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PREMANUFACTURED BAND SHAPED DRAINS

An Analysis and Evaluation of Effective Drain
Diameters of Band Shaped Drains. Results of a
Full Scale Laboratory Testing Programme.

by

Normand G. Castonguay, P. Eng.,

Submitted in partial fulfillment of the
requirements for the degree of
Master of Applied Science
under the supervision of
Dr. Bengt H. Fellenius
Professor

Department of Civil Engineering
Faculty of Science and Engineering
University of Ottawa

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a Jill,
et mes parents

SUMMARY

This report presents the results from a full scale laboratory testing programme conducted on premanufactured band shaped drains of three different geometrical designs.

The objectives of this study are to measure the efficiency of three drains as to their ability to drain a soil, and to determine the equivalent circle diameter and the equivalent sand drain diameter of each drain.

The usually applied theory for the design of drain projects builds on the theory developed by Barron (1948) and Kjellman (1948), which says that the time for reaching a certain degree of consolidation, i.e., drainage or pore pressure dissipation, is a function of among other factors the equivalent diameter of the bandshaped drain (Hansbo, 1979). The theory makes use of radial and horizontal flow toward a small circular drain in the centre of a cylinder of soil and build on the general consolidation theories. However, what assumption of the equivalent diameter to use for the bandshaped drain has not been clearly determined to date.

To achieve the above objectives, two full-scale test series accelerating the consolidation of soil by the aid of a bandshaped drain were performed in the laboratory. Test

Series One included a silty clay soil, and Test Series Two included a mixture of silica sand and bentonite. The soils for both test series were mixed with water to achieve isotropic soil conditions and placed in a one metre diameter steel cylinder to a height of one metre.

The instruments used to monitor pore pressures and settlements consisted of hydraulic pore pressure gauges and settlement gauges fabricated in the laboratory for the purpose of this study. Results of this study indicate that the performance of these gauges over an extended period of time are more reliable than electronic transducers. All readings were recorded with a precision of 2 mm. An accuracy of 20 mm of water column height was obtained with the pore pressure gauges, and 10 mm with the settlement gauges.

Laboratory tests performed on soil samples obtained from the soil mixtures prior to, and at the completion of the tests, indicated that isotropic soil conditions were obtained and that the coefficients of consolidation due to vertical and horizontal drainage were about the same.

Results of the analysis performed in the comparative study for Test Series One, confirmed that premanufactured band shaped drains accelerate the consolidation process. This analysis also indicated that a 100 mm wide studded drain (Drain A) is more efficient than a grooved drain of the same total width (Drain G) and that a half width Drain A (Drain M)

is about as efficient as the grooved drain (Drain G). Computing the efficiency in terms of time to complete the consolidation as compared to Drain A, it was found that Drain M and G are 85.9 and 91.8 per cent efficient, respectively.

The results of an effective drain diameter analysis performed in Test Series One, indicate that the effective drain diameter, d , is a function of both the drain geometry and its free or open surface. The equivalent circle drain diameters obtained from this analysis were 68 mm, 48 mm, and 36 mm for Drains A, G, and M respectively. From these equivalent circle diameters, the equivalent sand drain diameter for sand having a porosity, n equal to 0.4, for the sand, was found to be 169 mm, 119 mm, and 90 mm for Drains A, G and M, respectively.

Results of Test Series Two, indicated that arching was present in the sand-bentonite mixture and that drainage along the walls and the base of the test cylinders had occurred. Consequently, the results from the test cylinders varied from one another making it impossible to make any comparison and draw conclusions as to the drains efficiency and effective circle or sand drain diameter in this test.

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INTRODUCTION.

1.1 General

Fine-grained soils - clays and silts - cover most of the urban and agricultural areas of the world. The factors making these soils attractive for habitation are their ability to retain water, their usually horizontal or gently sloping surface, and the ease of excavation - to mention a few self evident factors. However, the very same factors also cause significant difficulties. Sometimes, too much water is retained soaking the ground and the excess water must be drained out before the area can be used, and water pressures deep in the soil will take a long time to dissipate. Often before the drainage and dissipation has been completed, the soil is weak and cannot be travelled on by equipment and is unstable both with regard to excavated and natural slopes. The drainage causes settlements, which create difficulties for buildings, roads, and other structures founded on the soils. In fact, the single most important factor in man's use of these areas is the drainage problem.

In the past, horizontal drainage was achieved by means of ditches. Often, these ditches were filled with free draining soils, such as sand and gravel, and covered. House foundations are still drained this way. Today, for

agricultural drainage and to replace some of the trench drains around foundations, it is common to use slotted plastic tubes, which can collect and lead off the water in the ground. Bandshaped drains (see below) are also used for this purpose.

Starting some 60 years ago, the draining of water from deep below the ground surface was achieved through vertical sand drains installed by means of punching or drilling a hole in the ground and filling it up with free draining sand (Porter, 1936). The sand drains could greatly accelerate the settlements in a local area and allow a road or a building to be finished much sooner than otherwise would have been possible. However, they were expensive and their installation sometimes so disturbed the soil that the final results were no improvement over the original conditions (Casagrande and Poulos, 1969). The use of vertical sand drains, therefore, came in disrepute in Canadian sensitive soils and were only rarely used in Canadian practice.

About 30 years ago, premanufactured bandshaped drains were developed that eliminated much of the disadvantages associated with the sanddrains (Kjellman, 1948). The bandshaped drain consists in principle of a filter surrounding a core that is studded or grooved. The filter gives free passage to water, but retains the soil particles. Once in the core, the water can travel freely in the channels created between the studs or grooves.

The first bandshaped drain - the Kjellman wick - was made of arsenic impregnated cardboard and had the dimensions 100 mm by 3 mm. Due to its low capacity and obvious environmental hazards, the Kjellman wick was abandoned. It was not until the late sixties that the modern bandshaped drain was invented consisting of a plastic core and a separate filter. At first, the filter was made of paper. Soon, however, the paper was discarded for synthetic material being more pervious, stronger, and durable.

The premanufactured drains very quickly became accepted as replacement for vertical sand drains in Europe and Asia. North American practice was a bit late in catching on. Until about 1980, the sand drain was still the most common vertical drain used in the USA. The Canadian practice, not having developed the use of sand drains in the first place, still regards the bandshaped drain as a novelty.

Today, there are several competing bandshaped drains in the international market. Most of them are based on the original Kjellman concept of a 100 mm wide strip with an outside filter and an inside core.

1.2 Statement of the Problem

The usually applied theory for the design of drain projects builds on a theory developed by Barron (1948) and Kjellman (1948), which says that the time for reaching a certain

degree of consolidation, i.e., drainage or pore pressure dissipation, is a function of among other factors the equivalent diameter of the bandshaped drain (Hansbo, 1979). The theory makes use of radial and horizontal flow toward a small circular drain in the centre of a cylinder of soil and build on the general consolidation theories. However, it is not clear how to determine the value of the equivalent diameter.

In engineering practice, there is often no distinction made between the equivalent sand drain diameter of a bandshaped drain and the equivalent circle diameter. Both of these equivalent diameters of a bandshaped drain are determined by a variety of approaches - for instance, as the average of the width and thickness of the drain, or as the diameter of a circle having the same circumferential surface as the bandshaped drain. The equivalent sand drain diameter is also determined as the diameter of a sanddrain having the same open or unobstructed circumferential surface area as the open surface area of the bandshaped drain (Fellenius, 1981).

The literature does not contain any reference to factual studies of the equivalent drain diameter of the bandshaped drain. This means that there is a lack of reliable input for use in the design of projects on bandshaped drains in general, because the different approaches give different values of equivalent diameter for commercially available drains. For instance, when using the approach of total

surface, all bandshaped drains have the same equivalent drain diameter (for the 100 mm width), while using the approach of the equal open surface, the equivalent diameter can vary by as much as a factor of two between drains.

Much work has been carried out in studying the filter characteristics of vertical drains and the flow characteristics inside the core of the drains (Hansbo, 1983, Vreeken et al., 1983, Jansen and Den Hoedt, 1983). However, to make full use of such information and to arrive at comparative figures for different drains, it is necessary to determine what approach to use for the equivalent diameter of the bandshaped drain and how to determine the efficiency of the drain.

