DAMAGES AND REMEDIAL MEASURES DUE TO EXPANSIVE SOIL ON THE BUILDINGS IN BISHOFTU TOWN

Mesfin Alemayehu Bejiga

A Thesis Submitted to
The department of Civil Engineering,
School of Civil Engineering and Architecture

Presented in Partial Fulfillment of the Requirement for the Degree of Master's in Civil Engineering (Specialization in Geotechnical Engineering)

Office of Graduate Studies
Adama Science and Technology University

February, 2023

Adama, Ethiopia

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DECLARATION

I declare that this Master Thesis entitled "**Damages and remedial measures due to expansive soil on the buildings in Bishoftu town**" is my original work. That is, it has not been submitted for the award of any academic degree, diploma or certificate in any other university. All sources of materials that are used for this thesis have been duly acknowledged through citation.

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I, the advisor of this thesis, hereby certify that I have read the revised version of the thesis entitled "**Damages and remedial measures due to expansive soil on the buildings in Bishoftu town**" prepared under my guidance by **Mesfin Alemayehu Bejiga** submitted in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering (specialization in Geotechnical Engineering). Therefore, I recommend the submission of revised version of the thesis to the department following the applicable procedures.

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APPROVAL PAGE

I, the advisor of the thesis entitled "**Damages and remedial measures due to expansive soil on the buildings in Bishoftu town**" and developed by **Mesfin Alemayehu Bejiga**, hereby certify that the recommendation and suggestions made by the board of examiners are appropriately incorporated into the final version of the thesis.

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We, the undersigned, members of the Board of Examiners of the thesis by **Mesfin Alemayehu Bejiga** have read and evaluated the thesis entitled "**Damages and remedial measures due to expansive soil on the buildings in Bishoftu town**" and examined the candidate during open defense. This is, therefore, to certify that the thesis is accepted for partial fulfillment of the requirement of the degree of Master of Science in Civil Engineering (specialization in Geotechnical Engineering).

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LIST OF ACRONYMS AND ABBREVIATIONS

AASHTO American Association of State Highway and Transportation Officials

ASTM American Society for Testing and Materials

A Area

A_c Activity

A^o Angstrom

C_c Compression index

C_r Recompression index

C_v Coefficient of consolidation

CE Compaction energy

CIS Corrugated iron sheet

DFS Differential free swell

DGR Dial gauge reading

DTA Differential thermal analysis

d_p Relative desired precision

d Drainage path in consolidation

E Easting

e Void ratio

e_o Initial void ratio

Fs Free swell

GPS Global positioning system

G_s Specific Gravity

H Height of sample in consolidation

HCB Hollow concrete block

H_s Active depth

H_d Depth of desiccation

LIR Load increment ratio

LL Liquid Limit

LI Liquidity Index

Ls Linear Shrinkage

L_o Original sample length

L_f Final oven dried sample length

N Northing

n	Sample size to be studied
p	Expected prevalence (sample proportion which are damaged)
PL	Plastic Limit
PI	Plasticity Index
P_c	Pre-consolidation Pressure
q	Expected non-prevalence (sample proportion which are not damaged)
r_b	Bulk Density
r_d	Dry Density
SL	Shrinkage Limit
SEM	Scanning Electron Microscope
S_p	Swelling potential
T_U	Time factor for U% consolidation
USCS	Unified Soil Classification System
V	Volume
V_w	Volume of the soil specimen read from graduated cylinder containing distilledwater
V_k	Volume of the soil specimen read from graduated cylinder containing kerosene
w	Water content
XRD	X-ray diffraction
z	Standard normal value
2D	Two Dimensional
3D	Three Dimensional

UNITS

Length		**Volume**		**Mass**		**Force**	
µm	Micrometer	ml	Milliliter	g	Gram	N	Newton
mm	Millimeter	l	Liter	kg	Kilogram	kN	Kilo newton
cm	Centimeter						
m	Meter						
km	Kilometer						

Pressure		**Energy**		**Temperature**	
kPa	Kilo pascal	kJ	Kilo joule	^oC	Degree centigrade

ABSTRACT

Damage caused by expansive soil on buildings constructed in the Bishoftu town are investigated and remedial measures proposed. Forty four buildings were randomly selected from different expansive soil locations. The investigation showed that out of those 44 buildings 36 or 82% of them are adversely affected. The extent of damage with respect to building type is very high on mud/cheka house (91%) and decreases when we proceed from G+0 HCB buildings (88%), G+1 buildings (67%) and G+2 buildings (50%) respectively. The damage has been analyzed and interpreted systematically with respect to type of foundation, wall, floor, pavement, door, window and fence. According to AASHTO classification system the study area expansive soil are grouped in A-7-5 and A-7-6 and with the USCS the soil are grouped in CH and MH. The test result showed that, w ranges from 26.73-49.03%, LL ranges from 61.6-94.4%, PI ranges from 23.8-54%, LS ranges from 10.84-22.36, FS ranges from 21.74-81.82%, G_s ranges from 2.67-2.72, % finer than 0.075mm ranges from 82.8-95.5% and OMC ranges from 28.6-37.83% and MDD ranges 1.22-1.42g/cm^3. Cv ranges from 0.334-3.049m^2/year, Cc ranges from 0.369-0.555 and e_o ranges from 1.37-1.68. The study showed that the main causes of failures are improper foundation design, construction with low quality materials, moisture fluctuation, poor surface drainage, damaged utility lines, presence of trees and vegetation close to buildings. For the past nearly 8 months 6 instruments 2Dimensional crack monitors on 3 cracked buildings were installed, which is used as a precautionary measure. Additional crack opening on Building ID No.26 at vertical crack 1.29mm was observed. Inserting helical bar along and across the crack, regulate and reduces crack movement. Nowadays people are aware that building a new house on black cotton soil on the study area, for G+0 buildings with trench excavation at 1m depth replacing the expansive foundation soils by non-heaving materials and providing plinth beam foundation is recommended. For G+1 and G+2 buildings with 1.5m and 2m excavation depth respectively a cushion of freely draining granular soil is placed and compacted which acts as a barrier and the effect of swelling on the foundation is reduced.

Keywords: *Expansive Soil. Damaged. Remedial Measures. 2-Dimensional Crack Monitor.*

CHAPTER 1

INTRODUCTION

1.1 General

In Ethiopia, all over almost and particularly in and around Bishoftu town the common soil is the black cotton clayey soil that seasonally swells and shrinks enormously, often inducing cracks in buildings and other civil engineering structures if no proper foundation treatment is done prior to construction.

Expansive soil is a clay soil that changes its volume when the water content of the soil changes. Usually, the soil will shrink when water (or moisture) content is reduced and swell when the water content increases. The degree of expansiveness depends on the content of the active clay mineral called Montmorillonite.

The stress caused by alternate heaving and shrinkage of the foundation soil creates stress on the structures. Usually, structures are not designed to withstand this stress or it is difficult to account quantitatively. As a result, the structures are damaged due to the additional stress. Damages due to expansive soils could occur on any type of buildings, road and other civil engineering constructions that are not properly designed and/or constructed.

However light structures like one or two story buildings, warehouses, retaining walls and buried facilities are more vulnerable to damages because these structures couldn’t exert a heavy pressure to counteract the uplift pressure from the expansive soils. Therefore, in this study more emphasis is given to lightweight buildings founded on expansive soil. This avoids the possibility of confusion whether damages are caused by structural faulty design or settlement.

The damages due to alternate heave and shrinkage or differential heave of the foundation soil are manifested through crack of floors and walls, stacked windows and doors, bulged floors and tilted walls and structures. The magnitude of the damages can be extended even to the extent of failure of one or all structures by decreasing the structural safety of the building. Maintenance and repair cost can also exceed the original cost of the foundation and creates financial burden to the homeowner. Generally, the damage will create economic loss for building owners and the country at large.

1.2 Statement of the problem

Ethiopia is one of the nations where expansive soil is a critical problem. In Bishoftu Town, recent construction areas are extensively covered with expansive soil. Most of the residents of Bishoftu town are low-income people. They usually construct lightweight buildings, which can't counteract the uplift pressure caused by the expansive soil. As a result cracks are bound to occur. Cracks don't only affect the structural safety and beauty of the building but also creates financial burden to the home owners.

In wet seasons, expansive soils will absorb water and swell up; as a result, the whole ground level rises. This increase in ground level is usually called free-field heave. However, if a structure is built on such a soil deposit, the foundations form an obstruction to the soil to freely move up and consequently, the soil applies an upward pressure on the foundation. This pressure that the soil applies on the foundation is called swell-pressure. If the footing transfers a downward stress which is smaller than the swelling pressure, the footing moves upward.

These upward and downward movements of foundations become cyclic seasonal movements during the entire lifespan of the structure. These cyclic movements tend to tear up the walls and eventually destabilize the whole structure. Light structures, such as single or double storied residential buildings, pavements, etc. which generally transmit smaller stresses to the soil than the swell-pressure are those that suffer the most damage. Once the structure develops cracks, it is hardly impossible to rehabilitate it without significant expense. Building a house for most people is a lifetime venture and if it occurs on an expansive soil, the investment needs to be safeguarded. Therefore, there is a need for the public to be aware of the implications of building on expansive soils, in order to identify the problem at an early stage rather than regret later.

1.3 Research question

- What are the properties of expansive soil?
- What are the problems caused by expansive soil?
- How much damage is caused by expansive soils?
- What remedial measures should be taken to reduce the potential impact of expansive soils?

1.4 Objective of the study

1.4.1 General objective

The general objective of this thesis is to investigate damage of lightweight buildings due to expansive soil in and around Bishoftu town and propose remedial measures for cracked buildings.

To achieve the main objective the research will have the following specific objectives.

1.4.2 Specific objectives.

- Identify the major causes for heaving and shrinkage of expansive soil, and evaluate the extent of damage of buildings in the study area.
- Propose remedial measures, with respect to expansive soil, for damaged buildings.
- Introducing low cost 2D crack monitor for damaged buildings.
- Propose preventive measures (methodologies and construction procedures) for new buildings to be constructed in the study area.

1.5 Scope of the study

The study addresses the general objectives and tries to investigate the cause of building failure due to expansive soil and finding the remedial measures. The study includes assessment of forty four randomly selected lightweight buildings founded on expansive soil in/around Bishoftu town. Simple random sampling method is adopted.

1.6 Significance of the study

After successive completion of this research, the output of the work may:

- Develop a better understanding of how expansive soil can cause damage.
- Installing 2D crack monitor is essential for monitoring the safety of buildings.
- Contribute as reference for future work.

CHAPTER 2

LITERATURE REVIEW

2.1 General

Soil is a mixture of various sizes of particles like gravel, sand, silt and clay. Gravel and sand are the coarse fractions and they are considered inert materials because of their insignificant activity. In contrast, clay and silty clays are particles of ultra-fine size in the form of platelets. They carry an unbalanced negative electric charge on their surface. This electric charge and large specific surface they possess render them highly active. They can absorb water as well as the positively-charged ions from the salts in water to neutralize the electric charge they carry on their surface. The amount of water absorbed depends on the type of the clay mineral present in the soil. Three most common minerals present in clay are Kaolionite, Illite and Montmorillonite and their capacity to absorb water increases in that order, therefore, the greater the percentage of montmorillonite mineral present, the greater would be the expansive nature of the soil (Venkataramana, 2003).

The predominant mineral in expansive soil is montmorillonite. Its basic structure is the aluminum octahedral sheet sandwiched between two silica tetrahedral sheets. Experience shows that swelling problem arises when a soil contains more than 20% montmorillonite or mixed layer montmorillonite, illite vermiculite (Venkataramana, 2003).

2.2 Occurrence and origin of expansive soils.

Expansive soils are known to occur in many parts of the world. Some of these countries are: Argentina, Australia, Burma, Canada, Cuba, Ethiopia, Ghana, India, Iran, Israel, Mexico, Morocco, Poland, South Africa, Spain, Turkey, U.S.A., U.S.S.R., Venezuela and Zimbabwe.

The parent materials of expansive soils may be classified into two groups. The first group comprises of basic igneous rocks. Here feldspar and pyroxene minerals of the parent rocks decompose to form montmorillonite, the predominant mineral of expansive soil and other secondary minerals. The second group comprises sedimentary rocks that contain montmorillonite as a constituent which breaks down physically to form expansive soils. In North America, bedrock shale found in the Pierre formation and more recent Larame and Denver Formations are examples of this type of rock. In Israel, there are the marls and lime stones and in South Africa, the shale of the Ecca Series.The expansive soils of Ethiopia are derived from both groups.

2.3 Distribution of expansive soils in Ethiopia

Expansive soil is known to be widely spread in Ethiopia. Although the extent and range of distribution of this problematic soil has not been studied thoroughly: the southern, south-east and south-west part of the city of Addis Ababa areas and central part of Ethiopia following the major trunk roads like Addis-Ambo, Addis-Woliso, Addis-Debre Berhan, Addis-Gohatsion, Addis- Modjo are some of the areas covered by expansive soils. Areas like some parts of Mekele, Gondor, Bahirdar, Debreberihan and Gambela are also known to be partly covered by expansive soils (Zewdie, 2004).

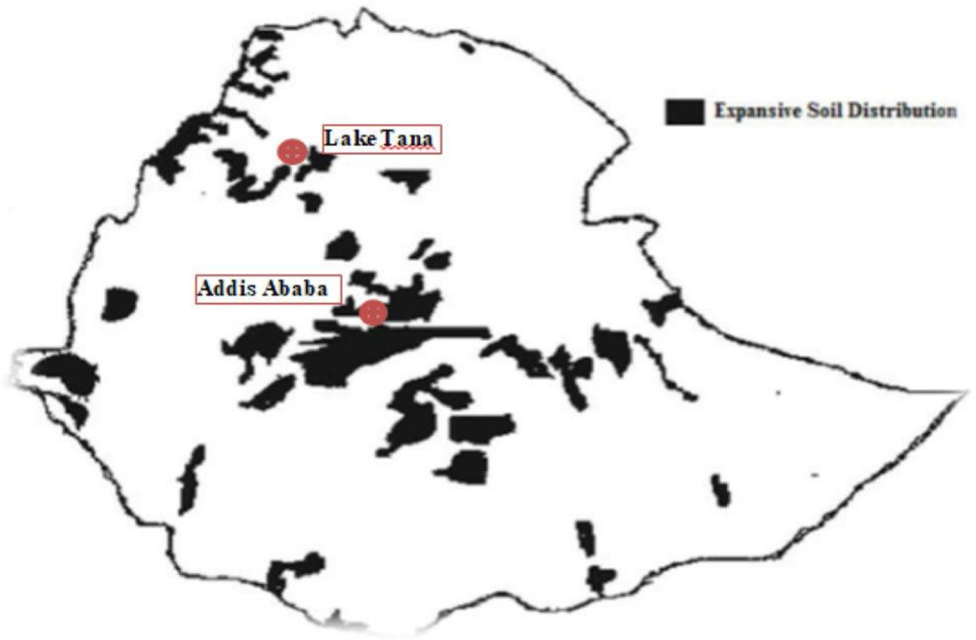


Figure 2. 1: Distribution of expansive soil in Ethiopia (Sisay, 2004).

Figure 2. 2: Views of expansive soil with polygonal pattern of surface cracks in the dry season (Chen, 2018).

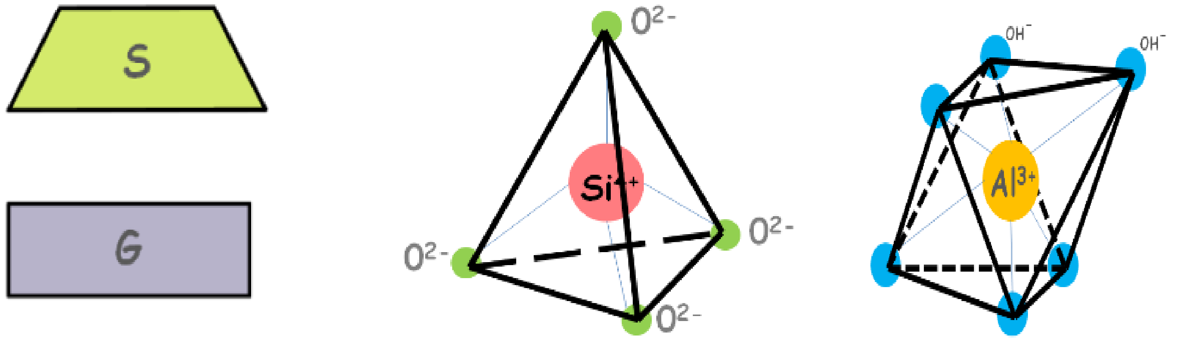
2.4 Clay Mineralogy

Three most common types of clay minerals are Montmorillote, Illite and Kaolinite (Murthy, 2002). All of these clay minerals have two basic atomic sheets:

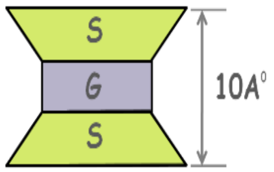
- ✓ Silica tetrahedral sheet
- ✓ Aluminum octahedral sheet

In a **silica tetrahedral sheet**, silica (Si) occupies the center positions and 4 oxygen ions (O) are strongly bonded to the core atoms. Silica tetrahedral sheet is symbolized with a trapezoid, of which the shorter face holds electrically unsatisfied oxygen atoms and the longer face holds electrically satisfied oxygen atoms.

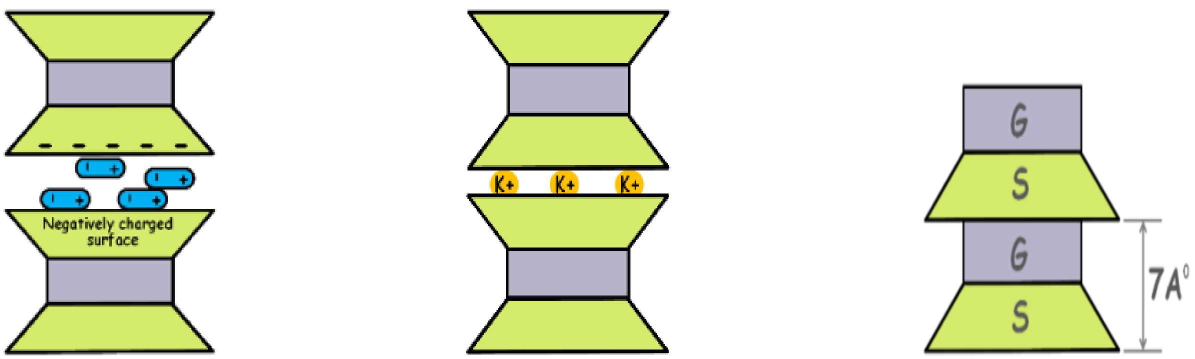
In **alumina octahedral sheet**, alumina (*Al*) ions are positioned at the center and hydroxyl ion (OH^-) bonded to the core atoms. Alumina octahedral sheet is symbolized with a rectangle with top and bottom faces having the same characteristics of exposed hydroxyl ions.



(a) Tetrahedral and Octahedral sheet (b) Tetrahedral unit (c) Octahedral unit



(d) Gibbsite sheet



(e) Mineral montmorillonite (f) Mineral illite (g) Mineral kaolinite

Figure 2. 3: A group of clay mineral (Murthy, 2002)

2.5 Identification of expansive soil

2.5.1 Field identification of expansive soils

Expansive soil deposits can be recognized in the field through visual inspections. The method is simple and easy to use. Some of the important field identification method that indicates the potential for expansiveness of a soil are the following:

- ✓ A shiny surface is easily obtained when a partially dry piece of the soil is polished with a Smooth object such as the top of a fingernail.
- ✓ The wet sample of the soil is sticky and it will be relatively difficult to clean the soil from the hands.
- ✓ The appearance of cracking in nearby structures.
- ✓ They usually have a color of black and gray.
- ✓ In the regions where there is seasonal moisture variation, open closed fissures, (a joint or similar discontinuity), slickenside (highly polished or glossy fissure surface) and shattering or micro-shattering, (presence of fissures forming granular fragments of clayey soils) may observe.

2.5.2 Experimental identification of expansive soils

There are three different methods of identifying potentially expansive soils in the laboratory (Chen,1988).

i. Mineralogical identification

Among the mineralogical identification methods are X-ray diffraction (XRD), differential thermal analysis (DTA), dye absorption, chemical analysis and Scanning electron microscope (SEM) methods. These methods are important in research laboratories in exploring the basic properties of clays. But they are costly and hence not commonly used in soil mechanics laboratories.

ii. Indirect methods

Indirect methods are used to investigate the swelling potential of a soil by examining other parameters, which indirectly yield excellent indices of expansive properties. Such tests are easy and can be performed in an average soil mechanics laboratory. The commonly used test here is the index property tests (consist of Grain size analysis, liquid limit, plastic limit, shrinkage limit, free swell and vertical swell). Also Cation Exchange Capacity (CEC) and Potential Volume Change (PVC) test can be used.

iii. Direct measurement

The most accurate and dependable method of determining the swelling potential and the swelling pressure of expansive clay is by direct measurement. As the name indicates, this type of test directly measures the pressure that a swelling soil exerts on any structure resting on it. It is a convenient and more reliable test because it directly tells the likely in-situ response of the soil for moisture variations. The test can be done by the use of a conventional one-dimensional consolidometer which is available in most soil mechanics laboratories. The method quantitatively evaluates the volume change characteristics of Expansive soil.

2.6 Classification of expansive soils

The different classification systems are categorized into two:-

- ✓ General classification systems: - which have evolved over many years and are based largely on correlation with actual performance
- ✓ Classification specific to Expansive soil:-These systems are based on indirect and direct prediction of swell potential, as well as combinations, to arrive at a rating.

2.6.1 General classification system

i. Unified Soil Classification System

The basis for USCS is liquid limit and plasticity index of a soil. The plasticity chart is a plot of PI and LL (in the ordinate and abscissa respectively) that describes the properties of clay and silt soils in terms of Atterberg limits. This chart consists of two lines namely A-line and U-line. The A-line is assumed to be a boundary between clay and silt soils. Which is defined by an equation $PI = 0.73(LL-20)$. In these classification system a correlation is made between swell potential and unified soil classification as follows below (Daniel, 2003) :-

Table 2. 1:Unified Soil Classification Systems

Category	**Symbol**	**Soil classification in unified system**
Little or no expansion	1	GW,GP,GM,SW,SP,SM
Moderate expansion	2	GW,SC,ML,MH
High volume change	3	CL,OL,CH,OH
No rating		PT

The above classification system can be summarized as follow:

- ✓ All sands and gravels, grouped in symbol number 1, exhibit little or no expansion.
- ✓ All clayey gravels and sands and all silts, grouped in symbol number 2, exhibit moderate volume changes.
- ✓ All clay soil and organic soils, in symbol number 3, exhibit high volume change.

ii. AASHTO classification

The AASHTO system uses similar techniques but the dividing line has an equation of the form PI = LL-30. It generally classifies a soil broadly into granular material and silt-clay material.

Soils classified under groups A-1, A-2 and A-3 are granular materials with 35% or less passing through a No. 200 sieve but A-1 & A-3 non-plastic. Soils with more than 35% passing a No. 200 sieve are classified under groups A-4, A-5, A-6 and A-7. These soils are mostly silt and clay type materials.

Group A-4: -The typical material of this group is a non-plastic or moderately plastic silty soil usually having 75 % or more passing a No. 200 sieve.

Group A-5:-The typical material of this group is similar to that described under Group A-4, except that it may be highly elastic as indicated by the high liquid limit.

Group A-6:-The typical material of this group is a plastic clay soil usually having 75 % or more passing a No. 200 sieve. Materials of this group usually have a high volume change between wet and dry states.

Group A-7:-The typical material of this group is similar to that described under Group A-6, except that it has the high liquid limits characteristic of Group A-5 and may be elastic as well as subject to high-volume change. Subgroup A-7-5 includes those materials with moderate plasticity indexes in relation to the liquid limit and which may be highly elastic as well as subject to considerable volume change. Subgroup A-7-6 includes those materials with high plasticity indexes in relation to liquid limit and which are subject to extremely high-volume change.

2.6.2 Classification specific to expansive soil

The above classification system may give an initial alert that the soil may have expansive character but does not provide useful information. A parameter determined from the expansive soil identification tests have been combined in a number of different classification schemes to give qualitative rating on the expansiveness of the soil. But the direct use of such classification systems as a basis for design may lead to an overly conservative construction in some places and inadequate construction in some areas (Nelson and Miller, 1991).

i. Classification based on indirect predictions of swell potentials

An indirect prediction of swell potential includes correlations based on index properties, swell, physical indicator and a combination of them. Some of classification systems are:-

Skempton's method (Mckeen, 1976)

Skempton classifies clays according to their activities. Activity is a measure of the water holding capacity of clay soil. Following his classification, three degree of colloidal activity has been established as indicated in Table 2.2.

$$A_c = \frac{PI}{\%\ \text{Clay}} \tag{2.1}$$

Table 2. 2: Degree of colloidal activity (Skempton's method)

Activity (A_c)	**Degree of Activity**
< 0.75	Inactive clay
0.75 – 1.25	Normal clay
> 1.25	Active clay

Altmeyer (1956)

Altmeyer suggested rating for degree of expansion based on volumetric Shrinkage Limit (SL) and Linear Shrinkage (Ls) as shown in Table 2.3.

Table 2. 3: Classification of expansive soil (Altmeyer, 1956)

Volumetric Shrinkage, %	**Linear Shrinkage,%**	**Degree of Expansion**
< 10	8 >	Critical
10 - 12	5 - 8	Marginal
> 12	< 5	Non Critical

Seed et al, (1962)

According to Seed et al, Plasticity Index is a parameter which can be used as a preliminary indicator of the swelling characteristics of a soil, as shown in Table 2.4.

Table 2. 4: Classification of expansive soil (Seed et al, 1962)

Plasticity Index	**Swell Potential**
0 – 10	Low
10 - 35	Medium
20 - 55	High
55 and above	Very High

Chen (1988) Method

In this method, a correlation is made between swell data and percent less than No. 200 sieve, liquid limit, and standard penetration resistance. The classification is as given in Table 2.5.

Table 2. 5: Classification of expansive soil (Chen, 1988) method.

< No.200 sieve, %	LL, %	Standard penetration Blows/ft	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

Rao (1991)

Besides using the index properties, the swell potential of clay soils can be indirectly estimated from the differential free swell (DFS) test (IS 2720 Part 40–1977 Determination of Free Swell Index of Soils). In this method, two oven-dried samples weighing 10g each and passing through 425um sieve are taken. One sample is put in a 100ml graduated glass cylinder containing kerosene (nonpolar liquid) and the other sample is put in a similar cylinder containing distilled water. Both the samples are stirred and left undisturbed for 24 hours and then their volumes are noted. The DFS is expressed as:

$$DFS = \frac{(V_w - V_k)}{V_k} 100\% \qquad (2.2)$$

Table 2. 6: Degree of expansiveness and free swell (Rao, 1991)

Degree of expansiveness	DFS (%)
Low	< 20
Moderate	20 – 35
High	35 – 50
Very High	>50

Anderson (1969)

The swelling potential of the samples are calculated according to the empirical formula of Anderson,

$$S_p = 0.23\ PI - 3.12 \qquad (2.3)$$

Anderson has correlated the swelling potential with degree of expansion given in Table 2.7.

Table 2.7: R/ship between swelling potential and degree of expansion (Anderson, 1969)

Degree of expansion	**Plasticity index (*PI*)**	**Swelling potential (S_p)**
Low	< 20	1.5
Medium	20 - 31	1.5 – 4.0
High	31 - 39	4.0 – 6.0
Very High	> 39	> 6.0

2.7 Soil moisture relationship of expansive soils

For swelling to take place, the environment of the expansive soil has to be changed. Environmental change can consist of pressure release due to excavation, desiccation caused by temperature increase, and volume increase because of the introduction of moisture. It is the effect of water on expansive soil that plays the major role.

2.7.1 Effect of moisture content

Moisture migration depends on the geological formation, climatic condition, topographic features, soil type and ground water level. The most common method of moisture transfer is by gravity. The seepage of surface water, precipitation and snow melting into the soil are common examples. Moisture migration can occur in all directions. In stiff clays and in shale bedrock, the flow occurs in the bedding plane or follows continuous fractures and fissures. In fine grained soils, a capillary force is a significant means of water transfer. Thermal gradients are also a cause for moisture migration through the liquid phase of the soils. Vapor and liquid moisture transfer under thermal gradient can be an important cause of the swelling of moisture deficient soils.

Moisture in the soil varies due to seasonal climatic changes, change of surface conditions and external influences. Climatic change consists of a change in atmospheric temperature, precipitation, evaporation, transpiration and relative humidity. Surface condition (changing of paved area into garden) and External influence which is resettlement of new houses around the existing building will disturb the moisture content of the foundation soil resulting in variation of moisture content which in turn results in the swelling and shrinkage of the soil. This alternate shrinkage and swelling of the foundation soil creates stress on the structures as a result, the structures be faced to additional stress which is not yet considered during the design (Sisay, 2004).

2.7.2 Factor affecting volume change

Swelling pressure is the pressure, which prevents the specimens from swelling. i.e., It is the pressure required to return the specimens back to its original state after swelling (Void ratio, height). Swell is influenced by initial moisture content, initial dry density, surcharge pressure, time allowed for swell, size and stratum thickness and degree of saturation

2.7.3 Effect of depth of moisture fluctuation

The thickness of the expansive soil layer below the ground surface and the depth to which the season moisture varies, called the ‘Active depth’, greatly influence the amount of heave the ground would undergo. Figure 2.3 shows the seasonal moisture variation and the active depth of homogeneous soil. Below the active depth, the soil water content is considered to remain constant; therefore, foundations situated within the active depth are subject to distress from soil volume change (Venkataramana, 2003)

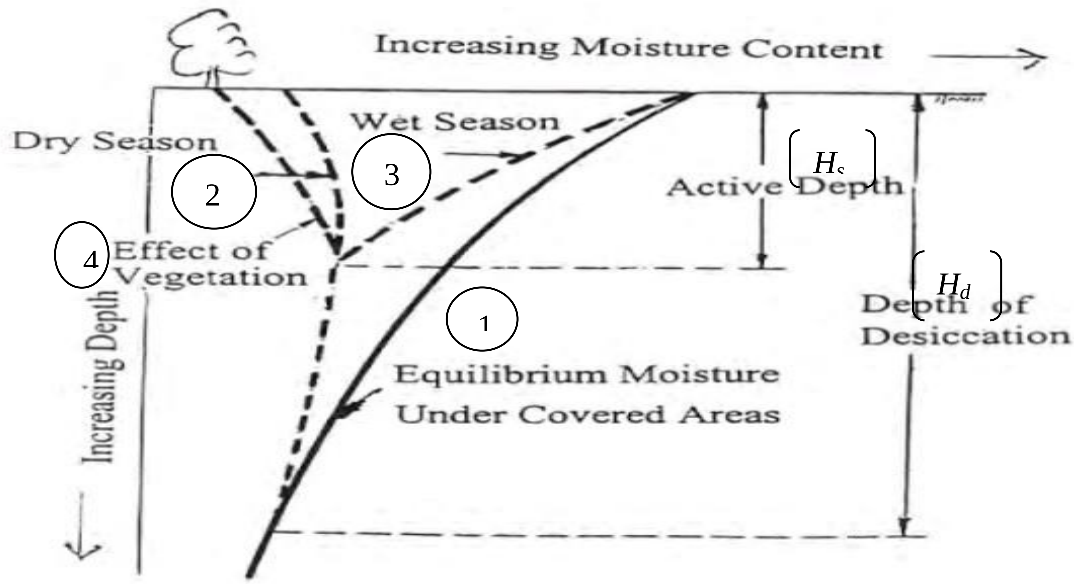


Figure 2. 3: Seasonal moisture variations (Venkataramana, 2003)

In a covered area, the moisture profile is shown by curve 1. There is no gain or loss of moisture to the atmosphere. The moisture content of the soil deceases with depth. Curve 2 indicates the moisture content variation in uncovered natural condition. Evaporation causes loss of moisture in the soil near the ground surface. However, the influence of evaporation decreases with depth and at some depth, H_d, the moisture content equilibrium remains the same as the covered condition. The value of H_d depends on the climate condition, the type of soil and the location of the water table. This depth represents the total thickness of material which has a potential to expand because of water deficiency. It is impossible to determine the value of H_d. the hotter and drier the climate, the greater the depth of desiccation. The maximum depth of H_d is equal to the depth of the water table and the minimum depth is equal to the seasonal moisture fluctuation depth described below.

During wet months with heavier precipitation and higher humidity, the moisture content of near surface soil increases and the moisture profile represented by curve 2 alters its shape to curve 3. Another case is planting of trees and shrubs which further reduce the moisture content of the soil and this condition is designated by curve 4.

The depth of seasonal moisture content fluctuation, H_s, indicated in Figure 2.3 depends on the variation of surface moisture, permeability of the soil, and climatic conditions. In areas where precipitation and evaporation are fairly constant, the H_s depth may be only a few meters. When a long drought is followed by an intense rainfall, the H_s depth will be very deep. It should be noted that man-made environments such as watering of lawns, discharge of roof drains, formation of drainage channels and swales, and possibility of utility line leakage will also increase the value of H_s (Chen, 1975).

2.8 Effects of foundation movement

The magnitude and intensity of structural damage is influenced by the intensity of contact pressure, the type of foundation and the relative stiffness of the super structure. In lightweight structures, the contract pressure is normally much smaller than the swelling pressure of the expansive soil. As a result, the whole building will be lifted differentially and creates stresses, which are not accounted for in the design. This stress creates cracks.

2.8.1 Cause of cracks on buildings constructed in expansive soil areas

Cracks caused by swelling soil are wide at the top and narrow at the bottom. The cause of foundation movement is moisture fluctuation of the foundation soil. Normally, the first sign of foundation movement for a structure founded on expansive soil is the cracking of the floor slab, door binding, window stacking and cracks appearing in the exterior and in the interior walls and even in the ceiling. It is important to differentiate the different types of cracks and their causes in order to know cracks caused by expansive soil heave.

It is difficult or costs a lot to make an absolute crack free building. The types and their causes are different. Some of the crack types and their causes are as follows (Sisay, 2004).

i. Crack due to shrinkage and expansion of the plaster work

Such cracks are hairline cracks, insignificant tilt of floors or change in levels and can be easily maintained. It mainly affects the aesthetic of the building .This type of cracks usually occurs and it is due to the stress induced by shrinkage or expansion of the mortar during drying.

ii. Crack due to structural failure

These cracks are significant cracks and caused due to improper design and/or quality control failure. Besides functions and cost, such cracks have psychological impact on the owners and can be encountered in high-rise buildings and in non-expansive soil areas. Such cracks occur very rarely.

iii. Crack due to foundation movement

These types of cracks are usually associated with expansive soil, which can exert a pressure which moves the structure. It is commonly observed in light weight buildings found on expansive soil areas. Such cracks are abundant.

2.8.2 Types of foundation distresses

Cracks are more readily noticed on buildings constructed in expansive soil areas .It is a rare case that cracks are caused on light weight buildings due to settlement. Settlement and shrinkage or heave crack can be distinguished by observing the width of the crack as the climate changes from season to season. If the crack width changes from season to season, the crack is the result of shrinkage and heave not settlement.

When a building is constructed on an expansive ground surface evaporation and temperature variations are retarded below it. As a result, patterns of soil movement under the building are identified on the basis of short-term and long-term effects. This soil movement will create the following 3 principal foundation distresses (Venkataramana, 2003; Sisay, 2004)

i. Center heave condition

In the long-term, moisture starts continuously accumulating under the center of the building being drawn up from the ground water at depth by capillary action until an equilibrium moisture condition is attained. This means that the soil under the building continuously swells with time and finally attains a mound shape while at the edges; the seasonal effects continue to occur. This is termed the central-heave condition. As a consequence of this, the foundation gets only a partial support from the soil at the center as shown in Figure 2.4. The height of the mound formation depends on the area of the soil covered by the building and also the active depth. Crack of wall start in the upper region and moves downward. Center lift distortion which is typically caused by heaving beneath the interior of the building, shrinking around of the perimeter of the building (Venkataramana, 2003)

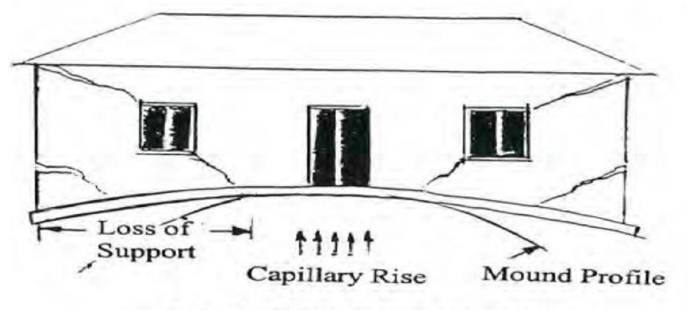


Figure 2. 4: Center heave condition (Venkataramana, 2003)

ii. Edge heave condition

In the short-term, i.e., just after the construction is complete, below the center of the building, moisture variation remains small while at the edges, seasonal moisture change continues to occur. If the building is constructed during the dry season, in the wet season that follows, soil around the building absorbs rainwater and swells pushing up the peripheral foundations. This effect is called the edge heave (Venkataramana, 2003; Solomon, 2015). Edge heave can also be initiated due to local effects, such as, sprinkling of water for vegetation around building, leaking utility lines and pounding of surface water due to poor drainage system.

In this case, the cracks develop from the lower region and transverse diagonally upward being wider at the bottom and narrower at top as illustrated in Figure 2.5.

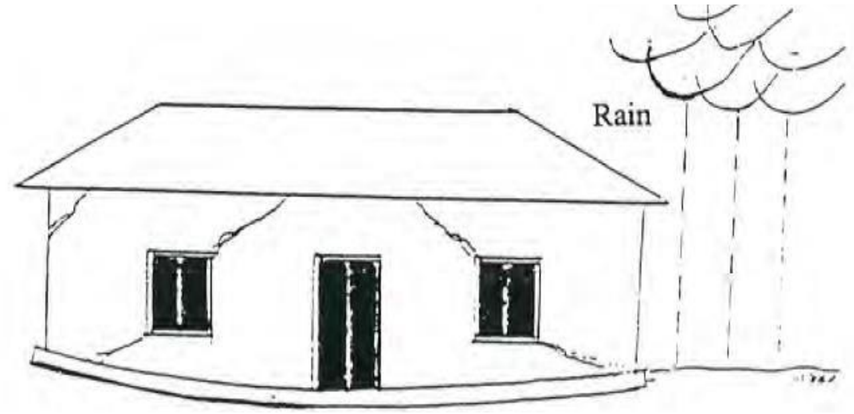


Figure 2. 5: Edge heave condition (Venkataramana, 2003)

iii. Edge Shrinkage

Like edge heave, edge shrinkage is also a short-term effect. If the building is constructed during the wet season, in the dry season that follows, soil around the building loses the absorbed rain water and the soil leaves the peripheral foundations. This effect is called the edge shrinkage. Presence of big trees close to the building will also cause edge shrinkage. Especially during the dry season when moisture available for roots to suck is the least, trees absorb water from the nearby foundation soil through their root system and cause shrinkage of soil.

In the edge shrinkage condition as shown in Figure 2.6 below, cracks propagate diagonally across from top to bottom being wider at the top and narrower towards the bottom of the structure (Venkataramana, 2003; Sisay, 2004)

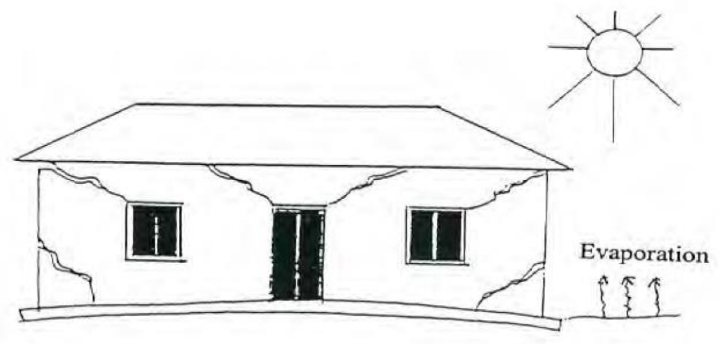


Figure 2. 6: Edge Shrink condition (Venkataramana, 2003)

2.9 Identifications of damages due to expansive soil

Different Buildings experience various levels of damages during their lifetime. Damage may occur within a few months following construction, may develop slowly over a period of about 5 years, or may not appear for many years until some activity occurs to disturb the soil moisture. (Jones and Holtz,1973).

The probability of damage increases for structures on swelling foundation soils if the climate and other field environment, effects of construction, and effects of occupancy tend to promote moisture changes in the soil. The differential movement caused by swell or shrinkage of expansive soils can increase the probability of damage to the foundation and superstructure. Differential rather than total movements of the foundation soils are generally responsible for the major structural damage. Differential movements redistribute the structural loads causing concentration of loads on portions of the foundation and large changes in moments and shear forces in the structure not previously accounted for in standard design practice.

The most obvious identifications of damage to buildings are doors & windows that get jammed, uneven floors and cracked foundations, floors, masonry walls, ceilings & pavement around buildings**.** The degree of damage based on observed cracks ranges from hairline cracks, severe cracks, very severe cracks to total collapse. The pattern of the cracks depends on whether it is a dooming heave or a dish shaped lift heave. In both cases vertical, horizontal and/or diagonal cracks will be developed along walls and floor area. This intern has a great effect on the functionality of doors and windows.

The classification of the damage is very important to assess whether the building calls for strengthening, repair, renovation or demolition. Various researchers put forward many definitions, specifications, classification and effect of damage in structures as given in Table 2.8.

Table 2. 8: Classification and effect of damages in buildings (Zumrawi et al, 2017)

Degree of damage	**Description of damage**	**Effect of damage on building**	**Crack width (mm)**
Insignificant	Hairline cracks	None	< 0.1
Very slight	Fine cracks	None	0.1 to 0.3
Slight	Cracks are visible and easily filled. Several slight fractures may appear inside of the building. Doors and windows may stick	Aesthetic only	0.3 to 2
Moderate	Cracks that require opening up and patching. Possible replacement of a small amount of brickwork. Doors and windows stick, service pipes may fracture.	May affect serviceability and stability of the building	2 to 5
Severe	Large cracks require extensive repair work involving breaking-out and replacing sections of walls. Windows and door frames distort and floor slopes are noticeable. Leaning or bulging walls. Beams lose some bearing. Utility service disrupted	Serviceability and stability of the building are at risk.	5 to 25
Very severe	Major repair involving partial or complete rebuilding. Beams lose bearing, walls lean badly and require shoring and windows are broken with distortion.	There is a danger of structural instability	>25

2.10 Possible causes of damage

2.10.1 Climate changes

Seasonal changes in rainfall were typically the principal cause of the change of soil moisture. This led to downward movement during summer and upward movement during winter. The consequent rising and settling of ground surface occurred in the dry and wet seasons resulting in seasonal subsidence and seasonal recovery respectively. The results and observations by most of the researchers suggested that expansive soils which experienced periodic swelling and shrinkage during alternate wet and dry seasons caused considerable damage to structures found on them. The damage to structures built on expansive soil in wet climates usually occurred during drought periods and damage to structures built in dry climates occurred during the rainy season (Osman et al, 2007).

