Geosynthetic tubes and mats : experimental and analytical studies

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**NANYANG
TECHNOLOGICAL
UNIVERSITY**

**GEOSYNTHETIC TUBES AND MATS:
EXPERIMENTAL AND ANALYTICAL
STUDIES**

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SCHOOL OF CIVIL & ENVIRONMENTAL ENGINEERING

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GEOSYNTHETIC TUBES AND MATS: EXPERIMENTAL AND ANALYTICAL STUDIES

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SUMMARY

Geosynthetic tube has been widely used in projects for coastal protection, dyke construction, flood control and dewatering waste sludge in many countries. In order to improve our understanding of the performances of geosynthetic tubes and geosynthetic mats and develop reliable design tools, both experimental and analytical/numerical studies were carried out. These included a series of model tests conducted on impermeable geosynthetic tubes, derivation of closed-form solutions for the design of impermeable geosynthetic tubes, evaluation of the effect of deformable foundation on the design of geosynthetic tubes, model tests and analytical analyses of 2-layer stacked impermeable geosynthetic tubes resting on rigid foundation, model tests of permeable geosynthetic tubes and methods of analysis, and methods to accelerate the dewatering process of waste sludge using permeable geosynthetic tubes.

Closed-form solutions for the design of water or slurry inflated impermeable geosynthetic tubes resting on rigid foundation were derived using force equilibrium methods. With these solutions, the tensile force along the tube, the contact width with ground, the width, height, perimeter, and area of cross-section of a geosynthetic tube can easily be calculated even without considering the whole cross-section. Relatively large scale model tests using geosynthetic tubes of up to 4 m long and 2 m wide were carried out. The results of the closed-form solutions agree well with the model test results. The e-log p method was proposed for the analysis of geosynthetic tubes or mats on deformable foundation. This method was evaluated against numerical results using FLAC and compared with the existing method using Winkler foundation. The comparisons show that the proposed e-log p method agrees better with the numerical results particularly when geosynthetic mat is analyzed.

One problem with the use of large geosynthetic tubes or mats is that seams will exist on the tubes or mats. The strength of the seams can be lower than 50% of the original geosynthetic material without seams. A new reinforced seaming method

was developed to improve the seam efficiency of two selected geosynthetic sheets with seams. This so-called “RJ” method increased their seam efficiency from the normal 50% to over 75%.

To accelerate the dewatering process of waste sludge using permeable geosynthetic tubes, the existing stacking method was evaluated using model tests. A new method, the so-called vacuum method, was proposed to improve the dewatering efficiency. The vacuum method was also verified using model tests. The model test results showed that the vacuum method could not only accelerate the dewatering process, but also reduce the tensile strength of the geotextile used for the tube. Theoretical methods for analyzing the cross-section and tensile forces at the end of filling and for calculating the deformation of the geosynthetic tubes during the dewatering process were proposed. The results of the theoretical methods agreed well with the model test results.

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LIST OF SYMBOLS

A	Area of cross-section of the geosynthetic tube
A_1	Area of cross-section of the top geosynthetic tube
A_2	Area of cross-section of the bottom geosynthetic tube
A_R	Area of reference cross-section
b	Contact width with ground
B	Contact width with ground surface
B	Width of the geosynthetic tube
b_R	Contact width with ground of reference cross-section
C_B	Factor of width
C_b	Factor of contact width with foundation
C_c	Compression index
C_L	Factor of perimeter
C_r	Recompression index
d	Thickness of geosynthetic material
D_e	Dewatering efficiency
E	Undrain modulus of clay
e_0	initial void ratio,
G	Shear modulus of soil foundation
G_s	Specific gravity of soil mass
H	Height of the geosynthetic tube
H_0	Height of the geosynthetic tube at the end of filling
H_1	Height of the top geosynthetic tube
H_2	Height of the bottom geosynthetic tube
H_f	Settlement of geosynthetic tube
H_F	Height of permeable geosynthetic tube at the end of filtration
H_{SF}	Final height of the stacked bottom tube
H_C	Height of permeable geosynthetic tube at the end of consolidation
H_{ft}	Trial settlement of geosynthetic tube during iteration
H_i	Thickness of each divided layer and
H_t	Trial height of the geosynthetic tube during iteration

H_w	Height of external water level
H_{w1}	Total head in the top geosynthetic tube
H_{w2}	Total height of the bottom geosynthetic tube
K	Bulk modulus of soil foundation
k	Height to width ratio
K_f	Modulus of subgrade reaction
L	Perimeter of cross-section
L_1	Perimeter of cross-section of the top geosynthetic tube
L_2	Perimeter of cross-section of the bottom geosynthetic tube
L_C	Length of the contact edge between two tubes
LL	Liquid limit
L_R	Perimeter of reference cross-section
L_t	Trial perimeter of cross-section during iteration
P'_f	Reaction force of soil subgrade
p'_f	Reaction pressure of soil subgrade
p_0	Pumping pressure of single tube
p_1	Pumping pressure in top tube
p_2	Pumping pressure in bottom tube
P_f	Surcharge force
p_f	Surcharge pressure
PI	Plastic index
PL	Plastic limit
p_T	Water pressure acting down on contact surface
p_T	Water pressure acting up on contact surface
P_w	External hydraulic water force
p_w	External hydraulic water pressure
Q	Factor of pumping pressure
q_i	Contact vertical surface pressure at point i
R	Radius of contact surface between two tubes
R^2	Coefficient of determination
R_{Nd}	Normalized rate of deformation
S_{ci}	Settlement of each divided layer

S_{ci}	Total settlement of each soil layer
S_u	Undrained shear strength
T	Tensile force along the geosynthetic tube per unit length
T_1	Tensile force along the top geosynthetic tube per unit length
T_2	Tensile force along the bottom geosynthetic tube per unit length
T_{max}	Maximum tensile force along the geosynthetic tube per unit length
T_{min}	Minimum tensile force along the geosynthetic tube per unit length
w_0	Water content of filling slurry
w_f	Water content of consolidated kaolin
γ	Unit weight of filling slurry
γ'	Effective unit weight of soil
γ_s	unit weight of soil
γ_{sat}	Saturated unit weight of soil
γ_w	Unit weight of water
Δe_i	Change of void ratio
δ_i	Vertical surface deflection at point i
$\Delta\sigma'$	Surcharge pressure
$\Delta\sigma_{zi}$	Overburden pressure due to the acting of geosynthetic tube
ν	Poisson's ratio
σ_0'	initial load pressure
σ_c'	Pre-consolidation pressure
AOS	Apparent pore opening size
GWL	Ground water level
HWG	Polypropylene heavy weight woven geosynthetic
LWG	Polypropylene light weight woven geosynthetic sheet
OCR	Over-consolidate ratio
PHD	Prefabricated horizontal drain
SSa	“Prayer” seam
SSd	“Butterfly” seam
SSn	“J” seam

CHAPTER 1

INTRODUCTION

1.1 Background

One of the biggest developments in geotechnical engineering in recent decades is the use of new materials, in particular, the use of geosynthetic materials. One of the notable applications of geosynthetic materials for river, coastal and other types of geotechnical constructions is the use of geosynthetic tubes. Since its early development in the eighties, it has been used in many projects in Australia, Brazil, China, France, Germany, Indonesia, Korea, Japan, Malaysia, Netherlands, USA and other countries (Davis et al., 1992; Bogossian et al., 1982; Miki et al., 1996; Nickels and Heerten, 1996; Leshchinsky et al., 1996; Saathoff et al., 2007; Katoh et al., 1994; Lee, 2009; Yan and Chu, 2010; Shin et al., 2002).

The applications of geosynthetic tubes include the following categories: (1) To elevate the height of existing dams or spillways (Perry, 1993), (2) To divert water for irrigation (Tam, 1997), (3) To construct breakwaters in beach (Alvarez et al., 2007), coastal erosion prevention and dike construction (Shin and Oh, 2007; Yan and Chu, 2010; Leshchinsky et al., 1996), (4) To control flood or water level (Biggar and Masala, 1998; Ennis, 1997; Fowler, 1997; Landis, 2000; and Plaut et al., 1998); and (5) To dewater sediment and industrial sludge. Geosynthetic tubes have also been proven useful in disaster mitigation and rehabilitation projects and have been used in many flood control and coastal protection projects (Chu et al., 2009; Fowler, 1997; Plaut and Suherman, 1998; Biggar and Masala, 1998)

The use of geosynthetic tubes offers the following advantages: (1) it is cost-effective. The high tensile strength of geosynthetic materials offers the best combination with soil. As a result, many geotechnical designs can be optimized by the use of geosynthetics. The use of a layer of geosynthetic to form a tube with soil (either sand or clay slurry) as fill materials is often more economical than the use of concrete or rocks required in the conventional design; (2) the construction process can be made simpler and faster (Oh and Shin, 2006); (3) it enables local soils and even slurry materials to be used as fill materials for the construction.

In terms of applications, geosynthetic tubes can be made into either permeable or impermeable type. The geosynthetics used can be either woven or nonwoven geotextiles (Austin and Theisen, 1996; Kim et al., 2004) or rubbers (Jongeling and Rövekamp, 1999). The fill materials can be air, water, clay slurry or sand. The tubes can be either in the sausage shape or flat. The latter has also been called geotextile mattress or geomat.

1.2 Objectives and Scope of works

The project aimed to achieve the following objectives:

- 1) To develop a simple analytical method for slurry inflated impermeable geosynthetic tubes and mats resting on rigid foundation;
- 2) To derive an analytical method for stacked impermeable geosynthetic tubes;
- 3) To develop new analytical methods to consider the influence of the settlement and stiffness of the foundation soil;
- 4) To develop methods to analyze the displacement and tensile stress of permeable geosynthetic tubes with possible incorporation of the consolidation process;
- 5) To improve the understanding of the performance and displacement characteristics of geosynthetic tubes through fully instrumented laboratory model tests.

The scope of works included analytical and numerical studies to develop better methods for designing, and experimental studies to carry out model tests to evaluate

the performance of geosynthetic tubes or geosynthetic mats, and use the results of model tests to verify the predictions of the analytical or numerical methods.

1.3 Outline of Thesis

This thesis is divided into nine chapters. Chapter 2 presents a literature review on previous analytical, experimental and numerical studies on geosynthetic tubes. The model test results for impermeable geosynthetic tubes and mats resting on rigid foundation are described in Chapter 3. The analytical methods for geosynthetic tube and mat resting on rigid foundation are presented in Chapter 4. An extension of the methods to cases for geosynthetic tubes resting on deformable foundation is also made and presented in Chapter 5. The analysis of stacked geosynthetic tubes are given in Chapter 6. Model tests on permeable geosynthetic tubes fill with clay slurry are discussed in Chapter 7. Two methods to improve the dewatering efficiency for using geosynthetic tubes to dewater sludge are proposed in Chapter 8. Finally, the conclusions and recommendations for future study are made in Chapter 9.

CHAPTER 2

LITERATURE REVIEW

2.1 Application of Geosynthetic Tubes

Geosynthetic tubes have been used in recent years in many projects including coastal protection, dyke construction, flood control and waste sludge dewatering in Australia, Brazil, China, France, Germany, Indonesia, Korea, Japan, Malaysia, Netherlands, USA and other countries (Davis et al. 1992; Bogossian et al. 1982; Yan and Chu, 2010; Saathoff et al. 2007; Shin et al. 2002; Katoh et al. 1994; Lee, 2009; Miki et al. 1996; Nickels and Heerten, 1996; Leshchinsky et al. 1996). Normally geosynthetic tubes are formed by sewing or gluing geosynthetic sheets together and then inflated with water, clay slurry, sand or waste sludge. The geosynthetic tubes are sometimes stacked together to form a higher dyke or other types of geotechnical structures (Kazimirowicz, 1994; Leshchinsky et al. 1996; Miki et al. 1996; Pilarczyk, 2000; Kim et al. 2005b; Chu et al. 2009; Yan and Chu 2010).

A literature review of the different applications, the analytical, experimental and numerical studies of the geosynthetic tubes are presented in this chapter.

2.1.1 Coastal Protection

In recent years, many methods have been developed to use geotextile or geosynthetic materials for the construction of coastal structures. One of the applications is to use geotextiles acting as formwork in mortar mattress (Silvester and Hsu, 1993) or concrete bags (Pilarczyk, 2000; Sprague and Koutsourais, 1992; Sprague and Fowler, 1994) as shown in Figure 2.1. The exterior geotextile materials

services primarily as a form until the internal cement mortar become hardening. The concrete bags can also be reinforced by driving steel rods through them (Pilarczyk, 2000).

Figure 2.1 Sketch of concrete mattress and bags (after Pilarczyk, 2000)

However, in some areas, it is imperative that any restoring action will not affect the natural environment or absorb any physical media modification. The sand or clay slurry inflated geosynthetic tube is the most ideal method for this purpose which have already been proposed and constructed in many water break projects as shown in Figure 2.2 (Alvarez, 2007; Harris and Sample, 2009; Oh and Shin, 2006; Pilarczyk, 2003; Saathoff et al. 2007; Shin and Oh, 2007).

(a) Breakwater in the sea (after Pilarczyk, 2003)

(b) Offshore breakwaters (after Harris and Sample, 2009)

Figure 2.2 Geosynthetic tubes used for break water application

Similar methods of using sand or clay slurry inflated geosynthetic tubes for dyke construction projects have been presented by Kazimirowicz (1994), Miki et al. (1996), and Leshchinsky et al. (1996). One example of the artificial islands constructed at the Amwaj Islands, Bahrain, is given in Figure 2.3 where geosynthetic tubes were inflated with the sand.

Figure 2.3 Use of geosynthetic tubes for the construction of artificial islands in Bahrain (after Ten Cate Niclon, <http://www.hastex.net>)

However, if sand is not readily available, it is possible to use clay slurry as filling material. One example was presented by Yan and Chu (2010) for dyke construction in Tianjin, China. As shown in Figure 2.4, the large flat geotextile bags in the form of geosynthetic mats, instead of geosynthetic tubes, were adopted for this project. The use of geosynthetic mat for dyke construction has following advantages, (1) geosynthetic mat has no lateral stability problems as the lateral dimension is very large compared with its height; (2) construction can be speedier as pumping can be carried out at a number of points; (3) the dyke made of geosynthetic mats can

accommodate relatively large differential settlement which may result in savings of the foundation treatment.

(a) Initial stage

(b) Intermediate stage

(c) Using bags for slope construction

(d) Using mattress for slope construction

(e) Final stage of the dyke

Figure 2.4 Dyke constructed using clay slurry filled geosynthetic mats
(after Yan and Chu, 2010)

Compared with the traditional ways of constructing shoreline structures using rock or precast concrete units, the use of geosynthetic tubes offers the following

advantages: (1) it is cost-effective; The high tensile strength of geosynthetic materials offers the best combination with soil. As a result, many geotechnical designs can be optimized by the use of geosynthetics. The use of a layer of geosynthetic to form a tube with soil (either sand or clay slurry) as fill materials is often more economical than the use of concrete or rocks in required in the conventional design; (2) the construction process can be made simpler and faster (Oh and Shin, 2006); (3) it enables local soils and even slurry materials to be used as fill materials for the construction. More examples of applications of geosynthetic tubes for coastal protection are shown in Figure 2.5.

Figure 2.5 Applications of geosynthetic tubes in coastal protection projects (after Pilarczyk, 2000)

2.1.2 Dewatering Sludge

In recent years, geotextile tubes are also being used in environmental applications for dewatering waste sludge such as digested biosolids, sewage sludge (Fowler, et al. 1996; Bowles and Fleischer, 1999), dredged materials (Moo-Young, et al. 2002a; Worley, et al. 2004; Zhu, et al. 2010), industrial solid wastes (Worley and Bass, 2004; Worley, et al. 2008), fly ash (Kutay, et al. 2005; Muthukumaran and Ilamparuthi, 2006) and coal slurry.

The waste sludge is firstly filled into the geotextile tube by pump. After that, under the action of pumping pressure and striction of geosynthetic sheet, the water seeped through the permeable geosynthetic sheet and left grained solids inside (Lawson, 2008). After dewatering, the water content of the solids inside of the geosynthetic tube is smaller such that it can be easily transported to a dry disposal area or can possibly be reused if the dewatered materials are some kinds of beneficial materials. Sometimes, the geotextile tubes have to be stacked together to save resting space and accerate the dewatering process as the example shown in Figure 2.6.

Figure 2.6 Photo of the gravity consolidated sludge (from <http://www.tencate.com>)

Traditionally, these high water content wastes are exposed to sunlight for the formation of desiccation crust. However, the process of sun drying is very slow and

only effective for the top several layers. The advantage and disadvantage of using permeable geosynthetic tubes for dewatering waste sludge are shown in Table 2.1.

Table 2.1 Advantages and disadvantages of geotextile tube dewatering method (after Andrews, 1999)

Advantage	Disadvantage
<ol style="list-style-type: none"> 1. Able to dewater large quantities of sediments concurrently 2. Low operating costs 3. Can accommodate high flow rates and rapidly varying flows and solids concentrations, such as those produced from a hydraulic dredge 4. Provides immediate containment of material and reduces odors 5. Provides a method of increasing solids content quickly and efficiently 6. Replace small sludge lagoons, aerators and settling tanks, thus reducing the construction, maintenance and closing costs 	<ol style="list-style-type: none"> 1. Periodic removal of liquids required for maximum consolidation if sufficient drainage away from the tube is not provided 2. Water soluble contaminants can leach out over time.

2.1.3 Flood Control

As global warming has caused the sea level to rise in recent years, the river or coastal related disasters such as tsunami and flood have become higher in frequency and stronger in intensity. The traditional method of building temporary barriers to prevent flooding is to use sandbags. But the filling, transportation and stacking these large amount sandbags need huge amount of manpower and time which are limited during a disaster. Furthermore, these sandbags are also difficult to be disposed after flood receded.

An alternative way to deal with this problem is to use water or slurry filled impermeable geomembrane tube which is made of very low permeability geosynthetic materials such as Butyl Rubber, EPDM Rubber, PVC, Polypropylene, Polyurethane, or Elvaloy (Fowler, 1997; Plaut and Suherman, 1998; Biggar and

Masala, 1998). One example is shown in Figure 2.7 to protect the house from the damage of flood. The advantage of this method is that the tubes can be inflated quickly and re-used in the future. However, the weakness of this method is that the dyke made of the geomembrane tubes cannot be too tall. The water pressure acting on one side may also cause stability problems. Several methods have been developed to solve this problem. Huong et al. (2002) used a wedged single or stacked geomembrane tubes to increase the lateral force. They concluded that the water-filled geomembrane tubes with wedges placed on the downstream side were effective in withstanding floodwaters because it could prevent the rolling of geomembrane tubes. Kim et al. (2004) attached a piece of geomembrane material and placed it with sand berm to the ground on the headwater side. The apron linked the geomembrane and increased the contact area between the geomembrane and the ground surface to stabilize the geomembrane tubes. Kim et al. (2005b) increased the lateral resistance by stacking the tubes in 1-2 shapes and hold them together with straps. The other improved methods such as baffle tube dam and sleeved tube dam were also presented by Kim (2003).

Figure 2.7 Using of water filled geomembrane tubes for flood control
(from <http://www.noaq.com>)

2.1.4 Rubber dams

Rubber dams are installed to function as low level weirs or barrages across a river to raise the water level for the use of diverting flow into a supply canal, conduit for irrigation, domestic, or industrial (Paul, 1998). The water or air inflated rubber bags or rubber dams have been also been used to provide a flexible and sometimes temporary barrier (Zhang et al. 2002; Chu et al. 2009).

The difference between rubber dams and impermeable geosynthetic tubes is that the rubber dam is normally supported and anchored on to a permanent concrete foundation as shown in Figure 2.8 which is used for Ramspol storm surge barrier in Netherlands. The Ramspol storm surge barrier is one of the largest rubber dams in the world so far. It used three identical inflatable rubber dams. The dimensions of each rubber dam were 75 *m* in length, 13 *m* in width and 8.35 *m* in height. A unique feature of this project is that a combination of air and water was used as the inflation medium. This minimized the dimensions of the rubber body and also allowed the height of the dam to be adjusted quickly. More information on this project can be found in Jongeling and Rövekamp (1999).

The advantage of the rubber dam is that it was easily deflated or inflated which can regulate easily the height of dams. The major disadvantage of the rubber dam is its vulnerability against sharp objects. If use ceramic chips as coating and stainless steel meshes against punctures to improve the strength of this material, the cost of the construction of the rubber dam is much higher.

When construction needs to be carried out in a lake or a river, impermeable geomembrane tubes filled with sand or water may be used to form a cofferdam as shown in Figure 2.9.

Figure 2.8 Rubber dam used for the Ramspol storm surge barrier in Netherlands (after Jongeling and Rövekamp, 1999)

Figure 2.9 The top view of temporary barrier using geosynthetic tubes (from www.aquabarrier.com)

2.2 Classification of Geosynthetic Tubes

Geosynthetic tubes can be grossly classified into three categories: (1) permeable geotextile tube or bag, (2) impermeable geomembrane tube and (3) inflatable rubber dam according to the different geosynthetic materials used for forming the tubes. The details of this classification are summarized in Table 2.2.

Table 2.2 Classification of Geosynthetic tubes (after Guo et al. 2009)

Types	Materials for the tube	Filling materials	Applications
Impermeable geomembrane tube	Very low permeability; liner or fluid containment materials is required; high tensile strength; made of butyl rubber, EPDM rubber, PVC, polypropylene, polyurethane, or Elvaloy	Water, clay slurry, sludge or other waste materials	Flood control(Fowler, 1997; Plaut and Suherman, 1998); Contain contaminated materials (Szyszkowski and Glockner, 1987); Form working table; Water level control (Sehgal, 1996); Water diversion (Tam, 1997)
Permeable geotextile tube or bag	Woven or non-woven geotextile; usually made of synthetic polymers- polypropylenes, polyesters, polyethylene, or polyamides with high tensile strength; high geotextile permeability and good soil retention characteristics.	Mainly sand, clay slurry, cement mixed clay, concrete, mortar or waste materials	Dykes (Fowler, 1997; Yan and Chu, 2010); Beach restoration (Oh and Shin, 2006; Alvarez et al. 2007); Coastal erosion prevention (Koerner and Koerner, 2006b; Shin and Oh, 2007); Underwater breakwater (Kim et al. 2004); Dewatering contaminated high water content waste (Fowler et al. 1996; Perry, 1993).
Inflatable rubber dam	Impermeable and high strength synthetics, like macromolecule compound materials.	Water and/or air	Small dams (Sehgal,1996); Height elevation for existing dams or spillways; Water diversion (Tam, 1997); Recreational basins; Contamination prevention, Groundwater supply; Hydroelectricity; Tidal control or flood control (Watson et al.1999a)

2.3 Analytical Methods

For a design purpose, the calculations of tensile forces and the cross-section of the geosynthetic tubes are required. A number of analytical solutions have been proposed as summarized below.

2.3.1 Impermeable Geomembrane Tubes

Analytical solutions for water filled impervious geosynthetic tubes resting on rigid foundation have been derived by Liu and Silvester (1977), Silvester (1986), Leshchinsky et al. (1996), Kazimirowicz (1994), Plaut and Suherman (1998), Antman and Schagerl (2005), Mal k (2007), Ghavanloo and Daneshmand(2009a), Mal k and Sysala (2010), and Cantr é and Saathoff (2010). A summary of some of the analytical methods for impermeable geosynthetic tubes on a rigid foundation are given in Table 2.3.

Table 2.3 Lists of analytical methods for impermeable geosynthetic tube on rigid foundation (after Guo et al. 2009)

Authors	Calculation Profile	Assumptions
Kazimirowicz (1994)	<p>where r = radius of curvature, N_ϕ = Tensile force; p_r = Hydraulic water pressure. Case A ($p_0 \gg \gamma H$) $r = H / 2$; Which means a circular cross section Case B ($p_0 \neq 0$) and C ($p_0 = 0$) $\frac{d^2 z}{dy^2} = -\frac{p_r}{N_\phi} \left[1 + \left(\frac{dz}{dy} \right)^2 \right]^{\frac{3}{2}}$</p>	<p>There is only the membrane state of stress at the covering materials;</p> <p>Plane strain state (2D);</p> <p>No concentrated loads acting on a structure;</p> <p>Shell's own weight is neglected;</p> <p>Filling material has a specific unit weight;</p> <p>Friction between the structure and subsoil is neglected.</p>

Continued

Authors	Calculation Profile	Assumptions
Leshchinsky et al. (1996)	<p> L = circumference of tube r = radius of curvature p_0 = pumping pressure γ = density of slurry </p> <p> $Ty'' - (p_0 + \gamma x)[1 + (y')^2]^{3/2} = 0$ </p>	<p>The geomembrane tube is 2D (plane strain) problem;</p> <p>The pressure loss due to drainage through the tube during filling is ignored;</p> <p>The sheet of tube is modeled as thin, flexible and has negligible weight per unit weight ;</p> <p>No shear stresses develop between the slurry and the geosynthetic</p> <p>The friction between tube and foundation was neglected. Then the tensile force is assumed as constant.</p>
Plaut and Suherman (1998)	<p> where: Q = The net tangential force per area; P = The internal pressure; T = The circumferential tension per width perpendicular to the cross-section; W = Width of tube; H = The height of tube; L = Total perimeter of cross-section; B = The contact length; s = The arc length from the origin lift-off point. </p> <p> $\begin{cases} \frac{dT}{ds} = Q, & T \frac{d\theta}{ds} = P \\ \frac{dx}{ds} = \cos \theta, & \frac{dy}{ds} = \sin \theta \end{cases}$ </p>	<p>The geomembrane tube is 2D (plane strain) problem;</p> <p>The sheet of tube is modeled as extensibility and negligible weight;</p> <p>No shear stresses develop between the slurry and the geosynthetic</p> <p>The friction between tube and foundation was neglected.</p>

Continued

Authors	Calculation Profile	Assumptions
Mal k (2007)	<p>Where α = the angle between the tangential direction and the axis y. H = the tensile force; ρ = density of slurry; g = the gravitational acceleration; p = the pressure of water or slurry at the point O.</p> $\begin{cases} x(\alpha) = -\int_{\frac{\pi}{2}}^{\alpha} \frac{H \sin \theta d\theta}{(p^2 + 2g\rho H(1 + \sin \theta))^{\frac{1}{2}}} \\ y(\alpha) = \int_{\frac{\pi}{2}}^{\alpha} \frac{H \cos \theta d\theta}{(p^2 + 2g\rho H(1 + \sin \theta))^{\frac{1}{2}}} \end{cases}$	<p>The geosynthetic is inextensible and flexible and its weight was neglected.</p> <p>The filling medium (water or slurry) behaves as an ideal liquid which generates hydrostatic pressure in every point and acts in the perpendicular direction to the geosynthetic.</p> <p>There is no friction between the foundation and the geosynthetic.</p>
Mal k and Sysala (2010)	$\begin{aligned} x(\theta) &= x_i + \int_{\theta_i}^{\theta} \frac{t \cos \delta d\delta}{(p_i^2 + 2tg\rho_{i+1}(\cos \delta - \cos \theta_i))^{1/2}} \\ y(\theta) &= x_i + \int_{\theta_i}^{\theta} \frac{t \sin \delta d\delta}{(p_i^2 + 2tg\rho_{i+1}(\cos \delta - \cos \theta_i))^{1/2}} \\ s(\theta) &= x_i + \int_{\theta_i}^{\theta} \frac{td\delta}{(p_i^2 + 2tg\rho_{i+1}(\cos \delta - \cos \theta_i))^{1/2}} \end{aligned}$ <p>where A_i points on the intersection between the cross-section curve and the horizontal lines separating the liquids; ρ_i = the density of slurry at the points A_i; θ_i = tangential angles to the cross-section curve at the points A_i; v = whole area occupied by the liquids. t = tension in the synthetic fabric; l = perimeter of the cross-section, b = contact edge with ground surface, h = height of tube.</p>	<p>The geosynthetic tube is considered as plane strain problem.</p> <p>The synthetic fabric is perfectly flexible, inextensible, and leak proof.</p> <p>The weight of the synthetic fabric is small compared to the weight of the liquids and is neglected.</p> <p>The liquids are separated in layers.</p> <p>The liquid with higher density is under the liquid with lower density.</p> <p>The pressure generated by the weight of the liquids acts in the perpendicular direction to the synthetic fabric.</p> <p>The tube rests on a rigid horizontal foundation and there is no friction between the tube and the foundation.</p>

Continued

Authors	Calculation Profile	Assumptions
Cantr�� and Saathoff (2010)	<p>Where T = circumferential tension per width perpendicular to the cross-section; W = width of tube; H = height of tube; L = total perimeter of cross-section; B = the contact length with ground surface.</p> $\begin{cases} T \frac{d\theta}{ds} = (P_{bot} - P_{atm}) - \gamma Y \\ \frac{dT}{ds} = 0, \frac{dx}{ds} = \cos \theta, \frac{dy}{ds} = \sin \theta \end{cases}$ $C_i = C_0 + C'_{i-1} \varepsilon_i$ <p>where C_i = cross-section in iteration step i, m; C_0 = circumference, unloaded m; B_{il}' = contact width with the ground from previous iteration step, m; C_{il}' = free circumference from previous iteration step, m.</p>	<p>The geomembrane tube is 2D (plane strain) problem;</p> <p>The sheet of tube is modeled as negligible weight;</p> <p>No shear stresses develop between the slurry and the geosynthetic;</p> <p>The friction between tube and foundation was neglected;</p> <p>The sheet of geosynthetic tube is elastic material and strain is considered to be constant.</p>

The solution of geomembrane tube resting on deformable foundation was presented by Plaut and Suherman (1998). The foundations were modeled with linear elastic Winkler Foundation. The contact friction with geomembrane tubes was not considered. They used non-dimensional parameters for calculation in which H and H_f were normalized by dividing them by the perimeter of cross-section, L , the pumping pressure, p , by γL , the tensile force, T , by γL^2 , and the modulus of subgrade reaction, K_f , by γ . The tube was modeled as an inextensible membrane and filled with an incompressible fluid. The geometry of geosynthetic tube was solved by partial differential equations with non-dimensional parameters as summarized in

Table 2.4. Plaut and Suherman (1998) also extend this method to analyze the geomembrane tube acted by external water on one side.

However, there is a lack of suitable analytical methods for the calculation of geosynthetic tubes resting on deformable foundation. The only method available is based on an over simplified Winkler model. The existing Winkler foundation method may not always be applicable directly to geosynthetic mat as the solution may not converge when the ratio between the height and width of the tube is very small.

Table 2.4 Analysis of impermeable geomembrane tube on deformable foundation

Authors	Calculation Profile	Assumptions
Plaut and Suherman (1998)	<p>Where: P = the net internal pressure; T = the circumferential tension per width perpendicular to the cross-section; W = width of tube; H = the height of tube; L = total perimeter of cross-section; B = the contact length; s = the arc length from the origin lift-off point.</p> $\begin{cases} \frac{dt}{ds} = \alpha k_f (y - h_f) \sin \theta \\ \frac{d\theta}{ds} = \frac{1}{t} [\alpha k_f (y - h_f) \cos \theta + h_{im} + h_f - y] \\ \frac{dx}{ds} = \cos \theta, \frac{dy}{ds} = \sin \theta \end{cases}$ <p>where, $t = T/\gamma L^2$, $x = X/L$, $y = Y/L$, $s = S/L$, $h_f = H_f/L$, $k_f = K_f/\gamma$</p>	<p>Filling material has a specific unit weight;</p> <p>Friction between the structure and subsoil is neglected;</p> <p>The geomembrane tube is 2D (plane strain) problem;</p> <p>The sheet of geosynthetic tube is modeled as inextensible weightless materials;</p> <p>The filled material and the stiffness coefficient of foundation are uniform.</p>

Through the review of the above solutions for impermeable geosynthetic tubes resting on rigid foundation, it can be seen that most of the existing solutions require the running of a computer program. This is not convenient for preliminary design where some trial and error processes or parametric studies are involved in the

selection of the dimensions of the geosynthetic tubes. Further study is required to develop a more convenient and effective method for preliminary design.

Some other special cases of impermeable geomembrane tubes to improve the lateral resistance of external water pressure were analyzed by some researchers. Breteler and van Wijhe (1994) have done extensive research on stacked tubes for Delft Hydraulics. The testing model tubes were stacked as breakwaters to fight various wave heights and currents acting on the side. The solution for three stacked tubes with external water level lower than it has been proposed by Klusman (1998). Plaut and Klusman (1999) presented formulation of stacked tubes resting on linear elastic Winkler Foundations. Two cases were considered in this paper, 1) one tube stacks on top of another and 2) one tube straddling two tubes resting on deformable foundation. Kim (2003) presented and analyzed four types of new methods as apron-tube dam, baffle tubes dam, sleeve tubes dam and stacked tubes dam to improve the stability of geomembrane tubes as shown in Figure 2.10. They used theoretical analysis, lab experiments and finite elements to prove these new methods. All these methods were carried out with these two or three stacked geosynthetic tubes of same perimeter.

Figure 2.10 Four types of water-filled geomembrane tubes (after Kim, 2003)

Klusman (1998) also analyzed the 2-layers stacked geosynthetic tubes resting on rigid foundation (Figure 2.11(a)) and on deformable foundation (Figure 2.11(b)). The whole cross-section of two tubes was divided into three sections in the case of rigid foundation and four sections of deformable foundation.

Figure 2.11 Calculation models of 2-layer stacked geomembrane tubes (after Klusman, 1998)

The geosynthetic tubes can be stacked in several layers to construct a higher dyke or to dewater waste sludge. Several methods have been proposed to analyze the cross-section and tensile force of the stacked geosynthetic tubes of the same perimeters. Usually, the dimensions of the geosynthetic tube at the bottom are larger than that on the top in order to form a slope to enhance the stability. Therefore, there is a need to develop a new analytical method for stacked geosynthetic tubes. Furthermore, the existing method cannot be used when geosynthetic mats are involved.

2.3.2 Permeable Geotextile Tubes

The analysis of a permeable geosynthetic tube is more difficult than that of an impermeable one as the consolidation process of the soil in the tube during or after filling needed to be modeled too. One analytical method proposed by Leshchinsky et al. (1996) was to assume that the consolidation process to be one-dimensional, that is, the width of the geosynthetic tube does not change during the consolidation process. Shin and Oh (2004) presented an approach to calculate the consolidation

process of geosynthetic tube called the settling and self-weight consolidation method. The details of these methods are summarized in Table 2.5.

Table 2.5 A list of analytical methods for permeable geosynthetic tube on rigid foundation (after Guo et al. 2009)

Authors	Calculation Profile	Assumptions
Leshchinsky et al. (1996)	$\left\{ \begin{aligned} \frac{\Delta h}{h_0} &= \frac{\Delta e}{1+e_0} = \frac{e_0 - e_f}{1+e_0} = \frac{G_s(w_0 - w_f)}{1+w_0 G_s} \\ w_0 &= \frac{G_s - \gamma_{slurry} / \gamma_w}{G_s(\gamma_{slurry} / \gamma_w - 1)} \\ w_f &= \frac{G_s - \gamma_{soil} / \gamma_w}{G_s(\gamma_{soil} / \gamma_w - 1)} \end{aligned} \right.$ <p>Where, Δh and h_0 are the decrease in height and initial height of the tube; G_s the specific gravity of solid; e_0 the initial void ratio; w_0 and w_f, the initial and final water content of filling material; γ_{slurry} and γ_{soil} are the unit weight of slurry and consolidated soil.</p>	<p>The width of tube is constant during the consolidation process;</p> <p>The consolidation process is 1-dimension;</p> <p>The filling material is full saturated;</p> <p>The filling material is uniform and during consolidation process its water content is uniform.</p>
Shin and Oh (2004)	<p>Direct calculation from measured soil properties and equations:</p> $C_F = \frac{-k}{\rho_f(1+e)} \frac{d\sigma'}{de} \quad (1)$ <p>where σ' is the effective stress, e the void ratio, k the permeability, and ρ_f the pore fluid density;</p> <p>1) By comparison of self-weight consolidation curve:</p> $C_F = \frac{Tz_0^2}{t} \quad (2)$ <p>where, T is time factor, t real time, z_0 total thickness of the soil and C_F coefficient of consolidation.</p> <p>2) Comparison of pore pressure dissipated curve:</p> <p>Plot the isochrones of excess pore water pressure (pwp) with factor T; Real pwp at a particular time are then compared with these isochrones to find a value of time factor T corresponding to the real time; Then C_F is calculated with Equation (2).</p>	<p>The large strain consolidation theory is used;</p> <p>The instant deposition of tube is a zero effective stress state;</p> <p>The filling material is uniform. During consolidation process, its water content, pore water pressure and density are uniform.</p> <p>The void ratio/effective stress and the permeability/void ratio relationship are linear, and that the coefficient of consolidation C_F is a constant.</p>

2.3.3 Rubber Dams

Many calculation methods have been developed based on the equilibrium cross-sections of liquid-filled rubber dams with or without external water acting on one side (Plaut, 1990; Plaut and Cotton, 2005; Ghavanloo and Daneshmand, 2009a & b;

and Watson, et al. 1999a). A summary of some of the analytical methods for rubber dam are given in Table 2.6.

Rubber dams are normally fixed onto a rigid concrete base, whereas a geosynthetic tube is resting on soil foundation. Therefore, the analytical methods for rubber dams and impermeable geomembrane tubes are different. The differences among the different analytical methods are mainly in the assumptions as shown in Table 2.6.

Table 2.6 A list of analytical methods for rubber dam on rigid foundation
(after Guo et al. 2009)

Authors	Calculation Profile	Assumptions
Plaut and Cotton (2005)	<p>Where, θ is the angle between tangent and horizontal; s is arc length; L is the total perimeter; B is the contact width; P is internal air pressure; μ is the mass per length of the tube; Q is the tension along the tube; The parameters are normalized as follows,</p> $x = X/L, \quad y = Y/L, \quad q = Q/\mu g L^2, \quad \sigma = S/L, \quad \omega = \Omega \sqrt{L/g}$ $\begin{cases} \frac{dx_d}{ds} = -\theta_d \sin \theta_e \\ \frac{dy_d}{ds} = -\theta_d \cos \theta_e \\ \frac{d\theta_d}{ds} = \frac{1}{q_e} [\omega^2 (x_d \sin \theta_e - y_d \cos \theta_e) - \theta_d \sin \theta_e - q_d \frac{(p + \cos \theta_e)}{q_e}] \\ \frac{dq_d}{ds} = -\omega^2 [\omega^2 (x_d \cos \theta_e + y_d \sin \theta_e) + \theta_d \cos \theta_e] \end{cases}$	<p>Problem solved: Dynamic quantities of air filled rubber dam</p> <p>The assumption of this paper:</p> <ul style="list-style-type: none"> • The tube is filled with air and its weight is considered; • The foundation is rigid and friction with tube is neglected; • The membrane is inextensible and with no bending stiffness.

Continued

Authors	Calculation Profile	Assumptions
Ghavanloo, and Daneshmand (2009a)	<p>Where, L = the total perimeter; C = the contact width between tube and inclined plane; P = internal air pressure; μ = the mass per length of the tube; t = the tension along the tube;</p> $\begin{cases} x = -\tau_0(1+p) \int_{-\frac{\pi}{2}}^{\alpha} \frac{\sin(\delta - \gamma_0)}{(p - \sin \delta)^2} d\delta \\ y = -\tau_0(1+p) \int_{-\frac{\pi}{2}}^{\alpha} \frac{\cos(\delta - \gamma_0)}{(p - \sin \delta)^2} d\delta \\ \tau_0 = [(1+p) \left(\int_{-\frac{\pi}{2}}^{\frac{3\pi}{2}} \frac{1 + \sin(\delta - \gamma_0)}{(p - \sin \delta)^2} d\delta \right)]^{-1} \end{cases}$ <p>where, The parameters are non-dimensional with $x=X/L$, $y=Y/L$, $p=P/\lambda g$, $\varpi = \Omega\sqrt{L/g}$, $\tau = \tau/\lambda gL$</p>	<p>Problem solved: Air filled rubber dam resting on inclined rigid plane foundation</p> <p>The assumption of this paper:</p> <ul style="list-style-type: none"> • The geomembrane tube is 2D (plane strain) problem and filled with air; • The membrane is inextensible and with no bending stiffness; • The tangential strain of tube equals to zero; • The internal air pressure is assumed constant within the cross-section area.
Ghavanloo, and Daneshm (2009b)	$\begin{cases} x(s) = \frac{p}{2(\pi - \psi_0)(p+1)} \left[\sin\left(\frac{2(\pi - \psi_0)(p+1)s}{p} + \psi_0\right) - \sin \psi_0 \right] + \frac{1}{2(p+1)} \\ y(s) = \frac{p}{2(\pi - \psi_0)(p+1)} \left[\cos \psi_0 - \cos\left(\frac{2(\pi - \psi_0)(p+1)s}{p} + \psi_0\right) \right] \end{cases}$ <p>where, x, y = the horizontal and vertical coordinate; p = the internal pressure; ψ_0 = the tangent angle at lift-off point; s = arc length.</p>	<p>Problem solved: Air filled rubber dam resting on rigid foundation</p> <p>The shape function of rigid foundation $f(x)$ should satisfy the following conditions:</p> <ul style="list-style-type: none"> • $f(x)$ is even function in $(-H, H)$; it is symmetric with Y axis; • $f(x)$ must be at least twice difference on $(-H, H)$; • $f'(x) \geq 0$

Continued

Authors	Calculation Profile	Assumptions
<p>Ghavanloo, and Daneshm (2009b)</p>	<p>Where the shape function of trapezoidal shaped rigid foundation is defined as:</p> $f(x) = -ax - \frac{ab}{2}, \text{ if } \frac{-b}{2} \geq x; 0, \text{ if } \frac{-b}{2} \leq x \leq \frac{b}{2}; ax - \frac{ab}{2}, \text{ if } x \geq \frac{b}{2}$ <p>if $p < p_0$</p> $\begin{cases} x(s) = \frac{p(1-b+b\sqrt{1+a^2})}{2(\pi-\psi_0)(p+\sqrt{1+a^2})} \left[\sin\left(\frac{2(\pi-\psi_0)(p+\sqrt{1+a^2})s}{p(1-b+b\sqrt{1+a^2})} + \psi_0\right) - \sin\psi_0 \right] + \frac{1+b(\sqrt{1+a^2}-1)}{2(p+\sqrt{1+a^2})} \\ y(s) = \frac{p(1-b+b\sqrt{1+a^2})}{2(\pi-\psi_0)(p+\sqrt{1+a^2})} \left[\cos\psi_0 - \cos\left(\frac{2(\pi-\psi_0)(p+\sqrt{1+a^2})s}{p(1-b+b\sqrt{1+a^2})} + \psi_0\right) \right] + \frac{a(1-b-pb)}{2(p+\sqrt{1+a^2})} \end{cases}$ <p>if $p \geq p_0$</p> $\begin{cases} x(s) = \frac{p}{2(\pi-\psi_0)(p+1)} \left[\sin\left(\frac{2(\pi-\psi_0)(p+1)s}{p} + \psi_0\right) - \sin\psi_0 \right] + \frac{1}{2(p+1)} \\ y(s) = \frac{p}{2(\pi-\psi_0)(p+1)} \left[\cos\psi_0 - \cos\left(\frac{2(\pi-\psi_0)(p+1)s}{p} + \psi_0\right) \right] \end{cases}$ <p>where, x, y = the horizontal and vertical coordinate; p = the internal pressure; ψ_0 = the tangent angle at lift-off point; s = arc length; p_0 is limit pressure defined as the pressure at which the contact width equal to b.</p>	<p>The assumption of this paper:</p> <ul style="list-style-type: none"> • The tube is 2D (plane strain) problem and filled with air; • The membrane is inextensible and with no bending stiffness; • The tangential strain of tube equals to zero; • The internal air pressure is assumed constant within the cross-section area.

