GROUND IMPROVEMENT WITH GEOTEXTILE REINFORCEMENT: CASE STUDIES – EMBANKMENT OVER SOFT CLAY IN AUSTRALIA AND SLUDGE POND CAPPING IN CHINA

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ABSTRACT

The use of geotextile reinforcement for ground improvement is discussed. Two mechanisms of geotextile reinforcement for ground improvement are presented. The first mechanism is activated at relatively small vertical deformations of the foundation. The second mechanism, commonly referred to as tensioned membrane effect, comes into play when large differential deformations of the foundation occur. Fundamental to design, the geotextile is required to carry tensile load, within a defined strain limit, over the design life of the structure. Partial factors of safety are applied to derive the allowable design strength for the geotextile. The relevant partial factors include those applied to creep, installation damage and environmental effects. The importance of geotextile laying direction is discussed. On-site seaming methods and guidance on achievable seam strengths are presented. Two case studies are described. The first case study at Cape Preston, Australia, involved the use of high strength woven polyester geotextile to reinforce a 7 m high embankment constructed over soft estuarial mud and was subject to intensive induced loading. This case study illustrated the activation of predominantly the first mechanism of geotextile reinforcement. The second case study involved the use of high strength woven polyester geotextile to reinforce a soil capping layer over an extremely soft wastewater sludge pond in Harbin, China. This case study illustrated the activation of the second mechanism of geotextile reinforcement.

Keywords: Ground improvement, woven polyester geotextile, reinforcement mechanism

INTRODUCTION

Soft clay foundations are often formed from fine soil particles transported and deposited by water (Jewell, 1996). Such soils are often layered due to different deposition sequence over long periods of time. Such ground conditions typically are low in shear strength and compressible. When fill is placed over such foundations, collapse of fill into the soft foundation can often be an issue. This collapse may be caused by self weight of the fill and/or induced loading from construction equipment and other overburden loads. Geotextiles may be used as reinforcement to improve foundation stability over soft foundations.

REINFORCEMENT MECHANISMS

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 (Yee, 2005). On soft ground the side-way movement of the foundation 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, 1988; Jewell, 1996). A summary of reinforcement mechanics of an embankment on soft ground is shown in Figure 1 (Jewell, 1996).

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.

When the foundation deformation response is non-uniform a different mechanism will develop. A non-uniform foundation deformation can occur as a result of non-uniform imposed loading and/or non-uniform foundation soil. By placing a reinforcement layer spanning the differentially deforming foundation, the reinforcement will act as a tensioned membrane to support load. A summary of reinforcement mechanics of a differentially deforming foundation subject to uniform vertical loading is shown in Figure 2. Examples of such applications include geotextiles spanning pile caps and voids or subsidence prone ground.

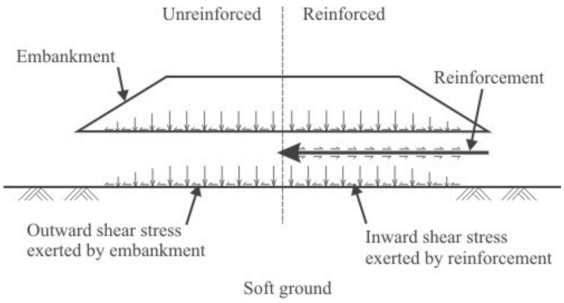


Fig. 1 A summary of reinforcement mechanics of an embankment on soft ground (adapted from Jewell, 1996)

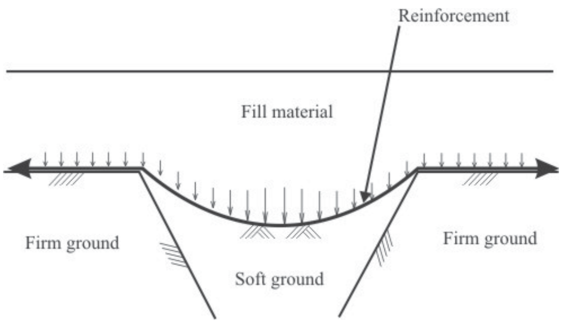


Fig. 2 A summary of reinforcement mechanics of a differentially deforming foundation subject to uniform vertical loading

An example of uniform foundation that undergoes non-uniform foundation deformation resulting from non-uniform imposed loading is the application of high strength geotextile for the reinforcement of veneer soil capping layer over sludge ponds. When a sludge pond needs to be capped over, a high strength 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 in-between the finger berms.

