ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

DEPARTMENT OF CIVIL ENGINEERING

INVESTIGATION OF DYNAMIC PROPERTY OF SOIL COMMONLY FOUND IN ARBA MINCH TOWN

A THESIS SUBMITTED TO THE ADDIS ABABA INSTITUTE OF TECHNOLOGY, SCHOOL OF GRADUATE STUDIES, OF ADDIS ABABA UNIVERSITY

IN PARTIAL FULFILLMENT OF THE REQUIRMENT FOR THE DEGREE OF MASTER SCIENCE IN CIVIL ENGINEERING

BY

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2013

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DECLARATION

I hereby declare that the work which is being presented in this thesis entitles "**INVESTIGATION OF DYNAMIC PROPERTY OF SOIL COMMONLY FOUND IN ARBA MINCH TOWN**" is original work of my own, has not been presented for a degree in any other university and that all sources of material used for the thesis have been duly acknowledged.

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This is to certify that the above declaration made by the candidate is correct to the best of my knowledge.

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ACKNOWLEDGEMENTS

First and for most, praise and thanks be to the Almighty God, for giving me his grace and strength to finish this work.

I am deeply indebted to my adviser Prof. Alemayehu Teferra for his encouragement, supervision, and guidance he has provided me throughout my research. His critical comment and kind support gives me strength right from inception of the topic to the last minute of the research.

I would like to extend my gratitude to my instructors, professor Dr.Ing Alemayehu Teferra and

Dr. Samuel Tadesse for giving me all the basics and their suggestions during progress report.

I gratefully acknowledge to Arba Minch municipality offices and Arba Minch University for their help during sample collection.

Last but not least, I would like to thank my Brothers, my course Mate, AAiT geotechnical engineering laboratory people and friends for their genuine support, socialization, and help each other.

Content

SYMBOLS AND ABBREVIATION

AAiT	Addis Ababa Institute of Technology
AMU-CSc	Arba Minch University Community School
a (k)	Parameter related to plastic index
A_{loop}	Area of hysteresis loop
ASTM	American Society for testing and Materials
e	Void ratio of soils
e_{max}	Maximum void ratio or void ratio in the loosest state
e_{min}	Minimum void ratio or void ratio in the densest state
G	Dynamic shear modulus
G_{max}	Maximum Dynamic shear modulus
Gs	Specific gravity of soil
MDD	Maximum dry density
OCR	Over consolidation ratio
OMC	Optimum moisture content
PI	Plastic index
w	Moisture content of soil
W_D	Dissipated energy
W_s	Maximum strain energy
γ	Shear strain
γ_c	Shear strain at the tip of hysteresis loop
γ_s	Unit weight of soil
γ_d	Dry unit weight of soil
γ_w	Unit weight of water
β	Damping ratio
v_s	Shear wave velocity
ρ	Density of soil
σ	Normal stress
σm	Mean normal stress
τ	Shear stress
τc	Shear stress at the tip of hysteresis loop

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ABSTRACT

The nature and distribution of earthquake damage is strongly influenced by the response of soils to cyclic loading. The better understanding of the dynamic properties of some particular natural soil such as the Mexico City clays or the San Francisco Bay mud, together with the actual ground motion measurements during the Michoacan-1985 and the Loma-Prieta earthquakes, have been greatly contribute to show off the importance of local site conditions and dynamic properties of natural soils. Of those dynamic properties, the shear modulus and damping characteristics of cyclically loaded soils are critical to the evaluation of Geotechnical Engineering problems. Generally, soil is a nonlinear material which causes nonlinear seismic loading responses of grounds especially for earthquake ground motions corresponding to strain level ($\gamma \geq 0.01\%$).

In this thesis, the shear modulus and damping ratio values of soils commonly found in Arba Minch were determined using cyclic simple shear testing machine on remolded samples. The tests were conducted as a function of cyclic strain amplitude of 0.01 %, 0.1 %, 1 %, 2.5 %, and 5% under the axial pressures of 100kPa, 200kPa and 400kPa. The test results revealed that the shear modulus reduction values are in good agreement with curves of local soils but slightly lower than other established literature value at the highest strain level (γ >0.1 %). The damping ratio value of the tests are generally in a good agreement with curves of local soils but slightly lower than the other literature values. This indicates that, the testing conditions appear to have significant effect on the damping ratio values but little effect on the shear modulus reduction values.

The cyclic simple shear test machine, currently functional in the laboratory, is capable of reproducing earthquake stress condition accurately. Moreover, the index properties of the soil are determined for characterization of the soil in the town to setup schedules of dynamic property investigations in this research.

CHAPTER 1

INTRODUCTION

1.1 Background

The ground response under seismic loading is most important in geotechnical investigation. The ground motion due to earthquake may lead to permanent settlement, tilting of footing, collapse structures ...etc [3].

The nature and distribution of earthquake damage is strongly influenced by the response of soils to cyclic loading. The behaviour of soils subjected to seismic loading is influenced by the stiffness, damping, Poisson's ratio, plasticity index, effective stress and density of the soil [4]. Of these, the stiffness and damping characteristics of cyclically loaded soils are critical for the evaluation of many geotechnical engineering problems [4]. Arba Minch is located in Gamo Gofa zone, the Southern Nations, Nationalities and Peoples Region. Geographically, Arba Minch is located at the floor of the southern part of the East African Rift between 6030'N to 6008'N latitude and 37033'E to 37037'E longitude at an elevation of 1285 above sea level.

The proximity of significant earthquakes to the major population centres like Arba Minch obviously leads to the question of how much damage will be sustained by the buildings and other infrastructures constructed or being constructed. Therefore, it is necessary to understand the ground responses of the soil deposits in which their dynamic property play a crucial role [23].

The dynamic properties of soil in Arba Minch have not been investigated so far. In this thesis, the shear modulus and damping ratio values of soils commonly found in Arba Minch are to be investigated in the laboratory using the cyclic simple shear testing machine on remolded samples. In addition, some important properties of the soil tests have been determined for better characterization of the soils.

1.2 Objectives

1.2.1 General objective

The main objective of this research is to determine the dynamic shear modulus and damping ratio values of soils found in Arba Minch town using cyclic simple shear testing machine, to provide useful guide in the selection of soil characteristics for analysis purpose, and to make a comparison of the results with literature values.

1.2.2 Specific objective

- To determine the index properties and classify the soils
- To determine field density, maximum dry density, field moisture content and pre-consolidation pressure from both disturbed and undisturbed soil sample.
- To determine shear modulus and damping ratio of specimens under remolded to field conditions

1.3 Materials and Methods

To meet the above objectives, the following methodologies have been employed:

- Previous studies and papers related to dynamic properties of soils have been reviewed.
- Field densities were determined using core cutter
- field moisture contents of the soils have been determined
- Samples are collected from 1.5m and 3m depth for characterization of the soil.
- The following tests have been conducted to characterize the soils
    - Atterberg limit
    - Specific gravity
    - Compaction
    - Particle size analysis
    - One-dimensional consolidation and Free swell
- The soil is groped in to silt soil (Sikela town) and clay soil (Secha town)

- As soils in Arba Minch are more of silty and clay type, disturbed soil samples have been collected at 3 m depth from five selected test pits of the area.
- In the laboratory, samples have been remolded to field condition for cycling loading test
- From cyclic simple shear machine test results, the shear stress is calculated simply by dividing the sheer force by the area of the specimen base, and the shear strain is calculated by dividing the shear displacement by the height of the specimen.
- Shear modulus and damping ratio values of the soil then have been calculated.
- Finally the results have been compared to known literature values and discussed.

1.4 Scope of the study

This thesis is limited to cyclic simple shear testing, index properties and one dimensional consolidation tests. The transportation problem and the soil character make the retrieval of undisturbed soil samples for laboratory test difficult. Hence, disturbed soil samples will be collected from the test pits.

1.5 Organization of the Thesis

This thesis has six Chapters and Appendices. In the first Chapter contains background of the thesis, objectives, materials and methods and scope of the thesis are presented. Chapter two covers a literature review regarding dynamic soil parameters and their determination using cyclic simple shear testing machine. The third Chapter is about sample collection and test results. Chapter four is about data analysis and discussion of test results of cyclic simple shear test. In Chapter five, comparison of shear modulus reduction and damping ratio values with literature are presented. Chapter six contains conclusions and recommendations. Appendices which contain test results and graphs are presented after the reference.

CHAPTER 2

LITERATURE REVIEW

2.1 Dynamic soil properties

2.1.1 General

Required inputs for seismic ground response analysis include stiffness and material damping information for each soil type at the site in question. The current state of practice for evaluating the response of soil deposits under seismic loading conditions shows much progress in recent years. Successful application of analytical procedures for determining ground response in specific cases, however, is essentially dependent on the incorporation of representative soil properties in the analysis. Thus, considerable effort has also been directed toward the determination of soil properties for use in these analytical procedures.

Therefore, to obtain the maximum benefit from any method of seismic analysis, an understanding of the dynamic response characteristics of soil is essential. Shear wave velocity, shear modulus, damping ratio, stress history, plasticity index and Poisson's ratio are important mechanical properties of Soil that control the dynamic response of soil under cyclic loading [7].

It has long been recognized that local soil conditions can significantly affect the ground response when seismic waves propagate upward through a soil profile [6]. The better understanding of the dynamic properties of some particular natural soils such as the Mexico city clays or the San Francisco Bay mud, together with the actual ground motion measurements during the Michoacan-1985 and the Loma-Prieta-1989 earthquakes, have greatly contributed to show the importance of local site conditions and especially the specific dynamic properties (Gmax, strain dependent shear modulus and damping) of natural soils [8]. Thus, the shear modulus and damping characteristics of cyclically loaded soils are critical to the evaluation of Geotechnical Engineering problems that involve dynamic loading of soils and soil–structure interaction systems at a wider strain level [10].

2.1.2 Cyclic stress strain behaviour of soil

The stress strain behaviour of soil under dynamic loading depends on the nature of the soil, the environment of the soil (static stress state and water content); and the nature of the dynamic loading (strain magnitude, strain rate, and number of cycles of loading).

The effect of percentage of fines, material grading, and plasticity index (PI) and stress history are significant on dynamic property of soil for small strain level ($\gamma \leq 0.01\%$). However, for problems that involve medium to large strains ($\gamma > 0.01\%$) the effects of loading frequency (f) and the number of cycles (N) on stress strain property of soil are significant (Darendeli 2001) [20]. For large strain level ($\gamma > 0.1\%$) the kind of behaviour is termed degraded hysteresis type. The manner in which the shear modulus and damping ratio change with cycles is considered to depend upon the manner of change in the effective confining stress during irregular time histories of shear stress application [20].

The stress strain response of soil is non-linear for large strain but linear for small cyclic shear strain. Earthquake-induced stresses and strains that produce cyclic shearing of the soil are generally considered to be medium to large strain ($\boldsymbol{\gamma} \geq 0.01\%$) level [9]. Geotechnical earthquake engineering analysis assumes that earthquake ground motions are generated by vertically-propagated shear waves which cause cyclic shearing of the soil [9]. The stress-strain response of soil to this type of cyclic loading is nonlinear and hysteretic and commonly characterized by a hysteresis loop [9]. The most common model used to represent the hysteretic behavior of soil in seismic analysis is the equivalent-linear model [9]. The equivalent-linear model characterized non-linear hysteretic soil behavior using an equivalent shear modulus, and damping ratio [9].

A wide variety of procedures, including laboratory and field tests have been used to determine both shear modules and damping characteristics of soils. Previous Cyclic laboratory studies have shown that, cyclic response of saturated soils manifested by the deformation of the soil skeleton [10]. These deformations are mainly due to breakage of particle bond, slippage at the particle contacts, and corresponding change of micro-structural repulsion forces and can be expressed in terms of the shear strain of the soil [10]. Hysteretic stress-strain relationships under moderate to relatively high strains may be determined in the laboratory by means of triaxial compression tests, simple shear tests or torsional shear tests conducted under cyclic loading conditions. In this study shear modulus and damping factors were determined using cyclic shear tests.

2.2 Some important properties of soil

2.2.1 Physical States and Index Properties of soil

Soils are aggregates of mineral particles, and together with air and/or water in the void spaces, they form three-phase systems. A large portion of the earth's surface is covered by soils, and they are widely used as construction and foundation materials [14]. Index Properties are properties of a soil which help to classify the soil to assess the engineering behaviour of the soil under study. The physical properties which show the state of the soil are soil color, soil structure, texture, particle shape, grain specific gravity, water content, density index, in-situ unit weight, consistency limits, and particle size distribution, and related indices [10].

2.2.1.1 Particle shape and size

The particle shape of coarse-grained soils may be described as 'angular', 'sub-angular', 'sub-rounded', 'rounded' and 'well-rounded'. Silt and clay constitute the finer fractions of the soil and one grain of this fraction generally consists of only one mineral. Microscopic studies of clay and silt soil indicates that the particles are angular, flake-shaped or sometimes needle-like shape [14].

2.2.1.2 Specific gravity

The specific gravity of soil (Gs) is defined as the ratio of the mass in air of a given volume of soil particles to the mass in air of an equal volume of gas free distilled water at a stated temperature (20°C). The specific gravity is determined by means of a calibrated pycnometer, by which the mass and temperature of a de-aired soil/distilled water sample is measured [4]. The specific gravity of the soil grains have value in computing the void ratio, degree of saturation and particle size by wet analysis when the unit weight and water content are known. Typical value of specific gravity are presented in Table 2.1 below.

Table 2. 1 Specific gravity value of some soil types

S.No	Soil type	Grain specific gravity
1	Quartz sand	2.64 - 2.65
2	Silt	2.68-2.72
3	Silt with organic matter	2.40 - 2.50
4	Clay	2.44-2.92
5	Bentonite	2.34
6	Loess	2.65-2.75
7	Lime	2.7
8	Peat	1.26 - 1.80

2.2.1.3 Moisture content

Moisture content of a soil has a direct bearing on strength and stability of fine-grained soils. The knowledge of water content is necessary for classification, for correlation studies and for the calculation of stability of all kinds of earth works [12]. The water content of soil can influence the behavior of cyclically loaded soil specimen, as it controls grain to grain slippage and pore water pressure development.

2.2.1.5 Consistency of clay soil

Figure 2.3 demonstrates the change in soil states with water content. If clay slurry is dried, the moisture content will gradually decrease, and the slurry will pass from a liquid state to a plastic state. With further drying, it will change to a semisolid state and finally to a solid state. A. Atterberg (In 1911), developed a method for describing the limit consistency of fine-grained soils on the basis of moisture content. These limits are the liquid limit, the plastic limit, and the shrinkage limit [13].

The Atterberg limits and related indices have proved to be very useful for soil identification and classification. The limits are often used directly in specifications for controlling soil quality for use in fills and in semi empirical methods of design Soils for different construction use [13].

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. A quantitative classification is given in Table 2.2.

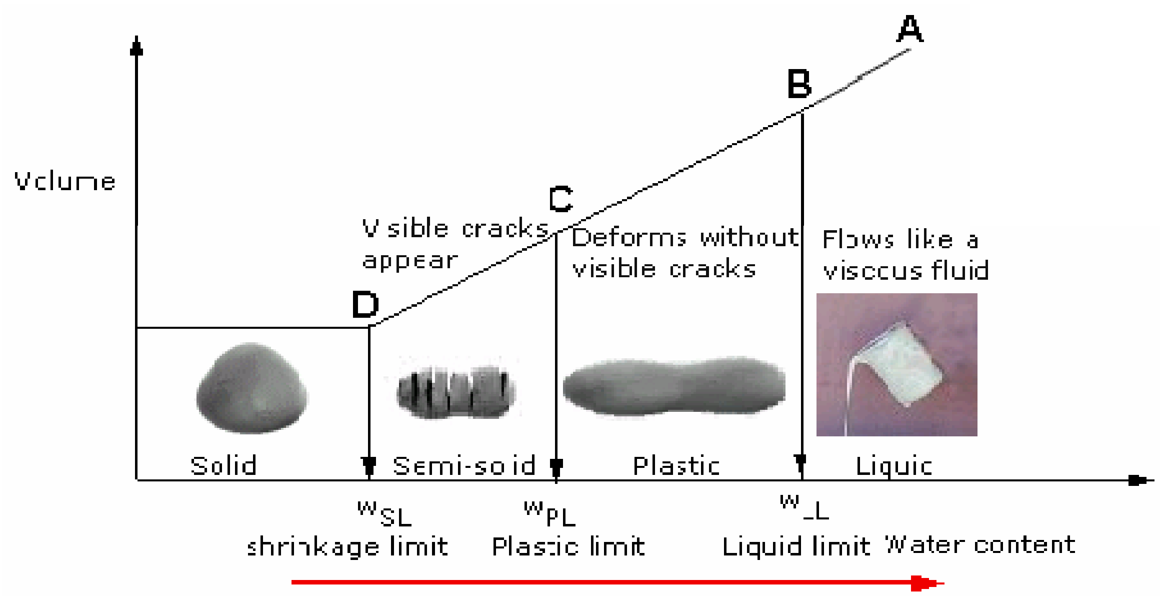


Figure 2. 1 Change in soil states as a function of soil volume and water content.

Table 2.2 Range of plasticity index with its consistency [12]

Plasticity index	plasticity
0	non – plastic
1 to 5	slight
5 to 10	low
10 to 20	medium
20 to 40	high
> 40	very high

2.2.1.6 In-situ unit weight

The in-situ unit weight refers to the unit weight of a soil in the undisturbed condition. The in-situ unit weight can be determined using either a sand-replacement or drive-cylinder method. According to ASTM-D 2937-94, drive-cylinder method is used to determine the in-place density of natural, inorganic soils which do not contain significant amount of particles coarser than 4.75 mm, and which can be readily retained in the drive cylinder. For this research, the Drive-cylinder method has been used to determine the field density of the soils in each test pits.

2.2.1.7 Grain size distribution

For a basic understanding of the nature of soil, the distribution of the grain size in a given soil mass should be known. The distribution of particle sizes larger than 75 μm is determined by sieving, while the distribution of particle sizes smaller than 75 μm is determined by a sedimentation process, using a hydrometer [12]. The grain-size distribution can be used to determine some of the basic soil parameters, such as the effective size, the uniformity coefficient, and the coefficient of gradation. Grain size distribution affects the seismic response in soil deposit [24].

2.2.1.8 Free swell

Some soils, particularly those clays containing montmorillonite, tend to increase their volume when their moisture content increases [2]. The amount of swelling and the magnitude of swelling are determined in the laboratory.

$$free\ swell = \frac{\text{Final volume} - \text{Initial volume of the soil}}{\text{Initial volume}} \text{X } 100\%$$

2.2.1.9 Compaction

Geotechnical engineers regularly recommend the highest practical soil compaction based on data correlating soil density by removal of air with increased mechanical strength [3]. Foundations of heavy buildings, highway roadbeds, and airport runways all require considerable levels of soil compaction for satisfactory performance [16]. Construction of earth-fill dams also involves heavy compaction to provide stable slope faces as well as a uniform and controlled rate of seepage through. In this study, test specimens have been compacted to field density and moisture content for cyclic shear testing to replicate the natural state of the soil samples.

2.2.1.10 One-Dimensional Consolidation Test

When a soil layer is subjected to a compressive stress, such as during the construction of a structure, it will exhibit a certain amount of compression. This compression is achieved through a number of ways, including rearrangement of the soil solids or extrusion of water [14]. The most often used method of consolidation testing is the one-dimensional consolidation test. In this study, the test specimens have been consolidated to determine the pre-consolidation pressure to use for simple cyclic shear test.

2.2.1.11 Soil Classification

A soil classification should permit the engineer to easily relate the soil description to its behavior characteristics. All soils are normally classified according to one of the following two systems.

Unified Soil Classification System (USCS): This system is used primarily for engineering purposes and is particularly useful to the Geotechnical Engineer. Therefore, they should be used for all structural-related projects; such as bridges, retaining walls, buildings, etc. Precise classification requires that a grain size analysis and Atterberg Limits tests be performed on the sample.

AASHTO Classification System: This system is used generally to classify soils for highway construction purposes and therefore will most often be used in conjunction with roadway soil surveys. Like the Unified System, this system requires grain size analysis and Atterberg Limit tests for precise classification

2.3 Shear modulus and Damping ratio

The stiffness and damping characteristics of cyclically loaded soil are critical to the evaluation of many geotechnical earthquake engineering problems at low strains ($\gamma \leq 10^{-3}$ percen^{t}), medium strains ($10^{-3} < \gamma < 10^{-1}$percent) and large strains level ($\gamma > 10^{-1}$ percent) [4].

2.3.1 Method of determining shear modulus and damping ratio

The selection of testing techniques for measurement of the shear modulus and damping ratio requires careful consideration and understanding of the specified parameters. There are various procedures, including laboratory and field tests that have been used to determine shear modulus and damping characteristics of soils. A summary of the procedure and the approximate range of strain within which they have been used are presented in the Table 2.3 [17].

Table 2.3 Test procedures for measuring moduli and damping characteristics [17]

general procedure	Test condition		
Determination of hysteretic stress-strain relationships	Triaxial compression	10^{-2} to 5 %	Shear Modulus; Damping
	Simple shear	10^{-2} to 5 %	Shear Modulus; Damping
	Tortional shear	10^{-2} to 5 %	Shear Modulus; Damping
Forced vibration	Longitudinal vibration	10^{-4} to 10^{-2}%	Shear Modulus; Damping
	Tortional vibration	10^{-4} to 10^{-2}%	Shear Modulus; Damping
	Shear vibration-lab	10^{-4} to 10^{-2}%	Shear Modulus; Damping
	Shear vibration-field	10^{-4} to 10^{-2}%	Shear Modulus
Free vibration tests	Longitudinal vibration	10^{-3} to 1 %	Shear Modulus; Damping
	Tortional vibration	10^{-3} to 1 %	Shear Modulus; Damping
	Shear vibration-lab	10^{-3} to 1 %	Shear Modulus; Damping
	Shear vibration-field	10^{-3} to 1 %	Shear Modulus
Field wave velocity Measurement	Compression wave	~5× 100^{-4} %	Shear Modulus
	Shear wave	~5× 100^{-4} %	Shear Modulus

2.3.2 Computation of Shear modulus and Damping ratio parameters

2.3.2.1 Shear modulus

Soil stiffness is represented by either shear-wave velocity or shear modulus. Because most soils have curvilinear stress-strain relationships, the tangent shear modulus (G_{tan}) varies through a cycle of loading but, its average value over the entire loop can be approximated by the secant shear modulus, G_{sec}, which is commonly called equivalent shear modulus (G) [4]. The relationship between maximum shear modulus (G_{max}), secant shear modulus (G_{sec}), shear strain ($\boldsymbol{\gamma}$), and shear stress (τ) is illustrated in Fig. 2.4. In addition, the figure shows the relationship between the stress–strain hysteresis loop for one cycle of loading and the material damping ratio. Using the equivalent-linear analysis method, the secant shear modulus can be determined by the extreme points on the hysteresis loop.

$$G_{sec} = \frac{\tau_c}{\gamma_c} \qquad \text{and} \qquad D = \frac{W_D}{4\pi W_S}$$

Where: τ_c is shear stress , γ_c is strain amplitudes at the tip and D is damping ratio

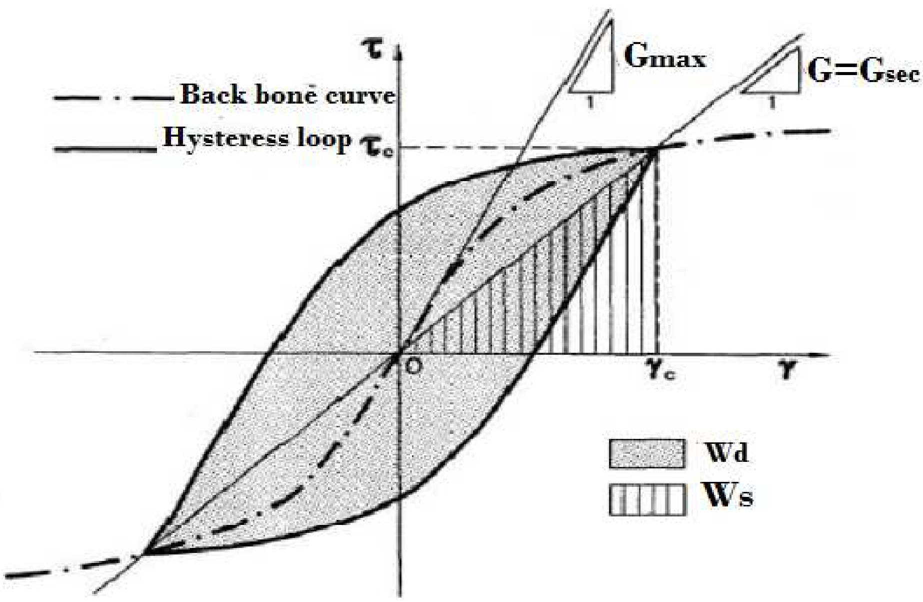


Figure 2. 2 Hysteretic stress-strain response of soil subjected to cyclic loading [9]

2.3.2.2 Damping ratio

The second key dynamic parameter for soils is damping. Two fundamentally different damping phenomena are associated with soils, namely material damping and radiation damping.

(i) Material damping

Material damping (or internal damping) in a soil occurs when any vibration wave passes through the soil. It can be thought of as a measure of the loss of vibration energy resulting primarily from hysteresis in the soil. Mechanisms that contribute to material damping are friction between soil particles, strain rate effect, and nonlinear soil behavior. As the soil elements loose stiffness with the amplitude of strain, its ability to dampen dynamic forces increases and damping decreases with confining pressure, void ratio, geologic age, and plasticity index and sometimes with cementation. [7]. The hysteretic damping ratio can be calculated by

$$D = \frac{W_d}{4\pi W_s}$$

Where W_d = energy dissipated in one cycle of loading, and Ws=maximum strain energy stored during the cycle. As noted in Fig.2.4, the area inside the hysteresis loop is Wd, and the area of the triangle is W_S. Theoretically, there should be no dissipation of energy in the linear elastic range for the hysteretic damping model. However, even at very low strain levels, there is always some energy dissipation measured in laboratory specimens. The damping ratio at very low strain levels is a constant value and is referred to as the small-strain damping ratio (D_{min}). At higher strains, nonlinearity in the stress–strain relationship leads to an increase in material damping ratio with increasing strain amplitude [18].

(ii) Radiation damping

Radiation damping is a measure of the energy loss from the structure through radiation of waves and it is a purely geometrical effect. The theory for the elastic half-space has been used to provide estimates for the magnitude of radiation damping, Whitman and Richart (1967) [2]. Radiation damping is frequency independent and only theoretical values for a particular type of footing and its usefulness may be for qualitative rather than quantitative assessments [2].

2.3.2.3 Normalized Shear Modulus and Material Damping Ratio Relationships

Normalizing shear modulus is a way to represent shear modulus degradation and expressed as (G/G_{max}). Characterization of the shear modulus of an element of soils requires consideration of both G_{max} and the manner in which the modulus ratio, G/G_{max}, varies with cyclic strain amplitude and other parameters. Small-strain shear-wave velocity (V_S) is directly related to small-strain shear modulus (G_{max}) by

$$G_{max} = \rho V_s^2$$

Where ρ =mass density of soil (total unit weight of the soil divided by the acceleration of gravity).

When shear wave velocity measurement are not available, G_{max} can be estimated in different ways. Laboratory test data suggest that, the maximum shear modulus can be expressed as [4]:

G_{max}= 625F (e)(OCR)^k(P_a^{1-n})(σ'_m)n^n (lb/ft^2)

where

OCR is over consolidation ratio of the soil

σ'_m is mean effective confining pressure

e is void ratio of the

P_a is atmospheric pressure in the same unit as σ'_m

n is often equal to 0.5

a = k are parameters that depends on the plasticity index of the soil

Figure 2.5, represents the strain-dependent shear modulus degradation factor (G/Gmax) and damping ratio (D) of saturated soils, [Vucetic, 1994].

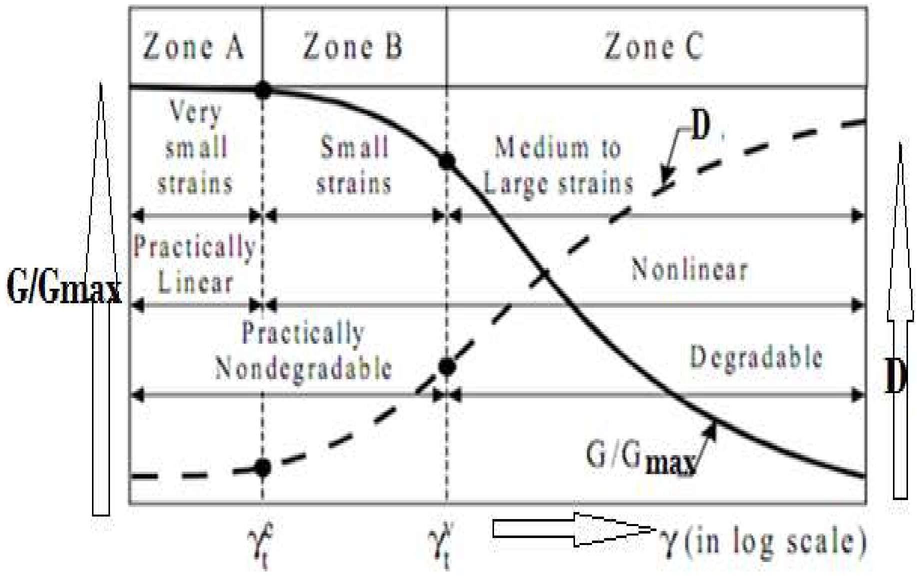


Figure 2. 3 Shear modulus reduction and damping ratio curves after [Vucetic, 1994] [9]

Table 2. 4 Typical soil behaviour

[Jardine, 1992]	[Vucetic, 1994]	Soil behaviour
Zone I	Zone A	Linear elastic
Zone II	Zone B	Non-linear elastic
Zone III	Zone C	Elastoplastic

γ_t^e is nonlinearity threshold shear strain γ_t^v is the volumetric threshold shear strain

2.3.3 Factors affecting Shear modulus and Damping ratio

The interest in the cyclic stress-strain characteristics of soil have already clarified in many aspects using extensive laboratory and field studies [14]. The Non-linear hysteretic behaviour of soil under cyclic loading dependent on some factors such as confining pressure, void ratio, geological age, cementation, overconsolidation and number of loading. Table 2.5 presents effect of each factor on shear modulus and damping ratio of soil.

Table2. 5 Effect of increase of various factors on G/Gmax, and D of normally consolidated and moderately over-consolidated clays [19]

Increasing factor	Maximum Shear modulus (G_{max})	Shear modulus degradation (G/G_{max})	Damping ratio (D)
Confining pressure (σ)	Increases with σ	Stays constant or increases with σ	Stays constant or decreases with σ
Void ratio (e)	Decreases with e	Increases with e	Decreases with e
Geological age (t)	Increases with t	May increase with t	Decreases with t
Cementation, (c)	Increases with c	May increase with c	May decrease with c
Overconsolidation (OCR)	Increases with OCR	Not affected	Not affected
Plasticity index (PI)	Increases with PI if OCR>1; stays about constant if OCR=1	increase with PI	Decrease with PI
Cyclic strain (γ_c)	-	Decreases with γ_c	Increases with γ_c
Number of loading Cycles (N)	Decreases after N cycles of large γ_c but recover later with time	Decreases after N cycles of large γ_c (G_{max} measured before N cycles)	Not significant for moderate γ_c and N

Figure 2.6 indicate that the rate of increasing damping ratio with strain becomes greater as the confining stress decreases. Similarly Figure 2.7 shows that the rate of reduction in shear modulus with strain becomes greater as the confining stress decreases.

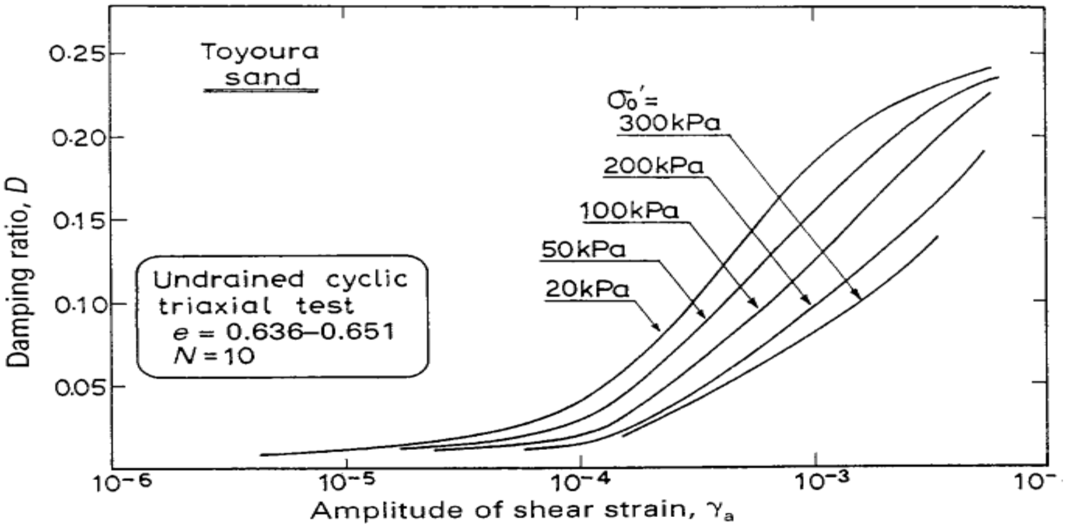


Figure 2.4 Effects of confining stress on the strain-dependent damping ratio [20].

