LATERAL VERSUS VERTICAL SWELL PRESSURES IN EXPANSIVE SOILS

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ABSTRACT

LATERAL VERSUS VERTICAL SWELL PRESSURES IN EXPANSIVE SOILS

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Expansive or swelling soils, exist in many part of the world, show excessive volume changes with increasing water content. As a result of this volume increase, expansive soils apply vertical and lateral pressures to the structures located or buried in these regions. Many researchs have been carried out on vertical swelling pressures helping to the engineers to design structures withstanding on these stresses. However, lateral swell behaviour of swelling soils have not been fully understood yet. Structures such as; basement walls, water tanks, canals, tunnels, underground conduits and swimming pools which will be built in expansive soils have to be designed to overcome the lateral swelling pressures as well as the other lateral pressures exerted by the soil. For this aim accurate and reliable methods are needed to predict the magnitude of lateral swelling pressures of expansive soils and to understand the lateral swelling behaviour of expansive soils.

In this experimental study, the lateral swelling behaviour of an highly expansive clay is investigated using a modified thin wall oedometer which was developed in the METU Civil Engineering Department Soil Mechanics Laboratory earlier.

Statically compacted samples were used in constant volume swell (CVS) tests to measure the magnitude of the lateral and vertical swelling pressures. To study the relationship between the lateral and vertical sweeling pressures, they were measured simultaneously. The samples having different initial water contents and different initial dry densities were used to study the effects of these variables on the vertical and the lateral swelling pressures.

It is observed that both lateral and vertical pressures increases with increasing initial dry density and they decrease with increasing initial water content. Swell pressure ratio, the ratio of lateral swelling pressure to the vertical one, is increasing with increasing initial water content. Time needed to obtain the magnitude of maximum lateral and vertical pressures decreases with increasing initial water content and increases with increasing initial dry density.

Key words : Expansive soils, lateral swelling pressure, vertical swelling pressure, constant volume swell, initial water content, initial dry density, swell ratio, swell time.

ÖZ

ŞİŞEN ZEMİNLERDE YANAL VE DÜŞEY ŞİŞME BASINCI İLİŞKİSİ

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Yüksek Lisans, İnşaat Mühendisliği Bölümü
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Dünyanın birçok bölümünde genleşen veya şişen zeminler su içeriğinin artması ile çok büyük hacim değişiklikleri gösterirler. Bu hacim artışının sonucu olarak, genleşen zeminler bu bölgelerin üzerine veya içine inşa edilmiş yapılara düşey ve yanal basınçlar uygular. Düşey yöndeki şişme basıncı ile ilgili yapılmış birçok araştırma mühendislere bu basınçlara dayanabilen yapıları dizayn etmelerinde yardımcı olmaktadır. Fakat, şişen zeminlerin yanal yöndeki şişme davranışı henüz tam olarak anlaşılamamıştır. Şişen zeminlerde inşa edilecek olan, özellikle; bodrum duvarları, su tankları, kanallar, tüneller, yanal zemin basınçları yanında yanal şişme basıncınada dayanacak şekilde tasarlanmalıdır. Bu amaçla, hassas ve güvenilir yöntemlere genleşen zeminlerin yanal yöndeki şişme basıncının büyüklüğünü ölçmek ve genleşen zeminlerin yanal yöndeki şişme davranışını anlamak için ihtiyaç duyulmaktadır.

Bu deneysel çalışmada, ODTÜ Zemin Mekaniği Laboratuvarında daha önce geliştirilmiş ince cidarlı odometre kullanılarak yüksek şişme özelliğine sahip kil numunesinin yanal şişme davranışı araştırılmıştır.

Yanal ve düşey yöndeki şişme basınçlarının büyüklüğünü ölçmek için sabit hacimde şişme deneyinde statik kompaksiyon ile hazırlanmış numuneler kullanılmıştır. Yanal ve düşey yöndeki şişme basınçları arasındaki ilişkiyi araştırmak için bu ikisine ait ölçümler eş zamanlı alınmıştır. Başlangıç su içeriği ve başlangıç kuru birim ağırlığının yanal ve düşey yönlerdeki şişme basınçları üzerindeki etkisini araştırmak için farklı başlangıç su içeriği ve başlangıç kuru birim ağırlığına sahip numuneler kullanılmıştır.

Yanal ve düşey yöndeki şişme basınçlarının artan başlangıç kuru birim ağırlığı ile arttığı ve artan başlangıç su içeriği ile azaldığı gözlemlenmiştir. Yanal yöndeki şişme basıncının düşey yöndekine oranı olan şişme basıncı oranı artan başlangıç su içeriği ile artmaktadır. Yanal ve düşey yöndeki şişme basınçlarının en yüksek değerini elde etmek için gerekli olan zaman artan başlangıç su içeriği ile azalmakta ve artan başlangıç kuru birim ağırlığı ile artmaktadır.

Anahtar Kelimeler : Şişen zeminler, yanal yöndeki şişme basıncı, düşey yöndeki şişme basıncı, sabit hacimde şişme, başlangıç su içeriği, başlangıç kuru birim ağırlığı, şişme oranı, şişme zamanı.

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NOMENCLATURE

σ = Tensile stress on the thin wall of the oedometer

C = Clay content

C_s = Swell index

D = Diameter of specimen

dR = Enlargement of the internal radius of the oedometer ring

E = Elasticity modulus of the oedometer ring material

ε = Microstrain (10^{-6})

e = Void ratio

G_s = Specific gravity

H = Height of specimen

I_p = Plasticity index

K_o = At rest lateral earth pressure coefficient

LL = Liquid limit

N = Normal force

P_h = Lateral swelling pressure

P_i = Internal pressure

P_v = Vertical swelling pressure

R = Internal radius of the oedometer ring

SL = Shrinkage limit

T = Time

t = thickness of the thin wall oedometer ring

W_i = Initial water content

W_f = Final water content

PVC = Potential volume change

γ_d = initial dry density

S_r = Swell ratio, ratio of the lateral swelling pressure to the vertical swelling pressure

CHAPTER 1

INTRODUCTION

Some partially saturated clayey soils are very sensitive to variations in water content and show excessive volume changes. Such soils, when they increase in volume under applied loads because of an increase in their water contents, are classified as expansive soils and exist on many parts of the world.

The swelling phenomenon is considered as one of the most serious challenges which the foundation engineer faces, because of the potential danger of unpredictable upward movements of structures founded on such soils. It is well known that movement in expansive soils does not take place in only one direction. Damage to buildings and structures caused by lateral expansion should not be ignored.

Any structure, located or burried in expansive clay, may be subjected to large magnitudes of lateral pressures due to development of lateral component of the swelling pressure. When the moisture content of the clays increase, they would tend to expand, developing swell pressures and thus increase the applied lateral pressures on the structures such as piles, earth retaining structures, water tanks, canals, side slopes and buried structures (pipes, tunnels, culverts, etc.). Such damage may occur even with little evidence of vertical swelling.

In practice the most common method is replacement of these types of clays, around the proposed structure, with non-expansive soils. The feasibility of this method depends on the geometry of the case under consideration. For the cases needing large volume of replacement this can be costly and it is not usually feasible to isolate the sides of such structures from the surrounding materials and sidewalls must be designed to cater for the anticipated pressures.

Structures like tunnels, underground conduits, canal linings, retaining structures etc. located in expansive soils may be designed to withstand high lateral stresses, for this aim the magnitudes of the lateral swell pressures should be determined.

Several laboratory and in-situ techniques are avaliable for measurements of lateral swell pressures. Thin wall oedometer testing technique is one of the most precise laboratory method for direct measurement of lateral swell pressures of both compacted and undisturbed samples.

The scope of this investigation is to utilize a thin wall oedometer to investigate the lateral swell pressures of an expansive clay. The samples are obtained from Ankara Metro Line Construction – Station D29 located on the Batıkent – Sincan line. Also the relationship between lateral and vertical swelling pressures, and the effects of initial water content, initial dry density and time are studied. The swelling tests are performed using constant volume swell technique and the variables considered in the experimental program include initial water content, initial dry density, time, surcharge (vertical) pressure. And the influence of these variables on the lateral swell pressure is reported on this study.

CHAPTER 2

REVIEW ON EXPANSIVE SOILS

2.1 GENERAL

The three most important groups of clay minerals are montmorillonite, illite and kaolinite, which are crystalline hydrous aluminosilicates. Montmorillonite is the clay mineral that presents most of the expansive soil problem. These type of soils are generally resulting due to weathering of rocks. G.W. Donaldson (1969) classified the parent materials that can be associated with expansive soils into two groups; basic igneous rocks and sedimentary rocks.

Absorption of water by clays leads to expansion. From the mineralogical standpoint, the magnitude of expansion depends upon the kind and amount of clay minerals present, their exchangeable ions, electrolyte content of aqueous phase, and the internal structure.Deformational behaviour of clay, under changes of moisture content and stress, is therefore a complex function of both the physico-chemical and the effective strength characteristics of the clay.

2.2 SWELL PARAMETERS

2.2.1 Swelling Potential

The available literature introduced various definitions for the swell potential. Of these the definition advanced by Seeds et al. (1962) , Chen (1975) have been widely used.

In 1962, Seed defined swelling potential as the percentage of swell of a laterally confined sample on soaking under 1-psi surcharge, after being compacted to maximum density at optimum water content in the standart AASHO compaction test.

Such a definition is expected to take care of the in-situ conditions such as overburden pressure, initial moisture content, confinement and density. The definition of swell potential introduced by Snethen (1984), unlike most of the previous definitions satisfies most of the in-situ conditions. This definition states that Swell potential is the equilibrium vertical volume change from an oedometertype test (i.e. lateral confinement), expressed as a percent of the original height of an undisturbed soil specimen from its natural moisture content and density to a state of saturation under an applied load equivalent to the in-situ overburden pressure.

2.2.2 Swelling Pressure

By definition, swelling pressure is the pressure required to maintain the initial soil volume when soil specimen is subjected to moisture increase. On the other words, the swell pressure is defined as the load at which the void ratio is equal to the initial void ratio.

2.3 FACTORS INFLUENCING SWELLING OF SOILS

The mechanism of swelling in expansive clays is complex and is influenced by a number of factors. Expansion is a change of particle spacing and this is a result of changes in the soil water system that disturb the internal stress equilibrium. Many of the factors influencing the mechanism of swelling also affect, or are affected by, physical soil properties such as plasticity or density.

The factors influencing the swell potential of a soil can be considered in three different groups; 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. The soil, environmental factors and stress conditions that affect swell behaviour are sumarized in Table 2.1 and Table 2.2.

A number of factors affects the swelling pressure of clays. In-situ or placement moisture content, in-situ density, method, and amount of compaction are some physical factors commonly recognized as affecting the swelling pressure. In addition, the soil type is itself a major factor because the phsico-chemical behaviour, and the interparticle and intra-particle forces and reactions are controlled by the soil type.

2.3.1 Effect of Clay Mineral Type

The mineralogical composition and clay fraction content are main factors governing the swelling characteristics of expansive clayey soils. However, the determination of mineralogical composition needs relatively sophisticated test equipment and elaborate test procedures which may not be generally available for practical purposes.

Clay minerals of different types typically exhibit different swelling potentials because of variations in the electrical field associated with each mineral. The swelling capacity of an entire soil mass depends on the amount and type of clay minerals in the soil, the arrangement ans specific surface area of the clay particles, and the chemistry of the soil water surrounding those particles. Three important structural groups of clay minerals are described for engineering purposes as follows;

i)- Kaolinite group – generally non-expansive

ii)- Mica-like group – includes illites and vermiculites, which can be expansive, but generally do not pose significant problems.

Table 2.1. Soil Properties that Affect Swell Behavior

Factor	Description	References
Clay mineralogy	Clay minerals which typically cause soil volume changes are *montmorillonites*, *vermiculites*, and some *mixed layer minerals*. Illites and Kaolinites are infrequently expansive, but can cause volume changes when particle sizes are extremely fine (less than a few tenths of a micron)	Grim (1968); Mitchell (1973, 1976); Snethen et al. (1977)
Soil water chemistry	Swelling is repressed by increased cation concentration and increased cation valence.For example, Mg^{2+} cations in the soil water would result in less swelling than Na^{+} cations	Mitchell (1976)
Soil suction	Soil suction is an independent effective stress variable, represented by the negative pore pressure in unsaturated soils. Soil suction is relaed to saturation, gravity, pore size and shape, surface tension, and electrical and chemical characteristics of the soil particles and water (see Chapter 4)	Snethen (1980); Fredlund and Morgenstern (1977); Johnson (1973); Olsen and Langfelder (1965); Aitchison et al. (1965)
Plasticity	In general, soils that exhibit plastic behavior over wide ranges of moisture content and that have high liquid limits have greater potential for swelling and shrinking. Plasticity is an *indicator* of swell potential	See Section 3.1
Soil structure and fabric	Flocculated clays tend to be more expansive than dispersed clays. Cemented particles reduce swell. Fabric and structure are altered by compaction at higher water content or remolding. Kneading compaction has been shown to create dispersed structures with lower swell potential than soils statically compacted at lower water contents	Johnson and Snethen (1978); Seed et al. (1962a).
Dry density	Higher densities usually indicate closer particle spacings, which may mean greater repulsive forces between particles and larger swelling potential	Chen (1973); Komornik and David (1969); Uppal (1965)

Table 2.2. Environmental Factors, Stress Conditions that Affect Swell Behaviour

Factor	Description
1. Initial moisture condition	A desiccated expansive soil will have a higher affinity for water, or higher suction, than the same soil at higher water content, lower suction. Conversely, a wet soil profile will lose water more readily on exposure to drying influences, and shrink more than a relatively dry initial profile. The initial soil suction must be considered in conjunction with the expected range of final suction conditions
2. Moisture variations	Changes in moisture in the active zone near the upper part of the profile primarily define heave. It is in those layers that the widest variation in moisture and volume change will occur.
2.1 Climate	Amount and variation of precipitation and evapotranspiration greatly influence the moisture availability and depth of seasonal moisture fluctuation. Greatest seasonal heave occurs in semiarid climates that have pronounced, short wet periods
2.2 Groundwater	Shallow water tables provide a source of moisture and fluctuating water tables contribute to moisture
2.3 Drainage and manmade water sources	Surface drainage features, such as ponding around a poorly graded house foundation, provide sources of water at the surface; leaky plumbing can give the soil access to water at greater depth
2.4 Vegetation	Trees, shrubs, and grasses deplete moisture from the soil through transpiration, and cause the soil to be differentially wetted in areas of varying vegetation
2.5 Permeability	Soils with higher permeabilities, particularly due to fissures and cracks in the field soil mass, allow faster migration of water and promote faster rates of swell
2.6 Temperature	Increasing temperatures cause moisture to diffuse to cooler areas beneath pavements and buildings
3. Stress conditions	
3.1 Stress history	An overconsolidated soil is more expansive than the same soil at the same void ratio, but normally consolidated. Swell pressures can increase on aging of compacted clays, but amount of swell under light loading has been shown to be unaffected by aging. Repeated wetting and drying tend to reduce swell in laboratory samples, but after a certain number of wetting–drying cycles, swell is unaffected
3.2 In situ conditions	The initial stress state in a soil must be estimated in order to evaluate the probable consequences of loading the soil mass and/or altering the moisture environment therein. The initial effective stresses can be roughly determined through sampling and testing in a laboratory, or by making in situ measurements and observations
3.3 Loading	Magnitude of surcharge load determines the amount of volume change that will occur for a given moisture content and density. An externally applied load acts to balance interparticle repulsive forces and reduces swell
3.4 Soil profile	The thickness and location of potentially expansive layers in the profile considerably influence potential movement. Gratest movement will occur in profiles that have expansive clays extending from the surface to depths below the active zone. Less movement will occur if expansive soil is overlain by nonexpansive material or overlies bedrock at a shallow depth

iii)- Smectite group – includes montmorillonites, which are highly expansive and are the most troublesome clay minerals.

Lambe&Whitman (1969) reported that swelling ability varies with the type of clay mineral and decreases in the order; montmorillonite, illite, attapulgite and kaolinite and also depends considerably on exchangeable ions. Particle features and engineering properties of the important clay minerals are summarized in Table 2.3.

2.3.2 Effect of Clay Content

El-Sohby and Rabba (1981) studied the effect of clay content on swelling and swelling pressure with different clay percentages being mixed, in one case with sand and, in the other, with silt. According to the experimental results; depending on the soil properties and the applied pressure, the swelling and the value of swelling pressure increases as the percentage of clay increases. The result of tests are showed in Figure 2.1, Figure 2.2 and Figure 2.3.

2.3.3 Effect of Initial Water Content

In order to study the effect of the initial water content on the amount of swelling and swelling pressure El-Sohby and Rabba (1981) prepared remoulded specimens having different initial water contents at the same initial dry density. The effect of initial water content on the amount of swelling under different vertical pressures, σ, is shown in Figure 2.4.

Table 2.3. Particle Features and Engineering Properties of Some Important Clay Minerals

Mineral Group	Basal Spacing (Å)	Particle Features	Interlayer Bonding	Specific Surface (m^2/g)	Atterberg Limits[a] LL (%)	PL (%)	SI (%)	Activity[b] (PI/% Clay)
Kaolinites	14.4	Thick, stiff 6-sided flakes 0.1 to 4 × 0.05 to 2 μm	Strong hydrogen bonds	10–20	30–100	25–40	25–29	0.38
Illites	10	Thin, stacked plates 0.003 to 0.1 × 1.0 to 10 μm	Strong potassium bonds	65–100	60–120	35–60	15–17	0.9
Montmorillonites	9.6	Thin, filmy, flakes >10 Å × 1.0 to 10 μm	Very weak van der Waals bonds	700–840	100–900	50–100	8.5–15	7.2

[a] LL, PL, SL, liquid, plastic, and shrinkage limits, respectively.
[b] From Skempton (1953).
Summarized from Mitchell (1976).

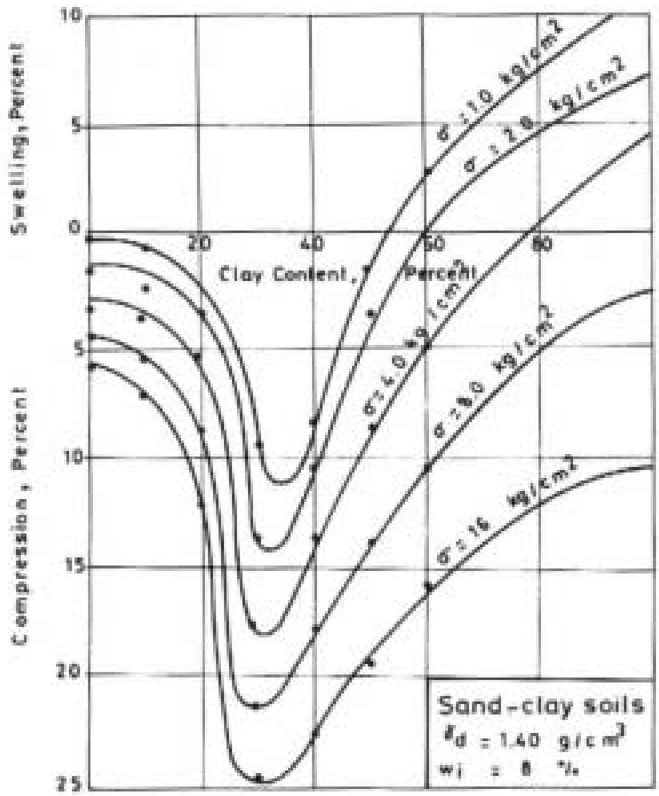


Figure 2.1 Effect of Clay Content on Swelling (After El– Sohby and Rabba,1981)

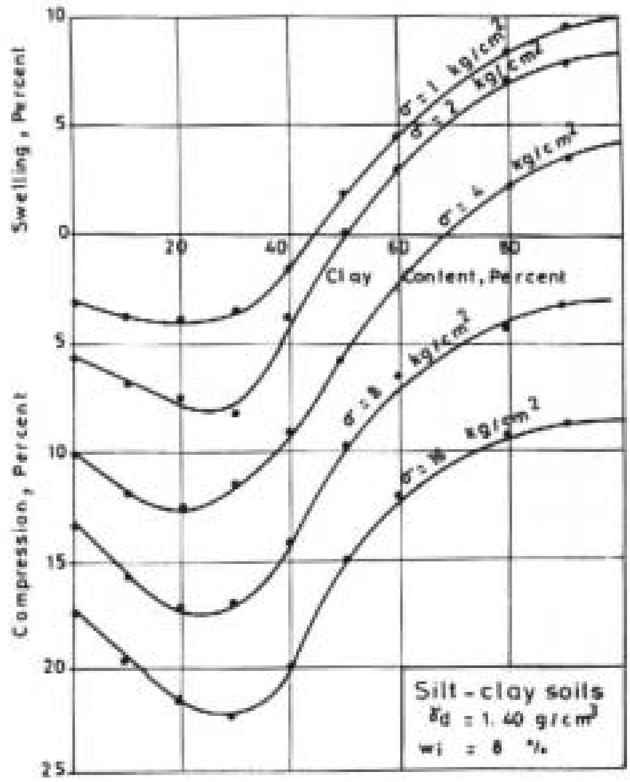


Figure 2.2 Effect of Clay Content on Swelling (After El– Sohby and Rabba,1981)

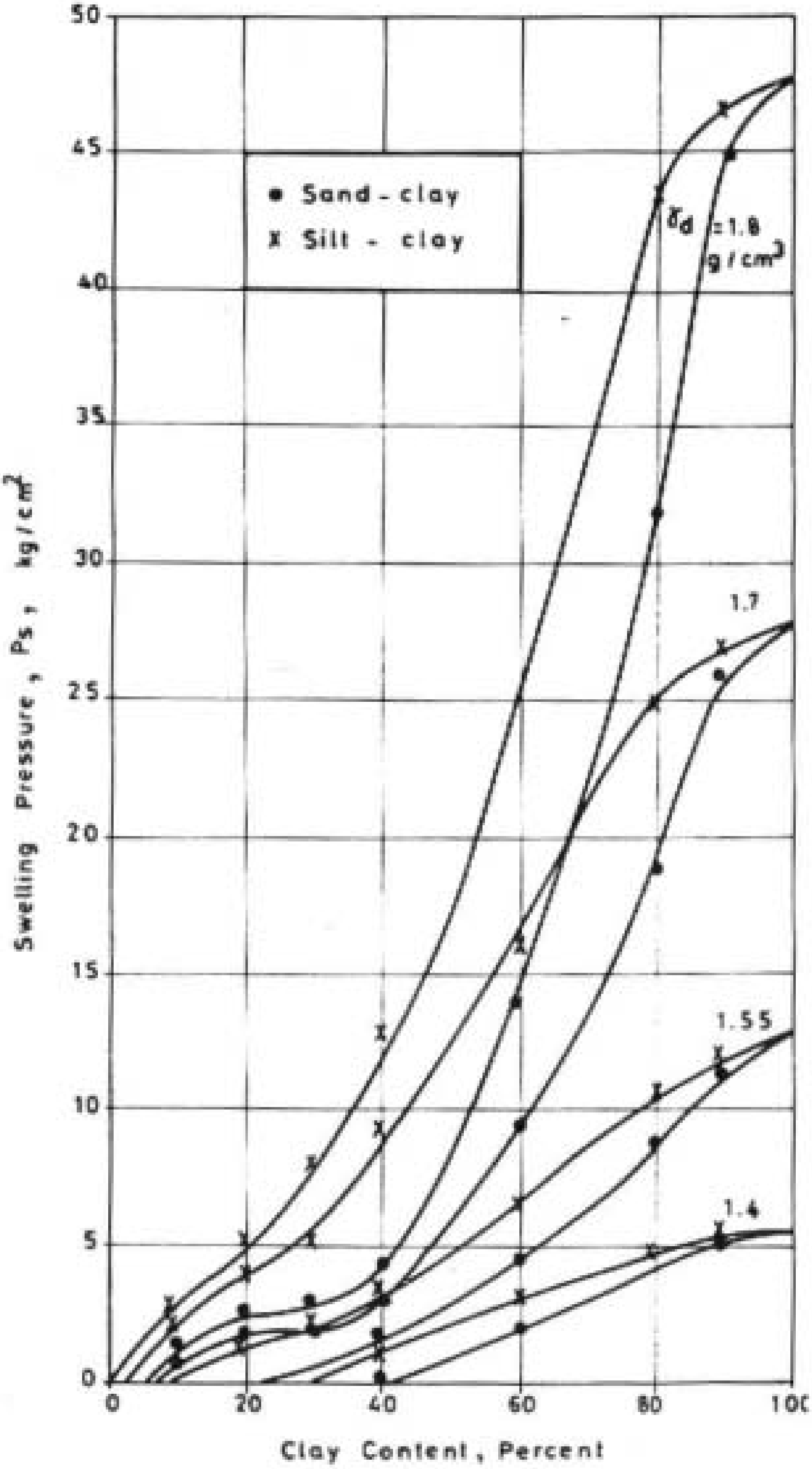


Figure 2.3 Effect of Clay Content on Swelling Pressure

(After El– Sohby and Rabba,1981)

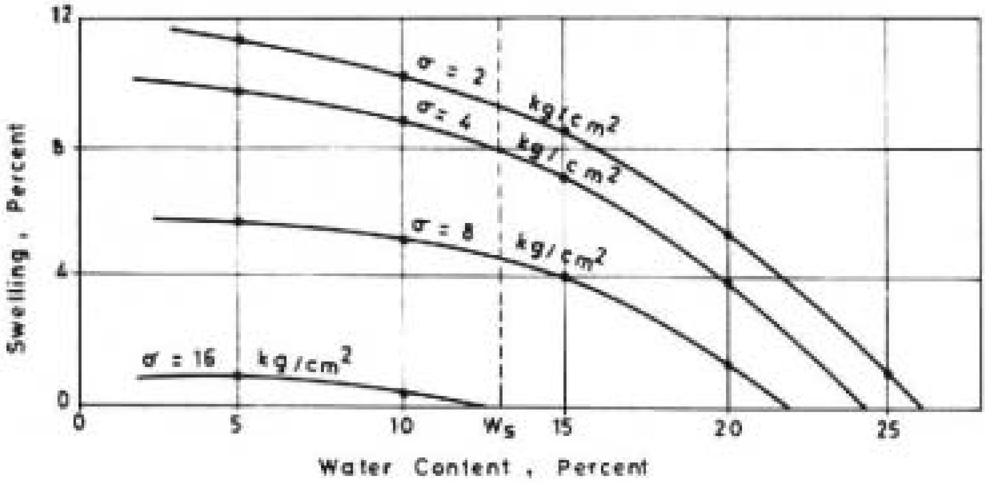


Figure 2.4 Effect of Initial Water Content on Swelling
(After El– Sohby and Rabba,1981)

It is evident from Figure 2.4 that the initial water content has a considerable influence on the percentage swelling of the remoulded samples. These results may be expected, since as the initial water content increases, for specimens having the same initial dry unit weight, the initial degree of saturation will also increase and the affinity of soil to absorb water will decrease. It follows that the amount of water absorbed for complete saturation will become smaller, and consequently the amount of swelling will decrease as the initial water content increases. The effect of the initial water content on the swelling pressure is shown in Figure 2.5.

As it can be seen from the figure as the initial water content increases the swelling pressure decreases, but the relationship is non-linear. From Figure 2.4 and Figure 2.5 it can be also noted that the initial water content has a small effect on both swelling and swelling pressure until it reaches the shrinkage limit. For values of initial water content exceeding the shrinkage limit the rates of decresases by increasing water content becomes steeper as shown in the figures. Similar results were noted by El-Ramlı (1965) and Edil and Analazy (1992).

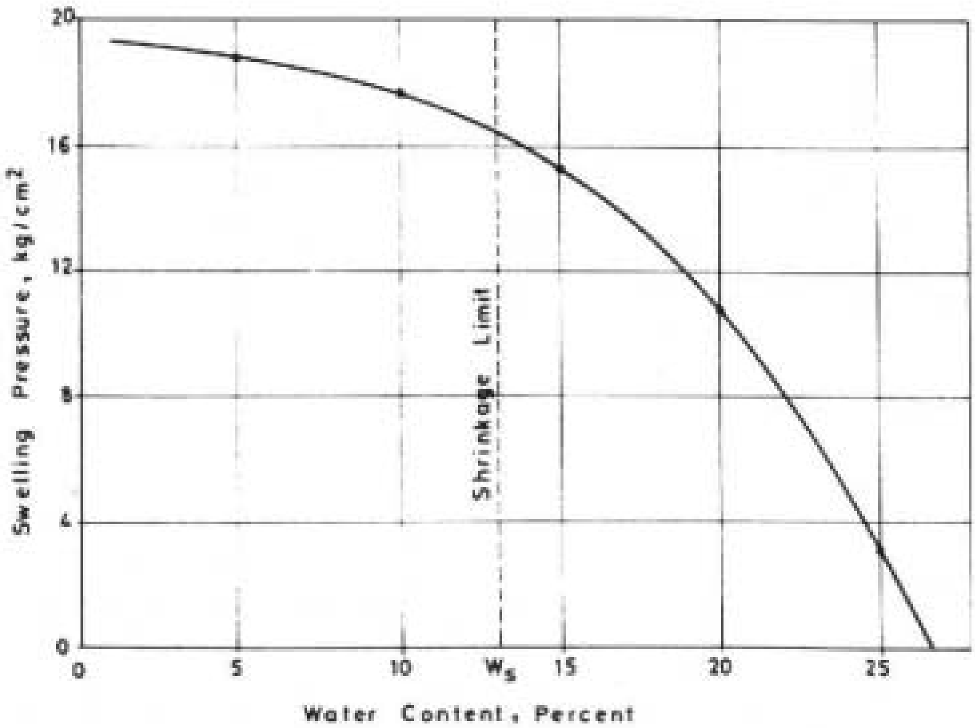


Figure 2.5 Effect of Initial Water Content on Swelling Pressure
(After El– Sohby and Rabba,1981)

2.3.4 Effect of Initial Dry Density

El-Sohby and Rabba (1981) carried out conventional oedometer test to study the effect of initial dry density on swelling and swelling pressure of soils. For each specimen the swelling under different pressures and, consequently, the swelling pressure is determined. The results of these test can be seen in Figure 2.6 and Figure 2.7.

The results of these tests showed that as the initial dry density increases both of the percent swell and the value of swelling pressure increase.

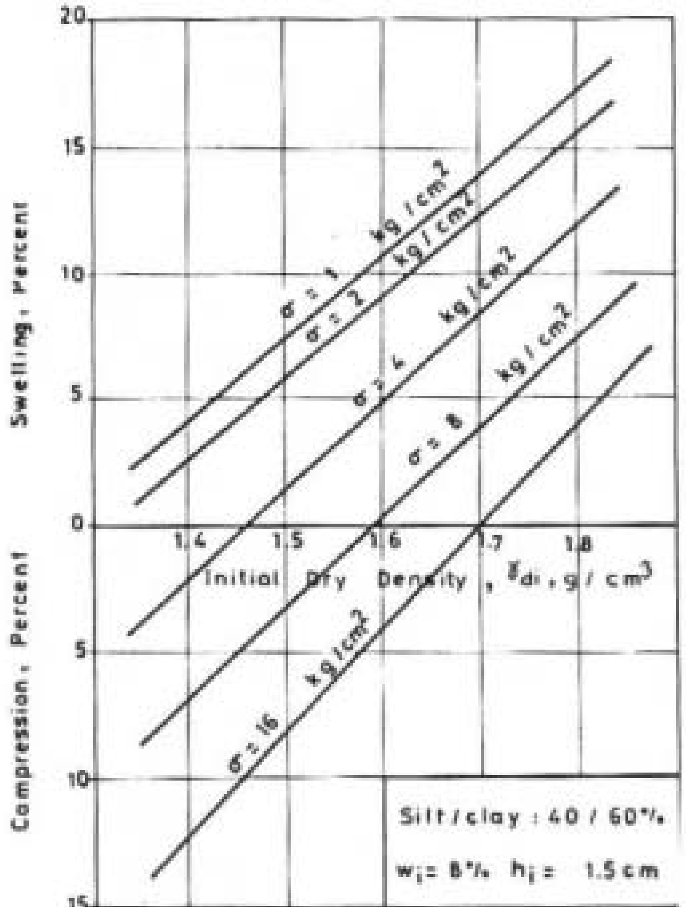


Figure 2.6. Effect of Initial Dry Density on Swelling Pressure

(After El– Sohby and Rabba,1981)

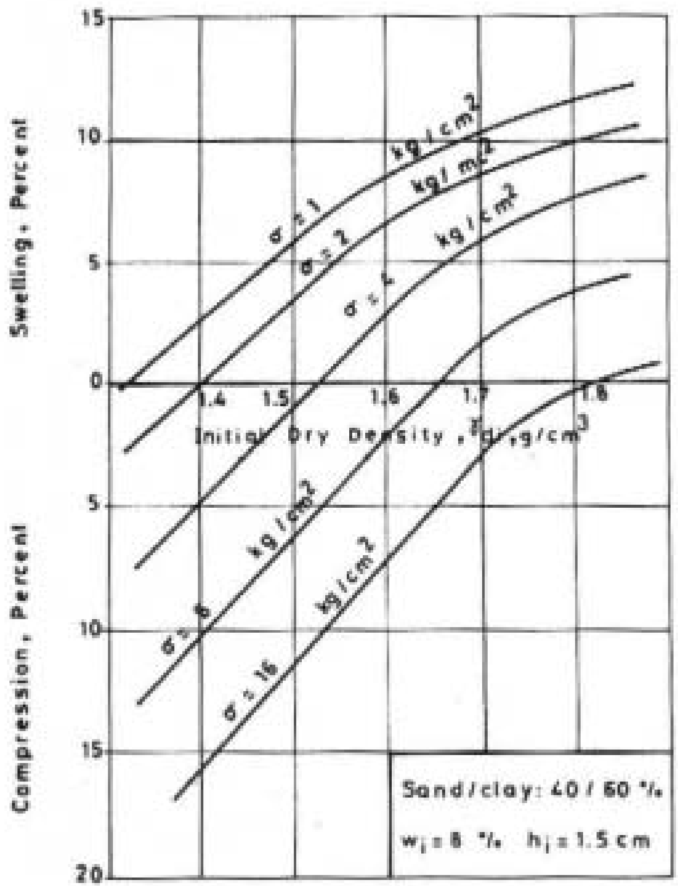


Figure 2.7. Effect of Initial Dry Density on Swelling

(After El– Sohby and Rabba,1981)

2.4 IDENTIFICATION AND CLASSIFICATION OF EXPANSIVE SOILS

In the design and construction of engineering structures, it is vital at the outset of an investigation to identify and classify soils that are potentially expansive. A first priority for a geotechnical engineer working in areas suspected for the presence of expansive soils, is to identify such soils and classify them according to their expected swelling behaviour. Failure to identify and classify these soils will result in extensive damages. Proper identification and classification will help in decision making during the stages of final soil investigation, design, execution and servicing of the engineering structure.

