EVALUATION OF RIVERBANK STABILIZATION USING ROCKFILL AND SOIL-CEMENT COLUMNS

by

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A Thesis
Submitted to the Faculty of Graduate Studies
in Partial Fulfillment of the Requirements
for the Degree of

DOCTOR OF PHILOSOPHY

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THE UNIVERSITY OF MANITOBA

FACULTY OF GRADUATE STUDIES

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ABSTRACT

The City of Winnipeg is located on glacio-lacustrine lake Aggassiz clay and silt sediments. The lacustrine high plastic clay soils are relatively weak in strength resulting in instability along many stretches of the major rivers.

Rockfill columns have become a recognized method for riverbank stabilization in Winnipeg. Although this technique has proved to be effective for stabilizing unstable riverbanks, monitoring of some cases in Winnipeg have shown that excessive displacements have been experienced even after the installation of rockfill columns. This has provided a need to improve understanding about how much movement a stabilized slope must undergo before sufficient shear resistance of the columns can be mobilized.

A study was carried out to evaluate the performances of rockfill columns and an alternative technique using soil-cement columns for improving the stability of riverbanks. The study was done in three phases: laboratory tests, numerical analysis, and statistical assessment.

Large-scale direct shear tests were carried out to characterize both the clay and the rockfill material. The rockfill and the clay together were also characterize to understand how the composite material responded to shear forces experienced in the field. Test results of both the clay and the clay-rockfill composite show that shear resistance in the rockfill column will mobilize only at shear strain greater than the shear strain corresponding to the peak strength of the clay.

Conventional laboratory direct shear, unconfined compression, and triaxial tests on soil-cement columns were carried out to determine the ideal cement content and the stress-strain and strength characteristics of cemented clay. It was found that cement content of 18% by weight of dry clay provided the optimum stiffness and strength of the soil-cement mixture. The soil-cement mixture has much higher stiffness and strength compared to rockfill material.

The results of the laboratory tests were fed into numerical modeling to predict how the stabilized riverbanks perform. The numerical modeling results provided ideas how to optimize design and installation of the two columnar inclusions. Statistical analysis examined key parameters in determining the factor of safety of stabilized riverbanks.

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Wisam Abdul Razaq

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LIST OF SYMBOLS AND ABBREVIATIONS

Symbols

a - angle of shear failure plane with horizontal
a_r - area replacement ratio
A_c - corrected cross-sectional area
A_r - horizontal area of rockfill column
A_{clay} - horizontal area of clay ground
c - cohesion of the soil
c_f - soil cohesion at surface of failure
c - mathematical term = $\frac{\pi}{4}$
d - rockfill column diameter
df - degree of freedom
D - sample diameter
D_r - relative density
e - current void ratio
e_{min} - minimum void ratio
e_{max} - maximum void ratio
E - Young's modulus
$f(j)$ factor as a function of j
G - shear modulus
g_{actual} - the actual gravitational acceleration
$g_{critical}$ - critical gravitational value
$g_{(j)}$ - gravitational acceleration variable
G_s - specific gravity
h_w - water head inside the rockfill column
k - hydraulic conductivity
K_o - earth pressure at rest
n - number of samples
P_x - load cell
q - deviator stress $= (\sigma_1 - \sigma_3)$
S^2 - deviation
SE - standard error
S_a - the actual strength parameters
$S_{(t)}$ - the variable shear strength
S1 - clear space between two columns
S2 - centre-to-centre distance between two columns
s - spacing between columns
t - equivalent thickness
u_w - water head above the slide surface
z - rockfill column length
τ - shear strength
τ_f - shear strength at surface of failure
τ_{rock} - shear strength of the rockfill column
τ_{clay} - shear strength of the native clay

τ_{av} - average shear strength
τ_p - load cell pressure
σ'_N - the effective normal stress
σ_N - the total normal stress
γ - unit weight
γ_{rock} - density of rockfill column
γ_{water} - water density
$\phi'_{critical}$ - critical friction angle
ϕ'_{cs} - friction angle at the critical state
ϕ'_{rock} - friction angle of the rockfill column
ϕ_f - friction angle at surface of failure
$\phi'_{transition}$ - transition friction angle
ν - Poisson's ratio
ν - Poisson's Ratio
$\bar{Y}$ - mean value of response
Δ - change
$\Delta\tau$ - change in shear strength
Δx - shear displacement of the large scale direct shear test
Δz - vertical displacement of the large scale direct shear test
ψ - dilation angle

Abbreviations

ASCE	- American Society of Civil Engineers
ANOVA	- analysis of variance
FEM	- Finite Element Method
CU	- consolidated undrained triaxial test
DMM	- Deep Mixing Methods
FEM	- Finite Element Method
FHWA	- Federal Highway Administration
GIM	- Gravity increase method
LEM	- Limit Equilibrium Method
FS	- Factor of safety
FLAC	- Fast Lagrangian Analysis of Continua
LVDT	- Linear variable displacement transducer
N	- number of runs
SRF	- shear reduction factor
SSR	- shear strength reduction
SS	- sum of squares
SS_{Model}	- sum of squares for significant factors
$SS_{Residual}$	- sum of squares for smaller effect factors
DOE	- Design of Experiment
RSM	- Response Surface Methodology

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Chapter 1

INTRODUCTION

1.1 Background

The City of Winnipeg is located on glacio-lacustrine Lake Agassiz clay and silt sediments. Winnipeg's riverbanks consist predominantly of lacustrine clay rich soils and alluvial (river) deposits of layered clays, silts and sands. There are over 240 km of waterfront property along the four primary rivers of Winnipeg, 45% of which are currently owned by the City of Winnipeg. The lacustrine high plastic clay soils are relatively weak resulting in instability along many stretches of the major rivers within the City of Winnipeg. These riverbank failures are generally deep-seated, extending 12 to 15 metres below the ground surface. Clay banks will typically move to achieve overall gradients in the order 6H:1V to 9H:1V, when at that stage, a state of quasi-equilibrium is established (Baracos and Graham 1981). A number of factors can reactivate these previously unstable banks, thereby triggering further movements. The principal factors controlling riverbank failure include groundwater conditions in the clay and underlying till, river hydraulics, and progressive soil weakening (Duncan and Wright 2005).

Rockfill columns (otherwise known as stone columns or rock caissons) have become a recognized method for riverbank stabilization in the City of Winnipeg (City of Winnipeg 2000, Yarechewski and Tallin 2003). Rock columns are large diameter holes drilled through the clay into the underlying till that are filled with crushed rocks or stones. The rockfill columns are generally located in the lower bank area of unstable riverbanks. There have been reports (e.g., Goughnour et al. 1991) where rockfill columns have been successfully used in stabilizing natural slopes. In Winnipeg, rockfill columns are constructed using a combination of *cased-borehole* and *vibro-compaction* methods. Densification of the crushed rocks, generally crushed limestone, is provided by an electrically- or hydraulically-actuated vibrator in the vibroflot. The vibroflot has the ability to penetrate the crushed rocks after they have been placed in the predrilled holes. The predrilled holes are supported with steel casing if the potential for sloughing exists. The columns are generally installed in a closely spaced and staggered grid pattern.

Rockfill columns improve the shearing resistance of the clay layers by removing columns of the weaker clay and replacing them with stronger rock. However, recent cases in Winnipeg have shown that movements occur following installation of rock columns. Uncertainty regarding the magnitude of the movements required to mobilize shearing resistance in the rock columns has resulted in situations where stability of riverbanks following remediation has been questioned. This has provided a need to improve our understanding of how much movement a stabilized slope must undergo before sufficient shear resistance of the rockfill column will be mobilized. The mobilization of shear resistance for granular materials such as crushed rocks is highly dependent on the shear displacement, applied normal stress (at the location of the failure plane) and relative density. A proper evaluation of the shear resistance mobilization in the rockfill columns and the clay-column composite will improve the assessment of the performance of rockfill columns. In addition to the physical requirement for movement to mobilize shearing resistance in rockfill columns, the method of installation can also impact the stress-strain behaviour of the system and therefore the movements required for shear mobilization. These questions are the basis for the objectives outlined in this research.

The research will examine alternative material mixtures to improve performance of rockfill columns for stabilizing natural riverbanks in Winnipeg. Juran and Riccobono (1991) reported positive effects of low level cementation on the mobilization of shear resistance in compacted sand. Providing artificial cementation through grouting or pre-mixing of cement with rockfill materials allows mobilization of shear resistance in rockfill columns at smaller strains.

The use of deep soil-cement mixing as an alternative columnar inclusion is investigated. Soil-cement columns are constructed by advancing augers into the ground while cement grout is injected into the clay soil through the augers.

Cement/lime column stabilized slopes have been previously studied and shown good promise for reducing movements of both natural slopes and embankment slopes (Rogers et al. 2000, Watn et al. 1999, Bergado et al. 1994).

1.2 Research objectives

Research objectives of this study are as follows:

1. To assess the displacements required to fully mobilize shear resistance in columnar inclusions.
2. To compare the performance of two types of columnar inclusions for riverbank stabilization, namely: rockfill columns and soil-cement columns.
3. To develop a measure of performance of these columnar inclusions following installation.
4. To investigate the influence of columns in a group.

1.3 Research methodology

The research methodology of this study is divided into three components as shown in Fig. 1.1. The first step was to conduct comprehensive laboratory tests using a large-scale direct shear test on fairly large sized, undisturbed clay-rockfill composite samples to evaluate the mobilized shear resistance in rockfill columns. The second step was to use the results in the numerical analysis and design to determine the factor of safety, and the associated displacements in the riverbank after rockfill column installation. Moreover, numerical analysis has been used to evaluate the optimum number of rockfill columns, optimum diameter, and the best column layout through the riverbank.

The third step of this study includes statistical analysis and modeling to asses the significance of factors influencing the factor of safety. By using this statistical method, not only each factor was analyzed separately, but the interactions between factors were analyzed to assess their influences on the results.

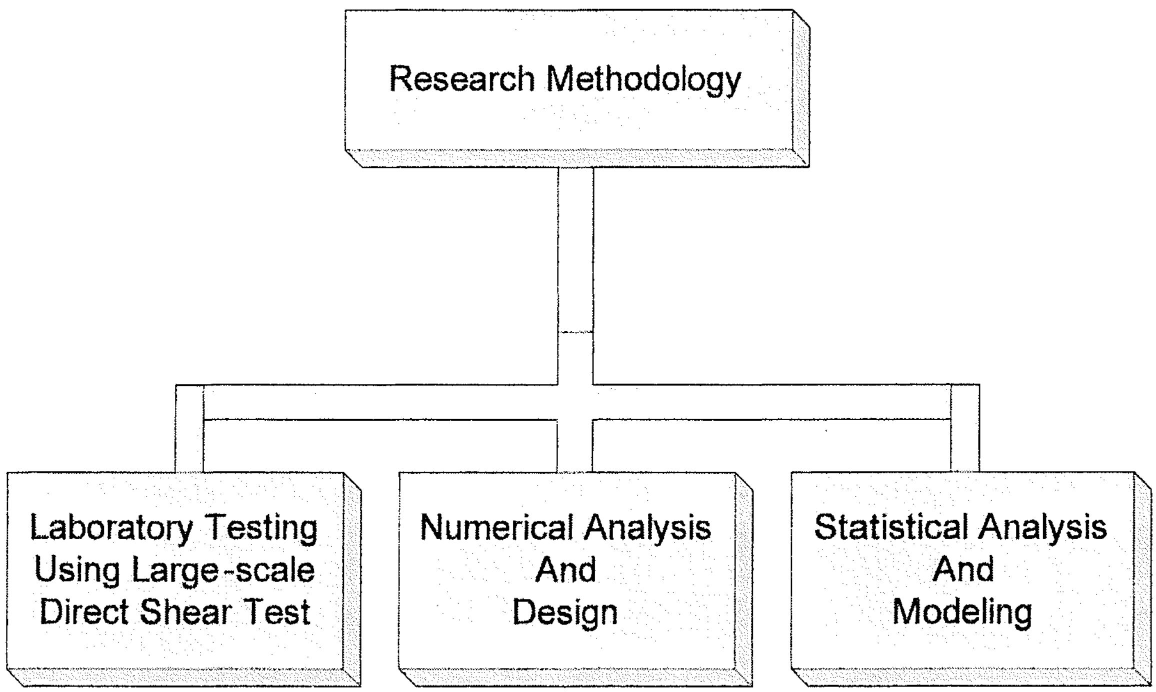

Figure 1.1 Components of research methodology

Finally, the expected outcomes of this study are as follows:

1. Development of new construction methods to reduce ground movements following installation of columnar inclusions.

2. Development of an understanding of what movements should be expected following placement of columnar inclusions.
3. Improvement of guidelines for design and construction of remedial work to increase stability and prevent riverbank failure.
4. Determination of an optimal diameter and spacing of columns that provide cost-effective design.
5. Examination of whether an alternative columnar inclusion using soil-cement columns is more effective for riverbank stabilization in Winnipeg, compared to commonly used rockfill columns.

1.4 Scope of thesis

In this work, a distinction is made between two methods of columnar inclusions, rockfill columns and soil-cement columns in evaluating the stability of a typical riverbank in Winnipeg stabilized with these columnar inclusions. This comparison offers the opportunity to decide the proper method that can be applied to stabilize instable riverbanks from a technical perspective. This thesis is divided into nine chapters, in which this chapter (Chapter 1) presents the introduction and objectives of the research work. Chapter 2 provides an overview of the literature related to the problem and the factors or variables that might have an influence on the problem of study. In addition, Chapter 2 demonstrates a general review of the soil improvement methods that might be used to stabilize unstable riverbanks.

Chapter 3 presents the procedure to investigate the engineering properties of rockfill columns using large-scale direct shear apparatus and test setups. The results and analysis of the laboratory large-scale direct shear test are presented in Chapter 4. Chapter 4 discusses the results of these tests on both rockfill columns, the native undisturbed clays, and composite clay-rockfill samples.

Chapter 5 discusses the performance and results of tests on soil-cement specimens. The parameters needed for assessing the stability and displacements of a typical riverbank in Winnipeg were determined from these test results.

Chapter 6 presents the numerical analysis and results of a typical riverbank stabilized with rockfill columns and soil-cement columns. It also discusses several scenarios that can occur, and their consequences on general performance. Chapter 7 assesses the performance of rockfill columns in stabilizing riverbanks in Winnipeg. It also investigates the influence of many variables that might contribute to the estimated factor of safety.

Chapter 8 recommends guidelines for the design and construction of rockfill columns for riverbank stabilization in Winnipeg. Finally, Chapter 9 includes a brief summary, conclusions, and discussion of the issues investigated in this study. In addition, this Chapter highlights the recommendations for further research related to the topic.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

This chapter reviews past studies related to this research. These related studies cover techniques used for slope stabilization, laboratory testing, numerical analysis and design, and statistical analysis and modeling.

The techniques used for slope stabilization are described with a brief review of studies made for each of them. More attention is given to the methods that increase the shear strength and release the excess pore water pressures, which are thought to be more applicable to Winnipeg riverbanks.

Studies related to laboratory testing include the properties of soil along riverbanks, shear strength, slope geometry, groundwater flow, and aquifer pressure in the till layer. The review shows that no large-scale laboratory shear tests on composite clay-rockfill samples were documented. This research attempts to fill this gap by characterizing the mobilization of shear resistance by using large direct shear test apparatus. This testing program will be discussed in detail in Chapter 3.

Different numerical analysis and design methods are reviewed. A comparison between the traditional Limit Equilibrium Methods (LEM) and the Finite Element Methods (FEM) that take advantage of the high computational power of today's computers will be discussed. The comparison shows that FEM will give more accurate results and do not require in-advance assumptions.

Two statistical methods are available: the Probability Method and the Design of Experiment Method. No historical study related to riverbank stability studies implements the Design of Experiment Method, which is considered to be more applicable to the subject of the research.

2.2 Lacustrine clay and riverbank failures

Four major rivers within the City of Winnipeg are located on soft lacustrine high plastic clays and alluvial deposits. The soil engineering properties used in historical studies related to Winnipeg are reviewed. These properties are compared to the soil engineering properties measured in laboratory tests carried out in this current study.

Winnipeg area soils can be divided into four layers: 1) the upper complex zone of mixed soils, 2) plastic glacial lacustrine clays, 3) dense basal and some water laid tills, and 4) Paleozoic dolomitic bedrock (Baracos et al. 1983). The bedrock carbonates have medium strength when intact, but have been greatly affected by preglacial erosion and weathering and glacial disturbance. Some of the lower tills are hard and dense, while others are noticeably softer. Because of successive ice advances, the tills may contain water bearing glacio-fluvial sands and gravels.

Lacustrine clays are generally weak in terms of strength; generally high to medium plastic which represents the main body of riverbanks in Winnipeg. Observations show that failure of riverbanks inside Winnipeg involves large sections of the slope. These bank slides are deep seated, extending 10-15 meters below ground surface, and extend horizontally in some locations as far as 60 m from the river's edge (Peterson et al. 1960). Peterson et al. (1960) stated that the soil properties of the high plastic clay foundation in Winnipeg are in the range of 36 to 81 for plasticity index (PI), 19 to 66 for the natural water content, and 14.74 kN/m^3 to 18.74 kN/m^3 (92 to 117 lb/ft^3) for the wet density. Quigley (1968) carried out several X-ray diffraction tests on both types of surface soil; brown colour or "chocolate clay", and dark grey-brown,"blue clay". He mentioned that the soil composition was 80% of interlayered montmorillonite-Illite, (5-10) % kaolinite, and (10-15) % quartz. These high contents of montmorillonite were thought to be the cause for the high plasticity index in the clay and the potential for swelling.

Mitchell and Soga (2005) pointed out that physicochemical behavior of clay plays a role in the soil behavior. The content of large clay minerals, particularly active clay minerals, leads to greater potential for swelling, creep, strain softening, and changes in behavior due to physicochemical affects. The activity of clay minerals is highly influenced (qualitatively) by the value of the plasticity index (PI), which, therefore is an essential indication of clay strength. They indicated that high plastic clays exhibit a very low value of hydraulic conductivity (permeability). Soils with high hydraulic conductivity express fewer problems because pore water pressure changes can occur rapidly and approache equilibrium conditions quickly.

Quigley (1980) reported that Winnipeg is built on swelling, plastic clays deposited in proglacial Lake Agassiz between 11,500 and 8,000 years ago. The lake covered 200,000 km^2. The average thickness of the clay at Winnipeg is 10-12 meter. The plastic clay is underlain by stiff basal tills. Render (1970) stated that the rock bed is karstified Ovdovician limestone that is imposes artesian water pressures to the bottom of the clay.

2.3 Shear strength of Winnipeg clay

The shear strength parameters of high plastic Winnipeg clay are reviewed. These parameters will be compared to the shear strength parameters measured in my studies. This will help in deciding the proper strength parameters (c' and ϕ') for the analysis and design in this specific soil type. The soil properties in the weak zone are also reviewed. Freeman and Sutherland (1974) reported that a layer of clay had been identified in Winnipeg along the clay-till interface with relatively low shear strength. They stated that the slip surface might tend to follow this weak clay layer parallel to the bedding. As will be shown later based on laboratory test program, it was reasonable to use the post peak strengths of this layer in designing stabilization measured for the Winnipeg riverbanks. This conclusion agrees with historical studies and recommendations by other researchers who carried out studies on high plastic clay soil. Details about the numerical analysis results of this research related to this subject will be given in Chapter 6.

Rivard and Lu (1978) and Lefebvre (1981) have shown that peak strengths of fissured plastic clays used in slope stability analysis did not produce satisfactory results. These slopes were analysed using peak parameters and showed a safe design; i.e. F.S. > 1, while in reality these slopes showed signs of instability. Duncan and Wright (2005) reported that using peak strengths for brittle soils can lead to inaccurate and unconservative assessments of stability. Graham (1986) stated that post peak strengths have been used with success in first-time slides in Winnipeg. Duncan and Wright (2005) stated that slopes in overconsolidated clays, particularly fissured clays, show that fully softened strengths are appropriate for these materials in cases where slickensides have not developed, and residual strengths are appropriate in conditions where these slickensides have developed. Generally the shear strength parameters used are c'=5 kPa and $\phi'=17^{\circ}$ for Winnipeg riverbanks, which are representative of the post peak parameters (Tutkaluk 2000, KGS Group 1994). However, in the weak zone located at or near the interface of the high plastic clay and the till layers, the values of the shear strength parameters were slightly lower with values of about c'=3 kPa and $\phi'=12^{\circ}$. These values fall within the upper bounds of residual strength for Winnipeg clays (Tutkaluk 2000).

2.4 Geometry of stable slope in Winnipeg riverbanks

Mishtak (1964) investigated the process of selecting a stable slope for the diversion channel (Floodway) to protect the Greater Winnipeg Area against flooding. He indicated that the slope inclination of 6H:1V for most of the floodway should be stable. This investigation was deemed to be an acceptable base for the design of the slope in the floodway. City of Winnipeg (2000) surveyed riverbanks along the Red river within its city limit, and showed that slopes are generally on the order of 6V:1H. Tutkaluk (2000) reported that the riverbank slopes according to the surveyed riverbanks in Winnipeg were 6H:1V, 10H:1V, and 4H:1V for the upper slope, mid slope, and the riverbank toe respectively.

Figure 2.1 shows the idealized riverbank cross-section used in this study, it represents the morphology, slope geometry, and the characteristics of most Winnipeg riverbanks. The cross section was chosen to be consistent with the one used in the analysis of Tutkaluk (2000). Slope height is the vertical distance from the horizontal ground surface at the toe to the horizontal ground surface at the crest, as illustrated in Figure 2.1. Mishtak (1964) reported that 80% of the outside edge of bends in the highly plastic clay are of height between 7.6-12.1 meters. Baracos and Graham (1981) studied landslides in Winnipeg, where the riverbank geometry used in their study considered a typical height of 9-11 meters. This range in slope height is consistent with the one reported by Tutkaluk (2000) wherein the typical height for a natural riverbank in Winnipeg is 12 m. He indicated that this value was selected on the basis of existing riverbanks in Winnipeg where top of bank and toe elevations range from 230 to 232 m and 218 to 220 m, respectively (geodetic elevation). Tutkaluk (2000) performed slope stability analysis of the Red riverbank. He considered that the geometric sections of riverbank consist of three sections: the upper bank, mid bank, and toe. The geometry of the upper bank section is known to be near the critical slope geometry in Winnipeg (6H:1V). The mid bank section is less steep than the upper bank and is influenced by the current regulation of river level. The toe section of the riverbank is usually the steepest slope. Tutkaluk indicated that the characteristics of the slopes used in his analysis were chosen to be consistent with riverbanks on the outside bends of rivers within the City of Winnipeg. These areas are typically the most active with respect to downslope

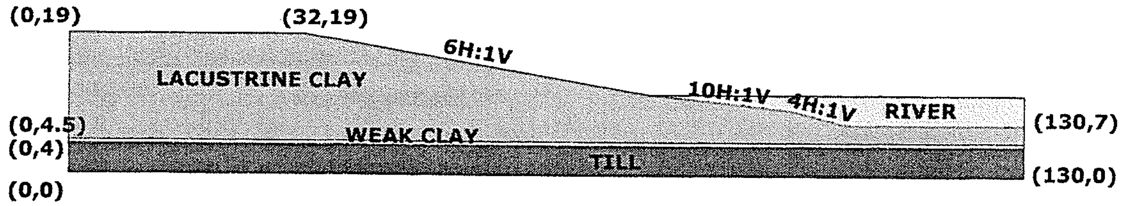

Figure 2.1 Typical Winnipeg riverbank cross section (after Tukaluk 2000, KGS Group 1994)

movement and failure. These values of slope geometry will be used in modeling the slope stability for my study.

The upper slope is the section located between the horizontal ground at the crest of the riverbank to an elevation where the regulated summer river level intersects the slope. This zone is significant and has a major contributing effect to the stability analysis of riverbanks. The mid bank slope represents the part of the bank that extends between the regulated summer river level and unregulated winter river level. Toe slope represents the section of the riverbank from the winter level of the unregulated river to the river bottom. This part is always submerged in the water and exposed to fluvial erosion throughout the year (Tutkaluk 2000).

River level is important in the stability analysis because it affects the computed factor of safety. The regulated river level was 223.9 m (geodetic elevation) as reported by KGS Group (1994).

2.5 Slope stabilization by various ground improvement techniques

Soil improvement technologies are necessary to improve poor and marginal soil in order to meet the engineering design requirements. Many ground improvement techniques and methods have been developed during the last four decades. These techniques assist in designing and implementing cost effective projects. Stabilization methods basically increase resisting forces, reduce driving forces, or both.

Soil improvement methods include but are not limited to the following; unloading, lightweight fill, soil and rock fill, counterberms, shear keys, mechanically stabilized embankment (MSE), pneusol (tire soil), drainage methods (e.g., surface drainage, catchment parameters, redirection of surface runoff, subsurface drainage, blankets, trenches, cut-off drains, horizontal drains, relief wells, and drainage tunnels or galleries), retaining walls method (e.g., conventional gravity or cantilever retaining walls, driven piles, drilled shaft walls, and tieback walls), and soil hardening methods (e.g., compacted soil-cement fill, electro-osmosis, thermal treatment, grouting, lime injection, preconsolidation) (ASCE Geo-support 2004, Abramson et al. 2002).

Abramson et al. (2002) indicated that there are many factors that affect selection of the appropriate method for slope stabilization. These factors include cost and time required for installation. The most effective method is one which provides improvement with the shortest time and least cost. Other factors that effect selection are:

1. The slope type; whether it is natural or man-made.
2. Soil type; cohesive soils or granular soils.
3. Topographical condition and ground water level.
4. Presence of aquifer pressure.
5. The shear strength of the ground.
6. Whether movements of the slope are allowed during or after construction.
7. The shear displacement required to mobilize the strength of the reinforcing elements.
8. The durability of the technique compared to the project life time.
9. Availability of the materials or equipments.
10. Others constraints; In addition to the technical constraints, there are site constraints, environmental constraints, aesthetic constraints, schedule constraints, and others such as politics and tradition (Abramson et al. 2002).

Taking these factors into account, rockfill columns and soil-cement columns have been evaluated for stabilizing riverbanks in Winnipeg.

2.6 Slope stabilization using stone or rockfill columns and methods of installation

It has been shown that rockfill columns can be used for slope stabilization (Aboshi et al. 1979, Diyaljee and Pariti 1990, Goughnour et al. 1990, Johnson 1994, Shin et al. 2000, Abramson et al. 2002, Yarechewski and Tallin 2003, Tweedie et al. 2004). The slope stabilization technique increases the factor of safety in two ways: (1) It increases the average shear resistance of a slope by displacing or replacing the weaken native soil with a series of closely spaced, large diameter columns, (2) the columns act as a drain; this helps in dissipating the pore water pressure, thereby increasing the strength along the failure surface and preventing landslides. While rockfill columns have been used in stabilizing riverbanks, majority of the applications were on embankments and foundations. Therefore, studies about slope stabilization at riverbanks using rockfill columns are limited. Most of the available literature focuses on their application to reduce liquefaction potential to support embankments, to minimize settlements, and or to increase bearing capacity. The current research tries to fill these significant gaps. Chapter 4 gives details about the tests and findings related to the investigation of shear resistance of rockfill columns for riverbank stabilization.

When used in embankment and foundation applications, several investigators (Bachus and Barksdale 1989, Bergado et al. 1994, and Abramson et al. 2002) indicated that rockfill columns are more effective for subsurface soils with shear strength in the range of 10-50 kPa. Soil in the lower range of these limits may not provide enough lateral support and thus too much rockfill materials will be required for the column to be effective. In contrast, subsurface soils in the higher range may not benefit from the placement of rockfill columns.

The construction of rockfill columns consists of (Abramson et al. 2002, Bergado et al. 1994):

(1) Forming a vertical hole in the underlying material, using either the vibro-replacement or the vibro-displacement techniques.

(2) Placing rockfill materials in the formed hole from the ground surface, as in the vibro-replacement technique, or by means of bottom feed equipment, as in the vibro-displacement technique.

(3) Compaction of the rockfill materials by penetration of each lift with the vibroflot, a process that drives the rockfill material laterally to the sidewalls of the hole, thus enlarging it.

To make piles or columns effective they must have the following characteristics (Aboshi 1979 and Poulos 1995): (1) They must be of relatively large diameter and relatively stiff; so that reasonably large stabilizing forces can be generated; (2) They must be extended enough below the critical failure surface so that the failure surface is not merely shifted downwards below the column tips without increasing the factor of safety. (3) Columns should be located in the vicinity of the centre of the critical shear failure to avoid merely relocating the shear failure surface behind, or in front of a group of columns. Bergado et al. (1994) defined the area of the rockfill column (horizontal projection) to the total reinforced area by the area of the equivalent cylindrical unit within the unit cell replacement ratio, a_r. and expressed as follows:

$$[2.1] \quad a_r = \frac{A_{rock}}{A_{rock} + A_{clay}}$$

where, A_{rock} = the horizontal area of the rockfill column,

A_{clay} = the horizontal area of the horizontal clay ground around the column. In terms of the column diameter (d) and column's spacing (S), area replacement ratio can be rewritten as follows:

$$[2.2] \quad a_r = c\,(\frac{d}{S})^2$$

where, c is a mathematical term, which depends on rockfill columns pattern used in the field:

$$[2.3] \quad c = \frac{\pi}{4} \quad \text{[for square pattern]}$$

$$[2.4] \quad c = \frac{\pi}{2\sqrt{3}} \quad \text{[for triangular pattern]}$$

Goughnour et al. (1990) reported successful stabilization of natural slopes using rockfill columns. Most recently, Yarechewski and Tallin (2003) evaluated the field performance of rockfill columns along riverbanks in Winnipeg. They stated that the rates of riverbank movements have decreased significantly compared to the pre-reinforcement rates. They performed a comparison between three different methods of slope stabilization. These methods were shear key, ribs, and rockfill columns. They pointed out that a shear key was more effective than rockfill columns at the Seine River Siphon because the constructed rockfill columns took almost twice as long to excavate and place the rockfill column materials. This led to longer period of reduced stability which directly increased the post-construction movements. It should be noted that the installation of rockfill columns did not include the use of steel casing.

Aboshi (1979), Goughnour at al. (1990), and Cornforth (2005) describe the method of calculating the degree of improvement supplied by the rockfill columns to stabilize the clay landslide. They suggest a simple way of calculating the extra strength provided by the stabilizer. The total shear resistance and the average shear strength of the composite soil at the potential shear failure can be calculated by back calculation. The average shear resistance of the composite soil is the shear resistance of the rockfill columns and the shear resistance provided by the clay soil at the slide surface:

[2.5] $\tau_{av} = \tau_{rock} (A_{rock} / A_{total}) + \tau_{clay} ((A_{total} - A_{rock}) / A_{total})$

[2.6] $\tau_{av} = \tau_{rock}\, a_r + \tau_{clay} (1 - a_r)$

The extra shear resistance provided by the rockfill columns can be calculated by summing the extra strength provided from the difference between the shear strength of the rockfill and the clay at the slip surface as shown in the Figure 2.2 (Cornforth 2005).

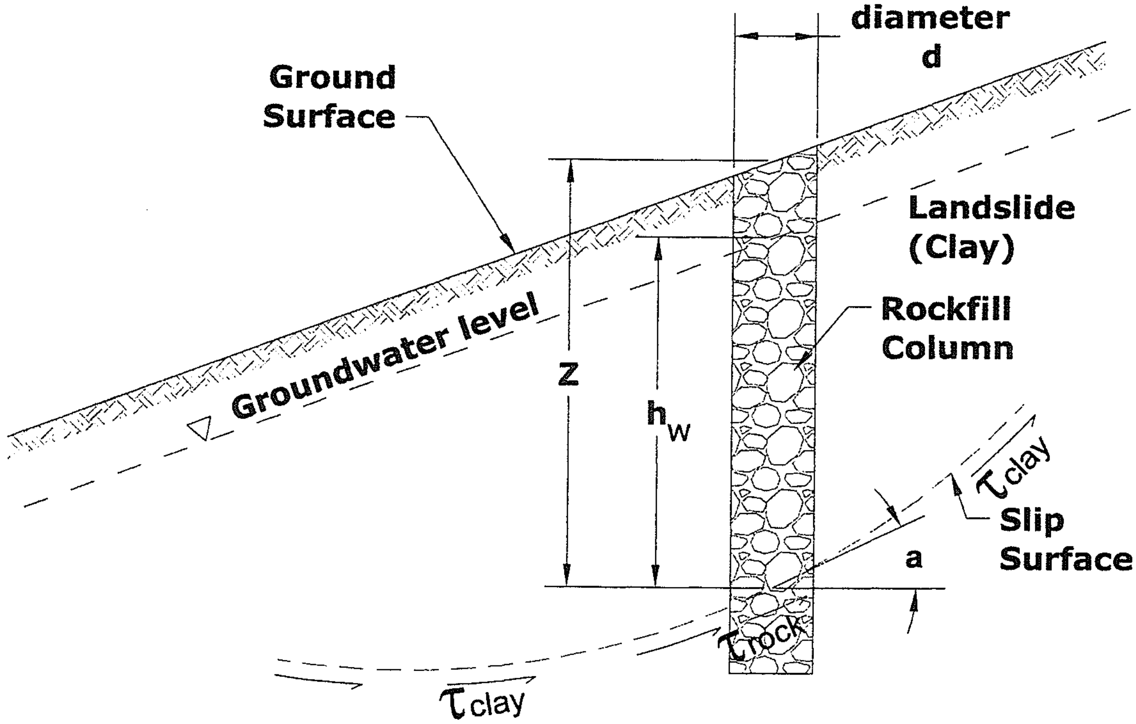

Figure 2.2 Schematic diagram showing the potential shear failure through rockfill column stabilization (after Cornforth 2005)

[2.7] $\tau = c' + \sigma_N' \tan\phi'$

[2.8] $\sigma_N' = \sigma_N - u_w$

$= (\gamma_{rock} * z) - (\gamma_w * h_w)$

[2.9] $\tau_{rock} = \sigma_N' * \cos\alpha * \tan\phi_{rock}'$

[2.10] $\tau_{extra} = \Sigma\{(\tau_{rock} - \tau_{clay})\frac{\pi}{4}d^2\}$

where, τ = Shear strength,

τ_{rock} = shear strength of the rockfill column,

τ_{clay} = shear strength of the native clay,

$\sigma_{N'}$ = the effective normal stress applied at the slide surface,

σ_N = the total normal stress,

u_w = water head above the slide surface,

γ_{rock} = density of rockfill column,

γ_w = water density,

ϕ'_{rock} = friction angle of the rockfill column.

Tweedie et al. (2004) used rockfill columns to mitigate movements of embankments on high plastic clay foundations in Alberta. They observed that the rate of movement following installation of rockfill columns was continuing to decrease which agreed with observations by Yarechewski and Tallin (2003). Despite continued slope movements during two years after installation, the rate of movements was steadily decreasing. Both of these cases reported relatively successful stabilization of slopes. However, both cases have shown that movements generally occurred following installation of rockfill columns as would be expected to mobilize the shear resistance in the rockfill materials. Uncertainty regarding the magnitude of movements required to mobilize shearing resistance in rockfill columns has resulted in situations where stability of riverbanks and slopes following remediation has been questioned. This has provided a need to improve our understanding regarding the movements that stabilized slopes must undergo before sufficient shear resistance of the rockfill column is mobilized. The lack of existing information regarding shear mobilization may in part be linked to the method of analysis traditionally used to establish the improvement in stability provided by rockfill columns.

2.6.1 Semi Rigid Columns

Field observations show that rockfill columns experienced continuous settlement or displacement after installation. As will be discussed later in Chapter 4, slope movements required to mobilize shear resistance in this technique may not be accepted from the technical, ecological and financial point of view (e.g., continuous displacement in the highway leads to continuous maintenance of embankments which is technically and financially unacceptable).

Therefore, attempts have been taken to improve the performance of rockfill columns by adding a small amount of bonding material such as lime or cement to form a more bonded matrix resulting in higher stiffness and more shear resistance. This method can be effective in reducing the movements required for shear mobilization.

Diyaljee & Pariti (1990) reported that stone column stabilization of a section of highway was not effective in controlling the continuous embankment settlement. They pointed out that lime-flyash injection and stone-cement columns have good results in stabilizing paved roadway in ten sites in Alberta highways. It was useful in resolving slope stability and bearing capacity problems. However, they did not present any quantitative evaluation of the material behaviour. They just reported qualitative description of its performance.

Heitz et al. (2005) reported application of grouted-stone columns in conjunction with geogrids to form a pile-geogrid reinforced embankment in Germany. This technique has proven to perform well in terms of both bearing capacity and serviceability aspects.

2.7 Slope stabilization by soil-cement columns

Deep Mixing Method (DMM), a widely used soil-cement column application method, is used to improve the engineering properties of soft alluvial clay ground, highly organic soils and loose sand deposits, and other soil types. It is expected that this method can be applicable to Winnipeg riverbanks because it does not require soil replacement. Soil replacement has two disadvantages; the ground movement following soil removal and the need for a better quality soil to replace the native soil.

DMM is an in-situ soil treatment whereby the native weak soil is blended with cementitious materials. Cementitious materials generally used are cement and/or lime and can be mixed in dry or slurry form (FHWA-RD-99-138 2000; FHWA-SA-98-086 R 2001). Binder materials are injected through hollow, rotated mixing shafts tipped with different types of cutting tools. These cutting tools, in some methods, inject fluid grout through nozzles. One to eight shafts (typically two to four) are usually used per carrier. The number of shafts used depends on the nature of the project.

Early applications of DMM involved soil treatment for transportation and harbor facilities in soft soils. Most of the past DMM applications concentrated on settlement control of soft soils supporting embankments, liquefaction mitigation, steel reinforced retaining walls, groundwater cutoff walls, and stabilization of contaminated soils. DMM has also been used to stabilize and support vertical excavations. DMM provides satisfactory performance under both static and dynamic loads. DMM has been expanded to slope stability applications; it improves the overall shear strength of the soil formation to adequately increase the factor of safety. DMM columns can also force the potential failure surface deeper as stated by Andromalos et al. (2000).

Abramson et al. (2002) has reported that slope stability can be improved by injection of lime columns to form columnar inclusions. These columns increase the shear strength of the composite ground. He mentioned that this technique is not effective in sandy soils, but mostly suited to clayey and silty soils. The lime injection column can also function as a vertical drain in the soil, as the mixed soil dissipates the excess porewater pressure. Broms (1999) stated that the shear strength of the stabilized soil, as determined by unconfined compression tests on clay-lime, increases with time and can be as high as 500-1000 kPa (5 to 10 tons/ft^2). Abramson stated that shear strength of the stabilized clay within the columns depends on: (1) the overburden pressure and, (2) the relative stiffness of the columns with respect to the surrounding unstabilized soil.

As mentioned earlier, soil-cement columns can force the potential shear failure surface below the tip of the columns', and thus will increase both the factor of safety as well as the resisting force due to soil weight. Soil-cement columns should be designed to provide appropriate shear strength, to overcome driving forces (soil and water lateral pressure), and to achieve the required safety factor. Chapter 5 gives details about the appropriate soil-cement ratio, tests conducted, and shear strength parameters that can be used for riverbank stabilization.

2.8 The effect of aquifer pressure on riverbank stability

The aquifer pressure in Winnipeg riverbanks varies seasonally. Studies pointed out that the aquifer pressure minimizes the Winnipeg riverbank's stability (Tutkaluk et al. 2002). Thus it should be considered in the analysis and design of riverbanks.

Soils and rocks that transmit water with ease through their pores and fractures are called aquifers. Its relative permeability is higher compared to the strata above (Abramson et al. 2002). Aquifers are either confined if they are between two impermeable strata, or unconfined if they are not overlain by a less permeable layer. Sometimes the only stabilization procedure required to remediate a slope is through the detection and drainage of an aquifer (Forres 2004).

Several investigators (Baracos 1978, KGS Group 1994, Tutkaluk 2000) stated that the movement of riverbanks increase during mid-Fall because of the drawdown of the water level in the river. This drawdown is also associated with the increase in piezometric elevation of an aquifer at the bedrock and lowers the factor of safety of the riverbank. This sudden increase in piezometric elevation is a function of the reduced water demand (industrial and commercial). This leads to increased pore water pressure and reduces both the effective stresses and the ultimate shear strength at the clay-till interface.

Baracos (1978) mentioned that the rates of slope movement increased during spring, the time of high snowmelt run-off and the lowest level of the river. He pointed out that higher rates of slope movement occurred in early spring, when rain fall combined with the low level of river during winter. This high rate of slope movement happened because the ground water level increases in spring, therefore the effective stress decreases. However, the high rate of slope movement may decrease as soon as the river level increases.

2.9 Factors contributing to slope failures

It is important to understand the causes of instability in riverbanks or natural slopes for two reasons. First, to avoid factors that lead to slope failure, and second, to attempt to treat critical riverbanks. Duncan and Wright (2005) stated that there is no single cause of failure, and this highlights the importance of considering and evaluating all potential causes of failure in order to develop an effective means of repairing and stabilizing slopes that have failed. Similarly, through the process of deciding the most appropriate method to stabilize slopes, it is important to anticipate all of the changes in properties and conditions that may affect a slope to ensure it will remain stable despite these changes.

Sowers (1979) illustrated the difficulties in attempting to isolate the cause of failure to one cause because in most slope failures there are generally several causes. Many agents may contribute to increase shear stresses at slopes or reducing the shear strengths of soils. Figure 2.3 illustrates these causes.

Duncan and Wright (2005) and Mitchell and Soga (2005) reported that physicochemical behavior of clays is important and may contribute to the process that reduces strengths of soils, especially with high plastic clays. They mentioned that the higher the plasticity index, the greater the potential for slope instability. Duncan and Wright (2005) pointed out that high plastic clay and heavily overconsolidated clays are subject to swell when those soils come in contact with water. Highly plastic clay is vulnerable to the development of slickensides, as a result of shear on a distinct plane of slip. As shear displacement takes place along distinct plane, plate-like clay particles tend to be realigned parallel to the plane of slip. Slickensided planes are weaker than the surrounding clay where clay particles are randomly oriented. Duncan and Wright (2005) reported that slopes of high plastic clays are highly susceptible to failure under sustained loads, which lead to persistent slope deformation (creep). Creep of soils is highly affected by cyclic variations such as freeze-thaw and wet-dry.

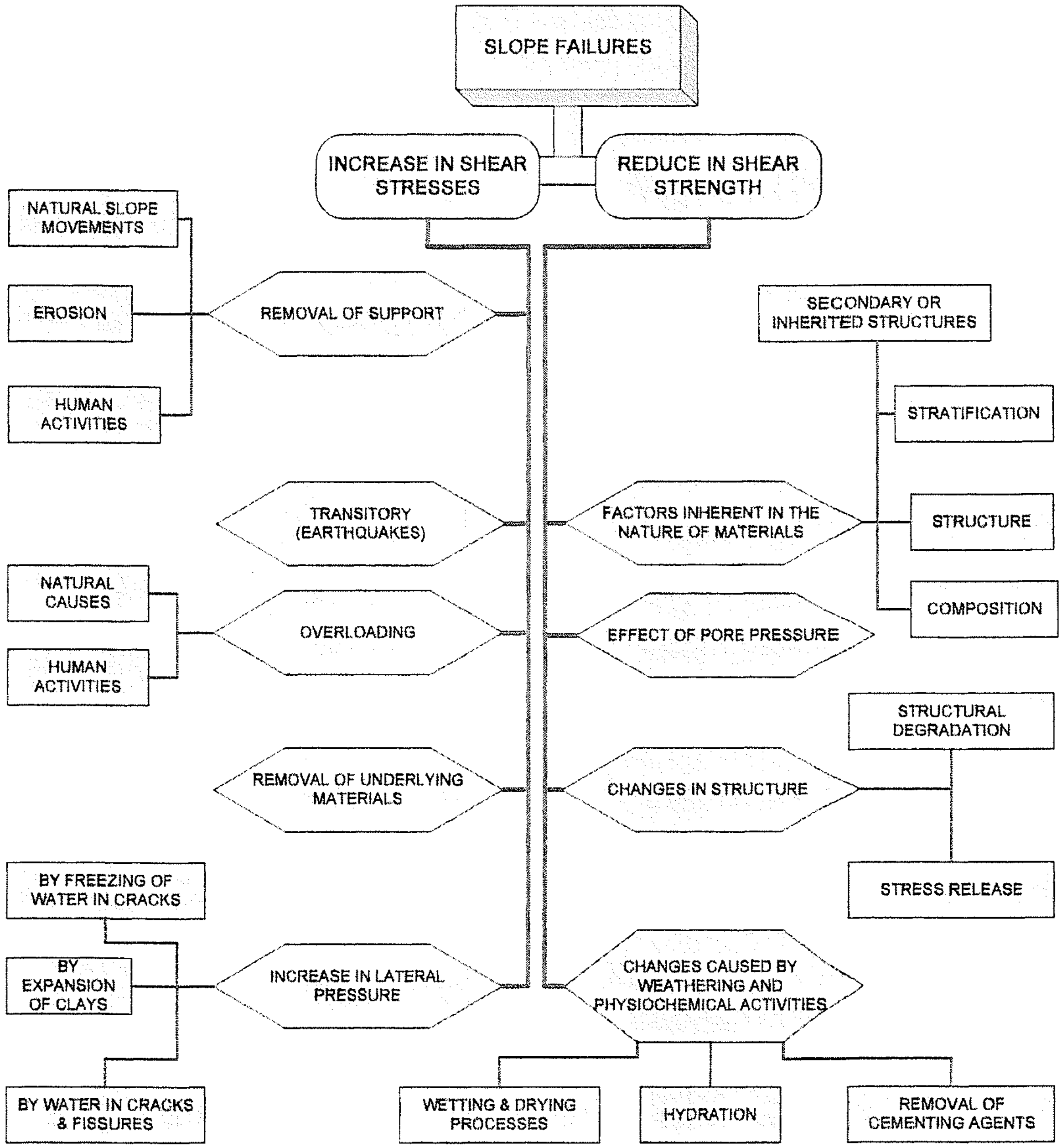

Figure 2.3 Flowchart of the causes of shear failure (after Highway Research Board, 1978)

2.10 Limit Equilibrium Method (LEM) and Finite-Element Method (FEM) of slope stability analysis

Generally the effects of implementing rockfill columns for slope remediation are analyzed using traditional limit equilibrium analysis (Tutkaluk 2000). Since slope stability applications are generally based on two-dimensional plane strain analysis, the rockfill column spacing and layout are transformed into an equivalent shear key of appropriate width. The shear key is then placed in the stability model with the appropriate properties based on the understanding of rockfill behaviour. The influence on the factor of safety is then calculated by examining slip surfaces passing through the rockfill materials consistent with the critical slip surfaces found prior to incorporating the rockfill materials. The limitation in this approach is that the relative displacement of the clay and rockfill to mobilize their respective shear resistance is not considered. This is simply a limitation of the LEM (Krahn 2003). As such, there is considerable need to establish the magnitudes of required movements to mobilize the assumed frictional strengths and to determine how the movements relate between the two materials. Only this understanding will lead to better defining appropriate strength selection criteria based on anticipated movements for limit equilibrium analysis.

Generally, factor of safety is a good indication of slope stability. This factor is also used for assessment of the degree of improvement following implementing of any slope reinforcing element such as columns. Although FEM is an accurate and powerful technique to analyze the stability of slopes, most geotechnical engineers still use traditional limit equilibrium methods because of simplicity and accuracy.

2.10.1 Limit Equilibrium Method (LEM)

Limit equilibrium analysis is one of the most popular methods for defining the stability of slopes by computing the factor of safety. The accepted factor of safety is determined for the surface that produces the minimum value, i.e. the surface which represents the critical surface. Yamagami et al. (2000) has applied a limit equilibrium approach to analyze slopes with stabilizing piles. Bishop's method was employed to find critical slip surfaces after installing piles in slopes. They concluded that pile installation produced a satisfactory design.

Turner and Schuster (1996) stated that the design of soil slopes and the mechanism of shear failure involve two categories: (1) evaluate the stability of slopes and (2) predict the slope deformation. Both of them are essential for analyzing the stability of slopes. The limit equilibrium method can be used to evaluate slope stability. However, finite element method is the only method that computes slope movements and stresses throughout the slope.

Turner and Schuster (1996) defined the factor of safety FS in LEM as the ratio of the magnitude of available shear strength divided by the shear strength required along a potential failure surface to maintain stability.

$$[2.11] \quad \text{Factor of Safety (FS)} = \frac{\text{shear strength of the soil}}{\text{shear stress required for equilibrium}}$$

Factor of Safety can be defined also as the ratio of total resisting forces to total driving forces over the critical slip surface, or the ratio of total resisting moments to total disturbing moments (in the case of circular slip surface).

$$[2.12] \quad FS = \frac{\text{resisting forces}}{\text{driving forces}} \text{ or}$$

$$= \frac{\text{resisting moment}}{\text{driving moments}}$$

In failed slopes, the factor of safety is usually considered to be equal to one (or less). For a typical safe slopes the accepted design value of factor of safety is in the range 1.25-1.5.

Limit equilibrium method or the method of slices are applied in frictional soils by subdividing the arc (or the slip surface) into slices. The normal stress and frictional strength can be computed at the base of these slices. Usually the arc is divided into 12-25 slices (Turner and Schuster 1996). LEM assumes the same value of factor of safety at all points on the slip surface; this is unlikely to be true. Therefore this assumption is one of the issues that casts doubt on its reliability. The critical slip surface is most likely to occur at the plane with minimum factor of safety. Various LEM can be used to analyze slopes. Different assumptions were placed for those methods to solve the equations. The characteristics of commonly used methods with regard to static equilibrium methods are summarized in Table 2.1.

Table 2.1 Characteristics of the methods used in limit equilibrium method (after Abramson et al. 2002)

Method	Force Equilibrium		Moment Equilibrium	Accuracy	Application
	x	y			
Ordinary method	No	No	Yes	low-very inaccurate	circular slip surfaces
Bishop's simplified	Yes	No	Yes	medium accuracy	-
Janbu's simplified	Yes	Yes	No	medium accuracy	any shape of slip surface
Lowe and Karafiath's	Yes	Yes	No	medium accuracy	any shape of slip surface
Modified Swedish (US Corps of Engineers)	Yes	Yes	No	medium accuracy	any shape of slip surface
Spencer's	Yes	Yes	Yes	accurate method	any shape of slip surface
Bishop rigorous	Yes	Yes	Yes	accurate method	circular slip surfaces
Janbu's generalized	Yes	Yes	No	accurate method	any shape of slip surface
Sarma's	Yes	Yes	Yes	accurate method	any shape of slip surface
Morgenstern-Price	Yes	Yes	Yes	accurate method	any shape of slip surface

2.10.2 Finite Element Method (FEM)

Using FEM became more creditable in slope stability analysis and design with the availability of more powerful and faster personal computers. The FEM has many advantages over the traditional limit equilibrium method. The main disadvantage of the traditional limit equilibrium methods is that many assumptions are needed to be assigned in advance. These assumptions include the failure surface location, the failure surface shape, and the forces and directions between slices. The FEM does not require any of these assumptions to be made in advance (Griffiths and Lane, 1999).

Other advantages of using the finite element method are:

(1) It can simulate engineering interventions such as geotextile, geodrains, retaining walls, soil nailing, etc.

(2) FEM can model progressive failure and simulate the field conditions more closely.

(3) FEM computes the equilibrium stresses, strains and the associated shear strengths in the slope body mass. Therefore the factor of safety can be calculated accurately.

There are some difficulties that limit the use of FEM. The traditional LEM is simple and more popular. Numerical analysis of FEM is more complicated, mesh generation and model setup can be difficult, and the analysis is much slower. FEM requires more soil parameters and boundary condition definitions. However FEM can be used to achieve an optimum design solution in terms of columns location, diameter, spacing and depth (Hammah et al. 2004, Baker 2003, Shukha and Baker 2003, Yamagami et al. 2000, Duncan 1996, Poulos 1995, Matsui and San 1992).

Matsui and San (1992) presented a finite element slope stability analysis using shear strength reduction (SSR) technique. They applied this method to reinforced cutting through the field test data. They concluded that the slip surface can be successfully traced for embankment and excavation slopes by the shear strength reduction technique. They stated that an agreement between the shear strength reduction technique and the modified Fellenius' method was satisfactory. Griffiths and Kidger (1995) stated that the FEM is able to give accurate estimates of collapse loads in geotechnical problems. Poulos (1995) used computer analysis for the response of a reinforced pile to laterally moving soil. He presented an approach for the design of piles to stabilize slopes.

Chow (1996) and Hassiotis et al. (1997) concluded that FEM can be used in the analysis and design of stabilized slopes. Numerical methods were used to analyze the stability of unreinforced and reinforced slopes including single and grouped of piles to achieve an optimum design solution; pile diameter, centre-to-centre distance, and location of the pile row. These parameters are required in determining strength so that the slope stability and the pile integrity are assured. Hassiotis et al. (1997) developed relationships between factor of safety and spacing between piles in terms of pile diameter. They indicated that a spacing equal to or less than 2.5 times the pile diameter should be sufficient to permit the piles to act as a group. Furthermore, they recommended that piles must be placed in the upper middle portion of the slope to provide the optimum safety factor.

Another numerical solution that uses the strength reduction technique is the finite difference method. Shukha and Baker (2003) applied the finite difference analysis with the strength reduction technique to perform slope stability analysis.

They used the computer software FLAC (Fast Lagrangian Analysis of Continua) as the modeling platform. The results of conventional limit equilibrium analysis matched those using FLAC.

Griffiths and Lane (1999) stated that finite element method is a very effective method for slope stability analysis of natural slopes and dams. They concluded that using the FEM in conjunction with an elastic-perfectly plastic (Mohr-Coulomb) stress-strain method has been shown to be a reliable and robust method for assessing the factor of safety for slopes. They have concluded that the Mohr-Coulomb criterion remains the one most widely used in geotechnical practice and was used throughout their analysis. Dawson et al. (2000) stated that the factor of safety of a slope can be computed accurately with a finite element or a finite difference code using soil shear strength reduction (SSR) in stages until the slope fails. They mentioned that the SSR method has a number of advantages over the method of slices for slope stability analysis. This technique is a general purpose tool that can be applied to almost any geotechnical stability problem. Duncan (1996) stated that the factor of safety for slopes can be calculated iteratively by dividing the shear strength of soil by a factor to bring the slope to the verge of failure. This scenario is defined as the shear strength reduction method. The factor used for the iteration is called the Shear Reduction Factor (SRF). Dawson et al. (1999) pointed out that the factor of safety for slopes computed with the finite element method using the shear reduction technique was comparable to the upper-bound values provided by the traditional limit equilibrium method.