Continued

Authors	Calculation Profile	Assumptions
Ghavanloo, and Daneshm (2009b)	<p>Where the parabolic function is presented by the simplest form, $f(x) = X^2$.</p> $\left\{ \begin{array}{l} x(s) = \frac{(1 - [\frac{c}{2}\sqrt{1+c^2} + 0.5\ln(c + \sqrt{1+c^2})])}{2(\pi - \psi_0)} \\ \quad \left[\sin\left(\frac{2(\pi - \psi_0)s}{(1 - [\frac{c}{2}\sqrt{1+c^2} + 0.5\ln(c + \sqrt{1+c^2})])} + \psi_0\right) - \sin \psi_0 \right] + \frac{c}{2} \\ y(s) = \frac{(1 - [\frac{c}{2}\sqrt{1+c^2} + 0.5\ln(c + \sqrt{1+c^2})])}{2(\pi - \psi_0)} [\cos \psi_0 - \\ \quad \cos\left(\frac{2(\pi - \psi_0)s}{(1 - [\frac{c}{2}\sqrt{1+c^2} + 0.5\ln(c + \sqrt{1+c^2})])} + \psi_0\right)] + \frac{c^2}{4} \end{array} \right.$	The same as above
	<p>Where r is radius of this circular foundation; α is central angle of the contact arc; β is the central of this rigid foundation.</p> $\left\{ \begin{array}{l} x(s) = \frac{1-r\alpha}{2(\pi - \psi_0)} \left[\sin\left(\frac{2(\pi - \psi_0)s}{1-r\alpha} + \psi_0\right) - \sin \psi_0 \right] + \frac{r \sin(\frac{\alpha}{2})}{2} \\ y(s) = \frac{1-r\alpha}{2(\pi - \psi_0)} \left[\cos \psi_0 - \cos\left(\frac{2(\pi - \psi_0)s}{1-r\alpha} + \psi_0\right) \right] + r \left(1 - \cos\left(\frac{\alpha}{2}\right) \right) \end{array} \right.$	

2.4 Experimental Studies

The experimental studies of geosynthetic tube include model test and the basic properties test of geosynthetic materials or filling slurry. The model test of geosynthetic tube inflated with dredged materials can be traced back to early 1980's in Brazil and France for land reclamation applications. The geosynthetic sheets used for seaming or gluing geosynthetic tubes must have enough tensile strength to sustain the pressure applied during pumping and stacking periods. For permeable geosynthetic tube, the geosynthetic sheet must also have enough retention properties to block the fine particles inside of tube from dissipating out. Therefore, the basic properties of geosynthetic material for impermeable geomembrane tubes involve only tensile strength, but for permeable geotextile tubes both tensile strength and filtration properties are involved.

2.4.1 Test of geosynthetic materials

(1) Tensile Strength Test

Several test standards were developed for the tensile strength testing of geosynthetic materials. The details of these testing standards for geotextile materials are summarized in Table 2.7 and for impermeable geomembrane (or plastic) materials are in Table 2.8. As geosynthetic specimens tend to contract (neck down) in the central area when stretching, specimen with a width of 20 *cm* and gauge length of 20 *cm* was suggested for tensile strength testing according to ASTM D 4595. The results achieved using this test standard can correlate better with the tensile strength values anticipated in the field (ASTM D4595).

Table 2.7 Test standards for tensile strength of geotextile material

	Name of standard	Specimen size ($W \times L^{(1)}$, <i>mm</i>)	Loading rate
ASTM	D4595-09	200×200	10±3 %/min
	D5034-09	100×75	300±10 mm/min
British Standard	EN ISO 10319: 1996	200×100	20±5 %/min

Note:

⁽¹⁾ The length of specimen, *L*, is the gauge length between clamps

Table 2.8 Test standards for tensile strength of geomembrane material

	Name of standard		Specimen size		Loading rate
			Dumb-bell	Rectangular ($W \times L^{(1)}$, mm)	
ASTM	D638-03		$13^{(2)} \times 57^{(2)}$		10 ± 3 %/min
	D882-02		---	$5 \sim 25.4 \times 50$	$25^{(3)}$ mm/min
British Standard	EN ISO 12311- 2: 2010	Method A:	---	50×200	100 ± 10 mm/min
		Method B	$6^{(4)} \times 115$	---	500 ± 50 mm/min
				15×170	200 ± 20 mm/min

Notes:

⁽¹⁾The length of specimen, L , is the gauge length between clamps;

⁽²⁾The width and length are determined by the thickness of specimen; more details can refer to ASTM D638-03;

⁽³⁾The load rate is determined by percent elongation at break when test for other than elastic modulus;

⁽⁴⁾The width is the smallest width; more details can refer to EN ISO 12311-2: 2010.

During a tensile strength test, the stress and strain relationship of woven materials may include some or all of the following four stages: (1) initial shrinkage stage, (2) elastic stage, (3) plasticity stage and (4) destroying stage, summarized by ASTM D4595 as shown in Figure 2.12. The characteristics of each stage during tensile tests are described as follows,

- (1) Initial shrinkage stage, OA : when pulling stress is not very high at the beginning of the test, the woven ribbons of geosynthetic materials slack, align, and shrink to be closer. This stage is the initial elongation of the specimen. The tangent modulus increases with the increasing of elongation. The corresponding modulus can be calculated as the slope of OA .
- (2) Elastic stage, AC : The woven ribbons of geosynthetic tube are stretched together. There is a linear stress-strain relationship. The tangent modulus of this stage is calculated as the slope of line AC .

- (3) Plasticity stage, *CF*: the woven permeable geosynthetic material exhibits a plastic characteristic as curve *CF* shown in Figure 2.12. The woven ribbons become loose with each other. The tensile forces increase with a slow rate till approach the peak value.
- (4) Destroying stage, *FH*: some of the woven ribbons of the specimen are stretched to be broken. The tensile force at this stage starts to decrease stage by stage. The stress-strain curve is in saw-tooth shaped or even drops suddenly.

Figure 2.12 Typical stress-strain relationship of geotextile material (ASTM, D4595)

(2) *Tensile Strength of Seams*

A common way to form permeable geosynthetic tubes is to sew two sides of a geosynthetic sheet together. The tensile strength of a sewn seam is influenced by several factors such as seam type, number of rows of stitch, seam location, stitch type, stitch count, thread type, sewing equipment and the properties of the geosynthetic sheets being sewn. In this chapter, only the first three main influence factors are discussed as follows,

- (1) Seam type: There are many seam types for seam constructions used depending on which kind of fabric is used or how much stress is placed on the seam. Some of the most commonly used seam types are “Prayer” seam (SSa), “J” seam (SSn) and “Butterfly” seam (SSd) as shown in Table 2.9.

- (2) Number of rows of stitching: The stretching force is acting on the first row of stitching. Therefore, the second row of stitching does not generally increase the strength of the seam significantly, but serves as a safety factor. However, this row of stitching could increase the tear resistance of sewn edges.
- (3) Seam location: The seam relative to the edge of nonwoven geosynthetic sheets should be sewn at least 2-5 cm from the edge which can have enough tear resistance to sustain the large tensile force. The properties of the fabric being sewn also influence the seam location.

Table 2.9 Commonly used federal seam types (after Ko, 1987; Mirafi, 2010)

Seam type	Section view	Front view
SSa ("Prayer" Seam)
SSn ("J" Seam)
SSd ("Butterfly" Seam)

The sewn geosynthetic sheets also have to sustain the pressure applied during pumping and stacking periods. The test standards for determination of joint or seam strength are summarized in Table 2.10. As geosynthetic specimens tend to neck down in the central area especially on sides of seam holes, the seamed geosynthetic

specimen with 20 cm in width, 20 cm in gauge length, and with the provision for the seam to extend for 25 mm on each side was suggested to be used in seam strength testing because this specimen correlated better with the tensile strength values anticipated in the field (ASTM D4884).

Table 2.10 Test standards for seam strength

	Name of standard	Specimen size ($W^{(1)} \times L^{(2)}$, mm)	Loading rate
ASTM	D4884-96-R2003	200×200	10±3 %/min
British Standard	EN ISO 10321 - 2008	200×100	20 ± 5 %/min

Notes:

⁽¹⁾The width of specimen, W , is the effective width, the geometry of seam specimen are shown in Figure 2.13;

⁽²⁾The length of specimen, L , is the gauge length between clamps.

Figure 2.13 Dimension of seam specimen (front view)

After geosynthetic sheets are sewn together, the seam strength is lower than the original unseamed geosynthetic specimen. Therefore, the same strength will reduce or even determine the tensile strength along the whole cross-section of the geosynthetic tube (Fowler, 1996). To qualify the seam strength, the seam efficiency is defined as the percentage ratio of the seam strength to the strength of unseamed geosynthetic sheet (Merkel, 1991; Chowdhary and Poynor, 2006; EN10321-2008).

Therefore, seam efficiency can be calculated by dividing of the seam tensile strength determined by ASTM D4884, over the tensile strength of the unjointed material determined by D4595, or by dividing tensile strength determined by EN10319 over that determined by EN10321. The seam efficiency of most commonly provided woven high-strength geosynthetic materials ranged from 40% to 60% tested by ASTM D4884 as summarized in Table 2.11 and Table 2.12 (Engineering bulletin, 2006; Gughelmetti et al. 1996; TenCateTM, 2010).

Table 2.11 Typical seam efficiency of geosynthetic materials according to ASTM 4884 (after TenCateTM, 2010).

Geotextile type	Seam efficient (%)
Nonwovens	90
Silt-film woven	40-60
Monofilament woven	40-60
High-strength PET woven	40-60

Table 2.12 Typical seam efficiencies obtained by Propex-Geosynthetics Company (after Engineering bulletin, 2006)

Propex Permeable geosynthetic	Seam Type	No. of Rows of Stitching	Seam efficient (%)
Light woven geosynthetic 135ST to 3155ST	Prayer	1	50-70
		2	60-85
Heavy woven geosynthetic 2×2 to 4×4	Prayer	1	40-75
		2	50-80
Nonwovens	Prayer	1	60-90

NOTES:

1. Values are representative of seams using propex permeable geosynthetic sewn in a clean environment with type 401 stitches and a stitch count of 5 to 6 stitches per inch. Actual project seam strengths depend on sewing conditions, materials, and other variables discussed in this Engineering Bulletin;
2. Strength values are reported in percent of propex's specified product strength as measured in accordance with ASTM D4595;
3. Seam strengths are measured in accordance with ASTM D4884.

(3) Filtration Test

There are three primary criteria for the design of geotextile filters: retention, permeability and clogging (Moo-Young and Tucker, 2002b). The retention criteria have been developed by Calhoun (1972), Ogink (1975), Millar et al. (1980) and Carroll (1983) for both woven and nonwoven geotextile materials. The apparent opening size (AOS) of the geotextile material and the grain size distribution of the filling materials will determine the mass of fine seeped out of geosynthetic tube. The smaller the AOS of the geotextile material and the larger the grain size of the filling material, the less particles dissipate out of the geotextile tube. The permeability criteria have also been summarized by Calhoun (1972), Giroud (1982), Carroll (1983), and Christopher and Holtz (1985). The methods of filtration test can be classified into three types according to the way of applying pressure. The 1st method is to test the filtration properties by hanging geotextile bag on a frame (Koerner and Koerner, 2006 a and b) as shown in Figure 2.14(a). The 2nd method is the vacuum filtration test which applying vacuum pressure to suck the excess water seepage out of fixed geotextile specimen as shown in Figure 2.14(b) (Moo-Young and Tucker, 2002b). The 3rd method is applying an air pressure on the top of sludge surface to force the excess water seepage through the fixed geotextile specimen as shown in Figure 2.14(c) (Moo-Young et al. 2002a; Muthukumaran and Ilamparuthi, 2006).

(a) Hanging bag test (Koerner and Koerner, 2006a)

(b) Vacuum filtration test (Horace and Tucker, 2002b)

(c) Air pressed filtration test (Moo-Young et al. 2002a)

Figure 2.14 Filtration test methods

2.4.2 Model Tests of Impermeable Geomembrane Tubes

The model test of impermeable geomembrane tubes includes the measurement of the cross-sectional geometry, the tensile force in geomembrane sheet and the pumping pressure inside. Since geosynthetic sheet is a thin and flexible material, a contact-free measure system has to be used to measure the cross-section of geomembrane tube. A highly accurate low-budget photogrammetric system was proposed for this purpose as shown in Figure 2.15 (Thomas and Cantr 2009; Cantr and Saathof, 2010). The tested geosynthetic tubes were in small-scale (2 m in length and 0.2 m in diameter). The highly accurate low-budget photogrammetric system included four digital cameras to capture approximately half of a tube's circumference during filling and dewatering while resting on a plane table. In their model test, the waterproof strain gauges were used to measure the strain of the geosynthetic sheets.

The experiments for improving the stability of single or stacked geomembrane tubes were also carried out by some researchers. Huong (2001) conducted the model test for wedged single geomembrane tube with different heights of external water blocked on the opposite side. Kim et al. (2004) presented an apron attached large scale geosynthetic tube resting on a soil tank. The soil in the tank was poorly graded sand. The displacement was obtained by using reference beams, a plumb bob and scales. Miki et al. (1996) used a circular loading plate pressed on the top of soil

filled tubes to model the stacked geosynthetic tubes system as shown in Figure 2.16. The tensile force along the geosynthetic tube was tested by the attached strain gauges.

Figure 2.15 Test setup to measure the geometry of geosynthetic tubes using highly accurate low-budget photogrammetric system (after Cantr é and Saathof, 2010)
(1): Geosynthetic tube; (2): Reference coordinates; (3): Plane table; (4): Cameras.

Figure 2.16 Schematic diagram of test (after Miki et al. 1996)

More research studies are required to better understand the performance of the impermeable geosynthetic tubes and establish simple and yet reliable design tools. The cross-section measurement of geosynthetic tube can use the contact-free laser displacement measuring system because the geosynthetic sheet is so thin and flexible that the contact equipment may influence its geometry. In the model tests,

the waterproof strain gauges have to be used to measure the tensile force of the geosynthetic sheets.

2.4.3 Model Tests of Permeable Geotextile Tubes

The model test of permeable geotextile tubes includes the tests of displacement of cross-sectional geometry, tensile force along the geotextile sheets and pumping pressure inside. Usually model tests of permeable geotextile tubes were carried out in-situ because the field test considers the influence of weather and waves which are not easily simulated in laboratory.

The engineering properties of dredged sediments ranged from beach sands to highly organic clays are usually indigenous to the environment from which they are dredged (Moo-Young and Tucker, 2002c). Therefore, for coastal erosion projects, the model test of permeable geotextile tube was usually filled with dredge sand or silty clay materials on situ (Shin et al. 2002; Shin and Oh, 2003; Shin and Oh, 2004; Oh and Shin, 2007). However, the consolidated period will more or less be affected by the properties of the filling materials. Based on the field test conducted by Shin and Oh (2003), the dredge sand filled geotextile tube dropped off about 40% in height within 2 days from the time after being filled. But for silty clay, the tube dropped off 50% within a month after it has been filled. The basic engineering properties of consolidated soil after the model test such as water content and shear strength also have to be determined. The undrained shear strength of consolidated soil was usually tested by the laboratory vane shear test (Shin et al. 2002).

The wave attack threatens the stability of geotextile tube and seepage of solid particles. The risk of destruction due to extraordinary wave conditions is permanent (Alvarez et al. 2007). The laboratory tests of sand filled geotextile tubes resting on rigid foundation confronted with wave were presented by Van Steeg and Vastenburg (2008). The models of geosynthetic tubes were carried out with its scale of approximately 1:4. The waves were varied with the range of 0.0 *m* to 1.5 *m* during the model test. They concluded that the migration of sand in geotextile tubes was a significant factor for the rolling or sliding of the tubes. A model units used in 2-

dimensional stability and laboratory wave transmission tests were conducted by Shin et al. (2002).

For using geotextile tubes to dewatering sewage sludge, the dewatering time and the water content of consolidated soil are the two important dewatering performances. Normal dewatering operations require the dewatering time to be between one and two months (Lawson, 2008). The longer dewatering time constrains the speed and volume of waste that can be treated. Therefore, the acceleration of this dewatering process is necessary. Currently, there are two kinds of accelerate methods: 1) adding chemical dewatering accelerant; and 2) using electro-osmotic potential. The chemical dewatering accelerant can accelerate the dewatering time for the organic waste such as sewage sludge, lagoon solids but has no benefit with fly ash (Worley, et al. 2004, and 2008). The electro-osmotic potential dewatering acceleration method utilizes an electric potential between anode and cathode to accelerate the pore water moving through the fine-grained particles. The later method has been widely used in many kinds of sludge such as sewage sludge (Miller et al. 1998; Jones et al. 2006), waste water sludge (Tyagi, 2006), lagoon sewage (Glendinning et al. 2006) and fine-grained residue from mine tailing (Fourie et al. 2002, 2004).

Normal dewatering operations require the dewatering time to be less than two months (Lawson, 2008). The normal dewatering process by stacking tubes may be too slow. Other accelerating methods like chemical dewatering and electro-osmotic dewatering method have their own limitations. A more efficiency method to improve the dewatering efficiency has to be developed.

2.5 Numerical Approaches

The numerical methods used for simulating the geosynthetic tubes can directly provide the geometry of the cross-section and the tensile force distribution in the geosynthetic tube. Some commercial computer software such as FLAC and ABAQUS can be used for the analysis.

The finite-difference program FLAC (Itasca Consulting Group, 2000) was used to analyze the geosynthetic tube. In most of the existing simulation methods, the geosynthetic tubes were modeled as linearly elastic beam elements (Kim, 2003; Kim, et al. 2005a; Kim, et al. 2005b; Huong, 2001)). The extension, bending effect and weight of geosynthetic sheet were neglected. The following one or two calculation steps has to be conducted: 1) to build the initial equilibrium between gravity, hydraulic pressure distribution and deformation of soil subgrade, 2) to apply the node forces onto the beam element and transfer the stress to the soil subgrade till the whole system balanced. The special shape or assembled geosynthetic tubes, such as attached with an apron (Kim et al. 2004), baffle tube dam, sleeved tube dam and the stacked tube dam (Kim, 2003; Kim, et al. 2005a; Kim, et al. 2005b) and a wedge holding on side (Huong, 2001), were also analyzed with this software.

The computer software ABAQUS-Standard finite element program (Hibbitt, Karlsson, & Sorensen, Inc.) was also used to calculate the geometry of the cross-section and the tensile force in geosynthetic tube. Seay and Plaut (1998) used it to model the geosynthetic tubes three-dimensionally. They modeled the geosynthetic tube in a flat shape at the beginning and then applied an internal hydraulic pressure to check the final shape of cross-section. The relationship between height of geosynthetic tube and pumping pressure obtained from this study was linear which was different with numerical result presented by Malik (2009). Two dimensional analysis of geosynthetic tube resting on rigid foundation was carried out by Cantré (2002). He modeled the initial geometry of stacked geosynthetic tubes with the formulation from Plaut and Suheman (1998) and then calculated with ABAQUS to get the balance shapes. The consolidation process of permeable geosynthetic tube was also modeled in this paper by assuming the modulus and saturation of slurry changed with the dissipation of the pore water pressure.

Both FLAC and ABAQUS have been used to calculate the cross-section and tensile force of geosynthetic tubes. FLAC has been used for 2-D analysis of geosynthetic tube. For 3-D analysis, ABAQUS has been adopted.

2.6 Summary and conclusions

Geosynthetic tubes are formed by sewing or gluing geosynthetic sheets together and then filled with water, clay slurry, or sand. The applications, classification, analytical solutions, experimental methods and numerical analysis of geosynthetic tubes are reviewed in this chapter. The following conclusions can be drawn based on the reviews.

- (1) Several solutions for impermeable geosynthetic tubes resting on rigid foundation have been proposed. However, most of the existing solutions require the running of a computer program. This is not convenient for preliminary design where some trial and error processes or parametric studies are involved in the selection of the dimensions of the geosynthetic tubes. Further study is required to develop a more convenient and effective method for preliminary design.
- (2) The geosynthetic mattress (or mats) is different from geosynthetic tube in that the horizontal dimension is much greater than the vertical one. Due to this difference, the existing analytical or numerical methods developed for the sausage shaped geosynthetic tubes may not be applicable directly to geosynthetic mats.
- (3) There is a lack of suitable analytical methods for the calculation of geosynthetic tubes resting on deformable foundation. The only method available is based on an over simplified Winkler model. The existing Winkler foundation method may not always be applicable directly to geosynthetic mat as the solution may not converge when the ratio between the height and width of the tube is very small.
- (4) The geosynthetic tubes can be stacked in several layers to construct a higher dyke or to dewater waste sludge. Several methods have been proposed to analyze the cross-section and tensile force of the stacked geosynthetic tubes of the same perimeters. Usually, the dimensions of the geosynthetic tube at the

bottom are larger than that on the top in order to form a slope to enhance the stability. Therefore, there is a need to develop a new analytical method for stacked geosynthetic tubes. Furthermore, the existing method cannot be used when geosynthetic mats are involved.

- (5) More research studies are required to better understand the performance of the impermeable geosynthetic tubes and establish simple and yet reliable design tools. The cross-section measurement of geosynthetic tube can use the contact-free laser displacement measuring system because the geosynthetic sheet is so thin and flexible that the contact equipment may influence its geometry. In the model tests, the waterproof strain gauges have to be used to measure the tensile force of the geosynthetic sheets.
- (6) Normal dewatering operations require the dewatering time to be less than two months (Lawson, 2008). The normal dewatering process by stacking tubes may be too slow. Other dewatering accelerating method like chemical dewatering and electro-osmotic method have their own limitations. A more efficiency method to improve the dewatering efficiency has to be developed.
- (7) Both FLAC and ABAQUS have been used to calculate the cross-section and tensile force of geosynthetic tubes. FLAC has been used for 2-D analysis of geosynthetic tube. For 3-D analysis, ABAQUS has been adopted.

CHAPTER 3

MODEL TESTS OF IMPERMEABLE GEOSYNTHETIC TUBES AND MATS ON RIGID FOUNDATION

3.1 Introduction

As reviewed in Chapter 2, impermeable geosynthetic tubes have been widely used for the construction of rubber dams (Ghavanloo and Daneshmand, 2009a and 2009b; Watson et al. 1999 a), water level control (Sehgal, 1996), flood control (Fowler, 1997; Plaut and Suherman, 1998) and water diversion (Tam, 1997) in many countries. However, there are still no established design procedures for the design of geosynthetic tubes for some of the above applications. Further studies are required to better understand the performances of the impermeable geosynthetic tubes and to establish simple and yet reliable design tools.

In this chapter, relatively large-scale model tests were carried out on water inflated impermeable geosynthetic tubes. The tests also provided data to verify numerical methods developed for the analysis of impermeable geosynthetic tubes. Before the model tests, the engineering properties of the geosynthetic sheets were tested. Model tests for 2-layer stacked geosynthetic tubes with one on top of another were also conducted.

3.2 Properties of Geosynthetic Sheets

The geosynthetic material used for forming the tube was 0.6-mm-thick polyvinyl chloride (PVC) membrane. The grids imbedded in the PVC membrane was DP18 nylon plastic tested by the 1100 DTEX standard (as provided by the supplier). The peel strength of the glued seams is 120N/5cm based on DIN53357 (IVK) European Standard.

The geosynthetic tube was also required to sustain the pressure applied during the filling or pumping period. Therefore, it is necessary to test the tensile strength of the membrane used to form the geosynthetic tube. The tested specimens with dimensions of $5 \times 100 \text{ cm}$ ($W \times L$) were prepared by the supplier directly. The two sides of the specimen were fixed into the whirling bars of the clamps as shown in Figure 3.1. The remained stretching length (gauge length) was around 20 cm because the whirling bars cannot exactly fix the values.

(a) Test setup

(b) destroying of the testing specimen

Figure 3.1 Photos of specimen for tensile strength testing

An INSTRON 5569 plastic test machine was used to test their tensile strength. The maximum stretch force of the machine was 50 *kN*. The displacement rate of INSTRON 5569 was in the range of 5 to 500 *mm/min*. The deformation of specimen during the test was recorded by the built-in sensor system. During the test, the pulling speed was controlled at 100 *mm/min* as required by standard BS EN 12311-2:2010.

The tensile tests results of the five specimens are shown in Figure 3.2. It shows that the tensile forces along the specimen increased linearly with respect to the axial deformation before the peak values. The peak tensile forces of these five specimens ranged from 2.0 to 2.8 *kN/5cm*. After the peak tensile forces, the geomembrane material broke suddenly with nearly horizontal fractures as shown in Figure 3.1(b).

Figure 3.2 Tensile test results of PVC membrane

3.3 Geosynthetic Tubes Used for Model Tests

Three geosynthetic tubes made of 0.6-mm-thick PVC membrane materials were used for the model tests with their dimensions shown in Table 3.1. The geometries of the three geosynthetic tubes were 1.0, 1.5 and 2.0 m in width and 2.0, 3.0 and 4.0 m in length, respectively. The length to width ratio of 2.0 was maintained. Then, the perimeters of the cross-sections were 2.0, 3.0 and 4.0 m, respectively.

Table 3.1 Dimensions of geosynthetic tubes

Model test No.	Width (m)	Length (m)
T1	1.0	2.0
T2	1.5	3.0
T3	2.0	4.0

There were three ports preinstalled on each geosynthetic tube. One port was located at one side and used for filling/drainage. Another port was on the top and used for air suction before filling tap water. The third port, also on top, was used for measuring the pumping pressure applied to the geosynthetic tube. The location and function of each of these ports are shown in Figure 3.3.

Figure 3.3 Ports location on the geosynthetic tubes

3.4 Model Test Set-up

3.4.1 Displacement Measurement

The profiles of the geosynthetic tubes after inflated were measured by the ILD1700-750 model laser sensor which had a range of 750 mm and resolution of 50 μm . The laser sensor which can move from left to right by manually control was hanged on the x-beam with an aluminum alloy bar and two pulleys. The x-beam was fixed on a frame as the photos shown in Figure 3.4. When the laser sensor moved horizontally along the x-beam, the desktop computer connecting with the laser sensor recorded the vertical distance between the sensor and laser point on the top surface of the geosynthetic tube. Before each model test, the distance between the concrete floor and laser sensor was measured. Then, the vertical position of a point on the top surface of the tube was calculated by the distance to the floor minus the laser sensor reading. The horizontal position of the testing point was measured by a ruler attached to the beam as shown in Figure 3.4.

Figure 3.4 Model test set-up and displacement measuring system

The laser sensor did not work for the bottom of the tube. To overcome this limitation, an indirect method by using a batten was introduced. The batten was firstly mounted on a cast iron support stands with rods. The test procedure was discussed as follows: (1) put the batten on the side of geosynthetic tubes and make sure the free edge was just contact with the tube; (2) took readings of a point on the batten using the laser sensor; (3) calculate the vertical position of the contacting point by the distance to the floor minus the laser sensor reading; (4) calculate the horizontal position by the reading of the ruler on the x-beam plus (or minus on the big scale side) the distance from the laser point to the free edge of the batten which was measured by an extra smaller ruler as shown in Figure 3.5.

This contact-free measuring system avoided the contact effect to the flexible geosynthetic tube and allowed its profile to be measured accurately. However, as the laser sensor could not move longitudinally, it could only measure one profile of the tube along a center cross-section such as the section A-A' shown in Figure 3.4.

Figure 3.5 Illustration of the measurement method for the points at the bottom of the cross-section

3.4.2 Tensile Force Measurement

(1) Set-up of Strain Gauge

The tensile force along the geosynthetic tube was measured by the strain gauges attached along its cross-section. The *WFLA-6-11-3L* type of strain gauge with the measured range of 2.0% ($20000 \times 1.0E-6$) was used in this test as a sketch shown in Figure 3.6(b). The entire strain gauge and wire connection were covered with transparent flexible epoxy resins. The strain gauge was in dimension of $25 \times 11 \times 1.5$ mm ($H \times W \times T$). The gauge factor of the strain gauge was 2.1.

Figure 3.6 Illustration of tensile force measurement method

Because of the symmetry of the cross-section, the strain gauges were only attached along its half circle as shown in Figure 3.6(a). The intervals between strain gauges

for Models T1 and T3 of 10 *cm*, and Model T2 of 15 *cm* were conducted. Therefore, there were in total 11 strain gauges used for T1 and T2. For Model T3, there were three strain gauges failed to attach to the geosynthetic sheet. Therefore, there were only 18 strain gauges left. All the strain gauges were connected to a data logger with cables as shown in Figure 3.6(c). An extender of the data logger was used to accommodate all the channels as shown in Figure 3.6(d).

It should be pointed out that the strain gauge factors were taken into consideration in the strain gauges readings. The value of this factor was calculated as $2.0/2.1 = 0.952$ which was input into the computer software.

It was not an easy task to stick the strain gauges onto the geosynthetic tube because its surface was flexible and smooth. The following procedure was adopted in this study: (1) polish an area little larger than strain gauges using sand paper; (2) clean the rough surface use acetone till there is no plastic dust or oil left; (3) stick the strain gauge with super glue onto the surface of geosynthetic tube; (4) put a polyethylene film and a glass on the surface of strain gauge; (5) compress the glass with dead weight till the super glue hardened.

(2) Strain Gauge Calibration

In the model tests, the strain gauges were attached onto the geosynthetic tube using super glue. When super glue was hardened, it increased the stiffness of the geosynthetic around the area where super glue was attached and thus affected the strain measurement using the strain gauges. Therefore, calibration tests have to be carried out to calibrate the super glue effect.

To ensure the reliability, the same calibration test was repeated on three specimens. The method for the attachment of strain gauges to the geosynthetic was exactly the same as discussed in last section. The width of tested specimens was 5 *cm*. The two sides of the specimen were fixed into a pair of holding clamps as shown in Figure 3.7. The section in between the clamps or the free stretching section was 30 *cm*. A

small hook was attached to the bottom clamp to allow dead weight to be hung. The dead weight used was 100g, 200g, 500g, 500g, 1kg, 1kg, 1kg, 2kg, 2kg, 2kg with each step last for 4 min. The stretching of the geosynthetic was measured over the 4 min at a time interval of 5 second.

Figure 3.7 Calibration test set-up of the strain gauges attached on PVC membrane

The force versus the strain gauge readings curves obtained from the three specimens are plotted in Figure 3.8. In order to transfer the unit of the calibration test results (from $kN/5cm$ to kN/m), the forces in this figure have been multiplied by 20. It can be seen that the force versus strain gauge relationship is linear. This was expected as the maximum strain experienced by the test specimens was only 0.5% as shown in Figure 3.8. It should be pointed out that, in all the following model tests, the maximum readings of strain gauge did not exceed this value. Therefore, the calibration relationship established by Figure 3.8 can be applied to all the model tests.

Figure 3.8 Calibration of the strain gauges attached on PVC membrane

(3) Bending Effect

During the model tests, the geosynthetic sheet would bend when the geosynthetic tube or mat was inflated. The bending of the inflexible geosynthetic sheet would cause the strain gauge to register some readings even when the geosynthetic is not stretched. A series of calibration tests were carried out to study how the amount of bending affect the strain gauge readings. A geosynthetic specimen with strain gauges installed was placed around a set of plastic cylinders of different diameters of 2.2, 4.6, 10 and 16.5cm, separately. As shown in Figure 3.9, the geosynthetic with strain gauge attached was bent to fit the curvature of the cylinder. No load was applied to the geosynthetic. The strain output was measured. For different curvatures, the outputs were different. The strain gauge readings versus the radius of the bending (or the radius of the cylinder) are plotted in Figure 3.10. It can be seen that the smaller the radius, the higher the strain gauge readings. A power relationship between the strain gauge readings and the radius of the bending can be established in Figure 3.10. When the radius is very large or the surface is flat, the strain gauge readings will be zero as expected. This relationship has been used in this thesis to correct for the bending effect of all raw strain gauge readings in all the model tests. It should be point that bending may not be cylindrical in shape. Nevertheless, the

above relationship provides a way to consider the bending effect on the strain gauge readings.

Figure 3.9 Bending effect test of the strain gauges attached on PVC membrane

Figure 3.10 Bending effect of the strain gauges attached on PVC membrane

3.4.3 Pumping Pressure Measurement

Since the pressures adopted for the model tests were small, the ordinary pore water pressure measurement method using transducers would not have sufficient accuracy. To overcome this problem, the pressure was measured directly by the water head. A

thin piezometer tube was connected to the tube via a port on top of the tube as the photo shown in Figure 3.11. The water head measured by this piezometer was the total head inside the tube. The pumping pressure was calculated by the unit weight of tap water multiply the difference between the total head and the height of geosynthetic tube, $p_0 = \gamma_w(h_T - H)$ where p_0 is pumping pressure; γ_w is unit weight of tap water; h_T is tested total head; H is height of the geosynthetic tube.

Figure 3.11 Photo of water head measurement

3.5 Determination of Reference Cross-section

In this section, attempts were made to determine and define the reference cross-section based on the observation of the model tests. The existence of a reference cross-section can be explained by assuming that the geosynthetic tube is inflated from a completely flat and empty state (no air or water inside). If a piezometer is installed as shown in Figure 3.12, the water level in the geosynthetic tube will be the same as the piezometer level (see Mat 1 and Mat 2 in Figure 3.9). However, this is only true when the geosynthetic tube has not been full inflated yet. When the tube is inflated to a certain height, extra pressure is required to inflate the tube even higher

(see Tube 3 in Figure 3.9). In this case, the piezometer level will be higher than the height of the tube. The difference of the pressure head on the two sides was defined as pumping pressure. The cross-section of the geosynthetic tube at which the piezometer level is about to be higher than the height of the tube is called the reference cross-section. In other words, the reference cross-section is the highest cross-section that can be achieved without a pumping pressure. Here, a pumping pressure is measured as a piezometer head above the top of the tube. It should be noted that, during the filling period, a total head is required to allow the water to flow into the tube. When the flow rate of the filling water was slow, the water flow will not influence the piezometer reading.

To summarize, a reference cross-section is defined as the highest cross-section that the geosynthetic tube can be inflated under zero pumping pressure. When a single geosynthetic tube is not fully inflated or lower than the reference cross-section, the top surface is flat as shown by Mat 1 and Mat 2 in Figure 3.9. We will call this special case the geosynthetic mat in the following discussion. Geometrically, geosynthetic mat is used to refer to a flat, mattress like configuration where its thickness is much smaller than any of its lateral dimension.

The width and height of the reference cross-section observed from these three models, T1 to T3, were shown in Table 3.2. The height to width ratio of reference cross-section was also calculated with their average value of 0.162. From this point of view, the reference cross-section can also be defined as the cross-section of geosynthetic container with height to width ratio of 0.162. The geosynthetic tube with height to width ratio less than 0.162 was defined as geosynthetic mats. Otherwise, it was defined as geosynthetic tube.

Figure 3.12 Schematic drawing of determination of reference cross-section

Table 3.2 Dimensions of reference cross-section from model tests

Tube No.	Width (m)	Height (m)	Height to width ratio	
			Test Results	Average
T1	0.927	0.148	0.160	0.162
T2	1.375	0.225	0.164	
T3	1.807	0.295	0.163	

3.6 Geosynthetic Mat

3.6.1 Test Procedure

Model tests on geosynthetic mats were carried out. During a test, the geosynthetic mat was inflated with tap water through a port shown in Figure 3.13(a). The test procedure adopted was as follows:

- 1) Suck the air and water out of the geosynthetic mat using a vacuum pump (Figure 3.13(b)) through the air suction port (Figure 3.13(a));
- 2) Move the laser sensor to the center of the tested section, such as the center of section A-A' in Figure 3.4, to measure the height of geosynthetic mat;
- 3) Open the tap valve slowly to let the tap water flow into the mats;
- 4) Turn off the tap valve and slightly sway the top caps to let the trapped air flow out after inflated the mat into a small height, such as 0.1 m in most of

the cases. The reason for removing all the air out of geosynthetic mat was that it may influence the final cross-section since its density is much smaller than that of water;

- 5) Turn on the tap valve gently and continue to inflate the mat until approached the design height.

A photo of the plane and front view of the Model T3, when inflated to a height of 0.2 m were shown in Figure 3.14(a) and (b) respectively.

(a)

(b)

Figure 3.13 Photos of (a) filling& dewatering port and (b) vacuum pump

(a) Plane view

(b) Front view

Figure 3.14 Test photos of Model T3 with height of 0.2 m

3.6.2 Cross-sections

The measured cross-sections of the geosynthetic mat in Model T1 when inflated to the height of 0.1 and 0.148 m are presented in Figure 3.15(a). It can be seen that the top lines on the cross-section were not horizontal but a little concaved because of the self-weight of two caps on the top surface as shown in Figure 3.4. The caps also affected the measurements in Model T2 as shown in Figure 3.15(b). However, the influence was no longer there for Model T3 as the caps were relocated to the bottom. Thus, smoother curves were obtained as shown in Figure 3.15(c). Those results

demonstrated the ability of the laser sensor in profiling the cross-section of the geosynthetic mat accurately.

Figure 3.15 The cross-sections of model tests of geosynthetic mats when inflated to different heights

3.6.3 Tensile Forces

The tensile force distributions measured along the geosynthetic mat are presented in this section. The bending effect of strain gauge was considered by the following steps. First, measure the radius of the strain gauge testing location from the tested cross-section which can easily be realized by computer software such as AutoCAD, (Autodesk, 2007). Second, calculate the corresponding value of bending effect using the measured radius and the equation from Figure 3.10. Third, subtract the reading of strain gauge by calculated bending value. Then the tensile force per unit length

can be calculated using the equation from the calibration strain gauge as discussed in Figure 3.8.

The model test results of tensile forces along the half cross-section of geosynthetic mats in model T1 are shown in Figure 3.16. It can be seen that the tensile force on the right corner up ground surface had the largest value, but that along the top horizontal section were almost uniform except the third strain gauge from left. The sharp values of the tensile force may due to the influence from the initial positions. The tensile forces along the contact edge with ground were smaller.

(a) Height of 0.1 m

(b) Height of 0.15 m

Figure 3.16 Test results of tensile forces along Model T1 (kN/m)

The model test results of the tensile forces along the half cross-section of Models T2 and T3, are also plotted, in the same way as Model T1, in Figure 3.17 and Figure 3.18, respectively. The tensile forces along the contact surface with ground surface reduced from bottom corner to the center. The little larger values locating on the right corner up ground surface were due to the influence from the bending of strain gauges. In these figures, the negative value of the tensile force means the geosynthetic material had a compression effect due to the influence from the initial position. These negative values were treated as bad point when drawing these figures.

In order to verify the accuracy of measured tensile forces, the comparisons between the measured tensile forces along mid-section with the hydraulic forces were illustrated in Table 3.3. The tensile forces on top and bottom points were horizontal which will be illustrated in section 4.5. The hydraulic forces along the mid-section of geosynthetic tubes were calculated by $0.5 \cdot \gamma \cdot H^2$ where γ is unit weight of filling tap water and H is height of geosynthetic tube. The details of this kind of free-body diagram method will be fully illustrated in section 4.5. As shown in Table 3.3, for model T1, the measured tensile forces along mid-section has very much difference with hydraulic force. For T2 and T3, the maximum difference was -60% where negative value means the measured tensile forces were smaller than the hydraulic forces.

Table 3.3 Comparison of the measured tensile forces along mid-section with hydraulic forces

Model test No.	Height (m)	Hydraulic Force (kN)	Model test (kN)			Difference (%)
			Top point	Bottom point	Total	
T1	0.1	0.05	0.115	0.129	0.244	388.0
	0.15	0.112	0.148	0.134	0.282	150.7
T2	0.1	0.05	0.066	0.002	0.068	36.0
	0.15	0.112	0.11	0.025	0.135	20.0
	0.225	0.253	0.143	0.12	0.263	3.9
T3	0.1	0.05	0.069	0.008	0.077	54.0
	0.2	0.2	0.086	0.009	0.095	-52.5
	0.295	0.435	0.163	0.011	0.174	-60.0

(c) height of 0.225 m

Figure 3.17 Model test results of tensile forces along Model T2 (kN/m)

(a) Height of 0.1 m

(b) Height of 0.2 m

(c) Height of 0.295 m

Figure 3.18 Model test results of tensile forces along Model T3 (kN/m)

3.7 Geosynthetic Tube

The performance of geosynthetic tubes under a pumping pressure was also studied. Due to the influence of the pumping pressure, the height of the tube and the tensile force induced along the geosynthetic tube were much higher than those of geosynthetic mat in which the pumping pressure was zero.

3.7.1 Test Procedure

The filling procedure of geosynthetic tube was the same as that of the geosynthetic mat discussed in Section 3.6.1 except that the inflated height was much larger. To avoid the explosion of the test model, the geosynthetic tubes for model T1 were inflated to a maximum height of 0.525 m, for Model T2 of 0.746 m and for Model T3 of 0.876 m.

A set of test photos from Model T3 are shown in Figure 3.19. The tested pumping pressures, denoted as p_0 , are also shown in their titles. It shows that when pumping pressure was small, such as 0.03 kPa in Figure 3.19(a), the Model T3 was only inflated to a relatively flat shape with a height of 0.42 m. However, with respect to

the increasing of pumping pressure, the cross-sections were in a sausage shape as shown in Figure 3.19(b) and (c).

(a) $H = 0.42 \text{ m}$, $p_0 = 0.03 \text{ kPa}$

(b) $H = 0.62 \text{ m}$, $p_0 = 0.49 \text{ kPa}$

(c) $H = 0.876 \text{ m}$, $p_0 = 2.91 \text{ kPa}$

Figure 3.19 Photos of filling process from Model T3

3.7.2 Cross-sections

The cross-sections of the Model T1 inflated with different pumping pressures are shown in Figure 3.20. It shows that the height and area of the cross-section increased with respect to the pumping pressures. The cross-sections were inflated into 77% of its maximum height ($L/\pi = 0.637 \text{ m}$) when pumping pressure was 4.3 kPa . The weight of black caps discussed in Section in 3.6.2 seemed to have no influence on the top curves of the cross-sections. This is because the pumping pressure acting on the inner top surface was greater than zero and could support the weight of black caps. The Model T1 was inflated till the pumping pressure of 6.86 kPa and height of 0.525 m as shown in Figure 3.20.

Figure 3.20 Tested cross-sections of Model T1

The measured cross-sections of Model T2 and T3, are plotted in Figure 3.21 and Figure 3.22, respectively. The relationships of cross-sections and pumping pressures are the same as those obtained from Model T1. The Model T2 was inflated till a pumping pressure of 5.74 kPa and height of 0.746 m. The Model T3 was inflated with a pumping pressure of 3.21 kPa and height of 0.876 m.

Figure 3.21 Tested cross-sections of Model T2

Figure 3.22 Tested cross-sections of Model T3

The model test results of relationships between pumping pressure, p_0 , and height of cross-sections, H , are plotted in Figure 3.23(a). It can be seen that, for each test, the height of the geosynthetic tube increases nonlinearly with respect to the pumping pressure. In these model tests, the change in the height was sensitive to a small change in pumping pressure particularly when the pressure was below 2 kPa. However, when pumping pressures were greater than 2 kPa, the change of height of geosynthetic tube became less visible.

(a) Pumping pressure vs. height curve

(b) Pumping pressure vs. width curve

Figure 3.23 Experimental study for pumping pressure

The width of cross-sections versus pumping pressures curves from these model tests are plotted in Figure 3.23(b). In contrast to the height changes, the width of cross-section nonlinearly decreased with respect to pumping pressure. This is easy to understand since the increase of height caused the cross-sections to approach the circle shape and the width decreasing. Similar to the height versus pumping pressure curve, the widths decreased sensitively when pumping pressure was under 2 kPa but slowly when that exceeded 2 kPa.

In order to compare the results of model tests on tubes of different perimeters, the height and width of geosynthetic tubes were normalized by dividing them over their perimeters, L . The pumping pressure was normalized by dividing that over $\gamma_w L$ where $\gamma_w = 9.81 \text{ kN/m}^3$ is the unit weight of water. The curves of normalized pumping pressure versus normalized height are shown in Figure 3.24. It can be seen that all the test data fall onto the same line indicating a unique relationship. Figure 3.24 shows that the change in height is sensitive to the change in pumping pressure particularly when the pumping pressure was small. The normalized pumping pressure versus normalized width curves are shown in Figure 3.25. The change in the width is also sensitive to the normalized pumping pressure particularly when $p_0/\gamma_w L$ is less than 0.1.

Figure 3.24 Normalized pumping pressure versus normalized height

Figure 3.25 Normalized pumping pressure versus normalized width

3.7.3 Tensile Forces

The model test results of tensile force along geosynthetic tubes were calculated using the same method as that for geosynthetic mats in section 3.6.3. The tensile forces distributions along the filled cross-sections of Model T1, T2 and T3 are shown in Figure 3.26, Figure 3.27 and Figure 3.28, respectively. The tensile forces along the whole cross-section of geosynthetic tubes were more uniform than those in geosynthetic mats especially that on the top surface. The higher the pumping pressure, the higher and more uniform the tensile force. The tensile forces along the contact surface with ground surface reduced from bottom corner to the center. The tensile force on the foot side up ground surface had the larger values because of the influence from the bending of strain gauge on this corner.