2.4.1 Identification of Expansive Soils

2.4.1.1 Direct Methods

The direct methods include mineralogical identification of the soil or quantitative evaluation of the swelling characteristics of an undisturbed sample in the laboratory. The direct methods usually need special equipment and are considered not suitable for quick identification of expansive soils. (Chen 1975)

Some of the laboratory tests that are used for the identification of expansive soils are; Improved swell oedometer (or free swell) test (ISO), Constant volume swell (CVS) test, Swell overburden (SO) test, soil suction tests and PVC meter test. These laboratory methods will be further discussed in the 'Prediction of swell parameters' section.

2.4.1.2 Indirect Methods (Empirical Formule)

The indirect methods for the identification of expansive soils rely on experience and simple indices such as Atterberg limits, clay content, initial water content, dry density and shrinkage limit. Such methods usually need simple apparatus and lend themselves for use by geotechnical engineers for quick and reliable identification of expansive soils. Some empirical formule reported by various authors in literature are are shown in Table 2.4

Table 2.4 Some Empirical Formule Reported by Various Authors in Literature (After EROL,1987, references are cited in EROL,1987)

	Reference	Description
1	Seed et al	$Sp=0.00216\ PI^{2.44}$
2	Van der Merve	$\Delta H= F\ e^{-0.377D}\ (e^{-0.377H}-1)$
3	Vijayvergia et al	$\text{Log } Sp= (0.44LL-\omega_o+5.5)/12$
4	Nayak et al	$Sp=(0.00229PI)(1.45C)/\omega_o+6.38$
5	Schnider et al	$\text{Log } Sp=0.9(PI/\omega_o-1.19$
6	Weston	$Sp=0.00411(LL_\omega)^{4.17}\sigma_v^{-3.86}\omega_o^{-2.33}$
7	Jhonson	$Sp=23.82+0.7346PI-0.1458H-1.7\omega_o$ $+0.00225PI\omega_o-0.0088PIH$
8	Komornik et al	$\text{Log } Ps=-2.132+0.0208LL+0.000665\gamma_d$ $-0.0269\omega_o$ Note: γ_d in kg/m^3

DEFINITIONS OF SYMBOLS			
Sp	Percent Swell	LL	Liquid Limit
PI	Plasticity Index	ω_o	Initial Water Content
ΔH	Total Heave	γ_d	Dry Unit Weight
F	Correction Factor for Degree of Expansiveness	C	Clay Percent
D	Thickness of Nonexpansive Layer	σ_v	Sucharge Load
H	Thickness of Expansive Layer	LL_ω	Weighted Liquid Limit

2.4.2 Classification of Expansive Soils

Many classification schemes provide an ‘expansion rating’ to provide a qualitative assessment of the degree of probable expansion. Expansion ratings may be such as; high, medium, and low, or critical and noncritical. A reasonable classification involves the use of soil properties and provides one or more of the following ratings;

i)- Ranges of the values for either probable percentage of volume change, or probable swelling pressure.
ii)- A qualitative expansion rating, i.e., low, medium, high, and very high expansion potential.
iii)- Some other classification parameter such as CH or A6 (in the USC and AASHTO schemes, respectively). These ratings are specific to the classification scheme, and defined for each individually.

It is important therefore to take into account the fact that two soils may have the same swell potential, according to their classification, but exhibit very different amounts of swell. (Seed et al., 1962)

2.4.2.1 USBR Classification System

United States Bureau of Reclamation (USBR) proposed a classification system for expansive soils. Colloid content less than 2 μm (percent by weight), plasticity index, shrinkage limits and percent swell values are used to classify the expansive soil. Table 2.5 shows the USBR classification system.

Table 2.5 USBR Classification System.

Data from index tests*			Probable expansion, percent total vol.change	Degree of expansion
Colloid content, percent minus 0.001 mm	Plasticity Index	Shrinkage limit		
> 28	> 35	< 11	> 30	Very high
20-13	25-41	7-12	20-30	High
13-23	15-28	10-16	10-30	Medium
> 15	<18	> 15	<10	Low

2.4.2.2 USAEWES Classification System

United States Army Engineer Waterways Experiment Station (USAEWES) recommended another classification system using liquid limit, plasticity index, in-situ soil suction and percent swell values. Table 2.6 shows the USAEWES classification system.

Table 2.6 USAEWES Classification System.

LL (%)	I_p (%)	$\tau_{initial}$ (Tons/sq.ft)	Potential swell (% swell)	Potential swell classification
< 50	< 25	< 1.5	< 0.5	LOW
50-60	25-35	1.5-4.0	0.5-1.5	MARGINAL
> 60	> 35	> 4.0	> 1.5	HIGH

2.4.2.3 Classification System Based on Activity

The activity method proposed by Seed, Woodward and Lundgen (1956) was based on remoulded, artificially prepared soils tested in standart AASHO compaction test under a surcharge of 1 psi. The activity for the artificially prepared sample was defined as;

Activity = Ip / C %

Where; C is the percentage clay size finer than 2 μm. The proposed classification chart is shown in Figure 2.8.

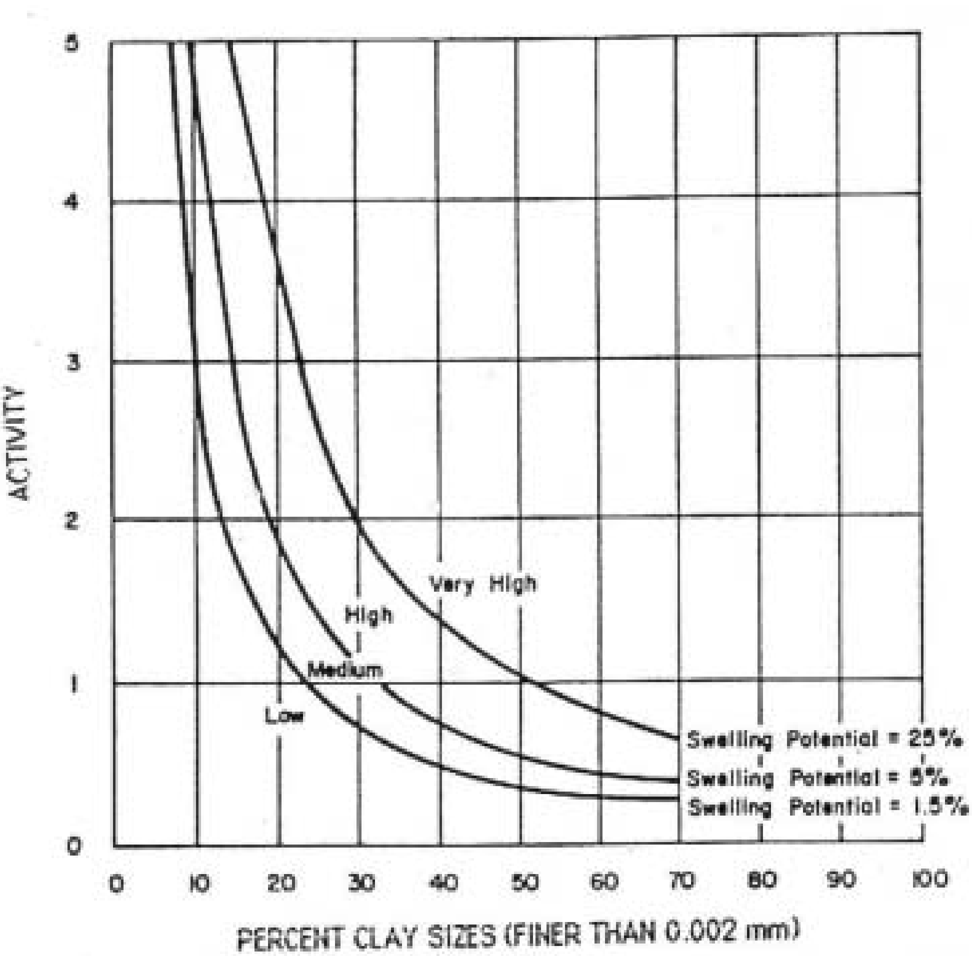


Figure 2.8 Classification Chart for Swelling Potential
(After Seed,Woodward&Lundgren, 1956)

According to Popescu (1986) swell potential of a clay can be estimated with using its clay percent and plasticity index. The proposed chart is shown in Figure 2.9.

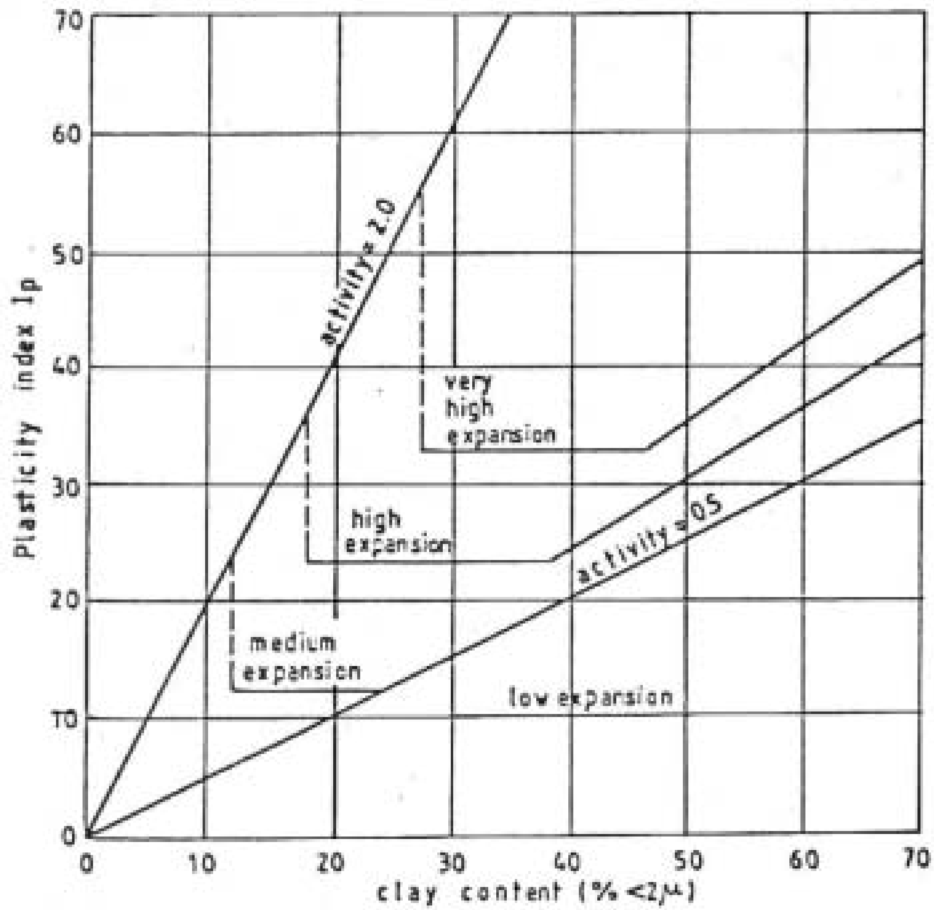


Figure 2.9 Estimation of Degree of Swell Potential (After Popescu, 1986)

2.5 PREDICTION OF SWELL PARAMETERS

Swelling potential and swelling pressure are called as swell parameters as discussed in section 2.2. Swell parameters can be measured by using various laboratory tests. Another way to predict approximate values of the swell parameters is to use the emprical formule,tables and charts avaliable in the literature with using laboratory determined index properties as discussed in section 2.4.1.2.

2.5.1 Measurement of Swell Parameters by Oedometer Tests

There are three types of oedometer tests performed in the laboratory to obtain swell behaviour and to measure the swell parameters. At these tests mainly undisturbed samples are used to obtain more reliable results, because undisturbed can reflect in-situ conditions. However, some of the in-situ conditions such as lateral confinement, availability of ground water, drainage conditions may not be well simulated at laboratory testing conditions.

2.5.1.1 Improved Swell Oedometer (or Free Swell) Test (ISO Test)

In these tests a predetermined surcharge pressure is applied to the specimen in dry, and kept for one hour for equilibrium. Then the sample is inundated with water and allowed to swell freely under a seating load of 7 kPa until primary swell is completed. Thereafter, it is loaded (after primary swell has occured) until its initial void ratio (height) is obtained. The swelling pressure can be taken as the pressure that brings the sample back to its initial height (i.e. at 0% vertical swell). Then the sample is unloaded in decrements to obtain the compression and the rebound characteristics (e - log P curve). The swell pressure on a e – log P curve can be seen on Figure 2.10 (Erol, 1987).

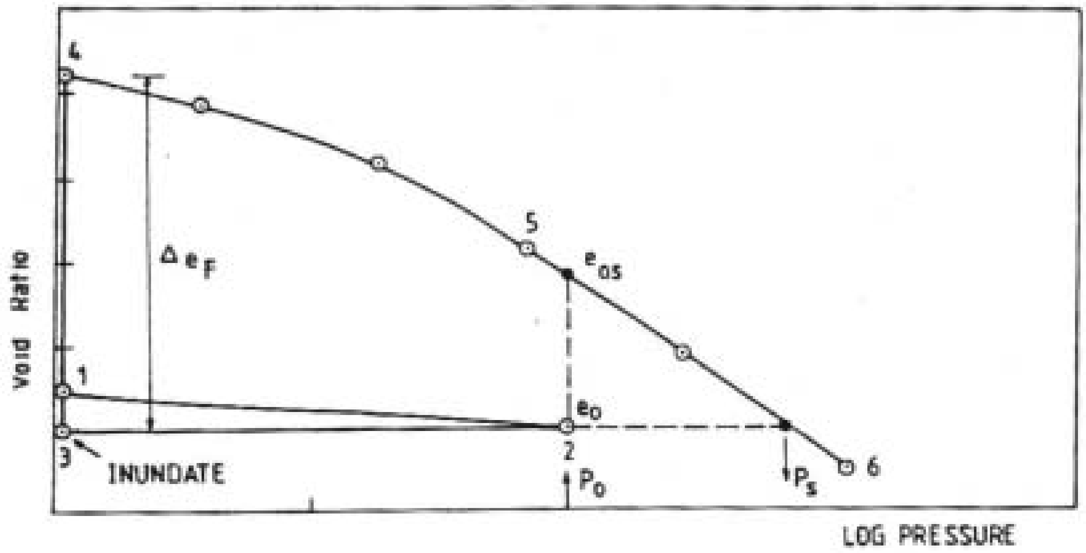


Figure 2.10 e - logP Curve in a ISO Test (After EROL, 1987)

2.5.1.2 Swell Overburden (SO) Test

In the swell overburden test, the specimen is loaded to a predetermined surcharge pressure in dry, and kept for one hour for equilibrium. Then the sample is inundated with water and allowed to swell until primary swell is completed. Specimen is then loaded for zero swell, i.e. until it reaches to its original height. Unloading can be performed to obtain rebound characteristics until equilibrium swell is obtained. Typical e – logP curve of SO test is shown in Figure 2.11.

2.5.1.3 Constant Volume Swell (CVS) Test

In this technique the specimen is placed into the oedometer ring and then water is added. This causes vertical deformations to develop but the deformations are prevented by applying small increments of vertical pressure in soaked conditions. The aim is to arrive at a load at in which there is neither swelling nor compression, maintaining the specimen at the original volume. The test continued until no swelling is registered.

The pressure developed by the specimen when saturation is completed represents a direct measure of swelling pressure.

After this stage the sample is unloaded in decrements to obtain the rebound characteristics.From e – logP curve, shown in Figure 2.12, swell pressure Ps is determined.

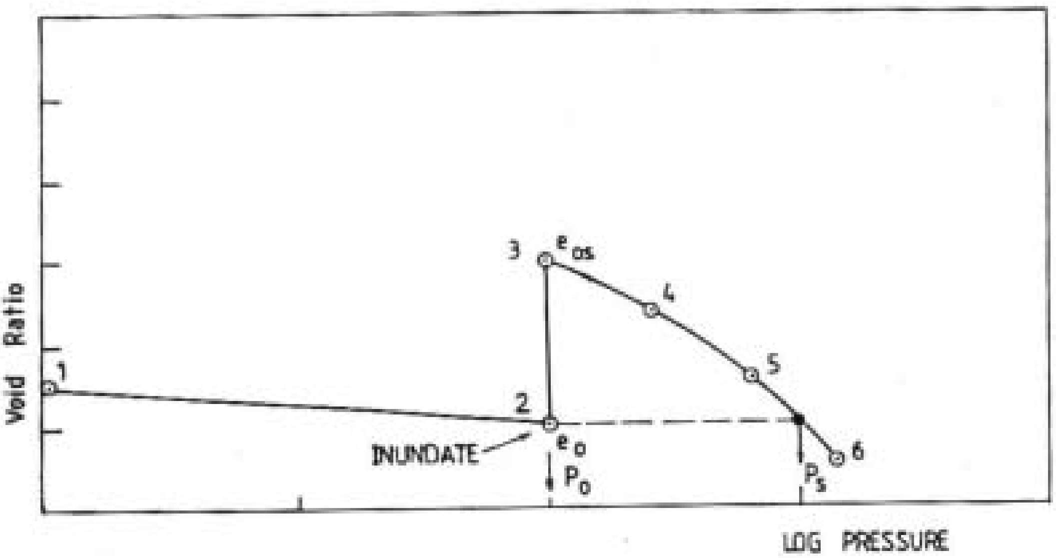


Figure 2.11 e - logP Curve in a SO Test (After EROL, 1987)

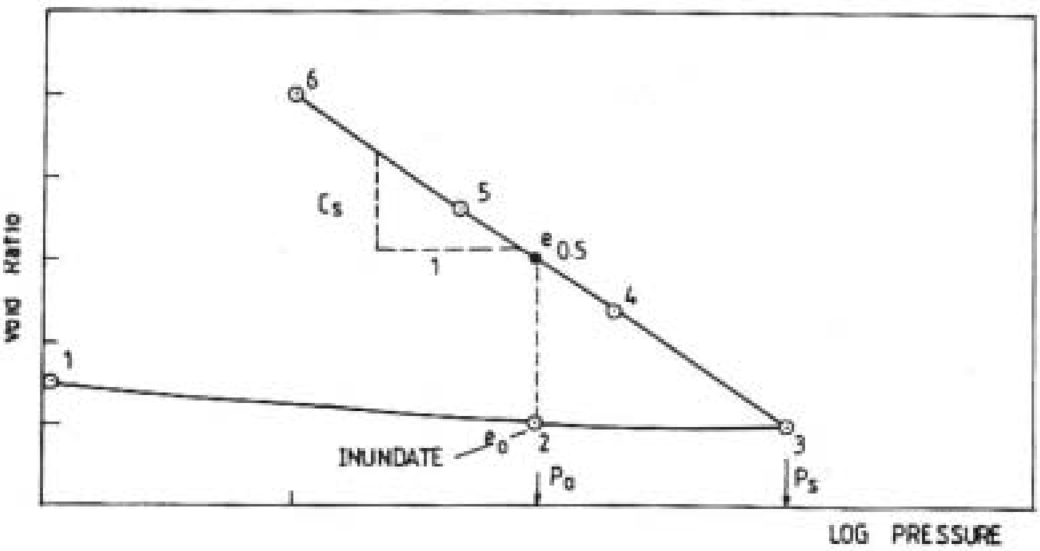


Figure 2.12 e - logP Curve in a CVS Test (After EROL, 1987)

2.5.2 PVC Meter

Potential volume change (PVC) determination was developed by T.W Lambe (1960) under the auspices of the Federal Housing Administration. The remoulded sample is firstly compacted in fixed ring consolidometer under a pressure of 55.000 ft-ibs. per cu.ft. Then the initial pressure of 200 psi was applied, and the water is added to sample. Vertical expansion of the sample was restrained by means of a proving ring and the proving ring reading is taken at the end of 2 hours. This reading is converted to pressure and it is called as swell index. The swell index can be converted to potential volume change by using Figure 2.13. The results of PVC meter tests should be used only to compare various swelling soils.

2.5.3 Soil Suction

In suction methods test equilibrium soil suction (negative pore pressure) is measured. Soil suction describes the interaction between soil particles and water which determines the physical behaviour of soil mass. (Snethen et.al,1980)

For this test Thermocouple Psychrometer test set-up or pressure plate can be used. The relative humidity of soil sample can be measured by using psychrometer method. Soil suction, which can be assumed equivalent to swell pressure, may be converted from the relative humidity using the principles of thermodynamics. The slpoe of the curve obtained from the percent water content vs. log soil suction plot is the most important parameter for the calculation of total heave by suction method. The less steeper curve represents higher swell potential. Soil suction method is more easier to use when compared with oedometer techniques because this technique takes shorter test period; having capabilities of simulating environmental factors and effects of lateral pressure on heave. (Erol, 1987)

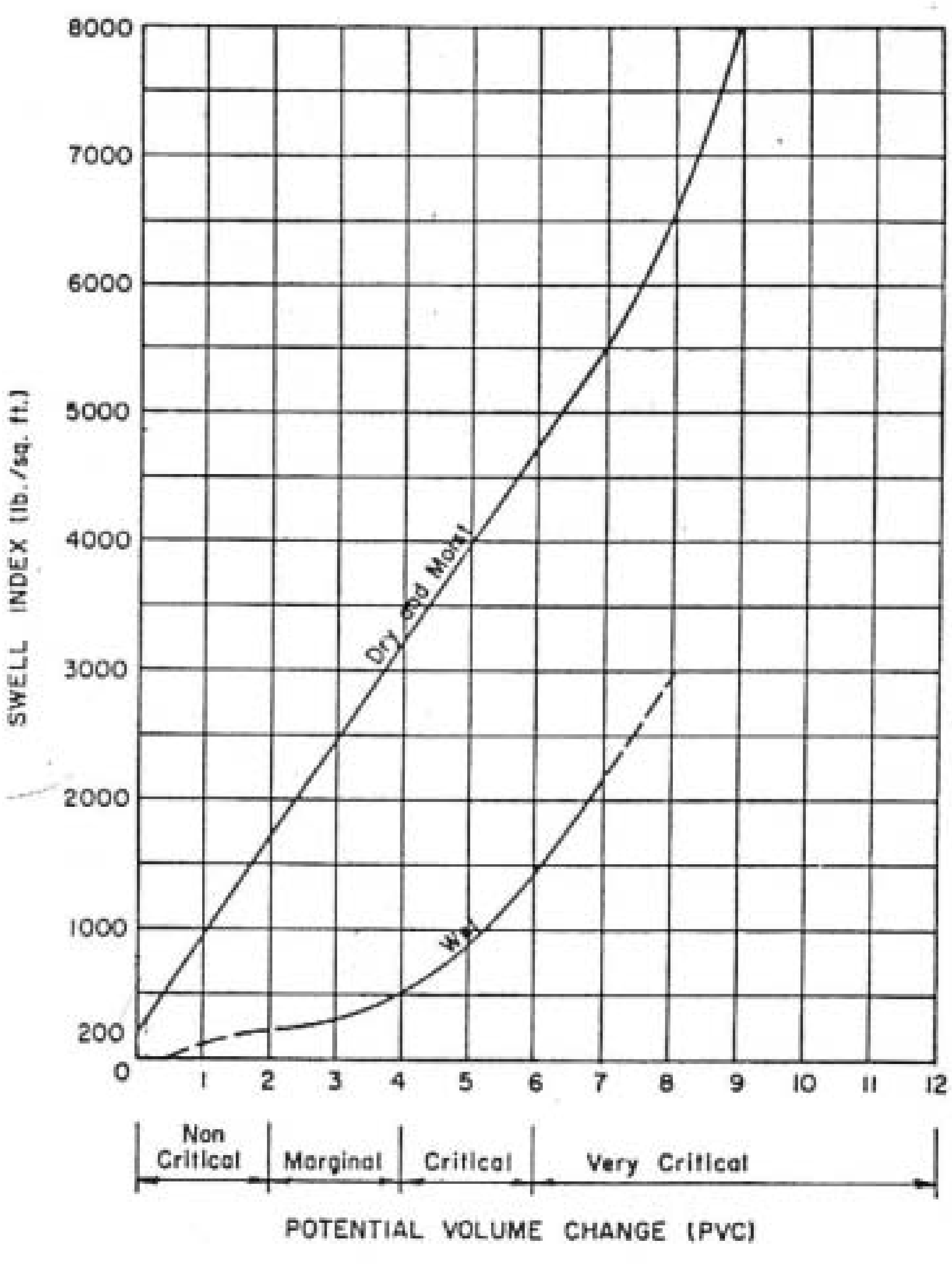


Figure 2.13 Swell imdex vs. Potential Volume Change. (From FHA Soil PVC Meter Publication, Federal Housing Administration Publication no.701, 1960)

2.6 LATERAL SWELLING PRESSURE

The problem of swelling pressures generated by expansive clays has received significant attention over the past two to three decades but most of the previous researchs concentrated on vertical swelling pressure properties and measurement and prediction of vertical swelling pressure. As it is well known clay shows anisotropic behaviour and the lateral swelling pressure may exceed the vertical swelling in many cases.

Any structure, located or burried in expansive clay, may be subjected to large magnitudes of lateral pressure as a result of increase in water content. Damage to the structures caused by lateral swelling pressures is well documented. Kassiff and Zeitlen (1962) discuss field observations of damage to pipelines buried in expansive clays. They found inequalities in the lateral and vertical swelling behaviour resulted in very large stresses in the pipeline. The current research on the subject indicate that the saturated expansive soil behaves unconventionally with respect to development of lateral pressure both at active and at rest conditions (Katti et.al, 1987).

Total lateral pressure, acting to a retaining structure which is built within expansive soil, consist of lateral earth pressure, surcharge pressure and lateral swelling pressure. Figure 2.14 shows a schematic diagram of lateral pressures acting on a sheet pile.

Measurement of swelling pressure against a 7.5 m retaining wall in stiff clay by Richards and Kuzeme (1973) indicated that lateral swell stresses were stabilizing at 1,3 to 5,0 times to the overburden pressures (Fourie, 1988).

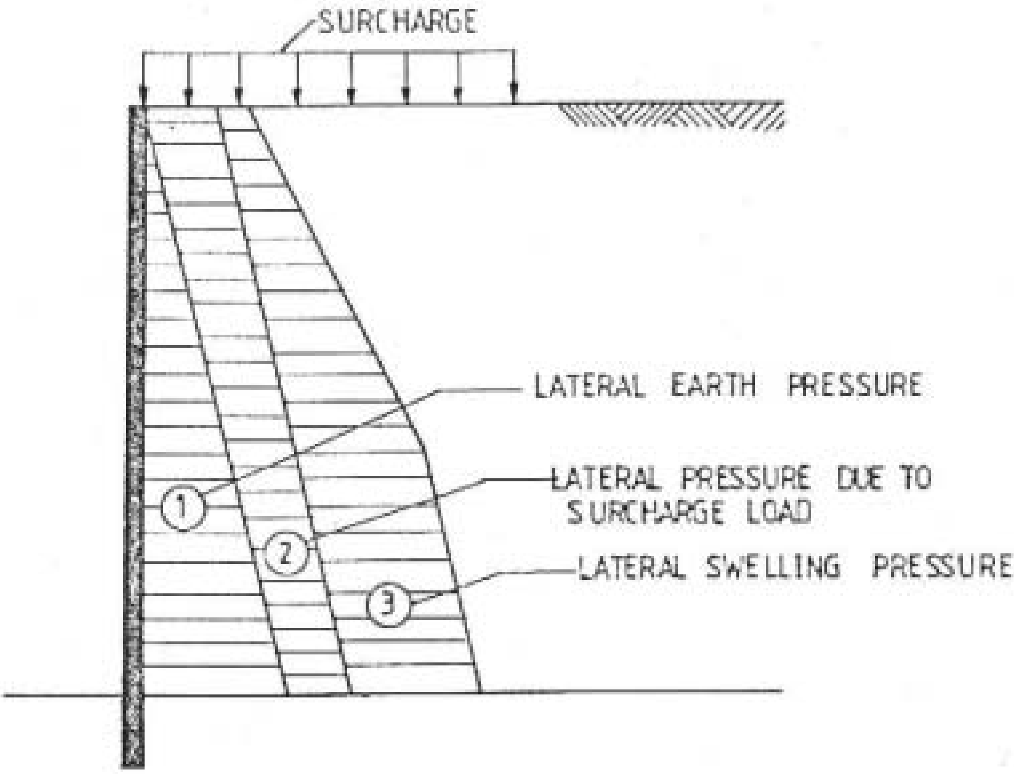


Figure 2.14 Schematic Diagram of Lateral Pressures Acting on a Sheet Pile.
(Ertekin, 1991)

2.6.1 Factors Influencing Lateral Swelling Pressure

Various researches on lateral swell pressure show that initial water content, initial dry density, surcharge load and the depth factor are the main factors that effects the lateral swell pressure.

2.6.1.1 Effect of Initial Water Content

Erol and Ergun (1994) indicated that both of the lateral swell pressure and vertical swell pressures decrease with increasing initial water contents as shown in Figure 2.15 and reveals larger swell pressure ratios which is the ratio of lateral swell pressure to the vertical swell pressure at higher initial water contents as shown in Figure 2.16.

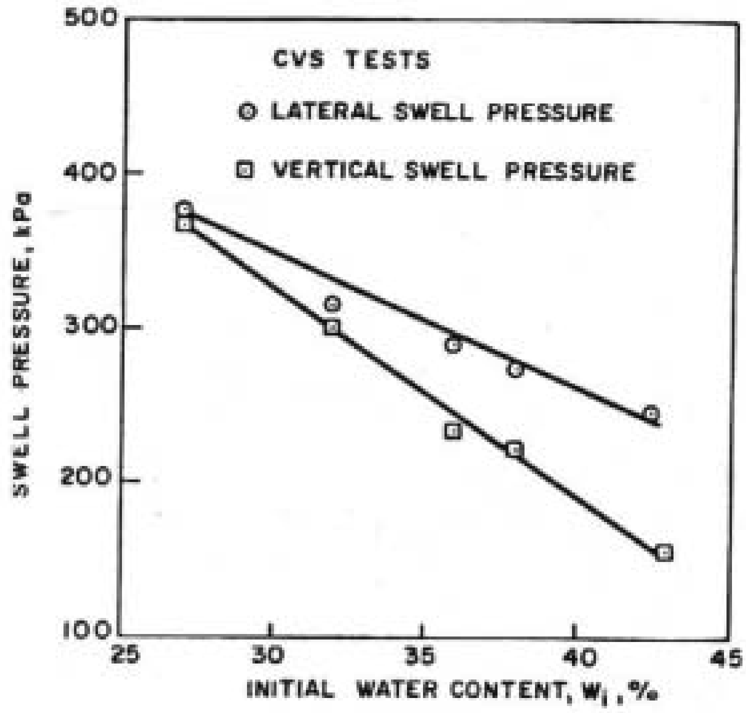


Figure 2.15 Effect of Initial Water Content on Swell Pressures
(After EROL and ERGUN, 1994)

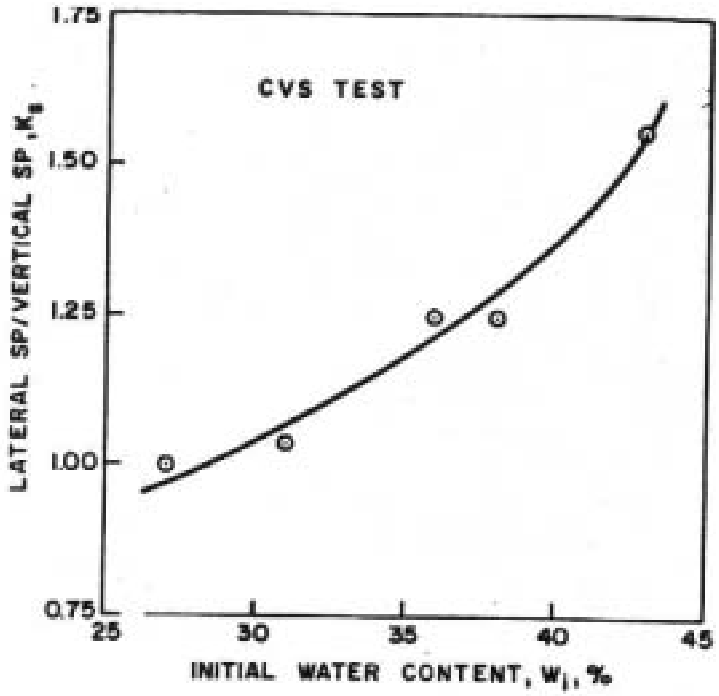


Figure 2.16 Effect of Initial Water Content on Swell Pressure Ratio
(After EROL and ERGUN, 1994)

2.6.1.2 Effect of Initial Dry Density

Chen (1974) and Ofer (1980) stated that the lateral swelling pressure, as happens for vertical swelling pressure, is related to the initial dry density. In a test conducted under a constant vertical surcharge load, the increase of lateral swelling pressure follows the same pattern as the increase of vertical swelling pressure.

2.6.1.3 Effect of Surcharge Load

Joshi and Katti (1984) indicated that saturating the sample under a higher surcharge and then reducing the surcharge to a lower one results in the development of increased lateral pressure as compared to the lateral pressure developed under lower initial surcharge. The constant lateral swelling pressure attained under any initial surcharge load is termed equilibrium lateral swelling pressure. The period for attaining equilibrium lateral swelling pressure depends on the magnitude of initial surcharge load. And equilibrium lateral swelling pressure increases rapidly with increasing initial surcharge load as seen in Figure 2.17 but after some point increase in lateral pressure becomes lesser. Joshi and Katti (1984) also showed that swell ratio decreases with increasing initial surcharge load as shown in Figure 2.18.

