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**Laboratory Investigation on Engineering Properties of Soil Due to
Cyclic Drying and Wetting**

**by
Samundra Kandel**

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ABSTRACT

Due to world-wide distributions and extensively used as construction materials, geotechnical engineers are interested in understanding the Engineering behavior of expansive soils which are sometimes referred in the literature as problematic soils. The climatic zones where expansive soils occur are often experienced by many Drying-Wetting cycles due to seasonal variations. This seasonal variation in the water content termed as Drying-Wetting cycle is regarded as the most destructive environmental factor that may cause many foundation diseases. Considering these, the purpose of this study is to understand the effects of drying-wetting cycles on shear strength(C, ϕ) characteristics of undisturbed soil. A series of unconfined compression test, CBR, Permeability tests are conducted on multiple drying-wetting soil specimens to analyze these engineering properties. The results of the tests conducted indicate about 22 % reduction in cohesion from 54.190 kN/m^2 to 42.342 kN/m^2 until fifth cycle of drying and wetting and a constant value of nearly equal to 42 kN/m^2 thereafter. Also the study of change in CBR of the soil specimen due to cyclic drying and wetting was conducted and found to decrease with the increase in number of cycles and vice versa in the case of coefficient of permeability. Dry density, Degree of saturation, Bulk density goes on decreasing with increase in number of Drying and Wetting cycles and obtain the constant value after the 5th cycle of drying and Wetting. However, void ratio increases with the number of Drying and Wetting cycles due to development of cracks.

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LIST OF ABBREVIATIONS

A	Activity of Soil
A _i	Cross Section Area of Soil Specimen after Different Deformations
C	Cohesion of Soil
CBR	California Bearing Ratio
C _c	Compression index
E	Initial Void Ratio
E _i	Axial Strain at different deformation during unconfined test
I _p	Plasticity Index
K	Coefficient of Permeability
LI	Liquid Limit
OH	Organic Soil of Medium to High Plasticity
OL	Organic Silt of Medium to Low Plasticity
PL	Plastic Limit
q _u	Unconfined Compressive Strength
SPT	Standard Proctor Test
S _t	Sensitivity
S _u	Undrained Shear Strength
UCS	Unconfined Compressive Strength
Ø	Internal Friction angle of soil

CHAPTER ONE: INTRODUCTION

1.1 Background

Expansive soils are those soils which have a tendency to shrink or swell with a change of moisture content. Expansive soils are predominant in many parts of the world. The variation of water content leads to cracking of the structures which are built on them. These problems due to expansive soil cause billions of dollars for their repair annually [Nelson & Miller]. These expansive behaviors not only cause the loss of property, but also loss of lives. The state Texas was used more than one billion dollar for the restoration of the foundation of the building, buried utilities, highway and Pavements of airports which were constructed on expansive soil. These soils are called regur soil in some part of the world.

Wetting and drying of the expansive soil causes the shrinkage and swelling of the soil which causes progressive damage to the structures. The influence of wetting and drying cycles has been well shown by Popesco, 1980, Chen and Ma, 1987, Subbarao and Satya Das, 1988, however the wetting and drying cycle does not affect the chemically stabilized expansive soils. To my best efforts in search of studies of effect on soil due to cyclic wetting and drying in Nepal, no such literatures have been found yet. Thus, this thesis is expected to be helpful as a literature related to such behavior of soil in context of Nepal.

The roads of Nepal have experienced a number of failures in embankments constructed of expansive soil. Failures of this type are especially frequent in embankments, roads, buildings constructed of the expansive soil. Effective stress shear strength parameters for the Expansive soil have been measured in previous laboratory studies and have been found to correspond to significantly higher strengths than those back-calculated for field conditions at the time of failure.

This report presents the results of a laboratory testing program which was initiated on 3rd, Jestha, 2073 to assess the possibility that the observed discrepancy between laboratory and apparent field strengths of the Expansive soil is due to repeated cycles of wetting and drying which occur in the field. Preliminary studies were necessary to develop procedures for preparation of compacted specimens and subjecting the specimens to repeated cycles of wetting and drying. The procedures and effects of wetting and drying on the moisture content void ratio, dry density, UCS, Cohesion,

coefficient of permeability and CBR value of specimens are to be summarized in this study. UCS tests, Permeability test, CBR tests are to be performed on specimens subjected to repeated cycles of wetting and drying; results from UCS tests, CBR tests, Permeability test are presented in this thesis work. The shear-strength parameters, CBR value, Coefficient of permeability value for different drying-wetting cycle are obtained from the UCS tests, CBR test, falling head permeability tests and are compared to each other and to the values measured previously in the laboratory for specimens which were not subjected to wetting and drying.

1.2 Importance of the study

According to the referenced literatures, a study published in 2009, attributed thirteen billion dollars (not adjusted to 2013 dollars) of annual damage to infrastructure in the United States to problems associated with expansive soils (Puppala and Cerato 2009). One-third of the damage was reported to appear in residential and commercial buildings (Wray and Meyer 2004). The remaining damage was done to roadways, bridges, and dams. Prior studies including Jones and Holtz (1973), Krohn and Slosson (1980), and Steinberg (1998) show, as a general trend, that the estimated damage associated with US infrastructure has increased with time. Despite of research gap in Nepal, such study related to damage of structures due to expansive soils have not been found yet, we can relate and learn the possibilities of damage from the cases of other countries.

The cost of repair of such infrastructures associated with expansive soils can be high alarming and thus identification of the behavior of such soils is important to avoid such mishaps.

Naturally, during monsoon and dry season, most soils on the top layer changes their physical state from wet to dry conditions. This can be attributed to repeated drying-wetting cycle when number of year passes. Several authors have shown that the influence of the first drying-wetting cycle on soil structure is greatest and decreases with subsequent cycles. Most of the previous investigations on the effects of multiple drying-wetting cycles on engineering properties are based on expansive soil or artificial soils (soil mixed with fly ash, cement, lime or organic polymers) or reconstructed soils for the purpose of slope stability analysis or soil solidification or stabilization. But the effects of multiple drying-wetting cycles on shear strength, CBR

value, Coefficient of permeability (k) of expansive soils are not well understood yet. Hence, it is utmost that a research on drying-wetting effect on the expansive soil is to be conducted for the safety of the structures built on it.

1.3 Behavior of Problematic Soils during seasonal variation

Expansive soils are soils that have a potential for shrinkage or swelling under changing moisture conditions (Nelson and Miller 1992). Changes in the moisture conditions of an expansive soil are due to the environmental conditions imposed at the ground surface, vegetation and groundwater table fluctuations. The environmental conditions imposed at the ground surface, may either increase or decrease the moisture content. An increase in the moisture content may be due to precipitation while a decrease in the moisture content may be due to elevated temperatures. During long periods without precipitation, an expansive soil will dry and cause the soil to shrink generally leading to both recoverable and irrecoverable negative volume change. When precipitation occurs after periods of no precipitation, the infiltrated water wets the expansive soil. The recoverability of the volume change in an expansive soil is dependent on the applied stress state and the stress history of the soil. The irrecoverable volume change is known as plastic strain and the recoverable volume change is known as elastic strain. The irrecoverable volume change is due to permanent changes in a metastable soil structure that resulted from the geological deposition of the soil.

In a laboratory setting, the volume change behavior is generally assessed by testing undisturbed or remolded samples for heave and collapse potential from ASTM D4546-08. Remolded samples are used only for assessing positive volume change or heave potential. To assess the heave/expansion potential of an undisturbed sample, the sample is first loaded to a predetermined net normal stress and the sample is inundated with water. The volume change that occurs due to the application of the normal stress and the inundation of water is recorded. The extent of the volume change is dependent on the initial stress state (i.e. density/moisture content and net normal stress). Expansive soils will exhibit positive volume change (gain in volume or swelling) when water is added to the soil as long as the applied net normal stress does not exceed a threshold pressure usually referred to as the swell pressure (i.e. applied net normal stress to achieve zero volume change). Also CBR, UCS and

Permeability test conducted in Drying-Wetting cycle results in reduction in UCS and CBR value and reverse result in case of permeability test.

When undisturbed samples are not collected and an assessment of engineering properties is needed for a borrow site (i.e. a location of suitable fill material for use at another site), remolded samples are used in a laboratory setting. The compaction specifications for the borrow material (e.g. moisture content range and minimum percent of the standard Proctor maximum dry density) and maximum swell potential is often governed by code specifications (i.e. city, county, state or engineering specifications). After the compaction specifications are determined, a representative sample of the borrow soil is then remolded, loaded to a specified net normal stress, and inundated with water. If swelling occurs, the volume change is recorded and the sample is then discarded after its final moisture content is obtained. However, this procedure of remolding and subjecting the soil to only one wetting cycle may not accurately assess the engineering properties and field irrecoverable and recoverable volume change behavior of the expansive soil. Therefore, the effects of successive wetting/drying on the Engineering properties of remolded or undisturbed expansive soils should be considered during the design and construction processes.

1.4 Causes of Expansive Soil Behavior

The amount of damage that expansive soils can inflict upon infrastructure has been linked to the type and the amount of clay minerals encountered in the soil (Mitchell and Soga 2005). Researchers have found that most expansive soils are comprised of clay minerals from the Smectite family (e.g. Montmorillonite) and Illite. The expansion of Smectite and Illite clay minerals is due to the large surface areas and a net negative electrical charge on the face of the clay particle. The negative electrical charge on the particle face will have a significantly larger affinity for water molecules when compared to other soil particles that do not have a net negative charge on the face (Mitchell and Soga 2005). Due to the affinity of water, soils with expansive clay minerals will be very sensitive to seasonal variation of moisture due to precipitation, evaporation from the soil surface, and/or evapotranspiration from vegetation. The hazard created by expansive soils will be the greatest in areas with pronounced wet and dry seasons (Fredlund and Rahardjo 1993). Therefore, infrastructure built in arid to semi-arid regions will be predisposed to expansive soil problems when compared to

humid regions that maintain a rather uniform soil moisture condition throughout the year (Handy 1995).

1.5 Research Objectives

The main objective of this research is to study the cyclic drying-wetting effect on the behavior of soil. The behavior shall be studied on the basis of following parameters:

- i) Coefficient of Permeability (k)
- ii) Unconfined Compressive Strength (UCS)
- iii) Cohesion of soil (c)
- iv) California Bearing Ratio (CBR) of soil.
- v) Dry Density (ρ_d)
- vi) Void ratio (e)
- vii) Degree of Saturation (S_r)
- viii) Bulk Density (ρ)

1.6 Overview of the methodology followed to accomplish the objectives

In order to accomplish the objectives of this dissertation, the following general tasks were completed:

1. Compile a literature review that described how multiple wetting and drying cycles affect the Dry density, Degree of saturation, Bulk density, Void ratio, Unconfined compression strength, CBR, and Coefficient of permeability (k), of expansive soil. Along with information on the wetting and drying cycle effect of compacted expansive clay; the literature review was to include previous research findings on the effect of shear strength, permeability, CBR value and other physical behavior of expansive soils.
2. Develop and conduct a comprehensive laboratory study to determine the engineering and index properties of soil, at multiple wetting and drying cycles.
3. Analyze the results of the comprehensive laboratory study by developing the following relationships/results obtained for each soil. These tasks were to include:
 - a. Void ratio vs no. of cycles
 - b. UCS vs no. of cycles

- c. Dry density vs no. of cycles
- d. Degree of saturation vs no. Of cycles.
- e. Bulk density vs no. of cycles.
- f. Untrained Cohesion vs no. of cycles.
- g. CBR value vs no. of cycles.
- h. Coefficient of permeability vs no. of cycles.

The flow chart of the study is as follows:

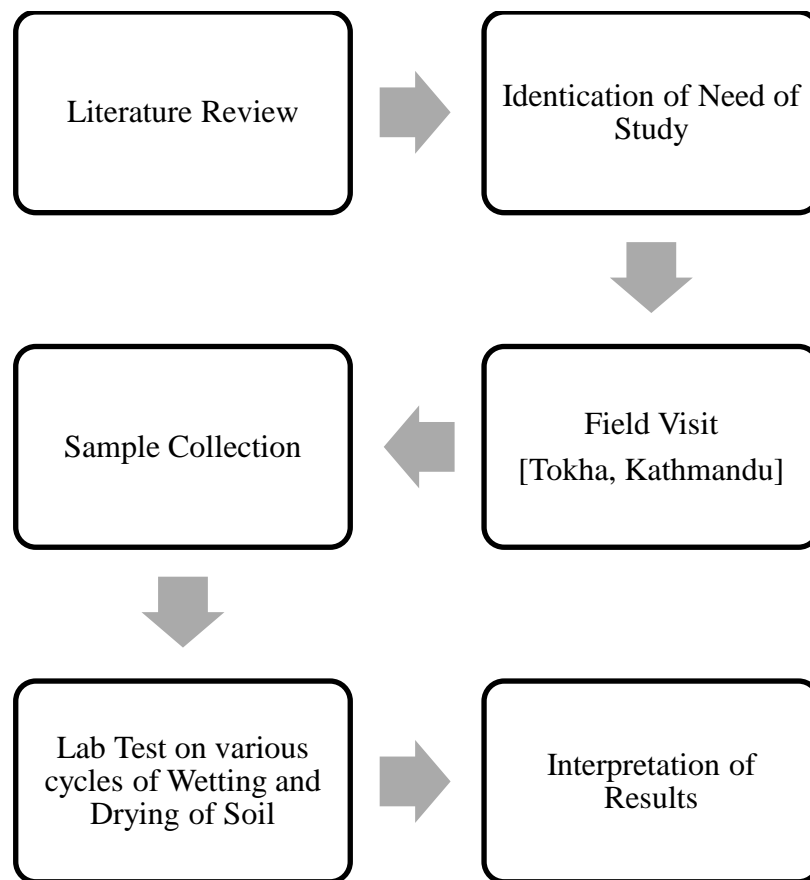


Figure 1.1: The flow chart of the study

1.7 Scope and Limitation of the study

This thesis studies about the engineering properties of soil obtain from only one site i.e Tokha, Kathmandu. The type of soil used in the study is expansive type and the results can be interpreted true only for the same type of soil. The properties studied are limited to the values of CBR, UCS, Coefficient of permeability (k), Cohesion(c), dry density, degree of saturation and void ratio. This study will be fruitful and will be

the first ever laboratory investigation in Nepal regarding the effect of multiple drying-wetting cycles. After the study, it will help to make some guidelines to know the actual discrepancy between the lab test and actual field test result.

1.8 Report Organization

Chapter 1, of this thesis includes the Introduction of the study of Cyclic drying and wetting. Importance of the study in the modern era, causes of volume change behavior of the expansive soil, causes of the expansive soil due to cyclic drying and wetting effect. Research objectives and the overview of the methodology followed to accomplish the objectives of the work are also presented in this chapter.

Chapter 2, of this report includes the Literature Survey in which the recent research on Cyclic effect of drying and wetting in various parts of the world are thoroughly studied and suitable methodology was selected to obtain the information on effect of cyclic drying and wetting on the soil at Tokha. Various engineering properties of expansive soil are also included in this chapter.

Chapter 3, of this report includes the Materials and the testing programme used to best replicate the field condition in the laboratory. The testing programme includes the index properties finding test, CBR, UCS, Falling head permeability test on multiple drying and wetting conditions.

Chapter 4, of this report includes the Result and Discussion of the laboratory test conducted on multiple number of drying and wetting cycles are presented in tabular form and the result obtained are then figured in graph and furthermore the discussion part of the result is also included in this chapter.

Chapter 5, of this report includes the Conclusion of the thesis work and nevertheless recommendation is also included in this chapter. Furthermore References and Annex are also included in this chapter.

CHAPTER TWO: LITERATURE REVIEW

2.1 Research on Effect of Drying-Wetting Cycle on Expansive Soil

Ekrem Kalkan (2011) black cotton soils having montomorollenate as main mineral which causes the vulnerable for expansive behavior and shrink properties by varying water percentage. It is also observed that wetting and dry cycle causes the cracks in soil and which leads to increase in the permeability of the soil, which causes distresses to footings, side drainages, and clay liner and side fill for waste nuclear deposits. To reduce the effect of cycles of wet and dry it is necessary to stabilize the soil with some stabilizing agent like chemicals and waste materials. In this study, black cotton soil material mixed using silica fume waste material and also observed. The natural clay sample mixed with silica fume and subjected to several wet and dry cycles and found strength improvement swelling reduction at each wet and dry cycle.

Abdulrahman H. T. Al Zubaydi (2011) on his research entitled “Effect of Wetting and Drying Cycles on Swell/Collapse Behavior and Cracks of Fine – Grained Soils” studied on the changes in swelling and collapsing properties with respect to number of wetting and drying cycles have been investigated. Also, a shrinkage cracks have been studied with aid of digital image after each drying cycle. Number of segments and area of cracks calculated with aid of AutoCAD package. Results indicated that, the collapse potential is influenced by soil type (soil composition) and applied loads. As the applied loads increase the collapse potential increases. For sandy soil the collapse potential decreased with increasing wetting and drying cycles, and for the clayey soils, swells potential decreased while collapse potential increased with these cycles. It has been shown that the cracks increase with wetting-drying cycles. Larger values of percent crack area to the initial sample area has been observed in the soil that contain more clay content than other types of soils.

Wang et.al.(2014) on their research entitled “Research on Wetting-Drying Cycles’ Effect on the Physical and Mechanical Properties of Expansive Soil Improved by OTAC-KCl” found that expansive soil experiences periodic swelling and shrinkage during the alternate wet and dry environments, which will result in severe damage to the slope stability. In this study, a promising modifier OTAC-KCl is introduced, which has a good diffusivity and is soluble in water or other solvents easily. Firstly, a reasonable combination of a meliorant 0.3% STAC and 3% KCl is chosen referring to

the free swell test. Then, the best curing period, 14days, is gotten from UCS tests. The effect of wetting and drying cycles on engineering properties of expansive soil improved by OTAC-KCl admixtures after 14-day curing is also studied accordingly. Both treated and untreated expansive soil samples are prepared for the cyclic wetting-drying tests which mainly include cyclic swelling potential and cyclic strength tests. Experimental results show that the swelling potential of expansive soil samples stabilized with OTAC-KCl is suppressed efficiently, and the untreated soil specimens will collapse when immersed in water while the treated specimens keep in good conditions. Moreover, expansive soil samples modified with 0.3% OTAC +3% KCl show enough durability on the swelling ability, shear strength, and unconfined compressive strength, which means, that both the physical and the mechanical properties of stabilized expansive soil have been improved effectively.

Hossain Md et.al.(2016) on their research entitled “Effect of Drying-Wetting Cycles on Saturated Shear Strength of Undisturbed Residual Soils” found that the deviator stress and initial stiffness of saturated soils increase and the volume of soils becomes contractive as the net normal stress increases but decrease with increasing drying-wetting cycle numbers. The cohesion (c_d) and internal friction angle (ϕ_d) decrease with increasing cycle number (N) but the attenuation rate of ϕ_d is less than c_d . The variations of c_d and ϕ_d with respect to drying-wetting cycles can be expressed by exponential function. The saturated shear strength and its attenuation rate due to drying-wetting cycles are analyzed. The effect is more significant for the first cycle and decreases with subsequent cycles and finally reaches to a constant state after 4 cycles. Furthermore, a mathematical function is proposed in this paper which can describe the saturated shear strength attenuation rate of drying-wetting cycle samples. Such studies are useful to understand the possible changes in shear strength behavior of residual soils below the engineering structures that are subject to periodic drying and wetting from climatic variations.

A.R Estabragh et. al (2014) on their research entitled “Laboratory investigation of the effect of cyclic wetting and drying on the behavior of an expansive soil” studied the behavior of an expansive soil through a number of experiments involving cycles of wetting and drying. They showed that the shrinkage was reversible after the soil reached the equilibrium condition where the deformations were the same and the wetting and drying paths (in the water content-void ration space) converged to an S-

shaped curve. They further showed that the samples with lower initial water content (on the dry side of optimum) have more swelling potential than samples with a higher initial water content (on the wet side of optimum).

Ziaie Moayed et.al (2013) on their research entitled “The Influence of Freeze-Thaw Cycles on CBR Values of Silty Soils Stabilized with Lime and Microsilica” studied the behavior of silty soils on CBR value through number of experiments involving cycles of freeze and thaw. They showed that Freezing- thawing cycles caused reduction in CBR values of the desired sample. But almost 90 percent of the decrease was occurred at the first cycle and there were no significant decrease happened on the CBR values on the next cycles.

M. T. Rayhani et.al (2008) on their research entitled “Physical modeling of desiccation cracking in plastic soils” studied the large scale experimental models of various natural clayey soils with various plasticity indices which were subjected to cyclic drying and wetting and hydraulic conductivity testing to better understand cracking behavior and self-healing in fine-grained soils. Experimental models in which cracks formed during drying were tested for soil hydraulic conductivity. The results indicated that cracking and hydraulic conductivity of clays are controlled by soil properties, especially plasticity and swelling. Cracking of the specimens resulted in an increase in hydraulic conductivity, sometimes as large as five to ten orders of magnitude. The hydraulic conductivity of highly plastic clays increased with an increase in number of drying and wetting cycles and then remains constant from 4th cycle of drying and wetting due to self-healing.

