DETERMINING THE BEHAVIOUR OF CLAY UNDER REPEATED LOADING CONDITION USING THE CRS-OEDOMETER

Thesis by

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A B S T R A C T

DETERMINING THE BEHAVIOUR OF CLAY UNDER REPEATED LOADING CONDITION USING THE CRS OEDOMETER

Over the past several years, considerable advance have been made in understanding and reasonably predicting the behaviour of soil during repeated loading with different types of apparatus. In this investigation, the consolidation characteristics of remoulded soft clay are studied under repeated loading conditions with the application of low constant rate of strain. Tests are carried out with CRS consolidation apparatus. In this research the variables chosen and their levels were as follows ; sustained load with (A: $\sigma_o = 0.5$, B: $\sigma_o = 1.0$, C: $\sigma_o = 2.0$ kg/cm), load intensity (dynamic load) in percent with (I: $\frac{\sigma_o}{\sigma_d} = 20\%$, II: $\frac{\sigma_o}{\sigma_d} = 40\%$, III: $\frac{\sigma_o}{\sigma_d} = 60\%$) and number of cycles with (α: N=3, β : N=4, γ: N=5).

The tests results with respect to chosen variables show that the dynamic void ratio decrease of a soil reach its maximum value under 1 kg/cm of sustained load. The kult and k_{max} values are mostly effected due to the increase of the sustained load.

Ö Z E T

Geçmiş yıllarda, çeşitli tipte aletleri kullanarak tekrarlı yükler altındaki zeminin davranışının anlaşılması ve sebebinin önceden tahmin edilebilmesi bakımından epeyce ilerleme kaydedilmiş bulunmaktadır. Bu araştırmada, sabit birim deformasyonlu tekrarlı yükler altındaki örselenmiş yumuşak kilin konsolidasyon karakteristikleri üzerinde çalışıldı. Deneyler CRS konsolidasyon aleti ile yapıldı. Bu araştırmada, seçilen değişkenler ve değerleri şunlardır: Devamlı yük (A: $\sigma_o = 0.5$, B: $\sigma_o = 1.0$, C: $\sigma_o = 2.0$ kg/cm^2), yük yüzdesi (dinamik yük) yüzde olarak (I: $\frac{\sigma_o}{\sigma_d} = 20$ %, II: $\frac{\sigma_o}{\sigma_d} = 40$ %, III: $\frac{\sigma_o}{\sigma_d} = 60$ %) ve tekrarların sayısı (α : N=3, β: N = 4, γ: N=5).

Seçilen değişkenlere göre deneylerin sonuçları, zeminin dinamik boşluk oranı azalmasının 1 kg/cm^2'lik devamlı yük altında maksimum değerine ulaştığını göstermektedir. k_{ult} ve k_{max} değerleri birinci derecede devamlı yükün artması ile etkilenmektedirler.

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a_v (cm^2/kg)	=	Coefficient of Compressibility
C_v (cm^2/kg)	=	Coefficient of consolidation
CRS	=	Constant Rate of Strain
e	=	Void ratio
e_o	=	Initial void ratio.
e_{od}	=	The volue of void ratio at the beginning of the dynamic load.
Δed	=	The difference between the values of void ratio at the beginning and end of dynamic loads.
g	=	Gravity.
H	=	Sample height.
i	=	Hydraulic gradient.
k (cm/min)	=	Coefficient of Permeability.
L.L	=	Liquid limit.
M	=	Modulus
m_v (cm^2/kg)	=	Coefficient of volume compressibility.
N	=	Number of cycles.
P.L	=	Plastic limit.
P.I	=	Plasticity index.
Tv	=	Time factor.
t	=	Time
u	=	Pore water pressure.

u_b = Pore water pressure at undrained end in oedometer.

v = Velocity of water.

w = Water content.

w_L = Liquid limit.

w_p = Plastic limit.

z = Depth.

$\bar{\epsilon}_1$ = Compressive strain.

γ_s = Unit weight of solid particles.

γ_w = Unit weight of water.

σ = Total stress.

σ' = Effective vertical stress.

σ_o = Sustained load.

CHAPTER 1 INTRODUCTION

One of the basic problems in foundation design of offshore structures, oil tanks, and silos is the persistent repeated loading of the subsoil. There are many engineering situations where soils are subjected to repeated loading essentially with undrained conditions. Much research related to the traffic loading problem has been performed on partially saturated compacted soils. And much of the research related to loading from vibrating machinery or from earthquakes has involved high frequencies and very low strains.

Such studies were performed using different types of test apparatus to clarify the nature of the phenomena affecting the overall deformation characteristics of soils.

Compression characteristics of soft clays are generally determined by oedometer tests. The oedometer apparatus, which usually contains the soil sample in a confining ring, has been used over a long period of time. The testing procedure proposed by Terzaghi has been widely used since 1925.

Strain rate effects have been evaluated and theories for evaluation of the test have been presented. The constant rate of strain tests with continuous loading have been used since last fifteen years.

In this investigation the data available from tests are used to continuously compute void ratio, e , the coefficient of consolidation, C_v , for the behaviour of clay under repeated loading conditions using a CRS oedometer apparatus.

The loads applied by the loading and unloading of oil tanks, silos etc. apply a slowly changing load to the foundation soil allowing consolidation to take place.

In this research we tried to simulate this effect of such kind of loading on soft soils, by using the CRS - oedometer apparatus for the application of repeated loading.

CHAPTER 2 PREVIOUS RESEARCH

2.A CLAY MINEROLOGY

2.A.1 Soil Structure and Fabric

Soil structure is both the geometric arrangement of the particles, or mineral grains, and the interparticle forces which may act upon them. Soil structure includes gradatition, arrangement of particles, void ratio, bonding agents, and associated electrical forces. The structure is the property which produces a response to external changes in the environment, such as loads, water, temperature and other factors.

Soil fabric is a more recently introduced term to describe the " structure" of clays. Fabric denotes the geometric arrangement of the mineral particles in a clay mass as observed optical or electron microscopes. The particles arrangement includes particle spacing and pore size distributions. (2)

2.A.2 Clay and Clay Minerals

Clay minerals are predominantly silicates of alimnum and/or iron and magnesium. Most of the clay minerals have sheet or layered structures. Soil masses generally contain a mixture of several clay minerals named for the predominating clay mineral with varying amounts of other nonclay minerals.

Clay minerals are very small (less than 2μm) and very electrochemically active particles which can be seen using an electron microscope only with difficulty. Inspite of their small size, however, the clay minerals have been studied extensively (Grimm, 1968; Mitchell, 1976) due to their economic importance, particularly in ceramics metal holding, oil field usage, and engineering soil mechanics. The clay mineral exhibit characteristics of affinity for water and resulting plasticity not exhibited by other materials even though they may be of the clay size or smaller.

There are two foundamental building blocks for the clay mineral structure. One is a slica units in which four oxygens form the tips of a tetrahedron and enclose a silicon atom, producing a unit approximately 4.6 Å.
(Angstrom unit Å 10^{-10}m.) (2)

2.A.3 General Clay Mineral Properties

Several characteristics are similar for all the clay minerals.

2.A.3.a Hydration

Clay particles are almost always hydrated, i.e., surrounded by layers of water molecules called adsorbed water. This layer is often at least two molecules thick and is called the diffuse layer. the double diffuse layer, or simply the double layer. This water is firmly attracted and/or contains metallic ions. A diffusion of the adsorb water cations from the clay mineral extends, outward from the surface of the clay into the adsorbed water layer. the effect of this is to produce a net (+) charge near the mineral particle and (-) charge a greater distance. This diffusion of cations is a phenomenon very similar to the diffused interface between a free water surface and the admosphere where the diffused material is water molecules. This water is often so firmly attracted it behaves more as a solid than as a liquid.(2

2.A.3.b Activity

The edges of all the clay minerals have net negative charges. This results in attempts to blance the charges by cation attraction. The attraction will be in proportion to the net charge deficiency and may be related to the activity of the clay. The activity may be defined as

$$\text{Activity} = \frac{\text{Plasticity index Ip}}{\text{Percent clay}}$$

where the percent clay is taken as the soil fraction $< 2\mu m$. Activity is also related to relative potential water contens. Typical activity values based on eq (1) are as follows

Kaolinite 0.4 - 0.5

Illite 0.5 - 1.0

Montmorillonite 1.0 - 7.0

Activity interms of volume change is a principal concern in evaluating the soil for use in earthworks and foundations.(2)

2.A.3.c Water Effects

The water phase of clay soils is not very likely to be chemically pure water. This water accounts for the plasticity properties of clay. In laboratory tests for the Atterberg limit it is specified by ASTM that distilled water be added as required. The use of distilled water, which is relatively ion free, may produce results somewhat different from those obtained using a more contaminated water such as may enter the soil insitu.

A particular phenomenon of clay is that a clay mass which has dried from some initial water content forms a mass which has considerable strength. If these lumps are broken down to elemantal particles, the material behaves as a cohesionless particulate medium.(2)

2.A.4 Forces of Water Retention in Soils

The potential of water can be specified without regard to the forces by which water is retained in soils. However, it is also necessary to understand these forces. They vary depending on the dominate force in clays.

CLAYS

Clays are characterized by the volume changes which accompany water content changes. Clays which are not aggregated into stable crumb units remain water-saturated to high values of suction. A decrease in water content is accompanied by an equal decrease in volume, and air water interfaces are not present except at the boundaries of the sample or at the cracks. Air enters the sample the volume has been reduced to where interparticle interference prevents shrinkage. On rewetting, the samples swell or increase in volume due to swelling forces. The forces of water retention are the same as the swelling forces. In the limitting case of high swelling clays in parellel orientation, soil suction is numerically equal to swelling pressure at any water content.

Subsoils with a high content of clay, especially if it is monmorillonite, have water retention characteristics which approach this discription.They have no crumb structure and are slowly permeable to water.(3)

2.A.5 Volume Change in Clay Soils

Swelling

Volume increase due to swelling does not always accompany water content increase on rewetting of a soil. A dry soil can take up water, with air in the voids being replaced by water, without a consequent increase in volume. This occurs typically for sandy and silty soils. Swelling requires a force of repulsion separating clay particles to increase the volume as the water content increase. (3)

2.A.5.a Soil Characteristics Affecting Swelling

The amount of swelling depends upon the clay minerals and their arrangement or orientation in the clay soil, as well as upon physical chemical properties such as valance of exchangeable cations, pore-water salt concentration and cementing bonds between clay particles and with decreasing valence of exchangeable cation. Specific differences between clays within a mineralogical group give rise to smaller differences in swelling.

Cementation between particles is a major factor in limiting volume increase of clays on swelling. Iron hydroxides, carbonates and various organic molecules are the cementing materials. It is not clear in many cases whether these materials bonds between particles to form a restrained to swelling, or whether they affect the physical-chemical properties of the surface in such a way as to reduce the swelling force iron salts dried into a clay can markedly reduce swelling; this occurs in some "non swelling" montmorillonites.

2.A.5.b Swelling Pressure of Soils

Swelling clays exert pressure against a confining load when water is available for a volume increase. This pressure exceeds usual loading, so that volume change is decreased but not prevented by the structure which the soil supports. Except for high swelling clays, the swelling pressure decreases rapidly with small volume increase, and the amount of swelling under load is usually small.

The dependence of swelling pressure on volume change makes a precise measurements of swelling pressure difficult. Unless special precautions are taken in the measurements to prevent volume change, measured pressures will underestimate the swelling pressure.

2.A.5.c Mechanism of Swelling

Swelling, or a volume change on wetting requires an attraction of water to the clay to provide an effective repulsion between clay particles. From thermodynamic considerations, water will move into the soil as long as the free energy of the water, in the soil is less than that of free water. Water fills the voids of sand without a significant increase in volume, but for clays this wetting usually results in a volume increase. The reason is sought in the surface properties of clays.

2.A.6 Compressibility of Clay

The compressibility of clay is the relationship between the effective stress (i.e., stresses between particles) developed in the clay soil and the corresponding decrease im void ratio. For soils that are partially saturated the decrease in void ratio due to a combination of solution of air into the pore water and extrusion of both water and air.The final equilibrium condition that must be achieved is such that the air and water in the soil voids do not carry any stress,i.e., that no pore pressure remains.

For fully-saturated clays, the mechanism by which compression occurs and during which generated pore pressures are dissipated to reach an ultimate state of zero pore pressure is called consolidation is the compression that results when a load applied to a saturated clays gives rise to a compression, the magnitude of which is determined when the generated pore pressures are fully dissipated. (3)

2.B Consolidation Theory

2.B.1 Definition

The conventional method for estimating both the rate and amount of settlement of a compressible soil subjected to one-dimensional strain is to take samples of the soil and perform "standard consolidation tests."

This type of tests is described in detail by lambe essentially, the load is applied in a series of increments with displacement recorded at appropriate time intervals. Based on the Terzaghi's theory of one-dimensional consolidation the displacement versus time curve for a given load allows the coefficient of consolidation, C_v, to be derived from which the rate of settlement in the field can be estimated. Each increment is allowed to remain on the sample for one day, at which time the sample has essentially come to "equilibrium" and the displacement corresponding to the one day reading is recorded. This allows the void ratio, e vs the normal effective stress, σ' to be calculated with corrections for sample disturbance, this e vs σ' relationship can be used to estimate the ultimate settlement for the field conditions.

Field evidence has indicated that while the ultimate settlement may be predicted with reasonable accuracy, estimates of the rate of settlement are likely to be very much in error.

There are a number of reasons why the rate of settlement may be in error.

(1) The samples tested may not be representative.

(2) Drainage may not be one-dimensional as assumed.

(3) The assumptions of the Terzaghi theory may not be valid.

2.B.2 Terzaghi's Theory of Consolidation

Terzaghi's theory of consolidation is based on the following assumptions.

(1) Material is continuous

(2) Material is homegeneous

(3) Fluid and solids are incompressible

(4) The soil is saturated (S=100%)

(5) Flow of water is one-dimensional

(6) Darcy's law is valid (v=k.i)

(7) There is a constant temperature.

The time-settlement relation using the above assumptions can be obtained as follows.

$$\frac{k}{\gamma_w} \frac{\partial^2 u}{\partial z^2} = \frac{\Delta \dot{V}}{V} = \frac{1}{1+e} \frac{\partial e}{\partial t} \qquad (1)$$

where

k = the coefficient of permeability

γ_w = the unit weight of water

u = the excess pore water pressure

z = the space variable

e = the void ratio

t = the time

$\frac{\Delta \dot{v}}{v}$ = the volumetric strain rate.

the above assumptions are perfectly reasonable except possibly for the assumption of the validity of Darcy's law at very low gradients that are likely to occur in the field.

Terzaghi's contribution essentially lay in relating void ratio and effective stress. Although he realized that the void ratio effective stress relationship was complex, to facilitate the solution of egn.1 he assumed the following simple relationship for a small increment of load.

$$\partial e = -a_v \partial \sigma' \tag{2}$$

where a_v the gradient of the e vs σ' curve.

There are two important concepts involved in this assumption.

(1) A linear relationship between void ratio and effective stress exists for small load increments.

(2) The void ratio depends only on the effective stress and not on time or strain rate. Itis this assumption that prevents the Terzaghi's theory from modelling "secondary compression".

Inserting the effective stress value $\sigma' = \sigma - U$ and egn. 2 into egn.1 yields the following equation.

$$\frac{k}{\gamma_w} \frac{\partial^2 U}{\partial z^2} = -\frac{a_v}{1+e} \left(\frac{\partial \sigma}{\partial t} - \frac{\partial U}{\partial t} \right) \tag{4}$$

In the load controlled test $\frac{\partial \sigma}{\partial t} = 0$, hence

$$\frac{k}{\gamma_w} \frac{\partial^2 U}{\partial z^2} = \frac{a_v}{1+e} \frac{\partial U}{\partial t} \tag{5}$$

Terzaghi further assumed that for any one increment

$\frac{k(1+e)}{a_v \gamma_w} = C_v$, would be constat, hence Eqn.5 reduce to

$$C_v \cdot \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} \qquad (6)$$

Equation (6) is the standard "heat equation" for which the solution is well known. Terzaghi, therefore, was responsible for the simplifying assumptions that left to Eqn.6. (1)

2.B.3 Evaluation of Strain Controlled Systems

If we investigate the same phenomena using strain controlled loading system the following calculations can be done.

Returning to the basic equation of consolidation given by Eqn.1.

$$\frac{k}{w} \cdot \frac{\partial^2 u}{\partial z^2} = \frac{\Delta \dot{v}}{v} = \frac{1}{1+e} \cdot \frac{\partial e}{\partial t}$$

Suppose a sample of soil is strained so slowly that change in void ratio is uniform throughout the sample, i.e., all elements of soil lose the same amount of water in any given time interval, then the right hand side of Eqn.1 is not a function of z and therefore Eqn.1 can be integrated quite simply.

For the boundary conditions

$u = 0$ at $z = H$

and $\frac{\partial u}{\partial t} = 0$ at $z = 0$

the following expression for the excess pore pressure at any time is obtained.

$$U(Z) = -\frac{1}{2} \cdot \frac{\gamma_w}{k} \cdot \frac{\Delta \dot{v}}{v} \cdot (H^2 - Z^2) \qquad (7)$$

The excess pore pressure at the base (z 0) is given by

$$U_b = -\frac{1}{2} \cdot \frac{\gamma_w}{k} \cdot \frac{\Delta \dot{v}}{v} \cdot H^2 \qquad (8)$$

and since the excess pore pressure varies with z according to a parabolic function, the average pore pressure throughout the sample is given by

$$U_{av} = -\frac{1}{3}\cdot\frac{\gamma_w}{k}\cdot\frac{\dot{\Delta v}}{v}\cdot H^2 \qquad (9)$$

The volumetric strain rate will be negative for all cases where the volume of the sample is decreasing with time and hence a positive excess pore pressure with result as expected.

If the excess pore pressure at the base of the sample is measured,then eqn.8 allows the coefficient of permeability to be calculated.

$$k = -\frac{1}{2}\cdot\frac{\gamma_w}{U_b}\cdot\frac{\Delta v}{v}\cdot H^2 \qquad (10)$$

Eqn.9 allows the average effective stress to be calculated if the total stress is known,as is generally the case,i.e., $\sigma_{av} = \sigma - U_{av}$.

Equations 7 to 10 inclusive are applicable to all cases of one-dimensional drainage.Examples of such cases wouldbee slow drained triaxiall tests with either one or three-dimensional strain, slow drained plane strain tests and strain-controlled consolidation tests. The only assumption other than the basic assumption of equation 1 is that tha strain rate sould be sufficiently slow that the change in void ratio is essentially constant throughout the sample.No assuption relating void ratio to effective stress is necesary.

The application of this theory has been tested experimentally for drained traxial tests (Byrne[2]) and plane strain tests.

2.B.4 Evaluation of Constant Rate of Strain Test

If a constant rate of compression is applied to the surface of a sample contained in an oedometer ring the total stress and effective stresses progresively increase as the test proceeds (Smith and Wahls,1969;Wissa et al.,1971).

Commencing with the linear form of eqn.

$$-\frac{k}{\gamma_w}\cdot\frac{\partial^2 u}{\partial y} = \frac{\partial \Delta v}{\partial t} = \frac{\partial \varepsilon_1}{\partial t} \qquad (\mathrm{I})$$

and differentiating eqn

$$\Delta v = \varepsilon_1 = \frac{e_o - e}{1+e} = m_v\left(\sigma_v' - \sigma_{vo}'\right) \qquad (\mathrm{II})$$

twice with respect to y after substituting for σ_v' and using eqn $\sigma_v = \sigma_v' + u + u_1$

we get

$$\frac{\partial^2 \varepsilon_1}{\partial y^2} = -m_v \frac{\partial^2 u}{\partial y^2} \qquad (\mathrm{III})$$

and substituting into (I) gives

$$C_v \frac{\partial^2 \varepsilon_1}{\partial y^2} = \frac{\partial \varepsilon_1}{\partial t} \qquad (\mathrm{IV})$$

which is the most convinient form for solution since boundary displacements rather than pore pressure are specified. It is also convenient to measure y vertically downwards from the original position of the upper surface.

The permeable upper surface is moving at a constant rate, r, given by the expression,

$$r = \frac{s}{H_t} = \frac{\bar{\epsilon}_1}{t} \qquad (V)$$

where s is the surface displcement, H the total thickness of the soil layer and t the time relative to the commencement of the test. $\bar{\bar{\epsilon}}_1$ is the average strain. The base of the layer is fixed and impermeable, the top permeable.

The solution for strain at time t is

$$\epsilon_1 \quad \epsilon_1 \left[1 + F\left(\frac{y}{H}, T_v\right)\right] \qquad (VI)$$

where

$$F = \frac{1}{T_v}\left[\frac{1}{3} - \frac{Y}{H} + \frac{1}{2}\left(\frac{Y}{H}\right)^2\right] - \frac{2}{\pi^2 T_v}\sum_{n=1}^{n=\infty}\frac{1}{n^2}\cos\frac{n\pi y}{H}\exp(-n\pi^2 T_v)$$

in which T_v is the usual definition of time factor

$$T_v = \frac{C_v \cdot t}{H_D^2} \qquad (VII)$$

with

$H_D = H$

This solution provides a technique for calculating the value of c for the range of effective stresses by applying a constant rate of displacement to the upper platen of a one-dimensional consolidation cell. In principle this technique has advantages in common with the constant rate of total stress test compared with the conventional oedometer test. It offers a grater degree of control of the testing conditions. Since

(a) the sample progresively instead of incrementally loading.

(b) the sample can be sealed

(c) the gradients of pore pressure are controlled, and the test can be completed much more rapidly than the standard oedometer test with the 24 hour loading periods.

At earlies time ($T_v < 0.5$) the ratio of strain at the base to the strain at the top is,

$$\frac{\epsilon_1(H,t)}{\epsilon_1(0,t)} = F_3 \qquad \text{(VIII)}$$

where F_3 is obtained from eqn.VI as

$$F_3 = \frac{T_v - \frac{1}{6} - \frac{2}{\pi^2}\sum \frac{\cos n}{n^2} \exp\left(-n^2\pi^2 T_v\right)}{T_v + \frac{1}{3} - \frac{2}{\pi^2}\sum \frac{1}{n^2} \exp\left(-n^2\pi^2 T_v\right)} \qquad \text{(IX)}$$

The strain ratio can also be expressed from eqn.II as

$$F_3 = \frac{\epsilon_1(H,t)}{\epsilon_1(0,t)} = \frac{(\Delta\sigma_{ye} - u_b)}{\Delta\sigma_{ye}} \qquad \text{(X)}$$

assuming m_v = constant where u_b is the base pore pressure and $\Delta\sigma_{ye}$ is the stress increment over time t.

Values of total stress increment, $\Delta\sigma_{ye}$, and base pore pressure, u_b , are continuously recorded throughout the test. At a time corresponding to $T_v < 0.5$ the value F_3 can be determined from eqn.X and the corresponding T_v can be obtained from eqn.IX and

$$C_v = \frac{T_v H_D^2}{t}$$

at larger times the expression VI simplifies to

$$\epsilon_1 = \bar{\epsilon}_1 \left[1 + \frac{1}{T_v}\left(\frac{1}{3} - \frac{Y}{H} + \frac{1}{2}\left(\frac{Y}{H}\right)^2\right]\right. \qquad \text{(XI)}$$

to within a very close approximation.

The strain at the top is

$$\bar{\epsilon}_1 = \left(1 + \frac{H^2}{3C_v t}\right) \qquad \text{(XIIa)}$$

and the base is

$$\bar{\epsilon} = \left(1 - \frac{H^2}{6C_v t}\right) \qquad \text{(XIIb)}$$

The strain difference is therefore

$$(\epsilon_{1.0} - \epsilon_{1.H}) = \frac{\bar{\epsilon}_1 H^2}{2C_v t} = \frac{r H^2}{2 C_v} \qquad \text{(XIII)}$$

If we continue to assume m is a constant the,

$$(\epsilon_{1.0} - \epsilon_{1.H}) = m_v\left[(\sigma_{ye} + \Delta\sigma_{ye}) - (\sigma_{ye} + \Delta\sigma_{ye} - U_b)\right] \qquad \text{(XIV)}$$

$$= m_v \cdot U_b$$

Hence from eqn.XIII and XIV

$$C_v = \frac{H^2}{2U_b}\left(\frac{r}{m_v}\right) \qquad \text{(XV)}$$

The rate of change of strain at any point is constant which means, over a time increment, Δt

$$m_v = r\left(\frac{\Delta t}{\Delta\sigma_{ye}}\right)$$

Hence

$$C_v = \frac{H^2}{2U_b}\left(\frac{\Delta\sigma_{ye}}{\Delta t}\right)$$

2.C Constant Rate of Strain Oedometer

The constant rate of strain test became a standard test at the Swedish Geotechnical Institute (SGI) in 1975.(6)

During the five years the CRS-test has been used at SGI much experience has been gathered. Related research concerning permeability, swelling , recompression and secondary deformation has been carried out.

Byrne and Aoki[4] compared the results from strain controlled tests with the standard load controlled test. The e vs log σ' curves are shown in fig. 2.1 . Unfortunately, because the initial void ratios are not quite the some curve crossing occur. However, it may be seen that higher strain rates cause higher effective stress for the same void ratio as was shown by Hamilton and Crawford[7] and that load controlled test agrees closely with tha lowest strain rate used.

They summarized that the merits of the method of the strain controlled consolidation test are

(1) Because of the low gradients that exist throughout the test, the simplification made by Terzaghi that $\partial e = -a_v \partial \sigma'$ within each load increment, can be replaced by the assumption, much more accurate in the proposed test method, that change in void ratio is uniform throughout the depth of the specimen.

(2) Because the strain rate is subsantially constant through the duration of the test, and under the control of the experimenter, secondary compression which is just a short coming of the Terzaghi simplification, can be seen as strain rate effect.

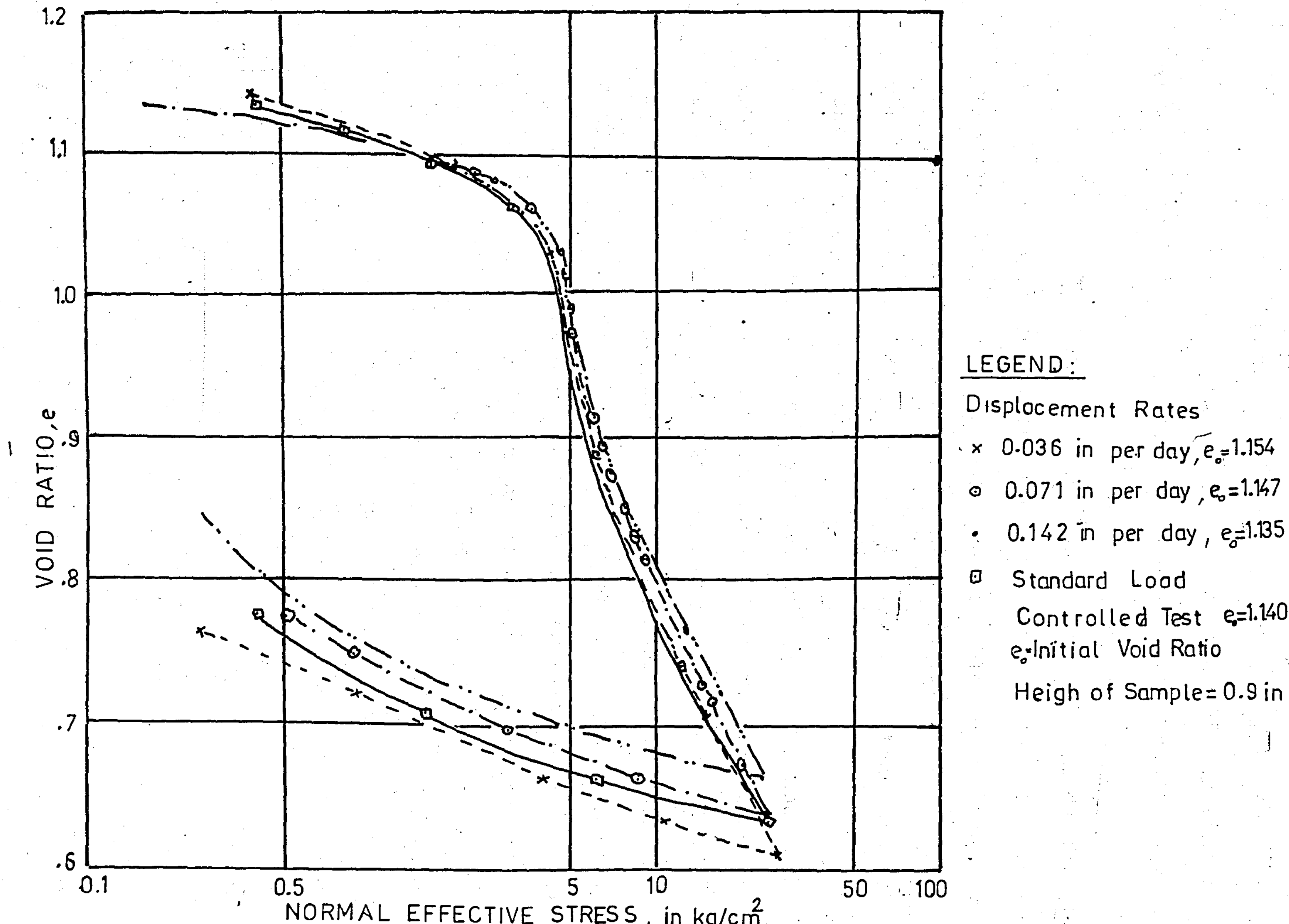


FIG_2.1 Void Ratio vs Normal Effective Stress.

(3) Using e vs k and e vs ζ' relationships determined over a range of strain rates, the complete time-displacement history of a load controlled specimen of any size can be predicted, i.e., primary and secondary compression can be predicted.

(4) If the Terzaghi assumptions are also made the theory allows a rapid method of determining the coefficient of consolidation, c_v. It has the considerable advantage that the gradients are low and that no squeezing of clay past the top stone take place as may occur in the load controlled test.

Kuantası Lee[8] demonstrates that consolidation with constant rate of deformation is a much more complicated process than has been suggested by the small strain theory. The present proposed method of interprating test results is applicable only to the case of a small β (normalized strain rate) value where β is a function of r, h_o, and c_v, and $\beta = r.h_o^2/c_v$, where r is deformation rate, h_o is initial thickness of specimen and c_v is coefficient of consolidation. This restricts the fastest strain rate can be used. It would be desirable to have an interpratation method independent of the strain rate used. Since such a method may eventually lead to a rapid consolidation test.

.2.D Effect of Repeated Loading

Estimation of the amount of permenant deformation occuring as a result of repeated loading of a structure is one problem for which the irrecoverable strains measured in cyclic loading tests provide valuable information. However estimation of the cyclic deformation occuring during the repeated loading is also important.

The features of the behaviour of clay under repeated loading on which information is required are

(1) The development of permanent (plastic), strains and pore water pressure with number of cycles.

