Dynamic Properties of Indian Standard Sand and Coal Ashes from Cyclic Simple Shear Tests

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Approval Sheet

This thesis entitled 'Dynamic Properties of Indian Standard Sand and Coal Ashes from Cyclic Simple Shear Tests' – by 'G. Thejesh Kumar' is approved for the degree of Master of Technology from IIT Hyderabad.

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Dedicated to

My family members

Abstract

Rapid increase in generation of ash products from coal-based thermal power plants have rendered these materials as an alternative to natural mineral resources in the field of geotechnical engineering. Fly ash and bottom ash are the two major ash materials produced from coal-based thermal power plants. When fly ash and bottom ash are mixed together and transported in the form of slurry and stored in lagoons, the deposit is termed as pond ash. Utilization of ash materials for various applications (reclamation, mine filling, retaining wall back fill material, and in pavements as a sub-base material, *etc.*) can alleviate disposal problems. When ash materials are used as a construction material and subjected to dynamic loading due to earthquakes, machine foundations, explosions, traffic movements, *etc.*, it is important to obtain the dynamic properties – shear modulus and damping ratio – at wide range of shear strains. The present study focuses on studying the behavior of bottom ash, fly ash, and pond ash subjected to cyclic loading using cyclic simple shear testing. In addition, the dynamic properties of ash materials are compared with Indian Standard (I.S.) sand. Dry sand and bottom ash specimens of 70 mm in diameter and 25 mm in height are prepared at relative densities equal to 30%, 50% and 75% and tested at a shear strain of 0.01% to 1% under vertical stresses equal to 100 kPa, 200 kPa, and 400 kPa. The ash materials are prepared at maximum dry density and optimum moisture content and tested at shear strain magnitudes ranging from 0.01% to 1% under vertical stresses equal to 40 kPa, 120 kPa, 200 kPa, and 350 kPa. Frequency range of 0.1 Hz to 2 Hz is maintained during testing. The influence of number of loading cycles, frequency, shear strain magnitude, relative density or void ratio, vertical stress, and sample dimensions on the secant shear modulus and damping ratio of ash materials and I.S. sand are studied. All specimens are subjected to 50 cycles of sinusoidal, and the variation of shear modulus and damping ratio with the number of loading cycles is also brought out. Among the various factors that influence the dynamic properties, results indicate that the shear strain is the most influencing factor, followed by vertical stress and relative density. The secant shear modulus decreases while the damping ratio increases with the

increase in number of loading cycles. However, the effect of number of loading on the damping ratio is found to be negligible. Frequency of loading and particle sizes are found to have an insignificant effect on the dynamic properties of the materials tested. Finally, the correlation proposed between the shear modulus and damping ratio of all tested materials can be used to determine the damping ratio from the shear modulus measured in the field.

Nomenclature

c - Cohesion

ϕ - Friction angle/Angle of shearing resistance

C_c - Coefficient of curvature

C_u - Coefficient of uniformity

D_{10} - Effective particle size

D_{50} – Average/Mean particle size

SP – Poorly-graded sand

R_d – Relative density

G_{max} – Maximum shear modulus

G_{sec} – Secant shear modulus

D – Damping ratio

Contents

Chapter 1

Introduction

1.1 Introduction and Motivation of the Study

1.1.1 Indian Standard Sand

The importance of dynamic properties of soil (shear modulus and damping ratio) has been widely recognized in the design of many geotechnical engineering problems from past five to six decades. Soil deposits, which are supporting the engineering structures will be subjected to dynamic loads either by earthquakes, explosions, machine foundations, pile driving, vehicular movements over ground, ocean wave storms, or other causes (Ramadan (2007)). The behavior of soil deposits and supporting structures to such loads depends to a large extent on the cyclic stress-strain characteristic of the soil in shear. Subsequently, in order to have a successful and safe design of foundations for the engineering structures, it is essential to know stress-strain properties of the foundation soil during cyclic loading. In addition to that, the shear modulus and the damping ratio of soils estimated at large strains would serve as a significant factor in the analysis of geotechnical engineering structures subjected to strong earthquake motions. Numerical soil models use the variation of shear modulus and damping ratio with strain level as basic input parameters for dynamic analyses. The recent advancement in the development of numerical soil models for non-linear dynamic responses of grounds due to strong earthquake motions have increased the demand for the dynamic soil properties corresponding to both small strain levels and large strain levels (Ravi Shankar et al. (2005)). Therefore, it would be necessary to evaluate the dynamic properties of soil deposits for a wide range of shear strains.

1.1.2 Ash Materials

The need for thermal power plants and other sources of power generation is increased due to rapid increase in industrialization, which demands huge amount of electricity. Possessing the advantages of occupying smaller space, economical in initial cost and minimal losses during power transmission, most of the countries are depending on thermal power plants, especially coal as a fuel. This results in generation of million tons of ash materials which is creating a lot of problems for its handling and disposal system all over the world. The fine texture ash materials entrained in the flue gases and captured in electro-static precipitator (ESP) is termed as fly ash. The ash which falls at the bottom of the boiler furnace is known as bottom ash and its particles are coarser in size compared to fly-ash particles. During the power generation from coal based thermal power plants, 80 to 90% of fly ash and 10 to 20% of bottom ash will be produced. When the fly ash and bottom ash are mixed together, transported in the form of slurry and stored in the lagoons, the deposit is termed as pond ash. To solve the problem of ash disposal to a great extent, the ash materials are widely used for the construction of fills and for filling low-lying areas as an alternative material to natural soils. Toth et al. (1978) determined that the structural fill made with fly ash could perform better than the fill constructed with natural materials and the physical behavior of fly ash is similar to that of silt, by studying the use of fly ash as a structural fill.

Coal based thermal power plants are one of the major sources for generating electric power in India. If thermal power plants use pulverized coal as the fuel then large quantities of ash will be generated as a by-product. According to the census conducted by Central Electricity Authority (CEA), New Delhi, India, nearly 163 million tons of fly ash is being produced annually across India (Central Electricity Authority 2014). This amounts to nearly 30 percent of fly ash that is produced across the world. Indian coal is categorized as low-grade coal with high ash content of nearly 40% in comparison to that of imported coal that contains ash content of only 10-15%, resulting in production of large quantity of ash during burning of Indian coal (Raja et al. 2006).

The CEA report on ‘fly ash generation at coal-fired thermal power stations’ stated that the utilization of fly ash was 100 million tonne over the production of 163 million tonne leading to its utilization of about 56% in the year 2012-13. The report also specified that 41.1% of fly ash is being utilized as binding material in the cement sector, 11.7% in reclamation of low lying area, 6% in roads and embankments, 10.3% in mine filling, and 9.94% in building materials like bricks, tiles etc. As per the report, the utilization of fly ash in cement sector has increased from 2.4 million tonne in 1998-99 to 41.3 million tonne in 2012-13. Similarly, utilization of fly ash in reclamation of low lying areas has increased from 4.1 million tonne in 1998-99 to 11.8 million tonne in 2012-13, and its utilization increased from 1.1 million tonne in 1998-99 to 6.0 million tonne in 2012-13 in the construction of roads and embankments. Overall, the fly ash utilization has increased from 9.6% during 1996-97 to 61.3% during 2012-13 (Central Electricity Authority 2014). The current percentage of utilization of fly ash in India is very low when compared to the other countries like Germany, Netherlands etc. where the utilization is above 90%.

In terms of volume, production of pond ash in thermal power plants will be large compared to fly ash and bottom ash (Subbarao and Ghosh 1997). Implementation of ash materials as a construction material in roadways, embankments, mine fillings and related peripheral projects has been a significant outlet for ash materials. Therefore, the wide applications of fly ash as a geotechnical material in highway embankment construction, retaining wall back fill material, reclamation of low lying areas, mine filling, etc., stressed the need for better understanding of the engineering behavior and dynamic behavior of the fly ash, especially if they are used in earthquake prone areas.

1.2. Objectives of the Study

The following are the objectives of this study:

1) To determine the shear modulus and damping ratio of I.S. sand and Ramagundam Bottom ash, for the specimens prepared at relative densities in between 30% to 75% and tested at various vertical stresses from 100 kPa to 400 kPa. The influence of number of loading cycles, frequency of loading, grain size, and sample height on dynamic properties also investigated.

2) To determine the shear modulus and damping ratio of Neyveli fly ash, Ramagundam bottom ash and Ramagundam pond ash for the specimens prepared at their compaction characteristics and tested at various vertical stresses from ranging from 40 kPa to 350 kPa. The influence of number of loading cycles, frequency of loading, and sample height on dynamic properties also investigated.

1.3. Report Outline

This report consists of six chapters, including this first Chapter covers the Introduction. The second chapter presents a brief literature review on dynamic soil characteristics of soils, shear modulus and damping values of sand, various studies on characterizing the ash materials and its static geotechnical properties and brief background about dynamic cyclic simple shear testing. The third chapter covers the properties of the tested materials. The fourth chapter briefly explains about the testing procedure with the dynamic cyclic simple shear device. The fifth chapter presents the laboratory test results and data validation with available literature. The sixth chapter gives the conclusions drawn from the obtained results.

Chapter 2

Literature Review

2.1 I.S. Sand

Geotechnical engineering problems associated with soil dynamics and cyclic loading are many and occupy the whole range of amplitude excitations from very small amplitudes of motion as in the case of some vibratory machine foundations up to the large amplitudes accompanying strong motion earthquakes and nuclear explosions. A solution to such problems needs a better understanding of the knowledge of dynamic soil properties and characteristics. Many studies have been conducted to characterize the factors that affect shear modulus and damping ratio of soils (e.g., Richart et al. (1970), Seed and Idriss (1970), Hardin and Drnevich (1972b), Iwasaki et al. (1978), Lee and Finn (1978), Zen et al. (1978), Kokusho et al. (1982), Seed et al. (1986), Sun et al. (1988), Vucetic and Dobry (1991), Ishibashi and Zhang (1993), Rollins et al. (1998), Stokoe et al. (1999), Darendeli (2001), Roblee and Chiou (2004), Stokoe et al. (2004)). The shear modulus degradation and damping curves most often used for dry cohesionless soils, such as sands, gravels and cohesionless silts, are those proposed by Seed and Idriss (1970) and Seed et al. (1986). Figure 2.1 shows the shear modulus degradation curve for sand proposed by Seed and Idriss (1970). Seed et al. (1986) plotted all the values of damping ratio previously determined by many investigators and an upper limit and lower limit was proposed as shown in figure 2.2. The extensive study of Hardin and Drnevich (1972a and 1972b) concluded that the primary factors affecting the shear moduli and damping factors are: strain amplitude, effective mean principle stress, void ratio or relative density, number of cycles of loading, degree of saturation (for cohesive soils). The less important factors are: octahedral shear stress, over-consolidation ratio, effective strength stress parameters and time effects.

Table 2.1 shows the effect of environmental and loading conditions on shear modulus and damping ratio of normally consolidated and moderately consolidated soils given by Hardin and Drnevich, (1972a and 1972b) and modified by Dobry and Vucetic (1987).

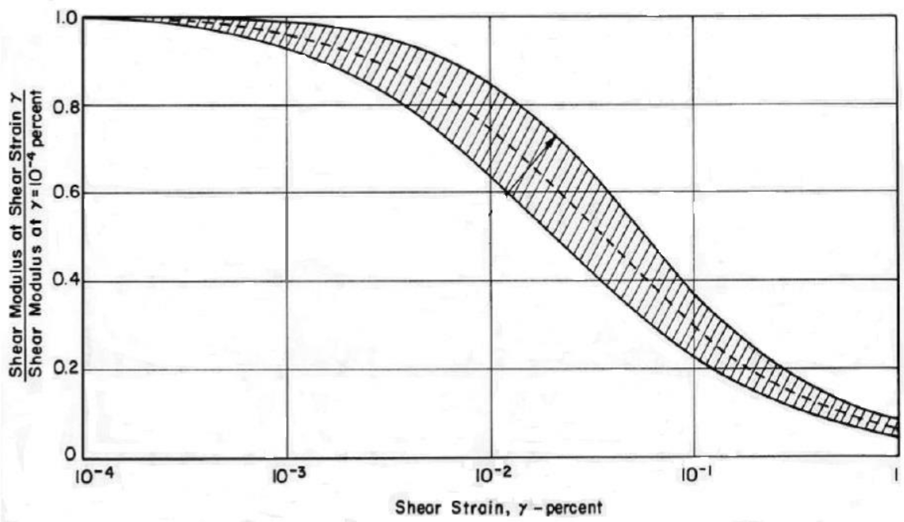


Figure 2.1: Variation of shear modulus with shear strain for sand [Seed and Idriss (1970)]

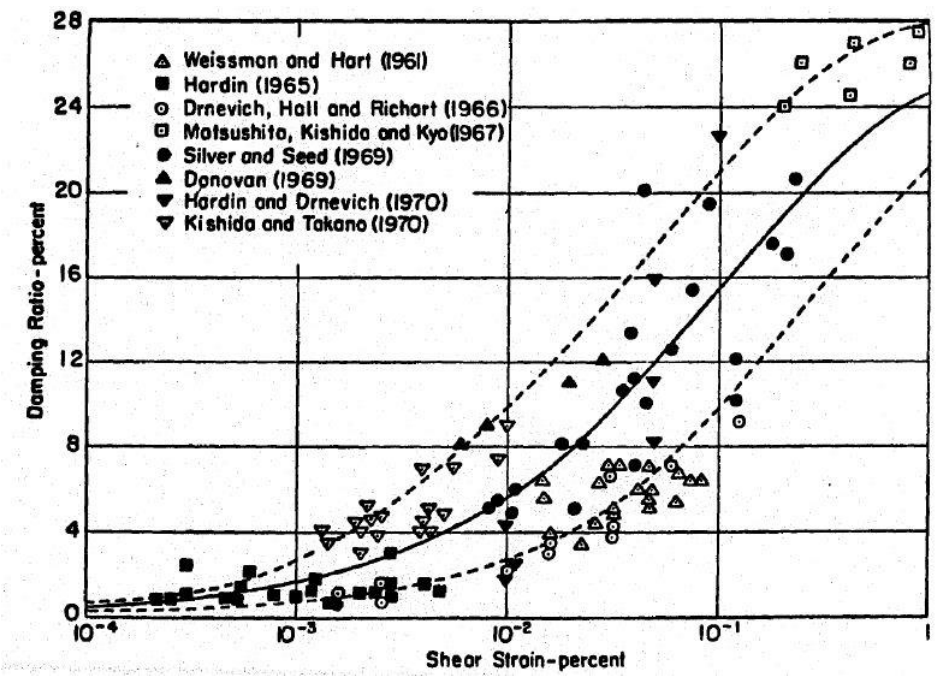


Figure 2.2: Variation of damping ratio with shear strain for sand [Seed and Idriss (1970)]

Many researchers have used cyclic triaxial or resonant column tests to determine dynamic properties (shear modulus and damping ratio) as functions of shear strain and effective stress for various materials. Seed et al. (1986), Rollins et al. (1998) studied gravels, Wilson

(1988), Kokusho (1980) studied sands, Hardcastle and Sharma (1998) studied loess, and Idriss et al. (1978), Kokusho et al. (1982), Vucetic and Dobry (1991) studied clays. Ellis et al. (1998) derived modulus and damping of very dense sand saturated with different pore fluids based on centrifuge testing. Elgamal et al. (2005) used centrifuge data to estimate stiffness, damping, and dilatancy characteristics of saturated dense Nevada sand.

Table 2.1: Effect of environmental and loading conditions on dynamic properties

Increasing Factor	**Shear Modulus**	**Damping Ratio**
Cyclic Strain, γ_c	Decreases with γ_c	Increases with γ_c
Confining Pressure, σ_m'	Increases with σ_m' (effect decreases with increasing PI)	Decreases with σ_m' (effect decreases with increasing PI)
Void Ratio, e	Increases with 'e'	Decreases with 'e'
Geologic Age, t_g	May increase with t_g	May decrease with t_g
Cementation, c	May increase with 'c'	May decrease with 'c'
Over Consolidation Ratio, OCR	Not affected	Not affected
Plasticity Index, PI	Increases with PI	Decreases with PI
Number of Loading Cycles	Decreases after N cycles of large γ_c for clays, for sands can increase (drained conditions) or decrease (undrained conditions)	Not significant for moderate γ_c and number of cycles, N

2.2. Ash Materials

Implementation of ash materials as a construction material in roadways, embankments, mine fillings and related peripheral projects has been a significant outlet for ash materials. Leonards and Bailey (1982) and Skarzynska et al. (1989) stated that ash collected from lagoons for construction of highway embankment or landfilling shows a large variation of engineering properties. The studies on usage of fly ash and bottom ash mixtures in highway embankments were done by researchers like Martin et al. (1990), Kim et al. (2005), Santos et al. (2011) and in sub-bases by Kumar and Singh (2008). Kim et al. (2005) performed studies on fly ash and bottom ash to determine the geotechnical properties and their suitability as a construction material in highway embankments. Three mixtures of fly and bottom ash with different mixture ratios, 50%, 75%, and 100% fly ash content by weight, were prepared for testing. Test results indicated that ash mixture characteristics are similar to that of conventional granular materials. The shear strength results indicated the peak internal friction angle in the range of 28° to 48°, fairly comparable with the internal friction angle of granular soils.

Sridharan et al. (1997), Das and Yudbhir (2005), Punthutaecha et al. (2006), Madhyannapu et al. (2008), and Jakka et al. (2010a) studied the static behavior of fly ash as a geotechnical material and determined its index and engineering properties. Few researchers (Gray and Lin (1972), Dey and Gandhi (2008), Mohanty et al. (2010), and Jakka et al. (2010b)) studied about the liquefaction potential assessment of different types of ash materials in different applications.

The research studies on the geotechnical engineering properties of bottom ash and the use of bottom ash, with or without admixtures, in geotechnical engineering applications is very limited compared to fly ash. Considering the grain size, bottom ash can generally be classified as coarse grained and its particles range in size from fine gravel to fine sand, with low percentages of silt-clay-sized particles. Even though variations in particle size distribution are possible in bottom ash samples taken from the same power plant at different times, it is usually considered as a well-graded material (Kumar and Stewart 2003). Seals et al. (1972) stated that dry bottom ash has a gray to black color and most particles are angular in shape with porous surface texture. Seals et al. (1972) performed standard proctor tests and results indicated that the maximum dry unit weight of bottom ash samples are similar to the values of maximum dry unit weight obtained for sands. Seals et al. (1972) also performed one dimensional compression tests on initially dry samples, one comparatively loose and the other relatively dense and determined that, at low stress levels and at the same relative density, the compressibility of bottom ash appeared comparable to that of sand.

Huang (1990) studied the use of bottom ash as a geotechnical material in the construction of highway embankments, subgrade, and sub-bases. Direct shear tests were performed on Indiana bottom ash and boiler slag at different densities to investigate the shear strength behavior. The experimental results concluded that friction angle varies in a wide range of 35° to 55°, depending on the density. Huang and Lovell (1990, 1993) also performed standard proctor tests and concluded that compaction curves for bottom ash samples were similar to those obtained for cohesionless materials (sands).

Pond ash, being artificial material, possess low unit weight and high porosity and the behavior of ash is expected to be different from natural soils (Datta et al. (1996), McLaren and DiGioia (1987), etc.). Several researchers (Sridharan et al. (1997), Das and Yudbhir

(2005), Punthutaecha et al. (2006), Madhyannapu et al. (2008), and Jakka et al. (2010a)) studied the behavior and determined various engineering properties of coal ashes. Pandian (2004) studies the characteristics of pond ash and concluded that pond ash is highly susceptible to liquefaction as its particles are fine grained, uniformly graded, and rounded.

Sridharan et al. (1997) investigated the geotechnical characterization of various ash ponds like fly-ash, bottom ash, and pond ash generated in India and stated that pond ashes, in general, possess low unit weight, good frictional properties, low compressibility and low permeability and they are well suited for their use as a structural fill. Mohanty and Patra (2014) studied the cyclic behavior and liquefaction potential of pond ashes collected from Talcher, Panki, and Panipat ash embankments located in India and concluded that the pond ashes from these embankments can be effectively used as a geomaterial in geotechnical applications.

Apart from laboratory experiments, field implementation of adopting ash materials as a geotechnical construction material has been recorded in many cases. Nearly 2 million tonnes of pond ash from National Thermal Power Corporation (NTPC), Badarpur thermal power station has been utilized in road embankment of Noida-Greater Noida express highway in India. Large quantity of ash has been utilized in railway embankment work in Tamluk-Digha section of Kharagpur division near Kolkata, India. More than 1.5 million tonnes of pond ash has been utilized by Delhi metro rail corporation, India in rail embankment. About 0.15 million tonnes of pond ash had been utilized in second Nizamuddin bridge embankment construction by Delhi public works department (PWD) at New Delhi, India. 0.275 million tonnes and 0.25 million tonnes of coal ash has been utilized in area development of stock yards and construction of railway embankment of steel authority of India limited, at Dankuni and Haldia in West Bengal, India respectively. For retrieving underground mines, bulk quantities of coal fly ash have been used to replace the conventionally used sand. During 1999–2000, the national thermal power corporation limited (NTPC), India used about 60,000 ton of ash for backfilling underground mines of Singareni Colliery Company Limited, Southern India, in collaboration with central mining research institute, India [Mathur (2000)].

2.3. Background of Dynamic Cyclic Simple Shear

During earthquakes, the behavior of soil deposits is a three dimensional complex phenomenon with three-directional excitation, non-homogeneous, anisotropic and non-linear. Simulating the erratic sequence of ground motions over a soil specimen in the laboratory is a difficult task. For simplifying the pattern, usually a simple model of a homogeneous horizontally layered soil profile is excited at its base by one-directional horizontal shaking, which involves only vertically propagating shear waves (Vucetic 1992). There are various types of laboratory devices, each type of testing performed to simulate certain conditions in the field within a specific strain range. Woods (1978) explained the advantages and disadvantages of various equipment used for determining the dynamic properties of soils and concluded that there is no ‘perfect’ laboratory test. However, for many researchers and investigators [Kjellman (1951), Hvorslev and Kaufman (1952), Roscoe (1953), Silver and Seed (1960), Thiers and Seed (1968), Seed and Peacock (1971), Silver and Seed (1971), Kovacs (1973), Sherif and Ishibashi (1976), Kovacs and Leo (1981), Vucetic (1992), Kramer (1996), Ramadan (2007), and etc.] simple shear testing appears to be the most appropriate equipment to reproduce the stresses experienced by an element of soil during earthquakes in the laboratory. Simple shear testing has been widely accepted because of the greater awareness of the importance of stress-strain anisotropy in geotechnical problems and its simplicity relative to other cyclic static devices like cyclic triaxial, cyclic torsional shear and resonant column.

2.3.1. History of Dynamic Cyclic Simple Shear

A number of different direct cyclic simple shear apparatus have been developed over the past few decades. Table 2.2 shows the various types of simple shear apparatus existing from past five to six decades. Several investigators (Cole (1967), Peacock and Seed (1968), Stroud (1971), Finn, et al. (1971), Seed and Peacock (1971), Budhu (1979), Airey (1984)) used Roscoe type device in their studies on liquefaction potential. The NGI type device has been used by many investigators (Bjerrum and Landva (1966), Carroll and Zimmie (1979), Shen, et al. (1978)). Vucetic and Lacasse (1982) tested larger sized specimens statically in the NGI device.

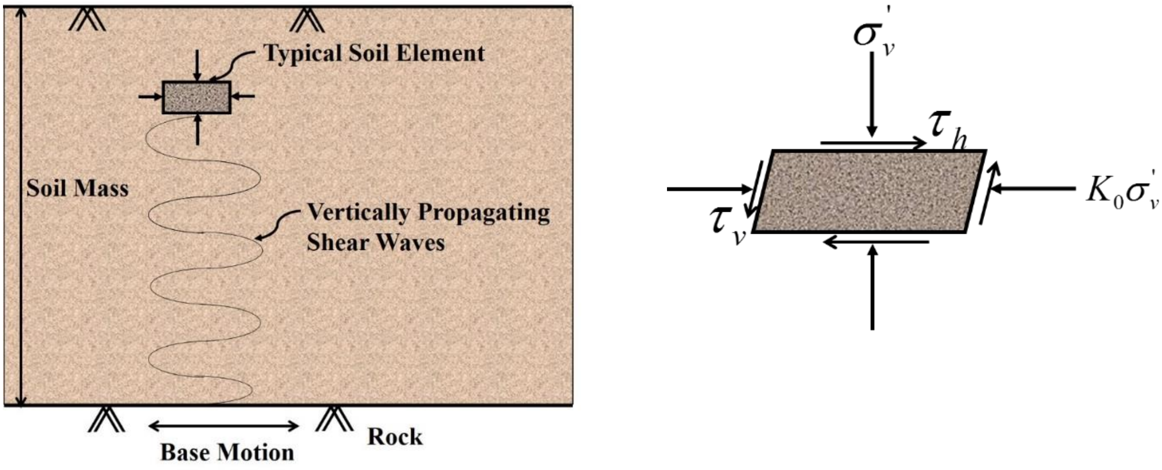
Apart from the devices listed in table 2.2, Franke et al. (1979), presented a device that uses a circular specimen that is laterally confined in a rubber membrane and placed in a pressure cell in which vertical and horizontal normal stresses can be applied. The device uses a specimen size of 2.95 in. (7.5 cm) in diameter and a height varying between 0.39-0.79 in. (1.0-2.0 cm). In addition, Villet et al (1985) developed at the University of California at Berkeley and Mao and Fahey (2003) developed at the University of Western Australia.

Table 2.2: Development of various types of cyclic simple shear equipments

DCSS Type	Introduced by	Specifications
Royal Swedish Geotechnical Institute (SGI) simple shear apparatus	Kjellman (1951) (Apparatus was built in 1936 but well described by Kjellman in 1951)	Diameter = 2.36 in. (6 cm) Height = 0.79 in. (2 cm) • Uses stacked rings to confine the specimen.
Cambridge-type simple shear apparatus	Roscoe (1953)	Square specimen 2.36 in. (6 cm) on each side and 0.79 in. (2 cm) thick.
Norwegian Geotechnical Institute (NGI) simple shear apparatus	Bjerrum and Landva (1966) (Apparatus was built in 1961 by NGI)	Diameter = 3.15 in. (8 cm) Height = 0.79 in. (2 cm) • Uses a wire-reinforced rubber membrane to confine the specimen.

The vertically propagating shear waves are considered as most important during seismic site response analysis as it causes horizontal shaking of the ground surface, which results in triggering much damage to infrastructure projects. According to Kramer (1996), the stress-strain conditions in the dynamic cyclic simple shear test correspond rather closely to those occurring during the propagation of shear waves through soil deposits. Similar to vertically propagating S-waves on a soil element, specimen will be subjected to stresses on top and bottom and under the condition of plane strain, simple shear tests allow the principal stress and/or strain axes to rotate. In practical engineering situations and during most in-situ tests, the principal axes also rotate (Rasmussen 2012). Ramadan (2007) stated that the dynamic cyclic simple shear apparatus is practically well in simulating the pure shear stress conditions which is applicable to a number of common field situations such as horizontal portions of the slope failure surface and foundation bearing capacity failure surface, behavior of soil surrounding vertically loaded piles, etc. Figure 2.3 illustrate a typical example of the pure shear stress conditions pertaining to a geotechnical earthquake engineering problem that can be simulated in the simple shear device.

The figure 2.3 (a) shows the transmission of shear waves from bedrock into overlying soil and figure 2.3 (b) shows an idealized stress-strain conditions of a soil element during earthquakes, in which σ_v' is the effective vertical stress, τ_h is the horizontal shear stress, τ_v is the vertical shear stress and K_0 is the earth pressure coefficient at rest conditions.



(a) **(b)**

Figure 2.3: a) Transmission of shear waves from bedrock into overlying soil
b) Idealized stress-strain conditions of a soil element during earthquakes

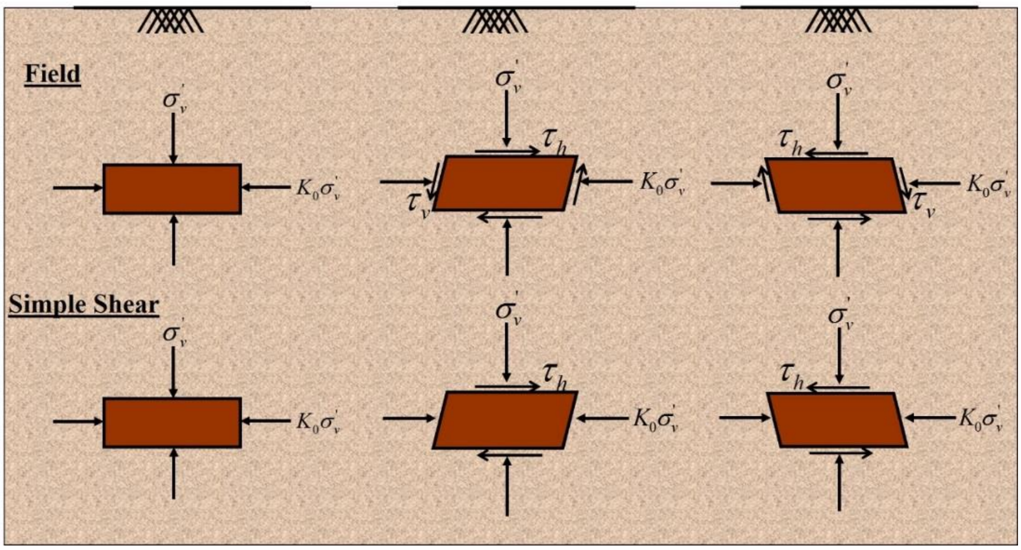


Figure 2.4: Boundary stress conditions in field and simple shear cases

On the other side, the dynamic cyclic simple shear also possess some drawbacks. Machine only applies shear stresses to the top and bottom of the specimen and there are no complimentary shear stresses on the sides of the specimen. Andersen et al. (1980) stated that the compliance in the test apparatus is insignificant for relatively soft clays but for dense sands it must be considered. Figure 2.4 shows the stress conditions developed on a soil element in the field and those imposed on the boundaries of the simple shear sample.

Several researchers studied about the nature of the non-uniform stresses in dynamic cyclic simple shear (DCSS) tests. Roscoe (1953) analyzed mathematically the stresses acting on a sample in the Cambridge simple shear device by performing analysis on an isotropic, linear elastic material and showed that tension develops in the upper leading edge and the lower trailing edge. He concluded that there will be no development of tension zones if a suitably large vertical load is applied to the specimen but still there will be the presence of non-uniform loading. Across the middle third of the specimen faces, the results showed that the shear stress is approximately uniform. Considering a nonlinear, anisotropic material within the Cambridge simple-shear apparatus, Duncan and Dunlop (1969) performed a finite element analysis and they concluded that stress non-uniformities are most severe near the ends of the sample. In addition to that, the stresses in the center of the sample are reasonably uniform and correspond closely to pure shear conditions. Many other researchers like Lucks et al. (1972), Prevost and Hoeg (1976), Shen et al. (1978), Wright et al (1978), Wood et al. (1979) and etc., studied the nature of the non-uniform stresses in dynamic cyclic simple shear tests. According to Kovacs and Leo (1981), Airey et al. (1985) and Kramer (1996), the uniformity in stress distribution improves by increasing the diameter/height ratio of the specimen and such effects are small at diameter/ height ratios greater than about 8:1.

Kovacs (1973) proved that the shear modulus decreases for a given length to height ratio as the sample size increases in plan dimension. He recommended, for faithful representation of the stresses imposed during field conditions, and to reduce or eliminate the boundary effects in simple shear testing, the use of a large sample length of 8 inches with a length to height ratio of at least 6:1. Vucetic (1981) and Vucetic and Lacasse (1981, 1982) investigated the influence of height to diameter ratio and membrane stiffness on the behavior of clay in the Norwegian Geotechnical Institute (NGI) simple shear apparatus.

Chapter 3

Material Properties

3.1. I.S. Sand

Grade-II Indian Standard (I.S.) sand (Ennore sand is the commercial name of I.S. sand) was employed as the testing material in all the tests. In order to study the influence of grain size characteristics on secant shear modulus and damping ratio of I.S. sand, grade-III I.S. sand was used to compare the results with grade-II I.S. sand. The table 3.1 shows the properties of grade-II and grade-III I.S. sand.

Table 3.1: Properties of materials used in the study

Property	**Standard**	**Value**	
		Grade - II	Grade - III
Specific Gravity	ASTM D854 (2014)	2.60	2.64
D_{10} (mm) D_{30} (mm) D_{60} (mm)	ASTM D6913 (2004)	0.28 0.41 0.55	0.12 0.20 0.33
Mean Particle Size (D_{50} in mm)	ASTM D6913 (2004)	0.50	0.28
Coefficient of Uniformity (C_u)	ASTM D6913 (2004)	1.96	2.75
Coefficient of Curvature (C_c)	ASTM D6913 (2004)	1.1	1.01
Minimum Dry Density (g/cc)	ASTM D4254 (2014)	1.53	1.51
Maximum Dry Density (g/cc)	ASTM D4253 (2014)	1.68	1.61

Grain size distribution curves of grade-II and grade-III I.S. sand are presented in figure 3.1 and sieve analysis test was performed in accordance with ASTM D6913 (2004). The boundaries for liquefiable soil range were proposed by Xenaki and Athanasopoulos (2003) and both grade-II and grade-III I.S. sands falls in the range of most liquefiable soil. From the grain size distribution curve, the coefficient of uniformity (C_u) was calculated as 1.96 and 2.75 and coefficient of curvature (C_c) was calculated as 1.10 and 1.01 for grade-II and grade-III I.S. sands respectively. Both grades of sand were classified as poorly-graded sand (C_c values are in between 1 to 3 and C_u values are less than 6) according to Unified Soil Classification System (ASTM D2487 (2011)).

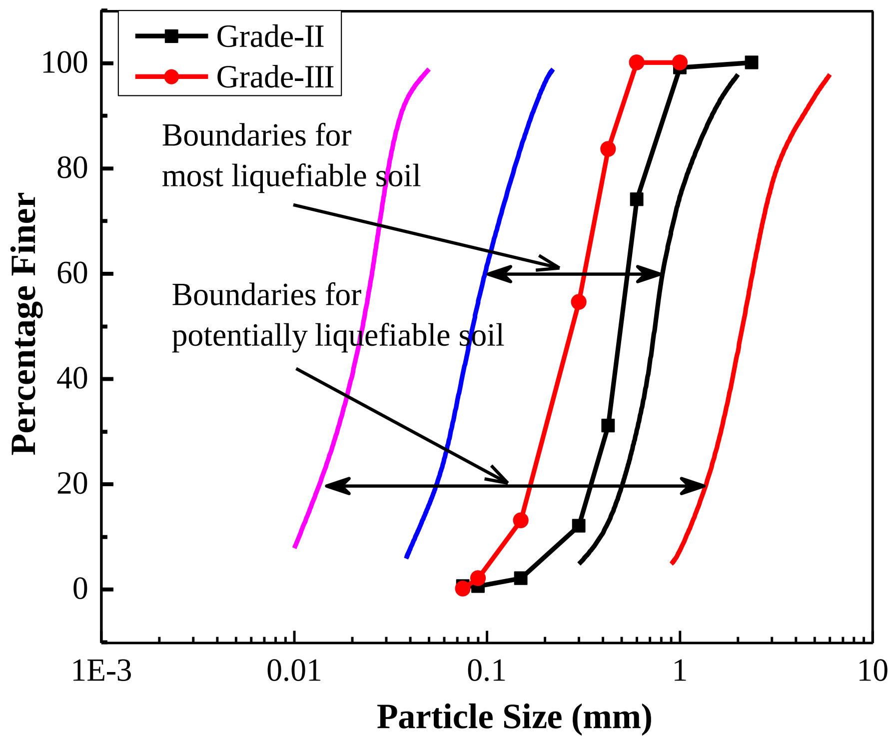


Figure 3.1: Grain size distribution curve of Grade-II and Grade-III I.S. sand

3.2. Ramagundam Bottom Ash

In this study, bottom ash obtained from NTPC Ramagundam, a part of National Thermal Power Corporation, located in Ramagundam, Telangana state, India was used. It is the first super thermal power station in India with a capacity of 2600 MW. Super Thermal Power Stations (STPS) or Super Power Stations are a series of ambitious power projects planned by the Government of India in 1990s.

3.2.1. Specific Gravity

The specific gravity of bottom ash was determined in accordance with ASTM D854 (2014), and the value obtained was equal to 1.88, an average value from three trial tests. According to Sridharan et al. (2001e), the specific gravity of typical Indian bottom ash lies in the range of 1.47 to 2.19.

3.2.2. Sieve Analysis

The Sieve analysis of bottom ash was conducted as per ASTM D6913 (2004). Figure 3.2 represents the grain-size distribution curve of bottom ash. The effective size and average size, D_{10} and D_{50}, of bottom ash, were equal to 0.10 mm and 0.25 mm, respectively.

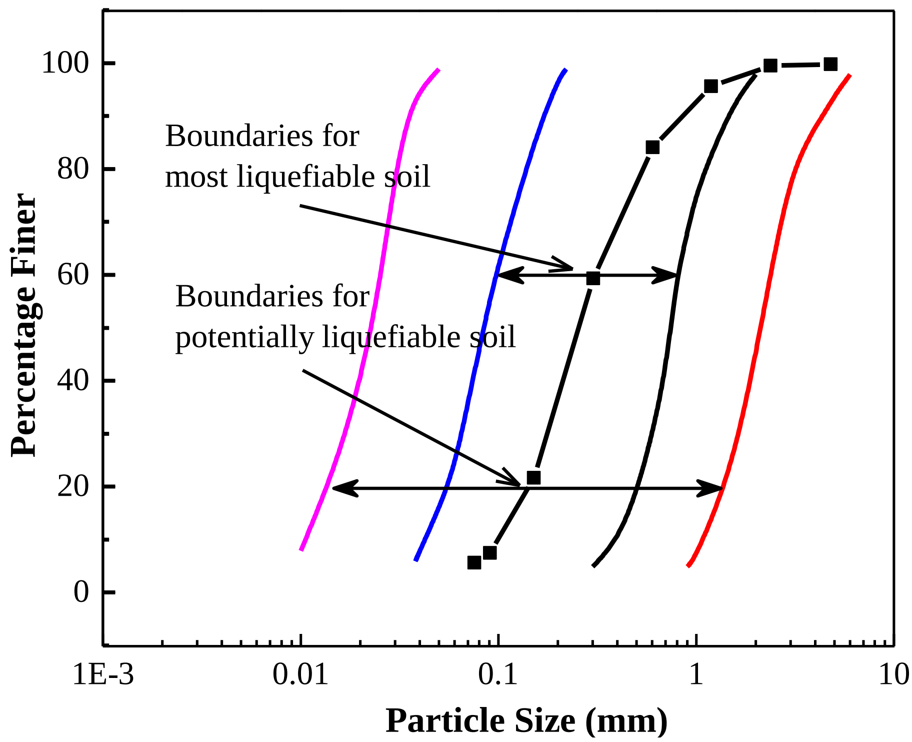


Figure 3.2: Grain size distribution curve of bottom ash

From the grain size distribution curve, the coefficient of uniformity (C_u) was calculated as 3 and coefficient of curvature (C_c) was calculated as 1.08. Bottom ash was classified as poorly-graded sand (C_c value is 1.08, ranged from 1 to 3 and C_u value is 3 which is less than 6) according to Unified Soil Classification System (ASTM D2487 (2011)). The boundaries for liquefiable soil range were proposed by Xenaki and Athanasopoulos (2003) and bottom ash considered in this study falls in the range of most liquefiable soil.

3.2.3. Maximum and Minimum Dry Densities

Maximum and minimum dry densities of bottom ash are determined as per ASTM D4253 (2014) and ASTM D4254 (2014) respectively and the values of maximum and minimum dry densities of bottom ash are 0.894 g/cc and 0.776 g/cc respectively.

3.2.4. Direct Shear Tests

The shear strength parameters (angle of shearing resistance, ϕ and cohesion, c) are determined by performing direct shear tests on dry bottom ash samples in accordance with ASTM D3080 (2011). The figure 3.3 shows the plot between shear stress and normal stress (shear strength envelope) obtained by direct shear tests. From figure 3.3, value of angle of shearing resistance is 41° and cohesion is 25 kPa. The reason for cohesion is the presence of finer bottom ash particles.

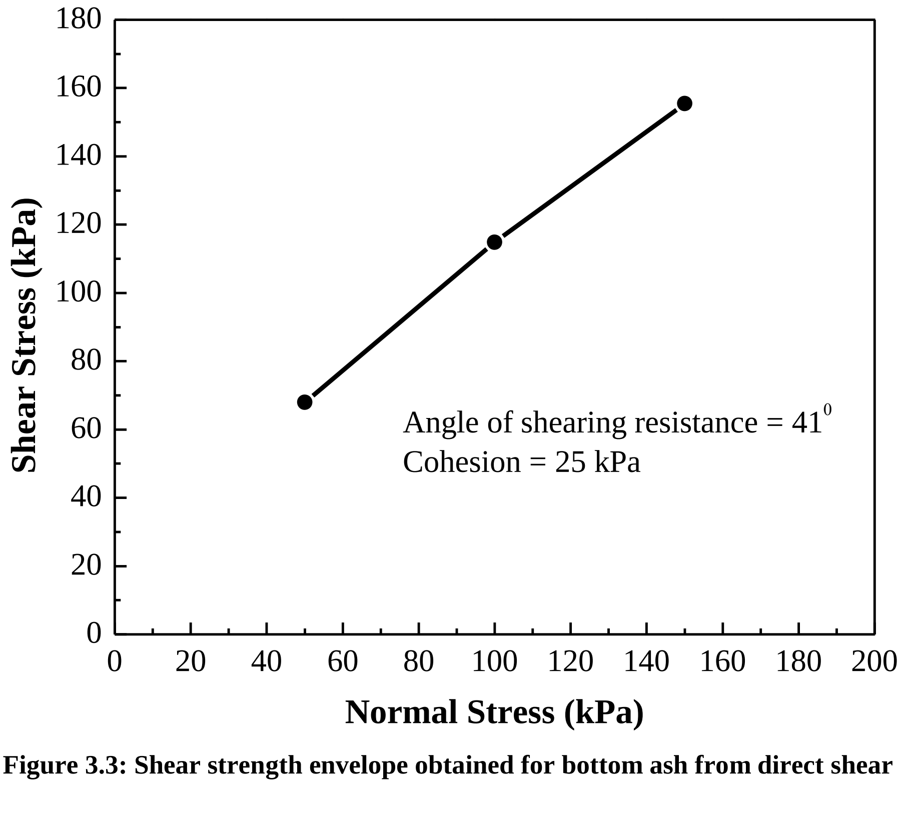


Figure 3.3: Shear strength envelope obtained for bottom ash from direct shear tests

3.2.5. Compaction Characteristics

Standard compaction test was conducted on bottom ash to determine the compaction characteristics, i.e., optimum moisture content (OMC) and the maximum dry density

(MDD). Compaction tests are performed in accordance with ASTM D698 (2012). Figure 3.4 shows the compaction curve of the bottom ash, from which MDD is determined as 1.12 g/cc at an OMC of 30.8%. Being bottom ash possess low specific gravity value compare to natural soils, OMC increased and MDD decreased in comparison to natural soils. Sridharan et al. (2001b) reported OMC in between 26% to 75.1% and MDD in between 0.58 g/cc to 1.12 g/cc during optimum state by testing various Indian bottom ash samples. According to Lovell et al. (1991), compaction curves for bottom ash exhibit maximum dry density at either an air-dried condition or a wet or flushed condition. Flushed conditions can be maintained in the field, producing a maximum dry density (Huang (1990)).

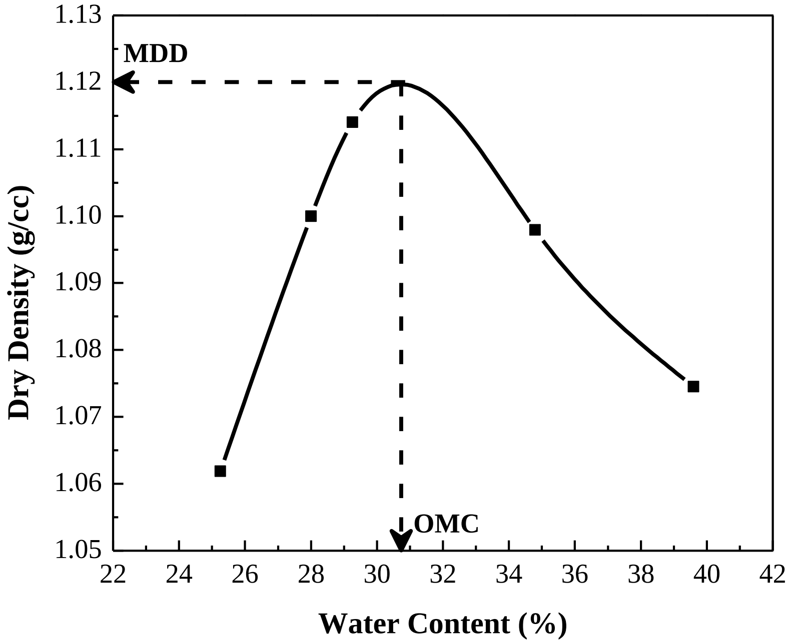


Figure 3.4: Compaction curve of bottom ash

3.3. Neyveli Fly Ash

In this study, fly ash obtained from Neyveli Thermal Power Station (NTPC), Chennai, India was used. NTPC is situated near lignite mines of Neyveli and capable of producing 2990 MW (as on December, 2014) with two distinct units. The fly ash was collected directly from the fly ash hoppers in air-tight containers.

3.3.1. Specific Gravity

The specific gravity of fly ash was determined in accordance with ASTM D854 (2014), and the value obtained was equal to 2.62, an average value from three trial tests.

3.3.2. Sieve Analysis

The Sieve analysis of fly ash was conducted as per ASTM D2487 (2011). Figure 3.5 represents the grain-size distribution curve of fly ash. The effective size and average size, D_{10} and D_{50}, of fly ash, were equal to 0.08 mm and 0.18 mm, respectively. From the grain size distribution curve, the coefficient of uniformity (C_u) was calculated as 2.5 and coefficient of curvature (C_c) was calculated as 1.225. Fly ash was classified as poorly-graded sand (C_c value is 1.225, ranged from 1 to 3 and C_u value is 2.5 which is less than 6) according to Unified Soil Classification System (USCS).

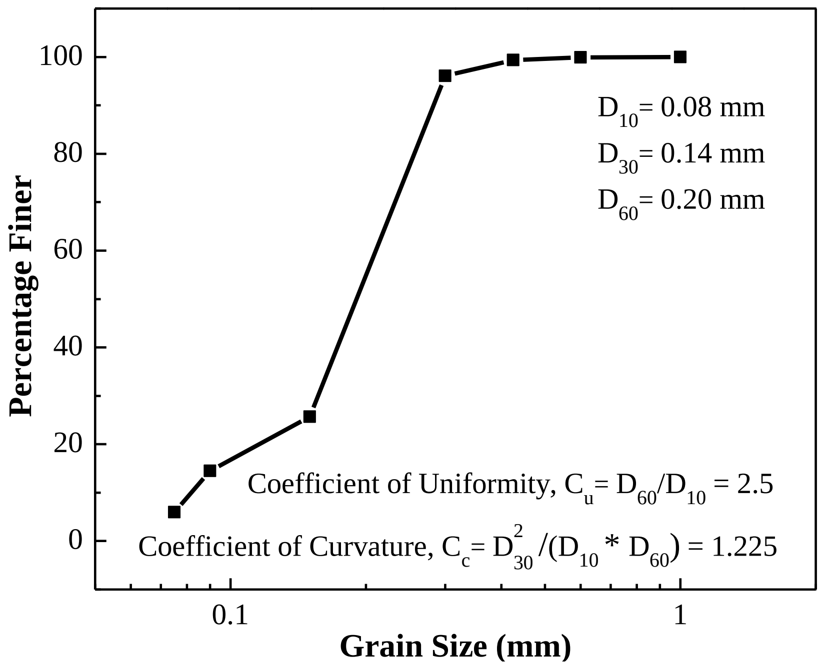


Figure 3.5: Grain size distribution curve of fly ash

3.3.3. Chemical Composition

Chemical composition of the fly ash sample was determined by doing X-ray Fluorescence (XRF) analysis. Table 3.2 shows the chemical composition of fly ash determined as per

ASTM specifications (ASTM C618-2012) and fly ash is classified as class-F fly ash as lime (CaO) content is less than 15% and contains greater proportions of silica (SiO_2), alumina (Al_2O_3) and iron (Fe_2O_3).