Schaefer V.R. (2015) on his research entitled “Mechanisms of Strength Loss during Wetting and Drying of Pierre Shale” , found significant fabric contrast after three cycles as the material’s structure became more massive. The most noticeable difference in the cycles was the particle settlement rates. Excessive cycling caused particles to stay up in suspension for weeks to months longer than the un-cycled material. This observation indicates clay aggregates are becoming smaller and going towards their unit-cell size. The reduction in size increased the clay fraction, contributing to the residual strength decrease

2.2 Engineering Properties of Expansive Soil

2.2.1 Cation Exchange Capacity

The cation exchange capacity or the CEC level is dependent on the isomorphous substitutions that occur with the clay minerals (Mitchell and Soga 2005). The isomorphous substitutions are due to tetrahedral and octahedral sheets containing cations instead of an idealized structure (i.e. aluminum in the places of silicon, magnesium instead of aluminum, etc.). When the isomorphous substitution occurs, multiple cations are replaced with other cations of other valences within the structure to maintain equilibrium within the clay structure. The ability to measure the cation replacement is computed as milliequivalents (meq) per 100 g of clay. The milliequivalents are determined by knowing the atomic weight, and the weight and valence of the element. The CEC value is a guide to estimate the predominant clay mineral. In addition, the CEC shows how stable the clay mineral is to isomorphous substitution. When the measurable CEC increases the isomorphous substitution within the clay mineral also increases. Table 2.1 outlines CEC values for common clay minerals. As one can see, Kaolinite has the lowest CEC, while Vermiculite has the highest CEC.

Table 2.1: CEC values for Common Clay Minerals (Mitchell and Soga 2005)

Clay Mineral	Meq/100 g
Kaolinite	3
Halloysite	12
Illite	25
Vermiculite	150
Smectite	85
Chlorite	40

2.2.2 Specific Surface Area

The specific surface area (SSA) is the measure of the surface area of a clay sample, which is determined by the amount of the polar molecule (i.e. glycol, glycerol, or ethylene glycol mono-ethyl ether (EGME)) retained under laboratory controlled conditions and it is then converted to SSA (Mitchell and Soga 2005). The SSA of the

sample is an indication the governing clay mineral within the test sample. Table 2.2 shows different ranges of SSA for common clay minerals.

Table 2.2: SSA values for Common Clay Minerals (Mitchell and Soga 2005)

Clay Minerals	SSA (m ² /g)
Kaolinite	10 to 20
Halloysite	35 to 70
Illite	65 to 100
Vermiculite	40 to 80
Smectite	50 to 800

As shown in Table 2.2, the range of SSA for the different minerals can overlap significantly. For example, if a soil sample has a SSA of 65 m²/g, it can be classified as Halloysite, Illite, Vermiculite, or Smectite. In this case, to define the governing clay mineralogy will require additional testing (i.e. X-ray diffraction).

2.2.3 Atterberg Limits

The concept of the Atterberg Limits was introduced to Geotechnical engineering by Arthur Casagrande in 1932. Under Casagrande, a uniform test method was developed to determine the liquid limit (LL), plastic limit (PL), and the plasticity index (PI) of soil. The liquid limit and the plastic limit correspond to different shear strengths of the soils. In 1948, Casagrande used the Atterberg Limits to create a soil classification system, which was then modified by the United Soil Classification System (USCS). Under this system, fines are classified as one of the following: non-plastic (NP), lean silt (ML), highly compressible silt (MH), lean clay (CL), and fat clay (CH). Most of the expansive soil exhibits characteristics of either a CL or CH soil.

Holtz and Kovacs (1981) overlaid different ranges of clay mineralogy on the USCS soil classification, which is shown in Figure 2.1. As one can see, both Kaolinite and Chlorite fall with the ML and MH soil classification ranges while the Illite and Montmorillonite fall within the CL and CH classification ranges.

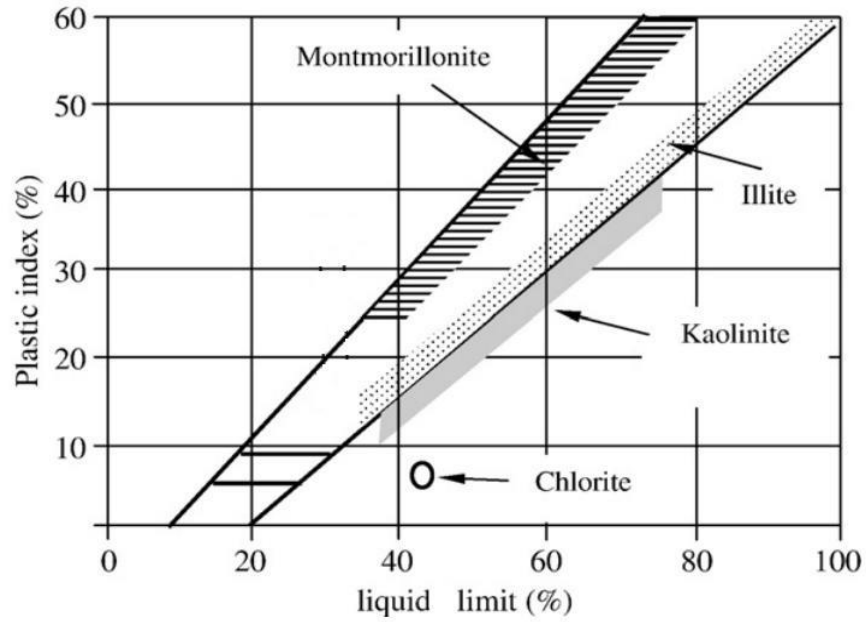


Figure 2.1: Location of clay mineralogy bands on USCS soil classification (modified after Holtz and Kovacs 1981)

Figure 2.1 is only a guideline for clay mineralogy classification based upon the Atterberg limits. There are a multitude of soils that do not fall within the clay mineralogy groupings that governing the behavior of the soil sample/stratigraphy.

2.2.4 Shrinkage Limit

The shrinkage limit of a soil is the point at which the soil will not exhibit volume change when moisture is removed from the soil sample. Shown in Figure 2.2 is a conceptual drawing of the shrinkage limit (SL).

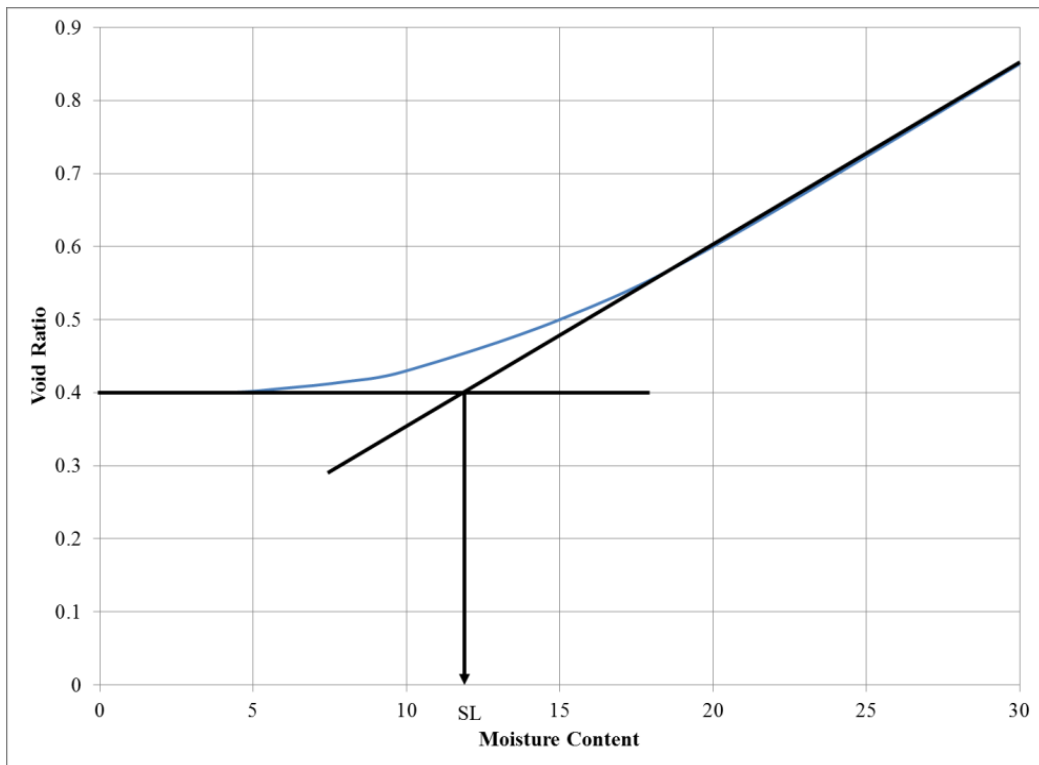


Figure 2.2: Determination of the Shrinkage Limit

2.2.5 Clay Fractions

Depending on the classification system the clay fraction size can vary. AASHTO states the clay fraction is determined by the percent passing 0.005 mm while USCS and USDA both state the clay fraction is determined by the percent passing 0.002 mm. When Skempton introduced the idea of activity the clay fraction that he specified is determined by the percent passing 0.002 mm, which is obtained from a hydrometer analysis.

2.2.6 Activity

In 1953 Skempton, introduced the concept of activity. The activity of a soil is defined by the plasticity index divided by the clay fraction, which is shown in Equation below. As the activity increases the higher the swell potential that will occur.

$$A = \frac{PI}{CF - 5}$$

2.2.7 Swelling Index, C_s

The swelling index is less than the compression index, most of the time by a considerable amount. When the soil is non-expansive the swelling index will be less than 0.1 and expansive soils will have a swelling index greater than 0.2 (Mitchell and Soga 2005). Shown in Figure 2.3 is an idealized effective stress void ratio curve for a compressible soil. Where the segment AB, in Figure 2.3, is the initial or virgin compression curve, segment BC is swelling, and segment CD is the recompression curve. Normally, when a soil does not exhibit a high degree of expansion (C_s less than 0.1) then segment BC is considered rebound. Table 4 shows typical swelling indexes for different minerals and sand consolidated under different pore fluids.

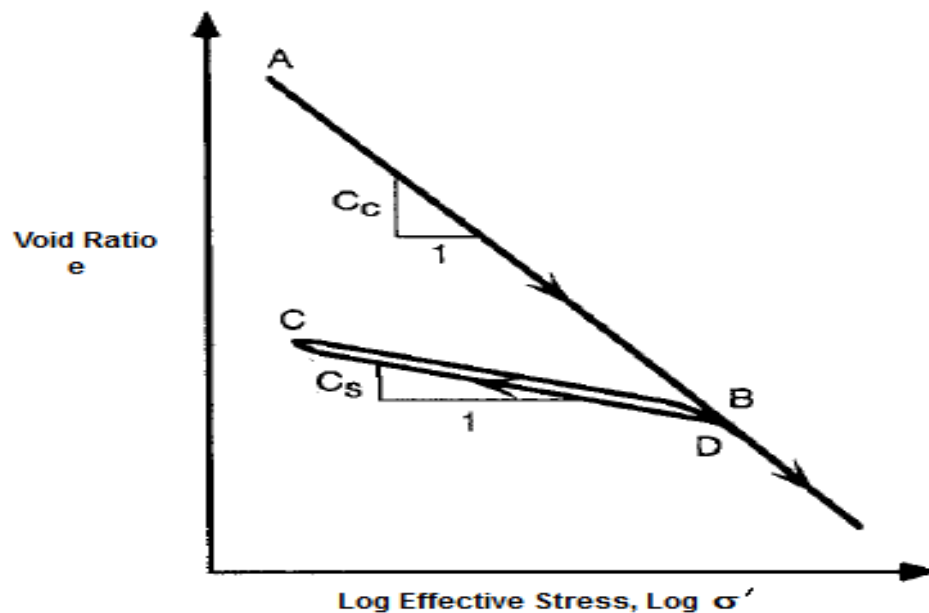


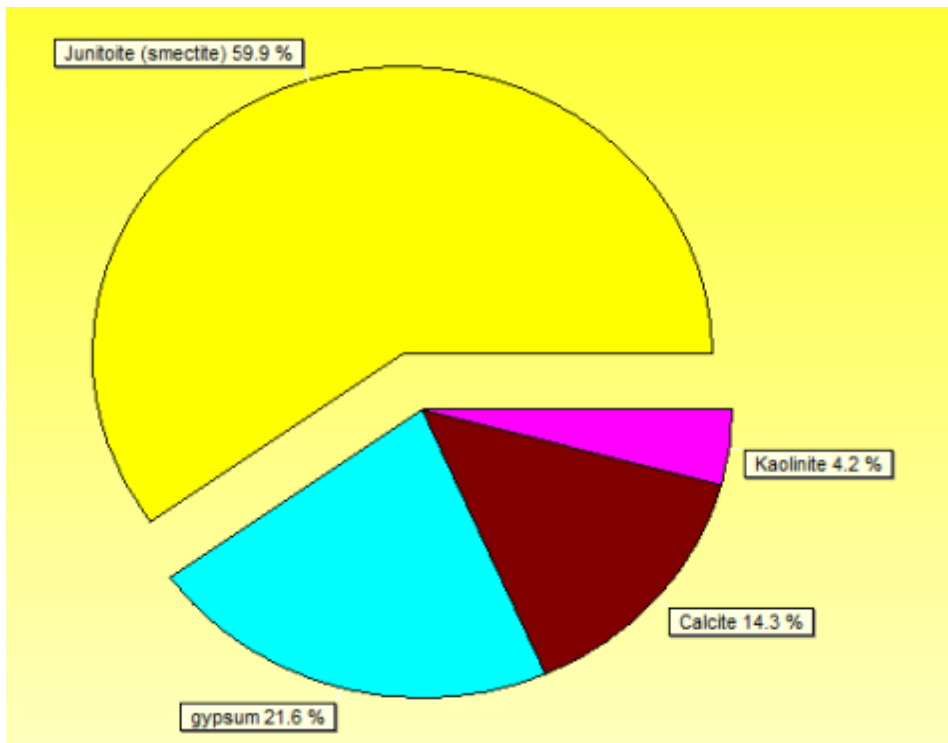
Figure 2.3: Idealized effective stress – void ratio for a compressible soil (Mitchell, J.K. and Soga, K. 2005)

Table 2.3: Swelling Indexes for Different Minerals (Olson and Mersi 1970)

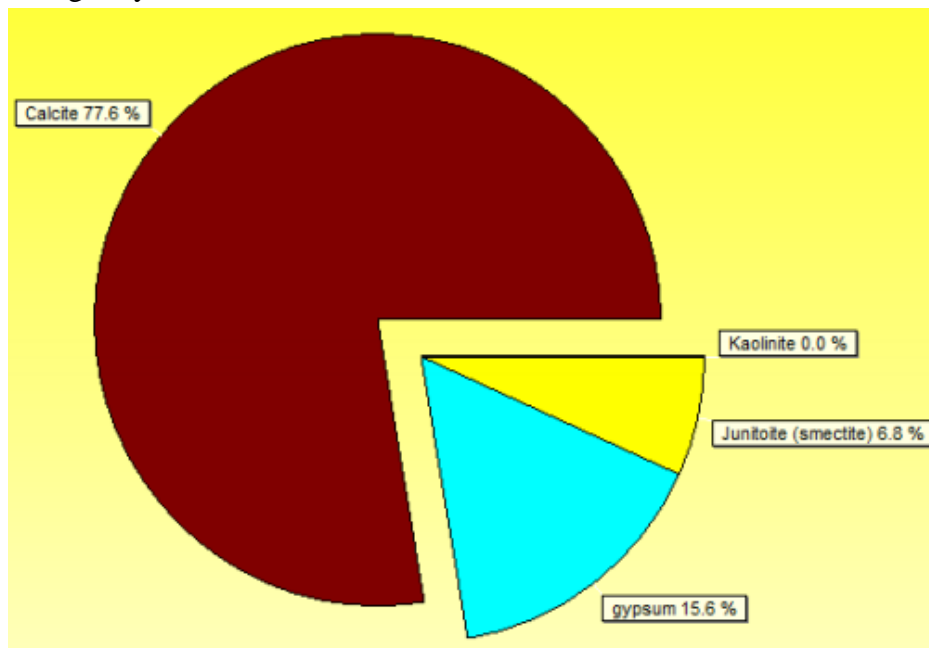
Mineral	Pore Fluid	Absorbed Cations	Electrolyte Concentration	e_0 @ 5kPa	Swelling Index
Sand	--	--	--	--	0.01 to 0.03
Kaolinite	H ₂ O	Na	1	0.95	0.08
	H ₂ O	Na	1 x 10 ⁻⁴	1.05	0.08
	H ₂ O	Ca	1	0.94	0.07
	H ₂ O	Ca	1 x 10 ⁻⁴	0.98	0.07
	Ethyl Alcohol	--	--	1.1	0.06
	Dry air	--	--	1.36	0.04
Illite	H ₂ O	Na	1	1.77	0.37
	H ₂ O	Na	1 x 10 ⁻³	2.5	0.65
	H ₂ O	Ca	1	1.51	0.28
	H ₂ O	Ca	1 x 10 ⁻³	1.59	0.31
	Ethyl Alcohol	--	--	1.48	0.19
	Dry air	--	--	1.46	0.04
Smectite	H ₂ O	Na	1 x 10 ⁻¹	5.4	1.53
	H ₂ O	Na	5 x 10 ⁻⁴	11.15	3.6
	H ₂ O	Ca	1	1.84	0.26
	H ₂ O	Ca	1 x 10 ⁻³	2.18	0.34
	Ethyl Alcohol	--	--	1.49	0.1
Muscovite	Water	--	--	2.19	0.42
	Dry air	--	--	2.29	0.41

2.2.8 Quantitative Clay Mineralogy

Quantitative clay mineralogy is determined by using various computer programs fitting the basal spacing of known clay minerals obtained from extensive X-ray diffraction (XRD) testing to the XRD signal of the soil sample. Depending on the methodology select for analysis, only certain clay minerals are selected to determine the quantitative clay mineralogy; however, in this process, three to four known clay minerals are used to determine the clay composition. When only three to four minerals are selected, some of more/less expansive minerals are left out of the quantification, which can lead to issues when comparing results from other laboratories or determination of the soil composition. Figure 5 shows an example of changing the number of minerals from four to twenty and computing the mineral composition of the sample (Shafer, Z. 2013).



a) Using only four mineral identification cards.



b) Using twenty different mineral identification cards.

Figure 2.4: Difference in clay mineralogy quantification techniques (Shafer, Z, 2013)

2.3 Guidelines of Swell Potential Determination Based on Engineering Properties

Various guidelines to determine the potential of swelling of expansive soils by measurable engineering soil properties are presented in this chapter. The most common soil properties used to determine the swell potential of expansive soils include activity, Atterberg limits, clay fractions, colloidal content, plasticity index, probable swell percent, shrinkage limit and the shrinkage index. There are numerous guidelines posted in the literature; however, the ones presented in this section are the major highlights that have been made throughout the past 50 plus years. Regardless of the swell potential classifications, it was found that once the swell potential is considered “medium” or “marginal”, the potential for a geotechnical hazard is significant.

In 1948, Skempton proposed a methodology to classify expansive potential for all types of soil. His methodology uses the percent of clay fraction (percent passing 0.002 mm) and the plasticity index. Shown in Figure 6 is the swell potential related to the plasticity index and the clay fraction.

