

**MEASUREMENT OF SUCTION
AND VOLUME CHANGE BEHAVIOUR OF
UNSATURATED SOILS**

SRI WIDIASTUTI

SCHOOL OF CIVIL & ENVIRONMENTAL ENGINEERING

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ABSTRACT

Soil suction is the key factor to understanding the behaviour of unsaturated soils. Soil suction or total suction consists of matric suction and osmotic suction. Both components are important to unsaturated soil mechanics. The total, matric and osmotic suctions of a soil specimen can be measured independently in the laboratory using different techniques. However, it is rare to measure total, matric and osmotic suctions for a single soil specimen. Generally, the total or matric suction is measured and the osmotic suction is inferred from the difference between total and matric suction. The accuracies in the measurement of the different suction components are different. As a result, the sum of the measured matric and osmotic suctions may not be equivalent to the measured total suction. To investigate the discrepancy associated with the measurement of suction components, independent measurements of total, matric and osmotic suction on compacted kaolin and residual soils were performed. The results showed that the measured total suction can be greater or less than the sum of matric and osmotic suctions depending on the relative contribution of the matric and osmotic suctions to the soil suction. A plot was developed to provide an estimate of the absolute error in suction measurement.

Volume changes in unsaturated soils are important because of their consequences in terms of settlement due to wetting. The amount of volume change depends on the soil type and structure, the initial soil density, the imposed stress state, and the degree of wetting. The volume change behaviour of compacted kaolin and residual soils were investigated by conducting single and double oedometer tests. The compacted kaolin and residual soils were considered to have slight to moderate collapse behaviour at the vertical pressure of 200 kPa.

increase as the dry density increases. The apparent preconsolidation pressure was found to be higher when the soils were at dry condition than at wet condition. The compression indices of compacted kaolin, residual soils were found to decrease as dry density increases. The compression indices of the wet specimens were found to be higher than the dry specimens. These observations can be explained by the presence of suction in the dry specimens. A general empirical compression equation that account for soil type, soil structure and degree of saturation was derived based on the double-oedometer test results. It has three empirical constants: α' , β' and η . The empirical constants α' and β' , dependent on the soil type and soil structure, were obtained from the wet compression curve of the double-oedometer tests. The empirical constant η was found to be a linear function of activity based on the data from the three soils studied. Using the wet compression curve and the empirical compression equation, the wetting-induced settlement of compacted soils can be determined at different vertical pressures. The procedure developed in this study enables a simple estimation of settlement in compacted soil under rainfall condition so it can be accounted for in the engineering design.

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LIST OF SYMBOLS

| | |
|------------------------|--|
| α | Contact angle |
| α' | Empirical constant |
| β | Empirical constant |
| ε | Empirical constant |
| ε_v | Volumetric strain |
| η | Empirical constant dependent on soil type |
| ρ_w | Water density (i.e., 998 kg/m ³ at = 20°C) |
| ω_v | Molecular mass of water vapor (i.e., 18.016 kg/kmol) |
| σ | Total stress |
| σ'_v | Effective vertical stress |
| σ_{mean} | Average total normal stress $[(\sigma_x + \sigma_y + \sigma_z)/3]$ |
| μ | Poisson's ratio |
| ψ | Total suction |
| π | Osmotic suction |
| Δe_c | Change in void ratio upon wetting |
| ΔH | The Field heave |
| A | Activity |
| CP | Collapse potential |
| C_s | The swell index |
| E | Modulus of elasticity or Young's modulus for the soil structure |

| | |
|-------|---|
| EC | Electrical conductivity |
| e | void ratio |
| e_o | The initial void ratio |
| e_1 | Void ratio at the beginning of saturation |
| g | Gravitational constant = 9.81 m/s^2 |
| H' | Modulus of elasticity for the soil structure with respect to a change in matric suction |
| H | The thickness of Swelling |
| h_c | Capillary height |
| i_e | Collapse potential of single oedometer test |
| i_m | Collapse potential of double oedometer test |
| P_a | Atmospheric pressure (=101.325 kPa) |
| PI | Plasticity index |
| P_o | The overburden pressure |
| P_s | The swell pressure |
| R | Universal (molar) gas constant [i.e., $8.31432 \text{ J}/(\text{mol K})$] |
| r | Radius of the tube |
| S | Degree of saturation |
| T | Absolute temperature [i.e., $T = (273.16 + t^\circ) \text{ (K)}$] |
| T_s | Surface tension of water |
| t | Temperature |
| u_a | Pore air pressure |
| u_w | Pore water pressure |

| | |
|----------------|--|
| $u_a - u_w$ | Matric suction |
| \bar{u}_v | Partial pressure of pore-water vapour (kPa) |
| \bar{u}_{v0} | Saturation pressure of pore-water vapour over a flat surface at the same temperature |
| V_0 | Initial overall volume of an unsaturated soil element |
| V_v | Volume of soil voids |
| v_{w0} | Specific volume of water or the inverse of water density (i.e., $1/\rho_w$) |
| w | Water content |

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Unsaturated soils have become an important topic in modern soil mechanics. Soil suction is one of the important stress parameters in unsaturated soils. The measurement of soil suction is crucial for engineering applications in unsaturated soils particularly for understanding the engineering behaviour of unsaturated soils. The engineering behaviour of unsaturated soils can be described using a plot in the void ratio, net normal stress and suction (or e-p-s) space.

Soil suction or total suction consists of matric suction and osmotic suction. Total suction, matric suction and osmotic suction can be independently measured in the laboratory using different techniques (Leong et al. 2003a). There are very few documented studies (Richards and Ogata, 1961; Krahn and Fredlund, 1972; Leong et. al. 2003a) on the measurement of total suction together with independent measurements of matric and osmotic suctions and their comparison. Richards and Ogata (1961) found that the sum of matric and osmotic suctions was not the same as the measured total suction. Krahn and Fredlund (1972) found that the difference between total suction measured using a psychrometer and matric suction measured using the null-type axis-translation apparatus was similar to the osmotic suction measured using the squeezing technique. Their data showed good agreement for Regina clay but a poorer agreement was obtained for Till. Leong et al. (2003a) found that the measured total suction was consistently greater than the sum of independently measured matric and osmotic suctions. The discrepancy between total suction and sum of matric and osmotic suctions increases as total suction increases.

Some unsaturated soils are known to have expansive and/or collapsible behaviour on wetting. These behaviours are accompanied by large volume change and can cause extensive damage to engineering structures. Change in volume lead to change in strength and deformation properties that in turn influence stability. Changes in volume are caused by changes in applied stress, chemical and moisture, environments and temperature (Mitchell, 1976). Fundamentally, volume changes are mostly caused by stress changes.

This research investigates the accuracy of soil suction measurement and the volume change behaviour of unsaturated soil when wetted.

1.2 OBJECTIVES AND SCOPE OF RESEARCH

The main purpose of this research is to study the engineering behaviour of unsaturated soils with emphasis on the measurement of soil suction and the volume change behaviour of unsaturated soils when wetted.

The research is divided into two parts. The first part covers independent suction measurements (total, matric and osmotic suctions) in the laboratory and the measurement accuracy of each suction component. The second part deals with the volume change behaviour of compacted soils using the oedometer test as the soil is being wetted.

1.3 ORGANIZATION OF THESIS

This thesis is organized into five chapters:

Chapter 1 gives an overview of the background, objectives, and scope of the study.

Chapter 2 presents a literature review and is divided into four parts. The first part presents a brief review of unsaturated soil mechanics. The review of residual soil is given in the second part. The third part reviews soil suction measurement and the fourth part reviews the volume change theory of unsaturated soils.

Chapter 3 presents the methodology and the test procedures of this study. This chapter describes the soil materials used, preparation of soil samples, all the tests that were carried out, and the devices used for suction measurements and volume change study.

In chapter 4, the results and discussion of the suction measurements and volume change tests of the soil samples are discussed.

Chapter 5 summarizes the findings in the research and presents recommendations for future research works.

CHAPTER 2

LITERATURE REVIEW

The literature review is divided into four parts. The first part presents a brief review of unsaturated soil mechanics. The review of residual soil is given in the second part. The third part reviews soil suction measurement and the fourth part reviews the volume change theory of unsaturated soil.

2.1 UNSATURATED SOIL MECHANICS

The concept of unsaturated soil mechanics is relatively new in explaining and solving some of the uncertainties of geotechnical engineering problems. It has become of significant concern in recent years. An unsaturated soil consists of three phases, namely solids, water and air. However, it may be more correct to include the air-water interface as a fourth and independent phase (Fredlund and Morgenstern, 1977). The broad spectrum of unsaturated soil behaviour is linked to the wide range of degree of saturation, S . Figure 2.1 shows classification of the regions within a saturated-unsaturated soil profile. The soil profile above the groundwater table is unsaturated and is called the vadoze zone. The vadoze zone is divided into three regions. The first region, directly above the groundwater table with degree of saturation near to 100%, is called the capillary fringe. Here, water fills most of the voids and the air phase is discontinuous. The thickness of this region may range from less than one metre to approximately 10 m depending on the soil type (Fredlund, 2000). The second region, above the capillary fringe, is the two phase zone where the degree of saturation ranges from about 20% to 80%. The water phase and the air phase are continuous in this region. The third region which is near the ground surface is dry soil with the degree of saturation close to 0%. Here, the water phase is discontinuous and air fills most of the voids.

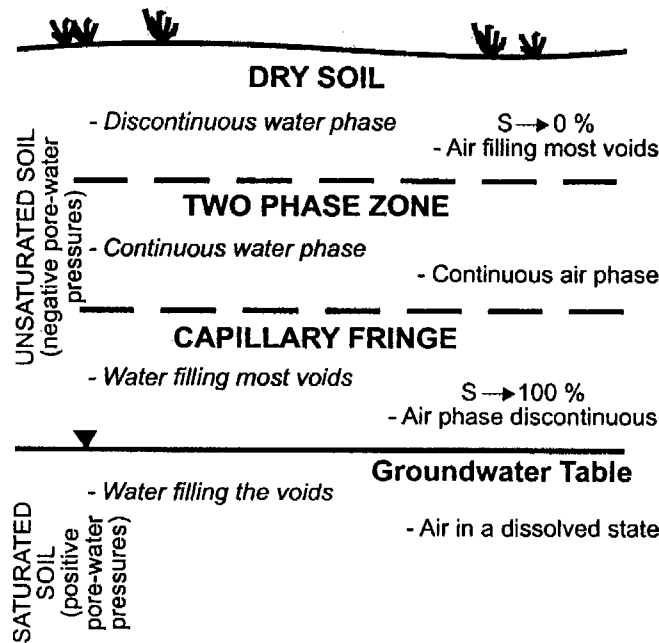


Figure 2.1 Classification of the regions within a saturated-unsaturated soil profile (from Fredlund, 2000)

Climate plays an important role in affecting the pore-water pressure in the soil, particularly in the vadose zone (Figure 2.2). Basically, there are two major fluxes of water flow across the ground surface due to climatic changes, i.e. upward flux of water out of the soil and downward flux of water into the soil. The upward flux is the result of extraction of water either by evaporation from the ground surface or by evapotranspiration from the vegetation. The downward flux is caused by rainfall and precipitation. During periods of high precipitation, soil saturates and the pores may be filled with water but during dry periods, soil desaturates and the pores may be almost completely filled with air. The difference between these two conditions will dictate the pore-water pressure conditions in the soil.

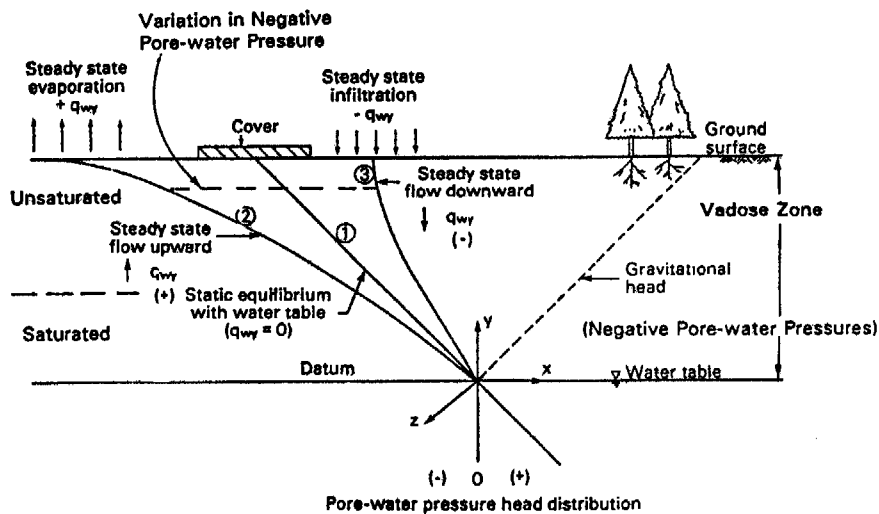


Figure 2.2 Illustration of possible negative pore-water profiles in the vadose zone
(from Fredlund and Rahardjo, 1993a)

The pore-water pressure profile will reach equilibrium condition at a hydrostatic condition where there is no flux across the ground surface (denoted as line 1 in Figure 2.2). The pore-water pressure will be negative above the ground water table and positive below it. When evaporation takes place at the ground surface, the negative pore-water pressure decreases further. Therefore, the pore-water pressure profile will be drawn to the left of line 1 (denoted as line 2 in Figure 2.2). The opposite condition i.e. when water enters at the ground surface such as infiltration, the pore-water pressure will be drawn to the right side of line 1 (line 3 in Figure 2.2). If the intensity of the rainfall is high enough to saturate the ground surface, the line might shift to a zero or positive value of pore-water pressure which is shown as flooding of desiccated soil (Figure 2.3). As a result, changes will occur in the volume and shear strength of the soil. Many soils exhibit swelling or expansion upon wetting.

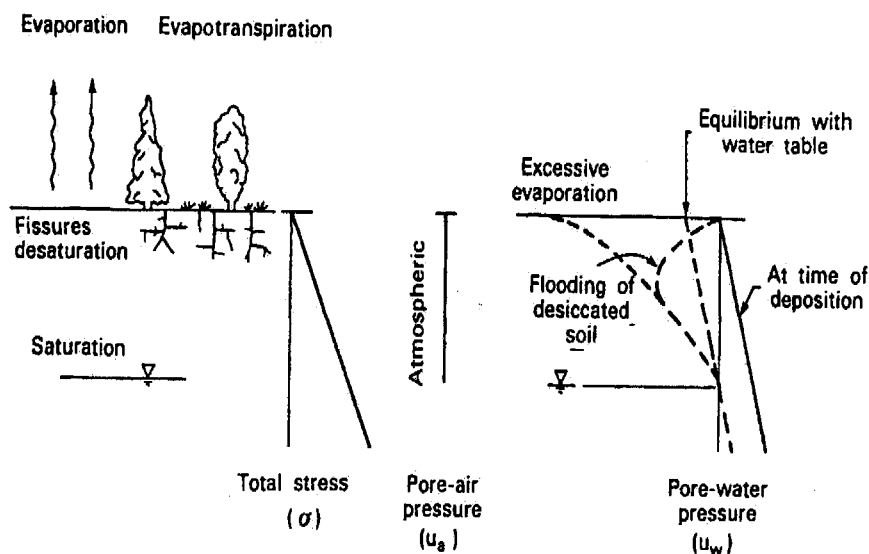


Figure 2.3 Stress distributions during desiccation of a soil
(from Fredlund and Rahardjo, 1993a)

2.2 RESIDUAL SOIL

There is no universal accepted definition of residual soil but a reasonably general definition would be a soil-like material derived from the in situ weathering and decomposition of rock which has not been transported from its original location (Blight, 1997). Residual soils are the product of the in situ weathering of the underlying rock and their occurrence is widespread. Residual soils are frequently involved in engineering construction and there is considerable experience of their behaviour. Basically, there are three types of weathering that cause the parent rock to become a residual soil namely: physical weathering, chemical weathering and biological weathering. Being formed in situ the residual soils have particular characteristics which distinguish them from material deposited from a fluid medium such as wind and water. Commonly, residual soils are product of the in situ weathering of igneous and metamorphic rocks while residual soils formed from sedimentary are rare. In the topics where there is

abundant rainfall and high temperature, the rate of weathering generally decreases with increase in depth. With increasing depth, there is an absence of organic matter and a reduction of seasonal moisture content fluctuations. Typically, a profile of residual soil will consist of three indistinctly divided zones (Vargas and Pitchler, 1957; Ruxton and Berry, 1957; Little, 1969) as shown in Figure 2.4. The upper zone consists of highly weathered and leached soil often reworked by burrowing animals and insects or by cultivation and intersected by root channels. The intermediate zone also consists of highly weathered material but exhibits some features of the structure of the parent rock and may contain core stones. In tropical regions, residual soil plays a very significant role in geotechnical engineering practice. The processes forming residual soils include incorporation of humus (decaying vegetation), physical and chemical weathering, leaching of insoluble materials, accumulation of insoluble residues, downward movement of fine particles and disturbance by root penetration, animal burrowing, free fall and desiccation (Working Party Group, 1995).

There are two main types of residual soil in Singapore, namely: residual soil derived from the Bukit Timah Granite and the sedimentary Jurong Formation, each covering about one-third of Singapore's land area. Figure 2.5 shows the geological map of Singapore and the distribution of the four major geological formations in Singapore (Bukit Timah Granite, Kallang Formation, Jurong Formation and Old Alluvium). The Bukit Timah Granite and its residual soil occupy the central part of Singapore whereas the sedimentary Jurong Formation and its residual soil occupy most of the western and southwestern parts of Singapore Island. Singapore residual soils are generally heterogeneous, highly textured in nature and relatively stiff, particularly in comparison to sedimentary soils.

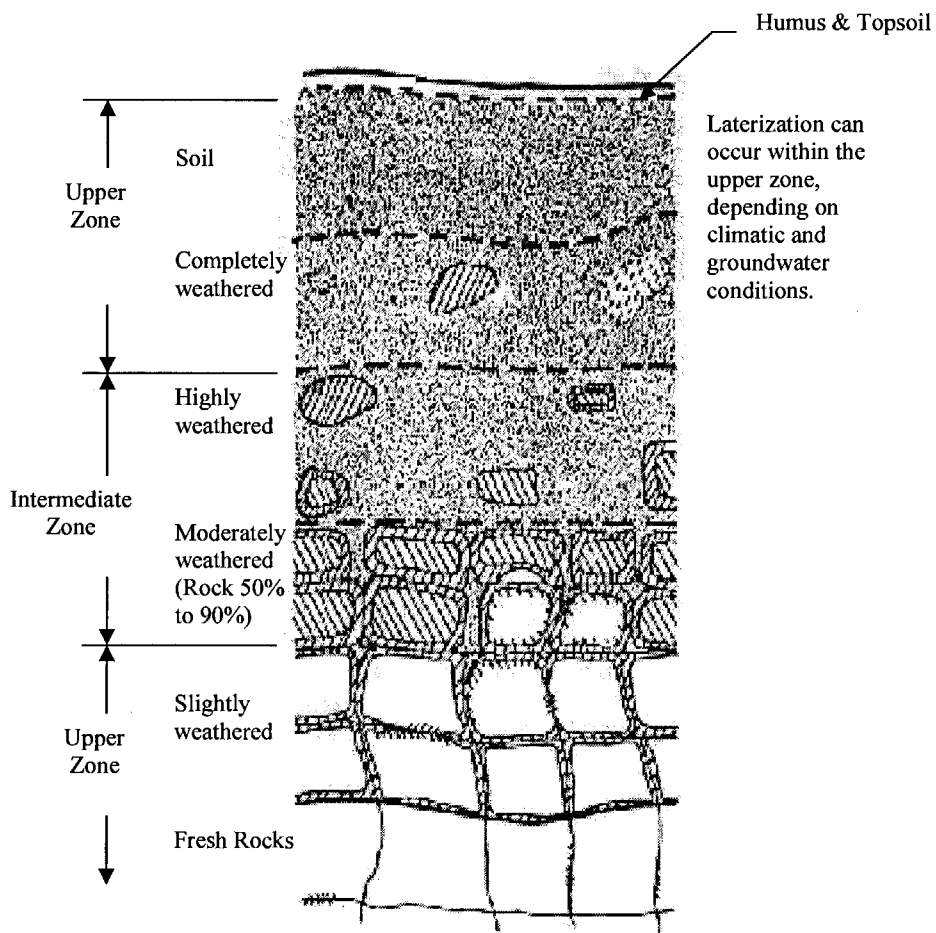


Figure 2.4 Schematic diagram of typical tropical residual soil profile
(from Little, 1969)

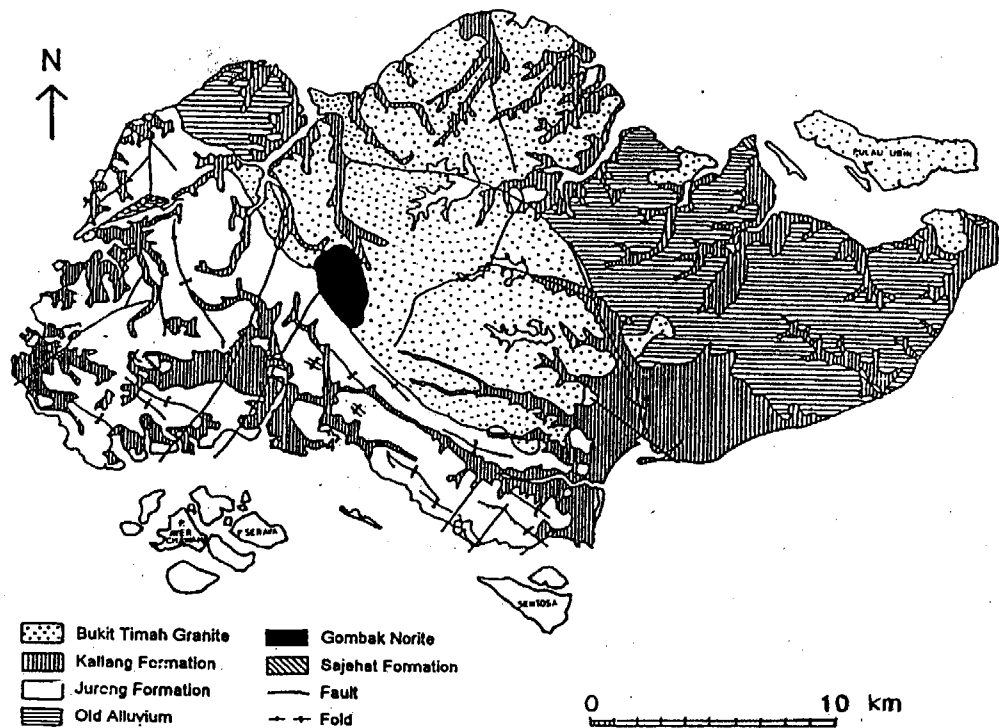


Figure 2.5 Singapore geologic map (from PWD, 1976)

Basic properties of Singapore residual soils have been reported by Poh et al. (1985), Yong et al. (1985) and Winn et al. (2001). Residual soils derived from Bukit Timah Granite formation is generally stiff to hard and it has sand contents from 10% to 90%. The liquid limit varies from 20% to 80%, the plastic limit varies from 20% to 40% while the specific gravity ranges from 2.55 to 2.75. Sedimentary Jurong formation is derived from the weathering of sedimentary rocks consisting of mudstone, sandstone, limestone, siltstone, shale and conglomerate. It has sand contents from 10% to 50%. The liquid limit varies from 30% to 65%, plastic limit varies from 25% to 50% while the specific gravity ranges from 2.60 to 2.75. In this research, two residual soils from the Jurong Formation were used.

2.3 SOIL SUCTION

In the unsaturated zone, the pore-water pressure situation is complex and contrary to the situation below the groundwater table. The pore-water pressure is lower than the atmospheric pressure (i.e. negative). This negative pore-water pressure is called soil suction. The concept of soil suction has been used by soil scientists for many years. The theoretical concepts of soil suction were originally introduced in soil physics in early 1900s by soil scientists (Buckingham, 1907; Gardner and Widtsoe, 1922; Richards, 1928) to describe the forces for migration and equilibrium of soil water, mainly for agricultural purposes. These concepts were later adopted in geotechnical engineering as a result of the need to establish the mechanics for unsaturated soils. Soil suction is an inherent property of soil with which a soil will absorb more water. Yong and Warkentin (1966) suggested that there are basically two common approaches to explain the interaction of soil water and soil. The first approach is based on thermodynamic principles and is called the potential energy approach. This approach considers the energy with which water is held in soil and can be determined by measuring the relative humidity of the soil sample. The second approach is based on the geometry of the soil particles and statics and is called the geometric or mechanical approach.

The term soil suction was used by Schofield (1935) to represent the pressure deficiency in the pore water of any soil (saturated or unsaturated) that has the capacity to absorb water if additional water was supplied at atmospheric pressure. Soil suction is commonly referred to as the free state of soil water (Edlefsen and Anderson, 1943). The free energy of soil water can be measured in terms of the partial vapour pressure of the soil water (Richards, 1965). In 1965, Aitchison proposed definitions for total, matric and osmotic suctions as suction components from a thermodynamic basis which are generally accepted in geotechnical engineering (Fredlund and Rahardjo, 1993a).

The engineering behaviour of an unsaturated soil is significantly influenced by the suction in the soil. Suction in unsaturated soil consists of two components, namely: matric suction and osmotic suction. Both components are important to unsaturated soil mechanics. The sum of the two components is called total suction which can be expressed in equation form as:

$$\psi = (u_a - u_w) + \pi \quad (2.1)$$

where:

- ψ = total suction
- $(u_a - u_w)$ = matric suction
- u_a = pore-air pressure
- u_w = pore-water pressure
- π = osmotic suction

Soil suction measuring techniques can be categorized as either direct or indirect measurements. Direct measurement measures the equilibrium state of a soil-water system without involving the external medium for moisture equalization. Indirect measurement involves another parameter (i.e. relative humidity, resistivity, conductivity and moisture content) which is related to suction through calibration against known values of suction. The measurement can be done in both the laboratory and in the field. There are several devices which can be used to measure soil suction (Figure 2.6). Each of these devices suffers from some limitations. Table 2.1 presents several suction measurement devices with ranges of measurements, advantages and limitations.

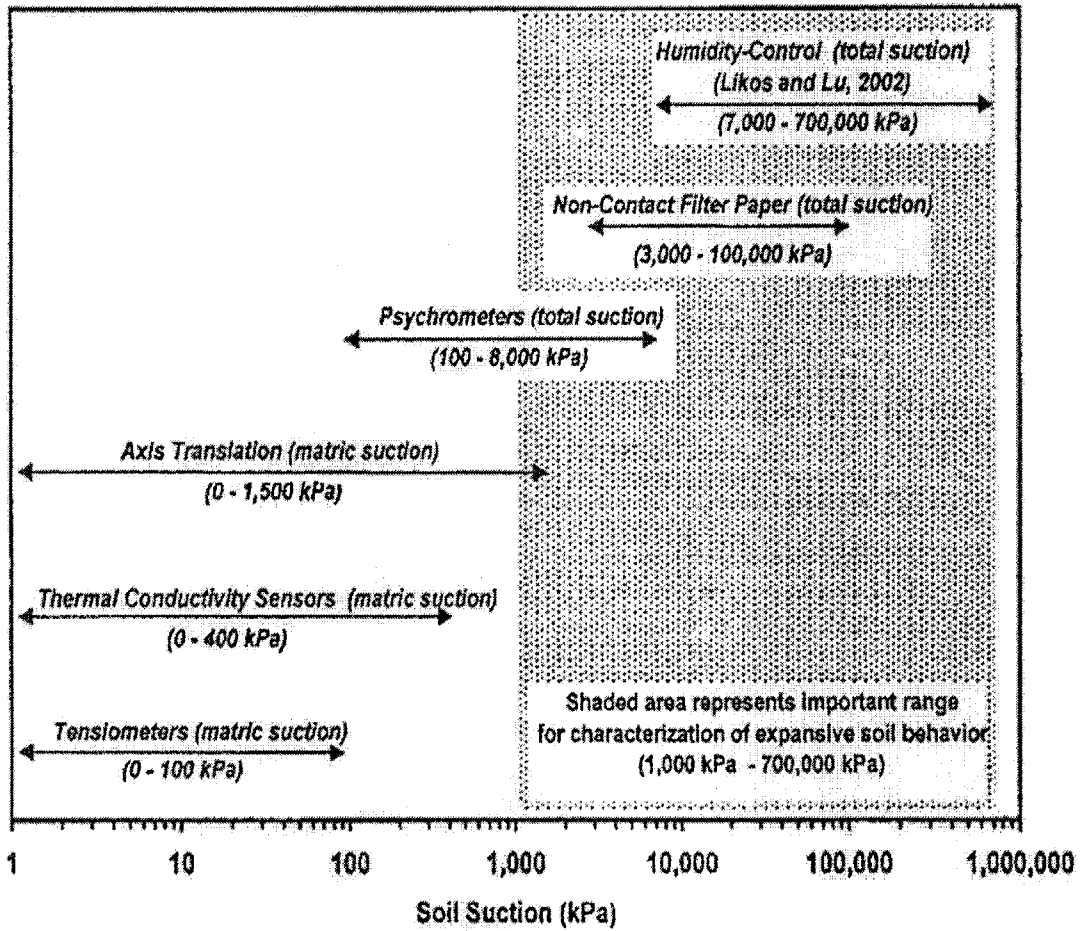


Figure 2.6 Approximate ranges for various suction measurement techniques (from Likos and Lu, 2002)

Table 2.1 Soil suction measurement devices (from Guan, 1996)

| Devices | Suction Component and Ranges | Advantages | Limitations |
|--|---|--|---|
| Ordinary Tensiometer and Suction Plate | matric, 0 ~ 100 kPa | <ol style="list-style-type: none"> 1. Low cost 2. Easy handling 3. Relatively quick response 4. Lab and field measurement | <ol style="list-style-type: none"> 1. Cavitation limit measurement less than 100 kPa 2. Difficulty with air bubbles |
| Osmotic Tensiometer | matric, 0 - 2000 kPa | <ol style="list-style-type: none"> 1. No cavitation 2. High suction measurement | <ol style="list-style-type: none"> 1. Poor reliability due to drift of reference pressure 2. Strict temperature control 3. Only for lab measurement |
| Pressure Membrane (Null Technique) | matric 0 ~ 2000 kPa | <ol style="list-style-type: none"> 1. Low cost 2. No cavitation 3. High suction measurement | <ol style="list-style-type: none"> 1. Long equilibrium time for high suction values 2. Deterioration of membrane and air diffusion difficulty 3. Only for lab measurement 4. Best for suction control rather than measurement |
| Pressure Plate (Null Type) | matric 0 ~ 2000 kPa | <ol style="list-style-type: none"> 1. No cavitation 2. High suction measurement | <ol style="list-style-type: none"> 1. Long equilibrium time for high suction values 2. Air diffusion difficulty 3. Only for lab measurement 4. Best for suction control rather than measurement |
| Filter Paper | total or matric, entire range of suction | <ol style="list-style-type: none"> 1. Low cost 2. Cover entire range measurement | <ol style="list-style-type: none"> 1. Strict control of environmental conditions 2. Difficult handling 3. Poor accuracy 4. User dependent 5. Generally for lab measurement |
| Psychrometer - Thermistor (high suction) - Thermocouple (low suction) Hygrometer | total 100 - 8000 kPa | <ol style="list-style-type: none"> 1. Wide range of measurement | <ol style="list-style-type: none"> 1. Constant temperature environment 2. Poor accuracy 3. Generally for lab measurement |
| Electrical Conductivity Sensors | matric 0 ~ 500 kPa and possibly up to 1,500 kPa | <ol style="list-style-type: none"> 1. May be good for clean sand and silts 2. Long term automatic monitoring 3. Lab and field measurements | <ol style="list-style-type: none"> 1. Sensitive to dissolved salts 2. Temperature dependent 3. Long equilibrium time 4. Deterioration of resistance block |
| Thermal Conductivity Sensors | matric 0 - 400 kPa | <ol style="list-style-type: none"> 1. Independent dissolved salts and ambient of temperature 2. Long term automatic monitoring 3. Lab and field measurement | <ol style="list-style-type: none"> 1. Long equilibrium time 2. Accuracy need improvement, especially for suction greater than 170 kPa 3. Deterioration of thermal block |

2.3.1 Total Suction

Total soil suction is a function of several variables including moisture content, pressure on soil, temperature, mineralogy and texture of soil particles, soil fabric and the amount of soluble matter in the soil water. Aitchison (1965) defined total suction as the equivalent suction derived from the measurement of the partial pressure of the water vapour in equilibrium with a solution identical in composition with the soil water, relative to the partial pressure of water vapour in equilibrium with free pure water.

Total suction of a soil can be described as the relative humidity of the pore-water vapour in the soil. From the thermodynamic standpoint, the relationship between soil suction and the partial pressure of the pore-water vapour can be written as:

$$\Psi = -\frac{RT}{v_{w0}\omega_v} \ln\left(\frac{\bar{u}_v}{\bar{u}_{v0}}\right) \quad (2.2)$$

where:

- Ψ = soil suction or total suction (kPa)
- R = universal (molar) gas constant [i.e., 8.31432 J/ (mol K)]
- T = absolute temperature [i.e., $T = (273.16 + t^\circ)$ (K)]
- t° = temperature ($^\circ\text{C}$)
- v_{w0} = specific volume of water or the inverse of water density (i.e., $1/\rho_w$) (m^3/kg)
- ρ_w = water density (i.e., 998 kg/m^3 at $= 20^\circ\text{C}$)
- ω_v = molecular mass of water vapor (i.e., 18.016 kg/kmol)
- \bar{u}_v = partial pressure of pore-water vapor (kPa)
- \bar{u}_{v0} = saturation pressure of pore-water vapour over a flat at the same temperature (kPa)

The term $\ln\left(\frac{\bar{u}_v}{u_{v0}}\right)$ in Equation 2.2 is referred to as the relative humidity,

R_h . Figure 2.7 shows a plot of Equation 2.2 at three different temperatures. According to Figure 2.5, total suction is equal to zero when the relative humidity, R_h , is equal to 100%. Generally, relative humidity less than 100% means that partial pressure is lower than the saturation pure water vapour pressure over a flat surface at the same temperature.

Lang (1967) suggested that suction can be generated by inducing a relative vapour pressure. The relative vapour pressure is related to the osmotic coefficient of the solution which depends on the number of mole solute in the solution. Therefore, a specific total suction can be generated in the vapour by adding a corresponding amount of solute into the solvent.

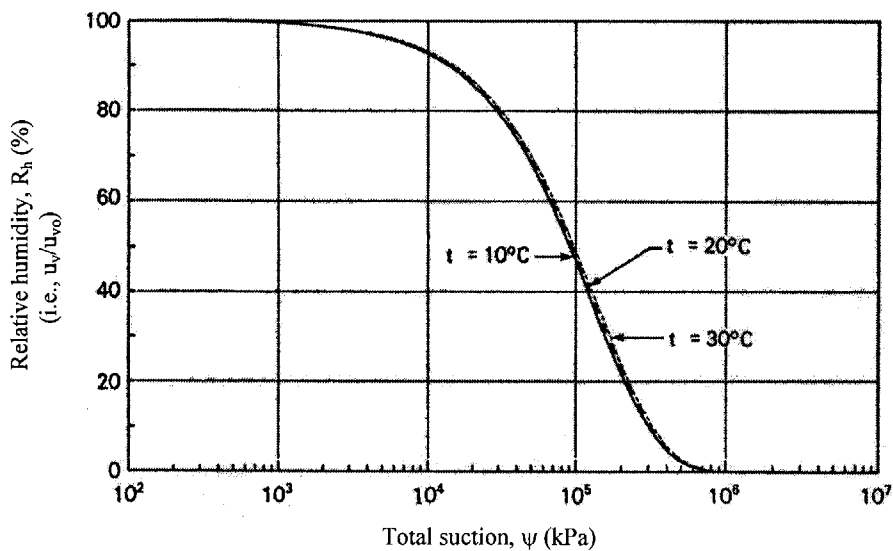


Figure 2.7 Relative humidity versus total suction relationship
(from Fredlund and Rahardjo, 1993a)

The measurement of total suction is conducted by measuring the relative humidity in the soil. The measurement can be conducted either by using psychrometer as a direct measurement or by using a filter paper as an indirect measurement. The psychrometer measures the relative humidity of the air above any material in an enclosed system. It works by measuring the temperature difference between a non-evaporating surface and an evaporating surface. The difference in temperature between these surfaces is related to the relative humidity. It can measure total or osmotic suction depending on whether the psychrometer is exposed to the soil or only to the soil pore water. The filter paper is a porous material and it has a specific suction versus water content characteristic relationship. When a non-contact filter paper is placed with a soil in a closed environment, vapour flow will occur between the soil and the filter paper until the suction of the filter paper becomes equal to the total suction in the soil. Filter paper method for total and matric suction measurement using Whatman No.42 and Schleicher and Schuell No. 589 (S&S 589) filter papers have been evaluated by Leong et al. (2002). The results showed that the response of Whatman No. 42 filter paper was more consistent than S&S 589 filter paper. Total and matric calibration curves were obtained for Whatman No. 42 and S&S 589 filter papers. It was found that the total suction calibration curve was not very sensitive to total suctions less than 1000 kPa.

Gee et al. (1992) used water activity meter for measuring low water potentials in soils ranging in wetness from near saturation to air dry (water potentials from -0.4 to \approx - 100 MPa). The meter senses the dew point of water vapour with a miniature cooled mirror located above a soil sample in a closed chamber. The readings for individual samples were obtained rapidly. The repeatability of measurements was found to be equal to or better than the precision of the water activity meter. They concluded that the water activity meter will be very useful in measuring water potentials of dry surface and it will also be useful for measuring water potential of soils that have been dried by evaporation.

Leong et al. (2003a) used a chilled-mirror dew-point technique with a selected temperature to measure the relative humidity of two residual soils from Singapore. The tests results showed that total suctions obtained using the device were always higher than the sum of the matric and osmotic suctions measured independently.

Polymer capacitance sensors were used by Albrecht et al. (2003) to measure the relative humidity of nine soil samples from western United States. The results showed that the polymer capacitance sensors can be used to obtain unbiased estimates of relative humidity for a broad variety of soils with a precision of approximately $\pm 3\%$.

2.3.2 Matric Suction

Matric suction is generally associated with the capillary model arising from the surface tension of water and adsorption forces (Fredlund and Rahardjo, 1993a). Matric suction ($u_a - u_w$) is defined as the difference between the pore-air pressure, u_a , and the pore-water pressure, u_w across the air water interface and inversely proportional to the radius of curvature of the water surface. Aitchison (1965) defined matric suction as the equivalent suction derived from the measurement of the partial pressure of the water vapour in equilibrium with the soil water, relative to the partial pressure of the water vapour in equilibrium with a solution identical in composition with the soil water. It is generally described using the capillary phenomenon as shown in Figure 2.8. This capillary water has negative pore-water pressure with respect to atmospheric air pressure.

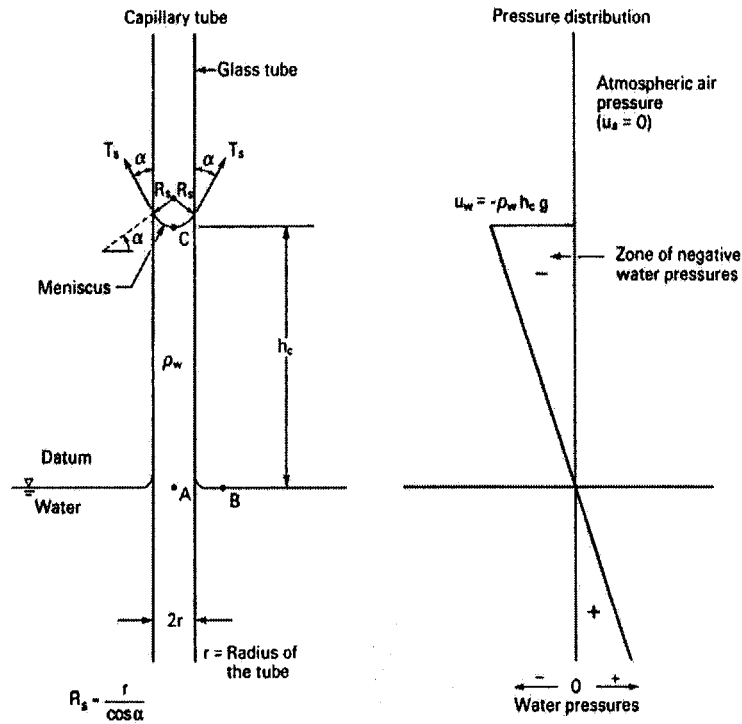


Figure 2.8 Capillary model of matric suction in soil
(from Fredlund and Rahardjo, 1993a)

At the interface, the force equilibrium is given by Kelvin's equation:

$$(u_a - u_w) = \frac{2T_s \cos \alpha}{r} \quad (2.3)$$

At point C, the pressure head is equal to the negative value of the elevation head where the water pressure is negative (i.e. $u_w = -\rho_w g h_c$) and the air pressure is atmospheric (i.e. $u_a = 0$). As a result, the matric suction at point C, given by the

difference between the air pressure and the water pressure, is positive which can be written as:

$$(u_a - u_w) = \rho_w g h_c \quad (2.4)$$

Substituting Equation (2.3) into Equation (2.4) gives the capillary height, h_c , as:

$$h_c = \frac{2T_s}{\rho_w g r} \quad (2.5)$$

Matric suction is one of the stress state variables that control the engineering behaviour of unsaturated soils. Matric suction is largely affected by the environmental changes. A change in matric suction would result in a change in the degree of saturation as the air spaces would fill up the water spaces where water have been forced out due to the greater pore-air pressure as compared to the pore-water pressure. At lower matric suctions, the degree of saturation is high. At higher matric suctions, more of the voids are filled with air and as such there is a smaller water volume for the water to flow through.

Matric suction can be estimated either by direct or indirect method. The tensiometer and the axis translation technique are often used. In a tensiometer, the negative pore-water pressure is transmitted directly to a pressure measurement system. It can be used in the field and in the laboratory. The interpretation of the data is simpler compared to other methods, but the main problem of this equipment is that cavitation will take place at a suction of about 90 kPa (Stannard, 1992). The null type axis translation technique was first developed by Hilf (1956) as a means of avoiding the cavitation problem (Figure 2.9). This technique is fairly accurate but best suited to soils with a continuous air phase as pointed out by Bocking and Fredlund (1980). The response time, however can be long, as shown in Figure 2.10. The method is not suitable for field installation since the

application of high air pressure in the field is not practical (Fredlund and Rahardjo, 1993a).

Matric suction can be determined from indirect methods using filter paper, thermal conductivity sensor, and electrical conductivity sensor and time domain reflectometry. In the filter paper method, the filter paper is placed in contact with the soil pore-water. The absorption capacity of the filter paper and the suction difference between the soil and the filter paper will cause a flow of water to occur until equilibrium. Once equilibrium is established, the water content of the filter paper is determined. The resulting moisture in the filter paper is a function of the matric suction of the soil. A thermal conductivity sensor uses the thermal properties of a porous material to measure matric suction indirectly. The sensor is placed in contact with the soil until equilibrium is reached. The thermal conductivity of a porous material increases with its degree of saturation. It can be used in the field, however, the measurement suffers from hysteretic effect. The CSI 229 thermal conductivity sensor was used by He (1999) to measure matric suction ranging from 8 to 1500 kPa. It was found that the sensitivity of the CSI 229 thermal conductivity sensor was lower for suctions greater than 300 kPa. The electrical conductivity of a porous material indicates the amount of water contained in the porous material and can be obtained by measuring the electrical resistance of the porous material. The electrical resistance of the porous material increases with its degree of saturation. In soil science, time domain reflectometry is used to measure water content in soil (Davis 1975, Davis and Annan 1977, Topp et al. 1980, Topp and Davis 1985, Dalton 1992). Theoretical analysis and experimental correlation have shown that the travel time of the pulse is proportional to the apparent dielectric constant, k_a , of the soil which is uniquely correlated with the volumetric water content of the soil (Davis 1975, Davis and Annan 1977, Topp et al., 1980).

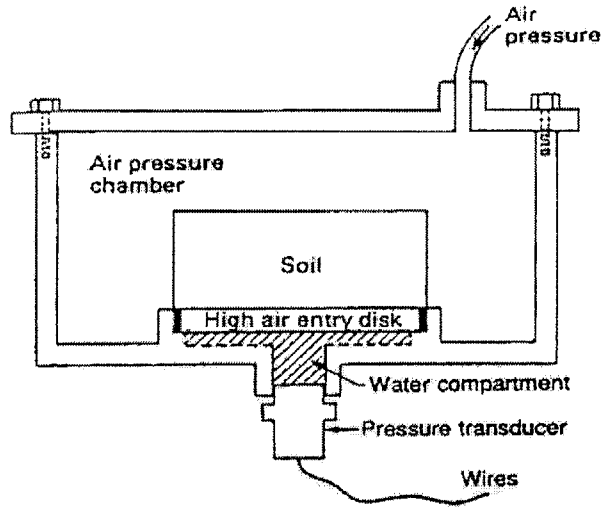


Figure 2.9 Measurement of matric suction using null-type axis translation technique (from Fredlund and Rahardjo, 1993a)

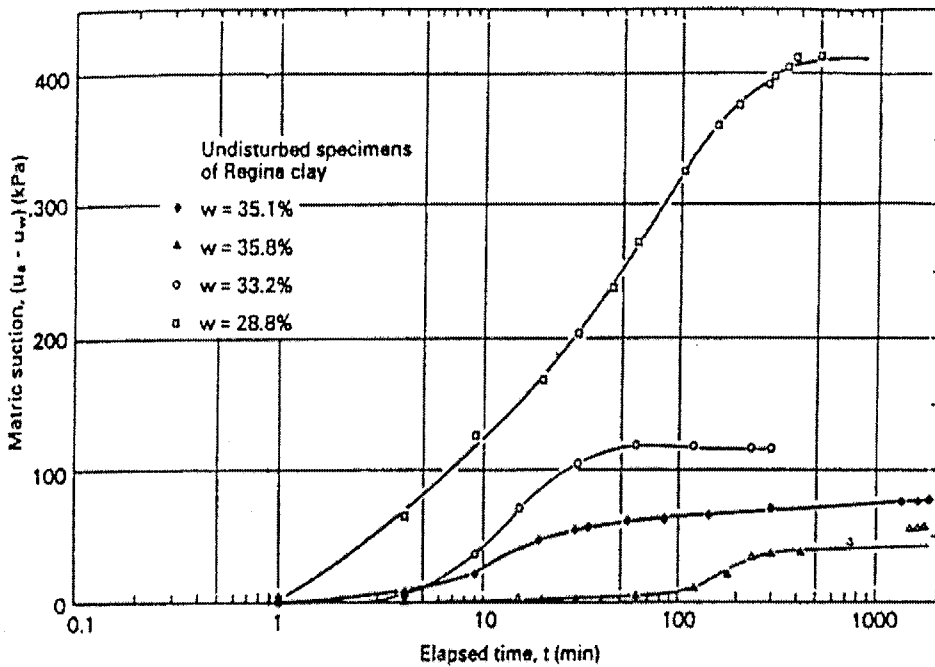


Figure 2.10 Response time for matric suction measurements on Regina clay using the null-type axis translation technique (from Widger, 1976)

2.3.3 Osmotic Suction

The pore-water in a soil generally contains dissolved salts. Increasing amount of dissolved salts in the soil's pore-water will cause a decrease in relative humidity. This is referred to as the osmotic suction, π . Aitchison (1965) defined osmotic suction as the equivalent suction derived from the measurement of the partial pressure of the water vapour in equilibrium with a solution identical in composition with the soil water, relative to the partial pressure of water vapour in equilibrium with the pure water.

The salt content of the soil can be estimated from an electrical-conductivity measurement on saturated soil paste or a more dilute suspension of soil in water. A better estimate of soluble salt can be obtained from the conductivity of a pore-water extract of the soil. Figure 2.11 shows the relationship of concentration of single salt solution to electrical conductivity. The relationships between osmotic pressure and electrical conductivity for pore water and single salt solutions are shown in Figures 2.12 and 2.13, respectively.

Osmotic suction has an effect on the mechanical behaviour of unsaturated soils. If the salt content in a soil changes, there will be a change in the overall volume and shear strength of the soil (Fredlund and Rahardjo, 1993a). In most geotechnical engineering problems, the change in osmotic suction can be neglected and is generally less significant than matric suction changes. Consequently, if the pore-air pressure is equal to the atmospheric pressure then the total pressure becomes equal to the negative pore-water pressure. The role played by osmotic suction in influencing the mechanical behaviour of a soil may or may not be the same as the role played by matric suction (Fredlund and Rahardjo, 1993a).

Osmotic suction measurement is conducted on the extracted pore water. Osmotic suction can be measured using a psychrometer, filter paper or electrical

conductivity meter. However it is necessary to extract the pore water from the soil to measure the osmotic suction. The extracted pore water is assumed to be identical in composition to the pore water in the soil. Theoretically, the measured osmotic suction should be equal to the difference between the measured total suction and the matric suction of the soil sample. Unfortunately these attempts to measure the osmotic component of total suction have not been very successful, as extracting the soil water from the soil and maintaining its composition is very difficult (Ridley, 1995).