Table 3.2: Chemical composition of fly ash

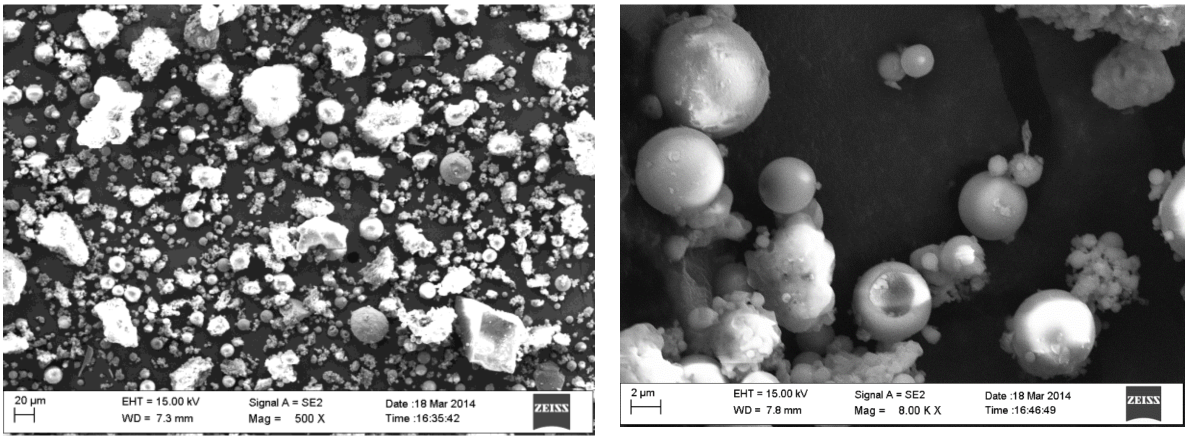
Chemical Compound	**Percentage**
MgO	1.85
Al_2O_3	32.34
SiO_2	40.6
CaO	11.9
K_2O	0.11
Fe_2O_3	9.6

3.3.4. Morphology

Scanning Electron Microscopic (SEM) studies were conducted to analyze the shape and surface of the fly ash particles. Due to the non-conductive nature of fly ash particles, gold coating was done on their surface. This procedure produced a clear scanning electron micrograph (SEM) image of the fly ash particles. Figures 3.6 shows the SEM images of fly ash particles. The SEM images are indicating that most of the fly ash particles are spherical in shape with varying sizes and the surfaces of the particles are observed to be smooth. The images of fly ash were taken at a magnification factor equal to 500x and 8000x. Being fly ash particles are smaller in size, high magnification factors were chosen for fly ash.

3.3.5. Compaction Characteristics

Standard compaction test was conducted on fly ash to determine the compaction characteristics, i.e., optimum moisture content (OMC) and the maximum dry density (MDD). Compaction tests are performed in accordance with ASTM D698 (2012). Figure 3.7 shows the compaction curve of the fly ash, from which MDD is determined as 1.39 g/cc at an OMC of 26%.



(a) (b)

Figure 3.6: SEM images of fly ash at magnification factors of (a) 500x, and (b) 8000x

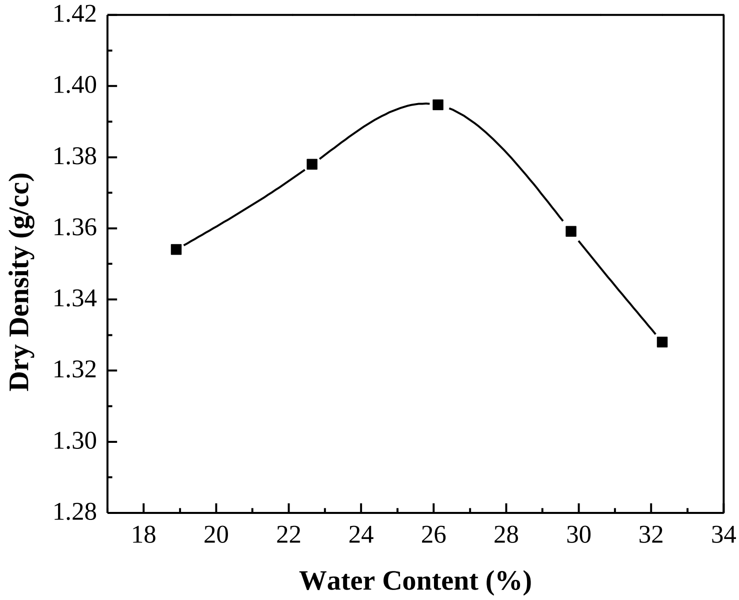


Figure 3.7: Compaction curve of fly ash

3.4. Ramagundam Pond Ash

In this study, pond ash obtained from Ramagundam Thermal Power Plant (RTPP), a part of National Thermal Power Corporation, located in Ramagundam, Telangana state, India is used. It is the first super thermal power station in India with a capacity of 2600 MW. Super Thermal Power Stations (STPS) or Super Power Stations are a series of ambitious power projects planned by the Government of India in 1990s. Figure 3.8 shows the collection of pond ash from RTPP using a back hoe. Pond ash was transported to the laboratory in the air-tight containers.

Figure 3.8: Collection of pond ash from RTPP using a back hoe

3.4.1. Specific Gravity

The specific gravity of pond ash was determined in accordance with ASTM D854 (2014), and the value obtained was equal to 2.01, an average value from three trial tests. According to Sridharan et al. (2001e), the specific gravity of typical Indian pond ash lies in the range of 1.64 to 2.66.

3.4.2. Sieve Analysis

The Sieve analysis of pond ash was conducted in accordance with ASTM D2487 (2011). Figure 3.9 represents the grain-size distribution curve of pond ash. The effective size and average size, D_{10} and D_{50}, of bottom ash, were equal to 0.08 mm and 0.21 mm, respectively. From the grain size distribution curve, the coefficient of uniformity (C_u) was calculated as 3.125 and coefficient of curvature (C_c) was calculated as 1.62. Pond ash was classified as poorly-graded sand (C_c value is 1.62, ranged from 1 to 3 and C_u value is 3.125 which is less than 6) according to Unified Soil Classification System (USCS).

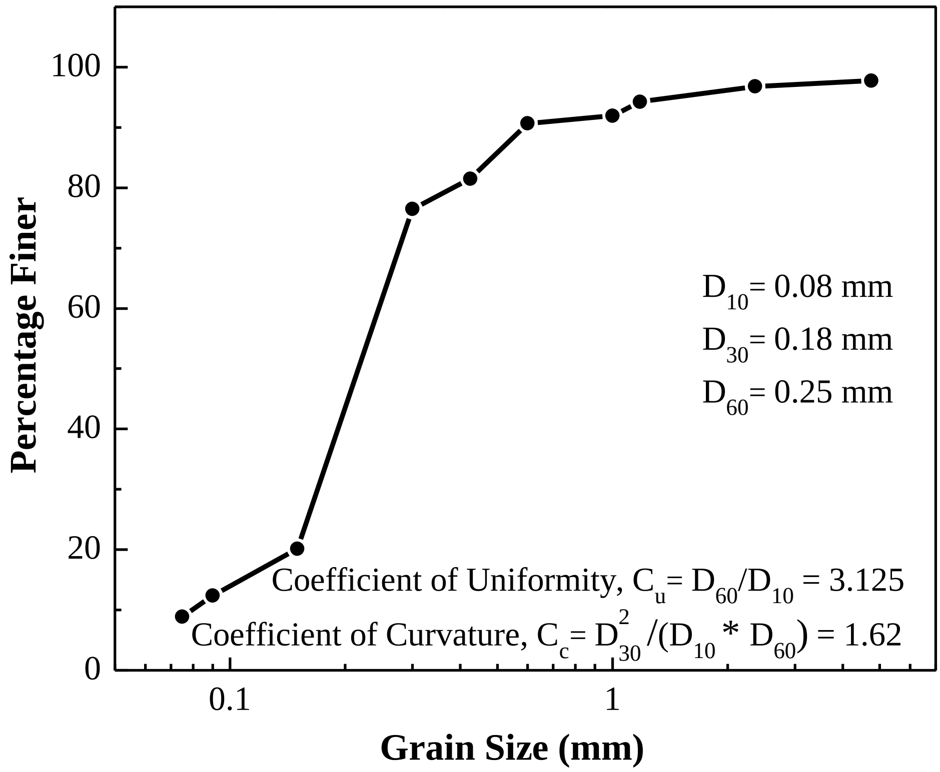


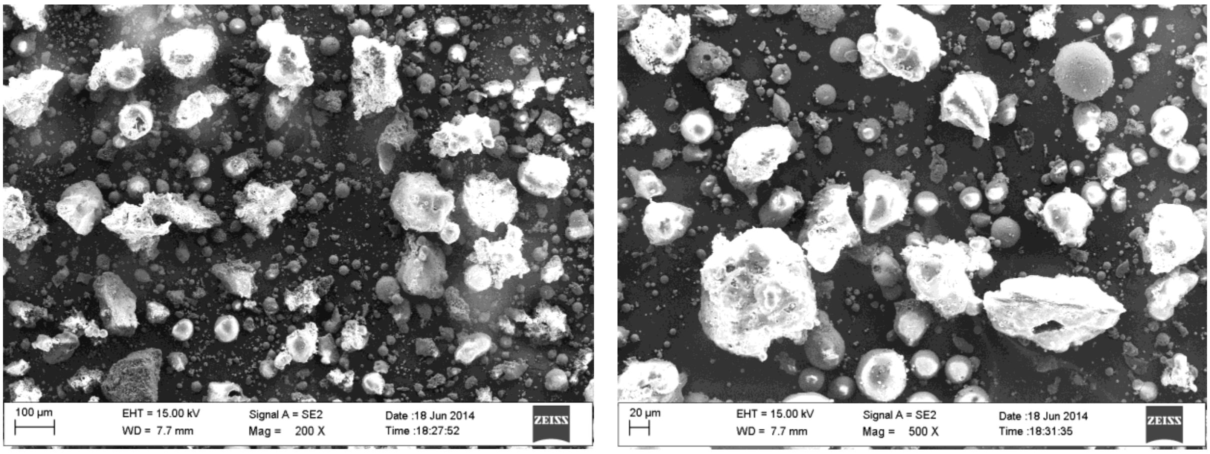
Figure 3.9: Grain size distribution curve of pond ash

3.4.3. Morphology

Scanning Electron Microscopic (SEM) studies were conducted to analyze the shape and surface of the pond ash particles. As pond ash particles are non-conductive in nature, gold coating was done on their surface and this will produce a clear scanning electron micrograph (SEM) image of the pond ash particles. Figures 3.10 (a) and (b) show the SEM images of pond ash particles. The images of pond ash were taken at a magnification factor equal to 200x and 500x. From the images, it was found that pond ash is composed of both angular and spherical particles with varying sizes and the surfaces of the particles are observed to be smooth.

3.4.4. Chemical Composition

Chemical composition of the pond ash sample was determined by doing X-ray Fluorescence (XRF) analysis. Table 3.3 shows the chemical composition of pond ash determined as per ASTM specifications (ASTM C618-2012) and pond ash is classified as class-F ash as lime (CaO) content is less than 15% and contains greater proportions of silica (SiO_2), alumina (Al_2O_3) and iron (Fe_2O_3).



(a) (b)

Figure 3.10: SEM images of pond ash at magnification factors of (a) 200x, and (b) 500x

Table 3.3: Chemical composition of pond ash

Chemical Composition	**Percentage**
SiO_2	59.0
Al_2O_3	19.5
Fe_2O_3	15.3
CaO	3.1
MgO	1.1
TiO_2	0.19
SO_3	0.18
$Na_2O + K_2O$	0.02

3.4.5. Compaction Characteristics

Standard compaction test was conducted on pond ash in accordance with ASTM D698 (2012) to determine the compaction characteristics, i.e., optimum moisture content (OMC) and the maximum dry density (MDD). Figure 3.11 shows the compaction curve of the pond ash, from which MDD is determined as 1.04 g/cc at an OMC of 15%. Being pond ash possess low specific gravity value compare to natural soils, OMC increased and MDD decreased in comparison to natural soils. Sridharan et al. (2001b) reported OMC in between 18.2% to 45.1% and MDD in between 0.9 g/cc to 1.72 g/cc during optimum state by testing various Indian pond ash samples.

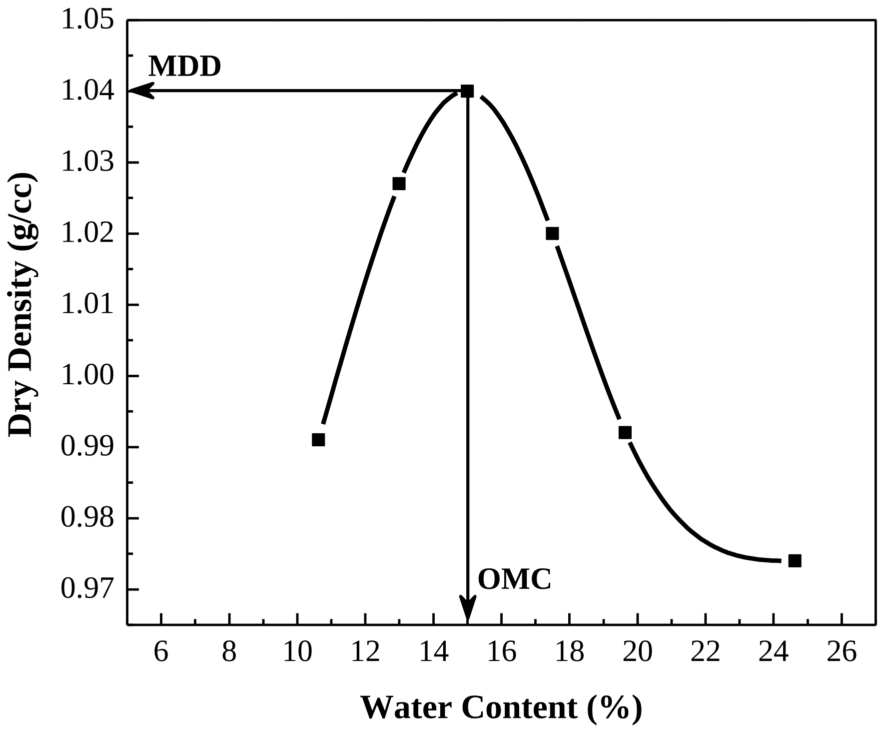


Figure 3.11: Compaction curve of pond ash

Chapter 4

Dynamic Cyclic Simple Shear Testing

4.1. Dynamic Cyclic Simple Shear Test – Sample Preparation

Different types of cyclic simple shear devices are available and developed by different manufacturers. In this research, Electro Mechanical Dynamic Cyclic Simple Shear (EMDCSS) system, manufactured by GDS instruments is used. The complete system is controlled by the GDSLAB software application program. ASTM D6528 (2007) 'cyclic simple shear tests for clays' specifies minimum specimen diameter (D) must be 45 mm, minimum specimen height (H) must be 12 mm and $H/D \leq 0.4$. Also, specimen height shall not be less than ten times the maximum particle diameter. For determining the dynamic properties of dry I.S. sand and bottom ash, 70 mm diameter and 25 mm height cylindrical soil specimens were prepared by tamping in three layers. Loose sand samples (relative density = 30%) and medium dense sand samples (relative density = 50%) and dense samples (relative density = 75%) are considered in this study. Each layer is compacted by hand tamping to the required density. For determining the dynamic properties of ash materials, 70 mm diameter and 25 mm height cylindrical soil specimens were prepared at optimum moisture content (OMC) and maximum dry density (MDD) of corresponding ash material. The soil sample is set up in the DCSS machine, which consists of top portion and bottom portion. The top portion holds the vertical ram and it is allows only vertical movement with the help of linear bearings, thus preventing horizontal movement. The bottom portion is mounted on roller bearings as in a standard shear box, which move only in the lateral direction. The soil sample is supported by a rubber membrane placed and secured with O-rings. To maintain a constant diameter throughout the test the soil sample is supported by a series of Teflon coated sample rings. During the shearing stage, the rings will slide over each other and 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 during shearing stage, resulting in a constant volume test. Figures 4.1, 4.2 and 4.3 show the important accessories for the specimen preparation, sequence of sample preparation for I.S. sand and bottom ash and for ash materials respectively for the testing.

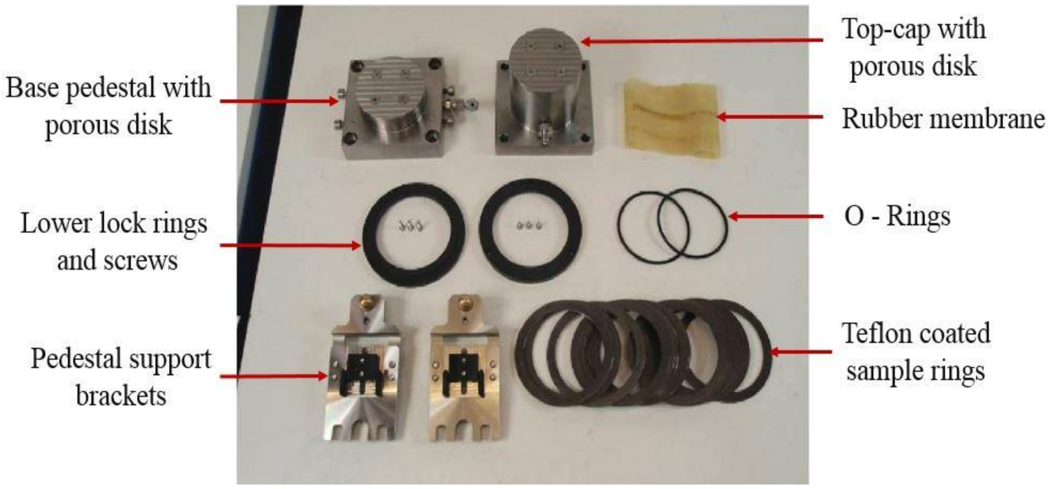


Figure 4.1: Accessories required for specimen preparation

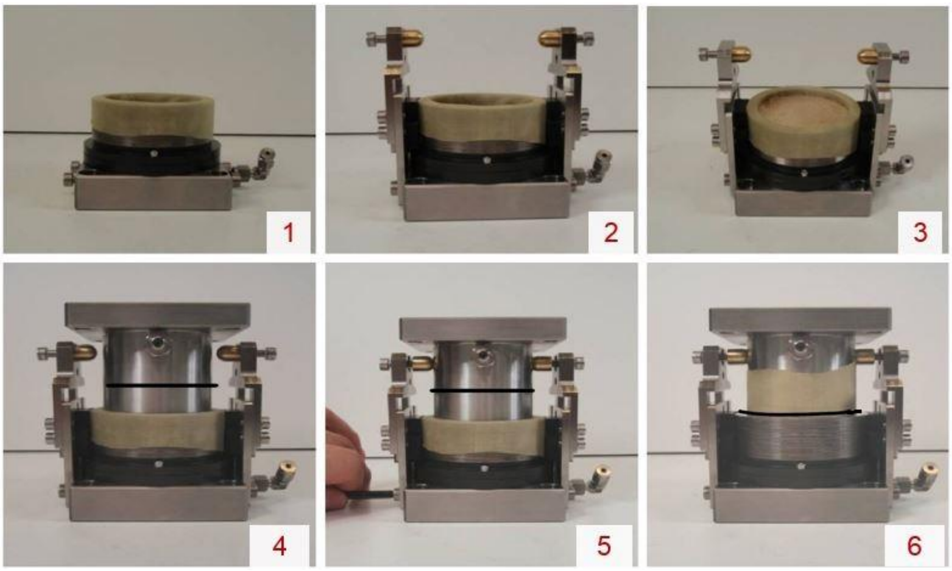


Figure 4.2: Sequence of sample preparation for I.S. sand and bottom ash

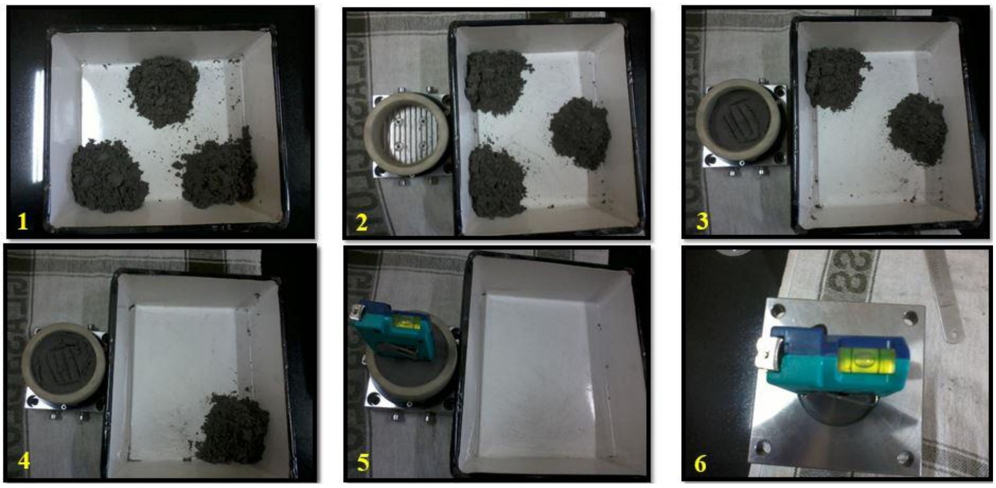


Figure 4.3: Sequence of specimen preparation for ash materials

4.2. Different Stages and Data Monitoring

4.2.1. Consolidation Stage

The consolidation stage is the application of vertical stress to the specimen while the lateral loading (shear) axis is held stationary/constant. Vertical stress and specimen displacements (axial and lateral) data are measured over time and logged by the system. As the test progresses, logged data can also be visualized in the form of charts and tables. The consolidation stage can be stopped automatically or manually by the operator once consolidation of the specimen is said to be completed. In the present work, the dry I.S. sand and bottom ash specimens were subjected to vertical stresses of 100 kPa, 200 kPa and 400 kPa and the ash specimens were subjected to normal stresses of 40 kPa, 120 kPa, 200 kPa and 350 kPa.

4.2.2. Cyclic Simple Shear Stage

In cyclic simple shear stage, the soil specimen will be subjected to a specified amplitude of lateral strain. Table 4.1 shows the range of shear strain amplitudes and corresponding shear strains used in the current study. The data logger will record lateral force, lateral displacement, axial displacement and axial force during the progress of test. I.S. sand specimens and ash specimens were subjected to 50 cycles of sinusoidal loading and for each cycle, 50 data capturing points are used to record the complete data of normal stress, axial displacements, lateral stress and lateral displacements for each cycle. Data logger provided by GDSLAB directly gives the values of shear stress and shear strain developed during the testing of the specimen. The plot of shear stress against the shear strain for each cycle is known as hysteresis loop of that particular cycle. The hysteresis loop will be used in the calculation of shear modulus and the damping ratio for each cycle. Figure 4.4 shows the ideal hysteresis loop and the procedure to determine shear modulus and damping ratio from the hysteresis loop, which was well explained by Vucetic (1992), Yimer (2010), Sheshov (2011) and etc.

Table 4.1: Shear strain magnitudes considered in the study

Magnitude(mm)	Shear strain (%)
0.0025	0.01
0.0125	0.05
0.025	0.1
0.125	0.5
0.25	1

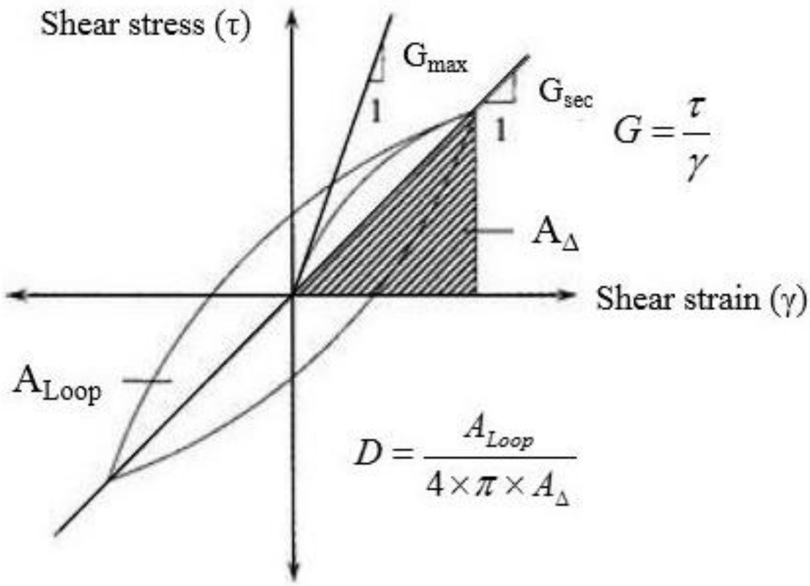


Figure 4.4: Ideal hysteresis loop and determination of dynamic properties from loop

4.3. Overview of the Testing Method

Dynamic cyclic simple shear testing is used in this research work to determine the shear modulus and damping ratio of Indian Standard Sand, Fly ash, Pond ash and Bottom ash. It has long been recognized, however, that there is no standard procedure to characterize the cyclic behavior of sands within the geotechnical engineering community. It is found that there are three factors that are more critical than others when trying to reproduce meaningful and comparable results [Jun Wang, 2005]:

- Specimen preparation method
- Shape of the loading pattern
- Initial state of sample tested

4.3.1. Specimen preparation method

The method of reconstituting samples has a strong effect on dynamic test results, and it is important to consider this effect in the interpretation of test data. In this experimental work, the tamping specimen preparation technique was used for preparation of sand and bottom ash in dry state and fly ash and bottom ash samples are compacted at their compaction characteristics (OMC and MDD). Wong et al. (1975) compared the effects of size considering 70 mm (2.8 in) and 300 mm (12 in.) diameter specimens with similar height-to-diameter ratios and showed that the 300 mm (12 in.) diameter specimen was approximately 10% weaker than the 70 mm (2.8 in.) diameter specimen. However, Mulilis et al. (1977) have observed that the variation of sample diameters does not significantly affect the cyclic strength.

4.3.2. Shape of the loading pattern

Three wave forms are commonly used in geotechnical research laboratories including triangular, square/rectangular-shaped and sinusoidal. Square-wave loading produces more severe conditions than sinusoidal loading and consequently may produce an apparently lower cyclic shear strength. Thiers (1965) have investigated that the triangular loading waveform gives 5 – 20% higher strength than the rectangular loading. Generally, sine wave is used for cyclic loading which gives approximately 30% higher cyclic strength than the rectangular or triangular wave forms [Mulilis et al. (1978)]. The sinusoidal wave form was recommended [Jun Wang (2005] and has been used in all tests conducted in this research work.

4.3.3. Initial state of sand tested

According to the literature on characterizing the behavior of saturated sands under cyclic loading, the main influencing factors have been identified as the initial effective confining pressure σ_0' and density [Jun Wang, 2005]. Void ratio (relative density) is one of the mechanical properties of specimen which is mainly influenced by the static/dynamic actions of loading. As the void ratio becomes lesser under the application of load, particles come closer to each other resulting in densification of sample. As the confining pressure increases, the shear modulus increases and damping ratios decreases because of the densification/compactness of sample. For this reason, in this study, specimens are tested under different vertical stresses and densities.

4.3.4. Other factors:

(a) Excitation Frequency

Based on the experimentation for a wide range of excitation frequency, Lee and Fitton (1969) reported that the lower loading frequency produced slightly lower strength.

However, Wong et al. (1975) and Wang (1972) reported that the slower loading frequency gives slightly higher strength, which is in contrast to the above mentioned finding. Based on the both resonant column and cyclic torsional shear tests performed by Lin et al. (1996, 1988) and cyclic triaxial tests by GovindaRaju (2005), it has been observed that the shear modulus is not significantly affected while damping ratios are significantly affected by the excitation frequency. Information available at the present time indicates that the dynamic behavior of soils is relatively insensitive to the frequency of applied cyclic loading within the range of 0.5 to 2 Hz.

(b) Percentage Fines

Hanumantharao and Ramana (2008) performed stress and strain controlled undrained cyclic triaxial tests on remoulded sand and sandy slit (silt content: 0 – 100%) specimens of 70 mm diameter and 140 mm height under a sinusoidal loading at 1 Hz frequency for evaluating the modulus reduction and damping curves and reported that the shear modulus is not significantly affected, although with the increase in silt content, the damping ratio was observed to decrease.

Chapter 5

Results and Discussions

5.1. I.S. Sand

5.1.1. Influence of number of cycles

The influence of number of cycles on the dynamic behavior of I.S. sand was investigated by subjecting the soil specimen to 50 cycles of sinusoidal loading. With the increase in the number of cycles, hysteresis loop is becoming flatter as shown in figure 5.1. The decrease in peak load as the number of cycles increases is conveyed by the progressive flattening of the shear stress versus shear strain curve. Figure 5.1 also conveys that, secant shear modulus, defined as the slope of a line through the end points of the hysteresis loop, decreases with the increase in number of cycles. Figure 5.2 shows the variation of secant shear modulus with the number of cycles, in which the reduction of secant shear modulus is higher at first few cycles. Figure 5.3 shows the influence on number of cycles on the damping ratio of I.S. sand. With the increase in number of cycles, the damping ratio increases in contrast to shear modulus, but the variation is not as much as secant shear modulus. Jafarzadeh et al. (2008) also stated that the rate of reduction of shear modulus will be higher for first few cycles and damping ratio increases with increase in number of cycles by conducting triaxial tests on the Hostun sand.

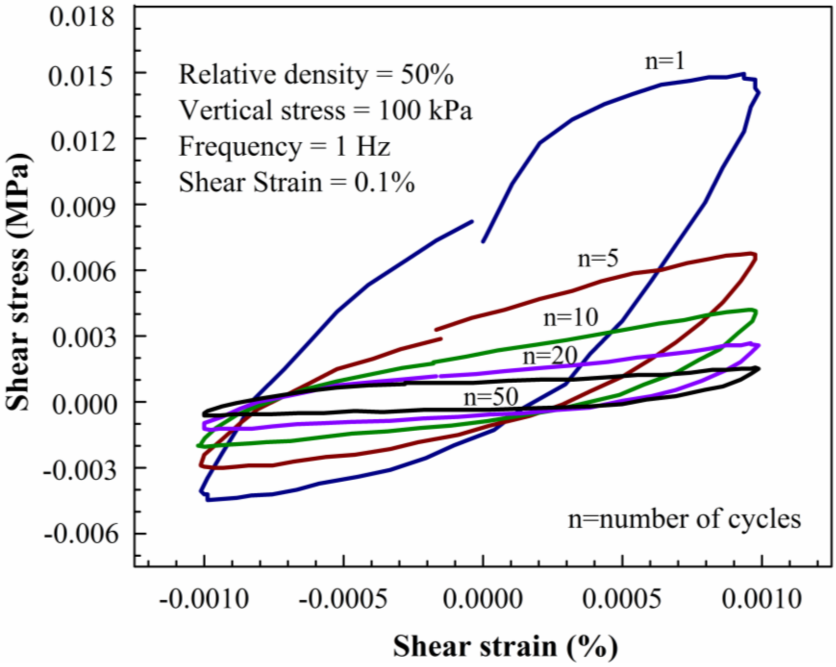


Figure 5.1: Influence of number of cycles on hysteresis loop for I.S. Sand

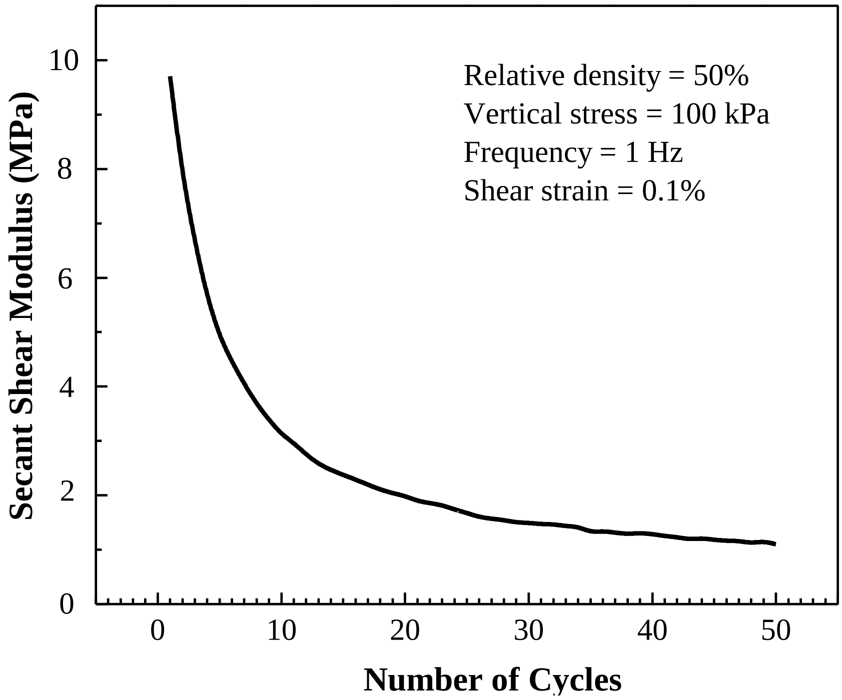


Figure 5.2: Influence of number of cycles on secant shear modulus

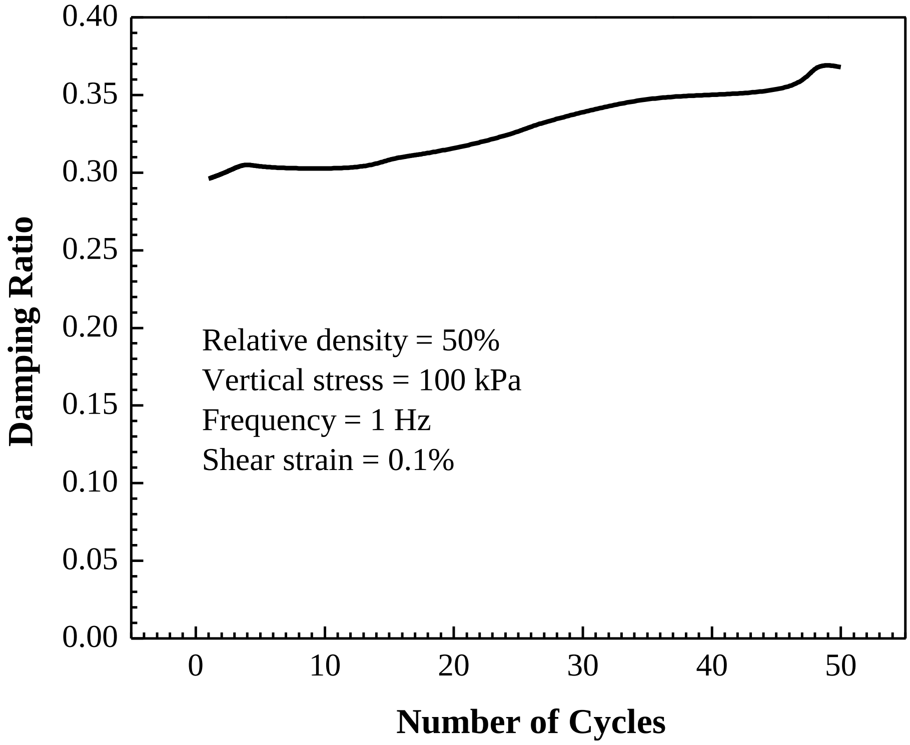


Figure 5.3: Influence of number of cycles on damping ratio

The extensive studies of Hardin and Drnevich (1972a and 1972b) and Dobry and Vucetic (1987) on normally consolidated and moderately consolidated soils to determine the factors influencing the dynamic properties of soils concluded that shear modulus will decrease with increase in number of loading cycles for undrained conditions and the damping ratio will not be influenced significantly by the number of loading cycles. Therefore, the results obtained in this study are in good agreement with the outcomes of Jafarzadeh et al. (2008), Hardin and Drnevich (1972a and 1972b) and Dobry and Vucetic (1987). Figures 5.1 to 5.3 are shown for fixed values of relative density equal to 50%, vertical stress equal to 100 kPa, frequency equal to 1 Hz and shear strain of 0.1%. Similar behavior can also be shown for other cases, i.e., when tested at relative densities of 30% and 75%, vertical stresses of 200 kPa and 400 kPa at various shear strains of magnitude 0.01%, 0.05%, 0.1% and 0.5%, maintaining 1 Hz frequency.

According to Das and Ramana (1993), the number of significant cycles would be less than 20 in most seismic events and suggested to consider the values of 5th cycle as the representative values of secant shear modulus and damping ratio for all practical purposes. Therefore, secant shear modulus and damping ratio values reported in the following sections corresponds to 5th cyclic loading.

5.1.2. Influence of Frequency

In order to study the influence of frequency on the dynamic properties of I.S. sand, the soil specimen was subjected to various frequencies ranging from 0.1 Hz to 2 Hz, preparing soil specimens at 50% relative density and subjecting samples to 100 kPa vertical stress and for a shear strain magnitude of 0.1%. Figures 5.4 and 5.5 show the variation of secant shear modulus and damping ratio with frequency respectively. With the increase in frequency from 0.1 Hz to 2 Hz, secant shear modulus of I.S. sand slightly increases and in contrast, damping ratio decreases. However, the change in damping ratio with frequency is considerably higher in comparison with secant shear modulus. A series of strain controlled cyclic triaxial element tests were performed by Ravishankar et al. (2005) on dry and saturated soil samples in medium to large shear strain levels and concluded that the effect of frequency is not significant on shear modulus but has some influence on the damping ratios of the soils for the range of frequencies tested (0.1 Hz to 2Hz). Therefore, the results obtained in this work are in good agreement with the conclusions of Ravishankar et al. (2005).

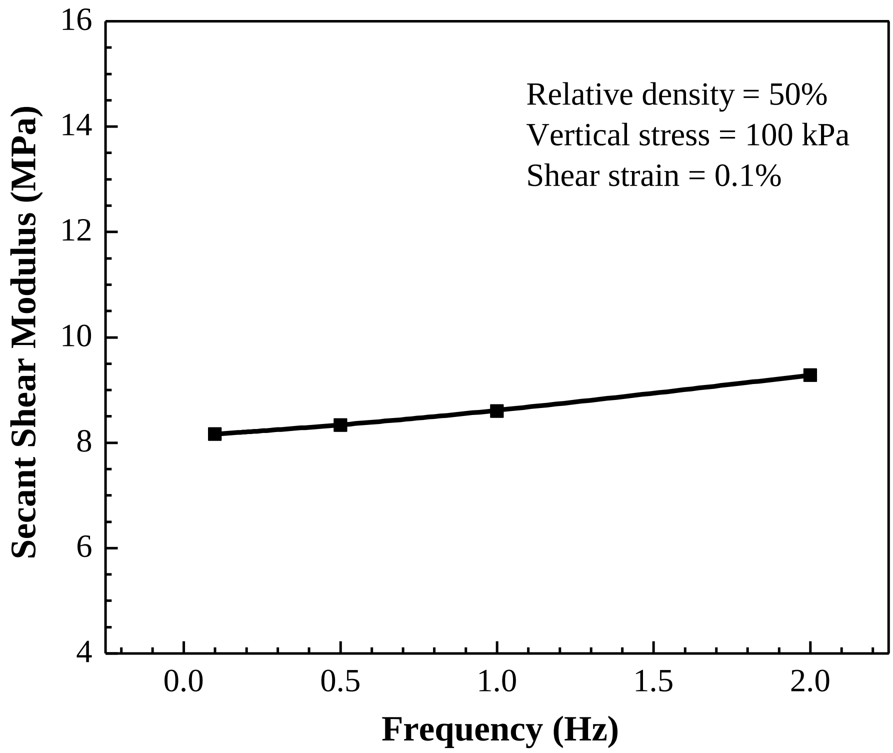


Figure 5.4: Influence of frequency on secant shear modulus

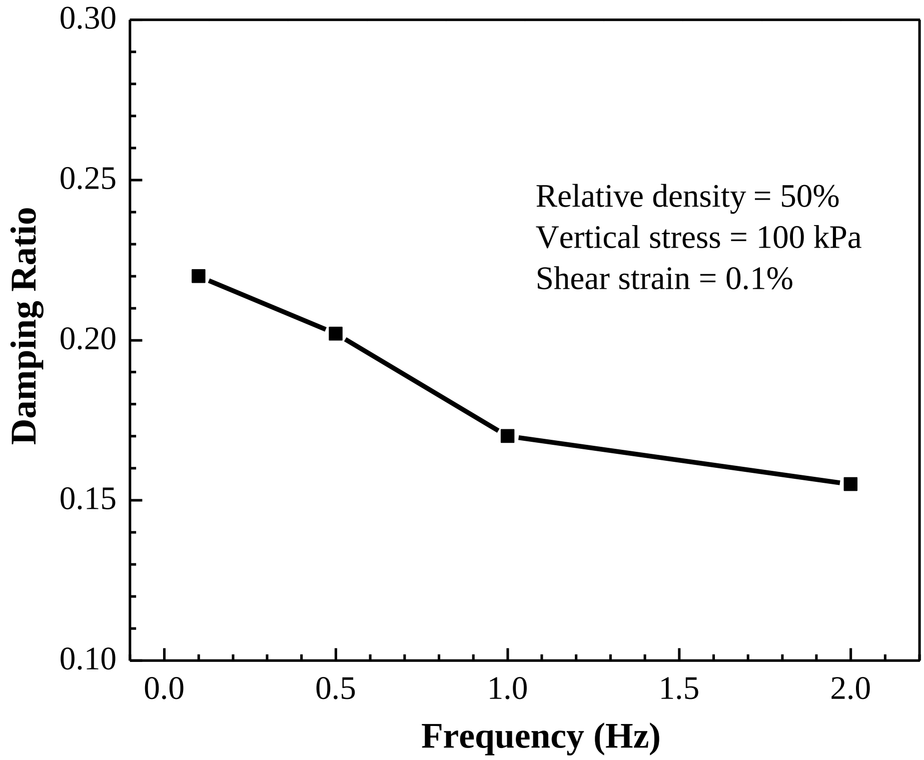


Figure 5.5: Influence of frequency on damping ratio

An average frequency value of 1 Hz was selected for performing the remaining tests to determine the influence of vertical stress, relative density and etc., on the dynamic properties of I.S. sand. Therefore, the values reported hereafter corresponds to a frequency of 1 Hz and 5th cyclic loading.

5.1.3. Influence of Grain Size

Influence of grain size on dynamic properties of I.S. sand was studied by testing two soil specimens having different particle sizes, namely grade-II I.S. sand (D_{50} = 0.50 mm) and grade-III I.S. sand (D_{50} = 0.28 mm). Properties of both grade-II and grade-III I.S. sand were mentioned in the table 3.1. Figures 5.6 and 5.7 show the variation of secant shear modulus and damping ratio with shear strain for two different grades of I.S. sand respectively. Figures are clearly indicating that there is no much influence on secant shear modulus, however, damping ratio is varying much with the change in grain size of the material. Moreover, the difference in shear modulus is decreasing with the increase in shear strain magnitude, i.e., at higher shear strains (> 0.1%), the variation is quite low.

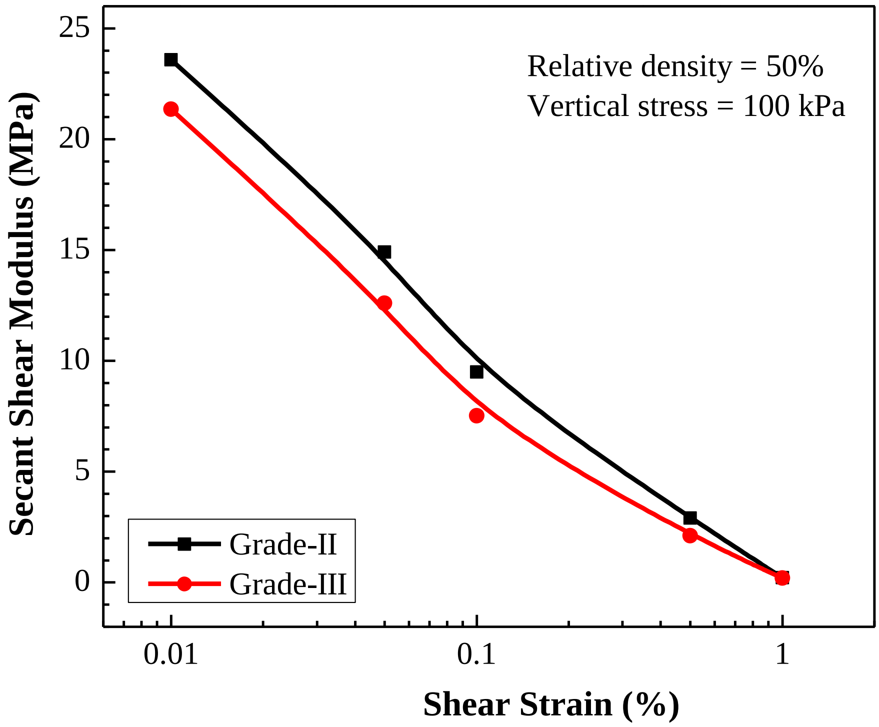


Figure 5.6: Influence of grain size on secant shear modulus

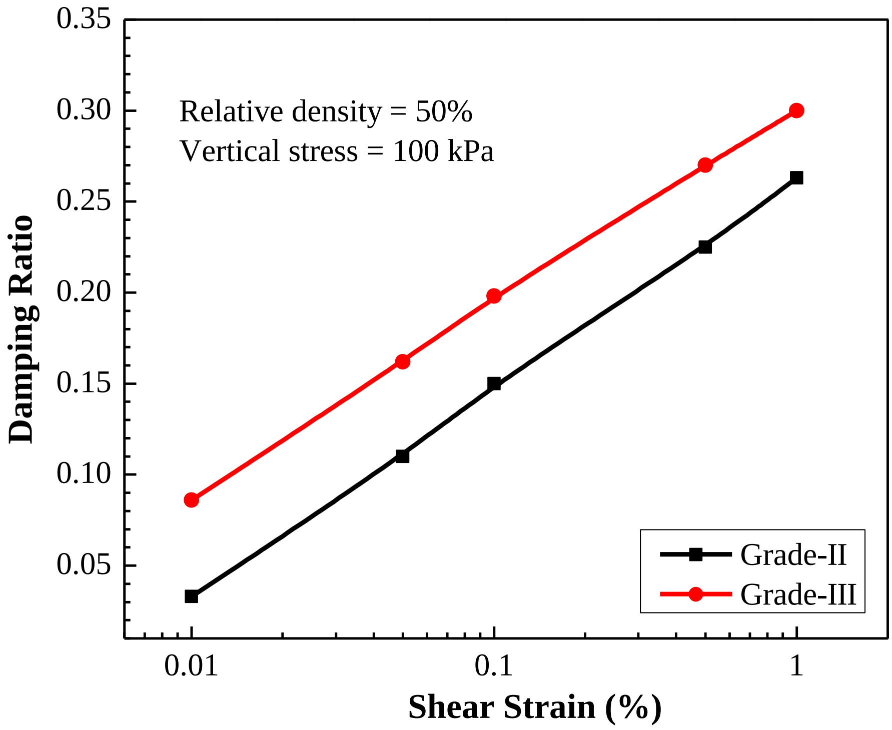


Figure 5.7: Influence of grain size on damping ratio

It is clear from the graphs, with the increase in magnitude of the shear strain, secant shear modulus decreases dramatically, and damping ratio increases. Similar behavior was observed by Hardin and Drnevich (1972b), Seed et al. (1986), Dobry and Vucetic (1987), Ravishankar et al. (2005), and etc., for the various tested materials like sand, clay, and gravel.

5.1.4. Influence of vertical stress

In order to study the influence of vertical stress on the dynamic properties of I.S. sand, soil specimens prepared at relative densities of 30%, 50% and 75% were tested at vertical stresses of 100 kPa, 200 kPa and 400 kPa. For a particular relative density and shear strain magnitude, with the increase in vertical stress secant shear modulus increases and damping ratio decreases. Figures 5.8 to 5.10 show the influence of vertical stress on secant shear modulus for relative densities of 30%, 50% and 75% respectively. Figures 5.11 to 5.13 show the influence of vertical stress on damping ratio for relative densities of 30%, 50% and 75% respectively. The studies of Silver and Seed (1971), Hardin and Drnevich (1972b), Dobry and Vucetic (1987), Ravishankar et al. (2005), Yimer (2010), etc., also concluded that with the increase in vertical stress, shear modulus increases and damping ratio decreases for various tested materials like sand, clay, and gravel at a particular shear strain magnitude.