1.3 Objectives of the Investigation

The objectives of this study are to measure the efficiency of three premanufactured band shaped drains, having different geometric designs, as to their ability to drain a soil, and to determine the equivalent circle and sand drain diameters of each drain.

1.4 Scope of the Investigation

An accurate experimental investigation into the understanding of the effective drain diameter of bandshaped drains should represent in-situ conditions and be able to eliminate many of

the factors which may obscure the results of the investigation. Such factors are discussed in the following chapter.

The completed investigation reported herein has been carried out at the University of Ottawa facilities and measures the efficiency of three bandshaped drains in terms of their ability to drain a soil. More precisely expressed, the efficiency is determined in terms of achieved degree of consolidation (pore pressure dissipation and settlement).

Three types of bandshaped drains have been investigated. The Alidrain (Drain A), a drain with a ratio of free surface over closed surface of about 10; the Mebra drain, a grooved drain (Drain G), a drain with a free surface ratio of about 1; and a half width Alidrain (Drain M) included as a special drain with a gross free surface equal to that of the Mebra drain and a free surface ratio equal to that of the Alidrain (Drain A).

The investigation consisted of two test series. In the first, natural Champlain clay obtained from a local site was used. This was remoulded with addition of water to make it more compressible. The soil used in the second series was a coarser soil, more similar to those found in deltaic areas around the world and made up of a mixture of silica sand, bentonite and water.

The tests were carried out in the laboratory, in full scale, placing the soil in large cylindrical containers and subjecting it to a uniform surcharge, which induced excess pore pressures in the soil matrix. The dissipation of the pore pressures and the compression (volume loss) resulting from the drainage of the water, were measured by means of specially developed and manufactured piezometers and settlement gauges.

In each test series, four containers were used, three of which had one of the drains placed in its centre. The fourth container served the purpose of reference to vertical drainage, only, to enable the results of the other three tests to be adjusted to consider the effect of horizontal drainage, only.

CHAPTER - 2 -

THEORY OF CONSOLIDATION BY VERTICAL DRAINS

2.1 Introduction

Theoretical design procedures were not available when sand drains were first used and the early installations were designed on an empirical basis. Theoretical design procedures based upon Terzaghi's theory of consolidation of compressible soils were developed by R. A. Barron during 1940-42. Prior to Barron's work, Rendulic (1935) formulated and solved the differential equation for consolidation by radial flow to a drain well. Carrillo (1942) worked on this same problem at about the same time as Barron. Barron's work, which was the most extensive, was done independently of Rendulic and Carrillo and was presented in the mid and late 40-ies (Barron 1944, 1948). The latter reference constitutes the most complete source available on the theory of vertical drains. The first generally available design information was presented by Terzaghi (1943). Terzaghi (1945) reviewed the Barron design procedures and presented revised design curves. Basically the same design procedures and design curves are used today, and they can be found in most text books dealing with soil mechanics (eg., Holtz and Kovacs, 1981).

2.2 Basic Theory of Consolidation

Consolidation analysis is concerned with the prediction of the time rate of volume change of a saturated compressible soil resulting from the application of an external load or from the dissipation of hydrostatic excess pressures. The compression of soil under load is obtained by conventional analysis of the performance of soil samples in laboratory consolidation tests. The parameters required for determination of the time rate of consolidation are normally obtained from the same laboratory tests. The general theory considers that drainage of pore water may occur in any direction. The problems of exclusively vertical drainage in the absence of vertical drains utilizing horizontal radial flow are special cases of the general theory of one-dimensional consolidation of soils, as developed by Terzaghi (1943). The general theory is also the base of the analysis of consolidation by means of vertical drains.

The following assumptions have been made in deriving the general theory of consolidation:

1. The voids in a soil are completely filled with water, which is an incompressible fluid.
2. The solid components of the soil are incompressible.
3. Darcy's law is valid.
4. The coefficient of permeability, k , is a constant.
5. The time lag of consolidation is governed entirely by the permeability of the soil.

Additional assumptions are that the clay is laterally confined, and an increase in the effective pressure from an initial value p'_0 to a final value p'_1 reduces the void ratio of the clay from an initial value e_0 to a final value e_1 ; the resulting ratio

$$a_v = \frac{e_0 - e_1}{p'_1 - p'_0} \quad (2.1)$$

is assumed to be a constant over the range of pressure p'_0 to p'_1 . The term a_v is called the "coefficient of compressibility".

For a decrease of the volume of voids, void ratio reduces from e_0 to e_1 in a block of soil of original volume $1 + e_0 = H_0$ with the volume of solids equal to unity. The decrease of the volume of voids per unit of initial volume is also expressed as vertical strain, \mathcal{E}_v ;

$$\mathcal{E}_v = \frac{H}{H_0} = \frac{e_0 - e_1}{1 + e_0} = \frac{a_v}{1 + e_0} (p'_1 - p'_0) = m_v (p'_1 - p'_0) \quad (2.2)$$

the value
$$m_v = \frac{\mathcal{E}_v}{p'_1 - p'_0} = \frac{a_v}{1 + e_0} \quad (2.3)$$

is called the "coefficient of volume decrease".

Considering an element of soil having its top at a distance z below some reference elevation and an area A and a height dz . As the pore water dissipates from the voids of the element during a one-dimensional vertical consolidation, the rate of

flow, Q , across its upper surface, A , is expressed by Darcy's law as

$$Q = kiA \quad (2.4)$$

where, k , is Darcy's coefficient of permeability, and i , is the hydraulic gradient also expressed as

$$i = -\frac{1}{\gamma_w} \frac{\partial u}{\partial z} \quad (2.5)$$

The difference between the flow through the top and bottom surfaces of the element determines the rate of loss of water from the element which amounts to

$$\Delta Q = -\frac{k}{\gamma_w} \frac{\partial^2 u}{\partial z^2} A dz \quad (2.6)$$

where γ_w is the unit weight of the water and u the excess water pressure.

Since it was assumed that the soil was completely saturated, and that both water and the solid soil particles are incompressible, the change in rate of flow ΔQ must be equal to $\partial V/\partial t$, i.e. The rate of volume change of the soil element is

$$\frac{\partial V}{\partial t} = \frac{\partial}{\partial t} \left(\frac{e}{1+e} A dz \right) = -\frac{A dz}{1+e} \frac{\partial e}{\partial t} \quad (2.7)$$

The rate of volume change (eq. 2.6 = eq. 2.7) becomes

$$\frac{k}{\gamma_w} \frac{\partial^2 u}{\partial z^2} = \frac{1}{1+e} \frac{\partial e}{\partial t} \quad (2.8)$$

Equation 2.8 is the foundation for all mathematical one-dimensional analyses of consolidation.

Considering the volume of the voids of a unit element, e , as a variable, the equation 2.8 can be written as

$$\frac{\partial u}{\partial t} = \frac{k}{a_v \gamma_w} \frac{1}{1+e} \frac{\partial^2 u}{\partial s_1^2} - \frac{k}{\gamma_w (1+e)^2} \left(\frac{\partial u}{\partial s_1} \right)^2 \quad (2.9)$$

where s_1 is taken as the independent variable instead of z where, for an element of soil dz , the corresponding height of the volume of solids is ds . This allows the use of limits of integration between $s_1 = 0$ and $s_1 = H_s$, which are fixed values, as compared to the limits of $z = 0$ and $z = H - f(\tau)$, which is a function of time.

The following simplifying assumptions are now made:

1. The variation in e will be limited to small values so that $(1 + e)$ can be considered a constant.
2. Since $\left(\frac{\partial u}{\partial s_1} \right)^2$ is small, the second term in eq. 2.9 is usually insignificant with respect to the first term and it is ignored.
3. For practical purposes, $\frac{k}{a_v \gamma_w (1+e)}$ is taken as a constant, which is approximately true for inorganic clays over a small range of e .