Hammah et al. (2005) confirmed the above observations in that finite element analysis using the shear reduction method enjoys many advantages. This method can predict stresses and deformations of the reinforced element, such as piles, anchors, and geotextile in various stages and bring them to failure.

Swan and Seo (1999) presented comparisons between the classical method (limit equilibrium analysis) and finite element method approaches for slope stability analysis. They applied both the *gravity increased method* and *strength reduction method* with finite element analysis. They stated that in purely cohesive soils, both methods yield identical results. But strength reduction method appears well suited for analyzing the stability of existing slopes in which unconfined active seepage is accuring.

The gravity increase method and the strength reduction method are briefly discussed here. Gravity increase method (GIM) is conducted by increasing the gravity load (g) until the slope becomes unstable, and the equilibrium solutions no longer exist (Swan and Seo 1999, Pham and Fredlund 2003):

[2.13] $g(j) = g_a{}^{*} f(j)$

[2.14] $FS_{gravity} = \frac{g_{critical}}{g_{actual}}$

where, g_{actual} = the actual gravitational acceleration,

$g(j)$ = gravitational acceleration variable, it is a function of the increasing factor j.

$g_{critical}$ = critical gravitational value that separates between the equilibrium solution zone and the non-equilibrium zone (equilibrium solution does not exist) as shown in Figure 2.4 (A).

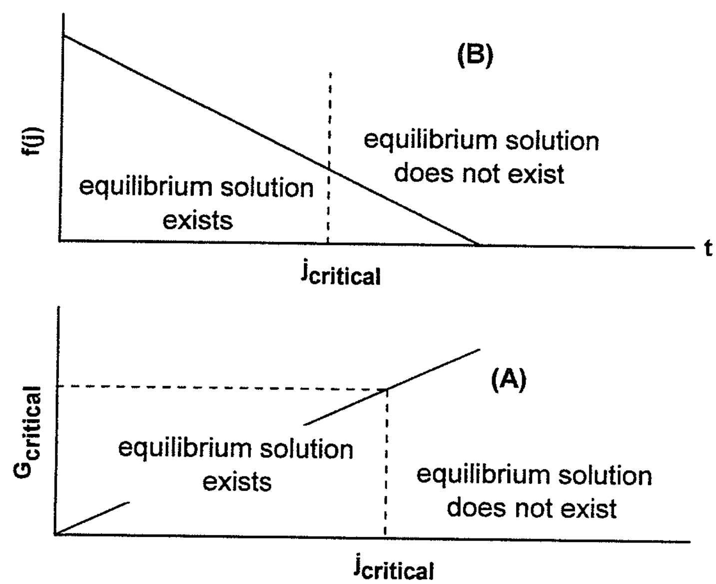

Figure 2.4 Critical boundaries for both gravitational and shear reduction methods (after Swan and Seo 1999)

The shear strength reduction method is conducted by decreasing the shear strength parameters of the soil until the slope becomes unstable and an equilibrium solutions no longer exist (Swan and Seo 1999, Griffiths and Lane 1999):

[2.15] $S(j) = S_a * f(j)$

[2.16] $FS_{SSR} = \dfrac{S_a}{S(j_{critical})}$

where, S_a = the actual strength parameters, f(j) = factor as a function of j, and S(j) = the variable shear strength, it changes as the factor of reduction changes.

Swan and Seo (1999) suggested that the SSR technique is suited for slope stability analysis of natural slopes because of the potential seepage in the slopes. On the other hand, the GIM approach is suited for stability analysis in embankments and fill applications because it closely simulated staged construction of embankments and fills. Therefore SSR was used in this study to analyze the stability of riverbanks. This study used plane strain condition and the Mohr-Coulomb constitutive model. This simplified condition and constitutive model are the only ones available in the computer software used in this study, Phase 2 which is a two-dimensional finite element program developed by Rocscience Inc. (Rocscience 2005).

The shear strength τ can be calculated through the Mohr-Coulomb model by using soil parameters such as cohesion, friction angle, and the applied normal stress:

[2.17] $\tau = c + \sigma_N \tan\phi$

At surface of failure the shear stress can be calculated again:

[2.18] $\tau_f = c_f + \sigma_N \tan\phi_f \qquad \{ c_f = \frac{c}{SRF} \ ; \ \tan\phi_f = \tan^{-1}(\frac{\tan\phi}{SRF}) \}$

The shear strength parameter c_f and $\tan\phi_f$ are reduced by a Shear Reduction Factor (SRF) until the slope is not stable anymore. One of the indications of slope instability in finite element analysis is through the non-convergence of results of stress distribution in Mohr-Coulomb criterion.

2.11 Probabilistic and statistical analysis of the stability of riverbanks

There are basically two distinct statistical studies. The first one is based on probability method which required that the acceptability of slope stability design should be based on the probability of slope failure. This type of assessment requires carrying out a sensitivity analysis, leading to an economical design. Instead of using a single value for each parameter involved in design, which leads to one single value of factor of safety, the influence of a series of values for each significant parameter on the factor of safety was studied. A more rational assessment of risks associated with each single type of stabilization can be achieved by combining both a sensitivity study and probability of failure.

The second statistical study is statistical analysis and modeling using Design of Experimental Method (DOE). DOE is a logical approach for determining cause and effect relationships. It allows determination of interactions between factors that would never be found through a one-factor-at-a-time method. This method helps in optimizing processes through Response Surface Methodology (RSM). This approach has not been used extensively in slope stability analysis compared to the probabilistic approach. This study attempts to use DOE in an assessment of the performance of rockfill columns for riverbank stabilization. Details of the approach are given in Chapter 7.

2.12 Summary

In this chapter, mechanisms related to the behaviour of natural riverbanks in Winnipeg have been reviewed to provide a background to the chapters that follow. Previous studies on the soft ground improvement techniques, especially rockfill columns and soil-cement columns as useful techniques to reinforce cohesive soft soils and natural slopes have also been reviewed.

The main points covered in this chapter include the following:

1) The geological structure and the engineering properties of Winnipeg soils are described. This includes the thicknesses, stratigraphy, and extent of soil strata.
2) Soil deposits including lacustrine high plastic clays and the weak soil underneath are described. Description of soils involved soil mineralogy,, index properties, soil classification and permeability. Recommended shear strengths of Winnipeg clays are discussed in terms of their appropriate use to design and analysis.
3) Geometry of Winnipeg riverbanks as reported by Mishtak (1964), City of Winnipeg (2000), and Tutkaluk (2000) have been shown generally to be about 6H:1V for equilibrium. Actual topographical mapping at the site is essential to landslide analysis and for designing the stabilization elements.

4) Field observations of riverbank landslides have shown that they incorporate large sections of the slope. The failure surfaces were deep seated, extending 10 to 15 meters below the ground surface, and extending horizontally as far as 60 meters from the river.

5) Groundwater conditions are one of the significant factors in slope stability. Higher groundwater table level increases the pore water pressure, which tend to reduce the strength. The regular groundwater table level is usually at 1 to 2 meters below ground surface. The influence of river level has also been investigated. The regulated river level of 223.9 m represents the lowest river level during summer. However, in the Fall due to low the river level associated with higher aquifer pressure, stability of riverbanks in Winnipeg will be at a minimum.

6) Slope stabilization methods generally increase stability by either increasing resisting forces, reducing driving forces, or by a combination of both functions. Rockfill columns and soil-cement columns were considered among techniques that comprise both functions. These are considered as the most efficient methods to stabilize riverbanks in Winnipeg for two main reasons: The first is due to deep-seated failure surface. Secondly, soil deposit consists of high plastic clays with low values of stiffness and shear strength.

7) Rockfill columns have become recognized as a cost-effective method for riverbank stabilization in Winnipeg for more than ten years. However, mixed results have been experienced. Some cases in Winnipeg have

shown that excessive displacements continue to occur even after the installation of rockfill columns.

8) Soil-cement columns or a Deep Mixing Method can be used to improve the average shear strength and shear stiffness of slopes of soft clay. This technique proved to be more effective over other traditional methods as it is more flexible in arranging the columns in different patterns.

9) Limit equilibrium methods (LEM) and the finite element method (FEM) can be used to assess the stability of natural and reinforced riverbanks. LEM is preferred by industries due to its simplicity and accuracy, while the FEM is more accurate as it is able to determine the failure surface location and shape, the equilibrium stresses and strains, and the associated shear strengths in the slope body mass.

10) The shear strength reduction method has been used and has proven to be creditable for estimating the factor of safety of slopes.

11) The influence of various factors on the calculated factor of safety may be analyzed using a Design of Experimental Method (DOE). The factor's interaction and main effect can then be assessed.

Chapter 3

LARGE-SCALE DIRECT SHEAR TEST ON ROCKFILL COLUMN MATERIALS

3.1 Introduction

Rockfill columns are generally used in soft clay foundations to reduce total and differential settlements and to increase bearing capacity (e.g. Bergado et al. 1994, Aboshi et al. 1979, Diyaljee and Pariti 1990, Goughnour et al. 1990). Little information is available with regards to the use of rockfill columns in the stabilization of natural slopes. The primary difference between embankments and natural slopes such as those commonly encountered in riverbanks is the way shear resistance is mobilized in the columns. In the case of embankment applications, the applied vertical stress from the weight of embankments can enhance the normal effective stress in the column. In the case of riverbank stabilization, the normal stress at any point in the column depends mostly from the weight of the column materials above that point. Therefore, riverbank failure mechanisms are expected to differ from those in the embankments.

In riverbanks stabilized with rockfill columns, the upper portion of the rockfill columns (during slope displacement) will tend to move with the slope soil and the lower portion of the rockfill columns will tend to stay at their original location as shown in Figure 3.1. This shear mode can be simulated in a large-scale direct shear test and the results can be used to predict the shear resistance and shear mobilization of the composite samples.

Large-scale direct shear tests were carried out using composite undisturbed-clay-rockfill samples to simulate and evaluate the performance of rockfill columns in stabilizing Winnipeg riverbanks. Large-scale apparatus is required to test the actual size of rockfill materials, and to measure the volume change during shearing.

The laboratory large-scale direct testing program for this research was conducted in three categories: on rockfill materials only, on undisturbed native Winnipeg clay, and on composite clay-rockfill samples. In the case of using composite samples, the tests simulated different cases including using cemented rockfill columns and rockfill columns in groups. It should be noted that Juran and Riccobono (1991) reported positive effect of low level cementation on the mobilization of shear resistance in compacted sand. Providing artificial cementation through grouting allows mobilization of shear resistance in rockfill

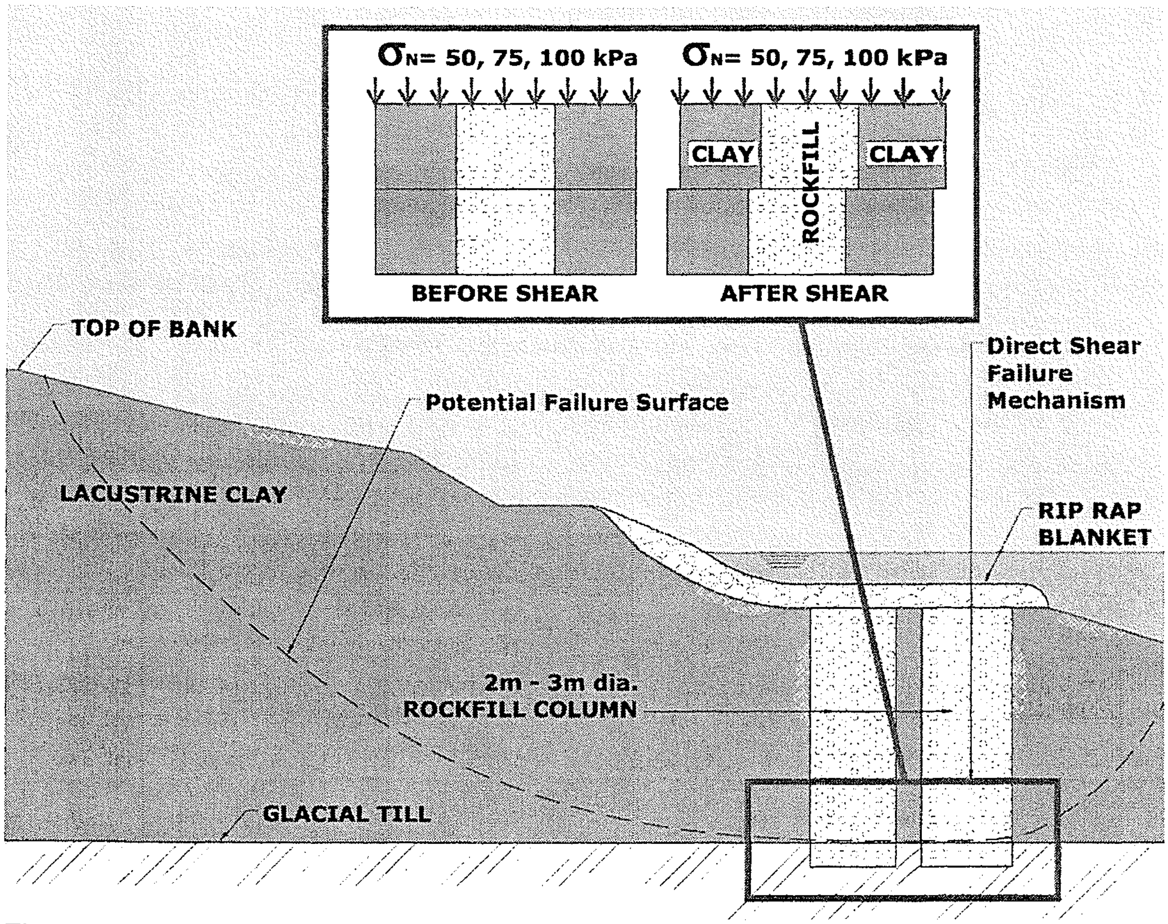

Figure 3.1 Typical riverbank cross-section stabilized with rockfill columns (after City of Winnipeg 2000)

columns at smaller strain. The higher mobilization of shear resistance at lower shear strain may reduce the movements of the riverbanks after column installation. The tests on group columns are done to investigate the effects of column spacing.

This chapter describes the materials used in the tests and the installation setups for each group of tests. Chapter 4 presents the results. These results will be used in the numerical analyses which are discussed in chapter 6.

3.2 Materials used in the tests

3.2.1 Rockfill materials

During the past ten years, limestone has been used successfully in rockfill columns for stabilization of riverbanks in Winnipeg. Limestone rockfill are taken from local quarries located northwest of Winnipeg. The crushed limestone has higher shear strength than the surrounding clay. Yarechewski and Tallin (2003) stated that crushed limestone was effective in increasing the stability of riverbanks in the City of Winnipeg. Three installation layouts have been used: 1) shear key, 2) rib type, and 3) rockfill columns. Crushed limestone originating from local quarries were used in this study.

Sieve analysis was carried out in general accordance with ASTM standards (ASTM D421-85 and D422-63, 1998) with slight modifications. The deviation from the ASTM Standard was the amount of material used in the sieve analysis. The ASTM Standard specifies the quantity of sample to be used in the test. However, the quantity of materials required for the test depends on the maximum particle size. The British Standard (BS 1377, 1990) specifies the minimum quantity of materials corresponding to the maximum size of the particles. For our material that has particle sizes of up to 100 mm, the minimum quantity is 150 kg. Therefore a sample of 240 kg of rockfill material was used in this study following the recommended procedure of using a quarter of a 1000 kg representative sample.

Sieve analysis was performed on two sets of samples. One sample was the original rockfill material used in the construction of rockfill columns in the field and the second was scaled-down mixture of rockfill materials to be used in clay-rockfill composite testing. Reducing the grain size distribution of the rockfill materials from its original size is necessary for testing the composite clay-rockfill specimens prepared for the direct shear test apparatus. Experimental and theoretical evidences (Marsal 1973; Gupta et al. 1995; Sitharam and Nimbkar 2000) suggest that as long as the grain size distributions are parallel to larger granular mixtures (of similar material) the stress-strain characteristics are relatively similar as well. Therefore, the reduced-scale sample was reproduced in such a way that its grain size distribution was parallel to that of the original rockfill material. Figure 3.2 shows the grain size distributions for the two samples. The maximum diameters in each of the original and reduced-scale materials are approximately 60 and 27 mm, respectively. This proportioning ensures that the maximum grain size of the rockfill materials would be at least 1/10th of the modeled rockfill column size in the direct shear test setup.

3.2.2 Native soil

Large undisturbed clay samples with diameter of about 67 cm were collected from a site located northwest of Winnipeg. These were obtained from depths of 12-15 m below the ground level. These levels coincide with the weakest zone in the riverbanks where shear failures are likely to occur. A large size of undisturbed clay is necessary to carry out direct shear tests on composite clay-

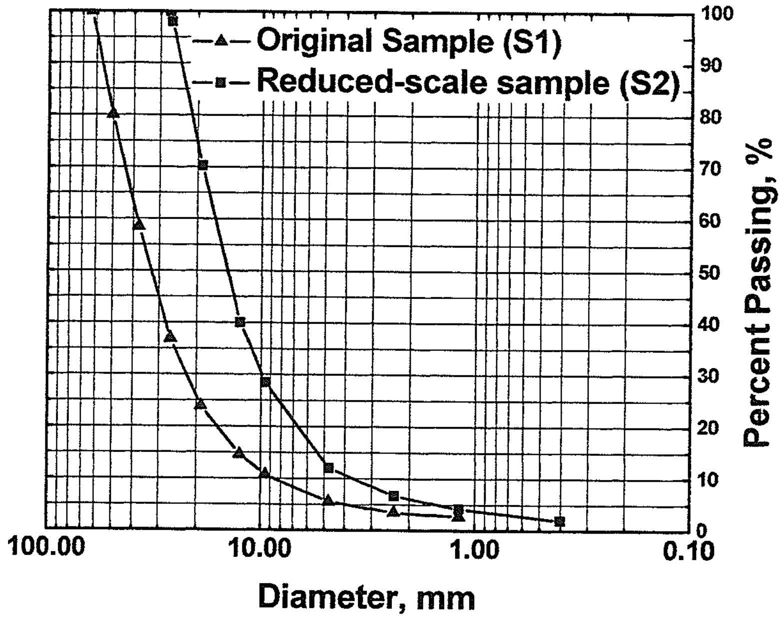

Figure 3.2 Grain size distribution of the original (S1) and scaled-down particle size (S2) samples of the rockfill soils

rockfill materials. The large sampler that was used to obtain the clay samples was designed and fabricated by a local drilling contractor[1]. Figure 3.3 shows the sampling of the large undisturbed lacustrine clay at depths of 12-15 m from the ground surface. To the author's knowledge, this is the largest undisturbed clay obtained at these depths.

Various supporting laboratory tests were conducted: Atterberg limit tests (ASTM D 4318), grain size distribution, hydrometer analysis, specific gravity (ASTM D 854), and water content determination (ASTM D 2216-90). The results are shown in Table 3.1:

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[1] Subterranean (Manitoba) Ltd.

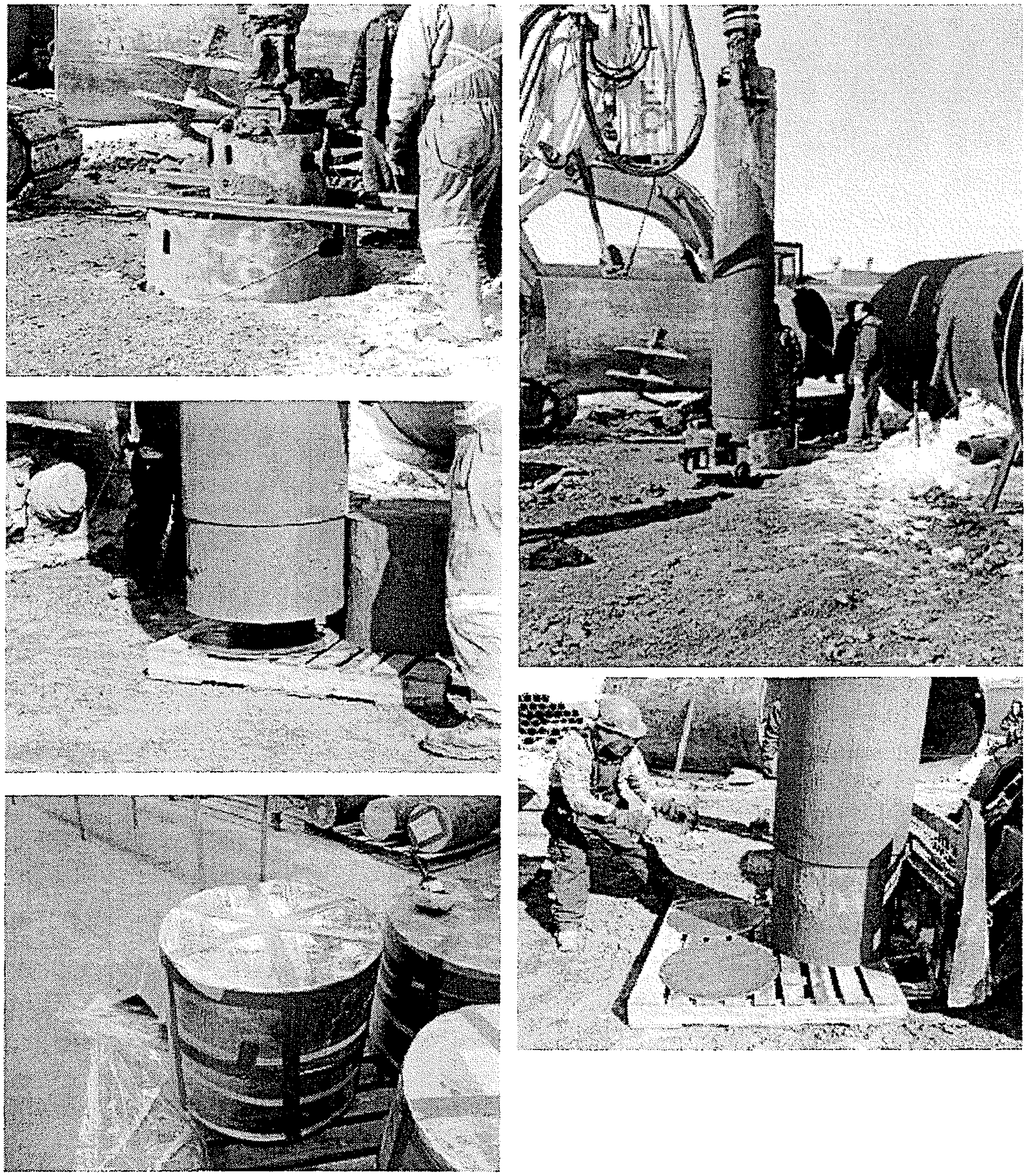


Figure 3.3 Sampling of large 'undisturbed' lacustrine clay

Table 3.1 Classification tests on Winnipeg clay

Test	Results
Natural water content	57-67 %
Liquid limit	76-80 %
Plastic limit	27-29 %
Plasticity index	50 %
Total unit weight	15.7-17 kN/m^3
Specific gravity	2.68
This soil is classified as "CH" according to the Unified Soil Classification	

3.3 Rockfill material density

The frictional strength and mobilization of shear resistance of the rockfill materials depend heavily on the relative density, D_r, of the material as placed. Relative density is defined as:

$$[3.1] \quad D_r = \frac{e_{max} - e}{e_{max} - e_{min}}$$

where, e_{max} = maximum void ratio (loosest condition),

e_{min} = minimum void ratio (densest condition),

e = current void ratio.

The determination of maximum and minimum dry densities (minimum and maximum void ratios) was carried out in general accordance with ASTM Standards (ASTM D 4253-93 and D 4254-91, 1996). One modification of the standard test was changing the size of the mould used in the test. The mould used has inner dimensions of 600 mm in diameter and 410 mm in height compared to the standard of 70 mm and 19 mm respectively. Three relative densities were specified for tests to cover the range of densities that may be expected using various placement methods. The three relative densities examined are as follows:

(1) Loose condition ($D_r < 15\%$)

(2) Medium-dense to dense condition ($D_r \sim 67\%$)

(3) Dense condition ($D_r > 90\ \%$)

The dry densities of both the original and scaled-down samples are summarized in Table 3.2.

Table 3.2 Unit weight of the original and reduced size samples

	Relative Density	Original Sample kN/m^3	Reduced-scaled Sample (27 mm maximum size) kN/m^3
1	Loose	14.7	15.0
2	Medium-Dense	17.0	17.3
3	Dense	18.6	19.1

3.4 Large direct shear test apparatus

The large-scale direct shear test apparatus shown in Figure 3.4 was used to measure the shear strength and mobilization of shear resistance of the rockfill materials and composite materials. The direct shear test closely simulates failure mechanisms in rockfill columns as shown in Figure 3.1. The 600 mm diameter shear box size of the large-scale apparatus allows large rockfill particle sizes up to 60 mm to be tested. The equipment is capable of performing direct shear tests in soil samples up to 600 by1200 mm in dimension.

This apparatus is equipped with four Linear Variable Displacement Transducers (LVDT's). Three Linear LVDT's are installed on the top cover to measure dilation and/or contraction. The fourth LVDT is connected to the upper box and measures shear displacements. The three top located LVDT's played a vital role in the findings of this research.

The apparatus is capable of performing both direct shear and pullout tests. In shear testing, the load is measured using a load cell attached to the front side of the upper box. A uniformly distributed normal stress can be applied on top of the sample with a rubber diaphragm using compressed air. Detailed description of the equipment is provided in Alfaro et al. (1995). In pullout testing, the upper and lower boxes are bolted together. The bolts are removed while running direct shear tests when the rollers on both sides of the box are engaged. The upper box can then be pulled or pushed relative to the lower box as shown in Figure 3.4. The rollers are equipped with a self-locking system that prevents any tendency for tipping the upper box with respect to the lower box as it is pulled or pushed during shearing. This ensures there are no eccentricities introduced due to the large-size of the apparatus. Vertical alignment of the pulling jack to accommodate engagement with respective pulling rods for pullout and direct shear tests is facilitated through an adjusting screw mounted on the reaction column. The pulling jack includes a universal joint that is free to rotate in both the horizontal and vertical directions.

Figure 3.5 shows the instrumentation layout used in the testing that includes measurement of horizontal displacements (Δx), shear load (P_X), vertical displacements (Δz) (dilatancy or contraction), and applied normal stress (σ_N). All instrumentation is connected to an electronic data acquisition system using "LABTECH" computer software to record the measurements at a selected interval of 5 seconds during all stages of testing. It should be noted that there is a layer of fine sand, about 20 mm in thickness, placed on top of the granular materials to prepare a plane surface and also to protect the rubber diaphragm.

Three normal stresses (50, 75, and 100 kPa) were used for the shear testing. These stresses were selected to represent a reasonable range of in-situ effective stress levels for rockfill columns, up to approximately 10 m in length, which would typically be utilized in the Winnipeg area (the length of rockfill columns varies according to the location along the bank and the thickness of the clay overburden

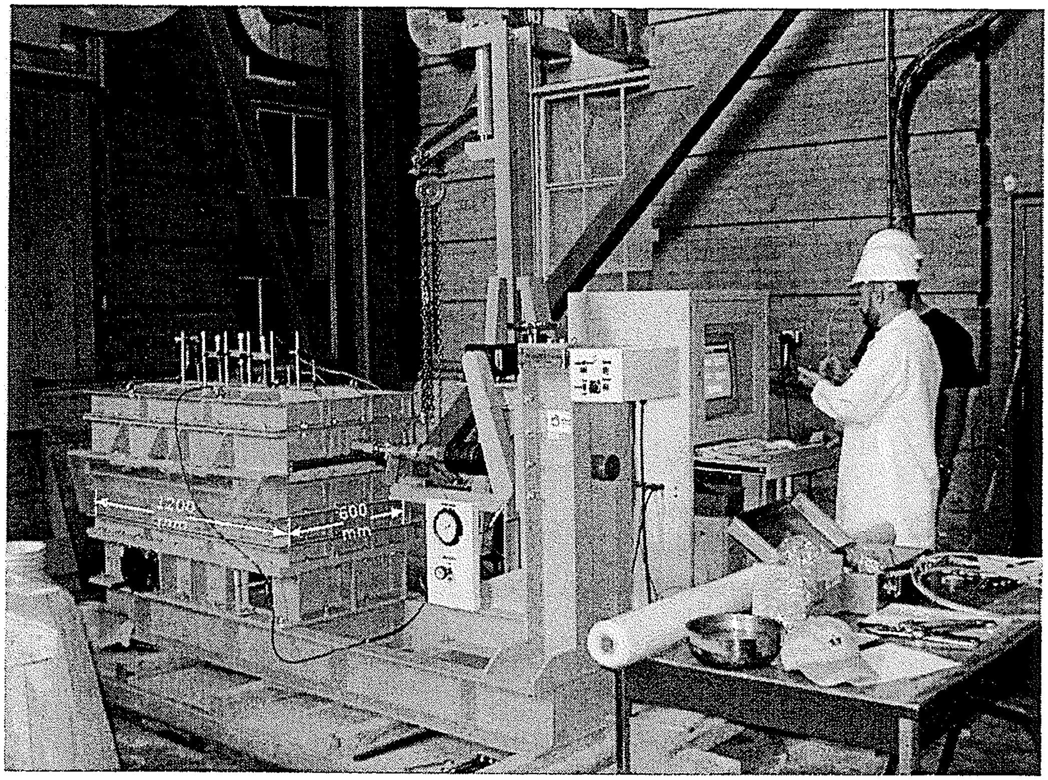


Figure 3.4 Photograph showing the large-scale direct shear test apparatus

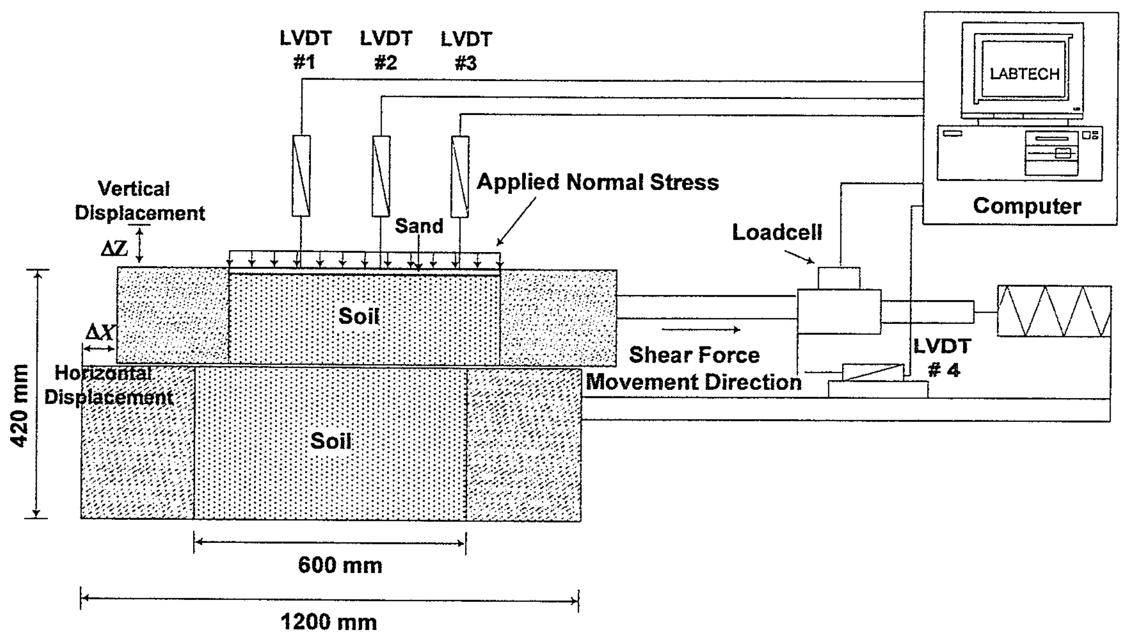


Figure 3.5 Schematic diagram of the large-scale direct shear test apparatus

at any one location). The shearing of each specimen was continued until the shear displacement reached approximately 50 mm which corresponds to approximately 8% shear strain. The mean shear stress is determined from the shear load (measured by the load cell) divided by the corrected cross–sectional area (Ac) of the test sample. That is,

$$[3.2] \quad \tau_p = \frac{(P_X)}{A_c}$$

The cross-section area (A_C) is the nominal area of the specimen corrected by the change of shear displacement of the sample:

$$[3.3] \quad A_c = \frac{D^2}{2}(\theta - \frac{\Delta x}{D}\sin\theta)$$

where, D = the internal diameter of the cylindrical box (mm),

Δx = shear displacement (mm), see Figure 3.5,

and θ is defined by:

$$[3.4] \quad \theta = \cos^{-1}\{\frac{\Delta x}{D}\}$$

Due to the large particle size and the angularity of the rockfill the shear response exhibits dilatant behaviour during shearing. The stress-strain characteristics can be described using a simple model following Coulomb's law. The net effect of ψ (dilation angle) due to increasing normal stress is that the failure envelope becomes curved. Using Coulomb's frictional law, the general form of the shear strength of soils are given in the following form:

[3.5] $\tau_f = \sigma'_{Nf} * \tan(\phi'_{cs} \pm \psi)$

Where the positive sign refers to soils in which the net movement of the particles is upward (dilation) during shearing and the negative sign refers to when the net particle movement is downward (contraction). Once dilation or contraction has taken place the material will tend towards a large strain frictional behaviour at constant volume. The friction angle at the critical state, ϕ'_{cs}, is a fundamental soil parameter while the friction angle at peak shear stress for dilating soil, ϕ'_p, is not a fundamental soil parameter but depends on the capacity of the soil to dilate (Budhu 2001). The general equation for the dilation angle, ψ is given as:

[3.6] $\psi = \tan^{-1}\{\Delta z/\Delta x\}$

where, Δz = vertical (normal) displacement,

Δx = horizontal displacement.

The transition friction angle lies on a straight line of gradient tan $\phi'_{transition}$ in the (τ, σ_N) space. Figure 3.6 shows the method of determining the transition friction angle. Atkinson (1992) stated that the point of zero dilation angle ψ = 0 (transition point and end point in Figure 3.6) represents the points of critical friction angle. The transition friction angle is equivalent to the corresponding friction angle at critical state as defined by Atkinson (1992) using the value of $\phi'_{transition}$ as an alternative method to determining $\phi'_{critical}$. This is important as the shear displacement corresponding to the critical state for densely compacted rockfill was found to be unattainable with the test equipment being used.

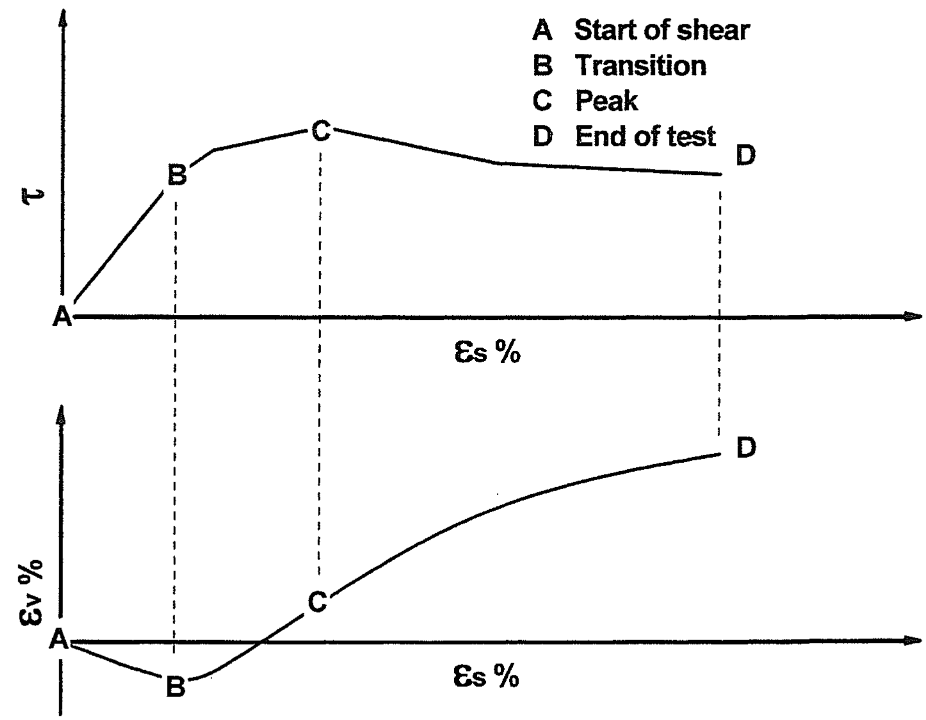


Figure 3.6 Shearing and dilation in shear test (after Atkinson 1992)

3.5 Test Conditions

The purpose of the experimental portion of this study is to determine if the presence of rockfill columns will enhance the engineering properties of the composite soil samples by providing an extra shear resistance to support riverbanks composed of high plastic clay. It examines how the presence of rockfill columns enhances the overall stability. The effects of column diameter, column relative density, and the applied normal stress will be studied. Other areas of interest are the area replacement ratio, cementation in rockfill, and the interaction of columns in group. It also investigates the shear mobilization of rockfill columns and the composite clay-rockfill materials.

In this study, large-scale direct shear tests have been conducted using six different conditions: (1) rockfill materials alone, (2) undisturbed clay 600 mm diameter, (3) undisturbed clay-rockfill composite, (4) remoulded clay 600 mm diameter, (5) remoulded clay-rockfill composite, (6) rockfill columns in groups, shear key layout and rib type tests.

3.5.1 Rockfill sample preparation

The rockfill materials were sheared under three different normal stresses: 50, 75, and 100 kPa. These stresses represent the stresses in the rockfill columns in the field equivalent to the depths below ground surface to a maximum of ten meters or about 100 kPa (refer to Figure 2.2 in Chapter 2). The samples were sheared up to 8% maximum displacement (the maximum limit of the apparatus). This level of strain was found to be sufficient to approach the peak strength of the densely compacted rockfill materials but insufficient to obtain the critical state strength; particularly at high level of normal stresses. Each sample was sheared at a rate of 2.4 mm/min, a slight modification of the ASTM standard wherein the rate of shearing is 10 mm/min. During each test, the horizontal displacement, vertical displacements, and shear stresses were recorded using the LABTECH electronic data acquisition system.

The relative densities that have been tested represent the different degrees of densification of materials at the construction site. The relative densities simulated in this study are: 14.7, 17.0, and 18.6 kN/m3. The process of densification involves compacting the rockfill material in three equal layers, where each layer was compacted using a vibrator probe (Fig. 3.7). The rockfill materials were placed in the shear mould in varying dry densities and normal stresses. Nine tests were conducted in total. The normal stress was applied by using a compressed air diaphragm. The tested soil sample was 600 mm in diameter and 410 mm in height. Three LVDT's located at the top were used to measure the vertical displacements of the rockfill materials during shearing (contraction or dilation). As in small-scale direct shear testing, vertical displacement measurements during shearing provide better understanding on the mobilization of shear resistance in rockfill column materials.

3.5.2 Undisturbed clay sample preparation

The principle behind the large-scale direct shear test for undisturbed clay samples is to measure the mobilization of shear resistance of the undisturbed lacustrine soft clay under the same boundary condition and scale effects as the undisturbed clay-rockfill composite test setup. As indicated earlier, large clay samples were obtained from 12-15 m below ground surface at a site in the northern part of the City of Winnipeg. The samples which have a diameter of 670 mm were cut in heights of 700 mm. The soil samples were cut out using a wire saw (see Figure 3.3). Each clay sample was placed on a circular . metal

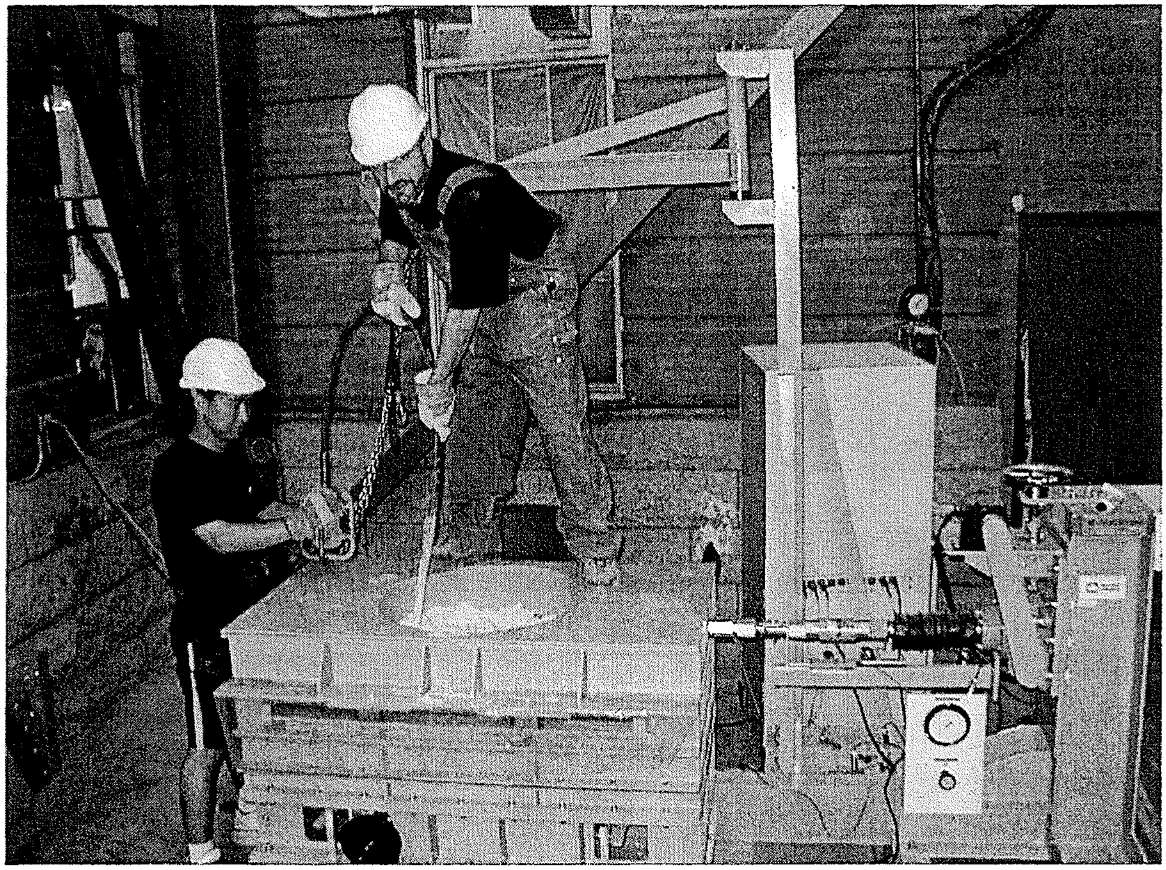


Figure 3.7 Densification of rockfilll materials in large direct shear box

plate, wrapped with multi layers of plastic sheets, and subsequently transported to the University of Manitoba Structures Laboratory. The samples were then waxed to preserve the moisture content (see Figure 3.8) and stored in the constant humidity room of the Structures Laboratory. A wooden platform had been used to move the samples from one place to another in the laboratory with the aid of a forklift.

In order to fit the clay sample to the size of the large direct shear box, an edged trimmer was designed and manufactured by Subterranean (Manitoba) Ltd. (Figure 3.9). To minimize the potential disturbance of the clay sample, the process of trimming was performed carefully and slowly. Excess soil around the sample was manually removed using a wire saw before the automatic trimmer was pushed downward, Figure 3.10 shows a clay sample after trimming.

A special metal frame, comprising of two circular metal plates (10 mm thickness) connected by four ended screw rods, was use to facilitate the handling of the large sample and placing it in position at the shear box (see Figures 3.11). The base metal plate under the clay sample remains in the shear box during testing, while the rest of the components of the frame are removed prior to testing (see Figure 3.12). The extra height of the clay sample is trimmed out as shown in Figure 3.13.

Figure 3.8 Waxing and rapping the undisturbed clay samples

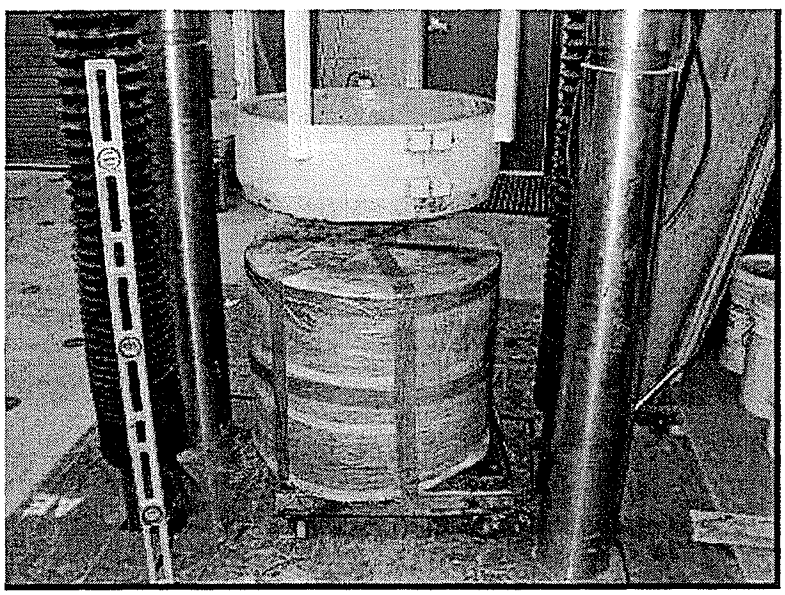


Figure 3.9 Trimmer used for the preparation of the large undisturbed clay samples

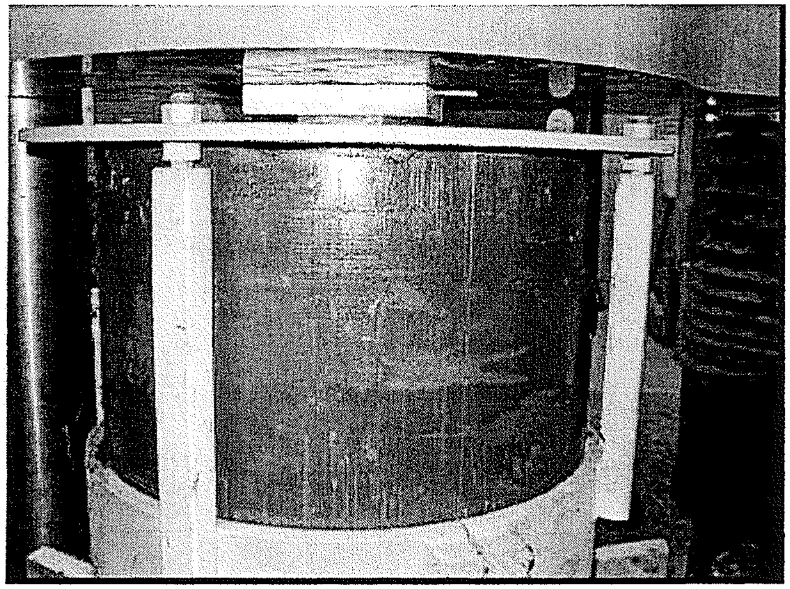


Figure 3.10 Clay sample after trimming

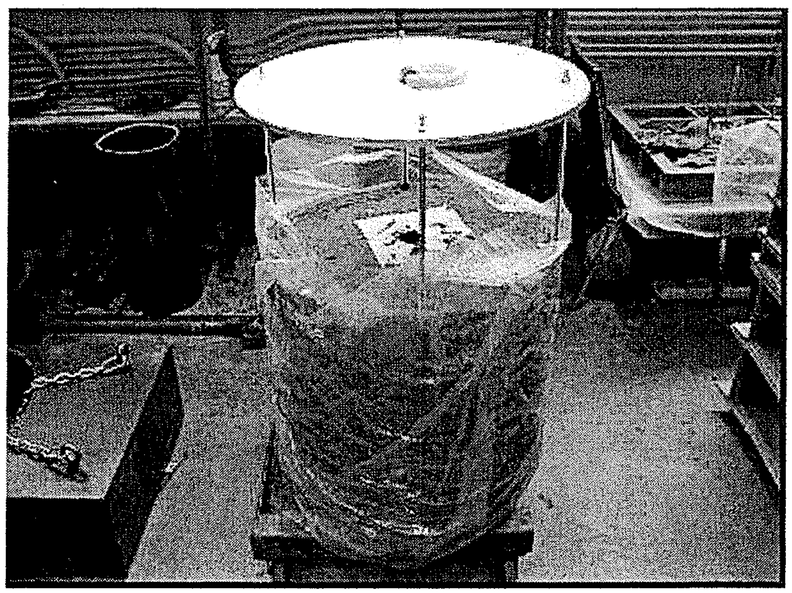


Figure 3.11 Moving an undisturbed sample to the shear box and a view of the metal frame

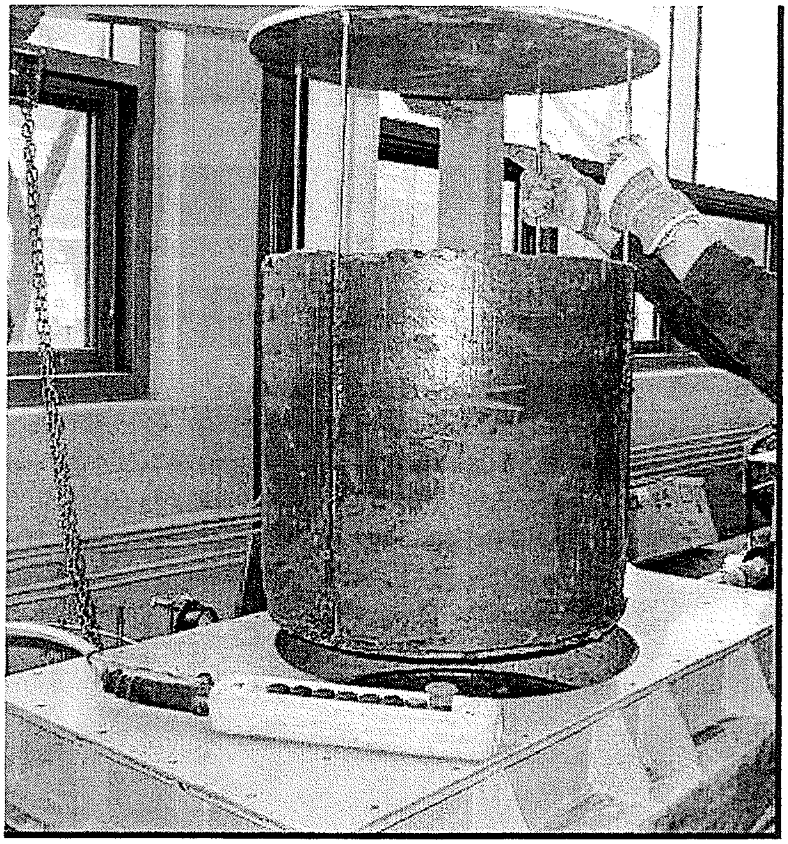


Figure 3.12 Placement of clay sample in the large-scale direct shear box

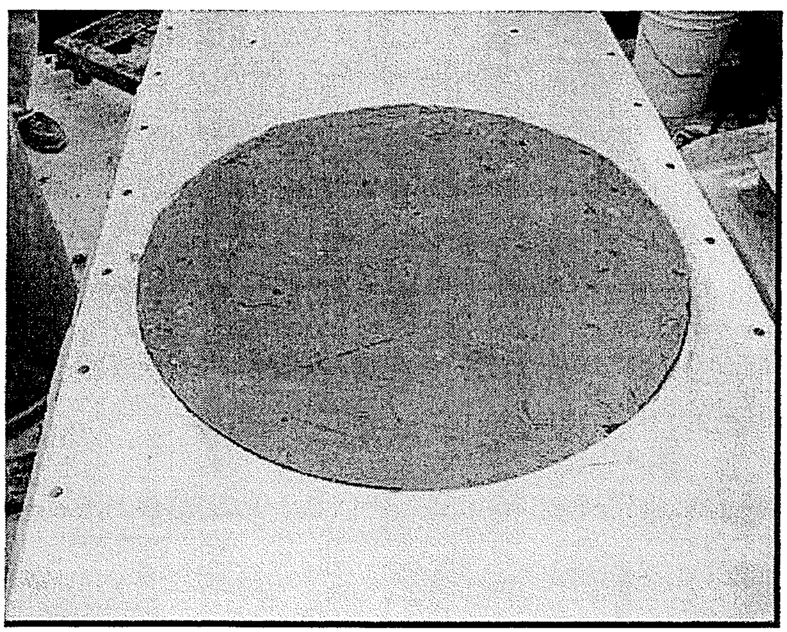


Figure 3.13 Photograph showing a trimmed clay sample fitted in the shear box

3.5.3 Clay-rockfill composite sample preparation

Tests on clay-rockfill composite were conducted to assess the shear mobilization of the composite material. The installation of rockfill column in the laboratory was done to closely simulate the field installation. In the field, rockfill column materials are installed either by a replacement or a displacement method. The experiment mimicked the replacement method which is the one used by local contractors in Winnipeg. The columns are installed by augering (drilling) a hole of about 2 meter diameter using a mechanical auger to a predetermined depth as shown in Figure 3.14. If there is a potential that the hole will collapse, a steel casing is driven before augering the clay. The casing is then removed incrementally as the rockfill column materials are compacted in place using a vibrator probe.

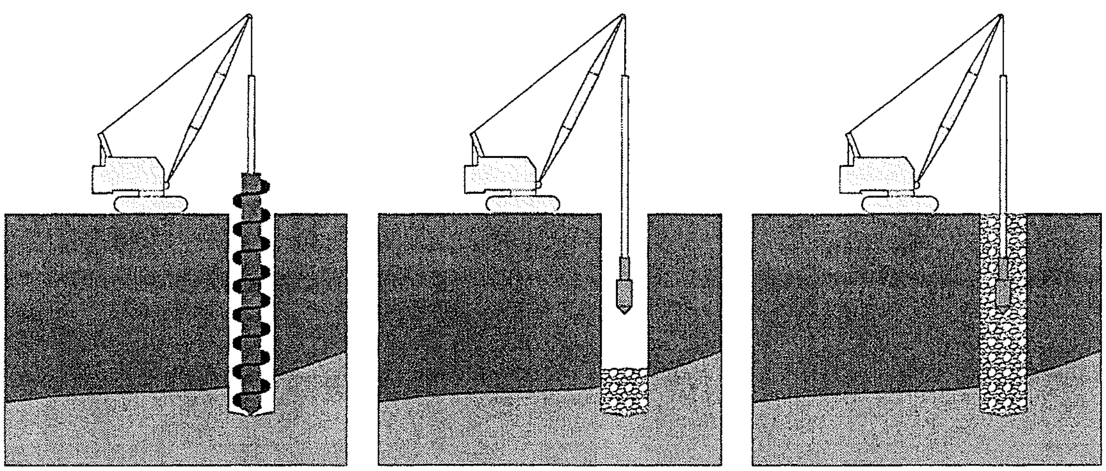


Figure 3.14 Rockfill columns installation process in Winnipeg

In the laboratory, the replacement method was simulated by manually augering the clay as shown in Figure 3.15. The augered hole was filled with rockfill materials in three equal layers, each layer being compacted with a vibrator to a target density.

