SHEAR STRENGTH PROPERTIES OF COMPACTED EXPANSIVE SOILS

A Thesis
Submitted to the Faculty of Graduate Studies and Research
In Partial Fulfillment of the Requirements
For the Degree of
Master of Applied Science
in Environmental Systems Engineering
University of Regina

By
Md Rashedul Hoque Chowdhury
Regina, Saskatchewan
October, 2013

Copyright 2013: R. H. Chowdhury
Md. Rashedule Hoque Chowdhury, candidate for the degree of Master of Applied Science in Environmental Systems Engineering, has presented a thesis titled, *Shear Strength Properties of Compacted Expansive Soils*, in an oral examination held on August 22, 2013. The following committee members have found the thesis acceptable in form and content, and that the candidate demonstrated satisfactory knowledge of the subject material.

External Examiner: Mr. Wayne Clifton, Clifton Associates Ltd

Supervisor: Dr. Shahid Azam, Environmental Systems Engineering

Committee Member: Dr. Guo H. Huang, Environmental Systems Engineering

Committee Member: Dr. Mohamed El-Darieby, Software Systems Engineering

Chair of Defense: Dr. Mohamed Ismail, Industrial Systems Engineering
ABSTRACT

Shear strength of compacted expansive soils affects the bearing capacity of foundations, slope stability of embankments, and design of earth retaining structures. The purpose of this research was to develop a clear understanding of the shear strength properties of compacted expansive soils. A typical expansive soil, namely, Regina clay, was selected for detailed laboratory characterization. Geotechnical index properties were determined for preliminary soil assessment. The specific gravity and clay size fraction (material finer than 0.002 mm) were found to be 2.74 and 60%, respectively. The liquid limit (77%) and plastic limit (27%) indicated a significant water adsorption and retention capacity of the soil. The compaction curve was determined for evaluating the behaviour of the expansive soil and obtaining samples for subsequent testing. The optimum water content and maximum dry density were found to be 24% and 1.6 g/cm$^3$, respectively. Direct shear tests were performed on compacted and natural samples to measure the stress-strain behaviour thereby determining the shear strength properties. Soils on the dry side of optimum showed a brittle stress-strain behaviour whereas those on the wet side of optimum showed a ductile behaviour. The strain at failure was found to be within 1.5 mm to 3.0 mm. The cohesion ($c'$) followed the compaction curve with a maximum value of 66 kPa at the optimum water content. Likewise, the friction angle ($\phi'$) decreased from 44$^\circ$ on the dry side reaching a minimum value of 27$^\circ$ at the optimum water content beyond which it was constant. The friction angle due to suction ($\phi''$) was measured using direct shear testing on unsaturated samples in conjunction with suction measurement as well as estimated using the soil water characteristic curve (SWCC) along with direct shear test on saturated samples and the plasticity index of the soil. The estimated values of the
corroborated well with the laboratory determined values following a linear relationship. Furthermore, the SWCC was found to be bimodal including two air entry values: a lower value of 10 kPa (for $w$, $\theta$, and $S$) followed by a higher value of 100 kPa (for $w$ and $\theta$) and 6000 kPa (for $S$). The swell-shrink curve was determined in conjunction with SWCC because of the volume change nature of the soil. An S-shaped swell-shrink curve was obtained with three distinct portions: structural shrinkage ($S = 100\%$ to $S = 85\%$); normal shrinkage ($S = 85\%$ to $S = 75\%$) and residual shrinkage ($S = 75\%$ to $S = 0$). The compaction curve was observed to be within the normal shrinkage zone. Finally, a parametric study was conducted using commercially available computer software. A 10 m high embankment was modeled using three slope angles ($45^\circ$, $60^\circ$ and $90^\circ$). The factor of safety was found to be more than 1 for the compacted soil at optimum water content regardless of the slope angles. A 6% change in water content on both sides of the optimum water content rendered the $90^\circ$ slopes unstable. Likewise, under saturated conditions, both natural and compacted slopes failed.
ACKNOWLEDGEMENTS

I express my sincere appreciation to the Natural Science and Engineering Research Council of Canada, the Saskatchewan Ministry of Highways and Infrastructure, and the University of Regina Faculty of Graduate Studies and Research for providing financial support in the course of this research.

I would like to thank and acknowledge the support of my supervisor, Dr. Shahid Azam, who provided the critical technical and theoretical expertise that this research needed to be completed.

Grateful thanks go to Mr. Pete Gutiw, our Laboratory Instructor, for his technical support in the laboratory.

I also thank my friends and colleagues, especially Dr. Raghunandan, for sharing knowledge and helping throughout this research.

Finally, the love and support of my wife, sisters, and, especially my parents whose blessings gave me the strength to finish my research.

The path we have been through has not been straightforward, yet everyone has been healthy and trying to be happy. You, my family, have been my greatest motivation to complete this research.
POST DEFENSE ACKNOWLEDGEMENTS

The time and inputs of Dr. Wayne Clifton (external examiner), Dr. Gordon Huang (supervisory committee member) and Dr. Mohammad El-Darieby (supervisory committee member) from the Faculty of Engineering, University of Regina, and Dr. Mohamed Ismail (thesis defense chair) from the Faculty of Engineering, University of Regina, are appreciated for serving on my thesis committee.
# TABLE OF CONTENTS

ABSTRACT.........................................................................................................................i

ACKNOWLEDGEMENTS.......................................................................................................iii

POST DEFENSE ACKNOWLEDGEMENTS........................................................................vi

TABLE OF CONTENTS......................................................................................................v

LIST OF TABLES................................................................................................................viii

LIST OF FIGURES............................................................................................................ix

LIST OF APPENDIX TABLES...........................................................................................xi

LIST OF APPENDIX FIGURES.........................................................................................xii

CHAPTER 1

INTRODUCTION................................................................................................................1

1.1. Problem Statement.....................................................................................................1

1.2. Research Objectives.................................................................................................4

1.3. Thesis Outline .........................................................................................................5

CHAPTER 2

LITERATURE REVIEW....................................................................................................6

2.1. General......................................................................................................................6

2.2. Compaction Curve..................................................................................................6

2.3. Soil Water Characteristics Curve..............................................................................8

2.4. Swell-shrink Curve................................................................................................12

2.5. Shear Strength for Expansive Soils........................................................................15

   2.5.1. Friction Angle................................................................................................15

   2.5.2. Cohesion.........................................................................................................15

   2.5.3. Friction Angle due to Suction......................................................................16
2.6. Factors Affecting Shear Strength of Expansive Soils ................................................18
2.7. Shear Strength of Saturated Soil.............................................................................21
2.8. Shear Strength of Unsaturated Soil......................................................................23
2.9. Direct Shear Testing of Unsaturated Expansive Soils...........................................25
2.10. Suction Characteristics......................................................................................27
2.11. Prediction of Unsaturated Shear Strength..........................................................28
2.12. Parametric Study............................................................................................29
2.13. Research Hypothesis.......................................................................................30

CHAPTER 3

RESEARCH METHODOLOGY....................................................................................31
3.1. General ..............................................................................................................31
3.2. Geotechnical Index Properties ...........................................................................35
    3.2.1. Water Content ....................................................................................35
    3.2.2. Dry Unit Weight ..................................................................................35
    3.2.3. Specific Gravity ..................................................................................35
    3.2.4. Grain Size Distribution ......................................................................37
        3.2.4.1. Sieve Analysis ...............................................................37
        3.2.4.2. Hydrometer Analysis .......................................................37
    3.2.5 Consistency Limits ...............................................................................38
3.3. Compaction Curve............................................................................................39
3.4. Soil Water Characteristics Curve......................................................................40
3.5. Swell-shrink Test.............................................................................................41
3.6. Direct Shear Test...............................................................................................42
3.7. Suction Measurement.......................................................................................44
3.8. Prediction of Unsaturated Shear Strength.......................................................45
3.9. Parametric Study ........................................................................................................47
  3.9.1. Slope Modeling ........................................................................................................47
  3.9.2. Material Properties .................................................................................................47
  3.9.3. Method of Analysis ...............................................................................................47

CHAPTER 4
RESULTS AND DISCUSSION .................................................................................49
  4.1. Geotechnical Index Properties .................................................................................49
  4.2. Compaction Characteristics Curve ..........................................................................49
  4.3. Soil Water Characteristic Curve .............................................................................52
  4.4. Swell-shrink Curve ..................................................................................................54
  4.5. Stress-strain Relationship .......................................................................................56
  4.6. Shear Strength Properties .......................................................................................58
  4.7. Prediction of Unsaturated Shear Strength ...............................................................62
  4.8. Physical model of compacted expansive soil ............................................................64
  4.9. Parametric Study .....................................................................................................67

CHAPTER 5
SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS ..................................69
  5.1. Summary ..................................................................................................................69
  5.2. Conclusions .............................................................................................................69
  5.3. Recommendations .................................................................................................71

REFERENCES ..............................................................................................................72

APPENDIX ....................................................................................................................79
**LIST OF TABLES**

Table 2.1: Shear strength parameters of unsaturated cohesive soils ........................................ 17

Table 2.2: Horizontal displacement rate and horizontal displacement at failure from several direct shear test ........................................................................................................ 26

Table 4.1: Geotechnical index properties of investigated soil ................................................. 50

Table 4.2: Shear Strength properties of investigated soil ..................................................... 59

Table 4.3: Factor of safety for the investigated slope with different material property ......... 68
LIST OF FIGURES

Figure 1.1: Canal Side slope failure at Deer Valley in Northwest of Regina......................2
Figure 1.2: Slope failure of natural deposit at Deer Valley in Northwest of Regina..........2
Figure 1.3: Ongoing infrastructure construction (mild slope) on local expansive soil.......3
Figure 1.4: Ongoing infrastructure construction (steep slope) on local expansive soil.......3
Figure 2.1: Mohr-Coulomb failure envelopes for saturated soil.................................22
Figure 2.2: Extended Mohr-Coulomb failure envelope for unsaturated soils...............22
Figure 2.3: Typical soil water characteristics curve......................................................9
Figure 2.4: Typical swell-shrink path..........................................................................13
Figure 3.1: Laboratory investigation program.................................................................32
Figure 3.2(a): Laboratory direct measurement of saturated shear strength.................33
Figure 3.2(b): Laboratory direct measurement of unsaturated shear strength..............33
Figure 3.2(c): Estimation of unsaturated shear strength..............................................33
Figure 3.3: Estimation of unsaturated shear strength property (φ_b).............................34
Figure 3.4: Laboratory direct shear test procedure.......................................................43
Figure 3.5: Fitting parameter versus plasticity index....................................................46
Figure 3.6: Schematics of parametric study procedure................................................46
Figure 3.7: Determination of critical slip surface..........................................................48
Figure 4.1: Compaction curve for the investigated soil..............................................51
Figure 4.2: Soil water characteristics curve for investigated soil..............................53
Figure 4.3: Swell-shrink curve of investigated soil.......................................................55
Figure 4.4: Stress strain relationship for the investigated soil.......................................57
Figure 4.5: Shear strength properties versus water content relationship.......................60
Figure 4.6: Relation between $\phi^b$ versus water content.......................................................63

Figure 4.7: Physical model describing behaviour of compacted expansive soils.........65
LIST OF APPENDIX TABLES

Table 1: Results from specific gravity determination for the investigated soil.................80
Table 2: Results from liquid limit determination for the investigated soil.....................81
Table 3: Results from plastic limit determination for the investigated soil....................81
Table 4: Results from the hydrometer analysis for the investigated soil.......................82
Table 5: Results from the standard proctor test for the investigated soil.......................83
Table 6: Results from the soil water characteristics curve and swell-shrink curve...........98
LIST OF APPENDIX FIGURES

Figure 1: Shear stress versus strain plot ($\rho_d = 1.46 \text{ g/cm}^3$ & $w = 16\%$).............................84
Figure 2: Shear stress versus normal stress plot ($\rho_d = 1.46 \text{ g/cm}^3$ & $w = 16\%$)................84
Figure 3: Shear stress versus suction ($\rho_d = 1.46 \text{ g/cm}^3$ & $w = 16\%$).................................85
Figure 4: Shear stress versus strain plot ($\rho_d = 1.54 \text{ g/cm}^3$ & $w = 21.5\%$)..........................85
Figure 5: Shear stress versus normal stress plot ($\rho_d = 1.54 \text{ g/cm}^3$ & $w = 21.5\%$).............86
Figure 6: Shear stress versus suction plot ($\rho_d = 1.54 \text{ g/cm}^3$ & $w = 21.5\%$).......................86
Figure 7: Shear stress versus strain plot ($\rho_d = 1.57 \text{ g/cm}^3$ & $w = 22.9\%$).............................87
Figure 8: Shear stress versus normal stress plot ($\rho_d = 1.57 \text{ g/cm}^3$ & $w = 22.9\%$)..............87
Figure 9: Shear stress versus suction plot ($\rho_d = 1.57 \text{ g/cm}^3$ & $w = 22.9\%$)............................88
Figure 10: Shear stress versus strain plot ($\rho_d = 1.59 \text{ g/cm}^3$ & $w = 23.9\%$)............................88
Figure 11: Shear stress versus normal stress plot ($\rho_d = 1.59 \text{ g/cm}^3$ & $w = 23.9\%$)...............89
Figure 12: Shear stress versus suction plot ($\rho_d = 1.59 \text{ g/cm}^3$ & $w = 23.9\%$)......................89
Figure 13: Shear stress versus strain plot ($\rho_d = 1.52 \text{ g/cm}^3$ & $w = 28.3\%$)............................90
Figure 14: Shear stress versus normal stress plot ($\rho_d = 1.52 \text{ g/cm}^3$ & $w = 28.3\%$)..............90
Figure 15: Shear stress versus suction plot ($\rho_d = 1.52 \text{ g/cm}^3$ & $w = 28.3\%$).....................91
Figure 16: Shear stress versus strain plot ($\rho_d = 1.48 \text{ g/cm}^3$ & $w = 30.2\%$).........................91
Figure 17: Shear stress versus normal stress plot ($\rho_d = 1.48 \text{ g/cm}^3$ & $w = 30.2\%$).............92
Figure 18: Shear stress versus suction plot ($\rho_d = 1.48 \text{ g/cm}^3$ & $w = 30.2\%$).....................92
Figure 19: Shear stress versus strain plot for saturated compacted soil ($\rho_d = 1.59 \text{ g/cm}^3$ & $w = 23.9\%$)...............................................................................................................93
Figure 20: Shear stress versus normal stress plot for saturated compacted soil ($\rho_d = 1.59 \text{ g/cm}^3$ & $w = 23.9\%$)...............................................................................................................93
Figure 21: Shear stress versus strain plot for natural soil ($\rho_d = 1.37 \text{ g/cm}^3$ & $w = 33\%$)..................94

Figure 22: Shear stress versus normal stress plot for natural soil ($\rho_d = 1.36 \text{ g/cm}^3$ & $w = 33\%$).........................................................................................................................94

Figure 23: Shear stress versus suction plot for natural soil ($\rho_d = 1.36 \text{ g/cm}^3$ & $w = 33\%$).........................................................................................................................95

Figure 24: Shear stress versus strain plot for saturated natural soil ($\rho_d = 1.36 \text{ g/cm}^3$ & $w = 33\%$)........................................................................................................................................95

Figure 25: Shear stress versus normal stress plot for saturated natural soil ($\rho_d = 1.36 \text{ g/cm}^3$ & $w = 33\%$).................................................................................................................................96
CHAPTER 1

INTRODUCTION

1.1. Problem Statement

The volume change properties of expansive soils (water addition causing swelling and water removal causing shrinkage) lead to severe management issues related to the integrity of civil infrastructure that generally involves soil compaction. Most of the infrastructure has been constructed on/with compacted soils. Shear strength of compacted soil is an important part of geotechnical engineering because of the role it plays in: (i) the evaluation of bearing capacity of foundations for residential and commercial facilities, (ii) the evaluation of stability of the slope for highway embankments, earth dams, artificial canals, excavations and (iii) the design of earth retaining structures like retaining walls, sheet piles and coffer dams. The difficulty lies in the evaluation of the shear strength, and more complex situations occur when the soil state is unsaturated. Compacted soils exhibit a relatively higher strength at the time of construction; however, their strength generally decreases with time. Although the province of Saskatchewan, including the provincial capital city of Regina, is considered as flat land, slope failures occur in the area. Figure 1.1 shows a slope failure of natural deposits, and Figure 1.2 shows a canal side failure at Deer Valley, northwest of Regina. All of these indicate instability in natural slopes. Furthermore, the provincial capital (Regina) is undergoing significant construction activities that include large-scale projects such as the Global Transportation Hub, with an area of $8 \times 10^6 \text{ m}^2$, and several tall buildings, along with the associated highway interchanges and buried infrastructure. Figure 1.3 and Figure 1.4 shows ongoing construction on the local soil for different slope angles. To ensure the stability of existing and new structures, there is a need to develop a clear understanding
Figure 1.1: Canal side slope failure at Deer Valley in Northwest of Regina

Figure 1.2: Slope failure of natural deposit at Deer Valley in Northwest of Regina
Figure 1.3: Ongoing infrastructure construction (mild slope) on local expansive soil

Figure 1.4: Ongoing infrastructure construction (steep slope) on local expansive soil
of the unsaturated shear strength properties of compacted expansive soils.