Figure 3.26 Tensile force test results of Model T1 (unit kN/m)

Figure 3.27 Tensile force test results of Model T2 (unit kN/m)

Figure 3.28 Tensile force test results of Model T3 (unit kN/m)

In order to verify the accuracy of measured tensile forces, the comparisons between the measured tensile forces along mid-section of T1 with the hydraulic forces were illustrated in Table 3.4. The tensile forces on top and bottom points were horizontal which will be illustrated in section 4.3. The hydraulic forces along the mid-section of geosynthetic tubes were calculated by $p_0 \cdot H + 0.5 \cdot \gamma \cdot H^2$ where γ is unit weight of filling tap water, p_0 is pumping pressure and H is height of geosynthetic tube. The details of this kind of free-body diagram method will be fully illustrated in section 4.3. As shown in Table 3.4, the maximum difference between tensile forces along mid-section and acting hydraulic pressure was -36.12% where the negative value means the measured tensile forces was greater than the hydraulic forces. However, when geosynthetic tube was inflated to higher cross-section ($p_0=6.86$ kPa in Table 3.4), the measured tensile forces was 28.85% higher than the hydraulic forces. The comparison of the measured tensile forces along mid-section of geosynthetic tube with hydraulic force of model T2 and T3 are listed in Table 3.5 and Table 3.6. The maximum difference between measured tensile forces and hydraulic forces were 48.56% and 81.67 for model T2 and T3, respectively.

Table 3.4 Comparison of the measured tensile forces along mid-section with hydraulic forces for Model T1

p_0 (kPa)	Height (m)	Hydraulic Force (kN)	Model test (kN)			Difference (%)
			Top point	Bottom point	Total	
0.12	0.262	0.375	0.37	0.14	0.51	-36.12
0.41	0.323	0.654	0.55	0.15	0.7	-7.02
0.706	0.357	0.889	0.69	0.15	0.84	5.54
1.16	0.407	1.300	0.91	0.16	1.07	17.72
4.3	0.491	3.317	1.84	1.25	3.09	6.84
6.86	0.527	5.004	2.27	1.29	3.56	28.85

Table 3.5 Comparison of the measured tensile forces along mid-section with hydraulic forces for Model T2

p_0 (kPa)	Height (m)	Hydraulic Force (kN)	Model test (kN)			Difference (%)
			Top point	Bottom point	Total	
0.1	0.366	0.706	0.45	0.06	0.51	27.80
0.31	0.449	1.147	0.66	0.06	0.72	37.24
0.54	0.495	1.492	0.82	0.07	0.89	40.37
1.12	0.566	2.236	1.07	0.08	1.15	48.56
3.51	0.691	4.813	2.07	0.97	3.04	36.84
5.74	0.748	7.091	2.86	2.37	5.23	26.24

Table 3.6 Comparison of the measured tensile forces along mid-section with hydraulic forces for Model T3

p_0 (kPa)	Height (m)	Hydraulic Force (kN)	Model test (kN)			Difference (%)
			Top point	Bottom point	Total	
0.03	0.419	0.890	0.191	0.013	0.204	77.09
0.08	0.465	1.118	0.191	0.014	0.205	81.67
0.2	0.567	1.721	0.455	0.027	0.482	71.99
0.5	0.625	2.266	0.45	0.03	0.48	78.81
1	0.714	3.263	0.61	0.05	0.66	79.77
3.21	0.877	6.661	1.24	0.23	1.47	77.93

3.8 Stacked Geosynthetic Tubes or Geosynthetic Mats

When a relatively high dike is to be constructed, geosynthetic tubes have to be stacked in multiple layers. Model tests of two layers stacked impermeable geosynthetic tubes were carried out in this section. There were totally three model tests conducted. The dimensions of each model are listed in Table 3.7.

Table 3.7 Tube dimensions for the stacked geosynthetic tube model tests

Model test No.	Tube on top $W \times L$ (m)	Tube at bottom $W \times L$ (m)	Height of bottom tube before filled (m)	Total height (m)
ST1-1	1.0 $\times$ 2.0	1.5 $\times$ 3.0	0.225	0.3
ST1-2				0.4
ST1-3				0.5
ST1-4				0.6
ST1-5			0.366	0.5
ST1-6				0.6
ST1-7				0.7
ST1-8				0.8
ST1-9			0.449	0.6
ST1-10				0.7
ST1-11				0.8
ST1-12			0.566	0.7
ST1-13				0.8
ST1-14				0.9
ST1-15			0.691	0.8
ST1-16				0.9

Table continued

<i>Model test No.</i>	<i>Tube on top W × L (m)</i>	<i>Tube at bottom W × L (m)</i>	<i>Height of bottom tube before filled (m)</i>	<i>Total height (m)</i>
ST2-1	1.5 × 3.0	2.0 × 4.0	0.200	0.3
ST2-2				0.4
ST2-3				0.5
ST2-4				0.6
ST2-5				0.7
ST2-6			0.295	0.5
ST2-7				0.6
ST2-8				0.7
ST2-9				0.8
ST2-10				0.9
ST2-11			0.465	0.6
ST2-12				0.7
ST2-13				0.8
ST2-14			0.465	0.9
ST2-15				1.0
ST2-16			0.625	0.7
ST2-17				0.8
ST2-18				0.9
ST2-19				1.0
ST2-20			0.714	0.8
ST2-21				0.9
ST2-22				1.0
ST3-1	1.0 × 2.0	2.0 × 4.0	0.295	0.4
ST3-2				0.5
ST3-3				0.6
ST3-4				0.65
ST3-5			0.465	0.5
ST3-6				0.6
ST3-7				0.7
ST3-8				0.8
ST3-9			0.625	0.7
ST3-10				0.8
ST3-11				0.8
ST3-12				1.0
ST3-13			0.714	0.8
ST3-14				0.9
ST3-15				1.0

3.8.1 Test Procedure

The laboratory test set-up, measurement methods of tensile forces and pumping water pressure for the model tests of stacked geosynthetic tubes were the same as those used for single geosynthetic tube as discussed in Section 3.4. The photos of Model Test ST3-1 to ST3-4, when the bottom tube was inflated to the height of 0.295 m and total heights to the range from 0.4 to 0.65 m , are shown in Figure 3.29. The details of the test procedure are summarized as follows:

- (1) lay the two tubes on the concrete floor with their central axis in the same line, such as the white adhesive taps as shown in Figure 3.29(a);
- (2) suck the air out of the two tubes with vacuum pump;
- (3) inflate the bottom tube with tap water to a designed height of 0.295 m as shown in Figure 3.29(b);
- (4) place and inflate the top tube till the total height became 0.4 m (Model Test ST3-1) as shown in Figure 3.29(c);
- (5) measure the cross-section, tensile forces of the Model Test ST3-1;
- (6) repeat step 4 and 5 to inflate the top tube till the total height reached 0.5 (Model Test ST3-2), 0.6 (Model Test ST3-3), and 0.65 m (Model Test ST3-4) as shown in Figure 3.29(d), (e) and (f);
- (7) drain off the water out of the top and the bottom tube after test.

(a) Photos of tubes after air suction

(b) Inflate the bottom tube to 0.295 m

(c) Total height of 0.4 m (Test ST3-1)

(d) Total height of 0.5 m (Test ST3-2)

(e) Total height of 0.6 m (Test ST3-3)

(f) Total height of 0.65 m (Test ST3-4)

Figure 3.29 Filling process of 2-layer stacked geosynthetic tubes

3.8.2 Model Test Results

(1) Observation of contact surface between two tubes

Based on the observations of the contact shape between the top and the bottom tubes from the model tests, three different types of contact configurations can be identified. The first one is the heavy top tube sink into the bottom tube as shown in Figure 3.30(a) or Figure 3.28. The main characteristic of this type is that the two sides of bottom tube are squeezed out. The contact surface of the bottom tube is concave and thus this case is named as the concave case. The second one is when the weight of top tube is smaller than the upwards pressure from the bottom tube as shown in Figure 3.30(c). In this case, the contact surface between the two tubes is convex and

this case is termed as the convex case. The third is when the internal water pressure of top tube acting down on the contact equals to the water pressure of the bottom tube acting up. The contact surface in this case should be horizontal by force equilibrium conditions. One example is shown in Figure 3.30(e). This case can be called horizontal contact case. A special case for each of the three cases is when the top tube is flat, or when the top tube is a geosynthetic mat as shown in Figure 3.30(b), (d) and (f).

In conclusion, there are totally three cases for 2-layer stacked tubes in terms of contact surface between the two tubes: the concave, convex, and horizontal contact. The three cases can be distinguished by knowing the pressures inside both tubes.

(a) Concave contact with tube on top

(b) Concave contact with mat on top

(c) Convex contact with tube on top

(d) Convex contact with mat on top

(e) Horizontal contact with tube on top

(f) Horizontal contact with mat on top

Figure 3.30 Experimental observation of different contact shapes

A sketch of two stacked tube and with measured total head in each tube are shown in Figure 3.31. The tested total head in each tube was independent of the testing points. The pressure head of top tube acting down at point B , h_T , can be calculated as the difference between the total head, H_{w1} , and the elevation head, H_B . Similarly, the pressure head of bottom tube acting up on point B , h_B , can be calculated as $(H_{w2} - H_B)$. The difference between the two water pressures acting up and down on the contact surface equals to the difference between the total head in these two tubes as derived in Equation (3-1).

$$h_T - h_B = (H_{w1} - H_2) - (H_{w2} - H_2) = H_{w1} - H_{w2} \quad (3-1)$$

Based on the discussed above, the concave case can be distinguished as $h_T > h_B$ or $H_{w1} > H_{w2}$. Similarly, the convex contact is distinguished as $h_T < h_B$ or $H_{w1} < H_{w2}$, and horizontal contact as $h_T = h_B$ or $H_{w1} = H_{w2}$.

Figure 3.31 Sketch of the tested total head

The total head in the top and bottom tubes of all the model tests are listed in Table 3.8. Based on the discussion above, we can determine the type of profile for the contact surface based on the total heads measured for the top and bottom tubes. As it was difficult to observe the pressures in both the top and bottom tubes to be exactly the same, a horizontal surface was assumed when the difference between the two heads divided by their average value was less than 1% ($((h_T - h_B)/((h_T + h_B)/2)) < 1\%$). One such an example is Model Test ST2-1. The total heads and the types of contact surface determined are given in Table 3.8 for all the model tests.

Table 3.8 Categories of all the model tests

Model test No.	H_{w1} (m)	H_{w2} (m)	Type of contact surface
ST1-1	0.300	0.290	Concave
ST1-2	0.400	0.350	
ST1-3	0.521	0.398	
ST1-4	0.756	0.438	
ST1-5	0.500	0.490	
ST1-6	0.611	0.554	
ST1-7	0.769	0.607	
ST1-8	0.879	0.681	
ST1-9	0.600	0.622	
ST1-10	0.769	0.688	
ST1-11	0.898	0.740	

Table Continued

<i>Model test No.</i>	<i>Hw1 (m)</i>	<i>Hw2 (m)</i>	<i>Type of contact surface</i>
<i>ST1-12</i>	<i>0.700</i>	<i>0.822</i>	<i>Convex</i>
<i>ST1-13</i>	<i>0.813</i>	<i>0.896</i>	
<i>ST1-14</i>	<i>0.986</i>	<i>0.966</i>	<i>Concave</i>
<i>ST1-15</i>	<i>0.800</i>	<i>1.192</i>	<i>Convex</i>
<i>ST1-16</i>	<i>0.938</i>	<i>1.279</i>	
<i>ST2-1</i>	<i>0.300</i>	<i>0.298</i>	<i>Horizontal</i>
<i>ST2-2</i>	<i>0.400</i>	<i>0.385</i>	<i>Concave</i>
<i>ST2-3</i>	<i>0.500</i>	<i>0.464</i>	
<i>ST2-4</i>	<i>0.615</i>	<i>0.557</i>	
<i>ST2-5</i>	<i>0.781</i>	<i>0.622</i>	
<i>ST2-6</i>	<i>0.500</i>	<i>0.490</i>	
<i>ST2-7</i>	<i>0.600</i>	<i>0.560</i>	
<i>ST2-8</i>	<i>0.715</i>	<i>0.635</i>	
<i>ST2-9</i>	<i>0.868</i>	<i>0.690</i>	
<i>ST2-10</i>	<i>1.155</i>	<i>0.737</i>	
<i>ST2-11</i>	<i>0.600</i>	<i>0.615</i>	<i>Convex</i>
<i>ST2-12</i>	<i>0.701</i>	<i>0.695</i>	<i>Horizontal</i>
<i>ST2-13</i>	<i>0.815</i>	<i>0.769</i>	<i>Concave</i>
<i>ST2-14</i>	<i>0.952</i>	<i>0.835</i>	
<i>ST2-15</i>	<i>1.143</i>	<i>0.895</i>	
<i>ST2-16</i>	<i>0.700</i>	<i>0.780</i>	<i>Convex</i>
<i>ST2-17</i>	<i>0.800</i>	<i>0.885</i>	
<i>ST2-18</i>	<i>0.909</i>	<i>0.971</i>	
<i>ST2-19</i>	<i>1.036</i>	<i>1.050</i>	
<i>ST2-20</i>	<i>0.800</i>	<i>0.980</i>	
<i>ST2-21</i>	<i>0.904</i>	<i>1.074</i>	
<i>ST2-22</i>	<i>1.029</i>	<i>1.158</i>	
<i>ST3-1</i>	<i>0.400</i>	<i>0.352</i>	<i>Concave</i>
<i>ST3-2</i>	<i>0.506</i>	<i>0.428</i>	
<i>ST3-3</i>	<i>0.738</i>	<i>0.453</i>	
<i>ST3-4</i>	<i>1.081</i>	<i>0.497</i>	
<i>ST3-5</i>	<i>0.500</i>	<i>0.498</i>	<i>Horizontal</i>
<i>ST3-6</i>	<i>0.600</i>	<i>0.551</i>	<i>Concave</i>
<i>ST3-7</i>	<i>0.719</i>	<i>0.578</i>	
<i>ST3-8</i>	<i>0.969</i>	<i>0.614</i>	
<i>ST3-9</i>	<i>0.700</i>	<i>0.731</i>	<i>Convex</i>
<i>ST3-10</i>	<i>0.806</i>	<i>0.777</i>	<i>Concave</i>
<i>ST3-11</i>	<i>0.950</i>	<i>0.816</i>	
<i>ST3-12</i>	<i>1.283</i>	<i>0.844</i>	
<i>ST3-13</i>	<i>0.800</i>	<i>0.887</i>	<i>Convex</i>
<i>ST3-14</i>	<i>0.899</i>	<i>0.934</i>	
<i>ST3-15</i>	<i>1.060</i>	<i>0.978</i>	<i>Concave</i>

There are too many model test results that it is difficult to discuss them one by one. Only several typical examples are presented in this section. More test results are given in Appendix A.

(2) Concave Case

The results of Model Test ST2-9 are shown in Figure 3.32. During this model test, the bottom tube was inflated firstly to a height of 0.295 m. Then the top tube was placed and inflated till the total height became 0.8 m. The total head in the top tube was 86.8 cm and that in the bottom tube was 69.0 cm. Thus the pressure in the top tube was greater than that acting up in the bottom tube, i.e. $86.8 > 69.0$ cm. Therefore, the contact surface between two the tubes should be concave down as the dash line shown in Figure 3.32(a).

It should be pointed out only the positions that could be directly reached by the laser sensor could be measured, as indicated by the open circles in Figure 3.32(a). The position of the contact interface between the two tubes could not be measured. The dashed line is only indicated based on the judgment on the shape of the curve made above.

(a) Cross-section

(b) Tensile force along the top and bottom tubes (kN/m)

Figure 3.32 Model test results of the Test ST2-9

The tensile force along the top mat of the tube is presented in Figure 3.32 (b) which was calculated with the same method as that for geosynthetic mats in section 3.6.3. The distribution was not uniform. The maximum value was located on the outside of the contact edge as marked as point A in Figure 3.32 (b). This maximum value was related to the maximum bending of the strain gauge at this location. These larger values may be because the influences of bending of strain gauges were not totally counteracted. Similar observation was made on the tensile forces along the bottom tube. It can be seen from Figure 3.32 (b) that largest tensile force occurred at the bottom corners marked as point B. This was due to the bending of the strain gauges too large. The tensile forces along the contact surface with ground surface reduced from bottom corner to the center

(a) Cross-section

(b) Tensile force along the top mat and the bottom tube (kN/m)

Figure 3.33 Model test results of the Test ST2-2

The test results of Model Test ST2-2 are given in Figure 3.33. The profile of this model test was different from that of Model Test ST2-9 in the cross-section of top tube. During this model test, the bottom tube was inflated firstly to a height of 0.2 m. Then the top tube was placed and inflated till the height became 0.4 m. The total head in the top tube was 39.4 cm which makes the pumping pressure in top tube was almost zero (i.e. $10 \times (0.4 - 0.394) / 100 = 0.0006 \text{ kPa}$). The top tube is therefore in flat shape as shown in Figure 3.33(a) which can also be called geosynthetic mat according to the terminology used in this study. The total head in the bottom tube was 38.5 cm. The pressure acting down on the contact surface was greater than that acting up (i.e. $39.4 \text{ cm} > 38.5 \text{ cm}$). Therefore, the contact surface between mat and tube should be in concave shape as shown in Figure 3.33(a).

The tensile forces along the top geosynthetic mat and the bottom tube are plotted in Figure 3.33(b). The tensile force along the top mat was almost uniform but with the largest value located on the outside of contact edge (point A) which may because the bending of strain gauge has not totally been eliminated. The tensile forces along the bottom tube were also uniform except those on the bottom corners were slightly larger (point B). The tensile forces along the contact surface with ground surface reduced from bottom corner to the center.

(3) Convex Case

Test results of the Model Test ST1-16 are shown in Figure 3.34. During this model test, the bottom tube was inflated firstly to a height of 0.691 m. Then the top tube was placed and inflated till the total height reached to 0.9 m. The tested total head in the top tube was 93.8 cm and that in the bottom tube was 127.9 cm. Thus the pressure in the top tube was smaller than that in the bottom tube, i.e. $93.8 < 127.9$ cm. Therefore, the contact surface between two the tubes should be convex as shown in Figure 3.34(a).

The tensile forces along the two geosynthetic tubes are presented in Figure 3.34(b). The tensile forces of the top tube along the contact surface with the bottom tube are slightly larger. The tensile forces along the contact surface with ground surface reduced from bottom corner to the center. The tensile forces at the bottom corners (Point B) were slightly larger because the influence from the bending of strain gauges.

(a) Cross-section

(b) Tensile forces along the top and bottom tubes (kN/m)

Figure 3.34 Model test results of the Test ST1-16

The test results of Model Test ST2-20 are given in Figure 3.35. During this model test, the bottom tube was inflated firstly to a height of 0.714 m. Then the top tube was placed and inflated till the height became 0.8 m. The total head in the top tube was 80.1 cm. The pumping pressure in top tube is 10 Pa, ($10 \times (80.1 - 80) / 100 \times 1000 = 10$ Pa). The top tube is then in flat shape as shown in Figure 3.35(a) which can also be called geosynthetic mat according to the terminology used in this study. The total head in the bottom tube was 98 cm. Therefore, the pressure head acting down on the contact surface was smaller than that acting up, i.e. 80.1 cm < 98 cm. The contact surface between mat and tube is convex as shown in Figure 3.35(a). The tensile forces along the mat and tubes are presented in Figure 3.35(b) which distributed similarly to those concluded from concave contact case.

(a) Cross-section

(b) Tensile forces along the top mat and the bottom tube (unit kN/m)

Figure 3.35 Model test results of the Test ST2-20

(4) Horizontal contact

The model test ST2-12 was carried out by inflating the bottom tube firstly to a height of 0.465 m . Then the top tube was placed and inflated till the total height was 0.7 m . The total head in the bottom tube was 69.5 cm and that in the top tube was 70.1 cm . The difference between these two total head was 0.6 cm ($70.1 - 69.5\text{ cm}$) which means the Model Test ST2-12 almost approached the horizontal contact as the dash line shown in Figure 3.36 (a). The tensile forces along the two tubes are presented in Figure 3.36(b). The distributions of tensile force along the two tubes were the same as those concluded from convex contact case.

(a) Cross-section

(b) Tensile forces along the top and bottom tubes (kN/m)

Figure 3.36 Model test results of the Test ST2-12

The test results of Model Test ST3-5 are shown in Figure 3.37. In this test, the bottom tube was inflated firstly to a height of 0.465 m . Then the top tube was placed and inflated till the total height become 0.5 m . The total head in the top tube was 0.499 m and that in the bottom tube was 0.498 m . The water head in the top tube was zero, i.e. $0.5-0.499=0.01\text{ m}$. Then the top tube is in flat shape as shown in Figure 3.37(a) which can also be called geosynthetic mat according to the terminology used in this study. The total head in the top tube was almost same as that in the bottom tube, i.e. $0.499-0.498=0.001\text{ m}$. Therefore, the contact surface was almost horizontal as shown in Figure 3.37(a). The tensile forces along the geosynthetic mat and tube were almost uniform as shown in Figure 3.37(b).

(a) Cross-section

(b) Tensile forces along the top mat and the bottom tube (kN/m)

Figure 3.37 Model test results of the Test ST3-5

3.9 Conclusions

A series of model tests for single and 2-layer stacked impermeable geosynthetic tubes inflated by water were carried out. The cross-sections and the tensile forces along the geosynthetic tubes were measured. Based on the results, the following conclusions can be drawn:

- (1) Based on the model tests on impermeable geosynthetic tubes, a reference cross-section is defined as the highest cross-section that the geosynthetic tube could be inflated without a pumping pressure. When the height of a

geosynthetic tube is lower than the reference cross-section, the top surface will be flat and this special configuration is called geosynthetic mat in this study.

- (2) The height of the geosynthetic tubes in the model tests varied with even a small change in pumping pressure particularly when the pressure was very small.
- (3) The tensile force of geosynthetic mat or tube at the two bottom corners just about to turn off from the ground had the largest value which possible due to the influence from the bending of strain gauges. The tensile forces along the contact surface with ground surface reduced from bottom corner to the center. Generally, for geosynthetic tubes, the larger the pumping pressure the more uniform the tensile forces.
- (4) For the 2-layer stacked impermeable geosynthetic tubes, the contact surface between the top and the bottom tubes can be convex, concave, or horizontal. The shape of the contact surface is controlled by the total head in the two tubes.
- (5) The tensile forces along the top geosynthetic tube were almost uniform. The slight larger value occurs at the two ends of the contact surface between the two tubes due to the influence from the bending of strain gauges. The tensile forces along their contact surface reduced from corner to the center.

CHAPTER 4

ANALYSIS OF GEOSYNTHETIC TUBE AND MAT ON RIGID FOUNDATION

4.1 Introduction

As reviewed in Chapter 2, several analytical solutions for geosynthetic tubes resting on rigid foundations have been proposed. However, most of the existing solutions require the running of a computer program. This is not convenient for preliminary design in which some trial and error processes or parametric studies are involved in the selection of the dimensions of the geosynthetic tubes or the types of geotextiles materials.

A geosynthetic mat is different from a geosynthetic tube as its horizontal dimension is much greater than the vertical one which makes it more stable than the sausage shaped geosynthetic tube. The existing analytical or numerical methods developed for the sausage shaped geosynthetic tube may not be always applicable directly to geosynthetic mat as the solution may not converge when the ratio between the height and width of the tube is very small as is typically the case for geosynthetic mats.

In this chapter, an analytical solution was proposed to calculate the required cross-sections and tensile forces of geosynthetic tubes. The proposed analytical solutions were also verified by comparing with the existing solutions and model test results. The analytical solution was also extended to geosynthetic mats. The solutions developed in this chapter are only applicable to the impermeable geosynthetic

tubes/mats or permeable geotextile tube/mat right at the time when filling is completed.

4.2 Basic Assumptions

In order to derive the solutions, the following assumptions were made:

- (1) The geosynthetic tube & mat is sufficiently long to be assumed as a plane strain problem;
- (2) The geosynthetic shell is thin and flexible so that its weight and extension can be neglected;
- (3) Frictions between the geosynthetic tube and the fill material, or that between the geosynthetic tube and the rigid foundation are neglected;
- (4) The tensile force along the geosynthetic sheet is constant;
- (5) All the geosynthetic tubes are inflated with the same material and no external water pressure is applied.

Some of the above assumptions were also adapted in the existing analytical solutions for geosynthetic tubes (Szyszkowski and Glockner, 1987; Kazimierowicz, 1994; Leshchinsky et al., 1996; Plaut and Suherman, 1998; Cantré 2002; Malik and Sysala, 2010; Cantré and Saathoff, 2010).

4.3 Analytical Solutions

4.3.1 Theoretical Derivations

As the condition is symmetric, only half of the simplified cross-section of a geosynthetic tube is needed as shown in Figure 4.3. The geosynthetic tube is assumed to be inflated with only one type of slurry material with a unit weight of γ . The coordinates are set up with x in the vertical direction and y in the horizontal direction. The origin of the coordinates is taken as the top point of the cross-section. The width of the cross-section is written as B . The height of the geosynthetic tube is denoted as H . The contact width with ground surface is presented as b . The tensile force along the geosynthetic tube per unit length is denoted as T .

An infinite small curve with a length of ds around point $S(x, y)$ can be treated as an arc with the center at random point C and radius of r as shown in Figure 4.1 (b). Then two geometrical equations relating the angle θ and the x and y coordinates are given in Equations (4-1) and (4-2). An infinite small curve with a length of ds around point $O(0,0)$ can be treat as an arc with the center at point $O(0,0)$ and radius of r_0 shown in Figure 4.1 (a). Due to symmetric of the arc along x-axis, the tangent line on point $O(0,0)$ is horizontal which is just the direction of tensile forces on point $O(0,0)$.

$$y' / \sqrt{1 + y'^2} = \sin \theta \quad (4-1)$$

$$y' = dy/dx = \tan \theta \quad (4-2)$$

$$\frac{dT}{ds} = 0 \quad (4-3)$$

$$\frac{d\theta}{ds} = \frac{1}{r} = \frac{1}{T} [P_0 + \gamma x] \quad (4-4)$$

Figure 4.1 The free body diagram of infinite unit

A free body diagram of a section from point O to a point $S(x,y)$ on the cross-section is used for force equilibrium analysis as shown in Figure 4.2(a). The angle between the tangent direction at point $S(x,y)$ and the x axis is denoted as θ . The hydraulic pressure acting internally on this point $S(x,y)$ is $p_0 + \gamma x$ where p_0 is the pumping pressure. The forces acting along the horizontal direction only involve the hydraulic

pressure and the tensile forces as shown in Figure 4.2(b). The forces equilibrium along horizontal direction yields,

$$T - T \sin \theta = p_0 x + \frac{\gamma x^2}{2} \quad (4-5)$$

(a) The free body diagram of curve OS

(b) Horizontal forces on curve OS

Figure 4.2 Free body diagram of curve OS

(a) Free body diagram

(b) Horizontal forces

Figure 4.3 Free body diagram of half cross-section

The free body diagram of this half cross-section is plotted in Figure 4.3(a). The forces acting on horizontal direction only involve the hydraulic pressure and the

tensile force as shown in Figure 4.3(b). The force equilibrium between these two forces yields the expression of tensile force as shown in Equation (4-6). It can be seen that the tensile force along the geosynthetic tube is related to the height of the cross-section, the pumping pressure and the unit weight of filling slurry.

$$T = (p_0 H + \frac{1}{2} \gamma H^2) / 2 \quad (4-6)$$

Deriving from Equations (4-5) obtains the expression of $\sin \theta$ shown as follows,

$$\sin \theta = 1 - (p_0 x + \frac{1}{2} \gamma x^2) / T \quad (4-7)$$

Deriving from Equation (4-7), the solution of the x -coordinate can be obtained as follows (as $x > 0$, the negative result is omitted),

$$x = \frac{1}{\gamma} \left[-p_0 + \sqrt{p_0^2 + 2\gamma T(1 - \sin \theta)} \right] \quad (4-8)$$

Combining Equations (4-1), (4-2) and (4-8) derives,

$$\frac{dy}{d\theta} = - \frac{T \sin \theta}{\sqrt{p_0^2 + 2\gamma T(1 - \sin \theta)}} \quad (4-9)$$

Integrating y in Equation (4-9) with respect to θ and the boundary condition, $\theta = 0$, $y' = 0$, yields the expression of y -coordinate of the geosynthetic tube as:

$$y = -\sqrt{\frac{T}{2\gamma}} \int (\sqrt{Q - \sin \theta} - \frac{Q}{\sqrt{Q - \sin \theta}}) d\theta \quad (4-10)$$

$$Q = \frac{1}{2\gamma T} (p_0^2 + 2\gamma T) \quad (4-10a)$$

where Q is denoted as the factor of pumping pressure.

Knowing the unit weight of the filling slurry, γ , the pumping pressure, p_0 , the height, H , or the perimeter, L , of cross-section, and combining with boundary conditions $x = 0$, $\theta = \pi/2$ and $x = H$, $\theta = -\pi/2$, the cross-section and tensile force can be calculated.

4.3.2 Calculation Procedure

Since Equation (4-10) includes the first and second elliptic integrals which have no closed-form solutions, a computer program was written using the adaptive Runge-Kutta-Merson method (RKM4) to solve the Equations (4-6), (4-7), (4-8) and (4-10).

The adaptive RKM4 method is a widely accepted method for the approximation of solutions of ordinary differential equations which will not be covered in this thesis. The standard programs for the RKM4 method had already been proposed by Press et al. (2007), Xu (1995), Christiansen (1970) and Lukehart (1963).

The calculation was carried out by taking the same inputs such as, the unit weight of the slurry, γ , the pumping pressure, p_0 , the height, H , or the perimeter, L . The unknown parameters were searched by the trial-and-error method which solves the problems by making a change if objectives do not happen, and then trying again. The procedure using trial-and-error method was outlined as follows which was similar with Leshchinsky et al. (1996),

- (1) Input the initial parameters: γ, p_0, H ;
- (2) Calculate the tensile force, T , using Equation (4-6) and the factor of pumping pressure, Q , using Equation (4-10a);
- (3) Calculate $\sin\theta$ using Equation (4-7),
- (4) Solve Equations (4-8) and (4-10) using RKM4 method;
- (5) If the perimeter of the geosynthetic tube, L , is taken as an input rather than the height, H , the iteration can be done as follows: assume $H_t = L/\pi$ as the input height to calculate the trial length, L_t . If $L_t \neq L$, then modify H_t and repeat the calculation until the difference between L_t and L is less than $1\text{E-}6$.

4.4 Comparisons

4.4.1 With Existing Solutions

To verify the reliability of the computer program, its results are compared with the existing solutions. The comparison between the results from the present method and those proposed by Leshchinsky et al. (1996) are shown in Table 4.1. The calculation was carried out by taking the same inputs such as $L = 9 \text{ m}$ and $\gamma = 12 \text{ kN/m}^3$. The percentage differences between these two sets of results were calculated by their differences dividing over the Leshchinsky's results. The maximum percentage difference between these two sets of results was 5% for the case with pumping pressure of 6.9 kPa .

The results of the present computer program were also compared with the results proposed by Kazimierowicz (1994), Silvester (1986) and Liu (1981) as shown in Table 4.2, Table 4.3 and Table 4.4, respectively. The input parameters and the results are listed in these tables.

Table 4.1 Comparison of present method with the results proposed by Leshchinsky et al. (1996), (For $L = 9 \text{ m}$, $\gamma = 12 \text{ kN/m}^3$), (after Guo et al. 2011)

p_0 (kPa)	Source	$H \text{ (m)}$		$B \text{ (m)}$		$A \text{ (m}^2\text{)}$		$T \text{ (kN/m)}$	
		Values	Diff. %	Values	Diff. %	Values	Diff. %	Values	Diff. %
0	Leshchinsky	0.90	0.00	4.09	0.24	3.36	3.87	2.50	4.00
	Present	0.90		4.08		3.23		2.40	
4.8	Leshchinsky	1.80	0.56	3.60	0.83	5.56	3.78	14.60	4.79
	Present	1.79		3.57		5.35		13.90	
6.9	Leshchinsky	2.00	5.00	3.64	4.12	5.76	3.47	18.10	2.76
	Present	1.90		3.49		5.56		17.60	
34.5	Leshchinsky	2.50	4.00	3.21	2.49	6.45	3.10	61.70	3.08
	Present	2.40		3.13		6.25		59.80	
52.4	Leshchinsky	2.60	3.85	3.13	2.24	6.51	2.76	87.50	1.71
	Present	2.50		3.06		6.33		86.00	
103.5	Leshchinsky	2.70	0.00	3.00	0.67	6.57	2.59	162.00	1.36
	Present	2.70		2.98		6.40		159.80	
122.8	Leshchinsky	2.70	0.00	2.96	0.00	6.57	2.44	189.70	1.11
	Present	2.70		2.96		6.41		187.60	
593.4	Leshchinsky	2.90	3.45	2.96	2.36	6.66	3.30	875.00	1.41
	Present	2.80		2.89		6.44		862.70	

Table 4.2 Comparison of present method with the results proposed by Kazimierowicz, (1994), (for $L = 3.6 \text{ m}$, $\gamma = 14 \text{ kN/m}^3$), (after Guo et al. 2011)

p_0 (kPa)	Source	H (m)		b (m)		T (kN/m)	
		Values	Diff. %	Values	Diff. %	Values	Diff. %
17.5	Kazimierowicz	1.00	1.50	0.46	1.09	11.80	0.85
	Present	0.99		0.46		11.90	
10.4	Kazimierowicz	0.90	0.33	0.64	1.25	6.80	0.49
	Present	0.90		0.65		6.77	
4.6	Kazimierowicz	0.80	1.75	0.84	2.14	4.00	2.83
	Present	0.81		0.82		4.11	
3.0	Kazimierowicz	0.70	8.29	0.96	3.65	2.70	15.48
	Present	0.76		0.89		3.03	

Table 4.3 Comparison of present method with the results proposed by Silvester (1986), (for $L = 3.6 \text{ m}$, $\gamma = 19.6 \text{ kN/m}^3$)

p_0 (kPa)	Source	H (m)		B (m)		b (m)		A (m ²)		T (kN/m)	
		Value	Diff. %	Value	Diff. %	Value	Diff. %	Value	Diff. %	Value	Diff. %
24.89	Silvester	1.00	1.50	1.27	1.89	0.46	0.43	1.05	3.81	17.41	2.30
	Present	0.99		1.25		0.46		1.01		17.01	
12.54	Silvester	0.90	0.33	1.32	1.44	0.65	0.77	0.99	2.02	10.05	3.78
	Present	0.90		1.30		0.65		0.97		9.67	
6.47	Silvester	0.80	1.25	1.39	2.16	0.82	0.00	0.95	2.95	5.76	2.08
	Present	0.81		1.36		0.82		0.92		5.88	
4.31	Silvester	0.70	8.57	1.45	3.45	0.94	1.06	0.89	2.25	4.15	7.23
	Present	0.76		1.40		0.93		0.87		4.45	
1.96	Silvester	0.60	8.33	1.50	2.27	0.94	14.89	0.81	0.25	2.51	8.76
	Present	0.65		1.47		1.08		0.81		2.73	
1.76	Silvester	0.50	28.00	1.55	5.16	0.94	17.02	0.74	6.76	2.01	24.38
	Present	0.64		1.47		1.10		0.79		2.50	

Table 4.4 Comparison of present method with the model test results presented by Liu (1981)($\gamma=10 \text{ kN/m}^3$)

$p_0 \text{ (kPa)}$	$L \text{ (m)}$	Source	$H \text{ (m)}$		$B \text{ (m)}$		$b \text{ (m)}$		$d \text{ (m)}$	
			Values	Diff. %	Values	Diff. %	Values	Diff. %	Values	Diff. %
1.56	0.93	Liu Present	0.23	0.43	0.34	0.88	0.18	3.17	0.09	5.56
			0.231		0.337		0.186		0.10	
0.16	0.93	Liu Present	0.16	4.38	0.34	14.12	0.31	2.58	0.05	4.00
			0.153		0.388		0.318		0.048	
1.04	1.04	Liu ⁽¹⁾ Present	0.24	0.83	0.41	4.63	0.25	1.60	0.09	1.11
			0.238		0.391		0.246		0.089	

Note: (1) The tube was filled with mortar and submerged in water. However, in the present calculation, the unit weight of filling slurry was still use 10 kN/m^3 because of internal and external hydraulic pressure difference.

4.4.2 With Model test Results

(1) Cross-section

The solutions of the proposed method are also compared with the model tests results presented in Section 3.7.2. During the analytical calculation, the unit weight of tap water was assumed as 9.81 kN/m^3 . The perimeters of the three models as shown in Table 3.1 were taken as inputs. The tested pumping pressures, p_0 , from model tests were also taken as input.

The cross-sections from the analytical method and model tests T1, T2 and T3 are compared and shown in Figure 4.4, Figure 4.5 and Figure 4.6, respectively. It can be seen that the cross-sections calculated from analytical method were little higher than those from model tests because the water heads were measured by piezometer tube. This measuring method has some limitations such as the verticality of piezometer tube and manmade reading errors. The method then makes the pumping pressure not accurate because it was calculated by the difference between tube height and total water head. However, generally, the cross-sections from analytical methods agreed well with those from model tests.

Figure 4.4 Comparison of the cross-sections between experimental and analytical results for Model T1

(a)

(b)

Figure 4.5 Comparison of the cross-sections between experimental and analytical results for Model T2

(a)

(b)

Figure 4.6 Comparison of the cross-sections between experimental and analytical results for Model T3

The heights of geosynthetic tubes versus pumping pressures curves from the analytical method are compared with those from the model tests as shown in Figure 4.7(a). It can be seen that the model test results were almost the same as those from

analytical method. The increase in the height of the cross-section is sensitive to the pumping pressure when it is less than 5 kPa but less sensitive when the pumping pressure is greater than 10 kPa. The comparisons of width versus the pumping pressures curves between model tests results and analytical methods are plotted in Figure 4.7(b). The decrease in the widths of the cross-sections is also sensitive to the pumping pressure when it is less than 5 kPa, but less sensitive when it is greater than 10 kPa.

(a) Relationship between H and p_0

(b) Relationship between B and p_0

Figure 4.7 Analytical and experimental studies of the effects of pumping pressure

(a) Relationship between normalized pumping pressure and height

(b) Relationship between normalized pumping pressure and width

Figure 4.8 Analytical and experimental studies of the effects of normalized pumping pressure

In order to compare the results of different model tests carried out with different tubes perimeters, the three model tests data, the height, H , width, B , and the pumping pressure, p_0 , were normalized with the same method as discussed in Section 3.7.2. The analytical solution was carried out by taking the perimeter of the cross-section, L , of 1 m and the unit weight of the filling slurry of 1.0 kN/m^3 .

A comparison between the analytical and model test results is shown in Figure 4.8. The normalized pumping pressures were ranged from 0.0 to 1.0. It shows that all

the test data form a single relationship and agree well with analytical results. The normalized height increases at a quicker rate at the beginning when the normalized pumping pressure is less than 0.2. On the other hand, the width of cross-section decreases with increasing in pumping pressure. The width decreases faster when the normalized pumping pressure was small.

(2) Tensile Forces

The tensile forces obtained from Model T1, T2 and T3 are compared with those from analytical solutions as shown in Figure 4.9, Figure 4.10 and Figure 4.11, respectively. The height of cross-section and the values of tensile forces from analytical method are also presented. The values of tensile forces from modeling tests of T1, T2 and T3 were already presented in Figure 3.26, 3.27 and 3.28, respectively.

The comparisons between the analytical and model test results indicate that the tensile forces measured at the bottom corner (such as point A in Figure 4.9(a)) from the model tests are slightly higher than those from analytical solutions due to the influence from bending of strain gauges. The tensile forces along the contact surface with ground (such as section B and C in Figure 4.9(a)) obtained from model tests are smaller than those from analytical solutions because of the influence of frictions between geosynthetic sheets and concrete floor.

Figure 4.9 Comparison of the tensile forces between experimental and analytical results for Model T1

Figure 4.10 Comparison of the tensile forces between experimental and analytical results for Model T2

Figure 4.11 Comparison of the tensile forces between experimental and analytical results for Model T3

4.5 Solution for $p_0 = 0$ Case

4.5.1 Theoretical Derivations

As discussed above, Equation (4-10) includes the first and second elliptic integrals and has no closed-form solutions. However, when the pumping pressure equals to zero or $p_0 = 0$, which is typically the case of geosynthetic mat, Equation (4-10) can be solved analytically. The theoretical derivations of this special case are discussed in this section.

Deriving from Equation (4-6) with $p_0 = 0$ gives the tensile force along the circle of geosynthetic mat shown as follows,

$$T = \frac{1}{4} \gamma H^2 \quad (4-11)$$

Deriving from Equation (4-10c) with $p_0 = 0$, and $Q = 1.0$ gives the differential equation for geosynthetic mat shown as follows,

$$y = -\sqrt{\frac{T}{2\gamma}} \int (\sqrt{1 - \sin \theta} - \frac{1}{\sqrt{1 - \sin \theta}}) d\theta \quad (4-12)$$

In order to eliminate the parameter θ in Equation (4-12), combining Equation (4-7) and (4-11) gives the expression of $\sin \theta$,

$$\sin \theta = 1 - \frac{2x^2}{H^2} \quad (4-13)$$

Equation (4-12) has closed-form solution and could easily be solved by combining Equations (4-13). After integrate, we have:

$$\begin{cases} x = 0, & y = 0 \\ x \neq 0, & y = H \left[\sqrt{1 - \frac{x^2}{H^2}} - \frac{1}{2} \ln \left(\frac{H}{x} + \sqrt{\frac{H^2}{x^2} - 1} \right) + C \right] \end{cases} \quad (4-14)$$

In order to solve Equation (4-14), one more boundary condition is required because the constant C has to be solved. For geosynthetic mat designing, the height and width are usually given as inputs. Then the connections between these inputs and differential equations can be established. The width of the cross-section, B , equals to two times of maximum y coordinate, y_{max} , ie, $B = 2y_{max}$. When $y = y_{max}$, $\theta = 0$. Applying $\theta = 0$ to Equation (4-13) leads to $x = H / \sqrt{2}$. By submitting $x = H / \sqrt{2}$ to Equation (4-14), the width of the geosynthetic mat can be derived as:

$$B = H((\sqrt{2} - \ln(\sqrt{2} + 1)) + 2C) \quad (4-15)$$

Defining the height to width ratio as k and $k = H/B$, the constant C is derived from Equation (4-15) as,

$$C = \frac{1}{2} \left[\frac{1}{k} - (\sqrt{2} - \ln(\sqrt{2} + 1)) \right] \quad (4-16)$$

Substituting Equation (4-16) into Equation (4-14), the analytic equation for a geosynthetic mat resting on the rigid foundation is obtained as:

$$\begin{cases} x = 0, & y = 0 \\ x \neq 0, & y = H \left[\sqrt{1 - \frac{x^2}{H^2}} - \frac{1}{2} \ln \left(\frac{H}{x} + \sqrt{\frac{H^2}{x^2} - 1} \right) + \frac{1}{2} \left(\frac{1}{k} - \sqrt{2} + \ln(\sqrt{2} + 1) \right) \right] \end{cases} \quad (4-17)$$

Using Equation (4-17), all the basic geometry parameters such as the area, A , the perimeter of cross-section, L , and the contact width with ground, b , are easily to be calculated. When $x = H$, the contact width with the basement, b , equals to two times of the corresponding y coordinate as shown in Equation (4-18). The perimeter and area of cross-section, L and A , were derived using integral methods as shown in Equations (4-19) and (4-20).

$$b = 2y_{x=H} = H\left(\frac{1}{k} - (\sqrt{2} - \ln(\sqrt{2} + 1))\right) \quad (4-18)$$

$$A = 2\int_0^H y dx = H^2\left(\frac{1}{k} - (\sqrt{2} - \ln(\sqrt{2} + 1))\right) \quad (4-19)$$

$$L = 2\int_0^H \sqrt{1 + y'^2} dx + b = 2H\left(\frac{1}{k} + 1 - (\sqrt{2} - \ln(\sqrt{2} + 1))\right) \quad (4-20)$$

Knowing the unit weight of the filling slurry, γ , and the height of cross-section, H , the tensile forces along the geotextile tube can be calculated with Equation (4-11). After knowing width, B , the cross-section and geometry parameters can be determined by the solutions given in Equations (4-17) to (4-20).

4.5.2 Parametric Studies

Using the closed-form equations, the parametric studies of geosynthetic mat resting on rigid foundation can be conducted to assess the influence of some control parameters.