(2) The variation of cyclic (elastic), strains and pore water pressures with number of cycles.

Güler E.[9] has conducted from a water content of twice the liquid limit to different consolidation pressure. Test results were given in fig. 2.2 . In this test, number of load application are N = 1, N = 6, and N = 11. Conclusions obtained from the oedometer tests are as follows

(1) The extra volume decrease that take place under the application of repeated loading increases with increasing intensity of the repeated load.

(2) The extra volume decrease also increases with the increasing number of load applications.

(3) The void ratio of soil can be brought to a value smaller than its value obtained from static consolidation by applying repeated loading.

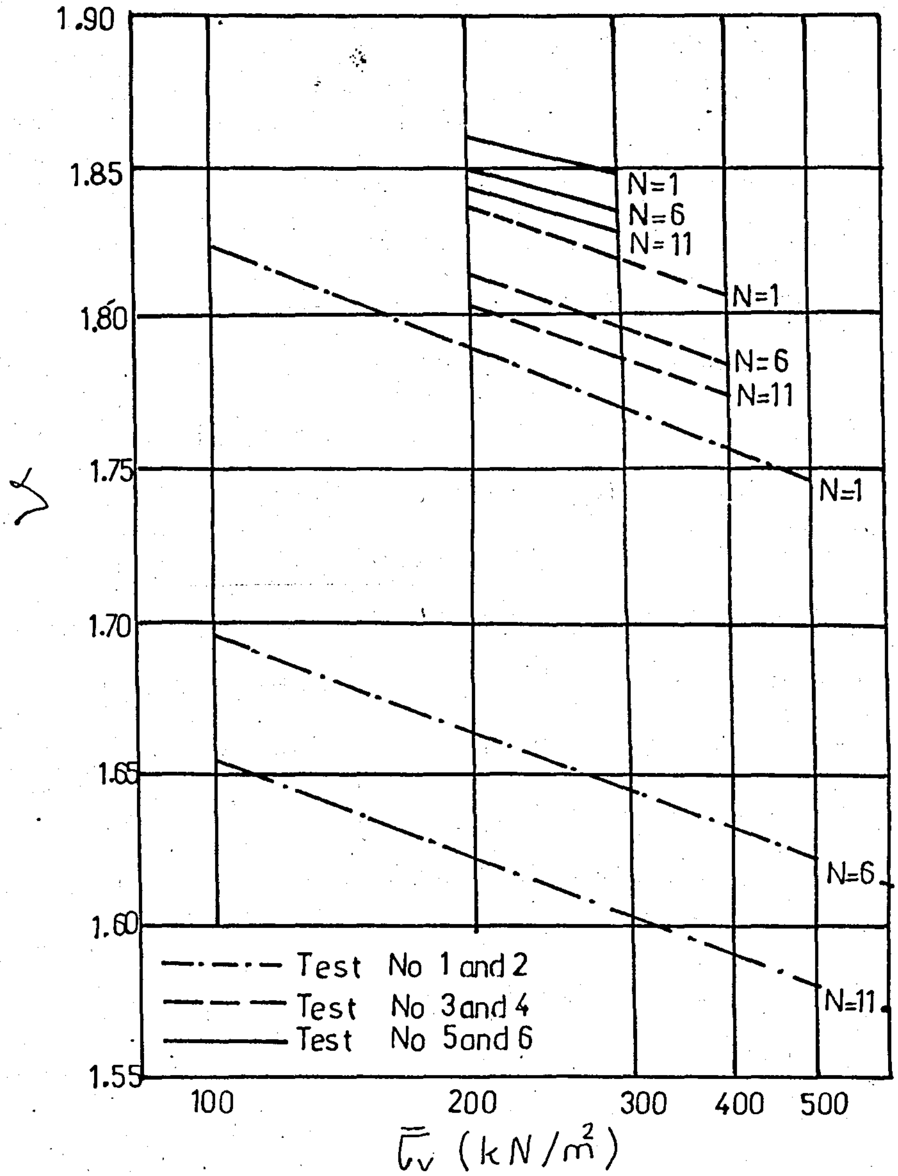


FIG-2.2 Results of Repeated Oedometer Tests.

Toğrol E. and Güler E.[10] have reported other similar tests with different load amplitude. Test results were given in fig. 2.3(a,b). In addition to above conclusions, some conclutions have been obtained which are as follows;

(1) If the repeated load is applied too rapidly to speed up the consolidation process, the load intensity has to be above a lower limit in order to obtain lower void ratio in comparison with the static loading. This lower limit is for example about %50 for "Topser Sarı Clay".

(2) The "Arnavutköy Kaolini" which has a higher plasticity index showed greater e values in comparison with the e values of "Topser Sarı Clay" so they may conclude that the application of repeated loading to increase the degree of consolidation can be applied more succesfuly on clays with high plasticity indexes.

Bishop and Henkel[11] tested heavily overconsolidated clay in the triaxial apparatus. The resulting negative residual pore pressure was then allowed to dissipate with consequent softening of the sample over several hours. Another pulse was then applied and the process repeated, the sample becoming progresively softer after each pulse.

Henkel(12) has reported other similar tests with different load amplitudes. Normally consolidated samples, for which cycles of undranied loading produce positive residual pore pressures, would strengthen if these pore pressures were allowed to dissipate.

Knight and Blight[13] tested a normally consolidated claysilt in the triaxial apparatus. Drainage was allowed

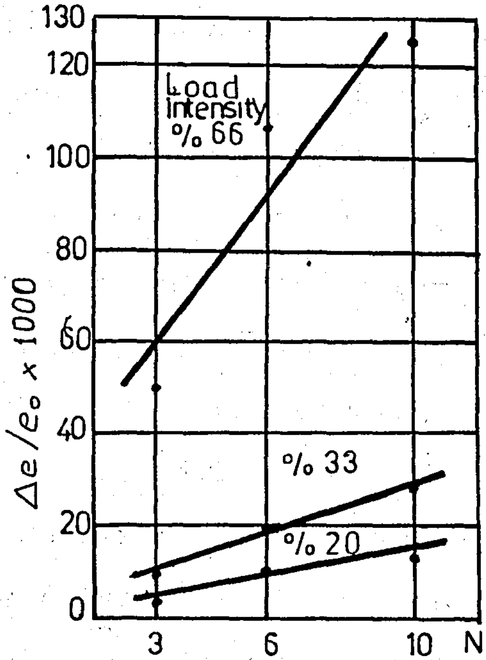


FIG. 3 Results of Tests Conducted on "Arnavutköy Kaolini"

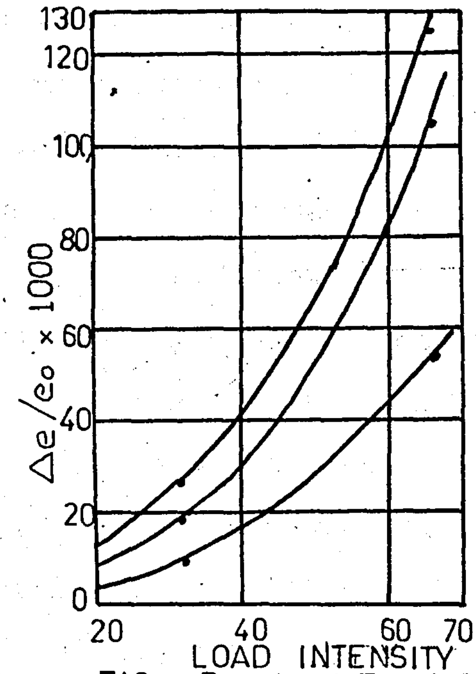


FIG. Results of Tests Conducted on "Arnavutköy Kaolini"

FIG.2.3.a

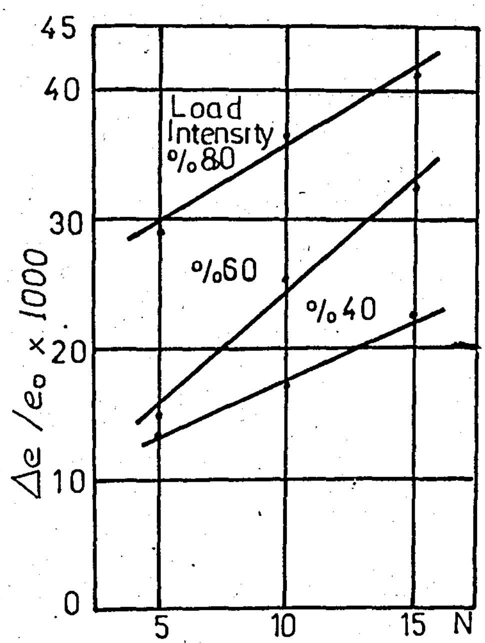


FIG Results of tests condacted on "Topser Sarı Clay"

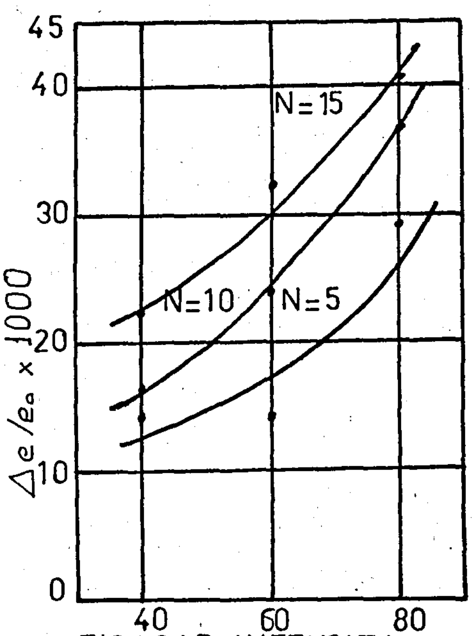


FIG LOAD INTENSITY

FIG Results of tests conducted on "Topser Sarı Clay"

FIG.2.3b

after each cycle of slow loading and unloading. It is not stated how cyclic stress compared with the static strength. However, they found that the residual positive pore pressure decreased, and the permanent deformation reached a limit as the mumber of cycles increased.

Sherif and Wu(14) report a series of repeated loading tests in the triaxial apparatus with two different frequencies and two different amplitudes. They showed the normally consolidated sample behaves a much faster development of strain with number of cycles than the other samples. These strains are permaent strains, that is the average of the strains at the two ends of a cycle. The amplitude of the cyclic strain was found to be independent of the number of cycles.

Wilson and Greenwood(15) found a linear relationship to exist between the cyclic strains and cyclic pore Pressures.

Hyde, Brown and Pell(16) have studied the effeet of strain under the repeated loading was not much affected by the rest periods.

Both Thiers,(17) and Taylor and Bacchus(18) show results from cyclic triaxial tests in which the stress amplitude increases with increasing strain amplitude at low numbers of cycles, but decreases with increasing strain amplitude at larger numbers of cycles.

Van Eekelen and Potts(19) noted that the behaviour of Drammen clay under cyclic loading may be described interms of single fatique parameter, i.e., the pore-water pressure u generated by cyclic loading.

CHAPTER 3 EXPERIMENTAL STUDY

3.1 EQUIPMENT

3.1.1 Equipment Requirements

The following items are requireds for the operation of the oedometer.

(1) The oedometer cell shown in fig3.1(The cosolidation ring used was 6.98 cm in diameter and 2 cm in height. The ring was sealed at the base so that drainage took place through the top porous stone only.). Pore pressure transducer is located in its base for measuring the pore pressure.

(2) Two self compensating constant pressure application systems are shown in fig3.2(Cell and back pressure were provided by them).

(3) A load frame or loading platform.

(4) Load cell mounted above the top stone (the normal load was measured by it).

(5) A dial gauge (the vertical displacement was measured by this).

(6) A millivoltmeter.

(7) Trimming equipment including wire saw.

(8) Slicon grease.

(9) Supply of distilled water.

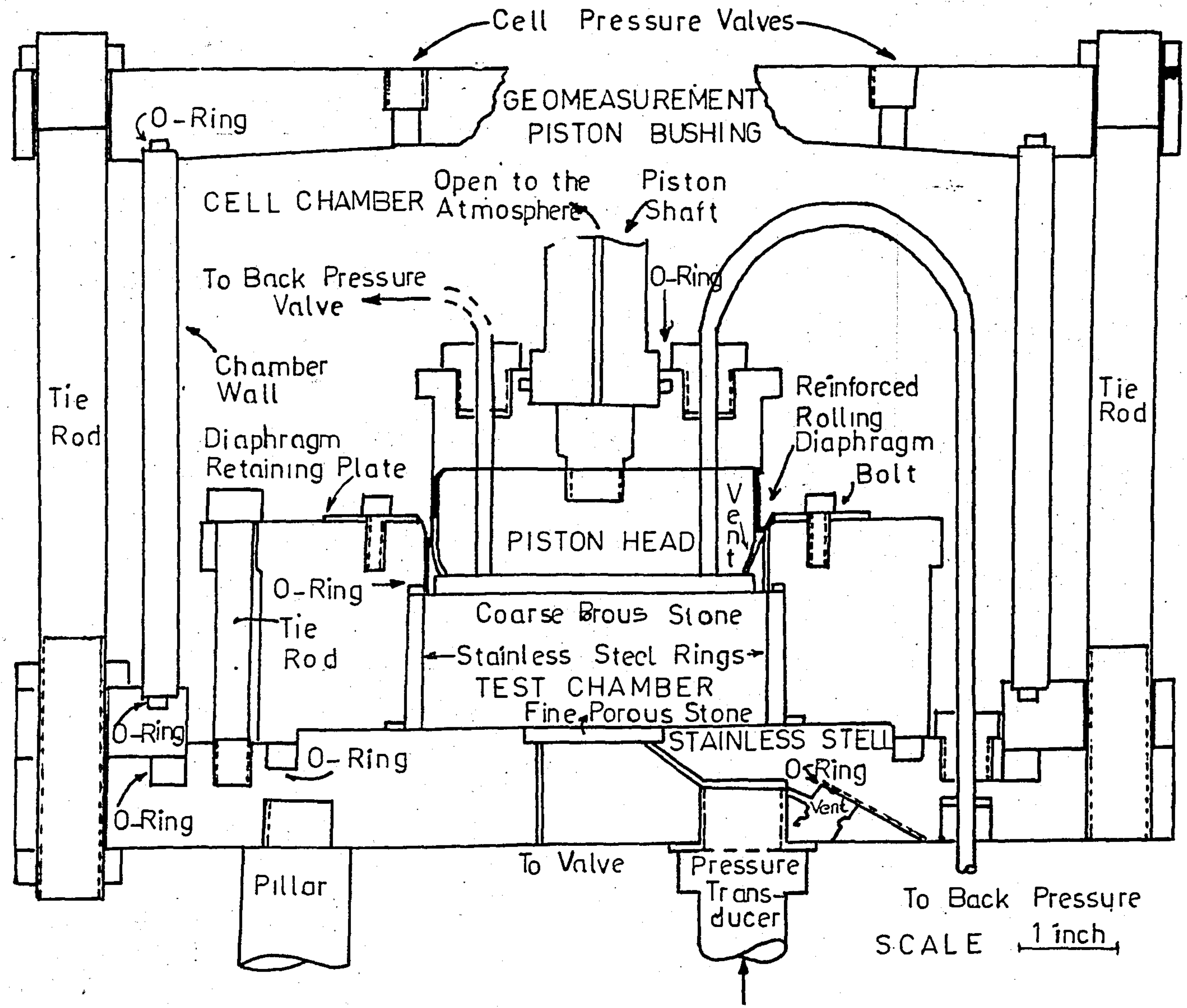


FIG-3.1 SCHEMATIC ASSEMBLY DRAWING OF OEDOMETER CELL

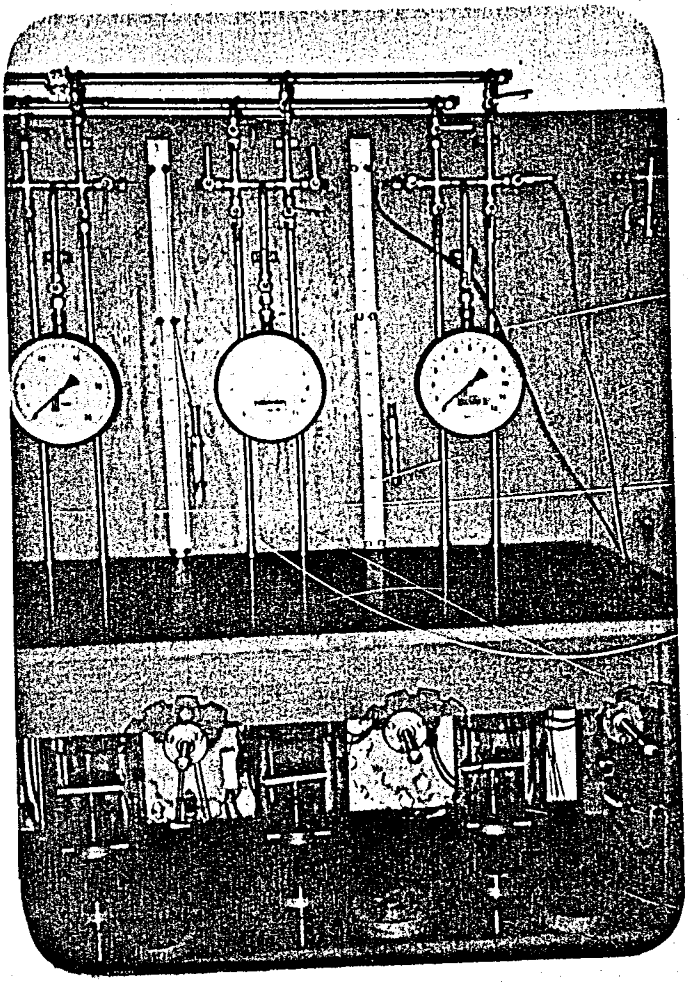


FIG-3.2 Self Compensating Constant Pressure Application Systems.

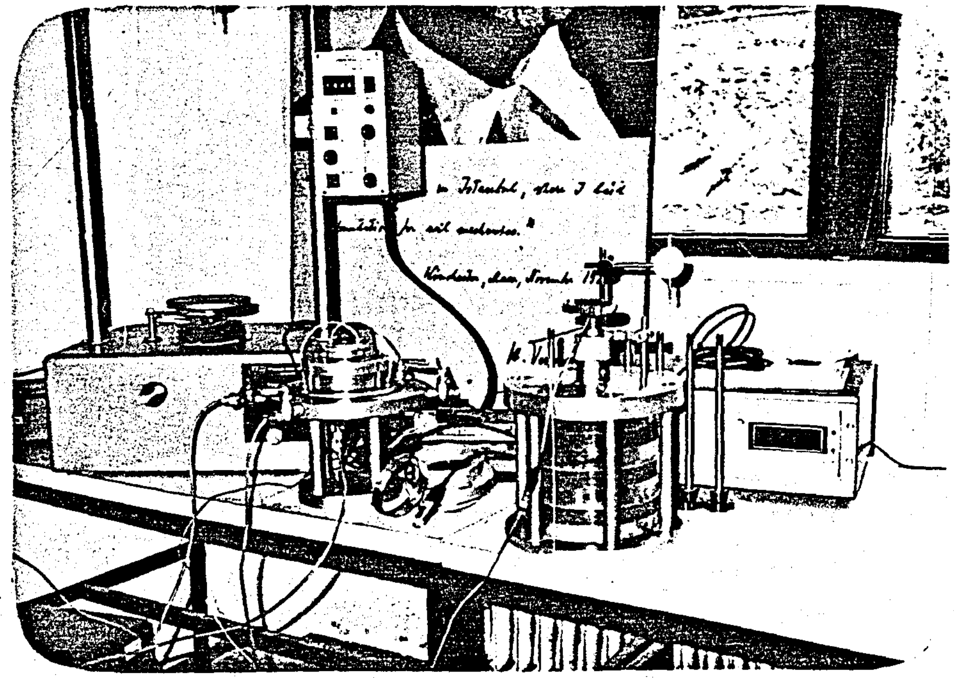


FIG-3.3 CONSOLIDOMETER

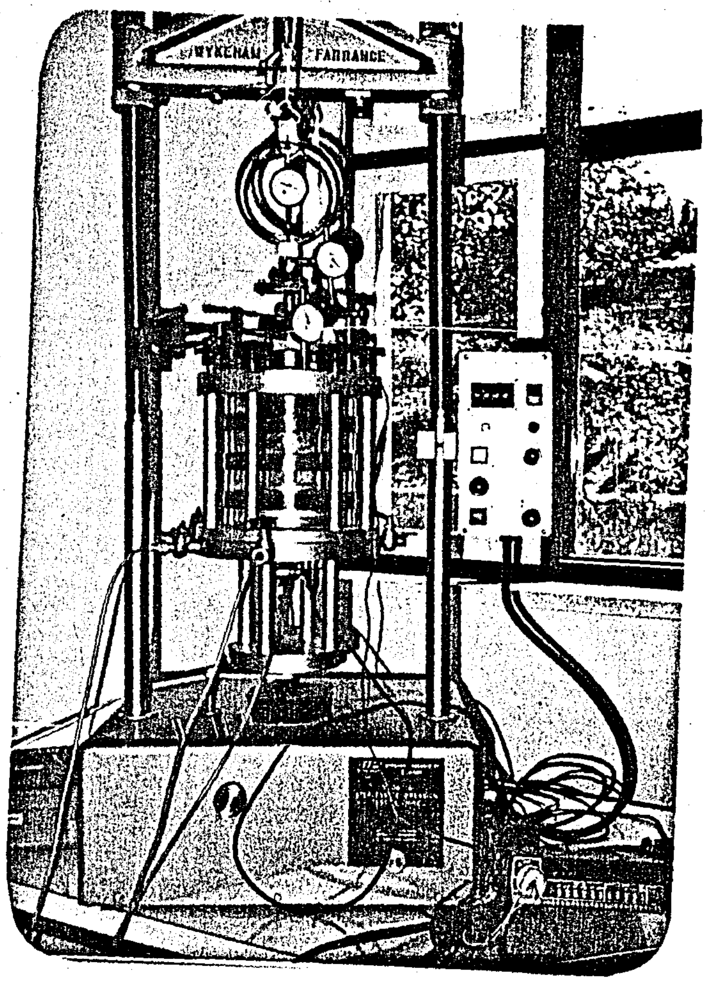
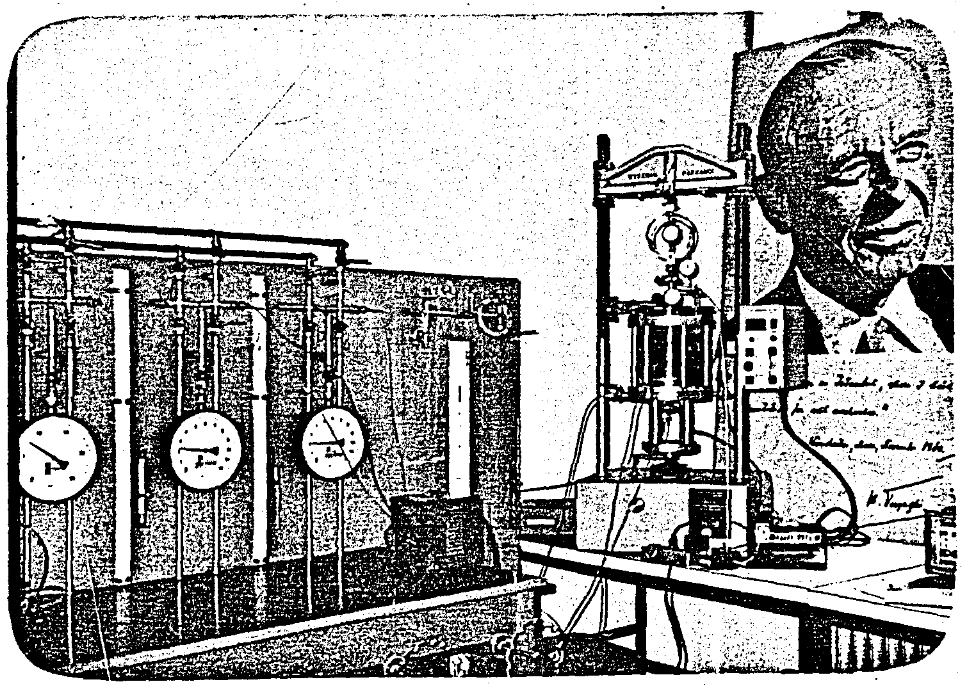


FIG_3.4 Oedemeter and CRS Loading Device.



FIG—3.5 Oedometer CRS Loading Device and Self Conpensating Constant Pressure Application System.

3.1.2 Mounting of Specimen

The test specimen which is in the stainless ring is then placed in the test chamber. Due to tight fit between the sample ring and the test chamber it is important that the test chamber and the part of the ring is not occupied by the specimen and be absolutely free of soil and dirt. During this operation the piston is heldiin its upstroke position by use of the spacer.

The test chamber containing the specimen is slid onto the cell base after the fine porous stone has been dried of free water. The unit should be slid on in a way that the soil specimen is always in contact with the base. At this time the residual pore pressure in the test specimen can be measured with the pore pressure transducer.

An O-ring is placed in the groove located at the bottom of the test chamber (between the sample ring and test chamber wall).

The test chamber unit should be correctly orientated with respect to the drainage lines while sliding it into its final position.

3.1.3 Assembly of Cell

The assembling of the cell involved the following steps.

(1) The unit containing the test chamber was bolted down.

(2) The spacer around the piston head was removed.

(3) The piston head was lowered until it is in contact with the soil.

(4) The drainage lines to the base of the cell chamber was reconnected.

(5) The cell chamber was assembled and it was bolted down.

(6) The piston shaft into the piston head was intruded.

(7) The cell chamber was filled with deaired water. (Distilled deaired water is preferable over tap water to minimize corrosion).

(8) The cell was placed in the load application device.

(9) The cell and back pressure lines were connected to the constant pressure supplies.

(10) The dial gauge and the dial reading were mounted.

(11) The load cell was mounted.

(12) All instruments were connected to the data acquisition channels.

a DC power supply

b load cell

c pore pressure tranducer.

3.2 MATERIAL AND SAMPLE PREPARATION

The material used in the tests are first dried in the oven and then sieved from a No:40 sieve.

To get the material in a condition suitable for testing it was first necessary to throughly mix the clay with sufficient water to bring the sample to a water content slightly below the liquid limit. The initial moisture content of the samples varied from 44 % to 46 % with the average value being 45 %.

Generally, material contins silt and fine sand. Index properties obtained from laboratory tests are ;

L.L. = 50.0 %

P.L. = 24.8 %

P.I. = 25.2 %

$\gamma_s = 2.70\ t/m^3$

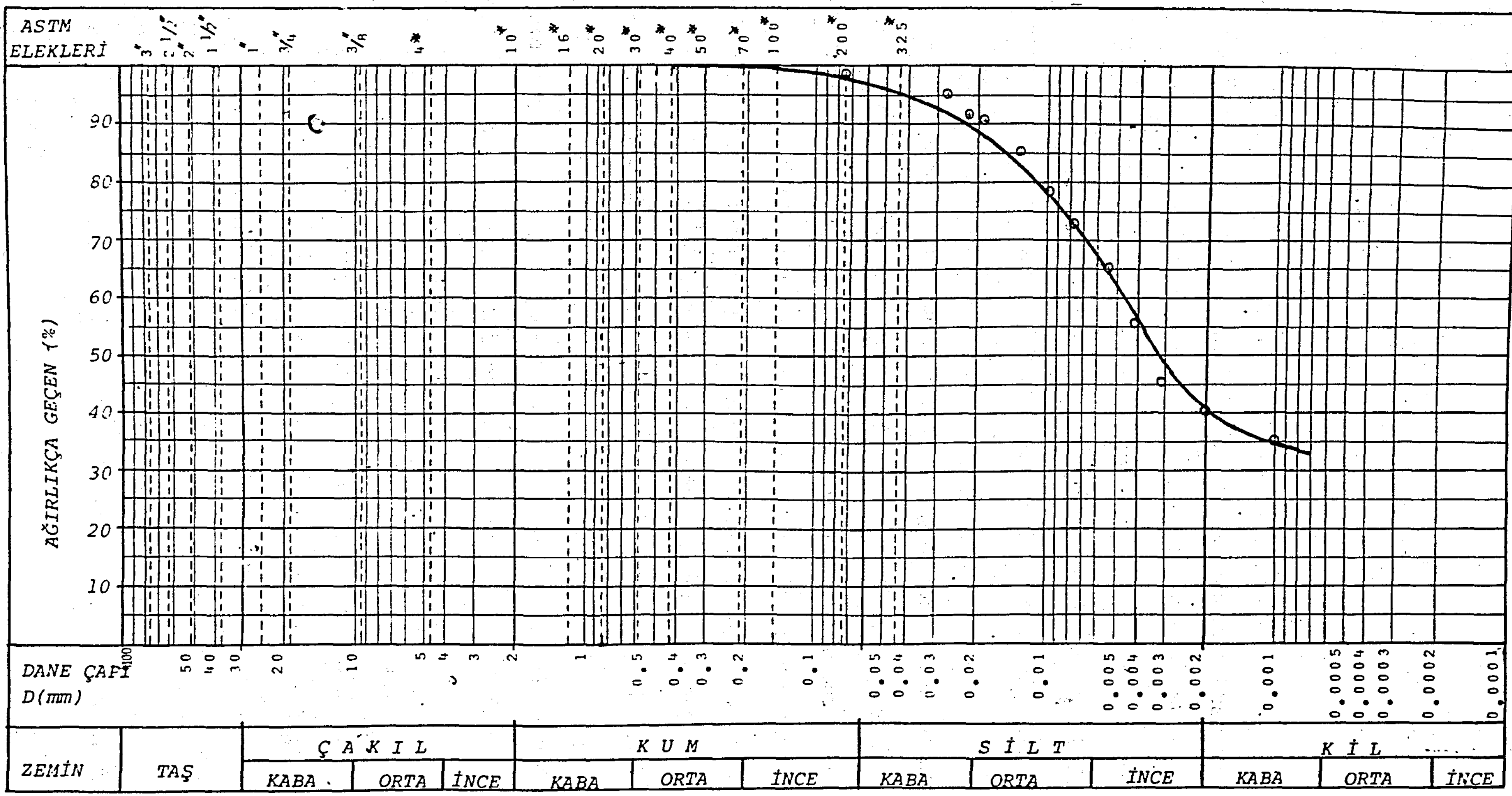


FIG.3.6 Grain Size Distribution Diagram for "Topser Sarı Clay."

3.3 TESTING

It is imperative that the water pressure in the cell chamber at all times be equal to or greater than the back pressure in order for for reinforced rubber membrane to be operative.

Using the load frame, the piston is brought into contact with the load cell without applying a significant force to the piston.

The test and cell chambers are connected hydraulically through the constant pressure manifold. The pressure is raised in both chambers until the desired back pressure is reached. There will be a net upwards force on the piston during this procedure which is taken up by the load cell.

Prior to starting the actual test, a small load should be applied to the piston head by increasing the cell pressure to insure that it is in contact with the soil. This is done by applying a pressure in the cell chamber slightly higher than that required to balance the net force on the piston caused by the back pressure.