Introducing these simplifications into eq. 2.9, the resulting expression is

$$\frac{\partial u}{\partial t} = c \frac{\partial^2 u}{\partial s_1^2} \quad (2.10)$$

since $e = \text{constant}$

$$s_1 = \text{constant times } z$$

The soil properties k , a_v , and γ_w in combination with $1 + e$ form a combined property called the "coefficient of consolidation"

$$c_v = \frac{k(1+e)}{a_v \gamma_w} \quad (2.11)$$

Substituting this combined property in Eq. (2.10) gives

$$c_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} \quad (2.12)$$

which is the customary form for the differential equation of consolidation as originally presented by Terzaghi. Although the individual terms in eq. 2.12 vary with outside pressure, c_v is nearly constant over the usual loading increments.

Similarly, the equations for consolidation by two dimensional and three-dimensional flow expressed in cartesian and cylindrical coordinates are, respectively;

$$\frac{\partial u}{\partial t} = c_v \left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial z^2} \right) \quad (2.13)$$

$$\frac{\partial u}{\partial t} = c_v \left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} + \frac{\partial^2 u}{\partial z^2} \right) \quad (2.14)$$

$$\frac{\partial u}{\partial t} = c_v \left(\frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} + \frac{\partial^2 u}{\partial z^2} \right) \quad (2.15)$$

If the coefficients due to vertical (c_v) and horizontal flow (c_h) differ, the equations must be rewritten and, expressed in cylindrical coordinates, the consolidation equation becomes

$$\frac{\partial u}{\partial t} = c_h \left(\frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right) + c_v \frac{\partial^2 u}{\partial z^2} \quad (2.16)$$

For the cases of two and three-dimensional flow, the assumptions are also made that the load distribution on the soil layer is unaffected by the consolidation process, and that change in strain does not affect change in stress.

These theories of primary consolidation do not include a treatment of the change in soil properties due to radial flow of water. Changes in soil properties in the horizontal direction may be of considerable importance in practical situations.

Consolidation by vertical flow alone involves only two variables, time and depth z . For consolidation by two-dimensional flow, the variables are x , z , and time, while for three-dimensional flow they are x , y , and z , and time. The

general solution for consolidation by two and three-dimensional flow, for a given set of boundary conditions may be obtained by the method of separation of variables as demonstrated by Carrillo (1942), whose relationship regarding the average values of pore water pressure which can be used for homogeneous soils is

$$\bar{U} = 1 - (1 - \bar{U}_h)(1 - \bar{U}_v) \quad (2.17)$$

2.3 Vertical Consolidation Due to Radial Flow

Barron (1944, 1948) presented the principal analytical studies of radial flow toward a drain well and the resulting consolidation of the clay. Barron considered two types of vertical strain which might occur in the clay layer; first the condition of "free vertical strain", which results from a uniform distribution of surface load; and, second, the condition of "equal vertical strains", which results from imposing the same vertical deformation at all points on the surface. For both strain conditions, Barron included an analysis of the effect of "smear" of the soil near the well boundary, and the effect of resistance to flow through the well itself.

2.3.1 Free Strain Consolidation

As indicated on Figure 2.1, for a triangular spacing of drain wells, a zone of influence exists which has a hexagonal

plan form. The hexagon represents a circle of equivalent diameter, d_e , which determines the outer limit of the zone of influence of each drain well. Thus, it becomes sufficient to consider the radial flow and resulting consolidation of a volume of soil of unit thickness which is contained between the distances d_e (diameter of well influence) and d_w (effective diameter of the drain well).

By eliminating the consideration of vertical flow, eq. (2.15) becomes

$$\frac{\partial u}{\partial t} = c_h \left(\frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right) \quad (2.18)$$

which is the equation for consolidation expressed in terms of radial coordinates. The boundary conditions which must be satisfied are:

- a) The initial pore water pressure, u_0 , is uniform throughout the soil mass when $t = 0$.
- b) The excess pore water pressure at the drain well surface r_w is zero when $t > 0$.
- c) The external radius, r_e , is considered impervious because of symmetry. Thus, $\frac{\partial u}{\partial r} = 0$ when $r = r_e$.

The solution of eq. 2.18, is subject to the boundary conditions indicated above and is quite involved and has been evaluated by Barron for a wide range of r_e/r_w ratios (n - values) and this information is reproduced in Figure 2.2.

2.3.2 Equal Strain Consolidation

In case of radial flow only, as is caused by the equal vertical strains throughout a horizontal ring of soil of thickness dz and bounded by $r_1 = r_e$ and $r_2 = r$, the change in volume of the ring due to a uniform change in thickness is

$$\Delta v = \Delta n * v = m_v (p_1 - p_0) * (r_e^2 - r^2) dz \quad (2.19)$$

after utilizing the expression for unit volume change, Eq. (2.2). Due to the volume change by equal vertical strains, the increase in effective pressure $\Delta p'$, is uniform throughout the ring. This is related to the increase in pressure on the ring, p , by

$$p' = p - u \quad (2.20)$$

where u is the average value of excess pore water pressure. Since the amount of water squeezed out of the ring of soil is equal to the change of volume, the flow of water out of the ring is given by the time rate of volume change or

$$Q = - m_v \pi (r_e^2 - r^2) dz \frac{\partial u}{\partial t} \quad (2.21)$$

The change of flow which occurs due to an increase in the size of the ring is

$$\frac{\partial Q}{\partial r} = m_v \pi * 2r dz \frac{\partial u}{\partial t} \quad (2.22)$$

From Darcy's law, the flow of water through the inner boundary of the ring is

$$Q = \frac{k}{\gamma_w} * 2 r * dr \frac{\partial u}{\partial r} \quad (2.23)$$

and the change of this flow in the radial direction is

$$\frac{\partial Q}{\partial r} = \frac{k}{\gamma_w} * 2\pi dz \left(\frac{\partial u}{\partial r} + r \frac{\partial^2 u}{\partial r^2} \right) \quad (2.24)$$

Combining Eqs.(2.22) and (2.24) gives

$$\frac{\partial u}{\partial t} = c_h \left(\frac{1}{r} \frac{\partial u}{\partial r} + \frac{\partial^2 u}{\partial r^2} \right) \quad (2.25)$$

Barron (1948) obtained the following solution of eq. 2.25

$$U_r = \frac{4u}{d_e^2 F(n)} \left[r_e^2 \ln\left(\frac{r}{r_w}\right) - \frac{r^2 - r_w^2}{2} \right] \quad (2.26)$$

in which: $\bar{u} = \bar{u}_o \ln^{-1}(\lambda)$ (2.27)

$$\lambda = \frac{-8 T_h}{F(n)} \quad (2.28)$$

and $F(n) = \frac{n^2}{n^2-1} \ln(n) - \frac{3n^2-1}{4n^2}$ (2.29)

The initial distribution of hydrostatic excess pressure is not uniform, but may be computed from Eq. 2.26 for $T_h = \lambda = 0$. Figure 2.3 shows the distribution of u_r/u_o along the radius of soil cylinders having values of $n (r_e/r_w)$ of 5, 10, 50 and 100.

2.3.3 Comparison of Free Strain and Equal Strain Consolidation

Curves showing the relation between the average pore water pressure, u , and the time factor, T_h , can also be obtained from Eq.(2.27). Such curves for $n = 5, 10, 20, 40$ and 100 , are shown on Figure 2.2 along with the corresponding curves determined by the "free strain" case.

The difference between the results obtained by the two rather extreme considerations, Free Strain and Equal Strain, is small particularly for the curves representing values of n greater than about 10. For $n = 5$, the discrepancy is about 4 percent for the first part of consolidation, but above about 50 percent consolidation, the curves are almost identical. For applications in practice, both Barron (1948) and Kjellman (1948) recommended the use of the equal strain solution. Therefore, the Barron solution to determine the time of consolidation is given by

$$t = \frac{D^2}{8 * c_h} \left[\frac{n^2}{n^2-1} \ln(n) - \frac{3}{4} + \frac{1}{4n^2} \right] \ln\left(\frac{1}{1-\bar{U}}\right) \quad (2.30)$$

Kjellman (1948), who was working with vertical drains at about the same time as Barron, obtained the same solution as Barron for equal-vertical strain, and simplified the equation (2.30) to yield the Kjellman - Barron solution

$$t = \frac{D^2}{8 c_h} \left[\frac{\ln(D/d)}{1-(d/D)^2} - \frac{3-(d/D)^2}{4} \right] \ln\left(\frac{1}{1-\bar{U}}\right) \quad (2.31)$$

which was simplified further for $D/d > 6$ to yield the Kjellman solution (Hansbo, 1979)

$$t = \frac{D^2}{8 c_h} \left[\ln(D/d) - \frac{3}{4} \right] \ln\left(\frac{1}{1-\bar{U}}\right) \quad (2.32)$$

Eq. 2.32 is commonly used in practice today for the design of premanufactured vertical drains.

