STRAIN-RATE INFLUENCE ON SHEAR STRENGTH CHARACTERISTICS OF COMPACTED FADAMA CLAY

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ABSTRACT

This study is concerned with the investigation of strain-rate effects on shear strength characteristics of saturated compacted Fadama Clay. The experimental program consisted of consolidated undrained triaxial test with controlled strain-rate. The rates of strain adopted are 0.0024, 0.006, 0.012 and 0.03 in/min. (0.08, 0.2, 0.4 and 1.0 %/min., respectively). This study mainly attempts to focus attention on strain-rate effects on rate of strength increase with respect to faster rates of deformation, strain at failure, pore pressure development and strength parameters. A mathematical equation is derived which best represents the relationship of the above parameters and rate of strain. In the analysis of test results, a direct functional relationships such as (1) strength increases with increasing rate of strain, (2) strength decreases with increasing time to failure, (3) failure strain increases with increasing rate of strain, (4) pore pressure increases with increasing time to failure and (5) pore pressure decreases with increasing rate of strain were obtained. A .simple method of Rate Process Theory is

described using Scott and Ko method (1968) for various cell pressures of 10, 30 and 45 psi. (68.95, 206.85 and 310.28 kN/m^2). The method used in the determination of the mineralogy of Fadama Clay is also described. Observations obtained in this study point to the fact that the properties of Shika Fadama Clay can be improved under proper conditions of compaction.

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CHAPTER 1

BACKGROUND OF THE STUDY

1.1 INTRODUCTION

Clays occur very widely in nature and are intensively used both directly and indirectly in many branches of engineering, as man extends his control over the world around him. The study of the properties and behaviours of the different clays is particularly important in rapidly developing countries where construction projects and highways are being built in large numbers. To point out these properties and behaviours, the writer has chosen a Fadama Clay, a common type of clay in Northern Nigeria, as a basis for determining the effect of strain-rate on the shear strength of this type of clay when saturated. Just like any other types of clay, the Fadama Clay undergoes various changes that it ultimately has little resemblance to its undisturbed state when used as raw material for construction. One of the processes of improving the strength characteristics of clay in general, is by compaction and this process is used in this study. Compacting the clay to its maximum dry density at optimum moisture content improves the

properties of the clay and make it suitable for the support of static loads as in building foundations, bridge abutments, dams and embankments, and for the support of moving loads and dynamic loads as in highways and airfields.

Furthermore, clay could be used in the construction of fills. Fills are used in building construction to provide level sites. If properly constructed, they can support heavy structures with safety and only nominal settlement. Fills are placed behind retaining walls to secure the required ground surface contour and to bridge the gap between the wall and the original soil.

Pavements themselves are sometimes constructed with clay. In cases such as this, the clay must be modified with admixtures and binders to give the pavements sufficient strength and resistance to deterioration. In many countries, earth is used in the construction of walls. Earth buildings are practical in areas where cheap construction labor and abundant supplies of clay and sand are available. They (earth buildings) are durable when well constructed and protected from rain wash and flooding.

1.2 PURPOSE OF THE STUDY

The purpose of this study is to investigate the rate

of strain effects on shear strength characteristics of one common clay in Northern Nigeria known as Fadama Clay. This investigation emphasises the need for a better understanding of time-rate effects on shear strength of this type of clay, since the results would be of considerable value in determining its behaviour. Consolidated undrained triaxial shear tests were carried out on clay samples compacted to their maximum dry density at optimum moisture content. These tests were carried out to analyse the influence of strain-rates of 0.0024, 0.006, 0.012 and 0.03 in/min. (0.08, 0.2, 0.4 and 1.0 %/min. respectively) on the following: (1) rate of strength increase, (2) strain at failure, (3) pore water pressure and (4) strength parameters, c' (effective cohesion) and ϕ' (effective angle of internal friction). Also, this investigation would indicate whether functional relationships exist for any of the above parameters and rate of strain. The investigation is also expected to solve other construction problems, such as stability of slopes and design of highway and airfield pavements.

1.3 REVIEW OF RELATED LITERATURE

1.3.1 Strain-Rate Effect On Shear Strength.

Early attempts of measuring the effect of rate of

loading on the shear strength of clays was carried out by the following researchers: Casagrande and Shannon (1948), Taylor (1948), Casagrande and Hirschfeld (1951), Tschebotarioff, Ward and de Phillippe (1953), Leonards (1955), Taylor (1957), Whitman (1957), Bjerrum, Simons and Torblaa (1958), Burmister (1959), Bishop, et. al. (1960), Bjerrum, Simons and Torblaa (1960), Casagrande and Hirschfeld (1960), Henkel (1960), Lowe and Karafiath (1960), Folque (1961), Healy (1963), Richardson and Whitman (1963), Ladd (1964), Olson and Parola (1967), Lee (1968), Daniel and Olson (1974), Sudhindra and Melkote (1974), Akai, Adachi and Ando (1975) and Balasubramanian (1975).

Casangrade and Shannon (1948) studied the effect of rate of loading on strength in connection with slope stability along the Panama Canal. From the results of unconfined and triaxial compression test, they concluded that the compressive strength of clay and shale loaded to failure at about 20 *mm/sec* were between 1.5 and 2.0 times the static (10 min.) strength. They also found that the compressive strength continuously increased with increased rate of loading. Casagrande and Wilson (1951) and Taylor (1957) showed that the shear strength of saturated clays increased with increasing strain-rate when tested under undrained and unconfined compression. They also

showed that for most clayey soils the strength decreased in proportion to the logarithm of time to failure. Their results show a continuous increase in compressive strength with rate of loading increase. Whitman (1957) at the Massachusetts Institute of Technology showed that for a number of clayey soils the rate of increase of compressive strength at strain rate of 1000 %/sec. and 0.03 %/sec. varied from 1.3 to 2.0. Similar studies on the influence of shear speed on the shear strength of cohesive soils were confined to the determination of undrained strength of saturated clays. For example, Crawford (1959) found a noticeable decrease in failure deviator stress $(\sigma_1 - \sigma_3)_f$ as the time to failure was increased from 0.1 to 10 hours.

Healy (1963) studies the results obtained from consolidated undrained triaxial compression tests on a saturated clayey silt at rate of strain up to 1.7 %/sec. He found that there is an increasing strength as strain-rate is increased. He concluded that the strength increase at the faster rate of strain of testing, was due to the lower pore pressure occurring in the central portion of the sample. It was mentioned by Healy that this was caused not only by non-uniform stresses in the triaxial specimen but also due to a strain-rate

effect on dilatant tendencies of the soil. Richardson and Whitman (1963) described an investigation to determine the effect of strain-rate upon undrained strength of saturated remoulded fat clay. The strain-rates used were 1 %/min. and 0.002 %/minute. They found out that the deviator stress is influenced by strain-rate, particularly at low values of strain and the increase in deviator stress was accompanied by a corresponding increase in the effective principal stress ratio. Olson and Parola (1967) also found that for dynamic shearing test on compacted clay there was a marked increase in strength as the time to failure decreased.

Taylor (1948), Casagrande and Hirschfeld (1951), Tschebotarioff, Ward and de Phillippe (1953), Leonards (1955), Bjerrum, Simons and Torblaa (1958), Burmister (1959), Bishop et. al. (1960), Bjerrum, Simons and Torblaa (1960), Casagrande and Hirschfeld (1960), Henkel (1960), Lowe and Karafiath (1960), Folque (1961), Ladd (1964), Lee (1968), Daniel and Olson (1974), Sudhindra and Melkote (1974), Akai, Adachi and Ando (1975), and Balasubramanian (1975) in separate investigations which were similar to the aforementioned studies, confirmed and supported the findings made by all of the above mentioned researchers that - " strength increases as rate of

strain increases ".

1.3.2 Strain Rate Influence on Pore Water Pressure.

Several researchers have investigated the effects of variation in strain rate on the stress and pore water pressure at failure. Crawford (1959) performed a series of consolidated undrained triaxial tests on samples of saturated sensitive clay. The tests carried out, demonstrated the significant influence of the rate of strain on deviator stress and pore water pressure at failure. For example, it was noted that under constant deviator stress the pore pressure continued to rise while under condition of constant strain, the pore pressure decreased. Crawford found in his investigation that the ratio of the excess pore water pressure to the deviator stress at failure which is equal to Skempton A factor at failure, A_f, consequently showed a significant increase as the time to failure increased. Lo (1961) found that when a load was applied to an undrained sample, a certain amount of pore water pressure was built up. When the load was removed, it was found that a large portion of the pore water pressure built up still persisted. Such a phenomenon, he maintained, can only be explained microscopically by the permanent deformation of the sample

when loaded. In other words, it was a strain effect.

Richardson and Whitman (1963) performed tests on clay obtained from a back-swamp alluvial deposit, such as can be found in the lower Mississippi River Valley. Special pore pressure measuring equipment was incorporated in the triaxial cell to measure the pore pressure at mid-height of the specimen. For strain of less than about 1 %/min. the measured pore pressure was independent of strain rate. For faster strain rate ($\geq$ 5 %/min.), the pore pressure decreased. The results indicated that behaviour of clay soils was basically different at small strains and at large strains. Healy (1963) found in his investigation that the pore pressure at mid-height of the sample was observed to decrease with increasing rate of strain and for a faster test, he also found the decrease to be less than that at the bottom of the sample.

Holzer, Hoeg and Arulanadan (1972) examined experimentally how the excess pore water pressure is related to the mechanism for undrained creep of San Francisco Bay mud. They found that the shear strains occurring during undrained creep are directly related to a gradual but significant increase in excess pore pressure that results in the reduction of the

effective stresses. The increase in magnitude of the pore pressure is a function of the initial consolidation stress and consolidation period (except immediately after the creep shear stress is applied).

Wilson and Greenwood (1974) carried out a consolidated undrained triaxial shear test on a normally consolidated lacustrine clay under repeated loading, to prove the existence of a relationship between pore water pressures and axial strains. The investigation on the behaviour of the clay involves study of the pore pressure and strain responses under repeated loading and comparing them with the responses from the more usual sustained loading. The results showed that when the deviator stress was being applied to the soil at constant rate of strain of 0.055 %/min., there was a continuous increase in the pore pressures. The pore pressures and axial strains continued to increase with time under the action of repeated loading. The immediate increase in the pore pressure and axial strain observed on first application of the load were attributed to the elastic compression of the soil structure. The pore pressures and axial strains continued to increase until the load was removed. They found that the behaviour of sensitive clay is greatly affected by the type of

loading applied and there is no unique effective stress path for the soil sample they studied.

An investigation of the strain-rate effect of clayey soils on pore pressures conducted by Taylor (1944) yielded findings which show that pore pressure tends to decrease at constant strain. Bishop and Henkel (1951) made a joint study of the same problem and came out with similar findings as Taylor. Another study that Bishop, Blight and Donald (1960) conducted fully supported the previous findings. These studies were later followed by separate investigations made by Lambe (1963), Blight (1963), Poorooshasb (1963) and Juarez-Badillo (1963). The general findings of their studies altogether confirmed the outcomes of the above mentioned researches.

1.3.3 Strain Rate Effect on Strength Parameters.

A review of some relevant studies conducted on strain rate effect on strength parameters was carried out by Gisbon (1953) on six remoulded and two undisturbed clays. He concluded after his investigation that the modified Coulomb-Hvorslev equation is a satisfactory empirical criterion applying to a wide range of saturated remoulded soils. He found out that the true cohesion depended only upon the water content

of the clay at failure while the true angle of internal friction was found to be approximately constant.

Jakobson (1953) investigated a clay layer that had been always under water (clay that had not been exposed to erosion or to suction due to plant roots) and found a shear strength of 0.06 to 0.08 kg/cm^2 at the surface. He proposed the term " Origin Cohesion " for this shear strength. Wu, Douglas and Goughnour (1962) studied the friction and cohesion of saturated clays to determine the influence of clay structure and method of loading. They observed that under a constant strain rate, the maximum cohesion was developed immediately, at a very slow strain while the friction continued to increase with strain. The cohesion decreased with time while the friction increased to resist the stress. The cohesion in the flocculated clay was non-viscous at low strains and stresses, and it did not dissipate under constant stress or constant strain conditions. At higher strains and stresses the nature of the cohesion changed and its behaviour was similar to that of the remoulded clay.

Roy and Sarathi (1975) found out that the effect of test speed on both cohesion and the angle of internal friction became more marked as the confining pressure increased. At a

low range of confining pressure, 15 to 30 psi (103.43 to 206.85 kN/m^2), the cohesion and angle of internal friction values remained nearly constant for all rates of strain (0.333 %/min., 0.083 %/min. and 0.033 %/min.).

1.3.4 Rate Process.

The study of absolute reaction rates resulting in the Rate Process Theory, originated by Eyring (1941) has been shown to be useful in the understanding of time-dependent phenomena. Eyring and his co-workers had developed the Rate Process Theory as part of the field of chemical rate reaction kinetics. This theory provides an insight into the mechanisms of theoretical aspects within which phenomena can be organised for study and it suggests functional forms for the influences of certain variables on soil behaviours. As a result, considerable progress has been made in the last 20 years in developing the theory to explain the mechanism of clay deformation by the application of the Rate Process Theory.

Mitchell and Campanella (1963) worked out a hypothesis based on the Rate Process Theory which relates effective stresses, deviator stress, soil structure, temperature, and frictional and cohesive soil properties with creep rate. A

specially designed triaxial compression apparatus which permits creep testing of clays under a range of controllable temperatures and pressures as well as the measurement of pore water pressures was used in their investigation. The apparatus used also permitted rapid temperature change during the test. When the clay was tested in the undrained condition, they found that the effective stresses and creep behaviour of saturated clays were very sensitive to temperature variations. A temperature increase led to a significant decrease in effective stresses and an increase in strain rate for creep under both a constant sustained deviator stress and a constant sustained effective principal stress ratio. For conditions of constant temperature, soil structure and effective stresses, the logarithm of the creep rate varied linearly with the deviator stress. Murayama and Shibata (1964) have given further consideration of the rate process when applied to clays. In their study, they considered the inter-relation of frequency distribution curves of the applied stress and the frictional resistance between particles. Wu, Resendiz and Neukirchner (1966) performed consolidation tests and applied Rate Process Theory to describe the deformation curves. They also found that the parameters of their model maintained reasonably

consistent values under different loading conditions.

Mitchell et. al. (1969), reviewed the use of the Rate Process Theory taking into consideration past developmental studies of other researchers on the theory. They maintained that, according to the Rate Process Theory, flow units (atoms, molecules and/or particles involved in a time dependent-flow or deformation process) " are constrained from movements relative to each other by virtue of energy barriers separating adjacent equilibrium positions ". To overcome these energy barriers and to be able to displace them, activation is required, that is sufficient energy should be acquired. They found that the measured value of the activation energy for creep is 30 to 45 K cal per mole. This is an indication that the interparticle bonds are effectively solid-to-solid and of the primary valence type. It was observed that the activation energy is dependent of water content, consolidation pressure and void ratio. The activation energy indicated that the resistance to shearing stress, or alternatively the creep rates of soil under shearing stresses, cannot be accounted for by postulating the presence of viscous water films between particles at point of closest approach. Further, by analysing the influence of stress and temperature on deformation rates,

a direct functional relationship can be established between strength and the number of inter-particle bonds per unit area. They found that there was a steady increase on the number of bonds with respect to the confining (cell) pressure. However, these influences of temperature should be of the same magnitude for different soils under different conditions. They concluded that influences of strain-rate and temperature on shearing resistance are predicted by the Rate Process Theory and this is most applicable in its functional forms.

Scott and Hon-Yim Ko (1969) carried out numerous tests to determine the suitability of the creep rate behaviour of soils. In their investigations, they found out that the experimental activation energy is dependent both on the magnitude of the creep stress and the magnitude of creep strain. Furthermore, they observed that the comparison of the experimental relationship between strain-rates, stresses and time for a particular soil can give an idea of the number of interparticle bonds per unit gross cross-sectional area of the soil. Consequently, they concluded that the application of the Rate Process Theory to the results of creep tests provides a means for obtaining information on both the strength and the number of interparticle bonds in the soil under different conditions.

CHAPTER 2

PHYSICAL PROPERTIES AND TESTING PROCEDURES FOR FADAMA CLAY

2.1 CLAY SAMPLES

2.1.1 Location.

The clay known as Fadama Clay from Shika, Samaru, Zaria, Nigeria, used in this study, is located (see Map, Figure 2.1) at a distance of about a kilometer from the main road, 14 kilometers from Zaria and almost 5 kilometers from Ahmadu Bello University, main campus. Samples were taken at random (about 20 meters apart) at a depth of one meter and below during wet season particularly in the month of July. Water table during sampling was encountered at 0.61 meter below the ground surface. Samples collected at depths of 1.00, 1.26 and 1.40 meters from site 1, 2 and 3 respectively, are believed to be below surface weathering effects.

2.1.2 Geology of Site.

The soil under study is formed from alluvial deposits left by the flowing water. The product of the most recent erosion cycle, Fadama soil consist of grey-brown sands, silts and clays. These alluvial deposits are known to contain a

substantial aeolian component due to the large quantities of dust spread over the country annually by the seasonal Harmattan winds. Based on the Pleistocene sequence of alternating humid and arid climatic periods, aeolian deposits are known to be about 7,000 years (Mortimore, 1970).

Fadamas fall under the classification of mineral hydromorphic soils whose development and characteristics are influenced by permanent or seasonal water logging. This influence is indicated by the presence of sticky clayey soil developed under the influence of excessive moistening in at least one horizon (Klinkenberg and Higgins, 1968).

On the lower slopes, mineral hydromorphic soils are commonly found and are developed on poorly drained sites. However, variation in drainage occur because of the variations in soil structure, organic matter content as well as the height of the water table (Ipinmidum, 1972). Generally, these soils have a weak to moderate, fine to medium sub-angular blocky structure except for the flood plain soils which are often structureless, especially if recently deposited.

The soil sample in this study was taken in an area where the mean rainfall is about 1100 mm. and this comes mainly

in six months between May and October and open water evaporation is about 1880 mm. per annum (Ipinmidum, 1973).

2.1.3 Description.

The Fadama soil used in this study, obtained below ground surface contained 46.0 to 47.0 % finer than 2 microns and the rest of the soil is 36.0 to 37.5 % silt and 15.5 to 17.0 % sand (Figure 2.2). The average liquid and plastic limits are 46.4 % and 20.5 % respectively, the natural moisture content is 41.5 %, the undisturbed density is 2358.7 kg/m^3 and the specific gravity is 2.60. The maximum dry density (MDD) ranges from 1659.67 kg/m^3 to 1691.71 kg/m^3 at optimum moisture content (OMC) of 19.5 % to 19.7 % (Figure 2.5).

Fadama soils are common in Northern Nigeria especially along major rivers such as the Niger, along small rivers and even in depressions which do not contain any stream. Generally, fadama soils are usually grey or pale brown varying at depth between grey, white pale brown and yellow brown. The texture of the upper horizon varies from clay to loamy sand but is normally loam or sand loam. They are quite deep and are usually over 150 cm. There are however, localities where shallow soils or even rock outcrops occur in the fadamas and

some small fadamas, particularly those without streams have shallow impermeable horizons.

Presently, fadama soils in Northern Nigeria are devoted to crop production with sugarcane, groundnuts, cotton, benni-seed or sesame as the major crops. The ability of the fadama soils to retain moisture during the dry season makes them suitable for the production of these agricultural crops. Further, the distinction between water in the stream channel and groundwater is important where the possibilities of irrigation are considered. In general, although the fadama is a blessing to Agricultural Engineers, it is a problem soil for Highway Engineers.

The Hydrometer Analysis and Sieve Analysis were used to determine the particle size distribution for Shika Fadama soil. For purposes of this study, the particle size distribution is very significant since it influences the strength and compressibility of the soil, both of which are important in the consideration of bearing and stability for engineering purposes.

An air dried soil sample was passed through a Standard Sieve No. 200. The materials passing through the sieve were subjected to a Hydrometer Analysis while the coarse materials

were subjected to a sieve Analysis. A 50 gm. dry weight passing through the No. 200 sieve were mixed with distilled water to form a thin paste. A deflocculating agent consisting of 25 % of 500 ml. solution containing 3.5 grams of sodium carconate and 16.5 grams of sodium hexametaphosphate was added to the paste to prevent the formation of soil flocs. The suspension consisting of the mixture of the 50 grams soil sample, distilled water and deflocculating agent was then dispersed in a dispersing cup and stirred by means of a mechanical stirring device. The dispersed sample was placed in a 1000 ml. cylinder and allowed to come to temperature equilibrium.

The percentage of the material content in Fadama Clay can be seen in Figure 2.2 and corresponding values from the three different sites are tabulated in Table 2.1.

2.2 MINERALOGY OF THE CLAY SAMPLE

In order to determine the soil composition and consequently the behaviour of the Fadama Clay, four methods were used in this study. The X-ray Diffraction Analysis was used to readily identify crystalline components. To supplement the information obtainable from the X-ray method, the Differential Thermal Analysis (D.T.A.) was used for the identification of non-crystalline constituents. To further supple-

ment the information from X-ray Diffraction and Differential Thermal Analysis, chemical analysis was conducted for Cation Exchange Capacity (C.E.C.) and Glycol Retention.

2.2.1 X-ray Identification.

X-ray diffraction analysis was performed in the Geology Department, Ahmadu Bello University, Zaria, using a Philips Diffractometer. Tests were performed on oriented samples which passed through a Standard Sieve No. 200 using copper radiation with a wave length of 1.54 Å. Test results are shown in Figure 2.3. It can be noted from the test results that Kaolinite is prominent in the Fadama soil with some quartz, illite, carbonate and chlorite. The presence of kaolinite was confirmed by a characteristic peak of 7.24 Å in the air dried preferred orientation (ADPO). The presence of quartz was likewise confirmed by the 4.26 Å reflection and by a narrow characteristic peak of 3.36 Å.

2.2.2 Differential Thermal Analysis (D.T.A.).

This analysis consisted of simultaneously heating a test sample (passing through Standard Sieve No. 200) and a thermally inert substance at constant rate which ranged from about 10°C to 870°C and continuously measuring differences in

temperature between the sample and the inert material. Differences in temperature between the sample and the inert substance indicate reactions in the sample as a result of the heating. Theory shows that the area of the reaction peak can be taken as a measure of the amount of mineral present in the sample.

Test results in Figure 2.4 showed an endothermic reaction peak of 580 - 600°C and exothermic reaction peak registered at 750 - 800°C. At 600°C, the slope ratio was found to be 1.60. Using Bramao et. al's criterion (1952) regarding the distinction of minerals present in the soil, the slope ratio obtained in this study is within the slope ratio for kaolinite which is between 0.78 and 2.34.

2.2.3 Cation Exchange Capacity (C.E.C.).

This particular test was performed in the Survey Division, Institute of Agricultural Research (I.A.R.), Ahmadu Bello University, Zaria. Test results showed that the Cation Exchange Capacity for Fadama Clay was 4.5 meq/100 g. According to Grim (1968), clay mineral having a cation exchange capacity ranging from 3 to 15 meq/100 g. soil as measured by the ammonium acetate method at pH 7, fall under the type of kaolinite clay.

2.2.4 Glycol Retention.

With the aid of the laboratory equipment of the Department of Chemistry, Ahmadu Bello University, Zaria, Ethylene Glycol Retention of the Fadama Clay was determined by using the Bower and Gschwend Method (1952). The test showed a glycol retention of 17.1 mg/g.

2.3 EQUIPMENTS

The Triaxial Apparatus, Leonard Farnell & Co. Triaxial Machine (see Plate 1, page 34), was used in this study. This apparatus measures stresses on the three principal planes. With this equipment, the pore water pressures can be conveniently measured and drainage can easily be controlled. Connections of the apparatus used for the pore water pressure measurement to the triaxial cell chamber could be seen in Plate 2, page 35. The triaxial machine have a gear box from which displacement rates (ranging from 0.0014 to 0.16 in/min.) can be obtained.

The British Standard Proctor mould, 4 inches (10.16 cm.) in diameter by 4 inches (10.16 cm.) height was used in compacting the clay sample.

2.4 PREPARATION OF SPECIMENS

The natural moisture content of the clay sample was

determined from the undisturbed sample. The disturbed samples were air dried, thoroughly mixed with water and compacted using Standard Proctor Method (ASTM.D.698.64T). Immediately after compaction, three cylindrical tubes of internal diameter of 1.5 inches (38 mm.) by 3 inches (76 mm.) length were pushed into the compacted soil and an extruding machine, hydraulically operated was used to removed the specimen from the cylindrical tube. The ends of the specimen were covered with filter paper cut to size followed by a porous disc. The specimen was fitted between end caps and covered with a rubber membrane. Rubber bands were used to fasten the membrane to the end plates. Then, the specimen was placed in the triaxial machine (see Plate 3, page 36). The cell chamber was properly set up and uniformly clamped down to prevent leakage of water during the test, making sure that the specimen was properly sealed with its end caps and correctly placed. Care was taken to prevent any part of the machine or cell chamber from jogging the specimen whilst it was being set up. Then the specimen was ready for saturation and consolidation.

2.5 CONSOLIDATED UNDRAINED TRIAXIAL TESTING PROCEDURE

A back pressure ranging from 10 to 30 psi. (68.95 to 206.85 kN/m^2) was applied to most specimens before consolida-

tion. All specimens were consolidated for one day. In every test that was performed, the cell pressure was kept constant using 10, 30 and 45 psi. (68.95, 206.85 and 310.28 kN/m^2) during shear and the change in pore pressure was also measured. The rates of strain (loading rate) for every individual test were 0.0024, 0.006, 0.012 and 0.03 in/min. (0.08, 0.2, 0.4 and 1.0 %/min. respectively). Immediately after every test, the specimen was taken out from the chamber, carefully handled and the weight of specimen was recorded before putting it into the oven for drying. This was done to check the actual value of saturation. Any error due to friction between the piston and specimen was believed to be negligible and hence no correction for it was made in this study. Likewise, bushing effect was minimized by the lubrication of the piston. In this study, five test series (Test Series 1 to 5) were perfc.med for every cell pressure (10, 30 and 45 psi) using rates of strain 0.08, 0.2, 0.4 and 1.0 %/min. Five Test Series were performed to establish the margin of error, thereby, enabling the writer to come up with the average values for test results. Results of the Test Series conducted are illustrated in Figures 1 to 40, Appendix A.

2.6 METHOD USED ON COMPUTERIZED RESULTS

In an attempt to find functional relationship which would best explain certain behaviour of Fadama Clay, the writer made use of the Cyber 72 Computer at Ahmadu Bello University, Zaria. By means of Bivariate Regression Analysis (Statistical Package for the Social Science, 1975) the following relationships which give the best fit to the results of (1) Deviator Stress versus Rate of Strain, (2) Deviator Stress versus Time to Failure, (3) Rate of Strain versus Time to Failure, (4) Pore Pressure Coefficient versus Rate of Strain and (5) Pore Pressure Coefficient versus Time to Failure were obtained and these results are presented and discussed in the next chapter.

2.7 SUMMARY OF THE PHYSICAL PROPERTIES OF SHIKA FADAMA CLAY

From the discussions previously presented in this chapter, the Shika Fadama soil consist of kaolinite. Using the Hydrometer and Sieve Analysis the summary of the particle size distribution of the soil under study is as follows:

Particle Size Distribution	Range: Minimum	-	Maximum
Sand (2 - 0.06 mm.), %	15.5	-	17.0
Silt (0.06 - 0.002 mm.),%	36.0	-	37.5
Clay (< 0.002 mm.),%	46.0	-	47.0

Natural Water Content	41.5 %
Liquid Limit, L.L.	46.4 %
Plastic Limit, P.L.	20.5 %
Plasticity Index, P.I.	25.9 %
Specific Gravity, G_s	2.60
Activity, P.I./% of clay	0.556
Natural Density, kg/m^3	2358.7 kg/m^3
Maximum Dry Density, MDD, kg/m^3	1659.67 - 1691.71
Optimum Moisture Content, OMC, %	19.5 - 19.7

Using the plasticity chart, Plasticity Index versus Liquid Limit (Atterberg limit, Casagrande, 1948), values obtained in this study for the plasticity index and liquid limit which are 25.9 and 46.4 % respectively, indicates that Fadama Clay falls under inorganic clays of medium plasticity.

The moisture/density relationship was determined by the used of Standard Proctor Method (ASTM.D. 698.64T). Figure 2.5 illustrates the results obtained for the three different sites where samples were collected. The values obtained for the maximum dry density at its optimum moisture content can be seen in Table 2.2.

TABLE 2.1

Particle Size Distribution

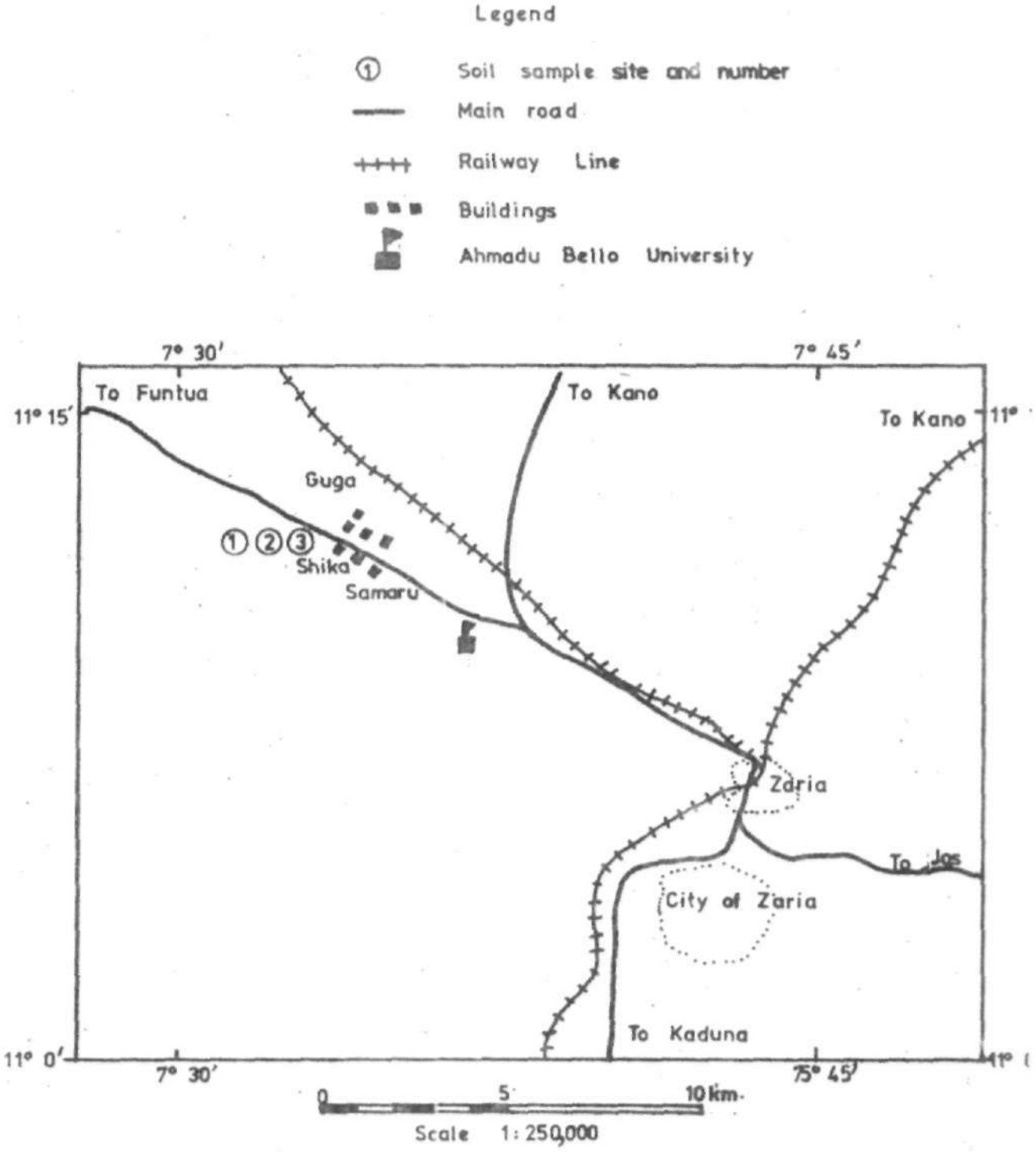
Sample Location	% Sand	% Silt	% Clay
Site 1	17.0	36.0	47.0
Site 2	16.8	37.2	46.0
Site 3	15.7	37.5	46.8

TABLE 2.2

Moisture/Density Relationship (Compaction Test)

Sample Location	Maximum Dry Density in pcf, (kg/m^3)	Optimum Moisture Content in %
Site 1	104.6 (1675.69)	19.6
Site 2	103.6 (1659.67)	19.7
Site 3	105.6 (1691.71)	19.5

Figure 2.1 Map of Zaria (portion) showing location of Soil Samples.



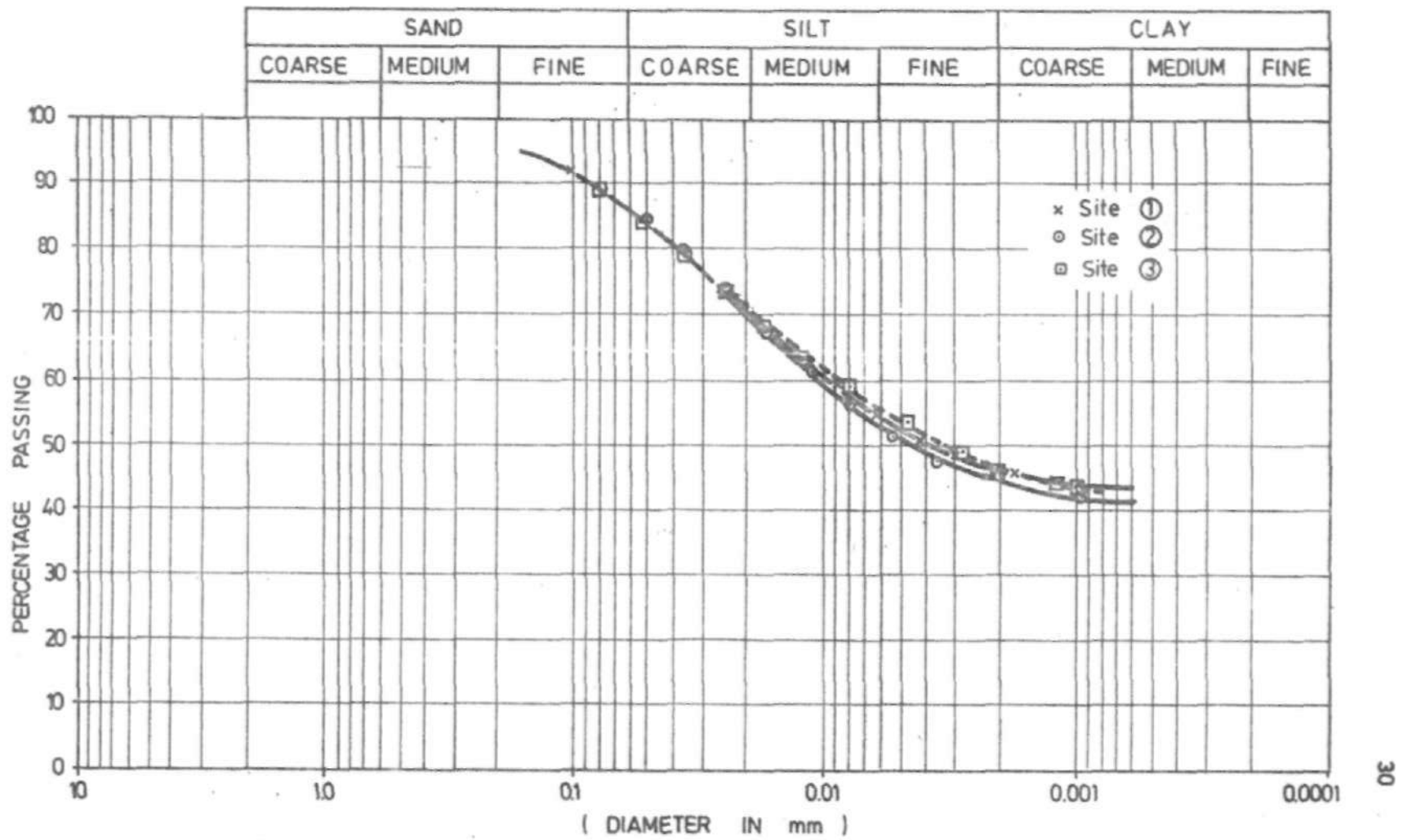


Figure 2.2 PARTICLE SIZE DISTRIBUTION.