2.10.2 Poor drainage system

Improper drainage is probably the most important factor contributing to soil volume change and subsequent damage to buildings. If water is allowed to stand in drainage ditches close to buildings, it can penetrate down and amplify heave (Nelson and Miller, 1991).

The following can be considered as the main causes of poor surface drainage:

- ✓ Surface runoff not properly drained away from the building
- ✓ Sprinkling of water for grass and shrub plantation
- ✓ Overflow from elevated and/or ground water tank
- ✓ Slope of surrounding area (outside the compound)

2.10.3 Presence of vegetation and big trees

Existence of vegetation, such as fast growing trees in the vicinity of compound walls can sometimes cause cracks in walls due to expansive action of roots growing under the foundation. Roots of some trees generally spread horizontally on all sides in the effective zone of the foundation soil when trees are located close to a building (Venkataramana, 2003).

Trees absorb water from the nearby foundation soil through their root system and cause shrinkage of soil especially during the dry season when moisture available for roots to suck is the least. This is the reason why big trees should not be located within a distance of 0.5 to 1 times their mature height from the structure, if big trees are felled just before construction, transpiration loss of moisture through leaves is discontinued and the soil moisture accumulates and allows the soil to swell. Therefore, trees have to be cut down far in advance of construction so that soil moisture condition reaches an equilibrium condition. If trees are retained, moisture barriers should be put in place as shown in Figure 2.7 (Venkataramana, 2003).

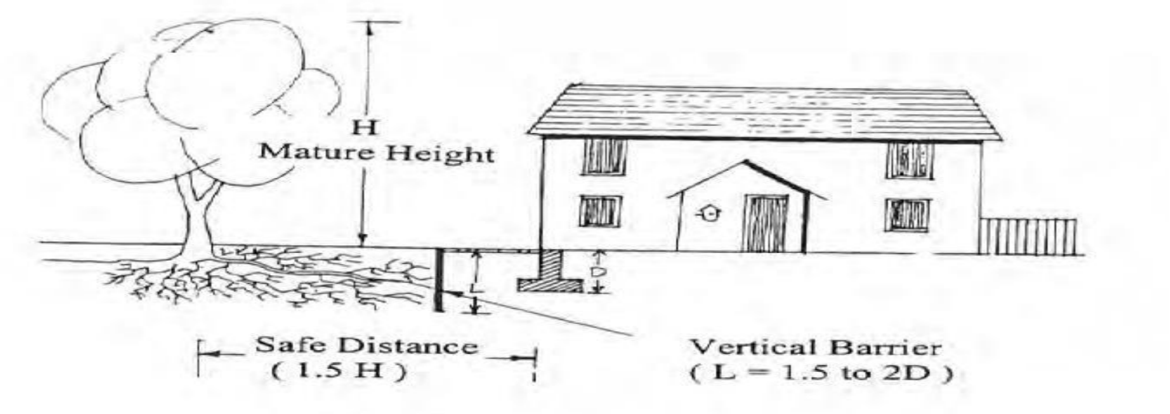


Figure 2. 7: Vertical moisture barriers (Venkataramana, 2003).

As shown in Figure 2.8 damage caused by a tree is similar to the edge-shirk condition because the soil at the periphery of the building is depleted of water by the tree roots.

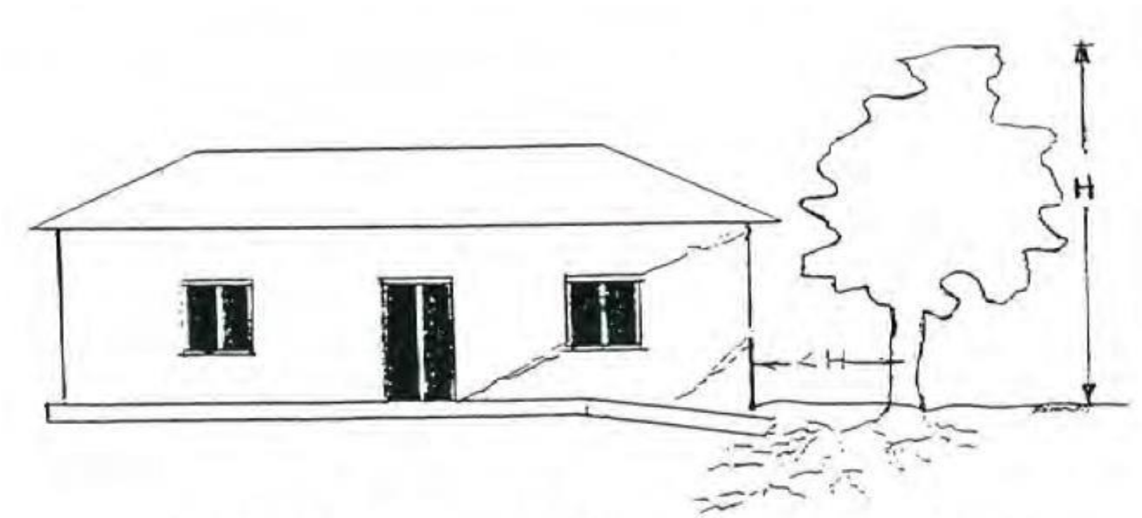


Figure 2. 8: Down warping of foundation due to trees close by (Venkataramana, 2003).

2.10.4 Damaged utility structures

Shallow plumbing pipes buried in the zone of seasonal moisture fluctuation, are exposed to enormous stresses by shrinking soils. If water or sewage pipes break, then the resultant leaking moisture can aggravate swelling damage to nearby structures. The effect of a leaking water line is dependent on the soil moisture condition in the supporting expansive soil mass prior to the leaking occurrence. Dishing of floor systems due to clay heave under the footings could occur when excessive water is present due to site leakage at the edges of the structure (Osman et al, 2007).

2.10.5 New construction near existing building

When areas are covered by new structures such as buildings, pavements, sidewalks or aprons, evaporation is blocked or is partially retarded. The moisture content beneath the covered area increases due to gravitational migration, capillary action, vapor and liquid thermal transfer and, in the course of several years, the depth of seasonal moisture content fluctuation H_s can approach to depth of desiccation H_d (Tessema, 1984). In the same manner if structures around the building have been demolished due to various reasons then the covered area around the building will be reduced. This leads to migration of moisture away from building foundation soil until new soil moisture equilibrium has been reached. Such conditions cause foundation distress depending on type of foundation, moisture content and swelling potential of the soil.

2.10.6 Creep action foundation on steep slope surface

Expansive soils on slopes tend to move down the slope due to soil creep, especially when the soil is wet. Soil creep is a very slow movement down the slope, which is not perceptible.

However, as shown in Figure 2.9, overtime it can be significant and can damage the foundations. The steeper the slope, the greater this effect is. Furthermore, on slopes, these soils can cause landslide problems. Slopes steeper than about 14^o (a residual friction angle of soil) have the tendency to slide, if conditions are favorable (Venkataramana, 2003).

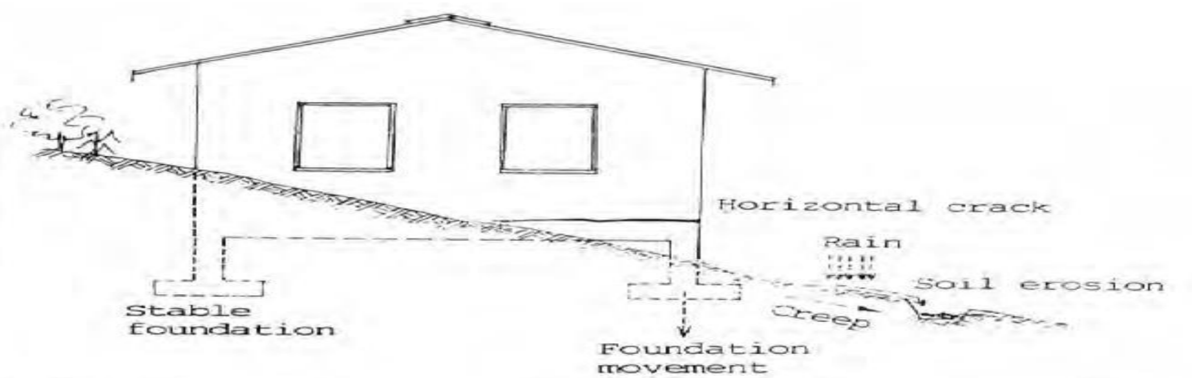


Figure 2. 9: Structural damages due to soil creep (Venkataramana, 2003).

2.10.7 Construction interruption

Due to financial problems of homeowners it is a common trend to construct foundation or substructure parts in one phase and superstructure in the next phase. Buildings are designed to the final stage or condition. Because of construction interruption, the dead load pressure which counterbalances the uplift force gets lower resulting in foundation movement. This results in a failure even if the foundation is designed properly with the precautions to be taken for expansive soil (Sisay, 2004).

2.11 Remedial measures

The remedial measures to be taken for a cracked building are simpler once the cause of foundation movement has been determined. Remedial works to be taken differ in each case. The commonly used remedial measures are as follows.

2.11.1 Minimizing moisture fluctuation

To keep the moisture content constant by keeping the soil in dry state or saturated state is not practical due to environmental and practical reasons. The problem can, therefore, be best tackled by minimizing the moisture gradient between the edges and the center of the covered area (Sharma, 1987).

To achieve this goal the following measures can be implemented;

- Cut trees and bushes located at a distance ranging from 1.0 to 1.5 times the height of tree,
- Provide positive drainage around the building,
- Maintain damaged gutters, down pipes, sewerage pipes and water supply pipes that are creating local wetting,
- Extend covered area around the building by providing an apron. An apron at the top of ground is not advisable because it is liable to damage by external forces or differential movement of the ground over which it rests. Therefore providing a waterproof apron of about 2 m width at a depth of about 50 cm is more appropriate (Sharma, 1987).
- Install horizontal barriers, vertical barriers, intercepting drains, peripheral drains or combination of them depending on source of water and surface drainage conditions (Tefera, 2008).

2.11.2 Increasing stiffness of the structure and/or resistance against uplift

In addition to minimizing moisture fluctuation, increasing the stiffness of the building and/or resistance against uplift pressure would also be a good solution for buildings damaged by cyclic heave and shrinkage of expansive soils. Such remedial measures vary depending on the type of foundation system and extent of damages. Most commonly used structural remedial measures are presented below with respect to foundation systems (Sisay, 2004).

For drilled pier foundation:

- Loosen soil around the pier to reduce the uplift pressure
- Reconstruct void space beneath the grade beams
- Eliminate the mushroom at the top of the pier
- Cut the top of the pier and adjust the pier by shims

- Remove all the backfill around the building and replace with compacted non expansive clay to protect surface water entering through the foundation soil
- Improve the drainage condition around the building by providing adequate slope away from the building and paving with concrete

For continuous footing foundation

- Provide voids beneath the footings at calculated interval to increase the dead load pressure
- Post-tension a foundation to provide structural stability by preventing unequal movement
- Under pile the structure with piers drilled into bedrock
- Reinforce existing foundation walls with new reinforced grade beam to tie the structure as in box construction

For individual pad foundation:

- Decrease pad size to increase the dead load pressure
- Underpin the pad with piers drilled into bedrock

For basement

- Saw cut the slab along the foundation wall to allow free slab movement
- Adjust screw jack on top of pipe column to re-level interior I-beam
- Provide slip joints to all internal slab bearing partition walls, including door frames and staircase walls.

2.11.3 Foundation practices on expansive soils

The following conventional foundation practices and innovative techniques can provide solutions to problematic soils (Chen, 2018).

i. Sub Excavating or replacing the expansive soil by cushions

In this technique the expansive soil is replaced either in part or full (Figure.2.10) with a material that does not undergo swell. The load of the cushion provides the load necessary to counter heave (Tefera, 2008; Chen, 2018).

Figure 2. 10: Sub excavating or replacing the expansive soil by cushions (Chen, 2018).

ii. Sand cushion method

The entire depth of the expansive soil stratum or a part thereof may be removed and replaced with sand cushion, compacted to the desired density and thickness. Swelling pressure varies inversely as the thickness of the sand layer and directly as its density. Therefore, generally sand cushions are formed in their loosest possible state without, however, violating the bearing capacity criterion. The basic advantage of the sand cushion method is its ability to adapt itself to volume changes in the soil. However, the sand cushion method has several limitations particularly when it is adopted in deep strata (Chen, 2018).

iii. Cohesive non swelling soils (CNS) layer method

Replacement by soils with relatively impervious material may, to a great extent, offset the disadvantages of the sand cushion method. Katti (1978) developed a technique whereby removal of about 1m of expansive soil and replacement by CNS layer beneath foundations has yielded satisfactory results. He successfully adopted it for prevention of heave and resultant cracking of canal beds and linings and recommends it for use in foundations of residential buildings also. The approximate value of CNS thickness to be used for soils in different swelling pressures is shown in Table 2.9.

Table 2. 9: Thickness of CNS layer for lining over expansive soil (Kartikey *et al*, 2012).

Swell pressure of soils (*kN/m*^{2})	**Thickness of *CNS* material (*cm*)**
50 – 150	75 – 85
200 – 300	90 – 100
350-500	105-125

iv. Providing a granular bed and a granular cover below and found foundation.

Figure 2.11 shows a strip footing under an outside wall constructed on an expansive soil. The excavation of the expansion soil is carried out for a width greater than the width of the footing proper. Generally, an extra width of 15*cm* is excavated on either side. A freely-draining granular soil, such as gravel, sand or a mixture of gravel or sand is placed in the trench up to the base level of the footing and compacted.

The foundation on expansive soil is usually constructed during hot season when the soil has shrunk to its minimum level. A cushion of freely draining soil below the foundation acts as a barrier and the effect of swelling on the foundation is reduced. A reinforced concrete apron of about 2m wide is provided around the outer walls of the building to prevent the entry of water into the foundation. Suitable arrangements should be made to drain the water away from the granular base during the raining season. The reinforced concrete beam should have an air gap over the ground surface to prevent effects of the soil. Arora (2009) suggests that instead of a reinforced concrete apron, a flexible waterproof apron made of bituminous concrete about 2*m* wide and 75*mm* thick can be constructed around the building. He stressed that the flexible apron should not crack when it is pushed up by the swelling soil. If it cracks or if their connection with the main structure is damaged, it should be properly sealed before the next rainy season. The flexible apron is covered with soil and given an outward slope of 1 in 30.

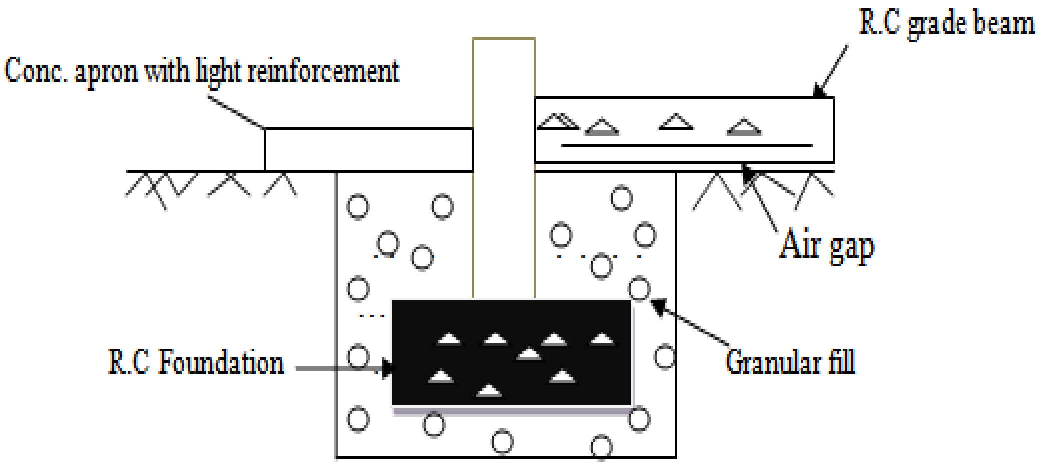


Figure 2. 11: Foundation on granular cover below and around the foundation (Chen, 2018).

2.11.4 Inserting Helical bar on the wall for cracked buildings

Helical bar: Is a stainless steel which is used in general for crack stitching and reinforcing masonry in both new and existing masonry structures. It is a remedial measure in order to reduce and regulate crack movement on buildings.

The 2D crack monitor is used only for 2D crack monitoring, across the crack (y-axis) and along the crack (x-axis). Depending on the installed instrument results if the wall of the building shows any additional crack opening or if the degree of damage shifts from moderate to severe, by inserting a 1m length helical bar on either side of the crack 50cm length left and right is recommended to reduce the crack movement. If the 2D crack monitor shows no crack difference from the initial or previous reading data then we can fix the wall by cement mortar.

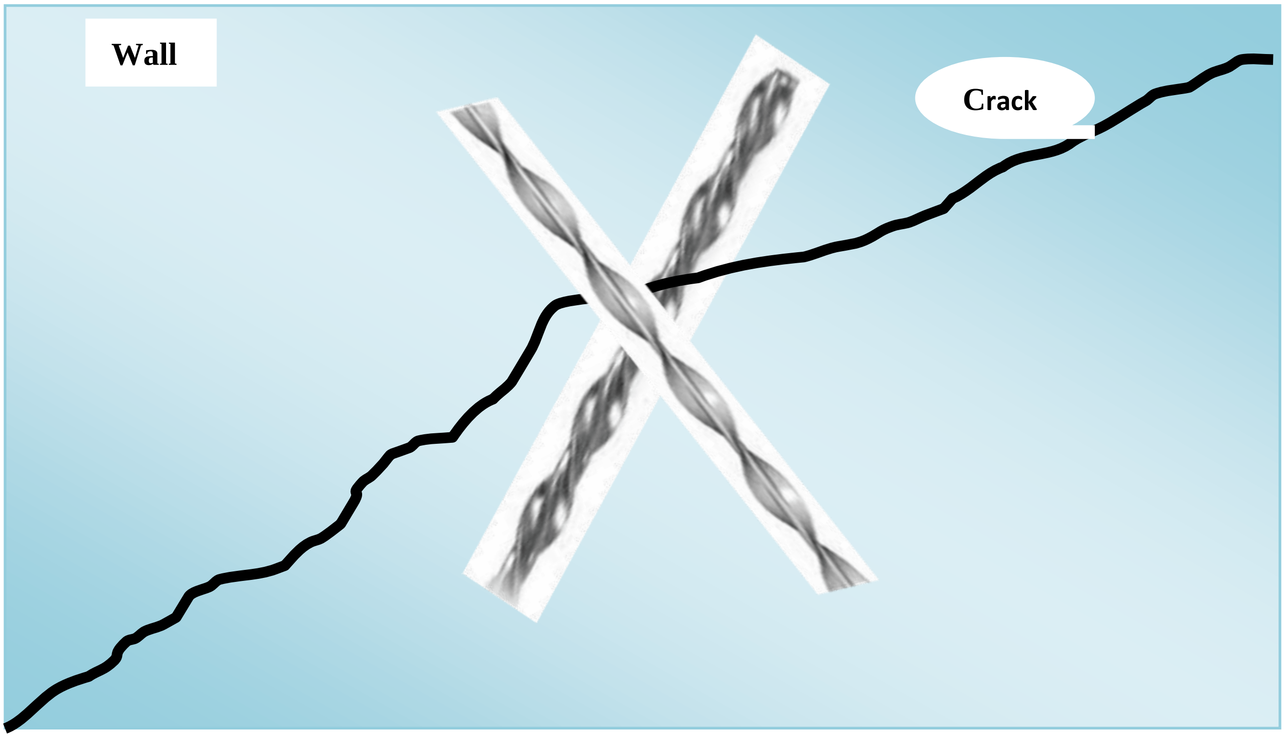


Figure 2. 12: Helical bar inserted along and across the crack for cracked buildings

2.12 Measurement of crack movement

2.12.1 Measurement of crack movement of barrage using 2D & 3Dcrack monitors

Barrages are basically meant for diversion of river water through canals for irrigation purposes. There are numerous barrages in India for various purposes. Some old barrages in India are in poor condition and have developed cracks and need regular monitoring for safety of structure. A comprehensive program of instrumentation and observation can play a major role in evaluating the stability of a structure. Proper guidelines for instrumentation of barrage are basic requirements for monitoring its performance and safety. *BIS* has framed guidelines for instrumentation of barrages and weirs in its code *IS* 14248:1995. These *BIS* standards recommend monitoring of barrage for uplift pressure, stress and strain, water levels, tilt and displacement. Barrage/ headwork has developed several cracks on dividing walls and piers on upstream and downstream sides. Central Soil and Materials Research Station had installed 27 instruments for crack movement monitoring at different locations on the upstream and downstream side of the barrage during 2004-05. Since then, data analysis has been done by Central Soil and Materials Research Station.

i. Instrumentation

There are multiple superficial cracks on dividing walls and piers on upstream and downstream sides of the barrage. Total 27 instruments (2D crack monitors: 15 nos.; 3D crack monitors: 12 nos.) have been installed for crack movement monitoring at different locations on upstream and downstream sides of barrage during 2004-05 (Sehra and Gupta, 2018).

ii. 2-Dimensional crack monitor

This instrument is used for monitoring of the relative movement of cracks in 2 directions along the crack (x-axis) and across the crack (y-axis).

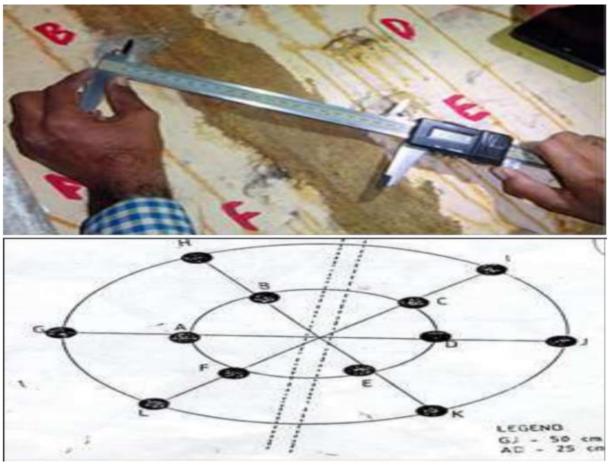


Figure 2. 13*:* 2D crack monitor with axis orientation for monitoring of crack movement

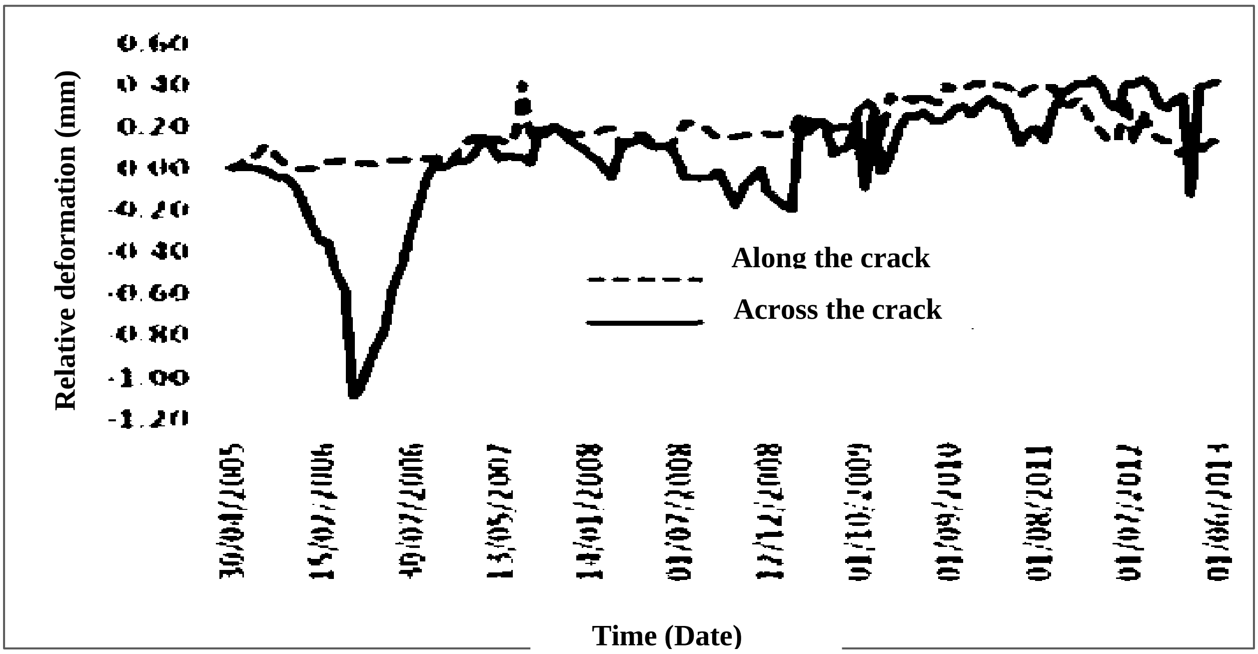


Figure 2. 14: 2D crack movement 2D/14, DP7, S1 (Sehra and Gupta, 2018)

In Y direction i.e. across the crack, movement of crack was ranging 5.18 mm (at 2D/15) to -0.41 mm (at 2D/14) during the monitoring period from April 2005 to June 2013.

2.12.2 Tewatia's 3-Pin Algorithm

Insert three pins A, B, and C on both sides of the crack as shown in Figure 2.20 below, so that AB is perpendicular to the direction of crack and BC is inclined at any known angle, α. Measure the original lengths of AB and BC. Suppose after time, t_1.Amoves to A_1 and C moves to C_1 so that BA_1 and BC_1 are the new lengths of BA and BC, respectively.

The basic assumption involved in the method is that the walls move as rigid bodies without any rotation so that AA_1 is equal and parallel to CC_1, and therefore the components of these along and across the crack, x and y say, are equal. The approximate value of $y(=BA_1-BA)$ is assumed. Now in triangle BCC_2: BC and $CC_2(=y)$ are known in magnitude and direction; therefore, the magnitude and direction of BC_2 can be known. In triangle BC_1C_2: as directions of C_1C_2 (which is parallel to crack) and BC_2 are known; therefore, the angle between them is known. As in this triangle magnitudes of two sides BC_1 and BC_2 and one angle are known; therefore, the third side $C_1C_2(=x)$ can be determined.

Using this x in the right-angled triangle BA_1A_2, BA_2 is determined as $(BA1^{2-}x^2)^{1/2}$. The new value of y is determined as $(BA_2\text{-}BA)$. As compared to the previous y, this y is closer to the true value. Using this y from triangles BCC_2 and BC_1C_2, the new value of x is determined as earlier. As compared to the previous x, this x is closer to the true value. Using this new x, the new y is determined from triangle BA_1A_2 as earlier, which is closer to the true value as compared to previous y. Repeating this procedure, after 5–6 iterations, the values of x and y converge to their true values (Tewatia, 1997).

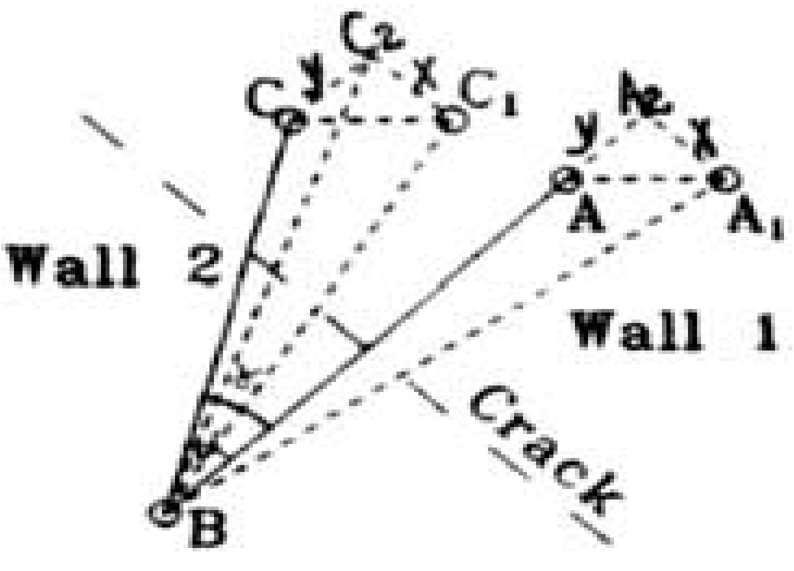


Figure 2. 15: The 3-pin method for 2D crack monitoring (Tewatia, 1997).

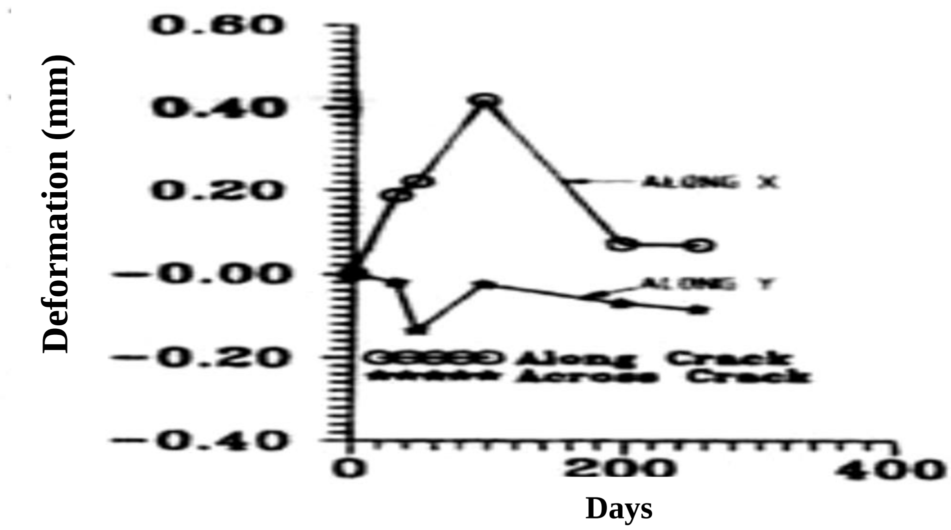


Figure 2. 16: A typical plot of crack monitoring at Sardar Sarovar Power House Cavern, India, using 3-pin method (Tewatia, 1997).

2.13 Review of previous studies about damage caused by expansive soil

Sisay (2004) Studied 96 randomly selected lightweight buildings founded on expansive soils of Addis Ababa were examined and detailed study of one selected building was also conducted. Out of the observed 96 buildings, 69 or (72%)are affected by the consequence of heave or shrinkage of the expansive soil. The problems observed on buildings vary from simple crack on walls to severe structural damages, which cause collapse of the whole building.

Solomon (2015) concluded that out of 78 randomly visited buildings constructed on expansive soil in/around Bahir Dar areas 68 or (87%) of them are damaged due to swelling and shrinkage cycle of the soil. In most cases more than one building component is affected.

CHAPTER 3

MATERIALS AND METHODS

3.1 General

Bishoftu, also known as Debre Zeit, is a town found in Oromia regional state, Ethiopia. It is situated a few dozens of miles (29.8 mil or 47.9 km) southeast of Addis Ababa. The city has been developing greatly since the second half of the 20^{th} century, and its population number has been increasing greatly since the 1990s. The area is famous for its lakes, including Lake Bishoftu, lake Hora, lake Kuriftu, lake Babogaya, and others. The site where primary data samples were collected is located in Bishoftu town around Sunshine, Babogaya, Air force, Ude/Denkaka, Kurkura and Near Dukem town. Laboratory tests were conducted according to American Society for Testing and Materials (ASTM), American Association of State Highway and Transportation Officials (AASHTO), British Standard (BS), Indian Standard (IS) or those procedures listed in standard textbooks or specification manuals. Based on the samples taken from the site, laboratory tests on the samples were conducted in Adama Science and Technology University Geotechnical laboratory room and Ethiopian Construction Design and Supervision Works Corporation. In the investigated area for cracked buildings six 2D crack monitor instruments are installed for 233 days of monitoring period.

3.1.1 Altitude, latitude and longitude

The latitude of Bishoftu is $8^0$40`N - $8^0$50`N and longitude $38^0$55`E - $39^0$5`E as shown in Figure 3.1. The town is the largest and most populated city, located in the East Shewa zone of Oromia National Regional State. The altitude of the city is 1,920 m above mean sea level.

3.1.2 Precipitation and Temperature Conditions

The climate here is mild, and generally warm and temperate. In winter, there is much less rainfall in Bishoftu than summer. Full summer season is the rainy season. Months with largest precipitation are August, July, September with 576mm precipitation. Most precipitation occurs in August with an average precipitation 220mm. The annual amount of precipitation in Bishoftu is 968mm. The average annual temperature is 25^oC in Bishoftu. The warmest month of the year is March, with an average temperature 27^oC. Usually July is the coldest month in Bishoftu, with average temperature 23^oC.

The difference between the hottest month: March and the coldest month: July is 4^oC. The difference between the highest precipitation (August) and the lowest precipitation (December) is 217mm, as shown below Figure 3.2.

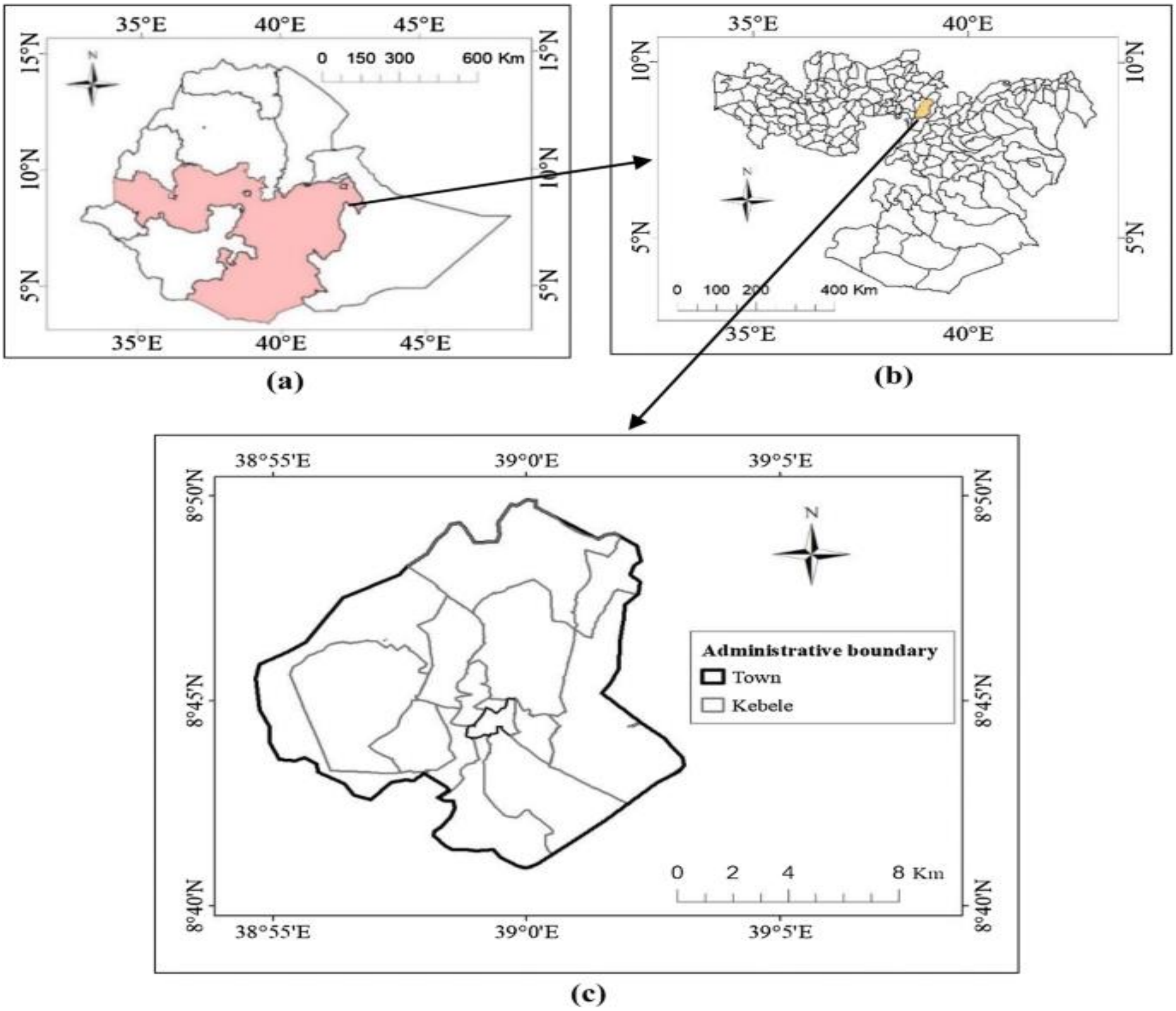


Figure 3. 1: Map of the study area (a) Ethiopia (b) Oromia region (c) Bishoftu town

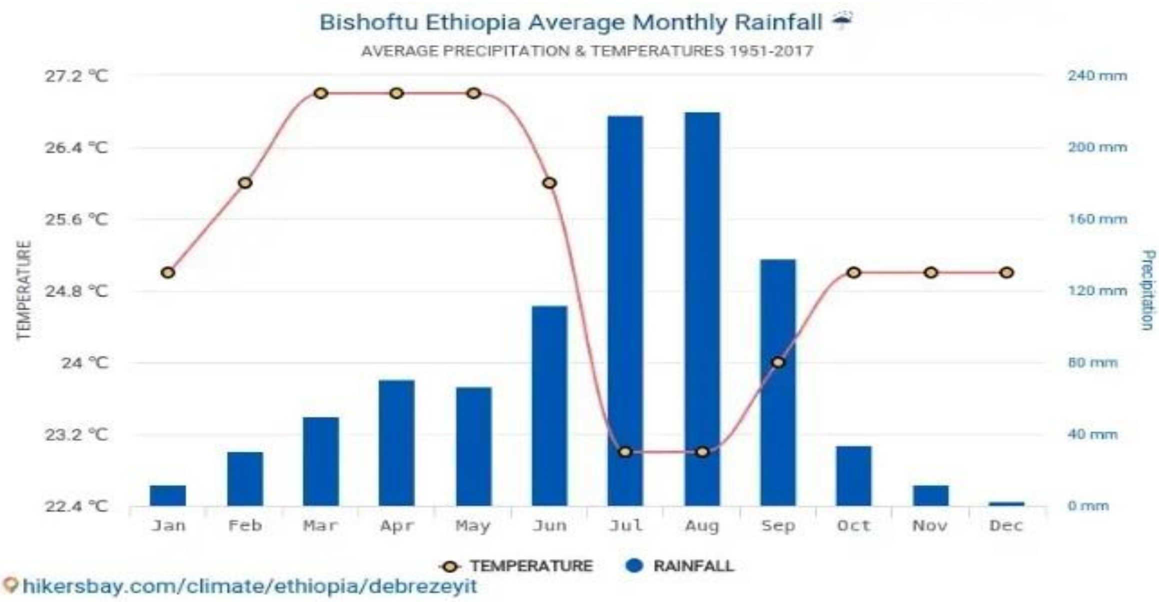


Figure 3. 2 A basic climate data set 1957-2017 for Bishoftu

3.2 Method of study

3.2.1 Site Reconnaissance

When selecting possible sampling sites, the major factor considered extremely important was that the site to be definitely located in the expansive soil region. To ensure samples covered a sufficient area of this formation, sites were identified by visual investigation and field identification of expansive soil; if the soils show expansive nature, the samples were collected.

The field investigation started with the site reconnaissance in order to collect information about the house construction and how the failure occurred from the owners to assess in investigating the source and reasons of failures. For each selected house the required data was first collected by conducting physical observation. This task has the following three major components:

i. Identifying construction material of each component of the building,

ii. Careful observation and analysis of extent of damage in each building element, and

iii. Studying the main cause of failures**.**

The second task was interviewing house owners or contractors who had been participating in construction of buildings or people directly involved during the construction period. Additionally, take information from Bishoftu town construction office design approval team about buildings constructed on expansive soil.

The last step was interpretation of collected data. Hence; damage is analyzed and interpreted systematically with respect to type of foundation, wall, floor, pavement, door/window and fence.

3.2.2 Fixed *2D* crack monitor for cracked buildings on the wall

Cracks may appear in civil engineering structures, such as buildings, the body of a dam, its galleries, adjoining tunnels, and power plants during or after construction. Swelling or poor soils in the foundations, redistribution of stresses in the tunnels, creep of the materials, earthquakes or other vibrations are just a few reasons for cracks. The relative movements of the walls along and across the crack, beyond certain limits may prove to be damaging (Tewatia, 1997). For considerations of safety and maintenance, the measurement of the magnitude and time rate of deformation of the cracks is essential. Instrumentation for deformation monitoring is of vital importance; in buildings it is necessary to monitor the relative deformation of crack at a certain time t. A 2D crack monitor is used for 2D crack monitoring, along the crack (x-axis) and across the crack (y-axis). In Bishoftu town there are no barrages, tunnels and big costly structures, the purpose of 2D crack monitor is within a low cost house owners can monitor their buildings.

Procedures for the 2D crack monitor installation

- ✓ Fix the two plates on the wall, plate A and plate B, need screws or adhesives.
- ✓ Stay the instrument for 24 hours after installing.
- ✓ Take the initial reading at a time t=0 day by digital vernal calipers, along the crack (x-axis) and across the crack (y-axis). The reading is taken in mm.
- ✓ Care should be taken before the initial reading has been taken, the instrument is fixed to the wall without any movement, when the reading is taken the caliper is fixed to its position properly.
- ✓ Take the next reading after 15 days or once a month.
- ✓ Data should be recorded starting from t =0 up to the required date.
- ✓ Interpret what looks like relative deformation versus time (days) along and across the crack within graphical presentation.
- ✓ Identify whether the crack is opening (+) or closing (-)
- ✓ Compare and contrast change in deformation (Δd) with the initial reading.

For the past 233 days or nearly 8 months a 2D crack monitor has been installed on cracked buildings. 6 instruments were installed on 3 buildings, these are Building ID No. 2 (1 instrument), Building ID No. 26 (3 instrument) and Building ID No. 40 (2 instrument) from June 4, 2022 up to January 22, 2023 monitoring period.

3.2.3 Method of sampling and sample size determination

Random sampling method is used for most scientific studies and implemented in this research too for sample size determination. According to this method sample size has been determined as follows (Smart, 2012).

$$n = \frac{z^2 p\, q}{{d_p}^2} \tag{3.1}$$

z = 95% confidence interval is used for most studies. This means that we can be 95% sure that the true population value for our damage assessment is within the limits of the interval calculated. At 95% confidence the value of z =1.96.

p = Information to determine the estimated prevalence can be obtained from various sources such as previous studies, surveillance data or rapid assessment results (Smart, 2012).

Since studies are done in our country, Sisay (2004) takes p = 0.5 (50%) which gives the maximum sample size because no other research is done on buildings which are damaged due to problems related to expansive soils in our country. In this study ninety-six randomly selected light weight buildings founded on expansive soils of Addis Ababa were examined. From the research he found that 72% of buildings are damaged due to problems related to expansive soils.

Solomon (2015) assessed the damages caused by expansive soil on buildings constructed in Bahir Dar, he used prevalence value (p) 0.72 (72%) and which gives a sample size of 78 buildings, out of this 87% of buildings are damaged due to problems related to expansive soils.

Depending on the above researches done previously estimated prevalence value (p) 0.87 (87%) was taken from Solomon study.

