ANALYSES ON THE STABILITY OF IRRIGATION CHANNEL EMBANKMENT WITH GEOSYNTHETIC REINFORCEMENT

by

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LIST OF SYMBOLS

Symbol	Description
σ'	Effective stress
σ	Total stress
u	Pore water pressure
S	Shear strength
c'	Effective cohesion
ϕ'	Effective angle of internal friction
F_s	Factor of safety
τ	Shear stress
i_e	Escape hydraulic gradient
i_{cr}	Critical gradient
G_s	Specific gravity
e	Void ratio
γ'	Submerged unit weight
γ_w	Unit weight of water
M_R	Resisting moment
M_D	Driving moment
T_{allow}	Maximum allowable tensile strength
σ_h	Horizontal stress
σ_v	Vertical stress
K_0	Relationship between horizontal and vertical stress
ν	Poisson's ratio
ε	Normal strain
γ	Engineering shear strain
u_a	Pore air pressure
u_w	Pore water pressure
E	Elastic modulus
S_r	Resisting shear force
S_m	Mobilized shear force
$S.F$	Safety factor

ϕ^b	Angle defining the increase in shear strength for an increase in suction
θ	Volumetric water content
EI	Flexural rigidity
A	Cross section area
EA	Axial rigidity
L	length
Π_a	Strain energy
u	Axial displacement along the geotextile
x'	Distance along the geotextile
ϕ	Internal friction angle
h	Total hydraulic head
k_x	Unsaturated hydraulic conductivities for the x-direction
k_y	Unsaturated hydraulic conductivities for the y-directions
m_w	Slope of the water volume characteristic curve
t	Time
n	Porosity
S	The degree of saturation
θ_r	Residual volumetric water content
θ_s	Saturated volumetric water content
Ψ	Negative pore water pressure
a	Curve fitting parameter
n	Curve fitting parameter
m	Curve fitting parameter
k_s	Saturated hydraulic conductivity
θ_p	volumetric water content at the halfway point of the volumetric water content function
Ψ_p	Suction at the halfway point of the volumetric water content function
M_v	Slope of pore water pressure line in positive pressure

LIST OF ABBREVIATIONS

Abbreviation	Description
USACE	United States Army Corps of Engineers
FEM	Finite Element Method
PVC	Polyvinyl Chloride
UV	Ultraviolet
HDPE	High Density Polyethylene
GCL	Geosynthetic Clay Liner
DID	Department of Irrigation and Drainage Malaysia
USDA	United States Department of Agriculture
BS	British Standards
USCS	Unified Soil Classification System

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ANALISA KESTABILAN TAMBAK TERUSAN PENGAIRAN BER TETULANG GEOSINTETIK

ABSTRAK

Dalam saliran pengairan, aras air dalam saliran selalunya berubah bergantung kepada keperluan air oleh petani. Perubahan aras air, terutamanya semasa aliran jatuhan deras boleh meningkatkan ketidakstabilan saliran pengairan. Tambak bagi saliran pengairan ini menghadapai dua masalah iaitu ketidakstabilan dan kadar resapan yang tinggi di bahagian hilir ban. Rancangan Pengairan Kerian melibatkan kawasan seluas kira-kira 23, 359 hektar dan merupakan rancangan ketiga terbesar di Malaysia. Panjang keseluruhan saluran utama dalam rancangan pengairan Kerian ialah 22.8 km. Semasa pemasangan meter aliran dalam struktur masukan air, aliran jatuhan deras menyebabkan beberapa bahagian ban saliran pengairan ini runtuh. Kajian kes telah dijalankan untuk menentukan sebab-sebab kegagalan ban saliran dan mencari sebab utama ketidakstabilannya. Cerapan data terperinci kawasan yang terlibat telah dikaji. Analisis lanjutan telah dijalankan dengan gabungan tiga model matematik; Seep/W, Sigma/W dan Slope/W untuk menentukan penyelesaian yang paling realistik. Penggabunagn model-model ini mampu mensimulasi hampir kesemua spesifikasi dan kesan-kesan geotekstil dalam menguatkan ban saliran pengairan. Analisa mendapati dengan menggunakan dua lapisan geotekstil bukan-tenun mampu menyelesaikan masalah ketidakstabilan. Selanjutnya, tambahan satu lapisan geomembran PVC perlu di dilapis dan dibengkokkan berhampiran hilir ban kerana ianya mampu mengurangkan resapan dengan berkesan. Dalam kajian ini, gabungan komprehensif oleh tiga model matematik, menghala kepada model dengan kedua-dua kesan tersebut. Dapat disimpulkan bahawa untuk mencapai permodelan yang paling realistik untuk penguat geotextile, kedua-dua kekuatan regangan dan kemampuan saliran geotekstil perlu diambil kira bersama.

