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Effect of Swell-shrink Characteristics on Landslides in Yazoo Clay

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Prepared by:

Sadik Khan, Ph.D., P.E.
John Ivoke, S.M.ASCE
Masoud Nobahar
Department of Civil and Environmental Engineering,
Jackson State University,
1400 J. R. Lynch St, Box 17068
Jackson, MS, 39217
Phone: 601-979-6373
Email: J00797693@jsums.edu

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University of Arkansas
4190 Bell Engineering Center
Fayetteville, AR 72701
479-575-6021

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ABSTRACT

Slope failures are frequent in highway embankments as well as in waterway infrastructures (levees) on expansive Yazoo clay in Mississippi which cause significant maintenance problems and require millions of state and federal dollars to fix it. After construction, the strength of the high plastic clay degrades with time due to the seasonal temperature and moisture variation, which is one of the significant factors of slope failure. However, no study is available on the strength reduction of Yazoo clay soil. The current study intends to investigate the repeated drop in the shear strength of the Yazoo clay soil with wet-dry cycles which cause slope failure. Representative Highly plastic Yazoo clay soil samples were collected from slope sites to investigate the soil mechanical properties. The high plastic Yazoo clay samples were tested at the laboratory to investigate the effect of wetting and drying cycles on the degradation of the shear strength. The study begins with the laboratory testing to quantify the progressive changes in the shear strength of the Yazoo clay soil. The test results also indicated that the highest shear strength ($c = 18.4$ kPa and $\varphi = 20.2^o$) was determined with the peak shear strength test, whereas the residual test generated the lowest strength ($c = 5.26$ kPa and $\varphi = 12.8^o$). Further laboratory testing was conducted to investigate the change in the void ratio of Yazoo clay soil under series of a wet and dry cycle, which eventually cause the changes in the shear strength. Reconstituted expansive clay soil samples were used for the experiment. The samples were subjected to 3, 5, and 7 numbers of wetting and drying cycles in an enclosed chamber. During the drying process, the temperature ranged from 120-125 deg. F to simulate the typical high summer temperature in Mississippi. The test results indicated that the void ratio increases with the progressive number of wet and dry cycles. With the continuous increments in void ratios, the soil has more void spaces which reduce the shear strength of the Yazoo clay. Further study was conducted using 2D Finite Element Methods in Plaxis 2D to investigate the progressive changes in the factor of safety of the slope constructed over the Yazoo clay soil. A highway slope over I220 near US 80 in Jackson, MS was considered as a reference slope. It was 3H: 1V slope with 9.2 m height. The factor of safety of the slope was determined based on the existing soil test data, with a peak shear strength. Later the topsoil layer which gets weathered due to the repeated wet-dry cycle was changed to fully soften, and residual shear strength and the effect of each shear strength (peak, fully soften and residual) on slope stability was evaluated. The slope stability analysis results indicated that the slope is stable at the dry condition, even with the residual shear strength. However, considering a perched water condition at the topsoil due to the effect of rainfall, soil slope failed at fully soften shear strength. Thus, the fully softened shear strength with the perched water condition due to rainfall triggers the slope failures in Yazoo clay.

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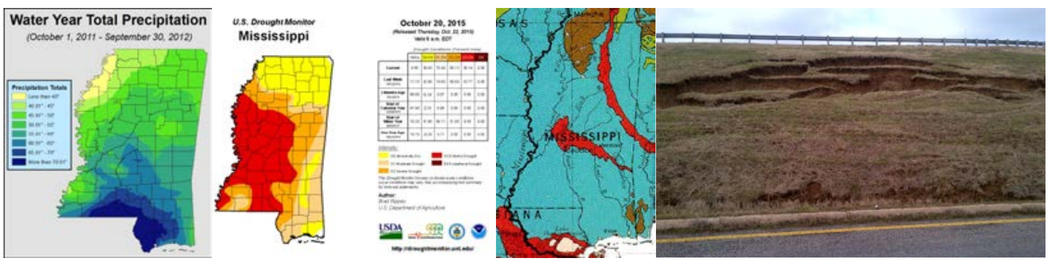
Chapter 1: INTRODUCTION

1.1 Research Challenge

Slopes and embankments are integral components of the maritime and multimodal transportation system. Failure of the slope often causes significant hazards to multimodal transportation infrastructure, and if not properly maintained, it requires more extensive and expensive repairs. Many of the slopes in multimodal transportation infrastructure (highway embankment and levees) of Mississippi are constructed using marginal highly plastic clay soil (Yazoo clay) which are highly expansive and are known to be susceptible to shallow landslide. Typically, shallow slope failure occurs due to an increase in pore water pressure and reduction of soil strength due to progressive wetting of near-surface soil. Moisture variations further intensify this condition due to seasonal climatic changes that result in cyclic shrink and swell of the high plastic clay soil. The variation of rainfall, temperature, and soil condition of Mississippi are presented in Figure 1.1, which eventually work as a stressor to cause shallow slope failure.



(a) (b) (c) (d)

Figure 1.1 a. Total Precipitation map, b. Drought map, c. Yazoo clay profile in Mississippi, d. Shallow slope failure

The Yazoo Formation containing Yazoo clay is geologically defined within the Jackson Group that have been identified throughout the southeastern and southwestern United States. The upper stratigraphy of the Jackson Group that contains Yazoo clay (or its geological equivalent) extends in regional locations across Alabama, Louisiana, and Mississippi. The regional extent of the Yazoo clay lies within the central Mississippi counties of Yazoo, Holmes, Hinds, Rankin, Madison, Scott, Newton, Smith, Jasper, and Wayne. The horizontal width of the surface outcrop varies from approximately 35 miles on the west to less than 10 miles on the east, whereas; the metropolitan Jackson area is located directly on top of the Yazoo clay (Lee 2012).

Yazoo Clay is highly plastic and subjected to high volume change (Douglas and Dunlap, 2000, Lee 2012). The average composition of the Yazoo clay is 28% smectite (probably montmorillonite), 24% kaolinite, 22% quartz, 15% calcite, 8% illite, 2% feldspar, and 1% gypsum based on recent x-ray diffraction results (Taylor 2005). In geotechnical engineering practice, the magnitude of the liquid limit and plasticity index and the proximity of the natural water content to the plastic limit reflects of the potential to shrink or swell upon changes in moisture content of clay soil. The weathered Yazoo clay usually has a liquid limit higher than 70% to 100% and plastic limits from 20% to 30%, results in a plasticity index greater than 50%. (Douglas and Dunlap, 2000). Yazoo Clay is indicated to have a very high shrink/swell potential, and moisture changes result in expansive, swelling, shrinkage, and otherwise, destructive behavior causes detrimental effect to the roads, foundations, and related infrastructure in the central Mississippi region

(Douglas and Dunlap, 2000; Lee 2012). The change of volume of the Yazoo clay between the liquid limit and oven dry moisture contents is ranged from 100 to 235 percent. On the other hand, swell pressures have been measured more than 25,000 psf (Johnson 1973).

Moderate to steep slope constructed on high plasticity clay is susceptible to the softening behavior at the topsoil due to wet-dry cycle (Khan et al., 2017). The fully softened shear strength corresponds to the shear strength of high plastic clay which develops over time due to the wetting and drying cycle (Wright, 2005). Skempton (1977) first proposed the concept of fully softened strength for natural and excavated slopes in London Clays. Skempton (1977) reported that over time the strength of slopes in the highly plastic London clay lost strength, eventually reaching what Skempton termed as “fully-softened” strength which lies between peak and residual strength as presented in Figure 1.2. Skempton (1977) indicated that the fully-softened strength is comparable to the shear strength of the soil in a normally consolidated state. Rogers and Wright (1986) conducted a study using the drained direct shear test to investigate the failure of slope constructed over highly plastic clay soil in Texas. During the study, authors subjected the high plastic clay soil sample at several repeated wetting and drying cycles. Rogers and Wright (1986) reported that cyclic wetting and drying of the soil would produce a significant shear strength loss, particularly regarding effective cohesion intercept, c’. The study also indicated that the loss in cohesion occurs within a relatively few numbers of cycles of wetting and drying. In fact, most of the loss in strength occurred in the first cycles.

The Yazoo clay soil has undergone the similar softening behavior, which has become a major driving factor on slope failure in highway embankment and waterway infrastructures in Mississippi. However, to date, no study is available that tested the development of the softening behavior in Yazoo clay soil in Mississippi. Hence, the current study intends to investigate the development of the softening behavior of the expansive clay soil and associated effect on the slope failure.

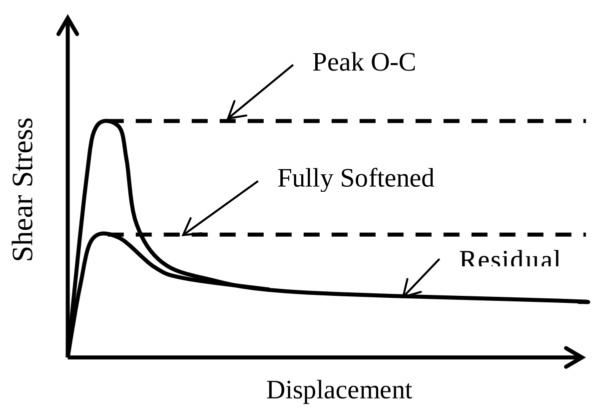


Figure 1.2 Comparisons of peak, residual and fully softened shear strength (redrawn after Skempton (1977))

1.2 The objective of the Study

The major objective of the current proposal is to study the failure mechanism of slopes on highway embankment and waterway infrastructures (levees) constructed using Yazoo clay soil, by investigating the degradation in the shear strength with wet-dry cycles. To undertake this objective, the current study focused on 2 major tasks as presented below.

a. Investigate the mechanism of the development of the fully soften shear strength of Yazoo clay with wet-dry cycles at the laboratory.
b. Effect of development of the fully soften strength along with slope failure in Mississippi.

1.3 Specific Tasks

To accomplish the project objective, the following 5 tasks have been conducted.

Task 1: Site Selection and Sample Collection

Task 2: Investigation of Shear Strength properties

Task 3: Investigation of Wet-Dry properties

Task 4: Effect of Wet-Dry Cycles and Rainfall on Slope Failure

Task 5: Data Analysis and Recommendation

Chapter 2: LITERATURE REVIEW

2.1 Identification of Expansive Soils

The identification of potential swelling or shrinking problems in the subsoil is essential for selecting the appropriate design and method of construction (Van Der Merwe, 1964; Hamilton, J. J 1966). Despite the lack of a standard definition of swell potential, there exists various geotechnical methods to identify the swelling potential of soil (Nelson and Miller, 1992). Surface examination as well as geological and geomorphological description can indicate expansive soils (Lucian, C. 2008). Identification is not just restricted to the present visual precursors of expansive soil, but also the careful review of the formation history of the grains. Generally, the soil textures are a result of geological history, soil composition, sedimentation, climatic and hydrological conditions, precipitation, and the pH levels.

The morphological description includes a host of things; such as groundwater table situation, the color of the soil, soil consistency, soil texture, soil structure, texture groups (Charles L. 2008). Most of the relevant physical and mechanical property indicators of swell potential are obtained by performing geotechnical index property tests such as Atterberg limits, unit weights, and grain size distribution. Other tests to determine the swell potential includes the volume change test (free swell and swell in odometer test), the coefficient of linear extensibility (COLE), mineralogical compositions by x-ray diffraction (XRD) test, and total suction test.

Lucian, C. (2008) stated that the geotechnical methods of identification with expansive soils can be broadly divided into direct and indirect methods. The direct method consists essentially of laboratory swell tests, while indirect methods are based on the correlation of measured soil properties, relying on the empirical correlations of geotechnical properties like moisture content, Atterberg's limits, and swell index.

2.1.1 *Identification by Atterberg Limits*

Casagrande (1932), In the year 1911, A. Atterberg (a Swedish soil scientist) proposed the Liquid Limit (LL), Plastic Limit (PL), and Shrinkage Limit (SL) in an effort to consistently classify the soils and understand the correlation between the limits and engineering properties like compressibility, shear strength and permeability. These limits represent the water holding capacity at different states of consistency. They are the most popular techniques for getting information on the expansive nature, mechanical aspects, and swelling behavior of clay soils (Williams, 1958). The most useful for classifying and identifying the relative swell potential of soil are liquid limits (LL) and plasticity index (PI).

The liquid limit is the water content at which a soil changes from the plastic state to a liquid state, while the plastic limit is the water content at which a soil changes from the plastic state to a semisolid state (Figure 2.1). The plasticity index is calculated by subtracting the plastic limit (PL) from the liquid limit (LL). i.e., PI = LL-PL. This number indicates the range over which the soils remain plastic. Soils that possess no clay minerals do not exhibit plasticity, they pass directly from their liquid limit to a semi-solid state when their moisture content is reduced. Clayey soils which is rich in smectite tend to absorb more water and thus exhibit greater swelling than non-expansive clays like chlorite, illite, and kaolinite. Generally, finer soils have a higher capacity to hold water due to their greater particle surface area. On the other hand, clayey soil which is rich in smectite retain their plasticity at lower moisture contents as opposed to non-expansive clays such as chlorite, illite, and kaolinite.

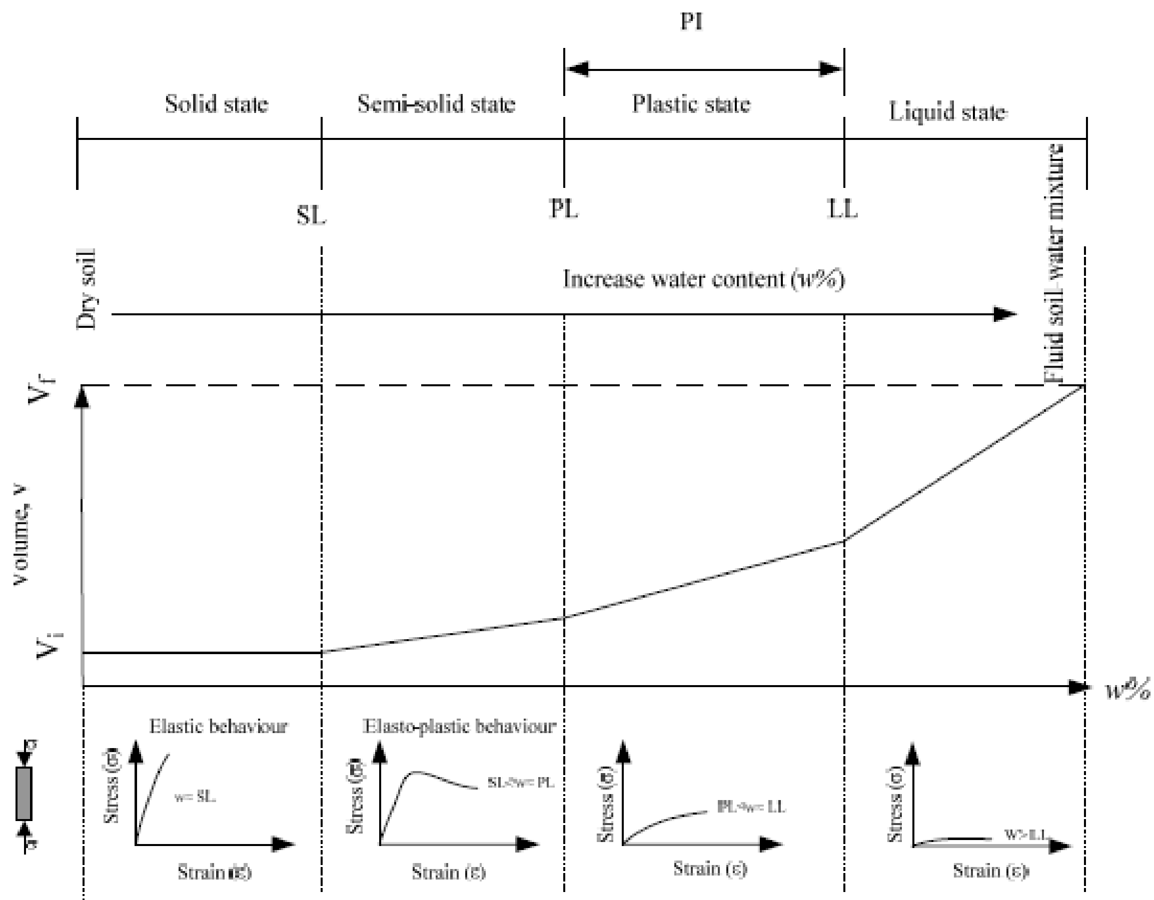


Figure 2.1 Atterberg limits description, volume change and generalized stress-strain response of expansive soils (After Holtz and Kovacs (1981))

2.2 Indirect Measurement of Potential Swell

2.2.1 *Classification of potential swell based on Casagrande's plasticity chart*

The plot of PL vs LL is used to detect the potential swell of soil and can be done according to Casagrande's plasticity chart (Figure 2.2). For example, a soil sample with a LL of 40% and PI of 25% plots in the zone typical for smectites (montmorillonite) implies that it has a high potential for swelling. Soils that plot above the A-line are heavy or plastic clays, and those which plot below it are organic soils, silts and clayey soils containing a sizeable portion of rock flour (BS 5930, 1981). The U-line indicates the upper boundary for natural soils. Thus no soil should plot above U-line.

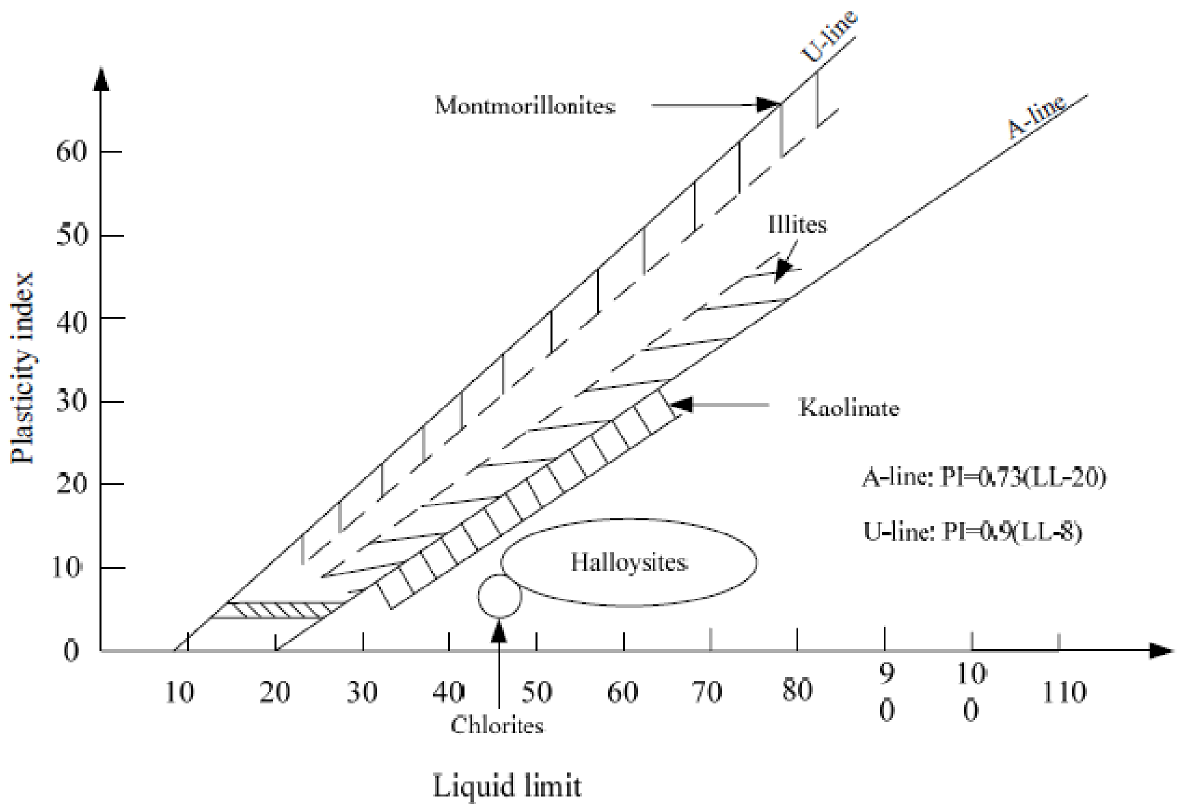


Figure 2.2 The plot of clay minerals on Casagrande's chart (Chleborad et al. (2005))

Table 2.1 Potential swell based on plasticity (Holtz and Gibbs (1956))

Classification of potential swell	Liquid limit (LL), %	Plasticity index (PI), %	Shrinkage limit (SL), %
Low	20-35	<18	>15
Medium	35-50	15-28	10-15
High	50-70	25-41	7-12
Very high	>70	>30	<11

2.2.2 *Classification of Potential Swell Based on Plasticity Table*

The change in Atterberg limits of a soil sample can be used to indicate the degree for potential swell as presented in Table 2.1. For example, a soil sample with LL exceeding 70% and PI greater than 35% is judged to have a very high swell potential. Presumably, the overlapping intervals in Table 2.2 account for the variations in the chemical properties of soils and their environment.

2.2.3 *Classification of Potential Swell Based on Advanced Physical Properties of Soils*

Skempton, 1954, Seed et al., 1960 and Van Der Merwe, 1964 developed useful empirical relationships between the expansion potential and physical properties of soils like clay contents, soil activity, plasticity index, (Figure 2.3). A preliminary classification based on percent clay fraction (soil particles < 0.002 mm or 2 μm in φ, usually determined in hydrometer test) and PI can be used to categorize probable severity.

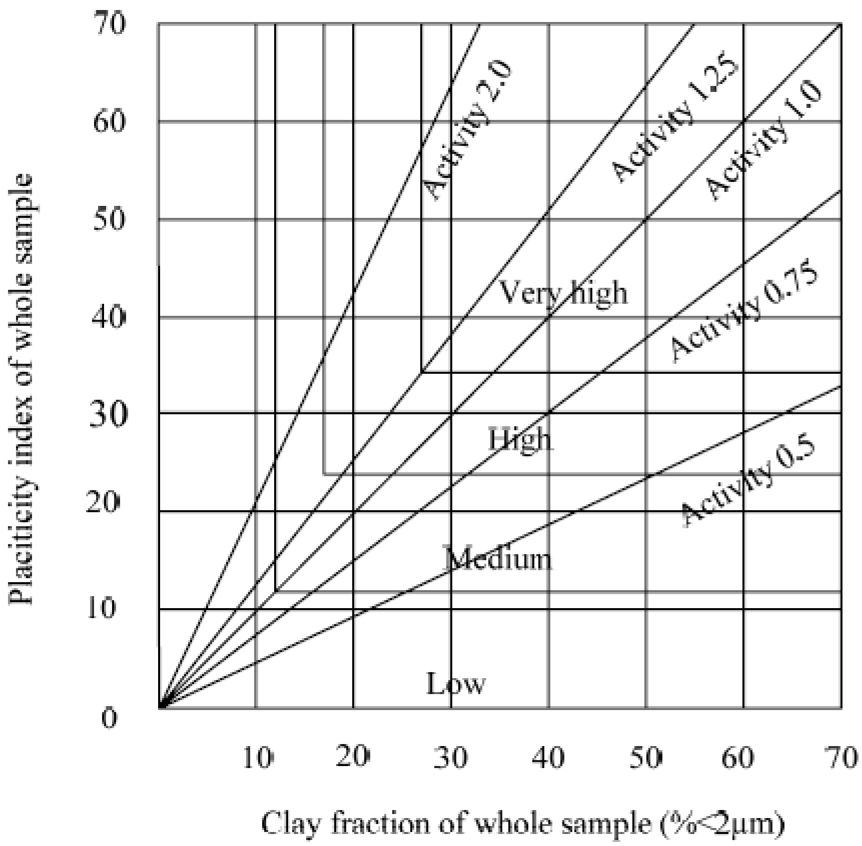


Figure 2.3 Chart for evaluation of potential expansiveness (Seed et al. (1960))

Generally, a soil having clay content higher than 30 percent and a plasticity index greater than 35 percent is considered to denote a very high potential for shrinkage or swelling (active soil). On the other hand, soil with clay content and a plasticity index more than about 10% to 20% may undergo at least slight swelling or shrinking in response to environmental changes (Lucian, C. (2008)). The activity in Figure 2.3 is taken as the dimensionless ratio of PI to colloids contents, both taken in percent. Thus;

Activity (Ac) = plasticity index (PI) in %/clay fraction finer than 2µm in % (2a)

Soil with activity less than 0.75 is inactive, indicating a low potential for volume change. In addition, soil with activity between 0.75 and 1.0 is actively signifying a high potential for volume change. Anything above 1.0 is very active, demonstrating very high potential for volume change.
Another way of identifying the expansive soil is to use the activity method quoted by Carter and Bentley (1991). The proposed classification chart is shown in Figure 2.4. The activity term in the Figure 2.4 is defined a bit differently from the equation (2a) which is as follows:

$A_c = PI/(C - 5)$ (2b)

where PI is plasticity index, and C is clay content.

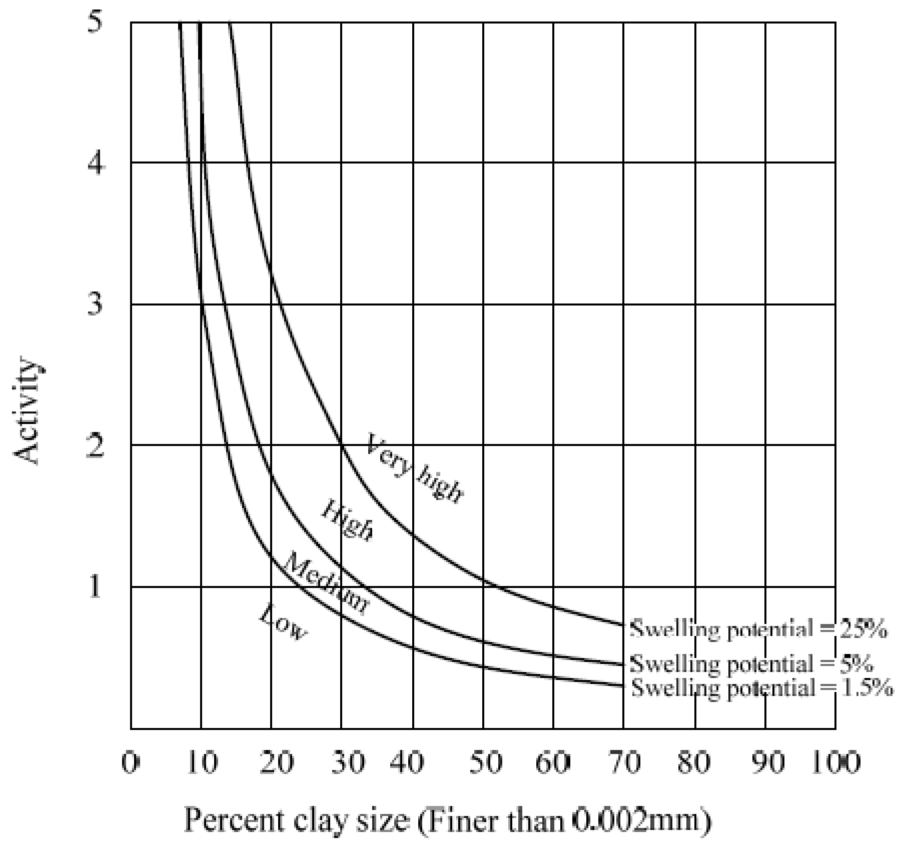


Figure 2.4 Classification chart for swelling potential proposed by Carter and Bentley (1991)

Several researchers have proposed empirical relationships to predict the swelling pressure of soils using soil characteristics like clay content, activity, and plastic limit. Carter and Bentley (1991) proposed an empirical equation (equation 2c-i) to calculate the potential swell (Table 2.2) as follows:

$$\text{Swell (\%)} = 60\kappa\, (PI)^{2.44} \qquad \text{(2c-i)}$$

where PI is the plasticity index, and κ is the constant, equal to 3.6 x 10-5

Seed et al. (1962) suggested that the swelling potential of clay soil is related to its activity and clay content by the following formula:

$$\text{Swell (\%)} = \kappa\, (A_c^{2.44})(C^{3.44})\, c \qquad \text{(2c-ii)}$$

Where Ac is the soil activity and C is the clay content

Table 2.2 Identification of potential swell based on plasticity (Carter and Bentley (1991))

Classification of Potential Swell	Plasticity Index (%)	Plasticity Index (%)
Low (0-1.5%)	0-15	0-15
Medium (1.5-5%)	10-30	15-24
High (5-25%)	20-55	25-46
Very high (>25%)	>40	>46

2.2.4 *Swelling Potential Determination Based on Suction Values*

Soil suction is a microscopic property that indicates the intensity or free energy level (force per unit area) of water that the soil attracts (Fredlund and Rahardjo, 1993; Bulut et al., 2001; Ridley et al., 2003; Rao and Shivananda, 2005; and Sreedeep and Singh, 2006). Soil suction is comprised of two components; osmotic and matric (capillary).