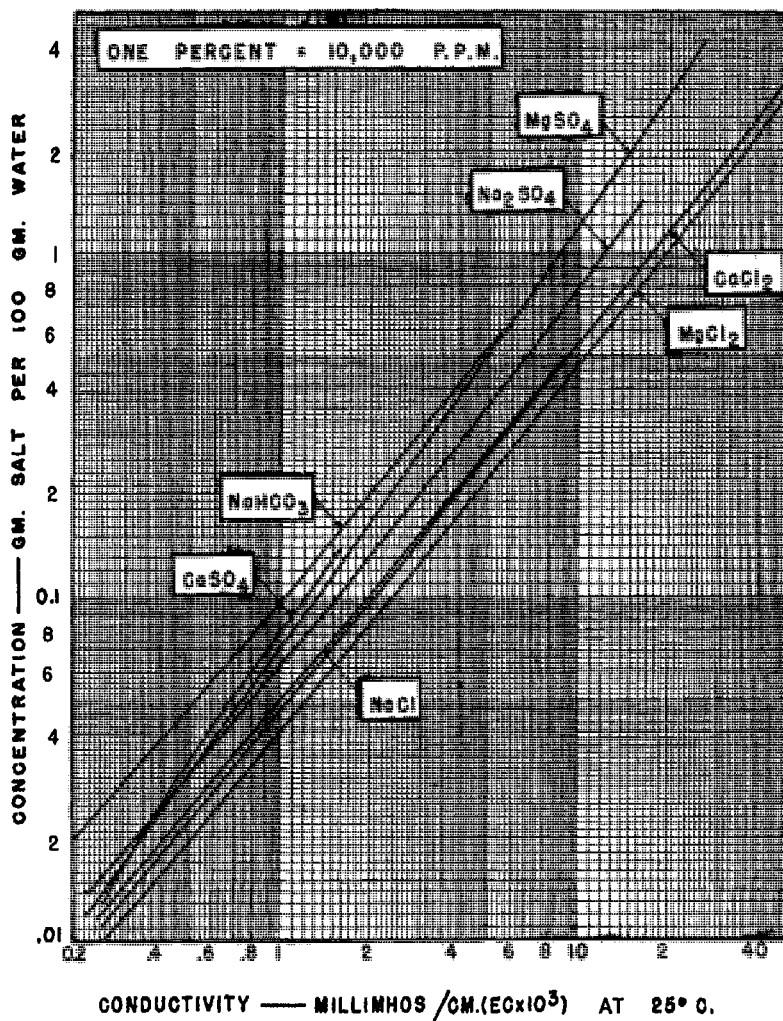


Figure 2.11 Concentration of single salt solutions in percent as related to electrical conductivity (from USDA, 1960)

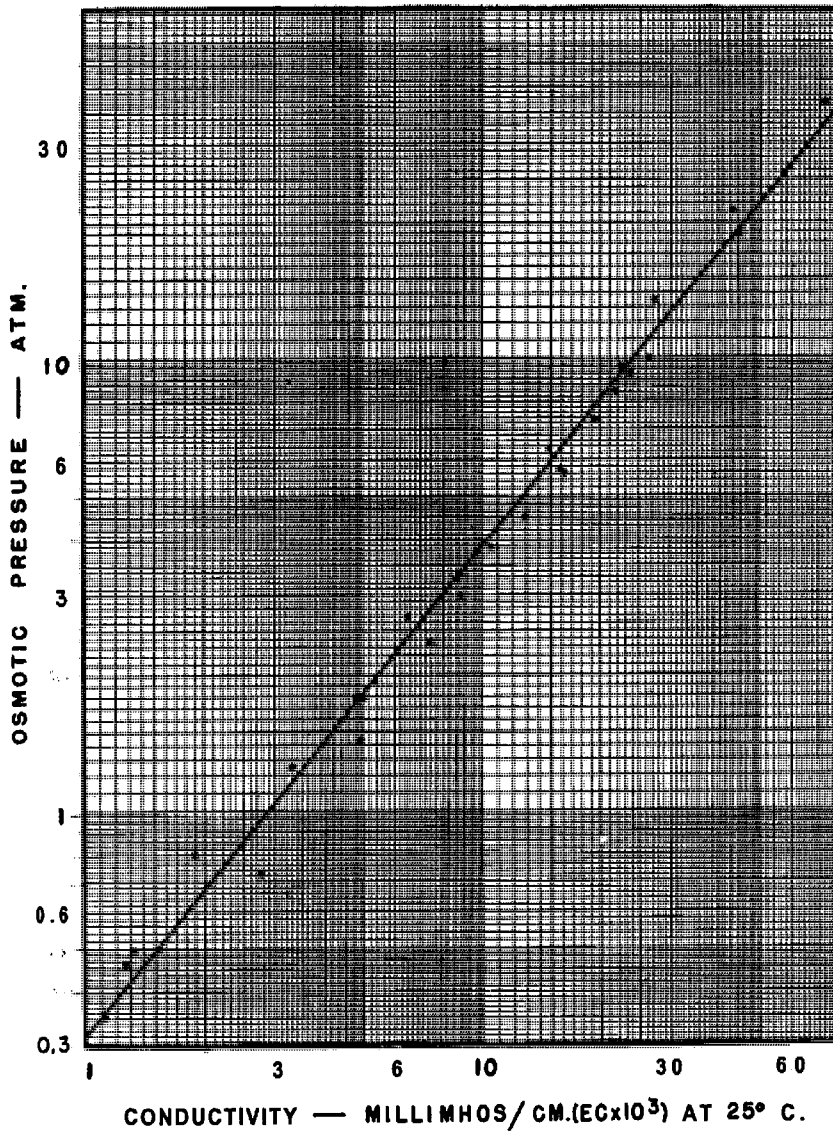


Figure 2.12 Osmotic pressure plotted against electrical conductivity for pore water containing dissolved salts (from USDA, 1960)

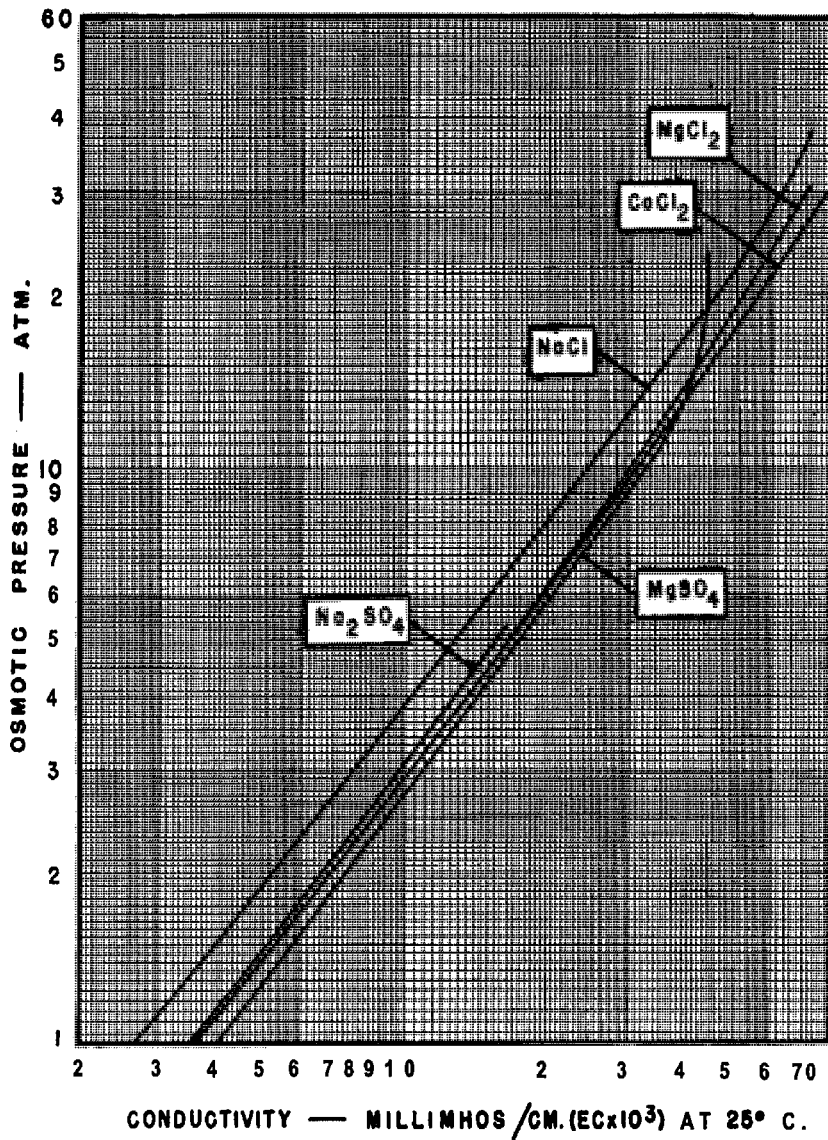


Figure 2.13 Osmotic pressure of single salt solutions plotted against electrical conductivity (from USDA, 1960)

There are several methods which can be used to extract the pore water from the soil samples, namely: dilution or saturation extract, leaching, centrifuging, immiscible liquid displacement, gas extraction method and pressurised squeezing (low and high pressure). Some of the pore water extraction methods mentioned above is more suitable to certain soil types. The methods are briefly discussed below.

Dilution or saturation extract method has been used for fine-texture soil which did not produce pore water when using other methods. In this method, an accurate determination of moisture content is needed and is based on the assumption that all salts present in the formation water are precipitated during drying and are redissolved when distilled water is added to the dried sample. Swarzenski (1959) carried out semi-quantitative analysis for Long Island soil samples using the dilution method and the centrifuging method. Swarzenski (1959) found that the dilution method is not suitable for certain quantitative analysis because of the possible errors inherent from laboratory procedure such as the dehydration temperature, limitations of laboratory equipment and technique, and other unknown factors. However, the results obtained using the dilution method may be considered a good semi-quantitative method in the determination of brackish and saline ground water zones. In summary, the dilution or saturation extract method may yield pore water that does not contain all the salts in the original pore water.

Leaching generally refers to the removal of a substance from a solid via a liquid extraction media. The desired component diffuses into the solvent from its natural solid form. The leaching method is generally used for preparing a soil sample with a particular pore water composition or for evaluating solid waste, rather than for pore water chemical analysis. Cameron (1911) found that leaching involves potential error not only by solubilization of solids, but also by changing sediment or water equilibria in soils. Parker (1921) found that calcium concentrations obtained from leaching techniques were twice as high as those

obtained by alcohol displacement method. Tsyba and Kriukov (1959) and Arslanbekova and Kriukov (1962) found leachate compositions of drill cores grossly different from those extracted by pressure filtration. The only constituents which can be safely determined by simple leaching techniques are relatively unreactive anions such as chloride as shown by Swarzenski (1959), Kullenberg (1952) and Arrhenius (1952).

Centrifuging has been used for fluid removal from various saturated or partially saturated geological materials for several decades. Centrifuging is the separation of solid or liquid particles of different densities by rapidly rotating the material in a suitable container in a horizontal circle. Tiensing et al. (2001) carried out tests for ecotoxicity assessment of two soils amended with Zn and Cd at different concentrations using two extraction techniques, centrifugation and Rhizon sampler. Their results showed that the concentrations Zn and Cd were significantly higher in soil solution extracted using the centrifugation technique compared with the Rhizon sampler technique. Edmunds and Bath (1976) investigated the properties of Cretaceous Chalk and Triassic sandstone aquifers of the U.K using the centrifuging method. From the investigation carried out, they concluded that absolute accuracy for some elements may be as low as $\pm 100\%$. Centrifuging was also employed by Swarzenski (1959) to determine the chloride content of water in core samples. The results obtained are in close agreement with known ground water salinities in the area. According to Swarzenski (1959), centrifuging method is unreasonable for formation sample which contain essentially fresh water because this method requires large amount of sample for the accurate determination of low chloride concentration. Elkhatib et al. (1987) developed a modified centrifuging technique for obtaining soil solutions. From their test results, it was found that coefficients of variation and standard deviations for ionic composition of soil solutions obtained by the modified centrifuging method were consistently lower than those obtained from the immiscible liquid displacement method.

Immiscible liquid displacement is used for quantitative interstitial water chemical analyses. According to Scholl (1963), immiscible liquid displacement is particularly appropriate for tightly-packed permeable sediments because other methods failed to extract the interstitial water in a virtually unmodified state and is recommended for work which requires quantitative knowledge of interstitial water chemistry. This method is seldom used because of its messiness.

Gas extraction method is used for semi-quantitative analyses and is applicable only for sandy or silty soils. Gas of relatively high humidity can be made to pass through water-saturated sediments to expel the interstitial water. The expulsion of gas is carried out in a filter press fitted with a gas-inlet valve (Lucsczynski, 1961; Siever, 1962). Scholl (1963) used the gas extraction method for extracting interstitial water from relatively small amounts of coarse-grained sediments for chemical analyses. The results showed that interstitial water extracted from highly permeable coarse-grained sediments with humidified air causes evaporation but this error is only about 0.3% and can be neglected. The gas extraction method does not readily purge interstitial water from sands with low permeability unless the driving pressures are higher than 20 psi (138 kPa).

Pressurised squeezing method has several favourable properties as a fluid extraction technique such as less contamination and the potential efficiency and convenience in removing fluid. The procedure for pore water recovery is to squeeze the sediment using low pressure mechanical squeezing (Siever, 1962), low pressure gas squeezing (Hartmann, 1965; Presley et al., 1967) or high pressure mechanical squeezing (Manheim, 1966). A low pressure squeezing technique was used by Siever (1962) for soft sediments such as recent ocean deposits. The main advantage of this method compared to others is that the equipment is portable and the pore water from soft sediments can be squeezed immediately after the soil sample has been recovered from the ground. Presley et al. (1967) used a simple and inexpensive squeezer on a 5 m core from Santa Catalina Basin, Southern California, to investigate the concentrations of various

transition elements, including iron, in the interstitial water of ocean sediments. From their study, it was concluded that the squeezer combines the efficiency of other models with simplicity, ease of operation, low cost of construction, and complete freedom from metallic contamination problems.

A high pressure squeezing method is applicable for stiffer materials. The working principle of this method is the same as for the low pressure mechanical squeezing except that the pressure applied to the piston is as large as needed to extract pore water from compacted sediments. A high-pressure squeezer developed by Manheim (1966) was successfully employed on oceanic drill-cores and has become the standard squeezer used in the Deep Sea Drilling Project. Morgenstern and Balasubramonian (1980) developed a squeezer similar to Manheim's. Figure 2.14 shows the high pressure squeezer used by Morgenstern and Balasubramonian (1980). Morgenstern and Balasubramonian found that high pressure mechanical squeezing was useful for extracting representative specimens of natural pore fluid from clay shales. A pore water squeezer similar to Manheim's was used by Iyer (1990) for the extraction of several clay shales and bentonitic clays from Western Canada. The results showed that the cation concentration of the water squeezed out at pressures less than 19,300 kPa remained constant and the concentration decreased at higher pressures. Krahn and Fredlund (1972) used a squeezer to obtain pore fluid from Regina clay for osmotic suction measurement. They concluded that values of the osmotic suction of the pore fluid obtained using the squeezing technique for the Regina clay gave better agreement with the difference between total and matric suctions compared to the values obtained from the saturation extract technique (Figure 2.15). The differences between total and matric suctions for the till sample deviate somewhat from the squeezing technique values particularly at lower water contents (Leong et al., 2003a). The difference in pore-water composition has been attributed to the double layer water. At high squeezing pressure, it is possible to squeeze out the double layer water together with the free water (Iyer 1990). For consistent results,

it is necessary to ensure that the squeezing pressure used does not affect the composition of the extracted pore water.

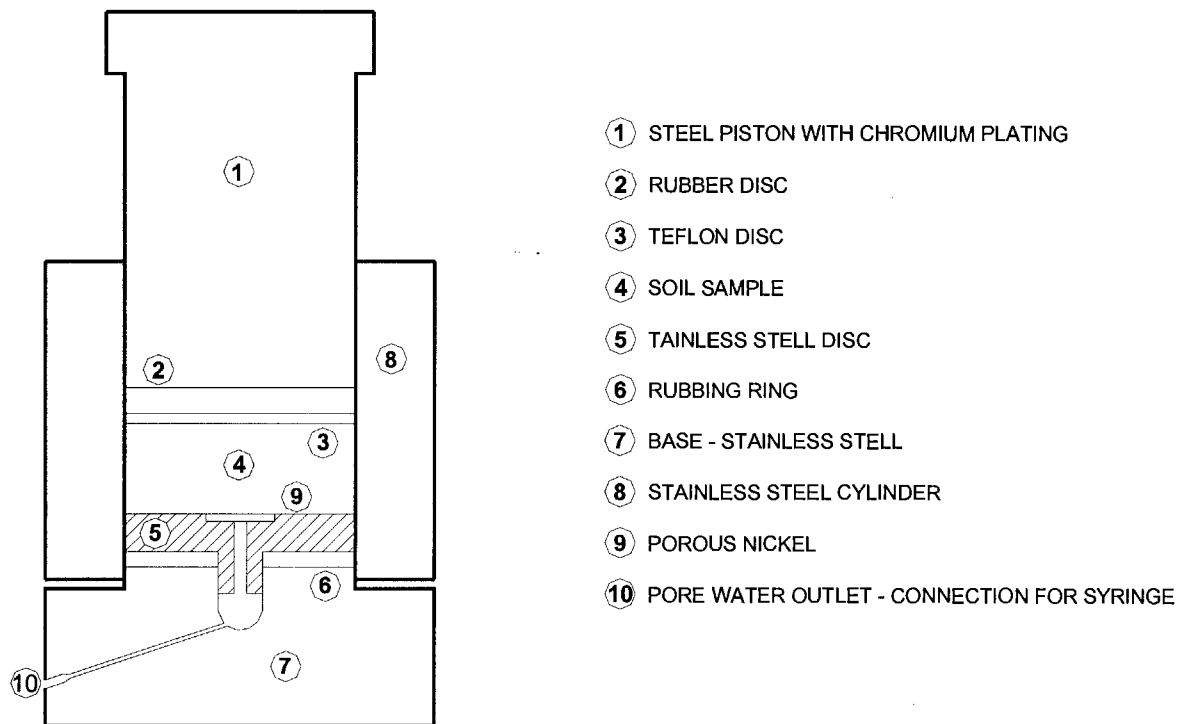
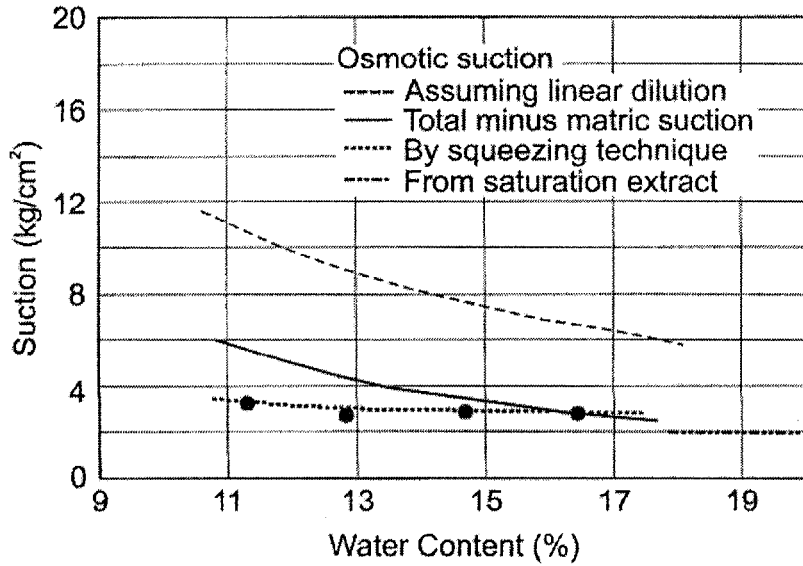
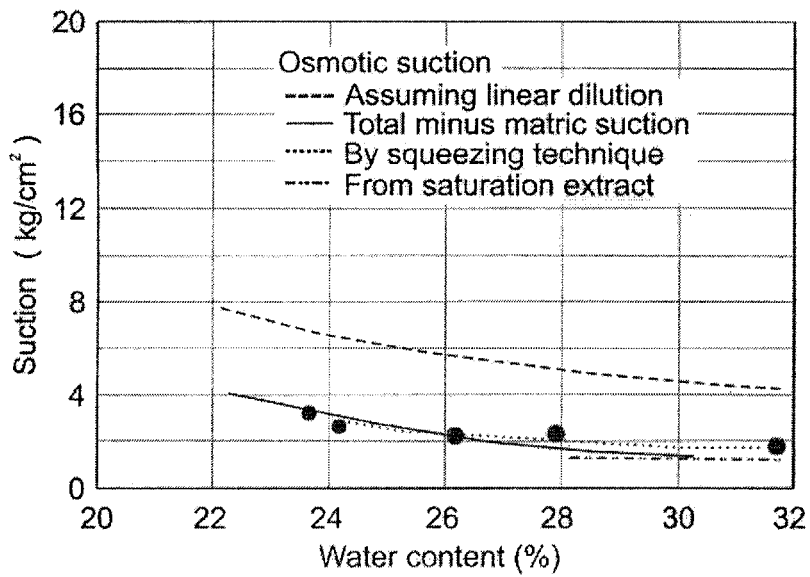


Figure 2.14 High pressure pore fluid squeezer of Morgenstern and Balasubramonian (1980)



(a)



(b)

Figure 2.15 Total, matric and osmotic suction measurements on: (a) glacial till and (b) compacted Regina clay (from Krahn and Fredlund, 1972)

2.4 VOLUME CHANGE THEORY

Volume changes in soils are important because of their consequences in terms of settlement due to compression and heave due to expansion. Many geotechnical problems require an understanding of the volume change behaviour of the soil due to various applied loads. Change in volume lead to change in strength and deformation properties that in turn influence stability. Changes in volume are caused by changes in applied stress, chemical and moisture, environments and temperature (Mitchell, 1976). Fundamentally, volume changes are mostly caused by stress changes. The amount of volume change depends on the soil type and structure, the initial soil density, the imposed stress state, and the degree of wetting. Volume changes of the soil due to the applied loads or environmental changes due to rainfall and evaporation are of interest to practising engineers.

In unsaturated soils, a major consideration for geotechnical engineers relates to the effect of wetting on engineering performance. A commonly cited problem with unsaturated soils is wetting-induced volume change (Houston and Houston, 1993). Some soils and conditions lead to increased volume (expansion) due to the addition of water giving rise to expansive or swelling soil problem. Other soils and conditions lead to decreased volume (collapse) upon wetting giving rise to collapsible soil problem. Many factors can affect the collapsible and swelling behaviour of soil, including natural moisture content, insitu density, mineral composition, type of cementing materials, structure and fabric in the undisturbed state, and the applied pressure when the soil is saturated. Soil with very low natural moisture content tends to either swell or collapse depending on the magnitude of pressure at which the soil is saturated. The mineral composition of the clay fraction also plays an important role. When montmorillonite is present in the clay fraction, the soil as a whole is expected to swell and exert higher swelling pressure. On the other hand, when clay minerals are inactive, clay

particle only act as a cementing material between the grains and the soil tend to lose its strength and cohesion when water is added.

In general, swelling of soils is due to the presence of expansive clay minerals, hydration of cations on the clay surface and release of intrinsic stresses by overconsolidation (Dhowian, 1990). Gromko (1974) has identified that among the clay minerals, illite, kaolinite, and montmorillonite possess the greatest ability to swell.

The wetting-induced collapse of compacted soils is now a well-recognized phenomenon. A collapsible soil is commonly referred to as a metastable-structured soil. The probable amount of collapse has become the major concern of researches on collapsible soil. Possible collapse mechanisms have been proposed by several researchers (Holtz and Hilf, 1961; Burland, 1964; Dudley, 1970; and Barden et al., 1973). According to Dudley (1970), Barden et al. (1973) and Mitchell (1976), four factors are needed to produce the collapse in soil structure:

1. An open, partially unstable, unsaturated fabric.
2. A high enough net total stress that will cause the structure to be metastable.
3. A bonding or cementing agent that stabilizes the soil in the unsaturated condition.
4. The addition of water to the soil, which causes the bonding or cementing agent to be reduced and the interaggregate or intergranular contacts to fail in shear, resulting in a reduction in total volume of the soil mass.

The collapse behaviour of compacted and cohesive soils depends on the percentage of fines particularly clay fraction, the initial water content, initial dry density, the energy and process used in compaction (Jennings and Burland, 1962; Barden et al., 1973; Hodek and Lovell, 1979). Alonso (1993) stated that the

microstructure is widely recognized as important in explaining the behaviour of collapsing soils despite the fact that it lacks a simple quantitative descriptor. Due to the behaviour of compacted collapsing soil, it is recognized that:

1. Any type of soil compacted at dry of optimum conditions and at a low dry density may develop a collapsible fabric of metastable structure (Barden et al., 1969, 1973).
2. A compacted and metastable soil structure is supported by microforces of shear strength that are highly dependent upon capillary action. The bonds start losing strength with an increase in the water content and at a critical degree of saturation the soil structure collapses (Jennings and Knight, 1957; Barden et al., 1973).
3. There is a gradual increase in compressibility as well as a gradual decrease in shear strength of a collapsible soil during the saturation process (Jennings and Burland, 1962; Barden et al., 1973).
4. The soil collapse progresses as the degree of saturation increases. There is a critical degree of saturation for a given soil above which negligible collapse will occur due to the magnitude of the prewetting overburden pressure (Jennings and Burland, 1962; Houston et al., 1993).
5. For a given mean normal total stress, the magnitude of axial collapse increases and the magnitude of radial collapse decreases with an increasing stress ratio (Lawton et al., 1991).

Matyas and Radhakrishna (1968) carried out test on two series of a collapsible soil. Their results showed that the compressibility of the soil is a function of matric suction. The soil structure was less compressible at high matric suctions. The collapse behaviour of compacted gneiss soil has been investigated by Pereira and Fredlund (2000). Their findings showed that the wetting-induced soil collapse caused two different and opposite effects. The first effect is the reduction of the void ratio with the accompanying increase in degree of saturation of the collapse

material. The second effect is the reduction in the water flow into the soil structure due to the increased amount of trapped air within the macrostructure of the soil.

Theories have been established in recent years (Fredlund and Morgenstern 1976, 1977; Alonso et al., 1985; Fredlund and Rahardjo, 1993a) that are consistent with a multiphase, continuum mechanics approach and describe the mechanical behaviours of an unsaturated soil as a function of two independent stress state variables. The volume change behaviour of unsaturated soils is a function of the stress state variables; namely, net normal stress ($\sigma - u_a$) and matric suction ($u_a - u_w$) where σ is the total normal stress, u_a is the pore-air pressure and u_w is the pore-water pressure. Therefore, the void ratio, e , must be related to $(\sigma - u_a)$ and $(u_a - u_w)$ (Ho et al., 1992).

The volume change of an unsaturated soil can be converted into prediction of swell or shrinkage by substituting the change in the stress state variables of an unsaturated soil. Mathematical as well as semi-empirical methods (Fredlund and Morgenstern, 1977) can be used for formulations, but the latter is more commonly used in conventional soil mechanics and will be used herein. In general, the change in volumetric strain, $d\varepsilon_v$, can be expressed as (Fredlund and Rahardjo, 1993a):

$$d\varepsilon_v = 3\left(\frac{1-2\mu}{E}\right)d(\sigma_{mean} - u_a) + \frac{3}{H}d(u_a - u_w) \quad (2.6)$$

$$d\varepsilon_v = \frac{dV_v}{V} \quad (2.7)$$

where:

- V_0 = initial overall volume of an unsaturated soil element
 V_v = volume of soil voids
 E = modulus of elasticity or Young's modulus for the soil structure
 μ = Poisson's ratio
 σ_{mean} = average total normal stress $[(\sigma_x + \sigma_y + \sigma_z)/3]$
 H = modulus of elasticity for the soil structure with respect to a change in matric suction

Other means of expressing the constitutive equation are as follow:

$$de = a_t d(\sigma_{\text{mean}} - u_a) + a_m d(u_a - u_w) \quad (2.8)$$

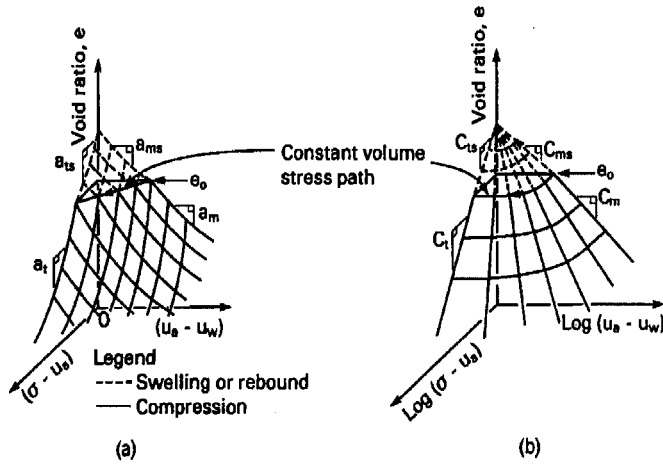
$$dw = b_t d(\sigma_{\text{mean}} - u_a) + b_m d(u_a - u_w) \quad (2.9)$$

where:

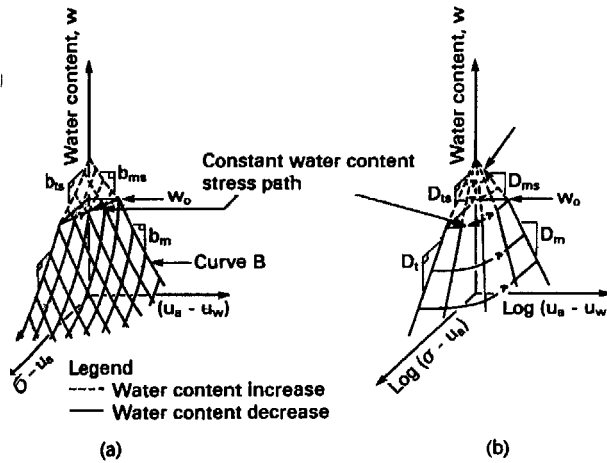
- a_t = coefficient of compressibility with respect to a change in net normal stress, $d(\sigma_{\text{mean}} - u_a)$
 a_m = coefficient of compressibility with respect to a change in matric suction, $d(u_a - u_w)$
 b_t = coefficient of water content change with respect to a change in net normal stress, $d(\sigma_{\text{mean}} - u_a)$
 b_m = coefficient of water content change with respect to a change in matric suction, $d(u_a - u_w)$

Experimental data are needed to verify the uniqueness of the constitutive equations when using the semi-empirical method. Experimental results are typically shown in a three-dimensional graph with the volumetric parameter plotted against net normal stress and matric suction (Fredlund and Rahardjo,

1993a). A unique constitutive surface is usually generated from such a plot. Examples of such graphs are shown in Figure 2.16.



(a) Void ratio constitutive surface for an unsaturated soil



(b) Water content constitutive surface for an unsaturated soil

Figure 2.16 Constitutive surfaces for an unsaturated soil
 (from Fredlund and Rahardjo, 1993a)

The constitutive surfaces are described by volume change indices, which can be obtained through experiments. The extremes of the surface are obtained by applying pure net normal stress and pure matric suction, respectively. The former is obtained when matric suction is zero, implying complete saturation of the soil. Hence the relationship between the void ratio and the water content plot can be easily related using $Se=wG_s$, where $S=1$ for full saturation. Curve B (Figure 2.16b), defines the other extreme of the constitutive surface, also known as the soil-water characteristic curve (SWCC). The two extreme curves can be used to estimate unsaturated soil properties. The indices of the curves can be defined as follows:

Coefficient of compressibility:

$$\begin{aligned}
 a_t &= \frac{\partial e}{\partial(\sigma - u_a)} & a_m &= \frac{\partial e}{\partial(u_a - u_w)} \\
 b_t &= \frac{\partial w}{\partial(\sigma - u_a)} & b_m &= \frac{\partial w}{\partial(u_a - u_w)}
 \end{aligned}
 \tag{2.10}$$

where:

- a_t = coefficient of compressibility with respect to a change in net net normal stress, $d(\sigma_{\text{mean}} - u_a)$
- a_m = coefficient of compressibility with respect to a change in matric suction, $d(u_a - u_w)$
- b_t = coefficient of water content change with respect to a change in net normal stress, $d(\sigma_{\text{mean}} - u_a)$
- b_m = coefficient of water content change with respect to a change in matric suction, $d(u_a - u_w)$

Compressive indices:

$$C_t = \frac{\partial e}{\partial \{\log(\sigma - u_a)\}} \quad C_m = \frac{\partial e}{\partial \{\log(u_a - u_w)\}} \quad (2.11)$$

$$D_t = \frac{\partial w}{\partial \{\log(\sigma - u_a)\}} \quad D_m = \frac{\partial w}{\partial \{\log(u_a - u_w)\}}$$

where:

- C_t = compressive index with respect to matric suction, $(u_a - u_w)$
- C_m = compressive index with respect to net normal stress, $(\sigma_a - u_a)$
- D_t = water content index with respect to matric suction, $(u_a - u_w)$
- D_m = water content index with respect to net normal stress, $(\sigma_a - u_a)$

The indices (a_t, a_m, C_t, C_m) and (b_t, b_m, D_t, D_m) obtained from experiments can be introduced into Equations 2.10 and 2.11 to determine the change in void ratio and water content, respectively. The general equation, Equation 2.6, can also be modified to describe the stresses imposed by different loading conditions.

For isotropic condition,

$$d\varepsilon_v = 3 \left(\frac{1-2\mu}{E} \right) d(\sigma_3 - u_a) + \frac{(1+\mu)}{H(1-\mu)} d(u_a - u_w) \quad (2.12)$$

For K_0 loading condition,

$$d\varepsilon_v = 3 \left(\frac{1-2\mu}{E} \right) \left(\frac{1+\mu}{1-\mu} \right) d(\sigma_y - u_a) + \frac{(1+\mu)}{H(1-\mu)} d(u_a - u_w) \quad (2.13)$$

Hence the total final volumetric strain of the soil can be obtained by summing each increment,

$$\varepsilon_v = \sum d\varepsilon_v \quad (2.14)$$

Conventional tests and equipments common to a soil mechanics laboratory can be used in the measurement of the volume change coefficients for unsaturated soil (Rahardjo et al., 1990; and Ho et al., 1992). The direct quantification of volume change that occurs when a soil undergoes collapse is usually obtained by conducting the one-dimensional oedometer consolidation test on soil specimens. Oedometer test is commonly preferred because it often yielded fairly good result in predicting the settlement and it is relatively inexpensive. Researches on the volume change that used oedometer test have been done from time to time. William (1975) found that the result obtained from oedometer test with Schmertman correction factors as well as Skempton-Bjerrum correction factors, could successfully predict the overconsolidation settlement for raft and spread foundations. Barksdale et al. (1982) conducted oedometer tests to predict the settlement of a large water tower resting on weathered biotite gneiss.

There are two methods to determine the amount of collapse using the oedometer apparatus (Lutteneger and Saber, 1988): (1) the single-oedometer test (load after wetting), and (2) the double-oedometer test (Jennings and Knight, 1957). Figure 2.17 shows plots of the single-oedometer test and double-oedometer collapse tests. The main source of concern regarding the single and double-oedometer tests would be the magnitude of applied pressure when free access water is available. In the single-oedometer test, the free access water is available only after the equilibrium of the required load is reached while in the double-oedometer test, the free access water is introduced after the equilibrium of the seat load (usually 5 kPa).

The single-oedometer test is the main method for evaluating the collapse susceptibility of the test soil and is described in ASTM D5333-92 (1998a). It is a single-specimen procedure, closely simulating actual field conditions. The single-oedometer test consists of incrementally loading a compacted specimen to the desired vertical stress, waiting for the vertical deformation to cease and then inundating the specimen with water. The deformation induced by the addition of water, divided by the initial height of the specimen defines collapse potential, CP. The change in volume will indicate either swell (increase) or collapse (decrease) of the soil due to wetting under a constant load.

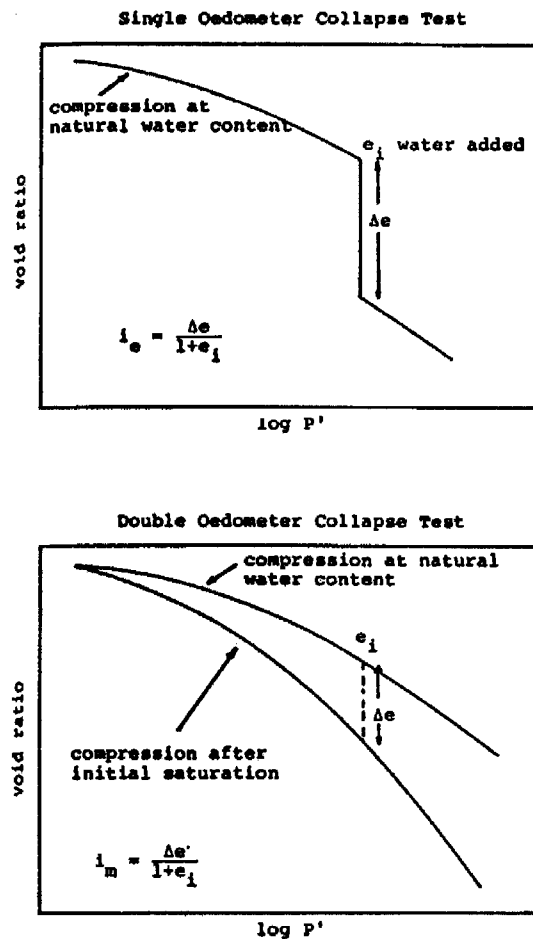


Figure 2.17 Single and double oedometer collapse tests
(from Lutenegeger and Saber, 1988)

Basma and Tuncer (1992) used single-oedometer tests on eight different soils to investigate the effect of soil type, compaction water content and applied pressure at wetting on collapse potential. Their results indicated that well-graded soils tend to collapse more than poorly graded ones under similar conditions. The most important factors affecting magnitude of collapse were initial moisture content, dry unit weight, stress at wetting, uniformity coefficient and, sand and clay size fraction. It was found that collapse potential decreases with an increase in: (1) difference between the sand and clay percentages, (2) compaction water content, and (3) initial dry unit weight. Lim and Miller (2004) conducted single-oedometer tests on compacted Oklahoma soils at 95% relative compaction and four different moisture contents: 4% and 2% below the OMC (optimum moisture content), OMC and 2% above the OMC. The results showed that the collapse index varies from 0 to 7.1% which encompass negligible to moderately severe collapse according to the collapse index criteria in ASTM D5333-92 (ASTM, 1998a).

Jennings and Knight (1957) used the double-oedometer test to predict the collapse potential of a soil. This method can be used to evaluate the response of a soil being wetted and loaded at different stress levels. The method requires testing two nominally identical samples. One sample was tested at the as-compacted moisture content while the other sample was initially inundated with water under a small seating load and allowed to swell until primary swelling was complete. The first sample is loaded in the as-compacted state by incremental vertical loads with the equilibrium deformation measured for each load level. The second specimen is first inundated and a similar loading procedure is carried out. The difference between the equilibrium deformations, at each stress level obtained by the two samples is used to define swell or collapse. The use of the double-oedometer test is beneficial as it requires fewer specimens and is less time consuming compared to the single-oedometer test.

Lawton et al. (1989) conducted oedometer tests to clarify the influences of compaction method, compaction water content, relative compaction, vertical stress level, and load-wetting sequence on post-compaction wetting induced volume changes in a moderately plastic clayey sand. The test program consisted mainly of double-oedometer collapse tests performed on an expansive soil taken from a deep fill in Southern California. The soil was compacted directly into oedometer rings using impact, kneading, or static method. The test results showed that at the stress levels where wetting induced volume changes were observed, the amount of swelling or collapse varied inversely with the compacted water content. At a given dry density, the overburden stress level at which the maximum amount of collapse took place also varied inversely with the compaction water content. Both swelling and collapse were reduced by compacting the soil at water contents on the wet side of the line of optimums for impact compaction. Collapse could also be reduced by compacting the soil to high levels of relative compaction. For a given compacted soil, Lawton et al. (1992) found that collapse potential depends on the prewetting moisture condition, dry unit weight of the soil and the total overburden stress. Generally, collapse potential increases with increasing total overburden stress, decreasing dry unit weight and decreasing prewetting moisture content.

Fredlund and Gan (1995) found that collapse potential decreases linearly with increasing initial water content for a constant initial dry unit weight and decreases linearly with increasing initial dry unit weight at a constant water content. Steadman (1987) studied the effect of fines content on the magnitude of collapse in sand-silt clay mixture based on double-oedometer test conducted on specimens prepared at 85% and 90% relative compaction. From their study, it was concluded that the magnitude of collapse increases as the fines fraction increases. Alwail et al. (1994) also found that collapse potential increases with the increase of clay size fraction and clay to silt ratio. Although both single-oedometer and double-oedometer tests have different sequence of loading and wetting of the

specimen, researchers found that the two methods generally agree in the collapse region (Booth 1977; Justo et al., 1984; Lawton et al., 1989; Miller et al., 1997).

More recently, the influence of suction on the stress-strain behaviour of unsaturated soil has been examined (Alonso et al., 1985; Cui and Delage, 1996; Delange et al., 1987; Wheeler and Sivakumar, 1993). It is known that suction has a significant influence on soil behaviour in terms of volume changes. Several researches have been performed using suction-controlled oedometer devices. Al Mukhtar et al. (1999) have conducted oedometer test using a relative humidity (suction) control oedometer device. They found that the oedometer test result under constant relative humidity showed that increasing axial stress leads to collapse of the soil structure, increases the degree of saturation and decrease the suction. Rampino et al. (2000) performed tests on compacted silty sand under suction-controlled condition using a stress-path triaxial apparatus and oedometer apparatus. The tests were carried out under suctions ranging from 0 to 400 kPa. The test results indicate that suction has a strong influence on compressibility, stiffness and shear strength. Furthermore, they found that the soil exhibited collapse behaviour upon wetting at low stress level. This indicates that suction has great influence on the mechanical response of the soil. Lloret et al. (2003) studied the mechanical behaviour of compacted bentonite in suction-controlled oedometers at high suctions and large vertical loads. The test results showed significant features of yield phenomena, dependence of swelling strains and compound stress paths in swelling pressure tests.

Generally, stress-strain relationship or void ratio versus logarithm of vertical pressure obtained from oedometer test is highly nonlinear (Figure 2.18). It can be seen in Figure 2.18 that the plot has two approximately straight line portions connected by a smooth transitional curve. The stress at which the transition occurs is an indication of the maximum vertical overburden pressure that the soil has sustained in the past and is known as the preconsolidation pressure, P_c' . For compacted soils, P_c' is the apparent preconsolidation pressure as

it is different from that of saturated clays or in-situ soils. There are several factors that can influence the measured P_c' value: sample disturbance, test equipment and procedure used to obtain the compression curve, environmental factors and the interpretative techniques used to estimate P_c' (Sridharan et al., 1991). Different techniques can be used to determine the preconsolidation pressure. The most popular is Casagrande's method, where P_c' is obtained from a construction at the point of the maximum curvature on the curve of e -log P . The accuracy of the determination of the preconsolidation pressure by Casagrande's method depends on the judgement of the point of maximum curvature.

The gradient line after P_c' represents the compression index, C_c , of the compression curve. The compression index, C_c , is used to calculate how much consolidation settlement will take place under a change of vertical load. The compression index, C_c , of unsaturated soils so far has generally been correlated with the liquid limit and other index properties which reflect the soil condition. Compression index, C_c , is a mechanical index of the soil; therefore it should represent the type, condition and mechanical structure of the soil.

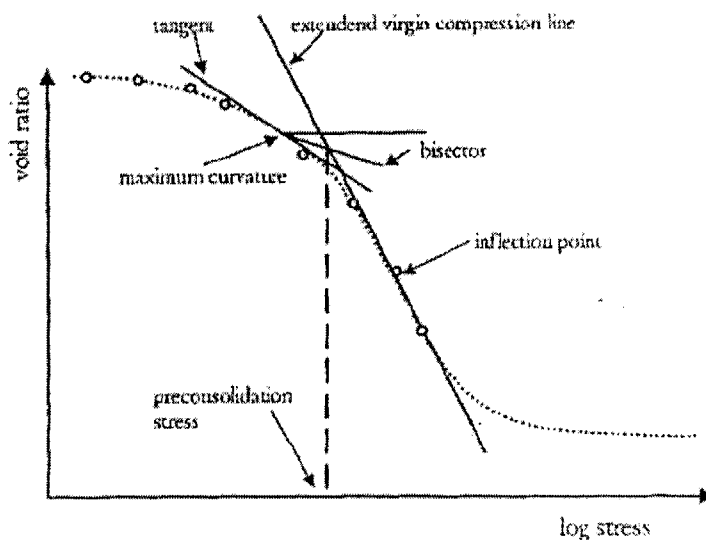


Figure 2.18 Determination of preconsolidation pressure according to the method of Casagrande (from Baumgartl and Kock, 2004)

2.5 SUMMARY

Total suction is the sum of matric suction and osmotic suction. Total suction, matric suction and osmotic suction can be independently measured in the laboratory using different techniques. Several studies on suction measurements have been done and it was found that the sum of matric and osmotic suctions may not be the same as the measured total suction. Total suction is determined by measuring the relative humidity in the soil sample. Matric suction is determined by measurement of the pore-water pressure while osmotic suctions measurement is conducted on the extracted pore water. A number of methods are available for the extraction of the pore water from soil samples: dilution or saturation extract, leaching, centrifuging, immiscible liquid displacement, gas extraction method and pressurised squeezing (low pressure and high pressure mechanical squeezing). Among these methods, pressurised squeezing technique was found to be suitable for a number of soil types, very suitable for kaolinite samples and for dry and stiff soils. Furthermore, pressurised squeezing technique is easy to use, the equipment can be cheaply fabricated, is direct and has been proven for osmotic suction measurement of extracted pore water from unsaturated soils. In this research, pressurised squeezing technique will be used to extract the pore water from the soil.

Volume changes in unsaturated soils are important because of their consequences in terms of settlement due to wetting. There are many factors contributing to volume change and the magnitude of volume change: soil type and structure, initial soil density, imposed stress state, and degree of wetting. The main laboratory apparatus for studying collapsible and expansive soils is the oedometer. For collapsible soils, single and double-oedometer tests are accepted by majority of engineering geologists and geotechnical engineers. Only the single-oedometer test is described in ASTM. In this research, both single and double-oedometer tests will be conducted to study the volume change behaviour of compacted soils. The oedometer apparatus used are not equipped with suction

application or suction measurement devices as a different approach will be used to evaluate wetting-induced settlement.

CHAPTER 3

MATERIALS AND TEST PROCEDURES

3.1 INTRODUCTION

This chapter describes the apparatus and the test procedures used to determine the soil suction components (total suction, matric suction and osmotic suction) as well as the volume change behaviour of compacted soils. Test procedures are done in accordance to ASTM standards where applicable. Figure 3.1 shows the tests performed in this study. Basic properties tests such as liquid limit, plastic limit, specific gravity, grain size distribution, hydrometer tests and compaction test with standard Proctor effort were done in this study, followed by suction measurement tests and single and double oedometer tests.

3.2 SOIL MATERIAL AND SAMPLE PREPARATION

Three soil samples, commercial kaolin and residual soils A and B, were used in this study. The kaolin sample used is refined kaolin made by Malaysia SDN BHD. The residual soils A and B were collected from a construction site at Nanyang Technological University campus. Residual soils A and B are from different strata of the sedimentary Jurong Formation (PWD, 1976).

The residual soil samples were air dried and sieved through No. 4 sieve as recommended by ASTM D422 (1998b) prior to test. Only soils passing the No. 4 sieve were used. Before sieving, the soil sample was disaggregated and mixed thoroughly to achieve a homogeneous state. Basic properties tests were performed to characterise the kaolin and residual soil samples and the results are summarised in Table 3.1. Liquid limit and plastic limit tests were obtained using the fall cone and plastic limit test, respectively. These tests were carried out in accordance to ASTM D4318 (1998c).