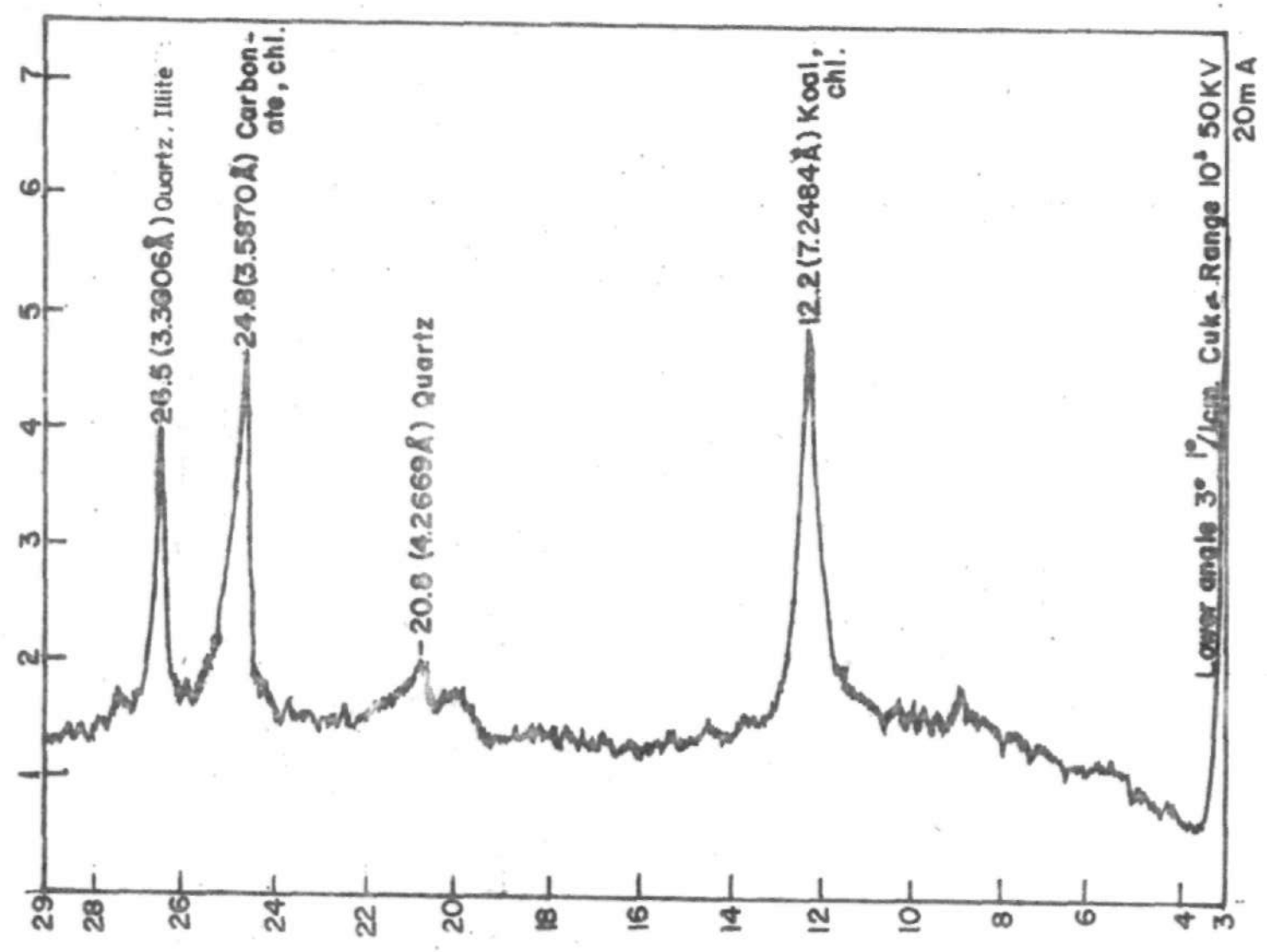


Figure 2.3 X-ray Diffraction for Fadama Clay.

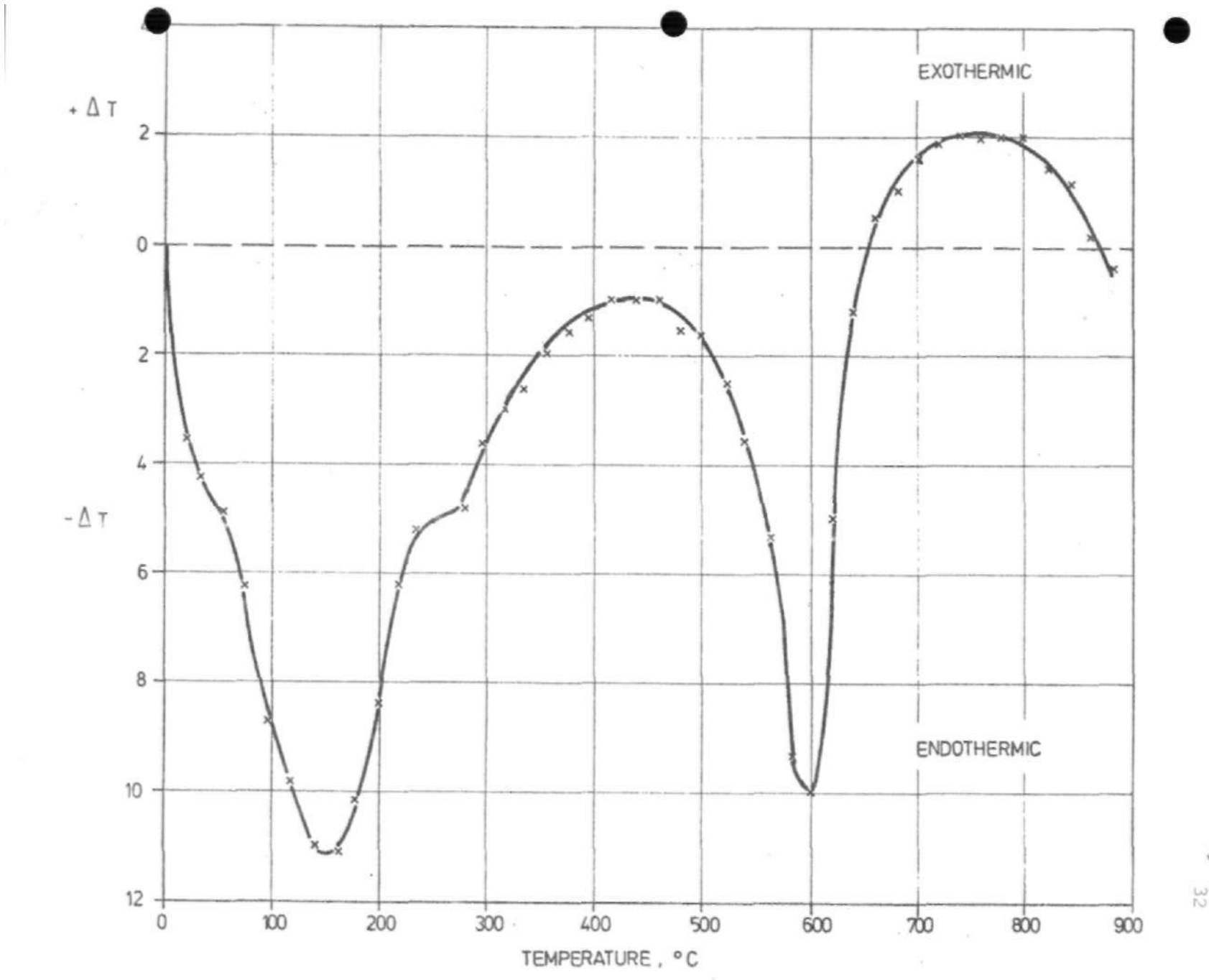


Figure 2.4 Differential Thermal Analysis.

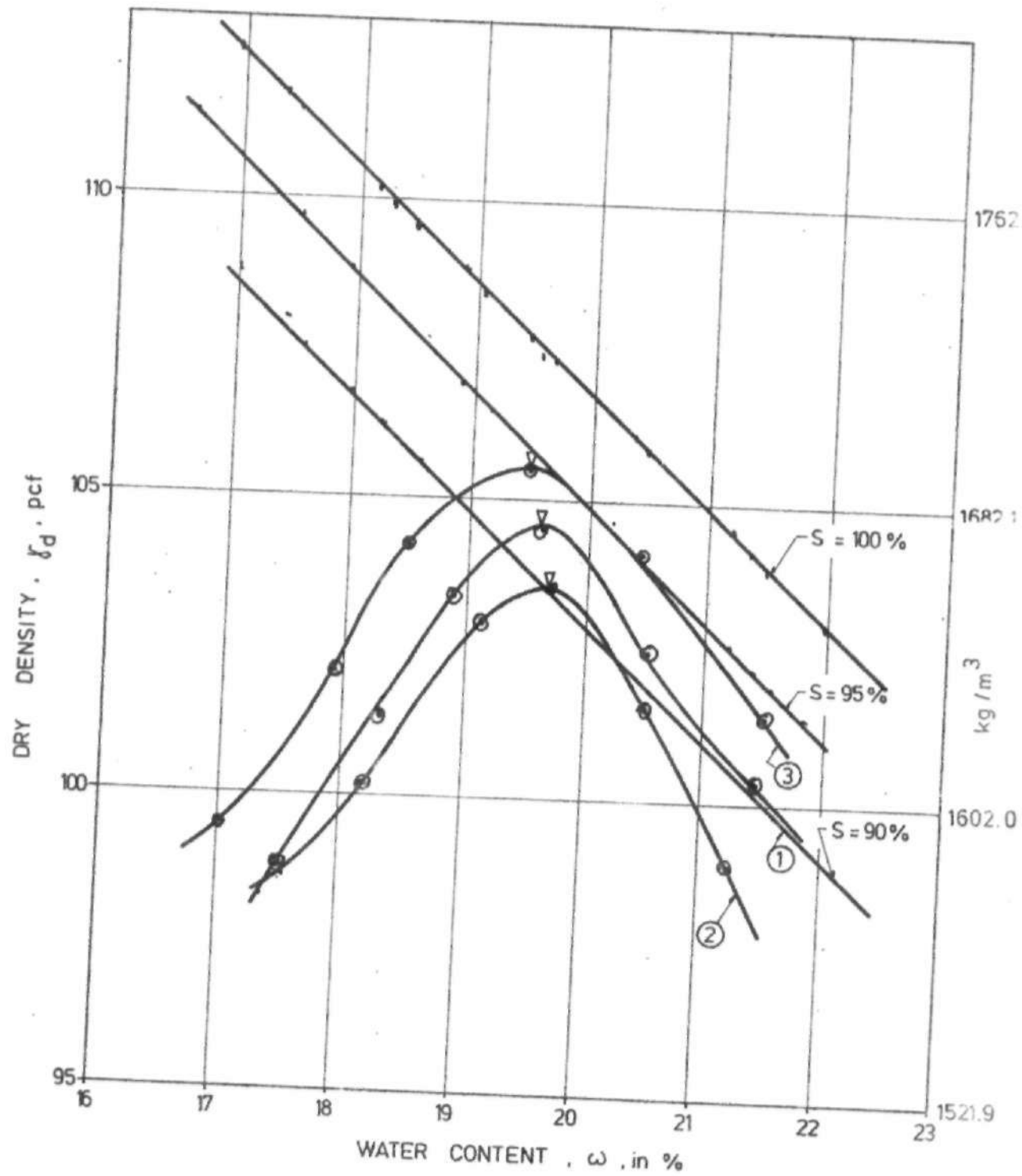


Figure 2.5 -- Moisture / Density Relationship.

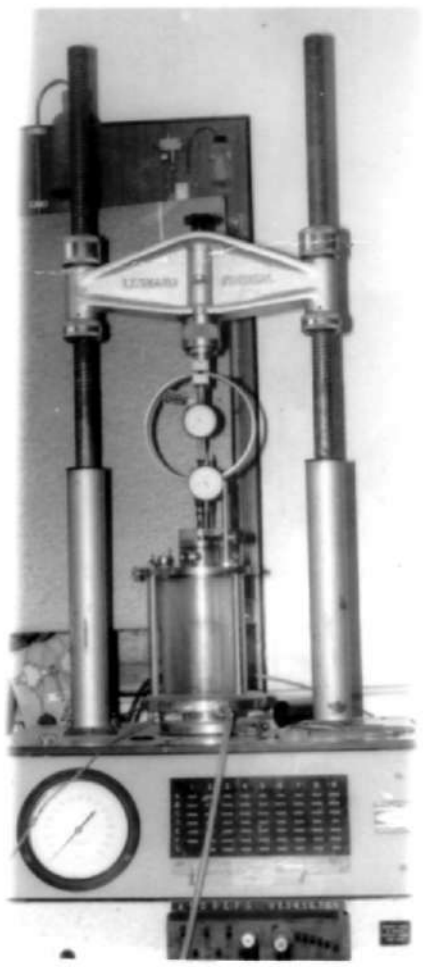


Plate 1 Leonard-Farnell & Co. Triaxial Machine

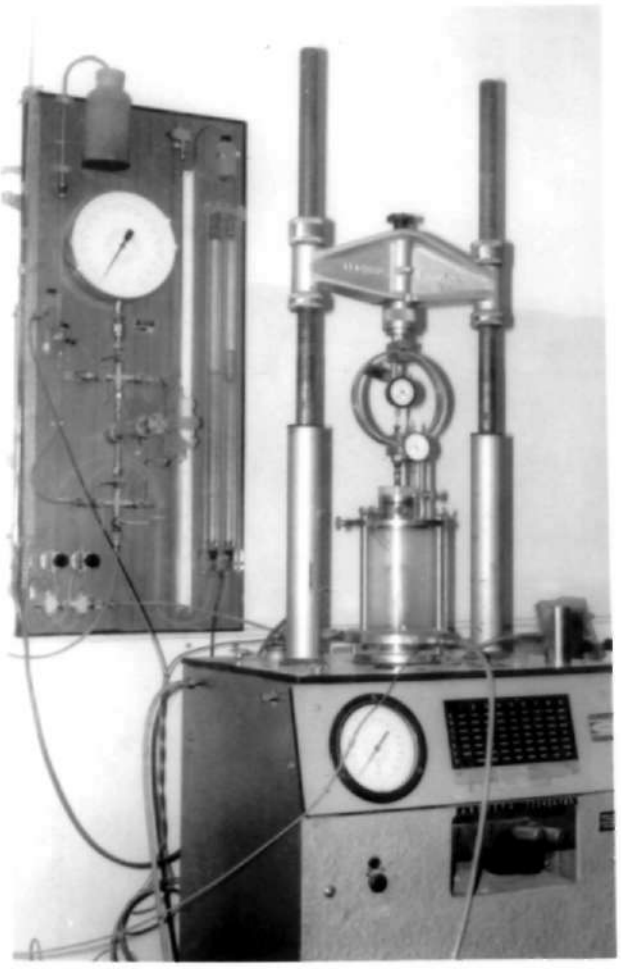


Plate 2 Triaxial Machine with the Pore Pressure Measurement Apparatus on the Left.



Plate 3 The Specimen Placed in the Triaxial Machine After Being Fixed with the Rubber Membrane.

CHAPTER 3

PRESENTATION AND INTERPRETATION OF TEST RESULTS

3.1 INTRODUCTION

In this chapter, the results obtained for the variation in strength of the Fadama Clay with increasing rates of strain are first discussed, and a mathematical equation is then derived which best represents the relationship. The results showing the functional relationship between strength and time to failure are also presented and a further mathematical equation is derived which best represent this relationship. The results showing the influence of failure strain at maximum deviator stress on the time to failure (for each cell pressure) are next presented and discussed. This is followed by the analysis of the results showing the functional relationship between time to failure and rate of strain.

In order to study the effect of the excess pore water pressure, it is first plotted against time to failure and then against rate of strain. This was carried out to establish the relationship between excess pore water pressure coefficient and

time to failure as well as the relationship between excess pore water pressure coefficient and rate of strain. The calculated strength parameters, c' and ϕ', are shown in Mohr-Coulomb diagram and their variation with varying rates of strain are discussed. An overall discussion and interpretation of the results is contained at the end of this chapter.

3.2 VARIATION IN STRENGTH WITH RATE OF STRAIN AND TIME TO FAILURE

3.2.1 Modulus of Elasticity.

The modulus of elasticity of soil for undrained loading is not a unique property but varies widely with stress level, stress history, time, type of loading and soil disturbance. From the figures of odd numbers (1, 3, 5, ...) in Appendix A, the average values (using secant modulus method) of modulus of elasticity at 2 % strain, 5 % strain, failure strain and half of the failure deviator stress, $(\sigma_1 - \sigma_3)/2$, are calculated and tabulated results can be seen in Table 3.01. It can be noted that at 2 % strain and 5 % strain, the modulus of elasticity, E, increases with increasing rate of strain. However, at failure strain a decrease in modulus of elasticity with increasing rate of strain was observed at 10 and 30 psi. (68.95 and 206.85 kN/m^2). At 45 psi. (310.28 kN/m^2) modulus of elasticity

showed an increasing trend with increasing rate of strain. At 10 and 30 psi. (68.95 and 206.85 kN/m^2) the decrease in modulus of elasticity with increasing rate of strain at failure strain (see Table 3.01) may be attributed to the possible effect of time to failure for breakdown of structural arrangements increases with increased time to failure. Secant modulus was applied in this study instead of tangent modulus because it is quite difficult to determine tangent modulus accurately from such test, since the slope of the stress-strain curve changes rapidly even at small strains.

Lambe and Whitman (1968) studied the behaviour of the modulus of soil at large loadings. They found that in general the modulus of the soil increases with increasing rate of strain. Results in this study conform to the general behaviour of the modulus of soil studied by Lambe and Whitman at cell pressure of 45 psi. (310.28 kN/m^2). Further, they also found that the modulus of the soil decreases with increasing deviator stress and soil disturbance. This study conforms to their findings only at cell pressures of 10 and 30 psi. (68.95 and 206.85 kN/m^2) at failure strain while at 2 % strain and 5 % strain the modulus of elasticity increases with increasing rate of strain.

3.2.2 Relationship Between Deviator Stress and Rate of Strain.

Tables 3.02 to 3.05 are summary of test results for various cell pressures (using 68.95, 206.85 and 310.28 kN/m^2) with their corresponding rates of strain (0.08, 0.2, 0.4 and 1.0 %/min.). Average values, extracted from the above tables can be seen in Table 3.06, which shows the relationship of strain-rate to the following parameters: (1) time to failure, (2) deviator stress at failure and (3) failure strain. The deviator stress plotted against the rate of strain at constant cell pressure is illustrated in Figure 3.01. The increase in deviator stress with increasing rate of strain were recorded for the rates of strain used in this study (see Table 3.07). The greatest overall increase in strength of 16.9 per cent was found for samples at a cell pressure of 45 psi. (310.28 kN/m^2). At cell pressure of 10 psi. (68.95 kN/m^2) the increase in strength is 10.3 per cent and for a cell pressure of 30 psi. (206.85 kN/m^2) increase in strength is 9.7 per cent. The overall mean increase in strength for the three values of cell pressure used was 12.3 per cent. At slower rate of strain (0.08 %/min.), Figure 1, Appendix A (Figure illustrating one of the test series in deviator stress versus axial strain, at

0.0024 in/min. rate of strain) shows that at cell pressures of 68.95, 206.85 and 310.28 kN/m^2, the maximum deviator stress (peak stress) were 200.0, 320.6 and 406.8 kN/m^2, respectively. At faster rate of strain, 1.0 %/min., Figure 37, Appendix A, shows that maximum deviator stresses were 220.6, 351.3 and 475.8 kN/m^2 at cell pressures of 68.95, 206.85 and 310.28 kN/m^2, respectively. It can be noted from the above test results, and in particular Figure 3.01, that strength increases as rate of strain increases and vice-versa. In other words, a direct functional relationship exist between deviator stress and rate of strain for a constant cell pressure.

The values obtained using Bivariate Regression Analysis are shown in Table 3.08. From the results, it could be deduced that the relationship between deviator stress and rate of strain is of the model: Stress = β_0 + β_1 log (Strain-Rate), where β_0 is a constant and β_1 is the coefficient of the log term. Further mathematical evaluations were undertaken to find out if there will be a common intercept and a single line equation to represent the relationship between deviator stress and rate of strain. Results from the computerized program shows that no functional

relationship exists between the deviator stress and strain rate when the cell pressure is not constant.

The results of this study conforms with the findings of Taylor (1943), Casagrande and Shannon (1948) and Richardson and Whitman (1963). Taylor (1943) investigated the variation in undrained strength with rate of loading using Boston Blue Clay. His results showed that an increasing rate of axial strain produced a steady increase in compression strength. Casagrande and Shannon (1948) also found that the compressive strength of Clay and Shale increased with rate of loading, when the failure time was less than 10 minutes. Studies on the relationship between maximum deviator stress and rate of strain were reported by Richardson and Whitman (1963), who used undrained saturated remoulded fat clay. They found that an increase in strain-rate caused an increase in effective deviator stress accompanied by effective principal stress ratio.

3.2.3 Relationship Between Deviator Stress and Time to Failure

Using Table 3.06, the deviator stress was plotted against time to failure at constant cell pressure (see Figure 3.02). This figure shows a slight but consistent decrease in

strength with a decrease in strain-rate (increase in time to failure). As the time to failure increases (rate of strain decreases) the deviator stress decreases. The rate of decrease of the deviator stress depends on the cell pressures (the rate of decrease of the deviator stress increases with increasing cell pressure). The results of this study showed a mean percentage decrease in strength of 5.5 per cent. The values obtained using Bivariate Regression Analysis in Table 3.09 shows the best fit equation to be a logarithm function in each of the cell pressures (68.95, 206.85 and 310.28 kN/m^2). The functional relationship is of the model: Stress = β_2 - β_3 log (Time) where β_2 is a constant and β_3 is the coefficient of the log term. However, no unique generalized equation was obtained indicating that no functional relationship exists between deviator stress and time to failure for a varying cell pressure for the Fadama Clay studied in this thesis.

The above behaviour has been confirmed by various authors working on different soil types. Studies by Taylor (1949) on remoulded Weald Clay showed that a decrease of strength of approximately 5 per cent was observed for a log cycle of time to failure, which is in close agreement with the

results in this study. Casagrande and Wilson (1951) found that for most clay soils the strength decreases in proportion to the logarithm of the time to failure. Their results are similar to those obtained in this study which was conducted with much longer test duration. As the test duration increased the deviator stress decreased at a diminishing rate. Roy and Sarathi (1976) investigated the strain-rate behaviour of compacted silt. They found that deviator stress decreased continuously with decreasing rate of strain for test duration shorter than 100 minutes.

3.3 FAILURE STRAIN INFLUENCE ON TIME TO FAILURE

Using data obtained from Table 3.06, failure strain (in per cent) was plotted against time to failure at constant cell pressure (see Figure 3.03). The decrease in strength for a log cycle of time to failure was then calculated and results are summarized in Table 3.10. It can be noted from the table that at the three cell pressures (68.95, 206.85 and 310.28 kN/m^2), failure strain decreased with increasing time to failure. It can also be observed that the rate of decrease in strength at a cell pressure of 45 psi. (310.28 kN/m^2) was 7.6 per cent which was more pronounced than at cell pressures of 10 and 30 psi. (68.95 and 206.85 kN/m^2) whose rate of

decrease in strength remained more or less constant at 4.5 per cent and 4.4 per cent respectively. When failure strain was plotted against the logarithm of strain-rate (Figure 3.04), failure strain at maximum deviator stress increased linearly for each of the cell pressures 10, 30 and 45 psi. (68.95, 206.85 and 310.28 kN/m^2) as the rate of strain increased. From Table 3.06, it can be noted that failure strain, $\in_f$, increases with increasing cell pressure at constant rate of strain, for example, 16 %, 18.4 % and 22.8 % for 10, 30 and 45 psi. cell pressures respectively. The maximum deviator stress, $(\sigma_1 - \sigma_3)_{max}$, is the measured strength which the sample can sustain at a point where the failure strain, $\in_f$, occurs. In Figure 39, Appendix A, at cell pressure (σ_3) of 45 psi.(310.28 kN/m^2), the value of the maximum deviator stress, $(\sigma_1 - \sigma_3)_{max}$, is 69.2 psi. (477.1 kN/m^2) and occur at 24 per cent axial strain. The ratio of effective major principal stress to the effective minor principal stress, known as the maximum effective principal stress ratio, $(\sigma_1'/\sigma_3')_{max}$, is an alternate measure of strength and this occurs at 22 per cent axial strain of the same figure. From Tables 3.02 to 3.05, it can be noted that with increasing rate of strain the effective principal stress ratio increases. The axial strain required to reach the maximum

stress difference (deviator stress) increases with increasing rate of strain. Hence, a direct relationship exist between the rate of strain and effective principal stress ratio. The criterion of failure in this study is selected to occur at maximum deviator stress on the stress-strain curve.

3.4 ANALYSIS ON TIME TO FAILURE AGAINST RATE OF STRAIN

From Table 3.06, time to failure was plotted against rate of strain and the graph is illustrated in Figure 3.05. The time to failure decreased with increased rate of strain. At lower rates of strain the time to failure seemed to be dependent on the cell pressure. However, as the rate of strain increased, the time to failure become independent of the cell pressure. For lower rates of strain (0.08 to 0.4 %/min.) there was a definite increase of time to failure. At a strain-rate of 0.03 in/min. (1.0 %/min.) the time to failure did not increase but remained nearly constant as the deviator stress increased. The results using Bivariate Regression Analysis is shown in Table 3.11. At each cell pressures (68.95, 206.85 and 310.28 kN/m^2) time to failure was best explained by reciprocal regression on the rate of strain variable. The results obtained in the relationship between time to failure and rate of strain is of the model: Time to Failure, t_f =

$\alpha_0 + \alpha_1$ /Strain-Rate, where α_0 is a constant and α_1 is the coefficient factor of 1/Strain-Rate.

Studies by Bjerrum, Simons and Torblaa (1958) also showed that for a soft marine clay there was a definite decrease in time to failure with increasing test duration which conforms to the results of this study.

3.5 EXCESS PORE WATER PRESSURE ANALYSES

3.5.1 Relationship Between Excess Pore Water Pressure and Time to Failure.

Table 3.12 shows the relationship of strain-rate to the following parameters: (1) pore pressure coefficient, (2) excess pore water pressure and (3) time to failure. The excess pore water pressure against time to failure at constant cell pressure of 10, 30 and 45 psi. (68.95, 206.85 and 310.28 kN/m^2, respectively) is illustrated in Figure 3.06. For consolidated undrained saturated compacted Fadama Clay, an increase in excess pore water pressure occurred with increase in time to failure. The increase was more pronounced with cell pressures of 45 and 30 psi. (310.28 and 206.85 kN/m^2) than with 10 psi. (68.95 kN/m^2). At strain-rate of 0.0024 in/min. (0.08 %/min.) Figure 8, Appendix A, shows that the maximum excess pore water

pressure at cell pressures of 10, 30 and 45 psi. (68.95, 206.85 and 310.28 kN/m^2) were 5.7, 9.6 and 13.0 psi. (39.3, 66.2 and 89.6 kN/m^2), respectively. At faster rate of strain 0.03 in/min. (1.0 %/min.) Figure 34, Appendix A shows that the maximum excess pore water pressure at cell pressures of 10, 30 and 45 psi. (68.95, 206.85 and 310.28 kN/m^2) are 36.5, 59.3 and 84.8 kN/m^2 respectively. Using Table 3.12, pore pressure coefficient was plotted against time to failure (see Figure 3.07). The increase in pore pressure coefficient with time to failure is more pronounced. Results obtained by the use of Bivariate Regression Analysis are shown in Table 3.13. The relationship between pore pressure coefficient and time to failure is of the model: Pore Pressure Coefficient, $\bar{A}_f = \psi_0 + \psi_1 \log$ (Time) where ψ_0 is a constant and ψ_1 is the coefficient factor of the log term.

For undrained saturated clays, Crawford (1959) maintained that pore pressure built up as a result of shearing strain. He found that under constant axial stress, pore pressure increases in direct proportion to strain. Lo (1969) agreed that pore water pressure continued to increase with time to failure. Wilson and Greenwood (1974) showed in their work that pore pressure and axial strain also gradually increased with time. The results of the foregoing researches are in

conformity with the results of this study.

3.5.2 Relationship Between Excess Pore Water Pressure and Rate of Strain.

Using data obtained in Table 3.12, excess pore water pressure was plotted against the rate of strain (see Figure 3.08). A decrease in excess pore water pressure occurred for all the cell pressures (68.95, 206.85 and 310.28 kN/m^2) with an increasing rate of strain. The smallest decrease of the excess pore pressure occurring at 10 psi. (68.95 kN/m^2) was 5.6 per cent. For the cell pressures of 45 and 30 psi. (310.28 and 206.85 kN/m^2), the excess pore pressure decreased by 6.6 and 6.8 per cent, respectively. Likewise, the pore pressure coefficient was plotted against the rate of strain (see Figure 3.09). It can be noted that pore pressure coefficient decreases with increasing rates of strain. The results of this study has been confirmed by Richardson and Whitman (1963) who conducted studies on the effect of pore pressure on strain-rate. The relationship between pore pressure coefficient and rate of strain is of the model: Pore Pressure Coefficient, $\tilde{A}_f = \psi_2 - \psi_3 \log$ (Strain-Rate) where ψ_2 is a constant and ψ_3 is the coefficient factor of the log term. At 10 psi. (68.95 kN/m^2), $\bar{A}_f = 0.17 - 0.13 \log \epsilon_R$, at 30 psi.

(206«85 kN/m^2), A = 0.17 - 0.17 log £ , and at 45 psi.C310.28 kN/m^2), A = 0.17 - 0.20 log <$£_R$.

Crawford (1959) maintained that stress and pore water pressure at failure depends on the rate of strain. Crawford (1963) found on his studies that pore pressure measurements at the base and within undisturbed specimens of sensitive clay during consolidation and shear indicate that increasing rate of strain during a test reduces the pore pressure slightly and causes increased shearing resistance. Richardson and Whitman (1963) found that for saturated remoulded fat clay, pore pressures decreased with increasing rate of strain, but for strains of less than 1.0 %/min* excess pore water pressure was independent of strain-rate. Wu (1971) summarising the findings of several researchers concluded that " when sheared in the undrained condition (such as in a consolidated undrained triaxial test), the pore pressure decreases with increasing strain-rate "«,

3.6 STRENGTH PARAMETERS, c* and j£

In this study, the normal and the shearing stress on the probable plane of failure was derived using Mohr-Coulomb diagram (see Figures 3.10 to 3.13). Using rates of strain of

0.08, 0.2, 0.4 and 1.0 %/min., the corresponding effective cohesion and effective angle of internal friction were obtained. These rates of strain adopted are believed to be relatively slow to allow equalization of pore water pressures at failure. A summary of important results is contained in Table 3.14. Included in the table are values obtained for effective cohesion (c') and effective angle of internal friction (ϕ') at different rates of strain. Likewise the value for the shear strength applying the Mohr-Coulomb shear stress equation, $\tau = c' + \sigma' \tan \phi'$, where τ is the shear strength, c' is the effective cohesion, σ' is the effective normal stress and ϕ' is the effective angle of internal friction. It can be seen from the table that with an increasing rate of strain, effective angle of internal friction also increases. However, it has been observed that effective cohesion (c') undergoes a reduction after attaining a peak value of 8.5 psi. (58.6 kN/m^2) at a very small strain (0.08 %/min.) and showed a slight decrease at faster rates of strain of 0.006 in/min. (0.2 %/min.). As the rate of strain is further increased, effective cohesion demonstrates an increasing value. The reduction in effective cohesion after reaching a maximum peak at a very small strain has been shown to have no

significant effect on the behaviour of the strength for Fadama Clay in this study. Shear strength increases as rates of strain are increased.

The results of this study is in close agreement with the findings obtained by various researchers on strain-rate effect on strength parameters, c' and ø'. Casagrande and Shannon (1948), Casagrande and Wilson (1951) and Taylor (1957) maintained that cohesion increases with increasing strain-rate when tested under undrained and unconfined compression. Gisbon (1953) who worked on six remoulded and two undisturbed clays, concluded that true cohesion depends only upon the water content of the clay at failure. Crawford (1959) found that cohesive strength of sensitive clays developed to a maximum at an axial compressive strain of less than 1.0 per cent, while the friction angle required a much greater strain. Wu et. al. (1962) who worked on remoulded laboratory flocculated, saturated clay found that effective cohesion first obtained its maximum value and then showed a slight decrease with an increase in the strain. Roy and Sarathi (1976) found out that the effect of test speed on both cohesion and the angle of internal friction became more marked as the confining pressure increased. At a low range of confining pressure of 15 and 30

psi. (103.43 and 206.85 kN/m^2), the cohesion and angle of internal friction values remained nearly constant for all rates of strain (0.333, 0.083 and 0.033 %/min.)

3.7 RATE PROCESS

The relationship derived from the rate of strain, $\dot{\epsilon}_R$, under constant shear force considering the net activation frequency in the direction of a distorting forces was obtained as follows using Scott and Ko Method (1969) on Rate Process Theory.

$$\dot{\epsilon}_R = 2 \text{ X } \frac{kT}{h} \exp(-\Delta F/RT) \sinh(f\lambda/2kT) \quad \text{.....eq.3.1}$$

where: X = function of the number of flow units in the direction of deformation and the average component of displacement in the same direction due to a single surmounting of the energy barrier.

k = Boltzmann's constant = 1.38×10^{-16} erg

$K^{-1} = 3.29 \times 10^{-24}$ cal $^{o}K^{-1}$

h = Planck's constant = 1.38×10^{-16} erg

R = universal gas constant = 1.98 cal ^{o}K erg sec^{-1}

T = absolute temperature ^{o}K

f = average shear force acting on each flow unit, dynes

λ = separation distance between successive equilibrium position, cm.

$\dot{\epsilon}_R$ = rate of strain

The following functions are treated constant with its equivalent symbols:

$$a = 2 X \frac{k T}{h} \exp(-\Delta F/RT) \quad \ldots\ldots\ldots \text{eq. } 3.2$$

$$b = \frac{\lambda}{2\ kT} \quad \ldots\ldots\ldots\ldots\ldots \text{eq. } 3.3$$

hence, eq. 3.1 becomes:

$$\epsilon_R = a \sinh f b \quad \ldots\ldots\ldots\ldots\ldots \text{eq. } 3.4$$

To obtain a significant creep rates in soil, the shear stresses must be large enough, that is, stress level of above 25 % of the strength or more. Then the value of the term ($f\lambda/2\ kT$) is greater than 1.0. In this case the hyperbolic sine in eq. 3.4 can be replaced by a simple exponential equation:

$$\epsilon_R = a \exp f b \quad \ldots\ldots\ldots\ldots\ldots \text{eq. } 3.5$$

If the maximum shear stress is D/2, where D is the principal stress difference (deviator stress), the relationship between the average shear force per flow unit and the deviator stress on the plane of maximum shear stress is approximated by : $f = D/2S$, where S is the bonds per unit area acted on by the sustained shear stress. Thus eq. 3.5 becomes:

$$\epsilon_R = a \exp Db/2S \quad \ldots\ldots\ldots\ldots\ldots \text{eq. } 3.6$$

i.e. $\ln \epsilon_R = \ln a + Db/2S$

$$D = \frac{2S}{b}(\ln \epsilon_R - \ln a)$$

$$D = -\frac{2S}{b} \ln a + \frac{2S}{b} \ln \epsilon_R$$

$$D = c + \frac{2S}{b} \ln \epsilon_R \quad \ldots\ldots\ldots\ldots \text{eq. 3.7}$$

In this study, computerization and analysis of the test results gives the constants in equation 3.7. These constants, c and 2S/b, are presented in Table 3.08 for the various cell pressures of 10, 30 and 45 psi. (68.95, 206.85 and 310.28 kN/m^2).

3.8 GENERAL DISCUSSION

Considering the results previously presented in this chapter the following observations have been found:

1. Strength increases with increasing rates of strain because of reduced time available for stress deformation. The test duration and rate of strain (rate of loading) significantly affect the strength of compacted clay. Most clay mineral particles encountered in Fadama Clay were thin plates, hence the surface area to mass ratio of most clay particles was sufficiently high that forces at particle surfaces become of significance in influencing the behaviour of the particles. Generally, when saturated clays are subjected to deviatoric

stress, a large number of interparticle contacts are disrupted, with some particles being oriented on the plane of incipient failure. In testing the present Shika Fadama Clay, at slower rate of strain, it was observed that strength decreased and this behaviour was attributed to the breakdown of structure of the soil. Also, test results show that the increased in strength at faster rate of strain was due to the increase in strength with time of the specimen. In this study, the strain-rate influence is shown and demonstrated in the functional relationship model that rate of strain increases as strength increases and vice-versa.

2. The relative decrease in strength and lower failure strain exhibited on the slowest rates of strain is indicative of increased brittleness built up in the specimen during the loading period. It was observed in this study that the strength decrease is substantially attributed to the breakdown in the soil structure. In strain-rate effects on strength, it was also observed that strength decreases for slower strain-rates. The variation in strength with time to failure is explained by the model demonstrating the findings in this study that as the time to failure increases, strength decreases. At failure strain, the rates of strain adopted in

this study are believed to be sufficiently slow for equalization of pore water pressure at failure. The time to failure at slower rates of strain seemed to be dependent on cell pressure up to 45 psi. (310.28 kN/m^2).

3. Strength decreases with increasing pore water pressure as a result of increase in time to failure. This result is attributed to the change in soil structure caused by the principal stress difference (deviator stress) and reaches equilibrium within the undrained test. The writer believes that the pore water pressure is influenced by the confining cell pressure due to structure weakening caused by the shear stresses. The decrease in pore pressure as a result of faster rates of strain might be due to the shorter time for the development of pore water pressure in the voids and incomplete destruction of the fabric/bonds in the soil. In this study it was also observed that pore water pressure continued to increase with time to failure, and the strength was consistent with the concept that pore pressure development is uniquely related to strain and independent of the rate of strain.

4. Strength in terms of effective stresses (cohesion and friction) showed a tendency to increase with increased rate

of strain. Shearing strength of Fadama Clay showed a tendency to increase with increased strain-rate (rate of loading). The increase in frictional resistance (the resistance to sliding of one surface on another) and the interlocking between particles increases the shearing strength of the soil. The magnitude of the resistance is proportional to the strength of the adhesive bond. Thus, frictional resistance depends directly on the effective stress. It appears that the magnitude of friction and the interlocking effects in a soil are dependent on soil composition. The strength contribution of the structural arrangement is practically independent of the change in effective stresses, whereas the frictional contribution is highly dependent on the magnitude of the effective stress. Test results in this study revealed the dependence of the angle of internal friction on the strain-rate. The angle of internal friction slightly increase with increased rate of strain.

5. Fadama Clay has been found to exhibit property changes with time (Rate Process Theory) after compaction and it was observed that among the changes that occurred was a strength increase with time at constant maximum dry density and optimum moisture content. The effects were significant at low

strain and the stress-strain modulus increased with decreasing time to failure. Once the load applied to the material was removed, the structure may no longer be in balance with the surroundings. If the interparticle force equilibrium was such that attraction was somewhat in excess of repulsion, there would have been a tendency towards flocculation of particles and aggregates and for a reorganization of the water-cation structure to a lower energy state.

TABLE 3.01

Relationship Between Rate of Strain and Modulus of Elasticity at 2 % Strain, 5 % Strain and Failure Strain.