ANALYSES ON THE STABILITY OF IRRIGATION CHANNEL EMBANKMENT WITH GEOSYNTHETIC REINFORCEMENT

ABSTRACT

In irrigation channels, the water level in the channel is always changing depend on the water demand by farmers. Changing of water level, especially during the rapid drawdown could increase the instability of irrigation channel embankments. The embankment of this irrigation channel faced two main problems which were instability and high seepage in toe. Kerian irrigation scheme covers an area of about 23,359 hectares which is the third largest granary areas of Malaysia. It is estimated that the total length of the main channel of the Kerian irrigation scheme is 22.8km. During the installation of a flow meter in the intake structure, the rapid drawdown caused several part of the irrigation channel embankment collapsed. Due to that, a case study had been conducted to determine the possible causes of failure of channel embankments and to find the main reasons of their instability. The detail data was monitored and collected of the affected site. Based on the information, further analysis has been conducted by integrating of three mathematical models which are Seep/W, Sigma/W and Slope/W to determine the most realistic solution. This integration was be able to simulate almost all specifications and effects of geotextile in reinforced irrigation channel embankment. In the analyses, it was found that utilizing two layers of nonwoven geotextile could solve the existing instability problem. Further, additional one layer of PVC geomembrane should be laid and bended near the toe of the embankment in order to decrease seepage significantly. In this study, a comprehensive integration of three models lead to a total solution that account all effects of geotextile together which are drainage ability, tensile strength and geotextile-soil interface friction. In sum, to reach the most realistic application of geotextile reinforcer, all effects of geotextile should be taken into consideration simultaneously in modeling practices.

CHAPTER 1
INTRODUCTION

1.1 Embankments in water conveyance routes

The construction of embankments is recorded in histories of most early civilization. Flood banks were constructed on the Yellow River in China as early as 600 BC, and their construction was brought under unified control by Han Dynasty in 69 BC. In Britain the Romans first built embankments to control flooding and subsequently many kilometers of banks were built to protect the low-lying marsh areas in the Fens and Somerset levels (Brookes, 1990).

Constructing embankments is to artificially increase the capacity of a channel, so that more flows which would normally have spread onto adjacent areas are now confined. They are one of the oldest forms of flood protection, used in either rural or urban areas provided that there is sufficient space for construction. Some of the great channels and rivers of the world have extensive embankment systems such as those that extend for more than 1000 km alongside the Nile, 700 km along the Hwang He, 1400 km on the Red River in Vietnam, and 458 km on the Narmada irrigation channel in India. They are key components in flood control systems along the lower courses of large rivers such as Mississippi, Missouri and Sacramento Rivers in the United States and are intended to protect major towns and cities which have become established on wide floodplains (Brookes, 1990).

1.2 Type of embankments

According to (Razvan, 1989), there are two types of embankments as follow:

1- Homogenous, basically of a single kind of material.

2- Zoned, consisting of a central impervious core, flanked by prisms of pervious material.

1.2.1 Homogenous embankments

Homogeneous embankments are constructed entirely or almost entirely of a single embankment material. So they are named homogeneous to distinguish them from zoned embankments which contain different materials in different portions of the embankment. Homogenous embankments have been built since the earliest times and are used today whenever only one type of material is economically available. Although some of the highest embankments and dams which are constructed are essentially homogenous, but homogenous configuration are used most often in embankments with low to moderate height. Low embankments are almost always made homogenous, because their construction tends to become unduly complicated if they are zoned (Razvan, 1989).

Sometimes large sections of a homogenous embankment are completely separated from each other by thin bands of more pervious material provided as internal drains. Such drains actually give the embankment many of the benefits of a zoned embankment (Razvan, 1989).

There are two groups of homogenous embankment:

1- Embankment of impervious materials

2- Embankment of pervious materials

1.2.1.1 Embankment of impervious materials

Clay and silt are not the only impervious materials used in embankments. Mixtures of coarse-grained soils with 10% of particles smaller than 0.074mm are virtually impervious (Razvan, 1989). So these soils can be used as impervious soils to construct embankments.

In the light of construction, the first trend would be to build the embankment of impervious materials is identifying abundant sources near the constructing site. The principal design problem for these embankments is controlling pore water pressure using drains and filters.

1.2.1.2 Embankment of pervious materials

These embankments are suitable for place that, pervious materials, mixture of sand and gravel are abundantly available and impervious materials are scarce near the site. An embankment built of pervious materials must be provided with watertight elements and as are shown in Figure 1.1 these are two kinds:

1- Impervious membranes, placed on the upstream slope
2- Impervious core or membrane, built inside the embankment

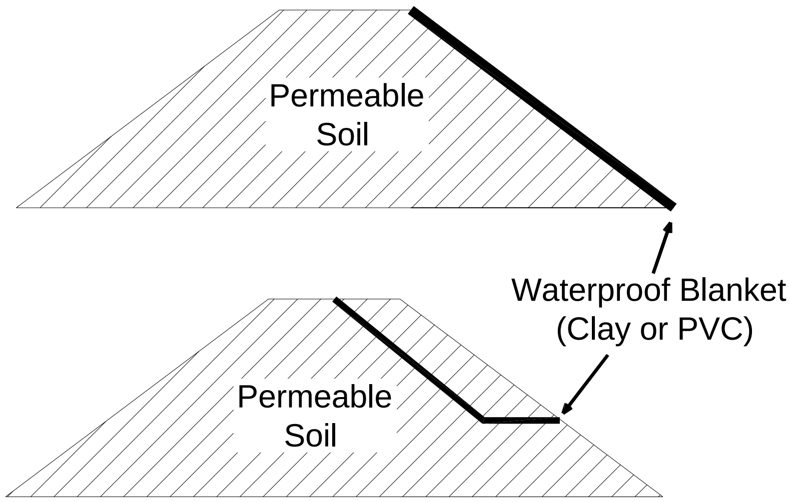

Figure 1.1: Embankment of pervious materials with impervious membrane

1.2.2 Zoned embankments

These types of embankments are made of permeable soil with an impermeable core. The permeable soil should be stable itself, and the core forms the actual seepage barrier (Waterways & Wetlands, 2006). While it is desirable to have soft and waterproof material for the core, it is equally desirable to have a strong and easily drained material for the shoulders which have to support the core. Coarse-grained soil is required, however several difficulties could arise if it is placed next to a soft clay core because the clayey materials could be lost into the interstices of the shoulder fill. It becomes usual to select the finest fraction from the borrow pit fill to place next the core and keeping the coarsest material for the outer parts of the shoulders. The success of this approach are very dependent on the types of soil available near the site that could be used as fill (Penman et al., 1999). Different kinds of zoned embankments are shown in Figure 1.2.