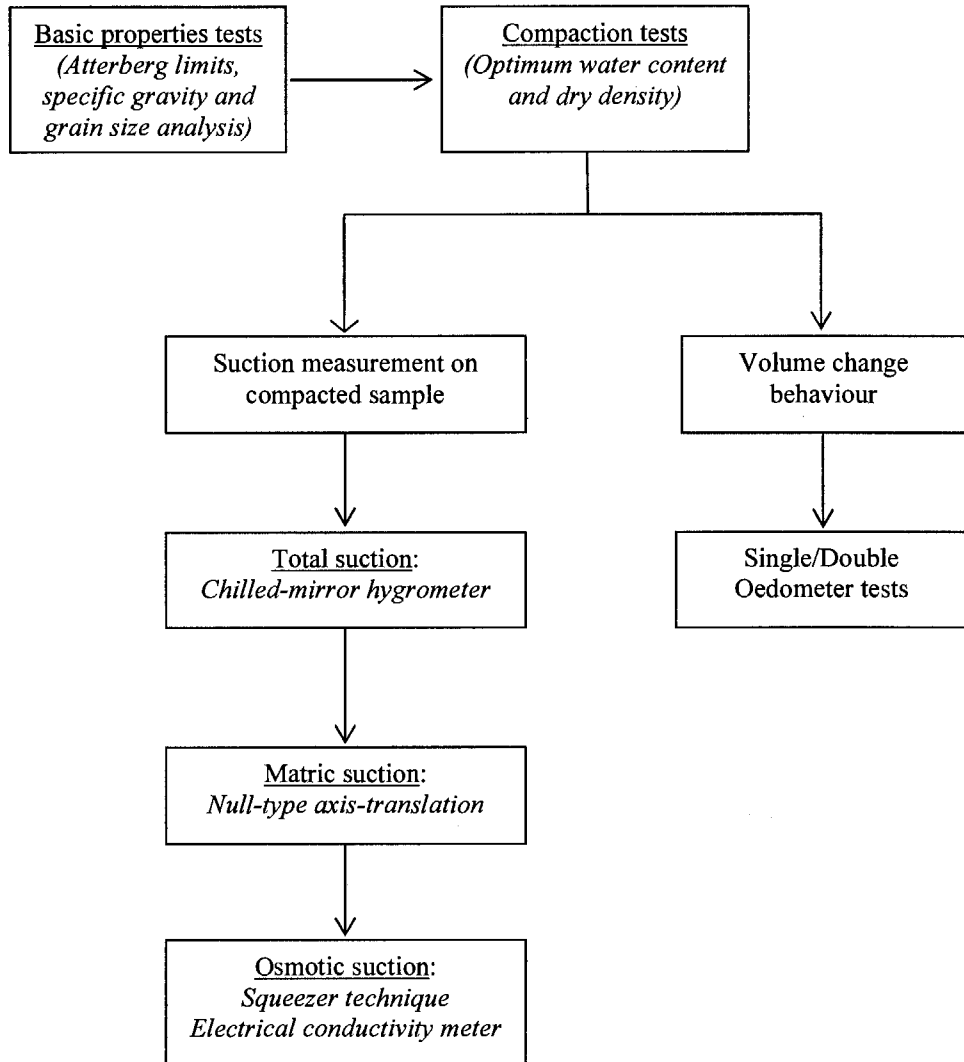


Figure 3.1 Flow chart showing experiments

As residual soil consists of both fine and coarse particles, mechanical sieving and hydrometer tests were carried out in accordance to ASTM D422 (1998b). As kaolin is a fine-grained soil, the hydrometer test was conducted to determine its grain size distribution. The grain size distribution curves of kaolin and residual soils are shown in Figure 3.2. Specific gravity of the kaolin and

residual soils was determined using the pycnometer in accordance with ASTM D854 (1998d). From Table 3.1, it may be observed that kaolin is a silt of low plasticity (MH) while both residual soils A and B are clayey sand (SC). From Figure 3.2, residual soil A is sandier than residual soil B and residual soil A is gap graded whereas residual soil B is well graded.

Table 3.1. Summary of soil properties used

| Soil Properties | Kaolin | Residual Soil | |
|-------------------------|--------|---------------|--------|
| | | Soil A | Soil B |
| Liquid limit | 61 | 29 | 35 |
| Plastic limit | 41 | 20 | 22 |
| Plasticity index | 17 | 9 | 13 |
| Specific gravity | 2.6 | 2.62 | 2.71 |
| Grain size distribution | - | 60% | 50% |
| | 92% | 20% | 39% |
| | 8% | 20% | 11% |
| USCS | MH | SC | SC |

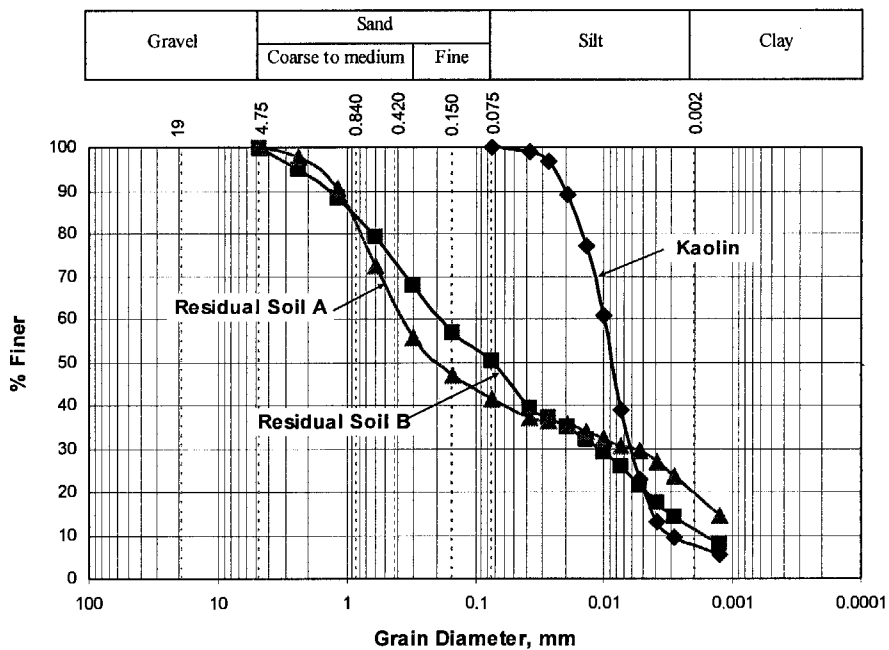
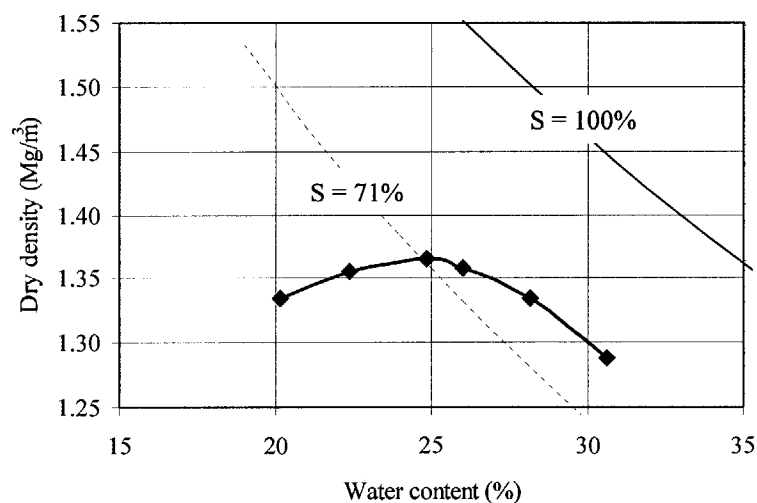


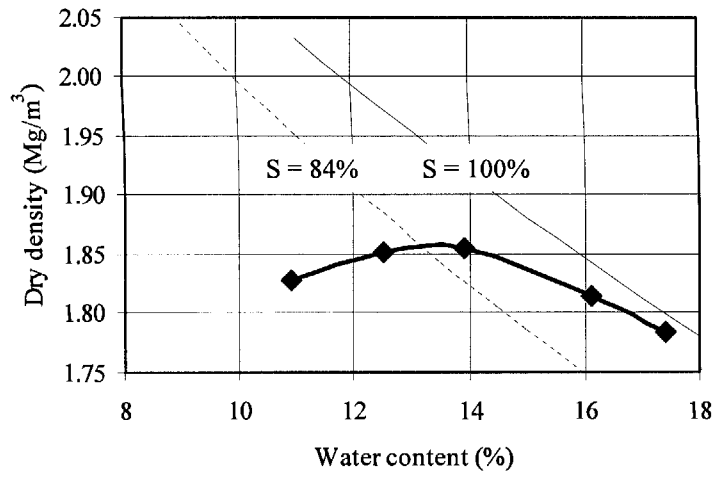
Figure 3.2 Grain size distributions of soil samples

For the suction measurement study, soil samples with different matric and osmotic suctions were required. A robust sample preparation method is to compact the soil mixed with water or salt solutions at various water contents. Different sample density would induce different matric suction in the sample while the different salt solution would induce different osmotic suction.

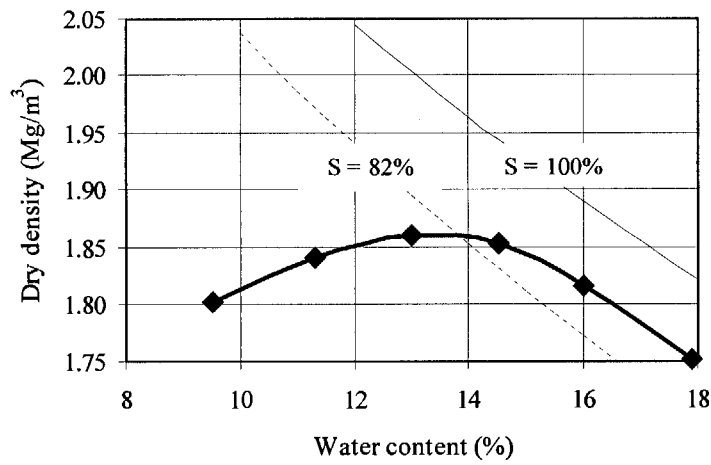
Before soil the samples were prepared, the compaction characteristics of the kaolin and the two residual soils were determined. Standard Proctor (ASTM D698, 1998e) tests were carried out in this study. The standard Proctor compaction curves of the kaolin and residual soils with various saturation lines are shown in Figures 3.3 (a), (b) and (c), respectively. The maximum dry density and the optimum water content of the kaolin under standard Proctor compaction effort are 1.37 Mg/m^3 and 24.5%, respectively. For the residual soil A, the maximum dry density and the optimum water content are 1.86 Mg/m^3 and 13.7% whereas for residual soil B, the maximum dry density and water content are 1.86 Mg/m^3 and 13.5%.



(a) Kaolin



(b) Residual Soil A



(c) Residual soil B

Figure 3.3 Compaction properties of soil samples

3.3 SUCTION MEASUREMENTS

This section briefly describes the devices used for suction measurements in this study. The devices described are a chilled-mirror hygrometer for total suction measurement, null-type axis translation apparatus for matric suction measurement and squeezing technique for osmotic suction measurement. The soil sample was mixed with tap water or salt solution of different sodium chloride concentration. The purpose of mixing the soil samples with two different fluids is to vary the osmotic suction of the soil samples. The concentration of dissolved salts in the pore-water which is related to the osmotic suction of the soil can be estimated from the electrical-conductivity measurement of the pore-water from the soil. The soil sample mixed with tap water has low electrical conductivity compared to the soil sample mixed with salt solution. As the concentration increases, the electrical conductivity value increases thus there will be variation of the osmotic suction. The sodium chloride concentrations of the salt solutions used for kaolin samples: 5.8 g/l, 14.0 g/l, 20.5 g/l and 32.0 g/l while for both residual soil A and B were 7.6 g/l, 16.0 g/l, 23.5 g/l and 32.5 g/l. The kaolin and residual soil samples were compacted at different water contents. The soil sample after compaction was then divided for total suction, matric suction, and osmotic suction measurements.

3.3.1 Total Suction

Total suction is obtained by measuring the relative humidity of the soil sample. Here, the relative humidity is measured using a chilled-mirror hygrometer (Aqualab series 3E). The schematic of the device is shown in Figure 3.4.

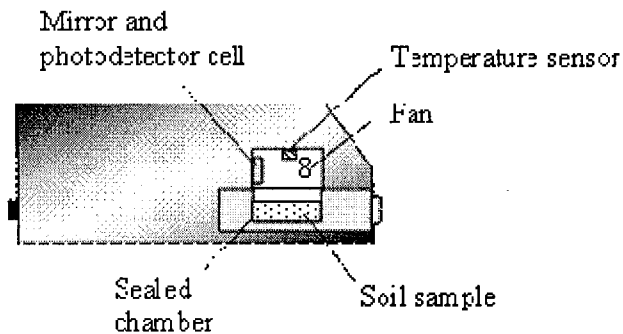


Figure 3.4 Schematic for chilled-mirror dew-point device
(from Leong et al., 2003a)

The device uses the chilled-mirror dewpoint technique to measure the relative humidity of a soil sample within a few minutes at a prefixed temperature. The reading is given in terms of water activity, A_w , in the range from 0.100 to 1.000. Water activity is equivalent to relative humidity. The chilled-mirror dew-point device has a relative humidity measurement range of 3 to 100% with a resolution of $\pm 0.1\%$ and an accuracy of $\pm 0.3\%$. In order to obtain accurate reading, the device should be allowed to warm-up for at least 15 minutes before use. This allows the air inside the device to equilibrate with the temperature of its surroundings. The device is calibrated using a standard solution (sodium chloride 6.0 molal) before the test is conducted.

For measurement, a prepared soil sample is put in a disposable sample cup (4.91 cm in diameter and 0.83 cm high) and is placed into the drawer in the sealed chamber of the device. The soil sample fills half the cup as overfilling will contaminate the sensor in the chamber. The sample is equilibrated with the headspace of the sealed chamber that contains a mirror and a means of detecting condensation on the mirror. The mirror temperature is precisely maintained by a thermopile (Peltier) cooling device. Detection of the exact point at which condensation first appears on the mirror is observed using a photoelectric cell and is shown by a beam of light directed onto the mirror and reflected into a

photodetector cell. The photodetector senses the change in reflectance when condensation occurs in the mirror. A thermocouple is used to record the temperature at which condensation occurs and is used to determine the vapour pressure of the overlying headspace. Relative humidity measured with the device has an accuracy of $\pm 0.3\%$. The relative humidity of the air in the chamber will be the same as the relative humidity in the soil at equilibrium condition. The measurement takes approximately 2 to 3 minutes to complete regardless of the relative humidity.

Total suction is determined using the thermodynamic relationship between total suction and soil gas relative humidity, R_h (Hillel 1980, Rawlins and Campbell 1986, Fredlund and Rahardjo 1993a) as follows:

$$\psi = -\frac{RT}{M_v} \ln\left(\frac{\bar{u}_v}{\bar{u}_{v0}}\right) = -\frac{RT}{v_{w0}\omega_v} \ln(R_h) \quad (3.1)$$

where:

- ψ = soil suction or total suction (kPa)
- R = universal (molar) gas constant [i.e., 8.31432 J/ (mol K)]
- T = absolute temperature [i.e., $T = (273.16 + t^\circ)$ (K)]
- t° = temperature ($^\circ\text{C}$)
- v_{w0} = specific volume of water or the inverse of water density (i.e., $1/\rho_w$) (m^3/kg)
- ρ_w = water density (i.e., 998 kg/m^3 at $= 20^\circ\text{C}$)
- ω_v = molecular mass of water vapour (i.e., 18.016 kg/kmol)
- \bar{u}_v = partial pressure of pore-water vapor (kPa)
- \bar{u}_{v0} = saturation pressure of pore-water vapour over a flat at the same temperature (kPa)

The calibration of the device was verified using standard solutions such as: magnesium nitrate ($\text{Mg}(\text{NO}_3)_2 \cdot 6\text{H}_2\text{O}$), sodium chloride (NaCl), potassium chloride (KCl), potassium nitrate (KNO_3) and potassium sulphate (K_2SO_4). Procedure for preparing the salt solutions are given in ASTM D5298 (1998f) and ASTM E104 (1998g). Table 3.2 shows the relative humidity of the salt solutions measured at a temperature of 25°C and the relative humidity of the saturated aqueous solutions given in ASTM E104 (1998g). The relative humidity readings from the device were similar to the values given in ASTM E104 (1998h) and also those reported by Leong et al. (2003a). Figure 3.5 shows the plots of the relative humidity against time for various salt solutions and saturated aqueous solutions. Calibration was also performed using various molal solutions of sodium chloride for relative humidities in the range of 93% to 99.3%. The relative humidity values were then calculated using Equation 2 and compare to the value obtained from Lang's equation. The suction values obtained from various molal solutions of sodium chloride are given in Table 3.3.

Table 3.2 Relative humidity from ASTM E104 (1998g) and measured relative humidity of various solutions at 25°C

| Solutions | Relative humidity from ASTM E104 (1998g): % | Relative humidity from Leong et al. (2003a): % | Measured relative humidity: % |
|--|---|--|-------------------------------|
| Magnesium nitrate ($\text{Mg}(\text{NO}_3)_2 \cdot 6\text{H}_2\text{O}$) | 52.9 ± 0.2 | 53.0 | 52.9 |
| Sodium chloride (NaCl) | 75.3 ± 0.1 | 75.3 | 75.2 |
| Potassium chloride (KCl) | 84.3 ± 0.3 | 84.4 | 84.5 |
| Potassium nitrate (KNO_3) | 93.6 ± 0.6 | 93.9 | 93.8 |
| Potassium sulphate (K_2SO_4) | 97.3 ± 0.5 | 97.7 | 97.7 |
| Pure distilled water | 100 | 99.9 | 99.9 |

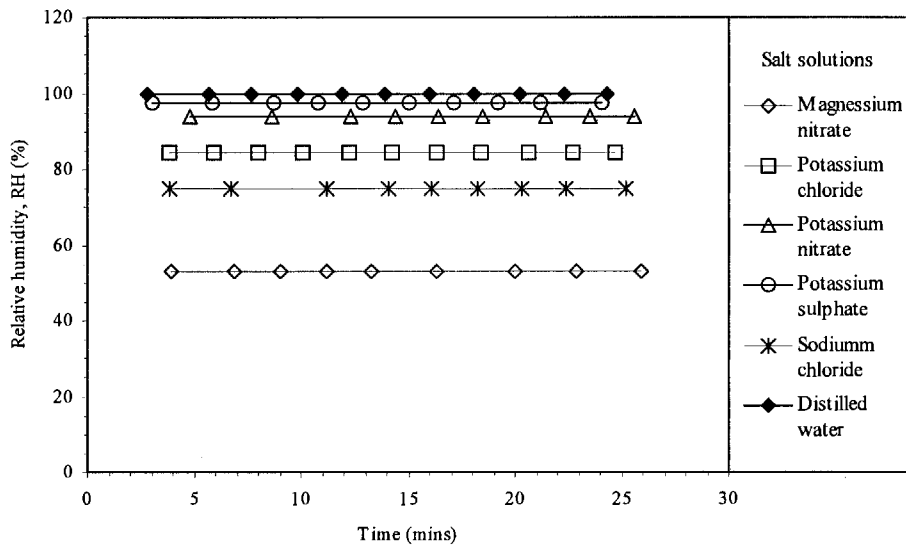


Figure 3.5 meter to various salt solutions

Table 3.3 Total suction of the sodium chloride solution at 25°C

| Molality | Salt content (g / l) | R _h | ES-12 (mS/cm) | Total suction (kPa) | | |
|----------|----------------------|----------------|---------------|---------------------|-------------|-------------|
| | | | | Eq. 2.2 | USDA (1960) | Lang (1967) |
| 0.2 | 1.17 | 0.993 | 19.2 | 967 | 927 | 915 |
| 0.5 | 2.93 | 0.984 | 43.9 | 2219 | 2400 | 2281 |
| 0.7 | 4.09 | 0.976 | 58.4 | 3343 | 3332 | 3210 |
| 1.0 | 5.85 | 0.967 | 78.7 | 4617 | 4696 | 4640 |
| 1.5 | 8.78 | 0.951 | 108.7 | 6913 | 6808 | 7134 |
| 1.7 | 9.95 | 0.946 | 119.9 | 7638 | 7621 | 8170 |
| 1.8 | 10.53 | 0.943 | 125.1 | 8076 | 8002 | 8700 |
| 1.9 | 11.12 | 0.941 | 129.2 | 8368 | 8305 | 9240 |
| 2.0 | 11.70 | 0.938 | 133.7 | 8807 | 8638 | 9780 |

3.3.2 Matric Suction

In this study, matric suction is measured using the null-type axis-translation apparatus as shown in Figure 3.6. The schematic drawing of the null-type axis-translation apparatus is shown in Figure 3.7. This method was first developed by Hilf (1956). The procedure involved a translation of the reference pore-air pressure. Hilf demonstrated that when the air pressure in the pressure chamber was increased, the pore-water pressure increased accordingly by the same amount. In other words, the matric suction of the soil remained constant although the specimen was measured at different ambient air pressure.

A soil specimen, 63 cm in diameter and 18 cm high, was cut from the compacted soil sample for matric suction measurement. The soil sample was put in the closed pressure chamber and a 1 kg metallic weight was put on top of the soil specimen. The surface of the soil specimen must be flat enough when it is in contact with the ceramic disk. A pore-water pressure transducer was connected to the water chamber of the pressure chamber through a tube filled with de-aired water. Once the soil specimen was placed, the system would start to register a negative pore-water pressure. The air pressure in the pressure chamber was raised incrementally. When equilibrium was reached, the air pressure and water pressure remain stationary. The difference between the air pressure and the water pressure in the chamber at equilibrium was taken as the matric suction of the soil specimen.

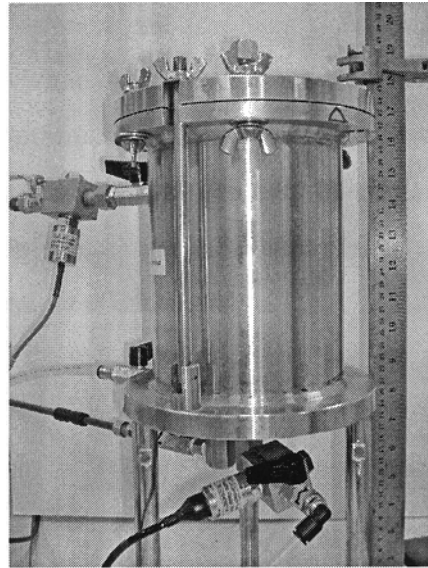


Figure 3.6 Null-type axis translation apparatus

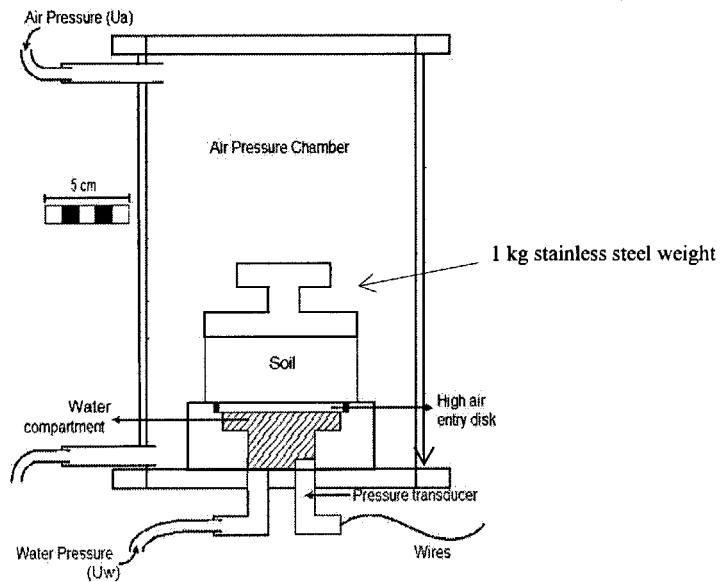


Figure 3.7 Schematic for null-type axis translation apparatus

3.3.3 Osmotic Suction

In this study, the osmotic suction was determined from the electrical conductivity of the pore water extracted from a soil sample and from the compaction water used for compacting the soil. The pore water of the soil sample was extracted using a pore fluid squeezer which consists essentially of a heavy-walled cylinder and piston squeezer as shown in Figure 3.8. The schematic drawing of the pore fluid squeezer is shown in Figure 3.9.

A small amount of soil sample is put into the squeezer and the piston is placed over it. Pressure of up to 30 MPa was exerted to the piston with a 10-tonne compression machine and the pore water was collected in a syringe. Depending on the water content of the soil specimen, the amount of water collected typically could be as little as 3 ml. Due to the small quantity of extracted pore water, the electrical conductivity of the extracted pore water is measured using a compact conductivity meter, Horiba B-173 (Figure 3.12). The Horiba B-173 conductivity meter can measure the electrical conductivity of a single drop of fluid (~ 1 ml). The Horiba B-173 conductivity meter has an automatic range changeover for the three subranges: (1) 0 to 199 $\mu\text{S}/\text{cm}$, (2) 0.20 mS/cm to 1.99 mS/cm, and (3) 2.0 mS/cm to 19.9mS/cm. The accuracy of the Horiba B-173 conductivity meter is $\pm 2\%$ full scale (over 10 mS/cm, the accuracy is $\pm 3\%$ full scale).

Due to the limitation in the measurement range of the Horiba B-173 conductivity meter, the portable conductivity probe, Horiba ES-12, was also used. The Horiba ES-12, as seen in Figure 3.13, can measure the electrical conductivity until 1999 mS/cm with the measurement range of: (1) 0 to 199.9 $\mu\text{S}/\text{cm}$, (2) 0 to 19.99 mS/cm, (3) 19.99 mS/cm to 199.99 mS/cm, and (4) 199.9 mS/cm to 1999 mS/cm. The resolution of the Horiba ES-12 conductivity meter is $\pm 0.5\%$ full scale. The Horiba B-173 electrical conductivity meter was used to measure the electrical conductivity of pore fluid extracted from soil samples mixed with tap water and salt solution of electrical conductivity less than 13 mS/cm, while the

Horiba ES-12 was used to measure electrical conductivity of salt solutions of higher electrical conductivity. The osmotic suction was estimated from electrical conductivity using Figure 2.11.

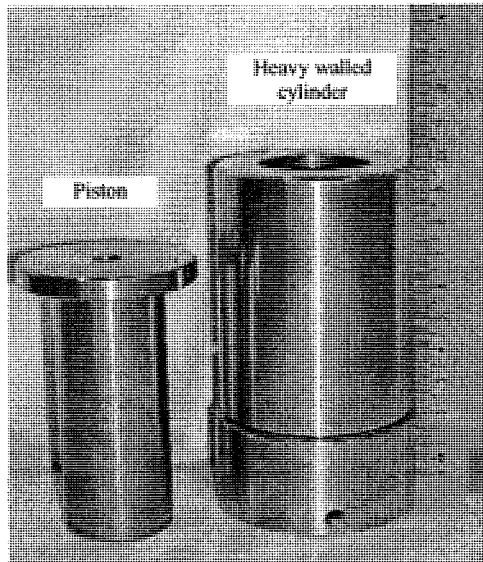


Figure 3.8 Pore fluid squeezer

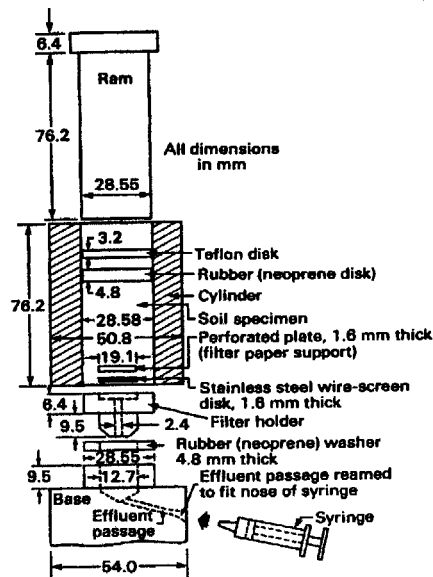


Figure 3.9 Schematic for pore fluid squeezer apparatus
(from Fredlund and Rahardjo, 1993a)

As the reliability of the measurement of osmotic suction measurement is dependent on the conductivity meters, (Horiba B-173, Horiba ES-12) and Figure 2.11, the reliability of the two conductivity meters and Figure 2.11 needs to be assessed. The measured electrical conductivities of the extracted pore water using Horiba B-173 for soil samples compacted at various water contents are compared with the electrical conductivities of the original solution as shown in Table 3.4. Table 3.4 shows that the variation in the electrical conductivities is comparable to the accuracy of the Horiba B-173. For the higher salt concentrations, the electrical conductivity of the soil sample pore water is taken as the electrical conductivity of the salt solution using the Horiba ES-12 conductivity meter as the measurement range of the compact conductivity meter (Horiba B-173) is exceeded at these salt concentrations.

To check the compact conductivity meters, sodium chloride solutions of different molality from 0.2 to 2 molality were prepared. Figure 3.12 shows the comparison of the measured electrical conductivities of the sodium chloride (NaCl) solutions of different concentrations with the relevant plot in USDA (1960). The electrical conductivities of the sodium chloride solution were measured using the Horiba B-173 and ES-12 conductivity meters. The osmotic pressures of the sodium chloride solution can be determined using the chilled mirror dew-point device. The measured osmotic pressures of the sodium chloride solutions are compared with the data given in USDA (1960) and shown in Figure 3.13. For the concentration range of the sodium chloride solutions used in the tests, the osmotic pressure π can be estimated using the following equation:

$$\pi = p_a (0.31EC^{1.15}) \quad (3.1)$$

where:

- p_a = the atmospheric pressure (=101.325 kPa)
- EC = electrical conductivity in mS/cm.

Equation 3 is also plotted in Figure 3.13.

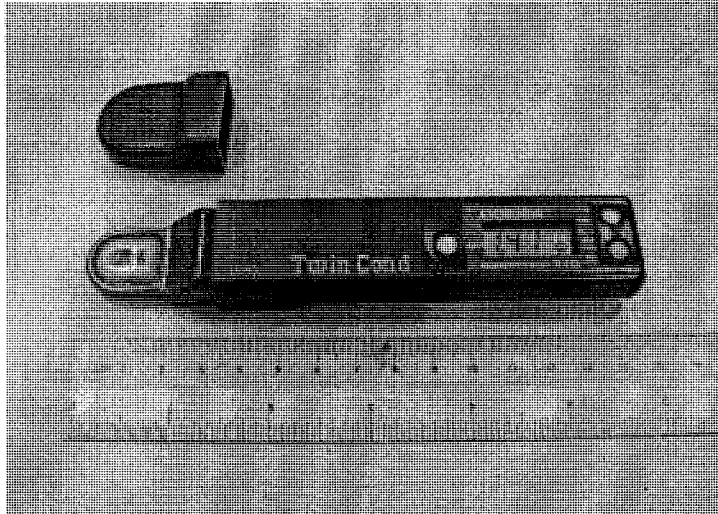


Figure 3.10 Conductivity meter B-173



Figure 3.11 Conductivity meter ES-12

Table 3.4 Comparison of electrical conductivities meter for extracted pore fluid and the original solution

| Kaolin | | | | |
|------------------------|----------------------------------|-------------------|------------------------------------|---------------------------------|
| Solution | Dry density (Mg/m ³) | Water content (%) | EC of extracted pore water (mS/cm) | EC of original solution (mS/cm) |
| Tap water | 1.25 | 20.1 | 0.1 | 0.1* |
| | 1.26 | 20.1 | 0.1 | |
| | 1.27 | 22.9 | 0.2 | |
| | 1.27 | 25.5 | 0.2 | |
| | 1.24 | 28.0 | 0.2 | |
| NaCl 5.8g/l | 1.24 | 20.2 | 8.9 | 9.9* |
| | 1.25 | 22.1 | 9.2 | |
| | 1.26 | 24.4 | 9.5 | |
| | 1.25 | 26.9 | 9.1 | |
| | 1.22 | 28.9 | 9.6 | |
| Residual soil A | | | | |
| Solution | Dry density (Mg/m ³) | Water content (%) | EC of extracted pore water (mS/cm) | EC of original solution (mS/cm) |
| Tap water | 1.91 | 9.4 | 0.2 | 0.1* |
| | 1.93 | 11.3 | 0.4 | |
| | 1.94 | 13.4 | 0.3 | |
| | 1.97 | 15.3 | 0.3 | |
| | 1.87 | 17.9 | 0.4 | |
| NaCl 7.6g/l | 1.91 | 9.4 | 12.0 | 12.8* |
| | 1.92 | 11.4 | 12.8 | |
| | 1.94 | 13.9 | 12.5 | |
| | 1.91 | 15.2 | 12.4 | |
| | 1.87 | 17.4 | 12.6 | |
| Residual soil B | | | | |
| Solution | Dry density (Mg/m ³) | Water content (%) | EC of extracted pore water (mS/cm) | EC of original solution (mS/cm) |
| Tap water | 1.91 | 10.0 | 0.2 | 0.1* |
| | 1.93 | 12.5 | 0.2 | |
| | 1.94 | 13.9 | 0.3 | |
| | 1.93 | 14.8 | 0.1 | |
| NaCl 7.6g/l | 1.90 | 9.2 | 12.2 | 12.8* |
| | 1.93 | 11.7 | 12.5 | |
| | 1.96 | 13.3 | 12.1 | |
| | 1.91 | 15.5 | 12.4 | |

Note: *Average of 3 readings

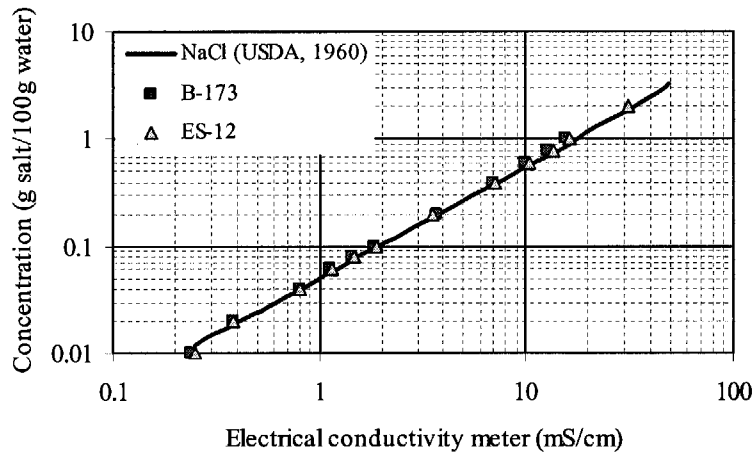


Figure 3.12 Relationship of electrical conductivity and salt concentration for various salts (from USDA, 1960).

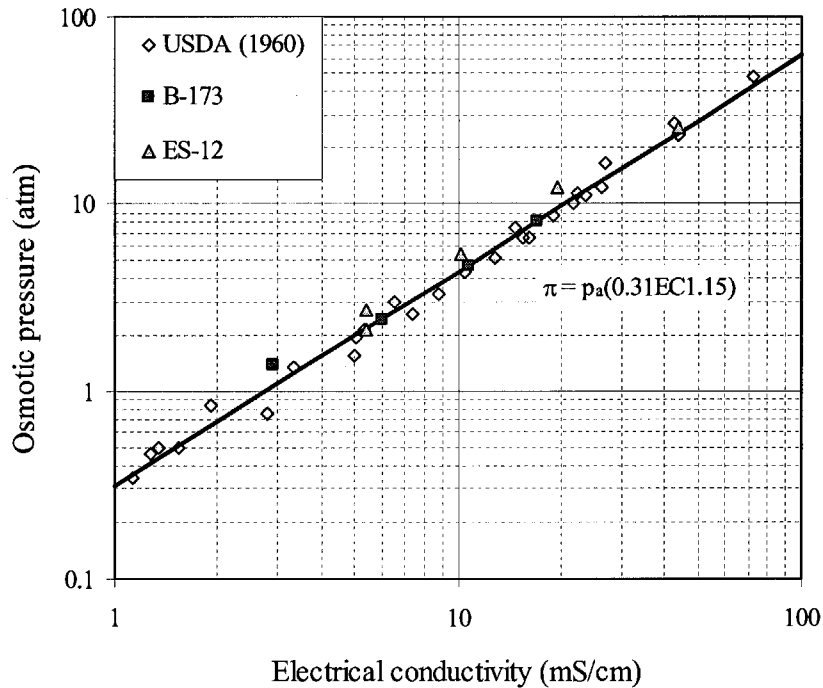


Figure 3.13 Relationship of electrical conductivity with osmotic suction for various salt concentrations (modified from USDA, 1960)

3.4 VOLUME CHANGE

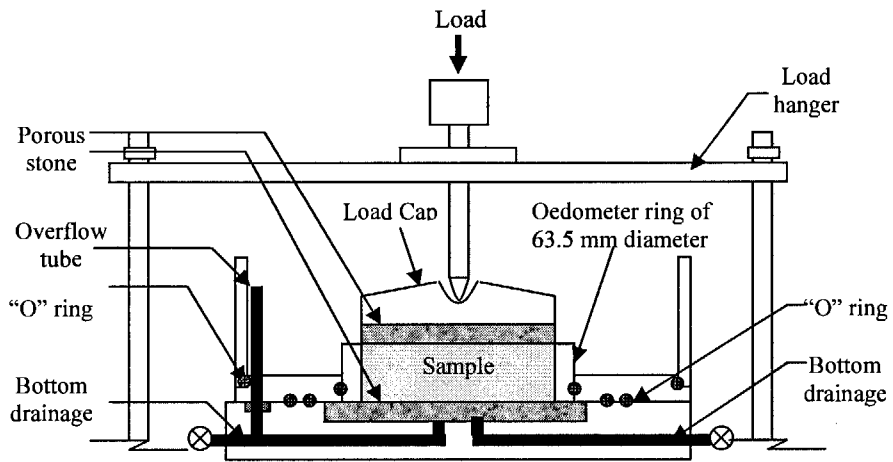
In this study, the volume change behaviour of compacted kaolin and residual soils were investigated by conducting single and double-oedometer tests.

3.4.1 Oedometer Test Apparatus

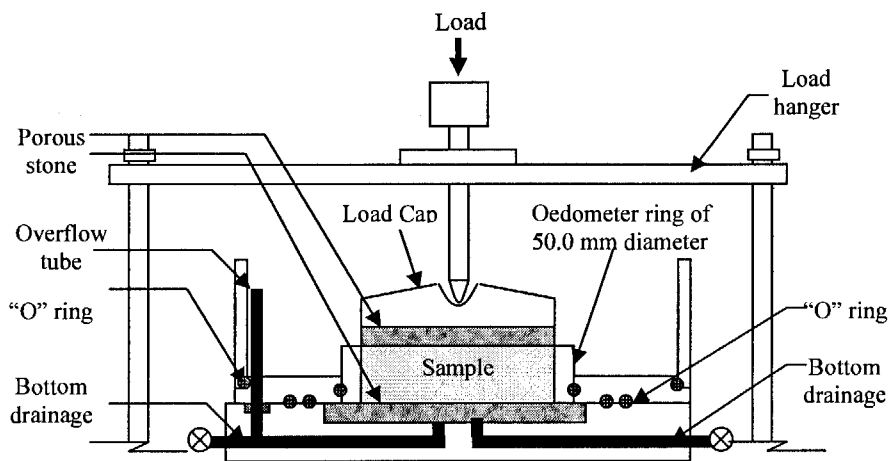
Four oedometer apparatus forming two complete sets of double-oedometer test apparatus (Set 1 and Set 2) were used. The two sets of double-oedometer apparatus have different specimen ring dimensions. The specimen ring dimensions are shown in Table 3.5. According to ASTM D2435 (1998h), “One-Dimensional Consolidation Properties of Soil”, the minimum specimen diameter and diameter to height ratio are 50mm and 2.5, respectively. From Table 3.5, both Set 1 and 2 rings satisfy the above requirements. The schematic representations of both set of double-oedometer test apparatus are shown in Figure 3.14. Each set of oedometer was fitted with a linear variable displacement transducer (LVDT) to monitor the settlement via a data acquisition system consisting of a data logger and a computer. The LVDT readings are recorded by the data acquisition system at intervals of 30 or 60 seconds. A schematic diagram of the apparatus is shown in Figure 3.15. The same oedometer apparatus was also used to conduct the single-oedometer tests.

Table 3.5 Dimensions of specimen rings in double-oedometer test apparatuses

| | Set 1 Oedometer Apparatus | Set 2 Oedometer Apparatus |
|--------------------------|------------------------------|------------------------------|
| Diameter | 63.5 mm | 50.0 mm |
| Height | 19.0 mm | 20.0 mm |
| Diameter to height ratio | 3.4 | 2.5 |



(a) Set 1 oedometer test apparatus with 63.5 mm diameter ring



(b) Set 2 oedometer test apparatus with 50.0 mm diameter ring

Figure 3.14 Set 1 and 2 oedometer test apparatuses

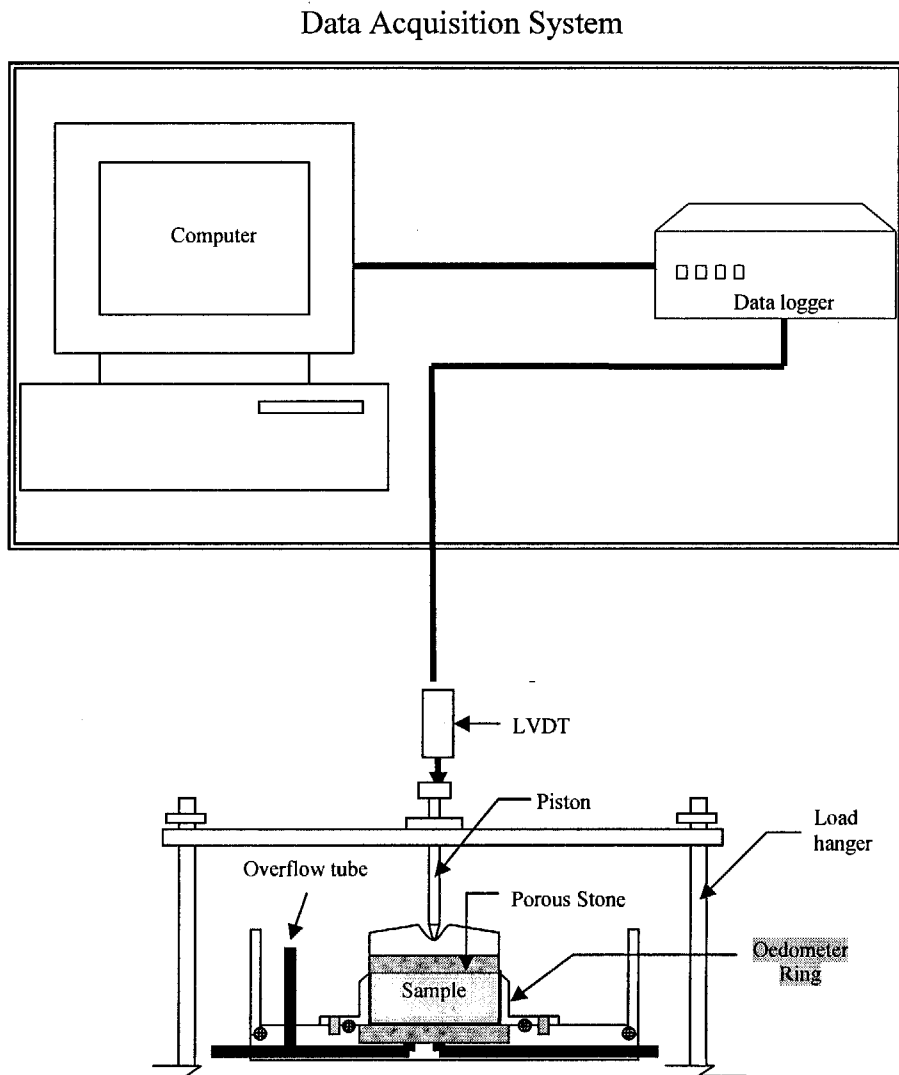


Figure 3.15 Schematic diagram of oedometer equipment with data acquisition system

3.4.2 Procedure of Single-Oedometer Test

Single-oedometer test is also referred as collapse potential test. The procedure described in ASTM D5333 (1998a), “Measurement of Collapse Potential of Soils”, was followed. However, after collapse the loading was continued till the loading reaches 1600 kPa. The more significant steps are summarized below for completeness:

- 1) Mix the dry soil sample at the desired water content.
- 2) Leave the soil sample for 24 hours in a sealed plastic bag to achieve moisture equilibrium.
- 3) Compact the soil according to standard Proctor Test (ASTM D698).
- 4) Extrude the compacted soil and prepare specimen in the oedometer ring.
- 5) Set up the oedometer test.
- 6) After the specimen is placed in the oedometer apparatus, covered it with a wet cloth to prevent evaporation.
- 7) A seating pressure of 5 kPa is applied on the specimen for 5 minutes.
- 8) After 5 minutes, take the deformation reading.
- 9) Take deformation reading at intervals of 30 seconds.
- 10) Leave the specimen for at least 24 hours.
- 11) Increase the loading to the next loading (a load increment ratio of 1 is used).
- 12) Repeat steps 10 and 11 till the required load (200 kPa) and inundate the specimen.
- 13) Take deformation readings at intervals of 30 seconds.
- 14) Leave the specimen for another 24 hours.
- 15) Increase the loading to the next loading (a load increment ratio of 1 is used).
- 16) Repeat steps 14 and 15 till the loading reaches 1600 kPa.

3.4.3 Procedure of Double-Oedometer Test

In this study, the double-oedometer tests were conducted in accordance to the procedure recommended by Jennings and Knight (1957). The equipment was set up with two-way drainage. The procedure is as follows:

- 1) Mix the dry soil sample at the desired water content.
- 2) Leave the soil sample for 24 hours in a sealed plastic bag to achieve moisture equilibrium.
- 3) Compact the soil according to standard Proctor Test (ASTM D698).
- 4) Extrude the compacted soil and prepare two specimens in the oedometer ring.
- 5) Set up the double oedometer tests.
- 6) A seating pressure of 5 kPa is applied on both specimens.
- 7) After 5 minutes, take both deformation readings and inundate one of the specimens with water.
- 8) Take deformation readings at intervals of 30 seconds.
- 9) Leave the specimens for at least 24 hours.
- 10) Increase the loading to the next loading (a load increment ratio of 1 is used).
- 11) Leave the specimen for another 24 hours.
- 12) Repeat Steps 10 and 11 till the loading reaches 1600 kPa.

CHAPTER 4

RESULTS AND DISCUSSIONS

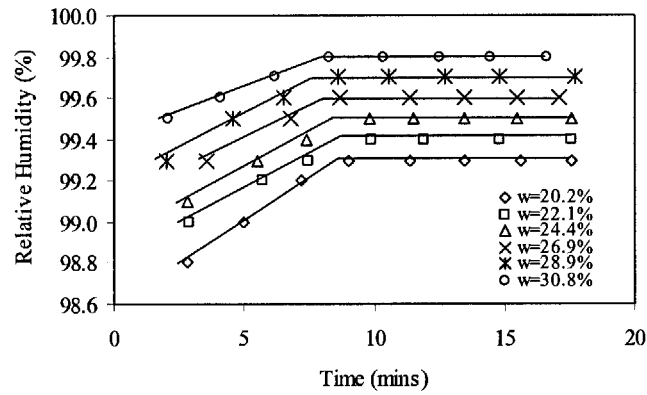
4.1 INTRODUCTION

This chapter present the results of the suction measurements and the oedometer tests on compacted kaolin and two compacted residual soil samples. The basic properties and compaction properties of kaolin and the residual soils were given in Chapter 3.

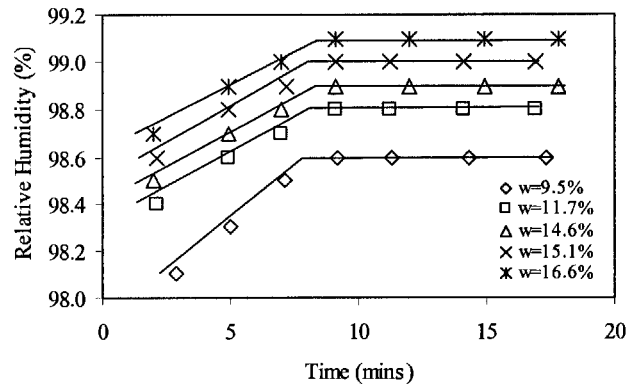
4.2 SUCTION MEASUREMENTS

4.2.1 Total Suction

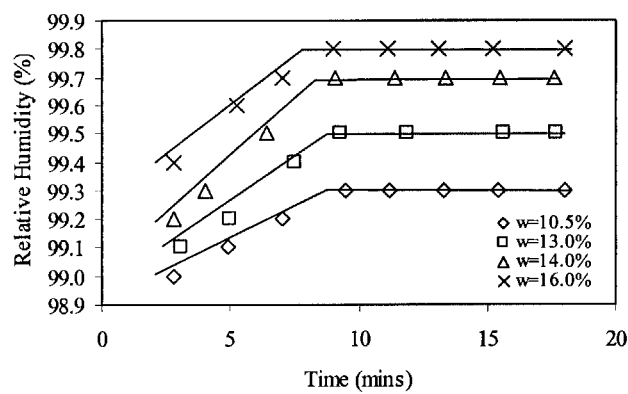
In this study, the total suction of the compacted kaolin and compacted residual soil samples were obtained by measuring the relative humidity of the soil sample using the water activity meter. Typical plots of the relative humidity readings with time at 25°C for the compacted kaolin and compacted residual soil specimens are shown in Figure 4.1. The compaction water contents are indicated on the figure. The results of all the tests are given in Appendix A. The test results showed that the relative humidity readings reached equilibrium in less than 15 minutes for all the soil specimens tested. The total suction for each specimen was then calculated using Equation 2.2. The experimental results showed that the relative humidity increases as the compaction water content increases, meanwhile the total suction decreases. Furthermore, the total suction increases as salinity of the pore water increases.



(a) Kaolin mixed with 5.8 g/l sodium chloride solution



(b) Residual soil A mixed with 16.0 g/l sodium chloride solution



(c) Residual soil B mixed with tap water

Figure 4.1 Typical relative humidity reading with time for kaolin, residual soils A and B compacted with standard Proctor effort

4.2.2 Matric Suction

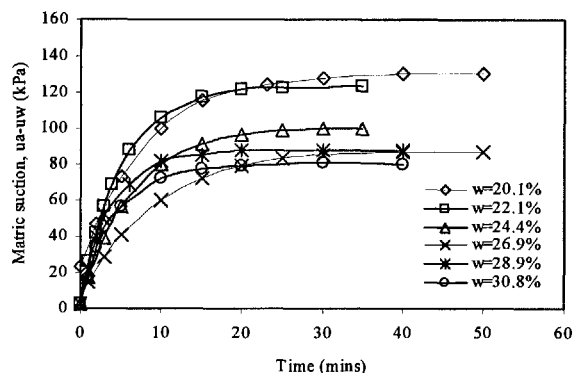
Matric suctions were obtained using null-type axis translation apparatus with a 15-bar air entry ceramic disk. Figures 4.2a, 4.2b and 4.2c show the typical response curves for matric suction measurement of kaolin and two residual soils samples compacted with standard Proctor effort, respectively. As seen in Figure 4.2, the response curves for matric suction measurement show a hyperbola. For kaolin, the response curve shows the fastest equilibration time followed by residual soil B then residual soil A. Test results of all the specimens are given in Appendix B.