Cell Pressure σ_3, psi. (kN/m^2)	Rate of Strain in/min	Modulus of Elasticity, E, psi.		
		2 % Strain	5 % Strain	Failure Strain
10 (68.95)	0.0024	705.0	436.0	181.3
	0.006	745.0	474.0	177.7
	0.012	970.0	518.0	177.3
	0.03	1155.0	577.0	165.0
30 (206.85)	0.0024	1005.0	610.0	252.7
	0.006	1300.0	744.0	248.4
	0.012	1105.0	778.0	241.0
	0.03	1710.0	878.0	220.0
45 (310.28)	0.0024	1460.0	866.0	258.8
	0.006	1715.0	934.0	265.2
	0.012	1775.0	972.0	269.4
	0.03	2360.0	1192.0	287.5

RATE OF STRAIN 0.0024 in/min

CELL PRESSURE σ_3 psi	TEST SERIES	DENSITY pcf		WATER CONTENT ω %		DEGREE OF SATURATION S %		VOID RATIO	DEVIATOR STRESS AT FAILURE	PORE WATER PRESSURE	EXCESS PORE WATER PRESSURE	PORE PRESSURE COEFFICIENT	TIME TO FAILURE	FAILURE STRAIN in	EFFECTIVE PRINCIPAL STRESS RATIO	EFFECTIVE PRINCIPAL MINOR STRESS
		DRY γ_d	BULK γ	BEFORE TEST ω_B	AFTER TEST ω_A	BEFORE TEST S_B	AFTER TEST S_A	e	$\sigma_1-\sigma_3$ psi	u psi	Δu psi	$\bar{A}_f = \Delta u/(\sigma_1-\sigma_3)$	t_f min	%	σ'_1/σ'_3	σ'_3 psi
10	1	104.49	125.28	19.9	20.7	94	97	0.552	29.0	6.7	5.7	0.197	200	16	8.8	3.3
	2	103.61	123.92	19.6	21.1	90	97	0.565	28.8	6.8	5.8	0.201	2125	17	9.0	3.2
	3	104.76	125.61	19.9	20.7	95	98	0.549	29.2	6.7	5.7	0.195	1815	15	8.8	3.3
	4	104.28	124.81	19.5	20.4	91	95	0.556	28.1	6.8	5.7	0.198	2125	17	9.4	3.1
	5	104.60	125.31	19.8	20.6	93	98	0.550	28.9	6.6	5.6	0.194	1825	15	8.5	3.4
	AVERAGE	104.35	125.0	19.7	20.7	93	97	0.554	29.0	6.7	5.7	0.197	200	16	8.9	3.3
30	1	104.61	125.43	19.9	20.7	94	97	0.551	46.5	12.1	10.0	0.215	225	18	2.8	16.1
	2	103.74	123.86	19.4	20.6	89	96	0.564	46.0	11.7	9.7	0.211	2375	19	2.5	16.3
	3	104.73	125.58	19.9	20.6	94	98	0.549	46.9	11.7	9.5	0.203	225	18	2.6	16.3
	4	104.24	124.57	19.5	20.5	91	96	0.556	46.3	11.8	9.6	0.207	2325	19	2.5	16.2
	5	104.59	125.19	19.7	20.7	93	98	0.552	46.8	11.7	9.7	0.207	225	18	2.5	16.3
	AVERAGE	104.38	125.0	19.7	20.6	92	97	0.554	46.7	11.8	9.7	0.209	230	18.4	2.5	16.2
45	1	104.78	125.62	19.8	20.7	94	98	0.549	59.0	18.0	14.0	0.237	275	22	2.2	27.0
	2	103.83	123.88	19.2	19.9	89	95	0.561	58.7	16.0	12.0	0.204	2875	23	2.0	29.0
	3	104.86	125.49	19.9	20.7	94	98	0.550	59.5	17.0	13.0	0.218	2875	23	2.1	28.0
	4	104.18	124.48	19.5	20.6	91	96	0.557	58.4	17.0	13.0	0.224	2875	23	2.1	28.2
	5	104.73	125.26	19.6	20.2	93	96	0.549	58.4	17.2	13.0	0.219	2875	23	2.1	27.8
	AVERAGE	104.48	125.0	19.7	20.4	92	97	0.553	59.0	17.0	13.0	0.220	285	22.8	2.1	28.0

Table 3.02 - Summary of Test Results at 0.0024 in/min. (0.08 %/min.) Rate of Strain.

RATE OF STRAIN 0.006 in/min

CELL PRESSURE σ_3, psi	TEST SERIES	DENSITY, pcf		WATER CONTENT ω, %		DEGREE OF SATURATION S, %		VOID RATIO	DEVIATOR STRESS AT FAILURE	PORE WATER PRESSURE	EXCESS PORE WATER PRESSURE	PORE PRESSURE COEFFICIENT	TIME TO FAILURE	FAILURE STRAIN IN	EFFECTIVE PRINCIPAL STRESS RATIO	EFFECTIVE PRINCIPAL MINOR STRESS
		DRY γ_d	BULK γ	BEFORE TEST ω_B	AFTER TEST ω_A	BEFORE TEST S_B	AFTER TEST S_A	e	$\sigma_1-\sigma_3$, psi	u, psi	Δu, psi	$\bar{A}_f = \Delta u_f/(\sigma_1-\sigma_3)_f$	t_f, min	%	σ_1'/σ_3'	σ_3', psi
10	1	104.92	125.27	19.4	20.4	93	96	0.550	29.5	6.6	5.6	0.190	85	17	8.7	3.4
	2	104.36	124.32	19.2	20.6	90	97	0.556	29.7	6.6	5.5	0.185	85	17	8.7	3.4
	3	104.38	125.05	19.8	20.7	93	97	0.554	29.3	6.4	5.4	0.184	80	16	8.1	3.6
	4	105.0	125.90	19.9	20.5	93	96	0.557	29.6	6.7	5.7	0.193	85	17	9.0	3.3
	5	104.25	125.10	20.0	20.8	94	97	0.556	29.4	6.6	5.6	0.190	80	16	8.6	3.4
	AVERAGE	104.57	125.0	19.7	20.6	93	97	0.555	29.5	6.6	5.6	0.188	83	16.6	8.6	3.4
30	1	104.83	125.38	19.6	20.4	93	97	0.547	47.6	11.5	9.5	0.200	95	19	2.6	18.5
	2	104.11	124.83	19.9	20.6	93	96	0.556	48.0	11.7	9.7	0.202	95	19	2.6	18.3
	3	104.48	125.17	19.8	20.6	93	97	0.553	47.9	11.6	9.6	0.200	95	19	2.6	18.6
	4	103.80	124.15	19.6	20.7	91	96	0.564	43.5	11.2	9.2	0.194	100	20	2.5	18.0
	5	104.25	124.89	19.8	21.0	93	98	0.557	47.4	11.5	9.5	0.200	100	20	2.6	18.5
	AVERAGE	104.29	125.0	19.7	20.7	93	97	0.555	47.7	11.5	9.5	0.199	97	19.2	2.6	18.6
45	1	104.88	125.44	19..	20.6	93	98	0.547	61.0	17.0	13.0	0.213	115	23	2.2	28.0
	2	104.38	124.72	19.5	20.7	91	97	0.554	60.6	16.9	12.9	0.213	115	23	2.1	28.2
	3	104.60	125.31	19.8	20.6	94	97	0.552	60.3	16.9	12.9	0.214	115	23	2.1	28.2
	4	103.66	124.50	20.1	20.7	93	96	0.565	61.7	17.0	12.9	0.209	115	23	2.2	28.3
	5	104.16	124.68	19.7	21.0	92	98	0.558	61.4	16.8	12.8	0.208	115	23	2.2	28.4
	AVERAGE	104.34	125.0	19.7	20.7	93	97	0.555	61.0	16.9	12.9	0.211	115	23	2.2	28.2

Table 3.03 - Summary of Test Results at 0.006 in/min. (0.2 %/min.) Rate of Strain.

RATE OF STRAIN 0.012 in/min																	
CELL PRESSURE	TEST SERIES	DENSITY , pcf		WATER CONTENT ω , %		DEGREE OF SATURATION S , %		VOID RATIO	DEVIATOR STRESS AT FAILURE	PORE WATER PRESSURE	EXCESS PORE WATER PRESSURE	PORE PRESSURE COEFFICIENT	TIME TO FAILURE	FAILURE STRAIN IN	EFFECTIVE PRINCIPAL STRESS RATIO	EFFECTIVE PRINCIPAL MINOR STRESS
		DRY	BULK	BEFORE TEST	AFTER TEST	BEFORE TEST	AFTER TEST									
σ_3 , psi		γ_d	γ	ω_B	ω_A	S_B	S_A	e	$(\sigma_1-\sigma_3)_f$, psi	u , psi	Δu , psi	$A_f = \Delta u_f/(\sigma_1-\sigma_3)_f$	t_f , min	%	σ_1'/σ_3'	σ_3' , psi
10	1	103.50	123.89	19.7	20.8	90	95	0.568	30.5	6.5	5.5	0.180	425	17	8.7	3.5
	2	103.56	124.17	19.9	20.9	91	96	0.567	30.6	6.6	5.3	0.173	425	17	9.0	3.4
	3	103.72	124.59	20.1	20.9	93	96	0.564	30.3	6.4	5.4	0.178	425	17	8.4	3.6
	4	104.13	125.06	20.1	20.7	94	97	0.558	30.7	6.7	5.7	0.186	450	18	9.3	3.3
	5	104.66	124.97	19.4	20.4	92	98	0.550	30.4	6.6	5.6	0.184	425	17	8.9	3.4
	AVERAGE	103.92	125.0	19.8	20.7	92	96	0.561	30.5	6.6	5.6	0.180	43	17.2	8.9	3.4
30	1	103.39	123.96	19.9	20.9	91	96	0.567	48.2	11.2	9.2	0.191	50	20	2.6	18.9
	2	103.69	124.22	19.8	20.9	91	96	0.565	48.3	11.6	9.4	0.195	50	20	2.6	18.6
	3	103.90	124.37	19.7	20.8	91	96	0.562	48.5	11.5	9.5	0.196	50	20	2.6	18.7
	4	104.25	125.00	19.9	20.6	93	97	0.556	47.9	11.5	9.4	0.196	50	20	2.6	18.6
	5	104.42	124.78	19.5	20.5	92	97	0.554	48.1	11.5	9.5	0.198	50	20	2.6	18.5
	AVERAGE	103.93	125.0	19.7	20.7	92	96	0.561	48.2	11.5	9.4	0.195	50	20	2.6	18.7
45	1	103.35	123.32	19.9	20.8	92	95	0.570	62.5	16.8	12.8	0.205	525	23	2.2	28.2
	2	103.75	124.29	19.8	20.8	91	96	0.564	62.7	16.7	12.7	0.203	525	23	2.2	28.4
	3	103.98	124.46	19.7	20.9	92	97	0.561	62.0	16.7	12.6	0.203	60	24	2.2	28.3
	4	104.44	125.12	19.8	20.4	93	98	0.553	63.4	16.5	12.5	0.197	525	23	2.2	28.6
	5	104.38	124.83	19.4	20.4	91	96	0.554	61.9	16.6	12.4	0.200	525	23	2.2	28.5
	AVERAGE	103.98	125.0	19.7	20.7	92	96	0.560	62.5	16.7	12.6	0.202	58	23.2	2.2	28.4

Table 3.04 - Summary of Test Results at 0.012 in/min. (0.4 %/min.) Rate of Strain.

RATE OF STRAIN 0.03 in/min

CELL PRESSURE σ_3, psi	TEST SERIES	DENSITY, pcf DRY γ_d	DENSITY, pcf BULK γ	WATER CONTENT ω, % BEFORE TEST ω_B	WATER CONTENT ω, % AFTER TEST ω_A	DEGREE OF SATURATION S, % BEFORE TEST S_B	DEGREE OF SATURATION S, % AFTER TEST S_A	VOID RATIO e	DEVIATOR STRESS AT FAILURE $(\sigma_1-\sigma_3)_f$ psi	PORE WATER PRESSURE u psi	EXCESS PORE WATER PRESSURE Δu, psi	PORE PRESSURE COEFFICIENT $\bar{A}_f=\Delta u/(\sigma_1-\sigma_3)_f$	TIME TO FAILURE t_f, min	FAILURE STRAIN ε_f %	EFFECTIVE PRINCIPAL STRESS RATIO σ_1'/σ_3'	EFFECTIVE PRINCIPAL MINOR STRESS σ_3', psi
10	1	103.43	123.50	19.4	20.9	89	95	0.569	32.0	6.5	5.4	0.169	19	19	9.1	3.5
	2	104.52	124.90	19.5	20.6	92	97	0.552	31.9	6.5	5.3	0.166	20	20	9.1	3.5
	3	103.91	124.80	20.1	20.6	93	95	0.561	31.8	6.3	5.2	0.164	20	20	8.6	3.7
	4	104.0	124.90	20.1	20.8	93	97	0.560	32.1	6.6	5.6	0.174	19	19	9.4	3.4
	5	104.69	125.00	19.4	20.5	92	97	0.550	32.2	6.5	5.5	0.171	19	19	9.2	3.6
	AVERAGE	104.11	124.50	19.7	20.7	92	96	0.558	32.0	6.5	5.4	0.169	19	19.4	9.1	3.5
30	1	104.60	125.38	20.1	20.5	94	96	0.554	51.0	11.0	9.0	0.167	23	23	2.7	18.6
	2	104.88	125.00	19.4	20.7	92	98	0.550	50.8	11.1	9.0	0.177	23	23	2.7	18.3
	3	103.75	124.60	20.1	20.8	93	96	0.564	50.5	10.8	8.4	0.174	23	23	2.6	18.7
	4	105.03	125.30	19.3	20.3	91	96	0.556	51.3	11.1	8.9	0.173	24	24	2.7	18.4
	5	104.47	125.16	19.8	20.4	93	96	0.553	51.4	10.8	8.6	0.171	23	23	2.7	18.4
	AVERAGE	104.47	125.10	19.7	20.5	93	96	0.555	51.0	11.0	8.9	0.172	23	23.2	2.7	18.5
45	1	104.57	125.07	19.6	20.7	93	98	0.552	69.0	15.8	12.0	0.174	24	24	2.3	30.0
	2	104.62	125.13	19.6	20.4	93	96	0.551	68.5	16.4	12.3	0.180	24	24	2.4	28.1
	3	103.67	124.40	20.0	20.6	92	95	0.564	68.8	16.2	12.2	0.177	24	24	2.4	28.3
	4	104.00	124.58	19.8	20.5	92	95	0.560	68.5	16.3	12.3	0.177	24	24	2.4	28.9
	5	104.58	125.18	19.7	20.4	93	96	0.551	68.2	16.2	12.2	0.176	24	24	2.4	28.0
	AVERAGE	104.29	124.90	19.7	20.5	93	96	0.556	69.0	16.2	12.2	0.177	24	24	2.4	29.3

Table 3.05 - Summary of Test Results at 0.03 in/min. (1.0 %/min.) Rate of Strain.

TABLE 3.06

Relationship Between Strain-Rate, Time to Failure,

Deviator Stress and Failure Strain

Cell Pressure σ_3, psi.	Rate of Strain $\dot{\epsilon}_R$ in/min	Time to Failure t_f,(min.)	Deviator Stress at Failure $(\sigma_1-\sigma_3)$,psi.	Failure Strain ϵ_f %
10	0.0024	200	29.0	16.0
	0.006	83	29.5	16.6
	0.012	43	30.5	17.2
	0.03	19.4	32.0	19.4
30	0.0024	230	46.5	18.4
	0.006	97	47.7	19.2
	0.012	50	48.2	20.0
	0.03	23.2	51.0	23.2
45	0.0024	285	59.0	22.8
	0.006	115	61.0	23.0
	0.012	58	62.5	23.2
	0.03	24	69.0	24.0

TABLE 3.07

Percentage Increase in Strength with Increasing Rate of Strain

Cell Pressure σ_3 (psi)	Deviator Stress (psi) at 0.0024 in/min. Rate of Strain	Deviator Stress (psi) at 0.03 in/min. Rate of Strain	% Increase
10	29.0	32.0	10.3
30	46.5	51.0	9.7
45	59.0	69.0	16.9
Mean Increase in Strength, %			12.3

TABLE 3.08

Equations Representing the Relationship Between Deviator Stress and Rate of Strain

Model	Cell Pressure, σ_3 (psi)					
	10		30		45	
	R^2	% Fit	R^2	% Fit	R^2	% Fit
Linear	0.8767	87.7	0.7998	79.98	0.8877	88.8
Quadratic	0.8617	86.2	0.7789	77.89	0.8917	89.2
Logarithmic	0.9931	99.3	0.9603	96.03	0.7228	72.3
Reciprocal	0.7446	74.5	0.7579	75.79	0.4313	43.1
Square Root	0.92.16	92.2	0.8573	85.73	0.8526	85.3
Best Fit Equation	$D= 32.8 + 1.4 \log \dot{\epsilon}_R$		$D= 51.9 + 2.2 \log \dot{\epsilon}_R$		$D= 71.5 + 4.8 \log \dot{\epsilon}_R$	

R^2 indicates the proportion of variation of deviator stress explained by the rate of strain.

TABLE 3.09

Equations Representing the Relationship Between Deviator Stress and Time to Failure

Model	Cell Pressure, σ_3 (psi)					
	10		30		45	
	R^2	% Fit	R^2	% Fit	R^2	% Fit
Linear	0.7745	77.4	0.7641	76.4	0.6792	67.90
Quadratic	0.5392	53.9	0.5425	54.3	0.4706	47.06
Logarithmic	0.9596	96.0	0.9928	99.3	0.9997	99.97
Reciprocal	0.8021	80.2	0.8790	87.9	0.9066	90.66
Square Root	0.9408	94.1	0.9390	93.9	0.9051	90.51
Best Fit Equation	$D = 32.5 - 1.4 \log t_f$		$D = 51.6 - 2.6 \log t_f$		$D = 70.9 - 4.8 \log t_f$	

R^2 indicates the proportion of variation of deviator stress explained by the time to failure.

TABLE 3.10

Rate of Decrease in Strength with Respect to

Time to Failure

Cell Pressure σ_3 (psi)	Deviator Stress (psi) at 10 minutes	Deviator Stress (psi) at 100 minutes	Rate of Decrease in Strength w.r.t. t_f, (%)
45	66.0	61.0	7.6
30	49.7	47.5	4.4
10	31.0	29.6	4.5
Mean % Decrease			5.5

TABLE 3.11

Equations Representing the Relationship Between Time to Failure and Rate of Strain

Model	Cell Pressure, σ_3 (psi)					
	10		30		45	
	R^2	% Fit	R^2	% Fit	R^2	% Fit
Linear	0.4150	41.5	0.4199	42.0	0.4417	44.2
Quadratic	0.3919	39.2	0.3967	39.7	0.3885	38.9
Logarithmic	0.6955	69.6	0.7004	70.0	0.6975	69.8
Reciprocal	0.9997	99.97	0.9994	99.94	0.8492	84.9
Square Root	0.4899	49.0	0.4947	49.5	0.5362	53.6
Best Fit Equation	$t_f = 2.127 + \frac{0.4836}{\epsilon_R}$		$t_f = 3.101 + \frac{0.5576}{\dot{\epsilon}_R}$		$t_f = 3.973 + \frac{0.694}{\epsilon_R}$	

R^2 indicates the proportion of variation of time to failure explained by the rate of strain.

TABLE 3.12

Relationship Between Rate of Strain, Pore Pressure Coefficient, Excess Pore Water Pressure and Time to Failure

Cell Pressure σ_3 (psi)	Rate of Strain (in/min)	Pore Pressure Coefficient $\bar{A}_f = \Delta u/(\sigma_1-\sigma_3)_f$	Excess Pore Water Pressure Δu, (psi)	Time to Failure t_f, (min)
10	0.0024	0.195	5.7	200
	0.006	0.188	5.6	83
	0.012	0.180	5.5	43
	0.03	0.169	5.4	19.4
30	0.0024	0.209	9.4	230
	0.006	0.199	9.5	97
	0.012	0.195	9.4	50
	0.03	0.172	8.8	23.2
45	0.0024	0.220	13.0	285
	0.006	0.211	12.9	115
	0.012	0.202	12.6	58
	0.03	0.177	12.2	24

TABLE 3.13

Equations Representing the Relationship Between Pore Pressure Coefficient and Time to Failure

Model	Cell Pressure, σ_3 (psi)					
	10		30		45	
	R^2	% Fit	R^2	% Fit	R^2	% Fit
Linear	0.8649	86.5	0.7659	76.6	0.7482	74.8
Quadratic	0.6550	65.5	0.5451	54.5	0.5194	51.9
Logarithmic	0.9298	93.0	0.9921	99.2	0.9914	99.1
Reciprocal	0.7257	72.6	0.8777	87.8	0.8621	86.2
Square Root	0.9807	98.0	0.9397	94.0	0.9358	93.6
Best Fit Equation	$A_f = 0.17 + 0.18 \log t_f$		$A_f = 0.17 + 0.18 \log t_f$		$A_f = 0.17 + 0.2 \log t_f$	

R^2 indicates the proportion of variation of pore pressure coefficient explained by the time to failure.

TABLE 3.14

Rate of Strain Effect on Shear Strength for Fadama Clay

Rate of Strain ϵ_R (in/min)	Effective Cohesion c', psi. (kN/m^2)	Effective Angle of Internal Friction ϕ', degree	Maximum Shear Strength, τ_{max}, psi. (kN/m^2)
0.0024	8.5(58.6)	22	27.89 (192.3)
0.006	8.0(55.2)	23	28.59 (197.1)
0.012	8.2(56.5)	24	30.01 (206.9)
0.03	8.3(57.2)	25	31.38 (216.4)

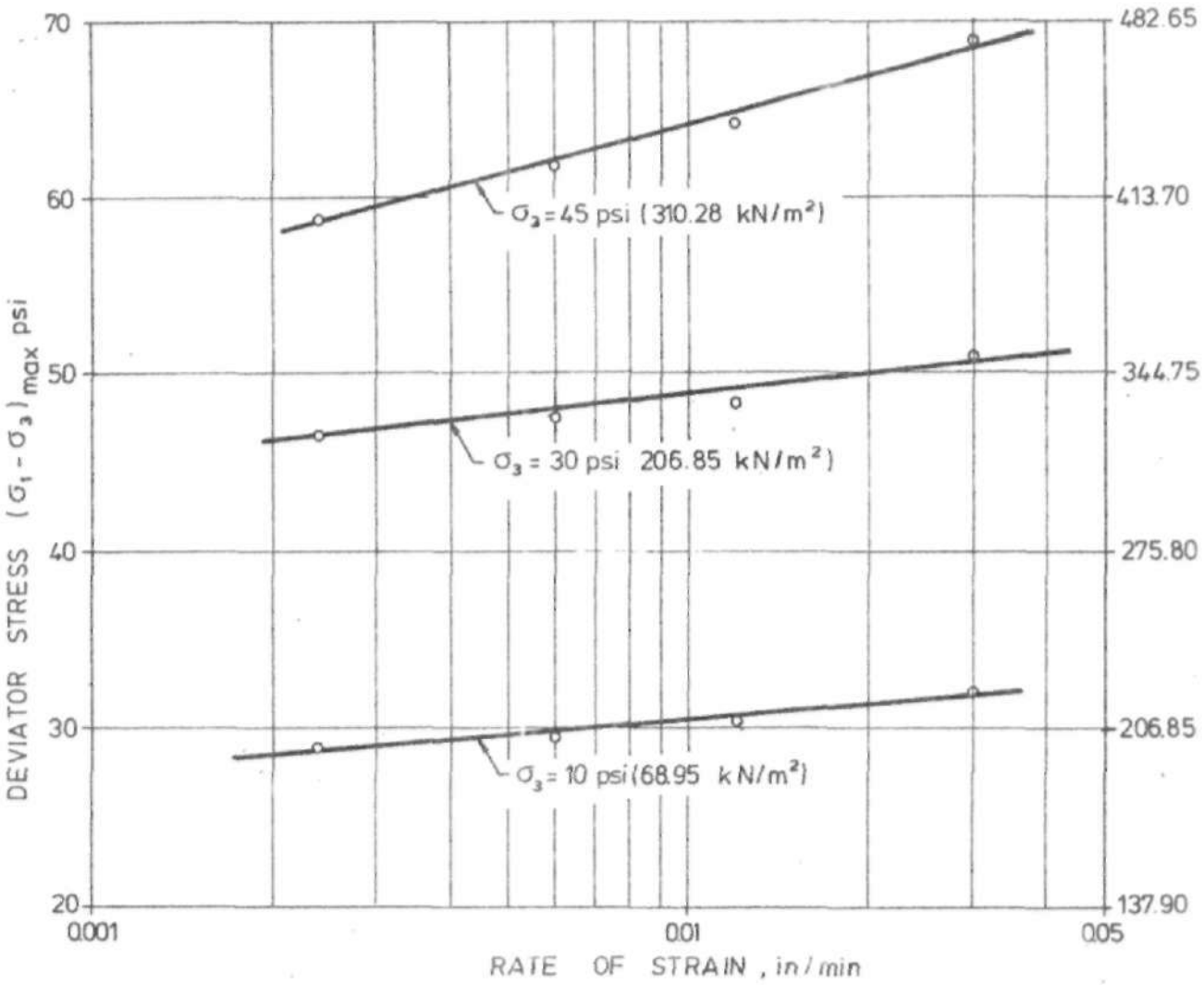


Figure 3.01 -- Influence of the Rate of Strain on Peak Deviator Stress for Compacted Fadama Clay (Failure at Peak Deviator Stress).

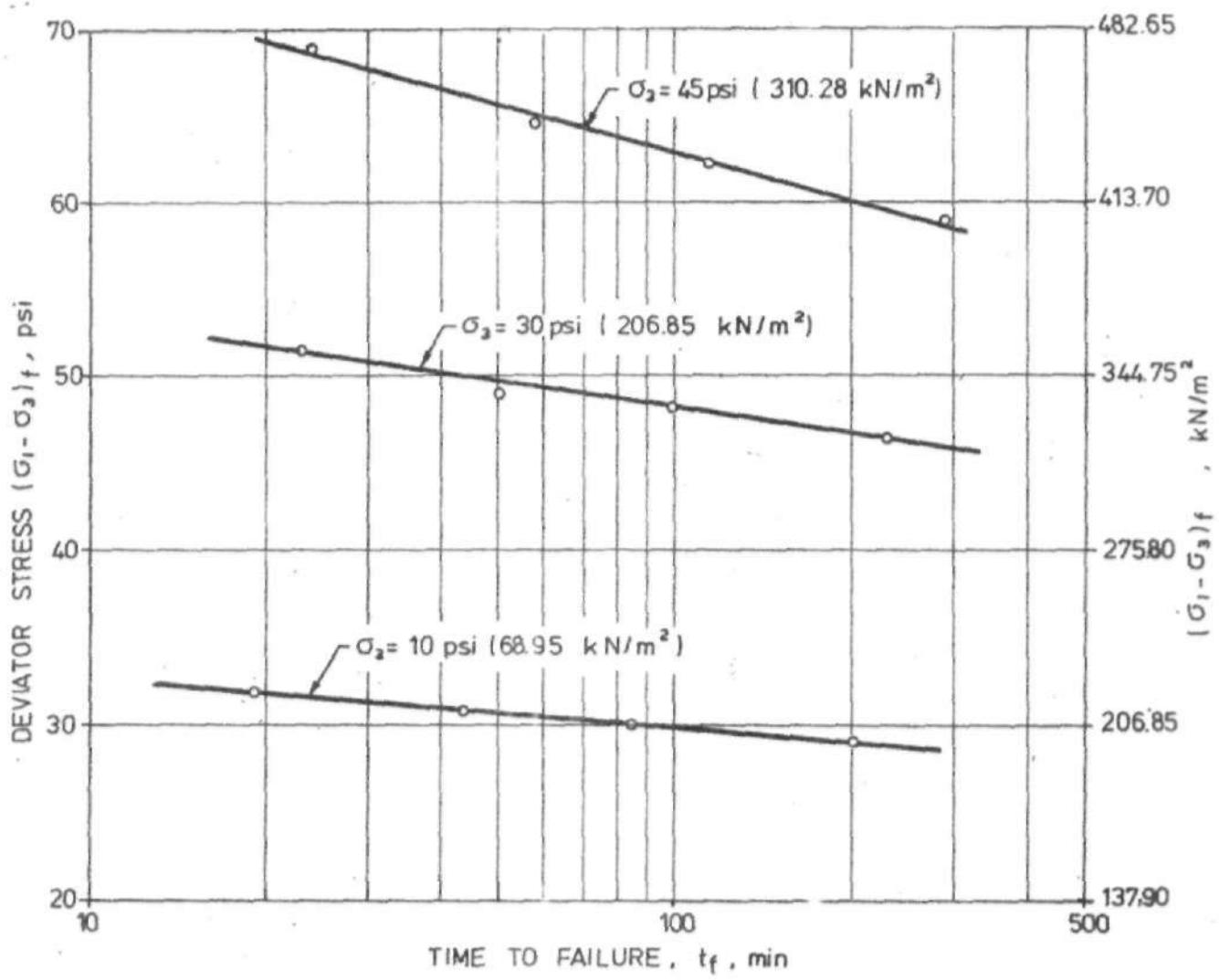


Figure 3.02 -- The Variation in Strength with Time to Failure for Consolidated Undrained Test on Compacted Fadama Clay.

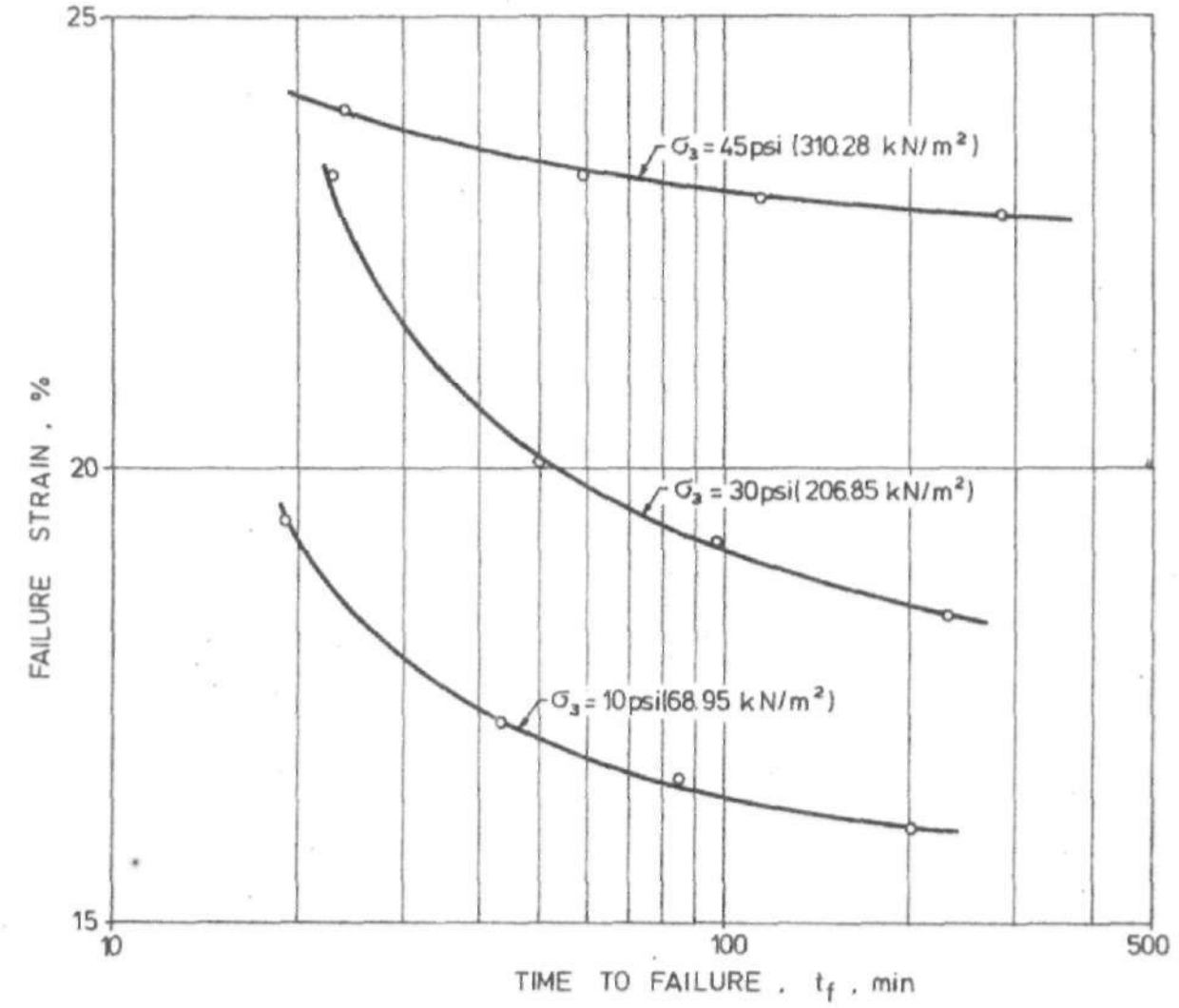


Figure 3.03 -- Influence of Time to Failure on the Failure Strain for Compacted Fadama Clay.

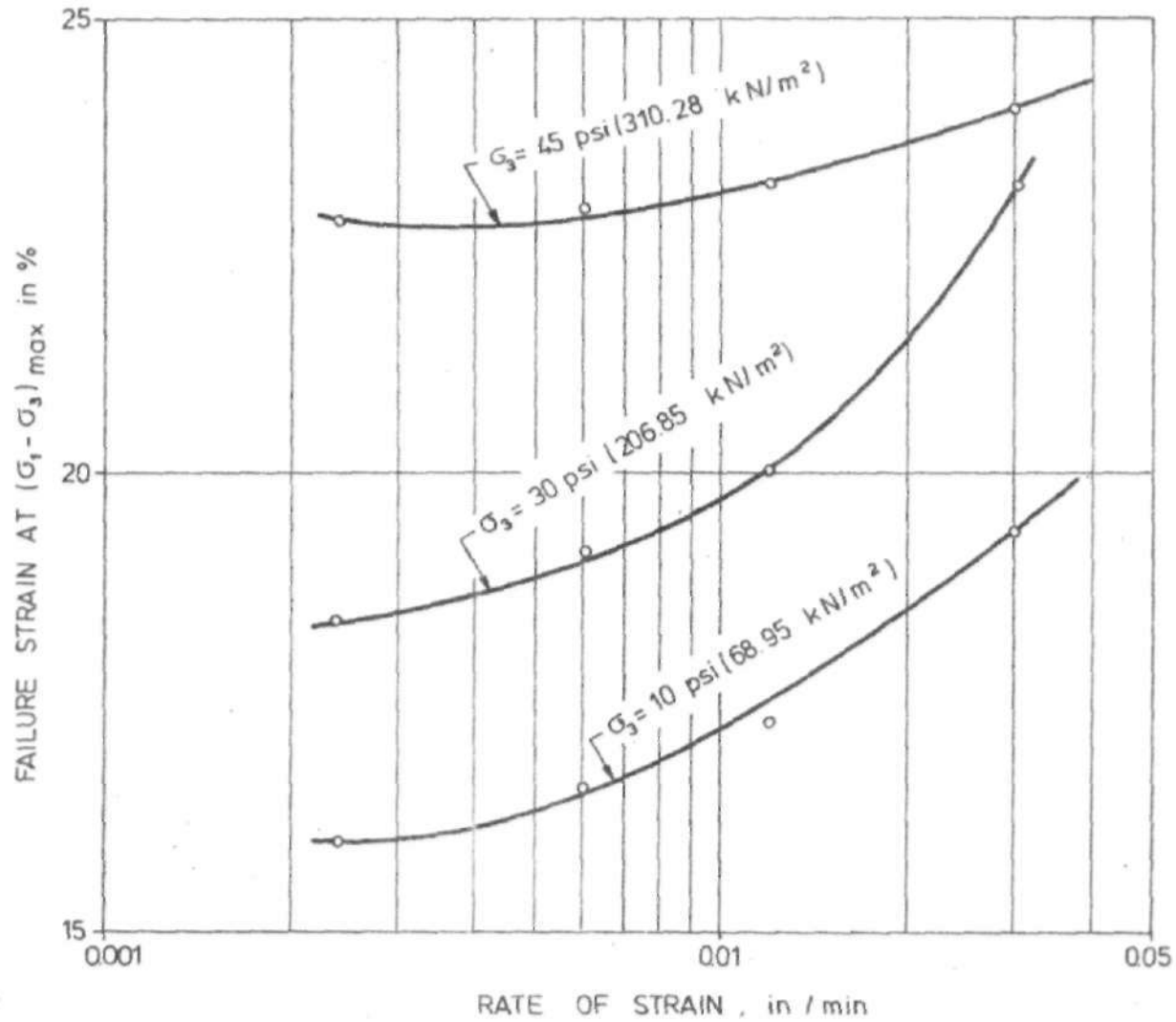


Figure 3.04 -- Influence of the Ratio of Strain on Failure Strain for Compacted Fadama Clay.