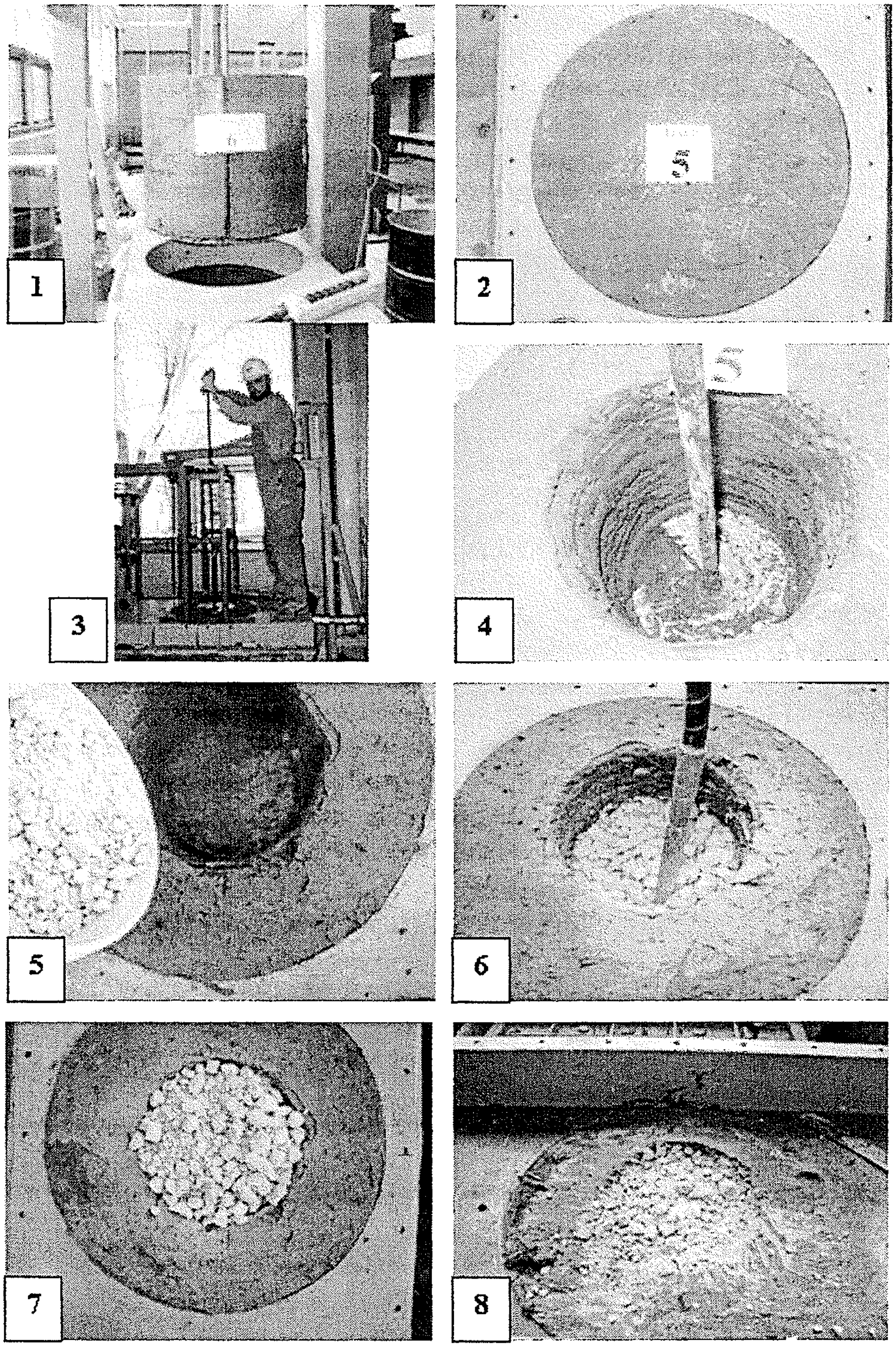


Figure 3.15 Steps showing the clay-rockfill composite sample preparation

3.5.4 Cemented rockfill column sample preparation

The goal of the cemented rockfill column tests was to study the influence of adding small amounts of cement to the rockfill material on the shear mobilization of rockfill columns. Type 50 Portland cement has been added at ratios of 0.5%, 2% and 5% cement by weight. Pozzutec 20^{+} accelerator was used to decrease the setting time and accelerate the curing time. A Pozzutec dosage of 16% of the total cement weight was used in the tests as recommended by the supplier's catalogue.

The setup for this test involves trimming out the undisturbed sample, placing the sample in the direct shear box, and augering the centre of the sample to the desired diameter (in this case only one size of hole: 270 mm). This is similar to the procedure mentioned in Section 3.5.3, except here we are adding cement to the rockfill materials. Dry cement was added to the rockfill, followed by adding water and Pozzutec. Pozzutec works effectively in cold temperatures and accelerates cement hydration but does not affect the final strength of the mixture. The densifying process was similar to the previous methods; the average dry density was 18.4 kN/m^3. A normal stress of 50 kPa was applied and the shear strain rate of 2.4 mm/min was used, similar to the previous tests. During shearing, horizontal and vertical displacements, and shear load were recorded.

3.5.5 Rockfill columns in group sample preparation

The aim of conducting these tests is to compare the performance of different layouts of rockfill columns on the shear mobilization. A group of rockfill columns was placed in two different arrangements; the first group was composed of five equal columns (168 mm diameter) arranged in two rows. One row with three columns located 130 mm from the far end, and a second row with two columns located (424) mm as shown in Figure 3.16. This arrangement is labelled as 'open-spacing' in the test result presentations. The spacing of these columns represents the wide range of area replacement ratios used in field installations of rockfill columns.

Five rockfill columns similar to the first arrangement were used in the second group of arrangement with the first row located in the same place as the first group. However, the second row of columns was located closer to the first row at a distance of 384 mm from the far end as shown in Figure 3.17. This arrangement is labelled as 'close-spacing' in the test result presentations.

The sample preparation for the tests on rockfill column in groups was different from the previous tests in that remoulded clay was used instead of undisturbed clay. This is because there were not enough undisturbed clay samples available.

To achieve the desired density the clay was compacted manually by tamping the clay in layers. The water content of the clay was identical to that of the undisturbed state. The process of installing rockfill columns was similar to that described in Section 3.5.4.

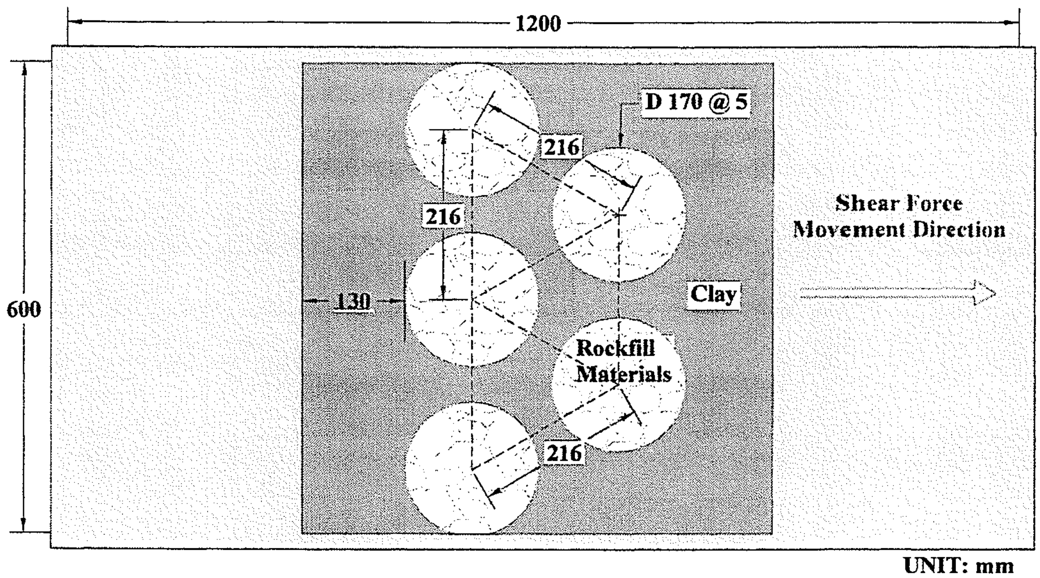


Figure 3.16 Rockfill columns in group: open-spacing

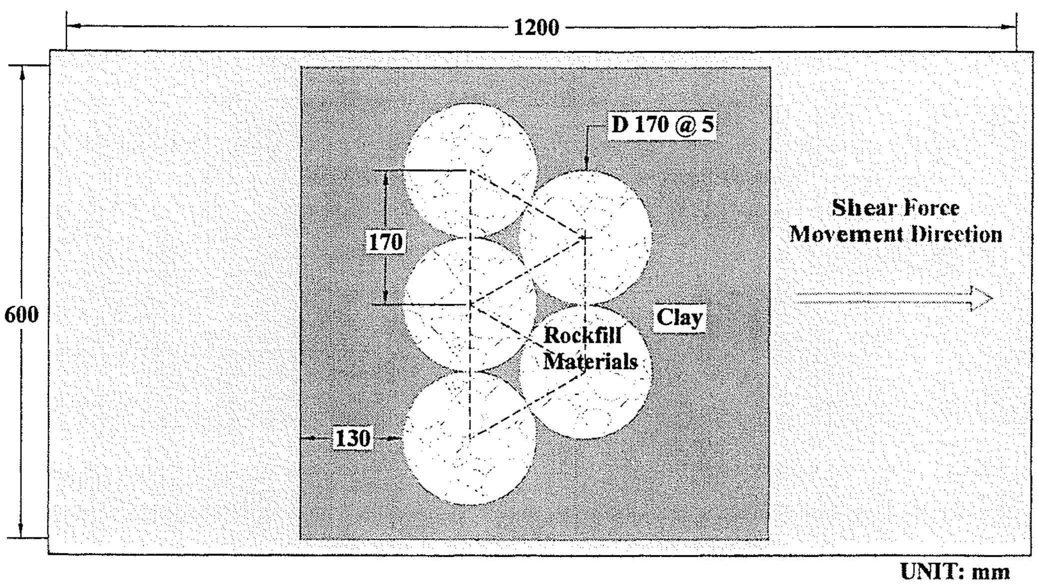


Figure 3.17 Rockfill columns in group: close-spacing

3.5.6 Shear key and ribbed-type sample preparation

Field observations by Yarechewski and Tallin (2003) showed different performance between various layouts of rockfill stabilization in Winnipeg riverbanks. Furthermore, Hassiotis et al. (1997) indicated that closely spaced piles provided interaction between them that can increase the factor of safety in slope stability. These tests attempt to compare the performance of the shear key and ribbed-type layouts with those of the rockfill columns in group. The installation of a shear key involves the following:

1) Placing a wooden framework in the shear apparatus perpendicular to the direction of movement with dimensions 315 mm x 600 mm x 410 mm.

2) Compacting clay soil in the direct shear apparatus (600 mm x 600 mm x 410 mm) in layers using a manual tamper as described in Section 3.5.5 to achieve the desired density.

3) Removing the framework to create a trench as shown in Figure 3.18.

4) Filling and compacting the rockfill materials in three equal layers.

The setup method for the installation of ribbed-type technique is similar to that of the shear key in terms of the area replacement ratio. The difference between them is the channel direction is parallel to the direction of shearing as shown in Figure 3.19.

The applied normal stress of 50 kPa was used. Shear strain rate of 2.4 mm/min and the maximum shear strain of 8% were used in these tests. These loading conditions are similar to previously described tests.

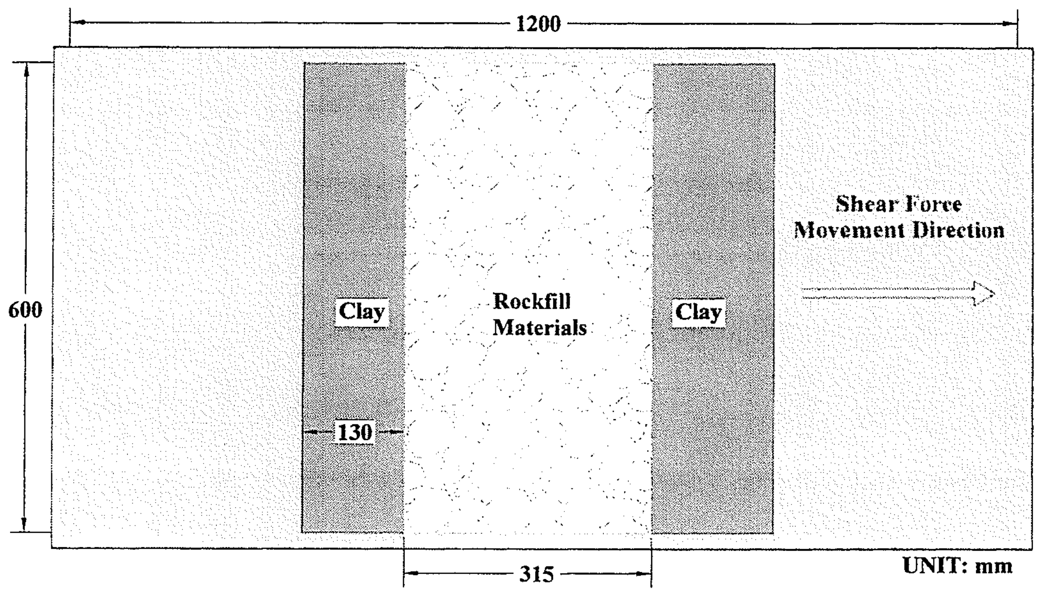


Figure 3.18 Plan view of the shear key layout

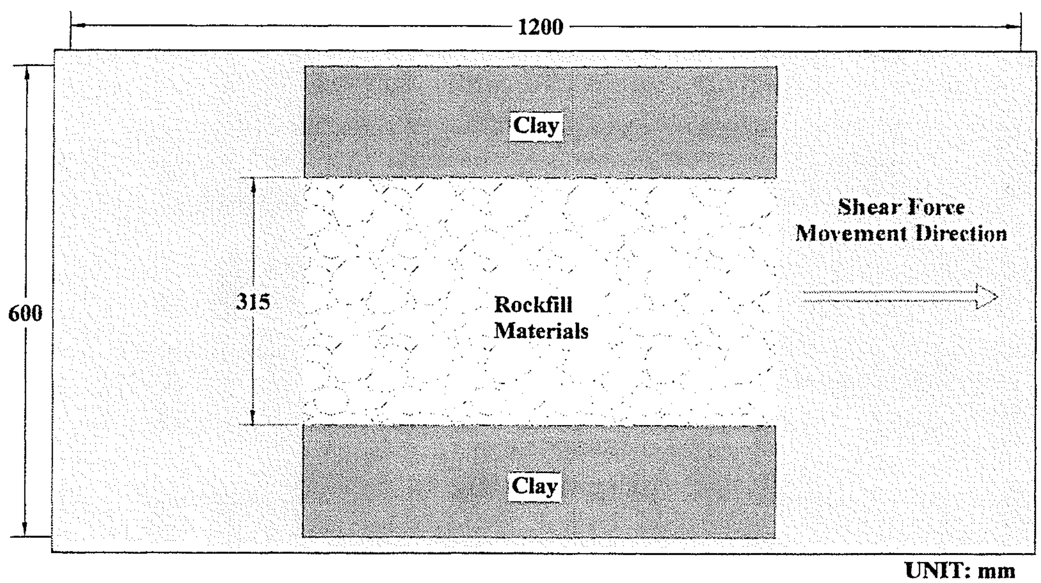


Figure 3.19 Plan view of the ribbed-type layout

3.6 Summary

The main points covered in this chapter can be summarized as follows:

1) A large-scale direct shear test is essential to determining the strength parameters of rockfill materials, as large-scale apparatus will accommodate the actual particle sizes of rockfill materials.
2) The mobilization of shear resistance of composite clay-rockfill samples is required to identify the stress-strain characteristics and thereby to obtain the failure strain and the degree of strain compatibility between the two types of soil.
3) Direct shear tests on composite clay-rockfill samples were conducted to assess the shear mobilization of the composite materials. The installation of a rockfill column in the laboratory was done to closely simulate the field installation. The replacement method using steel casing was used to construct the column.
4) The influence of adding a small amount of cement to the rockfill materials was studied. Large-scale direct shear tests of cemented rockfill were conducted to study the shear mobilization and shear stiffness.
5) Tests on rockfill columns in group were conducted in order to compare the performance of different layouts of columns on the shear mobilization.

Chapter 4

RESULTS AND DISCUSSIONS FROM TESTS ON ROCKFILL COLUMNS

4.1 Introduction

This chapter presents the results of tests on rockfill columns, which can be divided into four sections. The first section (Section 4.2) discusses the influence of normal stress and relative density on the mobilization of shear resistance of rockfill and clay materials. The second section (Section 4.3) evaluates the shear mobilization of clay-rockfill composite materials. The third section (Section 4.4) discusses the effects of area replacement ratios and low level cementation of the rockfill column. The last section (Section 4.5) compares the mobilization of columns in group compared to a single column and assesses the influence of installation layouts and procedures.

Large-scale direct shear tests were carried out for native clay, rockfill, and clay-rockfill composite. Conventional direct shear tests using traditional sample size on undisturbed native clays were also carried out under drained test conditions to determine the residual shear strength properties of clay. This will also be used in the numerical modeling to simulate the performance of the stabilized riverbank.

4.2 Direct shear tests on rockfill materials

4.2.1 Stress-strain characteristics of rockfill materials

Figures 4.1 shows the relationship between the shear stress, τ, and the shear strain, ε_S as determined from large direct shear test for densely compacted materials. Here, the shear strain was determined by dividing the shear displacement by the corrected length of the sample. This strain was used only for comparing the mobilization of shear resistances of different materials.

The corresponding results for loosely compacted materials are shown in Fig. 4.2. The tests were carried out under various applied normal stresses, σ_N, of 50, 75, and 100 kPa. The applied normal stress of 100 kPa is approximately equivalent to the effective stress condition in rockfill columns at about 10 m depth from the ground surface. Figures 4.1 and 4.2 illustrate that higher shear resistance is mobilized at higher applied normal stress as would be expected (Powrie 1997). These figures also show the shear stresses corresponding to the values taken for the determination of transition friction angles. The use of these values will be shown later. Note that for tests on the dense rockfill the mobilized shear stress is

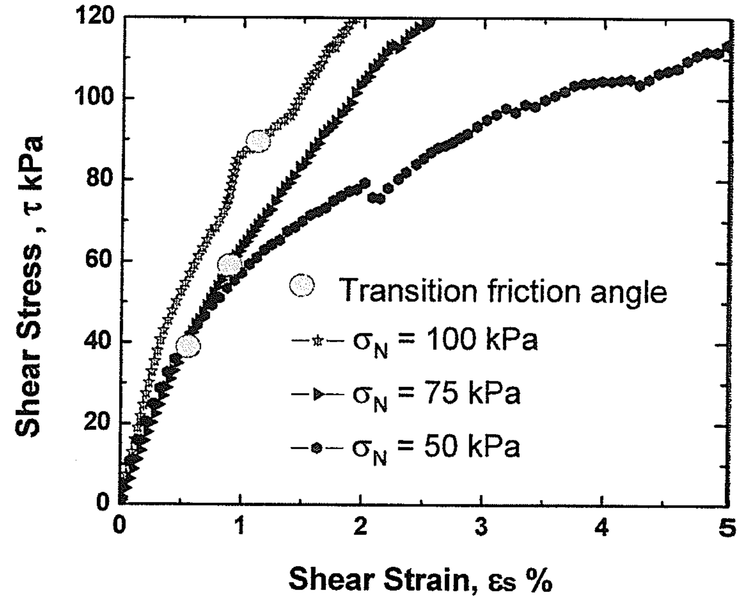


Figure 4.1 Shear mobilizations of dense rockfill materials

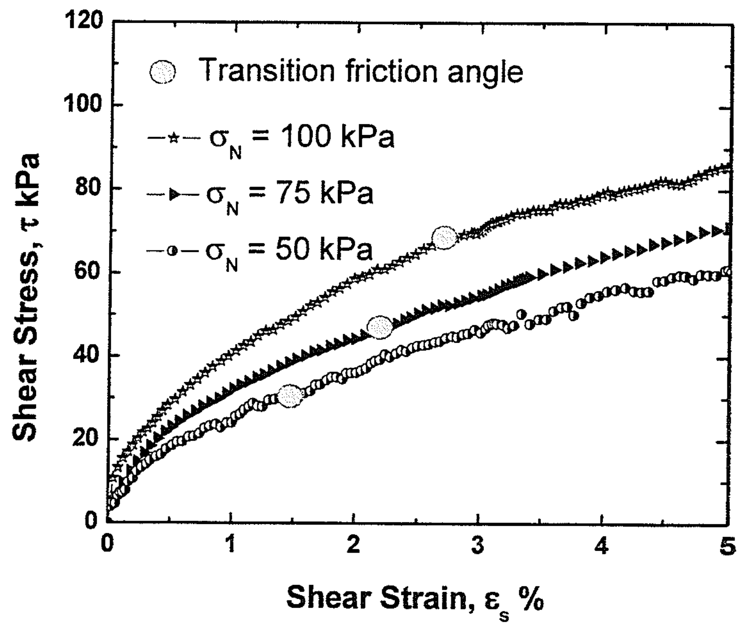


Figure 4.2 Shear mobilizations of loose rockfill materials

almost double its value in the loose condition at the same respective normal stress. The shear stiffness for dense rockfill was up to four times the corresponding value for the loose rockfill, depending on the shear strain level and applied normal stress. This indicates that the density achieved during placement of rockfill materials in columns has a significant impact on the movements required to mobilize the required shearing resistance.

The normalized shear load T/N versus normalized shear displacement, δ_h/D for dense and loose densities is shown in Figures 4.3 and 4.4. Here, T is the shear load, N is the normal load, δ_h is the shear displacement, and D is the diameter of circular cross-section soil sample. For example, the normalized shear load for the dense material was about 1.40 at 0.02 of normalized shear displacement, and the friction angle was about 60°. Similarly, in the loose condition at 0.02 of normalized shear displacement, the normalized shear load was only about 0.60 with friction angle of about 37°. The mobilization of the shear resistance is a function of the volume change during shear, which in turn depends on the density of the rockfill materials.

4.2.2 Volume change during the tests

When a densely compacted rockfill material starts shearing along its contact surface between upper and lower portions, particles lying above the contact surface will be forced to ride up which causes an expansion known as dilatancy. Conversely, when a loosely compacted rockfill material is sheared, contraction is expected.

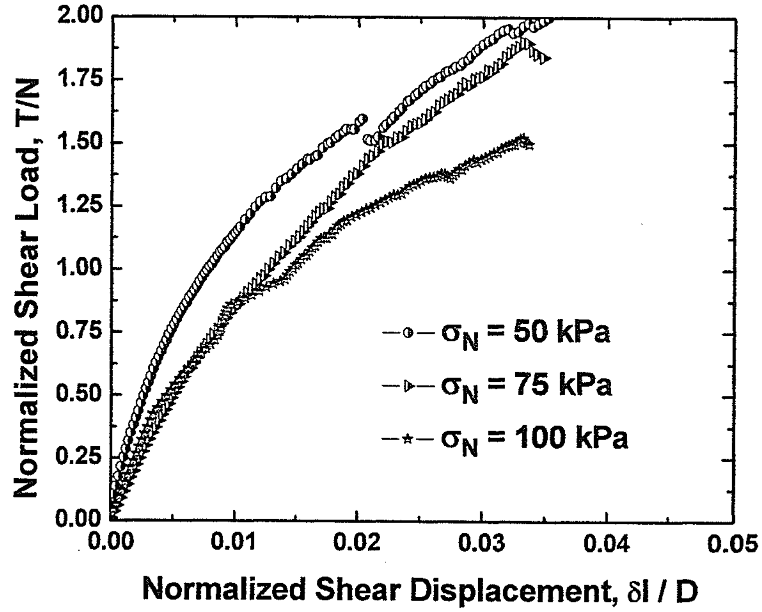


Figure 4.3 Normalized shear strength of dense rockfill materials

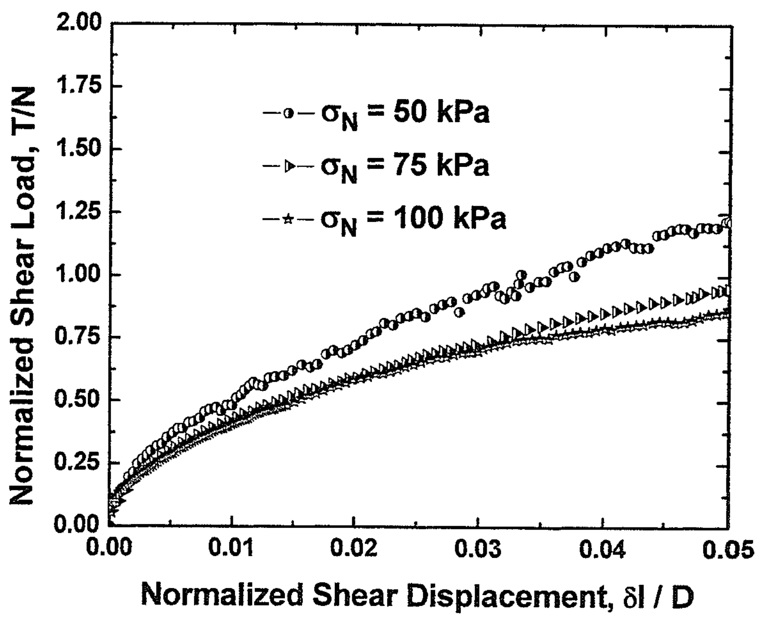
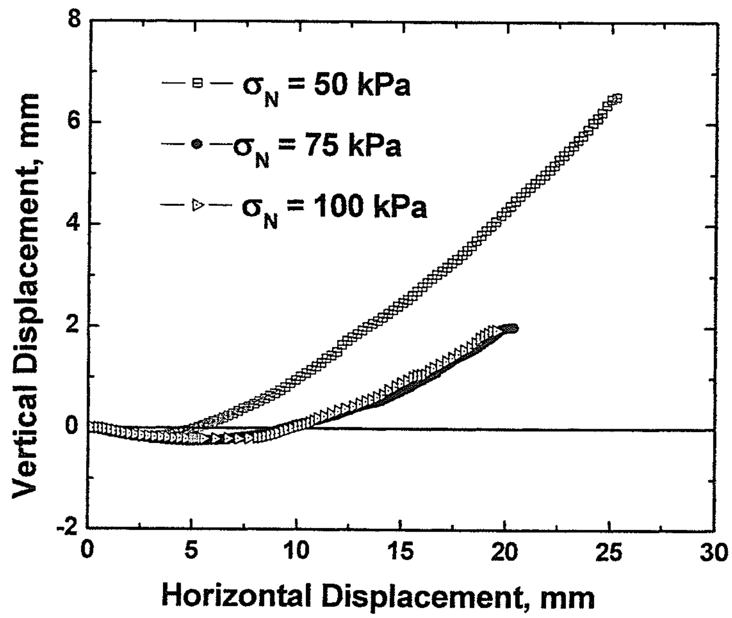


Figure 4.4 Normalized shear strength of loose rockfill materials

Three Linear Variable Displacement Transducers (LVDT's) located at the top cover of the large-scale direct shear test were used to measure the dilatancy. The vertical displacement (dilatancy or contraction as the case maybe) depends on the applied effective normal stress and on the relative density of rockfill soil. Vertical displacements (LVDT #1, #2 and #3 readings) plotted against the horizontal displacement (LVDT #4 readings) from the large-scale direct shear tests for dense and loose rockfill materials are shown in Figures 4.5 and 4.6, respectively. These figures show that the mobilized dilation angles decrease with the increase of the applied normal stress. As the applied normal stress increases, the tendency of the material to dilate decreases, as expected, based on conventional understanding regarding the behaviour of granular media.

The initially dense rockfill sample generally undergoes a very small compression at the start of shearing, but then begins to dilate as shown in Figure 4.5. For loose rockfill material (Figure 4.6), the initial shearing is accompanied by contraction then dilation at larger shear displacements. This observation deviated from the common understanding that for the loose condition the material will contract during shearing until it reaches the critical state where there is constant shear stress and constant volume during shearing. Dilation at large shear displacements even for loose granular materials may be attributed to progressive mobilization of shear for this particular test setup as will be shown later in the next paragraph. The progressive mobilization of shear would somehow result in rearrangement of particles. This rearrangement would lead to a transition from



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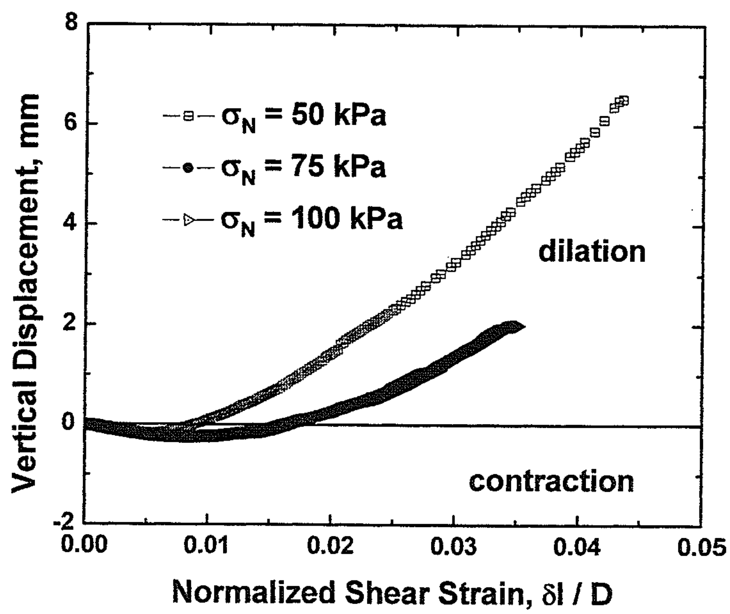
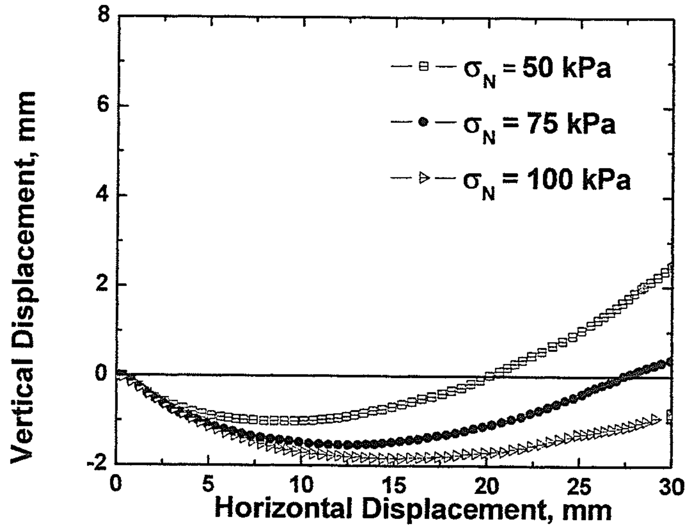


(b)

Figure 4.5 Volume change during shearing of dense rockfill materials (σ_N= 50, 75, 100 kPa), (a) Vertical displacements vs. horizontal displacement, (b) Vertical displacement vs. Normalized shear displacement



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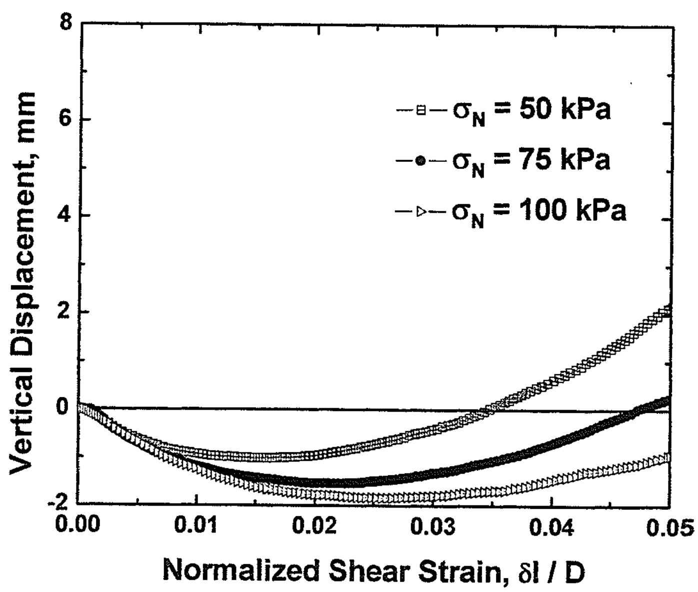
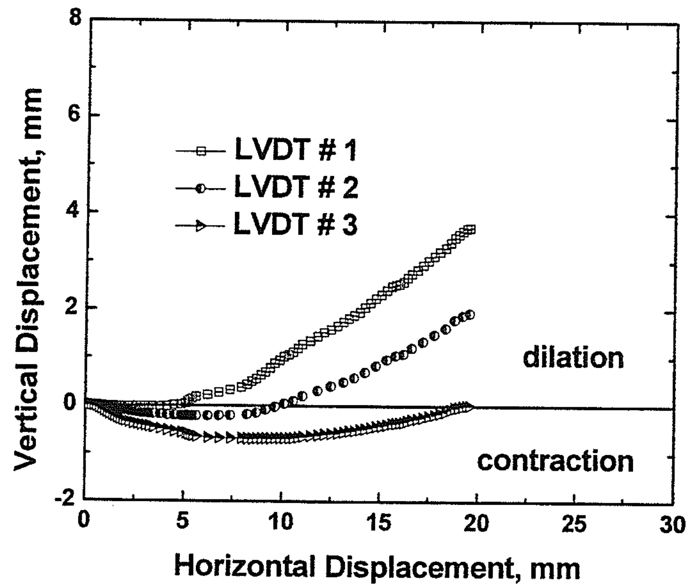


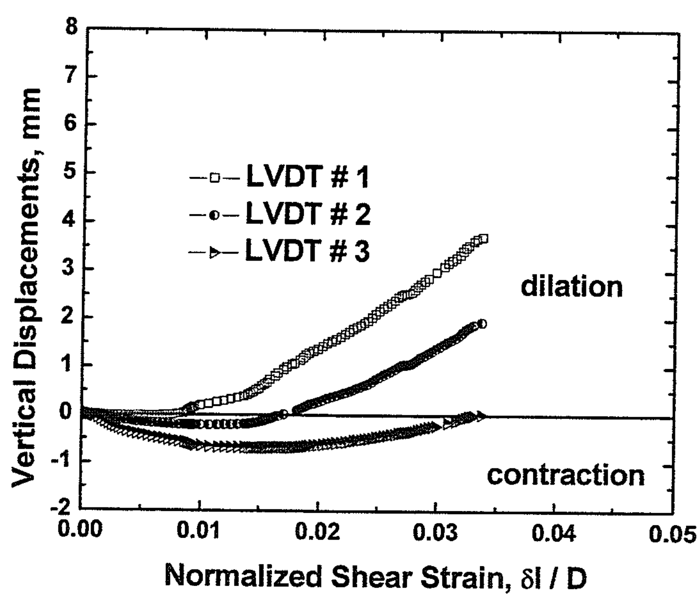
(b)

Figure 4.6 Volume change during shearing of loose rockfill materials (σ_N= 50, 75, 100 kPa), (a) Vertical displacements vs. horizontal displacement, (b) Vertical displacements vs. normalized shear displacement contraction to dilation. This transition period represents minimal volume change during shearing and this can be considered as the period of no volume change (i.e. corresponding to critical state). The measurements of vertical displacements during shearing are therefore important in evaluating the mobilized shear strength of rockfill material (see Section 4.2.3).

The LVDT's #1, #2, and #3 are equally spaced and arranged from the farthest to the closest from the direction of movement. Figure 4.7 reflects progressive mobilization of shear resistance from the back of direct shear apparatus to the front. The vertical displacement of #1 at a given horizontal displacement is higher than that of #2, and the vertical displacement of #3 is the lowest value. This means that there is a progressive mobilization of shear displacement along the length of sheared surface (i.e. from the back of the shear box to the front). The implication of this observation is that the shear mobilization of granular material is higher at a higher point on the slope and the lower point on the slope has lesser shear mobilization than the point above. This can be valuable information that will determine the best arrangement of rockfill columns at the riverbank. This behaviour was similar for both relative density conditions; dense and loose. It is interpreted that the readings of #2 (the middle LVDT) were considered to be representative of the vertical displacement during shearing as an approximate average along the entire length.



(a)



(b)

Figure 4.7 Volume change during shearing of three LVDTs along the upper surface of dense rockfill materials (σ_N= 100 kPa), (a) Vertical displacements vs. horizontal shear displacements (b) Vertical displacements vs. normalized shear displacements

4.2.3 Shear strength of rockfill materials

Figure 4.8 show that the transition friction angle $\phi'_{transition}$ for densely compacted rockfill lies approximately on a straight line of gradient tan $\phi'_{transition}$ that can extend to zero intercept in the (τ, σ_N) space. The transition friction angle is equivalent to the corresponding friction angle at the critical state as defined by Atkinson (1992). The reader is referred to Figure 3.6 with a discussion on Section 3.4 about using the value of $\phi'_{transition}$ as an alternative method for determining $\phi'_{critical}$. This is important as the shear displacement corresponding to the critical state for densely compacted rockfill was found to be unattainable with the test equipment being used. Note that the recommendation by Atkinson (1992) wherein the value of $\phi'_{transition}$ is equivalent to that of the value of $\phi'_{critical}$ has been used in the test results' interpretation in this study. This procedure of interpreting shear test results has also been employed by Graham et al. (2004). Using the approach used by Atkinson (1992), the critical friction angle ($\phi'_{critical}$) using the mobilized shear resistance at a transition point, was determined to be equal to 37° as shown in Figure 4.8.

The shear stress at critical state shown in Figure 4.8 was obtained by selecting the shear stress corresponding to shear displacement at the transition period from contraction to dilation in Figure 4.6. It can be seen in Figure 4.8 that the $\phi'_{transition}$ value for densely compacted rockfill is comparable with the $\phi'_{critical}$ determined for loosely compacted rockfill material. The test results presented here are consistent with the approach used by Atkinson (1992). Moreover, they illustrate the fact that regardless of initial relative density (i.e., densely compacted or loosely compacted), the $\phi'_{critical}$ represents a unique value. This value also represents the conservative value as it is the lowest value of strength (Bolton, 1991).

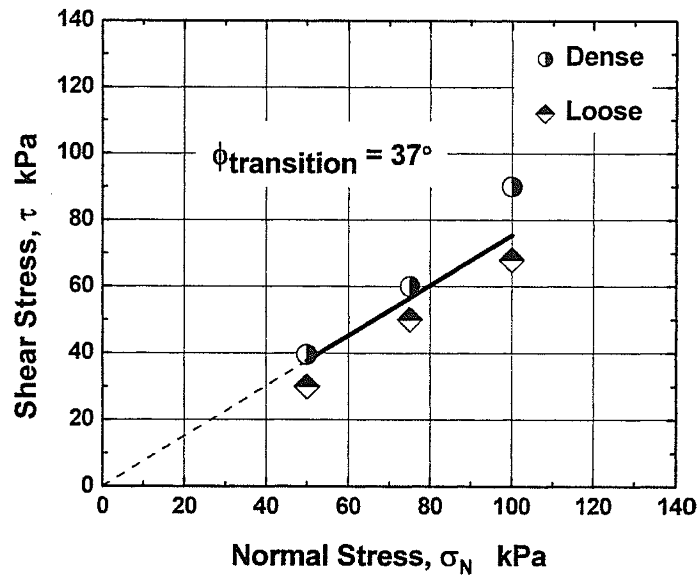


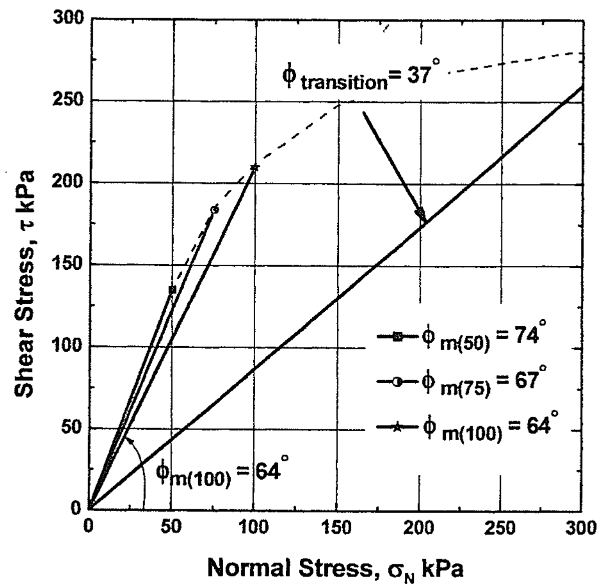
Figure 4.8 Transition friction angle of rockfill soil

There has been an on-going debate on the use of ϕ'_{peak} or $\phi'_{critical}$ in designing geotechnical structures (see Chapter 2). It should be noted that peak strength is considered transient, sustainable only while the soil is dilating (Powrie, 1997). Our laboratory tests show that the peak friction angle is decreasing with increasing effective normal stress as shown in Figure 4.9 (a and b). These figures illustrate that the envelope generated by the peak strength is curved until it approaches the critical state value. Figure 4.9 shows that for a particular rockfill material, the peak strength depends on the initial void ratio (density) and the effective normal stress applied at the plane of shearing. The lower the effective normal stress, the greater the dilation angle and mobilized shear strength. Figure 4.9 also shows that at 50 kPa applied normal stress the corresponding peak friction angle was 74° and this angle dropped to 67° at 75 kPa normal stress. The friction angle reached 64° at 100 kPa before it went into the critical state $\phi'_{critical}$ = 37°. This behaviour is consistent with the general observation for the shear strength of densely compacted granular material as shown in Figure 4.10 (Budhu, 2007).

Observations from our test results indicated that even for initially loose rockfill materials the granular material particles are able to change their packing (rearrangement) during shearing from contraction (negative dilatancy angle) at lower shear displacement to dilation (positive dilatancy angle) at larger shear displacement. The effect of this behaviour can be seen on the curved failure envelope of initially loose density samples shown in Figure 4.9 b.

4.2.4 Reduced-sized rockfill materials

To determine the shear mobilization of composite soil with a particular column diameter using large-scale direct shear test, a testing program has been designed in such a way that insitu rockfill particle sizes were scaled to conduct composite soil tests. This has been done such that the proportion of the size of rockfill particles to the size of the column used in the laboratory is relatively



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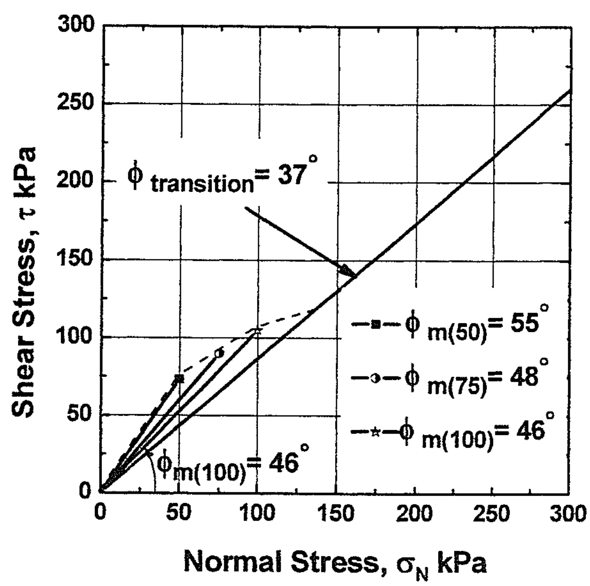

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Figure 4.9 Mobilized friction angles of (a)dense and (b) loose rockfill materials

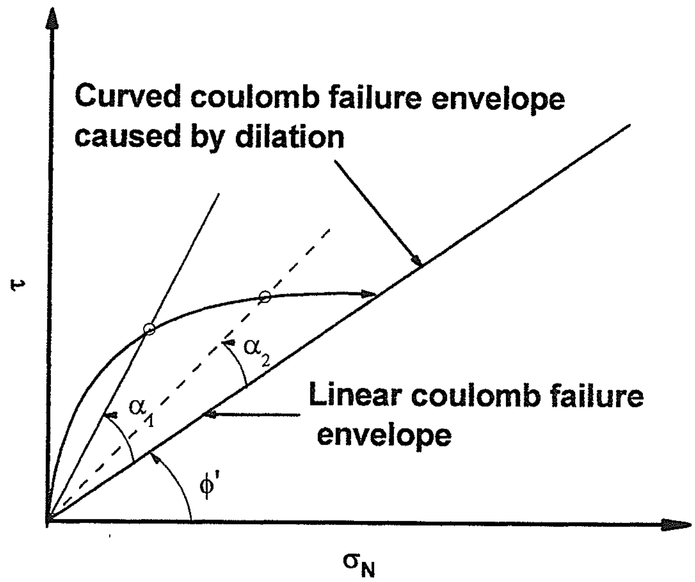


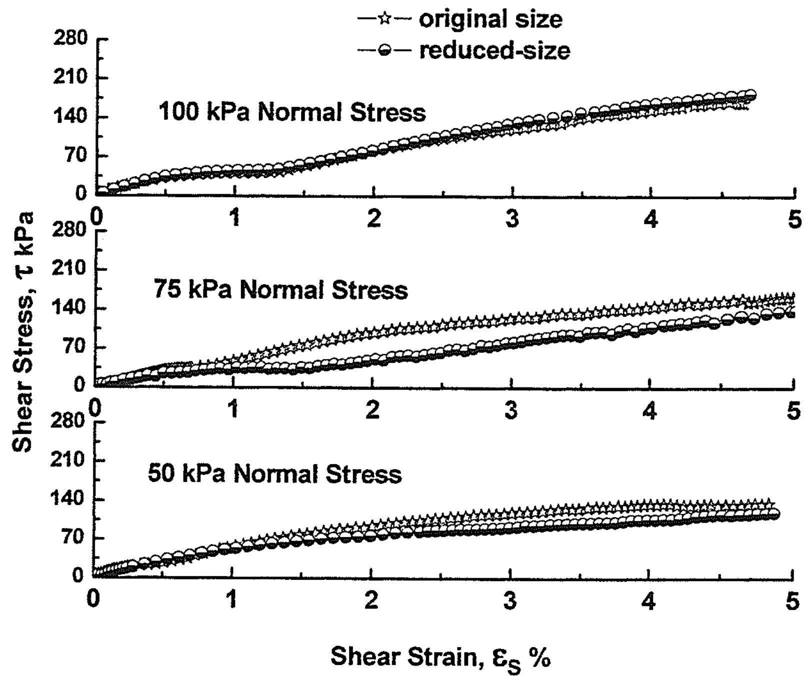
Figure 4.10 Effect of dilation on Coulomb's failure envelope (after Budhu 2007)

similar to that in the field. Therefore, the tests for clay-rockfill composite require reducing the sizes of the rockfill materials to maximum grain diameters of 16.8, 22, 27, and 32 mm corresponding to various sizes of rockfill columns. Furthermore, the mean size and grain size distribution of the rockfill material has to be taken into account to ensure that the effects of reducing particle sizes would have no effect on the test results sought for. Based on theoretical and experimental evidence (Marshal 1973; Gupta et al. 1995; Sitharam 2000) as discussed in Chapter 2, the stress-strain characteristics (and thus the strength) of granular materials are reasonably similar if their gradatsion are parallel. The results of two different grain sizes of parallel gradation of both the original and reduced size to maximum grain diameter 27mm and 60 mm are shown in Figure 4.11. Figure 4.11 shows the stress-strain relationships from reduced-size and original rockfill materials are indeed similar. This will help out to conduct tests on clay-rockfill composite tests.

4.2.5 Stress-Strain characteristics for undisturbed and remolded clay

Large-scale and conventional (small) direct shear tests have been conducted on undisturbed lacustrine Winnipeg clay. As indicated in Chapter 3, the primary purpose of performing large-scale direct shear tests is to determine the mobilization of shear resistance of undisturbed lacustrine soft clay under the same boundary condition and scale effects as the undisturbed clay-rockfill composite test setup. The results of the tests are also used in the determination of strength parameters for numerical analysis. Figure 4.12 shows the stress-strain characteristics of undrained direct shear tests at two applied normal stresses of 50 and 100 kPa. As expected, the results for the two normal stresses are similar given the fact that both were conducted under undrained shear (i.e., effective normal stress does not change with increase in applied normal stress).

As discussed in Section 2.3 (Chapter 2), using peak strengths for brittle soils can lead to inaccurate and unconservative assessments of stability. On the other hand post-peak strengths have been used with success in first-time slides in high-plastic clays (Duncan and Wright, 2005, Graham, 1986, Rivard and Lu, 1978, Lefebvre, 1981). The conventional direct shear test was conducted to determine the effective shear strength parameters under drained conditions. The test results for both peak and post-peak shear strength are presented in



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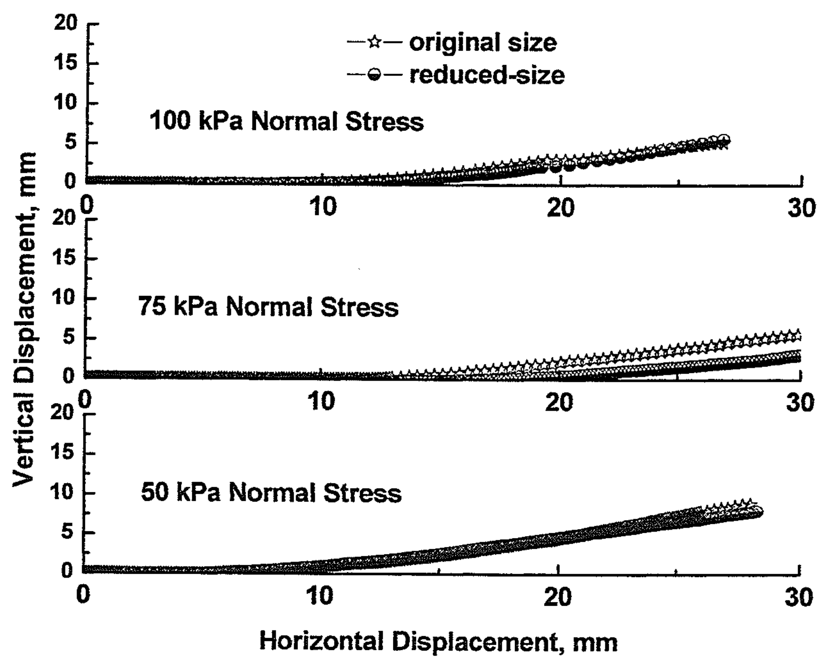


(b)

Figure 4.11 Effect of two grain size distributions on the soil characteristics, (a) Stress-strain characteristics, (b) Volume change during shearing of both materials (maximum sizes of 60 cm and 27 cm)

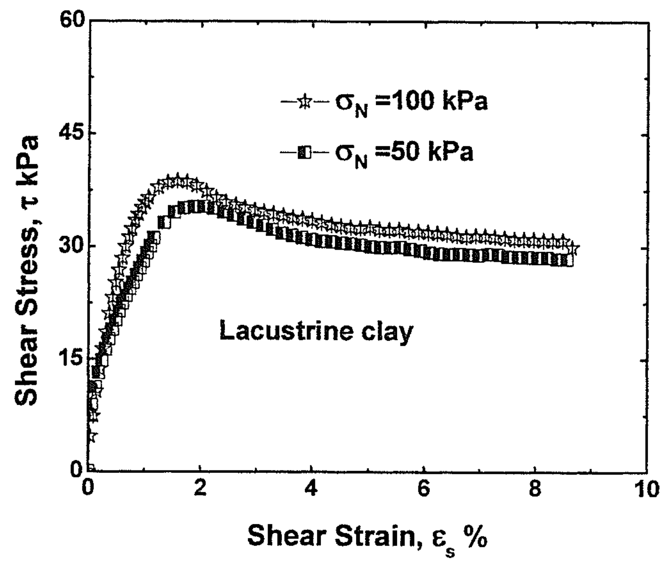


Figure 4.12 Shear stress versus shear strain of undisturbed lacustrine clay from large – scale direct shear tests under undrained condition

Figure 4.13 at normal stresses of 50, 100, and 200 kPa. The measured soil cohesion and angle of internal friction at the peak were 6 kPa and 16° respectively, while the post peak strength parameters (critical state strength parameters) were 4 kPa and 15°. This study uses the post-peak parameters as an input for the FEM analysis which is discussed further in Chapter 6. The importance of using post-peak parameters will be justified in more details in Chapters 6 and 8.

Figure 4.14 shows the residual shear strengths of drained direct shear tests of lacustrine clay. The strength parameters were found to be c'= 9 kPa, ϕ'= 8°. These parameters can be used for the numerical analysis, particularly for cases when the riverbank being stabilized had undergone failures.

The effective strength parameters were determined for peak, post-peak and residual shear strengths are shown in Figure 4.15.

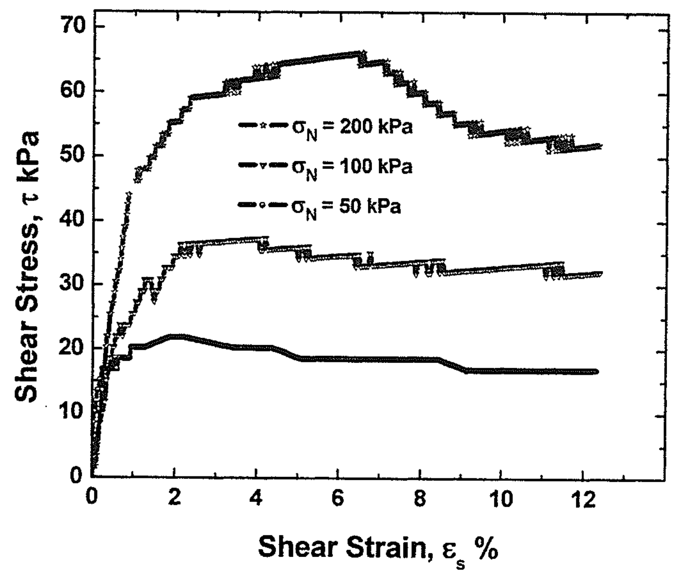


Figure 4.13 Peak and post peak shear strength of drained direct shear tests of undisturbed plain clay

4.3 Clay-Rockfill Composite Behaviour

The clay-rockfill composite material involves installing a single column of 270 mm diameter in undisturbed clay material. The stress-strain characteristics of undisturbed Winnipeg clay with and without rockfill column are shown in Figure 4.16. It is obvious that shear resistance of the clay-rockfill composite did not mobilize until the clay shear strength has been fully mobilized. This means that the shear resistance in the rockfill column will mobilize only at shear strains greater than the strain corresponding to the peak strength of the clay.