To saturated the test sample, cell pressure, 2.25 kg/cm^2 and back pressure, 2.0 kg/cm^2 were hydraulically applied along 24 hours. Therefore, test sample in the test chamber had been consolidated under 0.25 kg/cm^2. At 24 hours test sample was fully saturated. Any excess pore pressure caused by this operation was allowed to dissipate.

The chosen rate of strain was 0.002 mm/min. To determine this value we performed a series of test. To evaluate the test results with given reference equations, the maximum pore pressure values obtained from tests shall be smaller than 15 percent of the maximum reaching stress values. We have taken care that the maximum pore pressure values obtained from tests were not greater than 10 percent of the maximum reaching stress values to be on the safe side.

with respect to chosen minimum and maximum value of dynamic load, pushing the up or down button, repeated load was applied on the test specimen.

During the testing readings were taken on the data acquisition systems at desired time intervals.

3.4 EVALUATION OF TEST RESULTS

The available data obtained from the test is processed as follows.

The average effective stress in the sample during the test is calculated by the simple equation

$$\sigma' = \sigma - \frac{2}{3} u_b$$ (Smith and Wahls, 1969)[20]

where total pressure

and u_b pore pressure at the bottom of the sample measured with the pressure transducer.

This equation is not exact but the error is small provided that the pore pressure is low in relation to the vertical pressure. Considering the time dependency of the stress strain curve the pore pressure should in any case be kept low and thus the the formula can normally be used.

Tokheim and Janbu[21] (1976) give the equation

$$\sigma' = \sigma - \frac{4}{5} u_b \frac{5\dot{\sigma} - 4\dot{u}_b}{6\dot{\sigma} - 5\dot{u}_b}$$

where $\dot{\sigma} = d\sigma / dt$
$\dot{u}_b = du_b / dt$

where H is the sample height.

This equation is also valid for low pore pressure only. Sallfors[22] (1975) recommends that the pore pressure should not exceed 15 % of the total pressure.

Tokheim and Janbu[21] (1976) give the equation

$$c_v = \frac{M.H.\dot{\delta}}{2\,U_b} \; \frac{6\dot{\sigma}-5\,\dot{U}_b}{6\dot{\sigma}-4\,\dot{U}_b}$$

where $M = \text{Modulus}\,(d\sigma/d\varepsilon)$

$\dot{\sigma} = d\sigma/dt$

$\dot{U}_b = dU_b/dt$

$\dot{\delta} = dH/dt.$

The theory is limited to $\dot{U}_b/\dot{\sigma} < 0.35$ .The difference between this equation and $c_v = \frac{\Delta\sigma}{dt} \cdot \frac{H^2}{2U_b}$ is negligible for small pore pressures.From the CRS- test the coefficient of permeability can also be obtained continously and is calculated by the equation

$$k = \frac{d\varepsilon \,.\, H^2 \,.\, g \,.\, \rho_w}{dt \;.\; 2\,U_b}$$

Evaluations of c_v and k are based on the assumption of a parabolic pore pressure distribution in the sample,

CHAPTER 4 TEST RESULTS

Nine repeated loading tests were made. Variables chosen were (1) sustained load, (2) load intensity (dynamic load), (3) number of cycles, Sustained load, load intensity (which is the ratio of dynamic load to sustained load) and number of cycles were expressed by σ_0, $\frac{\sigma_0}{\sigma_d}$ and N respectively. In table- 4.1, the values of variables and results are given;

Test No	Sustained Load σ_0 kg/cm	Load Intensity $\frac{\sigma_0}{\sigma_d}$, %	Number of Cycle N	$\frac{\Delta e_d}{e_{od}}$ x1000	$k_{ult} \times 10^{-6}$ cm/min	$k_{max} \times 10^{-6}$ cm/min
1	0.5	20	4	5.2	2.11	24.7
2	1.0	20	3	57.0	2.02	21.1
3	2.0	20	5	14.8	0.75	7.65
4	0.5	40	3	12.4	1.56	2.82
5	1.0	40	5	114.0	0.80	24.55
6	2.0	40	4	97.2	0.38	2.1
7	0.5	60	5	25.0	1.15	17.9
8	1.0	60	4	82.6	1.65	20.34
9	2.0	60	3	226.7	0.75	8.3

Table- 4.1 The values of variables and results.

Variables, from 1st test to 9th test, were arranged to form a latin square in order to be able to apply variance analysis to the results. We denote sustained loads with (A: $\sigma_o = 0.5$, B: $\sigma_o = 1.0$, C: $\sigma_o = 2.0$ kg/cm^2), load intensity (dynamic loads) in percent with (I: $\frac{\sigma_o}{\sigma_d} = 20\%$, II: $\frac{\sigma_o}{\sigma_d} = 40\%$, III: $\frac{\sigma_o}{\sigma_d} = 60\%$) and number of cycles with (α: $N=3$, β: $N=4$, γ: $N=5$). Then we construct a 3 x 3 matrix with the test results to establish a latin square (Fig- 4.1).

	A	B	C
I	β	α	γ
II	α	γ	β
III	γ	β	α

Fig- 4.1 A Latin Square with Variables.

Where A, B and C : Sustained load

I, II and III: Load intensity

α, β and γ : Number of cycle.

If we superimpose the values of $\frac{\Delta ed}{e_{od}}$ x 1000 in latin square, where Δed is the difference between the values of void ratio at the begining and end of dynamic loads, and e_{od} is the value of void ratio at the begining of the dynamic loads respectively (for example see tab- 4.1) , fig-4.2 and table 4.2 are obtained as follows ;

	A	B	C	
I	5.2	57.0	14.8	77.0
II	12.4	114.0	97.2	223.6
III	25.0	82.6	226.7	334.3
	42.6	253.6	338.7	634.9

(a)

A	B	C	
12.4	57.0	226.7	296.1
5.2	82.6	97.2	185.0
25.0	114.0	14.8	153.8
			634.9

(b)

Fig- 4.2 The values of $\frac{\Delta ed}{e_{od}} \times 1000$ in latin Square .

	Sum of Squares	Degrees of Freedom	Mean Square	F	P
Sustained Load	15493	2	7746.5	1.58	0.25-0.50
Load Intensity	11105.5	2	5552.75	1.13	0.25-0.50
Number of Cycle	3729	2	1864.75	0.38	0.50-0.75
Remainder	9816.7	2	4908.35		
Total	40144.7	8			

Table- 4.2 Variance analysis of the values of $\frac{\Delta ed}{e_{od}} \times 1000$ w.r.t chosen variables.

With respect to table 4-2, due to repeated loading the magnitude of sustained load effects the variation of void ratio the most, load intensity and number of load cycles have lesser effect respectively.

As shown in fig 4-3- ,values of $\frac{\Delta e_d}{e_{od}}$ are increasing with increasing values of sustained load.

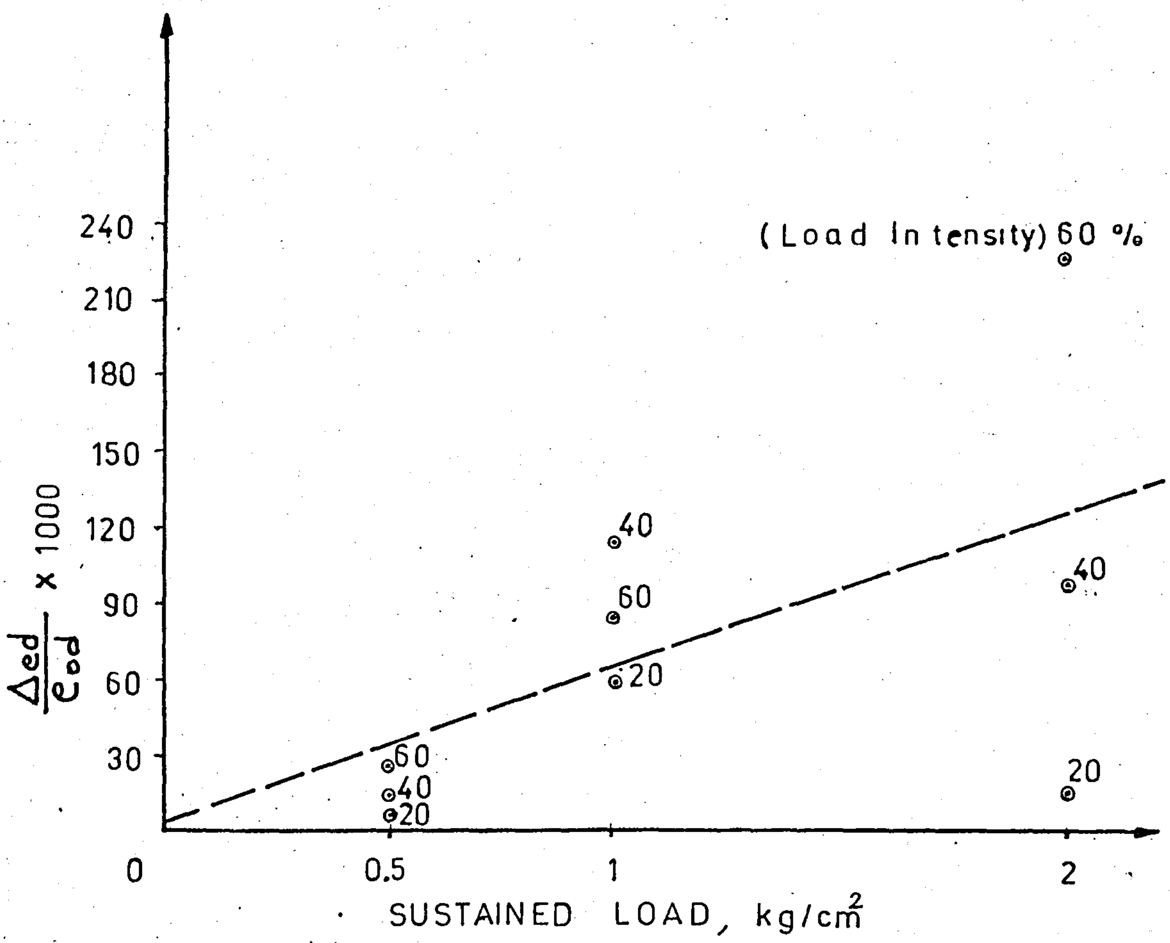


Fig-4.3 $\frac{\Delta e_d}{e_{od}}$ x 1000 versus Sustained Load.

If we superimpose the values of $k_{ult} \times 10^{-6}$ in latin square, where k_{ult} is the ultimate value of coefficient of permeability (for example see table- 4.1), figure- 4.4 and table- 4.3 are obtained as follows ;

	A	B	C	
I	2.11	2.02	0.75	4.88
II	1.56	0.80	0.38	2.74
III	1.15	1.65	0.75	3.55
	4.82	4.47	1.88	11.17

(a)

A	B	C	
1.56	2.02	0.75	4.33
2.11	1.65	0.38	4.14
1.15	0.80	0.75	2.7
			11.17

(b)

Fig- 4.4 The Values of k_{ult} in Latin Square.

	Sum of Squares	Degrees of Freedom	Mean Square	F	P
Sustained Load	1.72	2	0.86	57.3	0.01-0.025
Load Intensity	0.78	2	0.39	26.0	0.025-0.05
Number of Cycle	0.53	2	2.65	18.0	0.05
Remainder	0.03	2	0.015		
Total	3.06	8			

Table- 4.3 Variance analysis of the values of k_{ult} w.r.t chosen variables.

As shown in fig 4-5-,the values of kult are de- creasing with increasing values of sustained load.And as shown in figure 4-6.,The values of kult are decreasing with increasing percents of load intensity.

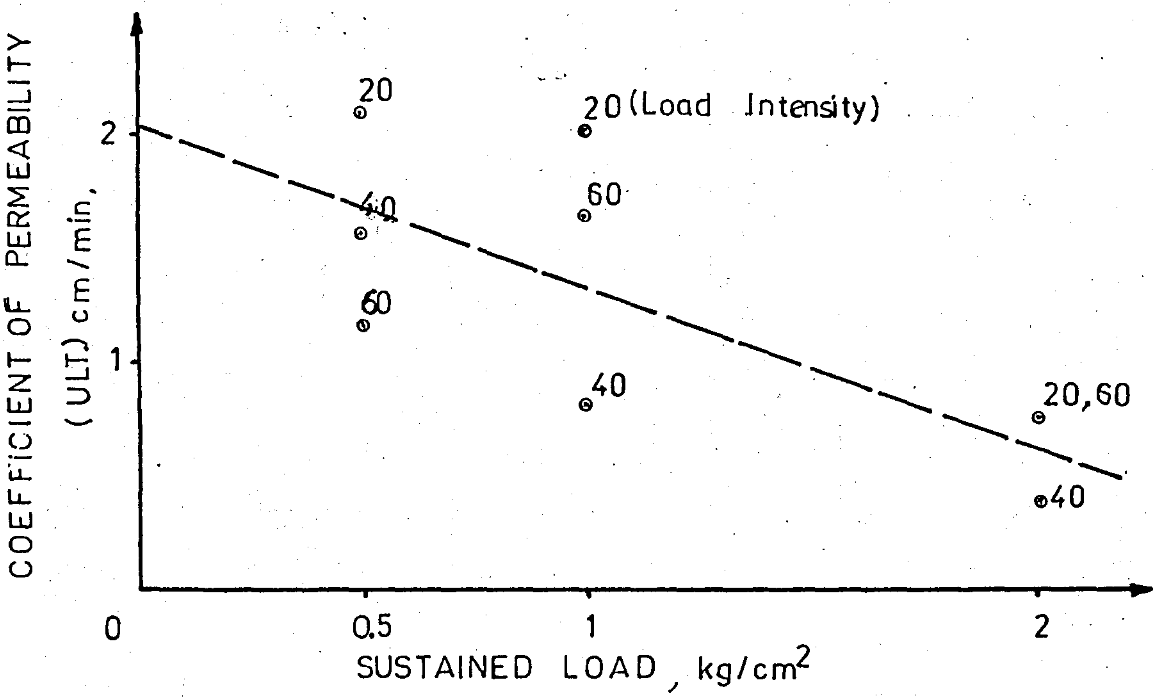


Fig-4.5 Coefficient of Permeability (Ult. Values) versus Sustained Load.

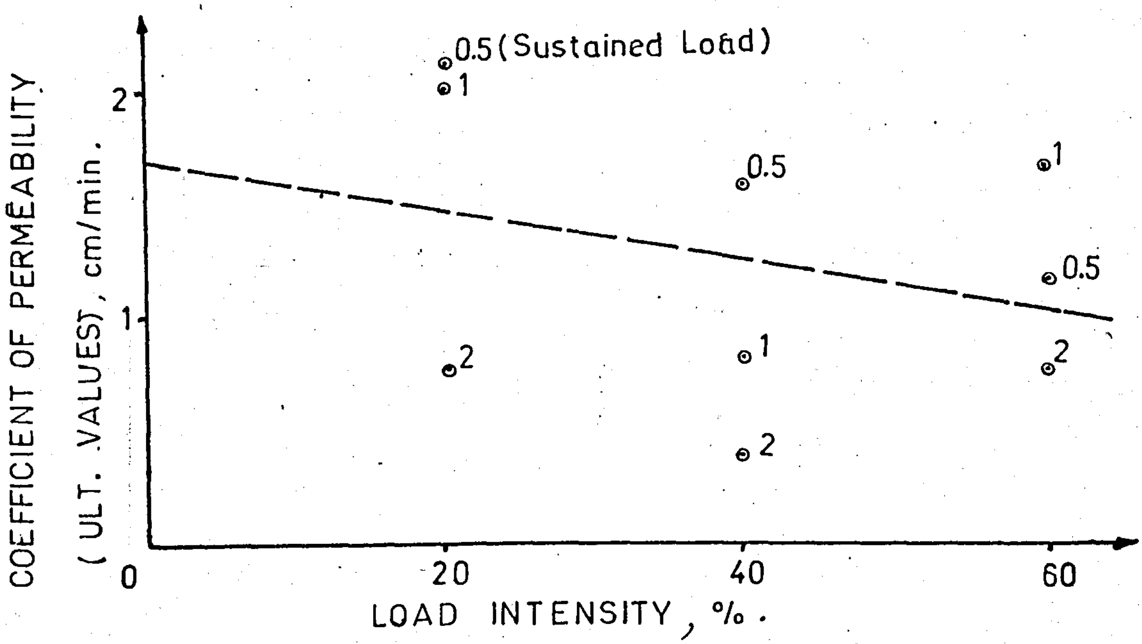


Fig-4.6 Cefficient of Permeability (Ult. Values) versus Load intensity.

If we superimpose the values of kmax x 10^{-6} in latin square, where kmax is maximum value of coefficient of permeability (for example see tab- 4.1), figure 4-7 and table 4-4. are obtained as follows.

	A	B	C	
I	24.7	21.1	7.65	53.45
II	2.82	24.55	2.1	29.47
III	17.9	20.34	8.3	46.54
	45.42	65.99	18.05	129.46

	A	B	C	
α	2.82	21.1	8.3	32.22
β	24.7	20.34	2.1	47.14
γ	17.9	24.55	7.65	50.1
				129.46

Fig- 4.7 The Values of k_{max} in Latin Square.

	Sum of Squares	Degrees of Freedom	Mean Square	F	P
Sustained Load	385.6	2	192.8	3.18	0.10-0.25
Load Intensity	101.6	2	50.8	0.84	0.50-0.75
Number of Cycle	61.2	2	30.6	0.50	0.50-0.75
Remainder	121.2	2	60.6		
Total	669.6	8			

Tab- 4.4 Variance analysis of the values of k_{max} w.r.t chosen variables.

As shown in figure 4-8-, the values of kmax are de- creasing with increasing values of sustained load. And as shown in figure 4-9., the values of kmax are decreasing with increasing percent of load intensity.

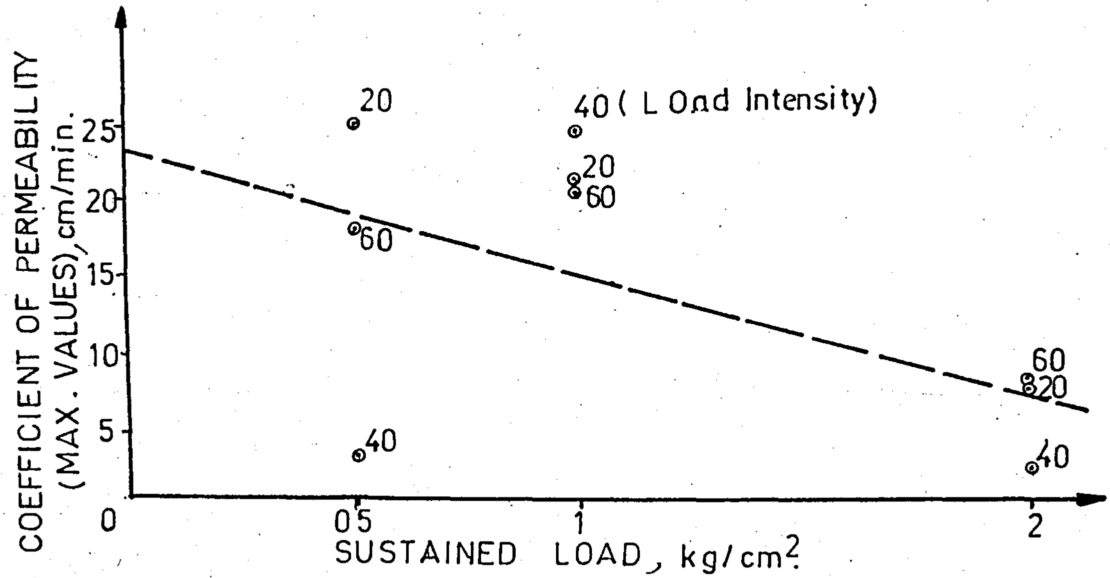


Fig- 4.8 Cefficient of Permeability (Max. Values) versus Sustained Load.

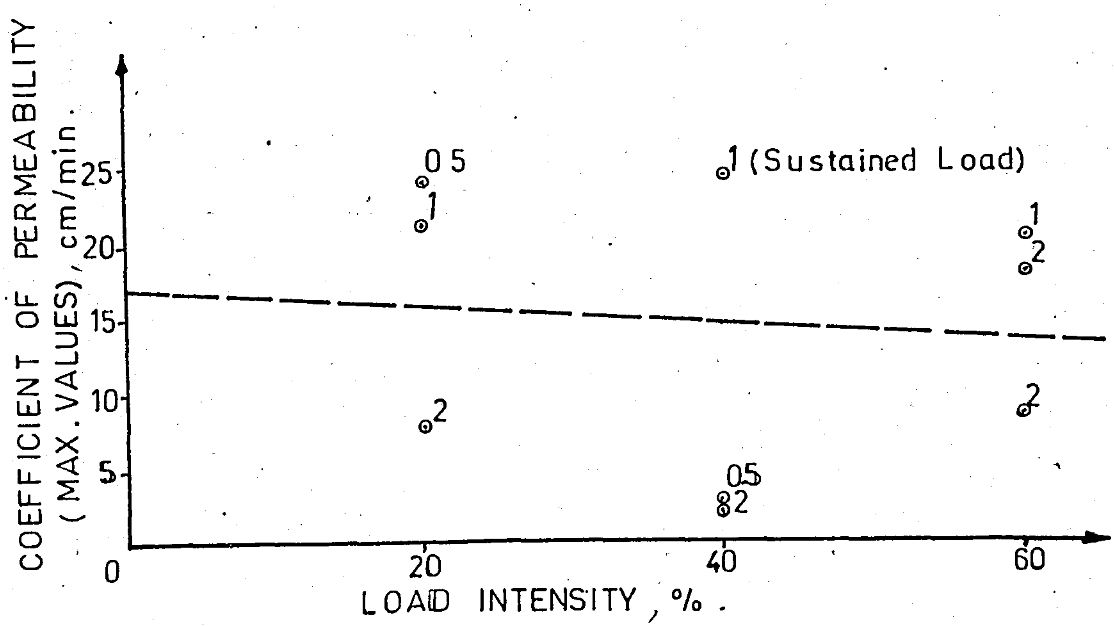


Fig- 4.9 Coefficient of Permeability (Max. Values) versus Load Intensity.

TEST 1

$w_i = 45\ \%$

$w_f = 44\ \%$

Sustained load, $\sigma_o = 0.5\ kg/cm^2$

Load Intensity, $\frac{\sigma_o}{\sigma_d} = 20\ \%$

Number of cycles, $N = 4$

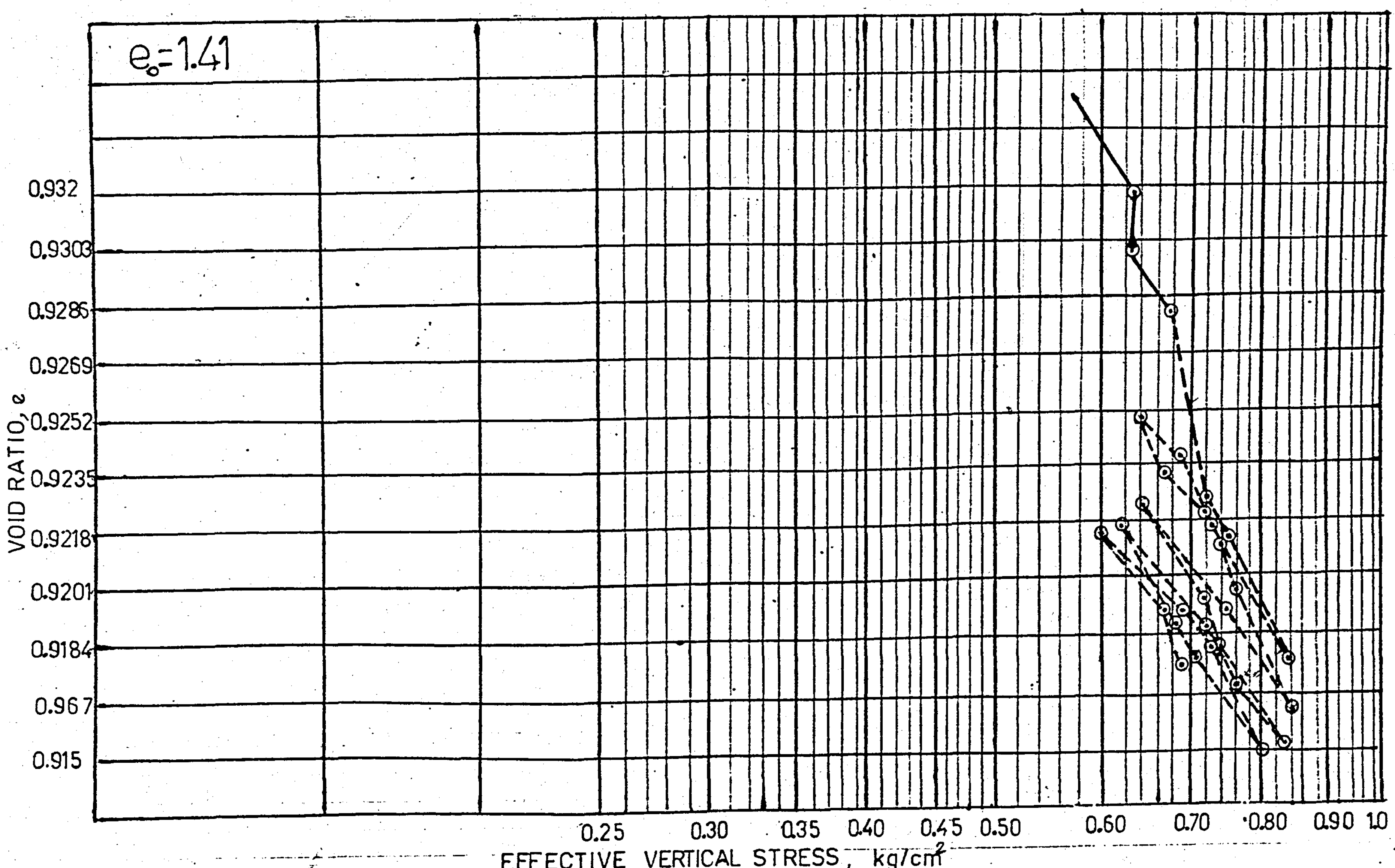


FIG - 4.10 Void Ratio versus Effective Vertical Stress

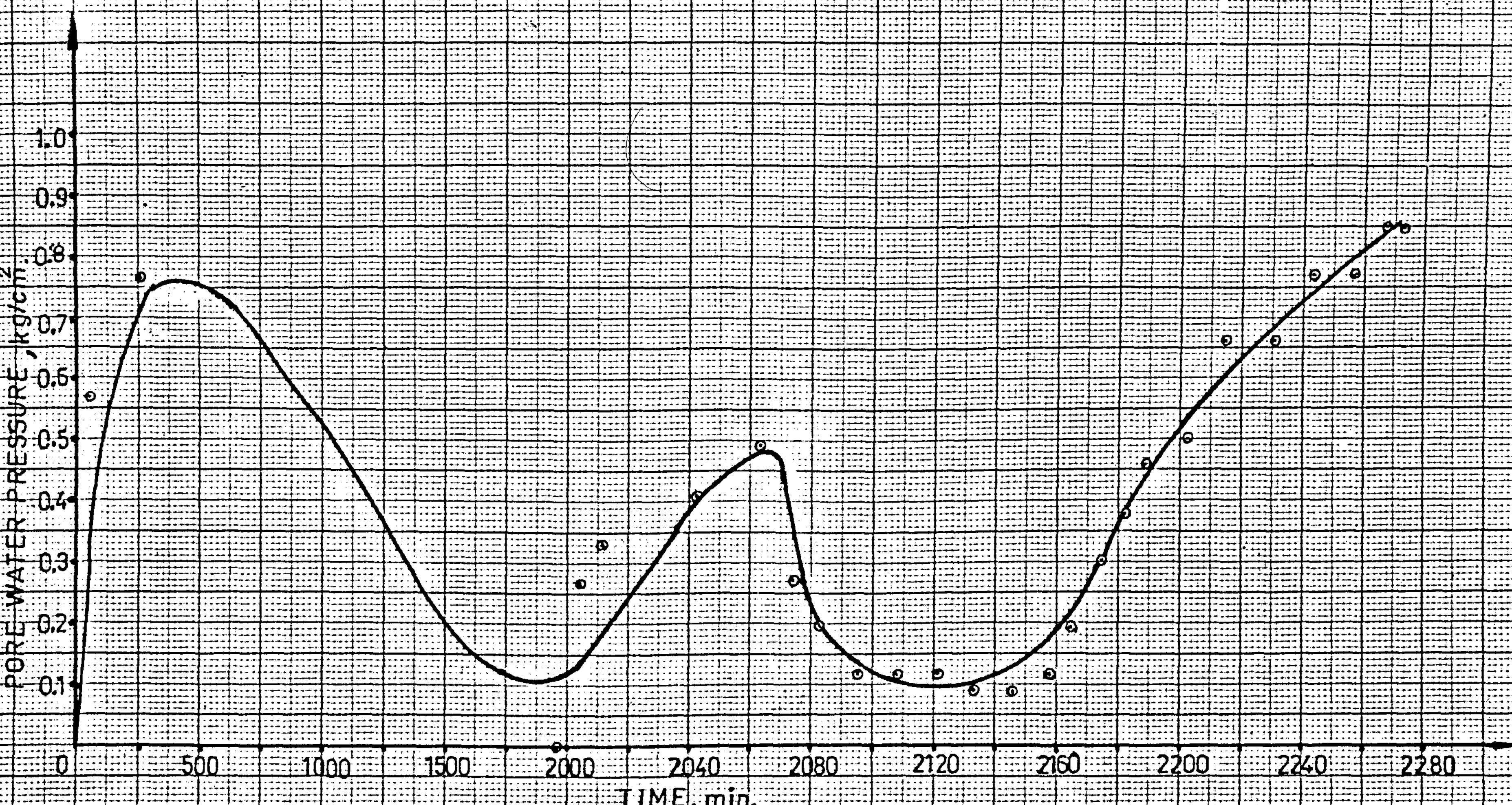
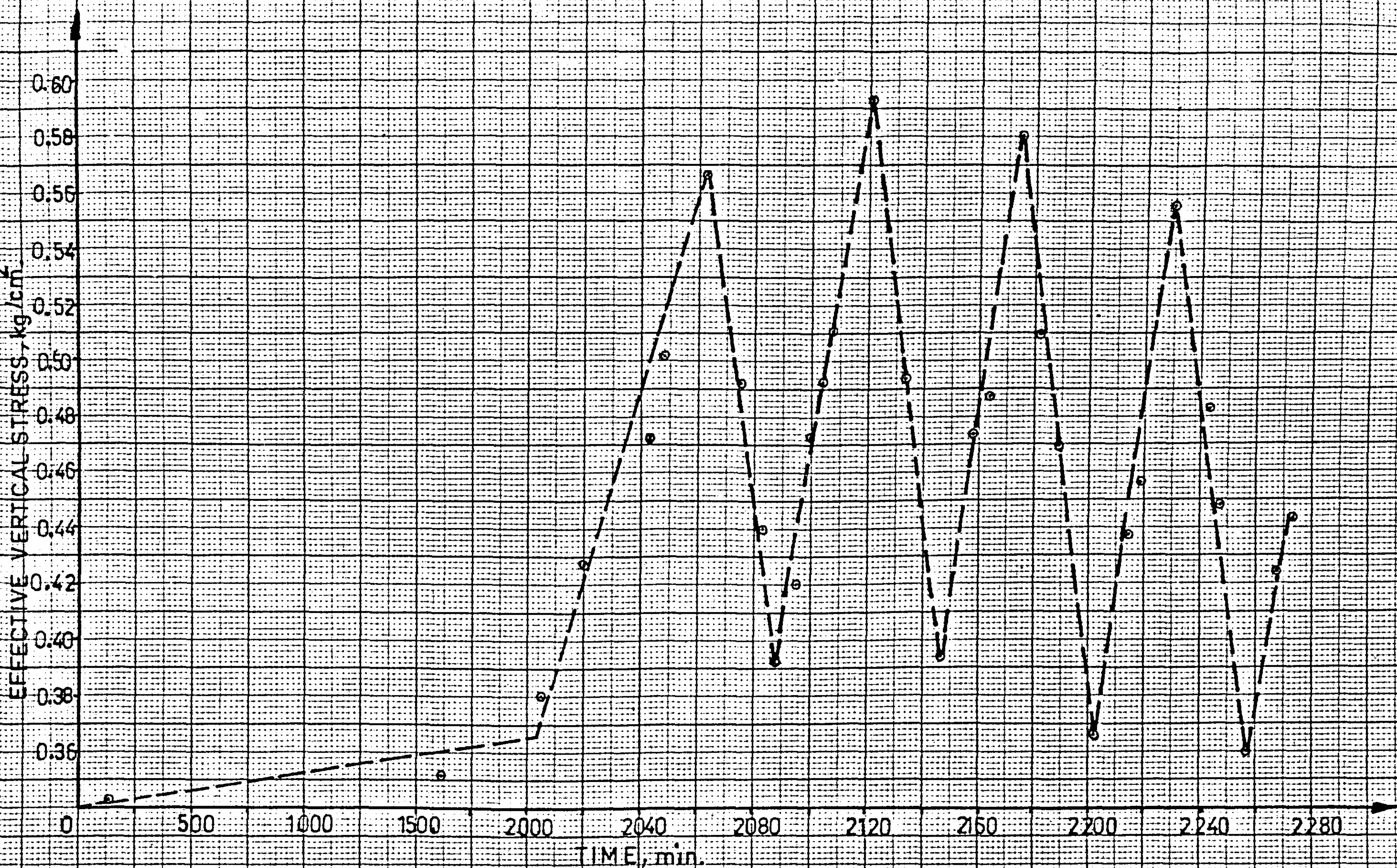


FIG-4.11 Pore Water Pressure versus Time.