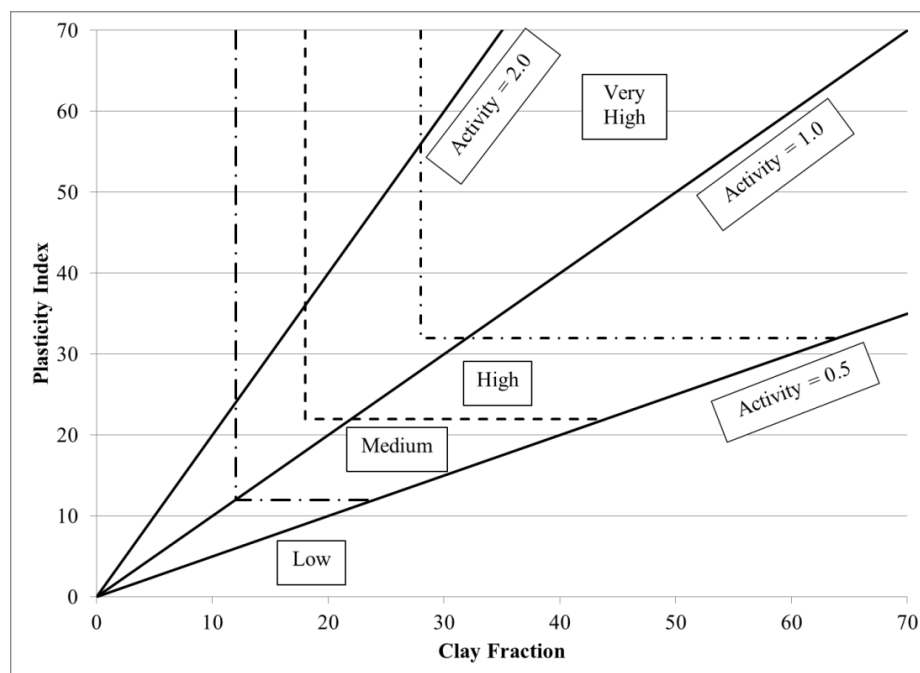


Figure 2.5: Swell potential related to PI and CF (Skempton 1953)

Table 2.4: Swell Potential Criteria per Holtz and Gibbs 1956

Swell Potential	% Swell under 1 PSI	Colloid Content	Plasticity Index	Shrinkage Limit
Low	< 10	< 17	< 20	> 13
Medium	20	12 – 27	12 – 34	8 – 18
High	30	18 – 37	23 – 45	6 – 12
Very High	> 30	27	> 32	< 10

Prior to the 1959 USBR method for determining the swell potential of questionable soils, Holtz, R.D. and Gibbs, H.J., in 1956, developed a method. This method was the basis of the USBR method. The Holtz and Gibbs swell potential criteria is shown in Table 2.4.

In 1955, Altmeyer had major criticisms of USBR method for classifying expansive soils and suggested a new method based on correlations between the linear shrinkage, shrinkage limit and the percent swell. His recommendations are shown in Table 2.5.

Table 2.5: Swell Potential Criteria per Altmeyer (1955)

Linear Shrinkage	Shrinkage Limit	Probable Swell	Degree of Expansion
< 5	> 12	< 0.5	Noncritical
5 – 8	10 – 12	0.5 – 1.5	Marginal
> 8	< 10	> 1.5	Critical

In 1965 Chen introduced a methodology for classify the swell potential, which Chen was tried to simplified the USBR method. The simplification of the USBR method was by eliminating the hydrometer analysis and replacing the colloidal content with percent passing the 200 sieve. Along with replacing the hydrometer data with sieve data, he also incorporated SPT field data and correlated the data to odometer swell data. Table 2.6 shows the laboratory and field data correlation to the degree of expansion.

Table 2.6: Swell Potential Criteria (Chen 1965)

P ₂₀₀	Liquid Limit	SPT (per ft)	Probably Expansion	Degree of Expansion
< 30	< 30	< 10	< 1	Low
30 – 60	30 – 40	10 – 20	1 – 5	Medium
60 – 95	40 – 60	20 – 30	3 – 10	High
> 95	> 60	> 30	> 10	Very High

In 1967, Terzaghi and Peck tried to relate the swell potential of an expansive soil with the plasticity index of the soil. Their results were determined by analyzing the results found from Seed, Woodward, and Lundgren 1962. The Terzaghi and Peck swell potential criteria is located in Table 2.7.

Table 2.7: Swell Potential Criteria per Terzaghi and Peck 1967

Plasticity Index	Swell Potential
0-15	Low
10-35	Medium
20-55	High
55 and greater	Very High

In 1969, Sowers first attempt to describe the potential volume change of an expansive material, Sowers only used the plasticity index. In 1970 Sowers and Sower included the shrinkage limit in describing the potential volume change. The addition of the shrinkage limit increased the accuracy of the prediction. Shown in Table 2.8 is the potential volume change per Sowers and Sowers.

Table 2.8: Potential Volume Change per Sowers and Sowers 1970

Shrinkage Limit	Plasticity Index	Potential Volume Change
>12	<15	Probably low
10 – 12	15 – 30	Probably Moderate
<10	>30	Probably high

In 1973, Dakshanamurthy and Raman proposed another method to predict swell potential using the same methodology that Casagrande outline back in 1948. Nevertheless, Dakshanamurthy and Raman used engineering judgment and posted literature values to generate the swell potential per the plasticity index – liquid limit chart. The chart that Dakshanamurthy and Raman proposed is shown in Figure 2.6.

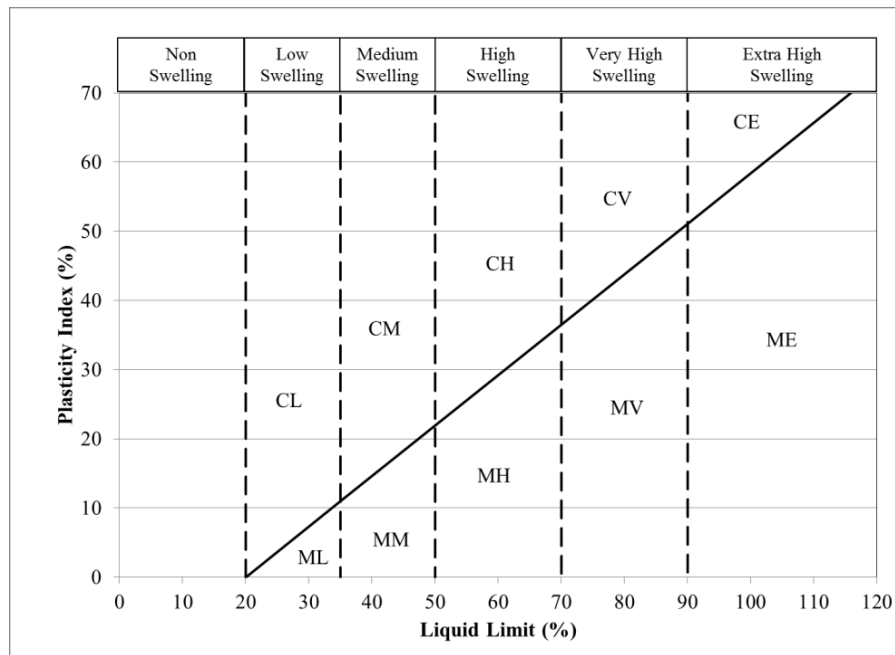


Figure 2.6: Swell potential per the plasticity chart (Dakshanamurthy and Raman 1973) Kay 1990 showed good correlation to the shrink-swell response for remolded compacted soils. His correlation for swell potential is based on Liquid Limit. The ranges for the LL correlation are shown in Table 2.9. The limits that Kay proposed are slight different than those proposed by Dakshanamurthy and Raman.

Table 2.9: Swell Potential Classification (per Kay 1990)

Soil Classification	LL Range
S (Slightly expansive)	<20
M (Moderately expansive)	20 to 40
H (Highly expansive)	40 to 70
E (Extremely expansive)	>70

As one can see the, the guidelines for swell potential ranges are different for the different authors as well as the engineering index property that is correlated to the swell potential. Nevertheless, these are the guidelines that have been set forth by the various authors have been used in by different agencies. The underlying factors that have been used for correlation include the Atterberg limits, the shrinkage limit up until the late-seventies and after the late seventies the matric suction of the soil was added. Thus from these guidelines we will be able to know the swelling and capacity

of soil sample and can know the level of expansive nature of the soil sample chosen for the study work.

CHAPTER THREE: MATERIALS AND TESTING PROGRAM

3.1 Sample preparation and Drying-Wetting Cycle

The preparation procedures include abstraction of disturbed and 14 nos. of undisturbed sample from the site of Tokha. The samples collected were at the depth of 2.5m below the ground level. Soil obtained for research work is classified as High plastic clay (CH) as per USCS classification system. The undisturbed samples were then submerged under water for 24 hours.

The schedule of cyclic drying-wetting and testing were carried out as follows:

Table 3.1: Schedule plan of cyclic drying-wetting cycle test

Day	6 th	7 th	8 th	9 th	10 th	11 th	12 th	13 th	14 th	15 th	16 th	17 th	18 th	19 th	20 th	21 th	22 th
Time	10 AM	10 AM	10 AM	10 AM	10 AM	10 AM	10 AM	10 AM	10 AM	10 AM	10 AM	10 AM	10 AM	10 AM	10 AM	10 AM	10 AM
Cycle	W	D	W	D	W	D	W	D	W	D	W	D	W	D	W	D	W
Sample no.	1	1	1	After wetting for 24 Hr, drying(40 ⁰ C) for 24 Hr, 24 Hr. rewetting and then the test to be done are:- UCS test, Falling head permeability test, CBR test													
Sample no.	2	2	2	2	2	After wetting for 24 Hr, drying(40 ⁰ C) for 24 Hr, 24 Hr. rewetting and then the test to be done are:-Falling head permeability test, unconfined compression test, CBR test											
Sample no.	3	3	3	3	3	3	3	Same as above tests									
Sample no.	4	4	4	4	4	4	4	4	4	Same as above tests							
Sample no.	5	5	5	5	5	5	5	5	5	5	5	Same as above tests					
Sample no.	6	6	6	6	6	6	6	6	6	6	6	6	6	Same as above tests			
Sample no.	7	7	7	7	7	7	7	7	7	7	7	7	7	7	7	Same as above tests	
Sample no.	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	8	Same as above tests

One of the samples was then tested for UCS following standard IS method naming it as a “0 cycle test”. Then remaining samples were oven dried at a constant temperature of 50±2 ° C for 24 hours. Once again the samples were kept under submerged condition for 24 hours and retested for UCS thus naming it as a “1 cycle test”. Similarly tests were conducted in similar fashion for 2 to 8 cycles of drying and wetting. CBR and Coefficient of Permeability test are done in remolded soil sample at optimum moisture content and is subjected to multiple number of drying and wetting cycles. These results are presented in Chapter 4.

The disturbed samples were used to find the index properties of the soil which includes water content, dry density, liquid limit, plastic limit, grain size distribution, compaction characteristics using IS codal provisions method for the classification of soil.

3.2 Testing Program

3.2.1 Determination of Specific Gravity of solids (Gs)

The specific gravity is defined as the ratio of density of the given soil solids at a given Temperature (270C) to the density of the distilled water at that temperature, both densities determined at same temperature. The specific gravity is determined by conducting the experiment by using pycnometer as per IS 2720 Part 3 Sec 2 1980. To determine specific gravity dryweight of pycnometer determined. Then 50gm soil added to the pycnometer and determines weight. Then added $\frac{3}{4}$ of water by weight and mixed by shaking. Then added water up to the mark and determined total weight then the soil was thrown away, then after cleaned pycnometer determined weight of water up to the mark level.

3.2.2 Grain size distribution

Percentage of different sizes of soil in a dry soil sample are determined by the sieve analysis which analyses in two stages, that is dry sieve analysis and sedimentation Analysis .The sieve analysis was determined if all soil particles which not passes through the Square opening 75 micron as per IS: 2720 part (IV) and the soil finer than 75 microns hydrometer analysis is conducted as per IS: 2720.

3.2.3 Determination of Liquid Limit

The liquid limit experiment conducted according to the code of IS 2720: PART 5:1985. 300 grams of soil sample was taken added water thoroughly. The paste was mixed thoroughly using knife plate for 10 minutes. By using casagrande apparatus liquid limit was determined. After mixing the soil thoroughly, the soil sample placed in the cup. A groove is formed at the center of the soil sample. By using of the crank-operated cam, the cup is lifted height of 10 cm and then dropped. The water content, to the soil, mixes to end the length of 12.7 mm along the bottom of the groove after 25 blows is defined as the liquid limit. It is very difficult to make the water content in the soil to get the required 12.7 mm closure of the groove in the soil pat at 25 blows.

Hence, at least three tests for the same soil were conducted at different water contents, and noted number of blows, N , required to achieve closure varying between 15 and 35. The water content with respect to number of blows drawn in the semi logarithmic graph and determined water content at 25 number of blows. The graph formed between water content and number of blows almost a straight line. This line known as flow line. The water content at 25 number of blows gives the liquid limit of the soil. The slope of the flow line is defined as the flow index.

3.2.4 Determination of Plastic Limit

Plastic limit is the water content below which the soil stops behaving as a plastic material. It begins to crumble when rolled into a thread of soil of 3mm diameter. At this water content, the soil loses its plasticity and passes to a semi-solid state. Plastic limit of the soil sample is conducted as per IS 2720

3.2.5 Compaction characteristics

Standard proctor test has been done according to the code IS 2720: PART VII: 1980. This test is used to determine the maximum dry density of the soil. In this method first weight the empty mould of base plate attach collar to the plate. Then apply thin layer of grease to the mould. Then take 2.4 kg of the soil and add the water up to 10% of the weight of the soil, then mix the soil thoroughly. Keep the soil under covered 20 to 30 seconds to allow the soil for full maturation.

Divide the soil into three equal parts and fill the mould with first and compact it to 25 numbers of blows. These blows should be evenly distributed whole area of the sample, so that we can achieve the better density. Repeat this procedure second and third part of the soil by taking precaution to scratch the previously compacted soil to achieve the homogenous density throughout depth of the soil. Remove the collar rotating it without disturbing the soil and then scratch the top soil up to the level. Then take the total weight of the soil and mould. Repeat this procedure by taking the fresh soil with varying the water content and determine the density of the soil in each case and then draw the graph between maximum dry density verses water content and then determine density of the soil.

3.2.6 Unconfined compression test

Unconfined compressive test has been done according to code of IS 2720: PART 3:1980. Unconfined compressive strength load per unit area at which unconfined cylindrical specimen of the soil fail in compression test. It was observed that force per unit area has not been obtained maximum value up to the 15% of the strain. In the experimental program, first it necessary to know the maximum dry density of the soil and optimum moisture content. The main intention of the unconfined compressive strength test is to find the unconfined compressive strength of soil that having enough cohesion to admit for determining in the unconfined state which is then possible to determine unconsolidated and un drained shear strength of the soil. The UCS test is determined as per IS: 2720 (Part 10) 1991. The soil sample was prepared from freshly developed soil sample and laid the samples for 7days in a constant water content desiccator .For purposes of testing black cotton soil specimen we use 5 to 20 KN proving ring depends on the strength of the soil. After knowing the volume of the mould, the soil will be compacted to into the mould up to the maximum dry density and optimum moisture content was used. Then after compacting the soil specimen will be removed from the mould which having 5 cm diameter and 10cm height. Then soil sample placed in the compressive testing machine without any side confinement and the stress strain values has been recorded. In the present experiment I used 5 KN proving ring and noted down the values obtained from the proving ring with changes of strain. I determined unconfined compressive strength for multiple number of drying and wetting cycles.

3.2.7 California Bearing Ratio

California Bearing Ratio (CBR) test is a type of test developed by the California Division of Highways in 1929. The test is used for evaluating the suitability of subgrade and the materials used in the sub-base and base courses. The test results have been correlated with the thickness of the various materials required for the flexible pavements.

CBR is expressed in percentage of force per unit area required to penetrate a soil mass with a circular plunger of 50 mm diameter at the rate of 1'25 mm/min to that required for corresponding penetration in a standard material. The ratio is usually determined for penetration of 2.5 and 5 mm. Where the ratio at 5 mm is consistently higher than that at 2'5 mm, the ratio at 5 mm is used. In this research work, I have

followed IS-2720 for conducting the test for multiple numbers of drying and wetting cycles.

3.2.8 Falling head permeability test

The coefficient of permeability is equal to the rate of flow of water through a unit cross-sectional area under a unit hydraulic gradient. It is represented by k , which represent the value of coefficient of permeability value. Falling head permeability test is used in this research to measure the permeability of relatively less pervious soils for multiple cycles of drying and wetting. Falling head permeability test is conducted as per IS-2720 in this thesis work.

CHAPTER FOUR: RESULT AND DISCUSSION

4.1 Index Properties

Plastic limit, Liquid Limit test, water content and core cutter test is done according to the codal provision of IS 2720: PART 5:1985. The following results were obtained and further mathematical calculation for plasticity index, liquidity index, consistency index were done and the results are summarized as below in table 11.

Table 4.1: Index properties of properties of Soil Sample

Soil	Depth (m)	Liquid Limit(%)	water content (%)	Plastic Limit(%)	Plasticity Index(%)	Liquidity index(%)	consistency index(%)
Tokha	2.5	52.43%	32.13%	28.60%	23.83%	14.82	85.178

It has been observed that as Liquid limit and Plastic limit of the soil is 52.43% and 28.60% and as per the USCS classification system the soil is classified as high plastic clay(CH), which shows expansive nature in the field.

Table 4.2: In-situ and dry density of soil sample

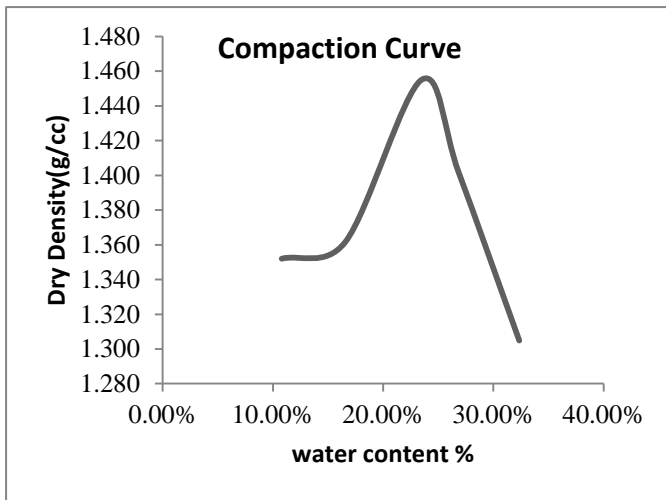
In situ density	1.718	gm/cc
dry density	1.300	gm/cc

4.2 Compaction Characteristics

Standard proctor test has been done according to code of IS 2720:PART VII:1980. The following data were observed from the test, which were used for calculating the optimum moisture content and maximum dry density. From the graph as shown in figure 8, OMC of the soil sample was found to be 23.50% and the corresponding dry density was found to be 1.462 g/cc.

Table 4.3: Standard proctor test data

water	10.80%	16.56%	23.50%	26.87%	32.35%
Density	1.498	1.587	1.797	1.778	1.727
Dry density	1.352	1.362	1.455	1.401	1.305



OMC	23.50%
maximum dry density	1.462 gm/cc

Figure 4.1: Compaction curve

4.3 Specific Gravity

Pycnometer test has been done according to code of IS 2720:PART VII:1985 for determining the specific gravity of solids. As per the test data the specific gravity of solids was found to be 2.51.

Table 4.4: Specific gravity test data

Pycnometer No.=	1.00	2.00
Wt. of pyc. + water +sample(W1) gm =	173.59	179.25
Temperature degree celcius=	25.00	25.00
Wt. of pyc. + water (W2) gm=	161.44	161.27
Net dry Wt. of sample(Ws) gm=	20.00	30.00
Specific gravity of water at T degree celcius(Gt)=	1.00	1.00
Specific gravity of solids (Gs)=	2.54	2.49
Average specific gravity of soils (Gs)=	2.51	

4.4 UCS Test Results on no. of Drying-Wetting Cycles

Unconfined compressive strength has been done according to the code of IS 2720: PART 3:1980. The test has been conducted for eight number of Drying and Wetting cycles. The results obtained from the test on multiple drying and wetting cycles indicates that the UCS value of the soil specimen was decreasing with the increasing number of cycles. Also the water content, dry density, degree of saturation, void ratio was also calculated during each cycle of drying and wetting and the results are

tabulated in table 15. From the result we can observe the reduction of about 22% in UCS value. And the void ratio goes on increasing till fifth cycle and then remain constant thereafter. Thus, after fifth cycle of drying and wetting there is no or slight change in the parameter as mention above.

Table 4.5 UCS data for number of cycles

Cycle no.	Mass of a soil sample(g)	Volume (cm ³)	Density (g/cc)	water content(%)	Dry density (g/cc)	Void ratio	Degree of saturation	Q _u (KN/m ²)	C _u (KN/m ²)
0	153.26	85.01	1.803	32.97%	1.356	0.851	97.197%	108.379	54.190
1	150.89	85.01	1.775	33.01%	1.334	0.881	94.037%	102.830	51.415
2	149.56	85.01	1.759	33.27%	1.320	0.901	92.635%	96.340	48.170
3	148.4	85.01	1.746	33.02%	1.312	0.913	90.804%	91.369	45.685
4	147.79	85.01	1.738	33.57%	1.301	0.929	90.742%	87.083	43.541
5	147.65	85.01	1.737	33.43%	1.302	0.928	90.383%	84.684	42.342
6	147.62	85.01	1.736	33.47%	1.301	0.929	90.397%	84.375	42.188
7	147.57	85.01	1.736	33.46%	1.301	0.930	90.320%	84.297	42.149
8	147.53	85.01	1.735	33.40%	1.301	0.930	90.191%	84.07	42.035

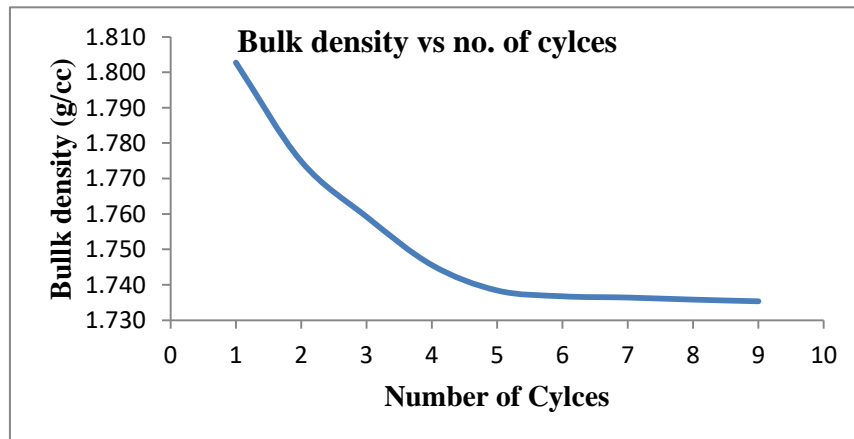


Figure 4.2: Bulk Density vs number of cycles

After the test it is clearly shown that, the bulk density of the soil sample decreases with increase in the number of the D-W cycles and after the 5th cycle the value remains constant. The value of bulk density decrease from 1.803 g/cc to 1.735 g/cc.