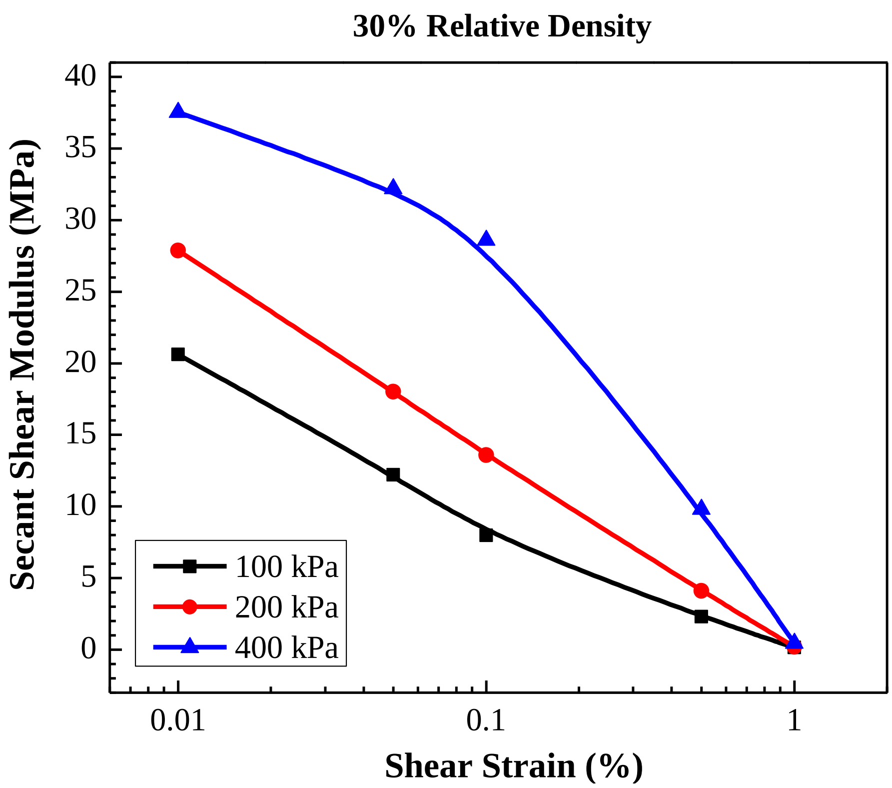


Figure 5.8: Influence of vertical stress on secant shear modulus for R_d = 30%

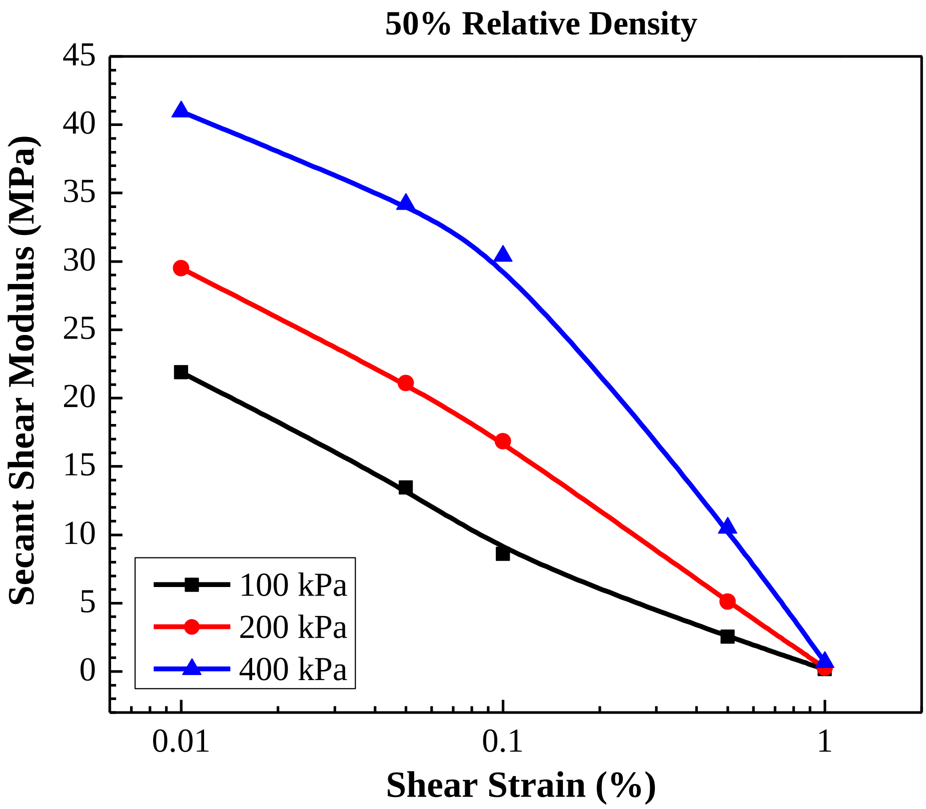


Figure 5.9: Influence of vertical stress on secant shear modulus for $R_d = 50\%$

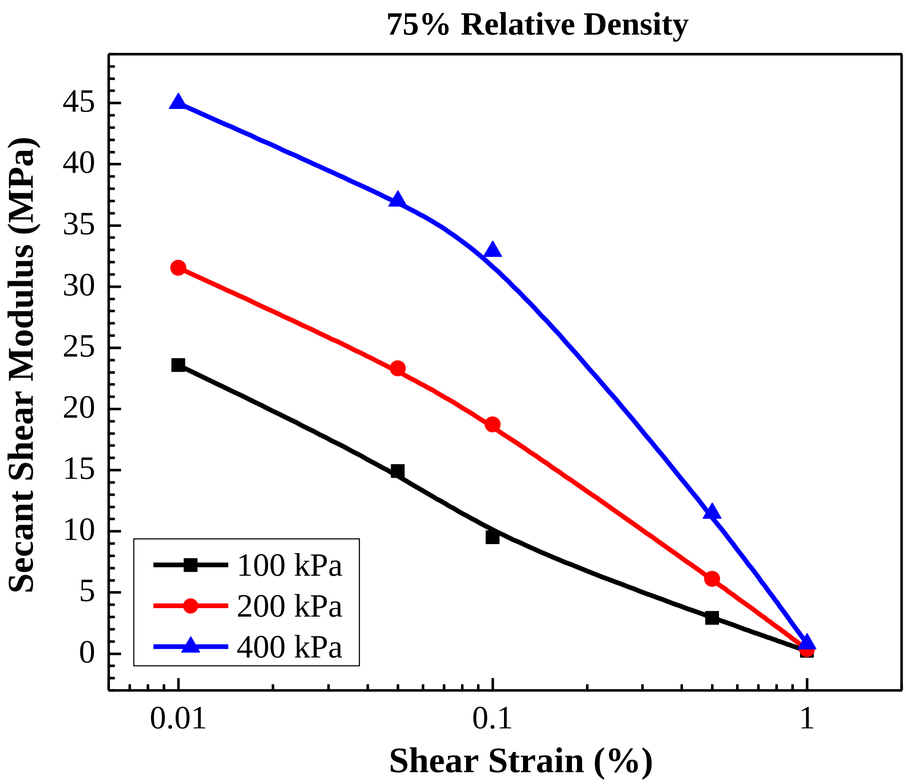


Figure 5.10: Influence of vertical stress on secant shear modulus for $R_d = 75\%$

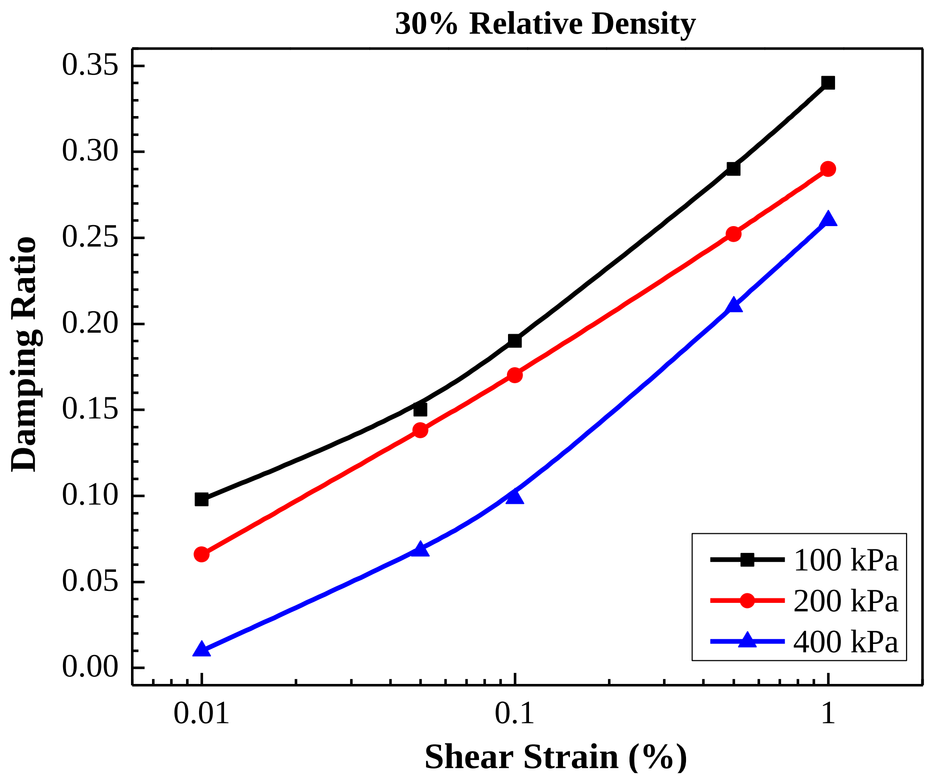


Fig. 5.11: Influence of vertical stress on damping ratio for R_d = 30%

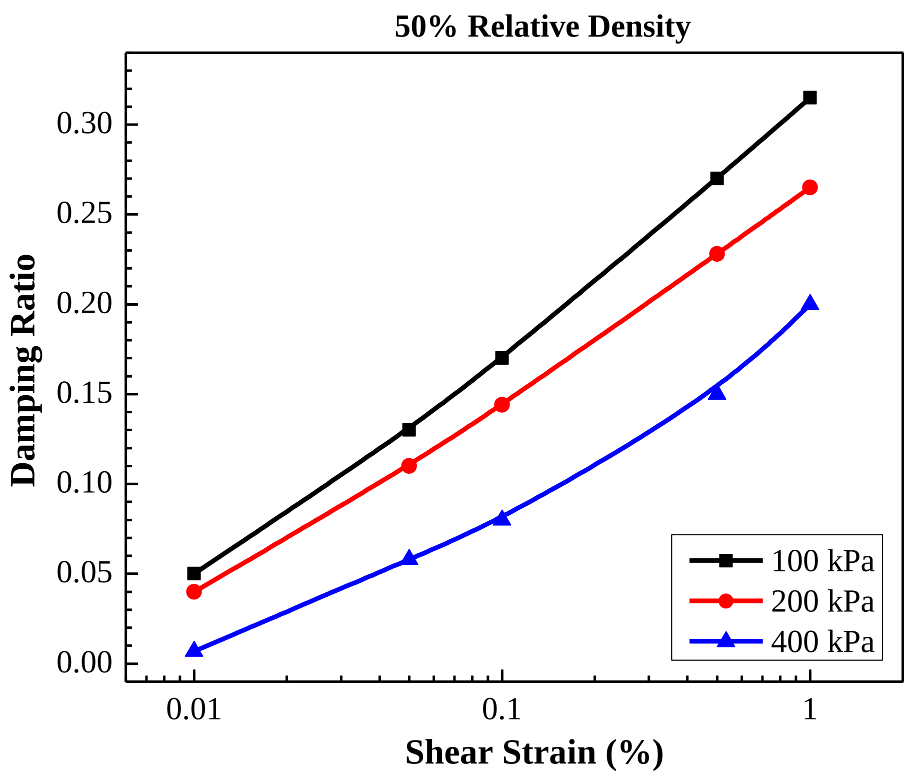


Figure 5.12: Influence of vertical stress on damping ratio for R_d = 50%

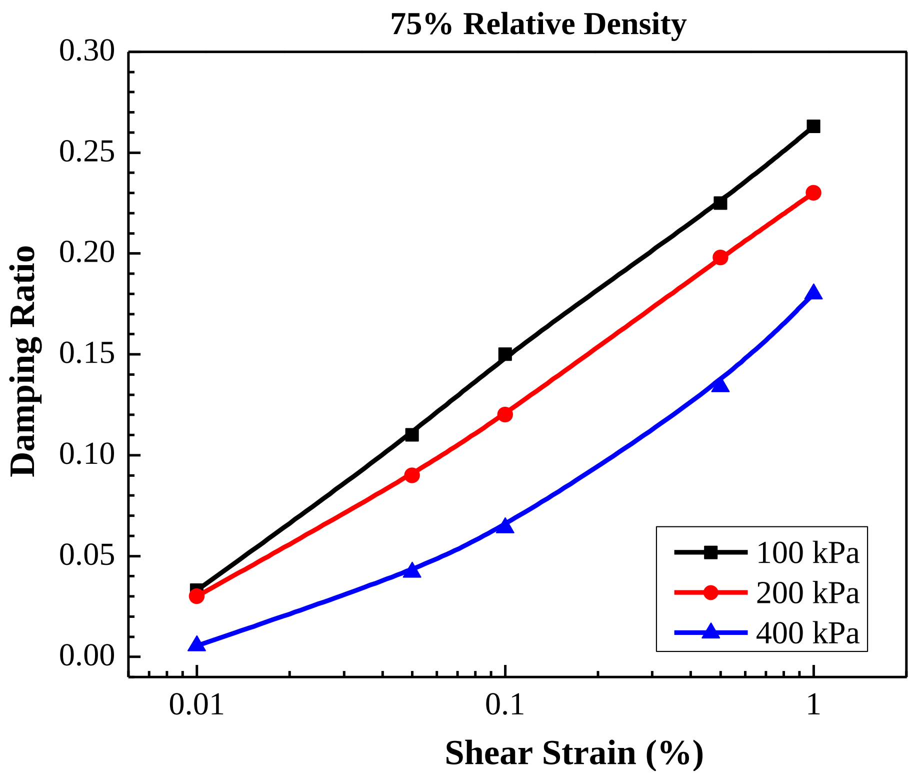


Figure 5.13: Influence of vertical stress on damping for $R_d = 75\%$

From the figures 5.8 to 5.13, it can be concluded that the secant shear modulus degradation curve (shear modulus variation with the shear strain) is attaining a shape of convex curve at higher vertical stresses and possess concave shape at lower vertical stresses. Similarly, damping ratio degradation curve (damping ratio variation with the shear strain) is attaining the shape of concave at higher vertical stresses with showing a linear variation at lower vertical stresses.

5.1.5. Influence of relative density

In order to study the influence of relative density on the dynamic properties of I.S. sand, soil specimen was tested at a relative density of 30%, 50% and 75% varying the vertical stresses from 100 kPa to 400 kPa. For a particular vertical stress and shear strain magnitude, with the increase in the relative density secant shear modulus increases and damping ratio decreases. Figures 5.14 to 5.16 show the influence of relative density on secant shear modulus for vertical stresses of 100 kPa, 200 kPa and 400 kPa respectively. Figures 5.17 to 5.19 show the influence of relative density on the damping ratio of I.S. sand for vertical stresses of 100 kPa, 200 kPa and 400 kPa respectively. Similar to the influence of vertical stress on dynamic properties, with the increase in relative density of sample secant shear modulus increases and damping ratio decreases at a particular vertical stress and shear strain. However, the influence of relative density on dynamic properties of sand is not as much as the vertical stress. The previous studies of Hardin and Drnevich (1972b), Seed et al. (1986), Yimer (2010), etc., also concluded that the influence of relative density is relatively small when compared to the influence of vertical stress on the dynamic properties of the soil. It can be understood from the figures 5.14 to 5.16 that the shear modulus degradation curves are of concave shape at lower vertical stresses (100 kPa) and of convex shape at higher vertical stresses (400 kPa). Similarly, damping ratio variation is somewhat linear at lower vertical stress of 100 kPa and the damping ratio degradation curve is concave at higher vertical stress of 400 kPa as shown in the figures 5.17 to 5.19.

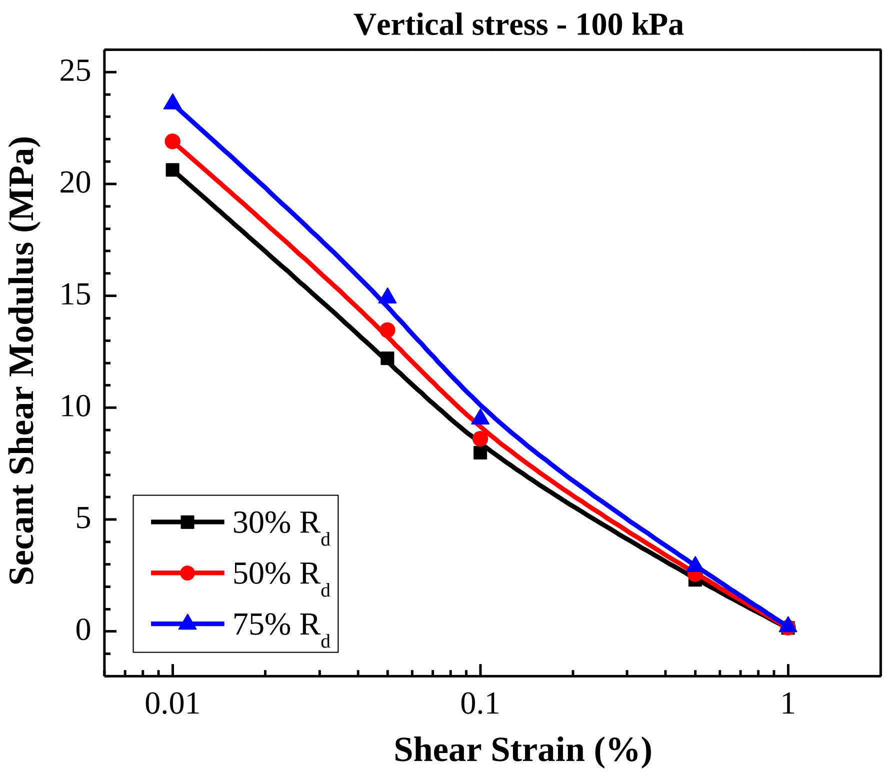


Figure 5.14: Influence of relative density on secant shear modulus at vertical stress of 100 kPa

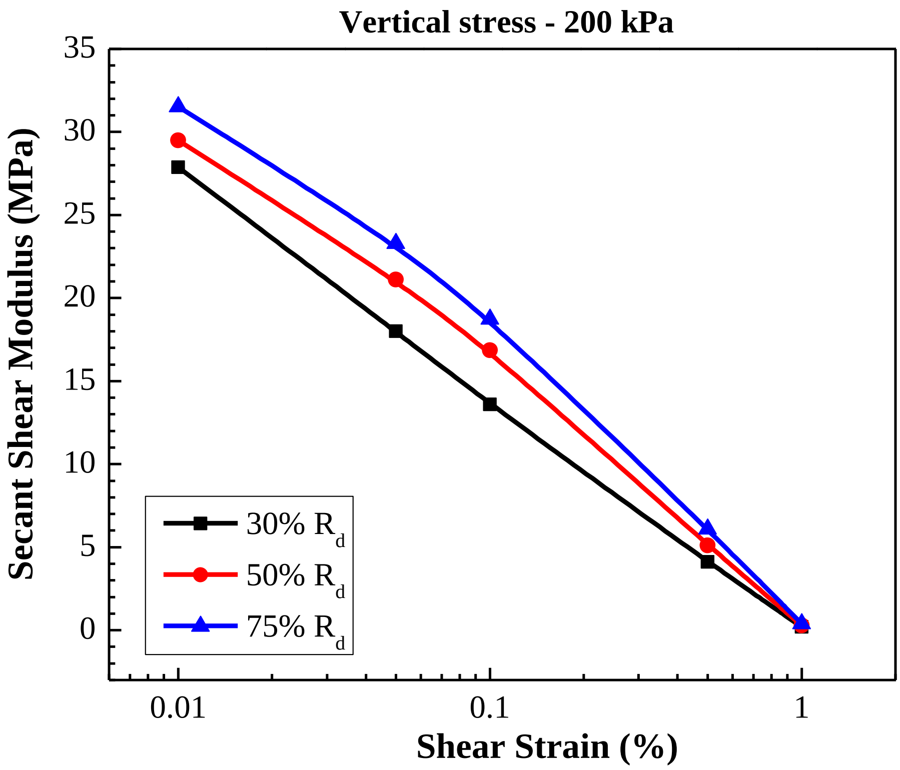


Figure 5.15: Influence of relative density on secant shear modulus at vertical stress = 200 kPa

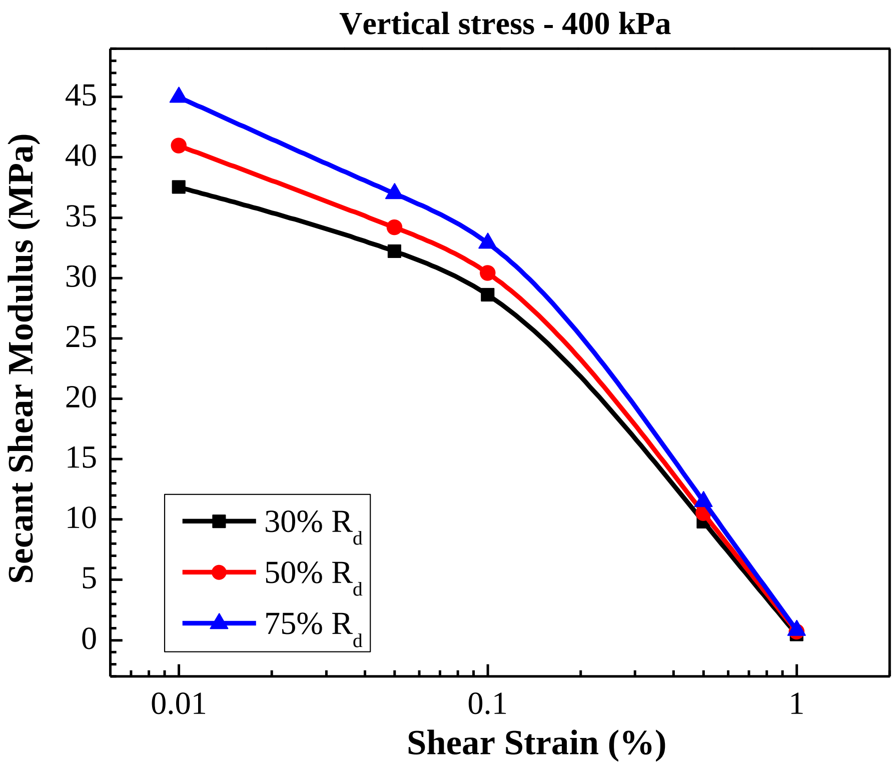


Figure 5.16: Influence of relative density on secant shear modulus at vertical stress = 400 kPa

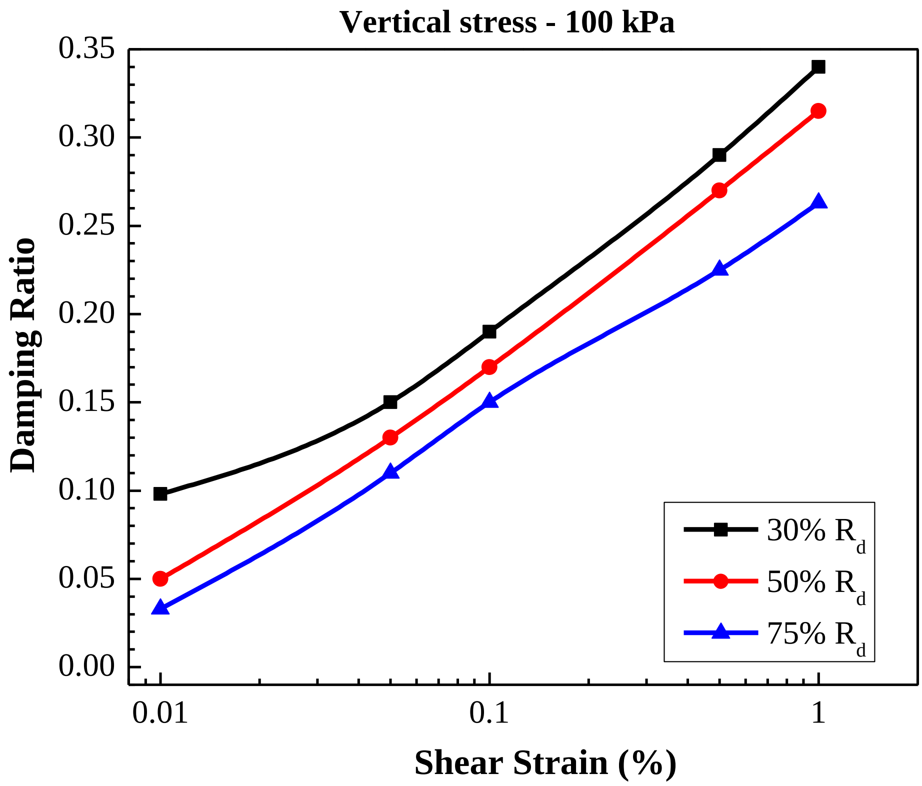


Figure 5.17: Influence of relative density on damping ratio at vertical stress = 100 kPa

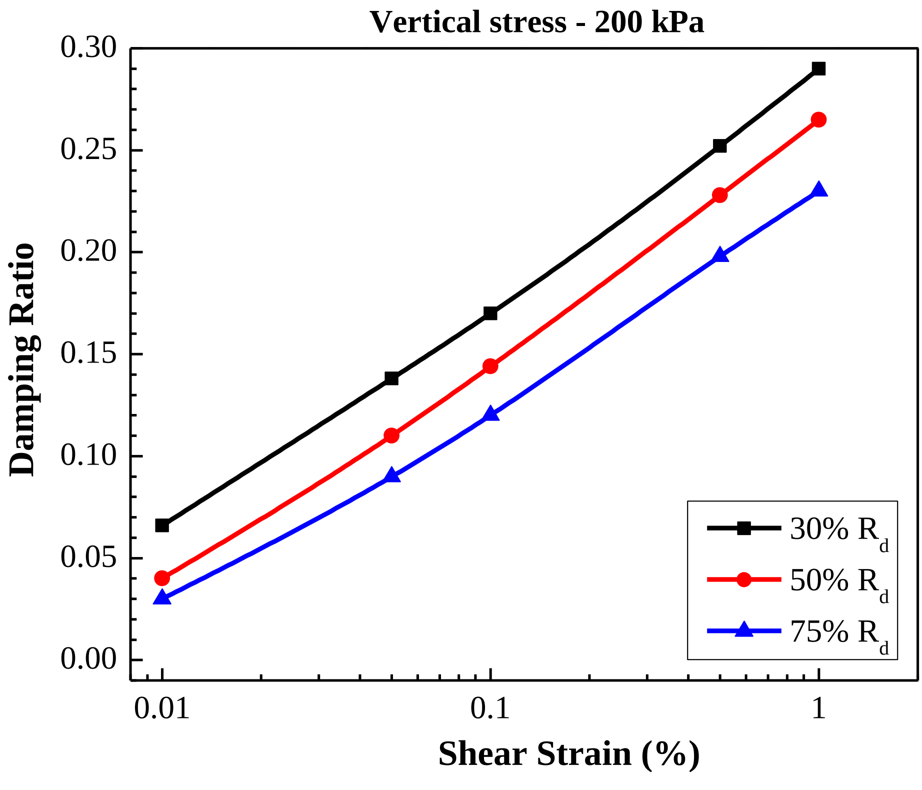


Figure 5.18: Influence of relative density on damping ratio at vertical stress = 200 kPa

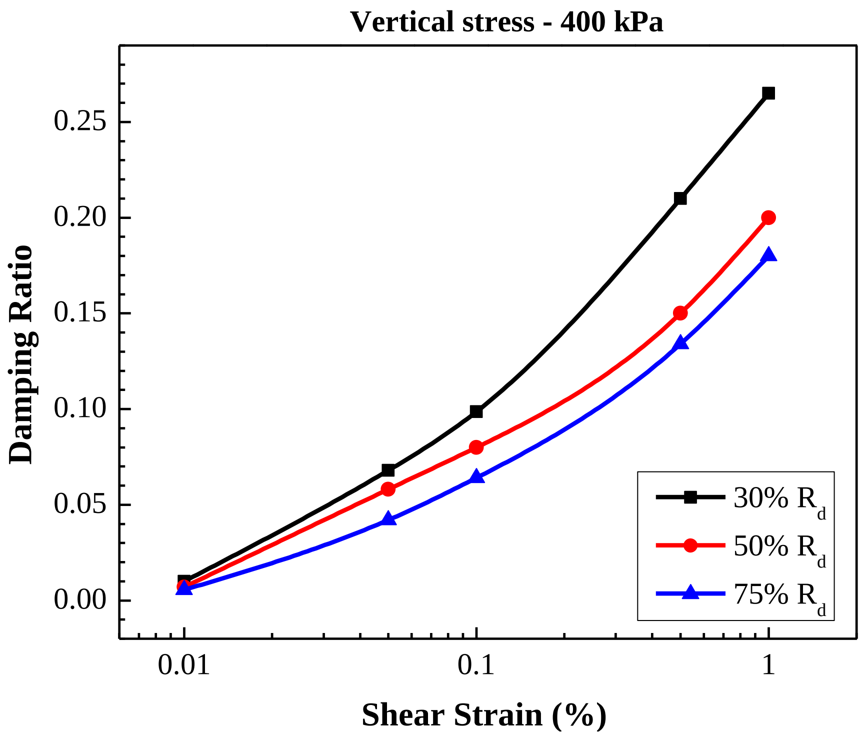


Figure 5.19: Influence of relative density on damping ratio at vertical stress = 400 kPa

5.1.6. Normalized modulus reduction curves and corrected damping ratio curves

According to Dobry and Vucetic (1987), the variation of both secant shear modulus (G_{sec}) and maximum shear modulus (G_{max}) depends more or less on the same parameters, such as vertical stress, void ratio or relative density, and over-consolidation ratio for clays (OCR). Therefore, it is convenient to present the reduction of G_{sec} in the normalized form with respect to G_{max}, i.e., in the G_{sec}/G_{max} versus shear strain format.

Mallick and Baidya (2014) proposed an equation (eq. 5.1) to determine G_{max} of Ennore sand (commercial name of I.S. sand) whose effective particle size is 0.34 mm. However, figure 5.6 conveys that the influence of grain size on shear modulus is minimal. Therefore, the Mallick and Baidya (2014) equation was used to calculate the G_{max} of I.S. sand.

$$G_{max} = 9280\left[\frac{(2.626-e)^2}{(1+e)}\right]\left(\sigma_0^{'}\right)^{0.4} \tag{5.1}$$

Where G_{max} = maximum shear modulus in kPa

e = void ratio of sand

$\sigma_0^{'}$ = effective confining pressure in kPa = $\dfrac{\sigma_v^{'} + 2\sigma_h^{'}}{3}$

($\sigma_v^{'}$ = effective vertical stress in kPa and $\sigma_h^{'}$ = effective horizontal stress in kPa)

$\sigma_h{}' = K_0\ \sigma_v{}'$

Where K_0 is the lateral earth pressure coefficient at rest and can be considered as 0.5 for all practical purposes.

Therefore, $\sigma_0{}' = \frac{2}{3}\sigma_v{}'$

Table 5.1 shows the values of G_{max} for various relative densities/void ratios and vertical stresses. The normalized shear modulus degradation curves are developed at various relative densities (30%, 50% and 75%) and at various vertical stresses (100 kPa, 200 kPa and 400 kPa) using the obtained G_{sec} values and computed G_{max} values. Figures 5.20 to 5.22 show the normalized shear modulus degradation curves for relative densities of 30%, 50% and 75% respectively at vertical stresses of 100 kPa, 200 kPa and 400 kPa. Figures 5.23 to 5.25 show the normalized shear modulus degradation curves for vertical stresses of 100 kPa, 200 kPa and 400 kPa respectively at relative densities of 30%, 50% and 75%.

Table 5.1: The values of G_{max} for various relative densities/void ratios and vertical stresses.

Relative Density (%)	Void Ratio	G_{max} (MPa)		
		$\sigma_v{}'$ = 100 kPa	$\sigma_v{}'$ = 200 kPa	$\sigma_v{}'$ = 400 kPa
30	0.61	125.68	165.83	218.82
50	0.59	129.801	171.25	226
75	0.56	136.22	179.74	237.18

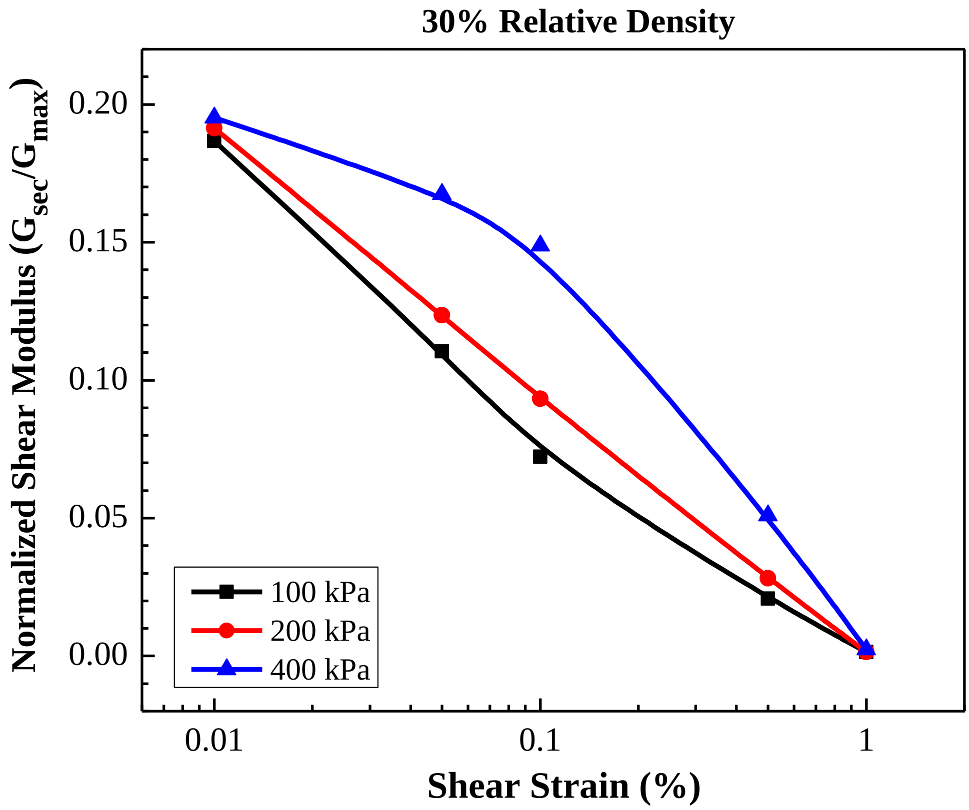


Figure 5.20: Variation of normalized shear modulus with shear strain at R_d = 30%

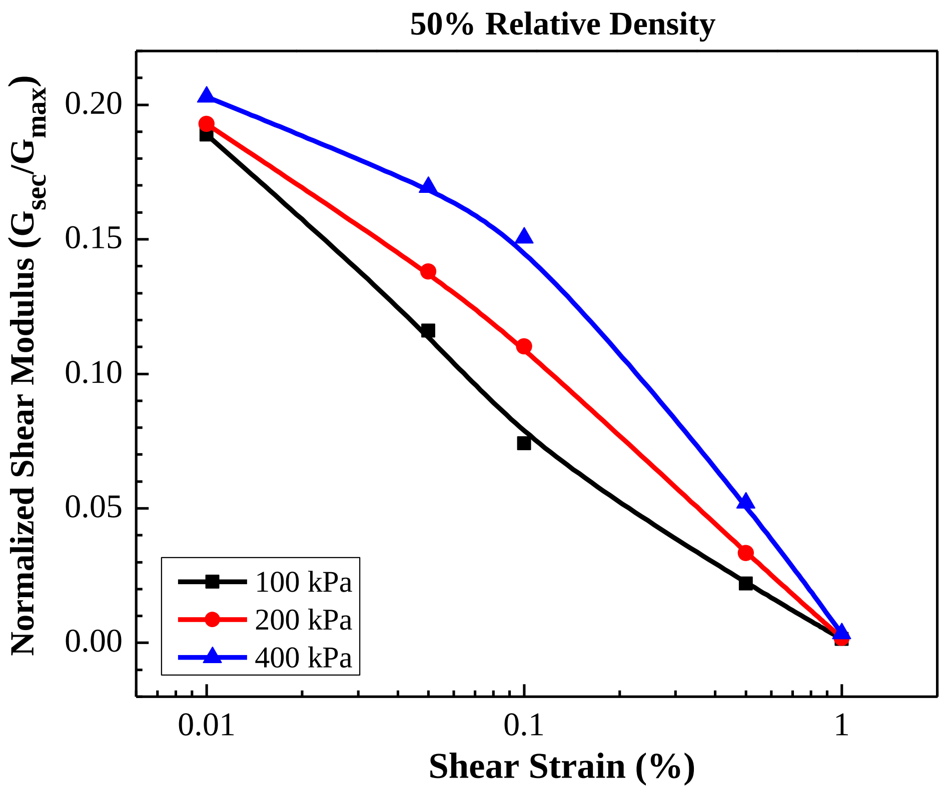


Figure 5.21: Variation of normalized shear modulus with shear strain at R_d = 50%

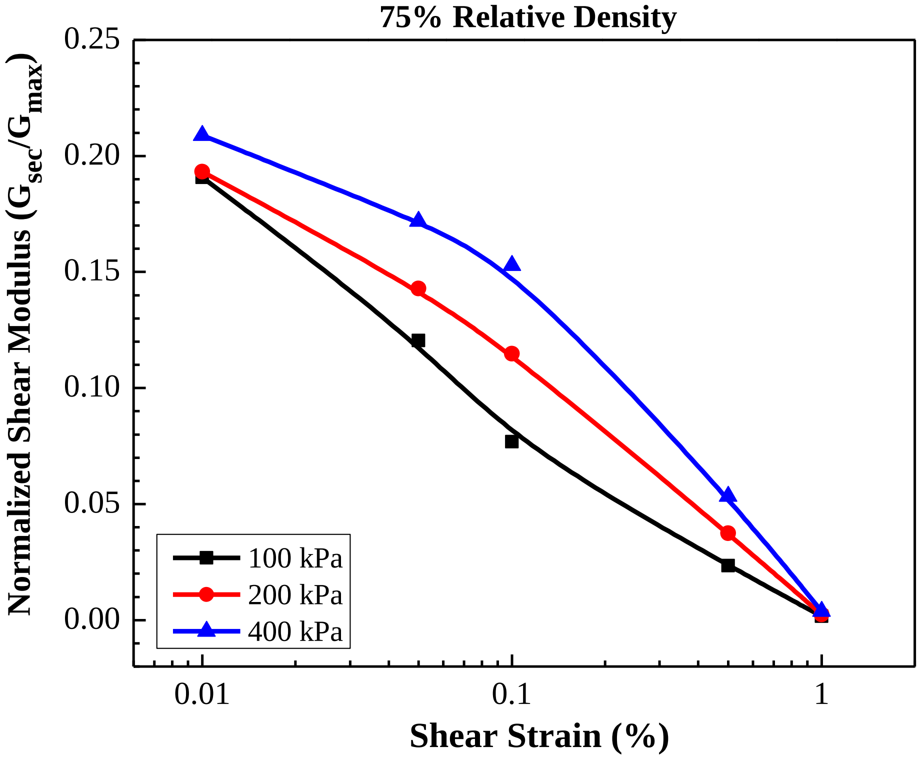


Figure 5.22: Variation of normalized shear modulus with shear strain at R_d = 75%

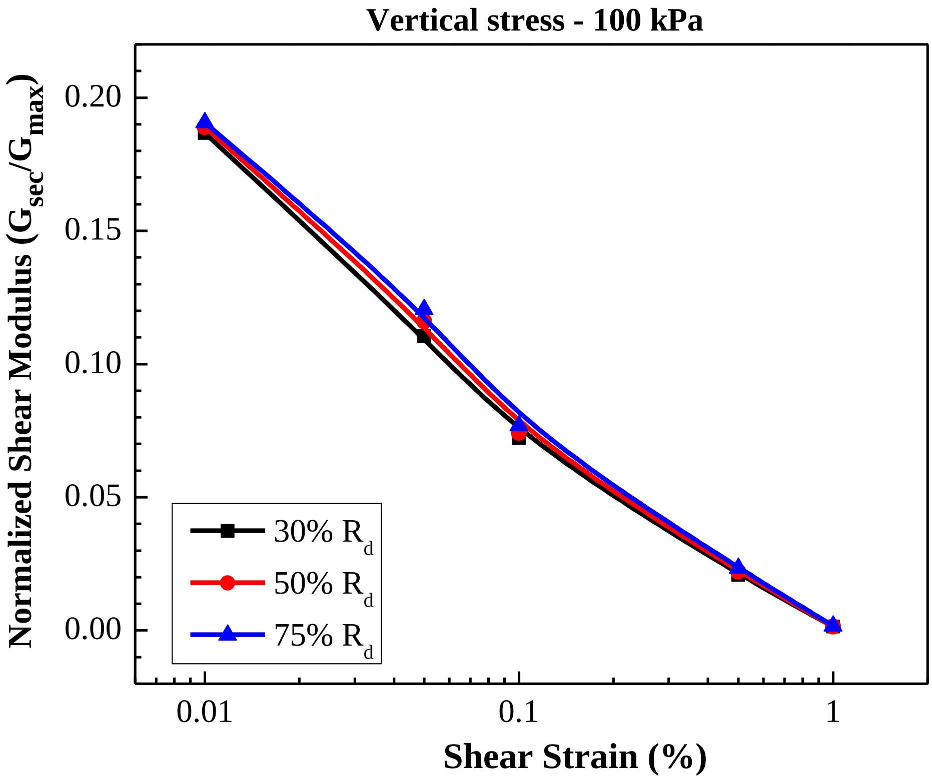


Figure 5.23: Variation of normalized shear modulus with shear strain at vertical stress of 100 kPa

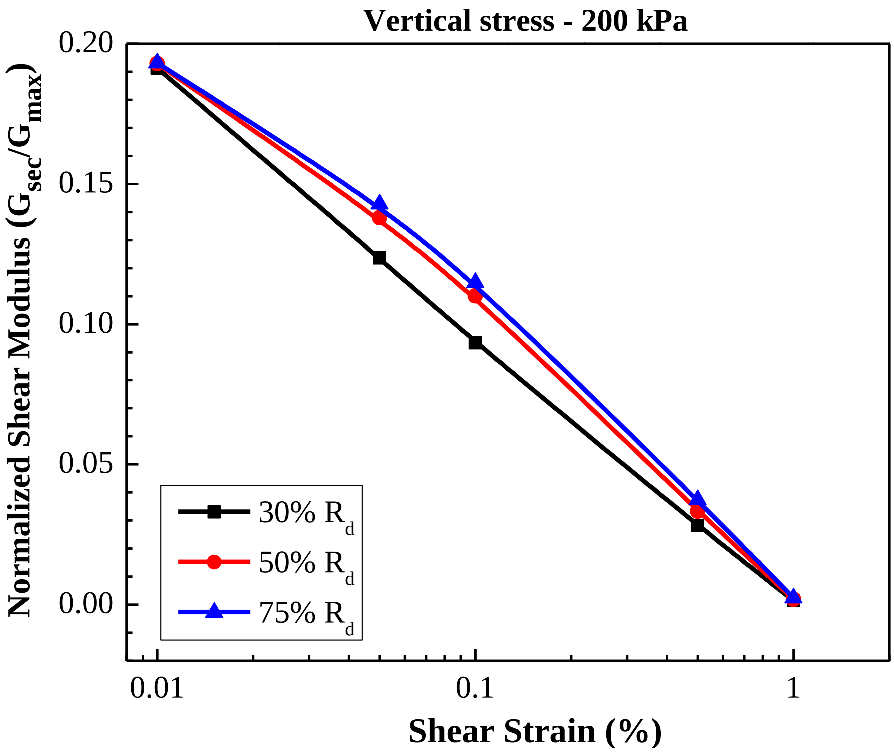


Figure 5.24: Variation of normalized shear modulus with shear strain at vertical stress of 200 kPa

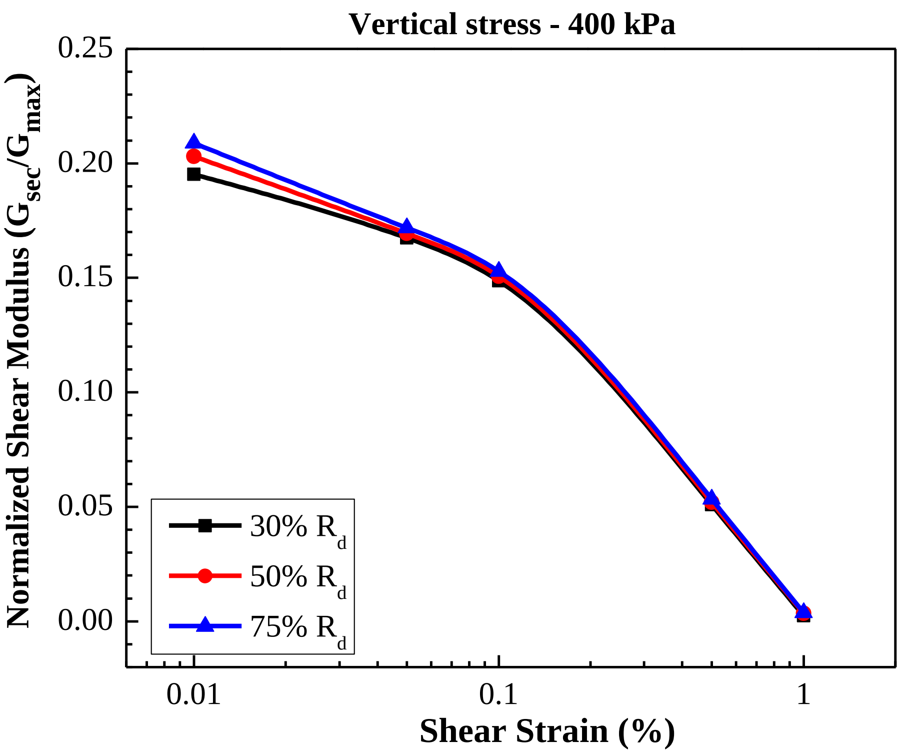


Figure 5.25: Variation of normalized shear modulus with shear strain at vertical stress of 400 kPa

The variation of normalized shear modulus with shear strain at various relative densities and vertical stresses is similar to variation of secant shear modulus with shear strain and well explained in the previous sections. From the figures 5.20 to 5.25 it is clear that the influence of vertical stress is higher compared to the influence of relative density on the dynamic properties of I.S. sand.

5.1.7. Comparison with Literature

Yimer (2010) determined the shear modulus and damping ratio of dry sand collected from Koka town, Ethiopia using cyclic simple shear testing. Yimer (2010) prepared sand specimens of size 70 mm diameter and 20 mm height at various relative densities and tested at various vertical stresses maintaining a frequency of 1 Hz. The secant shear modulus and damping ratio values obtained in this study are compared with the outcomes of Yimer (2010) as shown in figures 5.26 and 5.27, where the Yimer (2010) curves are for 40% relative density and vertical stresses ranging from 100 kPa to 400 kPa. The values obtained in this study are quite less than the values reported by Yimer (2010) but following the similar trend.

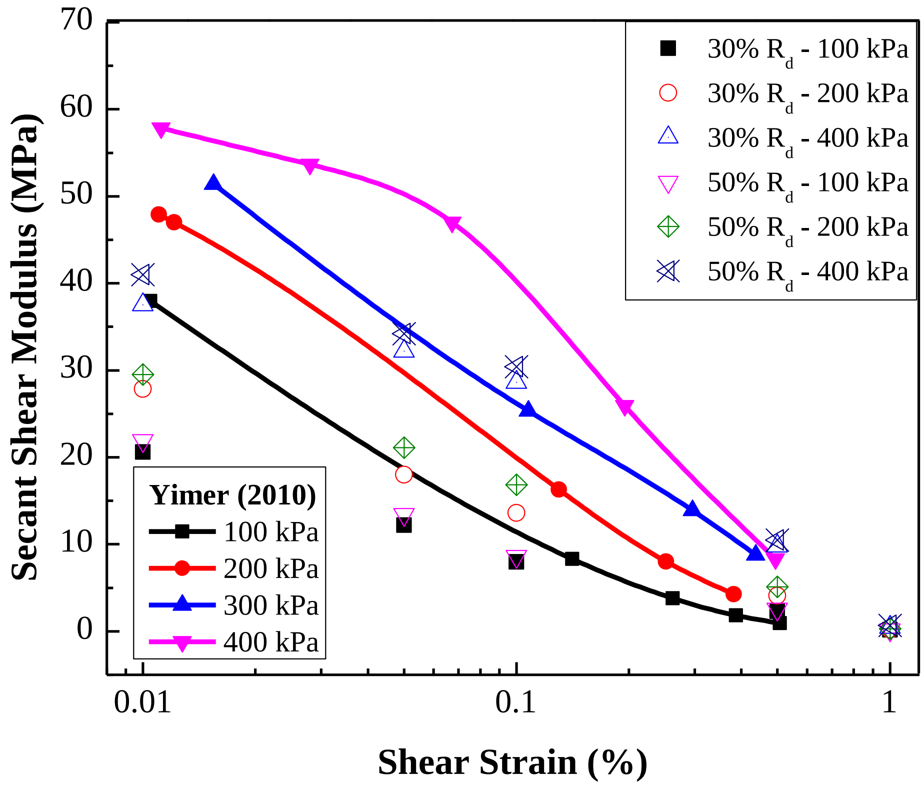


Figure 5.26: Validation of secant shear modulus with the results of Yimer (2010)

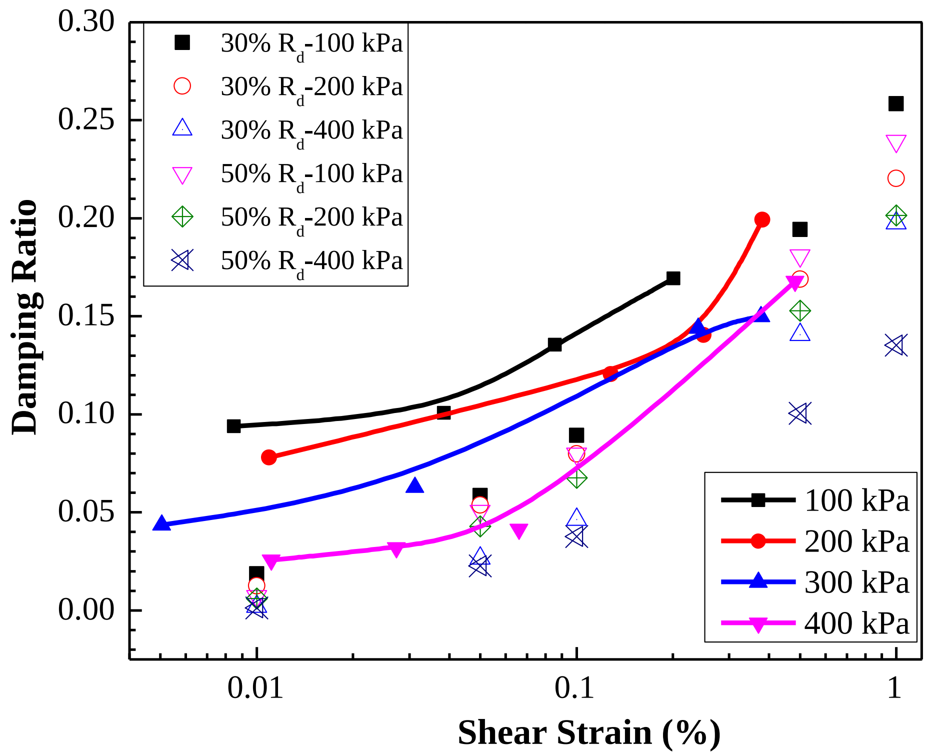


Figure 5.27: Validation of damping ratio with the results of Yimer (2010)

Seed and Idriss (1970) and Seed et al. (1986) proposed the range for normalized shear modulus and the damping ratio of sands respectively over a wide range of shear strains considering the studies of several researchers. Figures 5.28 and 5.29 show the overlying of results obtained in this study with the range of normalized shear modulus and damping ratio proposed by Seed and Idriss (1970) and Seed et al. (1986) respectively. At the shear strains of magnitude in between 0.01% to 0.1%, the values obtained in this study are falling very far away from the lower limit curve and values obtained for shear strain magnitude >0.1% are lying near to the lower limit curve. Laird and Stokoe (1993) performed experiments on sand samples in the laboratory at confining pressures of up to 5 MPa and concluded that, for the highly confined material, both the shear modulus and damping ratio values plot considerably outside the range proposed by Seed and Idriss (1970) and Seed et al. (1986) respectively. Therefore, at very high confining pressures, the use of the Seed and Idriss (1970) curves for cohesionless soils in dynamic response analyses would result in overestimating the capacity of soil to dissipate energy (Laird and Stokoe 1993).