There is no standard precision to use when planning a survey. Desired precision (d_p) depends on the objectives of the survey, estimated prevalence or rate, and resources (time and finance) available. Other things kept equal, the higher your desired precision, the larger is the sample size**.** For this study +/-10% precision value is selected. So the sample size would be:

$$n = \frac{1.96^2 x\ 0.87\ x\ (1-0.87)}{0.1^2} \quad = 44 \text{ buildings}$$

These 44 buildings are distributed over every expansive soil location in the study area, based on area coverage and number of buildings constructed in the area as shown below in Table 3.2.

3.2.4 Preparation of disturbed soil samples

The disturbed samples recovered were used to determine results for the Atterberg Limits, Free Swell, Specific Gravity, Linear Shrinkage and Particle Size distribution using both sieve and hydrometer methods. Breaking up the soil aggregates by rubber covered mallet. Then, sieve analysis is performed to separate the dried soils into different groups. The first group involves preparing uniform samples for Atterberg limits, Free Swell Index tests, Linear Shrinkage and others for Specific Gravity, Grain Size Analysis and for Scanning electron microscope in different sieve sizes.

i. Preparation of soil for Liquid Limit, Plastic Limit, Free Swell and Linear Shrinkage

The method adopted for all of these tests was the dry preparation method using a mechanical device. The sample that has been rubbed down is then sieved through a 425μm sieve (No.

40). After the entire sample has been sieved, the material which has passed through this sieve is then split to obtain a mass of at least 250g, so all four tests can be performed.

ii. Preparation of soil for Specific Gravity and One dimensional consolidation test

The sample that has been rubbed down is sieved through a 2mm sieve (No.10 sieve). After the entire sample has been sieved, the material which has passed through this sieve is oven dried and then weight 15g of soil sample for Specific gravity determination for each trial test and take 2500g of soil for one dimensional consolidation test.

iii. Preparation of soil for Compaction test

Take 2500g of air dried soil sample passing through a 4.75mm sieve (No.4 sieve). No further rubbing down the sample is needed due to fine grained soil and wide sieve size.

iv. Preparation of soil for Sieve and Hydrometer test using dry and wet sieve method.

For fine-grained soils that may contain a high fraction of clay soils, it was necessary to soak the soil prior to washing in order to keep the fine material from adhering to larger particles. Take 1kg of representative soil samples from 15kg soil of each test pit. Soak a 1kg of soil sample and wash through the 75µm sieve (Sieve No. 200). After washing, the material which has retained on 75µm sieve is air dried for mechanical sieve and 55g of the material which has passed through 75µm sieve is used for hydrometer test.

v. Preparation of soil for Scanning Electron Microscope (SEM)

From a sample of soil which has passed through 150µm sieve take 1g of soil for the Scanning Electron Microscope test. For investigating soil microstructure at 1000 times magnification SEM has great advantages. SEM is used to determine whether the soil is aggregate structure and sheet or plate structure.

Table 3. 1: Sample preparation and standard test methods

No.	Tests	Sieve No	Sieve Opening (mm) pass	Amount (g)	Standard Test Methods	Sample Type
1	Liquid limit, LL	40	0.425	100	ASTM D-4318	Air dried
	Plastic limit, PL	40	0.425	30	ASTM D-4318	Air dried
	Linear shrinkage, Ls	40	0.425	80	BS-1377	Oven dried
	Free swell, F_s	40	0.425	20	IS: 2720 (Part 40) 1977	Oven dried
2	Specific gravity, G_s	10	2	15	ASTM D-854	Oven dried
	Consolidation	10	2	2500	ASTM D-2435	Oven dried
3	Compaction	4	4.75	2500	ASTM D-698	Air dried
4	Mechanical sieve (Dry)	4 - pan	4.75 – pan	1000g - (>50% passing No. 200)	ASTM D422-63	Air dried
	Hydrometer (Wet)	200	0.075	55	ASTM D422-63	Air dried
5	Natural moisture content, w	Soil sample taken from site by plastic bag immediately		30 - 40	ASTM D-2216	
6	SEM	100	0.150	1		Air dried

3.3 Soil sampling

The expansive soil sample used for this research work is collected eight samples from six test pit, in and around Bishoftu town to determine the engineering properties of soil. The investigation involved collection of relevant geologic maps and associated reports and supplementary study materials from different sources. Regional geologic setting of the area is mainly referred from the country wide geologic map prepared by the Geological Survey of Ethiopia (Figure 2.1). As mentioned in Chapter 2 literature, the central part of Ethiopia following the major trunk roads like Addis-Modjo is one of the areas covered by expansive soils. So this is a justification for expansive soil is also found in and around Bishoftu town. Excavation work is done to a maximum depth of 2m.

Soil samples are taken from, in and around Bishoftu town in different area listed below:

- ✓ Sample 1 and 2 from the compound of individual house owners during toilet excavation around sunshine area at a depth of 1m and 2m respectively.
- ✓ Sample 3 and 4 from holes prepared for electric pole replacement digging at a time, at a depth of 1.5m each around Babogaya and Air force respectively.
- ✓ Sample 5 from Ude/Denkaka area which is far from the center of the town circle square, samples are taken from under construction buildings at a depth of 1.5m.
- ✓ Sample 6 and 7 around Kurkura from the new G+2 building during footing pad excavation work, at a depth of 1m and 2m respectively.
- ✓ Sample 8 which is test pit 6 is around Bishoftu-Dukem border or in front of Eastern Industrial Zone, from ditch work excavation at a depth of 1.5m.

The disturbed soil was placed in a tight plastic bag with a reference tag to describe the location of the sample and the depth taken. This was done to avoid the possibility of contaminating the sample being collected. From each test pit 15kg of soil sample was taken.
Samples taken from Ude/Denkaka and Near Dukem town are far from the center of the town Circle square, this makes in order to compare and contrast what look like the properties of expansive soil found in and around Bishoftu town. From investigation, expansive soil is dominated in the North, East and West part of the town. Soil samples and site description are given in Figure 3.1. The location name, the test pits designation, physical properties of the soil and the depth where the samples taken are shown in Table 3.1.

Figure 3. 3: Soil samples and site description

Table 3. 2: Description of test pit location and investigated buildings

Location	Test pit ID	Depth in (*m*)	Physical properties	Easting	Northing	Elevation	Construction status	Distance from center of the town (*km*)	No. of Investigated Buildings
Sunshine	Tp1	1.00	Black Clay	494258	969152	1898	Many Building	4.74	7
Sunshine		2.00	Gray Clay						
Babogaya	Tp2	1.50	Black Clay	501672	967831	1878	Some Building	3.68	6
Airforce	Tp3	1.50	Black Clay	501552	963105	1866	Some Building	5.22	6
Ude/ Denkaka	Tp4	1.50	Black Clay	503651	959347	1873	Many Building	8.65	7
Kurkura	Tp5	1.00	Black Clay	492598	968331	1917	Many Building	5.14	7
Kurkura		2.00	Gray Clay						
Near Dukem	Tp6	1.50	Black Clay	492108	970555	1922	Many Building	7.02	6
Danbi									3
Kebele 05									2
									Total 44

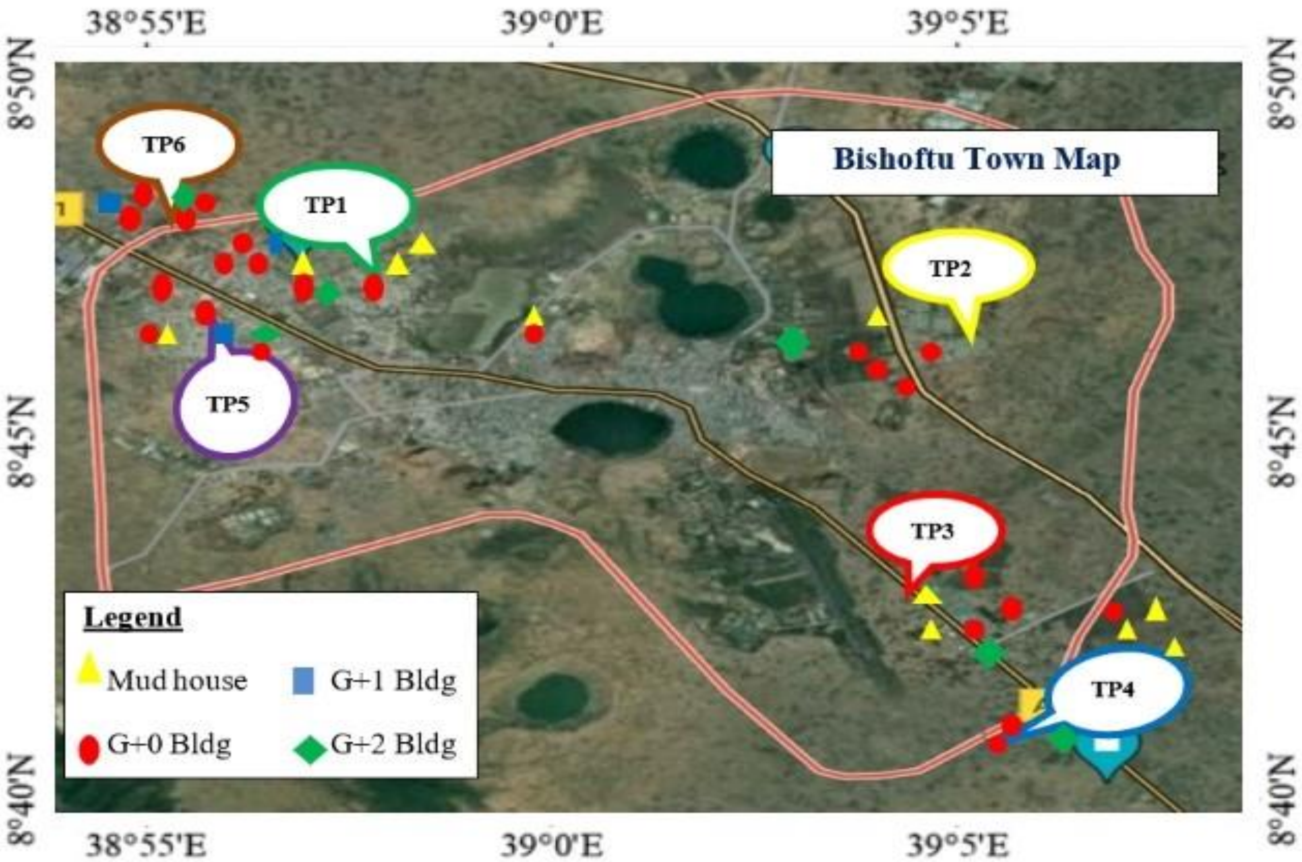


Figure 3. 4: Location of test pits and investigated buildings (*Google map*)

3.4 Materials

Table 3. 3: Laboratory and field materials used, to achieve the objectives of this research

No.	Laboratory test materials	
	Tests	**Materials needed**
1	Liquid Limit	Casagrand apparatus, with grooving tools, Sieve No. 40
		Balance, Cans, Oven (105-110^oC)
		Soil mixing equipment (Porcelain dish, Spatula, Plastic squeeze bottle)
	Plastic Limit	Glass plate, Oven (105-110^oC), Sieve No. 40, Balance, Cans
		3mm diameter welding rod for visual inspection
2	Free Swell	Distilled water, Kerosene
		Glass cylinder (100ml), Sieve No. 40, Balance
3	Linear Shrinkage	Half cylinder with Diameter = 12.5mm x 134.72mm length Lo
		Oven (105-110^oC), Sieve No. 40, Digital vernal calipers
4	Specific Gravity	Density bottle, Sieve No. 10
		Vacuum pump
5	Hydrometer analysis	Hydrometer (ASTM 152H), Thermometer, Sieve No. 200
		Sedimentation cylinder (1000ml)
		Hydrometer jar (for controlled temperature)
		Sodium hexametaphosphate ($NaPO_3$), Stop watch or Timer
	Mechanical sieve	Mechanical sieve shaker, Balance, Brush
		A stack of sieves (Pan, Sieve No.200 to No.4)
6	Compaction	Standard compaction test , Compaction rammer/manually operated
		Compaction mold with base plate and collar, Oven (105-110^oC)
		Metal straight edge (to level the mold containing soil), Cans, Sieve No. 4
		Large mixing pan and large spoon for dispensing soil, Balance
		Graduated cylinder (for adding a measured quantity of water in the soil)
	One dimensional consolidation	Consolidation device (including ring, porous stones and load plate), Dial gauge (0.002mm=1unit on dial gauge)
		Sample trimming device, Sieve No. 10, Balance

7		Loading device, Filter paper, Oven (105-110^oC)	
8	SEM	Sieve No. 100	
	Field materials		
1	15kg of soil samples from each test pit		
2	Sampling tools for disturbed soil samples		
3	GPS, Notecam/for picture taking		
4	Computer (Microsoft excel)		
5	2D Crack monitor	Two mica/glass or Aluminum plate with a dimension of 40mm width and 70mm length.	
		Digital vernal calipers	
		Screws or adhesives in order to fix the instrument on the wall	
6	Questioners or Standard formats to collect data from inspected buildings		
7	Interview with House owners, Construction office/Design approval team, Contractors		

Figure 3. 5: Laboratory tests and apparatus needed

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 General

The tests required for determination of engineering properties are generally elaborated and time consuming. Index properties of soil are properties, which are used to characterize the soils and determine their basic properties. Index properties include the Water content (also known as Moisture content), Specific gravity tests, Unit weight determinations/Compaction test, Particle size distributions, Free swell tests and Atterberg limits, which are used to classify the soil.

4.2 Laboratory Test Results

4.2.1 Results for Natural Moisture Content Tests

For many soils the water content may be an extremely important index used for determining the relationship between the way a soil behaves and its properties. The consistency of a fine grained soil largely depends on its water content. The moisture content test was carried out in the laboratory as per the procedure of ASTM D 2216. The laboratory test results of the natural moisture content of the study area falls in the range of (26.73 - 49.03) %, shown in appendix A.

4.2.2 Results for Atterberg Limit Test

The Atterberg Limits were determined for air-dried samples based on ASTM D 4318 standard test method for Liquid Limit, Plastic Limit, and Plasticity Index of soils. The laboratory test result of Atterberg limits of the soil under investigation and the classification made based on the results are summarized in Table 4.8 and Appendix B.

i. Liquid Limit

The liquid limit represents the moisture content at which any increase in moisture content will cause a plastic soil to behave as a viscous liquid. The liquid limit is defined as the moisture content at which a standard groove cut in a remolded sample will close over a distance of ½-inch (13mm) at 25 blows of the liquid limit device. The liquid limit value of the study area falls in the range of (61.6 - 94.4) %.

ii. Plastic Limit

The Plastic Limit (PL) is the water content, in percent, at which a soil can no longer be deformed by rolling into 3 mm diameter thread without crumbling. This test is somewhat more operator dependent than the liquid limit test; a visual detection of a 3 mm diameter is subject to some interpretation. Soil is set aside (20-30)g when blow count is around 50 blows (Bowles, 1978). Plastic limit value of the tested samples ranges from (27 - 40.7) %.

iii. Plasticity Index

The numerical difference between the liquid limit and the plastic limit termed as plasticity index indicates the magnitude of the range of moisture content over which the soil remains plastic. Plasticity index ranges from (23.8 - 54) %.

$$PI = LL - PL \tag{4.1}$$

vi. Results for Liquidity Index test

Liquidity Index (LI): The Atterberg limits are found for disturbed soil samples. These limits as such do not indicate the consistency of undisturbed soils. The index that is used to indicate the consistency of undisturbed soils is called the liquidity index or water plasticity ratio. The liquidity index is expressed as: -

$$LI = \frac{(w-PL)}{PI} \tag{4.2}$$

The value of Liquidity Index (LI) varies according to the consistency of soils as follows:-

LI	Description of soil strength
LI < 0	Semisolid state- High strength but brittle, i.e sudden fracture is expected
0 < LI <1	Plastic state- Intermediate strength
LI > 1	Liquid state- Low strength

Table 4. 1: Results for Liquidity Index of the tested samples

Test pit ID	Depth (*m*)	Natural Moisture Content, *w* (%)	Plastic Limit, *PL* (%)	Plasticity Index, *PI* (%)	Liquidity Index (*LI*)	Remark
Tp 1	1.00	40.54	29.80	30.00	0.36	Plastic state
Tp 1	2.00	42.02	36.20	47.20	0.12	Plastic state
Tp 2	1.50	29.83	30.70	30.90	-0.03	Semisolid state
Tp 3	1.50	26.73	35.00	54.00	-0.15	Semisolid state
Tp 4	1.50	30.5	27.00	38.50	0.09	Plastic state
Tp 5	1.00	47.93	33.40	23.80	0.61	Plastic state
Tp 5	2.00	49.03	40.70	53.70	0.16	Plastic state
Tp 6	1.50	35.97	37.80	52.20	-0.04	Semisolid state

4.2.3 Results for Free Swell Index

Free swell index is the increase in volume of a soil, without any external constraints on submergence in water. Free swell index is determined using Equation 2.2. According to IS:2720 (Part 40) 1977 test procedure the Free Swell Index results of the study area ranges from (21.74 - 81.82)% and the degree of expansiveness is from moderate, high and very high depending on the result of the tested soils, see in Appendix C.

4.2.4 Results for Linear Shrinkage

Linear shrinkage test is a laboratory test for measuring the linear shrinkage ratio of a soil sample that passes a 0.425mm sieve. Linear shrinkage is the decrease in length of a soil sample when oven-dried, starting with a moisture content of the sample at the liquid limit. The British standard BS 1377 uses a half-cylinder mold of Diameter=12.5mm x 140mm length L_o. The wet sample is oven dried and the final length L_f is obtained, from this the linear shrinkage L_s is computed as

$$Ls = \frac{(L_o - L_f)}{L_0} 100\% \qquad (4.3)$$

Table 4. 2:Results for Linear Shrinkage of the tested samples and degree of expansion

Sample No	Location	Depth in (m)	Test pit ID	Physical properties	Original sample length, L_o (mm)	Oven dried sample length, L_f (mm)	Linear Shrinkage (L_S) %	Degree of Expansion
1	Sunshine	1.00	*Tp1*	Black Clay	134.72	118.90	11.74	Critical
2	Sunshine	2.00	*Tp1*	Gray Clay	134.72	111.52	17.22	Critical
3	Babogaya	1.50	*Tp2*	Black Clay	134.72	117.31	12.92	Critical
4	Airforce	1.50	*Tp3*	Black Clay	134.72	113.89	15.46	Critical
5	Ude/Denkaka	1.50	*Tp4*	Black Clay	134.72	114.82	14.77	Critical
6	Kurkura	1.00	*Tp5*	Black Clay	134.72	120.12	10.84	Critical
7	Kurkura	2.00	*Tp5*	Gray Clay	134.72	104.60	22.36	Critical
8	Near Dukem	1.50	*Tp6*	Black Clay	134.72	107.62	20.12	Critical

4.2.5 Results for Specific Gravity Tests

In this research the specific gravity tests were determined using ASTM D854 standard test procedures. According to the laboratory test results, the specific gravity of black clay ranges from 2.67 - 2.72 and for gray clay ranges from 2.68 - 2.70. Therefore, the specific gravity of the investigated area soil ranges from 2.67 - 2.72 and the results are given in Appendix D.

4.2.6 Results for Grain Size Analysis (Sieve and Hydrometer Analysis Tests)

Grain size analysis tests were carried out in accordance with ASTM D 422-63 standard test method. According to ASTM D422-63 the distribution of particles, finer than 75µm can be done by hydrometer test and coarser than 75µm by mechanical sieve. The combined grain size distribution curve of test pit 2 for mechanical sieve & hydrometer tests is shown in Figure 4.1. The detail summary of laboratory test results of the Grain Size Analysis of the study area shows: - Gravel (0.20 - 2.30) %; Sand (4.00 - 16.20) %; Silt (20.44 - 39.69) %; Clay (50.06 - 73.96) % and finer than 0.075mm (82.80 - 95.50) %. The results are given in Appendix E.

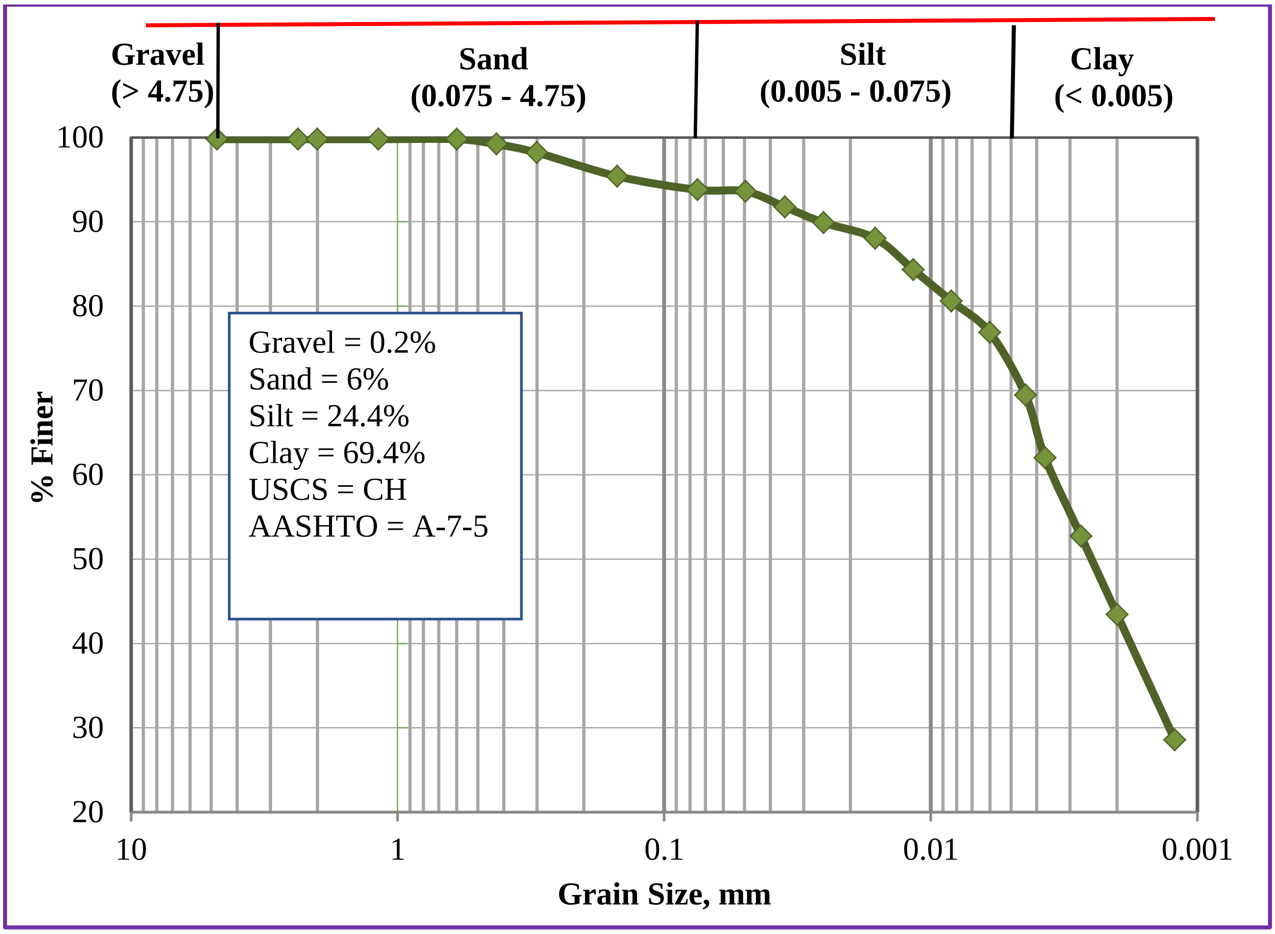


Figure 4. 1: Grain size distribution curve of test pit 2 @ 1.5*m*

4.2.7 Results for Compaction Tests

Compaction is the process of densification of soil by reducing air voids. The degree of compaction of a given soil is measured in terms of its dry density. The dry density is Maximum at the optimum moisture content. To obtain the maximum dry density and the optimum water content, a graph is drawn with axes of dry density and water content. Optimum moisture content of the tested samples ranges from (28.6 - 37.83) % and Maximum dry density ranges from (1.22- 1.42) g/cm^3. The tests are conducted in accordance with the ASTM D-698 testing procedure and the results are given in Appendix F.

Table 4. 3; Maximum dry density versus Optimum moisture content for all tests

Sample No	Test pit ID	Depth in (*m*)	Optimum moisture content (%)	Maximum dry density (*g/cm^3*)
1	*Tp1*	1.0	33.93	1.41
2	*Tp1*	2.0	32.71	1.22
3	*Tp2*	1.5	28.63	1.34
4	*Tp3*	1.5	37.83	1.27
5	*Tp4*	1.5	33.06	1.37
6	*Tp5*	1.0	32.73	1.42
7	*Tp5*	2.0	37.54	1.27
8	*Tp6*	1.5	30.54	1.31

Note: Soil samples at 1*m* depth have maximum dry density.

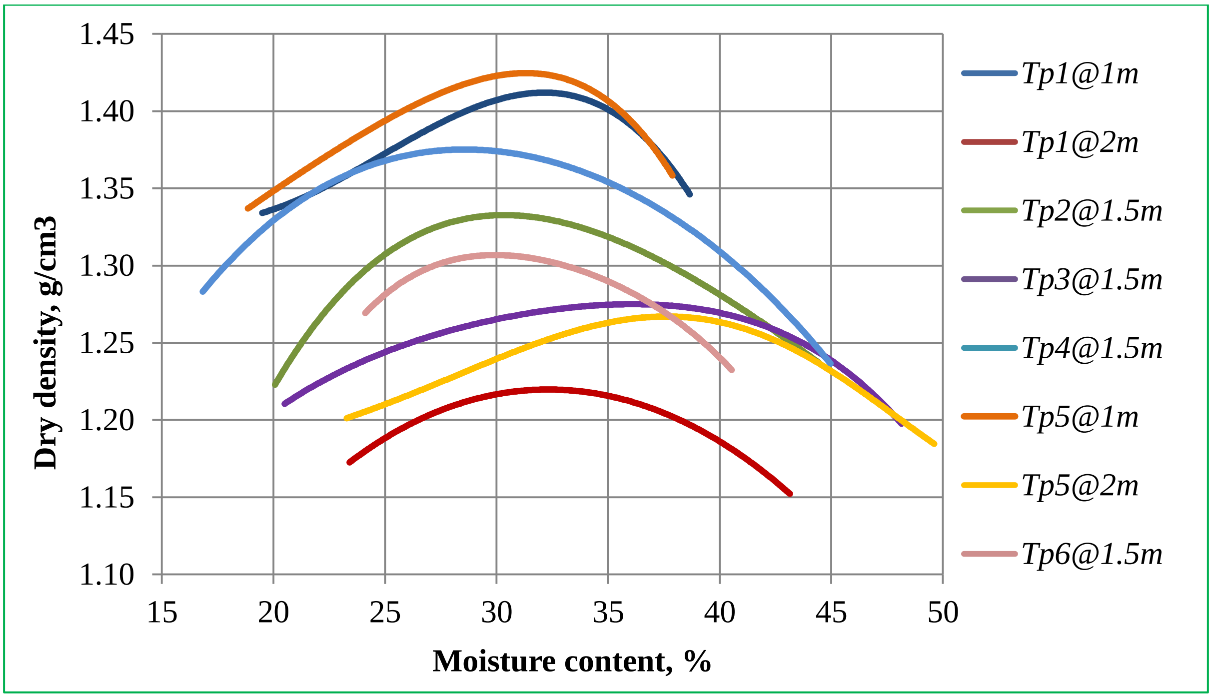


Figure 4. 2: Moisture content versus dry density for all tests.

4.2.8 Results for One Dimensional Consolidation Tests

This test is performed to determine the magnitude and rate of volume decrease that a laterally confined soil specimen undergoes when subjected to different vertical pressures. From the measured data, the consolidation curve (e-log p) can be plotted. This data is useful in determining the compression index (Cc), the re compression index (Cr) and the pre-consolidation pressure (Pc) or maximum past pressure of the soil. The data obtained can also be used to determine the coefficient of consolidation (Cv).

The consolidation properties determined from the consolidation test are used to estimate the magnitude and the rate of both primary and secondary consolidation settlement of earthen material. The data from the consolidation test are used to estimate the magnitude and rate of both differential and total settlement of a structure.

Primary consolidation when the rate of densification is controlled by the time required for expulsion of fluids, this volume change is called primary consolidation. Primary consolidation is started when the soil is fully saturated and completed when pore water expulsion stops. Primary consolidation refers to consolidation due to dissipation of excess water pressure, while Secondary consolidation refers to the creep process or readjustment of soil particles.

Secondary consolidation is the continued deformation of the soil structure after excess pore pressure has dissipated, as small numbers of particles move at random shear strains, in a Poisson process, to new final positions. Secondary consolidation is the volume change controlled by anything else, such as the sliding of particles over each other or compression of particles. It is a time dependent process and much slower than the primary consolidation.

In the laboratory test is conducted with an apparatus known as Consolidometer or Oedometer consisting essentially of a loading frame and consolidation cell in which specimen is kept. There are two types of cells: Fixed ring and Floating ring.

The test is based on the ASTM D-2435 standard test method for one dimensional consolidation properties of soils. A sample of clay is placed in a small consolidometer or oedometer and allowed to reach equilibrium under a small hanger load. A series of load increments are applied and the dial gauge reading is recorded at the start and after equilibrium is reached for each increment.

The most important properties furnished by a consolidation test are :-

- ✓ The C_v, which indicates the rate of compression under a load increment.
- ✓ The C_c, which indicates the compressibility of normally consolidated soil.

- ✓ The P_c, the maximum stress that the soil has felt in the past.

i. Square Root-Time Method (Taylor Method) to find the value of C_v (m^2/year)

To make a plot of dial reading versus √(time), locate the coordinates to draw the curve. To find D_o draw a straight line through the first several (6-8) plotted points and extend this line until it intersects the ordinate (y-axis) to give D_o (line A). Next take an abscissa (x) value 15 % greater than the value obtained by the intersection of the straight-line, and from Do draw a straight line through this point (line B). Where the plotted curve crosses the line B (1.15 offset line), the ordinate value is taken as D_{90}. Find the corresponding time for D_{90} to evaluate C_v, as shown,

$$Cv = \frac{0.848\ d^2}{t_{90}} \tag{4.4}$$

d is the average length of the sample.

For two-way drained d=H/2, for one way drainage d=H

ii. Finding the compressibility index C_c

From the plot of *e-log p*, C_c can be obtained from the slope of the straight line part as

$$Cc = \frac{(-\Delta e\ )}{Log\ (\frac{P2}{P1})} \tag{4.5}$$

iii. Finding the pre-consolidation pressure, P_c (Casagrande method)

From the plot of *e-log p*, casagrande proposed that the P_c could be estimated as follows: -

- ✓ At the sharpest part of the curve, the maximum curvature occurs, draw a tangent line.
- ✓ Through this point draw a horizontal line.
- ✓ Bisect the formed angle.
- ✓ Project the straight line (virgin compression line) portion of the *e-log p* plot back to intersect the bisector of angle at a point. The abscissa of this point is the P_c.

The results of the coefficient of consolidation (C_v), compressibility index (C_c) and initial void ratio (e_o) of the tested samples are;-

- ✓ C_v ranges from (0.334 – 3.049) $m^2/year$
- ✓ C_c ranges from (0.369 - 0.555)
- ✓ e_o ranges from (1.37 – 1.68)

Since the value of coefficient of consolidation C_v is taken the average value.

The results for one dimensional consolidation tests are given in Appendix G.

One dimensional consolidation test results of test pit 2 @ 1.5m depth is given below:-

The sample is remolded at Optimum moisture content.

Sample Type Remolded, Test pit 2@1.5*m*

Before Test

Weight of Sample & Ring (*g*)	119.80
Weight of Ring (*g*)	65.60
Weight of Sample (*g*)	54.20
Weight of Dry sample (*g*)	42.60
Weight of Initial Moisture (*g*)	11.60
Initial Moisture Content, M_o (%)	27.23
Initial Void Ratio : $e_o = (G_s/r_d)-1$	1.480
Height of Solids : $H_s = H_o/(1+e_o)$, (*mm*)	8.064
Initial Saturation: $S_o = (M_o x G_s)/e_o$, (%)	49.49

Specific Gravity (G_s)	2.69
Sample Diameter (*D*) (*mm*)	50.00
Sample Thickness (H_o) (*mm*)	20.00
Area (*A*) (cm^2)	19.64
Volume (*V*) (cm^3)	39.28
Bulk Density (r_b) (g/cm^3)	1.380
Dry Density (r_d) (g/cm^3)	1.085

After Test

Weight of Sample + Ring + Can (*g*)	122.80
Weight of Dry sample + Ring + Can (*g*)	108.20
Weight of Ring + Can (*g*)	65.60
Weight of Wet sample (*g*)	57.20
Weight of Dry sample (*g*)	42.60
Final Moisture Content, M_f (%)	34.27

Overall settlement (*mm*)	4.668
Volume change (cm^3)	9.17
Final Volume (cm^3)	30.11
Final Bulk Density (g/cm^3)	1.900
Final Dry Density (g/cm^3)	1.415

Sample Diameter (*D*) (*mm*)	50
Height of Solids : H_s=	8.064

Height H_o, (*mm*)	20
Initial Void Ratio : e_o=	1.48

Table 4. 4: Consolidation dial gauge reading for test pit 2 @1.5*m*

Time, min	**√Time,min**	**Dial gauge reading in *mm***						
		180kPa	**360kPa**	**720kPa**	**1440kPa**	**2880kPa**	**720kPa**	**180kPa**
0	0	0.000	1.402	2.418	3.564	4.462	5.592	5.132
0.25	0.5	0.466	1.726	2.650	3.726	4.580		
1	1	0.532	1.760	2.770	3.746	4.594		
2.25	1.5	0.598	1.786	2.792	3.762	4.612		
4	2	0.658	1.808	2.808	3.776	4.628		
6.25	2.5	0.702	1.826	2.826	3.792	4.652		
9	3	0.738	1.844	2.840	3.807	4.671		
12.25	3.5	0.788	1.860	2.855	3.822	4.700		
16	4	0.824	1.876	2.870	3.840	4.716		
25	5	0.904	1.910	2.900	3.872	4.764		
36	6	0.978	1.946	2.930	3.910	4.819		
49	7	1.048	1.998	2.964	3.948	4.888		
64	8	1.116	2.034	2.996	3.986	4.932		
81	9	1.150	2.082	3.034	4.030	4.974		
100	10	1.192	2.114	3.067	4.067	5.018		
121	11	1.222	2.164	3.100	4.107	5.036		
225	15	1.266	2.300	3.256	4.224	5.224		
400	20	1.316	2.348	3.402	4.368	5.350		
1440	38	1.402	2.418	3.564	4.462	5.592	5.132	4.668

Square Root-Time Method (Taylor Method) to find C_v ,(m^2/*year*) for *test pit 2@1.5m*

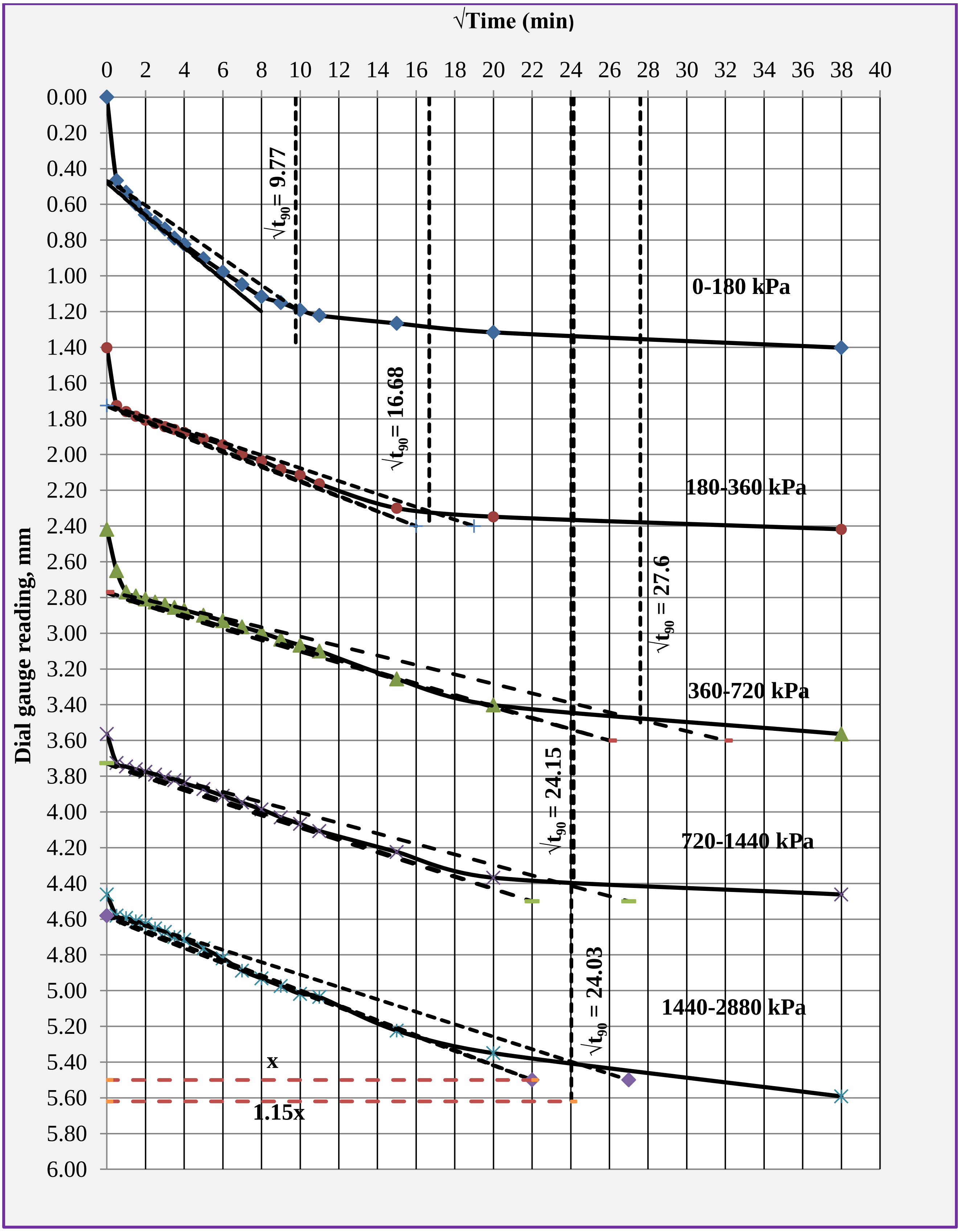


Figure 4. 3: Dial gauge reading versus √Time for test pit 2 @ 1.5*m*

Table 4. 5: Calculation of Coefficient of consolidation, C_v, for test pit 2 @ 1.5*m*

Pressure (*kPa*)	Cumulative compression (ΔH) mm	Consolidated height ($H=Ho-\Delta H$) mm	Void Ratio $e=(H-Hs)/Hs$	t_{90} (min)	$H = 1/2 (H1+H2)$	$C_v = (0.848*H^2)/t_{90}$ ($m^2/year$)
180	1.40	18.60	1.31	95.55	19.30	1.737
360	2.42	17.58	1.18	278.06	18.09	0.525
720	3.56	16.44	1.04	761.76	17.01	0.169
1440	4.46	15.54	0.93	583.22	15.99	0.195
2880	5.59	14.41	0.79	577.68	14.97	0.173
720	5.13	14.87	0.84			
180	4.67	15.33	0.90			

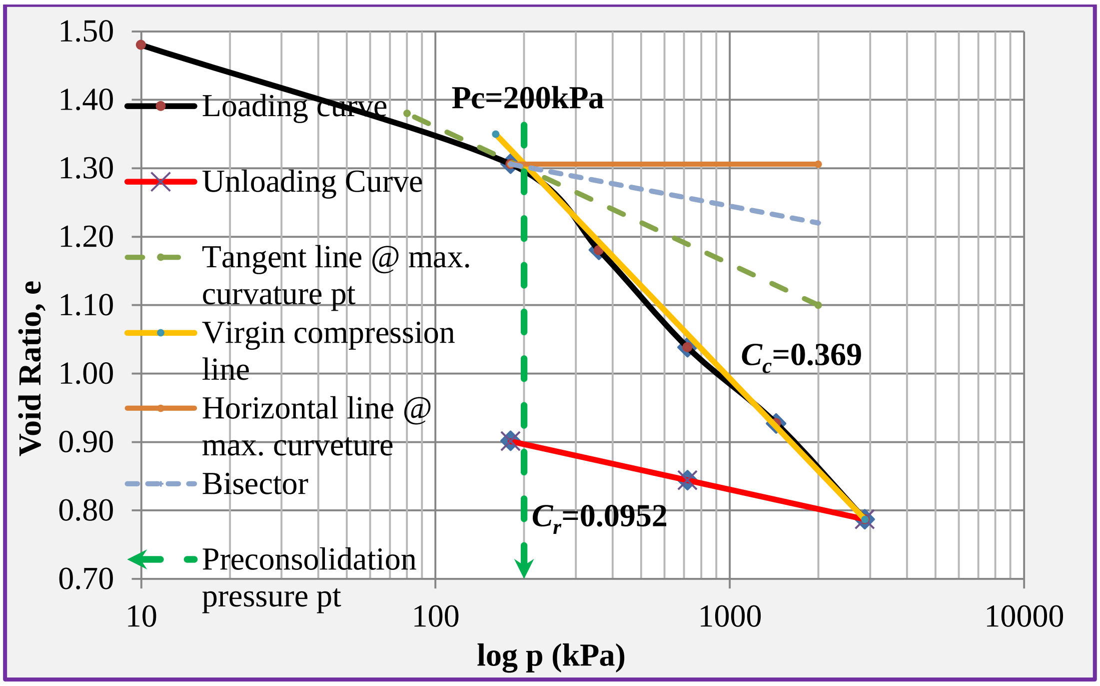


Figure 4. 4: *e-log p* curve for test pit 2 @ 1.5*m*

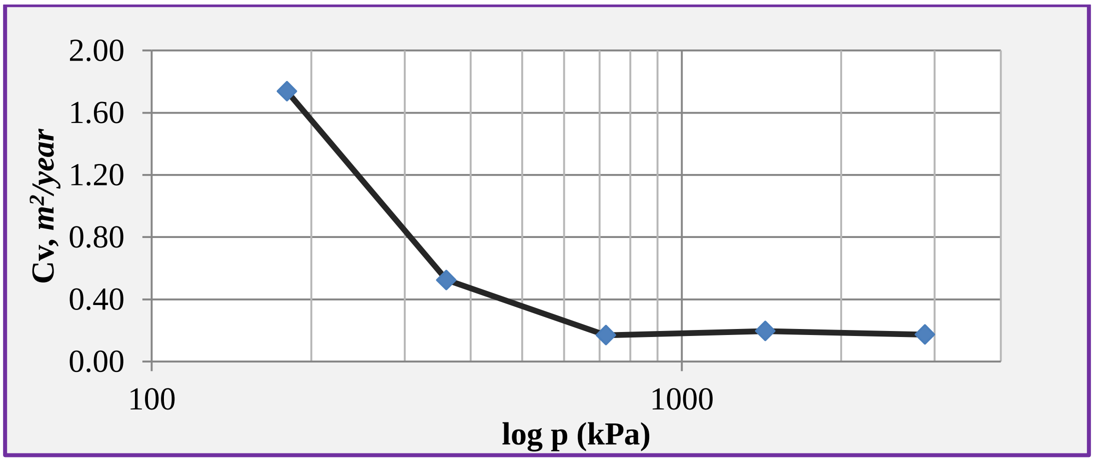
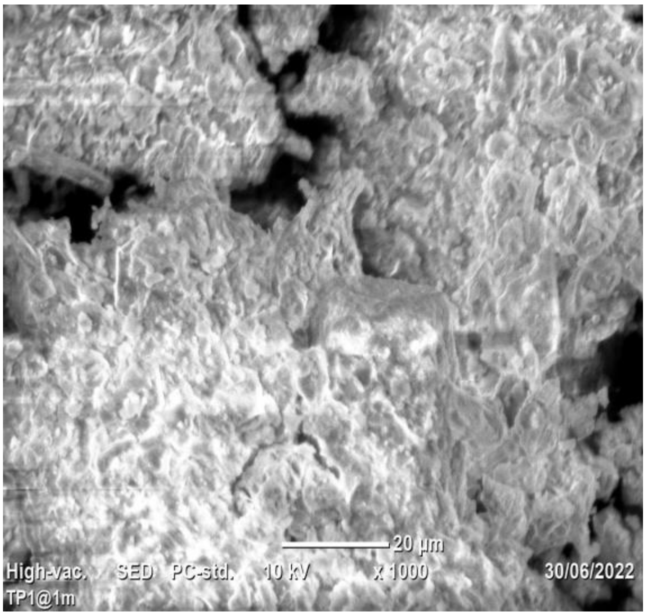


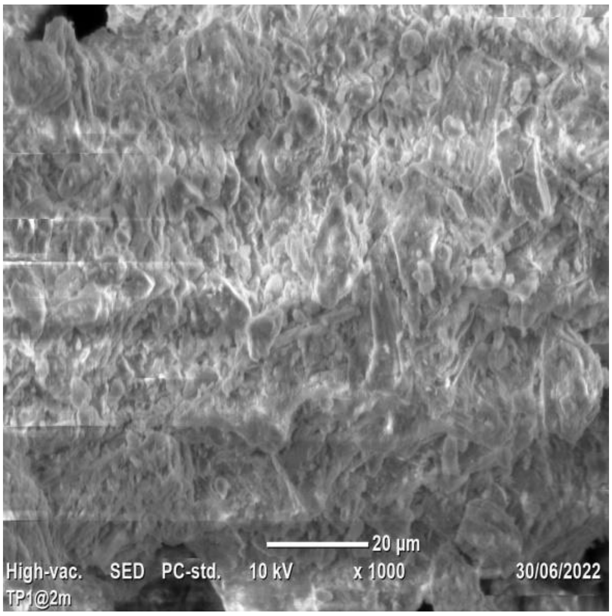
Figure 4. 5: C_v *versus log p* for test pit 2 @ 1.5*m*

4.2.9 Results of SEM for the tested soils

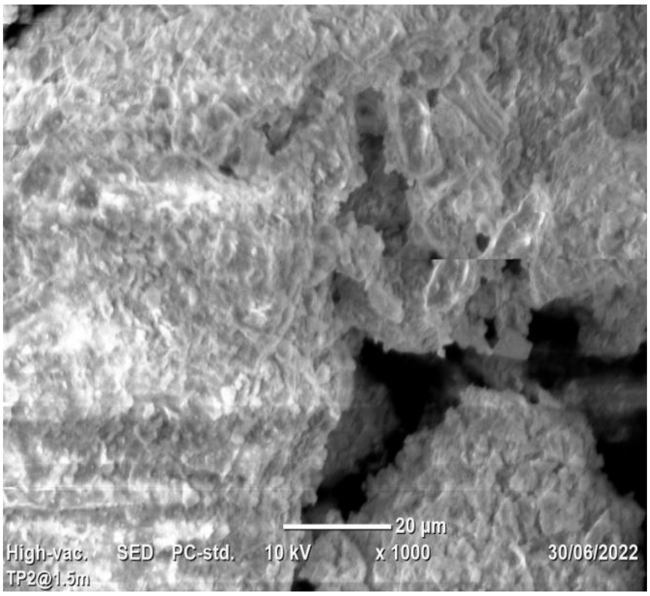
Scanning Electron Microscope (SEM) is an instrument that produces a largely magnified image by using electrons instead of light to form an image. A beam of electrons is produced at the top of the microscope by an electron gun. The electron beam follows a vertical path through the microscope, which is held within a vacuum. The primary reason for the SEM's advantage in investigating soil microstructure is the high resolution that can be obtained when examined.Thus, SEM has been an important instrument in investigating clays.