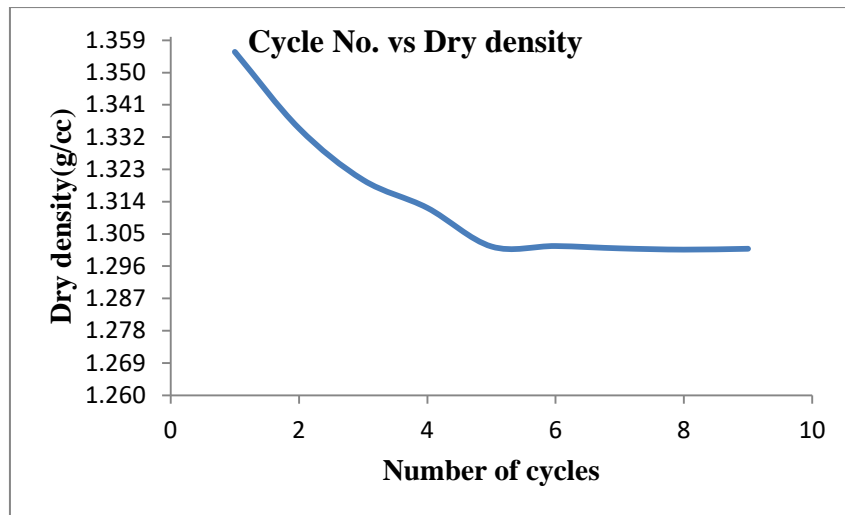


Figure 4.3: Cycle no. vs Dry density

The graph shown in figure 10 shows that there is a decrement in dry density with the increase in number of D-W cycles. The value of dry density decrease from 1.356 g/cc to 1.301 g/cc. The value of dry density also remains constant after 5th cycle of D-W cycles.

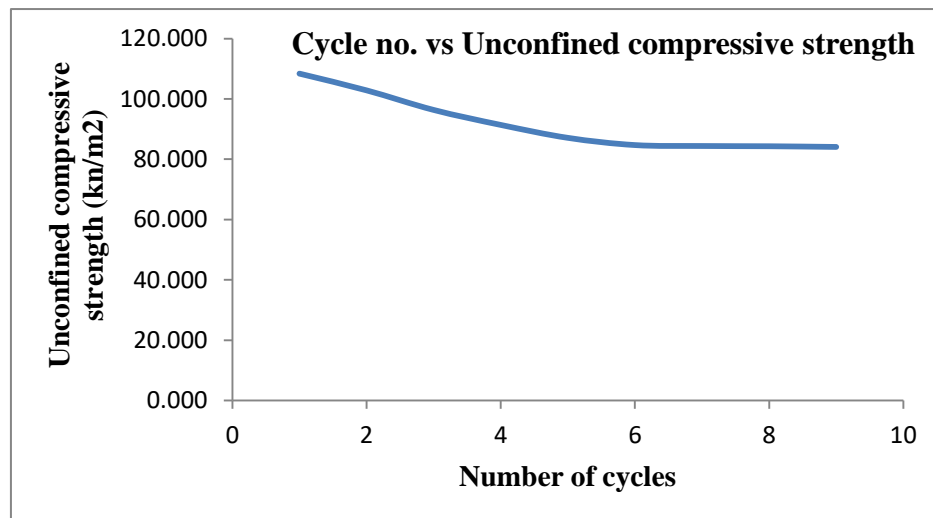


Figure 4.4: Cycle no. vs unconfined compressive strength

The graph shown in figure 11, shows that there is reduction in the unconfined compression strength of the soil sample from 108.379 KN/m² to 84.07 Kn/m², which is about 22.4% reduction after the multiple D-W cycles.

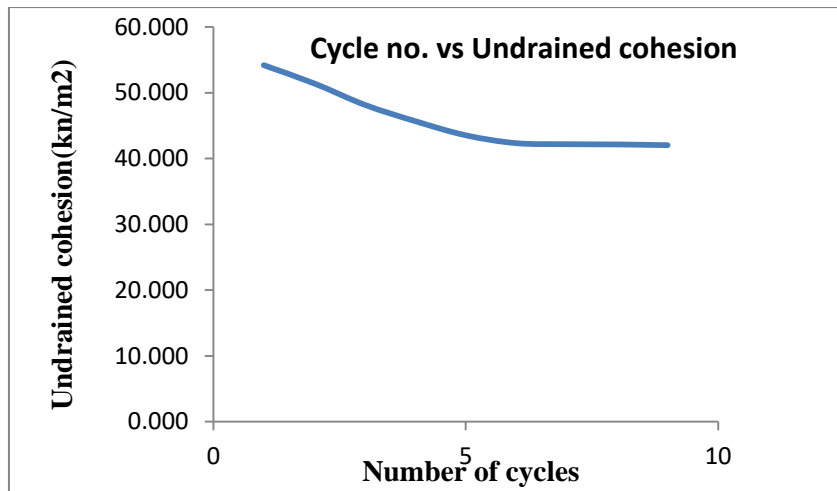


Figure 4.5: Cycle no. vs Undrained cohesion

The graph shown in Figure 12, shows that there is decrement in undrained cohesion value with the increase in D-W cycles. The value of undrained cohesion decreases from 54.190 kN/m² to 42.035 kN/m². And after 5th cycle of D-W the effect seems to be negligible i.e remains constant.

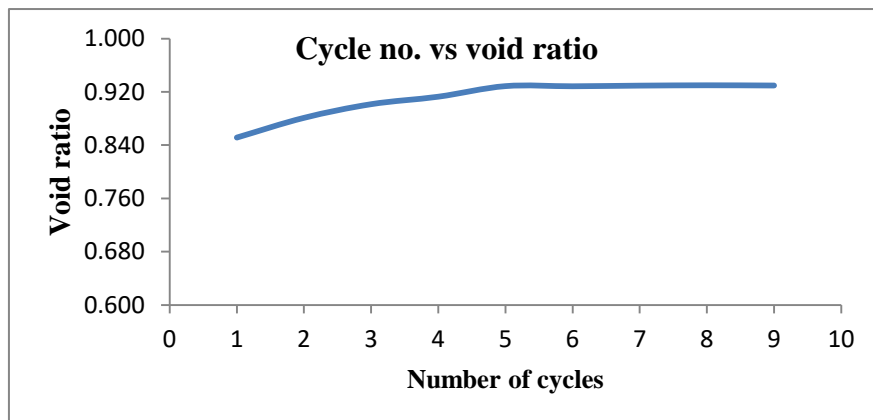


Figure 4.6: Cycle no. vs void ratio

The graph shown in figure 13, clearly shows that there is increase in void ratio with the increase in numbers of D-W cycles and the value remains constant after 5th cycle of D-W. The value of void ratio increase from 0.851 to 0.930, which means there is a increment of about 9.18%.

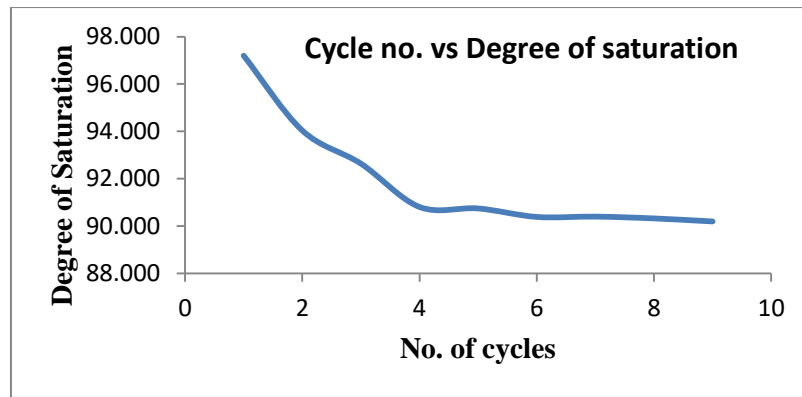


Figure 4.7: Cycle no. vs degree of saturation

The graph shown in figure:14, indicates that the degree of saturation goes on decreasing with the increase in number of D-W cycles. The value of degree of saturation decreases from 97.19% to 90.19% and the value remains almost constant after 5th cycles of D-W.

4.5 CBR Test Results on no. of Drying Wetting Cycle

CBR test has been conducted as per IS: 2720 (Part 16) – 1987 of the soil specimen on multiple number of Drying and Wetting cycles. The test results tabulated in table 16 indicate that there is decrement of CBR value from 4.6% to 2.19% with the increasing number of Drying and Wetting cycles. The decrease in CBR value stops from the fourth cycle and remains constant thereafter. Each result of CBR test with multiple number of Drying and Wetting cycles are tabulated in the following chapter.

Table 4.6: CBR test data at 0-Cycle of D-W test

Dial gauge reading	Penetration(mm)	Proving ring reading	Piston Load(kg)	Area of Piston(cm ²)	Penetration stress (Kg/cm ²)
0	0	0	0	1962.5	0
50	0.5	3	11.25	1962.5	0.005732484
100	1	6	22.5	1962.5	0.011464968
150	1.5	9	33.75	1962.5	0.017197452
200	2	14	52.5	1962.5	0.026751592
250	2.5	17	63.75	1962.5	0.032484076
400	4	20	75	1962.5	0.038216561
500	5	22	82.5	1962.5	0.042038217
750	7.5	25	93.75	1962.5	0.047770701
1000	10	27	101.25	1962.5	0.051592357
1250	12.5	29	108.75	1962.5	0.055414013

CBR =4.653%

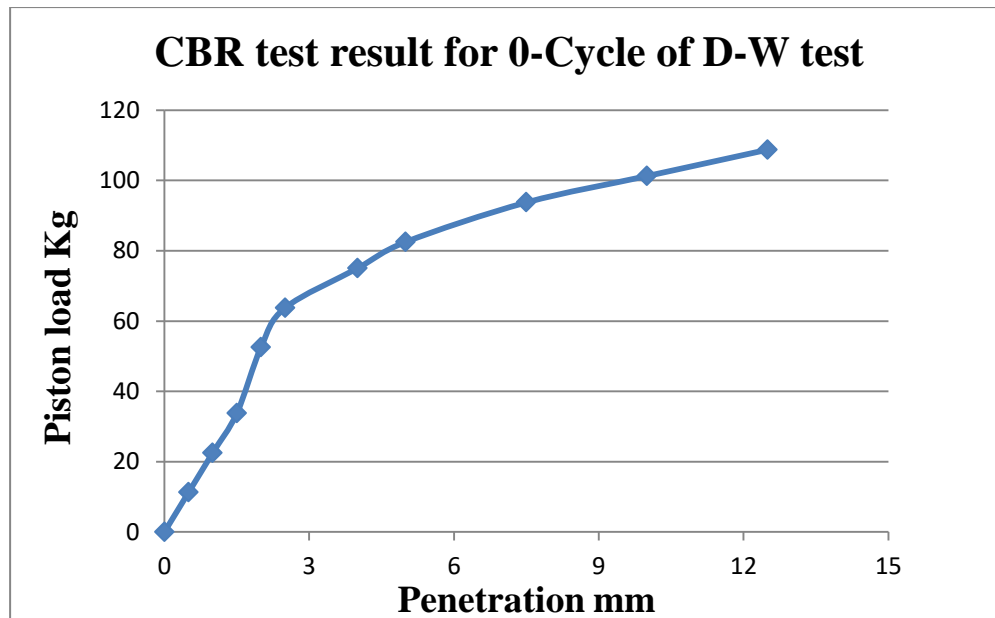


Figure 4.8: Piston Load Vs Penetration for 0-Cycles of D-W

Table 4.7 CBR test data for 1 cycle of D-W

Dial gauge reading	Penetration(mm)	Proving ring reading	Piston Load(kg)	Area of Piston(cm ²)	Penetration stress (Kg/cm ²)
0	0	0	0	1962.5	0
50	0.5	4	15	1962.5	0.007643312
100	1	6	22.5	1962.5	0.011464968
150	1.5	9	33.75	1962.5	0.017197452
200	2	11	41.25	1962.5	0.021019108
250	2.5	13	48.75	1962.5	0.024840764
400	4	15	56.25	1962.5	0.02866242
500	5	16	60	1962.5	0.030573248
750	7.5	18	67.5	1962.5	0.034394904
1000	10	21	78.75	1962.5	0.040127389
1250	12.5	23	86.25	1962.5	0.043949045

CBR=3.558%

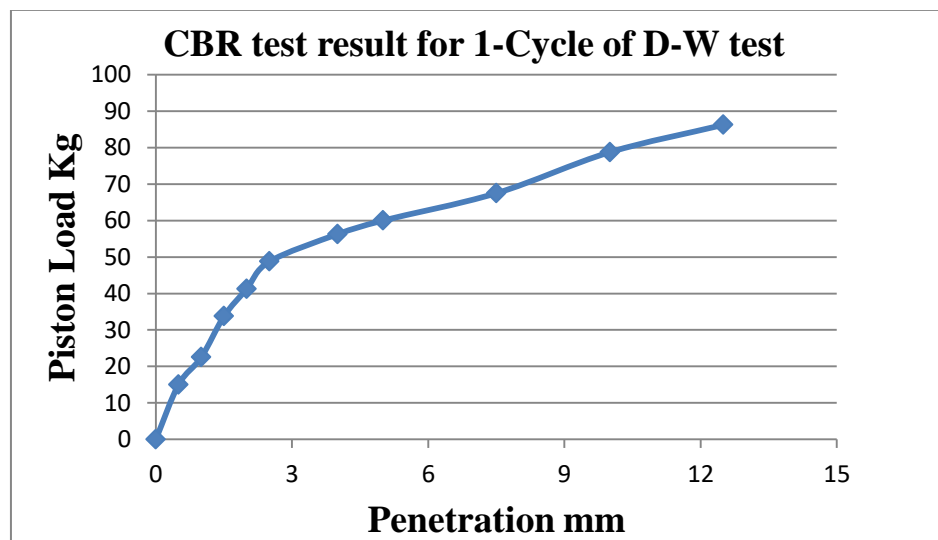


Figure 4.9 Figure 16 Piston Load Vs Penetration for 1-Cycles of D-W

Table 4.8 CBR test data for 2-cycle D-W

Dial gauge reading	Penetration(mm)	Proving ring reading	Piston Load(kg)	Area of Piston(cm ²)	Penetration stress (Kg/cm ²)
0	0	0	0	1962.5	0
50	0.5	3	11.25	1962.5	0.005732484
100	1	5	18.75	1962.5	0.00955414
150	1.5	7	26.25	1962.5	0.013375796
200	2	9	33.75	1962.5	0.017197452
250	2.5	11	41.25	1962.5	0.021019108
400	4	14	52.5	1962.5	0.026751592
500	5	15	56.25	1962.5	0.02866242
750	7.5	18	67.5	1962.5	0.034394904
1000	10	19	71.25	1962.5	0.036305732
1250	12.5	21	78.75	1962.5	0.040127389

CBR=3.011%

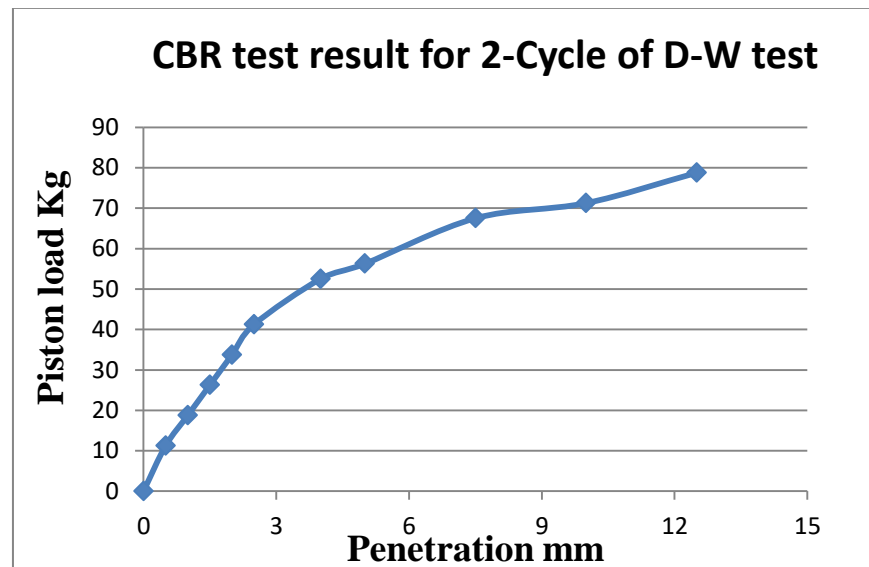


Figure 4.10 Piston Load Vs. Penetration for 2-Cycles of D-W

Table 4.9 CBR test data for 3-Cycle of D-W

Dial gauge reading	Penetration(mm)	Proving ring reading	Piston Load(kg)	Area of Piston(cm ²)	Penetration stress (Kg/cm ²)
0	0	0	0	1962.5	0
50	0.5	3	11.25	1962.5	0.005732484
100	1	5	18.75	1962.5	0.00955414
150	1.5	7	26.25	1962.5	0.013375796
200	2	9	33.75	1962.5	0.017197452
250	2.5	10	37.5	1962.5	0.01910828
400	4	12	45	1962.5	0.022929936
500	5	13	48.75	1962.5	0.024840764
750	7.5	15	56.25	1962.5	0.02866242
1000	10	17	63.75	1962.5	0.032484076
1250	12.5	19	71.25	1962.5	0.036305732

CBR=2.737%

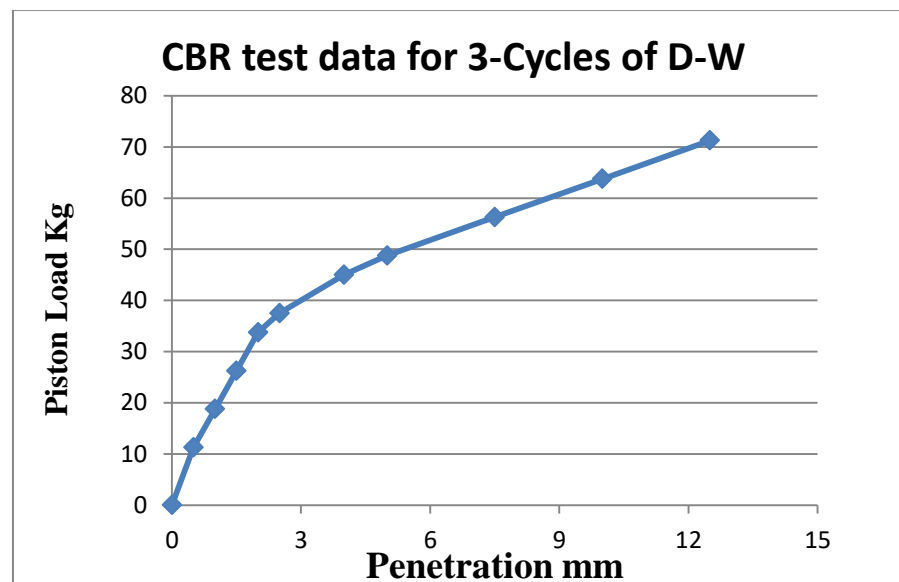


Figure 4.11 Piston load Vs. penetration for 3-Cycles of D-W

Table 4.10 CBR test data for 4-Cycle of D-W

Dial gauge reading	Penetration(mm)	Proving ring reading	Piston Load(kg)	Area of Piston(cm ²)	Penetration stress (Kg/cm ²)
0	0	0	0	1962.5	0
50	0.5	2	7.5	1962.5	0.003821656
100	1	4	15	1962.5	0.007643312
150	1.5	6	22.5	1962.5	0.011464968
200	2	7	26.25	1962.5	0.013375796
250	2.5	8	30	1962.5	0.015286624
400	4	10	37.5	1962.5	0.01910828
500	5	11	41.25	1962.5	0.021019108
750	7.5	14	52.5	1962.5	0.026751592
1000	10	16	60	1962.5	0.030573248
1250	12.5	18	67.5	1962.5	0.034394904