Expressing equation 2.32 in terms of degree of consolidation (U_h) due to pore water flow in the horizontal direction, one obtains the following relationship

$$U_h = 1 - \exp \left[- \frac{8 c_h t}{D^2 (\ln(D/d) - 0.75)} \right] \quad (2.33)$$

or

$$U_h = 1 - \exp \left[- \frac{8 T_h}{F(n)} \right] \quad (2.34)$$

where:

$$T_h = \frac{c_h t}{D^2}$$

$$F_n = \frac{n^2}{n^2 - 1} [\ln(n) - 0.75]$$

$$n = D/d$$

2.3.4 Effect of Peripheral Smear

The remoulded or smeared zone at the periphery of the drain well creates an additional resistance to the flow of water toward the drain. This retards the consolidation process.

The smeared zone will not be uniform or homogeneous with regard to the soil properties. Very likely, it consists of a thin layer of actual smear plus an adjacent region in which the soil has undergone a considerable amount of remoulding or disturbance. The amount of disturbance decreases with distance away from the well periphery and depends greatly on the mode of drain installation.

The effect of smear is taken into consideration when calculating the degree of consolidation (eq. 2.32) by considering (Barron, 1948) ;

$$U_n = \frac{n^2}{n-1} \ln\left(\frac{n}{s}\right) + \frac{k_c}{k'_c} \left[\ln(s) - 0.75 \right]$$

where: $s = d_s/d$
 d_s = diameter of disturbed zone
 k_c = permeability of undisturbed zone
 k'_c = permeability of disturbed zone

2.3.5 Effect of Well Resistance

The foregoing analyses have considered that there is unrestricted flow of the water through the drain. Actually, head losses will occur due to the resistance to flow of the drain backfill material or filters in the case of sand drains or band shaped drains, respectively. The magnitude of losses will depend upon the rate of flow, the size of the drain, and the permeability of the filter or the material filling the drain.

Barron (1948), developed a solution for the case of "equal vertical strain", with and without smear, where the permeability, k , of the consolidating soil is zero. The effect of drain resistance is shown on Figure 2.4 as applied to three conditions represented by $n = 15, 30,$ and 60 . The curves for excess pore water pressure versus time are shown as solid lines for each value of n for the condition of no smear and no well resistance. These curves were derived from consideration of radial flow only and are not influenced at all by changes in the thickness of the clay layer. For drain installations in practice for which n was about 10 to 15 and for d_e/H of 1.0 or less, the effect on the consolidation behavior of a clay layer should not be significant.

In conventional design procedures, neither smear nor well resistance is considered.

2.3.6 Equivalent Drain Diameter

In the use of prefabricated bandshaped drains, the required spacing of the drains is governed by two important parameters, viz. equivalent drain diameter (d) and the consolidation coefficient for pore water flow in the horizontal direction (c_h). There still exist today, no agreement on which method to use to calculate the equivalent diameter, and the difficulties in estimating the coefficient of consolidation are well known (Rowe and Shields, 1965) making it difficult to isolate any one of these two parameters.

Laboratory determination of the magnitude of c_h may result in erroneous values because:

- A) the scatter in the results of laboratory tests is high (inhomogeneity of soil)
- B) actual in-situ conditions often differ widely from laboratory conditions
- C) the magnitude of c_h and c_v changes with changing vertical pressure
- D) different values are found for virgin compression, re-compression and remoulded soil.

In field conditions, usually c_h is found to be 2 - 5 times higher than c_v .

A number of approaches, as listed below, are used today to determine the effective diameter (d) of a premanufactured drain;

$$d_1 = 2(a+b) \frac{1}{\pi} \quad (2.36)$$

$$d_2 = \frac{a+b}{2} \quad (2.37)$$

$$d_3 = \sqrt{\frac{4}{\pi} a*b} \quad (2.38)$$

$$d_4 = \sqrt{\frac{4}{\pi} (\text{open area of } a*b)} \quad (2.39)$$

$$d_5 = 2(a'+b') \frac{1}{\pi} \quad (2.40)$$

$$d_6 = \text{from field or laboratory observations} \quad (2.41)$$

- where:
- a = thickness of the drain
 - b = width of the drain
 - 2(a+b) = circumference of the drain
 - ab = cross sectional area of the drain
 - 2(a'+b') = free or open circumference of the drain
 - a'b' = free or open cross sectional area of the drain.

The equivalent circular drain diameter used by Kjellman (1948) was 50 mm for his 100 mm wide cardboard wick. Some references use the circumferential area of the filter jacket (equ. 2.36) which yields a value of 65 mm for a 100 mm wide bandshaped drain (Hansbo, 1979). Since the flow of water to a bandshaped drain is less favorable than a cylindrical drain

with the same width, some apply a factor of $\pi/4$, (Eq. 2.37) reducing the equivalent diameter to 51 mm (Jansen and Den Hoedt, 1983). Others indicate that the free or open area of the surface of the drain has to be considered (Eq. 2.40), and make a comparison not with the equivalent circular diameter but the equivalent sand drain diameter considering the open surface also of the sand drain and giving values that are 2 to 3 times as high as the values found with the previous methods (Fellenius, 1981).

2.4 Experimental Investigation

Although premanufactured drains are widely used today, there are still unsolved problems and controversial opinions concerning the mechanical and hydraulic characteristics as well as the durability of the bandshaped drains.

What is less known of prefabricated band-shaped drains are the problems concerning their biochemical and physical degradation with time after installation. On the basis of limited laboratory and field testing, it is estimated that non-impregnated filter paper may be effective up to 12 to 16 months. When the filter is made of impregnated paper, the duration may be extended at least 24 to 30 months (Jamiolkowski et al., 1976).

The mechanical properties of the band shaped drains are easily identified. However, there exists a lack of well

defined standard procedures of measuring these properties and little is known about the stresses in the drains during installation, when subjected to large lateral soil displacement and large settlements.

The most relevant features for the design and performance of band-shaped drains are the hydraulic properties, e.g. the discharge capacity, q_w , of the drain cross-section, the filter permeability, k_f , and the effective diameter of the band shaped drain. Most research thus far has been concentrated on the discharge capacity and filter permeability (Jamiolkowski et al., 1983, Hansbo, 1983).

For practical reasons, the determination of q_w cannot be made in the field, but must rely on laboratory tests. Hansbo (1983) suggests how the ideal test should be performed, and describes a number of tests which have been carried out by various institutions, namely; the Ontario Research Foundation (the ORF test), Chalmers University of Technology (the CTH and CTH revised test), Tokyu Construction Co. Ltd. (the Tokyu Construction test), the Dutch Study Center of Road Construction (the Delft test), the Department of Transportation, State of California (the California test), Moh and Consulting Engineers, Taiwan (the Taiwan test), and the Research Center of Hydraulics and Structures of ENEL in Italy (ENEL TEST). Hansbo advocates that only the CTH test which makes use of a conventional triaxial cell can truly measure q_w under in-situ conditions. However, the CTH test

does not measure the influence on q_w of drain folding caused by large settlements. Neither of the mentioned tests truly measures the effects of draining a large volume, representative of field conditions.

Tests to measure the permeability of the filter have been discussed by various authors (Den Hoedt, 1981, and Hansbo, 1983). The purpose of permeability tests is to investigate two mutually contradictory requirements:

- * Almost all fine particles should be prevented from entering the core.
- * The water should flow through the filter without any appreciable head losses.