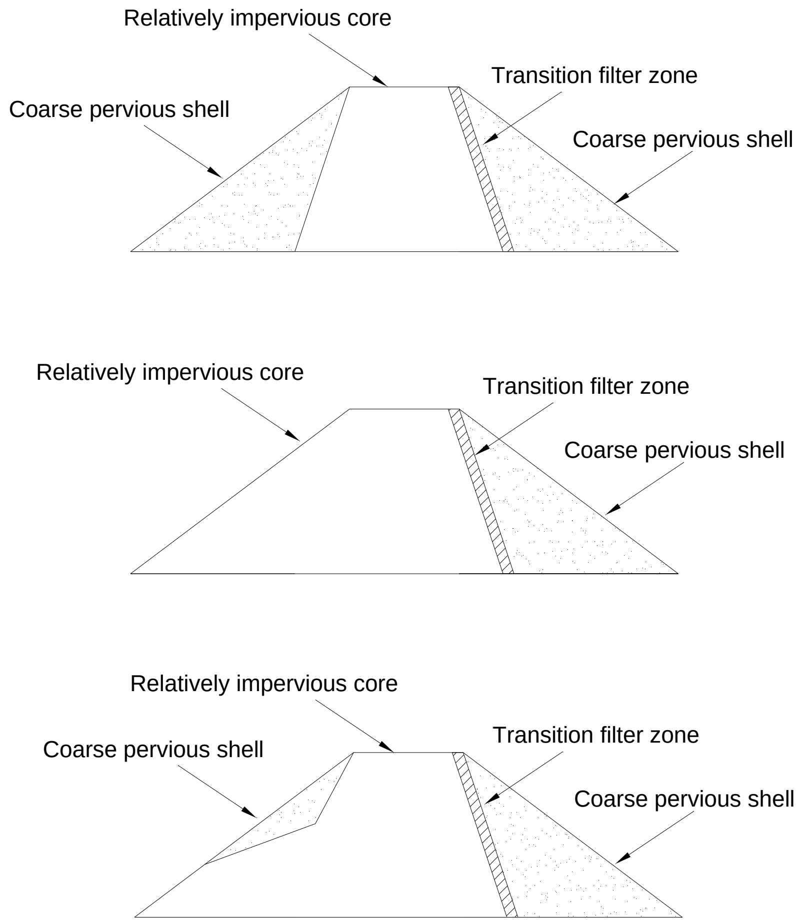

Figure 1.2: Some methods of constructing zoned embankments

1.3 Materials of channel embankment construction

The construction of embankment and the materials for construction are depending on many factors. There must be a clear need for the embankment, sufficient fund to build it, political stability, acceptable design, sufficient workforce as well as a suitable site and materials for construction (Penman et al., 1999).

Most materials of channel embankments are clay, silt, sand and gravel that form the main portion of the embankment body, concrete as an impermeable core or membrane and asphalt as an impermeable membrane.

In the last two decade geomembrane products have been used successfully as watertight elements on the upstream slopes of embankment of pervious soil (Razvan, 1989). Furthermore, geotextile and other geosynthetic materials are used in the embankment structure to provide the required stability or degree of protection.

1.4 Problems of old embankments

Large channels with high and wide embankments can control surrounding flow and reduce flood risk, however, they are expensive and have a considerable adverse to economic and beneficial land use, especially in places which lands are expensive.

Old embankments often made with mild slope to increase slope stability, decrease slope erosion, and deposits high flows. Based on this property they usually require too much land for construction. On the other hand if construct channel embankments with a minimum width and steep slope, collapse can occurs due to low stability, effect of pore water pressure in rapid drawdown and lack of water pressure force in empty condition. New embankments should therefore be more economical and maintained the required safety and functions.

1.5 Objective of the study

In this investigation, causes and conditions of channel embankments instability will be taken into consideration and a detail study on embankment of Kerian irrigation channel will be conducted as a case study.

- To study the causes of failure of channel embankments and finding main reasons of their instability.
- To determine the different existing and probabilistic conditions which collapse can occurs in irrigation channel embankment.

- To estimate the different safety factors of various conditions of embankment by using most appropriate mathematical model.
- To estimate the effects of reinforcing irrigation channel embankment by geosynthetic materials and determine on the usage of most appropriate geosynthetic.

To gain these goals, various geosynthetics such as geoweb, geomat and geotextile will be nominated and their usage will be described. Further, by using geosynthetics, new reinforced embankment with previous soil specifications and dimensions will be designed for this study.

By analyzing the new reinforced embankment and estimating its performance and and comparing with results of analyses of existing embankment, the benefits of using geosynthetic materials will be indicated. Further, it would be found that what proposed reinforcement method is the best for reinforcing Kerian irrigation channel embankment.