Matric suction creates a capillary phenomenon due to its nature of the soil texture and adsorptive forces of unsaturated soils; this varies with changes in moisture content of the soils. The osmotic suction is a result of the presence of dissolved salts in the pore fluid. The sum of the matric suction and osmotic suction equals to total suction. The relationship between the total, osmotic, and matric suctions under isothermal conditions is shown in equation (2.d) (Chen, 1998).

Total suction ht = ho+ hm (2.d)

(Assuming gravitational and external pressure effects are negligible)
Where h_o is the osmotic suction and
$h_m = (h_a - h_w)$ is the matric suction
h_a = pore-air pressure
h_w = pore-water pressure

The simplest method to conduct the suction test over a wide range of suction is by using filter paper by the ASTM D 5298. This involves collecting the undisturbed samples from ground profiles and taking them to the laboratory for testing. The samples are split across their diameters to form a series of soil disks, then the filter paper is inserted between the discs and sealed within an easily installed sensing chamber. The samples are then stored for an at least seven days. After the filter papers have reached suction equilibrium with the surrounding soil, the moisture content is carefully measured. The results are then related to a total suction value through calibration curves obtained from an established correlation for equilibrium filter papers, over the salt solution of known total suction. Figures 2.5 and 2.6 show the wetting curve constructed using NaCl salt solution and Schleicher & Schuell No. 589-WH filter papers. The curve has two regimes; the upper segment represents moisture retained in the soil by the surface adsorption processes, while the lower part represents moisture retained by surface tension and capillary forces between particles (ASTM D 5298). The suction is calculated either in log kPa (10 log $|$suction in kPa$|$) units (Figure 2.5) or in pF (10 log $|$suction in cm of water$|$) units (Figure 2.6). The two systems are approximately related by suction in log kPa = suction in pF-1 (Bulut et al., 2001). From the Figures 2.5 and 2.6, the relationships between suction in log kPa as well as pF are summarized in Table 2.3. Suction is zero in soils whose moisture is in balance with the free water and greater than zero in soils above the groundwater level. The maximum value of suction is reached at about pF = 7 corresponding to clay dried in an oven at 110°C (Trevisan, 1988). Once suction has been obtained, the swell can be readily calculated. Brackley (1980) proposed an empirical equation (equation 2.e) to calculate the swelling pressure based on suction values and effective overburden stress at the depth in question:

$$\text{Swell } \% = \frac{PI - 10\log_{10}\frac{S}{P}}{10} \qquad (2.e)$$

where S is the soil suction at the center of the layer, PI is the plasticity index, and P is the overburden plus foundation stress at that depth

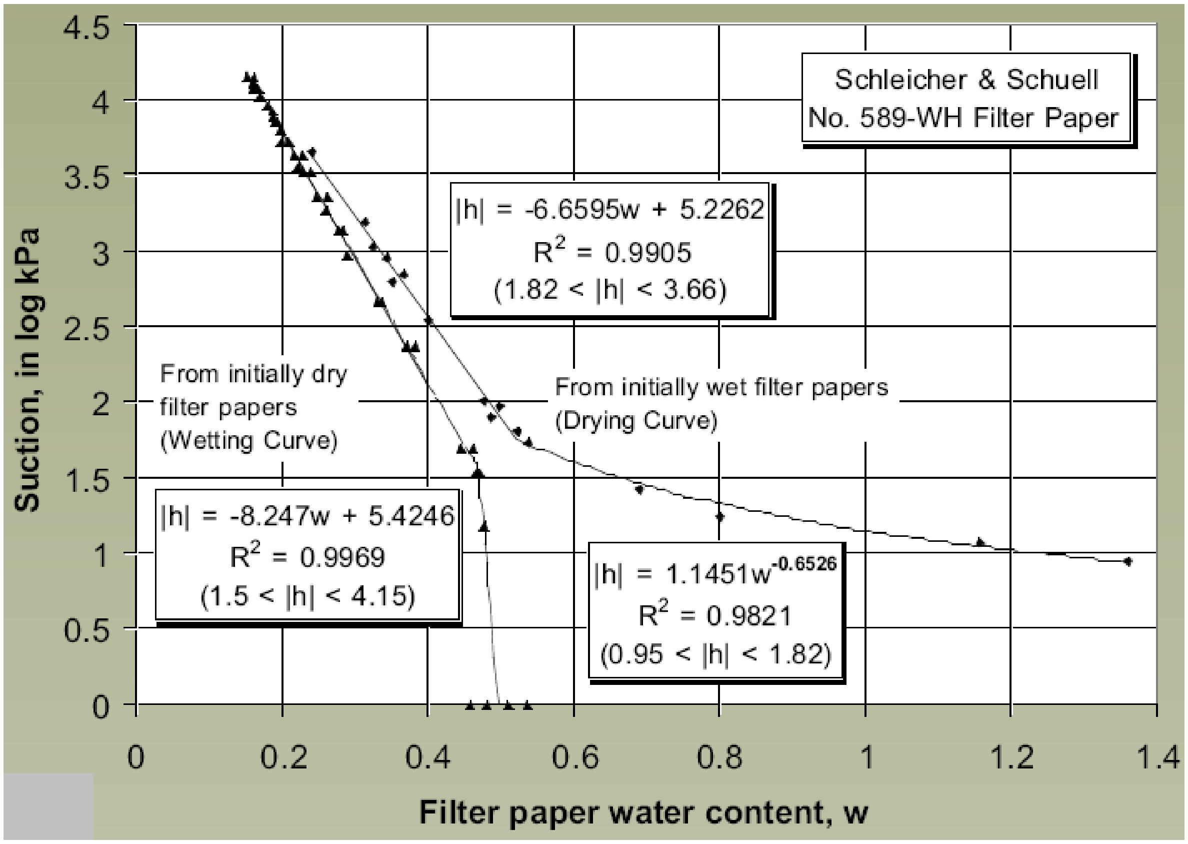


Figure 2.5 Filter paper drying calibration curve along with wetting suction curve for determination of suction in log kPa (Bulut et al. (2001))

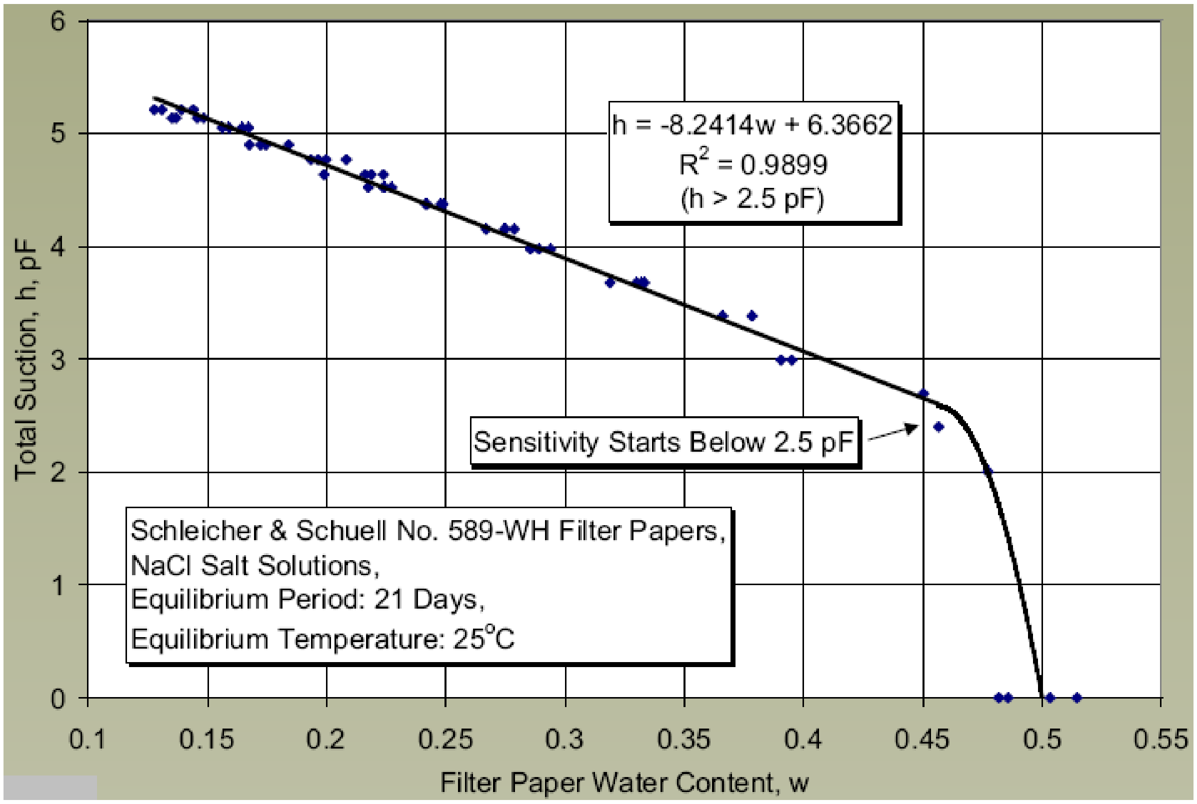


Figure 2.6 Filter paper wetting calibration curve for determination of suction in pF (Bulut et al. (2001))

Table 2.3 Filter paper calibration relationships

Filter paper	**log kPa (10 logs (│suction in kPa│)**	**pF (10 logs (│suction in cm of water│)**
Schneider & Schuell No. 589-WH	$h = 5.4246 - 8.247w$ R2 = 0.9969 $(1.5 < h < 4.15)$	$\|h\| = 6.3662 - 8.2414w$ R2 = 0.9899 (h > 2.5 pF)

2.3 Climate and hydrological condition

Climate, hydrological conditions, environmental conditions, topography, and geology govern the formation and behavior of soils. The climate, in particular, is one of the essential factors in soil profile development. It helps to change parent material into the soil. Climatic factors, such as precipitation, wind and sunlight, and temperature accelerate the formation of the basic material of soil. The soil is a mixture of rock fragments, minerals, air, water, and organic materials. Soils vary due to the different ingredients they contain, and climate contributes to those differences. For example, the climatic and topographical conditions under which smectite is formed is entirely different from that of kaolinite. The formation of smectite requires low relief, low permeability, low rainfall and low temperature. In these conditions, the environment offers itself extreme disintegration, intense hydration and restrained leaching appropriate for the formation of smectite rich, expansive soils (Tourtelot, 1973 and Azam et al., 1998). In contrast, high temperature, strong hydrolysis by high permeability and high rainfall intensities favor the formation of kaolinite (Tourtelot, 1973 and Weaver, 1989). Therefore, while expansive clays such as montmorillonite are more prevalent in drier environments, non-expansive clays, like kaolinite are more common in warm, humid environments.

2.4 Yazoo Clay

Yazoo clay soil is highly expansive and extended over central Mississippi, Alabama, and Southern Louisiana (Figure 2.7). Most of the structures are constructed on expansive Yazoo clay in Mississippi. The average composition of the Yazoo clay is 28% smectite (probably montmorillonite), 24% kaolinite, 22% quartz, 15% calcite, 8% illite, 2% feldspar, and 1% gypsum based on recent x-ray diffraction results (Taylor 2005). Surface exposures of Yazoo are weathered to a maximum depth of approximately 45 ft. Below ground surface. Landris et al. (2012) weathered Yazoo clay has a distinctive yellow-brown color while unweathered Yazoo clay is blue-gray. The expansive clay soil undergoes shrink-swell behavior due to seasonal moisture variation. Due to the high shrink-swell behavior, the shear strength of the soil reduces to fully soften or residual shear strength, which eventually leads to slope failure. These failures of structures and the embankment slope can be expensive to repair.

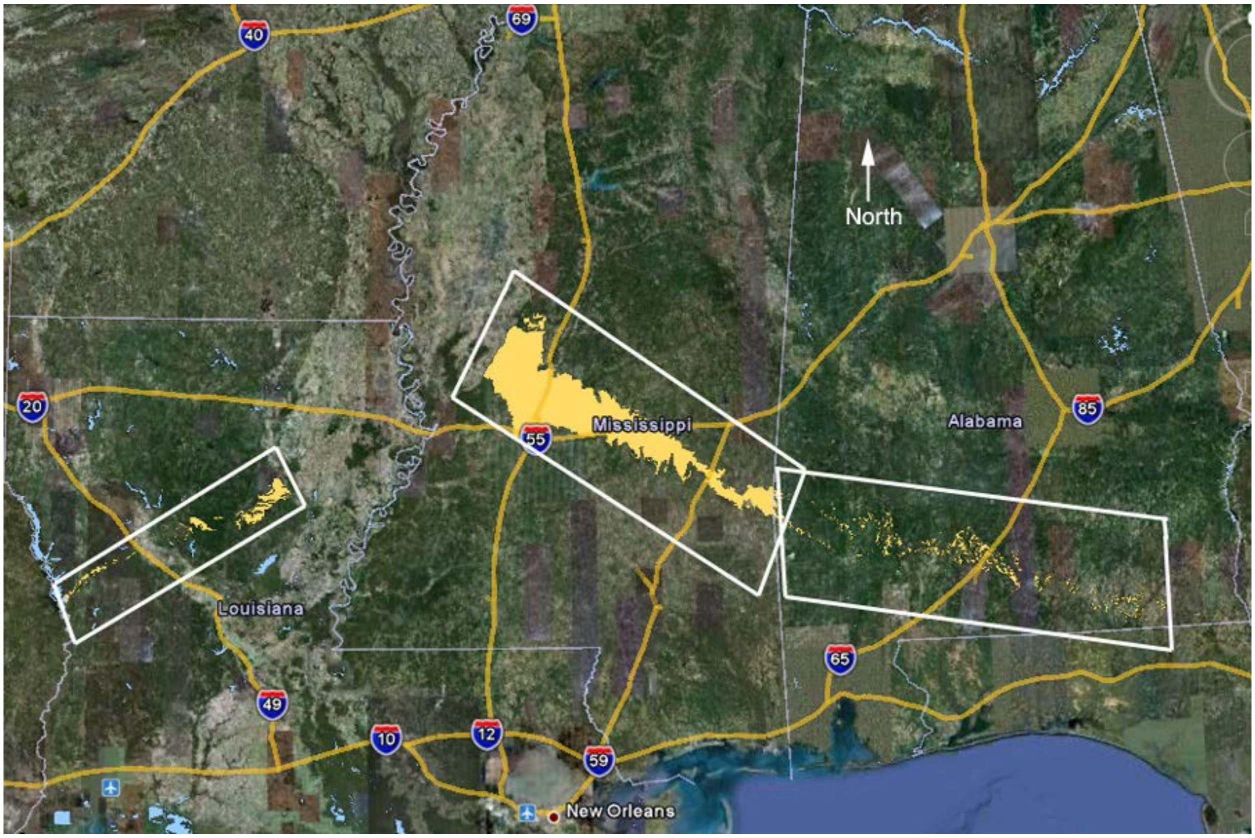


Figure 2.7 Boundary boxes of the Jackson Formation, including Yazoo clay and its geological equivalents, in Mississippi, Alabama, and Louisiana (after USGS 2010)

For compacted clay embankments, the main reason behind the progressive change in shear strength is as a result of weathering which will decrease the drained peak shear strength of compacted clays towards the fully softened shear strength to the residual strength. The concept of fully softened shear strength (FSS) was first proposed by Skempton and Henkel (1964) after investigating the cause of slope failures in cuts within London Brown clays (a stiff, fissured clay). From Skempton and Henkel's research, it was concluded that fully softened shear (critical) strengths should be used when analyzing slope stability for first-time slides in overconsolidated clays. In 1937 Taylor acknowledged that once a soil reached its peak strength, the resistance often fell to a lower value upon further shear deformation. This idea was reiterated in 1964 when Skempton defined this lower value of strength as the residual strength condition. Skempton (1964) concluded that the residual strength applies to slope stability analyses of natural slopes and excavations in stiff fissured over-consolidated clays as well as slopes in these materials that had experienced previous failures. In 1970 Skempton recognized there was a fully softened shear strength, which lies between the peak strength and residual strength. He concluded that the fully softened shear strength is numerically equal to the peak strength of the soil in its normally consolidated state. Initially, the fully softened shear strength was considered to apply primarily to slope failures in stiff fissured over-consolidated natural clay and shale deposits. However, subsequent research suggested that repeated wetting and drying can also reduce the strength of compacted high plasticity clays and shales to the normally consolidated, or fully softened state.

Following Skempton (1964, 1985), drained residual shear strength measured by laboratory tests has been successfully used for stability analyses of reactivated landslides (e.g., Skempton and Petley 1967; Hutchinson 1969; James 1970; Palladino and Peck 1972; Morgenstern 1977; Terzaghi et al. 1996). Mesri and Shahien (2003) have summarized laboratory and field experience to show that drained residual shear strength from laboratory tests is mobilized on the entire slip surface of reactivated landslides and the nearly horizontal lithological and structural discontinuity segment of first-time slope failures. Stability analyses by Huvaj-Sarihan (2009) for additional reactivated landslides support these conclusions. Though determination of soil strength parameters for shallow slope stability analysis is the most important task as the factor of safety will be significantly reduced (Rogers and Wright 1986). This research work tends to determine the progressive change in the mechanical properties of high plastic Yazoo clay which is very important for shallow slope stability analysis. Changes in basic soil properties due to weathering can provide important insight into the change in soil strength parameters. To simulate the loss of shear strength shallow slope failure condition, direct shear tests in the laboratory were conducted for peak, fully soften and residual conditions at low normal stresses (< 100kPa).

Expansive Yazoo clay soil is highly susceptible to the climate change, and it is dominant in central Mississippi and neighboring states. Yazoo clay undergoes volume change due to wetting or drying under different seasonal variation. These repeated volume changes can give rise to ground movements which may result in structural damages, resulting in the high cost of repair or reconstruction. During the wetting period, the highly plastic expansive clay soil absorbs water, and it swells. On the other hand, the soil shrinks during the drying period. However, there is limited research on the effect of different wet and dry cycle on the void ratio of expansive soils.

2.4.1 *Engineering Aspects of Yazoo Clay*

2.4.1.1 Weathered versus Unweathered Clay

Local geologists and engineers describe Yazoo clay as being either “unweathered” or “weathered.” Unweathered clay has a visually distinct blue color that grades into a gray-blue and gray, or it may have a green to grayish green color. Silt having a light gray color occurs locally in thin seams and lamina. Cycles of exposure to air, wetting, and drying tend to cause oxidation and acceleration of clay weathering.

Exposure to drying is accompanied by shrinkage and weathering causing mineralogical changes which in turn change the structural and strength characteristics of clay. Many types of clay lose their stability due to drying and tend to "slake" during rewetting (Mitchell, 1993). When air-dried Yazoo clay is wetted, it quickly slakes but is affected very little by water if at its natural water content. Detrimental swelling can be expected when Yazoo clay is allowed to dry below the optimum gravimetric water content (~15%) and is then wetted (Redus, 1962). As shown later in this report, Yazoo clay can appreciably swell when inundated from its natural water content state. Boston "blue clay" has a softer consistency, but its upper (assumedly weathered) component is a layer of hard yellow clay (Mitchell 1993). Yazoo "blue clay" is unweathered but is typically overlain with visually distinguishable (assumedly weathered) hard yellow clay. Yazoo clay exhibits weathering effects similar to other high-plasticity clays, in that drying (desiccation) generally increases strength, decreases compressibility, and increases swell potential. Yazoo clay is remarkably similar to another argillaceous sedimentary expansive soil, London Clay, assumed to originate during the same Eocene era (De Freitas and Mannion 2007). Its weathered upper consistency is soft to firm, with ochre staining due to oxidation of iron compounds. The upper 4 ft (1.21m) or so is the active zone. The lower-depth unweathered clay is blue-gray, firm to very stiff, and highly fissured. London clay is problematic as a shrink-swell material (Kovacevic et al. 2007; Hight et al. 2007; Jones and Terrington 2011), which is also similar to Yazoo clay. The unweathered Yazoo clay has structural breaks with slickenside (joints and fissures) features. These slickenside breaks are probably due to unloading after pre-consolidation or from shrinkage cracking during drying. Fissures have been found in normally consolidated clays at water contents well above their shrinkage limit (Lee, 2012).

The weathered Yazoo clay is generally found in a zone between the ground surface and the deeper unweathered clay. It has a visually-distinct color ranging from a limonite-stained orange to yellow. Near the surface, its consistency is usually soft and gummy, but it becomes firmer with depth. At the surface, caliche and gypsum crystals are common weathering features, and the clay may or may not be calcareous. At or near the surface most bedding features and fossils weather and become unrecognizable, but with depth, these features become gradually distinguishable. Near the surface, the fractured nature of the soft clay allows mixing with surface material, which can include loess silt, alluvial sands, and gravel. Thus, the near-surface weathered zone can have significantly altered the structural composition. Weathered Yazoo clay is marked by numerous fractures. These fractures allow water to penetrate the otherwise low-permeability clay and enhance weathering at depth (Lee 2012).

Martin's (2007) SEM study examined non-clay components in eight samples. Highly fractured Yazoo clay has surface coatings and vein fillings of secondary calcite, gypsum, manganese oxides, and iron oxides. Bedding planes may contain sand and silt seams or fossil layers. The SEM study observed these features. The Yazoo clay surface generally followed the contour of the ground surface. There was more elevation change in the NW-SE direction than in the N-S direction, and this elevation difference might be a primary indicator of Yazoo clay spatial variability. Figure 2.8 indicates that the weathered clay generally lies above the unweathered clay. Both were documented at just about any depth below the ground surface, and unweathered clay was found above the weathered clay (Lee, 2012).

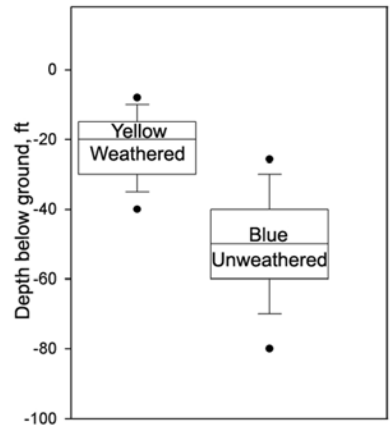


Figure 2.8 Box plots showing a range of depths for visually-classified samples (Lee (2012))

2.4.1.2 Geotechnical Index Properties

Lee (2012) performed a state study on the properties and characteristics of Yazoo clay soil in Mississippi. During the study, Yazoo clay soil samples from different locations were investigated and presented an average index property value. Table 2.4 lists the mean values for all the Yazoo clay soil data visually separated by sample color from that study. The ‘weathered’ samples were yellowish, and the ‘unweathered’ samples had a blue color. Weathered clay was visually identified in samples from the surface to 40 ft (12.19m) depths. Visually-identified unweathered clay was sampled and tested between depths of 25 ft and 80 ft below ground surface (Lee, 2012).

Lee (2012) also analyzed the correlations between geotechnical properties and available mineralogy data. The study indicated that the mineralogy of Yazoo clay includes quartz, clay, calcite, smectite, illite, and kaolinite content percentages. There was little correlation between sample depth (or elevation above mean sea level as shown in Figure 2.9) and regional VC. There also appeared to be little correlation between regional VC and visual color identification of weathering as a function of depth (or elevation) (Lee (2012)). Using visual color identification (yellow or blue) as the primary method to discriminate between weathered and unweathered clay may not be a reliable indicator for regional VC. Regression analysis, performed by Lee (2012), indicated almost no correlation between the averaged VC values as a function of depth. Averaged VC values did exhibit an observable pattern when grouped by depth intervals (Lee (2012)).

Table 2.4 Yazoo clay average index property values (Lee (2012))

Parameter	Weathered (yellow)		Unweathered (blue)		All	
	Mean	Stan Dev	Mean	Stan Dev	Mean	Stan Dev
ɣ dry, lbs/cu ft	82	9	82	9	82	9
ɣ wet, lbs/cu ft	112	10	114	9	113	10
Moisture Content %	38	9	39	9	39	9
Field Void Ratio	0.99	0.21	1.03	0.22	1.02	0.22
LL %	94	19	95	16	94	17
Pl %	35	8	37	8	36	8
PI %	59	16	58	13	59	14
VC %	140	39	138	38	138	39
*Clay %	53	21	65	14	60	18
* Calcite %	13	16	18	14	16	15
*Smectite %	45	18	48	13	46	15
*Illite %	16	17	11	10	13	14
*Kaolinite %	39	11	42	8	41	10

*XRD data

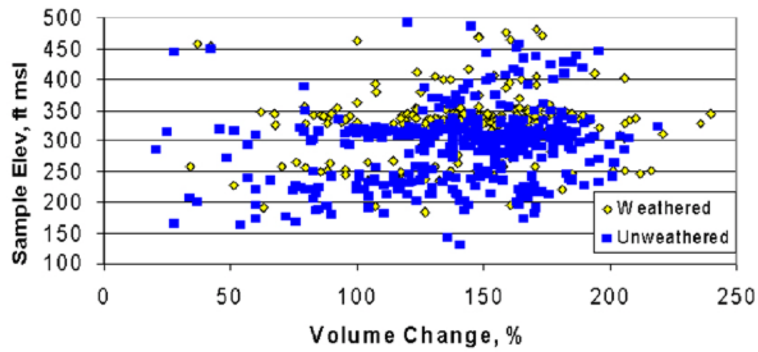


Figure 2.9 Volume change percent (VC%) values for all Yazoo clay data in the 5-county area of central Mississippi, plotted by elevation above mean sea level (MSL) (Lee (2012))

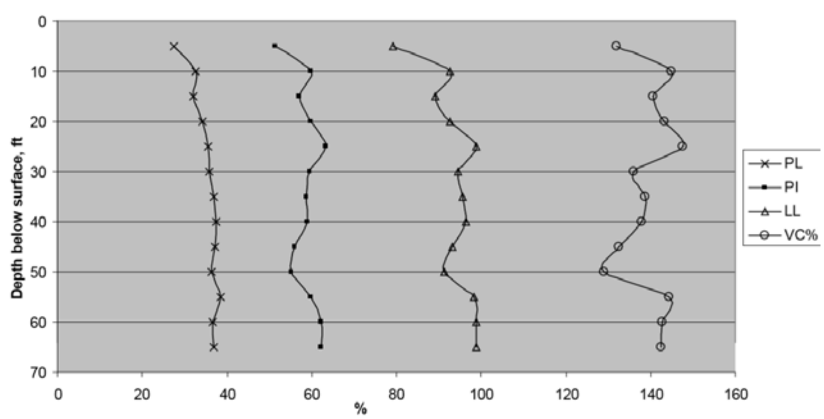


Figure 2.10 Regional weathered plus un-weathered Yazoo clay VC % and Atterberg limit values, averaged by 5-ft (1.524m) depth intervals (Lee (2012))

Based on the study performed and summarized by Lee (2012), the above Figure 2.10 shows that the PL values did not change very much by depth, but the LL (and thus the PI) values varied in concert with the VC% values. Although these data are regional, the following trends were noted:

- Average VC% and LL values were lowest above -10 ft (3.048m) and around 50 ft (15.24m).
- Average VC% and LL values were highest around -10 ft (3.048m), -25 ft (7.62m), and 55 ft (16.76m).
- These regional data indicated non-uniformity of Atterberg limits and expansive behavior patterns with depth.