Long-term stability of slopes depends on the soil strength achieved at the compaction of construction and afterward climate conditions and responding unsaturated property of soils. The presence of a hairline crack eventually weakens the slope. According to Terzaghi et al. (1996), almost every stiff clay is weakened by a network of discontinuities. If the surfaces of weakness subdivide the clay into fragments smaller than about 25 mm, a slope may become unstable during construction or shortly thereafter. On the other hand, if the spacing of the joints is greater, failure may not occur until many years after the cut is made. The reduction in strength with time, due to the presence of cracks, has been attributed to swelling and softening due to water infiltration in the hairline cracks, especially when stress relaxation and crack opening occurs in excavated slopes.

1.2. Research Objectives

The main objective of this research was to understand the shear strength properties of the compacted expansive soils. The key elements of this research are:

- To determine the geotechnical index properties for preliminary soil assessment and for subsequent analysis.
- To determine of the compaction curve for evaluating the behaviour of compacted expansive soils and obtaining samples for subsequent testing using the standard compaction effort.
- To measure the stress-strain behaviour and, thereby, determine the shear strength properties, friction angle ($\phi'$), and cohesion ($c'$) of compacted and natural samples using direct shear testing.
• To determine the unsaturated shear strength property (friction angle due to suction ($\phi_b$)) of compacted and natural samples using direct shear testing in conjunction with suction measurement.

• To estimate the unsaturated shear strength property (friction angle due to suction ($\phi_b$)) of compacted and natural samples using saturated shear strength properties ($c'$, $\phi'$) and plasticity index in conjunction with the soil water characteristic curve.

• To analyze slope stability using parametric study of saturated and unsaturated shear strength properties under both natural and compacted conditions.

1.3. Thesis Outline

The thesis is composed of five chapters. Chapter 1 gives the introduction of the research topic and establishes the need for and objectives of this work. The theoretical background of the present study and a review of the literature about shear strength properties of unsaturated expansive soil are given in Chapter 2. In Chapter 3, the experimental methods are explained by presenting the procedures of samples preparation followed by describing the testing program. The test results obtained in each experimental series with the interpretation and discussion of test results are presented in Chapter 4. Finally, the conclusions obtained from the present investigation and recommendations for future work are given in Chapter 5. This is followed by a list of references and an appendix.
CHAPTER 2

Literature Review

2.1. General

Expansive soils are found in various parts of the globe, including Argentina, Australia, Brazil, Canada, China, Cuba, Ethiopia, Ghana, India, Israel, Iran, Japan, Mexico, Morocco, Myanmar, Oman, Peru, Saudi Arabia, South Africa, Spain, Sudan, Turkey, United States of America, Venezuela, and Zimbabwe (Chen, 1988; Steinberg, 1998). Expansive soils are formed in areas where the annual evapotranspiration exceeds the precipitation (Chen, 1988). The relationship between the climatic similarity of the regions and the existence of expansive soils in the areas is evident. Slater (1983) described that the distribution of expansive soils is generally the result of geologic history, sedimentation, and local climatic conditions. This type of soil has been characterized by the presence of high amounts of expansive clay minerals. Expansive soils are typically consists of 2:1 expansive clay minerals such as montmorillonite. These minerals evolved in geologic time under an arid and semi-arid climate that, in turn, governs the volume change behaviour, that affects the shear strength property.

2.2. Compaction Curve

In geotechnical engineering, compaction is defined as the densification of soils by the application of mechanical energy (Holtz et al., 2011). Soil compaction is a general practice in geotechnical engineering to construction road, dams, landfills, airfields, foundations, hydraulic barriers, and ground improvements. Compaction is applied to the soil, with the purpose of finding optimum water content in order to maximize its dry density which eventually decreases long term compressibility, increases shear strength,
and sometimes reduces permeability. Proper compaction of materials ensures the durability and stability of earthen constructions. A typical compaction curve presents different densification stages when the soil is compacted with the same apparent energy input but different water contents. The water content at the peak of the curve is called the optimum water content (OWC) and represents the water content at which dry density is maximized for a given compaction energy. Several different methods are used to compact the soil in the field, such as tamping, kneading, vibration, and static load compaction. However, laboratory tests employ the tamping or impact compaction method using the type of equipment and methodology developed by Proctor (1933). This is known as the proctor test. Two types of compaction tests are commonly used in laboratory tests, (i) The Standard Proctor Test, and (ii) The Modified Proctor Test. The aforementioned parameters ($\rho_{\text{dmax}}$, $w_{\text{opt}}$) are not unique for various types of soils and vary with the type of soils and the compaction energy.

Zein (2000) showed that, compacted materials are highly aggregated on the dry side of optimum moisture content, albeit aggregation does not exist on the wet of optimum and also noted that there were no aggregations at optimum water content (and also wet of optimum). However, the degree of aggregation increased as the water content reduced below optimum moisture content. At moisture contents below 70% of OWC the material was completely aggregated with no matrix material.

The effect of compaction water content (at three points of the compaction curve: dry, OWC, and wet) on the microstructure of Jossigny silt (the clay fraction is 34%) was studied by Delage et al. (1996). At optimum water content, a more massive structure with less obvious aggregates occurred. The higher density is a result of lower resistance to
deformation of the aggregates, which deform and break down more easily; reducing in particular the interaggregate pores. On the wet side, due to hydration, the clay particles volume is much larger and forms a clay paste surrounding the silt grains.

According to Toll (2000) fabric plays a vital role in determining the engineering behavior of compacted soils. Clayey materials compacted dry of optimum moisture content develop an aggregated or ‘packet’ fabric. The presence of aggregations causes the soil to behave in a coarser fashion that would be justified by the grading. For soils compacted to degrees of saturation of 90% and over, the material would be expected to be non-aggregated. As the degree of saturation drops, the amount of aggregation increases rapidly and reaches a fully aggregated condition for degrees of saturation below 50%.

2.3. Soil Water Characteristics Curve

The soil water characteristic curve is an important constitutive feature of unsaturated soils, defining the relationship between the soil suction and the water content. The soil water characteristic curve (SWCC) gives the relationship between the amount of water in the soil (gravimetric or volumetric water content) and soil suction (matric suction at low suction and total suction at high suction). Many properties of an unsaturated soil such as the coefficient of permeability, shear strength and volume strain, pore size distribution, and the amount of water contained in the pores at any suction, can be estimated from the SWCC.

Figure 2.1 shows a typical soil water characteristics curve. To determine the SWCC, the most important part is to measure an accurate suction value corresponding to the water content. There are different procedure and instrument to measure the suction
Figure 2.1: Typical soil water characteristics curve
value either by direct suction measurement or applying by predefined suction. Pressure plate extractor, pressure membrane extractor, Chilled Mirror Hygrometer, centrifuge, hanging column, filter paper method are the instruments that are most commonly used for suction measurement. A typical curve during soil de-saturation usually consists of three stages: capillary saturation, de-saturation and residual saturation. The air-entry value ($AEV$) of the soil (bubbling pressure) is the matric suction where air starts to enter the largest pores in the soil. The residual water content is the water content where a large suction change is required to remove additional water from the soil. When the suction value exceeds the air-entry value ($AEV$), the degree of saturation decreases rapidly at relatively low suction values, and then, reduces more gradually when the suction becomes high. There is a large range of $AEVs$ corresponding to different void ratio values. The denser the soil, the higher the $AEV$, which implies that for soils with low void ratio values, small changes in degree of saturation can be assumed at low suctions (the soil can be treated as fully saturated). This might be a helpful observation when soils from different depth are being dealt with.

Regardless of the initial conditions of water content (dry of optimum, optimum and wet of optimum) and stress history, the soil-water characteristic behaviour appears to be similar at higher suctions (20000 kPa – 300000 kPa). In other words, as Vanapalli et al. (1999) explained, the inner forces between soil aggregates are very strong in resisting de-saturation behaviour at the higher suction values. Presumably, water films surrounding soil grain at these suctions are so thin that all the water is within the range of influence of osmotic and adsorptive fields. This is because soil structure (aggregation) seems to have negligible influence on the soil water characteristic behaviour in this high suction range.
It indicates that, the effect of initial water content and stress history is negligible at high suction value.

To discuss the influence of soil structure, Yan-hua et al. (2002) have compared the soil water characteristic curve of undisturbed soil sample with that of compacted soil sample. The curve of the compacted soil sample has an obvious inflection point. It can be considered approximately as two straight line segments. The first one is horizontal straight line segment and reflects the air entry stage of the soil sample (completely closed state). The other one is an oblique line segment and is corresponding to the interior connected state of unsaturated soil. For undisturbed sample, the curve shape is smooth and it does not have not obvious turn point.

The theoretical explanation is that the pore structure of undisturbed soil has certain directivity and, even, some connected channels exist in undisturbed soil because soil particles are arranged according to certain direction and order during the formation process of undisturbed soil. During the drainage process of soil samples, air enters to the bigger size channels at first and finally excretes the pore water in the channels that occupies a fairly large proportion in total pore water. With the increase in suction value increasing, water in channels decreases gradually and water in non-channel excretes slowly. So the soil has a high water-holding capacity. Therefore, undisturbed soil sample has a lower air entry value and a high water-holding capacity. For compacted soil sample, pores in soil are basically uniformly distributed and non-directional. The pore gradation of the soil is poorly graded. This is because it is difficult for air entry and, thus, soil has a high air entry value. The experiments showed that the soil structure has a great influence on soil water characteristic curve (Li et al., 2009).
2.4. Swell-shrink Curve

The change in void ratio with the change in water content for an expansive soil as a result of desiccation and water absorption provides an understanding of swelling and shrinkage behaviour (Hanafy, 1991). A significant amount of research previously conducted for the understanding of the soil volume change as a function of water content during shrinkage (Dasog et al., 1988). Figure 2.2 shows the relationship between void ratio and water content which is also known as swell-shrink curve. The shrinkage curve of a soil specimen shows different phases of deformation. Haines (1923) distinguished the different phases of shrinkage that resulting from the progressive drying of natural soils: (i) structural shrinkage, (ii) normal shrinkage, and (iii) residual shrinkage. During structural shrinkage, a few large, stable pores are emptied and the decrease in volume of the soil is less than the volume of water loss. During the normal shrinkage phase, the volume decrease is equal to the volume of water loss (the slope of the total specimen volume versus water content line is 45°). On further drying, the slope of the shrinkage curve changes and air enters the voids at the shrinkage limit or at the start of residual shrinkage. As the particles come in contact, the decrease in specimen volume is less than the volume of water loss. Finally, when all the particles come close together, no further shrinkage occurs while water is still being lost. This phase has been identified as the no-shrinkage stage by Stirk (1954).

Hanafy (1991) proposed a characteristic S-shaped curve to describe the potential volume change of an expansive clayey soil for the change in void ratio relative to changes in water content resulting from desiccation and water absorption. The S-shaped curve can be determined in the laboratory using conventional consolidation test equipme-
Figure 2.2: Typical swell-shrink curve
-nt by carrying out one complete swelling shrinkage test with two additional or partial tests. Expansive soil specimens, desiccated or partially desiccated with initial water content ($w$) and a corresponding natural void ratio ($e_o$) are used to trace the path during testing. The S-shaped curve can be used to classify the swelling potential of desiccated expansive clayey soils. Although swelling and shrinkage of expansive clays are interrelated, it is not certain that highly swelling clays will show equally high shrinkage upon drying (Mitchell, 1976 and Chen, 1988).

A typical swell-shrink test data depicted on a void ratio versus water content plot depicts an S-shaped curve (Tripathy et al., 2002). Theoretical lines for various degrees of saturation appear as straight lines with various slope angles emanating from the origin. This plot can be used to estimate both the degree of saturation and the void ratio of the soil for given water content. Ho et al. (1992) conducted shrinkage tests on compacted glacial till and compacted silt specimens. The test results showed that the shrinkage test curve moves away from the saturation line as the initial water content is lowered. Shrinkage tests on compacted soils have provided information on both volumetric and axial shrinkage (Kezdi, 1980; Sitharam et al., 1995). Subba Rao and Satyadas (1985) conducted shrinkage tests on compacted specimens and reported a curvilinear relationship between volumetric and linear shrinkage.

Padmanabha (1988) has shown that for compacted soils, the linear shrinkage generally lies within one quarter to one sixth of the volumetric shrinkage for initial and later parts of shrinkage. However, the specific ranges are not well defined. With the relationship between void ratio and the water content at various matric suctions, Fredlund and Rahardjo (1993) have stated the shrinkage and swelling relationship. The shrinkage
curve must be used in conjunction with the soil water characteristics curve, if along
drying path water content gradually decreases and the air-entry value of the soil is
indistinct.

2.5. Shear Strength for Expansive Soils

2.5.1. Friction Angle

Friction angle for a given soil is the angle on the graph (Mohr's Circle) of the shear stress
and normal effective stresses at which shear failure occurs. Friction angle of soil is
generally denoted by "φ". Gravels with some sand typically have a friction angle of 34° to
48°, loose to dense sand have 30° to 45°, silts have a friction angle of 26° to 35° and clay
have around 20°. Well graded soils have high values of friction angle. (Pa’lossy et al.,
1993).

The friction angle is a function of the characteristics like particle size, compaction
effort and applied stress level (Hawley, 2001; Holtz and Kovacs, 2003). Friction angle
increases with the increase in particle size (Holtz, 1960) whereas Kirkpatrick (1965)
made it more specific by indicating that the friction angle increases as the maximum
particle size increases. Friction angle also increases with the increase in angularity and
surface roughness (Cho et al, 2006). With an increase of density or decrease in void ratio,
friction angle increases (Bishop, 1996). Bhandary and Yatabe (2007) reported that
friction angle decreased with the increasing values of expansive mineral ratio (relative
amount of expansive clay mineral to non-expansive clay mineral).