1.6 Outline plan of research

Investigation procedure is including of many steps those are shown as the flow chart in Figure 1.3.

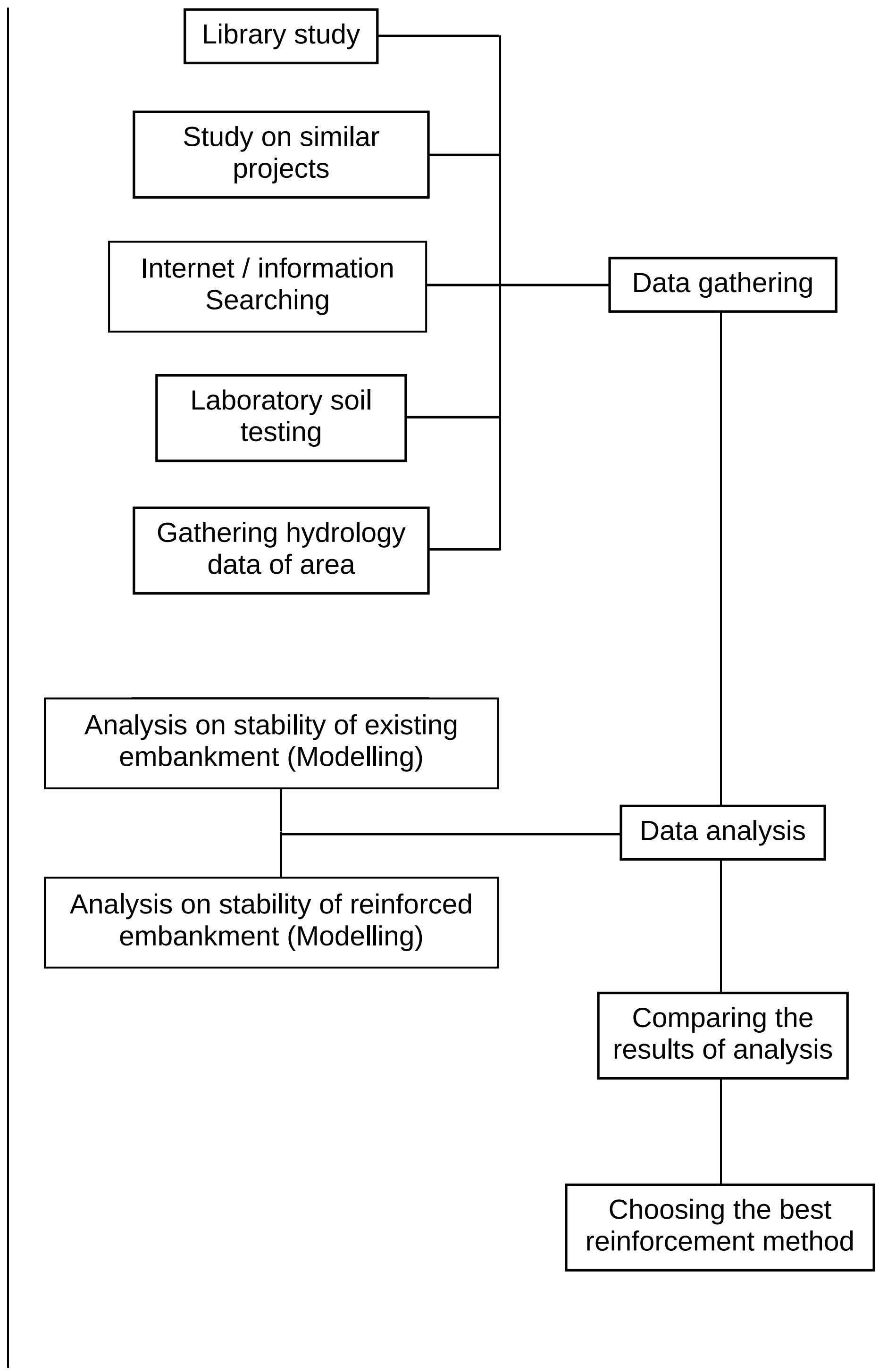

Figure 1.3: Outline plan of research

CHAPTER 2
LITRETURE REVIEW

2.1 Introduction

From old days, man used to protect surface water supplies specially rivers using embankments. He needs to protect the embankments against erosion and collapse. The response to this need is the main reason of developing many protection manners using natural materials along with modern materials and engineering systems.

The traditional methods are usually well integrated with the local environment, but they should be changed or extend by new materials and new systems because of changes in organizational structures, increasing demand for cost effectiveness, more required constructing speed, more stability and safety factors.

Nowadays, new materials and systems are being developed. Some of them have been adopted for embankment protection purpose and some of them have been designed specially for this goal. These materials are used in different situation of embankment instability based on the type of embankment destruction.

2.2 Channel embankment instability

There are many surface erosion and mass failure reasons which all of them lead to instability of channel embankments. That is why in this study, limitation of mass failure and collapse of embankment are the main target. The reasons of surface erosion are avoided and this study is toward in mass failure. There are several reasons of mass failure of channel embankments such as follows:

1- Surface water and ground water regime (Hemphill and Bramley, 1990)

2- Surcharge loading (Hemphill and Bramley, 1990)

3- Mechanical actions like freeze and thaw, animal and man drilling, boating and ice crash.