FIG–4.12 Effective Vertical Stress versus Time.

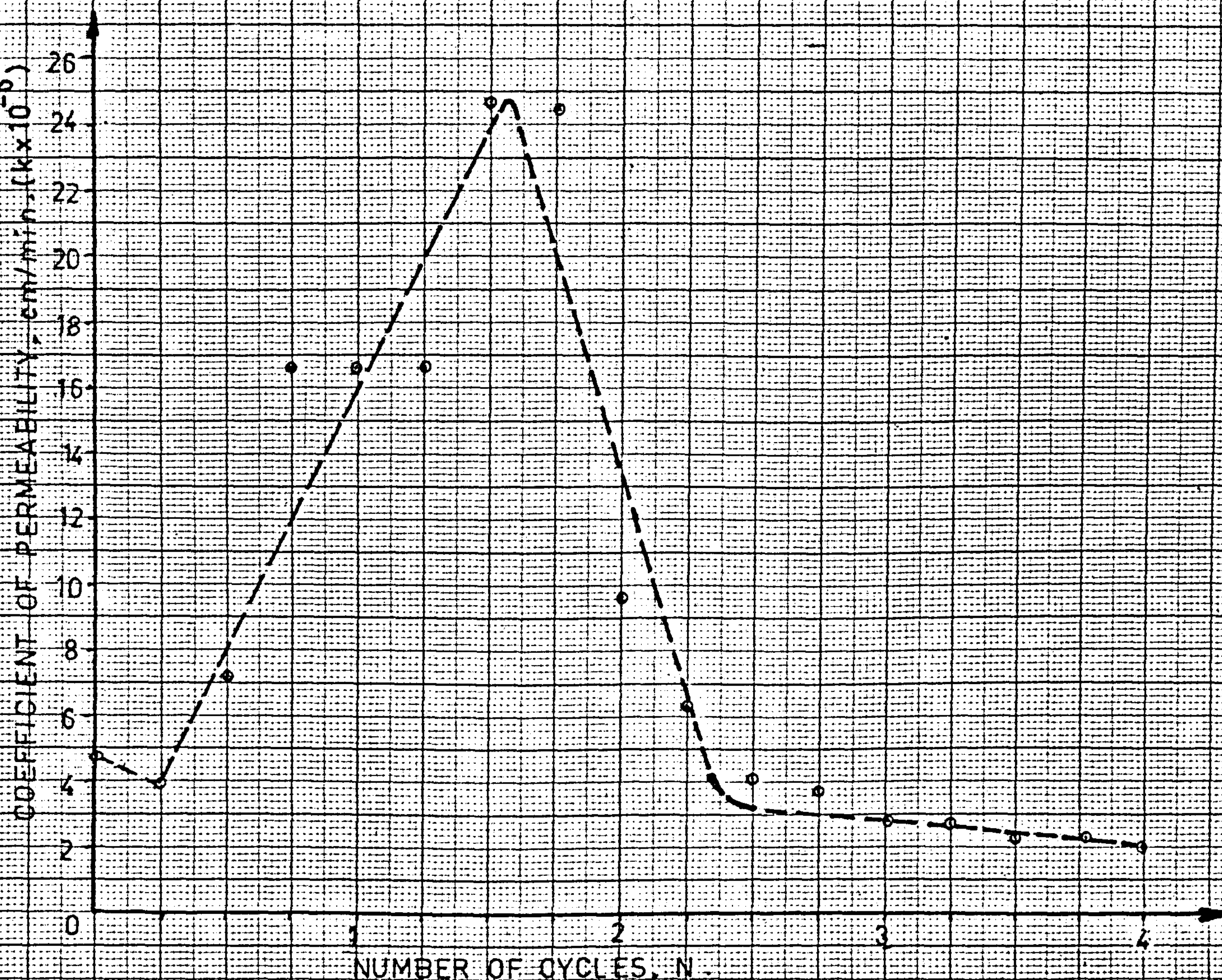


FIG- 4.13 Coefficient of Permeability versus Number of Cycles.

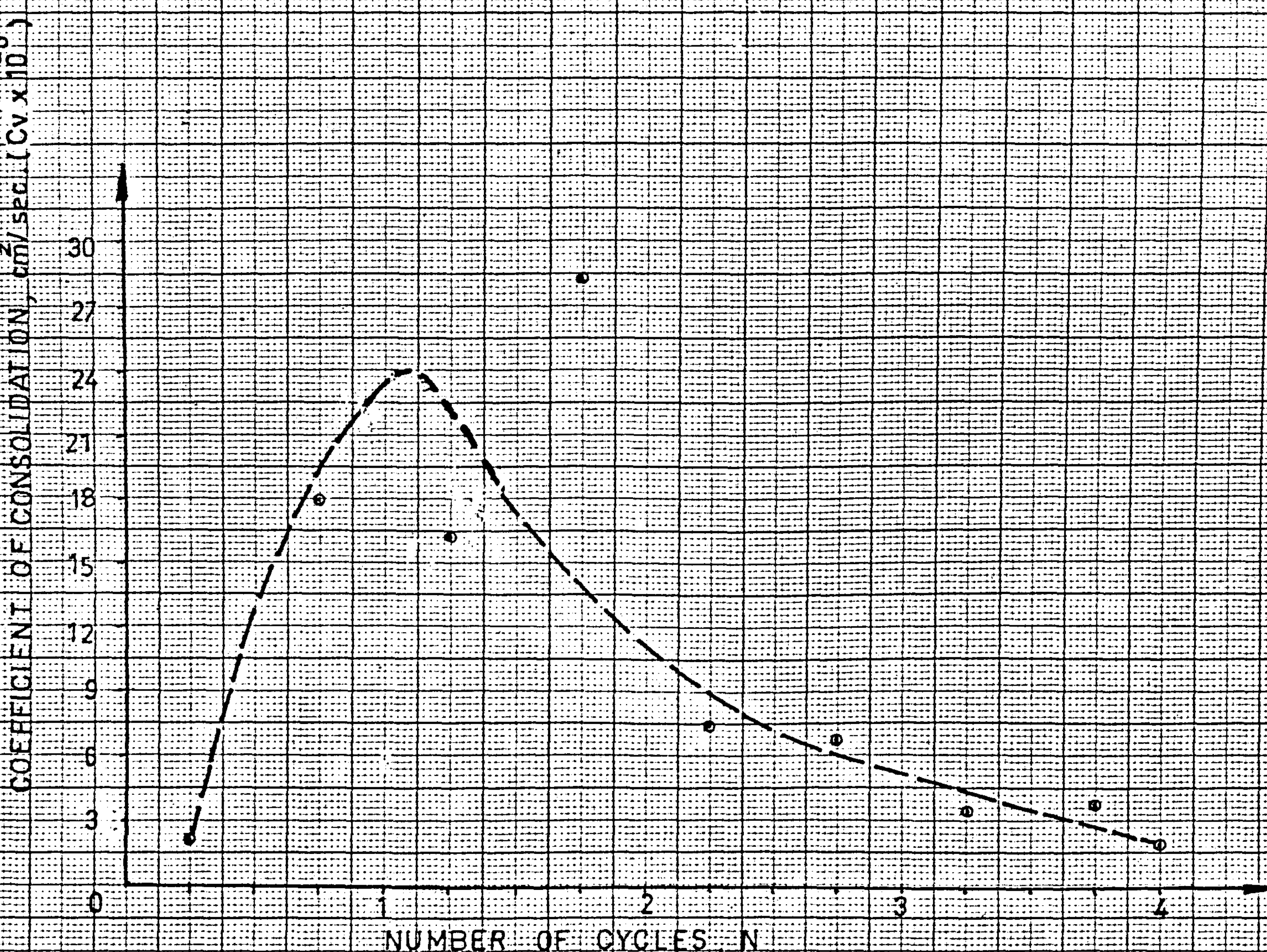


FIG- 4.14 Coefficient of Consolidation versus Number of Cycles.

TEST 2

$w_i = 44$ %

$w_f = 41$ %

Sustained Load, σ_o = 1.0 kg/cm^2

Load Intensity, $\frac{\sigma_o}{\sigma_d}$ = 20 %

Number of Cycles, N = 3

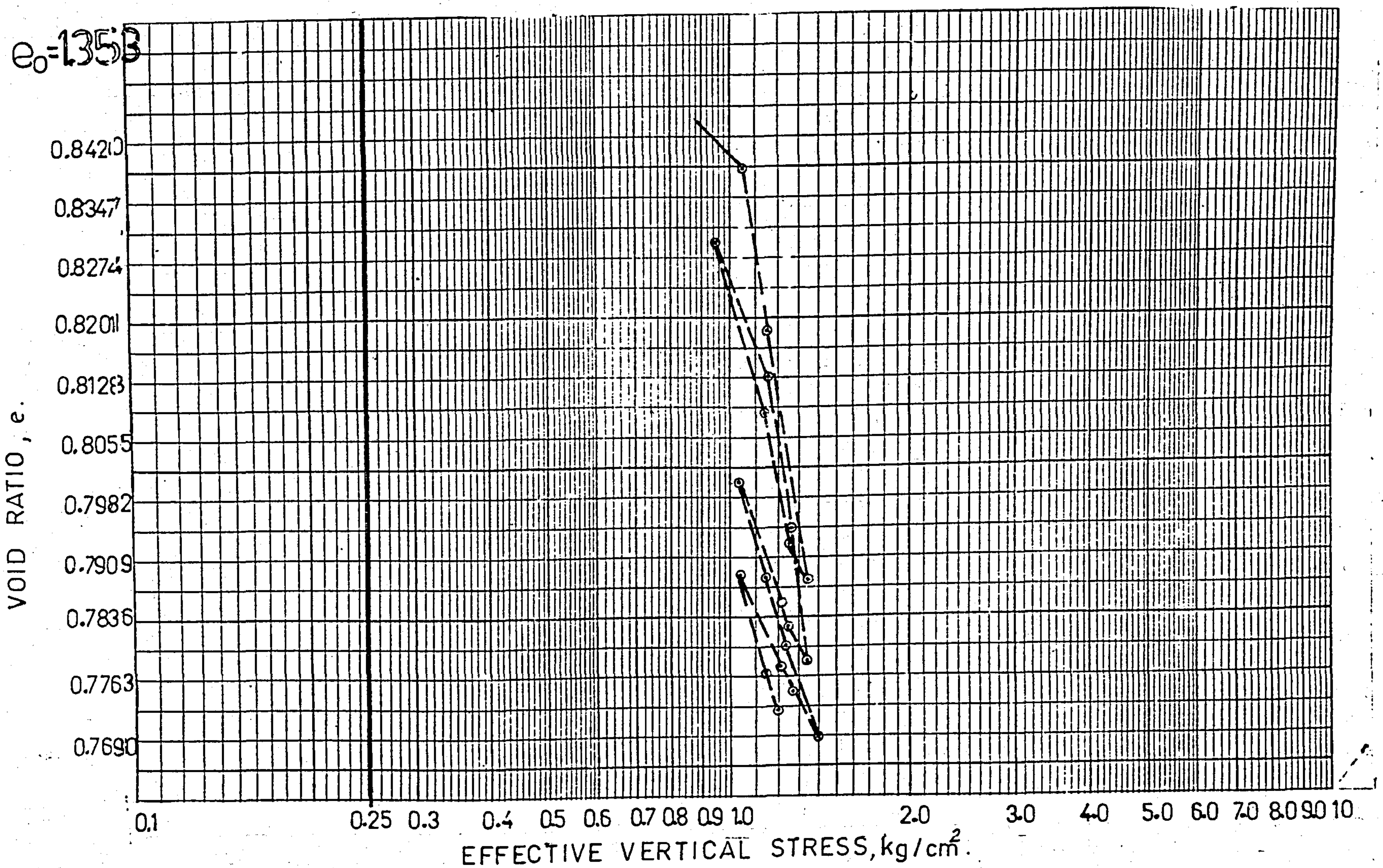


FIG-4.15 Void Ratio versus Effective Vertical Stress

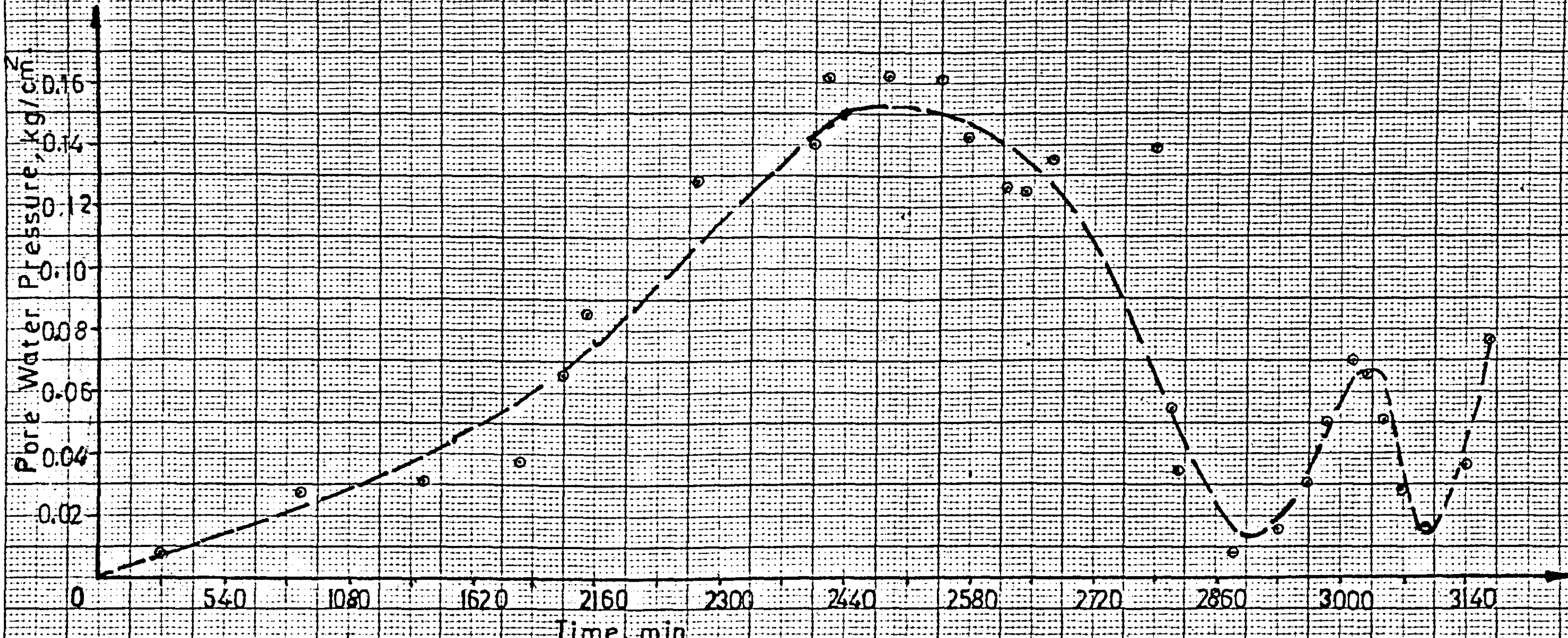


FIG-4.16 Pore Water Pressure versus Time

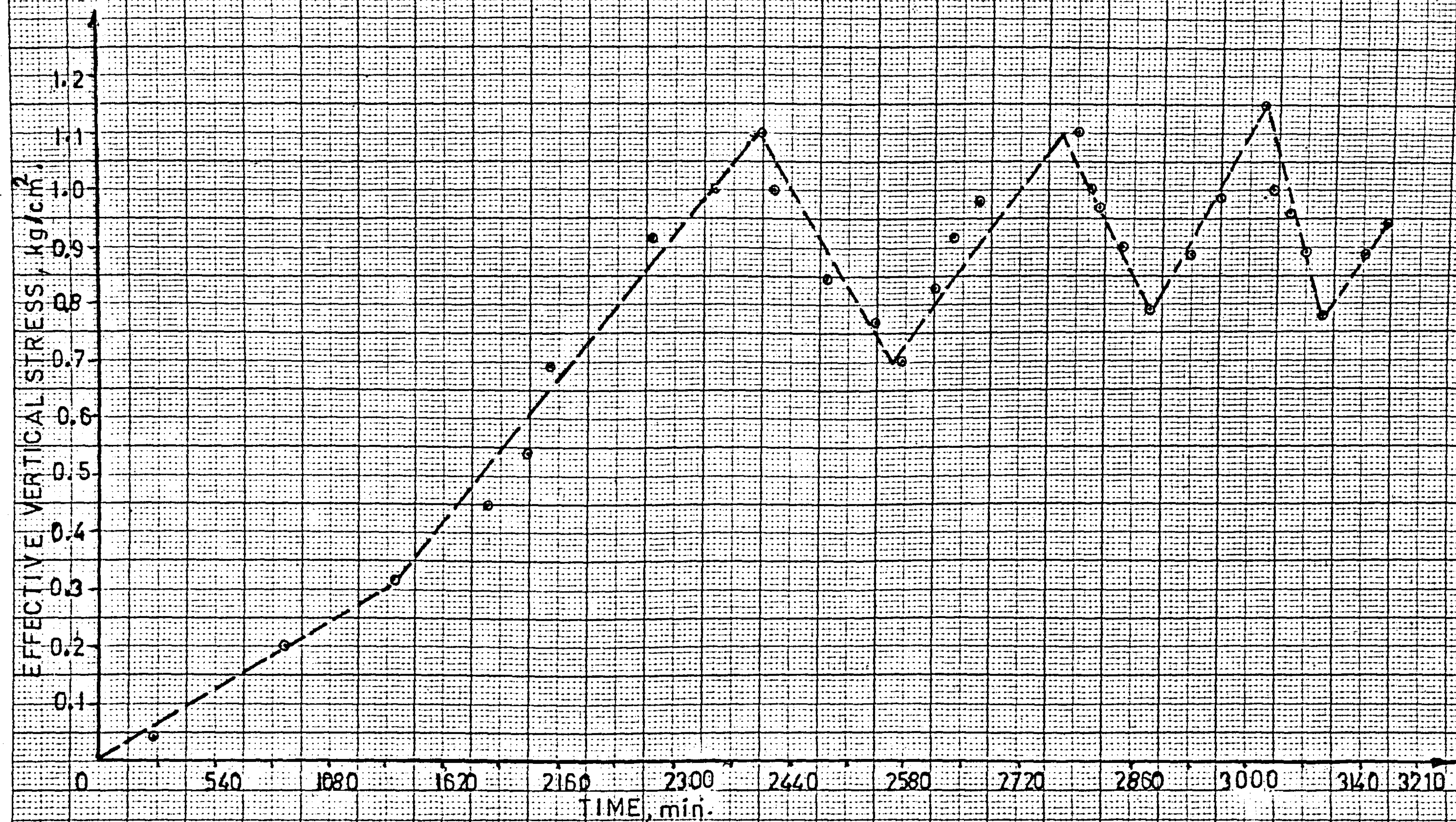
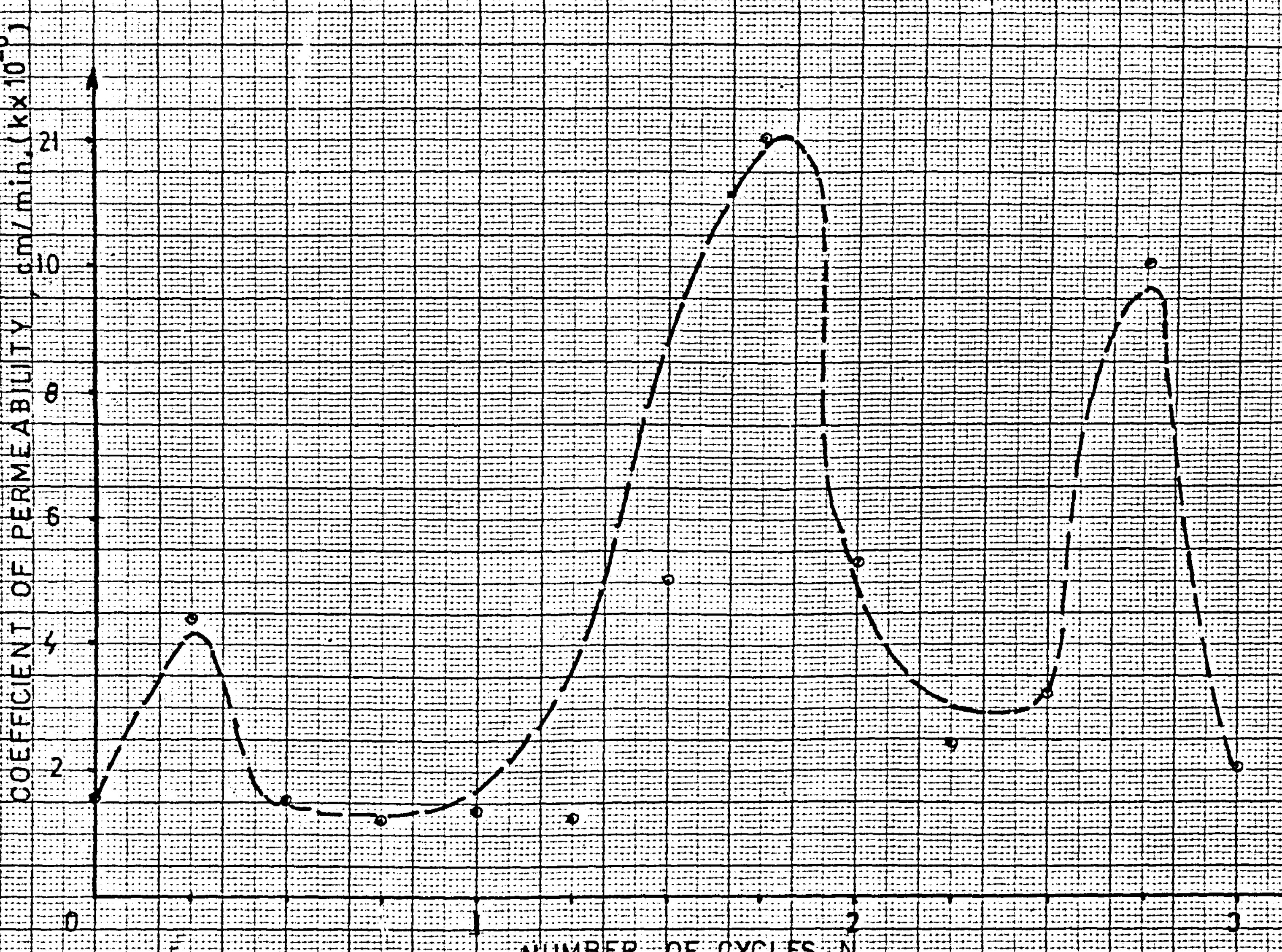


FIG 4.17 EFFECTIVE Vertical Stress versus Time



FIG—4.18 Coefficient of Permeability versus Number of Cycles.

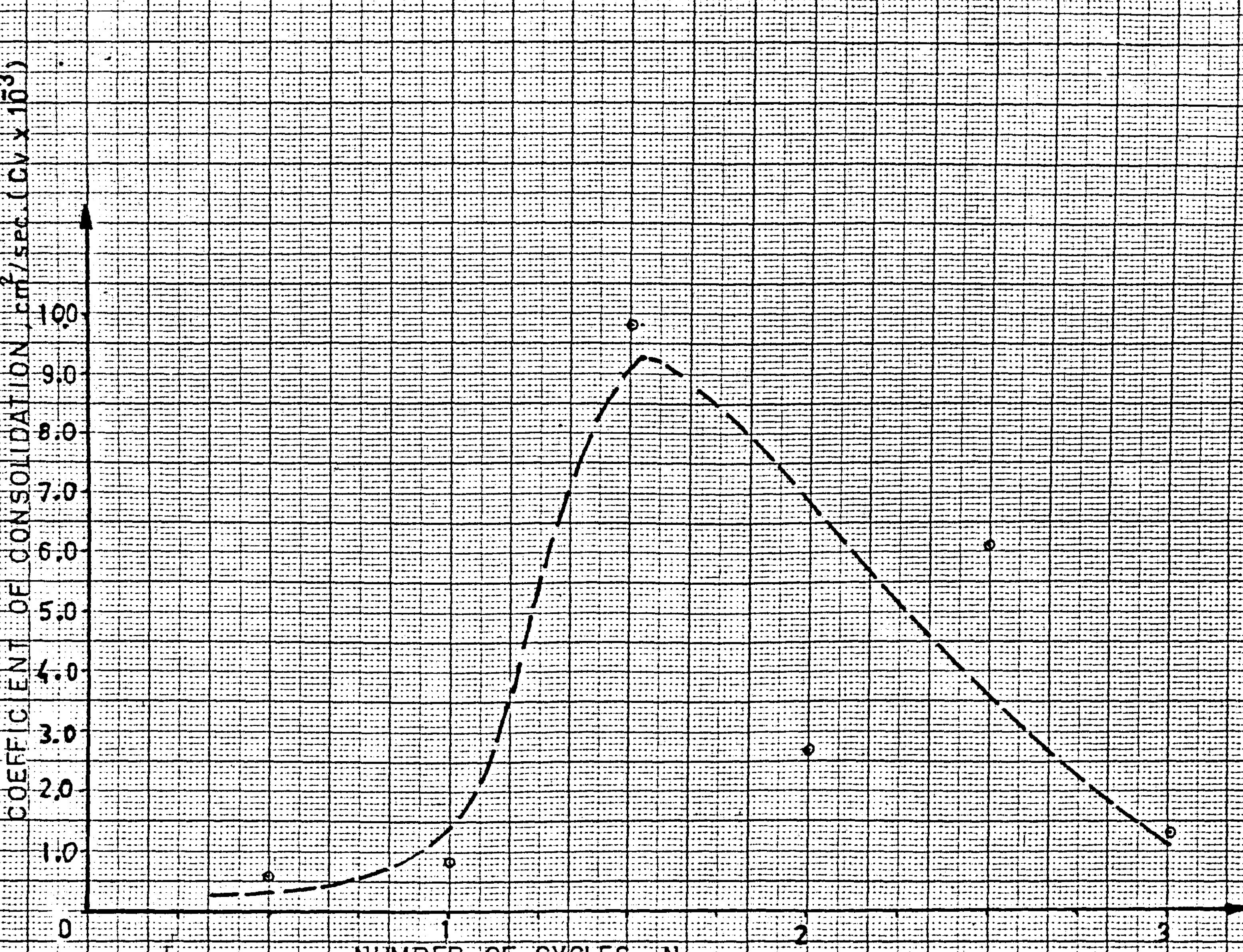


FIG-4.19 Coefficient of Consolidation versus Number of Cycles

TEST 3

$w_i = 45$ %

$w_f = 44$ %

Sustained Load, $\sigma_o = 2.0$ kg/cm^2

Load Intensity, $\frac{\sigma_o}{\sigma_d} = 20$ %

Number of Cycles, $N = 5$

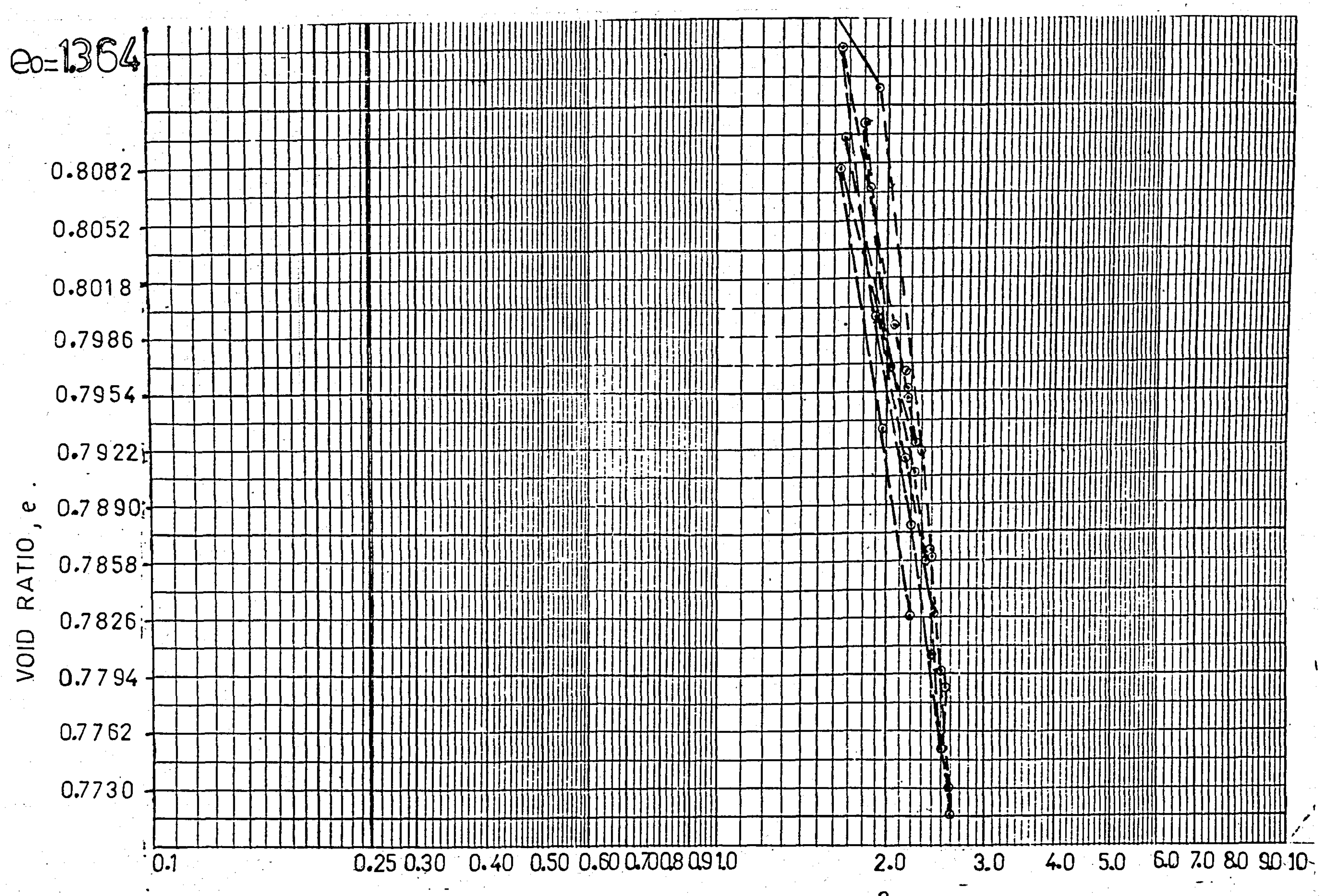


FIG-4.20 Void Ratio versus Effective Vertical Stress.