Lee (2012) indicated that the only significant geotechnical index property correlation was between dry density and natural water content (correlation coefficient R= 0.94). Sample weathering discrimination was irrelevant for this correlation. The high correlation was noted regardless of the degree of weathering. The best-fit non-linear regression equation (Figure 2.11) was:

$$Dry_{density}, pcf = 142.2e^{-0.0143w\%}$$

Where e=natural log base=2.178 and w%=water content percent.

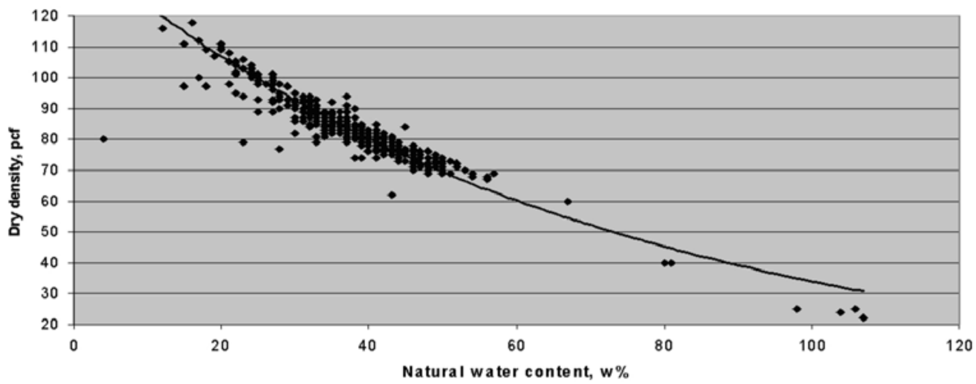


Figure 2.11 Dry density versus natural water content for all Yazoo clay data in the 5-county area of central Mississippi (Lee (2012))

2.4.1.3 Synopsis of Regional Observations

Lee (2012) has performed rigorous analyses on the Yazoo clay soil samples and develop several correlations between different soil parameters. According to the study of Lee (2012), the following correlations observed for weathered, unweathered, and visually non-discriminated Yazoo clay samples.

Strong relationships existed between average natural water content, LL, PL, and dry density. For example, knowing the natural water content averaged over any 5-ft (1.524m) depth interval for a Yazoo clay sample retrieved from less than 45 ft (13.72m) below ground surface enabled estimates such as:

$$LL\% = 15.17(w\%)^{0.49} \ \ (R = 0.90) \tag{2-f}$$

$$PL\% = 1.36(w\%)^{0.89} \ \ (R = 0.91) \tag{2-g}$$

$$Drydensity, pcf = 296.6(w\%)^{-0.35} \ \ (R = 0.95) \tag{2-h}$$

$$Wetdensity, pcf = 188.1(w\%)^{-0.14} \ \ (R = 0.80) \tag{2-i}$$

Lee (2012) indicated that for non-discriminated samples (i.e., hose not separated by visual degree of-weathering), averaged PL and clay content percent were strongly related to depth. They were also strongly related to averaged smectite, illite, and kaolinite percentages. For example, knowing the Yazoo clay sample depth (less than 45 ft (13.72m) below ground surface) enabled estimates of interval averaged values such as:

$$PL\% = 24.63(Depth, ft)^{0.11} \quad (R = 0.94) \tag{2-j}$$

$$Clay\% = -33.5 + 0.31(Depth, ft) + 2.33(PL\%) \quad (R = 0.97) \tag{2-k}$$

$$Smectite\% = -21.94 - 0.21(Depth, ft) + 2.18(PL\%) \quad (R = 0.91) \tag{2-l}$$

Lee (2012) indicated that VC was poorly related to any of the index or mineralogy properties. The regional data yielded poor correlations for Yazoo clay behavior (i.e. volume change percent) to geotechnical index or mineralogical properties, with the one exception being the dry density-natural water content relationship previously shown. The regional data yielded poor correlations between geotechnical index and mineralogical properties related to a depth below the ground surface unless those the values were depth-averaged in 5-ft (1.524m) intervals. Poor regional correlations to an elevation above MSL were also noted (Lee, 2012).

2.4.1.4 Useful Equations Site-specific data

Lee (2012) has performed rigorous analyses on the Yazoo clay soil samples and develop several correlations between different soil parameters. The following useful equations were derived from the lab data from the study site:

- If the natural water content % (in the range 25% to 50%) is known,

$$Dry\ density, pcf = 137.19e^{-0.012w\%} \ , e = 2.718 (R^2 = 0.85) \tag{2-m}$$

$$Free\ Swell\ \% = 0.9212w\%^{1.2737}\ for (25\% < w\% < 50\%); (R^2 = 0.90) \tag{2-n}$$

$$Iss\ 144 = 0.251w\% - 4 \quad (R^2 = 0.83) \tag{2-o}$$

Average total suction below the active zone depth as measured by the iButton,

$$\log psf = -0.079w\% + 6.47$$

- If the average total suction below the active zone depth is known,

$$\text{Potential combined shrink} - \text{swell vertical movement (strain) \%} = 51(\log \text{suction}, \text{psf}) - 222$$

- If the dry density γd, pcf is known,

$$Free\ Swell\ \% = 1636.4e^{-0.034\gamma d}\ (R^2 = 0.88) \tag{2-p}$$

- If the Free swell % is known,

$$LL\% = 6.6168FS\%^{0.5741} \quad (R^2 = 0.90)\ for\ (70\% < FS\% < 140\%) \tag{2-q}$$

If LL is known,

$$VC\ \% = 0.5665LL^{1.2368}\ (R^2 = 0.88) for\ (80 < LL < 120) \tag{2-r}$$

$$Free\ Swell\ \% = 0.08LL^{1.5754}\ (R^2 = 0.90) \tag{2-s}$$

$$w24hr\% = 2.5559LL^{0.8571} \quad (R^2 = 0.92) \tag{2-t}$$

- If the water content after 24 hours (w24hr %) is known,

$$LL = 0.5344w24hr\%^{1.0708} \ (R^2 = 0.92) \ for \ (100\% < w24hr\% < 150\%) \tag{2-u}$$

2.5 Slope Failure and Stabilization Methods

2.5.1 *Slope Failure*

Soil Slope failures are common occurrences globally. Usually, failures occur after prolonged rainfall events which lead to the reduction of soil strength (Titi and Helwany, 2007). Slopes failures can take place gradually or in so, e cases drastic failure can happen without any warning. Slopes are generally characterized as stable when the shear strength of the soil provides enough resisting force against the gravitational forces that are trying to move the soil mass downslope. Therefore, the stability of slope is governed by the balance between the driving and resisting forces. Changes in these forces may lead to the loss of stability and subsequent slope failure. An increase in driving (gravitational) forces can be triggered by changes in slope geometry, seepage pressure, or added surcharge from traffic loads on highway embankments (Titi and Helwany, 2007). On the other hand, reduction in resisting forces can take place due to increased pore water pressure as water perches on impermeable underlying soil layers and decrease in soil shear strength due to saturation of clayey soil after the prolonged event.

Slopes fail when the soil mass between the slope surface and slip surface moves toward downslope. According to Titi and Helwany, (2007), the soil movement and depth of slip surface depends on the type of soil, soil stratification, slope geometry, and presence of water. Abramson et al. (2002) described typical slides that can occur in clay soils, such as (1) translational, (2) plane or wedge surface, (3) circular, (4) noncircular, and (5) a combination of these types. The different slope failure types are illustrated in Figure 2.12.

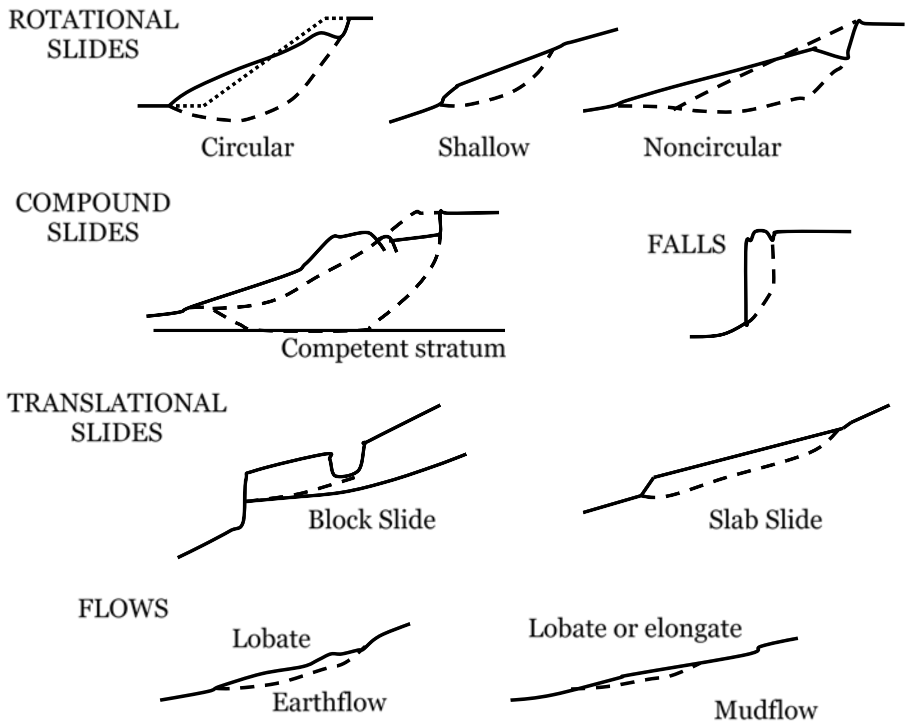


Figure 2.12 Types of Clay movement (Hossain et al. (2017))

Design of stable slope requires a rational selection and use of a factor of safety that accounts for the various uncertainties associated with the determination of soil strength, distribution of pore pressures, and soil stratification. It is suggested to consider a high factor of safety of slope when the level of soil investigation is of low quality, and the experience of the engineer is limited (Abramson et al., 2002).

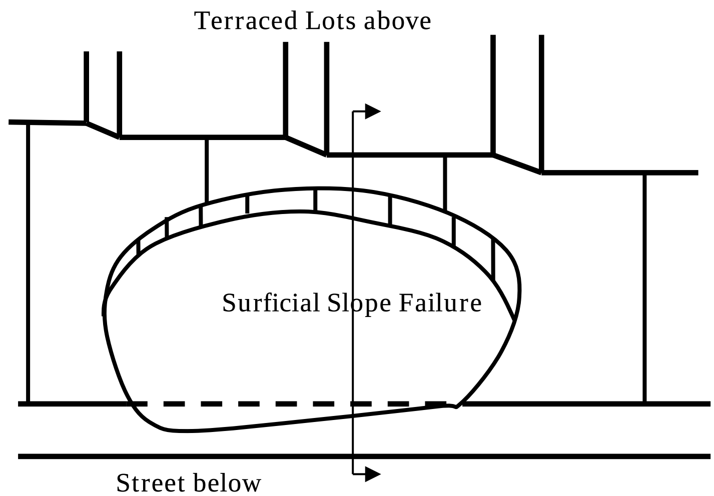
The factor of safety of slope is calculated by comparing the available shear strength along a potential slipping plane with the equilibrium shear stress that is needed to maintain a just-stable slope. The factor of safety is assumed to be constant along the slip surface and can be defined in terms of stresses (total and effective), forces, and moments, selecting a factor of safety for a typical slope design depends on many factors, including the level and accuracy of soil data, the experience of the design engineer and the contractor, level of construction monitoring and consequence of slope failure (risk level) (Titi and Helwany, 2007). For a typical slope design, the required factor of safety ranges between 1.25 and 1.50 (Abramson et al., 2002).

2.5.2 *Shallow Slope Failure*

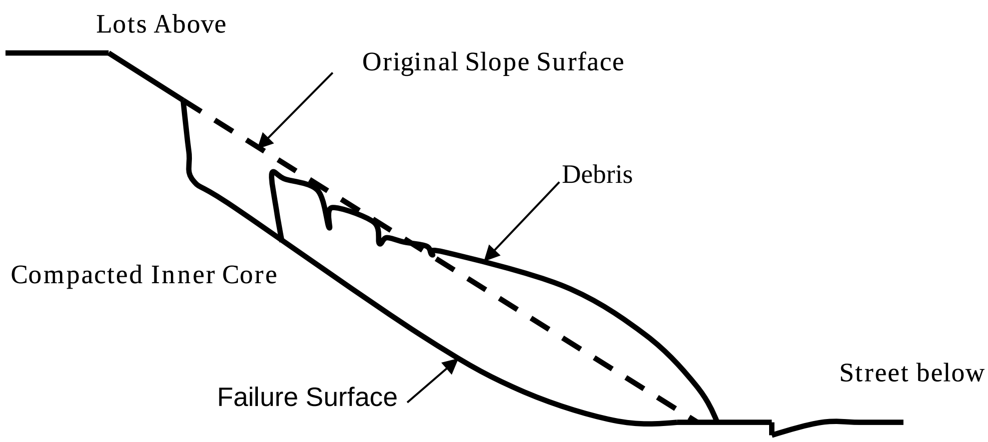
Surficial failures of slopes are quite common throughout the United States. Shallow slope failure refers to surficial slope instabilities along highway cut and fills slopes and embankments. These instabilities commonly occur in fine-grained soils, especially after prolonged rainfalls. The surficial failure by definition is shallow with the failure surface usually at a depth of 1.2 m (4 ft) or less (Day, R. W., 1989). In many cases, the failure surface is parallel to the slope face as illustrated in Figure 2.13.

Shallow slope failures generally do not constitute a hazard on human life or cause major damage. However, it can constitute a hazard to infrastructure by causing damage to guardrails, shoulders, road surface, drainage facilities, utility poles, or the slope landscaping (Khan et al., 2015, Khan et al., 2016, Hossain et al., 2015). In some cases, shallow slope failures can have an impact on regular traffic flow if debris flows onto highway pavements. Moreover, shallow slope failures can have an economic impact on the highway agencies at the local/district level. In general, the repairs of shallow slope failures are conducted at the district and local levels and often performed by maintenance crews as routine maintenance work. In many cases, such repairs may provide a temporary fix of slope failures as the slope failure generally reoccurs after a rainfall season (Titi and Helwany, 2007).

Slopes and embankment constructed on highly plastic clay soil are usually very strong immediately following construction. Skempton (1977) first reported that over time, the strength of slopes in the highly plastic London clay decreases, eventually reaching what Skempton termed a “fully-softened” strength. A few years after construction, shrink-swell behavior can reduce the shear strength of the top few feet of a slope, which is susceptible to moisture variations. After the soil reaches the fully-softened shear strength, water that infiltrates the soil during intense and prolonged rainfall events may cause excess pore water pressure and compound the overall problem. As a result, the factor of safety of the slope may reduce to unity and approach failure (Wright, 2005). These types of failures occur within the upper 0.91 m – 1.82 m (3 ft. - 6 ft.) of the slope and the failure surface is generally parallel to the slope face (Wright, 2005). The surficial failure may occur 3-7 years after construction. Surficial failure can even take decades to form, depending on the frequency of extreme weather conditions.



(a)



(b)

Figure 2.13 Typical surficial slope failures (Day, R. W. 1989, Hossain et al. (2017))

In general, the shallow slope failures vary in depth and extent of the failed area. The depth and extent of shallow slope failures largely depend on slope geometry, soil type, the degree of saturation of soil, seepage and climatic condition (Titi and Helwany, 2007). According to Abramson et al., (2002), many shallow slope failures occur when the rainfall intensity is larger than the soil infiltration rate, and the rainfall lasts long enough to saturate the slope up to a certain depth, which leads to the buildup of pore water pressure at that depth.

Shallow slope failures often are parallel to the slope surface and usually are considered as infinite slope failures. Various depths were reported in the literature based on case histories, but all studies indicated a shallow nature of surficial failures. Evans (1972), cited in Titi and Helwany (2007), defined the failure surface depth of shallow slope to be equal to or less than 4 ft (1.2m). According to Loehr et al. (2000), the depth of shallow slope failure as less than 10 ft. (3.05m), however, in general, it varies in between 3 ft.

(0.914m) to 6 ft. (1.83m). According to Titi and Helwany (2007), the recommended shallow failure depth ranges from approximately 2 (0.61m) to 4 ft (1.2m).

2.5.3 *Variation of Shear Strength of Highly Plastic Clay soil*

Moderate to steep slope constructed on high plasticity clay is susceptible to the softening behavior at the topsoil due to wet-dry cycle. The fully softened shear strength corresponds to the shear strength of high plastic clay seems to develop over time due to the wetting and drying cycle (Wright, 2005). Skempton, A. W. (1977) first proposed the concept of fully softened strength for natural and excavated slopes in London Clays. Skempton (1977) reported that over time the strength of slopes in the highly plastic London Clay lost strength, eventually reaching what Skempton termed as “fully-softened” strength which lies between peak and residual strength as presented in Figure 2.14. Skempton (1977) indicated that the fully-softened strength is comparable to the shear strength of the soil in a normally consolidated state.

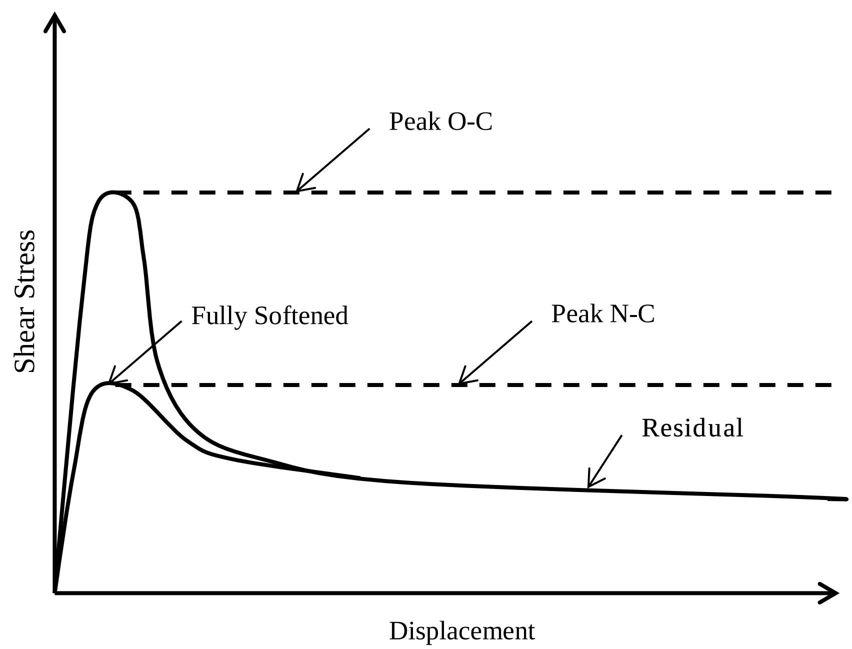


Figure 2.14 Comparisons of peak, residual and fully softened shear strength (Skempton (1970))

Rogers and Wright (1986) conducted a study to investigate the failure of slope constructed over highly plastic clay soil in Texas. The author reported the high plasticity of the clays experienced shrink-swell characteristics which make it probable that the repeated wetting and drying in the field might be one source of the softening. Accordingly, Rogers and Wright (1986) performed direct shear tests on specimens that were subjected to repeated cycles of wetting and drying. These tests were all performed on the clay from the Scott Street and I. H. 610 site in Houston, Texas identified as the red clay. Four series of drained direct shear tests were performed on specimens subjected to 1, 3, 9 and 30 cycles of wetting and drying. Shear strength parameters obtained from the study are summarized below in Table 2.5.

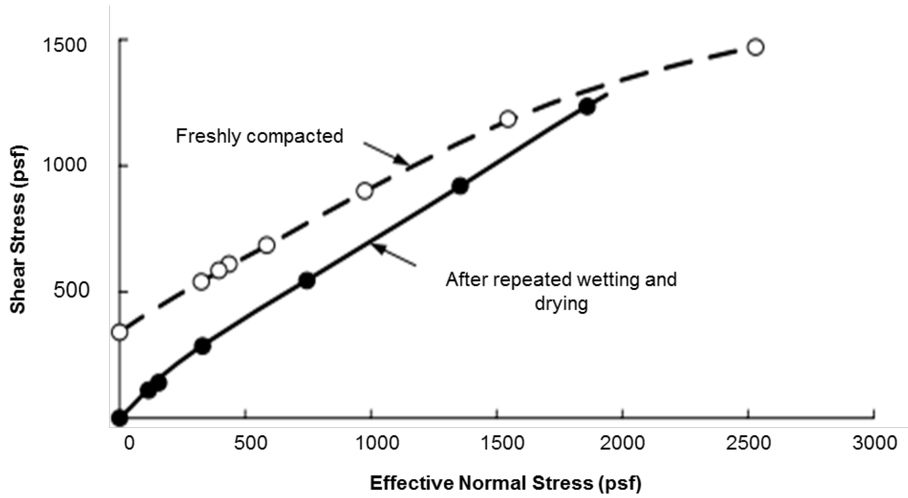
Table 2.5 Summary of Shear Strength Parameters from drained direct shear tests on specimens subjected to wetting and drying cycles (Rogers and Wright (1986))

Number of Wet-Dry Cycles	Cohesion, c (psf)	Friction Angle, Φ
1	29	23˚
3	77	26˚
9	33	25˚
30	0	27˚

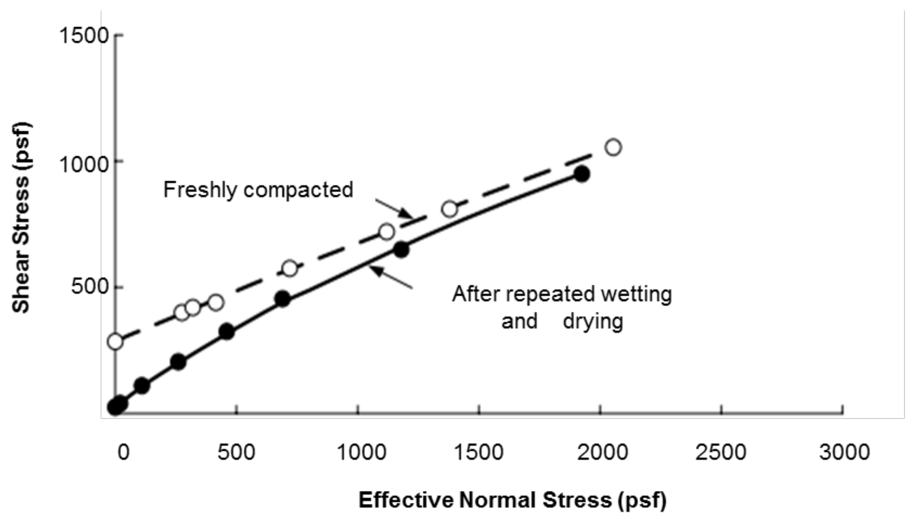
Rogers and Wright (1986) reported that cyclic wetting and drying of the soil produces a significant shear strength loss, particularly regarding effective cohesion intercept, c'. The direct shear test also indicated that the loss in cohesion occurs within a relatively few numbers of cycles of wetting and drying. In fact, most of the loss in strength occurred in the first cycles. However, Rogers and Wright suggested that the wetting and drying that specimen was subjected to in laboratory was much severe compared to that expected to occur during any wetting and drying cycles in the field. Even so, the effects of wetting and drying in laboratory and field are believed to be similar.

Kayyal and Wright (1991) developed a new procedure for triaxial specimens subjected to repeated cycles of wetting and drying. The procedure allowed the specimens greater access to moisture and exposure for drying. Also, the procedure allowed substantial lateral expansion and volume change to occur in the soil during drying. Two soils were tested during the study 1. Red clay or Beaumont clay from Houston, Texas and 2. Highly plastic clay soil from Paris, Texas.

Kayyal and Wright (1991) conducted several series of consolidated undrained compression tests with pore pressure measurement. Tests were performed on specimens subjected to repeated wetting and drying on specimens as well as on freshly compacted samples. Based on the study, the shear strength envelops for specimens of Beaumont clay and Paris clay tested in the as-compacted, and after wetting and, drying is presented in Figure 2.15. The results presented that both envelops were distinctly nonlinear. Also, the strength envelope for the specimens subjected to wetting and drying cycles lied significantly below the envelope for the specimens tested in the as-compacted condition at lower values of normal stress. Moreover, the intercept of the strength envelope for specimens subjected to wetting and drying is small and could be considered negligible.



(a)



(b)

Figure 2.15 Shear strength envelopes regarding effective stress a. Beaumont clay, b. Paris clay (Kayyal and Wright (1991))

Slopes constructed in the arid and semi-arid region remain in the unsaturated condition above the groundwater table. To accurately predict the mechanical behavior of unsaturated soil, two stress state variables are required of which the most widely used, is the combination of net normal stress (σ – up) and matric suction (a – UW). Based on these two stress state variables, Fredlund et al. (1978) proposed the following equation which is an extension of M-C theory to describe the shear strength of unsaturated soil:

$$\tau = c' + (\sigma - u_a)tan\varphi' + (u_a - u_w)tan\varphi^b \quad (2.1)$$

Where,

(σ – up) = net normal stress
(ua – uw) = matric suction
φ′ = angle of internal friction associated with the change in net normal stress
φb = angle representing the rate of change in shear strength relative to matric suction change

The first two terms on the right-hand side in equation (2.1) describes the conventional MC theory to determine the strength of saturated soil. The third term indicates the change in shear strength due to change in matric suction in unsaturated soil. The corresponding failure envelope for the extended M-C criterion is presented in three-dimensional stress space in Figure 2.16.