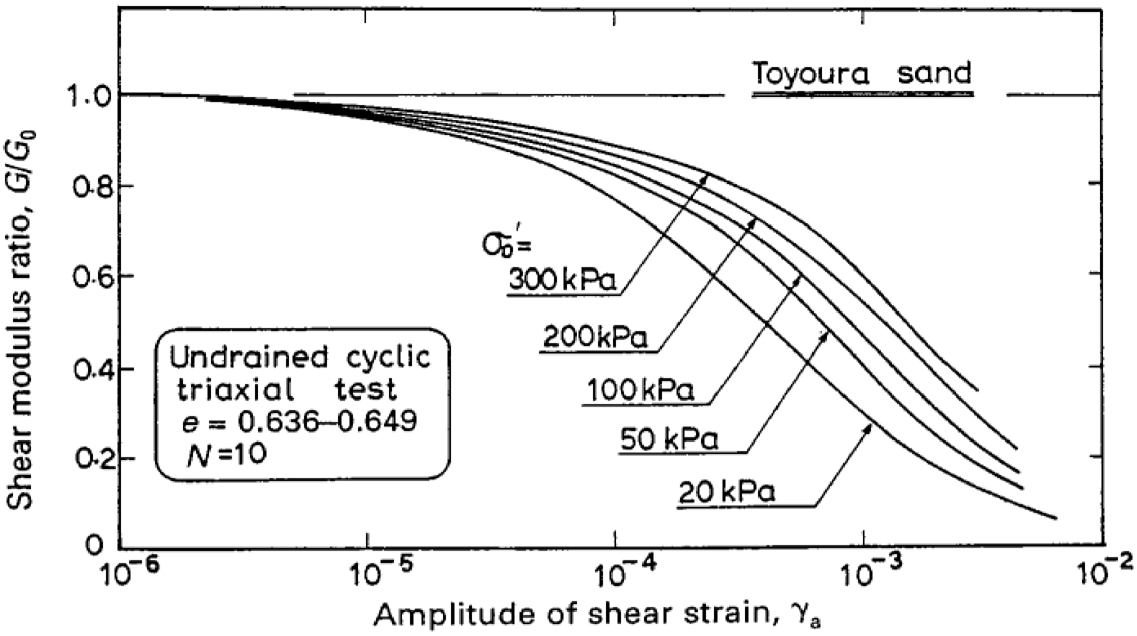


Figure 2. 5 Effects of confining stress on the strain-dependent shear modulus [20]

Figures 2.8 and 2.9 show that shear modulus and damping with strain is not significantly influenced by consolidation history. Thus whether a clay at a given site is in a normally consolidated state or in a state of overconsolidation, the modulus and damping ratio do decrease or increase with the same proportion over a wide range of shear strain.

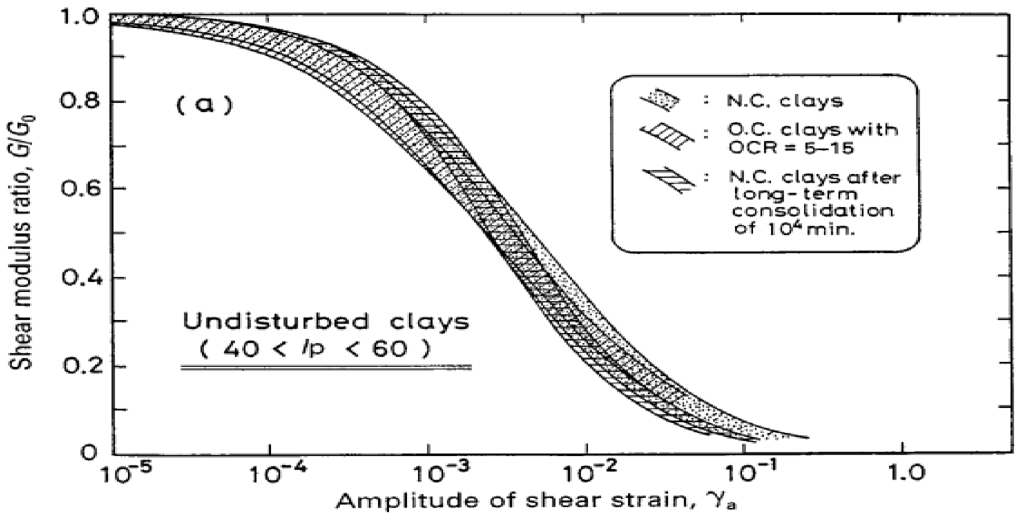


Figure 2.6 Effects of consolidation histories on strain-dependent modulus (Kokusho et al. 1982) [20]

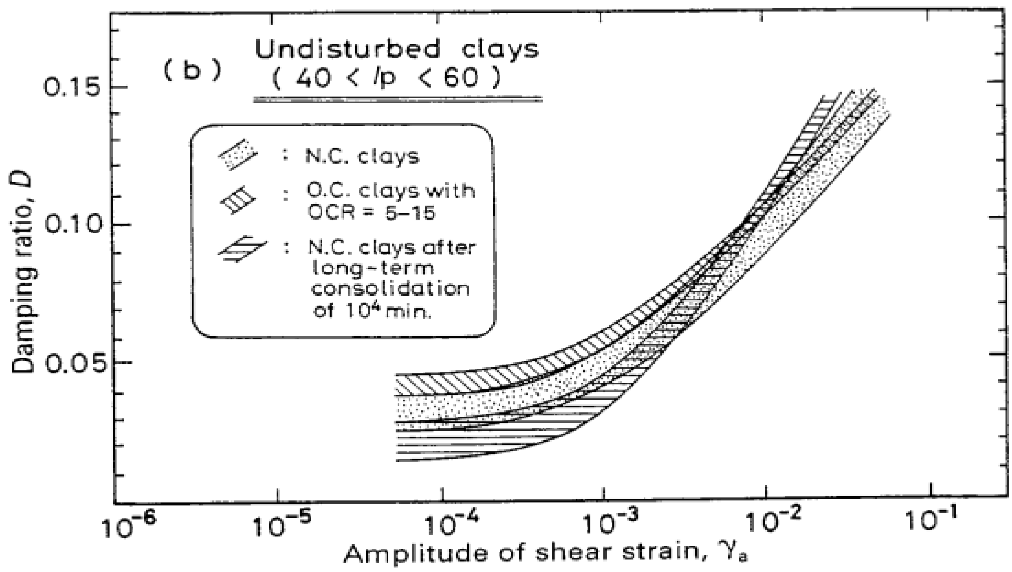


Figure 2.7 Effects of consolidation histories on strain-dependent damping ratio (Kokusho et al.1982) [20]

Figure 2.10 shows that for a given cyclic shear strain, as PI increases the value of G/G_{max} increases and damping ratio reduces. The soils having higher plasticity index tend to have a more linear cyclic stress-strain response at small strains, and to degrade less at larger strains than the soils having lower PI [19].

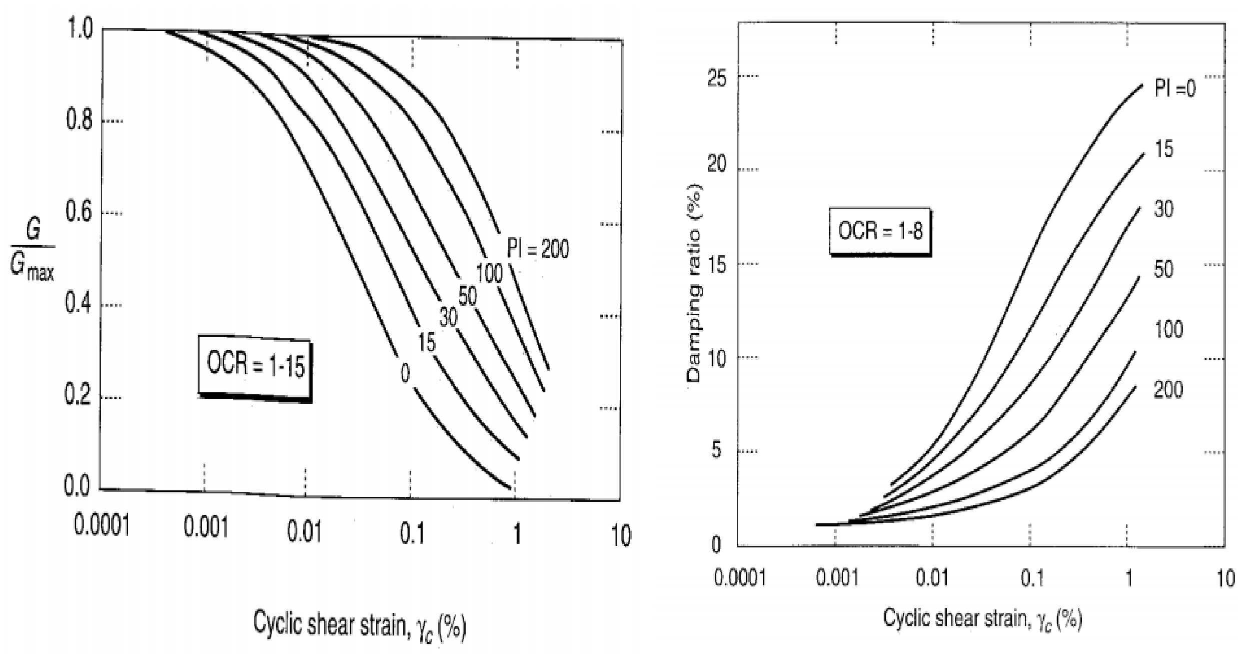


Figure 2. 8 Relations between G/Gmax versus cyclic shear strain, and Damping versus cyclic shear strain curves and soil plasticity for normally and overconsolidated soils [19].

2.3.4 Shear modulus and damping ratio values of soils

2.3.4.1 Clay

I. Shear Modulus

Cohesive soils with low plasticity exhibit a high shear modulus at low strain levels [19]. For high strain levels, all soils regardless of their plasticity tend to converge, because the shear modulus of low plasticity soils decreases rapidly with increases in shear strain, as compared to cohesive soils with high plasticity. At very low strain levels an approximately linear relationship between the shear modulus and shear strength exist for a number of clays, but nonlinear for medium to high strain level [17]. The modulus reduction curves for clay show a much larger scatter when shear strain increase and seem to be related to the characteristics of individual clay [6]. Figure2.11 shows shear modulus versus shear strain for undisturbed soft clays.

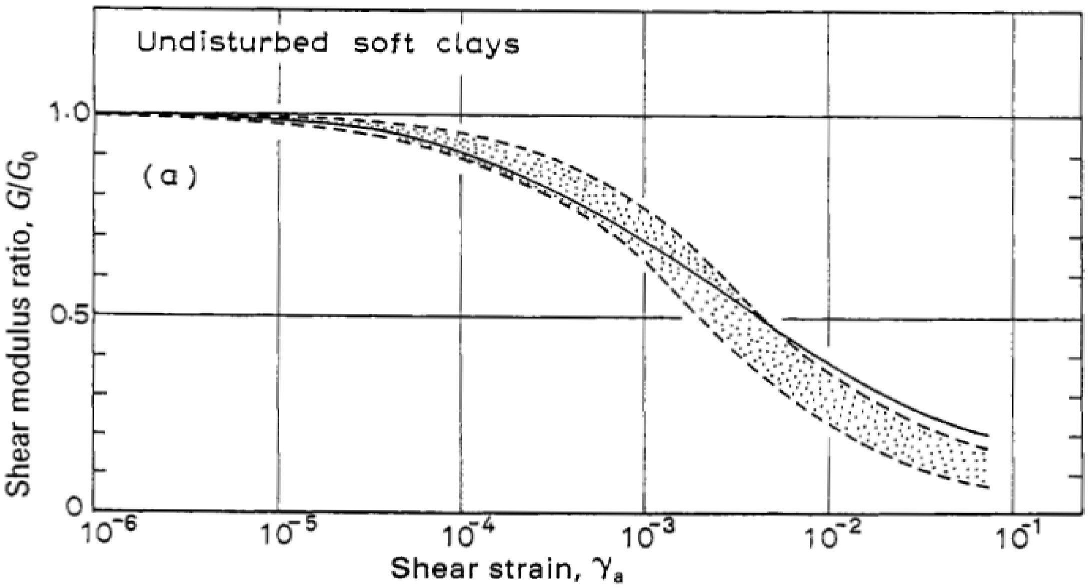


Figure 2.9 Strain-dependent shear modulus of soft clays [20].

II. Damping Ratio

Previously published data of damping ratios for saturated clay soils are limited and the results vary to such an extent that it is difficult to determine the main factors influencing the damping ratios of these soils. The basic relationships between damping ratio and strain levels for any particular clay soils is shown, in fig.2.12. This average relationship may well provide values of damping ratio with sufficient accuracy for many practical purposes [17].

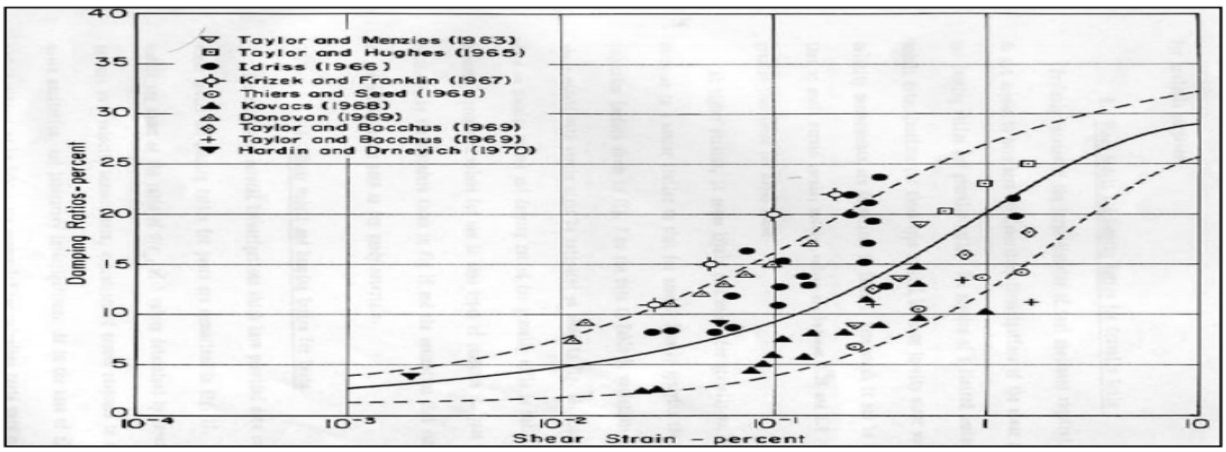


Figure 2.10 Damping ratio for saturated clay [17]

2.3.4.2 Sand

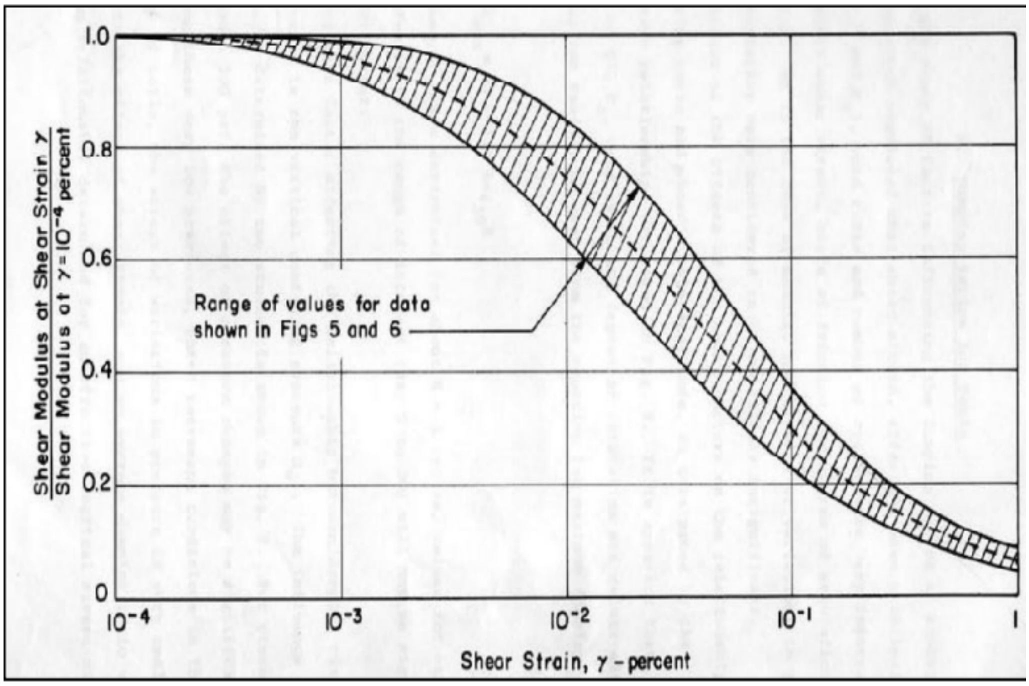


Figure 2.11: Variation of shear modulus with shear strain for sand [17]

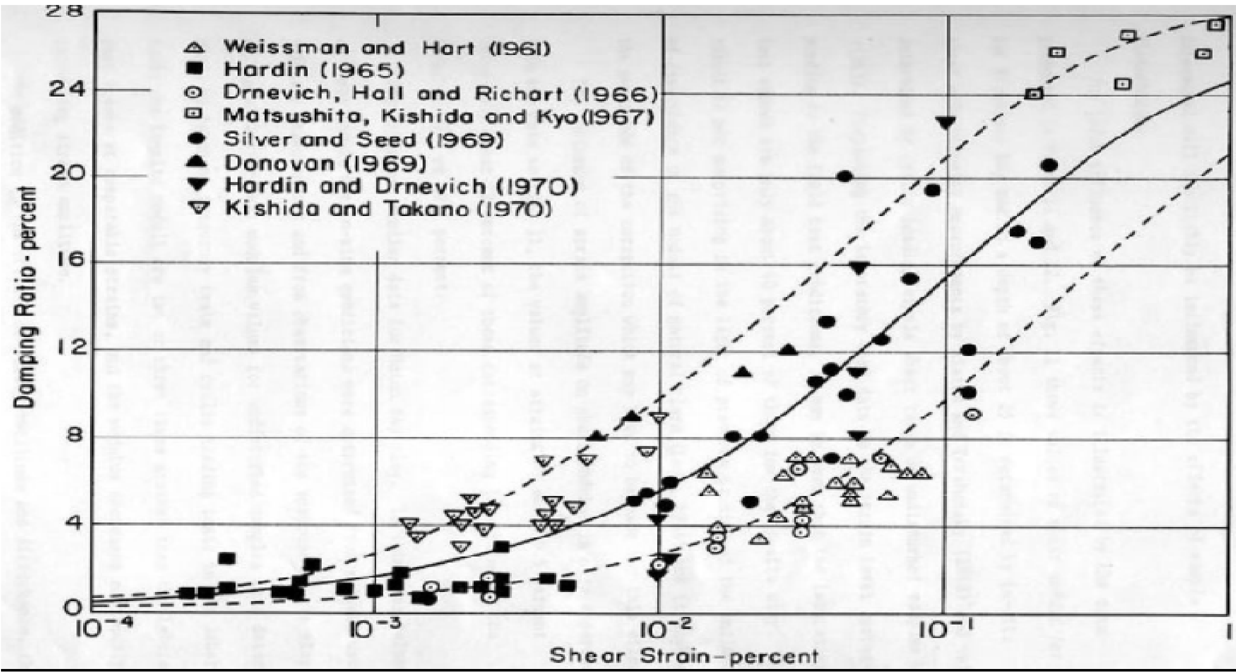


Figure 2.12 Variation damping ratio with strain for sand [17]

2.3.4.3 Other soil types

Properly established curves for dynamic properties of soils like silts are uncommon compared to sand and clay. Figure 2.15 and 2.16 show comparisons of soils in South of Tehran which is mainly silts with Vucetic and Dobry (1991) curves for plasticity indexes of 30, 15 and 0. The results show that, the location of shear modulus ratio values are comparable where as the damping ratio values are lower than that of Vucetic and Dobry curves. This may give us some indication about silt soil that it has lower damping ratio values as compared to literature [21].

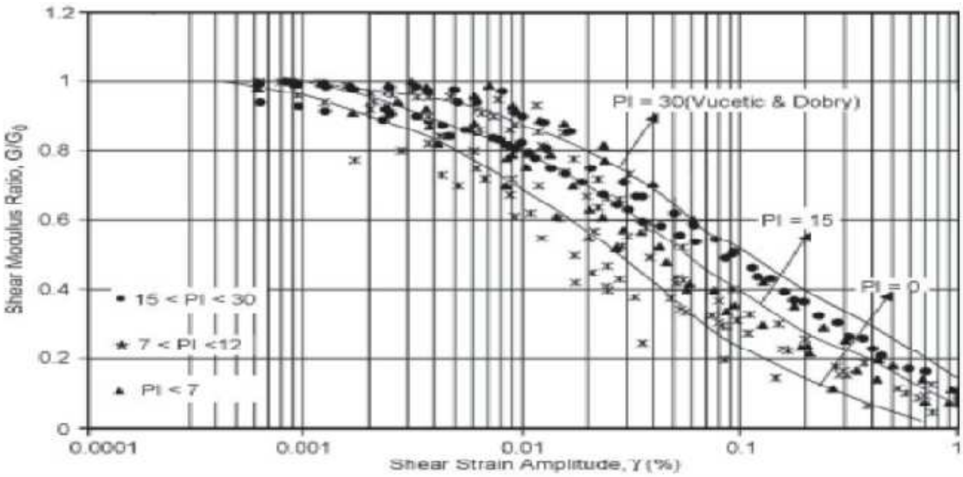


Figure 2.13: Shear modulus ratio vs. Shear strain curves of soils in South of Tehran as compared with Vucetic and Dobry (1991) curves [21]

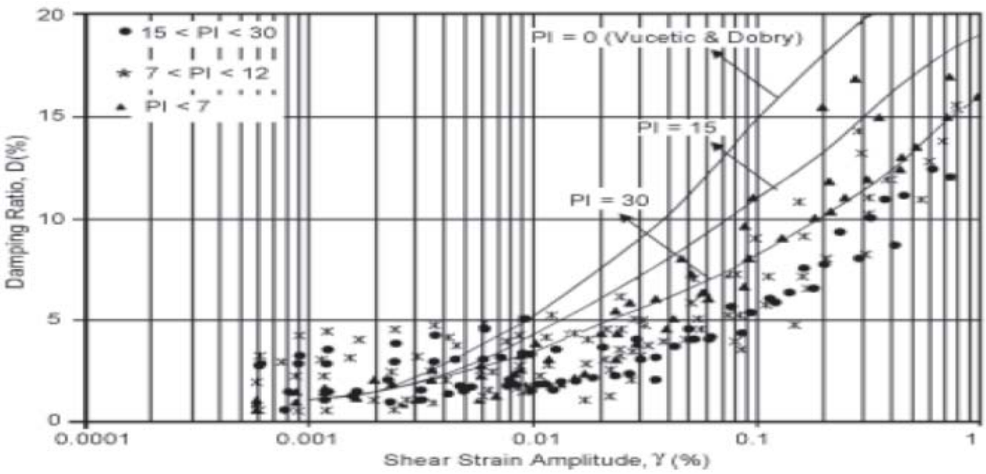


Figure 2.14: Damping ratio vs. Shear strain curves of soils in South Tehran as compared with Vucetic and Dobry (1991) curves [21]

CHAPTER 3

DATA COLLECTION AND TEST RESULTS

3.1 General

Soil samples have been taken from five test pits at 3m depth. During the field work, the field densities and natural moisture contents of the samples were determined using core cutter method. The locations of all test pits are shown in Figure 3.1. Based on laboratory test results of the study area, the soil is classifieds in to clay and silt soil based on USCS classification. Hence, two representative pits are selected to conduct cyclic simple shear testing.

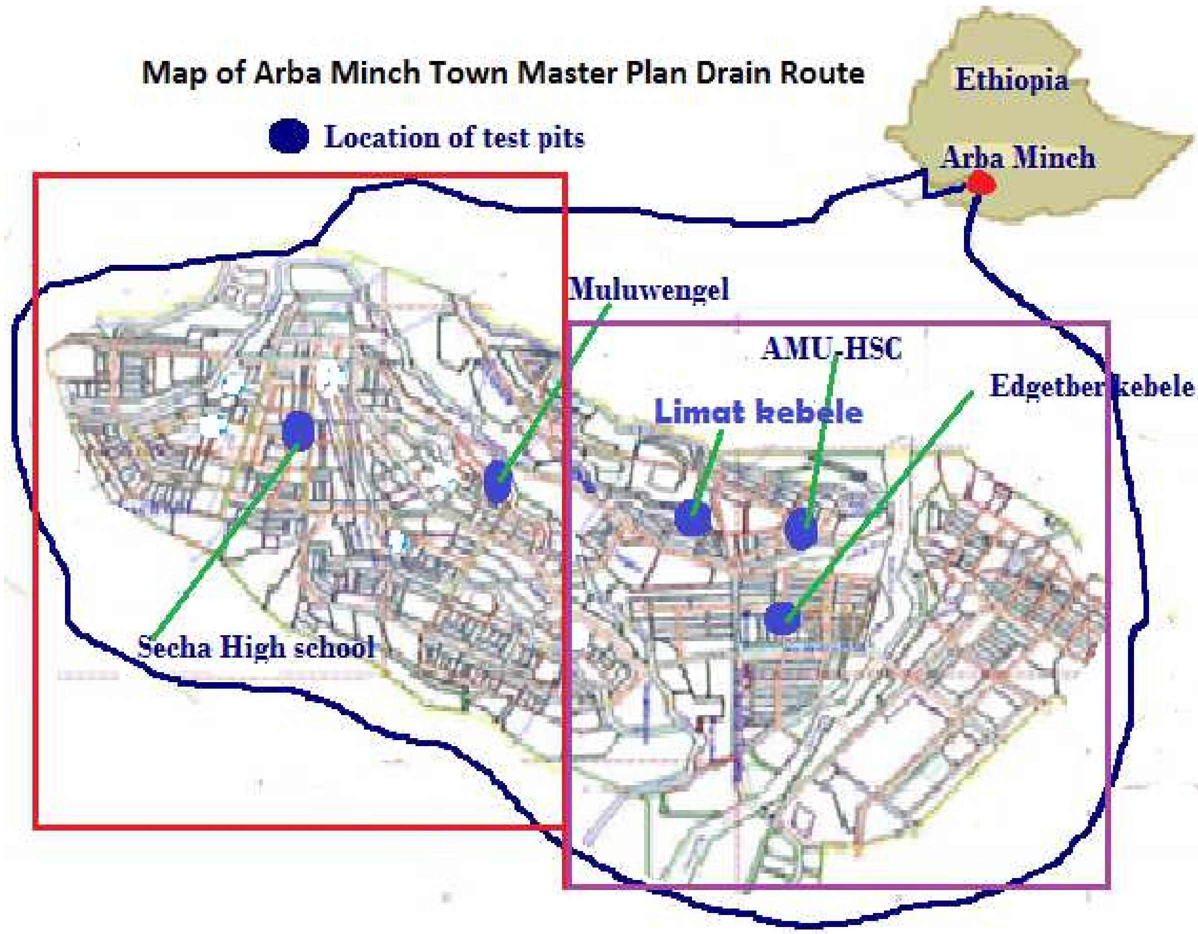


Figure 3.1 Location of test pits

3.2 Summary of some basic test results

Laboratory tests such as consolidation, compaction, free swell, Atterberg limit, sieve analysis are conducted to gain a better understanding on important properties of soils under consideration, and how the dynamic shear modulus and damping ratio is influenced by plasticity index, stress history, void ratio and grain distribution. Field density and field moisture content are determined to investigate field condition.

All tests were conducted according to ASTM procedures. ASTM D 2937-00 –field density of Soil, ASTM D 4318 - Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils, ASTM D698 - Standard Test Methods for Laboratory Compaction, ASTM D422 - Standard Test Method for Particle-Size Analysis of Soils ASTM D 854-00 – Standard Test for Specific Gravity of Soil Solids by Water Pycnometer. The sieve analysis was performed to determine the distribution of the coarser, larger-sized particles, and the hydrometer method was used to determine the distribution of the finer particles. Table 3.1 presents both laboratory and field test results.

Table 3 1 Field and maximum densities, water contents and specific gravities

Station	Field density in g/cm³	field moisture content in %	Field Dry Density	Specific gravity	MDD	OMC (%)	Maximum Bulk density	Free swell
Secha High School	1.58	26.30	1.18	2.71	1.32	42.42	1.95	68
Muluwengel	1.56	30.20	1.17	2.67	1.28	43.6	1.84	66
Edigetber kebele (stadium)	1.51	18	1.15	2.68	1.26	26.5	1.63	35
Limat-wezy kebele	1.52	17.12	1.16	2.69	1.27	25.81	1.64	35
AMU HSC	1.52	16.83	1.17	2.70	1.31	25.2	1.69	33

The grain size distribution curve drawn below helps to make contrast on the grain size distribution of different test pits.

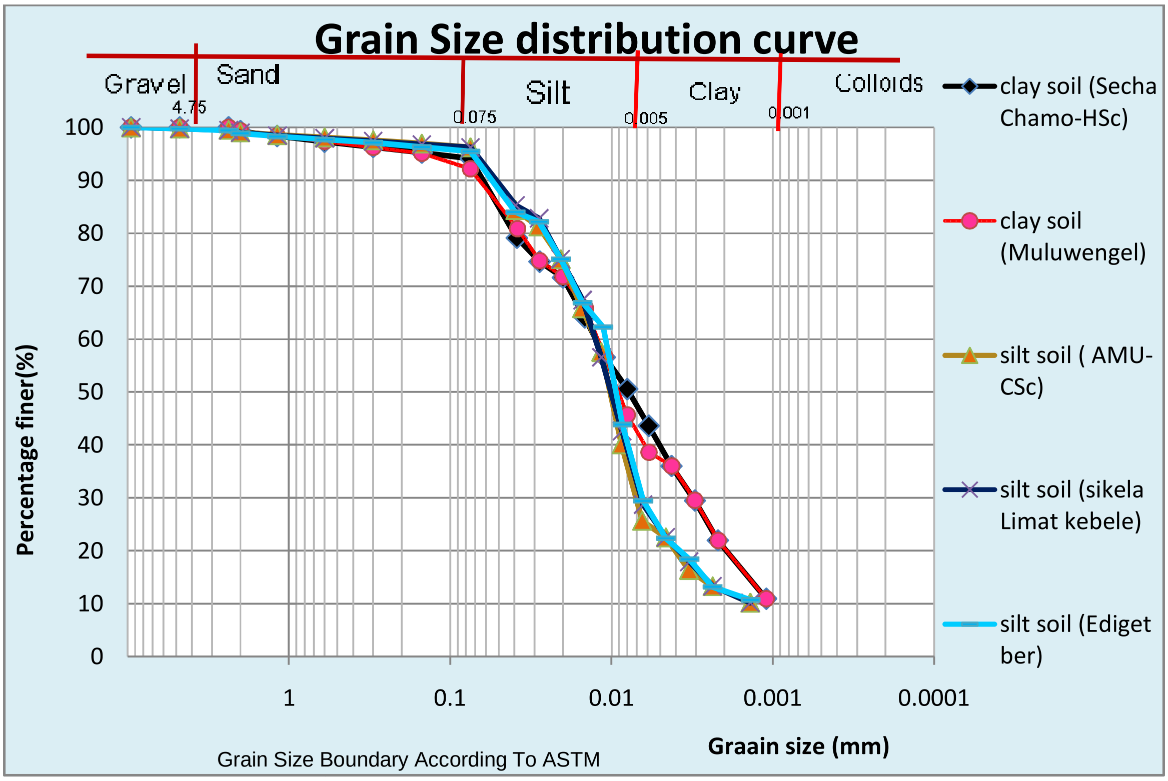


Figure 3. 2 Grain size distribution of all test pits

3.3 Soil Classification

The system of classifying soils for engineering purposes based on laboratory determination of particle-size characteristics, liquid limit, and plasticity index is termed as unified soil classification system. The unified soil classification system is popular for use in all types of engineering problems involving soils. Thus, USCS shall be used to get precise classification for further seismic investigation of the soil under study.