Figure 4.12 A group of cross-sections of a geosynthetic mat filled to different heights (For $L=9.0m$, $\gamma=12kN/m^3$, $H=0.2, 0.4$ and $0.682 m$, respectively)

The cross-sections of a geosynthetic mat inflated to different heights are analyzed as shown in Figure 4.12. The calculation was carried out by taking the same inputs such as $L = 9 m$ and $\gamma = 12 kN/m^3$. The heights of geosynthetic mats at $0.2, 0.4$ and $0.682 m$ were also taken as inputs. The corresponding height to width ratio, k , was calculated using Equation (4-20) with its value of $0.045, 0.092$ and 0.163 ,

respectively. Then the cross-section of these three cases can be calculated using Equation (4-17) as shown in Figure 4.12. It can be seen from Figure 4.12 that the higher the cross-section, the larger the area and the smaller the width.

For a given design height, the cross-section will take different perimeters when the widths are different. The cross-sections of three different geosynthetic mats with the same design height of 1.0 m are shown in Figure 4.13. In the calculation, the unit weight of slurry was taken as 12 kN/m^3 . The width of 10.0, 8.0 and 6.13 m are taken as inputs. The corresponding height to width ratios, k , calculated for the three tubes are 0.1, 0.125 and 0.163, respectively. The calculated perimeters of these three mats using Equation (4-20) are 20.9, 16.9 and 13.2 m, respectively. The cross-section with k of 0.163 has the smallest perimeter. The difference between the perimeters of these three cross-sections (i.e. $20.9 - 16.9 = 4 \text{ m}$) equals to two times the difference between their widths (i.e. $10.0 - 8.0 = 2.0 \text{ m}$). This conclusion can also be verified by Equation (4-21) which was derived from Equation (4-20).

$$L_1 - L_2 = 2(B_1 - B_2) \quad (4-21)$$

Figure 4.13 Profile of the cross-section of geosynthetic mat with height to width ratio, k (For $\gamma = 12 \text{ kN/m}^3$, $H = 1 \text{ m}$, $B = 10 \text{ m}$, 8 m and 6.13 m respectively)

4.5.3 A Case Study

The closed-form solution is adopted to analyze a case history presented by Fowler et al. (1996). An in situ model test was carried out by using the permeable geosynthetic tubes for dewatering the fine grained sewage sludge. They calculated

the consolidation process of the tested model with computer program GeoCoPS which was coded by Leshchinsky et al. (1996).

Calculations using the proposed closed-form solution were carried out by taking the same inputs such as the height and perimeter of geosynthetic mat, the unit weight of filling sludge which is listed in Table 4.5. The calculated geometry parameters such as the area of the geosynthetic mat, A , and the tensile force, T , are listed in Table 4.5. It can be seen that the percentage errors of area or width calculated from these two methods are less than 1.25%.

Table 4.5 Application of closed-form solution for a real case proposed by Fowler et al. (1996)

H (m)	γ (kN/m ³)	Source	A (m ²)		B (m)	
			Values	Diff.(%)	Values	Diff.(%)
0.82296	10.546	Closed-form Fowler	3.085	0.63	4.188	0.28
			3.066		4.176	
0.54864	10.791	Closed-form Fowler	2.207	-1.01	4.316	-0.3
			2.23		4.328	
0.4572	10.987	Closed-form Fowler	1.881	1.25	4.359	0.01
			1.858		4.359	

The comparison between the cross-sections of the geosynthetic mat obtained from the closed-form solution and those from Fowler et al. (1996) are shown in Figure 4.14. The cross-sections were picked proportionally based on the figures presented by Fowler et al. (1996). It can be seen that very good agreements have been achieved for these two methods.

(a) Geometry of cross-section for $H=0.823$ m

(b) Geometry of cross-section for $H=0.549\text{ m}$ (c) Geometry of cross-section for $H=0.457\text{ m}$

Figure 4.14 Application of close-form solution for a real case proposed by Fowler et al. (1996)

4.5.4 Comparisons with the Model Test Results

(1) Cross-Sections

The cross-sections obtained from the closed-form solution and from the model tests are compared as shown in Figure 4.15. The perimeters of the three models, as listed in Table 3.1, were taken as inputs in the calculation. The height to width ratio, k , is calculated by Equation (4-22) which was derived by substituting the height and perimeter of geosynthetic mat into Equation (4-20).

$$\frac{1}{k} = \frac{L}{2H} - 1 + (\sqrt{2} - \ln(\sqrt{2} + 1)) \quad (4-22)$$

As shown in Figure 4.15 (a), good agreements between the analytical and model test results are achieved. The cross-sections obtained from Model T1 and T2 are

slightly concave because of the caps on the top of the mat as already discussed in section 3.6.2.

(a) Cross-sections of Model T1

(b) Cross-sections of Model T2

(c) Cross-sections of Model T3

Figure 4.15 Comparisons of the cross-sections between the closed-form and model test results

The width of geosynthetic mats, B , obtained from model tests is also compared with those from closed-form solutions as shown in Figure 4.16. When the perimeter of cross-section is known as inputs, there is a linear relationship between height, H , and width, B , which can also be derived from Equation (4-15). The results obtained from the model tests almost located on the lines from closed-form solutions. Similarly, the contact width with ground, b , and the height, H , relations are linear as shown in Figure 4.17 which can also be derived from Equation (4-18).

Figure 4.16 Comparisons of widths between closed-form solutions and laboratory test results

Figure 4.17 Comparison of contact widths with ground between closed-form solutions and laboratory test results

(2) Tensile Force

In order to verify the accuracy of the proposed solutions, the tensile forces along Model T1 obtained from the closed-form solution are compared with those from model tests presented in Section 3.7.3. As shown in Figure 4.18(a), the tensile forces from closed-form solution were constant and much smaller than that from model tests. The tensile forces along model T2 and T3 obtained from closed-form solution and model tests are also compared as shown in Figure 4.19 and Figure 4.20. The same conclusions as Model T1 can also be drawn.

Figure 4.18 Comparison of the tensile forces between experimental and close-form results for Model T1

Figure 4.19 Comparison of the tensile forces between experimental and close-form results for Model T2

(a) $H = 0.1 \text{ m}$

(b) $H = 0.2 \text{ m}$

Figure 4.20 Comparison of the tensile forces between experimental and close-form results for Model T3

As the comparison discussed above, the data from model tests are quite scatter. The relationships between height of geosynthetic mats and tensile forces from all the model test data were compared with those from close-form results. As shown by Equation (4-11) or in graph by the solid line in Figure 4.21, the tensile forces and the height of geosynthetic mat relation is a second order polynomial. It can be seen that the tensile forces from close-form solutions almost range on two sides of the model tests results.

Figure 4.21 Relationship between the heights and the tensile forces

4.5.5 Calculation of Reference Cross-section

The reference cross-section has been defined in Chapter 3 as the highest cross-section that a certain geosynthetic tube can be inflated with zero pumping pressure based on experimental observation.

As discussed in the parametric study in Figure 4.12, for a given designing height, different widths induced different perimeters of cross-section. The geosynthetic mat with the reference cross-section has the smallest perimeter. The difference between the perimeters of cross-sections equals to two times of that between the widths as shown in Equation (4-21). Therefore, the main difference between the reference cross-section and the other cross-sections was only a horizontal section on top which can be treated as a horizontal stretch of the reference cross-section. For this purpose, it may be possible to choose a point that marks the boundary between the idealized horizontal section and the curved section where one of such a point is shown as point *C* in Figure 4.22.

Figure 4.22 Cross-section of geosynthetic mat resting on rigid foundation

Mathematically, the y coordinate of point *C* can be calculated as the limit value when x/H approaches zero using Equation (4-14). However, when x/H approaches zero, the limit of $-0.5\ln(2H/x)$ is $-\infty$ which makes it impossible to get the limit value, y_0 . The relationship between x/H and $-0.5\ln(2H/x)$ are plotted in Figure 4.23. From Figure 4.23, it appears that $x/H = 0.001$ represents a turning point. The value of $x/H = 0.001$ can be treated as the limit minimum value of x . Furthermore, $x = 0.001H$ is a small value that it is accurate enough in engineering applications. Substituting this

value to Equation (4-17) gives the limit value of y -coordinate, y_0 , as shown in Equation (4-23).

$$y_0 = H\left[\frac{1}{2k} - 3.0669\right] \quad (4-23)$$

For the reference cross-section, the point C locates on the original point (point A in Figure 4.22) or $y_C = 0$ because the reference cross-section had the smallest perimeter. By substituting $y_C = 0$ into Equation (4-23), the corresponding height to width ratio, k , was calculated as $k_0 = 0.163$. This value was also obtained from model test shown in Table 3.2. From this point of view, the reference cross-section can also be defined as the cross-section of geosynthetic tube with height to width ratio of 0.163.

Figure 4.23 Relationship between x/H and $-0.5\ln(2H/x)$

The width of reference cross-section can be calculated as $B_R = H/0.163$. A subscript “ R ” is used to refer to the values associated with the reference cross-section. Thus, b_R , L_R , A_R are corresponding to the contact width with ground, the perimeter and area of the reference cross-section. The value of b_R , L_R , A_R can be calculated from Equations (4-18), (4-19) and (4-20) by substituting the height to width ratio, $k_0 = 0.163$.

$$b_R = 5.602H \quad (4-24)$$

$$A_R = 5.602H^2 \quad (4-25)$$

$$L_R = 13.204H \quad (4-26)$$

4.6 Solution for Case $p_0 \neq 0$

4.6.1 Approximate Solutions

As mentioned in the section 4.3.1, the Equation (4-10) contains the first and second elliptic intervals and thus has no closed-form solutions. The equation has to be solved by numerical method. However, if some approximations can be made, it is possible to derive a closed-form solution. In this section, the following approximation is adopted to solve this problem.

$$\begin{aligned} \sqrt{Q - \sin \theta} &= \sqrt{Q-1} + \frac{1}{\sqrt{Q-1} + \sqrt{Q - \sin \theta}} (1 - \sin \theta) \\ &\approx \sqrt{Q-1} + \frac{1}{\sqrt{Q-1} + \sqrt{Q}} (1 - \sin \theta) \\ &= \sqrt{Q} - (\sqrt{Q} - \sqrt{Q-1}) \sin \theta \end{aligned} \quad (4-27)$$

Substituting Equation (4-25) into (4-10) and integrating with the boundary conditions of $x = 0$, $y = 0$ and $\theta = \pi/2$, the final shape equation of the cross-section is written as follows,

$$\begin{aligned} y &= \sqrt{\frac{T}{2\gamma}} (\sqrt{Q}(\theta - \frac{\pi}{2}) + (\sqrt{Q} - \sqrt{Q-1}) \cos \theta + \\ &\quad \frac{2Q}{\sqrt{1-Q+2\sqrt{Q(Q-1)}}} \tan^{-1}(\sqrt{\frac{2\sqrt{Q}-\sqrt{Q-1}}{\sqrt{Q-1}}} \tan(\frac{\pi}{4} - \frac{\theta}{2}))) \end{aligned} \quad (4-28)$$

Based on Equation (4-28), the basic geometry parameters like the width, B , the perimeter, L , and the area, A , of cross-section can be calculated. The width of the cross-section B equals to two times of the maximum y value. When $y = y_{max}$, $y' = 0$, $\theta = 0$. Submitting $\theta = 0$ to Equation (4-28) gives the width of tube as,

$$\begin{aligned}
 B &= \sqrt{\frac{2T}{\gamma}} \left[\left(1 - \frac{\pi}{2}\right) \sqrt{Q} - \sqrt{Q-1} + \frac{2Q}{\sqrt{1-Q+2\sqrt{Q(Q-1)}}} \tan^{-1} \left(\sqrt{\frac{2\sqrt{Q}-\sqrt{Q-1}}{\sqrt{Q-1}}} \right) \right] \\
 &= C_B \sqrt{\frac{2T}{\gamma}}
 \end{aligned} \tag{4-29}$$

where C_B is denoted as factor of width.

The contact width with ground b is the y -coordinate when $x = H$. When $x = H$, $\sin\theta = -1$ and $\theta = -\pi/2$. Submitting these values to Equation (4-28), the contact width of the geosynthetic tube is calculated as,

$$\begin{aligned}
 b &= \sqrt{\frac{2T}{\gamma}} \left(-\pi\sqrt{Q} + \frac{\pi Q}{\sqrt{1-Q+2\sqrt{Q(Q-1)}}} \right) \\
 &= C_b \sqrt{\frac{2T}{\gamma}}
 \end{aligned} \tag{4-30}$$

where C_b is denoted as the factor of contact width with foundation.

The perimeter of cross-section, L , is calculated as follows,

$$\begin{aligned}
 L &= 2 \int_{\frac{\pi}{2}}^{-\frac{\pi}{2}} \sqrt{\left(\frac{dx}{d\theta}\right)^2 + \left(\frac{dy}{d\theta}\right)^2} d\theta + b = \sqrt{\frac{2T}{\gamma}} \left(-\pi\sqrt{Q} + \frac{\pi(Q+1)}{\sqrt{1-Q+2\sqrt{Q(Q-1)}}} \right) \\
 &= C_L \sqrt{\frac{2T}{\gamma}}
 \end{aligned} \tag{4-31}$$

where C_L is denoted as factor of perimeter.

Based on Figure 4.3, the forces equilibrium on the contact edge with geosynthetic tube and the rigid foundation gives $\gamma A = N$ and $N = (P_0 + \gamma H) b$. Then the area of cross-section, A , is calculated as follows,

$$A = \frac{P_0 + \gamma H}{\gamma} b = C_b \frac{P_0 + \gamma H}{\gamma} \sqrt{\frac{2T}{\gamma}} \tag{4-32}$$

4.6.2 Charts Methods

Although Equations (4-29) to (4-32) are approximate, they define explicitly the relationships between the geometry parameters of the tube and the factor of pumping pressure Q . If some simple relationships or design charts between geometry parameters, b , L , B , A and the factor of pumping pressure, Q , could be established, Equations (4-29) to (4-32) can be used as an approximate solution for preliminary design. For this purpose, some parametric studies were carried out using the computer program proposed in Section 4.3 to establish the required relationships or design charts. The use of finite element analysis or other software may only be required for more sophisticated designs.

The design charts between the geometry factors, C_B , C_b , C_L and the factor of pumping pressure Q have been calculated and plotted in Figure 4.24. If the value of Q is between these intervals, linear difference formula can be used. The factor of pumping pressure was only calculated in the range of 1.0 to 10.0. It should be pointed out that the values of C_L , C_B or C_b when $Q = 1.0$ were not calculated by the computer program, but by the geometry parameters of reference cross-section calculated in section 4.5.5.

(a) Relationship between Q and C_L

(b) Relationship between Q and C_B (c) Relationship between Q and C_b

Figure 4.24 Designing charts method

When using the approximate method, the inputs parameters are the pumping pressure, p_0 , unit weight of filling slurry, γ , and height of geosynthetic tube, H . The details of calculation procedure are shown as follows:

- 1) Calculate the tensile force, T , using Equation (4-6) with the pumping pressure, p_0 , unit weight of filling slurry, γ , and height of geosynthetic tube, H ;
- 2) Calculate the factor of pumping pressure, Q using Equation (4-10a);
- 3) Check Figure 4.24 to get the desired factors such as factor of perimeter, C_L ,

factor of width, C_B , or factor of contact width, C_b ;

- 4) Calculate the desired parameter such as length, L , width, B , contact width with ground, b , or area of cross-section, A , using Equations (4-29) to (4-32).

The above calculation procedure can easily be realized by using Microsoft Office Excel. The advantage of Excel is that the linear difference equation can be solved automatically after changing the inputs, such as micro function $Qmatch()$ in Figure 4.25. There are two cases of calculation based on different inputs:

- 1) When H , γ and p_0 are taken as inputs, the calculation can be carryout directly using the calculation procedure as shown in Figure 4.25.
- 2) When L , γ and p_0 are taken as inputs, the “goal seek” function in Microsoft Office Excel 2010(**Menu/Data/What-if Analysis/Goal Seek**). The calculation procedures are shown in Figure 4.26.

Although this method is not as accurate as the computer program proposed in section 4.3.2, it is more convenient to be used for geosynthetic tube design when sophisticated software is not available.

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q
1	Charts Methods																
2																	
3	INPUTS				OUTPUTS												
4	γ	H	p_0		T	Q	B		L		b	A					
5	12	1	10		8	1.52083	B	C_B	L	C_L	b	C_b	1.162				
6	(kN/m ³)	m	kPa				1.395	1.20776	3.903	3.3803	0.634	0.54905	1.162				
7					kN/m		m		m		m		m ²				
8	Data from computer program																
9	Q	C_L	Q	C_B	Q	C_b	Notes										
10	1.00000	18.67328	1.00000	8.67616	1.00000	7.92242	1. input data of of the pumping pressure, p_0 , unit weight of filling slurry, γ , and height of geosynthetic tube, H										
11	1.00600	9.29226	1.00600	3.99218	1.00600	3.25369	2. Calculate tensile force, T , and factor of pumping pressure, Q . $T = (p_0 H + \frac{1}{2} \gamma H^2) / 2$										
12	1.01706	7.83738	1.01706	3.27401	1.01706	2.53554	$Q = (p_0^2 + 2\gamma T) / 2\gamma T$										
13	1.06329	6.03762	1.06329	2.40270	1.06329	1.67097											
14	1.13746	5.00516	1.13746	1.91990	1.13746	1.20078											
15	1.18377	4.62974	1.18377	1.74918	1.18377	1.04173	3. get the desired factors such as $CL=Qmatch(F5,A10:B34)$, $CB=Qmatch(F5,C10:D34)$ or $Cb=Qmatch(F5,E10:F34)$										
16	1.30681	3.98810	1.30681	1.46510	1.30681	0.77730	using the following micro function, $Qmatch()$										
17	1.38389	3.71750	1.38389	1.34886	1.38389	0.67063											
18	1.56957	3.26030	1.56957	1.15755	1.56957	0.50578	Function $Qmatch(\text{ByVal } xvalue, \text{ByVal } sordata \text{ As Range})$										
19	1.70547	3.02372	1.70547	1.06138	1.70547	0.42737	With sordata										
20	1.84318	2.83344	1.84318	0.98548	1.84318	0.36956	If $xvalue < .Cells(1, 1)$ Or $xvalue > .Cells(.Rows.Count, 1)$ Then										
21	2.12222	2.54244	2.12222	0.87203	2.12222	0.28605	$Qmatch = "Overflow"$										
22	2.26302	2.42750	2.26302	0.82810	2.26302	0.25925	Else										
23	2.54633	2.23833	2.54633	0.75691	2.54633	0.20895	Set $Bmatch = .Find(xvalue)$										
24	2.88844	2.06130	2.88844	0.69152	2.88844	0.16960	If $Bmatch$ Is Nothing Then										
25	3.11741	1.96442	3.11741	0.65624	3.11741	0.15233	$RowIndex = Application.Match(xvalue, .Columns(1), True)$										
26	3.40435	1.86072	3.40435	0.61885	3.40435	0.13084	$Qmatch = (.Cells(RowIndex + 1, 2) - .Cells(RowIndex, 2)) * (xvalue - .Cells(RowIndex, 1)) /$										
27	3.69192	1.77196	3.69192	0.58717	3.69192	0.11825	$(.Cells(RowIndex + 1, 1) - .Cells(RowIndex, 1)) + .Cells(RowIndex, 2)$										
28	3.97997	1.69487	3.97997	0.55988	3.97997	0.10552	Else										
29	4.55711	1.56686	4.55711	0.51502	4.55711	0.08303	$Qmatch = Application.VLookup(xvalue, sordata, 2, False)$										
30	5.13525	1.46414	5.13525	0.47943	5.13525	0.07117	End If										
31	5.42459	1.41984	5.42459	0.46419	5.42459	0.06471	End If										
32	6.00367	1.34213	6.00367	0.43760	6.00367	0.05881	End With										
33	7.30798	1.20555	7.30798	0.39132	7.30798	0.04488	End Function										
34	9.48423	1.04860	9.48423	0.33879	9.48423	0.03286	4. calculate the desired parameter as length, L , width, B , contact width with ground, b , or area or cross-section, A

Figure 4.25 Charts method using Microsoft Office Excel for geosynthetic tube calculation when H , γ and p_0 are taken as inputs

Figure 4.26 Charts method using Microsoft Office Excel for geosynthetic tube calculation when L , γ and p_0 are taken as inputs

4.7 Conclusions

In this chapter, analytical methods for geosynthetic tubes and mats were proposed by assuming the tensile forces along their cross-sectional circumference were constant. A computer program was written using the adaptive Runge-Kutta-Merson method (RKM4) to solve the proposed analytical equations. In order to verify the accuracy of the computer program, it was compared with Leshchinsky's (1996) and Kazimirowicz's (1994) solutions. The solutions were also compared with the model test results. The closed-form solution for geosynthetic mat was derived in this chapter too. Through all the analysis of geosynthetic tube and mats, the following conclusions can be withdrawn.

Geosynthetic tube

1. The difference between the results obtained from the proposed method and the solutions of Leshchinsky's (1996) and Kazimirowicz's (1994) is less than 5%. The cross-sections obtained from the proposed method agree well with those from model test results. The comparisons between the analytical and model test results indicate that the tensile forces measured at the bottom corner from the model tests are slightly higher than those from analytical solutions due to the influence from bending of strain gauges. The tensile forces along the contact surface with ground obtained from model tests are smaller than those from analytical solutions because of the influence of frictions between geosynthetic sheets and concrete floor.
2. When the height and width are normalized perimeter of cross-section, the normalized height or width versus the normalized pumping pressure curve becomes unique. The normalized height decreases at a faster rate when the normalized pumping pressure is less than 0.2.
3. A closed-form solution was also derived for the calculation of geosynthetic tube resting on rigid foundation based on some approximations. The equations

in these approximated equations express relationships between the geometry parameters and the factor of pumping pressure which make it possible to establish a series of basic charts for all the geometry parameters such as the contact width with ground, b , the width, B , height, H , perimeter, L , and area, A , of cross-section of a geosynthetic tube. With these design charts, the tensile force along the tube, T , and geometry parameters, b , L , B , A , can be calculated. The calculation can be implemented by using Microsoft Office Excel. Although this method is not as accurate as the computer program, it is convenient to be used for geosynthetic tube design when sophisticated software is not available.

Geosynthetic mat

4. When the pumping pressure, p_0 , equals to zero, the geosynthetic tube becomes flat and can be called geosynthetic mat in this study. A closed-form solution was derived for the calculation of the contact width with ground, b , the width, B , height, H , perimeter, L , and area, A , of cross-section of a geosynthetic tube under $p_0 = 0$ conditions. Using these formulas, the geometry parameters can be obtained directly without calculating the cross-section of the tube.
5. The cross-section of the geosynthetic mat obtained from the closed-form solutions agrees well with those from the experiments. The tensile forces from model test were quite scatter. However, the tensile forces from close-form solutions range on two sides of the model tests results.
6. For a given design height, the cross-section will take different perimeters when the widths are different. The difference between the perimeters of these three cross-sections equals to two times the difference between their widths. The cross-section with height to width ratio, k , of 0.163 has the smallest perimeter.

CHAPTER 5

ANALYSIS OF GEOSYNTHETIC TUBES & MATS ON DEFORMABLE FOUNDATION

5.1 Introduction

When geosynthetic tubes are used for dyke construction, they are often laid on soft ground where large settlement may take place. The ground settlement may influence the performances of the dyke made of geosynthetic tubes. However, there are still no suitable analytical methods available for the calculation of geosynthetic tubes resting on deformable foundation yet. The only method was proposed by Plaut and Suherman (1998) to analyze the geosynthetic tube resting on tensionless Winkler foundation. In the Plaut and Suherman's solution, the perimeter of the cross-section of the tube and the pumping pressure (expressed as pressure head) are assumed to be known. The height and width of the geosynthetic tube are the results of the solutions. However, in practice, the height of geosynthetic tube and the pumping water pressure are often the targeted design parameters. The perimeter of the cross-section of the tube and the tensile force of geotextile material required are calculated. The existing Winkler foundation method may not always be applicable directly to geosynthetic mat as the solution may not converge when the ratio between the height and width of the tube is very small. Furthermore, the parameter, K_f , is difficult to determine directly from laboratory test. A more suitable analytical method for geosynthetic tubes and mats resting on deformable foundation has to be developed.

In this chapter, three approaches were adopted to develop the solutions for geosynthetic tubes resting on deformable foundation: the Winkler foundation method, the e-log p method and the numerical method. These methods can be used for calculating impervious geosynthetic tube or mat or rubber dams.

5.2 Winkler Foundation Method

5.2.1 Winkler Foundation

The Winkler foundation was proposed by Winkler (1867) as a simple way to model the soil response to surface loads. In this method, the soil was modeled as a continuous layer of closely arranged, parallel, independent and linear-elastic springs as shown in Figure 5.1. The contact pressure between the foundation and the loading structure is assumed to be proportional to the corresponding displacement of the contact surface. The basic governing equation for this method is:

$$q_i = K_f \delta_i \quad (5-1)$$

where, q_i = contact vertical surface pressure at point i ; δ_i = vertical surface deflection at point i ; K_f = modulus of subgrade reaction.

Figure 5.1 Beam on Winkler foundation (after Subramanian et. al. 2005)

The modulus of subgrade reaction, K_f , related to the load-deformation characteristic of the foundation is elastic. The Winkler model has the following limitations (Subramanian et. al. 2005),

- K_f is assumed to be constant and independent of the loading intensity and pattern and the geometry of the foundation;
- A realistic determination of in situ rheological properties of the founding strata is difficult;
- The interaction between adjacent springs is ignored, even though there is very definite load spreading by vertical shear within an actual soil mass;
- Compatibility between deformation of the foundation structure and the founding stratum is ignored.

Figure 5.2 Factor for estimating elastic modulus (after TM 5-818-1)

In fact, the modulus of subgrade reaction, K_f , is not a property of soil layers but depends on foundation loads, foundation size and stratification of soil layers. There is no direct laboratory test for modulus of subgrade reaction. The standard bearing plate load test according to ASTM D 1196 is often used in the field tests. The most

commonly used calculation method is based on elastic modulus of soil layers (TM 5-818-1).

$$E = kS_u \quad (5-2)$$

$$K_f = \frac{E}{(1-\nu^2)\sqrt{A}} \quad (5-3)$$

where, E is undrain modulus of clay, k is calculation factor from Figure 5.2, S_u is undrained shear strength, ν is Poisson's ratio, and A is contact area of loading structure.

5.2.2 Geosynthetic Tube

A solution for geosynthetic tube resting on the Winkler Foundation is presented in this section. The geosynthetic tube is assumed to be watertight and inflated with water or slurry. The derivation processes are almost the same as that for geosynthetic tube resting on rigid foundation except that the foundation soil is modeled by Winkler Foundation with a modulus of subgrade reaction, K_f .

(1) *Derivation*

A simplified free body diagram of the half cross-section is shown in Figure 5.3. The coordinates are set up with y in the horizontal direction and x in the vertical direction. The origin is taken as the top center point of the cross-section. Then the cross-section of the tube can be considered symmetric with respect to x -axis. As shown in Figure 5.3, the angle between the tangent direction along the cross-section of geosynthetic tube and the y axis is θ . Two geometrical equations relating the angle between the tangent direction along cross-section and the x and y coordinates are given as follows,

$$\frac{dx}{ds} = \sin \theta \quad (5-4)$$

$$\frac{dy}{ds} = \cos \theta \quad (5-5)$$

(a) Free body of half cross-section

(b) Free body of force equilibrium

Figure 5.3 Forces of geosynthetic tube resting on Winkler foundation

By denoting p_0 as pumping pressure, the hydraulic pressure acting internally on the tube at a given depth can be calculated as $p_0 + \gamma x$. Based on the force equilibrium of the free body diagram shown in Figure 5.3(b), the force equilibrium equations on the normal and tangential directions of the calculation unit yields the expression of differential equations shown as follows.

$$\frac{dT}{ds} = \alpha K_f (x - H) \sin \theta \quad (5-6)$$

$$\frac{d\theta}{ds} = \frac{1}{T} [-\alpha K_f (x - H) |\cos \theta| + P_0 + \gamma x] \quad (5-7)$$

where $\alpha = 0$ for $x < H$, and $\alpha = 1.0$ for $x > H$.

The tensile force along the geosynthetic tube is calculated by the force equilibrium acting on the half cross-section. Due to the symmetry of the model, only half of the cross-section is analyzed as shown in Figure 5.3(a). The hydraulic pressure in horizontal direction acting on the free body is also shown in Figure 5.3(a). The settlement of geosynthetic tube is denoted as H_f . The reaction force of the Winkler foundation is vertical. Therefore, the forces acting along the horizontal direction only involve the hydraulic pressure and the tensile force. The equilibrium between these two forces yields the following equation:

$$T_{x=H+H_f} + T_{x=0} = p_0(H + H_f) + \frac{1}{2} \gamma (H + H_f)^2 \quad (5-8)$$

Integrating Equation (5-6) and combining Equations (5-4), (5-5) and (5-8), the tensile force along the geosynthetic tube is derived and shown in Equation (5-9) which indicates that the tensile force along the geosynthetic sheet above the ground surface, $T_{x=0 \sim H}$, is constant as $T_{x=0 \sim H}$ is independent of position x or y which can be calculated when H , H_f , K_f are known.

$$T_{x=0 \sim H} = \frac{1}{2} p_0(H + H_f) + \frac{1}{4} \gamma (H + H_f)^2 - \frac{1}{4} K_f H_f^2 \quad (5-9a)$$

$$T_{x=H \sim (H+H_f)} = K_f \left(\frac{1}{2} x^2 - Hx \right) + \frac{1}{2} p_0(H + H_f) + \frac{1}{4} \gamma (H + H_f)^2 - \frac{1}{4} K_f (H_f^2 - 2H^2) \quad (5-9b)$$

Combining Equations (5-7), (5-4) and (5-5), a nonlinear differential equation relating geometry of the cross-section of geosynthetic tube is derived as follows,

$$y'' = \frac{1}{T} [\alpha K_f (x - H) |y'| (1 + y'^2)^{\frac{1}{2}} - (P_0 + \gamma x)] (1 + y'^2)^{\frac{3}{2}} \quad (5-10)$$

Equation (5-10) has to be solved numerically because it contains an elliptical differential equation which does not have closed-form solution. The calculation was carried out with the inputs as follows, the unit weight of the filling slurry, γ , the height of geosynthetic tube, H , and the pumping pressure, p_0 . The tensile force, T , is calculated using Equations (5-9). The following two initial boundary conditions are needed to solve this equation:

- 1). when $x = 0$, $y = 0$, $dy/dx = \infty$, $\theta = 0$.
- 2). when $x=H+H_f$, $y = 0$, $dy/dx = -\infty$, and $\theta = \pi$.

Using the above initial boundary conditions and Equations (5-9) and (5-10), the relationship between the x and y coordinates that depicts the geometry of the cross-section of the geosynthetic tube can then be solved.

(2) Numerical Calculation

The numerical calculation procedure for geosynthetic tube resting on Winkler foundation is discussed in this section. A computer program is written with the RKM4 method to solve Equations (5-9), (5-10). The inputs are as follows, the unit weight of the slurry, γ , the modulus of subgrade reaction, K_f , the height of geosynthetic tube, H , and the pumping pressure, p_0 . The procedure is outlined as follows,

- (1) Input the initial parameters: γ , p_0 , K_f , and H ;
- (2) Calculate the tried tensile force on the top point of tube, T_t , using Equations (5-9) by assuming the tried settlement of geosynthetic tube is H_{ft} .
- (3) Input γ , H_{ft} , p_0 , H , K_f , and T_t into Equation (5-10) to calculate the x and y coordinates along the cross-section of the geosynthetic tube;
- (4) Adjust the value of H_{ft} if $y \neq 0$ or $dy/dx \neq -\infty$, when $x=H+H_f$. And repeat step (2) to (3) until $y = 0$ and $dy/dx = -\infty$. The final value of H_{ft} is the amount of settlement of the geosynthetic tube, H_f . T_t is the tensile force on the top point of tube, T . Using H_f , the x and y coordinates, the perimeter, the width, the contact width, and the tensile force are obtained;
- (5) If the length of the geosynthetic tube, L , is taken as an input, instead of height, H , the iteration is done as follows: assume $H=L/\pi$ as the input parameter to calculate the tried length, L_t . If $L_t \neq L$, then modify H and repeat the calculation until a good agreement between L_t and L is achieved.

(3) Verification

The results of the computer program are compared with the solutions given by Plaut and Suherman (1998). As reviewed in the Introduction, the Plaut and Suherman's method uses non-dimensional parameters in which H and H_f are normalized by the length of cross-section, L ; the pumping pressure, p , by γL ; tensile force, T , by γL^2 ; and modulus of subgrad reaction, K_f , by γ . The present method was made by taking the length of the cross-section of the tube, L , to be 1 m and the unit weight of the slurry to be 1.0 kN/m³. The analytical results obtained by both methods are shown in

Table 5.1. It can be seen that a good agreement between the two solutions is achieved.

Table 5.1 Comparison of present with Plaut and Suherman's solution (Plaut & Suherman 1998)

K_f (kPa/m)	Source	p_0 (kPa)		H (m)		H_f (m)		T_{max} (m)		T_{min} (m)	
		Values	%	Values	%	Values	%	Values	%	Values	%
10	Plaut	0.0565	1.33	0.1935	0.39	0.0264	8.71	0.02005	1.90	0.01656	0.06
	Present	0.0558		0.1943		0.0287		0.02043		0.01657	
25	Plaut	0.0514	1.95	0.1986	0.50	0.0104	9.62	0.01696	0.12	0.01562	0.06
	Present	0.0504		0.1996		0.0114		0.01698		0.01561	
50	Plaut	0.0498	0.18	0.2002	0.04	0.0051	15.69	--		--	
	Present	0.0499		0.2001		0.0059		0.01591		0.01527	
100	Plaut	0.0491	0.65	0.2009	0.16	0.0025	12.00	--		--	
	Present	0.0488		0.2012		0.0028		0.01533		0.01512	
200	Plaut	0.0487	0.41	0.2013	0.10	0.0013	15.38	--		--	
	Present	0.0485		0.2015		0.0015		0.01506		0.01487	

(4) Parametric Studies

Using the proposed method, some parametric studies of a geosynthetic tube resting on deformable foundation are conducted to assess the influence of some of the control parameters.

The effect of the modulus of subgrade reaction on the displacement of geosynthetic tube is studied. The parameter study is carried out on an impervious geosynthetic tube with perimeter of 9 m filled with slurry of unit weight of 12 kN/m^3 . Using a pumping pressure of 10 and 50 kPa, the cross-sections of the geosynthetic tube are calculated as shown in Figure 5.4. The solutions are given for modulus of subgrade reaction, K_f , of 100, 200, and $\infty \text{ kPa/m}$ respectively. It can be seen from Figure 5.4 that the height of the geosynthetic tube above the ground surface is influenced by the modulus of foundation. For a certain pumping pressure, the weaker the foundation the lower the height the geosynthetic tube. This is consistent with the intuitive expectation. Based on the solutions shown in Figure 5.4, the differences

between solutions with and without considering the modulus of subgrade reaction are evaluated.

(a) Pumping pressure, $p_0 = 10 \text{ kPa}$

(b) Pumping pressure, $p_0 = 50 \text{ kPa}$

Figure 5.4 Changes in the cross-section of geosynthetic tube with respect to pumping pressure, p_0 , and modulus of subgrade reaction, K_f (for $\gamma = 12 \text{ kN/m}^3$, $L = 9 \text{ m}$, $K_f = 100, 200$ and $\infty \text{ kPa/m}$, respectively).

The percentages of differences in the height of the tube versus the modulus of subgrade reaction K_f are plotted in Figure 5.5. It can be seen that when the modulus of subgrade reaction is less than 1000 kPa/m , the height of the geosynthetic tube is more influenced by the modulus of subgrade reaction. The differences become negligible when the modulus of subgrade reaction is greater than 1000 kPa/m . The settlement of the geosynthetic tube, H_f , is also plotted versus K_f in Figure 5.5(b). For the pumping pressures used, the settlement of the geosynthetic tube H_f varies dramatically with the modulus of subgrade reaction. However, when the modulus of subgrade reaction is greater than 1000 kPa/m , the influence of modulus becomes insignificant. When the settlement of the geosynthetic tube is small, the calculation method for geosynthetic tube on rigid foundation can be used as a good approximation for Equations (5-9), (5-10). It can be concluded that the deformable Winkler foundation method is only necessary to be used for geosynthetic tube resting on a very soft ground with K_f lower than 1000 kPa/m .

(a) Relationship between percentage error and modulus of subgrade reaction

(b) Relationship between the settlement H_f and modulus of subgrade reaction

Figure 5.5 Effect of modulus of subgrade reaction on the settlement of geosynthetic tube (For $\gamma = 12 \text{ kN/m}^3$, $L = 9 \text{ m}$).

The effect of pumping pressure on the geometry of the geosynthetic tube is also evaluated. Using the same inputs as Figure 5.4 such as $\gamma = 12 \text{ kN/m}^3$, $L = 9 \text{ m}$, $K_f = 100 \text{ kPa/m}$, the cross-section of the geosynthetic tube at p_0 of 0, 1, 5, 10, 20, 50 and 100 kPa are calculated and shown in Figure 5.6(a). The height of the geosynthetic tube versus pumping pressure is also plotted in Figure 5.6(b). It can be seen that the higher the pumping pressure, the higher the geosynthetic tube above the ground surface, and the greater the settlement below the ground surface. Figure 5.6 b shows that the influence of pumping pressure p_0 on H and H_f is greater when p_0 is smaller than 30 kPa in this case. When p_0 is greater than 30 kPa, the cross-section of the geosynthetic tube has reached almost a circular shape and then the pumping pressure will have little effect on the cross-section of the tube.

However, a higher p_0 will significantly increase the tensile force as shown in Figure 5.7. It is also seen in Figure 5.7 that tensile forces along the cross-section of the geosynthetic tube in the section above the ground surface are constant, whereas the tensile forces along the section below the ground surface varies. The lower the point on the geosynthetic tube, the greater the tensile force will be induced. The tensile force attains the highest value at the bottom of the geosynthetic tube. It should be

pointed out that the above results regarding tensile force in geotextile is only valid when the shear friction between geotextile and soil is ignored as is assumed in the Winkler foundation model. The conclusions are only valid for slurry filled impermeable geosynthetic tube resting on Winkler foundation. However, in practice, the friction between geotextile and soil will play a role and lead to a reduction in the tensile force in the geotextile along the portion in contact with the ground (Saathoff, et. al., 2007).

(a) Cross-section of geosynthetic tube changing with pumping pressure

(b) Relationships between height and pumping pressure

Figure 5.6 Effect of pumping pressure on the cross-section of geosynthetic tube resting on Winkler foundation for $\gamma = 12 \text{ kN/m}^3$, $L = 9 \text{ m}$, $K_f = 100 \text{ kPa/m}$.

Figure 5.7 Relationship between the height of a point on the cross-section and the tensile force of the geosynthetic tube (For $\gamma = 12 \text{ kN/m}^3$, $L = 9 \text{ m}$, $K_f = 100 \text{ kPa/m}$)

For a real project, the design target is normally the height of geosynthetic tube. Therefore, parametric studies are carried out to investigate the effect of modulus of subgrade reaction and pumping pressure. For a geosynthetic tube with a design height of 2 m under a pumping pressure of 10 kPa with slurry of a unit weight 12 kN/m^3 , the variation in the cross-section area and settlement is shown in Figure 5.8(a). The influence of the pumping pressure on the cross-sections of a geosynthetic tube with a fixed height of 2 m resting on soft foundation with a modulus of 100 kPa/m is shown in Figure 5.8(b). The solutions given in Figure 5.8(b) are for pumping pressures of $10, 30, 50 \text{ kPa}$ respectively. It can be seen that given the height of the tube being the same, the higher the pumping pressure, the smaller the perimeter as the tube is more circular. This is a different reflection of the fact shown in Figure 5.6(a) where it is seen that given the perimeter the same, the higher the pumping pressure, the higher the tube.

(a) Cross-section of geosynthetic tube with $p_0 = 10 \text{ kPa}$ (b) Cross-section of geosynthetic tube with $K_f = 100 \text{ kPa/m}$

Figure 5.8 Effect of modulus of subgrade reaction and pumping pressure on the cross-section of geosynthetic tube (for $\gamma = 12 \text{ kN/m}^3$ and $H = 2 \text{ m}$).

The influences of pumping pressure and modulus of the subgrade reaction of soil K_f on the maximum tensile force and the perimeter of the cross-section of a geosynthetic tube with a design height of 2 m are shown in Figure 5.9. It can be seen from Figure 5.9(a) that the tensile force increases with the increasing of pumping pressure and the decreasing of modulus of subgrade reaction. However, the effect of K_f becomes insignificant when $K_f > 1000 \text{ kPa/m}$. Figure 5.9(b) shows the perimeter length required achieving a geosynthetic tube of 2 m height. It can be seen that the required perimeter increases when the soil is getting softer.

(a) Modulus of subgrade reaction versus maximum tensile force

(b) Modulus of subgrade reaction versus perimeter of cross-section

Figure 5.9 Effect of modulus of subgrade reaction on the tensile force and perimeter (For $\gamma = 12 \text{ kN/m}^3$, $H = 2 \text{ m}$, $p_0 = 10, 20, 50 \text{ kPa}$ respectively).

For a given design height of 2.0 m, the variations of the maximum tensile force and perimeter of the cross-section inflated with different pumping pressure are plotted in Figure 5.10(a) and (b). As expected, the tensile force in the geosynthetic tube is affected considerably by the pumping pressure. As shown in Figure 5.10(a), the maximum tensile force increases almost linearly with the increasing of pumping pressure at different K_f values. Figure 5.10(b) shows the perimeter for a geosynthetic tube of 2m height when resting on a soil with different K_f reduces

quickly as the pumping pressures increases. However, the influence becomes insignificant when the pumping pressure is greater than 30 kPa . This is because when the pumping pressure is greater than 30 kPa , the shape of the geosynthetic tube reaches almost a perfect circle and then the pumping pressure will not affect much the cross-section of the tube anymore.

(a) Pumping pressure versus maximum tensile force

(b) Pumping pressure versus perimeter of cross-section

Figure 5.10 Effect of pumping pressure on the tensile force and perimeter (For $\gamma = 12 \text{ kN/m}^3$, $H = 2 \text{ m}$, $K_f = 100, 200, 500, 1000 \text{ kPa/m}$ respectively).

5.2.3 Geosynthetic Mat

(1) Basic Theory Derivation

Geosynthetic mat is a special case for geosynthetic tube. The derivation is similar to geosynthetic tube except for the difference of internal water pressure which makes the free body diagram slightly different from geosynthetic tube as shown in Figure 5.11.

Figure 5.11 Free body diagram of geosynthetic mat resting on Winkler foundation

Integrating $p_0 = 0$ to Equations (5-7) and (5-9) gives the expression of domain differential equation and tensile force along geosynthetic mats shown as follows.

$$\frac{d\theta}{ds} = \frac{1}{T} [-\alpha K_f (x - H) |\cos \theta| + \gamma x] \quad (5-11)$$

in which $\alpha = 0$ for $x < H$, and $\alpha = 1.0$ for $x > H$

$$T_{x=0-H} = \frac{1}{4} \gamma (H + H_f)^2 - \frac{1}{4} K_f H_f^2 \quad (5-12)$$

$$T_{x=H-(H+H_f)} = K_f \left(\frac{1}{2} x^2 - Hx \right) + \frac{1}{4} \gamma (H + H_f)^2 - \frac{1}{4} K_f (H_f^2 - 2H^2) \quad (5-13)$$

Equation (5-12) indicates that the tensile force along the geosynthetic sheet above the ground surface, $T_{x=0-H}$, is constant since $T_{x=0-H}$ is independent of position x or y .

The tensile force above the ground surface, $T_{x=0-H}$, can be calculated when H , H_f , K_f are known. Because the problem is symmetric as shown in Figure 5.12, the unit just on the center of the contact edge with ground surface (Figure 5.12(a)) must be horizontal. The water pressure acting down on top of this unit is of $\gamma(H+H_f)$. The pressure acting up by Winkler foundation on the bottom of this unit is $K_f H_f$. These two pressures balance vertically because the unit is horizontal. Then H_f is calculated as follows,

$$H_f = \gamma H / (K_f - \gamma) \quad (5-14)$$

Figure 5.12 Free body diagram to calculate H_f

Integrating $p_0 = 0$ to Equation (5-10), a nonlinear differential equation relating geometry of the cross-section of the geosynthetic mat resting on Winkler Foundation is derived as shown in Equation (5-15).

$$y'' = \frac{1}{T} [\alpha K_f (x - H) |y'| (1 + y'^2)^{\frac{1}{2}} - \gamma x] (1 + y'^2)^{\frac{3}{2}} \quad (5-15)$$

Equation (5-15) has to be solved numerically because it contains an elliptical differential equation which doesn't have closed-form solution. In Equation (5-15), the unit weight of filling slurry, γ , the height and width of the geosynthetic mat above the ground surface, H and B , are taken as inputs. The tensile force, T , is

calculated using Equations (5-12) and (5-13). The following two initial boundary conditions are needed to solve this equation:

- 1). when $x = 0$, $y = y_0$, $dy/dx = \infty$, $\theta = 0$.
- 2). when $x = H + H_f$, $y = 0$, $dy/dx = -\infty$, and $\theta = \pi$.