The variation of matric suction of the soil specimens with compaction water content and the trend line are shown in Figure 4.3. As expected, matric suction does not vary with the concentration of the salt solution used to compact the soil samples. The matric suction – water content relationships are related to the grain size distribution and the packing configuration of the soil particles. The scatter in the data are ± 5 kPa, ± 52 kPa, and ± 67 kPa for kaolin, residual soil A and residual soil B, respectively. The scatter in the data is attributed to error in the determination of the water content of $\pm 1\%$.

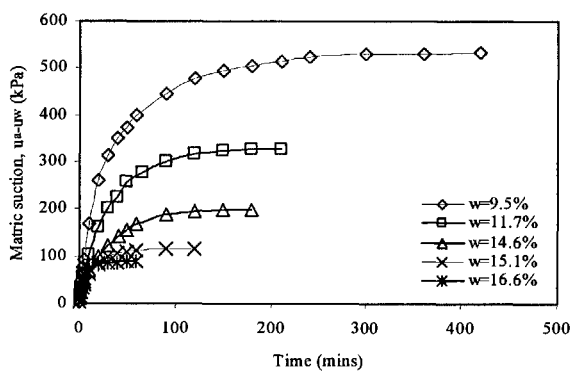
4.2.3 Osmotic Suction

In this study, the osmotic suction was determined from the electrical conductivity of the pore water extracted from a soil sample and from the solutions used to compact the soil. For the soil samples mixed with tap water and the lowest salinity sodium chloride solution, the osmotic suction was determined from electrical conductivity of the extracted pore water using a high pressure squeezer. For the soil sample mixed with sodium chloride solutions of higher salinities, the osmotic suction was determined by measuring the electrical conductivity of the sodium chloride solutions as the electrical conductivity exceeded the measurement

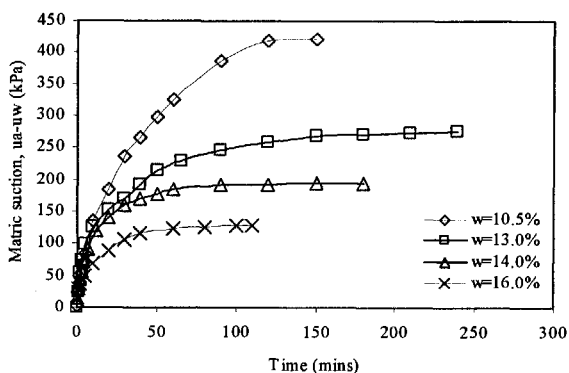
range of the compact conductivity meter (Horiba B-173). The osmotic suction was determined using Equation 3.1.



(a) Kaolin mixed with 5.8 g/l sodium chloride solution

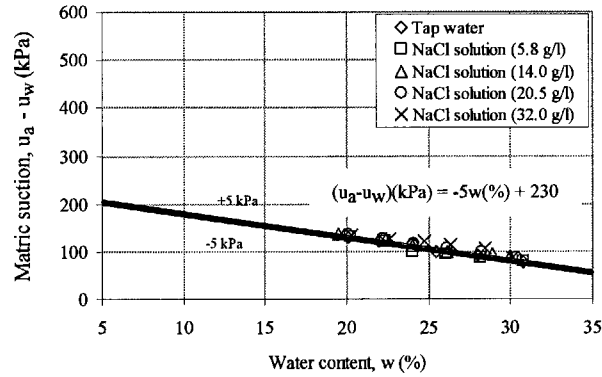


(b) Residual soil A mixed with 16.0 g/l sodium chloride solution

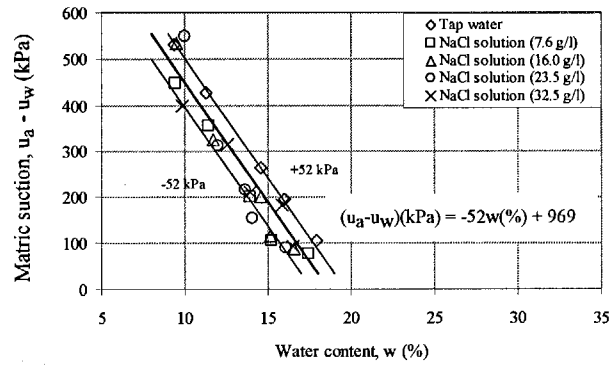


(c) Residual soil B mixed with tap water

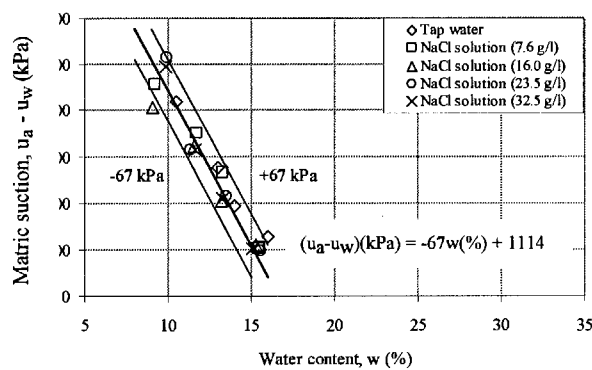
Figure 4.2 Typical plots of matric suction against time for compacted kaolin and residual soils A and B compacted with standard Proctor effort



(a) Kaolin



(b) Residual soil A



(c) Residual soil B

Figure 4.3 Matric suction of compacted soil samples at various water contents

The effect of squeezing pressure on the electrical conductivity of the pore fluid was also investigated. For the tests conducted, a squeezing pressure of up to 30 MPa was applied to the piston with a 10-tonne compression machine to extract 3 ml of pore water for samples compacted on the dry side of optimum, and a squeezing pressure of 10 MPa was needed to extract 3 ml of pore water for samples compacted on the wet side of optimum. For electrical conductivity reading by the Horiba BS-173 conductivity meter, less than 0.5 ml of pore water is required. For the Horiba ES-12 conductivity meter, 3 ml of pore water is insufficient for measurement. Therefore, the Horiba ES-12 conductivity meter was only used to measure the electrical conductivity of the solution used to prepare the soil samples. No significant difference in electrical conductivity reading was observed for squeezing pressures in the range of 10 to 30 MPa and between the extracted pore water and solution used for preparing the soil samples.

The suction measurement results for the compacted kaolin and two compacted residual soils are summarized in Tables 4.1, 4.2 and 4.3, respectively. Figures 4.4(a), (b) and (c) show the relationships between “matric + osmotic” suctions and total suction for the kaolin and the residual soil samples, respectively. It showed that the total suction values and “matric + osmotic” suction values were quite different. This is because the measurement accuracies associated with the measurement of matric suction, osmotic suction and total suction are different. As a result the calculated total suction value may not be equivalent to the directly measured total suction. The figures show that as osmotic suction increases, “matric + osmotic” suctions may exceed the total suction. For the tap water (i.e. low osmotic suction), the error could be as much as -75% for kaolin, -50% for residual soil A, and -60% for residual soil B. For the other sodium chloride solutions, the error is generally within $\pm 30\%$. Observations by Leong et al. (2003a) for compacted mudstone and sandstone residual soils from the Jurong formation of Singapore using distilled water showed that the sum of matric and osmotic suctions was always less than the measured total suction. Similarly, Leong et al. (2003a) used the water activity meter to measure total suction, compact electrical

conductivity meter to measure osmotic suction and null-type axis-translation apparatus to measure the matric suction. The results of these tests are shown in Table 4.4. Table 4.4 shows that the total suction for sandstone residual soil specimens was higher than the mudstone residual soil specimens. The total suction determined from the chilled mirror dew-point device for residual soil compacted with distilled water overestimated “matric + osmotic” suctions. Leong et al. (2003a) suggested that the following relationship be used to estimate “matric + osmotic” suctions from the total suction obtained from the chilled mirror-dew point device:

$$(u_a - u_w) + \pi = -0.00006\psi^2 + 0.5866\psi \quad (4.1)$$

Equation 4.1 is also plotted in Figure 4.4 and showed good agreement for soil samples compacted with tap water but is not applicable for soil samples compacted with salt solutions.

The discrepancy between “matric + osmotic” suctions and total suction has important implication in studies which involves suction measurement. Here the measurement accuracy of each individual suction component is needed in order to know how reliable each suction component is. The error, if one of the suction components is inferred from the measurement of the two other suction components, also needs to be known. For the total suction measurement, the accuracy is dependent on the accuracy of the chilled mirror dew-point device, i.e. $\pm 0.3\%$. The accuracy in terms of total suction measurement can be evaluated by differentiating Equation 3.1 with respect to relative humidity, i.e.

$$\frac{d\psi}{dR_h} = -\frac{RT}{v_{w0}\omega_v} \left(\frac{1}{R_h} \right) \quad (4.2)$$

Expressing the error in total suction measurement $\Delta\psi$ as a function of the total suction ψ gives:

$$\Delta\psi = \frac{\psi\Delta R_h}{R_h \ln R_h} \quad (4.3)$$

Table 4.1 Summary of suction measurement of kaolin specimens at different salinities and water content compacted with standard Proctor effort

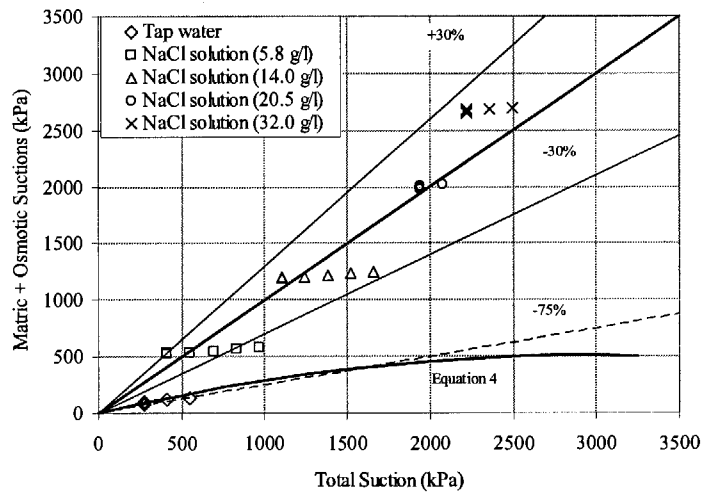
| Solution | Dry Density (Mg/m ³) | Water content (%) | Total suction (kPa) | Matric suction (kPa) | Osmotic suction (kPa) | “Matric + osmotic” (kPa) |
|---------------|----------------------------------|-------------------|---------------------|----------------------|-----------------------|--------------------------|
| Tap Water | 1.34 | 20.1 | 551 | 130 | 3 | 133 |
| | 1.34 | 22.0 | 413 | 121 | 3 | 124 |
| | 1.36 | 24.0 | 413 | 118 | 4 | 122 |
| | 1.35 | 25.5 | 275 | 100 | 3 | 103 |
| | 1.32 | 28.0 | 275 | 92 | 4 | 96 |
| | 1.27 | 30.8 | 275 | 76 | 4 | 80 |
| NaCl 5.8 g/l | 1.33 | 20.2 | 967 | 132 | 444 | 576 |
| | 1.33 | 22.1 | 828 | 123 | | 567 |
| | 1.34 | 24.4 | 690 | 100 | | 544 |
| | 1.34 | 26.9 | 551 | 94 | | 538 |
| | 1.31 | 28.9 | 413 | 87 | | 531 |
| | 1.28 | 30.8 | 413 | 80 | | 524 |
| NaCl 14.0 g/l | 1.33 | 19.5 | 1661 | 138 | 1108 | 1246 |
| | 1.33 | 22.4 | 1522 | 128 | | 1235 |
| | 1.35 | 24.0 | 1383 | 112 | | 1220 |
| | 1.34 | 26.1 | 1244 | 100 | | 1208 |
| | 1.32 | 28.9 | 1105 | 93 | | 1201 |
| | 1.29 | 30.0 | 1105 | 89 | | 1197 |
| NaCl 20.5 g/l | 1.34 | 20.1 | 2080 | 137 | 1883 | 2020 |
| | 1.36 | 22.2 | 1940 | 127 | | 2010 |
| | 1.36 | 24.1 | 1940 | 116 | | 2000 |
| | 1.35 | 26.1 | 1940 | 106 | | 1990 |
| | 1.34 | 28.2 | 1940 | 99 | | 1982 |
| | 1.31 | 30.4 | 1940 | 86 | | 1970 |
| NaCl 32.0 g/l | 1.33 | 20.3 | 2499 | 135 | 2561 | 2696 |
| | 1.33 | 22.6 | 2359 | 127 | | 2688 |
| | 1.34 | 24.8 | 2219 | 121 | | 2682 |
| | 1.33 | 26.3 | 2219 | 115 | | 2676 |
| | 1.31 | 28.4 | 2219 | 106 | | 2667 |
| | 1.29 | 30.1 | 2219 | 88 | | 2649 |

Table 4.2 Summary of suction measurement of residual soil A specimens at different salinities and water content compacted with standard Proctor effort

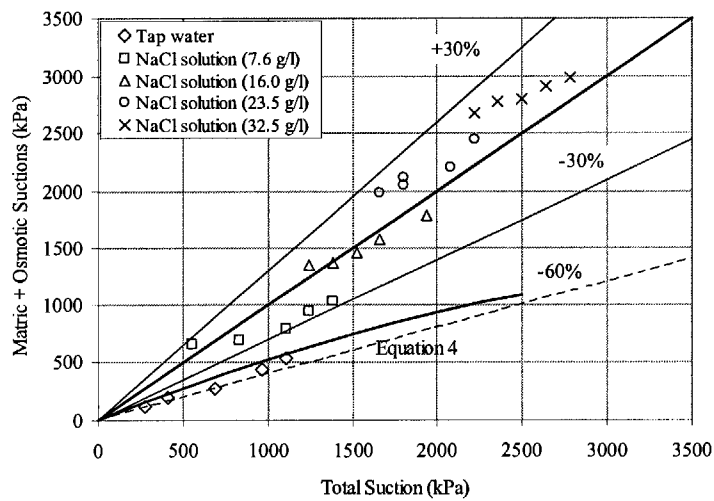
| Solution | Dry Density (Mg/m ³) | Water content (%) | Total suction (kPa) | Matric suction (kPa) | Osmotic suction (kPa) | “Matric + osmotic” (kPa) |
|------------------|----------------------------------|-------------------|---------------------|----------------------|-----------------------|--------------------------|
| Tap Water | 1.82 | 9.4 | 1105 | 532 | 5 | 537 |
| | 1.84 | 11.3 | 967 | 429 | 11 | 440 |
| | 1.83 | 14.6 | 690 | 265 | 8 | 273 |
| | 1.81 | 16.0 | 413 | 195 | 8 | 203 |
| | 1.78 | 17.9 | 275 | 105 | 11 | 116 |
| NaCl 7.6 g/l | 1.82 | 9.4 | 1383 | 450 | 586 | 1036 |
| | 1.83 | 11.4 | 1244 | 356 | | 942 |
| | 1.85 | 13.9 | 1105 | 200 | | 786 |
| | 1.82 | 15.2 | 828 | 105 | | 691 |
| | 1.79 | 17.4 | 551 | 78 | | 664 |
| NaCl 16.0 g/l | 1.82 | 9.5 | 1940 | 533 | 1260 | 1793 |
| | 1.84 | 11.7 | 1661 | 325 | | 1585 |
| | 1.85 | 14.6 | 1522 | 199 | | 1459 |
| | 1.84 | 15.1 | 1383 | 115 | | 1375 |
| | 1.81 | 16.6 | 1244 | 88 | | 1348 |
| NaCl 23.5 g/l | 1.83 | 10.0 | 2219 | 550 | 1897 | 2447 |
| | 1.84 | 12.0 | 2080 | 313 | | 2210 |
| | 1.85 | 13.7 | 1801 | 215 | | 2112 |
| | 1.85 | 14.0 | 1801 | 155 | | 2052 |
| | 1.81 | 16.1 | 1661 | 90 | | 1987 |
| NaCl 32.5 g/l | 1.80 | 9.8 | 2780 | 399 | 2589 | 2988 |
| | 1.83 | 12.5 | 2640 | 315 | | 2904 |
| | 1.86 | 13.9 | 2499 | 210 | | 2799 |
| | 1.83 | 15.9 | 2359 | 183 | | 2772 |
| | 1.80 | 16.5 | 2219 | 93 | | 2682 |

Table 4.3 Summary of suction measurement of residual soil B specimens at different salinities and water content compacted with standard Proctor effort

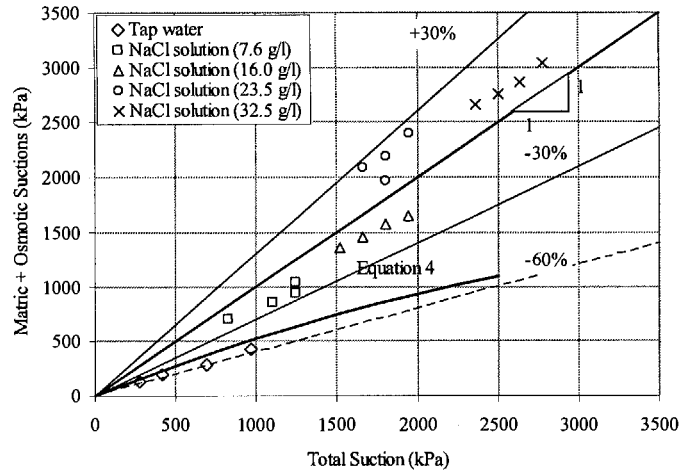
| Solution | Dry Density (Mg/m ³) | Water content (%) | Total suction (kPa) | Matric suction (kPa) | Osmotic suction (kPa) | “Matric + osmotic” (kPa) |
|---------------|----------------------------------|-------------------|---------------------|----------------------|-----------------------|--------------------------|
| Tap Water | 1.81 | 10.5 | 967 | 420 | 6 | 426 |
| | 1.83 | 13.0 | 690 | 275 | 6 | 281 |
| | 1.85 | 14.0 | 413 | 195 | 6 | 201 |
| | 1.82 | 16.0 | 275 | 127 | 6 | 133 |
| NaCl 7.6 g/l | 1.81 | 9.2 | 1244 | 457 | 586 | 1043 |
| | 1.84 | 11.7 | 1244 | 350 | | 936 |
| | 1.87 | 13.3 | 1105 | 265 | | 851 |
| | 1.85 | 15.4 | 967 | 102 | | 688 |
| NaCl 16.0 g/l | 1.81 | 9.1 | 1940 | 407 | 1249 | 1656 |
| | 1.83 | 11.6 | 1801 | 321 | | 1570 |
| | 1.86 | 13.2 | 1661 | 205 | | 1454 |
| | 1.82 | 15.3 | 1522 | 110 | | 1359 |
| NaCl 23.5 g/l | 1.82 | 9.9 | 1940 | 515 | 1876 | 2391 |
| | 1.85 | 11.4 | 1801 | 313 | | 2189 |
| | 1.86 | 13.5 | 1661 | 215 | | 2091 |
| | 1.82 | 15.6 | 1801 | 98 | | 1974 |
| NaCl 32.5 g/l | 1.80 | 9.9 | 2780 | 495 | 2550 | 3045 |
| | 1.84 | 11.6 | 2640 | 315 | | 2865 |
| | 1.86 | 13.3 | 2499 | 210 | | 2760 |
| | 1.82 | 15.1 | 2359 | 103 | | 2653 |



(a) Kaolin



(b) Residual soil A



(c) Residual soil B

Figure 4.4 Comparison of matric + osmotic suctions and total suction

Table 4.4 Suction measurements for mudstone and sandstone residual soils specimens (from Leong et al., 2003a)

| Specimen | Water content: (%) | Total suction: kPa (1) | Matric suction: kPa (2) | Osmotic suction: kPa (3) | Matric+Osmotic (4) = (1) + (3): kPa | (1) - (4): kPa | |
|-----------|--------------------|------------------------|-------------------------|--------------------------|-------------------------------------|----------------|------|
| Mudstone | MSSP1 | 11.4 | 688 | 355 | 10 | 365 | 323 |
| | MSSP2 | 14.1 | 413 | 310 | 10 | 320 | 93 |
| | MSSP3 | 16.8 | 275 | 175 | 10 | 185 | 90 |
| | MSSP4 | 20.6 | 137 | 30 | 10 | 40 | 97 |
| | MSEP1 | 11.2 | 688 | 420 | 10 | 430 | 258 |
| | MSEP2 | 14.0 | 413 | 260 | 10 | 270 | 143 |
| | MSEP3 | 16.5 | 275 | 140 | 10 | 150 | 125 |
| | MSEP4 | 20.2 | 137 | 45 | 10 | 55 | 82 |
| | MSMP1 | 11.1 | 550 | 475 | 10 | 485 | 65 |
| | MSMP2 | 13.4 | 413 | 240 | 10 | 250 | 163 |
| | MSMP3 | 16.3 | 275 | 130 | 10 | 140 | 135 |
| | MSMP4 | 20.2 | 137 | 30 | 10 | 40 | 97 |
| Sandstone | SSSP1 | 13.5 | 2495 | 1200 | 9 | 1209 | 1286 |
| | SSSP2 | 16.6 | 1650 | 690 | 9 | 699 | 951 |
| | SSSP3 | 19.3 | 825 | 475 | 9 | 484 | 341 |
| | SSSP4 | 22.8 | 275 | 120 | 9 | 129 | 146 |
| | SSEP1 | 13.2 | 2350 | 1050 | 9 | 1059 | 1291 |
| | SSEP2 | 15.7 | 1900 | 855 | 9 | 864 | 1036 |
| | SSEP3 | 18.25 | 1100 | 560 | 9 | 569 | 531 |
| | SSEP4 | 22.5 | 550 | 175 | 9 | 184 | 366 |
| | SSMP1 | 13.7 | 2350 | 950 | 9 | 959 | 1391 |
| | SSMP2 | 15.5 | 1800 | 775 | 9 | 784 | 1016 |
| | SSMP3 | 18.1 | 965 | 550 | 9 | 559 | 406 |
| | SSMP4 | 22.2 | 410 | 165 | 9 | 174 | 236 |

In Equation 4.3, ΔR_h is taken as $\pm 0.3\%$ and therefore the absolute error $|\Delta R_h|$ is 0.6%. For matric suction measurement, as long as good contact is ensured between the soil specimen and high-air entry ceramic disk, and the pressure plate is properly de-aired and saturated, the measurement error is attributed to the pressure recording device (pressure transducer). Taking nominal pressure transducer error of ± 5 kPa for the air and water pressure transducers, the matric suction measurement accuracy $\Delta(u_a - u_w)$ is of the order of ± 10 kPa or absolute error, $|\Delta(u_a - u_w)|$, of 20 kPa. For osmotic suction measurement, the accuracy is dependent on the accuracy of the conductivity meter used i.e., $\pm 0.5\%$ full scale. The error in osmotic suction measurement can be estimated by differentiating Equation 3.1 with respect to electrical conductivity i.e.,

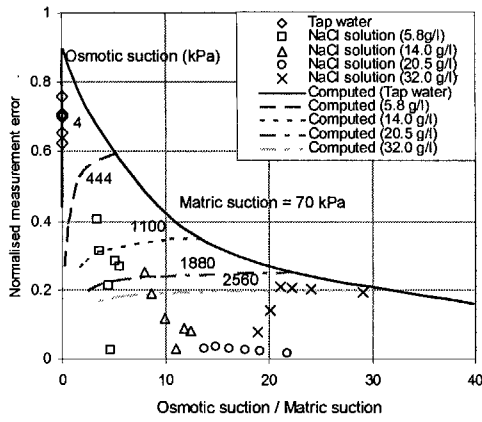
$$\frac{d\pi}{dEC} = p_a (0.345EC^{0.15}) \quad (4.4)$$

Expressing the error in osmotic suction measurement $\Delta\pi$ in terms of the measured osmotic suction gives:

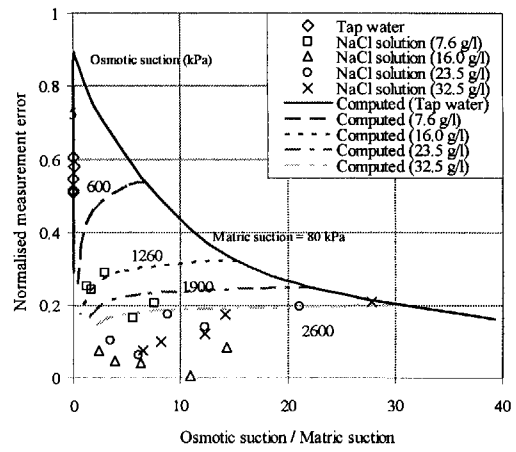
$$\Delta\pi = \frac{1.15\Delta EC}{EC} \pi \quad (4.5)$$

In Equation 4.5, ΔEC is taken as $\pm 0.5\%$ full scale for the Horiba ES-12 conductivity meter or absolute error $|\pi|$ of 1% full scale. Therefore Figure 4.3 is essentially a plot of $[(u_a - u_w) + \Delta(u_a - u_w) + \pi + \Delta\pi]$ versus $(\psi + \Delta\psi)$. The measurement error when comparing “matric + osmotic” suctions and total suction is therefore $[\Delta\psi - \Delta(u_a - u_w) - \Delta\pi]$.

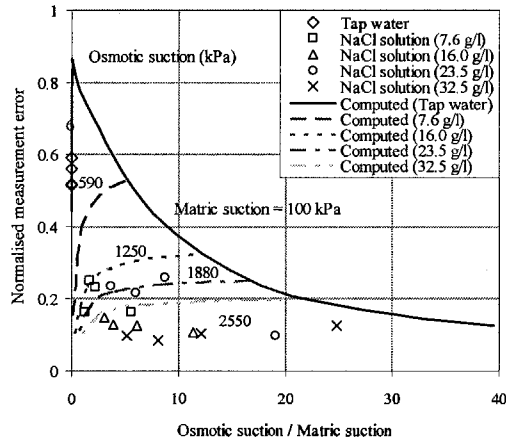
To evaluate the measurement error for various combinations of matric and osmotic suctions, the measurement error $[\Delta\psi - \Delta(u_a - u_w) - \Delta\pi]$ normalised with the total suction measurement $(\psi + \Delta\psi)$ is plotted against the ratio of osmotic suction and matric suction i.e. $[\pi + \Delta\pi] / [(u_a - u_w) + \Delta(u_a - u_w)]$ in Figures 4.5(a), (b) and (c) for kaolin and residual soils A and B, respectively. From Equations 4.3 and 4.5 and using $\Delta(u_a - u_w) = \pm 10$ kPa, the absolute measurement error can be estimated and is plotted in Figure 4.5. The upper bound of the error curve is shown as a bold solid curve. The upper bound curve y-values are given by the measurement error $[\Delta\psi - \Delta(u_a - u_w) - \Delta\pi]$ and the x-values are given by $[\pi + \Delta\pi] / [(u_a - u_w) + \Delta(u_a - u_w)]$ where $[(u_a - u_w) + \Delta(u_a - u_w)]$ is taken as the lowest matric suction value of the compacted soil samples in the series, i.e. 70 kPa for kaolin, 80 kPa for residual soil A and 100 kPa for residual soil B. The lighter curves in Figure 4.5 are for various osmotic suction values corresponding approximately to the measured osmotic suction at each sodium chloride concentration. In other words, the lighter curves are given by x-values $[\pi + \Delta\pi] / [(u_a - u_w) + \Delta(u_a - u_w)]$ where $[\pi + \Delta\pi]$ is constant and y-values $[\Delta\psi - \Delta(u_a - u_w) - \Delta\pi]$ where $\Delta\pi$ is constant as $[\pi + \Delta\pi]$ is constant. For a set of matric suction and osmotic suction values, the curves form an upper bound of the absolute measurement error that can be expected when comparing “matric + osmotic” suctions and total suction. Overall, the absolute measurement error decreases as the ratio of $[\pi + \Delta\pi] / [(u_a - u_w) + \Delta(u_a - u_w)]$ increases. At low salt concentrations (e.g. tap water), the difference between “matric + osmotic” suctions and total suction can be as high as 90% of the measured total suction. An idealised representation of the absolute measurement error for various values of osmotic suction and matric suction is shown in Figure 4.6. Plots similar to Figure 4.6 can be developed when using other techniques of suction measurement.



(a) Kaolin



(b) Residual soil A



(c) Residual soil B

Figure 4.5 Normalised measurement error versus ratio of osmotic and matric suctions.

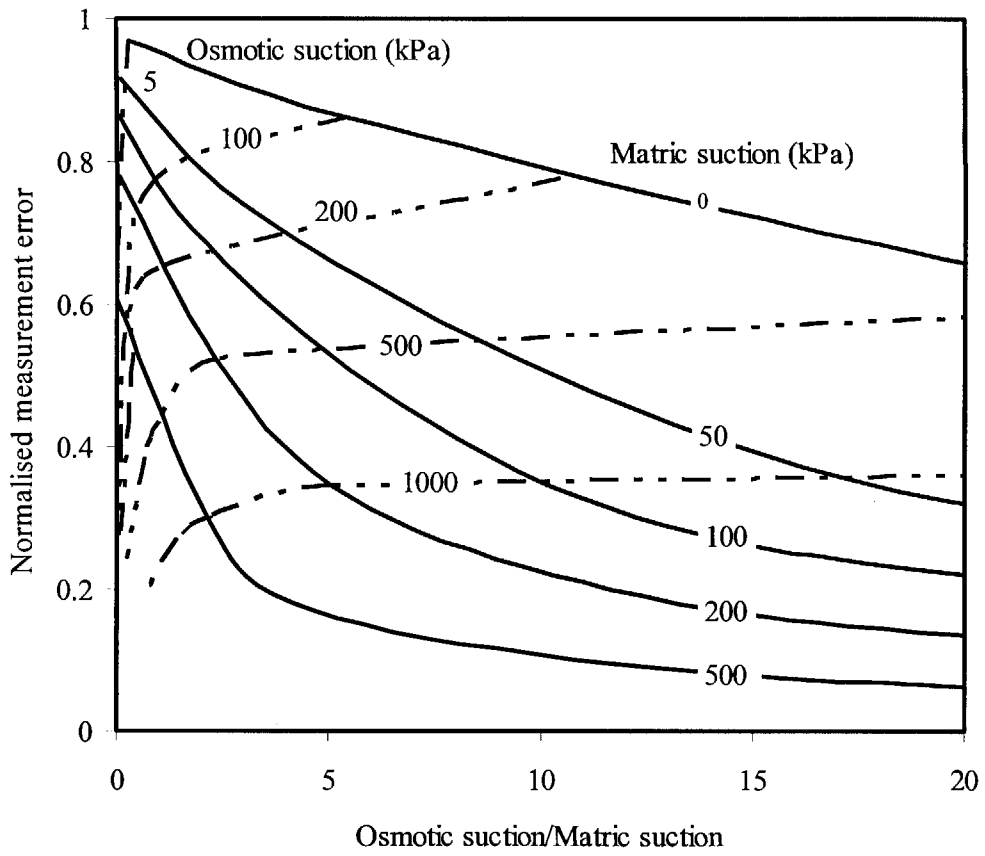


Figure 4.6 Family of normalized error curves for various osmotic and matric suction values

4.3 VOLUME CHANGE

4.3.1 Introduction

Single and double-oedometer test results for compacted kaolin, residual soil A and residual soil B are shown as e -log P curves in Appendix C and D. Single-oedometer tests were only performed on selected compacted samples of kaolin, residual soil A and residual soil B for the purpose of comparison with the double-oedometer tests. In the following sections, the single and double-oedometer tests are discussed.

4.3.2 Single-Oedometer Test

Single-oedometer tests were carried out in accordance to ASTM D5333-92 (1998a) on compacted samples of kaolin, residual soil A and residual soil B with initial water contents of 28.5%, 9.4% and 12.6%, respectively. In this test, the specimens were inundated at 200 kPa. According to ASTM D5333-92, the collapse potential at vertical stress of 200 kPa is termed as collapse index, I_e and is defined by:

$$I_e = \left(\frac{\Delta e}{(1 + e_o)} \right) 100\% \quad (4.6)$$

where:

Δe = change in void ratio as a result of wetting

e_o = initial void ratio

The classification of collapse index, I_e , according to ASTM D5333-92 is given in Table 4.5.

Table 4.5 Classification of collapse index, I_c (from ASTM D5333-92, 1998a)

| Degree of collapse | Collapse Index, I_c (%) |
|--------------------|---------------------------|
| None | 0 |
| Slight | 0.1 to 2.0 |
| Moderate | 2.1 to 6.0 |
| Moderately severe | 6.1 to 10.0 |
| Severe | > 10 |

The single-oedometer test results for compacted soils of kaolin, residual soil A and residual soil B are plotted together with the double-oedometer test results in Figures 4.7 to 4.9, respectively. The single-oedometer test result for kaolin specimen did not show much collapse after inundation with water (Figure 4.7). Based on Equation 4.6, the collapse index of the kaolin specimen is 0.2% at a vertical pressure of 200 kPa. According to Table 4.5, it is considered to have slight degree of collapse. Single-oedometer test result for residual soil A specimen (Figure 4.8) shows that the single-oedometer curve lied between the dry and wet curves of the double-oedometer test. The single-oedometer test curve did not meet the compression curve of the specimen in double-oedometer test under wet condition after inundation with water. Based on Equation 4.6, the collapse index of the residual soil A specimen is 3.3% at a vertical pressure of 200 kPa. According to Table 4.5, it is considered to have moderate degree of collapse. The single-oedometer test result for residual soil B specimen is shown in Figure 4.9. The single-oedometer test curve coincides with the wet compression curve of the double-oedometer test after inundation with water. The collapse index of the residual soil B specimen is 1.4% and it is considered to have slight degree of collapse. Overall, a comparison of the single-oedometer and double-oedometer tests demonstrated that the double- oedometer tests are valid for estimating wetting-induced settlement. The double-oedometer tests were found to provide similar wetting-induced settlement as the single oedometer test. Furthermore the double-oedometer tests provide the wetting-induced settlement at many different

stress levels using only two specimens. Many single oedometer tests will be required if the wetting-induced settlement is needed at many different stress levels. In the remaining sections of this chapter, only double-oedometer tests are considered.

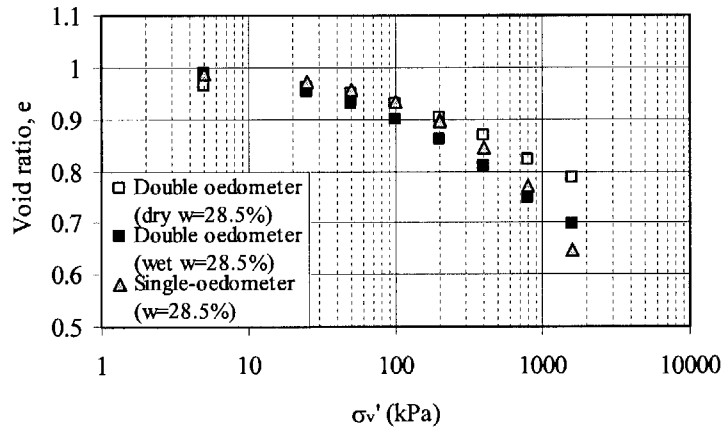


Figure 4.7 Comparison of single and double-oedometer tests results for compacted kaolin

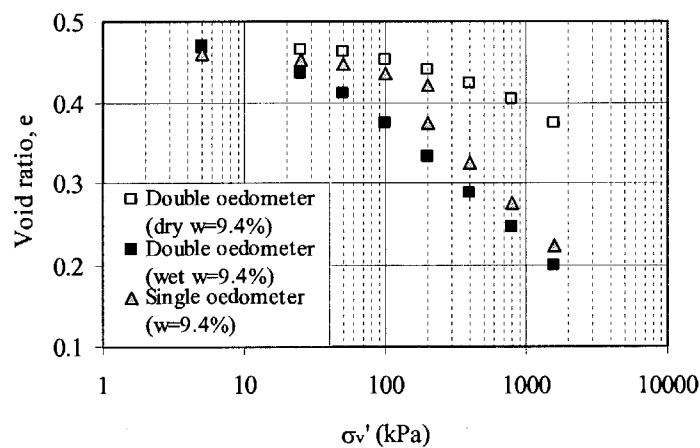


Figure 4.8 Comparison of single and double-oedometer tests results for compacted residual soil A

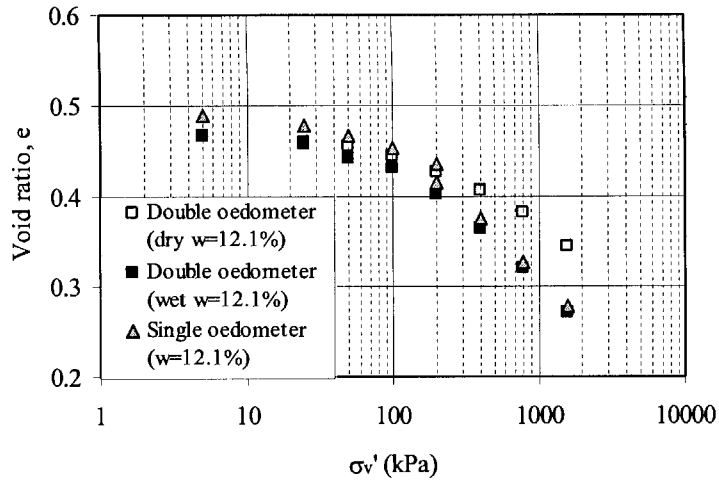
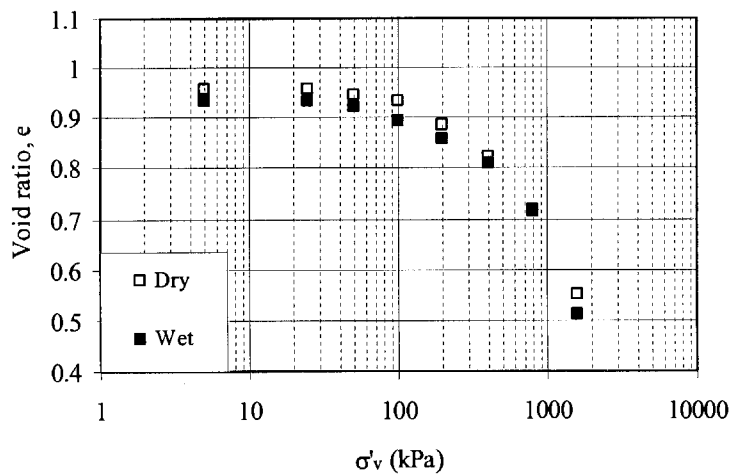


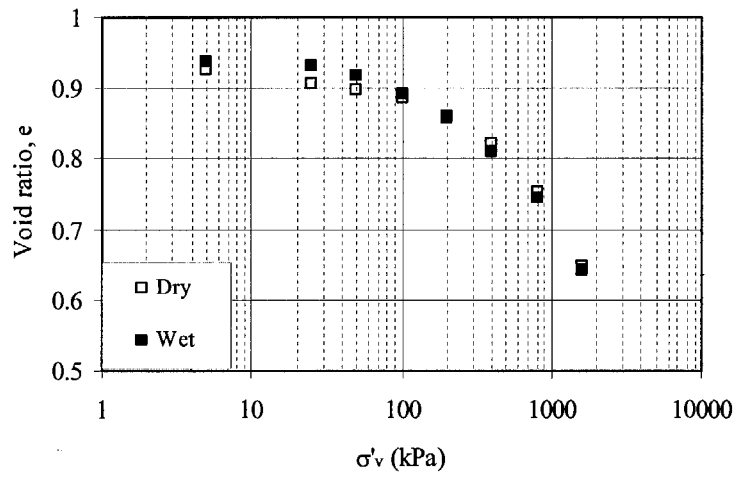
Figure 4.9 Comparison of single and double-oedometer tests results for compacted residual soil B

4.3.3 Double-Oedometer Test

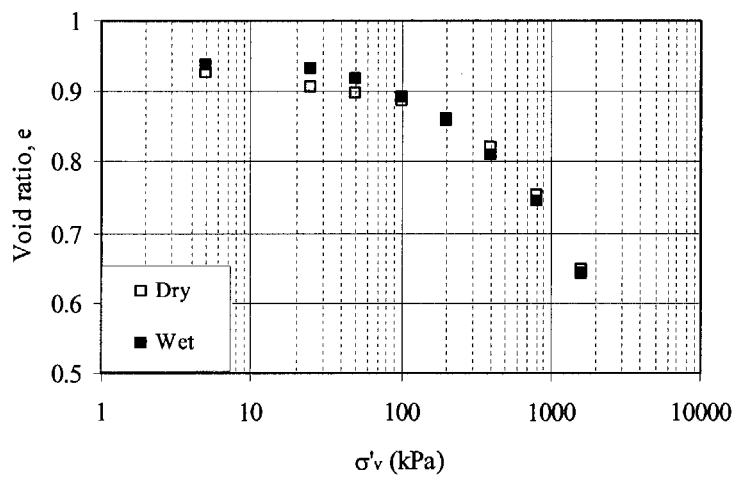
The double-oedometer test results for compacted samples of kaolin, residual soils A and B are shown in Figures 4.10 to 4.12, respectively. The figures showed that the dry soil specimens compression curve generally lies above the wet soil specimens compression curves.



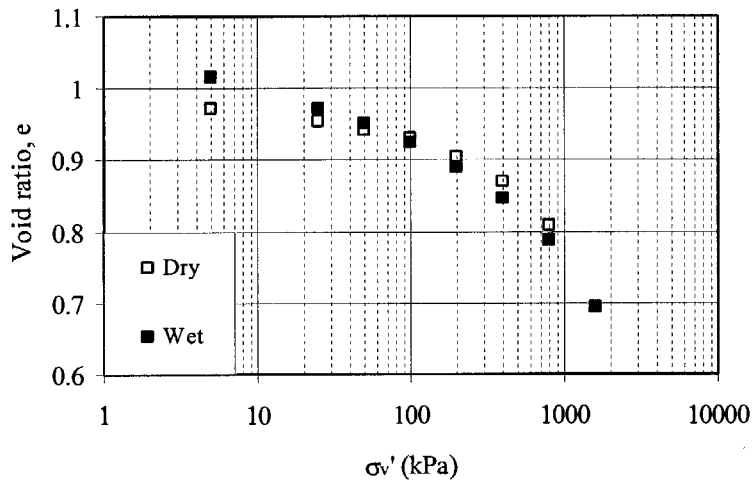
(a) $w = 21.5\%$



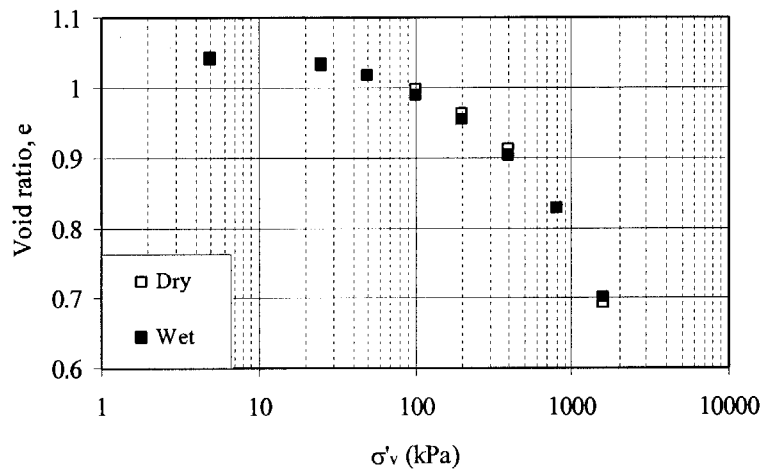
(b) $w=23.6\%$



(c) $w=26.5\%$

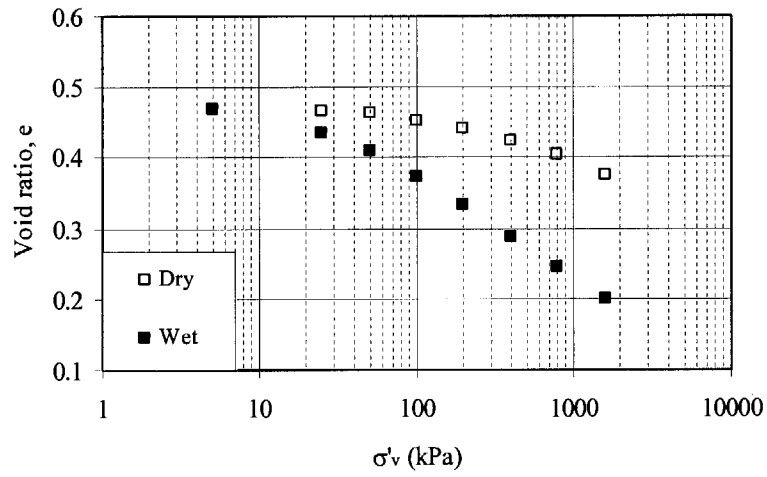


(d) $w = 27.3\%$

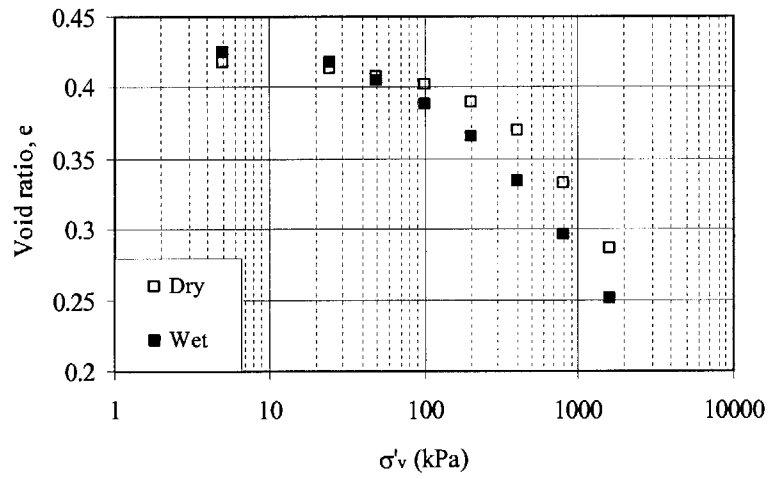


(e) $w = 30.1\%$

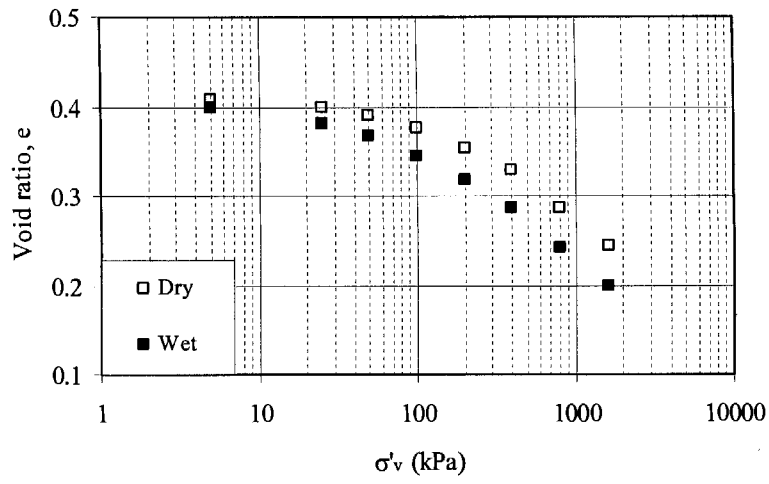
Figure 4.10 Double-oedometer test results for compacted kaolin