Table 3. 2 Atterberg limits and Classification

Test pit name	Secha High school	Muluwengel	Limat-wezy Keble	Edigetber Keble (stadium)	AMU CSC
LL	91	88	55	54	51
PL	37	38	42	41	39
PI	54	50	13	13	12
% passing 4.75mm sieve	100	100	100	100	100
% passing 0.075mm sieve	94.90	94.35	96.21	96.42	96.22
Percent silt size (0.075-0.005 mm) - ASTM	53.70	54.32	73	72.31	73.02
Percent clay size (≤ 0.005 mm) -ASTM	41.20	40.03	23.21	24.11	23.21
USCS Classification	Clay (CH)	Clay (CH)	Silt (MH)	Silt (MH)	Silt(MH)

3.4 Organic Content Test

Acording to the USCS classification, Limat-wezy, Edigetber kebele and AMU-community school test result shows the soil is either inorganic silt (MH) or organic silt(OH), an additional simple test was conducted to check whether the soils are organic or not. According to ASTM D2487 _ 98, a soil is an organic silt if the liquid limit after oven drying is less than 75% of the liquid limit of the original specimen determined before oven drying. However, the test results for Edigetber kebele and AMU-community school soils indicate that the LL ratio is 85 % and 87 % respectively. These values are greater than the 75 % boundary indicating that the soils are inorganic silts (MH).

3.5 One-dimensional Consolidation

Here, the main purpose of one dimensional odemeter test is to get better understanding on stress history of the soil under study. Inaddation, it helps to determine preconsolidation pressure which is one of the parameters required to compute the maximum shear modulus of the soil in this paper. The tests are based on ASTM D2435 - Standard Test Method for One-Dimensional Consolidation. The one-dimensional consolidation test results as shown figures 3.3 and 3.4 gives a preconsolidation pressure of 198KPa for the clay soi and 135 kPa for the silty soil .

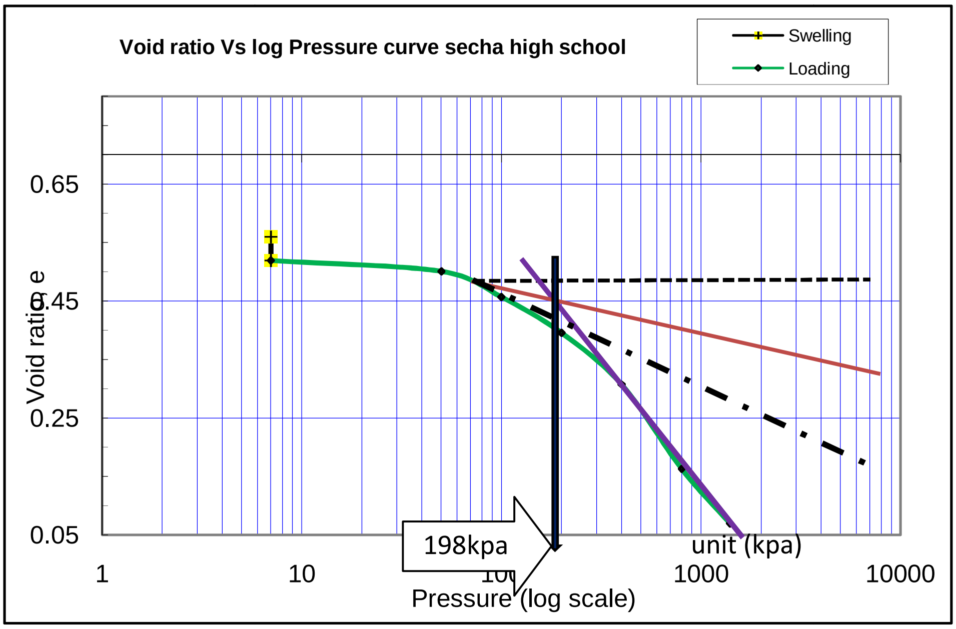


Figure 3.3 Void ratio vs. pressure (log scale) for clay soil

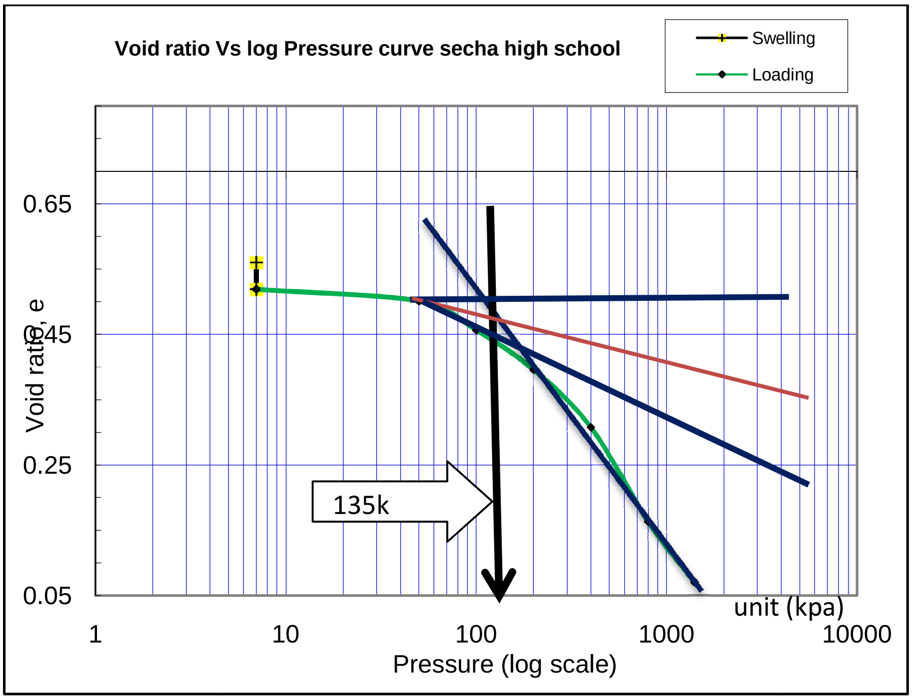


Figure 3.4 Void ratio verses pressure (log scale) for silt soil

3.6 Cyclic simple shear testing system

3.6.1 General overview of the machine

The cyclic simple shear apparatus is generally used for research into the dynamic field of soil behavior. This test most nearly duplicates the loading conductions thought to occur during an earthquake.

Nowadays there are different types of cyclic simple shear apparatuses in use. In this research the type of apparatus used is 31-WF7500 cyclic simple shear machine which is developed by the Controls Group. The complete system is controlled by the UTS004 software application program [24]. The cyclic simple shear machine is designed to allow a sample to be consolidated, drained and then sheared.

Figure 3.5 Cyclic simple shear machine (model 31-WF7500)

3.6.2 Stages and Setup of Cyclic simple shear testing

Sample preparation, consolidation and cyclic shearing are common procudure in the laboratory to carry out cyclic simple shear test. During simple cyclic shear test, specimens were initially consolidated to a vertical effective stress level with no applied static shear stress prior to commencement of constant volume (monotonic or cyclic) shear loading.

The sample is set up in the machine, which has a rigidly fixed top half and a moving bottom half. The top half houses the vertical ram. This is housed in a linear bearing to allow vertical movement and prevent horizontal movement. The bottom half is mounted on roller bearings as in a standard shear box. The sample is supported by a rubber membrane placed and secured with O-rings. To maintain a constant diameter throughout the test, the sample is supported by a series of slip rings. During shear the rings slide across each other as shown Fig. 2-17. During the shearing stage of the test the vertical height of the sample is maintained at a constant height by the vertical actuator in a closed control loop with the vertical displacement transducer. The rings maintain a constant sample diameter.

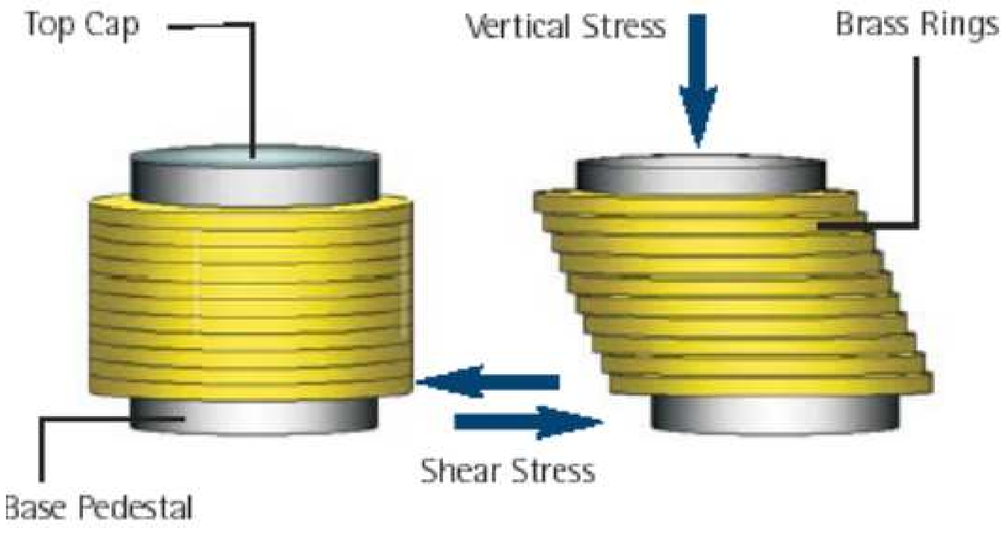


Figure 3 6: The movement rings during Shearing Stage

3.6.1 Specimen preparation

In order to conduct the cyclic simple shear test, the disturbed soil samples have been remolded to field condition (at field density and water content) to replicate the natural state. The specimen is cylindrical in shape with 20 mm height and 70 mm diameter. Once the specimen is prepared, it will be mounted on the cyclic simple shear test machine for testing.

3.6.2 Consolidation Stage

The consolidation stage is simply the application of a static axial loading stress to the specimen while the lateral loading (shear) axis is held stationary. Axial stress and specimen displacements (axial and lateral) data are measured over time and logged by the system. Logged data is also displayed to the operator in the form of charts and tables as the test stage proceeds. The consolidation stage is manually terminated by the operator once consolidation of the specimen is determined to be complete.

The effective pressure applied during consolidation stage of cyclic simple shear test is taken to be 100Kpa, 200Kpa and 400Kpa. The reason behind to select 100Kpa, 200Kpa and 400Kpa effective pressures is to account the stress history of the soil (normally-consolidated, preconsolidaed and partially-preconsolidaed) and to compare with the previous study results.

3.6.3 Cyclic Simple Shear Stage

The shear strain is induced by horizontal movement at the bottom of the sample relative to the top. The horizontal diameter of the sample remains constant throughout the test [22]. A lateral cyclic shear force or a displacement applied to the specimen, while the axial axis is either maintained at the specified stress, or optionally, the specimen height is maintained [21]. In this research, the cyclic shear test conducted is strain-dependent which allows a lateral cyclic displacement to be selected to the specimen.

The maximum and minimum levels of shear displacement allow the amplitude and offset of the displacement to be specified. The specimen's height is maintained constant during cyclic shear test so that, the test is conducted under constant volume condition. Both lateral force and specimen displacements are measured for each loading cycle. Measured data is obtained from 50 sample points captured over a single cycle period [25].

3.7 Presentation of Cyclic Shear Test results

3.7.1 Axial loads and Shear Strain Levels used

The cyclic simple shear testing machine enables one to conduct cyclic shear test within the strain levels of 0.01 to 5 percent. This test can also be conducted with different axial stress, which enables one to see its effect on the values of shear modulus and damping ratio. In this study, axial loads of 100 kPa, 200 kPa and 400 kPa were used. Table 3.5 below summarizes the axial stress and shear strain values used in this thesis. It should be noted here that, preparations of specimens and testing have been done for each strain level and axial load for all samples of representative test pits.

Table3. 3 Axial stress and shear strain values used for this thesis

Test-pit	Sample type	Axial stress (KPa)	Shear strain (%)				
Clay soil	Remolded to field condition	100	0.01	0.1	1	2.5	5
		200	0.01	0.1	1	2.5	5
		400	0.01	0.1	1	2.5	5
silt soil	Remolded to field condition	100	0.01	0.1	1	2.5	5
		200	0.01	0.1	1	2.5	5
		400	0.01	0.1	1	2.5	5

3.7.2 Shear stress and strain parameters

During the cyclic shear stage of the test both lateral force and specimen displacements are measured for each loading cycle with time. Measured data can be displayed to the operator on Microsoft Excel spreadsheet. From the lateral force and displacement recorded data, one can calculate the shear stress (τ) and shear strain (γ) values. Using the specimen height after consolidation (< 20 mm) and its diameter, 70 mm, the shear stress and shear strain of the specimen can be calculated based on the following equation.

$$\tau = \frac{Force}{Area} = \frac{shear\ force}{\pi * 35^2} * 10^3 \quad (MPa)$$

$$\gamma = \frac{\Delta l}{L} = \frac{Displacement}{Height\ after\ consolidation}$$

Table 3.9 below shows sample tabulation of shear strain and shear stress from the lateral force and specimen displacement taken from the 5^{th} cycle test result of silt soil with peak-to-peak cyclic strain amplitude of 2.5%. The loading frequency used in this study is 1 Hz, which is commonly used in laboratory tests. From the predefined shear shape option such as, sinusoidal, triangular etc, the loading cycle shape has been selected to be sinusoidal (see Figure 3.5) as it is the most common type of seismic wave shape for analysis [2].

Table 3. 4 Shear stress and shear strain values of the 5th cycle test result of silt soil with 2.5 % strain and 400 KPa axial loads.

Cycle No	Time in sec.	Lateral displacement (mm)	Shear force (kN)	$\gamma = \frac{\text{displacment}}{18.25}$	$\tau = \frac{Shear\ force}{\pi * 35^2} * 10^3$
5	0	-0.32071	-0.4509	-0.02036	-0.1172
	0.019	-0.30666	-0.4088	-0.01770	-0.1062
	0.038	-0.28661	-0.3516	-0.01654	-0.0914
	0.057	-0.25988	-0.2808	-0.01500	-0.0730
	0.076	-0.22652	-0.1993	-0.01307	-0.0518
	0.095	-0.18492	-0.1124	-0.01067	-0.0292
	0.114	-0.13574	-0.0251	-0.00783	-0.0065
	0.133	-0.04081	0.0298	-0.00235	0.0078
	0.152	0.034495	0.0871	0.00199	0.0226
	0.171	0.083155	0.1495	0.00480	0.0388
	0.190	0.121865	0.2120	0.00703	0.0551
	0.209	0.156345	0.2723	0.00902	0.0708
	0.228	0.188675	0.3285	0.01089	0.0854
	0.247	0.217715	0.3792	0.01256	0.0985
	0.266	0.243865	0.4233	0.01407	0.1100
	0.285	0.267095	0.4603	0.01541	0.1196
	0.304	0.286665	0.4896	0.01654	0.1272
	0.323	0.303085	0.5111	0.01749	0.1328
	0.342	0.323255	0.5187	0.01866	0.1348
	0.361	0.333045	0.5282	0.01922	0.1373
	0.380	0.337445	0.5322	0.01947	0.1383
	0.399	0.340705	0.5334	0.01966	0.1386
	0.418	0.341905	0.5312	0.01973	0.1380
	0.437	0.341825	0.5266	0.01973	0.1368
	0.456	0.340765	0.5199	0.01967	0.1351
	0.475	0.337215	0.5051	0.01946	0.1312
	0.494	0.329495	0.4793	0.01902	0.1245
	0.513	0.316535	0.4370	0.01827	0.1135
	0.532	0.296445	0.3790	0.01711	0.0985
	0.551	0.271095	0.3087	0.01565	0.0802
	0.570	0.237715	0.2268	0.01372	0.0589
	0.589	0.196515	0.1386	0.01134	0.0360
	0.608	0.147335	0.0507	0.00850	0.0132
	0.627	0.066245	-0.0157	0.00382	-0.0041

	0.646	-0.02468	-0.0771	-0.00142	-0.0200
	0.665	-0.08133	-0.1355	-0.00469	-0.0352
	0.684	-0.12001	-0.1952	-0.00693	-0.0507
	0.703	-0.15617	-0.2566	-0.00901	-0.0667
	0.722	-0.19236	-0.3164	-0.01110	-0.0822
	0.741	-0.22332	-0.3679	-0.01289	-0.0956
	0.760	-0.25079	-0.4129	-0.01447	-0.1073
	0.779	-0.27585	-0.4513	-0.01592	-0.1173
	0.798	-0.29648	-0.4819	-0.01711	-0.1252
	0.817	-0.31336	-0.5048	-0.01808	-0.1312
	0.836	-0.32592	-0.5199	-0.01881	-0.1351
	0.855	-0.33404	-0.5289	-0.01928	-0.1374
	0.874	-0.33879	-0.5332	-0.01955	-0.1386
	0.893	-0.34151	-0.5334	-0.01971	-0.1386
	0.912	-0.34188	-0.5312	-0.01973	-0.1380
	0.931	-0.34191	-0.5266	-0.01973	-0.1368

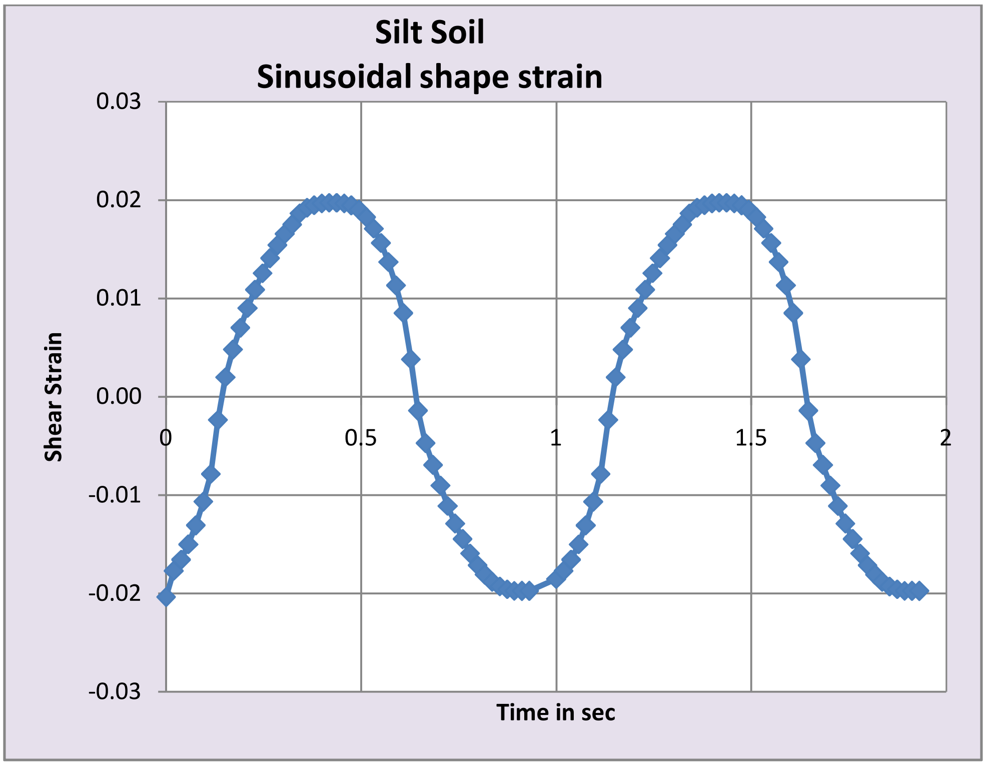


Figure3.7 Sinusoidal wave shapes of 2.5 % strain of silt soil for two cycles at 400KPa axial load

Table 3. 5 Shear stress and shear strain values of the 5th cycle test result of silt soil with 2.5 % strain and with 400 KPa axial loads.

Cycle No	Time in sec.	Lateral displacement (mm)	Shear force (kN)	$\gamma = \frac{\text{displacment}}{18.32}$	$\tau = \frac{Shear\ force}{\pi * 35^2} * 10^3$
5	0	-0.3726	-0.180985	-0.0193	-0.0470
	0.019	-0.3388	-0.125155	-0.0167	-0.0325
	0.038	-0.2948	-0.061445	-0.0145	-0.0160
	0.057	-0.2396	0.004495	-0.0118	0.0012
	0.076	-0.1735	0.065515	-0.0085	0.0170
	0.095	-0.0918	0.111025	-0.0045	0.0288
	0.114	0.0089	0.131975	0.0004	0.0343
	0.133	0.0856	0.151595	0.0042	0.0394
	0.152	0.1512	0.169345	0.0074	0.0440
	0.171	0.2041	0.187305	0.0100	0.0487
	0.19	0.2481	0.205935	0.0122	0.0535
	0.209	0.2851	0.225365	0.0140	0.0586
	0.228	0.3171	0.243475	0.0156	0.0633
	0.247	0.3439	0.259815	0.0169	0.0675
	0.266	0.3678	0.273365	0.0181	0.0710
	0.285	0.3878	0.283975	0.0191	0.0738
	0.304	0.4045	0.292255	0.0199	0.0759
	0.323	0.4169	0.298085	0.0205	0.0775
	0.342	0.4270	0.302115	0.0210	0.0785
	0.361	0.4333	0.302945	0.0213	0.0787
	0.38	0.4349	0.300345	0.0214	0.0780
	0.399	0.4336	0.294785	0.0213	0.0766
	0.418	0.4305	0.286945	0.0212	0.0746
	0.437	0.4231	0.273445	0.0208	0.0711
	0.456	0.4083	0.249695	0.0201	0.0649
	0.475	0.3845	0.216815	0.0189	0.0563
	0.494	0.3470	0.177445	0.0171	0.0461
	0.513	0.2924	0.137535	0.0144	0.0357
	0.532	0.2157	0.106965	0.0106	0.0278
	0.551	0.1158	0.089985	0.0057	0.0234
	0.57	0.0345	0.069445	0.0017	0.0180
	0.589	-0.0200	0.048895	-0.0010	0.0127
	0.608	-0.0684	0.015515	-0.0034	0.0040
	0.627	-0.1120	-0.020975	-0.0055	-0.0055

	0.646	-0.1563	-0.060255	-0.0077	-0.0157
	0.665	-0.2018	-0.098485	-0.0099	-0.0256
	0.684	-0.2450	-0.138455	-0.0120	-0.0360
	0.703	-0.2805	-0.172555	-0.0138	-0.0448
	0.722	-0.3130	-0.204445	-0.0154	-0.0531
	0.741	-0.3421	-0.232625	-0.0168	-0.0604
	0.76	-0.3671	-0.254825	-0.0180	-0.0662
	0.779	-0.3876	-0.273165	-0.0191	-0.0710
	0.798	-0.4044	-0.286415	-0.0199	-0.0744
	0.817	-0.4166	-0.295345	-0.0205	-0.0767
	0.836	-0.4253	-0.300795	-0.0209	-0.0782
	0.855	-0.4307	-0.302945	-0.0212	-0.0787
	0.874	-0.4342	-0.302655	-0.0213	-0.0786
	0.893	-0.4349	-0.299605	-0.0214	-0.0779
	0.912	-0.4348	-0.294765	-0.0214	-0.0766
	0.931	-0.4301	-0.282725	-0.0211	-0.0735

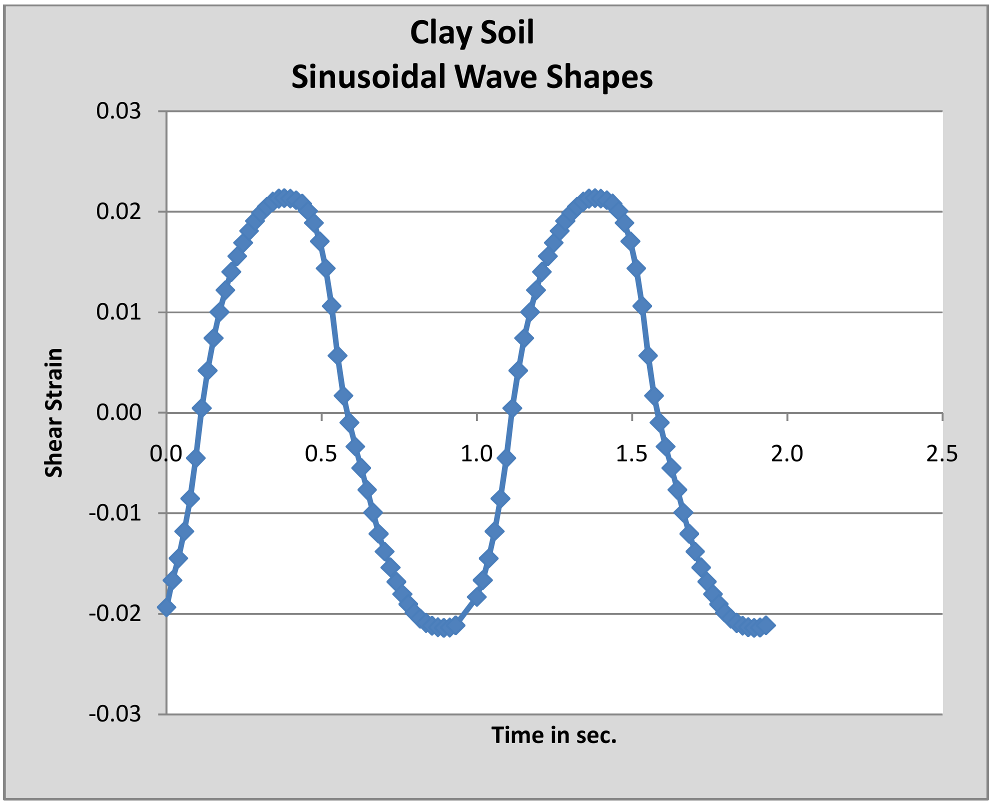
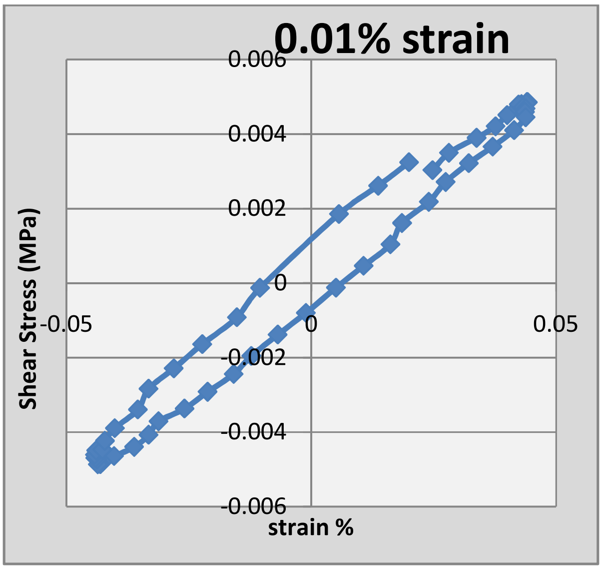
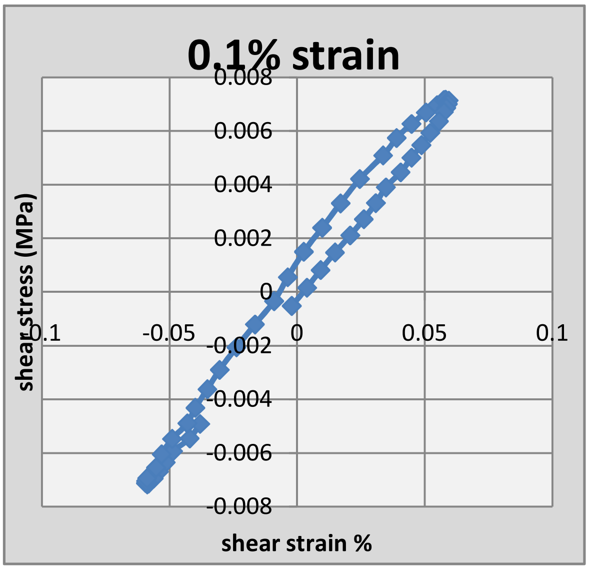


Figure 3. 8 Sinusoidal wave shapes of 2.5 % strain of clay soil for two cycles

3.7.3 Hysteresis loops of test results

The hysteresis loop of each cycle can be plotted using the results of shear strains and shear stresses that can be computed and presented with excel program. In most seismic events, the number of significant cycles is likely to be less than 20 [2]. For all practical purposes, the values determined at 5th cycles likely to provide reasonable values [3]. Figure 3.7 shows the hysteresis loops of the 5th cycle plotted for each strain level of silt soil tested under 400 KPa axial stresses. In this study, the number of cycles used in a taste is 40 cycles and Figure 3.7 shows the hysteresis loops of 40 cycles together in each strain levels of silt soil tested with 400 KPa. It has been shown in section 2.1.2 that the deformation characteristics of soils vary to a large extent depending upon the magnitude of shear strains to which the soils are subjected. And also the values of shear modulus and damping ratio will depend on the magnitude of the strain for which the hysteresis loop is determined. Figure 3.7 shows the hysteresis loops of the 5th cycles for each strain level test together and it shows how the value of shear strain affects the shape and size of the hysteresis loops. As the shear strain increases, the line connecting the tips of the hysteresis loop rotate clockwise implying a decrease shear modulus decreases. The width of the hysteresis loop becomes wider as the shear strain value increases. As a result, the damping effect of the soil increases.