2.5.2. Cohesion

Cohesion is one of the important components of shear strength soil mainly for fine
materials. Cohesion is the attraction by which soil particles are united throughout the
mass. Cohesion is the strength of soil which behaves like glue that binds the grains together. Cohesion of soil is usually denoted by "c". Rock has a cohesion value of 10,000 KPa, whereas silt has 75 KPa and clay has 10 to 20 kPa. Depending on the stiffness of the clay soft to high, cohesion varies from 0 to 766 kPa. Natural minerals that have been leached into the soil, such as caliches and salts, can provide a very strong cohesion. Heat fusion and long term overburden pressure will tend to fuse the soil grains together, producing significant cohesion. Table 2.1 summarizes several shear strength properties determined by researcher for different unsaturated cohesive soils.

2.5.3. Friction Angle due to Suction

Friction angle due to suction is the third component of shear strength which is associated with unsaturated soil mechanics. This is the cohesive strength associated with matric suction. This gives resistance of the particles to being pulled apart because of the surface tension of the thin layer of water surrounding each particle. The shear strength increases with respect to an increase in matric suction which is defined by the angle $\phi$. At zero suction, $\phi$ may assume to be the saturated friction angle (Gan and Fredlund, 1996). Then the $\phi$ decreases with the increase in matric suction. It appears that the angle is a function of matric suction. The shear strength contribution due to matric suction was initially assumed to be linear based on the analysis of limited results published in the literature. Later, experimental studies performed over a large range of suction values have shown that the variation of shear strength with respect to soil suction is non- linear (Escario and Juca, 1989).
Table 2.1: Shear strength properties of unsaturated cohesive soils

<table>
<thead>
<tr>
<th>Reference</th>
<th>Material</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>w (%)</th>
<th>$\gamma_d$ (g/cm$^3$)</th>
<th>Test Type</th>
<th>$c'$ (kPa)</th>
<th>$\phi'$ ($^\circ$)</th>
<th>$\phi^b$ ($^\circ$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ye et al. (2010)</td>
<td>Hubei clay</td>
<td>-</td>
<td>24</td>
<td>18</td>
<td>-</td>
<td>TR-SC</td>
<td>76.9</td>
<td>24.6</td>
<td>17.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(at 100 kPa suction)</td>
</tr>
<tr>
<td>Rahadrjo et al.</td>
<td>Jurong clay</td>
<td>36</td>
<td>22</td>
<td>16</td>
<td>1.71</td>
<td>TR-SC</td>
<td>80</td>
<td>31.5</td>
<td>29</td>
</tr>
<tr>
<td>(2004)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(at 50 kPa suction)</td>
</tr>
<tr>
<td>Taha et al. (2000)</td>
<td>Residual clay</td>
<td>-</td>
<td>-</td>
<td>21</td>
<td>1.38</td>
<td>TR-SC</td>
<td>58</td>
<td>26.5</td>
<td>17.8</td>
</tr>
<tr>
<td>Gan et al. (1988)</td>
<td>Glacial till</td>
<td>36</td>
<td>17</td>
<td>16</td>
<td>1.82</td>
<td>DS-SC</td>
<td>10</td>
<td>25.5</td>
<td>20.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(at 100 kPa suction)</td>
</tr>
<tr>
<td>Y. Hepig (2005)</td>
<td>Ningming clay</td>
<td>60</td>
<td>27</td>
<td>23</td>
<td>1.31</td>
<td>DS-NSC</td>
<td>36.4</td>
<td>15.6</td>
<td>-</td>
</tr>
<tr>
<td>Cokca et al. (2004)</td>
<td>Ankara clay</td>
<td>56</td>
<td>23</td>
<td>18</td>
<td>1.47</td>
<td>DS-NSC</td>
<td>40</td>
<td>44</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(at 2059 kPa suction)</td>
</tr>
<tr>
<td>Kong &amp; Tan (2000)</td>
<td>Hubei clay</td>
<td>82</td>
<td>35</td>
<td>29</td>
<td>1.87</td>
<td>DS-NSC</td>
<td>92.3</td>
<td>10.4</td>
<td>-</td>
</tr>
<tr>
<td>Shanker et al. (1989)</td>
<td>Black cotton soil</td>
<td>63</td>
<td>24</td>
<td>10</td>
<td>1.44</td>
<td>DS-NSC</td>
<td>3.9</td>
<td>21</td>
<td>-</td>
</tr>
</tbody>
</table>

TR-SC = Suction controlled triaxial test  
DS-SC = Suction controlled direct shear test  
DS-NSC = Suction not controlled direct shear test
2.6. Factors Affecting Shear Strength of Expansive Soils

Several factors affect the shear strength of clay, that includes:

(i) Clay content: The amount of clay fraction within the soil mass has its serious impact on the cohesion and the friction angle. For water content slightly above the optimum water content the increase in clay content improves the cohesion. This improvement may not be attained if the moisture is far above the optimum water content. The friction angle decreases with the increase in clay content.

(ii) Clay mineralogy: Presence of clay mineral reduces the shear strength of clay. Clay minerals are consistently weaker than natural rock-flour gouges composed of crushed granitic material. The expandable clay montmorillonite is by far the weakest of the clay minerals (Morrow et al., 1984). Swelling and shrinkage in expansive soils are of two extreme opposite effects on the shear strength. The shear strength is generally low for fully expanded clay while dry shrinking clay is capable of developing higher cohesion and friction angle.

(iii) Plasticity index: An increase in plasticity index reduces the shear strength. High plastic material has lower shear strength.

(iv) Water content: Cohesion increases with the increase of water content up to optimum water content, beyond which it reduces with the increase of water content. Friction decreases with the increase of water content and approaches a constant value near optimum water content. Generally, shear strength reduces with the increase of water content as shear strength contribution by suction reduces.

(v) Dry density: Shear strength increases with the increase of dry density.

(vi) Strain rate: The effect of shearing rate is significant and depends on the testing arrangements with regard to drainage conditions and the type of soil tested.
Generally, the strain rate is very low in clays to allow for dissipation of pore water pressure. Several days may be required to finish a single test. However, drained strength obtained in a test using a rate of 1.2 to 1.3 mm/min can give a better approximation for undrained specimen (Bowles, 1995). Ladd (1991) reported that undrained shear strength increases with the increase in shear strain rate. Boulanger and Idriss (2007) stated that the fast direct shear tests would underestimate the undrained strength.

(vii) Clay softening: Softening is the decrease in peak strength prior to failure. Softening is often invoked to justify slides in overconsolidated clay, but a general consensus about the mechanics of this phenomenon has not been achieved. Literally, softening is the result of water content increase due to a change of the state of stress. Terzaghi (1936), first, observed that fissured OC clays can experience some shear strength decay as a consequence of swelling induced by unloading. Some English authors attributed to softening the delayed failure of cuttings in London Clay. In one of his latest paper on this subject, Skempton (1977) remarked that delayed failure is caused by dissipation of negative excess pore pressures triggered by excavation, a process that does not affect the shear strength properties, but only the effective state of stress. However, he did not exclude that during the long-lasting phase of pore pressure equalization, some decay of the shear strength properties can take place. In another important paper touching the same issue, Morgenstern (1985) outlined that swelling can provoke a decrease in the dilative and brittle behaviour of clay, causing a decrease in the shear strength through a loss of its component associated with overconsolidation: therefore, the long-term strength, the so called fully-softened strength, could be very close to the critical value.
According to Terzaghi (1936), the mechanism of shear strength decrease in fissured stiff clays is due to opening of fissures, swelling of the adjacent clay under practically zero confining stress and reconsolidation of clay under its own weight. However, this mechanism does not apply to all cases, especially to slightly fissured clay. Similar considerations as those made by Terzaghi, were reported by Skempton (1970) tens of years after, but thinking to fissures induced, also in intact clay, by mechanisms of shear. Just to test, under the umbrella of the Critical Strength Theory, the effects of swelling on shear strength, Calabresi and Scarpelli (1985) and Rampello (1987) performed CIU triaxial tests on some Italian non-fissured or slightly fissured OC clays, pre-swelled in the laboratory under a very low confining stress. They noticed that swelling can bring about some decrease in cohesion due to increase in the water content. However, cohesion does not completely vanish and the soil behaviour remains dilative due to overconsolidation. It is worth mentioning that softening, as described above, has some similarities with other phenomena that are responsible for time-depending decay of shear strength, such as weathering, slaking, i.e. soil destructuration caused by cycles of wetting-drying or of freezing-thawing (Botts, 1998; Graham and Au, 1985) and fatigue (Lacerda, 1989; Eigengbrod et al., 1992). Through accumulated plastic strains, all these phenomena, generally concentrated in the most superficial soil layers, can determine a loss of that part of the shear strength that depends on interparticle bonding, causing a reduction in cohesion. Therefore, they affect only bonded clays.

Leroueil and Vaughan (1990) and Hight et al. (2002) assume that even simple swelling can provoke destructuration of bonded clays. This idea has been recently resumed by Takahashi et al. (2005) bearing on the results of laboratory tests on
undisturbed specimens of London clay. All mechanisms mentioned above show how complicated is the interpretation of slope instability in stiff overconsolidated clay and clay shale, since more than one of them can contemporaneously act in the same slope at same time. Furthermore, strain-softening (progressive failure) and rate effects can play an additional and significant role. However, laboratory data and field observations on highly fissured plastic clay shales of Italian Apennines show that a decrease in the shear strength could be caused by chemical-physical processes provoked by exposure of soil to fresh water.

2.7. Shear Strength of Saturated Soil

The shear strength of a saturated soil is described using the Mohr-Coulomb failure criterion and the effective stress concept (Terzaghi, 1936).

\[
\tau_f' = c' + (\sigma_f - u_w)f \tan \phi' 
\]

(2.1)

where, \(\tau_f\) = shear stress on the failure plane at failure, \(c'\) = effective cohesion, \((\sigma_f - u_w)f\) = effective normal stress on the failure plane at failure, \(\sigma_{ff}\) = total normal stress on the failure plane at failure, \(u_wf\) = pore-water pressure at failure, \(\phi'\) = effective friction angle.

The equation 2.1 denotes a straight line which commonly referred to as a failure envelope. This envelope represents possible combinations of shear stress and effective normal stress on the failure plane at failure. Each of the Mohr circle indicates different deviator stress that is represented by the diameter of the circle. The failure envelope indicates the shear strength of the soil for each effective normal stress as the pore water pressure acts isotropically on all planes. The shear stress described by the failure envelope indicates the shear strength of the soil for each effective normal stress. The failure envelope is obtained by plotting a line tangent to a series of Mohr circles representing
Figure 2.3: Mohr-Coulomb failure envelope for saturated soil

\[
\tau_{ff} = c' + (\sigma_f - u_w)\tan \phi'
\]

Figure 2.4: Extended Mohr-Coulomb failure envelope for unsaturated soil

\[
\tau_{ff} = c' + (\sigma_f - u_a)\tan \phi' + (u_a - u_w)\tan \phi_b
\]
failure conditions. The slope of the line gives the effective friction angle ($\phi'$), and its intercept on the ordinate is called the effective cohesion ($c'$). The direction of the failure plane in the soil is obtained by joining the pole point to the point of tangency between the Mohr circle and the failure envelope. The tangent point on the Mohr circle at failure represents the stress state on the failure plane at failure. This failure criterion is only applicable for the saturated soil.

2.8. Shear Strength of Unsaturated Soil

Bishop (1959) has modified the Terzaghi’s classic effective stress theory to define the shear strength of unsaturated soil. Terzaghi’s effective stress approach modified by Bishop has written as:

$$\tau_f = (\sigma_f - u_a)_f + \chi (u_a - u_w)_f$$

(2.2)

where, the effective stress parameter $\chi$ is generally considered to vary between zero and unity as a function of the degree of saturation. The difference $(\sigma_f - u_a)_f$ is the net normal stress and the difference $(u_a - u_w)_f$ is matric suction. For $\chi$ equal to zero (corresponding to perfectly dry conditions) and for $\chi$ equal to unity (corresponding to saturated conditions).

Following Bishop’s approach, the macroscopic engineering behaviour of unsaturated soil is described using the effective stress defined by the above equation within the established framework of saturated soil mechanics. Shear strength can be described by incorporating the modified effective stress expression into the classical Mohr–Coulomb failure criterion.

Fredlund et al. (1978) showed that the shear strength of unsaturated soils can be described by any two of the three possible stress state variables, $(\sigma - u_a)$, $(\sigma - u_w)$ and $(u_a - u_w)$ can be used for the shear strength equation. The stress state variables, $(\sigma - u_a)$ and
(\(u_a - u_w\)), have been shown to be the most advantageous combination in practice. Using these stress variables, the shear strength equation is written as follows:

\[
\tau_{ff} = c' + (\sigma_f - u_w) \tan \phi' + (u_a - u_w) \tan \phi_b
\]  \hspace{1cm} (2.3)

where, \(c'\) = intercept of the “extended” Mohr-Coulomb failure envelope on the shear stress axis, \((\sigma_f - u_w)\) = net normal stress state on the failure plane at failure, \(\phi'\) = friction angle, \((u_a - u_w)\) = matric suction on the failure plane at failure, \(\phi_b\) = angle indicating the rate of increase in shear strength relative to the matric suction.

Figure 2.4 shows a three dimensional plot of extended Mohr-Coulomb failure envelope rather than a two dimensional plot for saturated soil. The third axis indicates matric suction perpendicular to normal stress and shear stress. A comparison of equation 2.1 and equation 2.2 reveals that the shear strength equation for an unsaturated soil is an extension of the shear strength equation for a saturated soil. For an unsaturated soil, two stress state variables are used to describe its shear strength, while only one stress state variable (effective normal stress \((\sigma_f - u_w)\)) is required for a saturated soil. The shear strength equation for an unsaturated soil exhibits a smooth transition to the shear strength equation for water pressure \((u_w)\) approaches the pore-air pressure \((u_a)\) and the matric suction approaches zero. The matric suction component in not present in saturated soil, and, therefore, equation 2.1 reverts to the equation for a saturated soil.

Lamborn (1986) proposed shear strength equation for unsaturated soils by extending a micromechanics model based on principles of irreversible thermodynamics to the energy versus volume relationship in multi-phase material. The equation is as follows

\[
\tau_{ff} = c' + (\sigma_f - u_a) \tan \phi' + (u_a - u_w) \theta_w \tan \phi'
\]  \hspace{1cm} (2.4)
where, $\theta_w$ = volumetric water content which is defined as the ratio of the volume of water to the total volume of soil.

The volumetric water content, $\theta_w$, decreases as matric suction increases and it is a non-linear function of matric suction. However, the friction angle due with matric suction does not become equal to $\phi'$ at saturation unless the volumetric water content is equal to one. Equation of shear strength was also proposed by Satija (1978), Karube (1988) and Toll (1990). Most of the shear strength equations for unsaturated soils in the literature are either linear or bi-linear approximations. A non-linear model is more realistic and could provide a better approximation.

