ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

Relationship between Consolidation and Swelling Characteristics of Expansive Soils of Addis Ababa

By

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March 2005

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A thesis submitted to the school of graduate studies, Addis Ababa University, in partial fulfillment of the requirements for the Degree of Masters of Science in Civil Engineering

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Acknowledgement

I would like to thank my advisor, Dr. Messele Haile for his valuable advice and guidance for my work as well as providing the necessary materials for the research work.

I forward my gratitude and appreciation to the laboratory personnel of the Department of Civil Engineering of the Addis Ababa University for their commitment to spend their time and render assistance in the laboratory works of this thesis.

My thanks also go to Addis College, my employer, for their willingness to allow me attend my M. Sc. Study.

Last but not least, I would like to convey my gratitude to my parents, and friends. With out their support and encouragement I couldn't have this opportunity to complete my study.

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Notations

a_v	Coefficient of compressibility
C_c	Compression index
C_v	Coefficient of consolidation
e	Void ratio
Δe	Change in void ratio
H	Length of drainage path
k	Coefficient of permeability
m_v	Coefficient of volume compressibility
OCR	Over Consolidation ratio
P	Pressure due to applied load
P_c	Preconsolidation pressure
P_o	Initial overburden pressure
ΔP	Change in effective stress
S_c	Consolidation settlement
S_i	Immediate settlement
S_s	Secondary settlement
t_{50}	Time during which 50% of consolidation takes place
t_{90}	Time during which 90% of consolidation takes place
γ_w	Unit weight of water

Abstract

Clay soils are usually subjected to a time dependant strain under load. This strain is mainly a function of the rate of pore water pressure dissipation which in turn depends on the permeability of the soil mass. Therefore, consolidation is a term used to describe the volume change of the soil mass due to pore water pressure dissipation.

Among the clay soils in the world, expansive soils have a special mineralogical characteristic which leads to swelling when exposed to moisture change. This behavior is due to the presence of considerable amount of montmorillonite clay mineral. As a result of swelling, pressure is exerted by the soil on the structures built on it and causes an increase in volume of the soil mass which also lifts the structure. Consequently, cracking of buildings, breaking of pavements, damaging of utility pipes are common problems on structures built on such soils. But under the action of heavy loads, greater than the swelling pressure, the soil undergoes consolidation.

Thus, this paper is mainly intended to evaluate the swelling and consolidation behavior of expansive soils of Addis Ababa and their interdependence using one dimensional consolidation theory of Terzaghi. In the course of the research work consolidation-swell test was carried out on a number of samples of expansive soil obtained from different parts of Addis Ababa. The swelling and consolidation characteristic of the soil is determined from the laboratory tests. Ranges of values of the consolidation parameters are also obtained. The swelling behavior is found to influence the consolidation characteristics of expansive soils.

1-Introduction

1.1 General

Expansive soil, which is recognized by its considerable volume change upon exposure to moisture variation, has caused a number of problems in most structures constructed in Addis Ababa. Recent researches in assessing the failures caused on structures built on expansive soils showed that more than 60% of the structures are damaged due to causes associated with expansive soils.[14] The problems are either due to misunderstanding of the behavior of the soil or lack of information on the engineering properties of the soil.

Even though, expansive soils are found in different parts of the world, the engineering properties of the soil are not similar. The behavior of expansive soils varies from place to place depending upon the type of parent material, climate and topography. Therefore, due to the fact that the engineering properties of expansive soil of Ethiopia are different from the same soil in other locality, researches on the engineering properties of expansive soils of Ethiopia has been done. Some of the researches undertaken are 'The effects of initial moisture contents and initial dry density on the characteristics of expansive soils in Addis Ababa'(Yiheyis, H., 2001),'Examining the Swelling Pressure of Addis Ababa Expansive Soils'(Teklu, D., 2003), 'Assessment of Pavement Failure Founded on Expansive Soils in Ethiopia'(Tsegaw, M., 2003), 'Investigation on Index Properties of Expansive Soils of Ethiopia'(Legesse,M., 2004), 'Investigation into shear strength characteristics of expansive soil of Ethiopia'(Zewde,A.,2004),and 'Investigation of failures of buildings in Addis Ababa' (Sisay,A., 2004). In all the above researches, even though most of the engineering properties of expansive soils of Ethiopia are covered, consolidation characteristics have not been assessed.

1.2 Objective

Swelling and consolidation of expansive soils are phenomena, which take place due to moisture change and effective stress change in the soil mass, respectively. In condition where both moisture change and effective stress changes take place at the same time, there will be a tendency for swelling and consolidation; the volume change being dependent on the magnitude of the effective stress change. The main objective of this thesis is therefore, to identify the relationship between consolidation and swelling characteristics of expansive soils under different degree of saturation. In addition, the ranges of values of the consolidation parameters of expansive soil of Addis Ababa are determined.

1.3 Methodology

In order to achieve the objective of this thesis, researches accomplished on expansive soils of Addis Ababa are reviewed. In addition, consolidation and swelling tests are carried out on disturbed and undisturbed samples of expansive soil taken from different sites in Addis Ababa. In the course of the laboratory testing, soil samples with different degree of saturation are considered.

2-Literature Review

2.1 Expansive soil in general

Expansive soil is a term generally applied to any soil or rock material that has a potential for shrinking or swelling under changing moisture conditions. [2] Subsequent swelling and shrinkage of this soils due to change in moisture cause damages to civil engineering structures, particularly light buildings and pavements.

The origin of expansive soils is related to a complex combination of conditions and processes that result in the formation of clay minerals having a particular chemical makeup which, when in contact with water, will expand. The conditions and processes which determine the clay mineralogy include composition of the parent material and degree of physical and chemical weathering to which the materials are subjected [1].

There are two fundamental molecular structures as the basic units of the lattice structure of clay soils. These are the silica tetrahedron and the alumina octahedron. The silica tetrahedron consists of a silicon atom surrounded tetrahedrally by oxygen ions. The alumina octahedron consists of an aluminum atom surrounded octahedrally by six oxygen ions. When each oxygen atom is shared by two tetrahedral, a plate- shaped layer is formed. Similarly, when each aluminum atom is shared by two octahedron, a sheet is formed.

The silica sheets and the aluminum sheets combine to form the basic structural units of the clay particles. Various clay minerals differ in the stacking configuration.

The major component of expansive soils, montmorillonite is a three layer clay mineral having a single octahedral sheet sandwiched between two tetrahedral sheets to give a 2 to 1 lattice structure.

2.2 Engineering properties of expansive soils of Addis Ababa

Recent researches on the engineering properties of expansive soils of Ethiopia revealed that, the expansive soils in most parts of the country are classified as inorganic clays of high plasticity. (Fig.2.1). The clay content of the soil is found to be as high as 80%[5] and the amount of montmorillonite for Addis Ababa expansive soil is 70-80%[Table 2.1]. These soils have the ability to hold significant amount of water that affects the shear strength, as well as, shrinkage and swelling characteristics.

Table2.1: Mineralogical composition of Addis Ababa expansive soils. [7]

Clay miner	Mineral content (%)	
	Black clays	Gray clays
Montmorill	70-80%	70-75%
Illite	10-15%	25-30%
Kaolinite	10-15%	10-15%

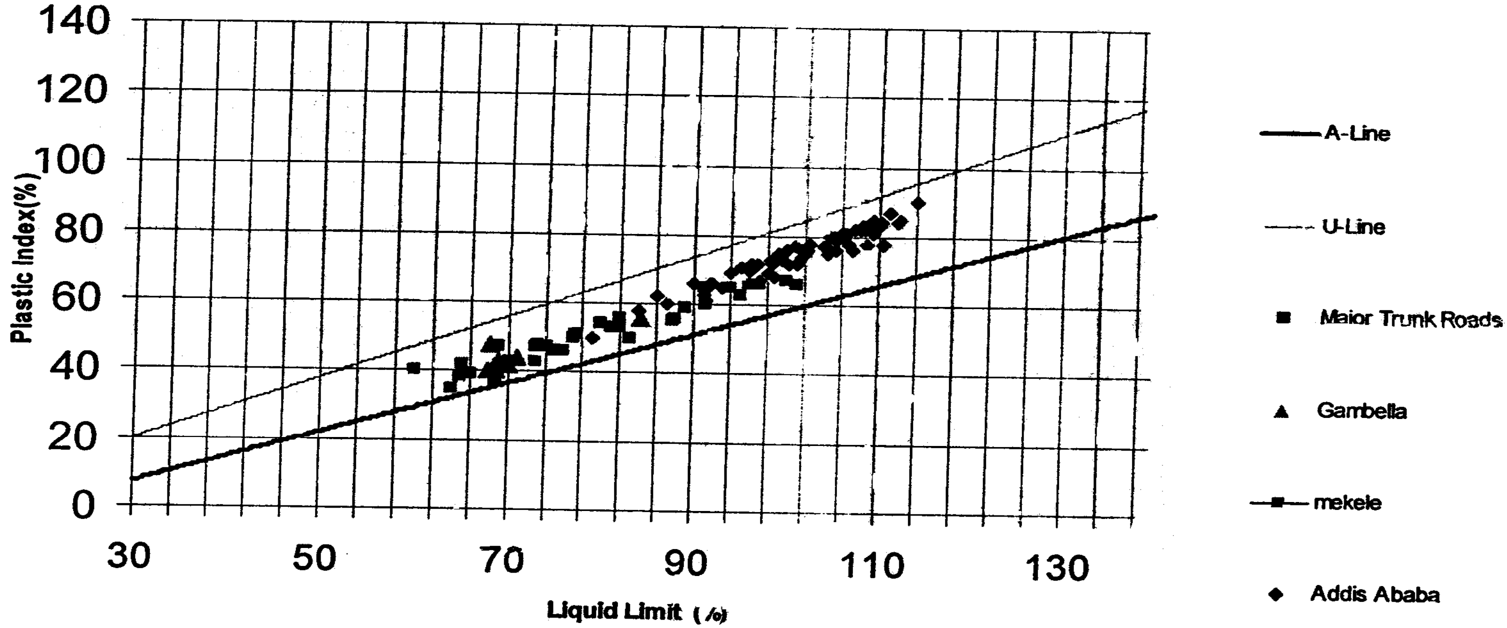


Fig. 2.1 Plasticity chart of expansive soils of Ethiopia [5]

From the researches mention above, the engineering properties of the expansive soil of Addis Ababa such as density, specific gravity, consistency index, swelling characteristics, and shear strength characteristics are obtained. Summaries of outputs of the researches are presented in Table 2.2 and Table 2.3.

Table2.2: Lithological properties of expansive soils of Addis Ababa [5]

Soil Properties	Standard Devia	Range of Val
Silty clay		
Moisture content	2.71	35.6-52.10
Bulk density(g/cı	0.03	1.60-1.76
Specific Gravity	0.03	2.70-2.88
Dry density(g/cm	0.04	1.10-1.30
Void ratio(e)	0.05	1.25-1.50
porosity(n)	0.11	0.53-0.61
Black clay		
Moisture content	5.24	34.00-42.60
Bulk density(g/cı	0.03	1.65-1.70
Specific Gravity	0.02	2.77-2.81
Dry density(g/cm	0.05	1.18-1.26
Void ratio(e)	0.11	1.21-1.38
porosity(n)	0.22	0.54-0.58
Gray clay		
Moisture content	3.50	38.90-45.10
Bulk density(g/cı	0.03	1.69-1.70
Specific Gravity	0.01	2.80-2.82
Dry density(g/cm	0.06	1.15-1.27
Void ratio(e)	0.09	1.26-1.43
porosity(n)	0.18	0.55-0.58

Table2.3 Range of values of index properties of expansive soils of Addis Ababa [5]

Soil t	Consistency limits			Grain size				
	liquid l	Plastic	Plasticity i	Clay co	Silt cont	Sand	Activity	Free swel
Black	98.50-10	25.50-26	75.47-76.40	75.40-79.	24.40-29.5	2.00-3.4	0.9-1.60	109.00-120
Grey	101.40-1	25.6-27.	68.30-74.90	71.10-75.	22.10-26.2	2.70-2.8	0.9-1.36	96.60-106.3
Silty	78.80-95	24.60-27	56.80-67.80	65.00-71.	36.15-46.4	2.20-3.8	0.89-1.10	86.00-117.0

2.3 Mechanics of swelling

2.3.1 General

Swelling of expansive soils will take place under change in the environment of the soil. Environmental change can consist of pressure release due to excavation, desiccation caused by temperature increase, and volume increase because of the introduction of moisture. By far the most important element and of most concern to the practicing engineer is the effect of water on expansive soil. There must be a potential gradient, which can cause water migration and a continuous passage through which water transfer can take place [1]. The potential gradient in expansive soils can be due to seasonal moisture fluctuation or thermal gradient, which can cause vapor and liquid moisture transfer. It is well recognized that the heaving of expansive soils may take place without the presence of free water. Vapor transfer plays an important role in providing the means for the volume increase of expansive soils.

2.3.2 Factors influencing Swelling

The mechanism of swelling in expansive soils is complex and is influenced by a number of factors. [2] Expansion is the result of change in the soil water system that disturbs the internal force equilibrium. The factors influencing the shrink-swell potential of a soil can be considered in three different groups. These are the soil characteristics that influence the basic nature of the internal force field, the environmental factors that influence the changes that may occur in the internal force system, and the state of stress.[2]

2.3.2.1 Soil characteristics

Soil characteristics may be considered either as microscale or macroscale factors. Microscale factors include the mineralogical and chemical properties of the soil.

Macroscale factors include the engineering properties of the soil which inturn dictated by the microscale factors.

i) Microscale factors (clay mineralogy and soil water chemistry):- clay minerals of different types typically exhibit different swelling potentials because of variation in the electrical field associated with each mineral. The swelling capacity of an entire soil mass depends on the amount and type of clay mineral in the soil, the arrangement and specific surface area of the clay particles, and the chemistry of the soil water surrounding those particles.

Soil water chemistry is important in relation to potential swell magnitude. Salt cations such as sodium, calcium, magnesium, and potassium are dissolved in the soil water and are adsorbed on the clay surfaces as exchangeable cations to balance the negative electrical surface charges. Hydration of these cations and adsorptive forces exerted by the clay crystals themselves can cause the accumulation of a large amount of water between the clay particles. In dry soils, salt cations are held close to the clay crystal surfaces by strong electrostatic forces. As water becomes available, cation hydration energies are sufficiently large to overcome interparticle attraction forces. Thus initially dessicated and densely packed particles are forced apart as adsorbed cations hydrate and become enlarged on the addition of water. [2]

ii) Macroscale factors (plasticity and density***):-*** Macroscale soil properties reflect the microscale nature of the soil. Because they are more conveniently measured in engineering work than microscale factors, macroscale characteristics are primary indicators of swelling behavior. Commonly determined properties such as soil plasticity and density can provide a great deal of insight regarding the expansive potential of the soils. Soil consistency, as quantified by the Atterberg limits, is the most widely used indicator of expansive potential. Most expansive soils can exist in a plastic condition over a wide range of moisture contents. This behavior results from the capacity of expansive clay mineral to contain large amount of water between particles and yet retain a coherent structure through the interparticle electrical forces. Soil plasticity, a useful indicator of swell potential, is influenced by the same microscale factors that control the swell potential. [2]

2.3.2.2 Environmental conditions

The potential for a soil to absorb or expel water will depend on the water content relative to the water deficiency of the soil. Initial moisture content influences the shrink-swell potential relative to possible limits, or ranges, in moisture content. Moisture content alone is not a good indicator or predictor of shrink-swell potential. Instead, the moisture content relative to limiting moisture contents such as the plastic limit and shrinkage limit must be known. Water content changes below the shrinkage limit produce little or no change in volume. There are indications that as a soil imbibes water, little volume change occurs at water content change above the plastic limit.

The availability of water to an expansive soil profile is influenced by many environmental and manmade factors. Generally, the upper few meters of the profile are subjected to the widest ranges of potential moisture variation. Also, overburden stress is low and the soil is not restrained against movement at shallow depths. This upper stratum (active zone) of the profile therefore exhibits the major part of the shrinking and swelling. Moisture variation in the active zone of a natural soil profile is affected by climatic cycles. Other obvious and direct causes of moisture variation result from altered drainage conditions or manmade sources of water, such as irrigation or leaky plumbing.

2.3.2.3 State of stress

Volume change is directly related to change in the state of stress in the soil. A reduction in the total stress due to excavation of overlying material will result in rebound and heave of the surface. Stress history is another factor which affects the swelling characteristics in that an overconsolidated soil is more expansive than the same soil of the same void ratio but normally consolidated. The thickness and location of potentially expansive layers in the profile also considerably influence potential volume change. Greatest movement will occur in profiles that have expansive clays extending from the surface to depths below the active zone.

2.4 Consolidation

2.4.1 Theories of compression and consolidation

Any structure built on the ground causes increase of pressures on the underlying soil layers. The soil layers are unable to spread laterally as the surrounding soil strata confines them. Hence there must be adjustment to the new pressure by vertical deformation. The compression of the soil mass leads to the decrease in the volume of the mass, which result in the settlement of the structure, built on the mass. The vertical compression of the soil mass under increased pressures is thus made up of the following components:

i. Deformation of the soil grains
ii. Compression of water and air with in the voids
iii. An escape of water and air from the voids

It is quite reasonable and rational to assume that the solid matter and the pore water relatively are incompressible under the loads encountered. The change in volume of the soil mass under imposed stresses must be only due to the escape of water and air.

Generally, the volume change in a soil deposit can be divided in to three stages:[12]

a) Initial consolidation:

When a load is applied to a partially saturated soil, a decrease in volume occurs due to expulsion of and compression of air in the voids. A small decrease in volume also occurs due to compression of solid particles. The reduction in volume of the soil just after the application of the load is known as initial consolidation or initial compression. For saturated soils, the initial consolidation is mainly due to compression of solid particles.

b) Primary consolidation:

After initial consolidation, further reduction in volume occurs due to expulsion of water from voids. When a saturated soil is subjected to a pressure, initially all the applied pressure is taken up by water as excess pore water, as water is almost incompressible as compared with solid particles. A hydraulic gradient develops and the water starts flowing out and a decrease in volume occurs. The decrease depends up on the permeability of the soil and is, therefore, time dependent. The reduction in volume is called primary consolidation.

In fine grained soils, the primary consolidation occurs over a long time. On the other hand, in coarse grained soils, the primary consolidation occurs rather quickly due to high permeability. As water escapes from the soil, the applied pressure is gradually transferred from the water in the voids to the solid particles.

c) Secondary consolidation

The reduction in volume continues at a very slow rate even after the excess pore water pressure developed by the applied pressure is fully dissipated and the primary consolidation is complete. This additional reduction in the volume is the called secondary consolidation. The causes for secondary consolidation are not fully established. It is attributed to the plastic readjustment of the solid particles and the adsorbed water to the new stress system. In most inorganic soil, it is generally small.

2.4.2 Factors Affecting the Consolidation Characteristics of Clay Soils

The consolidation behavior of clay soil in its natural state is highly dependent on stress history and permeability. The effects of these factors are explained below.

2.4.2.1 Stress History

The maximum stress to which the soil is subjected in the past influence the consolidation characteristics of the soil in its insitu condition. In remolded soils, because it has lost its structural characteristics as compared with its structure in its natural condition, it is inferred that a remolded soil is unsuitable for evaluating its stress history. [10]

As to the stress history, the insitu soil can be grouped in to two categories:

a) Normally consolidated soils

A normally consolidated soil is one whose present effective overburden pressure on the insitu prototype soil deposit is the maximum pressure to which the soil has ever been subjected at any time in the past history. In other words, the normally consolidated soil is one whose preconsolidation pressure is equal to its present effective overburden pressure.

b) Over-consolidated clay soil

Overconsolidated clay is one which has been completely consolidated under a large overburden pressure in the past that is larger than the present overburden pressure. The response of overconsolidated clays to applied loads is such that at early loading the soil shows relatively small decrease of void ratio with load up to the maximum effective stress to which the soil was subjected in the past. If the effective stress on the soil specimen is increased further, the decrease of void ratio with stress level will be larger.

2.4.2.2 Permeability

The expulsion of water from the voids of a saturated clay soil by an externally applied load in the consolidation process and the change in volume associated with such a process are essentially a hydraulic problem. Specifically, it is a problem of permeability of a soil to water. Therefore, the rate of consolidation depends on the permeability of the soil. The permeability of the soil by itself is a function of the soil type, size and shape of the soil particles (rounded, angular, or flaky), and thus, up on the size and geometry of voids. Also, the resistance is a function of the temperature of water (viscosity and surface tension effect). [10]

2.4.3 Theory of one-dimensional consolidation

The theory for the time rate of one-dimensional consolidation was first proposed by Terzaghi. The underlying assumptions in the derivation of the mathematical equation are the following:

1) The soil is homogeneous and isotropic
2) The soil is fully saturated
3) The soil particles and the water in the voids are incompressible. The consolidation occurs due to expulsion of water from the voids
4) Darcy's law is valid throughout the consolidation process
5) Soil is laterally confined and the consolidation takes place only in the axial direction. Drainage of water also occurs only in the vertical direction

The assumptions made by Terzaghi are not fully satisfied in actual field conditions. The results obtained from the use of the theory to practical problem are approximate. However, considering complexity of the problem, the theory gives reasonably accurate estimate of the time rate of settlement of a structure built on the soil.

The standard one dimensional consolidation test is usually carried out on saturated specimen using an Odometer [6]. In this test a small representative sample of soil s carefully trimmed and fitted into a rigid metal ring. The soil sample is mounted on a porous stone base and a similar stone is placed on top to permit water, which is squeezed out of the sample to escape freely at the top and bottom. Prior to loading, the height of the sample should be accurately measured. Also, a micrometer dial is mounted in such a manner that the vertical strain in the sample can be measured as loads are applied. The consolidation test apparatus is designed to permit the sample to be submerged in water during the test to simulate the position below a water table of the prototype soil sample from which the test sample was taken. Loads are applied in steps in such a way that the successive load intensity, P, is twice the preceding one; the load intensities commonly used being ¼, ½, 1, 2, 4,8,16 kg/cm^2. Each load is allowed to stand until primary consolidation is practically ceased. The dial readings are taken at elapsed time of 0, .0.25, 0.50, 1, 2, 4, 8, 15, 30,60minute.......24hours. After the greatest load required for the test has been applied to the soil sample, the load is removed in decrements to provide data for plotting the expansion curve of the soil in order to learn its elastic properties and magnitude of plastic or permanent deformation.

The consolidation characteristics (or parameters) of a soil which are the compression index, Cc, and the coefficient of consolidation, Cv, will be determined from the test. The compression index relates to how much consolidation or settlement will take place. The coefficient of consolidation relates to how long it will take for an amount of consolidation to take place [9]. The results of the odometer test are usually presented in the form of an e-P, e-log P, and dial reading- time plots.

2.4.3.1 Compression index

The compression index, Cc, is equal to the slope of the linear portion of the void ratio versus log pressure plot. Thus

$$C_c = \frac{\Delta e}{\log(\frac{P_0 + \Delta P}{p_0})}$$

The compression index is useful for the determination of the settlement in the field.

2.4.3.2 Coefficient of consolidation

A factor involved in characterizing the rate of consolidation of a soil is the one called the coefficient of consolidation, C_v, expressed as

$$C_v = \frac{(1+e)k}{a_v.\gamma_w} = \frac{k}{m_v.\gamma_w}$$

Because of the fact that during the process of consolidation k and m_v are assumed to be constant, the coefficient of consolidation C_v during the process of consolidation of the clay is constant. [10]

The coefficient of consolidation C_v as determined by Casagrande's semi logarithmic plot method is

$$C_v = \frac{(0.196).H^2}{t_{50}} .....[\frac{cm^2}{s}]$$

The C_v value as determined by Taylor's square root of time fitting method is

$$C_v = \frac{(0.848).H^2}{t_{90}} .....[\frac{cm^2}{s}]$$

2.4.3.3 Preconsolidation pressure

A soil may have been preconsolidated during the geologic past by the weight of an ice which has melted away, or by other geologic overburden or and structural loads which no longer exist. For example, thick layers of overburden soil may have been eroded or excavated away or heavy structures may have been torn down. Also capillary pressures which may have acted on the clay layers in the past may have been removed for one reason or another.

The practical significance of the preconsolidation load appears in calculating settlements of structures. [10]

The relative amount of preconsolidation is usually reported as the overconsolidation ratio (OCR) defined as

$$OCR = \frac{P_c}{p_o}$$

3-Laboratory testing

3.1 General

Laboratory determination of the consolidation characteristics of clay soils is usually carried out on saturated soil using an Odometer. The swelling characteristics of expansive soils can also be determined conveniently using an Odometer. In Odometer test, only one dimensional consolidation and swelling characteristics of the soil are determined. The main purpose of the consolidation test is to obtain information on the compression properties of a saturated soil for use in determining the magnitude and rate of settlement of structures. In general, the consolidation test of a soil should furnish the following information:

i. the relationship between time and percent consolidation
ii. the relationship between the increasing or decreasing load on the soil and the change in the void ratio of the soil

data on permeability of the soil as a function of that particular load

3.2 Sample preparation

For the purpose of the test, sampling of expansive soil was started in the month of May which is usually a dry month. The initial moisture content of the expansive soil in this month was about 40%. Because of the difficulty to find samples of different initial moisture content, it was necessary to prepare disturbed soil samples of lower initial moisture content. Therefore, for the laboratory consolidation testing, both disturbed and undisturbed samples are taken from different sites in Addis Ababa, which are covered by expansive soil.

Sample locations are selected in such a manner that to conform to the sites chosen for the researches on other engineering properties of the expansive soils of Addis Ababa mentioned in section 1.1 of this paper. The samples used in the laboratory tests along with their initial moisture content and dry density are presented in Table 3.1 and Figure 3.1.

Table 3.1: Sample of expansive soil used for consolidation -swell test and their characteristic prior to the test

Sample location	Pit No.	Sample No.	Depth of sampling	Sample type	moisture content	Dry density g/cm^3
Bole	1	1	1.5	Undisturbed	37.75	1.22
		2	1.5	Undisturbed	37.75	1.17
		3	1.5	Undisturbed	40.58	1.18
Gergi	1	4	2.2	Undisturbed	38.94	1.14
		5	2.2	Undisturbed	40.09	1.18
	2	6	2	Undisturbed	41.29	1.2
		7	2	Undisturbed	40.14	1.19
	3	8	1.8	Undisturbed	37.73	1.22
		9	1.8	Undisturbed	39.88	1.188
Bole	2	10	1.5	Disturbed	36.39	1.25
		11	1.5	Disturbed	36.39	1.34
		12	1.5	Disturbed	27.06	1.34
	1	13	1.4	Disturbed	32.13	1.286
		14	1.4	Disturbed	32.13	1.42
		15	1.4	Disturbed	25.93	1.33
Ayat	1	16	1.7	Disturbed	25	1.16
		17	1.7	Disturbed	25	1.43
		18	1.7	Disturbed	31.93	1.244
		19	1.7	Disturbed	31.93	1.38
		20	1.7	Disturbed	36.06	1.247

3.2.1 Undisturbed samples

Undisturbed samples of expansive soil are recovered from Bole and Gergi area with sampling tubes of diameter 89mm.The sample for odometer test is extracted from the tube sampler using the odometer ring which has a diameter of 7cm and a height of 2cm. The sample is then prepared for the test by trimming the ends.

3.2.2 Disturbed samples

Disturbed samples of expansive soil are also taken from Bole and Ayat area. The disturbed samples are air dried, sieved and soaked for 24 hours. The wet soil sample is then compacted to the required density which is in the range of the density of the natural deposit and the test sample is extracted using the ring of the odometer. The samples are also prepared to have different initial moisture content.

3.3 Test methods

In order to achieve the intended objectives, a consolidation- swell test is carried out on eleven disturbed and nine undisturbed samples of expansive soils obtained from six test pits in different parts of Addis Ababa where expansive soil is dominant. The ASTM procedures D2435-test methods for one-dimensional consolidation properties of soil, and D4546-Standard test method for one dimensional swell or settlement potential of cohesive soils, are followed.

The consolidation-swell test involves an initial loading of an unsaturated sample to a prescribed stress. The sample is then allowed to swell under that load when water is added. The initial load may represent overburden surcharge, overburden plus structural load, or some other arbitrary surcharge. After swelling, the sample is further loaded and unloaded in the conventional manner. [2] The swelling pressure is usually defined as the pressure required recompressing the fully swollen sample back to its initial volume. An idealized plot of consolidation swell test data is shown in Figure3.2.