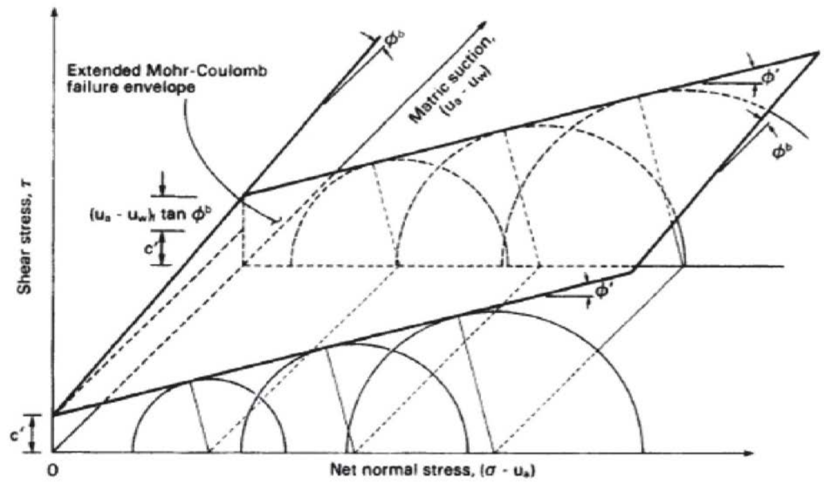
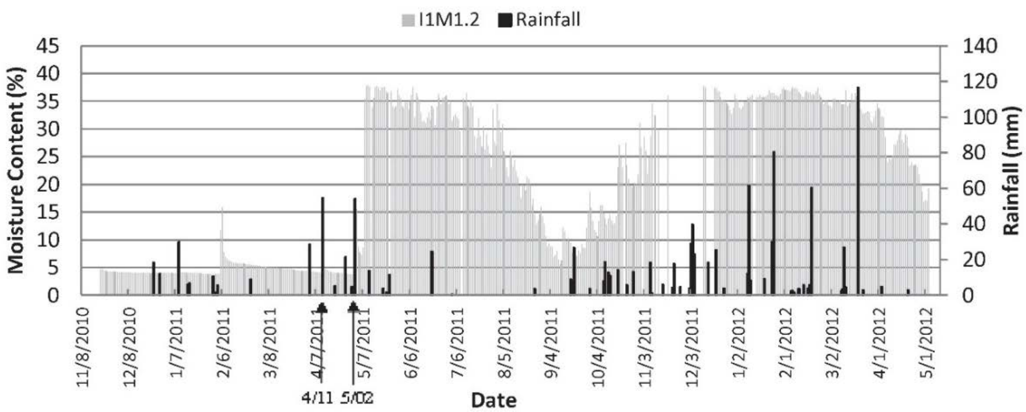


Figure 2.16 Extended Mohr-Coulomb failure envelope for unsaturated soils (Fredlund and Rajardjo (1993))

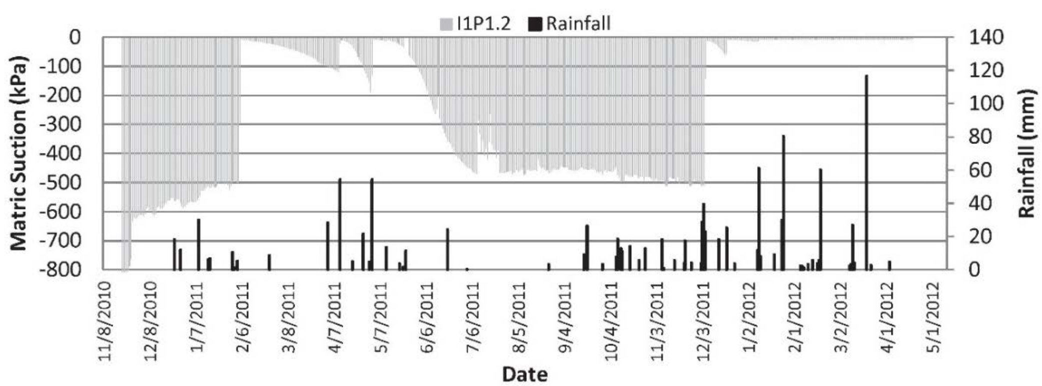
2.5.4 *Effect of Rainfall on Slope Stability*

Moderate to high-intensity rainfall is one of the major cause of the shallow slope failure. Rahardjo et al. (2001) investigated several shallow slope failures on residual soil that occurred after a rainfall event of 95 mm (3.75 in.) within 12 hours. The authors reported that the landslides were initiated by rainwater infiltration; that is, no changes in geometry or additional loading applied to the slopes could have initiated failure. Also, both the daily rainfall and the antecedent rainfall were important triggering factors for the occurrence of the landslides. Rahimi et al. (2010) conducted a study on rainfall-induced slope failure due to antecedent rainfall for high and low conductivity residual soils of Singapore. The authors applied three antecedent rainfall patterns to soil slopes and conducted a transient seepage analysis to investigate the effect of rainfall on the stability of the slope. Results from the study indicated that antecedent rainfall affected the stability of both high-conductivity and low-conductivity soil slopes, with the stability of the low-conductivity soil slopes being more significantly affected. Also, the stability of the slope was controlled by the amount of rainfall that infiltrated the unsaturated zone of the slope.

Hossian (2012) instrumented a highway slope in Dallas, Texas, constructed on highly plastic clay soil. During the study, moisture sensor and potential water probe were installed at the crest of the slope, to measure the moisture variation and associated change in matric suction due to rainfall. The study measures the moisture and suction variation between August 2010 and May 2012. Based on the study, a variation of in situ moisture content and matric suction profile at 1.2 m depth is presented in Figure 2.17 (a) and (b). The initial moisture content was relatively low (4%), and matric suction was high (-600 kPa). However, total precipitation of 181.9 mm was observed in one month between April and May 2011 with the highest precipitation of 54.2 mm during two rainfall events in April and May. As a result, the moisture content increased to a maximum of 37%, and matric suction decreased to -10 kPa. Based on the back-calculation of several slope failure constructed on Paris and Beaumont clay, Aubeny and Lytton (2004) suggested that the wet limit of suction at the surface during a slope failure is 2 pF (i.e., approximately -10 kPa).



(a)



(b)

Figure 2.17 In situ variation of moisture content and matric suction at 1.2 m depth near the crest of a highway slope in Dallas, Texas. (Hossian (2012))

During the relatively dry summer of 2011 in Texas, the moisture content decreased gradually from 37% to 10% and accordingly, matric suction increased from -100 kPa to -450 kPa. Therefore at 1.2 m, a variation of moisture content due to rainfall is evident. Moisture content in the soil started to increase with the rainfall observed between September and December 2011, but the suction in the soil remain unchanged. This may be due to change in soil moisture retention characteristic at the upper part because of cracking during the dry period. Zornberg et al. (2007) also reported a significant change in soil moisture retention characteristics due to the formation of cracks in high plasticity clays. The total rainfall observed between this period was 176 mm with the highest daily observed rainfall of 29 mm. Moisture content at the crest of slope at 1.2 m depth remained closed to 37% between the period of January to March 2012 as there was a high volume of rainfall observed in this period. The matric suction also remained unchanged to a value of -10 kPa due to the large volume of infiltration of rainwater during this period. Hossain (2012) also observed that infiltration of rainwater depends on the initial matric suction of the soil. During the rainfall event on September 2011, a total of 26.4 mm rainfall was observed, but both moisture content and matric suction remain unchanged.

The in situ matric suction before the rainfall was approximate -450 kPa. On the hand, same of amount of rainfall was registered in December 2011, and a drop-in suction was observed from -50 kPa to approximately -10 kPa. The different behavior for the same amount of rainfall may be due to low initial permeability as a result of high initial suction on September 11, 2011.

Khan et al. (2016) investigated the mechanism of failure of the highway slope on highly plastic clay soil in Texas. As a part of the study, authors conducted a flow analysis using finite element method to investigate the effect of rainfall on the variation of saturation and matric suction of a highway slope constructed over highly plastic clay soil in Texas. During the study, the authors investigate the effect of rainfall with three intensities of 0.0012 mm/sec (0.167 in./hour), 0.0019 mm/sec (0.271 in./hour), and 0.0022 mm/sec (0.3125 in./hour). The rainfall intensities were selected based on 2, 5, and 10-year periods of Texas rainfall data. The flow through the topsoil was determined for each of the intensities assuming rainfall durations lasting 3 hours, 6 hours, 12 hours, and 24 hours. In the flow analysis, the left and right boundaries were selected as the closed boundaries, and the top of the slope was selected as rainfall infiltration (Figure 2.18 a, b). A constant head of 6.1 m (20 ft.) was selected for the bottom boundary. The water table was assumed to begin at 15.24 m (50 ft.) below the top of the slope, to define the initial unsaturated condition.

Khan et al. (2016) conducted the flow analysis, considering Rainfall with uniform intensity and utilized the Van Genuchten model to define the flow parameters. The site investigation at the failed slope indicated that the soil has desiccation crack, up to 7 ft. (2.13m) depth during the summer. As a result, a high vertical permeability value of k_y = 1.063 m/day ($1.23*10^{-5}$ m/s) was used for the top 2.13 m (7ft.) of the slope to simulate the effect of the desiccation crack. In other clay layers, the permeability for both horizontal and vertical directions was selected as 0.0475 m/day ($5.5*10^{-7}$ m/s). Hence, the ratio of vertical to horizontal permeability became approximately 22. The Van Genuchten fitting parameters (α = 0.064, n = 1.219 and m = 0.1797), were utilized for the flow analysis. The variations of suction at the crest of the slope for a 12-hour rainfall are presented in Figures 2.18 (c) to 2.18 (f). The suction immediately dropped at the top after rainfall and continued to drop for two days, representing the accumulation of water at the corresponding depth. After 7 days, the suction had almost regained its original profile.

Based on the study conducted by Khan et al. (2016), the variations of change in suction at 0.91 m (3 ft.), 1.82 m (6 ft.), and 2.43 m (8 ft.) depths are presented in Figure 2.19. The change of suction refers to the change from the initial suction value before rainfall. The mentioned figure shows various rainfall durations with a 2-year return period (rainfall intensity of 0.0012 mm/sec [0.167 in./hour]) and a 10-year return period (0.0022 mm/sec [0.3125 in./hour]). The suction was observed to drop significantly with higher intensity and longer duration of rainfall. Moreover, the change in suction was more significant at depths of 0.91 m (3 ft.) and 1.82 m (6 ft.) when compared to the higher depth of 2.43 m (8 ft.).

The drop of suction was instantaneous at a depth of 1.82 m (6 ft.) for different rainfall intensities and durations, as depicted in Figure 2.19 (b) and 2.19 (e). In contrast, the change in suction took several days to weeks post-rainfall to achieve a steady value at 2.4 m (8 ft.) depth. The constant value of suction at 2.43 m (8 ft.) depth indicated that the percolated water could not drain out from the slope due to the very low permeability of the highly plastic clay soil.

The flow pattern through the topsoil was important to the investigation of the failure. The slope failed in October 2011, after a long drought condition followed by a rainfall event (Figure 2.19). The existence of shrinkage cracks allowed for easy rainwater intrusion at the slope crest in the top few meters immediately after rainfall. The low permeability of the highly plastic clay likely prevented downward movement of the water, creating a perched water zone near the crest of the slope. Hence, the slope stability analysis of the slope was extended to consider a perched water zone near the crest of the slope.

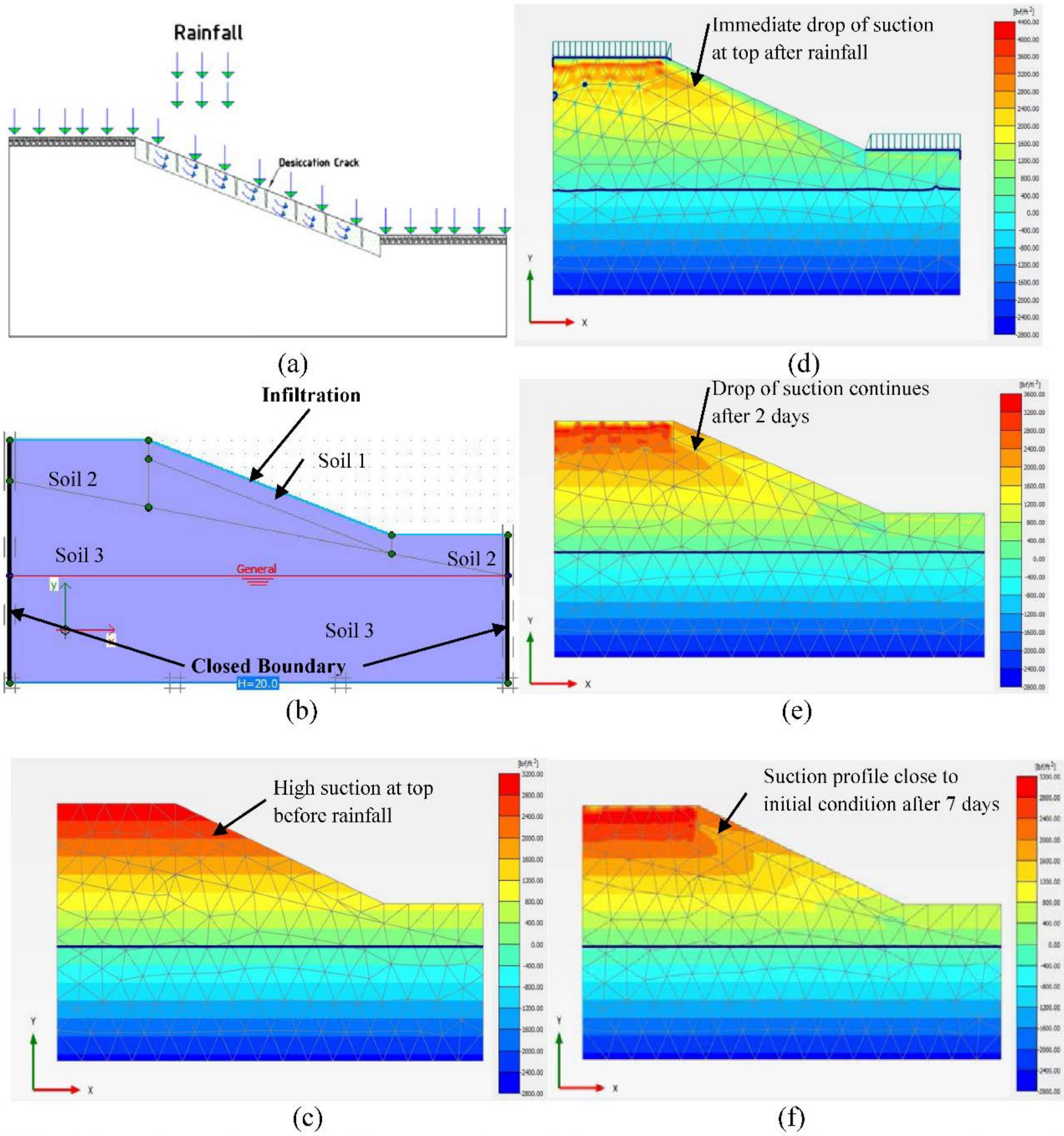


Figure 2.18 Variation of matric suction during moisture intrusion due to rainfall (Khan et al. (2016))

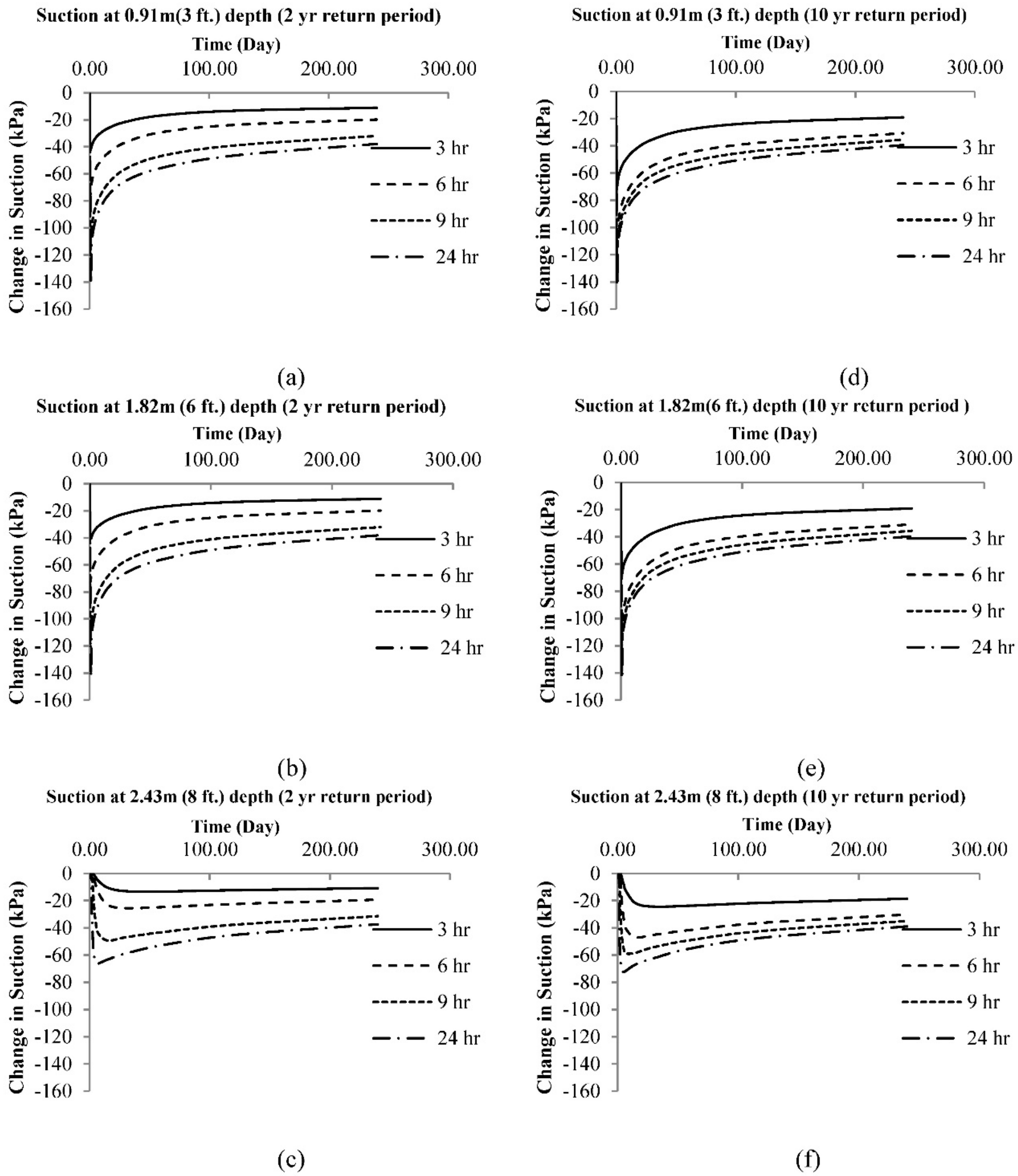
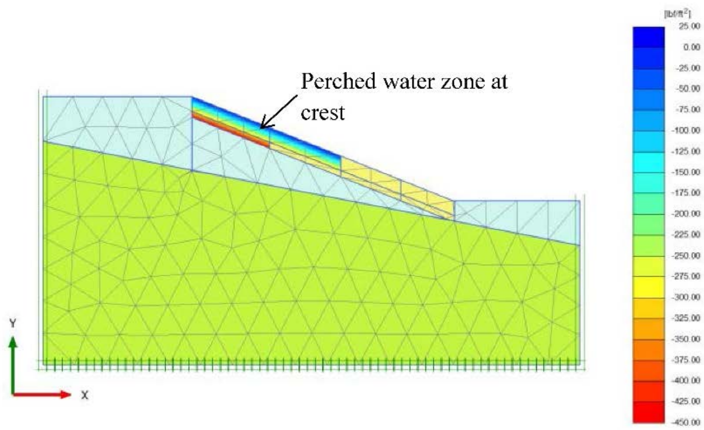


Figure 2.19 Variation of change of suction at the crest of the slope (Khan et al. (2016))

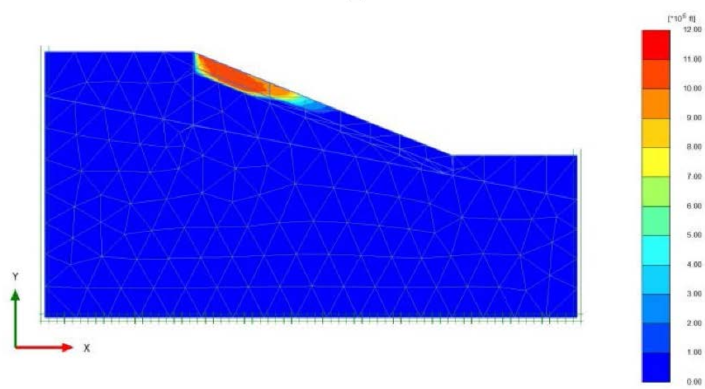
Khan et al. (2016) integrated the flow analysis results, in a failure investigation of a highway slope in Texas, constructed over highly plastic clay soil. It is evident that due to rainfall, water infiltrates and saturate the topsoil to create a perched water condition near the crest. Therefore, a perched water zone was applied in the crest (top 1/3 of the slope face) to evaluate the effect of rainfall on the safety of the slope in the presence of desiccation cracks. During this study, Khan et al. (2016) considered the fully softened shear strength at the topsoil. The soil model with the perched water zone is presented in Figure 2.20. The phi-c reduction analysis was conducted, and results indicated that the factor of safety of the slope reduced to 1.05

when considering the perched water condition. This reduced factor of safety was very close to failure. The failure plane of the slope is presented in Figure 2.20.

Khan et al. (2016) concluded that the slope failure took place due to the combined action of the perched water zone at the crest and the fully-softened condition resulting from seasonal wet-dry cycles. After a prolonged summer, rainwater intrusion via desiccation cracks formed a perched water zone at the topsoil. The combination of the desiccation crack and the perched water zone in the top few feet initiated the shallow slope failure.



(a)



(b)

Figure 2.20 (a) Initial soil model, including the perched water zone, (b) Slope stability analysis with fully-softened strength and perched water zone at the crest, FS = 1.05 for the perched water zone at crest (Khan et al. (2016))

Wright (2005) reviewed different research projects on slope and embankments constructed on highly plastic clay soil in Texas. Wright reported that the compacted highly plastic fills are generally very strong immediately after construction. In general, at the end of construction the factors of safety probably exceed 2, though, the soils tend to soften and weaken over time. As a result, the factor of safety decreases to values that approach 1, i.e., failure. The softening is probably enhanced by the repeated expansion and shrinkage which take place due to seasonal wetting and drying of the soil, respectively. The fully-softened strength of the soil is best characterized by a curved Mohr failure envelope. Also, the failure envelope for fully-softened conditions lies below the failure envelope for the soil immediately after compaction.

2.6 Impact of Rainfall

Most highway fill slopes in the Mississippi areas are constructed using in-situ high plasticity clay soil, which is highly expansive. These fill slopes have recurring shallow failures a few years after construction, causing a significant maintenance problem for the Mississippi Department of Transportation (MDOT). Shallow slope failures generally do not constitute a hazard to human life or cause significant damage; however, the highly plastic clay soil developed desiccation cracks. This may have significantly increased the permeability along the vertical direction of topsoil at the active zone. (Khan et al. (2017)). In some cases, infiltration of rainwater results in a reduction in soil shear strength and matric suction, which leads to a reduction in slope stability. The slope stability is reduced by both rainfall intensity and duration. (Hossain et al., 2013). Prolong drought creates significant shrinkage cracks, which provides a vertical preferential path during rainfall. The suction was observed to decrease significantly with higher intensity and longer duration of rainfall. (Khan et al. (2017)). In a case study, Yalcin (2007) showed that the rainfall also increases the water content in clays that leads to a reduction in the stability of natural slopes. As a result, the rainfall causes a decrease in shear strength by either reducing soil cohesion or through potential slip surfaces, and then this directly relates to excessive rainfall events.

Rahimi et al. (2010) conducted a study on rainfall-induced slope failure due to antecedent rainfall for high and low conductivity residual soils of Singapore. The authors applied three antecedent rainfall patterns to soil slopes and conducted a transient seepage analysis to investigate the effect of rainfall on the stability of the slope. Results from the study indicated that antecedent rainfall affected the stability of both high-conductivity and low-conductivity soil slopes, with the stability of the low-conductivity soil slopes being more significantly affected. Also, the stability of the slope was controlled by the amount of rainfall that infiltrated the unsaturated zone of the slope. Slopes constructed on highly plastic clay soil are usually very strong immediately the following construction.

Chapter 3: LABORATORY TESTING OF YAZOO CLAY SOIL

3.1 Introduction

The laboratory testing program was designed to determine properties relating to volume change behaviors of expansive Yazoo clay soils. A Highway slope located on in Jackson Mississippi has been selected for this study. Representative Yazoo clays soil samples collected from Geotechnical Engineering Company Burns Cooley and Dennis in Ridgeland; MS were collected and are investigated at the laboratory. The experimental program was mainly comprised of tests to determine basic soil properties, such as sieve analysis, hydrometer tests and Atterberg limits test, and engineering characteristics, including directional test and total soil suction measurement test. A summary of the laboratory procedures, equipment used, and results obtained are presented in the following sections. All representative soil samples were subjected to various physical property measurements.

3.2 Atterberg Limits Test

It has been found that the water contents corresponding to the transitions from one state to another usually differ for clays having different physical properties in the remolded state and are approximately equal for clays having similar physical properties. Therefore, the limiting water contents, or limits, may serve as index properties useful in the classification of clays. As the soil-water mixture passes from one state to another, there is no abrupt change in the physical properties. The Atterberg limit tests, therefore, are arbitrary tests that have been adopted to define the limiting values. The Atterberg limits vary with the amount of clay present, the type of clay mineral, and the nature of the ions adsorbed on the clay surface. Unlike finer soil particles, gravels and sands do not possess the required cohesiveness which permits the Atterberg limits tests to be performed. However, the finer sands and silts often contain sufficient clay coatings to allow the tests to be completed.

Atterberg tests are performed on only that soil fraction which passes through a No. 40 sieve (0.425 mm). Consistency limits (LL and PL) are significant to understand the stress history and general properties of the soil met with construction. An estimate of Plasticity Index is necessary to classify the soils, particularly in highly expansive clays. The Atterberg Limit test was performed in the Geotechnical Engineering Laboratory at Jackson State University. The test was conducted according to ASTM D4318. The moisture content of the soil samples for the liquid limit test is presented in Figure 3.1.

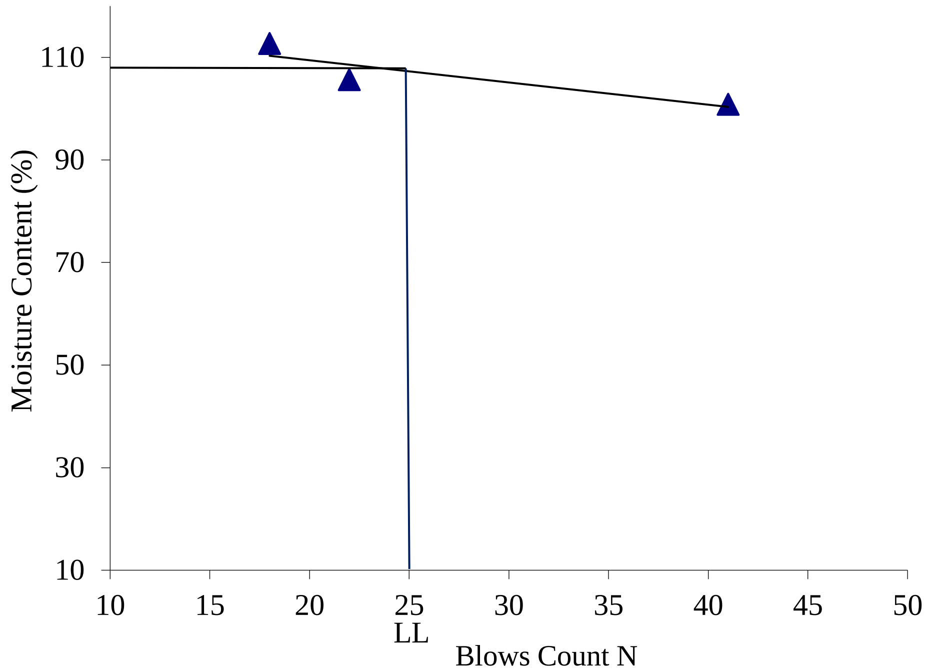


Figure 3.1 Graph of Liquid Limit Test

From the chart, Liquid Limit equals 108% at 25 number of blows. It is observed that the highest percentage of moisture content is 112.7 which is dependent on the number of blows N. It can be inferred that the higher the number of blows N, the lower the moisture content will be. This dependency is seen in the resulting graph, a linear graph (Figure 3.1). From the graph, the moisture content at which Yazoo clay soil will tend to behave like a plastic material is determined at 25 number of blow count as 108, which is the Liquid Limit (LL). The plasticity index of 84 was calculated, which is high and provides a clue that the sample is likely to be clay. In the grain-size classification, soils are designated according to the grain-size or particle-size. Terms such as gravel, sand, silt, and clay are used to indicate specific ranges of grain-sizes. Modern engineering classification systems are designed to allow an easy transition from field observations to basic predictions of soil engineering properties and behaviors. The soil is classified using ASHTOO classification system chart as inorganic clay of high plasticity (CH).

3.3 Sieve Analysis Test

Most granular soils and fine aggregates are mixtures of desirable coarse particles, sand, and undesirable clay or plastic fines. The sieve analysis test (Figure 3.2) which can also be referred as grain size test or sand equivalent test which is intended as a rapid field correlation test to indicate the relative proportions of clay-like or plastic fines and dust in granular soils and fine aggregates that pass the No. 4 (4.75 mm) sieve size. The test assigns an empirical value to the relative amount, fineness, and character of clay-like material present in a soil sample. At first, the sieve analysis test was performed according to ASTM C136. Later the retained soil over #200 U.S. sieve (75µm sieve opening) was washed according to ASTM D1140 (wash sieving) test method. The Yazoo clay soil has 96.8% passing No. 200 sieve. Therefore, hydrometer test is conducted to investigate the grain size distribution of fine content.