2.9. Direct Shear Testing of Unsaturated Expansive Soils

The direct shear test is particularly useful for testing of unsaturated soils due to the short drainage path in the specimen, in spite of having issues with stress concentrations, definition of the failure plane and the rotation of principal stresses. The shear strength testing of unsaturated soils is generally performed at a constant strain rate and, therefore, an appropriate strain must be selected before commencing a test. In undrained shear strength test, the selected strain must ensure equalization of pore pressure throughout the specimen. In drained shear, the selected strain must ensure complete dissipation of the pore pressures. The estimation of strain rate for testing soils in triaxial and direct shear can be made partly on the basis of the experimental evidence and partly on the basis of theory. The strain at failure depends on the soil type and the stress history of the soil. Table 2.1 present typical values of strain at failure, obtained from numerous direct shear testing programs on unsaturated soils. This information can be of value as a guide when attempting to establish a suitable strain rate.
Several researchers have done direct shear tests on unsaturated soils. Multistage direct shear tests have been performed on saturated and unsaturated specimens of a compacted glacial till by Gan et al. (1988). Consolidated drained direct shear tests were performed on five compacted specimens with a displacement rate of $1.7 \times 10^{-4}$ mm/s. The matric suction ranged from 0 to 500 kPa. The results fall within a band, forming curved failure envelopes. The angles corresponding to the failure envelopes are plotted with respect to matric suction. The $\phi'_b$ angles commence at a value equal to $\phi'$ at matric suction close to zero, and decrease significantly at matric suctions in the range of 50-100 kPa. The $\phi'_b$ angles reach a fairly constant value ranging from 5° to 10° when the matric suction exceeds 250 kPa. Zhao and Ng (2006) have performed suction controlled direct shear tests on natural and compacted unsaturated expansive clay of China with a clay content of 39%. Their investigation showed a non-linear relationship between shear strength and matric suction at lower suction range (0 to 200 kPa) for compacted and natural soils and at a particular suction natural soil possesses higher peak shear strength compared to the compacted soil. Kong and Tan (2000) have studied the shear strength characteristics of an expansive soil. The correlation between the shear strength properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Displacement rate, $dh$ (mm s$^{-1}$)</th>
<th>Displacement at failure, $dh_f$ (Mm)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Madrid gray clay</td>
<td>$1.4 \times 10^{-4}$</td>
<td>3.5-5</td>
<td>Escario (1980)</td>
</tr>
<tr>
<td>Madrid gray clay</td>
<td>$2.8 \times 10^{-5}$</td>
<td>6.0-7.2</td>
<td>Escario &amp; Saez (1986)</td>
</tr>
<tr>
<td>Red clay of Guadalix de la Sierra</td>
<td>$2.8 \times 10^{-5}$</td>
<td>4.8-7.2</td>
<td>Escario &amp; Saez (1986)</td>
</tr>
<tr>
<td>Madrid clayey sand</td>
<td>$2.8 \times 10^{-5}$</td>
<td>2.4-4.8</td>
<td>Escario &amp; Saez (1986)</td>
</tr>
<tr>
<td>Glacial till</td>
<td>$1.7 \times 10^{-5}$</td>
<td>1.2</td>
<td>Gan (1986)</td>
</tr>
</tbody>
</table>
and initial water content of unsaturated compacted expansive soil was established. Two of the data points were at dry of optimum two of them were near optimum and five of them were at wet of optimum. They found that the cohesion reduces with the increase of water content, the friction angle decreases significantly in the range of water contents less than the plastic limit (34.7%), whereas as the water content is more than the plastic limit, the friction angle tends to a constant value.

2.10 Suction Characteristics

The measurement of soil suction is crucial for applying the theory behind the unsaturated soils. Soil suction and positive pore water pressure are two similar important parameters with respect to describing the behaviour of unsaturated and saturated soils, respectively (Houston et al., 1994). Soil suction is nothing but the unit attractive force of the soil for water (McKeen, 1977). The macroscopic soil property in terms of the intensity or energy level with which soil attracts water is called the total soil suction.

Soil suction comprised of two major component- osmotic suction and matric suction. Osmotic suction is the attraction of water due to the presence of dissolved salts in the pore fluid whereas matric suction is the attraction of water due to the capillary nature of unsaturated soils. Matric suction arises because of capillary forces, soil texture, and adsorption forces of clay particles whereas osmotic suction is the differences between the salt concentration of one point to another in the soil mass. Matric suction is an important parameter that affects the shear strength of unsaturated soils, and is considered to be a component of cohesion in unsaturated soil shear strength (Lu and Likos, 2004). Total suction generally expressed in units of pressure (kilopascals), is fundamentally related to the water content of soil. Usually, soil suction increases with the decreases of water and
content. Therefore, the soil suction value in dry soil is high compared to moist soil. In engineering practice, the soil suction is usually calculated in pF (the common logarithm of the pressure exerted per square centimeter by a water column measured in centimeters).

2.11. Prediction of Unsaturated Shear Strength

The shear strength of an unsaturated soil can be determined using modified direct shear or triaxial shear equipment. Experimental studies related to the determination of the shear strength of unsaturated soils are time consuming and require extensive laboratory facilities (Escario, 1980; Escario and Jucá, 1989; Gan et al., 1988). In recent years, several semi-empirical procedures were proposed to predict the shear strength of unsaturated soils (Vanapalli et al., 1996; Fredlund et al., 1996; Khalili and Khabbaz, 1997; Oberg and Salfours, 1997; Bao et al., 1998). The proposed prediction procedures use the effective shear strength properties \((c', \phi')\) along with the soil-water characteristic curve data to predict the shear strength of unsaturated soils. Vanapalli et al. (1996) and Fredlund et al. (1996) have proposed a general, nonlinear function for predicting the shear strength of an unsaturated soil using the entire soil water characteristic curve (0 to 1,000,000 kPa) and the saturated shear strength properties as shown below:

\[
\tau_{ff} = [c' + (\sigma_f - u_a) \tan \phi'] + [(u_a - u_w) \{ (\Theta^k \cdot \tan \phi) \}]
\]

(2.5)

where, \(k\) = fitting parameter used for obtaining a best-fit between the measured and predicted values, and \(\Theta = \) normalized water content, \(\theta_a/\theta_s\).

The shear strength contribution due to suction constitutes the second part of equation 2.5, which is:

\[
\tau_{us} = [(u_a - u_w) \{ (\Theta^k \cdot \tan \phi) \}]
\]
Equation 2.5 can also be written in terms of degree of saturation \((S)\) or gravimetric water content \((w)\), to predict the shear strength yielding similar results.

2.12. Parametric Study

The slope stability analyses are performed to assess the safe and economic design of human made or natural slopes (embankments, road cuts, open-pit mining, excavations, and landfills). In the assessment of slopes, engineers primarily use the factor of safety (FS) values to determine how close or far slopes are from failure. When this ratio is greater than 1, resistive shear strength is greater than driving shear stress and the slope is considered stable. When this ratio is close to 1, shear strength is nearly equal to shear stress and the slope is close to failure, if factor of safety is less than 1 the slope should have already failed. Limit equilibrium types of analysis for assessing the stability of earth slopes have been in use in geotechnical engineering for many decades. The software Slope/w (Geo-slope 2004) allows geotechnical engineers to carry out limit equilibrium slope stability analysis of existing natural slopes, unreinforced man-made slopes, or slopes with soil reinforcement. The program uses many methods such as: Bishop’s Modified method, Janbu’s Simplified method, Spencer method, Morgenstern-Price method and others. Slope/w all these methods to be applied to circular, composite, and non-circular surfaces. This method considers not only the normal and tangential equilibrium but also the moment equilibrium for each slice in circular and non-circular slip surfaces. It is used for the estimation of factor of safety using the summation of forces tangential and normal to the base of a slice and the summation of moments about the center of the base of each slice. The equations were written for a slice of infinitesimal thickness. The force and the moment equilibrium equations were combined and a modified Newton-Raphson numerical technique was used to estimate for the factor of
safety, satisfying force and moment equilibrium. The solution required an arbitrary assumption regarding the direction of the resultant of the interslice shear and normal forces.

2.13. Research Hypothesis

This section presents a basis for a comprehensive laboratory investigation program. The whole laboratory investigation program that would be divided into five stages: geotechnical index properties, compaction curve, shear strength properties, suction measurement and soil water characteristics curve followed by swell-shrink curve. Geotechnical index properties would be done to determine the index properties of investigated expansive soil. The compaction curve of investigated soil would be done to determine the optimum water content and maximum dry density, and to obtain the compacted sample to investigate the behaviour of compacted expansive soil. The shear strength properties would be determined for natural and compacted unsaturated samples using consolidated drained direct shear testing. Total nine sets of tests would be conducted where each set comprised of three single stage test. Suction would be measured at the time of failure for each test on unsaturated sample to determine the friction angle due to suction. Soil water characteristics curve along with swell-shrink curve would be determined to predict the unsaturated shear strength. The research will focus on understanding the shear strength properties of compacted expansive soils through the laboratory test and parametric study. Overall, the output would be expected to introduce a simpler method to determine unsaturated shear strength and effect of shear strength properties on stability analysis of structures from a fundamental point of view.
CHAPTER 3

RESEARCH METHODOLOGY

3.1 General

The research methodology was divided into two parts. The laboratory testing program was focussed on the shear strength properties of local expansive soil. The estimation and parametric study was carried out using the laboratory determined soil properties.

Regina clay was obtained from near the intersection of Lewvan Drive and highway number 1 in southern Regina, Saskatchewan. These samples were retrieved from a depth of 1.5 m in accordance with ASTM D1452-07a. Then samples were preserved in the sealed container and carefully transported to the Saskatchewan advanced geotechnical laboratory (RIC 027) at the University of Regina, where they were stored at a constant temperature of 21 °C.

Figure 3.1 shows the laboratory investigation program and Figure 3.2 and Figure 3.3 show the stepwise procedure to determine all the test properties. Each of the test procedures is described in this section whereas the test data and example calculations are given in the Appendix. Tests performed on investigated soil consisted of the determination of geotechnical index properties, standard proctor test, direct shear test, soil water characteristics curve and swell shrink curve.
Figure 3.1: Laboratory investigation program
Figure 3.2: (a) Laboratory direct measurement of saturated shear strength, (b) Laboratory direct measurement of unsaturated shear strength, (c) Estimation of unsaturated shear strength
\[
\tan \phi_b = \left( \frac{\theta_w}{\theta_s} \right) \kappa \tan \phi' \quad \text{(Vanapalli, 1996)}
\]

- Friction angle
  - Determined from direct shear test on saturated sample
- Fitting parameter
  - \( I_p \) versus \( \kappa \) plot (Vanapalli and Fredlund, 2000)
- Saturated volumetric water content
  - Obtained from the soil water characteristic curve
- Volumetric water content \( \{w \ (\gamma_d / \gamma_w)\} \)
  - Obtained from compaction curve
- Friction angle due to suction

Figure 3.3: Estimation of unsaturated shear strength property (\( \phi_b \))
3.2. Geotechnical Index Properties

3.2.1. Water Content

Water content (w) is the amount of water present in the soil and is represented as percentage. Water content was determined according to ASTM D2216-05. The following equation was used to determine the water content:

\[ w(\%) = \left( \frac{M_{c_ms} - M_{c_ds}}{M_{c_ds} - M_c} \right) \times 100 \]

where, \( w \) = water content (\%), \( M_{c_ms} \) = mass of moisture can and moist soil, \( M_{c_ds} \) = mass of moisture can and dry soil, \( M_c \) = mass of moisture can

3.2.2. Dry Unit Weight

Dry unit weight (\( \gamma_d \)) is defined as the mass of soil solids divided by total volume of soil. It was determined by using the method described by Ito and Azam (2010) in accordance with ASTM D4943-08. The determination of soil volume was conducted through the water displacement method. Each soil specimen was coated with the molten microcrystalline wax (\( G_s = 0.91 \)) and permitted to cool at room temperature. The mass of soil samples before and after the wax coating were recorded. The soil sample upon solidification of wax was submerged into a 250 mL graduated cylinder filled with distilled water. The height of water in the beaker was also recorded before and after the submersion of the sample. Since, the volume of water displaced was equivalent to the volume of wax coated soil sample, the volume of soil sample was obtained by subtracting the volume of wax (mass/0.91) from the volume of wax coated sample.

3.2.3. Specific Gravity

Specific gravity (\( G_s \)) is the ratio of the mass of soil solid to the mass of an equal volume of distilled water at 4 °C. The specific gravity was determined by ASTM D854-10. A
clean and dry pycnometer was weighed to the nearest 0.01 g. Distilled water was de-aired using the vacuum pump and that was kept overnight to remove all the air bubbles. Then the distilled and de-aired water was added to the pycnometer up to the calibration mark of 500 ml. The mass of pycnometer and water and temperature were measured. Around 100g of soil was dispersed and with the distilled water soil was made to slurry. Slurry was poured into the pycnometer and remaining soil particle was carefully washed with spray squirt bottle to pour into the pycnometer. More water was added to make around two third volume of the pycnometer. Then vacuum pump was connected to the pycnometer and operated for 4 hours to remove entrapped air from the soil slurry. After the de-airing process was completed, the pycnometer was filled with de-aired distilled water to the calibration mark and weight of the pycnometer was measured. The soil slurry was transferred to the evaporating dish and it was kept in an oven that maintained the temperature at 110 °C. The following relationship was used to measure the specific gravity of investigated soil:

\[
G_s = \frac{\alpha M_s}{(M_{bw} + M_s - M_{bws})}
\]

where \(\alpha\) is temperature correction coefficient and is the ratio of the density of water (or \(G_w\)) at the test temperature T and at 20 °C.

\[
\alpha = \frac{\rho_w}{\rho_{20^\circ C}}
\]

where, \(M_s\) = Mass of soil solids, \(M_{bw}\) = Mass of pycnometer + distilled water to the calibration mark on the pycnometer, \(M_{bws}\) = Mass of pycnometer + distilled water + soil.
3.2.4. Grain Size Distribution

The grain size distribution (GSD) was determined in accordance with ASTM D422-63 (2007). GSD was done in two phases. In the first phase particle sizes larger than 75 μm (retained on the No. 200 sieve) was determined by sieve analysis and then in the second phase the distribution of particle sizes smaller than 75 μm was determined by a sedimentation process, using a hydrometer.

3.2.4.1. Sieve Analysis

Around 500g of oven dry soil specimen was taken and distilled water was added to the sample to make it slurry. The slurry was allowed to passed through the Sieve No. 200 (opening size = 0.075 mm) and spray squirt bottle was used on the sieve to make the procedure easy. The soil retained and passing from the sieve was transferred to the evaporating dishes and kept in oven at the temperature of 110 °C. The specimens were taken out of oven after drying and weighed. The fines content (%) was calculated from retained soil.

3.2.4.2. Hydrometer Analysis

The finer soil, mainly the clay fraction (the percent finer than 0.002 mm) which cannot be analyzed by sieve, is usually done by hydrometer analysis. After sieve analysis, the soil retained on the pan was dried and around 100 g of soil was taken for the hydrometer analysis. This sample was mixed with 125 mL of 4% NaPO₃ solution in a small evaporating dish and the dist was covered by wet paper towel to minimize evaporation. The mixture was kept for 16 hours to soak. After soaking, the mixture was transferred to a dispersion cup and water was added until the cup was about two-thirds full. Then the mixture was transferred to the sedimentation cylinder and agitated carefully for about 1
minute to make the mixture uniform. Then the cylinder was set for the hydrometer test and first reading was taken at an elapsed time of 30 seconds. At the same time water temperature was recorded. At least 15 seconds before the reading taken, the hydrometer was placed on the cylinder so that it can be settled down. Hydrometer and temperature readings were continued at approximate elapsed times of 2, 4, 8, 16, 30 and 60 minutes and then 2, 4, 8, 24, 48 and 72 hours.