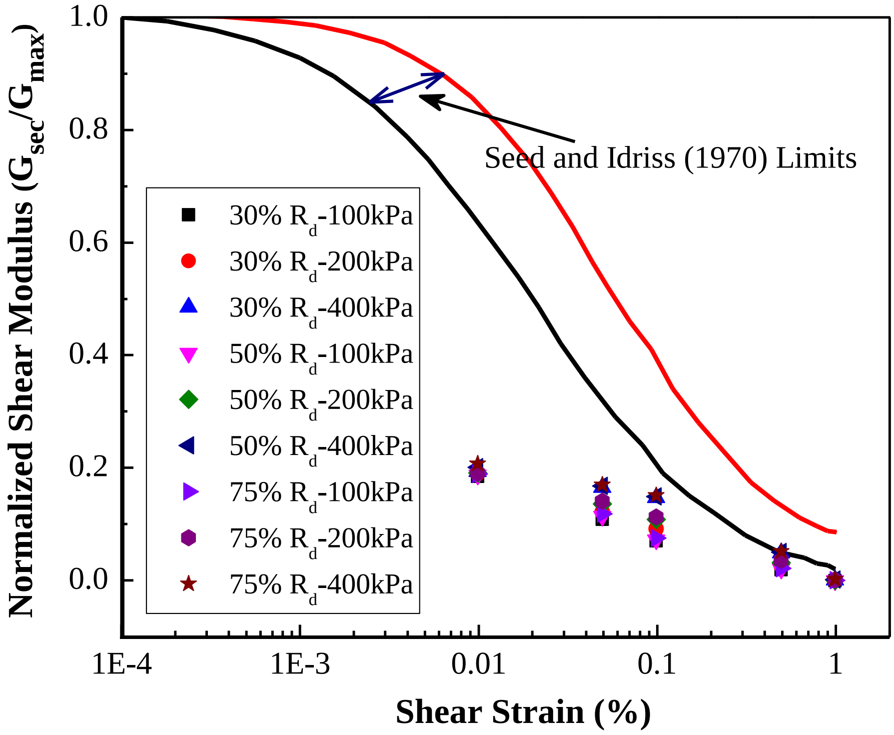


Figure 5.28: Validation of normalized shear modulus with the results of Seed and Idriss (1970)

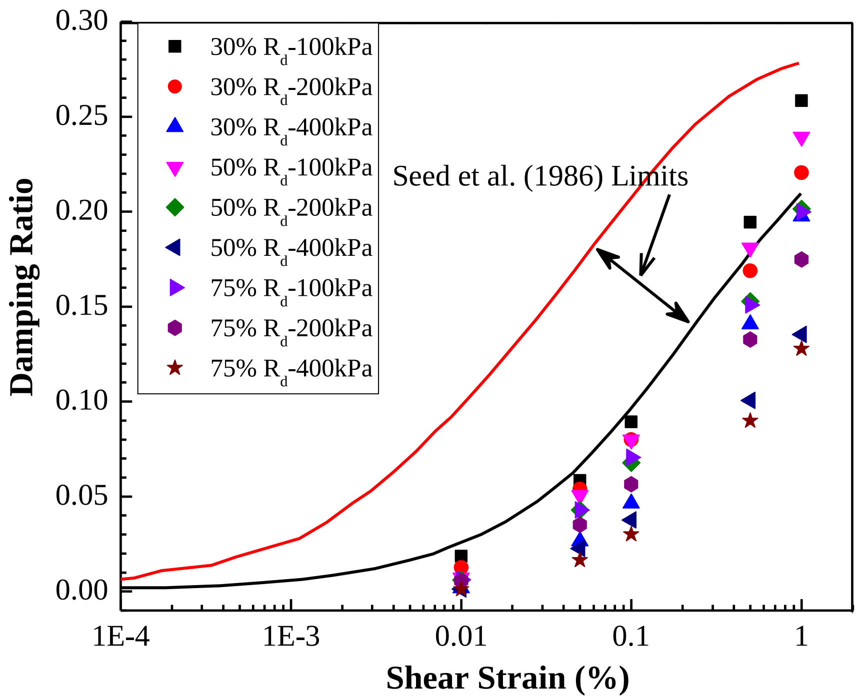


Figure 5.29: Validation of damping ratio with the results of Seed et al. (1986)

5.1.8. Relationship between normalized shear modulus and damping ratio

Many researchers (Hardin and Drnevich (1972), Tatsuoka et al. (1978), Ishibashi (1981) and Zhang and Aggour (1996)) proposed that the damping ratio could be expressed as a function of shear modulus by performing extensive studies on sandy soils. In this study, using the available data, authors tried to develop a correlation between damping ratio and normalized shear modulus by fitting a linear relationship between damping ratio (D) and $(G_{sec}/G_{max})^b$, where 'b' is the exponent which is varied till a linear relationship exists between them. Equation 5.2 shows the developed correlation and the figure 5.30 shows the correlation developed between the normalized shear modulus and damping ratio with a regression coefficient of 0.80.

$$D = 0.2831 - 0.6523 \times \left(\frac{G_{sec}}{G_{max}} \right)^{0.65} \tag{5.2}$$

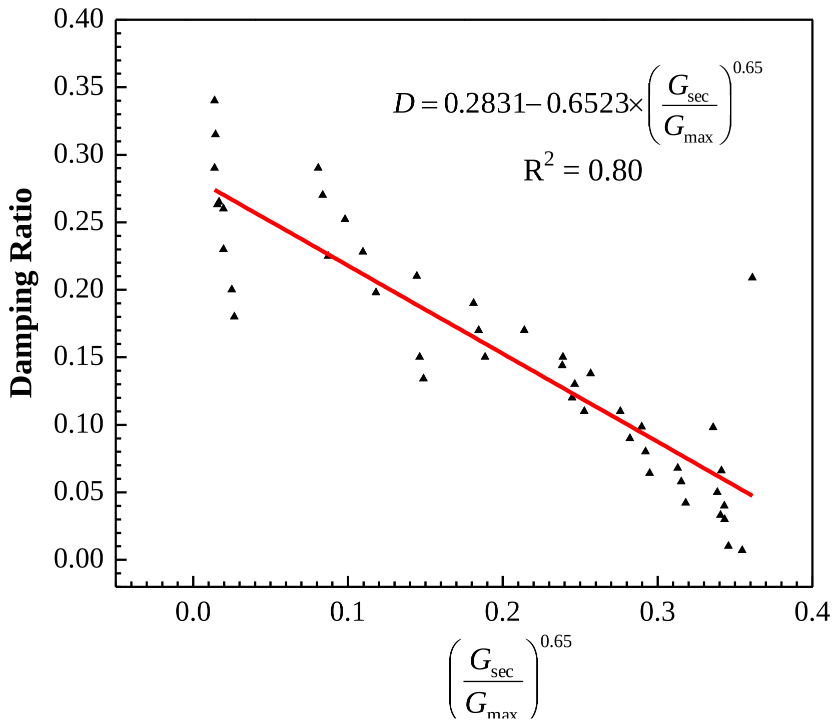


Figure 5.30: Correlation between normalized shear modulus and damping ratio of I.S. sand

Shear modulus can also be determined from the field tests like standard penetration test (SPT) using the existing correlations between the field SPT values and shear modulus developed by researchers like Ohta et al. (1972) and etc. Using the developed correlation in this study, the damping ratio of I.S. sand can be calculated from the field determined shear modulus. However, the developed correlation is applicable only for the range of shear strains and vertical stresses considered in the study.

5.2. Ramagundam Bottom Ash

5.2.1. Influence of number of cycles

The bottom ash specimen was subjected to 50 cycles of sinusoidal loading to determine the influence of number of cycles on secant shear modulus and damping ratio of the bottom ash. With the increase in the number of cycles, hysteresis loop is becoming flatter as shown in figure 5.31. The progressive flattening of the shear stress versus shear strain curve conveys the decrease in peak load as the number of cycles increases. From figure 5.31, it can be concluded that secant shear modulus, defined as the slope of a line through the end points of the hysteresis loop, decreases with the increase in number of cycles as shown in figure 5.32. Figure 5.33 shows the influence on number of cycles on the damping ratio of bottom ash. With the increase in number of cycles, the damping ratio more or less remains constant in contrast to secant shear modulus.

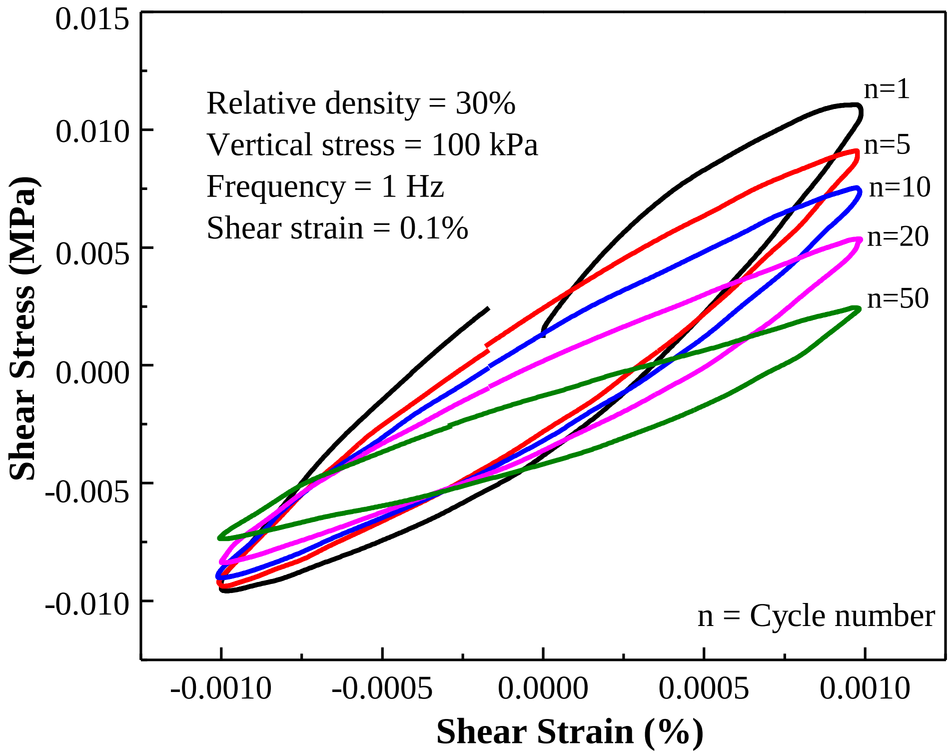


Figure 5.31: Influence of number of cycles on hysteresis loop

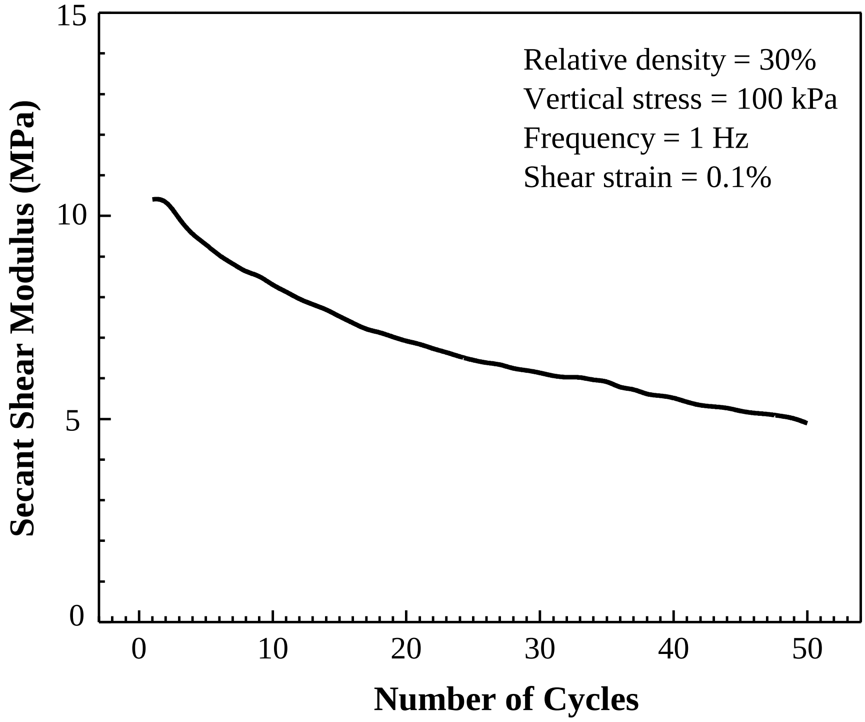


Figure 5.32: Influence of number of cycles on secant shear modulus

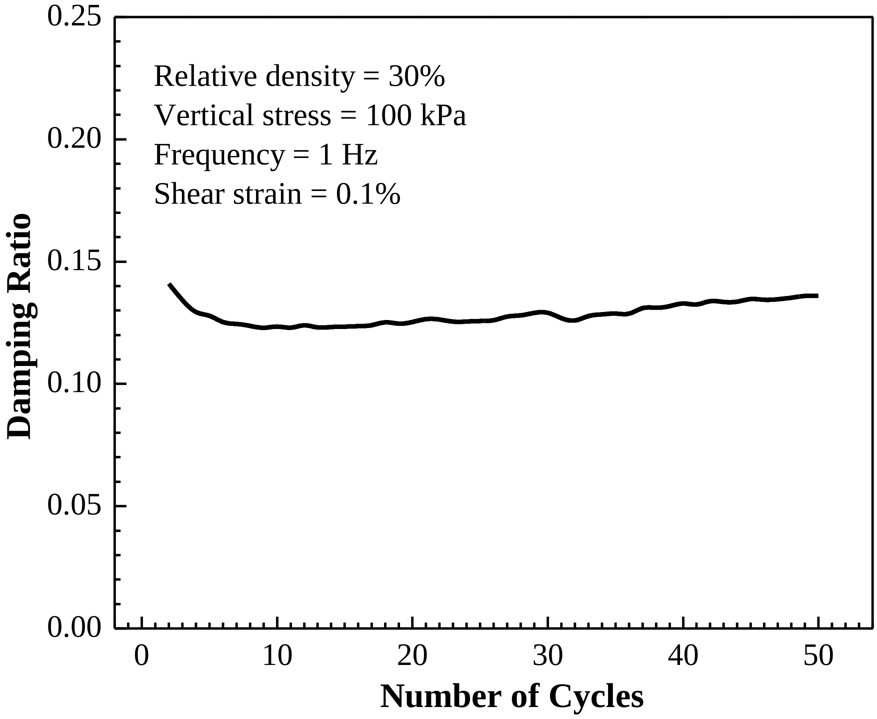


Figure 5.33: Influence of number of cycles on damping ratio

In order to determine the factors influencing the dynamic properties of soils, Hardin and Drnevich (1972) and Dobry and Vucetic (1987) performed extensive studies on normally consolidated and moderately consolidated soils and concluded that shear modulus will decrease with increase in number of loading cycles for undrained conditions and the damping ratio will not be influenced significantly by the number of loading cycles. Therefore, the results obtained in this study are in good agreement with the outcomes of Hardin and Drnevich (1972a and 1972b) and Dobry and Vucetic (1987). Figures 5.31 to 5.33 are shown for fixed values of relative density equal to 30%, vertical stress equal to 100 kPa, frequency equal to 1 Hz and shear strain of 0.1%. Similar behavior can also be shown for other cases, i.e., when tested at relative densities of 50% and 75%, vertical stresses of 200 kPa and 400 kPa at various shear strains of magnitude 0.01%, 0.05%, 0.5% and 1%, maintaining 1 Hz frequency.

Das and Ramana (1993) suggested to consider the values of 5th cycle as the representative values of shear modulus and damping ratio for all practical purposes as the number of significant cycles would be less than 20 in most seismic events. Therefore, secant shear modulus and damping ratio values reported in the following sections corresponds to 5th cyclic loading.

5.2.2. Influence of Frequency

The influence of frequency of loading on the dynamic properties of bottom ash was studied by subjecting the bottom ash specimen to various frequencies ranging from 0.1 Hz to 2 Hz. The specimens were prepared at 50% relative density and subjected to 200 kPa vertical stress and tested at a shear strain magnitude of 0.1%. Figures 5.34 and 5.35 show the variation of secant shear modulus and damping ratio with the frequency respectively. With the increase in frequency from 0.1 Hz to 2 Hz, secant shear modulus of bottom ash more or less remains constant and in contrast, damping ratio is changing considerably. The results of strain controlled cyclic triaxial element tests on dry and saturated soil samples in medium to large shear strain levels by Ravishankar et al. (2005) concluded that the effect of frequency is not significant on shear modulus but has some influence on the damping ratios of the soils for the range of frequencies tested (0.1 Hz to 2Hz). Therefore, the results obtained in this work are in good agreement with the conclusions of Ravishankar et al. (2005).

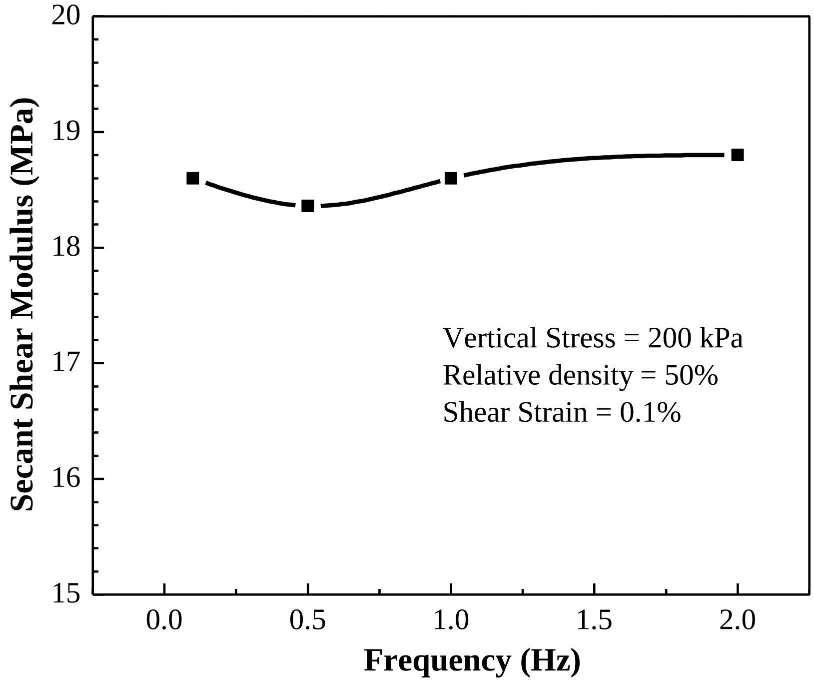


Figure 5.34: Influence of frequency on secant shear modulus

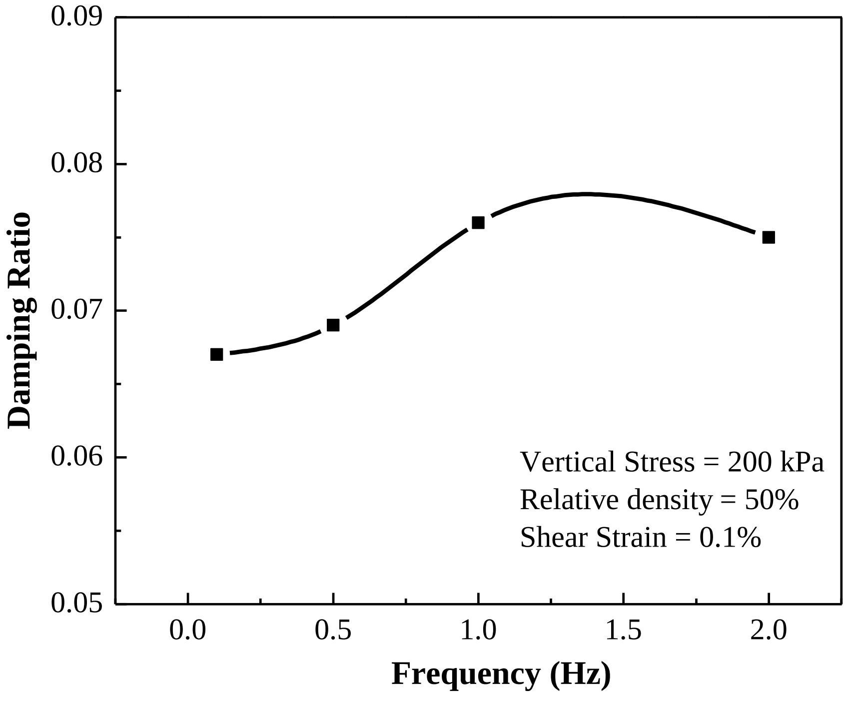


Figure 5.35: Influence of frequency on damping ratio

To determine the influence of vertical stress, relative density, and sample size an average frequency value of 1 Hz was selected for performing the remaining tests on the bottom ash. Therefore, the values reported hereafter corresponds to a frequency of 1 Hz and 5^{th} cyclic loading.

5.2.3. Influence of vertical stress and shear strain

In order to study the influence of vertical stress on the dynamic properties of bottom ash, specimens prepared at relative densities of 30%, 50% and 75% were prepared and tested at various vertical stresses of 100 kPa, 200 kPa and 400 kPa. For a particular relative density and shear strain magnitude, with the increase in vertical stress secant shear modulus increases and damping ratio decreases. Figures 5.36 to 5.38 show the influence of vertical stress on shear modulus for relative densities of 30%, 50% and 75% respectively. Figures 5.39 to 5.41 show the influence of vertical stress on damping ratio for relative densities of 30%, 50% and 75% respectively. The studies of Silver and Seed (1971), Hardin and Drnevich (1972b), Dobry and Vucetic (1987), Ravishankar et al. (2005), Yimer (2010), etc., also concluded that with the increase in vertical stress, shear modulus increases and damping ratio decreases for various tested materials like sand, clay, and gravel at a particular shear strain magnitude.

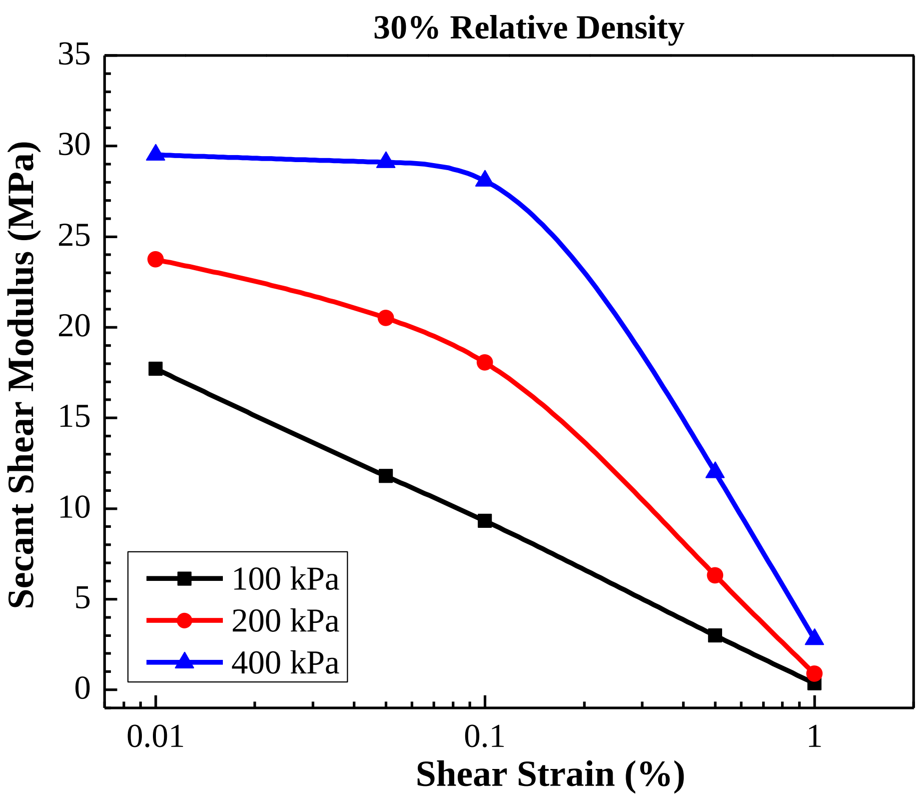


Figure 5.36: Influence of vertical stress on secant shear modulus for R_d = 30%

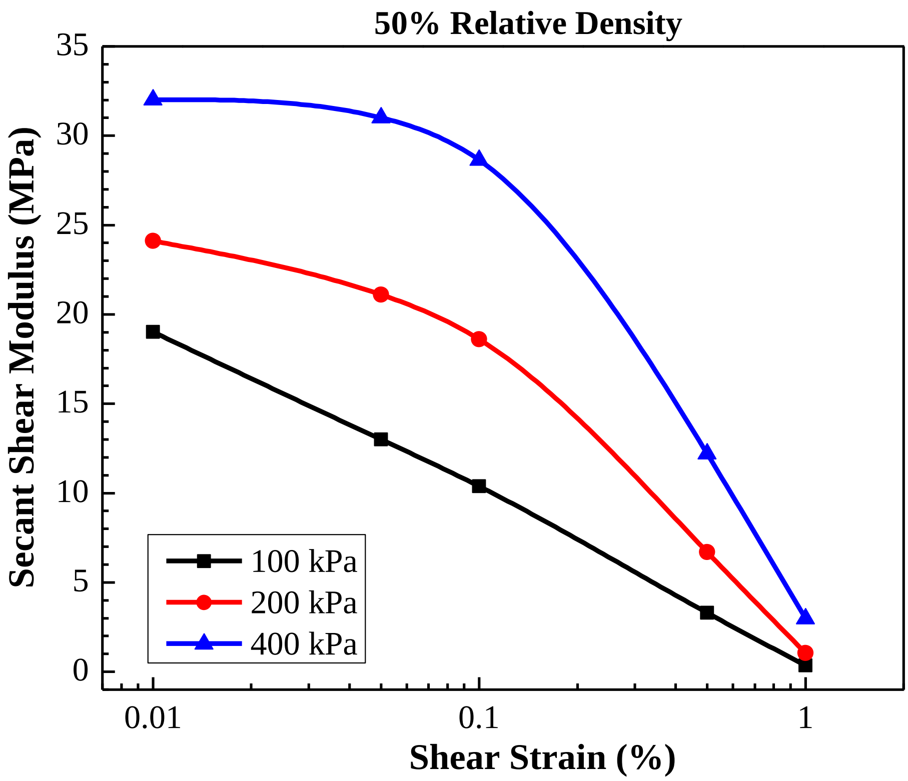


Figure 5.37: Influence of vertical stress on secant shear modulus for R_d = 50%

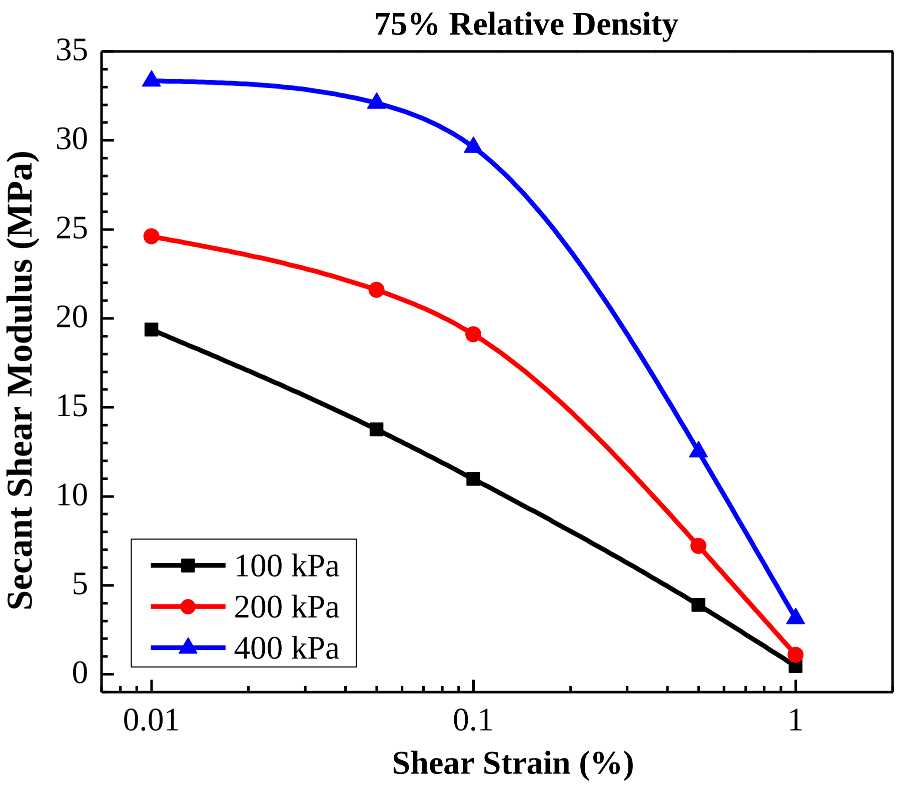


Figure 5.38: Influence of vertical stress on secant shear modulus for R_d = 75%

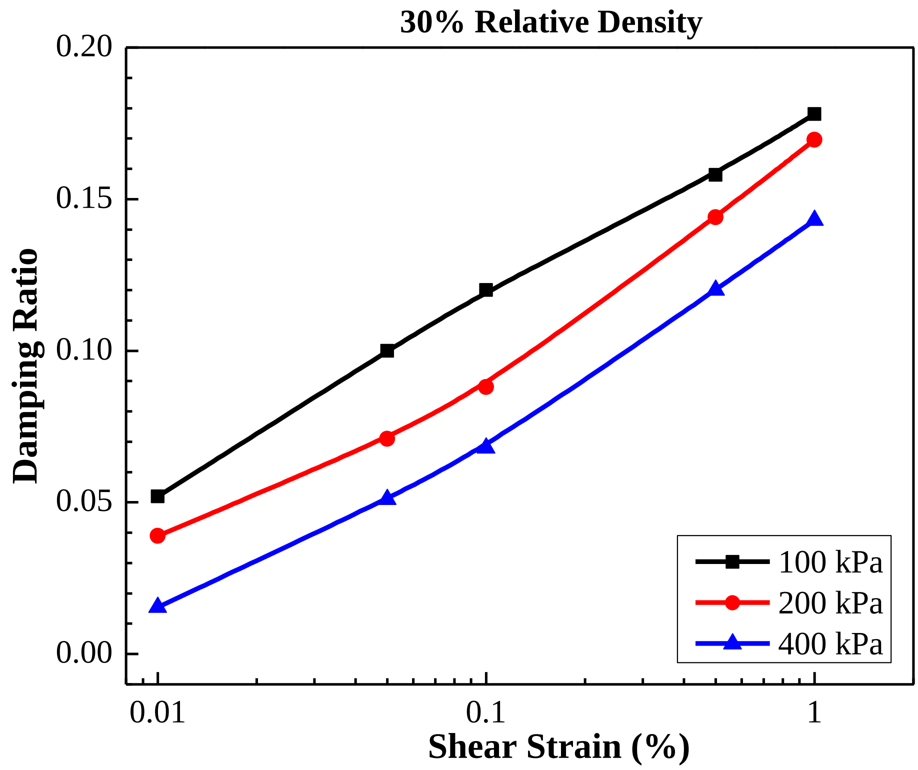


Figure 5.39: Influence of vertical stress on damping ratio for R_d = 30%

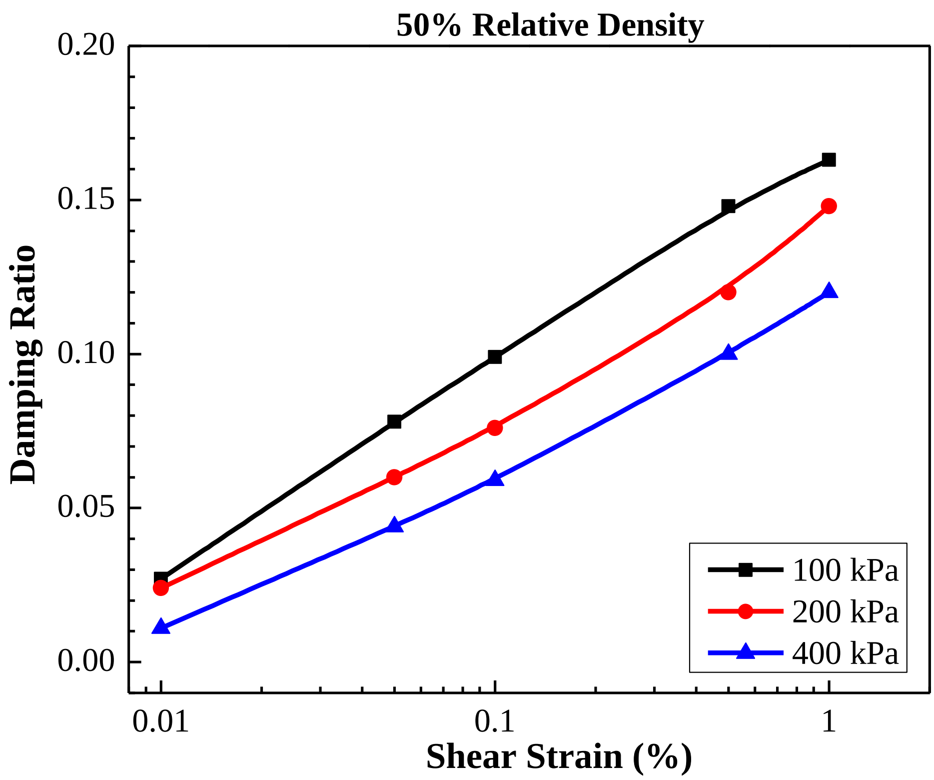


Figure 5.40: Influence of vertical stress on damping ratio for R_d = 50%

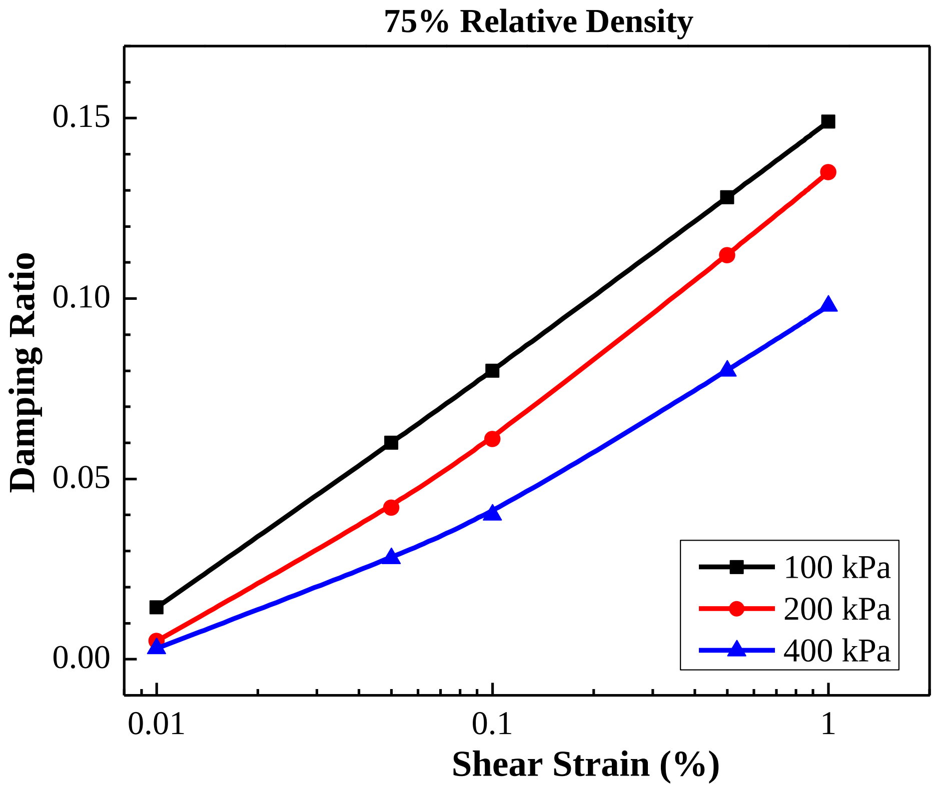


Figure 5.41: Influence of vertical stress on damping for R_d = 75%

From the figures 5.36 to 5.41, it can be concluded that the secant shear modulus degradation curve (shear modulus variation with the shear strain) is attaining a shape of convex curve at higher vertical stresses and possess concave shape at lower vertical stresses. Similarly, damping ratio degradation curve (damping ratio variation with the shear strain) is attaining the shape of concave at higher vertical stresses with showing a linear variation at lower vertical stresses.

It can also be concluded from the figures 5.36 to 5.41 that with the increase in shear strain magnitude, secant shear modulus decreases dramatically and damping ratio increases. Therefore, it can be concluded that shear strain is the most significant factor influencing the dynamic properties of the bottom ash. In addition, with the increase in the magnitude of shear strain, hysteresis loop is attaining the shape or reverse 'S' shape hysteresis loop as shown in figure 5.42. The figure 5.42 is shown for a specimen prepared at 50% relative density and tested at a vertical stress of 200 kPa. Similar behavior was observed even at other relative densities and vertical stresses.

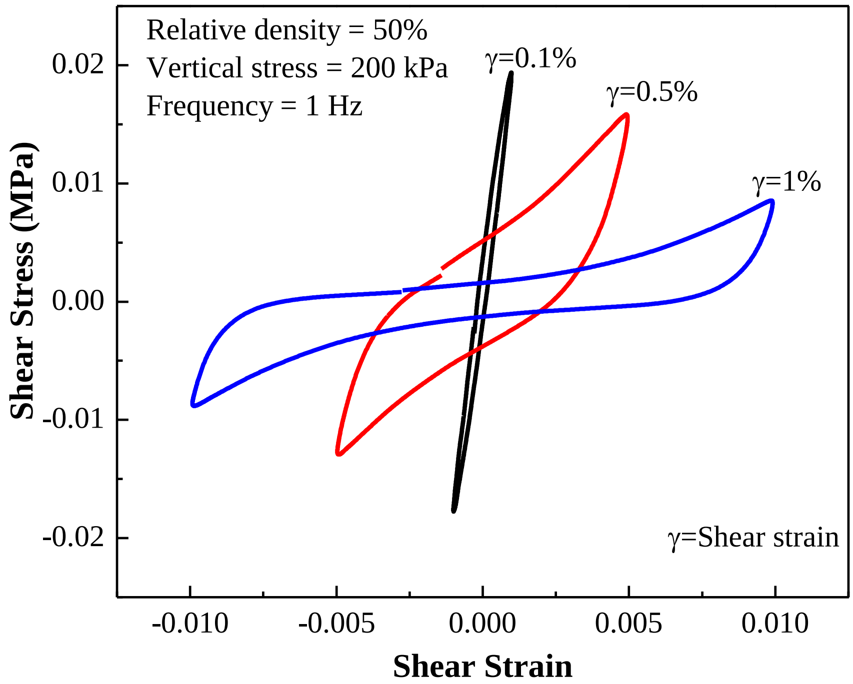


Figure 5.42: Influence of shear strain on hysteresis loop of bottom ash

5.2.4. Influence of Relative Density

In order to study the influence of relative density on the dynamic properties of bottom ash, specimens were prepared at different relative densities of 30%, 50% and 75% and tested at different vertical stresses varying from 100 kPa to 400 kPa. For a particular vertical stress and shear strain magnitude, with the increase in the relative density secant shear modulus increases and damping ratio decreases. Figures 5.43 to 5.45 show the influence of relative density on secant shear modulus for vertical stresses of 100 kPa, 200 kPa and 400 kPa respectively. Figures 5.46 to 5.48 show the influence of relative density on the damping ratio of bottom ash for vertical stresses of 100 kPa, 200 kPa and 400 kPa respectively. Similar to the influence of vertical stress on dynamic properties, with the increase in relative density of sample secant shear modulus increases and damping ratio decreases at a particular vertical stress and shear strain. However, the influence of relative density on dynamic properties of bottom ash, especially on secant shear modulus, is not as much as the vertical stress. The previous studies of Hardin and Drnevich (1972b), Seed et al. (1986), Yimer (2010), etc., also concluded that the influence of relative density is relatively small when compared to the influence of vertical stress on the dynamic properties of the soil. It can be understood from the figures 5.43 to 5.45 that the shear modulus degradation curves are of concave shape at lower vertical stresses (100 kPa) and of convex shape at higher vertical stresses (400 kPa). Similarly, damping ratio variation is somewhat linear at lower vertical stress of 100 kPa and the damping ratio degradation curve is concave at higher vertical stress of 400 kPa as shown in the figures 5.46 to 5.48.

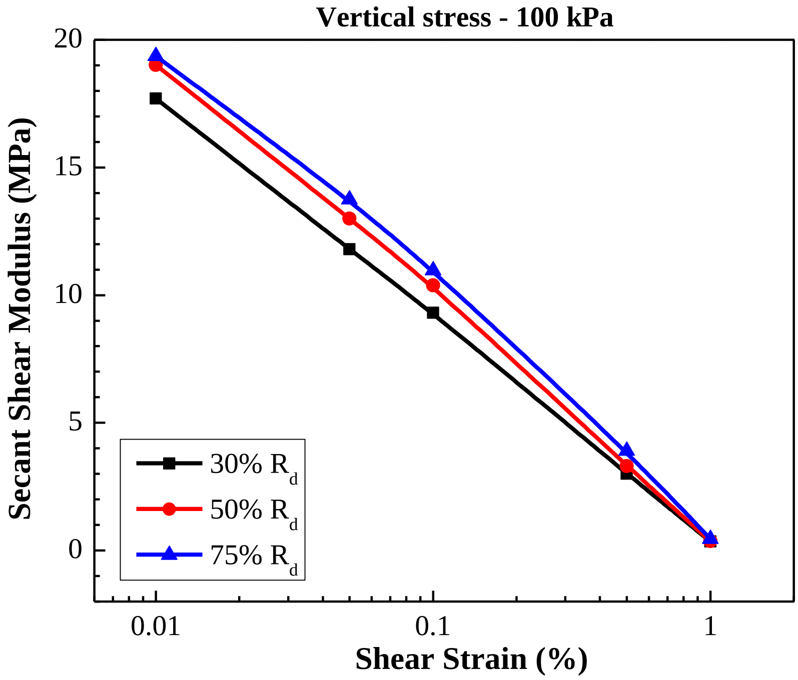


Figure 5.43: Influence of relative density on secant shear modulus at vertical stress = 100 kPa

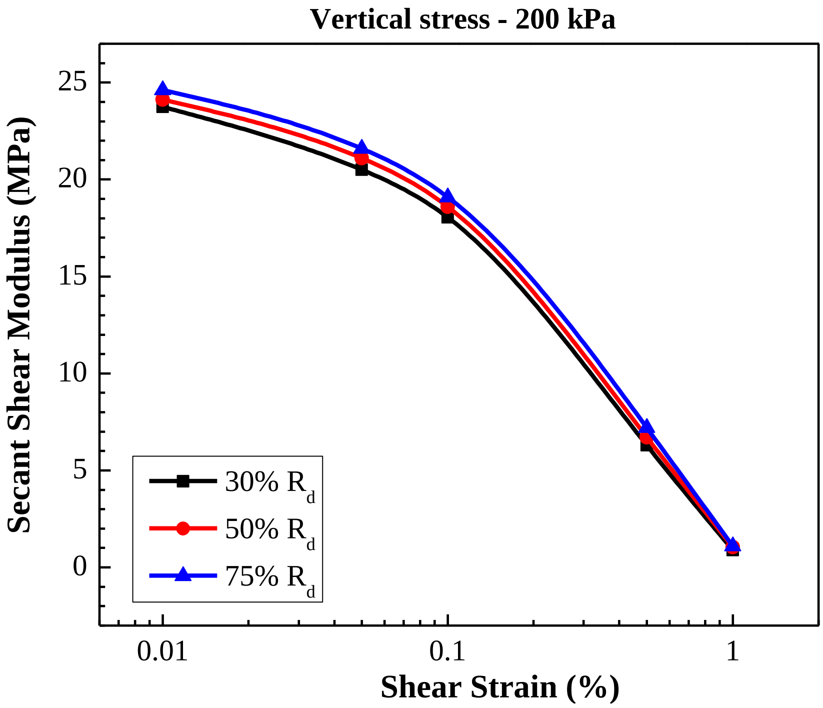


Figure 5.44: Influence of relative density on secant shear modulus at vertical stress = 200 kPa

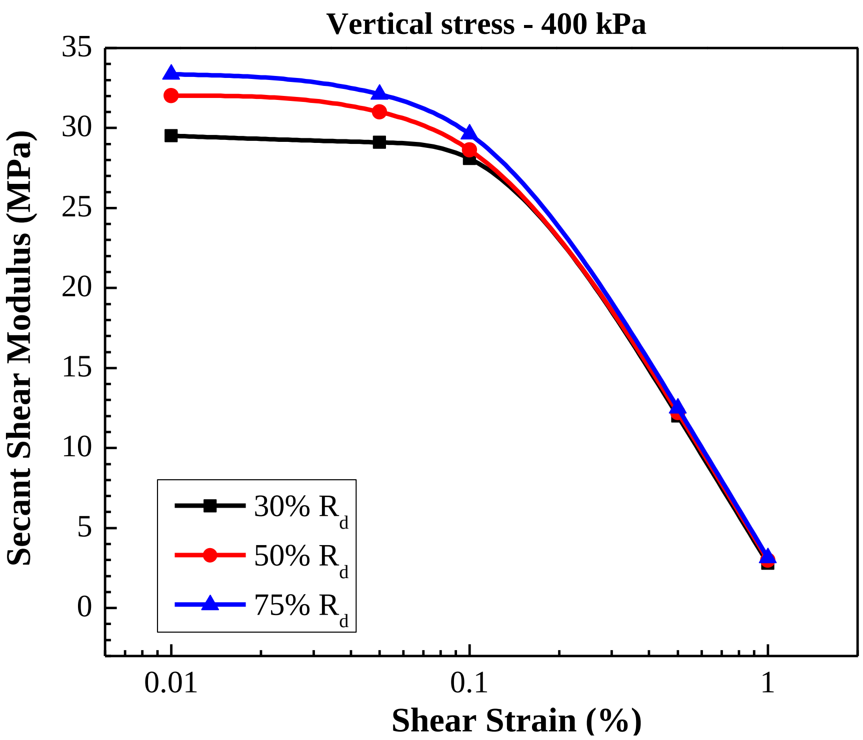


Figure 5.45: Influence of relative density on secant shear modulus at vertical stress = 400 kPa

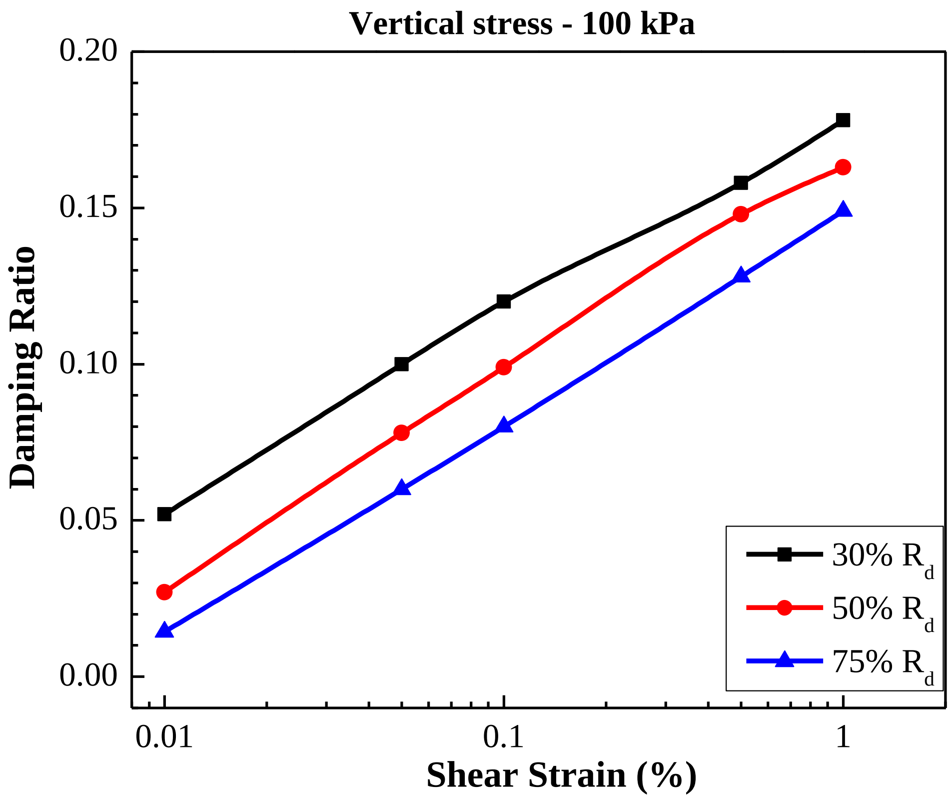


Figure 5.46: Influence of relative density on damping ratio at vertical stress = 100 kPa