CBR=2.190%

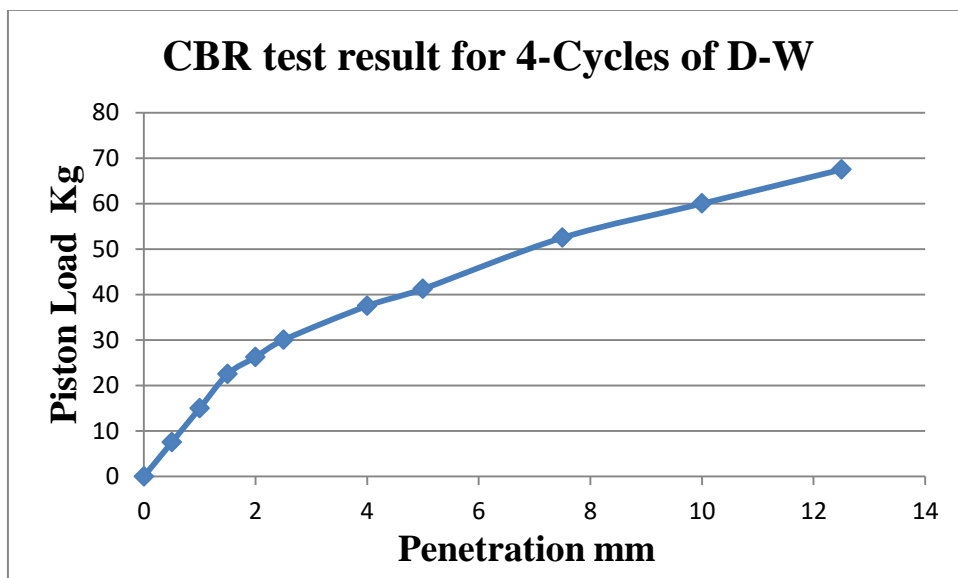


Figure 4.12 Piston Load Vs. Penetration for 4-Cycles of D-W

Table 4.11 CBR test data for 5- Cycle of D-W test

Dial gauge reading	Penetration(mm)	Proving ring reading	Piston Load(kg)	Area of Piston(cm ²)	Penetration stress (Kg/cm ²)
0	0	0	0	1962.5	0
50	0.5	2	7.5	1962.5	0.003821656
100	1	4	15	1962.5	0.007643312
150	1.5	6	22.5	1962.5	0.011464968
200	2	7	26.25	1962.5	0.013375796
250	2.5	8	30	1962.5	0.015286624
400	4	10	37.5	1962.5	0.01910828
500	5	11	41.25	1962.5	0.021019108
750	7.5	14	52.5	1962.5	0.026751592
1000	10	17	63.75	1962.5	0.032484076
1250	12.5	19	71.25	1962.5	0.036305732

CBR=2.190%

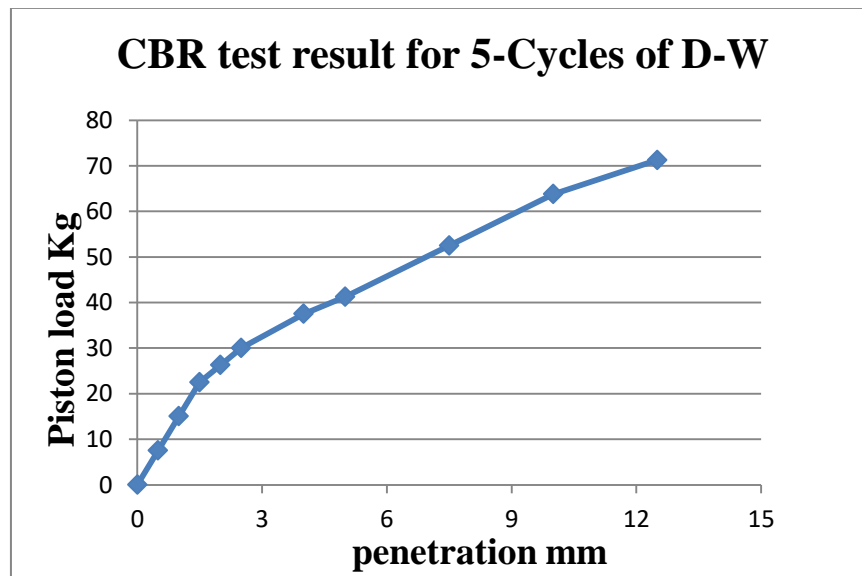


Figure 4.13 Piston Load Vs. penetration for 5-Cycles of D-W test

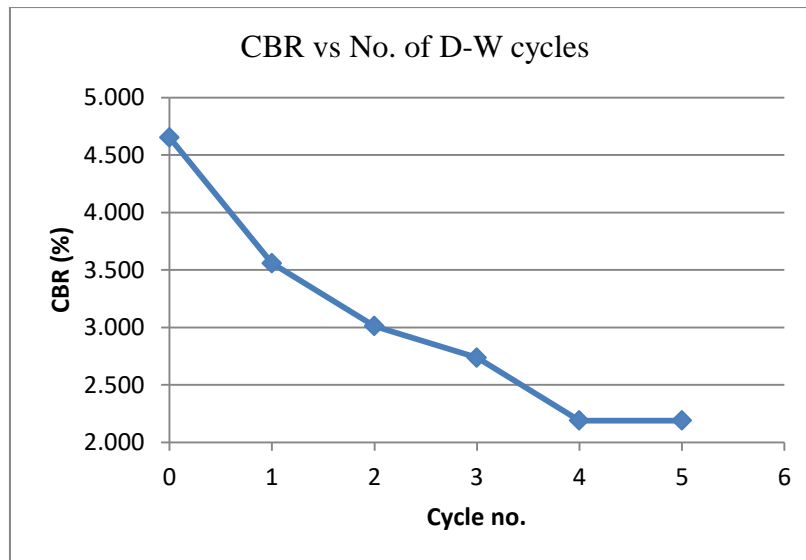


Figure 4.14: CBR Test Results on no. of Drying Wetting Cycle

4.6 Falling Head Permeability Test

Falling head permeability test method has been conducted as per codal provision of IS:2720 for six number of Drying and Wetting cycles. From the test result as shown in table 17, it has been found that the coefficient of permeability increased from $5.13122E-08$ cm/s to $5.93151E-08$ cm/s. This result indicates that there is increment of about 15.6% in the value of coefficient of permeability value. The test data for each cycles of Drying and Wetting are tabulated below.

Table 4.12 Falling Head Permeability test data at 0- Cycle of D-W

length of specimen(cm)	11.85	11.85	
Dia of permeater(cm)	10	10	
Area of specimen(cm ²)	78.5	78.5	
Volume of specimen(cm ³)	930.225	930.225	
Dia of stand pipe(cm)	0.9	0.9	
Area of stand pipe(cm ²)	0.63585	0.63585	
Specific gravity of solids	2.51	2.51	
Temperature °C	23	23	
Length of specimen(cm)	11.85	11.85	
Soil specimen compacted at optimum moisture content			
Sn no	observation and calculation	Test set	
		1	2
1	mass of mould +base plate (g)	8080	8080
2	mass of mould +base plate+ soil	9905	9905
3	initial head, h1(cm)	103	112
4	final head, h2 (cm)	94	102
5	head sqrt(h1*h2) (cm)	98.39715443	106.8831137
6	Time interval		
	h1 to square root (h1*h2)		
	Square root (h1*h2) to h2		
	h1 to h2 (sec)	172800	172800
9	Coefficient of Permeability (k) cm/s	5.07318E-08	5.18925E-08
Average value of Coefficient of Permeability (k) in cm/s			5.1312E-08

Table 4.13 Falling Head permeability test data at 1-Cycle of D-W

length of specimen(cm)	11.85	11.85	
Dia of permeater(cm)	10	10	
Area of specimen(cm ²)	78.5	78.5	
Volume of specimen(cm ³)	930.225	930.225	
Dia of stand pipe(cm)	0.9	0.9	
Area of stand pipe(cm ²)	0.63585	0.63585	
Specific gravity of solids	2.51	2.51	
Temperature °C	23	23	
Length of specimen(cm)	11.85	11.85	
Soil specimen compacted at optimum moisture content			
S.No	observation and calculation	Test set	
		1	2
1	Mass of mould +base plate (g)	8080	8080
2	Mass of mould +base plate+ soil	9905	9905
3	Initial head, h1(cm)	118	115
4	Final head, h2 (cm)	106	104
5	Head square root (h1*h2) (cm)	111.8391702	109.3617849
6	Time interval		
	h1 to square root (h1*h2)		
	Square root (h1*h2) to h2		
	h1 to h2 (sec)	172800	172800
9	Coefficient of Permeability (k) cm/s	5.95047E-08	5.57848E-08
Average value of Coefficient of Permeability (k) cm/s			5.76447E-08

Table 4.14 Falling Head Permeability Test at 2- Cycle of D-W

length of specimen(cm)	11.85	11.85	
Dia of permeater(cm)	10	10	
Area of specimen(cm ²)	78.5	78.5	
Volume of specimen(cm ³)	930.225	930.225	
Dia of stand pipe(cm)	0.9	0.9	
Area of stand pipe(cm ²)	0.63585	0.63585	
Specific gravity of solids	2.51	2.51	
Temperature °C	23	23	
Length of specimen(cm)	11.85	11.85	
Soil specimen compacted at optimum moisture content			
S.No	observation and calculation	Test set	
		1	2
1	Mass of mould +base plate (g)	8080	8080
2	Mass of mould +base plate+ soil	9905	9905
3	Initial head, h1(cm)	100	101
4	Final head, h2 (cm)	90	91
5	Head square root (h1*h2) (cm)	94.86832981	95.86970324
6	Time interval		
	h1 to square root (h1*h2)		
	Square root (h1*h2) to h2		
	h1 to h2 (sec)	172800	172800
9	Coefficient of Permeability (k) cm/s	5.84588E-08	5.78487E-08
Average value of Coefficient of Permeability (k) cm/s			5.81537E-08

Table 4.15 Falling Head Permeability test at 3-Cycle of D-W

length of specimen(cm)	11.85	11.85	
Dia of permeater(cm)	10	10	
Area of specimen(cm ²)	78.5	78.5	
Volume of specimen(cm ³)	930.225	930.225	
Dia of stand pipe(cm)	0.9	0.9	
Area of stand pipe(cm ²)	0.63585	0.63585	
Specific gravity of solids	2.51	2.51	
Temperature °C	23	23	
Length of specimen(cm)	11.85	11.85	
Soil specimen compacted at optimum moisture content			
S.No	observation and calculation	Test set	
		1	2
1	Mass of mould +base plate (g)	8080	8080
2	Mass of mould +base plate+ soil	9905	9905
3	Initial head, h1(cm)	99	90
4	Final head, h2 (cm)	89	81
5	Head square root (h1*h2) (cm)	93.86692708	85.38149682
6	Time interval		
	h1 to square root (h1*h2)		
	Square root (h1*h2) to h2		
	h1 to h2 (sec)	172800	172800
9	Coefficient of Permeability (k) cm/s	5.90818E-08	5.84588E-08
Average value of Coefficient of Permeability (k) cm/s			5.877E-08

Table 4.16 Falling Head Permeability test at 4-Cycle of D-W

Length of specimen(cm)	11.85	11.85	
Dia of permeater (cm)	10	10	
Area of specimen(cm ²)	78.5	78.5	
Volume of specimen(cm ³)	930.225	930.225	
Dia of stand pipe(cm)	0.9	0.9	
Area of stand pipe(cm ²)	0.63585	0.63585	
Specific gravity of solids	2.51	2.51	
Temperature °C	23	23	
Length of specimen(cm)	11.85	11.85	
Soil specimen compacted at optimum moisture content			
S.No	observation and calculation	Test set	
		1	2
1	Mass of mould +base plate (g)	8080	8080
2	Mass of mould +base plate+ soil	9905	9905
3	Initial head, h1(cm)	101	112
4	Final head, h2 (cm)	90.8	100.6
5	Head square root (h1*h2) (cm)	95.76429397	106.1470678
6	Time interval		
	h1 to square root (h1*h2)		
	Square root (h1*h2) to h2		
	h1 to h2 (sec)	172800	172800
9	Coefficient of Permeability (k) cm/s	5.90695E-08	5.95607E-08
Average value of Coefficient of Permeability (k) cm/s			5.9315E-08

Table 4.17 Falling Head Permeability test at 5-Cycle of D-W

length of specimen(cm)	11.85	11.85	
Dia of permeater(cm)	10	10	
Area of specimen(cm ²)	78.5	78.5	
Volume of specimen(cm ³)	930.225	930.225	
Dia of stand pipe(cm)	0.9	0.9	
Area of stand pipe(cm ²)	0.63585	0.63585	
Specific gravity of solids	2.51	2.51	
Temperature °C	23	23	
Length of specimen(cm)	11.85	11.85	
Soil specimen compacted at optimum moisture content			
S.No	observation and calculation	Test set	
		1	2
1	Mass of mould +base plate (g)	8080	8080
2	Mass of mould +base plate+ soil	9905	9905
3	Initial head, h1(cm)	101	112
4	Final head, h2 (cm)	90.8	100.6
5	Head square root (h1*h2) (cm)	95.76429397	106.1470678
6	Time interval		
	h1 to square root (h1*h2)		
	Square root (h1*h2) to h2		
	h1 to h2 (sec)	172800	172800
9	Coefficient of Permeability (k) cm/s	5.90695E-08	5.95607E-08
Average value of Coefficient of Permeability (k) cm/s			5.9315E-08

All the values of coefficient of permeability is then tabulated in Table number 29 as below:

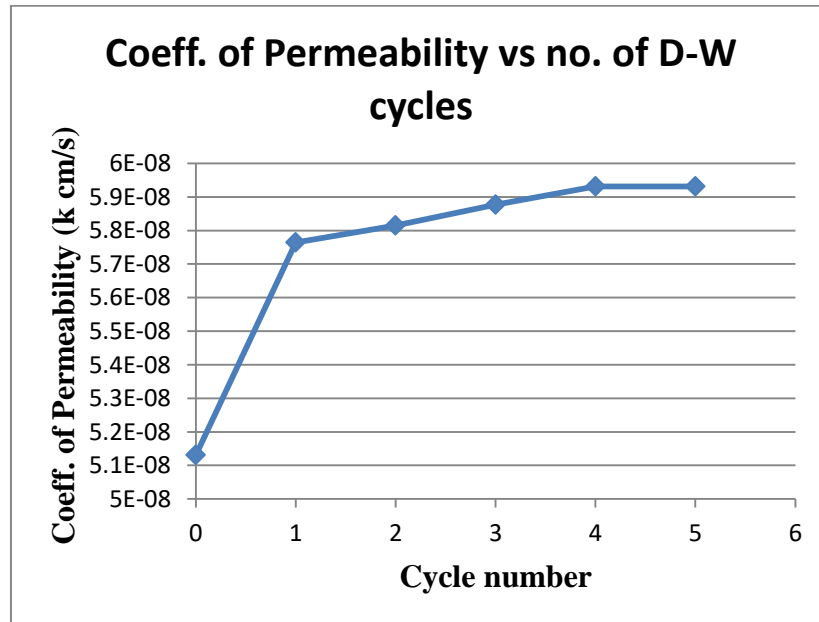


Figure 4.15: Falling Head Permeability Test

From the graph as shown in figure 16, it is clearly observed that the “k” value goes on increasing with the increase in the number of the D-W cycles and after 4th cycle of test, value of “k” is almost constant.

CHAPTER FIVE: CONCLUSION AND RECOMMENDATION

The Effect on soil sample from Tokha due to cyclic drying and wetting effects on Unconfined Compressive Strength, California Bearing ratio, Coefficient of Permeability, Dry density, Void ratio, Degree of Saturation and Bulk Density were investigated in this study and following conclusions were drawn:

- UCS value of the soil sample decreased with the increase in number of Drying and Wetting cycles, which indicate about 22% reduction in both undrained cohesion and UCS value of the soil. However, after 5th cycles of Drying and Wetting there was no effect of Drying and Wetting on the UCS and Undrained cohesion value of the soil sample from the site of Tokha.
- The value of void ratio increased from 0.85 to 0.92 until fifth cycle and then a constant value of nearly equal to 0.92 is obtained after 5th cycle, which indicates about 9% increment in the void ratio of the soil sample.
- The value of dry density reduced from 1.356g/cc to 1.301g/cc and it is observed that the value of dry density remains constant from 5th cycles of Drying and Wetting. Degree of saturation and bulk density goes on decreasing with the increase in the number of Drying and Wetting cycles
- There is a decrease of CBR value from 4.653% to 2.190% after 5th cycle of Drying and Wetting and the value remains constant from 4th cycles of Drying and Wetting.

From this study we can conclude that, the effect is more significant till fifth cycle and finally reaches to a constant state after fifth cycles.

Such studies are useful to understand the possible changes in behavior of expansive soil that are subjected to periodic drying and wetting from climatic variations and may provide the reference to the future geotechnical engineers and disaster reduction professionals to build up engineering structures safely and economically. Further laboratory testing is recommended to establish if all the observed losses in strength in the field are due to wetting and drying and to establish suitable procedures for measuring the effects of wetting and drying using generally more suitable laboratory tests. However, until such additional testing can be completed it is evident that the CBR value, UCS value, coefficient of permeability, dry density, degree of saturation and void ratio should not be relied upon for design when such soils will be subjected to repeated wetting and drying.

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ANNEX I: Calculation sheets of unconfined compression test

Table i: UCS test data for 0-Cycle of D-W

S.No.	Dial gauge Reading	Proving ring Reading	CALCULATION							Remarks
			Deflection, ΔL	Load (KN)	Axial Strain, ϵ	Axial Strain, e	Corrected Area, A_c	Area (m ²)	Stress = Load/A	
1	0	0	0.00	0.000	0.00000	0.000	11.34100	0.00113	0.000	0
2	20	5	0.20	0.007	0.00267	0.267	11.37132	0.00114	6.156	3.077918
3	40	7	0.40	0.010	0.00526	0.526	11.40101	0.00114	8.596	4.297867
4	60	9	0.60	0.013	0.00789	0.789	11.43125	0.00114	11.022	5.51121
5	80	11	0.80	0.015	0.01053	1.053	11.46165	0.00115	13.436	6.718056
6	100	13	1.00	0.018	0.01316	1.316	11.49221	0.00115	15.837	7.918405
7	120	15	1.20	0.021	0.01579	1.579	11.52294	0.00115	18.225	9.112257
8	140	17	1.40	0.024	0.01842	1.842	11.55383	0.00116	20.599	10.29961
9	160	19	1.60	0.027	0.02105	2.105	11.58489	0.00116	22.961	11.48047
10	180	21	1.80	0.029	0.02368	2.368	11.61612	0.00116	25.310	12.65483
11	200	24	2.00	0.034	0.02632	2.632	11.64751	0.00116	28.847	14.42368
12	220	27	2.20	0.038	0.02895	2.895	11.67908	0.00117	32.366	16.18278
13	240	30	2.40	0.042	0.03158	3.158	11.71082	0.00117	35.864	17.93214
14	260	33	2.60	0.046	0.03421	3.421	11.74272	0.00117	39.344	19.67175
15	280	33.5	2.80	0.047	0.03684	3.684	11.77481	0.00118	39.831	19.9154
16	300	35	3.00	0.049	0.03947	3.947	11.80707	0.00118	41.501	20.75028
17	320	37	3.20	0.052	0.04211	4.211	11.83951	0.00118	43.752	21.87591
18	340	39	3.40	0.055	0.04474	4.474	11.87212	0.00119	45.990	22.99505
19	360	42	3.60	0.059	0.04737	4.737	11.90492	0.00119	49.391	24.69568
20	380	44	3.80	0.062	0.05000	5.000	11.93789	0.00119	51.600	25.80019
21	400	46	4.00	0.064	0.05263	5.263	11.97106	0.00120	53.796	26.89821
22	420	47	4.20	0.066	0.05526	5.526	12.00440	0.00120	54.813	27.40662
23	440	50	4.40	0.070	0.05789	5.789	12.03793	0.00120	58.150	29.07476
24	460	54	4.60	0.076	0.06053	6.053	12.07165	0.00121	62.626	31.31303
25	480	56	4.80	0.078	0.06316	6.316	12.10556	0.00121	64.764	32.38181
26	500	58	5.00	0.081	0.06579	6.579	12.13966	0.00121	66.888	33.44409
27	520	61	5.20	0.085	0.06842	6.842	12.17395	0.00122	70.150	35.07488
28	540	65	5.40	0.091	0.07105	7.105	12.20844	0.00122	74.539	37.26929
29	560	68	5.60	0.095	0.07368	7.368	12.24313	0.00122	77.7579	38.87896
30	580	71	5.80	0.099	0.07632	7.632	12.27801	0.00123	80.958	40.47889
31	600	75	6.00	0.105	0.07895	7.895	12.31309	0.00123	85.275	42.63757
32	620	76	6.20	0.106	0.08158	8.158	12.34837	0.00123	86.165	43.08262
33	640	79	6.40	0.111	0.08421	8.421	12.38385	0.00124	89.310	44.65493
34	660	81	6.60	0.113	0.08684	8.684	12.41954	0.00124	91.308	45.65387
35	680	85	6.80	0.119	0.08947	8.947	12.45543	0.00125	95.541	47.77032
36	700	88	7.00	0.123	0.09211	9.211	12.49154	0.00125	98.627	49.31339
37	720	93	7.20	0.130	0.09474	9.474	12.52785	0.00125	103.928	51.96423
38	740	95	7.40	0.133	0.09737	9.737	12.56437	0.00126	105.855	52.92743
39	760	96	7.60	0.134	0.10000	10.000	12.60111	0.00126	106.657	53.32863
40	780	96	7.80	0.134	0.10263	10.263	12.63806	0.00126	106.345	53.1727
41	800	97	8.00	0.136	0.10526	10.526	12.67524	0.00127	107.138	53.56903
42	820	98	8.20	0.137	0.10789	10.789	12.71263	0.00127	107.924	53.9621
43	840	98.5	8.40	0.138	0.11053	11.053	12.75024	0.00128	108.155	54.07743
44	860	99	8.60	0.139	0.11316	11.316	12.78807	0.00128	108.379	54.190
45	880	98.5	8.80	0.138	0.11579	11.579	12.82613	0.00128	107.515	53.75744
46	900	98	9.00	0.137	0.11842	11.842	12.86442	0.00129	106.651	53.32538
47	920	97	9.20	0.136	0.12105	12.105	12.90293	0.00129	105.247	52.62369
48	940	97.5	9.40	0.137	0.12368	12.368	12.94168	0.00129	105.473	52.73658
49	960	97	9.60	0.136	0.12632	12.632	12.98066	0.00130	104.617	52.30858
50	980	96	9.80	0.134	0.12895	12.895	13.01988	0.00130	103.227	51.61338
51	1000	95	10.00	0.133	0.13158	13.158	13.05933	0.00131	101.843	50.92144
52	1020	90	10.20	0.126	0.13421	13.421	13.09903	0.00131	96.190	48.09517

