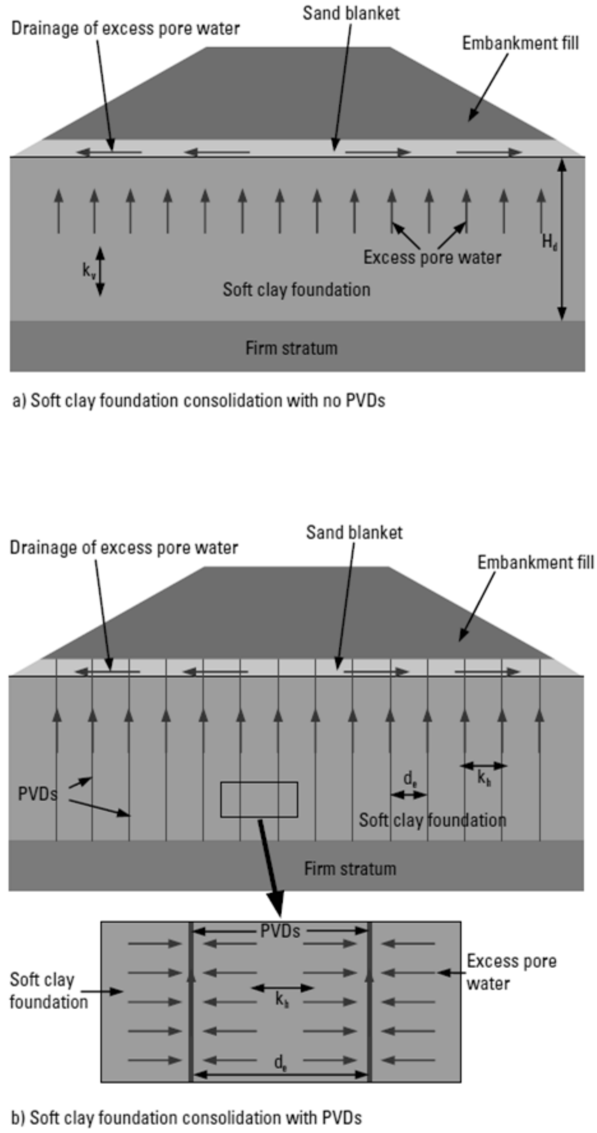


Figure 9 Drainage path for dissipation of excess pore water pressures for soft clay foundations under influence of embankment construction with and without PVD installation

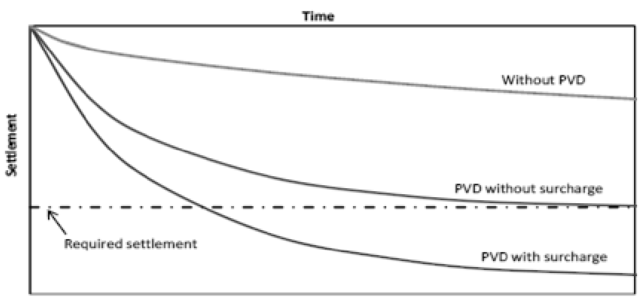


Figure 10 Acceleration of foundation settlement for embankment construction with PVD and surcharge (Montulet et al, 2013)

4.4 Geotextile reinforced piled embankments

Geotextile reinforced piled embankments are increasingly used to overcome combined stability and settlement problems. Different types of piles and structural columns have been used for this technique including concrete piles, timber piles, stone columns, cement stabilised columns, etc. The advantage of using this technique is that embankments can be constructed to any height, at any rate, with subsequent settlement issues. A significant portion of the embankment self weight is transferred directly onto the pile caps through soil arching. The loading from the unarched soil that is located outside the arching zone is generally small and a reinforcement geotextile is provided to carry this load through catenary action onto the pile caps (see Figure 11). The combined load is then transferred via the piles down to firm layers. In this way, soft ground layers are not stressed and the resulting embankment settlements are small.

For foundation piles with individual pile caps, reinforcement geotextile spanning is required in two directions. One layer of reinforcement geotextile is laid with the main strength direction as a continuous layer across the embankment. Another layer of reinforcement geotextile is laid with the main strength direction as a continuous layer lengthwise of the embankment. The reinforcement geotextile laid across the embankment needs to be stronger because this layer also need to counter the lateral force exerted by the embankment fill. The British design code BS 8006-1:2010 may be used to determine the required tensile strength of the reinforcement geotextiles.