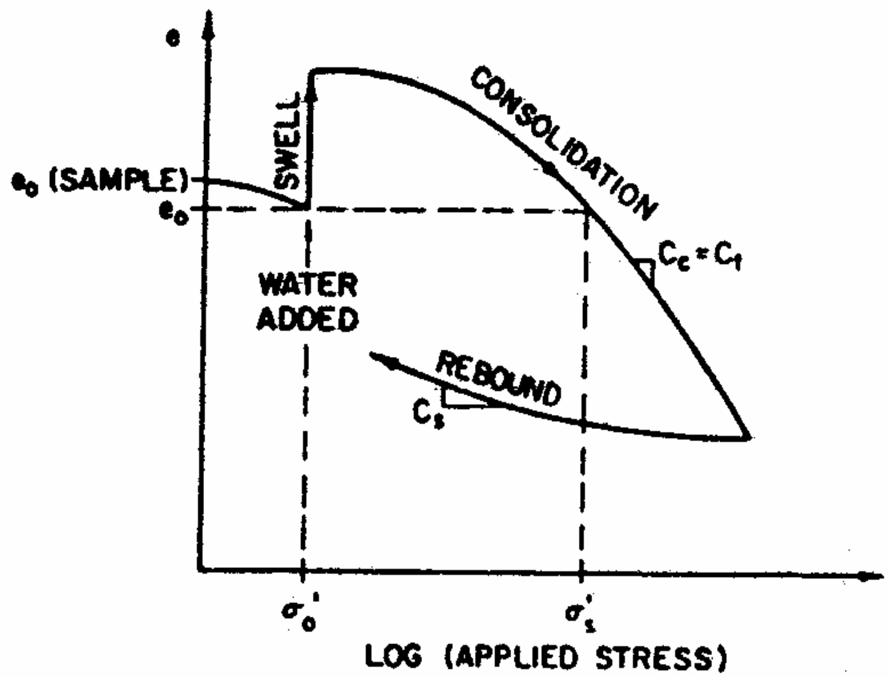


Fig 3.2 Typical plot of consolidation-swell test [2]

3.4 Test results

A consolidation-swell test is carried out on nine undisturbed and eleven disturbed samples of expansive soils of Addis Ababa. The test results obtained from consolidation swell test on disturbed as well as undisturbed samples of expansive soil of different density and initial moisture content are presented in the form of void ratio- log pressure, and dial reading- square root of time plots.

The test results are grouped into two for the purpose of the analysis. The first group includes test results of the undisturbed samples, typical of which are shown in Figure 3.3 and Figure 3.4. These results are used to determine the range of values of the consolidation parameters, Cc and Cv. the second group includes test results of both disturbed and undisturbed samples used to determine the relationship between consolidation and swelling characteristics of expansive soil of Addis Ababa. In this regard a number of pairs of soil samples of the same dry density and different moisture content (Figure 3.5) as well as soil samples of different dry density and the same moisture content (Figure 3.6) are considered. The test results presented in the main body of the paper are only the typical ones which can better explain the argument. The test results of all the other samples are in the appendices.

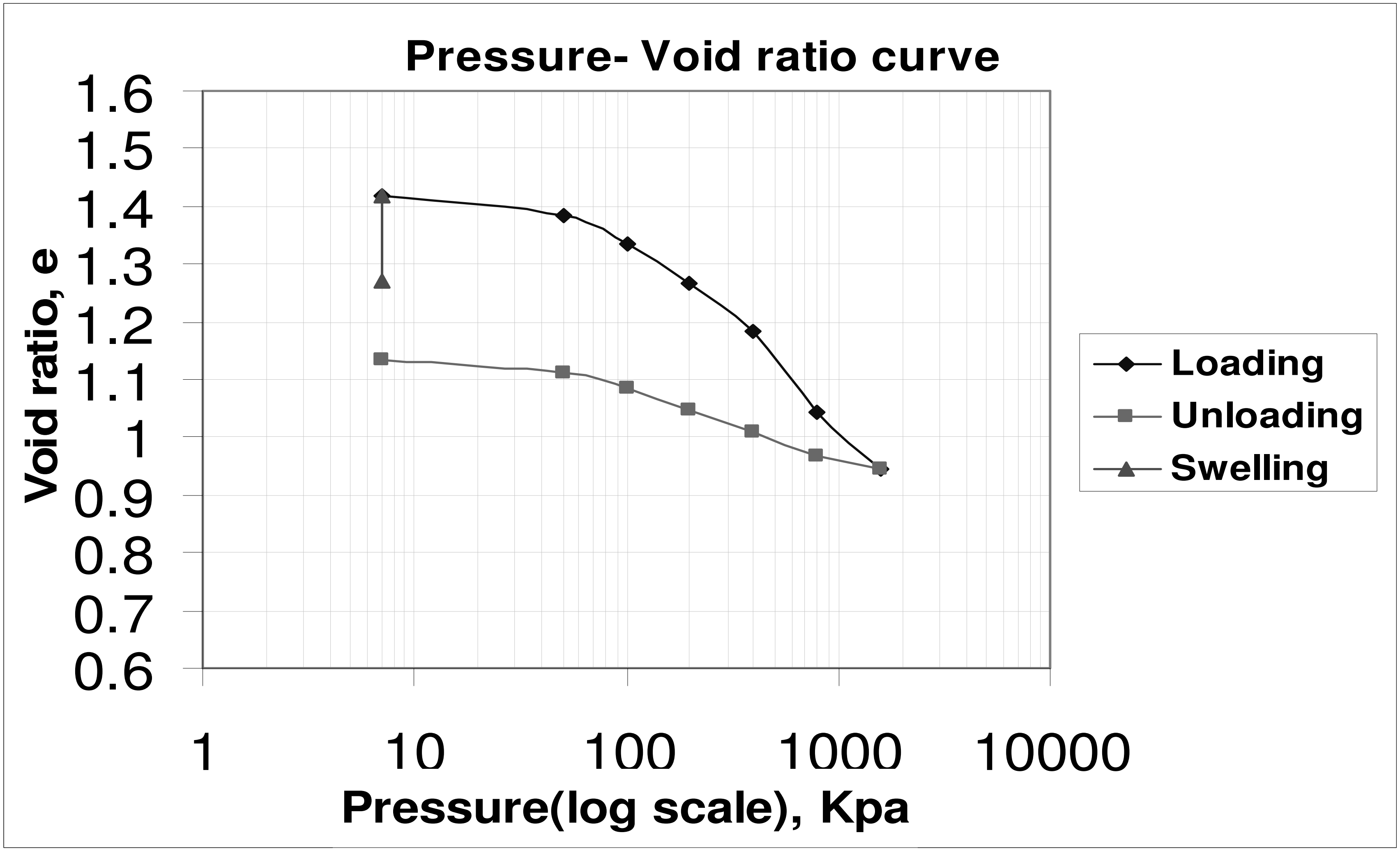


Fig 3.3 Typical plot of Void ratio versus log Pressure of undisturbed sample of expansive soil of Addis Ababa (Sample-2)

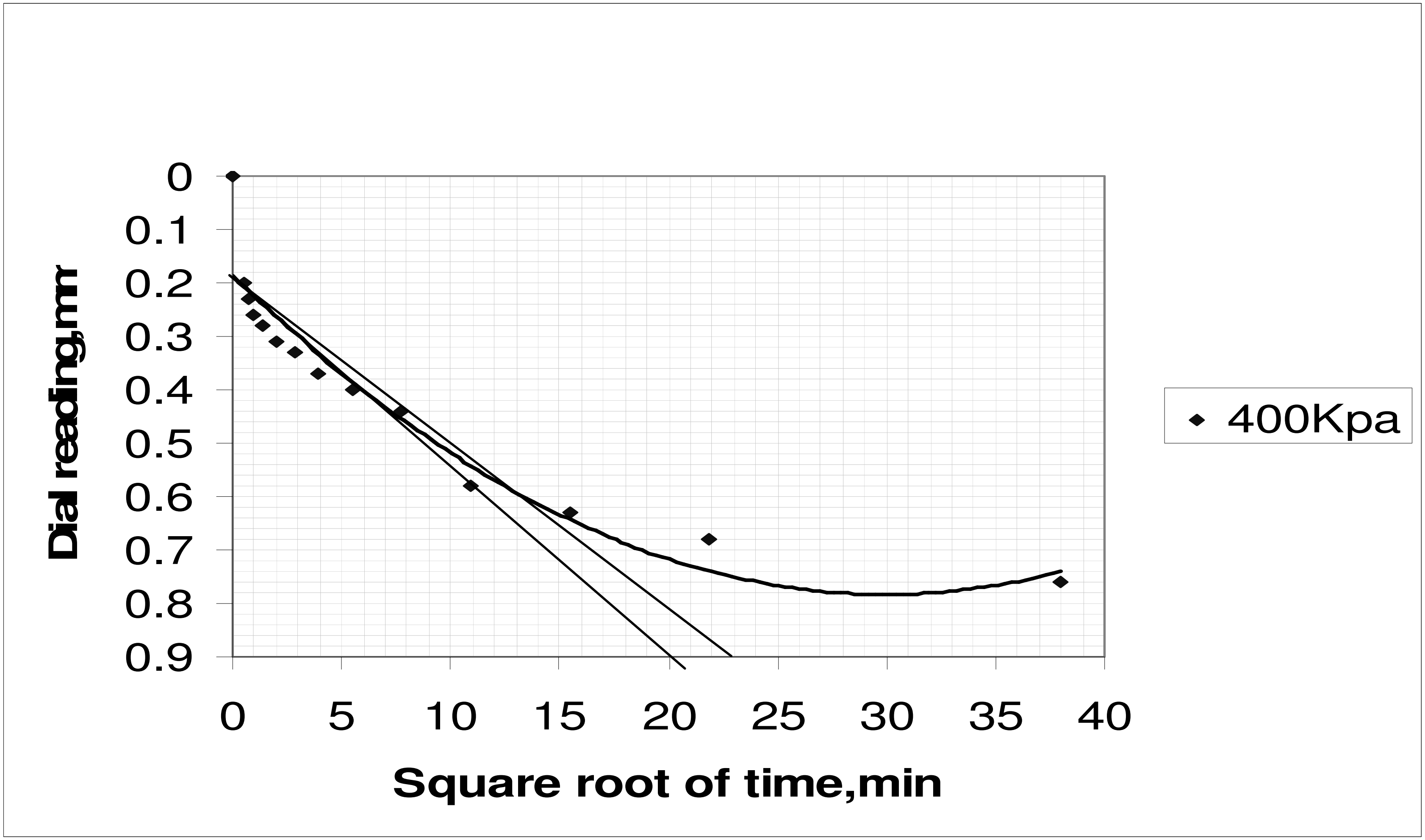
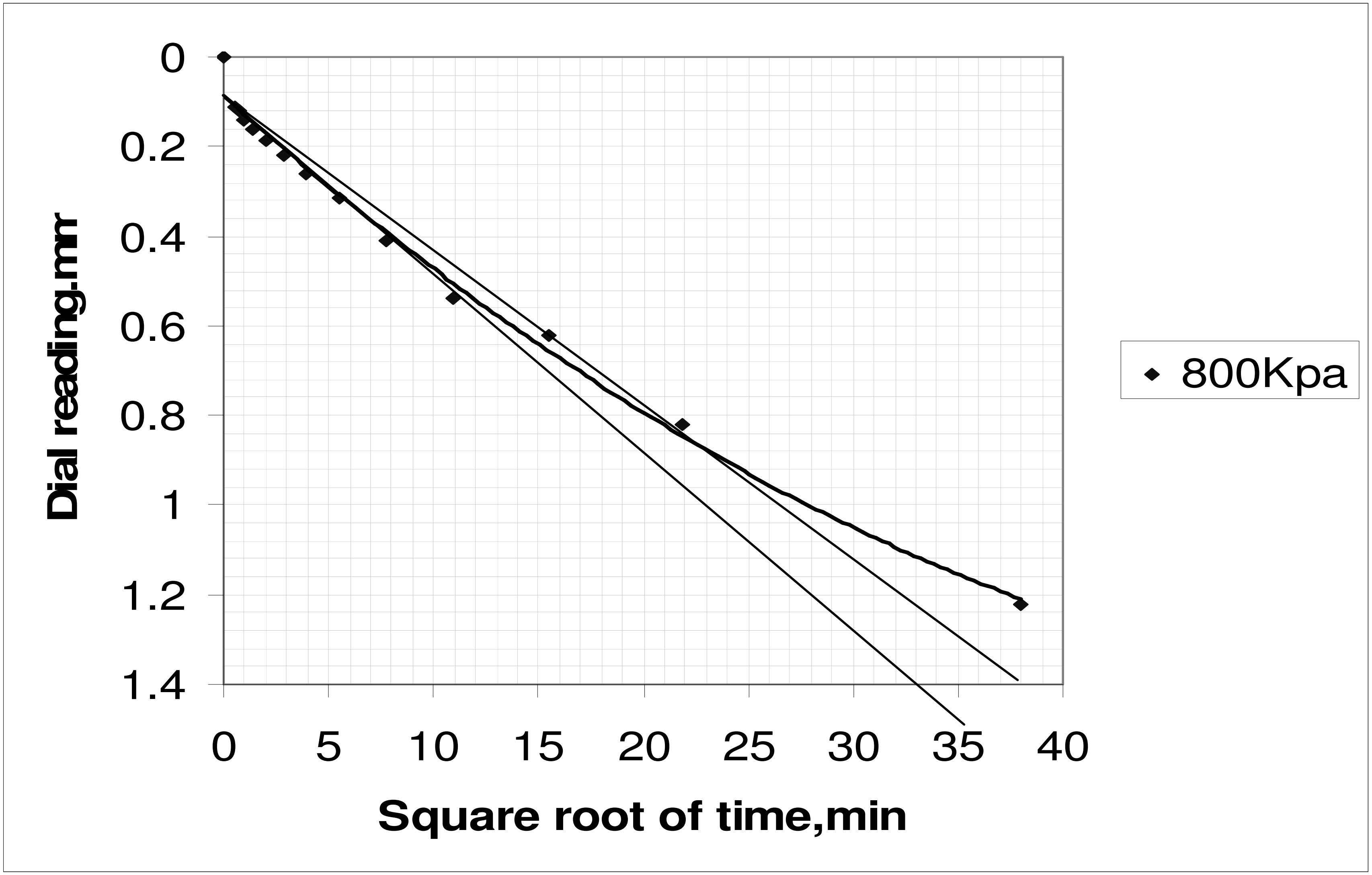


Fig 3.4a Typical plots of Dial reading versus Square root of time of undisturbed sample of expansive soil of Addis Ababa(P=400Kpa)



3.4b Typical plots of Dial reading versus Square root of time of undisturbed sample of expansive soil of Addis Ababa(P=800Kpa)

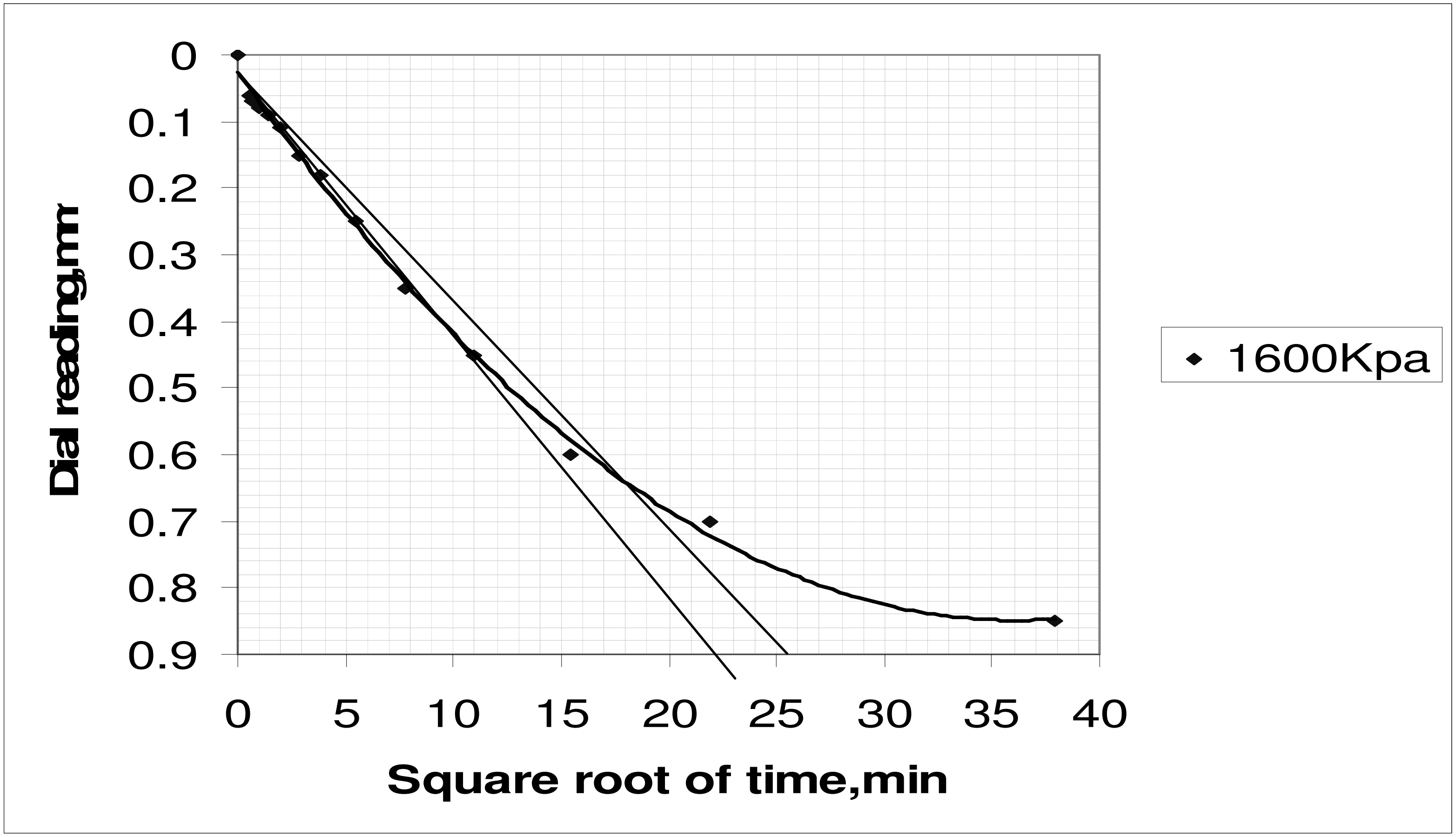


Fig3.4c Typical plots of Dial reading versus Square root of time of undisturbed sample of expansive soil of Addis Ababa(P=1600Kpa)

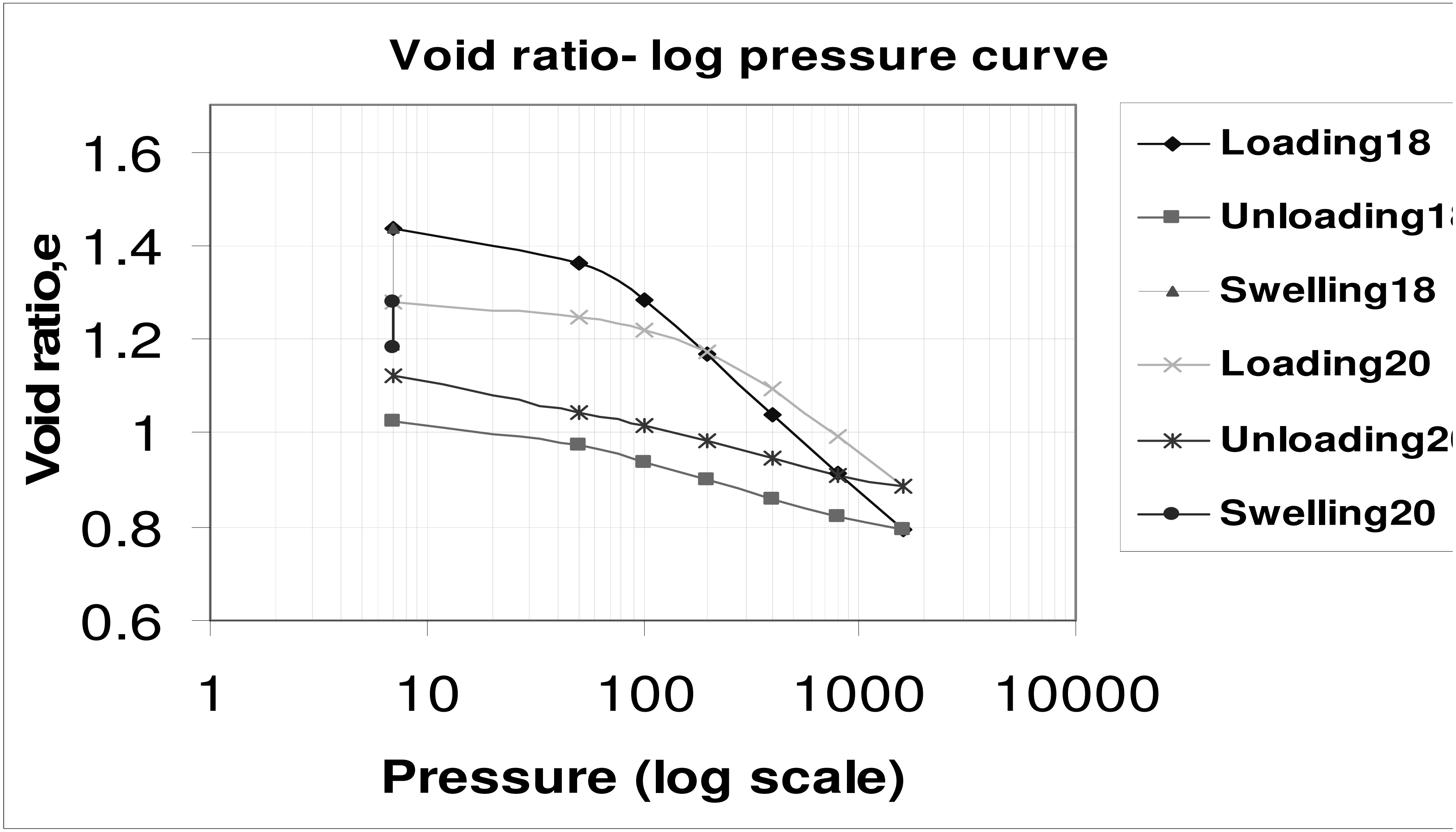


Fig 3.5 Plot of Void Ratio Versus Log-Pressure for soil samples of the same dry density and different moisture content Sample-18(ω=31.93%,ρ_d=1.244g/cm^3) and Sample-20 (ω=36.06%,ρ_d=1.247g/cm^3)

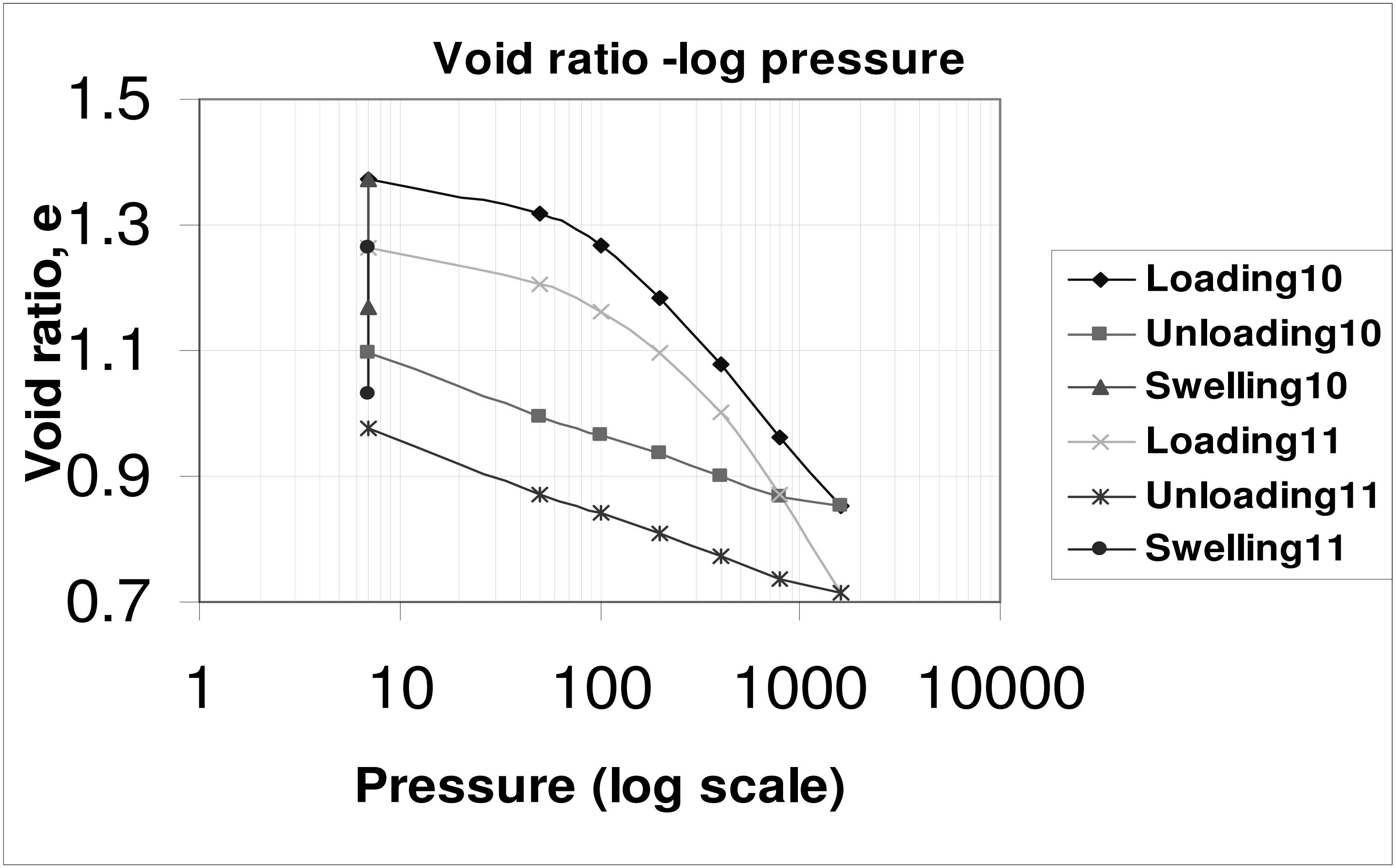


Fig 3.6 Plot of Void Ratio Versus Log-Pressure for soil samples of different dry density and the same moisture content for Sample-10(ω=36.39%,ρ_d=1.25g/cm^3) and Sample-11 (ω=36.39%,ρ_d=1.34g/cm^3)

3.5 Discussion on test results

The soil sample allowed to saturation in an oedometer undergoes swelling with a magnitude dependent on the change in moisture content and initial dry density. Early loading of the sample up to a pressure equal to the swelling pressure was to maintain the initial void ratio before saturation. Further loading beyond the swelling pressure has caused the soil to undergo consolidation from which the consolidation characteristic has been determined. During unloading there is a permanent deformation for each load decrement so that after the removal of all the applied loads the void ratio was less than the initial one. This indicates that the swelling due to the moisture change at the beginning of the test will not be observed after the sequential loading and unloading.

The insitu overburden pressure for the soil samples was in the range of 25-35kPa that is much less than the swelling pressure. For any load less than the swelling pressure consolidation will not take place. For consolidation to take place, the soil should be subjected to a pressure greater than the swelling pressure. The stress history of expansive soil should be determined not only based on the relative magnitude of the present overburden pressure and the maximum pressure to which the soil was subjected in its history but also on the swelling pressure. An expansive soil is said to be overconsolidated when the maximum pressure to which the soil was subjected in the past in greater than both the present overburden pressure and the swelling pressure otherwise, it is normally consolidated.

The consolidation swell test on undisturbed samples of expansive soil has shown that the e-logP curve for pressure greater than the swelling pressure is linear. It can be inferred from the trend of the curve that the insitu soil is normally consolidated.

Table 3.2 Consolidation and swelling test results

Sample location	Sample No.	Depth of sampling,m	initial moisture content, %	Dry density g/cm^3	Magnitude of swelling,mm	Compression index, Cc
Bole	1	1.5	37.75	1.22	2.07	0.344
	2	1.5	37.75	1.17	1.31	0.373
	3	1.5	40.58	1.18	1.12	0.345
Gergi	4	2.2	38.94	1.14	0.74	0.362
	5	2.2	40.09	1.18	1.75	0.381
	6	2	41.29	1.2	1.49	0.384
	7	2	40.14	1.19	1.26	0.358
	8	1.8	37.73	1.22	1.3	0.388
	9	1.8	39.88	1.188	1.63	0.336
Bole	10	1.5	36.39	1.25	1.87	0.377
	11	1.5	36.39	1.34	2.3	0.476
	12	1.5	27.06	1.34	4.69	0.479
	13	1.4	32.13	1.286	3.29	0.443
	14	1.4	32.13	1.42	3.35	0.395
	15	1.4	25.93	1.33	5.57	0.585
Ayat	16	1.7	25	1.16	2.16	0.579
	17	1.7	25	1.43	5.42	0.678
	18	1.7	31.93	1.244	2.3	0.443
	19	1.7	31.93	1.38	2.28	0.419
	20	1.7	36.06	1.247	0.92	0.342

In expansive soils, larger moisture change implies higher degree of disturbance in the soil structure. But the influence of the disturbance on the consolidation characteristics of expansive soil is not similar to non-expansive clay soils. In non-expansive clay soils, a soil subjected to more disturbances show a flatter e-log P curve. Whereas, the laboratory test results on expansive soil of Addis Ababa have shown that the soil subjected to larger moisture change (swelling) exhibit a steeper e-logP plot.(Fig.3.5, Fig.A.1, Fig.A.2 ,Fig.A.3, Fig.A.4)

Consolidation-swell test on remolded samples of expansive soil with different initial moisture content and dry density has given a result, which is different from the usual trend in non-expansive clay soils. Tests on remolded samples prepared to have the same dry density and different moisture content has shown a different consolidation characteristics, i.e. different compression index. (Fig.3.5, Table 3.2) In Figure 3.5 and Table 3.2 for instance, soil sample 18 and sample 20 had the same dry density but different initial moisture content. The corresponding compression indices obtained are 0.443 and 0.342, respectively. It can be easily observed that the samples subjected to more swelling show a larger value of compression index. The same observation can also be made on the other test results in appendix A. The effect of swelling on consolidation characteristics can also be explained by using soil samples of the same initial moisture content and different dry density (different swelling). Accordingly, tests on remolded samples of different dry density and the same moisture content have also indicated that the consolidation behavior of expansive soil is different from that of non expansive clay soils.(Fig3.6, Table 3.2) In non expansive clay soils, a soil with larger dry density will have compression index smaller than the same soil of lesser dry density, whereas, in expansive soils, a different consolidation characteristics is observed. As can be seen from Table 3.2, test results of sample 10 and sample 11 gave compression index values of 0.377 and 0.476, respectively. A higher value of compression index is observed in the denser soil (sample 11).