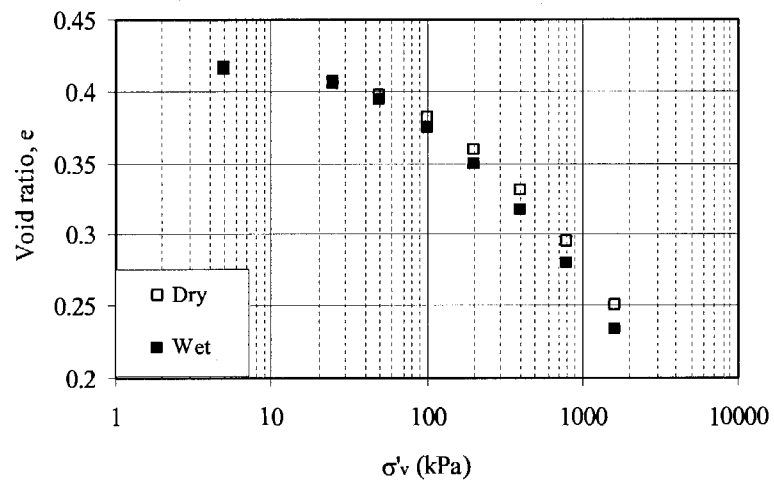
(a) $w = 9.4\%$



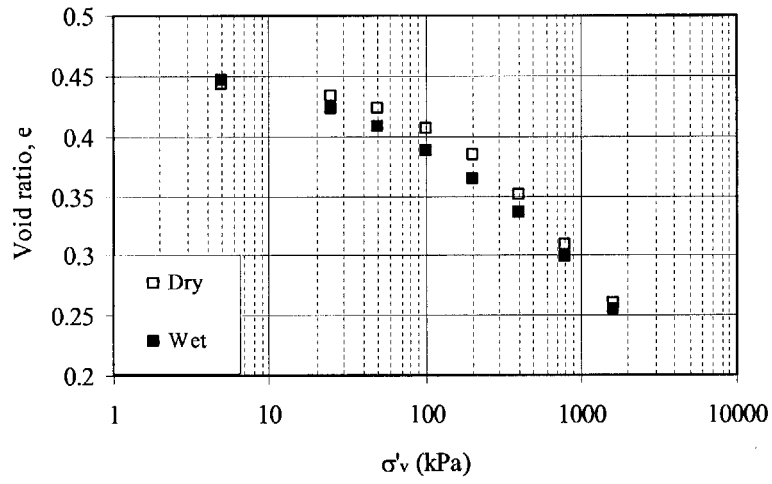
(b) $w = 11.4\%$



(c) $w = 13.8\%$

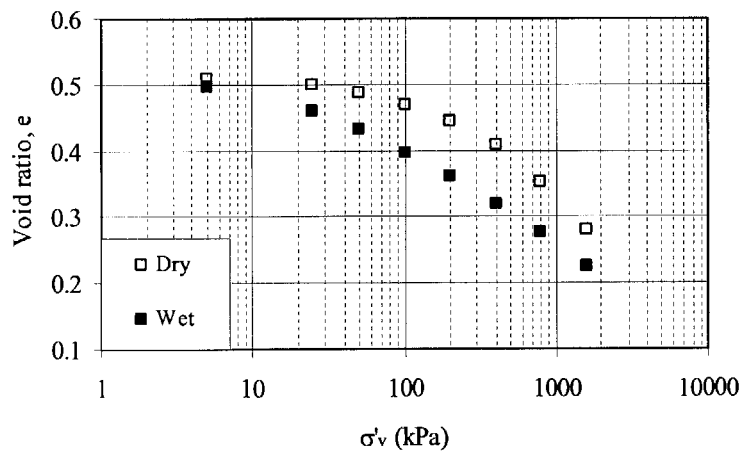


(d) $w = 15.3\%$

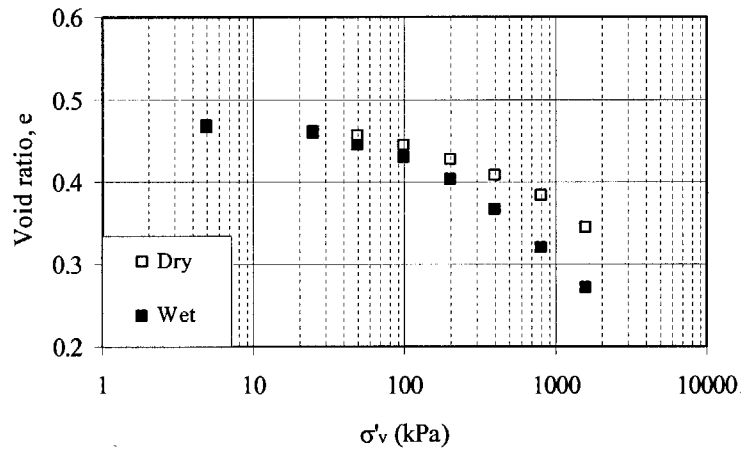


(e) $w = 16.7\%$

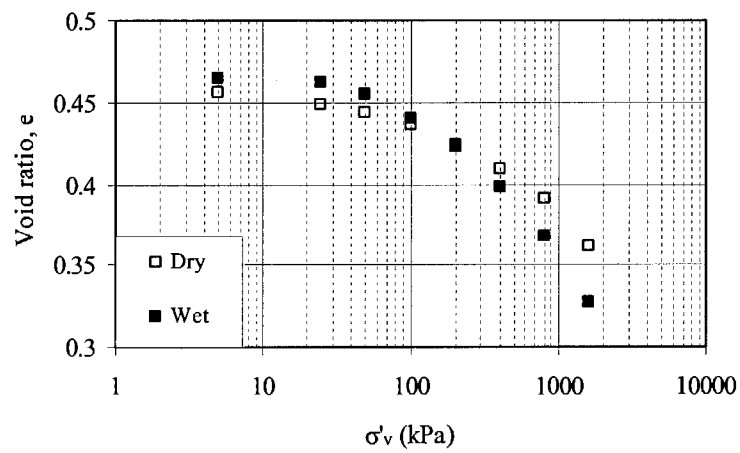
Figure 4.11 Double-oedometer test results for compacted residual soil A



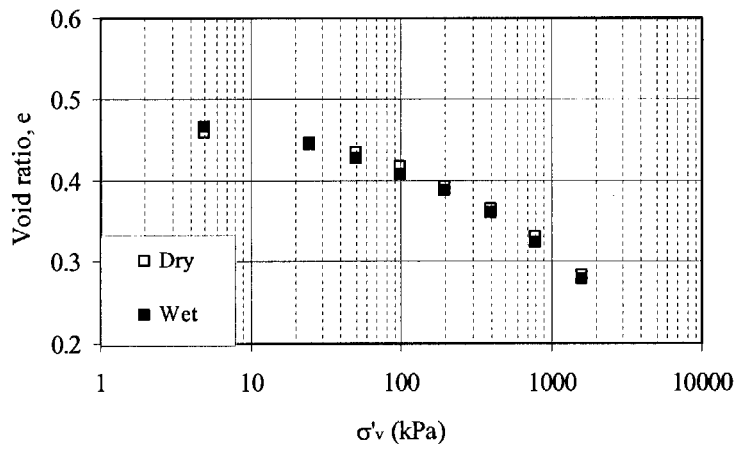
(a) $w = 9.7\%$



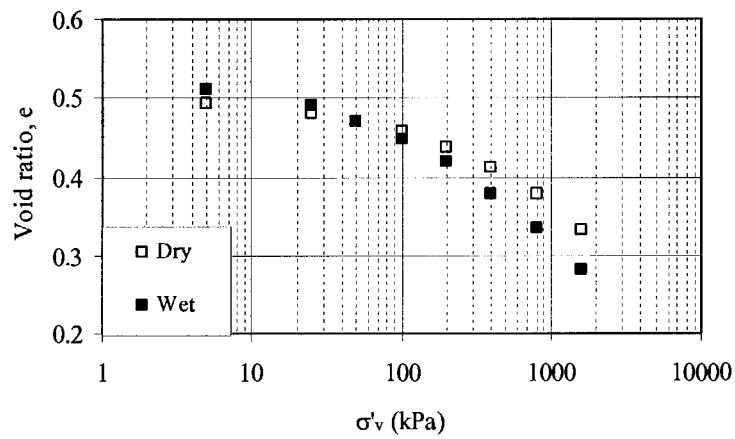
(b) $w = 12.1\%$



(c) $w = 13.2\%$



(d) $w = 15.2\%$



(e) $w = 17.5\%$

Figure 4.12 Double-oedometer test results for compacted residual soil B

4.3.3.1 Apparent Preconsolidation Pressure and Compression Index

The e-log P curves for compacted kaolin, residual soil A and residual soil B specimens have the same typical shape as saturated soils. Therefore, a preconsolidation pressure and a compression index can be determined from the e-log P curves. However, the preconsolidation pressure is not of the same sense as saturated clay or in situ soils. Therefore, the preconsolidation pressure of the compacted soils is termed as apparent preconsolidation pressure. In this study, the apparent preconsolidation pressure was determined using Casagrande's method (the most probable preconsolidation pressure). The apparent preconsolidation pressure and the compression indices of kaolin, residual soil A and residual soil B are summarised in Tables 4.6 to 4.8, respectively. The apparent preconsolidation pressures of kaolin specimens range from 280 kPa to 310 kPa for the wet condition and range from 320 kPa to 350 kPa for the dry condition. The apparent preconsolidation pressures of residual soil A specimens range from 60 kPa to 160 kPa for the wet condition and range from 180 kPa to 220 kPa for the dry condition. The apparent preconsolidation pressures of residual soil B specimens range from 90 kPa to 170 kPa for the wet condition and range from 170 kPa to 220 kPa for the dry condition. The difference in the P'_c value indicates that the P'_c value is associated with the compacted condition of the soil sample. Soil samples compacted at different water contents have different degree of saturation and void ratio which would directly affect the matric suction. The matric suction contributed to the strength of the soil structure and therefore samples compacted dry of optimum appear to have a higher P'_c than samples compacted wet of optimum. Figures 4.13 to 4.15 show the apparent preconsolidation pressure (P'_c) versus dry density (ρ_d) for the wet and dry conditions of compacted kaolin, residual soils A and B. In general, the apparent preconsolidation pressures for the dry and the wet conditions increase as the dry density increases. It also appears that the apparent preconsolidation pressure of the soil at low dry density tend to increase faster with increase in dry density. Figures 4.13 to 4.15 show that at the same dry density, the apparent preconsolidation pressure at dry condition is higher

than at the corresponding wet condition. This is due to the presence of suction in the dry specimens (Figure 4.3). Suction decreases as the water content and degree of saturation increases. The compacted specimens when inundated with water are expected to lose some or all its suction. This will account for the lower apparent preconsolidation pressures of the wet specimens. The difference in the apparent preconsolidation pressures between the wet and the dry specimens for compacted kaolin specimen ranged from 30 kPa to 40 kPa. For the compacted residual soil A, the difference in the apparent preconsolidation pressures between the wet and the dry specimens ranged from 20 kPa to 120 kPa. For the compacted residual soil B, the difference in the apparent preconsolidation pressures between the wet and the dry specimens is ranged from 20 kPa to 130 kPa.

Table 4.6 Apparent preconsolidation pressures and compression indices of kaolin specimens compacted with standard Proctor effort

| Water content, w (%) | Dry density, ρ_d (Mg/m^3) | Wet | | Dry | |
|------------------------|---|--|--------------------------|--|--------------------------|
| | | Apparent preconsolidation pressure (kPa) | Compression index, C_c | Apparent preconsolidation pressure (kPa) | Compression index, C_c |
| 21.5 | 1.342 | 300 | 0.4889 | 330 | 0.4128 |
| 23.6 | 1.358 | 300 | 0.3665 | 340 | 0.2526 |
| 26.5 | 1.346 | 310 | 0.4002 | 350 | 0.3237 |
| 27.3 | 1.315 | 300 | 0.3910 | 340 | 0.2987 |
| 30.1 | 1.275 | 280 | 0.3963 | 320 | 0.3327 |

Table 4.7 Apparent preconsolidation pressures and compression indices of residual soil A specimens compacted with standard Proctor effort

| Water content, w (%) | Dry density, ρ_d (Mg/m^3) | Wet | | Dry | |
|------------------------|---|--|--------------------------|--|--------------------------|
| | | Apparent preconsolidation pressure (kPa) | Compression index, C_c | Apparent preconsolidation pressure (kPa) | Compression index, C_c |
| 9.4 | 1.776 | 60 | 0.1539 | 180 | 0.0830 |
| 11.4 | 1.846 | 160 | 0.1497 | 220 | 0.1254 |
| 13.8 | 1.870 | 160 | 0.1442 | 200 | 0.1385 |
| 15.3 | 1.852 | 150 | 0.1470 | 180 | 0.1257 |
| 16.7 | 1.813 | 160 | 0.1371 | 180 | 0.1336 |

Table 4.8 Apparent preconsolidation pressures and compression indices of residual soil B specimens compacted with standard Proctor effort

| Water content, w (%) | Dry density, ρ_d (Mg/m ³) | Wet | | Dry | |
|----------------------|--|--|--------------------------|--|--------------------------|
| | | Apparent preconsolidation pressure (kPa) | Compression index, C_c | Apparent preconsolidation pressure (kPa) | Compression index, C_c |
| 9.7 | 1.789 | 90 | 0.4106 | 220 | 0.5052 |
| 12.1 | 1.854 | 150 | 0.2705 | 200 | 0.3525 |
| 13.2 | 1.859 | 170 | 0.3740 | 190 | 0.3576 |
| 15.2 | 1.849 | 150 | 0.1718 | 170 | 0.1331 |
| 17.5 | 1.793 | 150 | 0.4264 | 180 | 0.3327 |

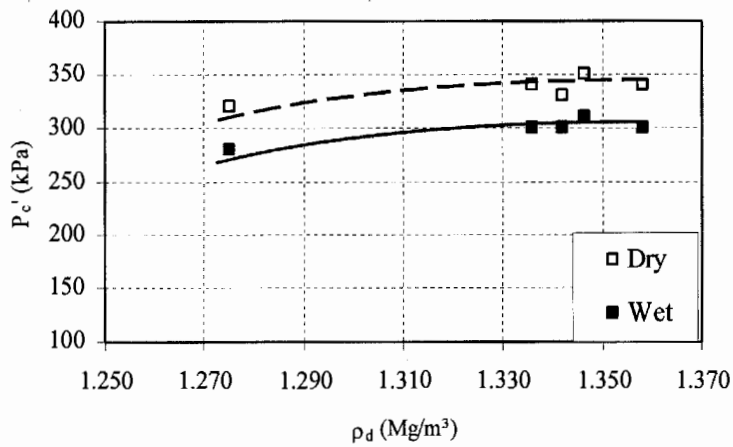


Figure 4.13 Apparent preconsolidation pressure of compacted kaolin

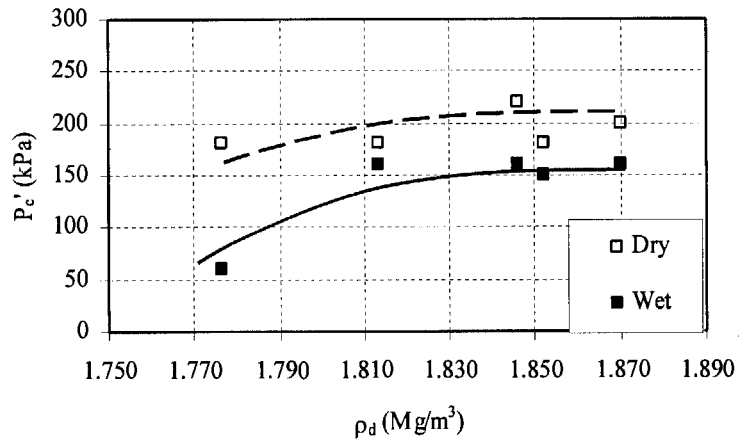


Figure 4.14 Apparent preconsolidation pressure of compacted residual soil A

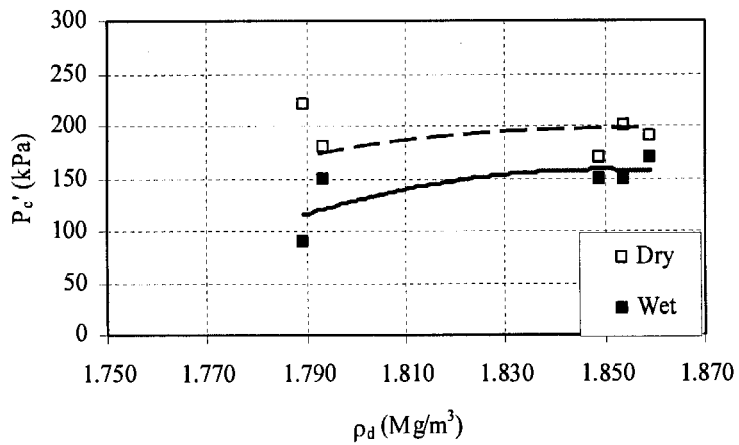


Figure 4.15 Apparent preconsolidation pressure of compacted residual soil B

The compression indices of compacted kaolin, residual soil A and residual soil B specimens are summarised in Tables 4.6 to 4.8, respectively. The compression indices C_c , of compacted kaolin specimens are in the range of 0.3 to 0.5 for the wet and dry conditions. The compression indices C_c , of compacted residual soil A are in the range of 0.1 to 0.15 for the wet and dry conditions. For compacted residual soil B, the compression indices, C_c , are in the range of 0.1 to 0.16 for the wet and dry conditions. The relationship between compression index and initial void ratio for compacted kaolin, residual soil A and residual soil B are shown in Figures 4.16 to 4.18, respectively. Generally, the compression index, C_c , increases as the initial void ratio, e_0 , increases. Figures 4.16 to 4.18 also show that the compression index of the wet specimen is higher than the corresponding dry specimen. This shows that the soil becomes more compressible when soaked with water due to a reduction of suction in the soil.

A summary of the compression indices of undisturbed Jurong Formation residual soils as reported in the literature is given in Table 4.9. The compression indices of undisturbed Jurong Formation residual soils with the bounding lines are shown in Figure 4.19. The compression indices of the two compacted residual soils in this study fall within the range of compression indices of undisturbed Jurong Formation residual soils as shown in Figure 4.19.

The compressibility of soils is an important engineering property. The compression index is used to estimate the amount of settlement under load. The compression index of the compacted soil at wet condition was found to be higher than at the dry condition. Therefore, compacted soil on wetting will experience settlement. In the next section, a method to estimate wetting-induced settlement is proposed.

Table 4.9 Compression indices of undisturbed Jurong Formation residual soils

| Residual soil | References | Compression index, C_c |
|------------------|------------------------|--------------------------|
| Jurong Formation | Dames and Moore (1983) | 0.003 – 0.108 |
| | Yong et al. (1985) | 0.10 – 0.60 |
| | Rahardjo (2000) | 0.05 |
| | Seah et al. (2001) | 0.1 – 0.6 |
| | Winn et al. (2001) | 0.1 – 0.6 |

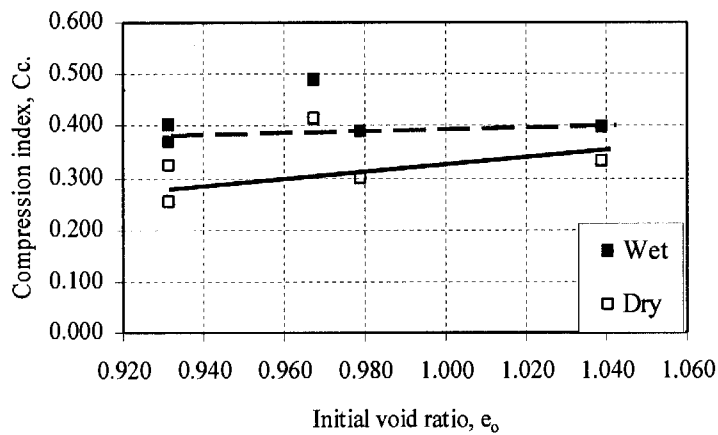


Figure 4.16 Relationship of compression index with void ratio for compacted kaolin

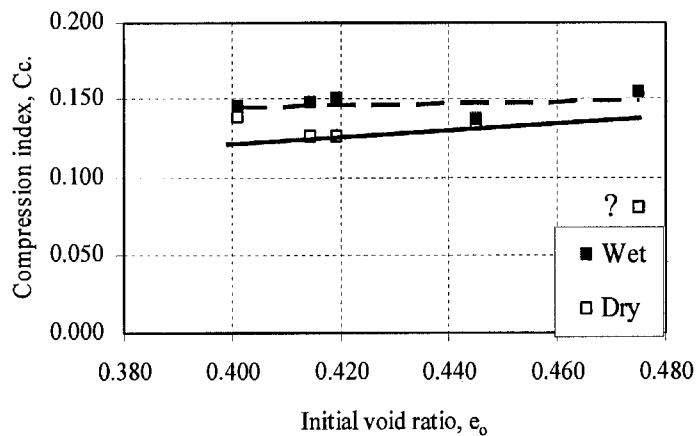


Figure 4.17 Relationship of compression index with void ratio for compacted residual soil A

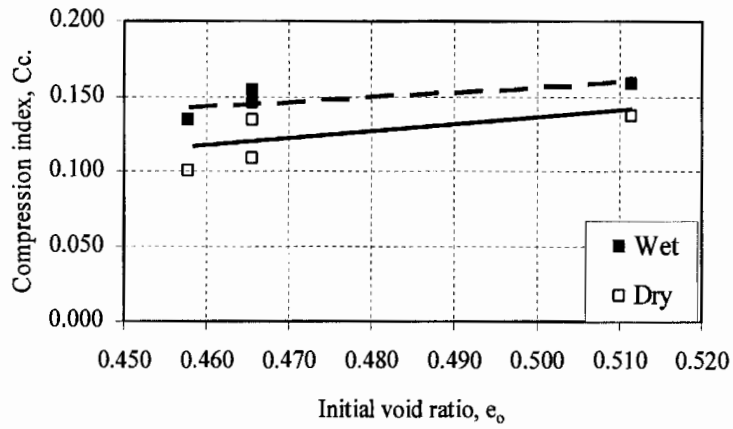


Figure 4.18 Relationship of compression index with void ratio for compacted residual soil B

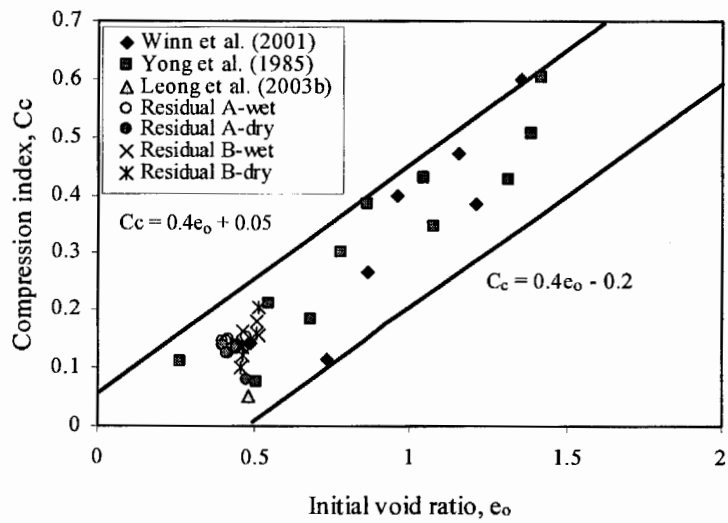


Figure 4.19 Relationship of compression index with initial void ratio of undisturbed Jurong Formation residual soils (modified from Leong et al., 2003b)

4.3.3.2 Estimation of Wetting-Induced Settlement of Compacted Soils

Carrier and Beckman (1984) have shown that the general compressibility relationship for saturated clays can be modeled using:

$$e = \alpha \left(\frac{\sigma'_v}{p_{\text{atm}}} \right)^\beta + \varepsilon \quad (4.6)$$

where:

- e = void ratio
- σ'_v = effective vertical stress
- p_{atm} = atmospheric pressure (~100 kPa)
- α = empirical constant
- β = empirical constant
- ε = empirical constants.

For normally consolidated clays, α , β and ε are given by:

$$\alpha = 0.0208\text{PI} \left(1.192 + \frac{1}{A} \right) \quad (4.7a)$$

$$\beta = -0.143 \quad (4.7b)$$

$$\varepsilon = 0.027\text{PL} - 0.0133\text{PI} \left(1.192 + \frac{1}{A} \right) \quad (4.7c)$$

where:

- PI = plasticity index
- A = activity (i.e. $\frac{\text{PI}}{\% \text{clay} < 2\mu\text{m}}$)

It is conceivable that any saturated soils under one-dimensional compression can also be modeled using Equation 4.6. Equation 4.6 can be normalized by the initial void ratio e_0 to give:

$$\frac{e}{e_o} = \frac{\alpha}{e_o} \left(\frac{\sigma'_v}{p_{atm}} \right)^\beta + \frac{\varepsilon}{e_o} \quad (4.8)$$

If it is assumed that when σ'_v tends to 0, $e = e_o$, then $\varepsilon/e_o = 1$ and Equation 4.8 can be shown to be:

$$\frac{e}{e_o} = 1 + \alpha' \left(\frac{\sigma'_v}{p_{atm}} \right)^\beta \quad (4.9)$$

where $\alpha' = \alpha/e_o$

Equation 4.9 can then be used to describe the one-dimensional compression of any saturated soils. The relationships given for the empirical constants α , β and ε in Equation 4.7 are dependent on Atterberg limits, water content at the start of the compression test, the activity and the pore-water composition (Carrier and Beckman, 1984). In other words, the empirical constants α , β and ε in Equation 4.7 are dependent on soil type and soil structure. Using the same reasoning, the empirical constants α' and β in Equation 4.9 account for soil type and soil structure as well. Therefore for compacted soils, α' and β values will be different for each compacted condition as the soil structure is different.

A slight modification of Equation 4.9 was found to give a better fit to the one-dimensional compression of the wet compacted soil specimens as shown below:

$$\frac{e}{e_o} = \left[1 + \alpha' \left(\frac{\sigma'_v}{p_{atm}} \right)^\beta \right]^{\beta'} \quad (4.10)$$

where α' and β' are empirical constants that account for soil type and soil structure. Figures 4.20 (a), (b), and (c) show the typical plot of compression curves of Equation 4.9 for compacted kaolin, residual soil A and residual soil B. The typical plot of compression curves of Equation 4.10 for compacted kaolin,

residual soil A and residual soil B are showed in Figures 4.21 (a), (b), and (c). For kaolin, the sum of squared residuals (SSR) for Equation 4.9 is 0.000698 and the SSR for Equation 4.10 is 0.000382. For residual soil A, the SSR for Equation 4.9 is 0.00053 and the SSR for Equation 4.10 is 0.000098. For residual soil B, the SSR for Equation 4.9 is 0.00045 and the SSR for Equation 4.10 is 0.00033. The smaller the SSR value the better the fit of the equation to the data. For all the three soil samples, the SSR value for Equation 4.10 is smaller than the corresponding SSR value for Equation 4.9. Thus, Equation 4.10 gives a better fit to the one-dimensional compression of the wet compacted soil specimens.

A modification of Equation 4.10 will be needed to describe the dry compression curve in the double-oedometer test. As the compacted soil specimens in the double-oedometer tests are nominally identical, one would expect α' and β' to be identical. A modification is thus needed to account for the degree of saturation, S_o , corresponding to void ratio, e_o . The following modification of Equation 4.10 is suggested to describe the dry compression curve:

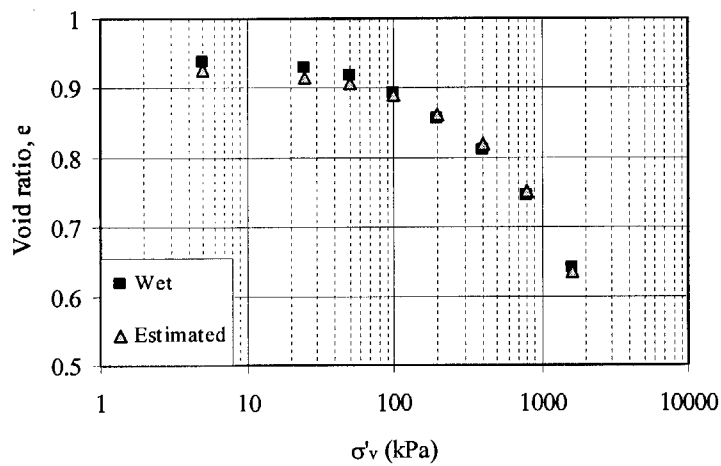
$$\frac{e}{e_o} = \left[1 + \alpha' S_o^\eta \left(\frac{\sigma'_v}{p_{\text{atm}}} \right) \right]^{\beta'} \quad (4.11)$$

where empirical constant η is dependent on soil type only.

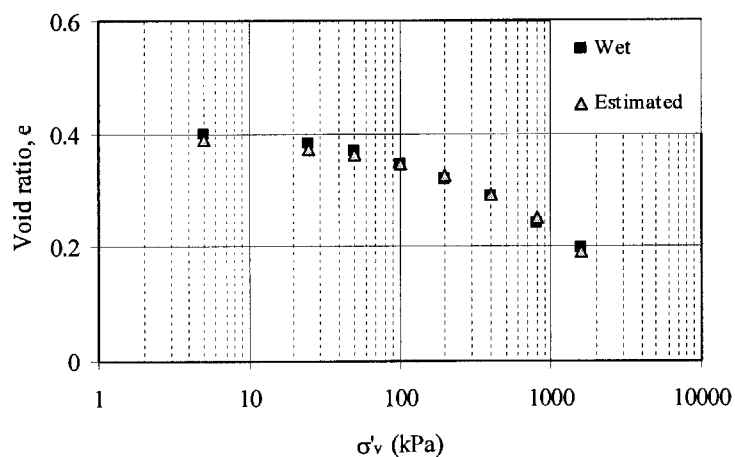
For the case of the wet compression curve in the double-oedometer test, $S_o = 1$ and Equation 4.11 reverts back to Equation 4.10. Therefore Equation 4.11 is a general equation for describing one-dimensional of soils of any degree of saturation as illustrated in Figure 4.22 for $\alpha' = 0.3$, $\beta' = -0.1$ and $\eta = 2.0$. Therefore the compression of soils at different degrees of saturation can be derived from the wet compression curve if η is known. The wetting-induced settlement can then be estimated by the difference in ordinates between the dry

compression curve and the wet compression curves at any applied vertical stress, σ'_v .

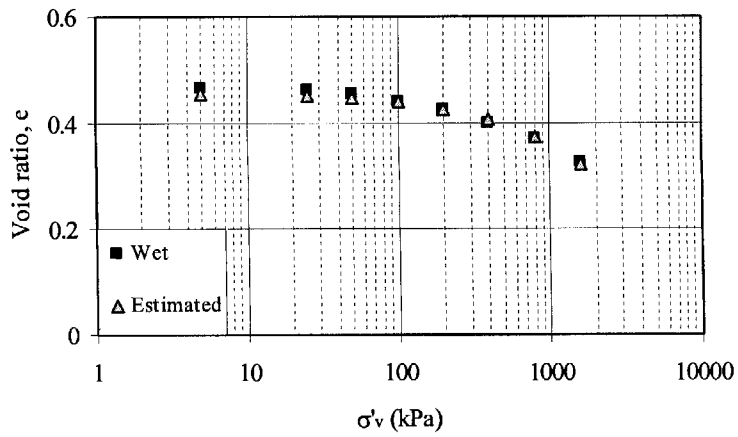
The empirical constants α' , β' and η for the compacted soils in this study are tabulated in Table 4.10. The value of η can be functions of any soil parameter that is characteristic of the soil type, such as Atterberg limits and activity. A linear relationship is obtained when η is plotted against activity as shown in Figure 4.22.



(a) Kaolin

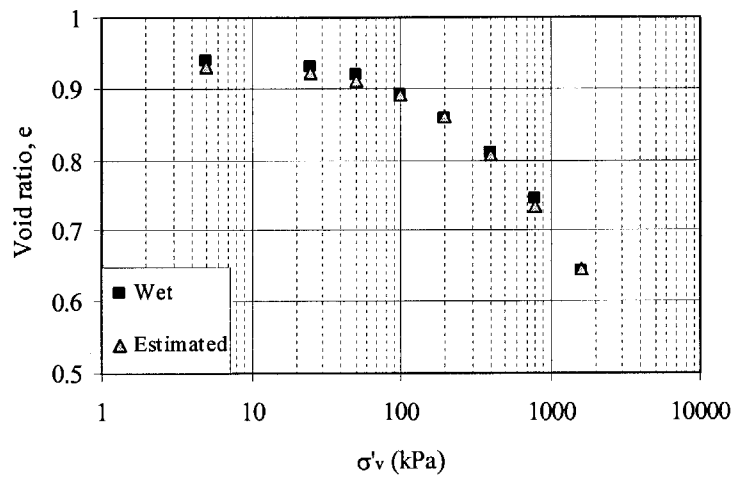


(b) Residual soil A

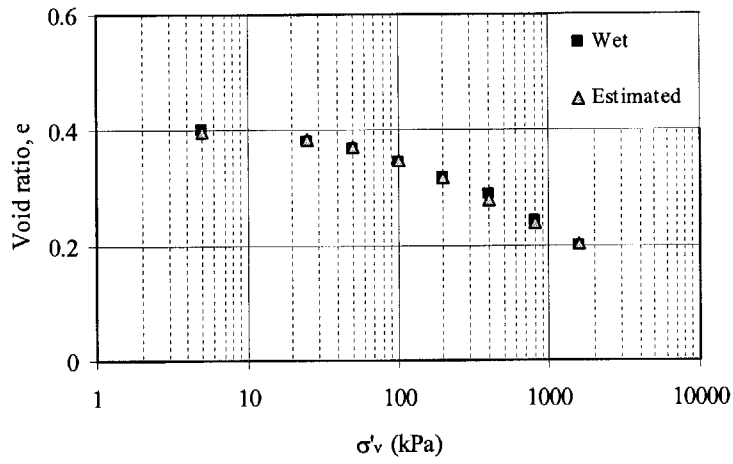


(c) Residual soil B

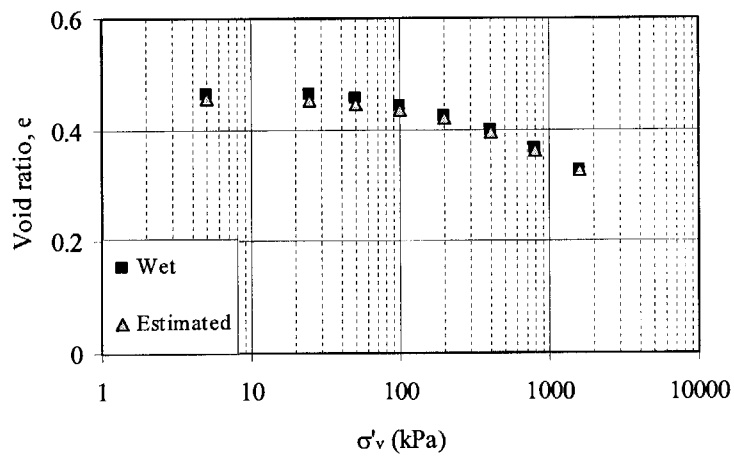
Figure 4.20 Typical compression curves of Equation 4.9 for compacted kaolin, residual soil A and residual soil B



(a) Kaolin



(b) Residual soil A



(c) Residual soil B

Figure 4.21 Typical compression curves of Equation 4.10 for compacted kaolin, residual soil A and residual soil B

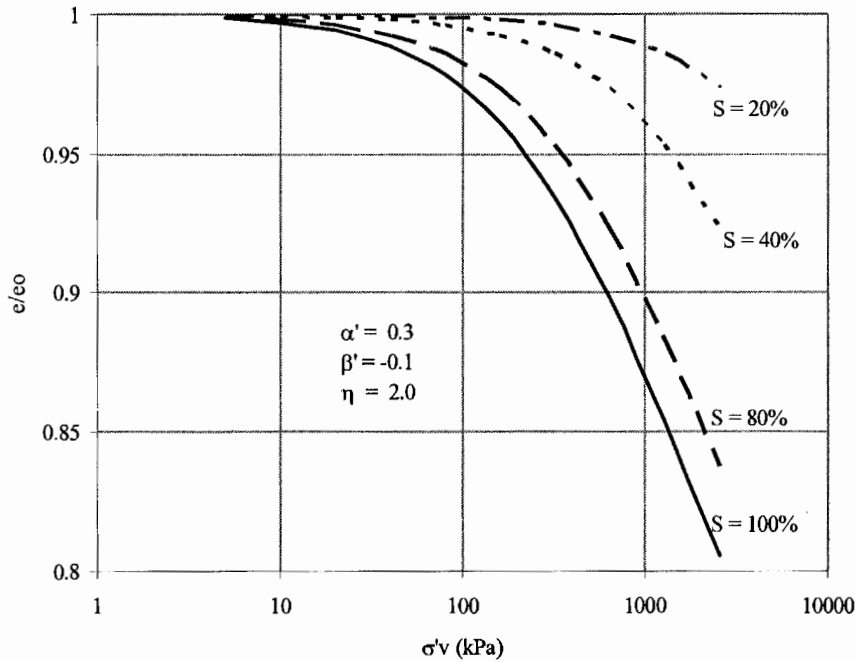


Figure 4.22 Compression curves from Equation 4.11 with different degrees of saturation

Table 4.10 Summary of empirical constants

| Soil Type | Compaction water content (%) | From wet compression curve | | From dry compression curve |
|-----------------|------------------------------|----------------------------|----------|----------------------------|
| | | α' | β' | η |
| Kaolin | 21.5 | 0.05 | -1.0 | 0.21 |
| | 23.6 | 0.16 | -0.29 | |
| | 26.5 | 0.12 | -0.36 | |
| | 28.5 | 0.17 | -0.33 | |
| | 30.1 | 0.14 | -0.33 | |
| Residual soil A | 9.4 | 1.43 | -0.27 | 3.65 |
| | 11.4 | 0.27 | -0.30 | |
| | 13.8 | 0.65 | -0.28 | |
| | 15.3 | 0.42 | -0.28 | |
| | 16.7 | 1.04 | -0.19 | |
| Residual soil B | 9.7 | 2.23 | -0.22 | 2.14 |
| | 12.1 | 0.33 | -0.29 | |
| | 13.2 | 0.28 | -0.20 | |
| | 15.2 | 1.06 | -0.17 | |
| | 17.5 | 0.72 | -0.23 | |

Double-oedometer data from Lawton et al. (1989) and Lim and Miller (2004) are used to verify the applicability of Equation 4.11. The properties of the soils used by Lawton et al. (1989) and Lim and Miller (2004) are summarised in Table 4.11. The values of α' and β' were derived from the wet compression curve and the value of η was obtained from Figure 4.23. The estimated dry compression curves are shown in Figures 4.24 and 4.25. Figures 4.24 and 4.25 showed that the estimated dry compression curves are in good agreement with the experimental dry compression curves. With the wet compression curve and the estimated dry compression curve, the wetting-induced settlement of compacted soils at any applied load can be estimated from the difference in ordinates between the curves.

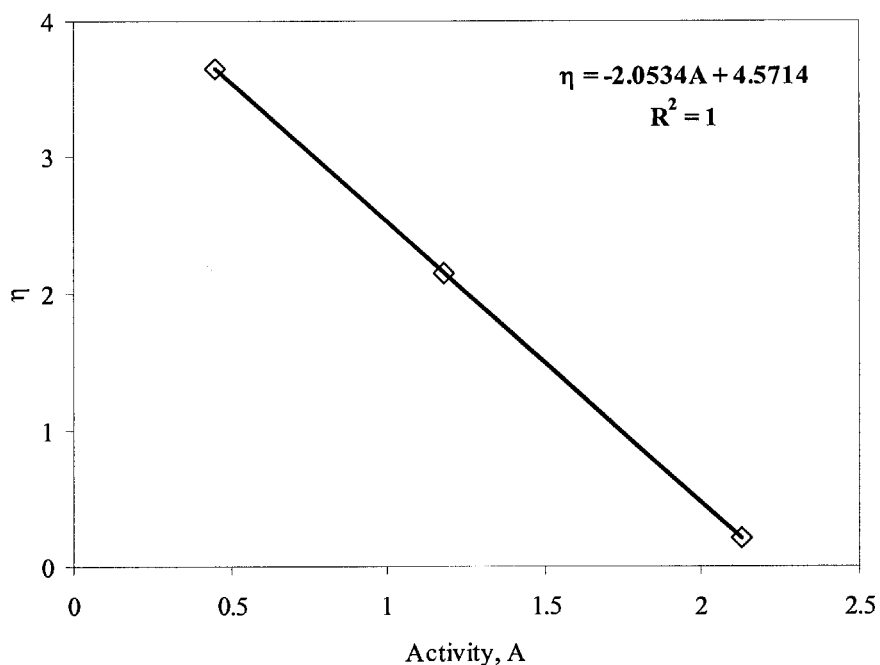


Figure 4.23 Relationship of η with activity A

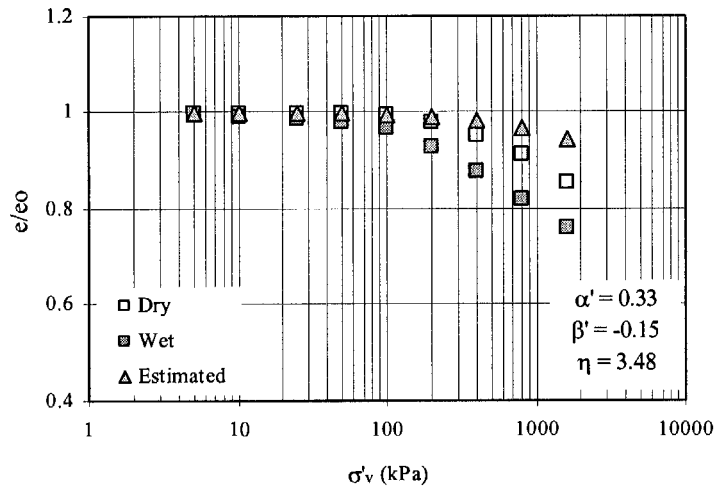
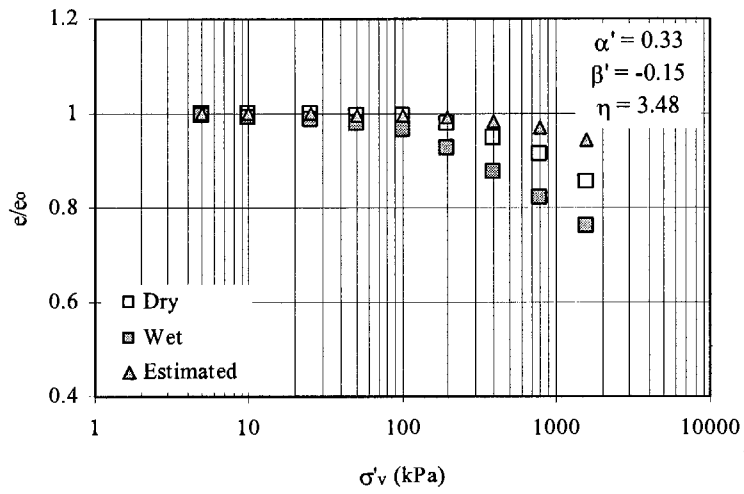


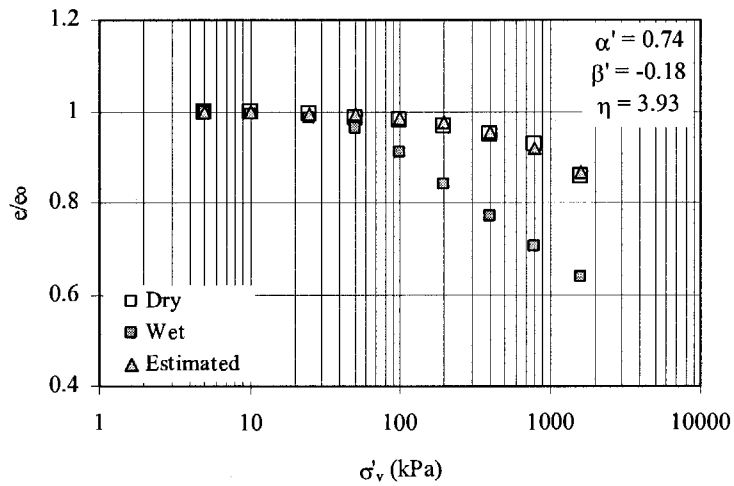
Figure 4.24 Estimation of dry compression curve for Lawton et al. (1989) clayey sand data

Table 4.11 Summary of soil properties from literature

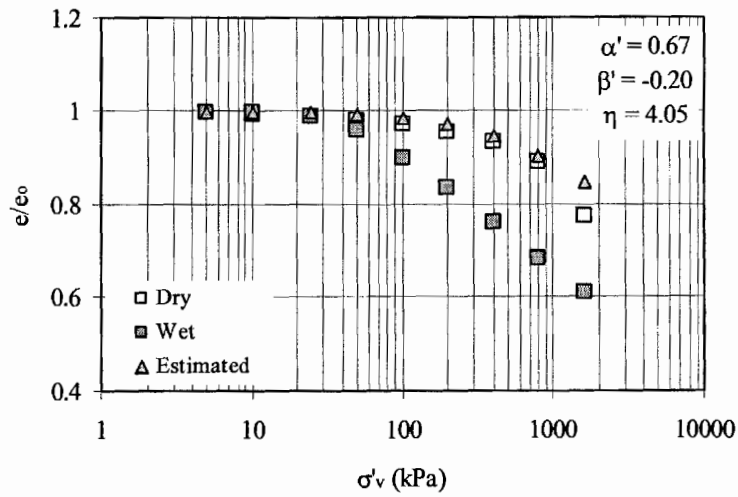
| Reference | Soil type | Soil Properties | |
|-----------------------|-------------------|------------------|------|
| Lawton et al. (1989) | Clayey Sand | Liquid Limit | 34 |
| | | Plasticity Index | 15 |
| | | % clay <2μm | 15 |
| | | Activity | 1.0 |
| | | USCS | SC |
| Lim and Miller (2004) | Minco Silt | Liquid Limit | 28 |
| | | Plasticity Index | 8 |
| | | % clay <2μm | 15 |
| | | Activity | 0.53 |
| | | USCS | CL |
| | Blaine Shale | Liquid Limit | 28 |
| | | Plasticity Index | 10 |
| | | % clay <2μm | 32 |
| | | Activity | 0.31 |
| | | USCS | CL |
| | Hennessey Shale 1 | Liquid Limit | 34 |
| | | Plasticity Index | 13 |
| | | % clay <2μm | 51 |
| | | Activity | 0.25 |
| | | USCS | CL |
| Boggy Shale | Liquid Limit | 45 | |
| | Plasticity Index | 24 | |
| | % clay <2μm | 48 | |
| | Activity | 0.50 | |
| | USCS | CL | |



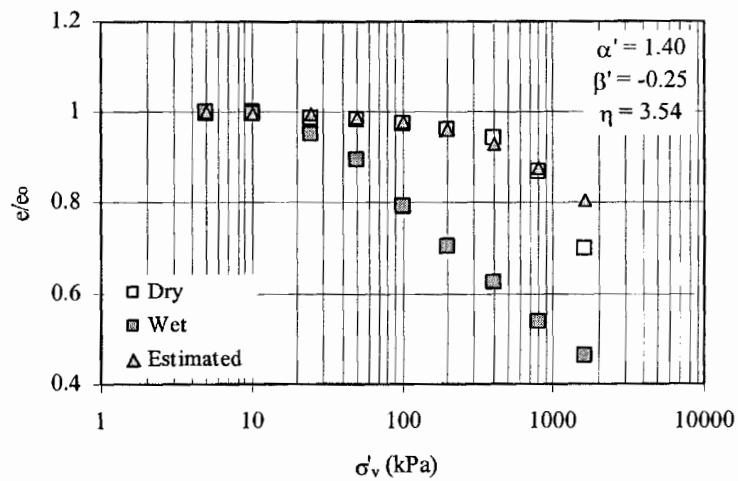
(a) Minco Silt



(b) Blaine Shale



(c) Hennessey Shale 1



(d) Boggy shale

Figure 4.25 Estimation of dry compression curves for Lim and Miller (2004) data

4.4 SUMMARY

Independent suction measurements (total, matric and osmotic) were performed on compacted soil samples at different water contents with four different sodium chloride concentrations. The compaction condition would produce soil samples of different matric suction whereas the sodium chloride solutions will change the osmotic suction of the soil samples. It was shown that total suction is not equal to “matric + osmotic” suctions. When the osmotic suction is low, matric + osmotic suction are lower than the total suction and when the osmotic suction increases, “matric + osmotic” suctions may exceed the total suction. From the matric suction measurement, it was found that the response curves generally show a hyperbola with a relatively fast equilibrium time. Matric suction decreased as the compaction water content increased. Osmotic suction increased as the sodium chloride concentration of the pore water was increased.

The accuracy of suction measurement is dependent on the technique used. In this study, the accuracy of the total and osmotic suction measurements is heavily dependent on the accuracy of the measurement device and the suction level. As suction increases, the error as a function of the measured suction decreases. A plot showing absolute measurement error for various matric and osmotic suction levels was developed for the suction measurement techniques used in this study. Similar plots can also be developed to provide error estimate for other suction measurement techniques.

The understanding of volume change behaviour of the soils is important in geotechnical engineering particularly their consequences in terms of settlement due to wetting. The volume change behaviour of compacted kaolin and residual soils were investigated by conducting single- and double-oedometer tests. Single-oedometer test conducted on compacted kaolin and residual soils specimens showed that the collapse potential for kaolin, residual soil A and residual soil B were 0.2%, 3.3% and 1.4% at a vertical pressure of 200 kPa, respectively. The

compacted kaolin and residual soils in this study were considered to have slight to moderate degree of collapse according to ASTM D5333-92 (1998a). The single- and double-oedometer test results were found to be compatible, indicating that the double-oedometer tests can be used to evaluate wetting-induced settlement.

Apparent preconsolidation pressure and compression index were determined from the double-oedometer test compression curves. The apparent preconsolidation pressure was determined using Casagrande's method (Casagrande, 1936). The apparent preconsolidation pressures for the dry and the wet conditions increase as the dry density increases. The apparent preconsolidation pressure of the dry specimens was found to be higher than the wet specimens. This shows that the higher suction in the dry specimens has the effect of increasing apparent preconsolidation pressure. Plots of the compression index with the initial void ratio of compacted kaolin and two residual soils in this study showed that the compression index increases as the void ratio increases. The compression index of the wet specimens was found to be higher than the corresponding dry specimen. This is again due to the suction in the dry specimens.

A general empirical compression equation for soils that account for soil type, soil structure and degree of saturation was proposed based on the double-oedometer test results. The compression equations have three empirical constants: α' , β' and η . The empirical constants α' and β' are dependent on soil type and soil structure, and can be derived from the wet compression curve. The empirical constant η was found to be a linear function of activity. The wetting-induced settlement of compacted soils at any degree of saturation can thus be obtained from an oedometer test of the wetted compacted soil and using the general compression equation. The proposed empirical compression equation was found to give good agreement with published data.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 INTRODUCTION

Soil suction is an important parameter in unsaturated soils. Soil suction or total suction consists of two components, namely, matric and osmotic suction. Independent measurements of total, matric and osmotic suction showed that total suction is not always equal to the sum of matric and osmotic suctions. In this study, a detailed evaluation of the discrepancy was conducted. Compacted soils are used in many engineering constructions. As compacted soils are unsaturated, wetting can lead to changes in volume especially under applied load. Some soils experienced swelling upon wetting while others experienced collapse upon wetting. In this study, the volume change behaviour of unsaturated soil as it is being wetted was investigated.