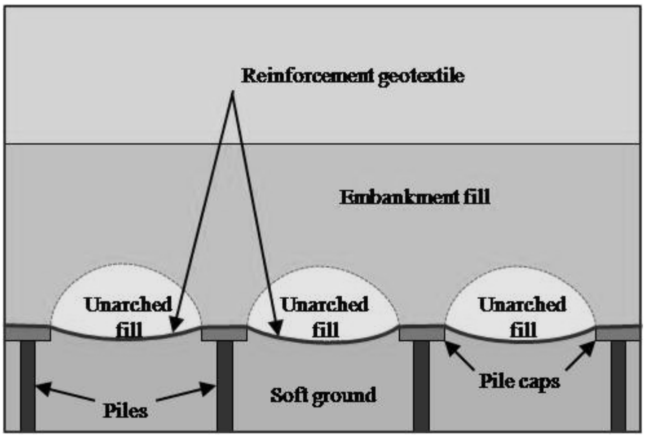


Figure 11 Load support mechanism with reinforcement geotextile in piled embankments

5. CASE STUDIES

5.1 Container Yard Subgrade Stabilisation, Port Klang, Malaysia

Pulau Indah is home to Westport, Malaysia. The general subsoil in this area is made up of very soft marine clay to depths of more than 30m, the water table is almost at ground level and the subgrade CBR of the top desiccated layer in general is about 3%. Due to the increase in container handling and storage demand, Infinity Logistics Sdn Bhd constructed additional load supporting platform areas. The original design involved a base course thickness of 1 m. However, when the heavy-reach stacker started operations on the platform severe rutting occurred despite the use of such a thick base course layer. Soft clay was also seen to have pumped up to the surface of the platform.

A reassessment of the situation led to the decision to use a subgrade stabilisation geotextile that has a combination of separation, filtration and reinforcement functions to overcome the problem. The Giroud-Han method was used to design the alternative solution. Figure 12 shows the printout of the spreadsheet software used for the design. It was found that with the use of the subgrade stabilisation geotextile the base course thickness could be reduced to 0.5 m. Figure 13 shows the cross section of the reinforced aggregate platform.

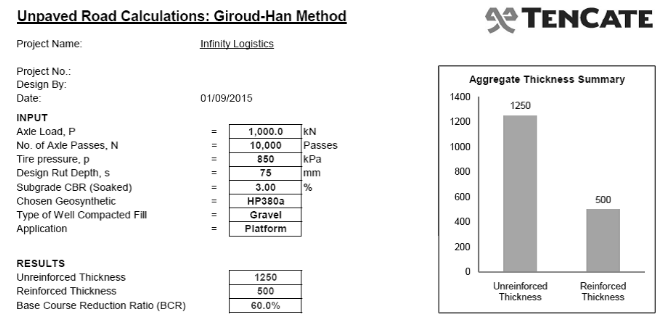


Figure 12 Design comparison between unreinforced and reinforced base course using Giroud-Han design method

Figure 13 Cross section showing use of Mirafi® HP380a to reinforce the base course aggregates

This resulted in overall cost savings despite having to pay for the subgrade stabilisation geotextile. The ground surface was stripped of vegetation and topsoil before the subgrade stabilisation geotextile was laid. The subgrade stabilisation geotextile were sewn up using a simple prayer seam at site. Aggregates of aggressive sizes of up to 150 mm in diameter was laid on top of the subgrade stabilisation geotextile and compacted. Figure 14 shows the construction of the reinforced aggregate platform. To date the container yard aggregate platform has performed to the client's satisfaction.

Figure 14 Placement of aggregates above Mirafi® 380a subgrade stabilisation geotextile

5.2 Sludge Pond Capping Project, Harbin, China

The Wenchang Wastewater Treatment Plant (WTP) in Harbin City generates sludge as a waste product of the wastewater treatment process. Over the years, the plant operator has been storing the waste sludge in a 70,000 m^2 trapezoidal shaped sludge pond. This sludge pond has reached its design capacity. Massive desludging, dewatering and disposal works would be needed in order to extend the lifespan of the pond. Alternatively, the plant owners decided to reclaim the sludge pond for further land development instead.