Figure 3.2 Sieve analysis at the Geotechnical Engineering Laboratory

3.4 Hydrometer Test

The hydrometer analysis, also called sedimentation method because it is based on the principle of sedimentation of soils in water. It is used to determine the grain size distribution for a fraction of the soil that is smaller than the No. 10 sieve. Fine soil particles are dispersed by soaking the soil sample in a dispersing agent and by rapid stirring to neutralize the charges between the soil particles. The test (Figure 3.3) uses a Type 152H calibrated hydrometer to give the mass of solids with a specific gravity equal to 2.65 in suspension and the settling velocity of the dispersed soil particles. Hydrometer test is used to determine what type of clay is predominant in a given soil sample (e.g., kaolinite, illite, montmorillonite, etc.). The hydrometer test was performed according to ASTM D7928-17 test method. The combined particle size distribution curve of the Yazoo clay soil is presented in Figure 3.4.

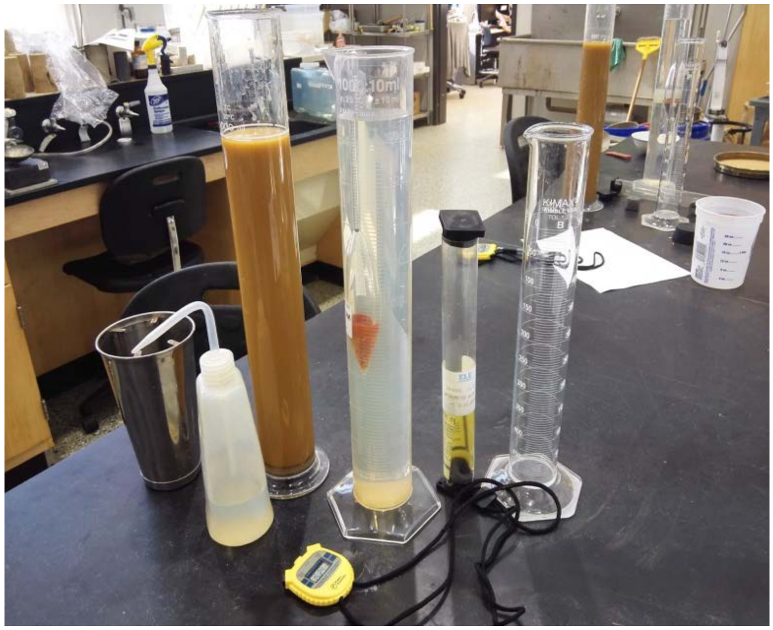


Figure 3.3 Hydrometer Test

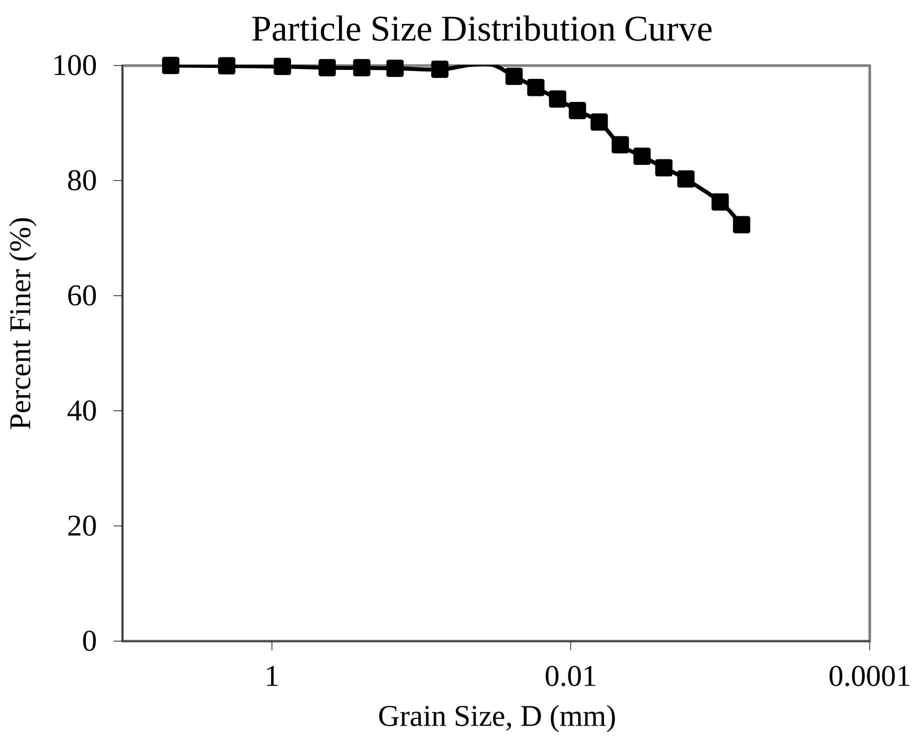


Figure 3.4 Combined Particle Size Distribution Curve of Yazoo Clay soil

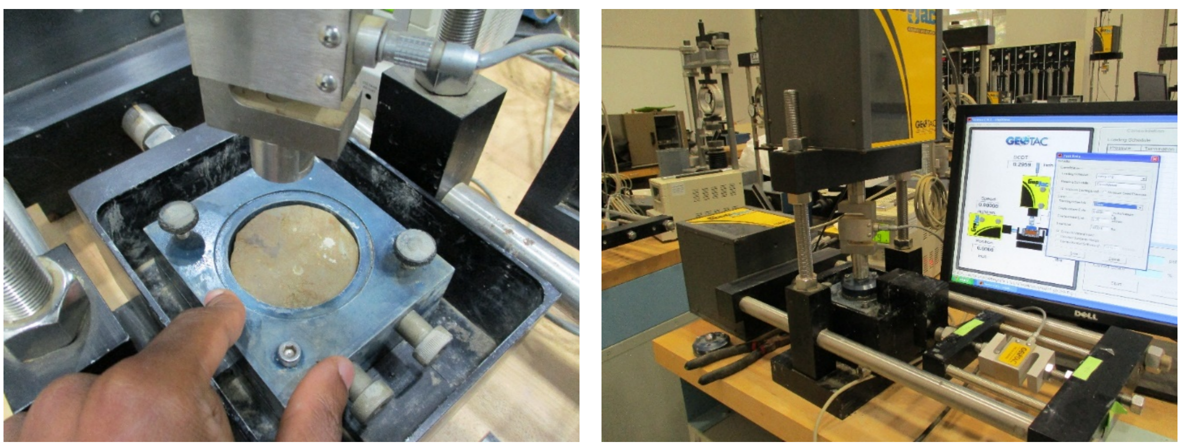
In Figure 3.4, the particle size distribution curve shows values ranging from 0.068mm to 0.00095mm. It can be inferred from the hydrometer readings, that the highest particle settlement occurred between 2 – 4mins which gave a hydrometer reading of 46 and 31 respectively. This resulted in a steep slope noticed in the graph with a corresponding effective length difference of 2.4 mm. Also, it can be induced that the soil particles settled to the bottom of the 1000 ml cylinder in an ascending (i.e., particles with smaller diameter settles first leaving those with a bigger diameter on top) order of its size particles. It is reflected in the graph, showing a steady increase in slope up to the point the big effective length difference was experienced.

Table 3.1 Soil Properties

Physical Properties	Values
Unified Classification	CH
Liquid Limit	108%
Plasticity Index	84%
Unit weight	21kN/m^3
Specific Gravity	2.68

3.5 Peak drained direct shear test

This test method covers the determination of the consolidated drained shear strength of cohesive Yazoo clay soil in direct shear. The direct shear test is performed following the ASTM D3080 standard by deforming a specimen at a controlled strain rate on or near a single shear plane determined by the configuration of the Geojac direct shear apparatus. Some test photos are presented in Figure 3.5. Generally, three or more specimens are tested, each under a different normal load, to determine the effects of shear resistance and displacement, and strength properties such as Mohr-Coulomb strength envelopes. Normal loads of 25 KPa, 50 KPa, and 100 KPa is applied to three different Yazoo clay soil samples of 2.5-inch diameter and 1 inch in height.



(a) (b)

Figure 3.5 Drained direct shear test Procedure (a) Sample (b) Testing

3.5.1 *Sample Preparation*

Undisturbed specimens from Shelby tube samples were used for the test. The soil sample is extruded from the sampling tube. The extruded sample was trimmed to a height of 38 mm to fit in the shear box of 63.5 mm diameter to fit into the shear box. The shear box is then assembled with the top and the bottom halves of the box screwed (or otherwise rigidly attached) together. The inside of the shear box is typically lightly greased to minimize side friction. The lower porous stone is placed in the shear box. The trimming ring is then carefully aligned with the top of the shear box. The sample is then slowly extruded into the shear box by pressing on its top surface, typically using the top porous stone or a suitable disk. The upper porous stone and loading cap are placed in the shear box, and the system to apply the normal load is brought into place, and a small normal load (seating load) is applied.

3.5.2 *Direct Shear Test Results*

The Peak drained friction angles for Yazoo clay soil is estimated using Figure 3.6 & 3.7. The results of the three series of direct shear tests conducted in this study are presented in plots of 25, 50 and 100 kPa shear stresses versus displacement respectively (Figure 3.6).

Based on the direct shear test results, the peak shear strength of the samples with the applied normal stress is plotted to develop the Mohr-Coulomb failure envelope, as presented in Figure 3.6. As presented in the failure envelope, the friction angle ϕ (20.2) and cohesion c (18.76 kPa) is determined.

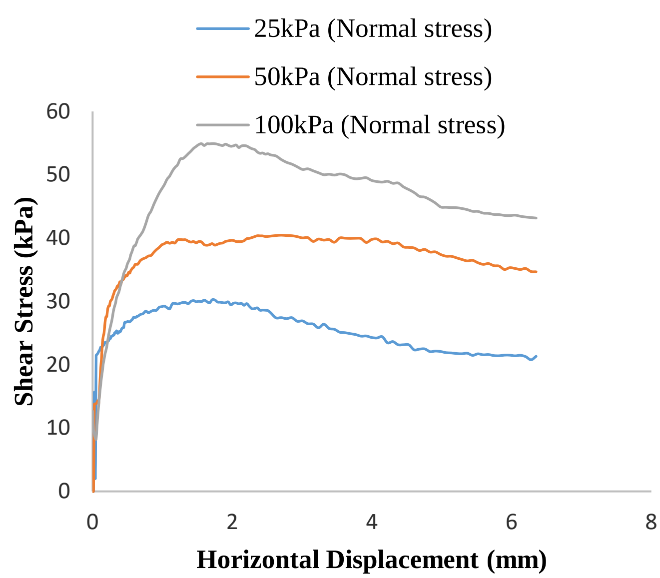


Figure 3.6 Shear strength vs. horizontal displacement curves for three applied normal stresses

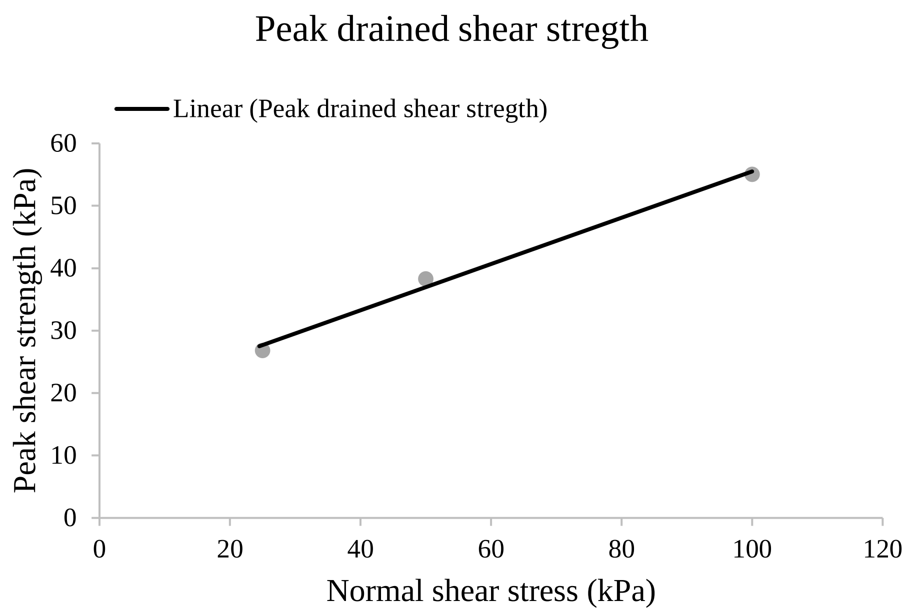


Figure 3.7 Mohr-Coulomb failure envelope for the drained shear strength of Yazoo clay

3.6 Fully Soften Test

3.6.1 *Sample Preparation*

A consistent method for reconstituting soil samples is needed to create normally consolidated samples to be used to predict fully softened shear strength (ERDC/GSL GeoTACS TN-13-1) was adapted and used to provide thoroughly disaggregated soil samples for direct shear testing. The following procedure was followed:

The samples were shaved or shredded at their natural water content. The shredded material was dried for at least 48 hours, to a constant weight at a temperature less than 50 °C and relative humidity below 30%. The sample was then soaked in distilled water for at least 48 hours. The resulting slurry of soaked material and water had a water content above 300%, or more than double the estimated liquid limit, whichever was greater. About 500 ml of the slurry was placed in a mechanical blender. The soil water slurry was then blenderized without interruption for 10 minutes. After mixing, it was washed through a No. 40 sieve into the plaster of Paris dishes lined with filter paper. These dishes helped wick out excess water from the slurry. A combination of this and air drying was used to bring the water content of the soil near the liquid limit. The resulting material was then combined into one homogenous sample by working with a steel spatula on a glass plate to make sure that no clay clumps, extraneous nodules or other ‘coarse,’ non-clay particles remained. The sample was placed in the shear box which was assembled with filter paper and porous on top and bottom and placed in the shearing device. The ASTM D3080-11 procedure was followed for the remainder of the procedure except where indicated otherwise. The soil sample was incrementally consolidated to the required confinement stress.

(a)

(b)

(c)

(d)

Figure 3.8 Sample preparation from the Slurry mix to fully soften shear strength, (a) Shredding, (b) Slaking, (c) Blending and (d) Sieving

Figure 3.9 shows a reduction in strength with accumulated displacement beyond the peak strength. The amount of horizontal displacement used in the direct shear testing was sufficient to mobilize post-peak shear strengths; this is in general agreement with the idealized response by Skempton (1970). As presented in the failure envelope, the friction angle ϕ (18.8) and cohesion c (10.8 kPa) is determined.

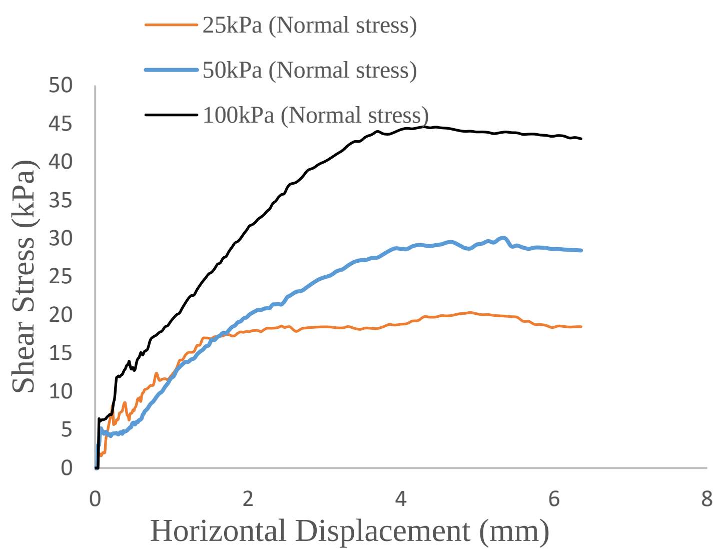


Figure 3.9 Shear strength vs. horizontal displacement curves for three applied normal stresses

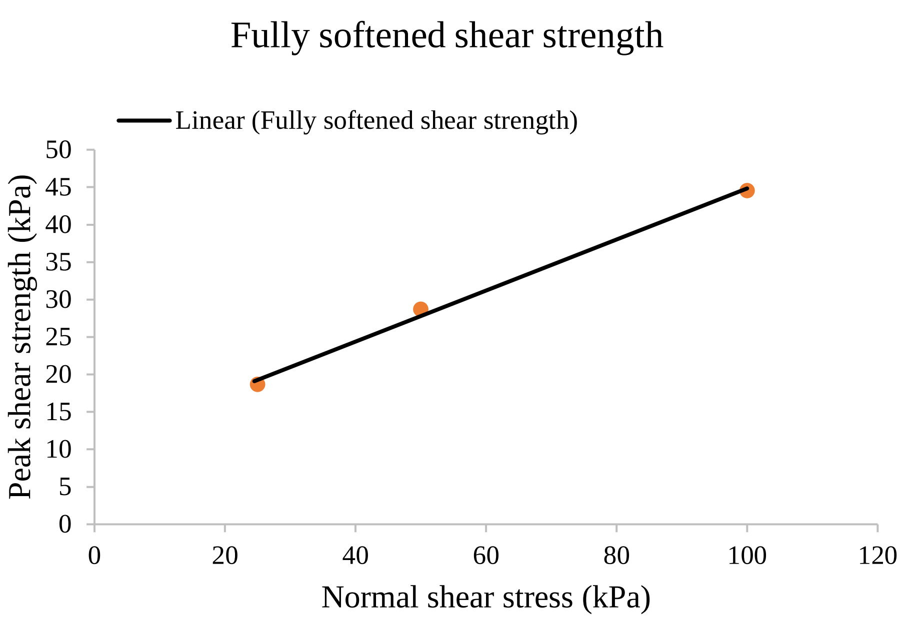


Figure 3.10 Mohr-Coulomb failure envelope for fully soften shear strength of Yazoo clay

3.7 Residual direct shear test

3.7.1 *Sample Preparation*

The most commonly used method for determining drained residual shear strength of stiff clays, shales, and mudstones is by drained multiple reversal direct shear tests or by drained ring shear tests, using either undisturbed or reconstituted specimens (e.g., Bishop et al. 1971; Chandler 1977; Bromhead and Dixon 1986; Stark and Eid 1994). For direct shear tests, the best procedure is to start with precut specimens, sometimes cut from intact, undisturbed samples, but more often prepared from reconstituted samples (Mesri and Cepeda-Diaz 1986). Undisturbed specimens from large undisturbed samples were used for the test, secured by undisturbed tube sampling procedures. The specimens were handled carefully to minimize disturbance, changes in cross-section, or loss of water content. The soil sample is extruded from the sampling tube and precut to fit into the direct shear box. Saturation of the sample together with the direct shear box was allowed for 24hrs. The specimen was consolidated for 24 hours. Afterward, the sample was sheared using a displacement/strain rate of 0.127mm/min. The device was modified to enable reversing of the upper one-half of the shear box to its original position after each run. Therefore, the test is conducted with the desired amount of accumulated one-displacement for each value of effective normal stress (or normal stress increment). Since the direct shear tests are conducted as drained tests (slow tests), it was essential to select an initial displacement rate that would allow for drained behavior during shear.

3.7.2 *Results and Discussion*

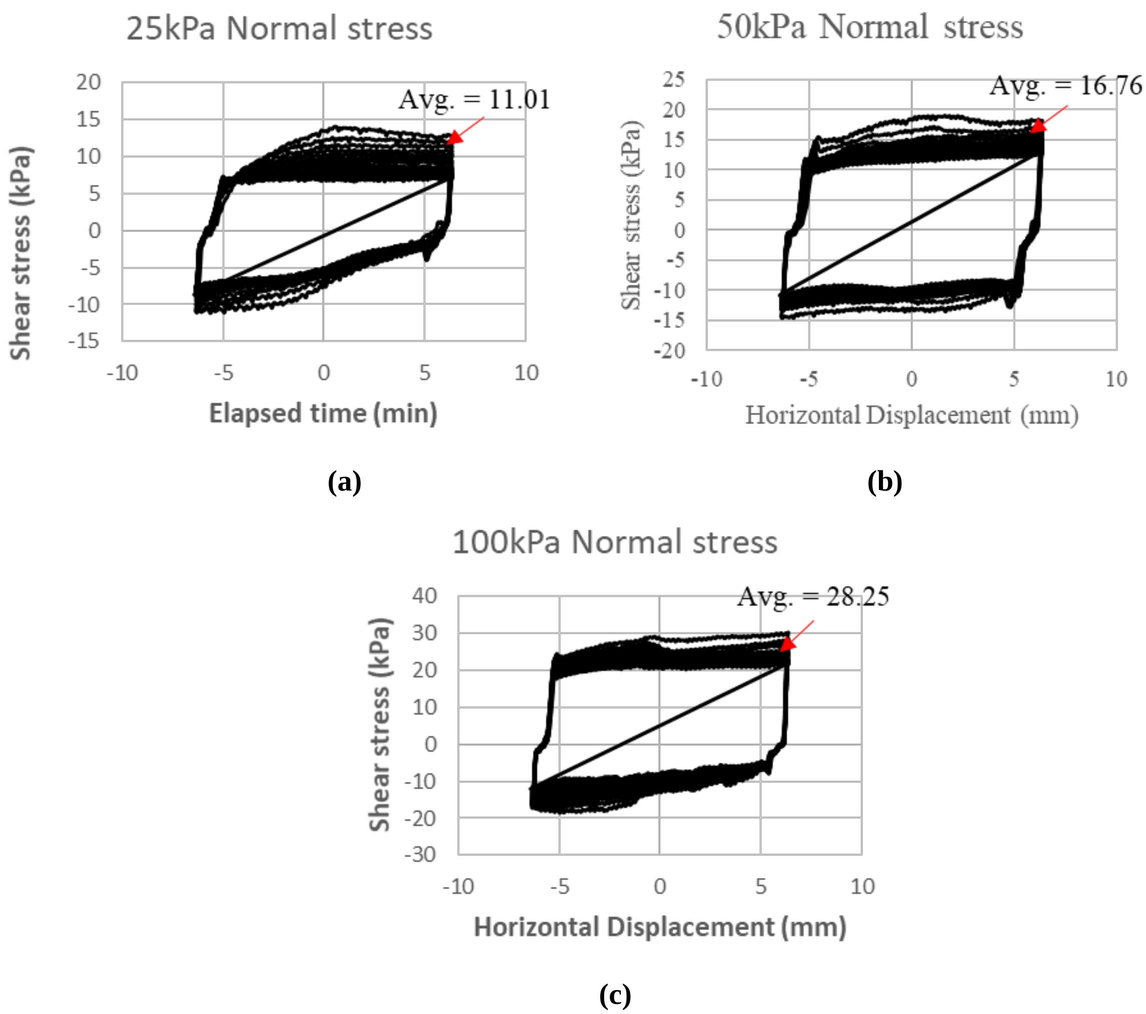
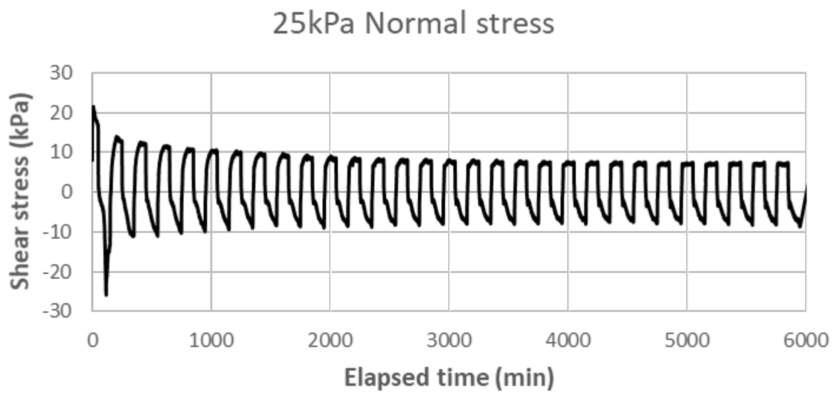
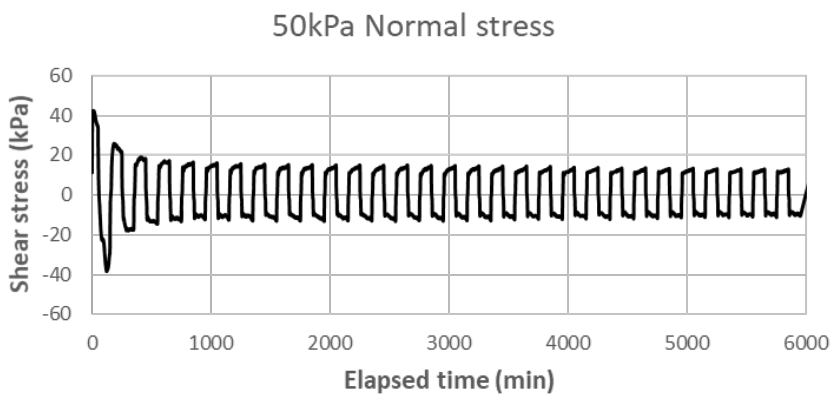


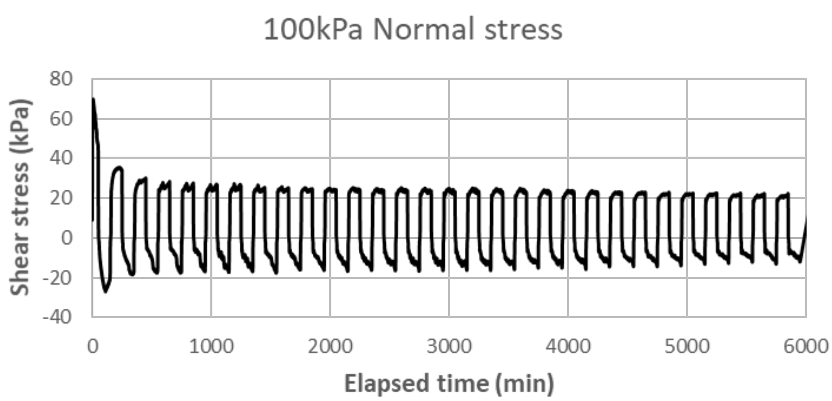
Figure 3.11 Plot of Shear Stress vs. horizontal displacement variations for (a) Normal Stress of 25kPa (b) Normal Stress of 50kPa and (c) Normal Stress of 100kPa



(a)



(b)



(c)

Figure 3.12 Plot of Shear Stress variations with time for (a) Normal Stress of 25kPa (b) Normal Stress of 50kPa and (c) Normal Stress of 100kPa

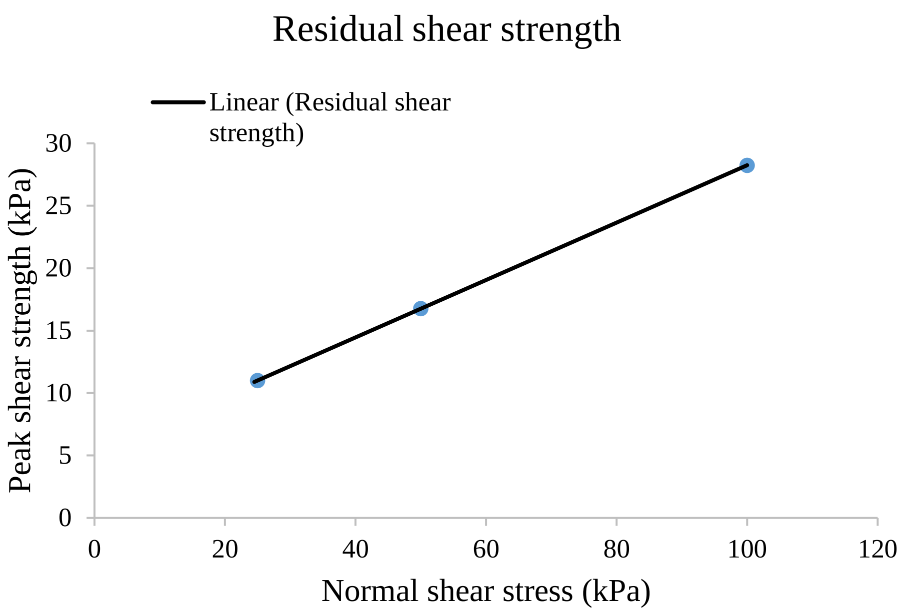


Figure 3.13 Mohr-Coulomb failure envelope for residual of Yazoo clay

3.8 Shear strength Analysis

The combined (peak, fully soften and residual) test results Figure 3.14 and Table 3.1, indicates that the highest shear strength (c = 18.4kPa and φ = 20.2deg.) was determined with the peak shear strength test, whereas the residual test generated the lowest strength (c = 5.3kPa and φ = 12.8 deg.).