3.2.5. Consistency Limits

The liquid limit is the water content at which soil changes from the liquid state to a plastic state or the minimum moisture content at which a soil flows upon application of very small shear force. Liquid limit ($w_l$) is the water content, in percent, of a soil at the arbitrarily defined boundary between the semi-liquid and plastic states whereas the plastic limit ($w_p$) is the water content, in percent, of a soil at the boundary between the plastic and semi-solid states.

The liquid limit, plastic limit and plasticity index were determined according to ASTM D4318–10. The liquid limit and plastic limits are used for soil identification and classification and for strength correlation. The liquid limit was determined by performing trials in which a portion of the specimen was spread in a brass cup, divided in two by a grooving tool, and then allowed to flow together from the shocks caused by repeatedly dropping the cup in a standard mechanical device. Several trial, were conducted and water content corresponding to 25 blows was determined.

About 20 g portion of soil was taken from the material prepared for the liquid limit test to determine the plastic limit of soil. The water content of the soil was reduced to a consistency at which it can be rolled without sticking to the hands by spreading on
the glass plate. The mass is rolled between the palm or fingers and the ground-glass plate with just sufficient pressure to roll the mass into a thread of uniform diameter throughout its length. The thread was further deformed on each stroke so that its diameter reached 3.2 mm (1/8 in.). Three trials were done for plastic limit test and finally the average value was taken for plastic limit.

3.3. Compaction Curve

The standard proctor compaction tests were done on the investigated soil according to ASTM D1557-09. Total six numbers of samples have compacted to get a proper compaction curve. Remoulded air dried sample was used for each compaction test. All the lumps of the soil were broken and sieved through a 0.075 mm opening sieve and collect the entire passed sample and stored in the container. Soil sample has dried on oven. Enough water was added to the oven dried soil (passed through 0.075 mm opening sieve) and mixed thoroughly to bring the water content up desired quantity. Soil was covered with aluminum foil and plastic wrap and kept overnight for the proper mixing of water. Weight of the (4 in proctor mold + base plate) was measured. After attaching the mould top extension soil sample will be poured into the mould in three equal layers. Each layer will be compacted with standard proctor compaction effort by 25 times before the next layer of loose soil was poured into the mold. After compaction of each layer, small amount of soil was kept for the water content. After removing the top extension, excess soil above the mould was trimmed. Compacted soil was removed from the mold with a 4 inch round dolly tamper. Weight of the (proctor mold + base plate+ compacted moist soil) was measured. Compacted soil was removed from mould carefully. Water content ($w$) and bulk density ($\rho$) was determined. Dry density ($\rho_d$) was determined using equation
3.1. Water content of the whole sample was the average of the water content measured in each three layers.

\[ \rho_i = \frac{\rho}{1 + \frac{w}{100}} \]  \hspace{1cm} (3.1)

Compacted sample was preserved in proper way using plastic wrap and wax. Compacted soil was covered by wrap in 3 layers and then soil specimen has coated with wax. Then the samples had preserved in a container and kept in an undisturbed location.

3.4. Soil Water Characteristics Curve

The soil water characteristic curve (SWCC) was determined in accordance with ASTM D6836-02. To develop the SWCC, a sample was taken from wet side of the optimum water content as after optimum the sample became close to saturation. To minimize disturbance during specimen preparation, a masking tape was used to protect the exposed surface of the core sample. The protected core sample was sliced to about 11 mm thickness using a knife. The slices were trimmed using knife to fit in retaining rings with a diameter of 40 mm. Predetermined suction values starting from 5 kPa to as high as 7000 kPa were applied using specialized equipment. Pressure plate extractor was used to apply suction up to 500 kPa and pressure membrane extractor was used for rest of the values up to 700 kPa. Beyond this limit, to determine suction chilled mirror hygrometer was used. Sub-samples retrieved from the compacted sample then samples were kept in bathtub with respective porous plate for saturation for seven days. After saturation the top surface was trimmed before putting into the extractor. These sub-samples were used to determine drying SWCC. Before starting the extractor the porous plate and the cellulose membranes were submerged into the distilled and de-aired for saturation. It was kept until
no gas bubbles appeared; however, usually it got fully saturated within 24 hours. Specimens along with the retaining rings were placed on the plates and membranes. Excess water was removed using a vacuum pump and each porous plate or membrane was placed in the designated pressure plate extractors. When the test preparation was completed, the pressure extractors were closed with tight sealing. Especially for higher range suction, the fitting of the O-rings between the vessel and the lid were thoroughly inspected before applying air pressure. For each suction value, the expelled water from the samples was periodically monitored in a graduated burette. When two consecutive readings over a 24 hour period were found to be close, the application of suction was terminated and the water content of the samples was immediately determined.

3.5. Shrinkage Test

The shrinkage test was conducted in accordance with ASTM D4943-08. To obtain the void ratio, the volume of soil specimens was determined using the water displacement method. Each specimen was coated with molten microcrystalline wax ($G_s = 0.91$) and allowed to cool down at room temperature. A 250 ml graduated cylinder was used for the determination of shrinkage. Cylinder was completely filled with water and the weight of $C_1$ (cylinder + water) was taken. After wax solidification, the sample was submerged in the cylinder that was filled with distilled water. After getting stable, the sample was removed and again the weight of $C_2$ (cylinder + water). The weight of (wax + soil) was also measured. The difference between $C_1$ and $C_2$ was the volume of the wax coated soil sample. This quantity was readily converted to water volume representing the volume of wax-coated soil. The volume of soil was obtained from the difference of volume of the wax-coated sample and the volume of wax (mass/0.91). A 7.4% correction was applied to
CHAPTER 3

account for the underestimation due to air entrapment at soil-wax interface in this method, as suggested by Prakash et al. (2008). The mass of the sample was also determined to estimate the bulk unit weight of the soil. Using basic phase relationships, the void ratio was determined from knowledge of the bulk unit weight of the soil.

3.6. Direct Shear Test

In this study, 9 sets of direct shear test had conducted where each set contained three individual tests with different initial condition. Six samples were obtained from compaction curve. Unsaturated shear strength tests were conducted on these compacted samples along with one natural sample. Compacted sample with maximum dry density and natural sample was saturated prior to direct shear test. 2560A pneumatic direct/residual shear apparatus was used for the tests. There were four transducer in the direct shear test equipment that included two LVDT (horizontal displacement transducer and vertical displacement transducer), load cell and pressure transducer. Humboldt integrated real time data acquisition system connected all the four transducers. Finally, test data collected and stored in the computer through data collection software and data acquisition system. Before commencement of the tests, all the transducers had calibrated.

Consolidated drained (CD) test was conducted on the both saturated and compacted samples. 4 in diameter circular direct shear ring was used in the shear box assembly. Preserved sample was carefully cut and trimmed to fit into the shear ring. Porous stone was placed at the top and bottom of the sample porous stones were kept 24
Figure 3.4: Laboratory direct shear test procedure
hours for saturation as dry porous stone could make an effect on suction. The sample was placed from the ring to the shear box using dolly tamper. The shear box assembly with sample was placed into the direct shear box. Horizontal and vertical transducers were adjusted with the shear box. The shear box was filled with water and kept the sample for saturation. The process of saturation took more than a week. Normal stress was applied on the sample and left the sample for consolidation for more than 24 hours. Three normal stress (75 kPa, 150 kPa and 250 kPa) was selected for the tests. The strain rate was selected to be $5 \times 10^{-3}$ mm/min. As drained test was conducted, the strain rate was selected slow enough so that the pore water pressure cannot develop. The strain rate was determined in accordance with ASTM D3080-98 and consolidation test result. Horizontal strain, vertical strain and shear stress data was collected and used for subsequent analysis.

For the unsaturated soils, sample was placed in the shear box with the same procedure. Normal stress and the strain rate were kept same as above. Normal stress was applied and kept for 24 hours for settlement, and then shear test started. The shear test continued until the shear stress reached the peak value and start dropping. Then test was stopped and sample was removed from the shear box. Along the failure plane several samples were taken and used for suction measurement.

### 3.7. Suction Measurement

The dew point potentiometer (WP4-T) was used for the suction measurement for the shear failed samples to determine the suction at failure point. The sampling cup was half filled with the investigated soil to ensure accurate suction measurements (Leong et al., 2003) by using about 5 mg of material with a known water amount. The unsaturated sample was placed in sealed measurement chamber, set at 25°C temperature, through a
sample drawer and was allowed to equilibrate with the surrounding air. Equilibration was usually achieved in 10 min to 20 min, as detected by condensation on a mirror and measured by a photoelectric cell. From the understanding of the universal gas constant, \( R \) (8.3145 \( \text{J/mol} \cdot \text{K} \)), sample temperature, \( T \) (\( ^\circ \text{K} \)), water molecular mass, \( X \) (18.01 kg/kmol), and the chamber relative humidity, \( p/p_o \), soil suction was calculated (\( \psi = RT/X \ln(p/p_o) \)) and displayed on the potentiometer screen.

### 3.8. Prediction of Unsaturated Shear Strength

Vanapalli et al., (1996) and Fredlund et al., (1996) have proposed a more general, nonlinear function for predicting the shear strength of an unsaturated soil using the entire soil water characteristic curve and the saturated shear strength properties as shown below:

\[
\tau = [c' + (u_a - u_w) \tan \phi'] + [(u_a - u_w)(\Theta^k)(\tan \phi')] \tag{3.1}
\]

where, \( \kappa = \) fitting parameter used for obtaining a best fit between the measured and predicted values and \( \Theta = \) normalized water content, \( \theta_w / \theta_s \).

Normalized water content, \( \Theta \) was determined from the soil water characteristics curve, for the compacted soil \( \text{SWCC} \) was determined in this research. From the consistency limit test, \( I_p \) was obtained and thus using the Figure 3.5 the value “\( \kappa \)” was determined. Saturated shear strength properties was determined from the single stage direct shear test on saturated specimens.
Figure 3.5: Fitting parameter versus plasticity index

Figure 3.6: Schematics of parametric study procedure
3.9. Parametric Study

Figure 3.6 schematically shows the flow chart of the parametric study program. The shear strength properties had obtained from laboratory direct shear test and material properties from the standard proctor test, then modeling and analyzing has done with the SLOPE/W of the GeoStudio 2004 version 6.02 (GeoStudio international ltd.)

3.9.1. Slope Modeling

The slope was modeled with a height of 15m and three different slope angles (45°, 60° and 90°). The model was divided into two region where upper one was the slope region and the lower one considered as the foundation layer.

3.9.2. Material Properties

While defining the material property the strength model has to define and the strength model was defined as Mohr-Coulumb. Material property was given same for both the region. The unit weight of the soil, cohesion and friction angle was defined as basic property and angle of friction due to suction along with soil unit weight was defined for advance property for unsaturated soil.

3.9.3. Method of Analysis

The model has solved using Morgenstern-price method with half-sign function. Before running the model, the critical slip surface was determined. A model was run for slope stability with a wide range of grid and radius to find the critical slip surface with minimum factor of safety. This FEM based software has analyzed 11 potential circular slip surface for 676 intersection points of the rotation grid resulted in 7436 trial slip surfaces had evaluated. The centre and radius of the slip surface corresponding to the minimum factor of safety has used for all other materials (with different $\rho_d$ & $w$)
$\gamma = 19 \text{ kN/m}^3$

$c' = 54 \text{ kPa}$

$\phi = 27.9^\circ$

$\phi^b = 20.6^\circ$

Figure 3.7: Determination of critical slip surface
4 Results and Discussion

4.1. Geotechnical Index Properties

Table 4.1 gives the geotechnical index properties of the investigated soil. The specific gravity of the soil was found to be 2.74 which is typical for sedimentary clays, that is, between 2.4 and 2.95 (Terzaghi et al., 1996). Materials finer than 0.075 mm and finer than 0.002 mm were respectively found to be 98% and 60%, thereby, confirming the fine grained nature of the soil. The liquid limit \( (w_l = 77\%) \) and plastic limit \( (w_p = 27\%) \) indicated a soil possessing significant water adsorption water and retention capacity. Overall, the soil was classified CH (clay of high plasticity) using the Unified Soil Classification System.

4.2. Compaction Curve

Figure 4.1 shows the compaction curve for investigated soil. The curve consisted of the six proctor points that are corresponding with the numbering in the figure. The maximum dry density was found to be 1.59 g/cm\(^3\) at the optimum water content of 23.9% which is close to plastic limit (27%). This closely matches with the model \( (OWC = 0.92 PL) \) used by Ridharan and Nagaraj (2005) for cohesive soils. The increase in \( \gamma_d \) with an increase in water content on dry side of optimum is due to expulsion of air from the pore space and re-arrangement of particles that decreases the pore space. Conversely, an increase in water content on wet side of optimum results in an increased volume of water \( (\gamma_w = 1 g/cm^3) \) which replaces the soil particles \( (\gamma_s = 2.74 g/cm^3) \).
Table 4.1: Geotechnical index properties of investigated soil

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity, $G_s$</td>
<td>2.74</td>
</tr>
<tr>
<td>-0.075 mm, (%)</td>
<td>98</td>
</tr>
<tr>
<td>-0.002 mm, (%)</td>
<td>60</td>
</tr>
<tr>
<td>Liquid limit, $w_l$ (%)</td>
<td>77</td>
</tr>
<tr>
<td>Plastic limit, $w_p$ (%)</td>
<td>27</td>
</tr>
<tr>
<td>Plasticity index, $I_p$ (%)</td>
<td>50</td>
</tr>
<tr>
<td>USCS Symbol</td>
<td>CH</td>
</tr>
</tbody>
</table>
Figure 4.1: Compaction curve for the investigated soil
4.3. Soil Water Characteristics Curve

The SWCC for investigated soil was determined on the wet of optimum ($w = 30.2\%$ and $\rho_d = 1.55 \text{ g/cm}^3$) close to the saturated condition thereby precluding the effect of hysteresis during pre-wetting. Figure 4.4 gives the SWCC of the investigated expansive soil. The SWCC of the investigated soil was determined as unsaturated strength properties of the soil can easily determined by using saturated shear strength properties and the SWCC in the empirical equation. The test data fitted well to a unimodal distribution with an air entry values of 10 kPa (for gravimetric water content ($w$) and volumetric water content ($\theta$)). But the degree of saturation ($S$) plot showed the bimodal distribution with two air entry value; a lower value of 10 kPa corresponding to drainage through inter-aggregate pores followed by a higher value of 7000 kPa associated with seepage through the soil matrix. The residual suction was found to be 8000 kPa (at $w = 5\%$) based on $w$ and $\theta$. Finally the abscissa joined at $10^6$ kPa under completely dry condition. The SWCC is in accordance with the soil fabric model of Gens and Alonso (1992) where expansive soils are composed of aggregates with intra-aggregate pores and inter-aggregate pores. The dual porosity structure of the compacted soil formed the bimodal distribution of the SWCC. The volumetric water content versus suction curve is useful for the determination of storage capacity of the soil using the part of the curve between the matrix air entry value and residual suction (Fredlund et al., 2012) and also important to determine normalized volumetric water content which has a relation between unsaturated shear strength.
Figure 4.2: Soil water characteristics curve for investigated soil
4.4. Swell-shrink Curve

Figure 4.3 shows the results of the swell-shrink test in the form of void ratio versus water content. Theoretical lines representing various degrees of saturation values were obtained from basic phase relationship and using $G_s = 2.74$. The same compacted sample as used in SWCC measurement was used for swell-shrink test. This test has conducted because of the investigated soil undergoes high swelling and shrinkage, so the SWCC should be read in conjunction with shrinkage curve. Several sub-samples were retrieved and saturated.