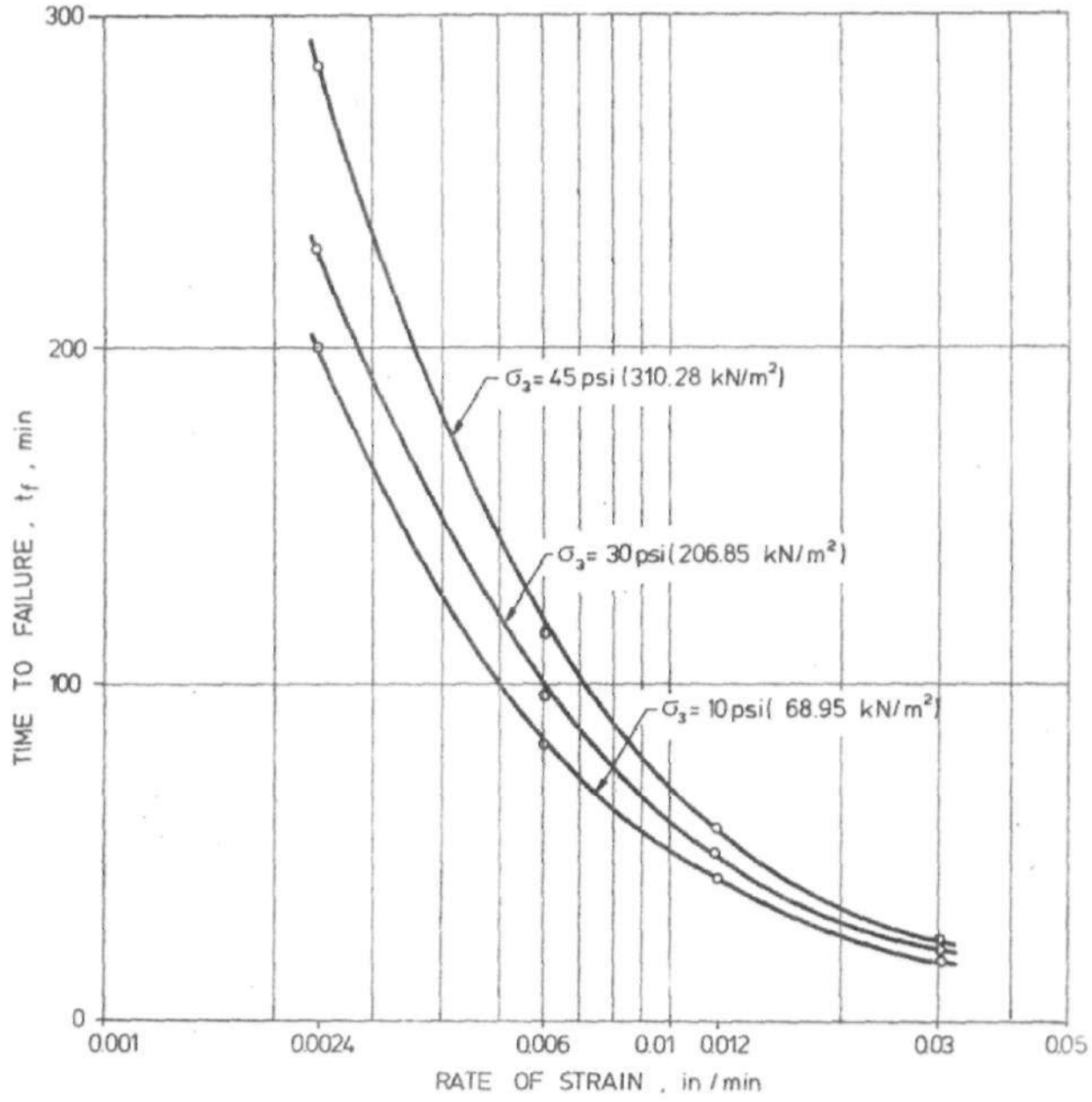


Figure 3.05 -- Influence of the Rate of Strain on Time to Failure.

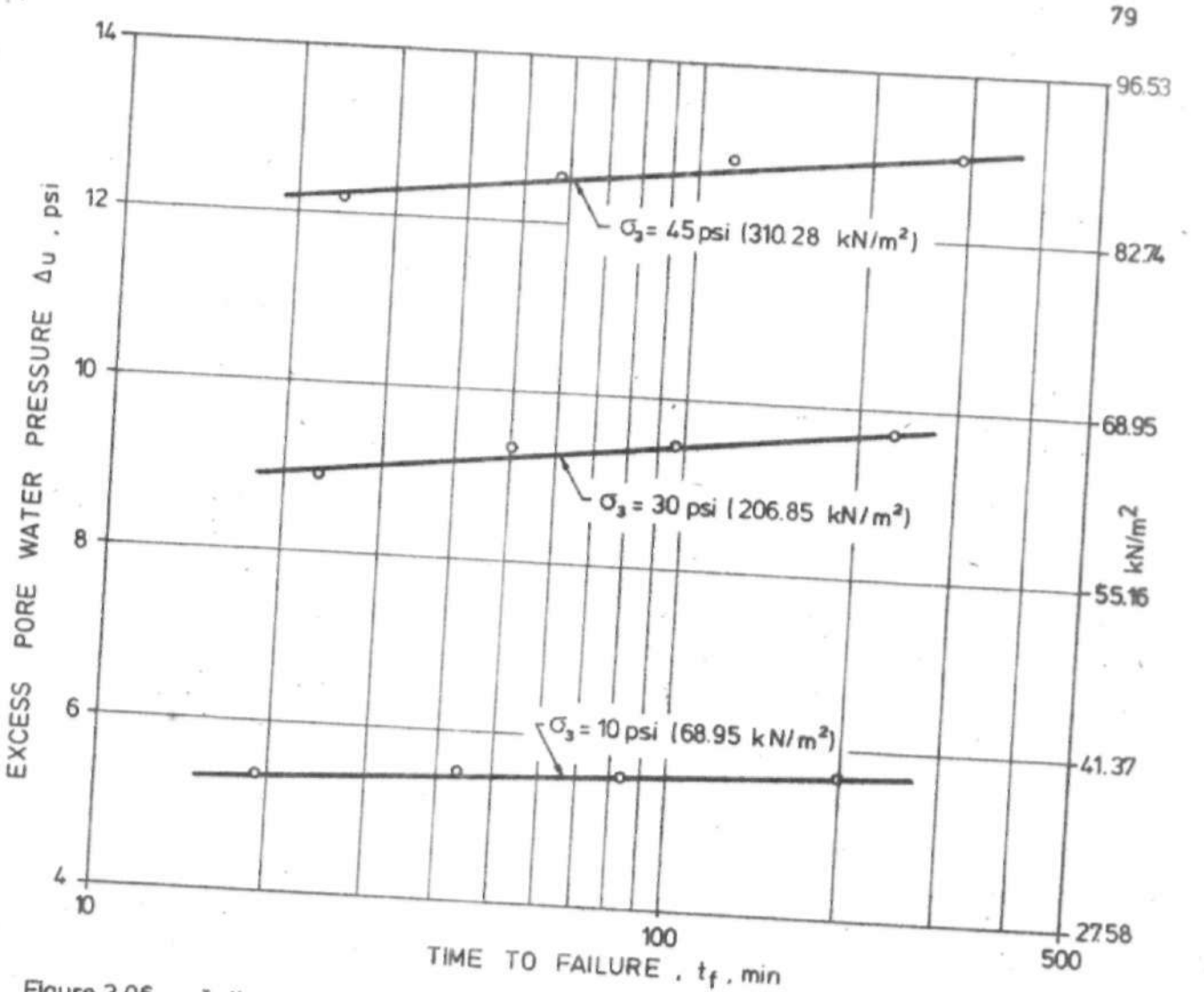


Figure 3.06 -- Influence of the Time to Failure on Excess Pore Water Pressure for Compacted Fadama Clay.

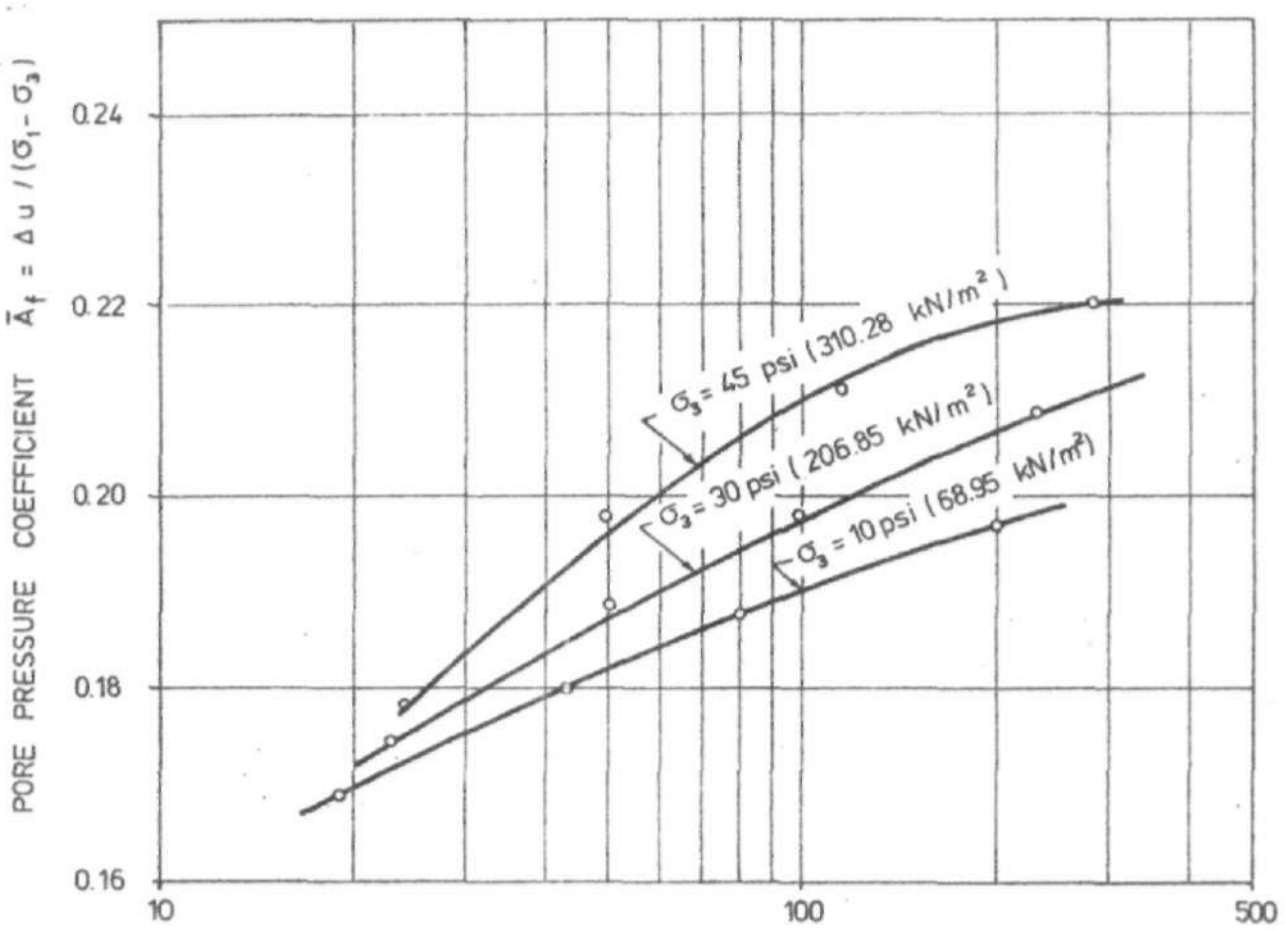


Figure . 3.07 -- Influence of the Time to Failure on Pore Pressure Coefficient at Failure for Compacted Fadama Clay.

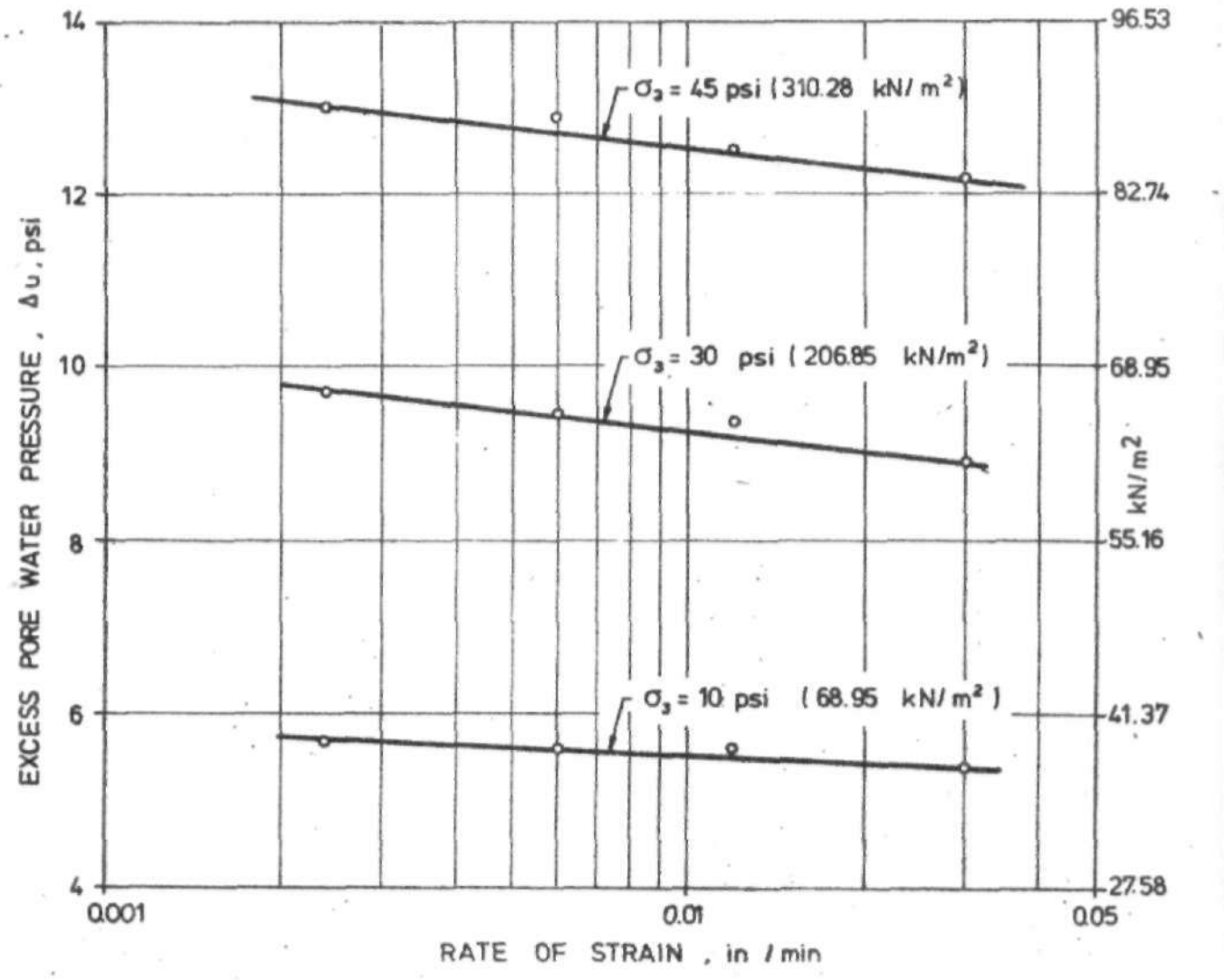


Figure 3.08 -- Influence of the Ratio of Strain on Excess Pore Water Pressure for Compacted Fadama Clay (Failure at Peak Excess Pore Water Pressure).

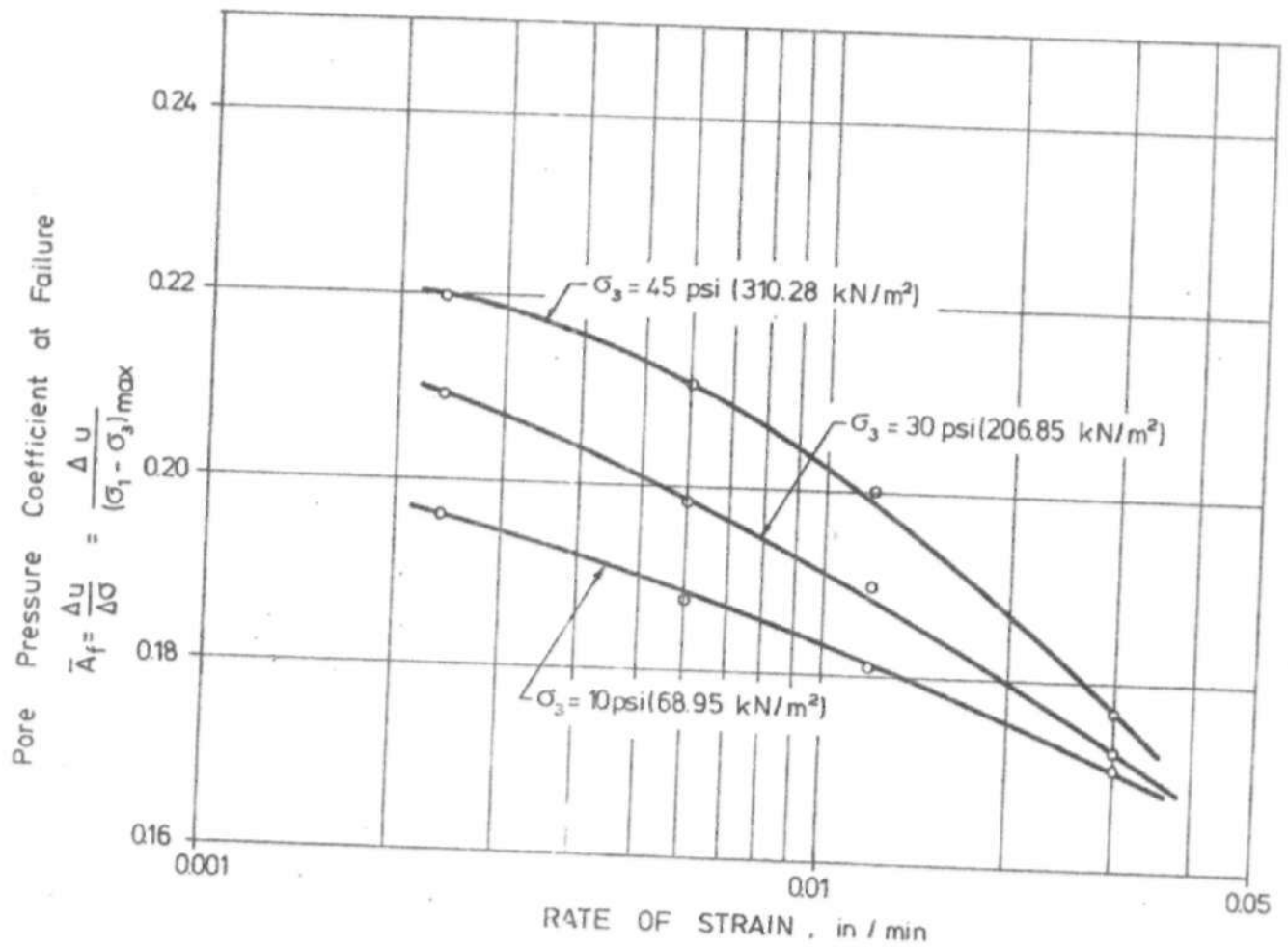


Figure 3.09 -- Influence of the Rate of Strain on Pore Pressure Coefficient at Failure for Compacted Fadama Clay.

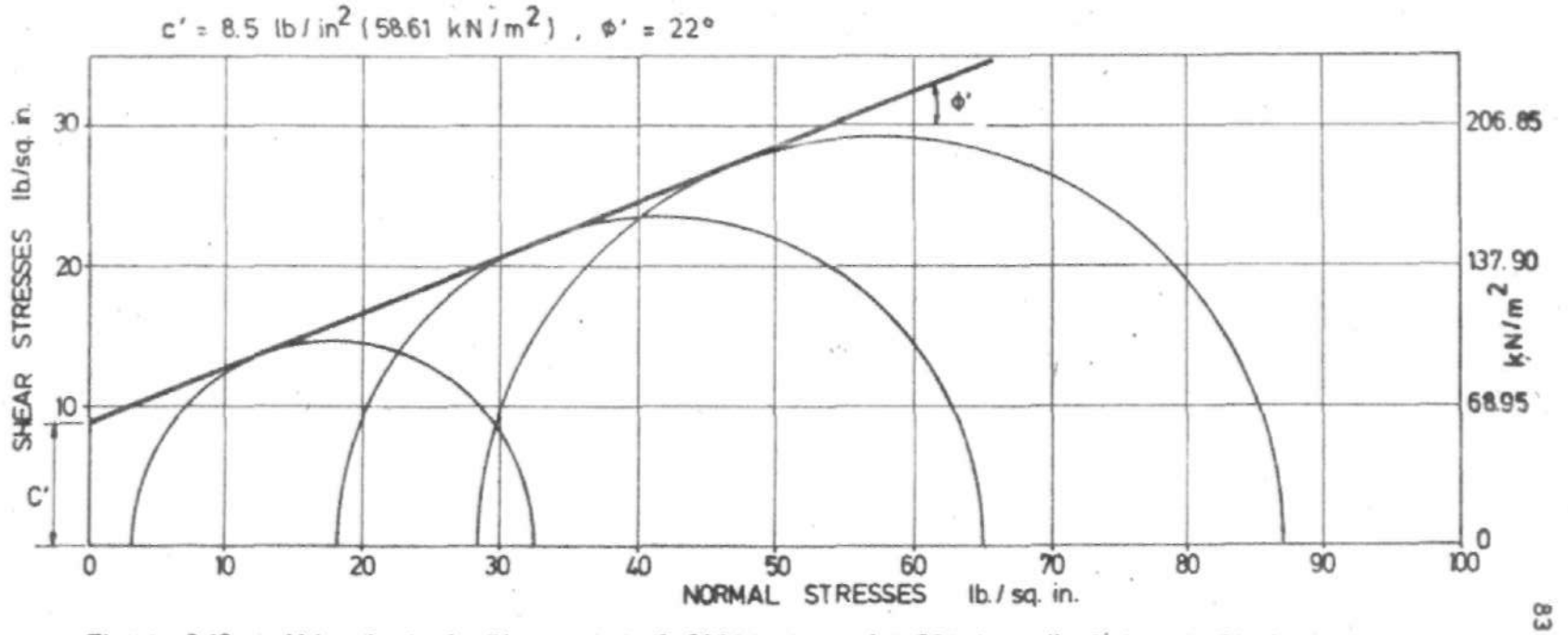


Figure 3.10 -- Mohr - Coulomb Diagram at 0.0024 in / min. (0.08 % / min.) Rate of Strain.

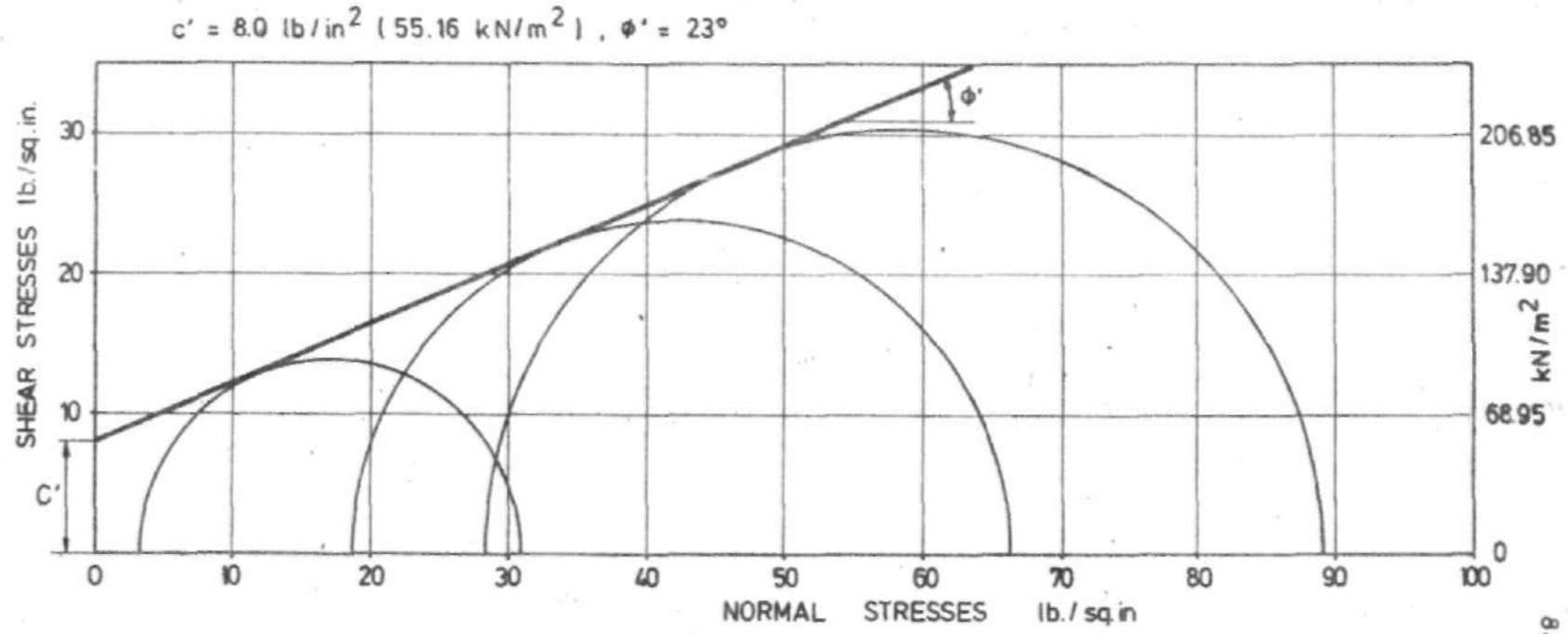


Figure 3.11 -- Mohr - Coulomb Diagram at 0.006 in/min. (0.12 %/min.) Rate of Strain.

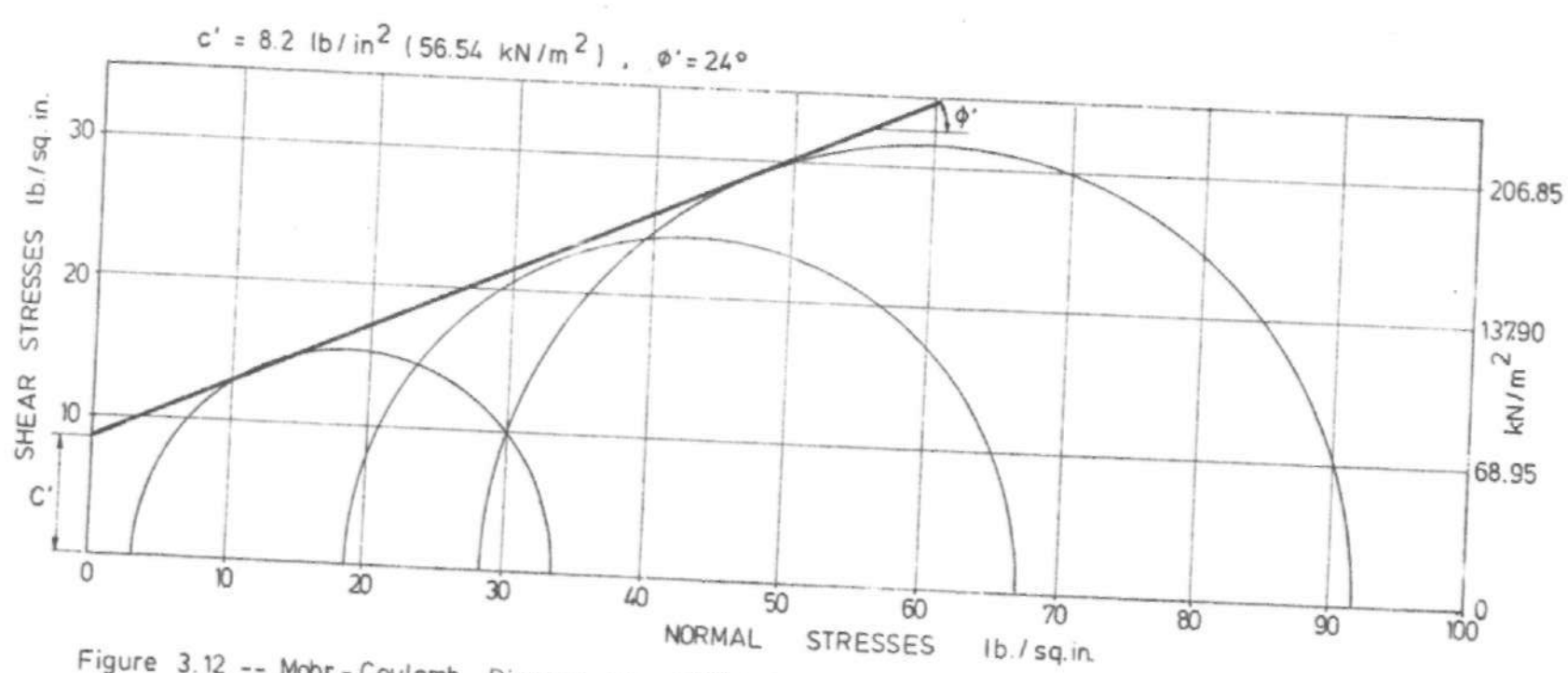


Figure 3.12 -- Mohr - Coulomb Diagram at 0.012 in / min. (0.4 %/ min.) Rate of Strain.

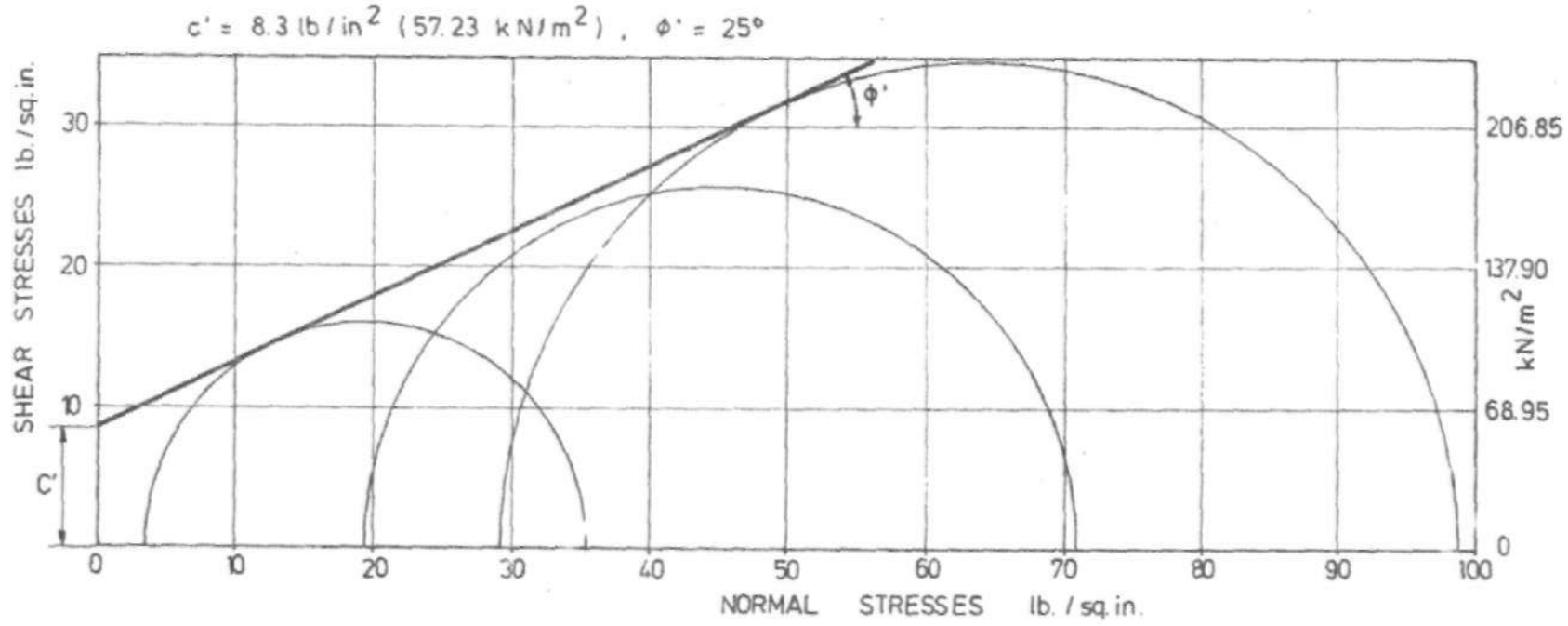


Figure 3.13 -- Mohr - Coulomb Diagram at 0.03 in/min. (1.0 %/min.) Rate of Strain.

CHAPTER 4

CONCLUSION AND RECOMMENDATION

4.1 CONCLUSION

In the preceeding chapters, the writer attempted to describe the composition and structure of compacted Fadama Clay and the manner in which the nature of such soil might influence its shear strength characteristics. Consideration of composition indicated that the strength of the compacted clay is likely to be influenced by the effective stresses in the soil, the time to failure, pore pressure development, the interparticle spacing at points of contact and the rate of loading. The strain-rate influence on the said factors is shown and demonstrated in the functional relationship models derived in this study.

From the analysis of results previously discussed, the relationship between the deviator stress and rate of strain is of the model: Stress = $\beta_0 + \beta_1 \log$ (Strain-Rate), where β_0 is a constant and β_1 is the coefficient of the log term.

The above model satisfies the findings in this study, that rate of strain increases as strength (deviator stress)

increases and vice-versa. It was also observed in this study of strain-rate effects on strength, that strength decreases for slower strain-rates. The findings in this study point to the fact that no functional relationship exist between deviator stress and strain-rate when cell pressure is not constant.

The variation in strength with time to failure is best explained by the model derived in this study as follows: Stress = $\beta_2 - \beta_1$ log (Time to Failure), where β_2 is a constant and β_1 is the coefficient of the log of time. The model clearly demonstrates the findings in this study that as the time to failure increases, strength decreases.

Considering the strain at failure, the rates of strain adopted in this study are believed to be sufficiently slow for equalization of pore water pressure at failure. At lower rates of strain, the time to failure seemed to be dependent on cell pressure up to 45 psi. (310.28 kN/m^2). However, as the rate of strain increased, the time to failure became independent of the cell pressure. The relationship between time to failure and rate of strain is best explained by the reciprocal regression of the rate of strain variable and the model derived is as follows: Time = $\alpha_0 + \alpha_1$ /Strain-Rate, where α_0 is a constant and α_1 is the coefficient factor of 1/Strain-Rate.

Experimental data revealed that for compacted Fadama Clay, the excess pore water pressure set up during the process of shearing increased with increased time to failure (slower rate of strain). The relationship between pore pressure coefficient and time to failure is of the model: Pore Pressure Coefficient, $\bar{A}_f = \psi_0 + \psi_1 \log$ (Time to Failure), where ψ_0 is a constant and ψ_1 is the coefficient of the log term. The relationship between pore pressure coefficient and rate of strain is of the model: Pore Pressure Coefficient, $\bar{A}_f = \psi_2 - \psi_3 \log$ (Strain-Rate), where ψ_2 is a constant and ψ_3 is the coefficient factor of the log term.

The present study also shows the dependence of the angle of internal friction (ϕ') on the strain-rate. The effective angle of internal friction increased for faster strain-rates. The effective cohesion (c') first attained its maximum value at slower rate of strain and decreased at faster strain-rate.

The above conclusions formulated from this study are intended to bring out the importance of the strain-rate effects on the shear strength characteristics of compacted Fadama Clay.

4.2 RECOMMENDATION

For further research on the strength characteristics of compacted Fadama Clay, it is recommended that (1) a comprehensive study of the fundamental nature of creep and creep-strength effects in clays, (2) a study of the influence of temperature, electrolyte concentration and other factors influencing interparticle forces on strength characteristics expressed in terms of effective stresses, and (3) a study of the influence of void ratio at failure on the strength characteristics in terms of effective stresses, be carried out.

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APPENDIX A

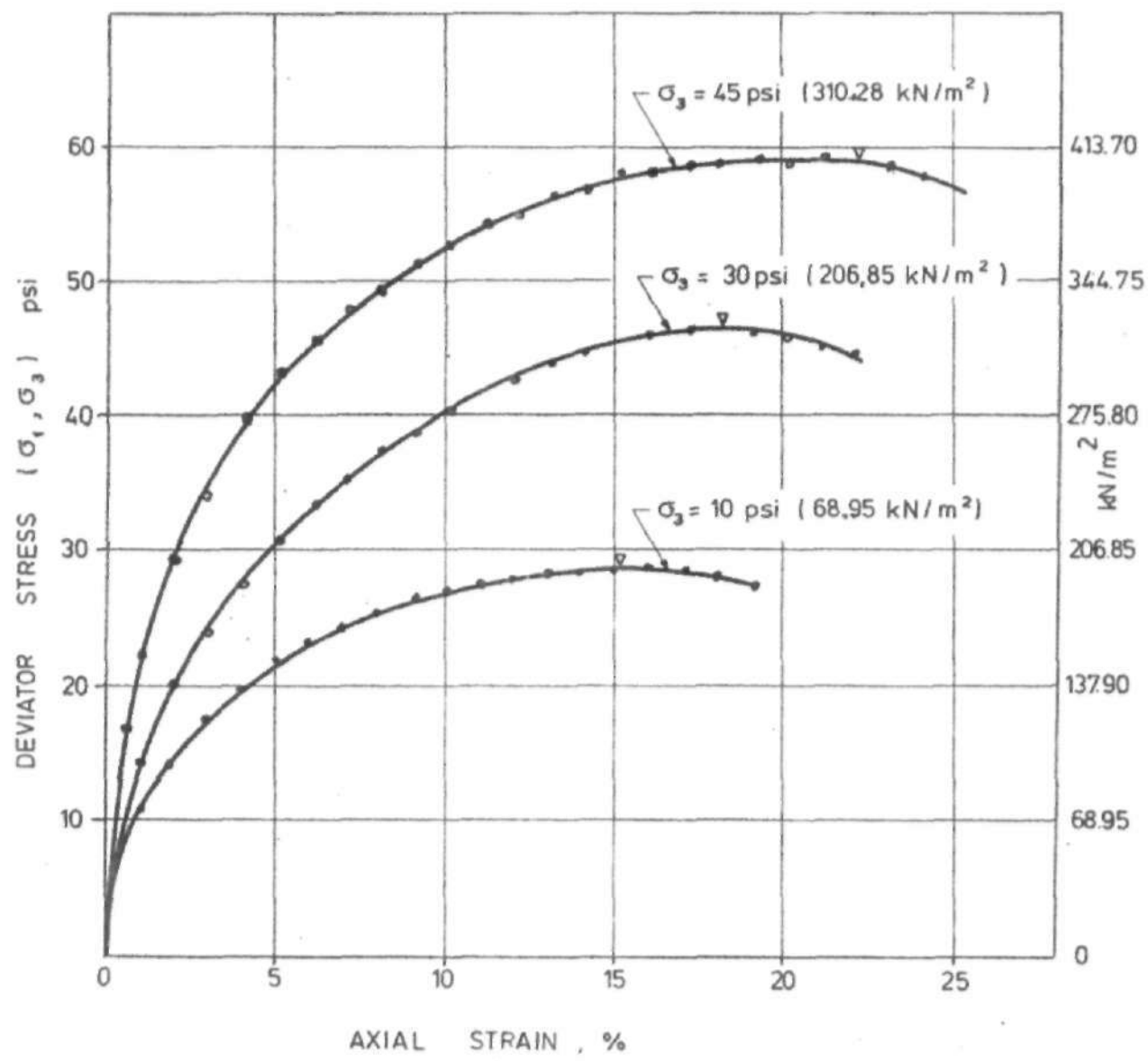


Figure 1 -- Deviator Stress Versus Axial Strain at 0.0024 in/min (0.08 %/min.) Rate of Strain - Test Series 1.