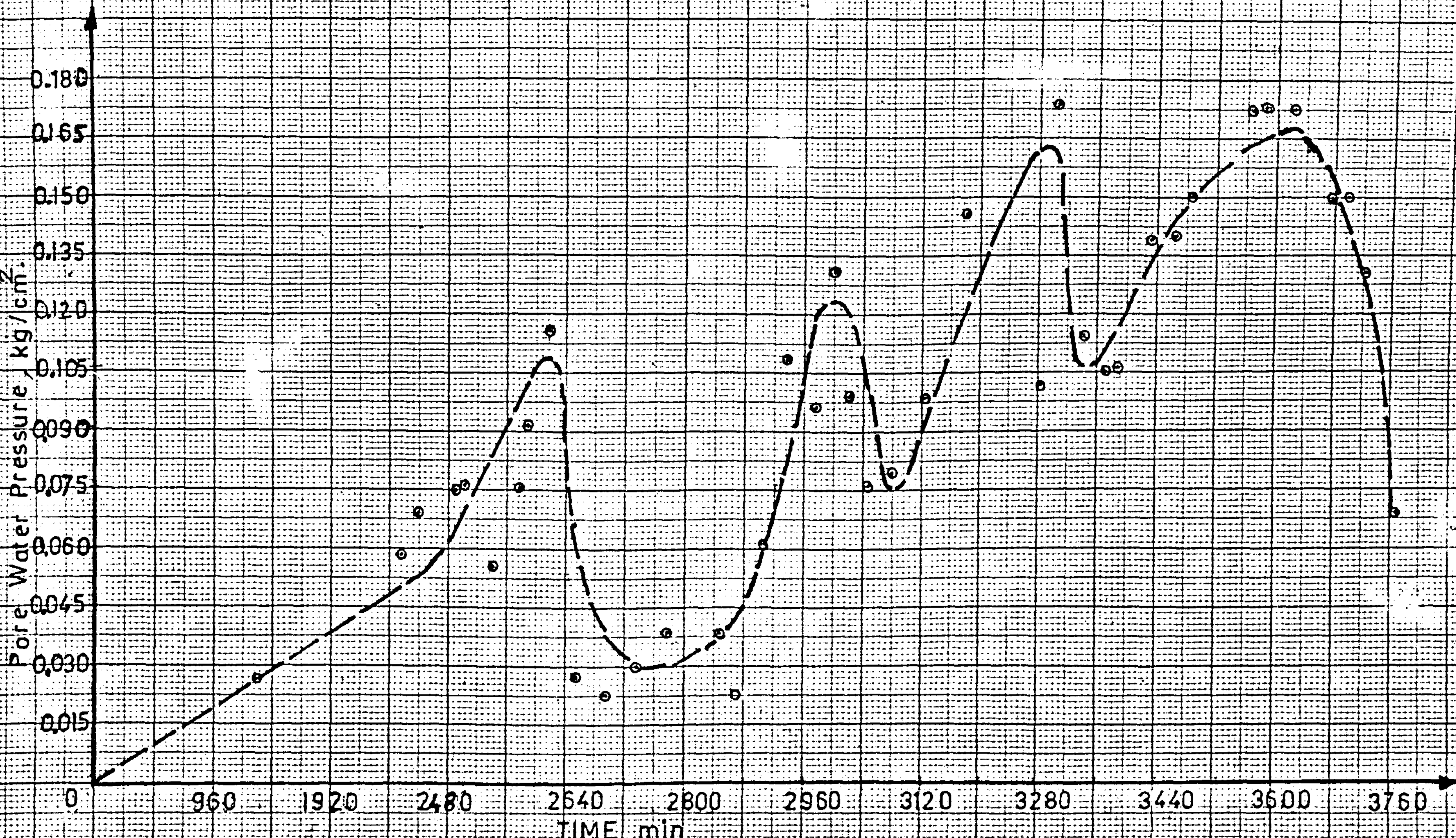


FIG-4.21 Pore Water Pressure versus Time

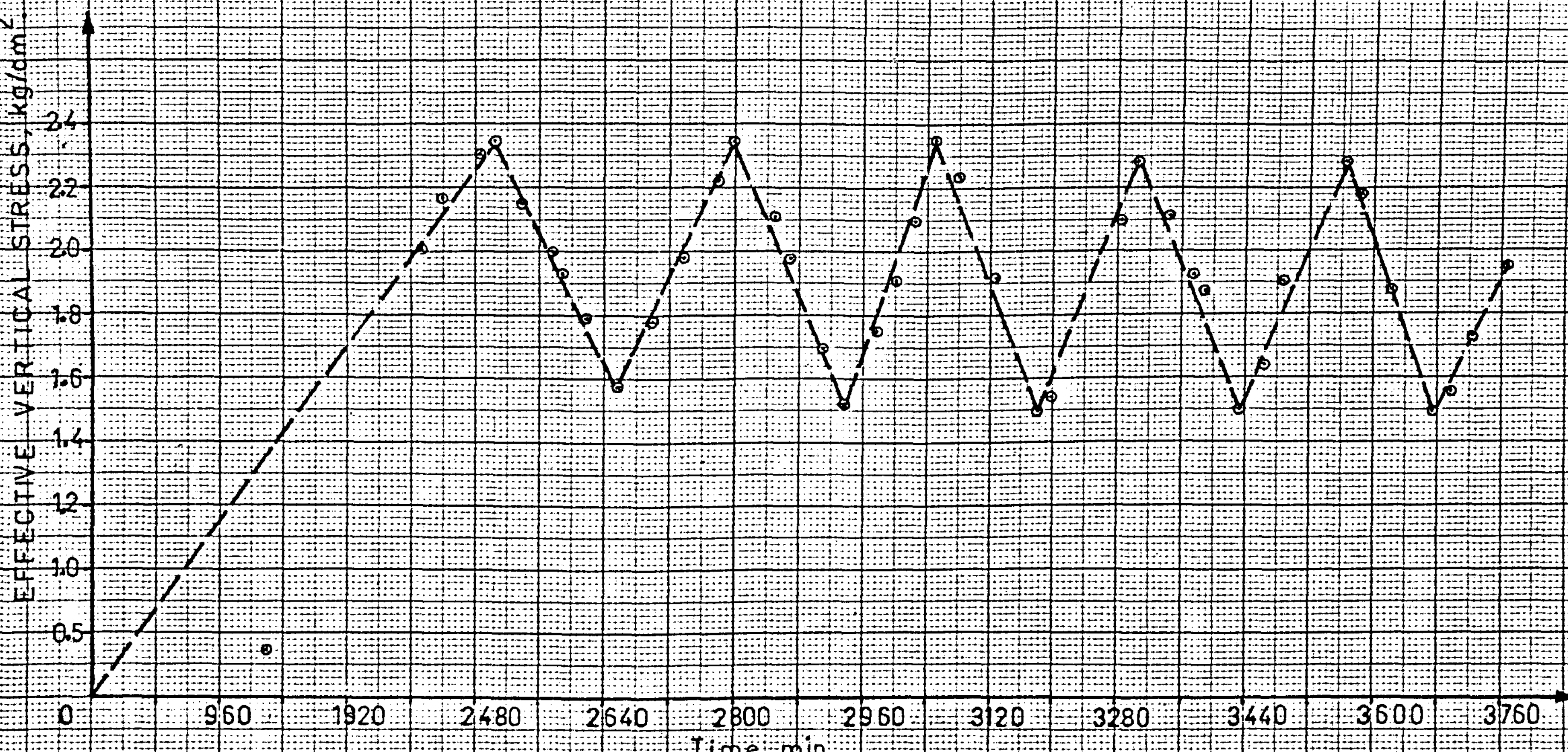


FIG. 4.22 Effective Vertical Stress versus Time.

FIG-4.23 Coefficient of Permeability versus Number of Cycles.

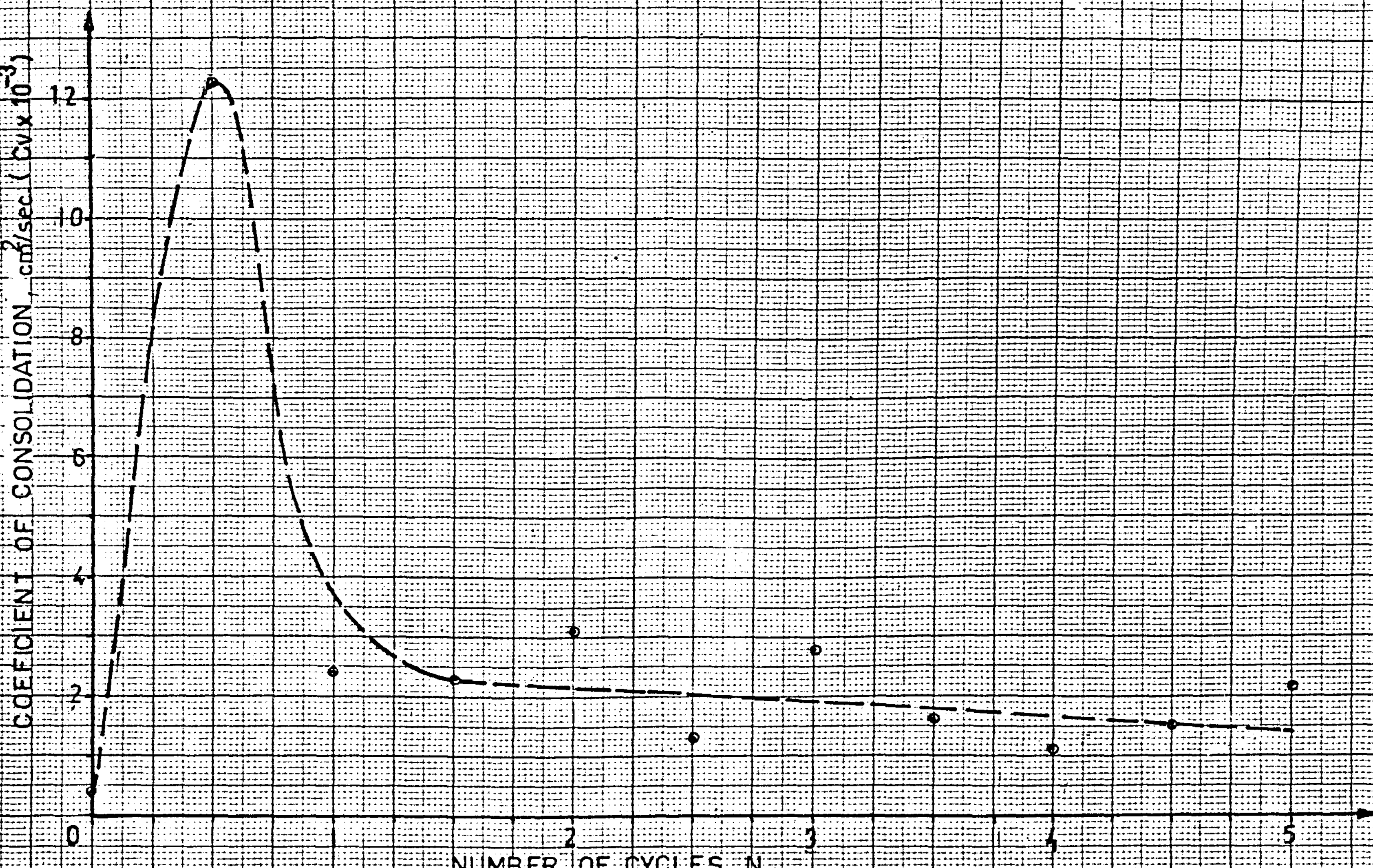


FIG 4.24 Coefficient of Consolidation versus Number of Cycles

TEST 4

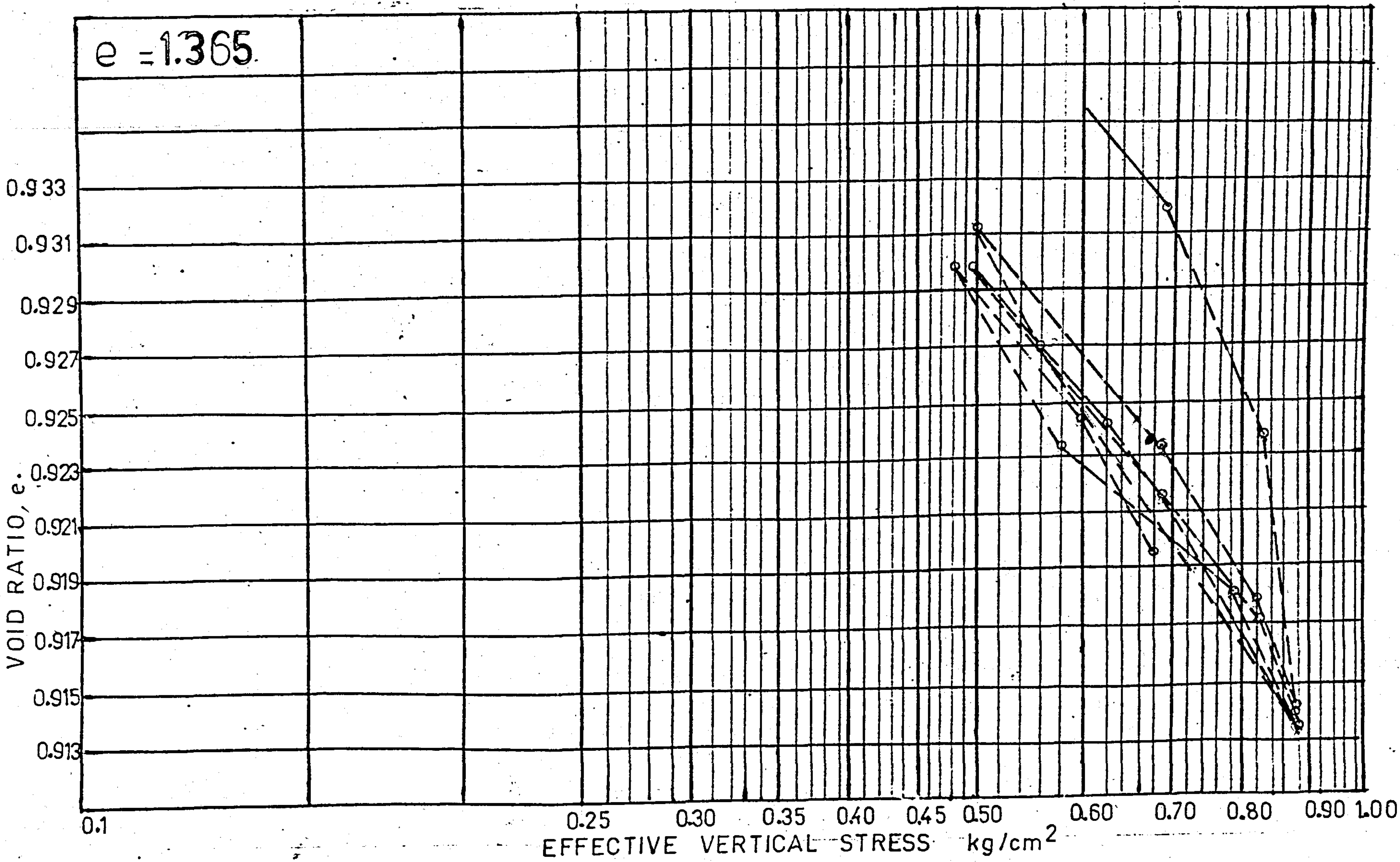
$w_i = 46\ \%$

$w_f = 43\ \%$

Sustained Load, $\sigma_o = 0.5\ kg/cm^2$

Load Intensity, $\frac{\sigma_o}{\sigma_d} = 40\ \%$

Number of Cycles, $N = 3$



FIG—4.25 Void Ratio versus Effective Vertical Stress

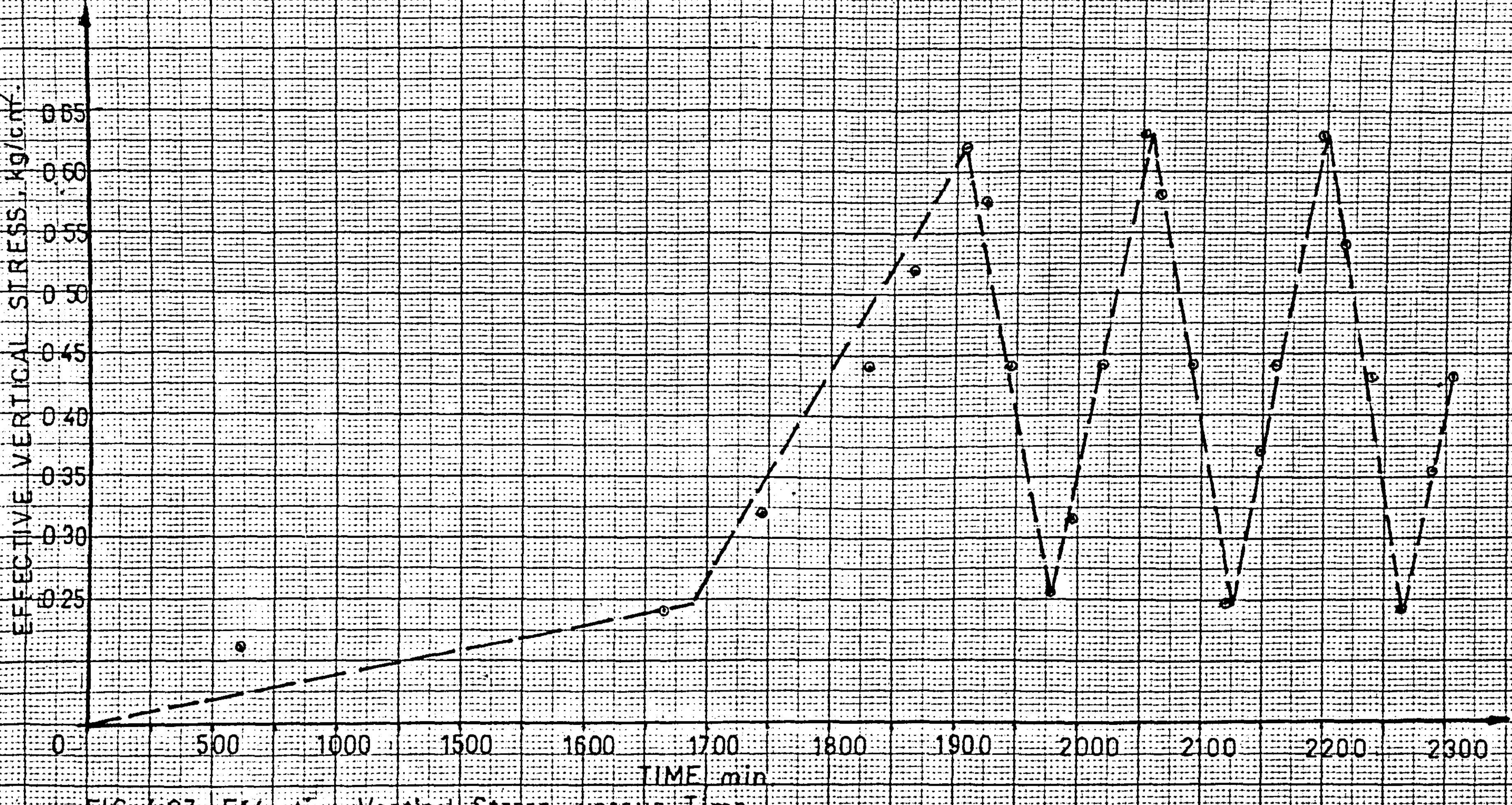


FIG 4.27 Effective Vertical Stress versus Time

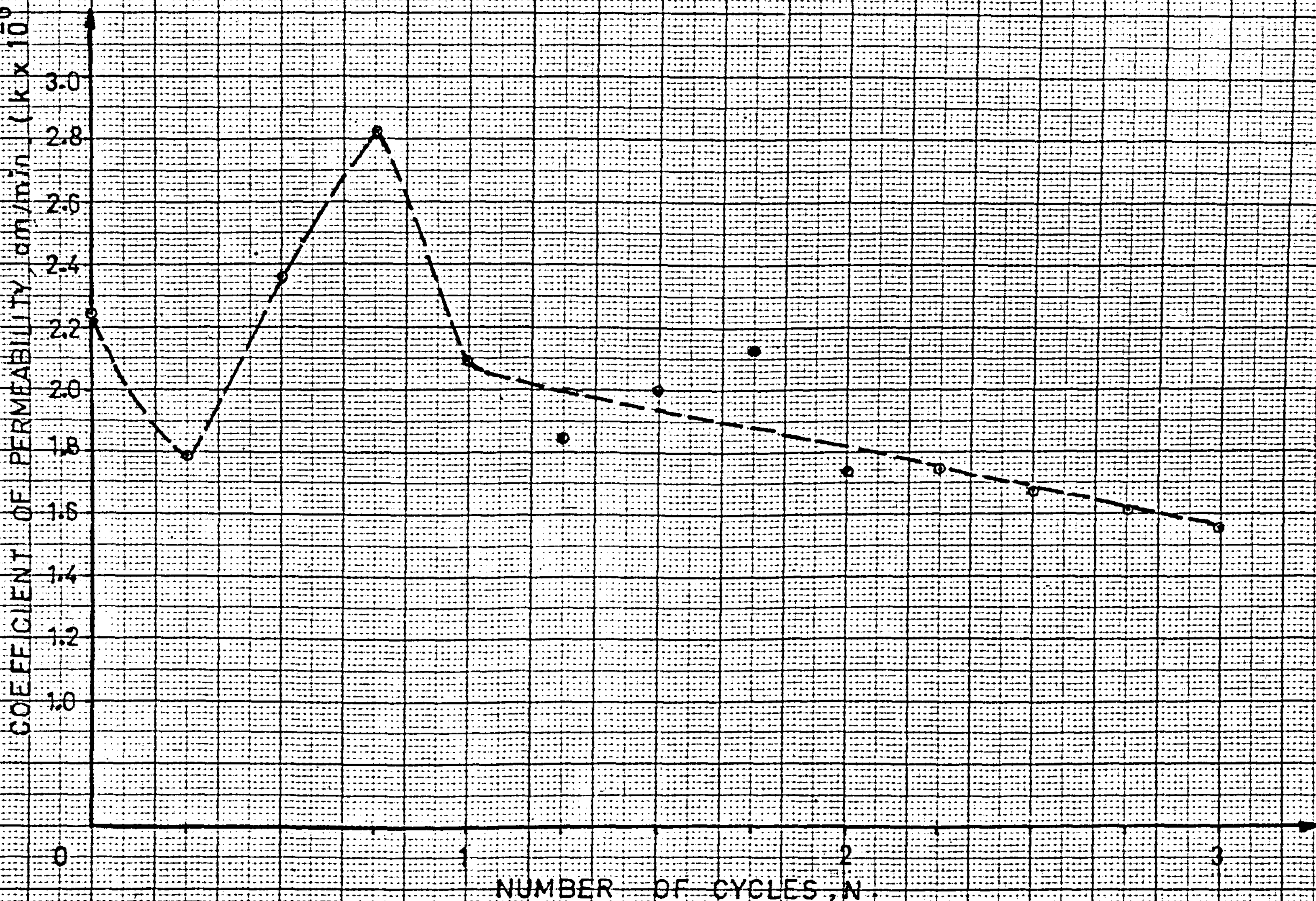


FIG-4.28 Coefficient of Permeability versus Number of Cycles.

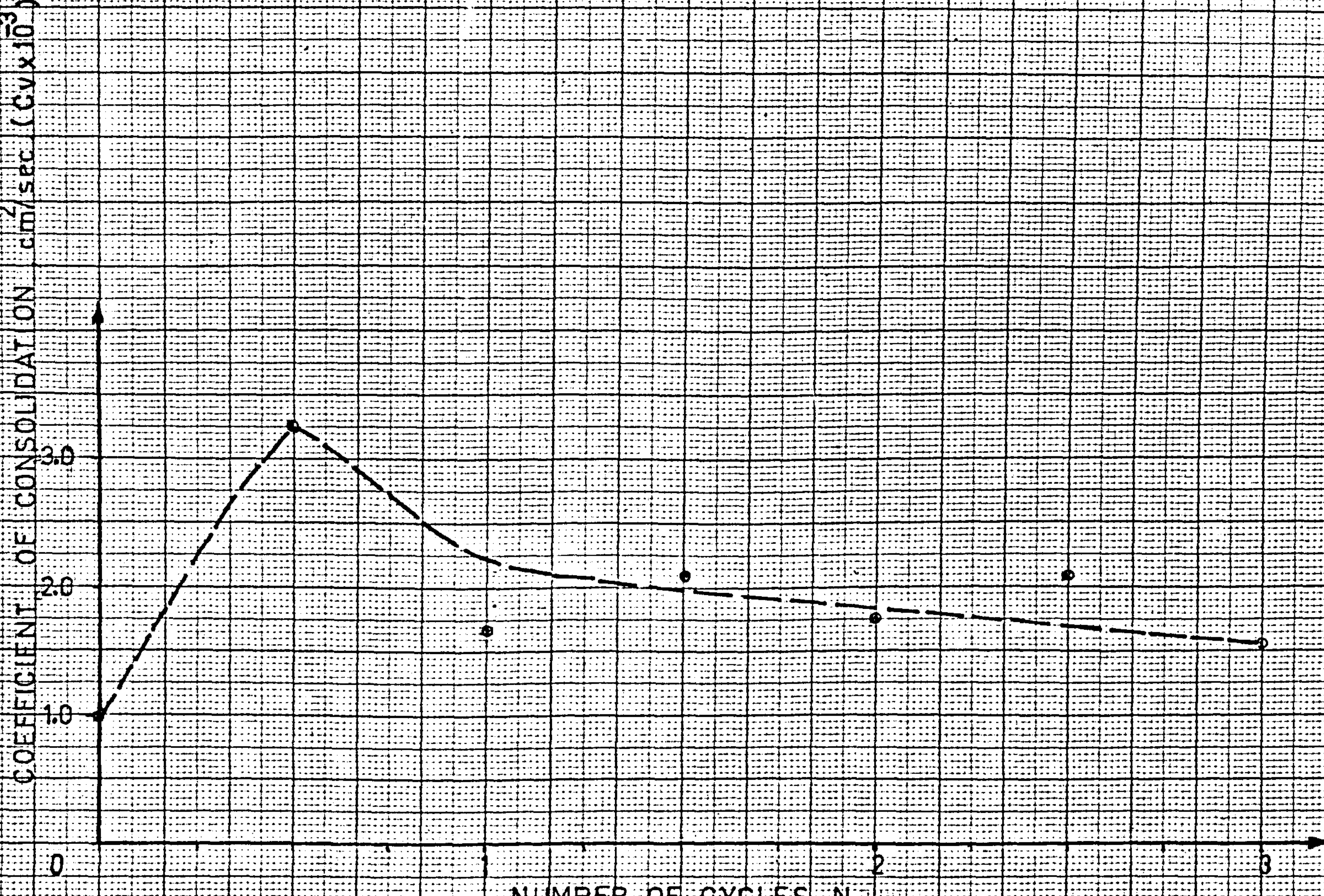


FIG-4.29 Coefficient of Consolidation versus Number of Cycles.