The observations made from the laboratory tests leads one to identify an additional factor, which governs the consolidation behavior of expansive soils. As can be seen from the test results, those factors that affect the swelling characteristics of expansive soils have also affected the consolidation characteristics of the expansive soil. Therefore, the additional factor anticipated to have an effect on consolidation characteristics of expansive soils is the swelling.

It is known that consolidation is the property of the soil mass that is highly dependent on permeability which inturn depends on the structural arrangement of soil particles. On the other hand, swelling is the property of the soil particle, which depends on the mineralogy of the soil particle. In effect, both phenomena bring about volume change in the soil mass. Swelling is a result of disturbance in internal stress equilibrium of the soil particle by change in moisture content. To attain new internal stress equilibrium, the soil particles start to swell and a new particle arrangement will take place in the soil mass. Therefore, Swelling brings about a soil of different particle arrangement and different consolidation characteristics. The assumption in the theory of consolidation that says that volume change in the soil particle is negligible will not be reasonable in case of expansive soil.

The undisturbed samples used in the test were found to have a dry density of $1.14 g/cm^3$ to $1.24 g/cm^3$ and moisture content with in the range 37.73% to 41.29%. The consolidation swell test on these samples has indicated that the compression index of the Addis Ababa expansive soil is with in the range of 0.3-0.4. This shows that there is no significant variation in the compression index from place to place in the given range of moisture content which is 37.5-41.5%. As compared to the compression index of red clay soils of Addis Ababa, the expansive soils have larger values of compression index. [16][17]

The coefficient of consolidation of the undisturbed samples varies from 0.139-$0.356 m^2$/year. It is found that the values of the consolidation coefficient of expansive soils of Addis Ababa are smaller than the corresponding values of red clay soil in Addis Ababa [16] [17] and in other parts of the world [Table 3.3] . Whereas, the values are closer to the coefficient of consolidation of wet expansive soil of Jordan.[Table 3.3]

It is also observed that the potential swelling of the expansive soil ranges from 3.7% to 10.3%. This does not mean that the expansive soil of Addis Ababa has low degree of expansion. But, the results are smaller due to the fact that the initial moisture content of the sample during the test was very high. Index property and free swell tests have shown that expansive soil of Addis Ababa has high degree of expansion. The swelling pressure of the soil is also varies from 80kPa to 300kPa. These values conform to the results obtained in recent researches on the swelling pressure of expansive soil of Addis Ababa.[4]

Table 3.3 Typical Values of the Coefficient of Consolidation (c_v) in the world.[15]

Soil Type		c_v (m^2/year)
Boston blue clay (CL) (Load, and Luscher;1965)		12 ± 6
Organic silt (OH) (Lowe, Zacheo, and Feldman;1964)		0.6 - 3
Glacial lake clays (CL) (Wallace, and Otto, 1964)		2 - 2.7
Chicago silty clay (CL) (Terzagi, and Peck; 1967)		2.7
Swedish medium sensitive clays (CL-CH) (Holts, and Broms; 1972)		0.1 - 0.2
San Francisco Bay Mud (CL) (Leonards, and Girault; 1961)		0.6 - 1.2
Mexico City clay (MH) (Leonards, and Girault, 1961)		0.3 - 0.5
Jordan (Amman city) clay (CH) (The Authors)	Wet soil	0.1 - 0.4
	Dry soil	0.1-0.2, 0.4-0.8, 0.8-1.6

4-Conclusion

1- It is known that the magnitude of the swelling of expansive soils varies with environmental conditions and it is observed from this research that the consolidation characteristic of expansive soils is influenced by the swelling. Therefore, the consolidation characteristic of expansive soil is not inherent property of the soil mass.

2- It is pointed out from this thesis that consolidation parameters are not constant for a given expansive soil. But the Atterburg limits are inherent property of the given soil. Therefore, empirical relations developed to determine compression index from the plasticity (Liquid limit) in non expansive soil will no more be applied for expansive soil.

3- This thesis has shown that the compression index, Cc, and coefficient of consolidation, Cv, of expansive soil of Addis Ababa as determined from the consolidation-swell test on undisturbed samples ranges from 0.3-0.4 and from 0.139-0.356 m^2/year, respectively in an initial moisture content range of 37.5-41.5%.

4- Laboratory tests conducted on expansive soil of Addis Ababa in this research work have shown that the value of the consolidation parameters of the soil are smaller than the values for clay soil in other parts of the world. This means that the soil has relatively lesser compressibility and the rate of consolidation is also smaller as well.

5- Recommendation

To increase the level of accuracy of the given range of values of consolidation parameters of expansive soil and to verify further the relationship between consolidation and swelling as well as to develop a quantitative relationship if any, it is recommended to carry out an extensive laboratory testing in future research programs with more number of samples.

6-References

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Appendix A

Consolidation- Swell test results of disturbed samples taken from expansive soils of Addis Ababa

The plots in appendix-A are presented in such a way that each pair in each plot shows the difference in consolidation characteristics of the samples with different dry density and different initial moisture content, keeping the other factors constant. Figures A.1, A.2, A.3, and A.4 show the influence of initial moisture content and Figures A.5, A.6, A.7, and A.8 show the influence of initial dry density on the consolidation characteristics of expansive soils. Figure A.9 is used to evaluate the sensitivity of the samples to remolding.

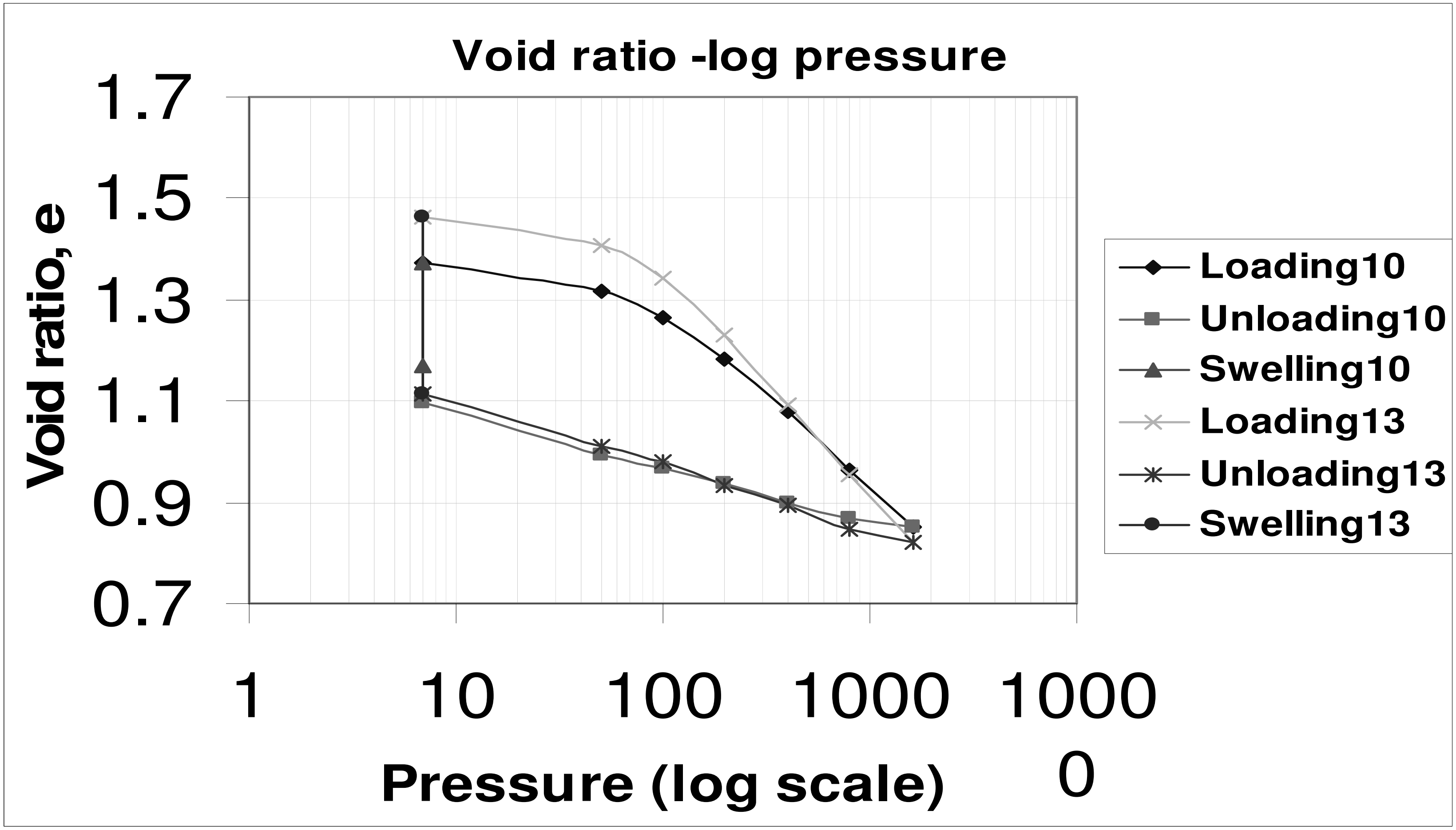


Fig A.1, Plot of Void Ratio Versus Log-Pressure for Sample-10(ω=36.39%,ρ_d=1.25g/cm^3) and Sample-13 (ω=32.13%,ρ_d=1.286g/cm^3)

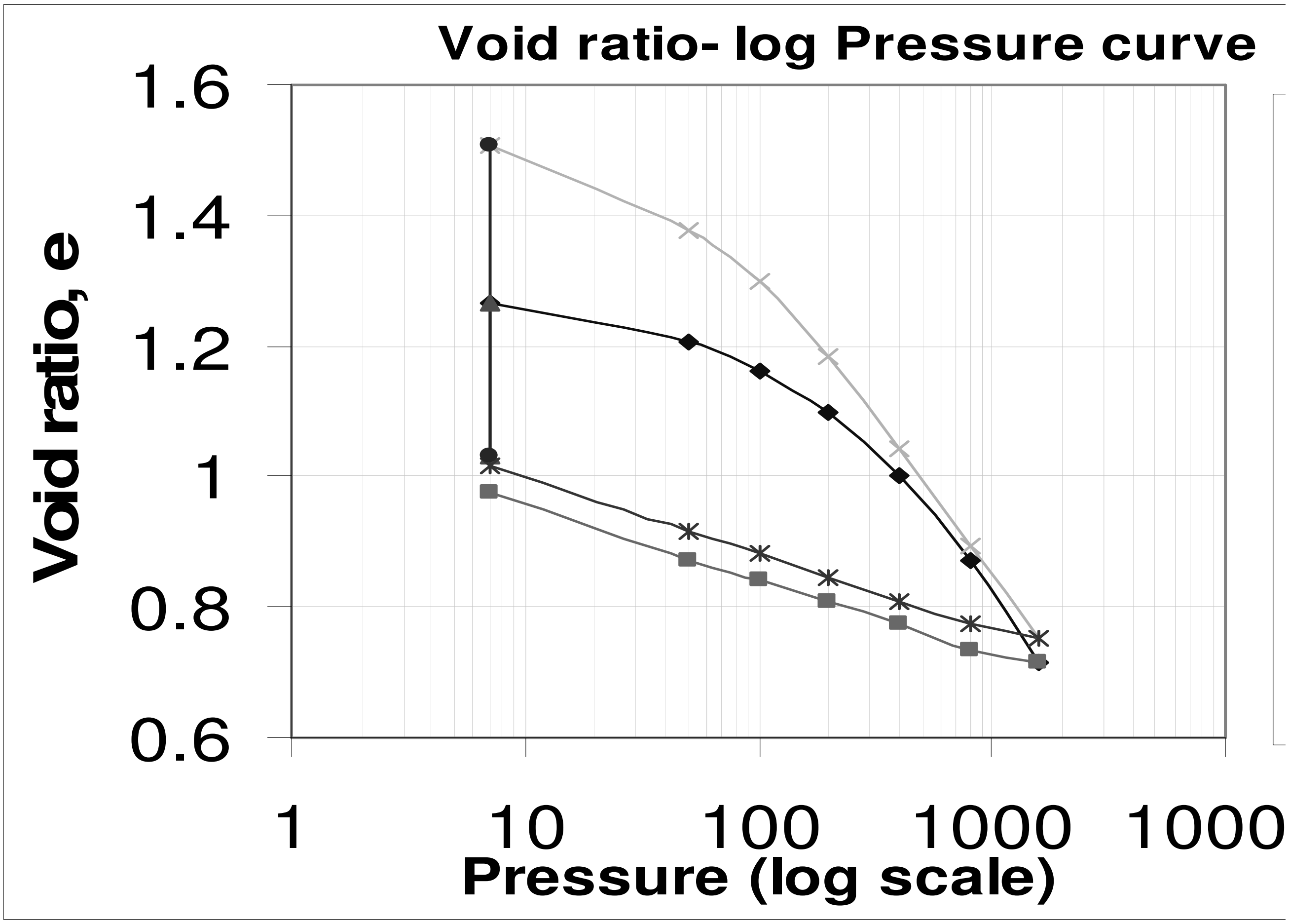


Fig A.2, Plot of Void Ratio Versus Log-Pressure for Sample-11(ω=36.39%,ρ_d=1.34g/cm^3) and
Sample-12 (ω=27.06%,ρ_d=1.34g/cm^3)

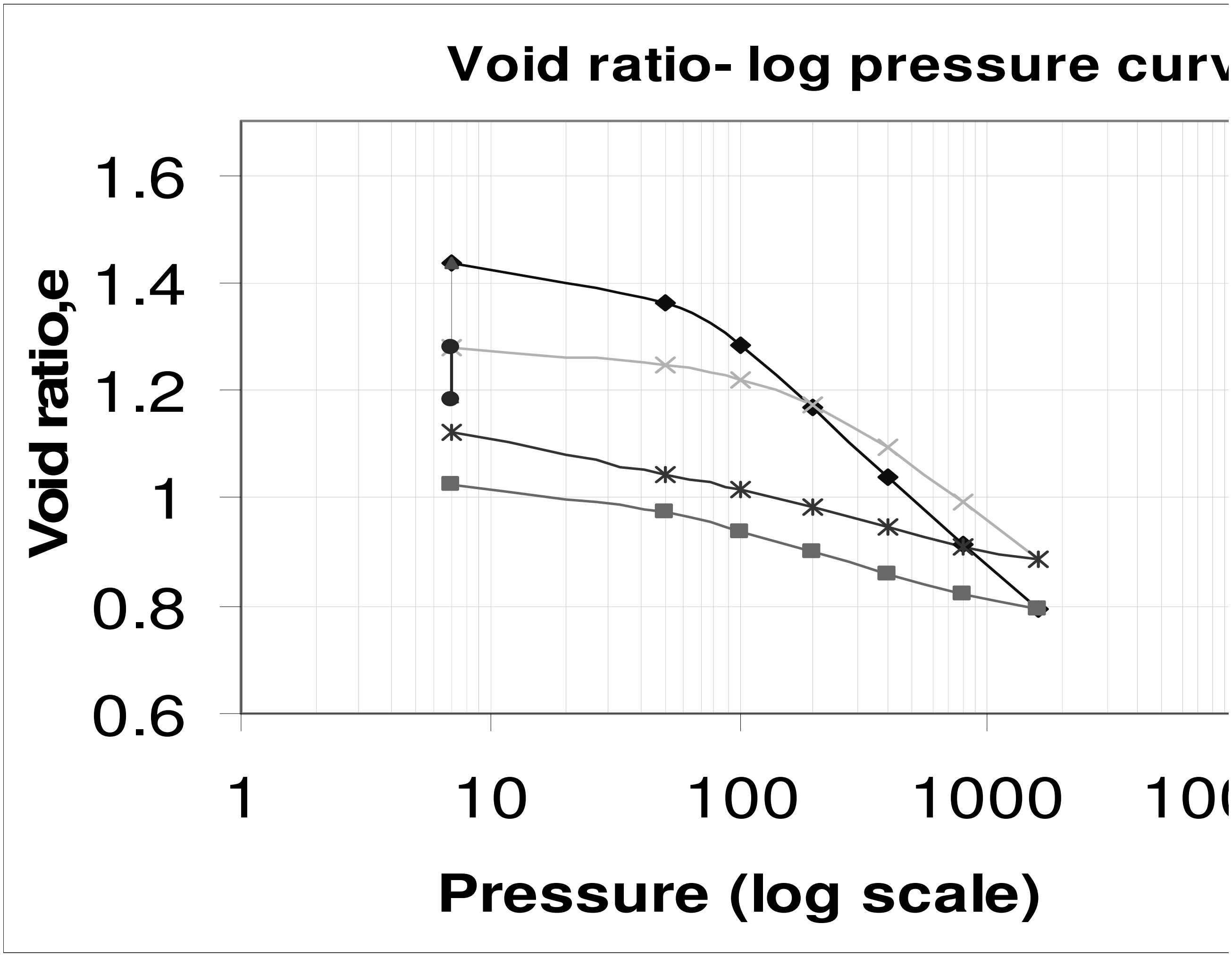


Fig A.3, Plot of Void Ratio Versus Log-Pressure for Sample-18(ω=31.93%,ρ_d=1.244g/cm^3) and
Sample-20 (ω=36.06%,ρ_d=1.247g/cm^3)

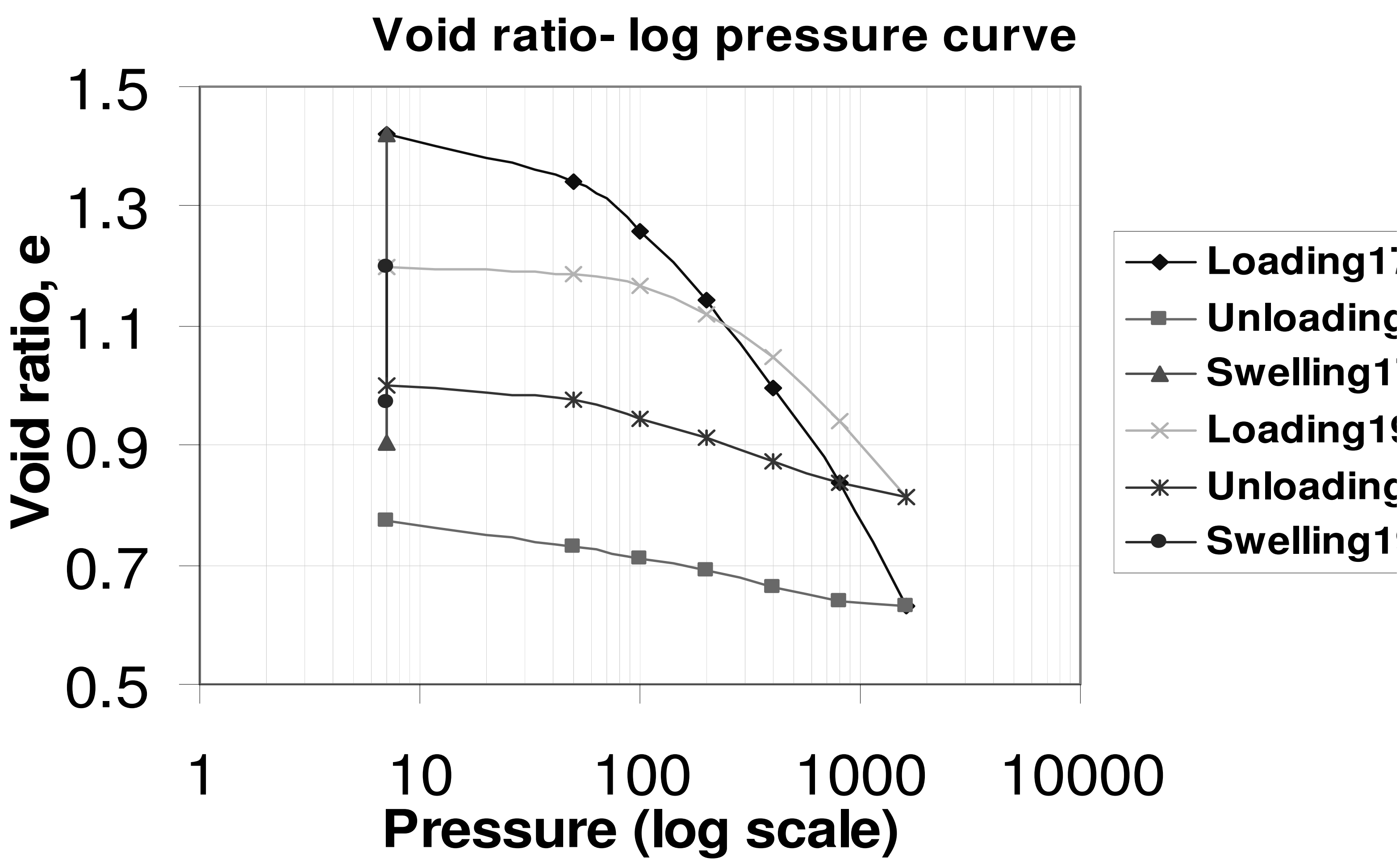


Fig A.4, Plot of Void Ratio Versus Log-Pressure for Sample-17(ω=25%,ρ_d=1.43g/cm^3) and

Sample-19 (ω=31.93%,ρ_d=1.38g/cm^3)

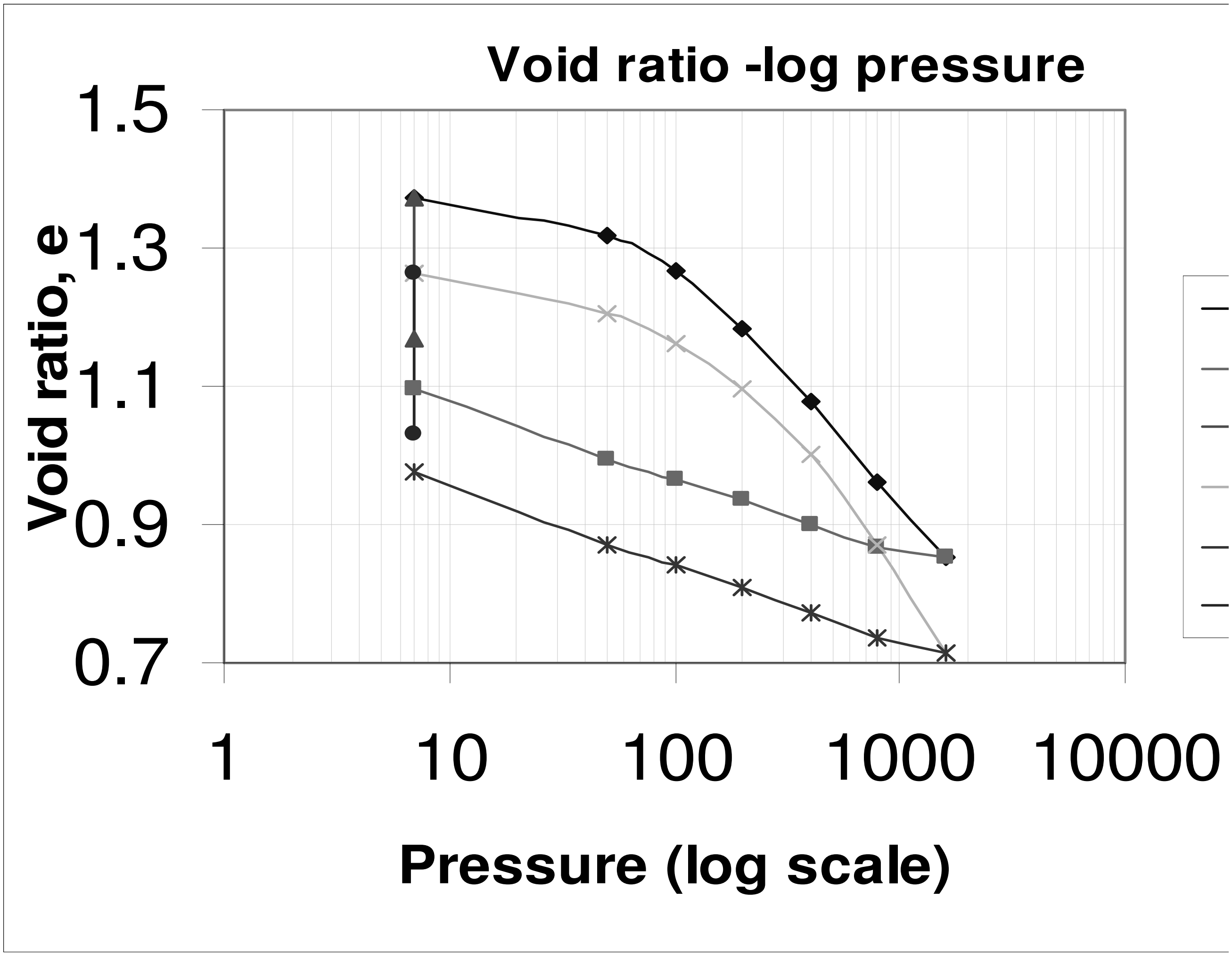


Fig A.5, Plot of Void Ratio Versus Log-Pressure for Sample-10(ω=36.39%,ρ_d=1.25g/cm^3) and
Sample-11 (ω=36.39%,ρ_d=1.34g/cm^3)

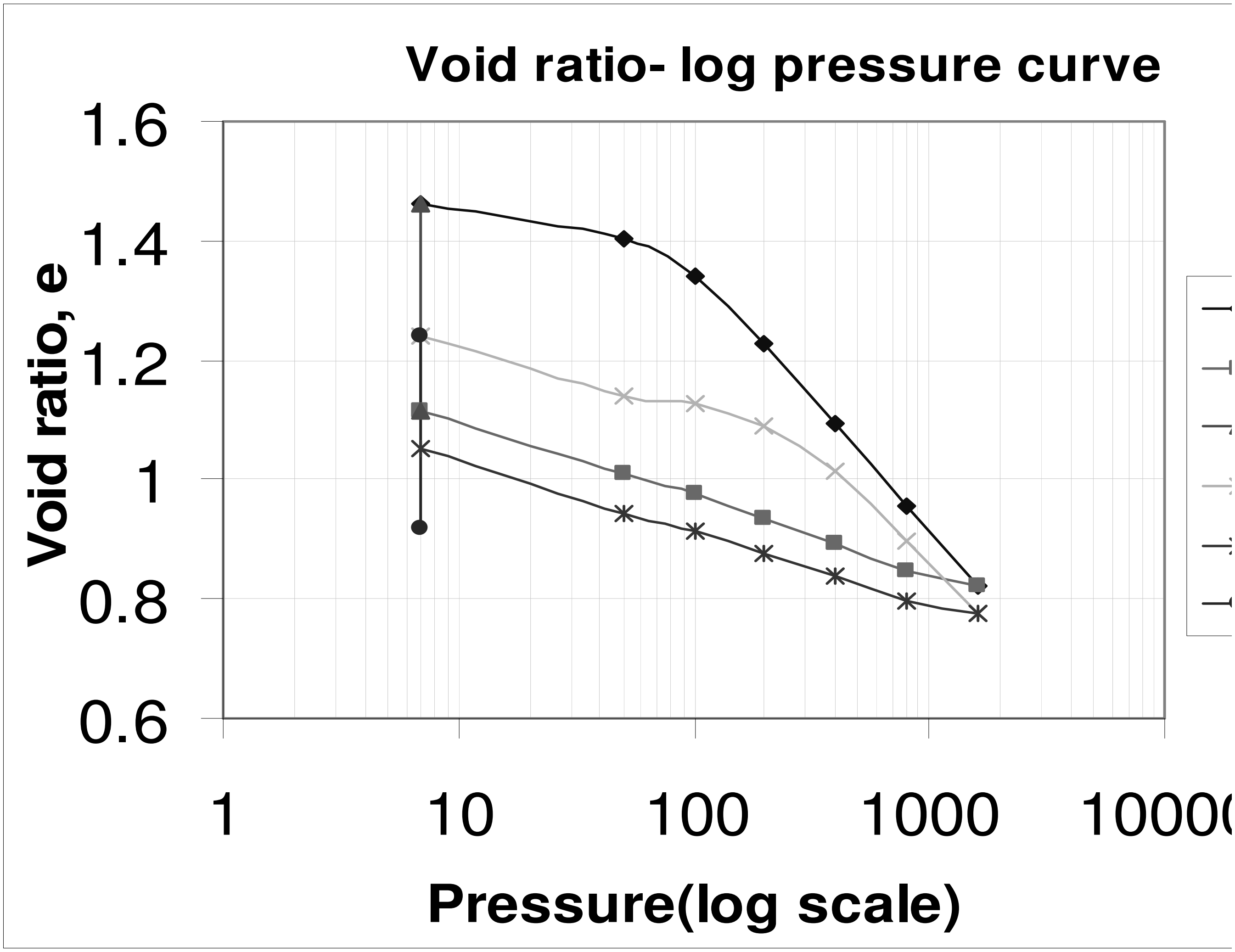


Fig A.6, Plot of Void Ratio Versus Log-Pressure for Sample-13(ω=32.13%,ρ_d=1.286g/cm^3) and
Sample-14 (ω=32.13%,ρ_d=1.42g/cm^3)