Among these mentioned reasons, the first reason is the most important among the others, because according to channel embankment histories, changing in water level, seepage and overtopping flow are the main reasons of most channel embankment collapse.

Although the last two reasons are rare, they can cause problems for channel embankments. For example, in 2004 at Llangollen Channel in Wales, badgers drilled the embankments and caused damage to the embankments and it was reported that the damage estimated was about 500,000 UK Pound (BBC, 2004).

Burrowing animals have been possible cause of piping failures in a number of small embankments but have not caused trouble in major embankments because animal holes do not penetrate to a great depth. The worst pests are muskrats, badgers, and ground squirrels (Sherard et al., 1963).

2.3 Mass failure in channel embankments

Failures of natural and man made embankment slopes are generally attributed to activities that result in either an increase in soil stress or a decrease in soil strength. The specific causes of slope instability are varied depend on the nature of the soil, pore water pressure, climate, and stress within the soil mass.

2.3.1 Surface water and ground water regime

The high pore water pressure in embankment material, especially after a rapid drawdown of the water level in channel, will decrease the effective stress in the materials and this can make the embankment disposed to mass failure. Further, high hydraulic conductivity from steady seepage can encourage piping and scouring the toe.

Rain water, overflowed and surface water can infiltrate into the embankment, especially through the cracks and chinks. Subsequently, they could cause for the increasing of unit weight of the soil and further increasing the pore water pressure. Therefore, the strength of the material will reduce and combine with increased weight, will trigger the mass failure.

Embankments composed of cohesive materials usually have problem on the effect of pore water pressure due to rapid drawdown. On the other hand, embankments composed of non-cohesive silty sands or sandy silts materials are most prone to piping or suffusion due to steady seepage. Similar failure rarely occur in embankments that composed from gravel or coarse medium sands, because the lift forces usually are less than submerged unit weight of material. It is important that where fine-grained are removed by suffusion the material with larger voids will be more susceptible to surface erosion (Wan and Fell, 2008; Fell, 2005).

2.3.2 Surcharge loading

Temporary or permanent loading on the top of the embankment will increase its susceptibility to mass failure. If loads be higher than acceptable load amounts on the slope, it can leads to mass failure of embankment. Surcharge loading leads to increasing in shear stresses within the embankment and its foundation due to the weight of the filled soil. If the shear stress force exceeds the strength of the materials, sliding of the embankment or its foundation may occur and resulting in the displacement of large portions of the embankment.

The embankment collapse due to this reason during embankment life is very rare. It is because the usage of embankment is clear before construction design, so the embankment designer will consider appropriate stability of embankment with regard to its usage and probabilistic load which will be applied on the crest of the embankment. However, the main collapse due to loading is during the embankment construction or exactly after construction. At the end of construction, due to the weight of saturated or semi saturated soil is increased, the water should be drained to consolidation of the embankment being completed. If water could not be drained, it will make excess pore water pressure. This excess pore water pressure will leads to instability of embankment. Actually in this condition of loading, collapse will be cause by the effect of pore water pressure, so that it can be counted as one of the condition of collapse.

2.3.3 Pore water pressure and piping

As mentioned, pore water pressure and piping can be counted as main reasons of mass failure of embankments. The comprehensive explanations of the failure by these two reasons are as following.

a. Failure by effect of pore water pressure

In high permeable soils, water can be drained easily. Due to this ability of drainage, during the rapid drawing down of water level, the ratio of pore water pressure in embankment material over the embankment external water level will be approaching zero. Since the pore water pressure is rather zero, consequently the change in effective stress will be equal to the change in the total stress.

In low permeability soils, water can not be drained easily. In contrast of the last condition, the pore water pressure in embankment material over the embankment external water level will not be zero during the drawing down of water level.

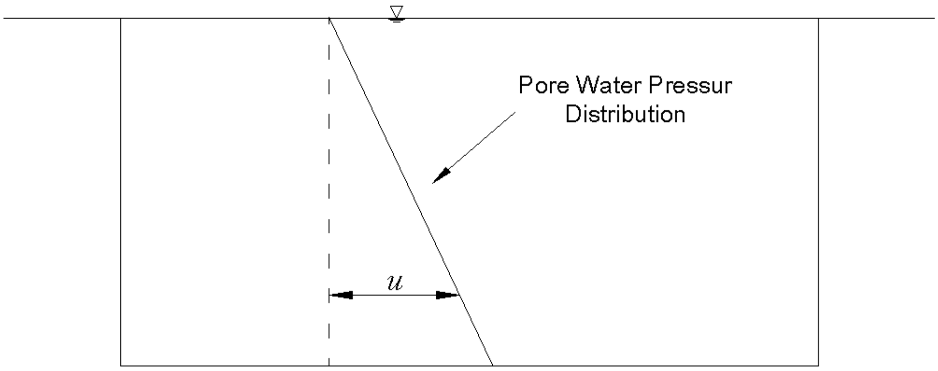

Figure 2.1: Distribution manner of pore water pressure

The equation between effective stress and total stress are as fallow:

$$\sigma' = \sigma - u \qquad \text{Equation 2.1}$$

Where, σ' is effective stress, σ is total stress, and u is pore water pressure.