It has been shown that the former requirement prevails in the selection of the filter material (Jamiołkowski et al., 1983). Figure 2.5 indicates the influence of finite drain permeability on consolidation rate. This figure shows that in the case of drains, where usually $q_w/k_h > 2000 \text{ m}^2$, the impact of finite well permeability on the consolidation rate is negligible.

Laboratory tests were performed by the ENKA laboratories in the Netherlands (Jansen and Den Hoedt, 1983) to try and clarify the different theories associated with the equivalent drain diameter. Two different types of drain core were examined consisting of a closed core and an open core, both enclosed in the same filter jacket and using the ENKA drain

tester (see Fig. 2.6). Three types of soils were used as described by their grain size distribution (Fig. 2.7). Test results obtained are shown in Fig. 2.8.

It was concluded from the Dutch tests, that an "open" drain core guarantees a much better discharge than a "closed" core in cases where there is an abundant flow of water toward the drain, as shown by soil III. Also, the permeability of the soil to be drained (soils I and II) is the most dominant factor that controls the flow of water into the drain, and not the inlet resistance of the drain itself. However, this test is not representative of in-situ conditions where the diameter of the soil cylinder dewatered by a single drain is much larger and the time rate of consolidation much greater. This is evident from the results of full scale field tests performed in central Sweden (Erikson and Ekstrom, 1983), who compared three different types of drains - one sand drain and two premanufactured band-shaped drains. Results as indicated in Figures 2.9 to 2.11, indicate little difference during the initial stage of consolidation. However, the deviation increased with time and drain spacing. Unfortunately, both the method of installation and the filter jackets for the band-shaped drains differed making it impossible to isolate the effect of the core.

Erikson and Ekstrom found the values of the horizontal coefficient of consolidation evaluated from the various settlement curves to vary widely and state the following

general conclusion; "The extent to which reality agrees with theory cannot be determined. The fact that the evaluated coefficients of consolidation change with the drain spacing casts suspicion on the simplifications on which the theory is based and the assumptions, such as the equivalent diameter of the drain, that must be assumed in the calculations".

Recently, Foot and Koutsoftas (1982), report the results of a very successful drainage project involving settlement observations in two areas where a band-shaped drain (Alldrain) had been installed at two different spacings, 1.5 m and 3.0 m. The drains were installed with a mandrel system designed to minimize the installation disturbance. The mandrel outside dimensions were 140 mm by 75 mm. At the smaller spacing, the evaluated c_h value was smaller than that evaluated from the larger spacing, and the authors concluded that the drain installation had disturbed the soil more at the smaller than at the larger spacing. It is difficult to accept that the installation disturbance caused by inserting the 0.14 m wide mandrel could disturb the soil 1.5 m, 11 mandrel diameters away. On the contrary, Fellenius and Samson (1976) and Bozozuk et al. (1978) indicate that the disturbance effect in soft sensitive clay is much more local and approximately limited to a zone of about three diameters.

Unfortunately, Foot and Koutsoftas (1982) do not quantify the analysis with regard to the data and assumptions used as input. For instance, it is not clear which approach for the

equivalent diameter they used in their analysis of the data. The evaluated c_h values depend on the various assumptions made and the conclusions could be disputed since the backcalculated c_h values can vary by a factor of two or more depending on what definition of equivalent drain diameter is applied in the analysis.

Hansbo and Torstenson (1977) presented results from a full scale test on the band-shaped drains (type Geodrain) compared with sand drains and found that when using the approach of equal total circumference, the evaluated c_h value was 50 percent larger than had been found for sand drains at the same site. However, as shown by Fellenius and Wager (1977), were one to use the approach of the open surface instead of the equivalent sand drain diameter, the evaluated c_h value would have been about equal to that evaluated from the sand drain data. The difference is important because the c_h value has a linearly proportional influence on the analysis and design of drain projects.

Most case histories which deal with the analysis of vertical drain projects, conduct a back-analysis of the settlement curves to calculate the coefficient of consolidation (c_h). They obtain such values by keeping the equivalent diameter (d) constant and neglecting the effect of smear, well resistance, and filter jacket. By doing so, c_h can no longer be considered as the coefficient of consolidation, but some other constant representing factors influencing radial flow,

such as the effect of the drain core. Therefore, it is not surprising, that some researchers obtain different c_h values for different drains installed with the same spacing, on the same site.

CHAPTER - 3 -

3.0 Testing Apparatus and Procedures

This chapter describes the test apparatus, the instrumentation and the procedures used in this investigation. A brief description of the evolution which preceded each phase of the investigation is also included.

3.1 Test Containers

A desirable aspect of laboratory testing is to perform a test which is representative of field conditions. To achieve this criterion requires measuring the consolidation of a soil cylinder, the diameter of which should be chosen about equal to the spacing between drains used on actual drainage projects, i.e., at least 1 metre.

3.1.1 Sand Box

The first study into the possibility of measuring the consolidation of such a large volume of soil, led to the use of the University of Ottawa's existing test box. The test box measures 1800 mm in width, 2135 mm in height and 15.0 m in length. This was ideal for a full scale test, placing several drains in a triangular pattern and measuring the consolidation of the soil cylinder drained by the central wick. Sand, which is stored in the box could easily be used

as surcharge.

The initial design required a 2.1 m by 1.8 m area, having a 1.5 m depth for a 5.7 m³ volume of soil for each test and 22.8 m³ for each series. With the intention of using manufactured clay (kaolinite), such a large volume rendered the option of using the box impractical and uneconomical.

3.1.2 Sono Tubes

The next approach was to limit the laboratory testing to a 1 m depth of soil contained in a large diameter cylinder. Some of the first ideas that were considered consisted of a flexible membrane held in place by large diameter rings and an external metal frame, large diameter steel drums and sono tubes.

The later option was attractive in that it provided a possibility of cutting the tube and obtaining a cross sectional view of the clay, the drain and the instruments, at the end of primary consolidation. Therefore, this option was chosen and sono tubes, .940 mm in diameter with 13 mm thick walls, having a wax lining on the inside surface were obtained and cut into 1.2 m lengths.

One of the difficulties associated with the open ended sono tubes, was to provide a leak proof base. As anticipated, and proven in a smaller scale test, the flexible sono tube is

susceptible to expansion, and leakage at the base would be inevitable unless the entire tube circumference was rigidly restrained. The bottom 50 mm length of the sono tubes were therefore embedded into a concrete base, and, after curing, the surface of the concrete inside the tube was coated with shellac.

Prior to filling the tubes with clay, they were filled with water to verify that the tubes would not leak. During the first two days, no leaks were observed at the base nor anywhere else along the tube wall. On the third day, however, it became apparent that the wall of the tube was absorbing water and losing its strength, creating an instability. An attempt to remedy the situation was made by placing a double lining of polyethylene plastic with non-hardening sealing compound between the wall of the tube and each lining. This was also repeated at the base. The attempt was fruitless, and the use of sono tubes was abandoned.

3.1.3 Steel Cylinders

In order to obtain leak proof large diameter cylinders, five specially manufactured steel drums, 1 m in diameter with a 1.2 m depth, were purchased. The thickness selected for the wall was 3 mm, sufficient to provide adequate stability and to allow handling of the cylinder without altering its cylindrical shape. Additional caution was taken by welding a 25 mm by 25 mm angle around the top perimeter. To safeguard

against leaking, the wall was welded throughout at the base and vertical joint. Also, all instrumentation would exit the cylinder from the top. Geometrical data on the test containers are shown on figure 3.1. The steel cylinders and the set-up for Test Series One and Two are also shown on Plate 1..

All cylinders were painted to minimize oxidation and to prevent the interaction of iron elements with soil particles. Since the bottom plates were not perfectly flat, the cylinders were placed on a thin layer of lean concrete. This minimized any strain deformation upon application of a load and enabled small plates glued to the base on which the instruments are attached, to remain intact.

3.2 Instrumentation

In each container, the pore pressures developing in the clay as the consolidation proceeds, were measured by means of 18 pore pressure gauges (piezometers) placed at different depths and distance from the drain. The volume loss resulting from the drainage of the water, was measured by means of 6 settlement gauges placed in the clay inside the test containers.