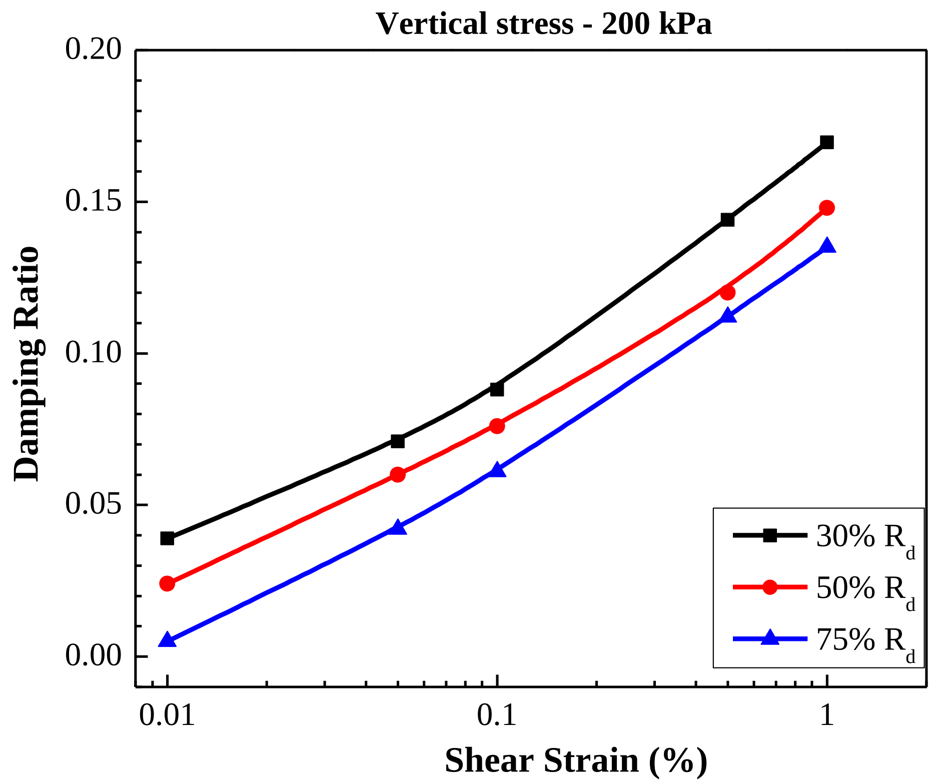


Figure 5.47: Influence of relative density on damping ratio at vertical stress = 200 kPa

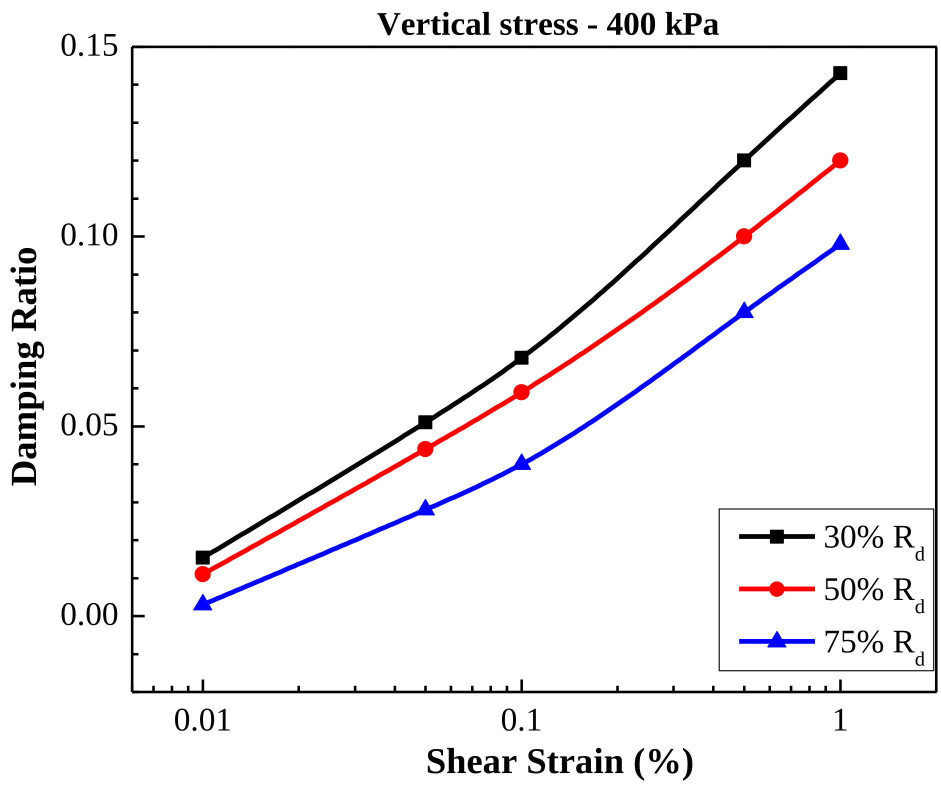


Figure 5.48: Influence of relative density on damping ratio at vertical stress = 400 kPa

5.2.5. Relationship between Secant Shear Modulus and Damping Ratio

Many researchers (Hardin and Drnevich (1972b), Tatsuoka et al. (1978), Ishibashi (1981) and Zhang and Aggour (1996)) proposed that the damping ratio could be expressed as a function of shear modulus by performing extensive studies on sandy soils. In this study, using the available data, authors tried to develop a correlation between damping ratio and normalized shear modulus by fitting a linear relationship between damping ratio (D) and G_{sec}^{b}, where 'b' is the exponent which is varied till a linear relationship exists between them. Equation 5.3 shows the developed correlation and the figure 5.49 shows the correlation developed between the normalized shear modulus and damping ratio with a regression coefficient of 0.85.

$$D = 0.1763 - 0.0214 \times G_{sec}^{0.57} \tag{5.3}$$

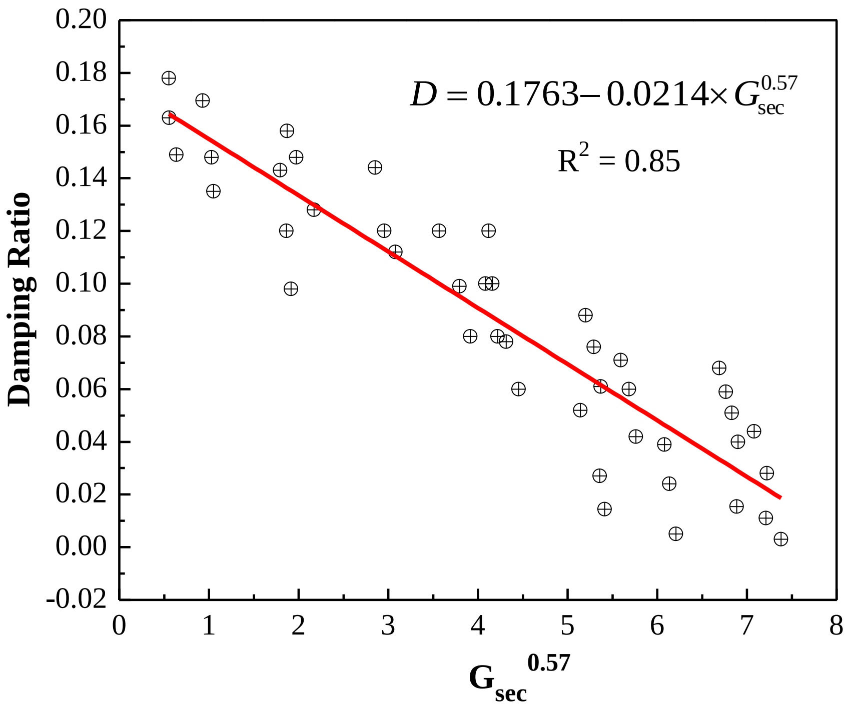


Figure 5.49: Correlation between secant shear modulus and damping ratio of bottom ash

Shear modulus can also be determined from the field tests like standard penetration test (SPT) using the existing correlations between the field SPT values and shear modulus developed by researchers like Ohta et al. (1972) and etc. Using the developed correlation in this study, the damping ratio of bottom ash can be calculated from the field determined shear modulus. However, the developed correlation is applicable only for the range of shear strains and vertical stresses considered in the study.

5.2.6. Influence of Sample Height

The influence of sample height on dynamic properties of bottom ash was studied by considering two sample heights, 25 mm and 20 mm, keeping the diameter of the specimen as constant (70 mm). Figures 5.50 and 5.51 show the influence of sample height on secant shear modulus and the damping ratio of bottom ash respectively. With the reduction in sample height, secant shear modulus is decreasing and damping ratio is increasing at a particular shear strain magnitude. Figures 5.50 and 5.51 also conveys that, with the increase in shear strain magnitude, shear modulus degradation curves for the two sample heights are converging and damping ratio curves for the two sample heights are diverging. Therefore, dynamic properties reported in this study are constrained to the sample of dimensions 70 mm in diameter and 25 mm in height.

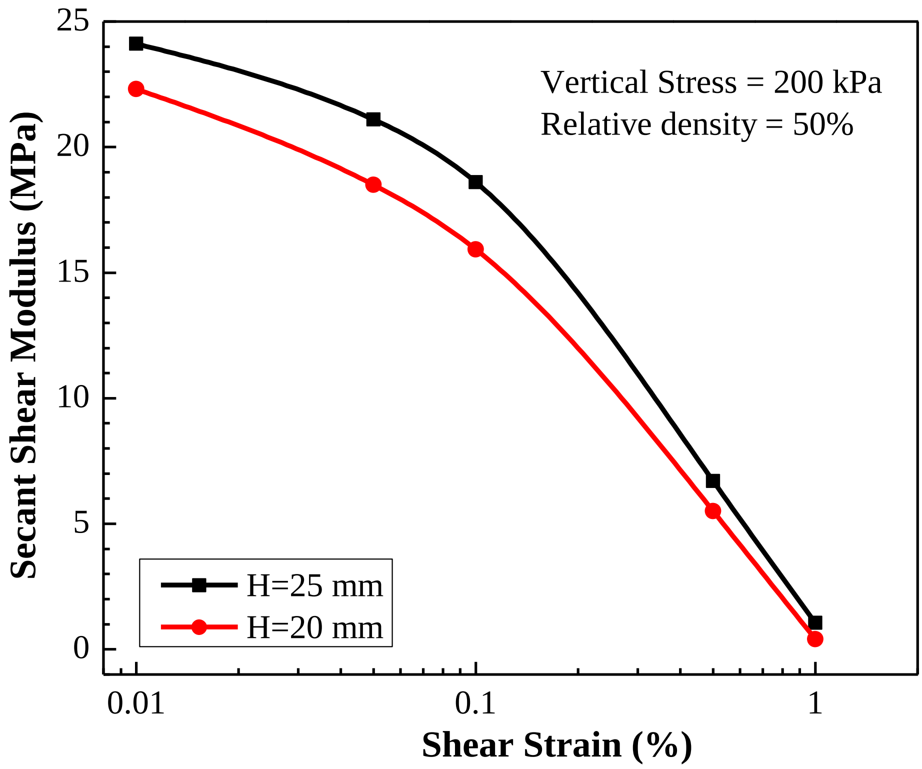


Figure 5.50: Influence of sample height on secant shear modulus

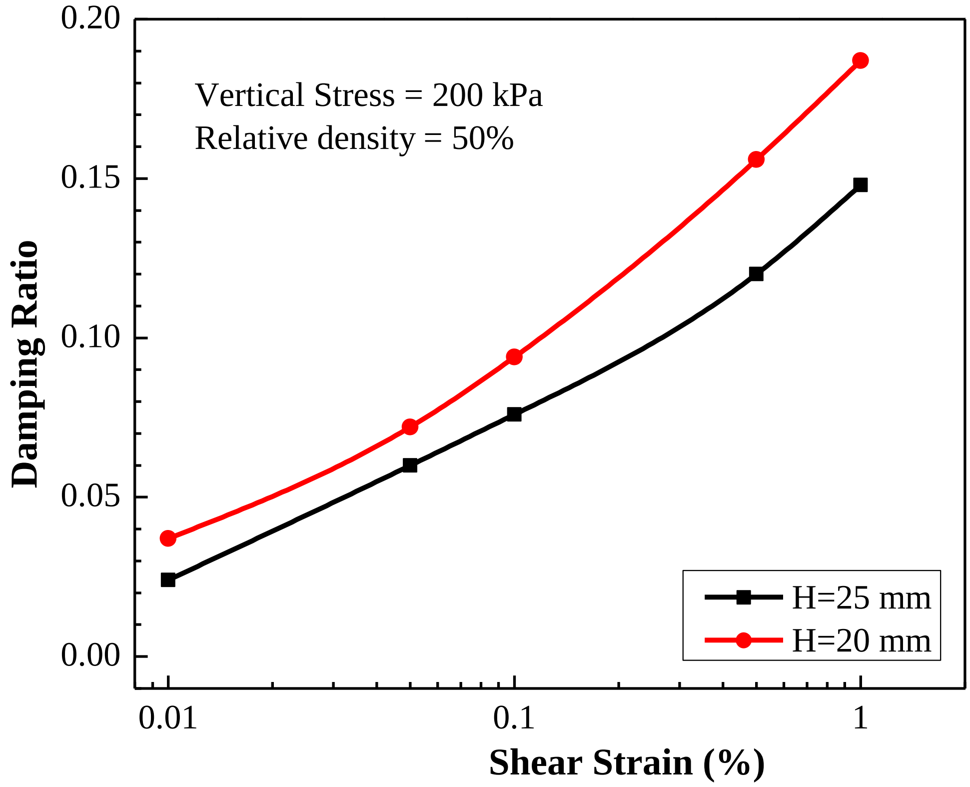


Figure 5.51: Influence of sample height on damping ratio

5.3. Neyveli Fly Ash

5.3.1. Influence of number of cycles

The influence of number of cycles on the dynamic behavior of fly ash was investigated by subjecting the fly ash specimen to 50 cycles of sinusoidal loading. With the increase in the number of cycles, hysteresis loop becomes flatter (figure 5.52). Figure 5.53 shows the variation of secant shear modulus with the number of cycles. It can be understood from figure 5.52, secant shear modulus, identified as the slope of a line through the end points of the hysteresis loop decreases with the number of cycles, as shown in figure 5.53. The progressive flattening of the shear stress versus shear strain curve conveys the decrease in peak load as the number of cycles increases. The percentage decrease in secant shear modulus for the first ten cycles is 36% and the percentage difference between the 11th cycle and 50th cycle is only 24.73%. By conducting triaxial tests on the Hostun sand, Jafarzadeh et al. (2008) also stated that the rate of reduction of shear modulus will be higher for first few cycles. Figure 5.54 shows the influence of number of cycles on the damping ratio. In contrary to the secant shear modulus, with the increase in number of cycles, the damping ratio more or less remains constant. Hardin and Drnevich (1972) and Dobry and Vucetic (1987) performed extensive studies on normally consolidated and moderately consolidated soils to determine the factors influencing the dynamic properties of soils and concluded that shear modulus will decrease with increase in number of loading cycles for undrained conditions and the influence of number of loading cycles on damping ratio is not significant. Therefore, the results obtained for fly ash are in good agreement with the results of Hardin and Drnevich (1972) and Dobry and Vucetic (1987). Figures 5.52 to 5.54 are shown for fixed values of vertical stress equal to 40 kPa, frequency equal to 1 Hz and shear strain of 0.1%. Similar behavior was observed for other cases also, i.e., when tested at vertical stresses of 120 kPa, 200kPa and 350 kPa at various shear strains of 0.01%, 0.05%, 0.5% and 1%, maintaining 1 Hz frequency.

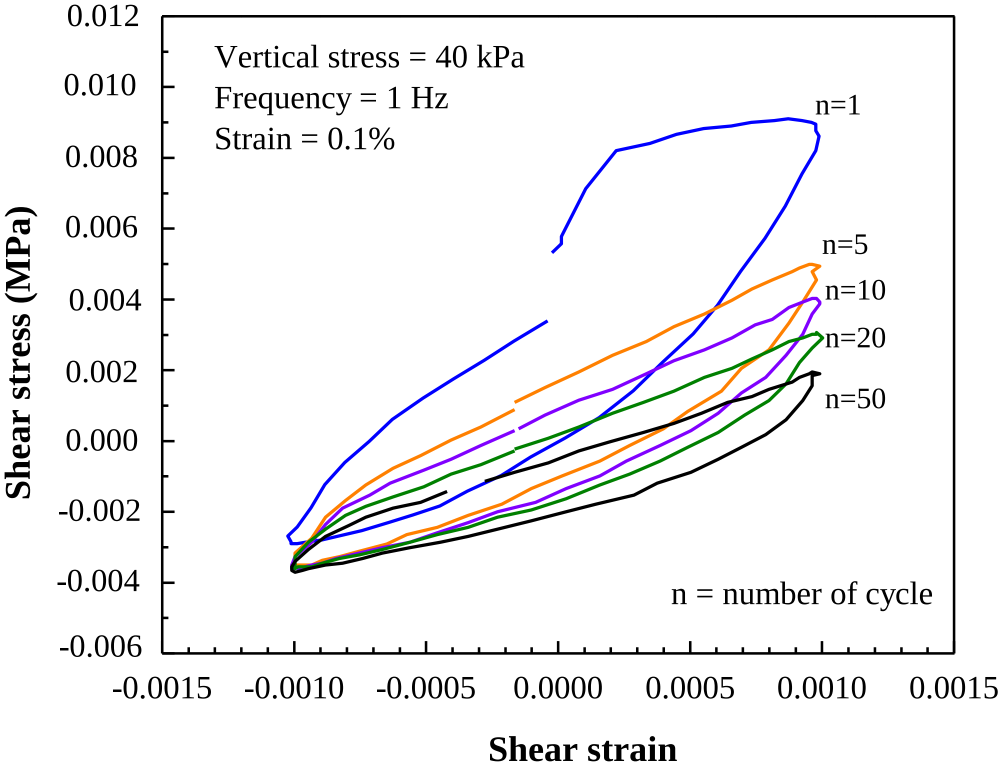


Figure 5.52: Influence of number of cycles on hysteresis loop

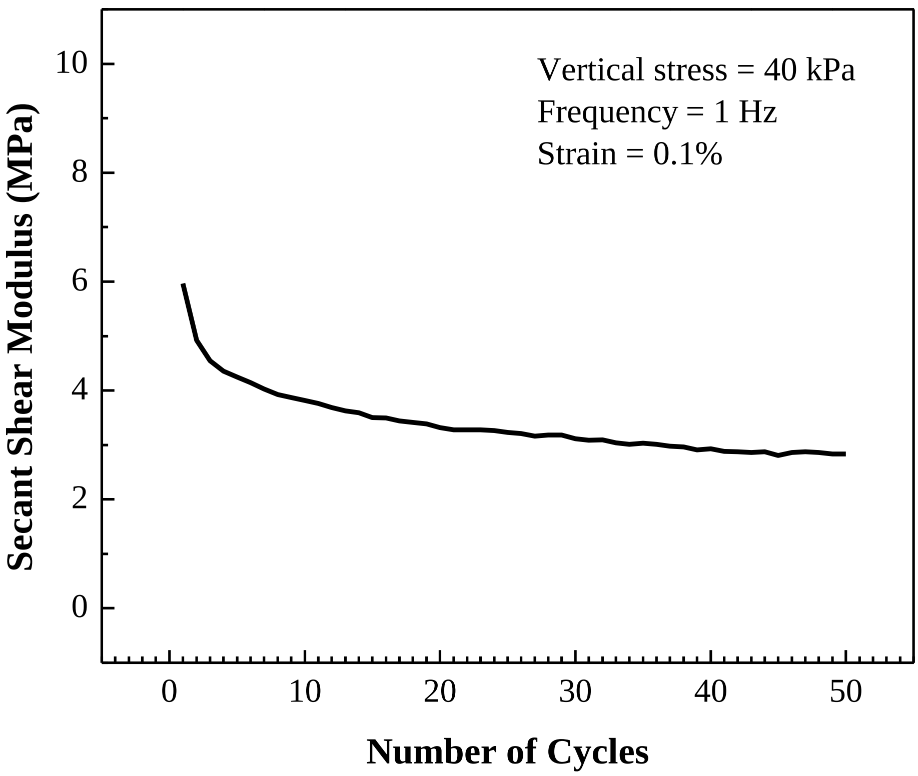


Figure 5.53: Influence of number of cycles on secant shear modulus of fly ash

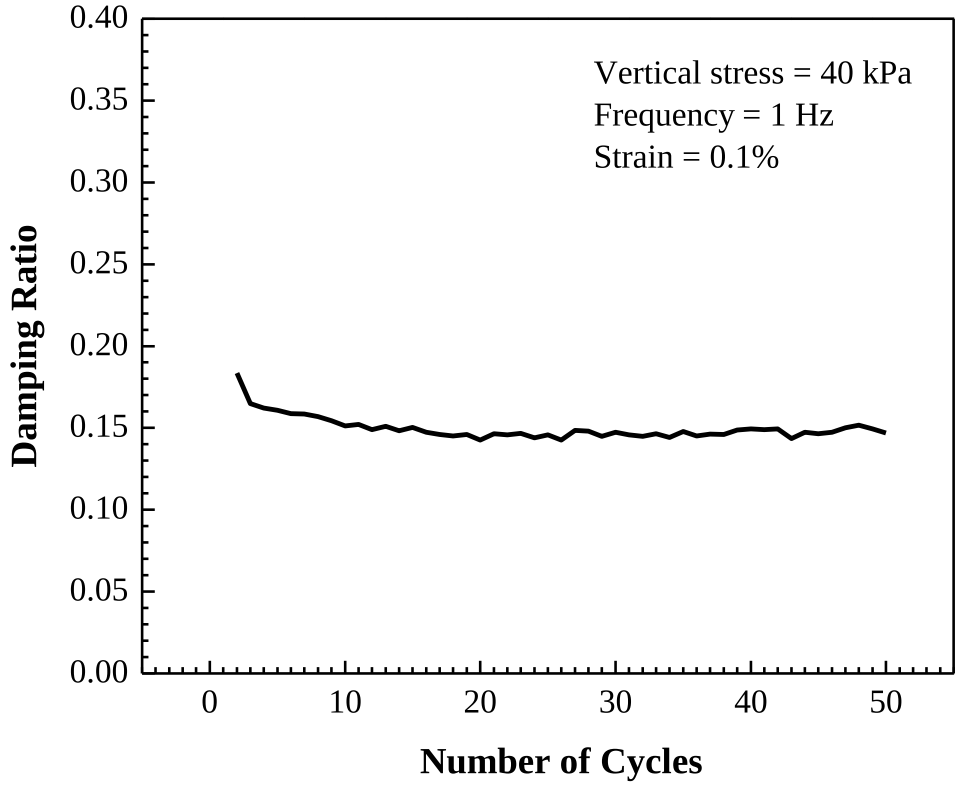


Figure 5.54: Influence of number of cycles on damping ratio of fly ash

According to Das and Ramana (1993), the number of significant cycles would be less than 20 in most seismic events. They suggested that the values of the 5th cycle will indicate the representative values of secant shear modulus and the damping ratio for all practical purposes. Therefore, secant shear modulus and damping ratio values reported in the subsequent sections corresponds to 5th cyclic loading.

5.3.2. Influence of Frequency

In order to study the influence of frequency on the dynamic properties of fly ash, the fly ash specimen was subjected to various frequencies ranging from 0.1 Hz to 2 Hz. Figures 5.55 and 5.56 show the variation of secant shear modulus and damping ratio with frequency respectively. With the change in frequency, secant shear modulus of fly ash more or less remains constant and in contrast, damping ratio value is varying with the frequency. Ravishankar et al. (2005) performed a series of strain controlled cyclic triaxial element tests on dry and saturated soil samples in medium to large shear strain levels and concluded that the effect of frequency is not significant on shear modulus but has some influence on the damping ratios of the soils for the range of frequencies tested (0.1 Hz to 2Hz). Therefore, the results obtained in this work are in good agreement with the Ravishankar et al. (2005).

Figure 5.55 and 5.56 are obtained when the fly ash specimen was tested at a vertical stress of 120 kPa and shear strain of 0.1%.

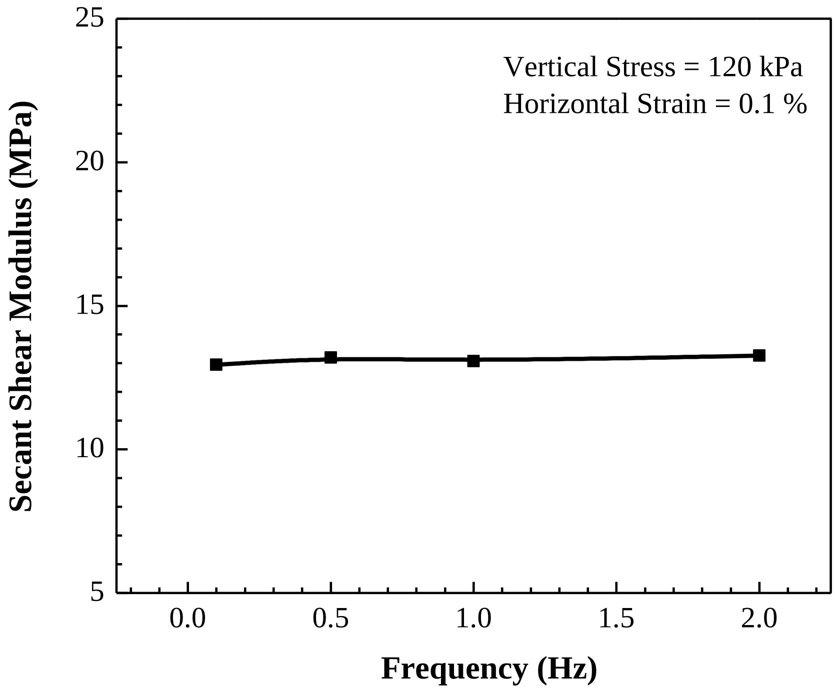


Figure 5.55: Influence of frequency on secant shear modulus of fly ash

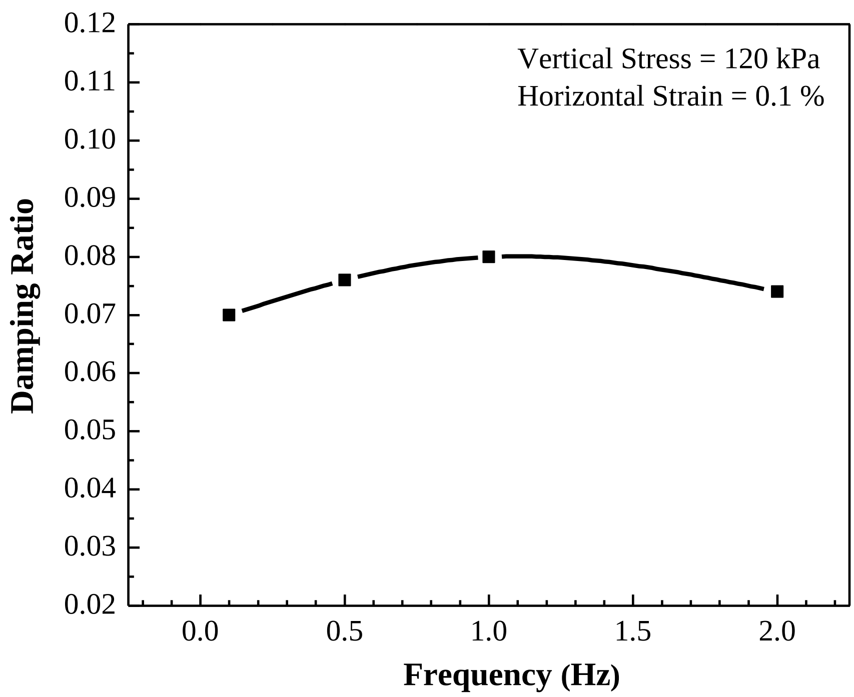


Figure 5.56: Influence of frequency on damping ratio of fly ash

An average frequency value of 1 Hz was selected for performing the remaining tests to determine the influence of vertical stress, shear strain magnitude and sample height on the dynamic properties of fly ash. Therefore, the values reported hereafter corresponds to a frequency of 1Hz and 5^{th} cyclic loading.

5.3.3. Influence of Vertical Stress

The prepared fly ash specimens were subjected to various vertical stresses (40 kPa, 120 kPa, 200 kPa and 350 kPa) in order to study the influence of vertical stress on the dynamic properties of fly ash at different shear strain magnitudes. The influence of vertical stress on secant shear modulus and damping ratio of fly ash were shown in figures 5.57 and 5.58 respectively. It can be concluded from the figures that for a particular shear strain, with the increase in the vertical stress, secant shear modulus increases and the damping ratio decreases. In addition, the increase in secant shear modulus with vertical stress is abruptly high between the shear strains of magnitude 0.01% to 0.1% compared to shear strains of magnitude greater than 0.1%. Similarly, the decrease in damping ratio with vertical stress is dramatically high between the shear strains of magnitude 0.01% to 0.1% compared to shear strains of magnitude greater than 0.1%. The studies of Silver and Seed (1971), Hardin and Drnevich (1972), Dobry and Vucetic (1987), Vucetic (1981), Ravishankar et al. (2005) and etc., also concluded that with the increase in shear strain, shear modulus increases and damping ratio decreases for various tested materials like sand, clay, and gravel.

Table 5.2 shows the change in secant shear modulus and damping ratio of fly ash with the increase in vertical stress from 40kPa to 350 kPa at various shear strains.

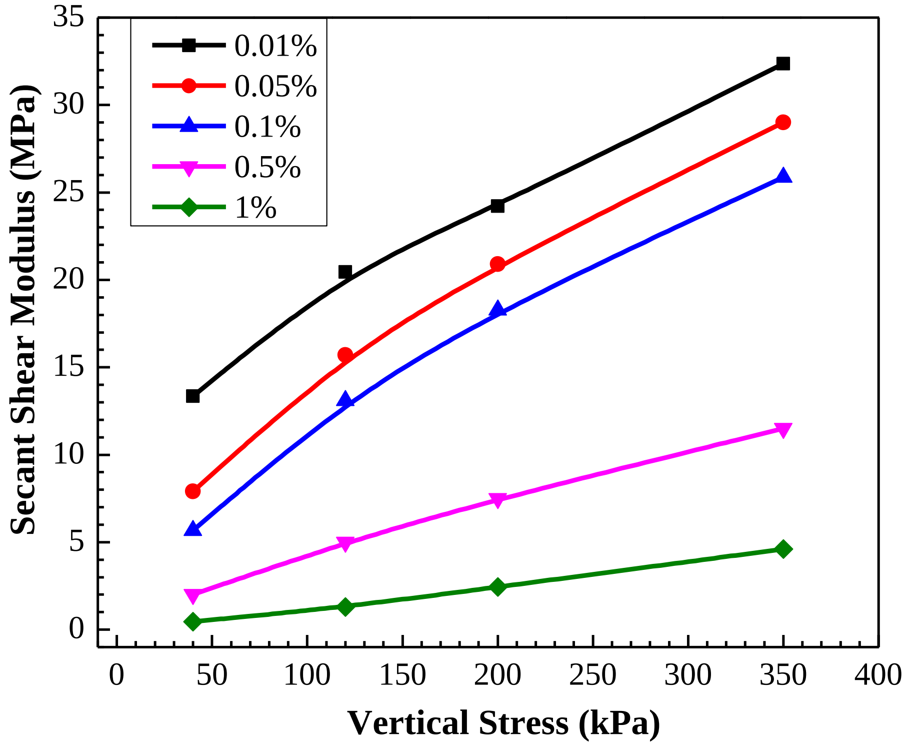


Figure 5.57: Influence of vertical stress on secant shear modulus of fly ash

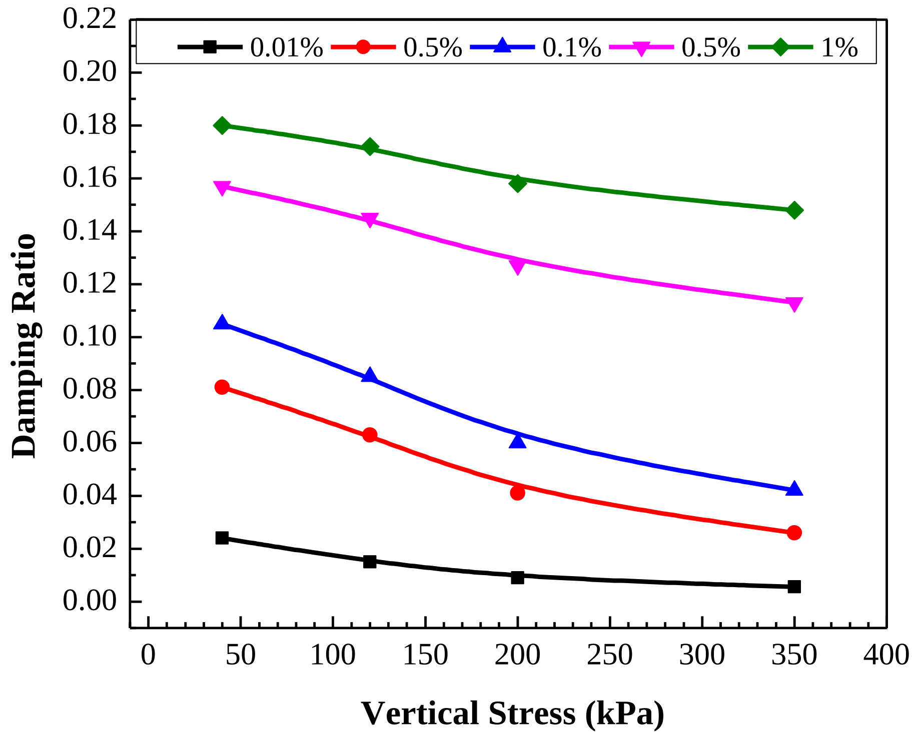


Figure 5.58: Influence of vertical stress on damping ratio of fly ash

Table 5.2: Influence of vertical stress on secant shear modulus and damping ratio

Parameter	Shear Strain (%)				
	0.01	**0.05**	**0.1**	**0.5**	**1**
% Increase in Secant Shear Modulus	58.74	72.76	78.05	82.61	90
% Decrease in Damping Ratio	77.08	67.9	60	28.02	17.77

From the table 5.2, we can conclude that with the increase in magnitude of shear strain, percentage increase in the secant shear modulus will rise and percentage decrease in damping ratio will decline if vertical stress increases from 40 kPa to 350 kPa.

5.3.4. Influence of Shear Strain

The influence of shear strain on dynamic properties of fly ash is studied by considering five shear strains of magnitude 0.01%, 0.05%, 0.1%, 0.5% and 1%. Figures 5.59 and 5.60 show the influence of shear strain on secant shear modulus and damping ratio respectively at various vertical stresses. It is clear from the graphs, with the increase in shear strain, secant shear modulus decreases, and damping ratio increases. Similar behavior was observed by Hardin and Drnevich (1972), Dobry and Vucetic (1987), Ravishankar et al. (2005) and etc., for the various tested materials like sand, clay, and gravel.

Table 5.3 shows the change in secant shear modulus and damping ratio with the increase in shear strain magnitude from 0.01% to 1% at various vertical stresses.

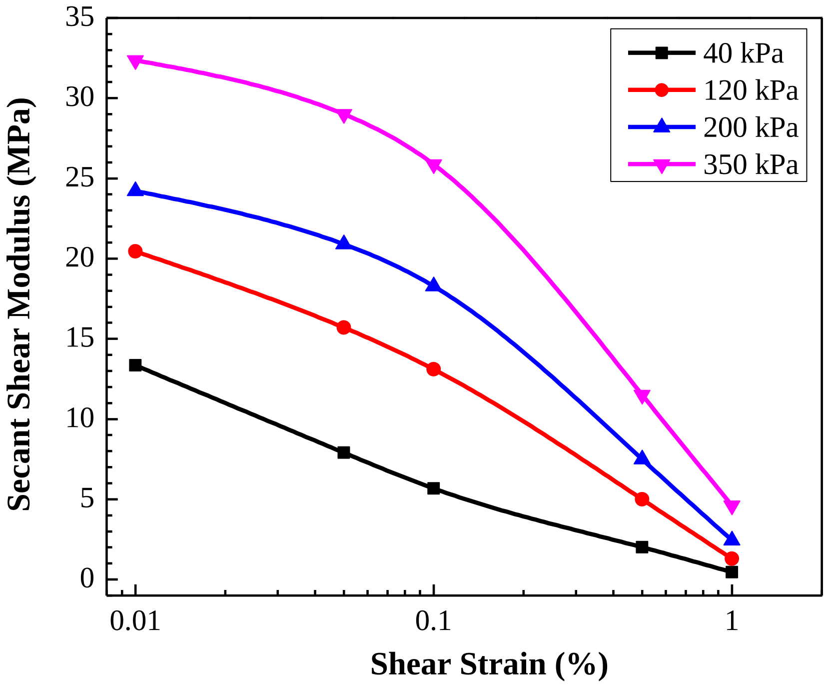


Figure 5.59: Influence of shear strain on secant shear modulus of fly ash at various vertical stresses

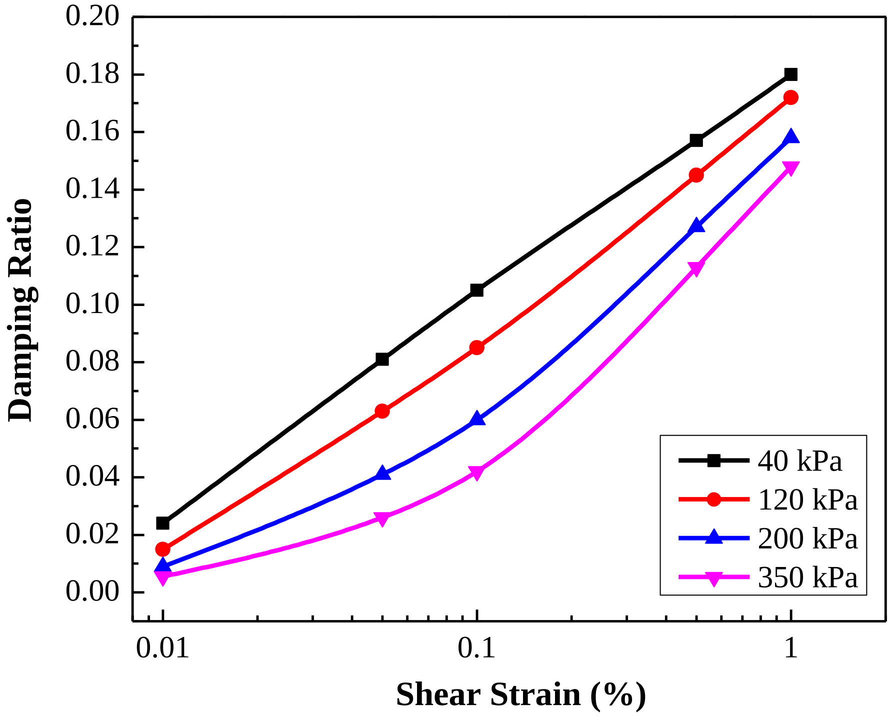


Figure 5.60: Influence of shear strain on damping ratio of fly ash at various vertical stresses

Table 5.3: Influence of shear strain on dynamic properties of fly ash

Parameter	Vertical Stress			
	40 kPa	**120 kPa**	**200 kPa**	**350 kPa**
% Decrease in Secant Shear Modulus	96.55	93.64	89.96	85.78
% Increase in Damping Ratio	86.67	91.28	94.30	96.28

From the table 5.3, we can conclude that, with the increase in shear strain magnitude from 0.01% to 1%, the percentage decrease in secant shear modulus will diminish and percentage increase in damping ratio will escalate with the increase in vertical stresses in between 40 kPa to 350 kPa. Apart from this, with the increase in the magnitude of shear strain, the hysteresis loops are becoming widened and attaining the reverse 'S' shaped hysteresis loop. This can be observed from the figures 5.61 and 5.62, where with the increase in shear strain magnitude from 0.1% to 1%, the hysteresis loop becomes widened and attained reverse 'S' shape. The figures are shown for the vertical stress of 40 kPa and similar behavior was observed at other vertical stresses also.

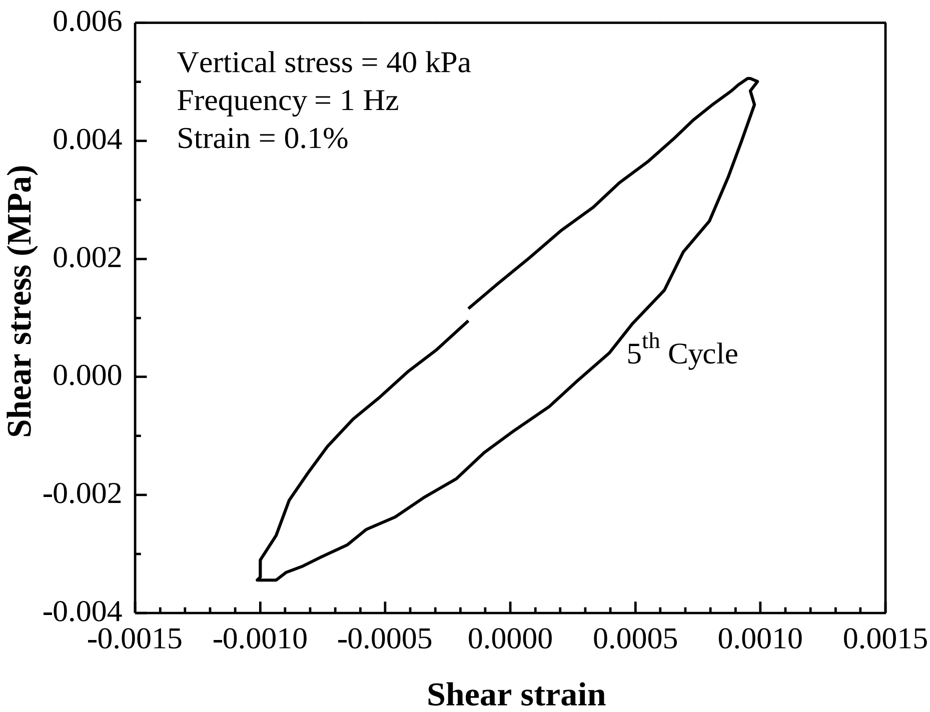


Figure 5.61: Hysteresis loop for the shear strain magnitude of 0.1% at vertical stress 40kPa

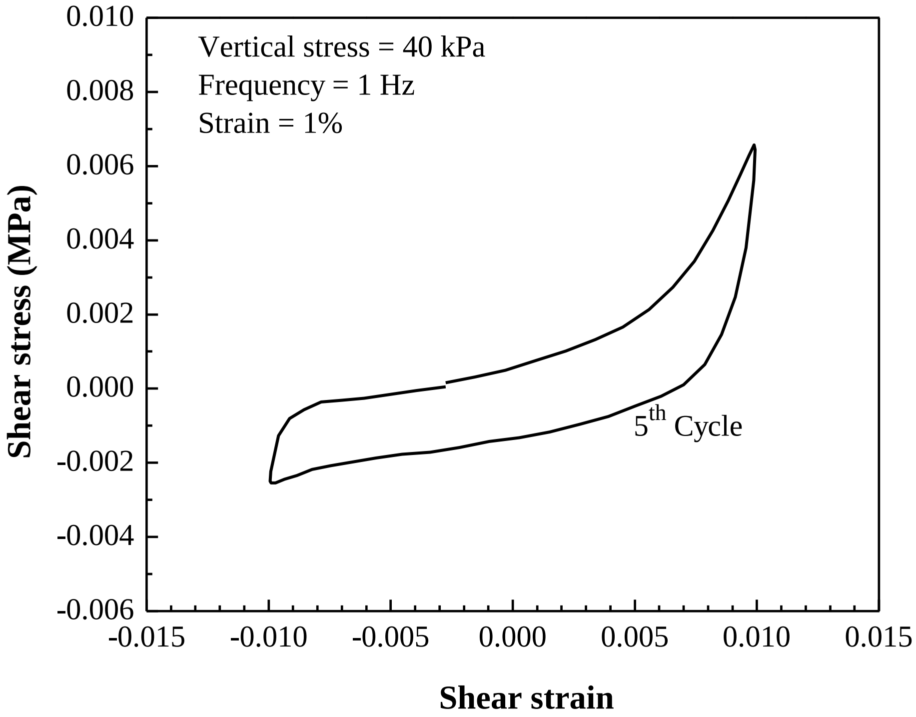


Figure 5.62: Hysteresis loop for the shear strain magnitude of 1% at vertical stress 40kPa

5.3.5. Relationship between Secant Shear Modulus and Damping Ratio

Hardin and Drnevich (1972), Tatsuoka et al. (1978), Ishibashi (1981) and Zhang and Aggour (1996) proposed that the damping ratio could be expressed as a function of shear modulus from their extensive studies on sandy soils. Considering the results obtained in this study, it was found that the damping ratio could also be expressed as a function of secant shear modulus. The authors tried to develop a linear relationship between damping ratio (D) and G_{sec}^{b}. The exponent 'b' was varied until a linear relationship exists between D and G_{sec}^{b}. The figure 5.63 shows the correlation developed between the secant shear modulus and damping ratio with a regression coefficient of 0.90 and equation 5.4 shows the developed correlation.

Shear modulus can also be determined from the field tests like standard penetration test (SPT) using the existing correlations between the field SPT values and shear modulus developed by researchers like Ohta et al. (1972) and etc. Using the developed correlation in this study, the damping ratio of fly ash can be calculated from the field determined shear modulus. However, the developed correlation is applicable only for the range of shear strains and vertical stresses considered in the study.

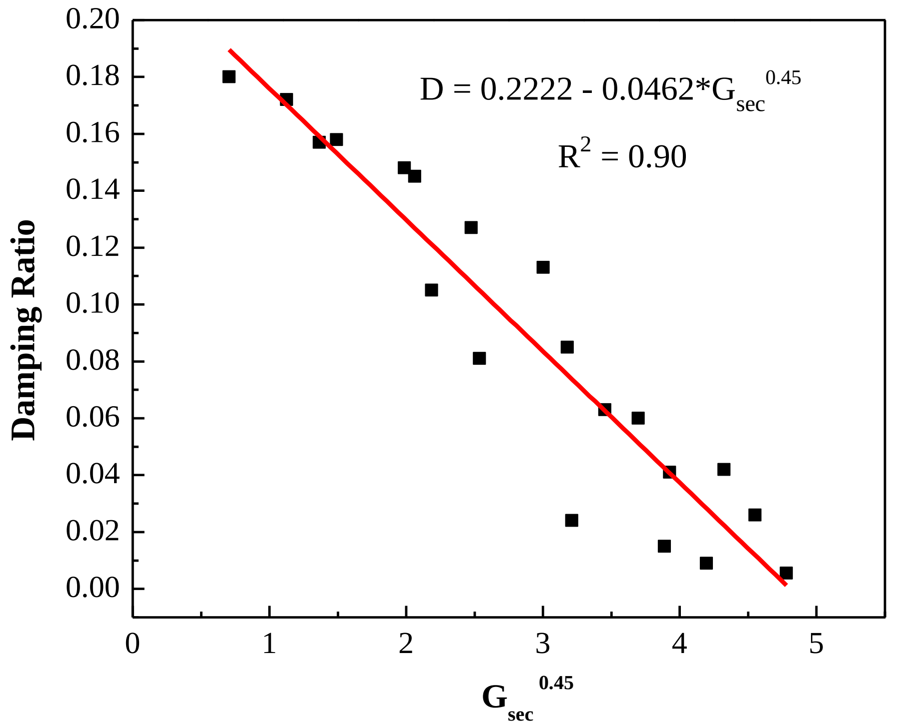


Figure 5.63: Correlation between secant shear modulus and damping ratio of fly ash

$$D = 0.2222 - 0.0462 \times G_{\text{sec}}^{0.45} \tag{5.4}$$

Where, G_{sec} is the secant shear modulus in MPa and D is the damping ratio.

5.3.6. Influence of Sample Height

The influence of sample height on dynamic properties of fly ash specimen was studied by considering two sample heights, 25 mm and 20 mm, maintaining the diameter of the specimen as constant (70mm). Figures 5.64 and 5.65 show the influence of sample height on secant shear modulus and damping ratio of fly ash respectively. From the figures, we can conclude that, with the decrease in sample height, secant shear modulus decreases and damping ratio increases for the same sample diameter. It is also observed from the figures, the sample height influence is enunciating higher in between the strains of magnitude 0.05% to 0.5% and the difference in dynamic properties with varying sample heights is comparatively less at strains of magnitude less than 0.05% and greater than 0.5%. To determine the influence of sample size, Wong et al. (1975) performed cyclic loading tests on Monterey sand considering 70 mm (2.8 in) and 300 mm (12 in.) diameter specimens with similar height-to-diameter ratios and concluded that the 300 mm (12 in.) diameter specimen was approximately 10% weaker than the 70 mm (2.8 in.) diameter specimen. Therefore, the dynamic properties and correlations developed in this study are pertaining to sample height of 25 mm and 70 mm diameter.