5.2 CONCLUSIONS

5.2.1 Suction Measurements

Based on the results of the suction measurement on compacted kaolin and two compacted residual soil samples, the following key conclusions were made:

- i) The results on independent measurement of suction components showed that total suction is not equal to the sum of matric and osmotic suctions. At low osmotic suctions, “matric + osmotic” suctions are lower than total suction. On the other hand, at high osmotic suctions, “matric + osmotic” suctions can exceed total suction. The accuracy of suction measurement is

dependent on the technique and the accuracy of the device used. As suction increases, the error as a function of the measured suction decreases.

- ii) Matric suctions for compacted soils were found to be linear functions of compaction water content and independent of the salinity of the solution used for compaction. As water content increases, matric suction decreases.
- iii) The electrical conductivity of the solution used to compact the soil samples were the same as the electrical conductivity of the extracted pore water using the high pressure squeezing technique. The electrical conductivity of the extracted pore water was not affected by the range of the squeezing pressure used, i.e. 10 to 30 MPa.
- iv) The equation proposed by Leong et al. (2003a) to correct the discrepancy between “matric + osmotic” suctions as total suction was found to be valid only for low osmotic suction.
- v) Plot of absolute measurement error for various matric and osmotic suction levels was developed for the suction measurement techniques used in this study. The plot provides a quantitative evaluation of the differences when matric or osmotic suction is inferred from total suction and osmotic or matric suction measurement.

5.2.2 Volume Change Behaviour

Based on the results of the oedometer tests on compacted kaolin and two compacted residual soil samples, the following key conclusions were made:

- i) Single-oedometer tests were conducted in accordance to ASTM D5333-93 (ASTM, 1998a) on compacted kaolin and residual soil samples. According

to the classification of collapse index given in ASTM D5333-92 (ASTM, 1998a), the degree of collapse of the compacted soils is considered slight to moderate at a vertical pressure of 200 kPa.

- ii) The single-oedometer tests showed that using double-oedometer tests to estimate wetting-induced settlement is valid.
- iii) The apparent preconsolidation pressures of compacted soil samples increase as the dry density increases. This is because as the dry density increases, the void ratio decreases with the result that the microforces at the particle contacts increase. Therefore the apparent preconsolidation pressure increases when the dry density increases.
- iv) The apparent preconsolidation pressure of the dry specimens was found to be higher than the apparent preconsolidation pressure of the wet specimens. The presence of suction in the dry specimens increases the microforces of the particle contacts and therefore the apparent preconsolidation pressure of the dry specimen is higher than the corresponding wet specimen
- v) The compression indices of kaolin, residual soils A and B were found to decrease as dry density increases. As dry density increases, the soil particles are closer together and the soil specimen becomes stiffer. Therefore the compression index decreases. The compression index of the wet specimens was found to be higher than the compression index of the corresponding dry specimen. The reduction of suction in the wet specimen decreases the microforces of the particle contacts and therefore the soil becomes more compressible.
- vi) An empirical compression equation was suggested to describe the wet compression curve of compacted soils. The empirical compression

equation with consideration of initial degree of saturation and activity can be used to estimate the dry compression curve. With the wet compression curve and the estimated dry compression curve, the wetting-induced settlement of compacted soils can be estimated. Comparison with published data showed good performance of the empirical compression equation.

5.3 RECOMMENDATIONS

Based on the results and findings, the following recommendations are suggested for future research:

- i) The influence of salt concentration of the pore water in the soil and the water in the water reservoir of the null-type axis translation apparatus on the matric suction measurement was not investigated. This should be investigated in future research.
- ii) The effect of squeezing pressure on the salt concentration of the extracted pore water was not fully investigated. This could be addressed in future research.
- iii) Plot of the measurement error for suction measurement techniques developed in this study should be validated with more tests and other suction measurement techniques.
- iv) The volume change behaviour is a function of two stress state variables: net normal stress ($\sigma - u_a$) and matric suction ($u_a - u_w$). In the oedometer tests, u_a is at atmospheric, i.e. 0 kPa and u_w is dependent on compaction condition. In this study, u_w was not measured. It would be advantageous if

the u_w of the compacted soil specimens was also measured during the oedometer test.

- v) An empirical compression equation was suggested to describe the compression curve of compacted soils that account for soil type, soil structure and degree of saturation. The applicability of the empirical compression proposed in this study need to be verified with more double-oedometer test data and for soils exhibiting a greater volume change on wetting.

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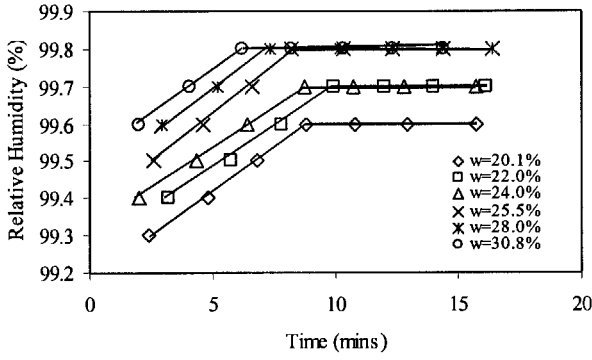
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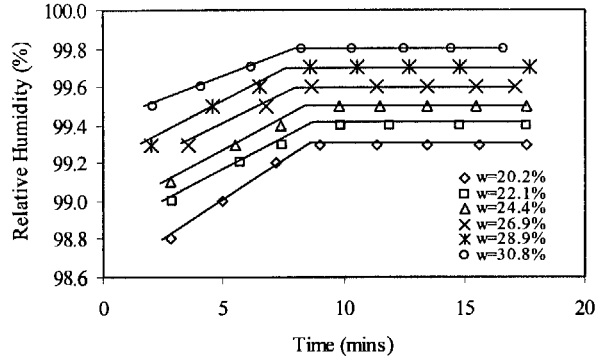
APPENDIX A

**Plots of relative humidity reading with time
for compacted kaolin, residual soil A and residual soil B**

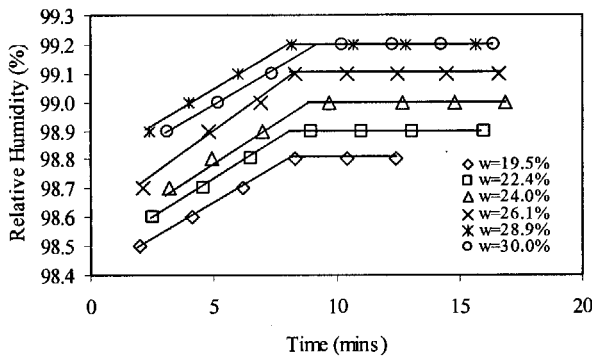
A.1 Kaolin



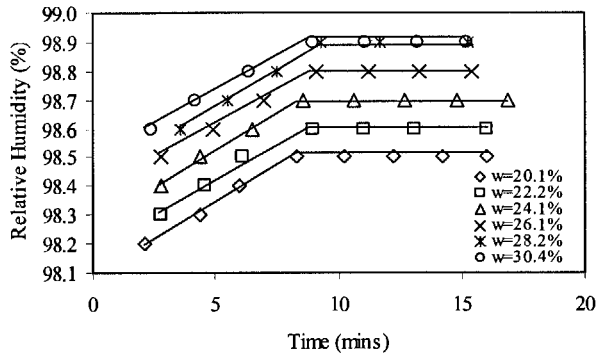
(a) Tap Water



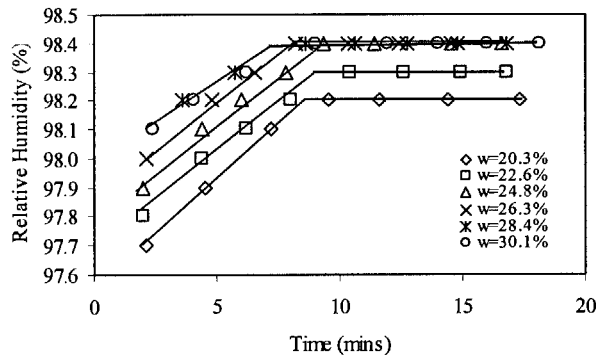
(b) 5.8 g/l NaCl solution



(c) 14.0 g/l NaCl solution

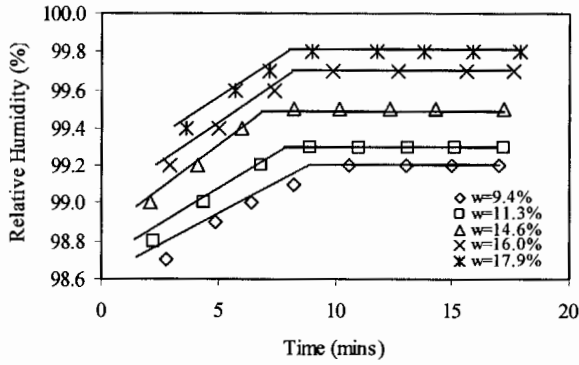


(d) 20.5 g/l NaCl solution

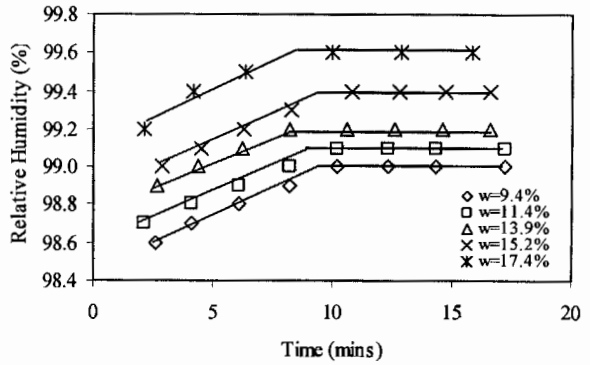


(e) 32.0 g/l NaCl solution

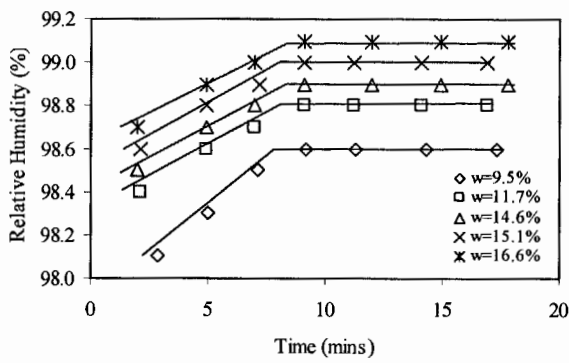
A.2 Residual Soil A



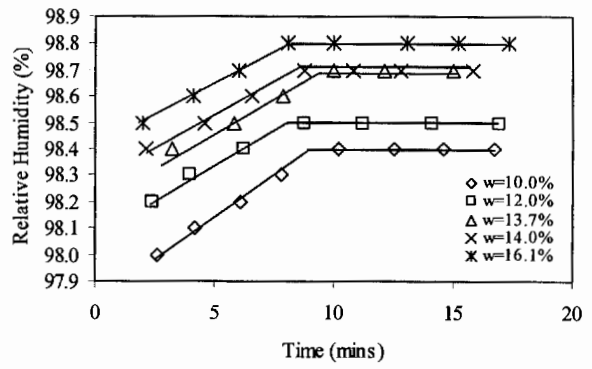
(a) Tap Water



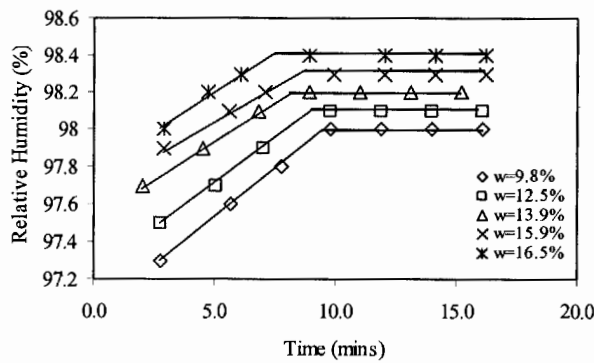
(b) 7.6 g/l NaCl solution



(c) 16.0 g/l NaCl solution

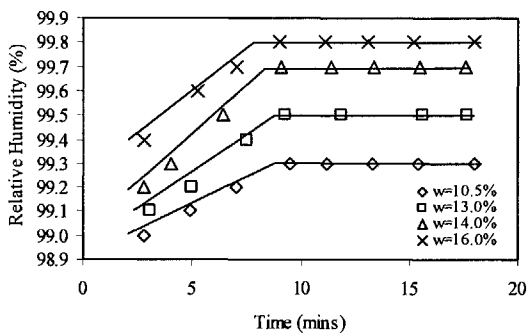


(d) 23.5 g/l NaCl solution

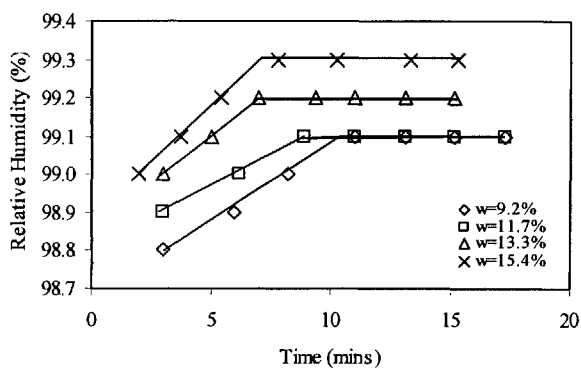


(e) 32.5 g/l NaCl solution

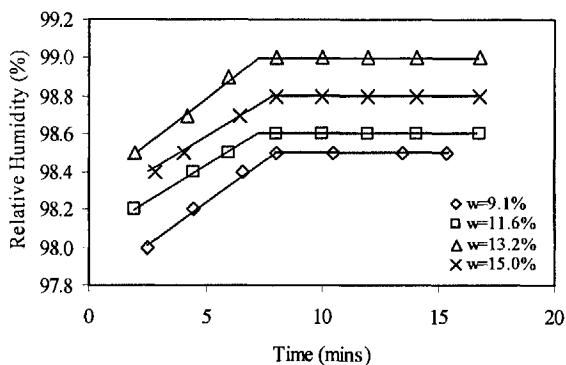
A.3 Residual Soil B



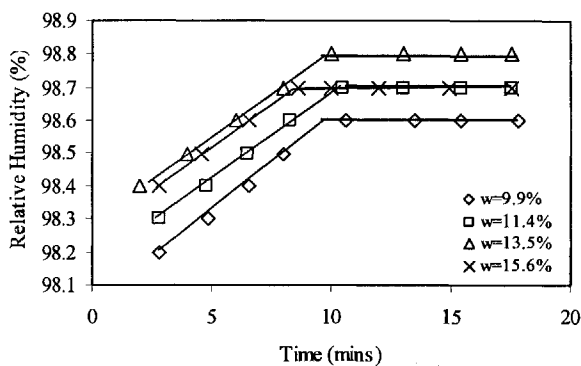
(a) Tap Water



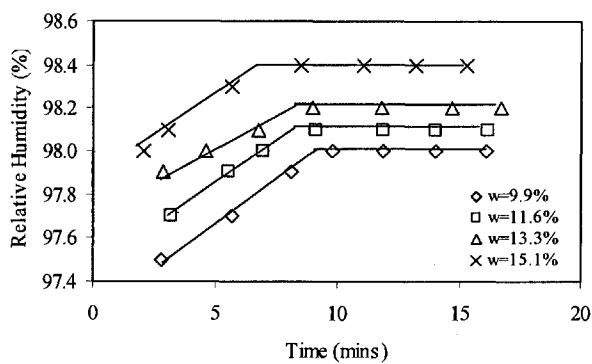
(b) 7.6 g/l NaCl solution



(c) 16.0 g/l NaCl solution



(d) 23.5 g/l NaCl solution

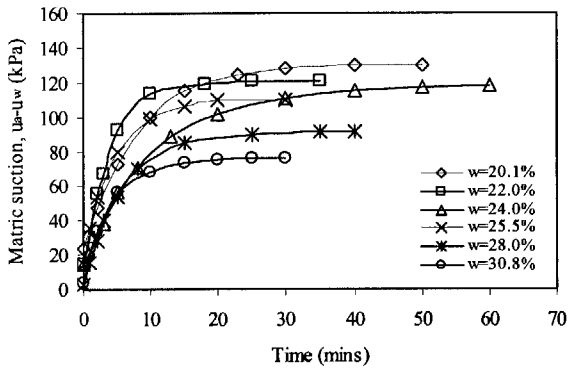


(e) 32.5 g/l NaCl solution

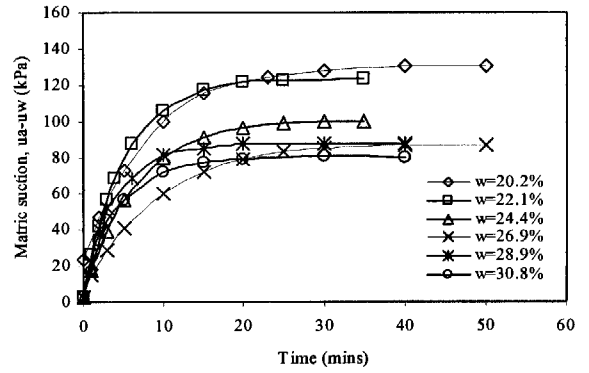
APPENDIX B

**Plots of matric suction against time
for compacted kaolin, residual soil A and residual soil B**

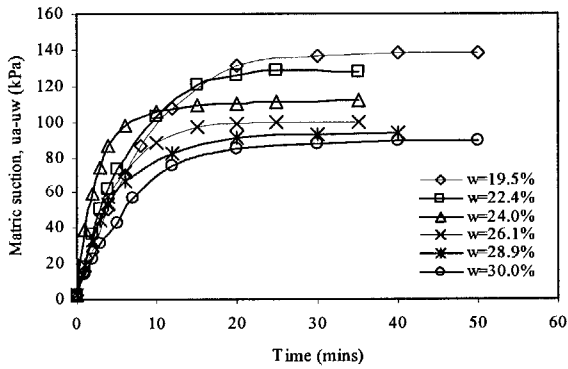
B.1 Kaolin



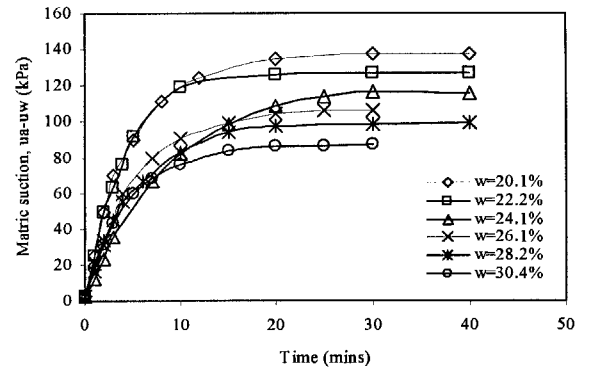
(a) Tap Water



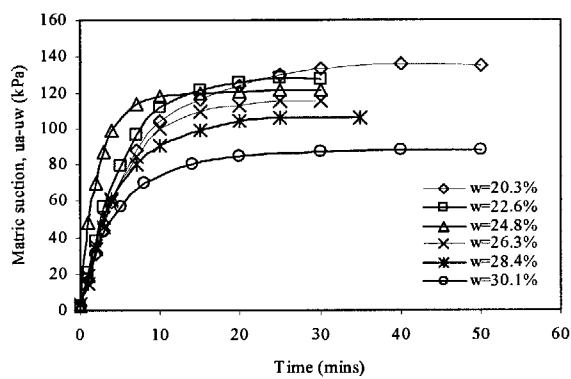
(b) 5.8 g/l NaCl solution



(c) 14.0 g/l NaCl solution

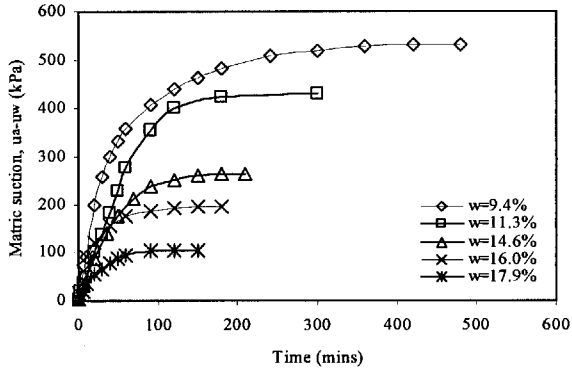


(d) 20.5 g/l NaCl solution

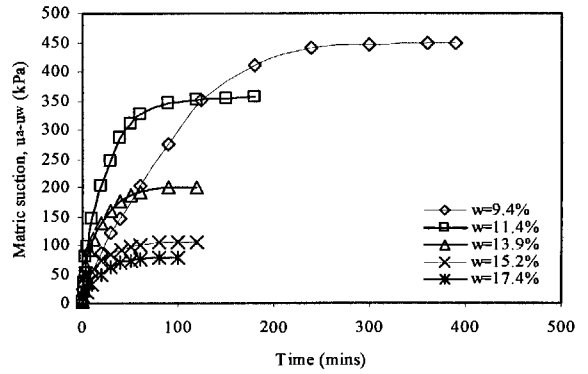


(e) 32.0 g/l NaCl solution

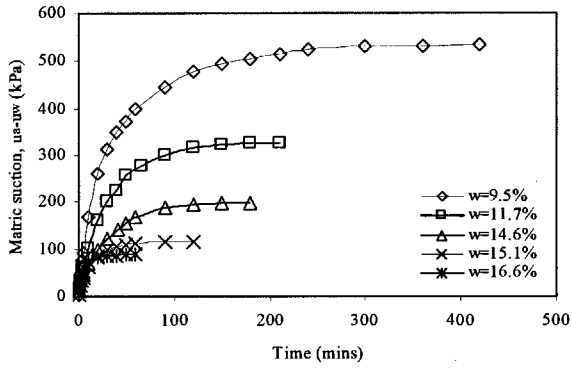
B.2 Residual Soil A



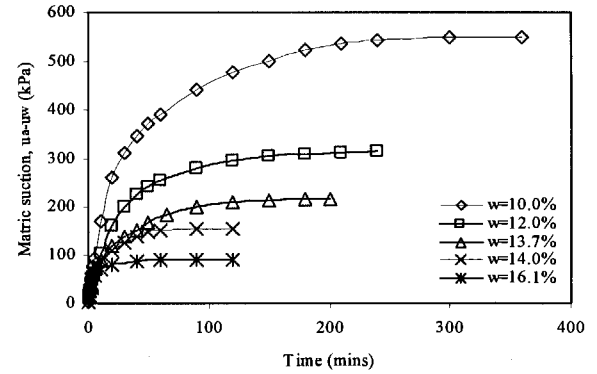
(a) Tap Water



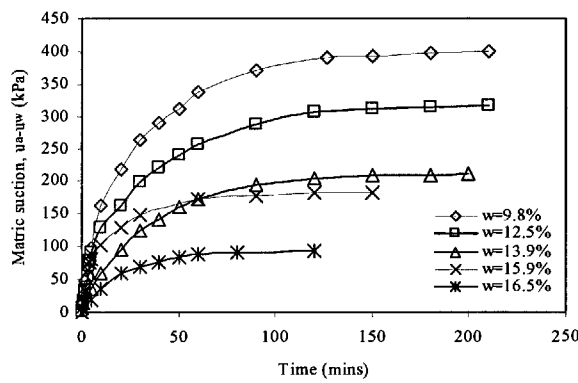
(b) 7.6 g/l NaCl solution



(c) 16.0 g/l NaCl solution

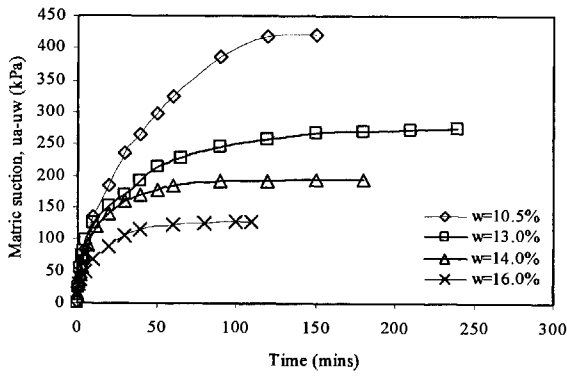


(d) 23.5 g/l NaCl solution

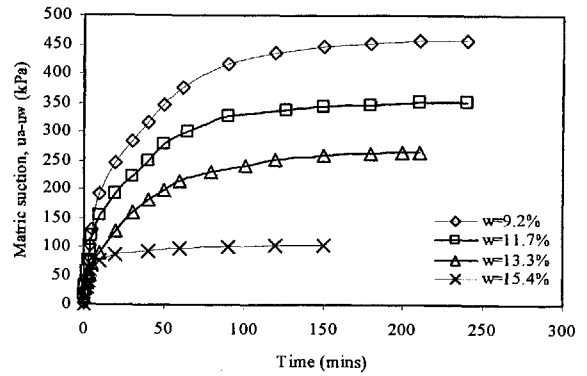


(e) 32.5 g/l NaCl solution

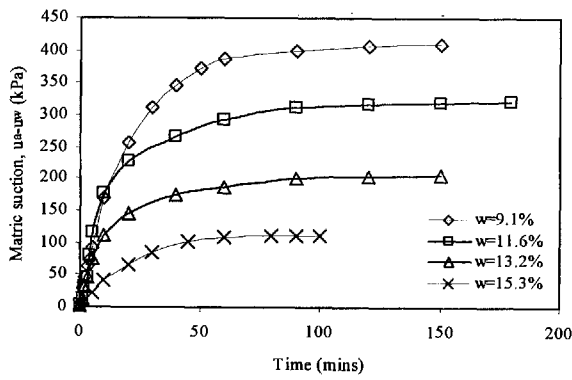
B.3 Residual Soil B



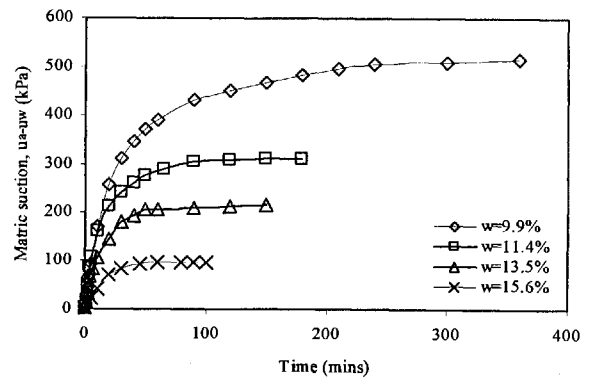
(a) Tap Water



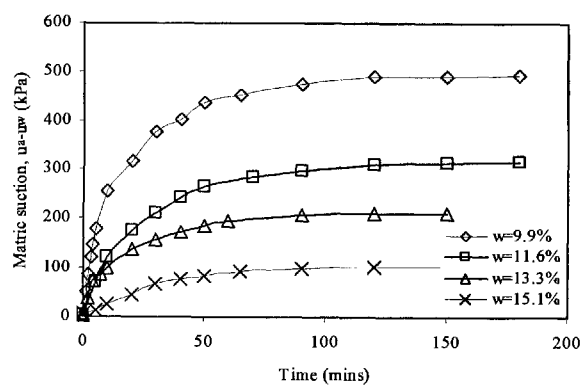
(b) 7.6 g/l NaCl solution



(c) 16.0 g/l NaCl solution



(d) 23.5 g/l NaCl solution

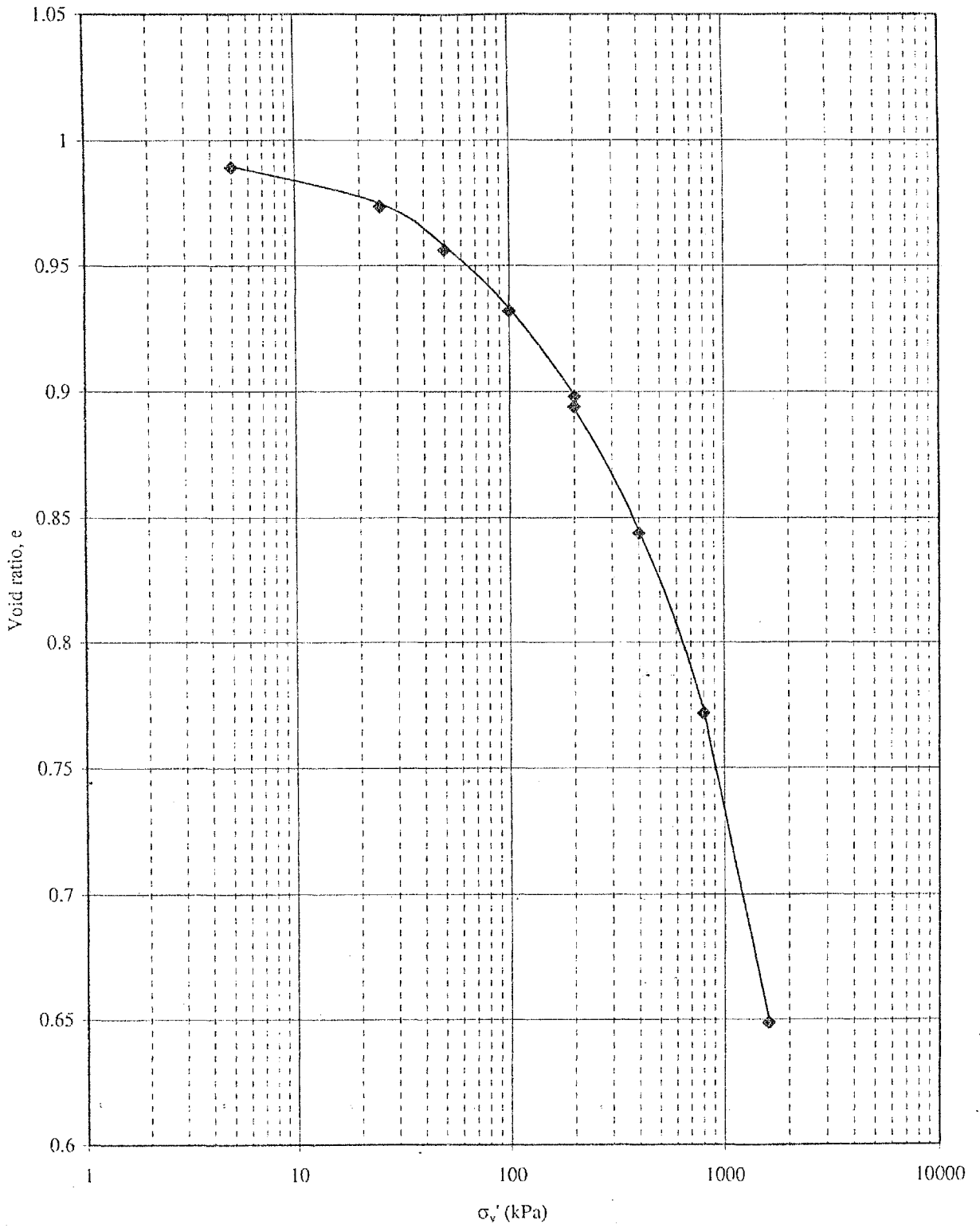


(e) 32.5 g/l NaCl solution

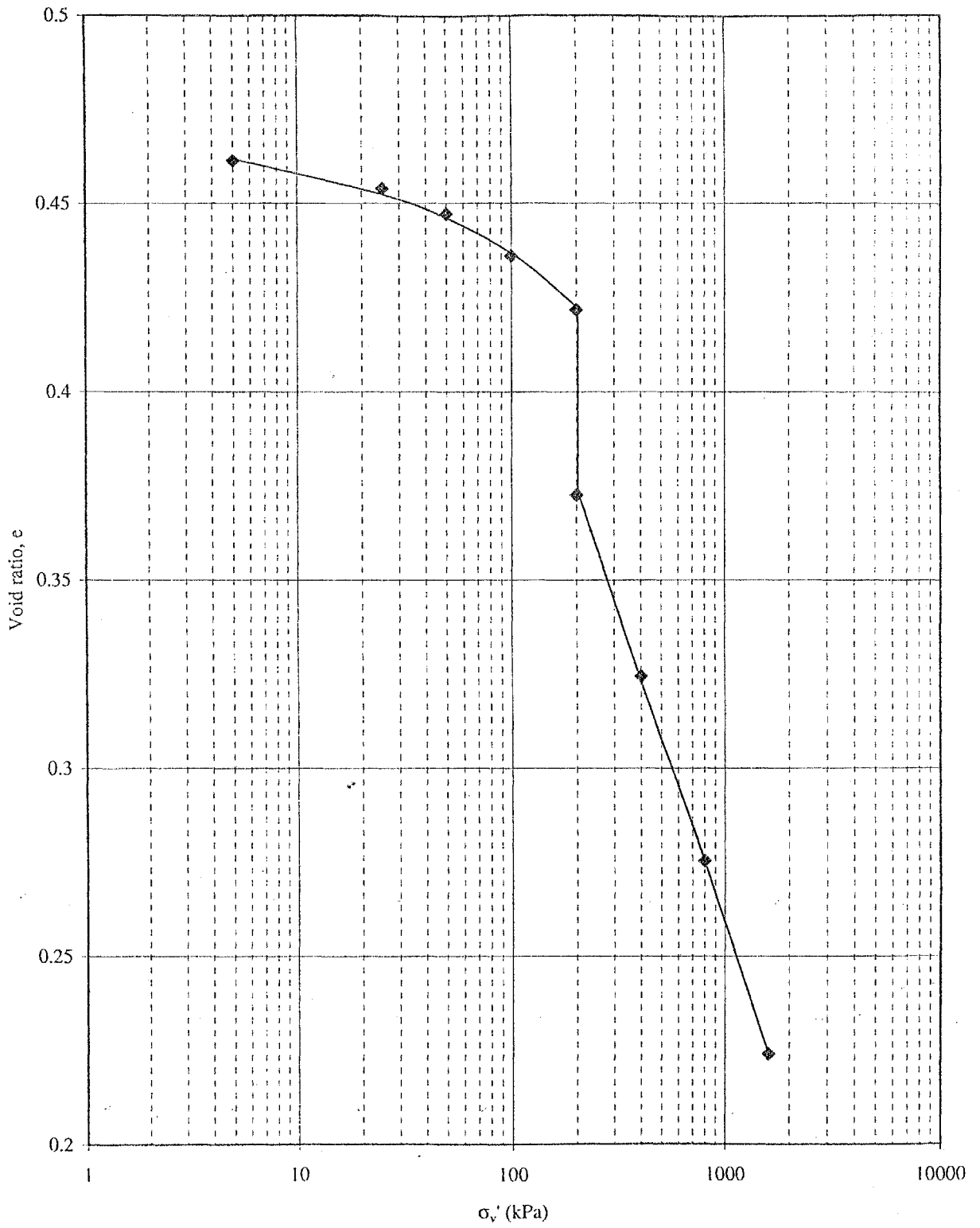
APPENDIX C

Single-oedometer tests for compacted kaolin, residual soil A and residual soil B

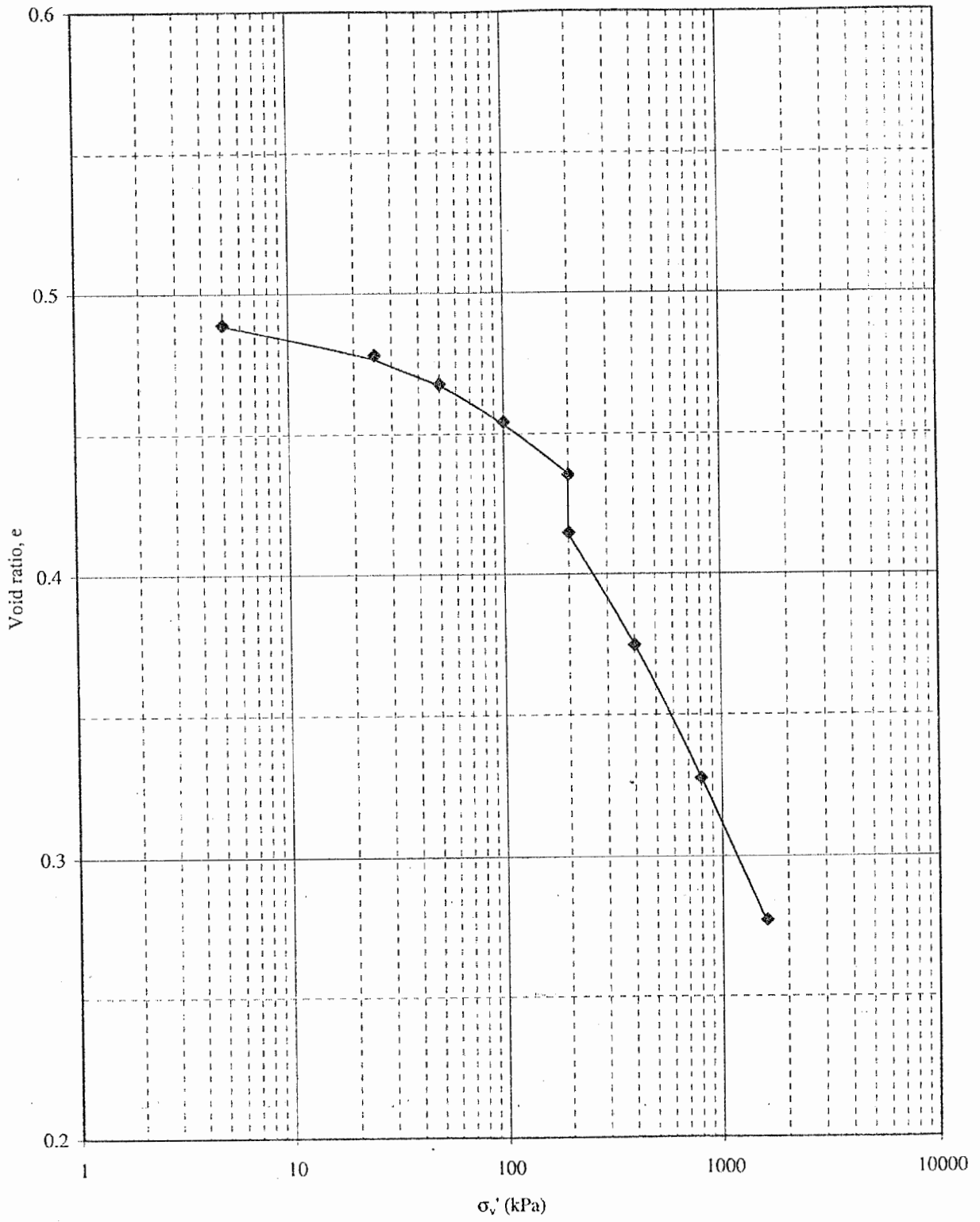
e-log P curve for compacted kaolin
(w=28.5%)



e-log P curve for compacted residual soil A
(w=9.4%)



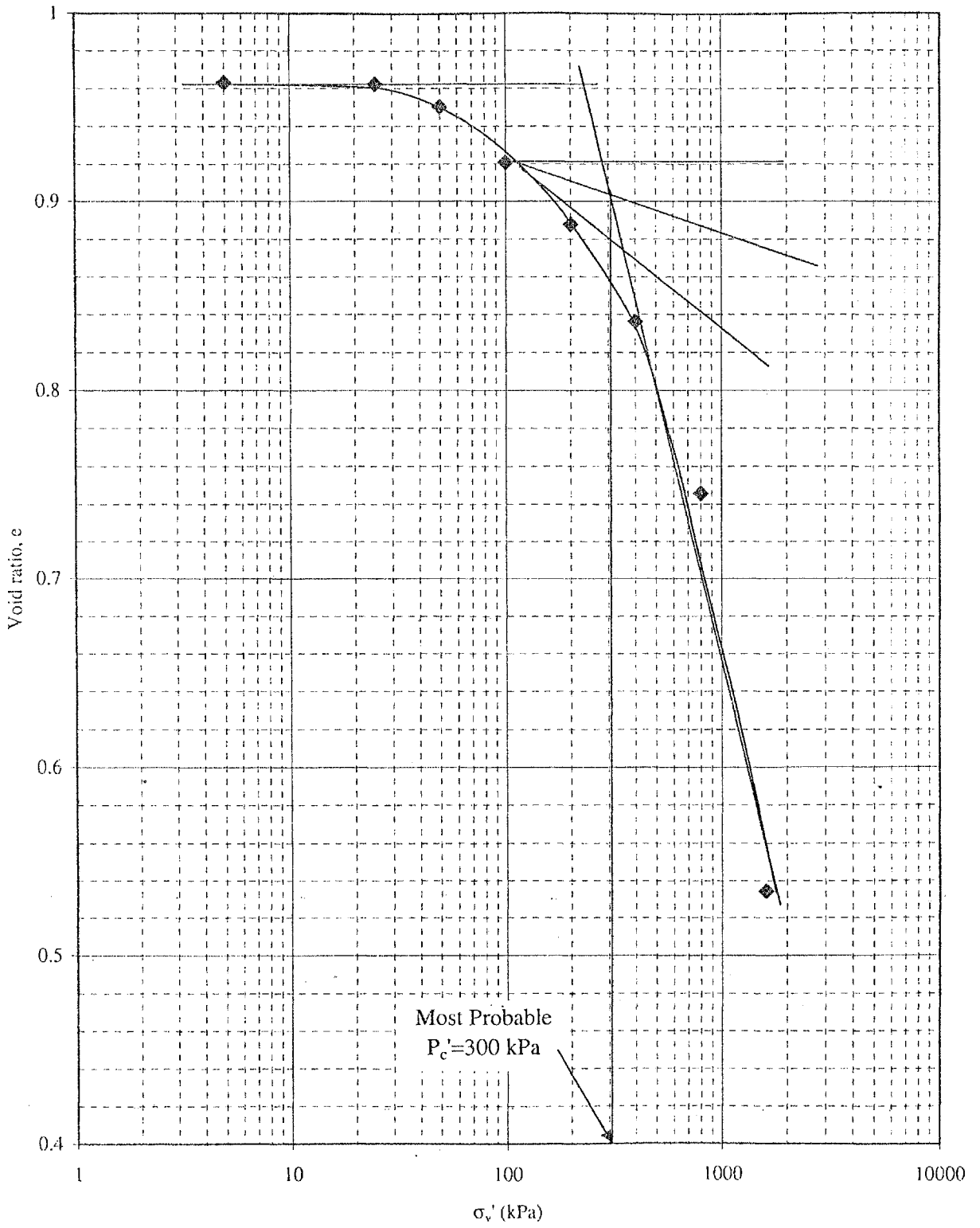
e-log P curve for compacted residual soil B
(w=12.1%)



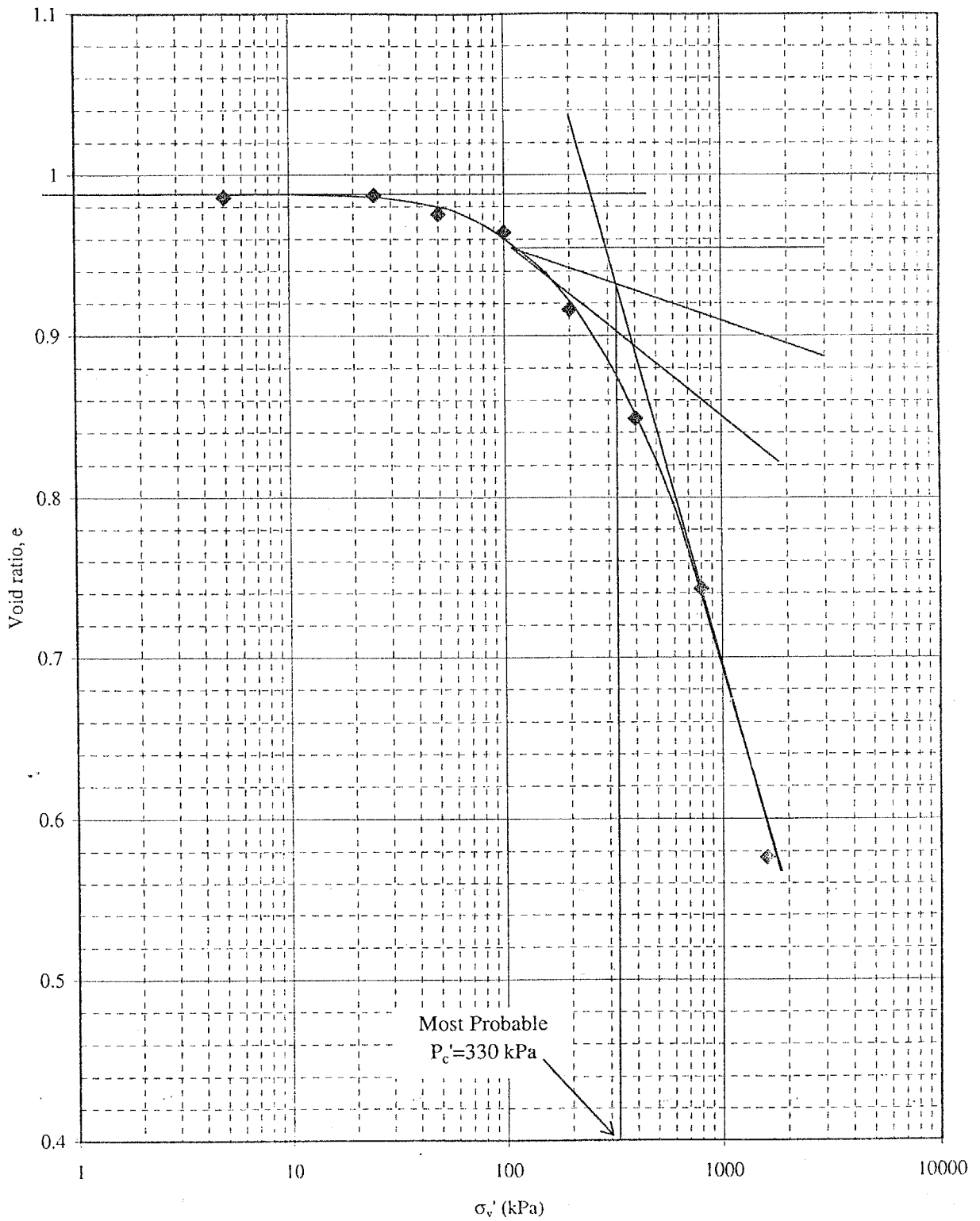
APPENDIX D

Double-oedometer tests for compacted kaolin, residual soil A and residual soil B

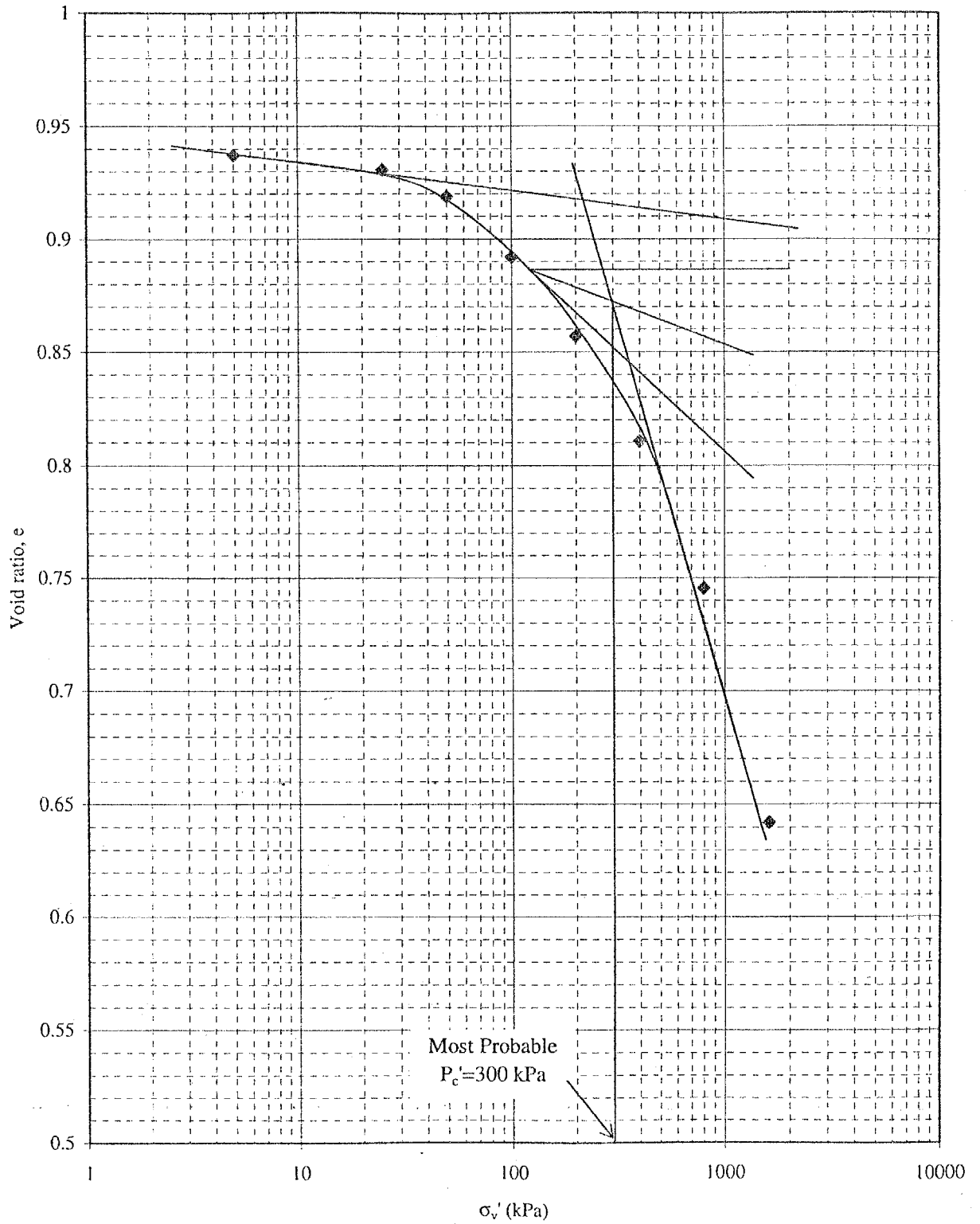
e-log P curve for compacted kaolin
(wet, $w=21.5\%$)