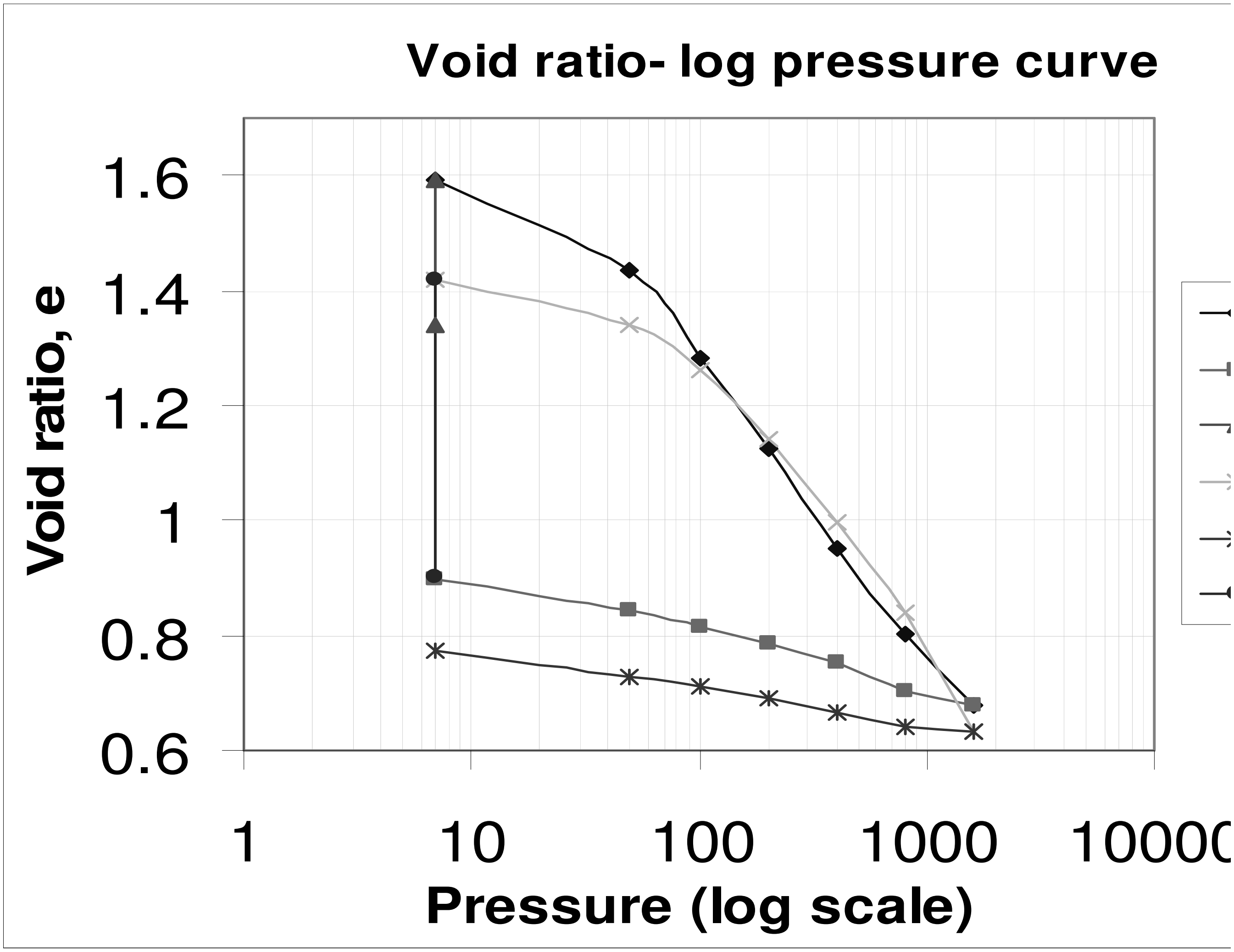


Fig A.7, Plot of Void Ratio Versus Log-Pressure for Sample-16(ω=25%,ρ_d=1.16g/cm^3) and
Sample-17 (ω=25%,ρ_d=1.43g/cm^3)

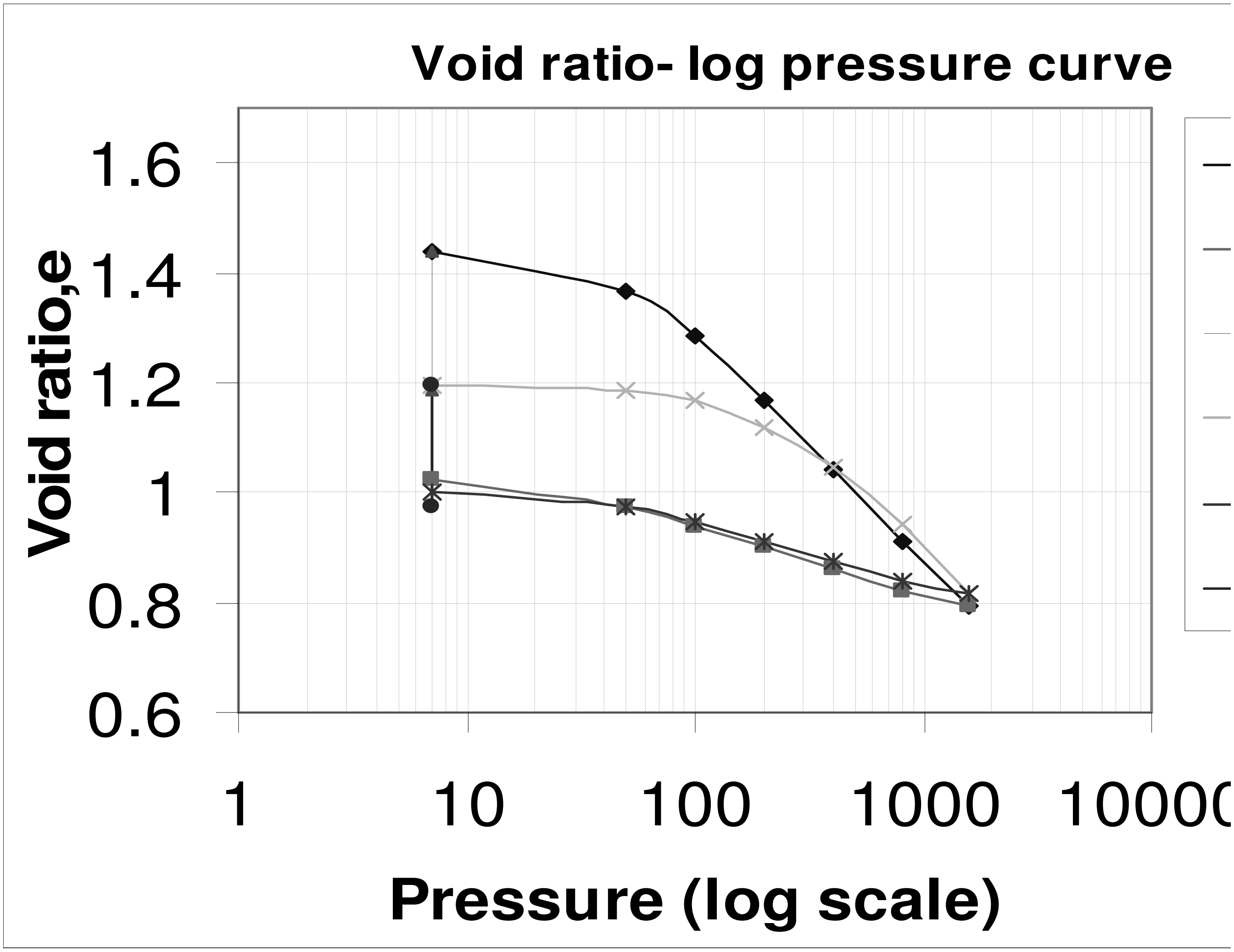


Fig A.8, Plot of Void Ratio Versus Log-Pressure for Sample-18(ω=31.93%,ρ_d=1.244g/cm^3) and Sample-19 (ω=31.93%,ρ_d=1.38g/cm^3)

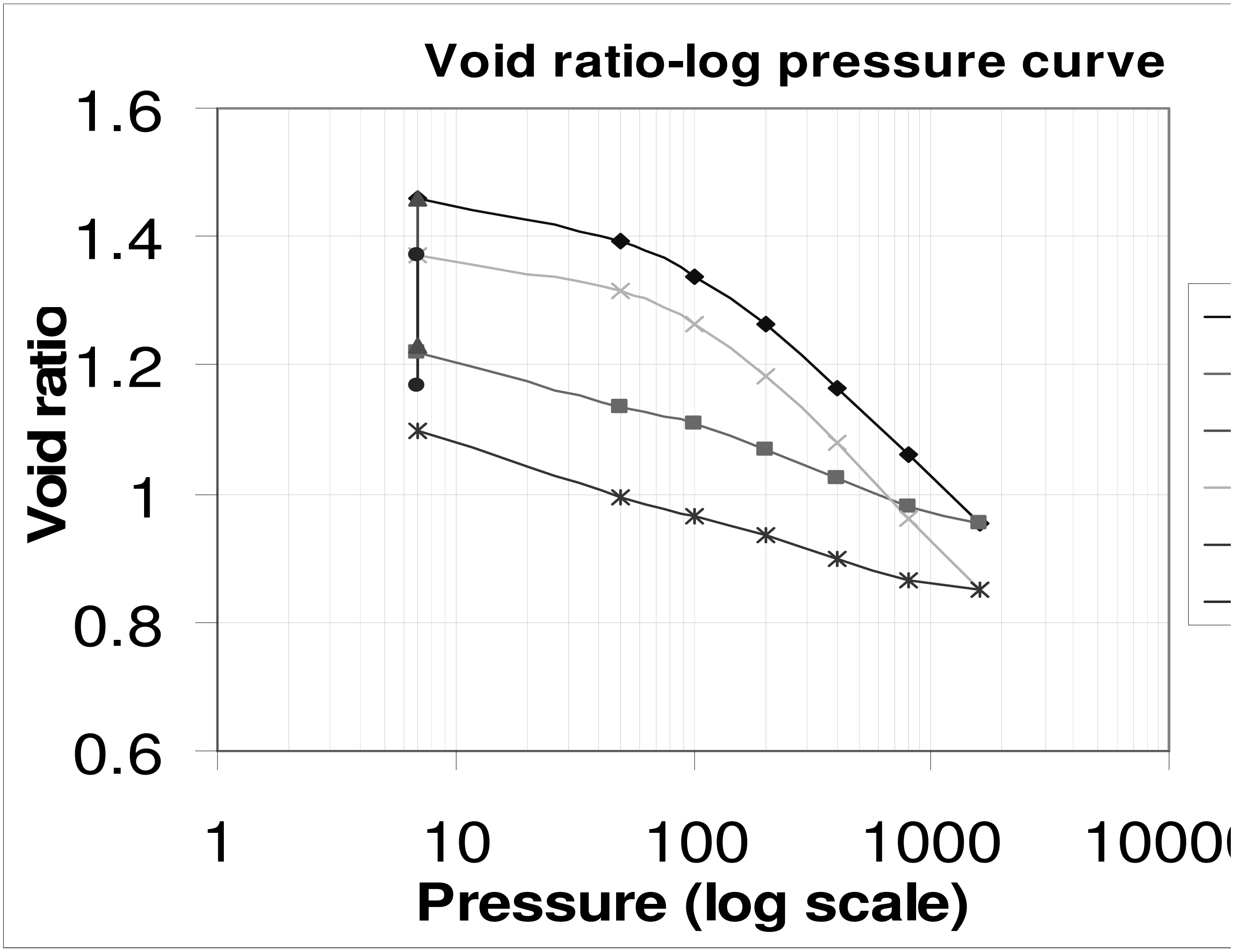
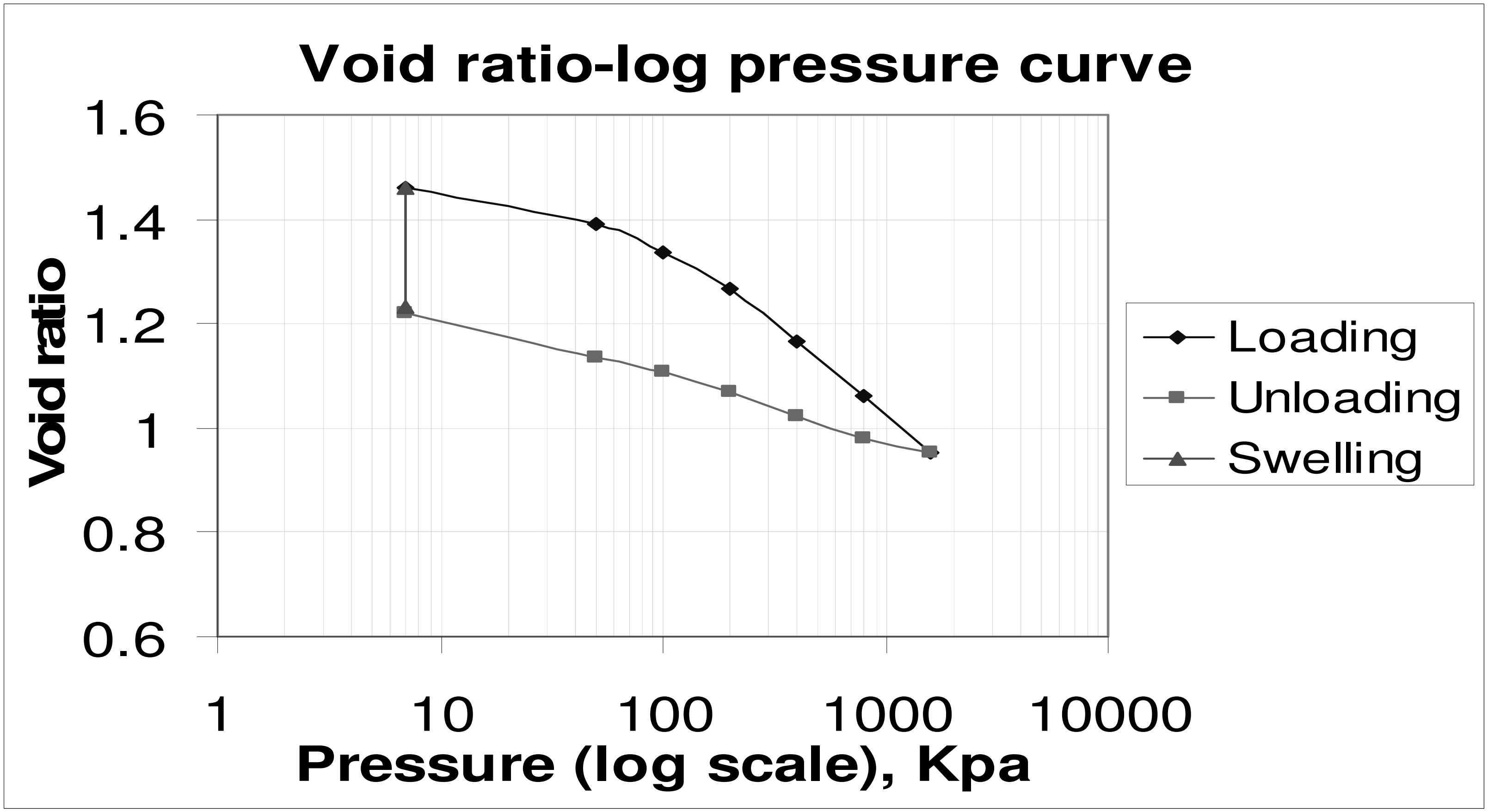


Fig A.9, Plot of Void Ratio Versus Log-Pressure for Sample-1 (Undisturbed, ω=37.75%,ρ_d=1.22g/cm^3) and
Sample-10 (Disturbed,ω=36.39%,ρ_d=1.25g/cm^3)

Appendix B

Consolidation-Swell test results of undisturbed samples taken from expansive soil of Addis Ababa



B.1 Plot of Void Ratio Versus Log-Pressure for Sample-1

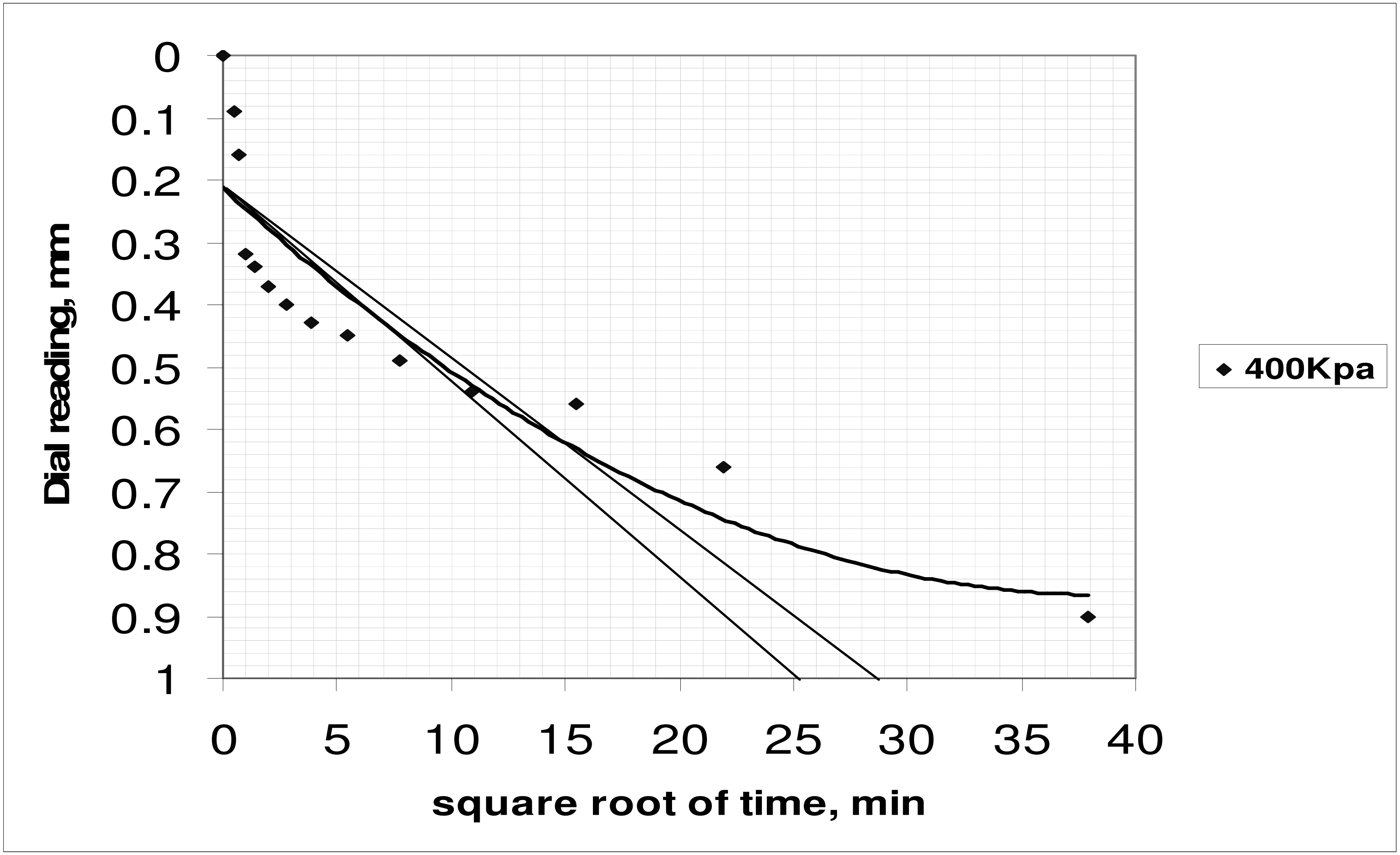


Fig B.1a Typical plots of Dial reading versus Square root of time of sample-1 (P=400Kpa)

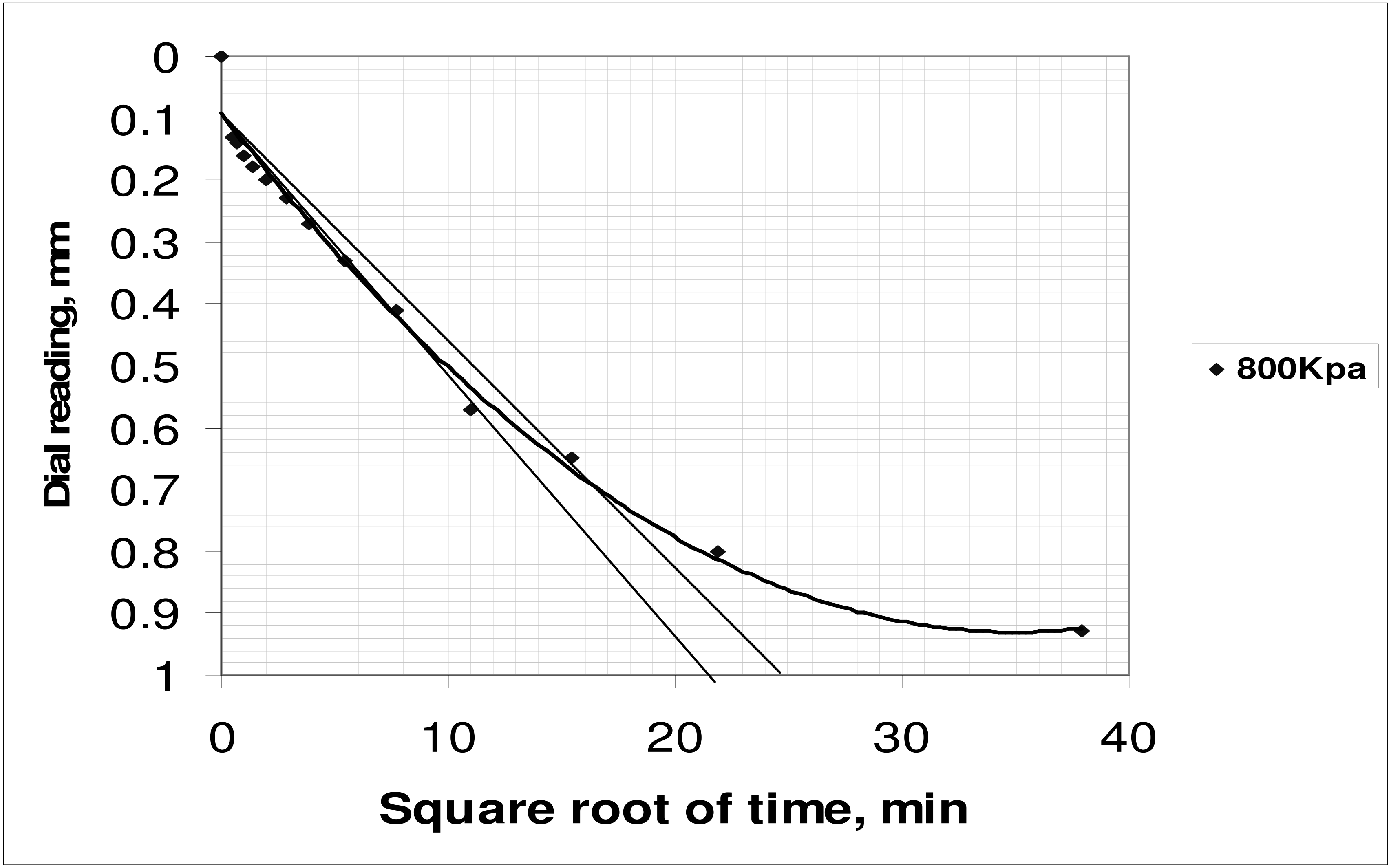


Fig B-1b, Typical plots of Dial reading versus Square root of time of sample-1 (P=800kpa)

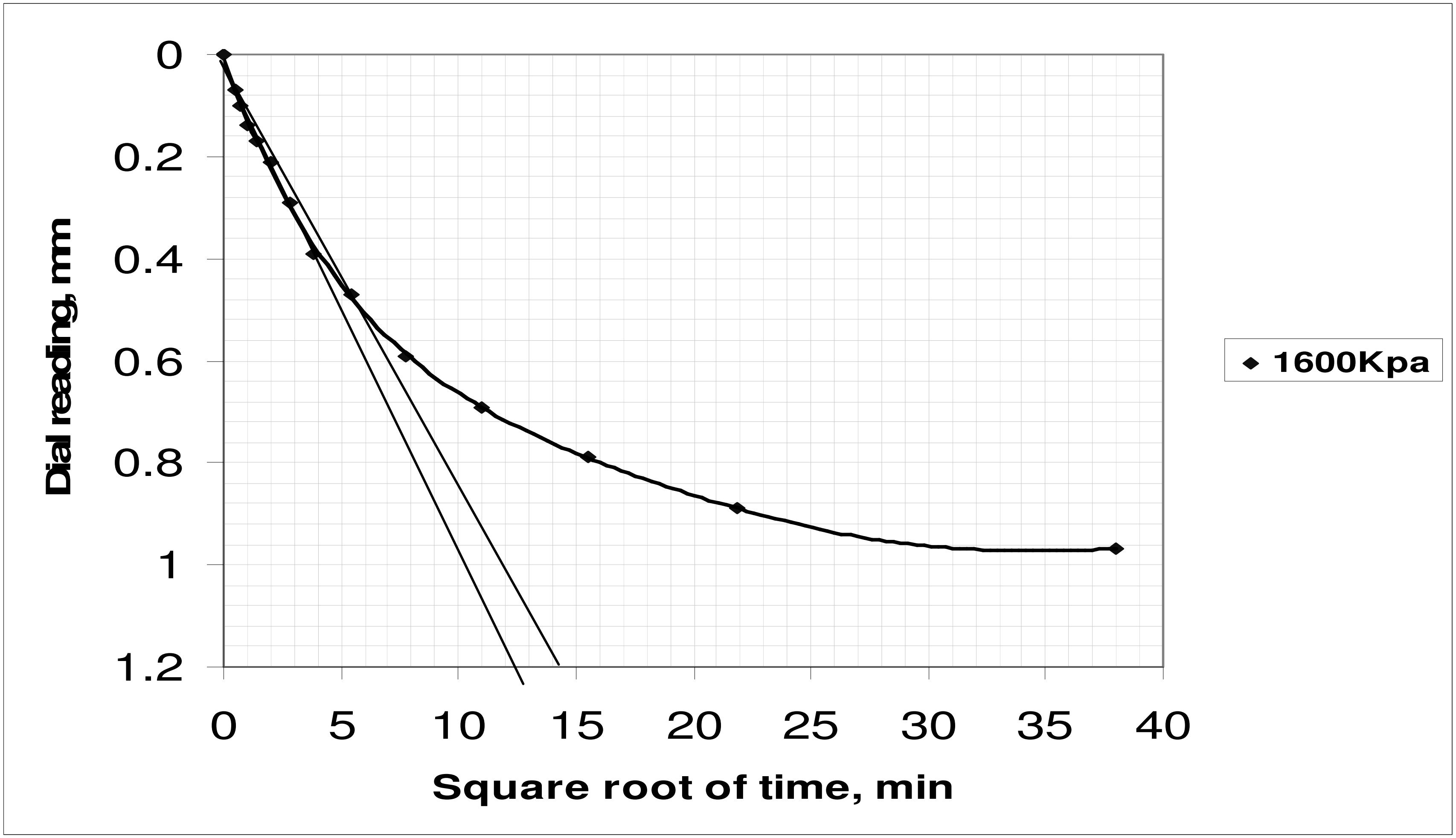
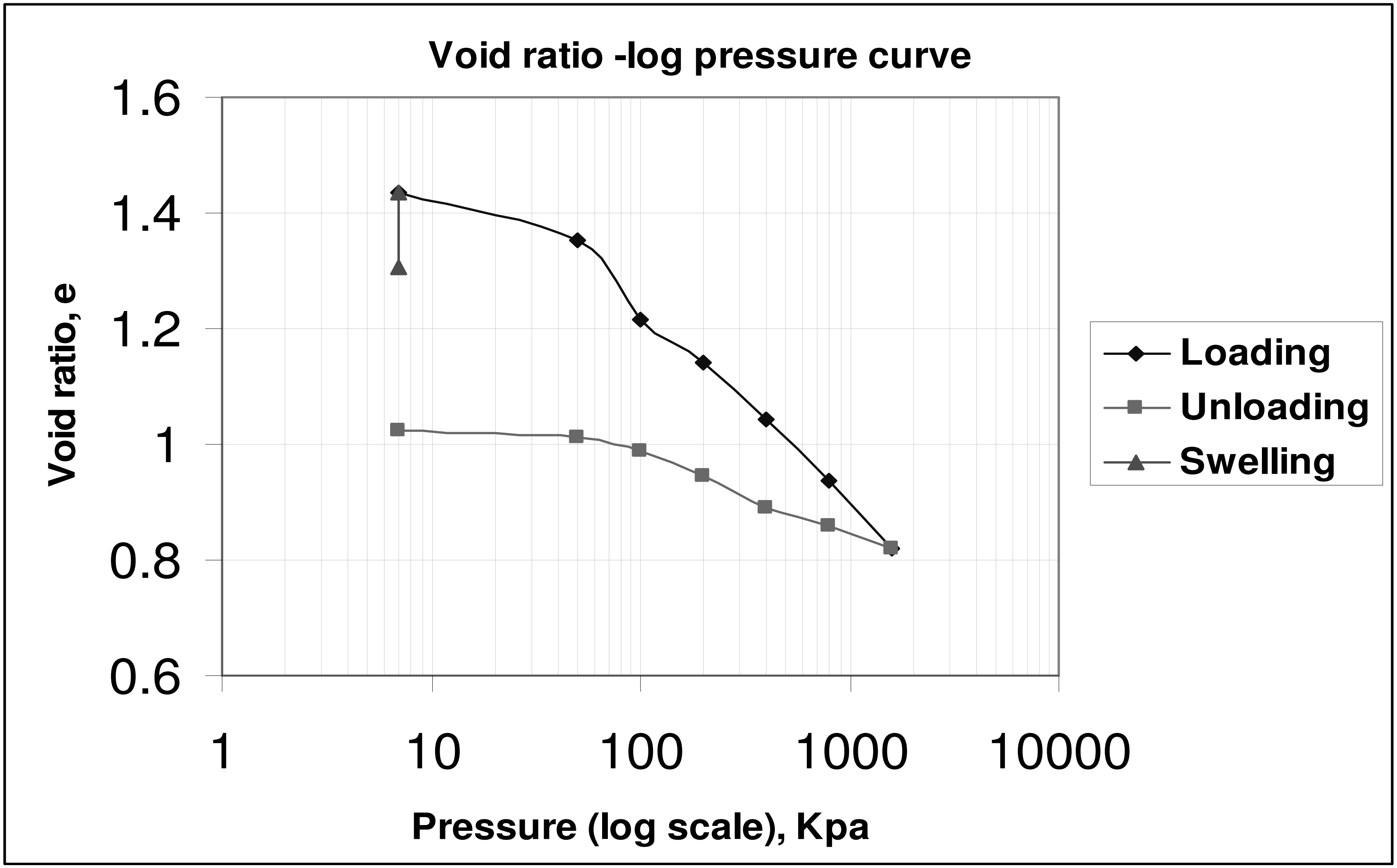