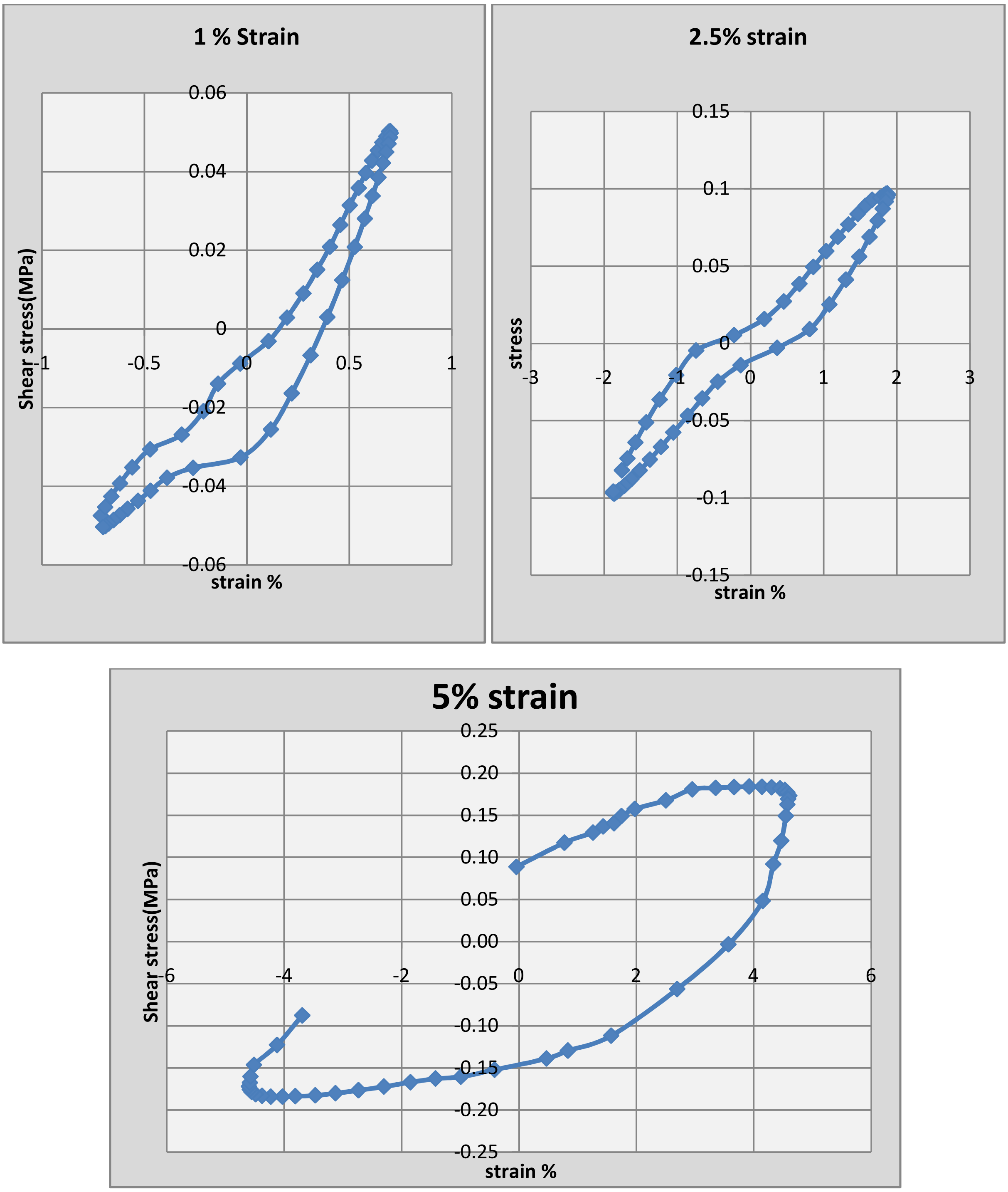


Figure 3. 9 Hysteresis loops of the 5th cycle of silt soil at 400 KPa of axial stress tests for each strain levels

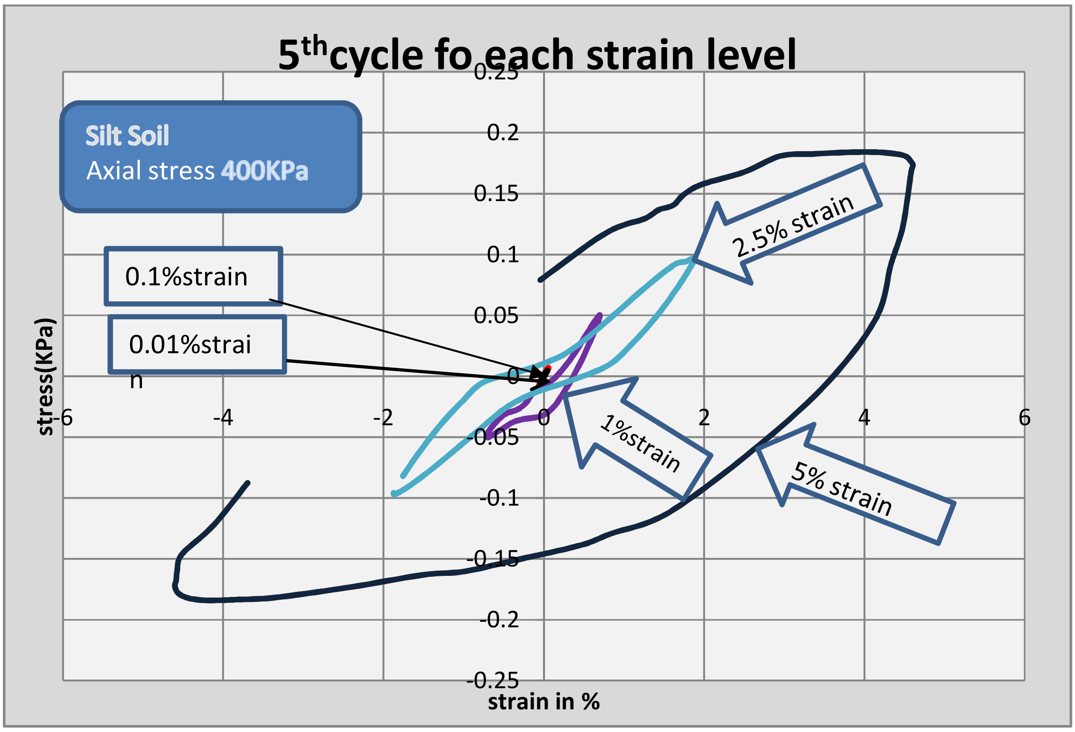
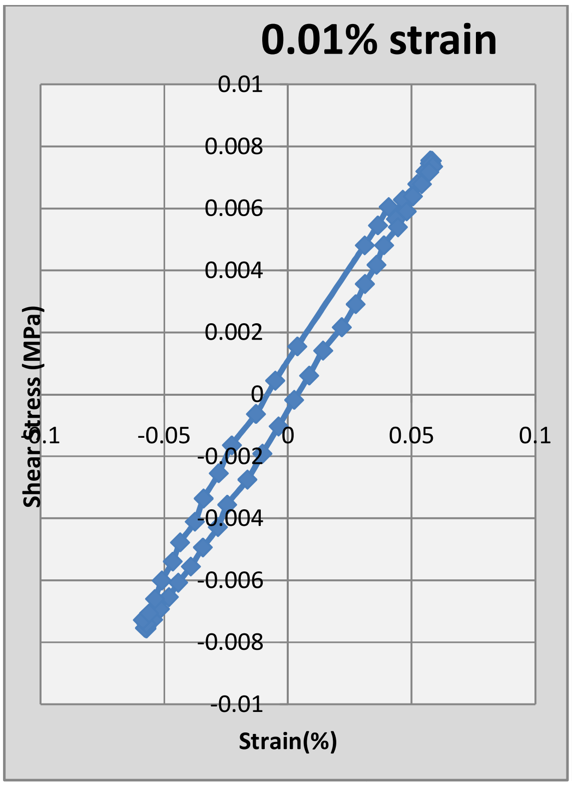
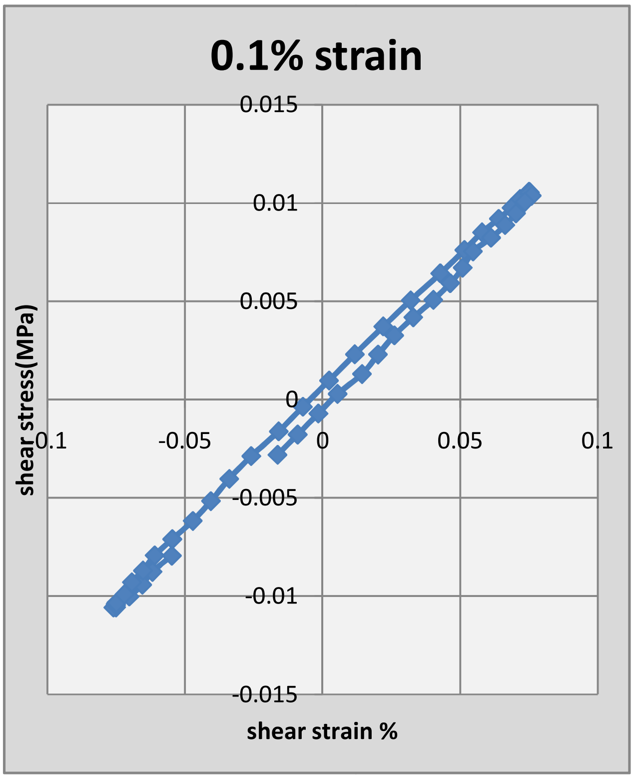


Figure3. 10 Hysteresis loops of the 5th cycle of silt soil at 100,200 &400KPa axial load & for each strain level in percent





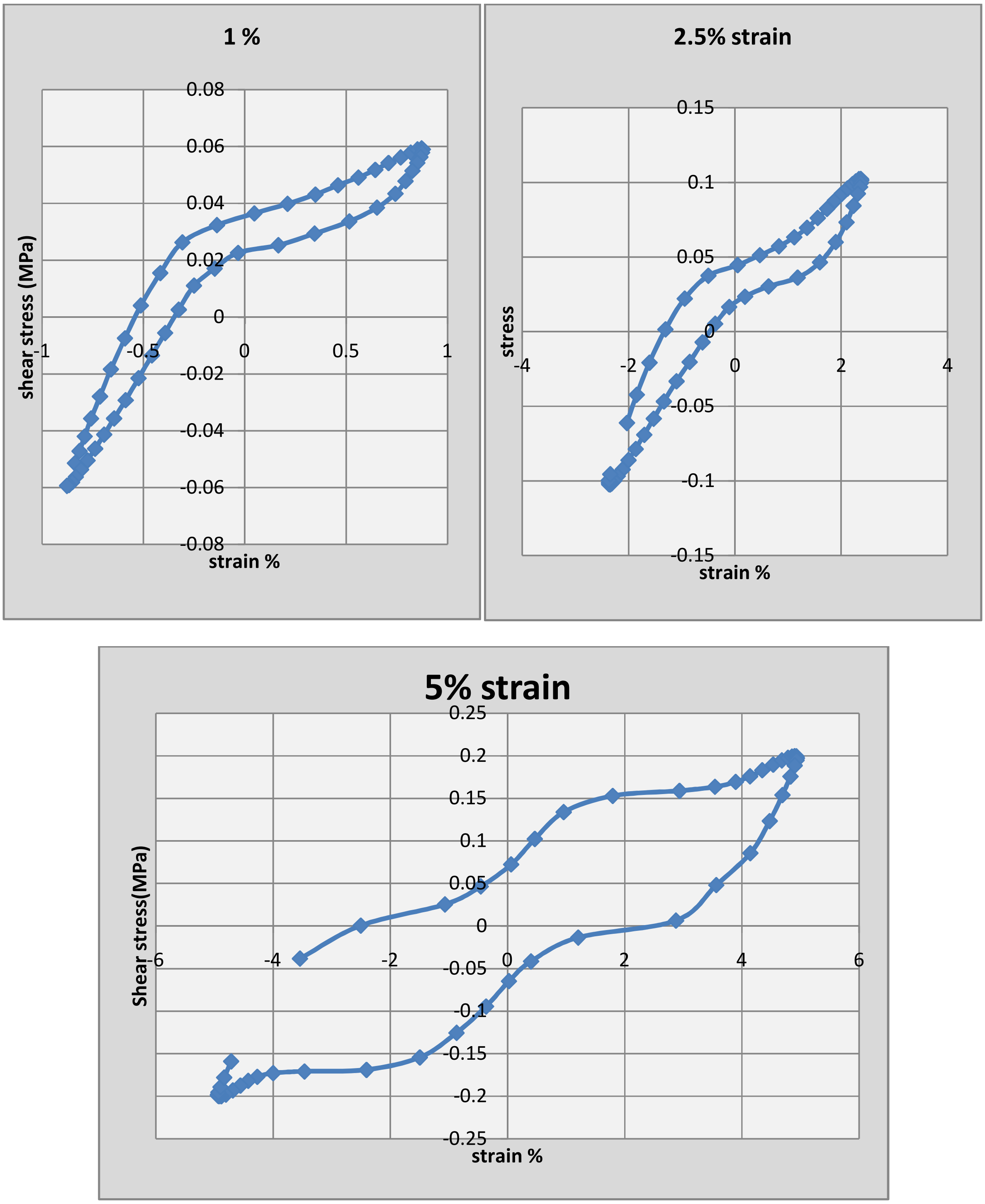


Figure3. 11: Hysteresis loops of the 5th cycle of clay soil for each strain levels in percent at 400KPa axial stress

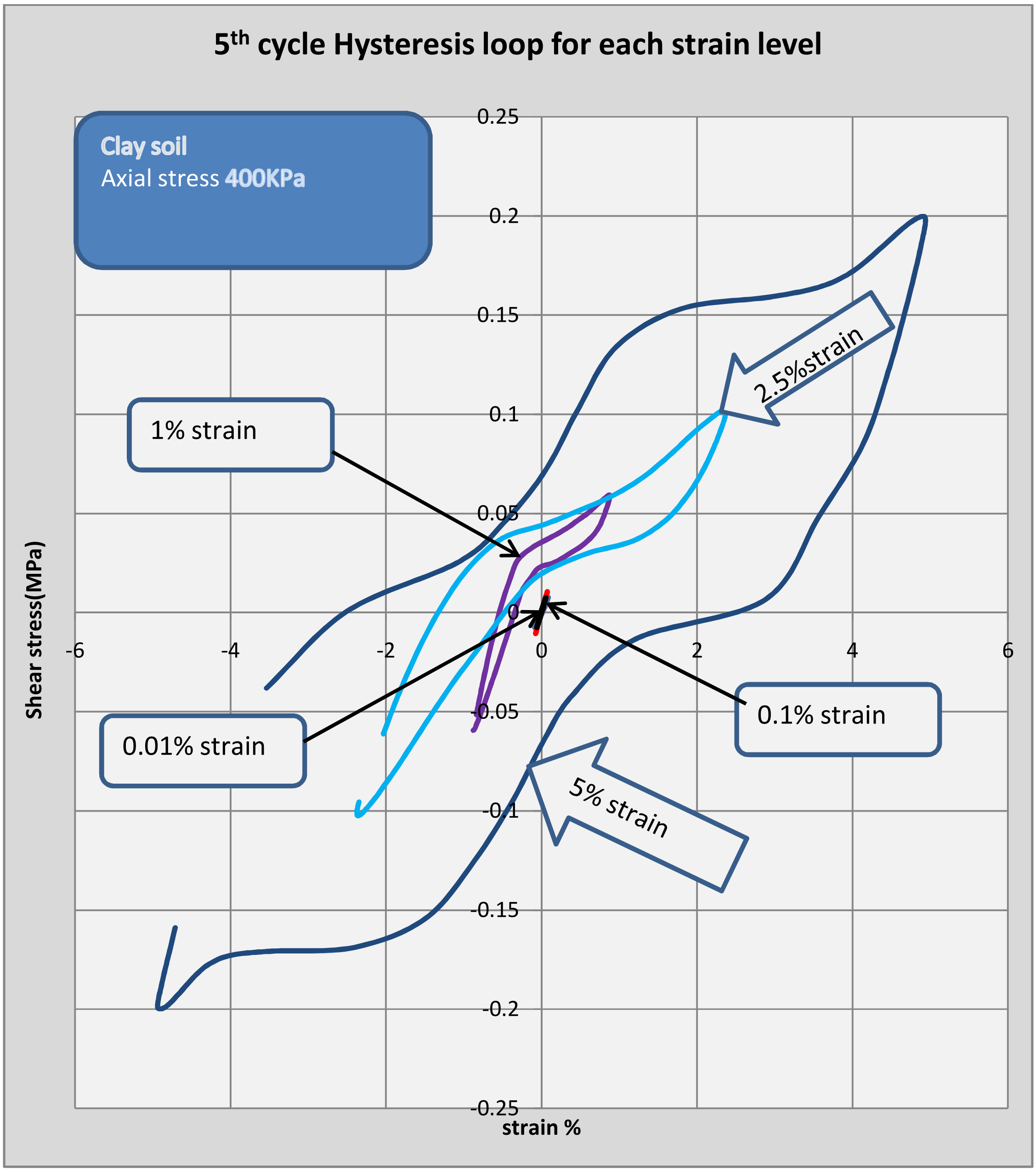


Figure 3. 12 Hysteresis loops of the 5th cycle of clay soil at 100KPa, 200Kpa & 400KPa axial load for each strain level in percent

CHAPTER 4
DATA ANALYSIS AND DISCUSSION OF TEST RESULTS

4.1 Computation of shear modulus and damping ratio values

Direct determination of stress-strain relationships are used for obtaining shear modulus and damping ratio by using cyclic simple shear test. As we have seen in section 3.3.4, the measured values of shear force and lateral displacement have been translated to shear stress and shear strain. And the hysteresis loops of each cycle can be plotted using the shear stress and strain values obtained from 50 sample points in a cycle. These 50 sample point's shear stress and shear strain values in a cycle have been analyzed for the determination of shear modulus and damping ratio values of the hysteresis loop. ASTM D3999 indicates that the shear modulus and damping ratio values determined at 5th cycle is likely to provide reasonable values for all practical purpose. Thus, in this research, the 5th cycle is selected for analysis purpose. For the determination of shear modulus, G, and damping ratio, D, a typical calculation is presented below.

$$G = \frac{\tau}{\gamma}$$

$$D = \frac{W_D}{4\pi W_S} = \frac{A_{loop}}{4\pi A_\Delta}$$

$$A_{loop} = 0.5 * (\tau_i - \tau_{i+1}) * (\gamma_i + \gamma_{i+1})$$

$$A_\Delta = 0.5 * X + Y$$

$$X = \gamma_{max} - \gamma_{center}$$

$$Y = \tau_{max} - \tau_{center}$$

Since, the center of the hysteresis loop is at the origin; both the stress and strain value at centre is zero. The shear strains and stresses are computed in Table 4.1. The shear modulus and damping ratio are calculated as indicated in Table4.2Using the shear strain and damping ratio values of Table 4.1, the hysteresis loop of a given cycle is plotted in Figure 4.1.

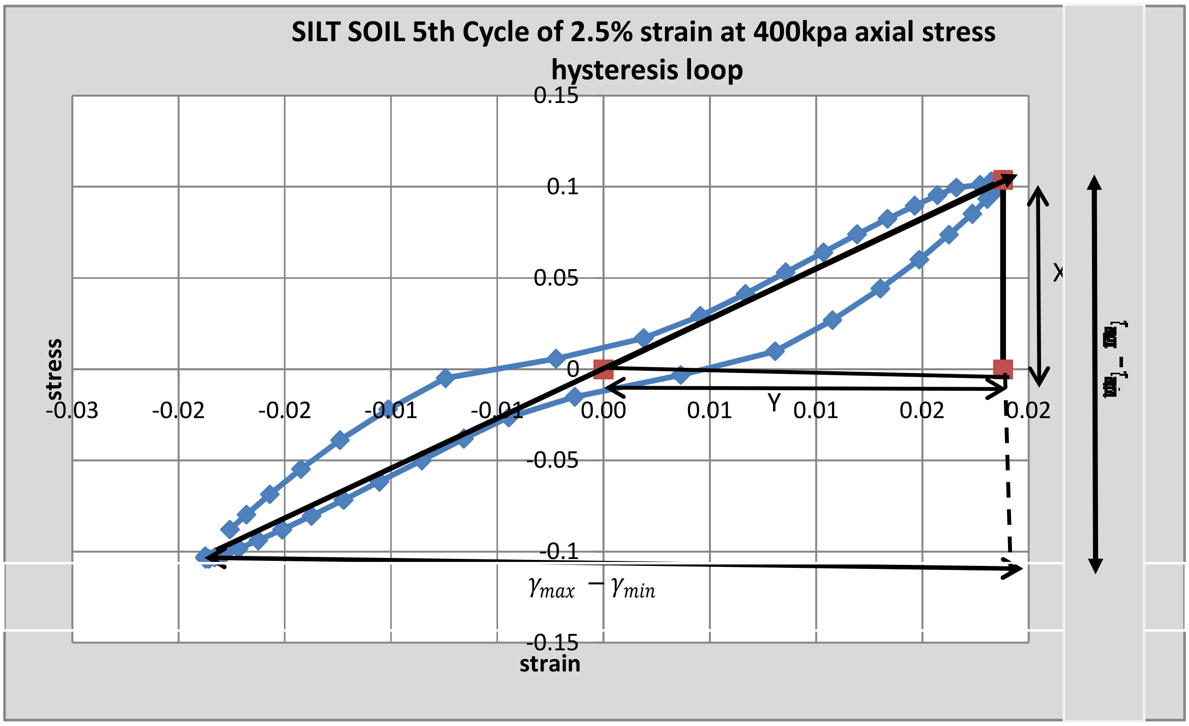


Figure 4.1 The hysteresis loop and triangle plotted using table 4.1 stress and strain values.

Table 4.1 computation of shear strains and shear stresses

Cycle No	Time in sec.	$\gamma = \frac{\text{displacment}}{18.25}$	$\tau = \frac{Shear\ force}{\pi * 35^2} * 10^3 (\text{MPa})$	$(\tau_i - \tau_{i+1}) * (\gamma_i + \gamma_{i+1})$
5	0	-0.017573	-0.087868	0.0003761
	0.019	-0.016803	-0.079664	0.0004827
	0.038	-0.015704	-0.068526	0.0005514
	0.057	-0.014240	-0.054715	0.0005639
	0.076	-0.012412	-0.038845	0.0005094
	0.095	-0.010132	-0.021898	0.0003985
	0.114	-0.007438	-0.004889	0.0001380
	0.133	-0.002236	0.005814	0.0000051
	0.152	0.001890	0.016979	-0.0001045
	0.171	0.004556	0.029136	-0.0001825
	0.19	0.006678	0.041322	-0.0002387
	0.209	0.008567	0.053068	-0.0002761
	0.228	0.010338	0.064020	-0.0002934
	0.247	0.011930	0.073903	-0.0002899
	0.266	0.013362	0.082499	-0.0002690
	0.285	0.014635	0.089706	-0.0002311
	0.304	0.015708	0.095418	-0.0001801
	0.323	0.016607	0.099598	-0.0000680

	0.342	0.017713	0.099684	-0.0000890
	0.361	0.018249	0.099790	-0.0000386
	0.38	0.018490	0.099791	-0.0000107
	0.399	0.018669	0.099846	0.0000213
	0.418	0.019735	-0.099696	0.0000444
	0.437	0.018730	-0.099684	0.0000654
	0.456	0.018672	0.098975	0.0001429
	0.475	0.018478	0.098431	0.0002451
	0.494	0.018055	0.093399	0.0003890
	0.513	0.017344	0.085157	0.0005063
	0.532	0.016244	0.073852	0.0005675
	0.551	0.014855	0.060165	0.0005933
	0.57	0.013025	0.044204	0.0005456
	0.589	0.010768	0.027008	0.0004300
	0.608	0.008073	0.009889	0.0002020
	0.627	0.003630	-0.003055	0.0000364
	0.646	-0.001352	-0.015032	-0.0000882
	0.665	-0.004456	-0.026415	-0.0001711
	0.684	-0.006576	-0.038046	-0.0002412
	0.703	-0.008557	-0.049998	-0.0002970
	0.722	-0.010540	-0.061662	-0.0003047
	0.741	-0.012236	-0.071695	-0.0003036
	0.76	-0.013742	-0.080461	-0.0002885
	0.779	-0.015115	-0.087960	-0.0002493
	0.798	-0.016245	-0.093921	-0.0001985
	0.817	-0.017170	-0.098376	-0.0001375
	0.836	-0.017858	-0.099684	-0.0000847
	0.855	-0.018303	-0.099790	-0.0000413
	0.874	-0.018564	-0.099791	-0.0000013
	0.893	-0.018713	-0.099846	0.0000213
	0.912	-0.019735	-0.099696	0.0000449
	0.931	-0.018730	-0.099684	0.0007925

The equivalent shear modulus of soil (G) can be determined from the shear modulus and confining pressure relationship or from the hysteresis loop directly. Because most soils have curvilinear stress-strain relationships, the tangent shear modulus (G_{tan}) varies through a cycle of loading but, its average value over the entire loop can be approximated by the secant shear modulus (Gsec,) which is commonly called equivalent shear modulus (G)[13]. Thus secant shear modulus describes the general inclination of the hysteresis loop.

In order to determine the shear modulus and damping ratio values of a hysteresis loop, consider the following procedure: The equivalent (secant) shear modulus, the maximum and minimum shear stresses and shear strains are obtained from the cycle. Then, the magnitude of the difference between the maximum and minimum shear stresses and shear strains are determined. Thus, the equivalent shear modulus of the hysteresis loop will be calculated by using based on figure 4.1

$$G = \frac{\tau_{max} - \tau_{min}}{\gamma_{max} - \gamma_{min}}$$

To determine the damping ratio value of the hysteresis loop, the area of the loop and the triangle should be first determined.

$$A_{loop} = 0.5 * \sum (\tau_i - \tau_{i+1}) * (\gamma_i + \gamma_{i+1})$$

The area of the triangle was calculated using the maximum values of shear stress and strain as:

$$A_{\Delta} = 0.5 * X + Y$$

Then, the damping ratio (D) value is calculated as:

$$D = \frac{A_{loop}}{4\pi A_{\Delta}}$$

Table 4.2 Typical calculation of shear modulus and damping ratio using

Calculation of Shear Modulus		Calculation of Damping ratio	
Calculation	Value	Calculation	Value
τ_{max}	0.099846	$A_{loop} = 0.5 * \sum (\tau_i - \tau_{i+1}) * (\gamma_i + \gamma_{i+1})$	0.0013
τ_{min}	-0.099846	$A_{\Delta} = 0.5 * X + Y$ =0.5*0.099846*0.01974	0.001
$\tau_{max} - \tau_{min}$	0.199	$\frac{A_{loop}}{4\pi A_{\Delta}} = D$	0.1514
γ_{max}	0.01974		
γ_{min}	-0.01974	$\beta(\%)$	10.30
$\gamma_{max} - \gamma_{min}$	0.040		
$G = \frac{\tau_{max}-\tau_{min}}{\gamma_{max}-\gamma_{min}}$	4.96MPa		

Based on Table 4.2, the values of shear modulus and damping ratio of each cycle in a test can be determined. In this study, a single specimen was tested up to 40 cycles and Table 4.3 below shows shear modulus and damping ratio values of each cycle of silt soil tested with 400 KPa axial loads.

Table 4.3 Values of shear modulus and damping ratio for silt soil tested at 400 KPa axial loads

Strain Level	0.01%	0.1%	1%	2.5%	5%	0.01%	0.1%	1%	2.5%	5%
No. Cycle	Shear Modulus G(MPa)					Damping Ratio β(%)				
1	15.5358	11.3832	7.6873	4.4016	1.6843	0.2309	2.7625	6.6833	11.2194	15.7684
2	15.6117	11.6757	7.8779	4.5572	1.7735	0.1141	2.5287	6.6138	10.9418	14.6161
3	15.7654	11.7297	8.4404	4.6094	1.8215	0.1148	2.4524	6.4891	10.6526	14.4927
4	16.2122	12.5596	8.5153	4.6433	1.8959	0.1134	2.4524	6.3138	10.3680	14.4854
5	17.1575	12.6919	8.6933	4.9556	2.0929	0.1156	2.3207	5.8972	10.3304	14.3869
6	17.2293	12.6400	8.8614	4.8629	2.1943	0.1086	2.2742	5.6841	10.2835	14.1163
7	17.7242	12.7388	8.8626	4.9164	2.3026	0.2070	2.5533	5.6100	10.2835	13.6621
8	17.8646	13.3657	8.9442	4.9601	2.3364	0.1070	2.2666	5.5095	10.2786	13.8089
9	17.8477	13.1350	8.9091	5.0376	2.3371	0.0930	2.2531	5.3739	10.1737	13.8492
10	18.2807	13.1435	9.0992	5.0455	2.3400	0.0972	2.2696	5.0380	10.0269	13.7092
11	17.5100	13.0971	9.2664	4.7560	2.3018	0.0901	2.1272	4.9753	9.7105	13.6591
12	17.6646	13.0403	9.3252	4.8066	2.3080	0.0880	1.9516	4.9721	9.7185	13.5891
13	17.8376	13.1879	9.3501	4.8279	2.3144	0.0857	1.8857	4.8975	9.8903	13.5438
14	18.2178	13.1993	9.3572	4.8560	2.3156	0.0824	1.8800	4.8751	9.7430	13.5429
15	18.2386	13.6048	9.3764	4.8629	2.3230	0.0792	1.8798	4.8595	9.9502	13.5247
16	18.3620	13.6495	9.4117	4.8821	2.3401	0.0792	1.8543	4.7919	9.9002	13.4645
17	18.4602	13.6285	9.4565	4.9071	2.3403	0.0758	1.8427	4.4576	9.8826	13.5356
18	18.5047	13.7285	9.4583	4.9120	2.3425	0.0566	1.7985	4.4530	9.0024	13.5803
19	18.5660	13.7318	9.5005	4.9547	2.3433	0.0521	1.9019	4.4524	9.6272	13.4774
20	18.6130	13.7232	9.8912	5.2066	2.3513	0.0421	1.8217	4.4451	9.5099	13.2875
21	17.6159	13.5546	9.2131	5.0973	2.3404	0.0464	1.7595	4.4312	9.0612	13.4738
22	18.6421	13.6370	9.2317	5.1143	2.3427	0.0501	1.7414	4.4229	8.9535	13.3494
23	18.6602	13.7320	9.2403	5.1224	2.3522	0.0490	1.6394	4.4074	8.9220	13.2374
24	18.6872	13.7747	9.2642	5.1360	2.3539	0.0481	1.5945	4.4068	8.8255	13.2274
25	18.7184	13.8238	9.2842	5.1523	2.3564	0.0470	1.5278	4.3975	8.7726	13.2064
26	18.7714	13.9787	9.2860	5.1644	2.3564	0.0448	1.5267	4.3974	8.7186	13.3701
27	18.7829	13.9798	9.2884	5.1848	2.3564	0.0403	1.5898	4.3656	8.4992	13.2592
28	18.8184	14.1343	9.3002	5.2147	2.3566	0.0392	1.4941	4.3575	7.9585	13.3768
29	18.8642	14.1823	9.3044	5.2355	2.3591	0.0359	1.5053	4.3482	7.9429	13.3103
30	18.9169	14.4514	10.2547	6.1094	2.3548	0.0367	1.5275	4.3171	7.8808	12.7563
31	18.2804	14.5985	9.5601	5.3959	2.3620	0.0311	1.5682	4.2876	8.9568	13.1765
32	19.0248	14.7308	9.5548	5.3777	2.3668	0.0282	1.5567	4.2857	8.9219	13.2015
33	19.0944	14.7502	9.5987	5.3671	2.3696	0.3104	1.5275	4.2823	8.7669	13.1881
34	19.1717	14.8135	9.5884	5.3298	2.5234	0.0307	1.5170	4.2706	7.9610	13.1759
35	19.1917	14.8607	9.5313	5.3668	2.5243	0.0289	1.5249	4.2651	7.9501	13.1613
36	19.2210	14.9763	9.5466	5.3501	2.5262	0.0238	1.4831	4.1583	7.9403	13.0891
37	19.2769	15.0643	9.5785	5.3450	2.5334	0.0376	1.4604	4.0666	7.8851	12.2145
38	19.2942	15.1050	9.5701	5.5089	2.5400	0.0360	1.4355	4.0586	7.8153	11.8941
39	19.3149	15.1193	9.5904	5.5275	2.5400	0.0359	1.4849	4.0489	7.7038	12.2294
40	19.4642	15.2287	10.5972	6.7998	2.5410	0.0369	1.4581	4.0124	7.5600	11.4051

Using the values of shear modulus and damping ratio, the shear modulus and damping ratio curves can be plotted. Figure 4.2 shows the shear modulus and damping ratio curves for some selected sites.