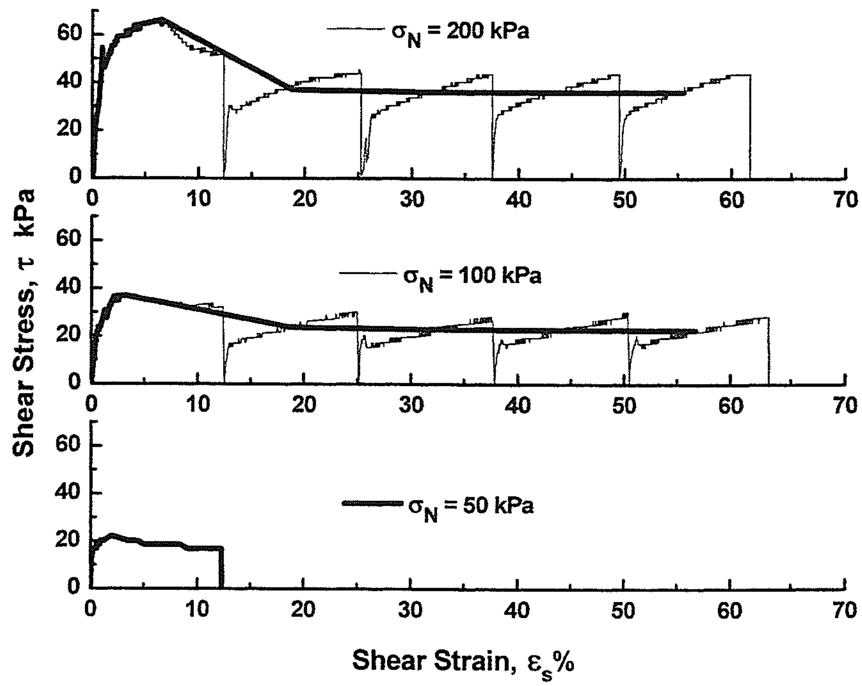


Figure 4.14 Drained direct shear tests of undisturbed lacustrine clay

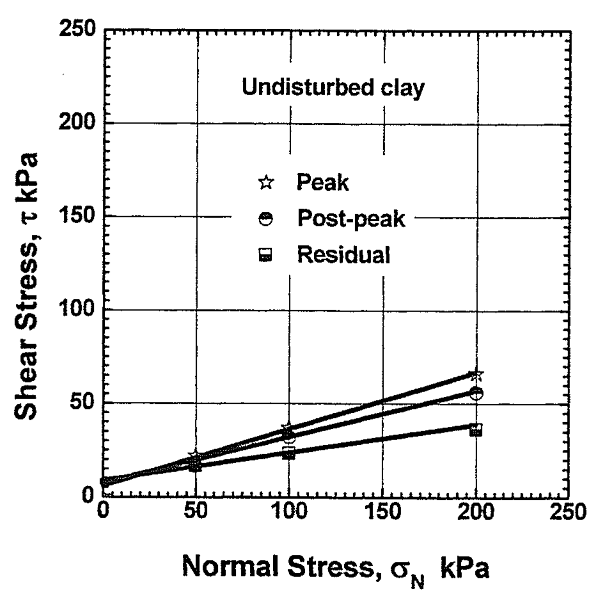


Figure 4.15 Peak strength parameters from drained direct shear tests

4.4 Factors affecting the shear mobilization of rockfill columns

4.4.1 Cementation effects in tests for rockfill materials alone and in clay-rockfill composite

The results of cemented rockfill material show that the shear strength using 0.5% cement content increased significantly as shown in the Figure 4.17. The increase in shear strength is due to the buildup of an internal confinement (bonding) or cohesion. This bond continues until the peak shear strength of cemented rockfill is attained. However, shear resistance started to decrease at larger shear strain and converged to that of the untreated rockfill material. In other words, when the peak shear strength of the cemented rockfill is attained, the intergranular cementation bonding starts to breakdown and the residual shearing strength approaches that of uncemented rockfill material (Juran and Riccobono 1991). This means that the low level cementation (0.5% cement content by dry weight of rockfill) is effective only at small shear strains and cannot be relied on at shear strains exceeding 6% strain. The shear stiffness of the cemented rockfill material at relatively small shear strain was up to twice that of the untreated rockfill material. It should be noted that intergranular cementation bonding has no effect on the friction angle, however it improves the cohesion intercept significantly (Balmer 1958, Dupas and Pecker 1979, Red and Clough 1982, Acar and El-Tahir 1986, Saxena et al. 1988, Juran and Riccobono 1991).

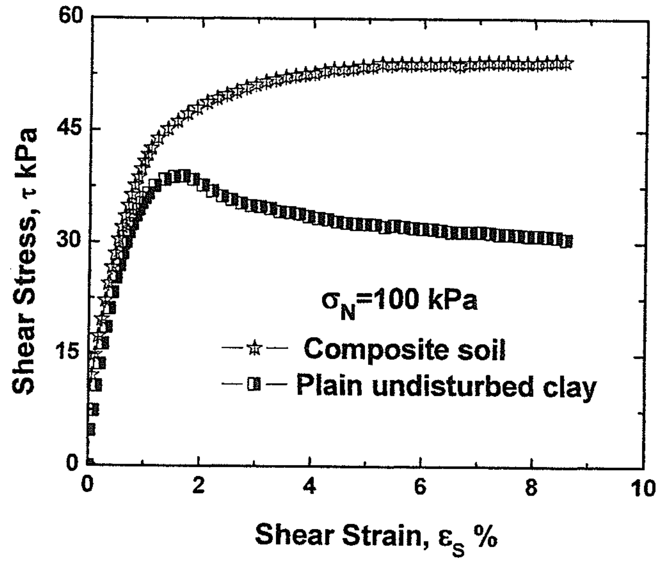
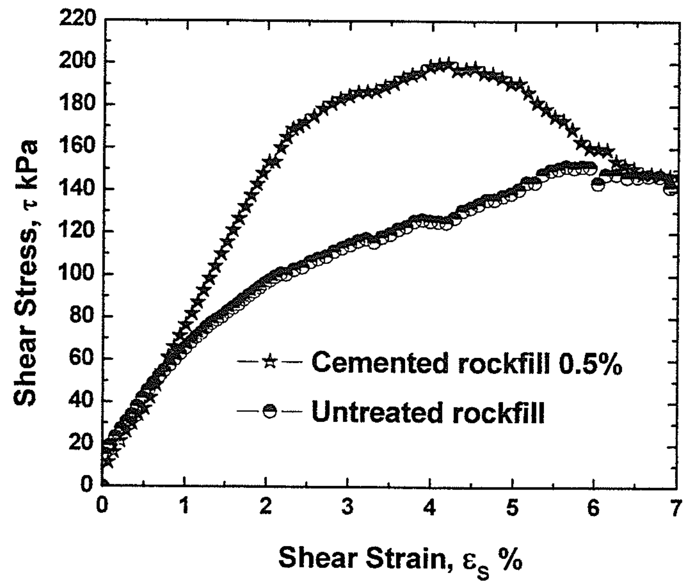


Figure 4.16 Shear stress vs. shear strain of undisturbed Winnipeg Clay with and without rockfill column at 100 kPa normal stress

The stress-strain characteristics of cemented soil-rockfill composite columns are shown in Figure 4.18 at 50 kPa applied normal stress. Two cement contents, namely: 2% and 5% were added to the rockfill column and the results of the shear tests were compared to that of the uncemented column. The results indicate that adding cement up to 5% by weight of the rockfill material leads to little increase in the shear stiffness of the composite material. Furthermore, the treatment has no effect on the ultimate shear strength of the composite. The most likely reason for the insignificant effect of cementation is the change in the mode of failure between the composite material with and without cementation. Without cementation in the rockfill material, the mode of failure is direct shearing through the clay and rockfill materials. With cementation, the dominating mode of



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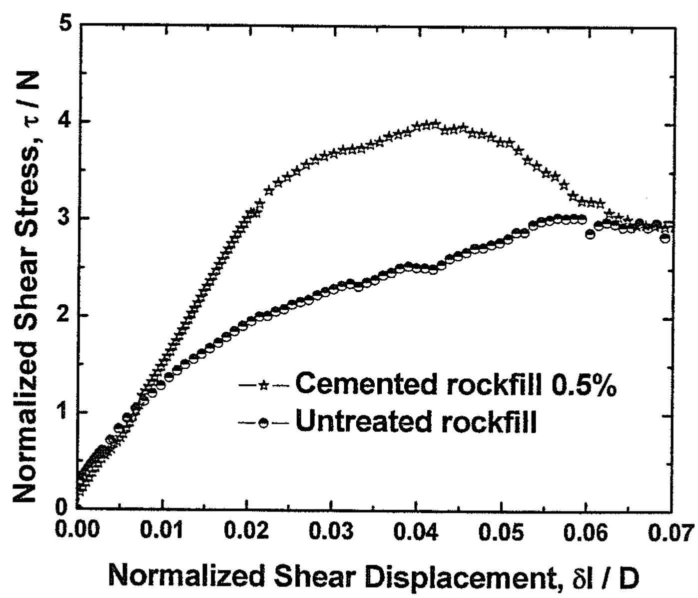
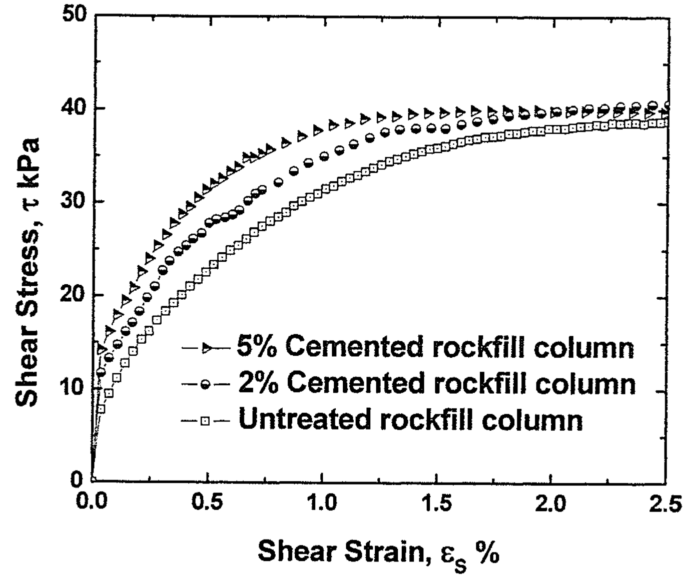


(b)

Figure 4.17 Shear mobilization of cemented rockfill materials, (a) shear stress vs. shear strain, (b) normalized shear load vs normalized shear displacement failure is a passive-type of failure which is governed by the shear strength of clay and is very much less than the shear strength of the rockfill. This mechanism allows the clay to move around the much stronger column material, similar to pile-clay interaction. This was verified by visual inspection of the failure mode after the tests as shown in Figure 4.19 where the clay moved around the intact cemented rockfill column. The implication of this finding is that the cemented rockfill columns should be arranged close enough to act as a group, while at the same time do not allow clay to move between the columns during slope movements.

It is apparent that adding low level cementation to rockfill materials has increased the particles' bonding, as a result the cohesion between the granular particles increased and enhanced the mobilization of shear resistance of rockfill columns. As will be shown later in Chapter 6, the back row of rockfill columns will experience larger shear displacement than lower level rows in a stabilized riverbank. Therefore increasing the shear stiffness by cementation may enhance the stability due to higher shear stiffness. This technique of providing high level cementation filling in the voids of the rockfill may also be recommended for cases where there is a possibility of silty soil infiltration (Chapter 6) inside the rockfill columns. This is the case in which aquifer pressure stimulates silt particles (inside the lenses of silty soils) surrounding the columns to migrate into the rockfill columns.



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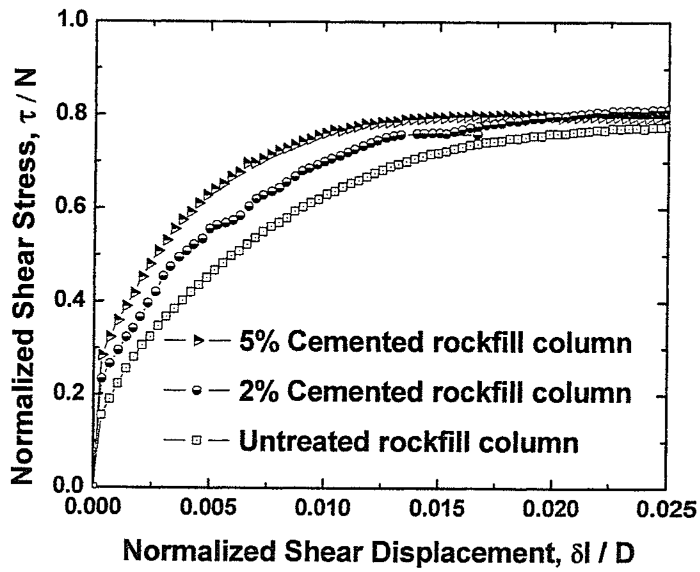
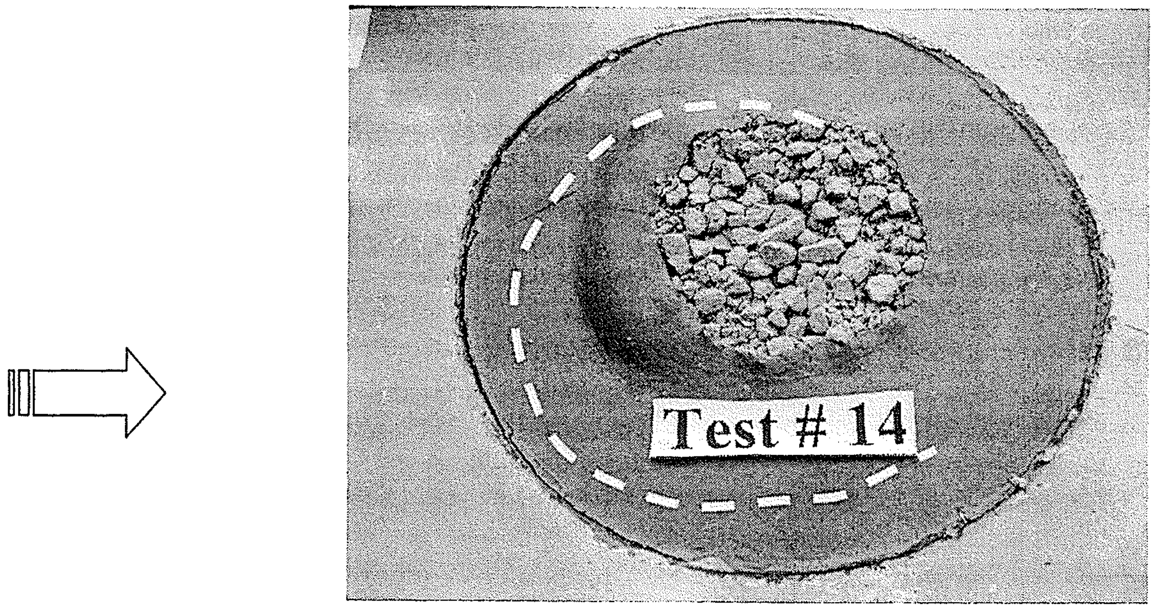


(b)

Figure 4.18 Shear mobilization of composite soil using cemented rockfill columns at 50 kPa normal stress, (a) shear stress vs shear strain, (b) normalized shear load vs normalized shear displacement



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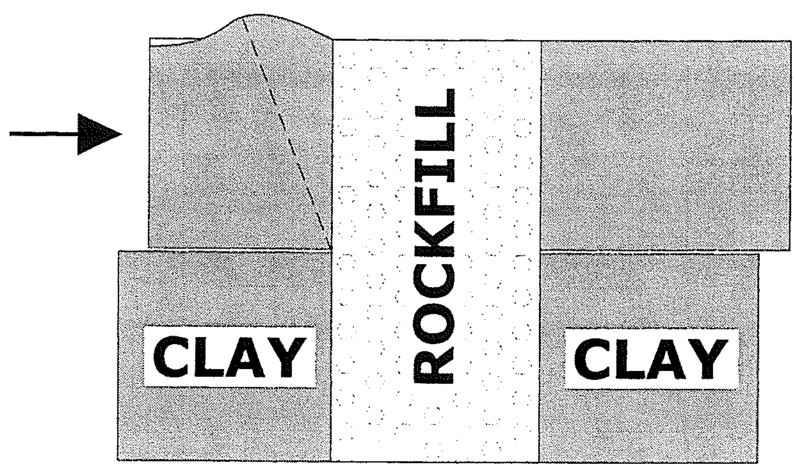


(b)

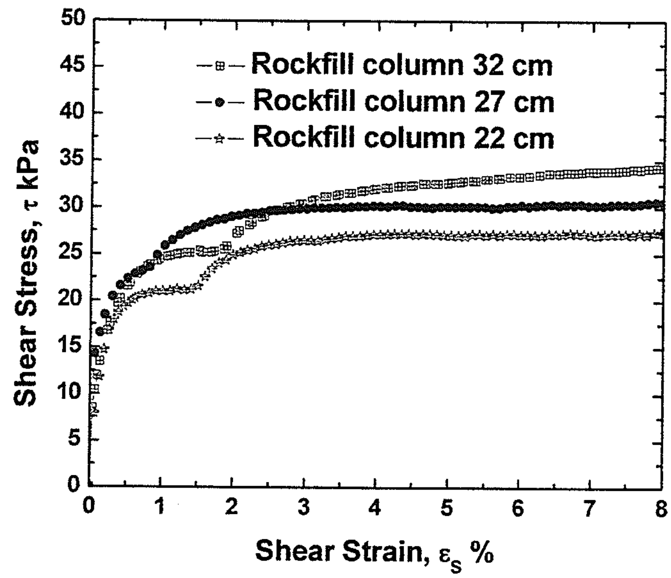
Figure 4.19 Mobilized shear strength of cemented undisturbed sample (2% cement ratio) by a passive failure, (a) Photo after test, (b) Conceptualized failure mode

4.4.2 Effects of various area replacement ratios

Tests were also performed to investigate the effects of the size of the rockfill column relative to the spacing. Due to the limited amount of large undisturbed clay samples, remolded soils have been used in conjunction with the rockfill materials to form a composite material. The results show that increasing the area replacement ratio increases the mobilized shear resistance of reinforced soils as expected. However, increasing area ratios did not increase the shear stiffness of the composite material particularly at smaller shear strain as shown in Figure 4.20 corresponding to the area replacement ratios of 14% (column diameter = 220 mm, with clay diameter of 600 mm), 22% (270 mm), and 30% (320 mm).

4.5 Behaviour of rockfill columns in group, shear key and ribbed-typed layouts

These tests examine the behaviour of remolded lacustrine clay stabilized by a group of rockfill columns and other layouts of rockfill material used for stabilization of riverbanks in Winnipeg. The primary aim of these tests is to investigate the influence of different layouts on the performance of stabilized riverbanks. Two tests were conducted using the same number of columns, column dimensions, and method of installation, but with different row location as shown in Figure 3.17 (Chapter 3). These two layouts will further confirm the progressive mobilization of shear resistance in rockfill columns as discussed in



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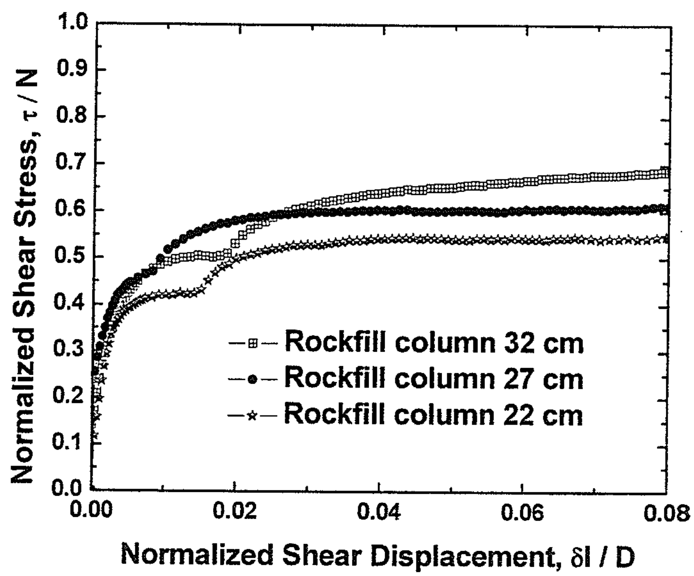


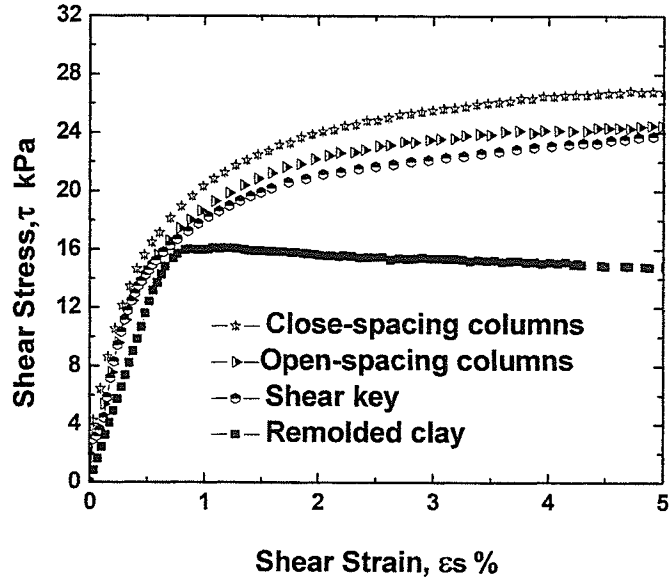
(b)

Figure 4.20 Mobilized shear resistance of clay-column composite at various area replacement ratios under σ_N = 50 kPa: (a) shear stress vs shear strain, (b) normalized shear load versus normalized shear displacement

Section 4.2.2. Figure 4.21 shows the comparison between the performances of two column spacings together with the performance of the shear key layout.

It can be seen that the shear mobilization of open spacing group columns is very similar to that of the shear key, although the replacement ratio for column groups was less than 60% compared to that of the shear key. The closely spaced column has greater shear resistance than the single shear key. This could be explained by the fact that unsupported trench excavation associated with shear key installations can lead to expansion of the surrounding soil and eventually reduction in strength particularly for high plastic clays with high overconsolidation. Duncan and Wright (2005) indicated that overconsolidated clays (as in the case of compacted clay in our test set-up) and stiff-fissured clays have brittle stress-strain characteristics, and they exert high horizontal stresses, often higher than the vertical stresses. When these materials are excavated, slopes rebound horizontally (expansion) causing weakening of the clay. On the other hand, column group installation could have lesser expansion compared to that of the shear key installation and better still when steel casings is used in the installation of rockfill columns as they support the surrounding native soil from lateral movements.

The comparison between the performance of shear key and ribbed-type layouts demonstrates that the ribbed-type has much greater shear strength than the shear key as shown in Figure 4.22. This is due to the fact that mobilization of shear resistance in shear key did not start until the full mobilization of shear



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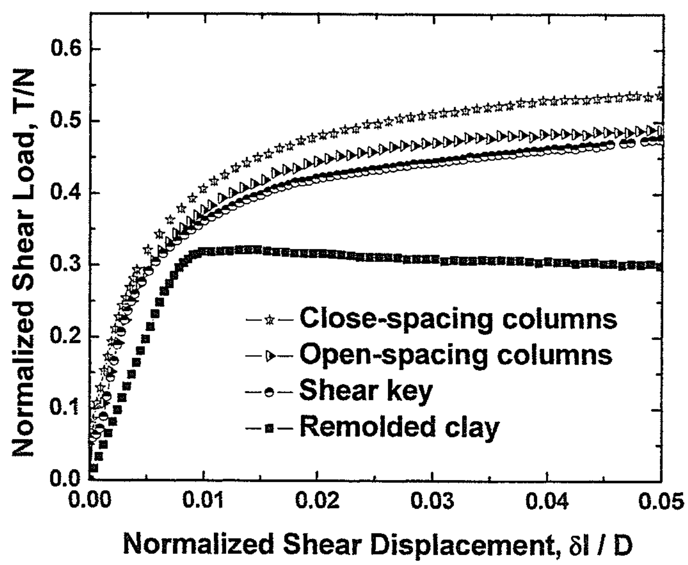

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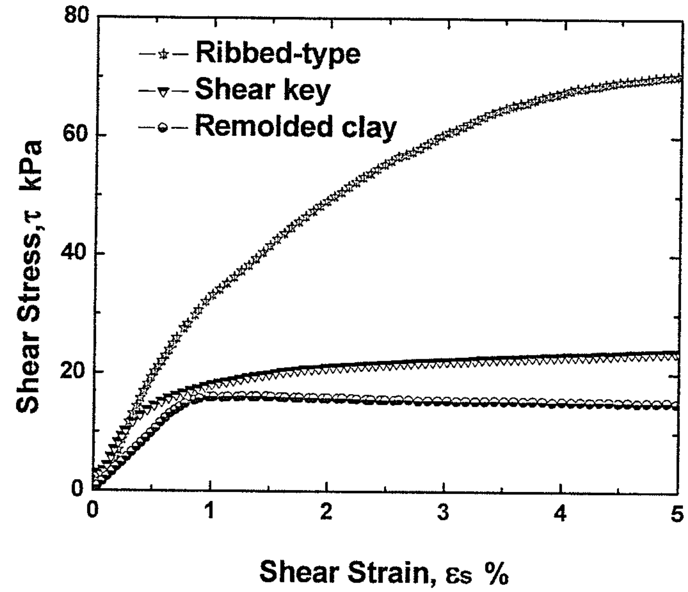
Figure 4.21 Shear mobilization of column groups for both open and close spacing, (a) shear stress vs shear strain, (b) normalized shear load vs normalized shear displacement



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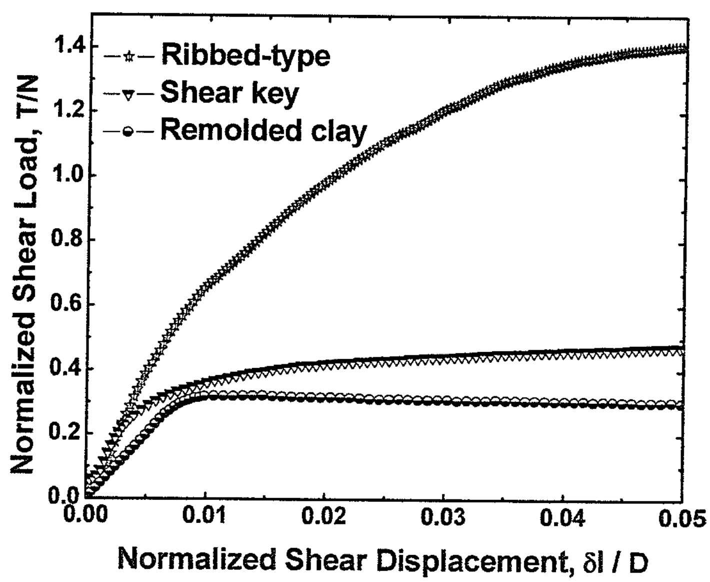

(b)

Figure 4.22 Shear mobilization of two techniques shear key and ribbed-type, (a) shear stress vs shear strain, (b) normalized shear load vs normalized shear displacement resistance in the clay. In contrast, the shear resistance of rockfill materials in ribbed-type layout mobilized right away as the composite material is sheared.

This finding has important implication in designing stabilization measures for riverbanks using rockfill materials. If rockfill columns are used, it is recommended that groups of columns will also be installed perpendicular to the length of the riverbanks in addition to those installed along the riverbanks as has been commonly done in local practice.

4.6 Summary

The results obtained in this chapter can be summarized as follows:

1. Results obtained from large direct shear tests of rockfill materials showed higher shear resistance mobilized at higher applied normal stress for both dense and loose samples.
2. Shear resistance of dense samples was almost double its value compared to that in the loose condition at the same normal stress.
3. Laboratory results demonstrated that the peak strength depends on the initial void ratio and the effective normal stress applied at the plane of shearing. As for deep-seated failure plane, higher effective normal stress is developed at the failure plane, and consequently a larger mobilized shear strength is generated.

4. There was improvement in shear stiffness of dense rockfill compared to the loose densities. Shear stiffness was about four times the corresponding value for the loose rockfill
5. Movements are expected to mobilize the shear resistance of rockfill columns. It should be noted that the density achieved during installation of rockfill columns has a significant impact on the movements required to mobilize the shearing resistance.
6. The mobilization of shear resistance is a function of volume change during shear, which in turn depends on the density of the rockfill materials.
7. Observations indicated that dense rockfill materials may undergo a very small compression at the start of shearing, but they begin to dilate during shearing. It should be noted that lesser applied normal stress leads to higher dilation values. On the other hand, loose rockfill materials exhibited contraction behaviour, except during large shear displacements, when the samples demonstrated dilation behaviour.
8. Volume changes during shearing exhibited progressive mobilization of shear resistance from the back of the direct shear apparatus to the front during monitoring of three LVDTs located on top of rockfill sample. This finding is significant and important in designing the rockfill column layout through riverbanks which will be proven in Chapter 6.
9. Critical state friction angle was estimated using the Atkinson (1992) method. Atkinson (1992) proved that the transition friction angle

calculated at low shear strain is equivalent to the critical state friction angle calculated at larger shear strains. This value was equal to 37° for the rockfill materials used in this study.

10. Results of undisturbed lacustrine direct shear tests at various applied normal stresses indicated that the cohesion can be taken as 6 kPa and friction angle as 16°. However, the corresponding post peak strength parameters were 4 kPa and 15° respectively.
11. Stress-strain characteristics of composite clay-rockfill samples revealed significant improvement in shear strength compared to undisturbed natural clay samples. The other observation was that there are some movements required to mobilize shear resistance in the rockfill column.
12. Cemented rockfill materials using low level cementation exhibited significant improvement in shear strength at low shear strain. However, the shear strength converged to that of the untreated rockfill materials at larger shear strains.
13. Composite clay-cemented/rockfill samples (i.e. rigid columns) showed little increase in shear strength compared to unreinforced samples. This may be due to change in the failure mechanism from direct shear to passive-type of failure.
14. Increasing the area replacement ratio increases the mobilized shear resistance of the composite samples.

Chapter 5

SOIL-CEMENT MIXING

5.1 Introduction

Soil-cement columns can be used to improve the overall shear strength of the soil mass, which can increase the factor of safety by forcing the critical slip surface to greater depth (Andromalos et al. 2000). This method has proven to be more effective over rockfill columns for stabilization of natural slopes in Columbia Slough, USA, where silt intrusion into the rockfill columns caused poor performance (Dailer and Yang 2002). Similar conclusions on the effectiveness of soil-cement columns in stabilizing soft clays in natural slopes to improve strength and deformation parameters have previously been reported by Christensen and Nordal (1999).

Soil-cement characteristics must be based on the treated soil for each particular soil type under natural conditions, specifically natural water content and Atterberg limits. The clay mineralogy and soil consistency may also influence the behaviour of the soil-cement mixture (Babasaki et al. 1997, Broms and Anttikoski 1984, Esrig 1999, Horpibulsuk and Rachan 2003, Rachan and Horpibulsuk 2003).

There was no information available regarding strength characteristics of soft high plastic clays in Manitoba treated with cement, although Ganapathy (1981) studied the mechanical behaviour at Winnipeg clays stabilized with lime. Therefore, this study was carried out to include investigation of the behaviour of soil-cement columns in high plastic clays as a structural element interacting with the surrounding native soils to support unstable riverbanks. The data will be used to evaluate the stability of stabilized riverbanks in Winnipeg using soil cement columns.

5.2 History of Soil-Cement Treatment

Historical evidence indicates that the soil mixing method was developed and put into practice during Babylon civilization (2000 BC). Soil mixed in-place with lime and fly-ash was used to increase stability of embankments. Soil mixing as a ground improvement method was used in 1950 in the United States by mixing cement grout with soil. The use of this technique was rare until another documented case in 1991 (Andromalos et al. 2000).

Since 1969 Japanese and Scandinavians have used soil-cement columns or Deep Mixing Method (DMM) to mitigate liquefaction, to construct retaining walls and to increase bearing capacity. The use of soil-cement columns for the stability improvement of embankments and natural slopes has only recently been started.

5.3 Cost-effectiveness

Soil-cement columns can be successfully installed up to 40-70 meters below ground surface depending upon the location and can be applied for most soil types and subsurface conditions (FHWA 2001). FHWA (2001) also reported that soil-cement columns can be applied for a wide range of soils from plastic clays to sand and gravel with cobbles. On the other hand, the rockfill column technique for ground improvement has a number of limitations such as depth of columns, subsurface conditions, installation patterns, and native soil strength.

There are many reasons why placing soil-cement columns can be more effective than rockfill columns for stabilizing embankments or slopes. Soil-cement column installation techniques eliminate the need for excavation and disposal of native soils, as well as the need to import good quality granular materials by utilizing the existing native soil. The drilling equipment used for deep soil mixing does not generate noise or vibration making the soil-cement columns as the preferred alternative for use in environmentally restricted areas (FHWA 2001). Soil-cement columns can have more efficient performance than other techniques because lateral support is maintained during installation (Andromalos et al. 2000).

Several authors reported benefits of using soil-cement columns over other techniques of ground improvement. Kivelő and Broms (1999) stated that soil-cement columns have economic benefits compared to the other conventional slope stabilization methods such as piles or soil replacement. Ribs of soil-cement columns proved to be cost competitive in comparison to traditional solutions in Norway (Watn et al. 1999). Soil-cement mixing has been more economical and has proven to achieve higher performance when compared to other traditional soil stabilization methods, such as rockfill columns, wick drains/preloading and conventional retaining walls (Andromalos et al. 2000). At the moment, installation of soil-cement columns is almost four times the cost of installing rockfill columns in Winnipeg. The difference in the cost of installation may be reduced when equipment for installing soil-cement columns becomes locally available.

5.4 Hardening mechanism of ordinary cement agent

Ordinary Portland cement manufactured by mixing gypsum with cement clinker is formed of minerals such as C3S, C2S, C3A & C4AF. These minerals react with water (H_2O):

[5.1] $C3S + 2(3CaO.SiO_2) + 6H_2O \rightarrow 3CaO.\ 2SiO_2.\ 3H_2O + 3Ca(OH)_2$

A product of cement hydration is calcium hydroxide, which provides strength. This high strength is a function of time; the greater the time the higher the strength. While the calcium hydroxide $Ca(OH)_2$ contributes to the pozzolanic reaction (Babasaki et al. 1997). It should be noted that pozzolanic reaction in the presence of calcium oxide, CaO, and water produces reaction products that are cementitious in nature. This creates denser soil resulting in increase in strength and reduction in permeability.

5.5 Shear strength and shear stiffness

Literature review was conducted to study the factors influencing the strength characteristics of soil-cement stabilised slopes, embankments and foundations, and to gain more detailed understanding of improved soil-cement mixing behaviour. Many articles have been reported regarding deep mixing method applications, most of those were published in Japan, Scandinavia, and in the United States.

Soil-cement columns have been successfully applied to improve slope stability by improving the overall shear strength of the soil mass. This improvement can force the critical potential slip surface to be deeper in the ground, which increases the overall factor of safety against slope failure. In order to design columnar inclusions of soil-cement, strength characteristics such as unconfined compressive strength and Young's modulus of elasticity need to be determined or estimated for both the native soil and soil-cement mixture (Andromalos et al. 2000). Andromalos et al. reported that soil-cement mixtures should be designed to achieve the required shear strength and factor of safety increase against slope failure.

Many factors influence the degree of improvement of soil-cement or soil-lime mixing. Babasaki et al. (1997) reported four categories of factors affecting improvement, as shown in Table 5.1.

Table 5.1 Factors influencing degree of improvement (after Babasaki et al. 1997)

I	Characteristics of hardening agent	Type Quality Mixing water and additives
II	Characteristics and conditions of soil	Characteristics of soil. Organic content pH of pore water Water content
III	Mixing conditions	Degree of mixing Period of mixing/remixing Quality of hardening agent
IIII	Curing conditions	Temperature Curing Time Humidity, repeated drying/ freezing Thawing etc.

FHWA (2001a) stated that the compressive shear strength properties of soil-cement mixture obtained from unconfined compression tests should be in the range of 10-50 times the compressive shear strength of the undisturbed native clays as shown in Equation 5.2:

[5.2] $C_{u_{col}} = 10 \text{ to } 50\, C_{u_{soil}}$

The initial tangent Young's modulus of elasticity of soil-cement mixture should be in the range 50 to 200 times the unconfined compressive strength of treated soils:

[5.3] $E_{col} = 50 \text{ to } 200\, q_{u_{col}}$

FHWA (2001a) also reported that the shear strength of mixed soils is equal to one half of their unconfined compressive strength:

$$[5.4] \quad \tau_{col} \approx 0.5\, C_{u_{col}}$$

The Young's modulus of elasticity of composite ground or stabilised foundation is a function of the modulus of the soil-cement column and the modulus of the native soil around the unit, and depends on the pattern of columnar inclusions:

$$[5.5] \quad E_{composite} = E_{clay}\,(1\text{-}a_r) + E_{col}\; a_r$$

FHWA reported that the composite ground (stabilized ground) properties are a function of the area replacement ratio (a_r), which is the ratio of the area in plan of the treated soil to the total soil mass. This ratio generally ranges between 20% and 35% for the case of soil-cement columns.

The basic mechanism of improvement of the deep mixing method is that by adding hardening agents (cement), the soil strength and stiffness is increased. For proper design of soil-cement mixtures, the testing of laboratory samples prepared by remoulding representative soil with hardening agents (lime or cement) at different ratios should be performed (Watn et al. 1999).

Generally, the literature agreed that the unconfined compression test is an acceptable laboratory test to evaluate the strength and modulus of elasticity of the particular soil and the hardening agents in a laboratory-mixed composite sample. Literature stated that with this test, it is possible to assess the stabilizing effect and by what degrees the mechanical properties of native soils are improved by hardening agents for each particular soil-cement ratio.

Lahtinen and Kujala (1990) showed that the stress-strain relationships for lime, lime/cement and cement stabilized soil can be evaluated by compression tests as shown in Figure 5.1.

Kivelő and Broms (1999) stated that the undrained shear strength failure criterion for the lime/cement columns is composed of two stages. Within the first stage the normal stress lies below the critical normal stress, and the undrained shear strength is composed of both cohesion and friction. The second stage covers the zone of normal stress equal and above the critical normal stress, where the undrained shear strength is governed by the cohesion only, as shown in Figure 5.2.

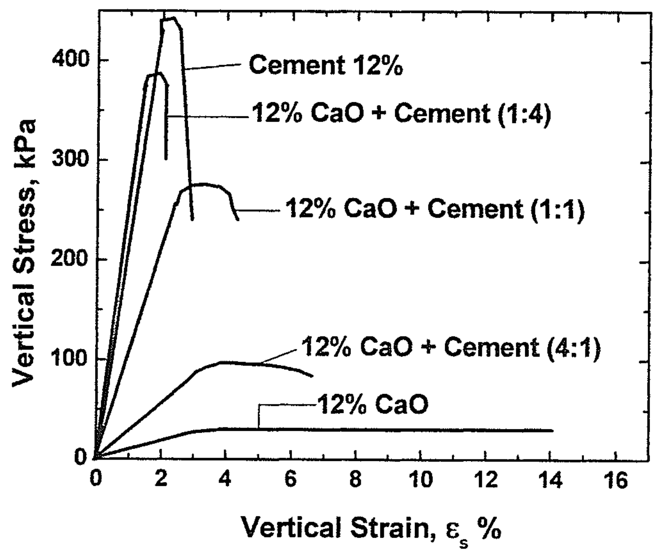

Figure 5.1 Stress-strain characteristics for lime, lime/cement, and cement treated soil (after Lahtinen & Kujala 1990)

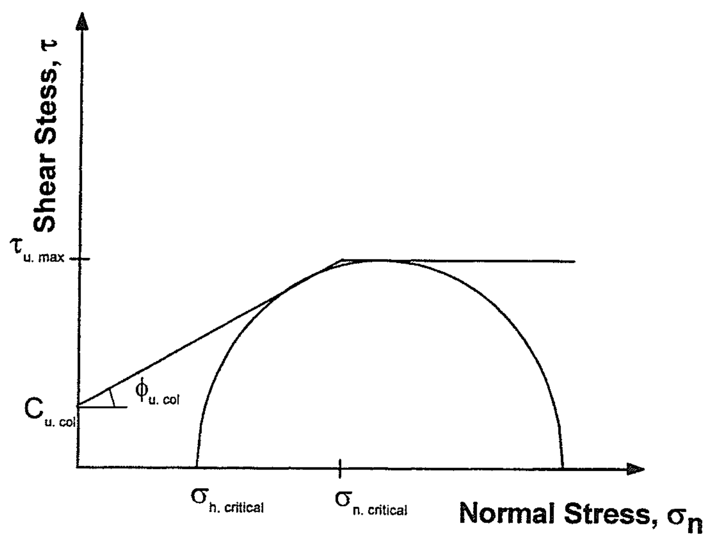

Figure 5.2 Undrained shear strength of lime/cement columns (after Kivelő and Broms 1999)

Kivelő and Broms (1999) further stated that the undrained short term shear resistance (S_a) of the soil-cement column along the potential failure surface can be evaluated as a function of the normal stress and the friction angle at the slip surface location:

$$[5.6] \quad S_a = \frac{\pi d^2}{4\cos\alpha} C_{u_{col}} + N \ \tan\phi_{u_{col}}$$

where, d= the column diameter, metre,

α = the angle between the slip surface and the horizontal, degrees,

N = the normal load in the column along the slip failure, kPa,

$C_{u_{col}}$ = cohesion, undrained shear strength of soil-cement mixture, kPa,

$\phi_{u_{col}}$ = the undrained angle of internal friction of the column material, degrees.

Core samples obtained from soil-cement columns in the field are also required to evaluate the laboratory soil mixture characteristics. Dailer and Yang (2002) evaluated the strength of core samples obtained from hardened soil-cement columns. The laboratory strength results ranged between 2.4 MPa to 9.0 MPa, with an average strength of 5.4 MPa, as shown in Figure 5.3.

FHWA (2001a) reported that the mechanical engineering properties that can be achieved by soil-cement columns is a function of the native soil (cohesive or granular), water content, and the soil chemistry. The range of values for both unconfined compressive strength (q_u) and Young's modulus of elasticity (E) of cohesive soils is given below:

[5.7] q_u = 200 to 3000 kPa

[5.8] E = 350 to 1000 *q_u (for laboratory samples)

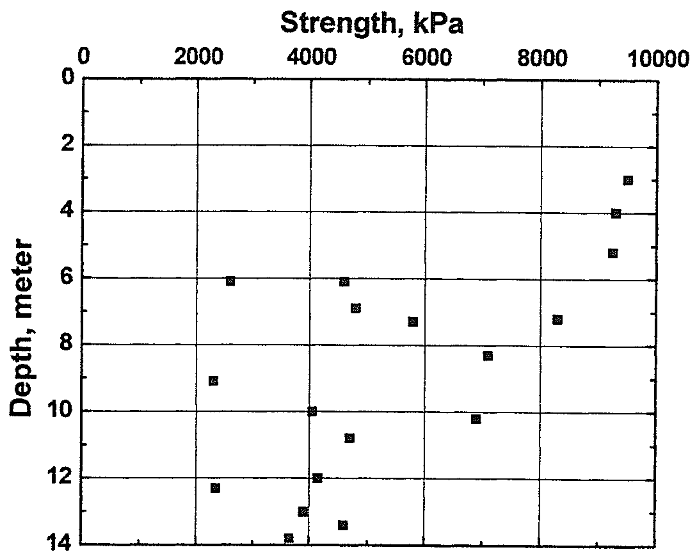

Figure 5.3 Strength of cemented columns obtained from core samples (after Dailer and Yang 2002)

Horpibulsuk and Rachan (2003) investigated the influence of clay-water/cement ratio (w_c/C) on the stress-strain relationship of cement admixed clay. Figure 5.4 shows that lower w_c/C ratios achieve the greatest undrained strength. They stated also that clay-cement mixtures having the same w_c/C ratio exhibit the same undrained shear behaviour.

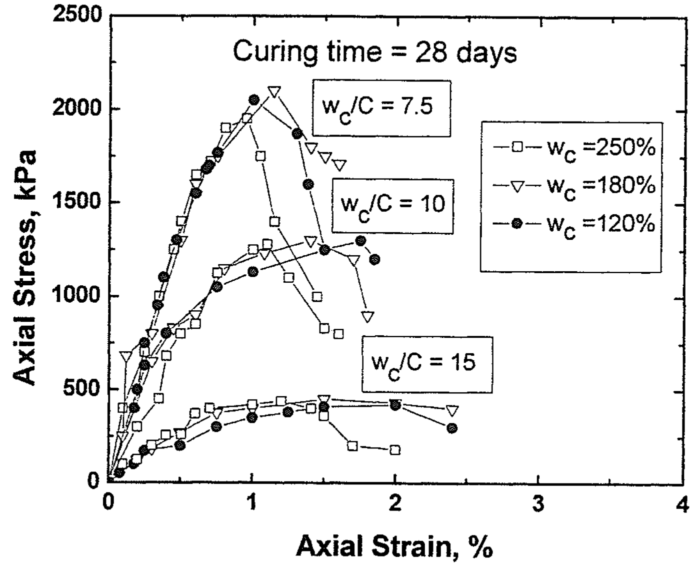

Figure 5.4 Stress- strain relationships of cement treated samples using unconfined compression testes (after Horpibulsuk and Rachan 2003)

5.6 Laboratory Tests

Laboratory tests were carried out on standard size test specimens of soil-cement mixtures to study their mechanical properties. As mentioned in Section 5.3, unconfined compression and unconsolidated undrained (UU) triaxial tests were conducted on laboratory prepared soil-cement specimens. The strength characteristics determined from these testing programs were used in numerical analyses performed to assess the overall performance of the soil-cement columns to stabilize unstable riverbanks in Winnipeg, presented later in Chapter 6.

The first stage of the laboratory testing program included performing unconfined compression tests on laboratory samples prepared by remoulding representative soil with cement at different soil/cement ratios. The objective of this stage was to determine the relationship between the soil/cement ratio and the resulting shear strength and stiffness. The second stage of the testing program involved conducting unconfined compression and unconsolidated undrained (UU) triaxial tests on soil-cement samples of equal soil/cement ratios, while varying other factors such as curing period, water content added during mixing, and the applied vertical stress during the curing period.

5.6.1 Unconfined compressive strength and sample preparation

Unconfined compression tests were conducted using triaxial test apparatus by setting the confining pressure equal to the atmospheric pressure, corresponding to a cell pressure equal to zero.

As mentioned previously in this chapter and in Section 2.7, the unconfined compression test is the most popular test that has been used in the literature to obtain the undrained shear strength of the cement-treated soils.

The laboratory testing program was conducted on soil-cement samples prepared in the laboratory by mixing high plastic clay soil at its natural water content with cement. The soil tested was obtained from 12-15 meters depth below ground surface (Section 3.3). The hardening agent used was ordinary Portland cement, which was mixed with the soil at ratios of 10%, 12%, 14%, 16%, 18%, and 20% by mass.

In order to study the influence of soil/cement ratio, other variables such as mixing conditions, curing time, curing temperature, and water/cement ratio were maintained constant. For easier handling of cement and to improve the degree of mixing, cement was first mixed with water to form a slurry and was then added to the remolded soil (Babasaki et al. 1997). The mixing water quantity and the type of additives are both significant factors in the performance of soil-cement columns. This part of the study the water/cement ratio used was 100% for workability reasons. This ratio have been also used elsewhere (Babasaki et al. 1997).

Soil improvement through admixture stabilization depends on the chemical reaction between cement and the remolded soil. Therefore, a longer mixing time (higher degree of mixing) leads to a higher strength (Babasaki et al. 1997). For this study, the soil and cement were mixed for 10 minutes for all samples as recommended by Babasaki et al. (1997).

All treated samples were compacted to the same wet density and water content. After compaction, the specimens were trimmed to the dimensions of 102 mm in height and 51 mm in diameter using the specimen trimmer, corresponding to a height-to-diameter ratio of 2. The samples were cured in the temperature-controlled room for a predetermined period of time.

Curing temperature is another factor that affects the rate of the chemical bonding reaction. The soil-cement samples were prepared at room temperature (20°C) and were subsequently cured at a controlled temperature of 8°C. Pozzutec 20$^+$ was added to the water/cement slurry at a proportion of 16% of the cement weight. This additive allows cement hydration to occur at temperatures as low as -10°C (Pozzutec 2006).

During the load application, the rate of vertical strain was 1% per minute. All instrumentation was connected to an electronic data acquisition (DA) system to record and save the data. The vertical strain was increased until the load reached a peak and subsequently decreased. For each test, vertical strain, ε, was calculated as follows:

[5.9] $$\varepsilon = \frac{\Delta l}{l}$$

where Δl = the vertical deformation of the specimen,

l = the original length of the specimen (≈ 102 mm).

The original cross sectional area (A_o) should be corrected (A_c) in order to calculate the applied vertical stress:

[5.10] $$A_o = \frac{D^2 \pi}{4}$$

[5.11] $$A_c = \frac{A_o}{1 - \varepsilon}$$

where, D = Specimen diameter (≈ 51 mm).

Finally the vertical stress acting on the specimen will be equal to:

$$[5.12] \quad \sigma = \frac{\text{vertical load}}{A_C}.$$

The peak vertical stress corresponds to the unconfined compressive strength.

5.6.2 Unconsolidated Undrained Triaxial Test (UU)

The unconsolidated undrained (UU) triaxial test is performed by applying a cell pressure and an increasing axial load during shearing. The back pressure (pore water pressure) was closed during shearing. Tests have been conducted by setting the confining pressure equal to 50 or 100 kPa to simulate the stress conditions in the field at the location of the failure plane and subsequently applying a vertical load to force shear failure.

Many authors have stated that for design purposes of soil-cement columns it is necessary to consider the normal stress at the failure plane in order to accurately estimate their strength.

As mentioned previously in Chapter 3, the failure plane usually occurred at depths up to 12-15 meters below ground surface. It is thus required to test samples under normal stresses consistent within this range. Ideally, the specimen should be mixed, cured and sheared under stress conditions consistent with the location of the failure plane. However, this is difficult to achieve for the following reasons. Firstly, the soil-cement mixture prior to curing is too soft to trim to the triaxial specimen dimensions. Secondly, a fully cured specimen is too hard to trim without damaging the sample. Thus, the specimen must be cured, at least partially, prior to trimming. The soil-cement mixture was cured under an applied normal stress in a mould for the first three days of curing. Subsequently, the samples were trimmed down to a diameter of 102 mm height and 51 mm diameter using the specimen trimmer, corresponding to a height-to-diameter ratio of 2. The trimmed specimen was then placed in the temperature-controlled room for the remainder of the curing period. Alternatively, the trimmed specimens could have been cured in the triaxial test apparatus for the remainder of the curing period. During shearing, the rate of vertical strain was 1% per minute. After recording the data, the stress-strain relationship can be interpreted as explained in the previous section to determine the peak shear strength.

5.7 Characteristics of the Treated Soil

To investigate the influence of the hardening agent on the improvement of soil-cement mixture, unconfined compression tests were conducted on reconstituted samples of high plastic clay (obtained from northwest of Winnipeg) mixed with various proportions of ordinary Portland cement.

The purpose of this part of study is to determine the ideal soil/cement ratio, which is marked by a plateau in shear resistance with increasing cement content. Unconfined compression tests were used to evaluate the shear strength and modulus of elasticity of the treated soil at varying soil/cement ratios.

Unconfined compressive strength, q_u, is plotted with respect to cement content in Figure 5.5. The results shown in this figure reaffirm that the cement content has a very large influence on the strength of the treated soil. Figure 5.5 shows that the compressive strength increases as the cement content increased up to 18% by dry weight of cement to dry weight of soil. Beyond this percentage of cement no significant improvement was gained in terms of compressive strength, thus the optimum soil/cement ratio for this particular soil may be interpreted as 18%. It may also be noted that the compressive strength increases with an approximately linear trend below a cement content of 18%.

Figure 5.6 shows that the elastic Young's modulus (E) increases with cement content. The elastic Young's modulus also increases approximately linearly with cement content, much like the compressive strength. Thus, increases in compressive strength accompanied increases in Young's modulus. The above discussions have many important practical implications for slope stability, as the compressive strength of the soil-cement reinforced columns are way beyond the strength of the native weak soils. As Winnipeg riverbanks soil is weak in strength with low friction angle, therefore strengthening the slip surface with a stronger element is required. A key consideration is the difference in strength between the existing shear surface and the cement-treated elements. In other words the larger the ratio between these two strengths the more improvement is gained.