Figure 3 shows the tensioned membrane effect experienced by a capping reinforcement geotextile due to differential deformation response of the pond sludge resulting from the load imposed by the finger berm above.

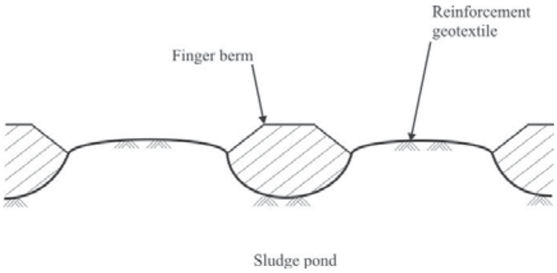


Fig. 3 Tensioned membrane effect due to differential deformation resulting from the load imposed by the finger berm above

GEOTEXTILE REINFORCEMENT

The term ‘geotextile’ is derived from ‘geo’ and ‘textile’ and may be simply defined as textile material used in a soil (geo) environment. The commonly used geotextiles today are either woven, nonwoven or knitted geotexiles. Woven geotextiles are manufactured through the weaving of tapes, fibres or yarns. Nonwoven geotextiles are manufactured by random placement of continuous or short fibres, which are then bonded by either a heat treatment or a needle-punching process. Knitted geotextiles as the name implies are formed by a knitting process that connects cross yarns to form a fabric.

The properties exhibited by the geotextile depend on the manufacturing process, polymer type, filament form, etc. Woven and knitted geotextiles generally exhibit relatively much higher tensile stiffness due to the alignment of filaments in the roll and cross-roll directions and are suitable for reinforcement applications. Nonwoven geotextiles on the other hand exhibit lower tensile strength and higher elongation due to the random placement of fibres and do not have the ideal properties for reinforcement applications.

Long Term Design Strength

When reinforced soil is stressed, deformations will occur. Deformation is necessary to mobilize shear strength in soil. Deformation is also required to mobilize tensile resistance of the reinforcing material. The contribution of geotextile as reinforcement may be either viewed as stress absorbing or strain alleviating.

Fundamental to evaluating the performance of reinforced soil foundations, the geotextile is required to carry tensile load, at defined strains, over the design life. The methodology used to assess the long term design strength for geotextile reinforcement is shown in Figure 4. Two fundamental characteristics act to reduce the load carrying capability over time. These are a reduction in strength due to visco-elastic (creep) nature of polymeric geotextiles and a reduction in strength due to installation damage and environmental effects.

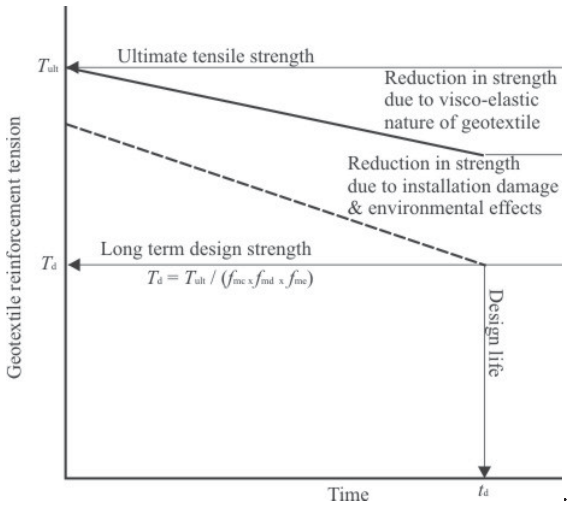


Fig. 4 Methodology used to derive the long term design strength of geotextile reinforcement

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 rupture, installation damage and environmental effects to derive at the long term design strength for the geotextile, given in Equation (1) as follows:

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

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

Polymeric geotextiles undergo differing amounts of strain over time due to their visco-elastic (creep) nature. This change in strain over different time periods is normally presented in terms of isochronous creep curves (see Figure 5). 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. Often, a limiting creep strain is defined for design.