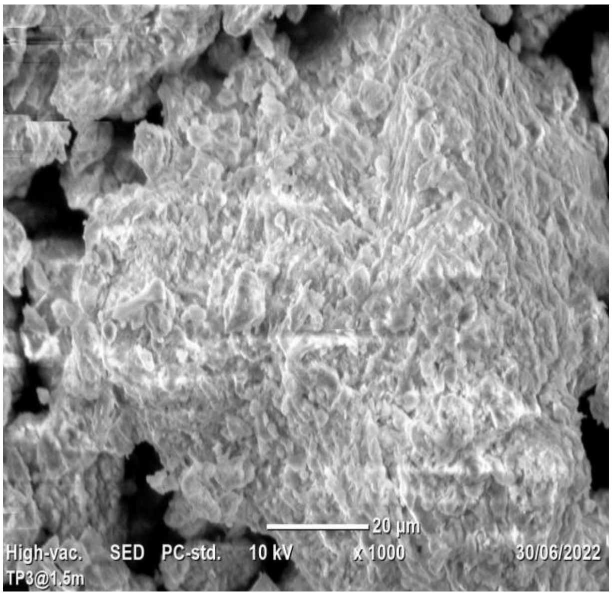
Tp 1 @1m



Tp 1@2m

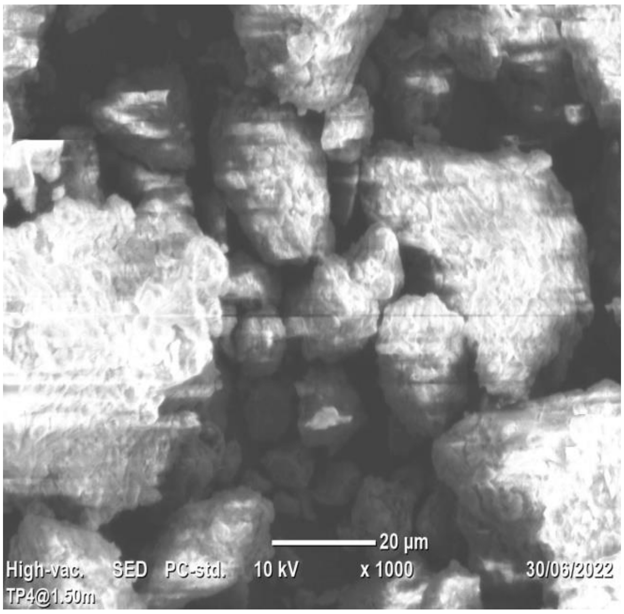


Tp 2 @ 1.5m

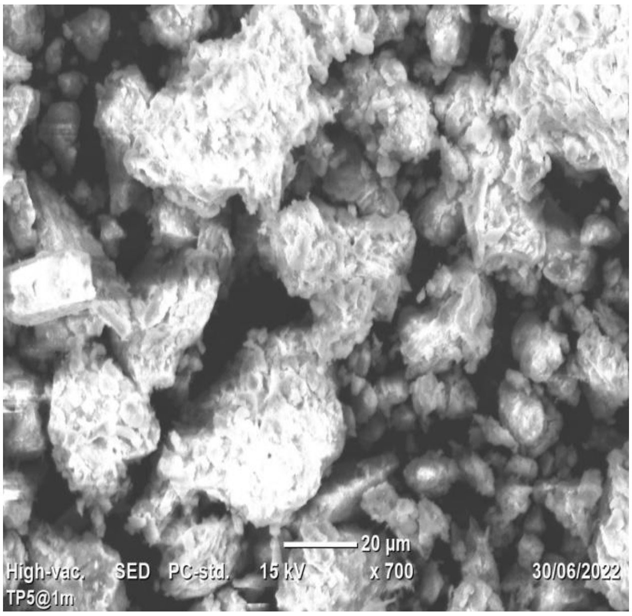


Tp 3 @1.5m

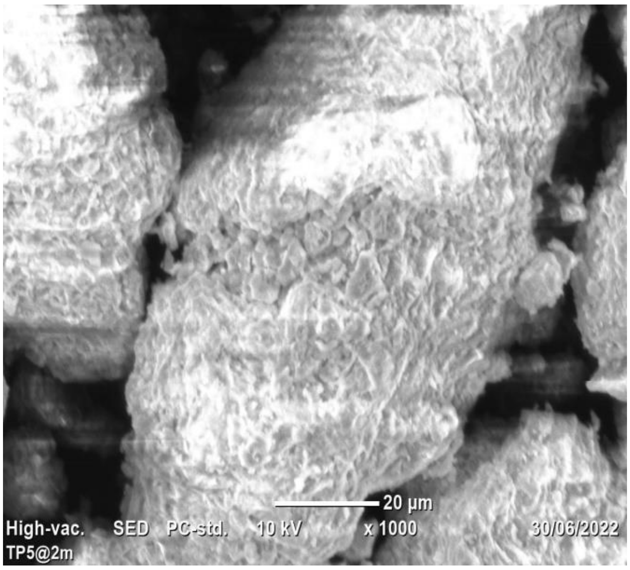
Figure 4. 6: SEM images of tested soils: Tp 1@1m,Tp 1@2m,Tp 2@1.5m & Tp 3@1.5m



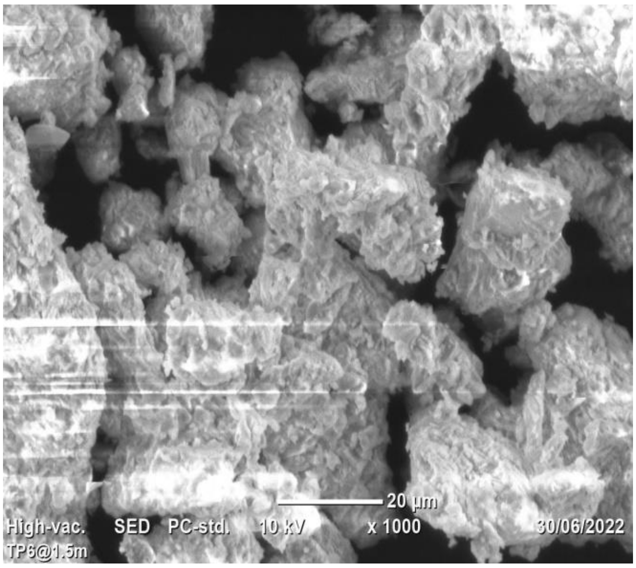
Tp 4 @1.5m



Tp 5 @1m



Tp 5@2m



Tp 6@1.5m

Figure 4. 7: SEM images of tested soils: Tp 4@1.5m, Tp 5 @1m, Tp 5@2m & Tp 6@1.5m

The crump or aggregate structures are main components in the SEM image of Tp 4@1.5m, Tp 5@1m, and Tp 6 @1.5m, while the sheet or plate structures, which are the normal form of clay minerals, are relatively few. On the contrary, the SEM image of Tp 1@1m,Tp 1@2m,Tp 2@1.5m, Tp 3 @1.5m and Tp5 @2m presents less particle groups but more strip and sheet structures. Moreover, the mean particle size of Tp 1@2m is smaller than the rest sample under the same condition of 1000 times magnification.

4.3 Soil Classification of the Study Area

Soil classification is an important aspect of laboratory tests, which tells the characteristics of the soil under interest. There are different methods of classification based on the identification tests performed on the soil. *USCS* and *AASHTO* methods are among the widely used schemes of soil classification. There are also other classification methods specifically proposed for expansive soils.

4.3.1 USCS and AASHTO Classification

i. USCS Classification

According to USCS classification scheme most of the soil of the study area falls in CH or OH region, which shows the soil is potentially expansive. Except soil sample 6, all soil samples are grouped in CH (Fat clay) soil. Soil sample 6 is grouped in MH.

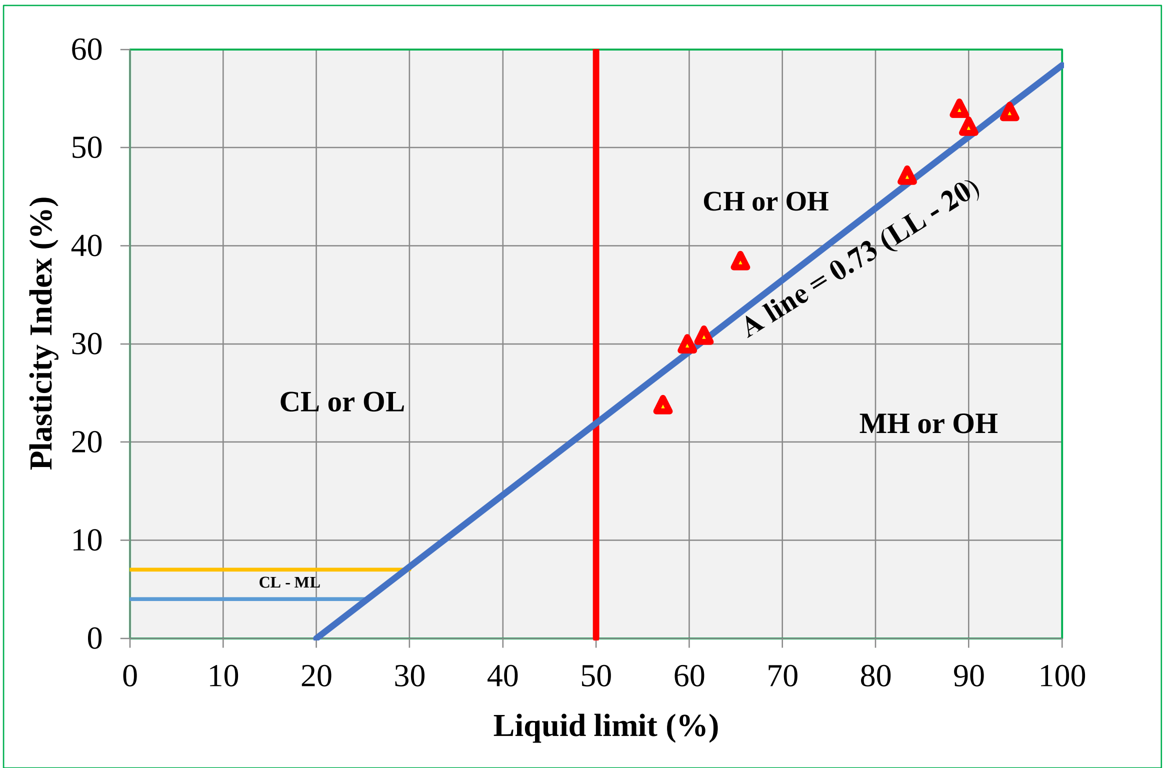


Figure 4. 8: Plasticity chart of the study area soil according to USCS

ii. AASHTO Classification System

According to AASHTO classification system the soil of the study area falls in the region of A-7-5 and A-7-6. Except soil sample 5 all samples are grouped in A-7-5. Soil sample 5 is grouped in A-7-6.

Since A-7-5 = PI <= LL-30 and A-7-6 = PI> LL-30

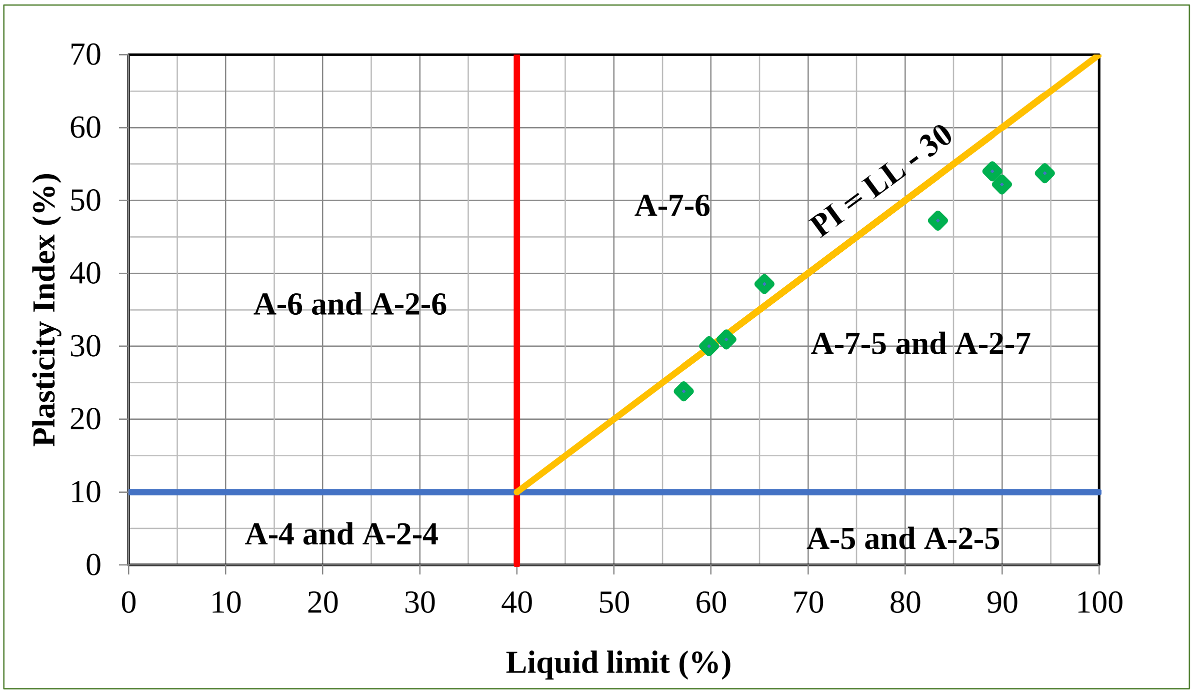


Figure 4. 9: Soil Classification of the study area according to AASHTO system

4.3.2 Other Classifications

i. Skempton's method

Activity which is defined as the ratio of the plastic index to percent of clay fraction finer than 0.002*mm* is one means of classifying expansive soils based on their index property. According to Skempton, clays of the study area are classified with respect to their activity in the range of active and normal clay. These values are shown below in Table 4.6 and Fig 4.10.

Table 4. 6: Activity test result of the study area

Sample No	Location	Depth (*m*)	Color	Test pit ID	*PI*	Clay % < 0.002*mm*	Activity =*PI*/Clay % < 0.002*mm*	Degree of Activity
1	Sunshine	1.00	Black	*Tp1*	30.0	30.5	0.98	Normal clay
2	Sunshine	2.00	Grey	*Tp1*	47.2	36.2	1.30	Active clay
3	Babogaya	1.50	Black	*Tp2*	30.9	43.4	0.71	Inactive clay
4	Airforce	1.50	Black	*Tp3*	54.0	29.2	1.85	Active clay
5	Ude/Denkaka	1.50	Black	*Tp4*	38.5	40.8	0.94	Normal clay
6	Kurkura	1.00	Black	*Tp5*	23.8	37.5	0.63	Inactive clay
7	Kurkura	2.00	Grey	*Tp5*	53.7	39.7	1.35	Active clay
8	Near Dukem	1.50	Black	*Tp6*	52.2	53.5	0.98	Normal clay

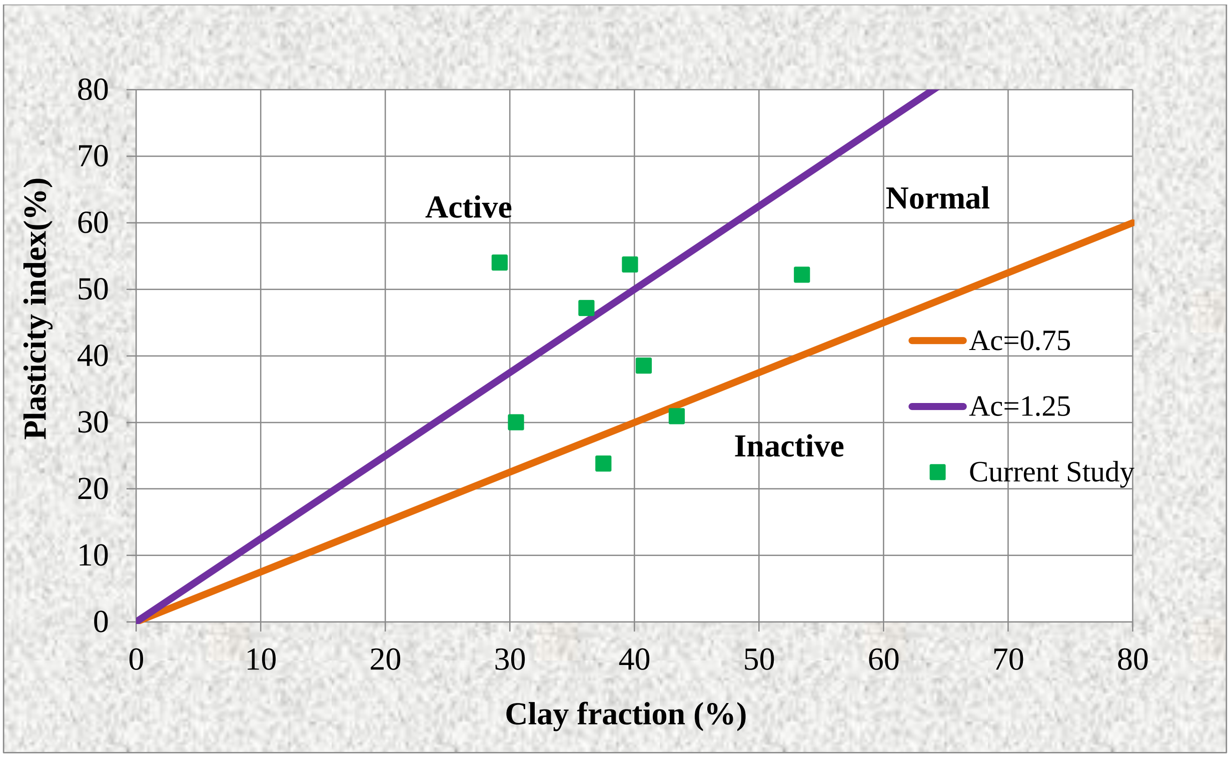


Figure 4. 10: Activity chart results of the study area

ii. Anderson

The swelling potential of the samples are calculated according to the empirical formula of Anderson, with equation 2.3.

Table 4. 7: Swelling potential and degree of expansion of the tested samples

Sample No	Location	Depth in (*m*)	Colour	Test pit ID	Plasticity Index (*PI*)	Swelling potential (S_p)	Degree of expansion
1	Sunshine	1.00	Black	*Tp1*	30.0	3.8	Medium
2	Sunshine	2.00	Grey	*Tp1*	47.2	7.7	Very High
3	Babogaya	1.50	Black	*Tp2*	30.9	4.0	High
4	Airforce	1.50	Black	*Tp3*	54.0	9.3	Very High
5	Ude/Denkaka	1.50	Black	*Tp4*	38.5	5.7	Very High
6	Kurkura	1.00	Black	*Tp5*	23.8	2.4	Medium
7	Kurkura	2.00	Grey	*Tp5*	53.7	9.2	Very High
8	Near Dukem	1.50	Black	*Tp6*	52.2	8.9	Very High

Table 4. 8: Geotechnical properties of the samples tested.

Test Pit ID	**Depth (*m*)**	**Free swell %**	**Linear shrinkage %**	***w* %**	***G_s***	**Grain Size Analysis**				**% Pass No 200 sieve**	**Atterberg Limit**			**Soil Classification**		**Compaction**		**Consolidation**		
						Gravel %	**Sand %**	**Silt %**	**Clay %**		***LL***	***PL***	***PI***	***AASHTO***	***USCS***	***OMC***	***MDD***	***Cv* m^2/*year***	***C_c***	***e_o***
Tp1	1.0	21.7	11.72	40.5	2.72	1.0	16.2	32.8	50	82.8	59.8	29.8	30.0	*A-7-5*	*CH*	33.9	1.41	3.049	0.389	1.49
Tp1	2.0	63.6	17.22	42.0	2.68	0.9	5.0	35.5	58.6	94.1	83.4	36.2	47.2	*A-7-5*	*CH*	32.7	1.22			
Tp2	1.5	59.1	12.92	29.8	2.69	0.2	6.0	24.4	69.4	93.8	61.6	30.7	30.9	*A-7-5*	*CH*	28.6	1.34	0.560	0.369	1.48
Tp3	1.5	81.8	15.46	26.7	2.67	0.5	4.0	39.7	55.8	95.5	89.0	35.0	54.0	*A-7-5*	*CH*	37.8	1.27			
Tp4	1.5	44.0	14.77	30.5	2.72	0.8	8.0	27.1	64.1	91.2	65.5	27.0	38.5	*A-7-6*	*CH*	30.0	1.38	0.688	0.472	1.37
Tp5	1.0	40.0	10.84	47.9	2.72	0.4	13.0	26.9	59.7	86.6	57.2	33.4	23.8	*A-7-5*	*MH*	32.7	1.42	2.816	0.555	1.41
Tp5	2.0	80.0	22.36	49.0	2.70	2.3	10.5	26.8	60.4	87.2	94.4	40.7	53.7	*A-7-5*	*CH*	37.5	1.27	2.665	0.526	1.68
Tp6	1.5	58.3	20.12	36.0	2.71	1.0	4.6	20.4	74	94.4	90.0	37.8	52.2	*A-7-5*	*CH*	30.5	1.31	0.334	0.487	1.52

4.4 Observation and damage analysis for the selected buildings

Hence; damage is analyzed and interpreted systematically with respect to type of foundation, wall, floor, pavement, door, window and fence. Detailed results are presented below.

Table 4. 9: Damage of buildings with respect to type of foundation system

Type of foundation	No. of visited buildings	No. of damaged foundations	Damaged foundations in %
Mat/Raft	0	0	0
Isolated footing	7	5	71
Combined footing	2	0	0
Masonry	22	14	64
Select Mat. plus shear wall beam	2	0	0
Wooden post with Masonry envelop (Mud house foundation)	11	9	82
Total	44	28	64

Among the foundation system mud house foundations constructed on expansive soil are highly susceptible to damage (82%) and decrease when we proceed to Isolated footing for G+1 and G+2 (71%) and masonry foundation (64%). Foundation with combined footing for G+2 and plinth beam foundation for G+0 HCB houses, which are recommended foundation type on expansive soil, are not damaged at all.

Table 4. 10: Damage of buildings with respect to wall type

Type of wall	No. of visited buildings	No. of damaged walls	Damaged walls in %
HCB	33	22	67
Masonry	0	0	0
Brick	0	0	0
Mud/Cheka	11	9	82
CIS	0	0	0
Total Building with wall	44	31	70

The extent of damage on Mud/Cheka house walls are (82%) and HCB wall are (67%) damaged. On Building ID No. 22 failure of one side of wall is observed.

Table 4. 11: Damage of buildings with respect to floor type

Type of floor	No. of visited buildings	No. of damaged floors	Damaged floors in %
Only concrete	4	2	50
Concrete + Screed	14	12	86
Concrete + Terrazzo	3	2	67
Concrete + Ceramic	20	12	60
Compacted soil	3	2	67
Total Building with floor	44	30	68

Floors with concrete plus screed are more affected as compared to the rest floor type, the main reason is all houses built with this type of floors and others do not have mesh on the ground floor/slab with reinforcement bar.

Table 4. 12: Damage of buildings with respect to pavement around the building

Type of pavement	No. of visited buildings	No. of damaged Pavements	Damaged pavements in %
Concrete + Terrazzo	10	6	60
Concrete	7	3	43
Stone	3	3	100
No pavement	24		
Total pavement around building	20	12	60
Total	44	12	27

Pavements around the building have a lion share in reducing moisture migration in the foundation soil and damages due to heaving. 24 buildings do not have pavement around the building, this is one of the main reasons why the damage summary is 82%.

Table 4. 13: Damage of buildings with respect to door/window

Type of door/window	No. of visited buildings	No. of damaged door & windows	Damaged door & windows in %
Metal	25	15	60
Metal + Wood	9	4	44
Wood	2	2	100
Aluminum	4	1	25
CIS	2	1	50
Not Installed	2		-
Total building with doors & windows	42	23	55
Total	44	23	52

Table 4. 14: Damage of buildings with respect to fence

Type of fences	No. of visited buildings	No. of damaged fences	Damaged fences in %
Masonry	1	1	100
HCB	7	4	57
Masonry + HCB	22	13	59
CIS	5	3	60
No fence	9		
Total building with fence	35	21	60
Total	44	21	48

Fences are liable to crack because of lightweight structures, around Ude/Denkaka area tilting of the fence wall almost at 5 degrees observed.

Table 4. 15: Extent of damaged buildings with respect to building elements

Building Element	No. of visited buildings	No. of damaged building elements	Damaged building elements in %
Foundation	44	28	64
Pavement around buildings	20	12	60
Floor	44	30	68
Walls	44	31	70
Door/Window	42	23	55
Fences	35	21	60

The extent of damage is very high on walls (70%) and decreases when proceed to floors, foundation, pavement around the buildings, fences and door/window in their respective orders. The degree of damage categorized according to (Zumrawi *et al*, 2017) by wall which is one of the building elements. In the investigated area 2 buildings with very severe crack of wall Near Dukem and Kurkura are observed, 13 buildings are severe, 17 buildings are moderate degree of damage and 4 buildings are categorized on slight degree of damage

Table 4. 16: Extent of damaged buildings with respect to building types

Type of Building	No. of visited buildings	No. of damaged buildings	Damaged building in %
Mud/Cheka house	11	10	91
G+0	24	21	88
G+1	3	2	67
G+2	6	3	50
Total	44	36	82

From the inspected buildings the extent of damage with respect to building type is very high on mud/cheka house (91%) and decreases when we proceed from G+0 hollow concrete block buildings (88%), G+1 buildings (67%) and G+2 buildings (50%) respectively. From this we can conclude that Mud/Cheka houses are highly susceptible to damage because they can't counteract the uplift pressure from the expansive soil.

Table 4. 17: Number of damaged buildings Summary

Type	No. of visited buildings	No. of damaged buildings	Damaged buildings in %
All Buildings	44	36	82

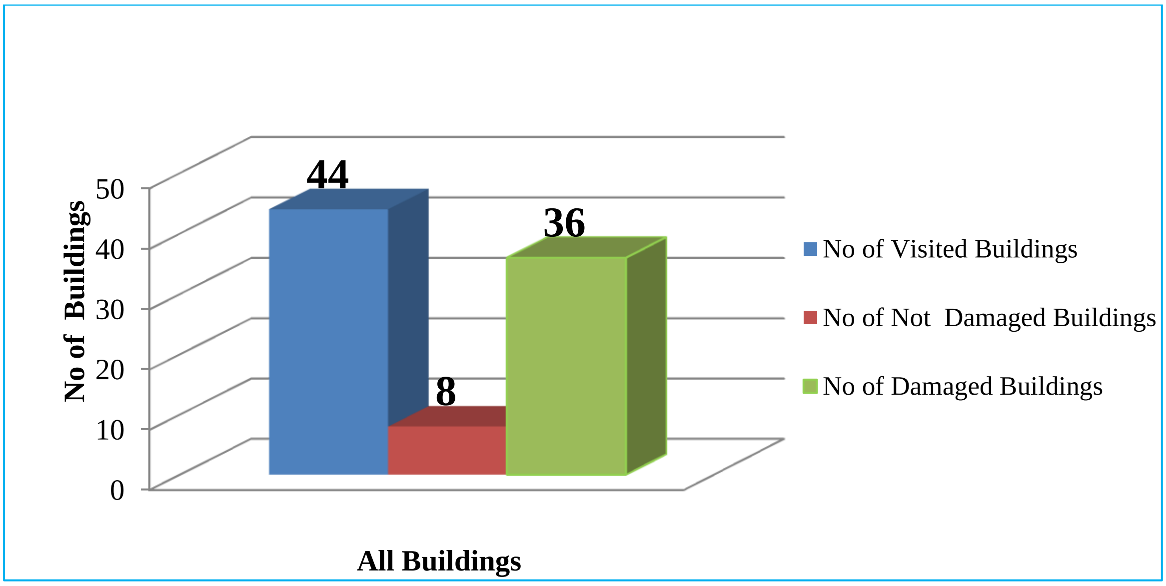


Figure 4. 11: Damage Summary

Table 4. 18: Investigated buildings with specific location and degree of damage.

Building ID No.	Building type	Easting (*E*)	Northing (*N*)	Elevation	Specific location	Degree of damage
1	Mud	493594	969102	1894	Sunshine	Moderate
2	G+0	493853	969522	1930	Sunshine	Moderate
3	G+0	493649	969287	1919	Sunshine	Moderate
4	G+0	493838	969508	1891	Sunshine	Slight
5	G+0	493721	969284	1909	Sunshine	Very Slight
6	G+1	493846	969504	1905	Sunshine	Very Slight
7	G+2	493758	969272	1914	Sunshine	Moderate
8	Mud	501290	966044	1902	Babogaya	Moderate
9	G+0	501150	966570	1876	Babogaya	Moderate
10	G+0	502542	967770	1869	Babogaya	Severe
11	G+0	502520	967963	1883	Babogaya	Moderate
12	G+0	499872	968075	1878	Babogaya	Moderate
13	G+2	499876	968074	1891	Babogaya	Very Slight
14	Mud	501023	962804	1882	Air Force	Moderate
15	Mud	501238	964001	1842	Air Force	Moderate
16	G+0	501264	962974	1855	Air Force	Moderate
17	G+0	500939	963888	1888	Air Force	Very Slight
18	G+0	500869	964029	1891	Air Force	Severe
19	G+2	502501	962193	1892	Air Force	Moderate
20	Mud	504676	959694	1852	Ude	Very Slight
21	Mud	504485	959520	1862	Ude	Severe
22	Mud	504570	959591	1860	Ude	Severe
23	G+0	506102	957214	1872	Ude	Severe
24	G+0	504562	959588	1858	Denkaka	Slight
25	G+0	504364	959500	1866	Denkaka	Severe
26	G+2	503078	961361	1918	Denkaka	Severe
27	Mud	492443	967935	1930	Kurkura	Very Severe
28	G+0	492515	968413	1902	Kurkura	Severe
29	G+0	492592	968500	1901	Kurkura	Moderate
30	G+0	492502	968383	1922	Kurkura	Moderate
31	G+0	492498	968323	1913	Kurkura	Severe
32	G+1	492431	968255	1915	Kurkura	Severe
33	G+2	492500	968212	1941	Kurkura	Very Slight
34	G+0	492101	970599	1882	Near Dukem	Severe
35	G+0	491984	970536	1912	Near Dukem	Very Slight

36	G+0	492025	970636	1907	Near Dukem	Very Severe
37	G+0	491990	970624	1927	Near Dukem	Severe
38	G+1	492106	970640	1913	Near Dukem	Moderate
39	G+2	492018	970710	1932	Near Dukem	Very Slight
40	Mud	497115	967134	1911	Kebele 05	Moderate
41	G+0	497118	967132	1911	Kebele 05	Moderate
42	Mud	494151	969258	1921	Danbii	Slight
43	Mud	494065	969253	1907	Danbii	Severe
44	G+0	495428	969009	1900	Danbii	Slight

Summary

Degree of damage	Crack width (mm)	No. of buildings
Very Slight	0.1 - 0.3	8
Slight	0.3 - 2	4
Moderate	2 - 5	17
Severe	5 - 25	13
Very severe	> 25	2
Total		**44**

Figure 4. 12: Damage of some inspected buildings with type of building and its elements.

4.5 Results for *2D* crack monitor fixed on the wall

Figure 4. 13: 2D crack monitor measuring across and along the crack Building ID No.26

Figure 4.13 shows the initial reading data taken on June 4, 2022 from Denkaka G+2 building (Building ID No. 26) at first floor horizontal crack.

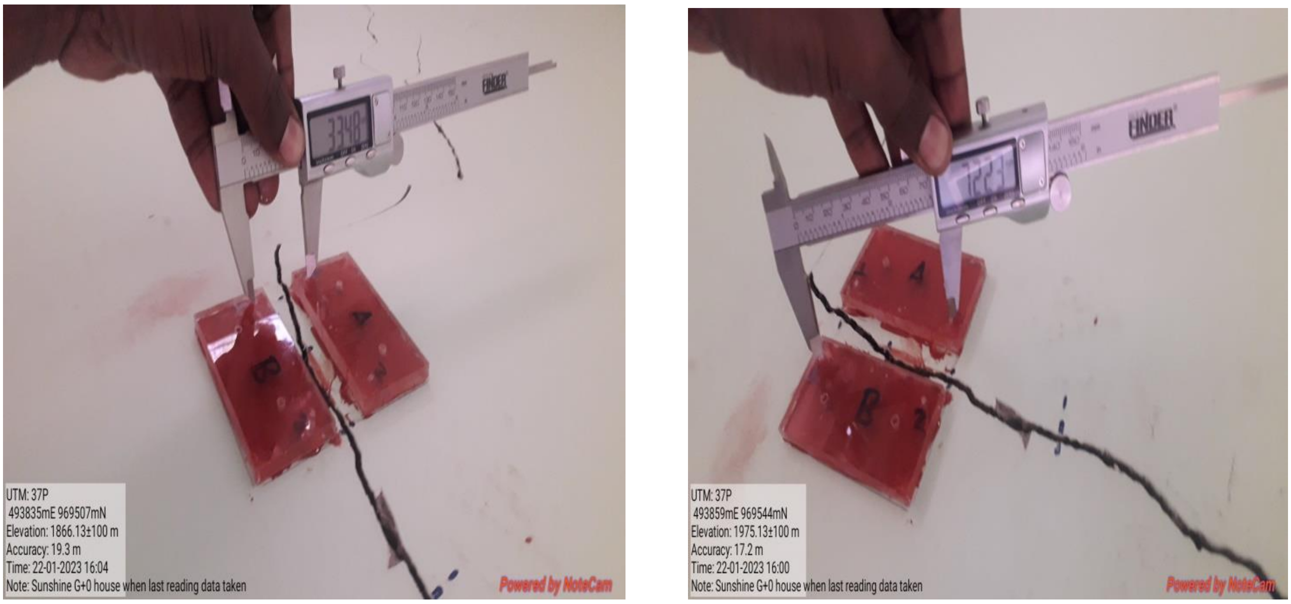


Figure 4. 14: 2D crack monitor measuring across and along the crack Building ID No. 2

Figure 4.14 shows the last reading data taken on January 22, 2023 from Sunshine G+0 house (Building ID No. 2) at horizontal crack, after 233 days. The rest results are given in Table and Figure in Appendix H.

Table 4. 19: Reading date versus Deformation Building ID No. 2 at horizontal crack.

Horizontal Crack	**Reading date**								
Across/Along the crack	**June 4,2022**	**June 19, 2022**	**July 10, 2022**	**July 24, 2022**	**August 7,2022**	**Sept. 3, 2022**	**Oct. 15, 2022**	**Nov. 20, 2022**	**Jan. 22, 2023**
Across the crack A_1B_1	33.18	33.13	33.01	32.97	32.89	32.91	33.03	33.32	33.48
Along the crack A_1B_2	72.17	72.23	72.18	72.24	72.36	72.01	72.09	71.88	72.23
Change in deformation(Δd)	***Δdo***	***Δd1***	***Δd2***	***Δd3***	***Δd4***	***Δd5***	***Δd6***	***Δd7***	***Δd8***
Across the crack A_1B_1	0	-0.05	-0.17	-0.21	-0.29	-0.27	-0.15	0.14	0.30
Along the crack A_1B_2	0	0.06	0.01	0.07	0.19	-0.16	-0.08	-0.29	0.06
Time (Date)	**0**	**15**	**36**	**50**	**64**	**92**	**134**	**170**	**233**

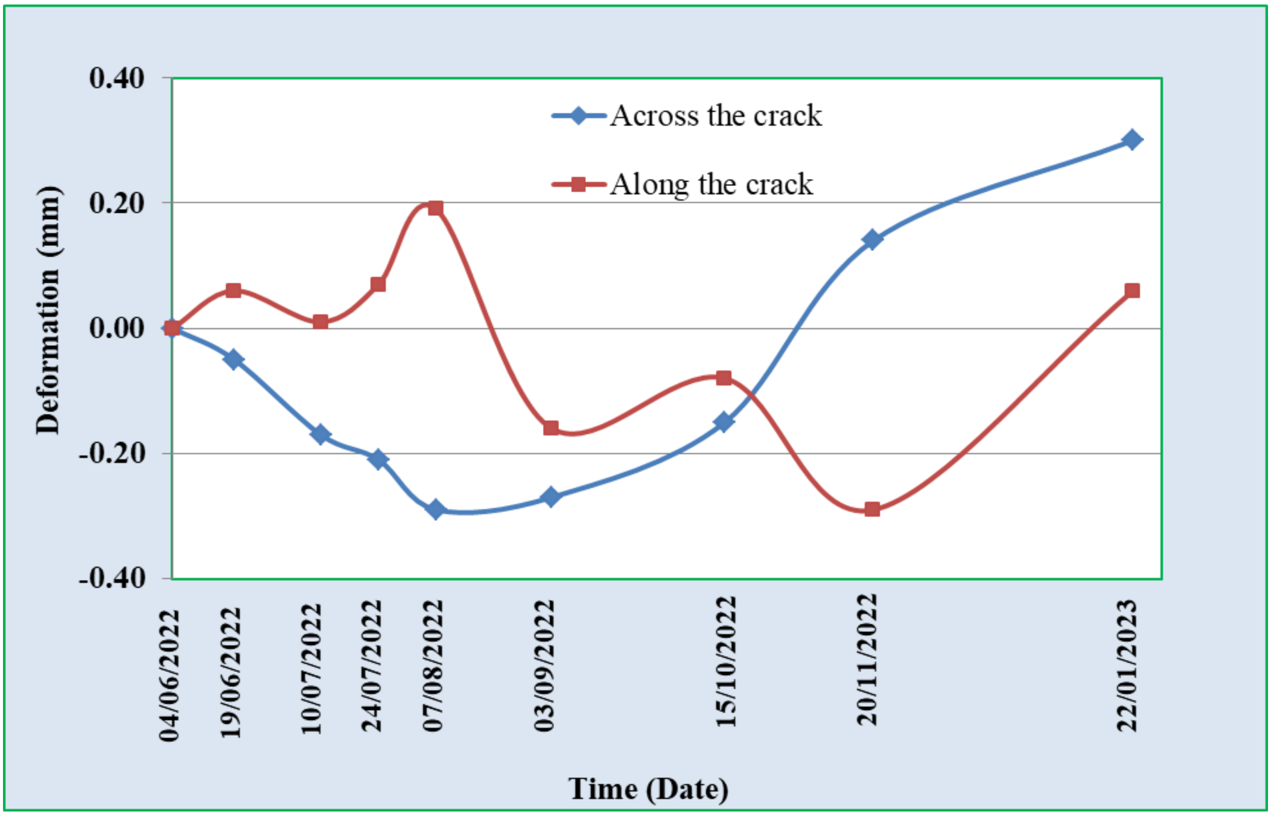


Figure 4. 15: Deformation versus Time Building ID No. 2 at horizontal crack

Observation from 2D crack monitor on Building ID No. 2, up to August 7, 2022 across the crack movement is decreasing (closing) and from August 7, 2022 which is the largest rainy month of the year, a 0.30mm additional crack opening was recorded up to January 22, 2023.