Geotechnical properties of the pond sludge at Wenchang WTP is not known or classified. The pond sludge is so soft that any solid object unexpectedly landing on the pond surface would generally sink in completely. The undrained shear strength of the pond sludge is estimated to be 1 to 2 kN/m^2 at best. Figure 15 shows the force diagram for the determination of the tension in the reinforcement geotextile. Conservatively, it was assumed that the geotextile would have to support up to 6 m of fill depth for the initial finger berm, which would be an upper-bound solution. The finger berm was assumed to have a berm width of 6 m. The density of the fill was estimated at 20 kN/m^3. This initial finger berm would have an upper bound buoyant weight, W_b, of 360 kN/m. In addition, a construction surcharge load, w_s, of 10 kN/m^2 over a width of 3 m was assumed in design. The total vertical load is therefore 390 kN/m and the geotextile tension, T, is half of that or 195 kN/m.

The allowable design strength of the reinforcement geotextile, T_d, must be equal or greater than the geotextile tension, T. The values of 1.45, 1.2 and 1.1 were adopted for f_{mc}, f_{md} and f_{me} respectively. From Equation 1, the short term ultimate tensile strength of the reinforcement geotextile, T_{ult}, must be equal or greater than 373 kN/m. Thus the reinforcement geotextile with tensile strength of 400 kN/m short term ultimate tensile strength was specified for this project.

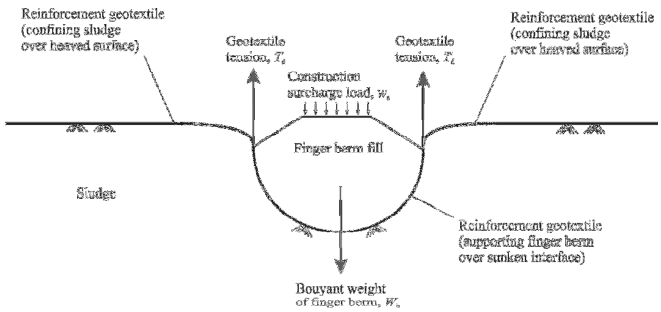


Figure 15 Force diagram for the determination of the tension in the geotextile (Yee et al, 2012)

No sewing was allowed in the warp direction (the principal direction) of the geotextile. The finger berm would induce tension in direction perpendicular to the axis of the finger berm. However, at the front end of the advancing finger berm, the geotextile would also be stressed in the direction longitudinal of the finger berm. As such two layers of the geotextile reinforcement would be installed, each to be laid perpendicular to one another. It was realized that this was a very conservative approach, but such conservatism was deemed necessary due to the lack of reliable design input information as well as a lack of precedence in design. Subsequent advancement of finger berms would be less critical because the confined sludge would have been pressurized by the imposition of the initial finger berm and this pressure counteracts further imposition of vertical loads.

Conventionally, pond capping works involve deployment of a prefabricated panel of geotextile that is large enough to cover the entire pond area. At Wenchang WTP this deployment methodology was not practical due to site constraints. Such a large prefabricated panel of geotextile would require a large fabrication work platform adjacent to the pond. Heavy machinery would also be needed to deploy the prefabricated panel of geotextile. There was a lack of space to enable neither geotextile cover prefabrication nor deployment to be done in the conventional way.

An innovative method for the deployment of geotextile reinforcement over the sludge pond was adopted. This innovative method involved the use of floating platforms placed over the sludge surface. These floating platforms were made of polystyrene foam slabs sandwiched between plywood panels. These floating platforms enabled workers to walk on top of them and conduct the laying work of the reinforcement geotextile as well as sewing adjacent rolls of geotextiles together.

Figures 16a to 16e show the construction sequence of laying the capping geotextiles over the sludge pond. Figure 16a shows the unrolling and laying of reinforcement geotextile. Figure 16b shows the extraction of floating platforms from underneath of reinforcement geotextile. Figure 16c shows the trenching of reinforcement geotextile to provide anchorage on the banks of sludge pond. Figure 16d shows the construction and advancement of the finger berm.

During the advancement of the first finger berm, the fill virtually sank all the way in because of the existence of slack in the reinforcement geotextile initially. The reinforcement geotextile only started picking up tension when the existing slack induced during laying has be taken up as a result of the sink-in of the initial finger berm. This effect is clearly evident in Figure 16d.

In between finger berms, the reinforcement geotextile confines the sludge below, resulting in an uplift pressure that can then support loading above. Figure 16e shows the ability of workers to walk directly on top of the geotextile. Small dump trucks were also able to move on top of a thin subbase layer placed above the geotextile in the areas between the finger berms. Once an initial layer of aggregate has covered the entire area, heavy machinery can move above to conduct the general earth filling and long term foundation treatment works.