Theoretically, the swell-shrink curve comprises of two straight lines: a sloped line closely following the $S = 100\%$ line that joins a horizontal line at a void ratio associated with the shrinkage limit of the soil. This J-shaped curve indicates that soils remain saturated up to the shrinkage limit. The investigated expansive soil exhibited an S-shaped curve with three distinct stages of shrinkage: an initial low structural shrinkage ($100\% \leq S \geq 80\%$) followed by a sharp decline during normal shrinkage ($80\% \leq S \geq 75\%$) and then by a low decrease during residual shrinkage ($S \leq 75\%$) up to complete desiccation of the soil. During structural shrinkage, water within the inter aggregate pores and some of the larger and relatively stable voids is removed such that the decrease in soil volume is less than the volume of water lost. During normal shrinkage, volume of water lost is equal to the volume decrease in soil which makes a parallel line to the $100\%$ saturation line. This suggests that drainage primarily takes place through the soil matrix in the normal shrinkage zone. During residual shrinkage, air enters the pores close to the shrinkage limit and pulls the particles together due to suction. This leads to a further decrease in soil volume albeit lower than the volume of water lost. Finally, the compaction curve was observed to be within the normal shrinkage zone.
Figure 4.3: Swell-shrink curve for investigated soil
4.5. Stress-strain Relationship

Figure 4.4 shows the stress-strain relationship of the investigated soil. All six samples were obtained from the compaction test. The strain at failure was found to be within 1.5 mm to 3.05 mm. On the dry side, the shear stress increases quickly with respect to the strain up to the maximum value, beyond which, it decreases at a faster rate. Flocculated structure was formed where soil particles are oriented randomly, with increasing stress it suddenly collapses. On the wet side, the stress gradually increases with respect to the strain to the maximum value, after which it gradually drops. Soil behaves differently as it travels more horizontal strain before failure. Soil makes dispersed structure where particles are more oriented in a parallel arrangement perpendicular to the direction of applied stress. Soils on dry side of optimum have brittle stress-strain behaviour whereas the wet side of optimum shows ductile behaviour (Fang, 1990).

As expansive soils mobilize both vertically and laterally, there is a good correlation between stress-strain and lateral earth pressure due to lateral swelling. Since soil layer is laterally confined, the lateral earth pressure is expected to be higher than the vertical overburden during swelling (Hong, 2008). Brackley and Sanders (1992) reported that the lateral pressure in surficial expansive clays (2 m deep) is up to four times the vertical pressure. However, shear strength contribution due to normal stress, reduced by the opposite shear stress due to lateral pressure. Thus the soil can fail at a lower strain. Such passive conditions imply localized shearing failure specially during alternate volume changes due to seasonal weather variations. The local shear failures cause damage to the lightly loaded structures.
Figure 4.4: Stress-strain relationship for the investigated soil
4.6. Shear Strength Properties

Table 4.3 summarizes the saturated and unsaturated shear strength properties of the investigated soil. The cohesion and friction angle is high for compacted soil compared to natural soil in saturated condition, which indicated the natural shear strength of the soil is lower than the compacted soil. At each given dry density, saturating the soil samples caused a clear reduction of the soil strength. The strength loss varied considerably depending on the compaction water content. The loss of the soil strength due to saturation was around 300% or more for samples compacted at water content less than optimum. The least strength loss was observed with samples compacted at water content greater than optimum (Rowshanzamir and Askari, 2010). When an unsaturated soil specimen is allowed to drain with respect to air phase under an applied stress, the resulting decrease in the void ratio can be expected to contribute to an increase in the shear strength.

Figure 4.3 gives the relationship between shear strength properties (cohesion, friction angle and friction angle due to suction) and water content. Cohesion increased with the increase of water content to a maximum value at around optimum water content and then decreased with further increase in water content which is similar to the shape of the compaction curve. Soil suction is the non-linear function of water content. So, suction and water content relation was also found to be similar to the shape of compaction curve. Cohesion was found lower at the drier side of optimum due to the presence of clay aggregate which made the soil mass more granular. Then cohesion value increases with the water content and reached maximum value at around optimum due to reduction of the size of clay aggregate. This is similar to the work done by Cokca et al. (2004). They mentioned that the cohesion at the drier side of optimum will be lesser than
Table 4.2: Shear strength properties of investigated soil

<table>
<thead>
<tr>
<th></th>
<th>w (%)</th>
<th>( \rho_d ) (g/cm(^3))</th>
<th>( c' ) (kPa)</th>
<th>( \phi' )</th>
<th>( \phi_b )</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Saturated shear strength properties</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compacted</td>
<td>23.9</td>
<td>1.59</td>
<td>27.9</td>
<td>32.3</td>
<td>-</td>
</tr>
<tr>
<td>Natural</td>
<td>33</td>
<td>1.36</td>
<td>16.3</td>
<td>23.4</td>
<td>-</td>
</tr>
<tr>
<td><strong>Unsaturated shear strength properties</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compacted (1)</td>
<td>18</td>
<td>1.46</td>
<td>23.7</td>
<td>43.5</td>
<td>6.0</td>
</tr>
<tr>
<td>Compacted (2)</td>
<td>21.5</td>
<td>1.54</td>
<td>48.3</td>
<td>34.6</td>
<td>8.3</td>
</tr>
<tr>
<td>Compacted (3)</td>
<td>22.9</td>
<td>1.57</td>
<td>56.7</td>
<td>32.3</td>
<td>13.0</td>
</tr>
<tr>
<td>Compacted (4)</td>
<td>23.9</td>
<td>1.59</td>
<td>65.7</td>
<td>29.2</td>
<td>14.3</td>
</tr>
<tr>
<td>Compacted (5)</td>
<td>28.3</td>
<td>1.52</td>
<td>54</td>
<td>27.9</td>
<td>19.8</td>
</tr>
<tr>
<td>Compacted (6)</td>
<td>30.2</td>
<td>1.48</td>
<td>32.7</td>
<td>27.2</td>
<td>20.6</td>
</tr>
<tr>
<td>Natural</td>
<td>33</td>
<td>1.36</td>
<td>39.3</td>
<td>27.9</td>
<td>21.8</td>
</tr>
</tbody>
</table>
Figure 4.5: Shear strength properties versus water content and suction relationship
that at optimum water content due to the ‘clay aggregation’ phenomenon where the soil mass exhibits a granular texture. Increase in water content above optimum reduced cohesion as excess water might developed thicker water film around the clay particle and thus increased the distance between particles. Seed et al. (1961) observed that cohesion on the wetter side of optimum is lesser than that at optimum water content due to the formation of thicker water films’ around clay particles in the ‘clay-water system’. Mitchell (1993) reported that the forces which generate cohesion become significant for separation distance less than 2.5 nm.

The angle of friction decreases rapidly with increasing moisture content and decreasing suction up to optimum moisture content (Figure 5). The dry side of optimum gives high values of friction angle which are not within the range expected for saturated clays of high plasticity. These high friction angles seem to be due to the clay aggregates formed on the dry side of optimum (Cokca et al., 2004). The granular structure and large size of aggregates within soil mass increased the interlocking between the clay particles and soil suction generated a resistance to slippage at the contacts between the particles (or aggregates) as the moisture content decreases on the dry of optimum. The reduction was due to the increased lubrication of the soil paste following water addition causing soil particles to slip and slide, resulting in a reduced friction angle. For higher water content, the soil particles dominated the behaviour of the soil mixture and the water acts as a lubricant, which decreases the friction angle as the water content increases. The lubrication occurs when the surface of the soil particle is wetted, causing the mobility of the absorbed film to increase due to increased thickness and greater surface ion hydration and dissociation (Mitchell, 1993). The role of clay aggregates on frictional behavior of
the clay is substantially reduced at about optimum moisture content. The decrease in angle of friction with increasing moisture content (at wet side of optimum), is attributed to decreasing suction values (close to saturated condition).

Friction angle due to suction increased with the increase of moulding water content and increased with the decrease of matric suction. At zero suction (saturated condition), $\phi_b$ may be assumed equal to the saturated friction angle (Gan and Fredlund, 1996). Then it decreases with the increase of matric suction. So, it appears that the angle is a function of matric suction. The maximum value of the $\phi_b$ was found to be equal to $20.6^\circ$ at a water content of 30.2% for compacted soil and the $\phi$ (saturated) was found to be equal to $32.3^\circ$. So, if the samples get saturated then both the values should be equal.

4.7. Prediction of Unsaturated Shear Strength

Unsaturated shear strength property means the cohesion component related to matric suction which is friction angle due to suction. Both the direct test method and empirical method has used to determine suction friction angle. One set of friction angle due to suction ($\phi_b$) was determined in the laboratory using axis translation technique and other set of Angle of friction due to suction ($\phi'$) was determined using soil water characteristics curve and fitting parameter. Figure 4.6 shows the relationship between friction angle due to suction ($\phi_b$) versus water content. From the investigation, measured value was found to be relatively higher than estimated value. Using all those point, a best fit linear equation has drawn which shows the variation of Angle of friction due to suction ($\phi'$) with the increase of water content. For the particular investigated zone of the investigated soil showed the liner behaviour of friction angle due to suction with respect to water content at low suction (890 kPa to 3900 kPa). At higher water content, friction angle due to
Figure 4.6: Relation between $\phi^b$ versus water content

$\phi^b = 1.26 \times w - 17.63$

$R^2 = 0.953$
suction ($\phi_b$) is close to saturated friction angle ($\phi'$), then it decreases with the decrease of water content. At zero suction (completely saturated soil), Angle of friction due to suction ($\phi_b$) may be assumed equal to the saturated friction angle (Gan and Fredlund, 1996).

4.8. Physical model of compacted expansive soil

Figure 4.7 gives a physical model to describe the physical and mechanical behaviour of compacted expansive soil. The effect of compaction on soil microstructure can be explained by the physical model. The model was developed with the help of previous work done by Lambe (1958), Holtz & Kovacs (1981), Gens and Alonso (1992), Frydman and Baker (2009) and Leroueil and Height (2013). Investigations and observations from this study also included in this model.

Microstructure of compacted fine grained soil can be classified by two main soil particle assemblages that consisted of flocculated and dispersed arrangements. Clayey soils compacted on the dry side of optimum water content have a flocculated structure with relatively large pore and clayey soils compacted on the wet side of optimum water content have a dispersed structure with smaller pores. The flocculated structure includes a random arrangement of particles with edge to face contacts, whereas the dispersed structure includes a better oriented arrangement with layered parallel soil particles. At the dry side of optimum, rigid microstructure formed due to particle aggregation. The size of the aggregates reduced with the increased water content.

The air phase is continuous on the dry side of optimum whereas air is occluded (the air is in the form of bubbles and so discontinuous) on the wet side of optimum. The interconnectivity of the pores decreased with increasing water content and the pores
<table>
<thead>
<tr>
<th>Soil structure</th>
<th>Phase diagrams and relationships</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Dry of optimum</strong></td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>aggregation</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>silt grains</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>continuous air phase</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>clay minerals</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>inter-aggregate pores</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>discontinuous air phase</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>clay minerals</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>water menisci</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>capillary water</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td><img src="image" alt="Diagram" /></td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>$V_{v1} = V_{v1} + V_{w1}$</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>$V_{v1} = V_{a1} + V_{w1}$</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>$M_{w1}$</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>$M_{s1}$</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>Continuous air phase</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>High air conductivity</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>Occluded water phase</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>High hydraulic conductivity</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td><strong>Wet of optimum</strong></td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>aggregation</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>silt grains</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>clay minerals</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>inter-aggregate pores</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>external adsorbed water</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>clay minerals</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>occluded air bubble</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>intra-aggregate adsorbed water</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td><img src="image" alt="Diagram" /></td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>$M_{w2}$</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>$M_{s2}$</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>Occluded air phase</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>Low air conductivity</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>Continuous water phase</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>Low hydraulic conductivity</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>Low swell-shrink</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>Low strain-stress modulus</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>Low shear strength</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>Low suction at failure</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>$M_{w2} &gt; M_{w1}$</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>$V_{a2} &lt; V_{a1}$</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>$V_{w2} &gt; V_{w1}$</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>$V_{v2} &gt; V_{v1}$</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
<tr>
<td>$w_2 &gt; w_1$</td>
<td><img src="image" alt="Diagram" /></td>
</tr>
</tbody>
</table>

Figure 4.7: Physical model describing behaviour of compacted expansive soils
became occluded. The amount and the size of the pores decreased with increasing water content up to the optimum and then increased with further increases in the water content at the wet side of optimum water content. However, the pores generally remained disconnected at high water contents, even though the pore sizes increased at these moisture levels.

The compacted soil is subjected to relatively high suctions on the dry side of optimum whereas on the wet side, suction is lower and consequently, the soil more deformable. As a result, the soil compacted on the dry side has an aggregated fabric with intra-aggregate pores (or micropores) and larger inter-aggregate pores whereas the soil compacted on the wet side is much more homogeneous with mostly micropores. A dual porosity structure (represented by inter-aggregate pores and intra-aggregate pores) formed during the compaction process. The change of the interaggregate pores is dominant during soil compaction and the change of the intra-aggregate pores is dominant during soil saturation and drying. The compaction compression stress can cause significant reduction in the volume of the interaggregate pores, but only slightly affects the intra-aggregate pores during compaction. The dual porosity structure evolutes during the saturation process, the soil aggregates might swell and result in larger and more intra-aggregate pores but smaller and fewer inter-aggregate pores. That indicated the presence of double porosity on the dry side as well as on the wet side of optimum.