Table 4. 20: Summary of selected buildings with Instrumentation

Type of Building	**Denkaka G+2 Building**			**Kebele 5 Mud/Cheka house**		**Sunshine G+0 HCB house**
Building ID. No	26			40		2
Date of Installation	June 03,2022	June 03,2022	June 03,2022	June 03,2022	June 03,2022	June 03,2022
Type of Crack	Diagonal	Horizontal	Vertical	Vertical	Vertical	Horizontal
Location of Instrument	Ground floor	1st floor	1st floor	Door corner	Window corner	Near to the Door
Northing, *N*	961356	961328	961332	967137	967133	969527
Easting, *E*	503065	503076	503069	497110	497117	493850
Elevation	1901	1901	1901	1911	1911	1930
Degree of damage	Moderate	Moderate	Severe	Moderate	Moderate	Moderate

4.6 Discussion

4.6.1 Possible cause of failure and proposed remedial measures

Based on the study results and available literature, the main cause of damages or failures in buildings founded on expansive soil areas are attributed to some factors such as poor surface drainage, trees and vegetation nearby buildings, damaged utility lines, ground water table fluctuation, improper foundation design, construction with low quality material and climate changes, and related remedial measures are presented below.

i. Poor surface drainage

Improper drainage is probably the most important factor contributing to soil volume change and subsequent damage to buildings. If water is allowed to stand in drainage ditches close to buildings, it can penetrate down and amplify heave. The main causes of poor surface drainage can be considered include: surface runoff not properly drained away from the building; sprinkling of water for grass and shrub plantation; overflow from elevated or ground water tank; and slope of surrounding area.

Remedies:

- ✓ In order to minimize problems created by such conditions positive slope around the house and proper outlets from the compound should be provided.

ii. Trees and vegetation nearby buildings

In this research some buildings are damaged due to trees/vegetation planted close to buildings. House owners do not have awareness about such conditions. It is common to make a garden in the compound, but shrubs and grass around the building disturbs the moisture equilibrium of the soil due to seasonal moisture fluctuation and results in crack. Most house owners have gardens in front of their homes and in the periphery of their fence.

Remedies:

- ✓ Removing trees and other plants from the building area. Awareness creation has to be done by the responsible governmental office, construction offices, contractors and academic institutions like Adama Science and Technology University.
- ✓ Plantation of trees, plants and hedges within 3m distance around the building should be avoided. This is because excessive watering of plants close to the building contributes to swelling.
- ✓ Provide vertical moisture barrier

iii. Damaged utility lines/water pipes

Shallow water pipes buried in the zone of seasonal moisture fluctuation, are exposed to enormous stresses by shrinking soils. If water or sewage pipes break, then the resultant leaking moisture can aggravate swelling damage to nearby structures. The effect of a leaking water line is dependent on the soil moisture condition in the supporting expansive soil mass prior to the leaking occurrence.

Remedies:

- ✓ Maintain any leaking pipes around the building
- ✓ Provide adequate outlet of all downspouts
- ✓ The sewer pipes with leak joints close to the foundation should be placed beyond the foundation media.

iv. Ground water table fluctuation

The main cause of failures in the study area is fluctuation of ground water table. Foundations of most buildings rest on 2m depth, which is an active zone. These buildings are constructed without taking any measures in response to shrink/swell property of the expansive soil. Foundation distress caused by heave and shrinkage cycles due to ground water fluctuation is the main causes for most damaged buildings.

Remedies:

- ✓ Provide positive drainage around the building

- ✓ Provide concrete or stone masonry aprons around the building
- ✓ Provide proper sub drain around the building below the lower floor slab
- ✓ Rain water collected from the roof should be discharged at a distance of about 1m from the structure.

v. Improper foundation design/Faulty Design

Assessment of foundation design sheets and reports showed that there is no consideration for checking the safety of building against uplift pressures. This indicates that there is either a knowledge gap or carelessness in design offices and/or designers. The authority who is in charge of approval of these designs also doesn't demand such requirements. Sometimes in the investigated area house builders or contractors do not have design documents for buildings below two stories which are vulnerable for damages due to their lightweight structures.

Remedies:

- ✓ The plinth beam should be separated from the natural ground by leaving an air gap of 8 to 10cm between the plinth beam bottom and natural ground. If the gap is not provided the plinth has to be designed for upward pressure due to swelling.
- ✓ Septic tanks should be provided far from the foundation region.
- ✓ Flooring shall be redone after removing existing filled up soil to about 1.5m from the floor level and replacing the same with well compacted non-expansive materials placed in layers not exceeding 30cm thickness.
- ✓ Sub excavating or replacing the expansive soil by cushions either in part or full with a material that does not undergo swell is essential because the load of the cushion provides the load necessary to counter heave.
- ✓ Construction of additional one or two floors above the existing building should be done so that the loading on the foundation would be more than the existing swelling pressure.
- ✓ Problems created by expansive soil heave can be properly addressed by considering the situation during the design phase and providing detailed drawings for house builders.
- ✓ Inserting helical bar along and across the crack, regulate and reduces crack movement.

vi. Construction with low quality materials

Using low quality materials for construction adversely affects the performance of the building. This sometimes occurs in the form of improper concrete mixture ratio (cement, sand and aggregate). Low quality materials for building may accelerate deterioration of the building and often result in cracking, low strength, shortened service life, or some combination of these problems. In the study area not only observed low quality material but also some house builders need sticks or steel in order to fix concrete into colon or beam instead of using mechanical vibration.

Remedies:

- ✓ Manufacturing companies especially cement and steel, should provide standards and quality materials.
- ✓ House owners must build their houses with good material not to regret letter and spend additional money for maintenance.
- ✓ Governmental institutions like the Construction office should regulate and control the quality of construction materials.

vii. Climate changes

In Bishoftu town most of the time the annual rainfall starts in the month of June and ends up to September, on average three up to four months are rainy months this means that full summer is a rainy season. The study results and observations indicated that expansive soils which experienced periodic swelling and shrinkage during alternate wet and dry seasons caused considerable damage to structures found on them. The damage to structures built on expansive soil in wet climates usually occurred during drought periods and damage to structures built in dry climates occurred during the rainy season.

Remedies:

- ✓ Construct foundation during dry season, where there is low moisture migration and when the expansive soil shrinks to its maximum level.

4.6.2 Proposed design and construction methodologies

In this research most buildings are G+0 buildings constructed from masonry foundation, HCB walls and Concrete floors. But the investigated area buildings resting on masonry foundation are liable to crack. There are also houses built with mud walls, some of these houses were constructed more than 10 years ago. Currently Bishoftu City Administration prohibits construction of mud houses. Therefore, there is no need to propose a proper foundation type for this kind of house. By considering design procedures followed in the study area, type of foundation used and amount of dead load imposed on foundation soil, design alternatives and detailed drawings (for single story buildings) have been presented below by dividing the building types into four categories.

i. Recommended foundation on expansive soil for single story (G+0) buildings

In the study area house owners and peoples involved in house construction came to know that expansive soil creates problems on buildings directly constructed on it. Some contractors replace the expansive soil by non-expansive soil at a depth of 70cm-100cm under masonry wall. This provision reduces the type and extent of damage as compared to buildings directly constructed on expansive soil.

The following cases are the major source of damage in the current practice;

- ✓ Partition walls are usually constructed directly on floor slabs.
- ✓ Only a few centimeters of soil under the floor slab has been replaced or not replaced at all.
- ✓ Void spaces are not provided between NGL and grade beam.
- ✓ External and internal walls are interlocked to each other at corners unless there is a column between them

Due to this the uplift force from expansive soil beneath the floor slab is transferred from the floor to grade beam, partition walls and external walls. Soil replacement under floor slab up to certain depth, allow the floor slab to float freely and let walls to act independently then the extent of damage will be minimized or eliminated.

Nowaday's most people are aware about how to build a new house on black cotton soil. In Bishoftu Town where construction status is dominated like around Kurkura, Sunshine, Near Dukem and Ude/Denkaka providing shear wall beam or plinth beam foundation around the perimeter of the building are preferable, which is recommended foundation for new G+0 buildings on expansive soil. Trench excavation work is done 1.0 - 1.2m depth below natural ground level and 50cm width. Void space/Air gap is provided by lean concrete with a thickness of 80-100mm between the plinth beam and the NGL. The non-expansive soil should be compacted in layers not exceeding 30cm thickness. The plinth beam is elevated 1m above natural ground level. During investigation, houses built with this mechanism do not show any damage to the structural element. Ground floor should be mesh by reinforcement bar $Ø_8$ 0r $Ø_{10}$ and concrete which reduces problems that come from heave condition and shrink/swell behavior of expansive soil.

Figure 4. 16: Plinth beam foundation for G+0 buildings

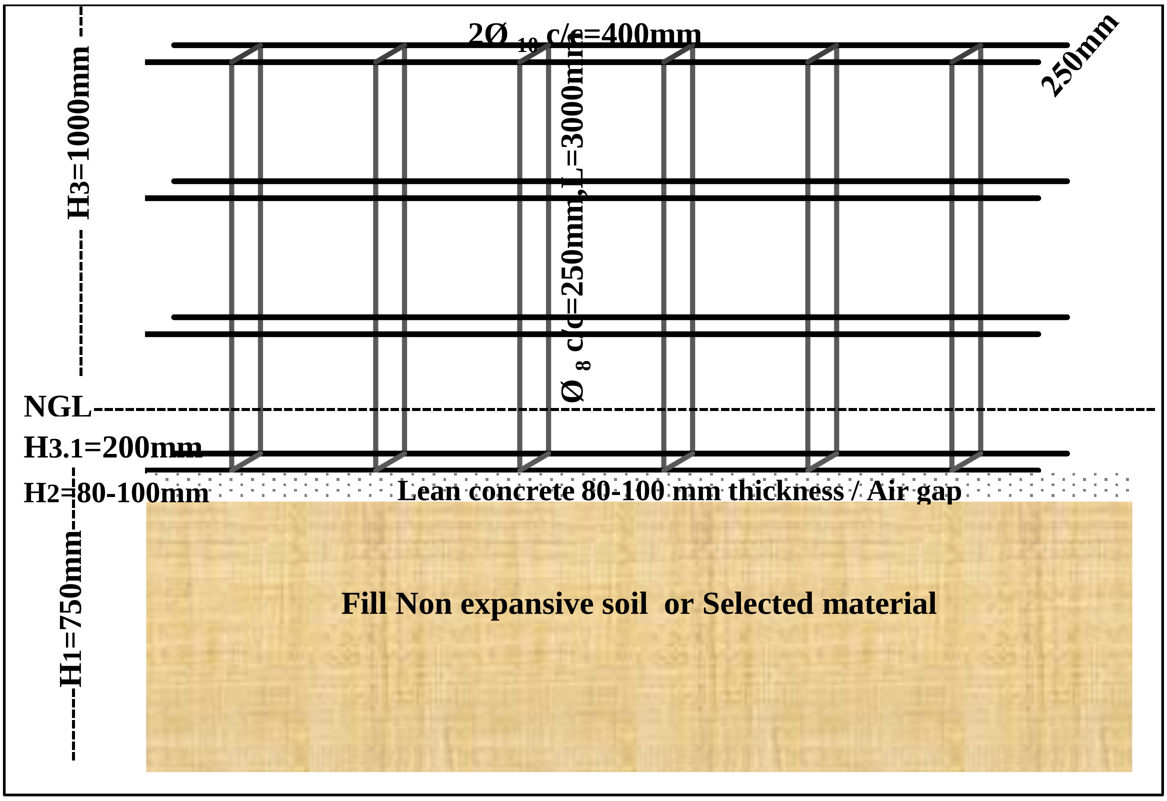


Figure 4. 17: Recommended foundation and detail drawings for G+0 buildings

ii. Recommended foundation on expansive soil for double story (G+1) buildings

These types of buildings don't have sufficient load to counterbalance the uplift pressure induced by expansive soil. Hence, in order to minimize the damage:-

- ✓ The foundation system should be isolated from the swelling effect of the soil,
- ✓ The foundation system should be rigid enough to avoid differential movement.
- ✓ Swelling potential of the foundation soil should be eliminated.

For two story (G+1) buildings it may be residential or office building and the degree of expansiveness of the soil ranges from moderate to very high then 1.5mx1.5mx1.5m width, length and depth respectively, isolated footing foundation with full replacement of the expansive soil by cushion is recommended type of foundation. Need C_{25} grade of concrete.

iii. Recommended foundation on expansive soil for three story (G+2) buildings

Isolated footing: For three story (G+2) buildings it may be residential or office building and the degree of expansiveness of the soil ranges from moderate to very high, then 2mx2mx2m width, length and depth respectively, isolated footing foundation with full replacement of the expansive soil by cushion is recommended. Need C_{25} grade of concrete.

Combined footing: In the study area some contractors built a foundation for a three story and above building by combined footing, it is preferable for two or four columns close to each other and also this type of footings are essential under stair case.

iii. Recommended foundation on expansive soil for multistory buildings

Tessema (1984) proposed to use under-reamed pile foundation with suspended floor slab and grade beam. Isolated footings and stiffened mat can also be used.

Isolated footing: In the study area the common foundation type for double story and higher buildings is isolated footing. These footings are placed between 2m to 3m depth below NGL. For these types of buildings damages are usually observed on ground floor slab, walls constructed directly on ground floor slab, pavement around the buildings and masonry under grade beam.

Rigid mat foundation: The other type of foundation practiced in Ethiopia is rigid mat. A heavily reinforced rigid mat is placed on leveled soil. The walls and columns are then built on the foundation. The rationale in this design is that the rigid mat will eliminate any differential heave and the structure will "ride on" without suffering any internal distress (Tefera, 2008).

4.6.3 Discussion with laboratory test results

According to the classification system stated in chapter 2, the soils are classified as follows:-

- ✓ According to the AASHTO classification system, except sample 5 all samples are grouped in A-7-5. But sample 5 is grouped in A-7-6.
- ✓ Based on the USCS, except sample 6 all samples are grouped in CH (Fat clay) soil. In this classification CH or OH groups have high volume change and sample 6 is grouped in MH.
- ✓ Seed et al, use plasticity index as a tool to indicate the swelling characteristics of a soil. Based on this classification all samples have high Swell potential.
- ✓ Altmeyer uses linear shrinkage for determination of degree of expansion. All samples have greater than 8% of linear shrinkage value, categorized under critical degree of expansion.
- ✓ Based on Chen method of soil classification, the samples have a high to very high degree of expansion.
- ✓ Rao, indirectly estimation of the swell potential of clay soil from the differential free swell test and the samples have moderate to very high degree of expansiveness.

The Coefficient of consolidation (C_v) of the investigated area soil ranges from (0.334 – 3.049) m^2/year.

Scanning Electron Microscope (SEM) results of sample 2 and sample 5; Sample 2 has less particle size of the rest sample under the same condition of 1000 times magnification, but more strip and sheet structures. In sample 5 aggregate structures are main components but the sheet or plate structures are few. Sample 2 has maximum liquid limit, plasticity index, free swell and linear shrinkage value 83.4%, 47.2%, 63.6% and 17.22% respectively as compared to sample 5 which have 65.5%, 38.5%, 44% and 14.77% value respectively.

4.6.4 Discussion with building damage and other structures

Among the foundation system mud house foundations constructed on expansive soil are highly susceptible to damage (82%) and decrease when we proceed to Isolated footing for G+1 and G+2 (71%) and masonry foundation (64%). Foundation with combined footing for G+2 and plinth beam foundation for G+0 HCB houses, which are recommended foundation type on expansive soil, are not damaged at all.

The degree of damage categorized according to (Zumrawi et al, 2017) by wall which is one of the building elements. The extent of damage on Mud/Cheka house walls are (82%) and HCB wall are (67%) damaged. In the investigated area 2 buildings with very severe crack of wall Near Dukem and Kurkura are observed, 13 buildings are severe, 17 buildings are moderate degree of damage and 4 buildings are categorized on slight degree of damage. Digital Vernier Calipers are used to measure the width of the crack.

Floors with concrete plus screed are more affected as compared to the rest floor type, the main reason is all houses built with this type of floors and others do not have mesh on the ground floor/slab with reinforcement bar.

Pavements around the building have a lion share in reducing moisture migration in the foundation soil and damages due to heaving. According to this research 24 buildings do not have pavement around the building, this is one of the main reasons why the damage summary is 82%.

Fences are liable to crack because of lightweight structures, around Ude/Denkaka area tilting of the fence wall almost at 5 degrees observed.

In this study more emphasis is given to light weight buildings founded on expansive soil. These structures are vulnerable to damages because they can't exert a heavy pressure to counteract the uplift pressure from the expansive soils. According to the inspected buildings the extent of damage with respect to building type is very high on mud/cheka house (91%) and decreases when we precede from G+0 hollow concrete block buildings (88%), G+1 buildings (67%) and G+2 buildings (50%) respectively. From this we can conclude that Mud/Cheka houses are highly susceptible to damage because they can't counteract the uplift pressure from the expansive soil. Severe damage of houses is observed around Kurkura, Ude/Denkaka, Sunshine and Near Dukem town where construction status is dominated.

In the investigated area expansive soil is not only damage buildings but also structures like:

Road: The old main road from Addis to Diredawa around Ude/Denkaka currently rehabilitation work is done as remedies. Different defects in this main road are observed; rutting, crocodile cracking and potholes around Air Force, Ude/Denkaka and Near Dukem also. Rutting is caused by lack of compaction, insufficient pavement thickness, weak asphalt mixtures and lateral movement of the materials due to traffic loading. Pothole is a hole in the road surface that results in gradual damage caused by traffic loading or bad weather.

Very light weight structure, wooden electric pole: Tilt at 45^0, sometimes completely falling down of wooden electric pole is observed around Ude/Denkaka, Air Force and Babogaya where black cotton soil is dominated. These wooden electric poles are not only affected by giving long service but also due to their lightweight structures, uplift pressure has its own factor. Now in this mentioned area the Ethiopian Electric Power Utility has replaced wooden electric poles into concrete poles as a remedy. Conduct circular excavation work with a diameter of 60 cm and 200 cm depth, then after inserting the concrete pole into the hole, backfill is done by concrete.

4.6.5 Discussion with instrumentation

Instruments play a vital role during or after construction. In crack monitoring, as the word used is monitoring and not measuring; so the purpose and methods are not to measure the absolute width, length and depth of cracks, it measures only deformation with respect to time is observed. If there is no deformation with time then nothing to worry about the cracks, they can be leveled and closed with cement mortar or some other material. The methods do not provide the absolute movements of the walls on both sides of the crack but their relative movements only with respect to each other.

On a few selected buildings where crack is observed on the wall the 2D crack monitor is installed. The results observed from the instrument are for 233 days (nearly 8 months), across the crack (width of crack) movement on Building ID No. 2 and Building ID No. 40 are closing when measured from time to time up to August 7, 2022, this is due to seasonal changes. Since the 2D crack monitor is installed during the wet or summer season. By increasing the number of days for further or detailed investigation it should be necessary what looks like the relative deformation with time during the rest of the Ethiopian season.

From instrumentation the maximum crack opening observed is 1.29mm on Building ID No. 26 at 1st floor vertical crack on November 20, 2022; at ground floor diagonal crack 1.05mm and at 1st floor horizontal crack 0.44mm additional opening of crack is observed on January 22, 2023 from the initial crack width reading data taken from June 4, 2022. The maximum crack closing observed on Building ID No. 40 at the vertical crack window corner is -0.37mm and decreases at door corner -0.23mm on September 3, 2022. Building ID No. 2 at horizontal crack -0.29mm closing of crack is observed on August 7, 2022 and 0.30mm additional crack opening is observed on January 22, 2023. The negative sign indicates closing of cracks.

The width of the crack is not only opening but also sometimes closing depending on weather conditions and types of structure. This opening and closing of cracks for a long time may dangerous to the structure. If the crack is stable there is not dangerous and also not opening and closing of crack is also not dangerous.

In Bishoftu town there are no barrages, tunnels and big costly structures, the purpose of 2D crack monitor is for monitoring buildings, light structures like mud/cheka houses and G+0 buildings when some building elements damage it can be simply maintained. In Ethiopia it is very necessary for multistory buildings and costly structures if the 2D and 3D crack monitor is applicable, it is easy to determine problems arising from structural or non-structural crack and also to investigate what type of soil the structure rests on. This method gives in built check on any human or instrumental error. Crack monitoring is essential for costly projects.

4.6.6 Discussion with type of crack

From the inspected buildings two types of crack are observed, non-structural and structural crack. The causes of non-structural crack are moisture changes, vegetation, thermal movement, foundation movement and settlement of soil. Improper or incorrect design, faulty construction, overloading, differential settlements (due to poor soil bearing capacity) are the cause of structural crack. During the investigation vertical crack, horizontal crack and diagonal crack were observed, where the degree of damage ranges from slight to very severe crack depending on the wall which is one of the building elements.

Horizontal cracks are the most serious type of foundation cracks and are always an indicator of structural foundation damage. Horizontal cracks are much more dangerous than vertical cracks because they can quickly lead to total foundation failure. Most of the time horizontal crack is observed between the grade beam or top beam and wall. On Building ID No. 34, settlement of foundation masonry, severe horizontal crack is observed. Continuous horizontal cracks which appear on walls indicate signs of displacement.

Diagonal cracks this type of foundation crack runs from about 30 to 75 degrees. Many times, diagonal cracks will be wider at one end than the other. Diagonal cracks are usually caused by differential settling of the foundation and can occur when the house is new or old. Differential settling may be caused by a change in the ground conditions under the foundation footings and also due to gutter downspouts drain too close to the house foundation on one side or at one corner. Very severe and severe diagonal crack is observed on Building ID No.36 and Building ID No. 43.

Vertical crack this type of foundation crack goes straight up and down or slightly diagonal within 30 degrees. Building ID No. 27 is where a very severe vertical crack is observed. In the investigated area sometimes, vertical crack is also observed at the corner, between colon and wall.

CONCLUSION AND RECOMMENDATION

Conclusion

From the results of the investigation the following conclusions are drawn:

1. Out of randomly visited 44 buildings constructed on expansive soil areas 36 or 82% of them are damaged due to swelling and shrinkage cycle of the soil. The damage has been analyzed and interpreted systematically with respect to type of foundation, wall, floor, pavement, door, window and fence.
2. Structures founded on masonry wall foundations resting on an expansive soil are liable to crack.
3. Providing a proper drainage system plays an important role in reducing damage due to heaving.
4. The extent of damage is very high on walls and decreases when we proceed to floors, foundation, pavements around the building, fences and door/window in their respective orders.
5. According to the inspected buildings the extent of damage with respect to building type is very high on mud/cheka houses and decreases when we precede from G+0 hollow concrete block buildings, G+1 building and G+2 buildings respectively. Mud houses are highly susceptible to damage because they can't counteract the uplift pressure from the expansive soil.
6. The study showed that the main causes of failures are improper foundation design, moisture fluctuation, poor surface drainage, presence of trees and vegetation close to buildings, damaged utility lines also have great contributions.
7. Depending on shrink/swell behavior of expansive soil, the linear shrinkage of all the tested samples are greater than 8%, these are categorized under critical degree of expansion and, the swelling potential and free swell results of all samples at 1.5m and 2m depth are categorized under very high degree of expansion.
8. Based on a different soil classification system stated in chapter two, the study area soil type is fat clay soil, this soil has high swelling potential. Therefore, care should be taken to design lightweight structures in this area.
9. From instrumentation (2D crack monitor) not only opening but also closing of crack is observed depending on seasonal changes and shrink/swell behavior of expansive soil.

Recommendation

1. The main cause of failure in the study area is moisture fluctuation and improper foundation design. Therefore, the following measures should be taken:
   i. For Existing buildings: - pavement around the building should be covered by providing concrete or stone masonry aprons, which reduces the seasonal moisture fluctuation.
   ii. For New buildings: If the uplift induced by the expansive soil which couldn't be counterbalanced by dead load of the structure then the soil below the foundation level shall be replaced by non-expansive material, this also increases the bearing capacity of soil, and provide plinth beam foundation around the perimeter of the building is recommended.
2. Remedies like provide positive drainage around the building, remove and re-compact non expansive material and provide concrete aprons around the building, cut trees and plants around the building and maintenance and regular inspection for utility lines should be implemented.
3. Install 2-dimensional crack monitor on existing buildings where crack is observed which is used as a precautionary measure. In a short time or days it is difficult to know the absolute width of crack, across the crack and along the crack movement on buildings, by increasing the number of days and comparing and contrasting with the rest season.
4. In this research work only eight samples of soil were collected from six test pits, it is recommended for further investigation on the engineering property of expansive soils of Bishoftu town in the future research programs with more samples.
5. A cushion of freely draining granular soil, such as gravel, sand or a mixture of gravel or sand is placed in the trench up to the base level of the footing and compacted, which acts as a barrier and the effect of swelling on the foundation is reduced.
6. In the investigated area it was observed that some buildings are settled, so it is mandatory to investigate the consolidation and settlement characteristics of these soils.
7. The result obtained may be used as a basis for further research in the area of expansive soil.

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APPENDICES

Appendix A: Natural Moisture Content Test Results

Parameters and formulas used

Mass of water= (mass of can + wet soil)-(mass of can + dry soil)

Mass of dry soil= (mass of can +dry soil)-(mass of can)

Water content, % = Mass of water*100/ Mass of dry soil

Average water content, % = (∑ Water content, %) / total number of trials

Table A.1: Determination of Water Content for test pit 1

	Test pit 1					
Depth (*m*)	1*m*			2*m*		
Can No	M_1	M_2	M_3	N_1	N_2	N_3
Mass of can $W_1(g)$	30.52	30.14	30.38	19.53	19.42	19.61
Mass of can + wet soil $W_2(g)$	62.47	62.56	62.13	50.20	51.70	53.20
Mass of can + dry soil $W_3(g)$	53.25	53.12	53.06	41.25	42.05	43.23
Mass of water $Ww(g) = W_2 - W_3$	9.22	9.44	9.07	8.95	9.65	9.97
Mass of dry soil $Ws(g) = W_3 - W_1$	22.73	22.98	22.68	21.72	22.63	23.62
Moisture Content w%	40.56	41.08	39.99	41.21	42.64	42.21
Average Moisture Content *w*,%	**40.54**			**42.02**		

Table A.2: Determination of Water Content for test pit 2 & 3

	Test pit 2			**Test pit 3**		
Depth (*m*)	1.5*m*			1.5*m*		
Can No	DU_1	DU_2	DU_3	AF_1	AF_2	AF_3
Mass of can $W_1(g)$	29.85	30.35	30.07	19.47	19.25	19.27
Mass of can + wet soil $W_2(g)$	62.32	64.26	65.12	51.14	52.64	54.14
Mass of can + dry soil $W_3(g)$	54.88	56.38	57.14	44.26	45.66	46.94
Mass of water $Ww(g) = W_2 - W_3$	7.44	7.88	7.98	6.88	6.98	7.2
Mass of dry soil $Ws(g) = W_3 - W_1$	25.03	26.03	27.07	24.79	26.41	27.7
Moisture Content w%	29.72	30.27	29.48	27.75	26.43	26.02
Average Moisture Content *w*,%	**29.83**			**26.73**		

Table A.3: Determination of Water Content for test pit 5

	Test pit 5					
Depth (*m*)	1*m*			2*m*		
Can No	K_1	K_2	K_3	K_4	K_5	K_6
Mass of can $W_1(g)$	19.30	19.58	19.53	19.70	19.46	19.60
Mass of can + wet soil $W_2(g)$	50.14	49.24	49.06	52.61	51.22	51.28
Mass of can + dry soil $W_3(g)$	40.13	39.63	39.51	41.67	40.79	40.95
Mass of water $Ww(g) = W_2 - W_3$	10.01	9.61	9.55	10.94	10.43	10.33
Mass of dry soil $Ws(g) = W_3 - W_1$	20.83	20.05	19.98	21.97	21.33	21.35
Moisture Content w%	48.06	47.93	47.80	49.80	48.90	48.38
Average Moisture Content *w*,%	**47.93**			**49.03**		

Table A.4: Determination of Water Content for test pit 4 & 6

	Test pit 4			Test pit 6		
Depth (*m*)	1.5*m*			1.5*m*		
Can No	U_1	U_2	U_3	ND_1	ND_2	ND_3
Mass of can $W_1(g)$	19.44	19.30	19.48	19.24	19.49	19.59
Mass of can + wet soil $W_2(g)$	50.62	52.34	51.76	51.42	52.86	50.73
Mass of can + dry soil $W_3(g)$	43.52	44.46	44.18	42.97	44	42.46
Mass of water $Ww(g) = W_2 - W_3$	7.1	7.88	7.58	8.45	8.86	8.27
Mass of dry soil $Ws(g) = W_3 - W_1$	24.08	25.26	24.7	23.73	24.51	22.87
Moisture Content w%	29.49	31.32	30.69	35.61	36.15	36.16
Average Moisture Content *w*,%	**30.50**			**35.97**		

Appendix B: Atterberg Limit Test Results

Representatives for Atterberg limit test results

Table B.1: Determination of Liquid Limit, Plastic Limit and Plasticity Index for test pit 1 @ 1*m*

Test pit 1 @ 1*m* depth	Liquid Limit (*LL*)			Plastic Limit (*PL*)		
Trial No	1	2	3	1	2	3
Can No	A_1	A_2	A_3	B_1	B_2	B_3
Mass of wet soil + can, *A*(*g*)	42.15	42.53	43.71	41.09	41.14	40.91
Mass of dry soil + can, *B*(*g*)	37.96	37.89	38.43	38.79	38.69	38.26
Mass of water, *C* = (*A* – *B*)	4.19	4.64	5.28	2.30	2.45	2.65
Mass of can, *D*(*g*)	30.35	30.21	30.16	30.51	30.37	29.99
Mass of dry soil *E*= (*B* -*D*), (*g*)	7.61	7.68	8.27	8.28	8.32	8.27
Water content % (*C* / *E* x 100)	55.06	60.42	63.85	27.78	29.45	32.04
Number of blows	32	24	19			
Result, %	**59.8**			**29.8**		
Plasticity Index (*PI*) = *LL* - *PL*	**30**					

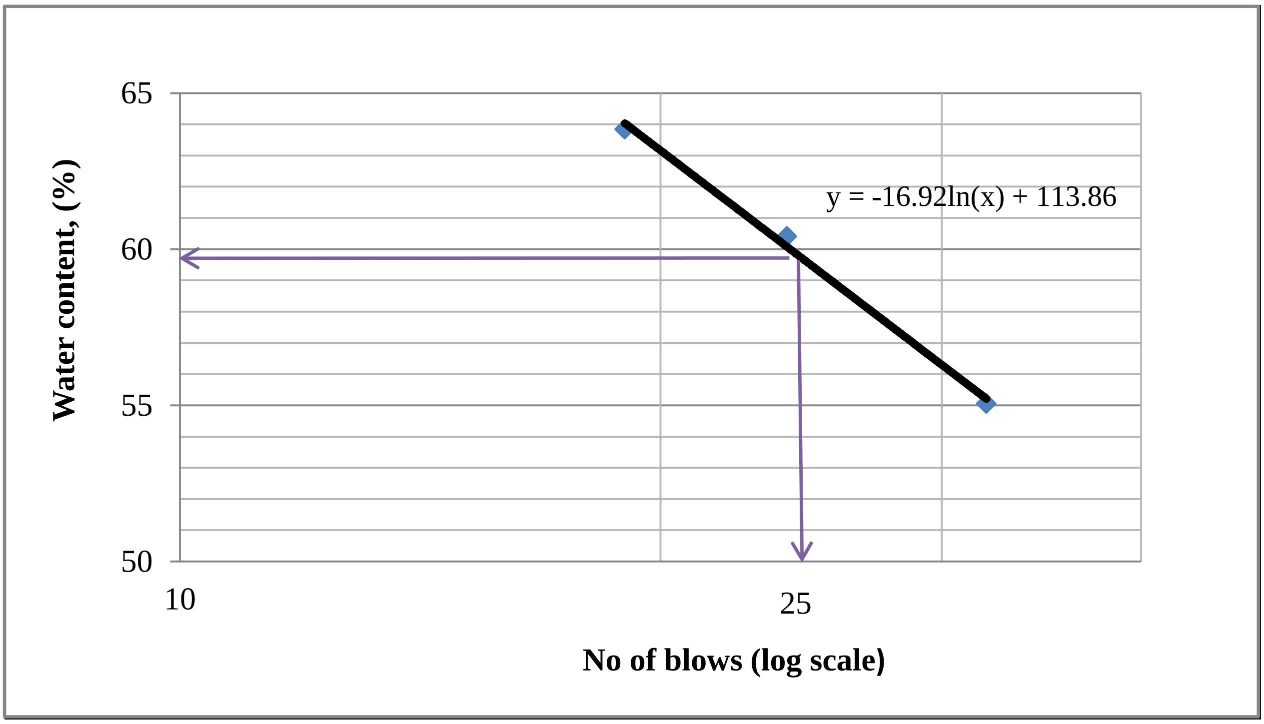

Figure B.1: Water content versus log of number of blows for test pit 1@ 1*m*

Table B.2: Determination of Liquid Limit, Plastic Limit and Plasticity Index for test pit 2 @ 1.5*m*

Test pit 2 @ 1.5*m* depth	Liquid Limit (*LL*)			Plastic Limit (*PL*)		
Trial No	1	2	3	1	2	3
Can No	C_1	C_2	C_3	D_1	D_2	D_3
Mass of wet soil + can, *A*(*g*)	33.30	32.92	32.45	29.38	30.09	28.58
Mass of dry soil + can, *B*(*g*)	28.21	27.88	27.32	27.15	27.74	26.28
Mass of water, *C* = (*A* – *B*)	5.09	5.04	5.13	2.23	2.35	2.30
Mass of can, *D*(*g*)	19.48	19.69	19.44	19.73	19.69	19.29
Mass of dry soil *E*= (*B* -*D*), (*g*)	8.73	8.19	7.88	7.42	8.05	6.99
Water content % (*C* / *E x* 100)	58.30	61.54	65.1	30.05	29.19	32.90
Number of blows	35	24	15			
Result, %	**61.6**			**30.7**		
Plasticity Index (*PI*) = *LL* - *PL*	**30.9**					

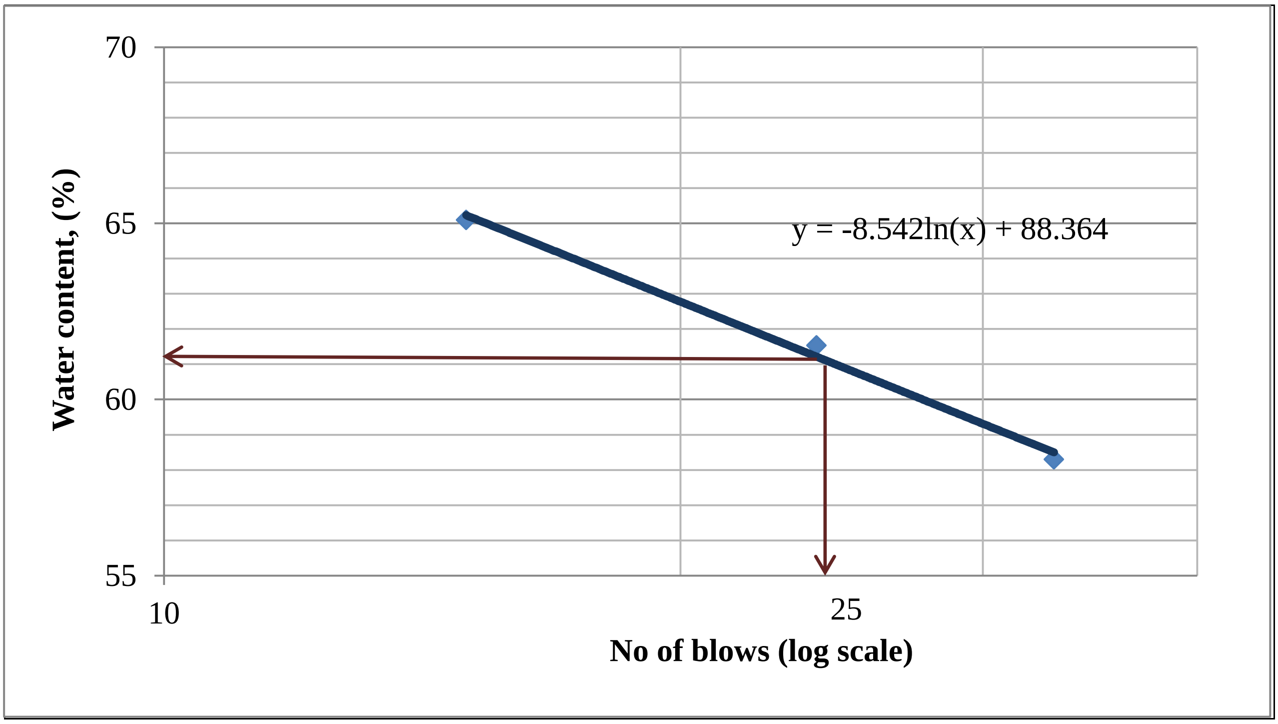

Figure B.2: Water content versus log of number of blows for test pit 2 @ 1.5*m*

Table B.3: Determination of Liquid Limit, Plastic Limit and Plasticity Index for test pit 4 @1.5*m*

Test pit 4 @ 1.5*m* depth	Liquid Limit (*LL*)			Plastic Limit (*PL*)		
Trial No	1	2	3	1	2	3
Can No	C_1	C_2	C_3	D_1	D_2	D_3
Mass of wet soil + can, *A*(*g*)	33.42	35.22	34.34	42.18	44.32	43.41
Mass of dry soil + can, *B*(*g*)	28.16	28.86	28.3	39.73	41.44	40.53
Mass of water, $C = (A - B)$	5.26	6.36	6.04	2.45	2.88	2.88
Mass of can, *D*(*g*)	19.46	19.22	19.68	30.52	30.63	30.28
Mass of dry soil $E = (B - D)$, (*g*)	8.7	9.64	8.62	9.21	10.81	10.25
Water content % ($C / E \times 100$)	60.46	65.98	70.07	26.60	26.64	28.10
Number of blows	32	23	18			
Result, %	**65.5**			**27**		
Plasticity Index (*PI*) = *LL* - *PL*	**38.5**					

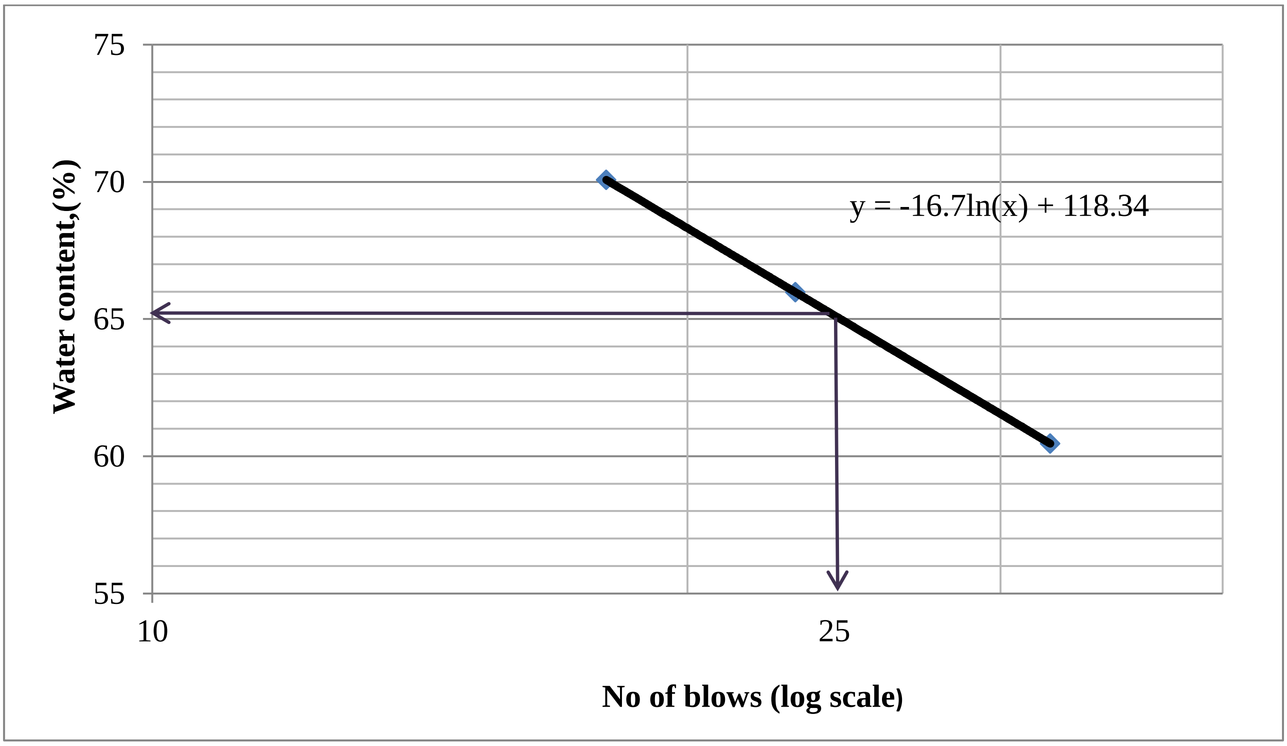

Figure B.3: Water content versus log of number of blows for test pit 4 @ 1.5*m*

Table B.4: Determination of Liquid Limit, Plastic Limit and Plasticity Index for test pit 5 @ 1*m*

Test pit 5 @ 1*m* depth	Liquid Limit (*LL*)			Plastic Limit (*PL*)		
Trial No	1	2	3	1	2	3
Can No	*P*	*Q*	*R*	*S*	*T*	*U*
Mass of wet soil + can, *A*(*g*)	40.55	41.13	40.32	42.38	43.80	40.40
Mass of dry soil + can, *B*(*g*)	36.77	37.14	36.68	39.22	40.49	37.74
Mass of water, *C* = (*A* – *B*)	3.78	3.99	3.64	3.16	3.31	2.66
Mass of can, *D*(*g*)	29.84	30.24	30.53	29.86	30.67	29.58
Mass of dry soil *E*= (*B* -*D*), (*g*)	6.93	6.90	6.15	9.36	9.82	8.16
Water content % (*C* / *E x* 100)	54.55	57.83	59.19	33.76	33.71	32.60
Number of blows	35	25	20			
Result, %	**57.2**			**33.4**		
Plasticity Index (*PI*) = *LL* - *PL*	**23.8**					