2.6.1.4 Effect of Depth Factor

Joshi and Katti (1984) performed large scale model studies to simulate the field conditions. The experiments showed that the lateral pressure near the surface was negligible, but it increased to 3,0 tsf (287 kN/m2), which is around the swelling pressure of soil within a shallow depth of 3,0 ft (0,92 m). With further depth the lateral swelling pressure appeared to remain the same until vertical pressure due to the weight of the soil is equal to the swelling pressure.

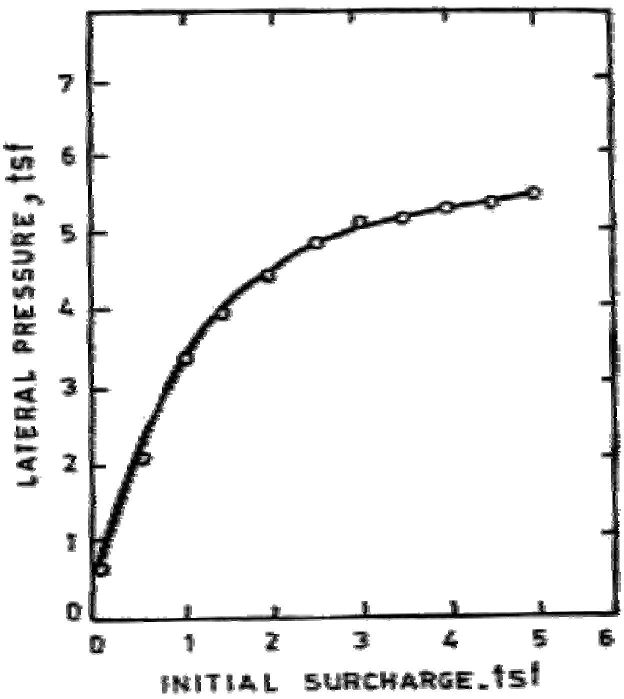


Figure 2.17 Effect of Initial Surcharge on Lateral Swell Pressure
(After Joshi and Katti, 1984)

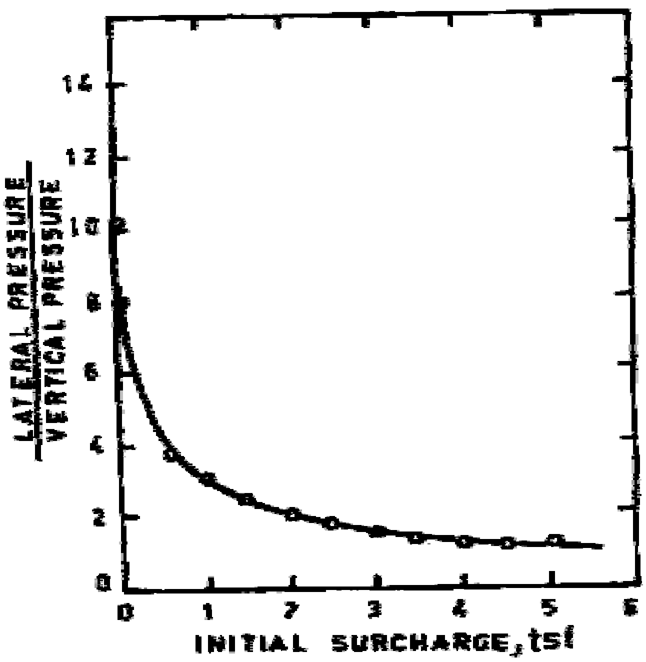


Figure 2.18 Effect of Initial Surcharge on Swell Pressure Ratio
(After Joshi and Katti, 1984)

2.6.2 Measurement of Lateral Swelling Pressure

2.6.2.1 Measurement of Lateral Pressure with Oedometer Ring

Komornik and Zeitlen (1965) modified the usual type of consolidation cell unit to study the lateral swell pressure developed during the saturation of expansive soils. This device consist of a special consolidation ring with a thin wall section in its central portion, allowing the use of electrical strain gages to determine the applied internal pressure. The result of these test are shown in Figure 2.19 in the form of percentage of vertical deformation vs. both vertical and lateral swell pressures.

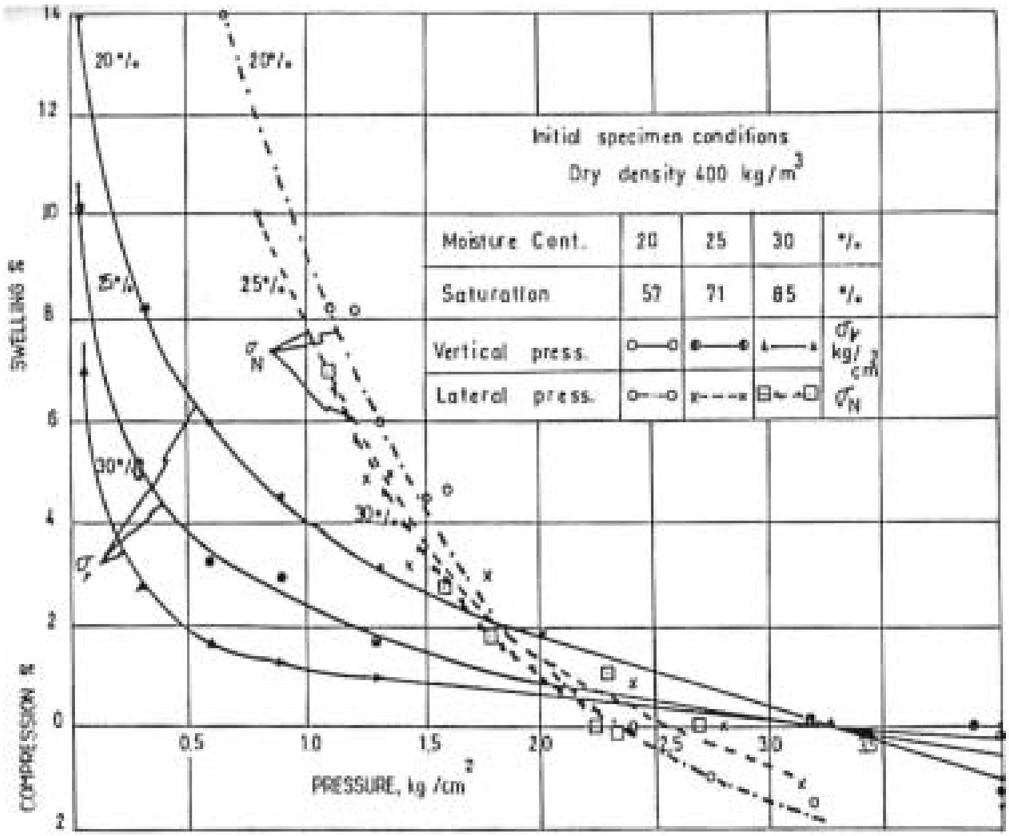


Figure 2.19 Typical Relationship Observed for Vertical Movements as a Function of Vertical and Lateral Pressures (After Komornik and Zeitlen, 1965)

Ofer (1980) developed lateral swelling pressure (LSP) ring which is a modified thin walled oedometer ring instrumented with strain gauges to work on a correlation between the amount of swell and the lateral swelling pressure developed in a clay sample. LSP ring is similar in principle to the apparatus developed by Komornik and Zeitlen (1965) for measuring the lateral soil swelling pressure in the laboratory. The ring is calibrated by clamping it between end plates, introducing air under pressure to the inner part of the ring and recording the corresponding strain with a digital strain indicator and a strain recorder. The effect of long term loading on the ring was investigated and it was found that extended periods of loading had no effect on the calibration of the ring. Tests were carried out under a constant vertical pressure of 19 kPa using compacted clay samples to various initial dry densities. Figure 2.20 shows typical density – per cent swell – lateral swelling pressure relationships obtained from a series of tests.

2.6.2.2 Measurement of Lateral Pressure with Using Modified Hydraulic Triaxial Apparatus

Fourie (1989) states a new laboratory technique that may be used to determine lateral swelling pressures in an expansive clay. The device used in this series of tests was the hydraulic triaxial apparatus, originally described by Bishop and Wesley (1975), which allows stress – controlled loading on both axial and radial pressures. Compacted samples were prepared for testing in the same way as for a conventional triaxial sample, with provision for radial as well as top and bottom drainage. A lateral strain belt of the type described by Bishop and Henkel (1962) was then assembled around the sample. After filling and pressurising the triaxial cell, any desired ratio of vertical to horizontal total stress could be imposed on the sample prior to wetting up. Water was introduced to the soil sample simultaneously through the top and bottom loading plates and hence through the radial drains.

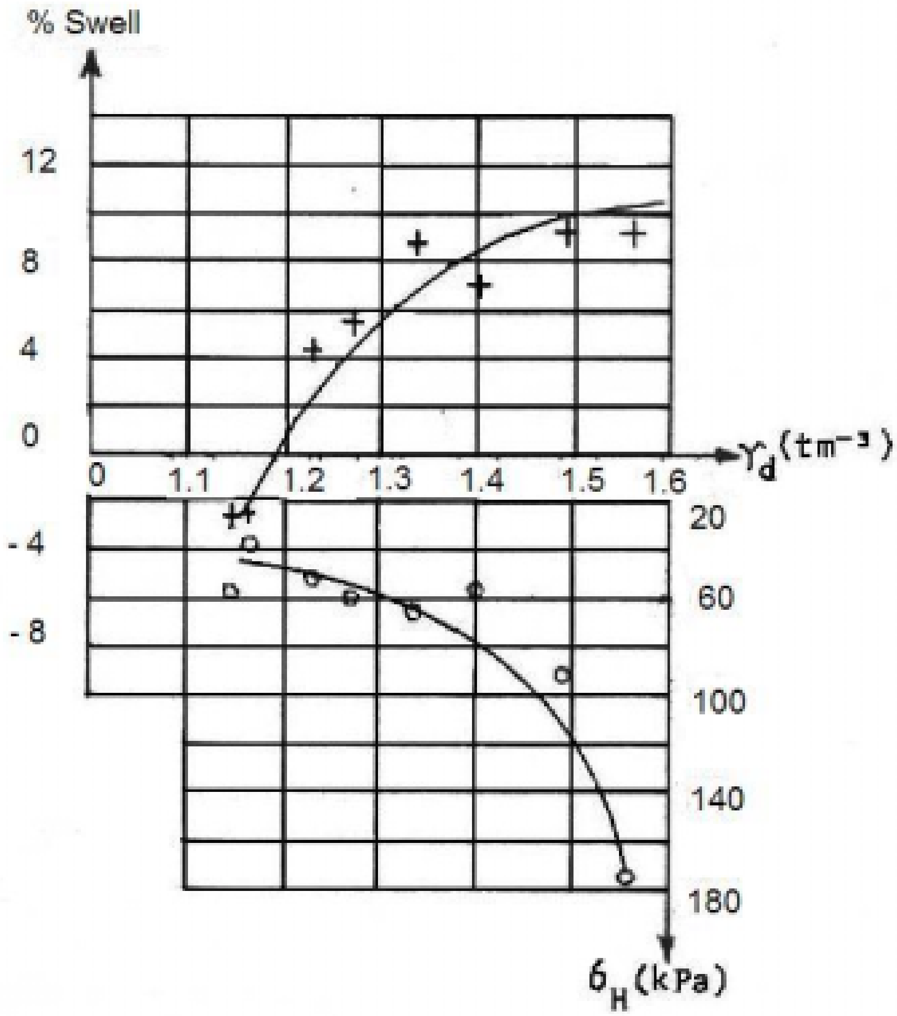


Figure 2.20 Density – percent Swell – Lateral Swelling Pressure Relation for the LSP Ring Test under σ_v = 19 kPa

The technique originally adopted of continuously increasing the cell pressure (and thus the horizontal total stress), if any increase in diameter was detected by the lateral strain belt upon ingress of water to the sample proved unsatisfactory. It was found to be impossible to avoid overcompensanting and thus compressing the sample beyond its original diameter (Fourie, 1989). The results obtained using this technique are shown in Figure 2.21.

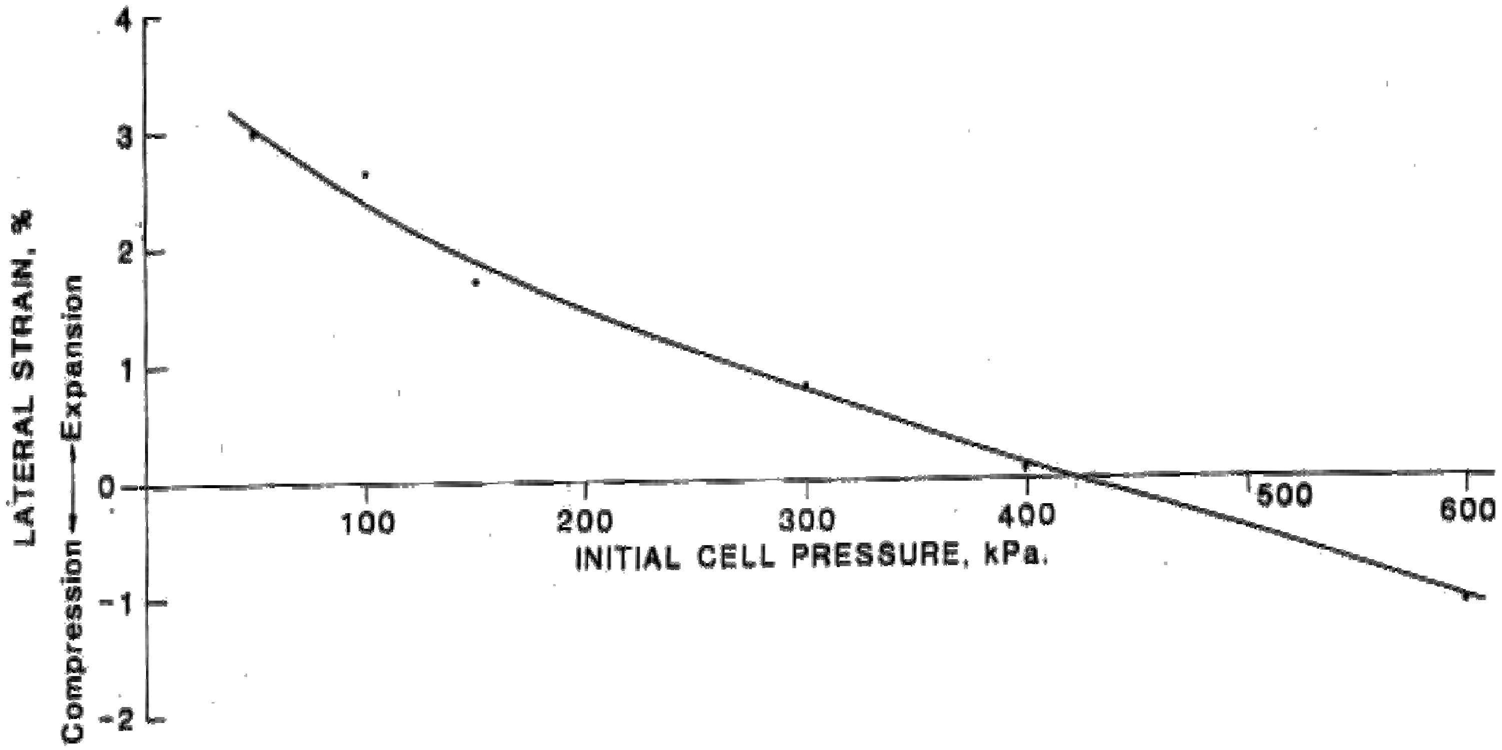


Figure 2.21 Variation of Final Measured Lateral Strain with Applied Initial Cell Pressure (After Fourie, 1989)

The vertical axis is the ultimate lateral strain (either expansion or compression) measured by the lateral strain belt. The point at which the curve intersects the zero lateral strain (i.e., the horizontal axis) is then interpreted as being the lateral swelling pressure developed under conditions of zero lateral strain. An advantage of using a hydraulic triaxial cell instead of a conventional cell is that a constant vertical stress similar to field conditions may be imposed on the soil sample. In addition lateral strains are measured directly, thus circumventing errors that may occur in deriving lateral strains from measured volumetric and axial strains (Fourie, 1989).

2.6.2.3 Large Scale Model Studies

Joshi and Katti (1984) carried large scale model studies to simulate the field conditions. Figure 2.22 shows tank and reaction frame of the large scale equipment used in this study.

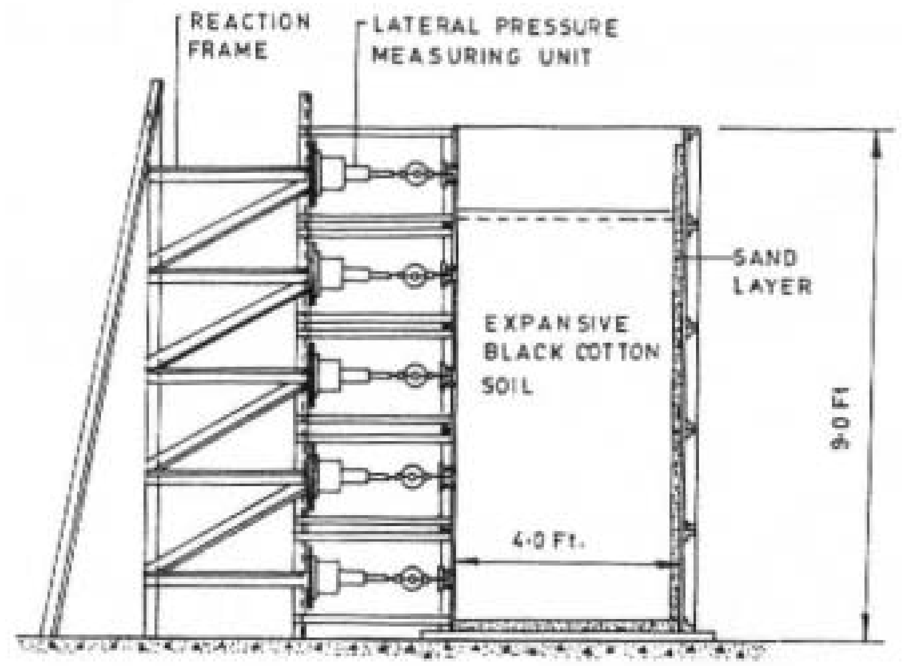


Figure 2.22 Tank and Reaction Frame of Large Scale Equipment (1 ft = 0.305 m)

The tank shown in Figure 2.22 made up of stiffened mild stell plates has internal dimensions of 0,92 m x 1,22 m in plan and 2,75 m in depth. The test set up has lateral pressure measuring units at various depths. The processed soil was compacted in 14 layers of 25.4 mm thickness at a void ratio of 1,0 in the container tank and the required surcharge was applied through the lever system. The soil was then allowed to saturate by providing water through the bottom sand layer and the side perforated perspex sheets. The developed lateral pressure and the vertical movement were recorded at regular time intervals till the soil – moisture eqiilibrium was established. Then the surcharge was gradually reduced in steps allowing sufficient time interval for the system to reach equilibrium under each reduced surcharge to obtain the rebound characteristics. The investigation regarding the development of lateral swelling pressure under dead load surcharges indicated certain general trends. In all cases the lateral swelling pressure increased rapidly with the time in the beginning of saturation process. Then the rate of increase slowed down and the lateral swelling pressure attained a peak value. With further increase in time lateral swelling pressure decreased to some extend and then remained constant (Joshi and Katti, 1984). A typical curve indicating the development of lateral swelling pressure in time is shown in Figure 2.23.

2.6.2.4 The In – Situ Lateral Swelling Pressure Measurements

2.6.2.4.a ISP Probe

Ofer (1980) stated an apparatus called in – situ lateral swelling pressure (ISP) probe to monitor the in – situ lateral swelling pressure developing during wetting of an expansive clay. This apparatus consists of pressure transducers, wetting rings located above and below the pressure transducers, and a cutting edge. A diagram of the in – situ swelling probe is shown in Figure 2.24.

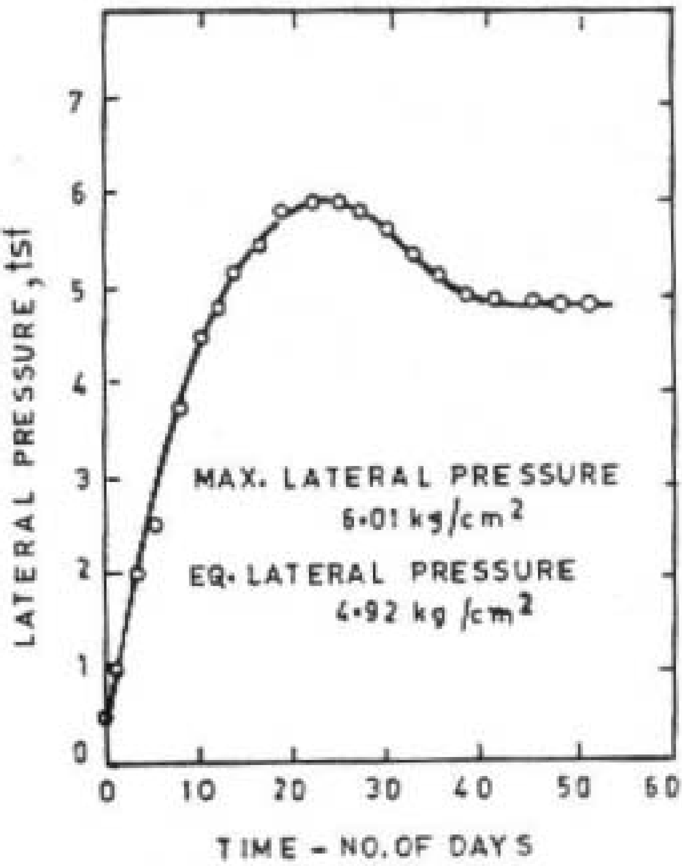


Figure 2.23 Development of Lateral Pressure with Time
(After Joshi and Katti, 1984)

The module is calibrated in an airtight cylinder which surrounds the module. Air pressure is introduced into the outer cylinder and the strains of the thin wall are recorded. For the in – situ tests the probe is placed in a predrilled hole, water is introduced into the surrounding clay and the resultant lateral swelling pressure is recorded. The ISP probe results shown in Figure 2.25 and Figure 2.26.

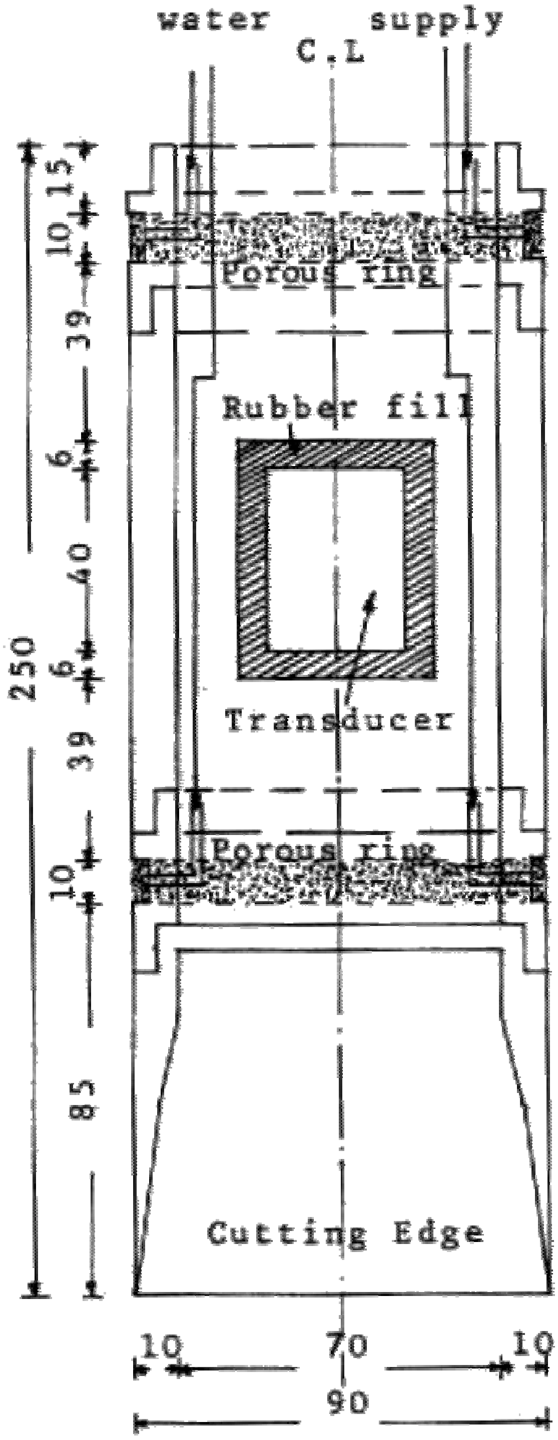


Figure 2.24 In – Situ Swelling Pressure Probe (After Ofer, 1980)

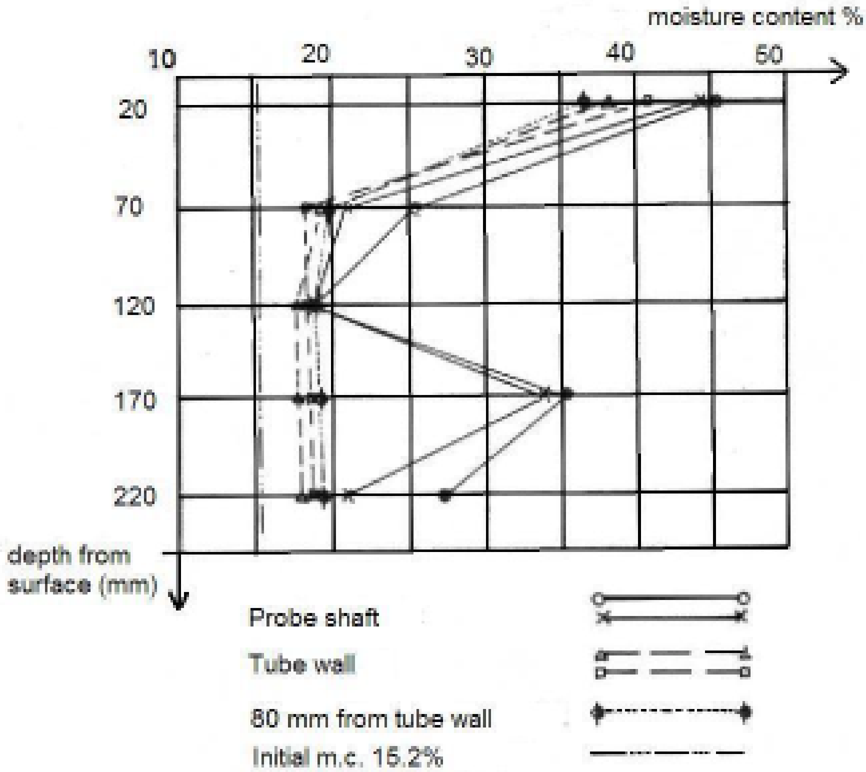


Figure 2.25 Moisture Content Profile at the end of the ISP Probe test (After Ofer, 1980)

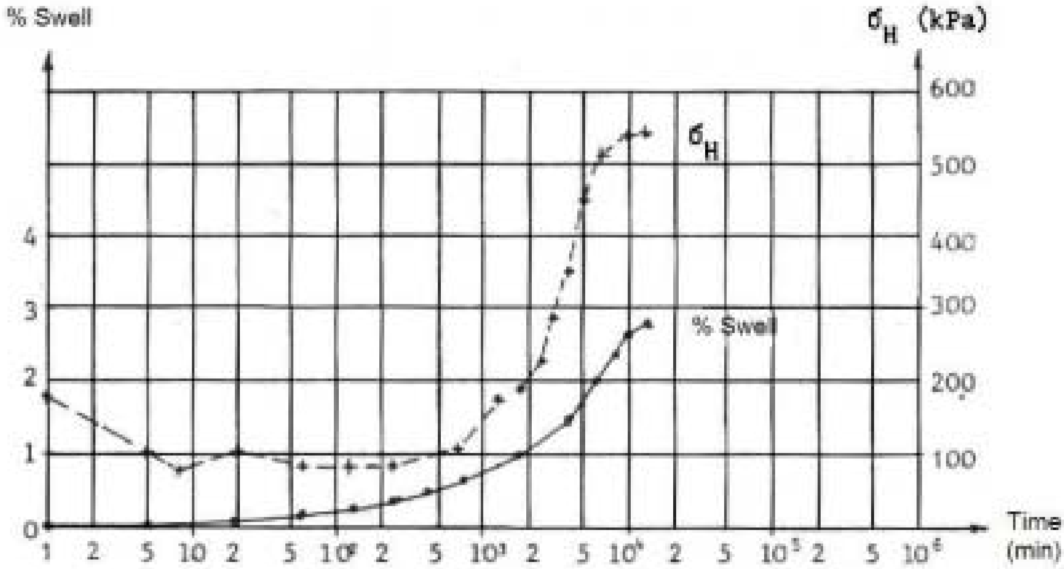


Figure 2.26 Time - % swell – Lateral Swelling Pressure Relationship fir the ISP Probe Test (After Ofer, 1980)

2.6.2.4.b Total Pressure Cell

Robertson and Wagener (1975) described total pressure cell for the in – situ measurement of both vertical and lateral swelling pressures. Two test pits were formed as illustrated in Figure 2.27.

The pits were formed in natural soil of medium dense and dense consistency. A gravel drainage layer was placed at the base of the pit from which water could be drained and the entire pit was lined with p.v.c. sheeting to control the entry of ground water. Clay was air dried in a shed, broken using hand stampers into pea size nodules, mixed with water to the optimum moisture content and compacted in 50 mm layers. Approximately 5 mm thick sand layers were placed between the consequtive clay layers. Terra Technology T-9010 pneumatic total pressure cells were installed at a depth of 1,0 m below the final clay pit surface. Figure 2.28 shows a general view of the pit at the time of cell installation and in Figure 2.29 two partially imbedded vertical cells can be seen. The shallow excavation made in the clay surface shown in Figure 2.29 is in preparation for the installation of a horizontally orientated cell. The clay pits were, at all times, covered to prevent rain water from entering. Water was removed from the drainage layer at the base of the pit to maintain a low ground water level and, with the p.v.c. lining, to control ground water entry into the clay pit. Auger holes are used to intersect the sand layers for water intake. Poor wetting occured in clay pit1 due to the trapped air in drainage paths. Therefore the wetting process is revised for clay pit2 and resulted in a successfull wetting process. Total pressure cell readings were taken at regular intervals from the time of installation. The variation with time of the average pressure readings from the three vertical cells and the horizontal cell in each pit is shown in Figure 2.30 and in Figure 2.31. After removal from the pits all the pressure cells effectively returned to the zero datum and it was concluded that the cell had functioned satisfactorily.