In high permeable soils:

$$\Delta u = 0$$
$$\Delta\sigma' = \Delta\sigma$$

In low permeable soils:

$$\Delta u \neq 0$$
$$\Delta\sigma' \neq \Delta\sigma$$

During flood conditions and when the channel is full of water, high water levels exist inside and outside the embankment. The water force outside the slope exerts a stabilizing pressure on the slope surface. The stabilizing pressure is diminishing when the water level drops down. If the water drops so rapidly, the pore water pressures within the embankment materials do not have enough time to change into equilibrium state with the drop in external water level. In this condition, since the pore water pressure in embankment materials are more than external water level, the risk of slope failure will increase significantly. This loading condition is called the rapid drawdown.

In low permeable soils, pore water pressure (u), leads to decrease in shear strength as follow:

$$S = c' + (\sigma - u)\tan\phi' \qquad \text{Equation 2.2}$$

where, S is shear strength, c' is effective cohesion, and ϕ' is effective angle of internal friction.

Finally low shear strength will reduce factor of safety of slope as follow:

$$F_s = \frac{S}{\tau} \qquad \text{Equation 2.3}$$

where, F_s is factor of safety, and τ is shear stress.

The rapid drawdown case is one of the most severe loading conditions that an earthen slope can experience. It is quite common in irrigation and stormwater drainage channels. Flooding in adjacent channel can leave water levels high in drainage channels, in which can drop relatively rapidly once floodwaters recede. While the development of deep seated failure surfaces is possible, the effect on earthen side slopes is most commonly seen in the form of relatively shallow slope failures. If these shallow slope failures left unattended, it will lead to the gradual deterioration of the channel embankment and could lead mass failure (Kerkes and Fassett, 2006).

Further, surcharge loading on an embankment consist of low permeability soils can be hazardous by effect of pore water pressure. After loading on embankment which is saturated or semi saturated, the water should go out from soil pores, to embankment materials approach to consolidation settlement. Because of lack of drainage ability in low permeable soils, this event leads to increase in pore water pressure as excess pore water pressure and as mentioned it will lead to reduce factor of safety.

Failure of embankment due to rapid drawdown can be occurred for both short and large embankments. The procedure of failing is the same, although the amount of collapse is related to embankment size. In earth dams as large embankments, rapid drawdown has made a lot of failures during embankment construction history.

For example, the San Luis Dam, completed in 1967, with a maximum height of 244 ft above the original ground surface and a volume of over 77,000,000 yd^3. It is the largest embankment dam by volume ever designed and constructed by reclamation. In September 1981, a rapid drawdown of the reservoir led to a slide in the upstream slope. The slide was about 1,300 ft long and with a total volume of about 1.4 million yd^3 (Lyman Wiltshire, 2002).

Further example, the Belle Fourche Dam (formerly Orman Dam), located about 10 miles northeast of Belle Fourche in USA , is a homogeneous earth fill embankment with 6,262 ft long and 122 ft high. In August 1931, More than 20 years after construction, rapid drawdown led to failure of the upstream slope. The reservoir was drawn down at an unprecedented rate and a shallow layer of soil slid down. The difference in elevation between the top and the bottom edges of the slide was approximately 45 ft. A movement of this type is similar to shallow slide resulting from heavy rain (Lyman Wiltshire, 2002).

A study of slides indicates that the majority of failures were caused by drawdown approximately between maximum water surface and mid-height of the embankment at average rates varying between 0.3 and 0.5 ft/day (Sherard et al., 1963). Most drawdown slides have developed when the water surface was lowered for the first time, though a few have occurred after many years of successful operation. In some of the latter collapse, the delay may have been due to decrease in the shear strength of the clay embankment or foundation with time. In every case studies by the authors, the slide was caused by drawdown which was either faster or over a greater range than had occurred previously.

b. Failure by effect of Piping

Piping, or progressive erosion, has caused a large number of catastrophic failures in contrast with any other action except that overtopping. Many of the modern techniques of embankment designing and construction have been developed to prevent it (Sherard et al., 1963). For example, many designs and techniques have been developed to provide dense and homogeneous cores which reduce the incidence of concentrated leak and resist piping when leaks do develop.

Darcy's Law predicts that under normal conditions, the volume of water that flows through a porous medium increases in direct proportion to the hydraulic head. Terzaghi (1929) asserted that the moment that the seepage pressure becomes equal to the force of gravity (effective stress), the discharge increases abruptly, because soil particles begin to be lifted apart and dispersed. Terzaghi defined the critical hydraulic gradient as that value of pressure head which equals the ratio between effective normal stress acting on the soil and the pore water pressure. When these values become equal, the effective stress becomes zero because the seepage pressure equals the submerged weight of the soil. The percolating water can then lifts particles of soil into suspension and transport them. This process is known as piping which means removing soil particles by water.

The rate of dissipation of water head per unit of length in the place where seepage occurs is i_e or Escape Hydraulic Gradient. The gradient which leads to start piping of particles is Critical Gradient i_{cr}. The critical gradient is depended on G_s (Specific Gravity of Solids) and e (Void Ratio). This parameter can be calculated as follow:

$$i_{cr} = \frac{\gamma'}{\gamma_w} = \frac{G_s - 1}{1 + e} \qquad \text{Equation 2.4}$$

where, γ' is Submerged Unit Weight, and γ_w is Unit Weight of Water.

If typical values of G_s and e for sand are used in the above equations, then i_{cr} will be approximately one (USACE, 1993).

Changing the i_e to i_{cr} leads to zero effective stress and as mentioned in Equation 2.1, Equation 2.2 and Equation 2.3, this event will lead to reduce factor of safety and subsequently instability of channel embankment.