All pore pressure gauges and settlement gauges were manufactured specially for the project and have a precision of about 2 mm water column change or settlement,

respectively. Laboratory testing has shown that the accuracy, when considering temperature, time lag, and reading precision, is about 20 mm (<2.5%) and 10 mm (<4.5%) of piezometers and settlement gauges, respectively. A detailed description of the instruments and their locations is presented in the subsequent sections of this chapter. To ensure longterm functioning of gauges in soil is difficult. Therefore, all gauges were doubled-up to enable a redundancy of data and to allow for the occurrence of gauge break-downs.

3.2.1 Piezometers

The pore pressure gauges consist of a typical manometer type piezometer as shown on Plates 2 and 3, and Fig. 3.2. A tube attached to a small porous membrane placed inside the test cylinder exits through the top, and is fixed to a panel for easy access. Upon the application of a load, coloured water inside the tube rises until the height of water is in equilibrium with the pore pressure in the soil matrix at the location of the porous membrane.

The porous membrane is a 100 mm thick section of 70 microns porous plastic tubing, having a 25 mm outside diameter and 13 mm inside diameter as shown on figure 3.2. Two 30 microns porous plastic plates, 3 mm in thickness, are fixed over the open surfaces of the tubing with epoxy sealing compound, preventing the inside of the piezometer tip from being contaminated by fine particles.

Prior to enclosing the porous tubing with the circular plates, a 2 mm diameter hole is drilled through the wall of the tubing and the end of a teflon tube, having a 2 mm outside diameter and 1 mm inside diameter (AWG 20), is inserted through the hole. The inside end of the teflon tube is then crimped by heating, to resist any pullout force, and fixed in place by sealing both the inside and outside orifices with epoxy sealing compound. When this procedure is completed, the circular plates are fixed in place, enclosing the system.

A small scale model study indicated the length of the tubing embedded in the soil, experienced bending due to consolidation of the soil layer. This downdrag acting on the teflon tubing could create excess pore pressures at the piezometer tip. To remedy this problem, a 150 mm length of tubing, nearest the porous membrane, is heated and shaped into a coil.

One of the main difficulties in designing the piezometers, was in selecting the size of teflon tubing. First, the volume of fluid required to record any significant change in pore pressure must be small. Minimizing this volume decreases the lag time (time for the soil matrix to receive or discharge water due to a decrease or increase in pore pressure). However, minimizing the size of tubing had the offsetting disadvantage of increasing the influence of capillarity.

Capillary rise (or depression) in a tube (Fig. 3.4) is due to surface tension of liquids in contact with air. From a free-body consideration, assuming the meniscus is spherical and equating the lifting force created by surface tension to the gravity force,

$$h = (2 \sigma \cos \theta) / (\gamma r) \quad 3.1$$

where σ = surface tension in units of force per unit length

γ = unit weight of liquid

r = radius of tube

h = capillary rise

θ = angle between the surface of the liquid and the surface of the tube at the point of contact

This expression can be used to compute the approximate capillary rise (or depression) in a tube and can be found in most text books on fluid mechanics. Fig. 3.5 indicates the effects of temperature and water quality on capillarity. R.G. Folsom, who determined the curve for tap water experimentally at the California Institute of Technology, found dirty water to give slightly lower values (Daugherty and Franzini, 1977). Table 3.1 lists values of capillary rise of water in a 1 mm inside diameter teflon tube, where it can be assumed that $\theta = 0$.

A series of tests were carried out in the laboratory to

verify the effects of capillarity. It was observed, that capillary rise occurred only when the water level in the tube decreased. When the fluid level rose, a depression was obtained instead. The effect of capillarity on a single tube with respect to a depression (Table 3.2), was greater with an observed maximum error of 39 mm, than the error obtained with a capillary rise (Table 3.3), having an observed maximum error of 23 mm.

These observations, inspired the addition of another tubing connected to the piezometer, via a Y-connector, at the base of the monitor panel. Thus, by fluctuating the level of fluid in the tubes, both the capillary rise and the depression could be obtained without adversely affecting the pressure at the piezometer tip. Taking the average of the two simultaneous readings, it is possible to minimize the error due to capillarity to 19 mm (Table 3.4).

An interesting problem observed during the testing of the instruments was the appearance of bubbles due to fluid breakup and possibly other sources. This effect is easily detected when observing the difference in fluid level of the two tubes. Laboratory analyses indicated that a single bubble could cause an error of as much as 20 mm (Table 3.5). To rid a single tube of these bubbles is very difficult. However, with two tubes this task is much easier.

To study the performance of the instruments in-situ, a small

scale model test was performed on a 550 mm thick layer of local clay (see Chapter 4 for soil properties), placed in a 400 mm diameter plastic barrel. Six piezometers were placed at different locations in the barrel. Five of these were connected to a set of two tubes, attached to a panel. The sixth piezometer (P-8) was connected outside the barrel to an electronic transducer. The electronic transducer was a PT25-10, capable of recording pore pressures with an accuracy of 53 mm (0.75%) and a repeatability within 7 mm (0.1%) as specified by the manufacturer.

Results of the study (Fig. 3.6) indicate that there is a lag of 20 minutes between the transducer readings and those obtained from the manometer type piezometers before registering the peak pressure following the application of a surcharge. Once the peak is obtained, there is little or no lag between the two types of piezometers (Figs. 3.6 and 3.7). The agreement of results between pairs of piezometers (P-8 and P-11, P-5 and P-7, P-6 and P-10) are within acceptable limits of variation. The small variations observed are probably due to slightly different positions of the instruments with respect to the drain wick and variations in pore pressures within the soil layer. Variations between piezometer P-11 and the electronic transducer (P-8), located at the bottom of the barrel and at equal distance from the drain wick, are greater, possibly due to the fluctuating datum of the electronic transducer as shown on Fig. 3.7, where a change in datum of as much as 309 mm has been

observed. This is an indication that the electronic instrument may be less reliable than the other manometer type piezometers.

3.2.2 Settlement Gauges

Measuring movements in the laboratory often requires the use of dial gauges, providing a high level of precision and accuracy. When movements are in excess of 50 mm, such precision is irrelevant and a simple ruler, able to measure within 1 mm (0.1 % strain in our case) is adequate.

The simplest method of measuring the settlement would be a system of gauges in direct contact with the soil, measuring directly the vertical movements of the soil at the surface and at depth within the soil layer. However, direct measurement using small diameter rods attached to plates placed within the soil, is impractical because of the obstructing surcharge. Electronic devices are excessively expensive and are often too sensitive to moisture.

Following a series of trials, the settlement gauge selected for the tests consisted of a small tube submerged in a container partially filled with alcohol, the level of which is monitored outside the test cylinder through the same tubing. The container of alcohol is fully enclosed within the soil mass, open to the atmosphere through two other small tubes. Details of a settlement gauge are given on Fig. 3.3.

The settlement gauge comprises of a 40 ml plastic container with a screw-on cap. A 75 mm plastic plate, shaped like a doughnut, is fixed to the container, glued against the base of the cap, to minimize the effect of potential rotation. Three teflon tubes (AWG 20) having a 2 mm outside diameter and 1 mm inside diameter are inserted into the container through holes drilled in the cap. Two of the tubes extend 10 mm into the container to serve as air lines and the third tube extends to the bottom to serve as the fluid line. All three tubes are heated and crimped at the inside surface of the cap to resist pullout forces and then glued in place with a plastic sealing compound.

To prevent that downward forces caused by the vertical displacement of the soil, be transmitted through the tubing to the container, a 150 mm section of the teflon tubing nearest the cap is heated and shaped into a coil. The tubes exit from the top of the test cylinder and are fixed to a separate panel outside the containers.

Unlike the piezometers, the settlement gauges are a closed system in which the fluid is not in contact with the soil. This enables the use of a fluid with a lower surface tension (σ) when in contact with air such as alcohol ($\sigma = 0.0223$ N/m), as compared to water ($\sigma = 0.0728$ N/m) at an air temperature of 20 °C. This results in reducing the effects of capillarity to a 15 mm rise as shown in Table 3.6. Results of tests shown in Table 3.6 also indicate that the capillarity effects

produces a rise both when the datum is increased or lowered, ranging between 10 and 15 mm.