Fig B-1c Typical plots of Dial reading versus Square root of time of sample-1 (P=1600)



B.2, Plot of Void Ratio Versus Log-Pressure for Sample-2

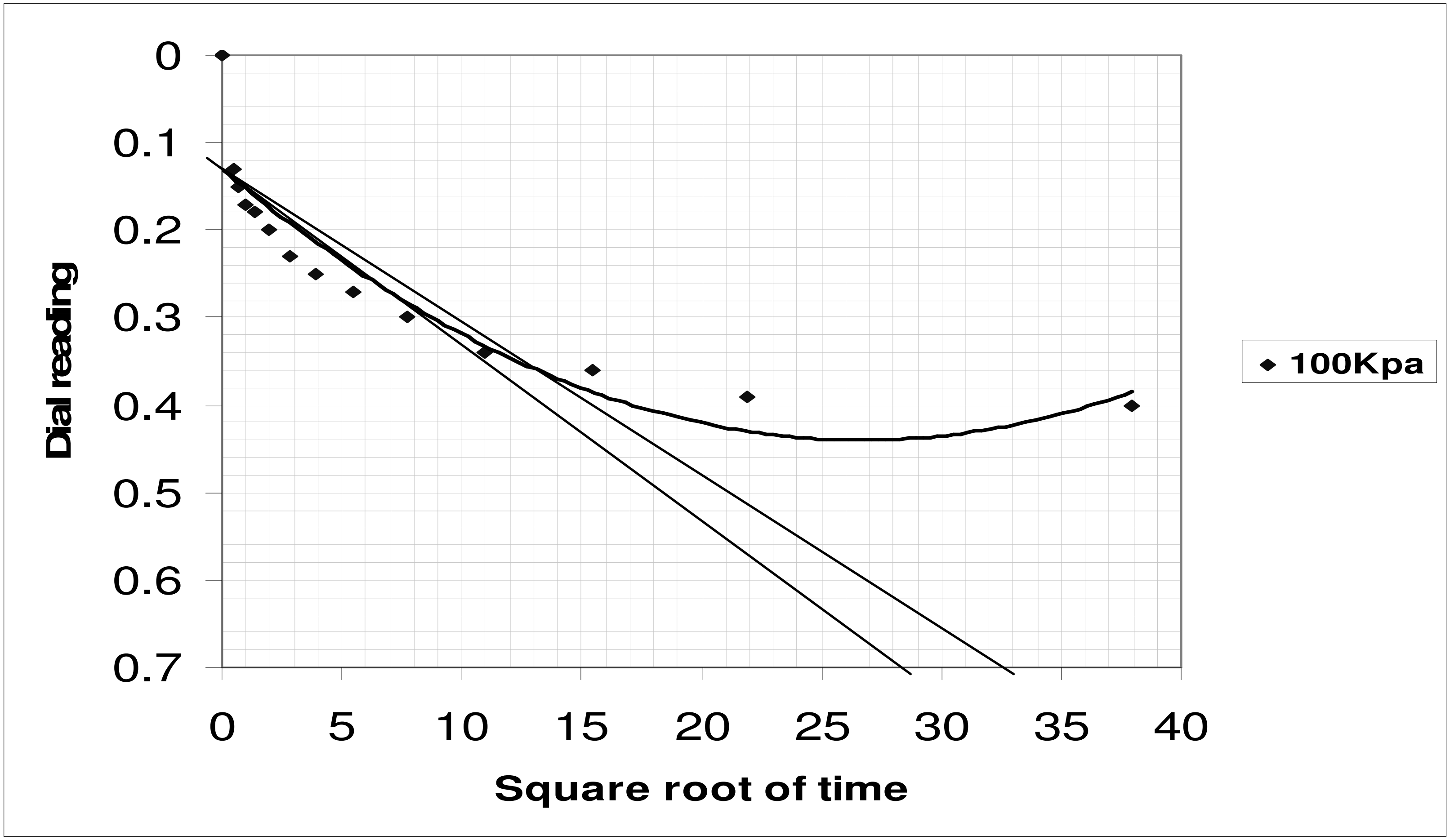


Fig B-2a, Typical plots of Dial reading versus Square root of time of sample-2 (P=100Kpa)

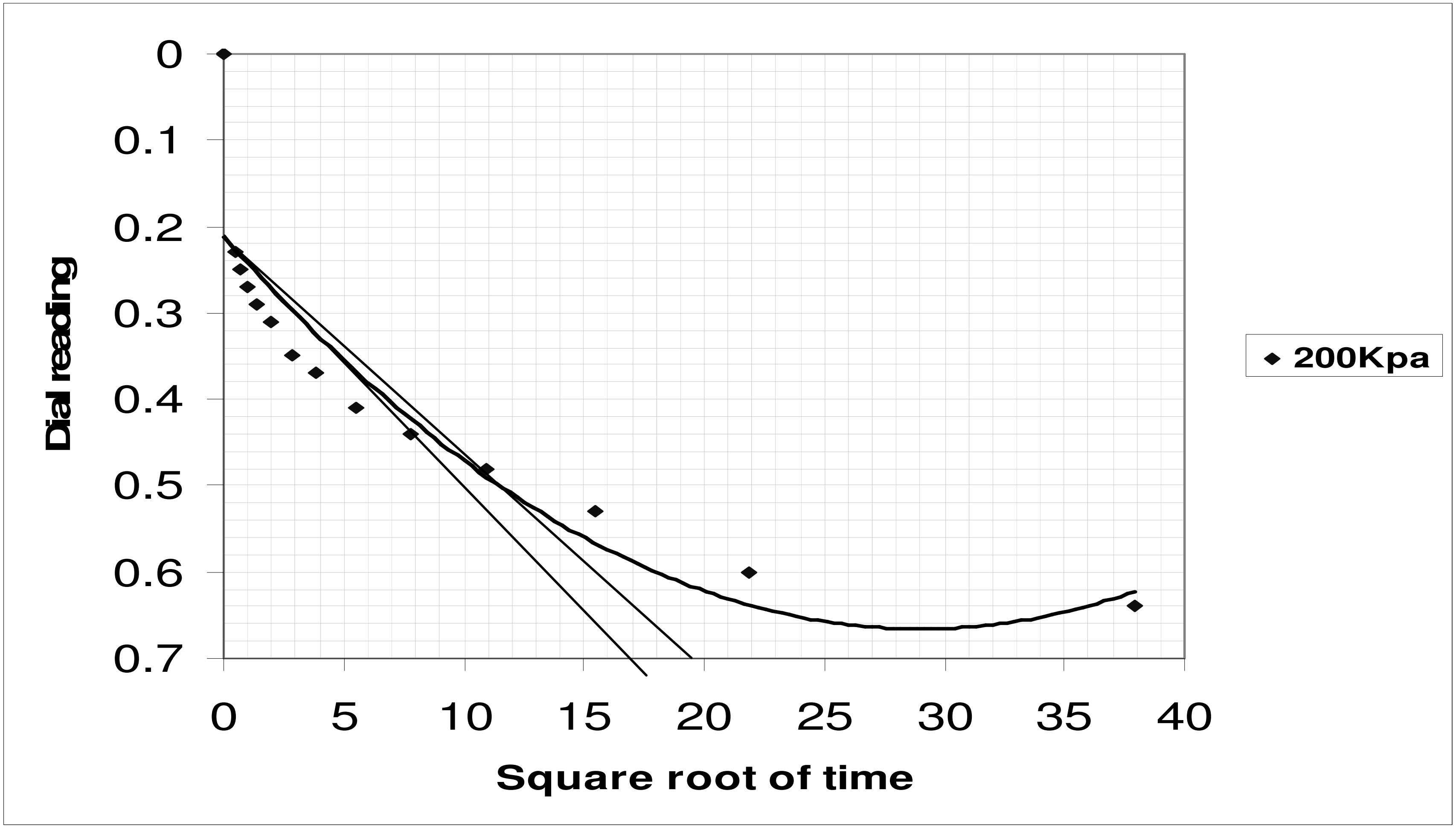


Fig B-2b, Typical plots of Dial reading versus Square root of time of sample-2 (P=200Kpa)

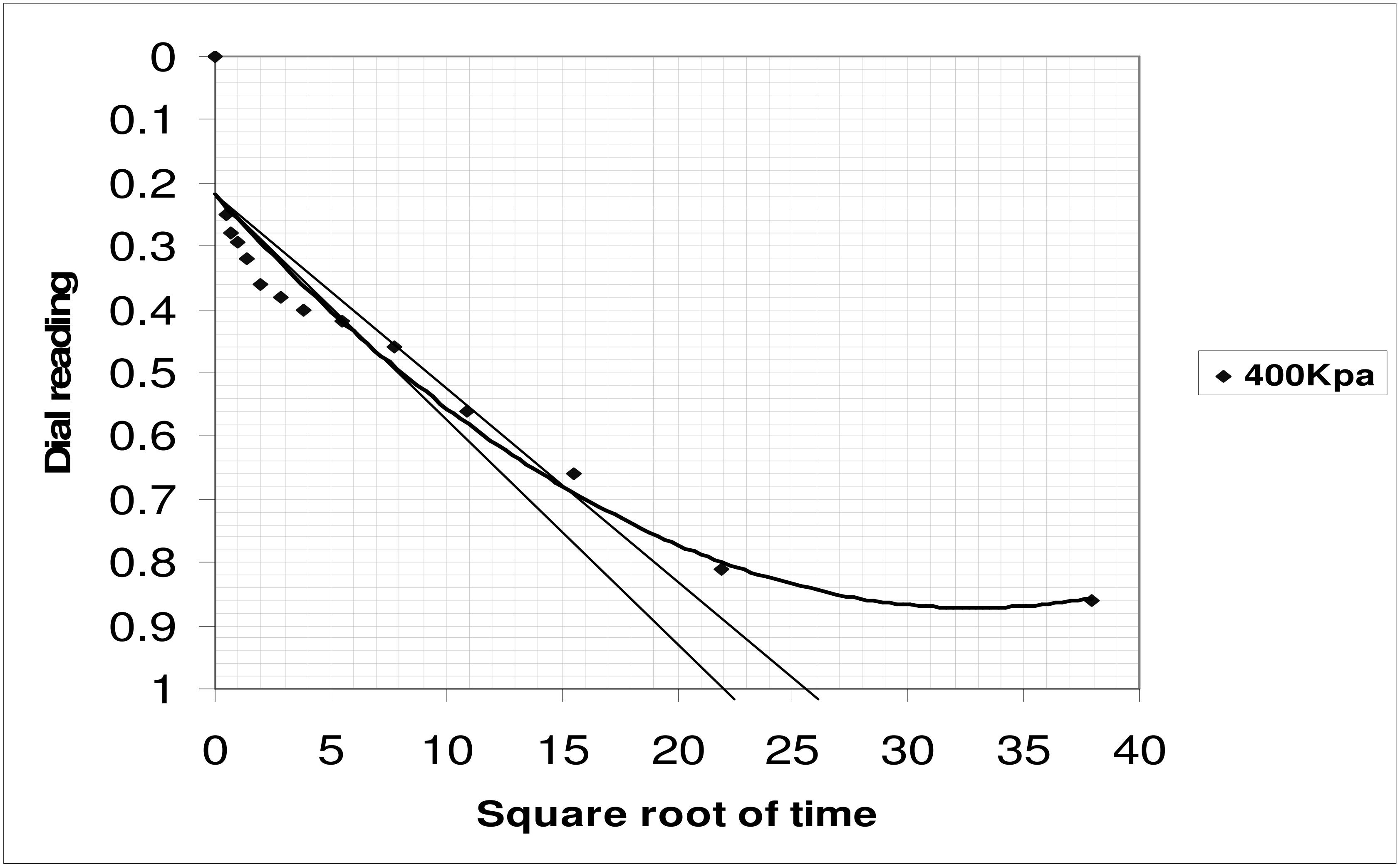


Fig B-2c, Typical plots of Dial reading versus Square root of time of sample-2 (P=400Kpa)

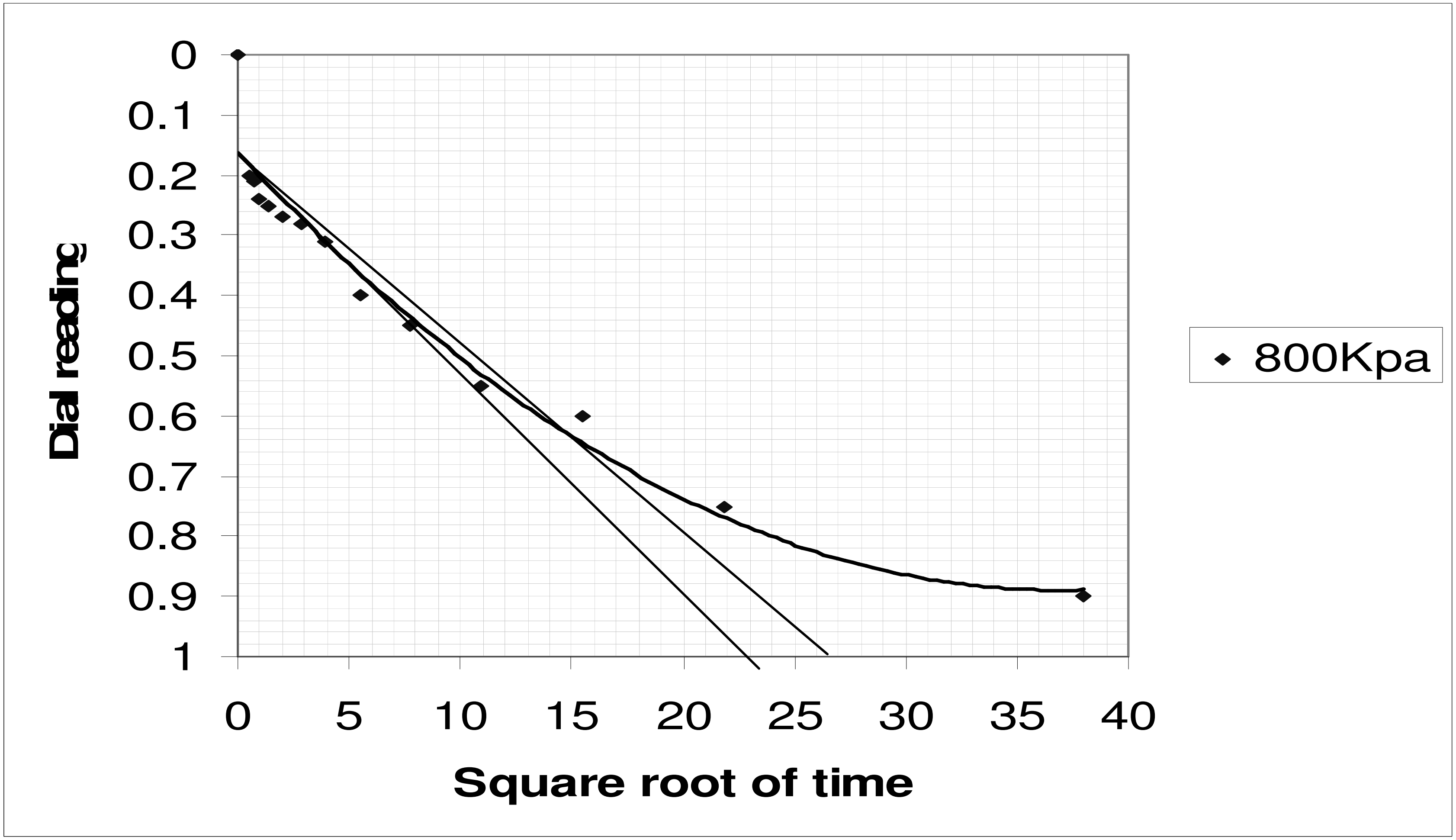


Fig B-2d, Typical plots of Dial reading versus Square root of time of sample-2 (P=800Kpa)

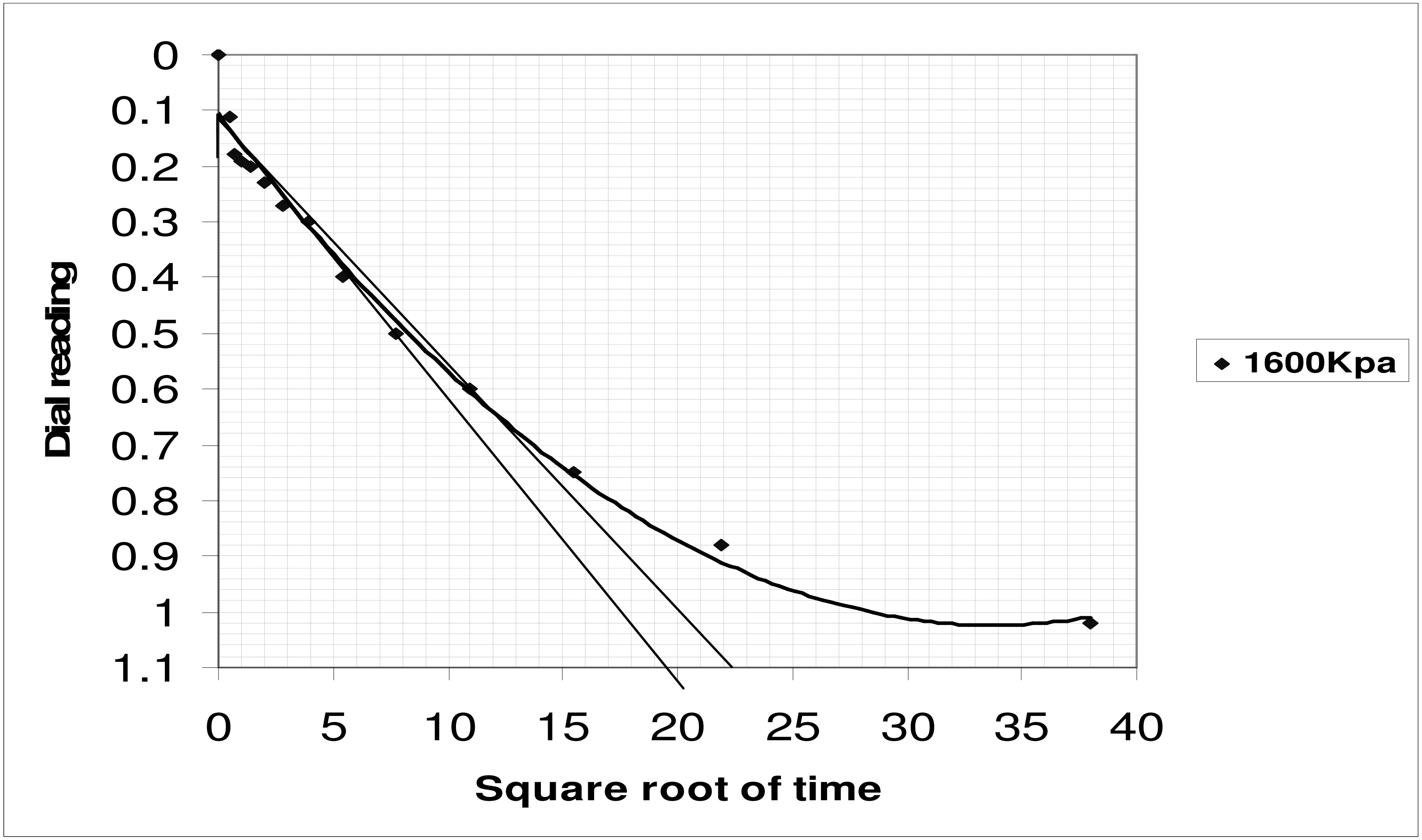


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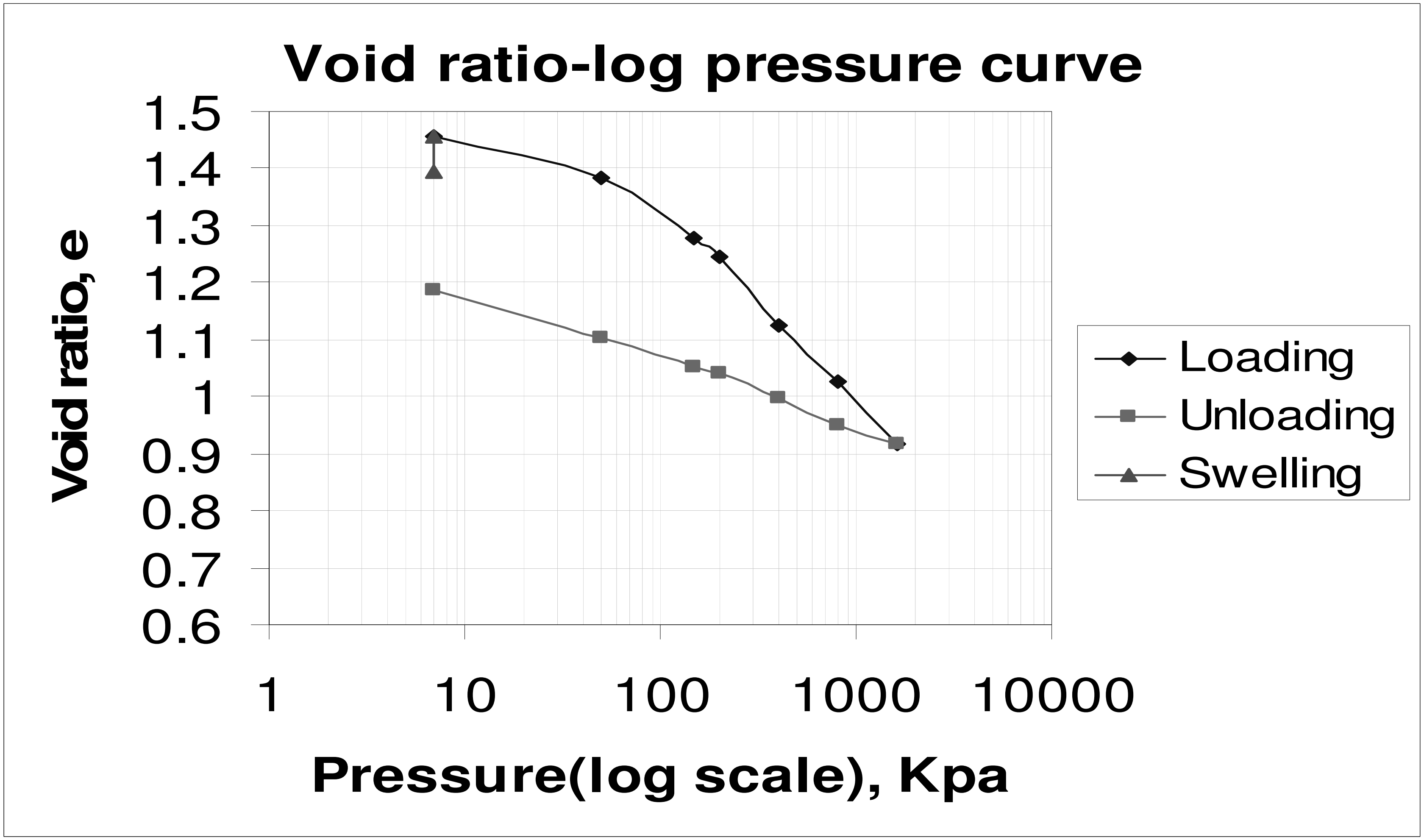
Fig B-2e, Typical plots of Dial reading versus Square root of time of sample-2 (p=1600Kpa)



B.3, Plot of Void Ratio Versus Log-Pressure for Sample-4

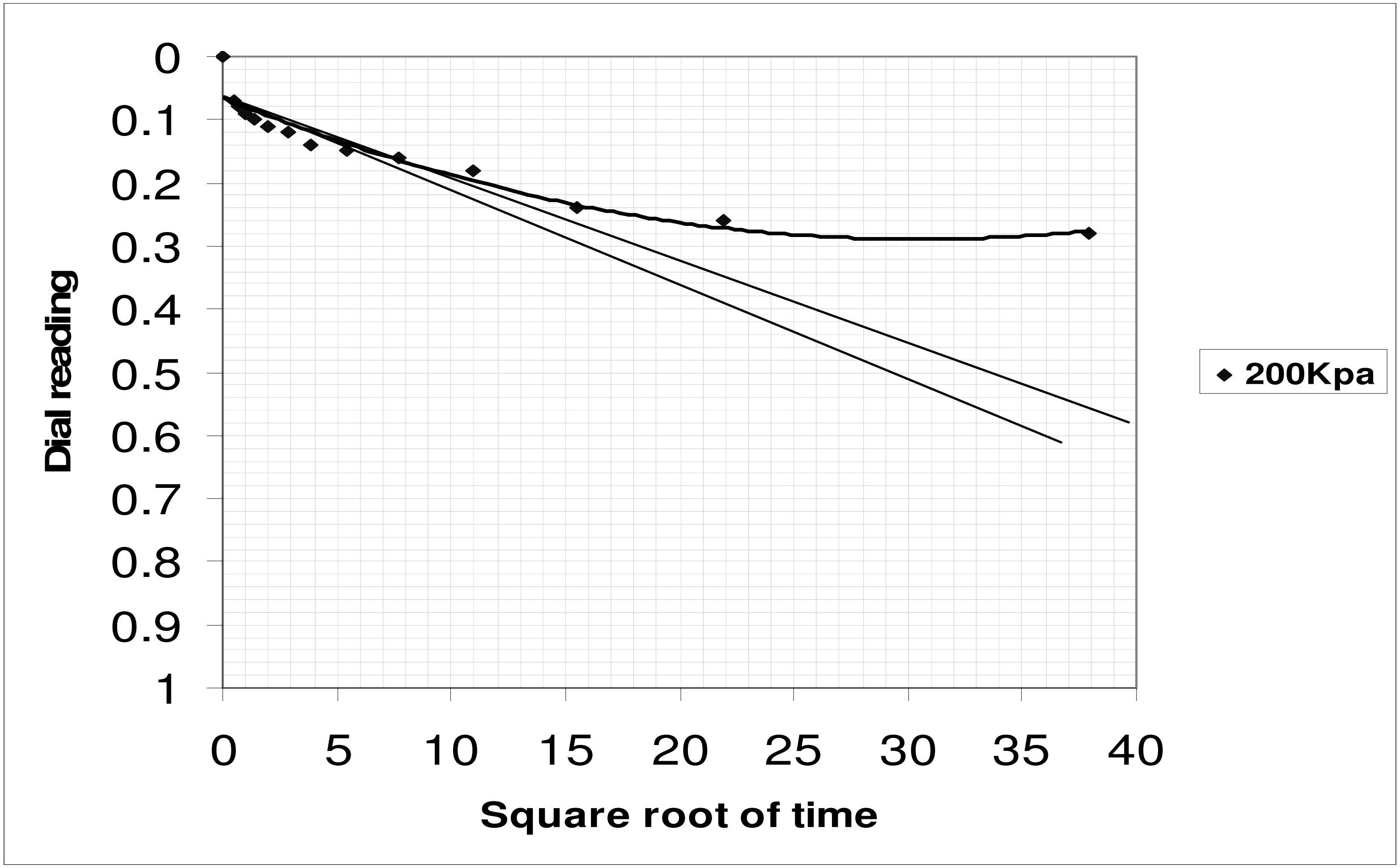


Fig B-3a, Typical plots of Dial reading versus Square root of time of sample-4 (p=200Kpa)