(2) Numerical Calculation

Similar to geosynthetic mats resting on rigid foundation, a reference cross-section can also be defined as the highest cross-section that can be inflated with zero pumping pressure resting on a certain deformable foundation. For a given deformable foundation, there is one and only one reference cross-section which is shown as *Mat 1* in Figure 5.13. The rest of the geosynthetic mat when filled to this height will be wider than *Mat 1* such as *Mat 2* in Figure 5.13. The Point A is the origin and on the top of the mat, CC' is the section assumed to be horizontal. In order to calculate the length of CC' , the width of cross-section should also be given as a input parameter. In the calculation procedure, the length of CC' is denoted as $2*y_0$.

A substitution method is developed to solve Equation (5-15). The unit weight of the slurry, γ , the modulus of subgrade reaction, K_f , the height of geosynthetic mat, H , the width of geosynthetic mat, B , or the perimeter, L , of the cross-section are taken as input parameters. The procedure is outlined as follows,

- (1) Input the design parameters: γ , K_f , H , B or L ;
- (2) Assume length of section AC equal to zero, $y_0 = 0$, at the start of calculation, and calculate the height of the mat below ground surface, H_f with Equation (5-14);
- (3) Calculate the tried tensile force, T_i , using Equation (5-12) and (5-13);
- (4) Calculate the x and y coordinates with Equation (5-15) with γ , K_f , H , H_f , T_i ; and the initial conditions. The results of perimeter and the width is defined as L_t and B_t , respectively;
- (5) Adjust y_0 and repeat step (3) to (5) if B_t or L_t , are different from inputs in step

(1), until they are matching in a allowed precision, $|1-B_t/B| < 10E-6$ or $|1-L_t/L| < 10E-6$;

Figure 5.13 Cross-section of geosynthetic mat resting on deformable foundation

(3) Parametric Studies

The analytical results of geosynthetic mats resting on deformable foundation using the Winkler foundation method are presented in this section. A group of geosynthetic mat with perimeter of 9 m, inflated to different heights is shown in Figure 5.14. The calculation is carried out by using the unit weight of filling material of 12 kN/m^3 , the modulus of subgrade reaction of 50, 100, 1000 kPa/m . For the modulus of subgrade reaction of 50 kPa/m and the height of geosynthetic mat of 0.2 to 0.591 m as shown in Figure 5.14(a), the settlement of geosynthetic mat increase from 6.3 to 18.6 cm. For the modulus of subgrade reaction of 100 kPa/m and the height of geosynthetic mat of 0.2 to 0.636 m as shown in Figure 5.14(b), the settlement of geosynthetic mat increase from 2.7 to 8.67 cm. However, for the modulus of subgrade reaction of 1000 kPa/m and the height of geosynthetic mat of 0.2 to 0.676 m as shown in Figure 5.14(c), the settlement of geosynthetic mat is very small ($< 1.0 \text{ cm}$).

(a) Cross-section of geosynthetic mat for $K_f = 50 \text{ kN/m}^3$

(b) Cross-section of geosynthetic mat for $K_f = 100 \text{ kN/m}^3$

(c) Cross-section of geosynthetic mat for $K_f = 1000 \text{ kN/m}^3$

Figure 5.14 The cross-section of geosynthetic mat resting on Winkler foundation with $L=9.0 \text{ m}$, $\gamma=12 \text{ kN/m}^3$

For a general case, the relationship between settlement of geosynthetic mat, H_f , and the modulus of subgrade reaction, K_f , is plotted in Figure 5.15. The calculation is carried out by taking unit weight of filling slurry of 12 kN/m^3 , the modulus of

subgrade reaction in the ranges of 20 to 5000 kPa/m , and the height of geosynthetic mat in the ranges of 0.1 to 2.0 m . It can be seen that the settlement of geosynthetic mat, H_f , is almost constant when K_f is greater than 1000 kPa/m . In the other words, the Winkler foundation method is only necessary to be used for the geosynthetic mats resting on a subgrade with modulus of subgrade reaction, K_f , less than 1000 kPa/m . When $K_f > 1000$ kPa/m , it is accurate enough to analyze the geosynthetic mat with the closed-form method proposed in Chapter 4.

Figure 5.15 Relationships between K_f and H_f when $\gamma=12$ kN/m^3

For a given design height of cross-section, different perimeters may be needed which are similar to geosynthetic mat resting on rigid foundation. When the subgrade is deformable, the modulus of subgrade reaction, K_f , also needs to be considered. A group of geosynthetic mats with a design height of 2 m and a width of 6 m , resting on deformable foundation is shown in Figure 5.16. The calculation was carried out by taking the unit weight of filling material of 12 kN/m^3 , and the modulus of subgrade reaction of 50, 100 and ∞ kPa/m . The perimeter of these three mats, L , were calculated as 26.34, 26.02 and 25.87 m . It can be seen that, for a certain height, the smaller the modulus of subgrade reaction, the larger the settlement of geosynthetic mat and the larger the perimeter of the cross-section needed.

Figure 5.16 The influence of modulus of subgrade reaction on designed geosynthetic mat (for $\gamma=12 \text{ kN/m}^3$, $H=2 \text{ m}$, $B=12 \text{ m}$, $K_f=50, 100$ and $\infty \text{ kN/m}^3$)

(4) Influence of K_f on Reference Cross-section

The value of the modulus of subgrade reaction influences the geometry of reference cross-sections which is defined as the highest of cross-section that the geosynthetic mat can be inflated with zero pumping pressure. Therefore, there is one and only one reference cross-section for a certain deformable foundation. The rigid foundation has one and only one reference cross-section because it is a special case for deformable foundation with modulus of subgrade reaction of infinite value, $K_f = \infty$.

Figure 5.17 The influence of modulus of subgrade reaction to the height of reference cross-section (for $\gamma=12 \text{ kN/m}^3$, $L=9 \text{ m}$)

As shown in Figure 5.17, the smaller the modulus of subgrade reaction, the larger the settlement of the reference cross-section and the larger the total height, $H + H_f$. The height of reference cross-section, H , increases with the increasing of modulus of subgrade reaction and it is almost constant when K_f is greater than 1000 kPa/m.

The variation of areas of these reference cross-sections is also plotted in Figure 5.18. Although the height of reference cross-section decreases when modulus of subgrade reaction decreases, the total area of reference cross-section increases as shown in Figure 5.18. Similarly, the area of reference cross-section is almost constant when K_f is greater than 1000 kPa/m.

Figure 5.18 The influence of modulus of subgrade reaction on area of reference cross-section (for $\gamma=12 \text{ kN/m}^3$, $L=9 \text{ m}$)

5.3 e-log p method

The limitation of Winkler foundation method is its inability to account for the variation of vertical pressure within the soil mass when a surcharge pressure is applied on top of soil layers. Furthermore, the parameter, K_f , is difficult to determine directly from laboratory or field test. In this section, one-dimensional consolidation settlement theory is used to analyze the settlement of each calculation

element. The surcharge distribution within the soil mass induced by the weight of geosynthetic mat is calculated with the Boussinesq solution.

It should be pointed out that the e-logp method is only as an approximate method because the e-logp curve is established under 1-D condition only. The advantage of this method is that the parameters used for calculation are easier to be obtained from laboratory test. The surcharge pressure distribution within soil mass, the over consolidation ratio and the external water level are considered. The e-logp method can be used to calculate the cross-section of impervious geosynthetic mats or rubber dams.

5.3.1 Introduction to e-logp Method

One dimensional consolidation settlement theory calculates the settlement of soil layers by assuming the soil deform vertically without any lateral deformations. By dividing the foundation into finite layers, the settlements of each soil layers are calculated by the change of void ratio.

$$S_c = \sum_{i=1}^n \Delta S_{ci} = \sum_{i=1}^n \frac{\Delta e_i}{1 + e_0} H_i \quad (5-16)$$

$$\text{NC clay: } S_c = C_c \frac{H}{1 + e_0} \log \frac{\sigma'_0 + \Delta \sigma'}{\sigma'_0} \quad (5-17)$$

$$\text{OC clay, } \sigma'_0 + \Delta \sigma' < \sigma'_c: S_c = C_r \frac{H}{1 + e_0} \log \frac{\sigma'_0 + \Delta \sigma'}{\sigma'_0} \quad (5-18)$$

$$\text{OC clay, } \sigma'_0 + \Delta \sigma' > \sigma'_c: S_c = \frac{H}{1 + e_0} (C_r \log \frac{\sigma'_c}{\sigma'_0} + C_c \log \frac{\sigma'_0 + \Delta \sigma'}{\sigma'_c}) \quad (5-19)$$

where ΔS_{ci} is the settlement of each divided layer, Δe_i is the change of void ratio, H_i is the thickness of each divided layer and e_0 is the initial void ratio, C_c is compression index, C_r is recompression index, $\Delta \sigma'$ is the stress increase due to surcharge pressure, σ'_c is pre-consolidation pressure, and σ'_0 is initial load pressure.

Many empirical relations for compression index, C_c , have been proposed basing on laboratory or in situ test results. The typical values of compression index, C_c , are in

the range of 0.15 to 4.0. For some special soil types like Mexico city clay or Peats, the value of compression index may range from 7 to 10 (Holtz and Kovacs 1981). Usually, the typical value of recompression index, C_r , is about 5% to 10% of compression index C_c .

5.3.2 Geosynthetic Tube

(1) *Derivation*

The basic formulas to calculate the geometry and tensile force of a geosynthetic tube on deformable foundation with e-log p method are derived in this section. As the condition is symmetric, only half of the simplified cross-section of a geosynthetic tube is needed as shown in Figure 5.19. The definitions of basic parameters are the same as Winkler foundation method as discussed in Section 5.2. The angle between the tangent directions along the cross-section of geosynthetic tube and x -axis is θ . Two geometrical equations relating the angle between the tangent direction along cross-section and the x and y coordinates are given as shown in Equations (5-20) and (5-21).

$$\frac{dy}{ds} = \sin \theta \quad (5-20)$$

$$\frac{dx}{ds} = \cos \theta \quad (5-21)$$

The hydraulic pressure acting internally on geosynthetic tube is $\gamma(H-y)$ at a given y coordinate. The external water level, denoted as H_w , is also considered here by assuming that the water heights on both sides of the geosynthetic tube are the same. The external hydraulic water pressure acting on the calculation point is $\gamma_w(H_w-y)$ where y is its y -coordinate and γ_w is the unit weight of external water. The foundation is divided into finite separate vertical slices. The connection between each slice is neglected. The calculation element of a single slice is shown in Figure 5.19(b). When the soil slice is acted by a surcharge pressure, p'_f , on top, the vertical surcharge pressure within soil mass is calculated with Boussinesq equation as shown in Equation (5-22) by assuming the soil mass is a semi-infinite,

homogeneous, isotropic and elastic medium. Each slice is divided into N horizontal layers along depth direction. The total settlement of each soil layer, S_{ci} , is calculated basing on the difference of void ratio as shown in Equations (5-17), (5-18) and (5-19).

$$\Delta\sigma_{zi} = \frac{2p_f}{\pi} \frac{1}{(-y_i)} \quad (5-22)$$

in which $\Delta\sigma_{zi}$ is extra overburden pressure due to the action of geosynthetic tube, and y_i is y -coordinated of central point on i_{th} layer.

The connection between the foundation settlement and geometry of geosynthetic tube is built by the same contact edge between geosynthetic tube and foundation. The value of total settlement of soil slice, S_c , equals to the magnitude of y -coordinate of geosynthetic tubes shell. For the calculation element of geosynthetic tube shown in Figure 5.19(b), the circumferential tensile stress per unit width along cross-section is denoted as T and the stress increase written as dT . Stress equilibrium on the normal and tangential directions of this unit yields,

$$\frac{dT}{ds} = \alpha p'_f \sin \theta \quad (5-23)$$

$$\frac{d\theta}{ds} = \frac{1}{T} (p_0 + \gamma(H-y) - p_w - \alpha p'_f \cos \theta) \quad (5-24)$$

where, α is a non-dimensional parameter, $\alpha = 1.0$ for $y > 0.0$, and $\alpha = 0$ for $y < 0.0$, and p'_f is the reaction pressure of soil subgrade; p_w is external hydraulic water pressure, $p_w = \gamma_w(H_w-y)$ for $y < H_w$ and $p_w = 0$ for $y \geq H_w$.

Equations (5-23) and (5-24) can only be solved numerically because it relates to the settlement of soil foundation. In these equations, the inputs parameters are the unit weight of filling slurry, γ , the height and width of the geosynthetic tube, H and B . The tensile force, T , and geometry of cross-section are calculated after combining with the following two initial boundary conditions:

(1). When $y = H$, $x = 0$, $\theta = \pi$.

(2). When $y = -H_f$, $x = 0$, $\theta = 0$.

(a) Free body of half cross-section (b) Free body of calculation unit

Figure 5.19 Free body diagrams of geosynthetic tube on deformable foundation.

(2) Numerical Calculation

The analytical calculation procedure for e-log p method is discussed as follows. There are three unknown parameters in Equations (5-23) and (5-24) such as settlement of geosynthetic tube, H_f , height of geosynthetic tube, H , and the initial circumferential tensile force per unit width, T_0 . The unit weight of the slurry, γ , the external water level, H_w , the height of geosynthetic tube, H , and the pumping water pressure, p_0 , are taken as input parameters. The basic parameters of soil subgrade such as initial void ratio, e_0 , unit weight of soil, γ_s , compression index, C_c and OCR are known.

A mathematical programming scheme of Complex Method proposed by Box (1965) is used for iterating these equations. The method is summarized as follows (Lipson, 1977),

- Starting with an initial feasible point (vertex) in n -space ($n=3$), randomly generate $(k - 1)$ additional points ($k = n$). If it is the i th design variable of the j th point, then m ($m=2n$) such variables can be generated to satisfy the upper and lower variable bounds t_{ui} and t_{li} , by

$$t_{ij} = t_{li} + r_{ij}(t_{ui} - t_{li}) \quad i = 1, 2, \dots, m \quad (5-25)$$

where r_{ij} is a random number uniformly distributed over the interval 0-1. If each point fails to satisfy the implicit constraints for the problem, it is moved halfway to the centroid of all the existing feasible points including the initial one until feasibility occurs.

- b) With this starting “complex” of k feasible points, identify the point with the worst (highest) function value and compute the centroid of all of the other points.

$$t_{ic} = \frac{\sum_{j=1}^k t_{ij} - t_{iw}}{k-1} \quad (5-26)$$

where t_{ic} and t_{iw} are the i_{th} variables of the centroid point and the worst point, respectively.

- c) Replace the worst point with a new point by “reflecting” it through the centroid as

$$t_{iw} = t_{ic} + \alpha(t_{ic} - t_{iw}) \quad (5-27)$$

where $\alpha=1.3$ which is a pre-selected scale factor (Xu, 1995). If this new design does not satisfy the variable bounds, any violated variables are set just inside the bounds themselves. If any implicit constraints are then violated, the point is moved halfway to the centroid of the remaining points until feasibility is achieved.

- d) Terminate the procedure when the pre-defined accuracy is achieved.

(3) Parametric Studies

The parametric studies of geosynthetic tubes resting on deformable foundation using e-log p method are presented in this section. As an example, a group of geosynthetic tubes resting on deformable foundation and filled to different heights are shown in Figure 5.20. The calculation is carried out by taking the unit weight of filling slurry of 12 kN/m^3 , the perimeter of 9 m , and pumping pressures of 1, 5, 10, 20, 50 and 100 kPa , respectively. The soil subgrade is 10 m in depth with the compression index C_c of 0.35, void ratio of 1.0 and the unit weight of 18 kN/m^3 . The ground water level reaches the surface of the subgrade. The over-consolidated

ratio of soil subgrade is 1.0. The subgrade is divided into 10 layers. The calculation results of cross-sections and height of these series of geotextile tubes are shown in Figure 5.20. As seen from Figure 5.20(a) when pumping pressure is greater than 20 kPa, the cross-section of the geosynthetic tube reaches 70% of the maximum height (L/π) and the pumping pressure is not sensitive on the cross-section of geosynthetic tube. The higher the pumping pressure, the higher the cross-section and the greater the settlement of geosynthetic tube as shown in Figure 5.20(b).

(a) Cross-sections of geosynthetic tube

(b) Relationship between the height of geosynthetic tube and pumping pressure

Figure 5.20 Effective of pumping pressure on the cross-section of geosynthetic tubes resting on deformable foundation (For $C_c=0.35$, $e_0=1.0$, $\gamma = 18 \text{ kN/m}^3$ of soil subgrade and $L = 9.0 \text{ m}$, $\gamma = 12 \text{ kN/m}^3$)

However, a higher p_0 , greater than 20 kPa, increases the tensile force significantly as shown in Figure 5.21. Therefore, there is an economic pumping pressure as 20 kPa in this special tube. Note that for other kind of geosynthetic tubes or other filling materials, a new threshold has to be calculated. If pumping pressure is lower

than this value, the inflated volume is very sensitive to pumping pressure. However, when pumping pressure is greater than this value, the inflated volume increases a little but the geosynthetic tube is taken a higher risk for explosion. It is also seen that the tensile forces along the cross-section of the geosynthetic tube in the section above the ground surface are constant and varies below the ground surface. The lower the point on the cross-section the greater the tensile force. The tensile force attains the highest value at the center of the contact edge with the ground surface. It should be pointed out that the above results regarding tensile force distribution in geosynthetic is only valid when ignoring the lateral reaction of soil subgrade and the friction between geosynthetic and soil.

Figure 5.21 Relationship between the height of the point on the cross-section and the tensile force of the geosynthetic tube (For $C_c=0.35$, $e_0=1.0$, $\gamma=18 \text{ kN/m}^3$ of soil subgrade and $L = 9.0 \text{ m}$, $\gamma = 12 \text{ kN/m}^3$ of geosynthetic tube)

The effect of ground water level on the displacement of geosynthetic tube is studied. The geosynthetic tubes can be set in place along shorelines and river fronts either to be left exposed to battle the elements, or incorporated into the environment as part of a man-made dune or riverbank. They can also be placed in the water to serve as jetties and groins. Then the external water level will influence the geometry or tensile force of the geosynthetic tubes. The calculated cross-section of geosynthetic tubes inflated with pumping pressure of 10 and 50 kPa and acted by external water level of 0.5 m are shown in Figure 5.22. The cross-sections with ground water level

locate on ground surface are also plotted in the figure to compare their differences. It can be seen that the external water level has little influence on the geometry of the cross-section especially when pumping pressure is 50 kPa as shown in Figure 5.22(b). The buoyancy of the external water makes the settlement decreasing and height of geosynthetic tube increasing. It can be concluded that using the method without considering the external water level to calculate the geosynthetic tube resting on deformable foundation is conservative.

(a) Pumping pressure of 10.0kPa

(b) Pumping pressure of 50.0 kPa

Figure 5.22 The cross-section of the geosynthetic tube for ground water level locates on 0.5m above ground surface (For $C_c=0.35$, $e_0=1.0$, $\gamma=18 \text{ kN/m}^3$ of soil subgrade and $L=9.0 \text{ m}$, $\gamma=12 \text{ kN/m}^3$)

The relationships between pumping pressure and the maximum tensile force for geosynthetic tube resting on deformable foundation without the acting of external water pressure are shown in Figure 5.23. The increase of pumping pressure makes the tensile force increasing heavily and they almost fit to a linear relationship.

Figure 5.23 The relationship between pumping pressure and maximum tensile force (For $C_c=0.35$, $e_0=1.0$, $\gamma=18 \text{ kN/m}^3$ of soil subgrade; $L=9.0 \text{ m}$, $\gamma=12 \text{ kN/m}^3$)

5.3.3 Geosynthetic Mat

(1) Derivation

The basic formulas using e-log p method for geosynthetic mats can be treated as a special case for geosynthetic tube with zero pumping pressure. Therefore, the derivation is almost the same as geosynthetic tube except for the internal water pressure which makes the free body diagram slightly different from geosynthetic tube as shown in Figure 5.24. The hydraulic pressure acting internally on geosynthetic mat is $\gamma(H-y)$ at a given y coordinate.

(a) Free body of half cross-section

(b) Free body of calculation unit

Figure 5.24 Forces of geosynthetic mat on deformable foundation

Integrating $p_0 = 0$ to Equation (5-24) gives the expression of domain differential equation shown as follows.

$$\frac{d\theta}{ds} = \frac{1}{T} [\gamma(H - y) - p_w - \alpha p_f' \cos \theta] \quad (5-28)$$

Equations (5-20), (5-21), (5-23) and (5-28) can be solved numerically by combining with the following two initial boundary conditions:

1). When $y = H$, $x = x_0$, $\theta = \pi$.

2). When $y = -H_f$, $x = 0$, $\theta = 0$.

(2) Numerical Calculation

Similar to geosynthetic mat resting on Winkler foundation, there also exists a reference cross-section for a geosynthetic mat resting on elastic foundation. There is one and only one reference cross-section for a certain geosynthetic mat and soil foundation. The geosynthetic mats lower than reference cross-section will have an extra horizontal section on the top with its length of $2 \cdot x_0$.

There are three unknown parameters in Equations (5-20), (5-21), (5-23) and (5-28), namely height of geosynthetic mat below ground surface, H_f , initial circumferential tensile stress, T_0 , and the top flat distance, x_0 . The unit weight of the slurry, γ , the external water level, H_w , the height, H , and the width of geosynthetic mat, B , are taken as input parameters. The basic parameters of soil subgrade such as initial void ratio, e_0 , unit weight of soil, γ_s , compression index, C_c and OCR are known. The procedure is outlined as follows,

- (1) Input the design parameters for the geosynthetic mat: γ , H , L ; and assume $x_0 = 0$, $T_0 = \gamma(L/\pi)^2/4$ at the start of calculation;
- (2) Calculate the settlement of geosynthetic tube, H_f , using the trial-and-error method.
 - a) Assume the value of settlement of geosynthetic mat to be H_{ft} . The surcharge pressure, p'_{f0} , at point O is $\gamma(H+H_{ft}) - \gamma_w(H_w+H_{ft})$ for $H_w > -H_{ft}$ and $p'_{f0} = \gamma(H+H_{ft})$ for $H_w < -H_{ft}$;
 - b) Calculate the surcharge pressure within soil mass using Equation (5-22);
 - c) Calculate the total settlement on the starting point O , S_{0t} , using Equations (5-16), (5-17), (5-18) and (5-19);

- d) If $(H_{ft} - S_{0t}) < 1.0E-6$, the trial height, H_{ft} , is settlement of geosynthetic mat, H_f . Otherwise, modified H_{ft} , and repeat the steps (a) to (d).
- (3) Calculate the x and y coordinates using Equations (5-20), (5-21), (5-23) and (5-28) with γ , H_f , x_0 , and T_0 ;
- (4) If $T_{y=-H_f} + T_{y=H} \neq \gamma(H+H_{ft})^2/4$, change the value of T_0 and repeat step(2) to (3) until $T_{y=-H_f} + T_{y=H} = \gamma(H+H_{ft})^2/4$.
- (5) If $x \neq x_0$ when $y = H$, change the value of x_0 and repeat steps (2) to (4) until $x = x_0$.

(3) Analytical Results

The analytical results of cross-section of geosynthetic mats resting on deformable foundation using the e-log p method are presented in this section. As an example, a group of geosynthetic mats resting on deformable foundation and filled to different heights are shown in Figure 5.25. The calculation is carried out by taking the unit weight of filling slurry of 12 kN/m^3 , the perimeter of 9 m and the heights ranged from 0.2 m to 0.6 m . The parameters of soil subgrade were the same as that used for geosynthetic tube calculation in section 5.3.2. The external water levels were considered on the ground surface. The settlement of geosynthetic mat increased with their values of 4.92 cm for $H = 0.2 \text{ m}$, 8.52 cm for $H = 0.4 \text{ m}$, and 11.46 cm for $H = 0.6 \text{ m}$, respectively.

Figure 5.25 The cross-section of geosynthetic mat resting on normal consolidated Marine Clay (For $L = 9.0 \text{ m}$, $\gamma = 12 \text{ kN/m}^3$ of geosynthetic mat and $C_c = 0.35$, $e_0 = 1.0$, $\gamma = 18 \text{ kN/m}^3$ of soil subgrade)

The tensile forces along the cross-section of Figure 5.25(a) are presented in Figure 5.26. It can be seen that the tensile forces along the cross-section of the geosynthetic mat in the section above the ground surface are constant. However, the tensile forces along the section below the ground surface varies. The lower the point on the geosynthetic mat, the greater the tensile force is induced. The tensile force attains the highest value at the center of contact edge with ground surface. It should be pointed out that the above results regarding tensile force distribution in geosynthetic is only valid when using e-log p method without considering the lateral forces and frictions.

Figure 5.26 The tensile force distribution along geosynthetic mat resting on normal consolidated Marine Clay (For $L = 9.0 \text{ m}$, $\gamma = 12 \text{ kN/m}^3$ of geosynthetic mat and $C_c = 0.35$, $e_0 = 1.0$, $\gamma = 18 \text{ kN/m}^3$ of soil subgrade)

(4) Influence of Compression Index

The influence of compression index to the settlement of geosynthetic mat is discussed in this section. The typical values of C_c range from 0.2~3.0 (Holtz and Kovacs, 1981). As shown in Figure 5.27, the calculation is carried out by taking unit weight of filling slurry of 12 kN/m^3 , the initial void ratio, e_0 , of 1.0, specific gravity, G_s , of 2.7, and effective unit weight of soil subgrade of 8.5 kN/m^3 , external water level on ground surface and the height of geosynthetic mat ranged from 0.1 to 2.0 m. It can be seen that H_f increased almost linearly when C_c is less than 3.0. In order to investigate the influence of void ratio, the value of 0.5 and 1.5 for mat

height of 0.6 m were also calculated as shown in Figure 5.27. It shows that the higher the void ratio, the higher the settlement of geosynthetic mats. This is not only due to the influence to the settlement calculation method when using Equation (5-16) to Equation (5-19) but also because the void ratio influenced the effective unit weight of soil layers.

It should be pointed out that, in the discussion regarding the relationship between C_c and H_f , the perimeter of geosynthetic mat is not taken into consideration because they are independent of perimeter.

Figure 5.27 Relationship between compression index, C_c , and settlement of geosynthetic tube, H_f (For $\gamma=12\text{ kN/m}^3$ and $G_s=2.7$, $OCR=1.0$, $H_w=0.0$ of soil subgrade).

5.4 Numerical Analysis

As discussed before, there are limitations in both the Winkler foundation and e-logp methods. The Winkler foundation method fails to consider the non-uniform distribution of surcharge pressure and non-uniform soil properties. Although the e-logp method could consider the non-uniform effects, it is only suitable for the one-dimensional consolidation settlement. To evaluate the influence of these limitations on the solutions, some numerical analyses are also carried out for comparison.

5.4.1 Outline of Modeling

The numerical analyses are divided into two static steps because gravity is considered in the numerical analysis. The first step is to build the initial equilibrium between gravity, hydraulic pressure distribution and deformation of soil subgrade. The second step is to apply the node forces onto the beam element and transfer the stress to the soil subgrade till the whole system balanced.

The geosynthetic tubes are modeled with 100 beam elements as shown in Figure 5.28. The foundation is modeled as 10 m depth and 20 m width with the minimum grid of $0.1\text{ m} \times 0.1\text{ m}$ right at the bottom of the geosynthetic tubes and range from 8 m in width and 4 m in depth. The rest of the grid is $0.2\text{ m} \times 0.2\text{ m}$. The edges between these two grids are attached with each other. The left and right sides of the grid are allowed to move freely in the vertical direction but fixed in the horizontal direction. The bottom of grids is fixed in both directions. The surfaces between geosynthetic tubes and foundation are modeled with contact pair without considering friction such as contact pair 1 shown in Figure 5.28.

Figure 5.28 Initial shapes of geosynthetic mat and subgrade meshing

In FLAC software, no hydraulic pressure could be applied on the beam element directly. A converted point load is applied on the nodes of beam element by assuming that the beam element is straight in each calculation step as shown in

Figure 5.29. A dynamic analysis is performed by FLAC to converge towards the equilibrium state. The load components at x, y direction in each calculation state is shown in Equations (5-29) to (5-32).

$$P_x = p_{n2} \left(\frac{y_{n+1} - y_n}{2} \right) + p_{n1} \left(\frac{y_n - y_{n-1}}{2} \right) \quad (5-29)$$

$$P_y = p_{n2} \left(\frac{x_{n+1} - x_n}{2} \right) + p_{n1} \left(\frac{x_n - x_{n-1}}{2} \right) \quad (5-30)$$

$$p_{n1} = p_0 + \gamma \left\{ H - \left(y_n - \frac{y_n - y_{n-1}}{4} \right) \right\} \quad (5-31)$$

$$w_{n2} = p_0 + \gamma \left\{ H - \left(y_n - \frac{y_{n+1} - y_n}{4} \right) \right\} \quad (5-32)$$

where γ is the unit weight of filling slurry, H is the height of geosynthetic tube, and p_0 is the initial pumping pressure and $p_0 = 0$, for geosynthetic mat, P_x is the horizontal force due to water pressure and P_y is vertical force, p_x, p_y are the water pressures acting on the two beams.

Figure 5.29 Hydraulic pressure transformation from pressures to point load (after Kim, 2003)

5.4.2 Basic Properties of Materials

The young's modulus of the whole soil subgrade is calculated from the e - $\log p$ curve on the medium height of soil layer when ΔP approached to zero as shown in Equation (5-33) (Helwany, 2007). The calculation is carried with the compression index, C_c , of 0.35 and the unit weight of soil subgrade of 18 kN/m^3 . The ground

water level located on the ground surface. The Poisson's ratio is taken as $\nu = 0.35$. The shear modulus, G and bulk modulus, K , are calculated with the Equations (5-34) and (5-35). The properties of soil subgrade are listed in Table 5.2 and Figure 5.30. The shear stiffness of the interface between geosynthetic mats and foundation is 50 MPa as suggested by Kim et al. (2004). The friction between tube and foundation is not considered.

$$E_u = \frac{(1+\nu)(1-2\nu)}{(1-\nu)} \frac{1+e_0}{\frac{Cc}{\Delta P} \log \frac{P_1 + \Delta P}{P_1}} \quad (5-33)$$

$$K = \frac{E_u}{2(1+\nu)} \quad (5-34)$$

$$G = \frac{E_u}{3(1-2\nu)} \quad (5-35)$$

The perimeter of the calculated geosynthetic tube or mat is 9 m . The thickness of the geosynthetic material, d , is taken as 0.3 mm . The moment of inertia of beam element is calculated by $1.0 \times d^3/12$, which is $2.025 \times 10^{-12} m^4$. The unit weight of filling material is 12 kN/m^3 .

Table 5.2 properties of soil layers

Soil Layer	Elastic Modulus E (kPa)	Bulk modulus, K (kPa)	Shear modulus G (kPa)
2	65.59	72.87	24.29
3	163.96	182.18	60.73
4	229.55	255.06	85.02
5	295.14	327.93	109.31
6	360.72	400.80	133.60
7	426.31	473.67	157.89
8	491.89	546.55	182.18
9	557.48	619.42	206.47
10	623.06	692.29	230.76

Figure 5.30 Properties of soil layers

5.4.3 Preliminary Study

The preliminary was carried out to invest the performance of beam elements. The geosynthetic tube is inflated with pressure of 130 kPa which is the maximum hydraulic pressure will be used in model simulation. The unit weight of filling material is 0 kN/m^3 which means the tube was filled with air. The basic parameters are exactly the same as discussed in section 5.4.2, i.e. $L=9.0\text{m}$, $d=0.3\text{E-}3 \text{ m}$ and $I=2.025 \times 10^{-12} \text{ m}^4$. The initial cross-section of membrane was in circle shape as shown in Figure 5.31.

Figure 5.31 Initial shapes of geosynthetic meshing

The tensile force along the beam elements can easily be calculated with theoretical method such as $p_0^*(L/\pi/2) = 186.3 \text{ kN/m}$. The elastic modulus of geosynthetic sheet used is $7.032\text{E}9 \text{ kPa}$ which is larger enough to make the extension of geosynthetic sheet to be negligible, i.e. $\varepsilon = p_0^*(L/\pi/2)/E/A = 8.83\text{E}-5$. The comparison of tensile force between the theoretical and numerical results is shown in Figure 5.32. It can be seen that the tensile force along beam element was total uniform with their values about 0.056% different from theoretical results.

The comparison of axial strain between the theoretical and numerical results is shown in Figure 5.33. It can be seen that the axial strain along beam element was total uniform with their values about 0.072% different from theoretical results.

(a) Contour plot of axial force along beam elements calculated by FLAC

(b) Comparison between theoretical and FLAC results

Figure 5.32 Comparison of tensile force between numerical and theoretical results

(a) Contour plot of axial strain on beam element

(b) Values of axial strain on beam element

Figure 5.33 Comparison of axial strain between numerical and theoretical results

As shown in Figure 5.34, the moment on two side of the nodes was not very uniform but with the largest values on four corners (nodes numbered 0, 25, 50 and 100). However, the maximum value is only $1.24\text{E-}4 \text{ Nm}$. It is reasonable to assume that the moment loads on beam element could be neglected.

(a) Contour plot of moment on beam node

(b) Values of moment on beam node

Figure 5.34 Moments of the beam nodes calculated by FLAC

5.4.4 Geosynthetic Tube

(1) Numerical Results

The geometry and tensile forces of numerical results of geosynthetic tubes resting on deformable foundation are discussed in this section. Figure 5.35 shows the balanced deformation of soil subgrade and geosynthetic tubes inflated with pumping pressure from 1 to 100 kPa . The cross-section is almost in a flat shape when pumping pressure is small (such as 1 kPa in this example). As the pumping

pressure increases, the geosynthetic tubes are inflated to almost a circle shape and a heavier geosynthetic tube induces a greater settlement.

Figure 5.35 Balance shapes of soil subgrade and geosynthetic tubes (For $C_c=0.35$, $e_0=1.0$, $\gamma=18 \text{ kN/m}^3$ of soil subgrade and $L=9.0 \text{ m}$, $\gamma=12 \text{ kN/m}^3$)

The effective stress distribution within the soil mass is presented in Figure 5.36. It can be seen that the effective stress of the soil layer below 5 m in this model test is almost similar to initial effective stress distribution which means the 10 m depth of soil subgrade is deep enough for modeling. The effective stresses increase heavily underneath the geosynthetic tube which makes the larger contour zone penetrate through the smaller zone especially on the top of soil subgrade. This part is settlement area because of the increased effective stress. The effective stress of the soil area on the side of geosynthetic tube decreases from the initial effective stress distribution which means these areas has uplifting effect. The higher pumping pressure caused larger uplifting area.

(a) Pumping pressure, $p_0 = 1 \text{ kPa}$

(b) Pumping pressure, $p_0 = 10 \text{ kPa}$

(c) Pumping pressure, $p_0=50 \text{ kPa}$

Figure 5.36 Contour of effective stress of soil subgrade and location of geosynthetic tubes (For $C_c=0.35$, $e_0=1.0$, $\gamma=18 \text{ kN/m}^3$ of soil subgrade and $L=9.0 \text{ m}$, $\gamma=12 \text{ kN/m}^3$)

(2) Comparison with the Winkler Foundation Method

The limitation of the Winkler foundation method is that it assumes the soil as infinite uncoupled linear “soil” springs thus cannot consider the nonuniform surcharge pressure distribution and nonuniform soil properties.

The numerical analysis is carried out on the uniform soil subgrade with the young's modulus of 295 kPa selected from the 5th layer shown in Table 5.2. The Poisson's ratio is 0.35 . The shear modulus, G , and bulk modulus, K , are calculated from Equations (5-34) and (5-35) with values of 328 and 109.3 kPa , respectively. The properties of geosynthetic beams are the same as those presented in Section 5.4.2.

The calculation using the Winkler foundation method is carried out with the modulus of subgrade reaction, $K_f = 111 \text{ kPa/m}$, calculated from Equation (5-3) by assuming the area of top foundation $A = 10 \text{ m}^2/\text{m}$. The unit weight of filling slurry is 12 kN/m^3 . The perimeter of the calculated geosynthetic tube is 9 m .

Figure 5.37 Comparison between the numerical and the Winkler foundation method

The cross-sections of geosynthetic tubes obtained from the Winkler foundation method and the numerical method are shown in Figure 5.37. It can be seen that the settlement obtained from the numerical results are slightly smaller than those from the Winkler foundation method.

(a) Height of cross-section versus pumping pressure

(b) Settlement of geosynthetic tube versus pumping pressure

(c) Width of cross-section versus pumping pressure

(d) Area of cross-section versus pumping pressure

Figure 5.38 Comparison between numerical and analytical results

The relationship between heights of geosynthetic tubes and pumping pressure from the Winkler foundation method and the numerical results are also plotted in Figure 5.38(a). The heights from the numerical results are lower than that from the Winkler foundation method. However, as shown Figure 5.38(b), the settlements of geosynthetic tube from the numerical results are greater than that from the Winkler foundation method because the Winkler foundation method cannot consider the nonuniform distribution of the surcharge pressure. The width and area of cross-section obtained from the Winkler foundation method and the numerical method agreed well with each other as show in Figure 5.38(c) and (d).

The comparisons between the tensile forces from the Winkler foundation method and the numerical method are shown in Figure 5.39. Note that the tensile force here is the axial force on the beam nodes which can be outputted directly by FLAC. The results show that the tensile force along geosynthetic tube from the numerical

results is almost uniform. However, in the results of Winkler foundation method, the largest tensile force located on the center of the bottom of cross. The Winkler foundation method is conservative in this respect.

Figure 5.39 Comparison between numerical and analytical results for the tensile force along geosynthetic sheet

(3) Comparison with e-log p Method

The results of e-log p method are compared with the numerical results in this section. The calculation is carried out with the same parameters as discussed in section 5.3.2.

The cross-sections inflated with different pumping pressure by the two methods are shown in Figure 5.40. The heights of geosynthetic tubes from the e-log p method are slightly higher than numerical results as shown in Figure 5.40 and Figure 5.41(a). The width and area of cross-section obtained from e-log p method and numerical method agreed well with each other as show in Figure 5.41(c) and (d). However, settlements of geosynthetic tube from the numerical results are slightly greater than that from the e-log p method. The settlement of geosynthetic tube calculated from the e-log p method is conservative.

Figure 5.40 Comparison of the cross-sections calculated from the numerical and the e-log p method (For $C_c=0.35$, $e_0=1.0$, $\gamma=18 \text{ kN/m}^3$ of soil subgrade and $L=9.0 \text{ m}$, $\gamma=12 \text{ kN/m}^3$)

(a) Height of cross-section versus pumping pressure

(b) Settlement of geosynthetic tube versus pumping pressure

(c) Width of cross-section versus pumping pressure

(d) Area of cross-section versus pumping pressure

Figure 5.41 Comparison of geometry parameters from the numerical and the e-logp method

The tensile forces along geosynthetic tube from the numerical method and the e-logp method are also compared as shown in Figure 5.42. For the results from the numerical method, the tensile forces along geotextile tube from numerical results are almost uniform. For the results from the e-logp method, the largest tensile force locates on the bottom of cross-section which is larger than that from numerical results. Therefore, the tensile forces from the e-logp method are more conservative.

Figure 5.42 Comparison between numerical and analytical results for the tensile force along geosynthetic tube.

5.4.5 Geosynthetic Mat

(1) Numerical Results

The cross-sections of the geosynthetic mat resting on deformable soil foundation inflated to different height are shown in Figure 5.43. As seen from these results, the top line of cross-section is almost straight line as assumed in previous analytical solutions. When the height of geosynthetic mat increases, this straight section decreases but the weight of the geosynthetic mat increases and induces a larger settlement. The effective surcharge pressures to the subgrade are also presented as shown in Figure 5.44. It can be seen that the surcharge pressure induced by the weight of geosynthetic mats only influenced the top 2 m of soil subgrade. For the soil layer deeper than 2 m from the ground surface, the effective stress is almost similar to initial effective stress distributions. The effective stresses increased

heavily underneath the geosynthetic mat but are almost constant on its sides. The lateral deformation of soil subgrade is so small that can be ignored.

Figure 5.43 Displace of soil subgrade and location of geosynthetic mats (For $C_c=0.35$, $e_0=1.0$, $\gamma=18 \text{ kN/m}^3$ of soil subgrade and $L=9.0 \text{ m}$, $\gamma=12 \text{ kN/m}^3$ for geosynthetic mat)

(a) Height of cross-section, $H=0.2m$ (b) Height of cross-section, $H=0.4m$ (c) Height of cross-section, $H=0.6m$

Figure 5.44 Contour of effective stress of soil subgrade and cross-sections of geosynthetic mats (For $C_c=0.35$, $e_0=1.0$, $\gamma=18 \text{ kN/m}^3$ of soil subgrade and $L=9.0 \text{ m}$, $\gamma=12 \text{ kN/m}^3$ for geosynthetic mat)

(2) Comparison with Winkler Foundation Method

The results of cross-sections and tensile forces of geosynthetic mat using the Winkler foundation method and the numerical method are compared in Figure 5.45 and Figure 5.46. The calculation is carried out with the same parameters as that presented in Figure 5.37. It can be seen that the settlement from numerical results are much larger than that from Winkler foundation method especially when height of the geosynthetic mat is of 0.6 m. The reason for this mismatch is that the weight of geosynthetic mat is supported by the whole soil layer in Winkler foundation method but only top several layers for numerical method.

Figure 5.45 Comparison of cross-section of geosynthetic mats between the numerical and Winkler foundation method (For $K_f=111\text{ kPa/m}$, $\gamma=18\text{ kN/m}^3$ of soil subgrade and $L=9.0\text{ m}$, $\gamma=12\text{ kN/m}^3$ for geosynthetic mat)

The comparison of the tensile force along geosynthetic mats from the Winkler foundation method and the numerical method are also shown in Figure 5.46. The results showed that the tensile forces along geosynthetic mats from the numerical results and the Winkler foundation method are not uniform but with the largest tensile force located on the center of the bottom of cross-section. There is significant difference between the two results especially for the geosynthetic mat with height of 0.6 m.

Figure 5.46 Comparison of tensile force between analytical and numerical results (For $K_f=111 \text{ kPa/m}$, $\gamma=18 \text{ kN/m}^3$ of soil subgrade and $L=9.0 \text{ m}$, $\gamma=12 \text{ kN/m}^3$ for geosynthetic mat)

(3) Comparison with e-logp Method

The comparisons of the results from the numerical and e-logp method are shown in Figure 5.47 and Figure 5.48. The calculation is carried out with the same parameters as in Figure 5.43. It can be seen that the cross-sections of geosynthetic mats gotten from the e-logp method and numerical method agree well with each other. We concluded that the one-dimensional consolidation settlement theory is suitable for the geosynthetic mats calculation.

The comparison of the tensile force along geosynthetic mats from the e-logp method and the numerical method are shown in Figure 5.48. The results showed that the tensile force along geosynthetic mat from the numerical results and the e-logp methods are not uniform anymore but with the largest value locating on the center of the bottom of cross-section. Generally, these two sets of results agree well with each other.

Figure 5.47 Comparison of cross-section of geosynthetic mats between numerical and analytical results (For $C_c=0.35$, $e_0=1.0$, $\gamma=18 \text{ kN/m}^3$ of soil subgrade and $L=9.0 \text{ m}$, $\gamma=12 \text{ kN/m}^3$ for geosynthetic mat)

Figure 5.48 Comparison of tensile force between analytical and numerical results (For $C_c = 0.35$, $e_0 = 1.0$, $\gamma = 18 \text{ kN/m}^3$ of soil subgrade and $L = 9.0 \text{ m}$, $\gamma = 12 \text{ kN/m}^3$ for geosynthetic mat)

5.5 Conclusions

The analytical and numerical methods for impermeable geosynthetic tubes and mats resting on deformable foundation were presented in this chapter. By assuming the impermeable geosynthetic mats is watertight and inflated with water or slurry, these methods can be used for calculating impervious geosynthetic tubes, mats or rubber dams. The solution may also be used for permeable geotextile tubes if the consolidation process can be ignored. Based on these theoretical analyses, the following conclusions can be made.