(a)

(b)

(c)

(d)

(e)

Figure 16 Wenchang WTP sludge pond capping; (a) unrolling of geotextile reinforcement, (b) extraction of floating platforms from underneath of geotextile reinforcement, (c) trenching at edge of pond, (d) advancement of finger berm, (e) filling between finger berms.

5.3 Ipoh-Padang Besar Double Track Railway, Malaysia

The Electrified Double Track Railway Project covers new track construction from Johor Bahru to the Malaysian-Thai border town of Padang Besar and will eventually form part of the Trans-Asia railway line spanning from Singapore to Kunming in China. The Ipoh to Padang Besar stretch is 330 km long and traverses a wide variety of geological and ground conditions.

Approximately 180 km of the railway alignment passes over alluvial river valleys and low-lying coastal plains where normally or slightly over-consolidated alluvial and marine clay deposits predominate. Commonly, these soft soil deposits are of 20 m or more in depth, exhibit undrained shear strengths ranging from 5 to 20 kN/m^2, and are essentially normally consolidated. Ground improvement works are required to ensure stability and post construction deformations are maintained within specific limits for the railway embankment.

The ground improvement works for the project involve the use of PVDs and preloading of the embankment by surcharging to ensure consolidation of the soft foundations occurs quickly within the construction period. To maintain stability of the surcharged embankment reinforcement geotextiles were used at the base of the embankment to achieve target factors of safety against global slip failures. Figure 17 shows the typical cross-section through the railway embankments. Construction of the basal reinforced embankment started with the stripping of vegetation and topsoil to provide a level surface.

A separator geotextile was placed on the ground level prior to the placement of a 500 mm thick layer of sand (see Figure 18a). This sand layer acts as a drainage blanket to drain out the excess pore water from the base of the embankment and also acts as a working platform to support the PVD installation equipment. The PVDs were installed through the sand layer and separator geotextile into the soft foundation soils on a 1.2 m square grid and to a depth that coincided with the bottom of the soft foundation layer (see Figure 18b). Following the installation of the PVDs a thin layer of sand was then placed over the sand platform prior to placement of the reinforcement geotextile. Woven polyester reinforcement geotextiles varying from 100 to 800 kN/m were used.

After the reinforcement geotextile was installed, general fill was placed and compacted in layers to construct the surcharged embankments (see Figure 18c). Typically, the surcharged embankments were 5 to 6 m in height, of which 2 to 3 m was surcharge. After the preloading period was completed, which was typically 3 to 6 months, the surcharge fill was trimmed to the final embankment height level before the ballast and tracking works were carried out.

The ground improvement works involving the use of basal reinforcement geotextiles and PVDs have helped to improved the soft foundation to support the railway embankment and accelerated the ground consolidation within the construction period so that very little post construction settlement will take place. The railway line is currently in full operation.

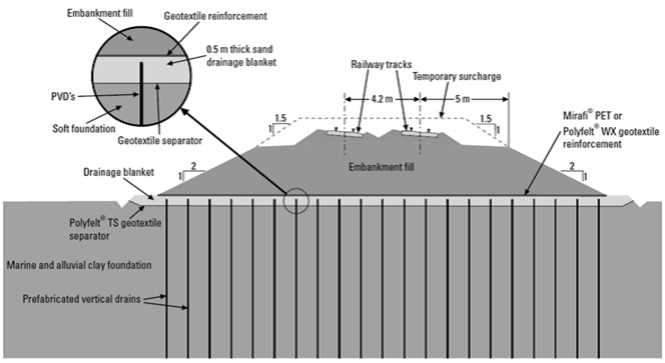


Figure 17 Typical cross-section through the railway embankments for Ipoh to Padang Besar Double Track Railway Project

(a)

(b)

(c)

Figure 18 Construction for Ipoh to Padang Besar Double Track Railway Project (a) installation of geotextile separator and placement of sand layer (b) PVD installation (c) placement of general fill above laid out reinforcement geotextile

5.4 Wat Nakorn-In Bridge Project, Bangkok, Thailand

The Wat Nakorn-In Bridge and Connecting Road Construction Project is a major infrastructure project and part of a larger master plan to ease traffic congestion on the west bank of the Chao Phraya River in the Greater Bangkok area. The main bridge crosses the Chao Phraya River midway between Rama VII Bridge and Nonthaburi Bridge. The project also involved a network of connecting roads that included the construction of other smaller bridges.