TEST 5

$w_i = 44\ \%$

$w_f = 40\ \%$

Sustained Load, $\sigma_o = 1.0\ kg/cm^2$

Load Intensity, $\frac{\sigma_o}{\sigma_d} = 40\ \%$

Number of Cycles, $N = 5$

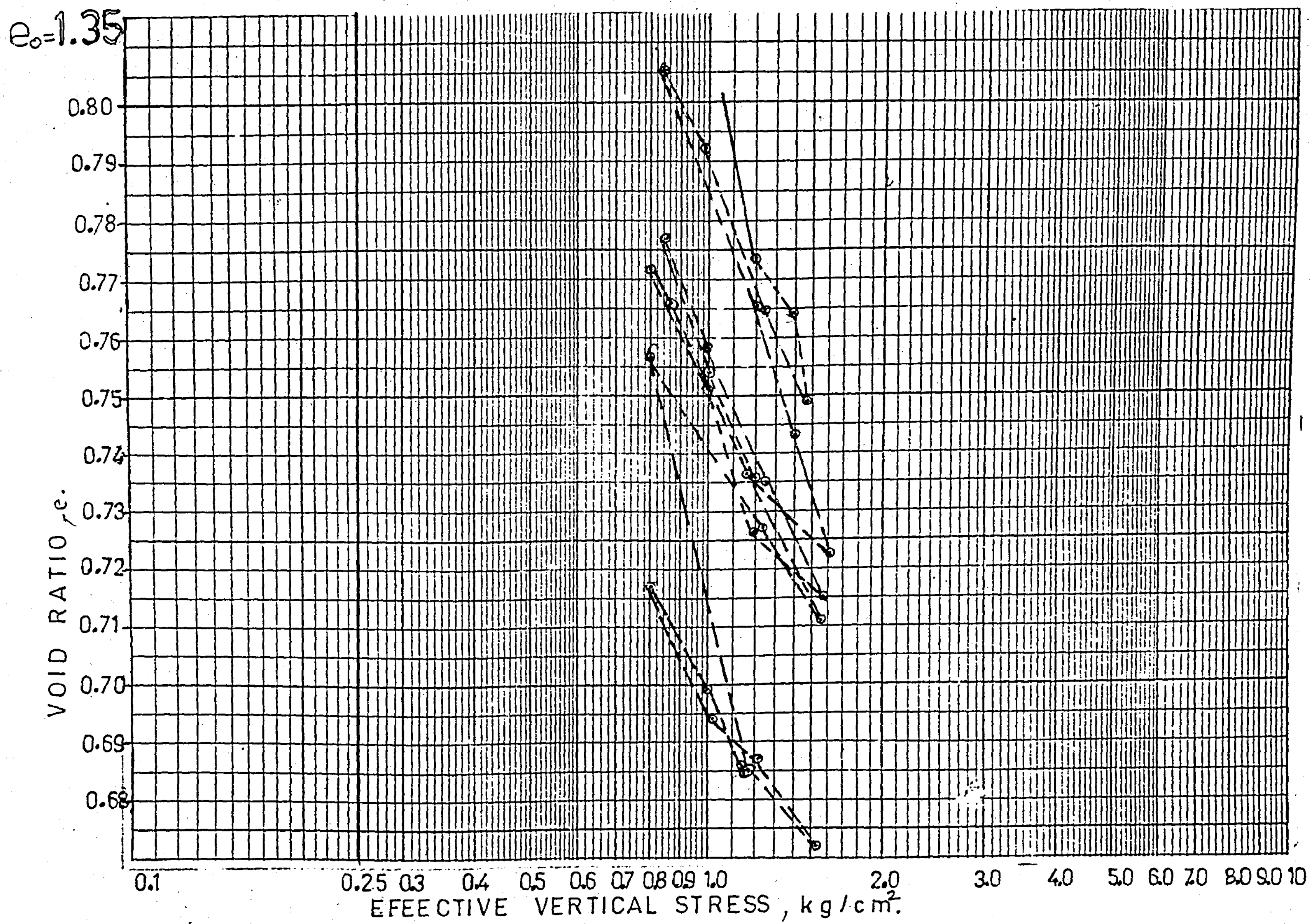


FIG_4.30 Void Ratio versus Effective Vertical Stress.

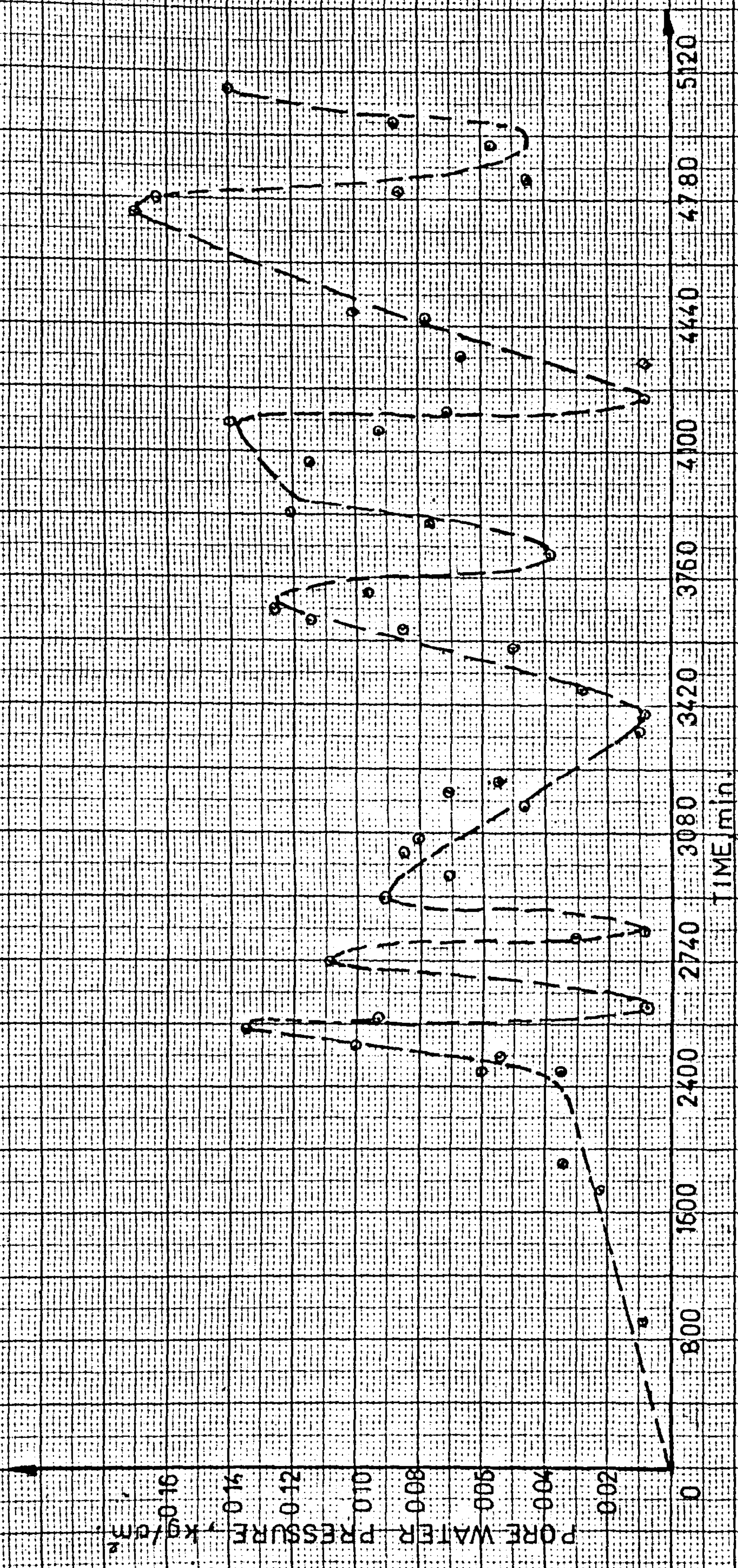


FIG-4.31 Pore Water Pressure versus Time

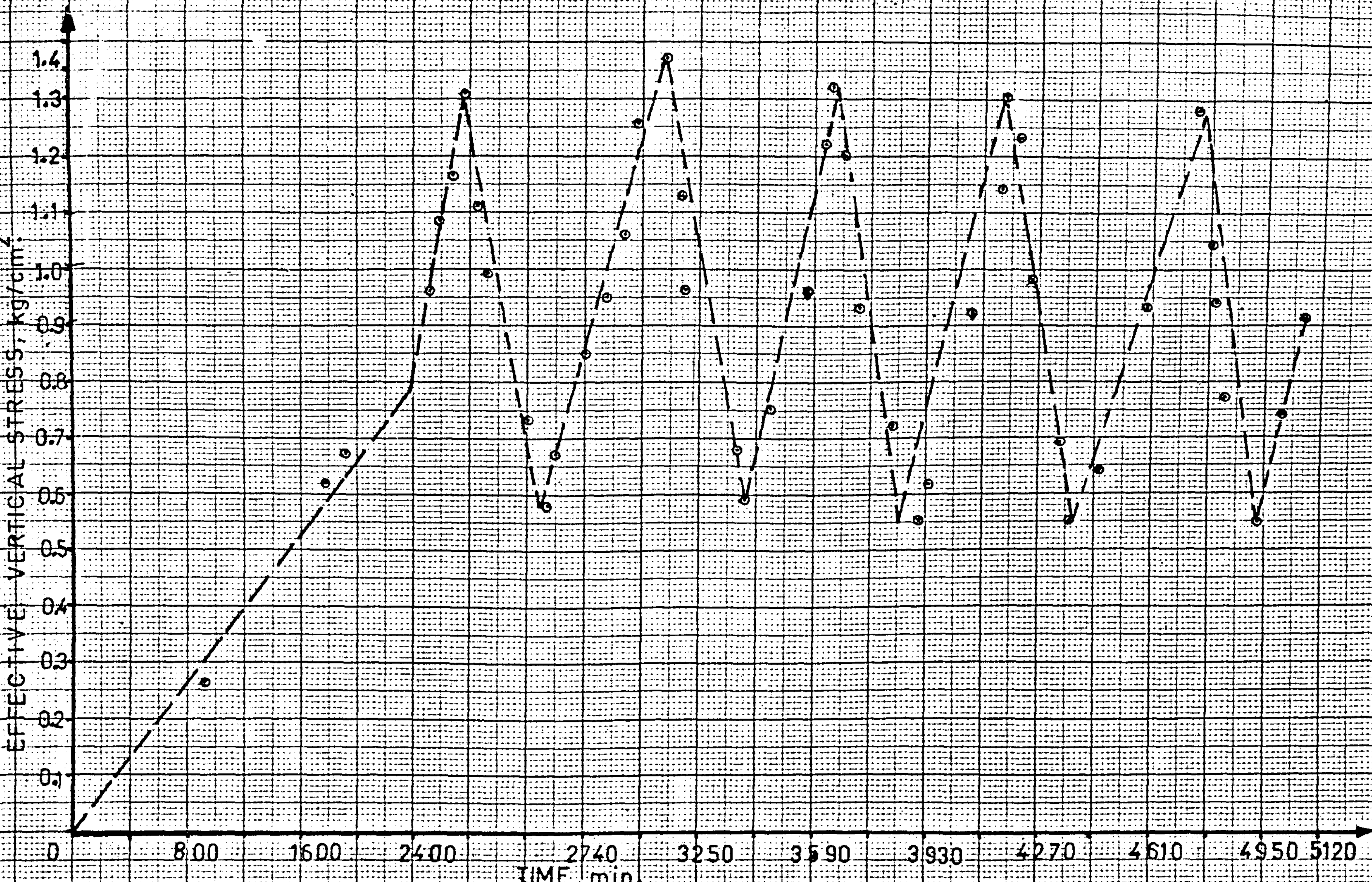


FIG-4.32 Effective Vertical Stress versus Time

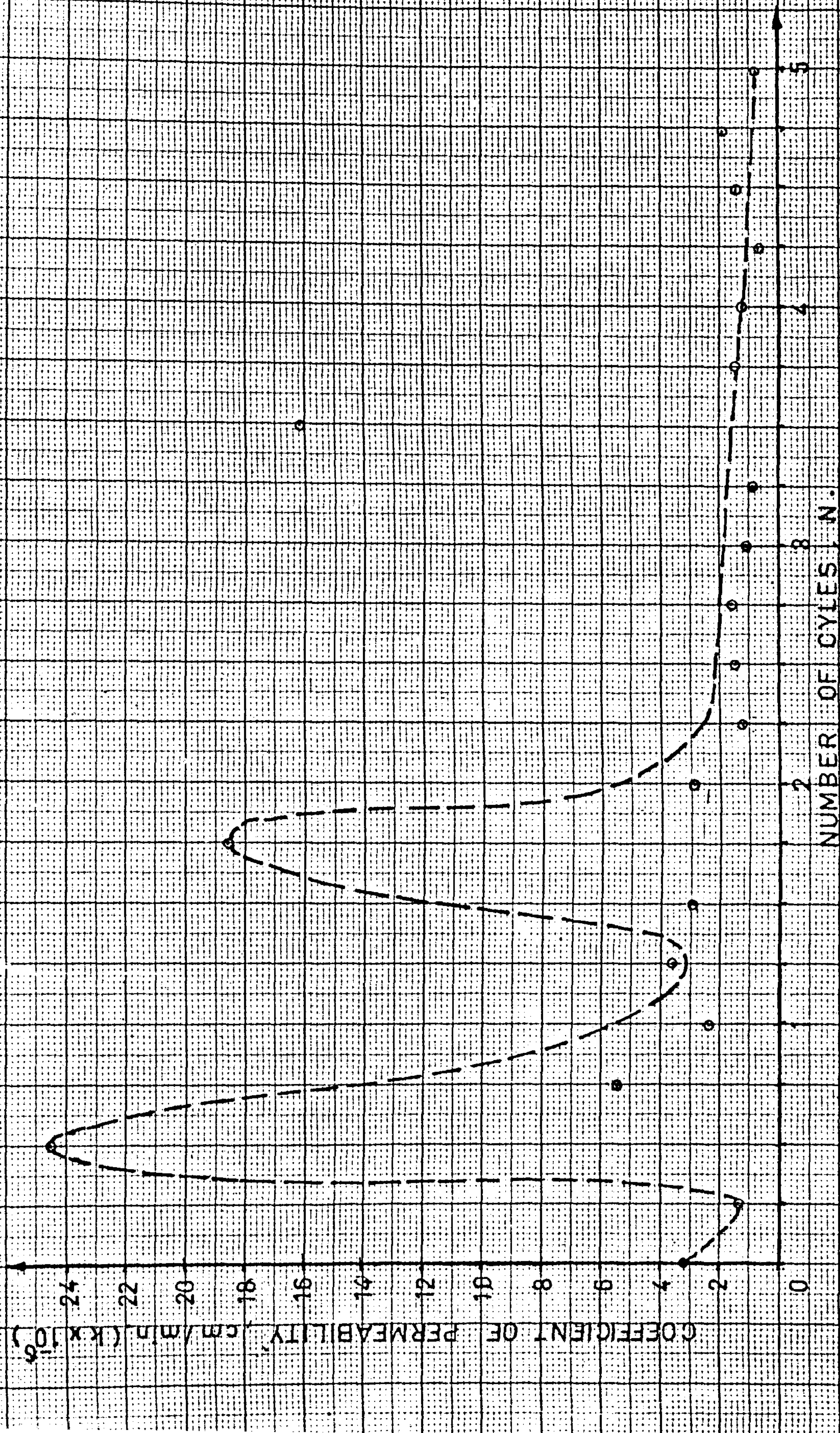


FIG 4.33 Coefficient of Permeability versus Number of Cycles.

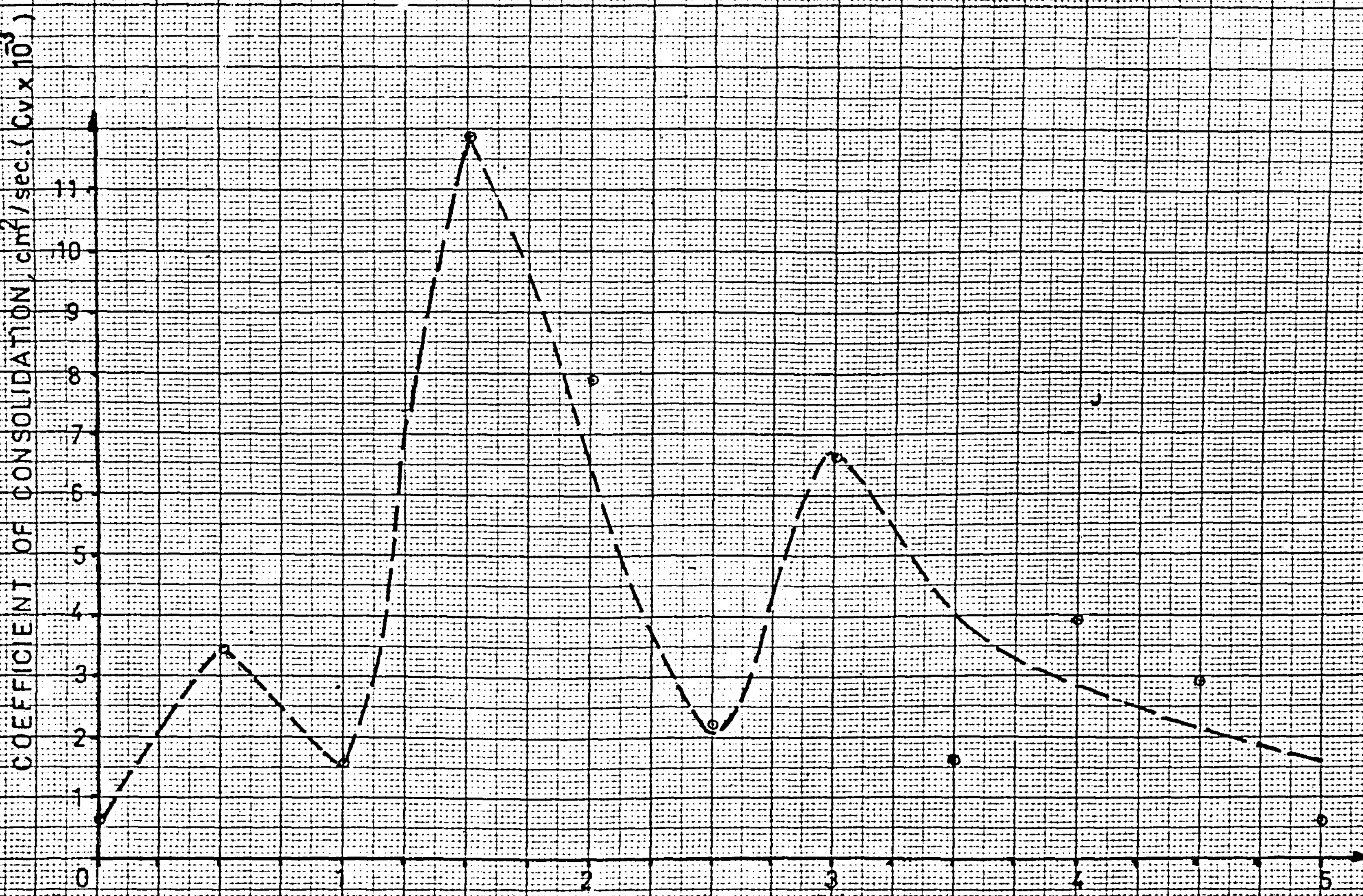


FIG-4.34 Coefficient of Consolidation versus Number of Cycles.

TEST 6

$w_i = 45$ %

$w_f = 41$ %

Sustained Load, $\sigma_o = 2.0$ kg/cm^2

Load Intensity, $\frac{\sigma_o}{\sigma_d} = 40$ %

Number of Cycles, $N = 4$

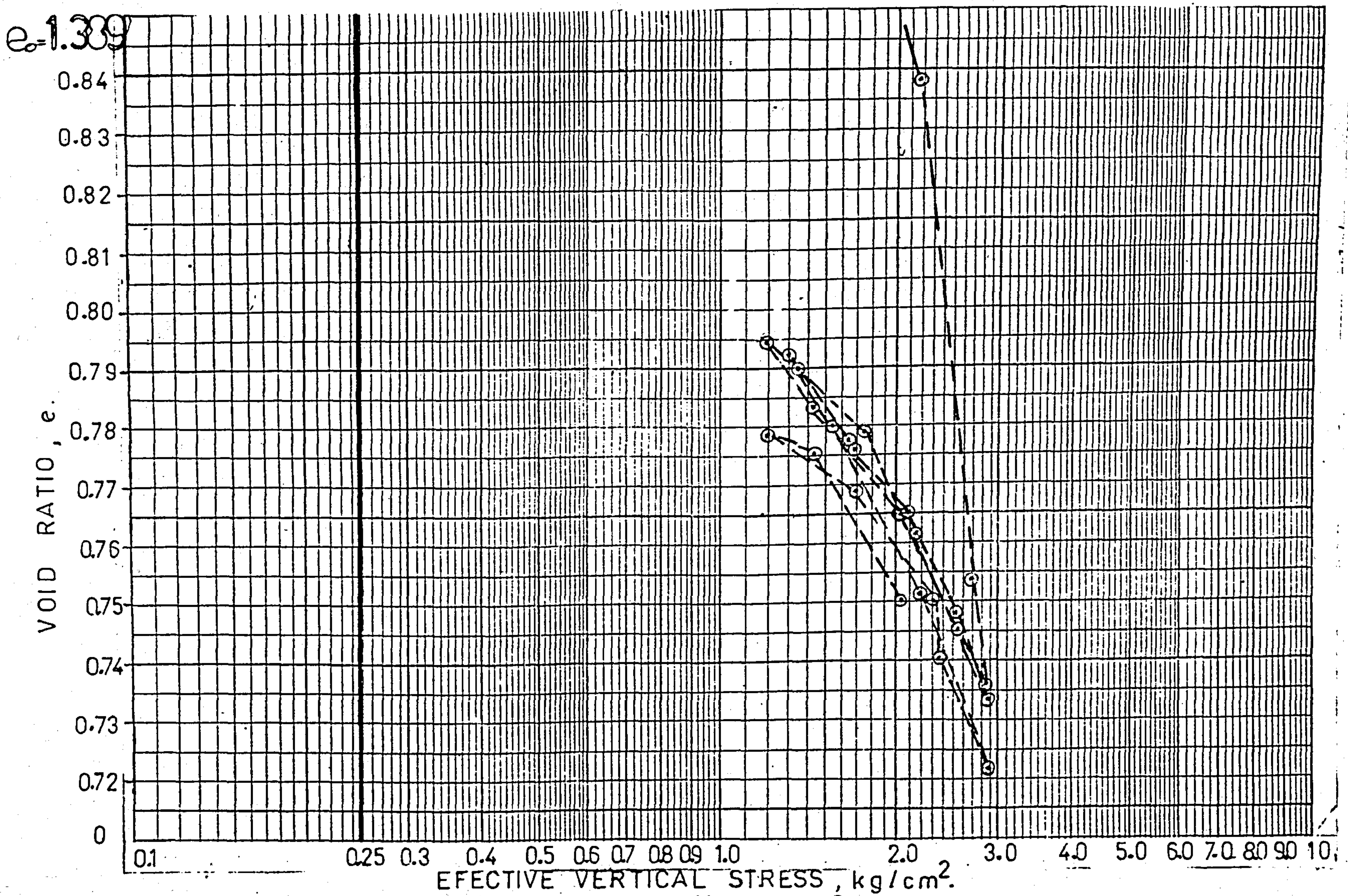


FIG-4.35 Void Ratio versus Effective Vertical-Stress.

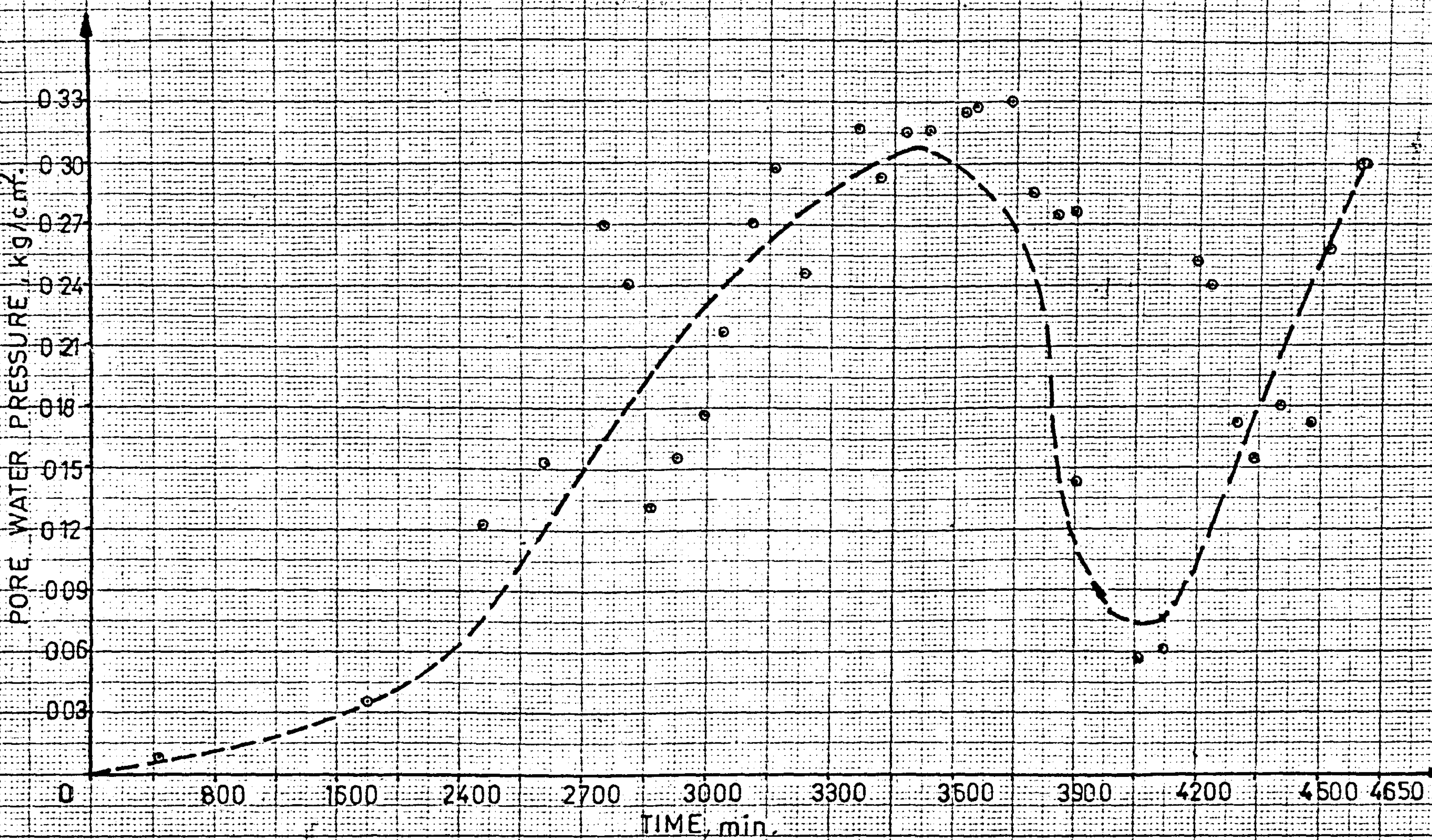


FIG.4.36 Pore Water Pressure versus Time.

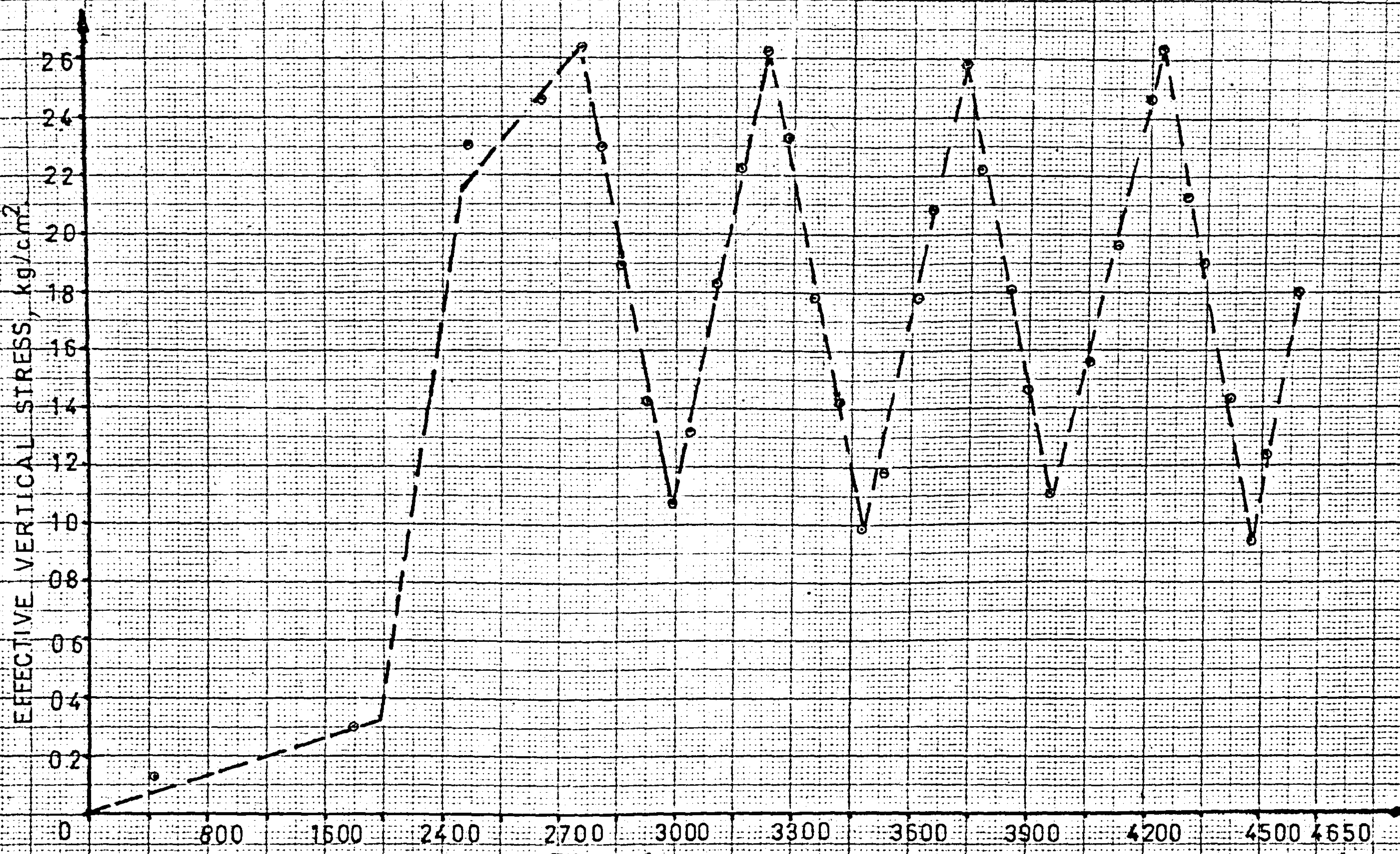


FIG_ 4.37 Effective Vertical Stress versus Time.