To obtain a higher degree of accuracy, alcohol was used in the settlement gauges instead of water. With the use of a small syringe, 15 ml of alcohol is injected into each container through the fluid line. Bubbles appearing in the line are easily eliminated by draining the line at any time during the test, and reinjecting fluid free of air. No evaporation of alcohol during both test series was observed. Any evaporation which may have resulted, was well within the instruments reading precision.

The average of four measurements of surface displacement, taken to the nearest mm with the use of a ruler at the top of the model-test cylinder, are presented on Fig. 3.8. Results indicate that readings from settlement gauges placed at the surface of the soil layer are in good agreement with these measurements, the end result differing by only 1 mm.

3.2.3 Instrument Location.

In all three drain barrels of each test series, eighteen piezometers were placed radially away from the edge and face of the drain. Two were placed at the bottom (0 mm elevation), and eight at both 300 mm and 600 mm elevations (see Figures 3.9, 3.11 and 3.12). In the reference barrel, only six piezometers were required, with two placed at each the

bottom, 300 mm and 600 mm elevations. In all test barrels, six settlement gauges were used, with two placed at each of the 300 mm, 600 mm and 1000 mm (surface) elevations (Fig. 3.9 and 3.10).

All instruments were fixed at their designated elevation with the use of a fine nylon string. Attached to the instrument, one string is fixed to a small plate glued to the bottom of the test barrel and another to a cross-member fixed at the top. The bottom plate is aligned with respect to the top cross-member, which is pre-marked with regard to each instrument location. An indication of the instrumental set-up is also shown on Plate 3.

All piezometers are saturated by boiling prior to installation. Saturation is maintained once the instruments are placed in the barrels by submerging them in a container of water, until the soil is ready to be placed. Immediately before placing the soil in the barrel, a positive head of water is applied, causing water to flow out of the porous membrane. The top string is then pulled and tied in place with the piezometer tip at its proper elevation.

3.3 Drains

The drains analysed consisted of a studded core (drain A), a grooved core (drain G), and a third drain (drain M) having half the width of drain A. Characteristics of the drains are

presented on Fig. 3.13 and 3.14 and in Table 3.7 .

Drains A, and G, have approximately the same total peripheral area and drain M has about half that area. Drains M, and G, have approximately the same free or open peripheral area, and drain A has about twice that area.

All drains were placed in position prior to placing soil. A tie wire, passing through the stabilizing rod (Fig. 3.1), was attached to the top of the drain and to the external frame, keeping the drain in tension. A porous sleeve was placed over the top of the drain, embedded in a 200 mm thick layer of sand in which the drain could travel, to minimize any buckling caused by the consolidating soil.

All drain cores were placed in a polyester filter (Alidrain filter) to isolate the effect of the filter in the analysis. Prior to loading, an impervious polyethelene plastic sleeve was placed over each drain to assure identical start-up conditions as shown on Plate 3. Immediately before applying a surcharge, the sleeve was removed.

3.4 Soil Preparation

One of the requirements of this project was to be able to reproduce the test with the same soil properties at any time in the future. This required the use of an artificial soil. A soil mixture consisting of kaolinite, bentonite, and cement

such as used by Tavenas et al. (1973), was investigated. The results of this investigation (Chapter 4), indicated that creating identical soil properties for the large volume required ($>3.2 \text{ m}^3$), would be very difficult and require a very long time. Therefore, it became attractive to use natural soil from a local excavation.

3.4.1 Test Series One

The soil used in Test Series One consists of a clay and silt, the properties of which are detailed in the Chapter 4. The soil was remoulded with an industrial hand drill and water was added to make it softer and more compressible and to insure 100 percent saturation. Once the initial remoulding process was completed, the soil was again remoulded using a drill press at a speed of 900 rpm. Water was added until the mixture attained an adequate consistency.

Samples were taken of the clay to monitor the difference in moisture between batches and between test barrels. The soil was then poured through a #4 sieve, to remove any gravel size material, into a holding container (fifth barrel).

When sufficient soil had been mixed to fill a 1 m height of a test barrel (about 20 batches), the material in the holding tank was again mixed with the industrial hand drill and then poured into the instrumented test barrel.

3.4.2 Test Series Two

For test series two, an attempt was made to produce a soil, representative of deltaic deposits. This objective was achieved by mixing sand and 2.5 percent bentonite. Properties for this soil mixture are presented in chapter 4.

A mortar mixer was used to mix the sand, bentonite and water. The mixer had a volume capacity of 0.15 m^3 and turned at a speed of 50 rpm. The volume of each batch consisted of half the mixer's volume capacity.

Following the mixing of each batch, the soil mixture was immediately poured into the test barrel. Moisture samples were taken from each mix to monitor changes in water content. An average of 10.5 batches were required for each test barrel.

3.5 Surcharge

Initially, it was decided to use a soil surcharge representative of field conditions consisting of 1 to 1.5 m of sand fill and placed in an extension to the test cylinder. Model tests on a smaller cylinder indicated that considerable arching would be created. Although the actual cylinders were of larger diameter, the arching of the sand would not be sufficiently diminished. Other problems, such as stability of the cylinders, the handling of instruments and the

construction of a joint between cylinders, rendered this approach impractical and, instead, a surcharge consisting of concrete cylinders was used.

To serve as an upper draining filter, a 200 mm thick crushed silica sand layer was placed between the concrete and the soil. Prior to placing the sand, a thin pervious filter - geotechnical textile - was placed on the soil to serve as a membrane separating the sand and the soil. The concrete cylinders were placed on a 19 mm thick plywood platform placed directly on the sand.

The concrete weights were 300 mm thick and 900 mm diameter cylinders. Their design included the addition of four symmetrical slots and steel reinforcing for ease of handling. A circular slot was designed at its centre, through which a 30 mm diameter steel pipe could be inserted to provide stability. The mass of each cylinder varied from 280.8 kg to 292.1 kg with an average of 286.0 kg. Table 3.8 lists each cylinder with their respective mass. The combination of concrete cylinders as indicated in Tables 3.8 and 3.10, were selected in order to minimize any load variance between tests. The mass of the pipe and plywood platform was measured at 11.4 kg. The sand placed above the soil consisted of a 200 mm thick layer of medium grained silica sand, having a dry unit weight of 14 kN/m^3 when placed in a loose state.

A surcharge of 10.1 kPa was obtained by utilizing two

concrete cylinders as shown on Plate 4. The surcharge was kept on until primary consolidation was achieved, or until it was decided that sufficient time had elapsed.

CHAPTER - 4 -

SOIL PROPERTIES

4.1 Introduction

For the purpose of comparative analysis, the soil used in this study must have identical properties for each test performed. Due to their nature of deposition and stress history, identical properties cannot be obtained in natural soil deposits unless their properties are physically and/or chemically homogenized.

4.2 Artificial Soil

To obtain a soil with homogeneous properties, it becomes necessary to create an artificial soil, whose mechanical properties can be reproduced. This approach has been studied by Tavenas et al. (1972, 1973) as part of research programmes on the use of common in-situ test methods in Champlain clays.

The materials selected by Tavenas et al. (1972, 1973) to simulate all mechanical properties of the clay, consisted of 66.7 percent Kaolinite type Kaolex D-6, 8.3 percent Bentonite type Black Hill 200 and Portland Cement, CSA type III, and a water content at mixing of 157 percent. As can be expected, the addition of cement to the soil mixture, has the effect of.

increasing soil strength with time. Also, although not indicated in the study conducted by Tavenas et al. (1972, 1973), it is a well known fact that the chemical reaction between cement and water (hydration) is a function of the initial water content. The presence of cement in the soil may therefore influence the results of the study since the drainage of the soil layer will cause the water content to vary between test barrels. This would reduce the usefulness of the soil mixture.

Other alternatives, are to use a natural soil and physically alter its mechanical properties, or to use an artificial soil without cement. Both have been considered in the study, where clay remoulded with water, and sand mixed with water and bentonite were used in the first and second series of tests, respectively.