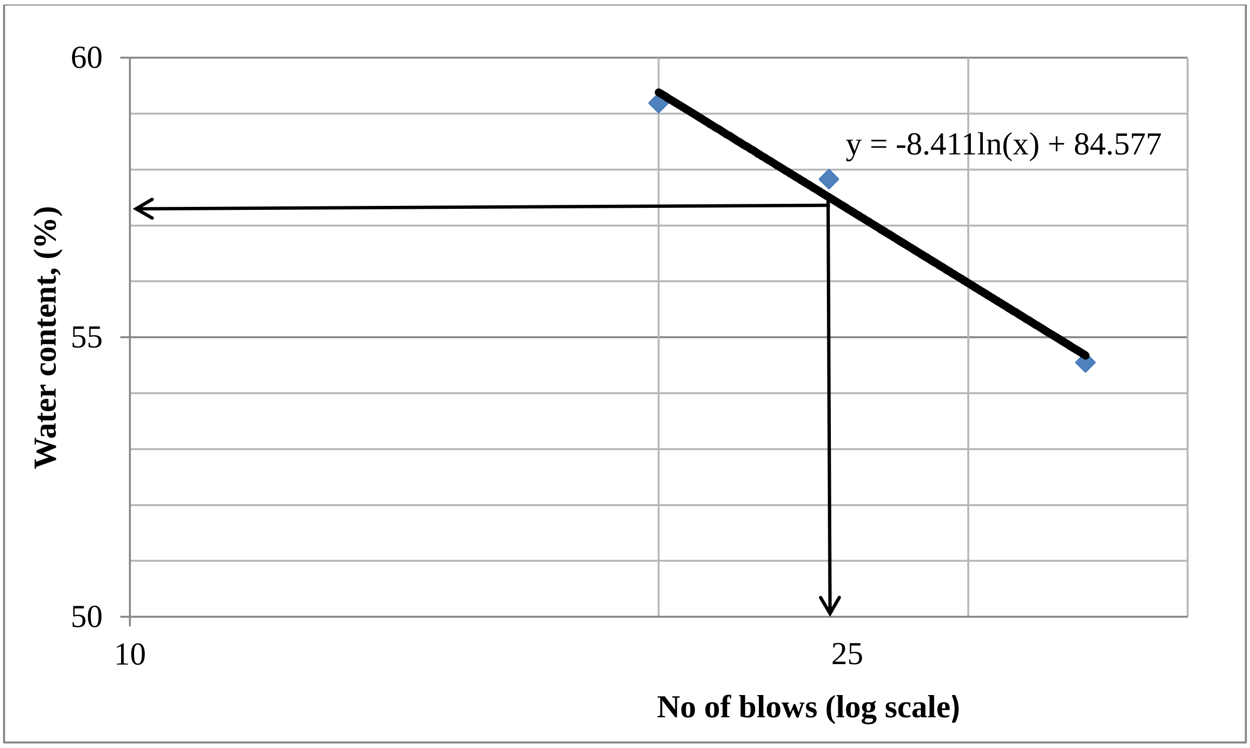

Figure B.4: Water content versus log of number of blows for test pit 5 @ 1*m*

Appendix C: Free Swell Test Results

Standards: IS 2720 Part 40–1977 for determination of Free Swell Index of Soils

Table C-1:-Results for Free Swell Index of the tested samples and degree of expansiveness

Sample No	Location	Depth in (*m*)	Test Pit ID	Reading After 24 Hours		Free Swell Index,%	Degree of Expansiveness
				Kerosene, V_k (*ml*)	Distilled Water, V_w (*ml*)		
1	Sunshine	1.0	Tp1	11.5	14	21.74	Moderate
2	Sunshine	2.0	Tp1	11	18	63.64	Very High
3	Babogaya	1.5	Tp2	11	17.5	59.09	Very High
4	Air force	1.5	Tp3	11	20	81.82	Very High
5	Ude/Denkaka	1.5	Tp4	12.5	18	44.00	High
6	Kurkura	1.0	Tp5	10	14	40.00	High
7	Kurkura	2.0	Tp5	10	18	80.00	Very High
8	Near Dukem	1.5	Tp6	12	19	58.33	Very High

Appendix D: Specific Gravity Test Results

Parameters and formulas used

- Density bottle (*100 ml in 20^{0}C*)

$$Gs = K * \frac{Ws}{(Ws + Wpw - Wpws)}$$

Table D-1 Density of water and correction factor *K* for Various Temperatures

Temperature, ^{0}C	Density of Water (*g/mL*)	Correction Factor *K*
16.0	0.99897	1.0007
16.5	0.99889	1.0007
17.0	0.9988	1.0006
17.5	0.99871	1.0005
18.0	0.99862	1.0004
18.5	0.99853	1.0003
19.0	0.99843	1.0002
19.5	0.99833	1.0001
20.0	0.99823	1.0000
20.5	0.99812	0.9999
21.0	0.99802	0.9998
21.5	0.99791	0.9997
22.0	0.99780	0.9996
22.5	0.99768	0.9995
23.0	0.99757	0.9993
23.5	0.99745	0.9992
24.0	0.99732	0.9991
24.5	0.99720	0.9990
25.0	0.99707	0.9988
25.5	0.99694	0.9987
26.0	0.99681	0.9986
26.5	0.99668	0.9984
27.0	0.99654	0.9983
27.5	0.99640	0.9982
28.0	0.99626	0.9980
28.5	0.99612	0.9979
29.0	0.99597	0.9977
29.5	0.99582	0.9976
30.0	0.99567	0.9974

Representatives for Specific gravity test results

Table D-2:- Specific gravity determination for test pit 1 @ 1*m*

Test pit 1	**Depth 1*m***		
A) Calibration of density bottle			
Density bottle No	*P5*	*P81*	*P29*
Weight of dry, clean density bottle, *Wp (g)*	53.26	49.69	52.51
Weight of density bottle + water, *Wpw (g)*	155.80	158.19	157.24
Observed temperature of water, T_i (^oC)	25	25	24.8
B) Specific gravity determination			
Determination No	1	2	3
Density bottle No	*P5*	*P81*	*P29*
Weight of density bottle + soil + water, *Wpws*(*g*)	165.33	167.66	166.72
Tempreture, T_x (^oC)	26	25.8	25.8
Weight of density bottle + water at T_x, *Wpw*(at *Tx*)(*g*)	155.80	158.19	157.24
Weight of dry soil, *Ws*(*g*)	15	15	15
Conversion factor, *K*	0.9986	0.9986	0.9986
Specific gravity of soil (G_s) at 20 ^oC, $G_s = KWs/(Ws+Wpw(\text{at } Tx)-Wpws)$	2.74	2.71	2.71
Average specific gravity of soil, G_s	**2.72**		

Table D-3:- Specific gravity determination for test pit 3 @ 1.5*m*

Test pit 3	**Depth 1.5*m***		
A) Calibration of density bottle			
Density bottle No	*P31*	*P32*	*P33*
Weight of dry, clean density bottle, *Wp (g)*	51.76	49.89	51.48
Weight of density bottle + water, *Wpw (g)*	157.17	154.98	159.67
Observed temperature of water, T_i (^oC)	25.6	25.4	25.6
B) Specific gravity determination			
Determination No	1	2	3
Density bottle No	*P31*	*P32*	*P33*
Weight of density bottle + soil + water, *Wpws*(*g*)	166.58	164.32	169.07
Tempreture, T_x (^oC)	26	26	26.2
Weight of density bottle + water at T_x, *Wpw*(at *Tx*)(*g*)	157.17	154.98	159.67
Weight of dry soil, *Ws*(*g*)	15	15	15
Conversion factor, *K*	0.9986	0.9986	0.9986
Specific gravity of soil (G_s) at 20 ^oC, $G_s = KWs/(Ws+Wpw(\text{at } Tx)-Wpws)$	2.68	2.65	2.67
Average specific gravity of soil, G_s	**2.67**		

Table D-4:- Specific gravity determination for test pit 4 @ 1.5*m*

Test pit 4	**Depth 1.5*m***		
A) Calibration of density bottle			
Density bottle No	*P28*	*P29*	*P30*
Weight of dry, clean density bottle, *Wp (g)*	52.11	51.03	51.13
Weight of density bottle + water, *Wpw (g)*	157.97	157.30	158.20
Observed temperature of water, T_i (^oC)	23.4	23.8	23.6
B) Specific gravity determination			
Determination No	1	2	3
Density bottle No	*P28*	*P29*	*P30*
Weight of density bottle + soil + water, *Wpws*(*g*)	167.46	166.77	167.70
Tempreture, T_x (^oC)	24	24.4	24.4
Weight of density bottle + water at T_x, *Wpw*(at *Tx*)(*g*)	157.97	157.30	158.20
Weight of dry soil, *Ws*(*g*)	15	15	15
Conversion factor, *K*	0.9991	0.9990	0.9990
Specific gravity of soil (G_s) at 20 ^oC, G_s = *KWs*/(*Ws*+*Wpw*(at *Tx*)-*Wpws*)	2.72	2.71	2.72
Average specific gravity of soil, G_s	**2.72**		

Table D-5:- Specific gravity determination for test pit 6 @ 1.5*m*

Test pit 6	**Depth 1.5*m***		
A) Calibration of density bottle			
Density bottle No	*P38*	*P39*	*P40*
Weight of dry, clean density bottle, *Wp (g)*	52.77	51.09	50.19
Weight of density bottle + water, *Wpw (g)*	158.89	159.35	157.13
Observed temperature of water, T_i (^oC)	23.2	22.6	22.8
B) Specific gravity determination			
Determination No	1	2	3
Density bottle No	*P38*	*P39*	*P40*
Weight of density bottle + soil + water, *Wpws*(*g*)	168.31	168.82	166.64
Tempreture, T_x (^oC)	23.8	23.2	23.2
Weight of density bottle + water at T_x, *Wpw*(at *Tx*)(*g*)	158.89	159.35	157.13
Weight of dry soil, *Ws*(*g*)	15	15	15
Conversion factor, *K*	0.9991	0.9993	0.9993
Specific gravity of soil (G_s) at 20 ^oC, G_s = *KWs*/(*Ws*+*Wpw*(at *Tx*)-*Wpws*)	2.69	2.71	2.73
Average specific gravity of soil, G_s	**2.71**		

Appendix E: Sieve and Hydrometer Test Results

Parameters and formulas used

- Hydrometer Type **ASTM 152 *H***
- Dispersing agent **Sodium hexametaphosphate (40*g/litre*)**
- Mass of soil **55*g***

Table E-1 Values of *K* for Use in Equation for Computing Diameter of Particle in Hydrometer Analysis

Temp. ^{0}C	**Specific Gravity (*X*), *K*=*X**10^{-2} or *K*=*X*/10^2**								
	2.45	2.5	2.55	2.6	2.65	2.7	2.75	2.8	2.85
16	1.510	1.505	1.481	1.457	1.435	1.414	1.394	1.374	1.356
17	1.511	1.486	1.462	1.439	1.417	1.396	1.376	1.356	1.338
18	1.492	1.467	1.443	1.421	1.399	1.378	1.359	1.339	1.321
19	1.474	1.449	1.425	1.403	1.382	1.361	1.342	1.323	1.305
20	1.456	1.431	1.408	1.386	1.365	1.344	1.325	1.307	1.289
21	1.438	1.414	3.91	1.369	1.348	1.328	1.309	1.291	1.273
22	1.421	1.397	1.374	1.353	1.332	1.312	1.294	1.276	1.258
23	1.404	1.381	1.358	1.337	1.317	1.297	1.279	1.261	1.243
24	1.388	1.365	1.342	1.321	1.301	1.282	1.264	1.246	1.229
25	1.372	1.349	1.327	1.306	1.286	1.267	1.249	1.232	1.215
26	1.357	1.334	1.312	1.291	1.272	1.253	1.235	1.218	1.201
27	1.342	1.319	1.297	1.277	1.258	1.239	1.221	1.204	1.188
28	1.327	1.304	1.283	1.264	1.244	1.255	1.208	1.191	1.175
29	1.312	1.29	1.269	1.249	1.23	1.212	1.195	1.178	1.162
30	1.298	1.276	1.256	1.236	1.217	1.199	1.182	1.165	1.149

Table E-2 Values of *L* (Effective depth) based on hydrometer and sedimentation cylinder of specified sizes (Diameters of particles for *ASTM* soil hydrometer 152*H*)

Actual Hydrometer Reading	**Effective Depth, *L*(*cm*)**	**Actual Hydrometer Reading**	**Effective Depth, *L*(*cm*)**
0	16.30	31	11.20
1	16.10	32	11.10
2	16.00	33	10.90
3	15.80	34	10.70
4	15.60	35	10.60
5	15.50	36	10.40
6	15.30	37	10.20
7	15.20	38	10.10
8	15.00	39	9.90
9	14.80	40	9.70
10	14.70	41	9.60
11	14.50	42	9.40
12	14.30	43	9.20
13	14.20	44	9.10
14	14.00	45	8.90
15	13.80	46	8.80
16	13.70	47	8.60
17	13.50	48	8.40
18	13.30	49	8.30
19	13.20	50	8.10
20	13.00	51	7.90
21	12.90	52	7.80
22	12.70	53	7.60
23	12.50	54	7.40
24	12.40	55	7.30
25	12.20	56	7.10
26	12.00	57	7.00
27	11.90	58	6.80
28	11.70	59	6.60
29	11.50	60	6.50
30	11.40		

Table E-3 Dry sieve analysis for test pit 1 @ 2*m*

Test pit 1				**Depth 2*m***	
Sieve *ASTM*	Sieve Opening (*mm*)	Weight Retained out of 1000*g*	% Retained	Cumulative % Retained	% Finer
3/8"	9.50	0	0.00	0	100.00
No.4	4.75	9	0.90	0.90	99.10
No.8	2.36	4	0.40	1.30	98.70
No.10	2.00	2	0.20	1.50	98.50
No.16	1.18	2	0.20	1.70	98.30
No.30	0.600	5	0.50	2.20	97.80
No.40	0.425	6	0.60	2.80	97.20
No.50	0.300	6	0.60	3.40	96.60
No.100	0.150	13	1.30	4.70	95.30
No.200	0.075	12	1.20	5.90	94.10

Table E-4 Hydrometer analysis for test pit 1 @ 2*m*

Elapsed Time, T (*min*)	Actual Hydrometer Reading, *Ra*	Corrected Hydrometer reading, *Rc*	Effective Depth *L*(*cm*)	$\sqrt{(L/T)}$	Coefficient, *K* from Table	Diameter of particle, *D*(*mm*)	% Finer	Adjust percent *PA*
0.5	46	44.35	8.8	4.1952	0.01253	0.053	88.10	82.90
1	45	43.35	8.9	2.9833	0.01253	0.037	86.12	81.03
2	44	42.35	9.1	2.1331	0.01253	0.027	84.13	79.17
5	42	40.35	9.4	1.3711	0.01253	0.017	80.16	75.43
10	40	38.35	9.7	0.9849	0.01253	0.012	76.18	71.69
20	38	36.35	10.1	0.7106	0.01253	0.009	68.24	67.95
40	36	34.35	10.4	0.5099	0.01253	0.006	68.24	64.21
80	33	31.35	10.9	0.3691	0.01253	0.005	62.28	58.60
120	29	27.35	11.5	0.3096	0.01253	0.004	54.33	51.13
240	25	23.35	12.2	0.2255	0.01253	0.003	46.39	43.65
480	21	19.35	12.9	0.1639	0.01253	0.002	38.44	36.17
1440	14	12.35	14	0.0986	0.01253	0.001	24.53	23.09

Table E-5 Dry sieve analysis for test pit 3 @ 1.5*m*

Test pit 3				**Depth 1.5*m***	
Sieve *ASTM*	Sieve Opening (*mm*)	Weight Retained out of 1000*g*	% Retained	Cumulative % Retained	% Finer
3/8"	9.50	0	0.00	0	100.00
No.4	4.75	5	0.50	0.50	99.50
No.8	2.36	0	0.00	0.50	99.50
No.10	2.00	2	0.20	0.70	99.30
No.16	1.18	3	0.30	1.00	99.00
No.30	0.600	5	0.50	1.50	98.50
No.40	0.425	4	0.40	1.90	98.10
No.50	0.300	6	0.60	2.50	97.50
No.100	0.150	9	0.90	3.40	96.60
No.200	0.075	11	1.10	4.50	95.50

Table E-6 Hydrometer analysis for test pit 3 @ 1.5*m*

Elapsed Time, T (*min*)	Actual Hydrometer Reading, *Ra*	Corrected Hydrometer reading, *Rc*	Effective Depth *L*(*cm*)	$\sqrt{(L/T)}$	Coefficient, *K* from Table	Diameter of particle, *D*(*mm*)	% Finer	Adjust percent *PA*
0.5	47	45.35	8.6	4.1473	0.01253	0.052	90.29	86.23
1	46	44.35	8.8	2.9665	0.01253	0.037	88.30	84.33
2	45	43.35	8.9	2.1095	0.01253	0.026	86.31	82.42
5	43	41.35	9.2	1.3565	0.01253	0.017	82.33	78.62
10	41	39.35	9.6	0.9798	0.01253	0.012	78.34	74.82
20	38	36.35	10.1	0.7106	0.01253	0.009	66.40	69.11
40	35	33.35	10.6	0.5148	0.01253	0.006	66.40	63.41
80	31	29.35	11.2	0.3742	0.01253	0.005	58.43	55.81
120	27	25.35	11.9	0.3149	0.01253	0.004	50.47	48.20
240	23	21.35	12.5	0.2282	0.01253	0.003	42.51	40.59
480	17	15.35	13.5	0.1677	0.01253	0.002	30.56	29.19
1440	9	7.35	14.8	0.1014	0.01253	0.001	14.63	13.98

Table E-7 Dry sieve analysis for test pit 5 @ 2*m*

Test pit 5				**Depth 2*m***	
Sieve *ASTM*	Sieve Opening (*mm*)	Weight Retained out of 1000*g*	% Retained	Cumulative % Retained	% Finer
3/8"	9.50	0	0.00	0	100.00
No.4	4.75	23	2.30	2.30	97.70
No.8	2.36	7	0.70	3.00	97.00
No.10	2.00	4	0.40	3.40	96.60
No.16	1.18	9	0.90	4.30	95.70
No.30	0.600	17	1.70	6.00	94.00
No.40	0.425	14	1.40	7.40	92.60
No.50	0.300	21	2.10	9.50	90.50
No.100	0.150	21	2.10	11.60	88.40
No.200	0.075	12	1.20	12.80	87.20

Table E-8 Hydrometer analysis for test pit 5 @ 2*m*

Elapsed Time, T (*min*)	Actual Hydrometer Reading, *Ra*	Corrected Hydrometer reading, *Rc*	Effective Depth *L*(*cm*)	$\sqrt{(L/T)}$	Coefficient, *K* from Table	Diameter of particle, *D*(*mm*)	% Finer	Adjust percent *PA*
0.5	47	46.00	8.6	4.1473	0.01267	0.053	90.98	79.33
1	46	45.00	8.8	2.9665	0.01267	0.038	89.00	77.61
2	45	44.00	8.9	2.1095	0.01267	0.027	87.02	75.88
5	44	43.00	9.1	1.3491	0.01267	0.017	85.05	74.16
10	43	42.00	9.2	0.9592	0.01267	0.012	83.07	72.44
20	42	41.00	9.4	0.6856	0.01267	0.009	77.13	70.71
40	40	39.00	9.7	0.4924	0.01267	0.006	77.13	67.26
80	36	35.00	10.4	0.3606	0.01267	0.005	69.22	60.36
120	32	31.00	11.1	0.3041	0.01267	0.004	61.31	53.46
240	28	27.00	11.7	0.2208	0.01267	0.003	53.40	46.57
480	24	23.00	12.4	0.1607	0.01267	0.002	45.49	39.67
1440	16	15.00	13.7	0.0975	0.01267	0.001	29.67	25.87

Table E-9 Dry sieve analysis for test pit 6 @ 1.5*m*

Test pit 6				Depth 1.5*m*	
Sieve *ASTM*	Sieve Opening (*mm*)	Weight Retained out of 1000*g*	% Retained	Cumulative % Retained	% Finer
3/8"	9.50	0	0.00	0	100.00
No.4	4.75	10	1.00	1.00	99.00
No.8	2.36	1	0.10	1.10	98.90
No.10	2.00	1	0.10	1.20	98.80
No.16	1.18	3	0.30	1.50	98.50
No.30	0.600	5	0.50	2.00	98.00
No.40	0.425	8	0.80	2.80	97.20
No.50	0.300	10	1.00	3.80	96.20
No.100	0.150	11	1.10	4.90	95.10
No.200	0.075	7	0.70	5.60	94.40

Table E-10 Hydrometer analysis for test pit 6 @ 1.5*m*

Elapsed Time, T (*min*)	Actual Hydrometer Reading, *Ra*	Corrected Hydrometer reading, *Rc*	Effective Depth *L*(*cm*)	√(*L*/*T*)	Coefficient, *K* from Table	Diameter of particle, *D*(*mm*)	% Finer	Adjust percent *PA*
0.5	50	48.70	8.1	4.0249	0.01267	0.051	96.11	90.73
1	49	47.70	8.3	2.8810	0.01267	0.037	94.14	88.87
2	48	46.70	8.4	2.0494	0.01267	0.026	92.16	87.00
5	47	45.70	8.6	1.3115	0.01267	0.017	90.19	85.14
10	46	44.70	8.8	0.9381	0.01267	0.012	88.22	83.28
20	45	43.70	8.9	0.6671	0.01267	0.008	82.30	81.41
40	43	41.70	9.2	0.4796	0.01267	0.006	82.30	77.69
80	41	39.70	9.6	0.3464	0.01267	0.005	78.35	73.96
120	38	36.70	10.1	0.2901	0.01267	0.004	72.43	68.37
240	34	32.70	10.7	0.2111	0.01267	0.003	64.53	60.92
480	30	28.70	11.4	0.1541	0.01267	0.002	56.64	53.47
1440	21	19.70	12.9	0.0946	0.01267	0.001	38.88	36.70

Table E-11 Combined sieve-hydrometer analysis for test pit 1 @1*m*, 1 @2*m*, 2 @1.5*m* & 3 @ 1.5*m*

Test pit 1 @ 1*m*		Test pit 1 @ 2*m*		Test pit 2 @ 1.5*m*		Test pit 3 @ 1.5*m*	
Sieve Opening, (*mm*)	% Finer	Sieve Opening, (*mm*)	% Finer	Sieve Opening, (*mm*)	% Finer	Sieve Opening, (*mm*)	% Finer
9.500	100.000	9.500	100.000	9.500	100.000	9.500	100.000
4.750	99.000	4.750	99.100	4.750	99.800	4.750	99.500
2.360	96.000	2.360	98.700	2.360	99.800	2.360	99.500
2.000	95.300	2.000	98.500	2.000	99.800	2.000	99.300
1.180	93.400	1.180	98.300	1.180	99.800	1.180	99.000
0.600	91.700	0.600	97.800	0.600	99.800	0.600	98.500
0.425	90.100	0.425	97.200	0.425	99.200	0.425	98.100
0.300	88.000	0.300	96.600	0.300	98.200	0.300	97.500
0.150	85.100	0.150	95.300	0.150	95.400	0.150	96.600
0.075	82.800	0.075	94.100	0.075	93.800	0.075	95.500
0.052	72.886	0.053	82.904	0.049	93.613	0.052	86.227
0.037	71.256	0.037	81.035	0.035	91.754	0.037	84.326
0.027	69.625	0.027	79.166	0.025	89.894	0.026	82.424
0.017	66.364	0.017	75.427	0.016	88.035	0.017	78.622
0.012	63.103	0.012	71.688	0.012	84.317	0.012	74.819
0.009	59.842	0.009	67.950	0.008	80.598	0.009	69.115
0.006	54.950	0.006	64.211	0.006	76.880	0.006	63.411
0.005	50.058	0.005	58.603	0.005	69.443	0.005	55.805
0.004	45.167	0.004	51.126	0.004	62.006	0.004	48.200
0.003	38.644	0.003	43.649	0.003	52.710	0.003	40.594
0.002	30.492	0.002	36.171	0.002	43.413	0.002	29.186
0.001	19.078	0.001	23.086	0.001	28.539	0.001	13.975

Table E-12 Combined sieve-hydrometer analysis for test pit 4 @ 1.5*m*, 5@ 1*m*, 5@ 2*m* & 6 @ 1.5*m*

Test pit 4 @ 1.5*m*		Test pit 5 @ 1*m*		Test pit 5 @ 2*m*		Test pit 6 @ 1.5*m*	
Sieve Opening, (*mm*)	% Finer	Sieve Opening, (*mm*)	% Finer	Sieve Opening, (*mm*)	% Finer	Sieve Opening, (*mm*)	% Finer
9.500	100.000	9.500	100.000	9.500	100.000	9.500	100.000
4.750	99.200	4.750	99.600	4.750	97.700	4.750	99.000
2.360	98.700	2.360	98.700	2.360	97.000	2.360	98.900
2.000	98.700	2.000	98.300	2.000	96.600	2.000	98.800
1.180	98.300	1.180	97.100	1.180	95.700	1.180	98.500
0.600	97.500	0.600	95.300	0.600	94.000	0.600	98.000
0.425	96.400	0.425	93.800	0.425	92.600	0.425	97.200
0.300	94.900	0.300	92.000	0.300	90.500	0.300	96.200
0.150	92.900	0.150	88.800	0.150	88.400	0.150	95.100
0.075	91.200	0.075	86.600	0.075	87.200	0.075	94.400
0.052	83.873	0.053	78.448	0.053	79.334	0.051	90.728
0.037	82.077	0.038	76.743	0.038	77.609	0.037	88.865
0.027	80.281	0.027	75.038	0.027	75.884	0.026	87.002
0.017	78.485	0.017	73.332	0.017	74.160	0.017	85.139
0.012	74.893	0.012	71.627	0.012	72.435	0.012	83.276
0.009	73.097	0.009	68.216	0.009	70.710	0.008	81.413
0.006	69.505	0.006	64.805	0.006	67.261	0.006	77.687
0.005	64.117	0.005	59.689	0.005	60.363	0.005	73.961
0.004	56.933	0.004	52.867	0.004	53.464	0.004	68.372
0.003	49.749	0.003	46.046	0.003	46.565	0.003	60.920
0.002	40.769	0.002	37.519	0.002	39.667	0.002	53.468
0.001	24.605	0.001	23.876	0.001	25.870	0.001	36.701

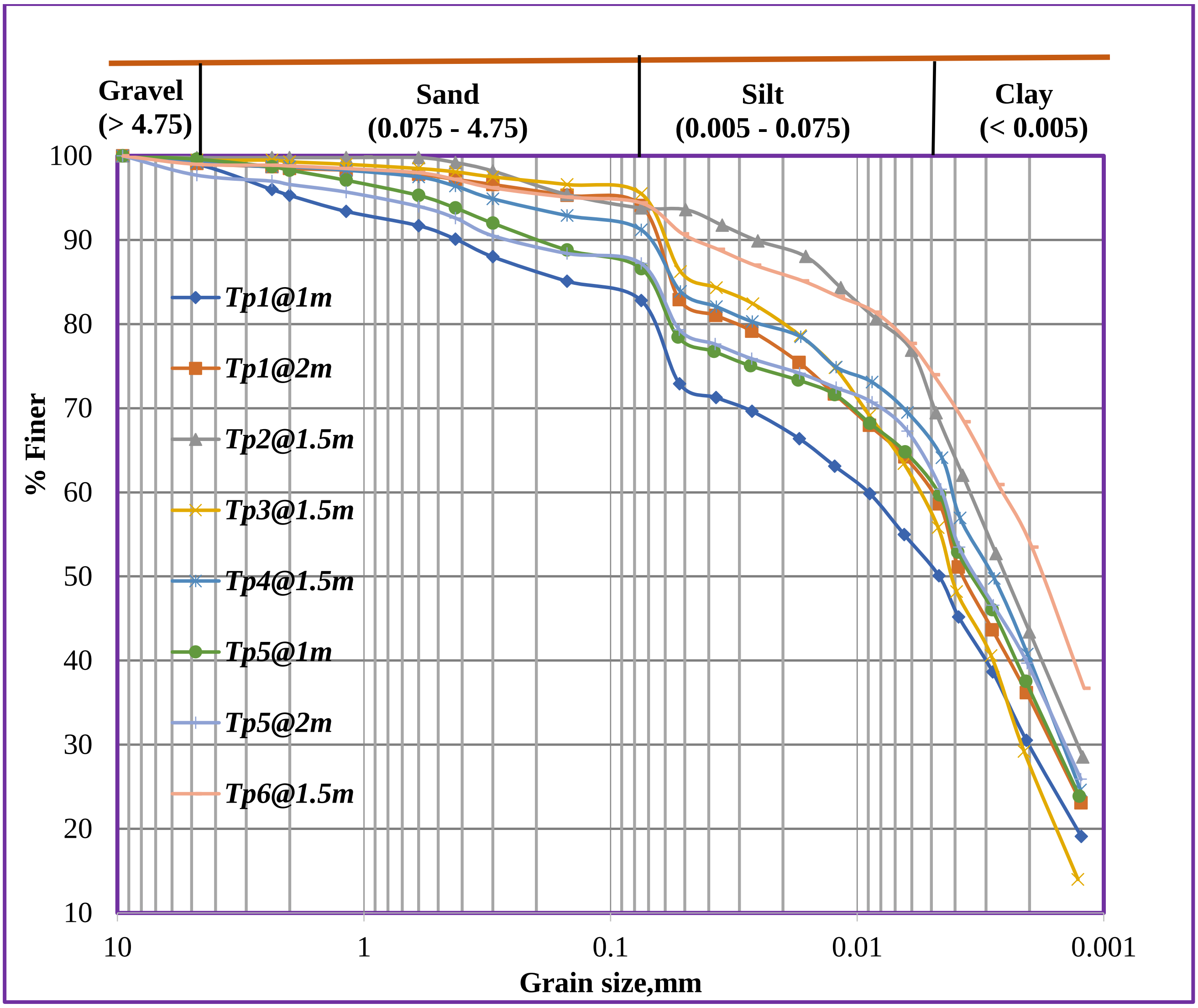

Figure E-1 Combined grain size distribution curve for all tests

Appendix F: Compaction Test Results

Parameters and formulas used

- ✓ Standard Compaction test
- ✓ Compaction mold with base plate and collar
- ✓ Compaction rammer (24.5*N*x0.305*m* drop)

Representatives for Compaction test results

Table F-1:- Compaction test results determination of test pit 1 @ 1*m*

Moisture content versus dry density ***Test pit 1 @ 1m***					
Trial No	*1*	*2*	*3*	*4*	*5*
Mass of Mold (*g*)	3755	3755	3755	3755	3755
Mass of Compacted Soil + mold, (*g*)	5260	5335	5438	5535	5517
Mass of Compacted Soil, M_s (*g*)	1505	1580	1683	1780	1762
Volume of mold, V_m (cm^3)	944	944	944	944	944
Bulk density, $\gamma b = \frac{Ms}{Vm}$ (g/cm^3)	1.59	1.67	1.78	1.89	1.87
Water content, w (%)	19.5	23.24	27.83	33.93	38.66
Dry density, $\gamma d = \frac{\gamma b}{1+w/100}$ (g/cm^3)	1.33	1.36	1.39	1.41	1.35

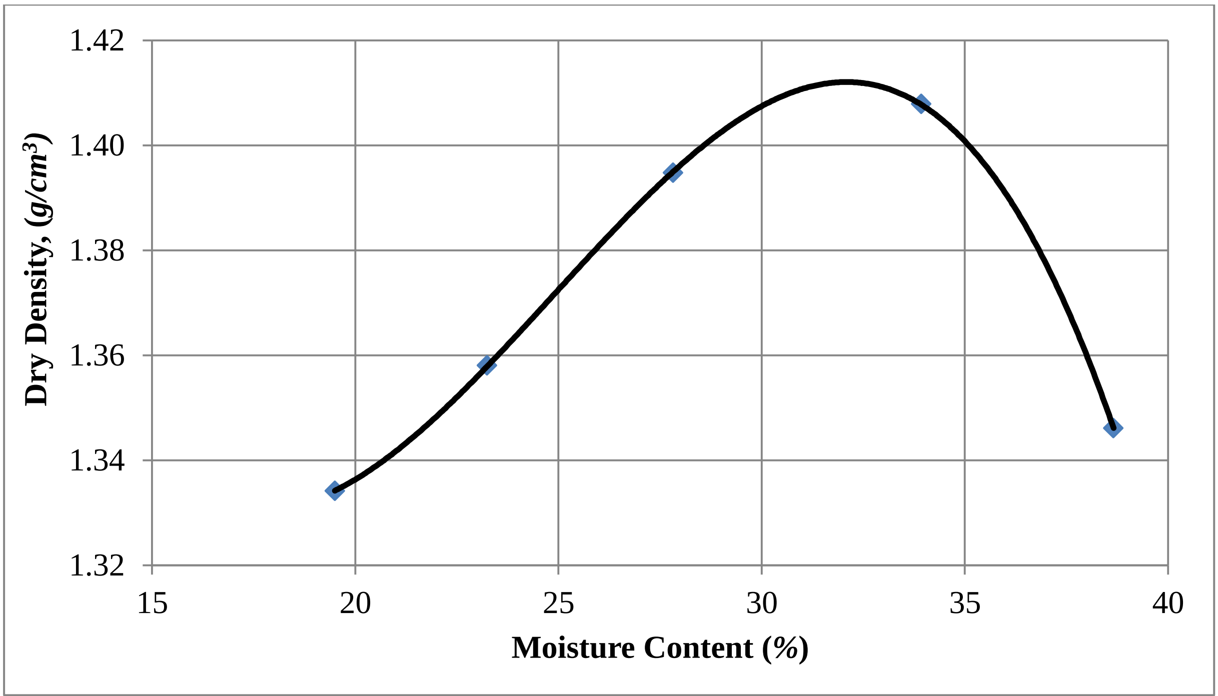

Figure F-1: Moisture content versus dry density for test pit 1 @ 1*m*

Table F-2:- Compaction test results determination of test pit 1 @ 2*m*

Moisture content versus dry density				***Test pit 1 @ 2m***	
Trial No	*1*	*2*	*3*	*4*	*5*
Mass of Mold (*g*)	3755	3755	3755	3755	3755
Mass of Compacted Soil + mold, (*g*)	5121	5215	5283	5318	5312
Mass of Compacted Soil, M_s (*g*)	1366	1460	1528	1563	1557
Volume of mold, V_m (cm^3)	944	944	944	944	944
Bulk density, $\gamma b = \frac{Ms}{Vm}$ (g/cm^3)	1.44	1.55	1.62	1.66	1.65
Water content, w (%)	23.41	27.95	32.71	37.38	43.15
Dry density, $\gamma d = \frac{\gamma b}{1+w/100}$ (g/cm^3)	1.17	1.21	1.22	1.21	1.15

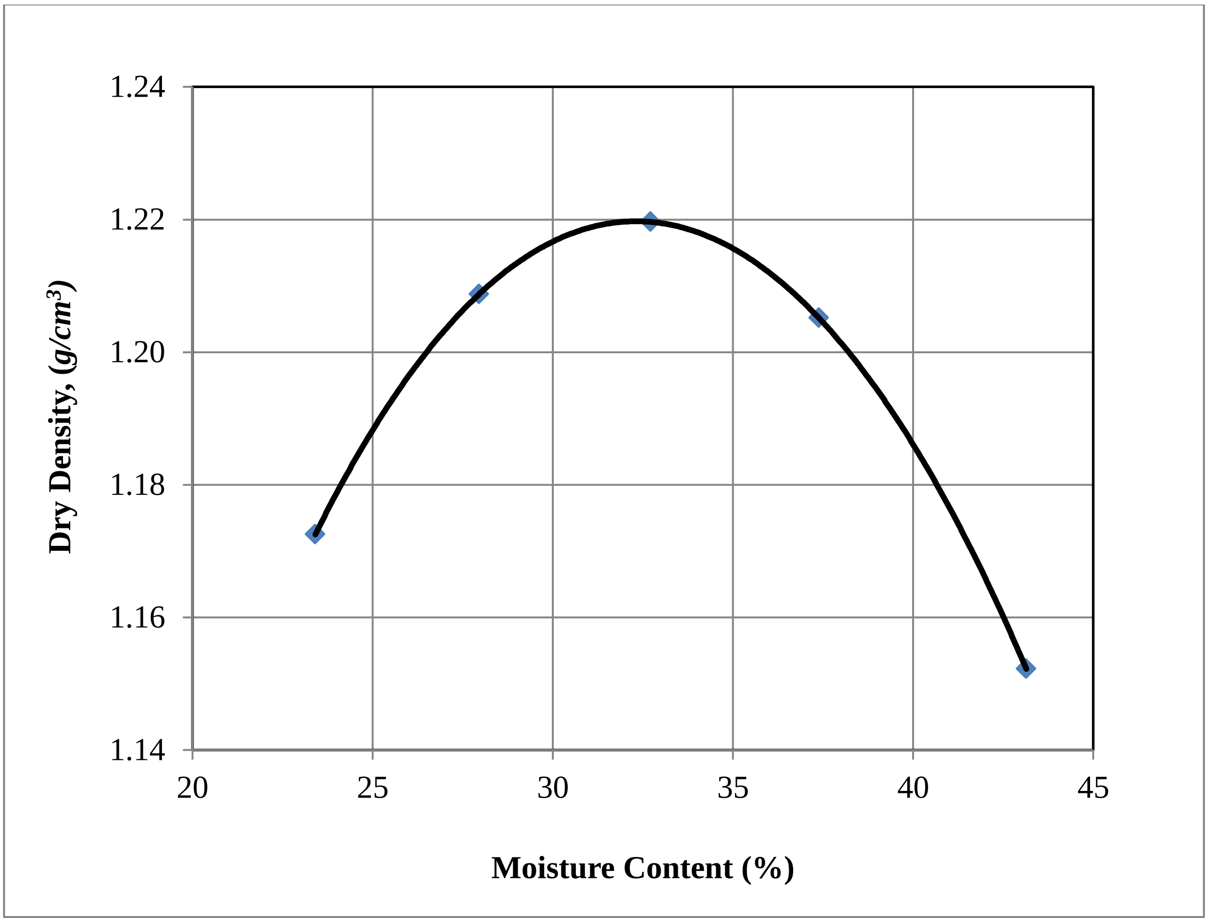

Figure F-2: Moisture content versus dry density for test pit 1 @ 2*m*

Table F-3:- Compaction test results determination of test pit 5 @ 1*m*

Moisture content versus dry density	***Test pit 5 @ 1m***				
Trial No	*1*	*2*	*3*	*4*	*5*
Mass of Mold (*g*)	3755	3755	3755	3755	3755
Mass of Compacted Soil + mold, (*g*)	5255	5346	5447	5537	5523
Mass of Compacted Soil, M_s (*g*)	1500	1591	1692	1782	1768
Volume of mold, V_m (cm^3)	944	944	944	944	944
Bulk density, $\gamma b = \frac{Ms}{Vm}$ (g/cm^3)	1.59	1.69	1.79	1.89	1.87
Water content, w (%)	18.86	22.68	27.16	32.73	37.89
Dry density, $\gamma d = \frac{\gamma b}{1+w/100}$ (g/cm^3)	1.34	1.37	1.41	1.42	1.36

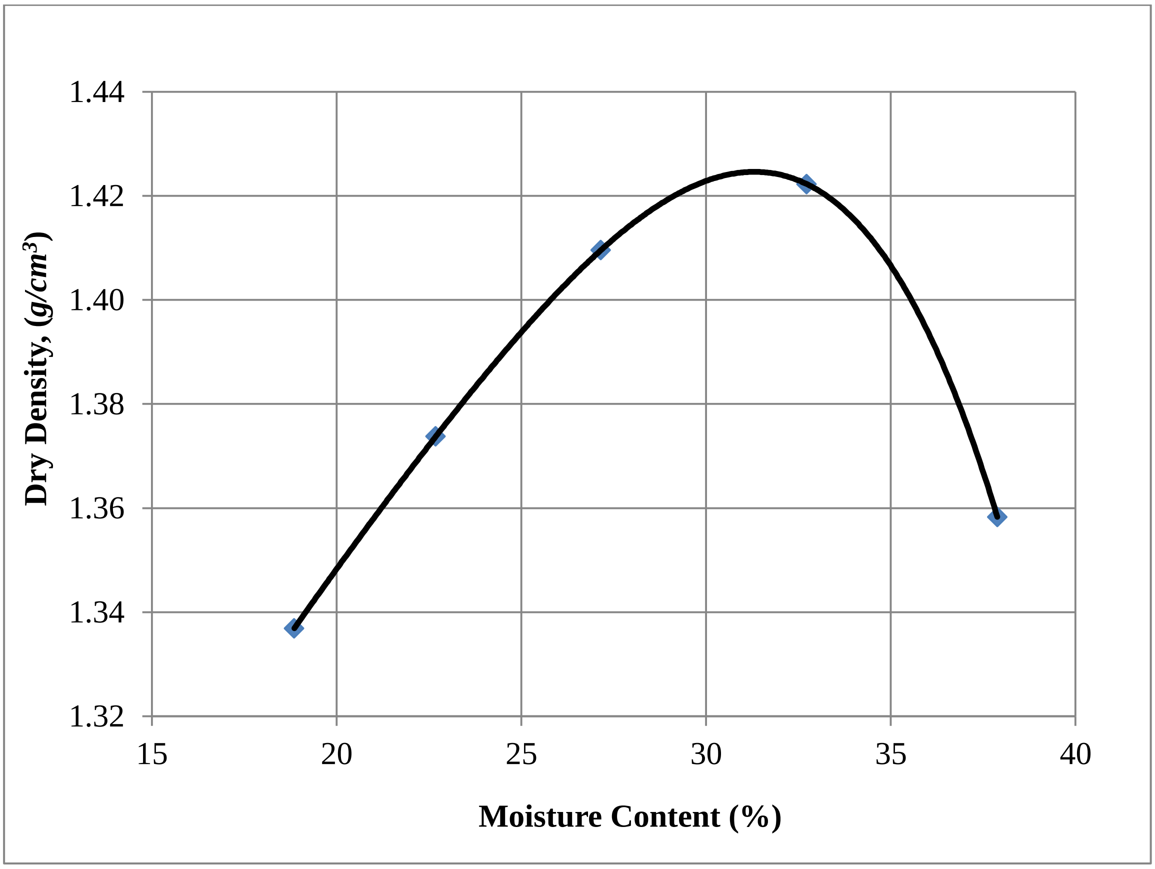

Figure F-3: Moisture content versus dry density for test pit 5 @ 1*m*

Table F-4:- Compaction test results determination of test pit 5 @ 2*m*

Moisture content versus dry density	***Test pit 5 @ 2m***					
Trial No	*1*	*2*	*3*	*4*	*5*	*6*
Mass of Mold (*g*)	3755	3755	3755	3755	3755	3755
Mass of Compacted Soil + mold, (*g*)	5153	5235	5295	5400	5440	5428
Mass of Compacted Soil, M_s (*g*)	1398	1480	1540	1645	1685	1673
Volume of mold, V_m (cm^3)	944	944	944	944	944	944
Bulk density, $\gamma b = \frac{Ms}{Vm}$ (g/cm^3)	1.48	1.57	1.63	1.74	1.79	1.77
Water content, w (%)	23.29	27.82	31.00	37.54	43.09	49.62
Dry density, $\gamma d = \frac{\gamma b}{1+w/100}$ (g/cm^3)	1.20	1.23	1.25	1.27	1.25	1.18

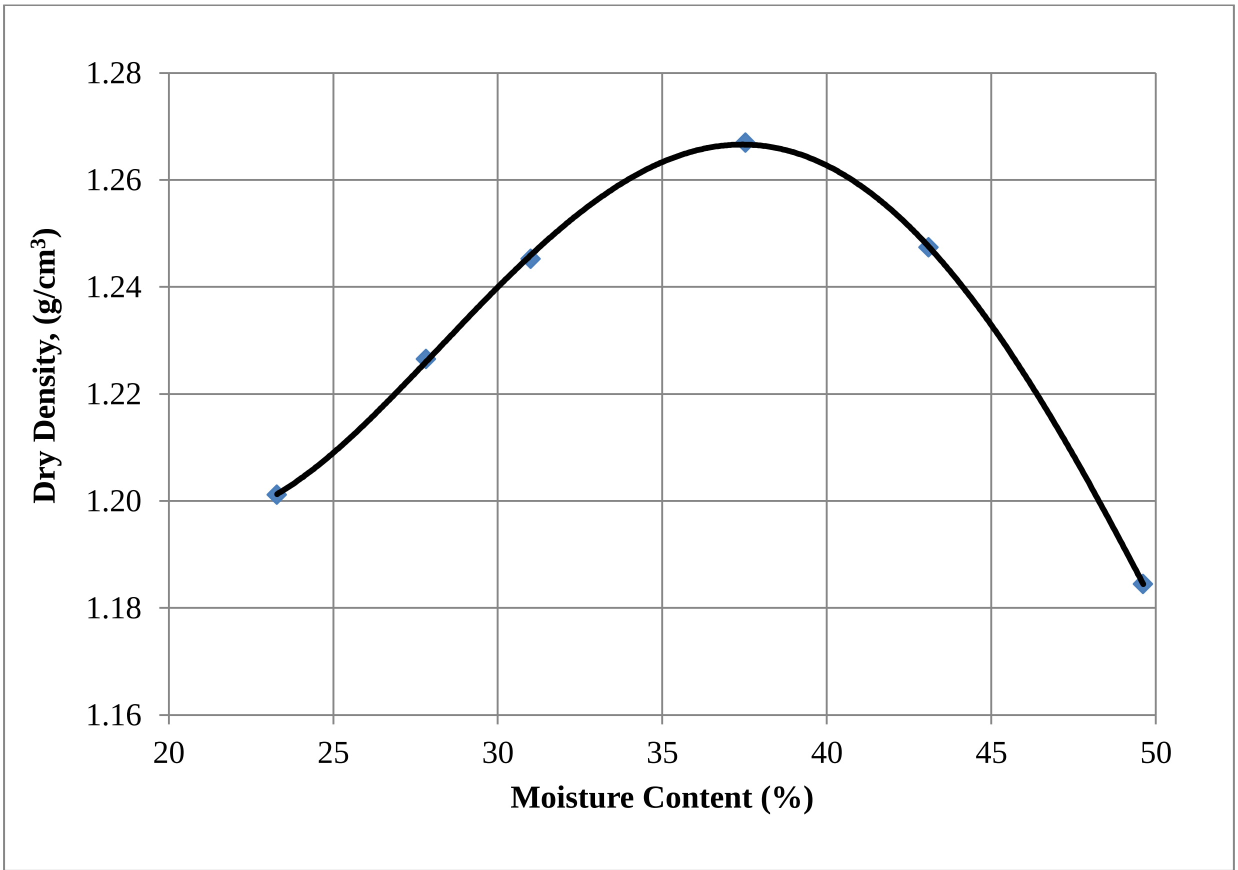

Figure F.4: Moisture content versus dry density for test pit 5 @ 2*m*

Appendix G: One Dimensional Consolidation test result

Parameters and formulas used

ASTM D 2435 Standard Test Method for One Dimensional Consolidation Properties of Soils. The laboratory consolidation test is performed on specimen with thickness of 20*mm*, placed in a confining metal ring ranging with diameter of 50cm and 75*mm*.