Swelling of compacted soils is high on the dry side of optimum. They have a relatively higher deficiency of water and therefore have greater tendency to absorb water. Shrinkage is just opposite phenomena of swelling. Soil compacted on wet side of optimum has greater tendency to shrink.
4.9. Parametric Study

The saturated and unsaturated shear strength properties determined in this research has been used for the parametric study. Table 4.3 summarizes the analyzed results from parametric study. Unsaturated and saturated compacted soils followed by natural soil slopes have been analyzed for three different slope angles. In both cases saturation condition is most unstable. Dry side and wet side of optimum, though both of them showed almost same range of factor of safety, dry side of optimum possess higher safety margin. Saturating the soil reduced the factor of safety upto 24% for natural soils and 67% for compacted soils. The maximum and minimum factors of safety were found to be 3.6 (for the soil compacted at optimum) and 1.07 (for natural soil) respectively at an angle of 45º. It also clear that compaction increased the factor of safety hence enhances stability of structures.
Table 4.3: Factor of safety for the investigated slope with different material properties

<table>
<thead>
<tr>
<th></th>
<th>w (%)</th>
<th>$\rho_d$ (g/cm$^3$)</th>
<th>c (kPa)</th>
<th>$\phi$ ($^\circ$)</th>
<th>$\phi_b$ ($^\circ$)</th>
<th>Slope Angle ($^\circ$)</th>
<th>Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Dry of Optimum</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18 - 24</td>
<td>1.46 - 1.59</td>
<td>23.7 - 64.7</td>
<td>29.2 - 43.5</td>
<td>6 - 14.3</td>
<td>45</td>
<td></td>
<td>1.3 - 3.6</td>
</tr>
<tr>
<td>45</td>
<td>1.59</td>
<td>64.7</td>
<td>29.2</td>
<td>14.3</td>
<td>45</td>
<td></td>
<td>3.6</td>
</tr>
<tr>
<td>60</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>90</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Optimum</strong></td>
<td>45</td>
<td>1.59</td>
<td>64.7</td>
<td>29.2</td>
<td>14.3</td>
<td>45</td>
<td>3.6</td>
</tr>
<tr>
<td>60</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>90</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Wet of Optimum</strong></td>
<td>24</td>
<td>1.48 - 1.59</td>
<td>32.7 - 64.7</td>
<td>27.2 - 29.2</td>
<td>14.3 - 20.6</td>
<td>45</td>
<td>1.4 - 3.6</td>
</tr>
<tr>
<td>24 - 30.2</td>
<td>1.48 - 1.59</td>
<td>32.7 - 64.7</td>
<td>27.2 - 29.2</td>
<td>14.3 - 20.6</td>
<td>45</td>
<td></td>
<td>1.4 - 3.6</td>
</tr>
<tr>
<td>45</td>
<td>1.59</td>
<td>64.7</td>
<td>29.2</td>
<td>14.3</td>
<td>45</td>
<td></td>
<td>3.6</td>
</tr>
<tr>
<td>60</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>90</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Compacted (Saturated)</strong></td>
<td>24</td>
<td>1.59</td>
<td>27.86</td>
<td>32.25</td>
<td>45</td>
<td></td>
<td>1.4</td>
</tr>
<tr>
<td>24</td>
<td>1.59</td>
<td>27.86</td>
<td>32.25</td>
<td></td>
<td>45</td>
<td></td>
<td>1.4</td>
</tr>
<tr>
<td>60</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>90</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Natural (Unsaturated)</strong></td>
<td>33</td>
<td>1.32</td>
<td>39.3</td>
<td>27.9</td>
<td>21.8</td>
<td>45</td>
<td>1.5</td>
</tr>
<tr>
<td>33</td>
<td>1.32</td>
<td>39.3</td>
<td>27.9</td>
<td>21.8</td>
<td>45</td>
<td></td>
<td>1.5</td>
</tr>
<tr>
<td>60</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>90</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Natural (Saturated)</strong></td>
<td>44</td>
<td>1.32</td>
<td>16.3</td>
<td>23.4</td>
<td>45</td>
<td></td>
<td>1.1</td>
</tr>
<tr>
<td>44</td>
<td>1.32</td>
<td>16.3</td>
<td>23.4</td>
<td></td>
<td>45</td>
<td></td>
<td>1.1</td>
</tr>
<tr>
<td>60</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>90</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.6</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
CHAPTER 5

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

5.1. SUMMARY

The purpose of this research was to determine the unsaturated shear strength properties and establish a clear understanding of the shear strength behaviour of compacted expansive soils. Unsaturated strength properties of compacted expansive soils have significant effects on the slope stability of natural and artificial structures. The main contributions of this research are summarized as follows:

1. A comprehensive research methodology was developed to determine and to predict unsaturated shear strength properties of expansive soils.
2. A conceptual model was developed to understand the behaviour of compacted expansive soils.

5.2. CONCLUSION

- The investigated expansive soil \( (G_s = 2.74 \text{ and material finer than } 0.002 \text{ mm } = 60\%) \) indicated significant water adsorption and retention capacity from its liquid limit (77%) and plastic limit (27%). The maximum dry density was found to be 1.59 g/cm\(^3\) at the optimum water content of 24% which is close to the plastic limit (27%).

- The strain at failure was found to be within 1.5 mm to 3.0 mm. Dry side of optimum showed brittle stress-strain behaviour whereas, wet side of optimum showed ductile behaviour.

- The cohesion \( (c') \) followed the compaction curve with a maximum value of 66 kPa at the optimum water content. Likewise, the friction angle \( (\phi') \) decreased
from 44° on the dry side reaching a minimum value of 27° at the optimum water content beyond which it was constant. The friction angle due to suction (\(\phi_b\)) is equal to the friction angle (\(\phi\)) at saturated condition, then decreased with decreasing of water content.

- The SWCC exhibited a single air entry value of 10 kPa (for gravimetric water content (\(w\)) and volumetric water content (\(\theta\))) and exhibited two air entry value; a lower value of 10 kPa corresponding to drainage through inter-aggregate pores followed by a higher value of 7000 kPa associated with seepage through the soil matrix (for degree of saturation(S)).

- The shrinkage path during progressive drying of the investigated expansive soil exhibited an S-shaped curve with three distinct portions: an initial low structural shrinkage (\(S = 100\%\) to \(S = 80\%\)) followed by a sharp decline during normal shrinkage (\(S = 80\%\) to \(S = 75\%\)) and then by a low decrease during residual shrinkage (\(S = 75\%\) to \(S = 0\)). The compaction curve was observed to be within the normal shrinkage zone.

- The estimated values of friction angle due to suction (\(\phi_b\)) corroborated well with the laboratory determined values, which validated the results. The friction angle due to suction (\(\phi_b\)) with water content relationship was found to be linear in the investigated zone.

- The maximum and minimum factor of safety were found to be 3.6 (for the soil compacted at optimum) and 1.07 (for natural soil) respectively at an angle of 45°. Compaction increases the safety margin and soil at dry of optimum has higher factor of safety than wet of optimum. The factor of safety was found to be more
than 1 for the compacted soil at optimum water content regardless of the slope angles. A 6% change in water content on both sides of the optimum water content rendered the 90° slopes unstable. Likewise, under saturated conditions, both natural and compacted slopes failed. Saturating the soil reduced the factor of safety upto 24% for natural soils and 67% for compacted soils. However, wet of optimum is better for construction as strength reduces less with the increase of water content.

5.3. RECOMMENDATIONS

1. Suction controlled direct shear test is highly recommended for accurate measurement of friction angle due to suction.

2. SWCC and swell-shrink curve showed the presence of fissure or cracks, so the effect of fissure on shear strength of expansive soils should be analyzed.

3. Slope stability analysis should be done with residual shear strength for long term stability
REFERENCES


REFERENCES


REFERENCES


REFERENCES


REFERENCES

of the 55th Canadian Geotechnical Conference, Niagara, Canada.


APPENDIX

The followings are the list of test results included in the appendix:

- Determination of specific gravity
- Consistency limits test
- Hydrometer analysis
- Standard proctor test
- Direct shear test
- Soil water characteristics curve
- Swell-shrink curve
APPENDIX

Specific Gravity

Table 1: Results from specific gravity determination for the investigated soil

<table>
<thead>
<tr>
<th>Test No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vol. of flask (ml)</td>
<td>500</td>
<td>500</td>
<td>500</td>
</tr>
<tr>
<td>Method of air removal</td>
<td>Vacuum</td>
<td>Vacuum</td>
<td>Vacuum</td>
</tr>
<tr>
<td>Wt of flask</td>
<td>195.56</td>
<td>183.34</td>
<td>195.56</td>
</tr>
<tr>
<td>Wt of flask + water + soil =</td>
<td>749.45</td>
<td>734.98</td>
<td>762.35</td>
</tr>
<tr>
<td>Temperature (C)</td>
<td>21</td>
<td>21</td>
<td>21</td>
</tr>
<tr>
<td>Wt of flask + water =</td>
<td>687.23</td>
<td>672.15</td>
<td>698.86</td>
</tr>
<tr>
<td>Wt of dish + dry soil</td>
<td>294.81</td>
<td>275.23</td>
<td>190.76</td>
</tr>
<tr>
<td>Wt of dish</td>
<td>196.2</td>
<td>176.5</td>
<td>191.2</td>
</tr>
<tr>
<td>Wt of dry soil =</td>
<td>98.61</td>
<td>98.73</td>
<td>99.56</td>
</tr>
<tr>
<td>Mw = Ms + Mbw - Mbws</td>
<td>36.39</td>
<td>35.9</td>
<td>36.07</td>
</tr>
<tr>
<td>a = pw/p20</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gs= aMs/Mw</td>
<td>2.71</td>
<td>2.75</td>
<td>2.76</td>
</tr>
<tr>
<td>Average Gs</td>
<td></td>
<td></td>
<td>2.74</td>
</tr>
</tbody>
</table>

Liquid limit

The following data is from the liquid limit tests conducted on three different samples.

The following equation was used to compute the liquid limits for three samples:

\[ LL = w_N(\%) \left(\frac{N}{25}\right)^{0.121} \]

where:

\[ LL = \text{liquid limit}, \]

\[ w_N(\%) = \text{moisture content, in percent, for 12.7 mm groove closure in the liquid limit device at } N \text{ (between 20 and 30) number of bows.} \]
Table 2: Results from liquid limit determination for the investigated soil

<table>
<thead>
<tr>
<th>Test No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass of Can, $W_1$ (g)</td>
<td>32.26</td>
<td>26.73</td>
<td>32.29</td>
</tr>
<tr>
<td>Mass of Can + Moist Soil, $W_2$ (g)</td>
<td>49.55</td>
<td>53.65</td>
<td>59.44</td>
</tr>
<tr>
<td>Mass of Can + Dry Soil, $W_3$ (g)</td>
<td>41.8</td>
<td>41.86</td>
<td>47.75</td>
</tr>
<tr>
<td>Water Content, w (%) = $\frac{[W_2-W_3]/(W_3-W_1)}{100}$</td>
<td>81.24</td>
<td>77.92</td>
<td>75.61</td>
</tr>
<tr>
<td>Number of Blows, N</td>
<td>16</td>
<td>23</td>
<td>29</td>
</tr>
<tr>
<td>Liquid Limit</td>
<td></td>
<td></td>
<td>77</td>
</tr>
</tbody>
</table>

**Plastic Limit**

The results from plastic limit tests are given in the tables below. The plastic limit was determined by taking an average value from three trials.

Table 3: Results from plastic limit determination for the investigated soil

<table>
<thead>
<tr>
<th>Test No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass of Can, $W_1$ (g)</td>
<td>32.26</td>
<td>26.73</td>
<td>32.29</td>
</tr>
<tr>
<td>Mass of Can + Moist Soil, $W_2$ (g)</td>
<td>45.56</td>
<td>39.76</td>
<td>42.63</td>
</tr>
<tr>
<td>Mass of Can + Dry Soil, $W_3$ (g)</td>
<td>42.77</td>
<td>36.94</td>
<td>40.44</td>
</tr>
<tr>
<td>Water Content, w (%) = $\frac{[W_2-W_3]/(W_3-W_1)}{100}$</td>
<td>26.5</td>
<td>27.6</td>
<td>26.9</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td></td>
<td></td>
<td>27</td>
</tr>
</tbody>
</table>

**Plasticity Index**

The plasticity index of the Regina clay was calculated as follows:

$\text{PI} = \text{LL} - \text{PL}$

$77 - 27 = 50$
Hydrometer Analysis

The followings are the results from the hydrometer analysis conducted on three samples. Hydrometer type 152H was used. For dispersive agent, 125 ml of sodium hexametaphosphate (40g/L) was used.

Table 4: Results from the hydrometer analysis for the investigated soil

<table>
<thead>
<tr>
<th>Date</th>
<th>Elapsed Time (min)</th>
<th>Hydrometer</th>
<th>Temp (°C)</th>
<th>Corr. Hyd. Reading</th>
<th>Hyd. Corr. only</th>
<th>Eff. Depth</th>
<th>Adjusted Dia. (mm)</th>
<th>% Finer</th>
<th>% Finer of Fine</th>
</tr>
</thead>
<tbody>
<tr>
<td>10-27-11</td>
<td>0</td>
<td>33.0</td>
<td>21</td>
<td>20</td>
<td>32.0</td>
<td>11.0</td>
<td>132.58</td>
<td>0.013</td>
<td>55.15</td>
</tr>
<tr>
<td>10-27-11</td>
<td>0.083</td>
<td>33.0</td>
<td>21</td>
<td>20</td>
<td>32.0</td>
<td>11.0</td>
<td>110.48</td>
<td>0.013</td>
<td>55.15</td>
</tr>
<tr>
<td>10-27-11</td>
<td>2</td>
<td>32.0</td>
<td>21</td>
<td>20</td>
<td>31.0</td>
<td>11.2</td>
<td>5.6065</td>
<td>0.013</td>
<td>53.19</td>
</tr>
<tr>
<td>10-27-11</td>
<td>4</td>
<td>31.0</td>
<td>21</td>
<td>20</td>
<td>30.0</td>
<td>11.4</td>
<td>2.8443</td>
<td>0.013</td>
<td>51.23</td>
</tr>
<tr>
<td>10-27-11</td>
<td>8</td>
<td>31.0</td>
<td>21</td>
<td>20</td>
<td>30.0</td>
<td>11.4</td>
<td>1.4221</td>
<td>0.013</td>
<td>51.23</td>
</tr>
<tr>
<td>10-27-11</td>
<td>16</td>
<td>29.0</td>
<td>21</td>
<td>20</td>
<td>28.0</td>
<td>11.7</td>
<td>0.7316</td>
<td>0.013</td>
<td>47.32</td>
</tr>
<tr>
<td>10-27-11</td>
<td>30</td>
<td>28.0</td>
<td>21</td>
<td>20</td>
<td>27.0</td>
<td>11.9</td>
<td>0.3956</td>
<td>0.013</td>
<td>45.37</td>
</tr>
<tr>
<td>10-27-11</td>
<td>60</td>
<td>27.0</td>
<td>21</td>
<td>20</td>
<td>26.0</td>
<td>12.0</td>
<td>0.2006</td>
<td>0.013</td>
<td>43.41</td>
</tr>
<tr>
<td>10-27-11</td>
<td>120</td>
<td>26.0</td>
<td>21</td>
<td>20</td>
<td>25.0</td>
<td>12.2</td>
<td>0.1016</td>
<td>0.013</td>
<td>41.46</td>
</tr>
<tr>
<td>10-27-11</td>
<td>240</td>
<td>24.0</td>
<td>21</td>
<td>20</td>
<td>23.0</td>
<td>12.5</td>
<td>0.0522</td>
<td>0.013</td>
<td>37.55</td>
</tr>
<tr>
<td>10-27-11</td>
<td>480</td>
<td>22.0</td>
<td>21</td>
<td>20</td>
<td>21.0</td>
<td>12.9</td>
<td>0.0268</td>
<td>0.013</td>
<td>33.64</td>
</tr>
<tr>
<td>10-28-11</td>
<td>1440</td>
<td>20.0</td>
<td>21</td>
<td>20</td>
<td>19.0</td>
<td>13.2</td>
<td>0.0092</td>
<td>0.013</td>
<td>29.72</td>
</tr>
<tr>
<td>10-29-11</td>
<td>1440</td>
<td>18</td>
<td>21</td>
<td>20</td>
<td>17.0</td>
<td>13.5</td>
<td>0.0094</td>
<td>0.013</td>
<td>25.81</td>
</tr>
<tr>
<td>10-30-11</td>
<td>2880</td>
<td>16</td>
<td>21</td>
<td>20</td>
<td>15.0</td>
<td>13.8</td>
<td>0.0048</td>
<td>0.013</td>
<td>21.90</td>
</tr>
</tbody>
</table>
Table 5: Results from the standard proctor test for the investigated soil