Winkler Foundation Method

- (1) The Winkler foundation method is only necessary to be used for analyzing the geosynthetic tubes resting on a deformable foundation with modulus of subgrade reaction, K_f , less than 1000 kPa/m . When $K_f > 1000 \text{ kPa/m}$, the foundation can be considered rigid. This conclusion is also applicable for geosynthetic mats. The threshold of this K_f is derived from a special case. For

other dimensions of geosynthetic tube or other filling, a new threshold has to be calculated.

- (2) The settlement from the Winkler foundation method is smaller than that from the numerical method using computer software FLAC. This is because the Winkler method cannot consider the non-uniform surcharge distribution within soil mass. The tensile forces predicted these analyses are inconstant but generally the analytical results were conservative.

e-log p Method

- (3) A method using the e-log p relationship as determined by one-dimensional consolidation tests was proposed. In theory, this method has the following advantages over the Winkler method: a). the parameters involved in this method such as compression index and over consolidated ratio can be obtained from laboratory tests; b) the deformation of soil is modeled as nonlinear; c) the variation of surcharge pressure within soil mass can be considered.
- (4) The largest settlement of the geosynthetic tube calculated by the e-log p method is when external water level locates on the ground surface. Therefore, the settlement of geosynthetic tube calculated by the e-log p method without considering external water level is conservative.
- (5) The settlements of geosynthetic tube calculated by the e-log p method are in good agreement with those obtained from the numerical results although the later are slightly greater. The settlements of geosynthetic mat calculated by the e-log p method also agreed well with those from the numerical results.

CHAPTER 6

ANALYSIS OF TWO-LAYER STACKED IMPERMEABLE GEOSYNTHETIC TUBES

6.1 Introduction

As reviewed in Chapter 2, geosynthetic tubes can be stacked in several layers to construct a higher dike or to dewater waste sludge. Two cases, (1) one tube stacks on top of another and (2) one tube straddles two tubes resting on rigid and deformable foundation were proposed and analyzed by Plaut & Suherman (1998) and Plaut & Klusman (1999). Klusman (1998) also analyzed the 2-layers stacked geosynthetic tubes with one on top of another. It was assumed in all the above methods that all the geosynthetic tubes have the same perimeter. Furthermore, the existing methods cannot be directly applied to the case when the pumping pressure of the top geosynthetic tube is zero. Therefore, there is a need to develop analytical methods that are suitable for more general conditions.

As discussed in Chapter 3, when two geosynthetic tubes are stacked one on top of another, the contact surface between the two tubes has to be in one of the three cases: horizontal, convex, or concave contact. Theoretical solutions for each case are derived and presented in this chapter. The solutions are developed by assuming the geosynthetic tubes are impermeable or watertight and inflated with same water or slurry. For this reason, the solutions developed in this chapter are only applicable to stacked impermeable geosynthetic tubes. Comparisons between the proposed

solutions and the experimental results and with the solutions obtained by Klusman (1998) are also made in this chapter.

6.2 Basic Assumptions

To derive the solutions, the following assumptions were made:

- (1) The geosynthetic tubes are sufficiently long to be assumed as a plane strain problem;
- (2) The geosynthetic shell is thin and flexible, and its weight and extension are neglected;
- (3) Frictions between the geosynthetic tube and the fill material, and that between the geosynthetic tube and rigid foundation are neglected;
- (4) The tensile stress along the circumference of geosynthetic tube is constant;
- (5) All the geosynthetic tubes are filled with the same kind of liquid material.

Some of the above hypotheses were also made in the existing theoretical solutions for analysis of the stacked geosynthetic tubes (Klusman, 1998; Plaut and Suherman, 1998; Plaut and Klusman, 1999).

6.3 Horizontal Contact

Two stacked geosynthetic tubes will form a horizontal contact when the water pressure acting down and up on the contact surface are balanced, or $p_1 + \gamma H_1 = p_2$, as shown in Figure 6.1. The calculation method for the top tube is the same as that for geosynthetic tube resting on rigid foundation as discussed in Chapter 4. It should be pointed out that, although the bottom tube has a horizontal section on its top surface, it is different from the geosynthetic mat because its pumping pressure here is not zero.

$$p_2 = p_1 + \gamma H_1 \quad (6-1)$$

Figure 6.1 Pressure acting down and up on the horizontal contact surface between stacked tubes resting on rigid foundation (for $p_1 > 0$, $p_2 > 0$)

6.3.1 Derivation

A simplified cross-sectional diagram of the two stacked tubes with horizontal contact surface is shown in Figure 6.2. The geosynthetic tubes are considered to be inflated with only one type of water or slurry with a unit weight of γ . The coordinates are set up with x in the vertical and y in the horizontal direction. The origin of the coordinate system is taken as the top center point of the cross-section. The height of the geosynthetic tubes on the top (Tube-1) is denoted as H_1 and that of the tube at the bottom (Tube-2) as H_2 . The pumping pressures in the top and the bottom tubes are defined as p_1 and p_2 , respectively. L_C is used to denote the length of the horizontal contact edge between two tubes, measured between point C and C' . The tensile forces along the tubes are denoted as T_1 along Tube-1 and T_2 along Tube-2, which are assumed to be uniform since the frictions are not considered.

Figure 6.2 Cross-section of two stacked geosynthetic tubes in horizontal contact surface (for $p_1 > 0$, $p_2 > 0$)

As shown in Figure 6.2, the cross-section of the two layers stacked geosynthetic tubes could be divided into four sections. OC is denoted the curve section of Tube-1. MC and ND are horizontal sections. CD is the curve section of Tube-2. The other side of the cross-section can be mirrored by x -axis because of the symmetry.

The tensile force along Tube-1 is calculated by the equilibrium of the forces acting on the curve OC . A free body diagram of section OC used for force equilibrium analysis is presented in Figure 6.3. As the problem is symmetric, the tensile force (T_1) at the top of the cross-section (point O) and that at the bottom (point C) are both in horizontal direction. The hydraulic pressure acting on the free body OC is shown in Figure 6.3(a). The forces acting along the horizontal direction only involve the horizontal component of hydraulic pressure and the tensile forces as presented in Figure 6.3(b). The equilibrium of the two forces acting on horizontal direction yields the expression of tensile force shown as follows:

$$T_1 = \frac{1}{2}(p_1 H_1 + \frac{1}{2} \gamma H_1^2) \quad (6-2)$$

(a) The free body diagram of curve OC (b) Horizontal forces acting on curve OC

Figure 6.3 Free body diagram of curve OC

Figure 6.4 presents the free body diagram of an arbitrary point $S_I(x, y)$ on section OC from Figure 6.2. The angle between the tangential direction of curve OC at point $S_I(x, y)$ and the y -axis is denoted as θ . The two geometrical equations relating the angle, θ , and the x and y coordinates are given in Equations (6-3) and (6-4). By denoting the internal pumping pressure of the Tube-1 as p_1 , the hydraulic pressure acting internally at point $S_I(x, y)$ can be calculated as $p_1 + \gamma x$ where x is the x -

coordinate. The equilibrium of the forces acting at the point in the normal and tangential directions yields the expressions of the differential equations shown as Equations (6-5) and (6-6).

Figure 6.4 The equilibrium of the forces acting on a point $S_I(x, y)$

$$\frac{dx}{ds} = \sin \theta \quad (6-3)$$

$$\frac{dy}{ds} = \cos \theta \quad (6-4)$$

$$\frac{d\theta}{ds} = \frac{1}{T_I} (p_I + \gamma x) \quad (6-5)$$

$$\frac{dT_I}{ds} = 0 \quad (6-6)$$

The following boundary conditions for section OC are introduced to solve Equations (6-2), (6-3), (6-4), (6-5) and (6-6):

- 1). when $x = 0$, $y = 0$, $\theta = 0$, as point O in Figure 6.3(b);
- 2). when $x = H_I$, $y = L_C$, $\theta = \pi$, as point C in Figure 6.3(b);
- 3). $L_C = \gamma A_I / (p_I + \gamma H_I)$, where A_I is the cross-sectional area of Tube-1.

Similar to section OC , the free body diagram of section CD from Figure 6.2 is plotted in Figure 6.5(a). The hydraulic pressure acting internally on a point $S_2(x, y)$ from Figure 6.2 is $p_2 + \gamma (x - H_I)$. Based on Equation (6-1), the internal pumping pressure of Tube-2 is calculated as $p_2 = p_1 + \gamma H_I$. Therefore, the water pressure

acting at point $S_2(x, y)$ is expressed as $p_1 + \gamma x$ which is the same as that applied at point $S_1(x, y)$ from curve OC . As a result, the differential equations on CD are the same as that on OC except the tensile force is T_2 as shown in Equation (6-7) and (6-8). The equilibrium of hydraulic water pressure and tensile forces in horizontal direction yields the expression of tensile force along Tube-2 as shown in Equation (6-9).

$$\frac{d\theta}{ds} = \frac{p_1 + \gamma x}{T_2} \quad (6-7)$$

$$\frac{dT_2}{ds} = 0 \quad (6-8)$$

$$T_2 = \frac{1}{2} (p_1 H_2 + \gamma H_1 H_2 + \frac{1}{2} \gamma H_2^2) \quad (6-9)$$

(a) The free body diagram of curve CD (b) Horizontal forces on curve CD

Figure 6.5 Free body diagram of curve CD

The boundary conditions needed to solve Equations (6-7), (6-8) and (6-9) are:

- 1). when $x=H_1$, $y = L_C$, $\theta = 0$, as point C in Figure 6.5(a);
- 2). when $x=H_1+H_2$, $y = b/2$, $\theta = \pi$, as point D in Figure 6.5(a), where b is contact width with ground.

In addition to the boundary conditions above, some other force equilibrium equations are also needed to be satisfied as shown in Equations (6-10) and (6-11).

$$\gamma(A_1 + A_2) = (p_1 + \gamma H_1 + \gamma H_2)b \quad (6-10)$$

$$\gamma A_1 = (p_1 + \gamma H_1)L_c \quad (6-11)$$

6.3.2 Numerical Calculation

In the case of horizontal contact between two stacked geosynthetic tubes, the differential Equations (6-3), (6-4), (6-5), (6-6), (6-7) and (6-8), as well as tensile force Equations (6-2) and (6-9) can be solved by combining all the initial and boundary conditions as discussed in previous sections. The input parameters are: the unit weight of filling slurry, γ , the perimeters of two geosynthetic tubes, L_1 and L_2 , and the heights of the bottom and top tubes H_2 & H_1 . The unknown parameters are as follows: contact length between two tubes, L_c , contact width with ground, b , pumping pressure of Tube-1, p_1 , and Tube-2, p_2 . It should be pointed out that the inputs parameters, H_1 , H_2 , and unknown parameters, p_1 and p_2 are not restricted. Any two parameters among those four can be used as inputs and then the other two will be taken as unknown parameters.

To solve the differential equations by iteration, a mathematical program called SGeoTub_H using Complex Method (Box, 1965) was developed. The method was summarized by Lipson (1977) as discussed in section 5.3.2. In order to achieve high accuracy, the calculation process terminated when the differences of the boundary conditions were less than $1.0E-5$.

6.3.3 $p_1 = 0$ Case

As shown in Figure 6.6, when the pumping pressure of top tube, p_1 , is zero, Tube-1 becomes flat which can also be called geosynthetic mat according to the terminology defined in Chapter 3. Substituting $p_1 = 0$ into Equations (6-5) and (6-6), the domain differential equations obtained are shown as Equations (6-12) and (6-13). The expression of tensile force along Tube-1 while $p_1 = 0$ is presented in Equation (6-14).

$$\frac{d\theta}{ds} = \frac{\gamma x}{T_1} \quad (6-12)$$

$$\frac{dT_1}{ds} = 0 \quad (6-13)$$

$$T_1 = \gamma H_1^2 / 4 \quad (6-14)$$

Figure 6.6 Cross-section of two stacked geosynthetic tubes in horizontal contact surface (for $p_1 = 0, p_2 > 0$)

The following boundary conditions are introduced to solve Equations (6-3), (6-4), (6-12), (6-13) and (6-14):

- 1). when $x = 0, y = y_0, \theta = 0$, as point A in Figure 6.6, where y_0 is a horizontal section on top of Tube-1;
- 2). when $x = H_1, y = L_C, \theta = \pi$, as point C in Figure 6.6;
- 3). $L_c = \gamma A_1 / \gamma H_1$, where A_1 is the cross-sectional area of Tube -1.

For the calculation of curve CD from Tube-2, the hydraulic pressure acting internally on the curve section is γx because the pumping pressure on top tube is zero. Substituting $p_1 = 0$ into Equations (6-7), (6-8) and (6-9) yields the differential equations for the bottom tube shown as follows:

$$\frac{d\theta}{ds} = \frac{\gamma x}{T_2} \quad (6-15)$$

$$\frac{dT_2}{ds} = 0 \quad (6-16)$$

$$T_2 = \frac{1}{2}(\gamma H_1 H_2 + \frac{1}{2} \gamma H_2^2) \quad (6-17)$$

The boundary conditions of Tube-2 are the same as the case of geosynthetic tube on top as discussed in Section 6.3.1. By substituting $p_1 = 0$ into Equations (6-10) and (6-11), the boundary conditions can be derived as shown below:

$$\gamma(A_1 + A_2) = \gamma(H_1 + H_2)b \quad (6-18)$$

$$\gamma A_1 = \gamma H_1 L_c \quad (6-19)$$

The differential Equations (6-3), (6-4), (6-12), (6-13), (6-15) and (6-16), the tensile forces (6-14) and (6-17) as well as force equilibrium can be solved by combining all the boundary conditions in the same way as discussed in Section 6.3.2 using computer program SGeoTub_H except to use the initial horizontal length of top tube, y_0 instead of the pumping pressure of the top tube, p_1 .

6.4 Convex Contact

Two stacked geosynthetic tubes will form a convex contact when the water pressure acting up on the contact surface, p_B , is greater than that acting down on the contact surface, p_T , i.e. $p_B > p_T$, as shown in Figure 6.7.

Figure 6.7 Pressure acting down and up on the convex contact surface between stacked tubes resting on rigid foundation (for $p_1 > 0$, $p_2 > 0$)

6.4.1 Derivation

In order to derive the solutions for the case of convex contact surface as shown in Figure 6.7, a simplified cross-sectional diagram as shown in Figure 6.8 defines all the parameters, the same as the case in Section 6.3.

Figure 6.8 Cross-section of two stacked geosynthetic tubes with convex contact surface resting on rigid foundation (for $p_1 > 0, p_2 > 0$)

Figure 6.9(a) presents a free body diagram of a point $S_2(x, y)$ on section CC' from Figure 6.8. The water pressure acting down on the point is $p_1 + \gamma x$ and that acting up is $p_2 + \gamma(x - H_1)$. The radius of curvature on the point can be derived as shown in Equation (6-20). It can be seen that the radius on this point $S_2(x, y)$ is independent of position x or y .

$$\frac{1}{R} = \frac{d\theta}{ds} = \frac{(p_1 + \gamma x) - (p_2 + \gamma(x - H_1))}{T_1 + T_2} = \frac{(p_1 - p_2 + \gamma H_1)}{T_1 + T_2} \quad (6-20)$$

Therefore, the section CC' is an arc with the radius of R and the central angle of 2α as shown in Figure 6.9(b). The radius, R , can be calculated with the pumping pressure of two tubes, p_1 and p_2 , the height of the top tube, H_1 , and the tensile force of two tubes, T_1 and T_2 . The value of α was connected with the cross-section of stacked geosynthetic tubes with the relationship of $\alpha = \theta_c - \pi$, where θ_c is the angle between tangential direction at point C and y -axis. For the convex contact, the angle θ_c was greater than π , i.e. $\theta_c > \pi$. The length of arc, L_c , equals to $2R(\theta_c - \pi)$.

It should be pointed out that the radius of the arc, R , in this case is defined to be negative in Equation (6-20), because the hydraulic water pressure acting down on the contact surface is less than that acting up, i.e. $p_1 + \gamma H_1 < p_2$. The details have been discussed in the definition of convex contact surface above. The formula of arc

CC' are shown in Equation (6-21) with the range of x from H_1 to $H_1 - R(1-\cos\alpha)$ and of y from $R\sin\alpha$ to $-R\sin\alpha$ where α is half the central angle of arc CC' .

$$(x - (-R + H_1))^2 + y^2 = R^2 \quad (6-21)$$

where, $H_1 < x < H_1 - R(1-\cos\alpha)$, $R\sin\alpha < y < -R\sin\alpha$ and $\alpha = \theta_c - \pi$, α is half of the central angle of CC' , R is the radius of arc CC' , θ_c is the angle between the tangent of point C and y -axis.

Figure 6.9 Free body diagram of curve CC' and a point $S_2(x, y)$

The tensile force along Tube-1, T_1 , is calculated by the equilibrium of the forces acting on the curve OC . A free body diagram of section OC from Figure 6.8 used for force equilibrium analysis is presented in Figure 6.10(a). In the horizontal direction, it only involves the horizontal component of hydraulic water pressure and tensile force as indicated in Figure 6.10(b). The equilibrium between these two forces yields the expression T_1 as shown in Equation (6-22). The basic differential equations for curve OC are the same as the case of horizontal contact surface as listed in Equations (6-3), (6-4), (6-5) and (6-6). It should be pointed out that the angle between the tangential direction of point C and the y -axis, θ_c , is larger than π as shown in Figure 6.10(a) in this case.

$$T_1 = (P_1(H_1 - R(1 - \cos\alpha)) + \frac{1}{2}\gamma(H_1 - R(1 - \cos\alpha))^2)/(1 + \cos\alpha), \quad R < 0 \quad (6-22)$$

where α is half of the central angle and $\alpha = \theta_c - \pi$, R is the radius of the convex arc.

(a) The free body diagram of curve OC (b) Horizontal forces on curve OC

Figure 6.10 Free body diagram of curve OC from Tube-1

The following boundary conditions for section OC are established to solve Equations (6-3), (6-4), (6-5), (6-6) and (6-22):

- 1). when $x = 0$, $y = 0$, $\theta = 0$, as point O in Figure 6.10(a);
- 2). when $x = H_1 - R(1 - \cos \alpha)$, $y = -R \sin \alpha$, $\theta = \theta_c$, as point C in Figure 6.10(a).

The free body diagram of section CD on Tube-2 in Figure 6.8 is plotted in Figure 6.11. The forces in the horizontal direction only involve the horizontal component of hydraulic water pressure and tensile forces. Based on the force equilibrium in horizontal direction, the tensile force along Tube-2 can be calculated as follows,

$$T_2 = (P_2(H_2 + R(1 - \cos \alpha)) + \frac{1}{2} \gamma(H_2 + R(1 - \cos \alpha))^2) / (1 + \cos \alpha), \quad R < 0 \quad (6-23)$$

where α is half of the central angle and $\alpha = \theta_c - \pi$, R is the radius of the convex arc.

The free body diagrams of an arbitrary point $S_3(x, y)$ in Figure 6.8 are plotted in Figure 6.12. The hydraulic pressure acting internally on this point is $p_2 + (\gamma x - H_1)$ in which p_2 is the pumping water pressure of Tube-2. The force equilibriums in the normal and tangential directions of points, $S_3(x, y)$, yield the differential equations of curve CD shown as follows,

$$\frac{d\theta}{ds} = \frac{p_2 + \gamma(x - H_1)}{T_2} \quad (6-24)$$

$$\frac{dT_2}{ds} = 0 \quad (6-25)$$

Figure 6.11 Free body diagram of curve CD

Figure 6.12 Free body diagram of point $S_3(x, y)$

The following boundary conditions for Tube-2 were established to solve Equations (6-24), (6-25) and (6-23):

- 1). when $x=H_1-R(1-\cos\alpha)$, $y = -R\sin\alpha$, $\theta = \theta_c-\pi$, point C in Figure 6.11;
- 2). when $x=H_1+H_2$, $y = b/2$, $\theta = \pi$, point D in Figure 6.11, where b is contact width with ground.

In addition to the boundary conditions discussed above, some other force equilibrium equations have to be satisfied shown as below,

$$\gamma(A_1 + A_2) = (p_2 + \gamma H_2)b \quad (6-26)$$

$$\gamma A_1 = 2 \int_0^{-R\sin\alpha} (p_2 + \gamma(x - H_1))dy \quad (6-27)$$

For the case of convex contact between two stacked geosynthetic tubes, the Equations (6-20) and (6-21) for contact arc, CC' , the differential Equations (6-3), (6-4) (6-5), (6-6), (6-24) and (6-25), as well as the tensile forces Equations (6-22), (6-23) can be solved by combining all the boundary conditions as discussed above. The input parameters are the unit weight of filling slurry, γ , the perimeter of cross-section of two geosynthetic tube, L_1 and L_2 , the height of the top tube, H_1 and of the bottom tube, H_2 . The unknown parameters include the radius or the length of contact surface, R or L_c , contact width with ground, b , pumping pressure, p_1 , of Tube-1 and p_2 of Tube-2. It should be pointed out that the inputs parameters, H_1 , H_2 , and unknown parameters, p_1 , p_2 , are not restricted. Any two parameters among these four can be used as inputs and then the other two will be taken as unknowns. Again the program SGeoTub_Cx is used to solve those equations.

6.4.2 $p_1 = 0$ Case

When the pumping pressure of top tube, p_1 , is zero, the Tube-1 becomes flat as shown in Figure 6.13 which can also be called geosynthetic mat according to the terminology used in this study. The radius of convex contact arc can be calculated by substituting $p_1 = 0$ into Equation (6-20) as shown in Equation (6-28). The equation of this arc is the same as the one of two stacked tubes, i.e. Equation (6-21). The differential equations of curve OC are the same as 2-layer stacked geosynthetic tubes in horizontal contact, i.e. Equations (6-12) and (6-13). Substituting $p_1 = 0$ into Equation (6-22) gives the expression of tensile force along Tube-1 as shown in Equation (6-29).

Figure 6.13 Cross-section of two stacked geosynthetic tubes in convex contact surface (for $p_1 = 0$, $p_2 > 0$)

$$\frac{1}{R} = \frac{d\theta}{ds} = \frac{\gamma x - (p_2 + \gamma(x - H_1))}{T_1 + T_2} = \frac{(\gamma H_1 - p_2)}{T_1 + T_2} \quad (6-28)$$

$$T_1 = \frac{1}{2} \gamma (H_1 - R(1 - \cos \alpha))^2 / (1 + \cos \alpha), \quad R < 0 \quad (6-29)$$

The following boundary conditions for Tube-1 are used to solve Equations (6-3), (6-4), (6-12) and (6-13):

- 1). when $x = 0$, $y = y_0$, $\theta = 0$, as point A in Figure 6.13, where y_0 is horizontal section on top of Tube-1;
- 2). when $x = H_1 - R(1 - \cos \alpha)$, $y = R \sin \alpha$, $\theta = \theta_c$, as point C in Figure 6.13.

The basic differential equations for the calculation of curve CD in Figure 6.13 are the same as two stacked tubes with convex contact as shown in Equations (6-24), (6-25) and (6-23). The following boundary conditions of Tube-2 are needed to solve these equations:

- 1). when $x = H_1 - R(1 - \cos \alpha)$, $y = R \sin \alpha$, $\theta = \theta_c - \pi$, as point C of Tube-2;
- 2). when $x = H_1 + H_2$, $y = b/2$, $\theta = \pi$, as point D of Tube-2.

In order to solve Equations (6-21) and (6-28), the differential Equations (6-3), (6-4), (6-12), (6-13), (6-24), (6-25) and tensile forces Equations (6-23) and (6-29), all the initial and boundary conditions are combined and applied in the same method discussed in section 6.4.1. The solving process is done by the computer program named SGeoTub_Cx except to use the initial horizontal length of top tube, y_0 instead of the pumping pressure of the top tube, p_1 .

6.5 Concave Contact

Two stacked geosynthetic tubes will form a concave contact surface when the water pressure acting down on the contact surface, p_B , is smaller than that acting up on the contact surface, p_T , i.e. $p_T > p_B$, as shown in Figure 6.14.

Figure 6.14 Pressure acting down and up on the concave contact surface between stacked tubes resting on rigid foundation (for $p_1 > 0, p_2 > 0$)

6.5.1 Derivation

In order to derive the solutions for the case of concave contact surface as shown in Figure 6.14, a simplified cross-section as shown in

Figure 6.15 is used to define all the parameters, the same as the case in Section 6.3.

It can be seen that the highest point on the bottom tube is no longer at point M but at point E and E' . In order to be consistent with the previous theoretical derivations, the pumping pressure on Tube-2, p_2 , is still defined as the water pressure on the top of the symmetric axis, i.e. point M in

Figure 6.15.

Figure 6.15 Cross-section of two stacked geosynthetic tubes in a concave contact (for $p_1 > 0, p_2 > 0$)

Figure 6.16(a) presents a free body diagram of a point $S_2(x, y)$ on section CC' from Figure 6.15. The water pressure acting down on the point is $p_1 + \gamma x$ and that acting up is $p_2 + \gamma(x - H_1)$. The radius of curvature on the point can be derived as shown in Equation (6-30). It can be seen that the radius on this point $S_2(x, y)$ is independent of position x or y .

$$\frac{1}{R} = \frac{d\theta}{ds} = \frac{(p_1 + \gamma x) - (p_2 + \gamma(x - H_1))}{T_1 + T_2} = \frac{(p_1 + \gamma H_1 - p_2)}{T_1 + T_2} \quad (6-30)$$

Therefore, the section CC' is an arc with the radius of R and the central angle of 2α as shown in Figure 6.16(b). The radius, R , can be calculated with the same method as the case of convex context discussed in section 6.4.1. It can be seen that the radius of the arc, R , is greater than zero in this case as the water pressure on top of the contact part is greater than that of the bottom. The value of α was connected with the cross-section of stacked geosynthetic tubes by the relationship of $\alpha = \pi - \theta_c$, where θ_c is the angle between tangential direction at point C and y -axis. For the concave contact, the angle θ_c was less than π , i.e. $\theta_c < \pi$. The length of arc, L_c , equals to $2 R (\pi - \theta_c)$. The range of x -value is from $H_1 - R (1 - \cos\alpha)$ to H_1 . The range of the y -value is from $-R \sin\alpha$ to $R \sin\alpha$. The formula of the arc, CC' , is shown in Equation (6-31).

$$(x + (R - H_1))^2 + y^2 = R^2 \quad (6-31)$$

where, $H_1 - R (1 - \cos\alpha) < x < H_1$, $-R \sin\alpha < y < R \sin\alpha$ and $\alpha = \pi - \theta_c$.

Figure 6.16 Free body diagram of curve CC'

For curve OC as shown in Figure 6.17(a), the basic differential equations are the same as the case of horizontal contact surface between two tubes which are listed in Equations (6-3), (6-4), (6-5) and (6-6). The tensile force, T_1 , is calculated by the equilibrium of the horizontal components of hydraulic water pressure and tensile force acting on curve OC as shown in Figure 6.17(b). The expression of tensile force along Tube-1, T_1 , is shown as follows,

$$T_1 = (P_1(H_1 - R(1 - \cos \alpha)) + \frac{1}{2} \gamma (H_1 + R(1 - \cos \alpha))^2) / (1 + \cos \alpha), \quad R > 0 \quad (6-32)$$

where α is half of the central angle of the arc and $\alpha = \pi - \theta_c$, R is the radius of CC' .

(a) The free body diagram of curve OC

(b) Horizontal forces on curve OC

Figure 6.17 Free body diagram of curve OC from Tube-1

The following boundary conditions of Tube-1 are established to solve Equations (6-3), (6-4), (6-5), (6-6) and (6-32),

- 1). when $x = 0, y = 0, \theta = 0$, as point O in Figure 6.17(a);
- 2). when $x = H_1 - R(1 - \cos \alpha), y = R \sin \alpha, \theta = \theta_c$, as point C in Figure 6.17(a).

The free body diagram of section CD is plotted in Figure 6.18. The expressions of differential equations of curve CD are the same as the case of convex contact which are listed in Equations (6-24) and (6-25). The expression of tensile force along Tube-2 is shown as follows,

$$T_2 = (P_2(H_2 + R(1 - \cos \alpha)) + \frac{1}{2} \gamma (H_2 + R(1 - \cos \alpha))^2) / (1 + \cos \alpha), \quad R > 0 \quad (6-33)$$

where α is half of the central angle of the arc and $\alpha = \pi - \theta_c$. R is the radius of CC' .

The following boundary conditions of Tube-2 can be established to solve Equations (6-24), (6-25) and (6-33):

- 1). when $x = H_1 - R(1 - \cos \alpha)$, $y = R \sin \alpha$, $\theta = \theta_c - \pi$, as point C in Figure 6.18(a);
- 2). when $x = H_1 + H_2$, $y = b/2$, $\theta = \pi$, as point D in Figure 6.18(a), where b is contact width with ground.

(a) The free body diagram of curve CD (b) Horizontal forces on curve CD

Figure 6.18 Free body diagram of curve CD

Similar to convex contact case, the Equations (6-30), (6-31) for contact arc, the differential Equations (6-3), (6-4) (6-5), (6-6), (6-24) and (6-25), as well as the tensile forces Equations (6-32), (6-33) can be solved by combining all the boundary conditions as discussed above. The inputs and outputs parameters are the same as that discussed for the case of convex contact in section 6.4.1. A computer program named SGeoTub_Cv is developed to solve those equations.

6.5.2 $p_1 = 0$ Case

When $p_1 = 0$, the top tube is flat geosynthetic tube as shown in Figure 6.19 which can also be called geosynthetic mat according to the terminology used in this study.

The derivation of the equations is the same as the case of convex contact which has already been presented in section 6.4.2 except the radius of the contact arc and tensile force along top tube are different. The radius of concave contact arc can be calculated by substituting $p_1 = 0$ into (6-30) as shown in Equation (6-34). The formula of the arc, CC' , is the same as two stacked tubes in concave contact as shown in Equation (6-31). The tensile force along the top geosynthetic tube was derived by substituting $p_1 = 0$ into Equation (6-32) as shown in Equation (6-35).

$$\frac{1}{R} = \frac{d\theta}{ds} = \frac{\gamma x - (p_2 + \gamma(x - H_1))}{T_1 + T_2} = \frac{(\gamma H_1 - p_2)}{T_1 + T_2} \quad (6-34)$$

$$T_1 = \frac{1}{2} \gamma (H_1 + R(1 - \cos \alpha))^2 / (1 + \cos \alpha), \quad R > 0 \quad (6-35)$$

Figure 6.19 Cross-section of two stacked geosynthetic tubes in concave contact surface (for $p_1 = 0, p_2 > 0$)

For sections OC and CD , the basic equations and boundary conditions are the same as 2-layer stacked geosynthetic tubes in convex contact discussed in section 6.4.2. The solving process is done by the computer program *SGeoTub_Cv* except to use the initial horizontal length of top tube, y_0 instead of the pumping pressure of the top tube, p_1 .

6.6 Comparisons

6.6.1 Comparisons with Existing Solutions

The results obtained from the proposed computer program are compared with the solutions given by Klusman (1998). In his method, the parameters were normalized

by dividing the height of two tubes, H_1 and H_2 by the length of cross section, L ; the water head, H_w , by L ; tensile force, T , by γL^2 . The unit weight of filling slurry was assumed to be 1.0 kN/m^3 . In order to compare with his results, the calculations were carried out by taking the perimeters of the two tubes, L , to be 1 m and the unit weight of the slurry to be 1.0 kN/m^3 . The total head in top and bottom tube, $H_{w1}=H_{w2}=0.6 \text{ m}$, was also taken as inputs. The pumping pressures were calculated by subtracting the total head to the trial height of two tubes during iteration. The comparison of the cross-sections with horizontal contact between Klusman's results and the presented theoretical method is made in Figure 6.20. It can be seen that a good agreement is achieved between the two solutions.

Figure 6.20 Comparison of the horizontal contact case with Klusman's method
(For $L_1=L_2=1.0 \text{ m}$, $H_{w1}=H_{w2}=0.6 \text{ m}$, $\gamma=1.0 \text{ kN/m}^3$)

The comparisons of the cross-sections with the convex contact between Klusman's results and the present theoretical method are shown in Figure 6.24. Two cases were compared: 1) $H_{w1}=0.4 \text{ m}$, $H_{w2}=0.6 \text{ m}$ and (b) $H_{w1}=0.5 \text{ m}$, $H_{w2}=0.6 \text{ m}$.

Figure 6.21 Comparisons of the convex contact with Klusman's method (For $L_1=L_2=1.0 \text{ m}$, $\gamma=1.0 \text{ kN/m}^3$, $H_{w2}=0.6 \text{ m}$, (a) $H_{w1}=0.4 \text{ m}$, and (b) $H_{w1}=0.5 \text{ m}$)

Figure 6.24 shows the comparison between Klusman's results and the presented method for the case of concave contact. Results based on two different inputs of H_{w2} values were compared: 1) $H_{w1}=0.6 \text{ m}$, $H_{w2}=0.4 \text{ m}$ and (b) $H_{w1}=0.6 \text{ m}$, $H_{w2}=0.5 \text{ m}$.

Figure 6.22 Comparisons of the concave contact with Klusman's method (For $L_1=L_2=1.0 \text{ m}$, $\gamma=1.0 \text{ kN/m}^3$, $H_{w1}=0.6 \text{ m}$, (a) $H_{w2}=0.4 \text{ m}$, and (b) $H_{w2}=0.5 \text{ m}$)

6.6.2 Comparisons with Experimental Results

In order to verify the validity and accuracy of the proposed solutions for 2-layer stacked geosynthetic tubes resting on rigid foundation, the solutions are compared with the model test results. Only several typical cases are presented in this section and more comparisons are made in Appendix A.

(1) Concave Contact

A comparison of the theoretical prediction and the test results of Model Test ST2-9 which was a concave contact case are shown in Figure 6.23. The detailed testing procedures and test results were already discussed in Sections 3.8.2. The prediction was carried out by taking some inputs such as the perimeters of top tube, $L_1 = 3 \text{ m}$, and bottom tubes, $L_2 = 4 \text{ m}$, the unit weight of water, $\gamma_w = 9.81 \text{ kN/m}^3$, the total head in the top tubes, $H_{w1} = 0.868 \text{ m}$ and the bottom tube $H_{w2} = 0.69 \text{ m}$. The pumping pressure on top and bottom tube were calculated taking their total head subtract the trial heights of two tubes during iteration. The contact length between two tubes, L_c , of 1.1064 m , the contact width with ground, b , of 1.6328 m , height of top tube, H_1 , of 0.6585 m and height of bottom tube, H_2 , of 0.13 m are the iterating results.

Figure 6.23 Comparison of the cross-sections between experimental and theoretical results for Model Test ST2-9

A comparison of predicted tensile forces and the rest results of Model Test ST2-9 is shown in Figure 6.24. The predicted tensile forces of the top and bottom tubes are

0.8962 and 0.8224 kN/m , respectively. The comparison in Figure 6.24 shows that the tensile force of the top tube is over predicted, whereas the tensile force of the bottom tube is under predicted.

Figure 6.24 Comparison the tensile forces between experimental and theoretical results for Model Test ST2-9

The theoretical prediction and the test results of Model Test ST2-2 for a concave contact surface are compared in Figure 6.25. In the analytical analysis, the following input parameters were used: the perimeters of the two geosynthetic tubes of 3 and 4 m respectively, the unit weight of water of 9.81 kN/m^3 , the total head for the top tube $H_{w1} = 0.4 \text{ m}$ and the total head for the bottom tube, $H_{w2} = 0.385 \text{ m}$. From the prediction, the contact length between the two tubes, are $L_c = 1.365 \text{ m}$, the length of the horizontal section, $y_0 = 0.734 \text{ m}$, the contact width with ground, $b = 1.67 \text{ m}$, and the height of bottom tube, $H_l = 0.1522 \text{ m}$ were obtained.

Figure 6.25 Comparison of cross-sections between experimental results and analytical results for Model Test ST2-2

Figure 6.26 Comparison of tensile forces between experimental results and analytical predictions for Model Test ST2-2

Figure 6.26 presents the comparison between tensile forces from theoretical prediction and laboratory test results. The tensile forces of top and bottom tube from theoretical prediction are 0.0627 and 0.319 kN/m respectively. It is found that the tensile forces recorded from the laboratory test are not in linear relationship but scattered in the diagram. The tensile force is overestimated by the analytical prediction, which is too conservative.

(2) Convex Contact

The theoretical prediction on convex contact was carried out in order to compare with the results of Model Test ST1-16 as shown in Figure 3.31. In theoretical prediction, the following input parameters were adopted: the perimeter of two geosynthetic tubes were 2 and 3 m respectively; the unit weight of filling water was 9.81 kN/m^3 ; the total height was 0.9 m and the total head of top and bottom tube were $H_{w1}=1.279 \text{ m}$, $H_{w2}=0.938 \text{ m}$.

After solving the case using SGeoTub_Cx, the output parameters calculated are listed as follows: The contact length between two tubes, L_c , was 0.5888 m , the contact width with ground, b , was 0.6635 m , and height of top and bottom tube were 0.2923 and 0.6406 m respectively. The cross-sections from analytical and experimental results are compared in Figure 6.27. It is noted that the prediction and

testing results are in very good agreement except the total height has a difference within 3.5%.

Figure 6.27 Comparison of cross-section between experimental results and analytical results for test model ST1-16

In Figure 6.28, tensile forces along the geosynthetic sheet were compared between analytical and experimental results. Based on the theoretical prediction, tensile forces of top and bottom tube were 0.3386 and 2.937 kN/m respectively. However, the data measured from the model test are scattered with the location of strain gauges, which is similar to the results presented earlier. It is observed that the theoretical prediction overestimated the tensile force of the bottom tube comparing to laboratory test. However, the tensile force along the top tube is underestimated by the prediction, as shown in Figure 6.28.

Figure 6.28 Comparison of tensile forces between experimental results and analytical results for test model ST1-16

Another comparison of theoretical prediction and the results of Model Test ST2-20 are shown in Figure 3.32. The following input parameters were used in the prediction based on the laboratory test: the perimeter of two geosynthetic tubes were 3 and 4 m respectively; the unit weight of filling water was taken as 9.81 kN/m^3 ; the total height was 0.8 m and the total head of bottom tube was $H_{w2} = 0.98 \text{ m}$. After iteration, as shown in Figure 6.29, the contact length between two tubes, L_c , calculated was 1.252 m; the contact width with ground, b , was 1.086 m; horizontal section, y_0 , was 0.67 m and height of bottom tube was 0.7313 m.

Figure 6.29 Comparison of cross-section between experimental and analytical results for Model Test ST2-20

Comparisons of tensile force along the geosynthetic sheet between analytical and laboratory results are made in Figure 6.30. The tensile forces of top and bottom tube determined from theoretical prediction were 0.128 and 2.087 kN/m respectively. Figure 6.30 shows that the theoretical prediction overestimated the tensile force of the bottom tube but underestimated that of the top tube. Similar observation was made on the other comparison on the case of convex contact, as presented in Figure 6.28.

Figure 6.30 Comparison of tensile forces between experimental and analytical results for Model Test ST2-20

(3) Horizontal Contact

As discussed in section 3.8.2, Model Test ST2-12 almost approached the horizontal contact. In fact, the total head in the top tube (0.701 m) was slightly greater than that in the bottom tube (0.695 m), the contact surface was actually concave. In order to take the difference in pumping pressures into consideration, the calculation was still based on the case of concave contact. The calculation was carried out by taking the perimeters of two tubes as $L_1 = 2$ m and $L_2 = 4$ m, unit weight of filling water as 9.81 kN/m^3 , total head on top and bottom tube to be 0.701 and 0.695 m. After iteration, as shown in Figure 6.31, the contact length between two tubes, L_c , was 1.197 m, the contact width with ground, b , was 1.489 m, and height of top and bottom tube were 0.313 and 0.345 m respectively.

Figure 6.31 Comparison of cross-section between experimental and analytical results for Model Test ST2-12

The predicted tensile force along the geosynthetic sheet is also compared with the tensile force measured in Model Test ST2-12 as shown in Figure 6.32. The predicted tensile forces for the top and bottom tubes are 0.231 and 0.962 kN/m respectively. Figure 6.32 shows that the tensile forces measured during laboratory test are almost uniform and agreed well with the theoretical prediction.

Figure 6.32 Comparison of tensile force between experimental and analytical results for Model Test ST2-12

One more theoretical prediction was conducted to compare with the results of Model Test ST3-15 as shown in Figure 3.34. The input parameters for theoretical prediction were: the perimeter of two geosynthetic tubes of 2 and 4 m respectively,

the unit weight of filling water of 9.81 kN/m^3 , the total height of 0.5 m and the total head of bottom tube, $H_{w2}=0.498 \text{ m}$. After iteration, as indicated in Figure 6.33, the following parameters were calculated: the contact length between two tubes, L_c , was 0.9086 m ; the contact width with ground, b , was 1.5313 m ; horizontal section, y_0 , was 0.663 m and height of bottom tube was 0.4118 m . By comparing the cross-sectional diagrams in Figure 6.33, it is noted that very good agreement was achieved between the theoretical prediction and laboratory measurements.

Figure 6.33 Comparison of cross-section between experimental and analytical results for Model Test ST3-15

A comparison of the tensile force along the geosynthetic sheet between the predicted and the measured from the test is presented Figure 6.34. The tensile force for the bottom tube was overestimated by the prediction. However, the tensile forces for the top tube were in good agreement between the two.

Figure 6.34 Comparison of tensile force between experimental and analytical results for Model Test ST3-15

6.7 Conclusions

Some theoretical solutions for two stacked impermeable geosynthetic tubes resting on rigid foundation were presented in this chapter. In these solutions, it is assumed that the geosynthetic tubes are impermeable and inflated with water or slurry and the frictions between geosynthetic tubes and filling material, or that between geosynthetic tubes and the foundation or between two geosynthetic tubes are neglected. As a result, the tensile force along the geosynthetic tube became constant. The solutions developed in this chapter are applicable to impermeable stacked geosynthetic tubes or those filled with sand or slurry where consolidation does not affect much the final geometry of the tubes.

The developed solutions are compared with the solutions of Klusman. It is seen that the prediction made using the proposed solutions matches very well with the solution by Klusman (1998). Comparisons between the predictions by the proposed solutions and the results of model tests were also made. It is observed that the cross-sectional diagrams agree well with each other. The predictions for the tensile force are also good except that the tensile force was over predicted for the bottom tube and under predicted for the top tube.

CHAPTER 7

MODEL TEST OF PERMEABLE GEOSYNTHETIC TUBE ON RIGID FOUNDATION

7.1 Introduction

The analysis of a permeable geosynthetic tube is more difficult than that of an impermeable one as the consolidation process of the soil in the tube during or after filling needed to be modeled too. One analytical method proposed by Leshchinsky et al. (1996) was to assume that the consolidation process to be one-dimensional, that is, the width of the geosynthetic tube does not change during the consolidation process. However, the validity of this assumption needs to be verified. The consolidation process of the soil during and after the filling of the tube needs to be studied through model tests, field tests or case histories.

An experimental study of permeable geosynthetic tubes resting on rigid foundation is presented in this chapter. Two types of woven geosynthetic sheets were selected for making the geosynthetic tubes for the model tests. The slurry made from Kaolin powder was used as filling material. Kaolinite was used because it had relatively high coefficient of consolidation. A study on the tensile strength of geosynthetic sheet with and without seams was also carried out.

7.2 Properties of Geosynthetic Sheets Used

Two types of woven geosynthetic sheets were used for the model tests. One was a black color, polypropylene light weight woven geosynthetic sheet (LWG) used for a real geosynthetic tube project in Tianjin, China (Yan and Chu, 2010); the other was a black color, heavy weight woven geosynthetic (HWG) supplied by Tok Si Engineering Co. Ltd, Taiwan. The basic properties of both geosynthetics are given in Table 7.1. The tensile stress elongation properties were tested by following ASTM D4595, permeability of the geosynthetics by ASTM D4491-99a, and apparent pore opening size (AOS) of O95 by ASTM D4751.

Table 7.1 Basic properties of two selected woven geosynthetic sheets

Geosynthetic Usage	Thickness (mm)	Mass Density (g/m ²)	AOS O_{95} (mm)	Permittivity (s ⁻¹)	Max. Elongation (%)
LWG	0.52	131	0.145	0.06	20×20
HWG	2.0	840	0.4	1.00	20×27

7.2.1 Tensile Strength Measurement

The permeable geosynthetic sheets used for geosynthetic tubes must have enough tensile strength to sustain the pressure applied during pumping and stacking periods. The deformation properties of the geosynthetic materials used also affect the final geometry of the geosynthetic tube after it was filled with slurry or sand.