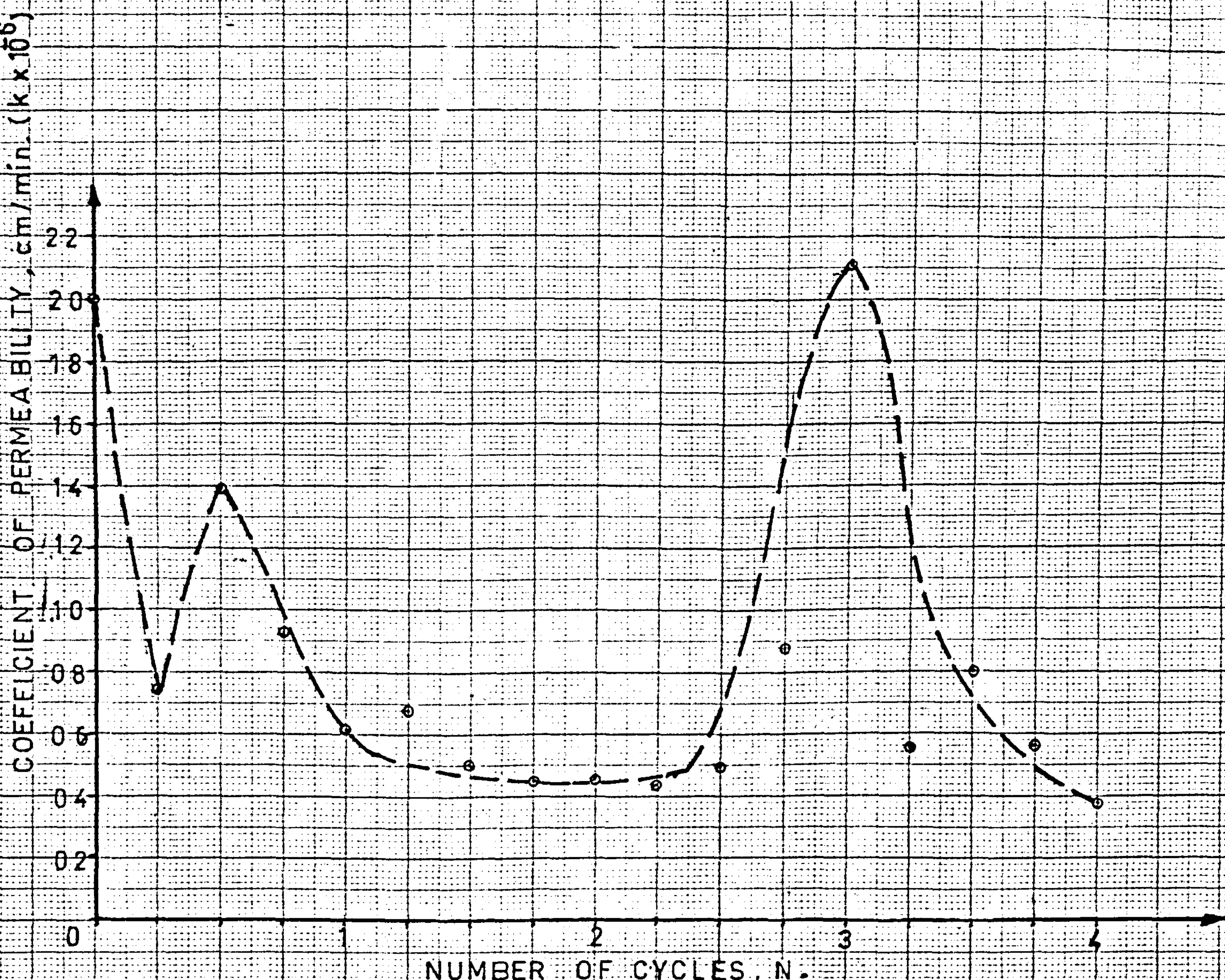


FIG 4.38 Coefficient of Permeability versus Number of Cycles.

FIG-4.39 Coefficient of Consolidation versus Number of Cycles.

TEST 7

$w_i = 44$ %

$w_f = 42$ %

Sustained Load, $\sigma_o = 0.5$ kg/cm^2

Load Intensity, $\frac{\sigma_o}{\sigma_d} = 60$ %

Number of Cycles, $N = 5$

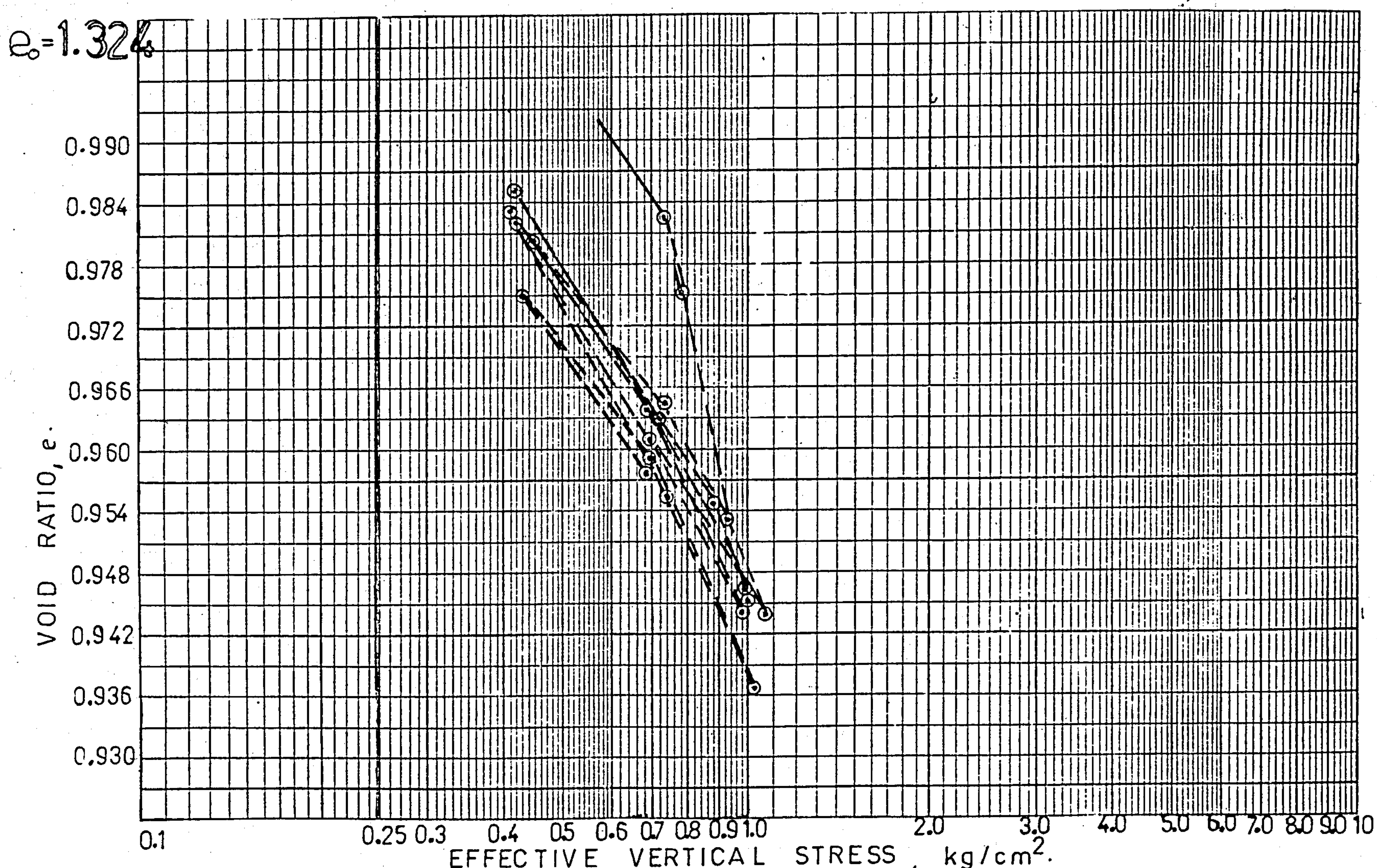


FIG_4.40 Void Ratio versus Effective Vertical Stress.

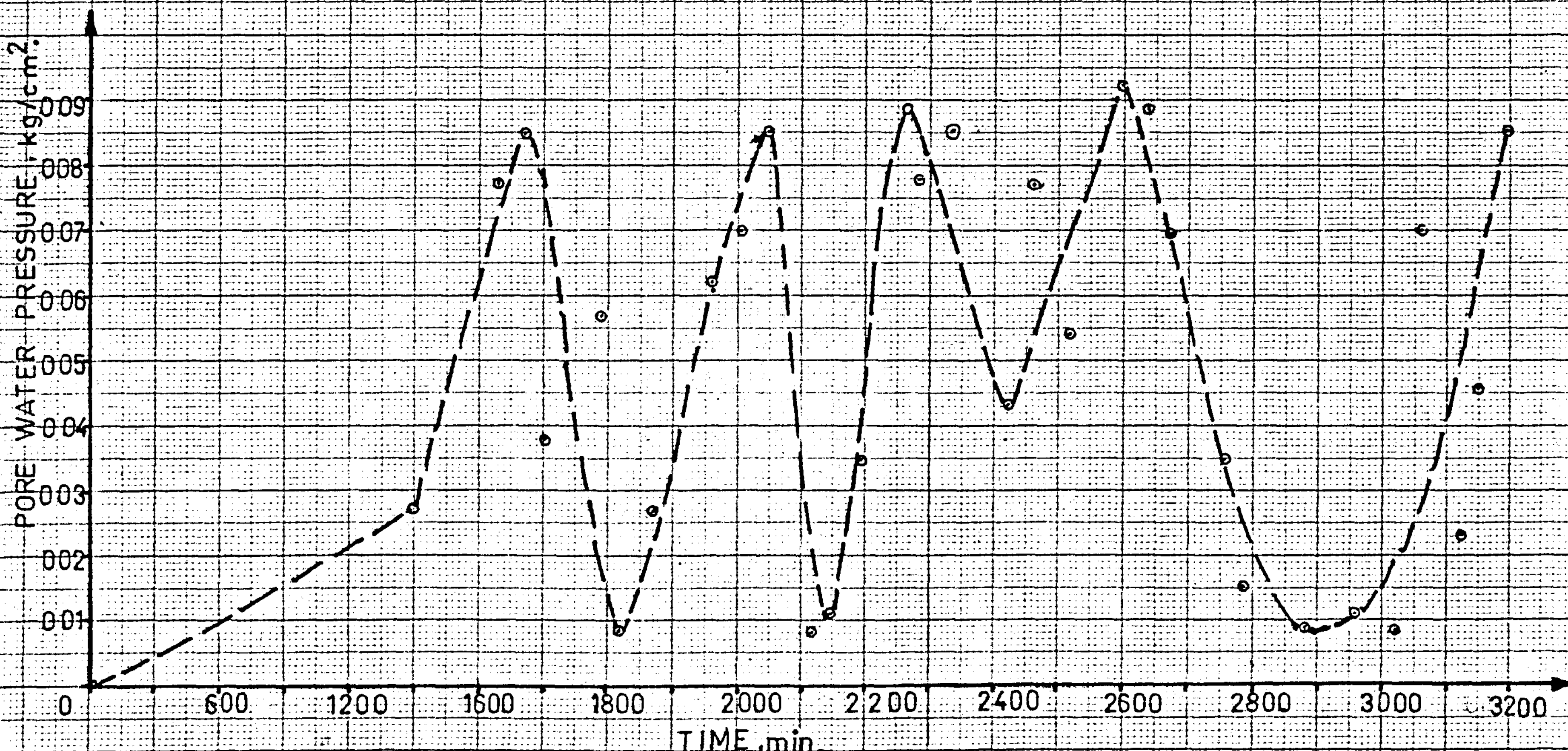
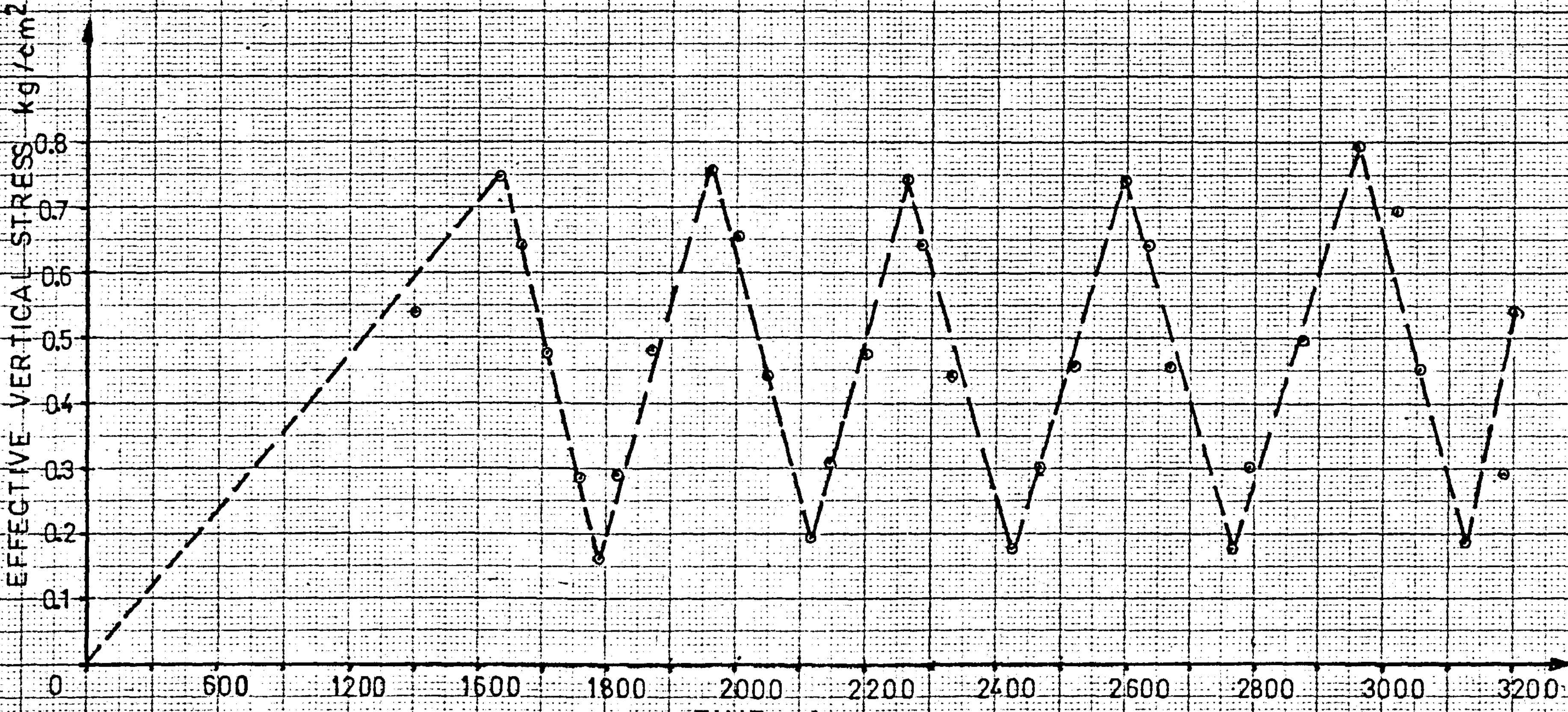


FIG_4.41 Pore Water Pressure versus Time.



FIG— 4.42 Effective Vertical Stress versus Time.

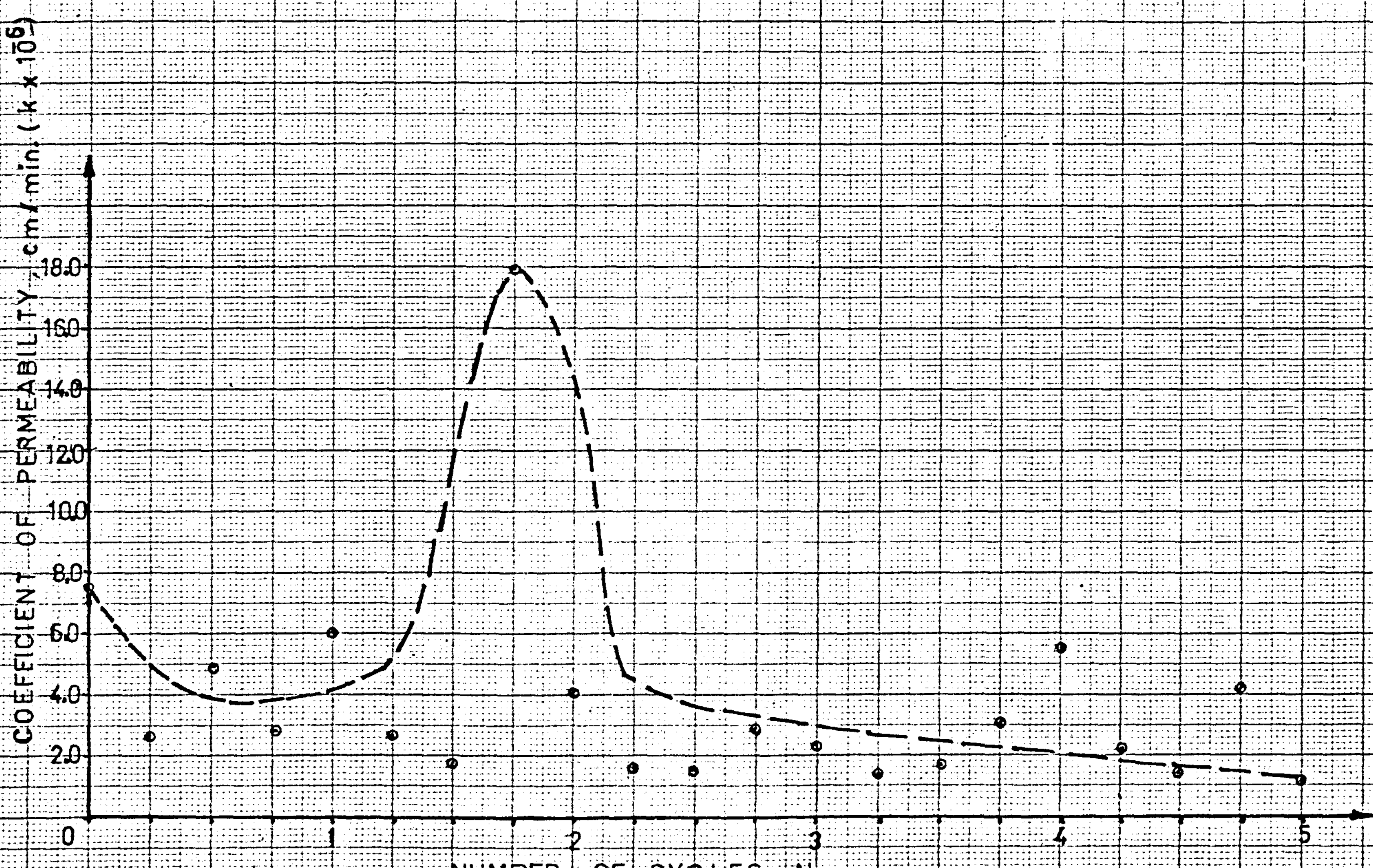


FIG. 4.43 Coefficient of Permeability versus Number of Cycles.

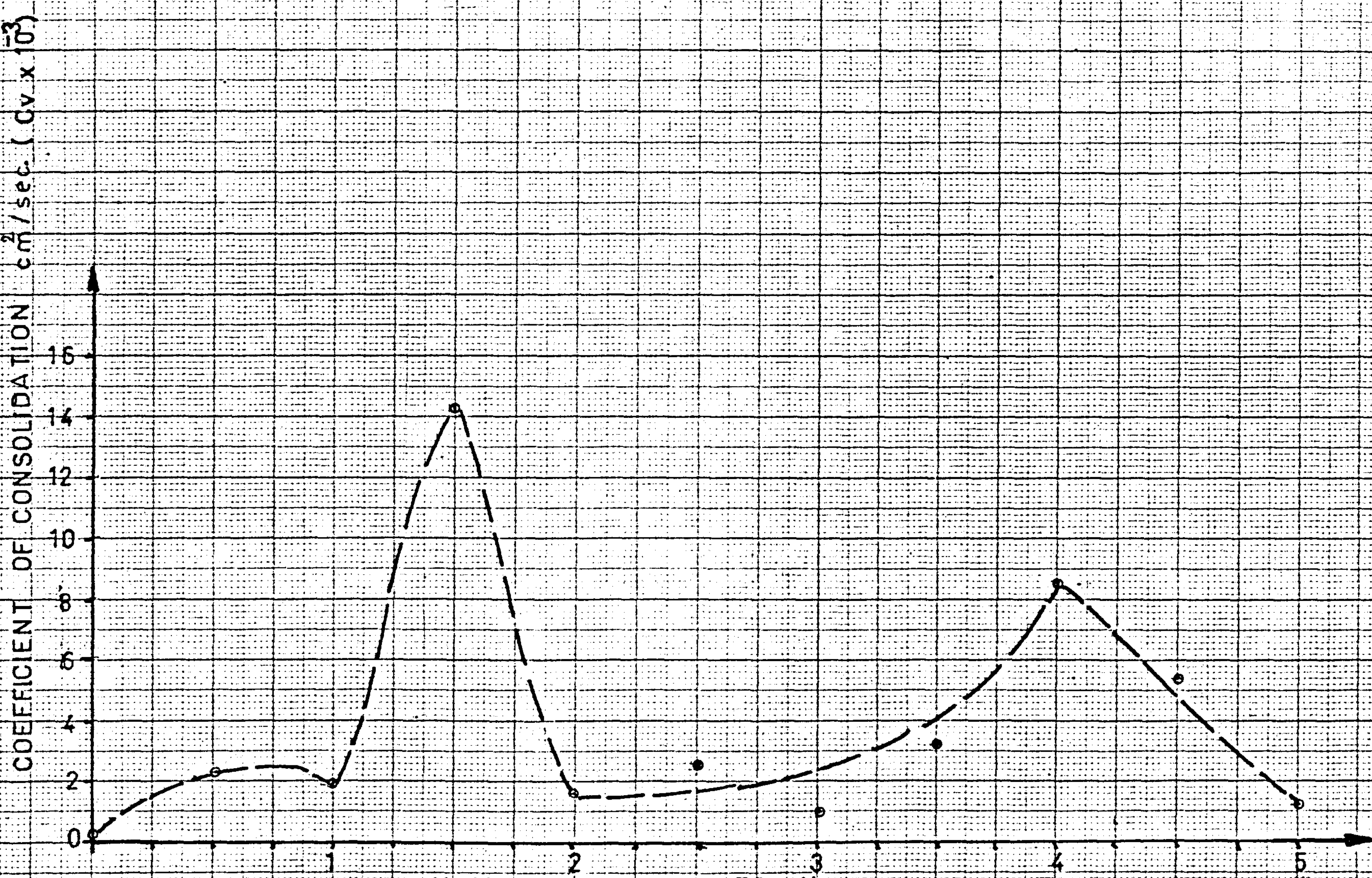


FIG. 4.44 Coefficient of Consolidation versus Number of Cycles.

TEST 8

$w_i = 44$ %

$w_f = 41$ %

Sustained Load, $\sigma_o = 1.0$ kg/cm^2

Load Intensity, $\frac{\sigma_o}{\sigma_d} = 60$ %

Number of Cycles, $N = 4$

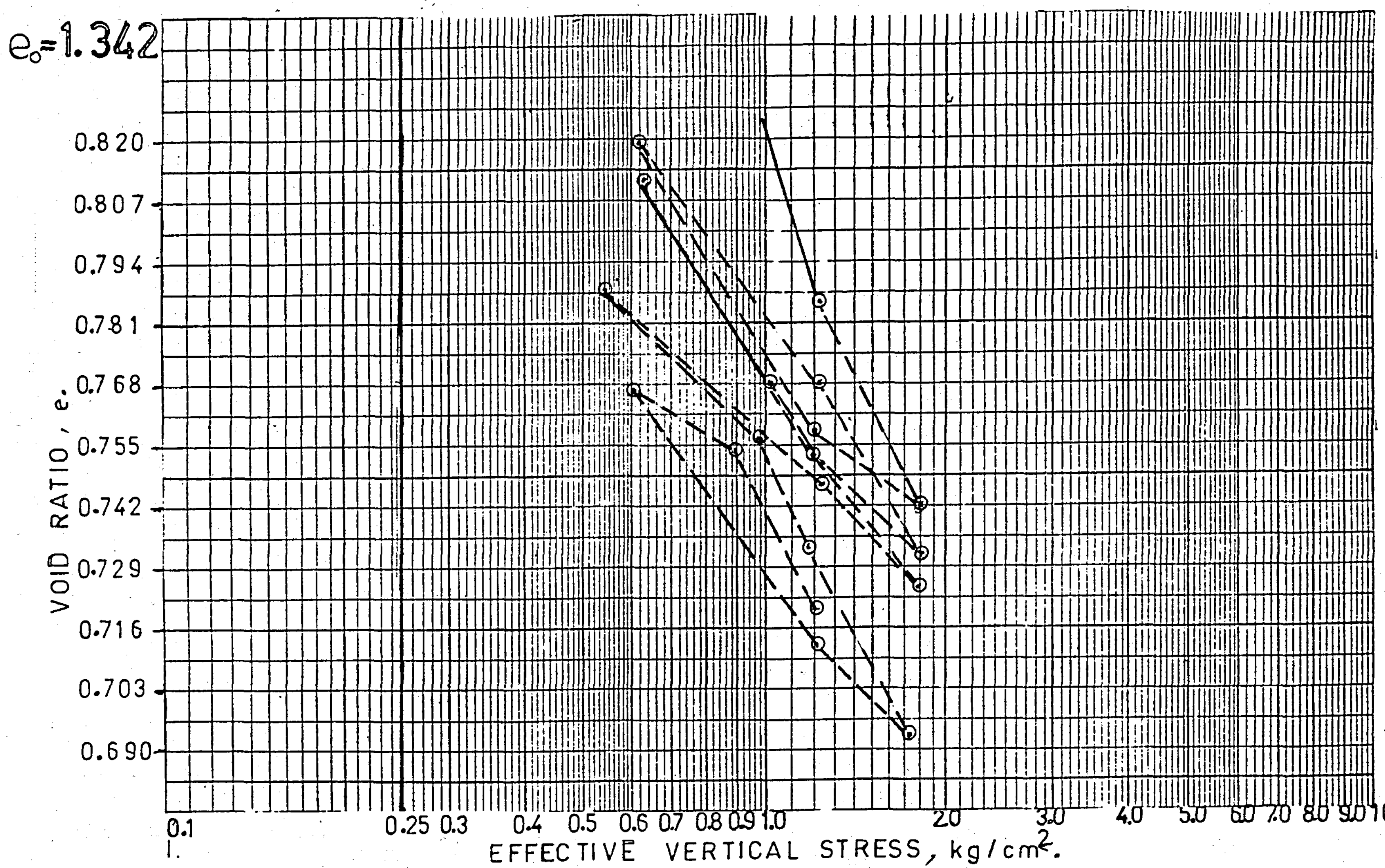


FIG-4.45 Void Ratio versus Effective Vertical Stress.

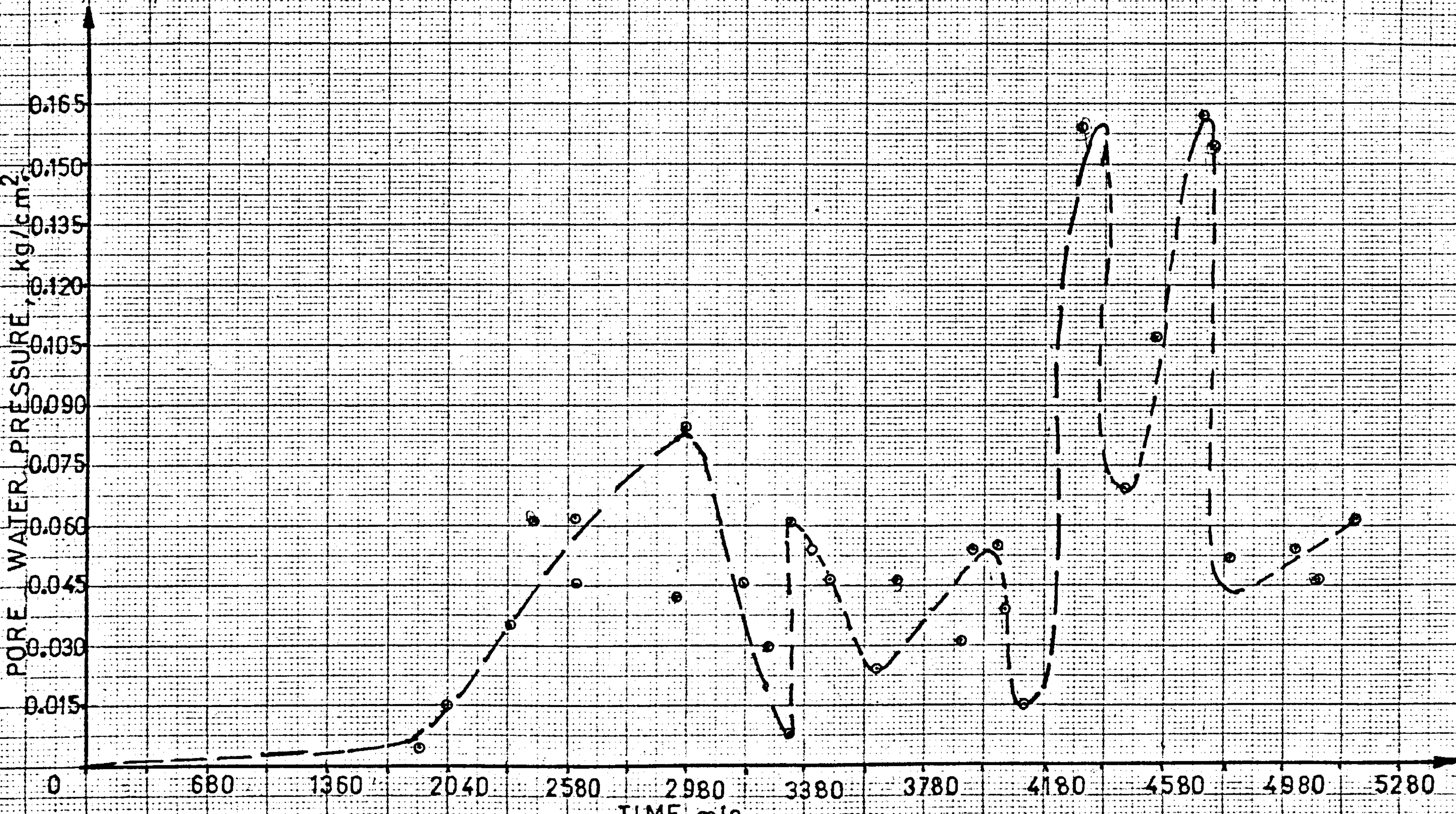


FIG-4.46 Pore Water Pressure versus Time.