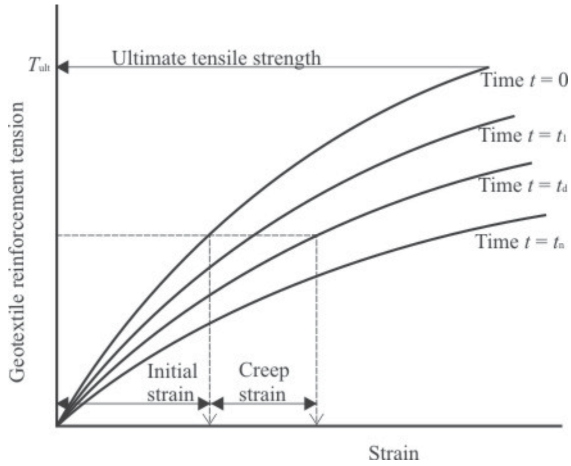


Fig. 5 Typical isochronous creep curves for geotextile reinforcements

Reinforcement/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 through friction 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:

$$T_{1}po = (2(_{1}po \tan [\![(_{1}s]\!] \ \gamma_{1}s \, zL_{1}po)/f_{1}po \qquad (2)$$

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:

$$\sigma_{1}sg = ((_{1}sg \tan [\![(_{1}s]\!] \ \gamma_{1}s \, zL_{1}sg)/f_{1}sg \qquad (3)$$

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

Laying Direction

Geotextiles used for reinforcement applications generally are manufactured with the principal strength direction along the roll direction. This principal strength direction must be laid to coincide with the direction of principal stress. If the application also has a second principal stress direction (usually perpendicular to each other) either the cross-roll edge seam strength should be strong enough to resist the mobilized stress or a separate continuous layer is laid.

Seaming

The decision to use an overlap or seam depends on the practicality of using an overlap and the comparative costs between using and overlap and seaming. When the ground is soft, large ground movement would require large overlaps. Typically seaming tends to be a more economical option over overlapping when ground CBR is 1 or weaker. Seaming may be mandatory when ground CBR is less than 0.5.

The types of commonly used on-site seams are the “prayer seam”, “J” seam and “butterfly seam”. 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 result in more overlap wastages but 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 unraveling should the thread be cut accidentally. For required seam strengths of more than 25 kN/m or when seaming heavier and higher strength geotextiles, the double thread, double chain stitch (type 401) which does not unravel easily, is generally used.

Thread is commonly available in Kevlar, nylon, polyester and polypropylene. Typically, polyester 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 when seaming adjacent panels of geotextiles together.

Table 2 Guidance for developing seam strengths (TC Mirafi, 2001)

Required seam strength (kN/m)	Suggested geotextile cross-roll tensile strength (kN/m)	Seam type P/J/BF*	Stitch type (single /double)
18	27	P/J	single
35	53 – 70	P/J	single
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

* P = prayer seam, J = “J” seam, BF = butterfly seam

REINFORCED EMBANKMENT, SINO IRON PROJECT, AUSTRALIA

Background

The Sino Iron Project is a world class, large-scale magnetite iron ore project located in Western Australia’s Pilbara region, 100 km southwest of Karratha (Loh and Suhendra, 2011; Yee and Loh, 2012). This project is the largest planned magnetite project in Australia with an estimated two billion tons of identified magnetite ore.