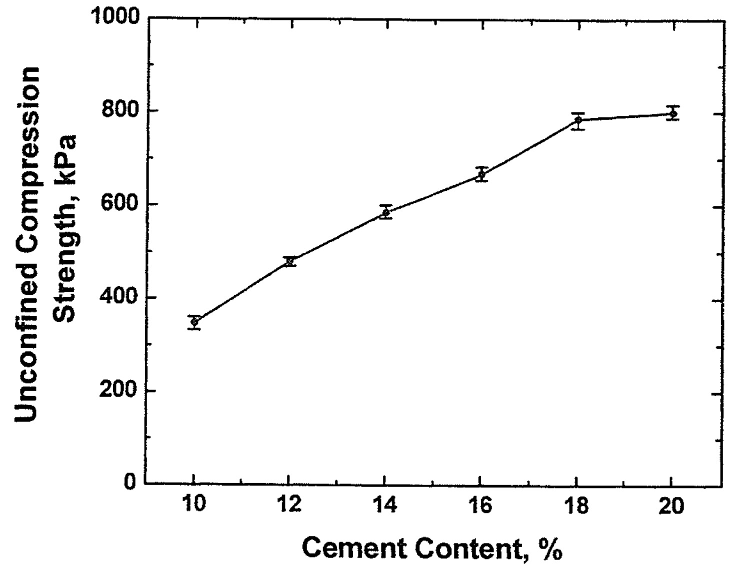

Figure 5.5 Unconfined compressive strength versus cement content of soil-cement specimens

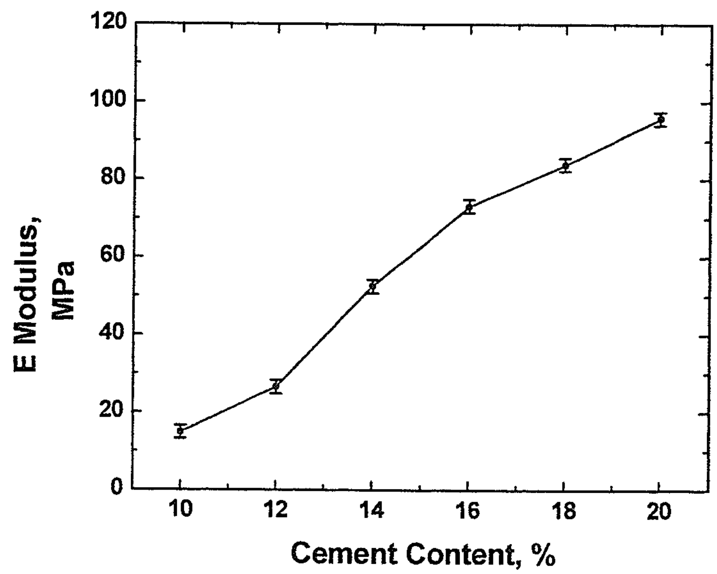

Figure 5.6 Relationship between Young's elastic modulus (E) and cement content

Similar studies have been conducted to improve the shear strength of Winnipeg lacustrine high plastic clay using lime in different ratios as reported by Ganapathy (1981). His study showed that the gain in strength of the soil-lime mixture was influenced by the amount of lime added, as in the case of this study but using cement. However Ganapathy (1981) indicated that the rate of gain in strength appears to decrease for higher lime contents. He reported that adding 10% of lime increased the unconfined compressive strength to 30 kPa for 28 days of curing. It should be noted that this value dropped when the moisture content exceed 39% (OMC). Many investigators have pointed out that the strength gain is much slower for lime-modified soils than for soil-cement mixtures.

The second stage of the testing program examined the influence of other factors on the strength characteristics of the soil-cement mixture, while the soil/cement ratio was held constant at the optimum ratio of 18%. The strength improvement from soil-cement mixing is derived from a time-dependent chemical reaction, which causes the unconfined compressive strength to be time dependent. Figure 5.7 shows the change of strength of soil-cement samples with time after soil mixing. The increase in strength with time is due to hydration of the hardening agent. The lower curve represents the compressive strength obtained from unconfined compression tests on samples cured under zero applied confining or vertical stresses. On the other hand, the upper curve represents the compressive strength obtained from unconsolidated undrained triaxial tests conducted on specimens initially cured under an applied vertical stress of 100 kPa. These results indicate that soil-cement mixtures cured at depth achieve a much larger increase in strength than when cured near the surface. Thus, a larger increase in strength can be achieved at the location of deep failure surfaces.

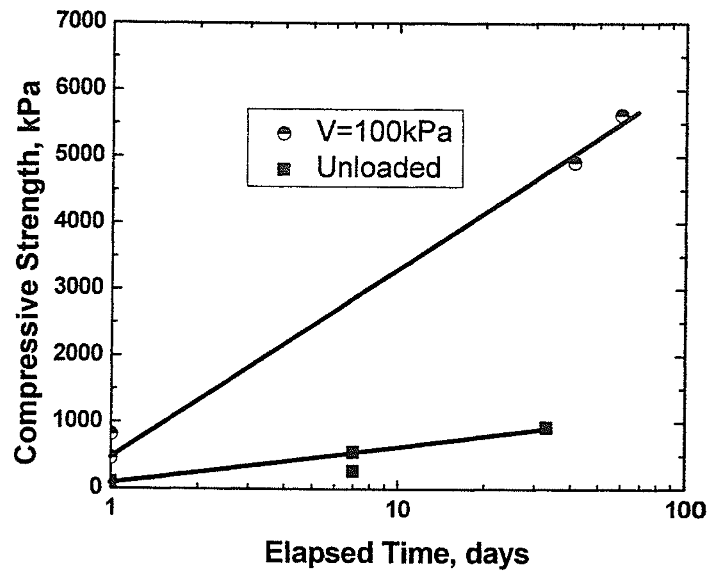

Figure 5.7 Compressive strength versus curing time of soil-cement mixture

Using results obtained from unconfined compression tests and unconsolidated undrained tests, the relationship between the strength and Young's modulus of elasticity of the soil-cement mixtures was investigated. Figure 5.8 shows the unconfined compressive strength and the initial tangent Young's modulus from laboratory prepared samples. These results show that the stiffness of the treated soil is proportional to the compressive strength. In other words, it shows a relatively linear relationship between the unconfined compressive strength and the modulus of elasticity. The relationship between the modulus of elasticity and the compressive strength can be expressed as:

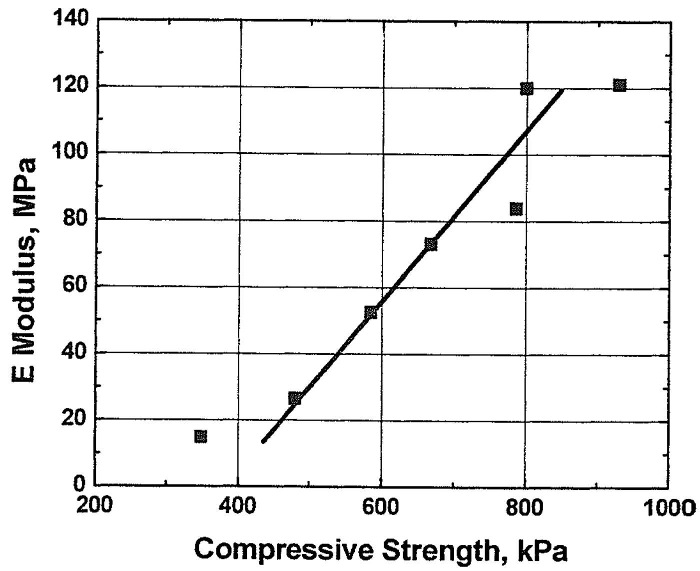

Figure 5.8 Relation between unconfined compressive strength and modulus of elasticity

[5.13] $E = 248\ q_u - 92200$

Axial strain at failure, ε_f, versus the unconfined compressive strength, q_u, of the soil-cement mixtures at different cement ratios is shown in Figure 5.9. As expected higher compressive strengths (because of higher cement ratios) lead to lower axial strain at failure. The axial strain values were between 0.75% and 2.30% for unconfined compressive strengths in the range between 100 kPa and 800 kPa.

Finally, the undrained strength parameters of soil-cement columns, c'_{col} and ϕ'_{col} are obtained by conducting UU tests at three levels of cell pressures 0, 50, and 100 kPa. Test results are shown in Figure 5.10 where the Mohr's stress circles and the failure envelopes based on total stresses have been plotted with respect to the peak strength values of the soil-cement columns. From these figures, it is

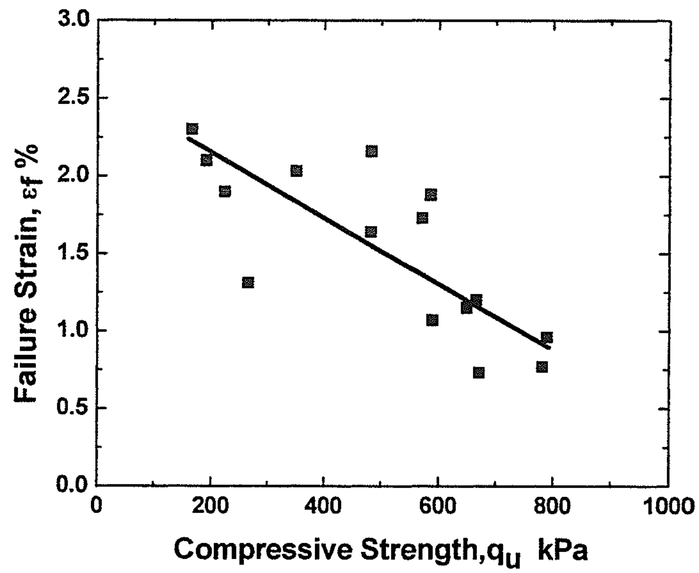

Figure 5.9 Relation between unconfined compressive strength and axial strain at failure for soil-cement samples

obvious that undrained shear strength is affected by the confining applied pressure, undrained shear strength increasing with increasing confining pressure. In other words the shear strength is affected by the normal stress on the failure plane. The undrained cohesion is 215 kPa and the corresponding friction angle is 30°.

5.8 Optimum Soil-water-cement ratio and mixing energy and duration

For cost savings on usage of cement and electrical power in the installation of soil-cement columns, the optimum soil-water-cement ratio should be determined.

In addition, the optimum required energy for mixing soil and cement should be evaluated as well. This can be done by fabricating a miniature soil-cement mixing

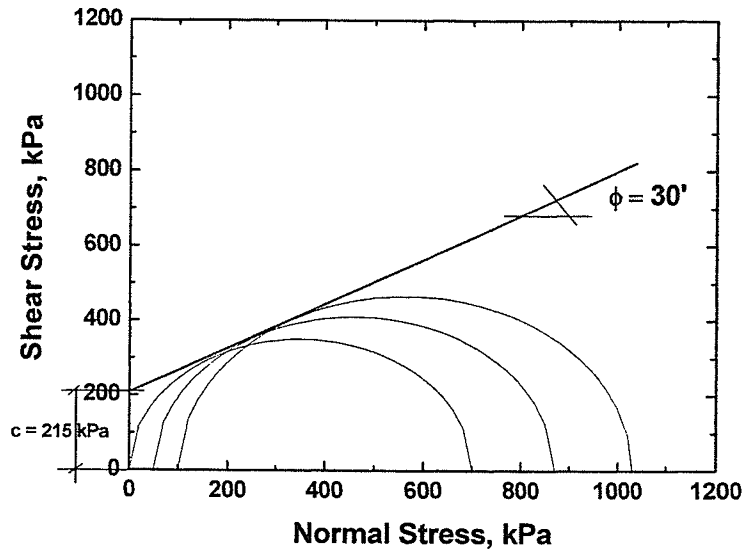

Figure 5.10 Shear strength parameters of soil-cement mixture

device for laboratory physical modeling with the same working principle as that of the machine installing actual soil-cement columns in the field. The miniature device is similar to that used by Shen et al. (2003) which include a slurry injection pump (for exerting pressure on the cement slurry to outlet), mixer (mixing head, head stand, auger motor for rotating blades) and controlling panel. It is envisaged that determing the pressure mixing process would yield cost savings and efficient installation of soil-cement columns.

5.9 Summary

(1) The literature recommends that the shear strength of soil-cement columns be measured and estimated using the unconfined compression test, regardless of the installation method used in the field. For the purpose of evaluating the stabilization improvement of riverbanks by soil-mixing columns, unconfined compression tests and unconsolidated undrained tests (UU) have been carried out in this study from samples prepared in the laboratory.

(2) Many factors have been investigated through this research in order to study their influence on the measured shear strength. These factors involve cement content, water content, curing period, and the confining stress applied during curing. Test results show that the undrained shear strength of soil-cement samples increased when vertical stress was applied during curing. This means that larger increases in strength may be achieved in soil-cement mixtures cured at greater depths, such as at the location of deep failure surfaces.

(3) In general the axial strain at failure of soil-cement specimens decreases with increasing compressive strength obtained from both unconfined compression tests and UU tests. Moreover, it was observed that the failure axial strain decreased with increasing curing time. The results obtained from these tests show axial strain values at failure were in the range of 0.75% to 2.3% for mixtures of 18% cement content (by dry weight) and compressive strengths exceeding 100 kPa.

(4) In summary, the results in this section were used to establish relationships between the strength and stiffness of the soil-cement mixture and other mixing and curing factors. The stiffness is an important consideration because it affects the mobilization of shear resistance in a slope matrix. Watn et al. (1999) found that the mobilization of strength in soil-cement columns is dependent on the relative stiffness between the column and the surrounding soil. The larger the relative stiffness of the column, the larger the mobilized stresses along a slip surface through the column material. In other words, the stiffer the column material, the more forces it attracts and resists. Thus, the difference in failure strain in the soil-cement column and the surrounding material is an important consideration in the strain incompatibility and strength mobilization in slope stability. The strain incompatibility and strength mobilization in slope stability are addressed in a later chapter, using the laboratory testing results from this section.

Chapter 6

NUMERICAL ANALYSIS AND RESULTS

6.1 Introduction

This chapter presents the numerical analysis of a typical riverbank in Winnipeg described in Chapter 2 (Sections 2.2 and 2.3). Two methods will be used to assess the performance of unreinforced and reinforced riverbank: Limit Equilibrium Method (LEM) and Finite Element Method (FEM). LEM has been used extensively by geotechnical engineers mainly because of its simplicity. As mentioned in Chapter 2, the FEM has many advantages. One of the significant advantages of FEM is its ability to compute the stresses, strains, and the associated mobilized shear resistance in the riverbank. A slope stability analysis is also available that combines LEM and FEM. The FEM is used to determine more accurately the stresses in the soil mass and then uses these stresses in the LEM to determine the factor of safety of the slope (GeoStudio 2005). In this combined FEM and LEM approach, the numerically computed stresses from

FEM are used to establish the resisting and driving shear forces along a potential slip surface and then use the summation of these forces to compute a safety factor. The pure FEM approach can be categorized into two general methods: shear strength reduction method (SSR) and gravity increase method (GIM). These two methods have been described in detail in Section 2.10.

A number of researchers (Griffiths and Lane 1999, Dawson et al. 2000, Hammah et al. 2005, Shukha and Baker 2003, Duncan 1996, Griffths 2006) reported several advantages of the SSR technique, including the ability to predict stresses and deformations of support elements, such as piles, anchors and geosynthetic reinforcement, even at the verge of failure. The failure occurs naturally through the zones within the slope soil matrix until the stresses developed by gravity can not be any more supported by the soil shear strength. The concept of failure in FEM is preserved locally and globally and no assumptions to be made in advance about slices and interslice forces. However, Krahn (2006) recommended that the use of SSR needed careful attention because of its inherent limitations that affect its usefulness in practice. One limitation of the SSR technique is that failure is based on non-convergence of the solution. He pointed out that this is very unconventional in engineering design analyses, in that engineers usually rely on converged solutions rather than uncoverged solutions. The second limitation as Krahn (2006) pointed out is that the strength of soil is reduced throughout the earth's structure by an equal amount. This means that the local safety factor is taken to be constant along the entire slip surface which is not the case in reality. The local safety factor in fact varies significantly along the potential failure surface.

The performance (both deformation and stability aspects) of the riverbank will be assessed using three approaches, namely: 1) LEM, 2) combined LEM and FEM, and 3) FEM-SSR.

6.2 Limit Equilibrium Method (LEM)

Factor of safety is calculated using the traditional limit equilibrium method. The Slope/W component of GeoStudio[2] and Slide 5, two commercial computer programs, which implement the limit equilibrium methods, were chosen. Results from both programs were compared. Figure 6.1 shows a typical riverbank geometry in the Winnipeg area. This geometry was used for all the analysis work done throughout this study. The water table was assumed at ground surface representing worst case condition, although typically it was about 2m below the ground surface (Tutkaluk 2000).

6.2.1 Using Slide 5 (Rocscience) Software

Rocscience Slide 5 is another software obtained by the Geotechnical Group at the University of Manitoba. It has a simple graphical user interface and allows the user to choose from several LEM methods. Two traditional limit equilibrium

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[2] Geoslope International Inc. 2004

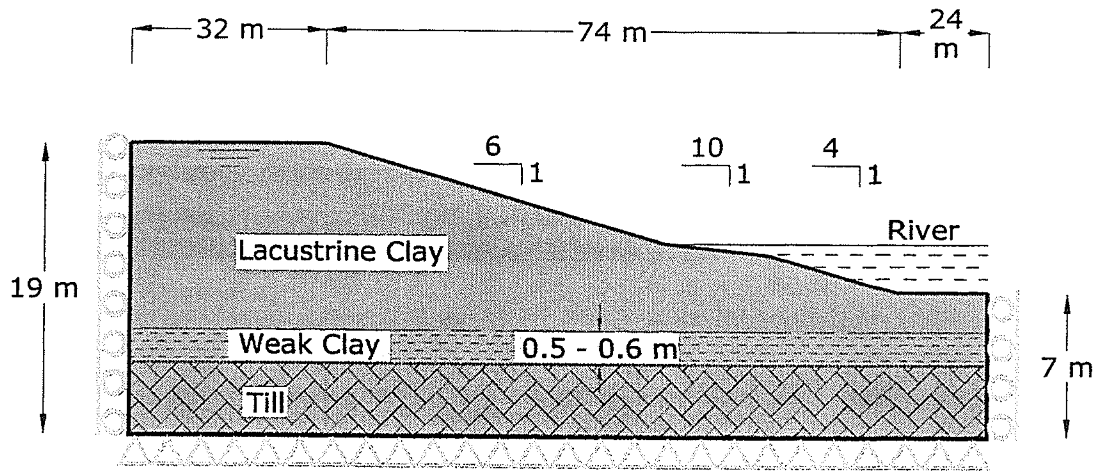

Figure 6.1 Typical geometry of Red riverbank in Winnipeg (after Tutkaluk 2000)

methods; Bishop's and Morgenstern and Price's methods were used to analyze the slope stability of the riverbank. These methods are the most popular in the industry and they take into account force and moment equilibrium (see Table 2.1 in Chapter 2). Two-dimensional plane strain has been assumed for all analyses. Results for the analysis of the stabilized riverbank will be discussed later in Section 6.4.

6.2.2 Using Slope/W (Geo-Studio) Software

GeoStudio is an integrated geotechnical software suite that can perform various types of geotechnical numerical analysis. Stability of the typical riverbank was analyzed using Slope/W software to verify the previous results obtained from Slide 5. Both software packages deal with most of the stability analysis problems encountered in geotechnical engineering practice. Stability analyses in Slope/W were also carried out utilizing Morgenstern and Price's and Bishop's methods.

6.3 Finite Element Method (FEM) using Shear Strength Reduction (SSR) Approach

There are two ways to estimate the factor of safety of slopes using FEM analysis: the shear reduction method, and gravity increase method. As mentioned in Chapter 2 (Section 2.10.2) the SSR method will be used in this study because it is more suited for analyzing the stability of natural slopes (Swan and Seo 1999). The main purpose of implementing numerical analysis is to estimate the total displacement of the natural and stabilized riverbanks in addition to estimating the factor of safety. A combined FEM and Limit-Equilibrium analysis using Geo-Studio (combined Sigma/W & Slope/W) was also conducted to verify the results obtained using the SSR method.

6.3.1 SSR Analysis Parameters and Boundary Conditions

Phase 2 software[3], an SSR-based FEM computer program, was chosen to perform the numerical analysis. The model used 6 noded triangular elements, and was set to a maximum of 500 iterations and a tolerance value of 0.001. These settings are recommended values given by Rocscience (2005).

The values of soil modulus for both high plastic clay and rockfill material are required for the soil constitutive model used to analyze the performance of the riverbank. Presently, the SSR method is limited to using elastic and elastic-perfectly plastic constitutive soil models. Values of equivalent Young's modulus,

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[3] Rocscience Ltd. 2005.

E, and Poisson's ratio, ν, in combination with shear strength parameters are required to compute the stresses and deformations in a riverbank. A rigorous elasto-plastic model such as Modified Cam Clay model suitable for high plastic clays is yet to be incorporated into the computer program.

One common laboratory strength test is the direct shear test, illustrated in Figure 6.2. In this figure, $\Delta\tau$ is the change in shear stress, Δx is the change in horizontal displacement, Δz is the change in vertical displacement, D is the sample diameter, and h is the sample height. Large direct shear testing was used in this study to accommodate the tests on clay-rockfill composite materials. To define the appropriate Young's modulus (E), results from large-scale direct shear tests were used to estimate the shear modulus (G) of dense rockfill materials. Rough estimates for the shear modulus (G) can be obtained using Figure 6.2 and the equation given below (Davis and Selvadurai 1996):

[6.1] Shear modulus, $G = \dfrac{\Delta\tau}{\Delta x} h$

The values of shear modulus were used to calculate the Young's modulus for both clay and rockfill soils using linear elastic assumptions for incompressible medium:

[6.2] $E = 2G(1+\nu)$

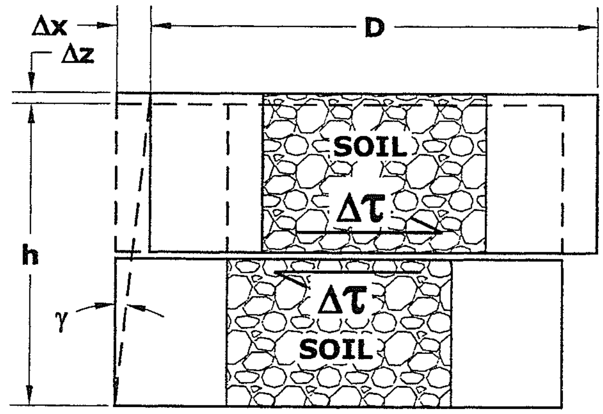


Figure 6.2 Direct shear test geometry after shearing (after Davis and Selvadurai 1996)

The elastic parameters taken from direct shear tests must be used with caution given the fact that the tests create nonuniformity of both stress and strain fields in the soil sample. Triaxial test may be the best tool for estimating elastic constants but large triaxial test equipment required for testing clay-rockfill composite material is not available in our laboratory.

The value of Young's modulus used in this analysis was the initial value. Hammah et al. (2005) pointed out that Young's modulus had minimal impact on factor of safety results. However, this value will significantly affect the computed deformations.

Poisson's ratio (ν) can be calculated using the Neutral Earth Pressure Method equation (Gregory 1973):

[6.3] $K_o = \dfrac{\nu}{1-\nu}$

The value of earth pressure, K_0 can be also calculated using the Jaky (1948) equation:

[6.4] $K_o = 1 - \sin\phi'$

6.4 Analysis of a typical riverbank in Winnipeg

6.4.1 Mechnism of riverbank failure

Movements in the riverbank may occur at low or high rates due to many factors. Progressive failure may also occur. This led to realization that the mobilized shear strength varies along the slip surface, especially in over-consolidated clays. As a result, shear stress along a potential shear failure plane will transfer progressively due to increasing shear strain.

Progressive failure does not only occur due to the soils physical and chemical properties (Abramson et al. 2002) but other factors such as slope topography, climatic factors, soil mineralogy; in our case a high plastic soft clay with high potential of swelling, ground water conditions, and heavy rainfalls and other factors can influence and cause a reduction in slope soil strength and consequently lead to riverbank failure.

Such unstable terrains are under continuous displacements, especially at point of maximum stream curvature (as pointed out by many investigators in Winnipeg). Displacements may occur at a rate enough to be of concern and consequently most favorable to slope-failure. Such situations, it is a mistake for such unstable riverbanks (FS $\approx$ 1) to be considered as stable and to accept the displacement values calculated as real values. As a matter of fact this value has no practical meaning and the fact is that the slope displacements will continue to occur until complete failure.

Therefore, realistic engineering evaluation of the expected movement of riverbanks stabilized with rockfill columns was simulated in this study through one stage modeling. At any time of riverbank stabilization there are movements occurring to mobilize the shear resistance of rockfill columns, irrespective of the amount of previous movements. However resistance of the native clay is dependent on the amount of previous movements. For example, for an ancient landslide, the soil strength parameters cannot be selected at their peak value, because it provides an overestimated value of factor of safety due to the fact that variable shear strain rates within riverbanks cause some terrain to mobilize shear resistance to its peak value, while lower shear strain rate leads to a lower shear resistance (Cornforth 2005, Gao et al. 2003). Post peak strength parameters may be more favorable for simulation of first slides while residual values may be used when designing a stabilization measure for a riverbank that has already failed.

6.4.2 Analysis of natural riverbank

Analysis of natural riverbank (before rockfill column installation), the critical strength reduction factor (SRF) was evaluated by assuming a ground water table at the ground surface. The slope soils consist of two types of soils. The calculated factor of safety of the natural riverbank was close to 1 (see detailed analysis in Section 6.5.1). This result was similar to the factor of safety determined using the LEM, which agree with documented field observations for a similar geometry and boundary conditions as mentioned earlier in Chapter 2. Therefore for a typical riverbank in Winnipeg with a similar geometry, stabilization is required.

6.4.3 Analysis of stabilized riverbank with rockfill columns

Hassiotis et al. (1997) recommended that reinforced concrete piles must be placed in the upper middle portion of the slope to provide the optimum improvement of factor of safety. Similarly, the Federal Highway Administration (1983) recommended that stone columns should be installed in the middle of the predicted slope failure (see Chapter 2). Preliminary design assumption, that the rockfill columns were placed at the mid-span of the potential failure surface. Alternative locations of the rockfill columns were also studied. A triangular grid pattern of rockfill columns was assumed with a centre-to-centre spacing of columns equal to twice the column diameter. Since the analysis was two-dimensional, the rockfill columns had to be represented by an equivalent strip element as shown in Figure 6.3. Calculation formula to represent this strip element is shown in Equation [6.5]:

[6.5] $t = \frac{A}{s}$

where, t = the thickness of equivalent strip element,

s = the spacing between columns,

A = the cross sectional area of the rockfill column.

The principal functions provided by rockfill columns are to provide stiffer and stronger material compared to the surrounding soil, and to act as a drain to relieve the pore pressure at the interface layer between the weak soil and the till. It was considered that hydraulic conductivity of rockfill columns is relatively high and for FEM analysis was considered equal to 0.02 meter/ sec.

The average shear strength along the potential slip surface will increase significantly as a result of rockfill column installation, serving to stabilize and reinforce the riverbank. The rockfill columns should extend deep into the stiff till to act as a key (pin) into the stiff layer in order to force the failure surface through the rockfill columns instead of beneath them at the rockfill-till interface.

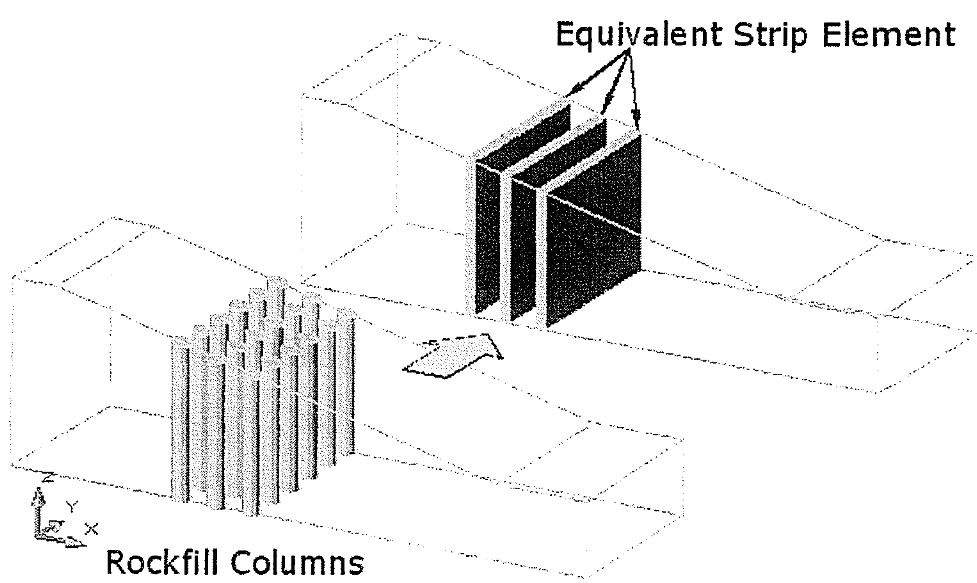


Figure 6.3 Equivalent strip elements of the rockfill columns

Engineering properties of rockfill materials, durability of rockfill materials, column's shear resistance, column's shear stiffness, relative density, column's diameter, and distance of column insertion into the stiff till layer are factors that may contribute to the effective performance of rockfill columns in riverbank stabilization (Abdulrazaq et al. 2005). One of the most significant factor to be investigated in this study is the amount of shear strain (and thus displacement) required to mobilize shear resistance of rockfill columns without exceeding the allowable maximum deformation in the stabilized riverbanks. Graham (1986) reported maximum shear strains in unstabilized riverbanks in the range of 1.75%-4% in brittle high plastic clays susceptible to strain softening. Therefore, the numerical analysis is vital to estimate the amount of shear strain required to mobilize shear resistance of rockfill columns in these soils, and to determine if the associated shear deformation is acceptable in terms of the serviceability requirements of the riverbanks or of the structures near riverbanks.

The soil properties used in this study (for analysis and design) are shown in Table 6.1. These have been determined from laboratory direct shear tests (both large-scale and conventional direct shear tests) conducted as part of this study. The parameters for rockfill materials were verified by simulating numerically the results of large-scale direct shear tests (Kim 2007). It was shown that the results from numerical simulation compared reasonably well with those from the large-scale tests.

Duncan (1996) stated that values of c' and ϕ' determined from direct shear tests have been found to be essentially the same as values determined from drained triaxial or consolidated undrained (CU) tests with pore water pressure measurements. It should be noted that the friction angle of the rockfill material is dependent on the stress and shear strain levels. The strength properties of this material were selected as shown just as a first approximation.

Table 6.1 Soil properties for both lacustrine clay and the weak soil used in the analysis and design

Type of Soil	E MPa	ν	γ kN/m^3	ϕ degrees	c kN/m^2
Clay	5	0.41	17.0	15	4
Weak Clay	3.5	0.41	15.7	12	3
Rock Column material	22	0.20	19.0	50	0
*Till Layer	170	0.20	22.0	50	0

* Based on Baracous et al. 1983

6.5 Analysis results

6.5.1 Stability and Deformation Analyses of Natural Riverbank

Figures 6.4 and 6.5 show the calculated factor of safety of a natural riverbank for both Slide 5 and Slope/W computer programs using Morgenstern and Price's method. As can be seen in these figures, the factors of safety were 0.986 and 0.972 from Slide 5 and Slope/W, respectively. Using Bishop's method, the factors of safety were 0.954 and 0.966 from Slide 5 and Slope/W, respectively. There was good agreement between the two software programs in terms of the estimated factors of safety and the potential failure geometry. The calculated factor of safety (expressed as critical strength reduction factor, SRF) using the shear strength reduction method for the natural riverbank was about unity, as shown in Figure 6.6. A combination of LEM and FEM using Geo-studio estimated the factor of safety at about 0.893 as shown in Figure 6.7. These results were similar to the factor of safety estimated using the LEM.

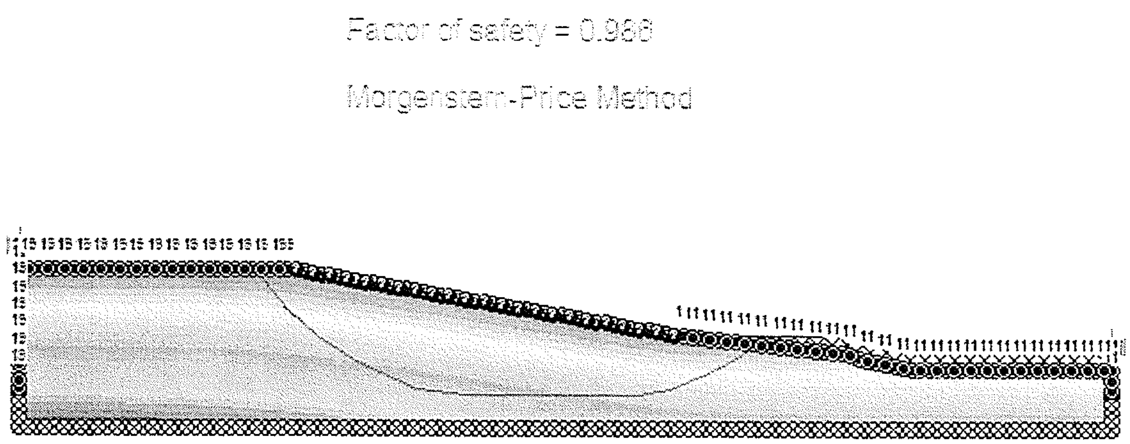


Figure 6.4 Slope stability analysis for natural riverbank using limit equilibrium method (Rocscience, Slide 5)

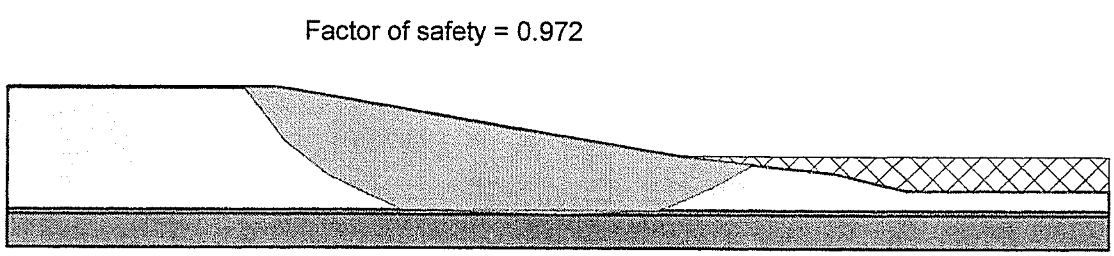


Figure 6.5 Slope stability analysis for natural riverbank using limit equilibrium method (Geo-Studio, Slope/W)

The analysis results indicated that the slope is unstable for the given slope geometry, ground water conditions, river level, soil properties and boundary conditions, which agree with field observations for Winnipeg riverbanks as discussed in Chapter 2. Results also showed that the potential shear failure plane passed through the weakest clay zone located at the clay till interface.

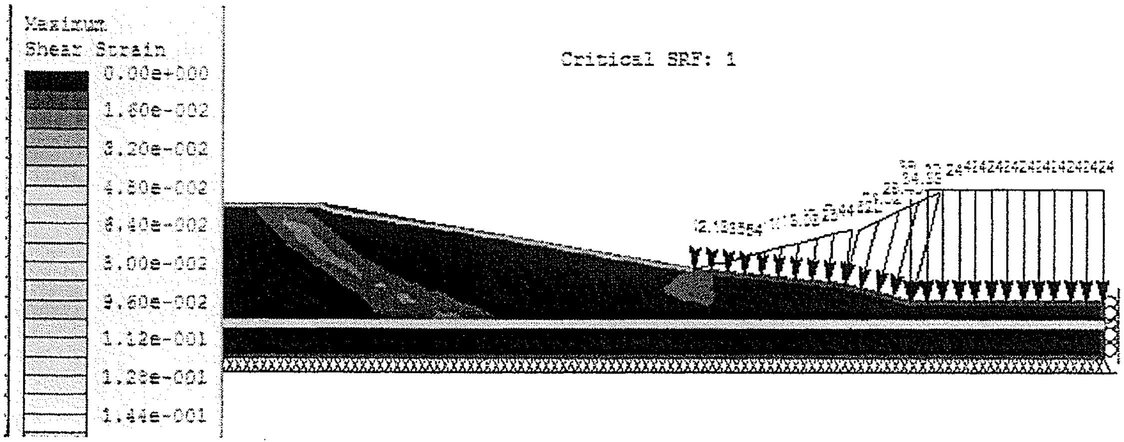


Figure 6.6 Slope stability analysis using FEM utilizing strength reduction method (Phase2)

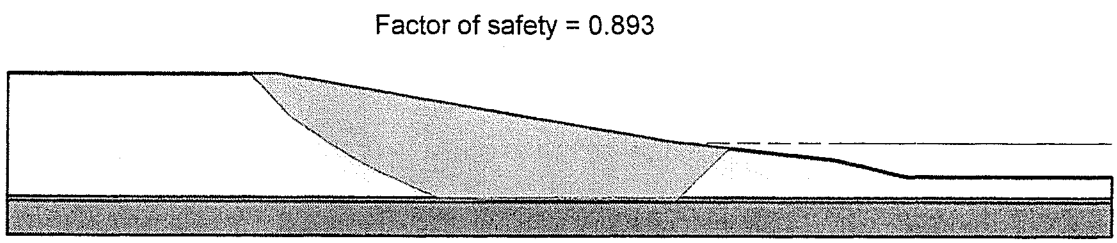
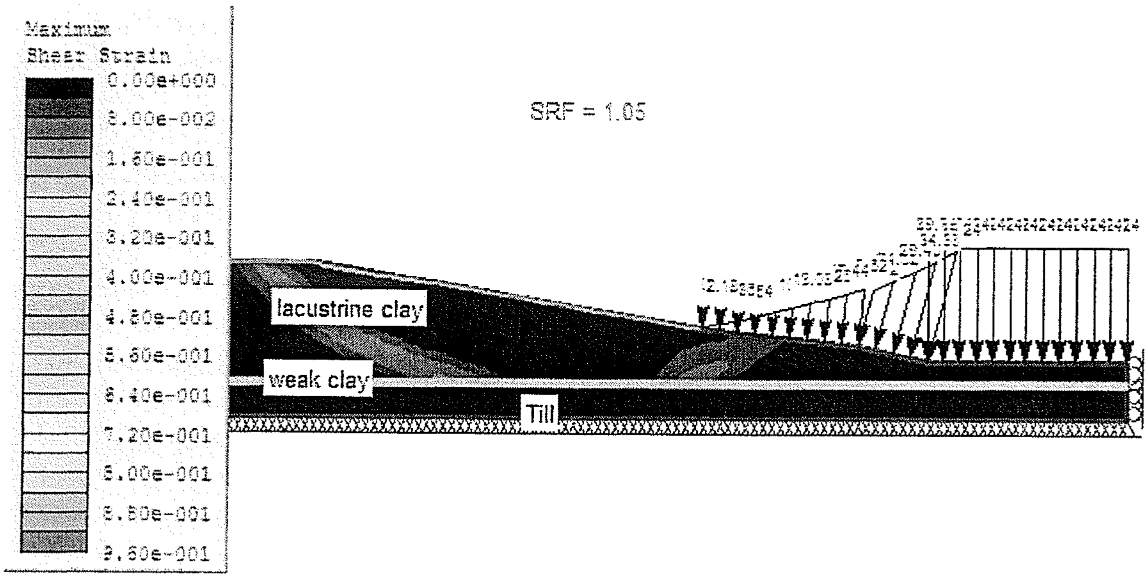
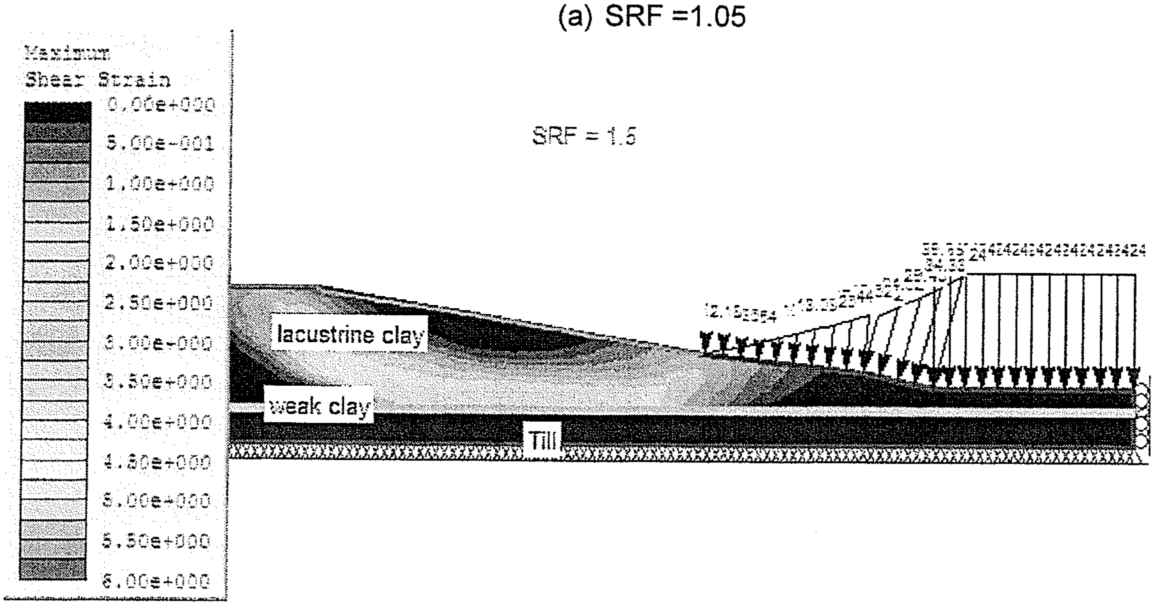


Figure 6.7 Slope stability analysis for natural riverbank using FEM with LEM (Sigma/W and Slope/W)

The failure slip surface predicted by the numerical analyses helped to determine the most efficient location to install the rockfill columns. It is interesting to note that as the shear strength of the clay is reduced, progressive movements of the riverbanks can occur as demonstrated in Figures 6.6 and 6.8. However, it should be noted that the shear strains (and thus deformations) calculated are meaningless as the riverbank has already failed. It simply shows the potential for progressive movements of the riverbank. Figure 6.9 and Figure 6.10 show the maximum total displacements in the natural riverbank obtained using Phase 2 and Sigma/W, respectively. Note that the results from Sigma/W only provides the



(a) SRF =1.05



(b) SRF =1.5

Figure 6.8 Failure patterns through various SRF values for an existing riverbank (Phase 2) individual y- and x-displacements while Phase 2 shows the resultant of these displacements. It is clear that values of the maximum displacements were closed, from both software programs using numerically computed stresses. However, the values obtained for unstable riverbanks are again meaningless, but they were calculated here for comparison purposes.

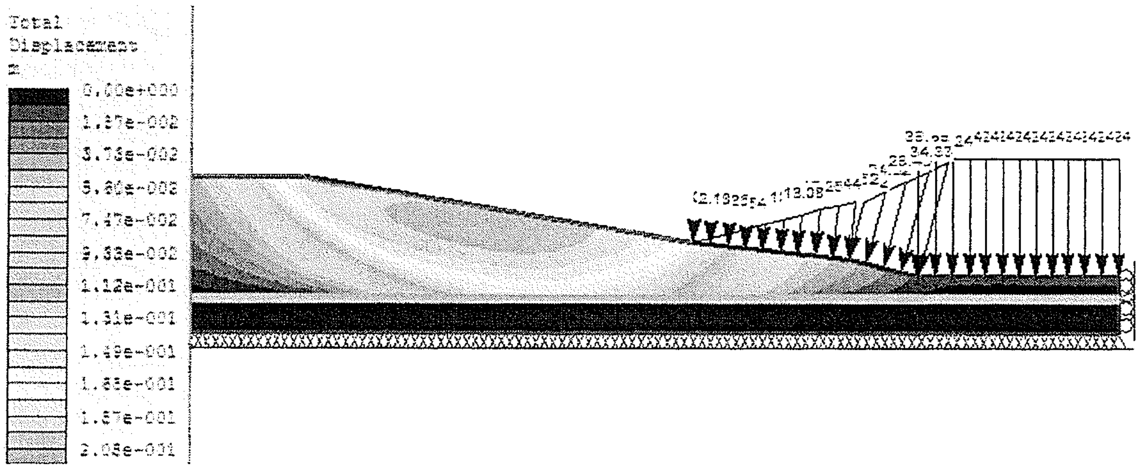
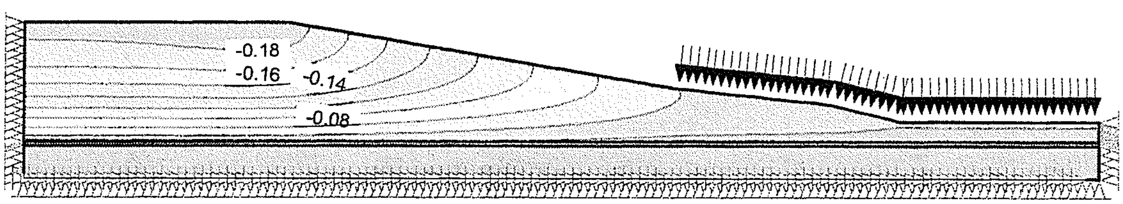
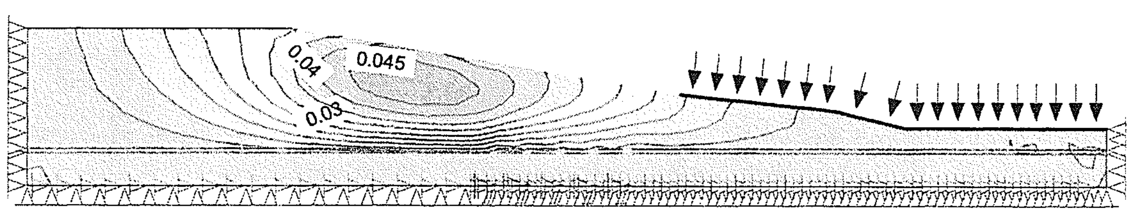


Figure 6.9 Displacement of natural riverbank using Phase 2



(a) Y-displacement, m



(b) X-dispalcement, m

Figure 6.10 Total y and x-displacements of natural riverbank using Sigma/W

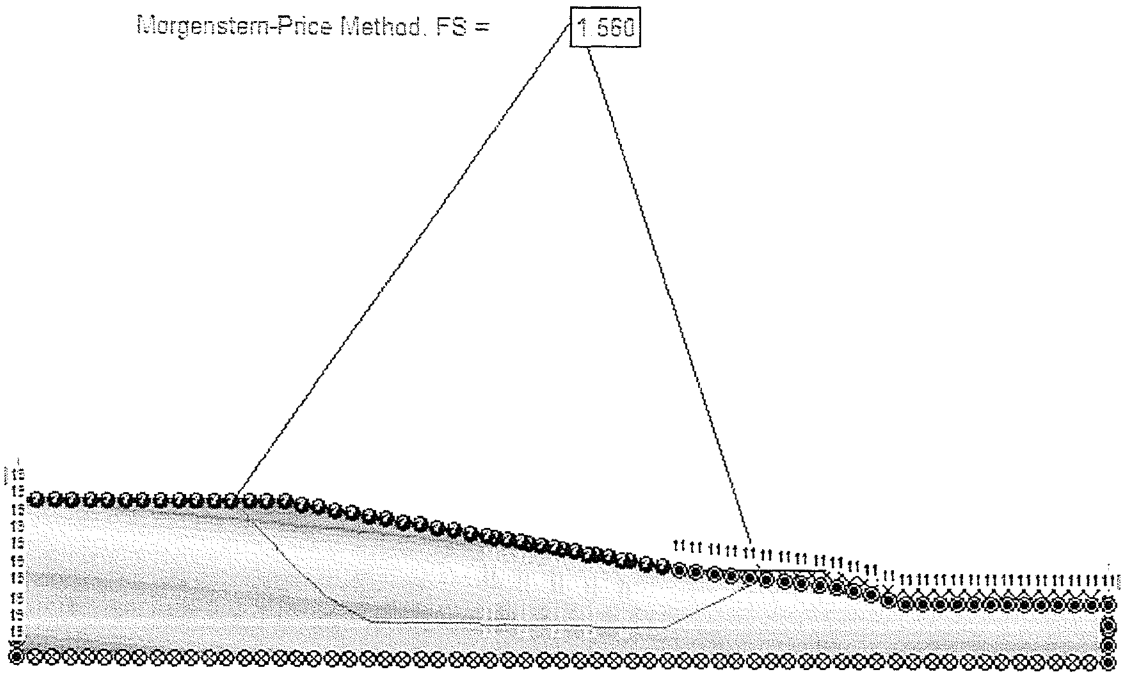


Figure 6.11 Slope stability analysis for stabilized riverbank using limit equilibrium method (Slide 5)

6.5.2 Stability Analysis of Rockfill Stabilized Riverbank

Figure 6.11 shows five rows of columns having a diameter of 2.3 m that were used in the riverbank being analyzed. The factor of safety using the limit equilibrium method (Slide 5) was calculated to be about 1.56 using the Morgenstern and Price method. On the other hand, the results from FEM using SSR method indicated a factor of safety equal to 1.35 as shown in Figure 6.12. This means that SSR provides a more conservative solution compared to LEM. The results of the comparison of factors of safety between LEM and FEM-SSR are consistent with those observed by Hammah et al. (2005) wherein the factors of safety calculated using FEM-SSR are generally 5% less than those calculated from LEM, for slopes stabilized with piles. The above results show, that the principle of replacing weak soil on the discrete shear zone of a landslide riverbank with well compacted rockfill columns improved the stability of the riverbank.

The maximum shear strain shown in Figure 6.12 is a good indication of the potential slip plane. It was observed that the potential slip plane moved from the interface between the till layer and the weak clay upward and through the rockfill columns. It was mentioned in Chapter 4 that the mobilized shear resistance of rockfill columns is highly influenced by the effective normal stress applied at the sheared plane. The increase in depth does not increase the shear resistance in the clay as much as it increases in the rockfill columns. This phenomenon forces the shear failure plane to move upward where the applied normal stress is less, as shown in Figure 6.12.

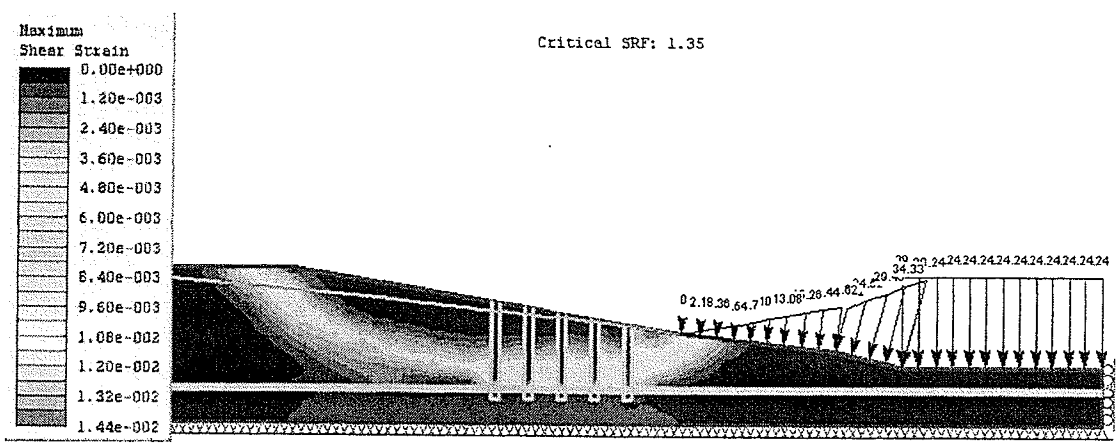


Figure 6.12 Stability analysis for stabilized riverbank using FEM /strength reduction technique (Phase 2)

The stability results obtained using Slide 5 and Phase 2 software programs have been justified in two ways. Firstly, similar analyses have been implemented using Slope/W and Sigma/W in order to perform LEM and FEM analysis. Results obtained showed strong agreement between the two packages of software programs. Figures 6.13 and 6.14 show that the factors of safety are equal to 1.415 using Slope/W (LEM method) and 1.391 using Slope/W in conjunction with Sigma/W (FEM method). Moreover, the modes of shear failure in all cases were similar. Secondly, good agreement was found between FEM and the field observations in terms of the size and depth of the shear failure plane in the natural riverbank in Winnipeg (see Peterson et al. 1960, Chapters 2).

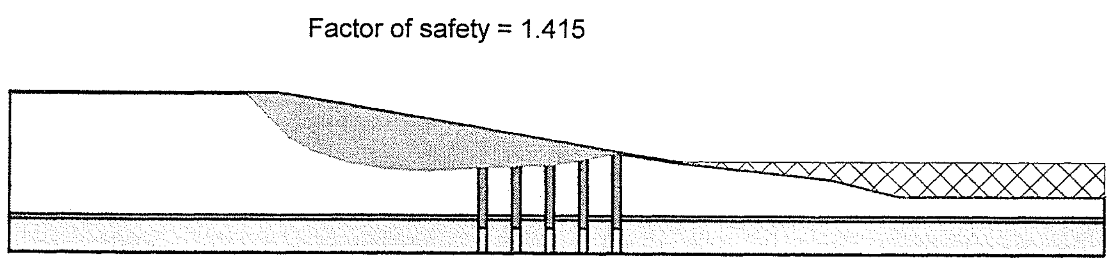


Figure 6.13 Slope stability analysis for stabilized riverbank using Limit Equilibrium Method (Slope/W)

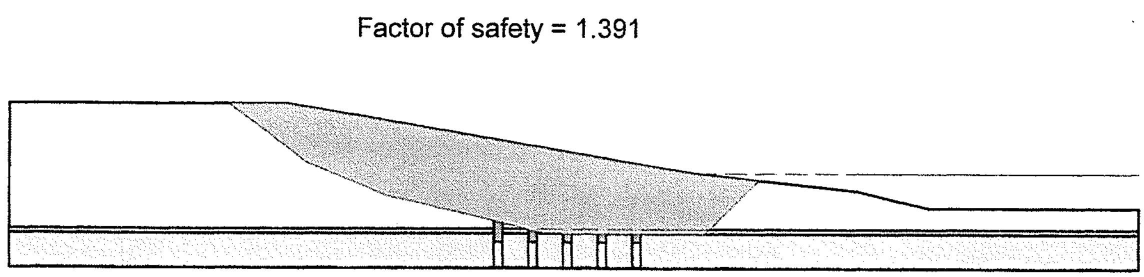


Figure 6.14 Slope stability analysis for stabilized riverbank using FEM with LEM (Sigma/W and Slope/W)

6.5.3 Deformation Analysis of Stabilized Riverbank

Finite element analysis was conducted using computer programs to perform a rational analysis of deformations in the riverbank reinforced with rockfill columns.

FEM with linear elastic-perfectly plastic stress-strain relationships have been applied in this study to predict riverbank movements after installation of rockfill columns. The linear elastic material model was selected for its simplicity and accuracy. Only two extra elastic parameters, Young's modulus E and Poisson's ratio, ν, are required to characterize the stress-strain behaviour of isotropic elastic materials (Duncan 1996). The strength of materials is governed by Mohr-Columb failure criterion.