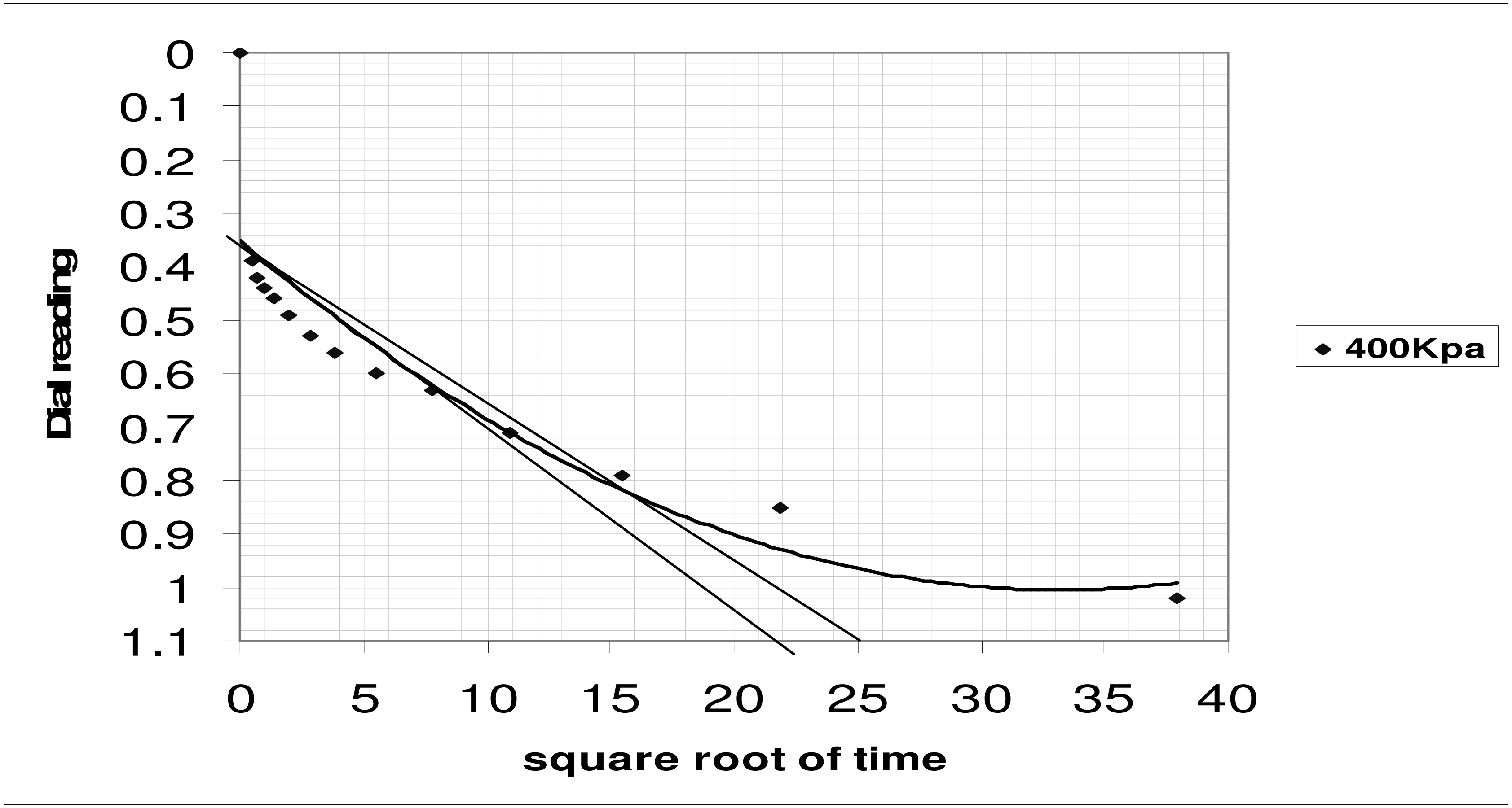


Fig B-3b Typical plots of Dial reading versus Square root of time of sample-4 9P=400Kpa)

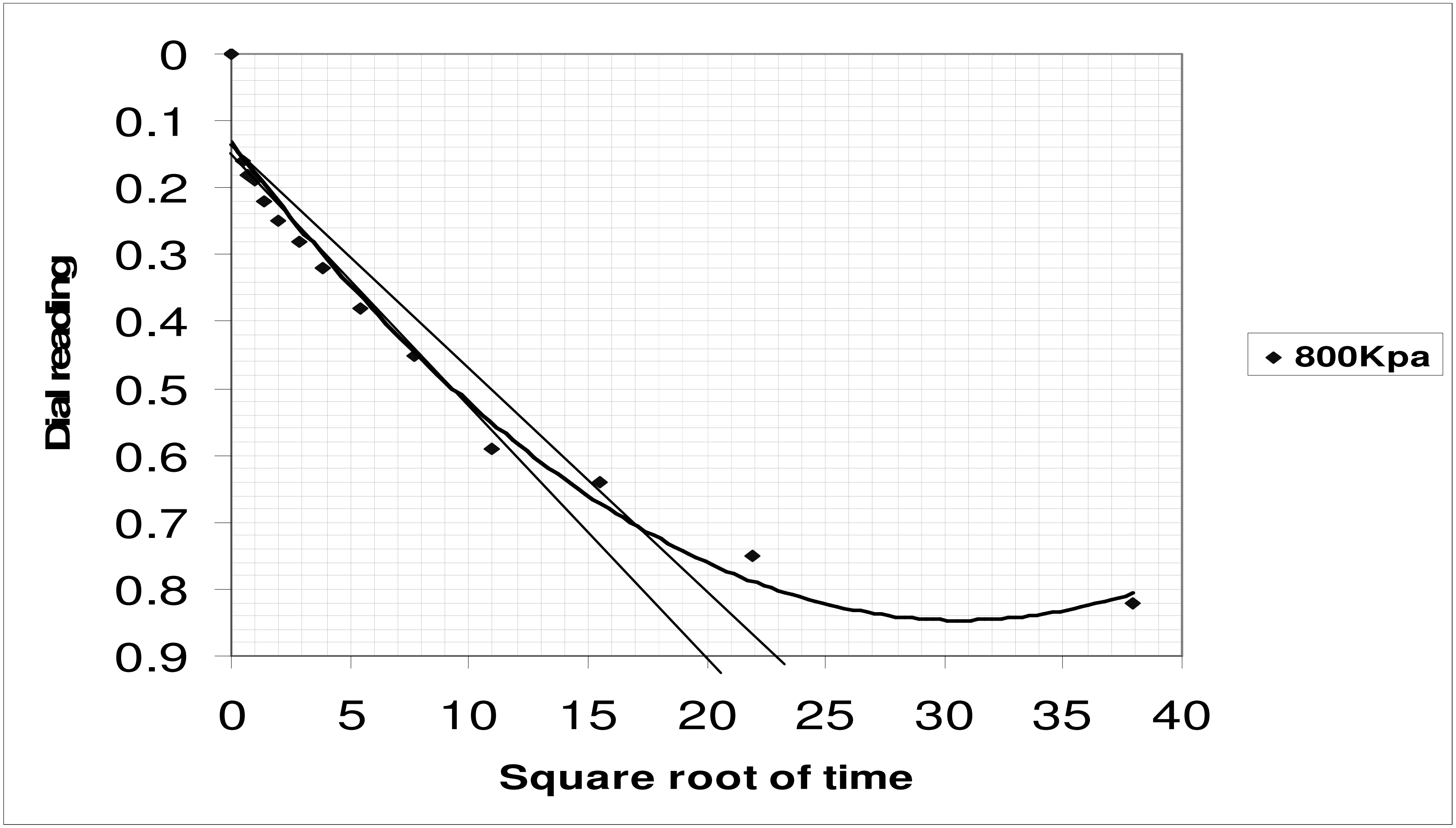


Fig B-3c, Typical plots of Dial reading versus Square root of time of sample-4 (P= 800Kpa)

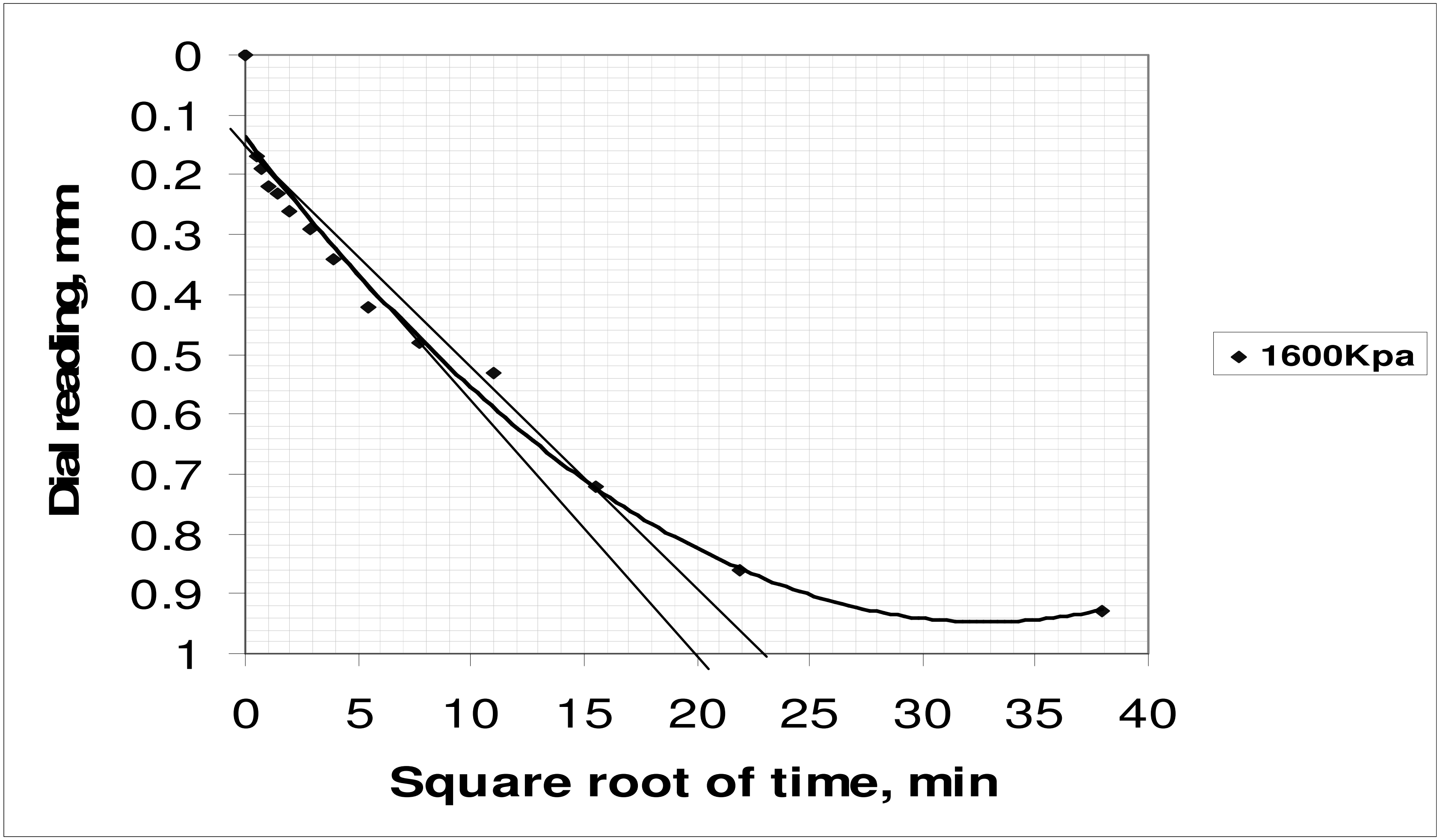
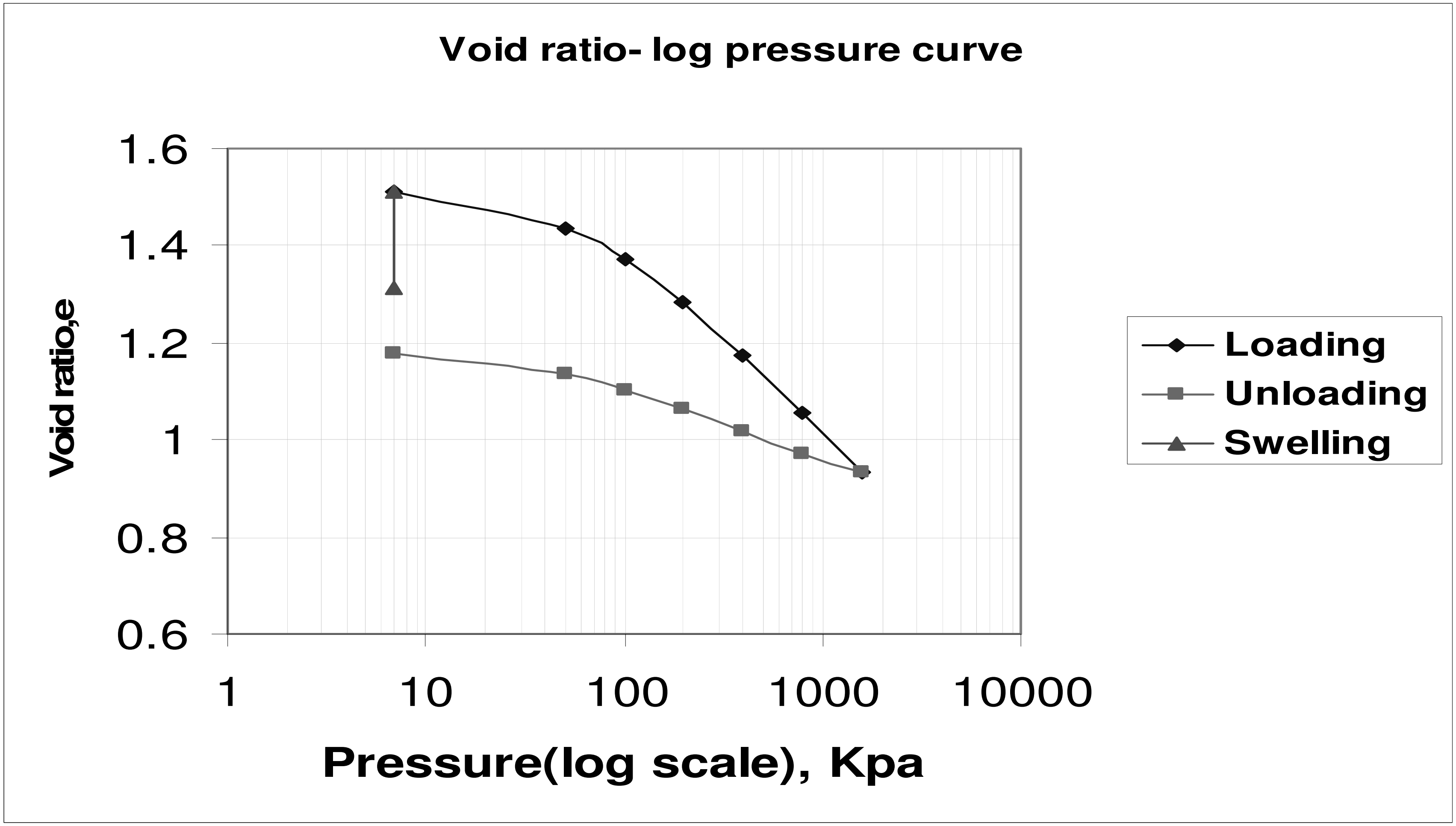


Fig B-3d, Typical plots of Dial reading versus Square root of time of sample-4 (P=1600Kpa)



B.4, Plot of Void Ratio Versus Log-Pressure for Sample-5

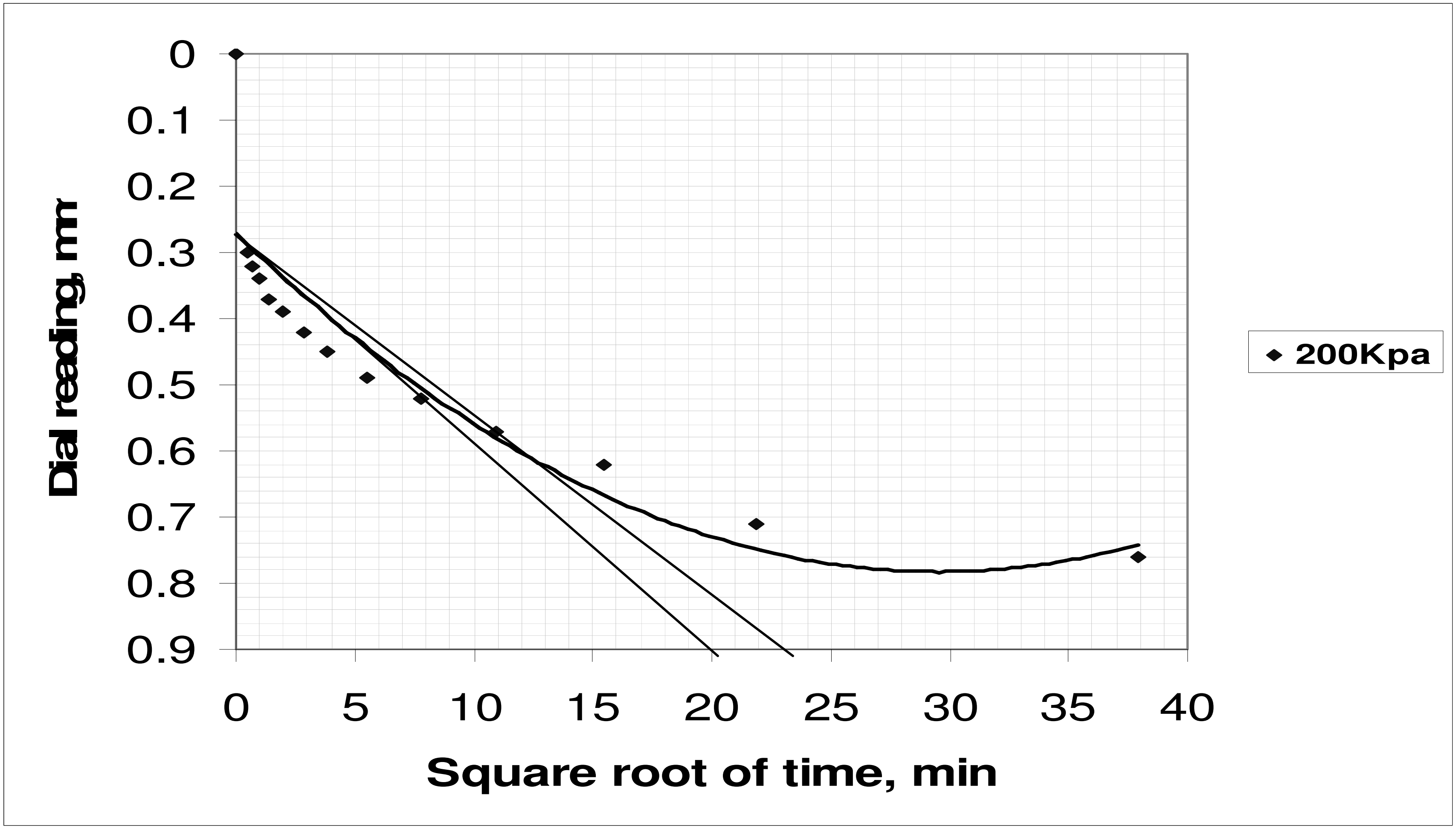


Fig B-4a, Typical plots of Dial reading versus Square root of time of sample-5 (P=200Kpa)

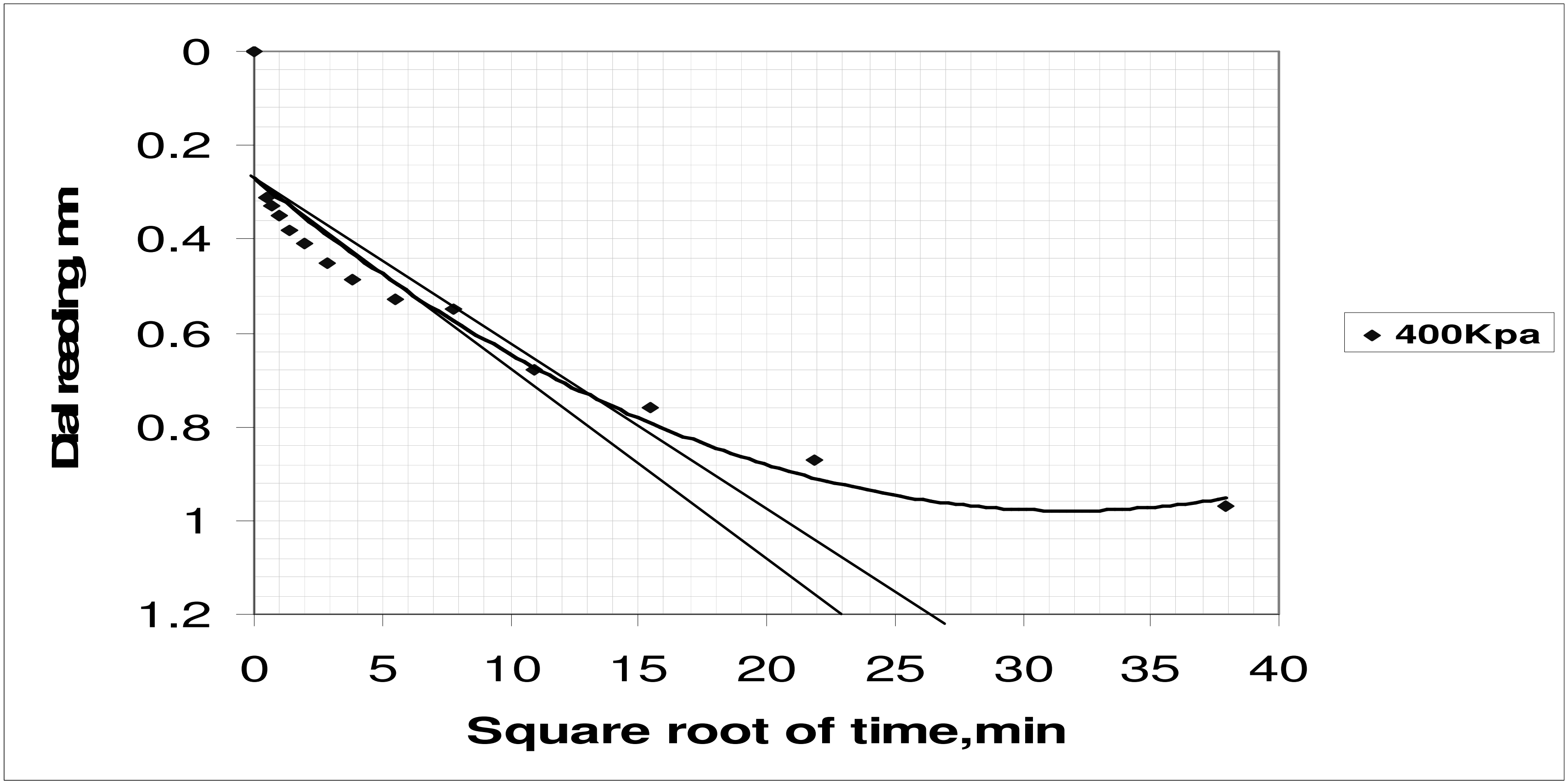


Fig B-4b, Typical plots of Dial reading versus Square root of time of sample-5 (P=400Kpa)

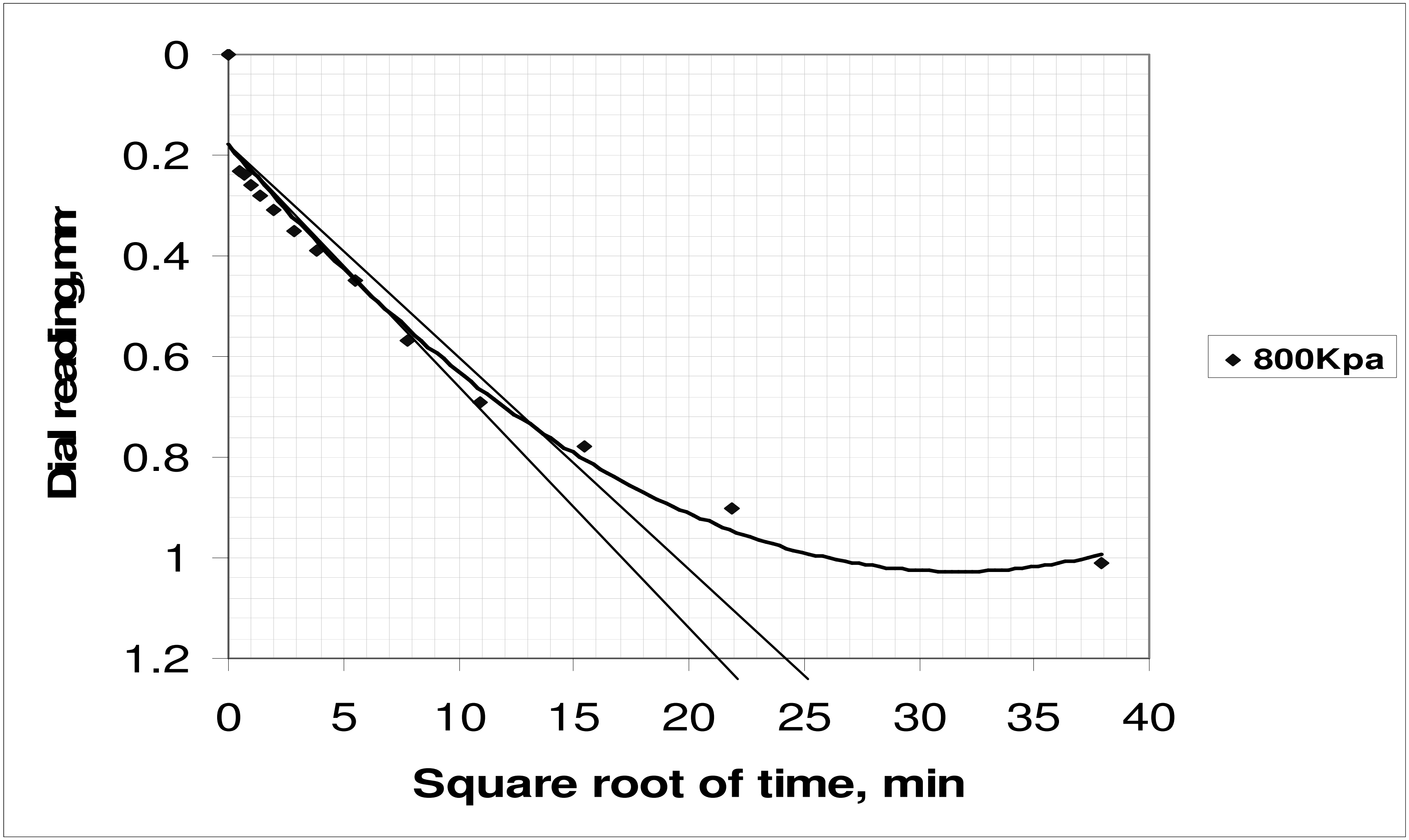


Fig B-4c, Typical plots of Dial reading versus Square root of time of sample-5 (P=800Kpa)