This project also involves construction of significant processing and support infrastructures including construction of ultra class open pit mine, 450 MW power station, 25 km length slurry pipeline and 51 gigalitre desalination plant to serve the project water needs without drawing on the region’s precious groundwater.

The mine will produce 27.6 million tons magnetite pellets and concentrate a year and the capacity will last for 25 years. Mine development and infrastructures as well as construction of a new deep water port with stockyard and transshipments facilities and a 28 km access route from the mine site to the new port are expected to cost USD 3.5 billion.

Access Route and Causeway over Mudflats

The access route must be completed in advance to allow transportation of all heavy equipments for the mining pit, power station, desalination plant and all related facilities for the project. During construction, the access route had to provide for the delivery of heavy over-dimensional loads of up to 2000 tons GVM, on a 17 m wide, multi-axle platform.

A key infrastructure component of the route is the 2.0 km long causeway embankment over the soft tidal mudflats between Cape Preston and the mainland. Figure 6 shows the project location, the layout of the access route and the location of the causeway.

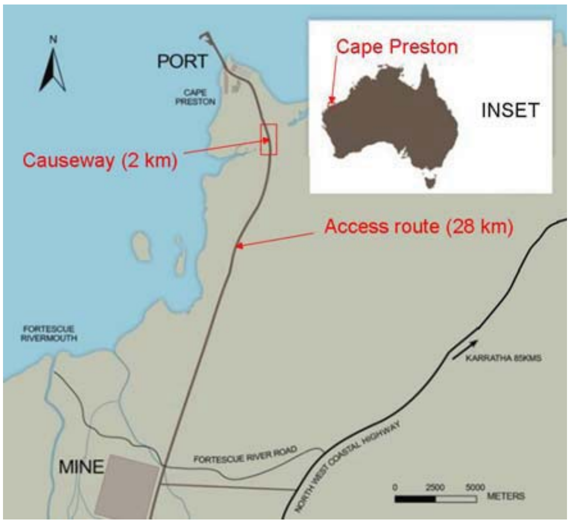


Fig. 6 Location of Sino Iron Project and causeway (with map of Australia as inset)

Subsurface conditions along the causeway comprise an estuarine mud flat underlain primarily by laterized coral over bedrock. The mudflat is flooded twice daily, with the minimum groundwater level being essentially at the surface of the mudflat. Over the southern part of the causeway alignment the soft mud is about 1.5 m to 2 m thick, increasing to about 4 m at the main creek where slope failures can be observed along the banks.

Beneath the soft clay is a thin alluvial layer of stiff clay and dense sand which overlies 2 m to 3 m of highly weathered, low to high strength carbonate rock with numerous clay seams. The calcareous rock was formed on andesitic bedrock, an igneous rock of volcanic origin. The causeway was planned for two stages of construction:

- Stage 1: construction up to level +4.50 m AHD (up to 3 m high embankment) for heavy construction equipment loads, and 2000 tons heavy platform loads of 90 kN/m^2 over a 16 m x 16 m footprint for delivery of mill components delivered from the port to the mine site,
- Stage 2: construction to final level of +6.90 m AHD for the long-term access to the port and potential future expansion of mine tenements, and similar heavy platform loads as for Stage 1.

The causeway embankment ranged in height from 1.5 m to 7 m, with a crest width of 32 m to be constructed with mine waste rock. The side slopes of the causeway were maintained at 1V:2H. At about mid distance of the causeway a 200 m long reinforced concrete bridge was constructed to enable river flows during both normal and flood periods. The estuarine mud is a very soft to firm silty clay of medium to high plasticity.

Investigation by in-situ cone penetration and vane shear tests resulted in a design undrained shear strength of 6 kN/m^2 from the surface to 1.5 m depth, increasing thereafter at 7 kN/m^2/m to a maximum of 20 kN/m^2 (Kerkovius and Semple, 2010). This foundation layer was unable to support the embankment with the required service loading.