Millions of dollars are spent annually around the world for upgrading earth embankments. Historically, around one in two hundred embankment dams have failed and one in sixty has experienced a piping incident necessitating repairs to the dam. Piping is among the most important causes of dam failure (Fell, 2005).

As a large embankment, in Idaho, USA, Teton earth dam was constructed between February 1972 and November 1975, with a maximum height of 305 ft above the streambed. Teton Dam failed catastrophically on June 5, 1976 due to the piping. The failure of this embankment dam killed 11 people, left 25,000 people homeless, inundated partially or completely an area of about 300 mi^2 that extended 80 miles downstream, and did property damage estimated at about $400 million (Lyman Wiltshire, 2002).

2.4 Analysis of channel embankment

By attention to reasons of failure of channel embankments, to attain stable and useful embankment, there are some analyses they should be done before embankment construction. Two most critical analysis which should be done for all channel embankments are seepage analysis and stability analysis.

2.4.1 Seepage analysis

The amount of water seep through and under an embankment, together with the manner of water distribution, can be estimated by using theories of flow through porous materials.

The computed amount of seepage is useful in estimating the loss of water from the channel. The estimated distribution of pressure in the pore water is used primarily in the analysis of stability against shear failure. Further, occasionally to study the hydraulic gradient at the point of seepage discharge which gives a rough idea of the piping potential.

The term of seepage usually refers to situations where the primary driving force is gravity controlled. Such as seepage losses from the irrigation channel, where the driving force is the total hydraulic head difference between the channel and external toe. Another cause of water movement in soils is the existence of excess pore water pressure due to external loading. This type of water flow is usually not referred to as seepage, but the fundamental mathematical equations describing the water movement are essentially identical. As a result, a formulation for the analysis of seepage problems can also be used to analyze the dissipation of excess pore water pressures resulting from changes in stress conditions.

Modeling the water flow through the soil with a numerical solution can be very complex. In addition, boundary conditions often change with time and cannot always be defined with certainty at the beginning of an analysis. In fact, the correct boundary condition can sometimes be part of the solution (Krahn, 2004a). Furthermore, when a soil becomes unsaturated, the coefficient of permeability or hydraulic conductivity becomes a function of the negative pore water pressure in the soil. The pore water pressure is the primary unknown and needs to be determined. Iterative numerical techniques are required to match the computed pore water pressure and the material property. This Iteration makes the solution highly non-linear (Krahn, 2004a). These complexities make it necessary to use some form of numerical analysis to analyze seepage problems.

Seepage in a channel embankment emerge on the external slope can soften fine grained fill in the vicinity of the landside toe. This action cause sloughing of the slope or even leads to piping of fine sand or silt materials. Seepage existing on the external slope would also result in high seepage forces and decrease the stability of the slope. In many cases, high water level in channel do not act against the embankment to lead this happen, but the possibility of a combination of high water level and a period of heavy precipitation may bring this. If the slope be very steep and flood stage durations and other pertinent considerations indicate a potential problem of seepage emergence on the slope, provisions should be incorporated in the embankment section. These provisions are such as horizontal and/or inclined drainage layers or toe drains to prevent seepage form emerging on the external slope. These require to select granular material and graded filter layers to ensure continued functioning, and therefore add an appreciable cost to the embankment construction, unless suitable materials are available in the borrow area with only minimal processing required. Where large quantities of pervious materials are available in the borrow areas, it may be more practicable to design a zoned embankment with large external pervious zone. This would provide an efficient means of trough seepage control and good utilization of available materials (USACE, 2000).

Nagahara et al. (2004), and Iryo and Rowe (2005), used FEM to analyze the effect of the drainage ability of geotextiles on stability of embankment. In research of Nagahara and colleagues, just the drainage ability of geotextile was taken into consideration. Iryo and Rowe first used FEM to model the drainage ability of geotextile, then, after estimating water surface in embankment, they used limit equilibrium method to consider the tensile strength of geotextile, however there was no consideration to soil-geotextile interface friction. Nagahara and colleagues reported that horizontal deformation of the case study embankment measured is much smaller than estimated by FEM and it is due to soil-geotextile interface friction that is generated in the field, is not modeled in their FEM analysis. Graph of FEM seepage analysis of Nagahara is shown in Figure 2.2.