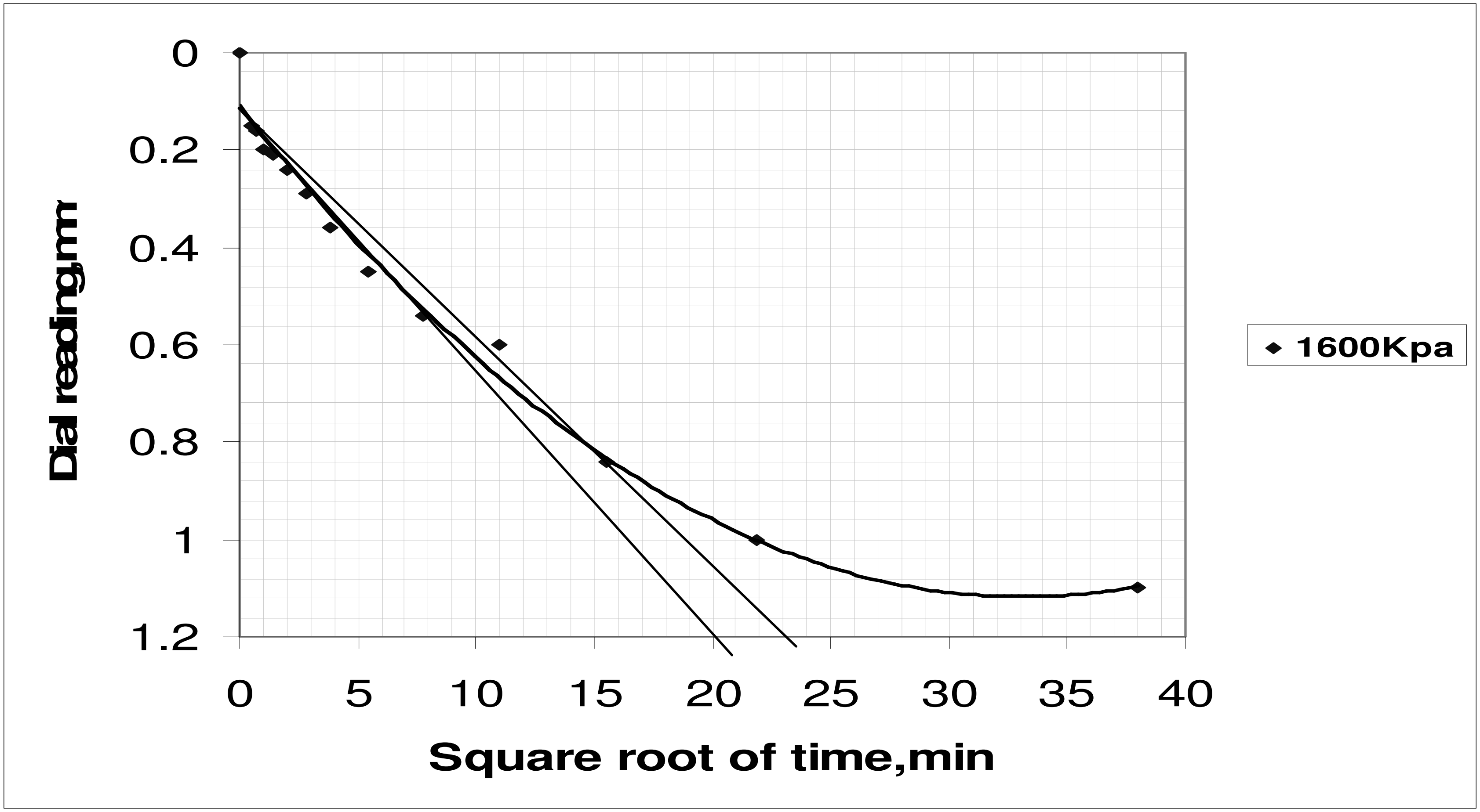
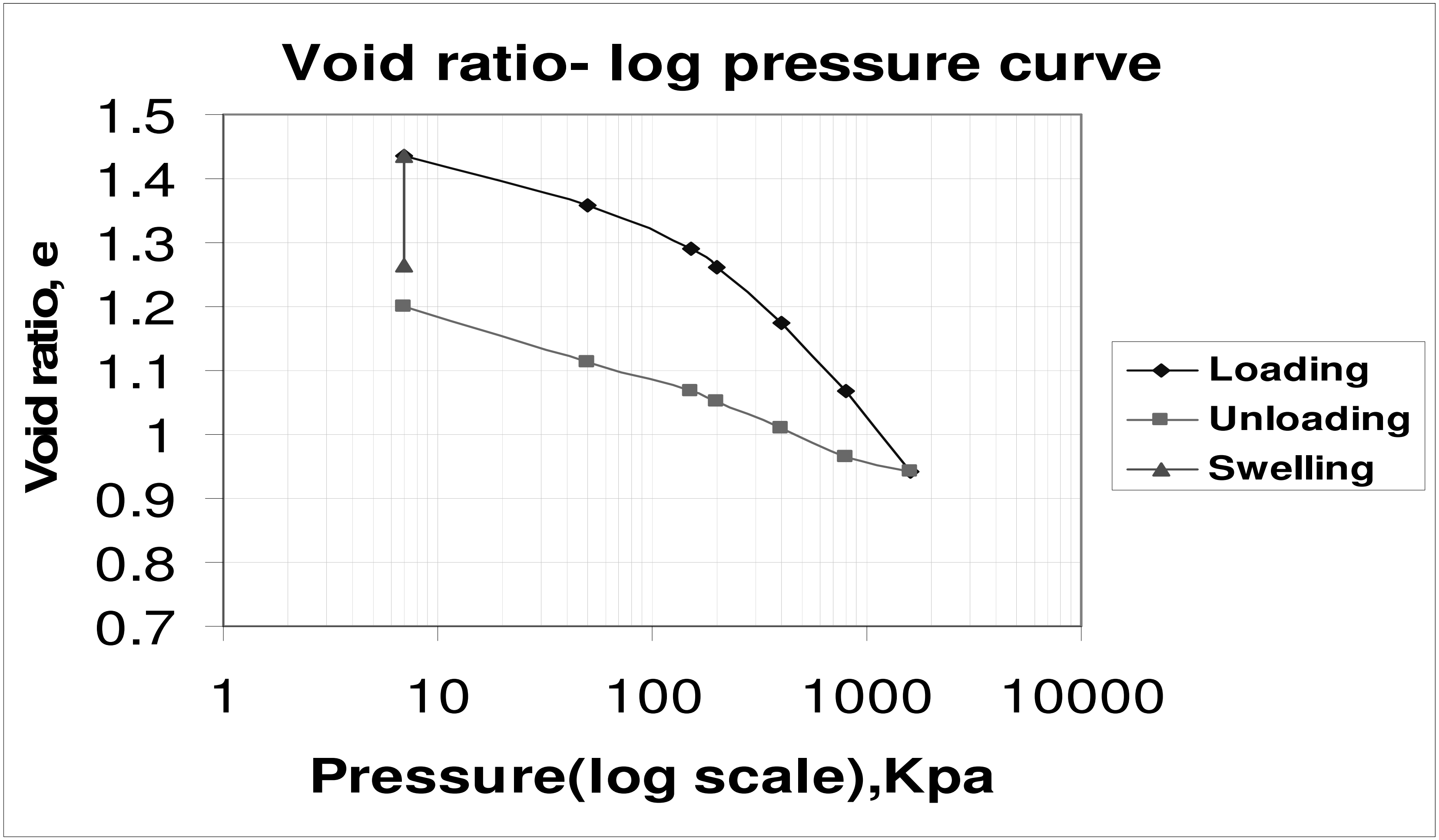


Fig B-4d, Typical plots of Dial reading versus Square root of time of sample-5(P=1600Kpa)



B.5 Plot of Void Ratio Versus Log-Pressure for Sample-6

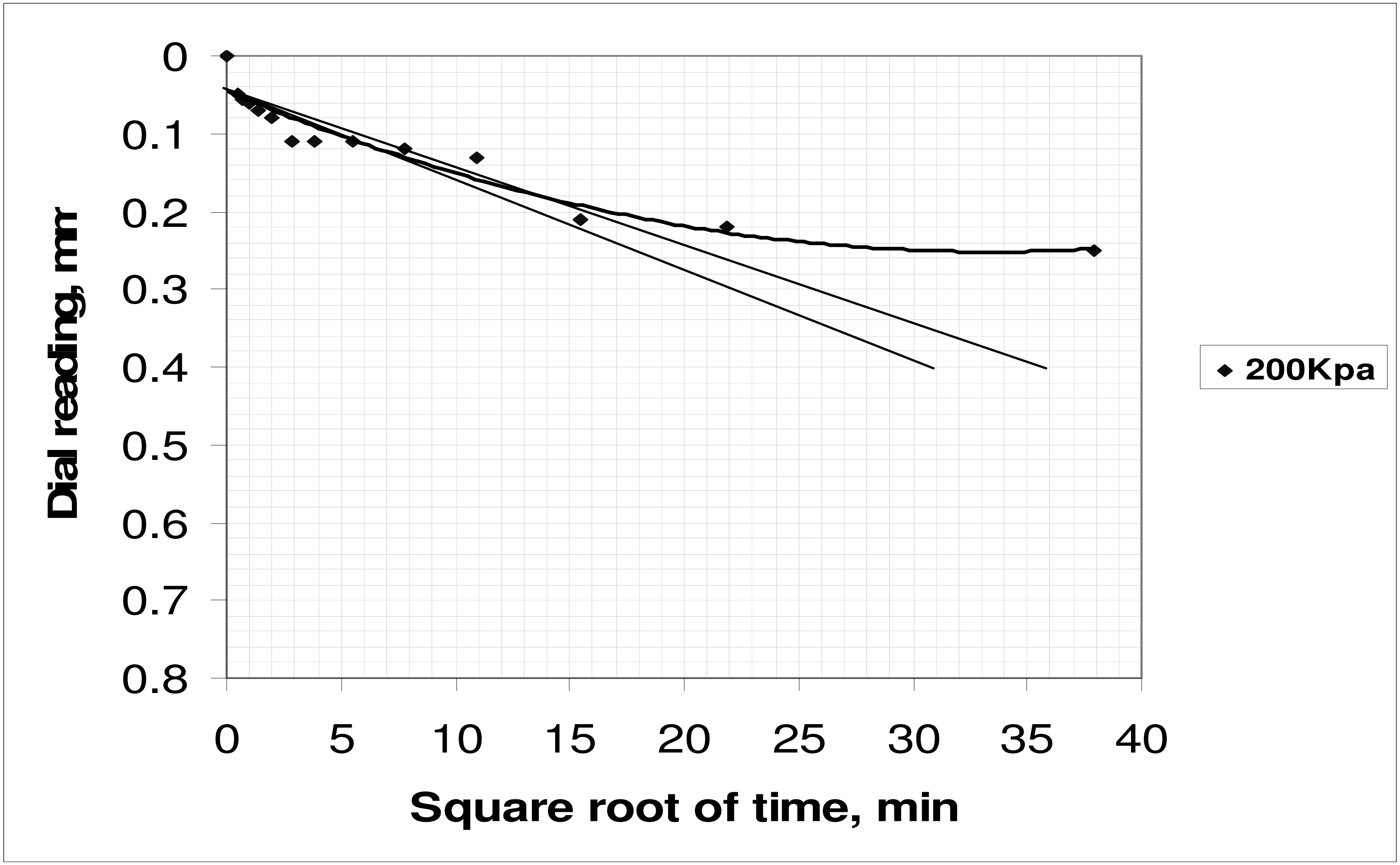


Fig B-5a, Typical plots of Dial reading versus Square root of time of sample-6 (p=200Kpa)

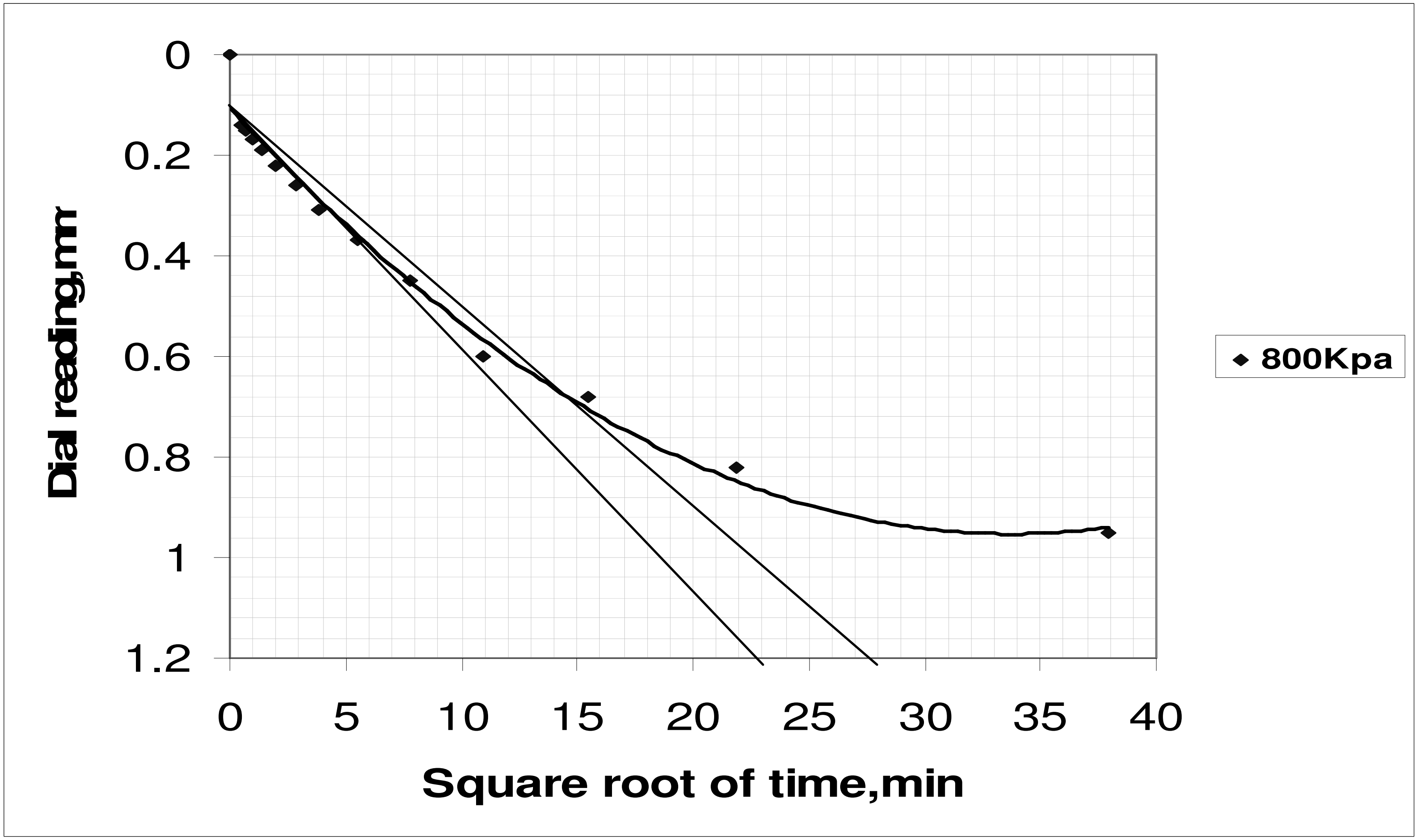


Fig B-5b, Typical plots of Dial reading versus Square root of time of sample-6 (P=800Kpa)

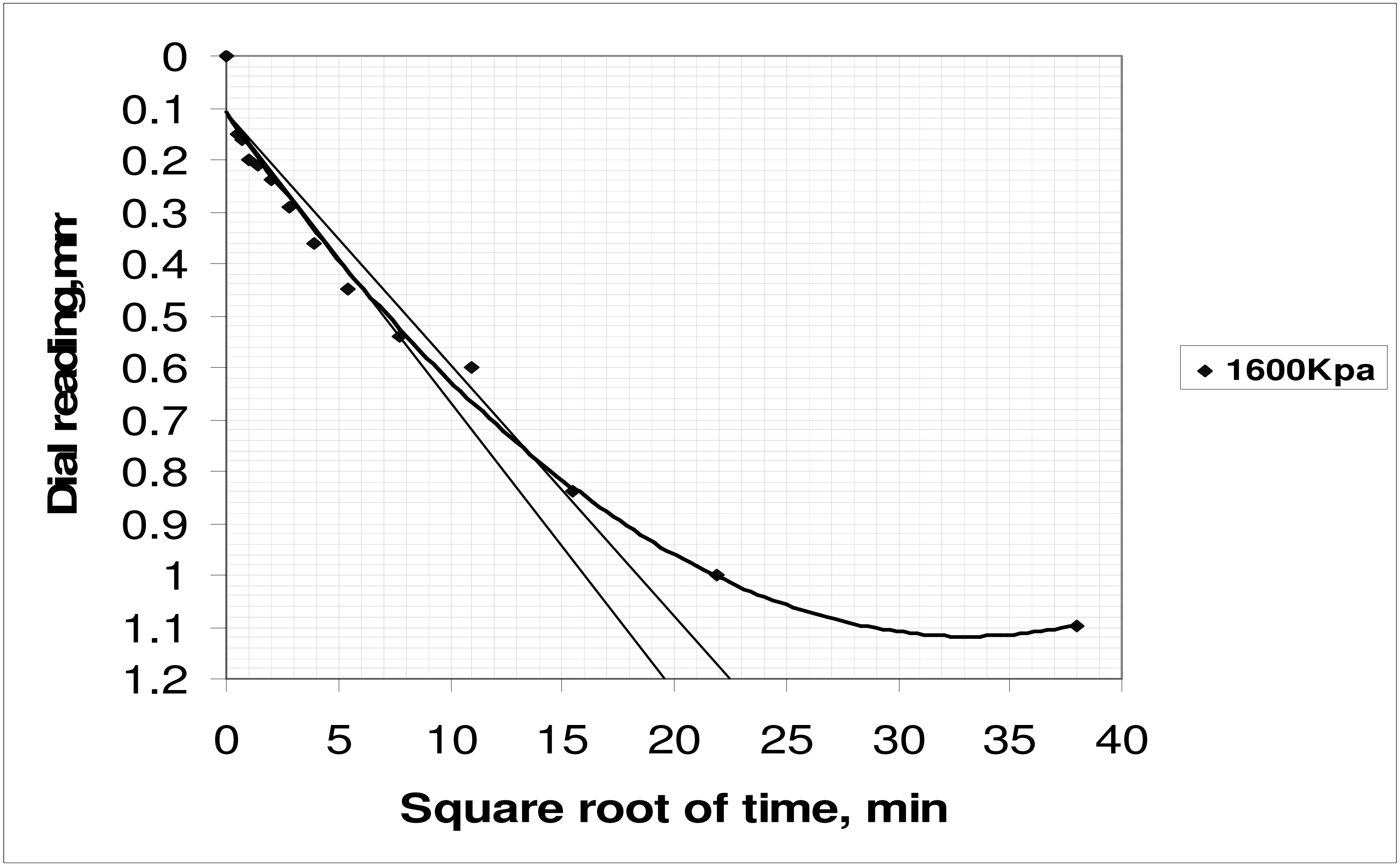
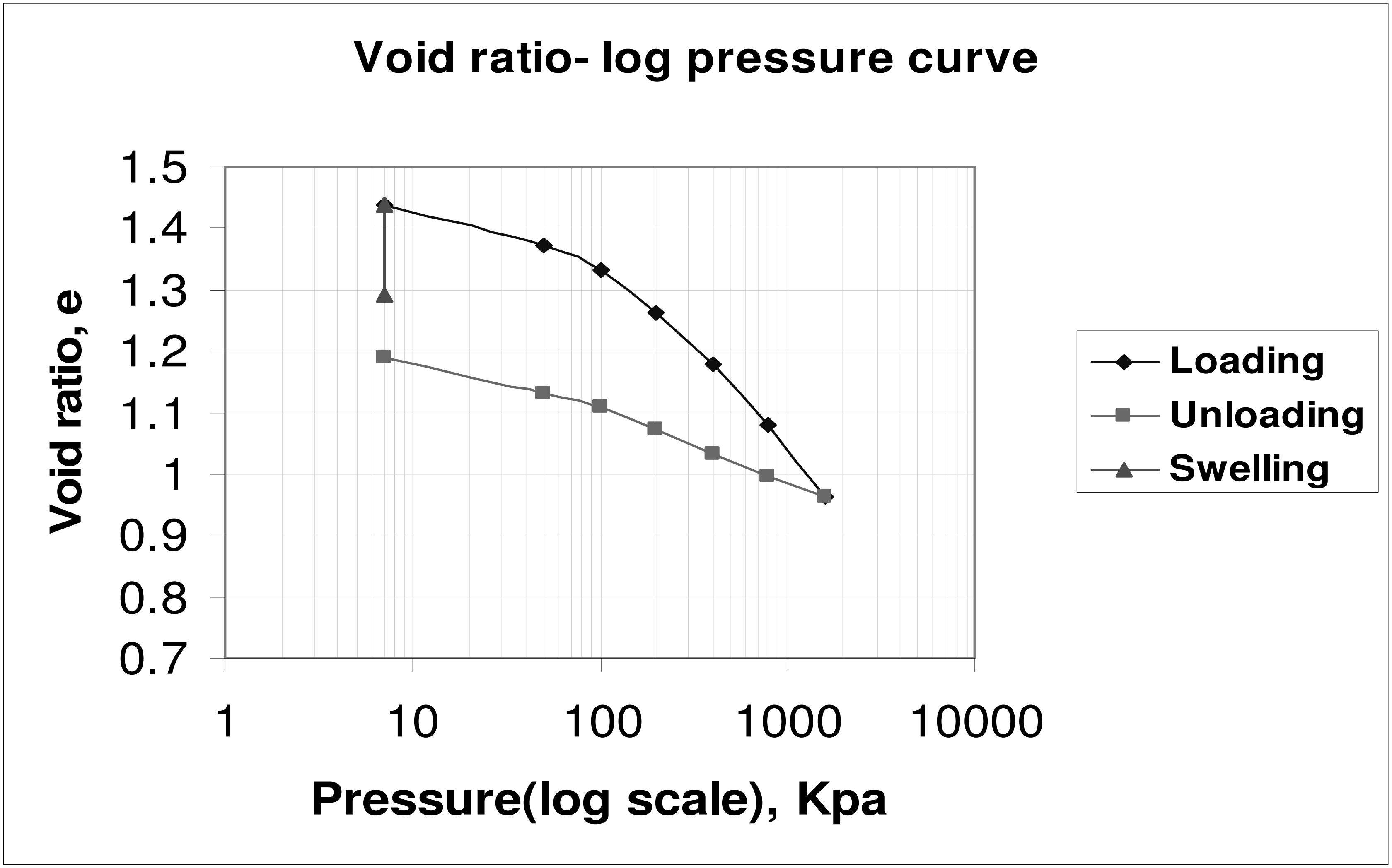


Fig B-5c, Typical plots of Dial reading versus Square root of time of sample-6 (P=1600Kpa)



B.6, Plot of Void Ratio Versus Log-Pressure for Sample-7

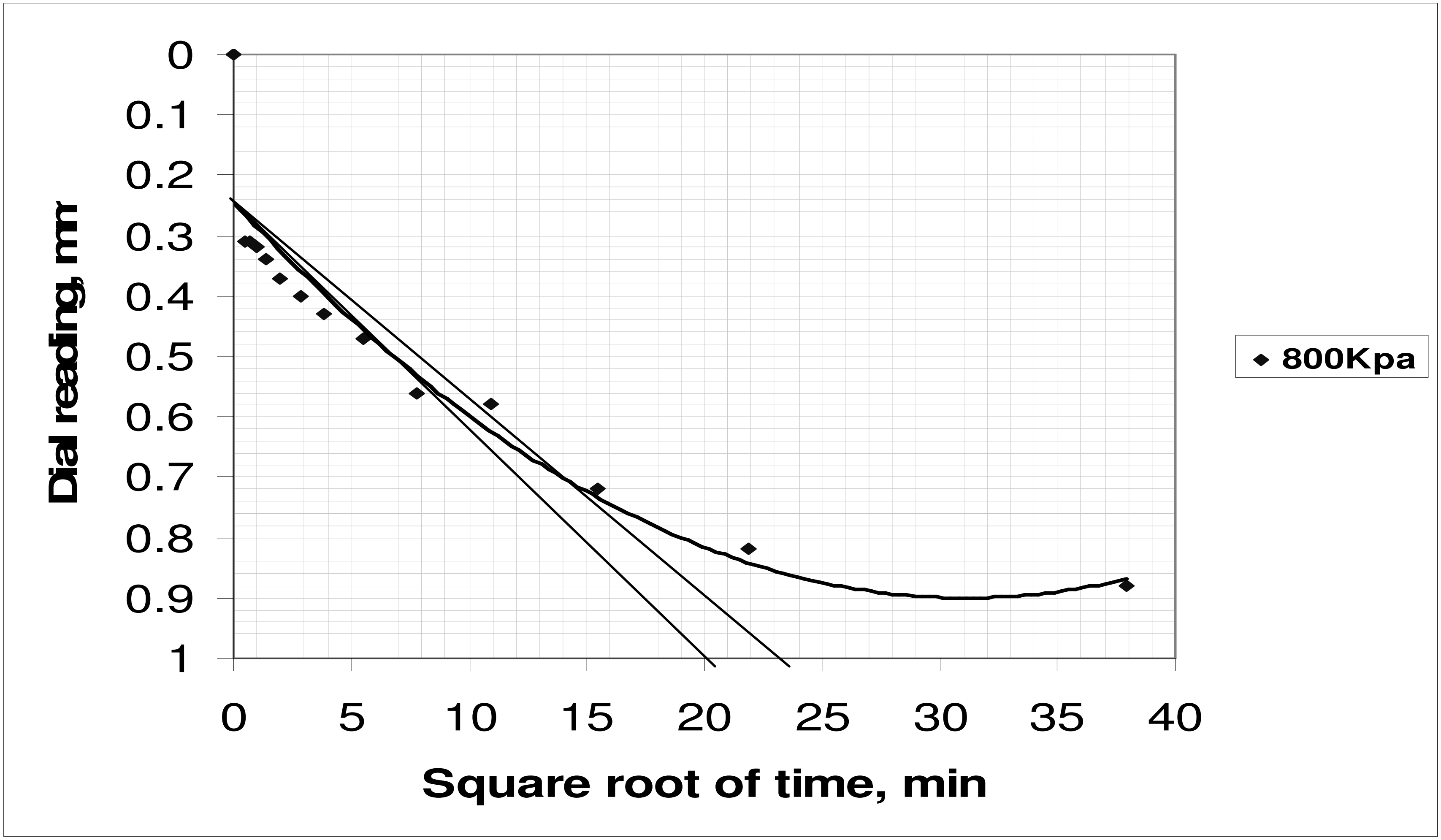


Fig B-6a, Typical plots of Dial reading versus Square root of time of sample-7(P=800Kpa)

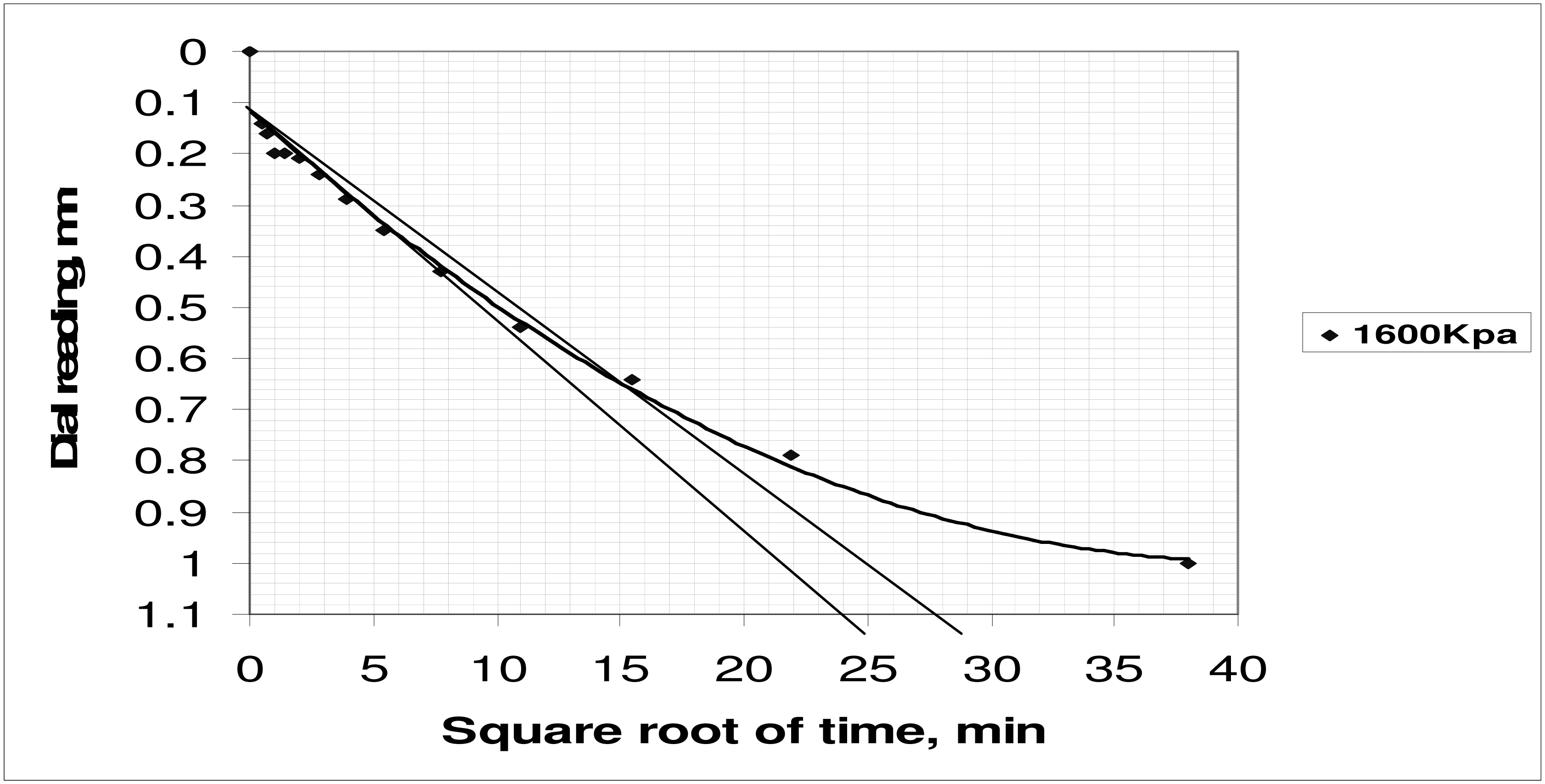
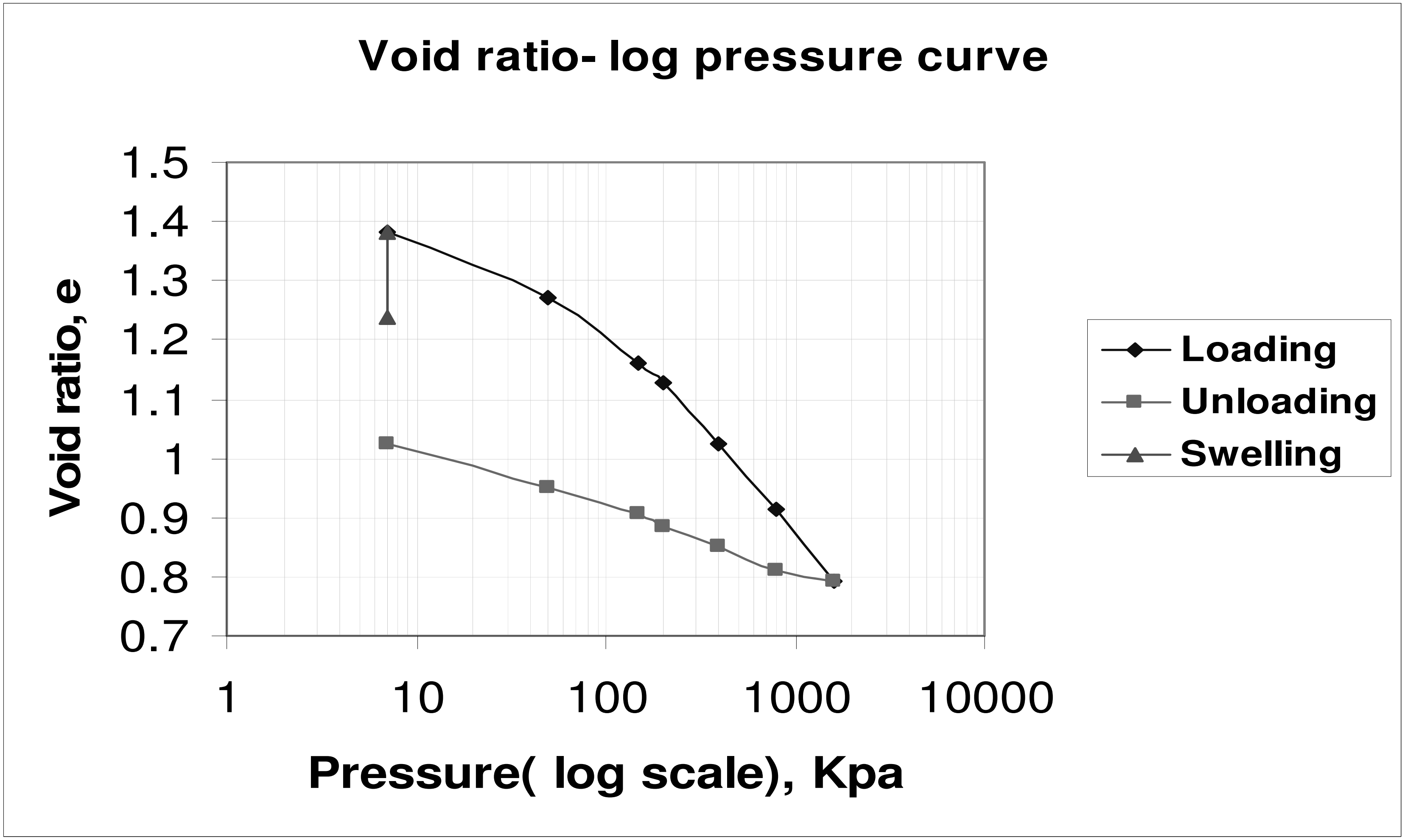