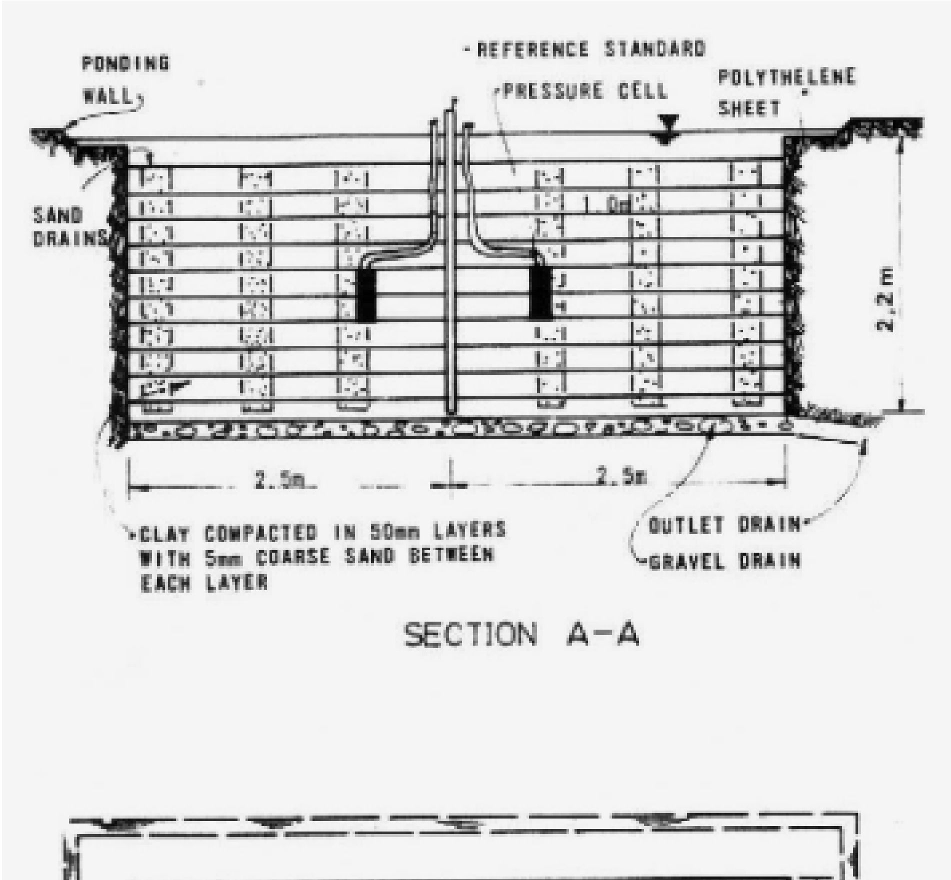


Figure 2.27 Test Pits for Total Pressure Cell Installation

(Robertson and Wagener, 1975)

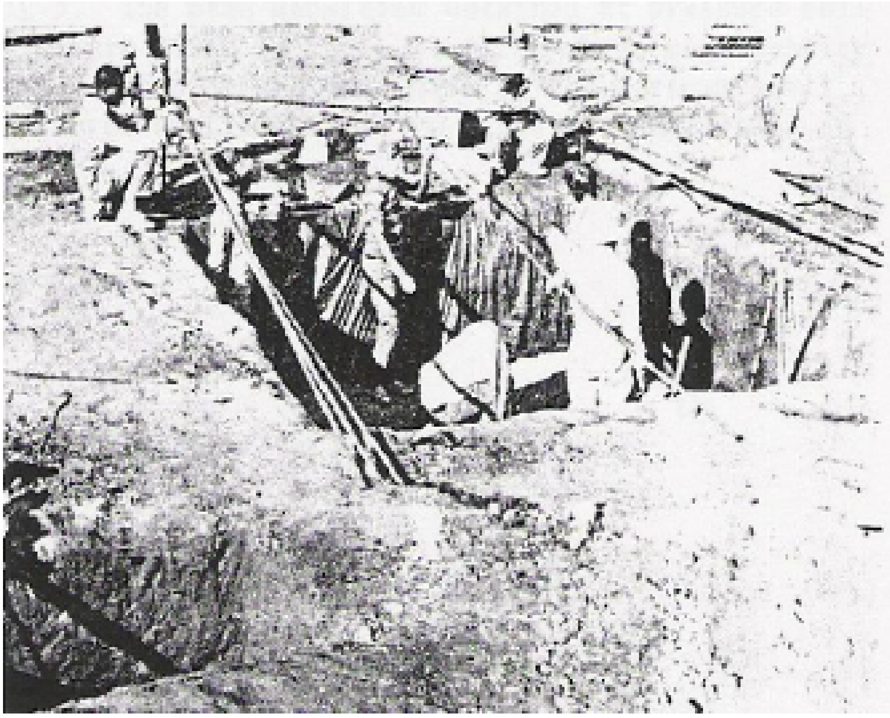


Figure 2.28 Total Pressure Cell Installation (Robertson and Wagener, 1975)

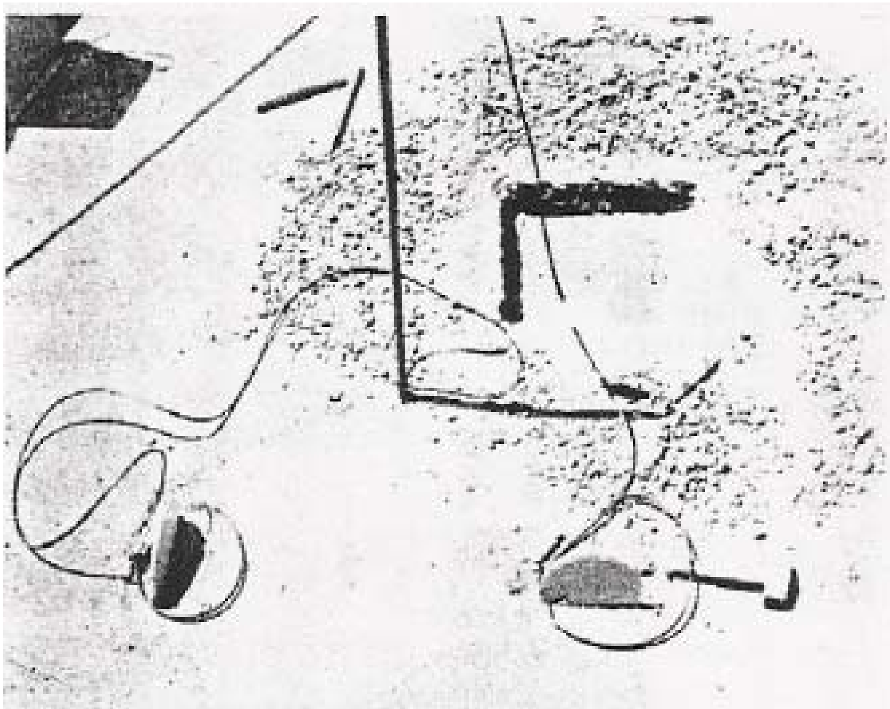


Figure 2.29 Vertical and Horizontal Total Pressure Cell Installation
(Robertson and Wagener, 1975)

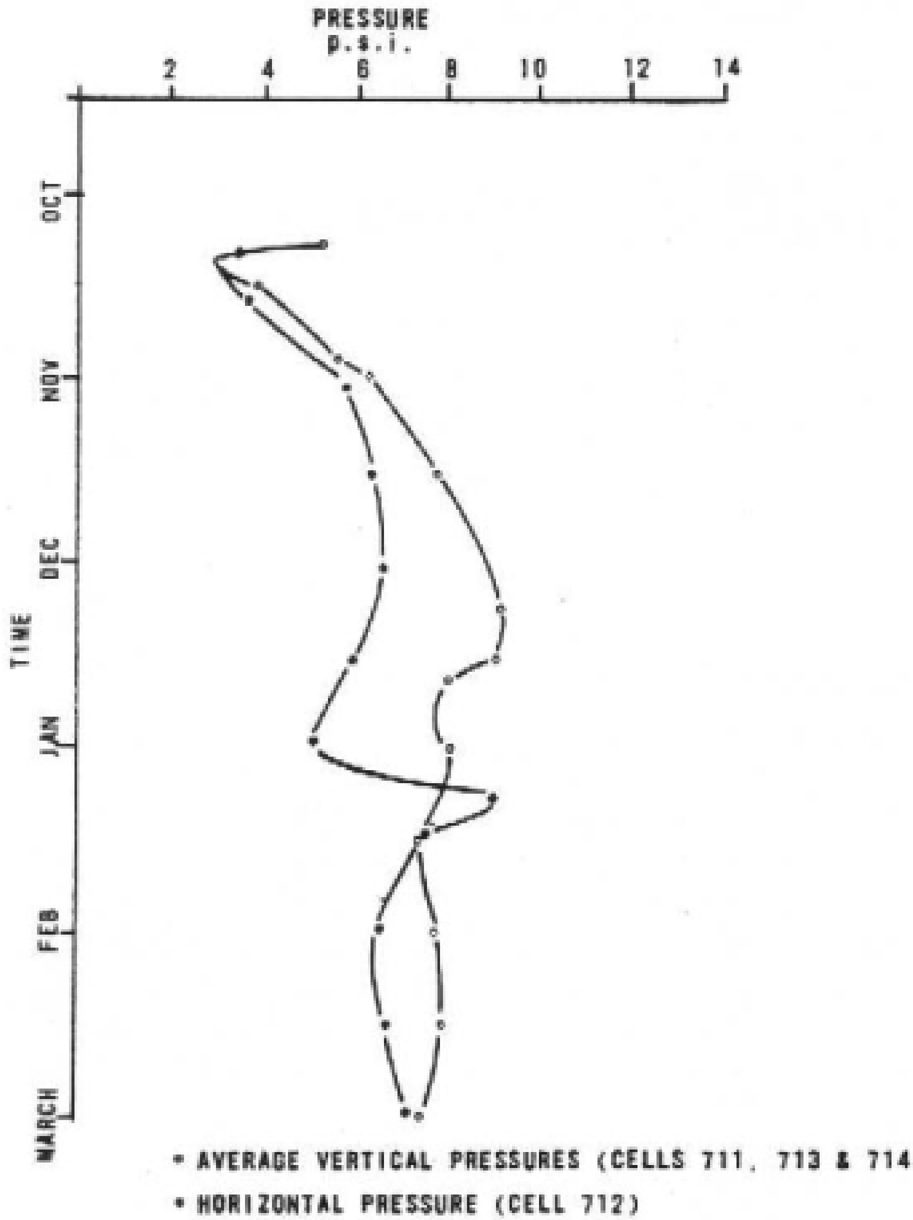


Figure 2.30 Pressure Measurement in Clay Pit 1 (Robertson and Wagener, 1975)

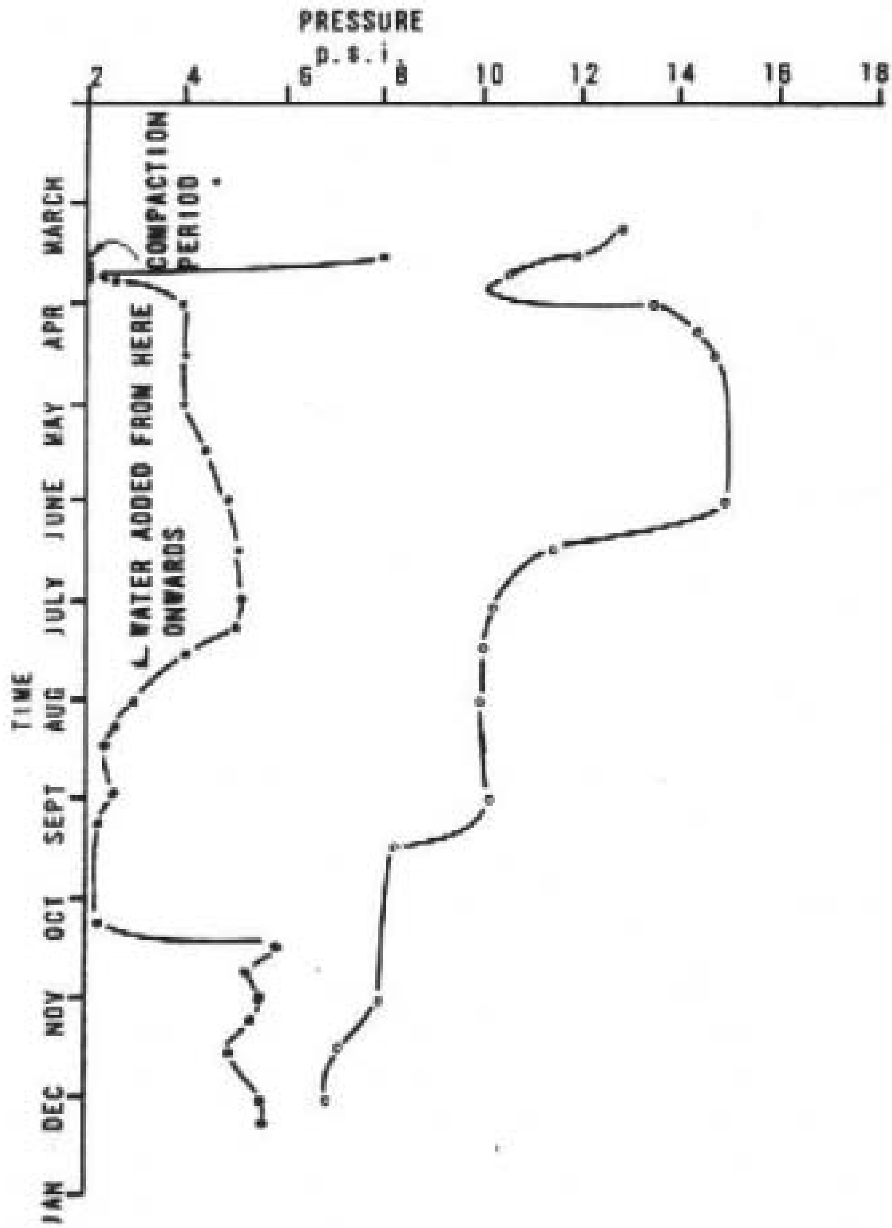


Figure 2.31 Pressure Measurement in Clay Pit 2 (Robertson and Wagener, 1975)

CHAPTER 3

DEVELOPMENT OF A MODIFIED THIN WALL OEDOMETER RING FOR LATERAL SWELL PRESSURE MEASUREMENT

3.1 INTRODUCTION

Prediction of the swell parameters can be done by using conventional laboratory oedometer tests, PVC meter, soil suction method and by using empirical correlations in the literature as previously mentioned in Section 2.5 and in Section 2.4.1.2 respectively. And the lateral swelling pressure is measured by using methods mentioned in Section 2.6.2. However, all of the laboratory and empirical techniques to measure both vertical and lateral swelling pressures do not reflect the actual in – situ conditions, and exaggerated results can be obtained. Since the lateral swelling pressure and the correlation between the lateral and vertical swelling pressures are important design parameters for structures located or buried in expansive soils, the laboratory measurement technique should have reasonable accuracy to measure lateral swelling pressure. This accuracy may be obtained by using lateral swelling pressure ring which is a modified thin wall oedometer ring instrumented with proper electrical strain gauges.

Komornik and Zeitlen (1965) described a modified oedometer ring instrumented with electrical strain wires around its central thin wall section to measure the applied lateral swelling pressure. Ofer (1980) modified this oedometer ring with electrical strain gauges configuration attached to the oedometer ring at the mid – height of thin wall section. Modified thin wall oedometer used in this study is developed by Ertekin (1991).

Main problem in measuring the lateral swelling pressure is to obtain the highest level of lateral pressure applied by the soil sample to the walls of the test instrument. It is only possible in the case of no lateral movement during swelling. For this aim the ring body should behave as a thin steel membrane which acts as a lateral restraint and also allows the strain control with back pressure. Ertekin (1991) modified an oedometer ring to achieve a precise lateral swell pressure measurement as explained in the next sections.

3.2 PROPOSED LATERAL SWELL PRESSURE TEST SET – UP

Lateral swell pressure test set – up mainly consists of:

- Thin wall oedometer ring body
- Casing
- Closing top and bottom caps (for use during calibration only)
- Inundation container
- Assembling pieces (for attachment, mid – contact and loading)
- Read – out unit (strain indicator)

3.2.1 Thin Wall Lateral Swell Pressure Oedometer Ring Body

A cross sectional diagram of lateral swell pressure oedometer ring is shown in Figure 3.1. The main ring is made of high quality alloy steel. Ring material is provided from the products of MKE (Makina Kimya Endüstrisi Kurumu). Material code Ç 4140 which is equal to DIN 42 Cr Mo 4. The internal diameter of the ring is 63,5 mm. The wall thickness and the height of the ring are 0,35 mm and 78 mm respectively. The outer diameter is 140,2 mm. The thin wall oedometer ring machined at the lathe in one operation to obtain a uniform wall thickness. The oedometer has four threaded holes drilled through the top collar of the ring for cable and strain gauge installations staggered as 90 degrees apart. And three holes staggered as 120 degrees for installation of pivot bolts which protect the thin wall of the oedometer against any shock.

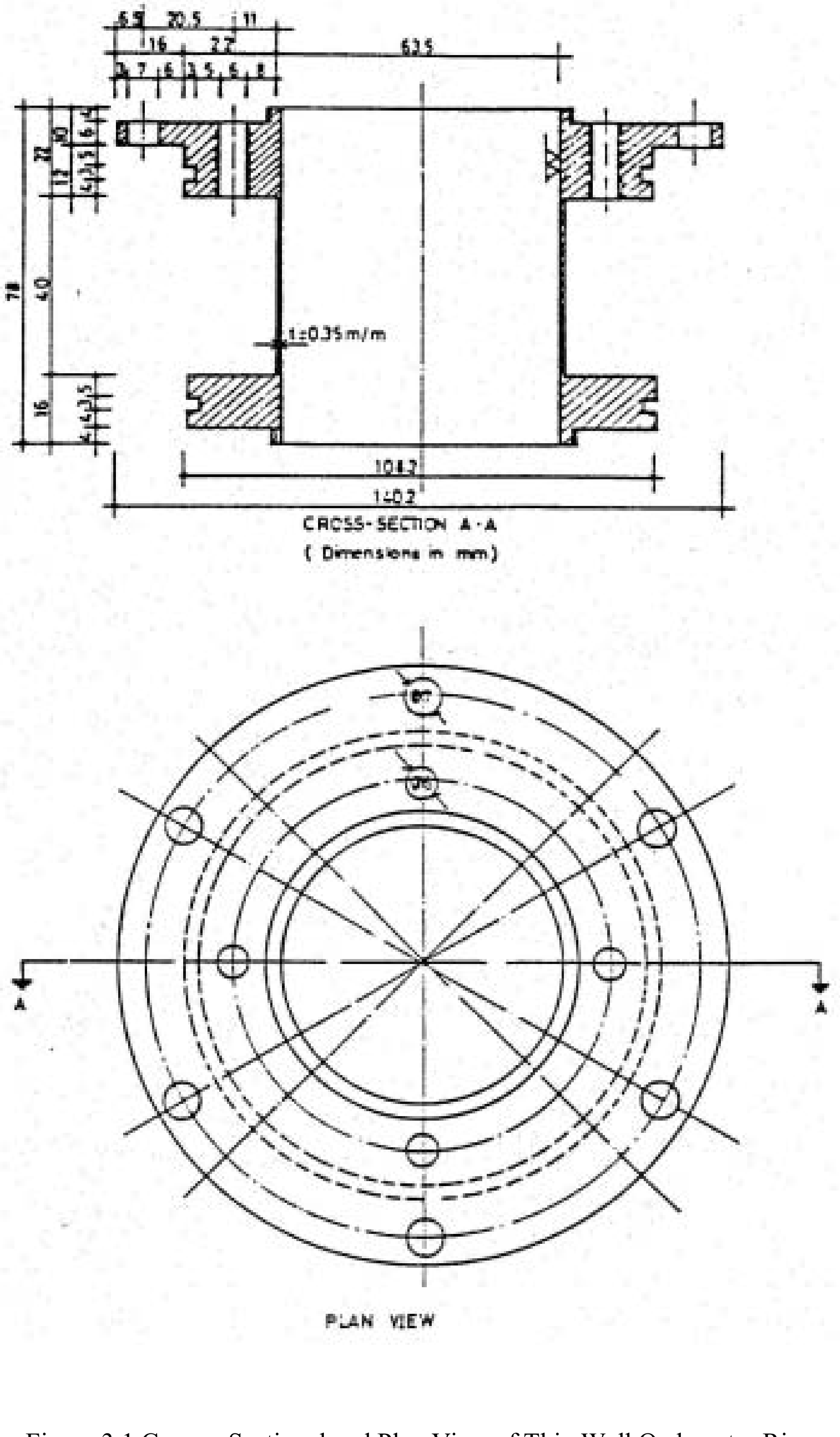


Figure 3.1 Cross – Sectional and Plan View of Thin Wall Oedometer Ring
(Ertekin 1991)

At the outer ends of the ring body there are screw – threads machined to hold the caps fluid tight in calibration. The caps have been machined to fit the two ends of the ring. They have inner o – ring groovs to provide fluid tightness when screwed on the ring. The top cap has two screw – threaded outlet for the installation of fluid supply and release valves for calibration purposes. In order to eliminate the internal stresses due to machining the ring body, it has been exposed to the head treatment and hardening process to avoid permenant deformations after release of the load while testing.No treatment have been applied to the other parts since they have no effect to stress – strain process. Finally, the test set – up has been galvanized for protection against rust. A cross sectional diagram of the complete test set – up is shown in Figure 3.2.

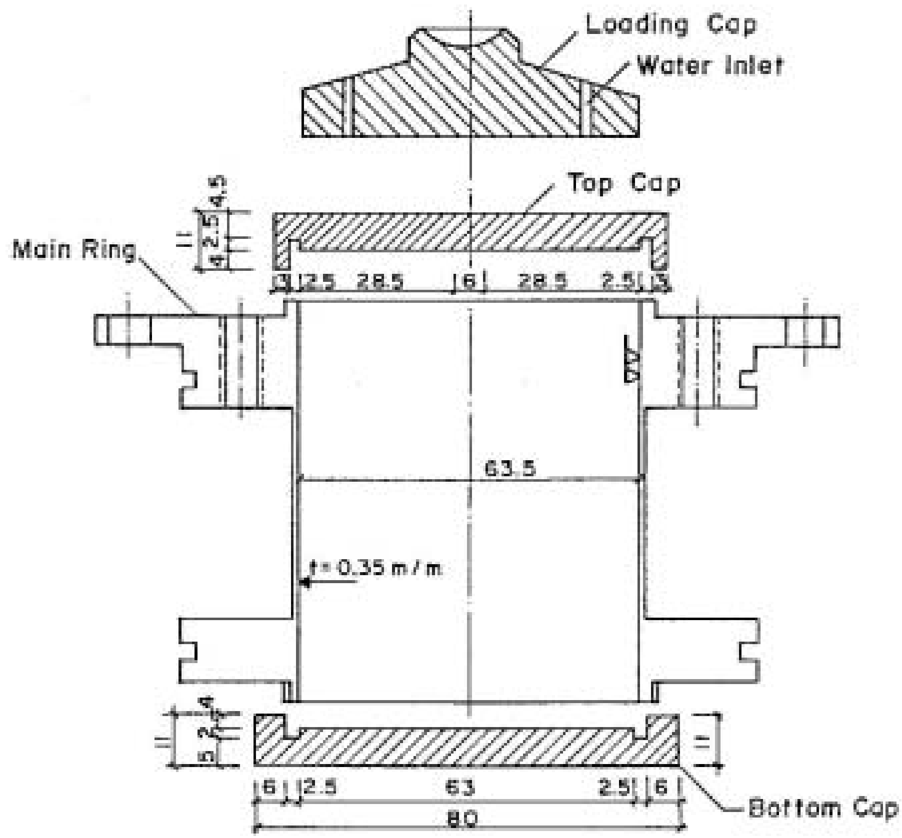


Figure 3.2 Lateral Swell Pressure Test-Set Up Ready for Instrumentation (Ertekin 1991)

3.2.2 Instrumentation of Lateral Swell Pressure Oedometer Ring with Electrical Strain Gauges

Stress in a material can not be measured directly, stress should be computed from other measurable parameters such as strain. With using measured strains in conjuction with other material properties the stresses for a given loading condition can be calculated. Strain can be measured with various methods such as mechanical, optical, acoustical, pneumatic and electrical methods. In this modified thin wall oedometer test set – up electrical strain gauges are used for lateral swelling pressure measurement.

Electrical strain gauges are the most important instruments which installed to the outer surface of the thin wall of the oedometer ring. Four 120 ohms electrical strain gauges are installed to the mid – height of the thin wall 90 degrees staggered apart. Figure 3.3 shows the installation of the strain gauges on the thin wall surface.

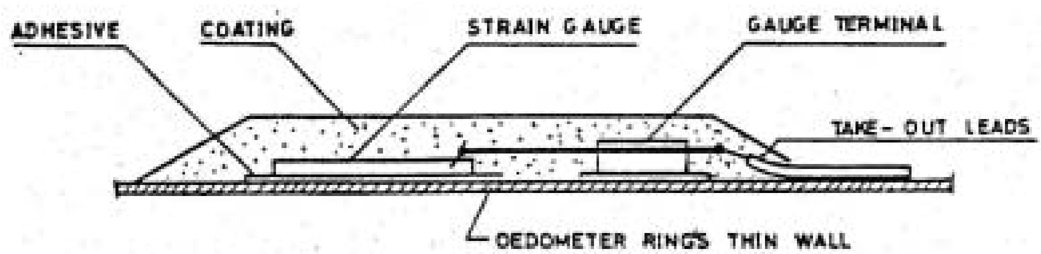


Figure 3.3 Installation of the Strain Gauges on the Thin Wall Surface

Fatigue life of the used strain gauges is 10^6 load – unload cycles. Cu – Ni alloy foil is the sensing element of the strain gauge which is fixed into epoxy carrier. Operational temperature range is between –20 to $+80^0$ C. Strain limit of this gauge type and gauge factor are 3% maximum and 2,1 respectively and the gauge length is 10 mm. Figure 3.4 shows a foil bonded resistance strain gauge and its nomenclature.

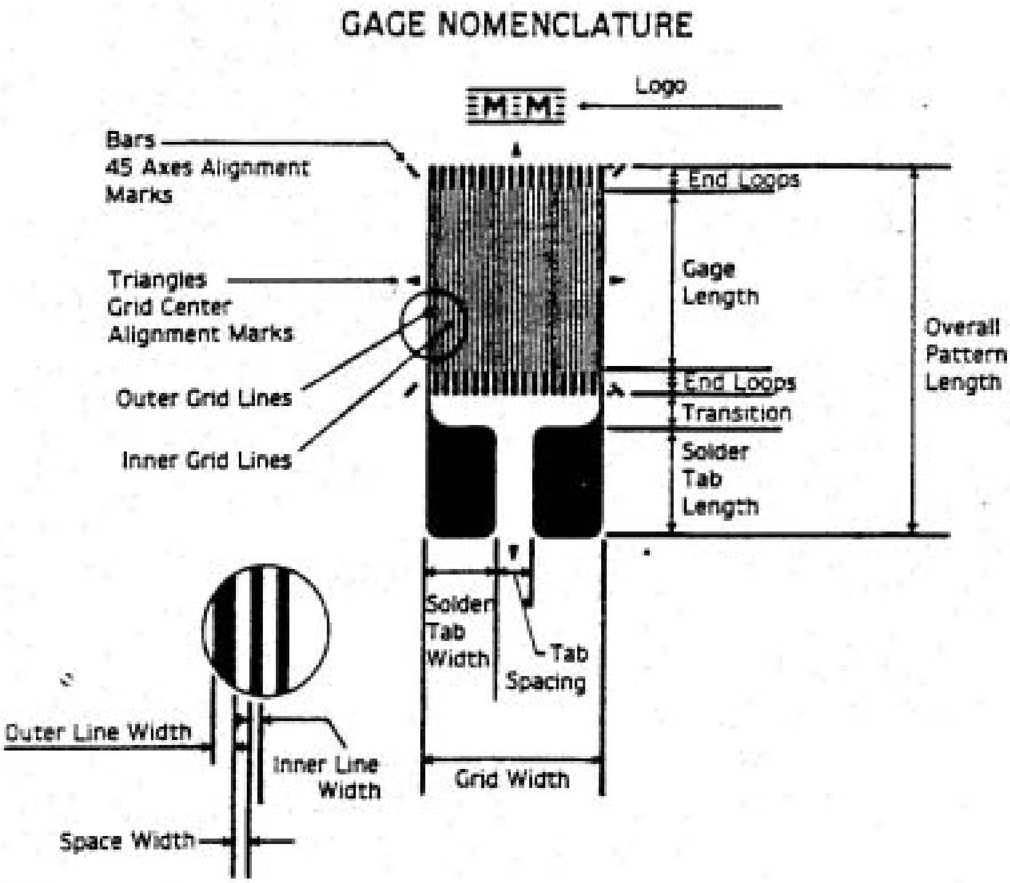


Figure 3.4 Strain Gauge Nomenclature

Electrical installation of the electrical strain gauges to the outer wall of the thin wall is so made to obtain Full Wheaston Bridge Configuration. One couple of the strain gauges installed parallel to the base, symmetrically placed, are called the active strain gauges while the other couple of the strain gauges installed perpendicular to the base, symmetrically placed, are called the dummy strain gauges which provide temperature compensation. Figure 3.5 shows the configuration of the Full Wheaston Bridge. And Figure 3.6 shows one of the active strain gauges on the thin wall oedometer ring.

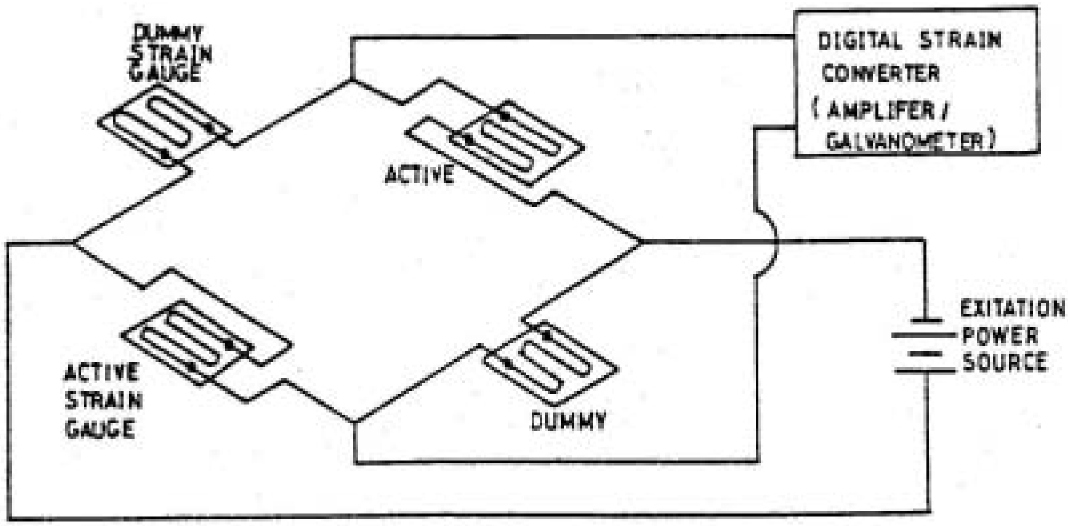


Figure 3.5 Full Wheaston Bridge Configuration

Figure 3.6 An Active Strain Gauge on the Thin Wall Oedometer

3.2.3 Read – Out Unit (The Strain Indicator)

The read – out unit is a needle market type nondigital B105 portable model of Automation Industries B.V Pekel Division and Holland made. It is a simple galvonometer equipped with an amplifier and a converter of electrical current intensity to microstrains. Five different strain measurements can be done at the same time with using its five channels. Sensitivity of the read – out unit is between 50 – 5000 and gauge factor is between 1,75 – 2,25. These factors changes with respect to the type of the strain gauges used. The read – out unit displays micro strains in the range of 000,00 to 999,99 having a sensitivy of 1,83 kPa which is surely efficient to read expected test measurements. A 220V adaptor is attached to the Read – out unit, which originally works with batteries, to provide more healthier voltage regulation and magnetic compensation from that the electrical strain gauges can easily be affected. Read – out unit with its adaptor is shown on Figure 3.7.

Figure 3.7 Read – Out Unit with Adaptor

This read – out unit and modified thin wall oedometer system simply works as follows to read the strains on the thin wall;

- When the test set – up is ready, before the inundation and pressure is applied, the reading needle is centered using three adjustment knobs on the read – out unit, and with the fourth knob the neddle set exactly to zero. And zero reading is taken from the numbers placed on the each knob.
- Inundation phase started with adding water to the system and load is applied slowly.
- When it is time to read, needle is again centered with using the knobs and reading is recorded.
- The micro strain occured on the thin wall due to the changes on the load system is the difference between the second and the first readings.
- The stress applied by the soil sample to the thin wall resulted from the load changes on the system for this stage of the test can be computed with using the micro strain recorded for this stage as explained above and the calibration data of the thin wall oedometer.

3.3 CALIBRATION OF THE LATERAL SWELL PRESSURE OEDOMETER RING

Lateral swelling pressure applied to the thin wall of the oedometer can only be computed from the strain amount recorded using read – out unit as explained above and the calibration data of the strain gauges. For this aim ‘Free Expansion Method’ is used. In this method a known value of fluid pressure is applied to the inside of the oedometer, it is best achieved with water, and since the water pressure is equal at all directions, the magnitude of the strain measured by the strain gauges corresponds to the known value of that lateral pressure.

For the calibration top and bottom of the oedometer ring are plugged tightly by the caps shown in Figure 3.8. The o – rings are placed on each cap to provide full water tightness. Water supply and release is done with using two exit points on the top cap as seen on Figure 3.8.

Figure 3.8 Bottom Cap (at the left) and Top Cap with supply (middle one) and release points

Calibration of the system is done by using the triaxial water pressure system as seen on Figure 3.9 (and calibration set – up is also seen). When the system is ready, the water is allowed to flow inside the oedometer ring from the supply point on top ring and after some water exits from release point, it means there is no air on the closed thin wall system, release valve is closed. Then water pressure is applied to the system, each strain corresponds to the each pressure increment is recorded. The water pressure is increased up to 500 kPa (5 kg/cm^2) , which is surely enough for the test conditions. And after reaching to the maximum pressure point, the reverse procedure is applied by decreasing water pressure gradually and recording again corresponding strains. In order to obtain an accurate calibration chart the load – unload cycle is repeated for four times, and for all times some water is released from release valve to be sure that there is no air bulbes occured inside of the thin wall. All calibration curves are shown in Figure 3.10. And the best fit of all of them and the calibration equation curve are shown in Figure 3.11.