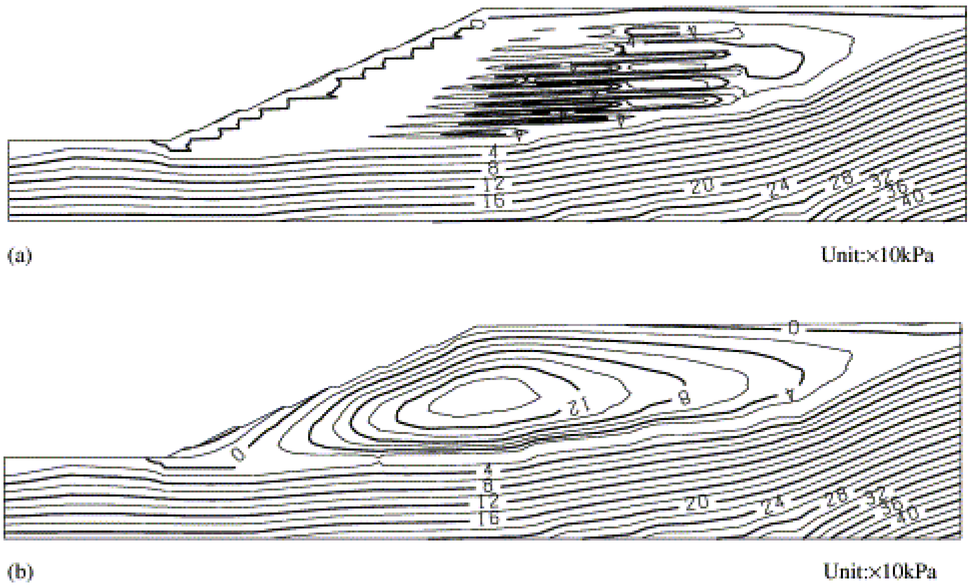

Figure 2.2: FEM Seepage analysis in embankment by Nagahara et al. (2004)

2.4.2 Stability analysis

Prior to 1935 few experienced engineers placed much reliance on theoretical embankment stability analysis. Before this time, earth embankment side slopes were selected wholly on the basis of past experience. Local rules evolved reflecting local experience, or the opinions of the principal designers in the area (Sherard et al., 1963).

In an influential series of articles covering the design and construction of earth embankments, Proctor (1933) suggested that slopes should vary from 2:1 to 4:1, depending on the foundation conditions. However, he proposed no specific means of analysis or slope selection.

At 1933, the consensus of opinion among eleven authorities representing Europe, Russia, and Japan was that designer should pattern the cross section of a successful embankment with similar dimensions (Terzaghi, 1933).

At 1956, Collin explained the first suggestions in engineering literature that earth slope could be analyzed on the basis of the results of laboratory tests of soil strength (Sherard et al., 1963). Another pioneer in recommending the use of a theoretical analysis was Bassell. In a small book Earth Dams, Bassel said that an embankment should have a base with that provides enough frictional resistance to prevent the embankment from sliding on its foundation under the water pressure (Sherard et al., 1963). He suggested a coefficient of friction between the embankment and foundation of 1.0 and a factor of safety of 1.0.

The modern sliding method of analysis was first applied to the analysis of a Quay wall failure in Sweden in 1916. Hence the common name of this method is Swedish circle analysis (Petterson, 1955). Over the next decade the applicability of the method to earth dam problems was hotly discussed, especially by European engineers, and a comprehensive discussion was published for the first time in English by Terzaghi (1929).

In the years flowing 1930 a number of investigators checked the sliding circle stability analysis by computing the factor of safety in earth slopes which had suffered shear failures. However, because little was known about soil shear strength at that time, these studies produced inconclusive results. One of the more valuable studies, published by Terzaghi (1933), was an analysis of a number of natural slope and embankment slides. Terzaghi showed that slides in some clay materials occurred at safety factors greater than unity when they were computed from the shear strengths measured in laboratory at that time.

During the next few decades, Fellenius introduced the Ordinary or Swedish method of slices at 1936. In the mid 1950 Janbu and Bishop developed some advances in Fellenius method. In the mid of 1960 Morgenstern, Price and Spencer developed iterative procedures by computer in slope stability analysis. In the early 1980, utilizing computers led to develop software products based on these techniques and routine use of mentioned methods in slope stability analysis. All of mentioned methods can be counted as slices method, so different solution techniques for the method of slices have been developed over the years. Basically, all are very similar. The differences between the methods are what equations of statics are included and satisfied, which interslice forces are included and what is the assumed relationship between the interslice shear and normal forces (Krahn, 2004b). Methods of slices are very simple and a quantitative index for stability. Further, estimating factor of safety can also be obtained; therefore they are very easily accepted by engineers (Krahn, 2004b).

Krahn (2003) explained that slices methods have some limitations. In complex conditions, it is often difficult to anticipate failure modes, particularly if reinforcement or structural members such as geotextiles, concrete retaining walls, or sheet piles are included. So the anticipation of slice method can be unrealistic in some complex conditions.

Development of the finite element method began in earnest in the middle to late 1950 for airframe and structural analysis and picked up a lot of steam at Berkeley University in the 1960 for use in civil and geotechnics engineering (Shen and Lal Kushwaha, 1998). The finite element analyses provide estimates of mobilized stresses and forces in soil structure. The finite element method is ideally suited for modeling complex problems and the estimated safety factors could be more realistic.

2.5 Safety factors and related estimations in channel embankments

Conventional analysis procedures characterize the stability of a channel embankment by calculating the safety factor. In slope design, and in fact generally in the area of geotechnical engineering, the factor which is very often in doubt is the shear strength of the soil. The loading is known more accurately because usually it merely consists of the self weight of the slope, and some times it combine with water force behind the embankment. The safety factor is therefore chosen as a ratio of the available shear strength to that required to keep the embankment stable. The critical slip surface is the one that has the minimum factor of safety and therefore, represents the most likely failure mechanism.

Comprehensive analysis to achieve stable channel embankment should be done for three conditions. In each of these conditions estimated safety factor should be equal or more than needed safety factor. Three different conditions are as follow:

- During and at the end of construction
- Steady state seepage
- Rapid drawdown

a. During and at the end of construction

This computation of stability should be performed to make confidence on stability of embankment during and at the end of construction. Consolidation analysis can be used to determine what degree of drainage may develop during the construction period. As a rough guideline, materials with values of permeability greater than 10^{-4} cm/sec usually will be fully drained throughout construction. Materials with