Fig B-6b, Typical plots of Dial reading versus Square root of time of sample-7 (p=1600Kpa)



B.7, Plot of Void Ratio Versus Log-Pressure for Sample-8

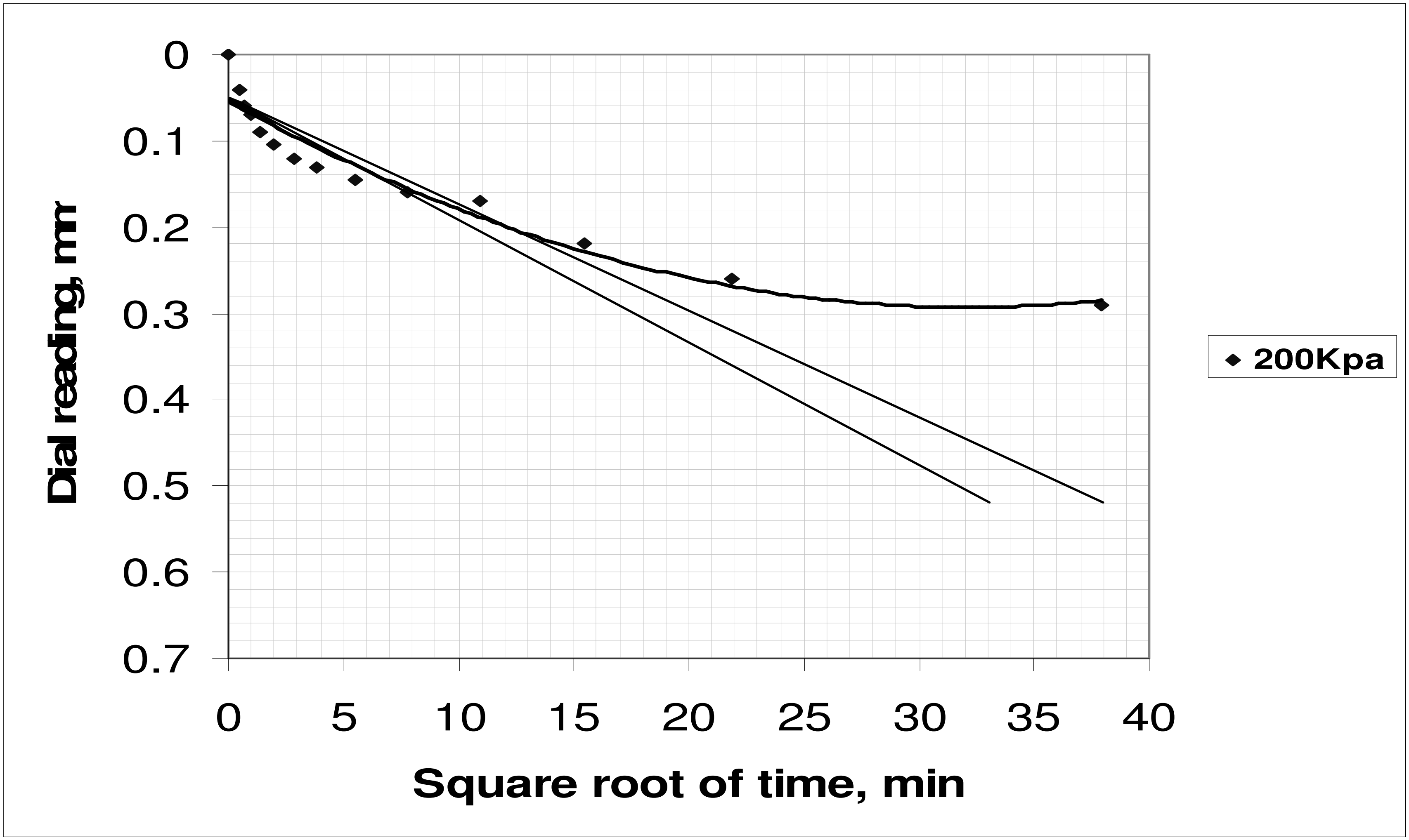


Fig B-7a, Typical plots of Dial reading versus Square root of time of sample8(P=200Kpa)

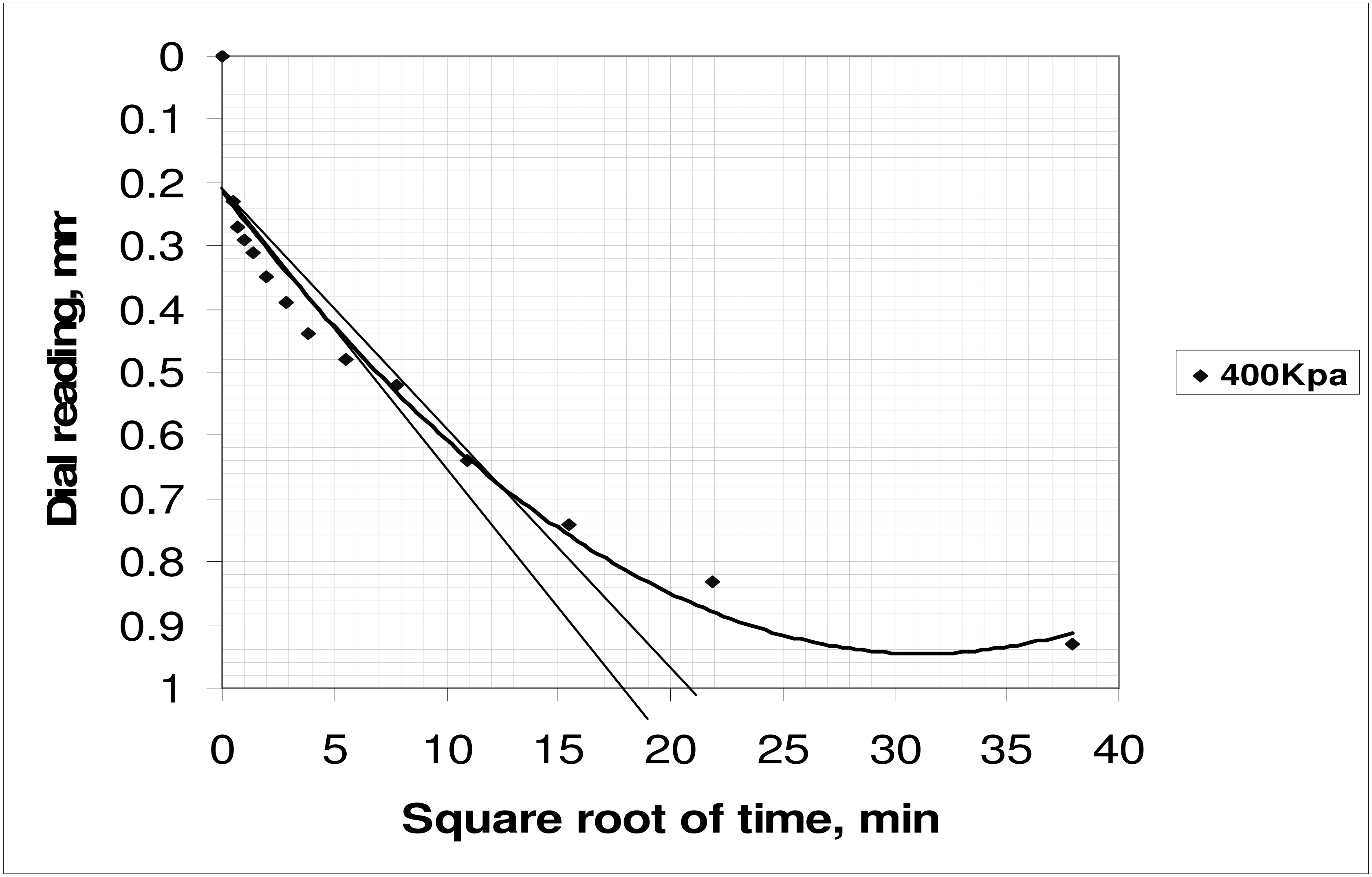


Fig B-7b, Typical plots of Dial reading versus Square root of time of sample-8(P=400Kpa)

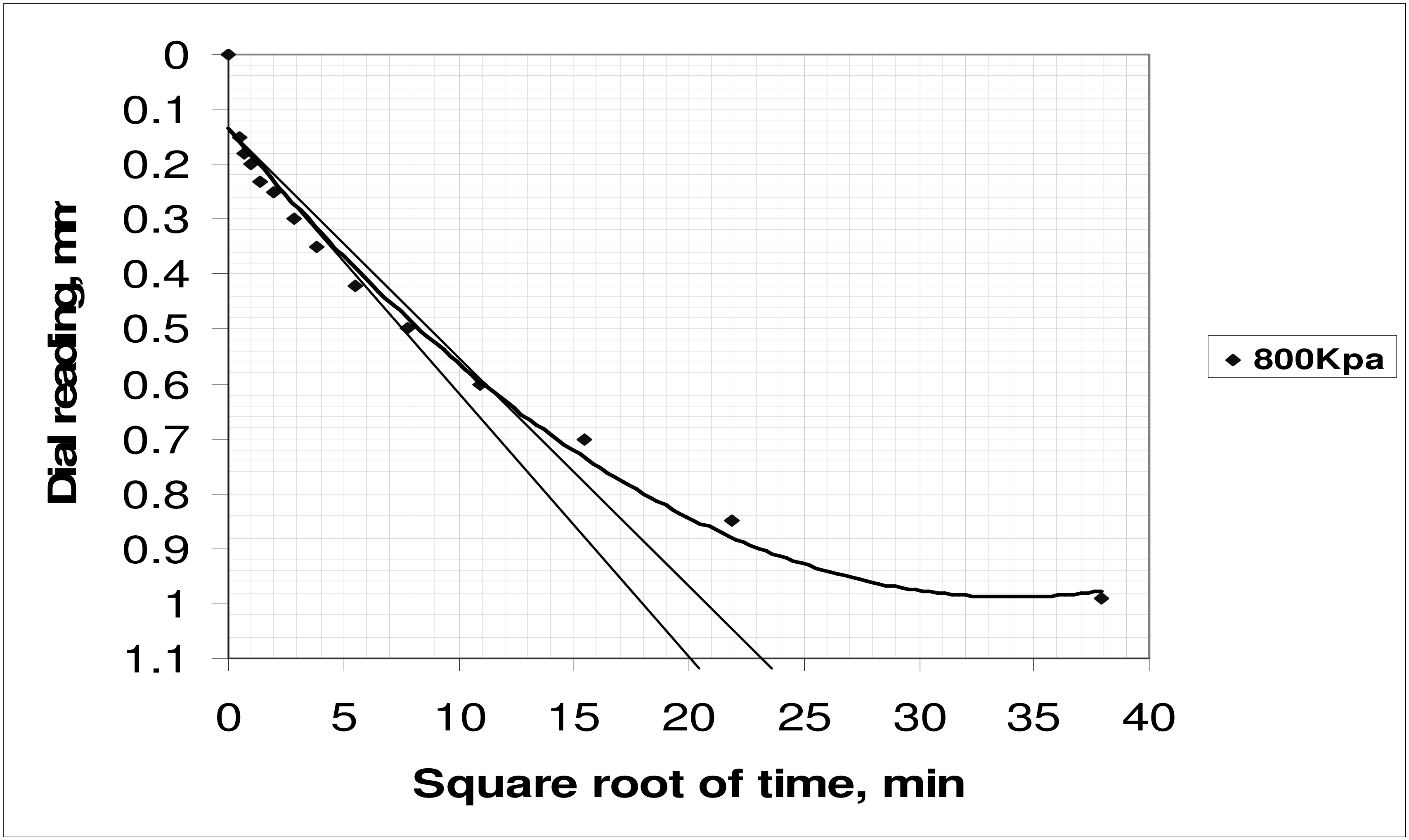


Fig B-7c, Typical plots of Dial reading versus Square root of time of sample-8 (P=800Kpa)

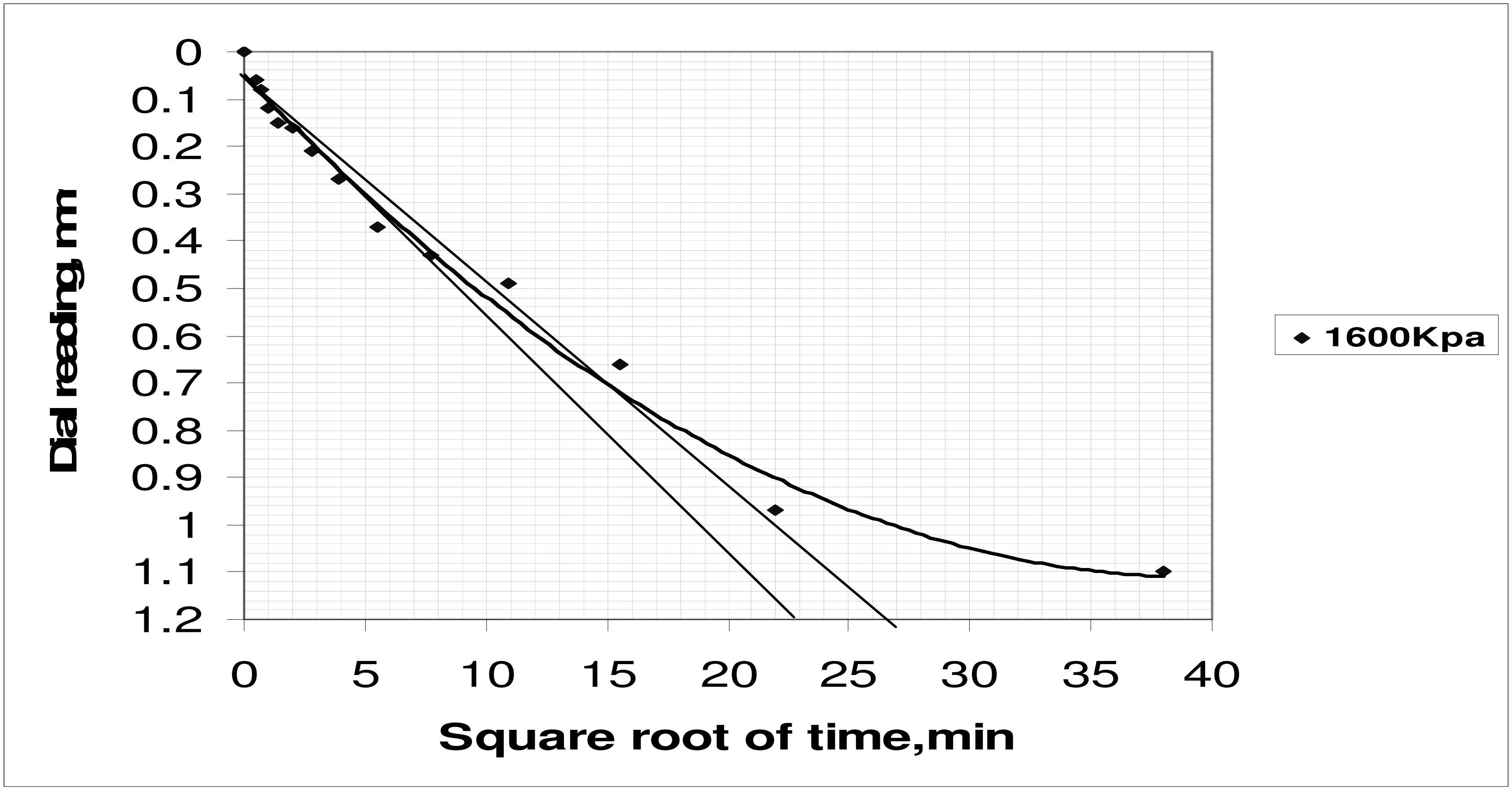


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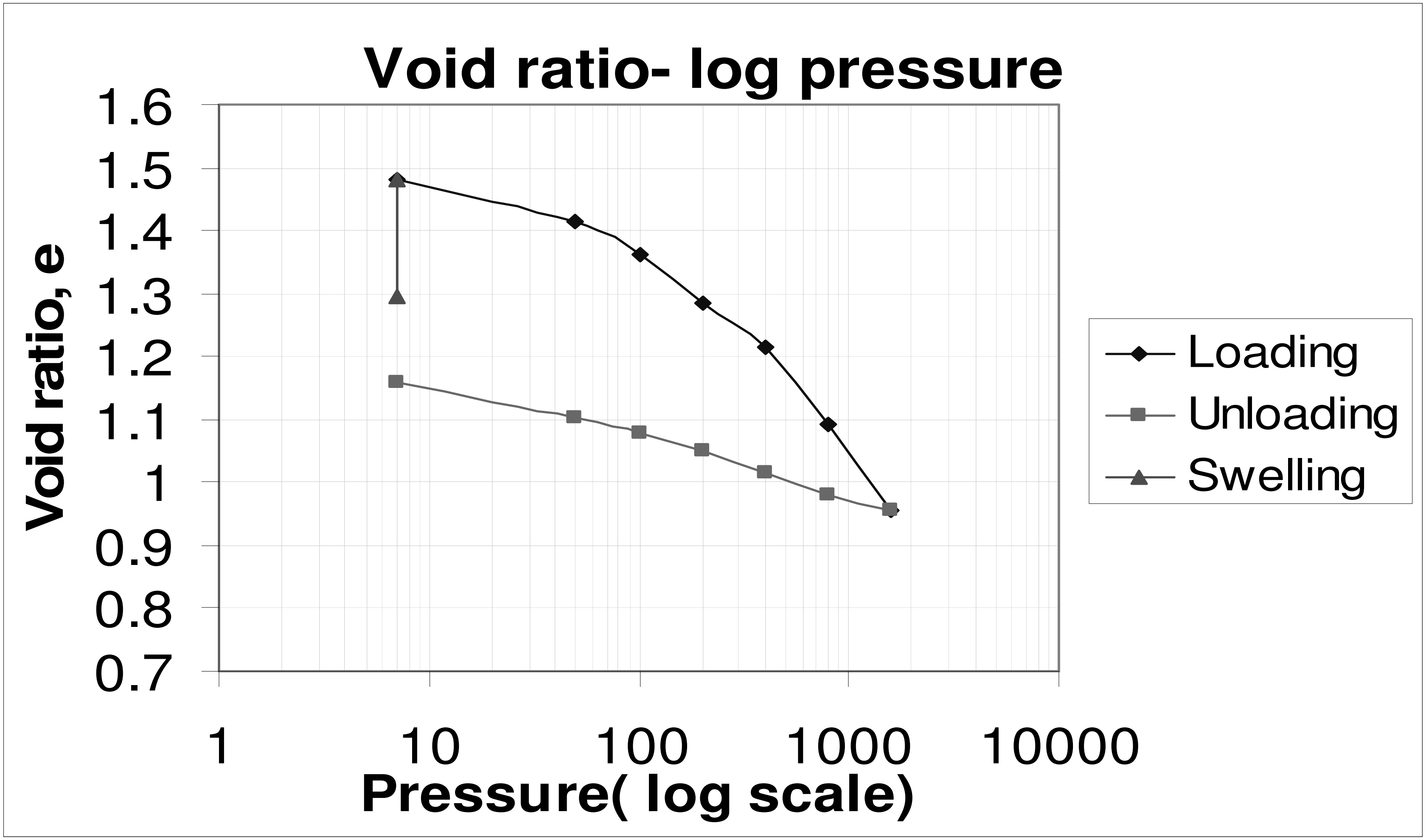
Fig B-7d, Typical plots of Dial reading versus Square root of time of sample-8(P=1600Kpa)



B.8 Plot of Void Ratio Versus Log-Pressure for Sample-9

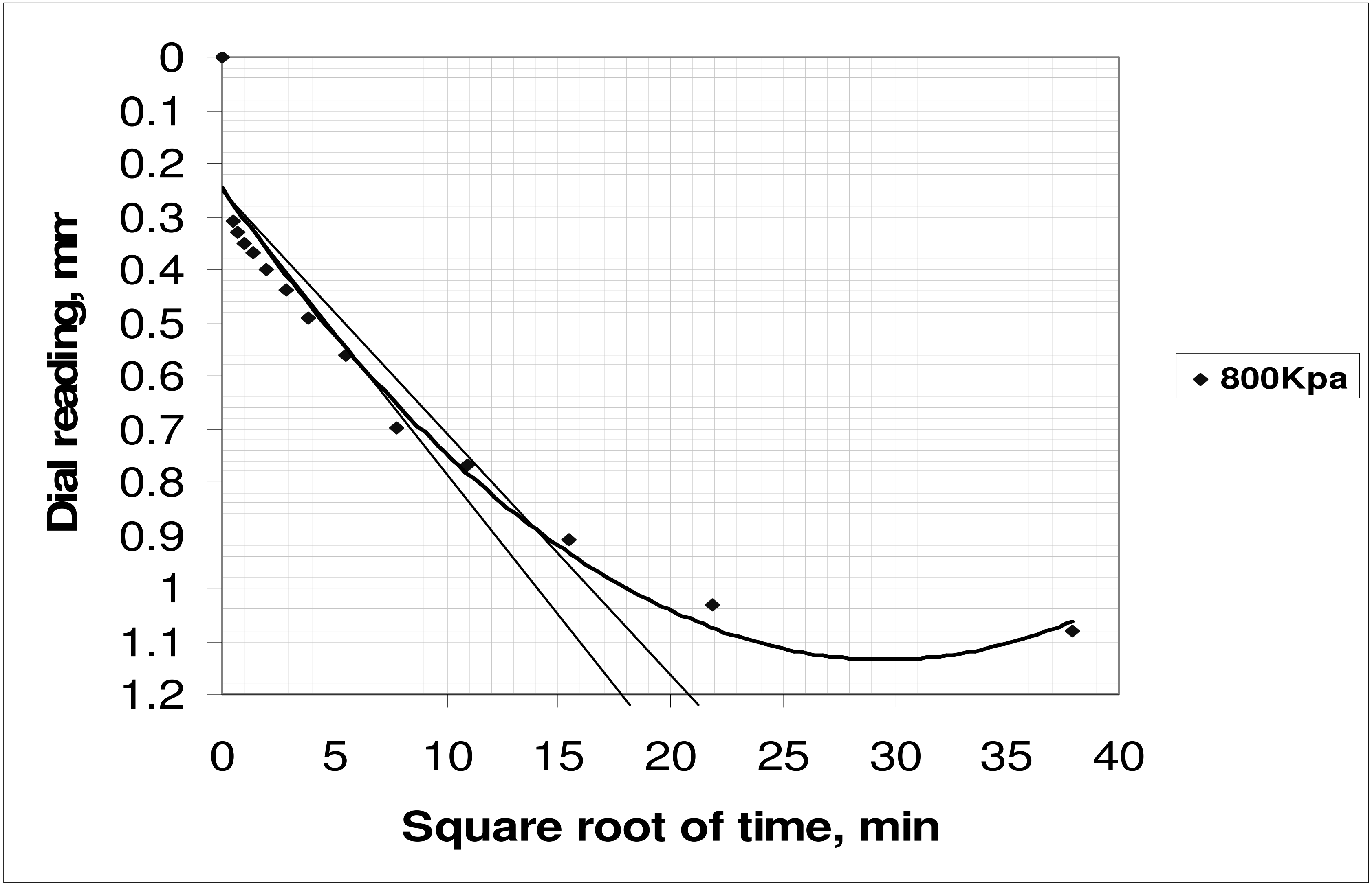


Fig B-8a, Typical plots of Dial reading versus Square root of time of sample-9 (P=800Kpa)

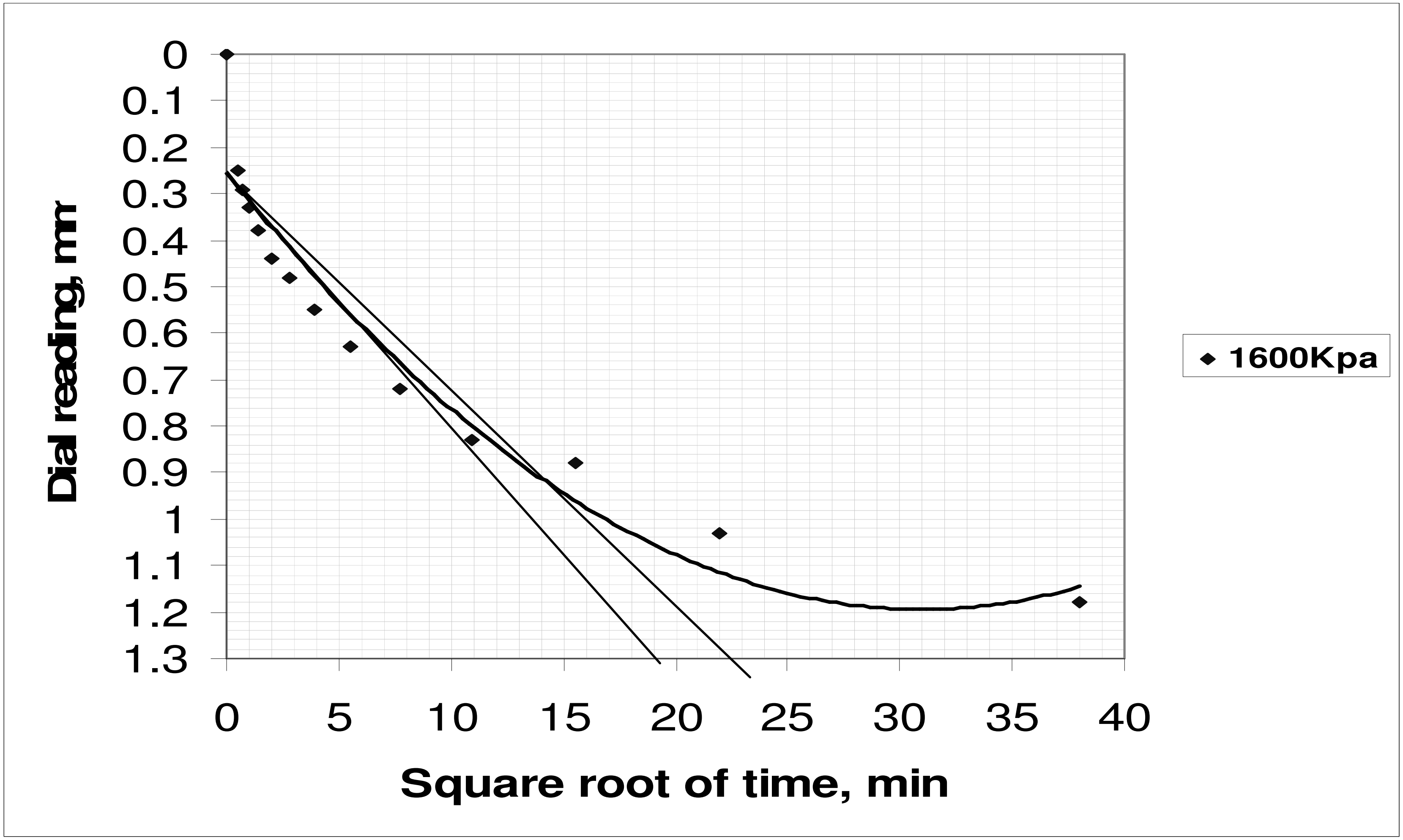


Fig B-8b, Typical plots of Dial reading versus Square root of time of sample-9 (P=1600Kpa)