1. Carefully trim a sample to fit the 'consolidation ring. Weight the sample and determine the sample height H_i and the diameter of sample.

2. Place the sample in the consolidometer with a saturated porous stone on each face. Place the consolidometer in the loading device and attach the dial gage. Apply a seating load, the consolidation test proceeds by applying loads in a geometric progression with a load ratio, $(P_2-P_1)/P_1=1$, with a typical load sequence as follows: Loading Procedure (recommended): The specimen is subjected to sequence loads that may be done by doubling the applied load every 24 hr. Load increment ratio (*LIR*) = $(P_2-P_1)/P_1$

For the preceding procedure the (*LIR*) = 1. In the oedometer and for a beam ratio = 1: 9 (i.e.,1*kg* applied on the beam = 9*kg* on the sample), the stress which correspond 1.25*kg* equal: $P/A=1.25\text{x}9/(\Pi/4)\text{x}(0.075)^2\ kg/m^2\text{x}1/101.9\ kN/kg = 25kN/m^2$

Applied weight on beam (*kg*) 1.25, 2.5, 5, 10, 20, 40 Applied stress on sample (kN/m^2) 25 50 100 200 400 800 Therefore, the following series of loads is applied: (25, 50, 100, 200, 400, 800, 1600 and sometimes 3200) *kPa* then zero the deformation dial gage. Then unloading the sample to (800, 400, 200, 100) *kPa*

3. At a convenient starting time, apply the first load increment and at the same time take deformation readings at elapsed times of (0.25, 1, 2.25 , 4, 6.25, 9, 12.25, 16, 25,… min, then, say, 1, 2, 4, h etc.). Take the reading of dial gauge at the time intervals above to plot *DGR* versus log t or *DGR* versus √ (time).

4. After 24h, change the load to the next value and again take elapsed-time interval readings as done above and continue change the load each 24h to take the readings of dial gauge and the elapsed time.

5. Place the sample in the oven at the end of the test to find the weight of soil solids (*Ws*).

Representatives for One dimensional consolidation test results

Before Test Sample type **Remolded, test pit 1 @ 1*m***

Weight of Sample & Ring (*g*)	232.50
Weight of Ring (*g*)	110.70
Weight of Sample (*g*)	121.80
Weight of Dry sample (*g*)	97.30
Weight of Initial Moisture (*g*)	24.50
Initial Moisture Content, M_o (%)	25.18
Initial Void Ratio : $e_o = (G_s/r_d)-1$	1.489
Height of Solids : $H_s = H_o/(1+e_o)$, (*mm*)	8.037
Initial Saturation: $S_o = (M_o x G_s)/e_o$, (%)	46.35

Specific Gravity (G_s)	2.72
Sample Diameter (*D*) (*mm*)	75.00
Sample Thickness (H_o) (*mm*)	20.00
Area (*A*) (cm^2)	44.18
Volume (*V*) (cm^3)	88.37
Bulk Density (r_b) (g/cm^3)	1.378
Dry Density (r_d) (g/cm^3)	1.101

After Test

Weight of Sample + Ring + Can (*g*)	239.70
Weight of Dry sample + Ring + Can (*g*)	208.00
Weight of Ring + Can (*g*)	110.70
Weight of Wet sample (*g*)	129.00
Weight of Dry sample (*g*)	97.30
Final Moisture Content, M_f (%)	32.58

Overall settlement (*mm*)	4.264
Volume change (cm^3)	18.84
Final Volume (cm^3)	69.53
Final Bulk Density (g/cm^3)	1.855
Final Dry Density (g/cm^3)	1.399

Sample Diameter (*D*) (*mm*)	75	Height H_o , (*mm*)	20
Height of Solids : H_s=	8.037	Initial Void Ratio : e_o=	1.49

Table G.1 Calculation of Coefficient of consolidation, *Cv*, for test pit 1 @ 1*m*

Pressure *kPa*	Cumulative compression (ΔH) *mm*	Consolidated height H = Ho-ΔH *mm*	Void Ratio *e* = *(H-Hs)/Hs*	t_{90} min	*H* = 1/2 (*H1*+*H2*)	$Cv = (0.848 * H^2) / t_{90}$ $m^2/year$
50	1.00	19.00	1.36	26.78	19.50	6.327
100	1.88	18.12	1.25	84.64	18.56	1.813
200	2.97	17.03	1.12	64.8	17.57	2.124
400	3.91	16.09	1.00	33.06	16.56	3.697
800	4.89	15.11	0.88	84.64	15.60	1.282
200	4.61	15.39	0.92			
50	4.26	15.74	0.96			

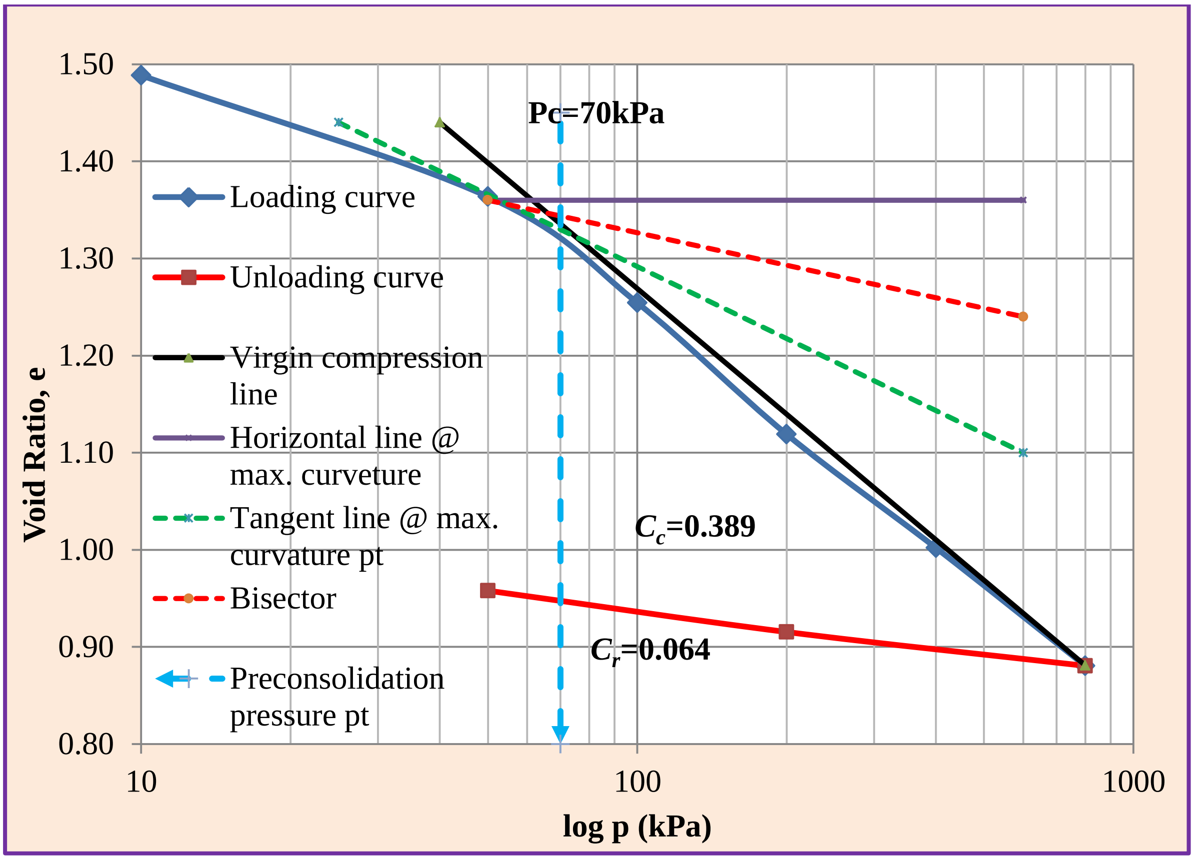

Figure G-1 *e-log p curve for test pit 1 @1m*

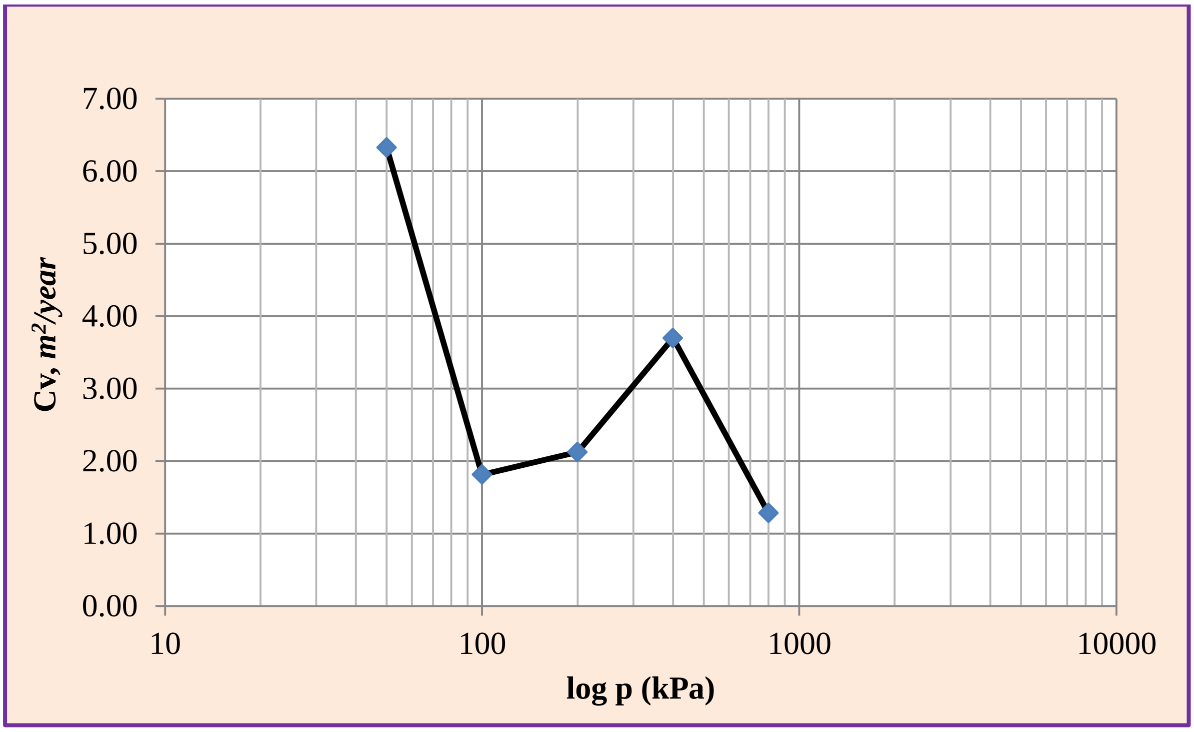

Figure G-2 *Cv versus log p for test pit 1 @ 1m*

Before Test Sample type **Remolded, Test pit 4 @1.5*m***

Weight of Sample & Ring (*g*)	128.60
Weight of Ring (*g*)	69.20
Weight of Sample (*g*)	59.40
Weight of Dry sample (*g*)	45.10
Weight of Initial Moisture (*g*)	14.30
Initial Moisture Content, M_o (%)	31.71
Initial Void Ratio : e_o= (G_s/r_d)-1	1.369
Height of Solids : H_s= H_o/(1+e_o), (*mm*)	8.442
Initial Saturation: S_o= (M_oxG_s)/e_o , (%)	63.00

Specific Gravity (G_s)	2.72
Sample Diameter (*D*) (*mm*)	50.00
Sample Thickness (H_o) (*mm*)	20.00
Area (*A*) (cm^2)	19.64
Volume (*V*) (cm^3)	39.28
Bulk Density (r_b) (g/cm^3)	1.512
Dry Density (r_d) (g/cm^3)	1.148

After Test

Weight of Sample + Ring + Can (*g*)	122.30
Weight of Dry sample + Ring + Can (*g*)	114.30
Weight of Ring + Can (*g*)	69.20
Weight of Wet sample (*g*)	53.10
Weight of Dry sample (*g*)	45.10
Final Moisture Content, M_f (%)	17.74

Overall settlement (*mm*)	4.004
Volume change (cm^3)	7.86
Final Volume (cm^3)	31.41
Final Bulk Density (g/cm^3)	1.691
Final Dry Density (g/cm^3)	1.436

Sample Diameter (*D*) (*mm*)	50
Height of Solids : H_s=	8.442

Height H_o , (*mm*)	20
Initial Void Ratio : e_o=	1.37

Table G.2 Consolidation dial gauge reading for test pit 4 @ 1.5*m*

Time, min	**√Time, min**	**Dial gauge reading in *mm***						
		180kPa	**360kPa**	**720kPa**	**1440kPa**	**2880kPa**	**720kPa**	**180kPa**
0	0	0.000	0.788	1.474	2.756	3.956	4.934	4.454
0.25	0.5	0.266	1.068	1.960	3.120	4.144		
1	1	0.300	1.124	2.002	3.146	4.164		
2.25	1.5	0.326	1.138	2.036	3.168	4.176		
4	2	0.354	1.160	2.066	3.186	4.194		
6.25	2.5	0.380	1.176	2.092	3.206	4.212		
9	3	0.402	1.196	2.118	3.224	4.232		
12.25	3.5	0.426	1.214	2.144	3.243	4.238		
16	4	0.448	1.242	2.166	3.262	4.266		
25	5	0.486	1.286	2.216	3.304	4.306		
36	6	0.522	1.320	2.264	3.344	4.346		
49	7	0.548	1.340	2.316	3.382	4.368		
64	8	0.592	1.356	2.358	3.428	4.402		
81	9	0.624	1.374	2.402	3.442	4.436		
100	10	0.648	1.390	2.442	3.488	4.466		
121	11	0.682	1.418	2.490	3.518	4.508		
225	15	0.714	1.440	2.654	3.628	4.618		
400	20	0.724	1.466	2.724	3.742	4.724		
1440	38	0.788	1.474	2.756	3.956	4.934	4.454	4.004

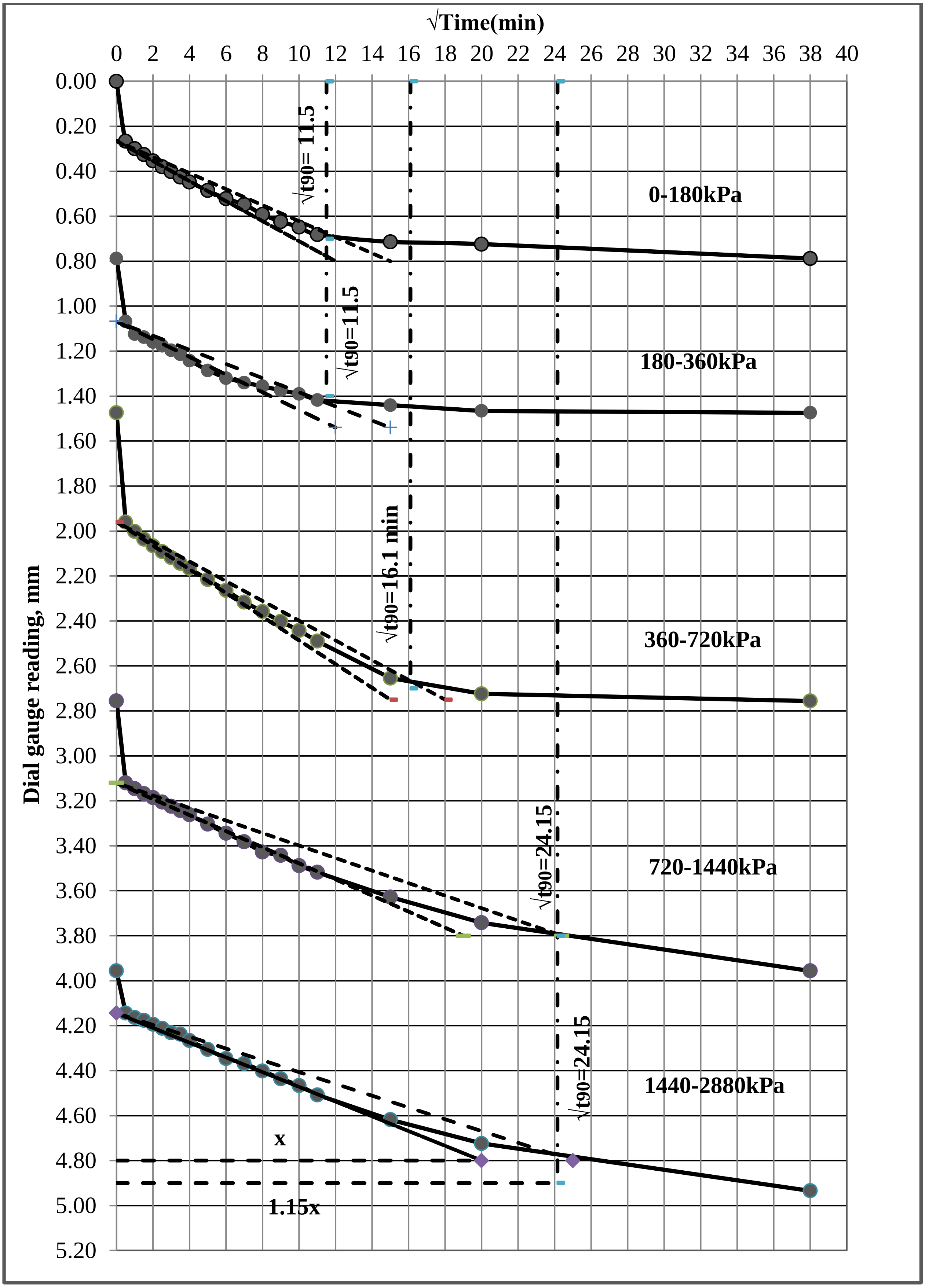

Figure G-3 Dial gauge reading versus √Time for test pit 4 @ 1.5*m*

Table G.3 Calculation of Coefficient of consolidation, C_v, *for test pit 4 @ 1.5m*

Pressure (*kPa*)	Cumulative compression (ΔH) *mm*	Consolidated height H=Ho-ΔH *mm*	Void Ratio e = (H-Hs)/Hs	t_{90} *min*	H = 1/2 ($H1$+$H2$)	Cv= (0.848*H^2)/ t_{90} m^2/year
180	0.79	19.21	1.28	132.25	19.61	1.295
360	1.47	18.53	1.19	132.25	18.87	1.200
720	2.76	17.24	1.04	259.21	17.9	0.550
1440	3.96	16.04	0.90	583.22	16.64	0.212
2880	4.93	15.07	0.78	583.22	15.56	0.185
720	4.45	15.55	0.84			
180	4.00	16.00	0.89			

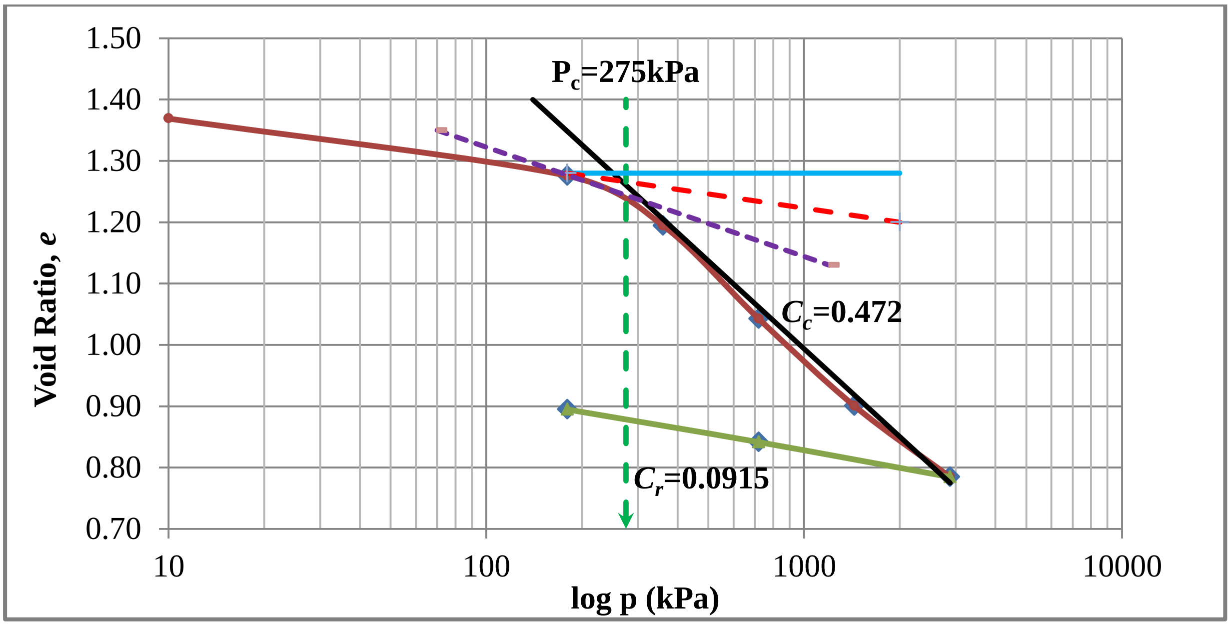

Figure G-4 *e-log p curve for test pit 4 @ 1.5m*

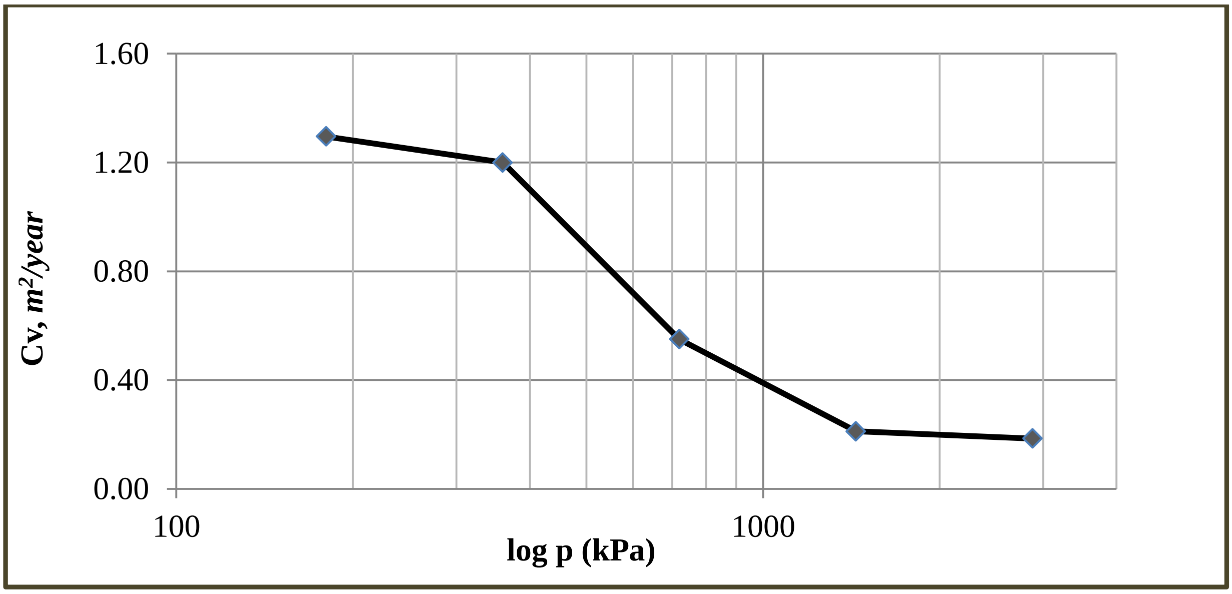

Figure G-5 C_v *versus log p for test pit 4 @ 1.5m*

Before Test Sample type **Remolded, Test pit 5 @ 1*m***

Weight of Sample & Ring (*g*)	235.40
Weight of Ring (*g*)	107.80
Weight of Sample (*g*)	127.60
Weight of Dry sample (*g*)	99.70
Weight of Initial Moisture (*g*)	27.90
Initial Moisture Content, M_o (%)	27.98
Initial Void Ratio : $e_o = (G_s/r_d)-1$	1.411
Height of Solids : $H_s = H_o/(1+e_o)$, (*mm*)	8.296
Initial Saturation: $S_o = (M_o x G_s)/e_o$, (%)	53.95

Specific Gravity (G_s)	2.72
Sample Diameter (*D*) (*mm*)	75.00
Sample Thickness (H_o) (*mm*)	20.00
Area (*A*) (cm^2)	44.18
Volume (*V*) (cm^3)	88.37
Bulk Density (r_b) (g/cm^3)	1.444
Dry Density (r_d) (g/cm^3)	1.128

After Test

Weight of Sample + Ring + Can (*g*)	240.50
Weight of Dry sample + Ring + Can (*g*)	207.50
Weight of Ring + Can (*g*)	107.80
Weight of Wet sample (*g*)	132.70
Weight of Dry sample (*g*)	99.70
Final Moisture Content, M_f (%)	33.10

Overall settlement (*mm*)	3.196
Volume change (cm^3)	14.12
Final Volume (cm^3)	74.25
Final Bulk Density (g/cm^3)	1.787
Final Dry Density (g/cm^3)	1.343

Sample Diameter (*D*) (*mm*)	75
Height of Solids : H_s=	8.296

Height H_o, (*mm*)	20
Initial Void Ratio : e_o=	1.41

Table G.4 Consolidation dial gauge reading for *test pit 5 @ 1m*

Time, min	√Time,min	Dial gauge reading in *mm*				
		50kPa	**100kPa**	**200kPa**	**400kPa**	**800kPa**
0	0	0.000	0.308	0.578	1.486	2.572
0.25	0.5	0.150	0.474	1.130	2.000	2.832
1	1	0.180	0.492	1.186	2.040	2.982
2.25	1.5	0.194	0.502	1.230	2.090	2.952
4	2	0.204	0.508	1.264	2.122	2.996
6.25	2.5	0.212	0.512	1.290	2.146	3.028
9	3	0.216	0.520	1.310	2.168	3.060
12.25	3.5	0.222	0.526	1.324	2.190	3.080
16	4	0.228	0.530	1.332	2.206	3.098
25	5	0.238	0.534	1.354	2.236	3.132
36	6	0.244	0.540	1.372	2.256	3.168
49	7	0.250	0.542	1.390	2.272	3.186
64	8	0.256	0.544	1.398	2.286	3.212
81	9	0.260	0.546	1.410	2.294	3.228
100	10	0.266	0.550	1.418	2.306	3.250
121	11	0.268	0.556	1.434	2.312	3.276
225	15	0.282	0.562	1.448	2.338	3.298
400	20	0.286	0.568	1.460	2.404	3.354
1440	38	0.308	0.578	1.486	2.572	3.652

Table G.5 Calculation of Coefficient of consolidation, C_v, *for test pit 5 @ 1m*

Pressure (*kPa*)	Cumulative compression (ΔH) *mm*	Consolidated height H=Ho-ΔH *mm*	Void Ratio e = (H-Hs)/Hs	t_{90} *min*	H = 1/2 ($H1$+$H2$)	C_v= (0.848*H^2)/ t_{90} m^2/year
50	0.31	19.69	1.37	55.88	19.85	3.142
100	0.58	19.42	1.34	47.61	19.56	3.581
200	1.49	18.51	1.23	26.78	18.97	5.988
400	2.87	17.13	1.06	160.02	17.82	0.885
800	3.65	16.35	0.97	259.21	16.74	0.482
200	3.42	16.58	1.00			
50	3.20	16.80	1.03			

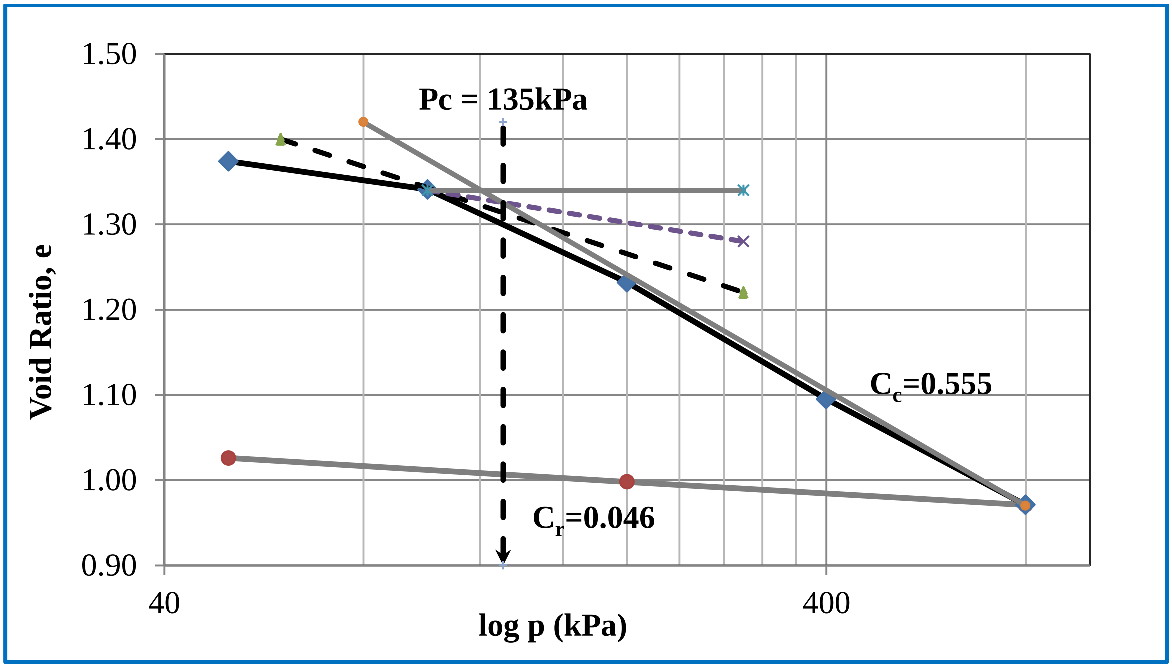

Figure G.6 *e-log p curve for test pit 5 @ 1m*

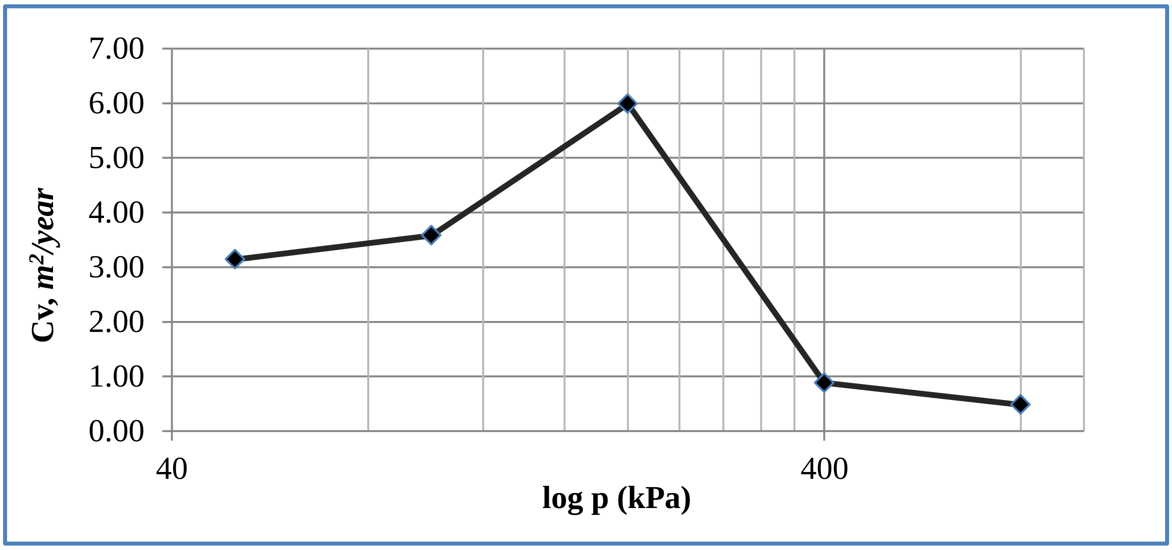

Figure G.7 C_v *versus log p for test pit 5 @ 1m*

Before Test Sample type **Remolded, Test pit 6 @ 1.5*m***

Weight of Sample & Ring (*g*)	123.90
Weight of Ring (*g*)	69.20
Weight of Sample (*g*)	54.70
Weight of Dry sample (*g*)	42.30
Weight of Initial Moisture (*g*)	12.40
Initial Moisture Content, M_o (%)	29.31
Initial Void Ratio : e_o= (G_s/r_d)-1	1.517
Height of Solids : H_s= H_o/(1+e_o), (*mm*)	7.947
Initial Saturation: S_o= (M_oxG_s)/e_o , (%)	52.38

Specific Gravity (G_s)	**2.71**
Sample Diameter (*D*) (*mm*)	50.00
Sample Thickness (H_o) (*mm*)	20.00
Area (*A*) (cm^2)	19.64
Volume (*V*) (cm^3)	39.28
Bulk Density (r_b) (g/cm^3)	1.393
Dry Density (r_d) (g/cm^3)	1.077

After Test

Weight of Sample + Ring + Can (*g*)	125.00
Weight of Dry sample + Ring + Can (*g*)	111.50
Weight of Ring + Can (*g*)	69.20
Weight of Wet sample (*g*)	55.80
Weight of Dry sample (*g*)	42.30
Final Moisture Content, M_f (%)	31.91
Sample Diameter (*D*) (*mm*)	50
Height of Solids : H_s=	7.947

Overall settlement (*mm*)	4.778
Volume change (cm^3)	9.38
Final Volume (cm^3)	29.90
Final Bulk Density (g/cm^3)	1.866
Final Dry Density (g/cm^3)	1.415

Height H_o , (*mm*)	20
Initial Void Ratio : e_o=	1.52

Table G.6 Consolidation dial gauge reading for test pit 6 @ 1.5*m*

Time, min	**√Time,min**	**Dial gauge reading in *mm***						
		180kPa	**360kPa**	**720kPa**	**1440kPa**	**2880kPa**	**720kPa**	**180kPa**
0	0	0.000	1.142	2.176	3.396	4.560	5.738	5.220
0.25	0.5	0.348	1.456	2.480	3.574	4.680		
1	1	0.382	1.496	2.510	3.596	4.698		
2.25	1.5	0.416	1.524	2.536	3.612	4.714		
4	2	0.450	1.546	2.554	3.63	4.734		
6.25	2.5	0.484	1.566	2.570	3.644	4.750		
9	3	0.510	1.584	2.586	3.658	4.772		
12.25	3.5	0.540	1.600	2.600	3.674	4.792		
16	4	0.572	1.616	2.614	3.69	4.810		
25	5	0.628	1.646	2.644	3.724	4.854		
36	6	0.682	1.678	2.674	3.757	4.902		
49	7	0.728	1.704	2.708	3.79	4.948		
64	8	0.776	1.730	2.738	3.832	5.008		
81	9	0.812	1.774	2.772	3.864	5.046		
100	10	0.848	1.814	2.806	3.910	5.086		
121	11	0.898	1.840	2.844	3.938	5.134		
225	15	1.010	1.956	2.969	4.064	5.288		
400	20	1.034	2.034	3.124	4.222	5.442		
1440	38	1.142	2.176	3.396	4.560	5.738	5.220	4.778

Square Root-Time Method (Taylor Method) to find C_v,(m^2/*year*) for test pit 6 @ *1.5m*

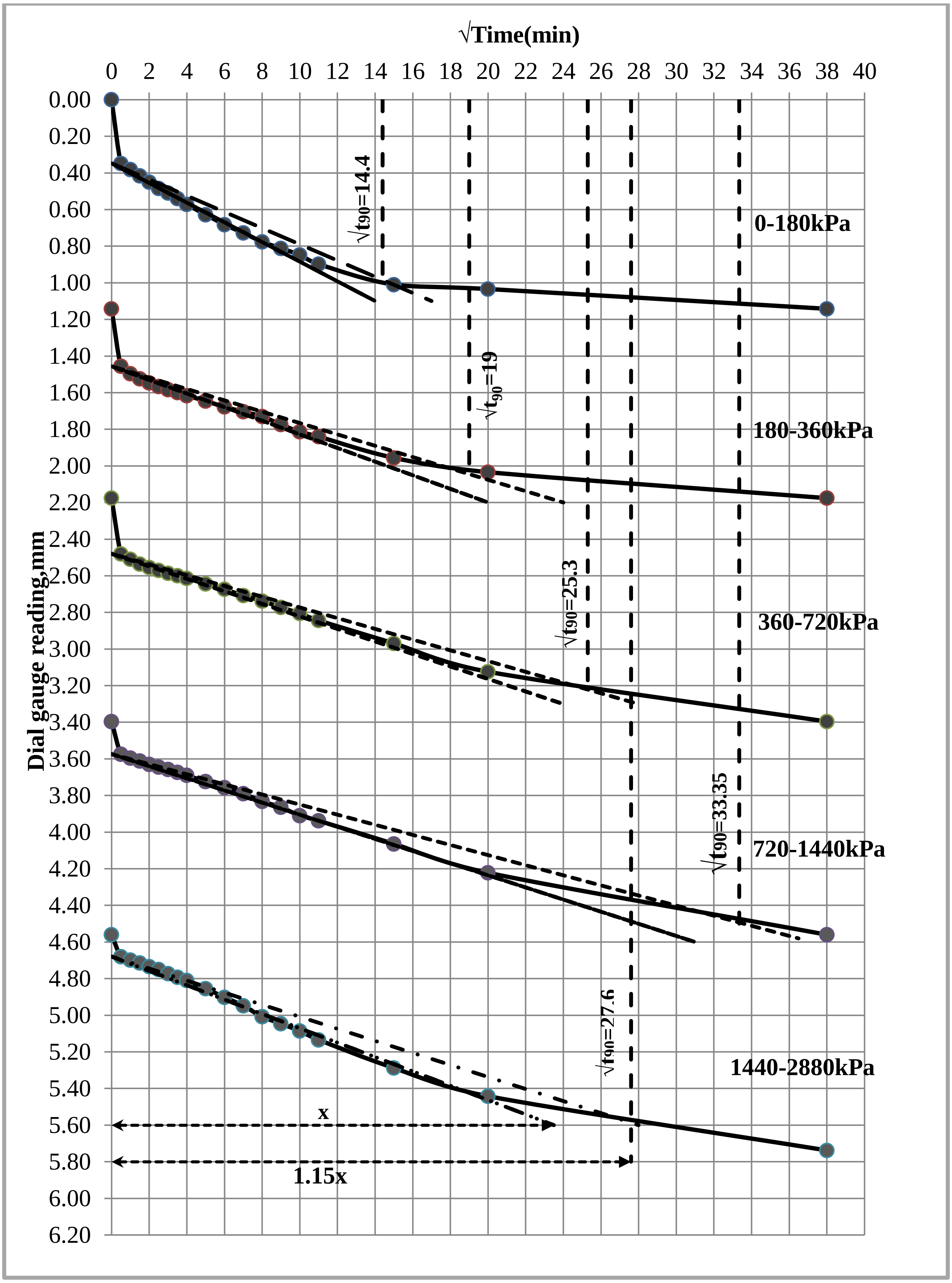

Figure G.8 Dial gauge reading versus √Time for test pit 6 @ 1.5*m*

Table G.7 Calculation of Coefficient of consolidation,C_v, for test pit 6 @ 1.5*m*

Pressure (*kPa*)	Cumulative compression (ΔH) *mm*	Consolidated height $H=Ho-\Delta H$ *mm*	Void Ratio $e = (H-Hs)/Hs$	t_{90} *min*	$H = 1/2 (H1+H2)$	$C_v = (0.848*H^2)/t_{90}$ m^2/year
180	1.14	18.86	1.37	206.64	19.43	0.814
360	2.18	17.82	1.24	360.05	18.34	0.416
720	3.40	16.60	1.09	640.09	17.21	0.206
1440	4.56	15.44	0.94	1112.22	16.02	0.103
2880	5.74	14.26	0.79	761.76	14.85	0.129
720	5.22	14.78	0.86			
180	4.78	15.22	0.92			

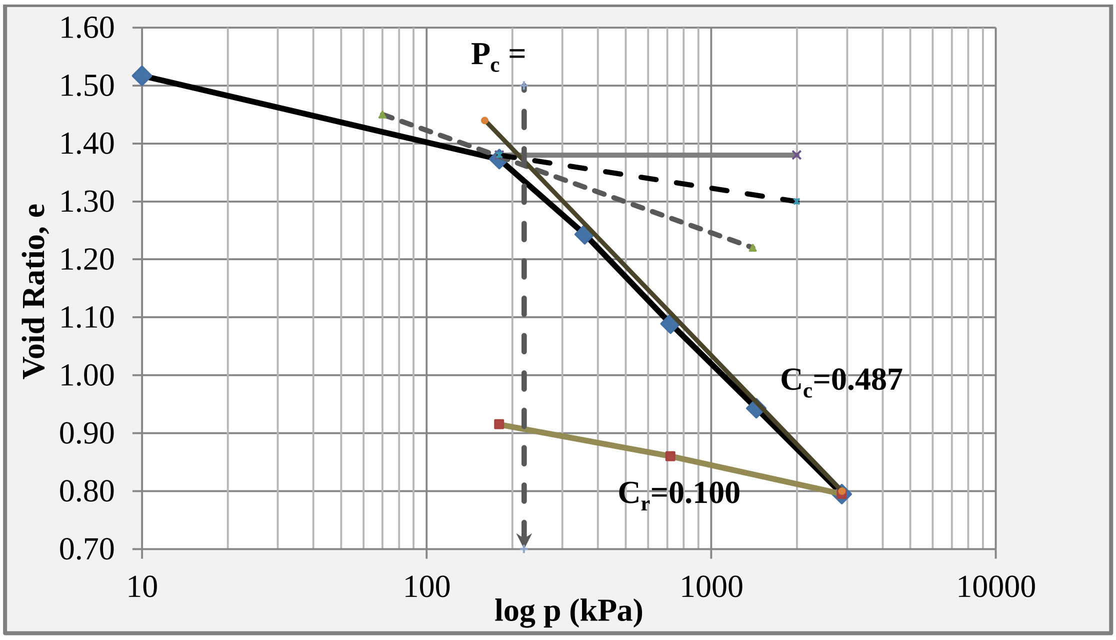

Figure G.9 *e-log p* curve for test pit 6 @ 1.5*m*

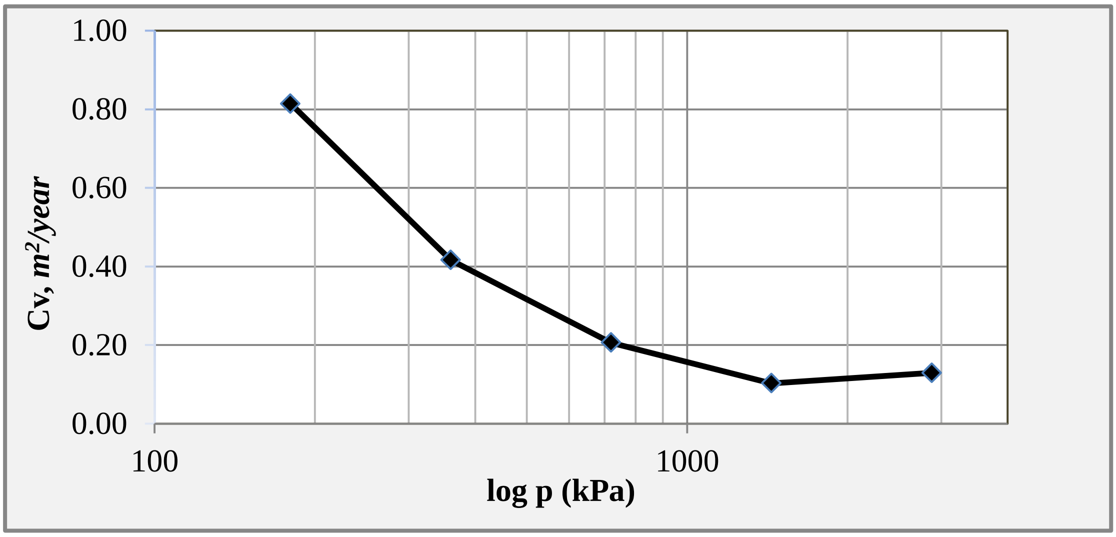

Figure G.10 C_v versus log p for test pit 6 @ 1.5m

Appendix H: Results for 2D crack monitor for 233 days monitoring period.