The Chao Phraya Plain is geologically a broad deep basin filled with sedimentary soil deposits which form alternate layers of clay and sand with gravel, down to about 1,000 m depth (Bergado et al. 1990). These sedimentary layers were deposited from the Quaternary Period and parts of the Chao Phraya Plain were still covered by shallow marine water from as recent as 5,000 years ago. The Chao Phraya Plain is approximately 300 km in the North-South direction and has a width of about 200 km. The three upper most layers consist of weathered clay, soft clay and stiff clay. The thickness of the layer of soft clay increases from North to South with about 15 m in Bangkok. In addition, the area has been undergoing subsidence as a result of groundwater extraction over time.

The embankments approaching the bridge abutments were designed with pile support to provide stability as well as to prevent large differential settlements between the embankment and bridge structure. The embankments approaching the bridge abutments were designed with steep side slopes due to lack of right of way. Figure 19 shows the longitudinal section of the embankment and the use of reinforcement geotextile with an ultimate tensile strength of 1,000 kN/m to span pile caps.

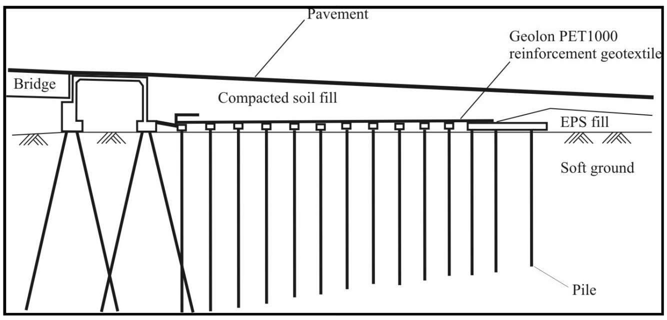
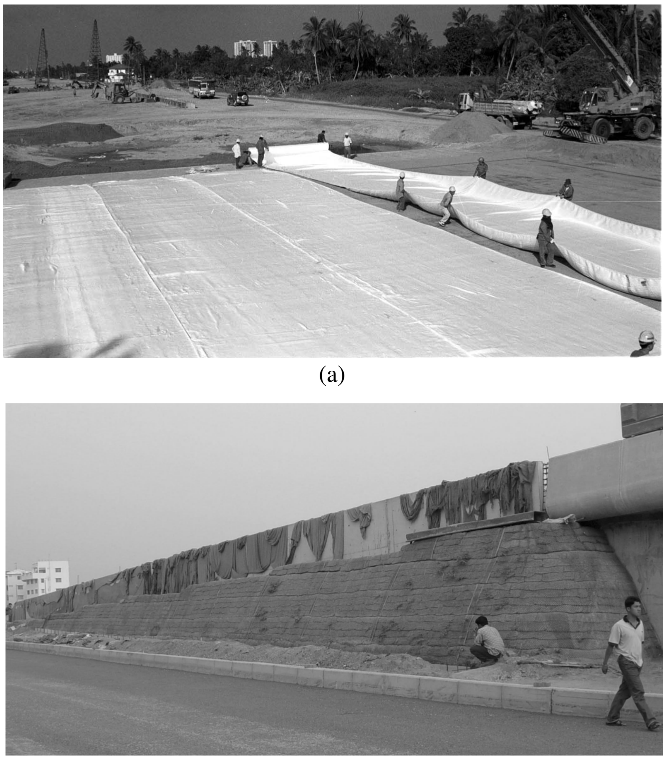


Figure 19 Longitudinal section of bridge approach embankment for Wat Nakorn-In Project (Yee et al, 2006)

Because of the overall size, the project was awarded in five contracts, each involving the construction of bridges and embankments using the ground improvement technique described above. The embankments can accommodate up to ten traffic lanes. Figure 20a shows the laying of the woven polyester reinforcement geotextile during construction for contract EW1. Figure 20b shows the completed reinforced piled embankment adjacent to the bridge abutment. The bridge approach embankments have shown no signs of significant differential settlements with the bridge structure till now.



(a)

(b)

Figure 20 Construction of geotextile reinforced piled embankment for Wat Nakorn-In Project (a) laying of reinforcement geotextile (b) completed embankment

6. CONCLUSION

Geotextiles are typically classified according to their manufacturing process and functionality. The application of reinforcement geotextiles in ground improvement works and the reinforcement mechanisms have been presented. Four Asian case studies have been reported.

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