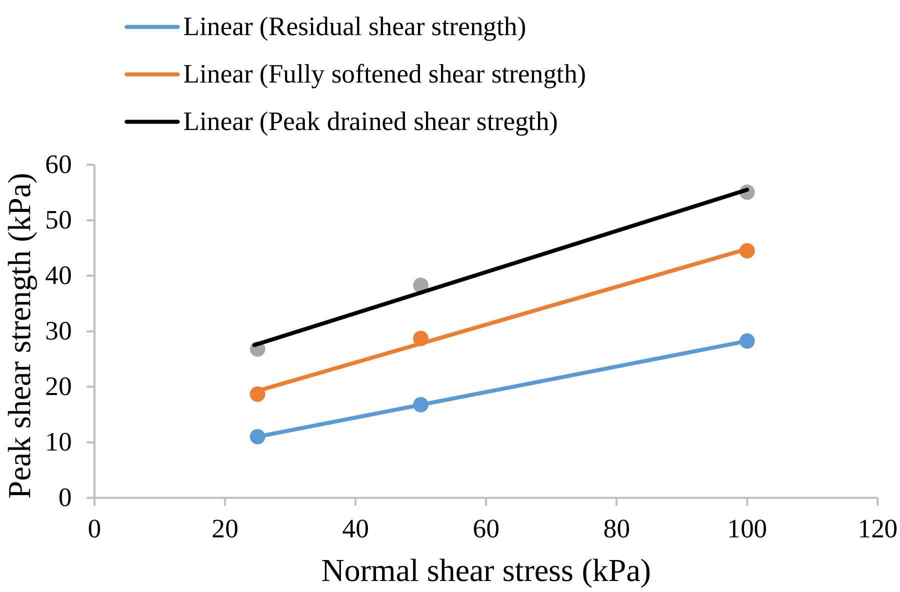


Figure 3.14 Mohr-Coulomb failure envelope for residual fully softened and drained shear strength of Yazoo clay

Table 3.2 Peak fully soften and Residual Yazoo clay test results

Tests	Cohesion (kPa)	Friction angle (deg.)
Peak Drained Direct Shear Test	18.4	20.34
Fully Softened Shear Strength test	10.8	18.8
Residual Direct Shear test	5.3	12.9

For the Yazoo clay soil that extends over central Mississippi, Alabama, and Southern Louisiana, a peak strength value is thought to be an unconservative estimate of the available in situ shear strength regarding the stability of cut slopes in these soils. This post-peak strength phenomenon can be explained by the progressive failure theory, which states that the peak strength is passed at any one point along a failure surface within a cut slope because of fissures or discontinuities that act as stress concentrators forcing that point to pass the peak strength with a given amount of displacement. Once the peak strength has passed at one point along the failure surface, the stress is shifted to another point, causing it to pass the peak, and so on. In this way, a progressive failure can be initiated, and the strength along the entire length (or majority) of a slip surface will decrease as a function of displacement to a lower strength range bounded by the softened and residual strengths. Skempton (1970) showed the idealized response of a stiff, fissured clay during a drained direct shear test which illustrated the reduction in shear strength as a function of displacement that would occur at one point and eventually progress to a portion or the entire length of a slip surface. The softened strength and zero cohesion residual strength are generally recognized as the upper and lower boundaries for the range of in situ strength mobilized for slope failures in fissured clays.

3.9 Wet-dry cycle test

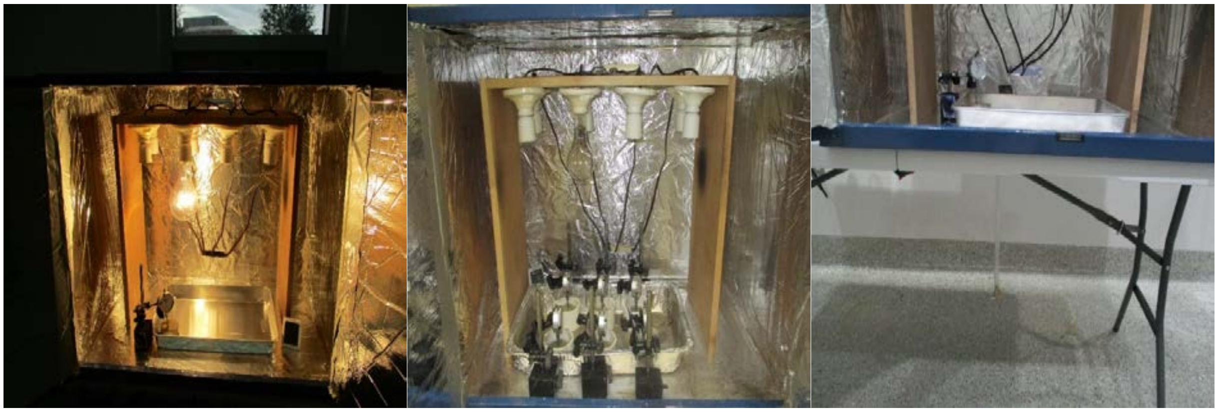
Expansive Yazoo clay soil is highly susceptible to the climate change, and it is dominant in central Mississippi and neighboring states. This high plastic clay undergoes volume change due to wetting or drying under different seasonal variation. From an engineering point of view, an essential characteristic of clay soils is their susceptibility to volume change due to wetting or drying which can occur independently of loading. These repeated volume changes can give rise to ground movements which may result in structural damages, resulting in the high cost of repair or reconstruction. It is generally accepted that the water content, void ratio and type of clay minerals in the soil are the main factors affecting the volume change potential of the soil (Jones and Jones, 1987, El-Sohby and Rabba, 1981, Fredlund and Rahardjo, 1993; Hanafy, 1991; Ho J.1992; Mitchell and Van Genuchten, 1992; Komine and Ogata, 1996; Marinho and Stuermer, 2000, Bell, 2000 and Ferber et al., 2009). When the water content of expansive clay changes due to drying or wetting, it leads to changes in the volume and void ratio of the soil. When a sample of dry, expansive clay is wetted, the increase in water content will cause an increase in the volume of voids (i.e., swelling). Hanafy (1991) suggested that two curves (wetting and drying) can be combined and integrated into one ‘S’ shaped curve at the equilibrium condition. It is possible to characterize the volume change in terms of the changes in the void ratio about changes in water content. Tripathy et al. (2002) and Estabragh et al. (2013) determined a characteristic ‘S’ shaped curve for two expansive soils. They used this curve to explain the potential of volume change regarding void ratio and water content resulting from wetting and drying cycles. Many researchers, including Wheeler et al. (2003), and Alonso et al. (2005) have studied the cyclic wetting and drying behavior of the expansive clayey soils by a suction control method. Wheeler et al. (2003) showed that irreversible compression occurs during drying stages of wetting-drying cycles. They studied the influence of a wetting-drying cycle on subsequent behavior during isotropic loading. Based on the results they proposed a model for the coupling of hydraulic hysteresis and mechanical behavior. Alonso et al. (2005) reported that samples experience progressive shrinkage during successive cycles of wetting and drying until a reversible elastic response occurs. This progressive shrinkage process leads to an increase in the over-consolidation ratio (OCR). Although many researchers have studied the behavior of expansive soils, the study of the wetting and drying paths in the form of variations of the water content and void ratio during cycles of wetting and drying is mainly limited to the work of Tripathy et al. (2002) and Estabragh et al. (2013).

Other researchers, including Sharma (1998), Wheeler et al. (2003), Sivakumar et al. (2006) and Jotisankasa et al. (2009), have studied the variations of specific volume(1+e) with suction. Their tests were conducted on unsaturated samples in a modified triaxial cell. It is indicated that during drying and wetting the values of specific volume for a given suction are not the same for wetting and drying because of the hysteresis phenomenon. The void ratio and water content of an expansive soil are changed during wetting and drying. Since suction is dependent on the water content, the variations of void ratio during wetting and drying can be a function of suction. A review of the literature indicates that there has been a considerable amount of research on the deformation behavior of expansive soils during wetting and drying condition but the investigation into the variations of void ratio, shear strength has been limited. Determination of soil strength parameters for slope stability analysis is the most important task as the factor of safety will be significantly reduced with the number of wetting and drying cycles (Rogers and Wright 1986). Wetting and drying (w-d) cycles reduce the expansive nature and increase the collapse tendency of embankment slopes made of high plasticity clay (Alonso and Gens 1995). Rogers and Wright (1986) also reported that wetting and drying could reduce the strength of high plastic clay which was reflected by the decrease of cohesion intercepts with only a minor change in friction angle. Reduction of soil strength due to weathering may reduce the factor of safety of a slope with time.

The Yazoo formation contains Yazoo clay, which is geologically defined within the Jackson Group, has been identified throughout the southeastern and southwestern United States. The metropolitan Jackson area is located directly on top of the Yazoo clay (Lee (2012)). Yazoo clay is indicated to have a very high shrink/swell potential, and moisture changes result in swelling and shrinkage behavior. This shrink-swell behavior causes detrimental effect to the roads, foundations, and related infrastructure in the central Mississippi region (Douglas and Dunlap 2000; Lee 2012). Although the Yazoo clay causes a significant problem in the deep southern states of United States, insufficient research work is conducted on the behavior of this expansive soil. The current study is focused on the volume change behavior of Yazoo clay. Therefore, the objective of this study is to investigate the effect of the wet-dry cycle on the void ratio of Yazoo clay. Changes in basic soil properties such as moisture content and the void ratio can provide a valuable insight into the change in soil strength parameters. Consolidated drained direct shear tests in the laboratory were conducted at normal stresses with the soil samples that went through different numbers of wet-dry cycles.

3.9.1 *Methodology*

Reconstituted Yazoo clay soil samples with a height of 1inches and 3 inches in diameter were used for this investigation. Collected soil samples from the boreholes were first broken down into small pieces and kept in the oven for 24 hrs. For drying. Oven dried soil was pulverized before grain size distribution analysis. Sample passing #40 sieve was used for the determination of liquid limit and plastic limit according to ASTM D4318. Samples are having a liquid limit of more than 50 which was separated and used for further investigations. Wet sieving method was used to separate the particles passing #200 sieve. A particle that retained on #200 sieve was used to determine the grain size distribution according to ASTM D422-63, and hydrometer analysis was used for the soil particle smaller than 0.075 mm.



(a) (b) (c)

Figure 3.15 Construction of the wet-dry cycle chamber

Reconstituted Yazoo clay soil samples with a height of 1inches and 3inches diameter were used for this investigation. The samples were air dried, pulverized and passed through a #40 U.S. sieve. Values of dry density 15.71 kN/m^3 (100 pcf) and optimum moisture content (15 %) were used as "target" values. A 2.5 kg hammer with an acrylic cylindrical face 62.8mm in diameter was utilized to compact the specimens with four equal lifts using six drops of the hammer at the height of 304.8 mm. After preparation, the mold was disassembled, and the specimen was placed in a stainless-steel ring with an inside diameter of 76.2 mm and height of 25.4mm. Compacted samples were prepared and transferred into a modified chamber for wetting and drying procedure. An acrylic pipe of 63.5mm inner diameter and 63.5mm height was used to keep the sample for wetting and drying. Porous stones were placed at top and bottom of the sample for the natural drainage of water. To prevent the clogging of the porous stone, the filter paper was used at top and bottom of the chamber, in between soil and porous stone. A schematic diagram of the wet-dry chamber is shown in Figure 3.16. To prevent the soil loss, porous stone at the bottom of the chamber was sealed tight to the acrylic pipe. Approximate height of the sample was about 25.4mm. The sample was placed in the chamber, filter paper reinforced with aluminum foil/porous stone was used as a top lid for the sample (Figure 3.16). After placing the compacted sample into the chamber, the whole set was immersed under water to saturate the sample for at least 24 hrs. The swelling or compression of the soil specimen was determined by observation of the dial gauge. Swelling or shrinkage was regarded as complete when no additional expansion or compression could be detected. The testing time, i.e., the time for a drying and wetting cycle depends on the permeability of the soil. The measurement of vertical soil movements determined the end of a shrinkage or swelling phase. Less permeable soils require more time for each cycle. After the desired number of wetting-drying cycles specimens were trimmed into a stainless-steel ring with a diameter of 63.5mm and a height of 25.4mm. Finally, they were extruded from the ring into the direct shear box for testing.

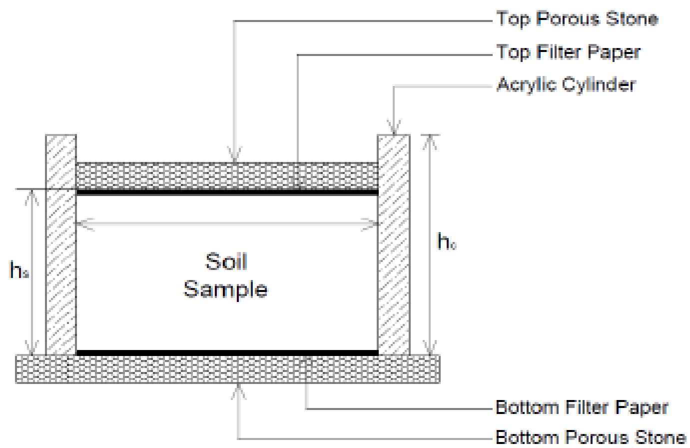


Figure 3.16 Construction of the wet-dry cycle chamber

The direct shear test apparatus used in this investigation had a pneumatic loading piston for applying the vertical load to the sample. This shear testing device has a large digital display for controlling load and displacement for both vertical and horizontal directions. Linear strain transducers were used to measure the vertical and shear displacements while the vertical load was measured with a pressure transducer. A load cell was used to measure the load in the horizontal direction. Specimens were consolidated vertically using a single load increment with a loaded hanger, and no free water was accessible to the specimen at the onset of consolidation. Maximum time for completion of primary consolidation was 6hrs but a 24hrs period of consolidation was used during the experiment. The rate of shear was set as 0.0508mm/hrs. To ensure adequate drainage. Each specimen was sheared up to deformation of 3.81mm.

Wetting was followed by a drying period by keeping the chamber at room temperature (120 - 125^{0}F) for 24 hrs. After drying sample was set for another wetting period before the placing in the shear strength test using a Geojac direct shear machine. Thus, a complete wet-dry cycle consists of wetting, drying, and rewetting process. These steps were repeated for further wet-dry cycles.

3.9.2 *Results and Discussion*

3.9.2.1 Effect of the wet dry cycle on vertical deformation

The vertical deformation of the samples is presented as the change in the height of the sample during either swelling or shrinkage. It is expressed as a deformation of the initial height of the sample at the beginning of the first swell-shrink cycle. By plotting the vertical deformation of a sample for several swell-shrink cycles, the change in the height of the sample during any of the swelling or shrinkage cycles can be observed. Typical wetting and drying curves of tested expansive Yazoo clay soil for 3, 5 and 7 number of cycles are shown in Figure 3.16. It is observed that in the beginning cycles, deformations during shrinkage are greater than deformations during swelling. However, the swelling decreases with the increasing number of cycles of drying and wetting.

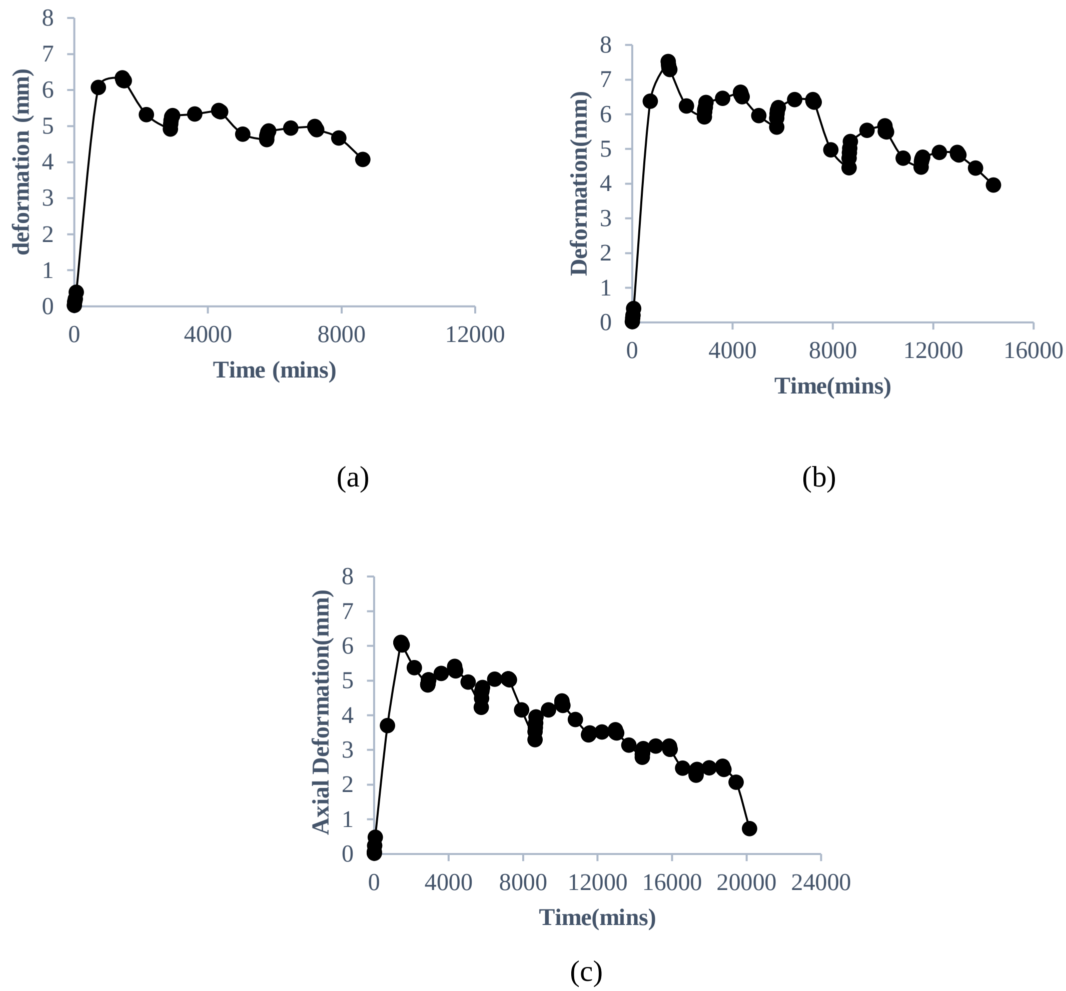


Figure 3.17 Vertical deformation curve of Yazoo clay sample at (a) 3N wet-dry cycle (b) 5N wet-dry cycle (c) 7N wet-dry cycle

It can be concluded that the shrinkage behavior changes with repeated cycles of wetting and drying. These findings are consistent with the results that were reported by Tripathy et al. (2002), Subba Rao and Tripathy (2003) and Alonso et al. (2005).

3.9.3 *Effect of the wet dry cycle on porosity and void ratio*

Basma et al. (1996) suggested that swelling is associated with a decrease and an increase in the apparent voids in soils due to partial and full shrinkage. The study indicated that the voids are reduced during the wetting and drying cycle for partial shrinkage. This reduction decreases the soil ability to attain further water upon rewetting and this, in turn, reduces the potential expansiveness of the clay. They also indicate that most of the free water will evaporate during full shrinkage and this increases the apparent voids. This, in turn, creates more pores to be filled with water when the samples are rewetted. Consequently, the potential swelling increases. This is consistent with observations made in this study for Yazoo clay soil. The swelling potential and rate of shrinkage increases as the number of wet-dry cycle increases although the latter rate dominates as shown in Figure 3.17.

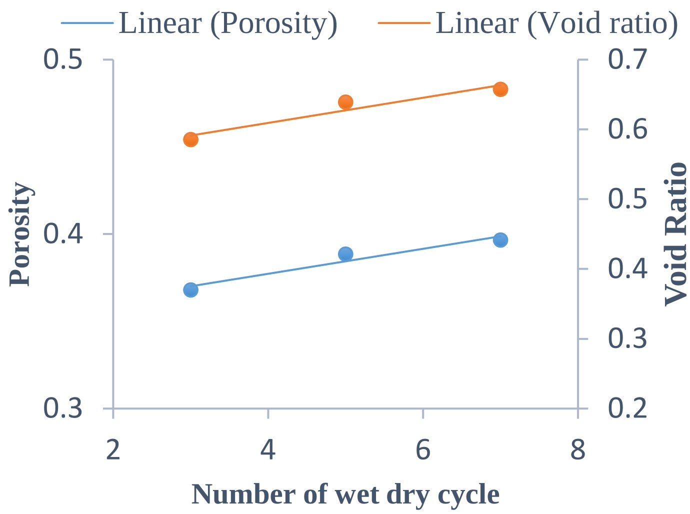
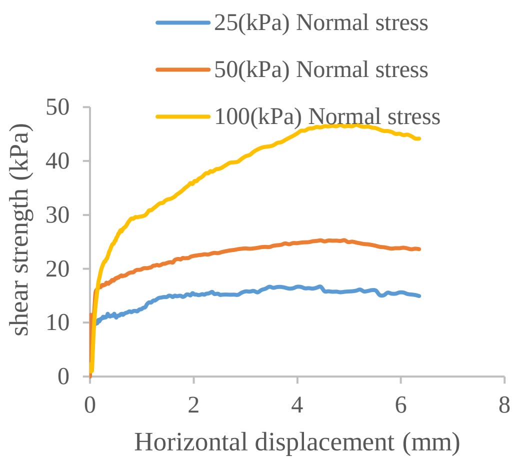


Figure 3.18 Change in porosity/Void ratio with series of wet-dry cycle

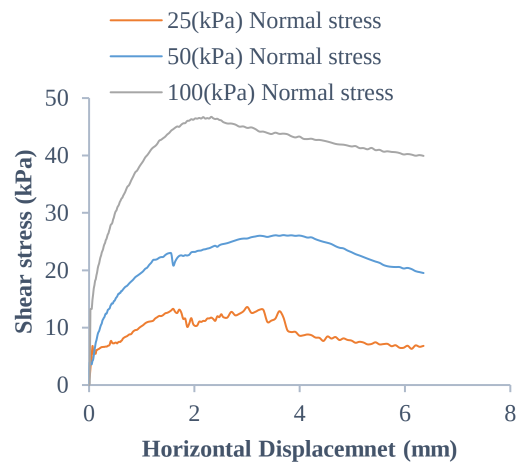
With the increase in the number of drying-wetting cycles, the compaction segment of curve grows significantly. Mainly because the sample grain skeleton structure under the action of a cycle is changing, microcracks and microcracks evolve, which continuously increase the porosity/ void ratio results.

3.9.4 *Effect of wet-dry cycle on shear strength*

Peak shear strength obtained from the compacted sample decreases with the increasing number of wet-dry cycles. Direct shear tests were conducted for compacted specimens those who underwent 3, 5 and 7 wet-dry cycles with three different normal stresses of 25, 50 and 100kPa as shown in Figure 3.18. Peak shear strength for the compacted stage was maximum and decreases with the increasing number of wet-dry cycles.



(a)



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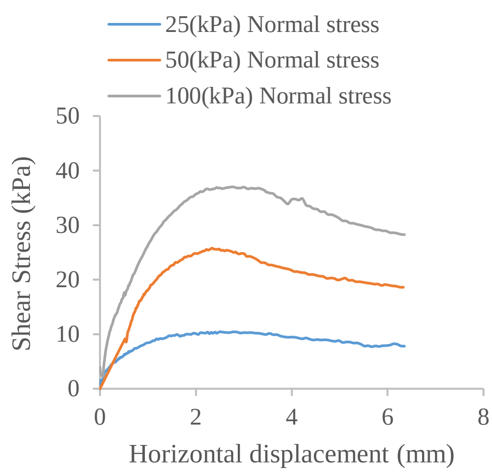


(c)

Figure 3.19 Shear stress vs. horizontal displacement curve of Yazoo clay sample at (a) 3N wet-dry cycle (b) 5N wet-dry cycle (c) 7N wet-dry cycle

The shear strength parameters (cohesion and friction angle) of the drying-wetting cycle specimens are determined using Mohr-Coulomb (M-C) failure envelop. Results obtained from the direct shear tests on wet-dry samples prepared from high plastic clay are shown in Figure 3.19. As the soil strength parameters decrease with the increasing number of wet-dry cycle, the position of M-C failure line also shifts downward. This indicates the dependency of failure envelope with void ratio/porosity. Increase in a number of wet-dry cycle increases the porosity and void ratio and as well reduces the shear strength envelope.

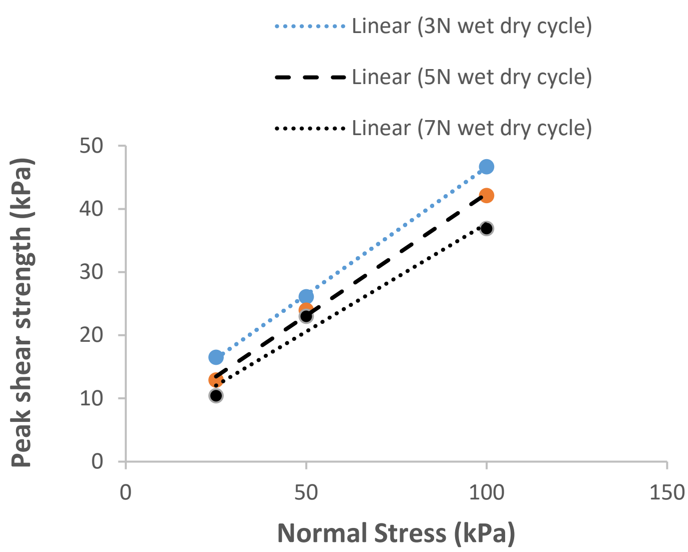
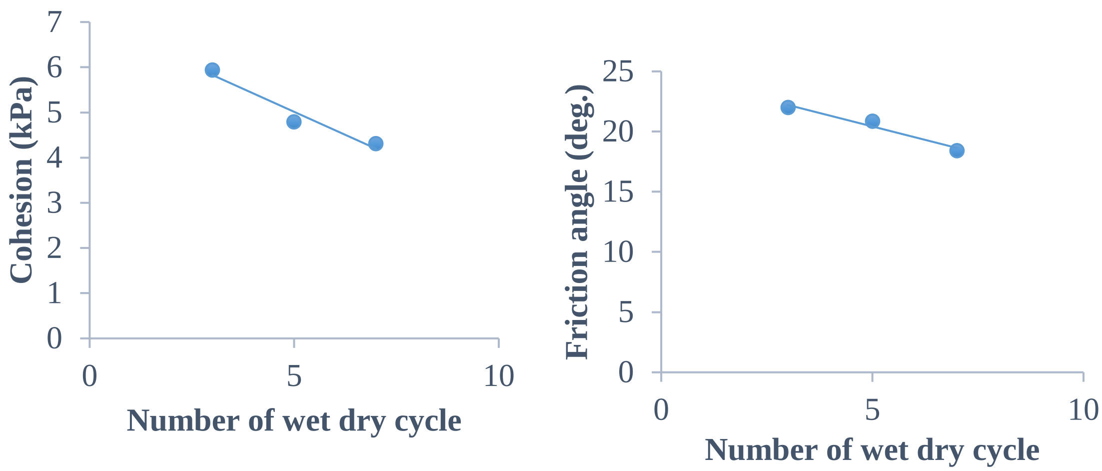


Figure 3.20 Mohr-coulomb failure envelopes for the sample subjected to different wet-dry cycles

Rogers and Wright (1986) performed some laboratory tests to understand the shear strength of high plastic clay used to construct embankment due to seasonal variation in Texas. They performed direct shear and triaxial tests for compacted Beaumont clay samples that went through series of the wet-dry cycle up to 30 cycles. The expansive Beaumont clay sample has an average liquid limit of 70 and plastic limit of 20 percent; the average plasticity index was 50 percent. Experimental results obtained by Rogers and Wright (1986), showed that increasing number of wet-dry cycle reduces the shear strength of high plastic soil as confirmed in this study and also after thirty drying and wetting cycle cohesion decreases to zero. Based on the direct shear test of results of the Yazoo clay soil samples, the variation in cohesion and the frictional angle is drawn based on the test results of wetting and drying cycles of 3, 5, and 7 times, respectively. The cohesion and internal friction angle of Yazoo clay, along with the change of drying-wetting cycles is shown in Figure 3.20. The results showed that with the increase of drying-wetting cycles, the cohesive force of c significantly reduced. Due to the 7 wetting and drying cycles, the cohesive force is reduced to a minimum 4.31 kPa than 5.94 kPa for 3 nos. of wetting and drying cycles. Interestingly, the internal friction angle exhibits similar variation to the cohesive force. Therefore, the friction between the particles decreased, specifically, φ become smaller.