Table H.1 Reading date versus Deformation Building ID No. 26 at horizontal crack

Horizontal Crack	**Reading date**								
Across/Along the crack	**June 4, 2022**	**June 19, 2022**	**July 10, 2022**	**July 24, 2022**	**August 7, 2022**	**Sept. 3,2022**	**Oct. 15, 2022**	**Nov. 20, 2022**	**Jan. 22, 2023**
Across the crack A_1B_1	34.11	34.27	34.40	34.56	34.45	34.38	34.48	34.53	34.55
Along the crack A_1B_2	70.74	70.83	70.88	70.78	70.86	70.79	70.54	70.66	70.77
Change in deformation(Δd)	***Δdo***	***Δd1***	***Δd2***	***Δd3***	***Δd4***	***Δd5***	***Δd6***	***Δd7***	***Δd8***
Across the crack A_1B_1	0	0.16	0.29	0.45	0.34	0.27	0.37	0.42	0.44
Along the crack A_1B_2	0	0.09	0.14	0.04	0.12	0.05	-0.20	-0.08	0.03
Time (Days)	**0**	**15**	**36**	**50**	**64**	**92**	**134**	**170**	**233**

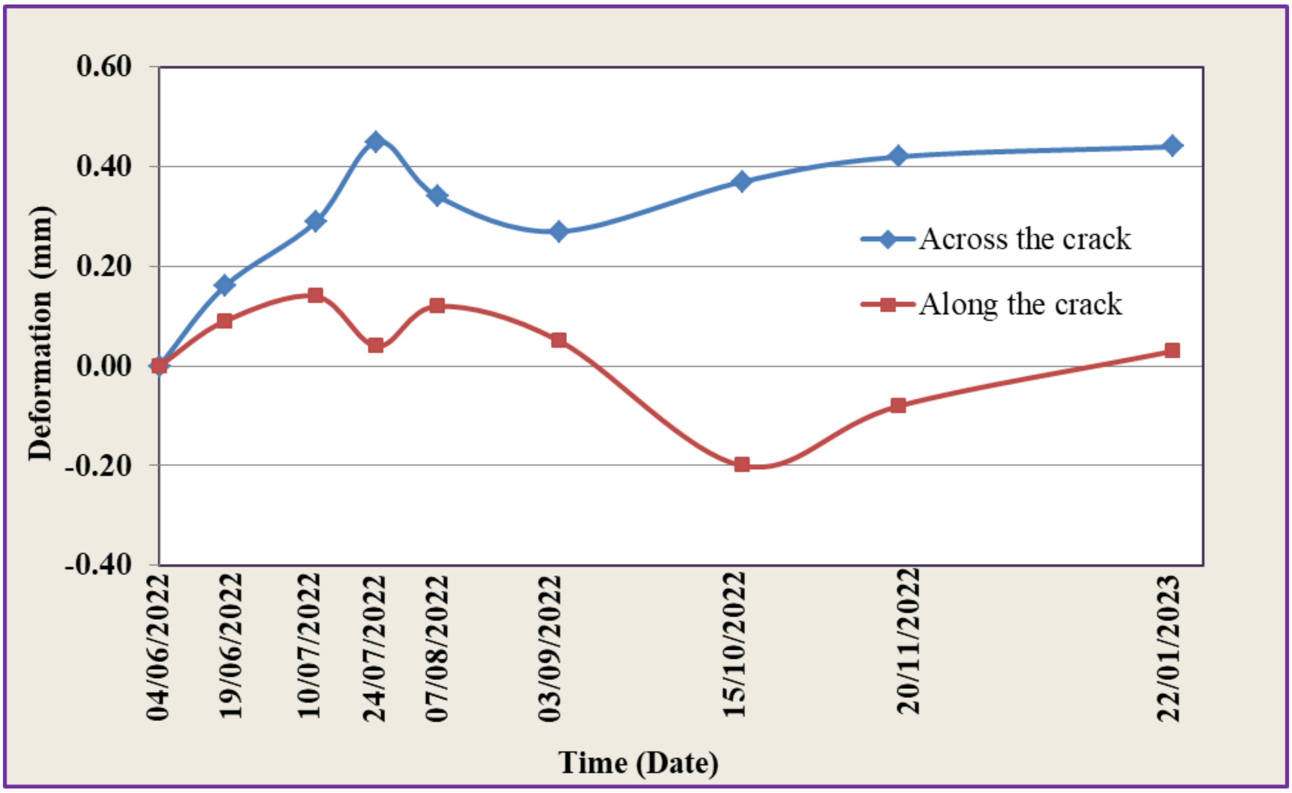

Figure H.1 Deformation versus Time Building ID No. 26 at horizontal crack

A 0.44mm crack opening was recorded on January 22, 2023.

Table H.2 Reading date versus Deformation Building ID No. 26 at diagonal crack

Diagonal Crack	**Reading date**								
Across/Along the crack	**June 4, 2022**	**June 19, 2022**	**July 10, 2022**	**July 24, 2022**	**August 7, 2022**	**Sept. 3, 2022**	**Oct. 15, 2022**	**Nov. 20, 2022**	**Jan. 22, 2023**
Across the crack A_1B_1	55.47	55.61	55.77	55.81	55.95	56.11	56.17	56.39	56.52
Along the crack A_1B_2	71.12	71.19	71.18	71.24	71.31	71.23	70.99	71.16	71.28
Change in deformation (Δd)	***Δdo***	***Δd1***	***Δd2***	***Δd3***	***Δd4***	***Δd5***	***Δd6***	***Δd7***	***Δd8***
Across the crack A_1B_1	0	0.14	0.30	0.34	0.48	0.64	0.70	0.92	1.05
Along the crack A_1B_2	0	0.07	0.06	0.12	0.19	0.11	-0.13	0.04	0.16
Time (Days)	**0**	**15**	**36**	**50**	**64**	**92**	**134**	**170**	**233**

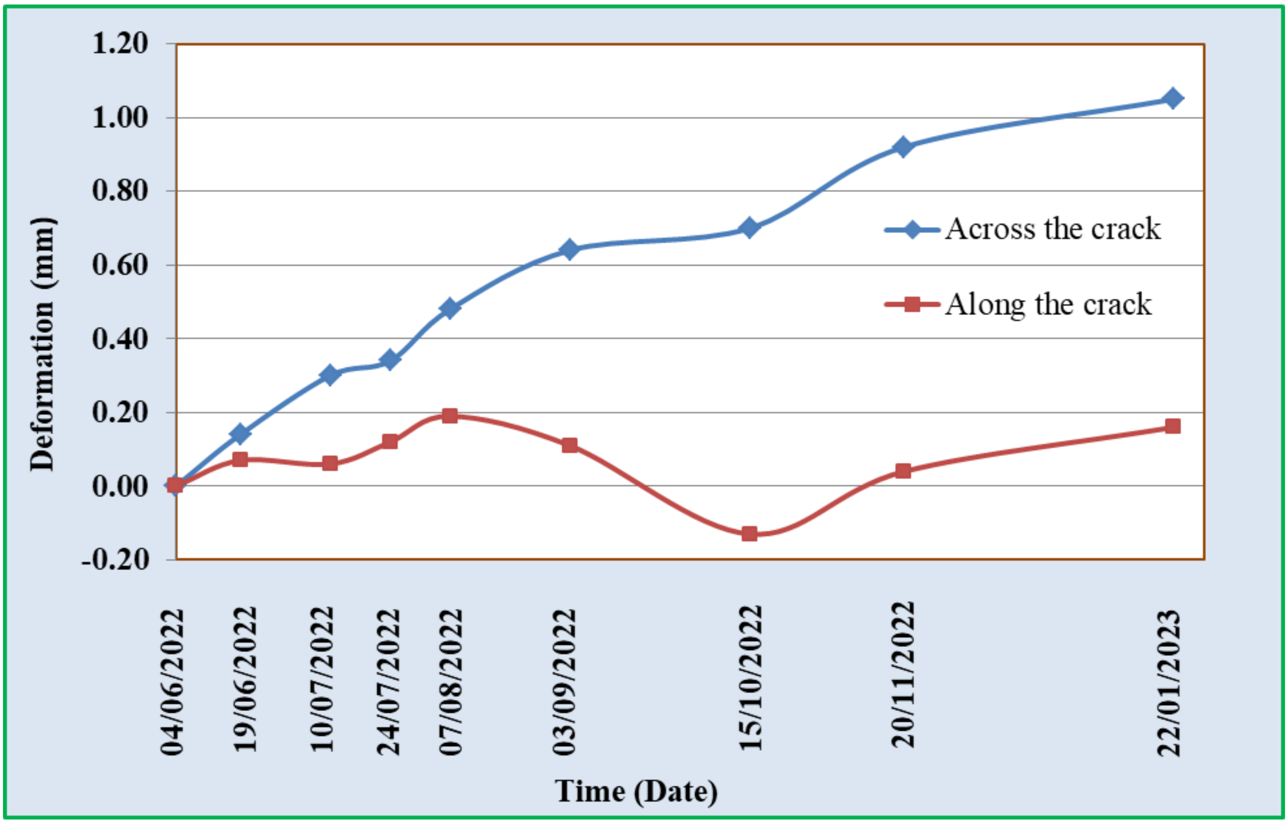

Figure H.2 Deformation versus Time Building ID No. 26 at diagonal crack

At the end of 233 days of monitoring period, a 1.05mm crack opening or across the crack movement was observed on Building ID No. 26 at diagonal crack.

Table H.3 Reading date versus Deformation Building ID No. 26 at vertical crack

Vertical Crack	**Reading date**								
Across/Along the crack	**June 4, 2022**	**June 19, 2022**	**July 10, 2022**	**July 24, 2022**	**August 7, 2022**	**Sept. 3, 2022**	**Oct. 15, 2022**	**Nov. 20, 2022**	**Jan. 22, 2023**
Across the crack A_1B_1	58.42	58.73	58.94	59.06	59.35	59.43	59.65	59.71	59.68
Along the crack A_1B_2	72.49	72.51	72.55	72.58	72.6	72.75	73.23	73.30	73.22
Change in deformation(Δd)	***Δdo***	***Δd1***	***Δd2***	***Δd3***	***Δd4***	***Δd5***	***Δd6***	***Δd7***	***Δd8***
Across the crack A_1B_1	0	0.31	0.52	0.64	0.93	1.01	1.23	1.29	1.26
Along the crack A_1B_2	0	0.02	0.06	0.09	0.11	0.26	0.74	0.81	0.73
Time (Days)	**0**	**15**	**36**	**50**	**64**	**92**	**134**	**170**	**233**

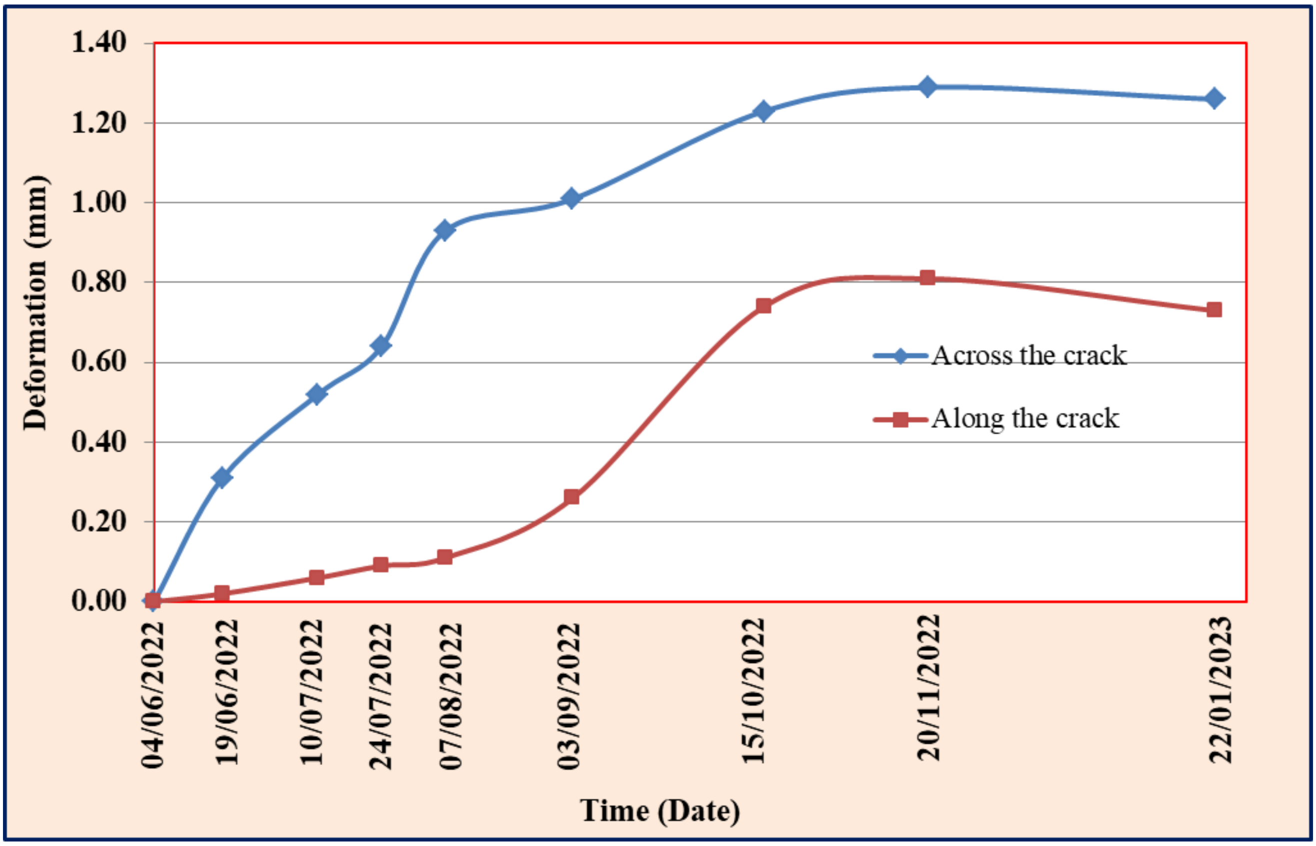

Figure H.3 Deformation versus Time Building ID No. 26 at vertical crack

During 233 days of monitoring period, additional crack opening on Building ID No.26 at vertical crack 1.29mm was observed.

Table H.4 Reading date versus Deformation Building ID No. 40 at vertical crack/door corner.

Vertical Crack/ Door corner	**Reading date**								
Across/Along the crack	**June 4, 2022**	**June 19, 2022**	**July 10, 2022**	**July 24, 2022**	**August 7, 2022**	**Sep. 3, 2022**	**Oct. 15, 2022**	**Nov. 20, 2022**	**Jan. 22, 2023**
Across the crack A_1B_1	30.43	30.34	30.60	30.49	30.38	30.20	30.24	30.32	30.67
Along the crack A_1B_2	67.52	67.47	67.55	67.49	67.42	67.36	67.44	67.49	67.65
Change in deformation(Δd)	***Δdo***	***Δd1***	***Δd2***	***Δd3***	***Δd4***	***Δd5***	***Δd6***	***Δd7***	***Δd8***
Across the crack A_1B_1	0	-0.09	0.17	0.06	-0.05	-0.23	-0.19	-0.11	0.24
Along the crack A_1B_2	0	-0.05	0.03	-0.03	-0.10	-0.16	-0.08	-0.03	0.13
Time (Days)	**0**	**15**	**36**	**50**	**64**	**92**	**134**	**170**	**233**

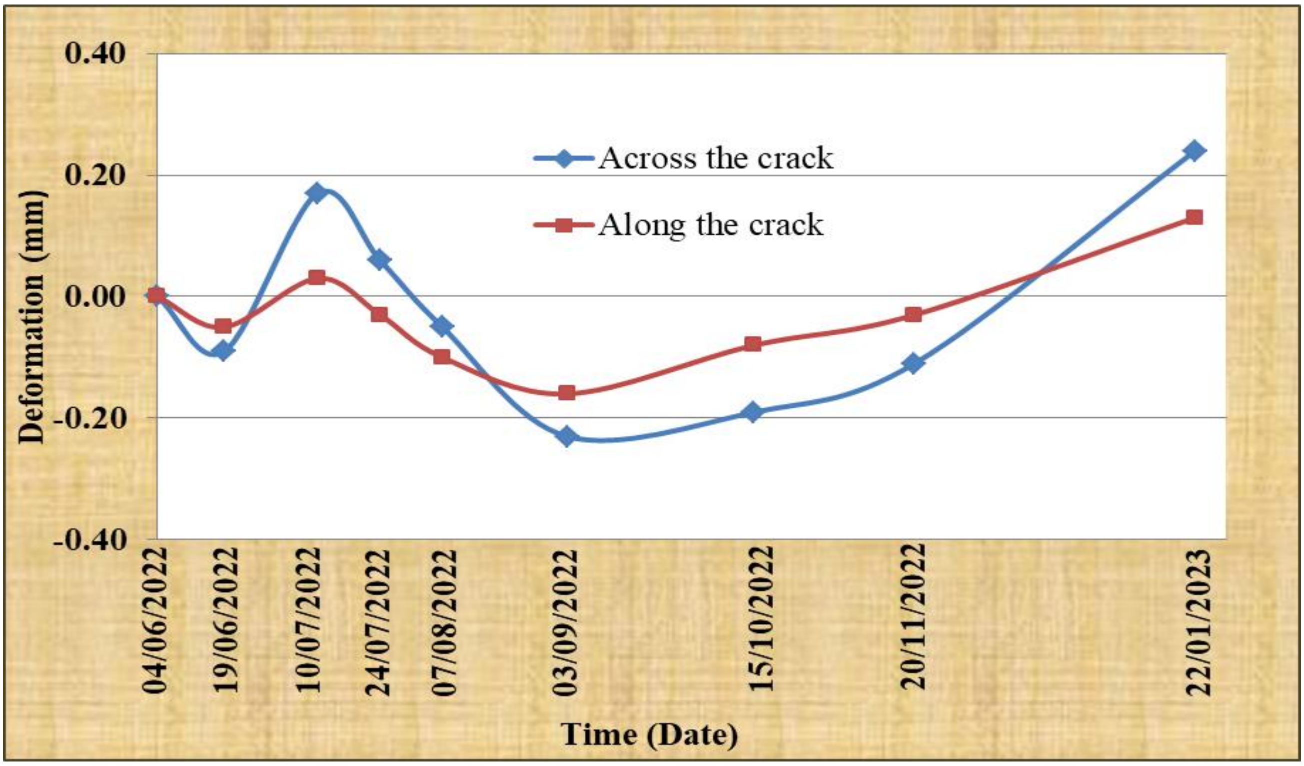

Figure H.4 Deformation versus Time Building ID No. 40 at vertical crack/door corner.

Depending on seasonal changes of Bishoftu town and shrink/swell behavior of expansive soil, opening and closing of crack was observed on building ID No. 40.

Table H.5 Reading date versus Deformation Building ID No. 40 at vertical crack/window corner.

Vertical Crack/ Window corner	**Reading date**								
Across/Along the crack	**June 4, 2022**	**June 19, 2022**	**July 10, 2022**	**July 24, 2022**	**August 7, 2022**	**Sept. 3, 2022**	**Oct. 15, 2022**	**Nov. 20, 2022**	**Jan. 22, 2023**
Across the crack A_1B_1	31.85	31.78	31.74	31.77	31.81	31.48	31.56	31.76	31.96
Along the crack A_1B_2	66.31	66.36	66.48	66.54	66.41	66.47	66.50	66.55	66.58
Change in deformation(Δd)	***Δdo***	***Δd1***	***Δd2***	***Δd3***	***Δd4***	***Δd5***	***Δd6***	***Δd7***	***Δd8***
Across the crack A_1B_1	0	-0.07	-0.11	-0.08	-0.04	-0.37	-0.29	-0.09	0.11
Along the crack A_1B_2	0	0.05	0.17	0.23	0.10	0.16	0.19	0.24	0.27
Time (Days)	**0**	**15**	**36**	**50**	**64**	**92**	**134**	**170**	**233**

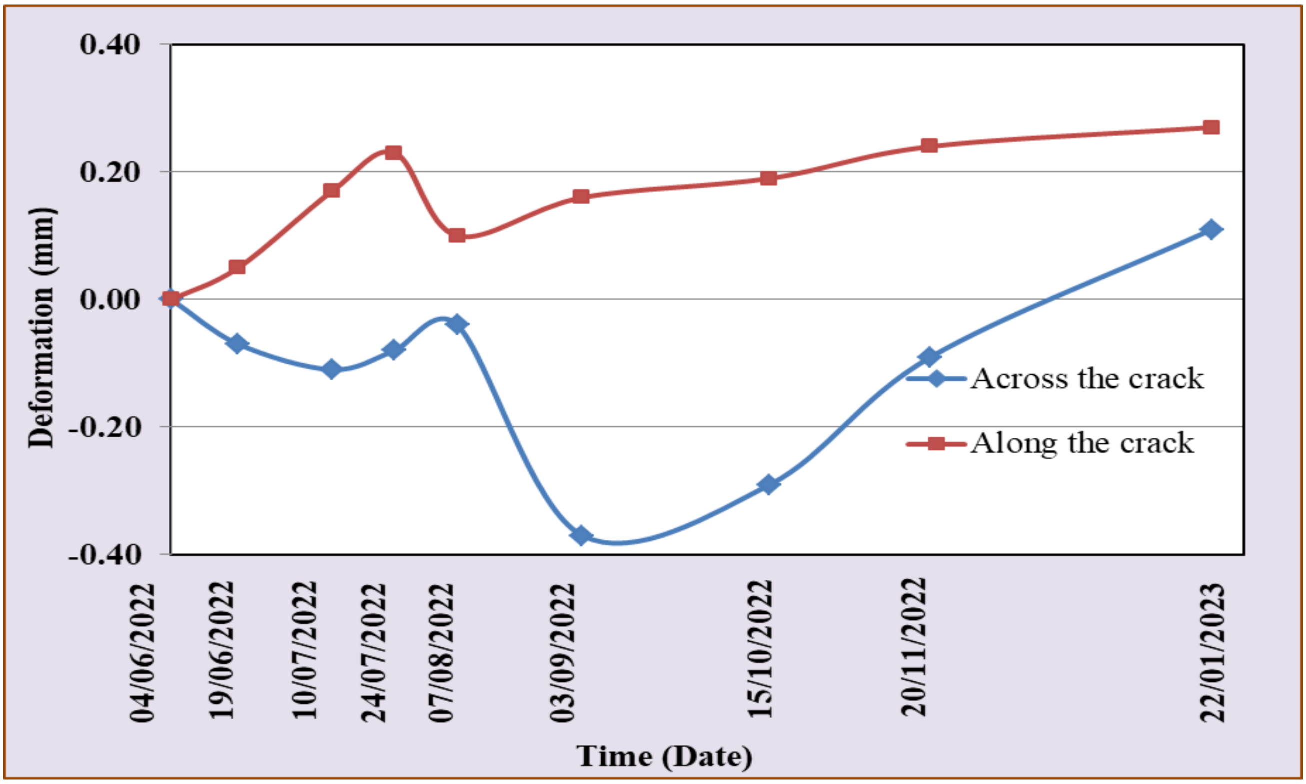

Figure H.5 Deformation versus Time Building ID No. 40 at vertical crack/window corner.

On September 3, 2022 a maximum crack closing (-0.37mm) was observed on Building ID No. 40 at vertical crack/window corner.

Appendix I: Standard format used to collect data from inspected buildings

Inspection Date:___________________

General Data

Building owner name

(Optional)____

Building location

Kebele__________ Northing ____________ Easting _____________ Elevation _________

Soil Data

Color of the soil:____________________

Liquid Limit (LL)___________________

Plastic Limit (PL)___________________

Plasticity Index (PI)__________________

Natural moisture content______________

Grain size analysis________________

MDD & OMC____________________

Clay content______________________

Specific gravity __________________

Free swell_______________________

Building Data

Type of building (No. of story)_________

Size of building_____________________

Foundation type_____________________

Foundation depth____________________

Building function _________________

Construction start date _____________

Construction end date______________

Construction Materials

Building element	**Construction materials**
Foundation	
Pavement around the building	
Floor	
Wall	
Ceiling	
Door	
Window	
Fence	

Extent of damage

Building element	**Damage description and its cause**
Foundation	
Pavement around the building	
Floor	
Wall	
Ceiling	
Door	
Window	
Fence	

Possible cause of damage

- ✓ Surface drainage condition

  Inside the compound

  ____

  Outside the compound

  ____

- ✓ Damaged utility lines/water pipes
- ✓ Trees and vegetation nearby the building
- ✓ Moisture fluctuation
- ✓ Improper foundation design
- ✓ Construction with low quality materials
- ✓ Others

Appendix J: AASHTO Soil classification ASTM D - 3282

Table J.1 Classification of Soils and Soil-Aggregate Mixtures

General Classification	Granular Materials (35% or less passing No. 200)							Silt-clay Materials (More than 35% Passing No.200)			
Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve analyses % passing											
No. 10 (2.00mm)	50 max										
No. 40 (425um)	30 max	50 max	51 min								
No. 200 (75um)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40 (425um)											
Liquid limit				40max	41min	40max	41min	40max	41min	40max	41min
Plasticity index	6 max		NP	10max	10max	11 min	11 min	10max	10max	11 min	11 min
Usual types of significant constituent materials	Stone fragments gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good					Fair to Poor					

Plasticity index of **A-7-5** subgroup is equal to or less than *LL minus* 30. Plasticity index of **A-7-6** subgroup is greater than *LL* minus 30.

Appendix K: Unified Soil Classification System (*USCS*) *ASTM D - 2487*

Fine grained soils (50% or more of material is smaller than No. 200 sieve size)

SILTS AND CLAYS Liquid limit less than 50%		ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravel clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silty clays of low plasticity
SILTS AND CLAYS Liquid limit 50% or greater		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SILTS		PT	Peat and other highly organic soils

Appendix L: Summary data of all inspected buildings

Bldg. ID No.	Bld Type	Construction Material							Description of Damage							Possible cause of damage
		Foundation	Pavement around bldgs	Floor/ Slab	Wall	Ceiling	Door & Window	Fence	Foundation	Pavement around bldgs	Floor	Wall	Ceiling	Door & Window	Fence	
1	Mud	Wooden post + Msnry	No pavement	Concrete + screed	Mud	Abujade	Metal	Msnry +HCB	Wooden post settle	No pavement	Crack in diff. direc	Moderate Crack	Slightly affected	No Damage	Cracked Msnry+ HCB	Moisture fluctuation, Poor subsurface drainage
2	G+0	Msnry	Concrete + Teraz	Concrete + Ceramic	HCB	Gypsum	Metal + wood	No fence	Foundation movement	Crack in diff. direc	Crack in diff. direc	Moderate Crack	Slightly affected	Not open & closed properly	No fence	Moisture fluctuation, Poor subsurface drainage
3	G+0	Msnry	Concrete + Teraz	Concrete + Ceramic	HCB	Gypsum	Metal + wood	Msnry +HCB	Edge Heave condition	*V* & *H* separation	Crack in diff. direc	Moderate Crack	Affected	Not open & closed properly	Cracked Msnry+ HCB	Trees close to the bldgs
4	G+0	Msnry	No pavement	Concrete + Ceramic	HCB	Gypsum	Metal	Msnry +HCB	No Damage	No pavement	Crack in diff. direc	Slight crack	No Damage	No Damage	Cracked Msnry+ HCB	Moisture fluctuation,
5	G+0	Select + Shear wall Beam	Concrete	Concrete + Ceramic	HCB	Gypsum	Metal	Msnry +HCB	No Damage	Fine cracks	No Damage	Fine cracks	No Damage	No Damage	No Damage	Not Damaged
6	G+1	Isolated footing	Concrete + Teraz	Solid Slab+ Ceramic	HCB	Concrete	Aluminium	HCB	No Damage	Fine cracks	No Damage	No Damage	No Damage	No Damage	No Damage	Not Damaged
7	G+2	Isolated footing	Concrete	Solid Slab+ Ceramic	HCB	Concrete	Metal + wood	HCB	Foundation movement	*V* & *H* separation	No Damage	Moderate Crack	No Damage	No Damage	Cracked HCB	Down pipe not installed properly, Trees close to bldgs
8	Mud	Wooden post + Msnry	No pavement	Concrete + screed	Mud	Chipwood	CIS	CIS	Wooden post settle	No pavement	Crack in diff. direc	Moderate Crack	Slightly affected	No Damage	Affected	Moisture fluctuation
9	G+0	Msnry	No pavement	Only Concrete	HCB	No ceiling	Metal + wood	Msnry +HCB	No Damage	No pavement	Crack in diff. direc	Moderate Crack	No ceiling	No Damage	Cracked Msnry+ HCB	Improper foundation design
10	G+0	Msnry	No pavement	Concrete + screed	HCB	Chipwood	Metal + wood	Msnry +HCB	Foundation movement	No pavement	Crack in diff. direc	Severe Crack	Slightly affected	Not open & closed properly	No Damage	Poor subsurface drainage

11	G+0	Msnry	No pavement	Concrete + screed	HCB	Abujade	Metal	Msnry +HCB	Cracked foundation msnry	No pavement	Crack in diff. direc	Moderate Crack	No Damage	Not open & closed properly	Cracked Msnry+ HCB	Improper foundation design
12	G+0	Msnry	Stone	Concrete + Terrazzo	HCB	Chipwoo d	Metal	Msnry +HCB	No Damage	Crack in diff. direc	Crack in diff. direc	Moderate Crack	Slightly affected	No Damage	No Damage	Down pipe not installed properly
13	G+2	Isolated footing	Concrete	Solid Slab	HCB	Gypsum	Metal	Msnry +HCB	No Damage	No Damage	No Damage	Fine cracks	No Damage	No Damage	No Damage	Not Damaged
14	Mud	Wooden post + Msnry	No pavement	Compacte d soil	Mud	Abujade	Metal	No fence	Cracked foundation msnry	No pavement	Crack in diff. direc	Moderate Crack	No Damage	Not open & closed properly	No fence	Ground water fluctuation,
15	Mud	Wooden post + Msnry	No pavement	Compacte d soil	Mud	Abujade	CIS	No fence	Wooden post settle	No pavement	Crack in diff. direc	Moderate Crack	No Damage	Not open & closed properly	No fence	Poor subsurface drainage
16	G+0	Msnry	No pavement	Concrete + screed	HCB	Chipwoo d	Metal	No fence	Cracked foundation msnry	No pavement	Crack in diff. direc	Moderate Crack	Slightly affected	No Damage	No fence	Improper foundation design
17	G+0	Msnry	Concrete	Concrete + Ceramic	HCB	Gypsum	Metal + wood	HCB	No Damage	Slight crack	No Damage	No Damage	No Damage	No Damage	No Damage	Not Damaged
18	G+0	Msnry	No pavement	Concrete + Ceramic	HCB	Gypsum	Metal	HCB	Cracked foundation msnry	No pavement	Crack in diff. direc	Severe Crack	Slightly affected	Not open & closed properly	Cracked HCB	Improper foundation design
19	G+2	Isolated footing	Concrete	Solid slab	HCB	Concrete	Alumi nium	HCB	Foundation movement	Crack in diff. direc	Crack in diff. direc	Moderate Crack	No Damage	Glass broken	Cracked HCB	Moisture fluctuation, Poor surface drainage
20	Mud	Wooden post+Msn ry	Concrete	Compacte d soil	Mud	No ceiling	Metal	No fence	No Damage	Slight crack	No Damage	Fine cracks	No ceiling	No Damage	No fence	Not Damaged
21	Mud	Wooden post + Msnry	No pavement	Only Concrete	Mud	Abujade	Metal	CIS	Wooden post settle	No pavement	No Damage	Severe Crack	No Damage	Not open & closed properly	Affected	Poor subsurface drainage
22	Mud	Wooden post + Msnry	No pavement	Concrete + screed	Mud	Abujade	Wood	No fence	Wooden post settle	No pavement	Crack in diff. direc	Severe Crack	No Damage	Not open & closed properly	No fence	Ground water fluctuation,
23	G+0	Msnry	No pavement	Concrete + screed	HCB	Gypsum	Metal	Msnry +HCB	Settle foundation msnry	No pavement	Crack in diff. direc	Severe Crack	Slightly affected	Not open & closed properly	Cracked Msnry+ HCB	Improper foundation design

24	G+0	Msnry	Concrete	Concrete + Ceramic	HCB	Gypsum	Metal	Msnry +HCB	No Damage	Slightly affected	Crack in diff. direc	Slight crack	Slightly affected	No Damage	Cracked Msnry+ HCB	Improper foundation design
25	G+0	Msnry	No pavement	Concrete + screed	HCB	No ceiling	Metal	Msnry +HCB	Cracked foundation msnry	No pavement	Crack in diff. direc	Severe Crack	No ceiling	Not open & closed properly	Cracked Msnry+ HCB	Water table fluctuation,
26	G+2	Isolated footing	No pavement	Solid Slab	HCB	No ceiling	Not install ed	Msnry +HCB	Foundation movement	No pavement	Crack in diff. direc	Severe Crack	No ceiling	Not Installed	Leaning walls	Moisture fluctuation, Poor subsurface drainage
27	Mud	Wooden+ Msnry	No pavement	Concrete + screed	Mud	Abujade	Metal	CIS	Cracked foundation msnry	No pavement	Crack in diff. direc	Very Severe Crack	No Damage	Not open & closed properly	No Damage	Poor subsurface drainage
28	G+0	Msnry	Concrete + Teraz	Concrete + Ceramic	HCB	Gypsum	Metal	Msnry	Settle foundation msnry	Crack in diff. direc	Crack in diff. direc	Severe Crack	Slightly affected	Not open & closed properly	Cracked Msnry	Trees close to bldgs, Poor subsurface drainage
29	G+0	Msnry	No pavement	Only Concrete	HCB	Abujade	Metal	CIS	Cracked foundation msnry	No pavement	Crack in diff. direc	Moderate Crack	No Damage	Not open & closed properly	Affected	Poor subsurface drainage
30	G+0	Msnry	No pavement	Only Concrete	HCB	No ceiling	Not install ed	No fence	No Damage	No pavement	Crack in diff. direc	Moderate Crack	No ceiling	Not Installed	No fence	Moisture fluctuation, Poor subsurface drainage
31	G+0	Msnry	Concrete + Teraz	Concrete + Ceramic	HCB	Gypsum	Metal	No fence	Cracked foundation msnry	Crack in diff. direc	Crack in diff. direc	Severe Crack	Slightly affected	Not open & closed properly	No fence	Improper foundation design
32	G+1	Isolated footing	Concrete + Teraz	Solid Slab	HCB	Gypsum	Metal	HCB	Foundation movement	Crack in diff. direc	Crack in diff. direc	Severe Crack	Slightly affected	Glass broken	Cracked HCB	Improper foundation design
33	G+2	Combined footing	Concrete + Teraz	Solid Slab	HCB	Gypsum	Alumi nium	HCB	No Damage	No Damage	No Damage	Fine cracks	No Damage	No Damage	Fine cracks	Not Damaged
34	G+0	Msnry	No pavement	Concrete + Ceramic	HCB	Gypsum	Metal	Msnry +HCB	Settle foundation msnry	No pavement	Slight crack	Severe Crack	Slightly affected	Not open & closed properly	Cracked Msnry+ HCB	Improper foundation design

35	G+0	Select + Shear wall Beam	Concrete + Teraz	Concrete + Ceramic	HCB	Gypsum	Metal + wood	Msnry +HCB	No Damage	No Damage	No Damage	Fine cracks	No Damage	No Damage	Slight cracks	Not Damaged
36	G+0	Msnry	No pavement	Concrete + screed	HCB	No ceiling	Metal	Msnry +HCB	Cracked foundation msnry + Gread Beam	No pavement	Slight crack	Very Severe Crack	No ceiling	Not open & closed properly	Cracked Msnry+ HCB	Improper foundation design, Poor subsurface drainage
37	G+0	Msnry	Concrete + Teraz	Concrete + Ceramic	HCB	Gypsum	Metal	Msnry +HCB	Cracked foundation msnry + Gread Beam	*V* & *H* separation	Crack in diff. direc	Severe Crack	No Damage	Not open & closed properly	Cracked Msnry+ HCB	Improper foundation design, Poor subsurface drainage
38	G+1	Isolated footing	No pavement	Solid Slab	HCB	Gypsum	Metal	Msnry +HCB	Foundation movement	No pavement	Slight crack	Moderate Crack	No Damage	No Damage	Cracked Msnry+ HCB	Poor subsurface drainage
39	G+2	Combined footing	Concrete + Teraz	Solid Slab	HCB	Gypsum	Alumi nium	Msnry +HCB	No Damage	No Damage	No Damage	Fine cracks	No Damage	No Damage	No Damage	Not Damaged
40	Mud	Wooden post+ Msnry	Stone	Concrete + Terrazzo	Mud	Chipwoo d	Metal + wood	Msnry +HCB	Edge Heave condition	Crack in diff. direc	Crack in diff. direc	Moderate Crack	Slightly affected	Not open & closed properly	No Damage	Trees close to bldgs, Poor subsurface drainage
41	G+0	Msnry	Stone	Concrete + screed	HCB	Gypsum	Metal + wood	Msnry +HCB	No Damage	Crack in diff. direc	Crack in diff. direc	Slight crack	Slightly affected	No Damage	No Damage	Poor subsurface drainage
42	Mud	Wooden post+ Msnry	No pavement	Concrete + screed	Mud	Abujade	Metal	CIS	No Damage	No pavement	Slight crack	Slight crack	Slightly affected	No Damage	No Damage	Poor subsurface drainage
43	Mud	Wooden post+ Msnry	No pavement	Concrete + screed	Mud	Abujade	Wood	No fence	Cracked foundation msnry	No pavement	Crack in diff. direc	Severe Crack	Slightly affected	Not open & closed properly	No fence	Poor subsurface drainage
44	G+0	Msnry	No pavement	Concrete + screed	HCB	Chipwoo d	Metal	Msnry +HCB	No Damage	No pavement	Crack in diff. direc	Slight crack	Slightly affected	Not open & closed properly	No Damage	Water table fluctuation