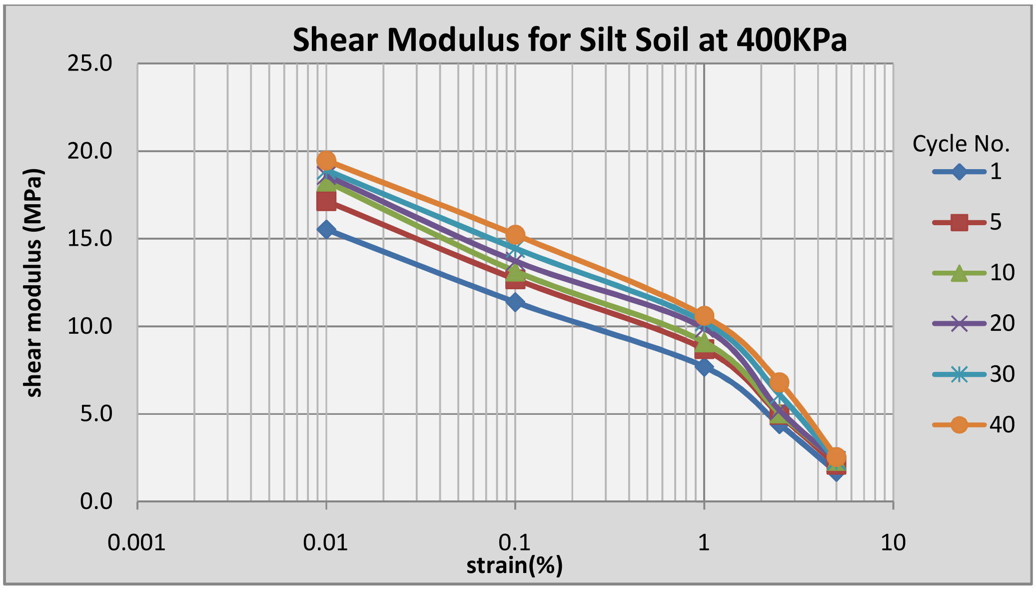


Figure 4.2 Shear Modulus curves of silt soil for selected cycles

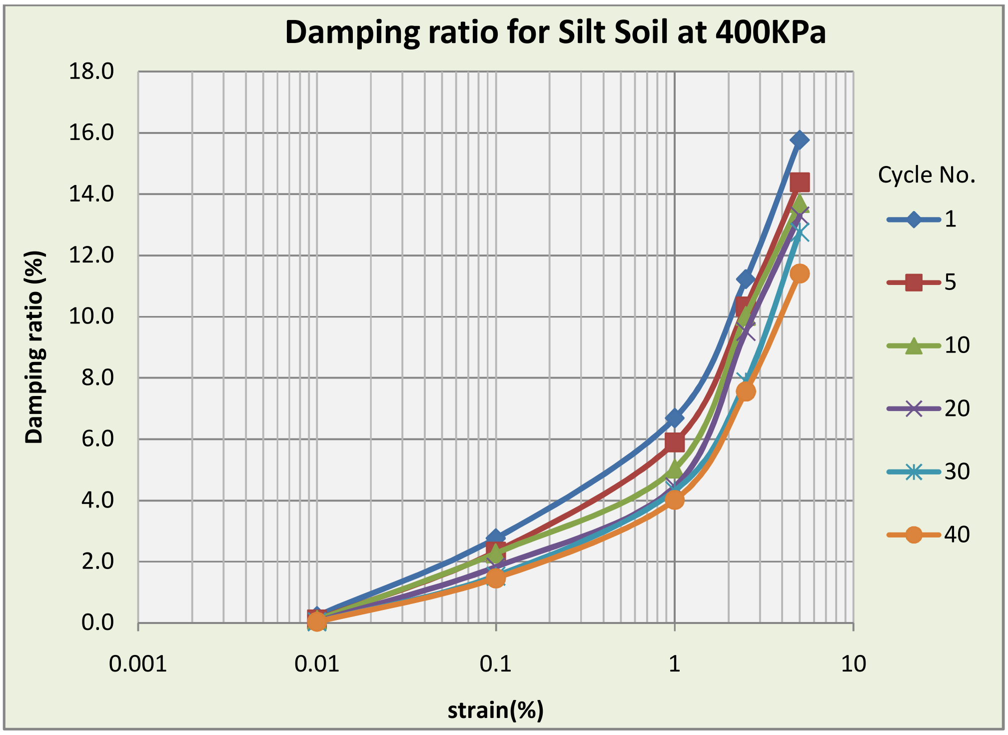
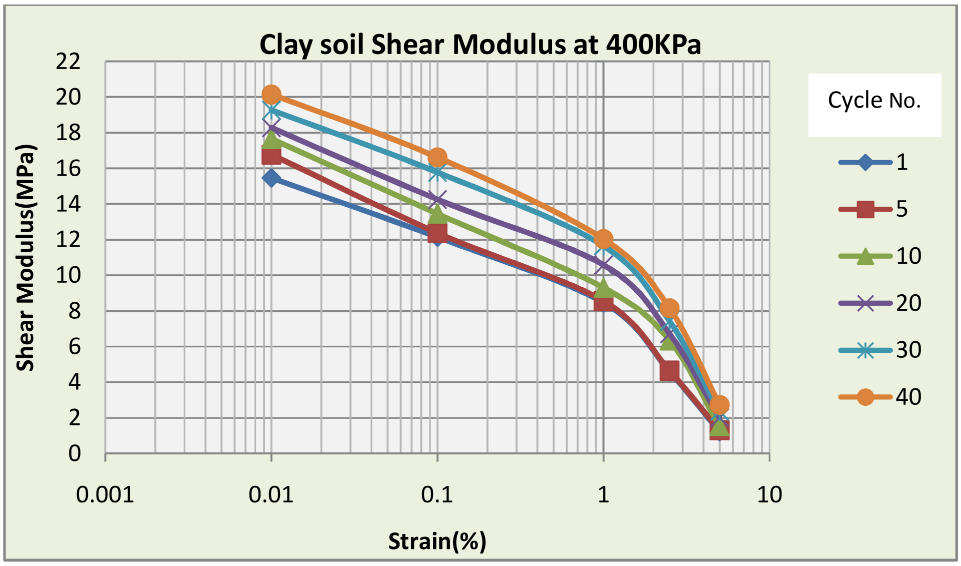


Figure 4 3 Damping Ratio curves of silt soil for selected cycles

Table 4. 4 Shear modulus and Damping ratio values of clay soil, at 400KPa axial stress

Clay soil, 400KPa										
Strain Level	0.01%	0.1%	1%	2.5%	5%	0.01%	0.1%	1%	2.5%	5%
No. Cycle	Shear Modulus G(MPa)					Damping Ratio β(%)				
1	15.4650	12.1386	8.4585	4.6041	1.2429	0.2209	2.3094	5.7188	10.8304	14.6127
5	16.7660	12.3680	8.5512	4.6421	1.3077	0.1133	2.2469	5.4473	10.2274	14.0222
10	17.6413	13.4498	9.2958	6.3419	1.5336	0.0948	2.0790	4.7283	9.6814	12.9067
20	18.2729	14.2506	10.5766	6.7117	2.0042	0.0814	1.5620	4.2671	9.3633	12.2808
30	19.2695	15.7776	11.6445	7.4225	2.3559	0.0569	1.3105	4.0936	8.3087	11.4658
40	20.1448	16.6251	12.0341	8.1499	2.7285	0.0261	1.1114	3.5717	7.3651	9.9707



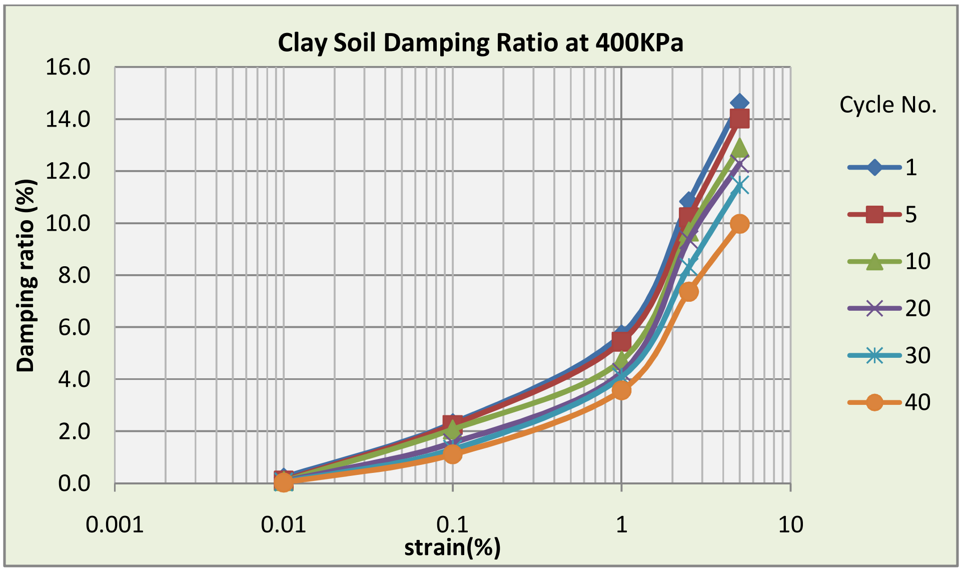


Figure 4.4: Shear Modulus and Damping Ratio curves of clay soil for selected cycles

4.2 Effect of number of cycles and Axial Loads

Previous study shows strength degradation would occur from cycle to cycle as cycling loading continues. As indicated in figures 4.2 to 4.4 the variation of both shear modulus and damping ratio for different cycles are nearly the same. The effect of number of cycles with increase axial loads shows slight difference on the shear modulus and damping ratio values.

As mentioned in the previous sections it is the consolidation stress which has a significant influence on shear modulus and damping values. In this research samples were consolidated under an axial stress of 100, 200 and 400KPa in order to evaluate the influence of consolidation stress. The variation of both shear modulus and damping ratio at different axial stress showed in figures 4.5 to 4.8.

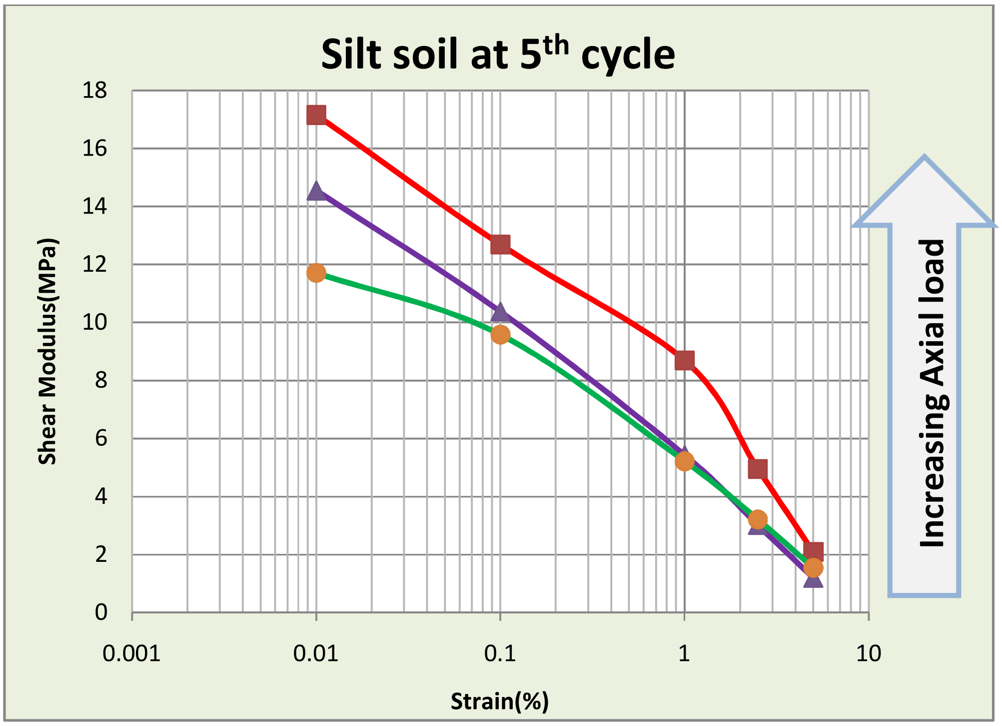


Figure 4. 5 Effect of axial loads on shear modulus of the Silt soil

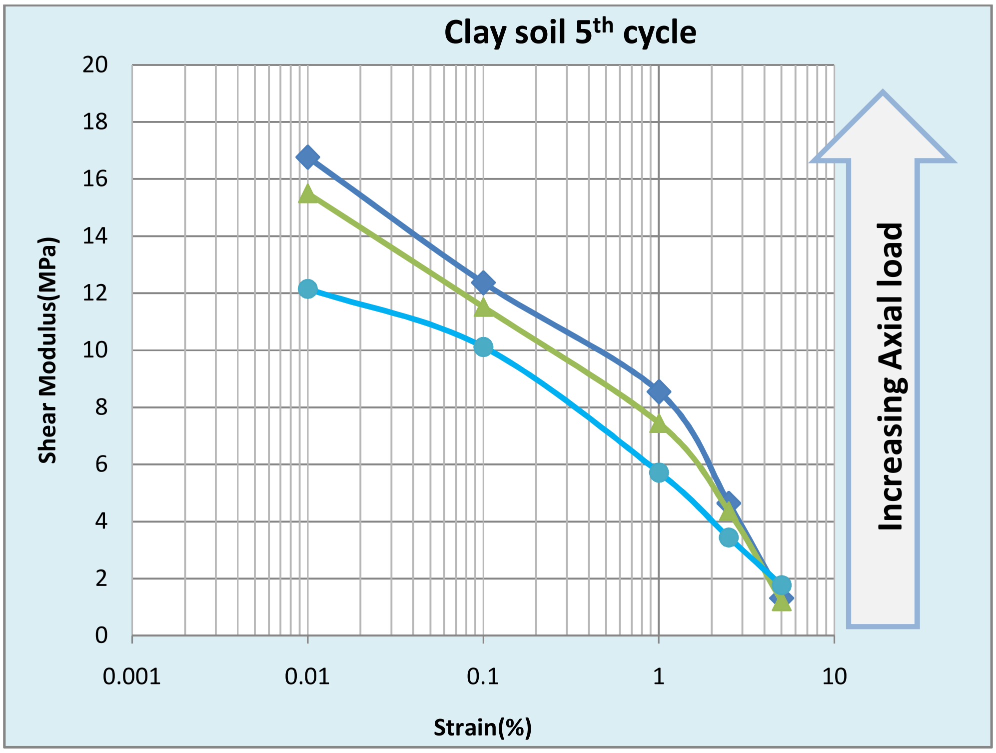


Figure 4 6 Effect of axial loads on shear modulus of the clay soil

Figures 4-5 and 4.6 (for both silt and clay soil), it can be seen that samples consolidated to a higher consolidation stress have higher shear modulus values.

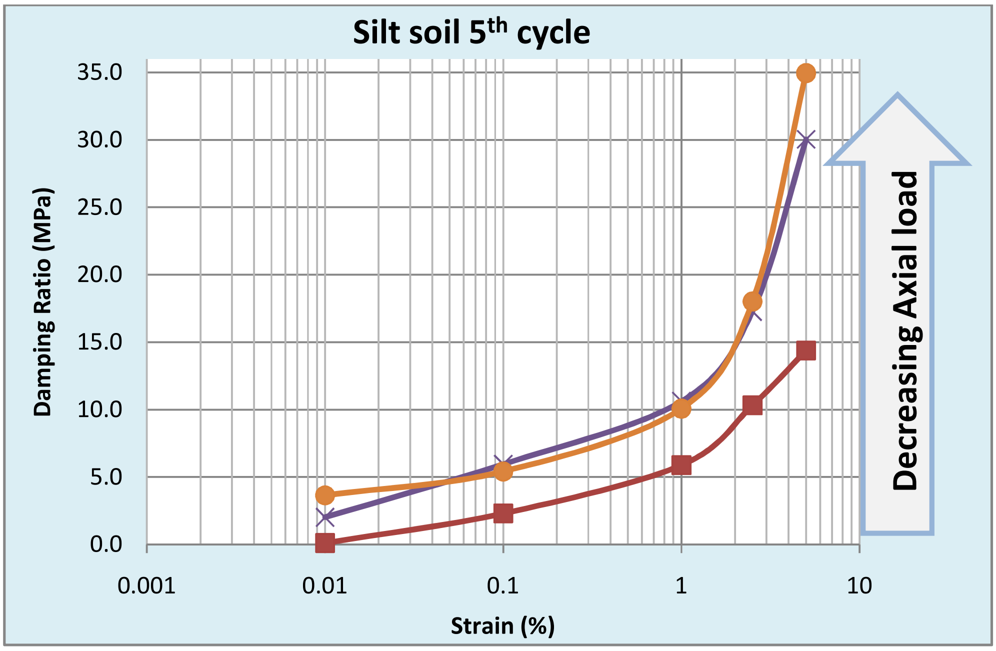


Figure 4 .7 Effect of axial loads on damping ratio of silt soil

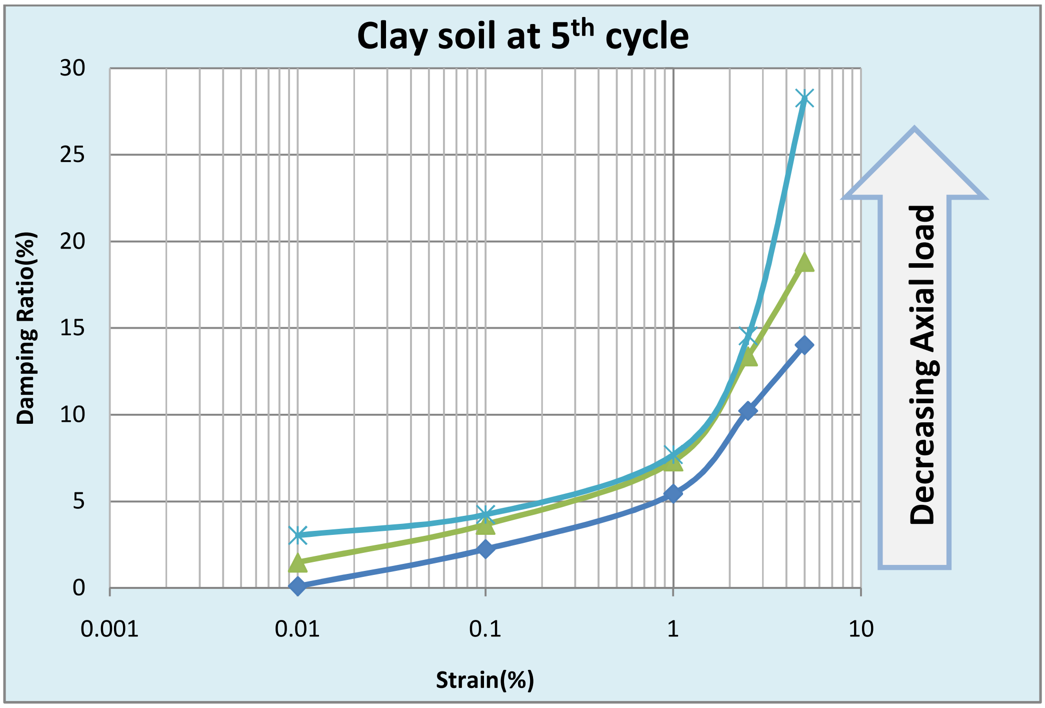


Figure 4. 8 Effect of axial loads on damping ratio of clay soil

4.3 Computation of maximum shear modulus

The maximum shear modulus, modulus of reduction and damping ratio of the soil are important parameters for dynamic analysis of the soil. The maximum shear modulus can be obtained from low strain seismic geophysical test which involves the measurements of body wave velocities which can be easily related to low-strain soil moduli or by using empirical correlations with index properties of soil. The maximum shear modulus, G_{max} which corresponds to very low strain levels cannot be determined using cyclic simple shear tests. Therefore, for the sake of comparison in this study, computation of Gmax is done using the expression in Equation (4.1) and presented below.

$$G_{max} = 14760\frac{(2.973-e)^2}{1+e}(OCR)^a(\sigma'_m)^{0.5}$$

Where G_{max} is the maximum shear modulus (lb/ft^2) and e is void ratio of the soil

$$e = \frac{G_s\gamma_w}{\gamma_d} - 12$$

Where; a is a parameter that depend on the plasticity index of the soil and determined using Table 4.5.

Table 4 .5 values of a with respect to plasticity index [26]

PI	a
0	0
20	0.18
40	0.30
60	0.41
80	0.48
≥ 100	0.5

The overconsolidation ratio (OCR) of soil is expressed as:

$$OCR = \frac{\sigma'_c}{\sigma'_p}$$

Where σ_c' is pre-consolidation pressure of a specimen

σ_p' is present effective vertical pressure

σ_m' is mean principal effective stress (lb/ft^2)

$$\sigma_m = \frac{\rho_1+\sigma_2+\sigma_3}{3}$$

Where σ_1 is the axial stress and $\sigma_2 = \sigma_3$ are the lateral confining stresses

Using K_0 loading condition, lateral stresses can be determined from the applied axial stress as:

$$\rho_3 = \sigma_2 = k_o\sigma_1$$

K_0 is coefficient of earth pressure at rest and estimated reasonably to 0.5 for these silt soils.

From Table 4.5, the value of a range from 0 to 0.41 as the PI value goes from 0 to 60. Linear interpolation was used for the determination of a for each test pit (see Table 4.8).

Table 4.6 Values of the parameters PI, a and e for all test pits

Parameter	Secha Chamo High School	AMU-community School(Sikela)
PI	54	12
a	0.377	0.108
e	1.3051	1.3077

Computation of G_{max} has been made for both silt and clay soil using the above procedures and tabulated in Table 4.7. For the sake of comparison G_{max} values for different soil types are presented in Table 4.8.

Table 4. 7 Typical Gmax values of soils [9]

Type of soil	Initial shear modulus Gmax (KPa)
Soft clays	2750 - 13750
Firm clays	6900 -34500
Silty sands	27600 -138000
Dense Sands and Gravel	69000 -345000

Table 4. 8 Computed G_{max} values with different axial stresses

Axial loads 100KPa		
Soil type	**Clay Soil**	**Silt Soil**
Pre-consolidation pressure(KPa)	198	135
σ_m(lb/ft^2)	1392.37	1392.37
OCR	1.98	1.35
Gmax(lb/ft^2)	859936.383	683674.849
Gmax(KPa)	**41173.9767**	**32734.5288**
Axial loads 200KPa		
Soil type	**Clay Soil**	**Silt Soil**
Pre-consolidation pressure(KPa)	198	135
σ_m(lb/ft^2)	2784.74	2784.74
OCR	0.99	0.675
Gmax(lb/ft^2)	936468.154	897125.8314
Gmax(KPa)	**44838.3378**	**42954.6171**
Axial loads 400KPa		
Soil type	**Clay Soil**	**Silt Soil**
Pre-consolidation pressure(KPa)	198	135
σ_m(lb/ft^2)	5569.48	5569.48
OCR	0.495	0.3375
Gmax(lb/ft^2)	1019811.025	1177218.613
Gmax(KPa)	48828.816	56365.5321

4.4 Modulus reduction (G/Gmax) values

The modulus reduction curves are the most widely used way of characterizing the modulus of soil under cyclic loading. Table 4.9 shows the modulus reduction values determined using the G values of 5th cycle and the calculated G_{max} values of table 4.8. The value of Gmax is used to obtain the normalized shear modulus which is then compared with those obtained from literature.

Table 4 9 Modulus ratio (G/Gmax) values

Axial Load	100KPa				
Shear Strain (%)	0.01	0.1	1	2.5	5
	G/Gmax				
Secha Chamo High School (Secha)	0.35412	0.27016	0.18044	0.10855	0.00559
AMU-Community School (Sikela)	0.3579	0.2928	0.1593	0.0982	0.0472
Axial Load	200KPa				
Shear Strain(%)	0.01	0.1	1	2.5	5
	G/Gmax				
Secha Chamo High School (Secha)	0.3458	0.2568	0.1661	0.0974	0.0269
AMU-Community School (Sikela)	0.3387	0.2416	0.1265	0.0763	0.0378
Axial Load	400KPa				
Shear Strain(%)	0.01	0.1	1	2.5	5
	G/Gmax				
Secha Chamo High School (Secha)	0.3274	0.2433	0.1551	0.0951	0.0268
AMU-Community School (Sikela)	0.27044	0.2252	0.1242	0.0779	0.0371

CHAPTER 5

COMPARISON OF TEST RESULTS WITH LITERATURES

5.1 INTRODUCTION

Previously, some researchers developed different modulus reduction and damping ratio for different soil types. Modulus reduction and damping ratio curves of sand and saturated clay developed by Seed and Indriss,(1970), modulus reduction and damping ratio curves of different PI soil developed by Vucetic and Dobry (1991), modulus reduction and damping ratio curves of silt soil developed by Ayalew ,and modulus reduction and damping ratio curves of silty clay soil developed by Abu are used to compare the test results of soils under study.

5.2 Shear modulus reduction

The computed shear modulus reduction values from Table 4.9 are plotted on the curves developed by Seed and Indris (fig. 5.1 -5.4). Similar plot is done on the curve of Vucetic and Dobry (fig. 5.5-5.6). Regarding local soils a comparsion is made with the curve done by Ayalew and Abu (fig. 5.7 -5.10).

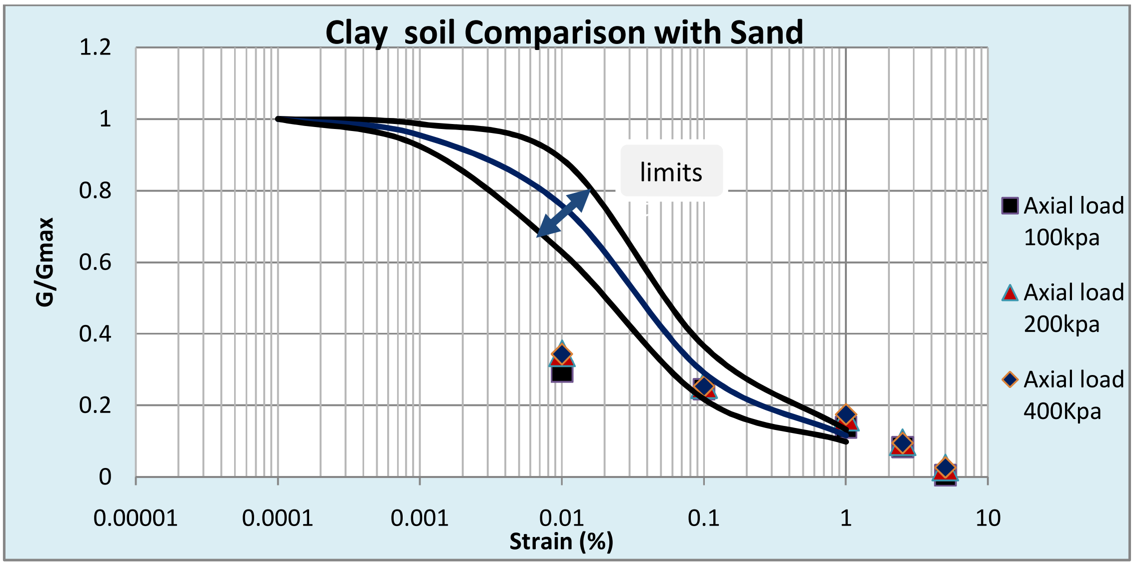


Figure 5. 1 location of modulus reduction values of Clay soil) as compared with curves developed for sand by Seed and Indriss (1970)

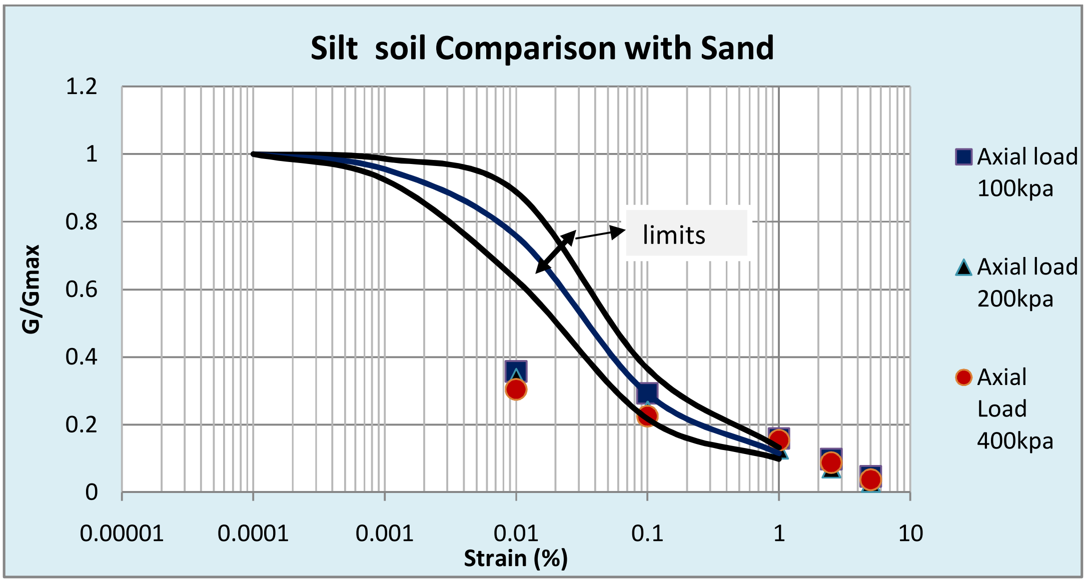


Figure 5. 2 location of modulus reduction values of Silt soil as compared with curves developed for sand by Seed and Indriss (1970)

From the above figure, it can be seen that the obtained normalized shear values are located lower than those suggested by literature at lower strains (γ=0.01%). For higher strains the obtained normalized shear values agree with those suggested by literature.

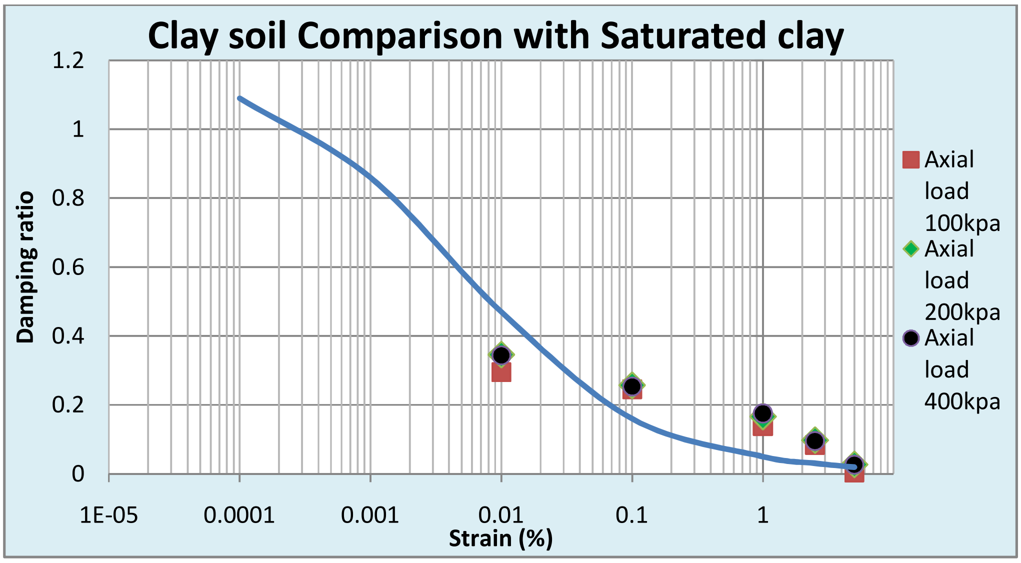


Figure 5. 3: Location of modulus reduction values of Clay soil as compared with curves developed for saturated clay by Seed and Indriss (1970)

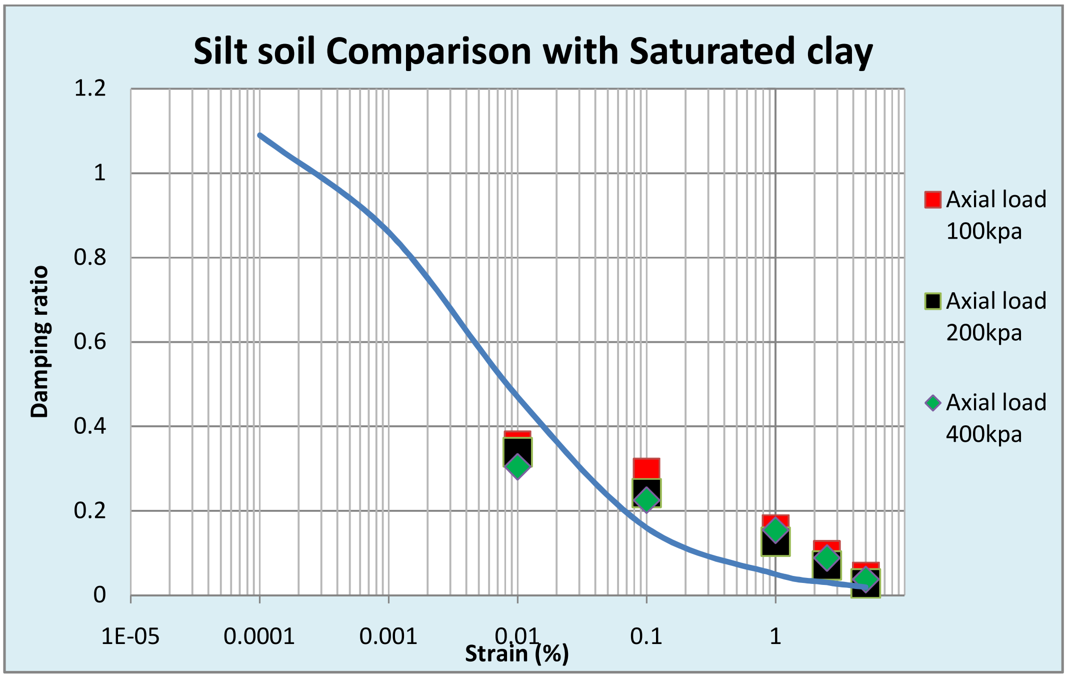


Figure 5. 4: Location of modulus reduction values of Silt soil as compared with curves developed for saturated clay by Seed and Indris (1970)

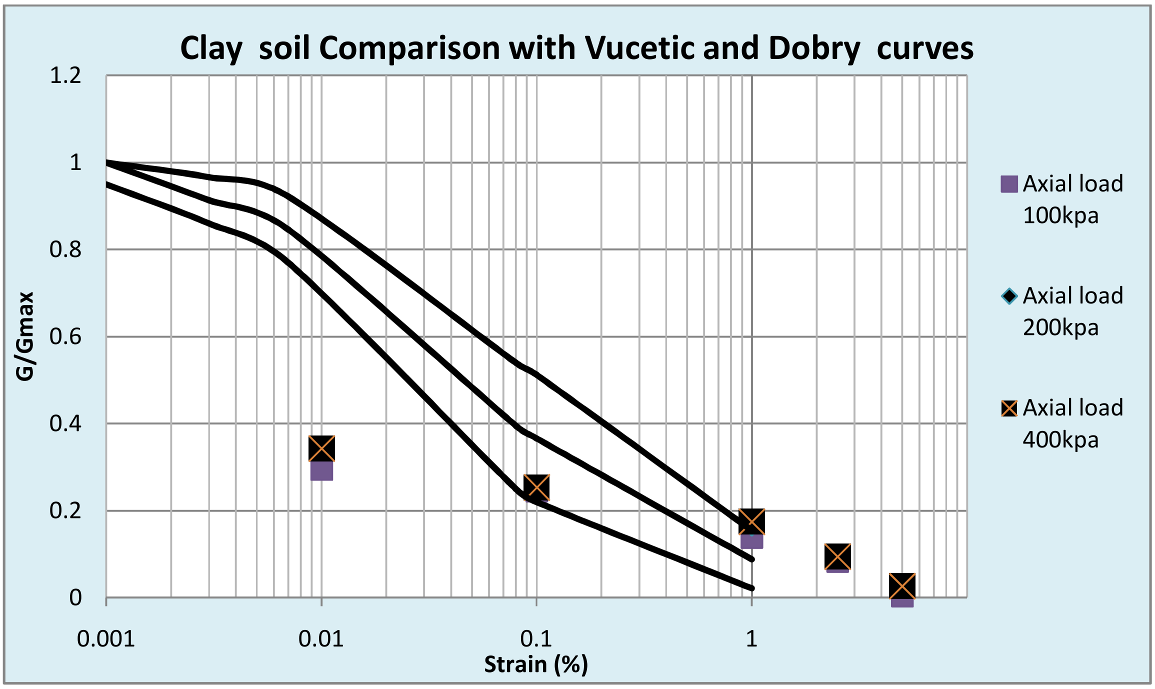


Figure 5 .5: Location of modulus reduction values of Clay soil as compared with curves developed by Vucetic and Dobry (1991)