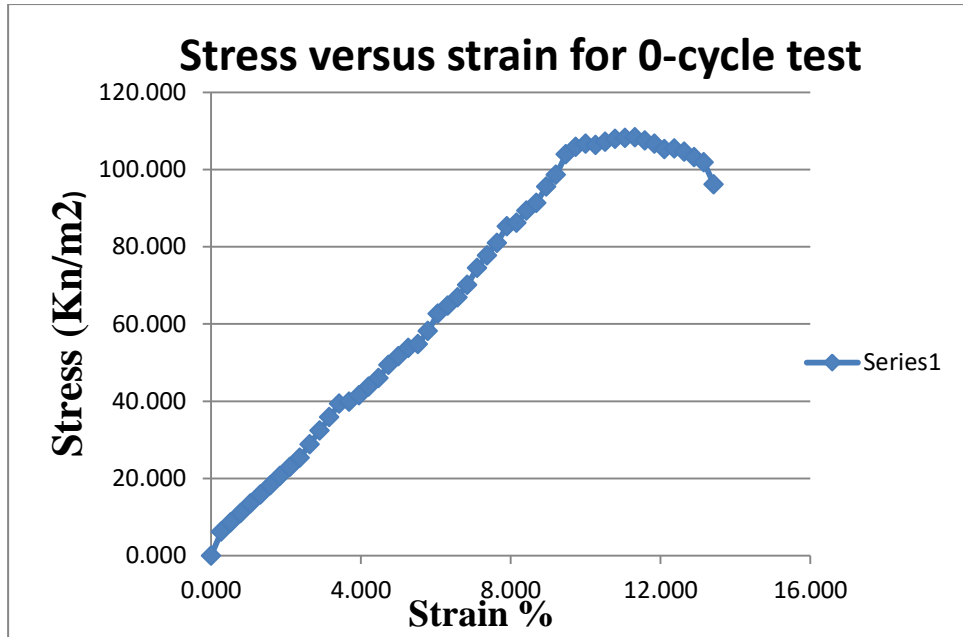


Figure i: Stress versus strain for 0-Cycle of D-W test

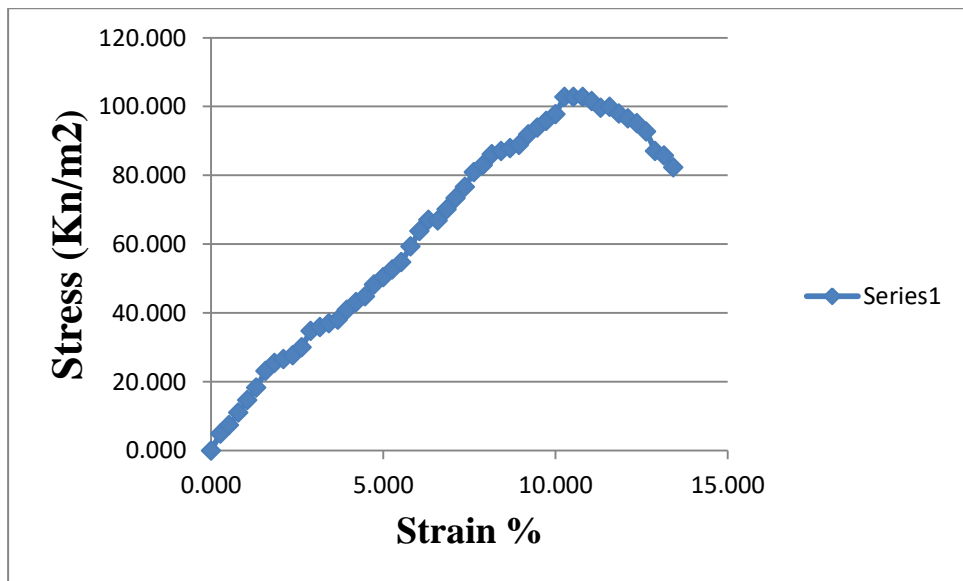


Figure ii: Stress versus strain for 1-Cycle of D-W

Table ii: UCS test data for 1-Cycle of D-W

S.No.	Dial gauge Reading	Proving ring Reading	CALCULATION							Remarks
			Deflection, ΔL	Load (KN)	Axial Strain,	Axial Strain, e	Corrected Area,	Area (m ²)	Stress =Load/A	
1	0	0	0.00	0.000	0.00000	0.000	11.34100	0.00113	0.000	0
2	20	4	0.20	0.006	0.00267	0.267	11.37132	0.00114	4.925	2.462334
3	40	6	0.40	0.008	0.00526	0.526	11.40101	0.00114	7.368	3.683886
4	60	9	0.60	0.013	0.00789	0.789	11.43125	0.00114	11.022	5.51121
5	80	12	0.80	0.017	0.01053	1.053	11.46165	0.00115	14.658	7.328788
6	100	15	1.00	0.021	0.01316	1.316	11.49221	0.00115	18.273	9.136621
7	120	19	1.20	0.027	0.01579	1.579	11.52294	0.00115	23.084	11.54219
8	140	21	1.40	0.029	0.01842	1.842	11.55383	0.00116	25.446	12.72305
9	160	22	1.60	0.031	0.02105	2.105	11.58489	0.00116	26.586	13.29317
10	180	23	1.80	0.032	0.02368	2.368	11.61612	0.00116	27.720	13.86005
11	200	25	2.00	0.035	0.02632	2.632	11.64751	0.00116	30.049	15.02467
12	220	29	2.20	0.041	0.02895	2.895	11.67908	0.00117	34.763	17.38151
13	240	30	2.40	0.042	0.03158	3.158	11.71082	0.00117	35.864	17.93214
14	260	31	2.60	0.043	0.03421	3.421	11.74272	0.00117	36.959	18.47953
15	280	32	2.80	0.045	0.03684	3.684	11.77481	0.00118	38.047	19.02366
16	300	34.5	3.00	0.048	0.03947	3.947	11.80707	0.00118	40.908	20.45385
17	320	36.5	3.20	0.051	0.04211	4.211	11.83951	0.00118	43.161	21.58029
18	340	38	3.40	0.053	0.04474	4.474	11.87212	0.00119	44.811	22.40543
19	360	41	3.60	0.057	0.04737	4.737	11.90492	0.00119	48.215	24.10769
20	380	43	3.80	0.060	0.05000	5.000	11.93789	0.00119	50.428	25.21383
21	400	45	4.00	0.063	0.05263	5.263	11.97106	0.00120	52.627	26.31347
22	420	47	4.20	0.066	0.05526	5.526	12.00440	0.00120	54.813	27.40662
23	440	51	4.40	0.071	0.05789	5.789	12.03793	0.00120	59.313	29.65625
24	460	55	4.60	0.077	0.06053	6.053	12.07165	0.00121	63.786	31.8929
25	480	58	4.80	0.081	0.06316	6.316	12.10556	0.00121	67.077	33.5383
26	500	58	5.00	0.081	0.06579	6.579	12.13966	0.00121	66.888	33.44409
27	520	61	5.20	0.085	0.06842	6.842	12.17395	0.00122	70.150	35.07488
28	540	64	5.40	0.090	0.07105	7.105	12.20844	0.00122	73.392	36.69592
29	560	67	5.60	0.094	0.07368	7.368	12.24313	0.00122	76.6144	38.30721
30	580	71	5.80	0.099	0.07632	7.632	12.27801	0.00123	80.958	40.47889
31	600	73	6.00	0.102	0.07895	7.895	12.31309	0.00123	83.001	41.50056
32	620	76	6.20	0.106	0.08158	8.158	12.34837	0.00123	86.165	43.08262
33	640	77	6.40	0.108	0.08421	8.421	12.38385	0.00124	87.049	43.52443
34	660	78	6.60	0.109	0.08684	8.684	12.41954	0.00124	87.926	43.96298
35	680	79	6.80	0.111	0.08947	8.947	12.45543	0.00125	88.797	44.39829
36	700	82	7.00	0.115	0.09211	9.211	12.49154	0.00125	91.902	45.95111
37	720	84	7.20	0.118	0.09474	9.474	12.52785	0.00125	93.871	46.93543
38	740	86	7.40	0.120	0.09737	9.737	12.56437	0.00126	95.827	47.91325
39	760	88	7.60	0.123	0.10000	10.000	12.60111	0.00126	97.769	48.88458
40	780	93	7.80	0.130	0.10263	10.263	12.63806	0.00126	102.830	51.41522
41	800	93	8.00	0.130	0.10526	10.526	12.67524	0.00127	102.830	51.41522
42	820	93	8.20	0.130	0.10789	10.789	12.71263	0.00127	102.830	51.41522
43	840	92.5	8.40	0.130	0.11053	11.053	12.75024	0.00128	101.567	50.78337
44	860	91	8.60	0.127	0.11316	11.316	12.78807	0.00128	99.624	49.81205
45	880	91.5	8.80	0.128	0.11579	11.579	12.82613	0.00128	99.874	49.93712
46	900	90	9.00	0.126	0.11842	11.842	12.86442	0.00129	97.945	48.97229
47	920	89	9.20	0.125	0.12105	12.105	12.90293	0.00129	96.567	48.28359
48	940	88	9.40	0.123	0.12368	12.368	12.94168	0.00129	95.196	47.59814
49	960	86	9.60	0.120	0.12632	12.632	12.98066	0.00130	92.753	46.37668
50	980	81	9.80	0.113	0.12895	12.895	13.01988	0.00130	87.098	43.54879
51	1000	80	10.00	0.112	0.13158	13.158	13.05933	0.00131	85.762	42.88121
52	1020	77	10.20	0.108	0.13421	13.421	13.09903	0.00131	82.296	41.14809

Table iii: UCS test data for 2-Cycles of D-W

S.No.	Dial gauge Reading	Proving ring Reading	CALCULATION						Stress = Load/Area (KN/m ²)	Remarks
			Deflection, ΔL (mm) *0.01	Load (KN) *0.0014	Axial Strain, $e = \Delta L/L_0$	Axial Strain, e (%)	Corrected Area, $A = A_0/(1-e)$	Area (m ²)		
1	0	0	0.00	0.000	0.00000	0.000	11.34100	0.00113	0.000	0
2	20	5	0.20	0.007	0.00267	0.267	11.37132	0.00114	6.156	3.077918
3	40	7	0.40	0.010	0.00526	0.526	11.40101	0.00114	8.596	4.297867
4	60	9	0.60	0.013	0.00789	0.789	11.43125	0.00114	11.022	5.51121
5	80	13	0.80	0.018	0.01053	1.053	11.46165	0.00115	15.879	7.939521
6	100	15	1.00	0.021	0.01316	1.316	11.49221	0.00115	18.273	9.136621
7	120	17	1.20	0.024	0.01579	1.579	11.52294	0.00115	20.654	10.32722
8	140	21	1.40	0.029	0.01842	1.842	11.55383	0.00116	25.446	12.72305
9	160	25	1.60	0.035	0.02105	2.105	11.58489	0.00116	30.212	15.10588
10	180	28	1.80	0.039	0.02368	2.368	11.61612	0.00116	33.746	16.87311
11	200	33	2.00	0.046	0.02632	2.632	11.64751	0.00116	39.665	19.83256
12	220	37	2.20	0.052	0.02895	2.895	11.67908	0.00117	44.353	22.17641
13	240	41	2.40	0.057	0.03158	3.158	11.71082	0.00117	49.015	24.50726
14	260	46	2.60	0.064	0.03421	3.421	11.74272	0.00117	54.842	27.42123
15	280	49	2.80	0.069	0.03684	3.684	11.77481	0.00118	58.260	29.12998
16	300	53	3.00	0.074	0.03947	3.947	11.80707	0.00118	62.844	31.42186
17	320	56	3.20	0.078	0.04211	4.211	11.83951	0.00118	66.219	33.10949
18	340	59	3.40	0.083	0.04474	4.474	11.87212	0.00119	69.575	34.78738
19	360	61	3.60	0.085	0.04737	4.737	11.90492	0.00119	71.735	35.86753
20	380	64	3.80	0.090	0.05000	5.000	11.93789	0.00119	75.055	37.52755
21	400	66	4.00	0.092	0.05263	5.263	11.97106	0.00120	77.186	38.59309
22	420	68	4.20	0.095	0.05526	5.526	12.00440	0.00120	79.304	39.65212
23	440	69	4.40	0.097	0.05789	5.789	12.03793	0.00120	80.246	40.12317
24	460	70	4.60	0.098	0.06053	6.053	12.07165	0.00121	81.182	40.59096
25	480	71	4.80	0.099	0.06316	6.316	12.10556	0.00121	82.111	41.05551
26	500	72	5.00	0.101	0.06579	6.579	12.13966	0.00121	83.034	41.51681
27	520	73	5.20	0.102	0.06842	6.842	12.17395	0.00122	83.950	41.97486
28	540	74.5	5.40	0.104	0.07105	7.105	12.20844	0.00122	85.433	42.71634
29	560	76	5.60	0.106	0.07368	7.368	12.24313	0.00122	86.906	43.45296
30	580	77	5.80	0.108	0.07632	7.632	12.27801	0.00123	87.799	43.89964
31	600	78	6.00	0.109	0.07895	7.895	12.31309	0.00123	88.686	44.34307
32	620	79	6.20	0.111	0.08158	8.158	12.34837	0.00123	89.567	44.78325
33	640	80	6.40	0.112	0.08421	8.421	12.38385	0.00124	90.440	45.22018
34	660	82	6.60	0.115	0.08684	8.684	12.41954	0.00124	92.435	46.2175
35	680	82	6.80	0.115	0.08947	8.947	12.45543	0.00125	92.169	46.08431
36	700	84	7.00	0.118	0.09211	9.211	12.49154	0.00125	94.144	47.07187
37	720	85	7.20	0.119	0.09474	9.474	12.52785	0.00125	94.988	47.49419
38	740	85	7.40	0.119	0.09737	9.737	12.56437	0.00126	94.712	47.35612
39	760	86.5	7.60	0.121	0.10000	10.000	12.60111	0.00126	96.103	48.05132
40	780	87	7.80	0.122	0.10263	10.263	12.63806	0.00126	96.376	48.18776
41	800	88	8.00	0.123	0.10526	10.526	12.67524	0.00127	97.197	48.5987
42	820	88	8.20	0.123	0.10789	10.789	12.71263	0.00127	96.912	48.45577
43	840	88	8.40	0.123	0.11053	11.053	12.75024	0.00128	96.626	48.31283
44	860	88	8.60	0.123	0.11316	11.316	12.78807	0.00128	96.340	48.16989
45	880	87	8.80	0.122	0.11579	11.579	12.82613	0.00128	94.962	47.48119
46	900	86	9.00	0.120	0.11842	11.842	12.86442	0.00129	93.591	46.79574
47	920	86	9.20	0.120	0.12105	12.105	12.90293	0.00129	93.312	46.65605
48	940	83	9.40	0.116	0.12368	12.368	12.94168	0.00129	89.787	44.8937