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volume of Moist Soil, V (cm(^3))</td>
<td>932.36</td>
<td>932.36</td>
<td>932.36</td>
<td>932.36</td>
<td>932.36</td>
<td>932.36</td>
</tr>
<tr>
<td>Weight of Moist Soil, M (g)</td>
<td>1603</td>
<td>1745</td>
<td>1799</td>
<td>1840</td>
<td>1817.3</td>
<td>1797</td>
</tr>
<tr>
<td>Water Content, w (%)</td>
<td>18</td>
<td>21.5</td>
<td>22.9</td>
<td>23.9</td>
<td>28.3</td>
<td>30.2</td>
</tr>
<tr>
<td>Bulk Density, (\rho_b = M/V) (g/cm(^3))</td>
<td>1.72</td>
<td>1.87</td>
<td>1.93</td>
<td>1.97</td>
<td>1.95</td>
<td>1.93</td>
</tr>
<tr>
<td>Dry Density, (\rho_d = \left[\rho_b/(1+w/100)\right]) (g/cm(^3))</td>
<td>1.46</td>
<td>1.54</td>
<td>1.57</td>
<td>1.59</td>
<td>1.52</td>
<td>1.48</td>
</tr>
<tr>
<td>Dry Unit Weight, (\gamma_d) (KN/m(^3))</td>
<td>14.29</td>
<td>15.11</td>
<td>15.4</td>
<td>15.63</td>
<td>14.9</td>
<td>14.52</td>
</tr>
</tbody>
</table>
Figure 1: Shear stress versus strain plot ($\rho_d = 1.46 \text{ g/cm}^3 \text{ & } w = 18\%$)

Figure 2: Shear stress versus normal stress plot ($\rho_d = 1.46 \text{ g/cm}^3 \text{ & } w = 18\%$)

\[
\tau = 0.95 \sigma + 23.7
\]
\[
c' = 23.7 \text{ kPa}
\]
\[
\phi' = 43.5^\circ
\]
Figure 3: Shear stress versus suction \( (\rho_d = 1.46 \text{ g/cm}^3 \text{ & } w = 18\%) \)

Figure 4: Shear stress versus strain plot \( (\rho_d = 1.54 \text{ g/cm}^3 \text{ & } w = 21.5\%) \)
Figure 5: Shear stress versus normal stress plot ($\rho_d = 1.54 \text{ g/cm}^3 \& w = 21.5\%$)

\[ \tau = 0.69 \times \sigma + 48.3 \]
\[ c' = 48.3 \text{ kPa} \]
\[ \phi' = 34.6^0 \]

Figure 6: Shear stress versus suction plot ($\rho_d = 1.54 \text{ g/cm}^3 \& w = 21.5\%$)

\[ \phi^b = 8.3^0 \]
Figure 7: Shear stress versus strain plot ($\rho_d = 1.57 \text{ g/cm}^3$ & $w = 22.9\%$)

Figure 8: Shear stress versus normal stress plot ($\rho_d = 1.57 \text{ g/cm}^3$ & $w = 22.9\%$)
Figure 9: Shear stress versus suction plot \( (\rho_d = 1.57 \text{ g/cm}^3 \& w = 22.9\%) \)

\[ \phi^b = 12.95^\circ \]

Figure 10: Shear stress versus strain plot \( (\rho_d = 1.59 \text{ g/cm}^3 \& w = 23.9\%) \)
Figure 11: Shear stress versus normal stress plot ($\rho_d = 1.59 \text{ g/cm}^3$ & $w = 23.9\%$)

\[ \tau = 0.63 \sigma + 60 \]
\[ c' = 60 \text{ kPa} \]
\[ \phi' = 32.2^\circ \]

Figure 12: Shear stress versus suction plot ($\rho_d = 1.59 \text{ g/cm}^3$ & $w = 23.9\%$)

\[ \phi^b = 14.3^\circ \]
Figure 13: Shear stress versus strain plot ($\rho_d = 1.52 \text{ g/cm}^3$ & $w = 28.3\%$)

Figure 14: Shear stress versus normal stress plot ($\rho_d = 1.52 \text{ g/cm}^3$ & $w = 28.3\%$)

\[ \tau = 0.53 \sigma + 54 \]
\[ c' = 54 \text{ kPa} \]
\[ \phi' = 27.9^\circ \]
Figure 15: Shear stress versus suction plot ($\rho_d = 1.52 \text{ g/cm}^3$ & $w = 28.3\%$)

Figure 16: Shear stress versus strain plot ($\rho_d = 1.48 \text{ g/cm}^3$ & $w = 30.2\%$)
Figure 17: Shear stress versus normal stress plot \((\rho_d = 1.48 \text{ g/cm}^3 \text{ & } w = 30.2\%)\)

\[
\tau = 0.51 \times \sigma + 32.7 \\
c' = 32.7 \text{ kPa} \\
\phi' = 27^\circ
\]

Figure 18: Shear stress versus suction plot \((\rho_d = 1.48 \text{ g/cm}^3 \text{ & } w = 30.2\%)\)

\[
\phi_b = 20.6^\circ
\]
Figure 19: Shear stress versus strain plot for saturated compacted soil ($\rho_d = 1.59 \text{ g/cm}^3$ & $w = 23.9\%$)

Figure 20: Shear stress versus normal stress plot for saturated compacted soil ($\rho_d = 1.59 \text{ g/cm}^3$ & $w = 23.9\%$)

\[ \tau = 0.63 \sigma + 27.9 \]

$\phi' = 32.2^\circ$

\[ c' = 27.9 \text{ kPa} \]
Figure 21: Shear stress versus strain plot for natural soil ($\rho_d = 1.37 \text{ g/cm}^3$ & $w = 33\%$)

Figure 22: Shear stress versus normal stress plot for natural soil ($\rho_d = 1.36 \text{ g/cm}^3$ & $w = 33\%$)

\[ \tau = 0.53 \sigma + 39.33 \]
\[ c' = 39.3 \text{ kPa} \]
\[ \phi' = 27.9^\circ \]
Figure 23: Shear stress versus suction plot for natural soil ($\rho_d = 1.36 \text{ g/cm}^3$ & $w = 33\%$)

Figure 24: Shear stress versus strain plot for saturated natural soil ($\rho_d = 1.36 \text{ g/cm}^3$ & $w = 33\%$)
Figure 25: Shear stress versus normal stress plot for saturated natural soil ($\rho_d = 1.36 \text{ g/cm}^3$ & $w = 33\%$)

\[ \tau = 0.43 \times \sigma + 16.3 \]
\[ c' = 16.3 \text{ kPa} \]
\[ \phi' = 23.3^\circ \]
Soil Water Characteristics Curve:

The following results were obtained for the determination of soil-water characteristics. The calculation for determining water content is used the same procedure described in the in situ water content section. The conversion for the volumetric water content was done according to the following equation:

\[
\theta = w\left(\frac{\rho_d}{\rho_w}\right)
\]

where:

- \(\theta\) = volumetric water content
- \(w\) = water content
- \(\rho_d\) = Dry unit weight
- \(\rho_w\) = Unit weight of water
### Table 9: Results from soil water characteristics curve and swell-shrinkage curve determination

<table>
<thead>
<tr>
<th>Pressure, Kpa</th>
<th>Sat</th>
<th>5</th>
<th>10</th>
<th>25</th>
<th>50</th>
<th>100</th>
<th>200</th>
<th>300</th>
<th>450</th>
<th>1000</th>
<th>2500</th>
<th>6000</th>
<th>7950</th>
<th>1630</th>
<th>2230</th>
<th>2670</th>
<th>3980</th>
<th>5340</th>
<th>6870</th>
<th>10000</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass of moist soil, g</td>
<td>11.2</td>
<td>11.2</td>
<td>14.5</td>
<td>26.3</td>
<td>25.4</td>
<td>2</td>
<td>15.3</td>
<td>15</td>
<td>23.6</td>
<td>18</td>
<td>9.1</td>
<td>20.6</td>
<td>19.6</td>
<td>5</td>
<td>22.6</td>
<td>8</td>
<td>23.0</td>
<td>19.5</td>
<td>21.7</td>
<td>5</td>
</tr>
<tr>
<td>Mass of moist soil + wax, g</td>
<td>17.0</td>
<td>17.0</td>
<td>19.3</td>
<td>32.1</td>
<td>29.5</td>
<td>6</td>
<td>20.8</td>
<td>6</td>
<td>28.4</td>
<td>2</td>
<td>31.95</td>
<td>12.8</td>
<td>1</td>
<td>30.0</td>
<td>4</td>
<td>26.3</td>
<td>9</td>
<td>28.5</td>
<td>8</td>
<td>32.35</td>
</tr>
<tr>
<td>Mass of wax, g</td>
<td>5.88</td>
<td>5.88</td>
<td>4.82</td>
<td>5.84</td>
<td>4.08</td>
<td>5.54</td>
<td>4.77</td>
<td>8.7</td>
<td>37.1</td>
<td>9.44</td>
<td>6.74</td>
<td>5.88</td>
<td>9.3</td>
<td>4</td>
<td>4.7</td>
<td>8.5</td>
<td>5.88</td>
<td>7.45</td>
<td>8.2</td>
<td>7.3</td>
</tr>
<tr>
<td>Mass of sringe + water, g</td>
<td>378.36</td>
<td>378.36</td>
<td>363.93</td>
<td>363.93</td>
<td>363.93</td>
<td>360.67</td>
<td>363.93</td>
<td>363.93</td>
<td>363.93</td>
<td>363.93</td>
<td>363.93</td>
<td>363.93</td>
<td>363.93</td>
<td>363.93</td>
<td>363.93</td>
<td>363.93</td>
<td>363.93</td>
<td>363.93</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Mass of sringe + water + wax soil</td>
<td>36.5</td>
<td>36.5</td>
<td>350.17</td>
<td>341.63</td>
<td>344.06</td>
<td>348.63</td>
<td>331.33</td>
<td>340.3</td>
<td>354.59</td>
<td>342.04</td>
<td>345.63</td>
<td>345.73</td>
<td>342.34</td>
<td>348.9</td>
<td>343.6</td>
<td>346.3</td>
<td>344.9</td>
<td>345.3</td>
<td>343.2</td>
<td></td>
</tr>
<tr>
<td>Mass of water, g</td>
<td>12.9</td>
<td>12.9</td>
<td>13.7</td>
<td>21.9</td>
<td>19.5</td>
<td>4</td>
<td>15.3</td>
<td>19.3</td>
<td>4</td>
<td>23.3</td>
<td>9.34</td>
<td>21.5</td>
<td>17.9</td>
<td>6</td>
<td>17.8</td>
<td>7</td>
<td>21.6</td>
<td>7</td>
<td>14.7</td>
<td>20.17</td>
</tr>
<tr>
<td>soil Gs</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>2.74</td>
<td>-</td>
</tr>
<tr>
<td>Wax Gs</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>0.91</td>
<td>-</td>
</tr>
<tr>
<td>specimen vol</td>
<td>6.44</td>
<td>6.44</td>
<td>8.46</td>
<td>15.5</td>
<td>15.0</td>
<td>5</td>
<td>9.20</td>
<td>14.0</td>
<td>9</td>
<td>13.66</td>
<td>5.26</td>
<td>11.1</td>
<td>8</td>
<td>10.5</td>
<td>6</td>
<td>11.4</td>
<td>0</td>
<td>11.33</td>
<td>9.53</td>
<td>10.6</td>
</tr>
<tr>
<td>bulk density</td>
<td>1.73</td>
<td>1.73</td>
<td>1.71</td>
<td>1.69</td>
<td>1.68</td>
<td>1.66</td>
<td>1.67</td>
<td>1.70</td>
<td>1.72</td>
<td>1.84</td>
<td>1.86</td>
<td>1.98</td>
<td>2.52</td>
<td>2.04</td>
<td>2.04</td>
<td>2.05</td>
<td>2.04</td>
<td>2.05</td>
<td>2.03</td>
<td>-</td>
</tr>
<tr>
<td>dry density</td>
<td>1.18</td>
<td>1.18</td>
<td>1.18</td>
<td>1.19</td>
<td>1.20</td>
<td>1.21</td>
<td>1.25</td>
<td>1.29</td>
<td>1.33</td>
<td>1.47</td>
<td>1.52</td>
<td>1.68</td>
<td>1.73</td>
<td>1.79</td>
<td>1.79</td>
<td>1.80</td>
<td>1.80</td>
<td>1.81</td>
<td>1.81</td>
<td>-</td>
</tr>
<tr>
<td>void ratio</td>
<td>1.24</td>
<td>1.24</td>
<td>1.22</td>
<td>1.19</td>
<td>1.16</td>
<td>1.14</td>
<td>1.11</td>
<td>1.09</td>
<td>1.09</td>
<td>1.06</td>
<td>0.86</td>
<td>0.8</td>
<td>0.63</td>
<td>0.53</td>
<td>0.53</td>
<td>0.52</td>
<td>0.52</td>
<td>0.51</td>
<td>0.49</td>
<td>0.5</td>
</tr>
<tr>
<td>degree of saturation</td>
<td>96.6</td>
<td>96.6</td>
<td>92.5</td>
<td>88.4</td>
<td>86.5</td>
<td>5</td>
<td>80.9</td>
<td>77.8</td>
<td>2</td>
<td>76.4</td>
<td>6</td>
<td>76.4</td>
<td>6</td>
<td>76.2</td>
<td>6</td>
<td>79.6</td>
<td>3</td>
<td>79.1</td>
<td>18</td>
<td>74.6</td>
</tr>
<tr>
<td>wt of can</td>
<td>32.3</td>
<td>32.3</td>
<td>32.2</td>
<td>32.2</td>
<td>32.2</td>
<td>32.2</td>
<td>32.2</td>
<td>32.2</td>
<td>32.2</td>
<td>32.2</td>
<td>32.2</td>
<td>32.2</td>
<td>32.2</td>
<td>32.2</td>
<td>32.2</td>
<td>32.2</td>
<td>32.2</td>
<td>32.2</td>
<td>32.2</td>
<td>32.2</td>
</tr>
<tr>
<td>wt of can+wet soil</td>
<td>53.4</td>
<td>53.4</td>
<td>72.2</td>
<td>70.3</td>
<td>46.9</td>
<td>4</td>
<td>51.8</td>
<td>54.8</td>
<td>55.24</td>
<td>54.6</td>
<td>54.4</td>
<td>55.3</td>
<td>45.7</td>
<td>46.9</td>
<td>54.5</td>
<td>49.0</td>
<td>53.0</td>
<td>48.9</td>
<td>53.0</td>
<td>48.9</td>
</tr>
<tr>
<td>wt of can+dry soil</td>
<td>46.7</td>
<td>46.7</td>
<td>60.0</td>
<td>59.1</td>
<td>42.7</td>
<td>4</td>
<td>46.6</td>
<td>49.1</td>
<td>49.7</td>
<td>49.6</td>
<td>49.9</td>
<td>50.9</td>
<td>43.6</td>
<td>44.8</td>
<td>51.7</td>
<td>47.0</td>
<td>50.5</td>
<td>45.8</td>
<td>47.9</td>
<td>5.8</td>
</tr>
<tr>
<td>water content</td>
<td>46.2</td>
<td>46.2</td>
<td>44.2</td>
<td>41.9</td>
<td>40.5</td>
<td>37.2</td>
<td>33.4</td>
<td>31.4</td>
<td>29.2</td>
<td>23.3</td>
<td>22.2</td>
<td>18.3</td>
<td>16.7</td>
<td>14.4</td>
<td>14.0</td>
<td>13.7</td>
<td>13.5</td>
<td>13.0</td>
<td>12.3</td>
<td>12.3</td>
</tr>
<tr>
<td>θ</td>
<td>54.8</td>
<td>54.8</td>
<td>52.5</td>
<td>49.9</td>
<td>48.5</td>
<td>9</td>
<td>45.1</td>
<td>42.1</td>
<td>40.4</td>
<td>39.1</td>
<td>37.2</td>
<td>33.8</td>
<td>30.7</td>
<td>29.0</td>
<td>25.8</td>
<td>25.4</td>
<td>24.9</td>
<td>24.3</td>
<td>23.6</td>
<td>22.3</td>
</tr>
</tbody>
</table>

98