(a) (b)

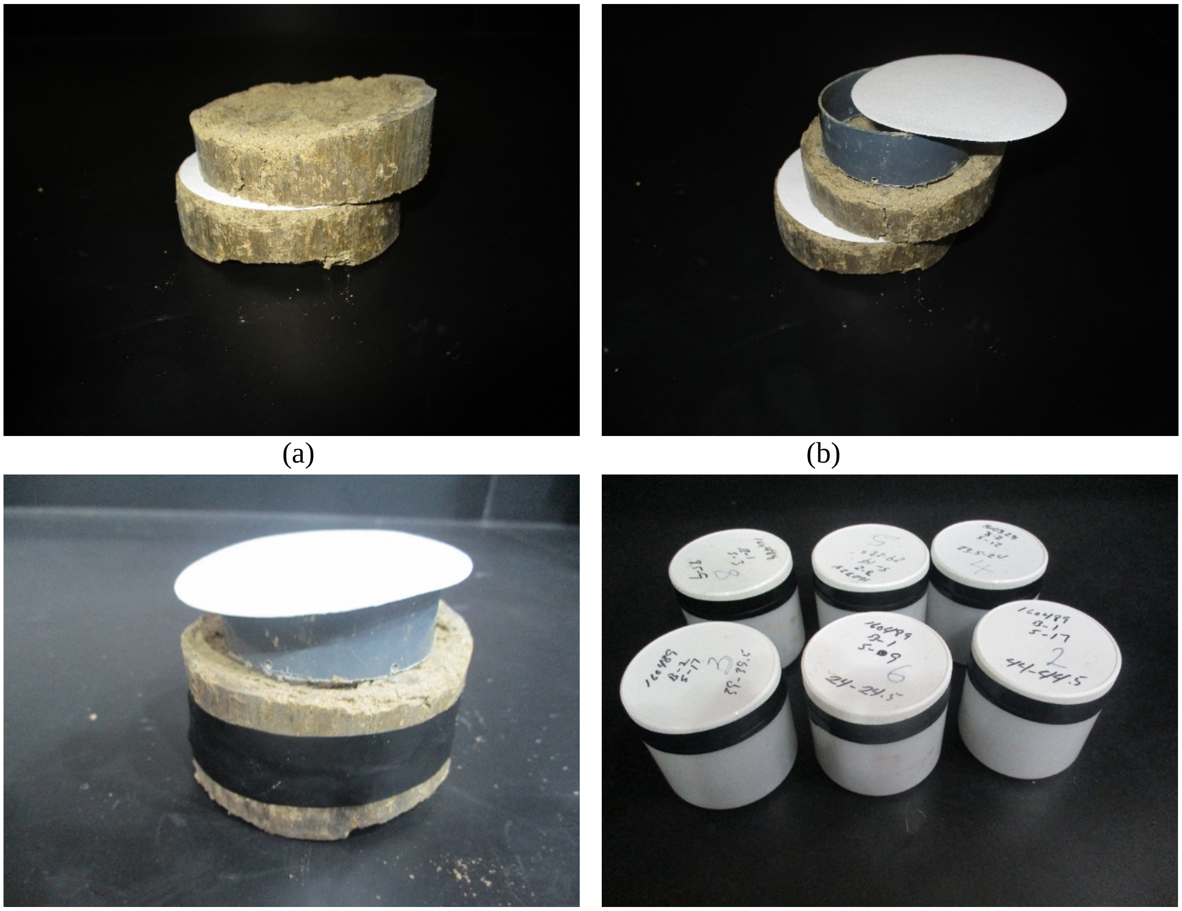
Figure 3.21 Effect of the wet dry cycle on soil strength of Yazoo clay (a) Effect on cohesion (b) Effect on friction angle

3.10 Soil Water Characteristics Curve of Yazoo Clay

Several methods in addition to the standard filter paper method are available for measuring soil total suction. These methods use devices that vary widely regarding the range of measurement, accuracy, precision, and reliability. Decagon's WP4 Dewpoint Potential Meter chilled-mirror technique and Wescor Inc. thermocouple psychrometers connected to a microvolt data logger were initially been considered for this study but were not selected. Pat-rick et al. (2007) compared the WP4 to filter paper and determined that the results were mostly the same when adequate equilibrium times were followed. The WP4 results were obtained in about 30 minutes each while the filter paper results were obtained in about a week each, but the cost of the WP4 was about 6000 times higher than the cost of filter paper. Mabirizi and Bulut (2009) compared the performance of filter papers, thermocouple psychrometers and chilled- mirror psychrometers for measuring the total suction of various high plasticity clay samples taken directly from Shelby tubes. The difference in suction readings was negligible at higher suction ranges. The filter paper technique cost less and reliably measured almost the entire range of total suction. If proper laboratory protocol is established, it provides very reliable total suction estimates compared to the thermocouple and chilled-mirror psychrometers. Other studies of the filter paper method (Houston et al. 1994, Leong et al. 2002, Bulut and Wray 2005) demonstrated that if a consistent and well-maintained laboratory testing protocol is followed, the filter paper technique is a very reliable method. The interactions of soil and water are influenced by numerous soil properties. The variability and energy state of soil moisture are factors that influence the hydraulic behavior of the soil. Since the deficit or excess of available water about that required by plants can affect production, it is essential to analyze the behavior of water storage in the soil to take actions that maximize productivity without wasting water. The filter paper method is a laboratory technique that has been in use for some time in the field of Civil Engineering, as noted by Chandler and Gutierrez (1986). The method was first formally described by Gardner (1937) and has recently been accepted as a standard method of measuring soil potential (ASTM (2003)), reaching far higher ranges of water potential in comparison to other techniques (Likos and Lu (2004)), and is based on the principle of moisture absorption by filter paper until there is a balance in potential between filter paper and soil (Ng and Menzies, 2007). The most commonly used filter papers are Whatman No. 42 and Schleicher & Schuell No. 589-WH (Leong et al., 2002; Bulut and Wray, 2005; Bulut and Leong, 2008). Among all known methods for measuring soil water potential, the filter paper technique is the only method to determine both the total soil water potential and matric potential (Bulut and Wray, 2005; Bulut and Leong, 2008; Beddoe et al., 2010). Houston et al. (1994) examined the influence of variations in particle size on the matric potential of soils by the filter paper technique. By the same method, Rao and Revanasiddappa (2000) investigated the effects of variations on matric potential on the behavior of compacted clay soils. This study aimed to evaluate a method using Whatman No. 42 filter paper to establish the soil water retention curve for five different soils and to compare results with those of the conventional method.

3.10.1 *Test Procedure*

Reconstituted samples of Yazoo clay were used for this experiment at 5% water content variation. Before starting to take measurements, all the items related to the experiment were cleaned carefully, and latex gloves were used throughout the experiment. Before taking the plastic containers, all small aluminum cans were weighed to nearest 0.0001 g. Accuracy were recorded on a filter paper water content measurement data sheet developed by Lee (1992). After that, all measurements were carried out by two persons. For instance, while one person was opening the sealed glass jar, the other person was putting the filter paper into the aluminum can very quickly (i.e., in a few seconds, usually less than 5 seconds). The weights of each can with wet filter papers inside were taken very quickly. The weights of cans and wet filter papers were recorded with corresponding can number with the top or bottom filter paper. The previous step was followed for every glass jar. Then, all cans were put into the oven with the lids half-open to allow evaporation. All filter papers were kept at 105 ± 5°C temperature for 24 hours inside the oven. Before taking measurements for the dried filter papers, the cans were closed with their lids and allowed to equilibrate for 5 minutes in the oven. Then a can was removed from the oven and put on an aluminum block for 20 seconds to cool down; the aluminum block acted as a heat sink and expedited the cooling of the can. After that, the can with dry filter paper inside was weighed again very quickly. The dry filter paper was taken from the can, and the cold can was weighed in a few seconds. Finally, all the weights were recorded on the data sheet. The moisture content of every filter paper was calculated by following the procedure described above. Several filter paper tests were performed by following the procedure described above for obtaining the total suction calibration curve.



(a) (b) (c) (d)

Figure 3.22 Sample Preparation (Rifat Bulut et al. (2001)) (a) Soil Sample (b) Sample Detail (c) Sample Detail (d) Samples

3.10.2 *Results and Discussion*

The total suction calibration curve for Yazoo Clay soil was obtained for the filter paper water contents and total (or osmotic) suction values (Figure 3.21). The reason for data scatter was most probably due to alternating condensation and evaporation during equilibrium. The values for matric suction as a function of moisture content followed the same trend found by Almeida et al. (2011) for one of the soils.

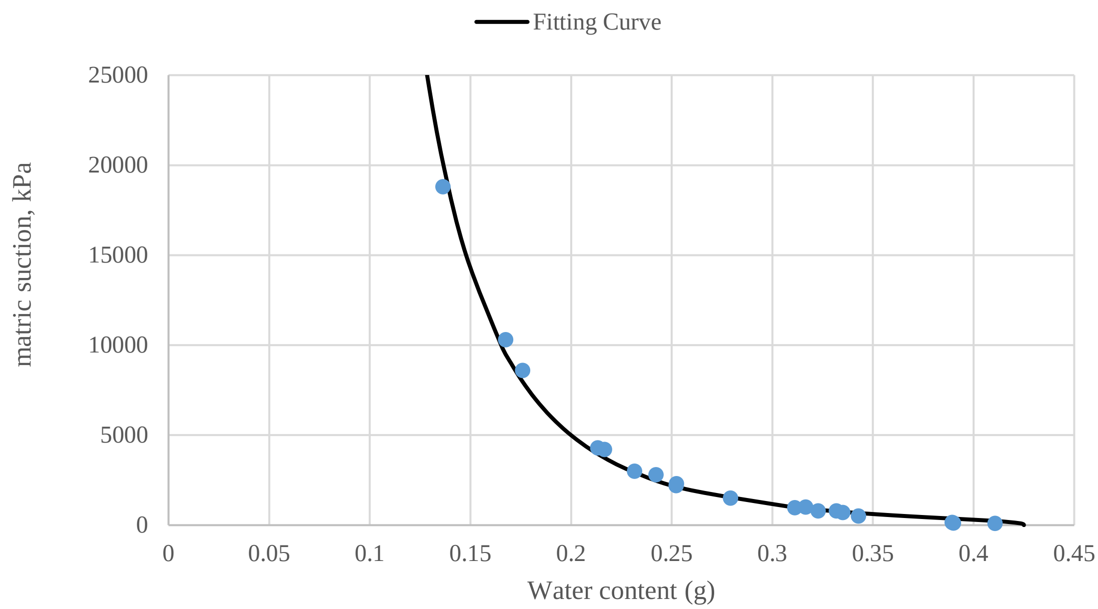


Figure 3.23 Yazoo clay SWCC

Chapter 4: CHANGE IN FACTOR OF SAFETY OF A HIGHWAY SLOPE MADE OF YAZOO CLAY

4.1 Progressive Change in Factor of Safety

Many of the slopes in multimodal transportation infrastructure (highway embankment and levees) of Mississippi are constructed using marginal highly plastic clay soil (Yazoo clay) which are highly expansive and are known to be susceptible to shallow landslide. The highly plastic clay soil undergoes repeated wet-dry cycles, which reduces the shear strength to fully softened shear strength. Previous studies in Texas have indicated that the combination of fully-softened shear strength and especially a perched water zone was found to be the typical result of highway shallow slope failures. However, limited analysis has been conducted on the triggering of shallow slope failures in Mississippi. The current study has focused on the progressive change in the factor of safety of slopes constructed out of Yazoo clay soil. To evaluate the slope failure mechanism in the maritime and multimodal transportation structures in Mississippi, a 2D Slope stability analysis was conducted using the Finite Element Method (FEM) in Plaxis 2D. A highway slope over I220 near US 80 in Jackson, MS was considered as the reference slope, which had progressive changes in the shear strength and failed in February 2015. It has a 3H: 1V slope with a height of 9.2 m. The factor of safety of the slope was determined based on existing soil test data using peak shear strength. Later, the topsoil layer, which gets weathered due to the repeated wet-dry cycle, was changed to fully softened, then to residual shear strength. The effect of each shear strength (peak, fully soften and residual) on the stability of the slope was then evaluated. The slope stability analysis results indicated that the slope is stable during dry conditions, even with the residual shear strength. However, while considering the effects of rainfall, a perched water condition was simulated on the topsoil. It should be noted that the slope geometry and soil profile applied in this analysis is also applicable for a levee slope constructed by Highly Plastic Yazoo clay soil. Site location and Laboratory Investigation

The slope is in the northern part of Jackson, Mississippi along US I220 N over US 80 near Metro Center. The site location is presented in Figure 4.1. The ratio of the of the failed slope is 3H:1V. Laboratory testing was designed to determine the expansive soil's properties, which sampled from this site.

Figure 4.1 Site location

During this current study, the changes in the shear strength of the Yazoo clay soil is investigated, as presented in chapter 3. The test results indicate a progressive change in the cohesion and friction angle, as presented in Table 4.1. The progressive changes in the shear strength of the slope were utilized to investigate the variations in values and simulate the failure condition.

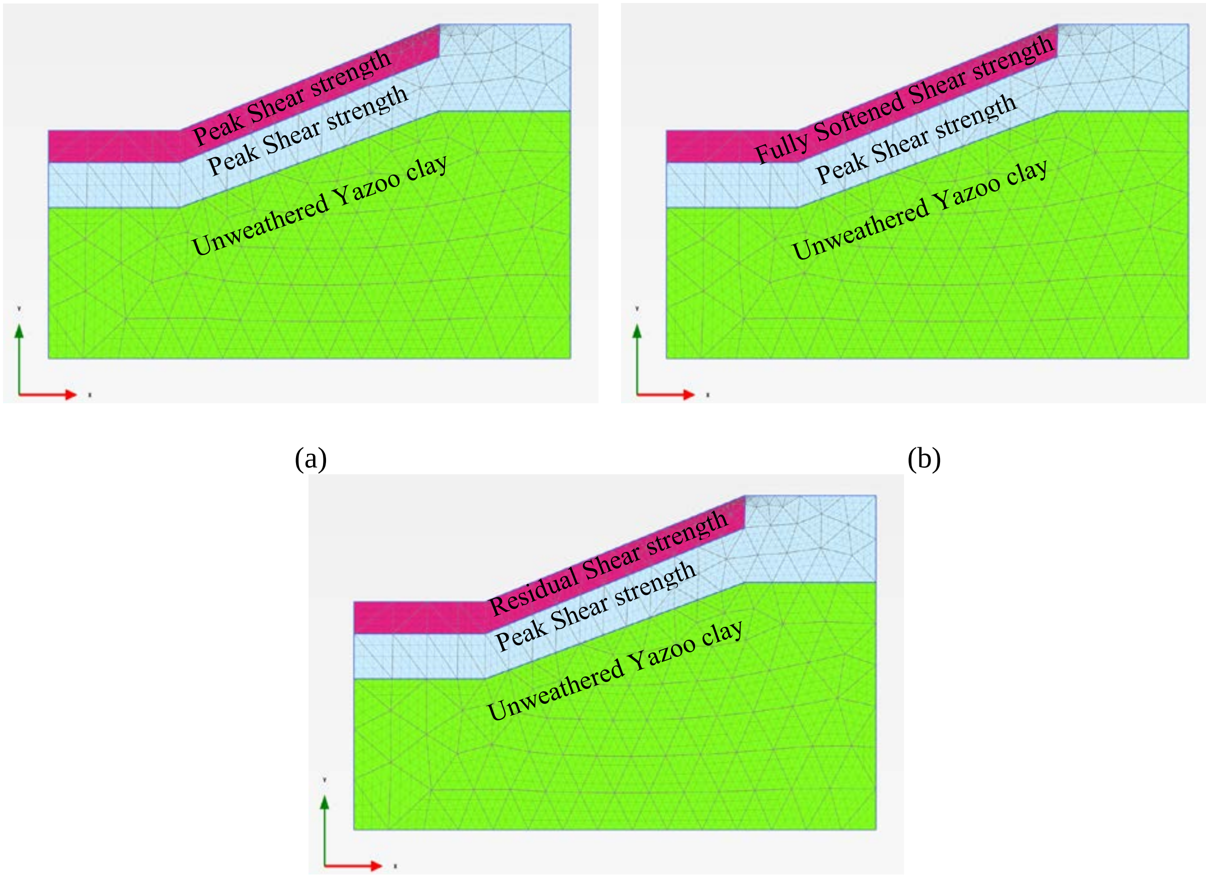
Table 4.1 Peak, Fully softened and Residual Yazoo clay test results

Tests	Cohesion (kPa)	Friction angle (deg.)
Peak Drained Direct Shear Test	18.4	20.2
Fully Softened Shear Strength test	10.8	18.6
Residual Direct Shear test	5.3	12.8

4.1.1 *Development of Finite Element Model*

The FEM program PLAXIS 2D was used to conduct deformation and safety analysis of the highway slopes over I220. A 15-node triangular element was used, which provides a fourth order interpolation for displacements, while the numerical integration involves twelve Gauss points. The Mohr-Coulomb model was considered as the material model, and undrained (A) type was selected for the drainage that the Undrained or short-term material behavior in which stiffness and strength are defined regarding effective properties. The undrained behavior of the material in Plaxis automatically applies a bulk stiffness to water which makes the soil incompressible, while also generating (excess) pore pressures even above the phreatic surface for unsaturated soil. However, during this study, the effect of matric suction was ignored to simulate the critical scenario. Based on the material stiffness, the Poison’s ratio was considered to be 0.3.

An existing soil test report indicated that the fill material of the slope has highly plastic Yazoo clay soil, which has progressive changes in shear strength. Therefore, during this study, different shear strength values for the topsoil layer was utilized considering three different cases in the active zone. The current study considered the depth of the active zone as 3 m. below the active zone, the shear strength of the soil does not change much. Therefore, below the active zone, the shear strength of the Yazoo clay soil remains constant. The different cases for this analysis are presented in Figures 4.2 and Table 4.3. As indicated in Table 4.2, the shear strength of the top layer was varied to peak, fully softened, and residual shear strength to simulate the progressive change in the factor of safety for the slope.



(a) (b)

(c)

Figure 4.2 Illustration of (a) Case 1, (b) Case 2, (c) Case 3

Table 4.2 Variation of Yazoo Clay shear strength in the soil profile

Layer	**Case 1**	**Case 2**	**Case 3**
Soil Layer 1	Yazoo clay (YC) with Peak Shear Strength	YC with Fully Softened Shear Strength	YC with Residual Shear Strength
Soil Layer 2	YC with Peak Shear Strength	YC with Peak Shear Strength	YC with Peak Shear Strength
Soil Layer 3	Unweathered YC	Unweathered YC	Unweathered YC

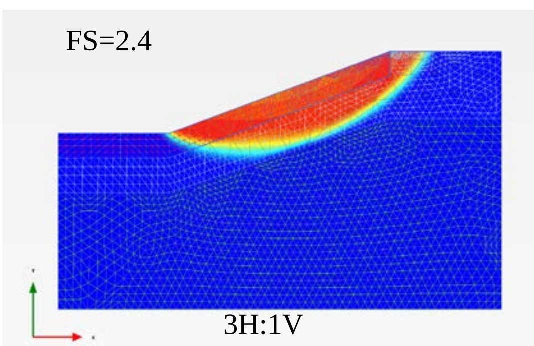
The soil parameters for the numerical analysis in PLAXIS 2D are presented in Table 4.3. The values of strength parameters selected for soil layers 2 and 3 were established from existing soil test reports.

Table 4.3 Soil parameters for FEM analysis

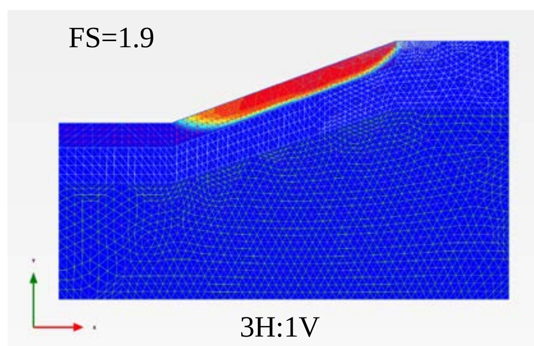
Parameter	Name	Unit	Soil 1 (YC)	Soil 2 (YC)	Soil 3 (Unweathered YC)
Bulk unit weight	Υ_{unsat}	kN/m^3	20	20	22
Saturated unit weight	Υ_{sat}	kN/m^3	21	21	22
Cohesion	C	kN/m^2	Presented in Table 1	18.4	3000
Friction angle	Φ	degree		20.2	35

4.1.2 *FEM Analysis Results*

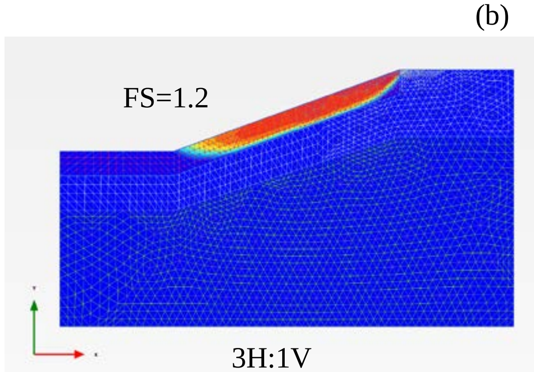
The factor of safety analysis for the slope was conducted using the shear strength reduction method. The slope stability of cases one to three in dry condition for 3H:1V slope ratios are presented in Figure 4.3. It should be noted that the water table of the soil was 10 m below the crest of the slope. Therefore, the safety analysis of the slope was conducted without any water table. Based on FEM results, it was observed that the factor of safety changed with different Yazoo clay (YC) shear strength values in the topsoil layer. The factor of safety for the three cases was observed to be 2.4, 1.9, and 1.2 for peak, fully softened, and residual shear strength, respectively, considering the dry condition. It is evident that in dry conditions, the slopes are stable at different phases of shear strength for the topsoil.



(a)



(b)



(c)

Figure 4.3 Total displacement changes-dry condition, 3H:1V (a) Case 1 (b) Case 2 (c) Case 3

Khan et al. (2017) conducted a failure investigation of a shallow slope on expansive soil in Texas. The study indicated that the failure of the slope occurred at fully softened shear strength, with the presence of a perched water condition. This was due to the infiltration of rainwater at the active zone of the slope. As indicated in Figure 4.3, the factor of safety of the dry slope was stable at even the residual state. The slope may have failed due to the presence of perched water, which might have been brought about by the infiltration of rainwater. To investigate further, the current study was extended, with a consideration of perched water in the active zone of the slope. Figure 4.4 shows the distribution of pore water pressure along with the shear strength of the topsoil layer. However, the pore water pressure was considered at the deeper soil layer, to match the field condition.

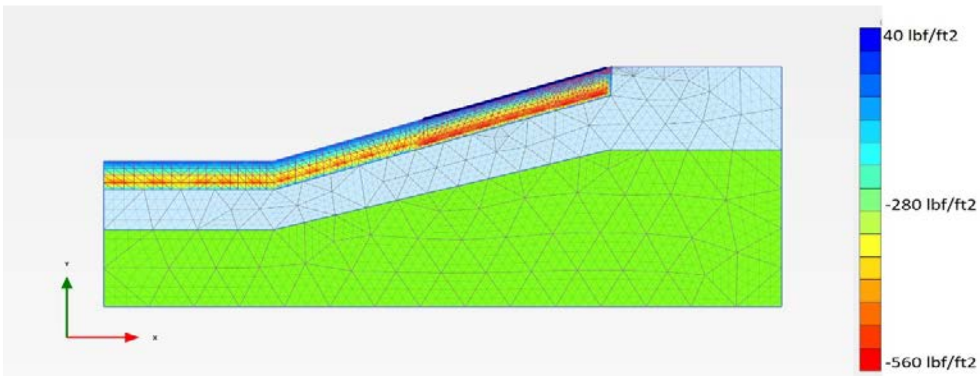


Figure 4.4 Distribution of pore water pressure

The slope stability analysis was conducted using the soil model presented in Figure 4.2. Also, a perched water zone was applied to the topsoil layer to evaluate the effect of rainfall on the safety of the slope with the presence of desiccation cracks (Figure 4.4). The depth of the perched water zone was about 3 m throughout the top portion of the slope. The slope stability analysis results included the effect of a perched water condition, presented in Figure 4.5. As indicated in the figure, the factor of safety for the slope with fully softened shear strength (case 2) reduced to 1.05 when the perched water condition was considered. This reduced factor of safety was very close to failure. Therefore, the slope failure took place due to the combined action of the perched water zone at the steep part of the slope and the fully-softened condition resulting from seasonal wet-dry cycles.