To construct the causeway a number of design and construction options were evaluated. These ranged from stage construction to soft soil replacement. The solution (based on economical and environmental viability) was to reinforce the rock fill with high strength reinforcement geotextiles to serve the engineering requirements of both Stage 1 and Stage 2.

Embankment Stability Analysis

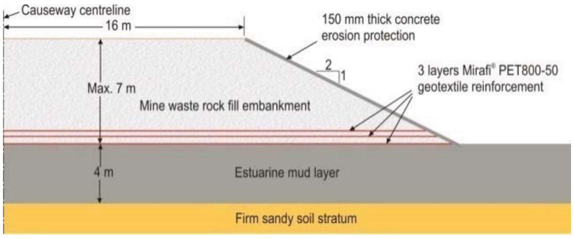
Figure 7(a) shows the typical cross section of the basal reinforced causeway embankment of Sino Iron Project. Embankment stability analysis was performed using the SLOPE/W computer software. Failure modes analyzed included rotational failure and translational failure. The lowest factor of safety for the 7 m high embankment and bridge abutment over the soft estuarine mud riverbank was found to be less than 1.

For this project the reinforced embankment is required to achieve minimum factor of safety of 1.4 against rotational circular failure and translational failure. To satisfy the design requirement, the causeway embankment was reinforced with three layers of high tenacity low creep polyester geotextile having ultimate tensile strength of 800 kN/m. Table 3 shows the derivation of long term design strength of the selected reinforcement geotextile having ultimate tensile strength of 800 kN/m.

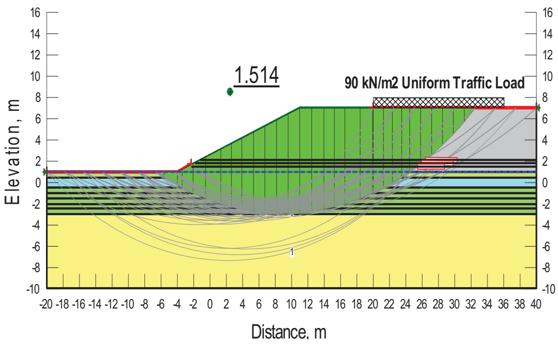
Table 3 Allowable long term design strength of reinforcement geotextile used for Sino Iron Project

Property of geotextile	Strength of geotextile	Partial factor	Reinforcement Geotextile
Ultimate tensile strength	T_u (kN/m)		800
Long term creep		f_{mc}	1.45
Construction damage (in contact with rock fill)		f_{md}	1.75
Environment degradation		f_{me}	1.1
Allowable long term design strength, $T_a = T_u / (f_{mc} \times f_{md} \times f_{me})$	T_a (kN/m)		287

The construction damage factor used for this project was 1.75, based on actual field test done at a previous project in Australia. Figure 7(b) shows the output of one such analysis using SLOPE/W.



(a)



(b)

Fig. 7 Basal reinforced causeway embankment of Sino Iron Project (a) typical cross section (b) output of embankment stability analysis using SLOPE/W

Embankment Construction

The project had an extremely tight time schedule. Planning work for the project started in March 2006 but actual construction on the project began in mid-2008. The lowest layer basal reinforcement geotextile was placed directly on the surface of the soft estuarine mud with rolls of geotextile laid out ninety degrees to the direction of the causeway embankment. No geotextile joins were allowed in this direction across the width of the embankment.

The first mine waste rock fill was placed on top of the reinforcement geotextile, spread out and compacted to construct an initial platform of 0.5 m thickness. On top of this fill platform the second layer of reinforcement geotextile was placed and then a 0.3 m thick fill layer placed on top. Finally, the third geotextile layer was placed and then the embankment was constructed to Stage 1 level which served as the interim haul road for the transportation of heavy equipment and machinery during the initial construction phase of the mine and infrastructures.