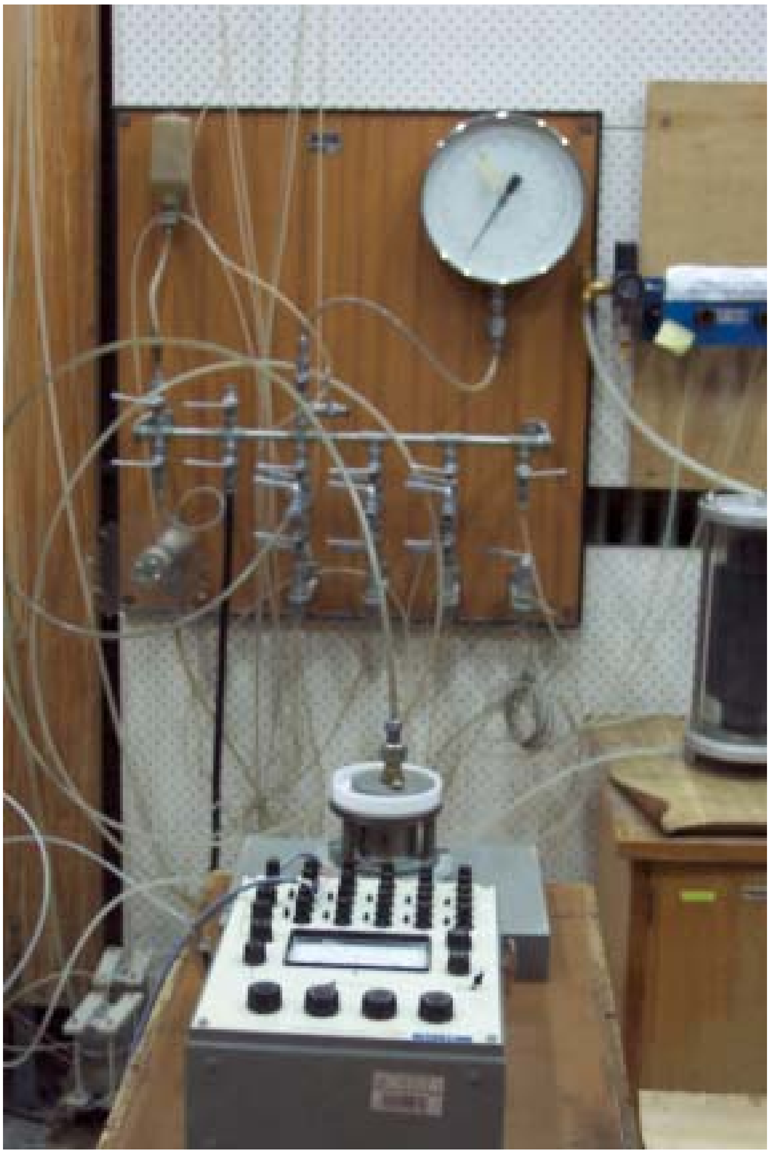


Figure 3.9 Triaxial Water Pressure System and Calibration Set - Up

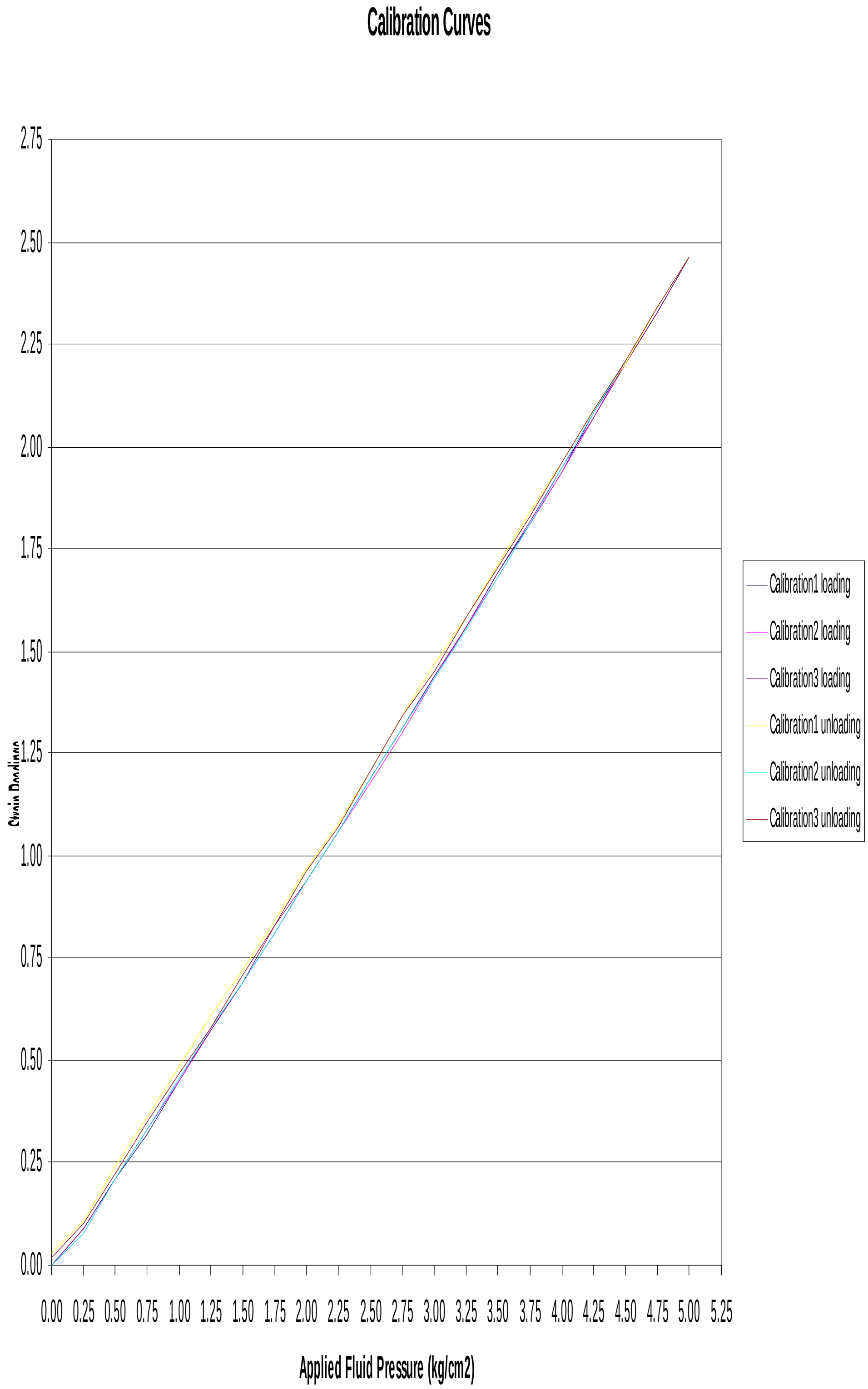


Figure 3.10 Calibration Curves

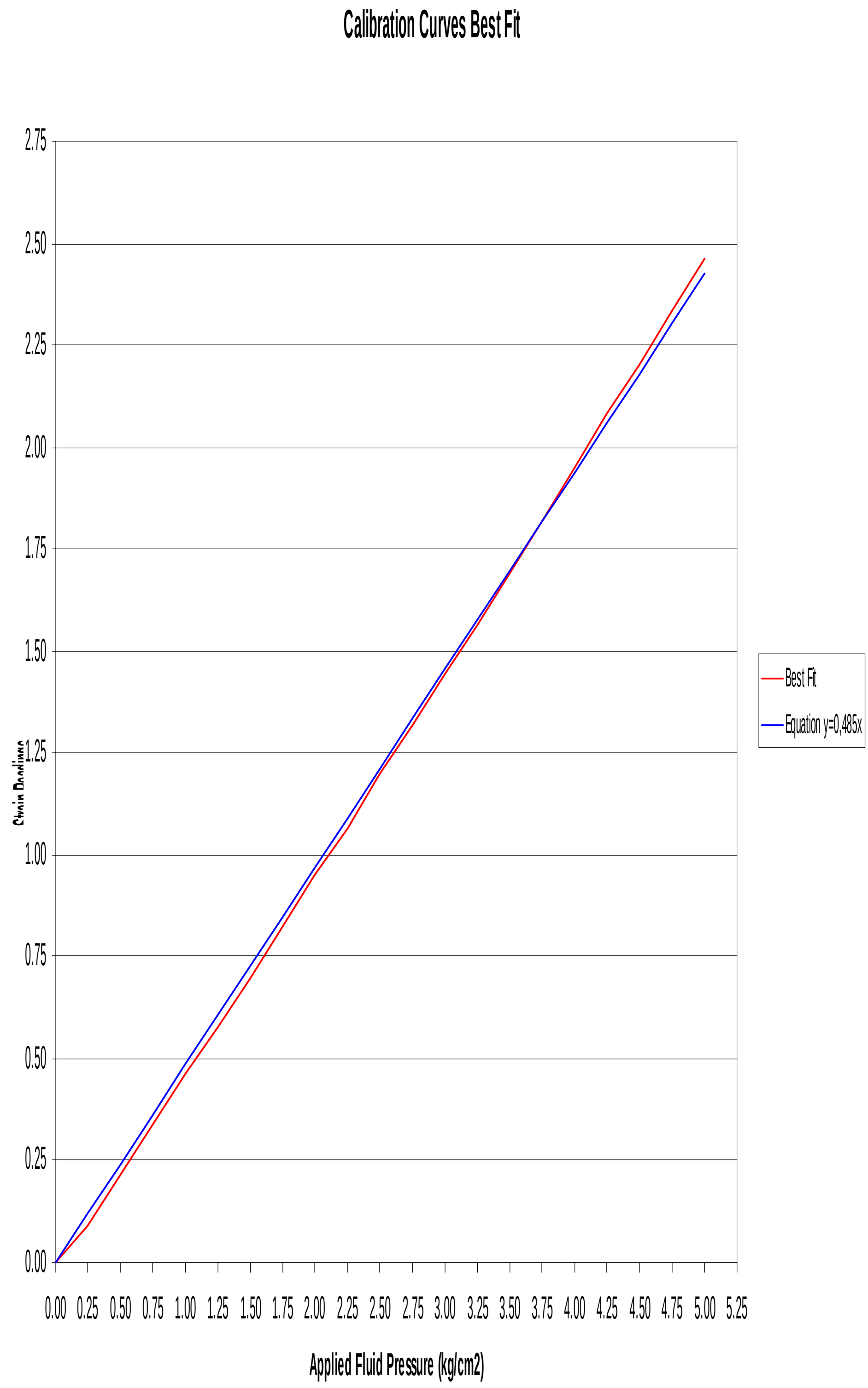


Figure 3.11 Best fit and Equation Curves for Calibration

The calibration data obtainbed from these four load – unload cycles indicated that all strain gauges produced a linear realtionship having a constant calibration constant. During the cycling loading no hysterises effects were noticed, this means that the electronics of the system is stable during any loading cycles without any significant drifts.

3.4 OPERATION RANGE OF THE MANUFACTURED LATERAL SWELL PRESSURE OEDOMETER RING

The thickness of the thin wall was chosen to satisfy that the ring was sufficiently thin so that the hoop deformations would be sensed by the strain gauges and also sufficiently thick to satisfy the K_o condition, i.e., no lateral deformation. The below computations are made to check K_o condition requirement is satisfied using the ring dimensions and properties based on the theory of elasticity. The highest safe level of internal pressure which can be applied to ring, the corresponding strains and the enlargement of the internal diameter can be calculated using principles of the strength of the material. Unit heigth is considered for the simplicity. Figure 3.12 shows stress distribution in the thin wall oedometer ring wall and the diagram for strength calculations. The definitions of notations used for the below calculations are as follows;

R : Internal radius of the ring

E : Modulus of elasticity of the ring material

t : Thickness of the ring wall

P_i : Internal pressure acting on the ring wall

δ : Tensile stress on the ring wall

ε : Strain on outer peripheral of the ring wall

dR : Enlargement of the internal radius

N : Total normal force acting on vertical cross sectional area

A : Vertical cross sectional area per unit height

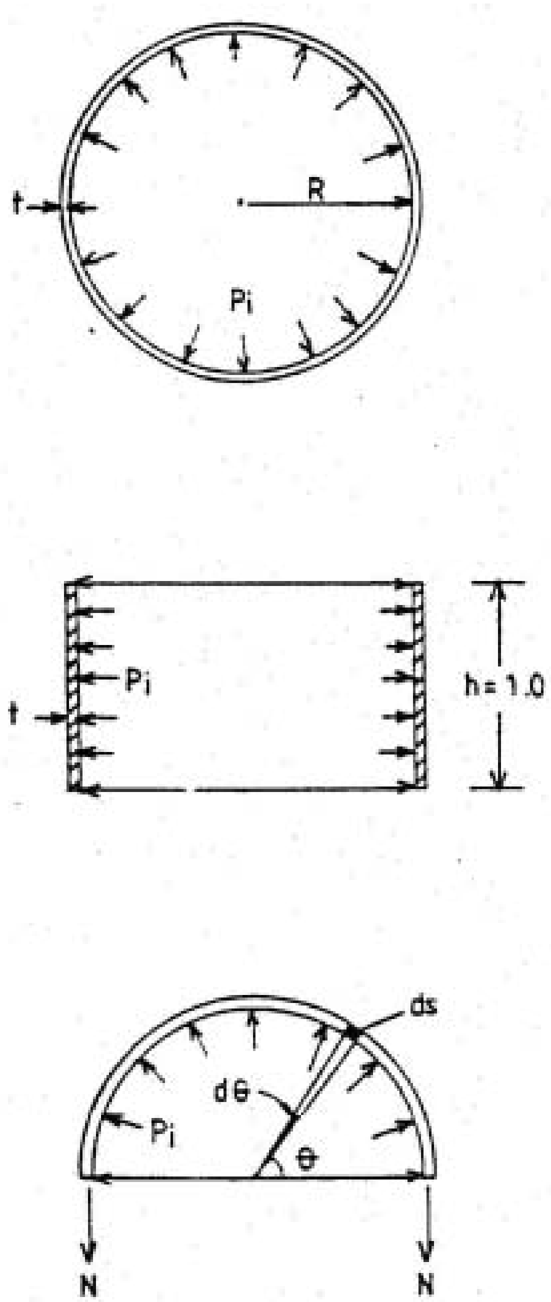


Figure 3.12 Stress Distribution in the Thin Wall Oedometer Ring Wall and Diagram for Strength Calculations

Equilibrium of the forces satisfies in the case of:

$$N = \text{ınt } P_i * \sin(\theta ds) \tag{3.1}$$

where,

$$ds = R\, d\theta \tag{3.2}$$

Substitution of 4.2 in 4.1 gives,

$$N = \text{int } P_i * \sin(\theta R d\theta) \tag{3.3}$$

Since,

$$\text{İnt } \sin(\theta d\theta) = 1 \text{ , then} \tag{3.4}$$

$$N = P_i * R \tag{3.5}$$

Tensile stress, δ in the ring wall is,

$$\delta = P_i * R/A \tag{3.6}$$

$$A = t*h \quad \text{or} \quad A = t*1 = t \qquad \text{for unit thickness} \tag{3.7}$$

$$\delta = P_i * R/A = P_i * R/t \tag{3.8}$$

Or in the other way of expression,

$$P_i = \delta t/R \tag{3.9}$$

Strain on the outer peripheral of the ring wall (the tangential strain ε);

$$\varepsilon = \delta/E \tag{3.10}$$

$$\varepsilon = P_i * R/E * t \tag{3.11}$$

Enlargement of the internal radius,

$$\varepsilon = ((R+dR)*2\pi - 2\pi)/2\pi * R \tag{3.12}$$

$$dR/R = P_i * R/E * t \tag{3.13}$$

$$dR = P_i * R^2/E * t \tag{3.14}$$

Using the material characteristics of the apparatus as:

Design tensile strength : $\delta_d = 55*10^4$ kPa

Modulus of elasticity : $E = 2,1*10^8$ kPa

Using these values;

Maximum allowable inner pressure without failure of inner thickness of $t = 0,35$ mm is;

$P_{max} = 6*10^3$ kPa

The maximum pressure applied to the thin wall in these tests is calibration pressure of 500 kPa, it is very small when compared to allowable pressure. The peripheral strain for this pressure is;

$\varepsilon = 2,17*10^4$ (217 μ strains)

The above calculations show that the manufactured thin wall oedometer ring is adequate for expected lateral swell pressure ranges of clayey soils.

CHAPTER 4

EXPERIMENTAL STUDY

4.1 PURPOSE

The purpose of this experimental study is to measure the lateral swelling pressure of clayey soil and to investigate the relationship between the vertical and the lateral swelling pressures and also investigate the effects of initial water content and initial dry density on these pressures by using the previously described thin wall oedometer test set – up.

The constant volume swell (CVS) tests are carried out. To investigate the effects of initial water content and initial dry density, a wide range of water content and densities are used for the test program. Changes in the lateral and vertical swelling pressures and the rebound characteristics are examined in these ranges.

4.2 MATERIAL INVESTIGATED

The clayey soil sample used in these study are obtained from Ankara Metro Line Construction – Station D29 located on the Batıkent – Sincan line. The soil sample is a plastic clay. The soil is classified as CH soil according to Unified Soil Classification System. Index properties of this clayey soil is shown in Table 4.1.

Table 4.1 Index Properties of Clayey Soil

Liquid Limit , LL	87,83 %
Plastic Limit , PL	32 %
Plasticity İndex , Ip	55,83 %
Shrinkage Limit , SL	15,9 %
Specific Gravity , Gs	2,6054
Percent Finer Than Sieve no.200	69,7 %
Percent Finer Than Sieve no.4	1,42 %
Percent Finer Than 2 μm , Clay Content , C %	44,17 %
Activity = Ip / C %	1,264
Optimum Moisture content , Wopt	36,5 %
Maximum Dry Density (Standart Proctor test) ,γd	12,65 kN/ m^3
Group Symbol (Unified Soil Classification System, 1957)	CH

The grain size distribution of the clay sample as a result of combined sieve and hydrometer analysis together is shown in Figure 4.1.

Plasticity characteristics of the clay is determined by using the index properties and the plasticity chart, and clay sample is on CV group (inorganic clays of very high plasticity) according to British System Plasticity Chart (BS 5930, 1981) as shown in Figure 4.2.

Swell classification of the clay sample is done by using the chart of Popescu (1986) and it is found that the clay sample is on the very high expansion region as shown in Figure 4.3.

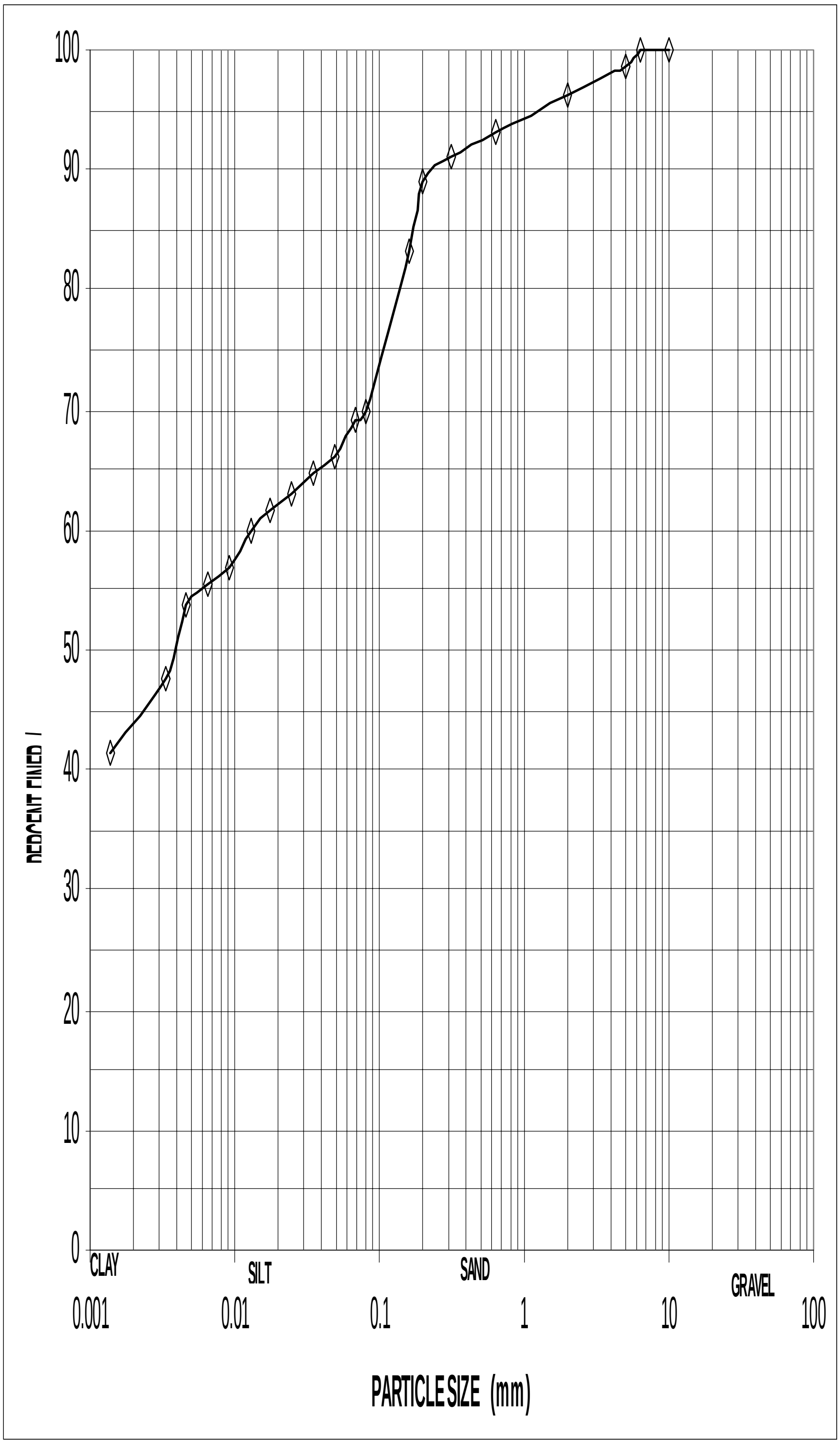


Figure 4.1 Grain Size Distribution of the Clayey Soil

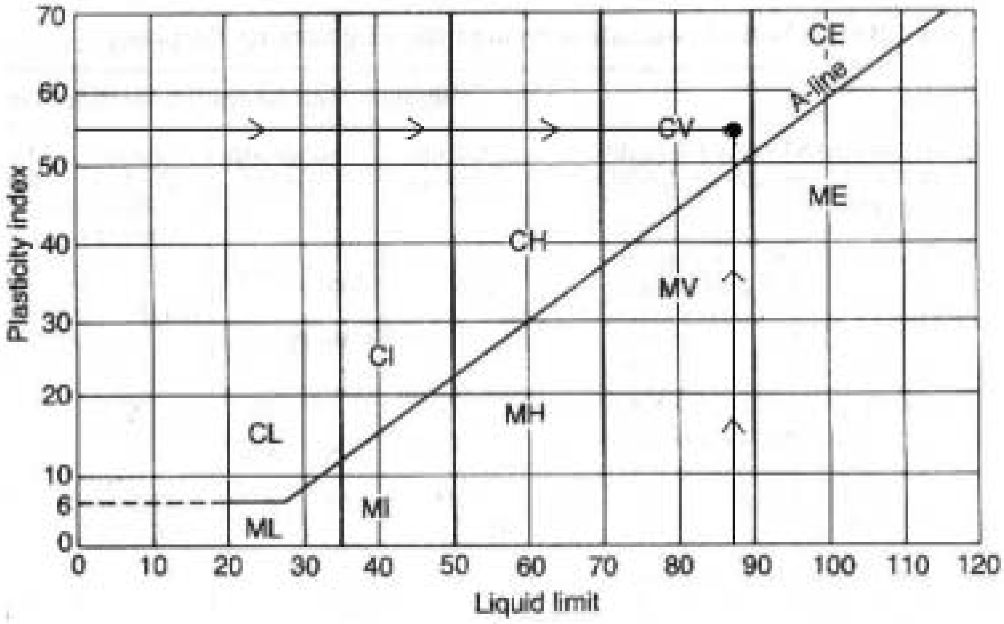


Figure 4.2 Plasticity Chart, British System (BS 5930, 1981)

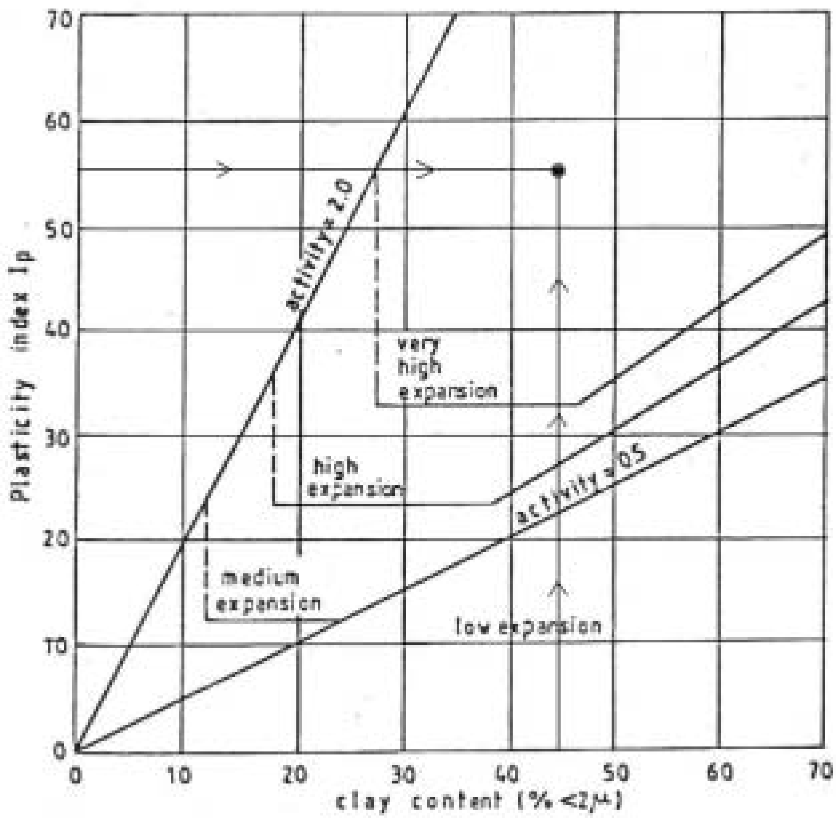


Figure 4.3 Swell Potetntial of the Clayey Soil (After Popescu, 1986)

4.3 PREPARATION OF SPECIMENS

The clayey soil is owen dried, crushed and pulverized. Clayey soil sieved through no.4 sieve and the finer fractions are used in these experimental work. To obtain the required initial water content and the initial dry density for each experiment set, calculated amounts of the clayey soil and the water is mixed carefully. The moist samples are wrapped into three sheet nylon bags and kept in dessicator for several days to provide homogeneous moisture distribution through the clay particles. The soil sample in nylon bag and dessicator are shown in Figure 4.4.

Figure 4.4 Soil Sample in Nylon Bag and in Dessicator

Before each set small samples are taken from these prepared soil to check the required initial moisture content for each set. The required initial dry density is obtained by compacting statically the calculated amount of the soil sample into the sampler ring (63,5 mm in diameter and 39,3 mm in height) with sampling hammer by using a hydraulic jack. Figure 4.5 shows the steps of placement of the sample into the sampler ring in figures of the first row and the compaction on the hydraulic jack in the second row.

Figure 4.5 Placement of the sample into the Sampler Ring
and the Compaction on the Hydraulic Jack

After compaction the sample and the ring are weighted together to check the requirement of initial dry density for each set. After this check, the specimen is placed into the thin wall oedometer statically. Figure 4.6 shows thin wall oedometer with placed sample and screwed into the inundation container.

Lateral swelling pressure can be measured more accurately if the strain gages on the thin wall of the oedometer are in the same plane with the mid – point of the compacted soil sample inside the ring. To achieve this goal, the test set up shown in Figure 4.7 is prepared by calculations. A hyperglass with pores, to provide water pass, is manufactured to rise the mid point of the soil sample into the mid - level of strain gauges. And a water cap is manufactured, on the top as shown in Figure 4.7, to protect the strain gauges from inundation water. It is a extra – protection because the strain gauges and cables are protecting by some izolation chemicals painted on them. There is one important point that as seen from the Figure 4.7 it is provided that porous stones are not in direct - contact with the thin wall of the oedometer, because it can be resulted in wrong readings of strain due to the possible adhesive effects of porous stones to the thin wall.

Figure 4.6 thin wall oedometer with placed sample
and screwed into the inundation container.

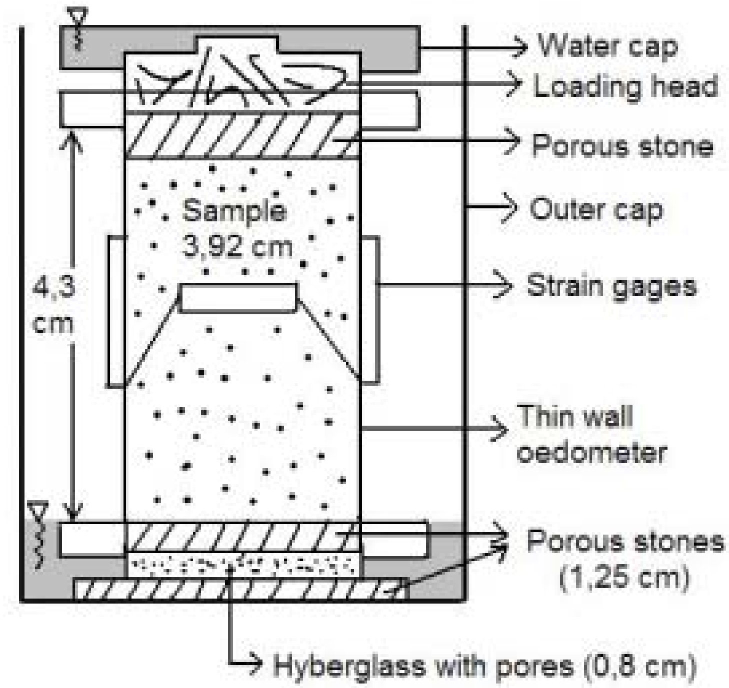


Figure 4.7 Cross Sectional View of Test Set - Up

4.4 ASSEMBLING OF THE TEST SET – UP

After the specimen is placed into the thin wall oedometer ring, the ring body is fixed into the inundation chamber. In this step there are two important points. Firstly, it must be assured that the required initial dry density is not disturbed while transfering the soil into the ring, it is checked by the red-pen lines in the ring. If the line is on the soil surface it means there is no placement problem. Secondly, while fixing the ring body into the inundation chamber, holding nuts have to be wrenched slightly so that no differential strains and stresses will occur on the thin wall. It can be checked from the display of the read – out unit. Two views of the assembled test set – up are shown in Figure 4.8.

Figure 4.8 Two Views of the Assembled Test Set - Up

4.5 TESTING PROCEDURE

To study the effect of the initial dry density, densities of 1.10, 1.15, 1.20, 1.25, 1.30 1.30 gr/cu.cm and to study the effect of the initial water content, water contents of 15, 20, 25, 30, 35, 40 percents are used in a cross table. The samples were 39,3 mm thick. All of the samples are tested in ‘Constant Volume Swell (CVS) Test’ according to ASTM standarts. While performing these tests lateral swelling pressure, vertical swelling pressure and time are recorded in the loading phase, strain with respect to the change in the load and time are recorded in the unloading phase.

4.6 TESTING TECHNIQUE

In constant volume swell (CVS) test, after assembling the test set – up and taking the zero reading from the read – out unit water is added to the reservours of the inundation container. After swelling is started, the loading on the system is started in small increments to obtain zero strain, i.e., to back into the original volume of the specimen. This swelling prevention lasts until the both lateral and vertical swell pressures are fully developed in this soaked condition. During tests, when zero strain is achieved, read – out unit readings, to calculate lateral swelling pressure, vertical loads and time are recorded. After fully development of the swell pressures, unloading phase is started with load decrements to obtain the rebound characteristics. Initial water content checks and final water content measurements are done for each set.

4.7 TESTING PROGRAMME

A total of 20 sets having different initial dry density and initial water content are carried out in CVS tests. Each set lasts 7 or 10 days together with unloading phase. Testing programme is completed nearly 4,5 months. Effects of initial water content and initial dry density, and the relationship between the lateral and vertical swelling pressures, by means of swelling ratio – S_r which is the ratio of the lateral swelling pressure to the vertical swelling pressure, are investigated throughout the experimental program.

4.8 TEST RESULTS

Results of constant volume swell (CVS) test results are summarized in Table 4.3. All data, including strain and final water content, at the end of each sets can be seen from Table 4.3. From Figure 4.9 to Figure 4.28 plots of lateral swelling pressure (Ph) versus vertical swelling pressure (Pv) and lateral and vertical swelling pressures versus time plots of sets 1 to 20 can be seen respectively. For each one, development of lateral and vertical pressures with respect to each other and with respect to time are plotted. To obtain rebound characteristics, void ratio versus logarithm of pressure curves are plotted also.

Table 4.2 shows set numbers of each set performed in this study with respect to their initial water content and initial dry density in a cross table.

Table 4.2 Set numbers

		Densities (g/cm3)				
		1,1	1,15	1,2	1,25	1,3
w_i (%)=	15	1	2	3	4	5
w_i (%)=	20	6	7	8	9	10
w_i (%)=	25	11	12	13	14	
w_i (%)=	30	15	16	17		
w_i (%)=	35	18	19			
w_i (%)=	40	20				

Table 4.3 Constant Volume Swell (CVS) Test Results

		γ =1,10 g/cm^3	γ =1,15 g/cm^3	γ =1,20 g/cm^3	γ =1,25 g/cm^3	γ =1,30 g/cm^3
W = 15%	P_v (kg/cm^2)	0.66	0.83	1.01	1.91	2.43
	P_h (kg/cm^2)	0.45	0.54	0.68	1.20	1.86
	S_r = Ph/Pv	0.69	0.64	0.68	0.63	0.76
	W_f	48.63	46.48	45.66	43.39	38.66
W = 20%	P_v (kg/cm^2)	0.52	0.80	0.90	1.60	2.08
	P_h (kg/cm^2)	0.41	0.47	0.62	1.05	1.69
	S_r = Ph/Pv	0.79	0.59	0.68	0.66	0.81
	W_f	49.99	47.87	43.80	40.89	38.76
W = 25%	P_v (kg/cm^2)	0.47	0.69	0.80	1.25	
	P_h (kg/cm^2)	0.37	0.45	0.58	0.82	
	S_r = Ph/Pv	0.79	0.65	0.72	0.66	
	W_f	50.12	48.76	43.51	42.50	
W = 30%	P_v (kg/cm^2)	0.40	0.63	0.73		
	P_h (kg/cm^2)	0.33	0.41	0.54		
	S_r = Ph/Pv	0.83	0.66	0.73		
	W_f	50.84	47.59	44.75		
W = 35%	P_v (kg/cm^2)	0.36	0.52			
	P_h (kg/cm^2)	0.31	0.37			
	S_r = Ph/Pv	0.85	0.71			
	W_f	49.95	47.57			
W = 40%	P_v (kg/cm^2)	0.31				
	P_h (kg/cm^2)	0.27				
	S_r = Ph/Pv	0.86				
	W_f	50.29				

Pressures decreases

Sr increases

Pressure increases

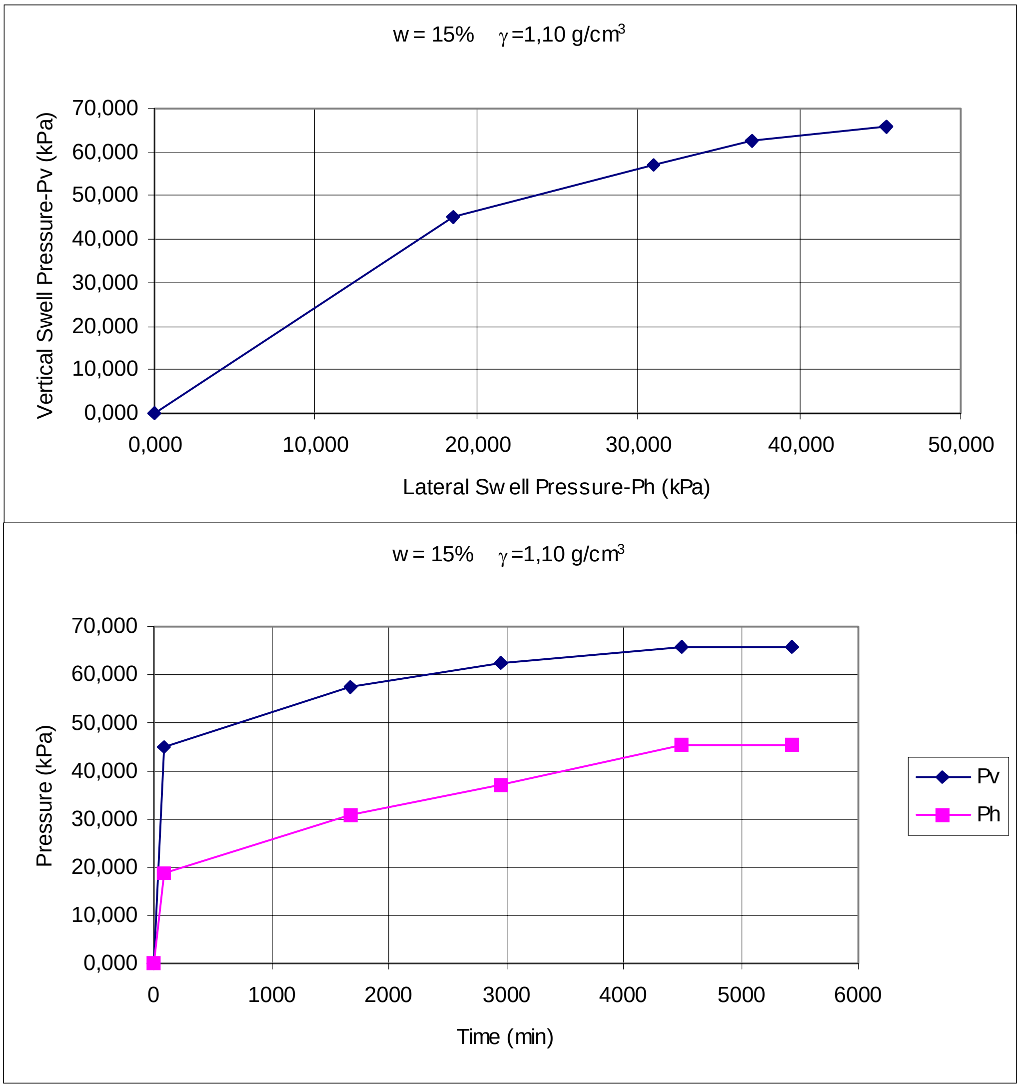
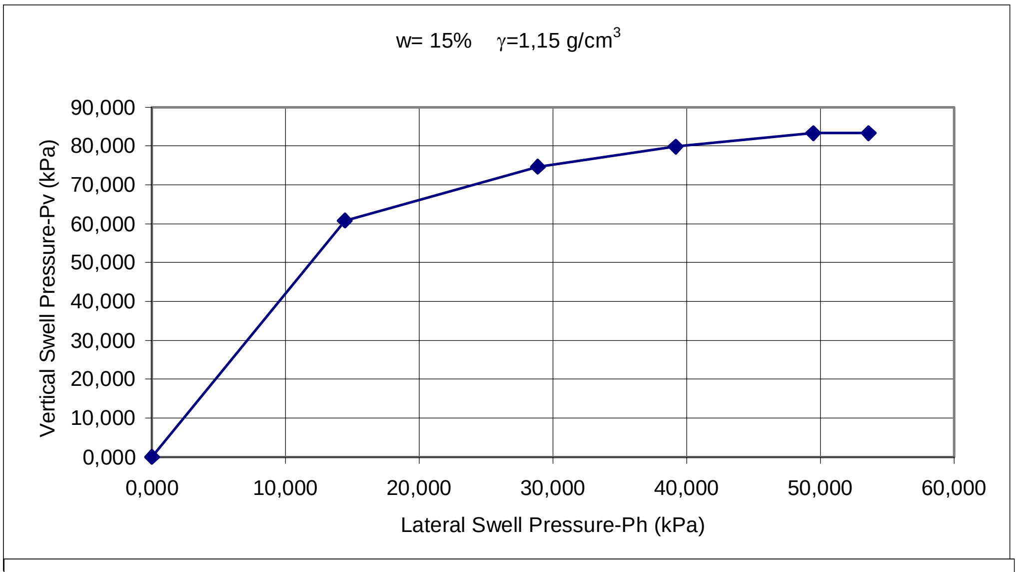


Figure 4.9 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time relationships in Series No:1 Test.