The total displacement at the critical strength reduction value, $SRF_{critical}$, under a stable condition is important in assessing the performance of the riverbank even if the factor of safety is acceptable. Figure 6.15 shows the total displacements contours in the riverbank corresponding to a computed factor of safety equal to 1.35 using Phase 2. The predicted total displacement for a typical riverbank in Winnipeg was about 9 cm. The maximum displacement occurred at mid-span between the columns and the crest of the slope, while the maximum displacement for the unreinforced riverbank occurred at the mid-span between the crest and the river. The calculated slope displacements were compared to field observations for the same rockfill column pattern and number.

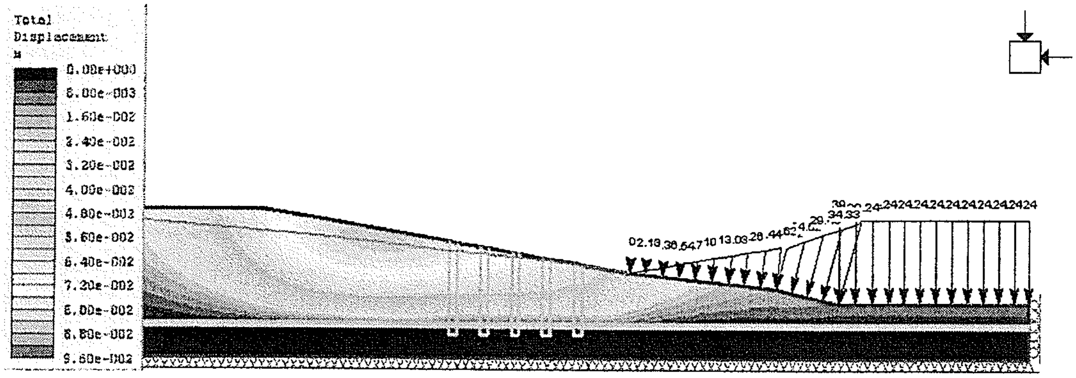
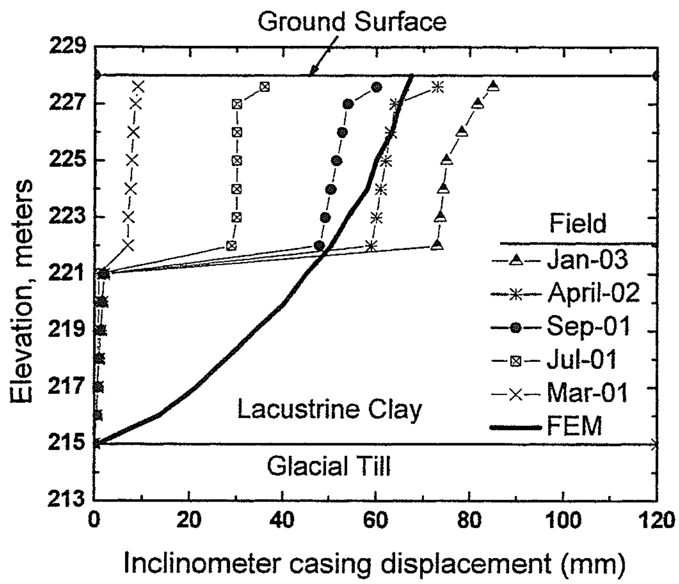
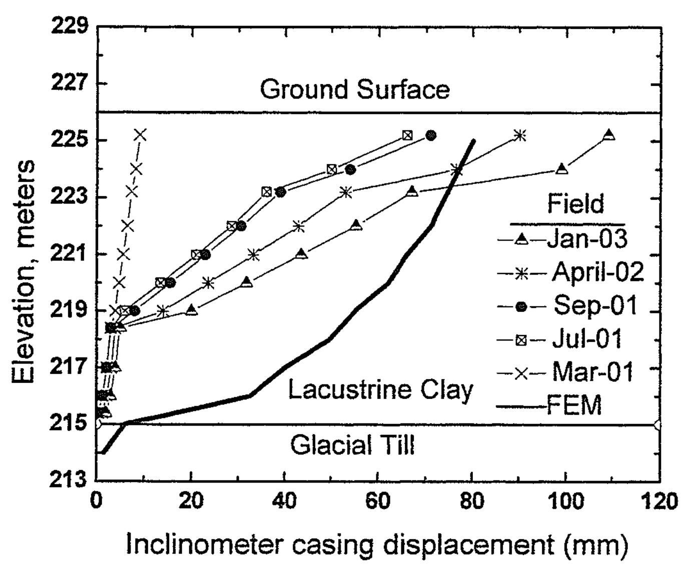


Figure 6.15 Total displacement of rockfill columns of stabilized riverbank using Phase 2

Figure 6.16 shows the FEM values in addition to the field observed displacements reported by Yarechewiski and Tallin (2003) for a riverbank in Winnipeg. It should be noted that the displacement readings from inclinometer mostly represented horizontal displacements of the slope, while the FEM results represented the displacement vector along the axis of the inclinometer. The displacement profiles were not properly simulated, but a reasonable agreement in the magnitude of displacement was found at the surface between the field observations and the numerical models even using a simplified geometry of riverbank used in the modeling and probably slightly different material properties. A separate research project is on-going to install rockfill columns in a fully-instrumented section along the riverbank and load it to failure. This will further validate and fine-tune the results from the large-scale laboratory testing and a numerical model developed using data currently available from the local consulting community.



(a) Post-installation displacements at riverbank crest



(b) Post-installation displacements between rockfill columns

Figure 6.16 Field observations and FEM results of the total displacement of rockfill columns of stabilized riverbank (after Yarechewiski and Tallin 2003)

It can be noted that riverbank reinforced with rockfill columns has significantly improved stability, as the applied lateral forces (driving forces) on the soil mass are shared between the columns and the ambient soil on the basis of their shear stiffness. In other words the applied driving stresses will transfer from the native soil to the rockfill columns that are relatively stiffer and stronger elements.

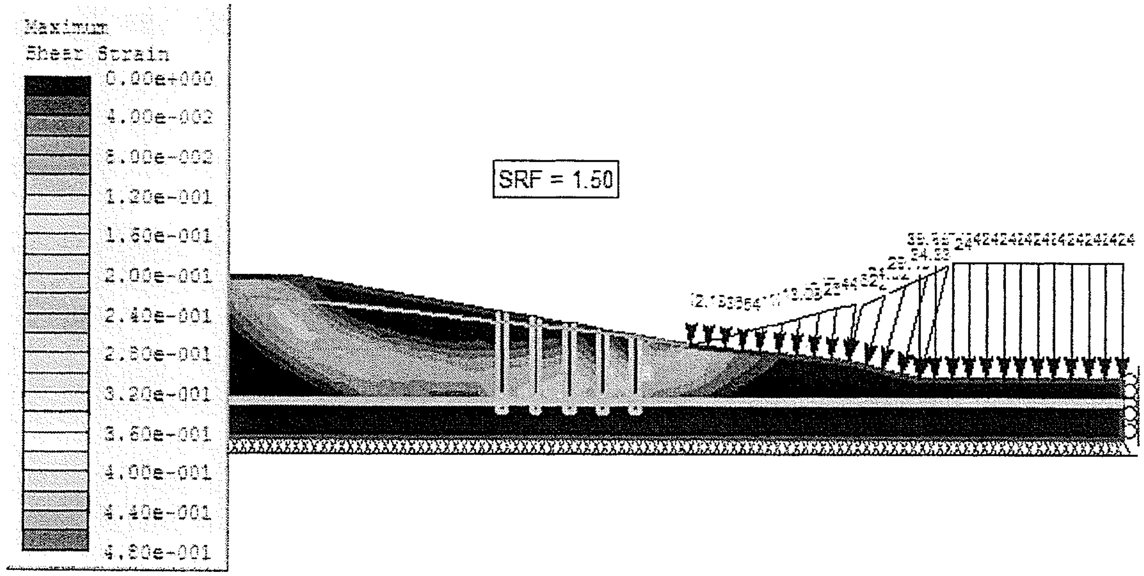
Combining Figures 6.12 and 6.17, the calculated shear strains of the riverbank are shown with increasing SRF. These figures demonstrate the progressive failure mechanism of the riverbank. The maximum shear strain increases until the increment becomes so high that the FEM solution will not converge, Figure 6.18 shows the total displacements of the stabilized riverbank below, within, and above the critical values. This figure shows that as the strength decreases, the maximum displacement increases. At some point (beyond the $SRF_{critical}$) the slope will fail, in which case the algorithm is unable to find a stress redistribution that satisfies the local and global equilibrium, and deformation increases rapidly. In other words, the maximum deformation keeps increasing until a particular stage is achieved where the increment becomes so high that the finite element analysis will not converge (Griffiths 2006).

6.6 Effect of rockfill columns geometry and layout for stabilizing natural riverbank in Winnipeg

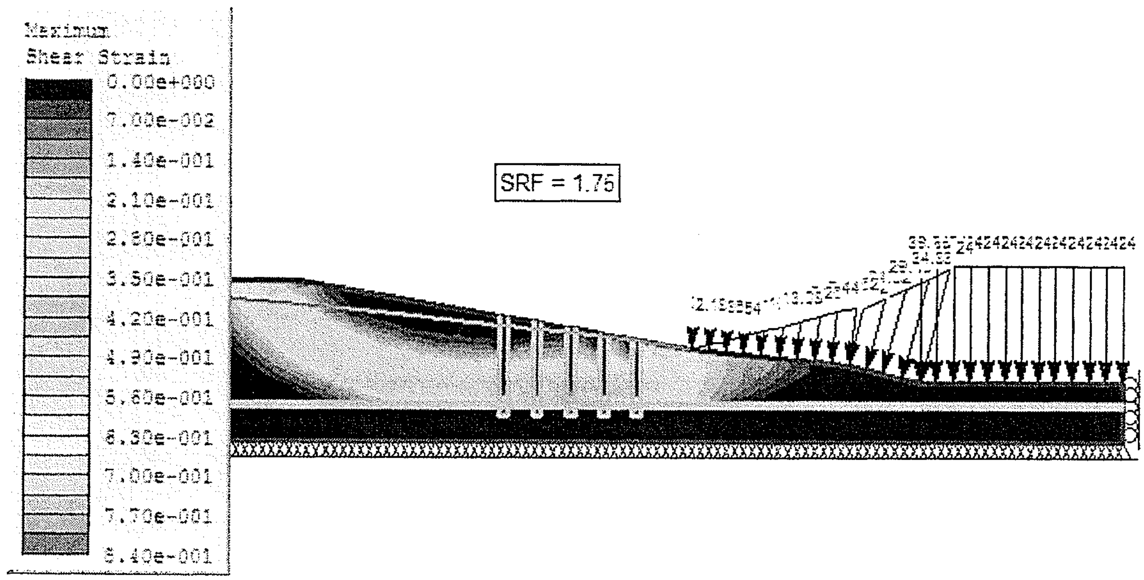
To illustrate the importance of properly selecting the locations of rockfill columns along the riverbank, another analysis was carried out with columns located further up and down in the slope. The results and discussions are presented in this section.

Two design considerations of riverbank stabilization works are examined. The first design consideration is the geometry such as spacing and configuration of columns along the riverbank to provide optimum factor of safety against instability. The second consideration is the slope movements that are expected following installation of rockfill columns. The assessment of the stability has been conducted using limit equilibrium methods (LEM) to calculate the factor of safety.

LEM can also be used for evaluating the degree of improvement achieved by introducing slope reinforcing element such as rockfill columns. The traditional limit equilibrium method (LEM) is the most popular method used to examine the impact of proposed stabilization works because of its accuracy and simplicity. However, the finite element method (FEM) has become increasingly popular for slope stability analysis and remedial design, because it provides valuable information about the magnitude of movements in the slopes. FEM enjoys many advantages including the ability to predict stresses and associated deformations with and without reinforcement (Matsui and San 1992, Duncan 1996, Griffiths and Lane 1999, Dawson et al. 2000, and Hammah et al. 2004).



(a) SRF = 1.50



(b) SRF = 1.75

Figure 6.17 Failure patterns through various SRF values, (a) SRF=1.50, (b) SRF=1.75

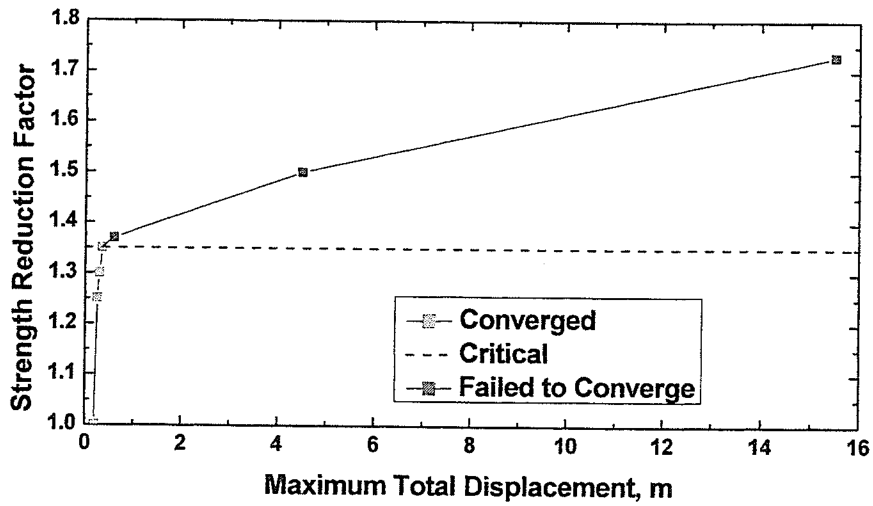


Figure 6.18 Total displacements of stabilized riverbank at the critical SRF value (Phase 2)

Required rockfill column lengths vary depending on the column location in the slope. The columns typically penetrate a stiff till layer to completely eliminate failures in the soft clay mass. Riverbank failure mechanisms are expected to differ from those in engineered embankments. The primary difference between embankments and natural slopes such as those commonly encountered in riverbanks is the way shear resistance is mobilized in the columns. In the case of embankment applications, the applied vertical stress from the weight of the embankments themselves can enhance the normal effective stress in the column. In the case of riverbank stabilization, the normal stress at any point in the column relies mainly on the weight of the column materials above that point.

Having established the similarity of estimated FS from LEM and FEM, LEM was used in this study for the evaluation of stability of stabilized riverbanks. FEM was used to investigate the displacements of the riverbank. The Mohr-coulomb failure criterion was used for LEM analysis of riverbank stability, while an elastic-perfectly plastic constitutive model was used for FEM. The effect of the number of stabilized rows, the diameter and spacing of columns, and the column location relative to the slope crest was investigated. Each combination of these parameters resulted in a different failure surface and computed factor of safety. As mentioned earlier in this chapter the analysis of a natural riverbank was showing unstable condition, in which the potential shear failure plane could pass through the weakest clay zone at the clay till interface.

Figure 6.19 illustrates, that the vicinity of the centre of the potential shear failure is the best position for rock columns to maximize the factor of safety. These results confirm the conclusions by Hassiotis et al. (1997) that the optimum location of reinforcement is at mid-span of the potential shear failure, to achieve the maximum stability.

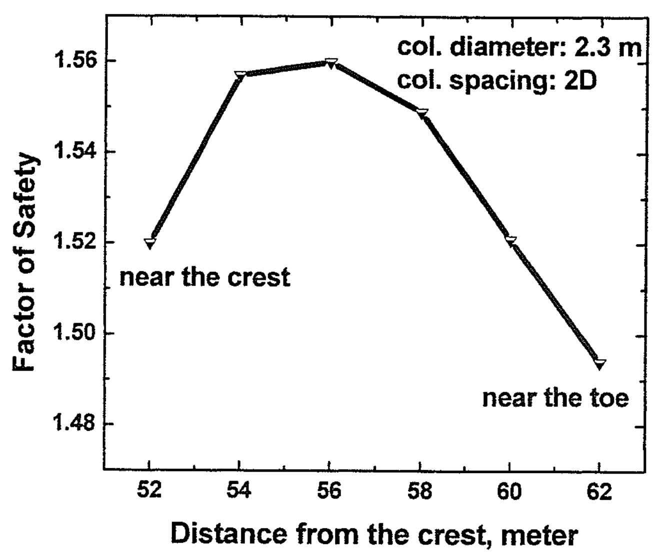


Figure 6.19 Effect of columns location on the estimated factor of safety

The influence of the number of reinforced column rows on the calculated factor of safety has been studied as shown in Figure 6.20. The factor of safety increases with the number of rows of columns as expected. Five rows of columns were chosen as the base case of this study since this gives a generally acceptable factor of safety equal to 1.5 (using LEM). The rows were positioned at their optimum location; that is, at the mid-span of the potential failure surface. Finally, the rockfill column diameter has been investigated as shown in Figure 6.21. The factor of safety increased proportionally with increases in the column's diameter.

When the spacing between rows of rockfill columns is altered, the location of the potential slips failure changes. This is significant since for the same number of column rows but with a closer spacing, this provides higher shear strength and therefore factor of safety. The effect of the ratio of the clear distance between columns (S2) to the centre to centre distance (S1) has been investigated (see Figure 6.22 for parameter definition). As shown in the Figure 6.23, the lower the spacing between column rows (lower S2/S1 ratio), the higher resulting factor of safety.

The FEM with SSR technique was used to compare the mode of failure for the S_2/S_1 ratio of 0.30 to that of 0.60. As shown in Figure 6.24, at the lower value of S_2/S_1 the columns work as a group, while at the higher ratio the columns behave individually as illustrated in Figure 6.25. This is demonstrated by the higher concentration of shear strain in the vicinity of the columns for $S_2/S_1 = 0.3$ compared to that for $S_2/S_1 = 0.6$. Another finding observed from FEM analysis is

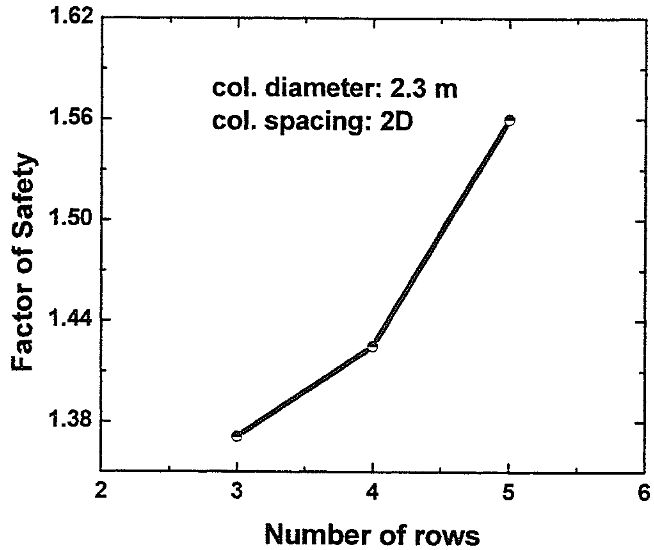


Figure 6.20 Rockfill column's rows versus factor of safety

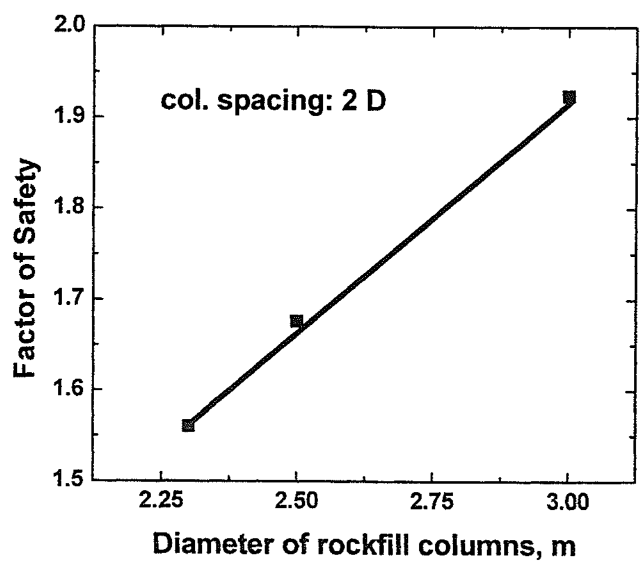


Figure 6.21 Rockfill columns diameter versus factor of safety

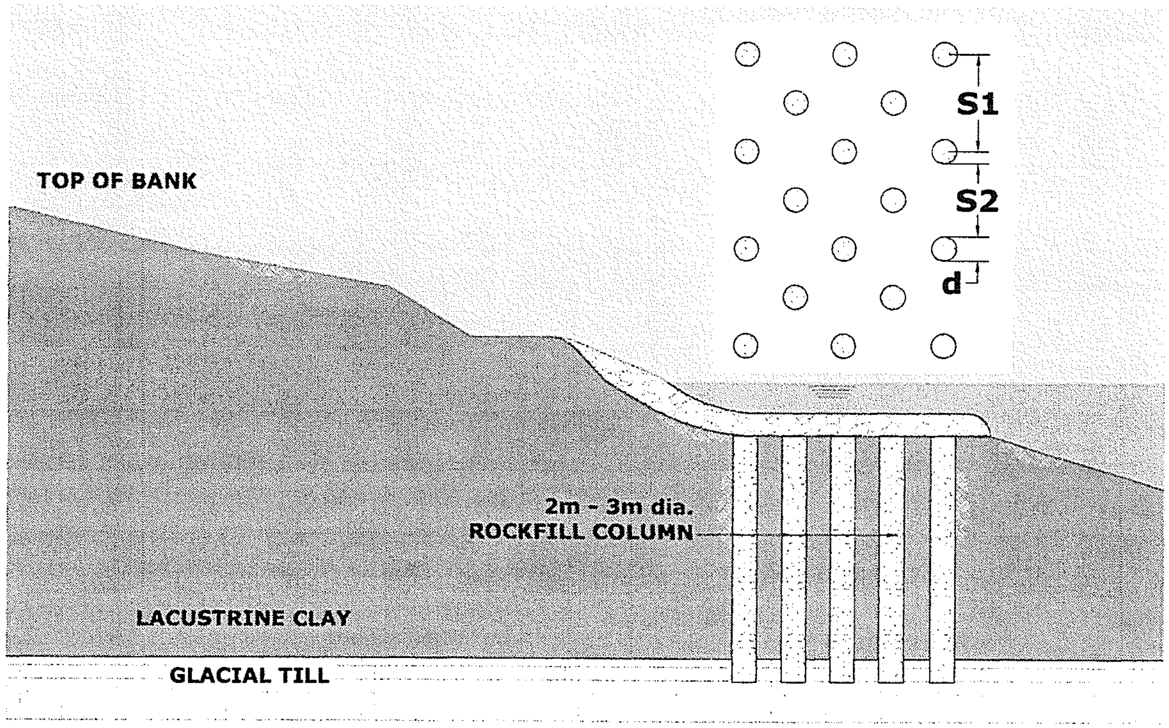


Figure 6.22 Rockfill columns pattern along riverbank

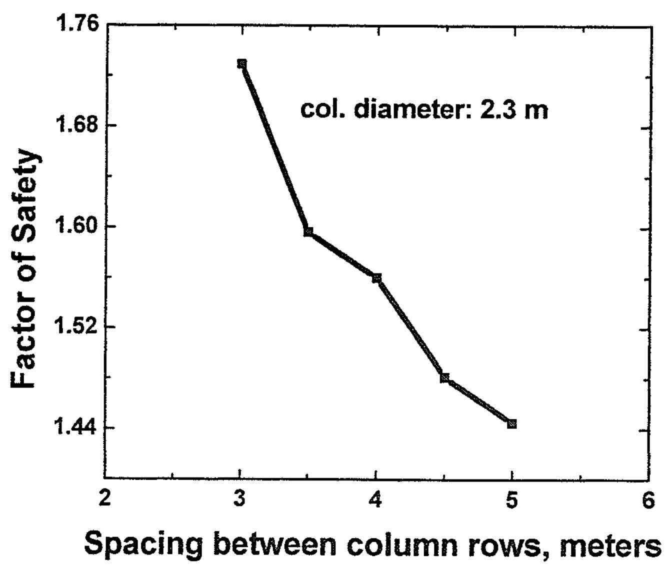


Figure 6.23 Spacing between rows versus factor of safety for a rockfill column diameter of 2.3 meters

that the calculated maximum shear strain is reduced with the closer spacing of rows. This is important as lesser slope displacements are required to mobilize the necessary shear resistance for stabilization for closer rockfill columns.

6.7 Effect of low level cementation in rockfill columns for riverbank stabilization

Recent observations for stabilized riverbanks in Winnipeg have shown that movements occurred following installation of rockfill columns. Uncertainty regarding the magnitude of movements required to mobilize shearing resistance in rockfill columns has been a topic of discussion among local geotechnical engineers. Moreover, the amount of movements to generate sufficient shear resistance in the rockfill columns may not satisfy the serviceability requirements of the structures adjacent to the riverbanks. Accordingly, there is a need to improve the performance of these columns to reduce the magnitude of riverbank movements required to mobilize sufficient shear resistance of rockfill columns. As described previously, small amounts of Portland cement were added to rockfill materials in an attempt to improve their strength properties. The laboratory testing results showed that, the added cement increased bonding between rockfill particles. This resulted in increased cohesion between the rockfill particles. As demonstrated in Chapter 4 and from the results reported by Juran and Riccobono (1991), there was a positive effect of low level cementation on the mobilization of shear resistance in compacted sand. This section describes the

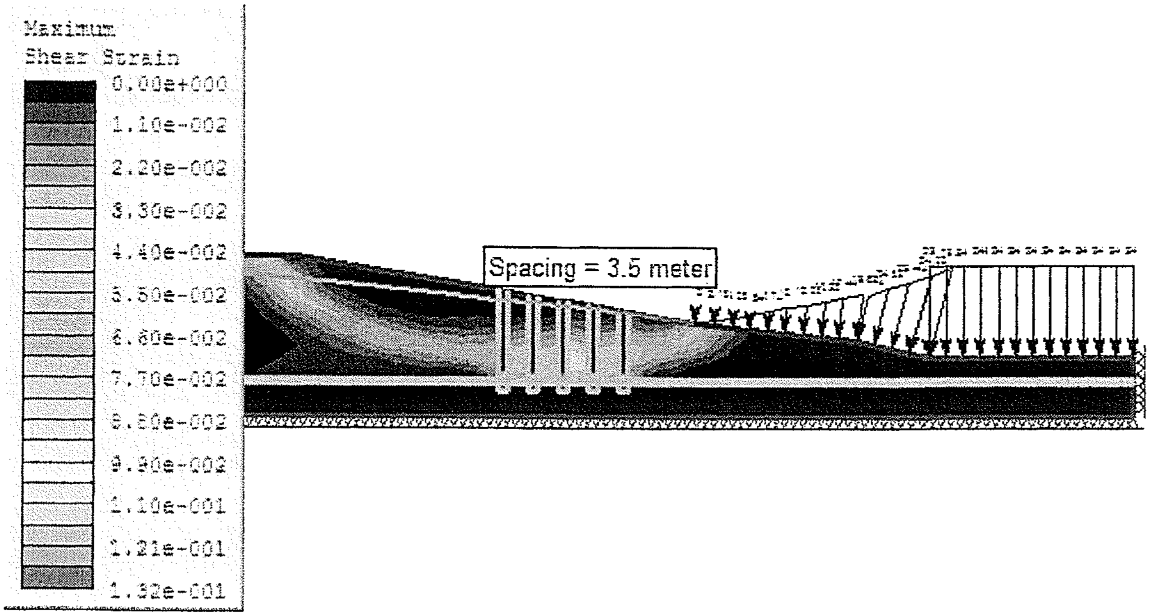


Figure 6.24 Maximum shear strain for the spacing ratio of S_2/S_1 = 0.30

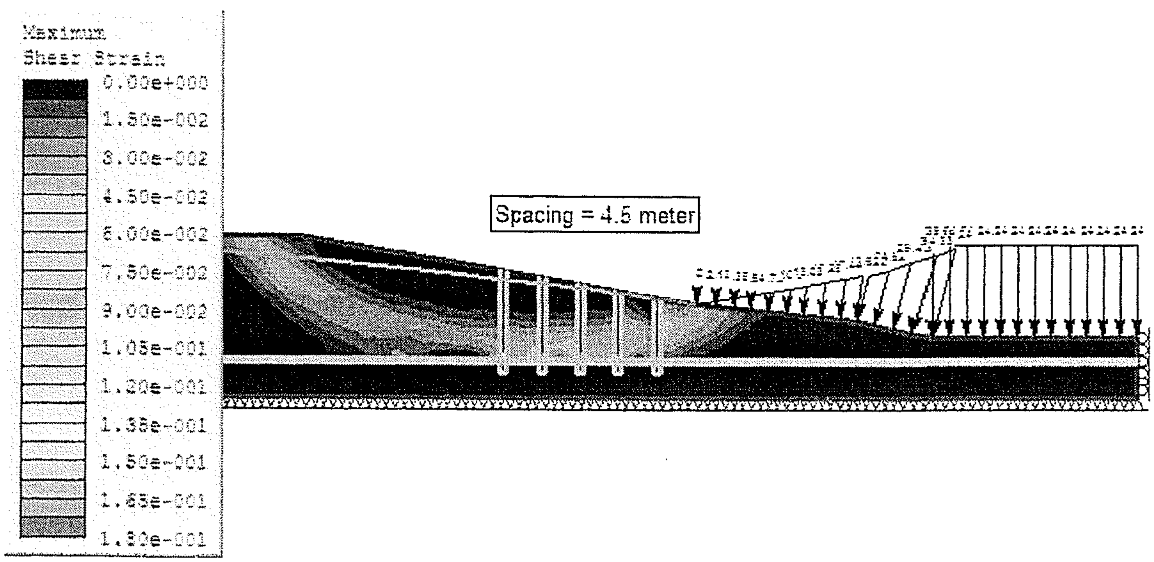


Figure 6.25 Maximum shear strain for the spacing ratio of S_2/S_1 = 0.60

results of numerical investigations on the effects of adding small amount of cement in rockfill columns for riverbank stabilization.

The shear parameters derived from laboratory test results (Chapter 4) were used in numerical modeling. The finite element method (FEM) was used to assess the performance of cemented and uncemented rockfill columns in the stabilization of riverbanks. Phase 2 (computer program) was used to conduct the FEM modeling using SSR technique. The settings for the numerical model were identical to the ones used in previous sections (see Section 6.3). For simplicity in the analysis, the constitutive relationships used for clay and rockfill materials are linear elastic with Mohr-Coulomb yield criterion. Values of equivalent Young's modulus, E, and Poisson's ratio, ν, in combination with shear strength parameters are required to compute the stresses and deformations in the riverbank. Thus the same parameters used in previous sections will be used here as well. The soil properties used in this section are summarized in Table 6.2, as determined from large-scale laboratory and conventional direct shear tests.

Table 6.2 Soil properties of the riverbank soils and cemented rockfill columns

Type of soil	E MPa	ν	γ kN/m^3	ϕ degrees	c kPa
Clay	5	0.41	17	15	4
Weak clay	3.5	0.41	15.7	12	3
Cemented rockfill column	30	0.20	19.0	50	90
Till layer	170	0.20	22.0	50	0

A typical geometry of Winnipeg riverbank (Figure 6.1) has been used in the numerical investigation. The factor of safety using LEM increased to 1.909 with the installation of similar diameter of cemented rockfill columns placed in five rows near the mid-span of the potential slip surface, as shown in Fig. 6.26. The potential displacements of the riverbank stabilized with this configuration of cemented-rockfill columns are shown in Figure 6.27. The addition of low level cementation had increased the factor of safety significantly. However, there was only a slight reduction in displacements in the cemented model (maximum displacement of 8 cm) as compared to the uncemented case. This is attributed to the fact that there is no significant difference in terms of the measured Young's modulus at this particular cement ratios compared to the uncemented rockfill.

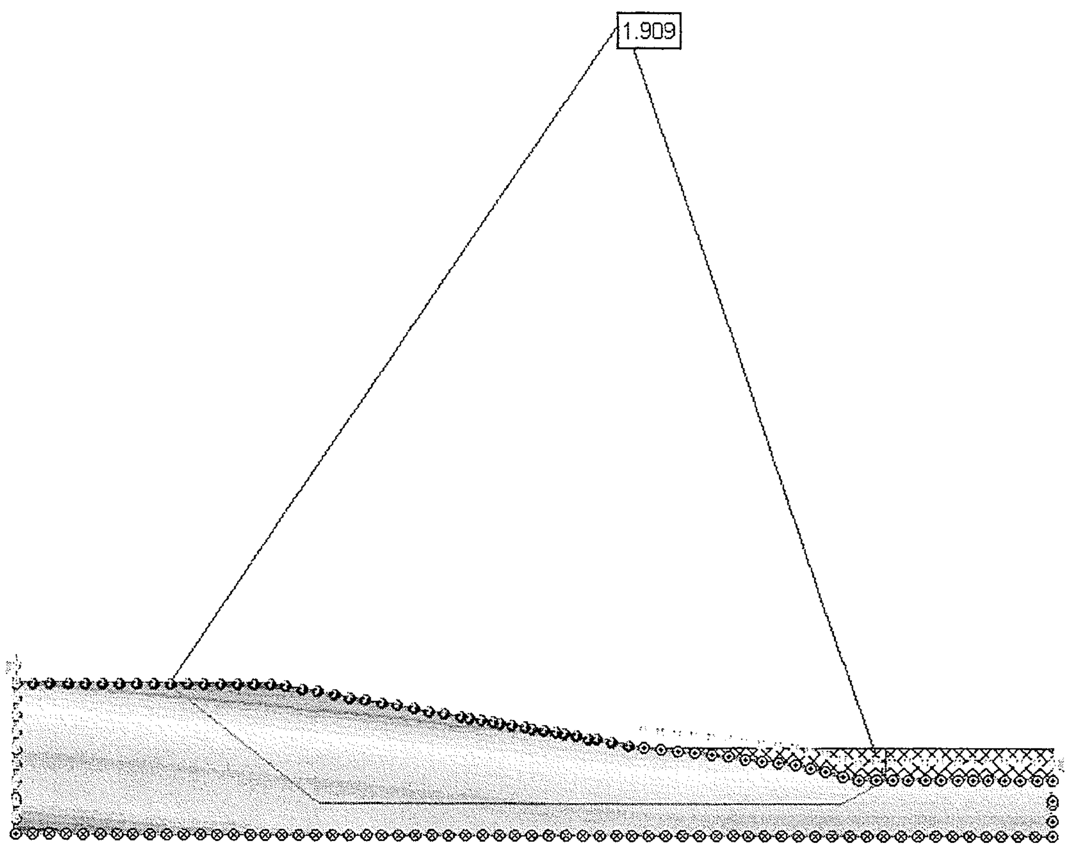


Figure 6.26 Stability of riverbank stabilized with cemented rockfill columns using LEM

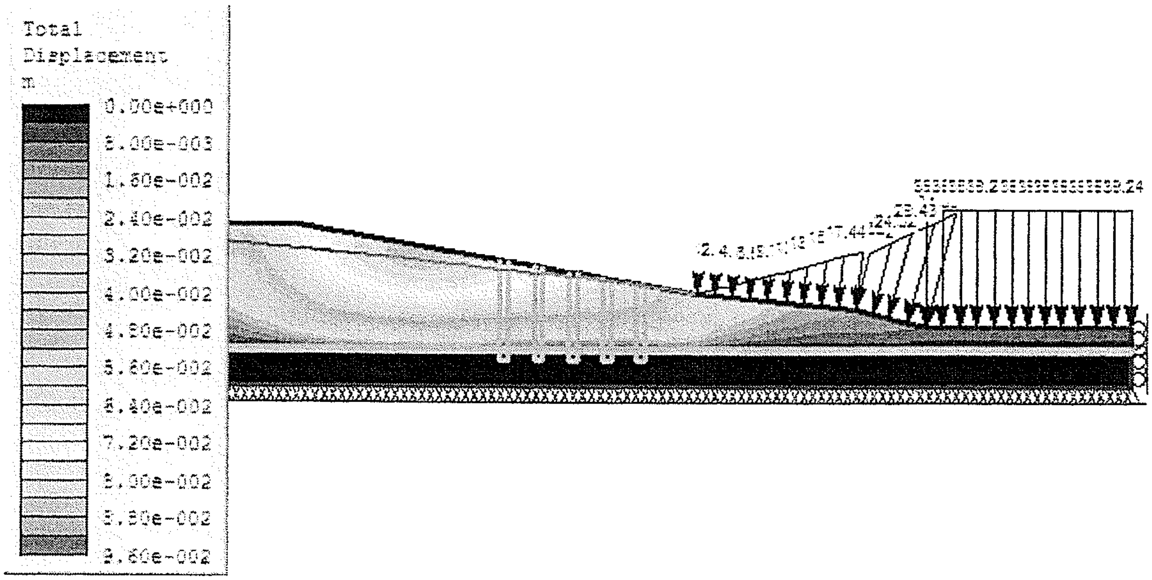


Figure 6.27 Total displacement of cemented-rockfill columns stabilization using Phase 2

6.8 Numerical analysis of soil-cement columns

An alternative method of strengthening riverbanks is the use of soil-cement columns. One of the advantages of using soil-cement columns is that they tend to provide higher shear stiffness and shear resistance compared to other techniques. Furthermore, soil-cement columns provide more flexibility in configuration patterns as will be shown later in Section 6.8.3. The analysis of the riverbank stabilized using soil-cement columns was performed using both FEM-SSR and LEM. These methods are used to determine the factor of safety and estimate the general stability. The shear strength reduction method (SSR) has the ability to obtain stresses and deformations of piles (Duncan 1996). The SSR method and column group analysis were used to evaluate effects of soil-cement columns and to evaluate the effect of different geometry patterns on riverbank stability. The first pattern involves columns in triangular patterns installed in 5 rows of staggered layout. The second pattern was more innovative, using the design concept found in Chapter 4 where the first upper row of columns in the slope mobilizes much higher shear resistance than the lower level row of columns. In this pattern, the soil-cement columns in the first row were arranged immediately adjacent to each other (wall type) to prevent the surrounding clay from squeezing between them. This is defined here as "wall type". Moreover, they are connected by a 1 metre thickness of cement stabilized clay functioning as a cap. This continuous cap is used to connect the top of individual columns together and also to act as a relieving platform for the clay under surcharge loads (Nicholson et al. 1998).

Andromalos et al. (1999) stated that in order to design soil-cement columns, strength characteristics are required to be determined or estimated, such as unconfined compressive strength, Young's modulus and shear strength of both the native soil and the soil-cement columns. The material properties such as unit weight, cohesion, friction angle, and shear stiffness were determined from laboratory unconfined compression test and unconsolidated undrained (see Chapter 5). Young's modulus and Poisson's ratio were obtained as described earlier in Section 6.3.1.

6.8.1 Analysis of stabilized riverbank with soil-cement columns

Nicholson (1999) found that high strength soil-cement columns behave as piles. Duncan (1996) and Hassiotis (1997) have pointed out that piles can be used successfully to reinforce slopes. The basic concept of stabilizing riverbank using soil-cement columns is to provide stronger element and higher stiffness than the ambient native soils, and to act as a block (a group of columns acting as one body) to resist driving forces. To achieve this target, soil-cement columns have been designed to extend into an underlying till layer with a high shearing resistance. Consequently, failure is forced through the soil-cement columns and failures that undermine the reinforced block are avoided. It does not mean, however, that the stability of the riverbank should not be checked with respect to the potential shear failure plane passing below the soil-cement reinforced zone.

The engineering properties of both native soils and soil-cement columns, shear resistance, relative stiffness and diameter of soil-cement columns, area replacement ratio, moment capacity of the columns, shearing resistance of the underlying soil layer, and geometry and layout of the columns are all contributing factors to the effectiveness of soil-cement stabilization of riverbanks. The main purpose of the numerical analyses carried out in this study was to evaluate the factors that influence the overall stability and associated displacements of riverbank stabilized by soil-cement columns. At the same time, this study compares the performance of soil-cement columns and rockfill columns in stabilizing riverbanks.

The soil properties used in this study are shown in Table 6.3, which were obtained from laboratory tests for the materials concern. A two-dimensional plane strain model was used in this analysis. The Mohr-Coulomb failure criterion was used to model the strength of the soil-cement columns and the surrounding native soils, consistent with the available constitutive models used in SSR finite element analysis. The rows of singular cemented columns have been represented by continuous strips of soil-cement materials running parallel to the direction of the river. Five rows of 2.3 m diameter columns were used at the optimum mid-slope location on the riverbank. The centre-to-centre spacing between columns was set at space equal to twice the column's diameter, 2D.

Table 6.3 Material properties of the native clay and soil-cement mixture

Type	E MPa	ν	γ kN/M^3	ϕ' degrees	c' kPa^2
Clay	5.0	0.41	17	15	4
Weak clay	3.5	0.41	15.7	12	3
Till layer	170	0.20	22	50	0
Soil-cement	120	0.33	16.5	30	215

6.8.2 Analysis results

As discussed previously in Section 6.4, the typical natural riverbank shown in Figure 6.1, indicated unstable condition. Therefore, stabilization measures are required. When reinforcing with soil-cement columns the calculated factor of safety for the riverbank using finite element method and utilizing SSR was 1.86 as shown in Figure 6.28. These results show a significant improvement in the computed factor of safety. The associated displacements were also reduced as shown in Figure 6.29, yet they were found to be similar to the values obtained from riverbank stabilized with rockfill columns (refer to Figure 6.15). As in rockfill columns, the lateral displacements of the soil-cement columns located uphill

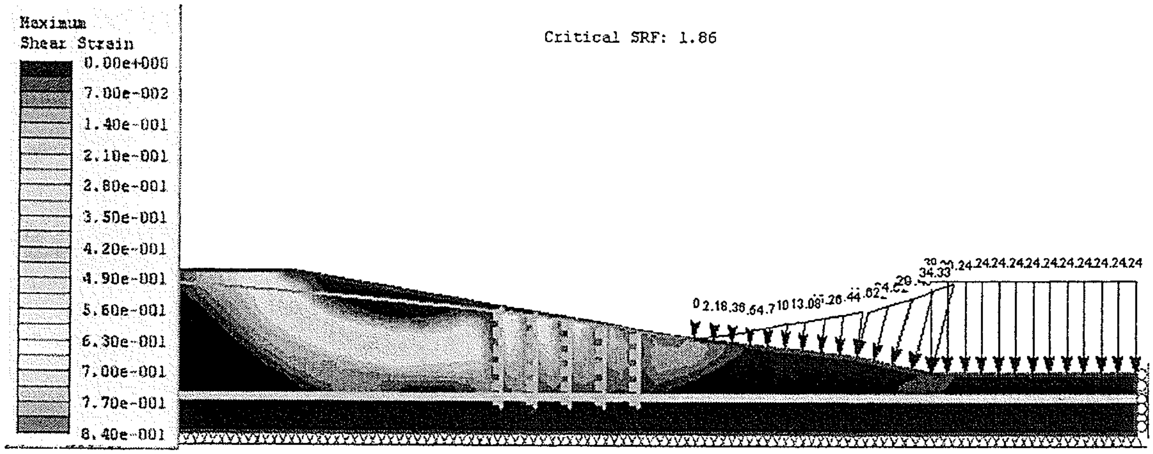


Figure 6.28 Stability analysis of riverbanks stabilized with soil-cement columns

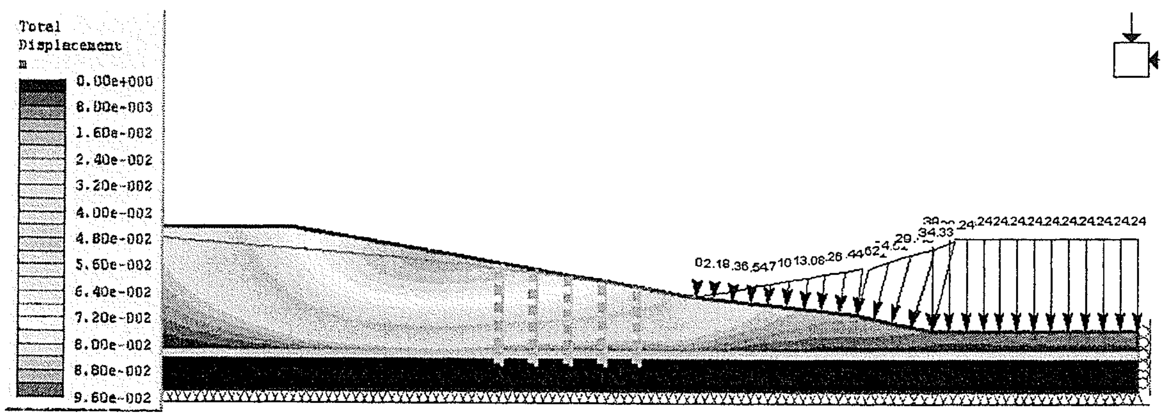


Figure 6.29 Total displacements of soil-cement columns stabilized riverbank

(near the crest side) showed higher displacements than the columns located downhill (near the toe side of the slope) as shown in Figure 6.30. These results agree with a similar observation of natural slopes reinforced with piles as reported by Gao et al. (2003), that showed progressive mobilization of column (pile) resistance. To optimize the mobilization of all the columns in the riverbank, a layer of the same soil-cement mixture was constructed on top of the columns to act as a beam (cap) in order to connect individual columns and make them work as a group, as shown in Figure 6.31. The design arrangement can only be done on soil-cement columns that have bending stiffness. Consequently, mobilization of shear resistance of all columns became almost simultaneous. The placement of this beam enhanced the factor of safety to 1.91 (Figure 6.32). The design achieved two important goals. First, the wall type of soil-cement columns was helpful in preventing the clay soil from squeezing in between and around columns a problem that is associated with cemented rockfill columns as they can not be installed tangent or secant to each other. Secondly, it reduced maximum deformation because of near uniform mobilization of shear resistance in all columns (Figures 6.33).

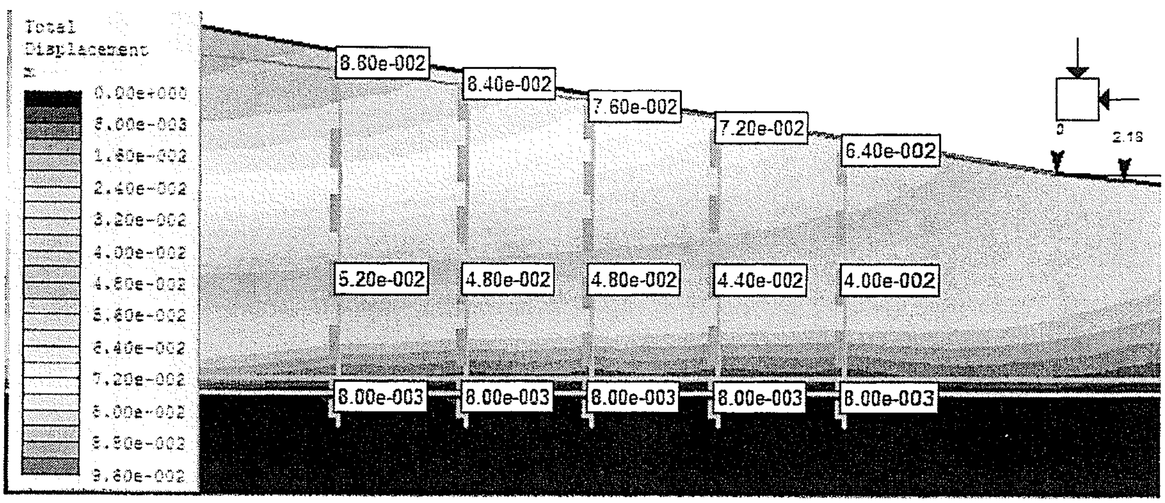
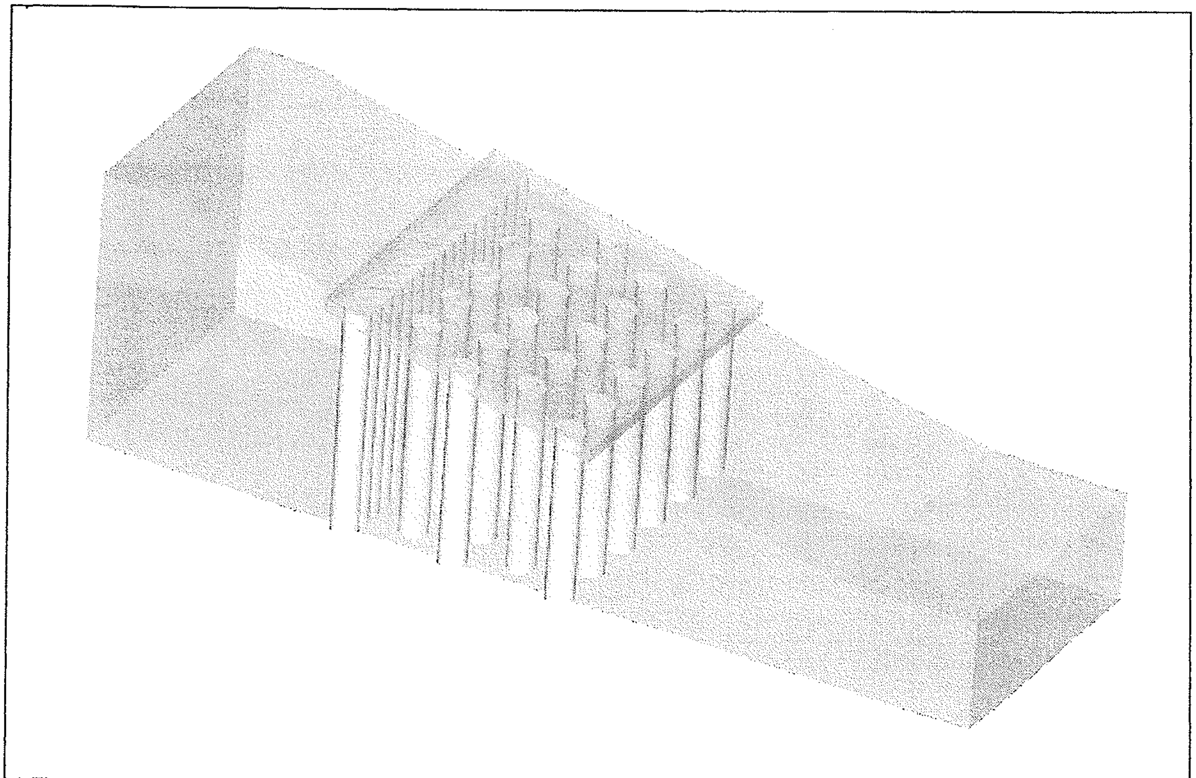


Figure 6.30 Lateral deformation of soil-cement columns



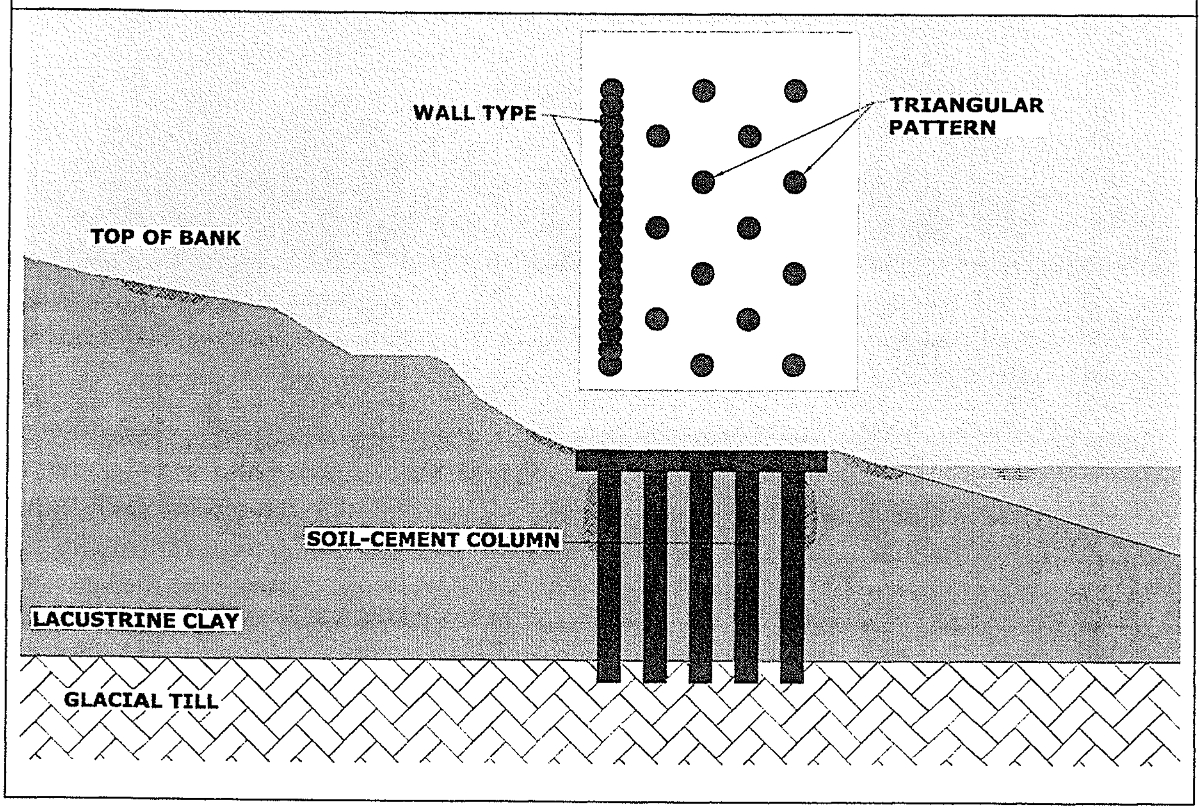


Figure 6.31 Stabilization of riverbank of soil-cement columns with blanket

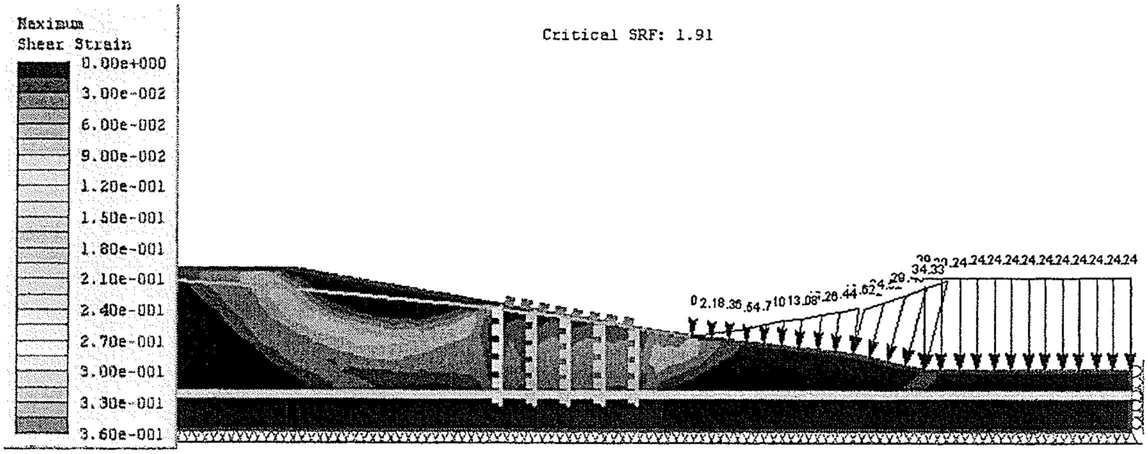


Figure 6.32 Stability of stabilized riverbank of soil-cement columns with blanket

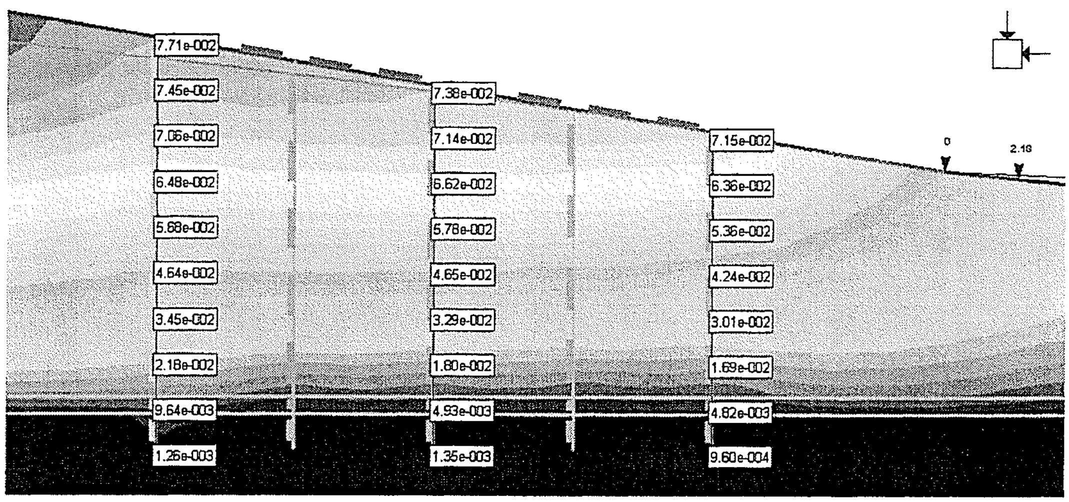


Figure 6.33 Lateral deformation of soil-cement columns along reinforced columns after blanket of soil-cement construction

6.8.3 Effect of different configuration of column layout on the performance of the stabilized riverbank

One of the major concerns in the design of columnar inclusions, whether they are soil-cement or rockfill columns, is the behaviour of the surrounding soil and how it relates to the behaviour of the reinforced elements (columns) or vice-versa. The design layout and configuration should be such that the compatibility between the

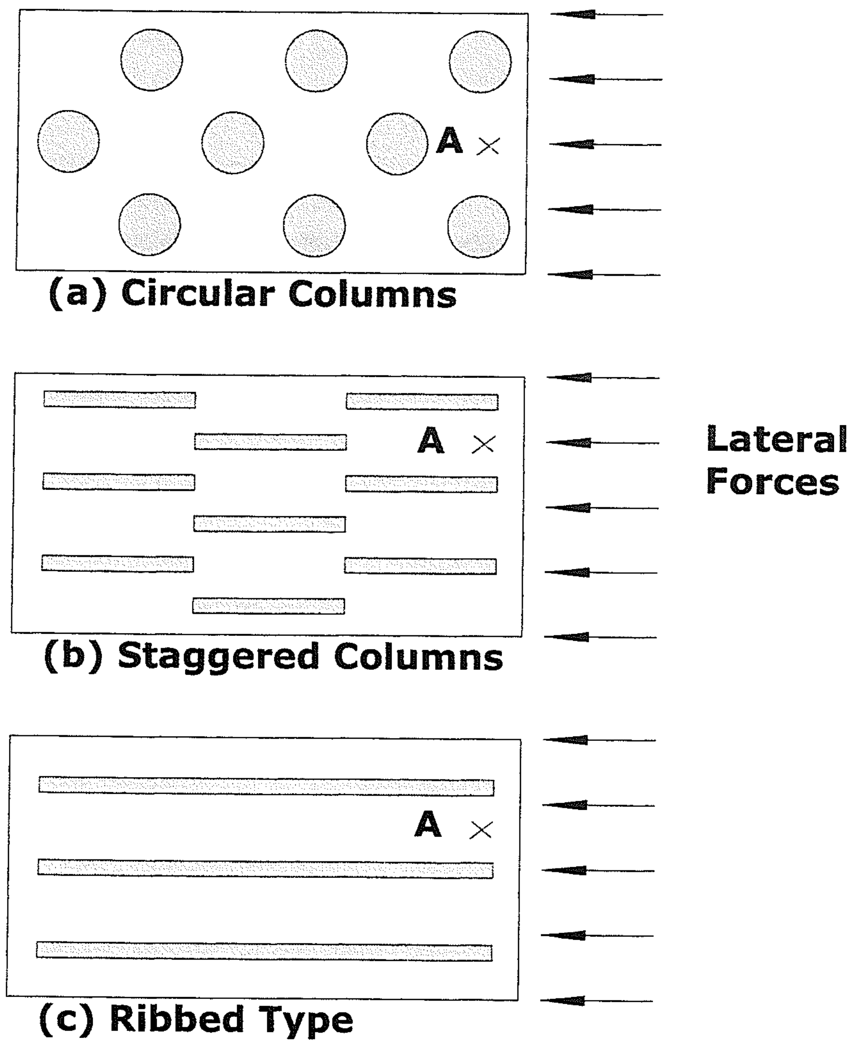


Figure 6.34 Effect of different configuration on the lateral displacements

native soil and the columns is enhanced, so they behave interactively as a composite material. The other concern is that, clay squeezing around the columns due to lateral earth pressures should be avoided. The finite element method (Phase 2) has been used to study the influence of different configurations of cemented rockfill columns on the calculated displacements corresponding to lateral loading as shown in Figure 6.34. Figure 6.34 shows a horizontal section (top view) passing through three different configurations with the same area replacement ratio: a) column groups, b) staggered ribs, c) continuous ribs (ribbed type). The engineering properties of the soil-cement material assumed in this analysis are the same as in the previous section (Table 6.3). In this particular analysis the soil-cement columns were assumed fixed, and the surrounding soil was allowed to displace (represented by the displacement of point "A") during the application of horizontal pressure. The pressure-displacement relationships of these configurations are presented in Figure 6.35. It is clear that the clay shows greatest resistance to squeezing in the case of column groups arranged in a triangular pattern,. Moreover, ribbed-configuration provided the least resistance to clay squeezing as shown in Figure 6.35.