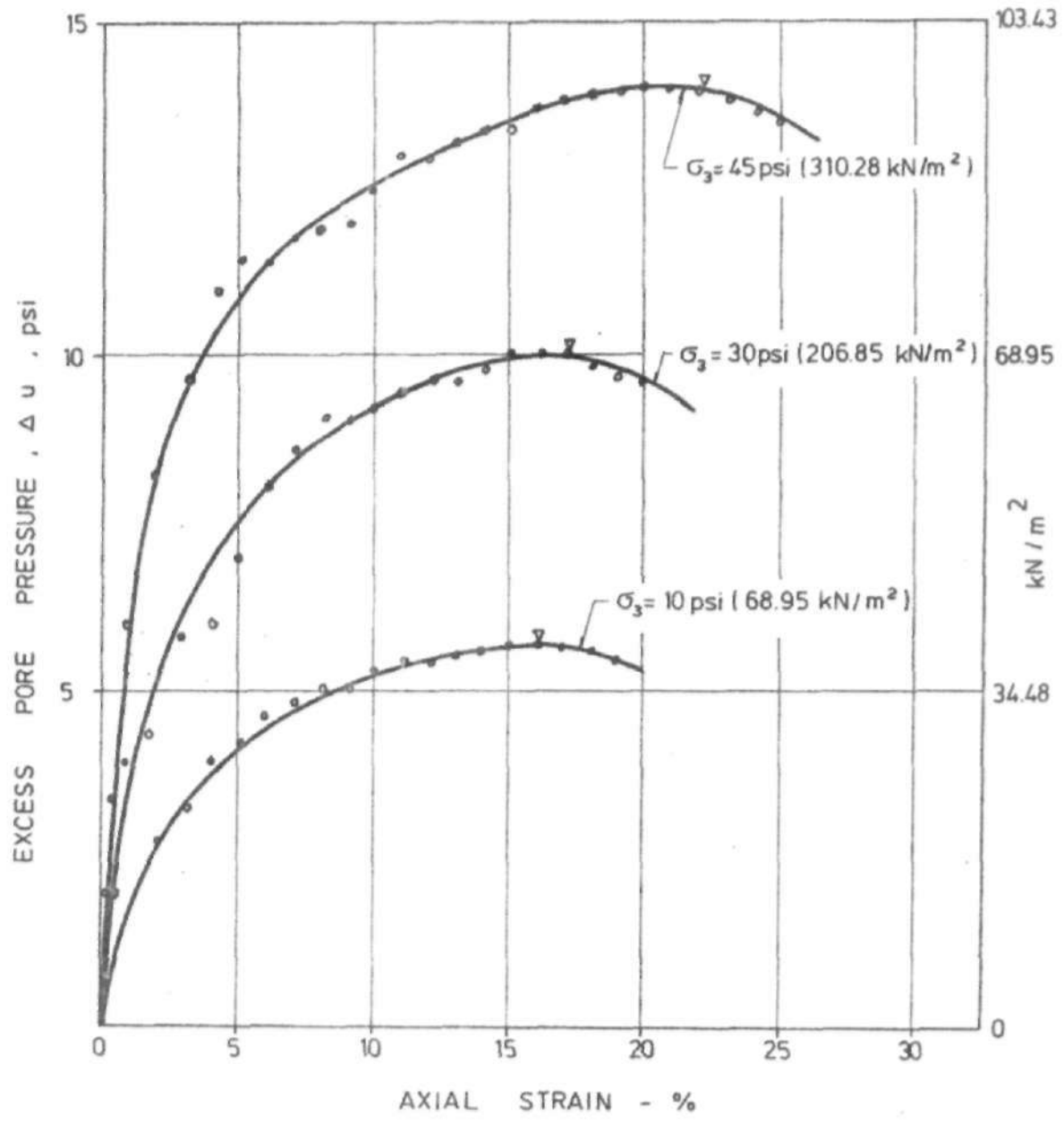


Figure 2 -- Excess Pore Pressure Versus Axial Strain at 0.0024 in/min (0.08% /min.) Rate of Strain - Test Series 1.

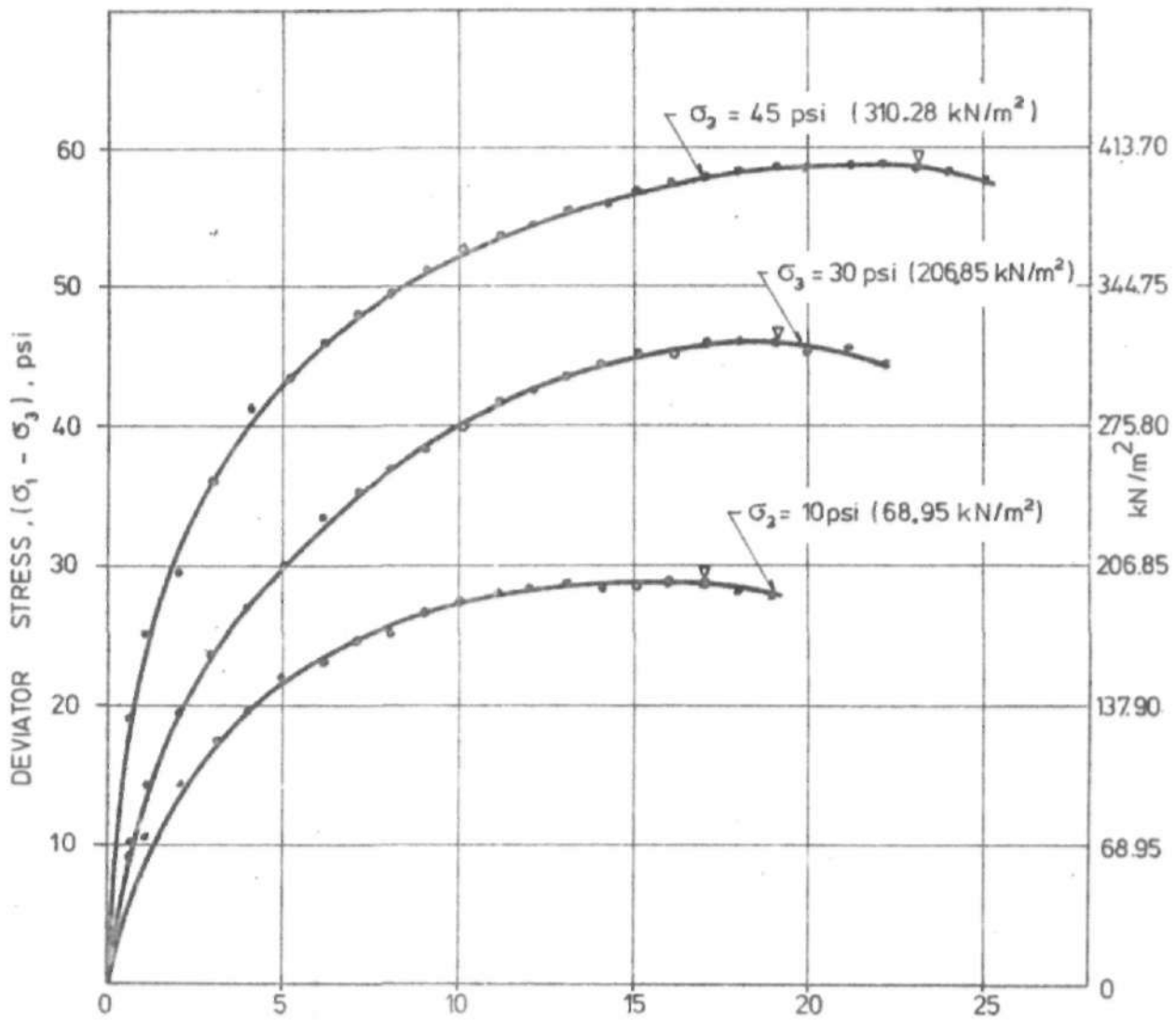


Figure 3 -- Deviator Stress Versus Axial Strain at 0.0024 in / min (0.08%/min.). Rate of Strain - Test Series 2.

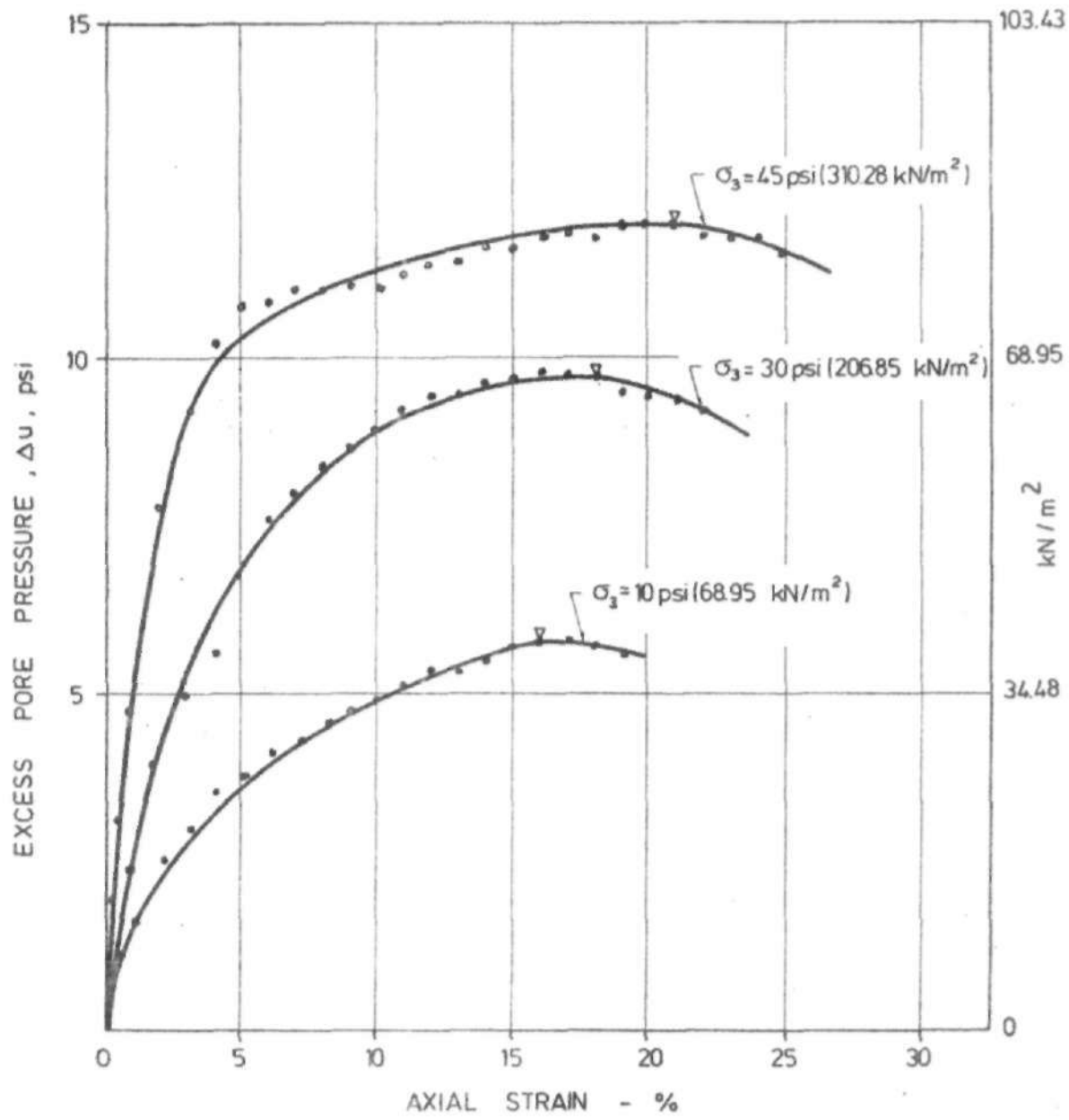


Figure 4 -- Excess Pore Pressure Versus Axial Strain at 0.0024 in / min (0.08 % / min.). Rate of Strain - Test Series 2.

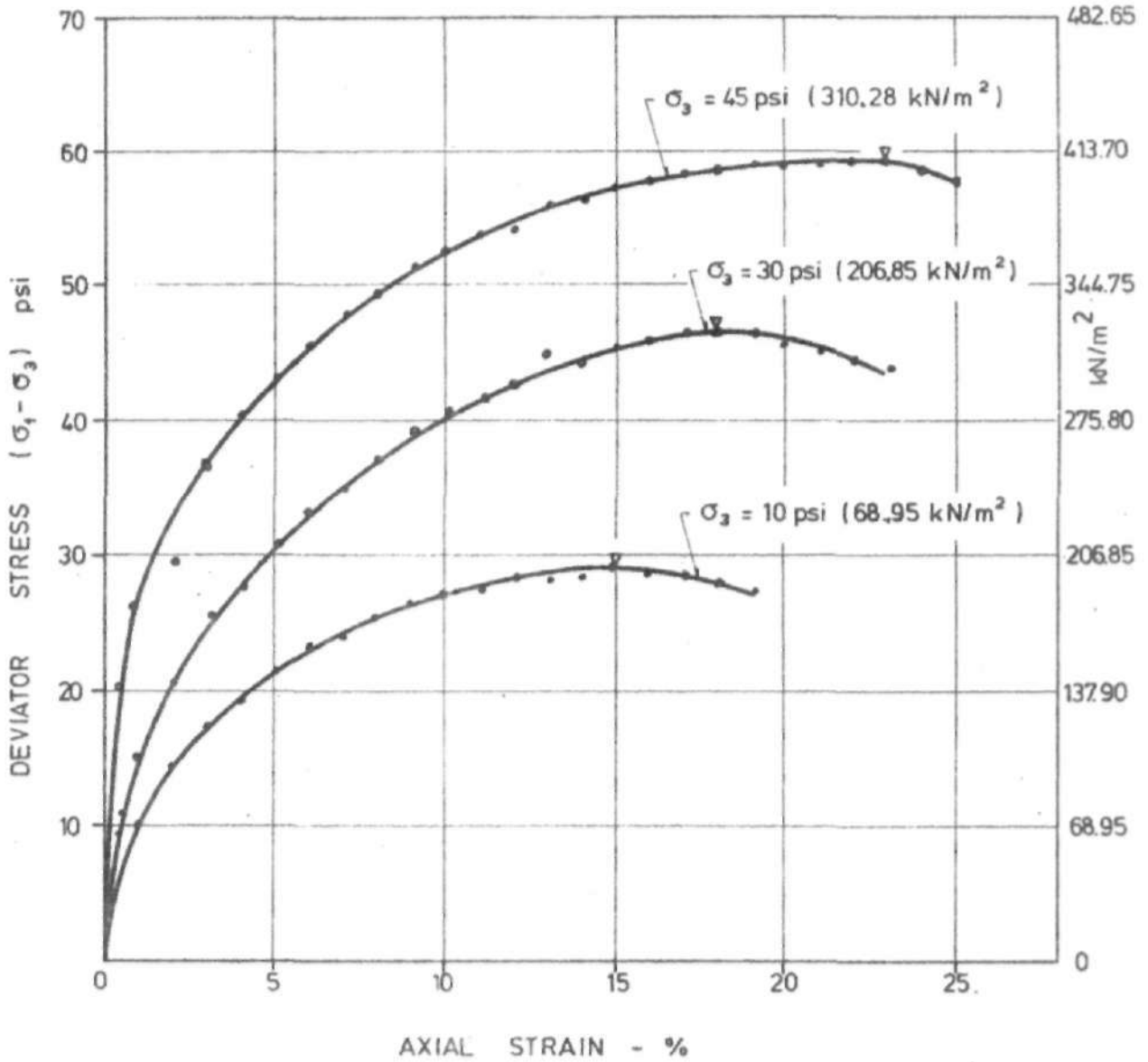


Figure 5 -- Deviator Stress Versus Axial Strain at 0.0024 in/min (0.08%/min.) Rate Strain - Test Series 3.

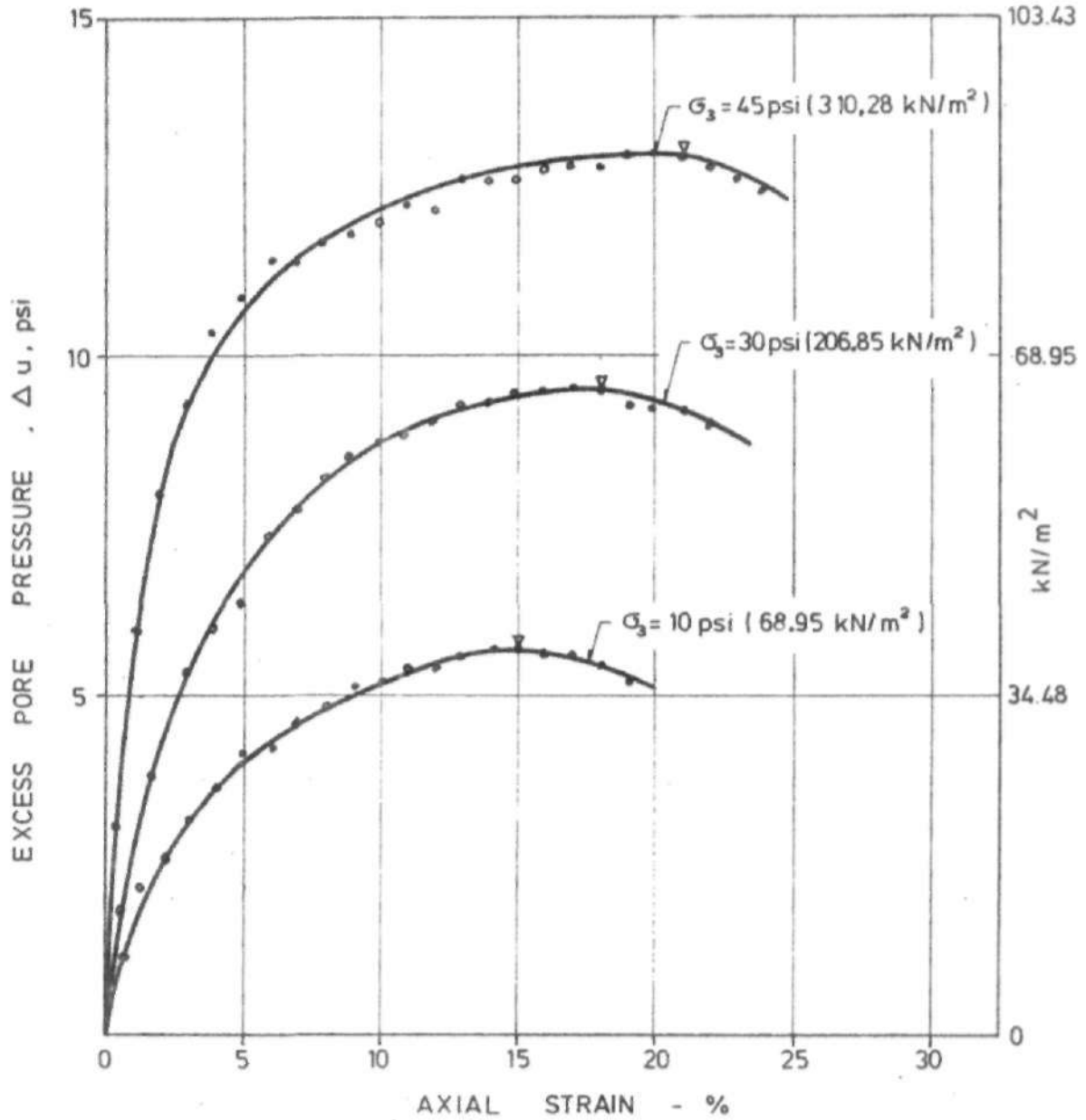


Figure 6 -- Excess Pore Pressure Versus Axial Strain at 0.0024 in/min (0.08%/min.). Rate of Strain - Test Series 1.

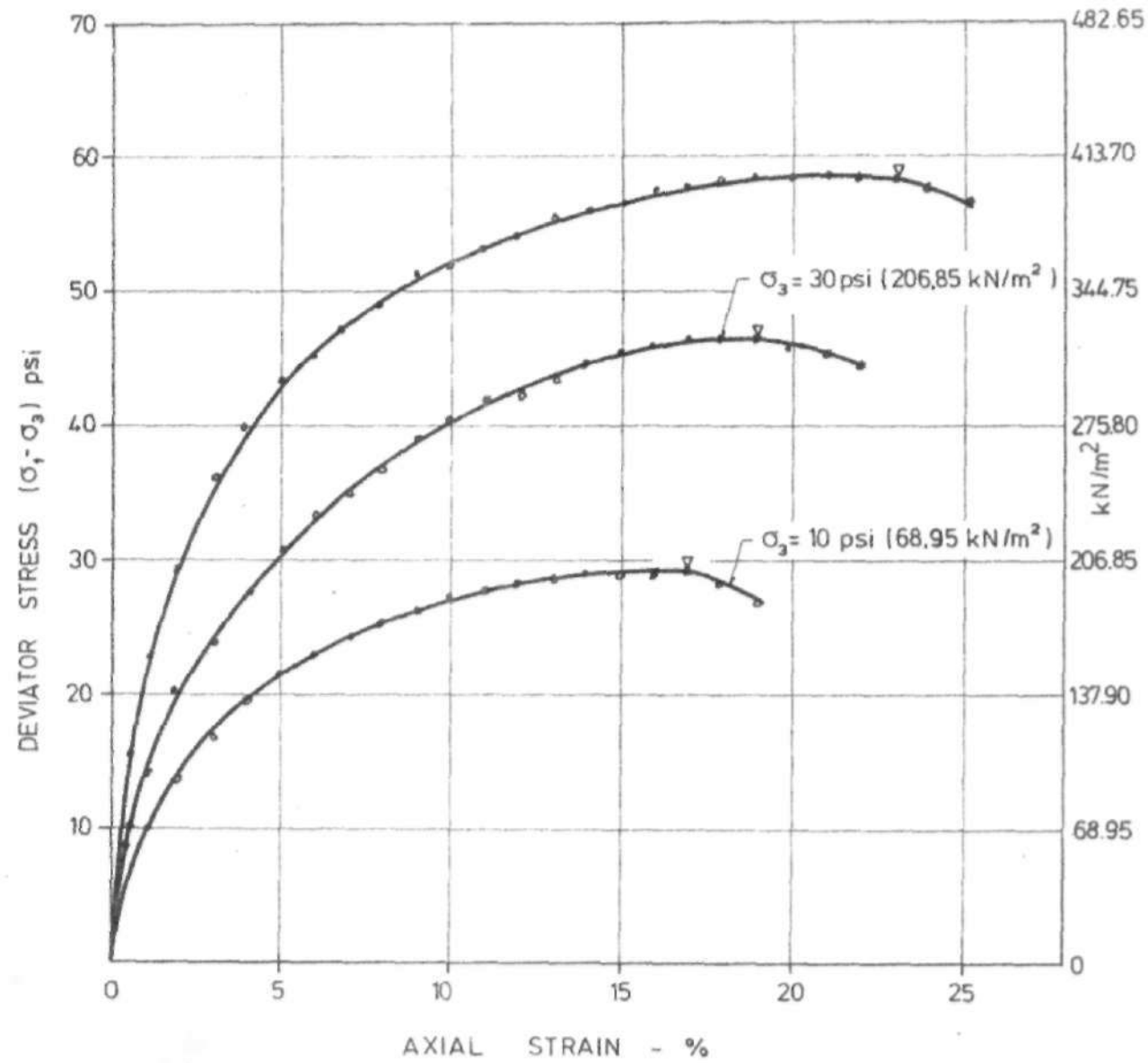


Figure 7 -- Deviator Stress Versus Axial Strain at 0.0024 in/min (0.08%/min.) Rate Strain - Test Series 4.

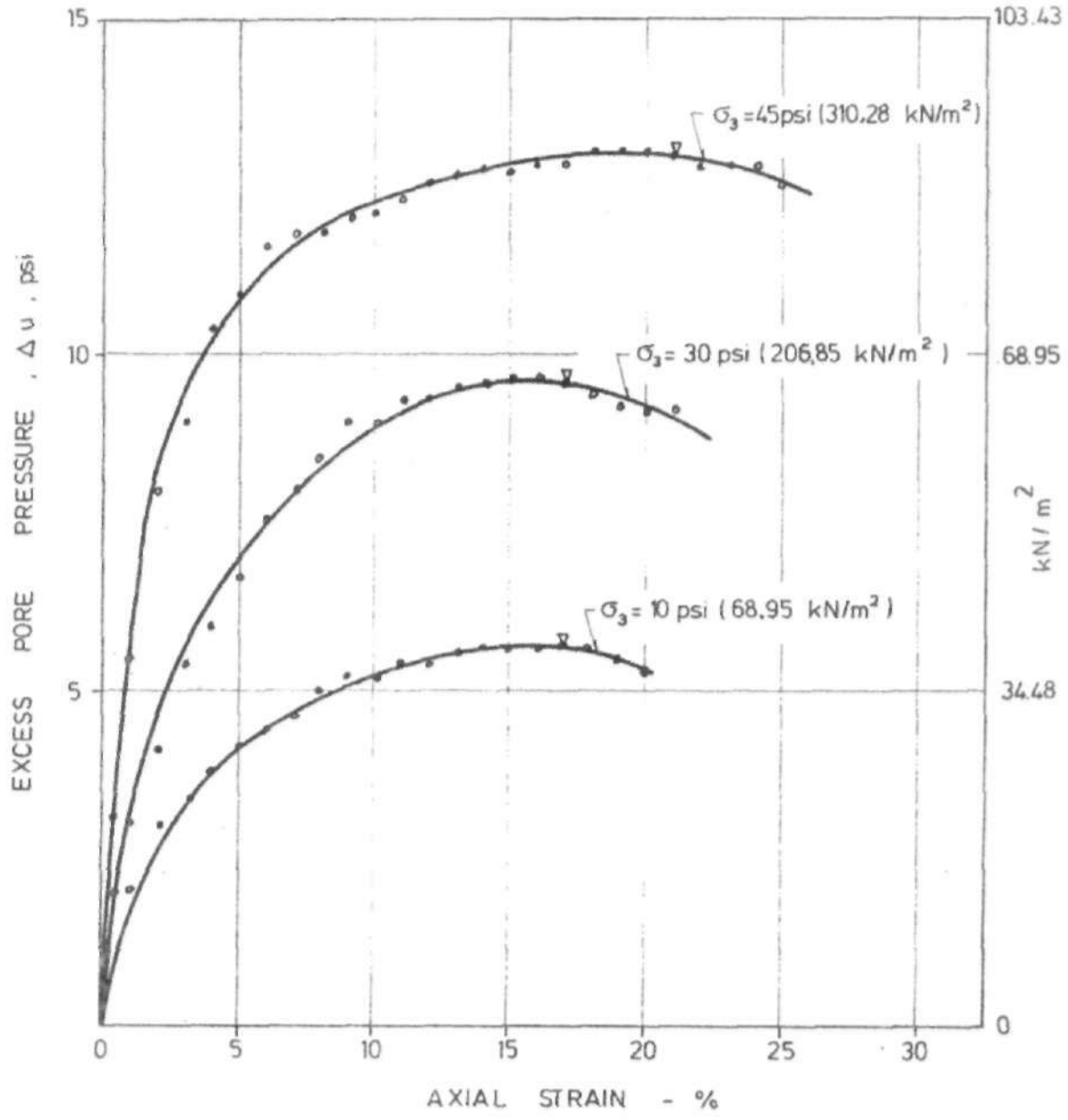


Figure 8 -- Excess Pore Pressure Versus Axial Strain at 0.0024in/min (0.08 % / min.). Rate of Strain - Test Series 4.

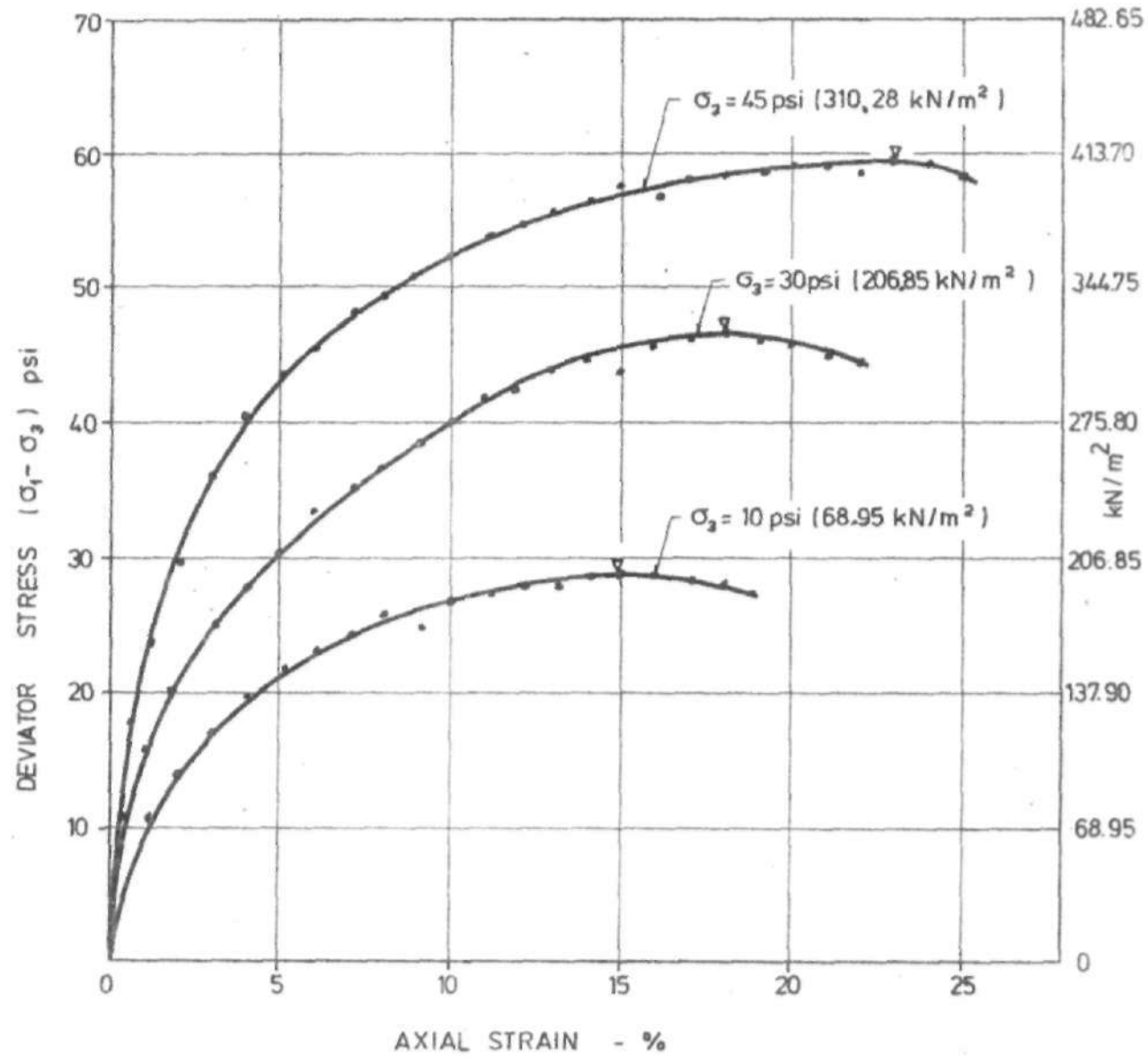


Figure 9 -- Deviator Stress Versus Axial Strain at 0.0024 in/min (0.06% / min.). Rate of Strain - Test Series 5.

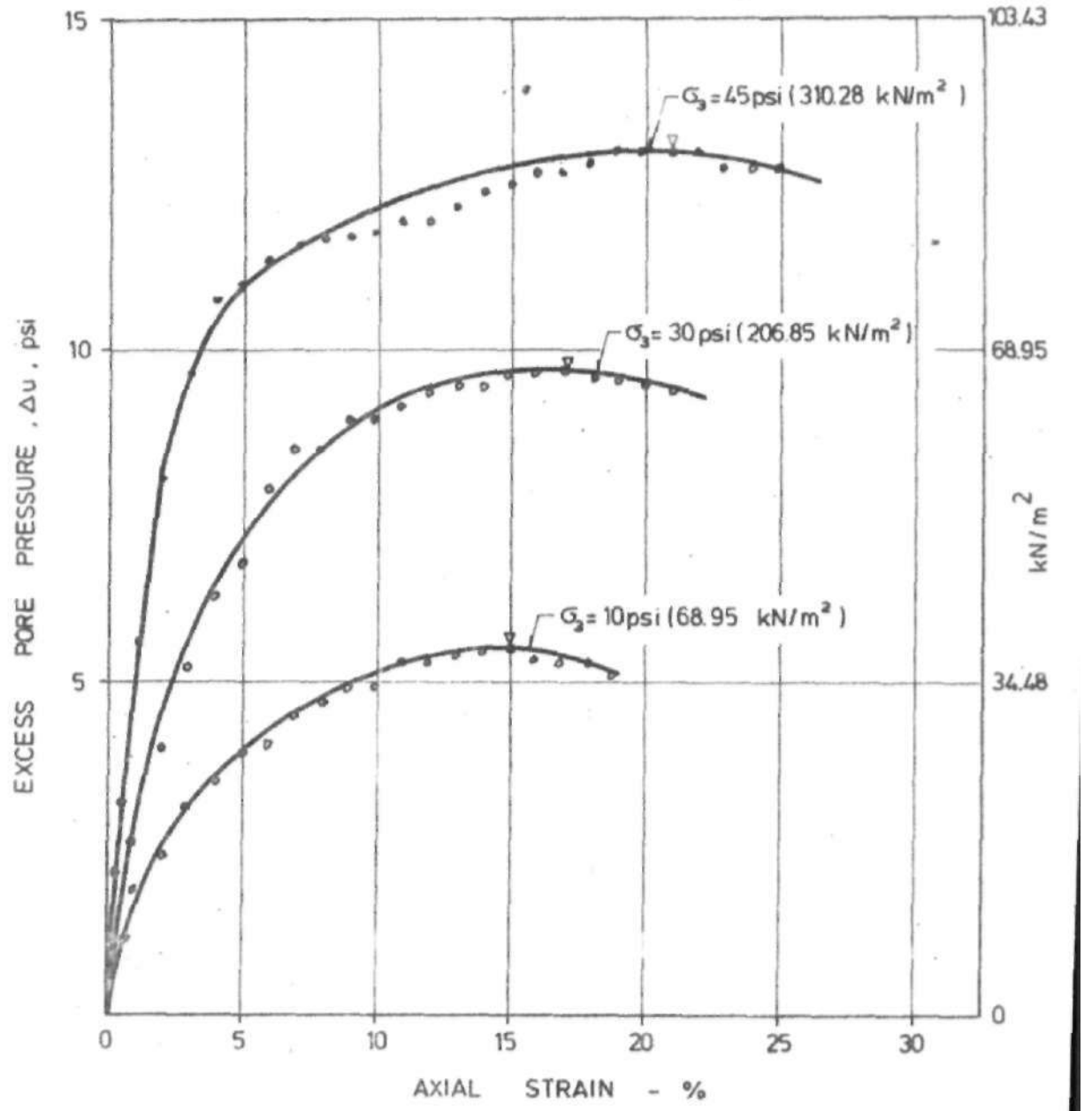


Figure 10 -- Excess Pore Pressure Versus Axial Strain at 0.0024 in/min (0.08 % / min.). Rate of Strain - Test Series 5.

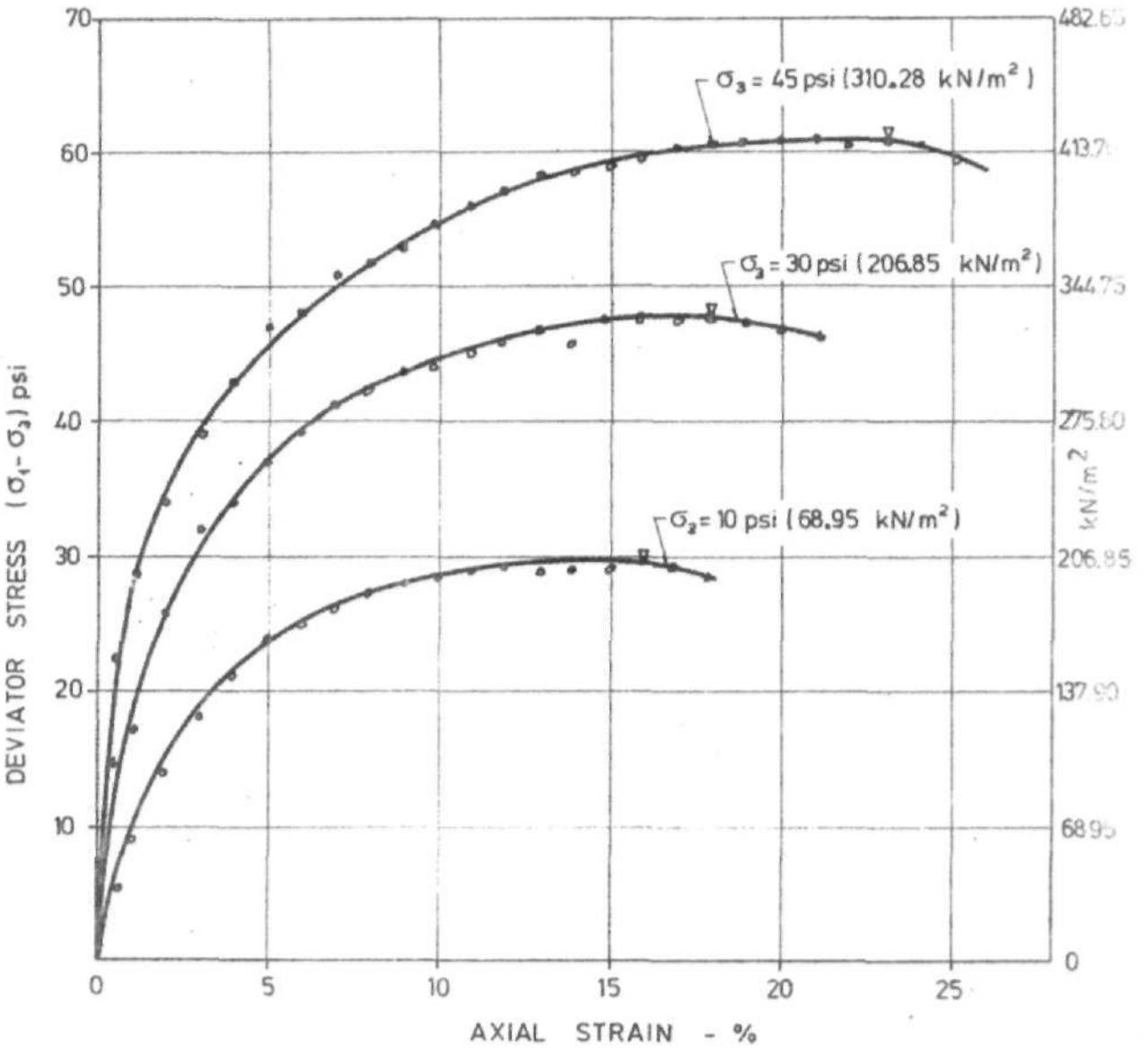


Figure 11 -- Deviator Stress Versus Axial Strain at 0.006 in/min. (0.2% /min.) Rate of Strain - Test Series 1.