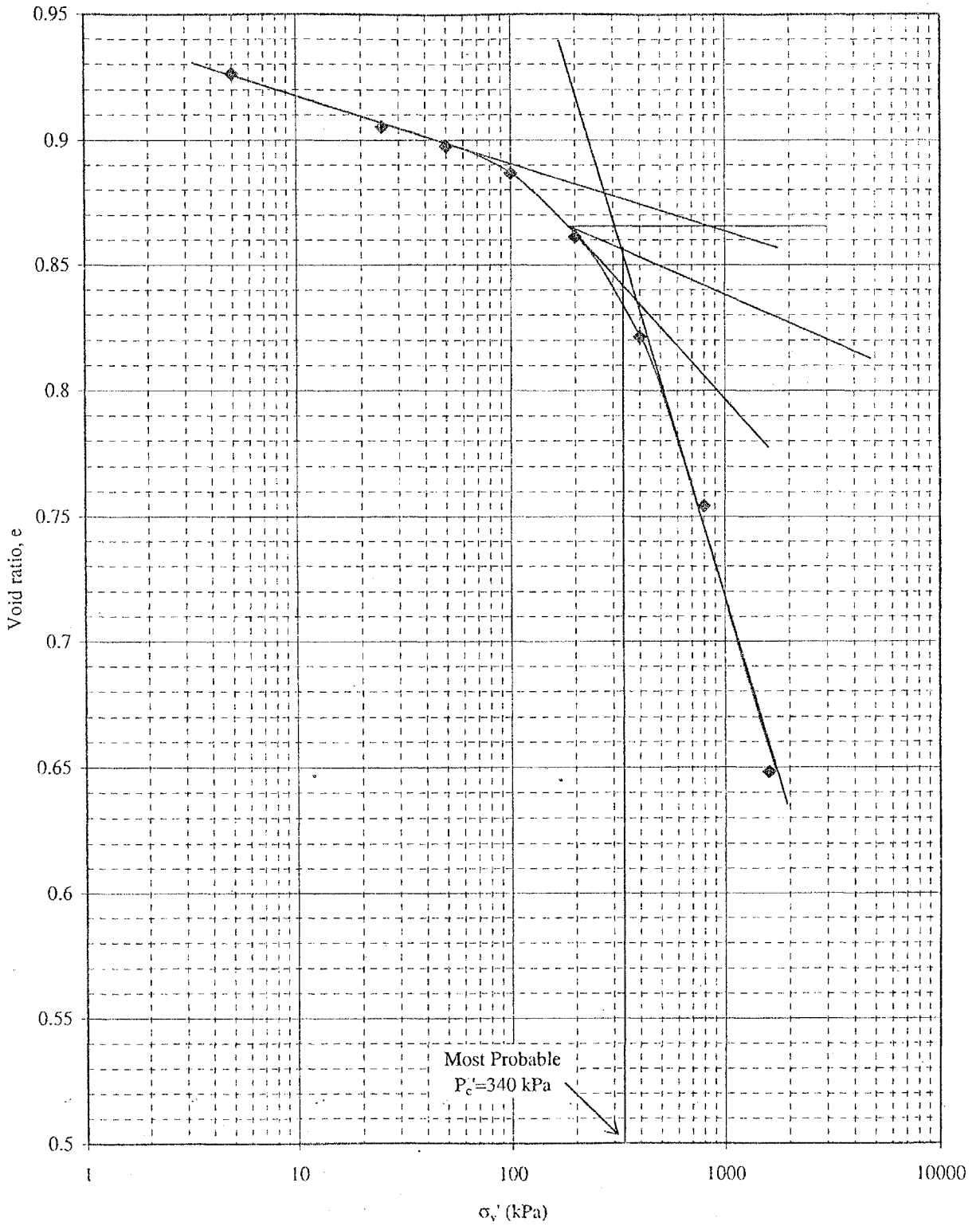
**e-log P curve for compacted kaolin
(dry, w=21.5%)**



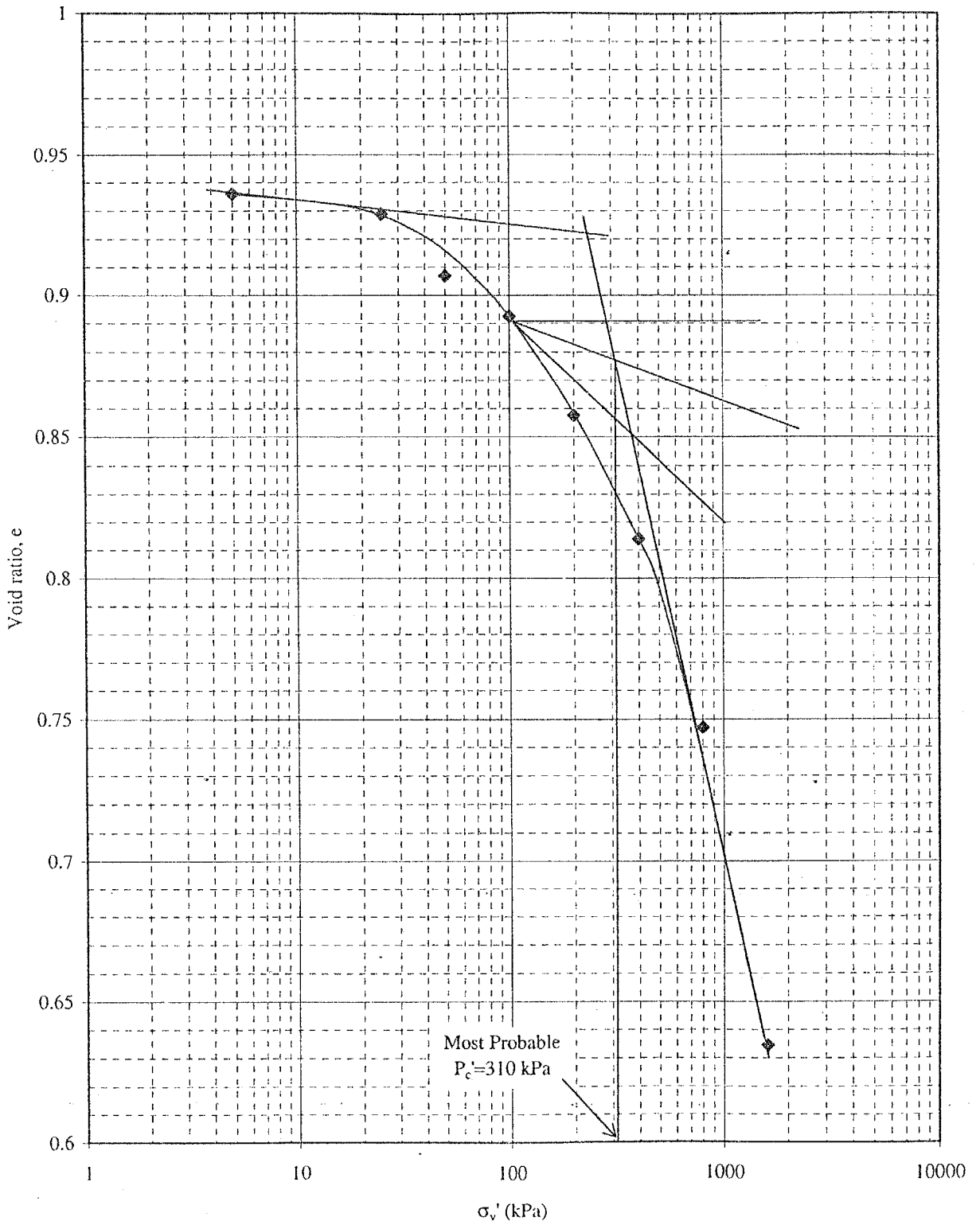
e-log P curve for compacted kaolin
(wet, w=23.6%)



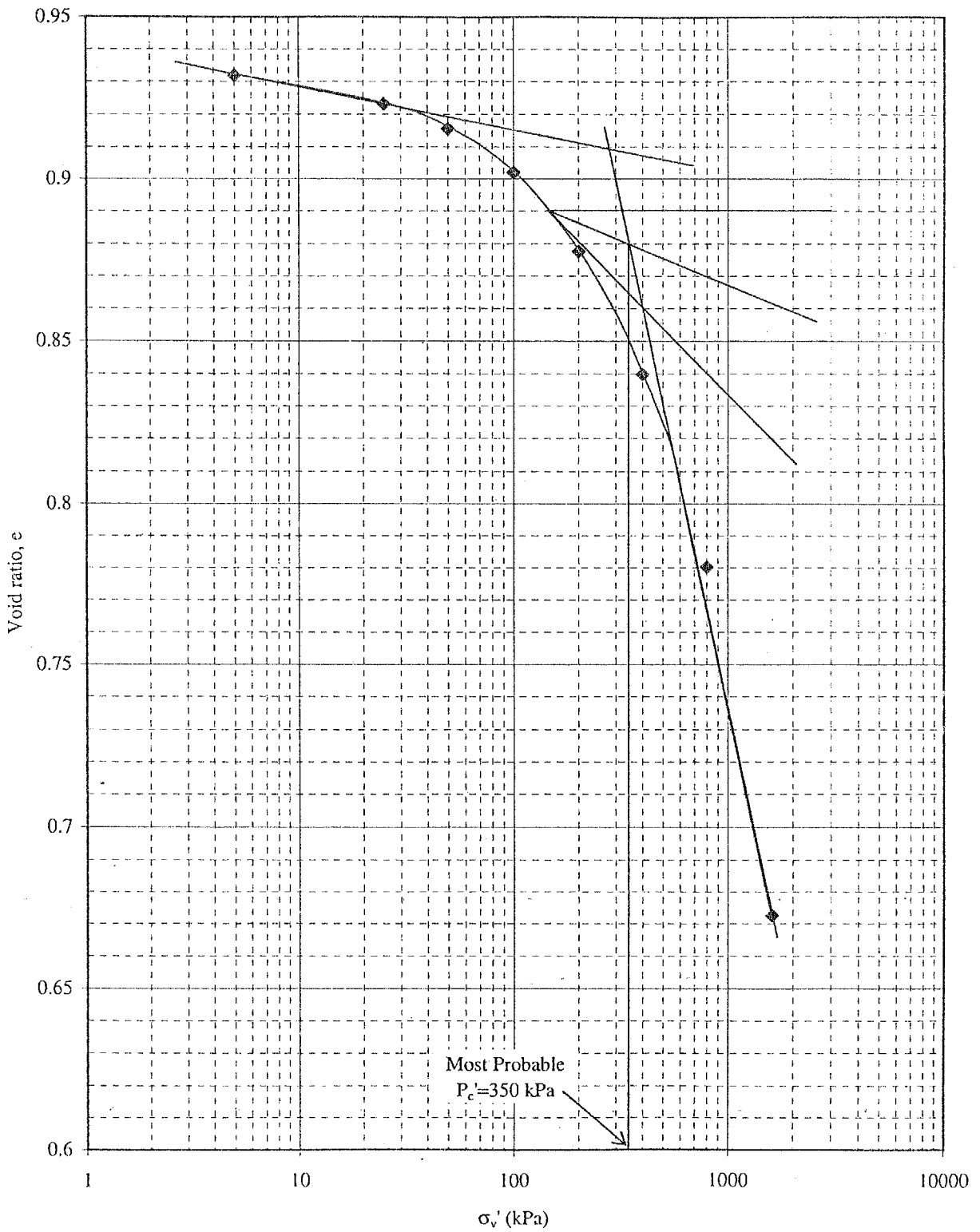
e-log P curve for compacted kaolin
(dry, w=23.6%)



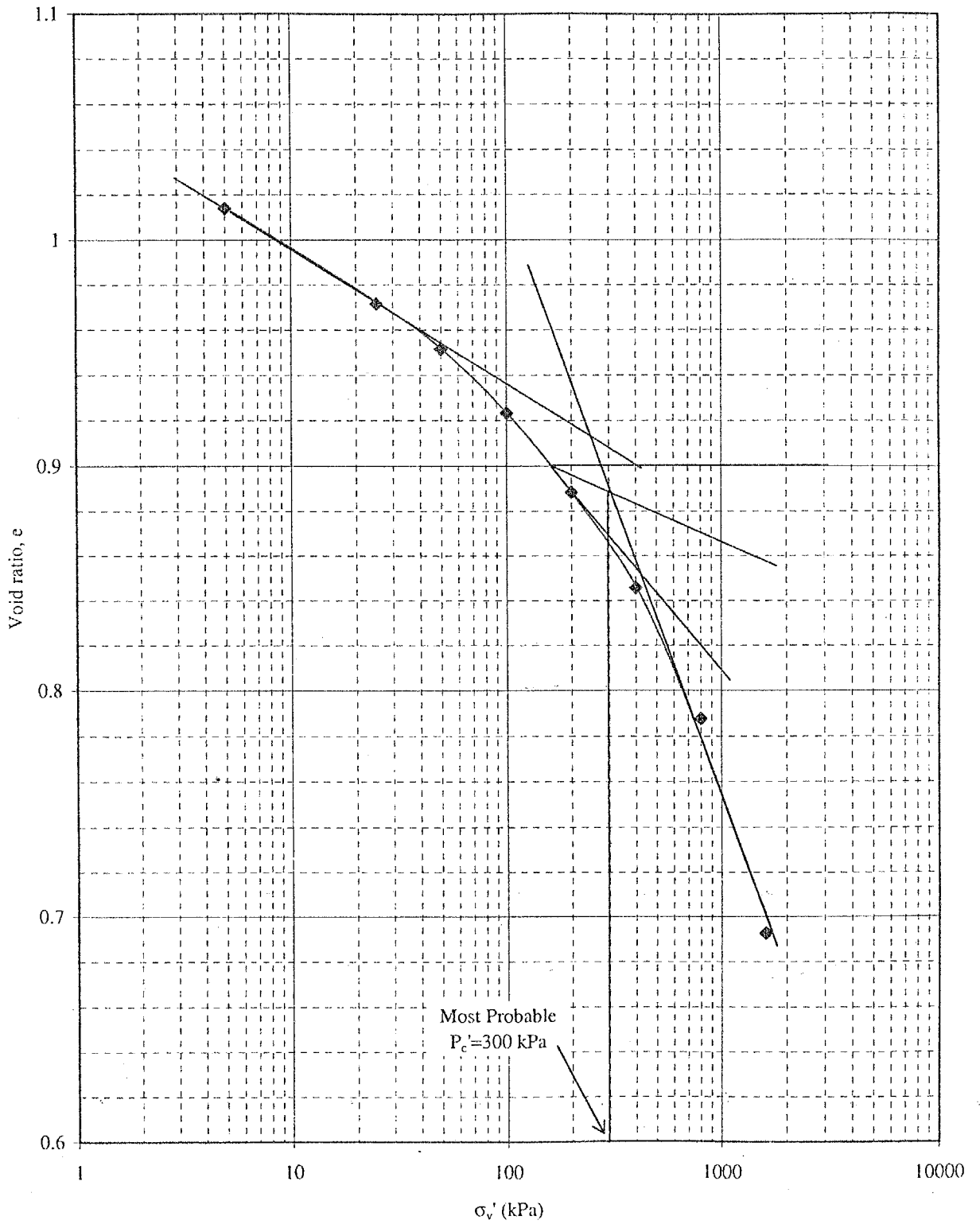
e-log P curve for compacted kaolin
(wet, $w=26.5\%$)



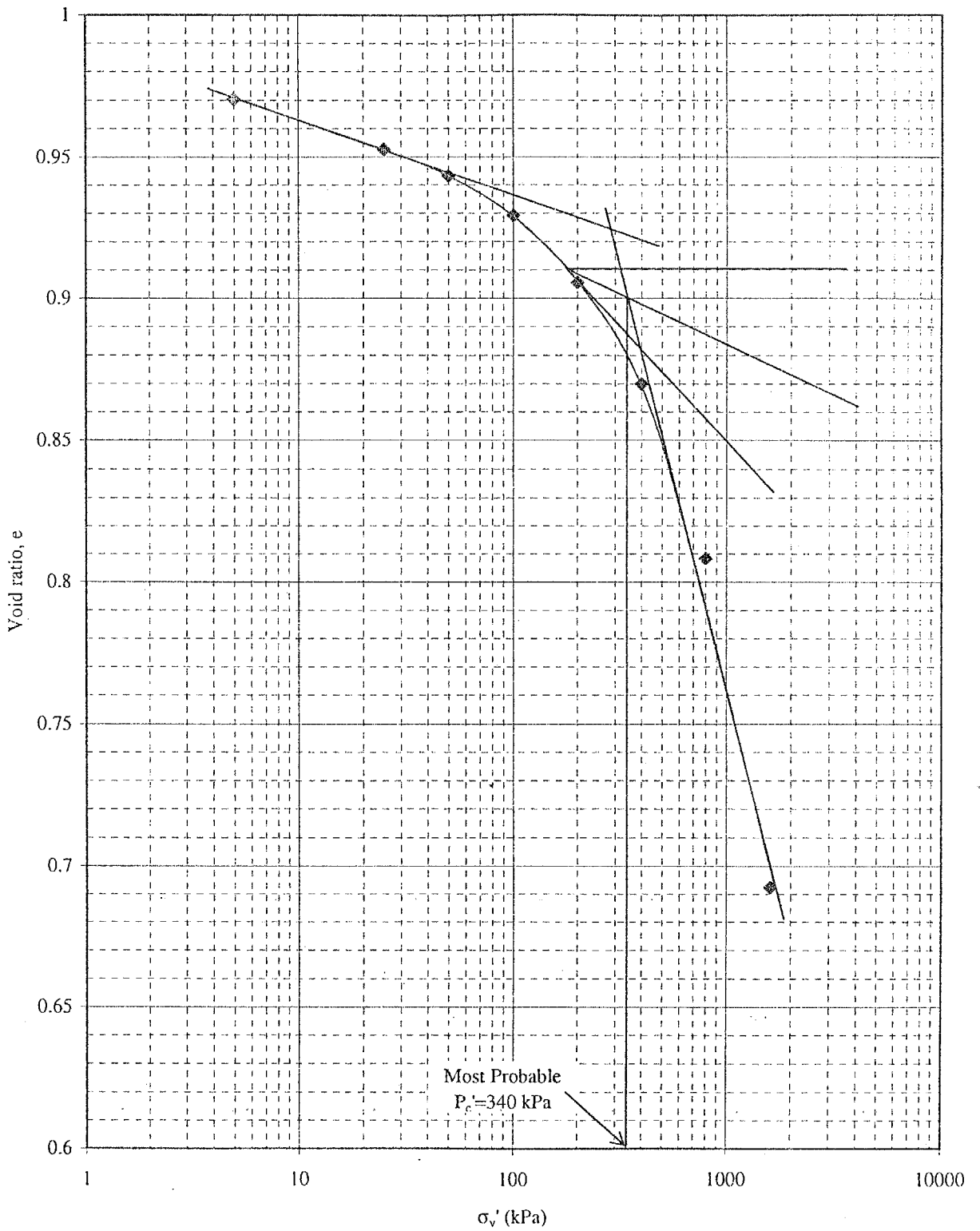
e-log P curve for compacted kaolin
(dry, w=26.5%)



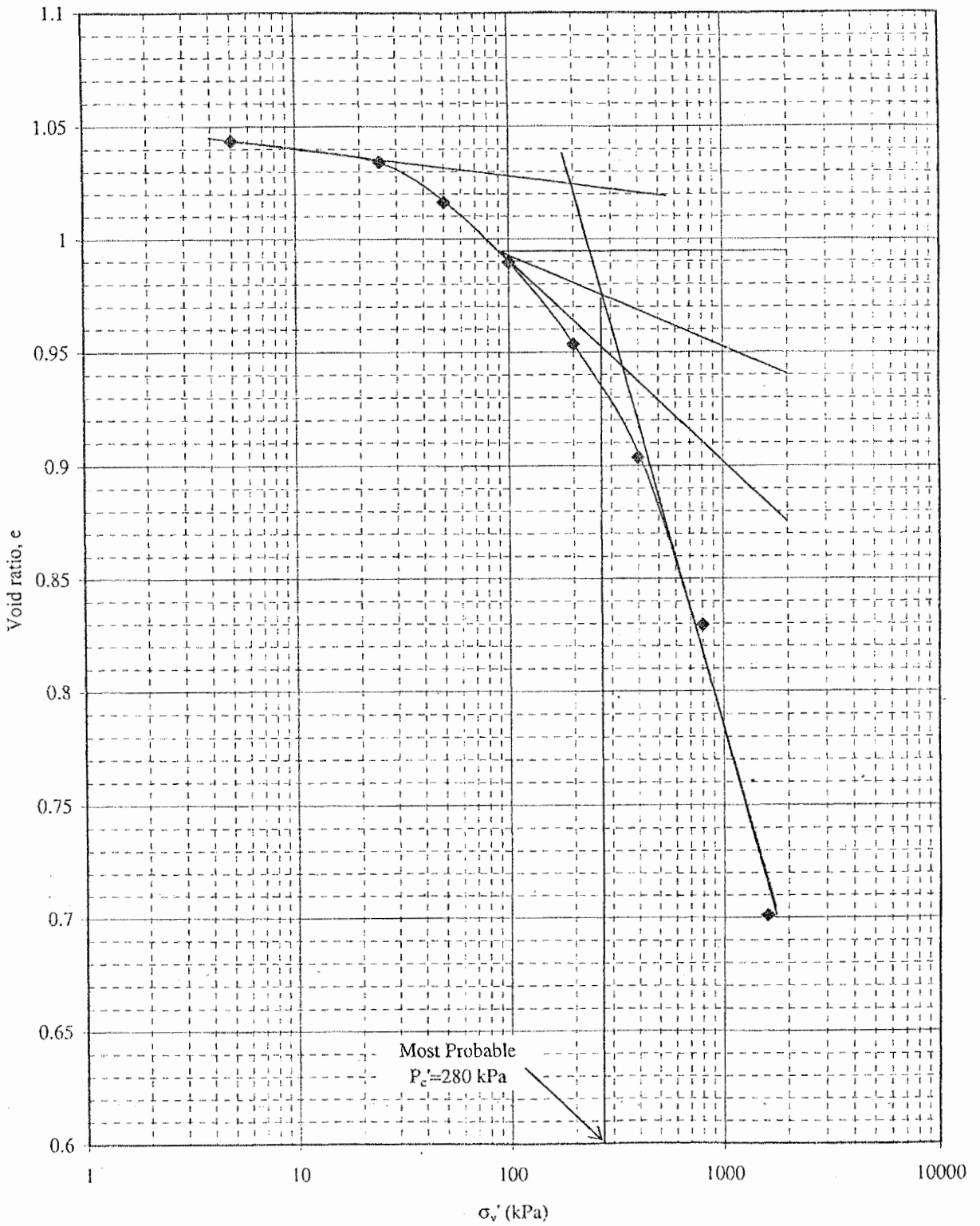
e-log P curve for compacted kaolin
(wet, $w=27.3\%$)



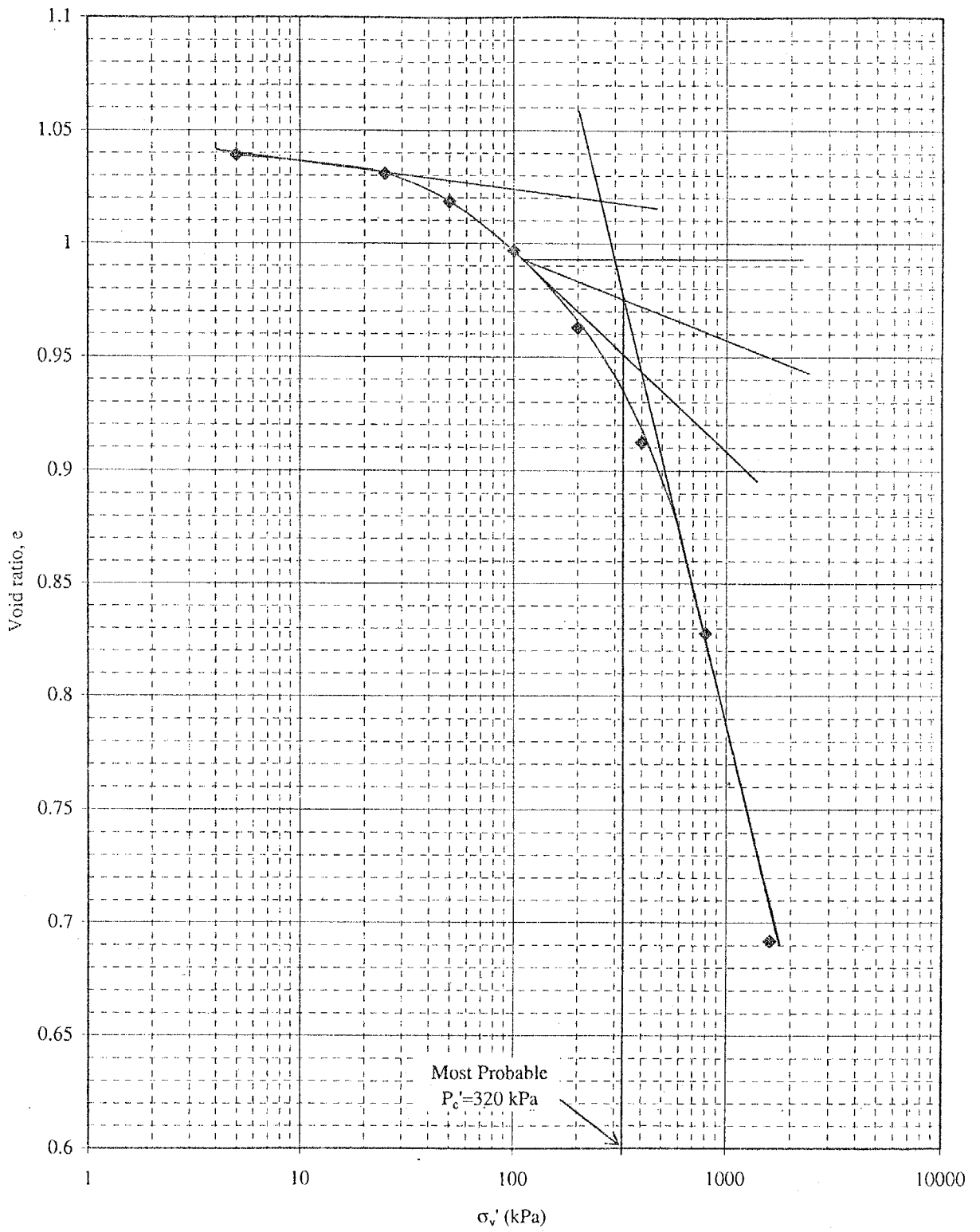
e-log P curve for compacted kaolin
(dry, $w=27.3\%$)



**e-log P curve for compacted kaolin
(wet, w=30.1%)**

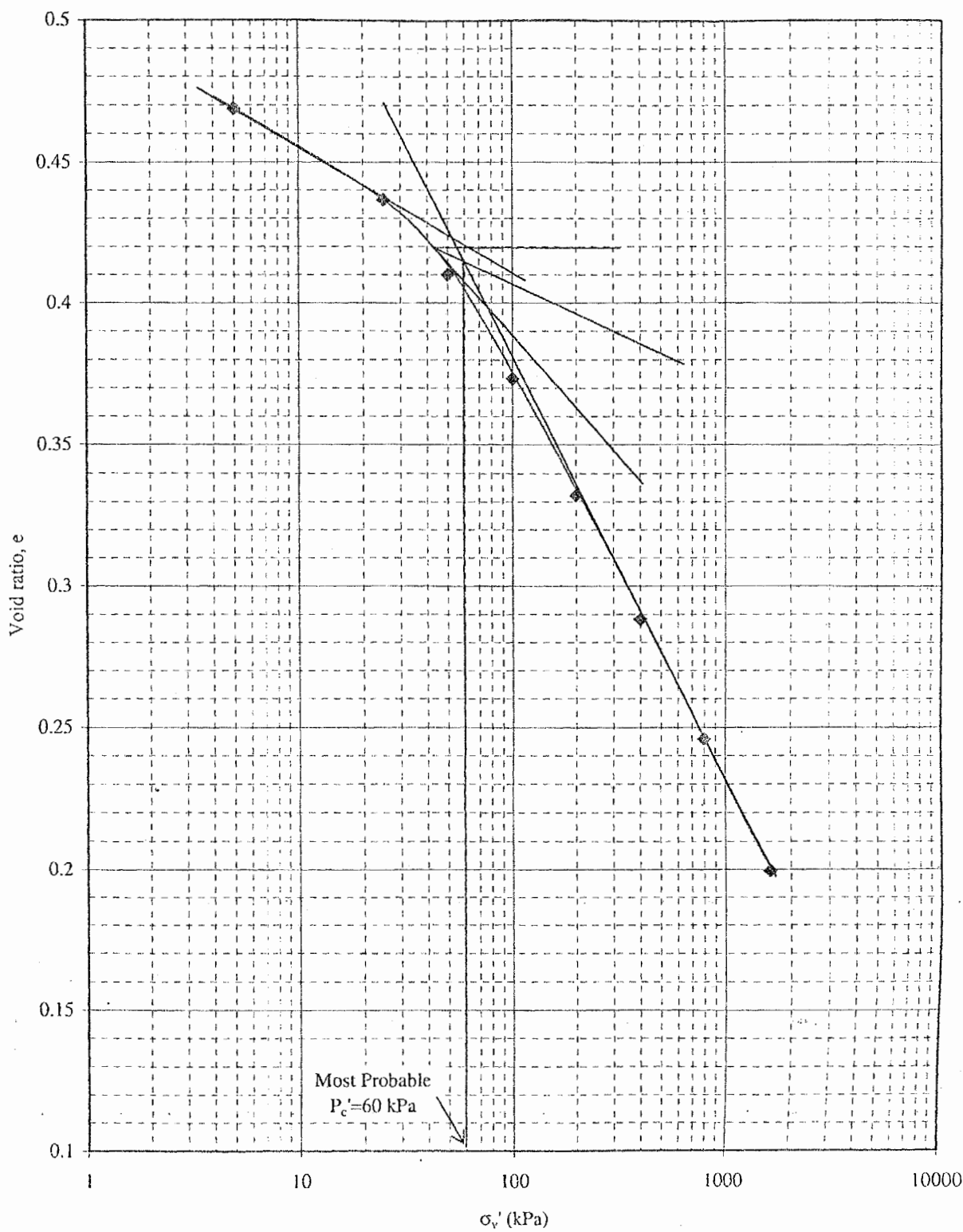


**e-log P curve for compacted kaolin
(dry, w=30.1%)**

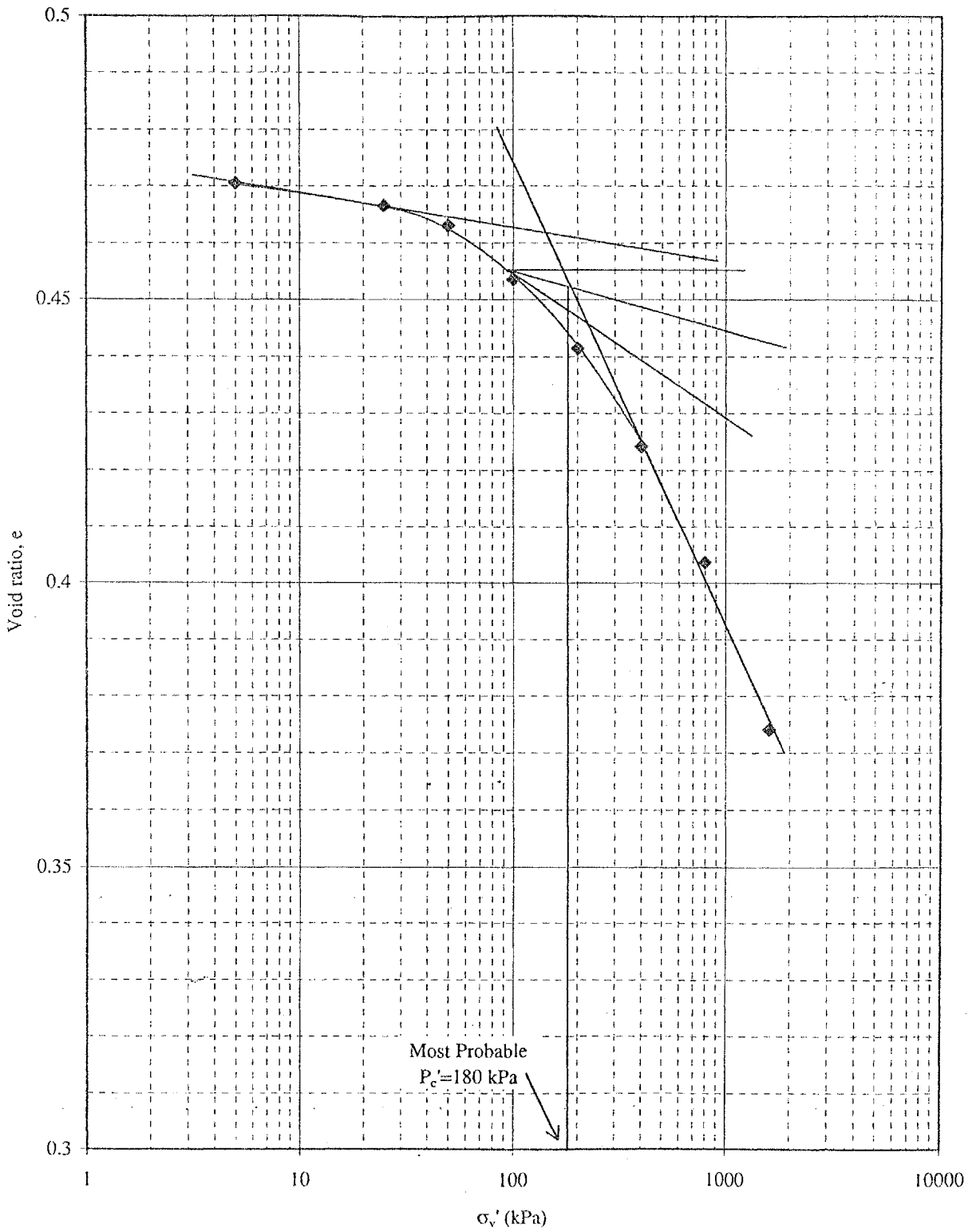


residual Soil A

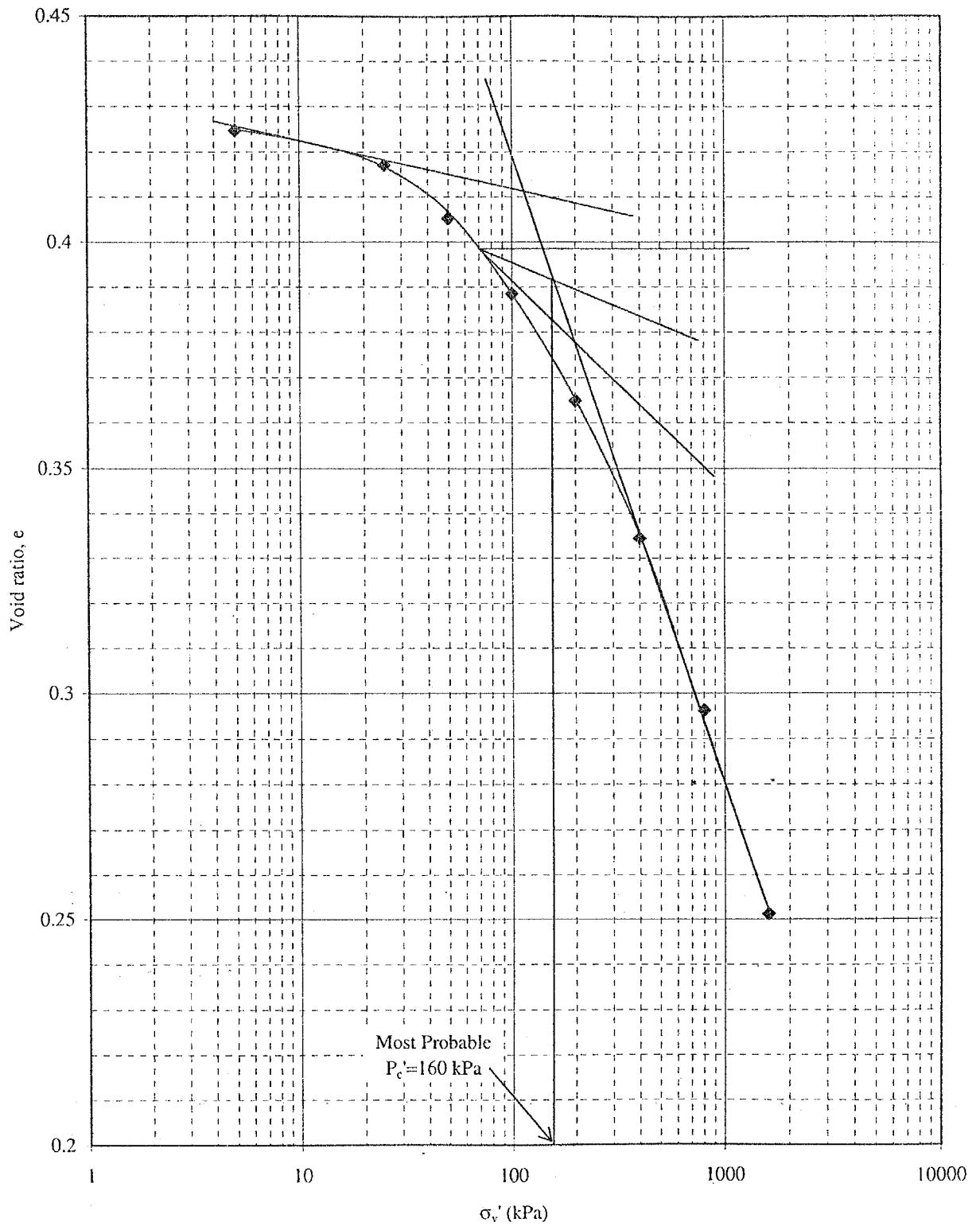
e-log P curve for compacted residual soil A
(wet, $w=9.4\%$)



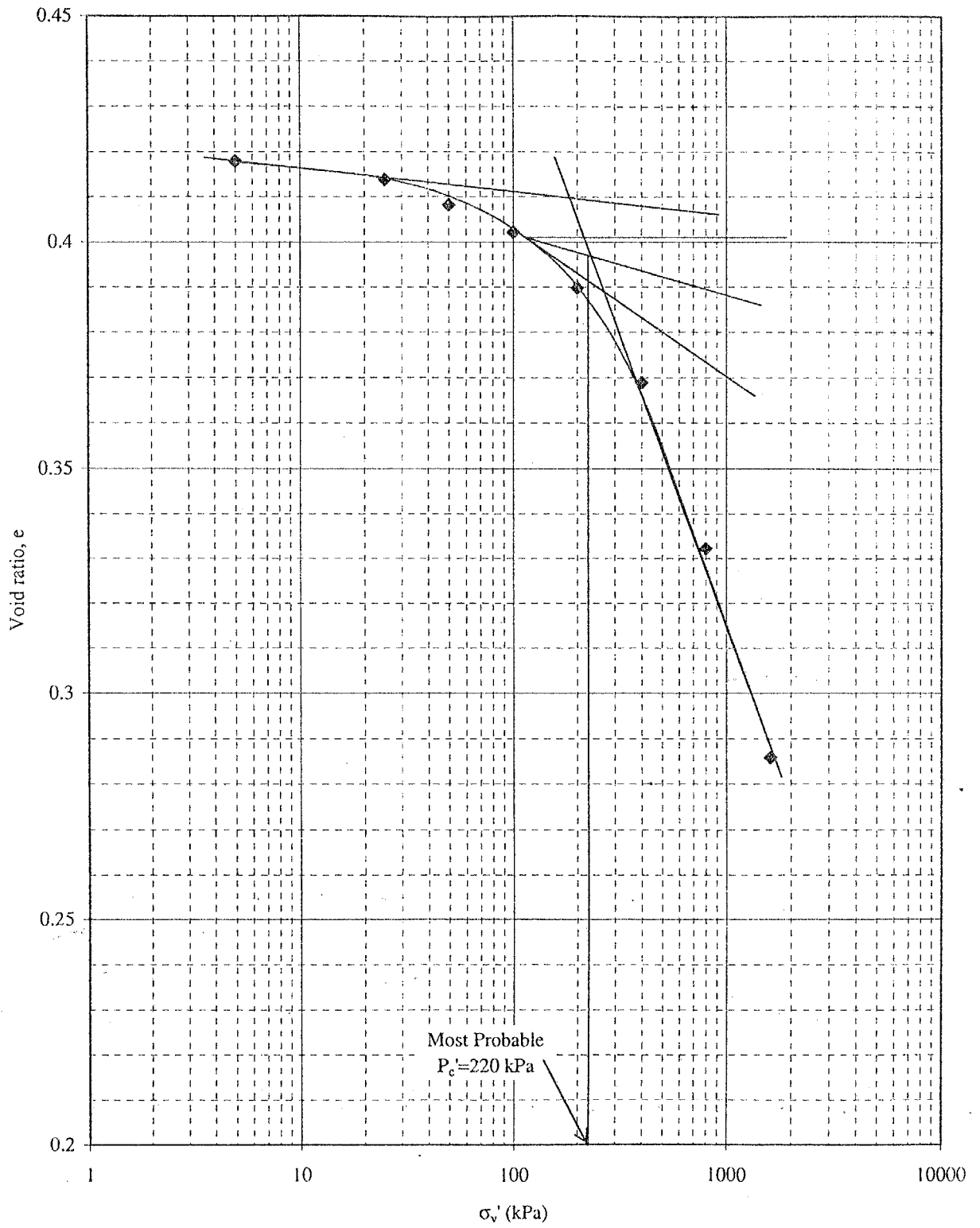
e-log P curve for compacted residual soil A
(dry, w=9.4%)



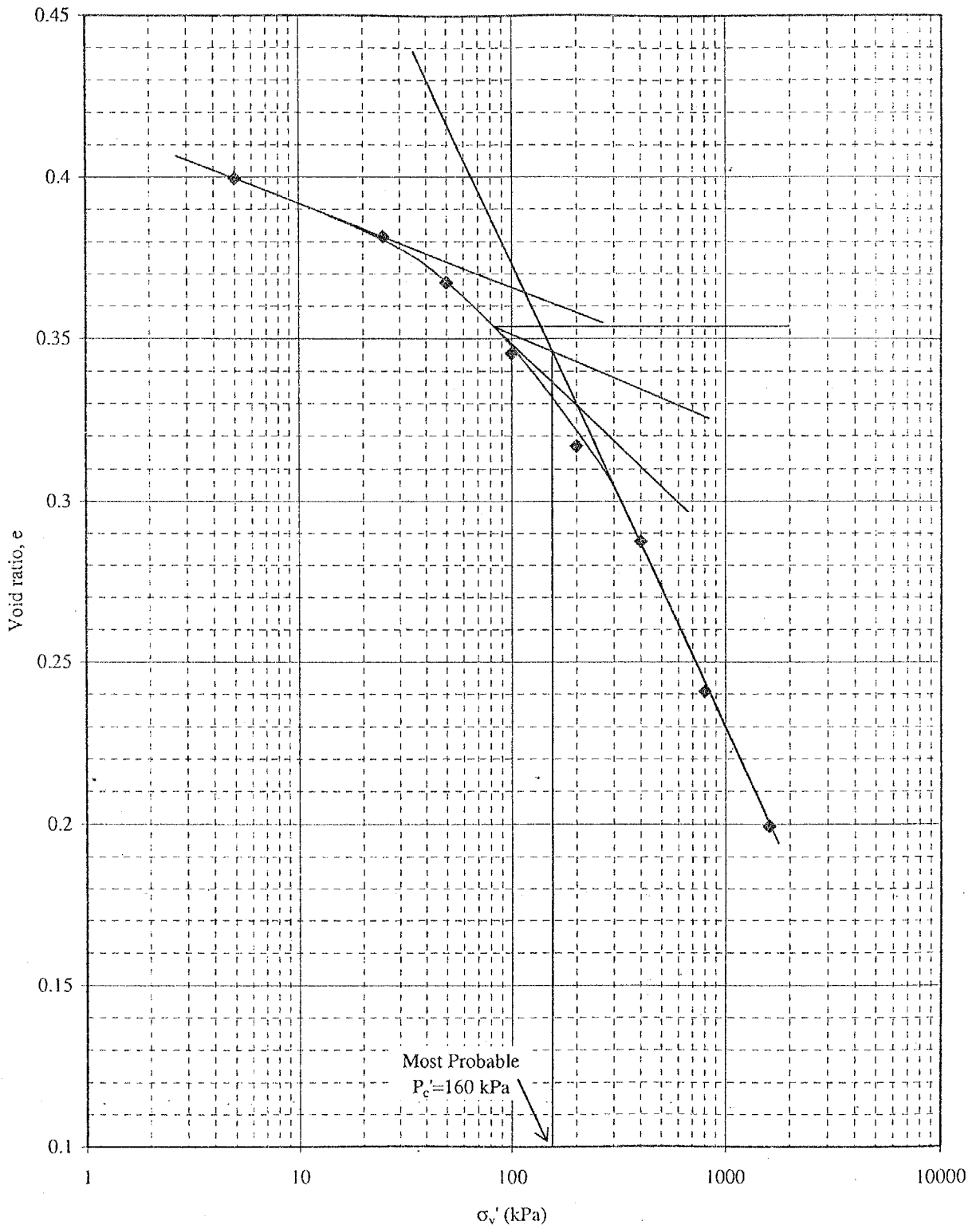
e Log P curve for compacted residual soil A
(wet, $w=11.4\%$)



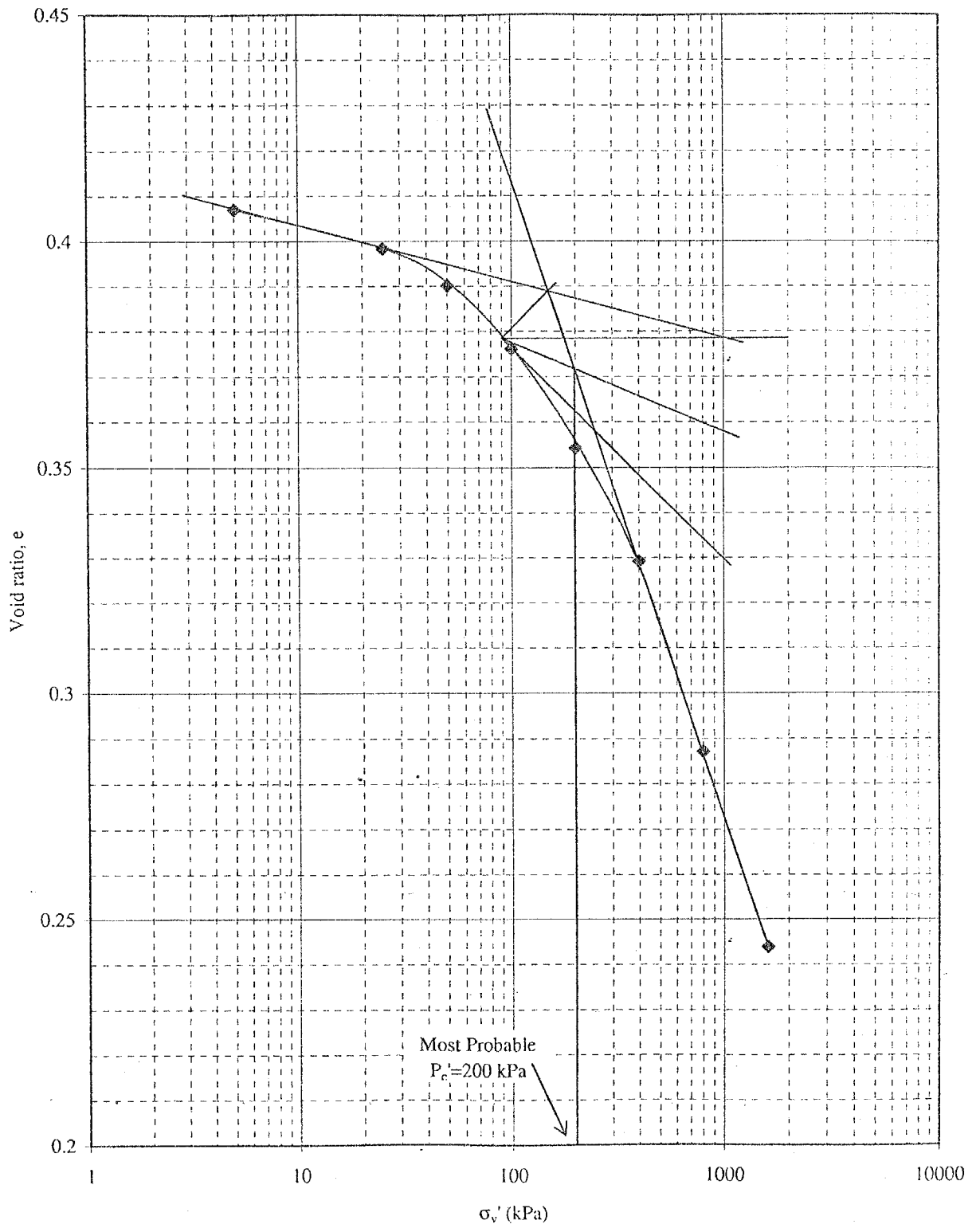
e-log P curve for compacted residual soil A
(dry, w=11.4%)