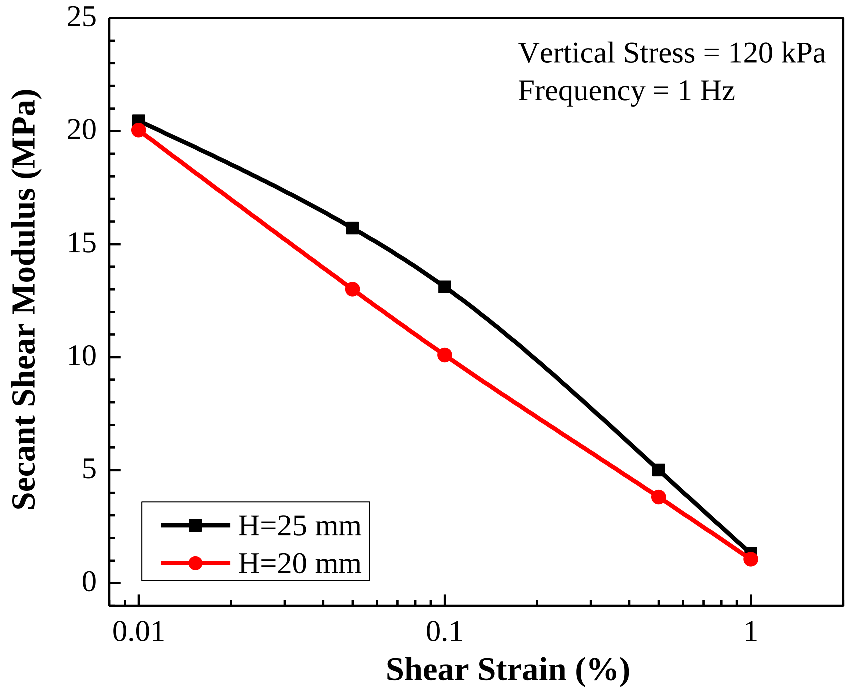


Figure 5.64: Influence of sample height on secant shear modulus of fly ash

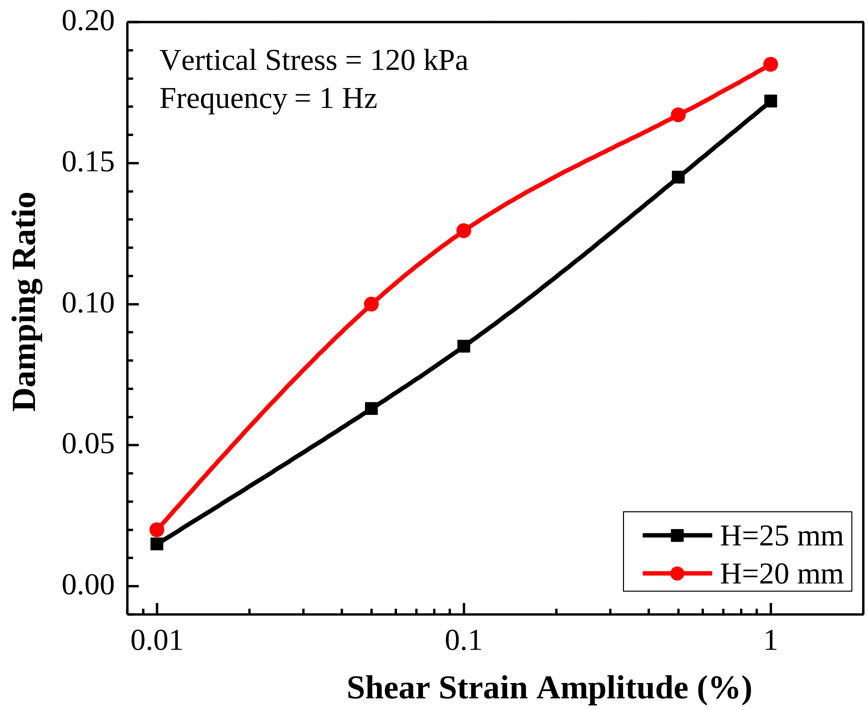


Figure 5.65: Influence of sample height on damping ratio of fly ash

5.4. Ramagundam Bottom Ash

5.4.1. Influence of number of cycles

Bottom ash specimen was subjected to 50 cycles of sinusoidal loading for determining the influence of number of cycles on the secant shear modulus and damping ratio of bottom ash. Hysteresis loops becoming flatter with the increase in the number of cycles (figure 5.66). From the figure 5.66, secant shear modulus, defined as the slope of a line through the end points of the hysteresis loop decreases with the number of cycles, as shown in figure 5.67. The progressive flattening of the shear stress versus shear strain curve conveys the decrease in peak load as the number of cycles increases. The percentage decrease in secant shear modulus for the first fifteen cycles is 16.3% and the percentage difference between the 15th cycle and 50th cycle is only 12.9%. Jafarzadeh et al. (2008) also stated that the rate of reduction of shear modulus will be higher for first few cycles by conducting triaxial tests on the Hostun sand. The influence of number of cycles on the damping ratio was shown in figure 5.68. With the increase in number of cycles, the damping ratio more or less remains constant in contrast to the secant shear modulus.

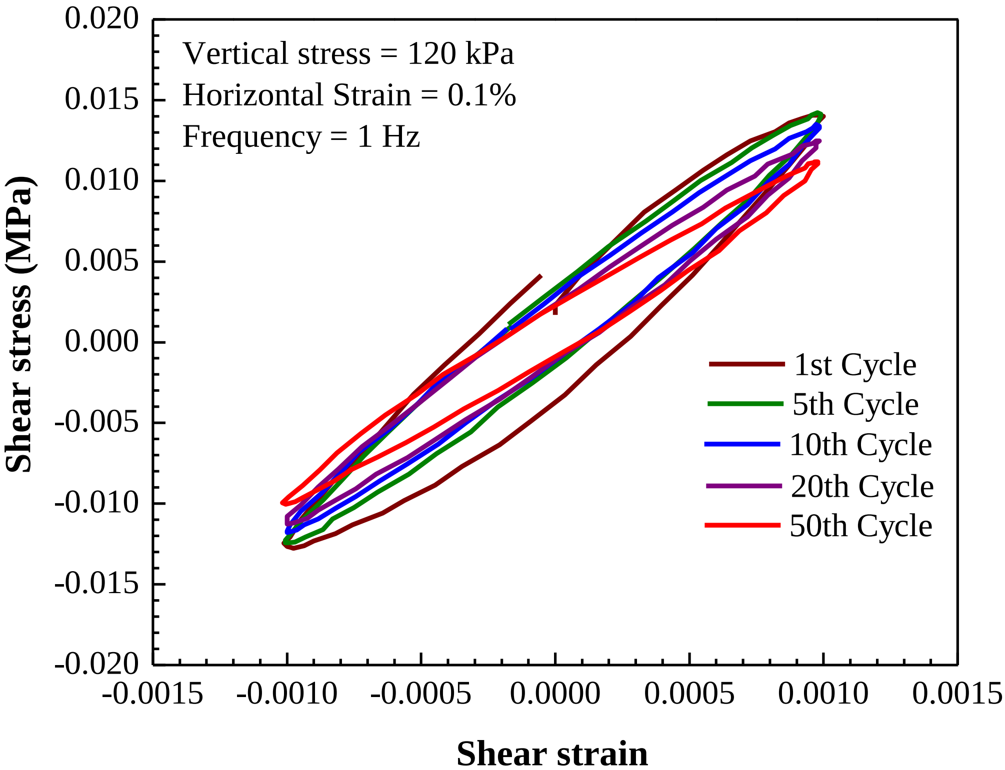


Figure 5.66: Influence of number of cycles on hysteresis loop

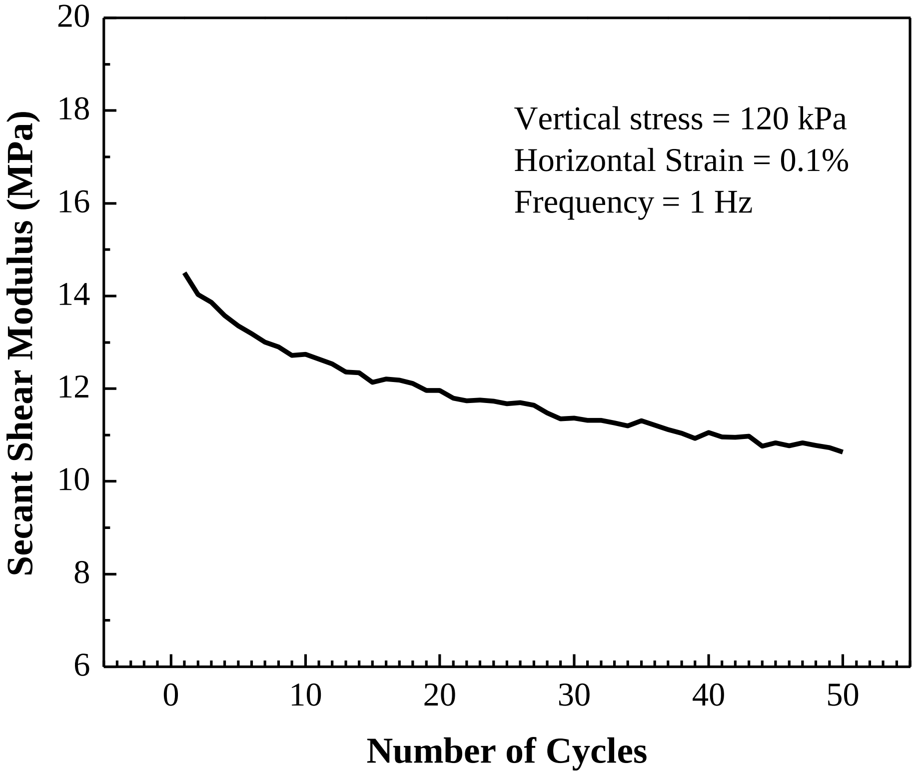


Figure 5.67: Influence of number of cycles on shear modulus of bottom ash

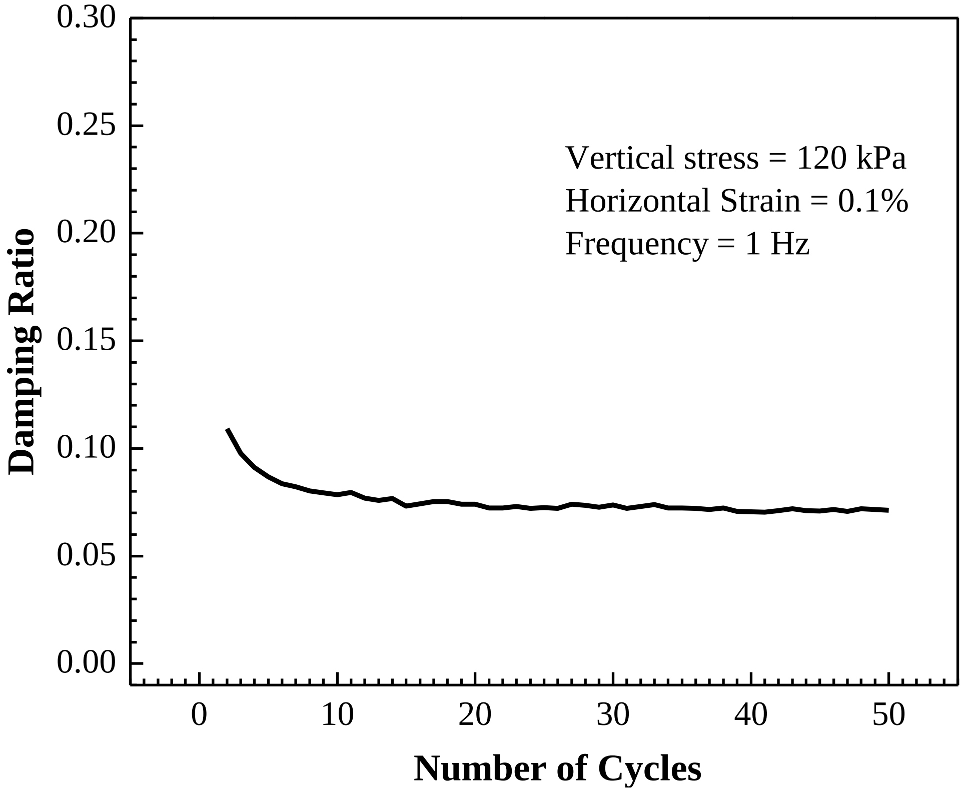


Figure 5.68: Influence of number of cycles on damping ratio of bottom ash

To determine the factors influencing the dynamic properties of soils, Hardin and Drnevich (1972) and Dobry and Vucetic (1987) performed extensive studies on normally consolidated and moderately consolidated soils and concluded that shear modulus will decrease with increase in number of loading cycles for undrained conditions and the influence of number of loading cycles on damping ratio is not significant. Therefore, the results obtained for bottom ash are in good agreement with the results of Hardin and Drnevich (1972) and Dobry and Vucetic (1987). Figures 5.66 to 5.68 are shown for fixed values of vertical stress equal to 120 kPa, frequency equal to 1 Hz and shear strain of 0.1%. Similar behavior was observed for other cases also, i.e., when tested at vertical stresses of 40 kPa, 200kPa and 350 kPa at various shear strains of magnitude 0.01%, 0.05%, 0.5% and 1%, maintaining 1 Hz frequency.

In accordance with Das and Ramana (1993), the number of significant cycles would be less than 20 in most seismic events and suggested that the values of the 5th cycle will indicate the representative values of secant shear modulus and the damping ratio for all practical purposes. Therefore, shear modulus and damping ratio values reported in the following sections corresponds to 5th cyclic loading.

5.4.2. Influence of Frequency

The bottom ash specimen was subjected to various frequencies ranging from 0.1 Hz to 2 Hz in order to study the influence of frequency on the dynamic properties of bottom ash. Figures 5.69 and 5.70 represent the variation of secant shear modulus and damping ratio with frequency respectively. With the change in frequency, damping ratio value is varying and in contrast, secant shear modulus of bottom ash more or less remains constant. A series of strain controlled cyclic triaxial element tests on dry and saturated soil samples in medium to large shear strain levels were performed by Ravishankar et al. (2005) and concluded that the effect of frequency is not significant on shear modulus but has some influence on the damping ratios of the soils for the range of frequencies tested (0.1 Hz to 2Hz). Therefore, the results obtained in this work are in good agreement with the outcomes of Ravishankar et al. (2005). Figure 5.69 and 5.70 are obtained when the bottom ash specimen was tested at a vertical stress of 120 kPa and shear strain of 0.1%. Similar kind of behavior was also observed when testing was done at other vertical stresses and shear strains.

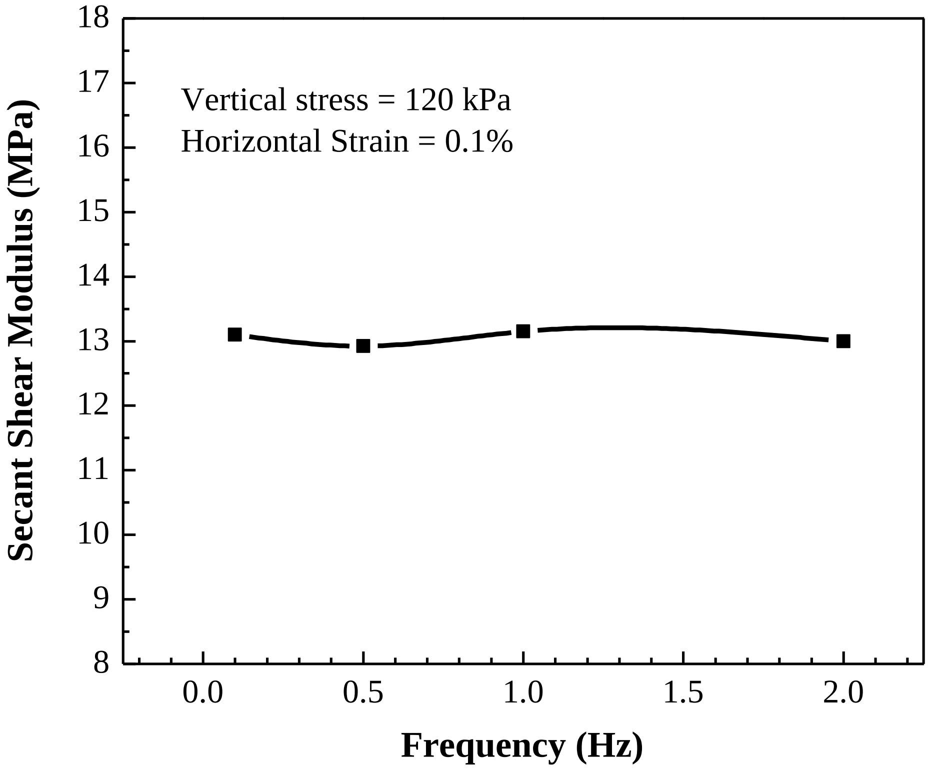


Figure 5.69: Influence of frequency on secant shear modulus of bottom ash

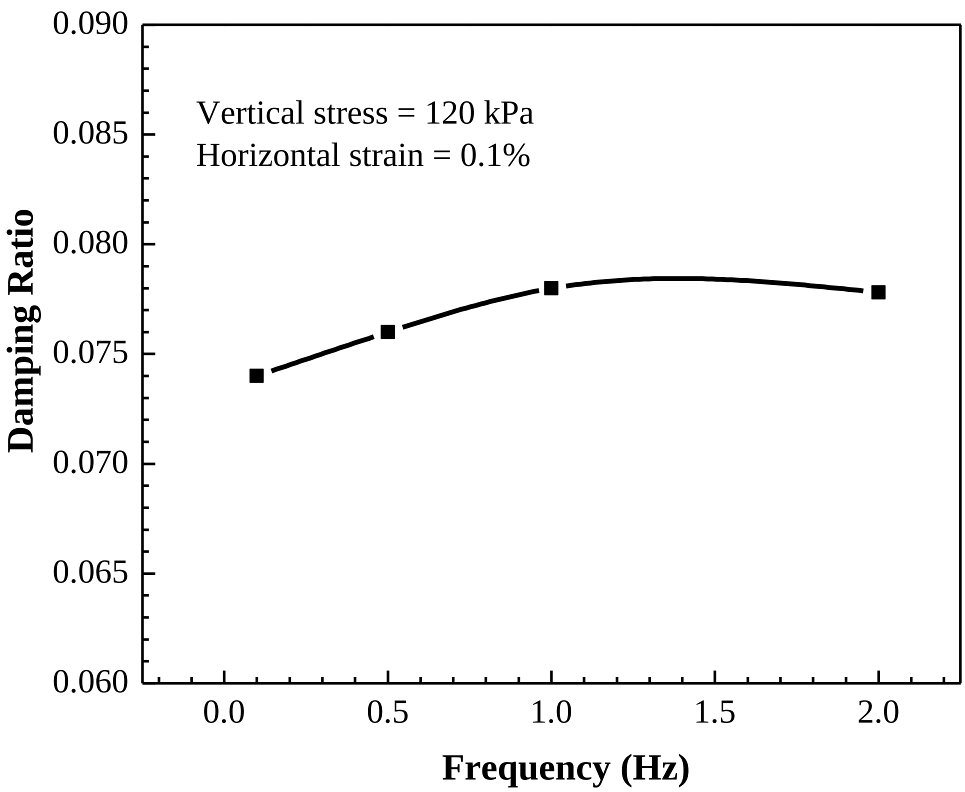


Figure 5.70: Influence of frequency on damping ratio of bottom ash

To determine the influence of vertical stress, shear strain magnitude and sample height on the dynamic properties of bottom ash, an average frequency value of 1 Hz was selected for performing the remaining tests. Therefore, the values reported hereafter corresponds to the 5^{th} cyclic loading and at a frequency of 1 Hz.

5.4.3. Influence of Vertical Stress

In order to study the influence of vertical stress on the dynamic properties of bottom ash, the specimen was subjected to vertical stresses of 40 kPa, 120 kPa, 200 kPa and 350 kPa. Figures 5.71 and 5.72 show the influence of vertical stress on secant shear modulus and damping ratio of bottom ash respectively at various shear strains. From the figures, it can be concluded that for a particular shear strain, with the increase in the vertical stress, secant shear modulus increases and the damping ratio decreases. In addition to that, the increase in secant shear modulus with vertical stress is abruptly high between the shear strains of magnitude 0.01% to 0.1% compared to shear strains of magnitude greater than 0.1%. Similarly, the decrease in damping ratio with vertical stress is dramatically high between the shear strains of magnitude 0.01% to 0.1% compared to shear strains of magnitude greater than 0.1%. It is also interesting to note that, at low normal stress (40 kPa) and at low shear strain (0.01%), bottom ash is exhibiting very high secant shear modulus value and very low damping ratio value in contrast to its behavior at other normal stresses and other shear strain magnitudes. The studies by several researchers - Silver and Seed (1971), Hardin and Drnevich (1972), Dobry and Vucetic (1987), Vucetic (1992), Ravishankar et al. (2005) and etc., also concluded that with the increase in shear strain, shear modulus increases and damping ratio decreases for various tested materials like sand, clay, and gravel.

Table 5.4 shows the change in secant shear modulus and damping ratio of bottom ash with the increase in vertical stress from 40kPa to 350 kPa at various shear strains.

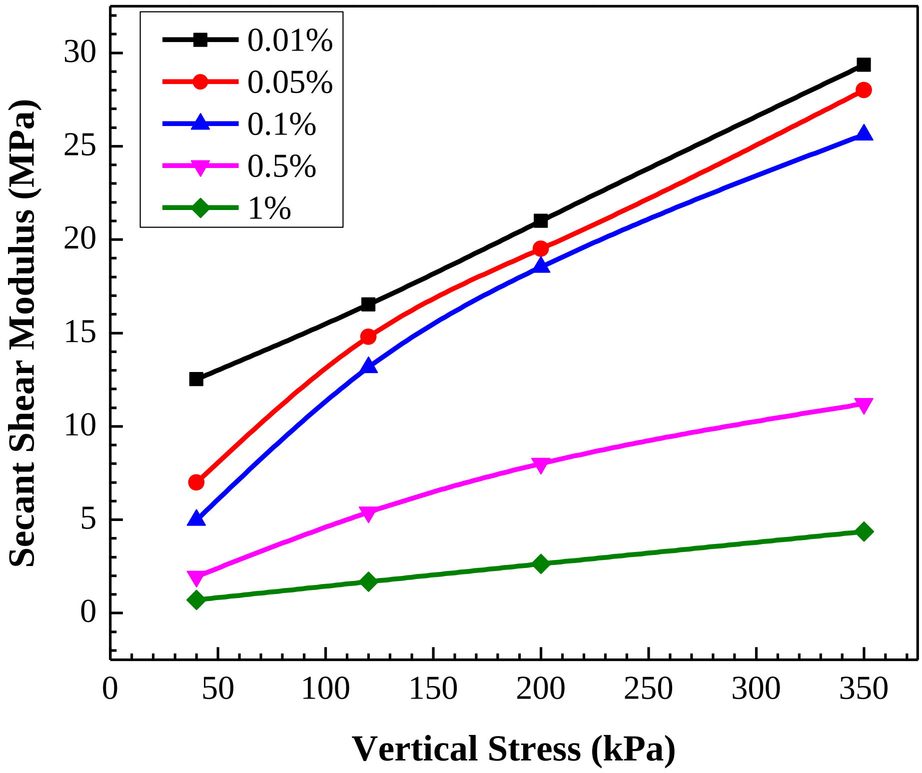


Figure 5.71: Influence of vertical stress on secant shear modulus of bottom ash

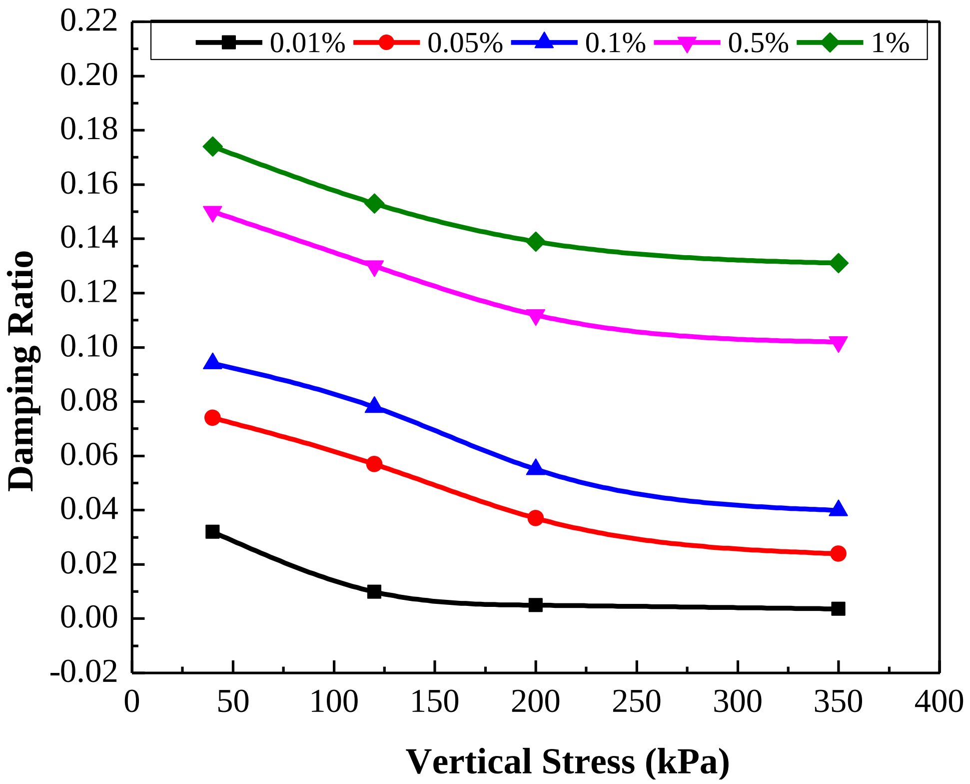


Figure 5.72: Influence of vertical stress on damping ratio of bottom ash

Table 5.4: Influence of vertical stress on secant shear modulus and damping ratio

Parameter	Shear Strain (%)				
	0.01	**0.05**	**0.1**	**0.5**	**1**
% Increase in Secant Shear Modulus	57.35	75	80.55	82.59	83.77
% Decrease in Damping Ratio	88.75	67.57	57.55	32	24.71

From the table 5.4, we can conclude that with the increase in magnitude of shear strain, percentage increase in the secant shear modulus will rise and percentage decrease in damping ratio will decline if vertical stress increases from 40 kPa to 350 kPa.

5.4.4. Influence of Shear Strain

Five shear strains of magnitude 0.01%, 0.05%, 0.1%, 0.5% and 1% are considered to study the influence of shear strain on dynamic properties of bottom ash. The influence of shear strain on secant shear modulus and damping ratio of bottom ash were shown in figures 5.73 and 5.74 respectively at various vertical stresses. From the graphs it can be concluded that, with the increase in shear strain, secant shear modulus decreases, and damping ratio increases. Similar behavior was observed by several researchers [Hardin and Drnevich (1972), Dobry and Vucetic (1987), Ravishankar et al. (2005) and etc.] for the various tested materials like sand, clay, and gravel.

Table 5.5 shows the change in secant shear modulus and damping ratio with the increase in magnitude of shear strain from 0.01% to 1% at various vertical stresses.

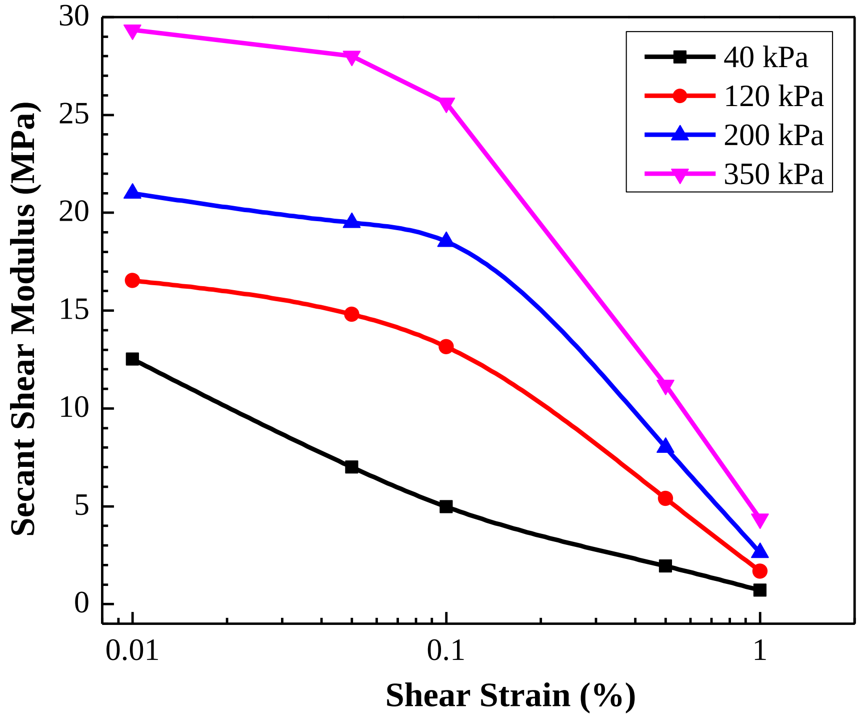


Figure 5.73: Influence of shear strain on secant shear modulus of bottom ash at various vertical stresses

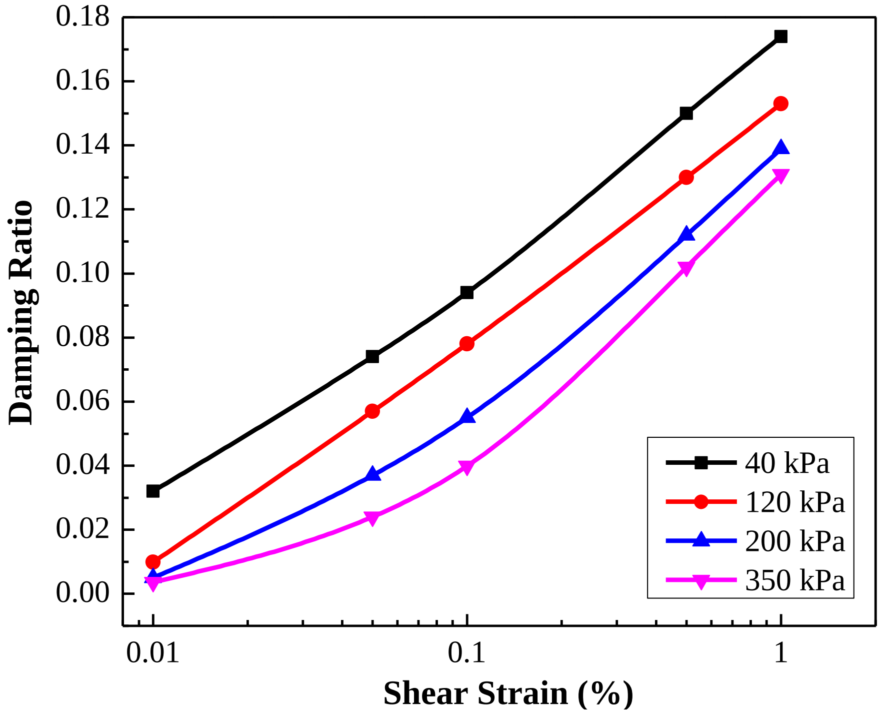


Figure 5.74: Influence of shear strain on damping ratio of bottom ash at various vertical stresses

Table 5.5: Influence of shear strain on dynamic properties of bottom ash

Parameter	Vertical Stress			
	40 kPa	120 kPa	200 kPa	350 kPa
% Decrease in Secant Shear Modulus	94.36	89.81	86.92	85.16
% Increase in Damping Ratio	81.61	93.53	96.40	97.25

From the table 5.5, we can conclude that with the rise in vertical stress, percentage decrease in the secant shear modulus will diminish and percentage increase in damping ratio will escalate if shear strain increases from 0.01% to 1%. Apart from this, with the increase in the magnitude of shear strain, the hysteresis loops are becoming widened and attaining the reverse 'S' shaped hysteresis loop. With the increase in shear strain magnitude from 0.1% to 1%, the hysteresis loop becomes widened and attained reverse 'S' shape as shown in the figures 5.75 and 5.76. The figures are shown for the vertical stress of 120 kPa and similar behavior was observed at other vertical stresses also.

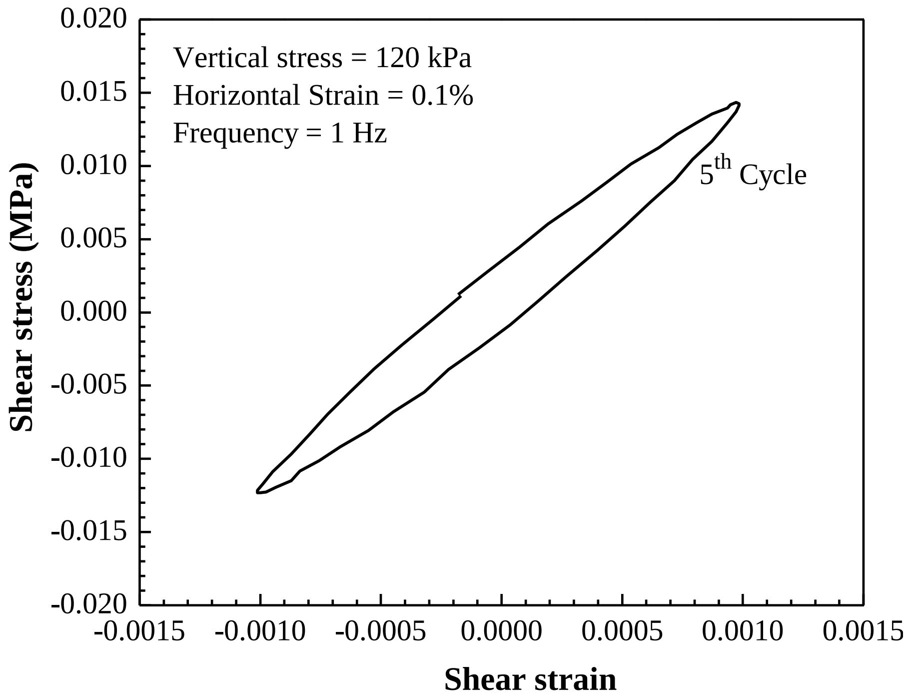


Figure 5.75: Hysteresis loop for the shear strain magnitude of 0.1% at vertical stress 120 kPa

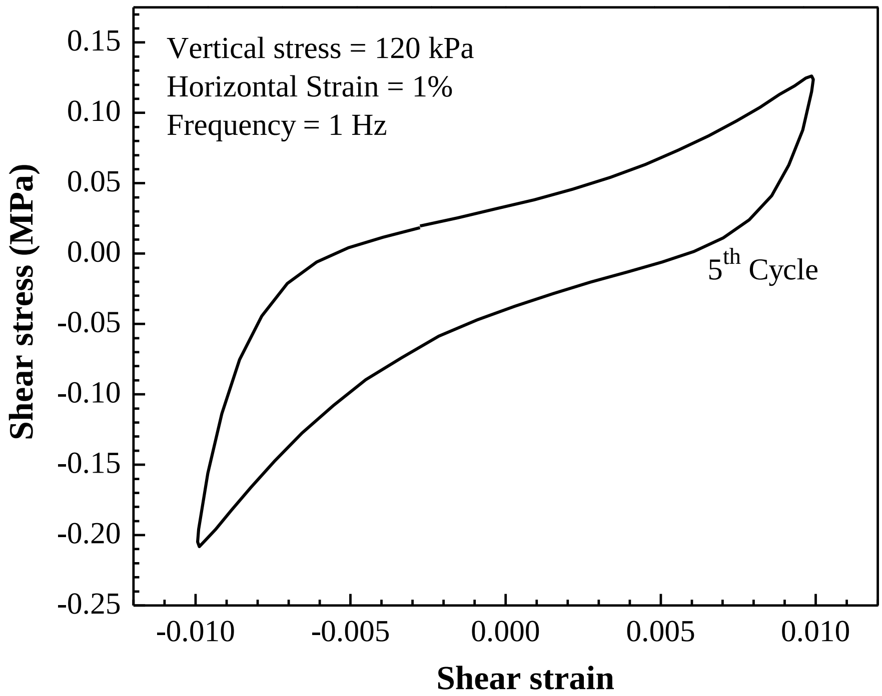


Figure 5.76: Hysteresis loop for the shear strain magnitude of 1% at vertical stress 120 kPa

5.4.5. Relationship between Secant Shear Modulus and Damping Ratio

Through their extensive studies on sandy soils, Hardin and Drnevich (1972), Tatsuoka et al. (1978), Ishibashi (1981) and Zhang and Aggour (1996) proposed that the damping ratio could be expressed as a function of shear modulus. In this study also, authors tried to fit a linear relationship between secant shear modulus and damping ratio. A relationship between damping ratio (D) and G_{sec}^{b} was developed by varying the exponent 'b' until a linear relationship exists between D and G_{sec}^{b}. Equation 5.5 shows the developed correlation and the figure 5.77 shows the correlation developed between the secant shear modulus and damping ratio with a regression coefficient of 0.87.

Researchers like Ohta et al. (1972) and etc., developed the correlations between the standard penetration test (SPT) value and shear modulus. Therefore, shear modulus can be determined easily from various field tests but not the damping ratio. Using the developed correlation in this study, the damping ratio of bottom ash can be calculated from the field determined shear modulus. However, the developed correlation is applicable only for the range of shear strains and vertical stresses considered in the study.

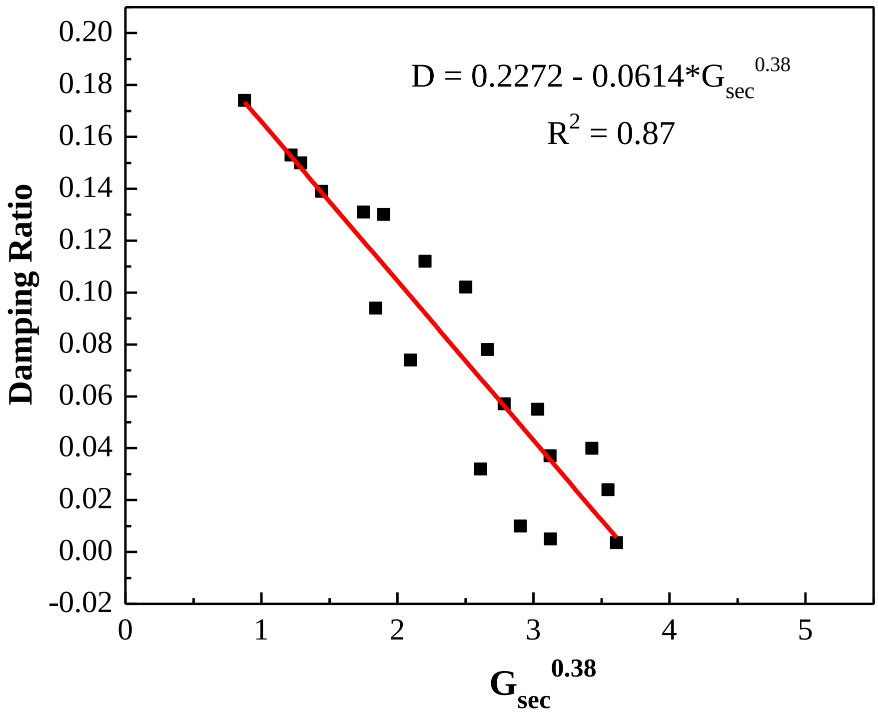


Figure 5.77: Correlation between secant shear modulus and damping ratio of bottom ash

$$D = 0.2272 - 0.0614 \times G_{\sec}^{0.38} \tag{5.5}$$

Where, G_{sec} is the secant shear modulus in MPa and D is the damping ratio.

5.3.6. Influence of Sample Height

Two specimen heights, 25 mm and 20 mm are considered to study the influence of sample height on secant shear modulus and damping ratio of the bottom ash, maintaining constant diameter of 70 mm. Figures 5.78 and 5.79 show the influence of sample height on secant shear modulus and damping ratio of bottom ash respectively. From the figures, we can conclude that with the decrease in sample height, secant shear modulus decreases and damping ratio increases for the same sample diameter. It is also calculated from the figures, the sample height influence on secant shear modulus is increasing with the increase in shear strain magnitude. The percentage difference is 16.51% at 0.01% shear strain magnitude, 23.13% at 0.1% shear strain magnitude and 33.42% at 1% shear strain magnitude. Similarly, the rate of change in damping ratio of bottom ash with varying height of specimen is decreasing. The percentage difference in damping ratio between sample heights 25 mm and 20 mm is 41.07% at 0.01% shear strain magnitude, 24.27% at 0.1% shear strain magnitude and 14.04% at 1% shear strain magnitude. Therefore, the dynamic properties and correlations developed in this study are pertaining to sample height of 25 mm and 70 mm diameter.

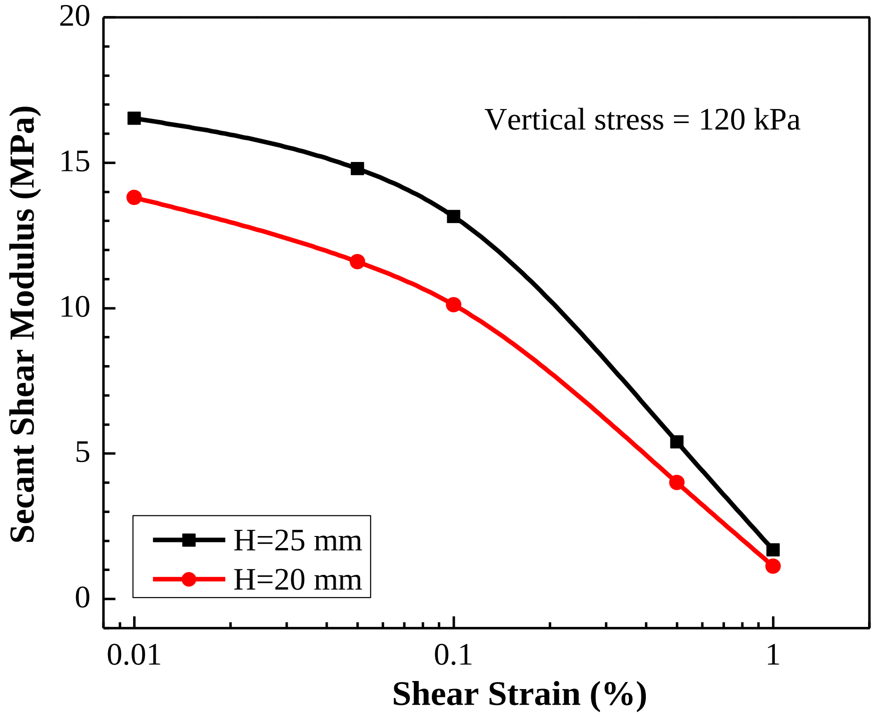


Figure 5.78: Influence of sample height on secant shear modulus of bottom ash

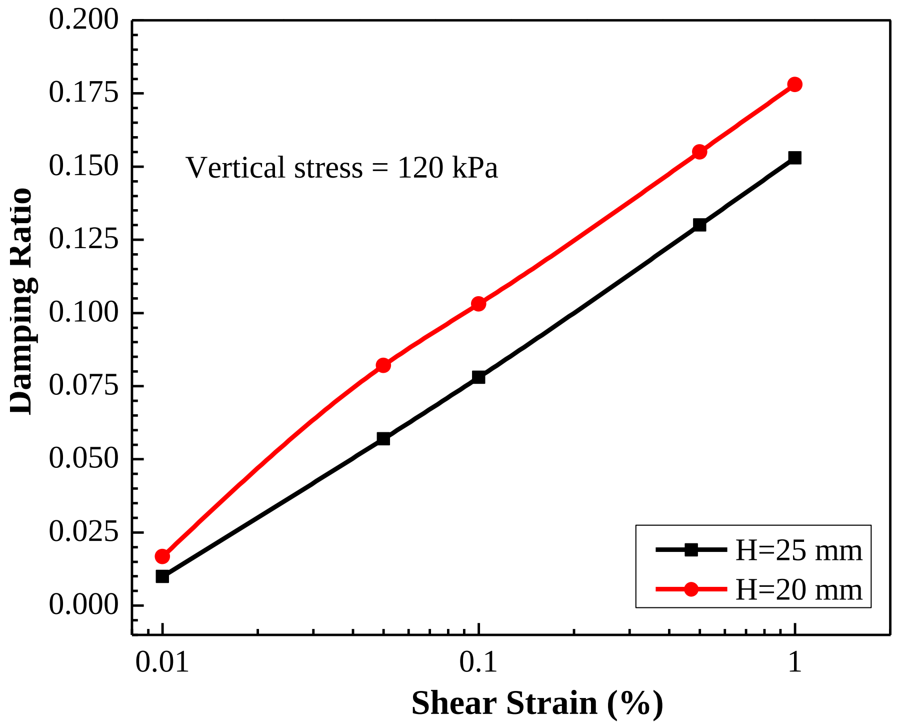


Figure 5.79: Influence of sample height on damping ratio of bottom ash

5.5. Ramagundam Pond Ash

5.5.1. Influence of number of cycles

The data obtained from data logger when the pond ash specimen subjected to 50 cycles of sinusoidal loading was used to determine the influence of number of cycles on the secant shear modulus and damping ratio of pond ash. With the increase in number of cycles, hysteresis loop is becoming flatter as shown in figure 5.80. The progressive flattening of the shear stress versus shear strain curve conveys the decrease in peak load as the number of cycles increases. Figure 5.80 also conveys that, secant shear modulus, defined as the slope of a line through the end points of the hysteresis loop, decreases with the increase in number of cycles. Figure 5.81 shows the variation of secant shear modulus with the number of cycles. The percentage decrease in secant shear modulus for the first ten cycles is 54% and the percentage difference between the 11^{th} cycle and 50^{th} cycle is only 28.7%. Jafarzadeh et al. (2008) also stated that the rate of reduction of shear modulus will be higher for first few cycles by conducting triaxial tests on the Hostun sand. Figure 5.82 shows the influence of number of cycles on the damping ratio. In contrast to secant shear modulus, damping ratio more or less remains constant with the increase in number of cycles, with slight decrease at the first few cycles.