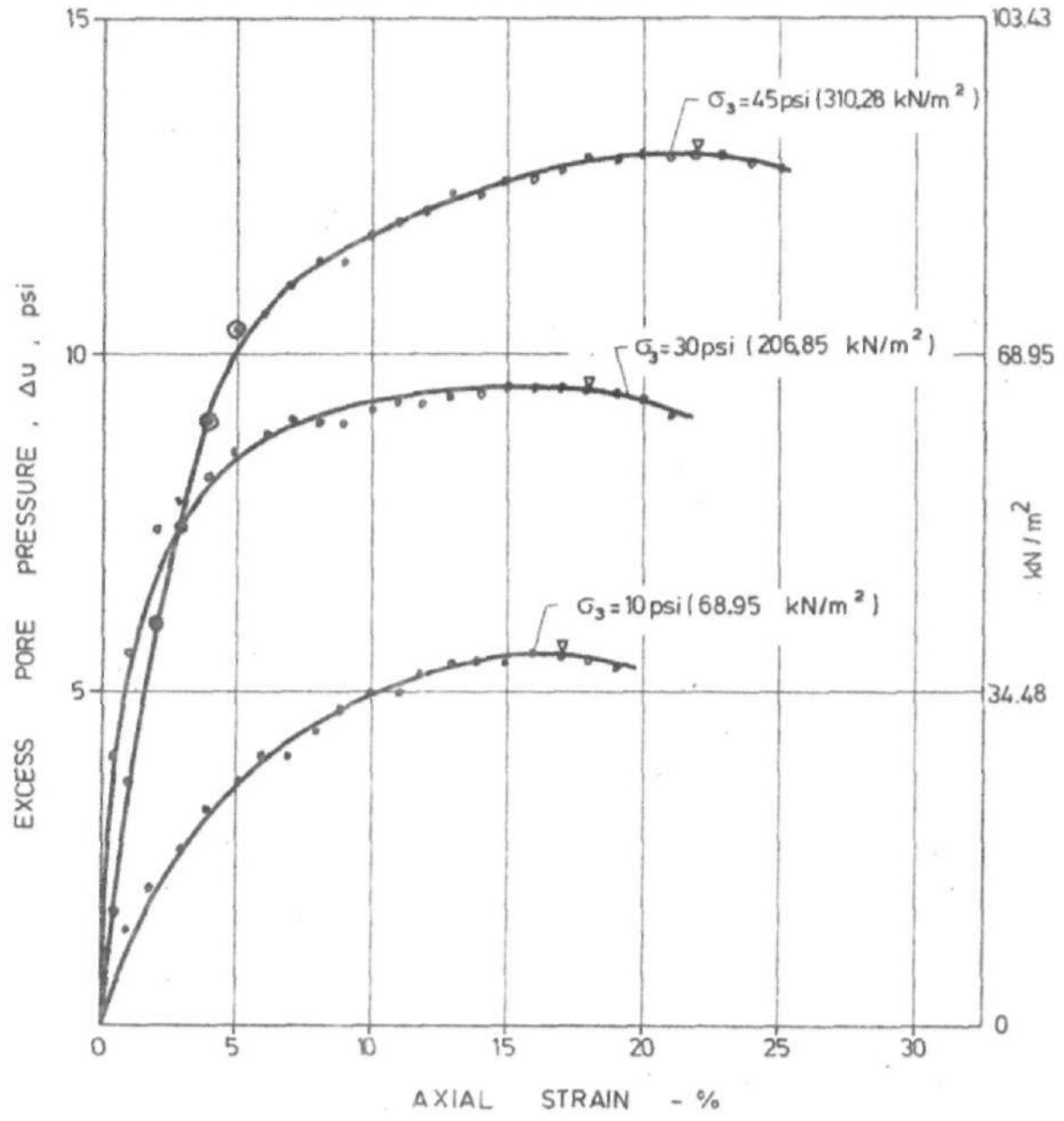


Figure 12 -- Excess Pore Pressure Versus Axial Strain at 0.006 in/min. (0.2 %/min.) Rate of Strain - Test Series 1.

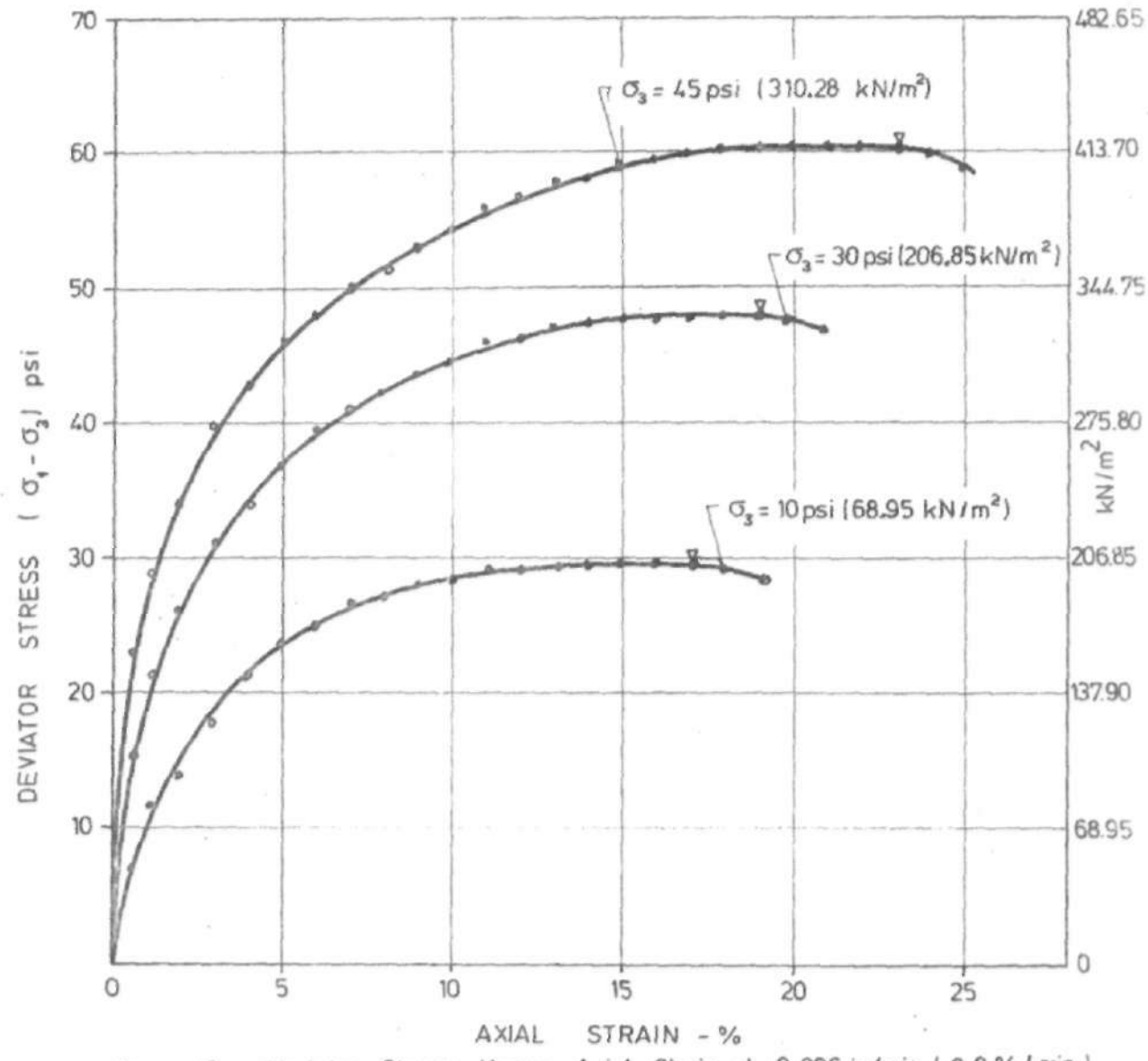


Figure 13 -- Deviator Stress Versus Axial Strain at 0.006 in/min. (0.2 % / min.) Rate of Strain - Test Series 2.

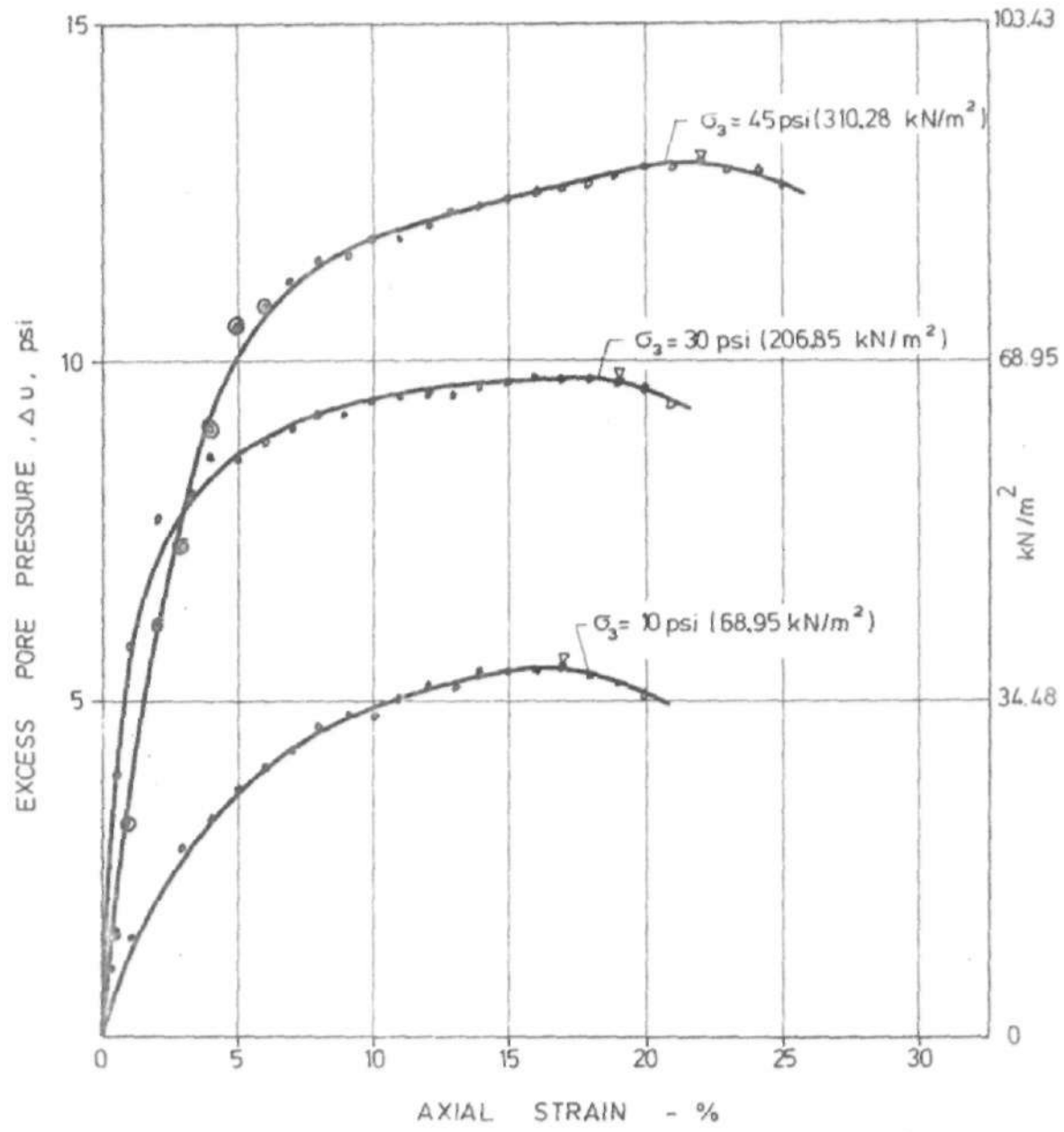


Figure 14 -- Excess Pore Pressure Versus Axial Strain at 0.006 in/min. (0.2 % / min.). Rate of Strain - Test Series 2.

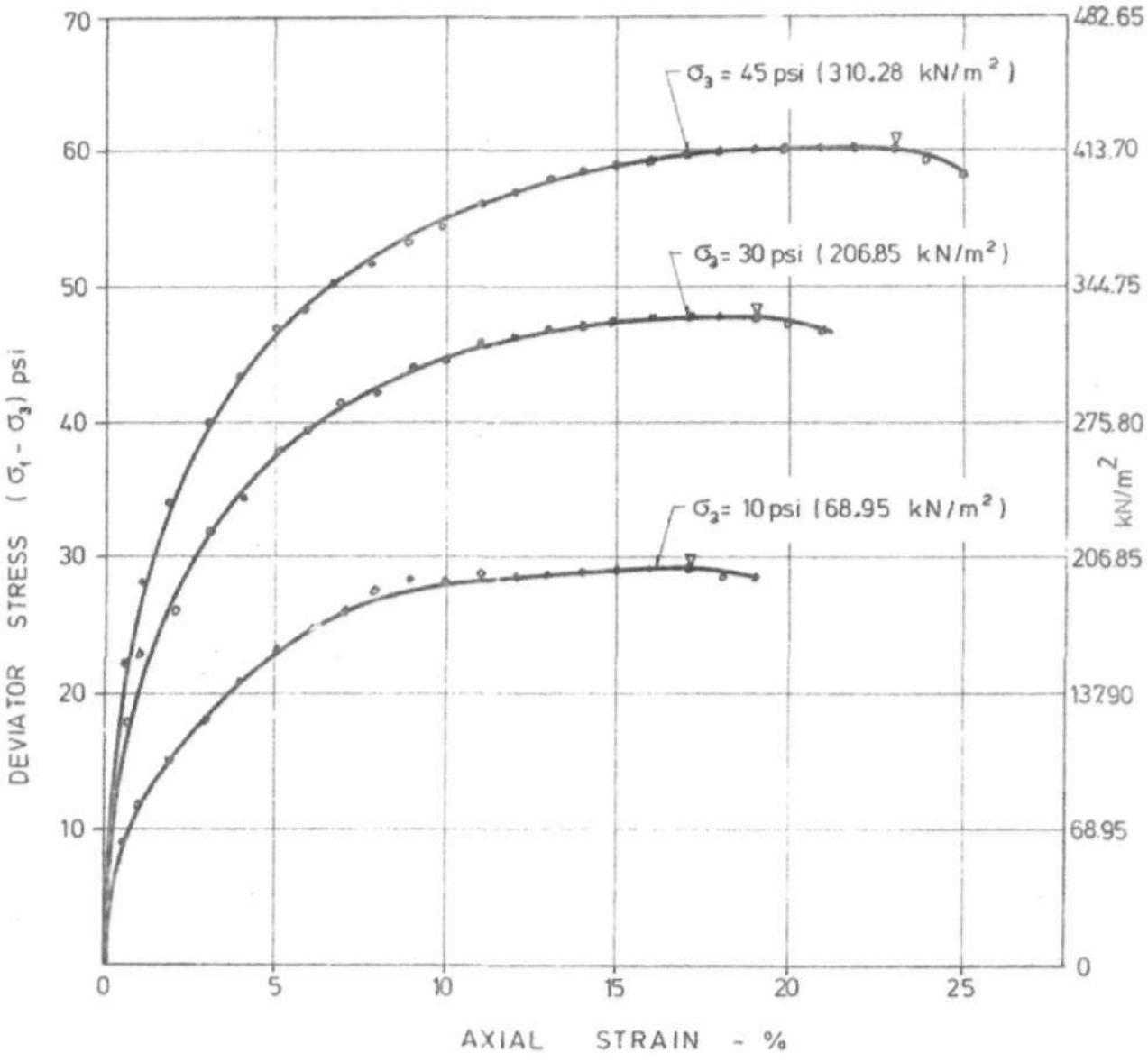


Figure 15 -- Deviator Stress Versus Axial Strain at 0.006 in/min. (0.2% /min.) Rate o' Strain - Test Series 3.

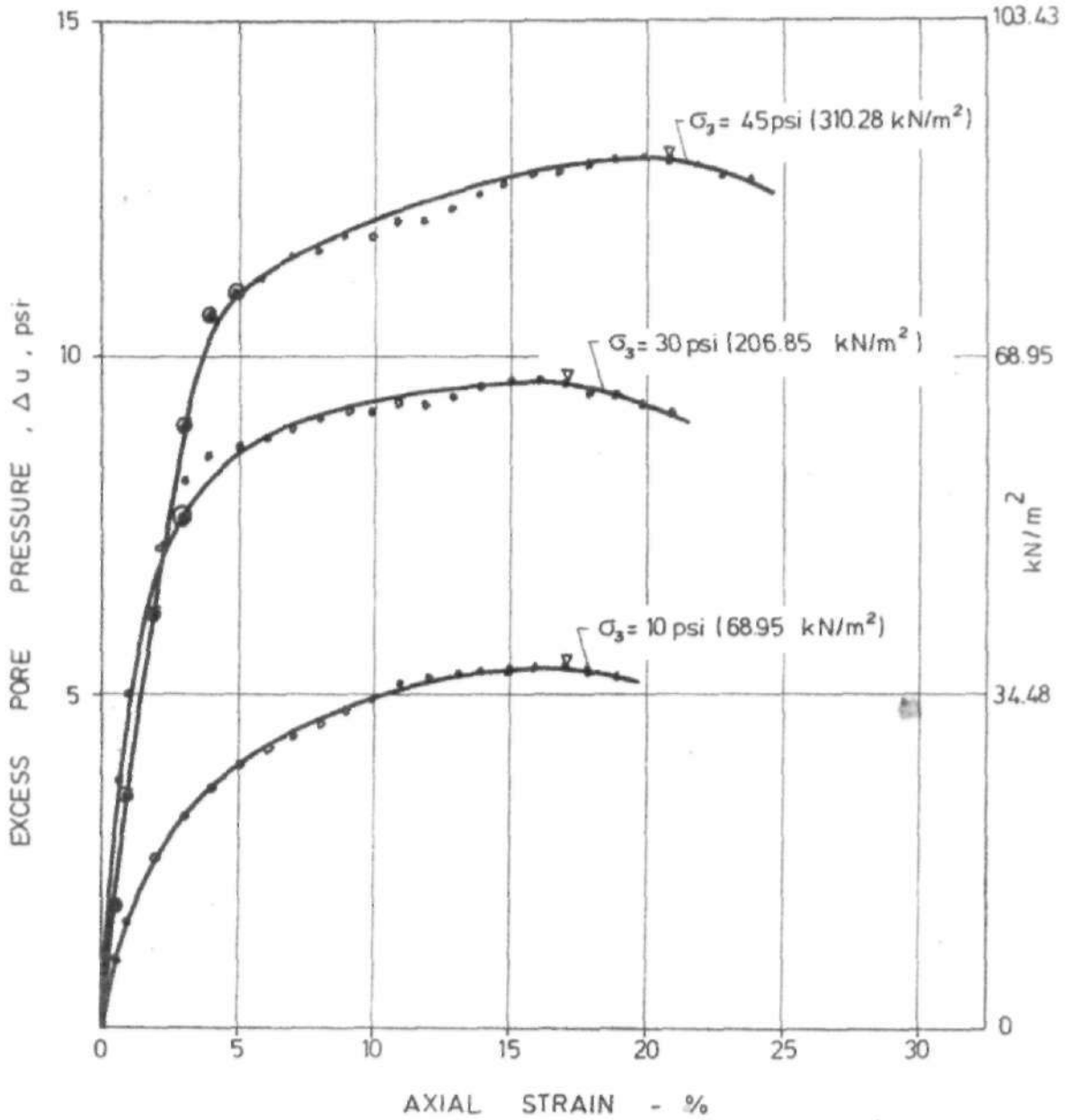


Figure 16 -- Excess Pore Pressure Versus Axial Strain at 0.006 in/min. (0.2%/min.). Rate of Strain - Test Series 3.

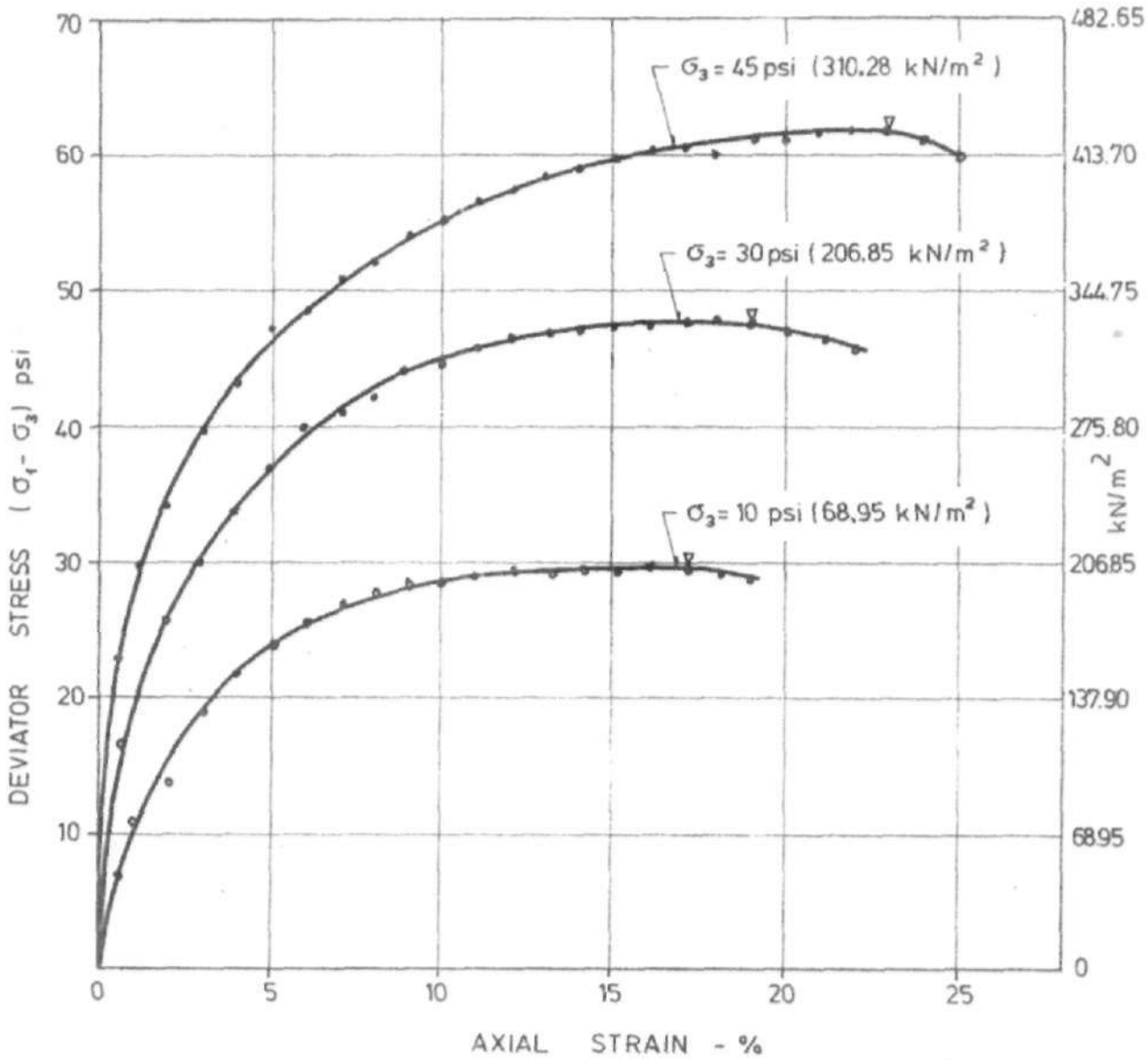


Figure 17 -- Deviator Stress Versus Axial Strain at 0.006 in/min. (0.2 % / min.) Rate of Strain - Test Series 4.

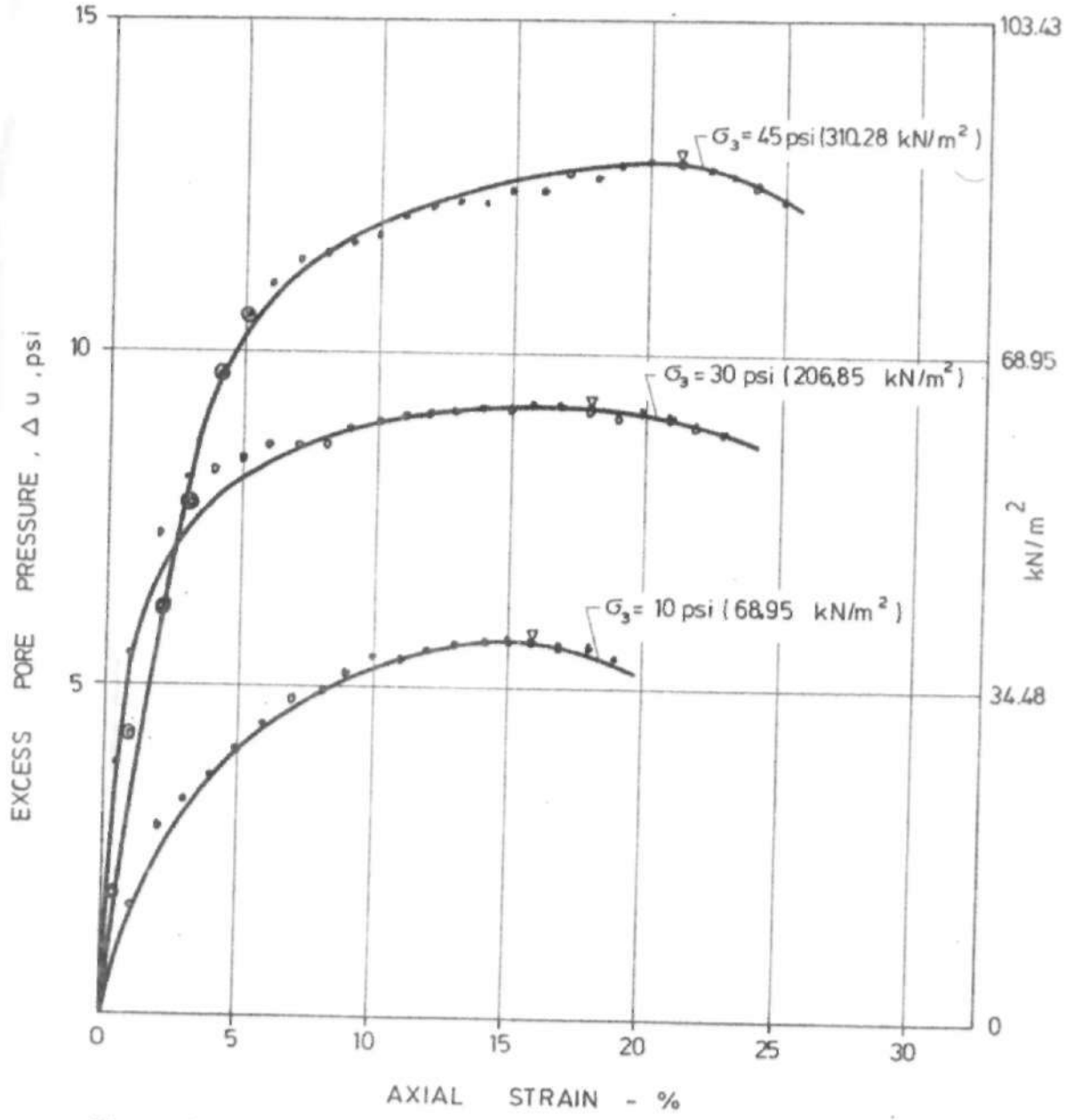


Figure 18 -- Excess Pore Pressure Versus Axial Strain at 0.006 in/min. (0.2%/min.). Rate of Strain - Test Series 4.

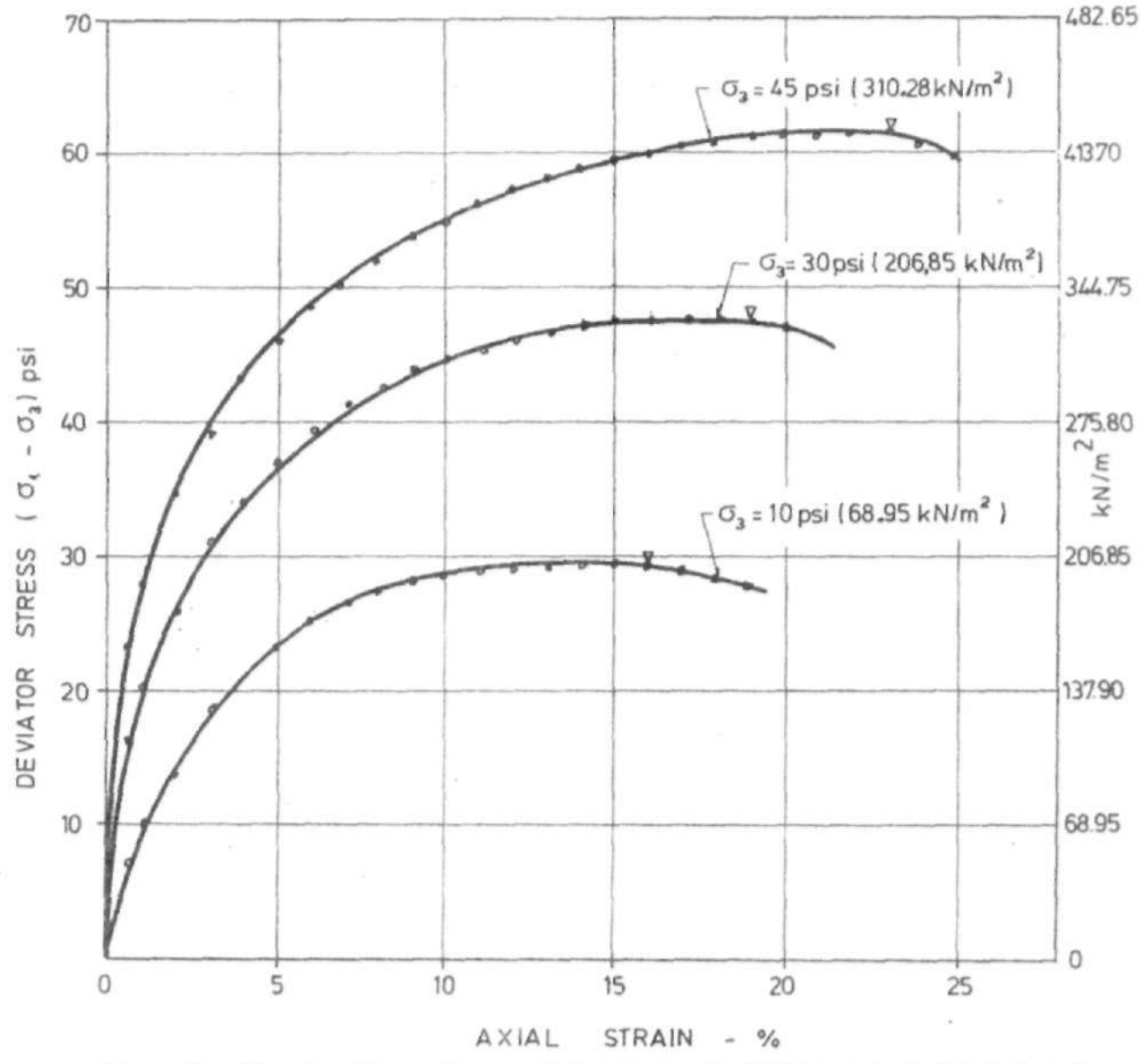


Figure 19 -- Deviator Stress Versus Axial Strain at 0.006 in/min. (0.2 % / min.) Rate of Strain - Test Series 5.

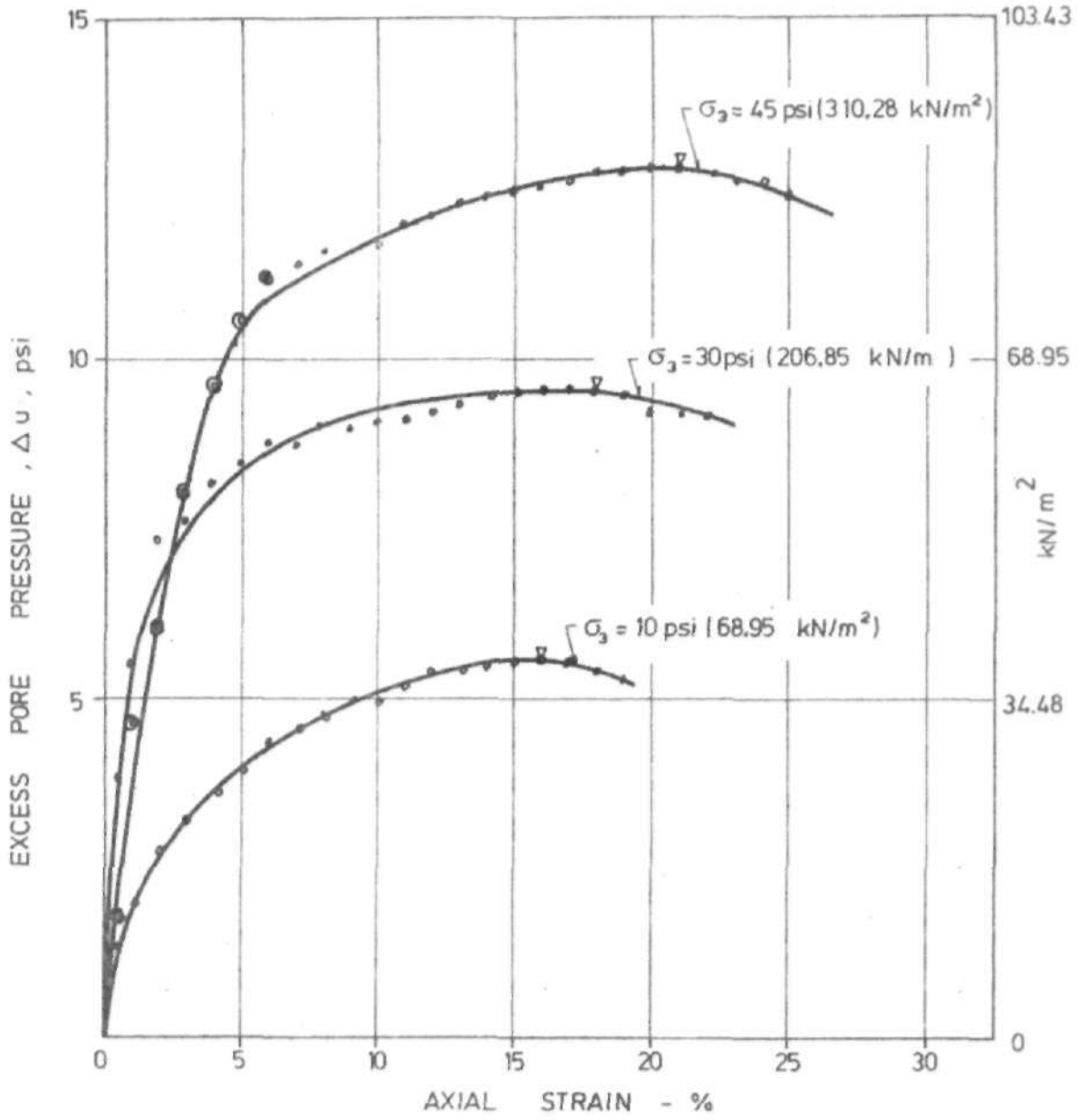


Figure 20 -- Excess Pore Pressure Versus Axial Strain at 0.006 in/min. (0.2%/min.). Rate of Strain - Test Series 5.