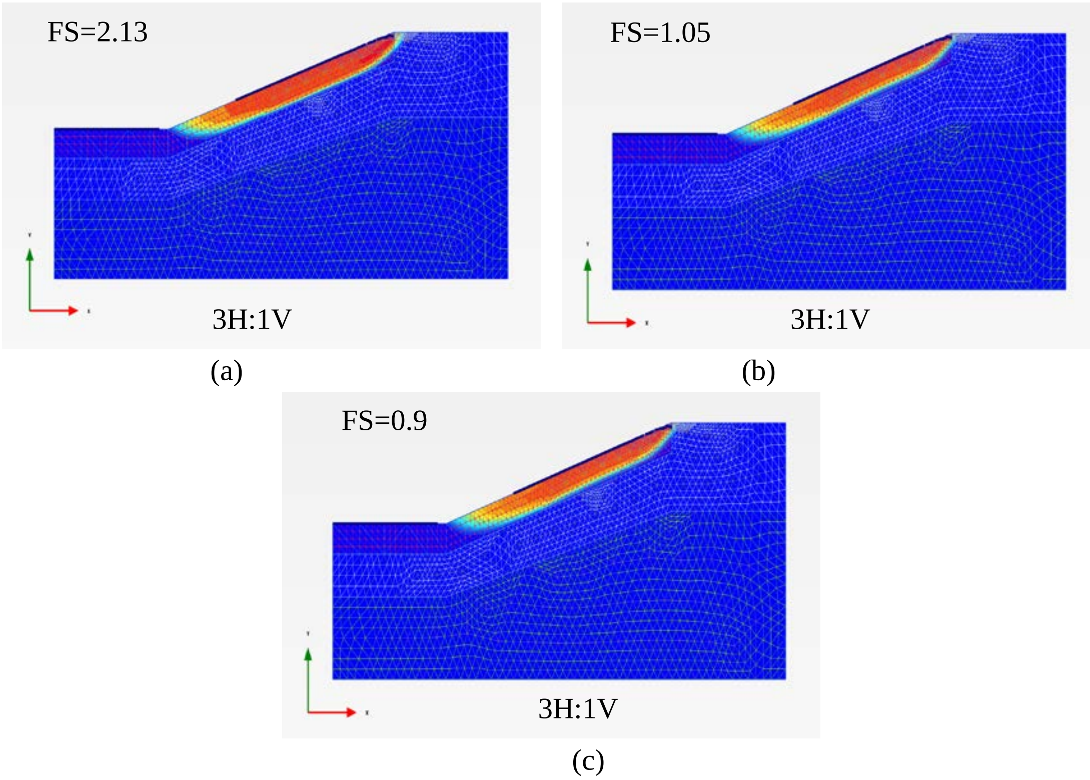


Figure 4.5 Total displacement changes including perched water zone, 3H:1V (a) Case 1 (b) Case 2 (c) Case 3

Figure 4.6 shows the progressive variation of the factor of safety for cases one to three, comparing both dry and perched water zone formation conditions. From the mentioned figures, it can be observed that the factor of safety reached a minimum value after the topsoil layer shear strength was replaced with the residual value. Before any rainfall event, the factor of safety of the slope was 2.4 which decreased to a value of 1.9 (20 %) after case one was applied. On the other hand, this value decreased to 1.2 (50%) after case two was applied. More analysis of the perched water zones showed that initially, the safety factor was 2.13, which reduced to 1.05 (51 %) after case one was applied. Similarly, this value decreased to 0.9 (58%) after case two was applied. It is therefore shown that during a longer period, the soil shear strength parameter decreased from peak to residual. This reduction in soil strength has a detrimental effect on slope stability.

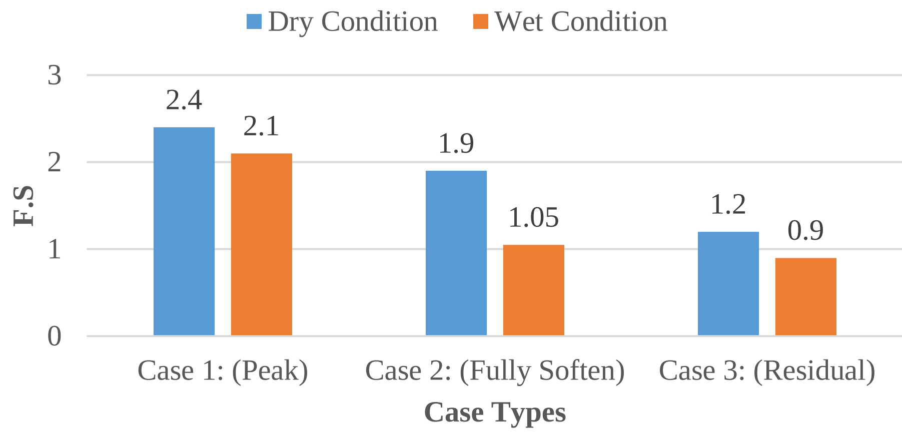


Figure 4.6 Progressive change in Factor of safety

4.2 Effect of Rainfall on Slope Failure

NOAA collects the rainfall data all over the US and develops the precipitation pattern of any locality based on the historical data (NOAA Atlas 2014). The PDS based intensity duration and frequency (IDF) curve of precipitation, based on NOAA Atlas 2014 of Jackson, Mississippi, was collected. In this parametric study, 100 years return period opts, which indicated that the high-intensity rainfall ranges between 2.79 in/hour (for 30 mins. duration) and 20.3 in/hour (for 30 days duration).

The different volume of rainfall (4.97 in and 10.7 in) and duration of rainfall (2 hr. and 3 days) are selected, based on a return period of 100 years from the PDS based IDF curve of Jackson, MS. Moreover, the fill slope constructed on Yazoo clay soil with slope ratio of 3H: 1V is selected. The FEM matrix for the selected slope inclinations and rainfall intensities are presented in Table 4.4.

Table 4.4 Selected Precipitation pattern for FEM analysis

Rainfall Duration	Rainfall Volume (inch/hr)	Rainfall Volume (mm/hr)
30-min	2.79	70.86
60-min	3.88	98.55
2-hr	4.97	126.2
6-hr	7.04	178.81
12-hr	8.17	207.51
1 day	9.22	234.18
3 days	10.70	271.78

The FEM program PLAXIS 2D was used to conduct the coupled flow-deformation analysis. A 15node triangular element was used, which provides a fourth order interpolation for displacements, and the numerical integration involves twelve Gauss points. The Van Genuchten model is considered as the hydraulic model. The soil parameters, as shown in Table 4.3, were used in the numerical analysis using PLAXIS 2D. They were established from existing soil test reports. Precipitation of different intensities was applied to the soil model to assess the flow behavior during rainfall. The analysis was carried out at three rainfall intensities. The rainfall intensities were selected based on 100-year periods of Mississippi rainfall data.

In this numerical analysis, four different cases (Case i, Case ii, case iii, and Case iv) are considered for values of the shear strength of fully softened of the topsoil layer of the soil above layers profile. For the case one, the peak shear strength values are assigned for the topsoil strength. For the case two, the shear strength related to the three wet and dry cycles shear strength values are assigned for the topsoil strength. For the case three, the shear strength related to the five wet and dry cycles shear strength values are assigned for the topsoil strength. Finally, for the case four, the shear strength related to the seven wet and dry cycles shear strength values are assigned for the topsoil strength. This study aims to evaluate the progressive change in the factor of safety in topsoil layer considering four different cases with four different values in soil shear strength regarding the number of wet and dry cycles.

The flow through the topsoil was determined for each of the intensities assuming rainfall durations lasting 30 min, 60 min, 2 hours, 6 hours, 12 hours, 1 day, 3 days and 7 days. The representative soil model is presented in Figure 4.7. The boundary condition as outlined in the mentioned figure is infiltration for topsoil which allows simulating water ponding at the topsoil. During the dry period, the highly plastic clay soil developed desiccation cracks which might have significantly increased the permeability along the vertical direction of the topsoil at the active zone. However, due to the desiccation crack, the permeability along the horizontal direction might have did not affect and could have remained unchanged (Khan et al. (2017)). Therefore, a high vertical permeability value of k_x=k_y= 8.194 ft/day (0.034 cm/sec) was used for the top mentioned part for each one of slopes to simulate the effect of the desiccation crack, as selected from CPT. In other clay layers, the permeability for both horizontal and vertical directions are selected as 3.861 ft/day (0.0013 cm/sec) and 0.0087 ft/day ($3.06 * 10^{-6}$ cm/sec) respectively. The water table was placed at 3 m (10 ft), below the ground surface.

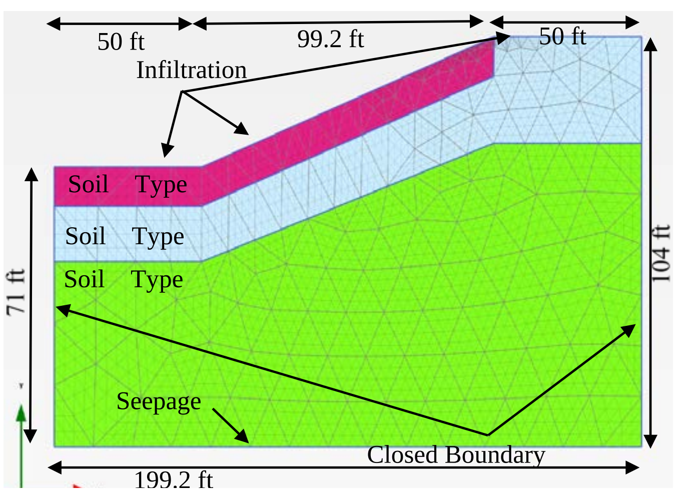


Figure 4.7 The boundary condition for the soil model

4.2.1 *Flow Analyses Results*

The variations of suction at the slope with 3H: 1V ratio for the initial phase, 30 min, 60 min, 12 hr, and 3 days rainfall intensities are presented in Figure 4.8 (a) to (e). The mentioned figure is for 126.2 mm/hr rainfall intensity (RI) for the case we have fully softened shear strength with 3 number of the wet and dry cycle for the soil layer one. As indicated, that the suction immediately dropped at the toe of the slope after rainfall and continued to drop during rainfall, representing the accumulation of water at the corresponding depth. It is also observed that the suction increase has continued for few hours, even after the rainfall. It is also noticed that after few days of rainfall, the suction had increased, and it almost regained its original profile for the top part. This trend of the suction profile is similar to other cases that we have different shear strength with 5 and 7 wet and dry cycles.

It should be noted that during the FEM analysis, infiltration boundary was used at the topsoil, which allowed ponding of water to simulate realistic behavior. The ponding status is established when the rainfall intensity is equal to the infiltration capacity. From FEM analysis results, it can be seen that ponding occurrence exists in almost all surficial soil for the toe part of the slope with different rainfall intensities. However, for 12-hrs and 3 days rainfall intensity, ponding condition can be found hardly at the slope, due to the low intensities. In particular, ponding has affected the matric suction according to the mentioned rainfall intensities. In other words, ponding occurrence decrease amount of suction at the topsoil at the toe.

Based on Figure 4.8, the change in suction was significant during the period. The maximum change in suction was observed within 3 days of rainfall duration. For example, the matric suction value is about 2400 lb/ft^2 (114.9 kN/m^2) for 3H:1V slope, which is higher than the matric suction values of lower rainfall duration. As is seen, the matric suction value of the crest raised from 600 lb/ft^2 to 800 lb/ft^2 (28.73 kN/m^2 to 38.3 kN/m^2) on the first hour of rainfall. The suction was observed to increase with higher intensity and longer duration of rainfall, and it continues with slight changes. Moreover, the change in suction was more significant at the crest of the slope, when compared to the middle and toe of the slope. The drop of suction was integrated at the toe of the slope for the depth for different rainfall intensities and durations. In contrast, the change in suction continued several days to weeks of post-rainfall to reach a steady value at the crest.

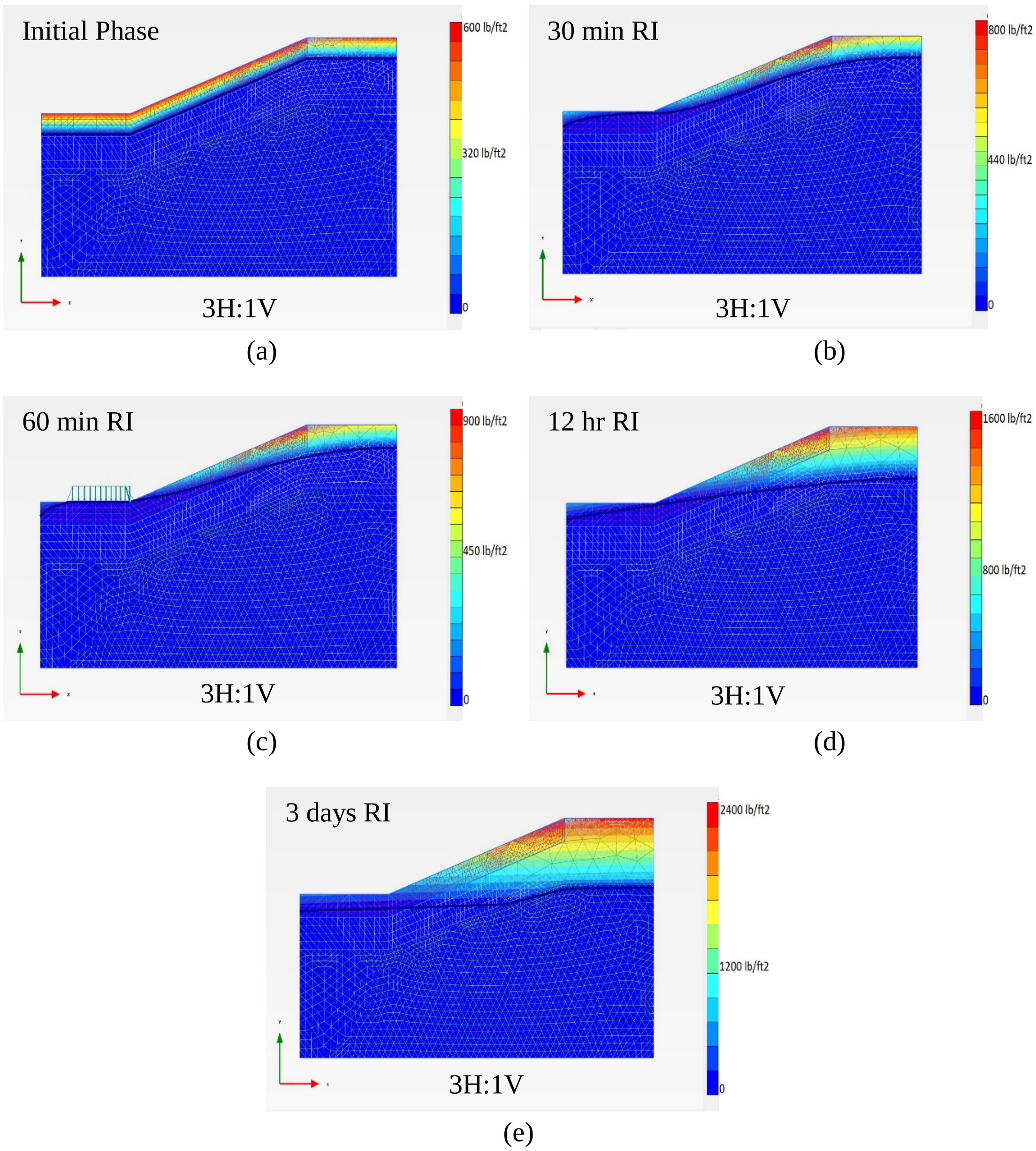


Figure 4.8 Suction profile for 126.2 mm/hr rainfall intensity with 3 number of wet and dry cycles of shear strength in the top first layer (a) before rainfall (b) during rainfall-30 min (c) during rainfall-60 min (d) during rainfall-12 hr (e) after rainfall-3 days

4.2.2 *Stability Analysis Results*

The factor of safety of a slope is defined based on shear strength reduction method as the factor in which the original shear strength parameters can be reduced to bring the slope to the point of failure (Griffith and Lane (1999)). During this study, stability analysis was conducted considering the unsaturated moisture and matric suction variation of the soil, as investigated by the fully-coupled flow analysis. The slip surface of 126.2 mm/hr rainfall intensities with one-day rainfall duration for the slope ratio 3H: 1V considering four cases are presented in Figure 4.9. Based on FEM results, it can also be observed that the factor of safety changed with different rainfall durations.

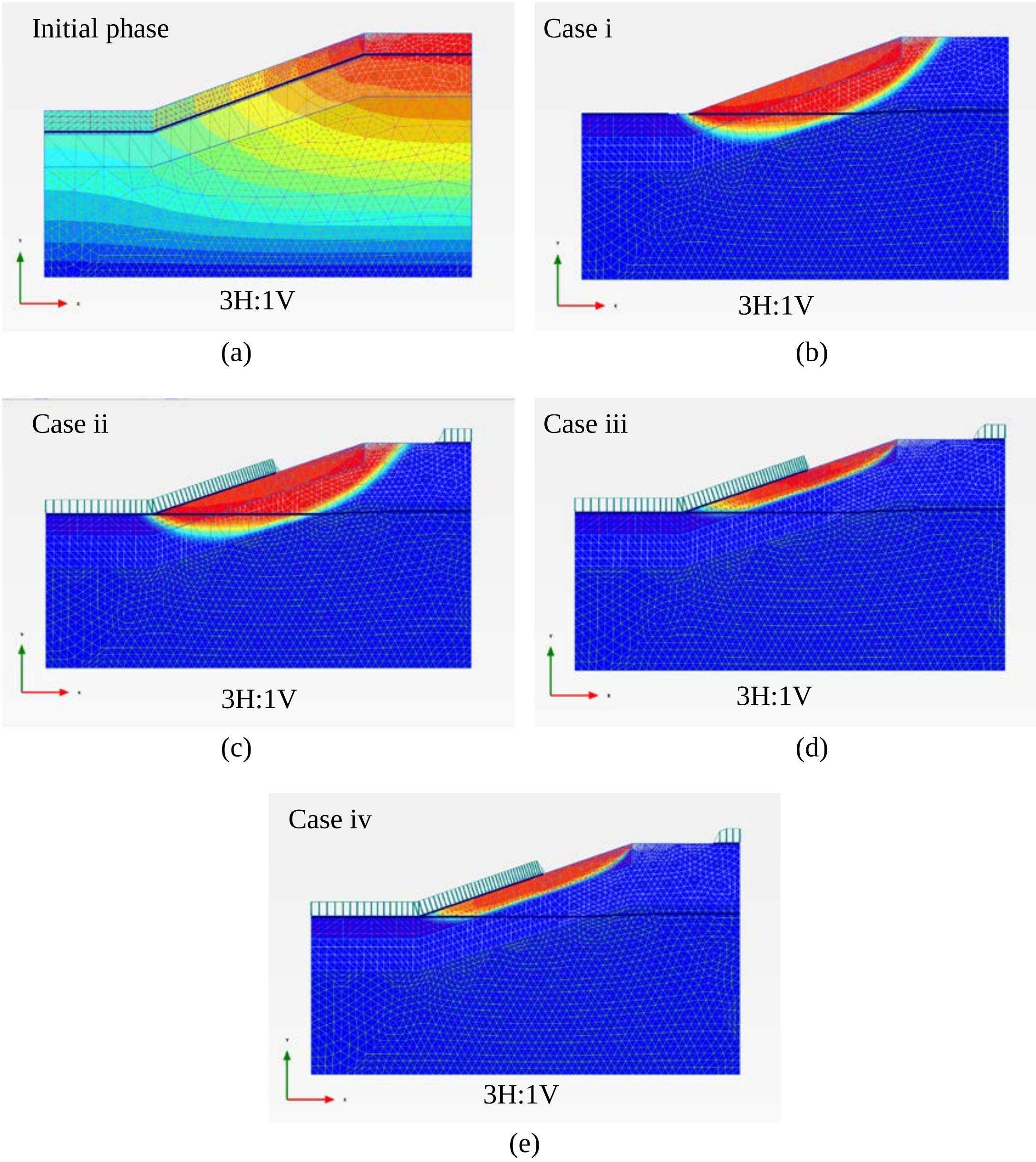


Figure 4.9 Total displacement change for 126.2 mm/hr rainfall intensity for one day rainfall duration for 3H:1V slope ratio (a) prior to rainfall (b) case i (c) case ii (d) case iii (e) case iv

Figure 4.10 shows the change in slip failure surface for case iv. In this part, the soil shear strength is with seven wet and dry cycles, and the factor of safety varied from 1.28 to 1.2. As is seen, the soil strength in the top layer with seven cycles of wet and dry decreased and it is very susceptible to failure. This reduced factor of safety was very close to failure. Therefore, the slope failure took place due to the reduction in fully softened shear strength with an increase of a number of wet and dry cycles at the steep part of the slope and the fully-softened condition resulting from seasonal wet-dry cycles.

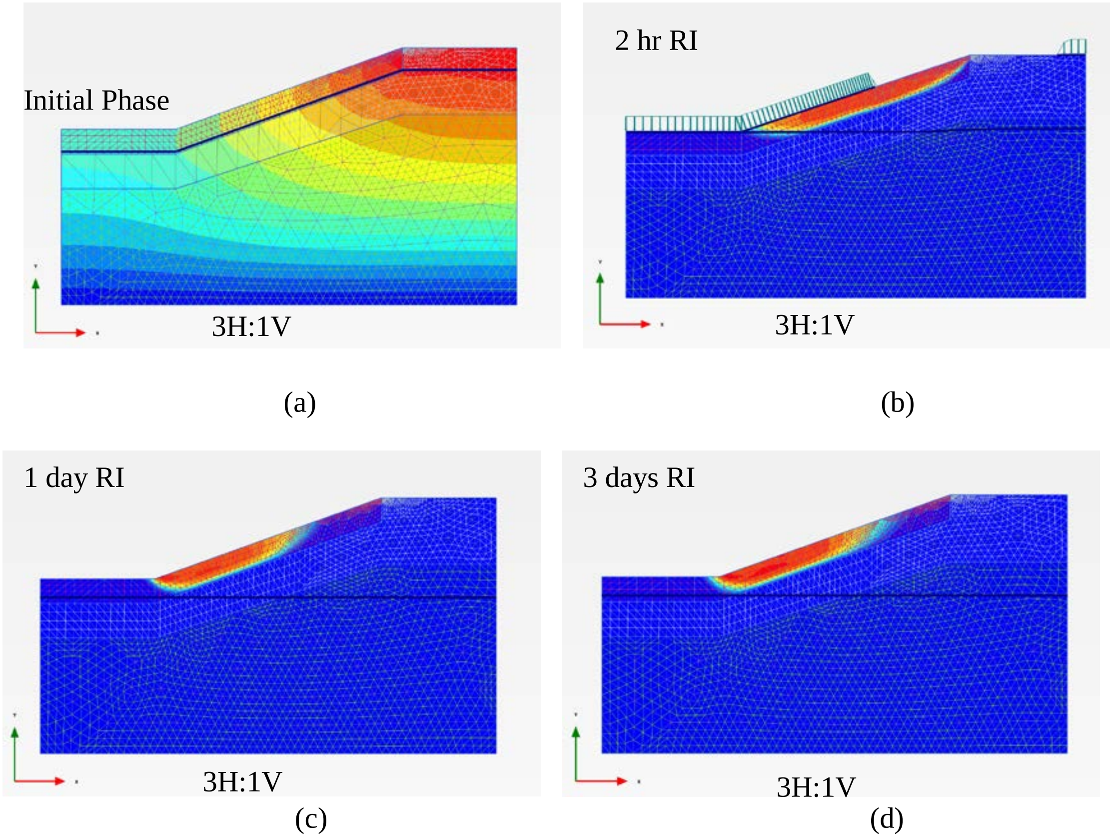


Figure 4.10 Case iv total displacement change for 126.2 mm/hr rainfall intensity for 3H:1V slope ratio (a) prior to rainfall (b) 2 hr rainfall duration (c) 1 day rainfall duration (d) 3 day rainfall duration

The factor of safety of the slope ratio 3H:1V with consideration of the mentioned four cases were presented in Figure 4.11. The mentioned figure shows the progressive variation of the factor of safety for cases one to four. The amount of factor of safety indicates that the slope is not stable in fully-softened condition with the presence of matric suction at the crest of the slope. Also, the failure surface is observed as deep-seated in case I and it changed to shallow failure surface in cases ii to iv which occurred due to the presence of matric suction. As the rainfall in influencing the matric suction value at the topsoil, almost a large change in the factor of safety occurred in shallow slip surface failure. Consequently, From the mentioned figure, it can be observed that the factor of safety reached a minimum value after the topsoil layer shear strength was replaced with the seven wet and dry cycles value.

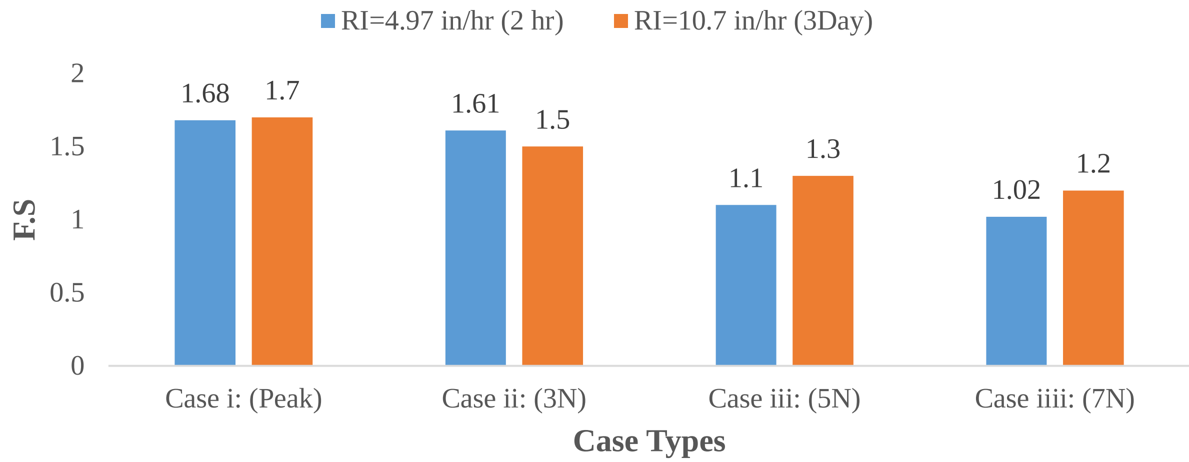


Figure 4.11 Progressive change in Factor of safety

Chapter 5: IMPACTS AND BENEFITS OF IMPLEMENTATION

5.1 Relevance to the needs of Mississippi

The existence of Yazoo clay soil in Mississippi frequently causes many failures in highway slopes, embankments and maritime waterway infrastructure, such as levees, due to the shrink-swell behavior and excessive rainfall. Each year, fixing the slope failure requires significant maintenance budget. Through this study, understanding of the slope failure mechanism will improve the design practice to implement useful slope stabilization technique, which will enhance the effective use of the slope maintenance budget.

5.2 Implementation of Results

Expansive soils cover more than 25% of the total area of the United States and are responsible for premature shallow slope failure of highway fill slopes, levee, dam, and embankments. According to Federal Highway Administration (FHWA), expansive soils are a very significant problem in many parts of the United States and are responsible for the application of premature maintenance and rehabilitation activities on many miles of roadway and maritime infrastructures each year. Investigation of the failure mechanism, the laboratory testing and the finite element analysis results identified the critical condition of the slope failure in Mississippi. The test results of the wet-dry cycles investigate the progressive changes of the shear strength and worst-case scenario of the slope failure. This test data will help to identify when the slope failure on expansive soil may occur, which will help to manage their maintenance budget better to restrict/repair slope failure.

Chapter 6: RECOMMENDATIONS AND CONCLUSION

Slope failures are typical and recurring in most of the highway slopes and maritime waterway infrastructure such as Levees in Mississippi due to the existence of the highly plastic Yazoo clay soil. Based on the results of the direct shear tests performed in this study, the following conclusions are advanced:

1. Peak drained shear strengths apply to first-time slides in natural and excavated slopes of stiff fissured over-consolidated material where there has also not been any prior large shear displacement. The Peak shear strength is also applicable to compacted slopes of high plasticity clays and shales exposed to wetting and drying.
2. Fully softened shear (critical) strengths should be used when analyzing slope stability for first-time slides in overconsolidated clays as it serves as a border strength between stability and failure.
3. The results of direct shear tests conducted on the consolidated residual soils investigated showed a reduction in strength with accumulated displacement beyond the peak strength.
4. The data generated in this study were found to be in general agreement with the trend of decreasing residual friction angle with increasing plasticity index reported in the literature.
5. It is evident that the wet-dry cycles reduce the shear strength of soil. This changes in the shear strength should be considered for the design and construction of fill embankment with marginal high plastic clay soil.
6. The numerical analysis presented in this study focused on observing the progressive change in the factor of safety. Based on the numerical analysis results, it is evident that the progressive change in the shear strength caused gradual changes in the factor of safety of the slope. Moreover, a fully softened condition with a presence of perched water, up to 3 m deep, initiated the slope failure.
7. The numerical analysis on the progressive changes in the factor of safety indicated that with 5 to 7 number of wet-dry cycles, the shear strength of the slope reduces significantly. Moreover, with the presence of rainfall, the slope is subjected to fail.

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