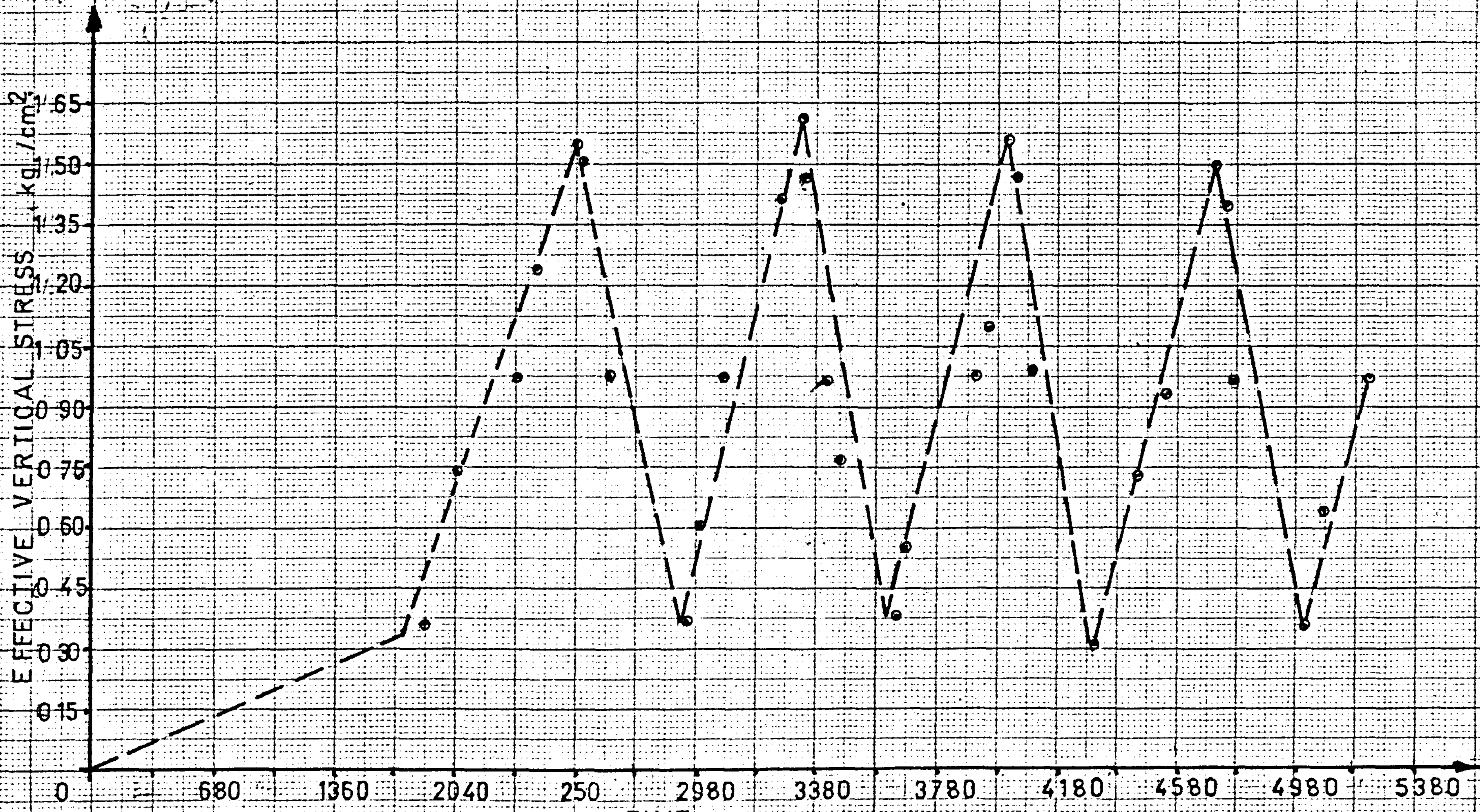


FIG-4.47 Effective Vertical Stress versus Time

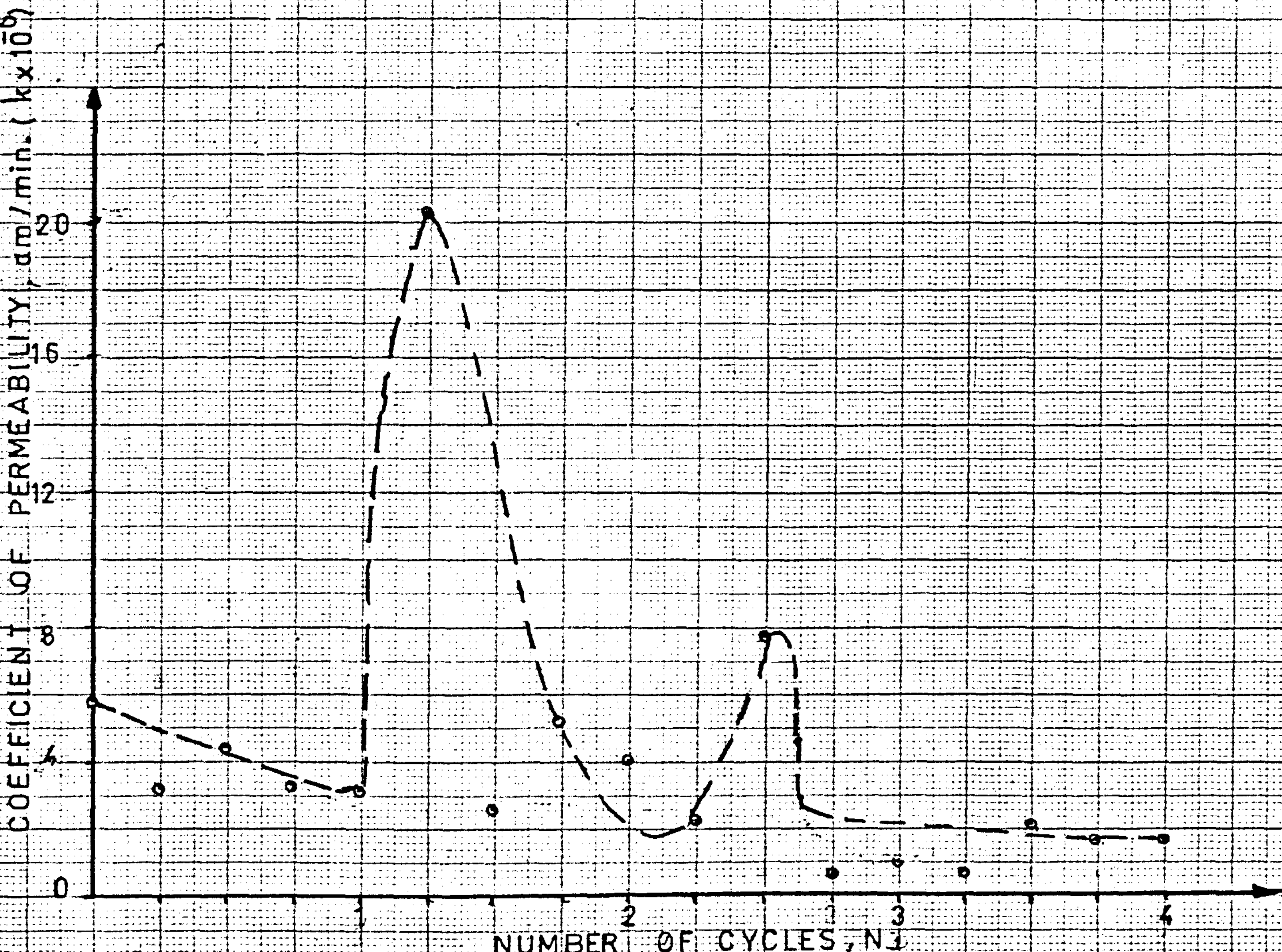


FIG_4.48 Coefficient of Permeability versus Number of Cycles

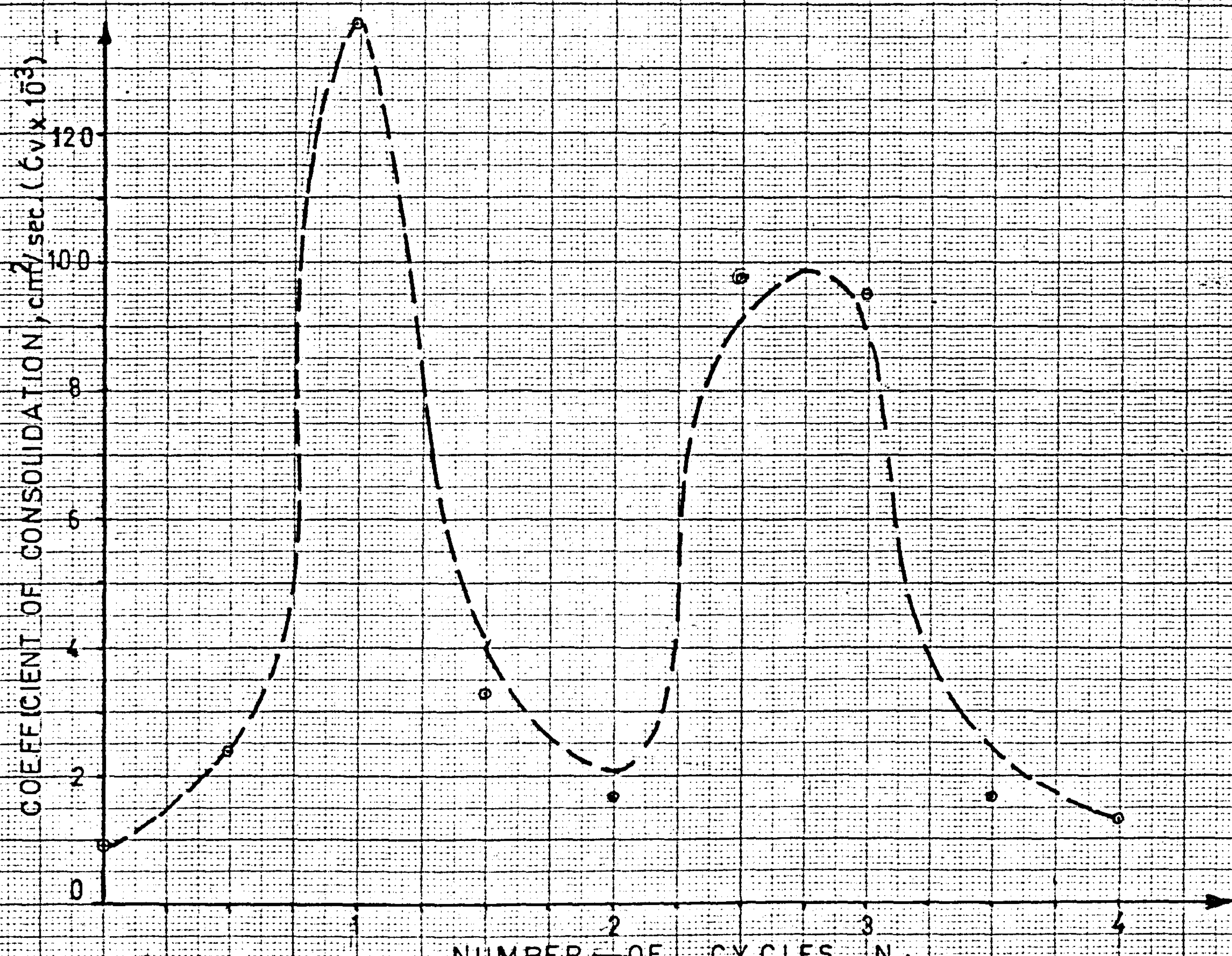


FIG. 4.49 Coefficient of Consolidation versus Number of Cycles

TEST 9

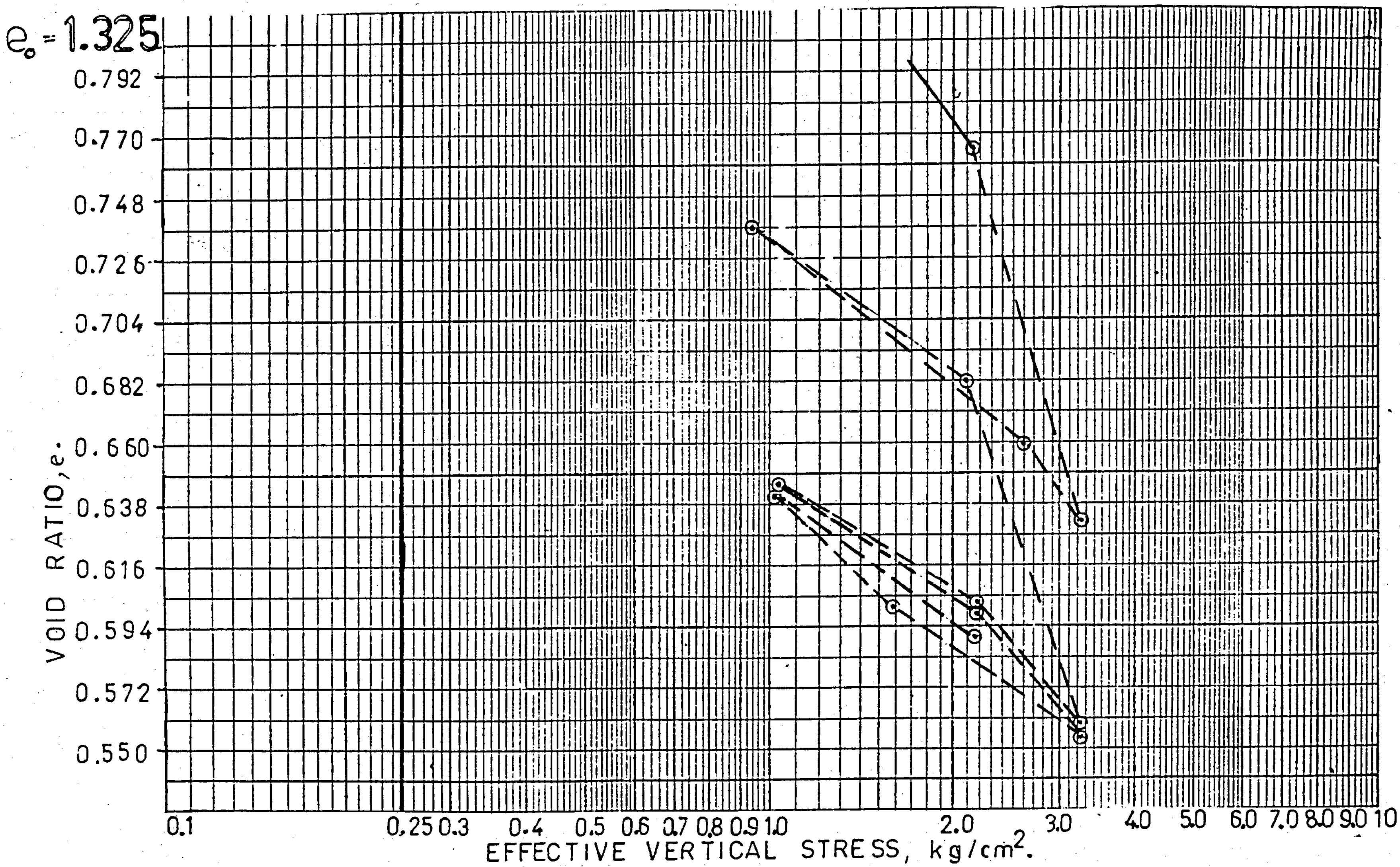
$w_i = 45$ %

$w_f = 43$ %

Sustained Load, $\sigma_o = 2.0$ kg/cm^2

Load Intensity, $\frac{\sigma_o}{\sigma_d} = 60$ %

Number of Cycles, $N = 3$



FIG–4.50 Void Ratio versus Effective Vertical Stress.

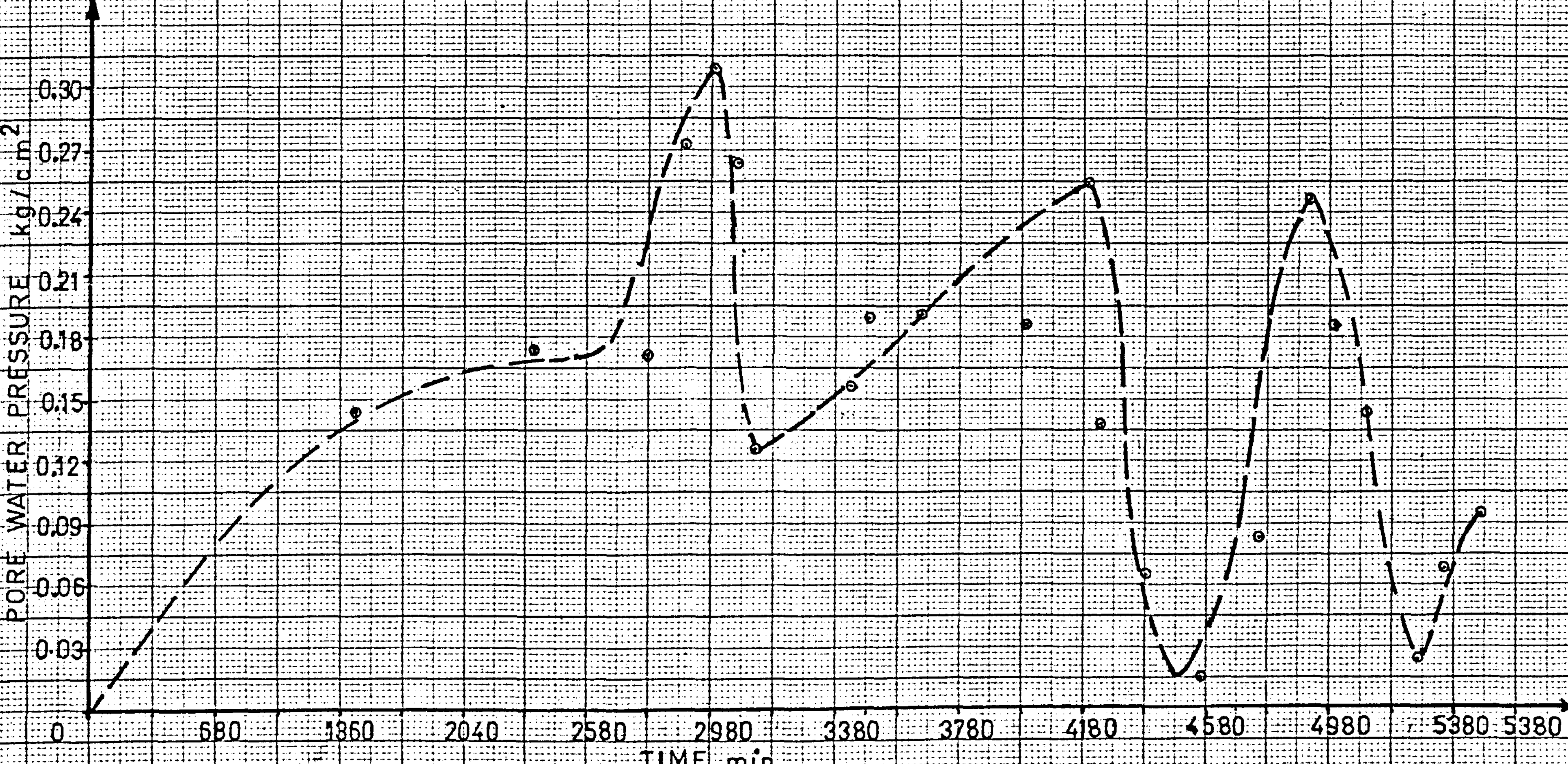


FIG-4.51 Pore Water Pressure versus Time

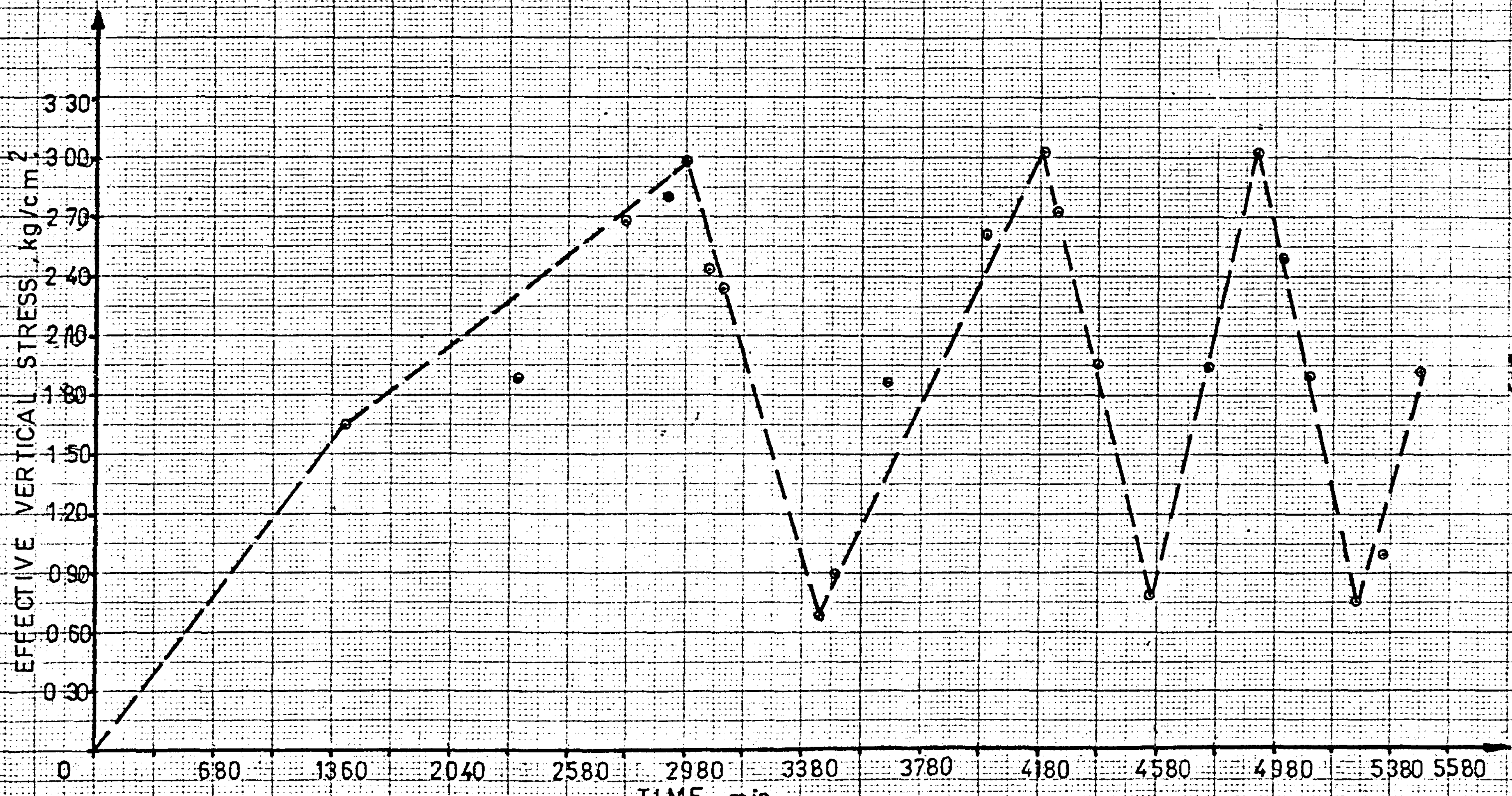
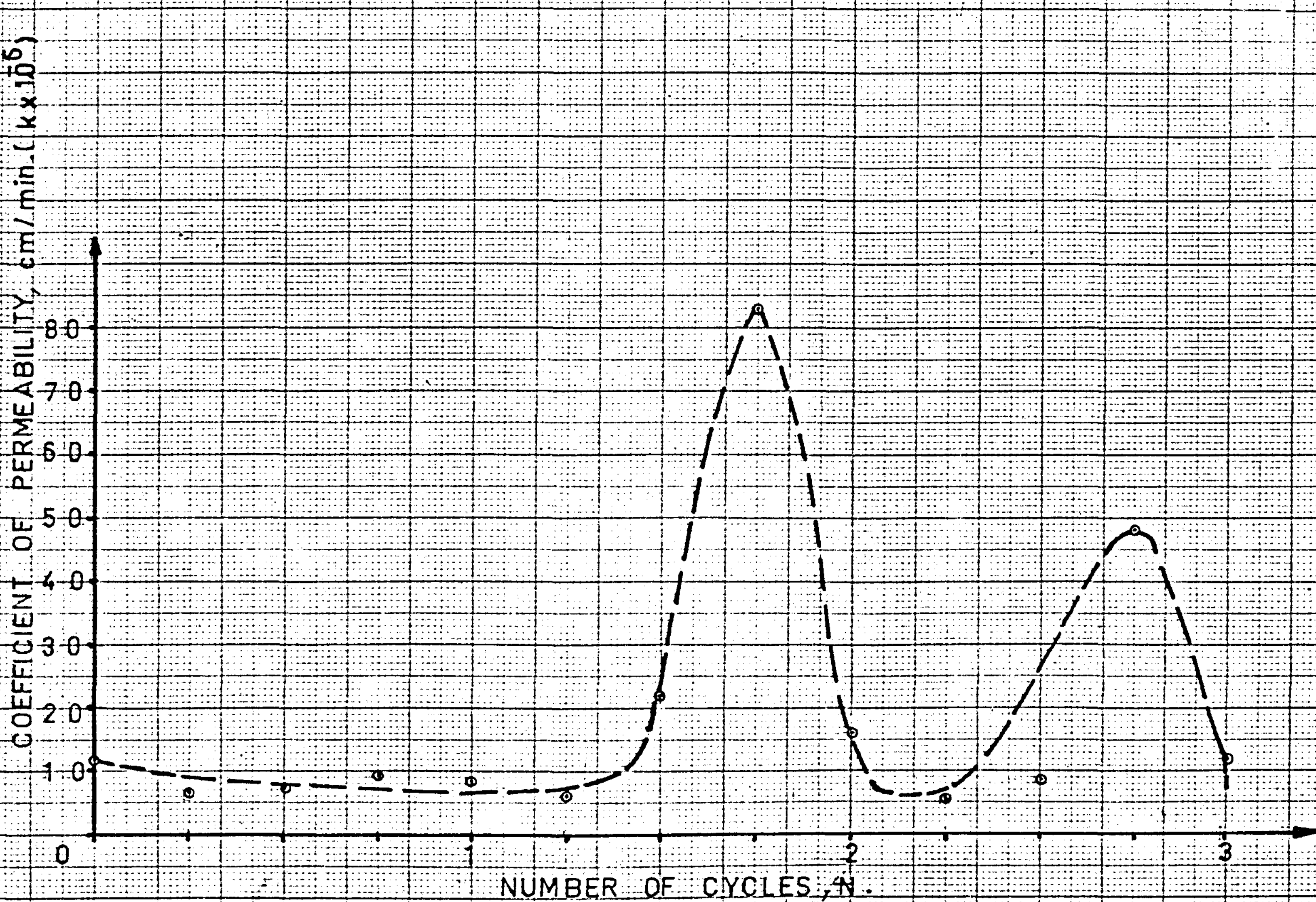


FIG.-4.52 Effective Vertical Stress versus Time.



FIG—4.53 Coefficient of Permeability versus Number of Cycles.

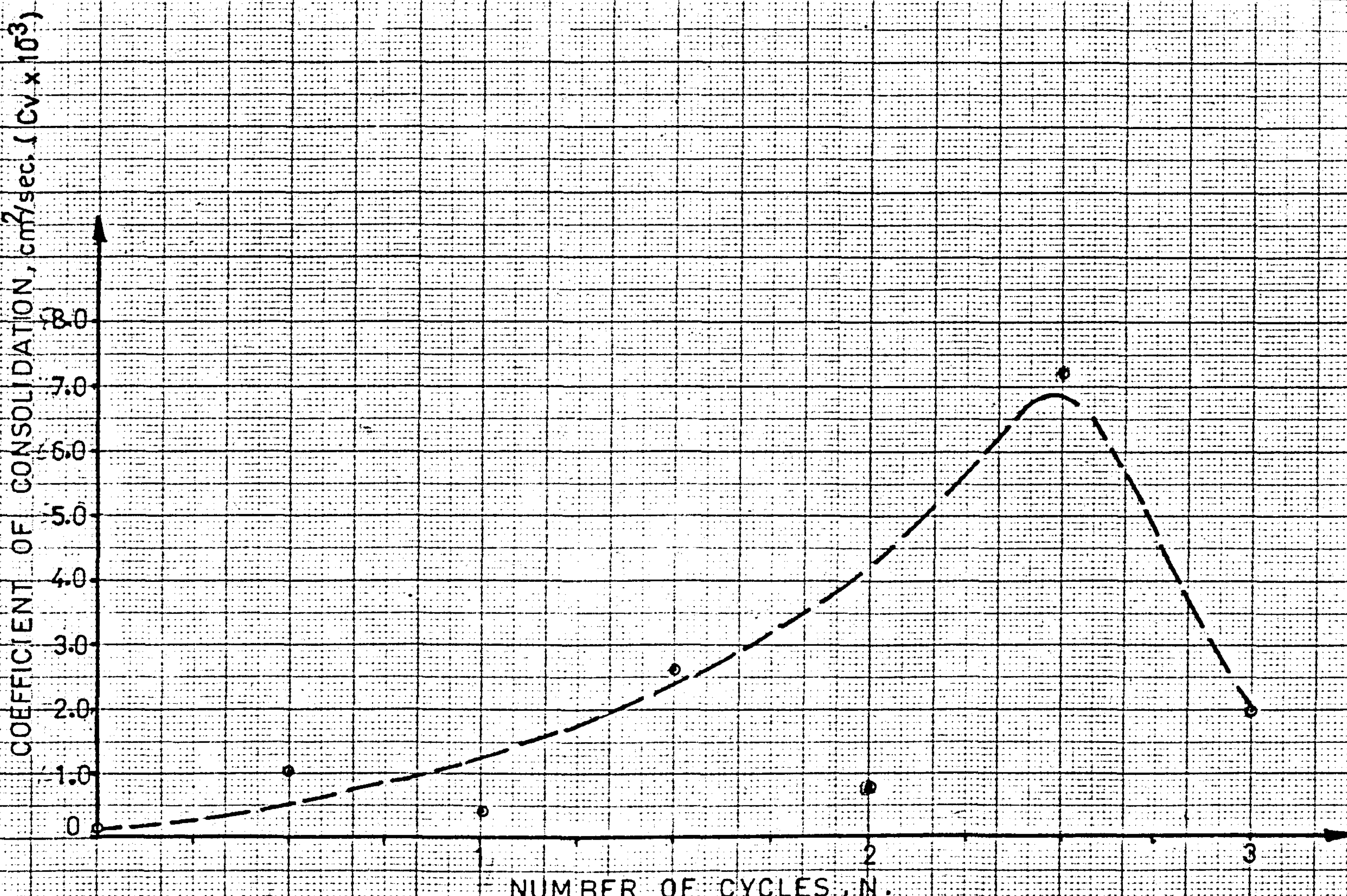


FIG_4.54 Coefficient of Consolidation of Number of Cycles.

CHAPTER 5 CONCLUSSION

This investigation demonstrates that consolidation with constant rate of strain repeated loading is to predict the behaviour of clay under oil tanks, silos, etc. Though chosen rate of strain was very small. The time of testing has been reduced considerably when compared with standard oedometer tests. Pore water pressure distributions with respect to time throughout the tst is very interesting. In the case of reloading, pore water pressure initially increases and after a period of time, it suddenly begins to decrease. In the case of unloading, inversely the pore water pressure initially decreases and after a period of time, it suddenly begins to increase. Due to above different manner of pore water pressure, the values of coefficient of permeability are not chancing linearly as expected. Another reason of the nonlinearity is the effect of swelling in the case of unloading. Therefore great diferences in the values of k_{ult} and k_{max} occur specially in the second or third cycle.

Conclusions obtianed with respect to chosen variables are as follows:

(1) The maximum value of Δed (the difference between the values of void ratio at the begining and end of dynamic loads) is obtained under the application of repeated loading when sustained load is equal to 1 kg/cm^2.

(2) The values of Δed also increase with increasing

intensity of repeated loading.

(3) Final void ratio after repeated load application is less thanthe void ratio obtained from the static test.

(4) The values of k_{max} are approximately 10 times greater than the values of k_{ult} .

(5) The values of k_{max} and k_{ult} decrease with increasing values of sustained load.

(6) The minimum values of k_{ult} and k_{max} are obtained load intensity under the application of 40 % .

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