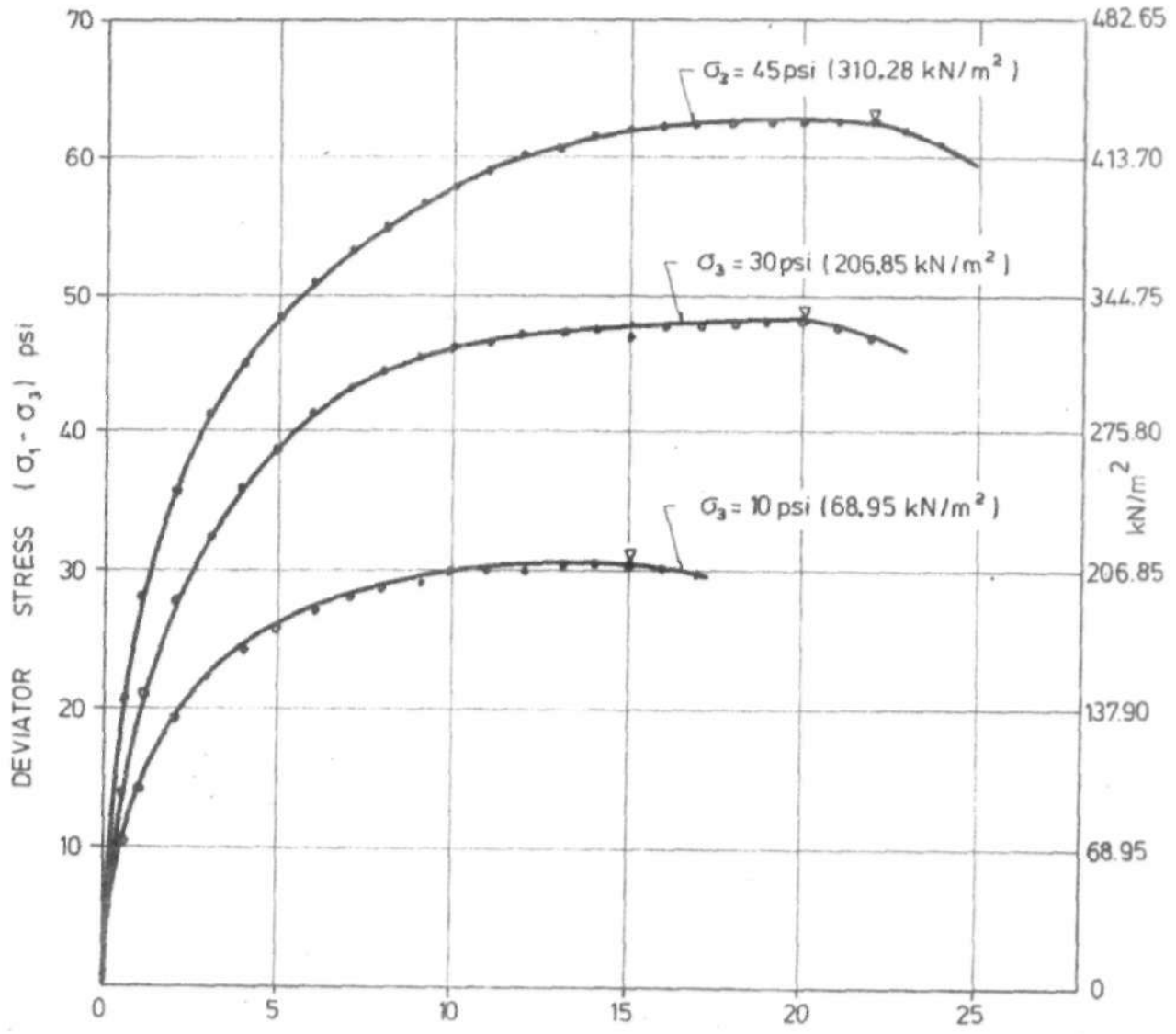


Figure 21 -- Deviator Stress Versus Axial Strain at 0.012 in/min. (0.4 %/min.) Rate of Strain - Test Series 1.

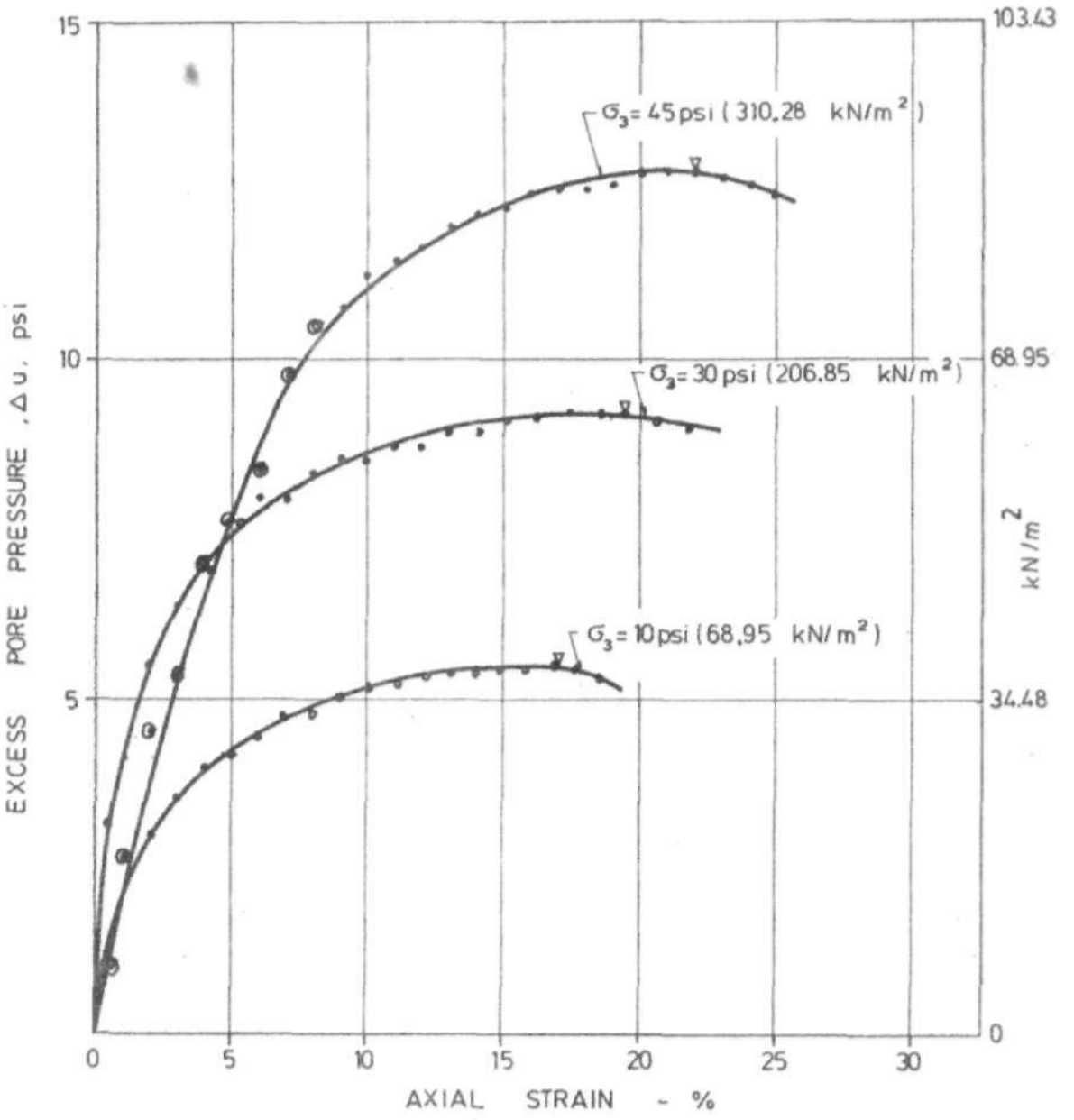


Figure 22 -- Excess Pore Pressure Versus Axial Strain at 0.012 in/min. (0.4 %/min.). Rate of Strain - Test Series 1.

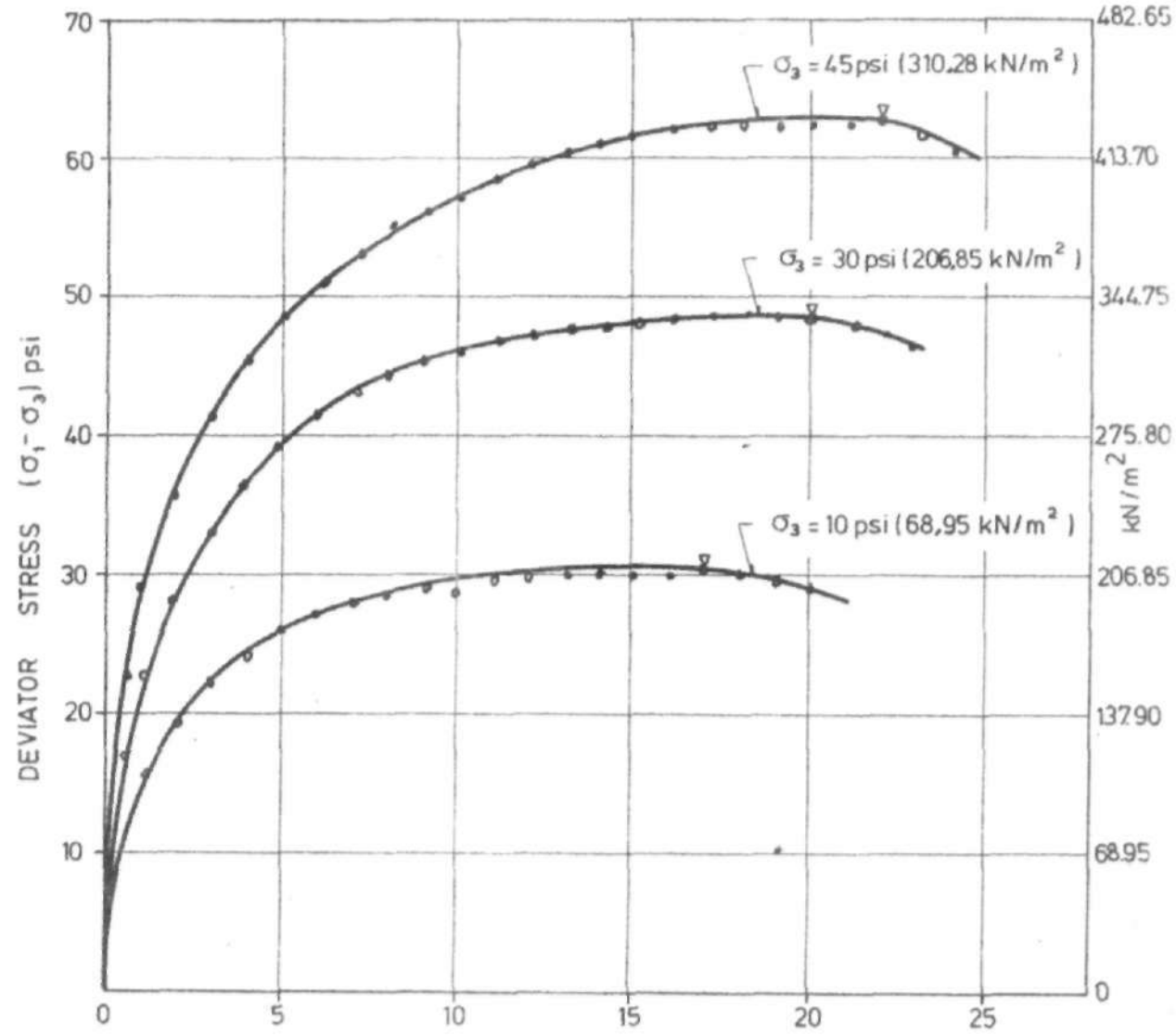


Figure 23 -- Deviator Stress Versus Axial Strain at 0.012 in/min. (0.4%/min.). Rate of Strain - Test Series 2.

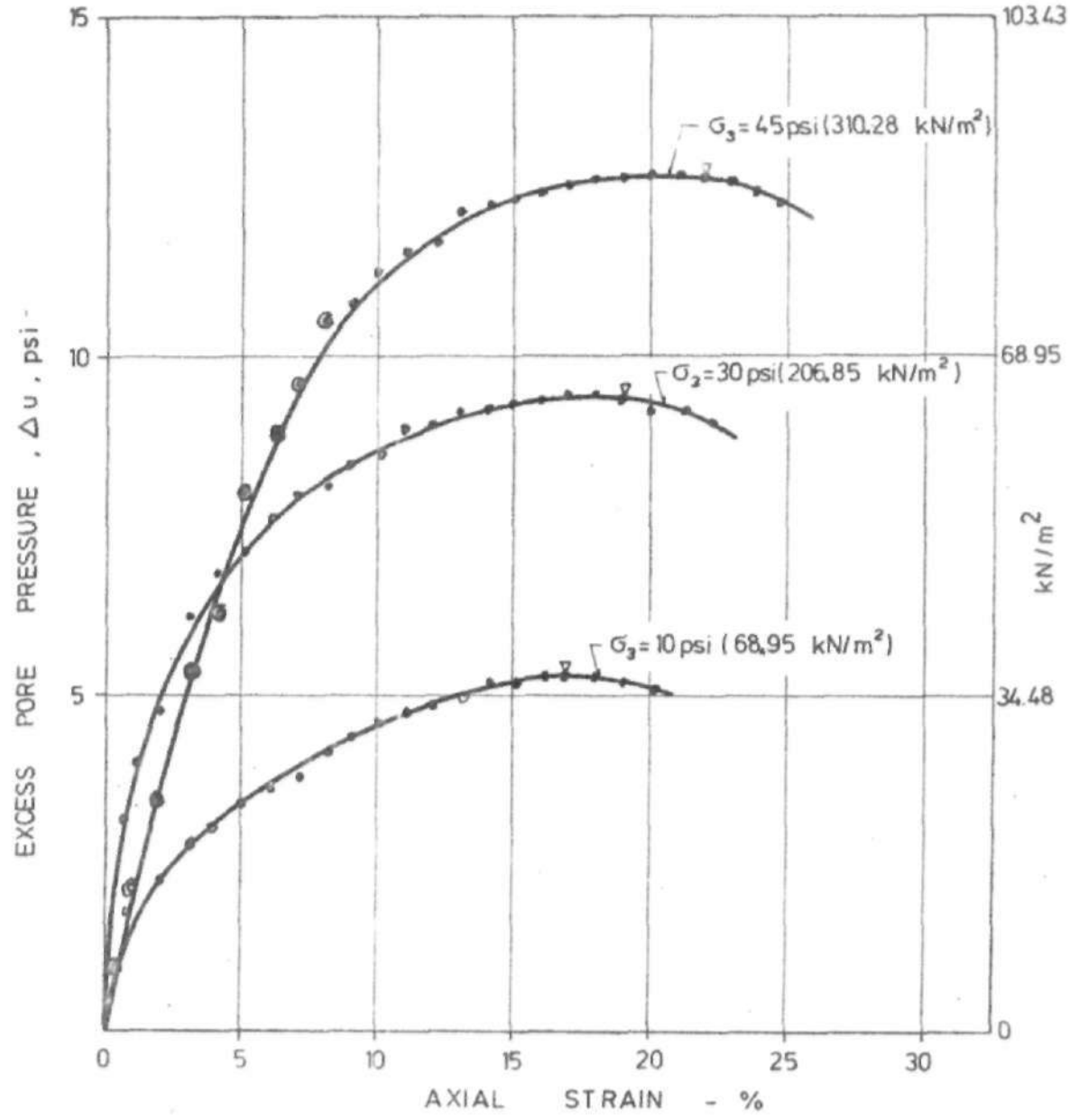


Figure 24 -- Excess Pore Pressure Versus Axial Strain at 0.012 in/min. (0.4%/min.). Rate of Strain - Test Series 2.

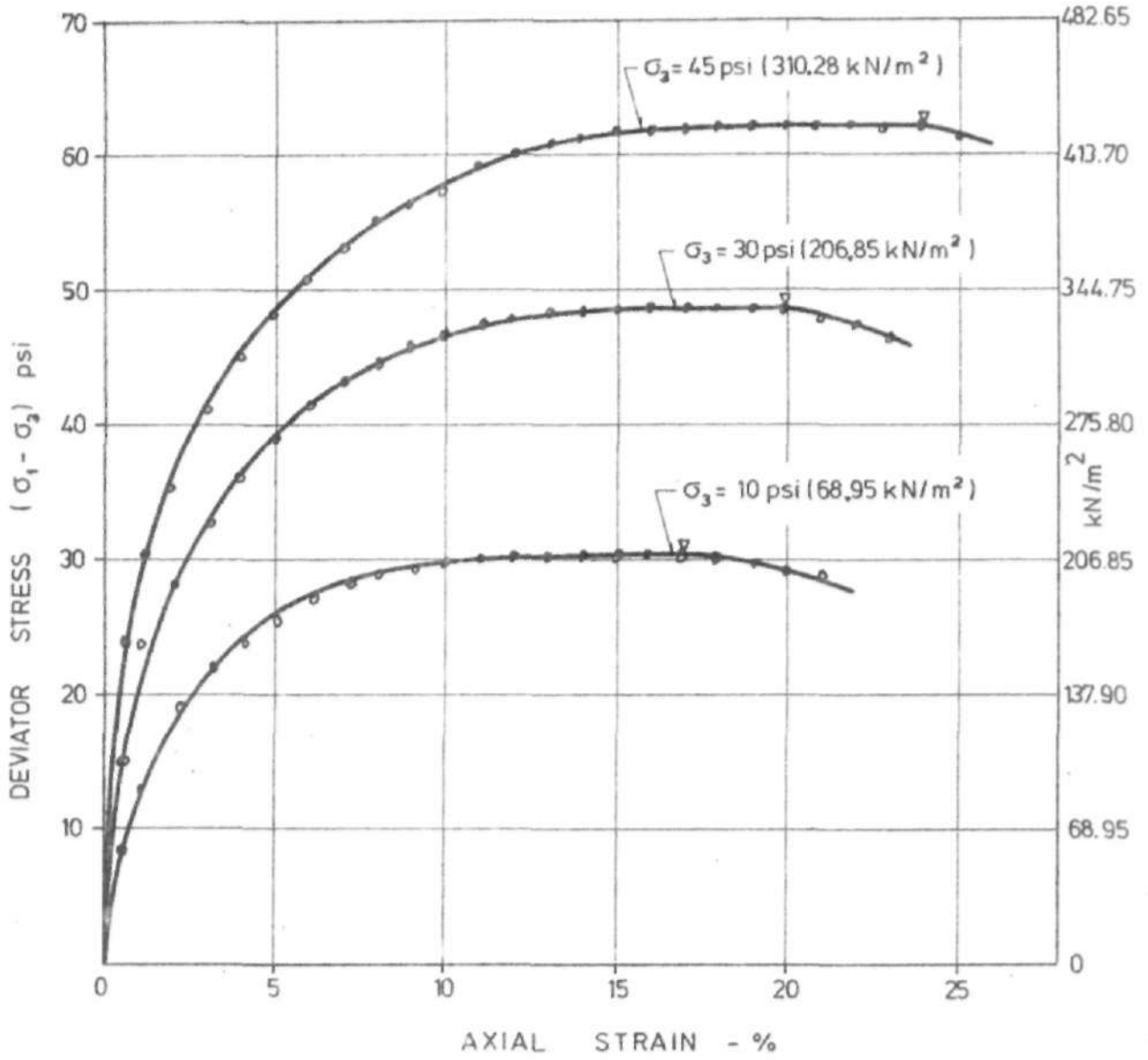


Figure 25 -- Deviator Stress Versus Axial Strain at 0.012 in/min. (0.4%/min.). Rate of Strain - Test Series 3.

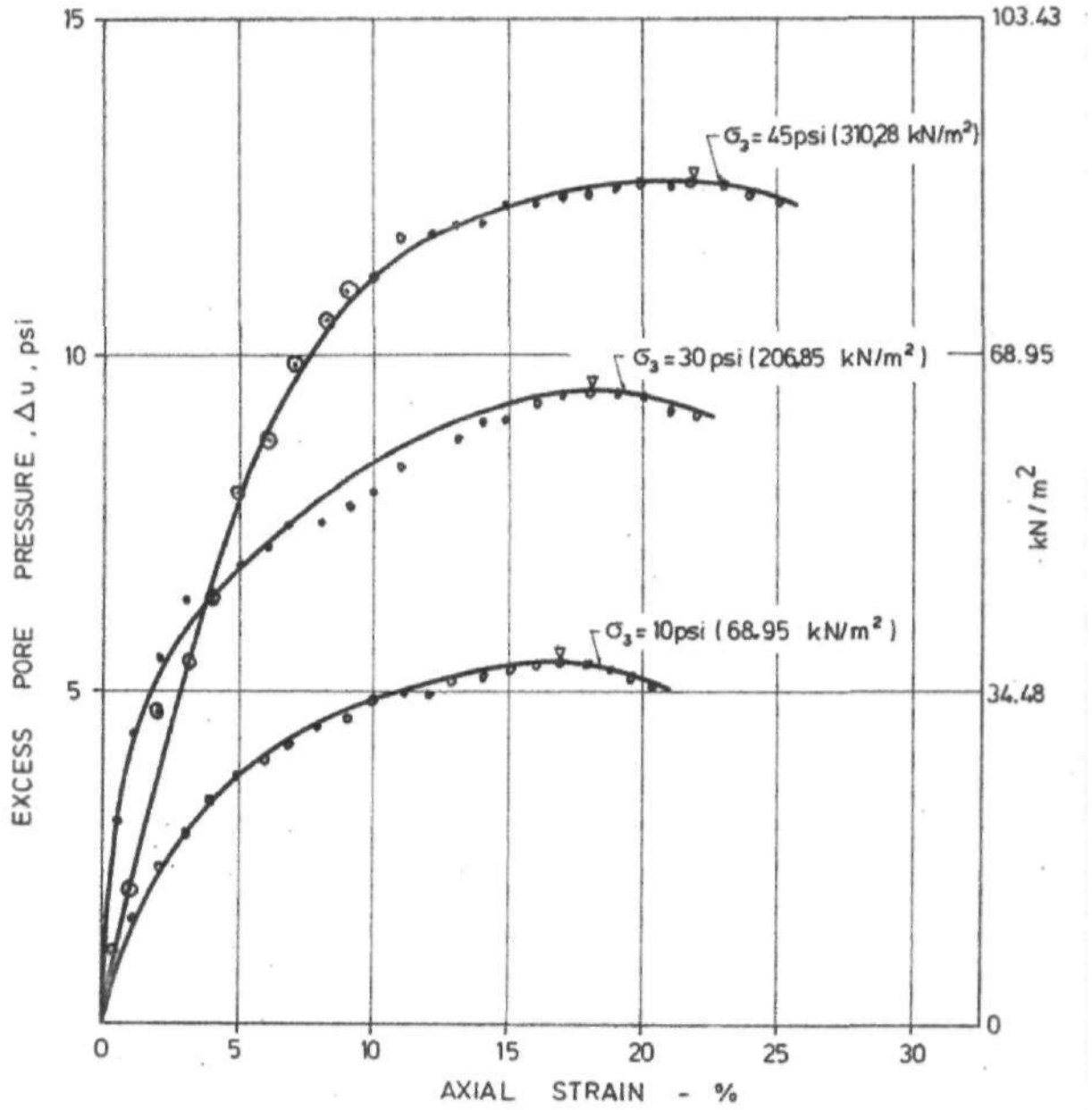


Figure 26 -- Excess Pore Pressure Versus Axial Strain at 0.012 in / min. (0.4%/min.). Rate of Strain - Test Series 3.

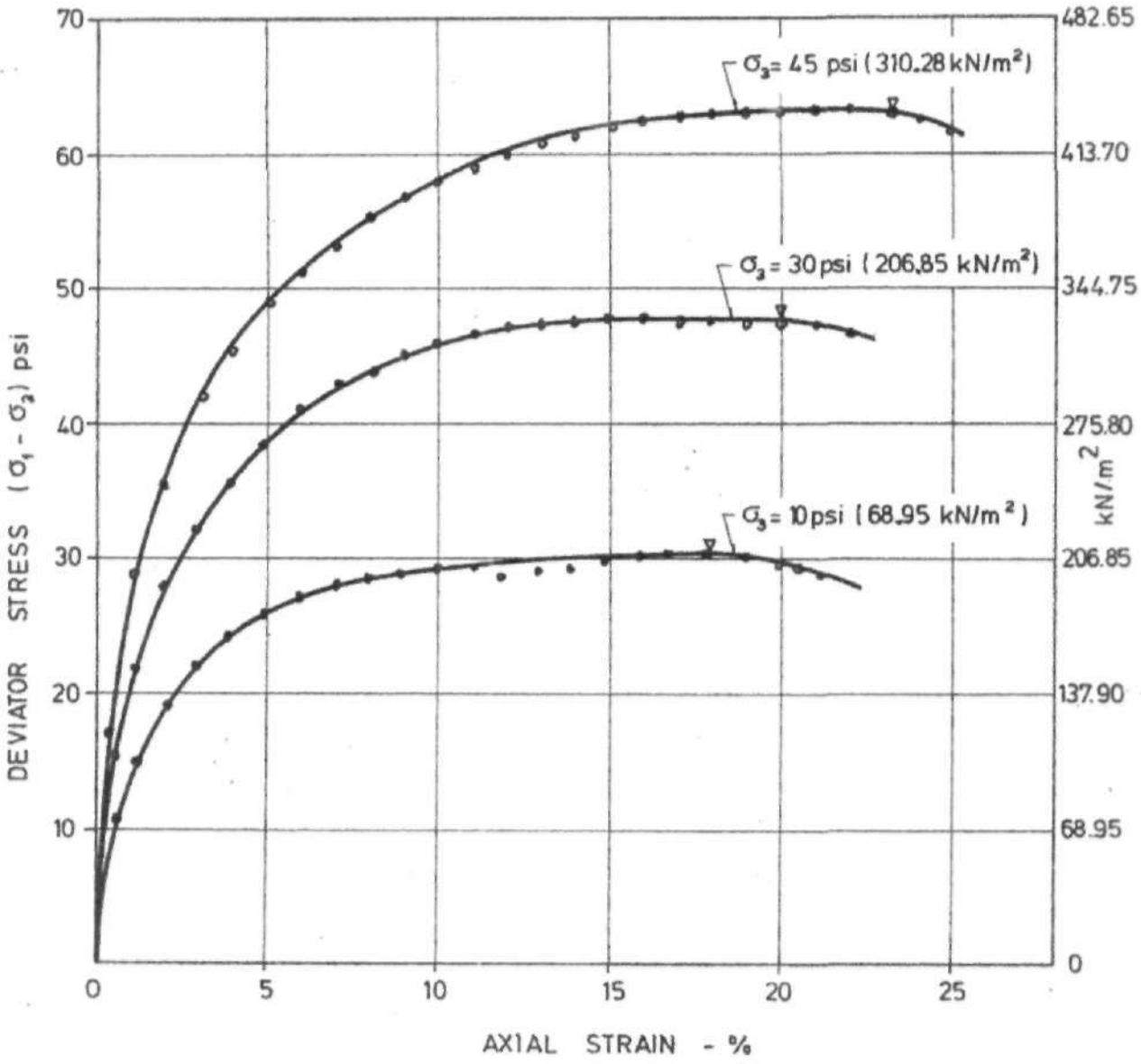


Figure 27 -- Deviator Stress Versus Axial Strain at 0.012in/min.(0.4%/min.). Rate of Strain - Test Series 4.

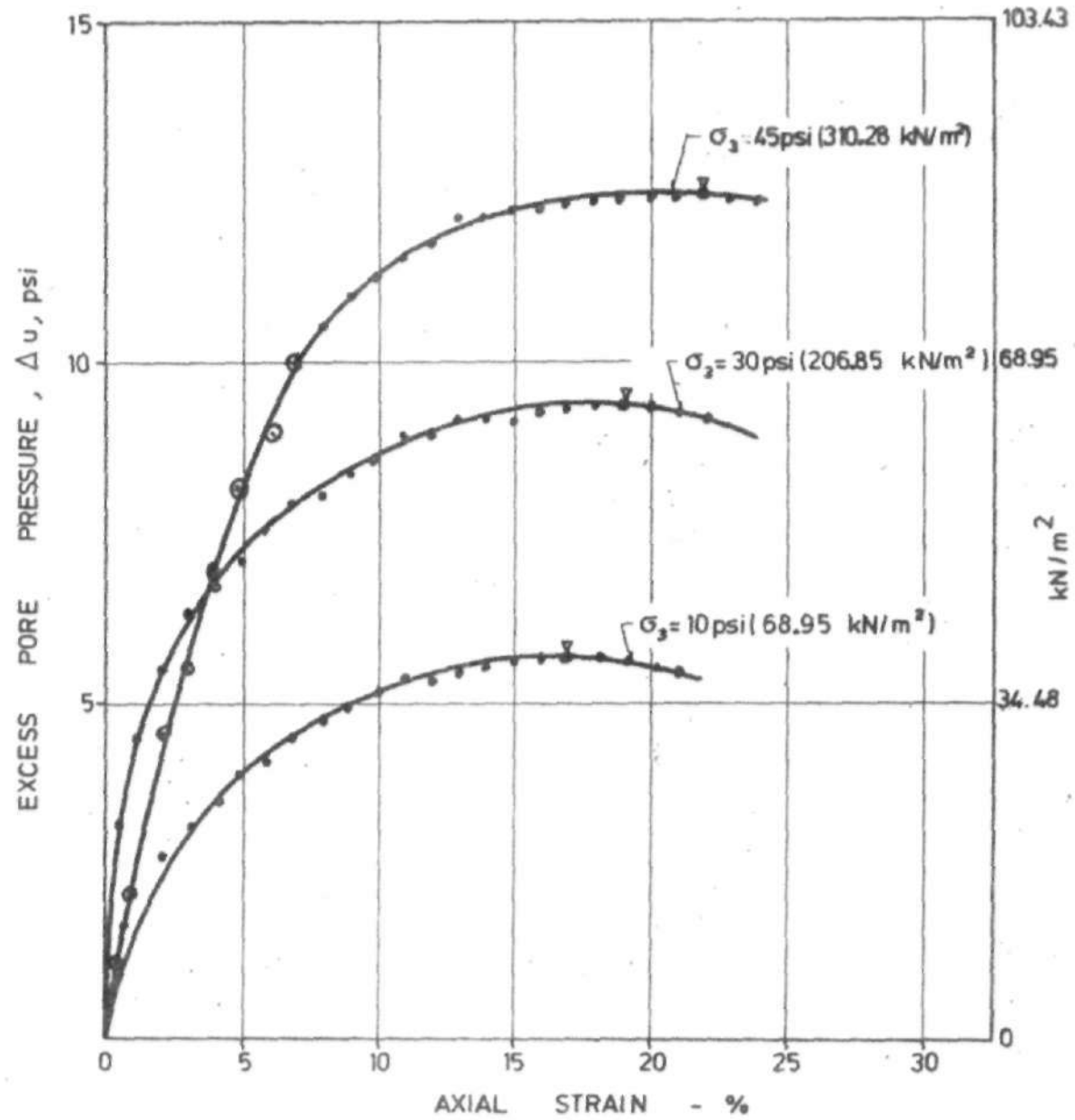


Figure 28 -- Excess Pore Pressure Versus Axial Strain at 0.012in/min (0.4%/min.). Rate of Strain - Test Series A

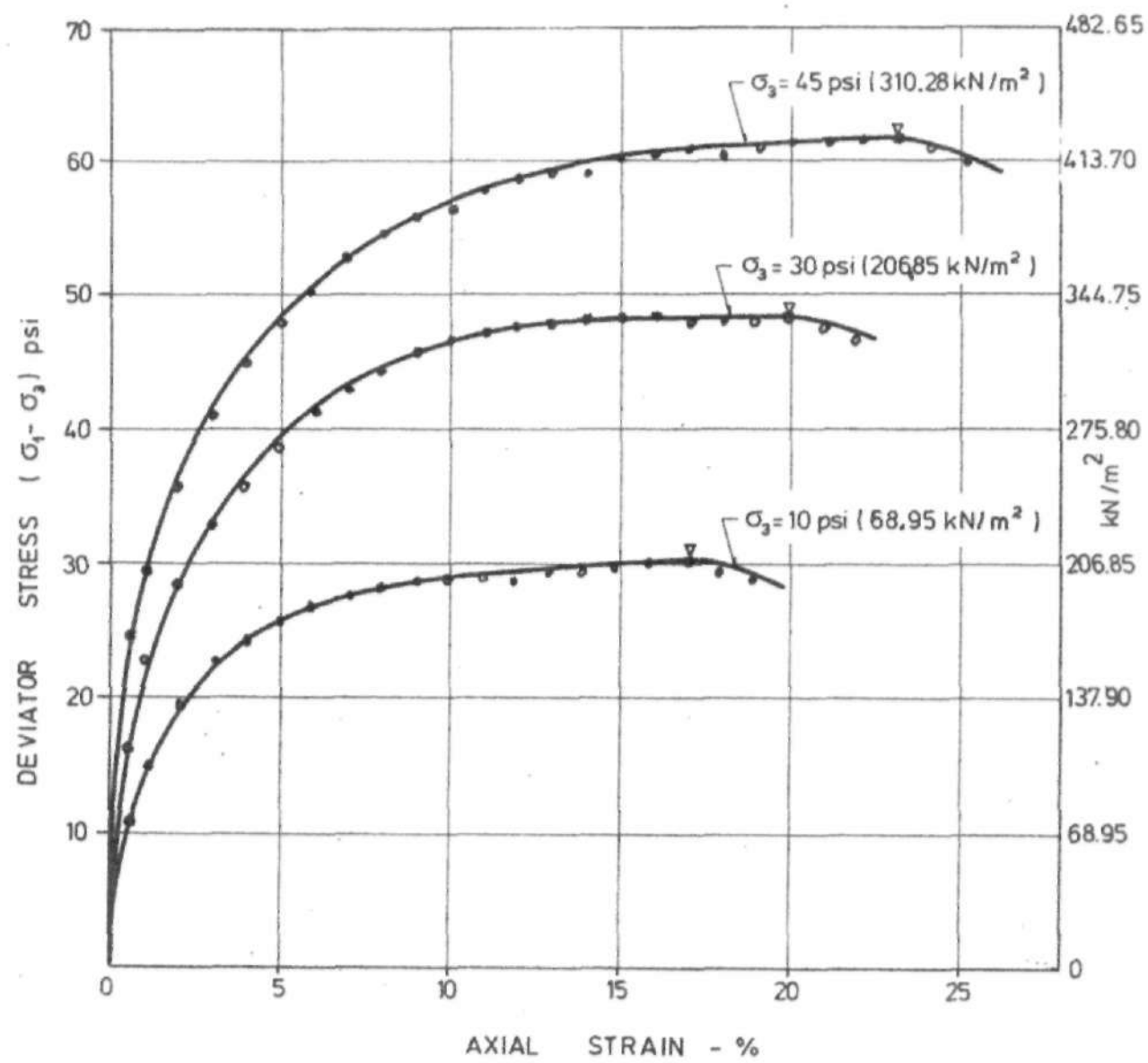


Figure 29 -- Deviator Stress Versus Axial Strain at 0.012 in/min. (0.4 % / min.). Rate of Strain - Test Series 5.

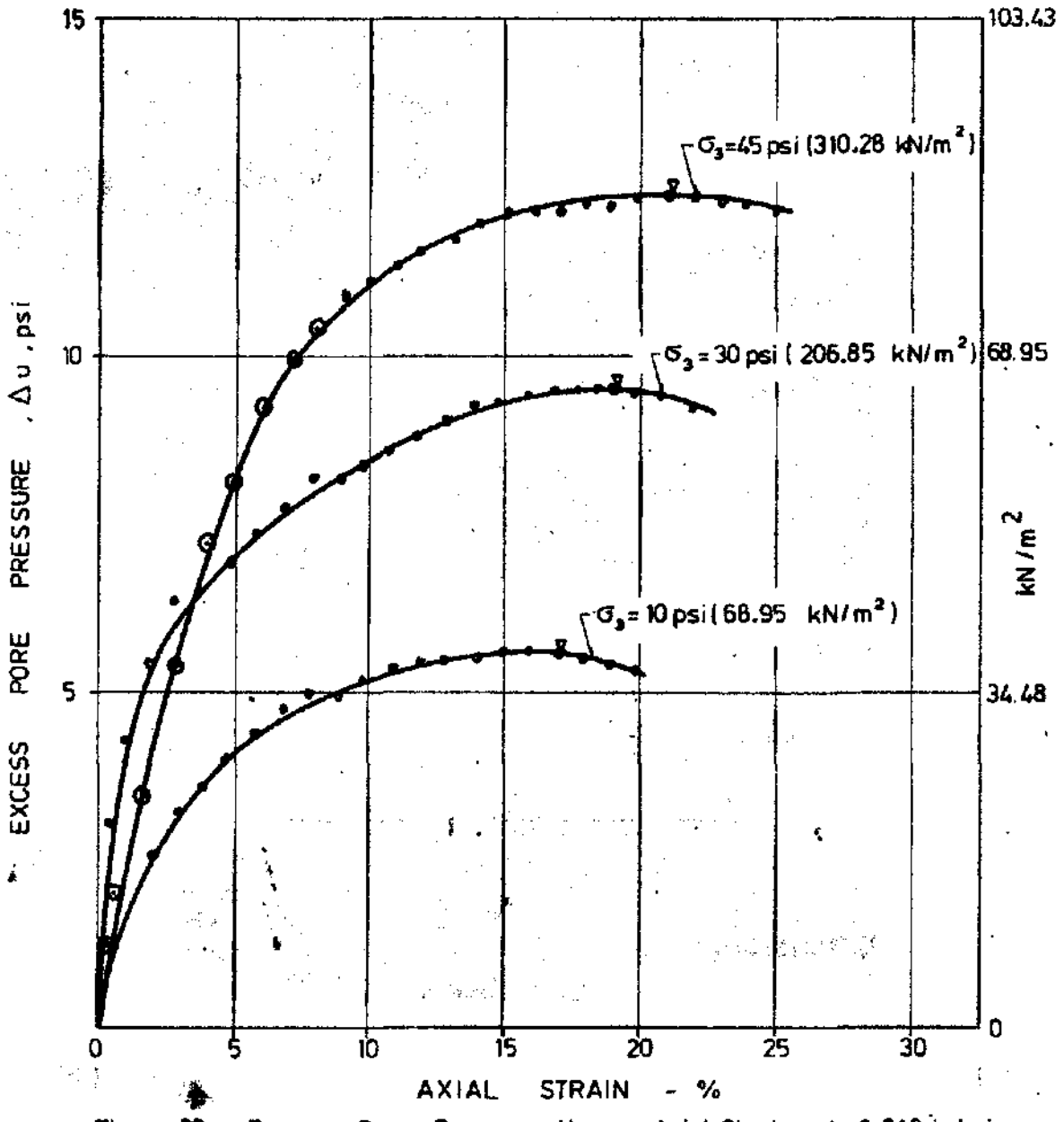


Figure 30 -- Excess Pore Pressure Versus Axial Strain at 0.012 in/min. (0.4%/min.). Rate of Strain - Test Series 5.