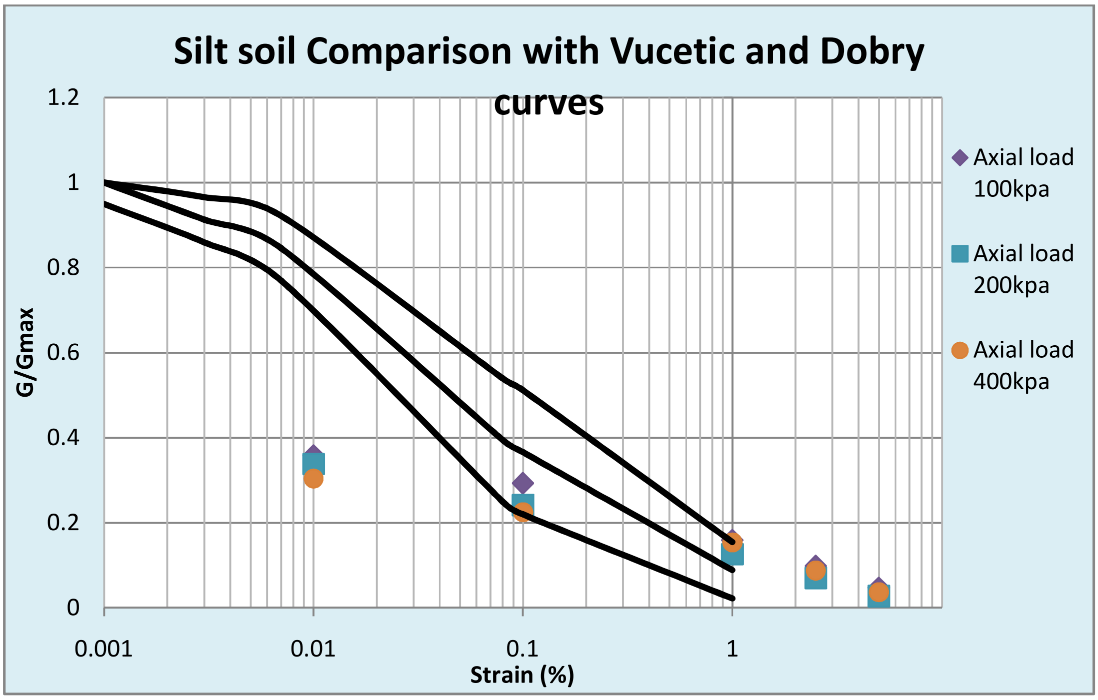


Figure 5 .6: Location of modulus reduction values of Silt soil as compared with curves developed by Vucetic and Dobry (1991)

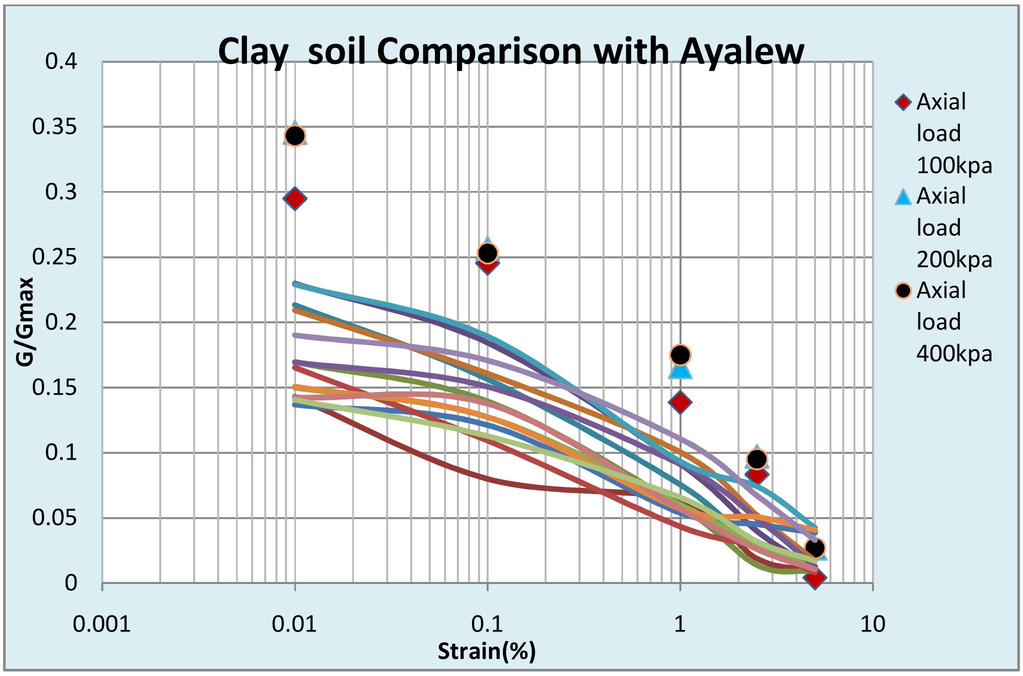


Figure 5. 7 Location of modulus reduction values of Clay soil as compared with Ayalew's curves developed for silt soil (2012)

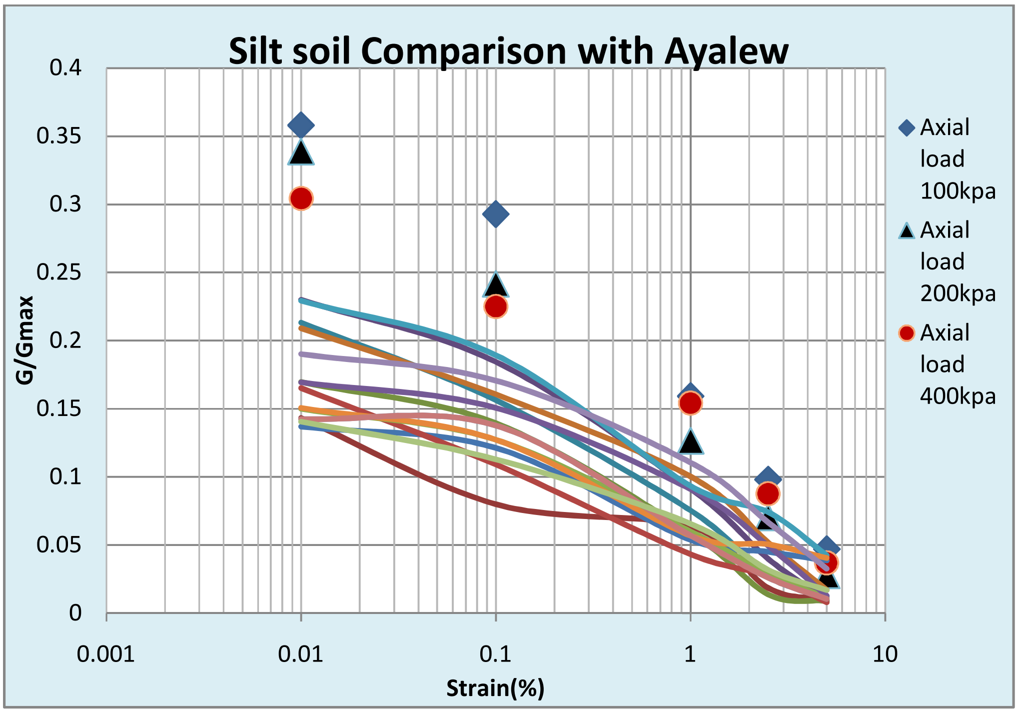


Figure 5. 8 Location of modulus reduction values of Silt soil (Sikela AMUCSc) as compared with Ayalew's curves developed for silt soil (2012)

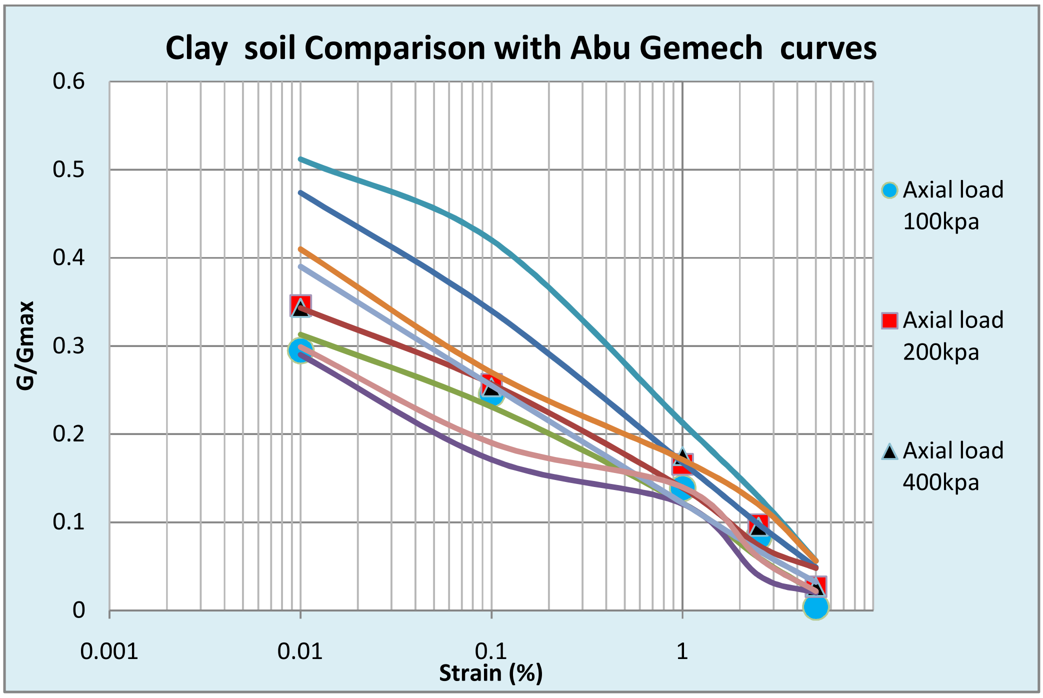


Figure 5. 5 Location of modulus reduction values of Clay soil as compared with ABu's curves developed for silt clay soil (2011)

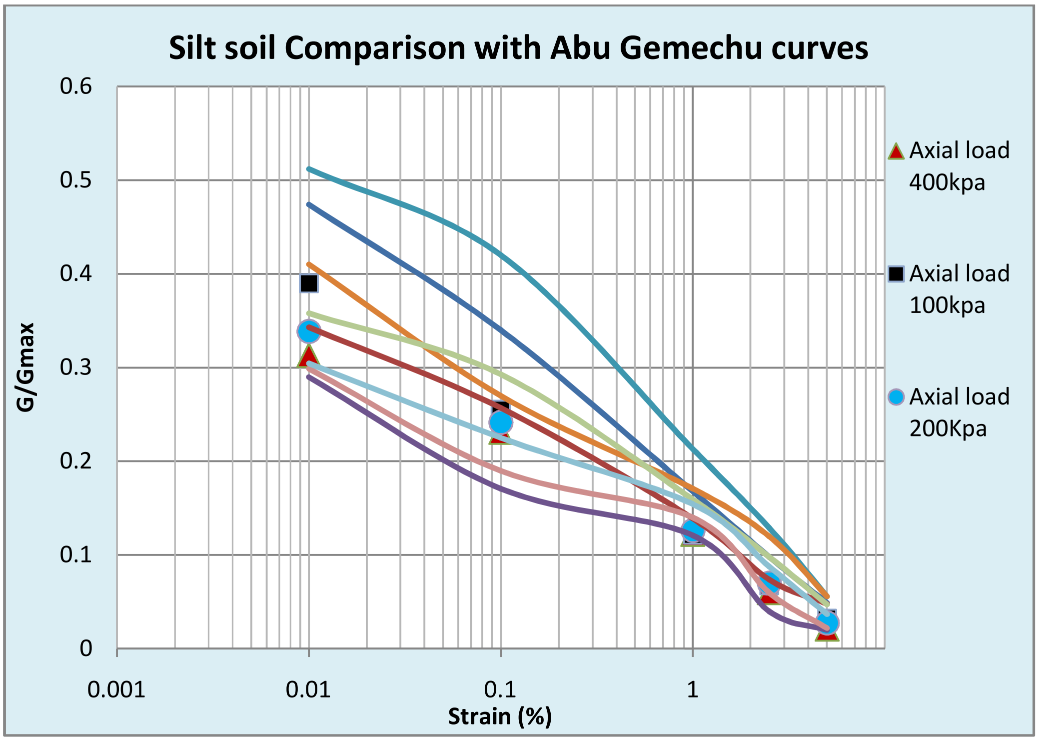


Figure 5 6: Location of modulus reduction values of Silt soil (Sikela AMUCSc) as compared with Abu's curves developed for silt soil (2011)

At lower strains ($\gamma \leq 0.01\%$) as shown (figures 5.5 and 5.6) that the obtained normalized shear values are located lower than those curves developed by Vucetic and Dobry (1991). Similalrly, figures 5.1 to 5.4 show similar trainds compared with the curve (at lower strains, $\gamma \leq 0.01\%$) doveloped by Seed and Indris (1970). However, at higher strains ($\gamma \geq 0.01\%$) the computed normalized shear values agree with those suggested by Seed and Indris (1970), and Vucetic and Dobry (1991). Regarding coparision with local soils (figures 5.7 & 5.10) the computed normalized shear values slightly higher than the curves developed by Ayalew for silt soil of Awassa (2012) but well fitted with curves developed by Abu for silty clay soil of Adama (2011).

5.3 Damping ratio

The computed damping ratio values of silt and clay soil (Tables 4.3 - 4.4) can be plotted and compared with different damping ratio curves from literatures. Figures 5.11 - 5.20 show location of damping ratio test results as compared with different literature curves.

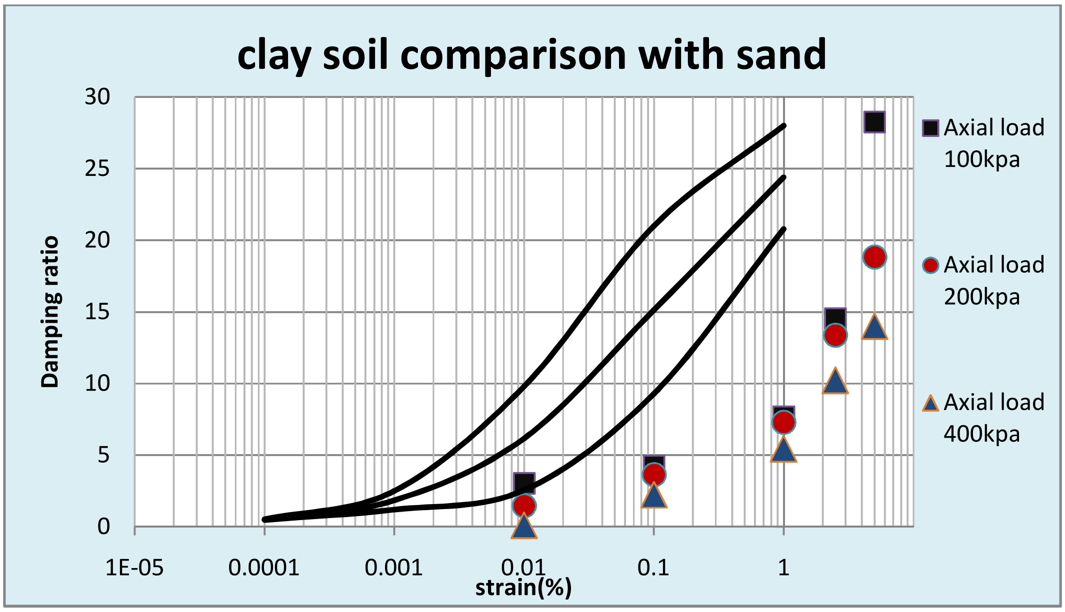


Figure 5 .11 Location of damping ratio values of Clay soil as compared with curves developed for sand by Seed and Indriss (1970)

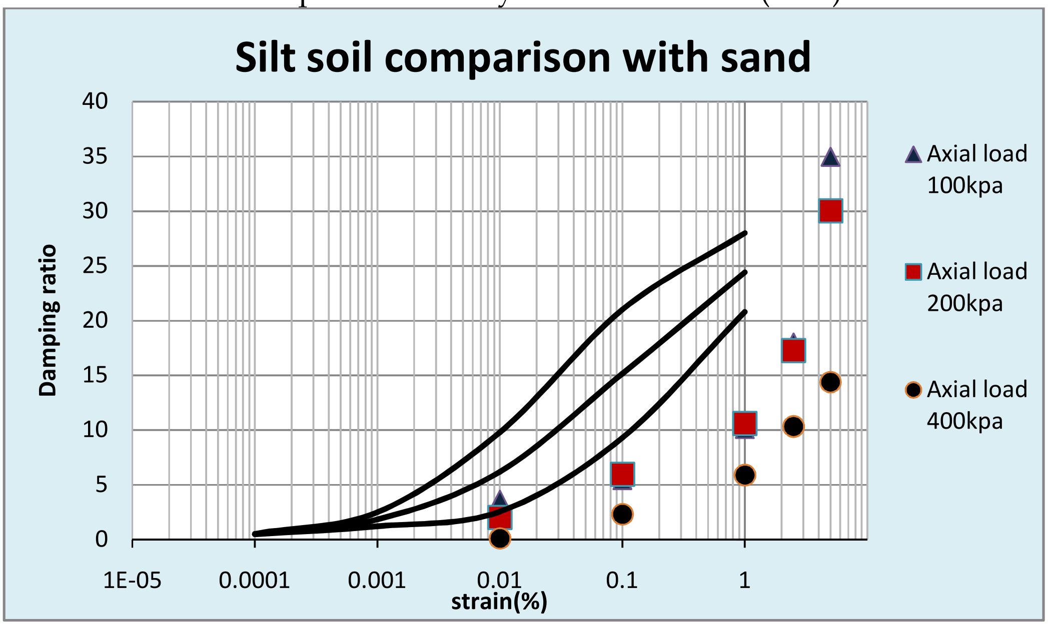


Figure 5.7 Location of damping ratio values of Silt soil as compared with curves developed for sand by Seed and Indriss (1970)

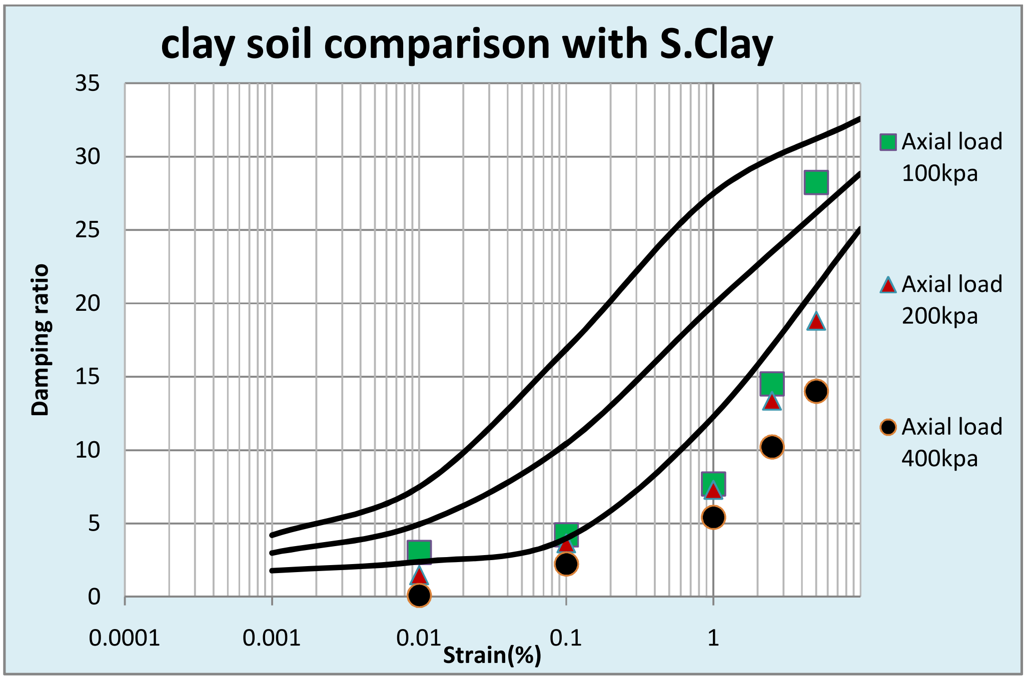


Figure5.8 Location of damping ratio values of Clay soil as compared with curves developed for saturated clay by Seed and Indriss (1970)

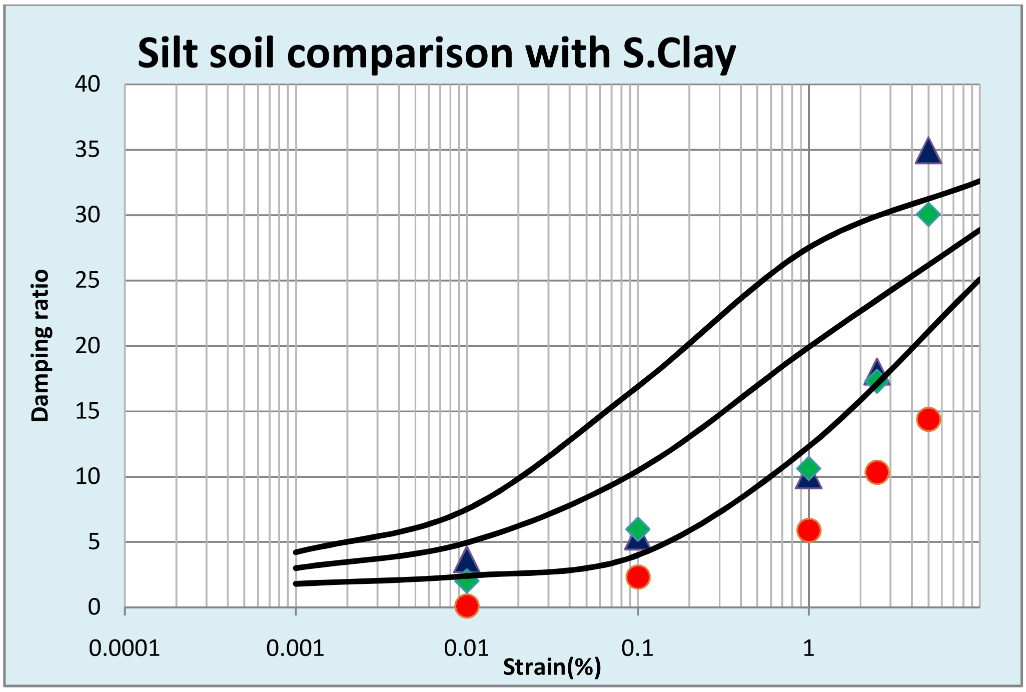


Figure 5.9 Location of damping ratio values of Silt soil as compared with curves developed for saturated clay by Seed and Indriss (1970)

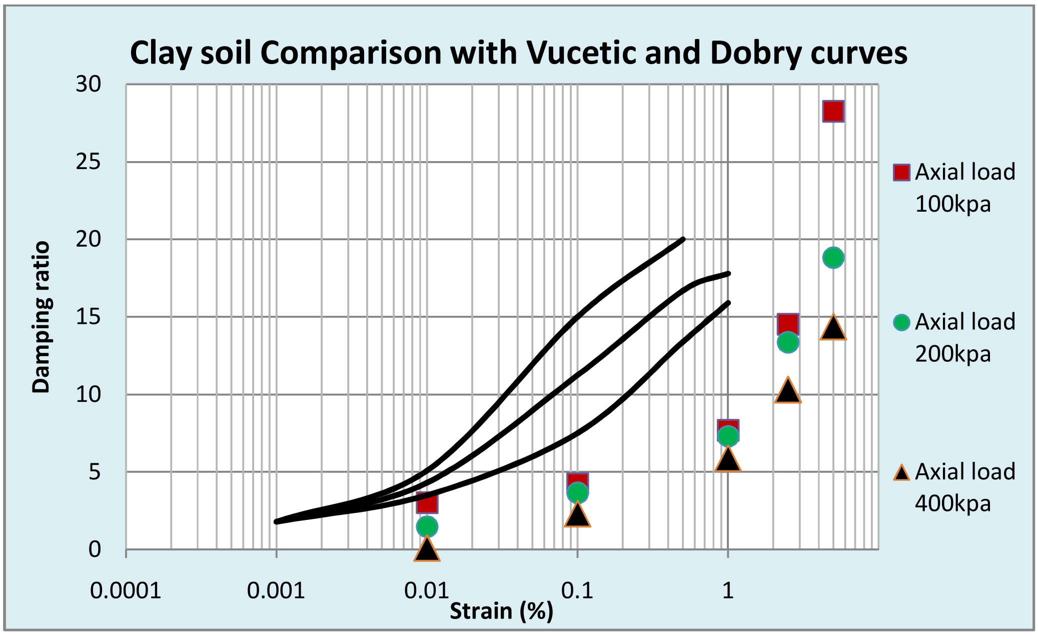


Figure 5.10 Location of damping ratio values of Clay soil as compared with curves developed by Vucetic and Dobry (1991)

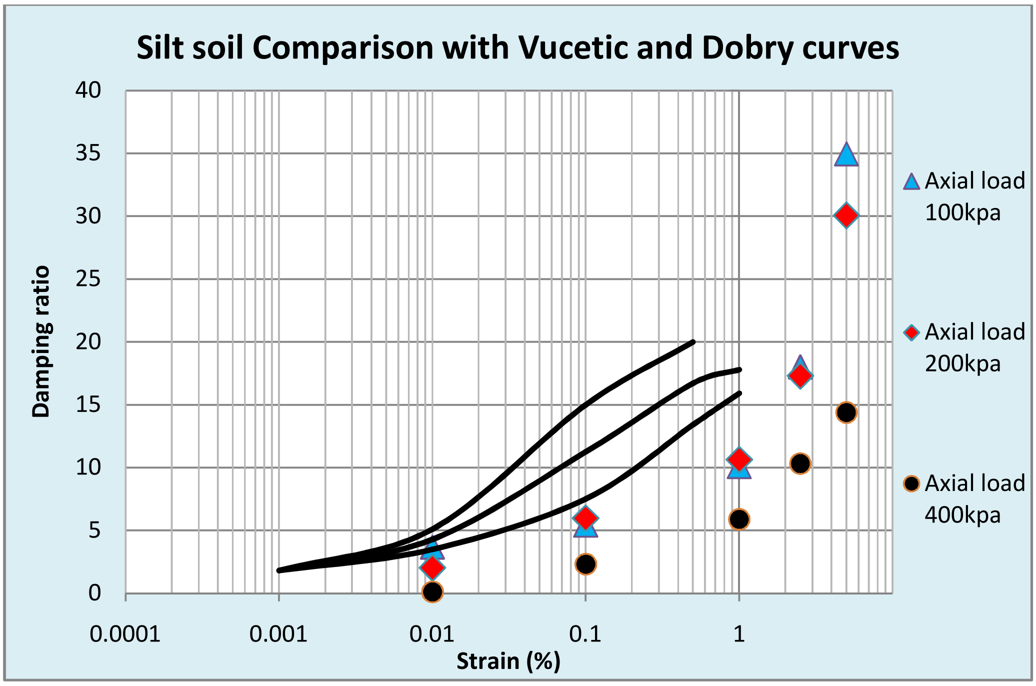


Figure 5. 11 Location of damping ratio values of Silt soil as compared with curves developed by Vucetic and Dobry (1991)

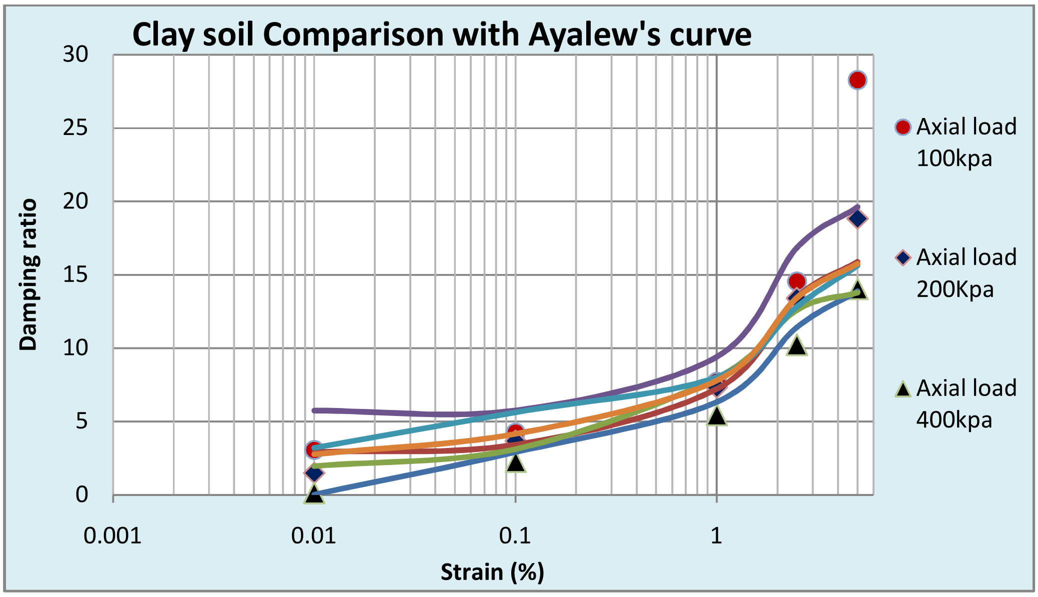


Figure 5.12: Location of damping ratio values of Clay soil as compared with curves developed by Ayalew (2012) for silt soil of Awassa

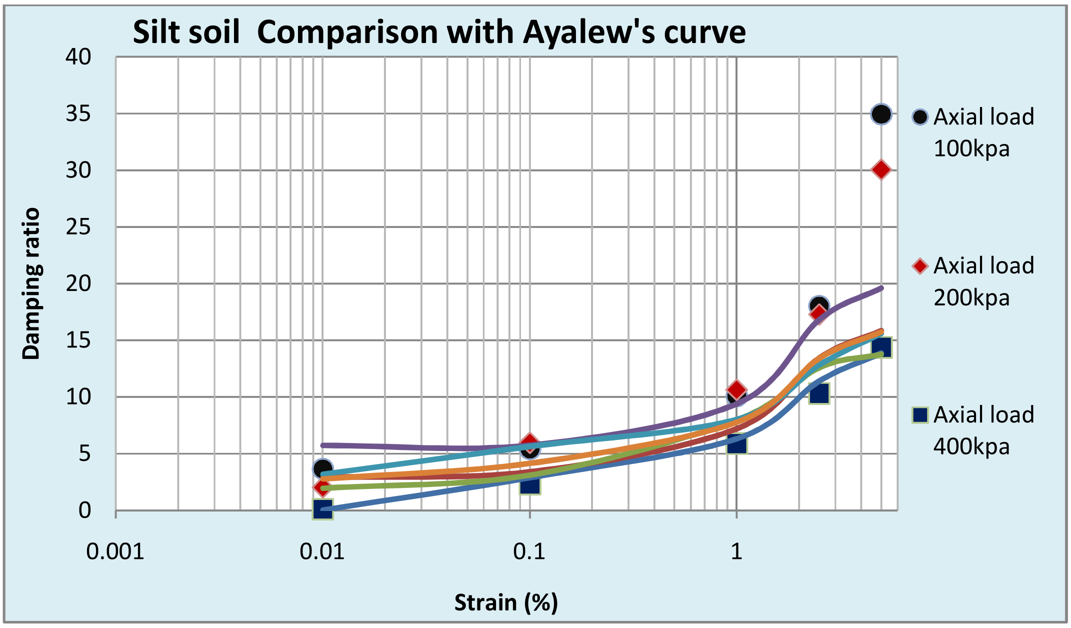


Figure 5.13: Location of damping ratio values of silt soil as compared with curves developed by Ayalew (2012) for silt soil of Awassa

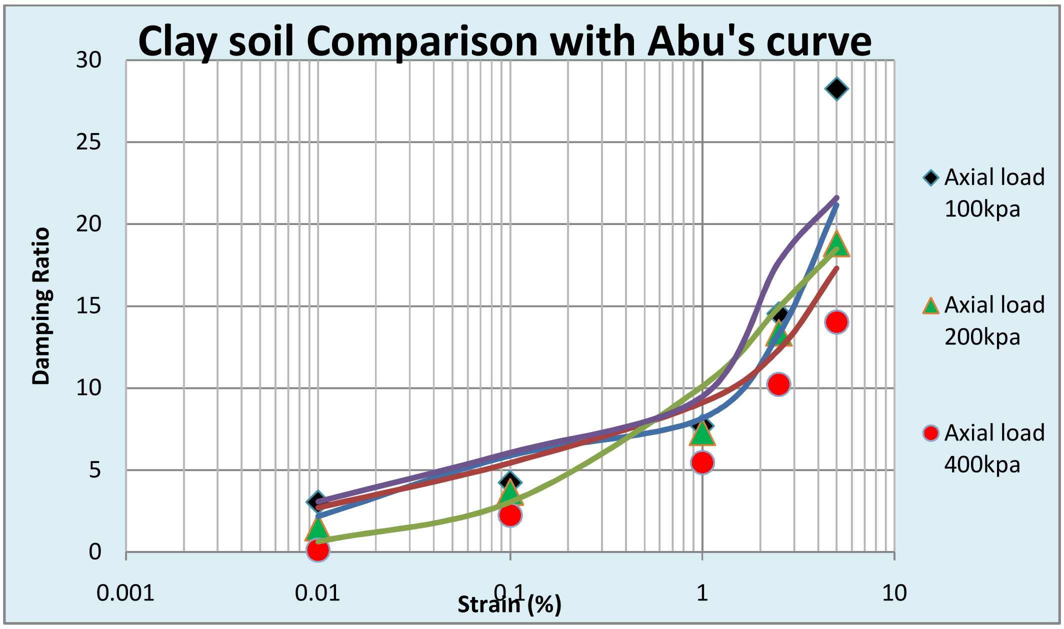


Figure 5.14 : Location of damping ratio values of Clay soil as compared with curves developed for silt by Abu (2011)

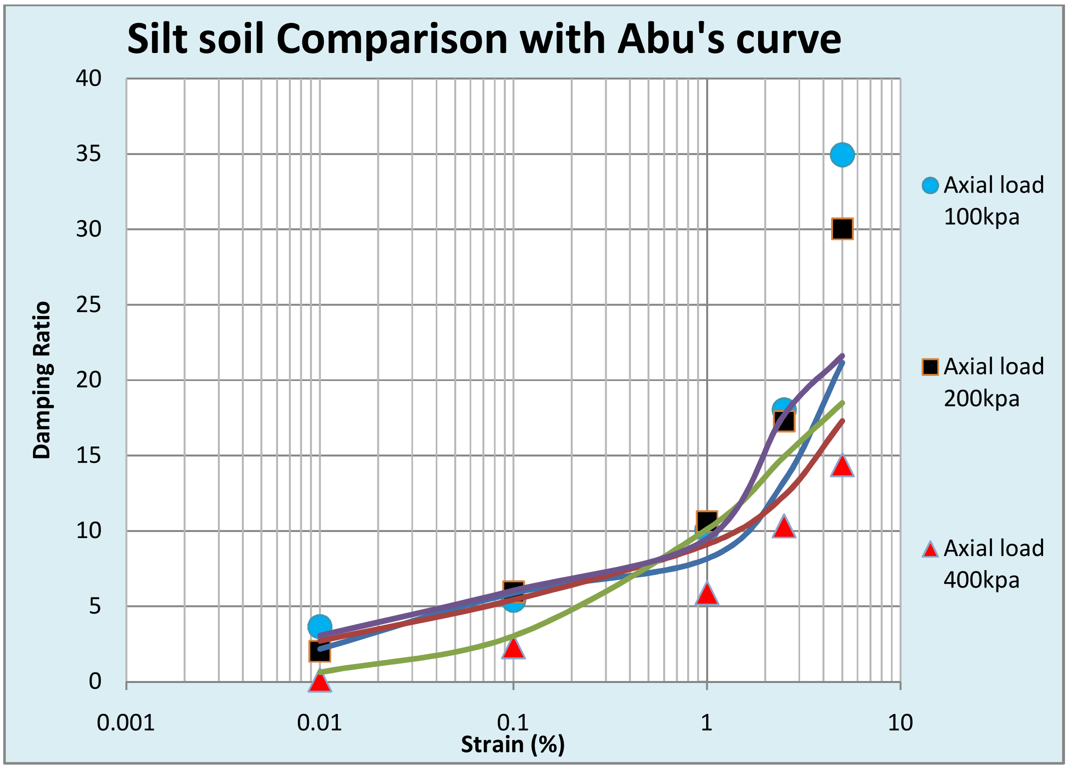


Figure 5.15: Location of damping ratio values of Silt soil as compared with curves developed for silt by Abu (2011)

The observed trends of damping ratio-shear strain (%) development under cyclic loading for both silt soil and clay soil are generally slightly less as compared to those previously noted literature curves developed by Seed and Indriss(1970) shown in figures 5.11-5.14. Similar trained are seen on curves developed by Vucetic and Dobry (1991) as shown figures 5.15 - 5.16. Regarding comparison with local soil damping ratio literature curves, both silt soil and clay soil are similar with silty clay soil (Adama town) and silt soils (Awassa town) as shown in Figures 5.17 – 5.20.