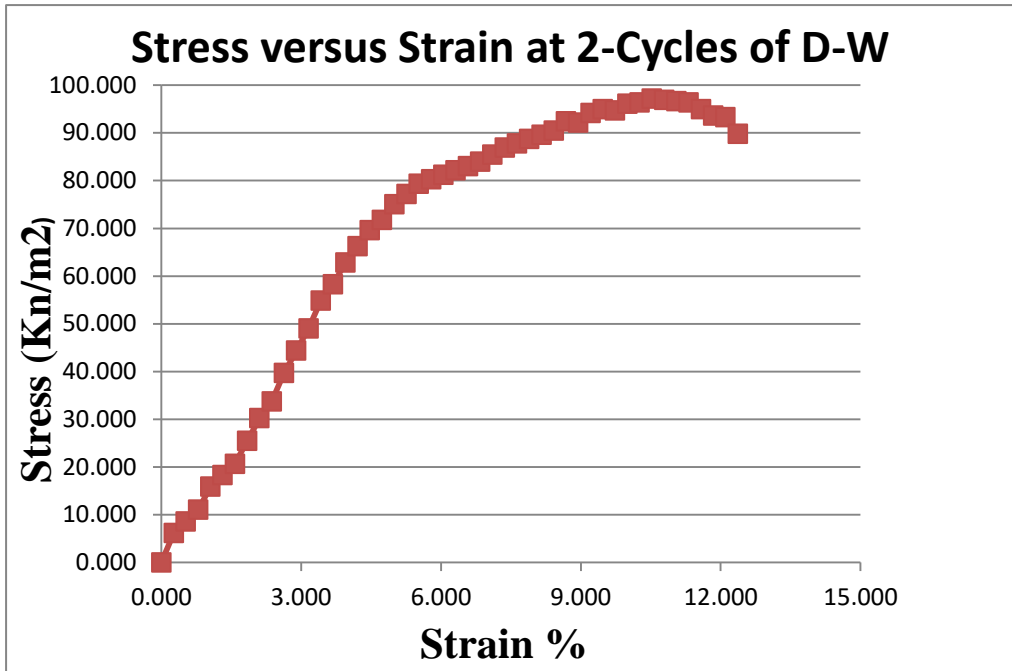


Figure iii: Stress versus Strain at 2-Cycles of D-W

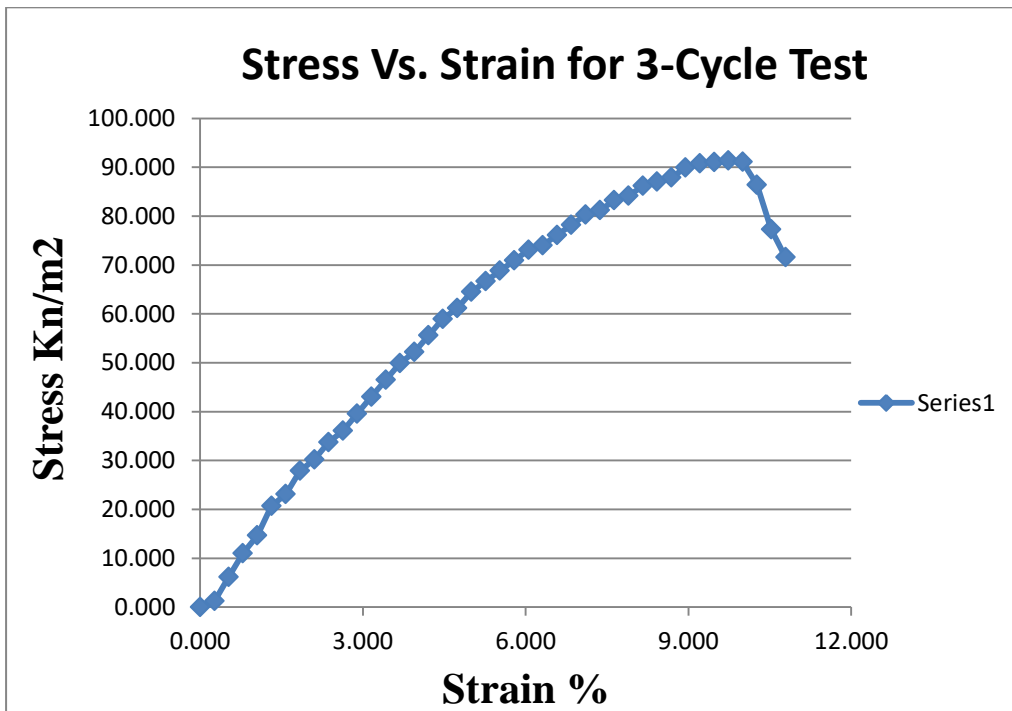


Figure iv: Stress Vs. Strain for 3-Cycle of D-W test

Table iv: UCS test data for 3-Cycle of D-W

S.No.	Dial gauge Reading	Proving ring Reading	CALCULATION							Remarks
			Deflection, ΔL	Load (KN)	Axial Strain,	Axial Strain, e	Corrected Area,	Area (m ²)	Stress =Load/A	
1	0	0	0.00	0.000	0.00000	0.000	11.34100	0.00113	0.000	0
2	20	1	0.20	0.001	0.00267	0.267	11.37132	0.00114	1.231	0.615584
3	40	5	0.40	0.007	0.00526	0.526	11.40101	0.00114	6.140	3.069905
4	60	9	0.60	0.013	0.00789	0.789	11.43125	0.00114	11.022	5.51121
5	80	12	0.80	0.017	0.01053	1.053	11.46165	0.00115	14.658	7.328788
6	100	17	1.00	0.024	0.01316	1.316	11.49221	0.00115	20.710	10.35484
7	120	19	1.20	0.027	0.01579	1.579	11.52294	0.00115	23.084	11.54219
8	140	23	1.40	0.032	0.01842	1.842	11.55383	0.00116	27.870	13.93477
9	160	25	1.60	0.035	0.02105	2.105	11.58489	0.00116	30.212	15.10588
10	180	28	1.80	0.039	0.02368	2.368	11.61612	0.00116	33.746	16.87311
11	200	30	2.00	0.042	0.02632	2.632	11.64751	0.00116	36.059	18.0296
12	220	33	2.20	0.046	0.02895	2.895	11.67908	0.00117	39.558	19.77896
13	240	36	2.40	0.050	0.03158	3.158	11.71082	0.00117	43.037	21.51857
14	260	39	2.60	0.055	0.03421	3.421	11.74272	0.00117	46.497	23.24844
15	280	42	2.80	0.059	0.03684	3.684	11.77481	0.00118	49.937	24.96856
16	300	44	3.00	0.062	0.03947	3.947	11.80707	0.00118	52.172	26.08607
17	320	47	3.20	0.066	0.04211	4.211	11.83951	0.00118	55.577	27.78832
18	340	50	3.40	0.070	0.04474	4.474	11.87212	0.00119	58.962	29.48083
19	360	52	3.60	0.073	0.04737	4.737	11.90492	0.00119	61.151	30.5756
20	380	55	3.80	0.077	0.05000	5.000	11.93789	0.00119	64.500	32.25024
21	400	57	4.00	0.080	0.05263	5.263	11.97106	0.00120	66.661	33.33039
22	420	59	4.20	0.083	0.05526	5.526	12.00440	0.00120	68.808	34.40405
23	440	61	4.40	0.085	0.05789	5.789	12.03793	0.00120	70.942	35.47121
24	460	63	4.60	0.088	0.06053	6.053	12.07165	0.00121	73.064	36.53187
25	480	64	4.80	0.090	0.06316	6.316	12.10556	0.00121	74.016	37.00778
26	500	66	5.00	0.092	0.06579	6.579	12.13966	0.00121	76.114	38.05707
27	520	68	5.20	0.095	0.06842	6.842	12.17395	0.00122	78.200	39.09987
28	540	70	5.40	0.098	0.07105	7.105	12.20844	0.00122	80.272	40.13616
29	560	71	5.60	0.099	0.07368	7.368	12.24313	0.00122	81.188	40.59421
30	580	73	5.80	0.102	0.07632	7.632	12.27801	0.00123	83.238	41.61914
31	600	74	6.00	0.104	0.07895	7.895	12.31309	0.00123	84.138	42.06906
32	620	76	6.20	0.106	0.08158	8.158	12.34837	0.00123	86.165	43.08262
33	640	77	6.40	0.108	0.08421	8.421	12.38385	0.00124	87.049	43.52443
34	660	78	6.60	0.109	0.08684	8.684	12.41954	0.00124	87.926	43.96298
35	680	80	6.80	0.112	0.08947	8.947	12.45543	0.00125	89.921	44.9603
36	700	81	7.00	0.113	0.09211	9.211	12.49154	0.00125	90.781	45.39073
37	720	81.5	7.20	0.114	0.09474	9.474	12.52785	0.00125	91.077	45.53854
38	740	82	7.40	0.115	0.09737	9.737	12.56437	0.00126	91.369	45.68473
39	760	82	7.60	0.115	0.10000	10.000	12.60111	0.00126	91.103	45.55154
40	780	78	7.80	0.109	0.10263	10.263	12.63806	0.00126	86.406	43.20282
41	800	70	8.00	0.098	0.10526	10.526	12.67524	0.00127	77.316	38.65806
42	820	65	8.20	0.091	0.10789	10.789	12.71263	0.00127	71.582	35.79119

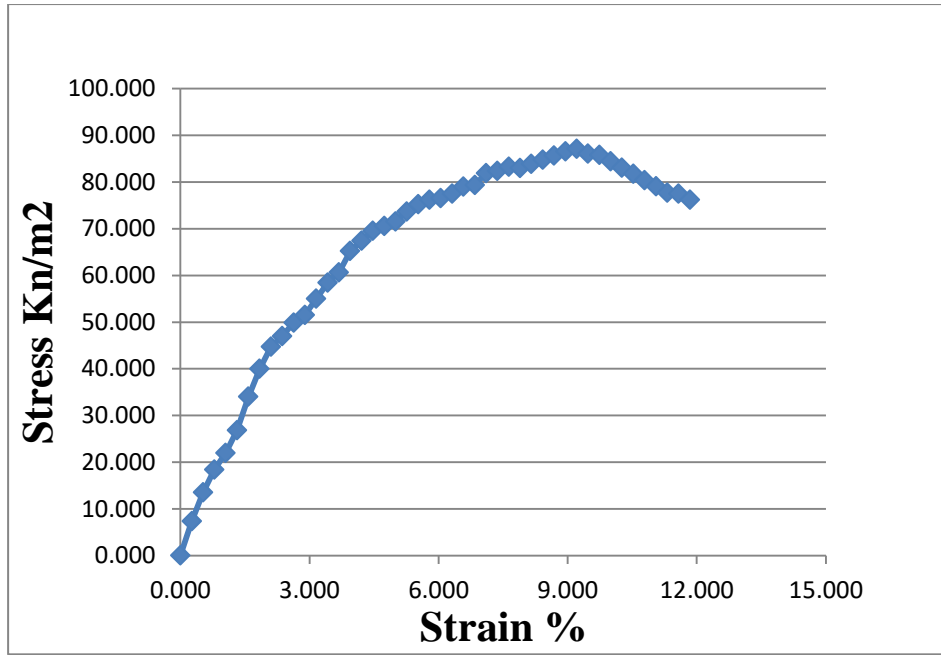


Figure v: Stress Vs. Strain for 4-Cycle of D-W

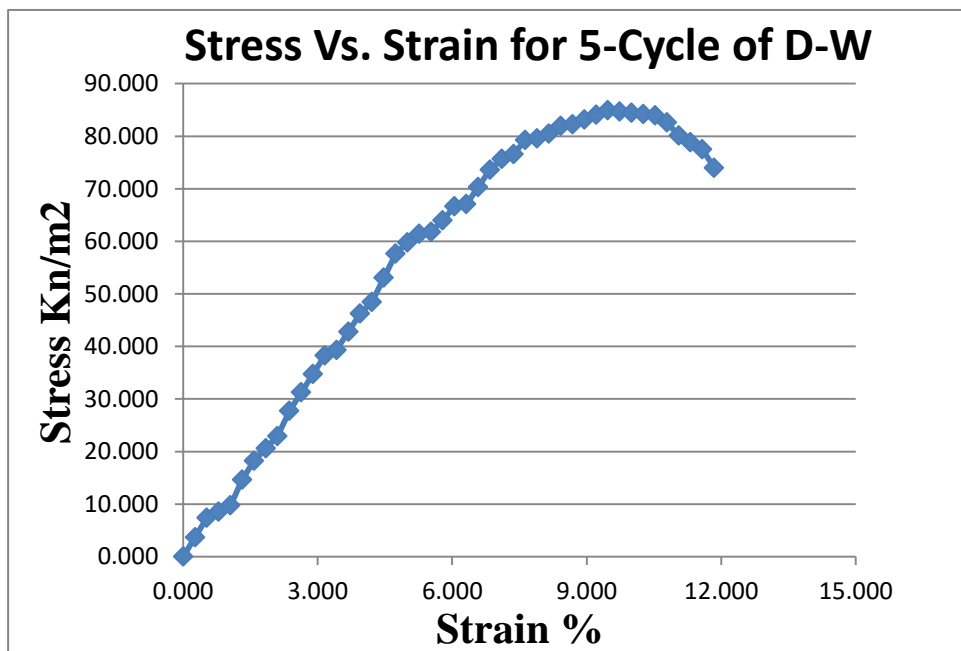


Figure vi: Stress Vs. Strain for 5-Cycle of D-W test

Table v: UCS test data for 4-Cycle of D-W

S.No.	Dial gauge Reading	Proving ring Reading	CALCULATION							Remarks
			Deflection, ΔL	Load (KN)	Axial Strain,	Axial Strain, e	Corrected Area,	Area (m ²)	Stress =Load/A	
1	0	0	0.00	0.000	0.00000	0.000	11.34100	0.00113	0.000	0
2	20	6	0.20	0.008	0.00267	0.267	11.37132	0.00114	7.387	3.693501
3	40	11	0.40	0.015	0.00526	0.526	11.40101	0.00114	13.508	6.75379
4	60	15	0.60	0.021	0.00789	0.789	11.43125	0.00114	18.371	9.18535
5	80	18	0.80	0.025	0.01053	1.053	11.46165	0.00115	21.986	10.99318
6	100	22	1.00	0.031	0.01316	1.316	11.49221	0.00115	26.801	13.40038
7	120	28	1.20	0.039	0.01579	1.579	11.52294	0.00115	34.019	17.00955
8	140	33	1.40	0.046	0.01842	1.842	11.55383	0.00116	39.987	19.99336
9	160	37	1.60	0.052	0.02105	2.105	11.58489	0.00116	44.713	22.3567
10	180	39	1.80	0.055	0.02368	2.368	11.61612	0.00116	47.004	23.50183
11	200	41.5	2.00	0.058	0.02632	2.632	11.64751	0.00116	49.882	24.94095
12	220	43	2.20	0.060	0.02895	2.895	11.67908	0.00117	51.545	25.77258
13	240	46	2.40	0.064	0.03158	3.158	11.71082	0.00117	54.992	27.49595
14	260	49	2.60	0.069	0.03421	3.421	11.74272	0.00117	58.419	29.20957
15	280	51	2.80	0.071	0.03684	3.684	11.77481	0.00118	60.638	30.31896
16	300	55	3.00	0.077	0.03947	3.947	11.80707	0.00118	65.215	32.60759
17	320	57	3.20	0.080	0.04211	4.211	11.83951	0.00118	67.401	33.70073
18	340	59	3.40	0.083	0.04474	4.474	11.87212	0.00119	69.575	34.78738
19	360	60	3.60	0.084	0.04737	4.737	11.90492	0.00119	70.559	35.27954
20	380	61	3.80	0.085	0.05000	5.000	11.93789	0.00119	71.537	35.76845
21	400	63	4.00	0.088	0.05263	5.263	11.97106	0.00120	73.678	36.83886
22	420	64.5	4.20	0.090	0.05526	5.526	12.00440	0.00120	75.222	37.61121
23	440	65.5	4.40	0.092	0.05789	5.789	12.03793	0.00120	76.176	38.08793
24	460	66	4.60	0.092	0.06053	6.053	12.07165	0.00121	76.543	38.27148
25	480	67	4.80	0.094	0.06316	6.316	12.10556	0.00121	77.485	38.74252
26	500	68.5	5.00	0.096	0.06579	6.579	12.13966	0.00121	78.997	39.49863
27	520	69	5.20	0.097	0.06842	6.842	12.17395	0.00122	79.350	39.67486
28	540	71.4	5.40	0.100	0.07105	7.105	12.20844	0.00122	81.878	40.93888
29	560	72	5.60	0.101	0.07368	7.368	12.24313	0.00122	82.332	41.16596
30	580	73	5.80	0.102	0.07632	7.632	12.27801	0.00123	83.238	41.61914
31	600	73	6.00	0.102	0.07895	7.895	12.31309	0.00123	83.001	41.50056
32	620	74	6.20	0.104	0.08158	8.158	12.34837	0.00123	83.898	41.94887
33	640	75	6.40	0.105	0.08421	8.421	12.38385	0.00124	84.788	42.39392
34	660	76	6.60	0.106	0.08684	8.684	12.41954	0.00124	85.671	42.83573
35	680	77	6.80	0.108	0.08947	8.947	12.45543	0.00125	86.549	43.27429
36	700	77.7	7.00	0.109	0.09211	9.211	12.49154	0.00125	87.083	43.54148
37	720	77	7.20	0.108	0.09474	9.474	12.52785	0.00125	86.048	43.02415
38	740	77	7.40	0.108	0.09737	9.737	12.56437	0.00126	85.798	42.89908
39	760	76	7.60	0.106	0.10000	10.000	12.60111	0.00126	84.437	42.2185
40	780	75	7.80	0.105	0.10263	10.263	12.63806	0.00126	83.082	41.54117
41	800	74	8	0.104	0.10526	10.526	12.67524	0.00127	81.734	40.86709
42	820	73	8.20	0.102	0.10789	10.789	12.71263	0.00127	80.393	40.19626
43	840	72	8.40	0.101	0.11053	11.053	12.75024	0.00128	79.057	39.52868
44	860	71	8.60	0.099	0.11316	11.316	12.78807	0.00128	77.729	38.86434
45	880	71	8.80	0.099	0.11579	11.579	12.82613	0.00128	77.498	38.74902
46	900	70	9.00	0.098	0.11842	11.842	12.86442	0.00129	76.179	38.08956

Table vi: UCS test data for 5-Cycle of D-W test

S.No.	Dial gauge Reading	Proving ring Reading	CALCULATION							Remarks
			Deflection, ΔL	Load (KN)	Axial Strain,	Axial Strain, e	Corrected Area,	Area (m ²)	Stress =Load/A	
1	0	0	0.00	0.000	0.00000	0.000	11.34100	0.00113	0.000	0
2	20	3	0.20	0.004	0.00267	0.267	11.37132	0.00114	3.694	1.846751
3	40	6	0.40	0.008	0.00526	0.526	11.40101	0.00114	7.368	3.683886
4	60	7	0.60	0.010	0.00789	0.789	11.43125	0.00114	8.573	4.286497
5	80	8	0.80	0.011	0.01053	1.053	11.46165	0.00115	9.772	4.885859
6	100	12	1.00	0.017	0.01316	1.316	11.49221	0.00115	14.619	7.309297
7	120	15	1.20	0.021	0.01579	1.579	11.52294	0.00115	18.225	9.112257
8	140	17	1.40	0.024	0.01842	1.842	11.55383	0.00116	20.599	10.29961
9	160	19	1.60	0.027	0.02105	2.105	11.58489	0.00116	22.961	11.48047
10	180	23	1.80	0.032	0.02368	2.368	11.61612	0.00116	27.720	13.86005
11	200	26	2.00	0.036	0.02632	2.632	11.64751	0.00116	31.251	15.62565
12	220	29	2.20	0.041	0.02895	2.895	11.67908	0.00117	34.763	17.38151
13	240	32	2.40	0.045	0.03158	3.158	11.71082	0.00117	38.255	19.12762
14	260	33	2.60	0.046	0.03421	3.421	11.74272	0.00117	39.344	19.67175
15	280	36	2.80	0.050	0.03684	3.684	11.77481	0.00118	42.803	21.40162
16	300	39	3.00	0.055	0.03947	3.947	11.80707	0.00118	46.243	23.12174
17	320	41	3.20	0.057	0.04211	4.211	11.83951	0.00118	48.482	24.24088
18	340	45	3.40	0.063	0.04474	4.474	11.87212	0.00119	53.065	26.53275
19	360	49	3.60	0.069	0.04737	4.737	11.90492	0.00119	57.623	28.81162
20	380	51	3.80	0.071	0.05000	5.000	11.93789	0.00119	59.810	29.90477
21	400	52.5	4.00	0.074	0.05263	5.263	11.97106	0.00120	61.398	30.69905
22	420	53	4.20	0.074	0.05526	5.526	12.00440	0.00120	61.811	30.90533
23	440	55	4.40	0.077	0.05789	5.789	12.03793	0.00120	63.964	31.98223
24	460	57.5	4.60	0.081	0.06053	6.053	12.07165	0.00121	66.685	33.34258
25	480	58	4.80	0.081	0.06316	6.316	12.10556	0.00121	67.077	33.5383
26	500	61	5.00	0.085	0.06579	6.579	12.13966	0.00121	70.348	35.17396
27	520	64	5.20	0.090	0.06842	6.842	12.17395	0.00122	73.600	36.79987
28	540	66	5.40	0.092	0.07105	7.105	12.20844	0.00122	75.685	37.84267
29	560	67	5.60	0.094	0.07368	7.368	12.24313	0.00122	76.614	38.30721
30	580	69.5	5.80	0.097	0.07632	7.632	12.27801	0.00123	79.247	39.6237
31	600	70	6.00	0.098	0.07895	7.895	12.31309	0.00123	79.590	39.79506
32	620	71	6.20	0.099	0.08158	8.158	12.34837	0.00123	80.496	40.24824
33	640	72.5	6.40	0.102	0.08421	8.421	12.38385	0.00124	81.962	40.98079
34	660	73	6.60	0.102	0.08684	8.684	12.41954	0.00124	82.290	41.14484
35	680	74	6.80	0.104	0.08947	8.947	12.45543	0.00125	83.177	41.58828
36	700	75	7.00	0.105	0.09211	9.211	12.49154	0.00125	84.057	42.02846
37	720	76	7.20	0.106	0.09474	9.474	12.52785	0.00125	84.684	42.34195
38	740	76	7.40	0.106	0.09737	9.737	12.56437	0.00126	84.684	42.34195
39	760	75.5	7.60	0.106	0.10000	10.000	12.60111	0.00126	83.881	41.94075
40	780	75.5	7.80	0.106	0.10263	10.263	12.63806	0.00126	83.636	41.81811
41	800	75	8	0.105	0.10526	10.526	12.67524	0.00127	82.839	41.41935
42	820	75	8.20	0.105	0.10789	10.789	12.71263	0.00127	82.595	41.29753
43	840	73	8.40	0.102	0.11053	11.053	12.75024	0.00128	80.155	40.07769
44	860	72	8.60	0.101	0.11316	11.316	12.78807	0.00128	78.823	39.41173
45	880	71	8.80	0.099	0.11579	11.579	12.82613	0.00128	77.498	38.74902
46	900	68	9.00	0.095	0.11842	11.842	12.86442	0.00129	74.003	37.00129