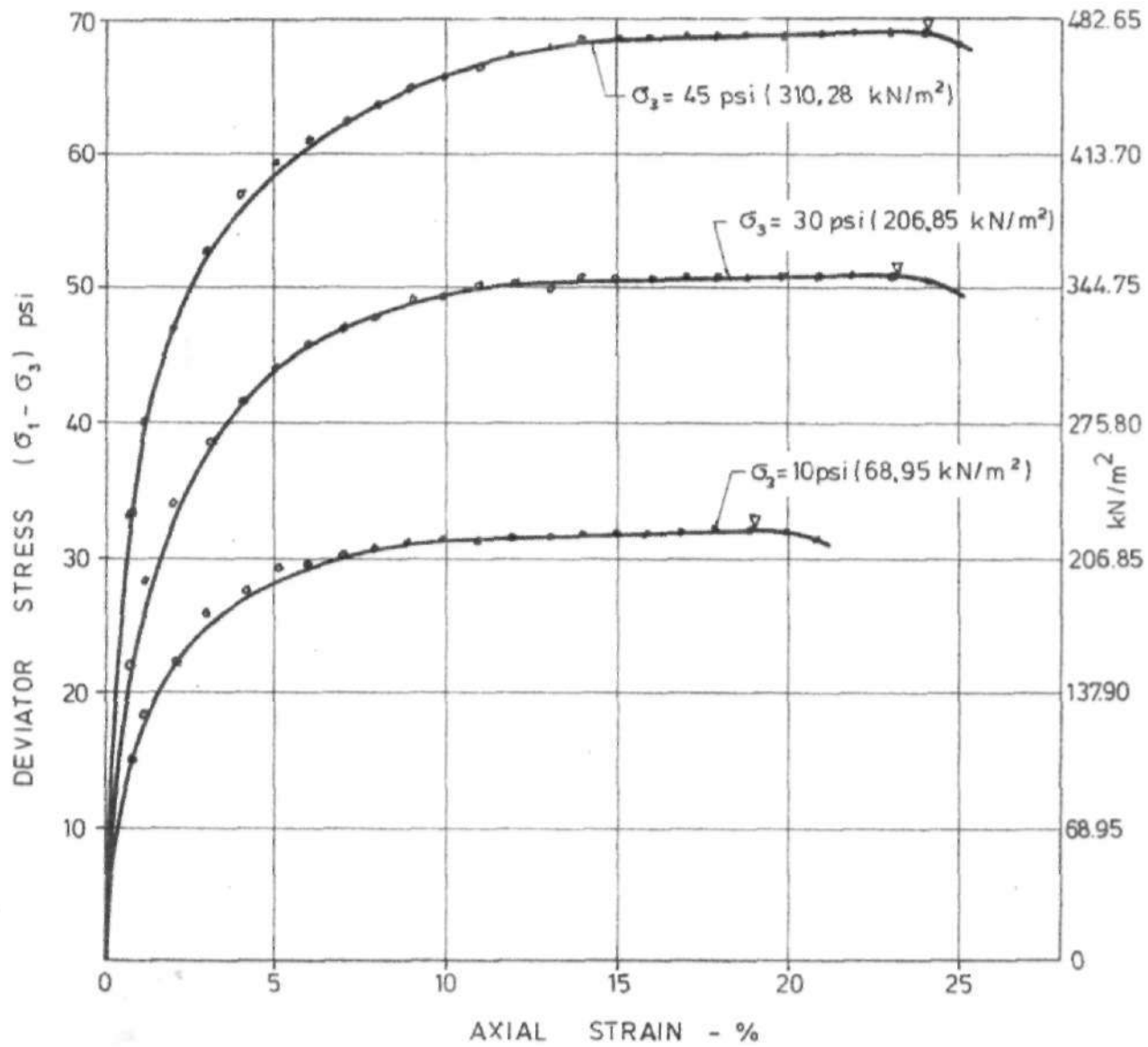


Figure 31 -- Deviator Stress Versus Axial Strain at 0.03 in/min. (1.0 %/ min.). Rate of Strain - Test Series 1.

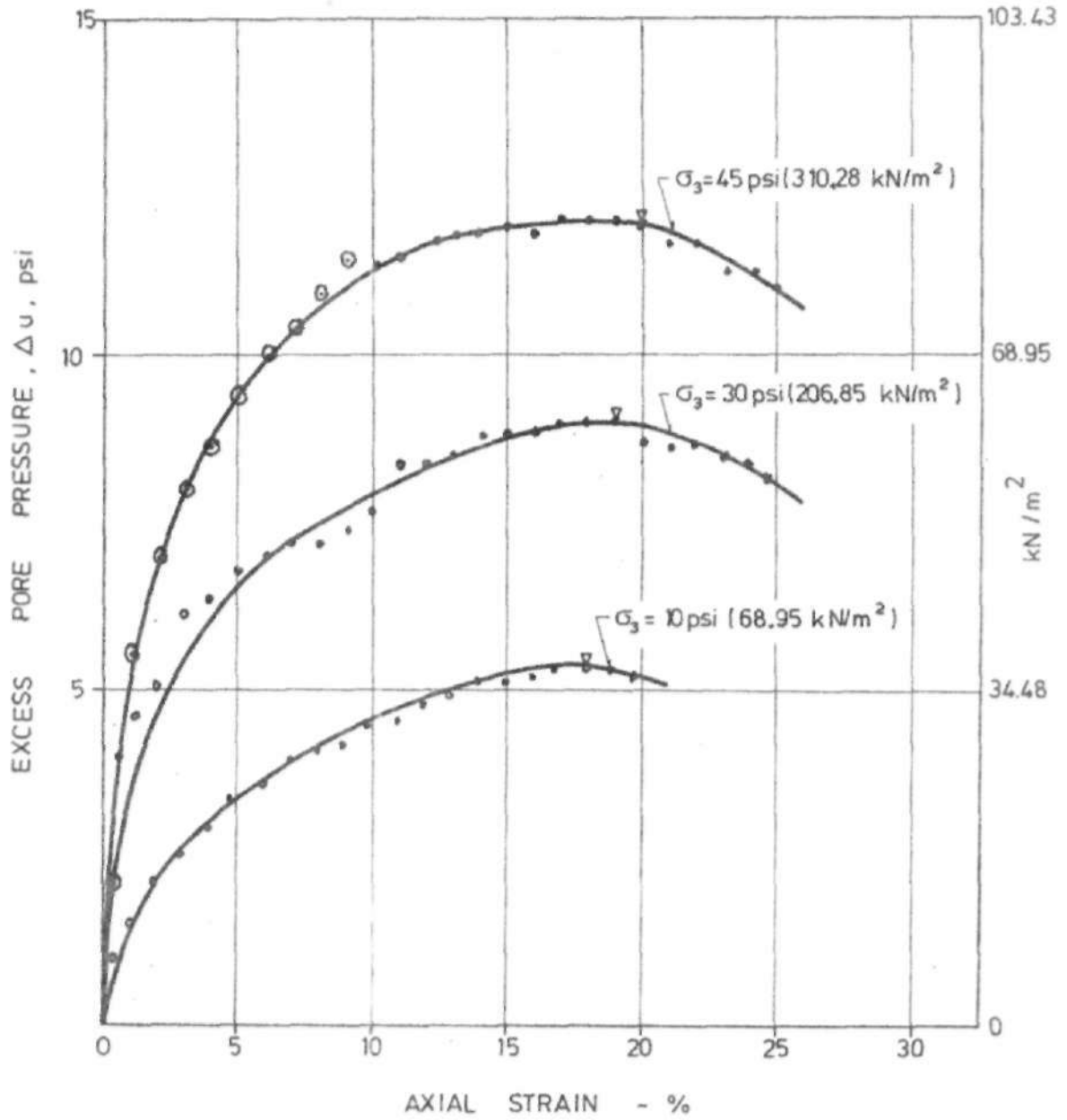


Figure 32 -- Excess Pore Pressure Versus Axial Strain at 0.03 in/ min. (1.0 %/ min.). Rate of Strain - Test Series 1.

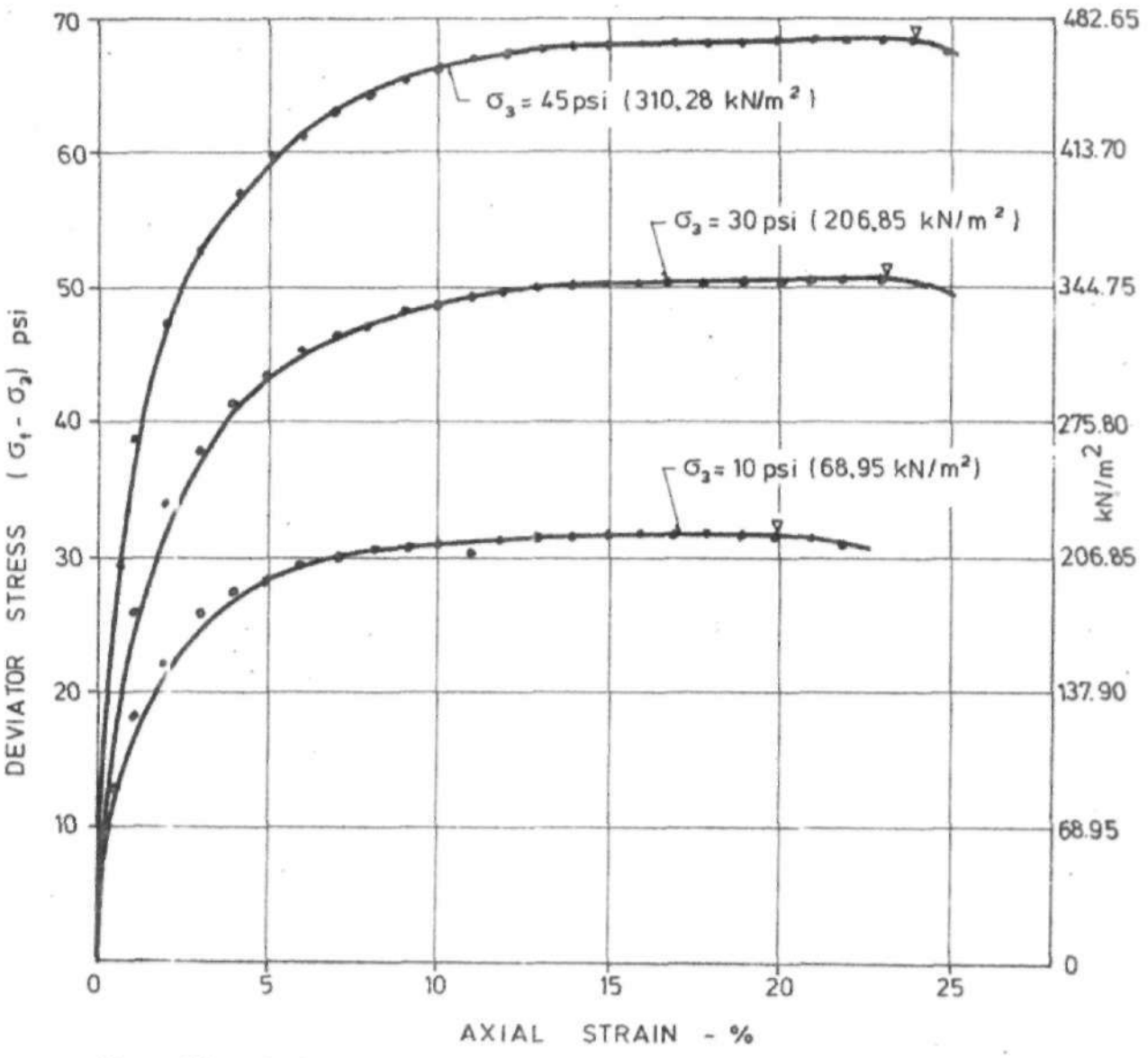


Figure 33 -- Deviator Stress Versus Axial Strain at 0.03 in/min. (1.0 %/min). Rate of Strain - Test Series 2.

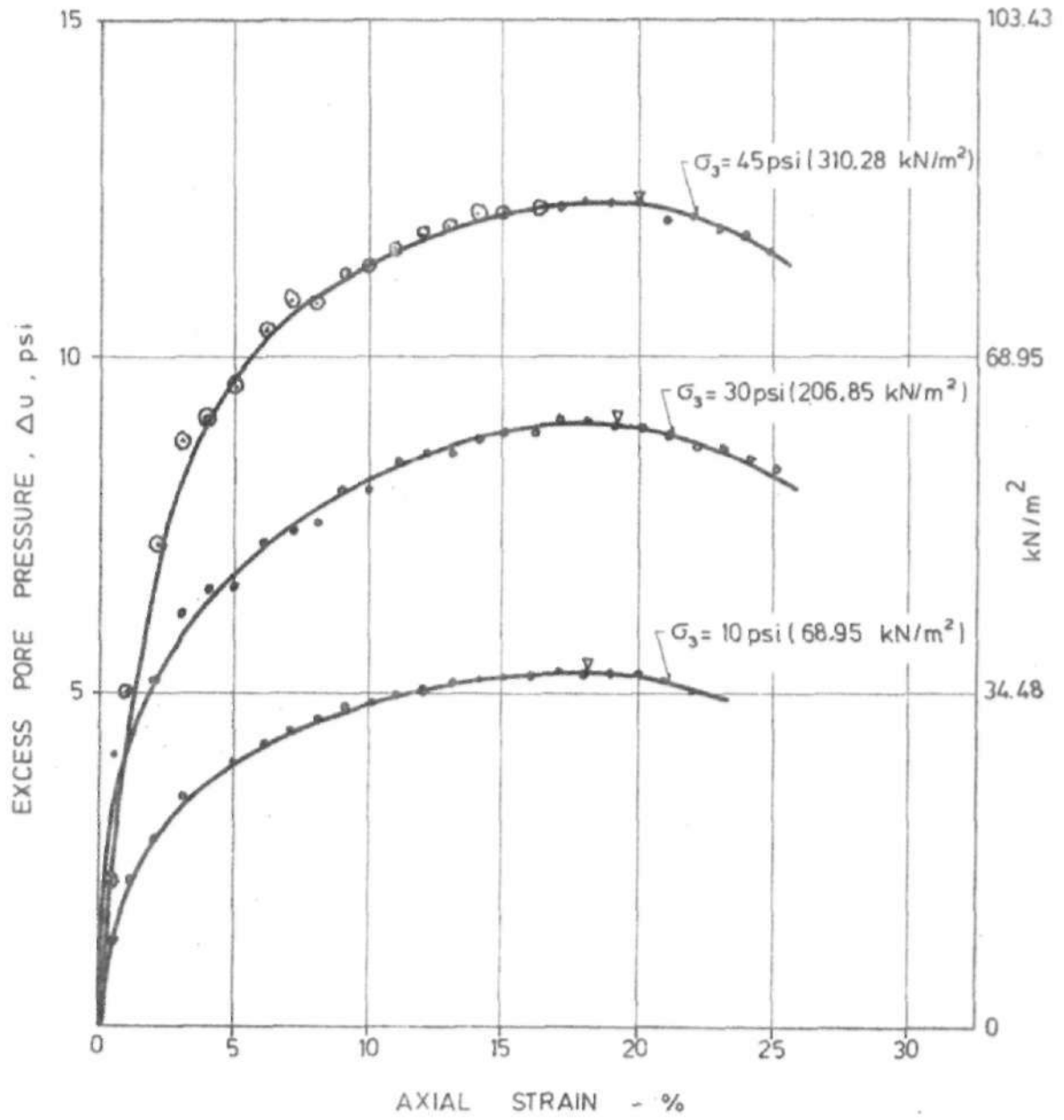


Figure 34 -- Excess Pore Pressure Versus Axial Strain at 0.03 in/min. (1.0 %/min.). Rate of Strain - Test Series 2.

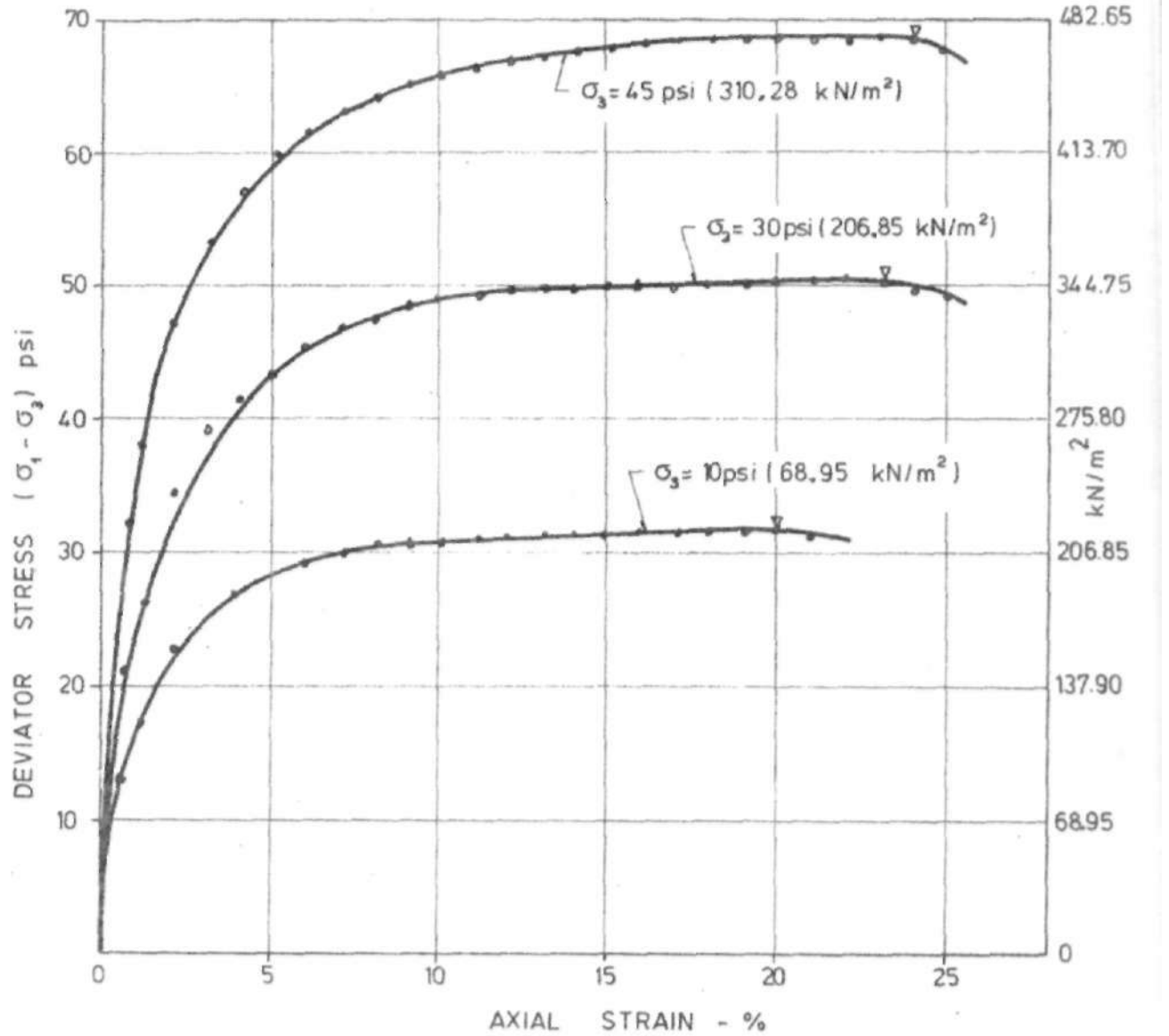


Figure 35 -- Deviator Stress Versus Axial Strain at 0.03in/min. (1.0 % / min.). Rate of Strain - Test Series 3.

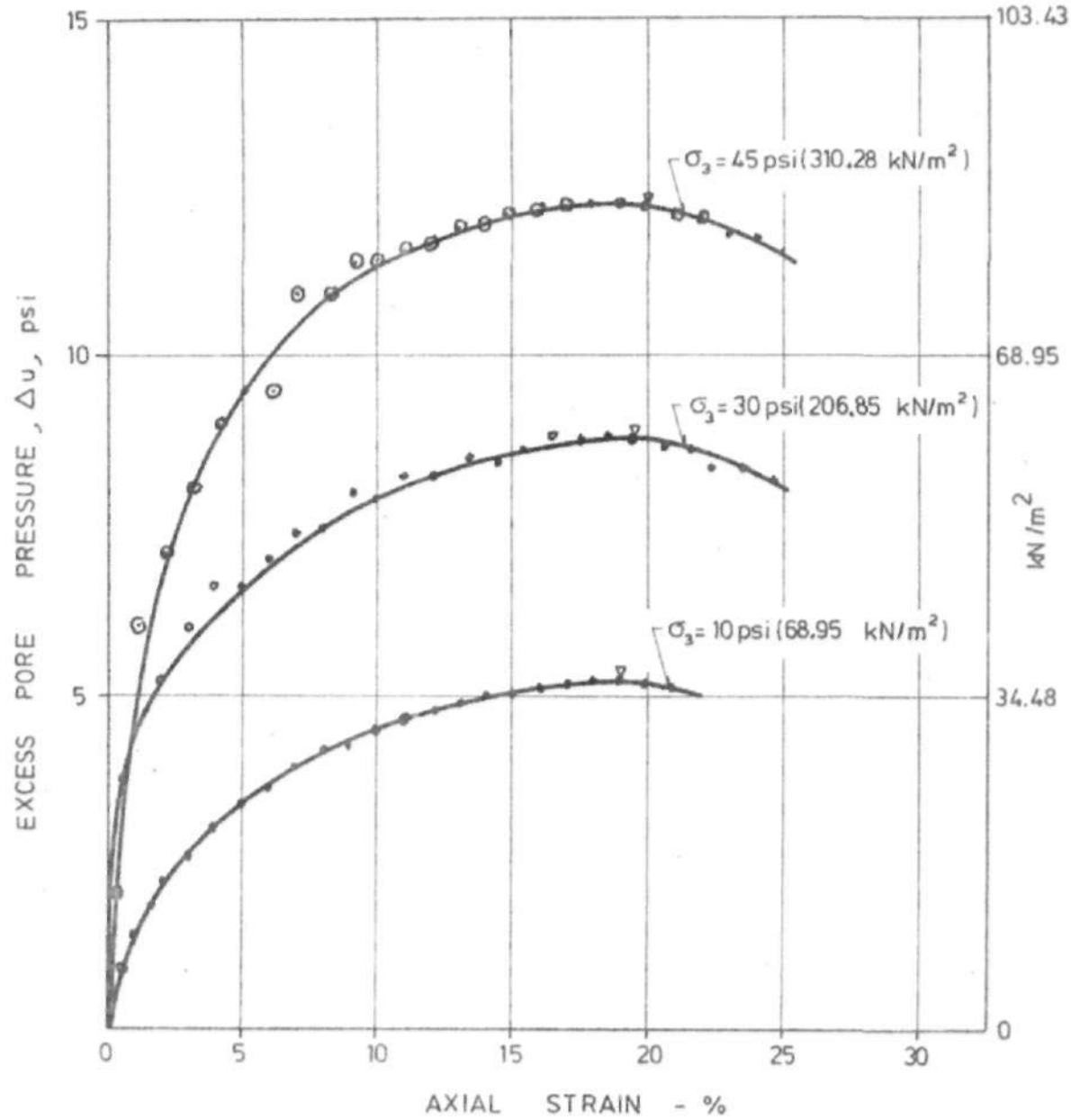


Figure 36 -- Excess Pore Pressure Versus Axial Strain at 0.03 in / min. (1.0 % / min.) Rate of Strain - Test Series 3.

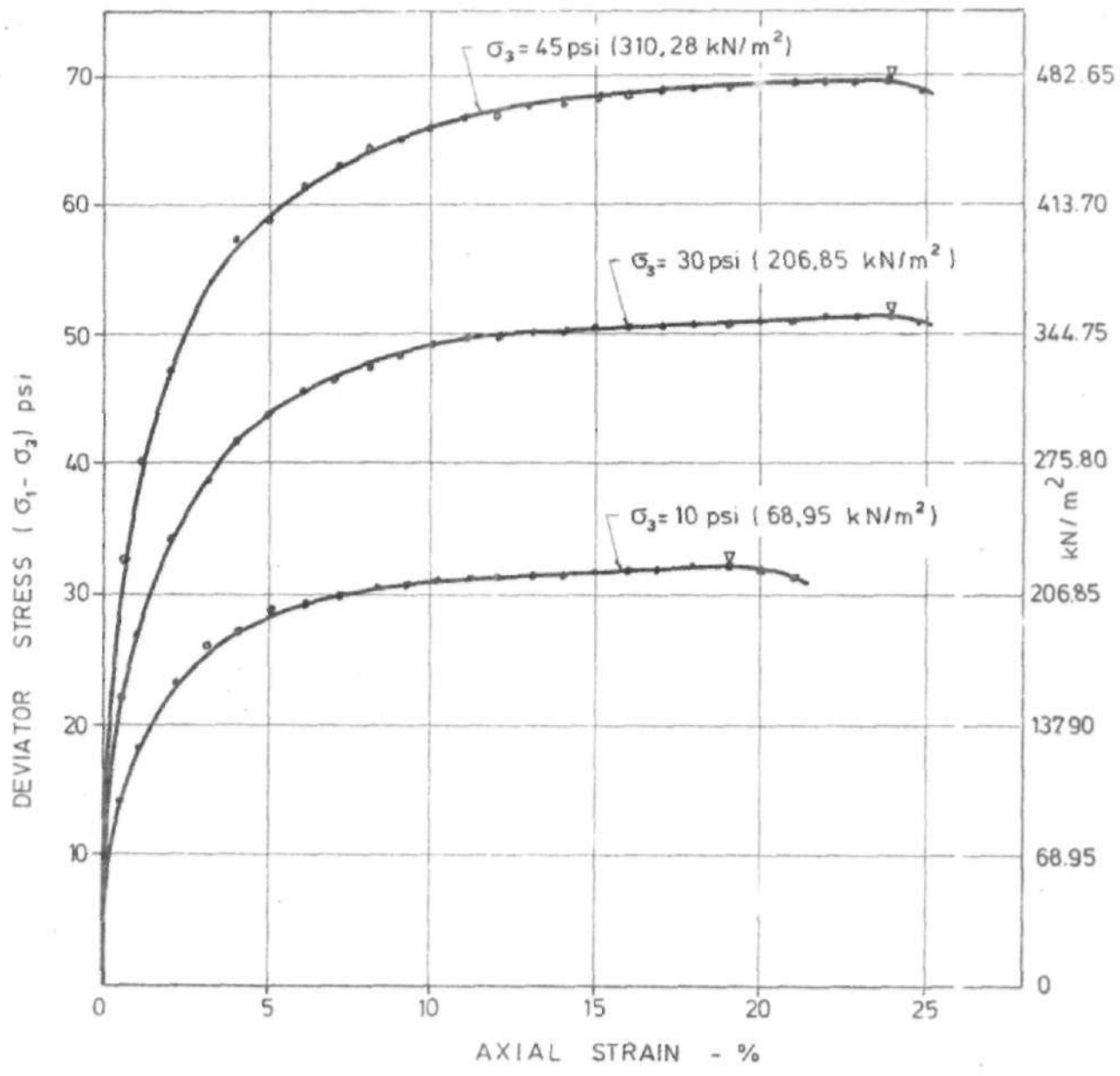


Figure 37 -- Deviator Stress Versus Axial Strain at 0.03 in/min. (1.0 %/min.). Rate of Strain - Test Series 4.

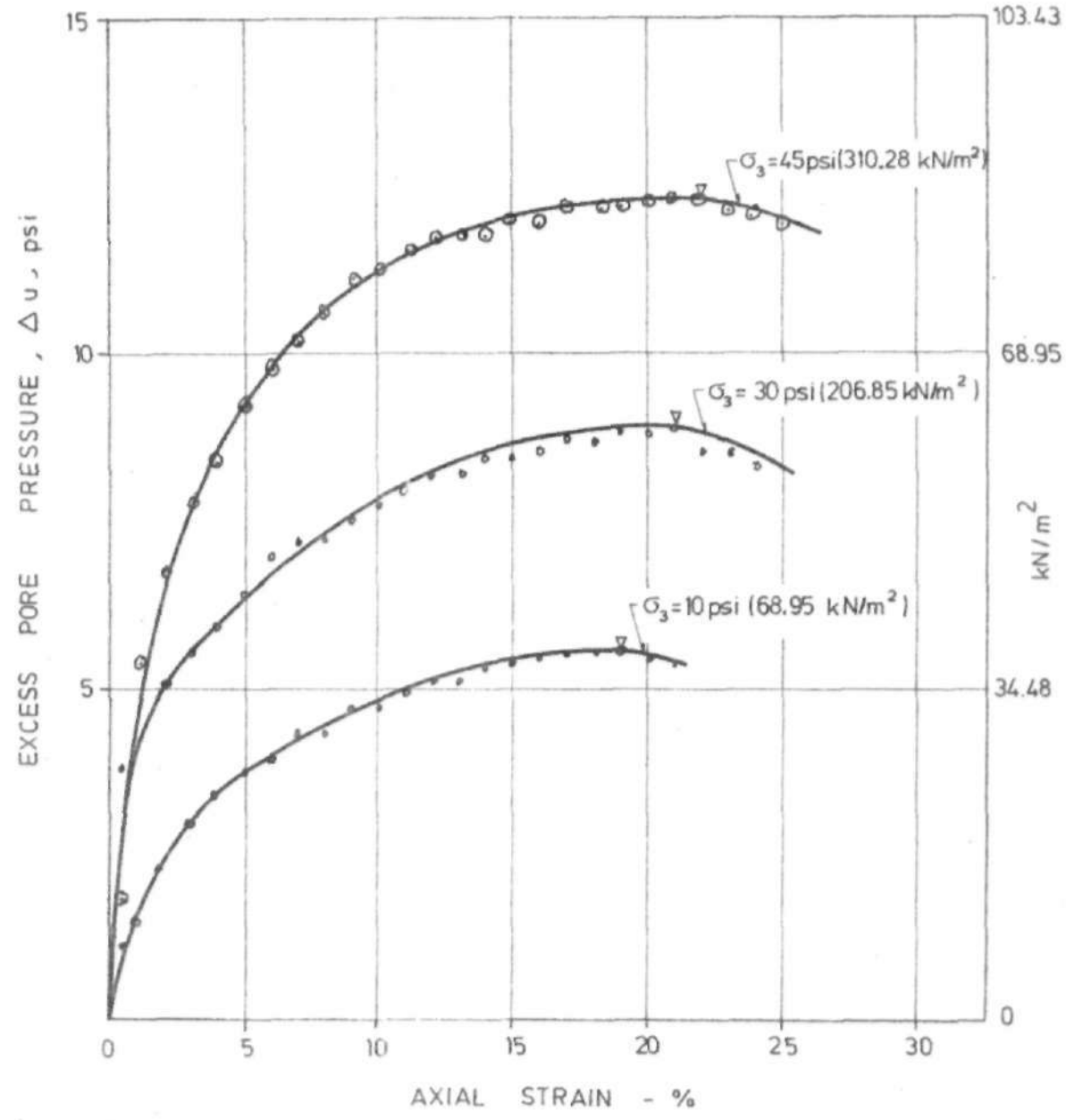


Figure 38 -- Excess Pore Pressure Versus Axial Strain at 0.03 in/min. (1.0 % / min.). Rate of Strain - Test Series 4.

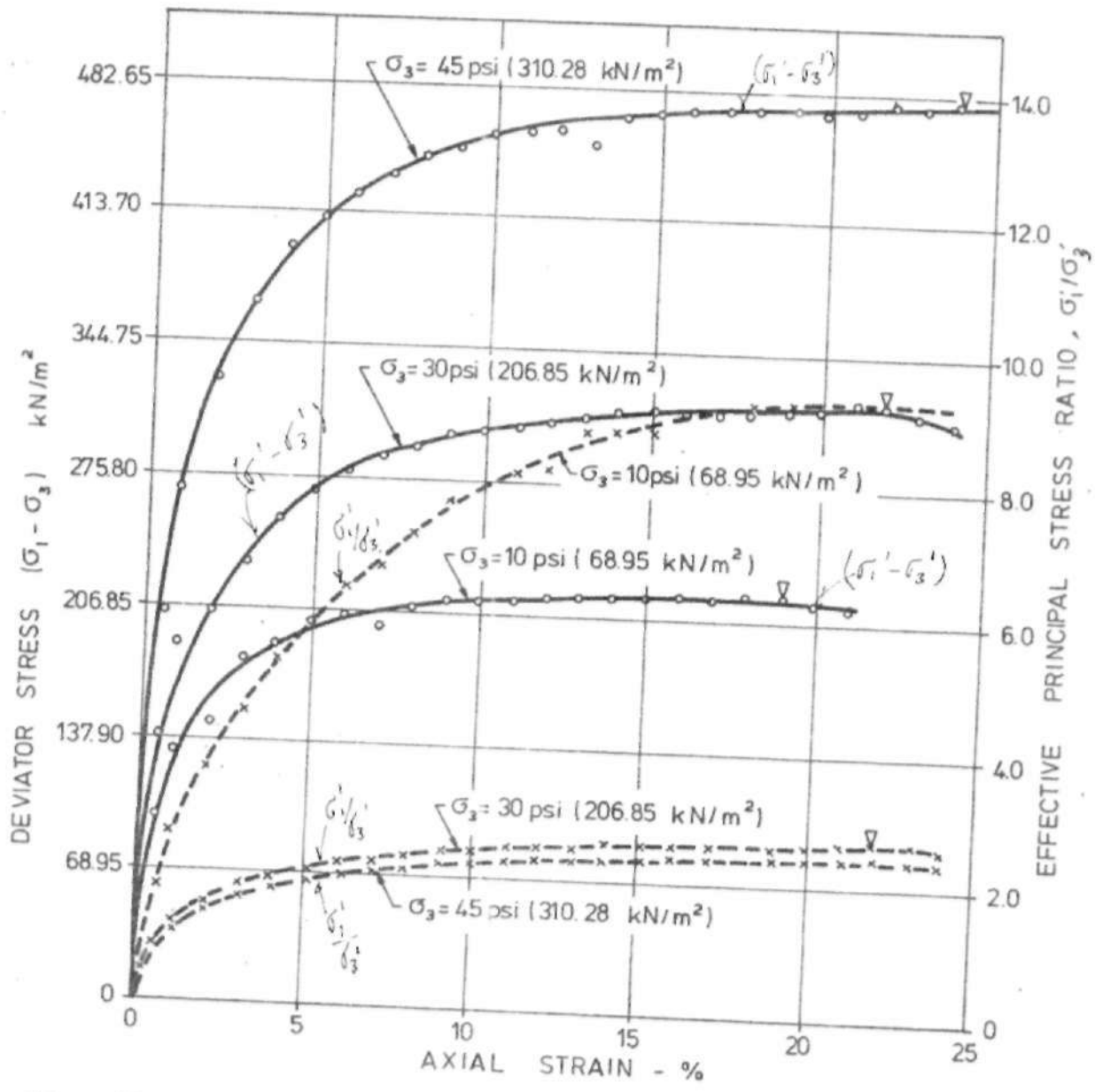


Figure 39 -- Deviator Stress and Effective Principal Stress Ratio Versus Axial Strain at 0.03 in / min (1.0 %/ min). Rate of Strain - Test Series 5.

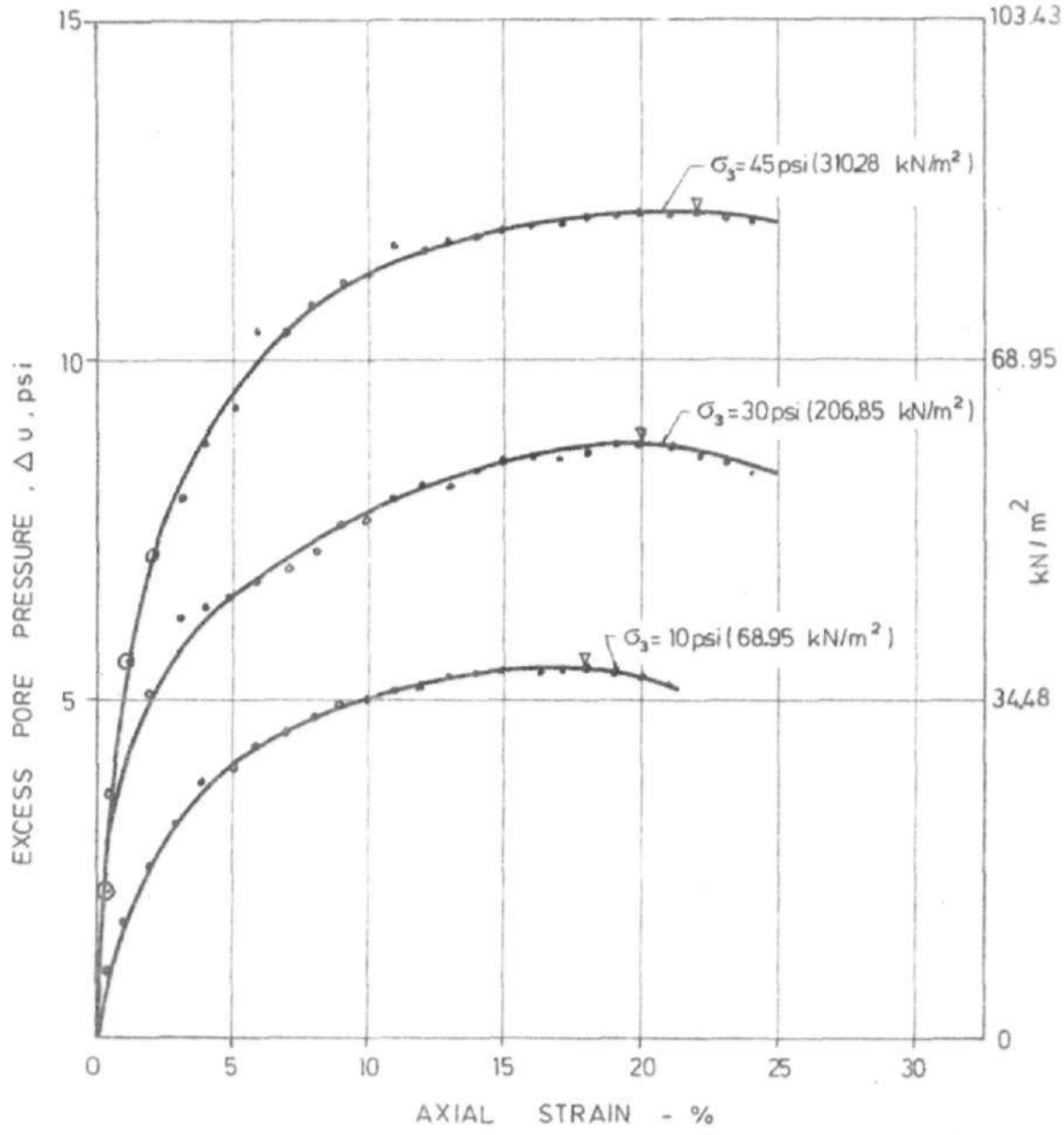


Figure 40 -- Excess Pore Pressure Versus Axial Strain at 0.03 in / min. (1.0 % / min.). Rate of Strain - Test Series 5.