In reality, the position of damping ratio curves for silt soil test result would be expected to be in between the curve for sand and clay. However, this is not the case for the silt soil considered in this study compared to curves developed by Seed and Indriss (1970) and Vucetic and Dobry (1991) as shown in figures 5.13 -5.16. The test results lie around the lower side of the literature curves for clay soil. This divergence may be due to the following factors.

- For this study samples are remolded to field density and moisture content which cause significant changes to the particle structure and different to the conditioned of the soil in the literature.
- All laboratory devices have its own limitations and effect on the values, the cyclic ring shear test has some limitations, such as stress and strain non-uniformities associated with some specimen dimensions, difficulties in performing undrained testing, and friction that develops along the walls of the specimen confining rings. Most previous tests were conducted using cyclic triaxial testing machine. Thus, the type of the testing machine may cause variation on the values.
- The difference in testing conditions, as it was not possible to fully saturate the soil during consolidation.
- And other unknown factors.

In general the damping ratio results are relatively lower than literature results caused by the above and other factors.

CHAPTER 6

CONCLUSION AND RECOMMENDATION

6.1 Conclusion

This study has investigated of dynamic property of soil of Arba Minch Town from five test pits. A series of cyclic simple shear tests for two types of soil (clay and silt) have been performed using cyclic simple shear testing machine on remolded samples to determine the shear modulus and damping ratio of the soils. In addition, index property tests were conducted in the laboratory. From the index property, the soils are classified in to silt (Sikela town) and clay (Secha town). Field density and natural moisture content are determined so as to replicate some natural state during remoulding the sample in the laboratory. Based on the investigation obtained the following conclusions may be drawn.

1. The shear modulus and damping ratio obtained for both the silt and clay soil show similar trained compared with silty clay in Adama and silt in Awassa.
2. The shear modulus and damping ratio curves show similar trained comparing with the curves presented in the literature by Seed and Indriss (1970). Similar trained observed with curve developed by Vucetic and Dobry (1991).
3. The location of the shear modulus curves for both silt and clay soil (at a strain $\gamma \leq 0.01\%$) are lower than the literature curves developed by Seed and Indriss (1970). This shows that the testing conditions and sample preparation appear to have significant effect on the shear modulus values at small strain ($\gamma \leq 0.01\%$).
4. The location of the damping ratio curves for both silt and clay are generally lower than the literature curves developed by Seed and Indriss (1970), and Vucetic and Dobry (1991). This shows that testing condition and sample preparation appear to have significant effect on the damping ratio compared with its effect on the shear modulus values for all strain level.
5. The computed maximum shear modulus for silt is higher than for clay soil.

6. As the axial load increases, shear modulus increases while damping ratio decreases for both silt and clay.
7. The maximum shear modulus (G_{max}) increased with increasing confining pressure.
8. As the strain rate increases, shear modulus decreases while damping ratio increase

6.2 Recommendation

From this study the following recommendation can be drawn:

- The test should be repeated using cyclic triaxial testing machine to have a better understanding and characterization of the dynamic properties of the soils by comparing with the literature.
- Better to carry out tests with different moisture content to observe the effect of consistency on the shear modulus and damping ratio of the soil.
- Conducting field tests like standard penetration test helps to see the consistency of laboratory test results with field condition and estimate the potential sample disturbance effect on results of laboratory shear strength tests values.

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APPENDICES

LIST OF APPENDICES

Appendix – A .1 Test Results of Index Properties of Soil

Table A .1 : liquid limit and plastic limit test values

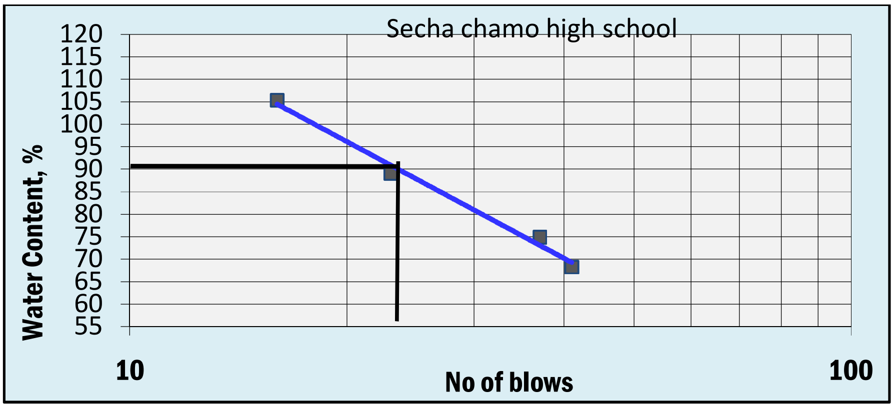
	Secha Chamo High School						
	Liquid Limit				Plastic Limit		
Trial No	1	2	3	4	1	2	3
No of blows	43	35	22	14	-----	------	------
Water content,(%)	66.19	83.87	90.24	114.28	29.71	36.88	43.82
Average (%)	For 25 blows------91				37		

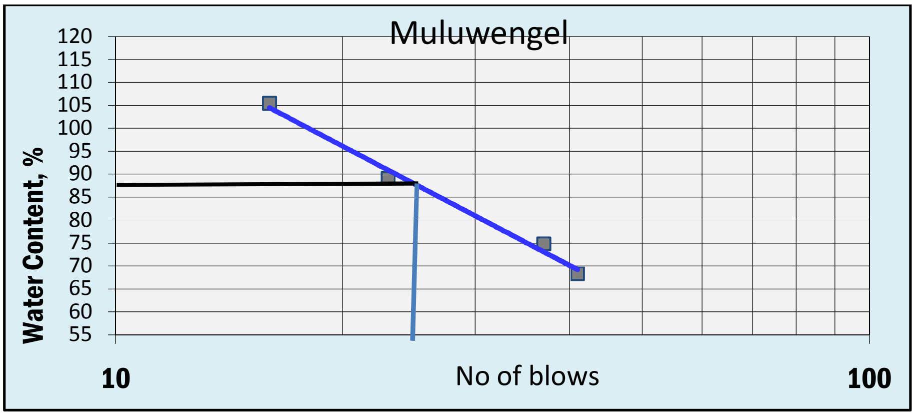
	Muluwengel						
	Liquid Limit				Plastic Limit		
Trial No	1	2	3	4	1	2	3
No of blows	41	37	23	16	-----	------	------
Water content,(%)	68.32	74.86	89.12	105.41	30.12	37.69	46.82
Average (%)	For 25 blows------88				38		

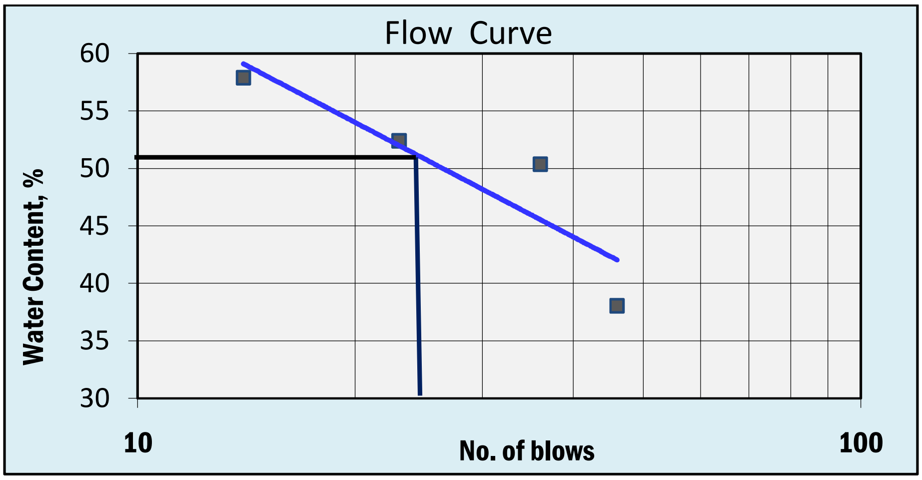
	AMU-CSc						
	Liquid Limit				Plastic Limit		
Trial No	1	2	3	4	1	2	3
No of blows	39.74	47.69	55.64	60.94	-----	------	------
Water content,(%)	46	36	23	14	35.1	38.61	43.29
Average (%)	For 25 blows------51				39		

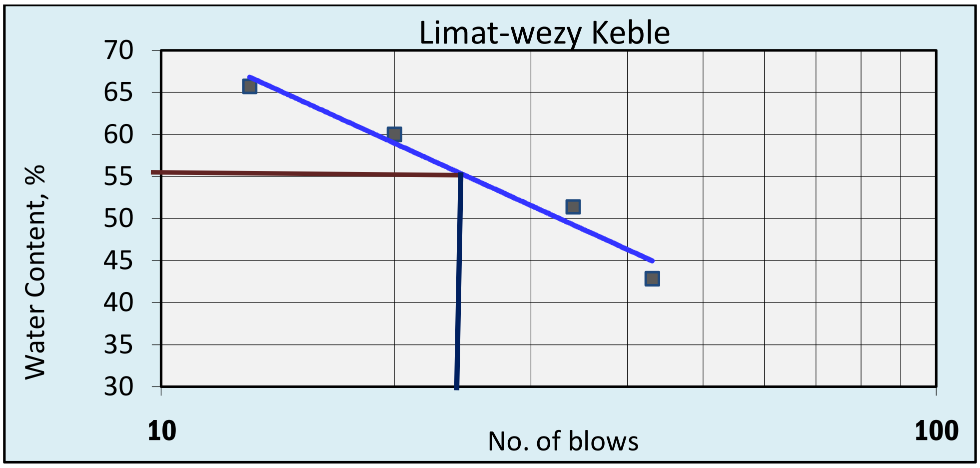
	Limat-wezy Keble						
	Liquid Limit				Plastic Limit		
Trial No	1	2	3	4	1	2	3
No of blows	42.86	51.43	60.00	65.71	-----	------	------
Water content,(%)	43	34	20	13	37.8	41.58	46.62
Average (%)	For 25 blows------55				42		

Edigetber Keble (stadium)							
	Liquid Limit				Plastic Limit		
Trial No	1	2	3	4	1	2	3
No of blows	42.08	50.49	58.91	64.52	-----	------	------
Water content,(%)	45	35	22	14	36.90	40.59	45.51
Average (%)	For 25 blows------54				41		









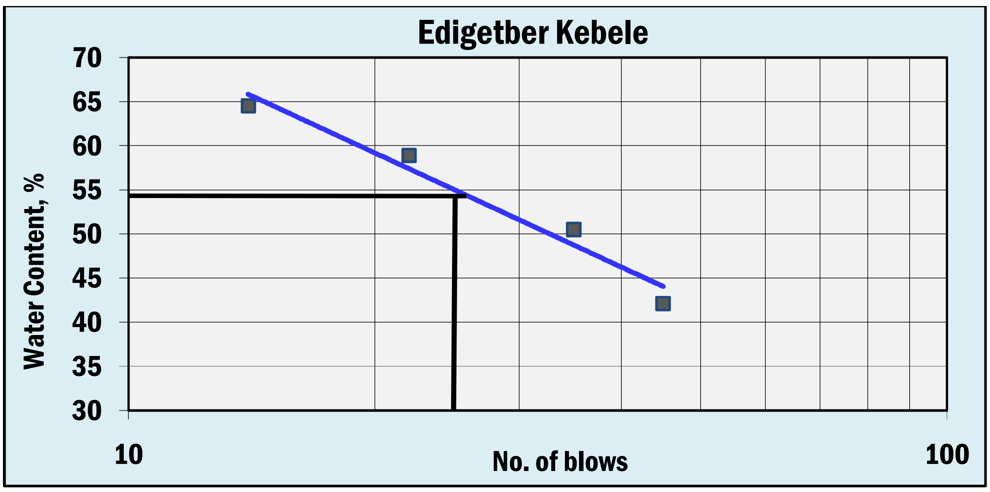
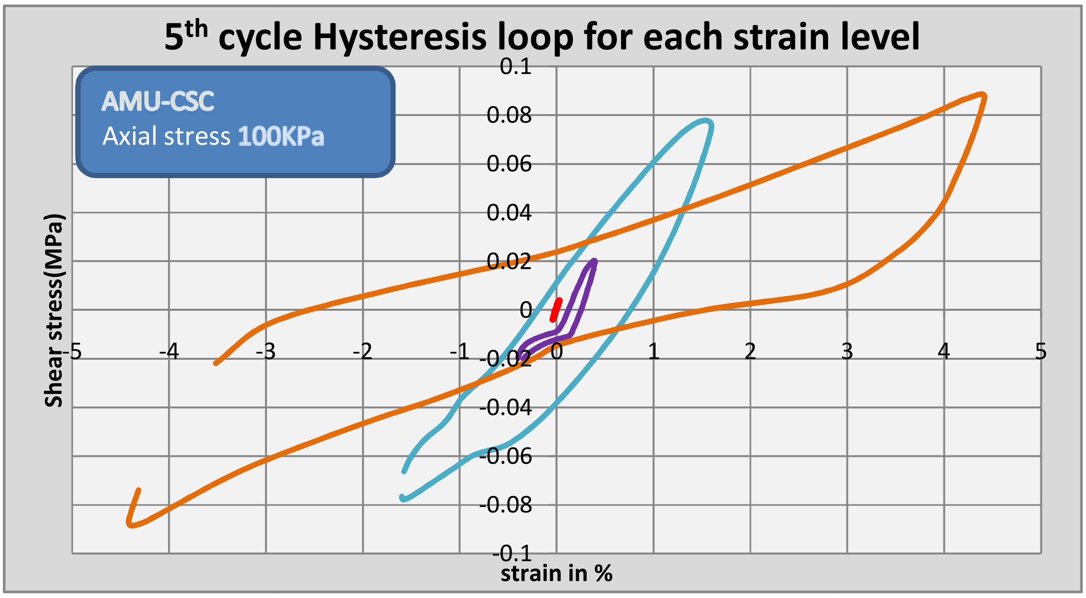


Figure A 1: Liquid limit graphs

Spendix B 1 Hysteresis loop of the 5th cycles of each test pits



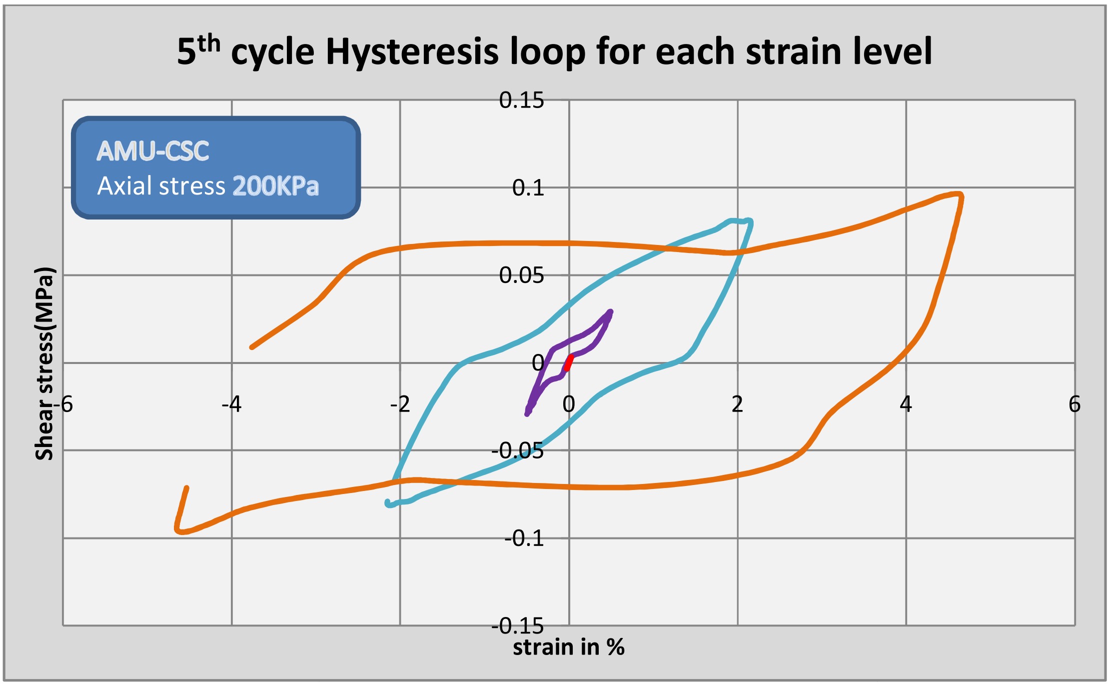
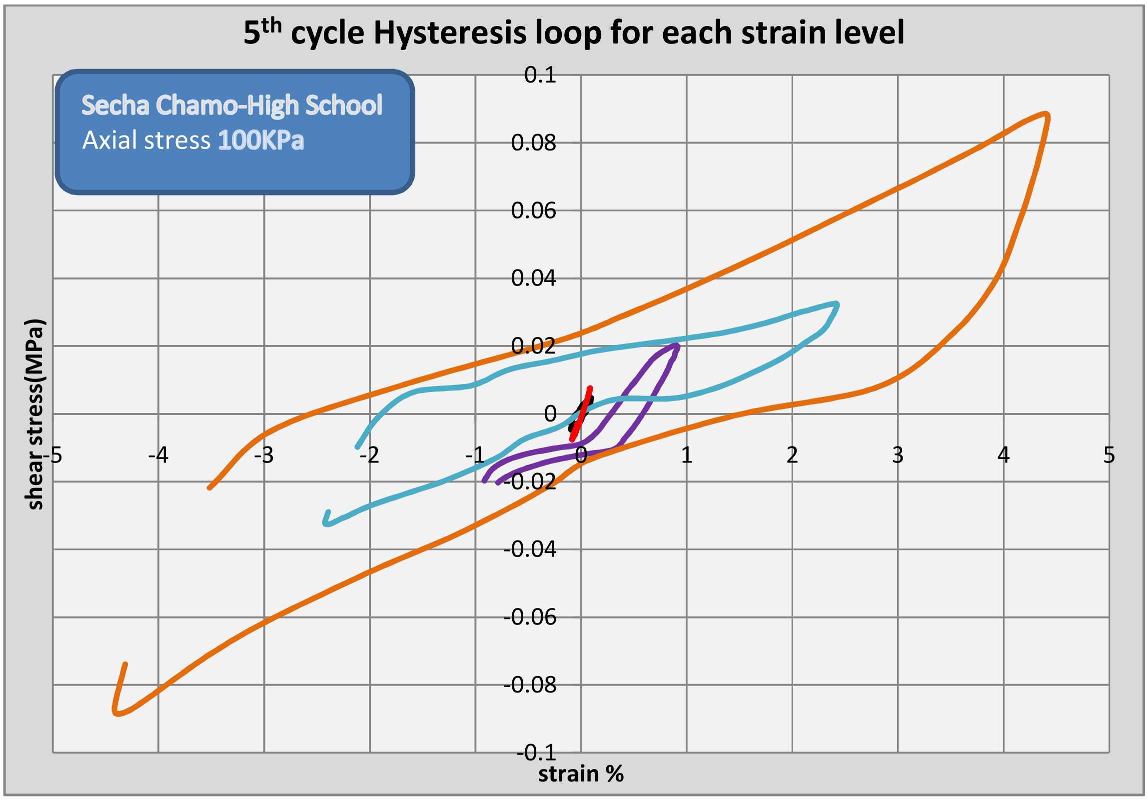


Figure B 1: Hysteresis loops of each strain levels with axial stresses indicated for silt soil



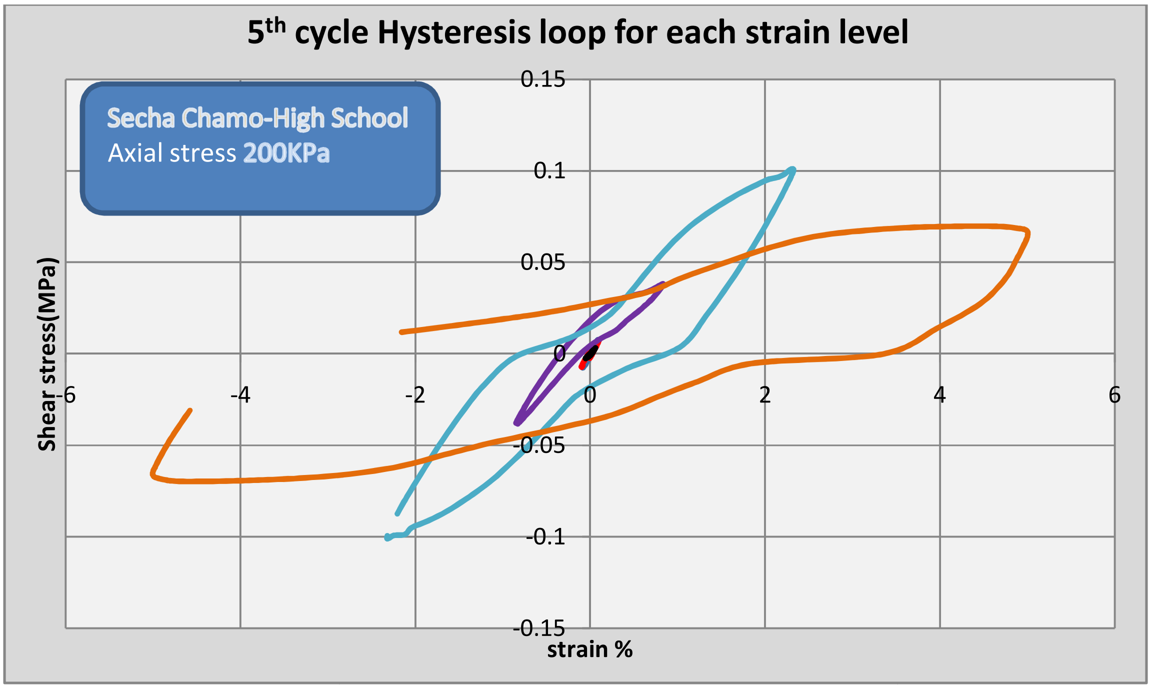


Figure B 2:Hysteresis loops of each strain levels with axial stresses indicated

Appendix – C. 2 SHEAR MODULUS AND DAMPING RATIO VALUES OF SELECTED CYCLES

Table C 1 Shear modulus and Damping ratio values of AMU-Community School (silt soil) pit

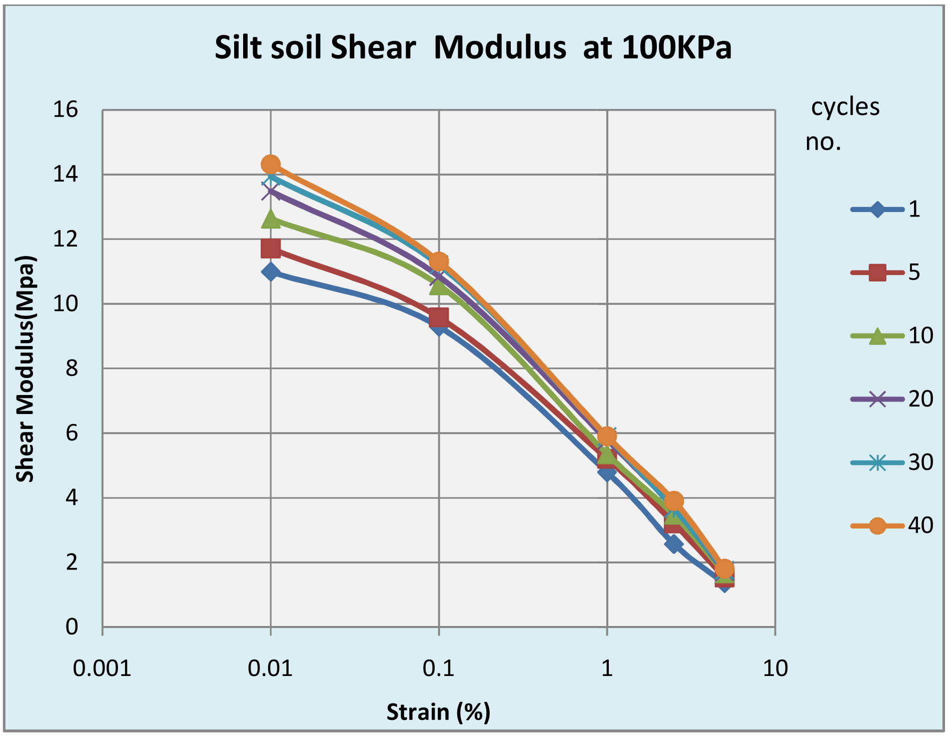
AMU-Community School pit,100KPa										
Strain Level	0.01%	0.1%	1%	2.5%	5%	0.01%	0.1%	1%	2.5%	5%
No. Cycle	Shear Modulus G(MPa)					Damping Ratio β(%)				
1	10.9914	9.2963	4.7957	2.5758	1.3651	4.1482	6.5363	11.6687	18.1908	38.3090
5	11.7161	9.5853	5.2161	3.2141	1.5446	3.6572	5.4187	10.0878	18.0283	34.9570
10	12.6423	10.5862	5.3605	3.4834	1.6744	3.6797	5.1873	8.5846	16.5474	33.1679
20	13.4862	10.8491	5.7949	3.6944	1.7238	3.0612	3.5846	8.4778	15.6242	30.2538
30	13.9445	11.2133	5.8955	3.6772	1.7679	2.0263	3.6194	7.1554	14.0497	26.1496
40	14.3146	11.3119	5.9033	3.9062	1.8124	1.9069	3.3739	6.9518	13.4994	24.9741

AMU-Community School pit,200KPa										
Strain Level	0.01%	0.1%	1%	2.5%	5%	0.01%	0.1%	1%	2.5%	5%
No. Cycle	Shear Modulus G(MPa)					Damping Ratio β(%)				
1	13.6480	10.1366	5.1012	2.8844	1.1413	2.2232	6.6345	11.5946	18.1599	33.0478
5	14.5508	10.3792	5.4336	3.0186	1.1944	2.0257	5.9707	10.6309	17.2875	30.0479
10	15.7611	11.1388	5.5718	3.4208	1.3756	2.2386	5.0993	10.1714	16.6995	28.3520
20	16.0936	11.6890	6.8953	3.6715	1.4926	1.7962	4.4475	8.6150	13.7984	27.1374
30	17.1207	13.5034	6.7877	3.8010	2.1425	1.7529	2.7874	7.5206	11.5978	24.5783
40	17.9597	13.5389	7.2431	4.1800	2.2780	1.3344	2.0631	6.1638	9.5414	19.9321

Table C.2 Shear modulus and Damping ratio values of clay soil (Secha Chamo High School pit)

Secha Chamo High School pit,200KPa										
Strain Level	0.01%	0.1%	1%	2.5%	5%	0.01%	0.1%	1%	2.5%	5%
No. Cycle	Shear Modulus G(MPa)					Damping Ratio β(%)				
1	14.0731	11.0444	6.9337	3.7278	1.1980	1.5708	3.8201	7.4510	13.6759	21.4663
5	15.5045	11.5166	7.4456	4.3652	1.2040	1.4841	3.6609	7.3099	13.3784	18.8300
10	16.2369	11.9875	7.9849	4.6859	1.3273	1.4446	3.4746	7.0942	12.0833	16.5236
20	16.8911	12.5330	8.4291	5.1862	1.4320	1.2039	3.0586	6.0167	10.5498	15.0264
30	17.4135	13.5932	9.1237	5.5440	1.6111	1.1300	2.8580	5.2763	9.3475	13.1892
40	18.5119	14.0629	9.5550	5.8535	1.8137	1.0008	2.3354	4.1548	8.0100	11.1266

Secha Chamo High School pit, 100KPa										
Strain Level	0.01%	0.1%	1%	2.5%	5%	0.01%	0.1%	1%	2.5%	5%
No. Cycle	Shear Modulus G(MPa)					Damping Ratio β(%)				
1	11.3703	9.7882	5.3632	3.0039	1.6932	3.1279	4.8725	8.0098	15.6923	31.0421
5	12.1497	10.1142	5.7148	3.4373	1.7699	3.0458	4.2436	7.6981	14.5443	28.2645
10	13.2613	10.7376	6.0879	3.7686	1.8526	2.7900	4.0034	7.2212	13.6605	26.3497
20	13.8227	11.4217	6.6905	3.8679	1.9121	2.2614	3.6355	6.5423	12.5238	22.3038
30	14.2469	11.8407	7.1601	4.2251	2.0033	2.1436	3.2307	6.0035	12.3218	20.1398
40	14.8904	12.1275	7.5453	4.4884	2.1544	1.9033	3.0226	5.5353	11.9526	18.7488