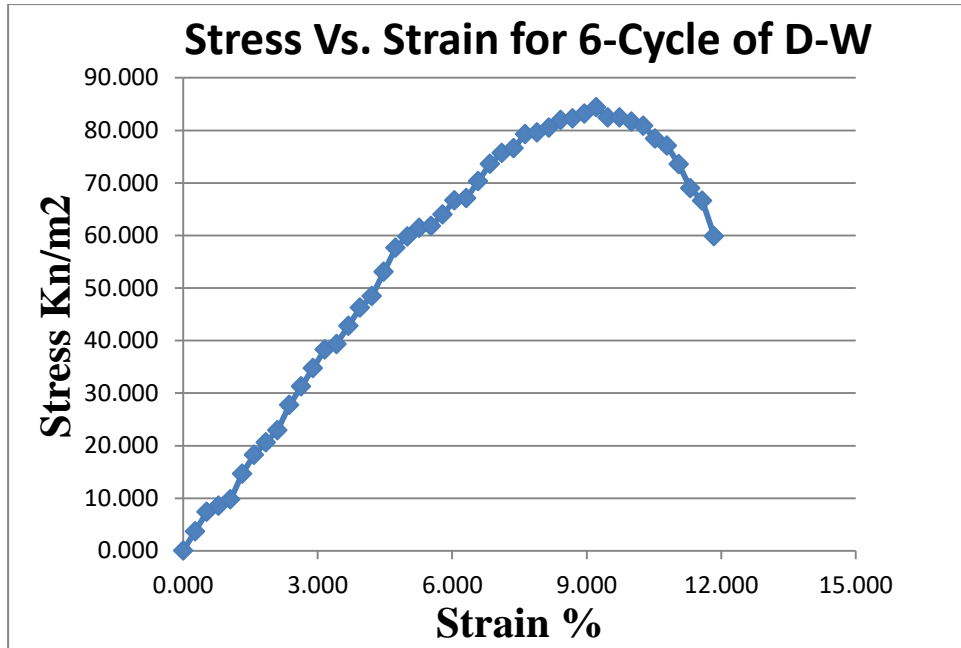


Figure vii: UCS test data for 6-Cycles of D-W test

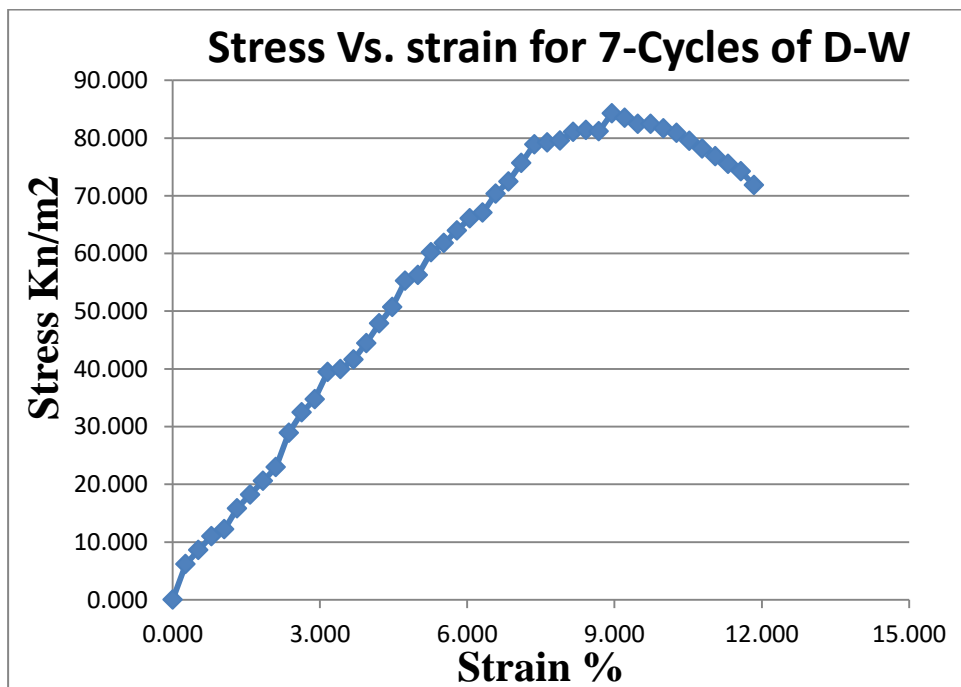


Figure viii: Stress Vs. Strain for 7-Cycles of D-W test

Table vii: UCS test data for 6-Cycles of D-W test

S.No.	Dial gauge Reading	Proving ring Reading	CALCULATION							Remarks
			Deflection, ΔL	Load (KN)	Axial Strain,	Axial Strain, e	Corrected Area,	Area (m ²)	Stress =Load/A	
1	0	0	0.00	0.000	0.00000	0.000	11.34100	0.00113	0.000	0
2	20	3	0.20	0.004	0.00267	0.267	11.37132	0.00114	3.694	1.846751
3	40	6	0.40	0.008	0.00526	0.526	11.40101	0.00114	7.368	3.683886
4	60	7	0.60	0.010	0.00789	0.789	11.43125	0.00114	8.573	4.286497
5	80	8	0.80	0.011	0.01053	1.053	11.46165	0.00115	9.772	4.885859
6	100	12	1.00	0.017	0.01316	1.316	11.49221	0.00115	14.619	7.309297
7	120	15	1.20	0.021	0.01579	1.579	11.52294	0.00115	18.225	9.112257
8	140	17	1.40	0.024	0.01842	1.842	11.55383	0.00116	20.599	10.29961
9	160	19	1.60	0.027	0.02105	2.105	11.58489	0.00116	22.961	11.48047
10	180	23	1.80	0.032	0.02368	2.368	11.61612	0.00116	27.720	13.86005
11	200	26	2.00	0.036	0.02632	2.632	11.64751	0.00116	31.251	15.62565
12	220	29	2.20	0.041	0.02895	2.895	11.67908	0.00117	34.763	17.38151
13	240	32	2.40	0.045	0.03158	3.158	11.71082	0.00117	38.255	19.12762
14	260	33	2.60	0.046	0.03421	3.421	11.74272	0.00117	39.344	19.67175
15	280	36	2.80	0.050	0.03684	3.684	11.77481	0.00118	42.803	21.40162
16	300	39	3.00	0.055	0.03947	3.947	11.80707	0.00118	46.243	23.12174
17	320	41	3.20	0.057	0.04211	4.211	11.83951	0.00118	48.482	24.24088
18	340	45	3.40	0.063	0.04474	4.474	11.87212	0.00119	53.065	26.53275
19	360	49	3.60	0.069	0.04737	4.737	11.90492	0.00119	57.623	28.81162
20	380	51	3.80	0.071	0.05000	5.000	11.93789	0.00119	59.810	29.90477
21	400	52.5	4.00	0.074	0.05263	5.263	11.97106	0.00120	61.398	30.69905
22	420	53	4.20	0.074	0.05526	5.526	12.00440	0.00120	61.811	30.90533
23	440	55	4.40	0.077	0.05789	5.789	12.03793	0.00120	63.964	31.98223
24	460	57.5	4.60	0.081	0.06053	6.053	12.07165	0.00121	66.685	33.34258
25	480	58	4.80	0.081	0.06316	6.316	12.10556	0.00121	67.077	33.5383
26	500	61	5.00	0.085	0.06579	6.579	12.13966	0.00121	70.348	35.17396
27	520	64	5.20	0.090	0.06842	6.842	12.17395	0.00122	73.600	36.79987
28	540	66	5.40	0.092	0.07105	7.105	12.20844	0.00122	75.685	37.84267
29	560	67	5.60	0.094	0.07368	7.368	12.24313	0.00122	76.614	38.30721
30	580	69.5	5.80	0.097	0.07632	7.632	12.27801	0.00123	79.247	39.6237
31	600	70	6.00	0.098	0.07895	7.895	12.31309	0.00123	79.590	39.79506
32	620	71	6.20	0.099	0.08158	8.158	12.34837	0.00123	80.496	40.24824
33	640	72.5	6.40	0.102	0.08421	8.421	12.38385	0.00124	81.962	40.98079
34	660	73	6.60	0.102	0.08684	8.684	12.41954	0.00124	82.290	41.14484
35	680	74	6.80	0.104	0.08947	8.947	12.45543	0.00125	83.177	41.58828
36	700	75	7.00	0.105	0.09211	9.211	12.49154	0.00125	84.375	42.1875
37	720	74.5	7.20	0.104	0.09474	9.474	12.52785	0.00125	82.455	41.22768
38	740	74	7.40	0.104	0.09737	9.737	12.56437	0.00126	82.455	41.22768
39	760	73.5	7.60	0.103	0.10000	10.000	12.60111	0.00126	81.659	40.82973
40	780	73	7.80	0.102	0.10263	10.263	12.63806	0.00126	80.867	40.43341
41	800	71	8	0.099	0.10526	10.526	12.67524	0.00127	78.421	39.21032
42	820	70	8.20	0.098	0.10789	10.789	12.71263	0.00127	77.089	38.54436
43	840	67	8.40	0.094	0.11053	11.053	12.75024	0.00128	73.567	36.78363
44	860	63	8.60	0.088	0.11316	11.316	12.78807	0.00128	68.971	34.48526
45	880	61	8.80	0.085	0.11579	11.579	12.82613	0.00128	66.583	33.29141
46	900	55	9.00	0.077	0.11842	11.842	12.86442	0.00129	59.855	29.92751

Table viii: UCS test data for 7-Cycles of D-W test

S.No.	Dial gauge Reading	Proving ring Reading	CALCULATION							Remarks
			Deflection, ΔL	Load (KN)	Axial Strain,	Axial Strain, e	Corrected Area,	Area (m ²)	Stress =Load/A	
1	0	0	0.00	0.000	0.00000	0.000	11.34100	0.00113	0.000	0
2	20	5	0.20	0.007	0.00267	0.267	11.37132	0.00114	6.156	3.077918
3	40	7	0.40	0.010	0.00526	0.526	11.40101	0.00114	8.596	4.297867
4	60	9	0.60	0.013	0.00789	0.789	11.43125	0.00114	11.022	5.51121
5	80	10	0.80	0.014	0.01053	1.053	11.46165	0.00115	12.215	6.107324
6	100	13	1.00	0.018	0.01316	1.316	11.49221	0.00115	15.837	7.918405
7	120	15	1.20	0.021	0.01579	1.579	11.52294	0.00115	18.225	9.112257
8	140	17	1.40	0.024	0.01842	1.842	11.55383	0.00116	20.599	10.29961
9	160	19	1.60	0.027	0.02105	2.105	11.58489	0.00116	22.961	11.48047
10	180	24	1.80	0.034	0.02368	2.368	11.61612	0.00116	28.925	14.46266
11	200	27	2.00	0.038	0.02632	2.632	11.64751	0.00116	32.453	16.22664
12	220	29	2.20	0.041	0.02895	2.895	11.67908	0.00117	34.763	17.38151
13	240	33	2.40	0.046	0.03158	3.158	11.71082	0.00117	39.451	19.72536
14	260	33.5	2.60	0.047	0.03421	3.421	11.74272	0.00117	39.940	19.96981
15	280	35	2.80	0.049	0.03684	3.684	11.77481	0.00118	41.614	20.80713
16	300	37.5	3.00	0.053	0.03947	3.947	11.80707	0.00118	44.465	22.23244
17	320	40.5	3.20	0.057	0.04211	4.211	11.83951	0.00118	47.891	23.94526
18	340	43	3.40	0.060	0.04474	4.474	11.87212	0.00119	50.707	25.35351
19	360	47	3.60	0.066	0.04737	4.737	11.90492	0.00119	55.271	27.63564
20	380	48	3.80	0.067	0.05000	5.000	11.93789	0.00119	56.291	28.14567
21	400	51.5	4.00	0.072	0.05263	5.263	11.97106	0.00120	60.229	30.1143
22	420	53	4.20	0.074	0.05526	5.526	12.00440	0.00120	61.811	30.90533
23	440	55	4.40	0.077	0.05789	5.789	12.03793	0.00120	63.964	31.98223
24	460	57	4.60	0.080	0.06053	6.053	12.07165	0.00121	66.105	33.05264
25	480	58	4.80	0.081	0.06316	6.316	12.10556	0.00121	67.077	33.5383
26	500	61	5.00	0.085	0.06579	6.579	12.13966	0.00121	70.348	35.17396
27	520	63	5.20	0.088	0.06842	6.842	12.17395	0.00122	72.450	36.22488
28	540	66	5.40	0.092	0.07105	7.105	12.20844	0.00122	75.685	37.84267
29	560	69	5.60	0.097	0.07368	7.368	12.24313	0.00122	78.901	39.45071
30	580	69.5	5.80	0.097	0.07632	7.632	12.27801	0.00123	79.247	39.6237
31	600	70	6.00	0.098	0.07895	7.895	12.31309	0.00123	79.590	39.79506
32	620	71.5	6.20	0.100	0.08158	8.158	12.34837	0.00123	81.063	40.53168
33	640	72	6.40	0.101	0.08421	8.421	12.38385	0.00124	81.396	40.69817
34	660	72	6.60	0.101	0.08684	8.684	12.41954	0.00124	81.162	40.58122
35	680	75	6.80	0.105	0.08947	8.947	12.45543	0.00125	84.297	42.1485
36	700	74.5	7.00	0.104	0.09211	9.211	12.49154	0.00125	83.497	41.74827
37	720	74	7.20	0.104	0.09474	9.474	12.52785	0.00125	82.455	41.22768
38	740	74	7.40	0.104	0.09737	9.737	12.56437	0.00126	82.455	41.22768
39	760	73.5	7.60	0.103	0.10000	10.000	12.60111	0.00126	81.659	40.82973
40	780	73	7.80	0.102	0.10263	10.263	12.63806	0.00126	80.867	40.43341
41	800	72	8	0.101	0.10526	10.526	12.67524	0.00127	79.525	39.76258
42	820	71	8.20	0.099	0.10789	10.789	12.71263	0.00127	78.190	39.09499
43	840	70	8.40	0.098	0.11053	11.053	12.75024	0.00128	76.861	38.43066
44	860	69	8.60	0.097	0.11316	11.316	12.78807	0.00128	75.539	37.76957
45	880	68	8.80	0.095	0.11579	11.579	12.82613	0.00128	74.223	37.11174
46	900	66	9.00	0.092	0.11842	11.842	12.86442	0.00129	71.826	35.91301

Table ix: UCS test data for 8-Cycles of D-W test

S.No.	Dial gauge Reading	Proving ring Reading	CALCULATION							Remarks
			Deflection, ΔL	Load (KN)	Axial Strain,	Axial Strain, e	Corrected Area,	Area (m ²)	Stress =Load/A	
1	0	0	0.00	0.000	0.00000	0.000	11.34100	0.00113	0.000	0
2	20	2	0.20	0.003	0.00267	0.267	11.37132	0.00114	2.462	1.231167
3	40	5	0.40	0.007	0.00526	0.526	11.40101	0.00114	6.140	3.069905
4	60	9	0.60	0.013	0.00789	0.789	11.43125	0.00114	11.022	5.51121
5	80	10	0.80	0.014	0.01053	1.053	11.46165	0.00115	12.215	6.107324
6	100	13	1.00	0.018	0.01316	1.316	11.49221	0.00115	15.837	7.918405
7	120	15	1.20	0.021	0.01579	1.579	11.52294	0.00115	18.225	9.112257
8	140	17	1.40	0.024	0.01842	1.842	11.55383	0.00116	20.599	10.29961
9	160	21	1.60	0.029	0.02105	2.105	11.58489	0.00116	25.378	12.68894
10	180	22	1.80	0.031	0.02368	2.368	11.61612	0.00116	26.515	13.25744
11	200	25	2.00	0.035	0.02632	2.632	11.64751	0.00116	30.049	15.02467
12	220	29	2.20	0.041	0.02895	2.895	11.67908	0.00117	34.763	17.38151
13	240	32	2.40	0.045	0.03158	3.158	11.71082	0.00117	38.255	19.12762
14	260	34	2.60	0.048	0.03421	3.421	11.74272	0.00117	40.536	20.26787
15	280	36	2.80	0.050	0.03684	3.684	11.77481	0.00118	42.803	21.40162
16	300	38	3.00	0.053	0.03947	3.947	11.80707	0.00118	45.058	22.52888
17	320	40	3.20	0.056	0.04211	4.211	11.83951	0.00118	47.299	23.64964
18	340	44	3.40	0.062	0.04474	4.474	11.87212	0.00119	51.886	25.94313
19	360	47	3.60	0.066	0.04737	4.737	11.90492	0.00119	55.271	27.63564
20	380	50	3.80	0.070	0.05000	5.000	11.93789	0.00119	58.637	29.3184
21	400	52.5	4.00	0.074	0.05263	5.263	11.97106	0.00120	61.398	30.69905
22	420	54.5	4.20	0.076	0.05526	5.526	12.00440	0.00120	63.560	31.78001
23	440	55.5	4.40	0.078	0.05789	5.789	12.03793	0.00120	64.546	32.27298
24	460	57	4.60	0.080	0.06053	6.053	12.07165	0.00121	66.105	33.05264
25	480	59	4.80	0.083	0.06316	6.316	12.10556	0.00121	68.233	34.11655
26	500	61	5.00	0.085	0.06579	6.579	12.13966	0.00121	70.348	35.17396
27	520	63	5.20	0.088	0.06842	6.842	12.17395	0.00122	72.450	36.22488
28	540	65	5.40	0.091	0.07105	7.105	12.20844	0.00122	74.539	37.26929
29	560	67	5.60	0.094	0.07368	7.368	12.24313	0.00122	76.614	38.30721
30	580	68	5.80	0.095	0.07632	7.632	12.27801	0.00123	77.537	38.76851
31	600	69.5	6.00	0.097	0.07895	7.895	12.31309	0.00123	79.022	39.51081
32	620	70.5	6.20	0.099	0.08158	8.158	12.34837	0.00123	79.930	39.9648
33	640	71	6.40	0.099	0.08421	8.421	12.38385	0.00124	80.266	40.13291
34	660	72	6.60	0.101	0.08684	8.684	12.41954	0.00124	81.162	40.58122
35	680	73	6.80	0.102	0.08947	8.947	12.45543	0.00125	82.053	41.02627
36	700	74.5	7.00	0.104	0.09211	9.211	12.49154	0.00125	84.070	42.035
37	720	74	7.20	0.104	0.09474	9.474	12.52785	0.00125	81.898	40.94912
38	740	73.5	7.40	0.103	0.09737	9.737	12.56437	0.00126	81.898	40.94912
39	760	73	7.60	0.102	0.10000	10.000	12.60111	0.00126	81.104	40.55198
40	780	73	7.80	0.102	0.10263	10.263	12.63806	0.00126	80.867	40.43341
41	800	72	8	0.101	0.10526	10.526	12.67524	0.00127	79.525	39.76258
42	820	71	8.20	0.099	0.10789	10.789	12.71263	0.00127	78.190	39.09499
43	840	67	8.40	0.094	0.11053	11.053	12.75024	0.00128	73.567	36.78363
44	860	63	8.60	0.088	0.11316	11.316	12.78807	0.00128	68.971	34.48526
45	880	61	8.80	0.085	0.11579	11.579	12.82613	0.00128	66.583	33.29141
46	900	55	9.00	0.077	0.11842	11.842	12.86442	0.00129	59.855	29.92751

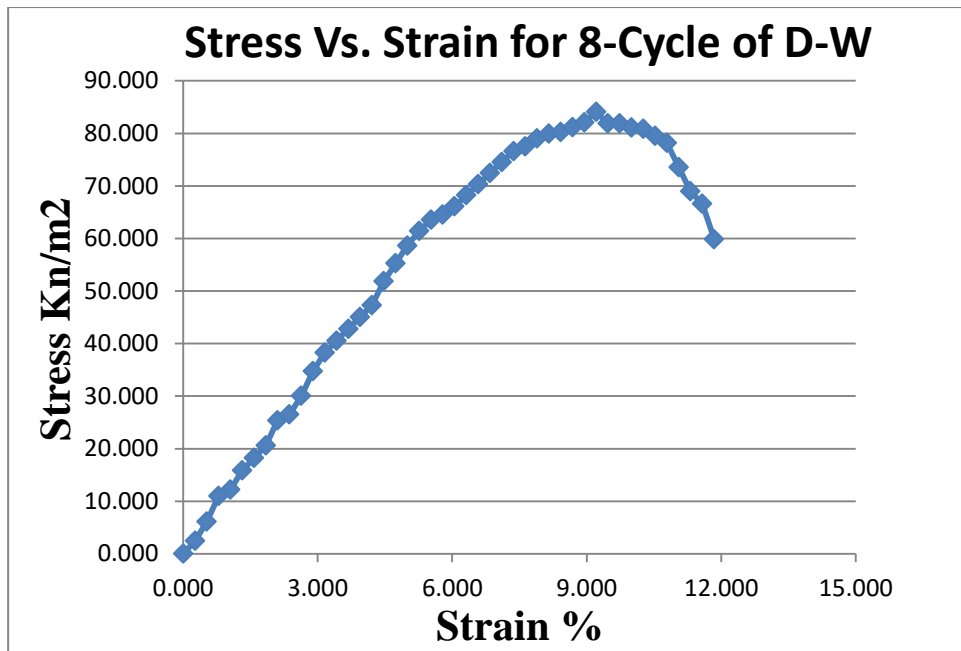


Figure ix: Stress Vs. Strain for 8-Cycles of D-W test

ANNEX II: Glimpse of laboratory works



Figure x: Wetting Phase of Soil Samples



Figure xi: Drying Phase of Soil Samples



Figure xii: Soil Sample after UCS test



Figure xiii: Water Content Determination for PL test



Figure xiv: Compacting the Soil Sample for SPT



Figure xv: Preparation of CBR test in Lab.



Figure xvi: Soil sample ready for CBR test after dynamic compaction



Figure xvii: Soil sample after oven drying



Figure xviii: weighing the soil sample for water content determination



Figure xix: Soil sample for wet sieving



Figure xx: Soil sample weighing for density measurement



Figure xxi: Soil sample in compaction mould ready for SPT test



Figure xxii: Soil sample after immersion in water for 24 hours