An INSTRON 5569 plastic test machine was used to test the tensile strength of the geosynthetic sheet. The maximum stretch force that could be applied by this machine was 50 kN. The displacement rate of test machine ranges from 5 to 500 mm/min. The displacement during the test was recorded by a built-in sensor system. The permeable geosynthetic sheets were cut into specimen with width of 20 cm and stretching length of 20 cm according to the ASTM D4595. The pulling speed was controlled as 20 mm/min (10%/min) in this test as stipulated by ASTM D4595. Three specimens each for LWG and HWG were tested in both longitude and

latitude directions. The specimen cutting directions are shown in Figure 7.1(b). The specimens were first cut to 21 cm in width and then trimmed to 20 cm by tearing off the fabric at each side. Each side of specimen was sealed by heating. The total length of the specimens was 40 cm. The top and bottom 10 cm was fixed into the wedge clamps as shown in Figure 7.1(a). Thus the gauged length was 20 cm.

(a) Specimen dimension

(b) Specimen cutting directions along the front view of tube surface

Figure 7.1 The tensile test installation of woven geosynthetic materials

Photos of a specimen under testing at different stages are shown in Figure 7.2. The tensile strength test results of LWG material on longitude and latitude directions are shown in Figure 7.3. It can be seen that the slope of initial portion, OA in Figure 7.3, is not constant but increasing with extension. However, the slope of AC, is almost constant. The reason for this phenomenon was that the woven ribbons of geosynthetic materials need a time to slack, align, and shrink to be closer as reviewed in Chapter 2.

According to ASTM 4595, the force-extension relationship of LWG material can be divided into three different stages, such as initial shrinkage stage, OA, elastic stage, AC, and failure stage, CD, as shown in Figure 7.3. It can be seen that the tensile strength for LWG material in the latitude direction was about the same as that in the longitude direction. The average peak tensile strength in the latitude direction was $4.2 \text{ kN}/20\text{cm}$ and in the longitude direction was $4.4 \text{ kN}/20\text{cm}$.

Figure 7.2 The photos of LWG specimen during tensile test

The tensile strength tests for the HWG materials had similar phenomena with those for the LWG materials as shown in Figure 7.4 except that the peak tensile strengths were much greater than those for the LWG. The average peak tensile strength on the latitude direction was $26 \text{ kN}/20\text{cm}$ and on longitude direction was of $29 \text{ kN}/20\text{cm}$.

Figure 7.3 Tensile test results of LWG

Figure 7.4 The tensile test results of HWG materials

7.2.2 Tensile Strength of Seams

As mentioned in the literature review, the tensile strength of the sewn seams depends on seam type, number of rows of stitching, seam location, stitch type, stitch count, threading type, sewing equipment and the properties of the geosynthetic sheets being sewn. Some of the most commonly used seam types are “Prayer” seam (SSa), “J” seam (SSn) and “Butterfly” seam (SSd). In this study, the seam type, “J” seam, was adopted because it offered higher seam strength among the different seam types. Two rows of stitching were adopted leaving about 5 cm from the edge of specimen as shown in Figure 7.5(b).

The tensile strength of seamed geosynthetic specimen was measured using wider width specimens according to ASTM D4884-09. Each specimen was firstly cut into a square shape of 25×25 cm as shown in Figure 7.5(a). Then the specimen was trimmed to the final dimensions as shown in Figure 7.5(c). The top and bottom 10 cm was fixed into the wedge clamps. Then the total gauged length of the tested specimen was 20 cm.

(a) Single geosynthetic sheet (b) Side view of “J” seam (c) Dimension of specimen

Figure 7.5 Specimen preparing for tensile strength test of seams

Because the original tensile strength in the latitude has little difference from that in the longitude direction, only seams along the longitude direction were tested. The

photos before and after test for the LWG specimen are shown in Figure 7.6. It can be seen that the specimen was ruptured on the seam sides. The failure in the seam was due to not only the breakage on the side of the pin holes between seaming stitches but also the shrinkage of the geosynthetic sheet and the tensile stress concentration. Furthermore, the seams made the geosynthetic sheets bent to one side of the pin holes and the tensile force at this part was not parallel to the specimen direction. All the above factors caused stress concentration and made the specimen fail earlier.

Figure 7.6 Photos “J” seamed LWG specimen before and after test

The tensile force versus extension curves for the LWG is shown in Figure 7.7. The average tensile strength of seamed LWG specimen was only about $2.5 \text{ kN}/20 \text{ cm}$, which was 56.8% of the original light woven geosynthetic sheets without seam (average tensile strength of $4.4 \text{ kN}/20 \text{ cm}$).

The photos before and after test for the HWG specimen are shown in Figure 7.8. The same failure phenomena and mechanisms as that for the LWG specimen can be seen from Figure 7.10. The tensile force versus extension curves for HWG is shown in Figure 7.9. It can be seen that the average tensile strength of the seamed HWG specimen was $15 \text{ kN}/20 \text{ cm}$, which was 51.7% of the original geosynthetic without seam (i.e. $29 \text{ kN}/20 \text{ cm}$).

Figure 7.7 Tensile force versus time curve for “J” seamed LWG specimen

(a) The HWG specimen before test

(b) The HWG specimen after test

Figure 7.8 Photos “J” seamed HWG specimen before and after test

Figure 7.9 The tensile strength of “J” seamed HWG specimen

7.2.3 Tensile Strength of Reinforced Seams

To improve the tensile strength of the seams, a new reinforced seam method is proposed in this section. The main purpose of this proposed method was to improve the tensile strength of the seam on the pin-hole side which was the weakest part in the traditional seam methods. The two layers of geosynthetic sheets were sewn together with a sewing machine first to double its thickness. For the three commonly used seam types, the “Prayer” seam (SSa), “J” seam (SSn) and “Butterfly” seam (SSd), the reinforced seam configurations are shown in Table 7.2.

In the laboratory tests, the reinforced seam method was applied to “J” Type. For the convenience, it reinforced “J” type seam will be called “RJ” type in the following discussion. Both LWG and HWG specimens were sawn into “RJ” seams with two rows of stitching. The length of reinforce part was 20 cm with 15 cm for seaming and another 5 cm for force transferring as shown in Figure 7.10(a). The dimensions of the reinforced specimen, as shown in Figure 7.10, were the same as that without

reinforcement. ASTM D4884 was still adopted for the testing of the tensile strength of the reinforced seams.

Table 7.2 Reinforced seam types

Seam type	Reinforced “ <i>Prayer</i> ” seam	Reinforced “ <i>J</i> ” seam	Reinforced “ <i>Butterfly</i> ” seam
Sketch

(a) Single geosynthetic sheet (b) Side view of “J” seam (c) Dimension of specimen

Figure 7.10 Specimen preparing for tensile strength test of “RJ” seam

The photos before and after test for the “RJ” seam used for the LWG specimens are shown in Figure 7.11. It can be seen that the weakest part of the specimen was not at the sides of the pin holes anymore, but at the edges of the reinforced area. This is because the thickness of specimen transferring from the double layers to the single layers induces a stress concentration.

Figure 7.11 Photos of tensile strength test for the RJ seamed LWG

The tensile test results of “RJ” seams for the LWG specimen is shown in Figure 7.12. The average tensile strength of the “RJ” seams for the LWG specimen was about $3.5 \text{ kN}/20 \text{ cm}$, or 79.5% of the original LWG specimen without seam.

Figure 7.12 Tensile test results of “RJ” seamed LWG specimen

Figure 7.13 Photos of tensile strength test for the RJ seamed HWG

Figure 7.14 Tensile test results of "RJ" seams for HWG specimen

The photos before and after test for the "RJ" seams for the HWG specimen are shown in Figure 7.13. It can be seen that the specimen was also broken on the side of the reinforced area which was the same as that for the "RJ" seams for the LWG specimen. The results of the tensile strength tests for the "RJ" seams for the HWG

are shown in Figure 7.14. The average tensile strength of the “RJ” seams for HWG was 22 kN/m/20 cm, or 75.8% of the original geosynthetic sheets without seams.

The comparison between the seam efficiency of “J” seam and “RJ” seam for both LWG and HWG geosynthetics are listed in Table 7.3. It can be seen that the seam efficiency of the geosynthetic sheet with “RJ” seam can still be improved. The seam efficiencies of the “RJ” seam were only about 79.5% for the LWG and 75.8% for the HWG because the reinforced parts were sewn together with a sewing machine which caused damages of the ribbons in the geosynthetic sheets. If it was possible for this part to be double knitted, the seam efficiency would be higher.

Table 7.3 Basic properties of two selected woven geosynthetic sheets

Geosynthetic Usage	Tensile strength without seam (kN/20cm)	“J” Ream		“RJ” seam	
		Tensile strength (kN/20cm)	Seam efficiency %	Tensile strength (kN/20cm)	Seam efficiency %
LWG	4.4	2.5	56.8	3.5	79.5
HWG	29	15	51.7	22	75.8

7.3 Preparation of Slurry

In the model tests carried out, the filling slurry was made of Kaolin powder and water. Kaolin was chosen as the filling material because it was relatively quick to consolidate. Kaolin was also commercially available and its grain size consistency could be maintained. The kaolin used was supplied by Kaolin Malaysia Sdn. Bhd. The other basic engineering properties of kaolin are presented in Table 7.4.

Table 7.4 Engineering properties of Kaolin

Properties	Values
Specific Gravity	2.61
Liquid limit, LL (%)	61
Plastic limit, PL (%)	38
Plasticity Index, PI (%)	23

7.4 Model Test of HWG Tubes

7.4.1 Model Test Setup

The model test setup consisted of four components: data recording system, geosynthetic tube, displacement measurement system and filling system. A panoramic view of the setup is shown in Figure 7.15. The data loggers recorded the readings of strain gauges and water pressure transducers. The displacement measuring system measured the geometry change of the geosynthetic tube. The filling system was used to fill the geosynthetic tube with the mixed slurry.

Figure 7.15 Panoramic view of the model setup

(1) Filling System

The kaolin powder was mixed with tap water into a slurry form. Firstly, the weighted dry kaolin powder and water were putted into a big mixer and blended for about 60 min as shown in Figure 7.16(a). After mixing, the desired slurry was

transferred from the mixer to a filling tank as shown in Figure 7.16(b). This procedure was repeated several times until sufficient slurry was made. The filling tank had a height of 1.0 m and a diameter of 1.0 m.

Figure 7.16 Equipment used for mixing and filling of Kaolin slurry

After the desired amount of slurry was placed in the filling tank, the top of the tank was sealed using a cap as shown in Figure 7.17(a). The compressed air was then applied to press the piston down as shown in Figure 7.17(b) and force the slurry to be displaced into the geosynthetic tube through a soft plastic tube connected to the bottom of the filling tank as shown in Figure 7.17(b) and (c). An air pressure of 50 kPa was used to fill the tube. The cross-section of the geosynthetic tube was monitored.

Figure 7.17 Filling system of geosynthetic tubes

(2) *The HWG Tubes*

The first series of model tests were carried out on geosynthetic tubes made of the heavy woven geosynthetic (HWG) sheet. The dimensions of the HWG tubes and water content of the filling slurry are given in Table 7.5. The range of water content of the filling slurry was from 70% to 100%.

Table 7.5 Geosynthetic tubes information

Model test No.	Materials	Width (m)	Length (m)	Water Content of filling slurry (%)
H1	HWG	1.0	2.0	71.6
H2	HWG	1.0	2.0	80.6
H3	HWG	1.0	2.0	89.2
H4	HWG	1.0	2.0	103.6

The experiments were performed in a box of 3.0 m wide, 4.0 m long and 0.4 m high as shown in Figure 7.18. The box was placed on the concrete floor. Two concrete beams and two wooden boards were used to form a box. Another flat wooden board was placed on the floor as the base for the geosynthetic tube. The box and the base were lined with double layers of plastic sheets to make it water proof and contain water that would seep out during the test.

Figure 7.18 Photo of the HWG tube and container

(3) Displacement Measuring System

The displacements of the HWG tubes during a test were measured by an automatic running scanner mounted on the frame as shown in Figure 7.19. The frame used was different from that used for the impermeable geosynthetic tube tests presented in Chapter 3. Two standing steel truss columns were fastened to the loading bearing concrete floor using bolts. A scanner was mounted on a horizontal track which was supported by the two columns as shown in Figure 7.19. The scanner was attached to a motor so its movement could be controlled. The controller box for the scanner is

shown in the left bottom of Figure 7.19. The ILD1700-750 model laser sensor, the same as that used for the model tests presented in Chapter 3, was used.

The laser sensor which was connected to a desktop computer recorded the time and the vertical distance to the top of the geosynthetic tubes at that time. The horizontal coordinate was calculated by the speed of scanner, 5 cm/s, multiply the recording time. Before the model test, the distance from wooden board to the laser sensor was scanned. Then the vertical coordinates of testing points on the HWG tube could be calculated as the difference between the distance to wooden board and the laser sensor readings. This contact-free distance measuring system was effective and accurate in measuring the displacement of the geosynthetic tube.

Figure 7.19 Photo of cross-section measurement system

(4) Data Recording System

The WFLA-6-11-3L strain gauge was used for measuring the tensile forces along the HWG tube. This was the same type as that used in model tests presented in Chapter 3. The positions of the strain gauges installed on the tube are shown in Figure

7.20(a). A photo of the strain gauges attached to the surface of the geosynthetic tube is shown in Figure 7.20(c).

Figure 7.20 Photo of attached strain gauges and data logger

The miniature pore water pressure transducers (PPTs) used in this model test were Druck[®] PDCR 81 type with an external diameter of 6.4 mm and height of 11.4 mm as shown in Figure 7.21 (b). Such a miniature size was necessary to minimize the influence of the measuring device to the overall soil behavior during model test. The PPTs has 5 m long and 2.3 mm diameter integral Teflon vented cable. Before a model test, all PPTs were calibrated by using water pressure generated in a triaxial cell. The reaction of the PPTs to the applied pressure was linear and the capacity was 300 kPa. Two PPTs were used to test the pore water pressure during filling and dewatering stage. One was preinstalled on the inner top surface as shown in Figure 7.21(a). Another was at the bottom as shown in Figure 7.21(c).

Figure 7.21 Miniature pore water pressure transducer PDCR 81

7.4.2 Strain Gauges Calibration

As discussed in Section 3.4.2, the hardened super glue stiffened the area where the strain gauge was attached as shown in Figure 7.22. A simple strain gauge calibration test was also carried out on three specimens by following the method similar to that described in Section 3.4.2 and shown in Figure 7.22. The method for the attachment of the strain gauge was exactly the same as that discussed in Section 7.4.1. The force versus the strain gauge readings curves obtained from the three specimens are plotted in Figure 7.23. It can be seen that the tensile force versus strain gauge readings relationship is a second order polynomial. This was expected as the woven ribbons need some time to slack, align, and shrink to sustain load. As shown in Figure 7.23, the tensile modulus increases with the increasing of elongation. There are two ways to fit the calibration curves, 1) to separate the curve

into two linear stages or 2) to fit it in a second order polynomial. In this study, the second method was adapted because the R^2 is very close to 1.0. Therefore, the tensile force in the geosynthetic tube in all the model tests can be calibrated using the second order polynomial equation as shown in Figure 7.23. It should be point out that, in all the follow model tests, the maximum readings of the strain gauge did not exceed 3500 which is the maximum value in Figure 7.23.

Figure 7.22 Calibration test set-up of the strain gauges attached on HWG

Figure 7.23 Calibration of the strain gauges attached on HWG

As discussed in Section 3.4.2, during the model test, the geosynthetic sheet would bend when the geosynthetic tube was inflated. The bending caused the strain gauge reading to increase even without tensile force. Calibration tests were carried out to correct the bending effect. The detail was discussed in Section 3.4.2 and shown in Figure 7.24. The strain gauge readings versus the radius of the bending (or the radius of the cylinder) are plotted in Figure 7.25. Similar to section 3.4.2, a power relationship between the strain gauge readings and the radius of the bending can be established as shown in Figure 7.25.

It should be pointed out that the relationship may not fully correct the bending effect of all raw strain gauge readings during the model tests as the radius on the strain gauge location on the flexible geosynthetic tube were changing during dewatering stage. Therefore, the tensile forces tested in this chapter may not be the absolute value of tensile force but can be used to identify the changing tendency of tensile forces.

Figure 7.24 Bending test of the strain gauges attached on HWG specimen

Figure 7.25 Bending effect of the strain gauges attached on HWG specimen

7.4.3 Test Results

The experimental results of Model Test H1 for a HWG tube (see Table 7.5) are presented in Figure 7.26. The height versus time curve during the self-weight consolidation process is shown in Figure 7.26(a). It can be seen that the height increased quickly during the filling stage as shown as OA and AB in Figure 7.26(a). There are two “steps” in the curves because the filling had to be done in two stages as the capacity of the filling tank was not big enough to allow the tube to be filled in one go. The tube was inflated to a height of 0.23 m during the first stage and 0.47 m during the second stage. After fully inflated, the tubes underwent a dewatering stage, as shown as BC in Figure 7.26(a). The height decreased quickly at the beginning as a result of dewatering, but approached a constant value after about 9 hours.

The tensile force per unit length can be calculated using the equation from the calibration of strain gauge attached on HWG sheets as discussed in Figure 7.24. The bending of stain gauge basing on the equation from Figure 7.25 was considered by same method used in section 3.6.3. The tensile forces versus dewatering time curve are shown in Figure 7.26(b). The tensile forces increased quickly with the increase of tube height of the tube during the filling stage. The tensile forces at the testing

points along the geosynthetic tube decreased until reached a constant value during the dewatering stage. The dewatering period lasted about 9 hours after the tube was fully inflated. Because of the internal lateral pressure acting by the soil mass, the reduction in the tensile force along the side of geosynthetic tube at the B2 to B6 positions was slower after self-weight consolidation, whereas the tensile forces along the bottom surface, B7 and B8, of geosynthetic tube became nearly zero.

The pore water pressure variations at the top and bottom inner surface of the tube with time curves are shown in Figure 7.26(c). It can be seen that right after the tube was fully filled, the pore water pressure at the top and bottom reached the peak (point B in Figure 7.26(c), with their value of 3.3 and 10.1 kPa , respectively. The difference between the pore water pressure at the top and bottom inner surface was due to the difference in elevation, i.e. 0.47 m in Figure 7.26(a). During the dissipation stage shown as BC in Figure 7.26(c), the pore water pressure on the top inner surface dropped very quickly with time. On the other hand, the pore water pressure at the bottom maintained for a while before it decreased. The difference between the pore water pressures on top and bottom after dewatering was 4.01 kPa which was consistent with the difference in elevation between the two testing points, i.e. 0.378 m as shown in Figure 7.26(a).

(a) Displacement vs. time of model H1

(b) Tensile force vs. time for test model H1

(c) Pore water pressure vs. time

Figure 7.26 Laboratory test results of model H1

The experimental results of Model Test H2 are presented in Figure 7.27. The tube was inflated in two stages: one to a height of 0.2 m and another to 0.37 m. It can be seen from Figure 7.26 that the behavior Model Test H2 was almost the same as that for Model Test H1 except that the pore water pressure at the bottom inner surface dropped quicker as shown in Figure 7.27(c). Right after the tube was fully inflated (marked as point B in Figure 7.27(c)), the pore water pressure at the top and bottom reached the peak with their value of 2.0 and 8.6 kPa, respectively. The difference between the pore water pressures on top and bottom tubes after dewatering was 3.5 kPa which was due to the difference in elevation between the testing points, i.e. 0.254 m in Figure 7.27(a).

In Model Test H3, the tube was inflated in two stages too, one to a height of 0.2 m and another to 0.46 m, as shown in Figure 7.28(a). Model H3 behaved almost the same as model H1 except that the pore water pressure on the top and bottom inner surface had a time delay as shown in Figure 7.28(c). The model H3 was fully inflated at 1.8 h. But the pore water pressure on the top and bottom inner surface only peaked at 10 and 4 h respectively. The peak values were 4.9 and 9.0 kPa, respectively as shown as the point B in Figure 7.28(c). The difference between the pore water pressures on the top and bottom after dewatering was again due to the difference in elevation between the two points.

In model Test H4, the tube was inflated to a total height of 0.45 m in three stages as shown in Figure 7.29(a). The model H4 was fully inflated at 7.0 h. The pore water pressure at the top and bottom reached the peak with their value of 2.0 and 12.0 kPa, respectively (see point C in Figure 7.29(c)).

(a) Height of H2 vs. time

(b) Tensile forces vs. time

(c) Pore water pressure vs. time

Figure 7.27 Laboratory test result of model H2

(a) Height of H3 vs. time

(b) Tensile forces vs. time

(c) Pore water pressure vs. time

Figure 7.28 Laboratory test results of model H3

(a) Height of H4 vs. time

(b) Tensile forces vs. time

(c) Pore water pressure vs. time

Figure 7.29 Laboratory test results of model H4

7.5 Model Test of LWG Tubes

7.5.1 Model Test Setup

Similar to the model tests on HWG tubes, the model test set up for the LWG tubes had the similar components except a platform weighing machine and LVDTs were used to measure the weight and displacements of the tube respectively. A panorama view of the setup is shown in Figure 7.30.

Figure 7.30 Panorama view of the LWG tube test setup. (1) Platform weighing machine, (2) The plastic box, (3) Data Logger, (4) LVDT, (5) The filling tube, (6) The LWG tube, (7) Wooden board, (8) Filling Tank, (9) Strain gauge, (10) Rubber pad.

The model tests on the LWG tubes were performed in a box with dimension of $1.0 \times 1.5 \times 1.0 \text{ m}$ (W×L×H) as shown in Figure 7.31. A wooden board covered by plastic membrane was also used as the rigid foundation. A platform weighing machine with a capacity of 300 kg was placed at the bottom of the tube to measure the total weight change of the tubes. From the weight change, the volume change of the water content change of the soil can be estimated.

(a) Plan view the model test setup

(b) Side view on section A-A

Figure 7.31 Sketch for the LWG tube model test setup (unit m)

The filling tank for inflating the LWG tubes was the same as that used in the model tests for the HWG tubes except the volume of the tank was smaller as shown in Figure 7.30. The tensile force along the LWG tubes and pore water pressure on top and bottom inner surface were measured with the same methods as discussed in Section 7.4.1.

Six model tests were carried out for the LWG tubes. The detail of the model tests is shown in Table 7.6. All the tubes were sawn using the LWG material with “RJ” seam proposed in section 7.2.3. The range of the water content of the filling slurry was from 130% to 170% as shown in Table 7.6.

Table 7.6 Information of the LWG tubes

Model test No.	Materials	Width (<i>m</i>)	Length (<i>m</i>)	Filling Water Content (%)
L1	LWG	0.51	1.10	130.8
L2	LWG	0.47	1.25	141.8
L3	LWG	0.48	1.08	147.2
L4	LWG	0.45	0.87	159.9
L5	LWG	0.60	1.2	144.0
L6	LWG	0.60	1.2	168.4

The displacements of the LWG tubes were measured using LVDTs as shown in Figure 7.30. The LVDTs is a linear position sensor that produces an electrical signal proportional to the displacement of the testing points. In order to reduce the influence of the reacting force of the LVDTs to the flexible geosynthetic tube surface, circular black rubber pads with diameter of 5 *cm* were used at the contact points as the photos shown in Figure 7.30.

Two types of LVDTs were used for the model tests. One was LVDTs with a measurement range of 20 *cm* and another with a measurement range of 5 *cm*. Three units of the former type LVDT were mounted on aluminum alloy bars and connected with the top beam to measure the displacements on the top surface of the tube (see Figure 7.30). Two units of the later type were installed on both the left and the right of the tube (see Figure 7.30) to measure the change in the width. The practical accuracies of all the five LVDTs were 0.1 *mm*. The sketch of all the LVDTs positions are shown in Figure 7.32.

Figure 7.32 Sketch of the LVDTs mounted positions (unit *cm*)

7.5.2 Test Results

The displacements of the tube in Model Test *L1* during the filling and dewatering stages are shown in Figure 7.33(a). There is a horizontal step at the beginning of the curve. This was caused by the lack of contact of LVDTs to the surface of the tube. The tube in Model Test *L1* was inflated to a height of 0.236 *m* as the end of filling, (see point A in Figure 7.33(a). After filled, the tube underwent a dewatering stage, as shown as AB in Figure 7.33(a). The height decreased quicker at the beginning, but approached a constant value of 0.188 *m* after about 6 hours.

The change in the weight of the model *L1* as measured by the platform weighing machine is shown in Figure 7.33(b). It can be seen that the model *L1* was inflated with 110 *kg* slurry at the end of filling (see point A in Figure 7.33(b). During the dewatering stage, the weight dropped quickly at the beginning and then slowed down and approached a constant value of 98 *kg* (see AB in Figure 7.33(b).

(a) Displacement changes of model L1

(b) Weight changes of model L1

Figure 7.33 Test results of the model L1

The test results of Model Test L2 to L6 are shown in Figure 7.34 to Figure 7.38, respectively. It can be seen that the results of Test L2 to L6 are similar to that of Test L1 except the height and weight at the end of filling or dewatering were different. A summary of the results are given in Table 7.7.

Table 7.7 Displacement of the LWG tubes

Model test No.	End of filling		End of dewatering	
	Height (m)	Weight (kg)	Height (m)	Weight (kg)
L1	0.236	110	0.188	97
L2	0.154	83	0.104	70
L3	0.102	76.8	0.072	52.8
L4	0.129	62.5	0.067	52
L5	0.224	147.6	0.165	125.6
L6	0.209	125	0.137	101.5

(a) Displacement changes of model L2

(b) Weight changes of model L2

Figure 7.34 Test results of the model L2

(a) Displacement changes of model L3

(b) Weight changes of model L3

Figure 7.35 Test results of the model L3

(a) Displacement changes of model L4

(b) Weight changes of model L4

Figure 7.36 Test results of the model L4

Figure 7.37 Test results of the model L5

Figure 7.38 Test results of the model L6

7.6 Observation of Displacement Stages

A typical displacement versus time curve for a geosynthetic tube during a model tests is shown in Figure 7.38. The first part of the curve from O to B is the filling stage and the second part of the curve from B to C is the dewatering stage.

Figure 7.39 Typical displacement of geosynthetic tubes

During the filling stage, the high water content slurry was filled into the geosynthetic tube which increased the height of the tube rapidly. At the end of filling process, as marked by point B in Figure 7.39 and Figure 7.40(b), the cross-section and the tensile force induced in the geosynthetic tube were the highest. During this stage, water and some fines seeped out as shown in Figure 7.40. The amount of fines seeped out was determined by the AOS of geosynthetic sheet, particle size of filling soils and the filling time.

(a) Photo of H3 in the first filling

(b) Photo of H3 at the end of filling

Figure 7.40 Photos of model H3 in filling stage

Figure 7.41 Photos of model H3 in dewatering stage

During the dewatering stage, water dissipated through the permeable geosynthetic sheet. Water and some soil particles dissipated through geosynthetic sheet quickly at the beginning which was shown as the quick drop in the height of the geosynthetic tube. During the model tests, this stage lasted only about 1-2 hours depending on the water content of filling slurry, AOS of geosynthetic sheets and particle size of kaolin. After some time, only clean water seeped out of the geosynthetic tube as shown in Figure 7.41(b). The dewatering process was almost stopped about 9 hours after filling as shown in Figure 7.39 as point C.

The dewatering stage was a complex process that that is hard to explain in general. However, it should involve at least one of the following stages: filtration stage and consolidation stage.

The filtration stage is characterized by a quick dissipation of water and some fines through the tube. At this stage, the slurry flow velocity through a soil-geosynthetic system is a function of the resistances offered by the soil, thickness of the geosynthetic and the hydraulic gradient (Rushton et al., 1996). After some time, a filtration cake may be formed. For cohesive, fine particles soils, the soil particles carried by water may accumulate onto the fabric grids to form a low permeability layer as shown schematically in Figure 7.42. However, for non-cohesive soils or sand, a filter is formed by stopping larger particles on one side of the geosynthetic sheet which then stop smaller particles. As time going on, the interface layer will

become thicker and thicker which makes the inside soil particles harder to move through the drainage pipe (Rushton et al., 1996).

The self-weight consolidation starts to dominate after the pumping pressure becomes zero. The consolidation involves the dissipation of pore water pressure due to the self-weight of soil particles and the confining pressure asserted by the geosynthetic tubes. During this stage, only clean water seeps out of the geosynthetic tube as shown in as shown in Figure 7.41(b). After the self-weight consolidation, the whole dewatering process stops.

It should be pointed out that there is no clear separation of the two stages and both may co-exist particularly during the transition from the filtration to the self-weight consolidation stage.

Figure 7.42 Filtration process of slurry

7.7 Displacement Analysis

7.7.1 At the End of Filling

As discussed before, the height of geosynthetic tube increased very quickly during the filling stage. Although some water and fine particles seep out of the tube, the amount of dissipation is negligible as the time taken for dissipation is limited. Furthermore, the pumping pressure inside the geosynthetic tube could be maintained because the filling is continued. Therefore, the cross-section and tensile force at the point right after filling, i.e., point B in Figure 7.39, can be calculated with the theory discussed in Chapter 4 by assuming the tube is impermeable during the filling process.

A comparison of the experimental results for Model Test H1 at the end of filling and the analytical solutions is shown in Figure 7.43. In the calculation, the tube was assumed to be impermeable and inflated with slurry. The water content of the slurry was taken as 71.6%. The specific unit weight of kaolin was 2.61. The perimeter of the tube in Test H1 was 2 m and the pumping pressure was 3.3 kPa (see point B in Figure 7.26(b)). The unit weight of the slurry was calculated as follows,

$$\gamma_{sat} = \frac{1+w}{1+wG_s} G_s \gamma_w \quad (7-1)$$

where γ_{sat} is saturated unit weight of filling slurry, w is water content of the filling slurry, G_s is specific unit weight of kaolin and γ_w is unit weight of water.

The cross-section of the tube in Model Test H1 at the end of filling is compared with the analytical solutions in Figure 7.43. A good agreement can be seen except the width of the observed cross-section was a little larger. Comparisons of the experimental results with analytical solutions at the state immediately after filling for Model Test H2, H3 and H4 are shown in Figure 7.44, Figure 7.45 and Figure 7.46 respectively. The inputs were similar to those for Model Test H1 with their values listed in Table 7.8. In general, the agreements are acceptable.

Table 7.8 The parameters used for theoretical calculation

Model test No.	Water Content %	Unit weight kN/m^3	Pumping pressure kPa	Height, m	
				Test	Theoretical
H1	71.6	15.6	3.3	0.483	0.463
H2	80.6	15.2	2.0	0.368	0.427
H3	89.2	14.8	5.0	0.458	0.498
H4	103.6	14.3	2.0	0.45	0.431

Figure 7.43 Comparison between the experimental and analytical results of model H1 at the end of filling

Figure 7.44 Comparison between the experimental and analytical results of model H2 at the end of filling

Figure 7.45 Comparison between the experimental and analytical results of model H3 at the end of filling

Figure 7.46 Comparison between the experimental and analytical results of model H4 at the end of filling

7.7.2 During the Dewatering Stage

The displacement of geosynthetic tubes during a dewatering stage is accompanied by the dissipation of pore water pressure due to the self-weight of soil particles and the confining pressure asserted by the geosynthetic tubes. The whole process was time related because of the influence of permeability of the geosynthetic sheet used

and that of the filled soil itself. The whole dewatering process can be divided into two stages such as filtration stage and consolidation stage as discussed in Section 7.6. If a relationship between time and displacement on these two stages can be established, the displacement of geosynthetic tube at any time can be calculated.

(1) 1-D Method

A one dimensional method to calculate the displacement of permeable geosynthetic tube during dewatering stage was proposed by Leshchinsky et al. (1996). In this method, the width of geosynthetic tube is assumed to be constant. The total volume change of the tube is only due to its displacement in the vertical direction. The change ratio of the height of cross-sections equals to the volume change ratio and the water content change ratio as shown in Equation (7-2):

$$\Delta H = \frac{G_s(w_0 - w_f)}{1 + w_0 G_s} H_0 \quad (7-2)$$

where G_s was the specific gravity of soil, w_0 was water content of filling slurry, w_f was the water content of consolidated kaolin and H_0 was height of the geosynthetic tube at the end of filling.

The 1D assumption in the above method can be examined using the model test results. The normalized widths of the tube versus time curves for Model Test H1, H2 and H3 during the dewatering stage are shown in Figure 7.47. The width of geosynthetic tube is normalized by dividing the width of the tube at the end of filling to facilitate comparison. The time was taken from the end of filling. It can be seen that the width increased 6 to 8% initially and maintained constant for the rest of the dewatering stage. This 6 to 8% increase in the width is insignificant. Therefore, the assumption of a 1D displacement during the dewatering stage is reasonable.

Figure 7.47 Width changes of the HWG tubes

For Model Tests L1 to L4 on LWG tubes, the total weight was measured. By assuming the change in the weight was completely due to water dissipation, the total weight change can be used to calculate the change in the average water content of the soil. : It has to be noted that the water contents of kaolin in different position are different. The water content of kaolin next to geosynthetic sheet will be lower than that at the center of the tube. The tested average water content can only show a changing tendency of the entire tube.

The average water contents variation with time curves are shown in Figure 7.48. In this plot, the time is taken from the end of filling. It can be seen that during the dewatering stage, the average water contents dropped very fast at the beginning and then slow down till approached to the constant value. This is consistent with the width change as shown in Figure 7.47.

Figure 7.48 Average water content of the LWG tubes

The displacements of the LWG tubes L1 during the dewatering stage using the 1-D method were calculated by integrating the average water content from Figure 7.48 into Equation (7-2). The time is taken from the end of filling. The rate of displacement was calculated by the displacement divided by the height of the tube at the end of filling. As shown in Figure 7.49, the rate of displacement calculated by the 1-D method is about 15.57% which was smaller than that measured from the test results.

Figure 7.49 Compare 1-D method with test results for model L1

The same can be observed for Model Tests L2, L4, L5 and L6. The differences are 11.5, 26.0, 7.3, and 9.7% as shown in Figure 7.50, Figure 7.51, Figure 7.52 and Figure 7.53, respectively. It should be pointed out the differences for these series of tests are bigger because the sizes of the tubes are smaller and the difference should be smaller for the prototype. Overall, the 1-D method tends to underestimate the amount of volume change by about 12% because of the omitting of the change in width. It should be point out that the mismatch was not only due to the lack of consideration of the changing of width but also because of the assumption that the change in the weight was completely due to water dissipation when calculation the average water content.

Figure 7.50 Compare 1-D method with test results for model L2

Figure 7.51 Compare 1-D method with test results for model L4

Figure 7.52 Compare 1-D method with test results for model L5

Figure 7.53 Compare 1-D method with test results for model L6

(2) Curve Fitting Method

Another problem with the 1-D method is that the final water content of the soil has to be known. This is not convenient for design. A different method is proposed as follows.

A curve fitting method, the so-called first order rate equation (FORE) method was proposed by Handy (2002). He claimed that the FORE method can be applied to

many geotechnical phenomena so long as they are consistent (Handy, 2002). By applying the FORE method, a relationship between the height of the tube h and time t can be established as Equation (7-3). From Equation (7-3), the height of geosynthetic tube, h , with respect to dewatering time, t , can be calculated as Equation (7-4):

$$\log(-h + H_{\infty}) = -at - b \quad (7-3)$$

$$h = -10\exp(-at - b) + H_{\infty} \quad (7-4)$$

where h is the height of geosynthetic tube, t is dewatering time, H_{∞} is values of h when $t = \infty$, a and b is scale parameter that governs the rate of change of h with respect to t .

By adjusting H_{∞} , h , a and b , the FORE method could generate a wide variety of interpolating curves that can be used to fit the experimental data.

The FORE method was used for Model Test H3. The time is taken from the end of filling, such as point B in Figure 7.28(a). This is the case for all the rest of the data analysis. The testing data within the dewatering time from 0.0 to 0.8 hour (after point B in Figure 7.28(a)) was selected for curve fitting. As shown in Equation (7-3), the height of geosynthetic tube when $t = \infty$, H_{∞} , and scale parameter a and b are unknown. The value of the final height, H_{∞} , in the filtration stage was denoted as H_F . With reference to Handy (2002), the procedure for determining H_F , a and b in the FORE method is as follows,

- (1) Select the experimental data for curve fitting such as time from 0.0 to 0.8 hours;
- (2) Use a trail $H_F = 0.0$ at the beginning to calculate $\log(h - H_F)$ where h is the height of geosynthetic tube from experimental data;
- (3) Linear regress $\log(h - H_F)$ and t using equation $\log(h - H_F) = -at - b$ and get the value of a , b and R^2 where R^2 is coefficient of determination; The equations for the linear regression can be found in any mathematical text books which will not covered in this thesis;

- (4) Change the value of trail H_F and repeat step 2) to 3) till R^2 got the largest value.
The infinite height, H_F , was obviously less than the height when $t = 0.8 h$.

Figure 7.54 shows the results of linear regression for the displacement of model H3 in filtration stage using the FORE method. It can be seen that the coefficient determination R^2 was of 0.991, scale parameter a of -0.665, and b of -1.04 with respects to the infinite height in filtration stage of 0.357 m.

Figure 7.54 Curve fitting using FORE method for displacement of model H3 in filtration stage

The FORE procedure was also applied to the consolidation stage in the Model Test H3. The test data with 1.9 to 21.4 h after the time of filling (after point (B) shown in Figure 7.28(a) were selected. The final height of model H3 was denoted as H_C . The regression curve is shown in Figure 7.55. The coefficient determination R^2 was of 0.964, scale parameter a of -0.165, and b of -1.036 with respecting to the final height in consolidation stage of 0.2978 m. The value of H_C from this method matches the experimental value of 0.295 m very well (see Figure 7.28(a)). The displacements of model H3 during the complete dewatering stage are plotted in Figure 7.56 with logarithmic time scale. It can be seen that the FORE method can fit the experimental data well except a significant overlapping between two the

equations. This is reasonable because the consolidation stage must start simultaneously with filtration stage.

Figure 7.55 Curve fitting using FORE method for displacement of model H3 in consolidation stage

Figure 7.56 Logarithmic time scale plot of relationship between displacement and time of model H3 during dewatering stage

The overlapping between the two stages cannot be overcome no matter which kind of curve fitting method was used (Handy, 2002). However, the height during

filtration stage, H_F , can be used as the height separating the two stages as shown in Figure 7.56. It can be defined that the filtration stage is the height between H_0 to H_F and the consolidation stage between H_F to H_C . Different regression curves or equations can be used for different stages. Figure 7.56 is also re-plotted into a linear time scale as shown in Figure 7.57.

Figure 7.57 Linear time scale plot of relationship between displacement and time of model H3 during dewatering stage

(3) Curve Fitting for the HWG Tubes

In order to compare all the experimental data of displacements of geosynthetic tubes, their heights were normalized by divided the heights at the end of filling, H_0 . Then the rate of displacement, R_d was calculated as $1-h/H_0$ where h is height of geosynthetic tube at dewatering time t .

The displacement of the geosynthetic tubes is influenced by the water content of the filling slurry. To consider the influence of water content, the rates of displacements are normalized by dividing it by the void ratio of the filling slurry, e_0 , as shown in Equation (7-5). The void ratio is related to water content as shown in Equation (7-6).

$$R_{Nd} = (1 - \frac{h}{H_0}) / e_0 \quad (7-5)$$

$$e_0 = w_0 Gs \quad (7-6)$$

where R_{Nd} is the normalized rate of displacement, h is the height of the tube at time t , H_0 is the height of the tube at the end of filling; e_0 and w_0 are the void ratio and water content of filling slurry respectively.

The normalized rates of displacement versus time curves are shown in Figure 7.58 for all the HWG tubes. The time is taken from the end of filling. It can be seen that all the displacements curves fall within a narrow band.

Figure 7.58 Relationships between normalized rate of displacement and the logarithmic time scale for the HWG tubes during dewatering stage

The normalized rate of displacement for the HWG tubes is also fitted using the FORE method. Similar to the FORE fitting for Test H3, the mean values of the test data is fitted and plotted in a logarithmic time scale as shown in Figure 7.59. It can be seen that the permeable HWG tubes was in the filtration stage when the normalized rate of displacement was less than 0.0835 and in the consolidation stage when the rate was between of 0.0835 and 0.1385. In the filtration stage, the coefficient determination R^2 was of 0.998, scale parameter a of -0.5184, and b of -

1.099. In consolidation stage, the coefficient determination R^2 was of 0.995, scale parameter a of -0.1495, and b of -1.03. The Figure 7.59 was also plotted in a linear time scale as shown in Figure 7.60.

Figure 7.59 The logarithmic time scale plot of curve fitting for the mean values of the normalized rate of displacement of HWG tubes with the FORE method

Figure 7.60 Linear time scale plot of curve fitting for the mean values of the normalized rate of displacement of HWG tubes with the FORE method

Similar to the curve fitting for the mean values, the FORE method is also used the minimum and maximum values shown in Figure 7.61 and Figure 7.62. The details

of the equations are also shown in these two figures. The two figures were also plotted in a linear time scales as shown in Figure 7.63 and Figure 7.64.

Figure 7.61 The logarithmic time scale plot of curve fitting for the maximum values of the normalized rate of displacement of HWG tubes with the FORE method

Figure 7.62 The logarithmic time scale plot of curve fitting for the minimum values of the normalized rate of displacement of HWG tubes with the FORE method

Figure 7.63 Linear time scale plot curve fitting for the maximum values of the normalized rate of displacement of HWG tubes with the FORE method

Figure 7.64 Linear time scale plot curve fitting for the minimum values of the normalized rate of displacement of HWG tubes with the FORE method

All these curve fitting results using FORE method are summarized in in Equation (7-7) and (7-8) and Table 7.9. It can be seen that the parameters in the fitted equations are about in maximum $\pm 40\%$ difference of the mean values in the filtration stage and $\pm 17\%$ in the consolidation stage. It should be noted that the

equations can only be used for a general case to calculate the displacement of the HWG tubes inflated with the kaolin because the equation is derived from the normalized experimental data. For the other kind of filling slurry or geosynthetic materials, these equations may not be applicable because their permeability will be different.

Filtration stage

$$\begin{aligned} \text{Maximum, } R_{Nd} &= 0.092 \cdot 10^{(-0.691t-1.085)}, 0.092 > R_{Nd} > 0.0 \\ \text{Mean, } R_{Nd} &= 0.0835 \cdot 10^{(-0.5184t-1.099)}, 0.0835 > R_{Nd} > 0.0 \\ \text{Minimum, } R_{Nd} &= 0.096 \cdot 10^{(-0.2989t-1.001)}, 0.096 > R_{Nd} > 0.0 \end{aligned} \quad (7-7)$$

Consolidation Stage

$$\begin{aligned} \text{Maximum, } R_{Nd} &= 0.15 \cdot 10^{(-0.1678t-1.0264)}, 0.15 > R_{Nd} > 0.092 \\ \text{Mean, } R_{Nd} &= 0.1385 \cdot 10^{(-0.1495t-1.03)}, 0.1385 > R_{Nd} > 0.0835 \\ \text{Minimum, } R_{Nd} &= 0.115 \cdot 10^{(-0.15897t-1.1131)}, 0.115 > R_{Nd} > 0.096 \end{aligned} \quad (7-8)$$

Table 7.9 Curve fitting parameters for the normalized rate of displacement from the HWG tubes

		a	b	Finite Value	Difference %		
					a	b	finite value
Filtration stage	Maximum	-0.6910	-1.0852	0.0927	33.31	-1.26	10.91
	Mean	-0.5184	-1.0990	0.0835			
	Minimum	-0.2989	-1.0011	0.0959	-42.34	-8.91	14.79
Consolidation stage	Maximum	-0.1678	-1.0264	0.1501	12.21	-0.29	8.41
	Mean	-0.1495	-1.0293	0.1385			
	Minimum	-0.1590	-1.1131	0.1149	6.32	8.14	-17.03

(4) Curve Fitting for the LWG Tubes

The normalized rate of displacement, R_{Nd} , for the LWG tubes versus time curves are shown in Figure 7.65.

Figure 7.65 Relationships between normalized rate of displacement and time for the LWG tubes during dewatering stage

The curve fitting for the mean values of the test data was conducted and plotted with logarithmic time scale as shown in Figure 7.66. It can be seen that the LWG tubes was in filtration stage when the normalized rate of displacement less than 0.0528 and in consolidation stage when that range of 0.0528 to 0.0775. In filtration stage, the coefficient determination of the fitting curves, R^2 , was of 0.998, scale parameter a of -0.4683, and b of -1.309. In consolidation stage, the coefficient determination R^2 was of 0.996, scale parameter a of -0.1148, and b of -1.3438. The Figure 7.66 was also plotted in a linear time scale as shown in Figure 7.67.