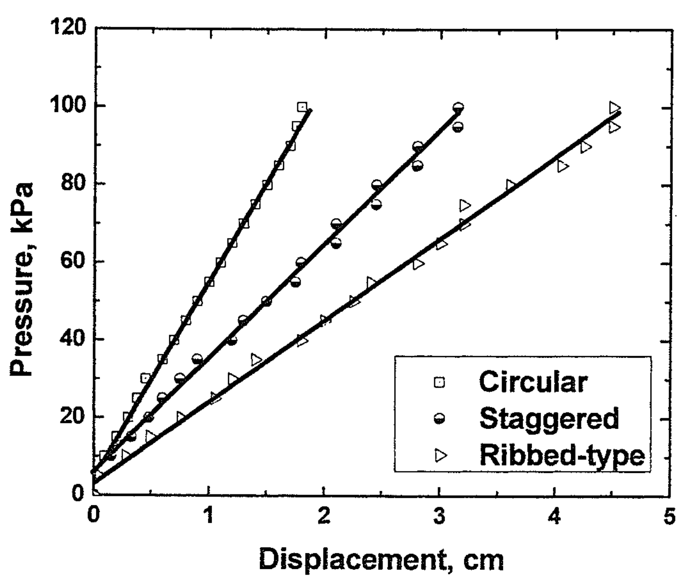


Figure 6.35 Displacements at the riverbank stabilized with cemented rockfill columns using FEM

6.8.4 Influence of the strength characteristics of till layer on the stability of stabilized riverbanks

Soil variability may influence the mode of failure of the typical riverbank. One of the factors is the strength characteristics of the glacial till layer. Previous studies (Baracos et al. 1978, Baracos et al. 1983a, and Baracos et al. 1983b) reported the characteristics of the till layer in Winnipeg as shown in Table 6.4.

Table 6.4 Geotechnical properties of the glacial till in Winnipeg (After Baracos et al. 1983 a and b)

Geotechnical properties	Typical Range
Unit weight (moist)	23.6 kN/m^3
Unit weight (dry)	22.0 kN/m^3
Pressuremeter modulus	170 – 240 MN/m^3
Residual angle	31°

The residual angle of shearing resistance was about 31° (Baracos et al. 1978). However, glacial tills in some areas are soft, clayey, water-laid tills which have unconfined compressive strengths under 50 kPa (Baracos 1960). Therefore, the stabilized riverbank should be checked with respect to potential shear failure planes that pass below the tip of the column and through the potentially weak till.

In this Section, a potential failure plane passing below the tip of the column is investigated. To achieve this condition of failure mode, lower till properties than those presented in the Table 6.4 are assumed.

Limit equilibrium methods (using Slide 5) have been used to investigate the influence of till layer parameters on the general riverbank stability. The analysis showed that using friction angle values less than 31° resulted in a significant reduction in the calculated factor of safety, while values above 31° had a limited influence on the calculated factor of safety. The factor of safety was as low as 1.35 for a till friction angle of 20° as shown in Figure 6.36 and a factor of safety at 1.56 for friction angle of 31°. Figure 6.36 shows that the potential failure plane can pass through beneath the columns.

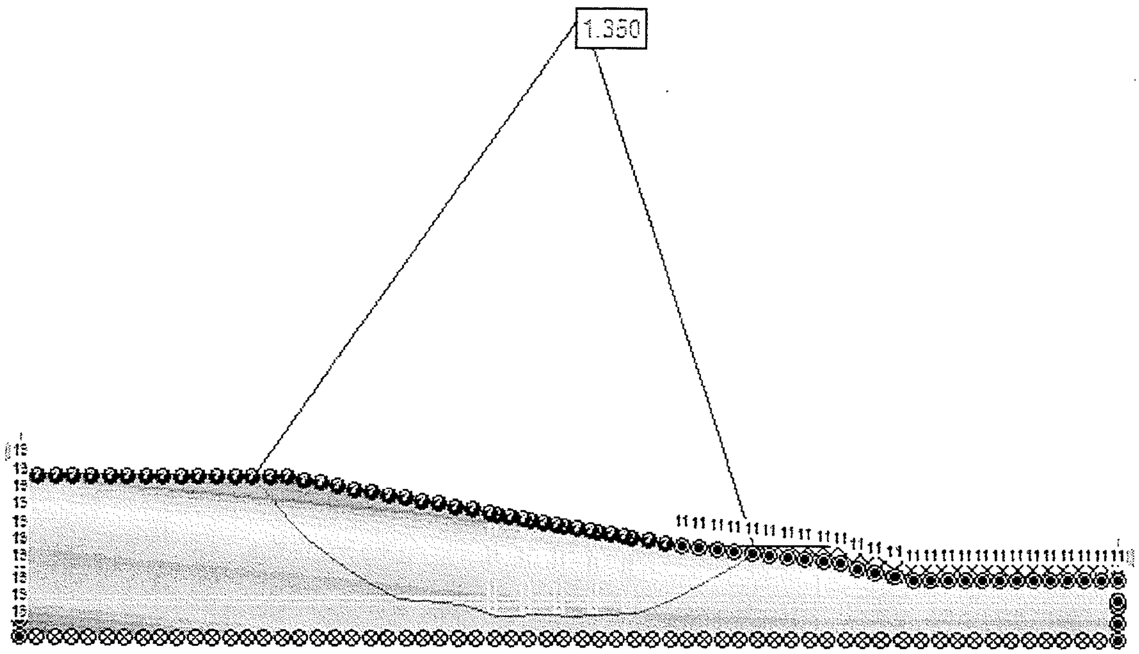


Figure 6.36 Effect of till layer strength characteristics on the mechanism of failure

6.8.5 Influence of the presence of silt lenses on the stability of riverbanks stabilized with rockfill columns

Previous studies reported the presence of lenses of soft silty soils or silty sand within the riverbank clay deposit (Baracos et al 1983). Moreover, during spring time, the rainfall and snow melting will generate water flow through the zone of silty sand. Abramson (2002) reported that a stratum of silty fine sand could be an aquifer if it were confined between clay strata as shown in Figure 6.37. This aquifer zone could provide a preferential seepage path into the rockfill columns, and possibly cause silt infiltration. It should be noted that these lenses of silty soils may have less effect on the stability in the case of a natural riverbank. However, these lenses might play a major role after installation of rockfill columns. Firstly, silt infiltration into the rockfill material may weaken the rockfill column matrix. Secondly, the removal of silt particles from the original silty zone may result in a very loose weak layer, and cause a progressive failure mechanism.

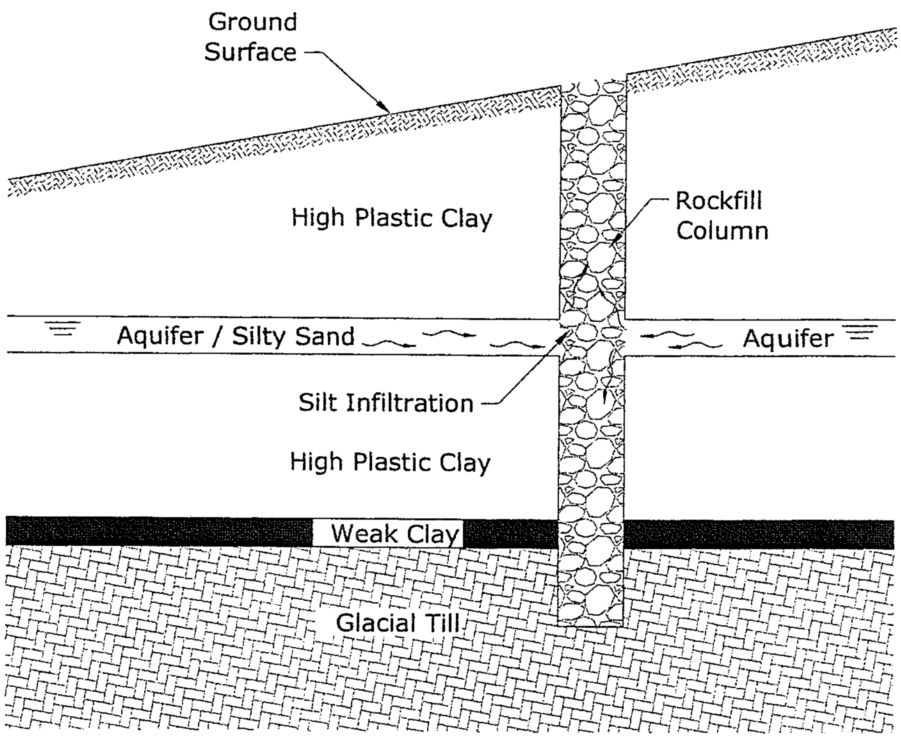


Figure 6.37 Riverbank with lenses of silty soils

It is difficult to simulate the above-indicated condition. The purpose of this simulation is to evaluate how migration of silt particles into the rockfill columns might reduce the stability of the stabilized riverbank. Here, the silt lenses assumed to be located at the mid-height of the rockfill columns and were also simulated as a weak layer at the clay-till interface (c' = 3 kPa and ϕ' = 12°). Moreover, the migration of silt into rockfill columns did take into account the contamination of rockfill materials with the silt. This is simulated by reducing the angle of internal friction of the rockfill material to 36° from 50°. This reduction needs to be examined by laboratory tests and requires further investigations. Figure 6.38 shows that for the lenses at 5 meter below ground surface, the factor of safety had dropped drastically to 1.19.

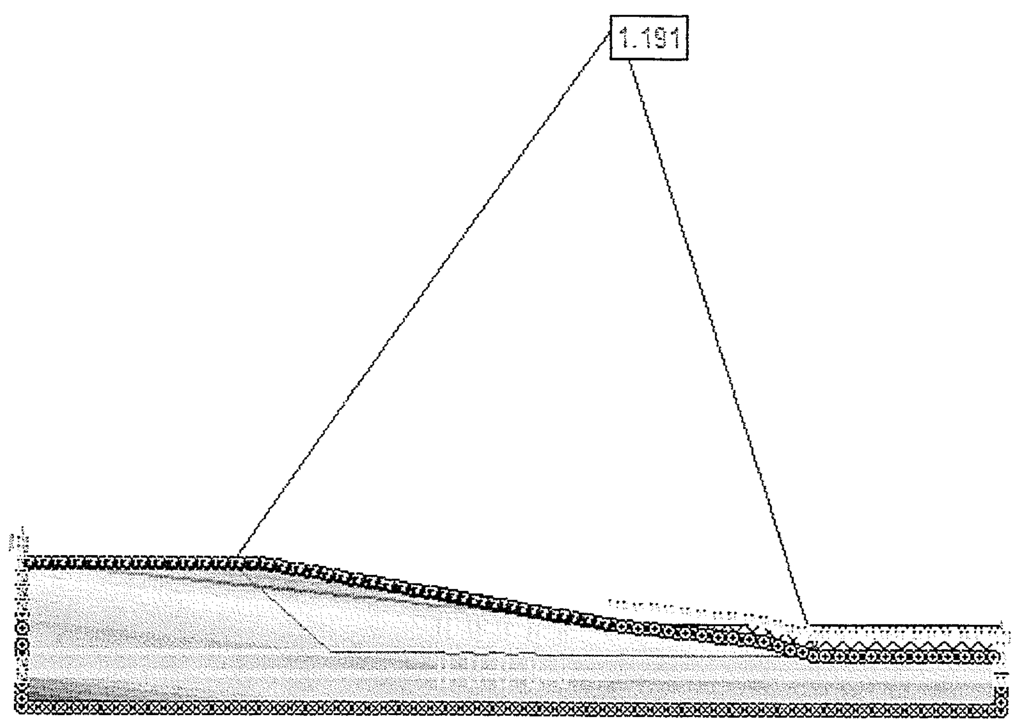


Figure 6.38 Effect of lenses of silt layer on the failure mechanism at rockfill columns riverbanks

6.9 Summary

(1) Limit equilibrium methods can be used to successfully evaluate the stability of natural and stabilized riverbanks. Additionally, the finite element method with the strength reduction method can also be used to evaluate riverbank stability. The use of the finite element method to estimate displacements associated with the stabilization of riverbanks was also assessed. The results seem reasonable in comparison with the measured displacements in one of the rockfill column stabilized riverbanks. However, further verification is required to improve confidence in the numerical model and the assumptions made.

(2) Stability analysis shows that the optimum location for a group of columns will be in the vicinity of the centre of the potential shear failure.

(3) Columnar inclusions will increase the factor of safety significantly, and the potential shear failure will move from the zone of weak clay at the clay-till interface upward into the clay zone.

(4) The centre-to-centre spacing between columns plays a major role in increasing the factor of safety and reducing the associated displacements. Furthermore, large spacing between columns is inefficient because it may allow clay to squeeze through between columns.

(5) This study has generally assumed that the installation of rockfill columns into a stiff till layer will force the potential shear failure to pass

through the columns. However, variation in till strength characteristics have shown a proportional reduction in the calculated factor of safety when the till friction angle is reduced below 31°.

(6) Existing lenses of silty soils may cause piping and ultimately lead to another failure mechanism.

(7) Cemented rockfill columns increase stability and reduce the associated displacements required to mobilize the required shear resistance when compared to uncemented rockfill columns. Moreover, the addition of cement mixtures in the rockfill may inhibit the intrusion of silt particles from the surrounding soil.

(8) Soil-cement columns can be an effective alternative in stabilizing Winnipeg riverbanks as they increase the average shear strength significantly, and reduce the associated displacements. Moreover, joining the soil-cement columns by a continuous top cap causes them to behave more as a group, and thus results in more effective performance.

Chapter 7

EVALUATION OF KEY FACTORS AFFECTING THE STABILITY OF STABILIZED RIVERBANKS USING DOE

7.1 Introduction

Various factors can affect the stability of stabilized riverbanks. These factors are evaluated using a more structured statistical analysis called the Design and Analysis of Experiments (DOE). Application of the basic idea and methods contained in DOE have increased greatly over the recent years, and it is safe to claim that it is one of the most important concepts in applied statistics (Anderson and Whitcoub 2000, Clarke and Kempson 1997, Mason et al. 1991, and Montgomery 2005). The first application of DOE was in agricultural research, and was quickly extended to industrial research. Developments have since spread to many other fields of study (Montgomery 2005). DOE is excellent for screening many factors in order to identify the vital few. They often reveal interactions that would never be found through one-factor-at-a-time method. Two-level factorials are efficient, producing maximum information with a minimum number of runs.

DOE is being used in this study to evaluate key factors affecting the factor of safety at the stabilized riverbank. In this study, the following factors or variables are being evaluated:

1) Number of columns
2) Diameter of the columns
3) Spacing between columns
4) Stiffness ratio (shear stiffness of column relative to the surrounding clay)
5) Slope height
6) Slope angle

These variables are studied for rockfill columns. It is envisaged that the method of statistical analysis will help determine the factors (and interactions of multiple factors) that contribute significantly to the performance of stabilized riverbanks. Statistical analyses will be conducted using a commercially-available statistical program called Design Ease 6.0.11 [4] that runs on a personal computer. The following procedures have been followed in this study:

1) Assign total numbers of effects (independent variables) that need to be studied, n.

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[4] State-Ease, Inc. 2005

2) The number of observations required is 2^n. This means for five effects, there will be $2^5 = 32$ observations required for the two level factorial design.
3) For each of these 32 observations, there is a specific value for each effect, and there is a single response value (e.g., factor of safety).
4) The main effect and interactions between effects will be studied. A prediction equation for the response (e.g., factor of safety) will be obtained.
5) In case the target response is not yet achieved, an improvement to the formula can be made to hit the target.

7.2 Basic Statistics for DOE

The analyses of experiments are conducted using many statistical terms. These terminologies will be briefly described and related to the parameters relevant to this study.

Mean value

Mean is calculated by adding up the response (e.g., factor of safety) and dividing by the number of observations, n.

[7.1] $$\ddot{Y} = \frac{\sum_{i=1}^{n} Y_i}{n}$$

where, $\ddot{Y}$ is the response, n is the sample size, and i is the individual response.

Degree of freedom (df)

Defined as one less than the number of observations.

[7.2] $df = n - 1$

Deviation

Various (S^2) equals the sum of the squared deviations from the mean, divided by the degree of freedom (df).

$$[7.3] \quad S^2 = \frac{\sum_{i=1}^{n} (Y_i - \ddot{Y})^2}{n - 1}$$

Standard error

It is calculated as shown in the following equation and it is used to manage expectations.

$$[7.4] \quad SE \equiv S\ddot{Y} \approx \sqrt{\frac{S^2}{n}}$$

Confidence interval (CI)

It represents the number of standard deviations from the estimated mean.

$$[7.5] \quad CI = \ddot{Y} \pm t^{*}SE$$

where, t is a factor that depends on the confidence desired and the degree of freedom generated for the estimate of error.

Normality

Normality is a measure of the percentage of individuals expected to fall below a given level, benchmarked in terms of standard deviation. At the benchmark of zero standard deviation, the cumulative probability is 50%, while it is 84% at a value of 1 standard deviation. These rules have been obtained by measuring the proportion of the area under the normal curve that falls below any given level.

Normality or normal distributed data is useful to understand the factors under study and their interactions. Both full-normal plots and half-normal plots can be obtained to check our data using two different templates.

Two-level factorial design

Two-level factorial design involves factors (effects) at two levels. The simple way to describe the number of observations required is through two levels for each factor. Assuming there are two factors A and B. We need to measure the number of observations required. In this case there are $2^2 = 4$ observations required and these will be as follows: (1) low-A & low-B, (2) high-A & low-B, (3) low-A & high-B, and (4) high-A & high-B. The points for the factorial designs are labelled in standard order, i.e. starting with all low levels and ending with all high levels.

In advancing one step further, the effects of each factor will be examined. Moreover the factorial design provides contrasts of averages, thus providing statistical power to the estimated effects.

Main effects

The main effect is a mathematical calculation of average response at the high levels minus average response of the low levels for each single effect:

$$[7.6] \quad \text{Effect (A)} = \frac{\sum Y_+}{n_+} - \frac{\sum Y_-}{n_-}$$

where,

Y = the response (e.g., factor of safety),

n_+ = number of runs at high levels,

n_- = number of runs at low levels.

Let us say we have three factors A, B, and C, the negative sign in the equation below represents the low level of the factor, while the positive sign represents the high level of the factor. So the effects of factor A can be calculated as fallows:

$$[7.7] \quad \text{Effect A} = \frac{A^+ + A^+B^+ + A^+C^+ + A^+B^+C^+}{4} - \frac{A^-B^-C^- + B^+ + C^+ + B^+C^+}{4}$$

The same process is followed in calculating the main effect of other factors (i.e., B and C). The lower the value of the calculated effects means the less effects on the calculated response (insignificant).

Two level factorial design reveals "interactions" of factors. The effects caused by interaction of factors will be investigated. Again for three factors the interactions of AB, AC, and BC besides the interaction ABC can be determined. The level of interaction (e.g., - or +) can be determined by multiplying the signs of the parent terms contributing in that interaction (e.g. level of interaction AB sign = A (sign) * B (sign)).

In this study, a stability analysis of natural riverbank performed by Tutkaluk et al. (2002) was used (see Table 7.1) to illustrate the procedure of determining normality and variance of the factor of safety. The factor of safety was computed with varying four factors: 1) slope angle, 2) slope height, 3) aquifer pressure, and 4) location of ground water table. The results of Tutkaluk's studies have been plotted consistent with the concept described above. The half-normal plot, full-normal plot, plots of residuals, and the analysis of variance (ANOVA) table were obtained.

Full-normal plot

Full-normal plot is less sensitive for selecting small effects (see Figure 7.1). However this method shows the difference between positive and negative effects. The results showed that the effects at the right side of the line represent the positive effects factors, while the effects at left side of the line represent the negative effects factors.

Table 7.1 Factor of safety with various factors (after Tutkaluk et al. 2002)

Run	*Factor-A* Slope H:V	*Factor-B* Slope height (m)	*Factor-C* Aquifer pressure head (m)	*Factor-D* Location of G.W.T "D"	Response Factor of safety
1	6:1	16.00	2.00	20.00	2.03
2	4:1	4.00	6.00	20.00	1.9
3	4:1	4.00	2.00	16.00	2.17
4	4:1	16.00	2.00	16.00	1.45
5	6:1	16.00	6.00	16.00	1.52
6	4:1	16.00	2.00	20.00	1.45
7	6:1	4.00	2.00	16.00	2.85
8	4:1	4.00	2.00	20.00	2.1
9	4:1	16.00	6.00	20.00	1.12
10	4:1	16.00	6.00	16.00	1.12
11	6:1	16.00	2.00	16.00	2.03
12	6:1	4.00	6.00	20.00	2.50
13	6:1	4.00	6.00	16.00	2.69
14	6:1	16.00	6.00	20.00	1.52
15	4:1	4.00	6.00	16.00	2.10
16	6:1	4.00	2.00	20.00	2.78

Half-Normal Plot

The half-normal plot of effects helps in simplifying to see the significant factors. In order to plot the effects, it is required to convert them to absolute values, a more sensitive scale for detection of significant outcomes. The Y-axis of the half-normal plot represents the cumulative probability of getting a result at or below

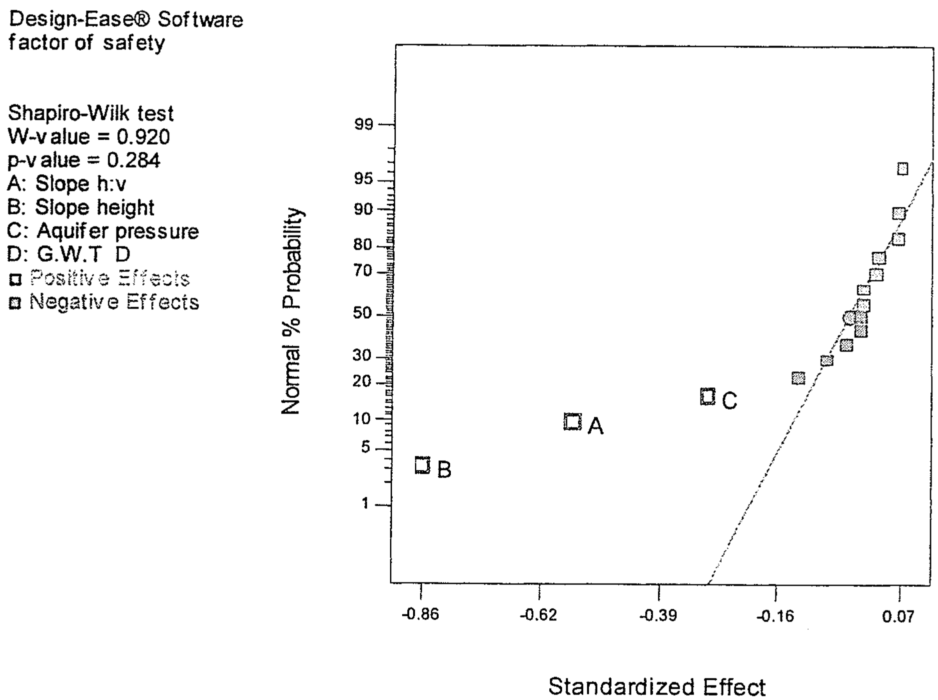


Figure 7.1 Full-normal plot

any given level. The scale of the vertical axis will be adjusted to account for using the absolute value of the effects. The Y-axis is usually divided from 0 to 100% cumulative probability scale into number of equal segments equal to the (number of factor + interactions).

The half-normal plot, is done by plotting the absolute values of the effects on the X-axis versus the cumulative probabilities on the Y-axis using half-normal paper (see Figure 7.2). Half-normal plots can now be generated using statistical software.

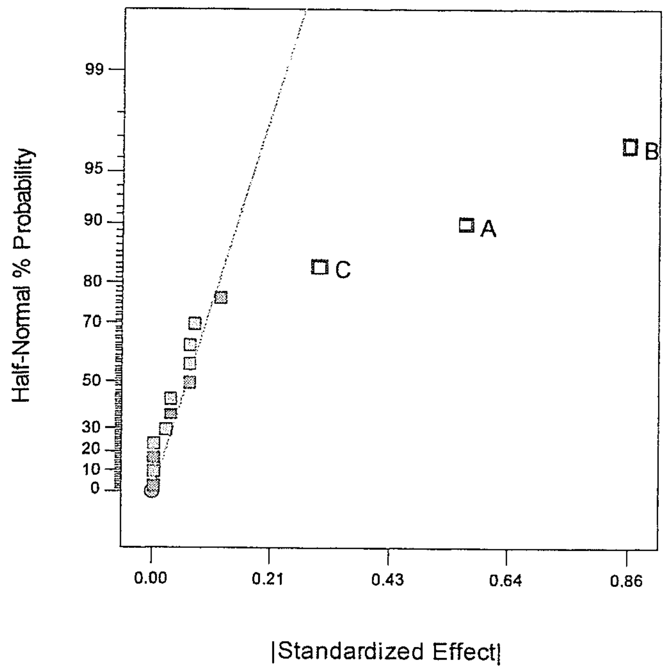


Figure 7.2 Half-normal plot

7.3 Interpretation of the results from basic statistics

Insignificant factors are identified based on the plot shown in Figure 7.1. These are the data points that fall in line with the normality line. These factors will be used later as an estimate of error for analysis of variance (ANOVA). Invariably, the significant factors are data points that are away from the normality.

Analysis of variance

Analysis of variance (ANOVA) can be determined by computing the sum of squares (SS), which are related to the effects as follows:

[7.8] $SS = \frac{N}{4}(\text{Effects})^2$

where, N = number of runs,

Sum of squares for significant factor called SS_{Model} can be calculated as follows:

[7.9] $SS_{Model} = [SS]_{all\,significant\,factors}$

Sum of squares for smaller effects, which fall on the near-zero line, will be pooled together and used as an estimate of error called 'residual':

[7.10] $SS_{Residual} = [SS]_{all\,small\,factors}$

Both sum of squares for model and residuals can be seen in the second column of the ANOVA tables (see Table 7.2 and Table 7.3). The third column in Table 7.3 displays the degrees of freedom (df). The fourth column is the mean square (MS), which was obtained by dividing the sum of squares over the degree of freedom. The fifth column is the F value, which represents the ratio of the mean square (MS) of each effect over the $MS_{Residual}$. The critical F-values can be obtained from the table in statistical books. The F value for this example must be compared to the reference distribution of F at the same degree of freedom (df), that is, equal to 3 for numerator and equal to 12 for the denominator. For this case, it can be seen that the actual F value exceeded the critical value. From the results, it is more than 99% confident that these three factors are significant, which match with the results viewing from the half-normal plot.

The sixth term in the ANOVA table (Table 7.3) is labelled “Cor Total”. This is the total sum of squares corrected for the mean. If ‘Prob>F’< 0.05, it means that the particular factor (effect) is significant.

Table 7.2 The contribution of the effect list

	Term	Effect	SumSqr	% Contribution
Require	Intercept			
Model	A-Slope h:v	-0.564	1.27126	27.02
Model	B-Slope height	-0.856	2.93266	62.34
Model	C-Aquifer pressure	-0.299	0.35701	7.59
Error	D-G.W.T D	-0.066	0.01756	0.37
Error	AB	0.074	0.02176	0.46
Error	AC	0.066	0.01756	0.37
Error	AD	-0.001	0.00001	0.00
Error	BC	-0.121	0.05881	1.25
Error	BD	0.066	0.01756	0.37
Error	CD	-0.031	0.00391	0.08
Error	ABC	0.024	0.00226	0.05
Error	ABD	0.001	0.00001	0.00
Error	ACD	-0.001	0.00001	0.00
Error	BCD	0.031	0.00391	0.08
Error	ABCD	0.001	0.00001	0.00
	Lenth's ME	0.120		
	Lenth's SME	0.245		

Table 7.3 ANOVA table

Source	Sum of Squares	df	Mean Square	F Value	p-value Prob > F	
Model	4.561	3	1.520	127.29	< 0.0001	significant
A-Slope h:v	1.271	1	1.271	106.44	< 0.0001	
B-Slope height	2.933	1	2.933	245.54	< 0.0001	
C-Aquifer pressure	0.357	1	0.357	29.89	0.0001	
Residual	0.143	12	0.012			
Cor Total	4.704	15				
The Model F-value of 127.29 implies the model is significant.						

Predictive equation of factor of safety incorporating significant effects

After testing the model in the ANOVA, a mathematical equation can be obtained. This equation can be used to predict a given response. In this study, the term of the equation on the effect of 3 significant factors (A, B, and C), can be written as follows:

[7.11] $Y = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \beta_3 X_3$

The above equation can be written for the factor of safety in term of coded factors with significant factors as follows; slope angle (A), slope height (B), and aquifer pressure (C) as shown in Equation 7.12:

[7.12] $\text{Factor of safety} = +1.96 + 0.28A - 0.43B - 0.15C$

The value for the (β_0) of 1.96 represents the average of actual responses. The coefficients can be directly compared to assess the relative impact of factors. For this particular case the factor B is causing the bigger effect on the factor of safety than the other factors. The factors in the coded equation can be changed to coded levels (lower level or higher level) in predicting the overall response. The factor of safety can be predicted by substituting the level (- or +) of each factor (A, B, and C).

In terms of actual factors, the final equation is given as:

[7.13] $\text{Factor of safety} = +1.561 + 0.282\,\text{slope} - 0.071\,\text{slope height} - 0.075\,\text{aquifer pressure}$

where, Slope = 6 or 4 to represent 6H:1V for low level, and 4H:1V for high level as shown in Figure 7.3, slope height = 4 or 16 meter to represent low and high levels respectively, aquifer pressure head = 2 or 6 meter to represent low and high levels respectively.

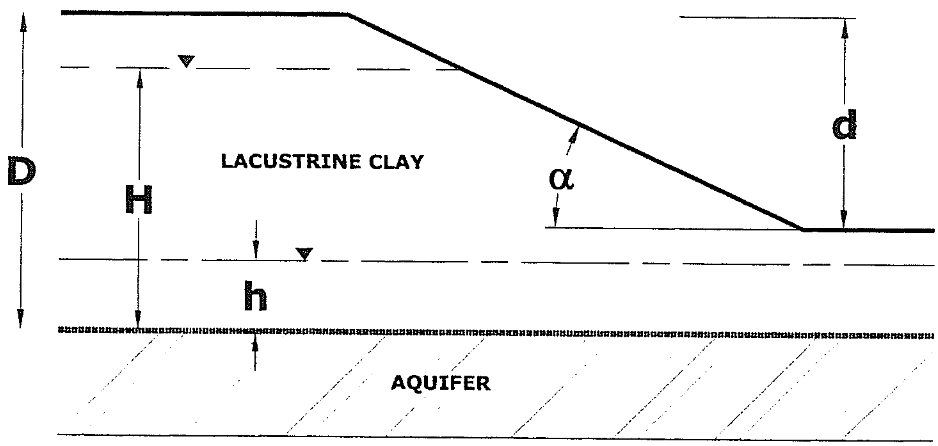


Figure 7.3 Typical of natural riverbank (after Tutkaluk et al. 2002)

Examination of residuals

The statistical assumptions are that residuals are normally distributed and independent with constant variance. So there is a need for checking the validation of these assumptions. It can be done by checking the following plots:

1) Normal plot of residuals,
2) Residuals versus predicted level.

Residuals can be calculated by subtracting the predicted value from the actual value to see the plotted points.

[7.14] $\text{Residual} = \text{Actual value} - \text{Predicted value}$

If the residuals lie on a straight line of relationship then they will be approximately considered as normally distributed as shown in Figure 7.4.

The second recommended checking can be determined from plotting residual versus the predicted values as shown in Figure 7.5. This plot shows a random scatter of the data, in other words constant range of residuals across the graph.

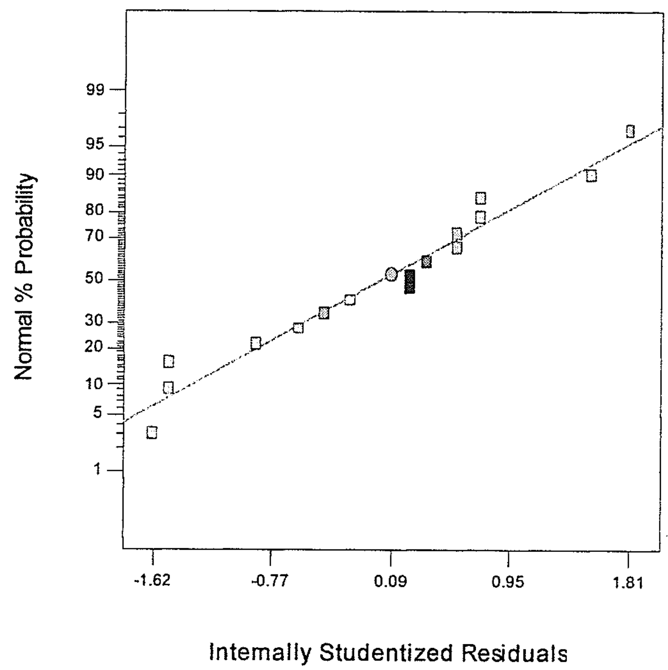


Figure 7.4 Plot of normal percent of probability versus residuals factor of safety

7.4 Application of DOE method in stability analysis of stabilized riverbanks

Design of experiments (DOE) methodology can be used to analyse the numerical results of relationships between the factor of safety and a group of variables for riverbank stabilized with rockfill columns. Statistical analysis and design of two-level factorial design could be applied to the stability analysis performed in Chapter 6 to better understand the overall response of the system.

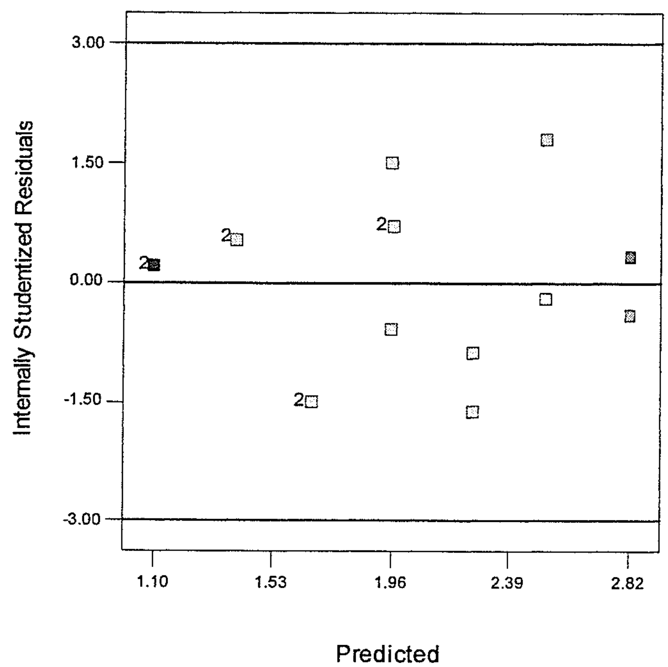


Figure 7.5 Residuals versus predicted factor of safety

Using the factorial design method helps to identify key factors that affect the factor of safety of stabilized riverbank. The engineer can then make reasonable improvements to the design.

Four parameters are selected as variables. These parameters are diameter of rockfill column, number of row of columns, column's strength parameters, and angle of the till as summarized in Table 7.4. There are other influencing variables that are kept as constant during the analysis. As explained earlier in this chapter, two level factorial design involves assigning the low level and high level for each parameter as shown in Table 7.4.

Table 7.4 Low and high level values of the selected variables

Factor	Units	DOE Notation	Low level	High level
Diameter	meter	A	2	3
No. of rows	numeric	B	4	5
Rockfill column characteristics (cohesion)	kPa	C	rockfill	cemented-rockfill
			0	95
Till friction angle	degree	D	31°	40°

The ranges between low level and high level for the variables were based from the field data of typical rockfill columns in Winnipeg. For the columns diameter, both 2 meters and 3 meters have been used in practice. The same is true for spacing, where the distance between columns are typical values used in the field. Two types of reinforced columns parameters have been used for comparison, rockfill columns and cemented/rockfill columns. The other parameter selected in this study is the lacustrine till layer strength characteristics. The till friction angle in the range of 31° to 40° has been applied based on a study conducted by Baracos (1983). Rockfill column characteristics used in this analysis were selected based on the results obtained from laboratory work on rockfill and cemented rockfill materials. Finally, the number of rows required to stabilize a particular riverbank geometry is four and five rows based on the factor of safety values above 1.5.

7.5 Results of two factorial design for stabilized riverbank

Two-level factorial design helps to screen many factors to investigate the key parameter(s) and if there is any interaction between factors. The analysis was done by Design-Ease computer software utilizing the full factorial design method. In order to study the full factorial design of four factors, 2^4 = 16 runs are required to perform the analysis. Perform the stability analysis for each run following the values for each particular run to obtain the response. Table 7.5 shows design layout in standard order including factor values and the corresponding responses obtained. Analysis of various methods (ANOVA) is used to investigate the impact of factors on the obtained factor of safety as shown in Table 7.6. Values of "Prob > F" less than 0.05 indicate model terms are significant. In this analysis columns diameter and type of column whether it is cemented rockfill or not, factors A and C respectively, are the most significant factors. Other factors such as B, D, AD, CD, and ACD, which represents number of rows the till stiffness, and their interactions are also significant. Factors that have "Prob > F" values greater than 0.10 indicate they are not significant. Additionally, the interaction effects and main effects mentioned above have positive effects on the response value. Figure 7.6 shows the positive and negative effects of significant factors (i.e. the factors that lie away from the straight line). Similar results can also be found from the half-normal probability plot that the significant factors are away from the half normal straight line while the non-significant factors fall along the straight line on the half-normal probability plot as shown in Figure 7.7. Another observation is the largest effect is the point farthest away from the half-normal line. Similar

Table 7.5 Influence of several parameters on the calculated factor of safety of stabilized riverbank

Run	*Factor A* Diameter meter	*Factor B* No. of rows No.	*Factor C* Column strength	*Factor D* Till stiffness	Response Factor of safety
1	2	5	rockfill	30	1.53
2	2	5	rockfill	40	1.55
3	3	4	cemented	30	1.71
4	3	5	rockfill	40	1.95
5	2	4	cemented	40	1.71
6	3	5	rockfill	30	1.93
7	3	4	rockfill	40	1.76
8	3	4	cemented	40	2.41
9	2	4	rockfill	40	1.41
10	2	4	rockfill	30	1.41
11	2	5	cemented	30	1.91
12	3	5	cemented	30	1.777
13	2	5	cemented	40	1.909
14	3	4	rockfill	30	1.756
15	2	4	cemented	30	1.708
16	3	5	cemented	40	2.848

conclusion is observed through the ANOVA table for the contributions of all main and interaction factors.

Here, it is important to check the validity of the statistical model. Figure 7.8 shows the normal probability plot of the studentized residuals. Studentized residuals represent the number of standard deviations of the actual values from their respective predicted values. The normal probability plot indicates that the residuals follow a normal distribution. For non-abnormalities (or normality), the relationship will follow a straight line as shown in Figure 7.8.

Table 7.6 The contribution of the effect list

	Term	Effect	SumSqr	Contribution %
Model	A-diameter	-0.10628	0.04518	33.93
Model	B-col.No.	-0.05409	0.01170	8.79
Model	C-rockfill strength	-0.09463	0.03582	26.90
Model	D-Till strength	-0.04902	0.00961	7.22
Error	AB	0.00662	0.00018	0.13
Model	AC	0.02955	0.00349	2.62
Model	AD	-0.04734	0.00896	6.73
Error	BC	0.00227	0.00002	0.02
Error	BD	-0.00677	0.00018	0.14
Model	CD	-0.04614	0.00851	6.40
Error	ABC	0.00320	0.00004	0.03
Error	ACD	-0.00508	0.00010	0.08
Model	BCD	-0.04782	0.00915	6.87
Error	ABCD	-0.00388	0.00006	0.05
Error		-0.00557	0.00012	0.09
	Lenth's ME	0.11394		
	Lenth's ME	0.23131		

Table 7.7 ANOVA table for stabilized riverbank

Source	Sum of Squares	df	Mean Squares	F value	p-value Prob>F	
Model	0.13242	8	0.01655	163.62	< 0.0001	significant
A-Diameter	0.04518	1	0.04518	446.57	< 0.0001	
B-col. No.	0.01170	1	0.01170	115.66	< 0.0001	
C-rockfill strength	0.03582	1	0.03582	354.03	< 0.0001	
D-Till Strength	0.00961	1	0.00961	95.02	< 0.0001	
AC	0.00349	1	0.00349	34.52	0.0006	
AD	0.00896	1	0.00896	88.60	< 0.0001	
CD	0.00851	1	0.00851	84.16	< 0.0001	
ACD	0.00915	1	0.00915	90.42	< 0.0001	
Residual	0.00071	7	0.00010			
Cor total	0.13313	15				
The model F-value of 163.6 implies the model is significant.						

Residuals versus predicted plots are another diagnostic tool to validate the model. Figure 7.9 shows that the size of studentized residual is independent of its predicted value. In other words the spread of the studentized residuals is approximately the same across all levels of the predicted values. It is obvious that the residuals do not reveal any problems, such that the model is validated.

It should be noted that we cannot interpret main effects plots as long as they are involved in interactions. Figure 7.10 shows the effect of interactions of both rockfill column diameter and strength characteristics on the obtained factor of safety. This relationship is assumed linear. The analysis illustrated the major influence of both those factors on the measured factor of safety. However, the interaction of those factors was not much significant. It is clear that increasing column diameter has a similar influence on the rockfill column whether it is cemented or uncemented column.

Interpretation of the interaction of both column diameter and till friction angle, AD, is significant as shown in Figure 7.11. It is obvious that for a stiffer till layer, the improvement in response was significant for increasing the rockfill column diameter. For a less stiff till layer, increasing the column diameter has a limited effect on the calculated response.