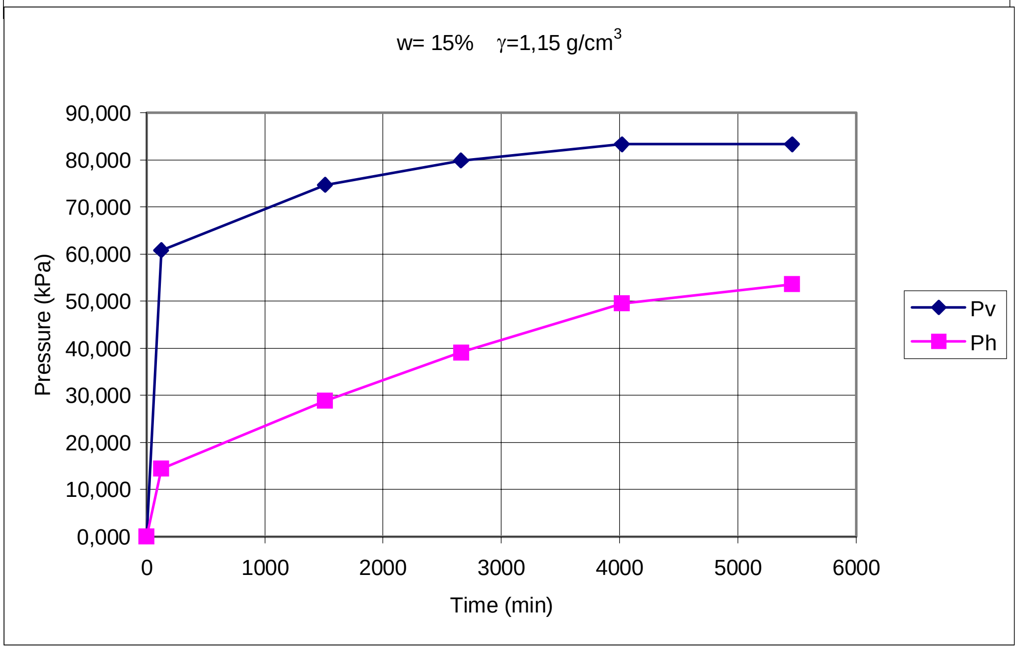


Figure 4.10 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time relationships in Series No:2 Test.

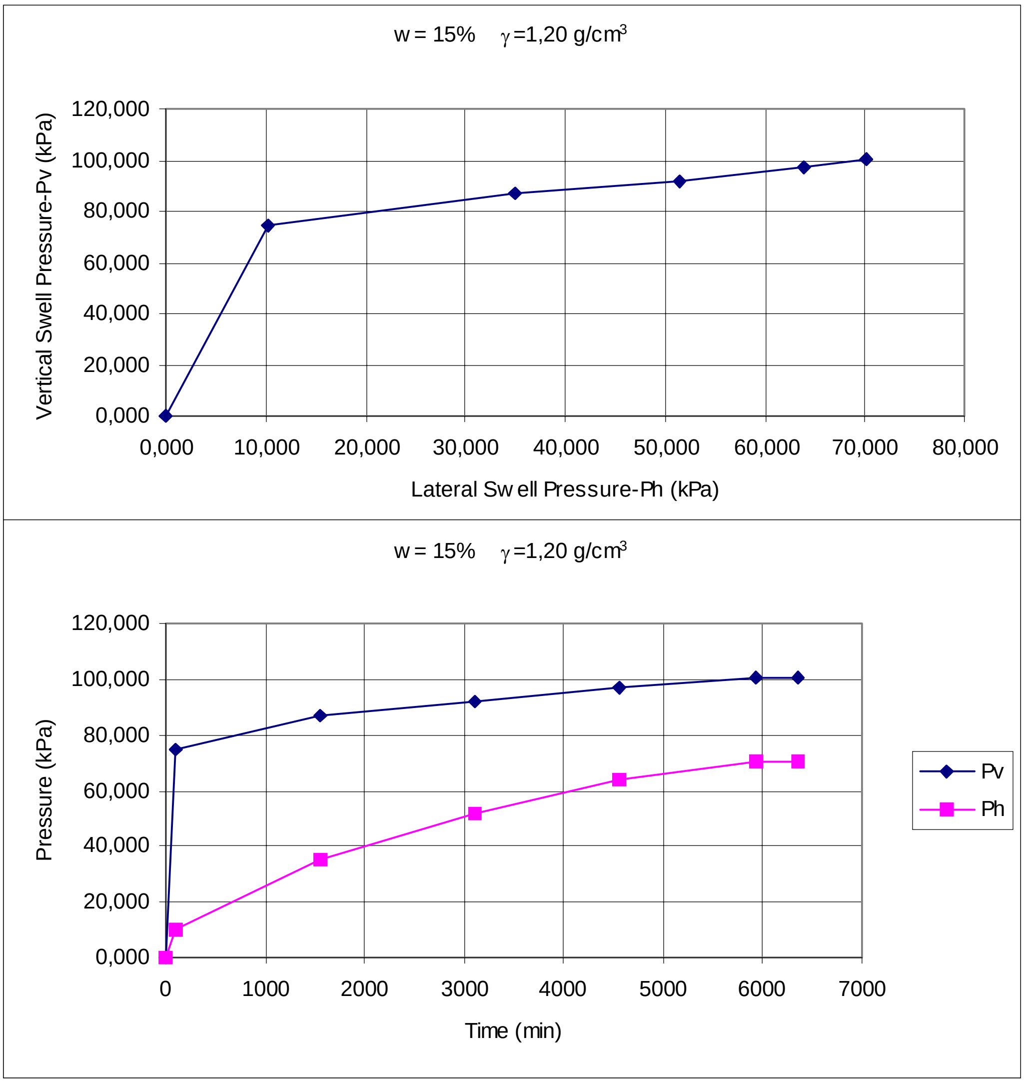


Figure 4.11 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time relationships in Series No:3 Test.

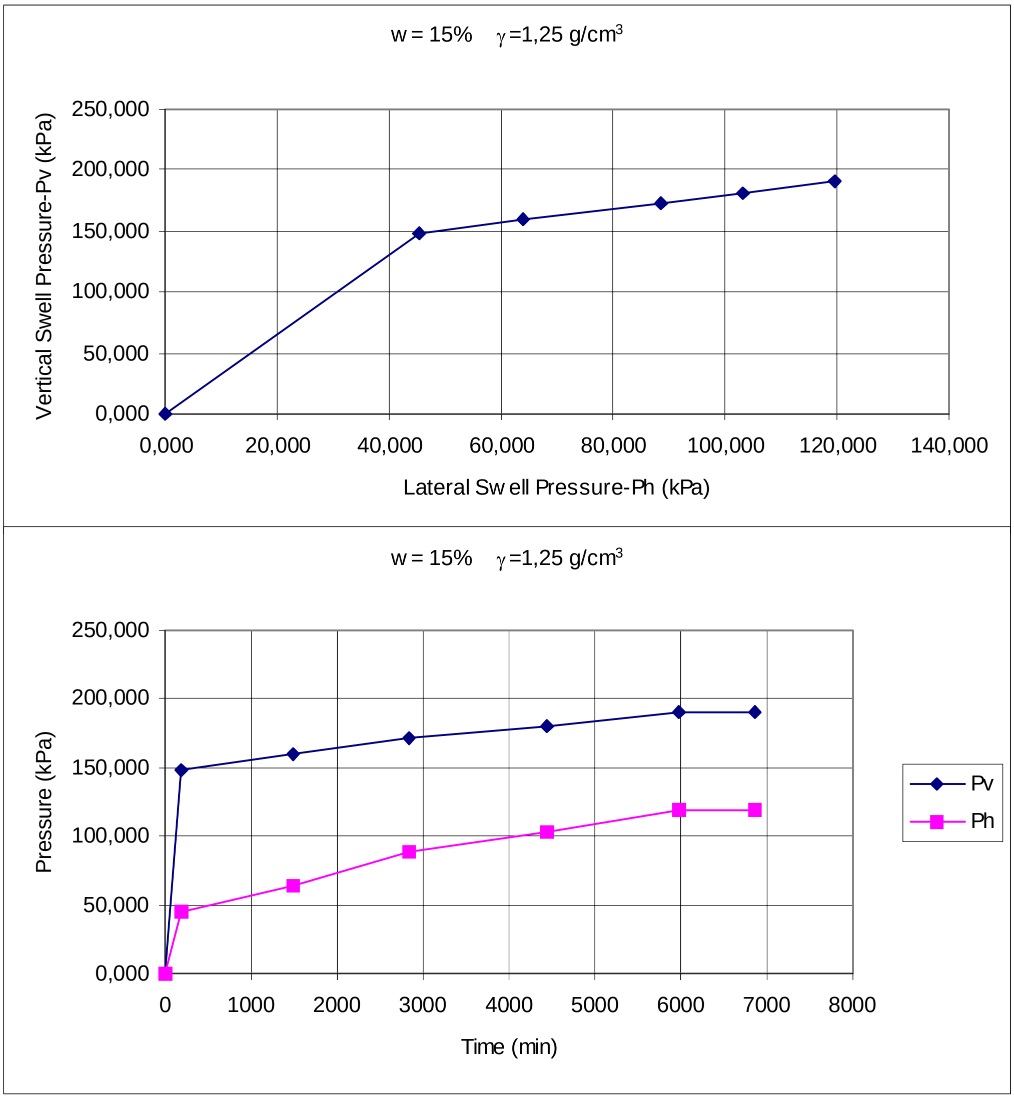


Figure 4.12 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time relationships in Series No:4 Test.

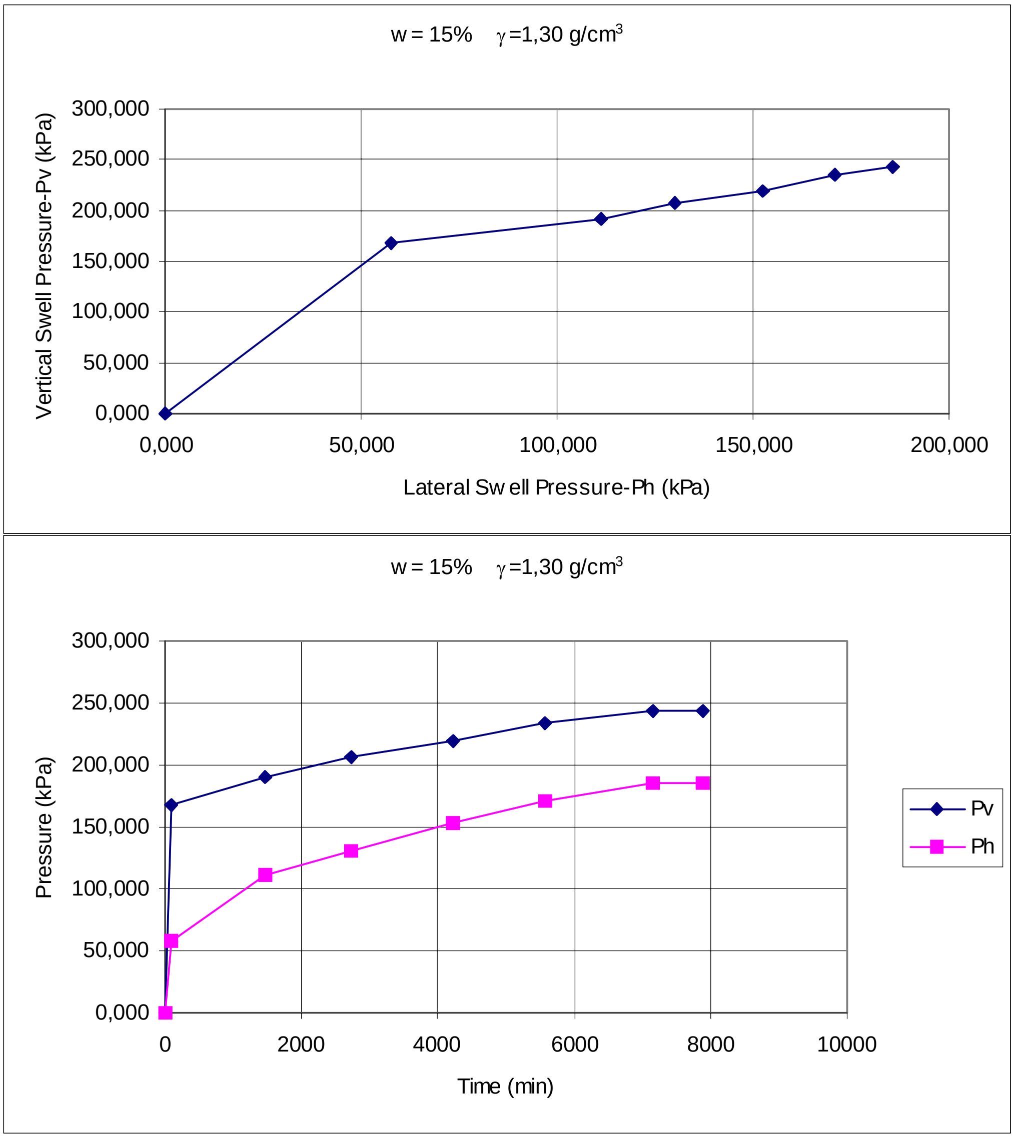
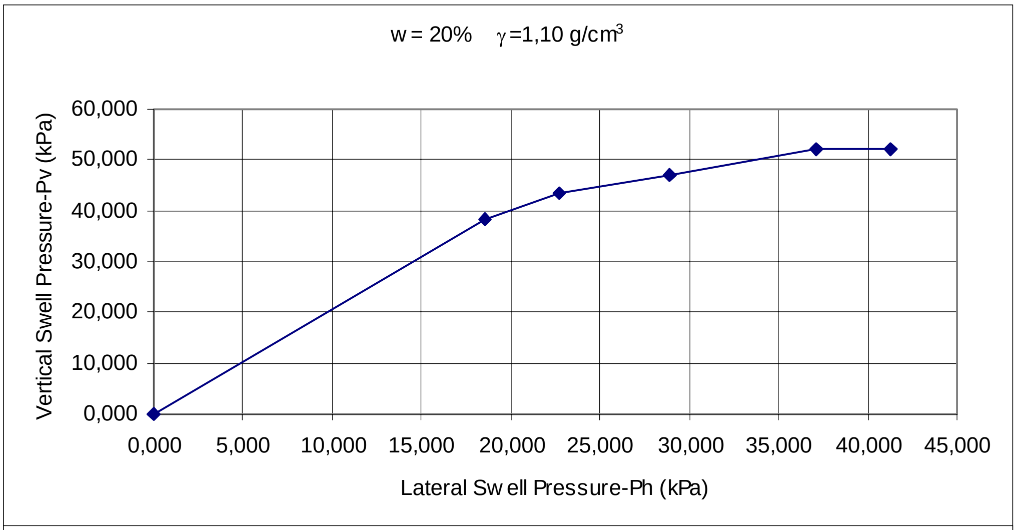


Figure 4.13 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time relationships in Series No:5 Test.



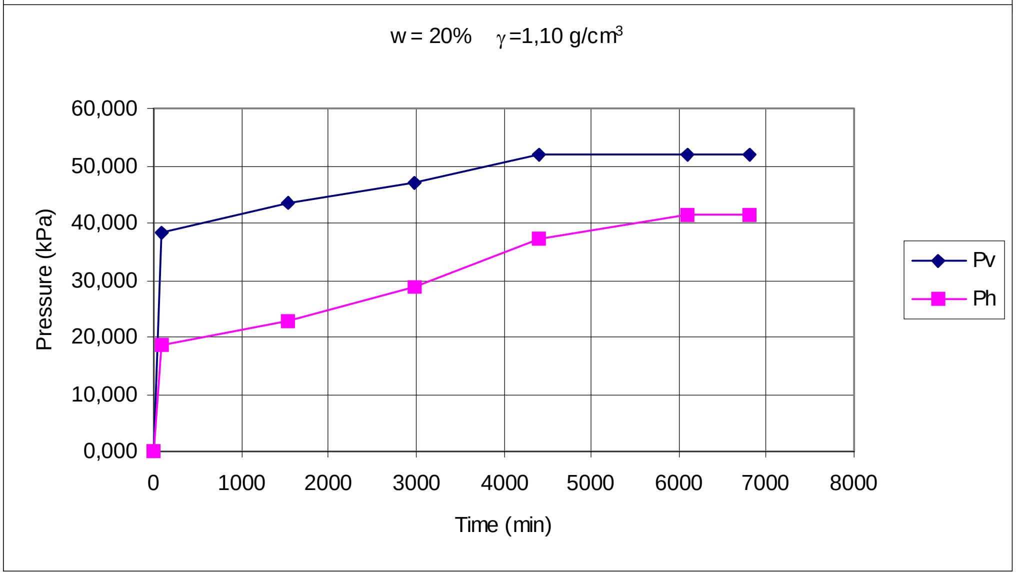


Figure 4.14 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time relationships in Series No:6 Test.

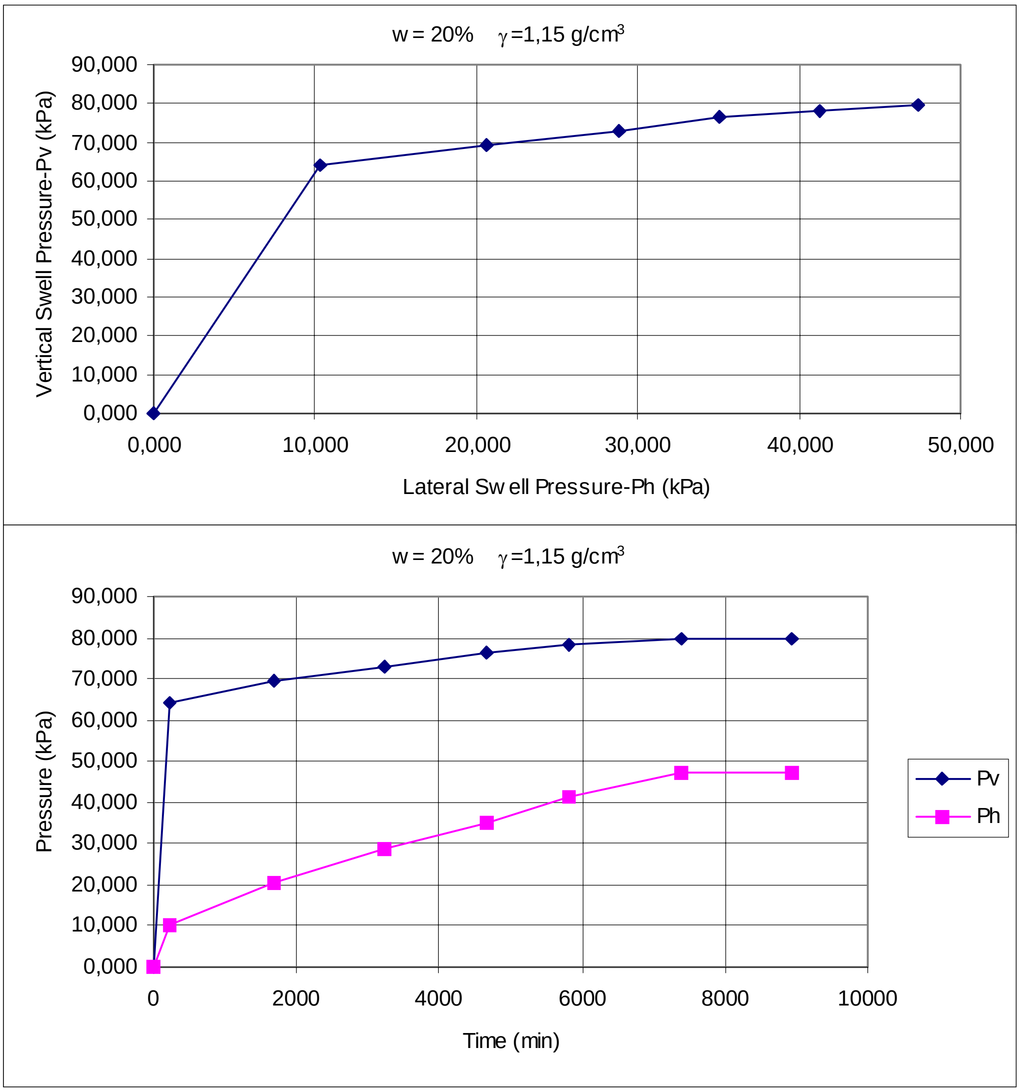


Figure 4.15 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time relationships in Series No:7 Test.

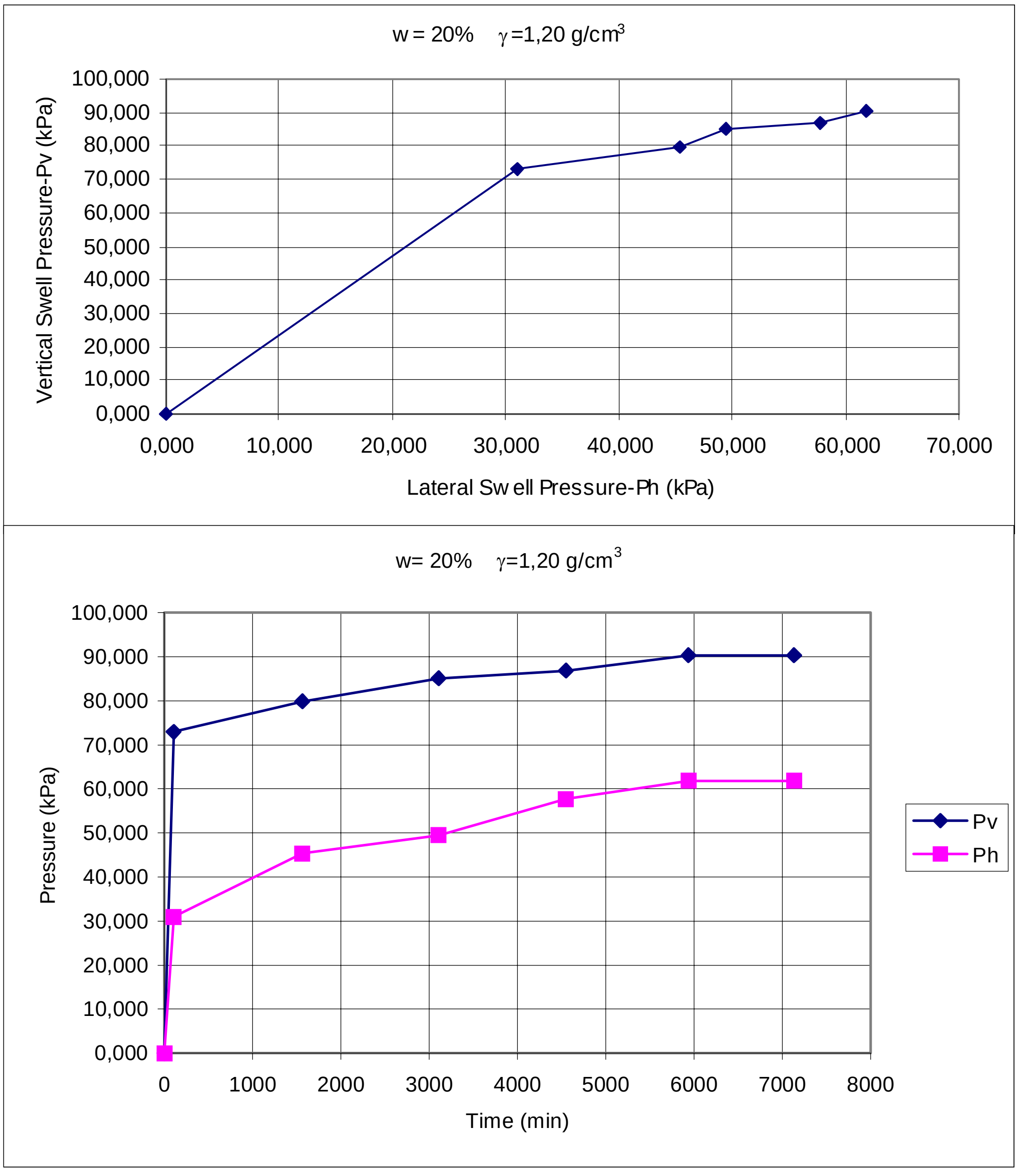


Figure 4.16 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time relationships in Series No:8 Test.

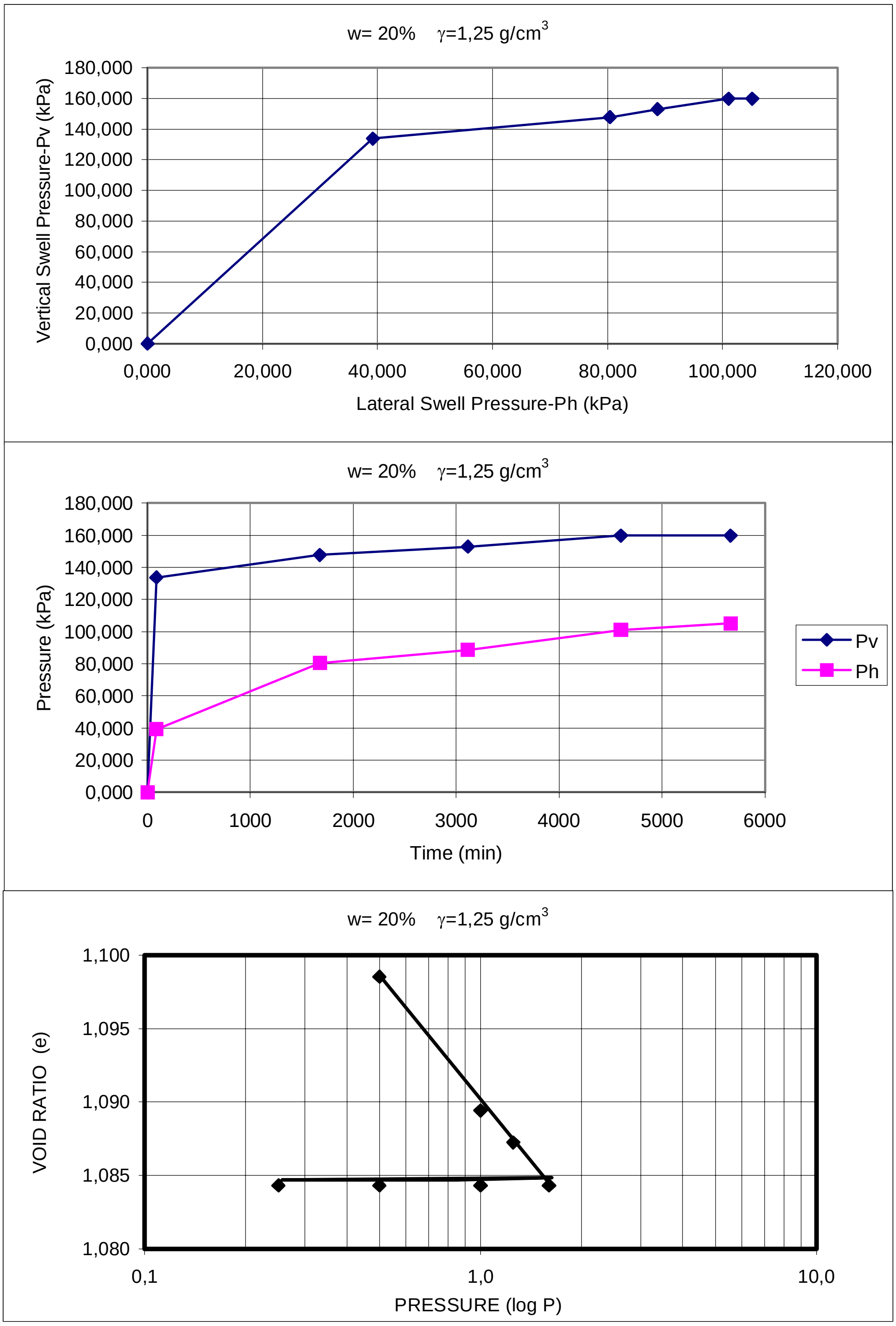


Figure 4.17 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time, and void ratio vs. logarithm of presssure relationships in Series No:9 Test.

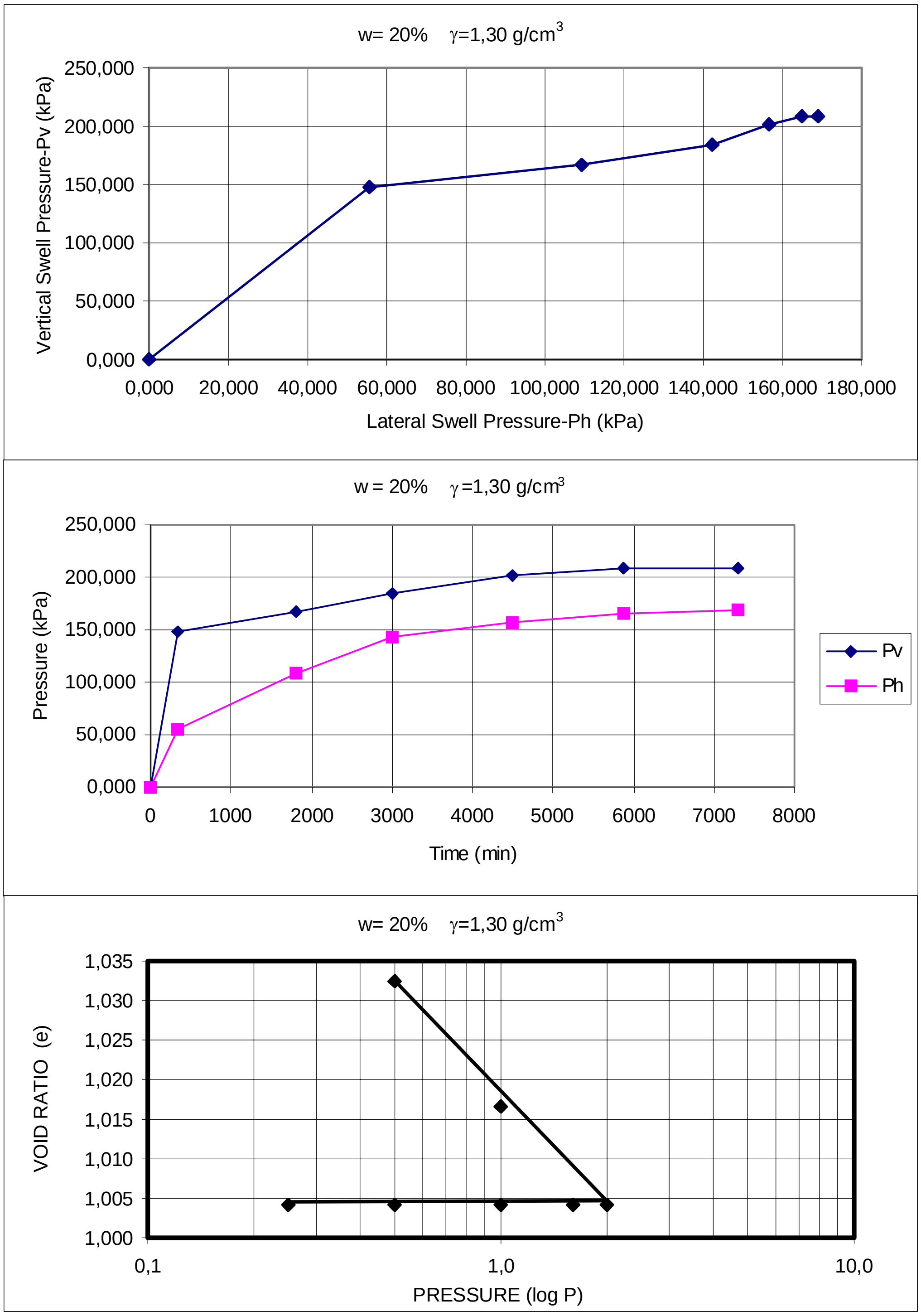


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Figure 4.18 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time, and void ratio vs. logarithm of presssure relationships in Series No:10 Test.