Figure 7.66 Curve fitting for the mean values of the normalized rate of displacement of LWG tubes with the FORE method

Figure 7.67 Linear time scale plot curve fitting for the mean values of the normalized rate of displacement of LWG tubes with the FORE method

Similar to the curve fitting for mean values, the FORE method is also applied to the minimum and maximum values as shown in Figure 7.68 and Figure 7.69. The details of the fitting equations are also shown in these two figures. The two figures are also plotted in linear time scale as shown in Figure 7.70 and Figure 7.71.

Figure 7.68 Curve fitting for the maximum values of the normalized rate of displacement of LWG tubes with the FORE method

Figure 7.69 Curve fitting for the minimum values of the normalized rate of displacement of LWG tubes with the FORE method

Figure 7.70 Linear time scale plot curve fitting for the maximum values of the normalized rate of displacement of the LWG tubes with the FORE method

Figure 7.71 Linear time scale plot curve fitting for the minimum values of the normalized rate of displacement of LWG tubes with the FORE method

The results of the curve fitting using the FORE method for the HWG tubes are summarized in Equation (7-9), (7-10) and Table 7.10. The differences between the measured and predicted values are about $\pm 30\%$ in filtration stage and $\pm 60\%$ in the consolidation stage.

Filtration stage

$$\text{Maximum, } R_{Nd} = 0.0695 - 10^{(-0.511t-1.201)}, 0.0695 > R_{Nd} > 0.0$$

$$\text{Mean, } R_{Nd} = 0.0528 - 10^{(-0.4683t-1.309)}, 0.0528 > R_{Nd} > 0.0 \quad (7-9)$$

$$\text{Minimum, } R_{Nd} = 0.039 - 10^{(-0.555t-1.426)}, 0.039 > R_{Nd} > 0.0$$

Consolidation Stage

$$\text{Maximum, } R_{Nd} = 0.1211 - 10^{(-0.04488t-1.2918)}, 0.1211 > R_{Nd} > 0.0695$$

$$\text{Mean, } R_{Nd} = 0.0775 - 10^{(-0.1148t-1.3438)}, 0.0775 > R_{Nd} > 0.0528 \quad (7-10)$$

$$\text{Minimum, } R_{Nd} = 0.059 - 10^{(-0.1041t-1.601)}, 0.059 > R_{Nd} > 0.039$$

Table 7.10 Curve fitting parameters for the normalized rate of displacement from the LWG tubes

		<i>a</i>	<i>b</i>	<i>Finite Value</i>	Difference %		
					<i>a</i>	<i>b</i>	finite value
Filtration stage	Maximum	-0.5109	-1.2010	0.0695	9.11	-8.26	31.56
	Mean	-0.4683	-1.3091	0.0528			
	Minimum	-0.5552	-1.4257	0.0391	18.57	8.91	-25.98
Consolidation stage	Maximum	-0.0449	-1.2918	0.1212	-60.89	-3.87	56.43
	Mean	-0.1147	-1.3438	0.0775			
	Minimum	-0.1041	-1.6013	0.0589	-9.31	19.16	-23.91

7.8 Conclusions

Model tests were carried out on two types of permeable geosynthetic tubes. In measuring the tensile strengths of the geosynthetic sheets with seams, a new reinforced seaming method is proposed to improve the seam efficiency for the two selected geosynthetic materials. The theoretical methods for analyzing the cross-section and tensile forces at the time of end of filling, or that for calculating the displacement during the dewatering process were proposed and compared with the experimental results. The following conclusions can be drawn from the studies presented in this chapter.

- (1) Based on the tensile strength testing of the LWG and HWG materials, the strain stress relationship of geosynthetic sheets can be divided into three stages: shrinkage stage, elastic stage and failure stage.
- (2) The seam efficiencies using the “J” seam type were only 56.8% and 51.7% for the LWG and HWG specimens respectively. The seam efficiency can increase to 79.5% and 75.8% when the proposed reinforced seam method “RJ” is used. The reinforced seaming method avoids the pin hole weakening, stress concentration and shrinkage on the seamed areas. All of these advantages make the tensile strength increasing then non-reinforced samples.

- (3) Based on the model test results, the displacement of the permeable geosynthetic tube can be divided into two stages: the filling stage and the dewatering stage. The dewatering stage involves at least one of the following two stages: the filtration stage and the consolidation stage.
- (4) The cross-sections of geosynthetic tube at the end of filling can be analyzed using the impermeable membrane theory presented in Chapter 4 by assuming that the seepage of solid or water during filling stage is negligible. The cross-sections predicted using this method agreed reasonably well with the experimental observations.
- (5) The one-dimensional displacement theory proposed by Leshchinsky et al. (1996) for dewatering stage underestimated the experimental values. The mismatch was not only due to the lack of consideration of the changing of width but also because of the assumption that the change in the weight was completely due to water dissipation when calculation the average water content.
- (6) A method to estimate the displacement of permeable geosynthetic tube using the first order rate equation (FORE) method is proposed. The prediction using this method matches the experimental data well.
- (7) The displacement of geosynthetic tube normalized by the height at the end of filling and void ratio of the filling slurry versus time curves fall into a narrow band especially for HWG tubes. Thus, the relationship established by curve fitting may be able to be used for prediction when similar conditions.

CHAPTER 8

DEWATERING OF SLURRY USING PERMEABLE GEOSYNTHETIC TUBES

8.1 Introduction

As discussed in Chapter 2, the permeable geosynthetic tubes provide an ideal medium for the dewatering of digested bio-solids, dredged material, industrial solid wastes, fly ash and coal slurry materials in dike construction, and waste dewatering projects. However, according to the experimental data presented in Chapter 7, the dewatering process of permeable geosynthetic tube is time consuming. Furthermore, the water content of soil after dewatering is still high and sometimes even around the liquid limit. Therefore, the geosynthetic tube dewatering method is not efficient particularly when a single tube is used. This situation may be improved if a method to accelerate the dewatering process can be developed to improve the dewatering efficiency.

In this chapter, two methods to improve the dewatering efficiency are proposed. The first is the stacking method in which permeable geosynthetic tubes are stacked together to use the weight of the tubes on top to preload the tubes below. Although the method of stacking permeable geosynthetic tubes is easy and economical, it is impossible to refill the bottom tube after consolidation. The second method is to accelerate the dewatering process by applying vacuum pressure on the soil inside the geosynthetic tube. Large scale laboratory model tests were carried out to study the dewatering process and evaluate the dewatering efficiency of both.

8.2 Stacking Method

8.2.1 Model Test

Two sets of model tests on stacked permeable geosynthetic tubes (SPT) filled with slurry were tested. The details of the dimensions of the tubes and water content of the filling slurry are shown in Table 8.1.

Table 8.1 Information of stacked geosynthetic tube and filling slurry

Model test No.	Materials	Dimension of top tube($W \times L$, m)	Dimension of bottom tube ($W \times L$, m)	Water Content (%)
SPT1	HWG	0.70×1.35	1.0×2.0	71.6
SPT2	HWG	0.75×1.50	1.0×2.0	89.2

The tensile forces along the circumference of the tube, the pore water pressures on top and bottom of the inner surface of the tube and the displacement of the geosynthetic tubes were measured using the same equipment and method as discussed in Chapter 7 except two sets of strain gauges were used for both the top and bottom geosynthetic tubes as shown in Figure 8.1.

Figure 8.1 Schematic drawing of the location of strain gauges

The model testing setup and testing procedure for Test SPT1 are shown in Figure 8.2. The bottom tube was filled and self-weight consolidated first in the same way as for the case of a single geosynthetic tube as shown in Figure 8.2(a) and (b). After the bottom tube was consolidated under self-weight, another tube was placed on top

and fully inflated with the same type of slurry. The height of the tube after inflation was 0.677 m as shown in Figure 8.2(c) and (d).

(a) Filling of bottom tube

(b) Consolidated bottom tube

(c) Filling of top tube

(d) After filling of the system

Figure 8.2 Filling process of Test SPT1

The contact surface between the two stacked geosynthetic tubes was determined by inserting a set of bended iron bars as shown in Figure 8.3. The bended right-angle part can be scanned by the laser sensor. The height of contact surface can then be calculated by the reading of the laser sensor subtracting the height of the iron bars. The iron bars were small enough so their disturbance to the top tube was negligible.

Figure 8.3 Test method of contact surface between two tubes

8.2.2 Test Results

The measured cross-sections of the Test SPT1 are shown in Figure 8.4. Due to the limitation of the laser sensor, only the top profile of the stacked geosynthetic tubes could be recorded. The bottom geosynthetic tube of model SPT1 was inflated to almost sausage shape after filling for 4.51 *hour* as shown in Figure 8.4(a). Because of the dewatering process of the inflated slurry, the height of the bottom tube reduced until the top surface became flat as shown in Figure 8.4(a) by the profile measured at 44.6 *h*. At time of 99.03 *h*, a top tube was placed on to the bottom tube and inflated with slurry. The water content of the slurry was 71.2%. The height of the top tube after inflation was 0.677 *m* as shown in Figure 8.4(b). Under the weight of the top tube, the bottom tube compressed, but only by 2 *cm*, as shown in Figure 8.4(b). The top tube consolidated till the top surface was flat as shown in Figure 8.4(b) as the profile at time 113.2 *h*.

(a) Cross-sections of bottom tube before stacked

(b) Profile of stacked geosynthetic tubes

Figure 8.4 Cross-section changes with time for the Test SPT1

The displacements of the Test SPT1 tube versus time curve during filling and dewatering processes are presented in Figure 8.5. It can be seen that the self-weight consolidation of the bottom tube almost stopped 11h after filling. After the top tube was placed, the bottom tube settled about 2 cm in about 3h which was measured with the method discussed in Figure 8.4. The small settlement of the bottom tube implies that the effect of the stacking of the top tube on the consolidation of the bottom tube was limited. Thus, stacking may not be effective in improving the dewatering effect of the bottom tube.

The tensile forces versus time curves of the bottom tube in Test SPT1 are plotted in Figure 8.6. Due to the lateral force or expansion by the internal solid filling material, the tensile force on the side of the tube, B2 to B6, increased to the largest values at the end of filling and then reduced during dewatering. Before stacked, the strain gauge B0 and B1 located on the top surface of the bottom tube reduced rapidly and even fell below zero. However, the tensile force at the base, B7 to B8, almost kept constant during filling and dewatering and did not seem to be influenced by the construction process. After stacking of the top tube, the similar phenomena were observed.

Pumping pressure is a control factor for the cross-section and tensile forces along a geosynthetic tube. For impermeable geosynthetic tubes, the internal water pressure can be maintained constant because there is no water dissipation. However, for permeable geosynthetic tubes, the pumping pressures or the excess pore water pressure in the slurry dissipated during dewatering process. The pumping pressures versus time curves during the whole testing process of Test SPT1 are shown in Figure 8.7. It can be seen that, before the placement of the top tube, the pore water pressure at the top and bottom of the inner surface of the bottom tube reached almost a constant value at the end of self-weight consolidation. After the placement of the top tube, the pore water pressure increased and dissipated subsequently with time till to a constant value as shown in Figure 8.7.

Figure 8.5 The height changing of Test SPT1

Figure 8.6 Tensile force changing with time of bottom tube of Test SPT1

Figure 8.7 Pore water pressure on top and bottom inner surface of bottom tube from Test SPT1

Test SPT2 was conducted using the same method as that for Test SPT1 except the water content of filling slurry of 89.2%. The measured cross-sections for Test SPT2 are shown in Figure 8.8. The bottom tube in Test SPT2 was inflated in 1.87 hours and dewatered for 21.43 h (or total time 23.3 h) as shown in Figure 8.8(a). At time of 96.93 h, the top tube was placed on top of the bottom tube and inflated till the total height of 0.606 m as shown in Figure 8.8(b).

Figure 8.8 Cross-section changes with time for the Test SPT2

The displacements of the tube in Test SPT2 during filling and dewatering processes are shown in Figure 8.9. It can be seen that the self-weight consolidation process of the bottom tube were completed in about 6 hours after filling. After the placement of the top tube, the bottom tube settled about 1.9 cm in about 4 hours. The tensile

forces versus time curves for the bottom tube are plotted in Figure 8.10. The pumping pressures versus time curves during the whole test are shown in Figure 8.11.

Figure 8.9 The height changing of the Test SPT2

Figure 8.10 Tensile force changing with time of bottom tube of the Test SPT2

Figure 8.11 Internal pore water pressure on top and bottom inner surface of bottom tube of the Test SPT2

8.2.3 Dewatering Efficiency

The review presented in Chapter 2 has shown that there is no unified theory to predict the dewatering efficiency (Somasundaran, 2006). In this study, the dewatering efficiency, D_e , was calculated basing on the changes of displacement using Equation (8-1):

$$D_e = (H_0 - h) / (H_0 - H_{SF}) \quad (8-1)$$

where H_0 is the initial height of the bottom tube, h is the height at time t , H_{SF} is the final height of the bottom tube.

The dewatering efficiency for the Model Test SPT1 during self-weight dewatering stage was calculated with Equation (8-1) by taking H_0 = the height of bottom tube at the end of filling, and H_{SF} = the height of bottom tube before stacked. The dewatering time, t , was taken from the end of filling. Figure 8.12(a) shows the dewatering efficiency versus time curve for Model Test SPT1 during self-weight dewatering stage. It takes about 3.6 hours for the 80% dewatering efficiency. Similar to the calculation of self-weight dewatering stage, the dewatering efficiency was calculated by taking H_0 = the height of bottom tube before stacked, and H_{SF} = the final height of bottom tube after test. The dewatering efficiency versus time curve for Model Test SPT1 during the stacking period is also shown in Figure 8.12(a). The time was taken from the time when the top tube was placed. It can be seen that for the stacking emthod, only 1.56 hours was required to achieve 80% dewatering efficiency. Therefore, the stacking method is more efficient than the self-weight consolidation method as far as dewatering is concerned.

Figure 8.12(b) shows the dewatering efficiency versus time curves for Model Test SPT2 for both the self-weight dewatering and the stacking periods. The calculation method for Model Test SPT2 was the same as that used for Model Test SPT1. The time taken to achieve 80% dewatering efficiency was 4.0 hours for self-weight consolidation and 1.5 hours for the stacking period.

(a) Model Test SPT1

(b) Model Test SPT2

Figure 8.12 The comparisons of dewatering efficiencies during self-weight dewatering and stacked periods

8.2.4 Properties of Kaolin after Test

The use of permeable geosynthetic tubes for dewatering normally serves two objectives, 1) reduction of the water content of the soil and 2) improvement of the shear strength. Once the slurry is dewatered, the engineering properties of the soil will be improved.

The engineering properties of the kaolin soil in the tube in the two model tests after dewatering were studied. At the end of the test, the permeable geosynthetic tube was cut off from one side in order to take soil samples for testing. The basic steps are shown in Figure 8.13(a) to (d).

Figure 8.13 Testing photograph of basic properties of Kaolin

The water contents of the consolidated kaolin on different positions of the cross-section of the bottom tube from Test SPT1 are shown in Figure 8.14. It can be seen that water content distributions in the kaolin soil are not uniform. The values are higher in the center with an average of 52%, and lower on two sides with an average of 50%. The water content of the soil along the top surface of the top tube had the lowest value of about 46%.

The water contents of kaolin in the bottom tube in Test SPT2 are shown in Figure 8.15. The average water content was 50.5%, on the top, 52% on the two sides, and 54% at the center of the cross-section. The uneven water content distribution was due to not only the filter cake effect, but also the difference in the drainage length.

Figure 8.14 Water contents of consolidated kaolin in the bottom tube of Test SPT1

Figure 8.15 Water contents of consolidated kaolin in the bottom tube of Test SPT2

The lab vane shear test was selected to measure the shear strength of the consolidated kaolin “in-situ”. The shear vane with diameter of 33 mm and height of 50 mm was used as shown in Figure 8.16. A correction factor of 1.346 was used according to standard BS1377-7 (1990). The tests were done along five cross-sections along the bottom tube as shown in Figure 8.17. For each cross-section, three tests were carried out. The results of this lab vane shear tests are presented in Table 8.2. The average shear strength was 18.2 *kPa* for the bottom tube from the Test SPT1 but only 8.5 *kPa* for that from the Test SPT2.

Figure 8.16 Photos of lab vane shear

Figure 8.17 The tested points of vane shear method

Table 8.2 The vane shear strength of consolidated kaolin on bottom tube

<i>Tube No.</i>	<i>Testing Profile</i>	<i>Vane shear strength on side, kPa</i>	<i>Vane shear strength on center, kPa</i>	<i>Vane shear strength on side, kPa</i>
<i>SPT1</i>	<i>1</i>	<i>16.5</i>	<i>23.1</i>	<i>19.8</i>
	<i>2</i>	<i>17.6</i>	<i>24.2</i>	<i>13.2</i>
	<i>3</i>	<i>11.0</i>	<i>19.8</i>	<i>22.0</i>
	<i>4</i>	<i>11.0</i>	<i>11.0</i>	<i>19.8</i>
	<i>5</i>	<i>19.8</i>	<i>22.0</i>	<i>22.0</i>
<i>SPT2</i>	<i>1</i>	<i>8.4</i>	<i>10.0</i>	<i>9.2</i>
	<i>2</i>	<i>12.6</i>	<i>8.4</i>	<i>7.5</i>
	<i>3</i>	<i>8.4</i>	<i>6.7</i>	<i>8.4</i>
	<i>4</i>	<i>6.7</i>	<i>8.4</i>	<i>8.4</i>
	<i>5</i>	<i>7.5</i>	<i>8.4</i>	<i>9.2</i>

8.3 Vacuum Method

The second way to accelerate the dewatering process of soil inside a geotextile tube is to use vacuum pressure. A new vacuum method is proposed in this section by applying the vacuum pressure inside the soil mass. To develop this vacuum method, model tests were carried out. The model test procedure and test results are presented in the following.

8.3.1 Test Procedure

Two HWG tubes, Test VIT1 and VIT2, and two LWG tubes, Test VIT3 and VIT4, were selected for model tests. The details of their dimensions and water content of filling slurry are listed in Table 8.3. The tensile force, pore water pressure on the top and bottom of the tube and the displacement for Test VIT1 and VIT2 were measured with the same equipment as that used for the model tests for single HWG tubes in Section 7.4. The installations of waterproof strain gauges, hydraulic transducers, PCDR, and scanner were also the same as single permeable HWG tubes.

Table 8.3 The information of the vacuum improved geosynthetic tubes

Model test No.	Materials	Width (<i>m</i>)	Length (<i>m</i>)	Water content of filing slurry (%)
VIT1	HWG	1.0	2.0	80.6
VIT2	HWG	1.0	2.0	103.6
VIT3	LWG	0.6	1.2	144.0
VIT4	LWG	0.6	1.2	168.4

The filling and test procedures were also the same as that for single permeable tubes except a frame and prefabricated horizontal drain (PHD) as shown in Figure 8.20(a) & (b) were preinstalled inside the geosynthetic tube. The PHD included a 5-*cm*-diameter perforated tube covered by a 0.3-*mm*-thick geotextile sheet as a filter (Figure 8.18(a)). In order to prevent the PHD from contacting directly the geosynthetic sheet, the PHD was tied to a metal frame as shown in Figure 8.18(b)&(c). The dimension of the frame was 80×28×9 *cm* (L×W×H) as shown in Figure 8.18(b). A small plastic red tube was connected to the PHD for the application of vacuum pressure and discharge of water dissipated under the vacuum

load as shown in Figure 8.18(c) & (d). A vacuum tank, as shown in Figure 8.19(a), was also used to maintain the vacuum pressure and store water discharged from the tube. It should be pointed out that the *PHD* and frame was totally covered by the slurry during filling as shown in Figure 8.20. Therefore, the PHD is completely sealed.

(a) Photos of PHDs

(b) Dimensions of the frame and PHD (unit, *cm*)

(c) PHD and frame

(d) The suction tube

Figure 8.18 The installation of the PHDs and the frame

(a) Vacuum tank

(b) Vacuum pump

Figure 8.19 The photos of vacuum tank and vacuum pump

(a) Panorama view of kaolin covered PHDs

(b) Cross-section view of kaolin covered PHDs

Figure 8.20 The installation of vacuum improved permeable geosynthetic tube

For Tests VIT3 and VIT 4, the PHD used was the same as that for Tests VIT1 and VIT2 except the length of PHD was 60cm. The dimensions of the frame used to support the PHD are $70 \times 27.5 \times 7$ cm ($L \times W \times H$) as shown in Figure 8.21. The rest of the testing setup and test procedure are the same as those used for the model tests presented in Section 7.5.

Figure 8.21 The PHD and frame used for Test VIT3 and VIT4

Figure 8.22 The water tank and suction tube used for Test VIT3 and VIT4

8.3.2 Test Results

For Test VIT1, the pore water pressures versus time curves measured at the top and bottom of the tube and at the sides and on the top of the PHD are plotted in Figure 8.23 and Figure 8.24, respectively. The pore water pressure on the top and bottom peaked at 2.0 and 8.6 *kPa* respectively at the end of filling as shown in Figure 8.23. The internal pore water pressure reached almost a constant value of 3.2 *kPa* at the bottom and 0.3 *kPa* on the top at the end of filling ($t = 5h$). After the application of vacuum pressure, the internal pore water pressures at the sides and on the top of the PHD decreased suddenly. As seen from Figure 8.24, the applied vacuum pressure at the side of PHD was -72.4 *kPa* at the beginning but maintained at -30 *kPa* most of the time because of the difficulties in sealing the vacuum pressure.

The displacement versus time curve for Test VIT1 is presented in Figure 8.25. The vacuum pressure was applied at time 42.1 *h* when the self-weight consolidation was totally stopped. It can be seen that after the application of vacuum pressure the Test VIT1 settled about 3.9 *cm* in about 2 hours.

The tensile forces measured in Test VIT1 versus time curves are shown in Figure 8.26. It can be seen that the tensile force at the side of the tube, B2, B4 and B5,

increased to the peak at the end of filling and then reduced during dewatering. Due to the lateral pressure acting by the inner soil, the tensile force at B2, B4 and B5, still kept in the relative large values during this self-weight dewatering stage. The tensile force at points B7 to B9 along the base almost kept constant during filling and self-weight dewatering process which was mainly due to the frictions. After the vacuum pressure was applied, the tensile forces along the geosynthetic sheet reduce suddenly especially at B1 to B5 and then reached a constant value. The tensile forces at B7, B8 and B9 at the bottom were little influenced by the vacuum.

Figure 8.23 Time versus internal pore water pressure on top and bottom inner surface of the Test VIT1

Figure 8.24 Applied vacuum pressure versus time of the Test VIT1

Figure 8.25 Displacement versus time curve of the Test VIT1

Figure 8.26 Tensile forces versus time curve for the Test VIT1

The pore water pressure changes versus time curves in Test VIT2 are shown in Figure 8.27. After the application of vacuum pressure, the pore water pressure at the bottom of the geosynthetic tube reduced from a constant value, 6 *kPa*, to about 2 *kPa*. For the water pressure on top of the tube, its value changed from 1.5 *kPa* to about 0.0. The vacuum pressure applied to the PHD was -75 *kPa* at the beginning and then maintained at -15 *kPa* most of the time as shown in Figure 8.28. The displacements of Test VIT2 during the filling and dewatering processes are shown in Figure 8.29. It can be seen that the self-weight consolidation of the tube were almost stopped at 22 hours. After the application of vacuum, the tube settled about 4 *cm* in 4 hours. The tensile forces versus time curves for Test VIT2 are plotted in

Figure 8.30. The changes of the tensile forces as measured by the strain gauges are the same as that for Test VIT1.

Figure 8.27 Curve of the time versus pore water pressure on top and bottom of inner surface of the Test VIT2

Figure 8.28 Applied vacuum pressure versus time curve for the Test VIT2

Figure 8.29 Displacement versus time curve for the Test VIT2

Figure 8.30 Tensile forces versus time curve for the Test VIT2

Similar observation was also made for Test VIT3. The applied vacuum pressure in PHD was -65.8 kPa at the beginning and maintained -46.3 kPa all the time. The displacements versus time curve of Test VIT3 during filling and dewatering processes are shown in Figure 8.31. After the application of vacuum, the tube VIT3 settled about 2.5 cm in about 7 hours. The average water content of Test VIT3 versus time curve is presented in Figure 8.32. The calculation and testing method for average water content was the same as that used in Figure 7.46. It can be seen that, before the application of vacuum pressure, the average water content decreased

from 144% at the end of filling to 107.6% after self-weight consolidation. At the end of the dewatering, the average water content reduced to the final value of 78.6%.

Figure 8.31 Displacement versus time curve for the Test VIT3

Figure 8.32 Average water content versus time curve for the Test VIT3

Similar observation was made for Test VIT4. The vacuum pressure applied was -76.3 kPa at the beginning and maintained at -52.5 kPa . The displacements versus time curve for Test VIT4 during the filling and dewatering processes are shown in Figure 8.33. After the application of vacuum, the tube settled about 2.6 cm in about 5 hours. The average water content of Test VIT4 versus time curve is presented in

Figure 8.34. The average water content decreased from 168.4% at the end of filling to 117.8% after self-weight consolidation. After the application of vacuum pressure, the average water content reduced to 75%.

Figure 8.33 Displacement versus time curve for the Test VIT4

Figure 8.34 Average water content versus time curve for the Test VIT4

There is an interesting phenomenon that should be pointed out here. The kaolin inside the geosynthetic tube was shrinking during the vacuum stage. The geosynthetic tube was sucked inwards on to the soil surface as shown in Figure 8.35 which made the tensile force along the whole geosynthetic sheet much smaller.

Therefore, comparing to the stacking method, the vacuum method could effectively avoid the risk of tensile strength failure of tubes.

(a) LWG tube

(b) HWG tube

Figure 8.35 Photos of the geosynthetic sheets sucked inwards onto the soil surface

8.3.3 Dewatering Efficiency

The dewatering efficiencies versus time curves for Test VIT1 for both self-weight consolidation and vacuum dewatering process are shown in Figure 8.36. The calculation method used for Model Test SPT1 was the same as that discussed in Section 8.2.3 except that the height of tube before the application of vacuum pressure instead of that before stacking of the top tube was used. The time was taken from the moment where the vacuum pressure was applied. The time taken to achieve 80% dewatering efficiency was 4.13 hours for self-weight consolidation and 2.6 hours for the vacuum method. Therefore, the vacuum dewatering method is more efficient than the self-weight dewatering method.

Similar observations were made for Test VIT2, VIT3 and VIT4. The dewatering efficiency versus time curves for Model Test VIT2, VIT3 and VIT4 during both self-weight consolidation and vacuum stage are shown in Figure 8.37, Figure 8.38 and Figure 8.39, respectively. The time taken to achieve 80% dewatering efficiency was 3.6 hours for Tests VIT1 and VIT2 during the self-weight consolidation and 2.0 hours for vacuum dewatering method. Similarly for Tests VIT3 and VIT4, the time taken to achieve 80% dewatering efficiency was 7.65 hours for self-weight consolidation and 1.68 for vacuum period as summarized in Table 8.4.

Figure 8.36 Dewatering efficiency of Model Test VIT1 during self-weight and vacuum applied stages

Figure 8.37 Dewatering efficiency of Model Test VIT2 during self-weight and vacuum applied stages

Figure 8.38 Dewatering efficiency of Model Test VIT3 during self-weight and vacuum applied stages

Figure 8.39 Dewatering efficiency of Model Test VIT4 during self-weight and vacuum applied stages

Table 8.4 Dewatering efficiency of the vacuum improved geosynthetic tubes

Tube No.	Vacuum Pressure (kPa)	Displacement due to self-weight consolidation		Displacement due to vacuum pressure	
		$\Delta H(\text{cm})$	Time for $D_e=80\%$	$\Delta H(\text{cm})$	Time for $D_e=80\%$
VIT1	-72.4 ~ -39.5	17.7	4.13	3.8	2.30
VIT2	-76.3 ~ -52.5	11.4	3.15	3.1	1.68
VIT3	-65.8 ~ -46.3	6.2	7.80	2.7	1.86
VIT4	-73.7 ~ -52.6	7.1	7.50	2.6	1.50

8.3.4 Properties of Kaolin after Test

The water contents of the consolidated kaolin collected from different positions were measured. For HWG tube, VIT1 and VIT2, the average water content for all the tested points were 44.03% and 48.08%, respectively. For LWG tube, VIT3 and VIT4, that was 66.87% and 68.6%, respectively as summarized in Table 8.5. The shear strengths of consolidated kaolin were conducted by the UU, UC and lab vane shear tests with their average values shown in Table 8.5. For UU or UC tests, three specimens from different parts of geosynthetic tubes are collected. For the lab vane shear test, the test locations were the same as that shown in Figure 8.17.

Table 8.5 The basic properties of consolidated kaolin

Model test No.	w/c of filling slurry %	w/c of consolidated kaolin %	UC		UU		Vane shear	
			C_u kPa	w/c %	C_u kPa	w/c %	C_u kPa	w/c %
VIT1	80.6	44.03	20.3	45.41	-	-	26.5	44.03
VIT2	103.6	48.08	19.6	49.93	-	-	25.1	48.08
VIT3	144.0	66.87	-	-	20	68.57	-	-
VIT4	168.4	68.60	22.5	71.6	-	-	-	-

8.4 Conclusions

In this chapter, two types of dewatering methods, the stacking method and vacuum method, are evaluated through laboratory model tests. The former uses the weight of the top tube to preload the bottom tube. The later applies vacuum via horizontal drains to consolidate the soil. Through these model tests, the following conclusions can be drawn.

Stacking method

- (1) The stacking method is easy and inexpensive. However, it is impossible to refill the bottom tube after consolidation.

- (2) Using model tests of 2-layers geosynthetic tubes, the time taken to achieve 80% dewatering efficiency was 4.0 hours for self-weight stage and 1.5 hours for the stacking period.
- (3) After placing the top tube, the internal pore water pressure at the top and bottom of the tube increased suddenly which induces additional tensile forces along the bottom of the tube increase. The magnitude of the tensile forces is linked to the filling height of the top tube and the supporting of filled soil mass.
- (4) The water contents of the consolidated soil in the bottom tube were not uniform but with the higher value in the center and lower at the outsides. This is not only due to the filtration cake effect, but also the drainage length.

Vacuum method

- (5) The advantage of the vacuum method is that the geosynthetic tube can be refilled after dewatering. Compared to the two layer stacking method, the vacuum method resulted in lower water content and higher shear strength.
- (6) The model tests of HWG tubes indicated that the time taken to achieve 80% dewatering efficiency was 3.6 hours for self-weight stage and 2.0 hours for the vacuum applied period. However, for LWG tubes, the time taken to achieve 80% dewatering efficiency was 7.65 hours for self-weight consolidation and 1.68 for vacuum applied period.
- (7) After the application of vacuum pressure, the kaolin inside of the geosynthetic tube shrunk and the geosynthetic tube was sucked inwards on to the soil surface which made the tensile force along the whole geosynthetic sheet to reduce. Compared with the stacking method, the vacuum method could reduce the requirement on tensile strength of the geotextile used for the tube.

CHAPTER 9

CONCLUSIONS AND RECOMMENDATIONS

9.1 Conclusions

Geosynthetic tubes or mats have been widely used in projects for coastal protection, dike construction, flood control and dewatering waste sludge in many countries. The geosynthetic tubes can either be impermeable or permeable. For the former, air, water or slurry is used to inflate the tubes. For the later, concrete mortar, sand or mud is used to fill the tubes which allow pore water pressure to be dissipated. Model tests and analytical or numerical studies for both cases were carried out to improve the understanding of the performance of geosynthetic tubes or mats and develop reliable design tools. These included a series of model tests conducted on impermeable geosynthetic tubes, derivation of closed-form solutions for the design of impermeable geosynthetic tubes, evaluation of the effect of deformable foundation on the design of geosynthetic tubes, model tests and analytical analyses of 2-layer stacked impermeable geosynthetic tubes resting on rigid foundation, model tests of permeable geosynthetic tubes and methods of analysis. Geosynthetic tubes were also used to dewater waste sludge. Methods to accelerate the dewatering process of kaolin sludge using permeable geosynthetic tubes were also studied. The following conclusions can be made from the studies:

Impermeable geosynthetic tube & mat on rigid foundation

- (1) Based on the model tests on impermeable geosynthetic tubes, a reference cross-section is defined as the highest cross-section that the geosynthetic tube could be inflated without a pumping pressure. When the height of a geosynthetic tube is lower than the reference cross-section, the top surface will be flat and this special configuration is called geosynthetic mat in this study.
- (2) The height of the geosynthetic tubes in the model tests varied with even a small change in pumping pressure particularly when the pressure was very small. The tensile force of geosynthetic tube at the two bottom corners just about to turn off from the ground had the largest value which possible due to the influence from the bending of strain gauges. Because of the influence from the friction between geosynthetic sheets and concrete floor, the tensile forces along the contact surface with ground surface reduced from bottom corner to the center. Generally, for geosynthetic tubes, the larger the pumping pressure the more uniform the tensile forces.
- (3) Closed-form solutions for the design of impermeable geosynthetic mats resting on rigid foundation were derived. Using the solutions, the tensile force along the mats and the geometry of the cross-section including width, height, perimeter, and area can be calculated. The cross-section of the geosynthetic mat obtained from the closed-form solutions agrees well with those from the experiments. The tensile forces from model test were quite scatter. However, the tensile forces from close-form solutions range on two sides of the model tests results.

Impermeable geosynthetic tube & mat on deformable foundation

- (4) Most geosynthetic tubes were used over soft ground. The displacement of the foundation soil may affect the design of the geosynthetic tube. The use of

Winkler foundation to model the effect of deformation of foundation soil was evaluated. The study shows that the effect of the deformation of foundation soil is only need to be considered when the modulus of subgrade reaction, K_f , is less than 1000 kPa/m . When K_f is greater or equal to 1000 kPa/m , the foundation can be considered rigid. This conclusion is also applicable to geosynthetic mats too.

- (5) A e-log p method was also proposed to model the deformation of the foundation soil. In this method, the e-log p relationship determined by one-dimensional consolidation tests was used. In theory, this method has the following advantages over the Winkler method: a). the parameters involved in this method such as compression index and over consolidated ratio can be obtained from laboratory tests; b) the deformation of soil is modeled as nonlinear; c) the variation of surcharge pressure within soil mass can be considered. The prediction of this method also shows a better agreement with the model test results than the Winkler method.

2-layer stacked impermeable geosynthetic tube & mat

- (6) Based on the model test results, the deformation pattern of the 2-layer stacked impermeable geosynthetic tubes can be classified into three cases: horizontal contact surface, convex contact surface and concave contact surface. Analytical solutions for each case were developed. The cross-sections given by the analytical solutions agree well with the model test results. The predicted tensile force also agree well with the model test results except that the tensile force was grossly overestimated for the bottom tube and underestimated for the top tube.

Permeable geosynthetic tube on rigid foundation

- (7) A new reinforced seaming method was developed to improve the strength of seamed geosynthetic sheets. This so-called “RJ” method increased the seam

efficiency of the two selected geosynthetic materials from the normal 50% to over 75%.

- (8) Based on the model test observations, the displacement of the permeable geosynthetic tube can be divided into two stages: the filling stage and the dewatering stage. The dewatering stage involves at least one of the following two stages: the filtration stage and the consolidation stage.
- (9) Theoretical methods for analyzing the cross-section at the end of filling were proposed and compared with the model test results. The cross-sections predicted using this method agrees well with the experimental observations.
- (10) A method to estimate the displacement of permeable geosynthetic tube during the dewatering stage was proposed. In this method, the first order rate equation (FORE) method was adopted. The prediction using this method matches the experimental data well. The displacement of geosynthetic tube normalized by the height at the end of filling and void ratio of the filling slurry versus time curves fall into a narrow band. Thus, the relationship established by curve fitting may be able to be used to predict the displacement of the tube during dewatering under similar conditions.

Dewatering of slurry using permeable geosynthetic tube

- (11) The stacking method for dewatering of waste sludge was evaluated using model tests. Using 2-layers geosynthetic tubes, it was observed that the time taken to achieve 80% dewatering efficiency was 4.0 hours for self-weight stage and 1.5 hours for the stacking period. After placing the top tube, the internal pore water pressure at the top and bottom of the tube increased suddenly which induced additional tensile forces along the bottom of the tube. The magnitude of the tensile forces was linked to the filling height of the top tube and the supporting of filled soil mass. The water contents of the

consolidated soil in the bottom tube were not uniform but with the higher water content in the center and lower at the boundary.

- (12) A vacuum method for dewatering was proposed. In this method, vacuum was applied to consolidate the soil inside the tube. The advantages of the vacuum method are that the dewatering process can be accelerated and the geosynthetic tube can be refilled after dewatering. Compared with the two layer stacking method, the vacuum method resulted in consolidated soil with lower water content and higher shear strength. Under the vacuum pressure, the tensile force in the geosynthetic sheet reduced. Thus the vacuum method could reduce the requirement of tensile strength of the geotextile used for the tube.

9.2 Recommendations

The present work attempted to investigate the behavior of the permeable and impermeable geosynthetic tube & mat based on model tests, analytical/numerical analysis. Some recommendations and suggestions for further studies are proposed as follows.

- (1) All the theoretical derivations of impermeable geosynthetic tube were made based on the assumptions that the tensile forces along the whole cross-section were constant. However, this assumption is only valid when the friction between geosynthetic sheets and foundation are neglected. Theoretical studies to consider the frictions between tubes and ground should be carried out. Experimental studies on the influence of the interface friction on the design of geosynthetic tube or mat should also be carried out. One way is to compare the behavior of two identical tubes, but one with rough interface with the base and another with relatively smooth base.
- (2) Super glue is normally used to stick strain gauges onto the geotextile. However, this study shows the use of super glue stiffens the attached areas locally. This

will affect the strain measurements when the strain gauge bends together with the geotextile. A better method for using strain gauges to measure tensile force along the geotextile tube in the bending sections should be developed and adopted for future work.

- (3) The e-log p method is an improvement over the Winkler method as the non-linear stress-deformation behavior of foundation soil can be considered. The soil parameters are also much easier to determine. However, the e-log p method is only valid for one-dimensional conditions. A more reliable method has to be proposed.
- (4) Kaolin was used in the model tests in this study because the same physical and mechanical conditions of the soil prepared from kaolin could be ensured. However, model tests with the soil to be used in practice should be carried out to verify the findings obtained from tests using kaolin can be extended to that using other soils.
- (5) The reinforced seam method developed in this study avoids the pin hole weakening and increases the tensile strength of the seamed geotextile. However, the study was verified for two types of geotextile materials. Further verification of the suitability of the method to other types of geotextile materials should be carried out in future works.
- (6) The usefulness of the vacuum dewatering method proposed in this study should also be verified using different types of wastes or clay sludge.

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**APPENDIX A MODEL TEST RESULTS OF 2-LAYER
STACKED IMPERMEABLE GEOSYNTHETIC TUBES**

(a) Cross-section

(b) Tensile force

Figure A 1 Comparison between test and analytical results of ST1-1

(a) Cross-section

(b) Tensile force

Figure A 2 Comparison between test and analytical results of ST1-2

(a) Cross-section

(b) Tensile force

Figure A 3 Comparison between test and analytical results of ST1-3

(a) Cross-section

(b) Tensile force

Figure A 4 Comparison between test and analytical results of ST1-4

(a) Cross-section

(b) Tensile force

Figure A 5 Comparison between test and analytical results of ST1-5

(a) Cross-section

(b) Tensile force

Figure A 6 Comparison between test and analytical results of ST1-6

(a) Cross-section

(b) Tensile force

Figure A 7 Comparison between test and analytical results of ST1-7

(a) Cross-section

(b) Tensile force

Figure A 8 Comparison between test and analytical results of ST1-8

(a) Cross-section

(b) Tensile force

Figure A 9 Comparison between test and analytical results of ST1-9

(a) Cross-section

(f)

Figure A 10 Comparison between test and analytical results of ST1-10

(a) Cross-section

(b) Tensile forces

Figure A 11 Comparison between test and analytical results of ST1-11

(a) Cross-section

(b) Tensile force

Figure A 12 Comparison between test and analytical results of ST1-12

(a) Cross-section

(b) Tensile force

Figure A 13 Comparison between test and analytical results of ST1-13

(a) Cross-section

(b) Tensile force

Figure A 14 Comparison between test and analytical results of ST1-14

(a) Cross-section

(b) Tensile force

Figure A 15 Comparison between test and analytical results of ST1-15

(a) Cross-section

(b) Tensile force

Figure A 16 Comparison between test and analytical results of ST1-16

(a) Cross-section

(b) Tensile force

Figure A 17 Comparison between test and analytical results of ST2-1

(a) Cross-section

(b) Tensile force

Figure A 18 Comparison between test and analytical results of ST2-2

(a) Cross-section

(b) Tensile force

Figure A 19 Comparison between test and analytical results of ST2-3

(a) Cross-section

(b) Tensile force

Figure A 20 Comparison between test and analytical results of ST2-4

(a) Cross-section

(b) Tensile force

Figure A 21 Comparison between test and analytical results of ST2-5

(a) Cross-section

(b) Tensile force

Figure A 22 Comparison between test and analytical results of ST2-6

(a) Cross-section

(b) Tensile force

Figure A 23 Comparison between test and analytical results of ST2-7

(a) Cross-section

(b) Tensile force

Figure A 24 Comparison between test and analytical results of ST2-8

(a) Cross-section

(b) Tensile force

Figure A 25 Comparison between test and analytical results of ST2-9

(a) Cross-section

(b) Tensile force

Figure A 26 Comparison between test and analytical results of ST2-10

(a) Cross-section

(b) Tensile force

Figure A 27 Comparison between test and analytical results of ST2-11

(a) Cross-section

(b) Tensile force

Figure A 28 Comparison between test and analytical results of ST2-12

(a) Cross-section

(b) Tensile force

Figure A 29 Comparison between test and analytical results of ST2-13

(a) Cross-section

(b) Tensile force

Figure A 30 Comparison between test and analytical results of ST2-14

(a) Cross-section

(b) Tensile force

Figure A 31 Comparison between test and analytical results of ST2-15

(a) Cross-section

(b) Tensile force

Figure A 32 Comparison between test and analytical results of ST2-16

(a) Cross-section

(b) Tensile force

Figure A 33 Comparison between test and analytical results of ST2-17

(a) Cross-section

(b) Tensile force

Figure A 34 Comparison between test and analytical results of ST2-18

(a) Cross-section

(b) Tensile force

Figure A 35 Comparison between test and analytical results of ST2-19

(a) Cross-section

(b) Tensile force

Figure A 36 Comparison between test and analytical results of ST2-20

(a) Cross-section

(b) Tensile force

Figure A 37 Comparison between test and analytical results of ST2-21

(a) Cross-section

(b) Tensile force

Figure A 38 Comparison between test and analytical results of ST2-22

(a) Cross-section

(b) Tensile force

Figure A 39 Comparison between test and analytical results of ST3-1

(a) Cross-section

(b) Tensile force

Figure A 40 Comparison between test and analytical results of ST3-2

(a) Cross-section

(b) Tensile force

Figure A 41 Comparison between test and analytical results of ST3-3

(a) Cross-section

(b) Tensile force

Figure A 42 Comparison between test and analytical results of ST3-4

(a) Cross-section

(b) Tensile force

Figure A 43 Comparison between test and analytical results of ST3-5

(a) Cross-section

(b) Tensile force

Figure A 44 Comparison between test and analytical results of ST3-6

(a) Cross-section

(b) Tensile force

Figure A 45 Comparison between test and analytical results of ST3-7

(a) Cross-section

(b) Tensile force

Figure A 46 Comparison between test and analytical results of ST3-8

(a) Cross-section

(b) Tensile force

Figure A 47 Comparison between test and analytical results of ST3-9

(a) Cross-section

(b) Tensile force

Figure A 48 Comparison between test and analytical results of ST3-10

(a) Cross-section

(b) Tensile force

Figure A 49 Comparison between test and analytical results of ST3-11

(a) Cross-section

(b) Tensile force

Figure A 50 Comparison between test and analytical results of ST3-12

(a) Cross-section

(b) Tensile force

Figure A 51 Comparison between test and analytical results of ST3-13

(a) Cross-section

(b) Tensile force

Figure A 52 Comparison between test and analytical results of ST3-14

(a) Cross-section

(b) Tensile force

Figure A 53 Comparison between test and analytical results of ST3-15