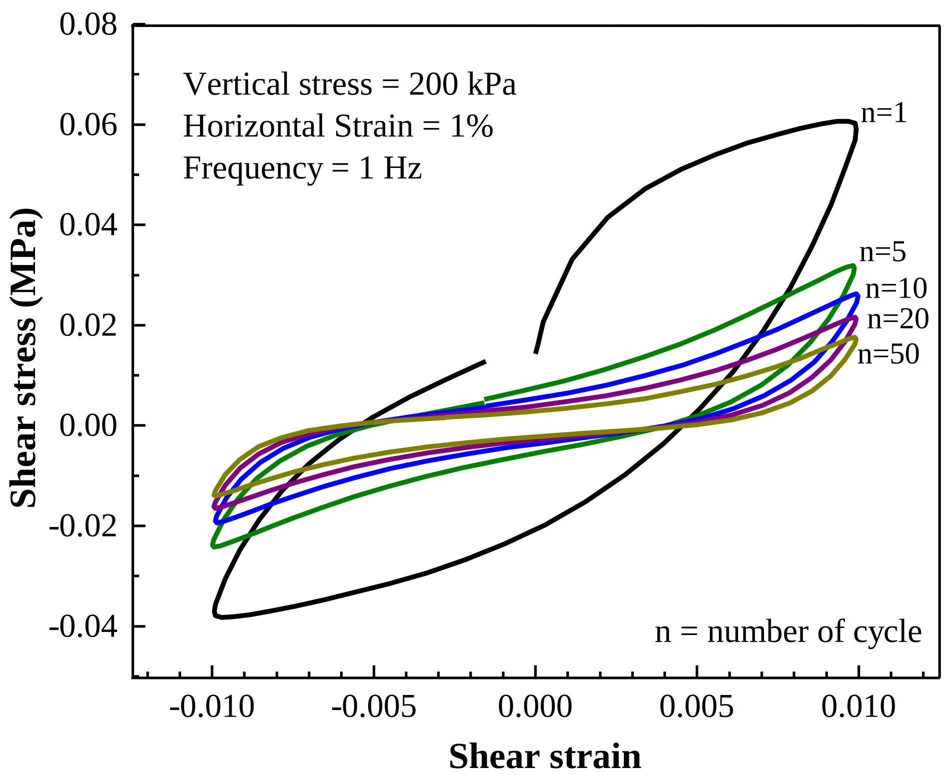


Figure 5.80: Influence of number of cycles on hysteresis loop

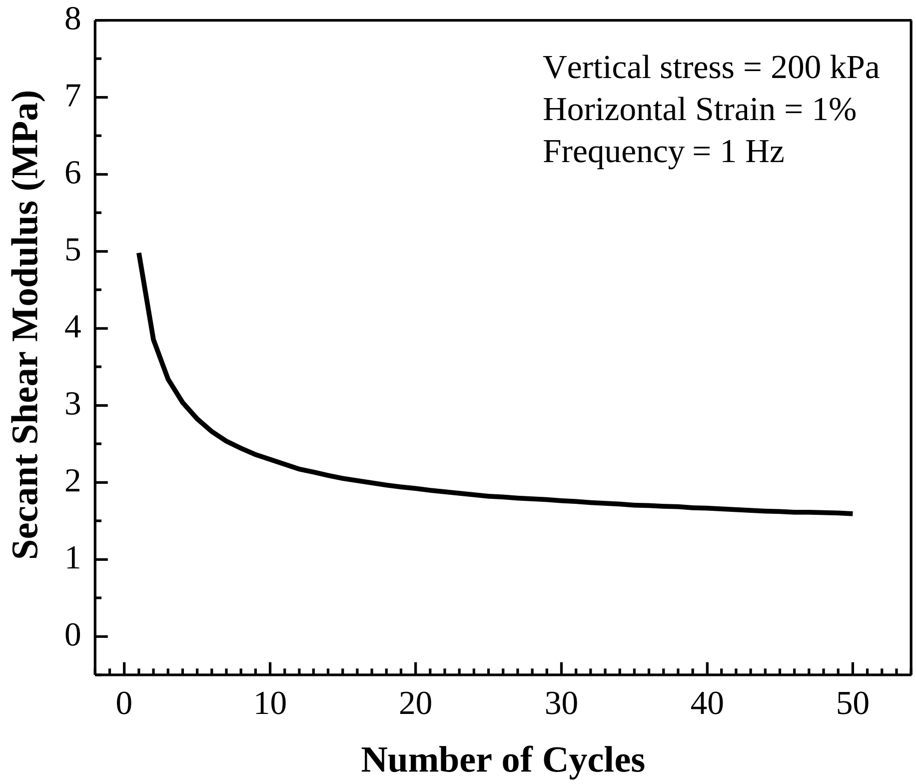


Figure 5.81: Influence of number of cycles on shear modulus of pond ash

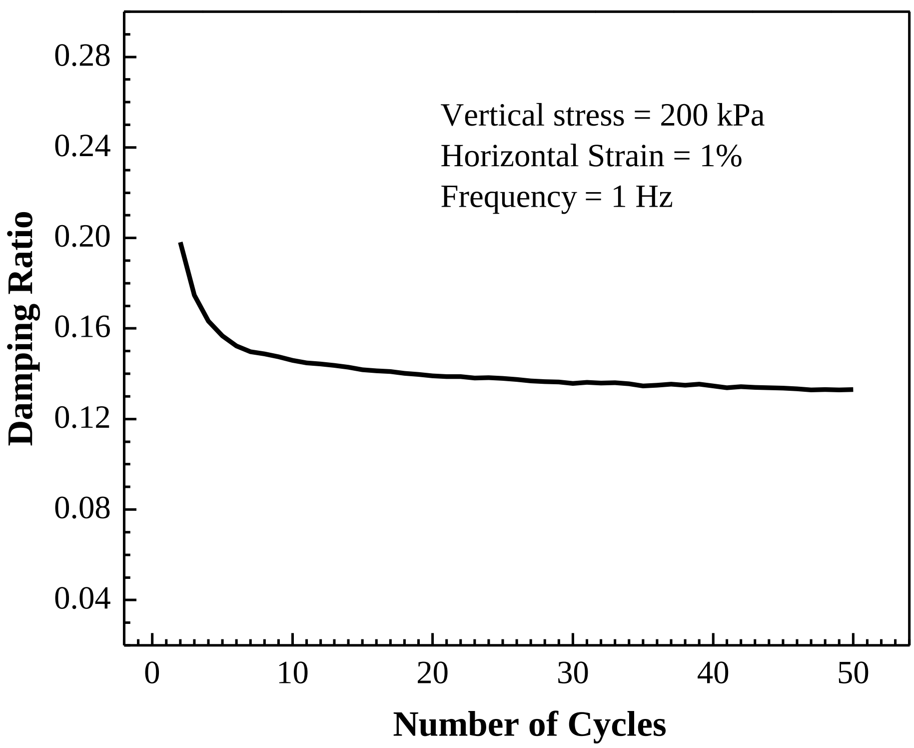


Figure 5.82: Influence of number of cycles on damping ratio of pond ash

The extensive studies of Hardin and Drnevich (1972) and Dobry and Vucetic (1987) on normally consolidated and moderately consolidated soils to determine the factors influencing the dynamic properties of soils concluded that shear modulus will decrease with increase in number of loading cycles for undrained conditions and the damping ratio will not be influenced significantly by the number of loading cycles. Therefore, the results obtained in this study are in good agreement with the outcomes of Hardin and Drnevich (1972) and Dobry and Vucetic (1987). Figures 5.80 to 5.82 are shown for fixed values of vertical stress equal to 200 kPa, frequency equal to 1 Hz and shear strain of 1%. Similar behavior can also be shown for other cases, i.e., when tested at vertical stresses of 40 kPa, 120 kPa and 350 kPa at various shear strains of magnitude 0.01%, 0.05%, 0.1% and 0.5%, maintaining 1 Hz frequency.

The number of significant cycles would be less than 20 in most seismic events, Das and Ramana (1993) suggested that the values of the 5^{th} cycle will indicate the representative values of secant shear modulus and the damping ratio for all practical purposes. Therefore, shear modulus and damping ratio values reported in the following sections corresponds to 5^{th} cyclic loading.

5.5.2. Influence of Frequency

In order to study the influence of frequency on the dynamic properties of pond ash, the pond ash specimen was subjected to various frequencies ranging from 0.1 Hz to 2 Hz keeping the vertical stress as 120 kPa and maintaining shear strain magnitude of 0.1%. Figures 5.83 and 5.84 represent the variation of secant shear modulus and damping ratio with frequency respectively. With the change in frequency, damping ratio value is varying and in contrast, secant shear modulus of pond ash more or less remains constant. A series of strain controlled cyclic triaxial element tests were performed by Ravishankar et al. (2005) on dry and saturated soil samples in medium to large shear strain levels and concluded that the effect of frequency is not significant on shear modulus but has some influence on the damping ratios of the soils for the range of frequencies tested (0.1 Hz to 2Hz). Therefore, the results obtained in this work are in good agreement with the conclusions of Ravishankar et al. (2005).

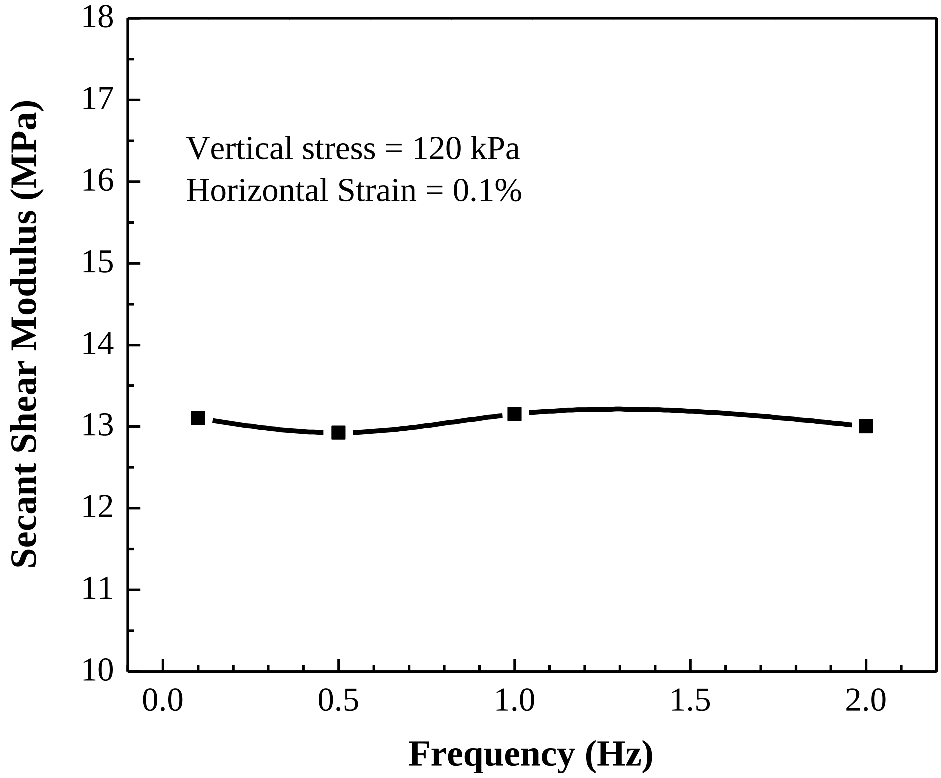


Figure 5.83: Influence of frequency on secant shear modulus of pond ash

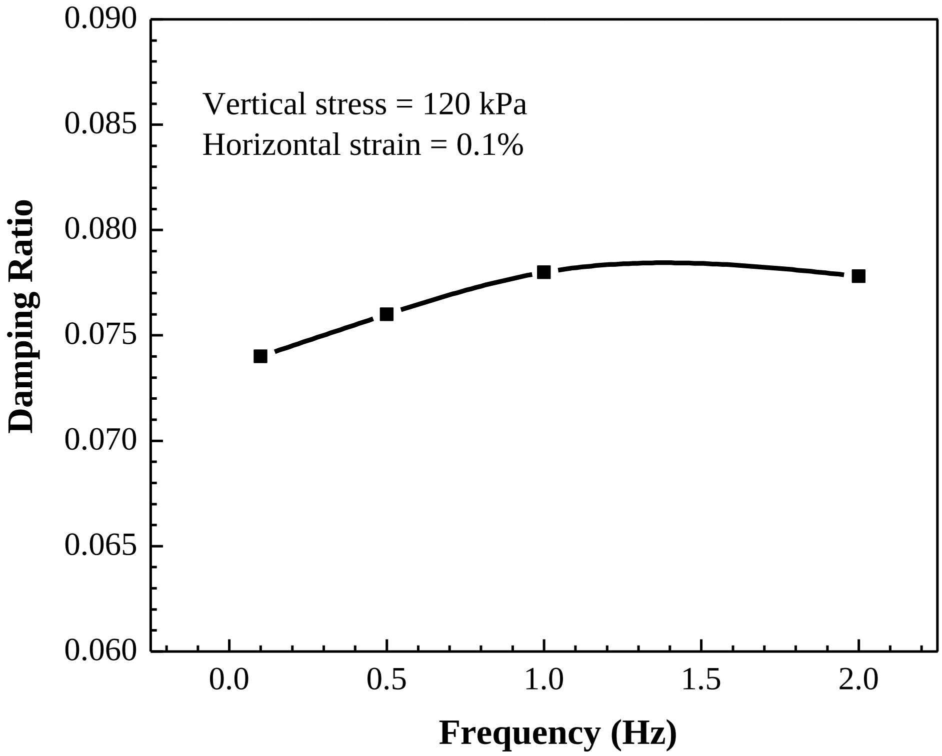


Figure 5.84: Influence of frequency on damping ratio of pond ash

For performing the remaining tests to determine the influence of vertical stress, shear strain magnitude and sample height on the dynamic properties of pond ash, an average frequency value of 1 Hz was selected. Therefore, the values reported hereafter corresponds to the 5th cyclic loading and at a frequency of 1 Hz.

5.5.3. Influence of Vertical Stress

The prepared pond ash specimen was subjected to vertical stresses of 40 kPa, 120 kPa, 200 kPa and 350 kPa in order to study the influence of vertical stress on the dynamic properties of pond ash. The influence of vertical stress on secant shear modulus and damping ratio of pond ash were shown in figures 5.85 and 5.86 respectively at various shear strains. Figures 5.85 and 5.86 are clearly conveying that, for a particular shear strain, with the increase in the vertical stress, secant shear modulus increases and the damping ratio decreases. Also, the increase in secant shear modulus with vertical stress is abruptly high between the shear strains of medium magnitude (0.01% to 0.1%) compared to high shear strain magnitudes (> 0.1%). Regarding the influence of vertical stress on damping ratio, the decrease in damping ratio with vertical stress is dramatically high between the shear strains of medium magnitude (0.01% to 0.1%) compared to shear strains of higher magnitude (>0.1%). It is also worthy to note that, at low vertical stress of 50 kPa and low shear strain magnitude of 0.01%, secant shear modulus value is quite high and damping ratio value is pretty low compared to values at other vertical stresses and shear strain magnitudes.

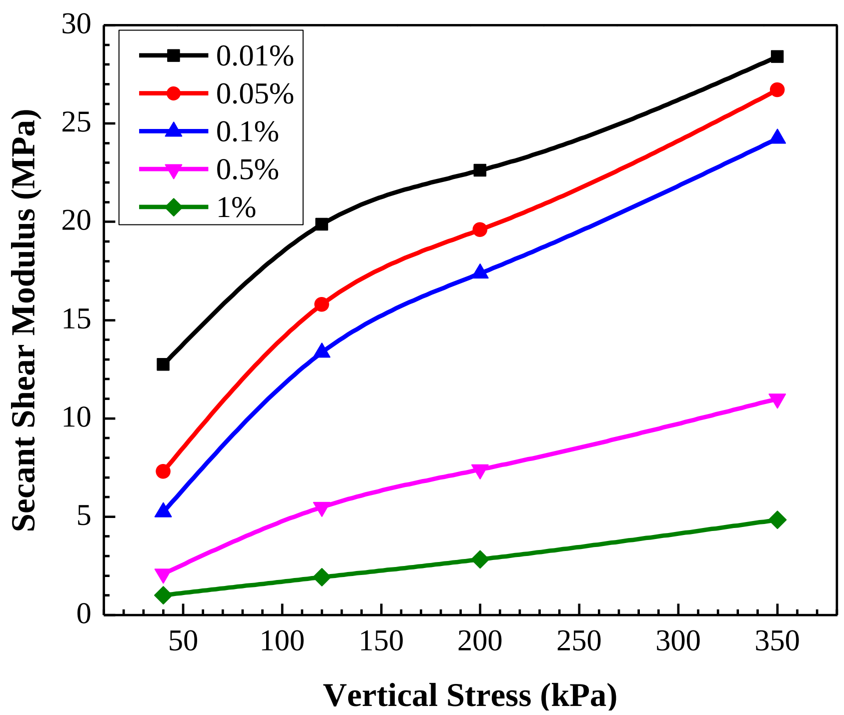


Figure 5.85: Influence of vertical stress on secant shear modulus of pond ash

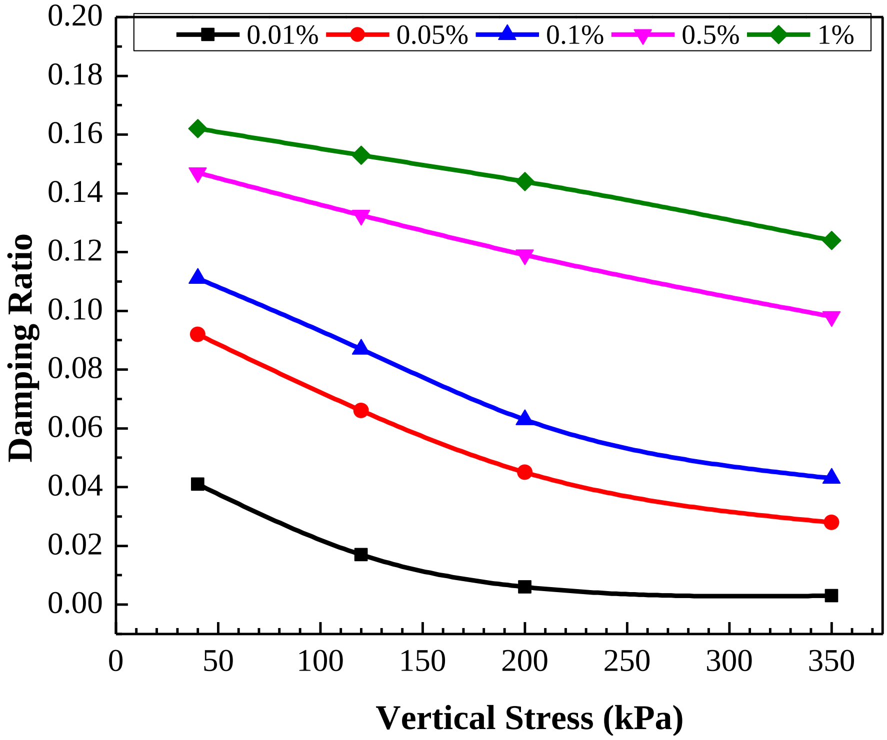


Figure 5.86: Influence of vertical stress on damping ratio of pond ash

The similar behavior, i.e., with the increase in vertical stress, secant shear modulus increases and damping ratio decreases at a particular shear strain for various tested materials like sand, clay and gravel by researchers like Silver and Seed (1971), Hardin and Drnevich (1972), Dobry and Vucetic (1987), Vucetic (1992), Ravishankar et al. (2005), etc. Table 5.6 shows the change in secant shear modulus and damping ratio of pond ash with the increase in vertical stress from 40kPa to 350 kPa at various shear strains.

Table 5.6: Influence of vertical stress on secant shear modulus and damping ratio

Parameter	**Shear Strain (%)**				
	0.01	**0.05**	**0.1**	**0.5**	**1**
% Increase in Secant Shear Modulus	55.12	72.66	78.41	80.91	79.14
% Decrease in Damping Ratio	92.68	69.56	61.26	33.33	23.46

From the table 5.6, we can conclude that with the increase in shear strain, percentage increase in the secant shear modulus will rise (from 55.12% to 79.14%) and percentage decrease in damping ratio will decline (from 92.68% to 23.46%) if vertical stress increases from 40 kPa to 350 kPa.

5.5.4. Influence of Shear Strain

The influence of shear strain magnitude on dynamic properties of pond ash are studied by considering shear strains of magnitude 0.01%, 0.05%, 0.1%, 0.5% and 1%.

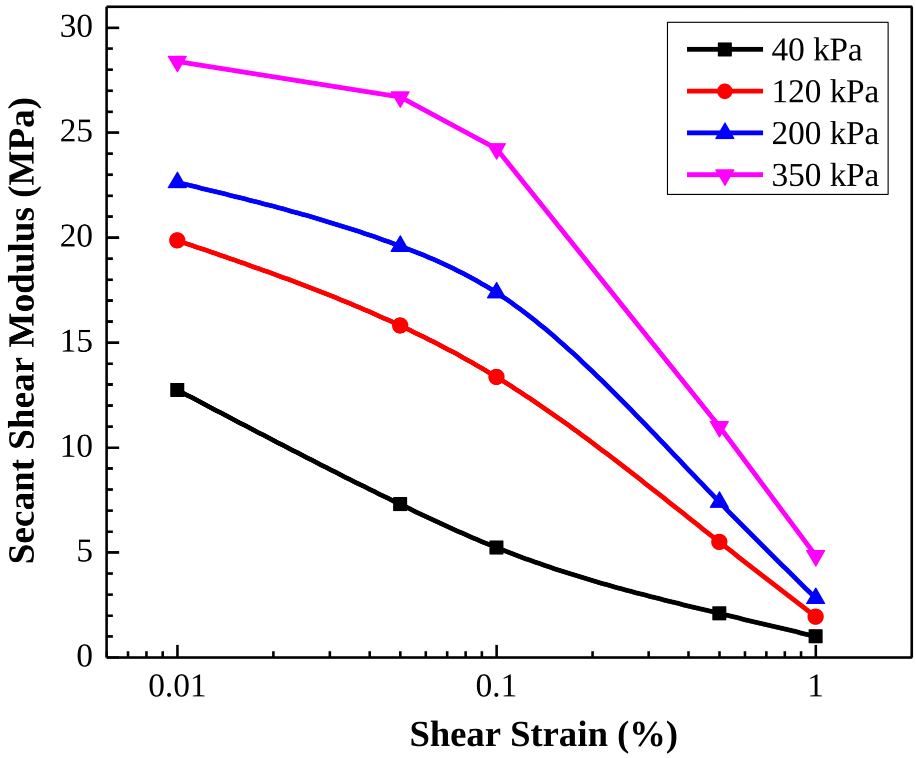


Figure 5.87: Influence of shear strain on secant shear modulus of pond ash at various vertical stresses

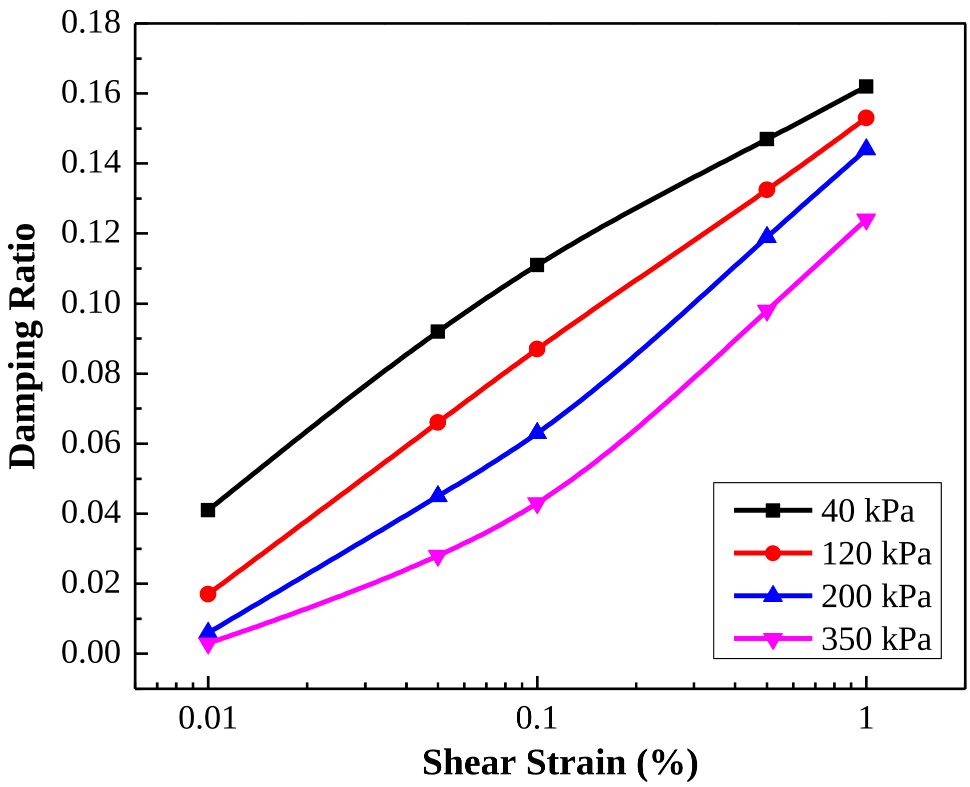


Figure 5.88: Influence of shear strain on damping ratio of pond ash at various vertical stresses

Figures 5.87 and 5.88 show the influence of shear strain on secant shear modulus and damping ratio of pond ash respectively at various vertical stresses. It can be concluded from the graphs, with the increase in shear strain, secant shear modulus decreases, and damping ratio increases. Hardin and Drnevich (1972), Dobry and Vucetic (1987), Ravishankar et al. (2005), etc., also observed similar behavior for the various tested materials like sand, clay, and gravel. Table 5.7 shows the change in secant shear modulus and damping ratio with the increase in shear strain from 0.01% to 1% at various vertical stresses.

Table 5.7: Influence of shear strain on dynamic properties of pond ash

Parameter	**Vertical Stress**			
	40 kPa	**120 kPa**	**200 kPa**	**350 kPa**
% Decrease in Secant Shear Modulus	92.07	90.26	87.5	82.95
% Increase in Damping Ratio	74.7	88.89	95.83	97.58

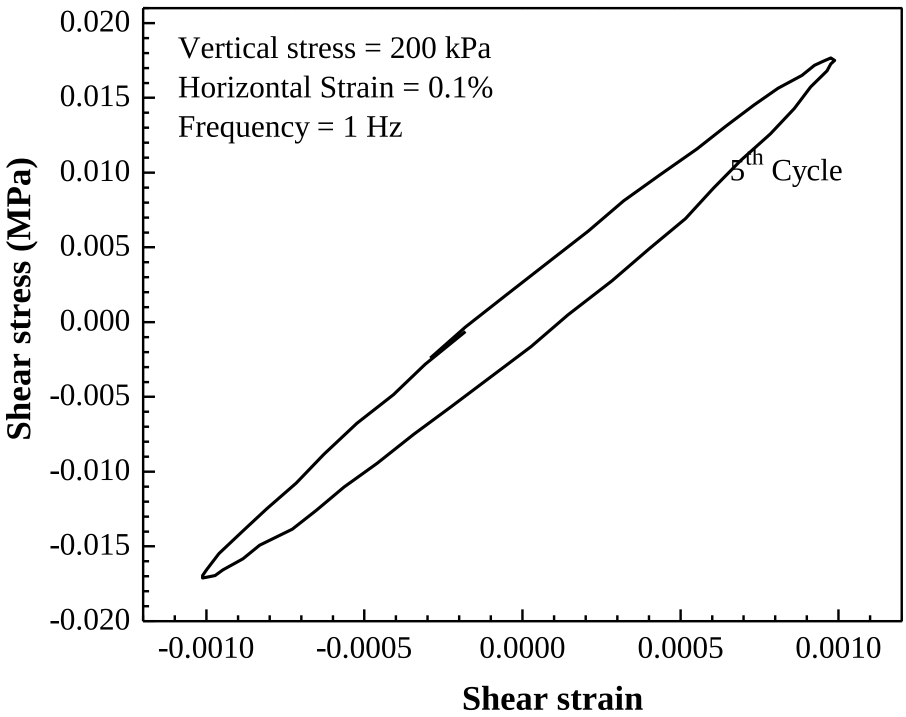


Figure 5.89: Hysteresis loop for the shear strain magnitude of 0.1%

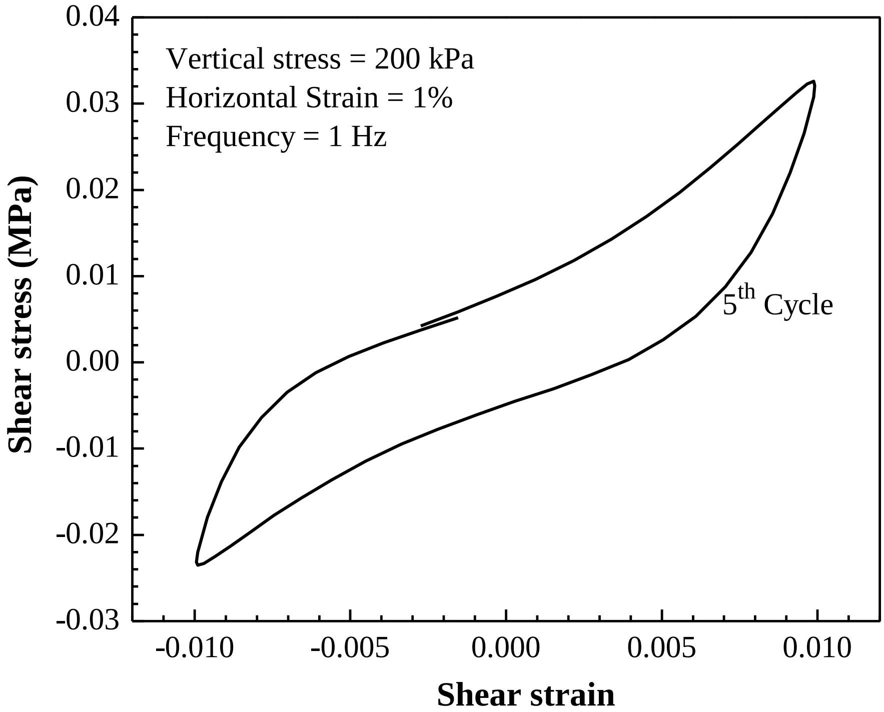


Figure 5.90: Hysteresis loop for the shear strain magnitude of 1%

It can be concluded from table 5.7, with the increase in vertical stress, percentage decrease in the secant shear modulus will reduce and percentage increase in damping ratio will escalate if shear strain increases from 0.01% to 1%. It is also observed, with the increase in the magnitude of shear strain, the hysteresis loops are becoming widened and attaining the reverse ‘S’ shaped hysteresis loop. This can be observed from the figures 5.89 and 5.90, where with the increase in shear strain magnitude from 0.1% to 1%, the hysteresis loop becomes widened and attained reverse ‘S’ shape. The figures are shown for the vertical stress of 200 kPa and similar behavior was observed at other vertical stresses also.

Mohanty and Patra (2014) studied the cyclic behavior and liquefaction potential of pond ashes collected from Talcher, Panki, and Panipat ash embankments located in India by performing cyclic triaxial tests on reconstituted pond ash samples at a relative compaction varying from 94 to 99%. They also studied the influence of loading frequency, confining pressure, and relative compaction on cyclic behavior and liquefaction potential of reconstituted pond ashes. The results indicated that maximum shear modulus (G_{max}) ranges from 10.5 MPa to 4.6 MPa over the shear strain magnitudes of 0.6% to 1.2%, tested at frequencies of 0.3 Hz and 1 Hz, at a relative compaction varying from 94 to 99% and at a confining pressure of 100 kPa for all the three pond ashes. Similarly, the damping ratio lies in between 0.09 and 0.52 for the testing conditions mentioned above. The secant shear modulus value obtained in this study at 1% shear strain is 1.01 MPa and 1.94 MPa at vertical stresses of 40 kPa and 120 kPa respectively, when tested at a frequency of 1 Hz. Secant shear modulus (G_{sec}) values obtained in this study at a frequency of 1 Hz, shear strain magnitudes in between 0.5% to 1%, and at vertical stresses of 40 kPa and 120 kPa are less than G_{max} values reported by Mohanty and Patra (2014) at a confining pressure of 100 kPa. Damping ratio values obtained in this study at a frequency of 1 Hz are 0.162 and 0.153 at vertical stresses of 40 kPa and 120 kPa respectively, which lies in the range (0.09 – 0.52) reported by Mohanty and Patra (2014) in between the shear strains of magnitude 0.6% to 1.2% at a confining pressure of 100 kPa. Therefore, results obtained in this study are in good agreement with the published results of Mohanty and Patra (2014).

5.5.5. Relationship between Secant Shear Modulus and Damping Ratio

Hardin and Drnevich (1972), Tatsuoka et al. (1978), Ishibashi (1981) and Zhang and Aggour (1996) performed extensive studies on sandy soils and proposed that the damping ratio could be expressed as a function of shear modulus. In this study, using the available data, authors tried to develop a correlation between damping ratio and secant shear modulus. Authors tried to fit a linear relationship between damping ratio (D) and G_{sec}^{b}, where ‘b’ is the exponent which is varied till a linear relationship exists between them. Equation 5.6 shows the developed correlation and the figure 5.91 shows the correlation developed between the secant shear modulus and damping ratio with a regression coefficient of 0.92.

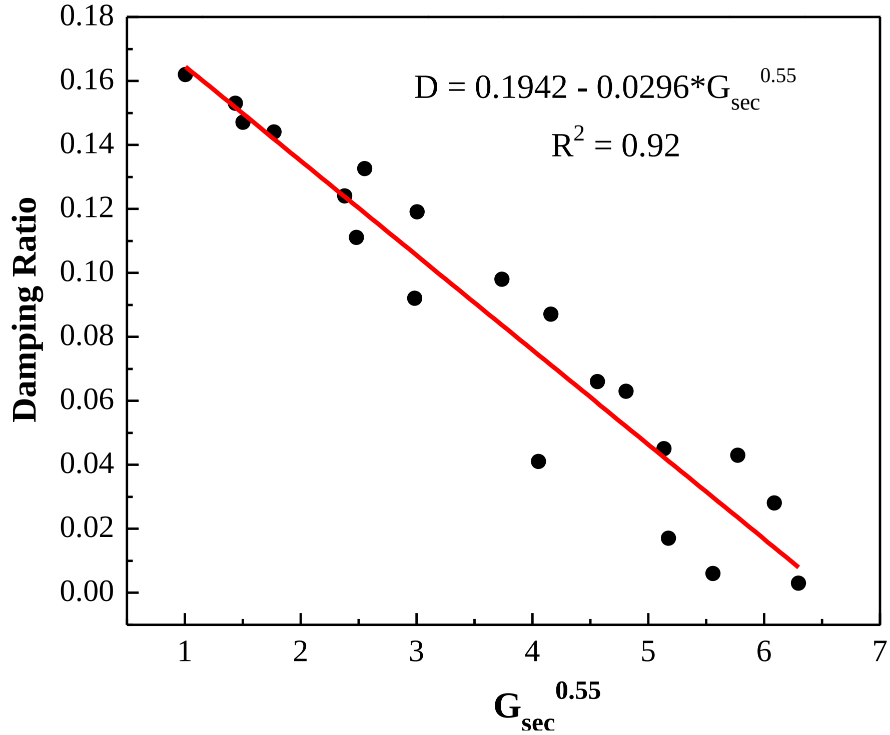


Figure 5.91: Correlation between secant shear modulus and damping ratio of pond ash

$$D = 0.1942 - 0.0296 \times G_{\sec}^{0.55} \tag{5.6}$$

Where, G_{sec} is the secant shear modulus in MPa and D is the damping ratio.

Researchers like Ohsaki and Iwasaki (1973) and etc., developed the correlations between the standard penetration test (SPT) value and shear modulus. Therefore, shear modulus can be determined easily from the various field tests, however, determining damping ratio is still tricky. The developed correlation in this study will be helpful in determining the damping ratio of pond ash from the field determined shear modulus value. However, the developed correlation is applicable only for the range of shear strains and vertical stresses considered in the study.

5.3.6. Influence of Sample Height

Two aspect ratios (D:H), 7:2.5 and 7:2 were considered with a constant diameter of 70 mm and two specimen heights, 25 mm and 20 mm respectively to study the influence of specimen dimensions on dynamic properties of pond ash. The pond ash specimens were tested at a vertical stress of 120 kPa and 1 Hz frequency. The influence of sample height on secant shear modulus and damping ratio of pond ash were shown in figures 5.92 and 5.93 respectively. It is clear from the graphs, secant shear modulus values are lower and damping ratio values are higher for 20 mm sample height than 25 mm high sample. Even though figure 5.92 looks like, with the increase in shear strain magnitude the degradation curves are approaching each other, but the percentage difference between the values is increasing. The percentage difference is 8.2% at 0.01% shear strain magnitude, 20.4% at 0.1% shear strain magnitude and 22.9% at 1% shear strain magnitude. In contrast, the rate of change in damping ratio with varying height of specimen is decreasing. The percentage difference in damping ratio between sample heights 25 mm and 20 mm is 26.1% at 0.01% shear strain magnitude, 19.44% at 0.1% shear strain magnitude and 17.3% at 1% shear strain magnitude. Therefore, the dynamic properties and correlations developed in this study are pertaining to sample height of 25 mm and 70 mm diameter.

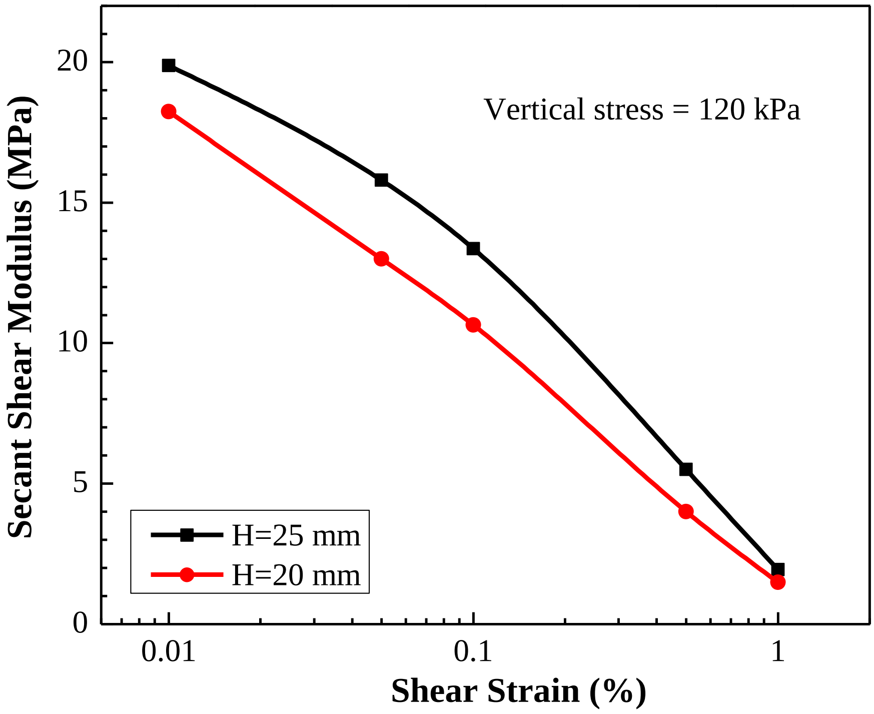


Figure 5.92: Influence of sample height on secant shear modulus of pond ash

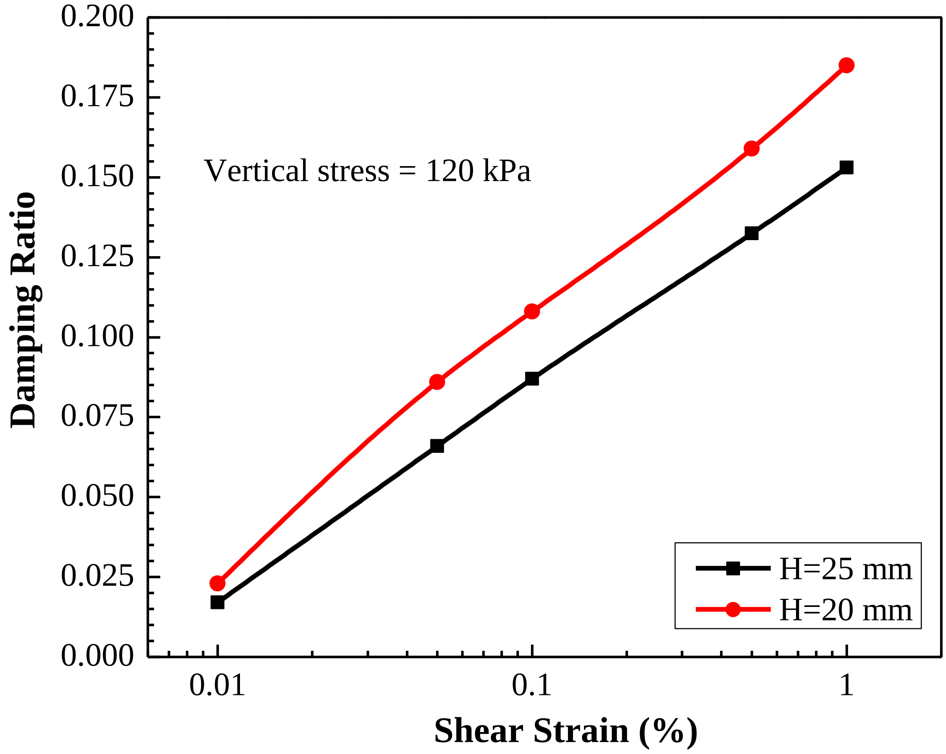


Figure 5.93: Influence of sample height on damping ratio of pond ash

5.6. Comparative Studies

5.6.1. I.S. Sand Vs. Bottom Ash

The dynamic properties obtained for both I.S. sand and bottom ash are compared and concluded that, up to medium strain levels (strain magnitude <0.1%) I.S. sand have higher secant shear modulus value compared to bottom ash and at higher strain levels (strain magnitude >0.1%) bottom ash possess higher secant shear modulus value compared to the I.S. sand. Considering damping ratio, usually I.S. sand is showing higher damping ratio values compared to bottom ash. However, up to medium strain levels (strain magnitude <0.1%) damping ratio value of bottom ash determined at vertical stresses in between 100 kPa to 400 kPa at various relative densities possess high damping ratio values rather than I.S. sand damping ratio value determined at high vertical stress (400 kPa) and at different relative densities. Figures 5.94 to 5.99 show the variation of secant shear modulus and damping ratio of both I.S. sand and bottom ash with vertical stresses at relative densities of 30%, 50% and 75% respectively. Figures 5.100 to 5.105 show the variation of secant shear modulus and damping ratio of both I.S. sand and bottom ash with relative densities at vertical stresses of 100 kPa, 200 kPa and 400 kPa respectively.

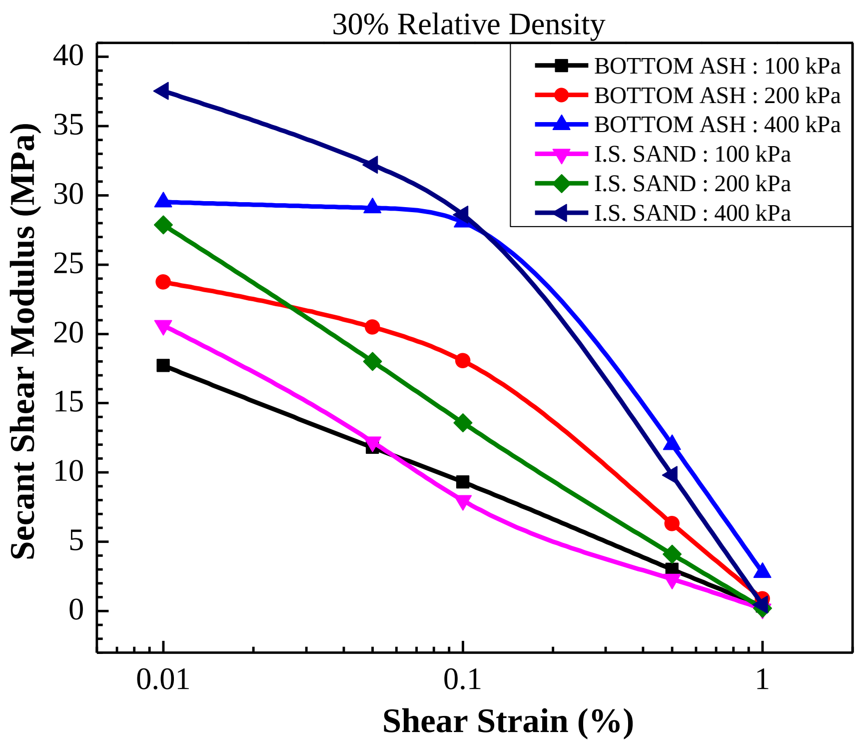


Figure 5.94: Comparison of secant shear modulus of I.S. sand and bottom ash at 30% relative density

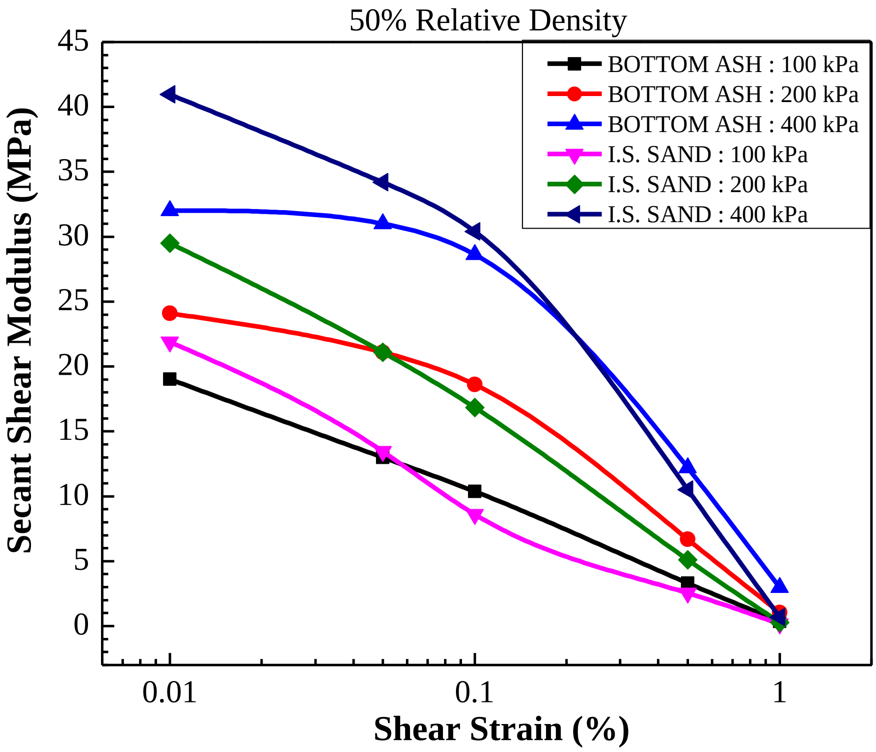


Figure 5.95: Comparison of secant shear modulus of I.S. sand and bottom ash at 50% relative density

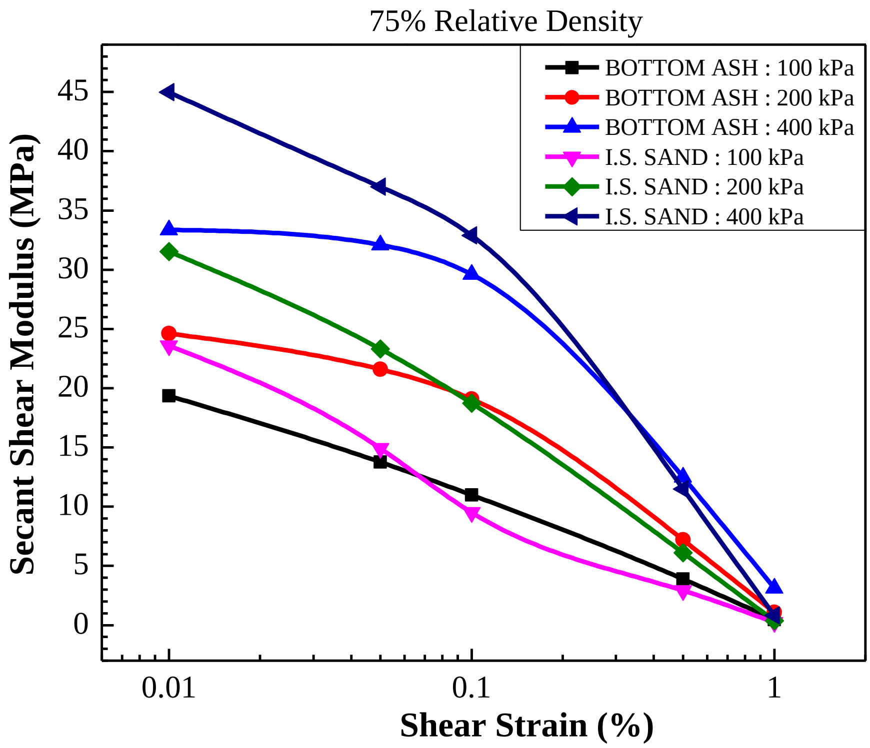


Figure 5.96: Comparison of secant shear modulus of I.S. sand and bottom ash at 75% relative density

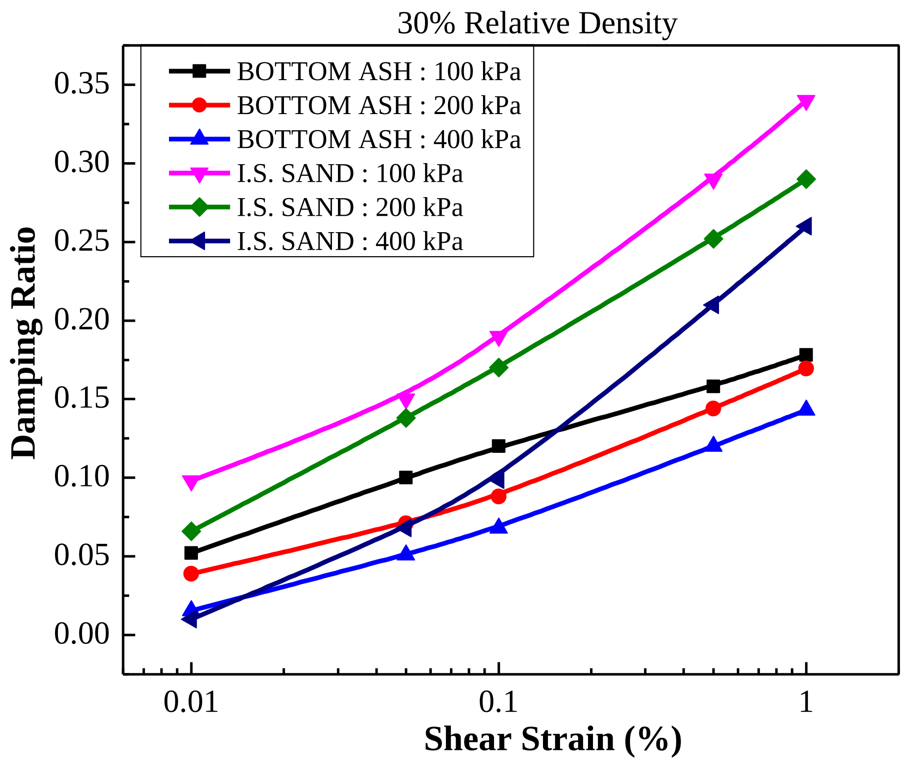


Figure 5.97: Comparison of damping ratio of I.S. sand and bottom ash at 30% relative density

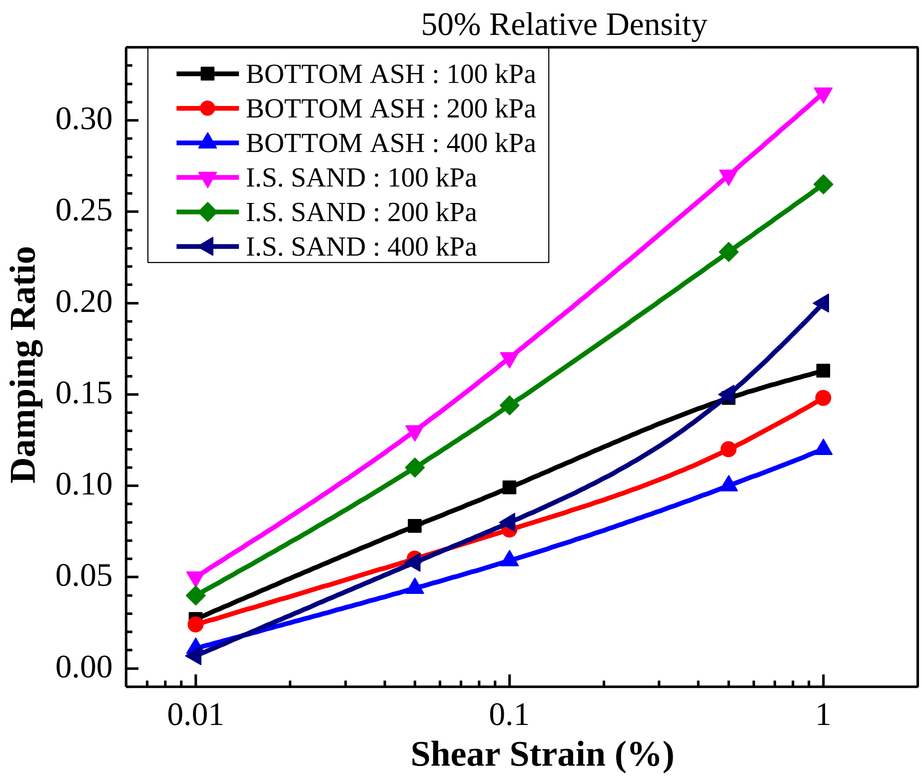


Figure 5.98: Comparison of damping ratio of I.S. sand and bottom ash at 50% relative density