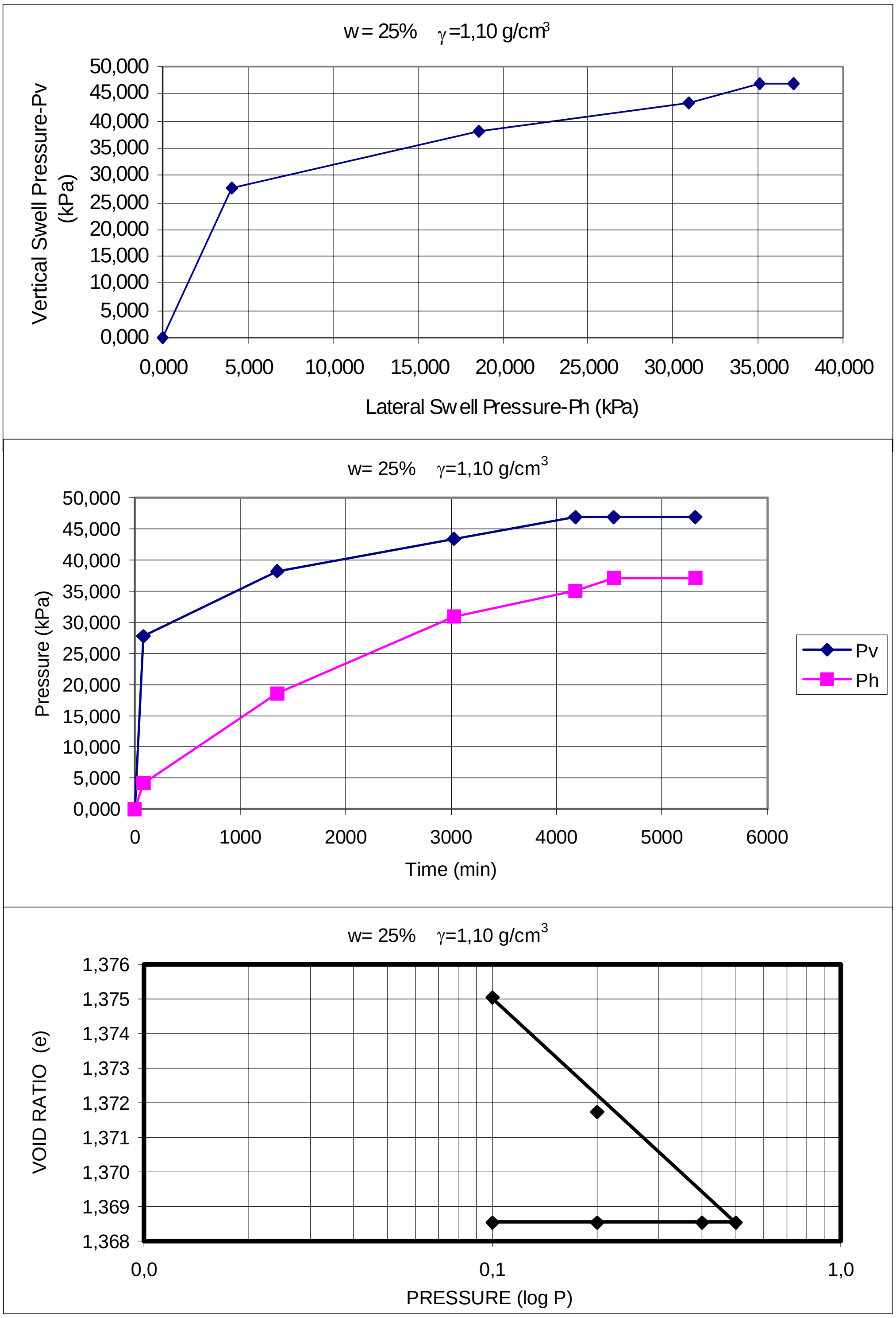

Figure 4.19 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time, and void ratio vs. logarithm of presssure relationships in Series No:11 Test.

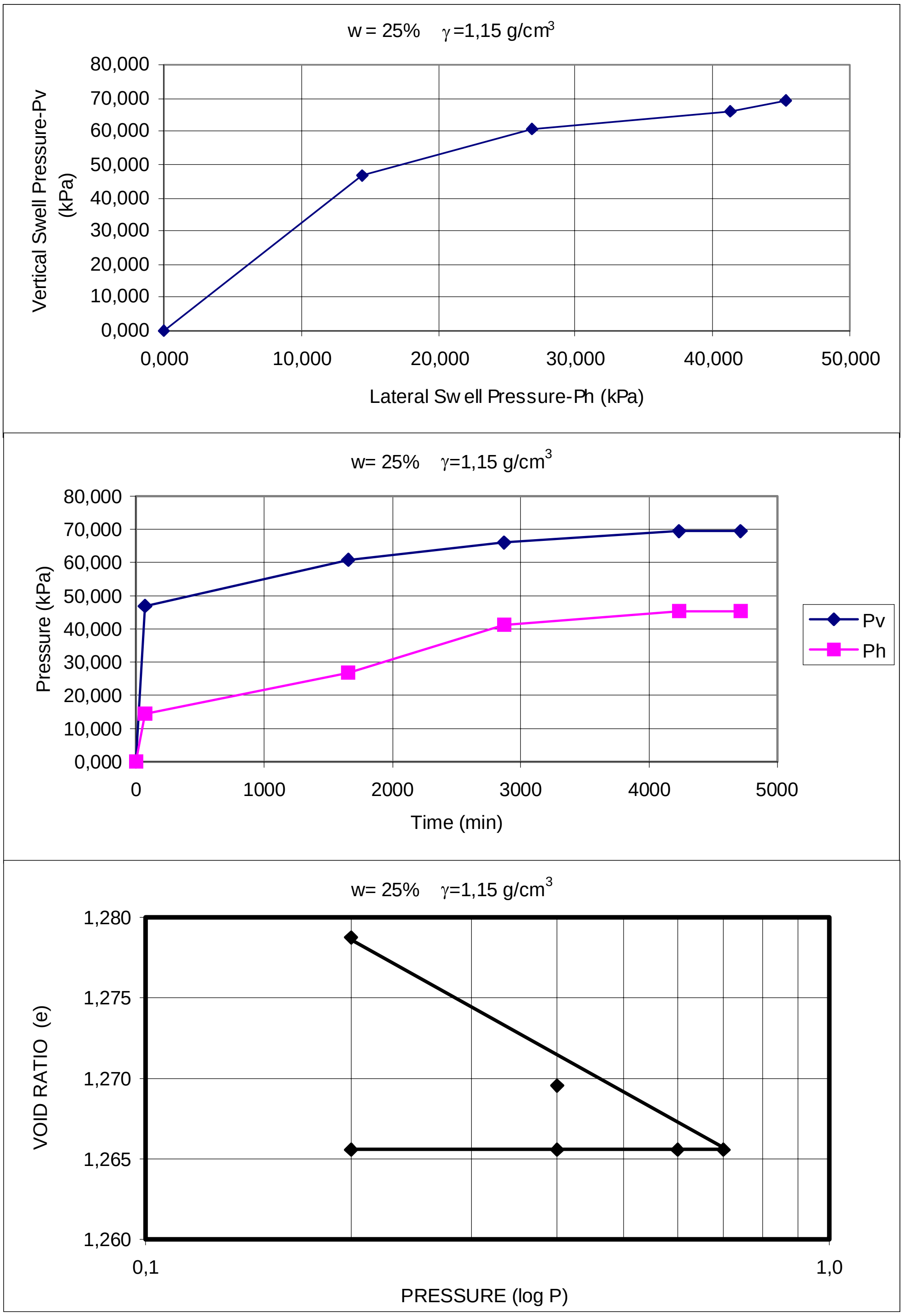

Figure 4.20 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time, and void ratio vs. logarithm of presssure relationships in Series No:12 Test.

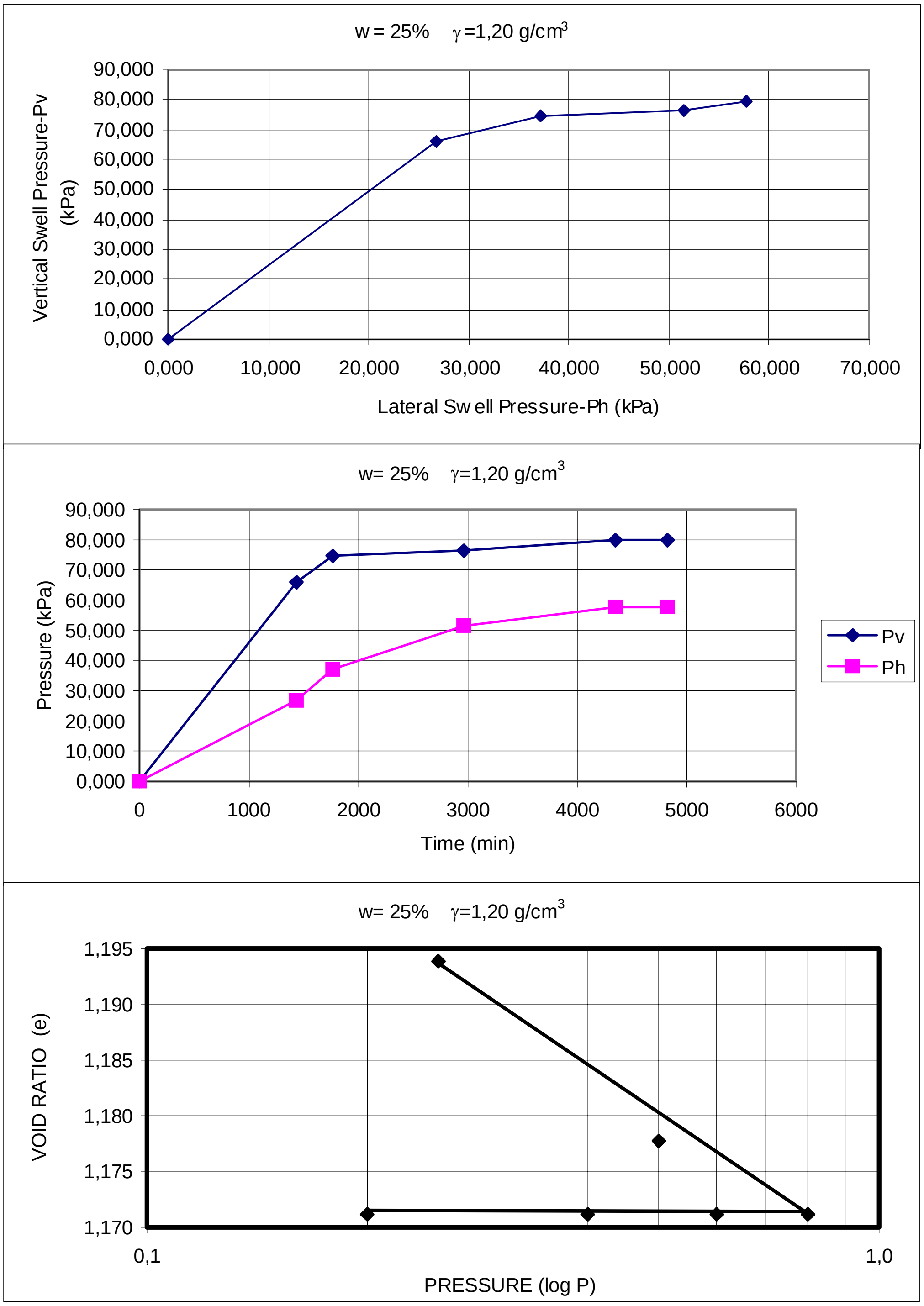

Figure 4.21 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time, and void ratio vs. logarithm of presssure relationships in Series No:13 Test.

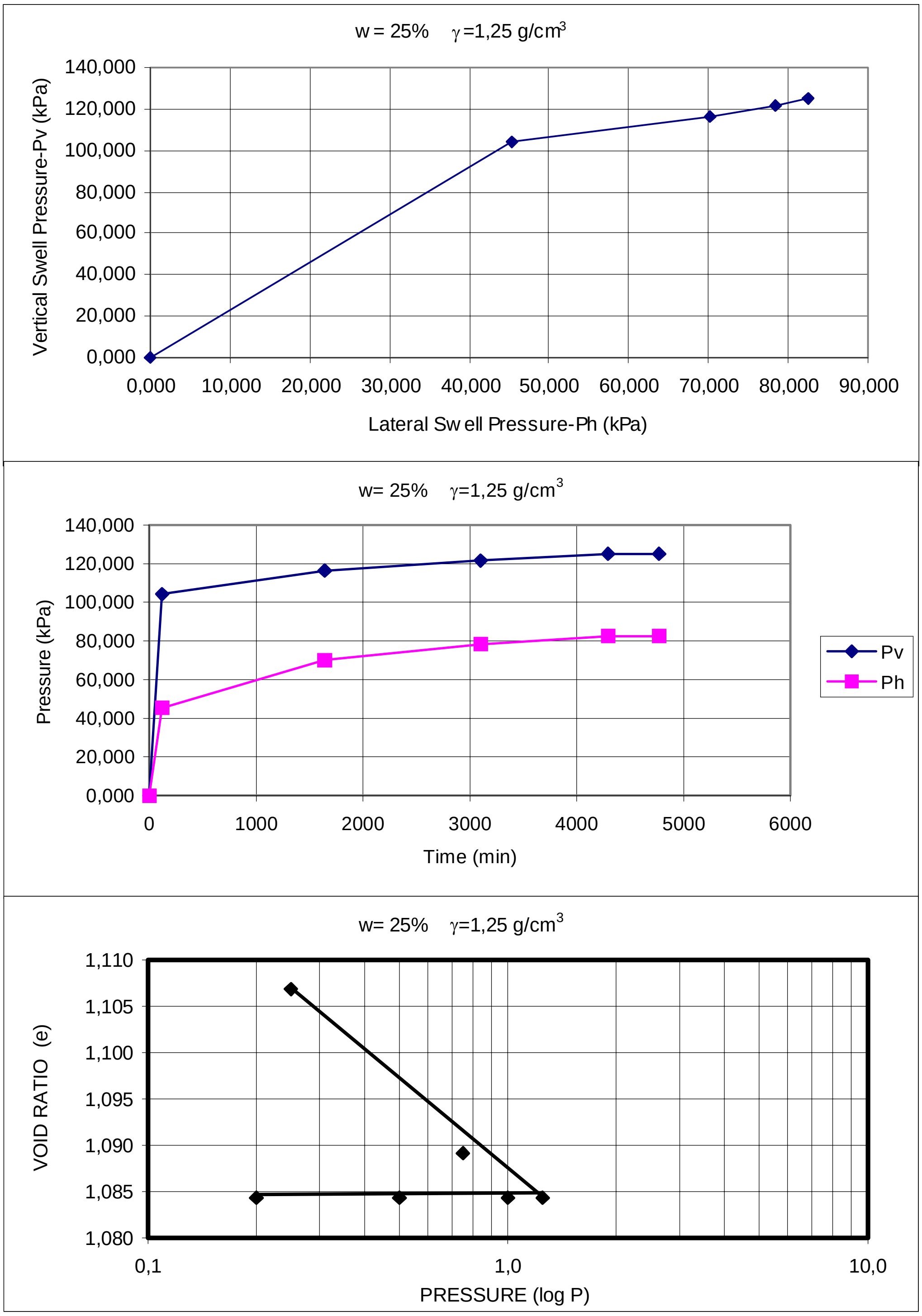

Figure 4.22 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time, and void ratio vs. logarithm of presssure relationships in Series No:14 Test.

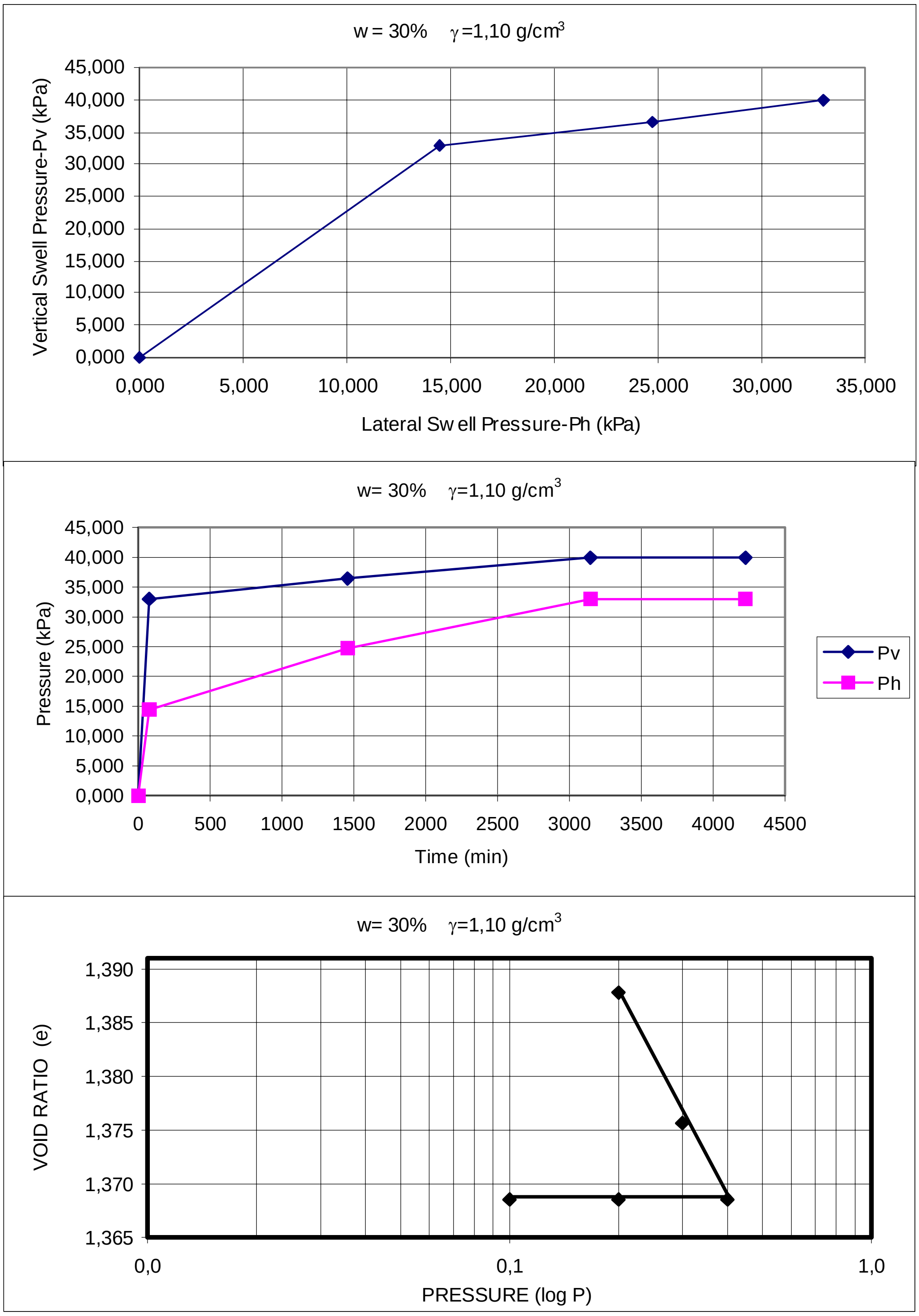

Figure 4.23 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time, and void ratio vs. logarithm of presssure relationships in Series No:15 Test.

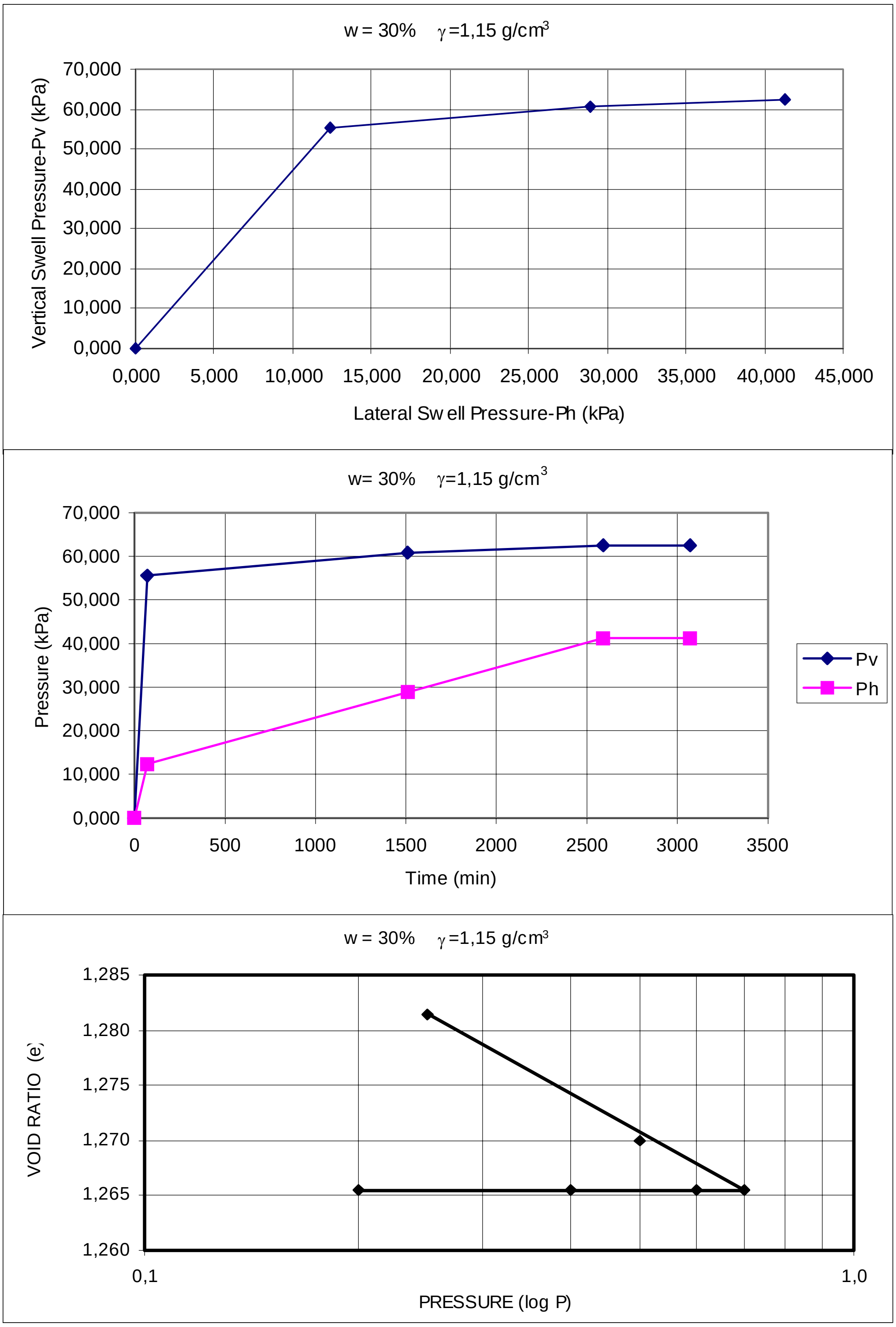

Figure 4.24 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time, and void ratio vs. logarithm of presssure relationships in Series No:16 Test.

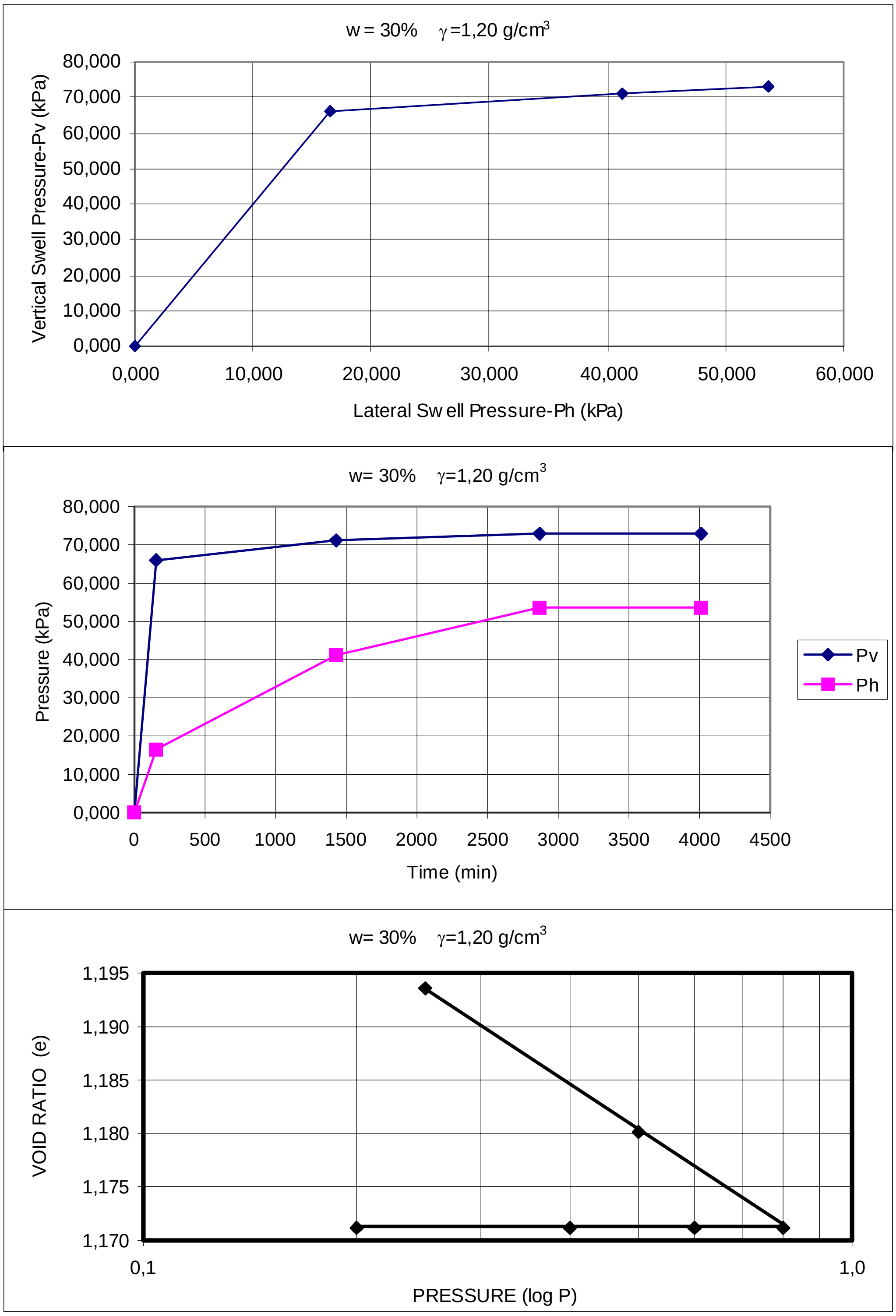

Figure 4.25 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time,and void ratio vs. logarithm of pressure relationships in Series No:17 Test.

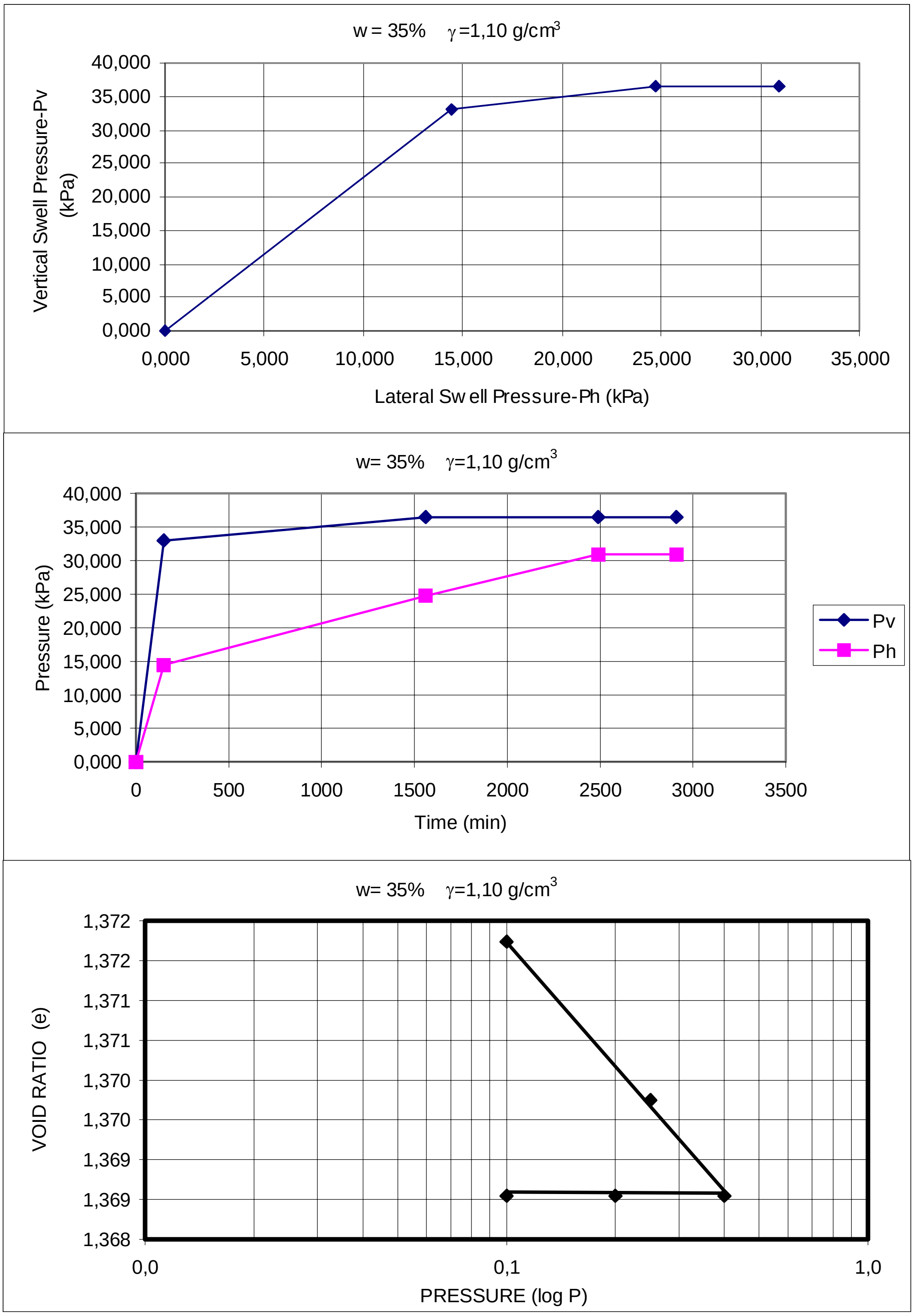

Figure 4.26 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time,and void ratio vs. logarithm of pressure relationships in Series No:18 Test.

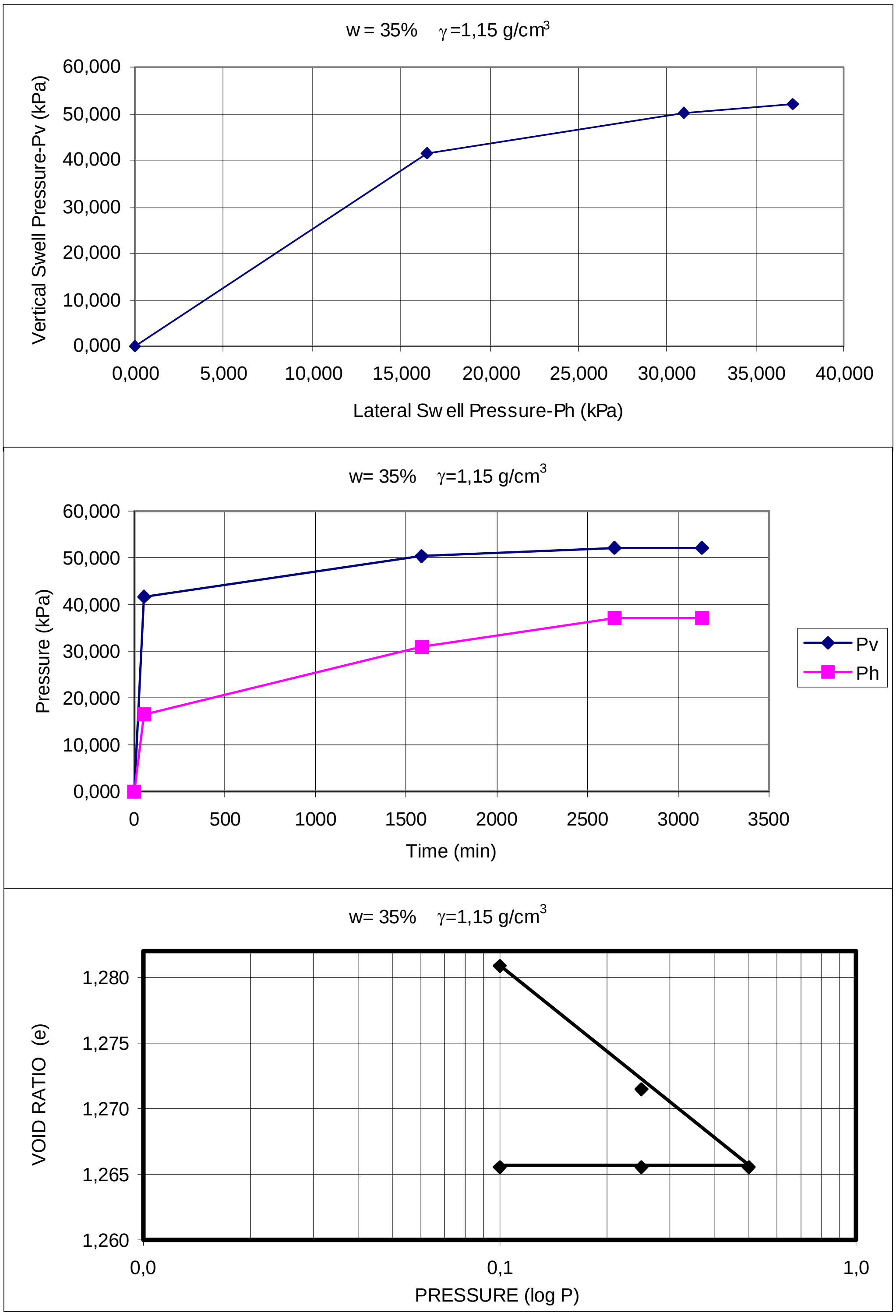

Figure 4.27 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time,and void ratio vs. logarithm of pressure relationships in Series No:19 Test.