Where the causeway embankment abutted the central bridge structure another three layers of reinforcement geotextile, placed coincidentally with the cross-wise layers, were used at the base of the 7 m high abutments to ensure adequate stability in the vicinity of the main river channel. These three layers were placed 40 m into the causeway embankment to ensure the bridge abutments had adequate stability. Figure 8(a) shows the layout of reinforcement geotextile on site. Figure 8(b) shows view of the reinforced embankment under construction. Figure 8(c) shows the transportation of the giant grinding mill along the access route. Figure 8(d) shows view of the bridge under construction.

(a)

(b)

(c)

(d)

Fig. 8 Sino Iron Project (a) layout of geotextile reinforcement (b) reinforced embankment under construction (c) transportation of the giant grinding mill along the access route (d) bridge under construction

Stage 2 of causeway embankment construction involved filling to the final design height. Construction works were scheduled for completion in September 2010 and targeting first production by end of 2010. The use of basal reinforcement has enabled the causeway to be constructed quickly, directly on the estuarine mud foundation, without soil replacement. Consequently, the impact on the environment has been reduced to a minimum. No significant embankment deformations have been observed.

SLUDGE POND CAPPING AT WENCHANG WASTEWATER TREATMENT PLANT, HARBIN CITY, CHINA

Background

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. Figure 9 shows the Google satellite view of the 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.

Fig. 9 Sludge pond at Wenchang WTP

Sludge Properties

The nature of the sludge generated from a wastewater treatment plant depends on the level of treatment that has been instituted. Primary treatment is the first step in wastewater treatment. During primary treatment raw sewage is grated to remove large debris and then screened to filter out smaller items. Brief residence in a grit tank allows sand and gravel to settle for removal. The waste stream is then pumped into primary sedimentation tank where about half of the suspended, organic solids settle to the bottom as sludge. Secondary treatment consists of biological degradation of the remaining suspended solids, which will then settle out in a sludge settling tank. Sludge is also generated within specific processes of the treatment e.g. thickeners, etc.

Solids concentration of sludge generated from a wastewater treatment plant can vary anything between 0.5 to 16 percent (Metcalf and Eddy, 2003). In geotechnical terms, the moisture content of sludge would vary from 500 to 20000 percent. In the pond, the sludge would thicken towards the bottom as solids in suspension and colloids settle out gradually over extended period of time. Even the bottom solids would generally have only 25 to 30 percent solids concentration as a maximum. That is equivalent to 230 to 300 percent in moisture content. In addition, the sludge consists of largely biosolids. The specific gravity of the solids in sludge generally range from 1.2 to 1.4 but can be higher when lime is added to the primary tanks for phosphorus removal. The relative density of the sludge can vary from 1 to 1.05.

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 for normally consolidated clay is found to depend on the current vertical effective stress (Ladd et al, 1977) and can be estimated to be between 0.22 to 0.25 times the vertical effective stress (Jewell, 1996). Assuming the relative density of sludge is 1.05, the buoyant relative density would be 0.05. Therefore the effective vertical stress at 1 m depth would only be 0.5 kN/m^2. Thus the shear strength profile of the pond sludge would have an upper-bound value of about 0.13 kN/m^2 at 1 m depth and linearly increasing by about 0.13 kN/m^2 for every depth increase of 1 m. This is based on the assumption that the pond sludge has fully gained normally consolidated shear strength. The pond sludge is most likely still under-consolidated.

Capping Layer Reinforcement Design

The situation was unique and there was no design precedent to adopt. The pond has been filled with sludge to depth of 6 m. A conservative approach was adopted in the selection of geotextile as capping layer reinforcement. For the design of the soil capping layer, the pond sludge was assumed to have zero shear strength. It was anticipated that the first finger berm would settle the most. 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 of 10 kN/m^2 was assumed in design.

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.