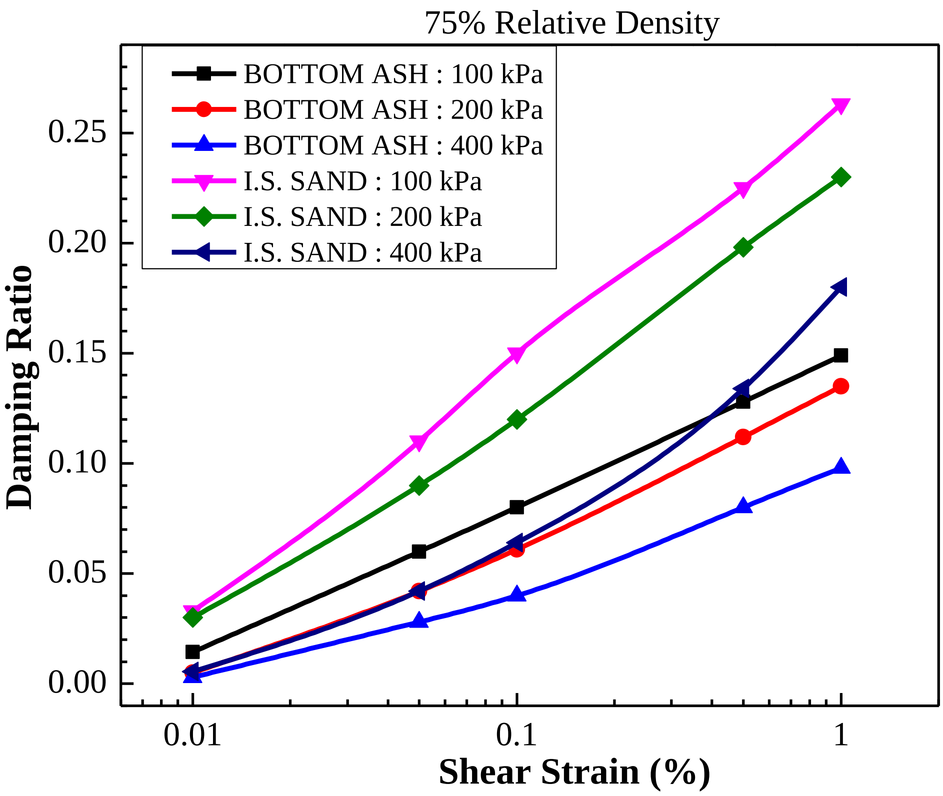


Figure 5.99: Comparison of damping ratio of I.S. sand and bottom ash at 75% relative density

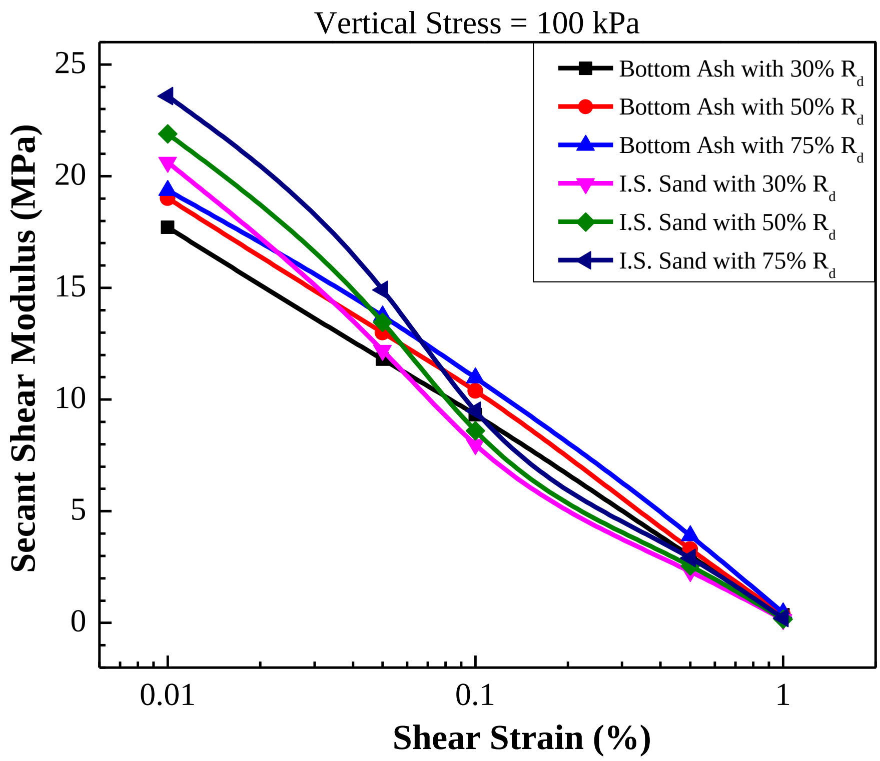


Figure 5.100: Comparison of G_{sec} of I.S. sand and bottom ash at 100 kPa vertical stress

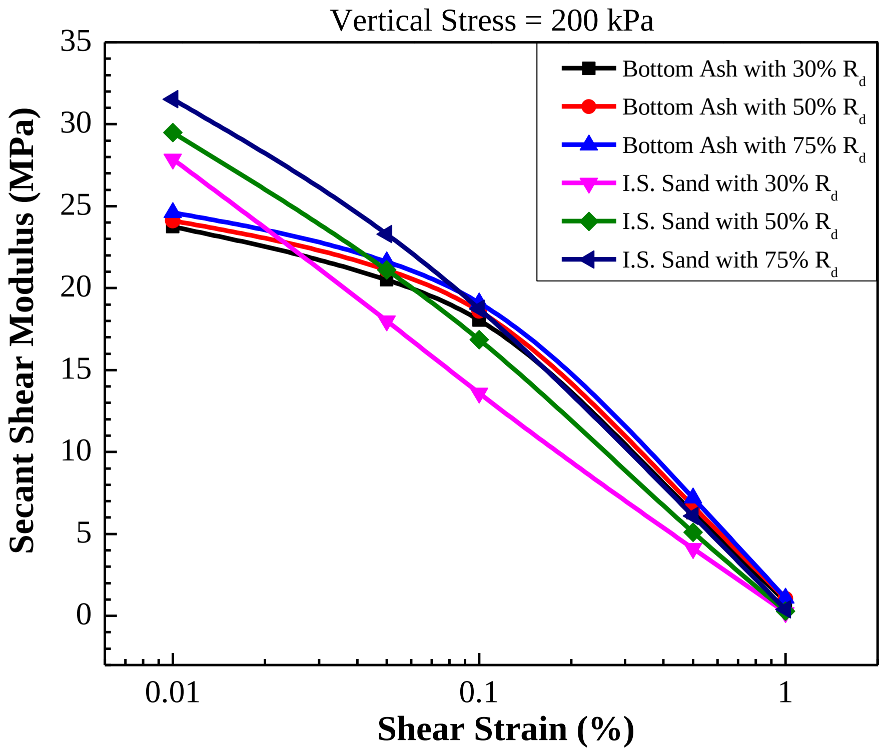


Figure 5.101: Comparison of G_{sec} of I.S. sand and bottom ash at 200 kPa vertical stress

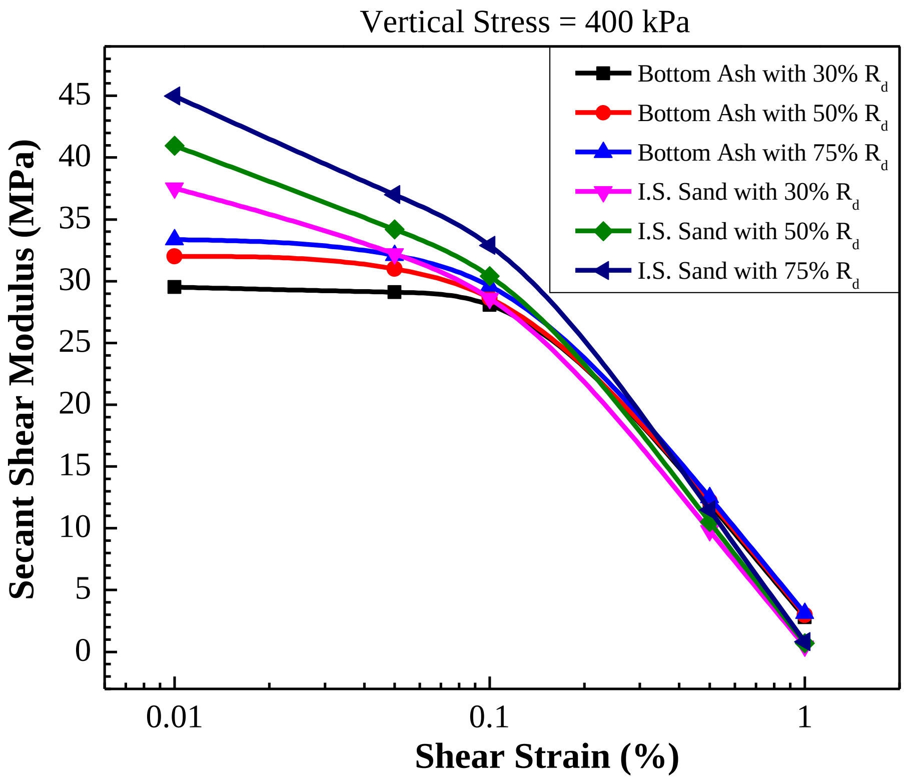


Figure 5.102: Comparison of G_{sec} of I.S. sand and bottom ash at 100 kPa vertical stress

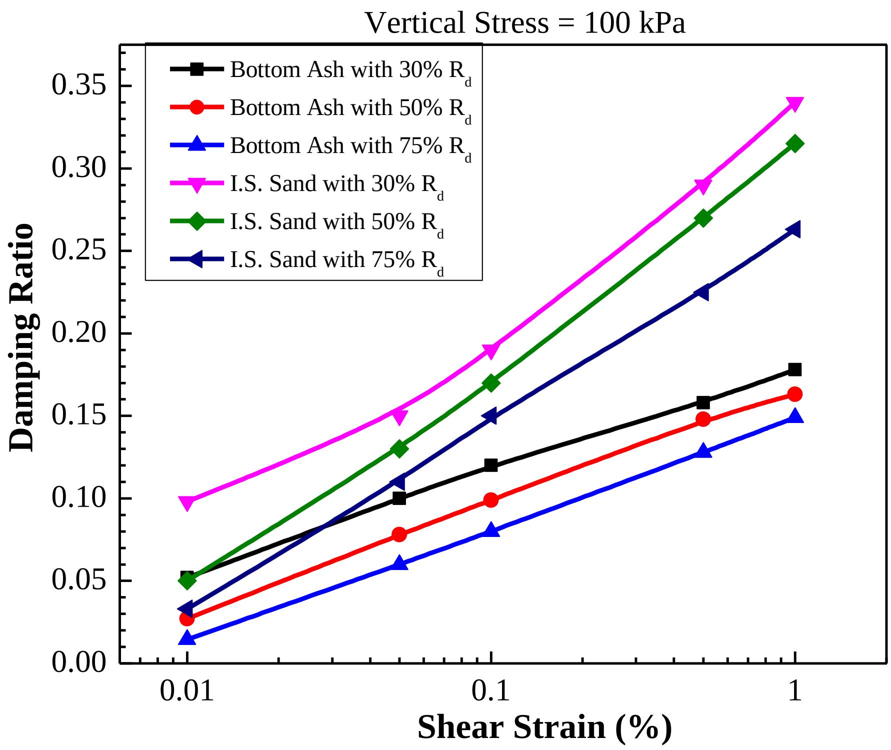


Figure 5.103: Comparison of damping ratio of I.S. sand and bottom ash at 100 kPa vertical stress

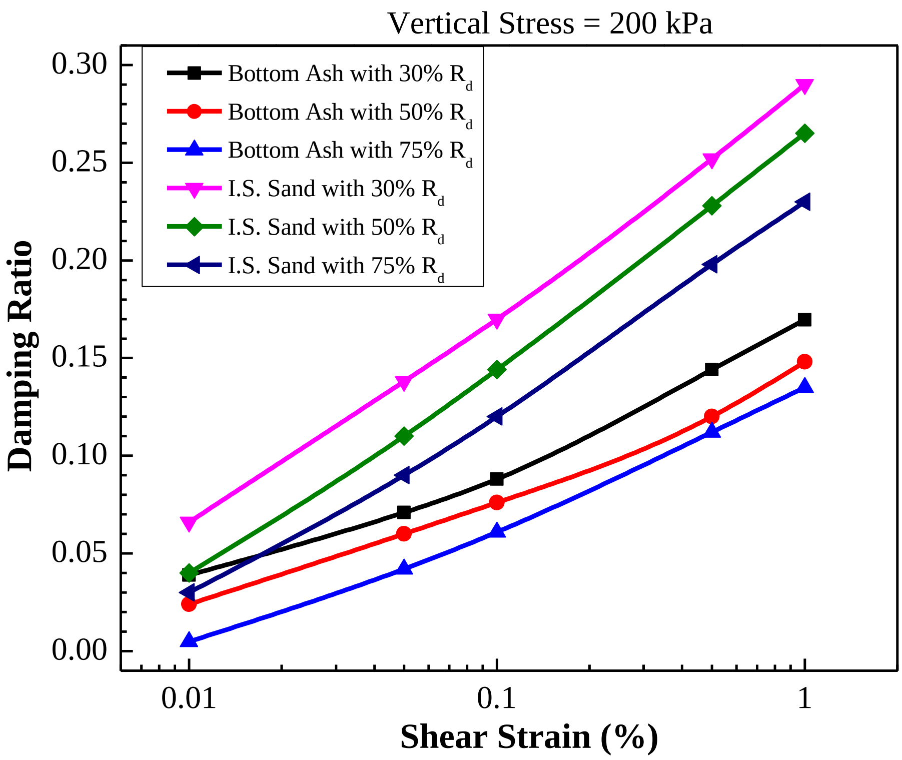


Figure 5.104: Comparison of damping ratio of I.S. sand and bottom ash at 200 kPa vertical stress

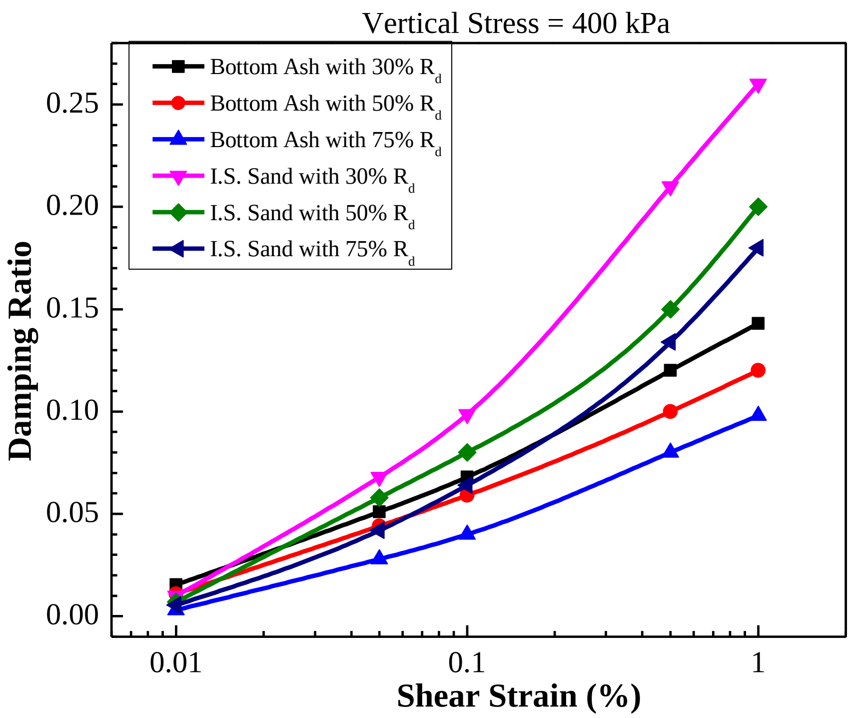


Figure 5.105: Comparison of damping ratio of I.S. sand and bottom ash at 400 kPa vertical stress

5.6.2. Comparison of Dynamic Properties of Various Ash Materials

The dynamic properties obtained for bottom ash, pond ash, and fly ash are compared at various vertical stresses. At strain magnitude of 0.01%, fly ash is exhibiting higher values of secant shear modulus compared to other ash products namely bottom ash and pond ash. But, at shear strain magnitude of 1%, pond ash is exhibiting higher values of secant shear modulus compared to remaining ash products at vertical stresses of 40 kPa, 120 kPa, 200 kPa and 350 kPa. At intermediate strain levels, the secant shear modulus of pond ash (mixture of fly ash and bottom ash) is always in between the secant shear modulus values of fly ash and bottom ash. In contrast to secant shear modulus, pond ash is exhibiting higher damping ratio values compared to fly ash and bottom ash at shear strain magnitude of 0.01% and at shear strain magnitude of 1%, fly ash possesses higher damping ratio values compared to pond ash and bottom ash. Overall, bottom ash is exhibiting lower values of damping ratio compared to fly ash and pond ash at various shear strain magnitudes and at various vertical stresses. Figures 5.106 to 5.113 show the variation of secant shear modulus and damping ratio with vertical stresses of 40 kPa, 120 kPa, 200 kPa, and 350 kPa respectively for different ash materials. From the figures, it is clear that there is considerable change in secant shear modulus values at low shear strain (0.01%) and damping ratio values at high shear strain (1%) and there is no much difference in secant shear modulus and damping ratio of various ash materials at high shear strains and at low shear strains respectively.

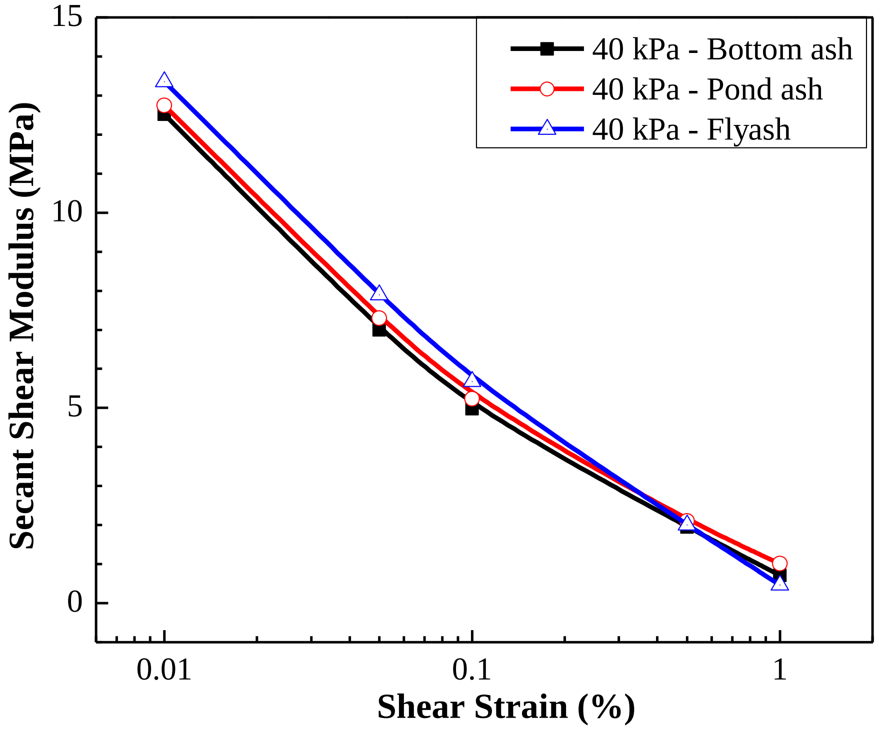


Figure 5.106: Secant shear modulus for ash materials at 40 kPa vertical stress

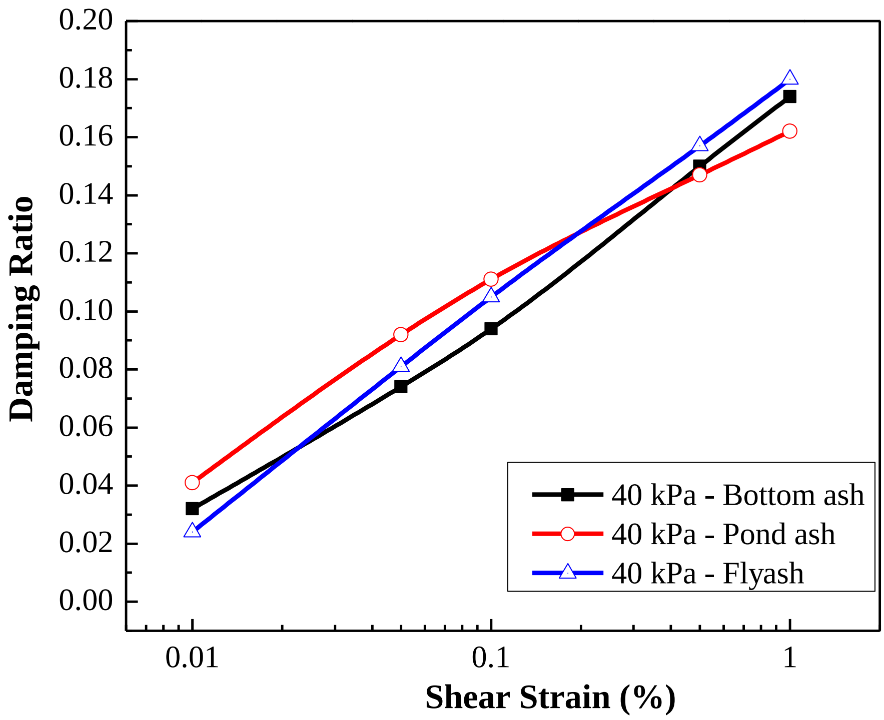


Figure 5.107: Damping ratio for ash materials at 40 kPa vertical stress

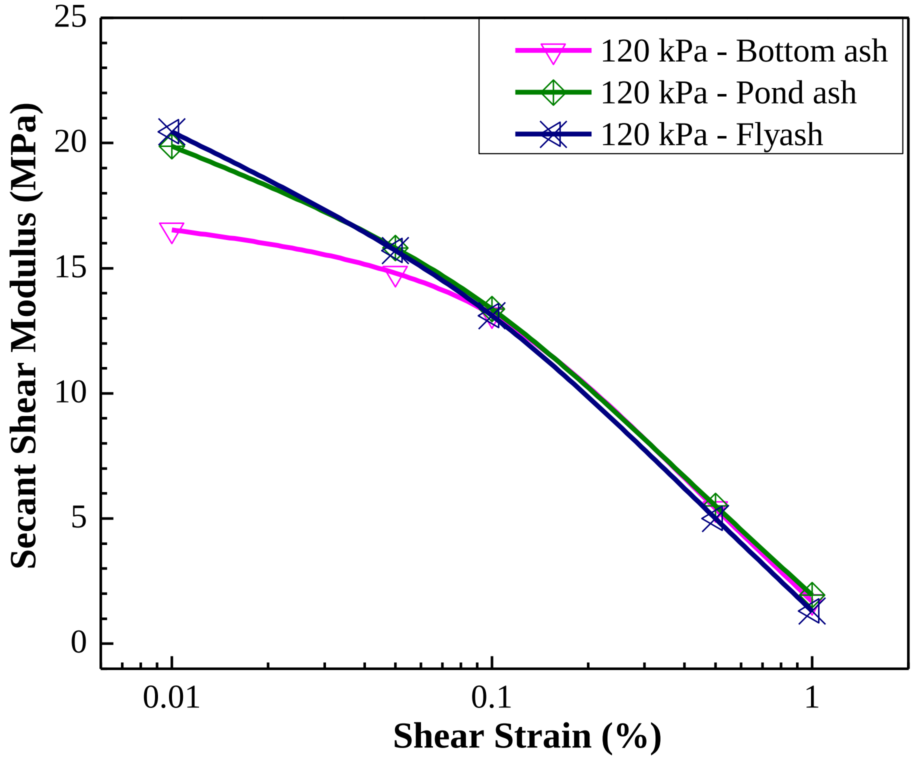


Figure 5.108: Secant shear modulus for ash materials at 120 kPa vertical stress

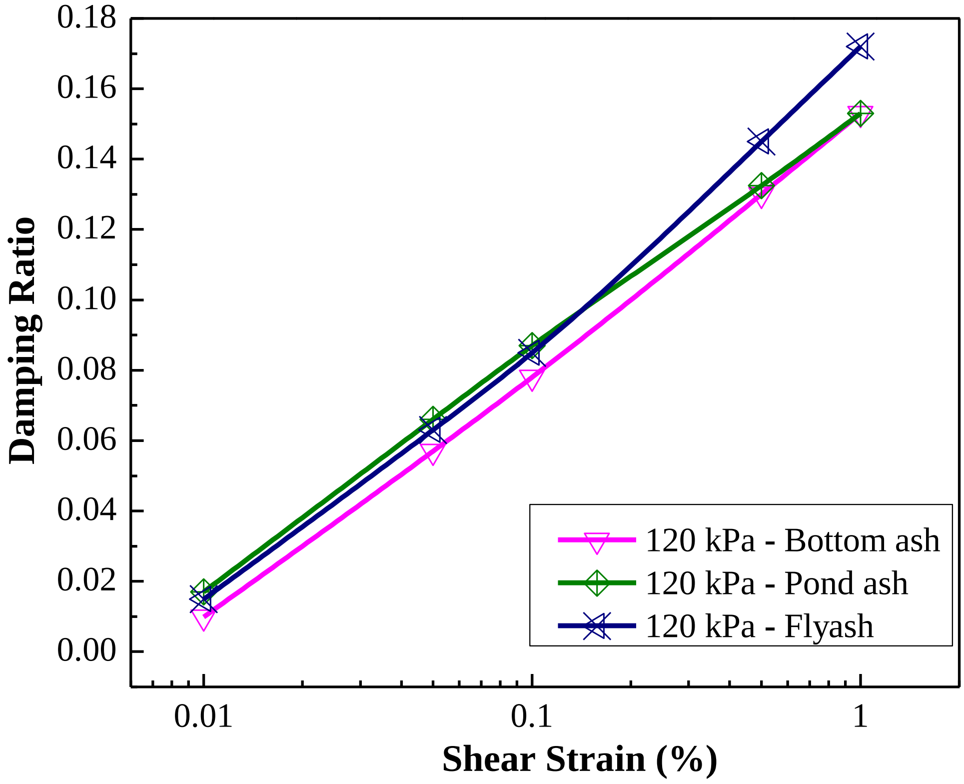


Figure 5.109: Damping ratio for ash materials at 120 kPa vertical stress

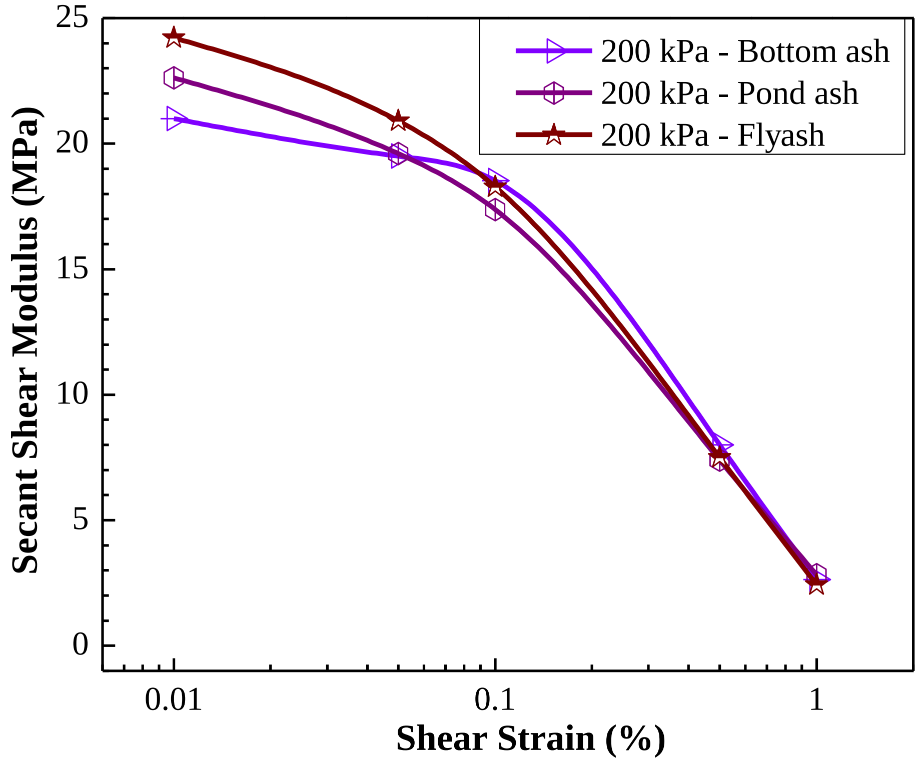


Figure 5.110: Secant shear modulus for ash materials at 200 kPa vertical stress

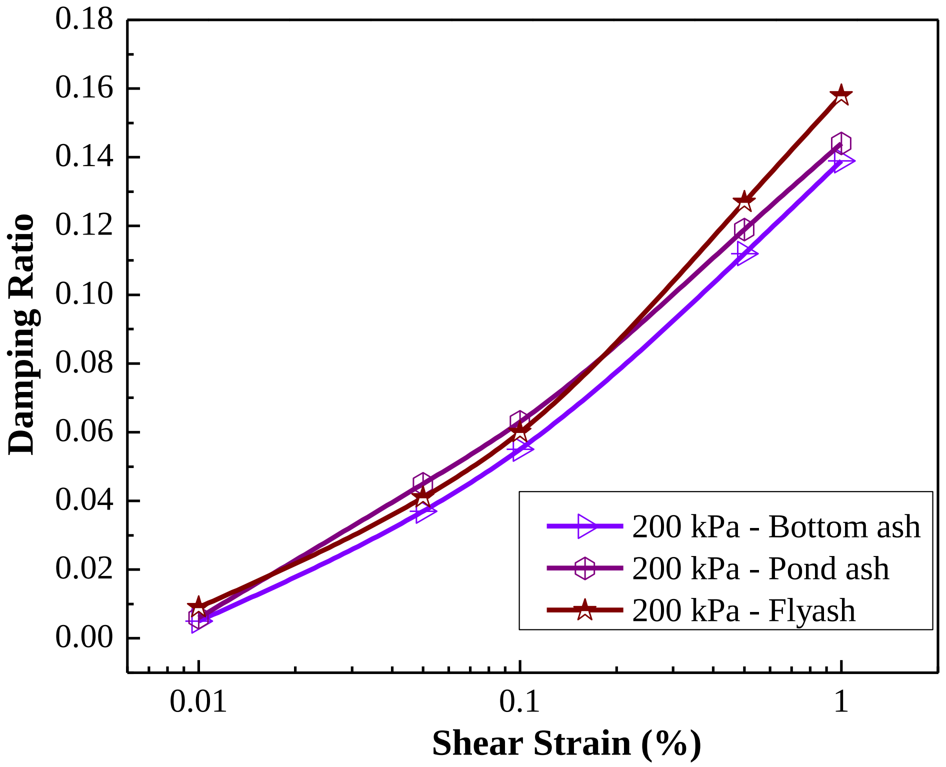


Figure 5.111: Damping ratio for ash materials at 200 kPa vertical stress

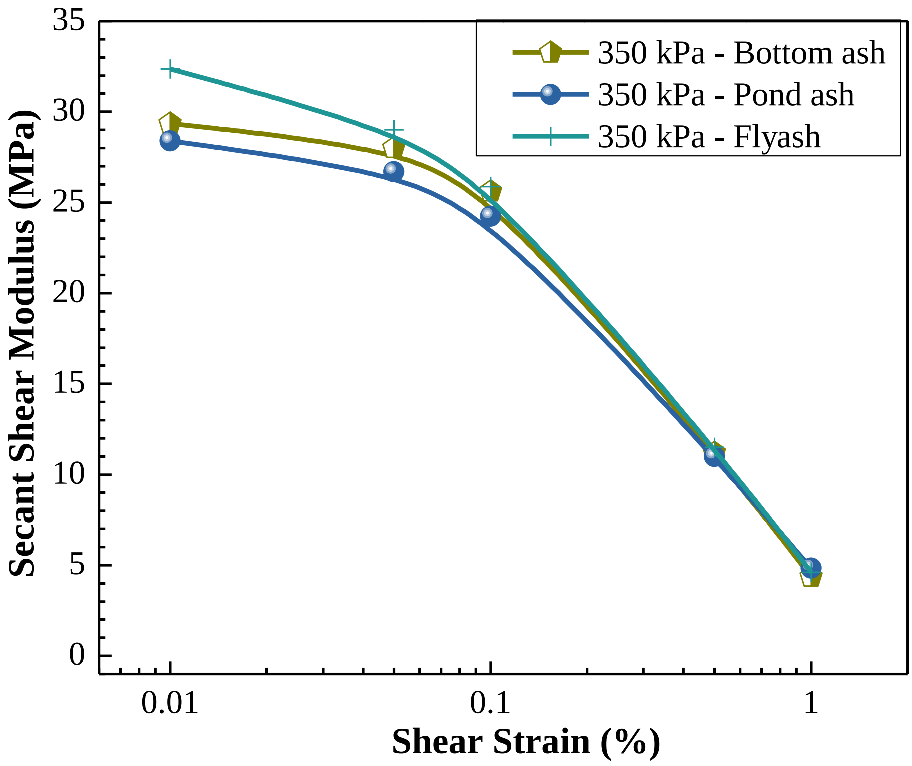


Figure 5.112: Secant shear modulus for ash materials at 350 kPa vertical stress

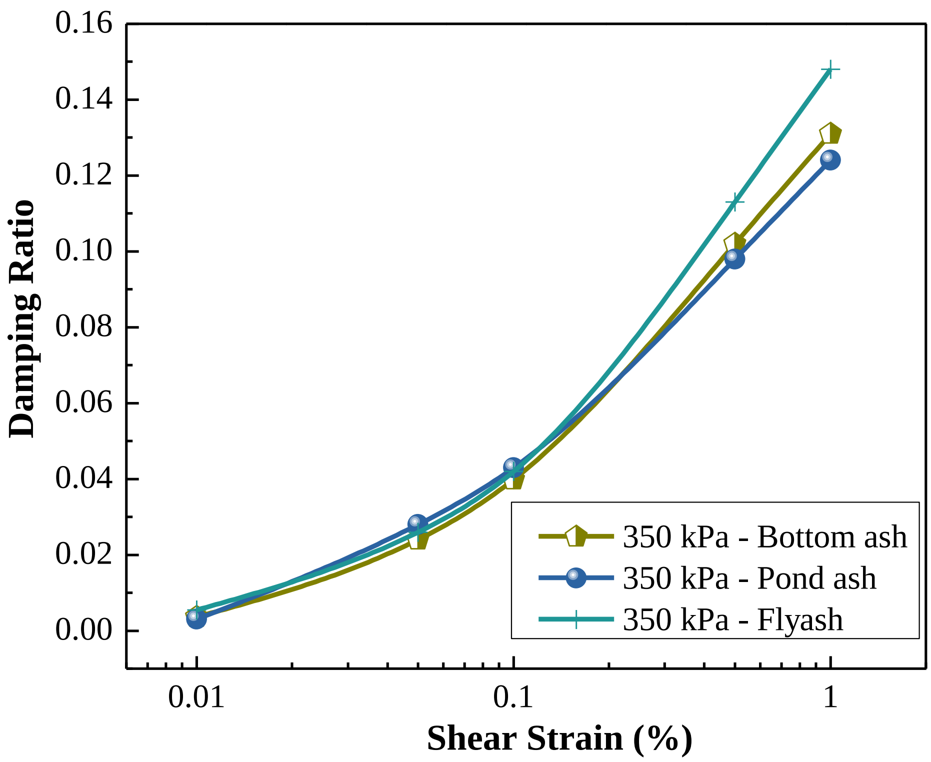


Figure 5.113: Damping ratio for ash materials at 350 kPa vertical stress

Chapter 6

Conclusions

6.1. I.S. Sand

Cyclic simple shear tests were performed on Indian Standard (I.S.) sand to determine the dynamic properties, i.e., secant shear modulus and damping ratio and the various parameters influencing the dynamic properties of I.S. sand. Sand specimens of size 70 mm in diameter and 25 mm in height are prepared at relative densities of 30%, 50% and 75% and tested at various vertical stresses of 100 kPa, 200 kPa and 400 kPa. The influence of number of cycles, frequency of loading, shear strain, grain size, vertical stress and relative density on dynamic properties of I.S. sand were studied. The following are the major conclusions of this study:

- The influence of number of sinusoidal loading cycles on dynamic properties of I.S. sand was studied by subjecting the sample to 50 cycles and results indicated that the secant shear modulus of I.S. sand decreases with the increase in number of loading cycles and the decrease is exceptionally high at first few cycles. In contrast, the damping ratio of I.S. sand increases with the increase in number of loading cycles but the variation is not as much as the secant shear modulus.
- Frequency at which specimen was tested varied from 0.1 Hz to 2 Hz to study the influence of frequency on dynamic properties of I.S. sand and it can be concluded from the results that frequency of load application has no significant effect on secant shear modulus but the damping ratio varies considerably with the change in frequency of loading.
- Dynamic properties of I.S. sand are highly influenced by the shear strain magnitude and it is considered as most influencing factor compared to other factors. Shear modulus decreases intensely and damping ratio increases with the increase in magnitude of shear strain, at a particular vertical stress and relative density.

- Grain size of the sand has no significant impact on the secant shear modulus values but damping ratio is varying considerably with the change in grain size of the testing material.
- With the increase in relative density of sample, shear modulus increases and damping ratio decreases at a particular vertical stress. Similarly, with the increase in vertical stress, shear modulus increases and damping ratio decreases at a particular relative density. However, the change in dynamic properties value due to change in vertical stress is higher than the change in the relative density of the sample prepared. In other words, vertical stress is the significant factor influencing dynamic properties of I.S. sand compared to the relative density.
- A linear relationship was developed between normalized shear modulus and damping ratio of I.S. sand with a regression coefficient of 0.80 for the range of shear strain magnitudes, relative densities and vertical stresses considered in this study. This correlation will be useful in the determination of damping ratio from field measured shear modulus values.

6.2. Bottom Ash (Dry)

The dynamic behavior of bottom ash collected from NTPC Ramagundam, India was studied by performing dynamic cyclic simple shear tests on bottom ash, as bottom ash is widely used as a fill material for retaining walls, embankments, for mine fillings and in pavements sub-base layer. Tests were performed on bottom ash samples to determine the dynamic properties, i.e., secant shear modulus and damping ratio and the various parameters influencing the dynamic properties of bottom ash. Bottom ash specimens of size 70 mm in diameter and 25 mm in height were prepared at relative densities of 30%, 50% and 75% and tested at various vertical stresses of 100 kPa, 200 kPa and 400 kPa. The influence of number of cycles, frequency of loading, shear strain, grain size, vertical stress and relative density on dynamic properties of bottom ash were studied. The following are the major conclusions of this study:

- Bottom ash specimens were subjected to 50 cycles of sinusoidal loading to determine the influence of number of cycles and results indicated that the secant shear modulus of bottom ash decreases with the increase in number of loading cycles and in contrast, the damping ratio of bottom ash more or less remains constant with the increase in number of loading cycles.
- Bottom ash specimens were tested over a frequency range of 0.1 Hz to 2 Hz to study the influence of frequency on dynamic properties of bottom ash and it can be

concluded from the results that frequency of load application has no significant effect on secant shear modulus but the damping ratio varies considerably with the change in frequency of loading.

- Analyzing the data, shear strain magnitude is considered as the most significant factor influencing the dynamic properties of bottom ash. Secant shear modulus decreases intensely and damping ratio increases with the increase in magnitude of shear strain, at a particular vertical stress and relative density. The hysteresis loop is attaining the shape of reverse 'S' shape hysteresis loop with the increase in shear strain magnitude.
- The increase in vertical stress on bottom ash specimen prepared at a particular relative density causes secant shear modulus to increase and damping ratio to decrease. Similarly, with the increase in relative density, secant shear modulus increases and damping ratio decreases when tested at a particular vertical stress. However, the change in dynamic properties value due to change in vertical stress is higher than the change in the relative density of the sample prepared. In other words, vertical stress is the most significant factor compared to the relative density.
- A linear relationship was developed between secant shear modulus and damping ratio of bottom ash with a regression coefficient of 0.80 for the range of shear strain magnitudes, relative densities and vertical stresses considered in this study. This correlation will be useful in the determination of damping ratio from field measured shear modulus values.
- Two sample heights, 25 mm and 20 mm are considered for determining the influence of sample height, keeping the diameter as constant (70 mm) and results concluded that the secant shear modulus and damping ratio determined in this study are found to be dependent on sample height.

6.3. Neyveli Fly Ash

The present work highlighted the importance of determining dynamic properties of fly ash and the advantages of cyclic simple shear testing for determining dynamic properties compared to other equipments. Fly ash specimens prepared at optimum moisture content and maximum dry density are subjected to wide range of shear strains (0.01% to 1%), vertical stresses (40 kPa to 350 kPa) to determine secant shear modulus and damping ratio. The influence of number of cyclic loadings, frequency, sample size on the dynamic properties are also studied. The following are the major conclusions of this study:

- Secant shear modulus will decrease with the number of loading cycles and the decrease will pronounciate higher at first few cycles and there will be no significant effect of number of loading cycles on the damping ratio of fly ash.
- The frequency at which specimen was tested has no much influence on secant shear modulus of fly ash. However, the damping ratio varies with the frequency at which specimen was tested.
- The increase in vertical stress will result in the increase of secant shear modulus and decrease of the damping ratio at a particular shear strain. The increase of secant shear modulus and decrease of damping ratio with the increase in vertical stress is higher at shear strains of 0.01% to 0.1% compared to the strains of magnitude greater than 0.1%.
- Among various factors influencing dynamic properties of fly ash, shear strain magnitude is considered as most significant factor. At a particular vertical stress, with the increase in shear strain secant shear modulus decreases and damping ratio increases. With the increase in vertical stress, the percentage decrease in the secant shear modulus will diminish and the percentage increase in damping ratio will escalate if shear strain increases from 0.01% to 1%.
- The correlation was developed between secant shear modulus and damping ratio of fly ash with a regression coefficient of 0.90 for the range of shear strains and vertical stresses considered in the study. This equation will help in the easy determination of damping ratio from field measured secant shear modulus values.
- Influence of sample height on the dynamic properties of fly ash was studied and found that there will be considerable difference in values of secant shear modulus and damping ratio in between the strains of magnitude 0.05% to 0.5% by changing the sample height from 25 mm to 20 mm, maintaining the constant diameter of 70 mm.

6.4. Ramagundam Bottom Ash

In the present study, the importance of determining dynamic properties of bottom ash and the advantages of cyclic simple shear testing for determining dynamic properties compared to other equipments are discussed. In order to determine the dynamic properties, bottom ash specimens (prepared at optimum moisture content and maximum dry density) are subjected to shear strains of magnitude ranging from 0.01% to 1%, vertical stresses in between 40 kPa to 350 kPa. Apart from that, the influence of number of loading cycles, frequency, sample size on secant shear modulus and damping ratio are also studied. The following are the foremost conclusions of this study:

- The number of loading cycles has no significant influence on damping ratio of bottom ash. However, with the increase in number of loading cycles, secant shear modulus will decrease and the decrease will pronounciate higher at first few cycles.
- Secant shear modulus is not changing much with the change in frequency at which specimen was tested, but the damping ratio of bottom ash varies with the frequency.
- For a specimen tested at a particular shear strain, secant shear modulus is increasing and damping ratio is decreasing for the tested bottom ash with the increase in vertical stress. The change in dynamic properties is much prominent at shear strains of magnitude 0.01% to 0.1% compared to the strains of magnitude greater than 0.1%.
- Dynamic properties of bottom ash are highly influenced by shear strain and it is considered as most influencing factor compared to other factors. Secant shear modulus decreases dramatically and damping ratio increases with the increase in magnitude of shear strain, at a particular vertical stress. With the increase in vertical stress, the percentage decrease in the secant shear modulus will diminish and the percentage increase in damping ratio will escalate if shear strain increases from 0.01% to 1%.
- In order to determine the damping ratio from field measured shear moduls, a correlation was developed between secant shear modulus and damping ratio of bottom ash with a regression coefficient of 0.87 for the range of shear strains and vertical stresses considered in the study.
- Two sample heights, 25 mm and 20 mm are considered for determining the influence of sample height, keeping the diameter as constant (70 mm) and results concluded that the secant shear modulus and damping ratio determined in this study are found to be dependent on sample height.

6.5. Ramagundam Pond Ash

This study aims at determining the dynamic properties (secant shear modulus and damping ratio) of pond ash, as the pond ash is widely used as a fill material in embankments and retaining walls, mine filling, sub-base material in pavements, etc., applications in geotechnical engineering. This study also highlighted the advantages of dynamic cyclic simple shear testing over regular testing methods like cyclic triaxial, resonant column, etc. Pond ash specimens (prepared at optimum moisture content and maximum dry density) are subjected to vertical stresses in between 40 kPa to 350 kPa and shear strains of magnitude ranging from 0.01% to 1% to develop modulus degradation curves and damping ratio curves for pond ash. In addition, the influence of number of loading cycles, frequency, sample size on secant shear modulus and damping ratio are also studied. The following are the major conclusions of this study:

- Secant shear modulus of pond ash decreases with the increase in number of sinusoidal loading cycles and the decrease is exceptionally high at first few cycles. However, the number of loading cycles has no significant influence on damping ratio of pond ash.
- Frequency of load application has no significant effect on secant shear modulus of pond ash. But, the damping ratio varies considerably with the change in frequency of loading.
- From the results, it can be concluded that the secant shear modulus increases and damping ratio decreases with the increase in vertical stress at a particular shear strain magnitude. The change in dynamic properties is much prominent at medium shear strains of magnitude (0.01% to 0.1%) compared to the higher magnitudes of shear strain (>0.1%). It is also concluded, with the increase in shear strain, percentage increase in the secant shear modulus will increases and percentage decrease in damping ratio will decreases if vertical stress increases from 40 kPa to 350 kPa.
- Similar to conventional natural geotechnical materials (sand, clay and gravel), dynamic properties of pond ash are highly influenced by shear strain and it is considered as most influencing factor compared to other factors. Secant shear modulus decreases intensely and damping ratio increases with the increase in magnitude of shear strain, at a particular vertical stress. With the increase in vertical stress, the percentage decrease in the secant shear modulus will reduce and the percentage increase in damping ratio will increase if shear strain increases from 0.01% to 1%.
- A correlation was developed between secant shear modulus and damping ratio of pond ash with a regression coefficient of 0.92 for the range of shear strains and vertical stresses considered in this study. This correlation will be useful in the determination of damping ratio from field measured shear modulus. This equation is valid for the range of shear strain magnitudes and vertical stresses considered in this study.

- Two aspect ratios (D:H), 7:2.5 and 7:2 are considered to study the influence of sample dimensions on dynamic properties of pond ash. The results indicated that, with the reduction in sample height to 20 mm, secant shear modulus values will be lower and damping ratio values will be higher than 25 mm sample height, considering constant diameter in both cases (70 mm). Therefore, dynamic properties and correlation developed in this study are suitable only for the sample dimensions of 25mm height and 70 mm diameter.

6.6. Comparative Studies

(a) **I.S. Sand and Bottom Ash**

- I.S. sand have higher secant shear modulus value compared to bottom ash up to medium strain levels (strain magnitude <0.1%) and at higher strain levels (strain magnitude >0.1%) bottom ash possess higher secant shear modulus value compared to the I.S. sand.
- I.S. sand is showing higher damping ratio values compared to bottom ash. However, up to medium strain levels (strain magnitude <0.1%) damping ratio value of bottom ash determined at vertical stresses in between 100 kPa to 400 kPa at various relative densities possess high damping ratio values rather than I.S. sand damping ratio value determined at high vertical stress (400 kPa) and at different relative densities.

(b) Among Ash Materials

- At strain magnitude of 0.01%, fly ash is exhibiting higher values of secant shear modulus compared to other ash products namely bottom ash and pond ash.
- At shear strain magnitude of 1%, pond ash is exhibiting higher values of secant shear modulus compared to remaining ash products at vertical stresses of 40 kPa, 120 kPa, 200 kPa and 350 kPa.
- At intermediate strain levels, the secant shear modulus of pond ash (mixture of fly ash and bottom ash) is always in between the secant shear modulus values of fly ash and bottom ash.
- Pond ash is exhibiting higher damping ratio values compared to fly ash and bottom ash at shear strain magnitude of 0.01% and at shear strain magnitude of 1%, fly ash possesses higher damping ratio values compared to pond ash and bottom ash.
- Bottom ash is exhibiting lower values of damping ratio compared to fly ash and pond ash at various shear strain magnitudes and at various vertical stresses.

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