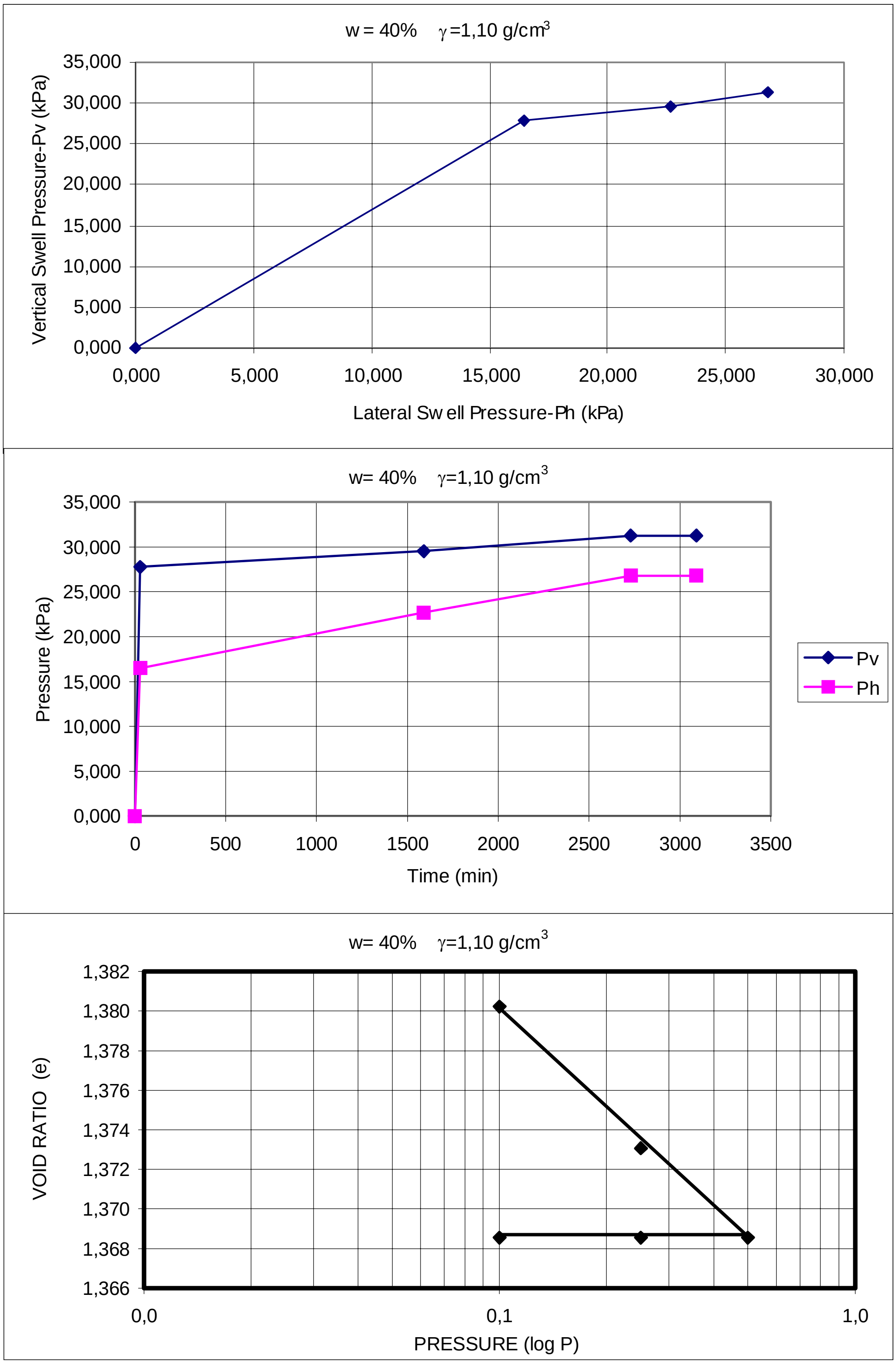

Figure 4.28 Lateral vs. vertical swelling pressures, lateral and vertical swelling pressures vs. time,and void ratio vs. logarithm of pressure relationships in Series No:20 Test.

CHAPTER 5

DISCUSSION OF TEST RESULTS

5.1 LATERAL SWELL BEHAVIOUR IN CVS TESTS

Brackley (1975) stated that free swell (swell of the soil under a negligible load of one kilopascal) is controlled predominantly by the initial moisture content, but that swell pressure (pressure required to hold the soil at constant volume when water is added) is a function of density. During constant volume swell (CVS) tests it has been found that both of initial dry density and initial water contents affect the lateral and vertical swelling pressures. The detailed discussions of test results will be presented in the proceeding sections.

5.1.1 Effect of Initial Water Content

Swelling mechanism depends on the amount of water absorbed by the soil mass. As the initial water content increases, for specimens having the same dry unit weight, the initial degree of saturation will also increase and the affinity of soil to absorb water will decrease. It follows that the amount of water absorbed for complete saturation will become smaller, and consequently the amount of swelling will decrease as the initial water content increases (El – Sohby and Rabba, 1981). As the absorbed water is decreased for the same initial dry density the interparticle forces developed during swelling will become smaller. This in turn will be resulted in smaller swell pressures.

The vertical swelling pressure decreases as the initial water content increases. Figure 5.1 shows relationship between the vertical swelling pressure and the initial water content for different initial dry densities.

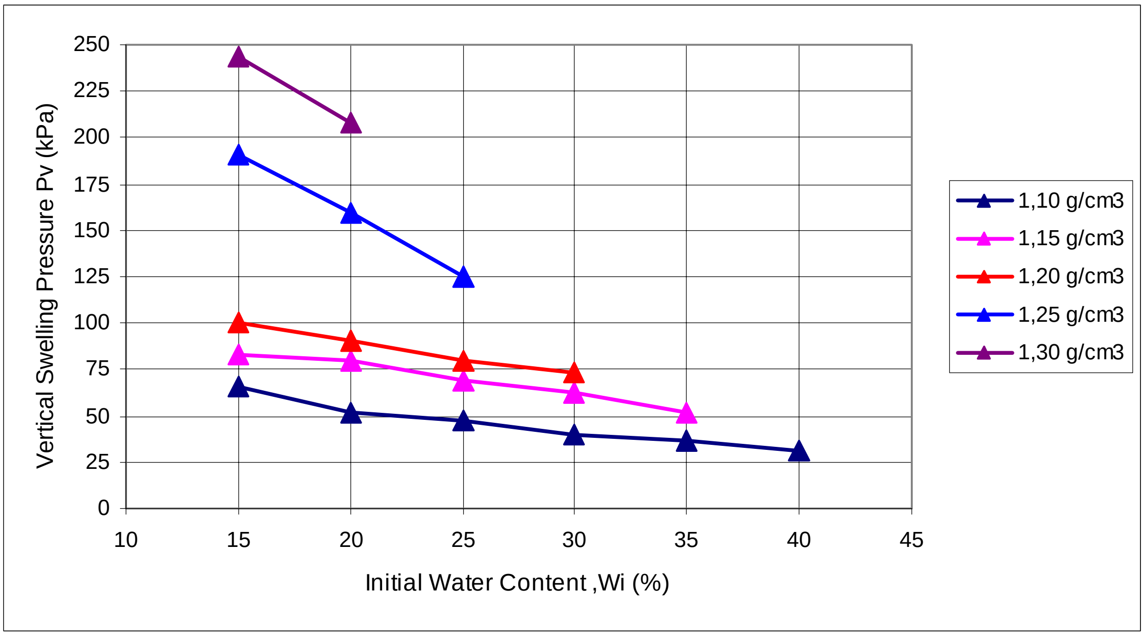

Figure 5.1 Relationship between the vertical swelling pressure and the initial water content for different initial dry densities.

The lateral swelling pressure decreases as the initial water content increases. Figure 5.2 shows relationship between the lateral swelling pressure and the initial water content for different initial dry densities.

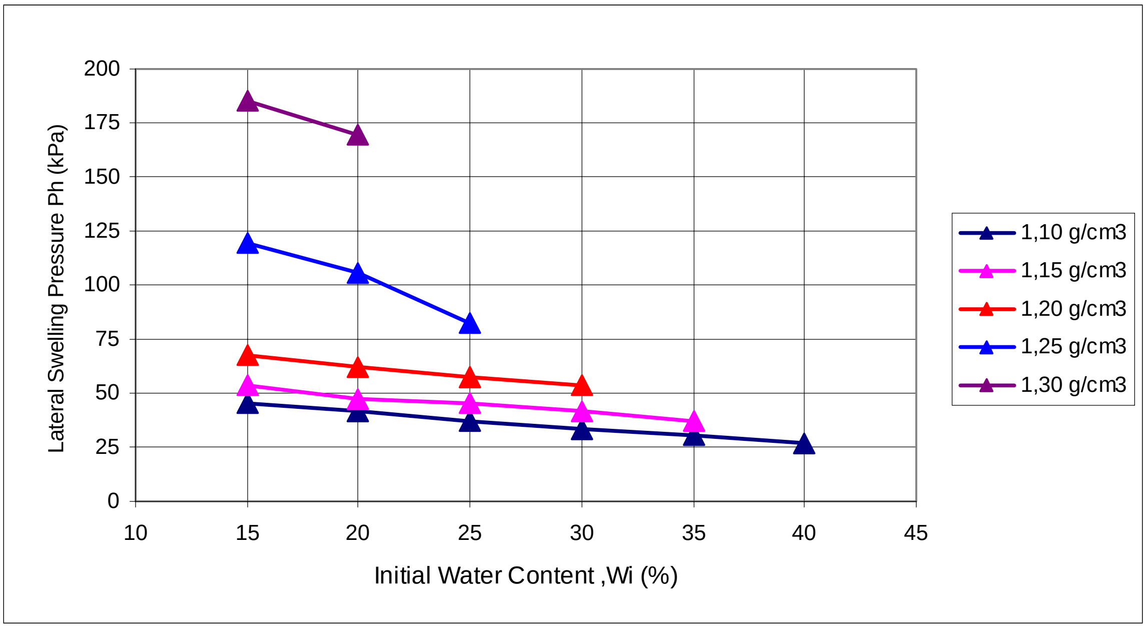

Figure 5.2 Relationship between the horizontal swelling pressure and the initial water content for different initial dry densities.

5.1.2 Effect of Initial Dry Density

Swelling pressure in both lateral and vertical directions are results of interparticle forces developed during swelling as a result of water absorption. As the density increases, for the same initial water content, there will be smaller volume for water particles to move. During water absorption into the soil media water particles will apply more force to the surrounding soil particles to achieve the complete saturation for higher initial dry densities. And consequently as the initial water content increases the swelling pressures in both directions will be higher as a result of this increased interparticle forces.

The vertical swelling pressure increases as the initial dry density increases. Figure 5.3 shows relationship between the vertical swelling pressure and the initial dry density for different initial water contents.

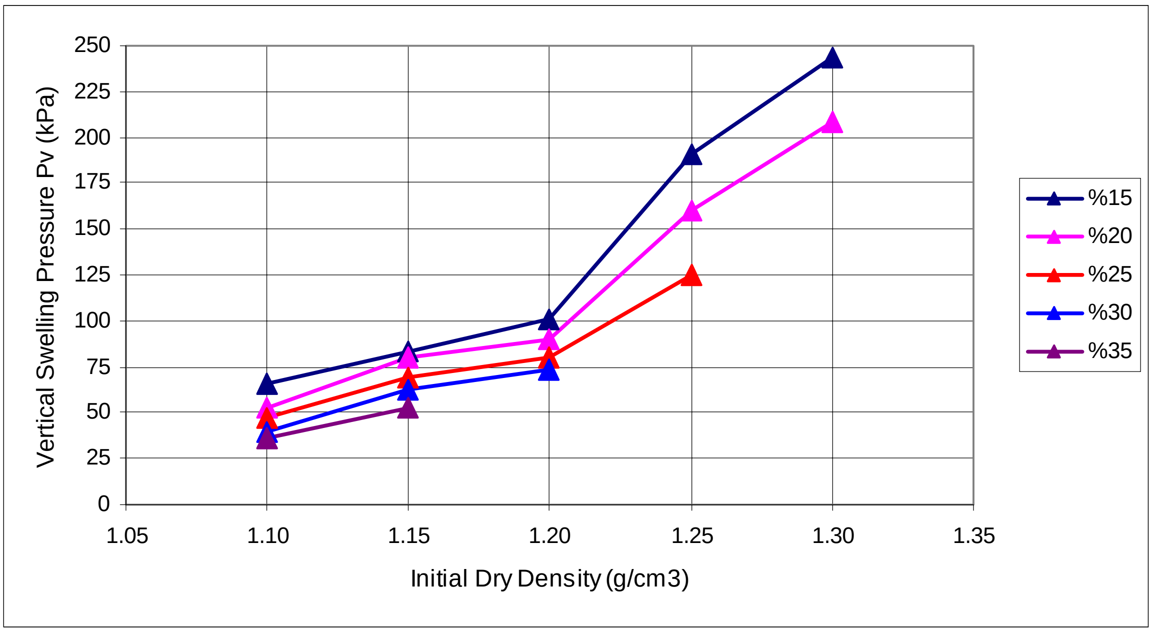

Figure 5.3 Relationship between the vertical swelling pressure and the initial dry density for different initial water contents.

The lateral swelling pressure increases as the initial dry density increases. Figure 5.4 shows relationship between the lateral swelling pressure and the initial dry density for different initial water contents.

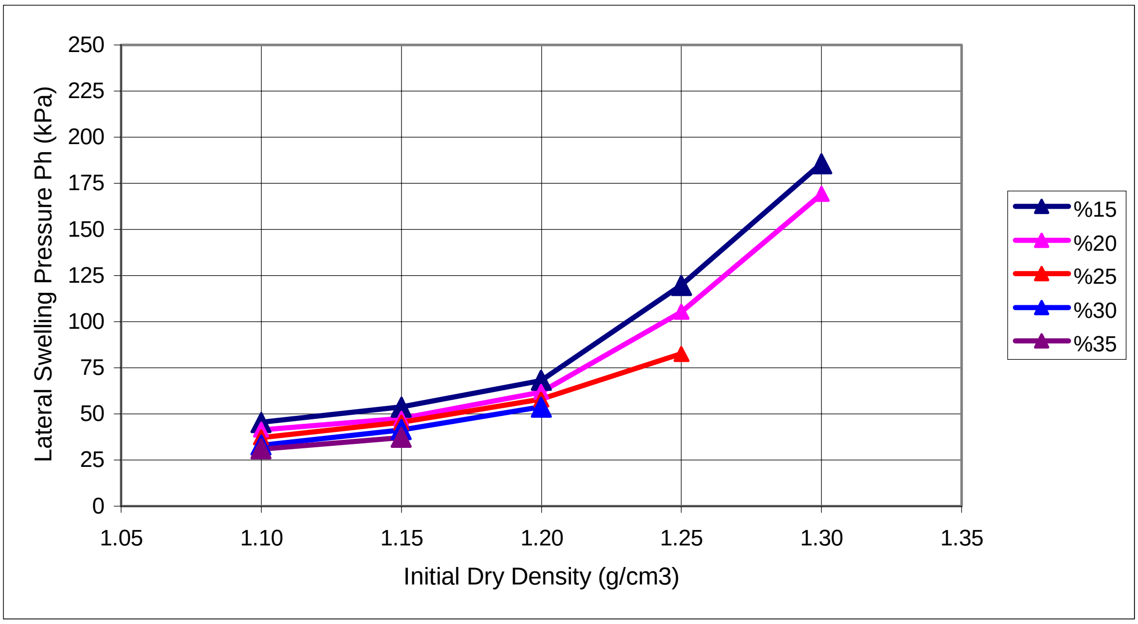

Figure 5.4 Relationship between the horizontal swelling pressure and the initial dry density for different initial water contents.

The rate of incease in the lateral and vertical swell pressures is higher at dry densities exceeding 1.2 g/cm^3 as shown in Figures 5.3 and 5.4.

5.2 COMPARISON OF LATERAL AND VERTICAL SWELLING PRESSURES

Swell pressure ratio - S_r is the ratio of lateral swelling pressure to the vertical swelling pressure as described in the previous sections. It is found that swell pressure ratio changes from 0,59 to 0,86 for the soil example used in the experimental work. The test results indicate that the magnitude of vertical swelling pressures are higher than the magnitude of the lateral swelling pressures for that soil sample under this test conditions. Erol and

Ergun (1994) reported that at the end of the CVS tests with thin wall oedometer technique, using similar testing conditions with this study, swell pressure ratios are changing between 1,0 and 1,55. This is a result of different swelling characteristics of different soil samples. In Table 5.1 comparison of these samples can be seen.

Table 5.1 Comparison of tested sample with the one used by Erol and Ergun (1994)

	Tested Sample	Erol and Ergun (1994)
Liquid Limit , LL	87,83 %	110 %
Plasticity Index , PL	32 %	85 %
Clay Content , C %	44,17	61 %

The data trends indicate that the swell pressure ratio increases with both increasing initial water content and the initial dry density. However this increase in swell pressure ratio is more dependent on dry density changes.

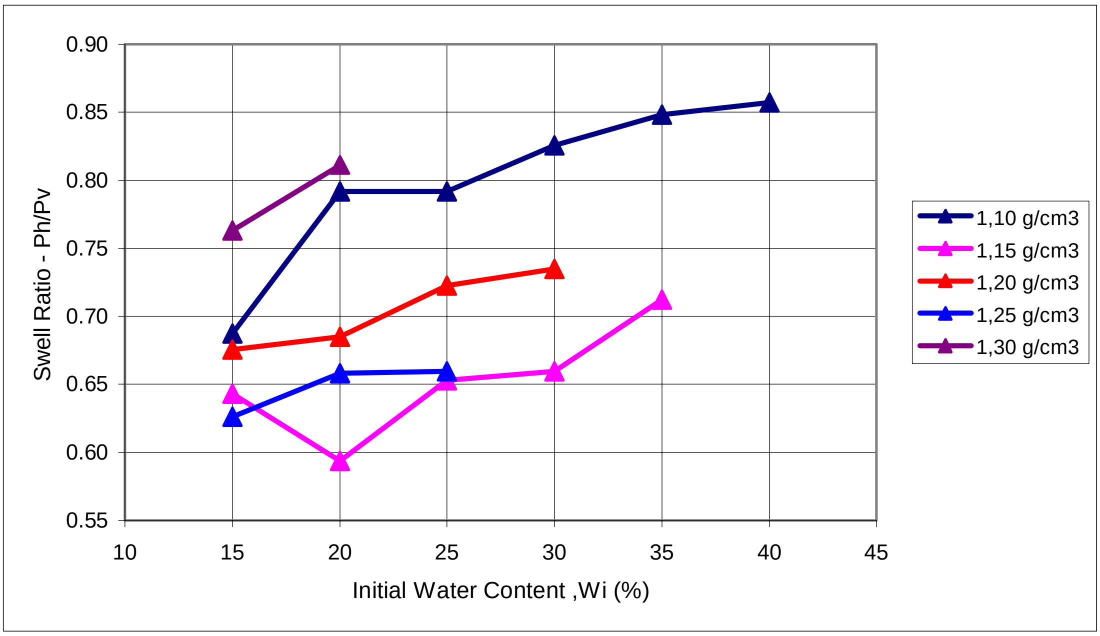

Figure 5.5 Relationship between the swell pressure ratio - S_r and the initial water content for different initial dry densities.

5.3 REBOUND CHARACTERISTICS DURING CVS TESTS

Rebound characteristics were obtained by unloading the specimens at the end of swelling tests. From this unloading data e – log p curves were drawn as it can be seen from Figures at Section 4.8. It is found that the e – log P behaviour in the unloading phase is a straight line in all tests performed in this study. Robertson and Wagener (1975) stated that; each clay type exhibits a unique void ratio – swell presssure relationship which depends not only on its clay minerology but also on the spatial arrangement of clay particles and packets, and if the void ratio of the clays as existing in the field, are measured, then the potential maximum swell pressure can be predicted from the void ratio – swell pressure relationship. The slope of e – log p curve during unloading phase of constant volume swell (CVS) test is defined as swell index, C_s. Swell index is used to determine swell under vertical surcharge. It is observed that swell index, C_s is increasing with increasing initial water content as shown in Figure 5.6 for initial dry density of 1,20 g/cm^3 and in Figure 5.7 for initial dry density of 1,25 g/cm^3. And the magnitudes of C_s are given in Table 5.2 and in Table 5.3 respectively.

Table 5.2 Swell index values for initial dry density of 1,20 g/cm^3

Initial Water Content (%)	25	30
Initial Dry Density (g/cm^3)	1,20	1,20
Swell index, C_s	0,044	0,050

Table 5.3 Swell index values for initial dry density of 1,25 g/cm^3

Initial Water Content (%)	20	25
Initial Dry Density (g/cm^3)	1,25	1,25
Swell index, C_s	0,028	0,032

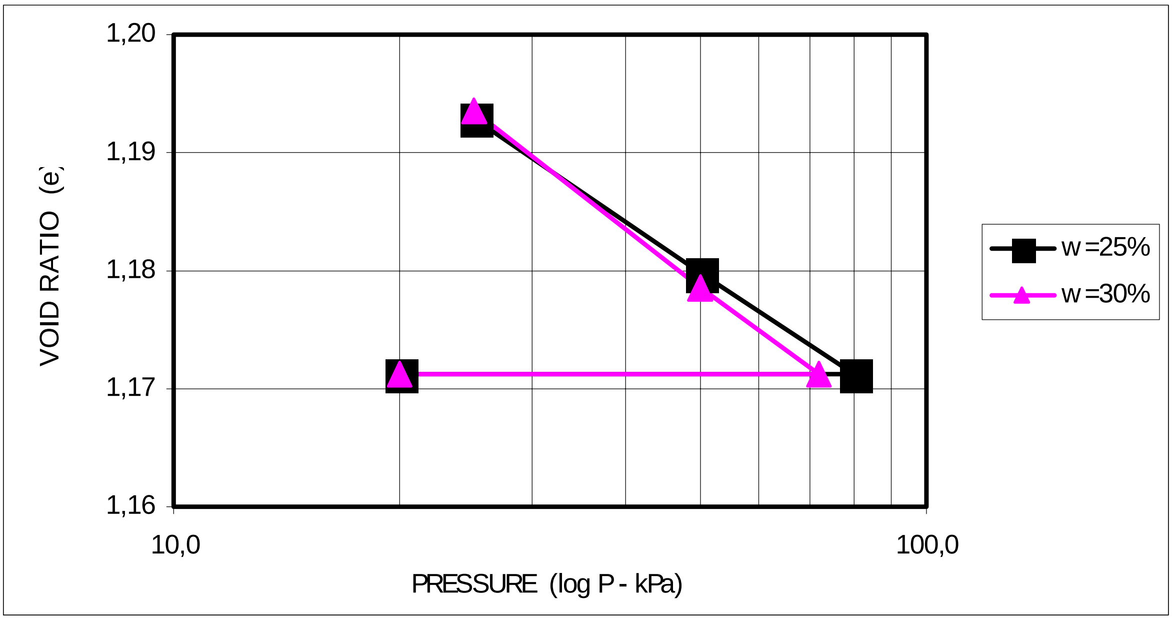

Figure 5.6 e – log p curves for initial water contents of 25% and 30% for initial dry density of 1,20 g/cm^3

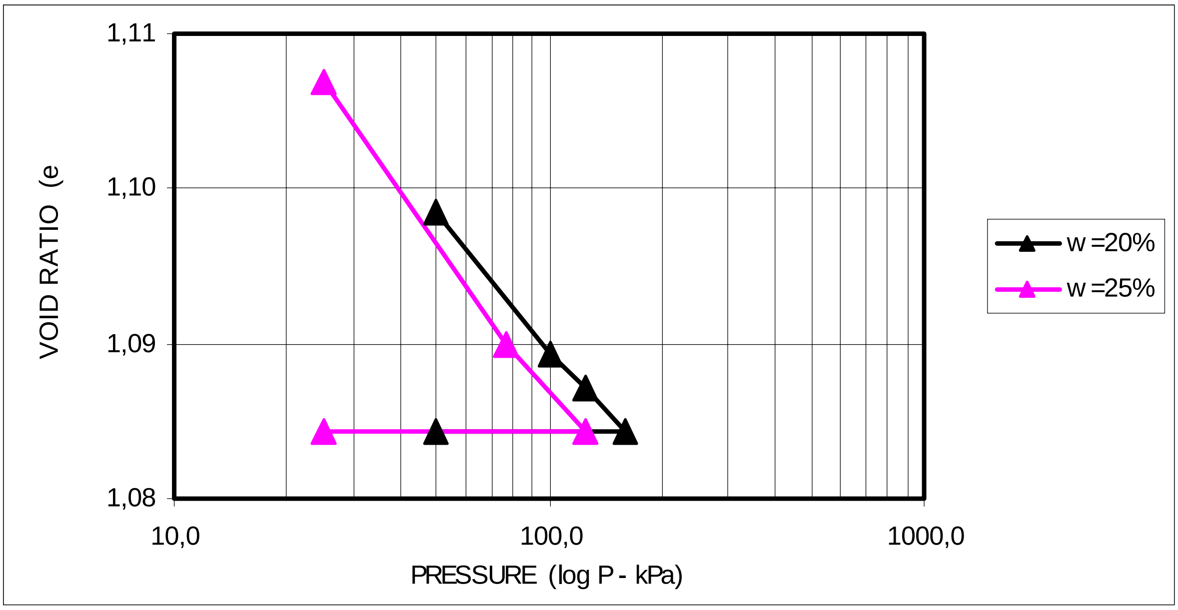

Figure 5.7 e – log p curves for initial water contents of 20% and 25% for initial dry density of 1,25 g/cm^3

5.4 RATE OF DEVELOPMENT OF SWELL PRESSURES

As explained before the main reason of swelling behaviour is water absorption of soil mass in time. And the soil suction governs the time needed for water to be absorbed to reach to the complete saturation. Seed (1961) explained this behaviour as; when a compacted soil specimen is exposed to water, time is required for the movement of water into the specimen under the hydraulic gradient set up by the negative water pressure within the soil relative to free water i.e., the soil suction. The amount of swell that occurs within a given period of time depends on the quantity of water that enters the soil; thus, the rate of swell is proportional to the hydraulic gradient and the conductivity. These quantities, in turn, are influenced by the soil structure, or by the treatment during compaction. It is found that rate of development of lateral and vertical swell pressures for a compacted specimen is different. Figure 5.8 and Figure 5.9 shows development of vertical and lateral swell pressures in time respectively for initial water content of 15 % and for five different initial dry densities.

As it can be seen from both figures that at the time of inundation, there is a sharp increase in both vertical and lateral swelling pressures. A gradual decrease in rate of swell pressure is observed, attaining equilibrium in 5000 to 8000 minutes. It is also observed that the vertical swelling pressure attains equilibrium faster than the lateral swelling pressure. This behaviour may be due to the measuring system characteristics. Because the sample is wetted from top and bottom but the lateral strain measurements are done at the middle of the sample by the strain gauges. The complete saturation takes time. It can be sad that vertical swelling pressure development may be faster than lateral swelling pressure development due to the longer time required for the development of lateral swelling pressure in the middle of the sample.

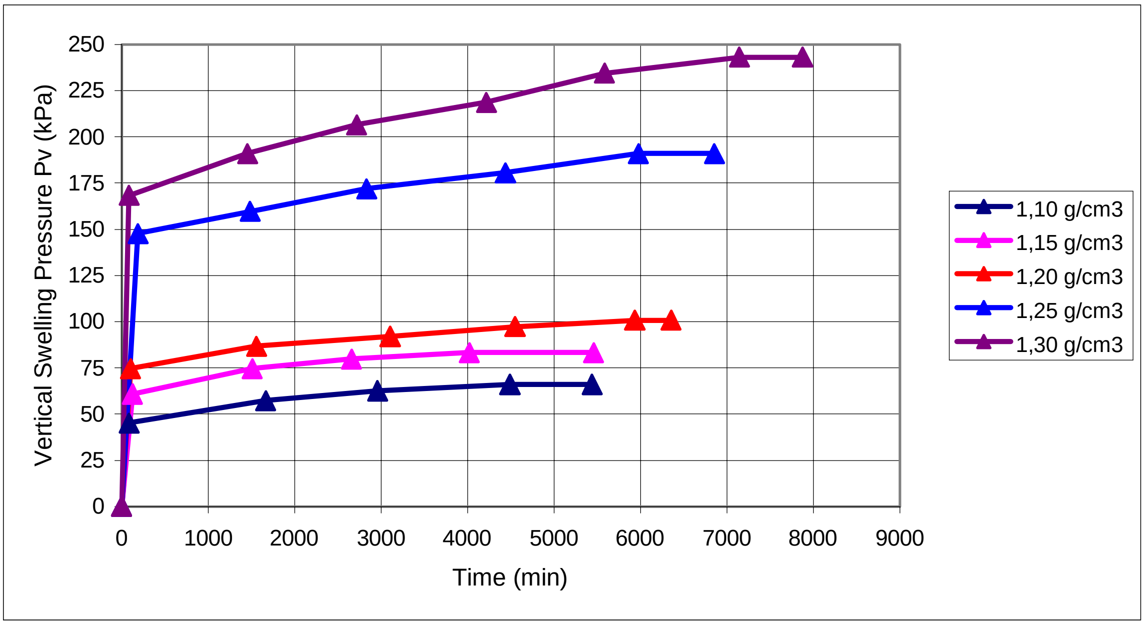

Figure 5.8 Development of vertical swell pressures in time for initial water content of 15 % and for five different initial dry densities.

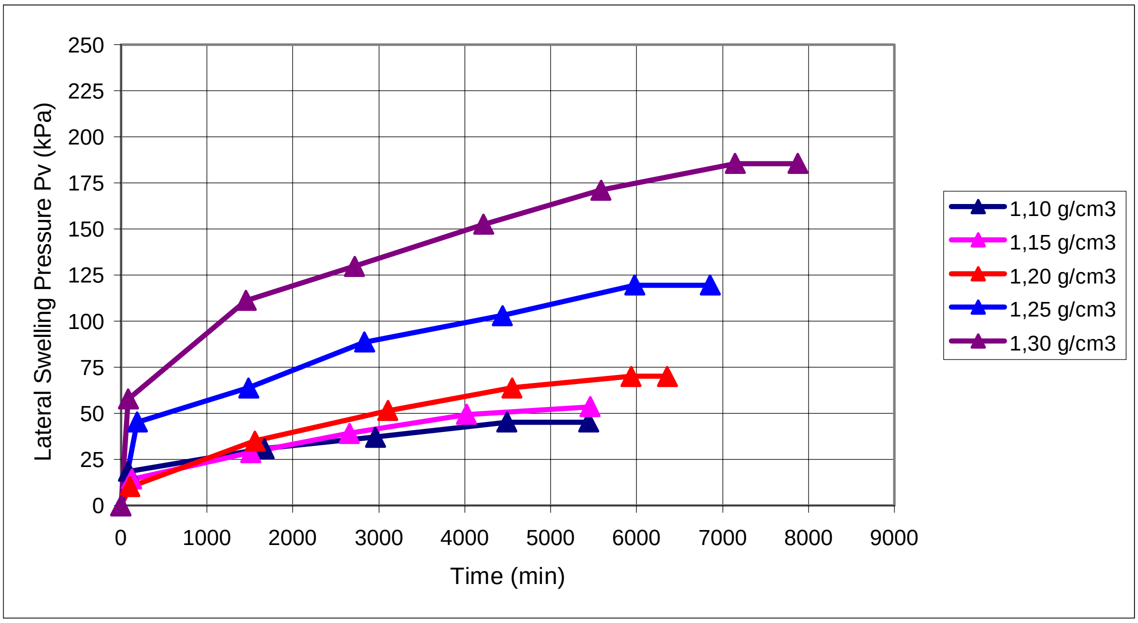

Figure 5.9 Development of horizontal swell pressures in time for initial water content of 15 % and for five different initial dry densities.

CHAPTER 6

CONCLUSIONS

A thin wall oedometer test set – up containing strain gauges at the mid point of its thin wall of 0,35 mm thickness to measure the lateral strains developed at the mid point of the sample and also containing a read – out unit which is used to convert the lateral strains to the lateral stresses after calibration process. Vertical swelling pressures are also measured for statically compacted clayey soil samples.

Constant volume swell (CVS) tests are performed, with unloading steps, on samples having different initial water contents and different initial dry densities. The following conclusions can be drawn from the test results:

1. Lateral and vertical swelling pressures decrease with increasing initial water content for the samples having the same initial dry unit weight.

2. Lateral and vertical swelling pressures increase with increasing initial dry unit weight for the samples having the same initial water contents.

3. There is a linear relationship between the void ratio and the logarithm of vertical swelling pressure. Swell index, C_s is increasing with increasing initial water content for samples having the same initial dry density.

4. The magnitude of lateral swelling pressures developed in CVS tests were smaller than the magnitudes of the vertical swelling pressures, and the swell pressure ratios vary over a range from 0,59 to 0,86 for the clay sample and for the initial water content and initial dry density ranges used in these

experimental work. Swell ratio, Sr is increasing with increasing initial water content for samples having the same initial dry density.

5. Vertical swelling pressures attain equilibrium faster than the lateral swelling pressure due to the characteristics of the test set – up.

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