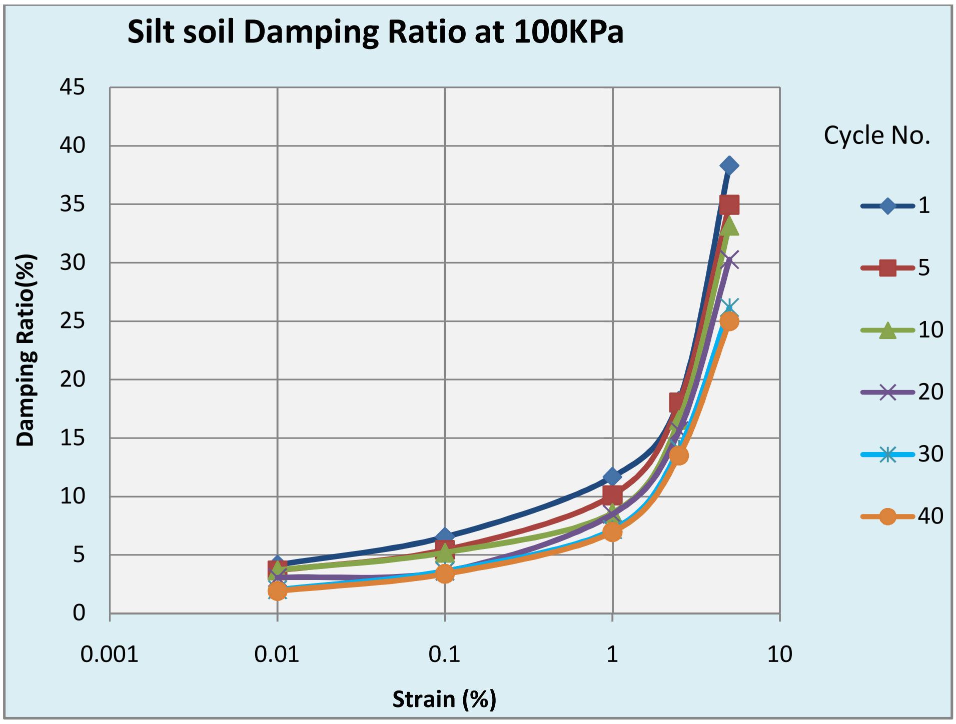
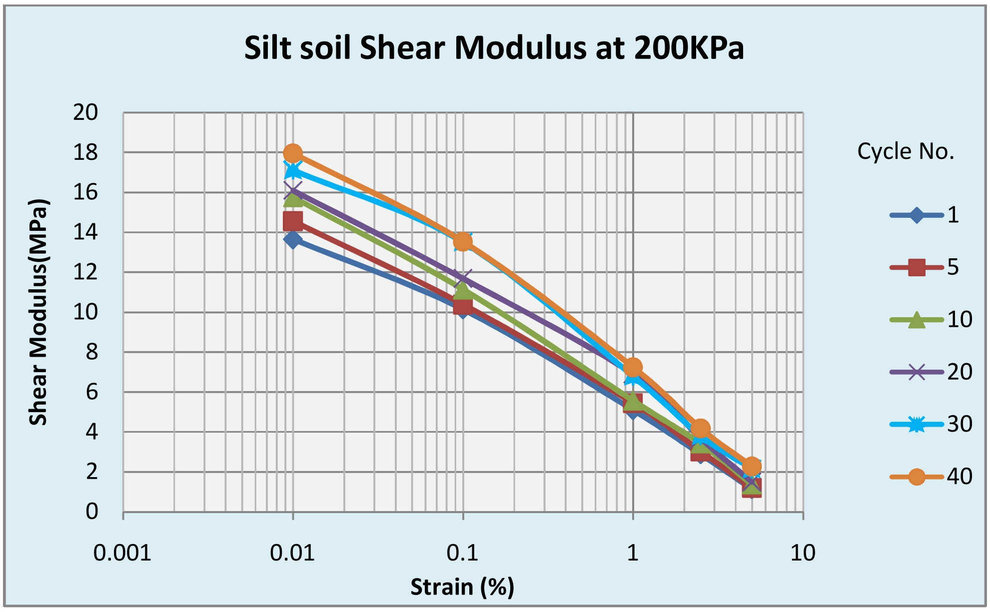


Figure C 1: Shear Modulus and Damping Ratio curves of Silt soil for selected cycles at 100Kpa



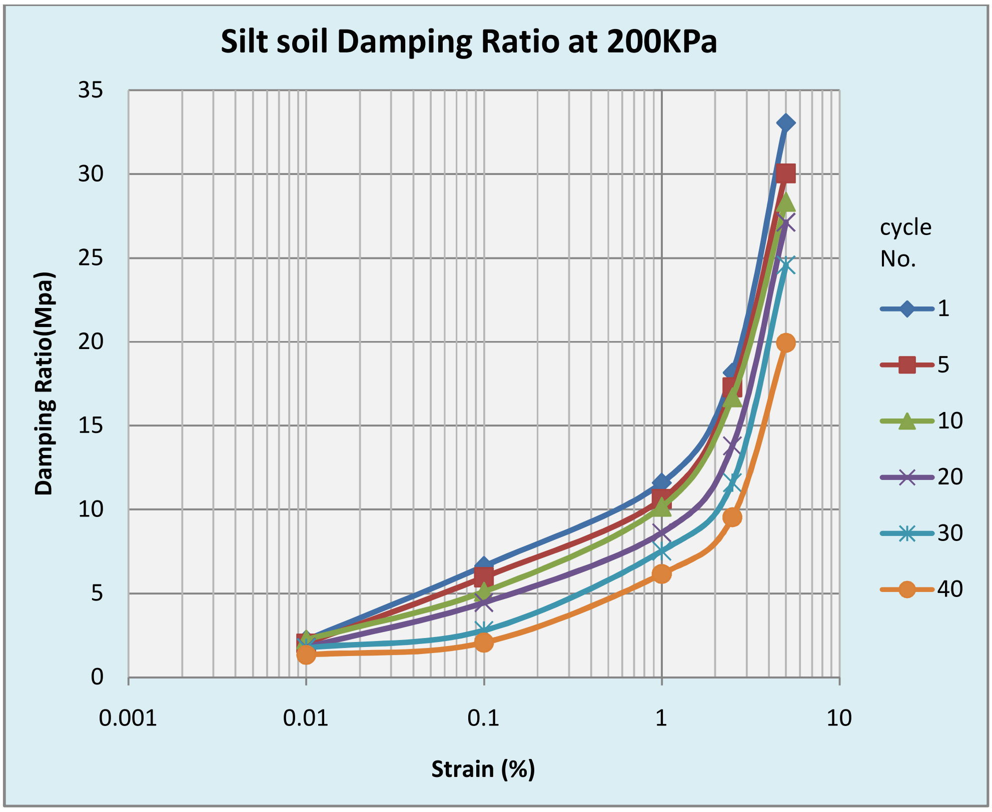
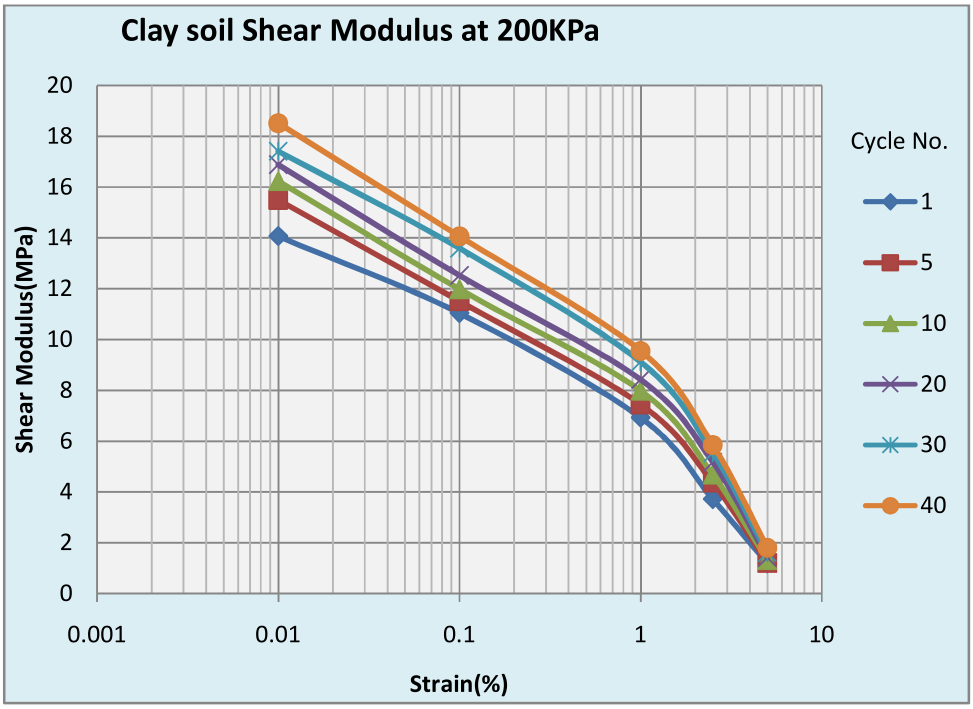


Figure C 2 Shear Modulus and Damping Ratio curves of Silt soil for selected cycles at 200Kpa



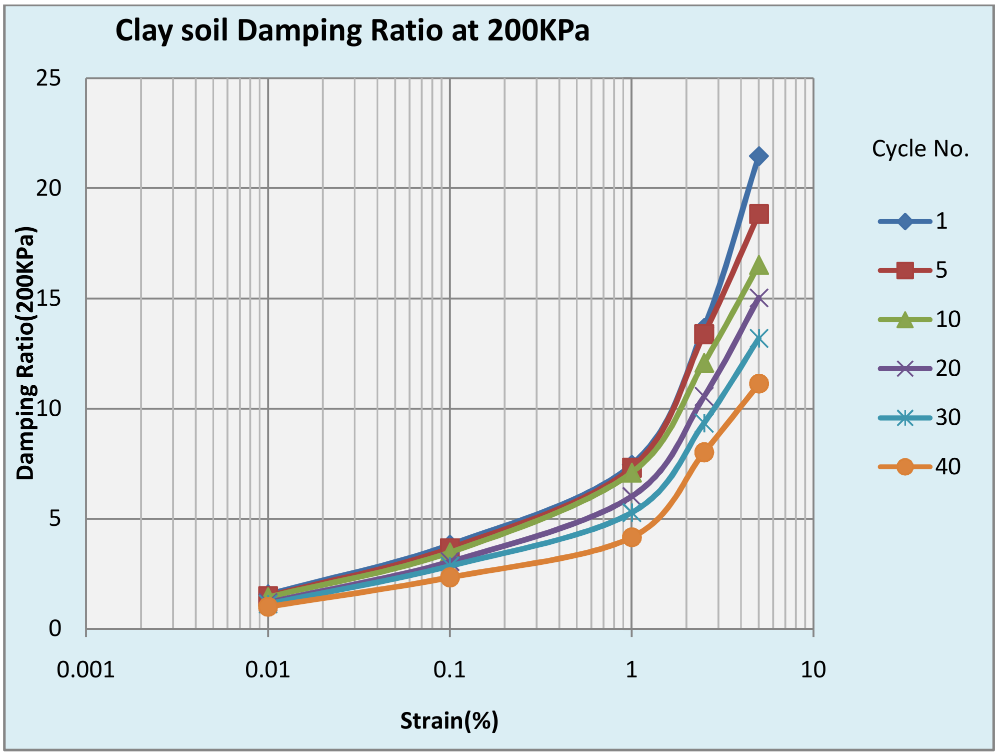
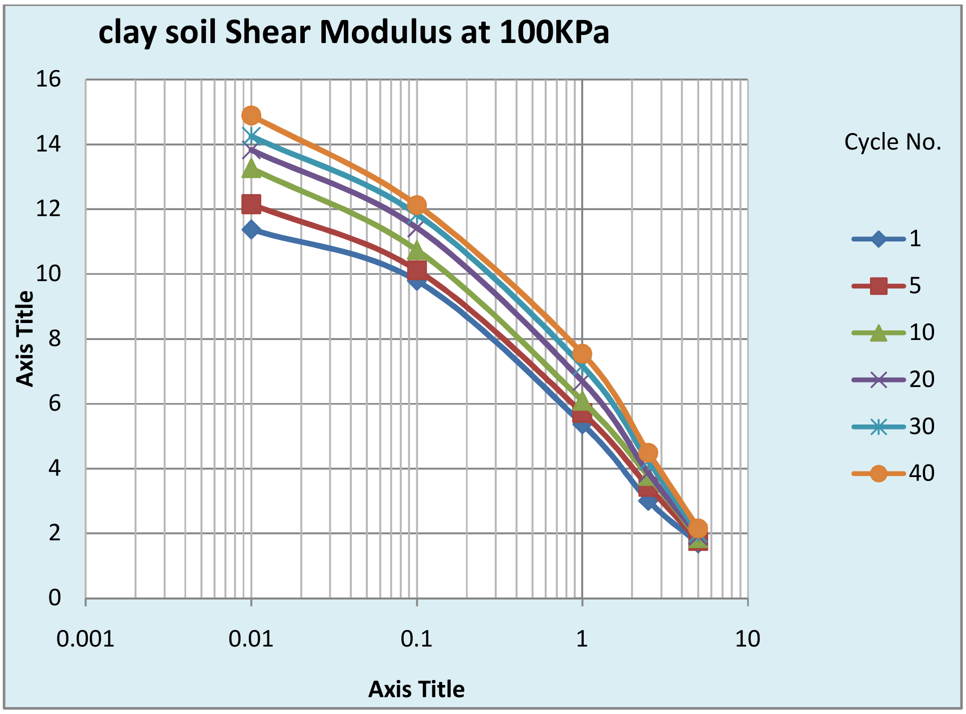


Figure C 3: Shear Modulus and Damping Ratio curves of clay soil for selected cycles at 200Kpa



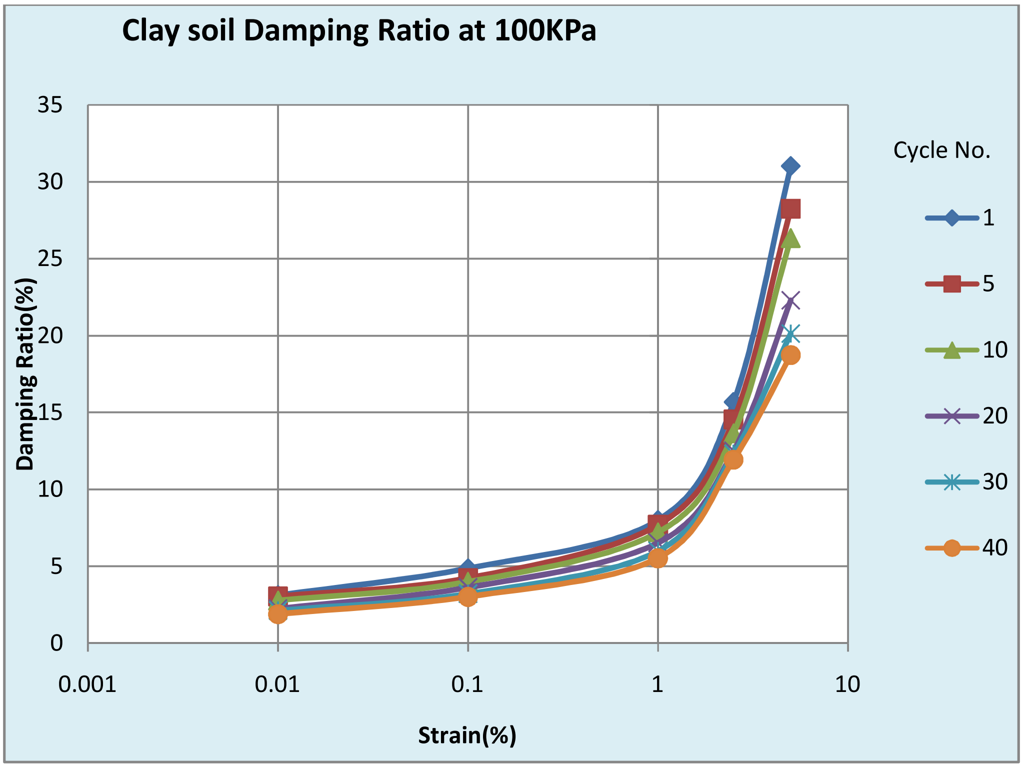


Figure C 4: Shear Modulus and Damping Ratio curves of clay soil for selected cycles at 100Kpa

Appendix – D 3 Shear modulus and Damping ratio curves under different axial loads and number of cycles for both silt and clay

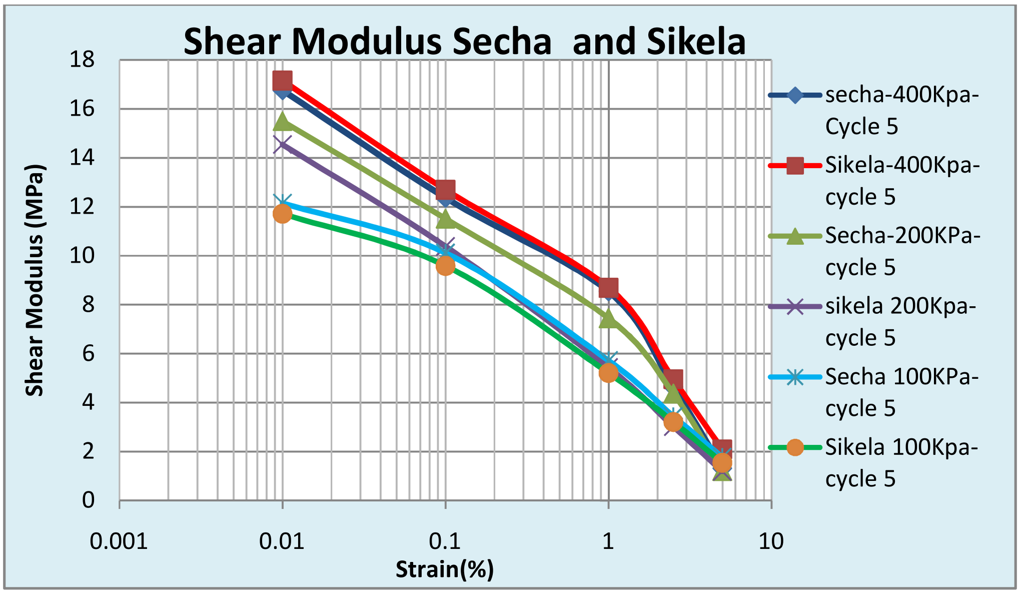


Figure D 1: Shear modulus curves for different axial loads of silt and clay soil

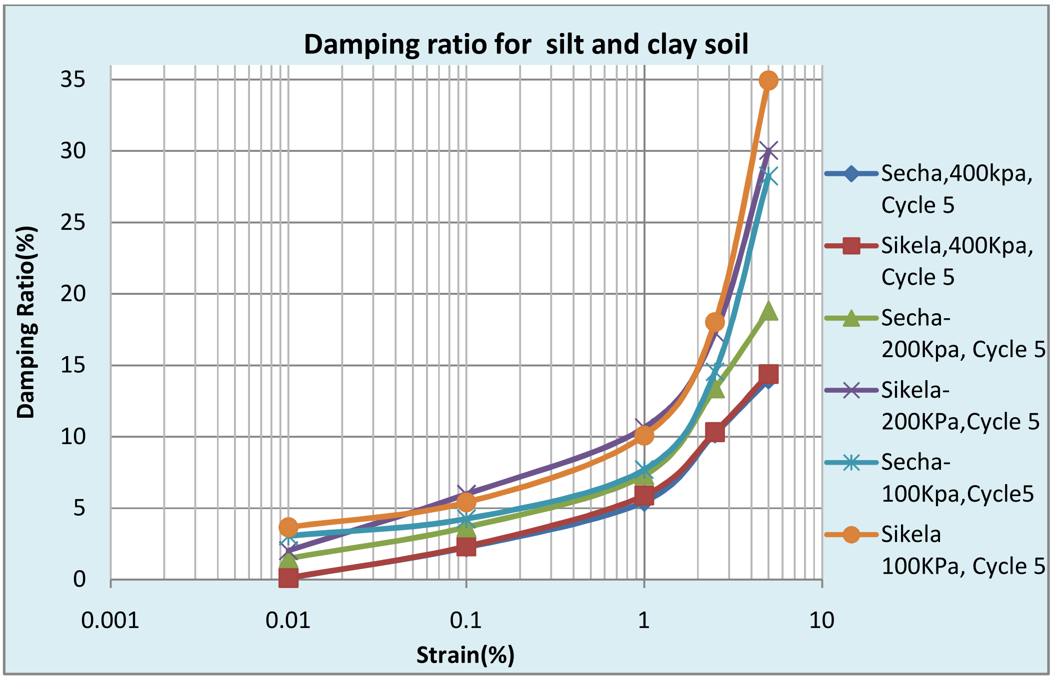
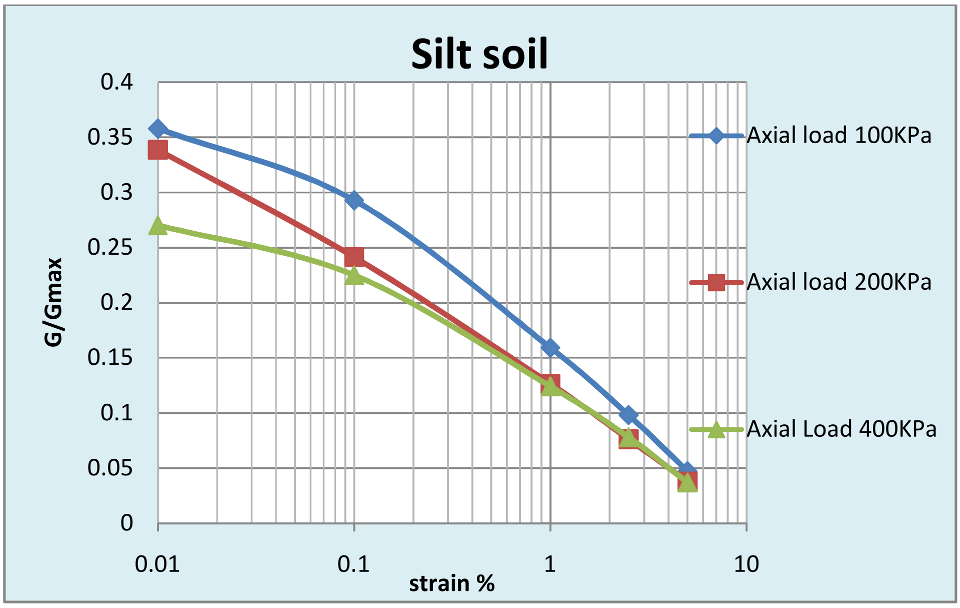


Figure D 2 : Damping ratio curves for different axial loads of silt and clay soil



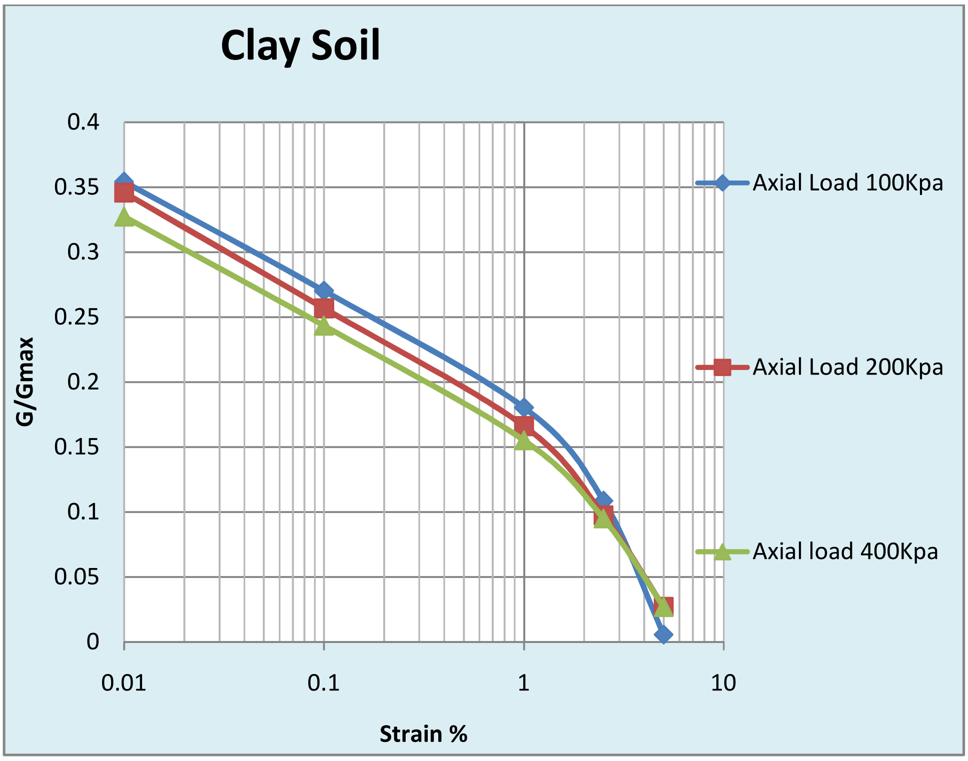


Figure D 3 Shear modulus reduction curve

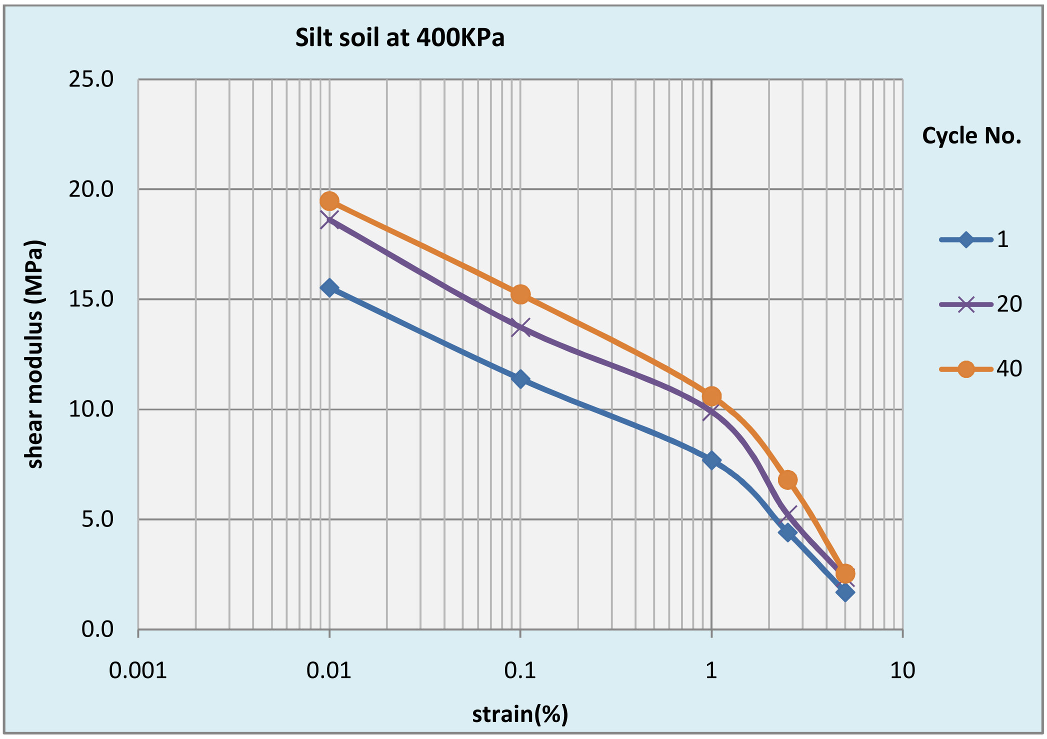


Figure D 4 Effect of number of cycles on shear modulus of the silt soil

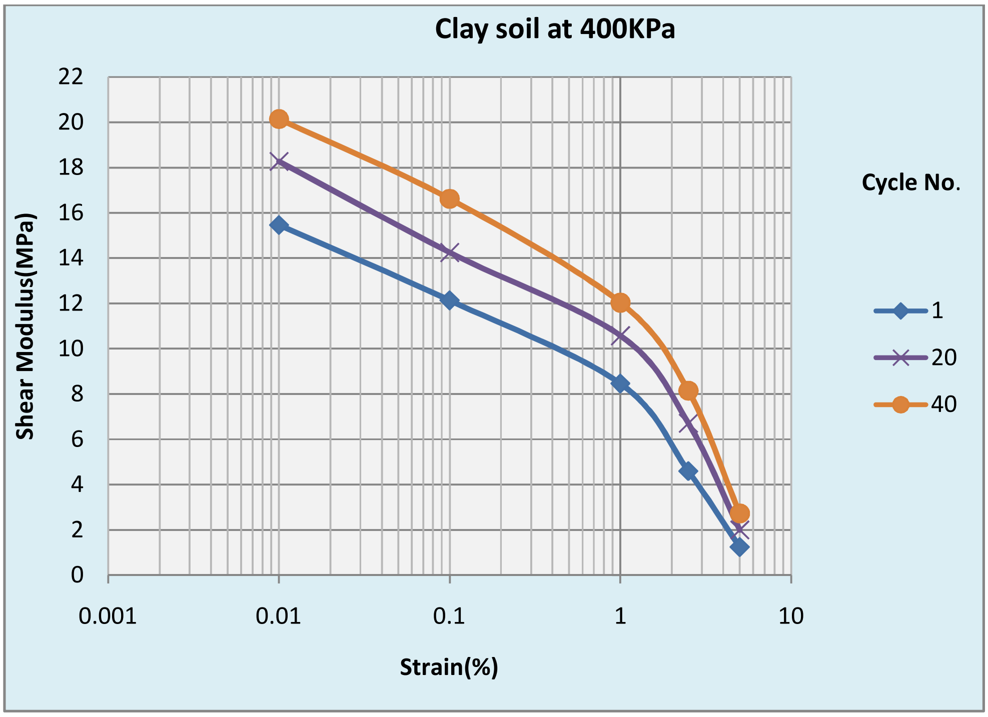


Figure D 5 Effect of number of cycles on shear modulus of the clay soil

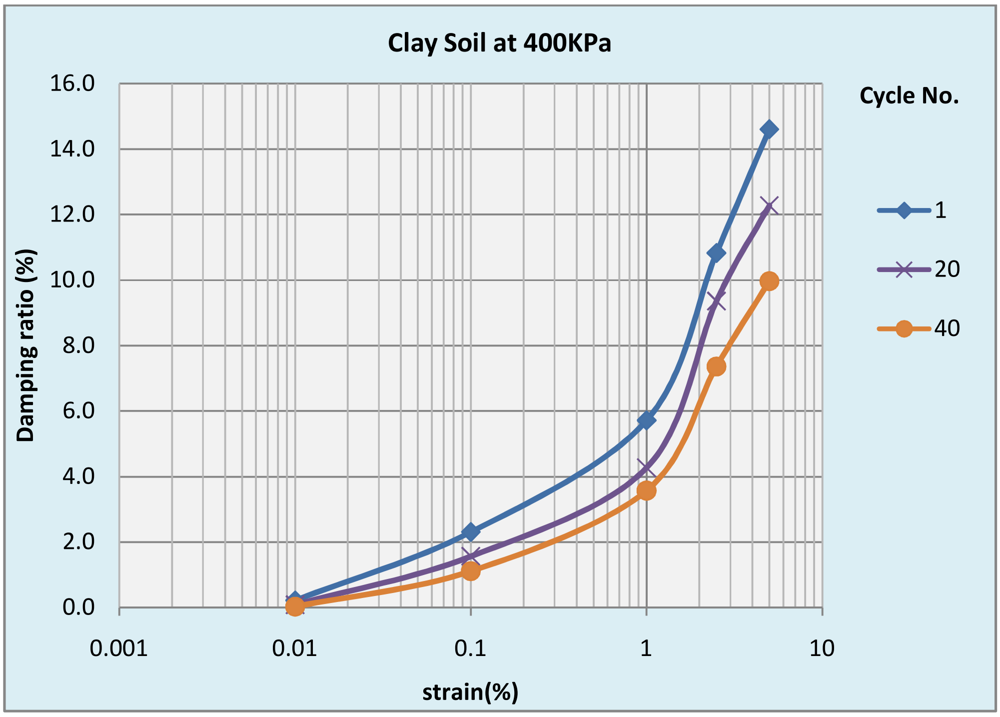


Figure D 6 Effect of number cycles on damping ratio of clay soil

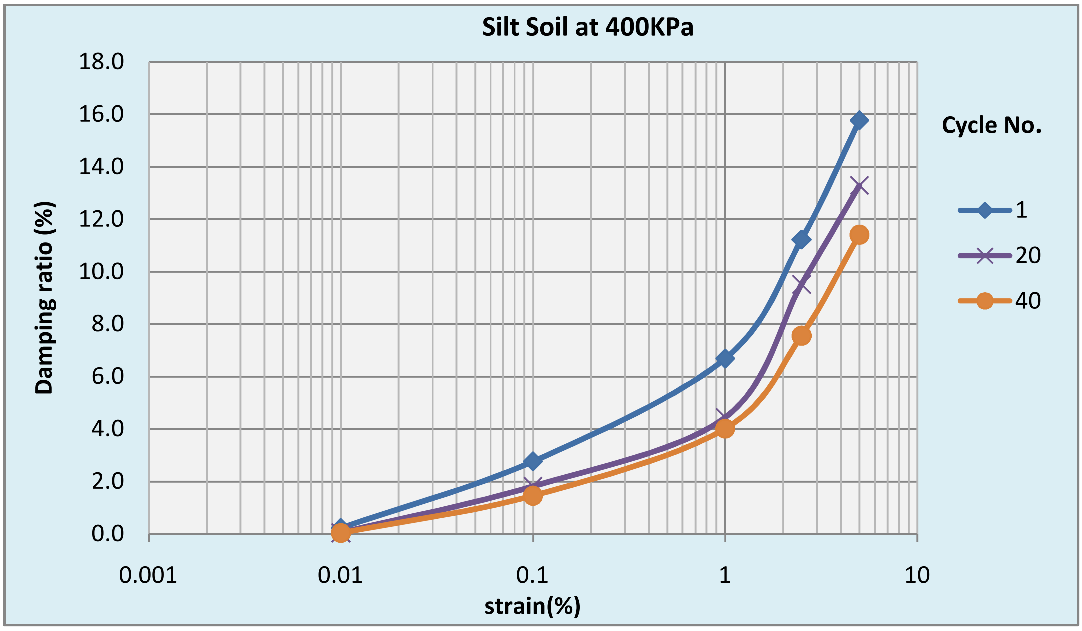


Figure D 7 Effect of number of cycles on damping ratio of silt soil