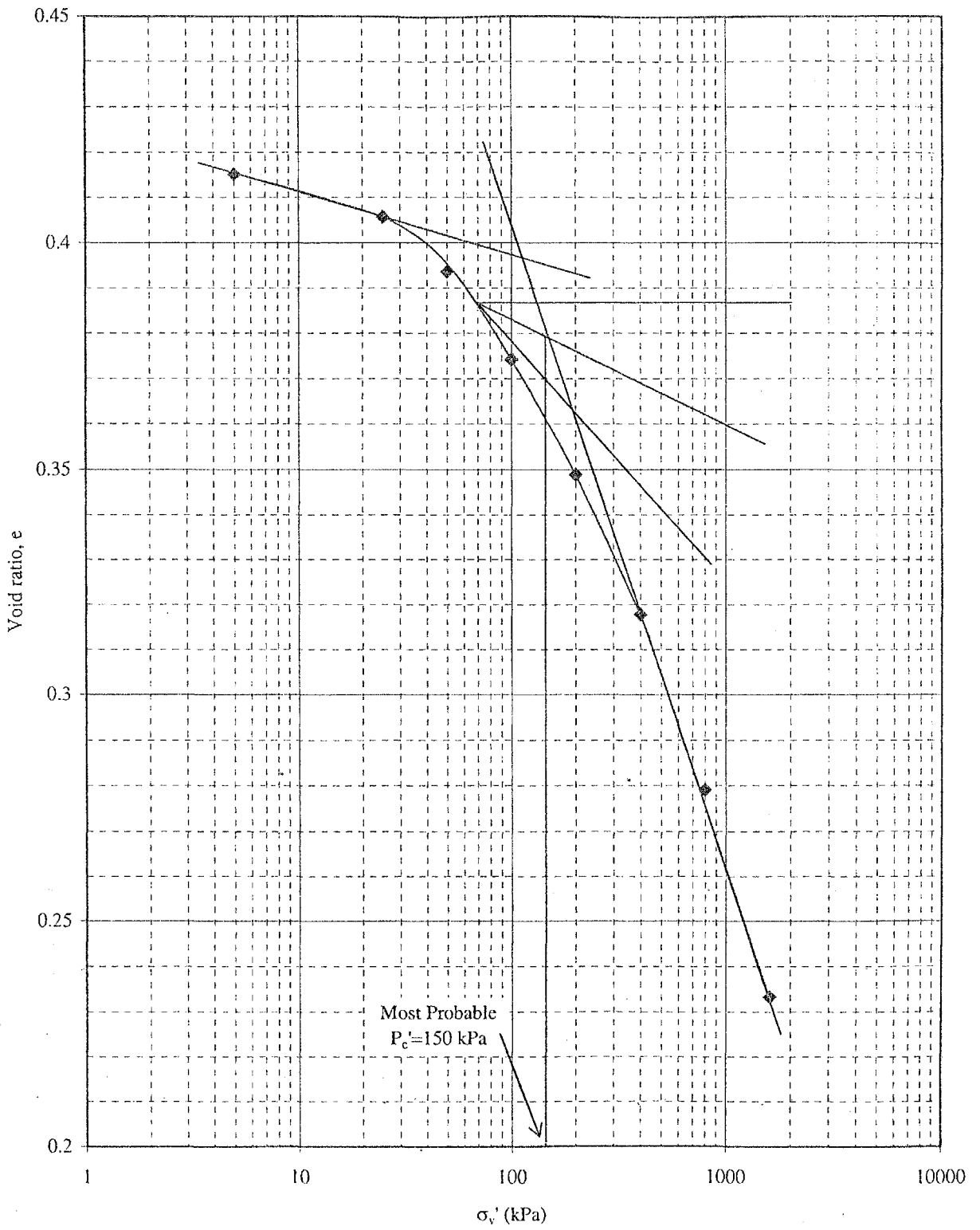
e-log P curve for compacted residual soil A
(wet, $w=13.8\%$)



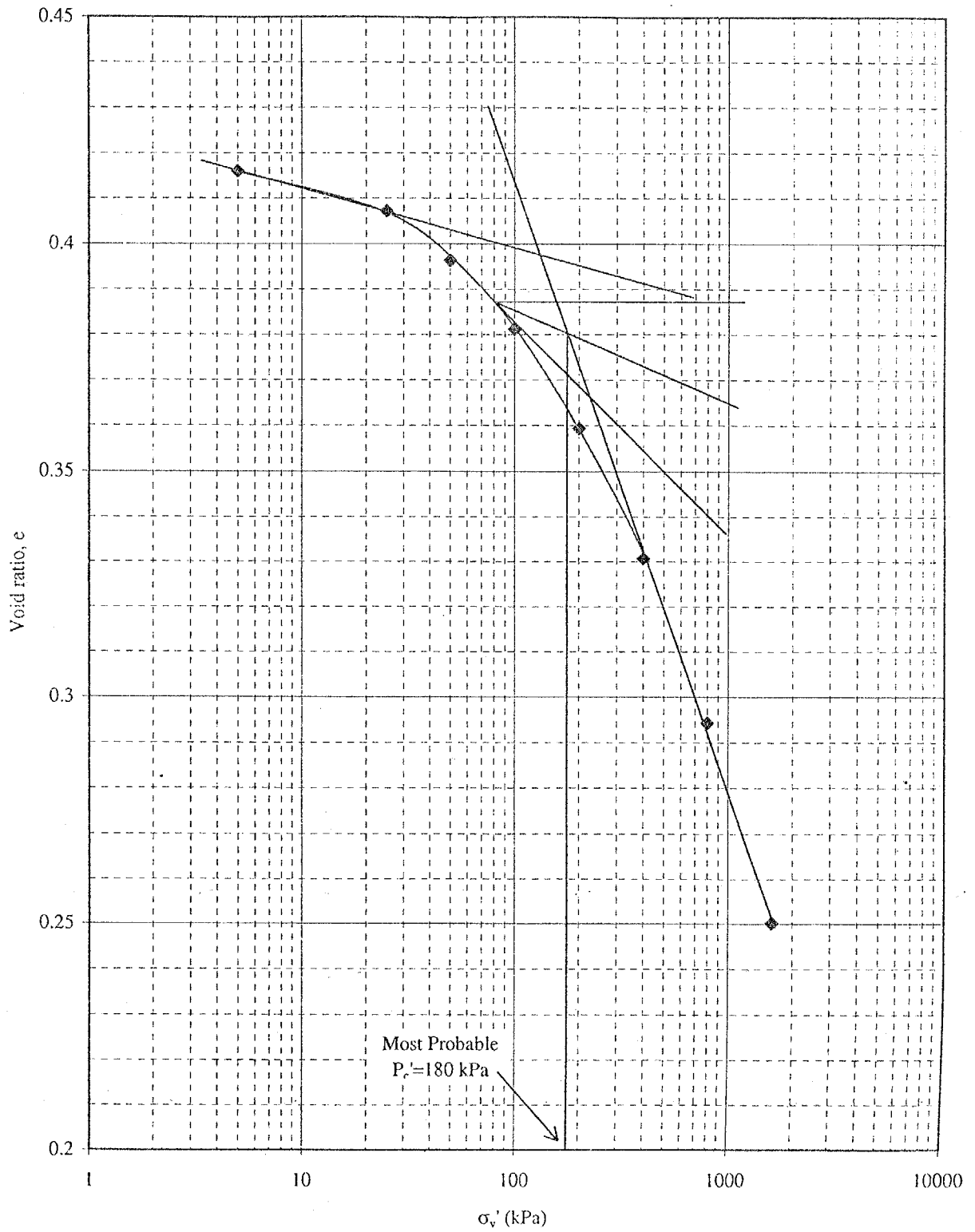
e-log P curve for compacted residual soil A
(dry, w=13.8%)



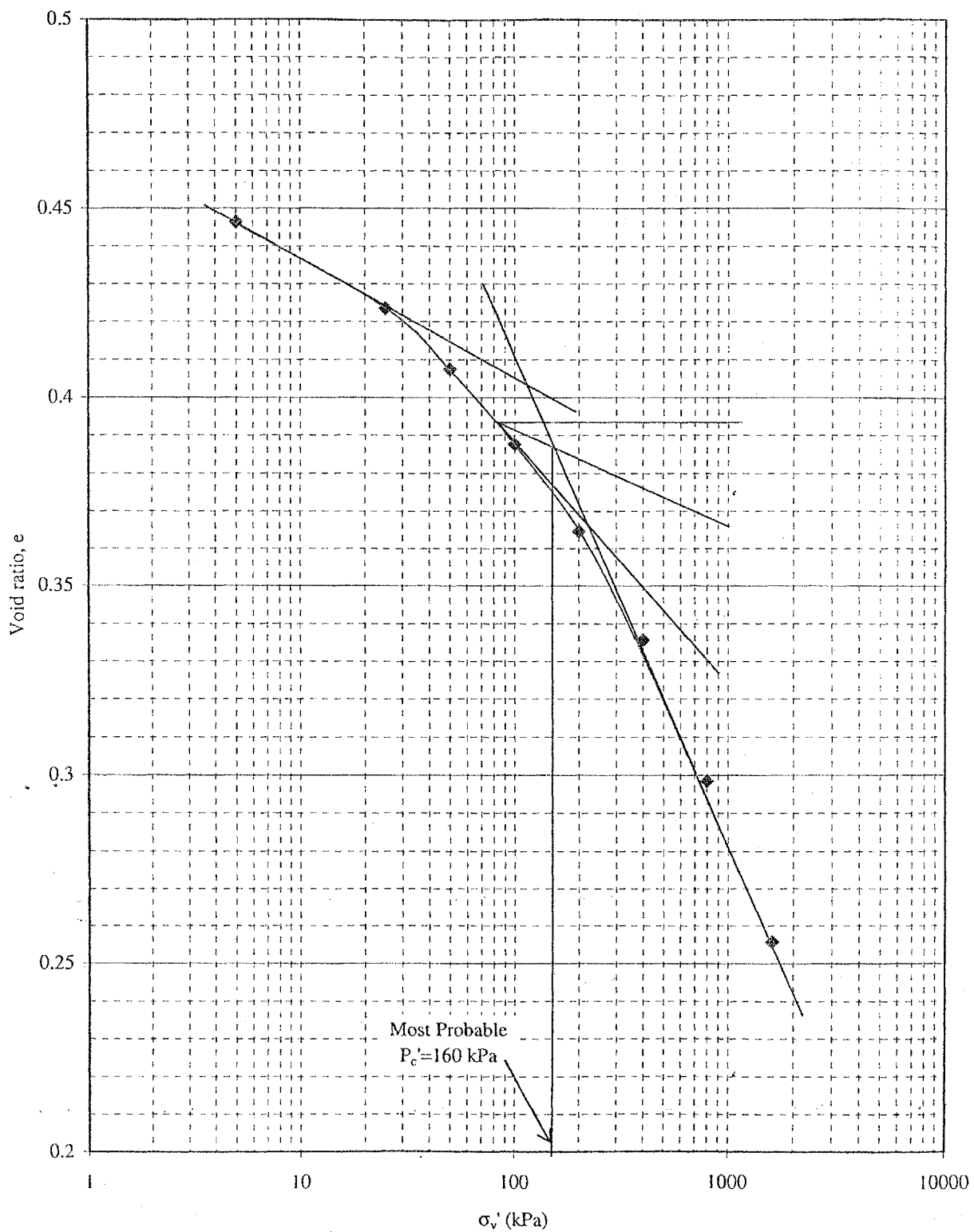
e-log P curve for compacted residual soil A
(wet, $w=15.3\%$)



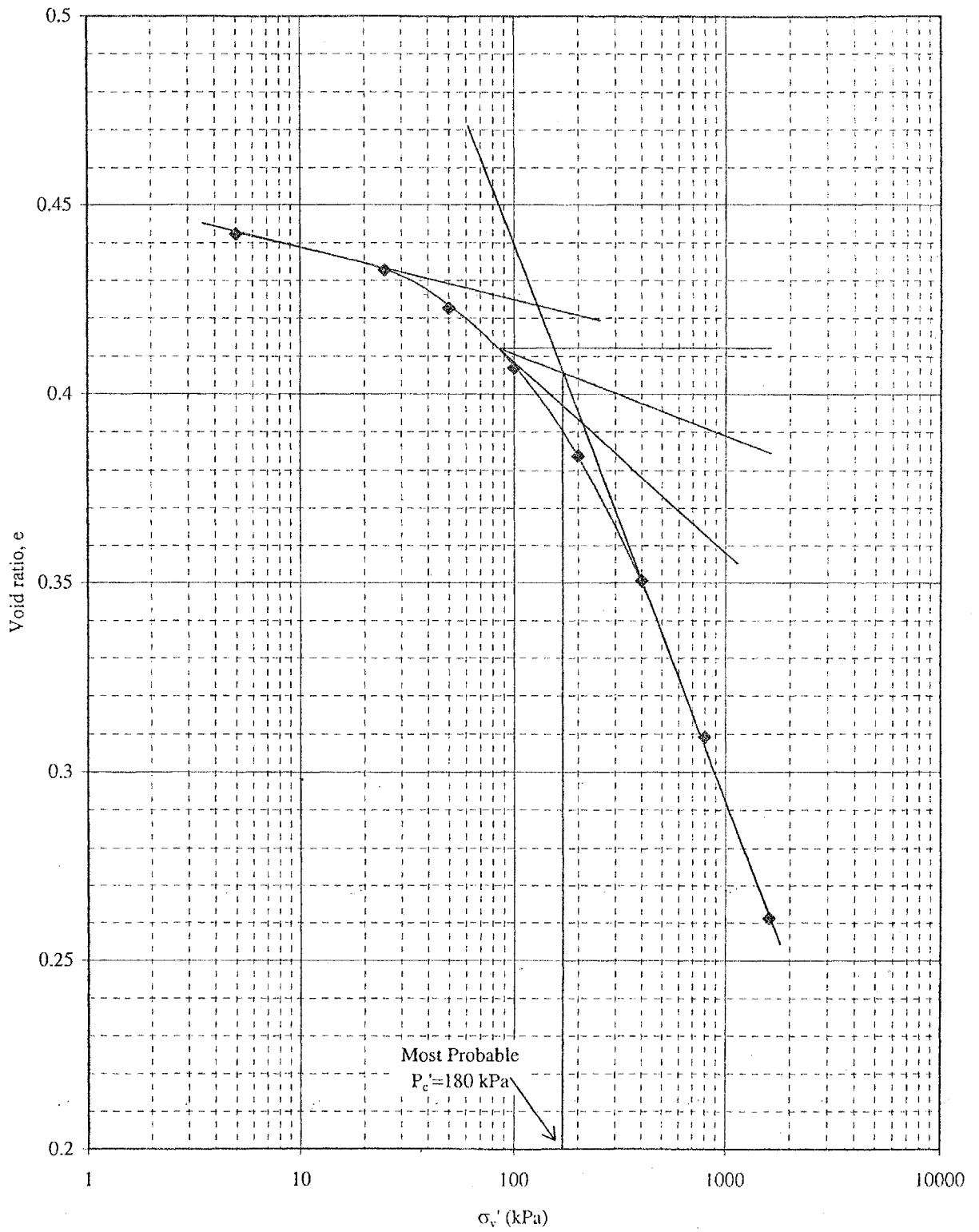
e-log P curve for compacted residual soil A
(dry, w=15.3%)



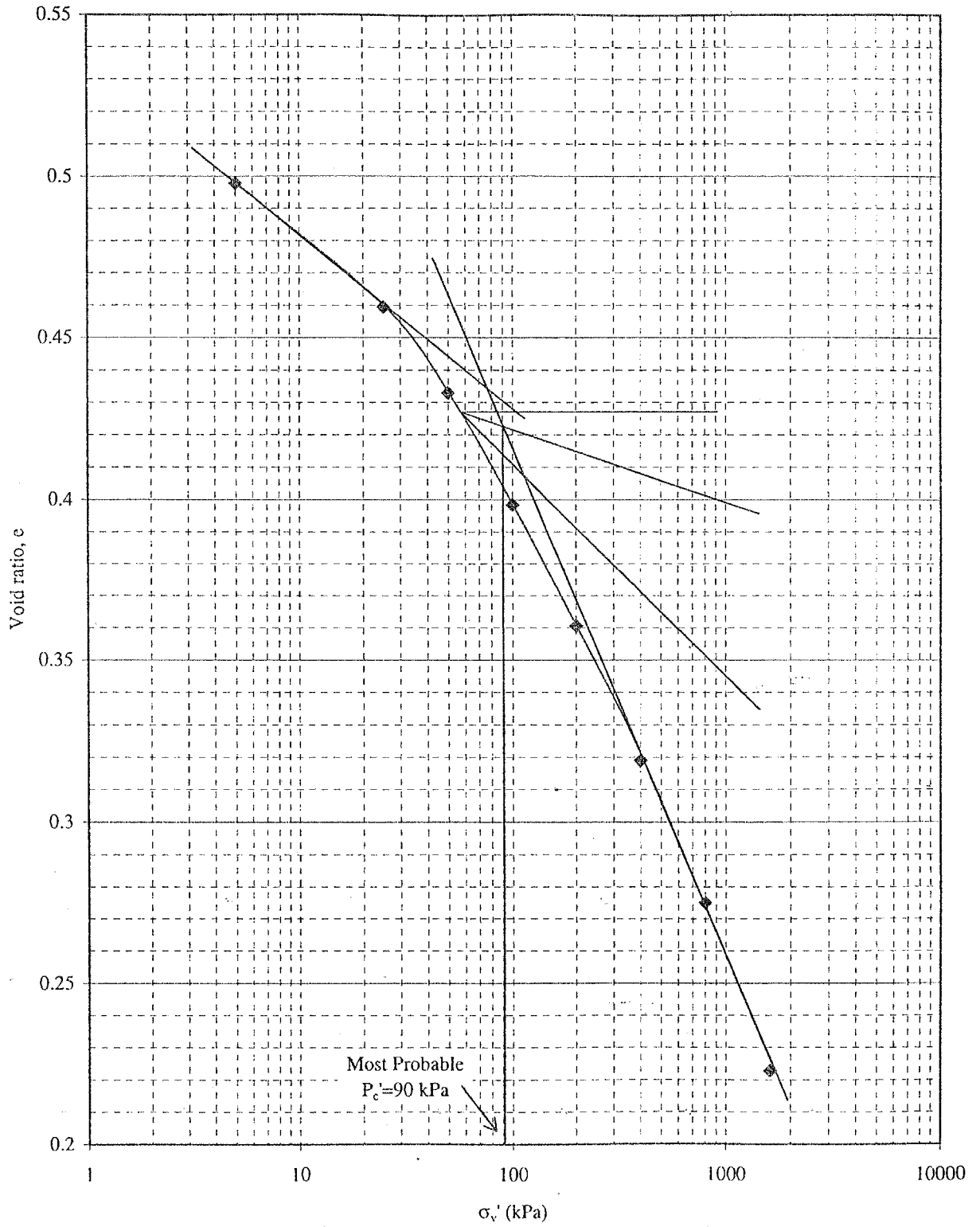
e-log P curve for compacted residual soil A
(wet, w=16.7%)



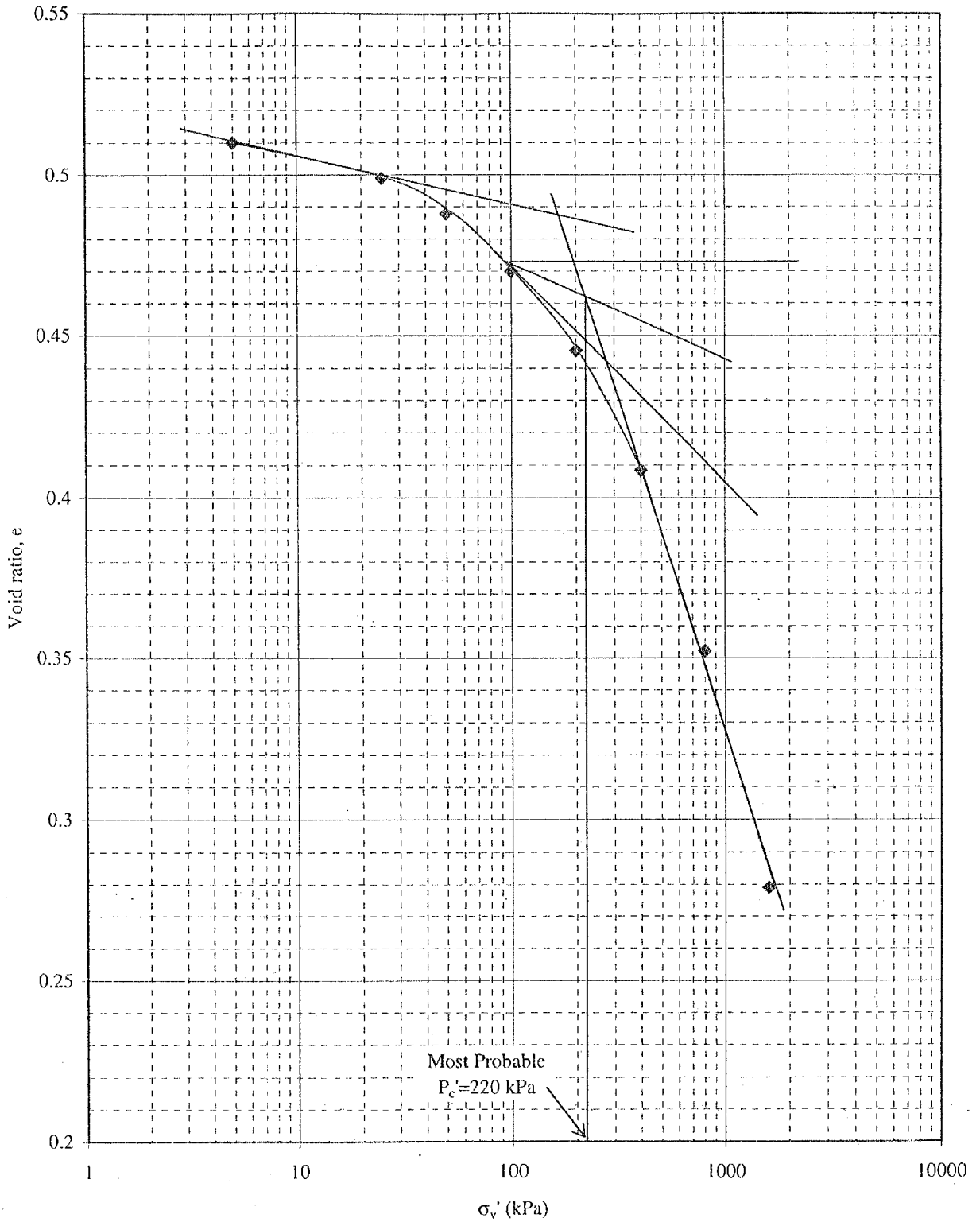
**e-log P curve for compacted residual soil A
(dry, w=16.7%)**



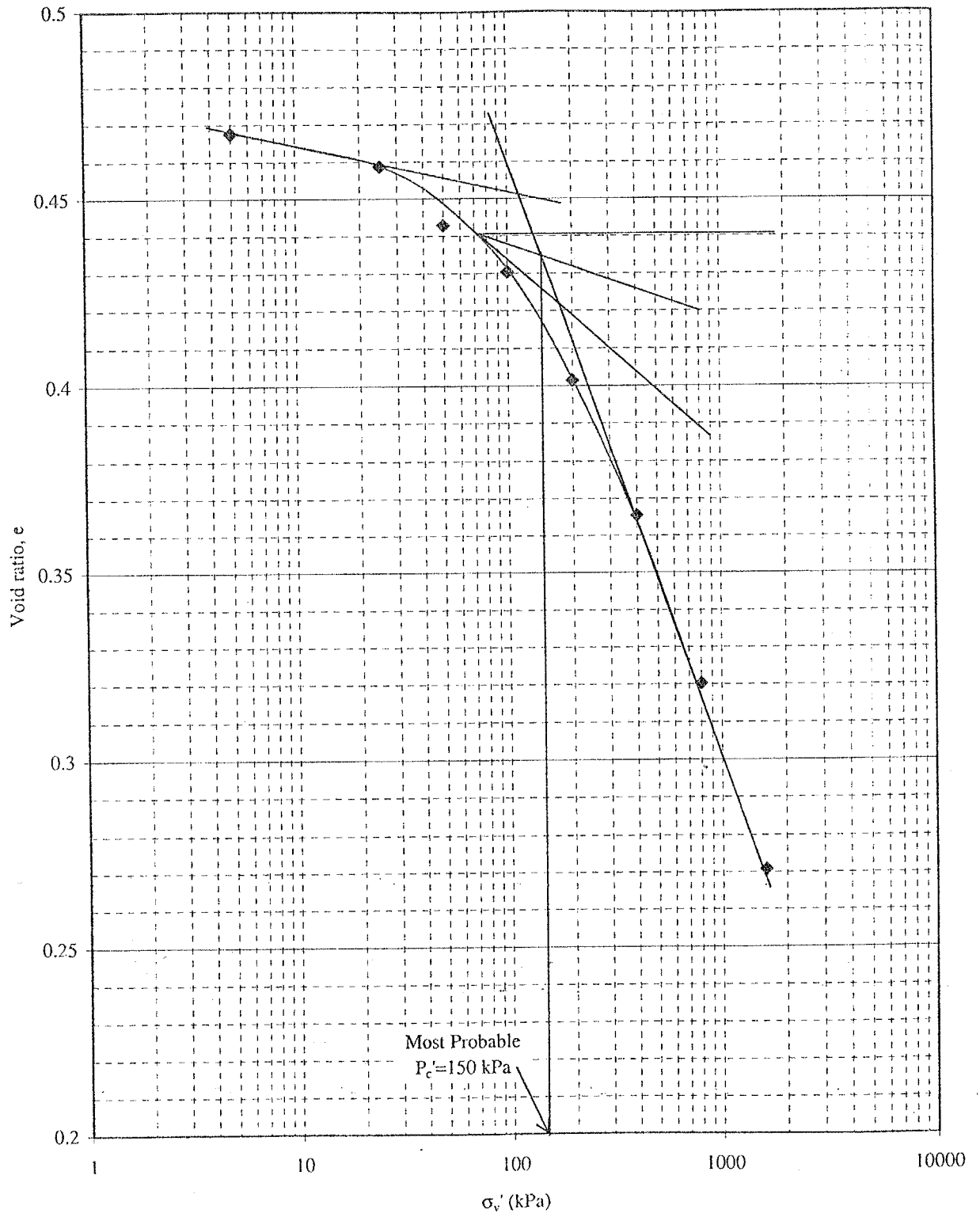
e-log P curve for compacted residual soil B
(wet, $w=9.7\%$)



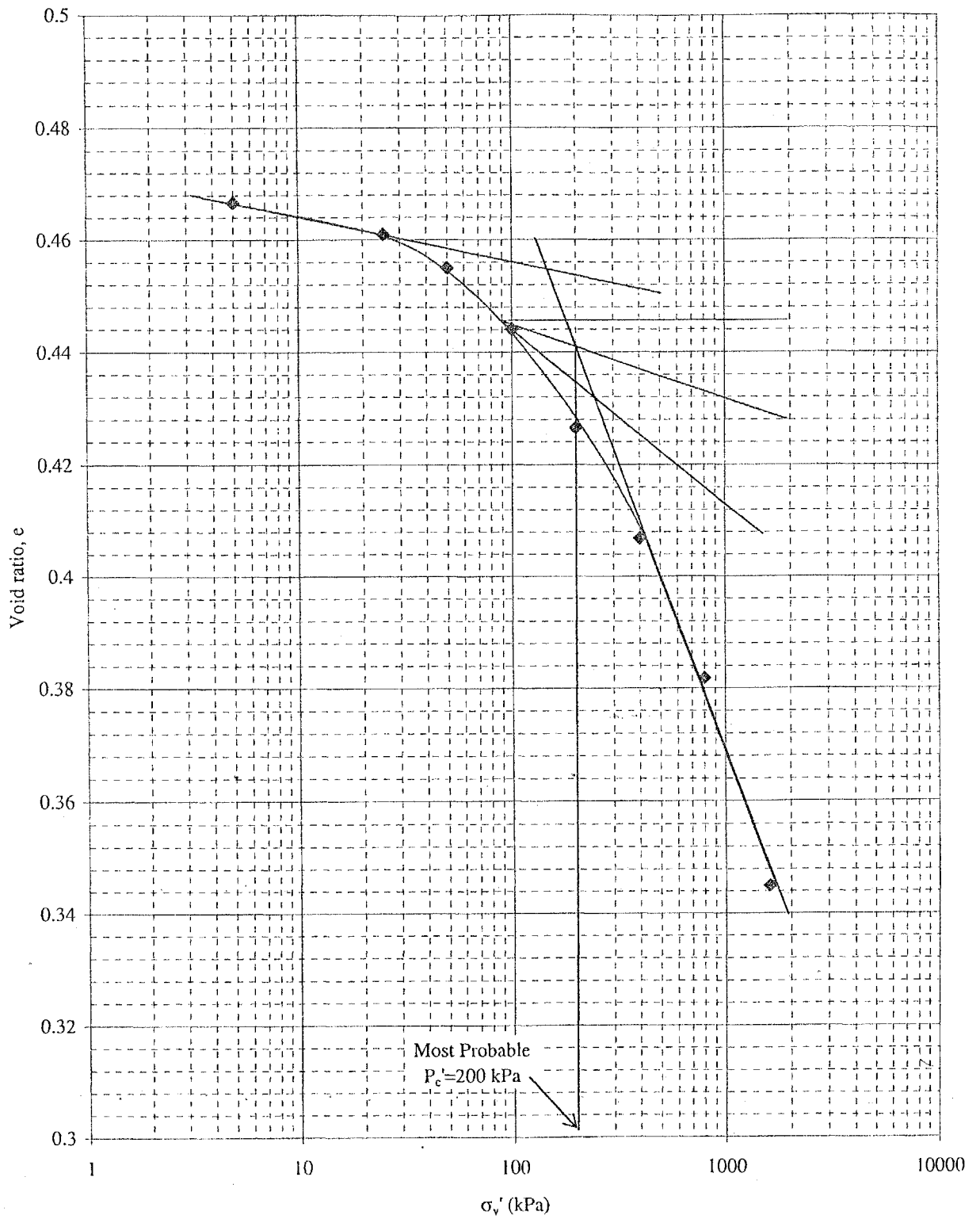
e-log P curve for compacted residual soil B
(dry, $w=9.7\%$)



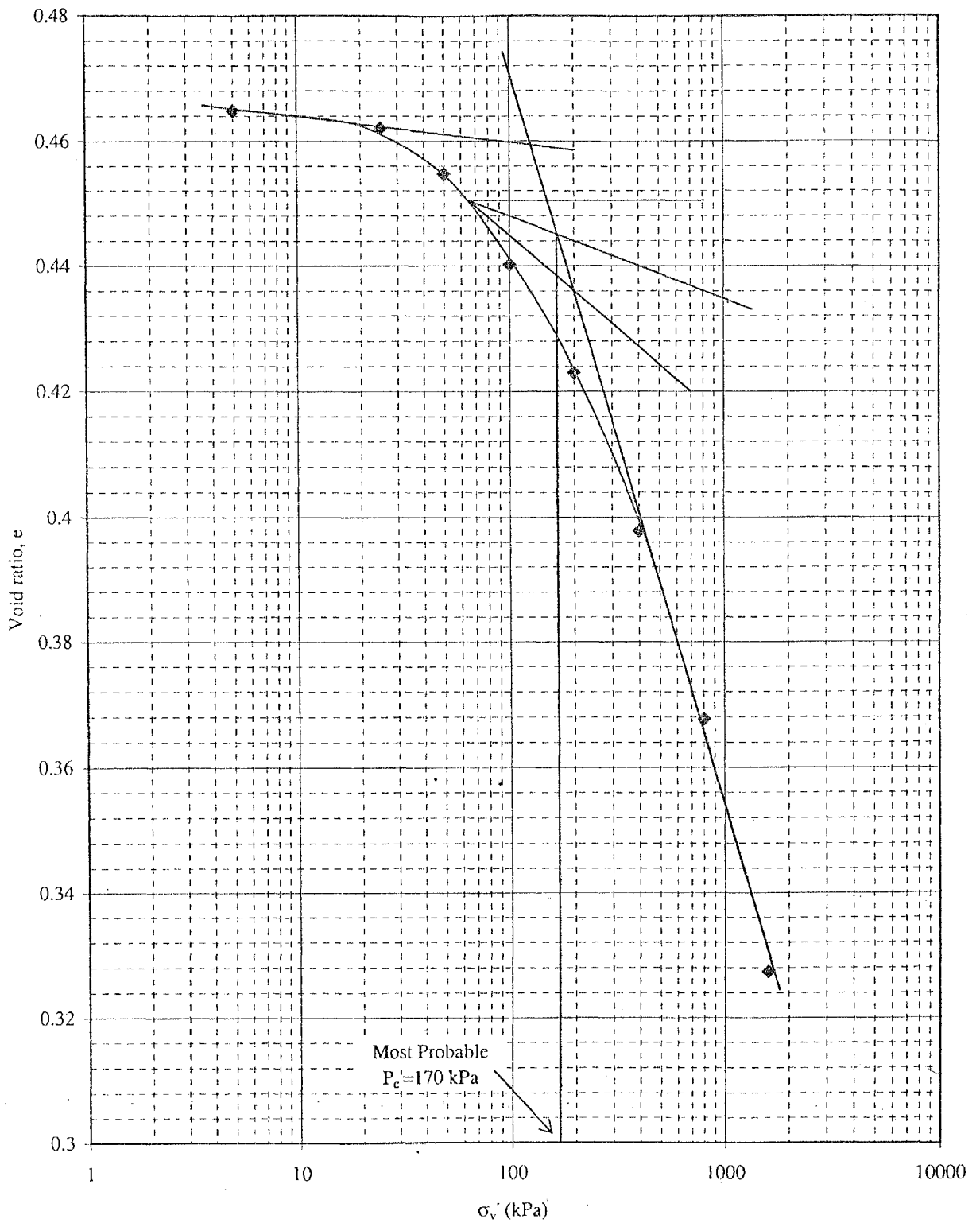
**e-log P curve for compacted residual soil B
(wet, w=12.1%)**



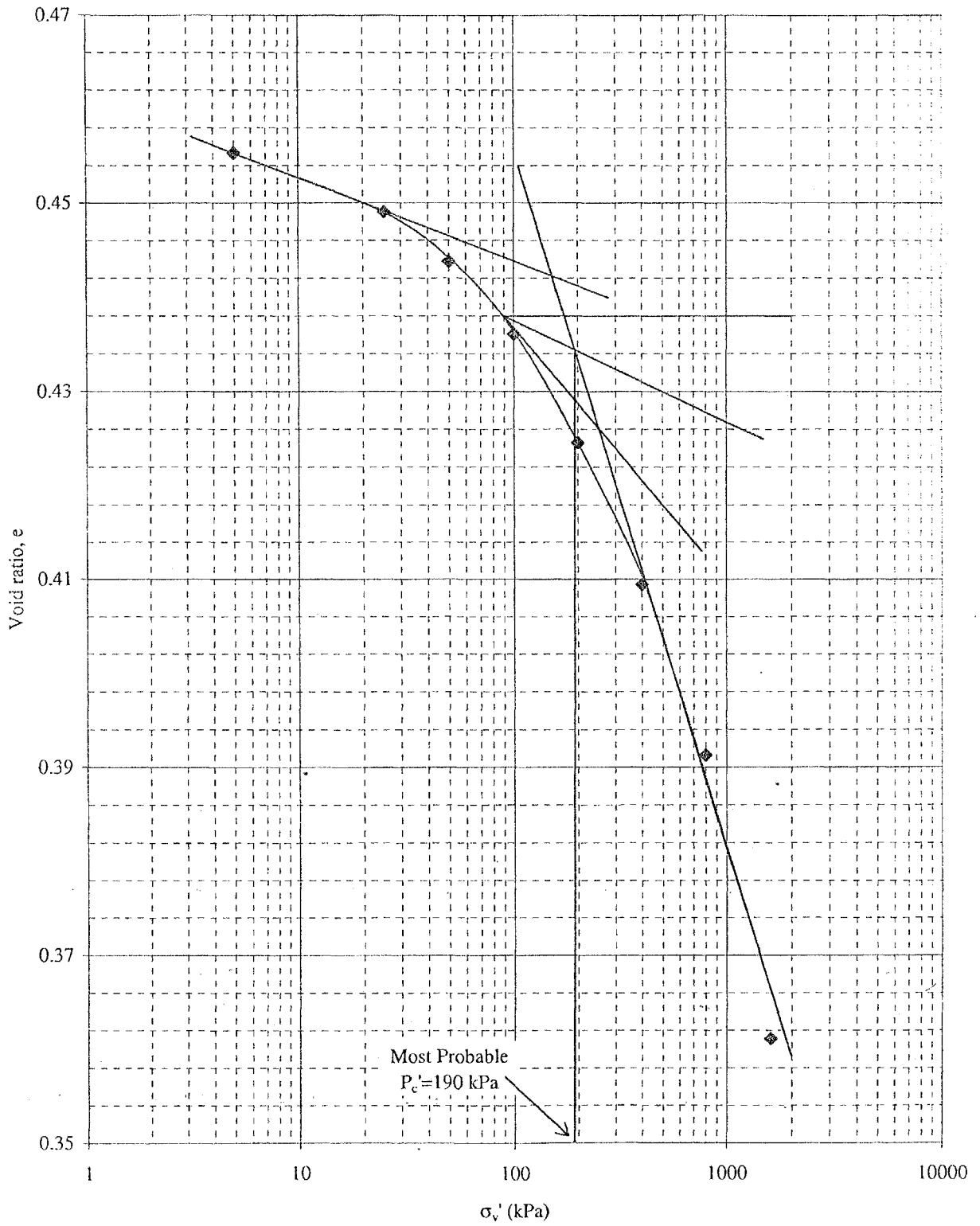
**e-log P curve for compacted residual soil B
(dry, w=12.1%)**



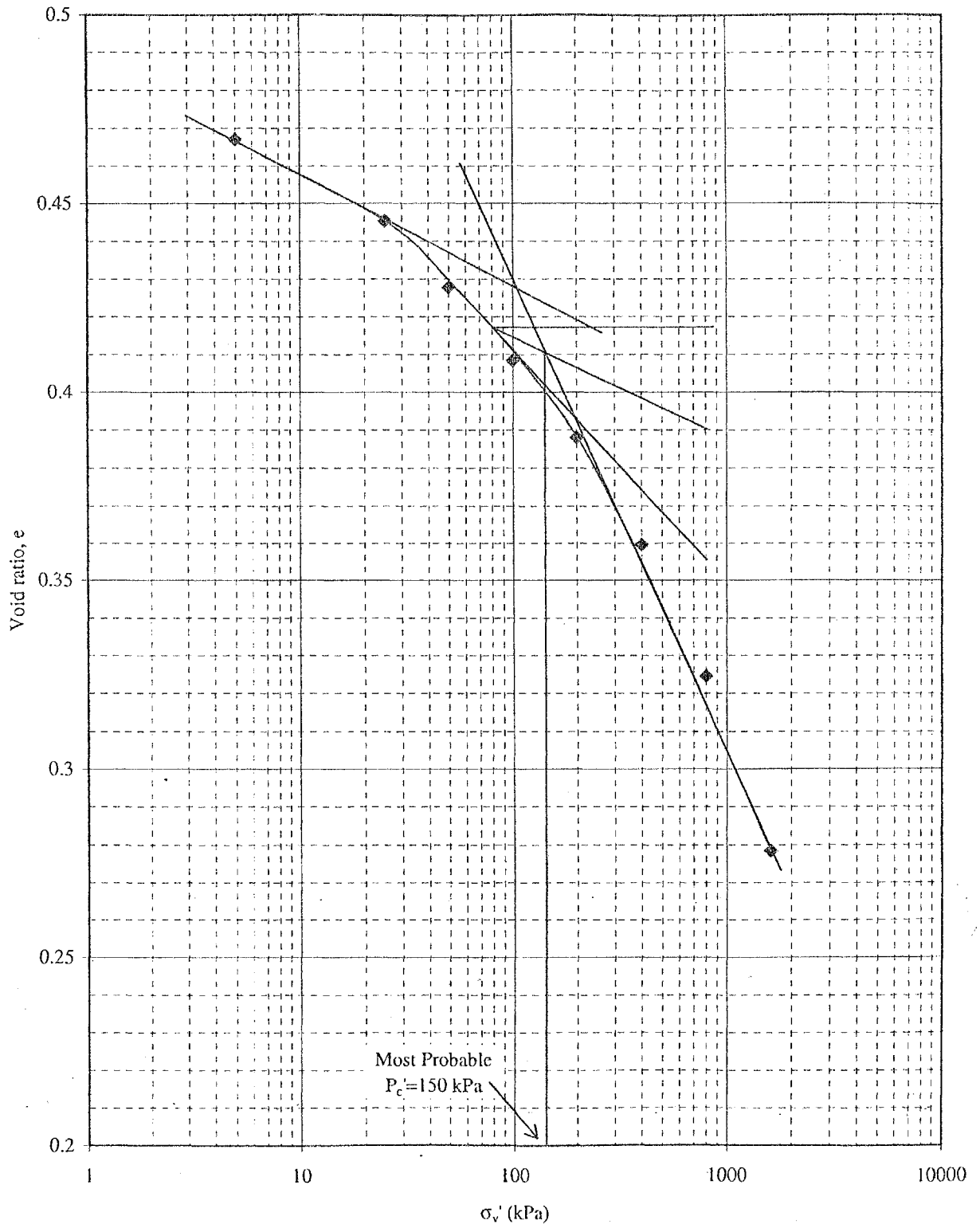
**e-log P curve for compacted residual soil B
(wet, w=13.2%)**



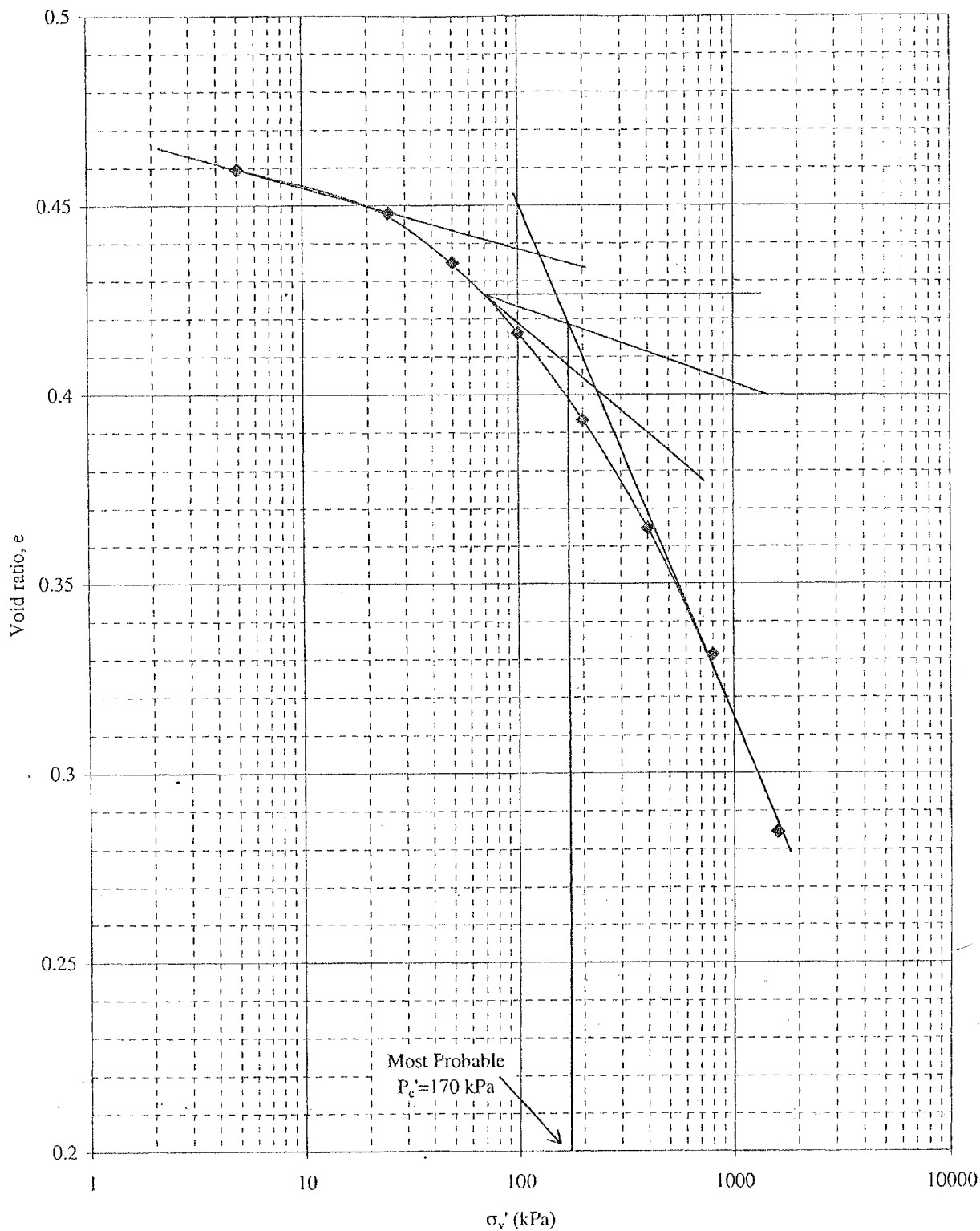
**e-log P curve for compacted residual soil B
(dry, w=13.2%)**



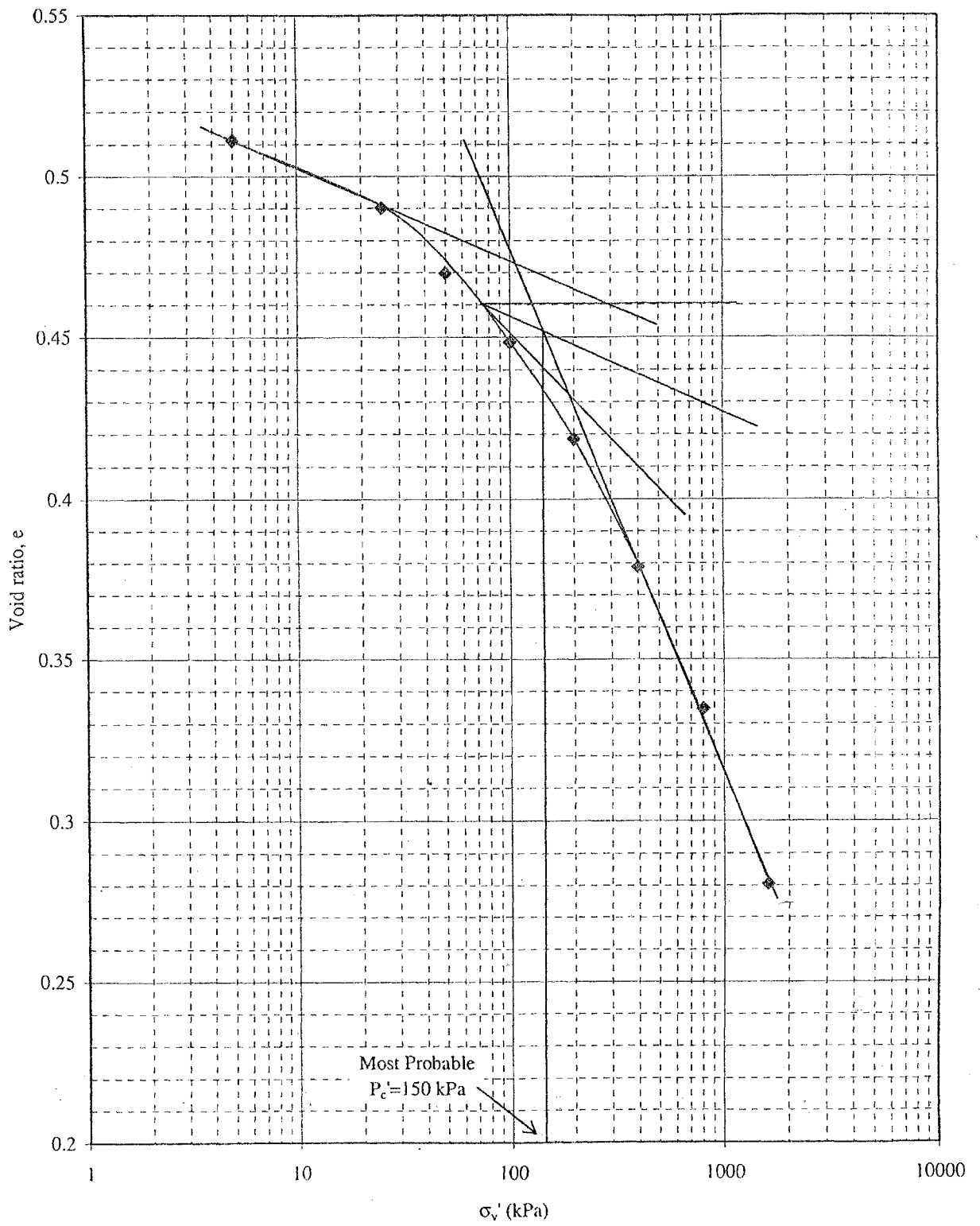
e-log P curve for compacted residual soil B
(wet, w=15.2%)



e-log P curve for compacted residual soil B
(dry, $w=15.2\%$)



e-log P curve for compacted residual soil B
(wet, $w=17.5\%$)



e-log P curve for compacted residual soil B
(dry, w=17.5%)