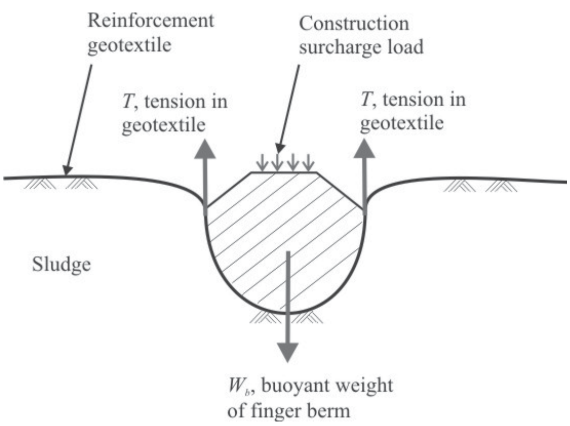


Fig. 10 The force diagram for the determination of the tension in the reinforcement

The design strength of the reinforcement geotextile, T_d, should be greater than the tension in the geotextile, T. Figure 10 shows the force diagram for the determination of the tension in geotextile. Equation 1 can then be rewritten as follows:

$$T_{ult} \geq f_{mc} f_{md} f_{me} T \qquad (4)$$

The worst case scenario for the tension in geotextile, T, was 195 kN/m. The values of 1.45, 1.2 and 1.1 were adopted for f_{mc}, f_{md}, f_{me} respectively. Equation 4 requires a minimum value of 373 kN/m for T_{ult}. Thus the reinforcement geotextile with tensile strength of 400 kN/m ultimate tensile strength was specified for this project. 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 stress in the direction longitudinal of the berm. As such two layers of the geotextile reinforcement would be installed, each to be laid perpendicular to one another.

Capping Layer Construction

Conventionally, pond capping works involve deployment of a prefabricated panel of geotextile that is large enough to cover the entire pond area and including edge extras for the purpose of anchoring in trenches. At Wenchang WTP this methodology was not practical to adopt due to site constraints. Such a large prefabricated panel of geotextile would require a fabrication 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 both prefabrication and deployment to be done in this way.

The method adopted for the deployment of geotextile reinforcement involved seaming of rolls of geotextile after they have been laid out over the sludge pond. To do that, floating platforms were placed above the sludge pond. These floating platforms were made of polystyrene foam slabs sandwiched between plywood panels. These floating platforms were laid out to enable workers to walk on and conduct the work of laying of the geotextile reinforcement as well as seaming adjacent rolls of geotextiles together.

Figures 11(a) to 11(f) show the construction sequence of laying the capping geotextiles over the sludge pond. Figure 11(a) shows the unrolling and laying into position of geotextile reinforcement. Figure 11(b) shows the extraction of floating platforms from underneath of geotextile reinforcement. Figure 11(c) shows the trenching of geotextile reinforcement to provide anchorage on the banks of sludge pond.

Figure 11(d) 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 geotextile reinforcement initially. The geotextile reinforcement only started picking up tension when the existing slack induced during laying has be taken up as a result of sinking in of the initial finger berm. This effect is clearly evident in Figure 11(e).

(a)

(b)

(c)

(d)

(e)

(f)

Fig. 11 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) sinking in of initial finger berm, (f) filling between finger berms

As adjacent finger berms are advanced, the sinking of fill material downwards is reduced. This is because as the geotextile is tensioned as a consequence of more finger berms being deployed, the tensioned membrane effect helped support the finger berms and reduce deformations. In between finger berms, the geotextile reinforcement confines the sludge below, resulting in an uplift pressure that can then support loading above.

Figure 11(f) shows the ability of workers to walk directly on top of the geotextile as well as allow small dump trucks to transport aggregates for filling 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 works.

CONCLUSIONS

The use of geotextile reinforcement for ground improvement was discussed. Two mechanisms of geotextile reinforcement for ground improvement were discussed; one associated with small differential settlements and the other when significant differential settlements developed. Two case studies were presented; one for each of the reinforcement mechanisms discussed.

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