It can be noticed that the interaction of both columns type and till stiffness, or CD effect, has a positive effect at the high level of both as shown in Figure 7.12. For stiffer till the increase in rockfill column strength increased the factor of safety

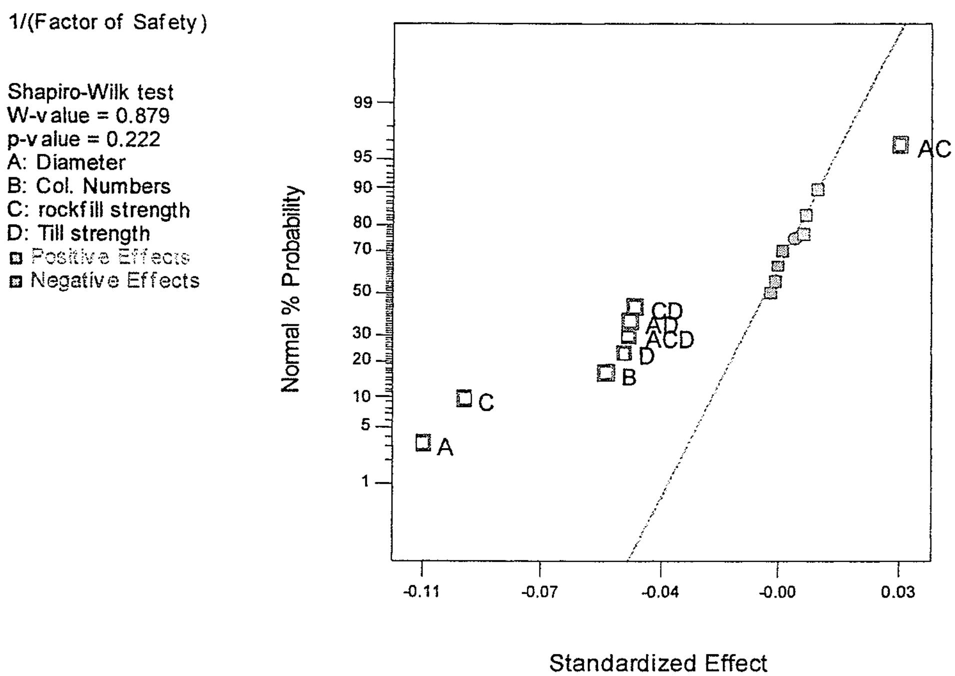


Figure 7.6 Normal plot of stabilized riverbanks

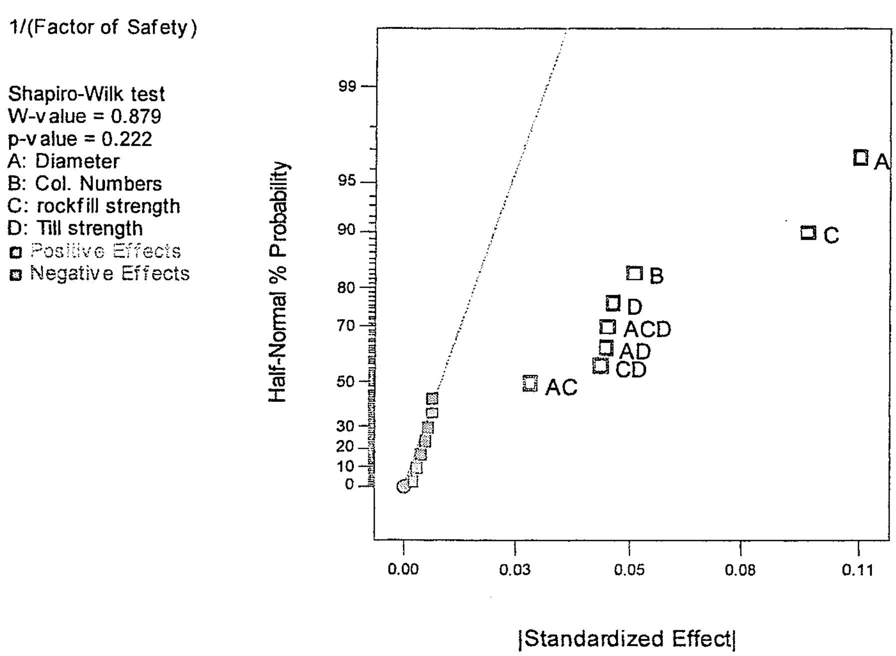


Figure 7.7 Half- normal plot

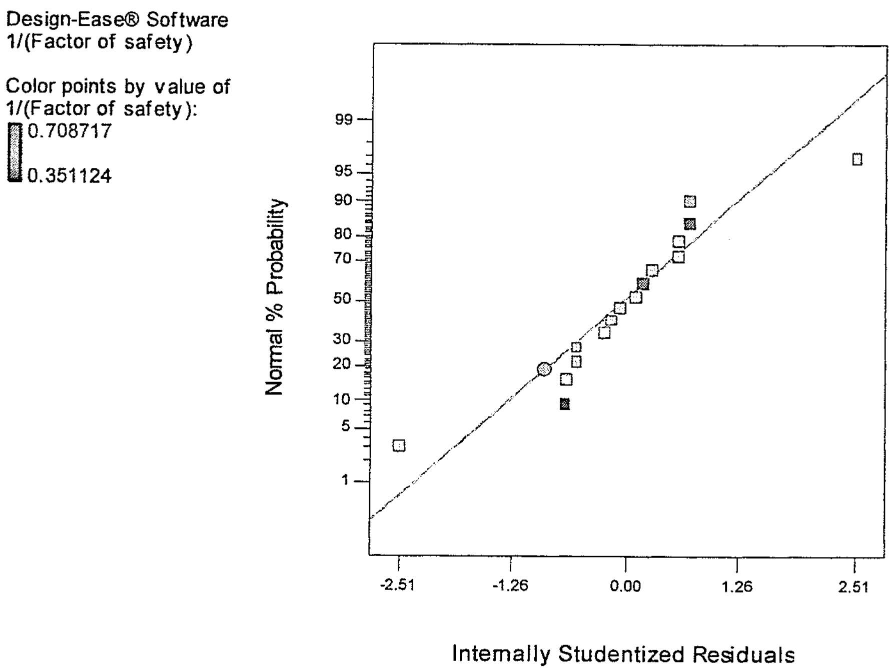


Figure 7.8 Diagnostic the results using studentized result for the normal plot of residuals

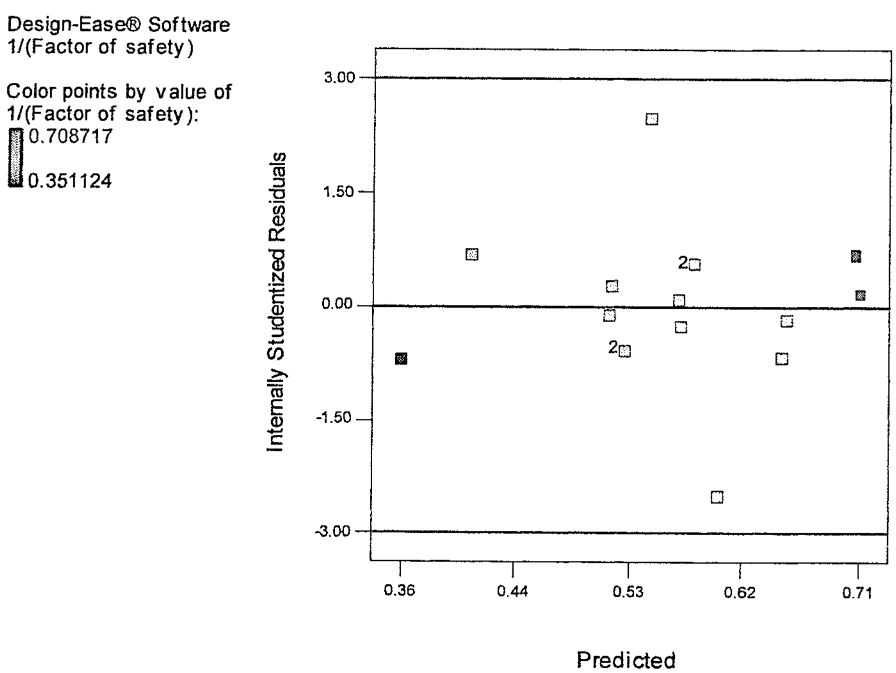


Figure 7.9 Diagnostic the validity, residuals versus the predicted values

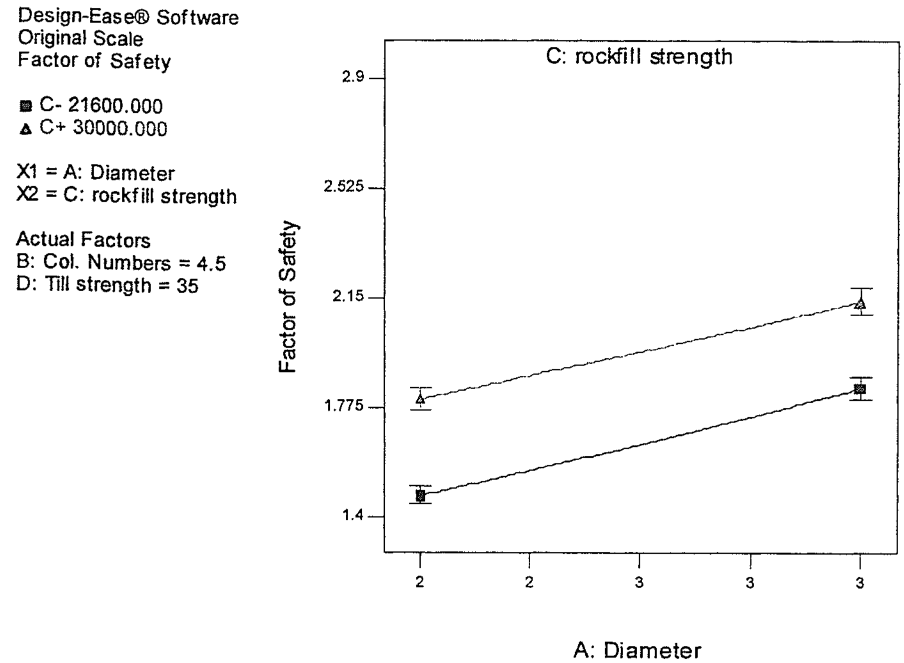


Figure 7.10 Influence of the interaction factor AC versus factor of safety

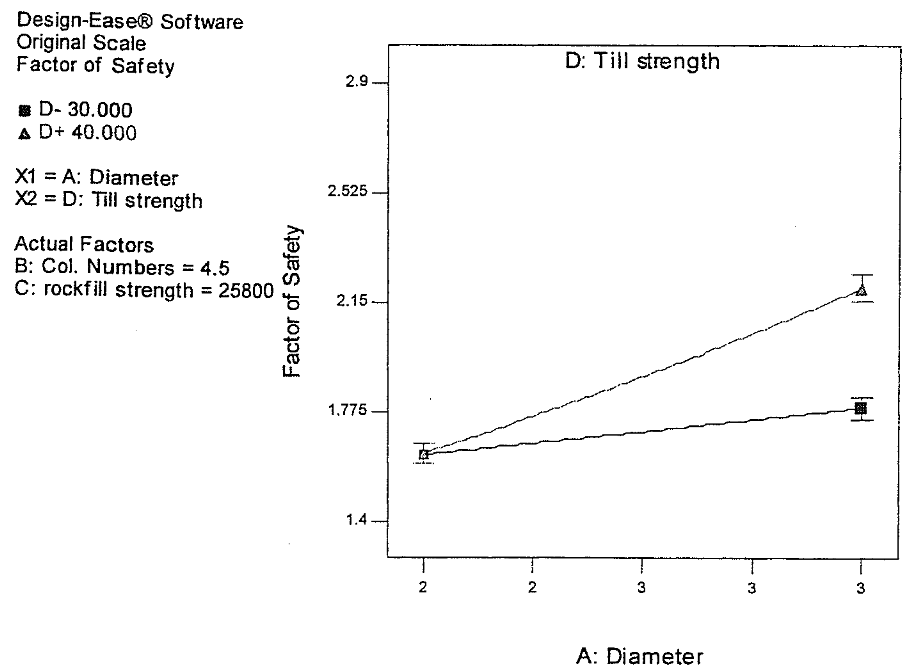


Figure 7.11 Influence of the interaction factor AD versus factor of safety

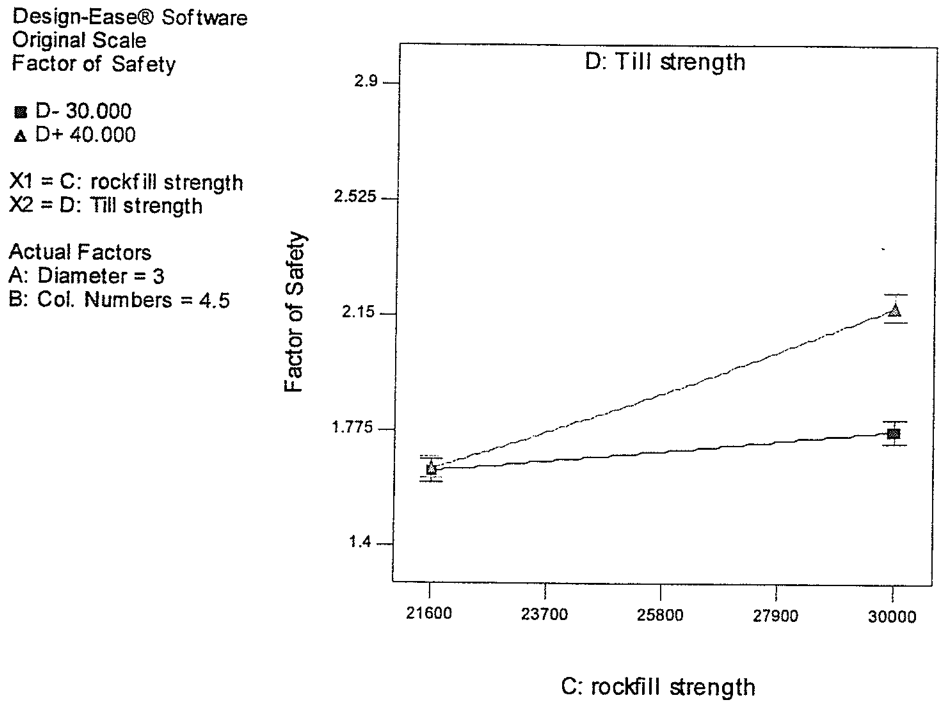


Figure 7.12 Influence of the interaction factor CD versus factor of safety

significantly. At low level of till stiffness, increase in the column strength has a limited effect on the calculated factor of safety.

There is a triple interaction factor of interest that showed a significant effect on the factor of safety, namely column diameter, column type, and the till stiffness, factor ACD. Figure 7.13 shows that till layer stiffness is the key role player for the triple interaction factor. The factor of safety was affected significantly by the till layer stiffness especially in the case of semi rigid columns (cemented rockfill columns). It is obvious that at low level till stiffness (at residual value) the higher value of column's diameter has the same stability result as the low level, as long as the till stiffness is at its low level.

From a geotechnical engineering point of view, the presented results and interpretations are interesting. It is clear that the main effect column diameter (A) is the most significant factor if it was considered individually. While analysis through DOE revealed that, the interaction of columns diameter, column type, and the stiffness of till layer (AD, CD, AC, and ACD) are highly influenced with the till layer stiffness. It is obvious that factor D (till characteristics) is the major role player.

Based on the ANOVA table, the following regression models in terms of coded factors and actual factors are determined:

[7.15] $$\frac{1}{\text{Factor of safety}} = 0.564 - 0.053A - 0.027B - 0.047C - 0.025D + 0.024AD - 0.023CD - 0.024ACD$$

Factor of Safety can be estimated for rockfill columns using actual factors equation:

[7.16] $$\frac{1}{\text{Factor of safety}} = 1.212 - 0.14\,\text{Diameter} - 0.054\,\text{No.of rows} - 0.00053 * \text{Till stiffness} + 9.677\,\text{Diameter} * \text{Till stiffness}$$

Factor of Safety can be estimated for cemented rockfill column using actual factors equation:

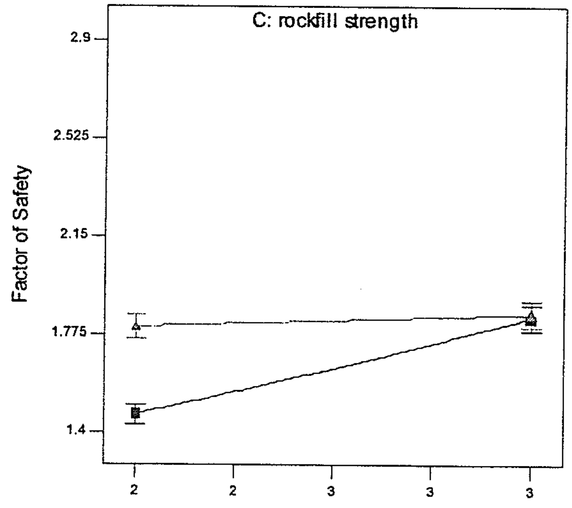
C = 21600 kPa

C = 30000 kPa

Actual factors:

B: col. No. =4.5

D: Till strength = 31.5°



(a) Interaction of ACD factor at 30° of till friction angle

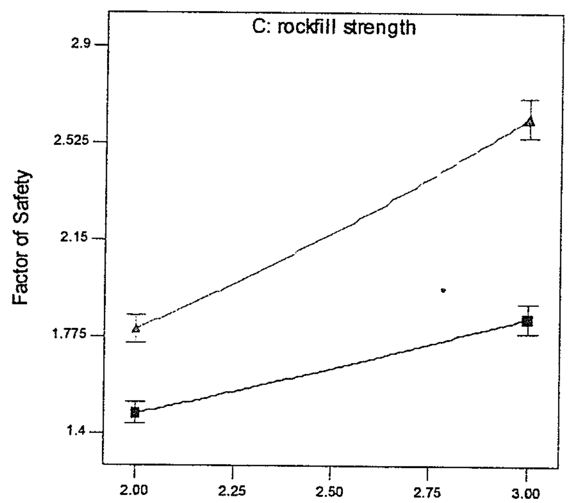
C = 21600 kPa

C = 30000 kPa

Actual factors:

B: col. No. =4.5

D: Till strength = 40°



(b) Interaction of ACD factor at 40° of till friction angle

Figure 7.13 Interaction ACD versus factor of safety

$$[7.17] \quad \frac{1}{\text{Factor of safety}} = -0.381 + 0.59\,\text{Diameter} - 0.054\,\text{No. of rows} + 0.038 * \text{Till stiffness} - 0.019\,\text{Diameter} * \text{Till stiffness}$$

7.6 Summary

(1) The Design of Experiments (DOE) method of statistical analysis can help in interpreting the results of the stability analysis of stabilized riverbanks. Two-level factorial design has been used to recognize key factors. These include the main factors and the interactions between the effects. Factor D (till layer stiffness) proved to be the most significant effect on the response determined.

(2) Regression models in terms of the actual factors and coded factors are determined. It is possible to determine the factor of safety for each design combination (diameter, number of columns row, column strength characteristics, and till stiffness) by using the regression models utilizing actual factor values.

Chapter

8

GUIDELINES FOR DESIGN AND INSTALLATION OF ROCKFILL AND SOIL-CEMENT COLUMNS

8.1 Introduction

Guidelines for proper construction methods are recommended based on the results achieved in previous chapters and from literature. This chapter provides providing step-by-step procedures for the design and installation rockfill and soil-cement columns of stabilized riverbanks.

The causes and the nature of riverbank instability should be understood before selecting the type of reinforcement elements. The choice of ground improvement techniques often hinges on a combination of technical, environmental and economic issues. Mostly there are multiple contributing factors that cause or may cause riverbank instability. So in order for the stabilization technique to perform effectively, it is important to identify contributing causes of riverbank failure. The selected remediation technique depends on the factors that may affect the overall stability, e.g. shape and location of potential failure surface, material strength, hydraulic conditions, presence of aquifer pressure, lenses of silty soils, very weak clay zones, location of the riverbank, and/or others.

Various methods of riverbank stabilization were discussed in order to select the most proper method to stabilize riverbanks in Winnipeg. Among all possible methods, it was found that rockfill columns considered feasible, and may be more economical than other techniques. Rockfill columns are effective for many reasons; they provide a structure with higher shear strength and stiffer than the surrounding soft clay along a potential shear failure surface, and thus contribute to overall stability. They also provide drainage path to relieve the excess pore water pressure of subsurface soils generated after events such as melting snow, heavy rainfall, rapid drawdown, etc. However recent observations of stabilized riverbanks showed some cases of mixed results. Large slope displacements occurred after installation of rockfill columns.

8.2 Construction of rockfill columns in Winnipeg

Rockfill columns are installed to increase the shear resistance of riverbanks. The installation of rockfill columns involves replacing or displacing the native soils with a series of closely spaced cylindrical large diameter columns of well compacted stone, rock, or crushed rock. The rockfill columns generally replace 20-35% of the native soils with cylindrically shaped compact rockfill inclusions, thereby improving the shear resistance (Bachus and Barksdale 1989). Rockfill columns generally improve stability in several ways:

(1) Increase the average shear strength of the composite ground.

(2) Increase the average shear stiffness of the composite ground.

(3) Provide a drainage path that for relieving any excess pore water pressures, thereby increasing the strength of the surrounding soils.

(4) For the case of the displacement type of installation, the columns have the further beneficial effect of increasing the strength of the surrounding soil by densification and increase the lateral stress (Duncan and Wright 2005, Abramson et al. 2005).

The installation method of rockfill columns has a significant impact on their performance. In general, the installation procedure involves replacement or displacement of weak soils with columns of well compacted stones/rockfill that penetrate down to a firm layer. A number of methods have been used around the world. For the purpose of this study, only the installation methods used in Manitoba will be presented. Advantages and disadvantages associated with each method are given.

In general, rockfill columns involve boring a hole beyond of the depth where the maximum shear strain is expected. Often the rockfill columns are anchored to a much stiffer layer to provide better reinforcement of the weak layer. If the hole can provide support by itself, uncased-bored columns are usually used. Otherwise, cased-bored columns are used to protect the hole's sidewall from sloughing into the hole. A mechanical rotary drill is used to auger and extracts the native soil. After completing the hole, the rockfill materials are placed in the hole and a vibrating probe is generally used to densify the rockfill materials. Rockfill columns that are 2.10 and 3.0 m in diameter are usually installed in Winnipeg.

8.3 Recommended design guidelines to stabilize riverbanks in Winnipeg using rockfill columns

8.3.1 Assumptions

To study the influence of rockfill columns on the general stability of riverbanks, some assumptions were made. Firstly, no increase in shear strength and shear stiffness of the interface layer surrounding the columns as a result of densification of rockfill materials because the size of the interface layer has a minor contribution compared to the size of the potential shear plane. Secondly, the column was inserted deep enough inside the stiff till layer to ensure the critical failure surface will pass through the stabilizing columns rather than pass below the tip of the columns.

8.3.2 Proposed rockfill column design and construction guidelines

Figure 8.1 outlines the design and construction of rockfill columns. The detailed procedures are given as follows:

Step 1. Establish the geometry, external loading, topographical conditions, and groundwater conditions. These include the following:

- Slope angle
- Slope height
- Soil type(s) present in the riverbank
- Location of the weak clay zone and its thickness
- Location of water table level
- Aquifer pressure in the till layer, if available, and its variation over a year
- River water level and its variation over a year
- Groundwater conditions including the rainfall condition
- External loads such as buildings and other infrastructure

Step 2. Determine the engineering properties of the in-situ (native) soils. These include the lacustrine clay layer, the weak clay above the till layer, and the glacial till layer as shown in Figure 8.2. Silt lenses and/or other soil types are embedded in the lacustrine clay layer, these should be taken into account in the analysis. The required engineering properties for each soil type include:

a. Strength parameters for both short and long term design S_u, c', and ϕ'
b. Unit weight γ_{sat} and γ_{dry}
c. Young's modulus of elasticity, E
d. Poisson's ratio, ν
e. Hydraulic conductivity, k

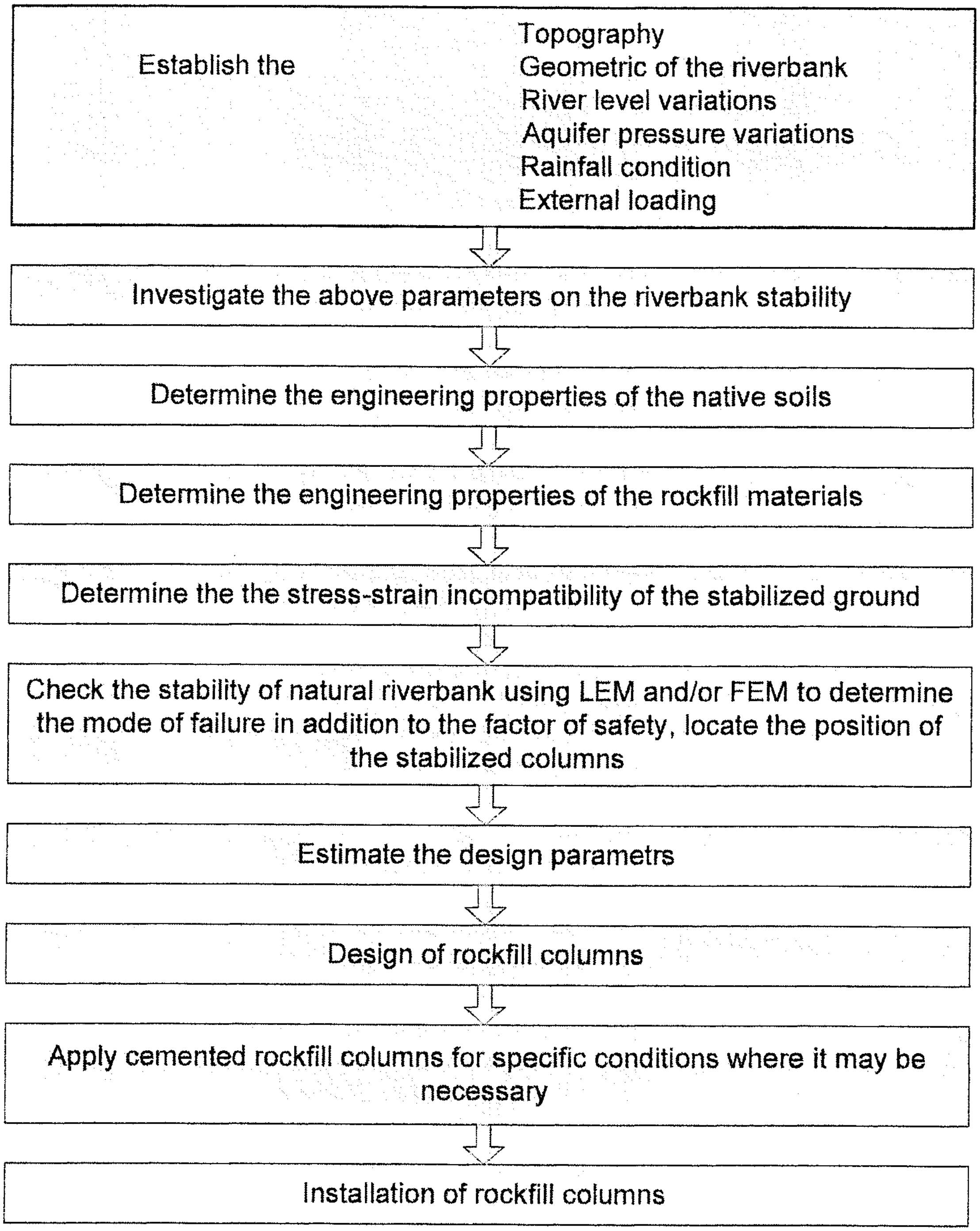


Figure 8.1 Flowchart for design and construction of rockfill columns

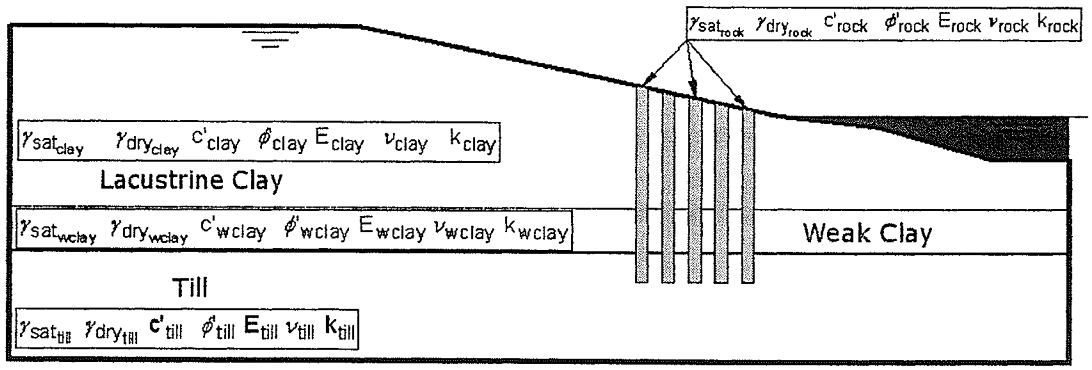


Figure 8.2 The engineering parameters required for modelling

Step 3. Investigate the influence of the variation of the above parameters on the riverbank stability, shear strength and shear stresses. Choose influences that represent the worse case scenario.

Step 4. Determine the engineering properties of the rockfill materials including;

a. grain size distribution

b. relative density

c. effective strength parameters; c' and ϕ'

d. long-term durability (based on ASTM C-88)

e. Young's modulus of elasticity, $\left(G=\frac{\Delta\tau}{\Delta x}h, E=2G(1+\nu)\right)$

f. Poisson's ratio, ν, $\left(K_o=\frac{\nu}{1-\nu}\right.$, where $\left.K_o=1-\sin\phi'\right)$

Step 5. Determine the stress-strain incompatibility of the materials in the composite ground; e.g. peak strengths of the stabilized column and the native soils cannot be mobilized simultaneously if shear strain at failure varies.

Stress-strain relationships of both the native soil and column materials are vital to the design of stabilization measures. Figure 8.3 indicates that the peak strength of the rockfill columns occur at a higher level of shear strain than the native soil. This shows that it is better to use post-peak strength parameters of the soil rather than peak values.

Step 6. Check the stability of the natural riverbank using either LEM and/or FEM. The following steps may be followed:

a. Conduct a stability analysis using limit equilibrium analysis and/or the finite element method. It is noted that for riverbanks in Winnipeg, the potential shear failures are generally deep seated and go through the weak clay zone at the clay-till interface as shown in Figure 8.4. The shear failure plane also moves though the crest, and down to the toe of the riverbank.
b. If the calculated factor of safety indicates that the riverbank requires stabilization, determine the critical shear failure surface in order to place the rockfill columns in the appropriate locations. The vicinity of the mid-span of the potential failed mass is the optimum location of the reinforced columns (as shown in Figure 6.8).

Step 7. Estimate the design parameters

Other parameters required for the analysis and design of stabilized riverbanks using rockfill columns include:

a. Young's modulus of elasticity, E, is obtained after measuring the shear modulus ,G, the following equation (Davis and Selvaduri 1996):

[8.1] $G = \frac{\Delta\tau}{\Delta x} h$ (see Chapter 6 for detail)

The Young's modulus can then be calculated using an assumption of linear elastic:

[8.2] $E = 2G(1+\nu)$

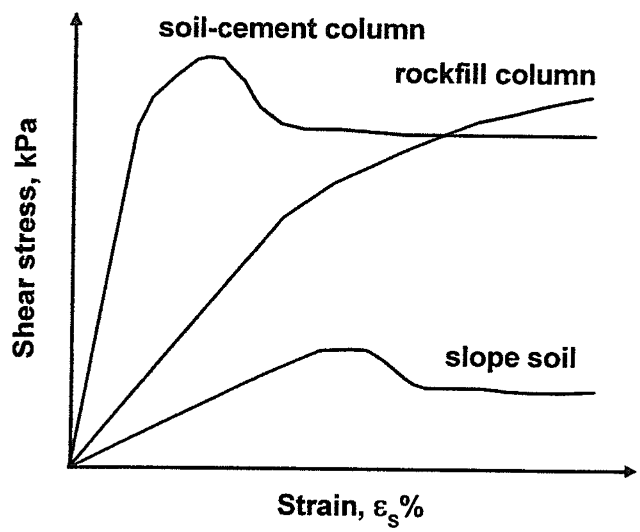


Figure 8.3 Stress-strain incompatibility of different materials

The Poisson's ratio, ν, can be calculated using Gregory (1973) equation:

[8.3] $K_o = \frac{\nu}{1-\nu}$ (see Chapter 6)

K_o can be calculated using Jaky (1984) equation:

[8.4] $K_o = 1 - \sin\phi'$ (see Chapter 6)

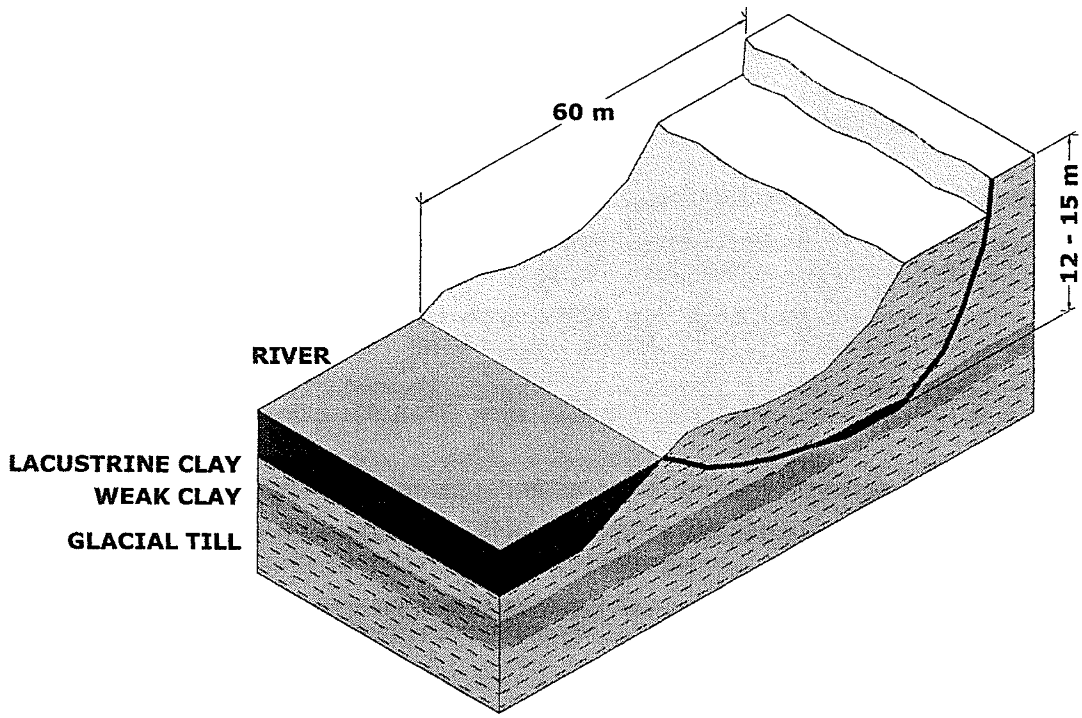


Figure 8.4 Failure mode of a typical riverbank in Winnipeg after field observations

Step 8. Design of rockfill columns

The limit equilibrium and finite element methods (shear strength reduction method) can be used to calculate the number of columns required, which depends on the required factor of safety, say FS ≥ 1.5 for the optimum location in mid span of failure surface.

Notes:

I. The center-to-center distance between rows of columns should be no more than twice the diameter of columns.

II. The columns should be designed such that they are anchored in a stiff till layer.

III. The maximum displacements calculated at the riverbank crest should not exceed the acceptable displacements required for structures along the riverbank.

Step 9. Installation of rockfill columns:

The replacement method is the most common technique for the installation of rockfill columns in Winnipeg. The installation process should include the following steps:

I. Driving of Casing: Casing supports the sidewall of the hole from extra displacements usually generated during installation and prevents stress relief around the borehole wall. If there is sufficient stability of the side wall, casing is not necessary and the hole can be made by direct augering.

II. Augering: A rotary mechanical auger is generally used to drill the hole. Augering can be performed in lifts to avoid high suction generated by the subsurface high plastic clays. The borehole must pass through the till layer until a hard layer is reached. During augering, it is important to watch the borehole log to observe where lenses of silty soils are present. If so, the grouting of rockfill columns at those locations may be done to prevent silt infiltration inside the rockfill column which may lead to poor performance of the rockfill columns.

III. Backfilling of suitable rockfill materials into the borehole is performed as follows:

a. Place dumping crushed limestone (up to 100 mm is size) into the borehole in lifts. Notes:

   1. Durability tests are required to ensure proper quality or the rockfill materials. The percent weight loss of crushed limestone materials shall not be more than 15% for the Magnesium Sulphate Soundness Test (ASTM C-88 2005).
   2. Check for rock contamination with the native soils otherwise the strength parameters of the rockfill used for analysis may not represent the condition at the site.

b. Densification of rock should be carried out using a vibrator probe in lifts to increase the friction angle. Rockfill material densification is significant as shear resistance in a dense condition is twice the value than for a loose conditions when the rockfill material is only dumped in the hole (Chapter 4).

c. Backfilling of rockfill materials continues to the ground surface. The installed columns should be covered with a 30 to 50 cm angular limestone riprap blanket for protection and to improve drainage. The riprap will protect the surface from strong river currents or waves.

IV. Mixing rockfill with cement

1. Low level cementation

Rockfill column techniques can be improved in terms of shear resistance, shear stiffness and particle-to-particle bonding by mixing rockfill materials with cement at a low level (1% cement by weight of the rockfill materials). Addition of low level cementation can be carried out by premixing the rockfill materials with cement and water. A higher level of cementation can be done by grouting the rockfill columns when these are installed (Broms1984).

2. Cement grouting

   Grouting of rockfill columns should be considered when there are lenses of silty soils within the clayey soil as infiltration of this silt may endanger the strength of the rockfill columns. Silt infiltration may also change the failure mode. The silt lenses will now become weaker (due to silt particles migration into the columns) and the column strength can also be reduced.

It is expected that rockfill columns with low level cementation will have no significant negative effect on the hydraulic conductivity of the treated soil. The applications of cemented rockfill columns have additional advantage in the following conditions:

i. Cemented rockfill columns are preferable for use in short lengths for two reasons. First, there is less confined support supplied by the surrounding soils in cases of soft clays. Second, the mobilization of shear resistance of rockfill columns is highly affected by the column weight above the failure plane (FHWA 1983).

ii. It is recommended that the first row of columns from the crest be cemented as these columns are exposed to the highest lateral pressure and thus experience more displacements than others.

iii. Use cemented rockfill columns for riverbanks where the bottom soil layer is not firm enough, which leads to the possibility of failure plane passing below the column tip. The cement injection for the till layer may also be recommended to create a good bearing strata.

iv. Where continuous displacement is experienced in a slope where rockfill columns have been installed, the rockfill columns can be grouted afterward with cement (Broms 1984). It is well known that the mechanisms of rigid cemented column performance are similar to piles or piers (FHWA 1983). If the designer has restrictions in cementing the granular columns, it is recommended to stiffen the lower part of the rockfill columns at the weak zones, where high lateral shear stresses are anticipated. In other words, cement may be added to the rockfill material only at depths where a weak layer is found.

V. It should be recognized that riverbank geometry is not only important for the design for the number of rows of columns required to reach the acceptable factor of safety, but also to determine a more efficient layout of columns that satisfy the stability and the allowable displacements, thus leading to an optimum design. For example, using rockfill columns installed in a shear key layout pattern (parallel to river) may not be enough to achieve acceptable design. In such a situation, a combination of two column layouts; shear key type and rib-type (perpendicular to river), may be required as shown in Figure 8.5.

VI. Quality control and quality assurance:

Full-time inspection by a qualified geotechnical engineer is mandatory to ensure that the specified design and construction guidelines are followed.

8.4 Typical construction of soil-cement columns

As explained previously, soil-cement columns are an in-situ soil treatment technique in which the soil is blended with reagent (cement, lime, lime-cement, and others) to increase the shear strength through improving in-situ soil properties, without excavation or removal of the native soils. A column is formed by mixing cementitious materials with soft soils. A column can be installed individually or with other columns in different patterns such as wall type, grid type, block type and area types.

The basic concept and procedures are very similar for all techniques, whereby cement and/or lime is injected and mixed using either the slurry state or dry powdered state. The injection process is performed through hollow, rotated mixing shafts using 2 to 8 mixing shafts with cutting tools as shown in Figure 8.6.

It should be noted that the suitability of each method, (both dry mixing and wet mixing) for stabilizing slopes depends on the moisture content of the native soil.

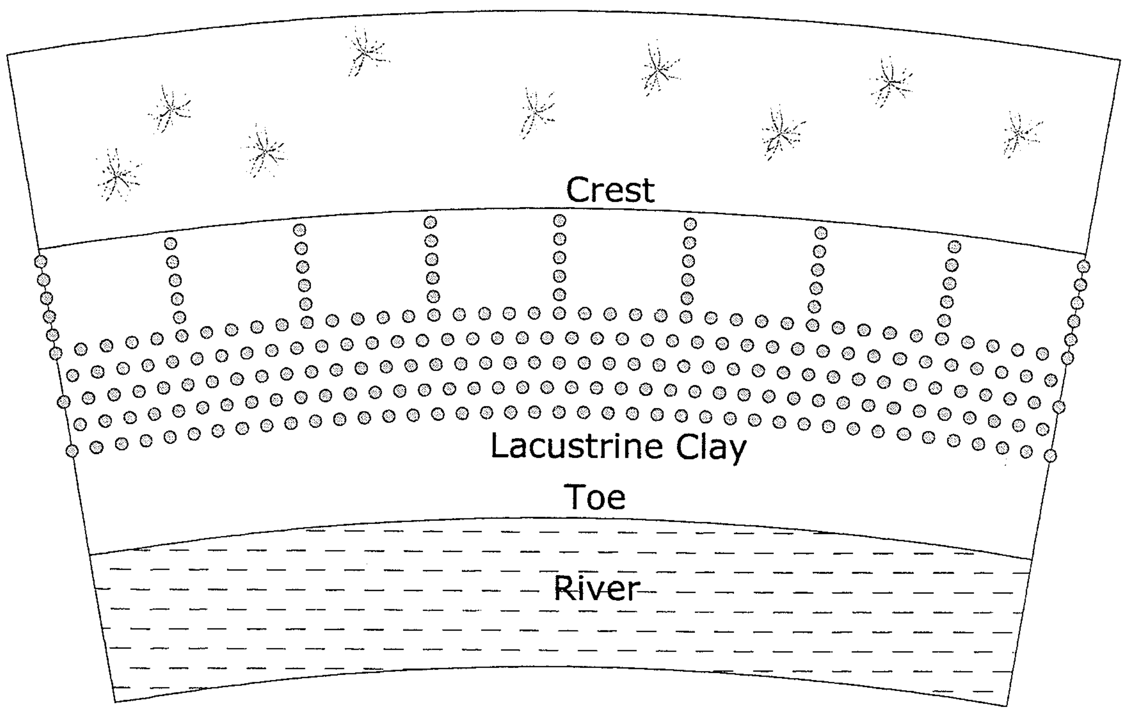


Figure 8.5 Stabilization of riverbank using rockfill columns following shear key and rib-type patterns

For soils of 60% to 200% or more of moisture content, the dry mixing method should be used. Otherwise, the wet mixing method is preferable.

Construction of a soil-cement column involves driving the mixing shafts tipped with cutting tools into the soil. As the shafts advance into the ground, cement grout is injected through the hollow stems of the cutting tools and pumped into

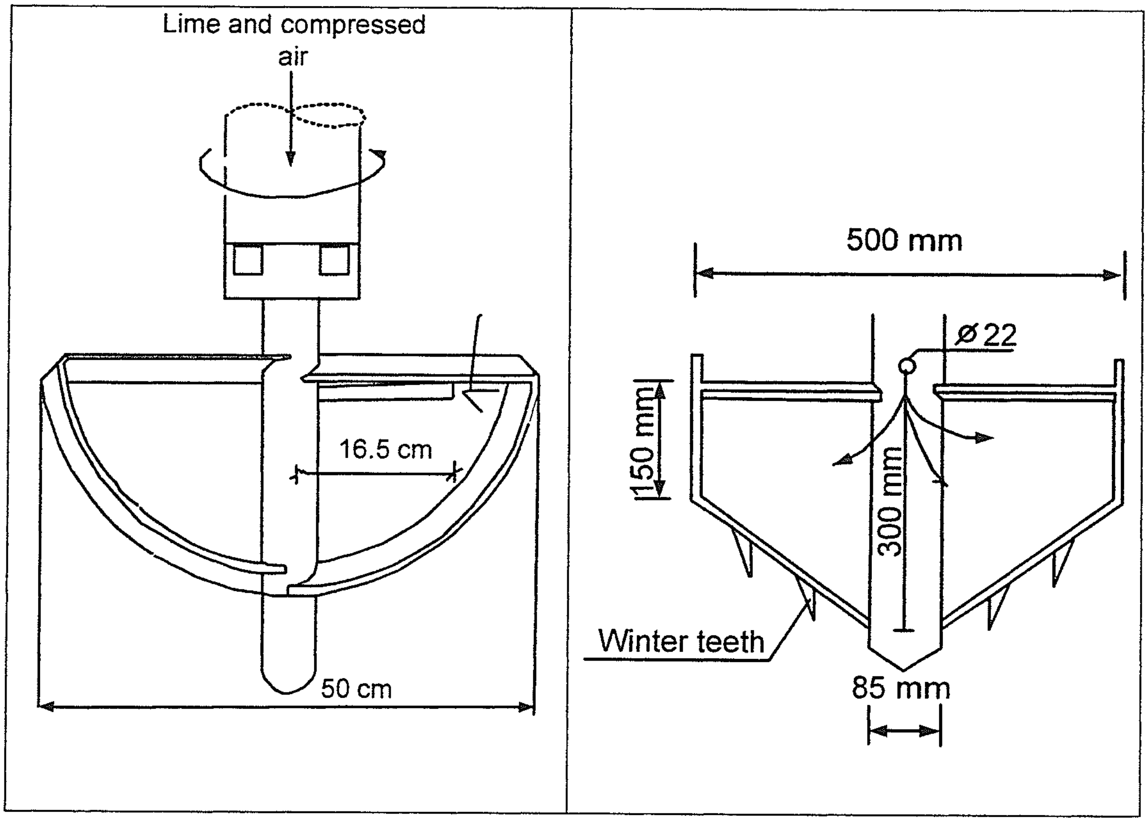


Figure 8.6 Cutting tools for the construction of soil-cement columns (after FAHWA 2001a)

the soil. The soil blends with the grout using rotating mixing blades, paddles, and discontinuous auger flights to form uniform column properties. The mixing shafts are advanced into the soil until the design depth is reached. Then, the withdrawing operation starts while the mixing and injection process continues up to the ground surface. In some methods, the process of cement slurry injection is accompanied by fluid grout injection at high pressure through nozzles in the mixing tools. This is done to enhance the injecting stage, thereby obtaining a high shear strength and a high uniformity.

8.5 Recommended design guidelines to stabilize riverbanks using soil-cement columns

8.5.1 Assumptions

To stabilize riverbanks using soil-cement columns, assumptions are made similar to the ones for rockfill columns. It is assumed that reinforced columns are installed deep enough up to a stiff till layer, and to ensure that potential slip surfaces pass through the columns rather than below the tip.The columns in this technique can be simulated as liners (beam element) where moment resistance is considered.

8.5.2 Proposed soil-cement column design and construction guidelines

Columnar inclusions, either rockfill columns or soil-cement columns, follow similar steps regarding design guidelines. Therefore, steps one to three, regarding the in-situ native soils and riverbank geometry, are generally similar in both techniques. Furthermore, the evaluation of stability of natural riverbanks is the same for both techniques.

Step 1. Establish the geometry, external loading, topographical conditions, and groundwater conditions. These include the following:

- Slope angle
- Slope height
- Soil type(s) present in the riverbank, and their physical, chemical and mineralogical properties
- Moisture content of the soils

- pH-value of the pore water in the soil
- Location of the weak clay zone and its thickness
- Location of water table level
- Aquifer pressure if available and its variation over a year
- River water level and its variation over a year
- External loads such as buildings and other infrastructure
- pH-value of the ground water table

Step 2. Investigate the influence of the variation of the above parameters on the riverbank stability, shear strength and shear stresses. Choose parameter values that represent the worse case scenario.

Step 3. Determine the engineering properties of the in-situ (native) soils. Similar to the guidelines of rockfill columns.

Step 4. Determine the engineering properties of the soil-cement column including;

a. Quality of hardening agent, cement
b. Mixing water; water/cement ratio and additives if any are present
c. Mixing time for each particular soil type, and degree of mixing
d. Site temperature, humidity and curing time
e. The effective strength parameters; $c^{'}$ and $\phi^{'}$ at elapsed time of 28 day and 90 days. However, the parameters for 28 days are usually used for design purposes

f. Hydraulic conductivity, k, of the soil-cement column

Step 5. Determine the stress-strain incompatibility of the materials comprising the composite ground

Stress-strain relationships of the soil slope and soil-cement columns are vital to select the proper engineering parameters of the native soils. It is clear from Figure 8.3 that mobilization of shear resistance of soil-cement columns is accomplished at an earlier stage of shear strain, than in the native soil. This properly requires the use of post peak parameters of the native soils, for proper simulation of field conditions, rather than the peak values.

Step 6. Check the stability of the natural riverbank using either LEM and/or FEM. The following steps may be followed: Similar to the guidelines of rockfill columns.

Step 7. Estimate the design parameters

Other parameters required for the analysis and design of stabilized riverbanks using soil-cement columns include estimation the Poisson's ratio, ν. This can be calculated using Gregory (1973) equation as explained earlier with rockfill columns.

Step 8. Design of soil-cement columns

The limit equilibrium and finite element methods can both be used to calculate the factor of safety as explained with rockfill columns guidelines except the followings:

I. To mobilize shear resistance in all columns more uniformly, a blanket soil-cement layer can be constructed on top of the constructed columns to connect them so they respond as a group during loading. A one meter thick soil-cement blanket may be constructed to achieve this goal.

II. The first row of columns from the crest can be installed to form a walll type due to the fact that the first row is subjected to a higher lateral forces than the succeeding columns..

Step 9. Installation of soil-cement columns:

The wet deep mixing method may be preferable to the dry mixing method for construction in Winnipeg. This recommendation is due mainly to the high plastic clay and the moisture content that is usually in the range of 47% to 67%. The following steps may be followed to ensure the applicability for these ground conditions:

I. Slurry cement is injected into the soil using mixing shafts. These shafts are rotated and advanced into the soil layers to blend and mix the native soil with the cementitious materials. It is also essential to know that shafts should be kept in vertical alignment to prevent unmixed zones between column sets.

II. The optimum cement ratio for this particular soil type and moisture content was 18% at 100% water/cement ratio.

III. The process of mixing cement with the high plastic clay needs to be done thoroughly to reach a high uniformity, which leads to high shear strength.

IV. Drilling and mixing tools should be selected properly to achieve full mixing with no clay lumps remaining inside the soil-cement columns. Moreover, it should be recognized that the drill head is extremely important to ensure the capability of mixing, especially for this type of soil. Cement end mixing tools can be used for a higher performance and to eliminate the suction that may develop during the blending and mixing process. However, for different soil types, different end mixing tools can be used.

V. Designing the penetration and extraction rates of mixing tools is important to ensure a homogenous mixture, in which consistent engineering properties can be obtained. Therefore, field trials of the soil-cement column are essential to obtain the ideal mixing penetration rate, an RPM value, which achieves the degree of uniformity required.

VI. Quality control and quality assurance:

Just as in rockfill columns, full-time inspection by a qualified engineer is mandatory to ensure that the design and construction guidelines are being followed.

Chapter 9

CONCLUSIONS AND RECOMMENDATIONS

9.1 Concluding Remarks

9.1.1 Large scale direct shear tests of rockfill materials and clay-rockfill composite

The main conclusions that can be drawn from the results of the laboratory testing program can be summarized as follows:

(1) Large-scale direct shear equipment has proven to be valuable for evaluating shear mobilization in rockfill columns. This large scale apparatus provides an opportunity to test the engineering properties of scaled particle sizes of rockfill as well as clay-rockfill composite specimens.

(2) Mobilization of shear resistance in rockfill columns is dependent on the relative density of the material and the applied normal stress. Higher relative

densities lead to higher shear resistances. For example, under a normal stress σ_n = 100 kPa (equivalent to an effective stress of 10 m below the ground surface, assuming water table is on the ground surface), the shear resistance in the dense condition is almost twice as high as in the loose condition.

(3) The mobilized shear resistance of rockfill materials can be very high for densely compacted material. It is dependent on the dilatancy of the material during shearing; higher applied normal stress results in lower dilatant behaviour of the material and therefore lower friction angle. In other words, the friction angle of the rockfill material over the range in normal stresses that do not suppress dilatancy is very dependent to the applied normal stress. For example, the mobilized shear resistances at 2-5% shear strain are found to be in the range of $\phi_m = 64°$ for normal stress σ_n = 100 kPa and $\phi_m = 74°$ for normal stress σ_n = 50 kPa. The corresponding results for loosely compacted materials are in the range $\phi_m = 46°$ for normal stress σ_n = 100 kPa and ϕ_m = 55° for normal stress σ_n = 100 kPa.

(4) The critical friction angle is considered to be a unique parameter independent of the applied normal stresses. The shear resistance corresponding to the critical state is normally mobilized at larger shear strains that is beyond the limit of the large-scale direct shear apparatus. The method of Atkinson (1992) using transition friction angle based on smaller shear strains was used to estimate the critical state friction angle. This value is $\phi_{cs} = 37°$.

(5) During shearing, dilatancy of the rockfill material is greater at the backside of the shear box compared to the front in the direction of movement. This means that the mobilization of shearing progresses along the shear plane from the back to the front. In the field situation, this means that shear resistance in a rockfill column group will be mobilized first in columns located at the front (uphill) then progress towards those at a lower level on the slope.

(6) Test results of both the native clay and clay-rockfill composite show that mobilization of shear resistance in the rockfill column is limited until the peak strength of the clay is mobilized at about 2% shear strain. This means that the shear resistance in the rockfill column will mobilize only at shear strains greater than the strain corresponding to the peak strength of the clay. This behaviour of the clay-rockfill composite can be referred to the stress-strain incompatibility of native clays and the rockfill materials.

(7) Large scale direct shear tests on clay-rockfill composite show that increasing the area replacement ratio increases the overall shear strength of the composite material proportionally.

(8) Rib-type layout shows much greater mobilization of shear resistance than the shear key. This is due to the fact that mobilization of shear resistance in a shear key does not start until full mobilization of shear resistance in the clay.

9.1.2 Rockfill columns with cementation

The mobilization of shear resistance of rockfill columns depends on the magnitude of the overall riverbank movements. This study attempts to reduce the movements required to mobilize shearing resistance in the columns by adding small amount of cement. Results obtained by conducting large scale direct shear tests on cemented rockfill materials and on clay-rockfill composite show that:

(1) The peak shear strength of the rockfill column using 0.5% cement content is increased, but at larger shear strain (6%) the strength is almost the same as that of the untreated rockfill material because the cement bonding between rockfill particles is broken. The shear stiffness for cemented rockfill material at lower shear strain up to about 2% is twice that of untreated rockfill material.

(2) Adding small amount of cement increased the apparent cohesion of the rockfill material while the friction angle was similar to that of untreated material.

(3) Large scale direct tests on composite soils show that adding 5% of cement by weight to the rockfill column will slightly increase the shear stiffness, but this has no significant effect on the shear stress compared to an untreated single rockfill column. This finding may be attributed to the change of mode of failure from direct shear type of failure to a passive type of failure by squeezing the clay around the column. The practical implication of this finding is to arrange the rockfill columns very closely, such that they act as a wall.

9.1.3 Soil-cement columns

An alternative columnar inclusion in the form of soil-cement column has also been investigated to stabilize riverbanks by providing stiffer and stronger columns than rockfill columns. The research investigated the stress-strain characteristics of soil-cement mixtures in the laboratory. This has been done by conducting unconfined compression tests which are a widely used test to determine the stress-strain characteristics of soil-cement columns. The test has also been used to determine the optimum cement content for soil-cement columns in the high plastic clay in Winnipeg.

(1) It was found that the cement content of 18% by weight of dry soil provided the optimum stiffness and strength of the soil-cement mixture.
(2) The results show that the modulus of elasticity was approximately E =120 MPa and the unconfined compression strength was about q_u =800 kPa.
(3) Soil-cement mixture has a much higher stiffness and strength compared to rockfill columns.

9.1.4 Analysis and design of columnar inclusion for riverbank stabilization

The performance of natural and stabilized riverbanks was investigated using numerical analysis. Both limit equilibrium method (LEM) and finite element method (FEM) of analyses were used. Based on the results of the investigation, the following conclusions can be drawn:

(1) Both LEM and FEM have been shown to be satisfactory in obtaining the factor of safety of natural and stabilized riverbanks. FEM has an added benefit of estimating deformations associated with the stabilization of riverbanks.
(2) The optimum location for columnar inclusions was found to be in the vicinity of the centre of the critical shear failure surface.

(3) Given the same number of columns, the spacing between column rows can influence the calculated factor of safety and slope displacements. Closer spacing leads to higher factors of safety and lower slope movements. It was found that centre-to-centre spacing of equal to or less than twice the diameter of the column provided good results.

(4) The finite element method was used to assess the performance of rockfill columns with the addition of low level cementation. The factor of safety increased from 1.35 and 1.56 to 1.82 and 1.91 using FEM and LEM respectively. The calculated maximum displacement was about 8.5 cm, which is slightly less than the value for untreated rockfill columns of about 9 cm. It is apparent that adding cementitious material increased the particle bond and as a result the cohesion between the granular particles increased and enhanced the mobilization of shear resistance of the rockfill columns, even in low applied normal stress environments. This technique is most useful for columns near the crest of the slope, which are exposed to a large displacement, or where there is the possibility of silt infiltration into the rockfill columns.

(5) Mixtures of Winnipeg clay and cement have shown higher stiffness and strength compared to those of rockfill materials under field stress levels. Therefore the calculated factor of safety of riverbanks stabilized with soil-cement columns was found to be higher than those for slopes stabilized with rockfill columns. The corresponding displacements of riverbanks with soil-cement columns were also found to be lower.

(6) The advantages of using soil-cement columns in terms of field applications include: a) less stress relief in the soil during column installation as the technique does not require augering open holes, and b) soil-cement columns can be installed tangentially or even in an overlap manner (secant columns) which is not possible for rockfill columns. Closely spaced columns generally work as a group and therefore provide better shear resistance than the collective action of individual columns.

(7) One of the variables that might affect the stability of stabilized riverbanks in Winnipeg is the existence of lenses of silty soil. This layer, when it is between two layers of high plastic clay, can potentially create a problem in the performance of rockfill columns.

(8) Another factor that may influence the performance of stabilized riverbanks is the strength of the till layer where columns are usually anchored. Based on previous studies, the friction angle of this layer is close to 40° and the residual value is about 31°. With this range of friction angle (31° to 40°), the analysis showed that the potential failure surface passed above the clay-till interface and through the column. However, if there are factors that cause the friction angle of the till to decrease further to less than 31°, there is a possibility that the failure surface might pass through the tip of the column, assuming the columns are anchored one meter into the till layer.

9.2 Recommendations for Future Research

Recommendations are outlined below in terms of the needs for further research:

(1) Perform fully-instrumented full-scale field tests of slopes reinforced with rockfill columns to validate the results from large-scale laboratory tests and numerical analyses. These tests lead to a better understanding of the slope movements required to mobilize shearing resistance of stabilized riverbanks.

(2) Further full-scale field testing can be carried out to study the effectiveness of low level cementation. In addition, rigid rockfill columns can be used. In this technique, traditional rockfill columns will be injected with cement similar to grouting to improve their performance.

(3) Perform in-situ full-scale test to stabilize riverbank using soil-cement column. This will determine the applicability of this technique to stabilize riverbanks consisting of high plastic clay.

(4) Conduct laboratory tests on soil-cement columns using a miniature soil mixing tool to investigate the mixing of cement with on-site soils including optimum soil-water-cement ratio, mixing energy and duration. Better precision on the mixing process will lead to cost-savings in usage of cement and electrical power.

(5) Conduct drained large scale triaxial tests with pore pressure measurement to determine more accurately the elastic properties of rockfill materials, cemented rockfill materials, and soil-cement mixtures.

(6) Carry out three-dimensional numerical analyses so that the appropriate geometry and layout of the columns can be properly determined and for assessing the accuracy of results (stability and displacement) determined by 2-D analysis.

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