4.3 Test Series One

In the first series of tests, a natural clay was obtained locally from a construction site at the intersection of Merivale Road and Clyde Avenue in Ottawa. The following information outlines the soil characteristics determined in the laboratory prior to and subsequent to consolidation.

4.3.1 Characteristics of the Remoulded Soil

Fig. 4.3.1 presents a typical partical-size distribution of

the soil used in the laboratory testing as obtained from hydrometer tests (ASTM D-422, 1980). Results of these tests indicate that the soil consists of 3 to 4 percent sand, 39 to 46 percent silt and 51 to 57 percent clay. These grain size proportions classify the soil as a clay and silt with a trace of sand. From the aspect of engineering behaviour, clay content of 51 to 57 percent is dominant and, therefore, the soil is called clay.

A measure of the engineering behavior of this fine grained soil is better defined by the Atterberg limits. Atterberg limits are water contents at certain limiting or critical stages in soil behavior (Holtz and Kovacs, 1981). As presented in Table 4.2.1, the plastic limit (ASTM D424-59) of the soil used in the tests is 21.4 and the liquid limit (ASTM D423-66) is 50.3 to 52.3. Consequently, the plasticity index is 28.9 to 30.9.

A value of relative density, D_r , is necessary to compute the void ratio of a soil, and is useful in determining the unit weight of a soil. The relative density of the clay sample was determined to be 2.80. (The relative density - this term was formerly called "specific gravity" - is the unit weight of a material divided by the unit weight of distilled water at 4 degrees celsius).

Of great importance to this investigation, is permeability of the soil. Soil permeability determined by the falling head

method (ASTM D2434-65T, Bowles, 1970), were $1.5 \cdot 10^{-8}$ to $5.2 \cdot 10^{-8}$ m/sec .

Also of importance to this investigation, is the the coefficient of consolidation, c_v , which defines the material properties that govern the consolidation process, as defined by Equation 2.11. Because of the low consistency of the remoulded clay, the determination of c_v in the laboratory required the use of an apparatus called the "Rowe Cell" (Taylor, 1977). A number of tests were performed. However, due to the sensitivity of the remoulded clay sample and the lack of accuracy of the apparatus at low stress levels, only one test provided results that could be correlated to the behavior of the clay in the test barrels. The test was performed on a 57.2 mm thick sample placed in a 152.3 mm inside diameter cell. The initial moisture content was measured at 112.4 percent and the initial void ratio, e_0 , was 3.05 with a degree of saturation of 100 percent. The applied total stress (σ) increments were 6.5, 10, 20 and 40 kPa with an applied pore water pressure (back pressure), u , of 5 kPa. Plotting the void ratio versus the logarithm of effective stress (σ'), the coefficient of compressibility, a_v , as defined by equation 2.1, is found to be $-25.52 \cdot 10^{-6}$ m^2/N . At an effective stress of 15 kPa, representative of the effective stress acting on a soil element located at the mid section of the clay layer in our test cylinders, c_v was measured at $11.03 \cdot 10^{-8}$ m^2/sec , with reference to the Taylors method of analysis (Taylor, 1948).

The water content of samples taken from each batch of remoulded soil were recorded and the results are summarised in Table 4.3.1. Moisture samples of the remoulded soil as it was being poured in the test cylinders were also taken. The average water content varies from 111.9 to 115.2 percent between test cylinders.

4.3.2 Characteristics of the Consolidated Soil

In order to properly assess the characteristics of the consolidated soil, it is necessary to obtain undisturbed soil samples from the test cylinders. With the use of proper recovery equipment and precautionary methods, the degree of disturbance was kept to a minimum. This is best achieved with the use of special piston samplers. Attempts with a Swedish piston-sampler indicated that the adhesion of the clay to the inside of the sampler wall was not sufficient to break the suction created below the sampler as it was withdrawn. Therefore, air was supplied to the tip of the sampler with the aid of a small diameter tube with the purpose of breaking the vacuum. Once the sampler was in place, the small diameter (2mm) tube was inserted along its outside. As the sample is retrieved, the vacuum below the tip is broken through the air tube. (This procedure is only possible because of the laboratory conditions and the limited sampling depths).

A series of soil properties were measured on the recovered samples. Results of tests are presented in Tables 4.3.2-a and

4.3.2-b.

Water contents obtained at various depths and locations in each barrel, range from 64.12 to 80.73 percent, a reduction of between 26.37 to 53.0 percent from the initial water contents. Lower values were obtained at the top of the clay layer. It should also be noted at this point that the degree of consolidation is not the same for all barrels since soil sampling occurred prior to the end of primary consolidation.

The void ratio, e , varied between 2.04 and 2.30 .

The degree of saturation, the ratio of the volume of water to the total volume of voids in a soil mass, was measured between 94.0 and 100 percent.

Soil density, a measure of soil mass per unit volume, was measured both wet and dry. The wet total density of the clay, ρ_t , ranged from 1507 to 1586 kg/m³ and the dry density, ρ_d , ranged from 849 to 920 kg/m³ .

Tests on undisturbed samples were performed using a fixed ring consolidometer. To measure the soil properties of a sample having consolidated due to water flowing across its horizontal plane, samples from the 50 mm diameter liner tubes ("burks") contained inside the Swedish piston sampler during sampling were carefully trimmed and placed in a 44.55 mm diameter ring with the horizontal plane remaining

horizontal. To measure the soil properties of a sample consolidated due to water flowing across its vertical plane, samples were carefully trimmed and placed in a 38.20 mm diameter ring, with the vertical plane becoming then horizontal.

Tests were performed on samples obtained at a depth ranging from 360 to 530 mm below the clay surface and 110 to 150 mm from the band shaped drain. The estimated effective stress at this depth ranges from 11.9 to 13.2 kPa. Therefore, a loading increment of 5, 10, 15, 25, 45 and 90 kPa was selected and the following properties were measured (refer to Table 4.3.2-b):

- The coefficient of consolidation, c_v , due to pore water flow in the vertical direction, measured at a stress of 15 kPa ranged from $0.42 \cdot 10^{-8}$ to $13.36 \cdot 10^{-8} \text{ m}^2/\text{s}$. The coefficient of consolidation, c_h , due to pore water flow in the horizontal direction, measured at the same stress level (15 kPa), ranged from $1.02 \cdot 10^{-8}$ to $3.25 \cdot 10^{-8} \text{ m}^2/\text{s}$, with one test yielding a questionable value of $47.31 \cdot 10^{-8} \text{ m}^2/\text{s}$. Thus, the test results indicate that the vertical and horizontal coefficients are about equal.

- The preconsolidation pressure, σ_p , ranged between 10.4 and 14.0 kPa, which is in close agreement with the effective overburden pressure of 11.9 to 13.2 kPa.

- The coefficient of permeability, k , obtained from equation 2.11 varied from $1.18 \cdot 10^{-10}$ to $37.52 \cdot 10^{-10}$ m/s in the vertical direction, k_v , and from $2.60 \cdot 10^{-10}$ to $29.1 \cdot 10^{-10}$ m/s in the horizontal direction, k_h . The results indicate that the vertical and horizontal coefficients of permeability are about equal. This was expected since, according to equation 2.11, the coefficient of permeability is proportional to the coefficient of consolidation.

- The coefficient of compressibility, a_v , ranged between values of $-6.6 \cdot 10^{-6}$ and $-28.7 \cdot 10^{-6}$ m^2/N .

- The void ratio, e , at the stress level of 15 kPa was measured between 1.92 and 2.05 with one questionably high value of 2.22.

A number of attempts were made to measure the permeability of the consolidated clay samples in the "burks". However, voids created during sampling between the sample and the wall of the burk, resulted in erroneously high permeability. Other attempts were made and one yielded acceptable results. This test consisted of obtaining a sample with a 26.6 mm inside diameter tube, inserted into the 50 mm diameter soil sample and performing a falling head test. This test yielded a soil permeability, k_v , of $69.1 \cdot 10^{-10}$ m/s which is similar to the results of the test on a remoulded sample at the same stress level.

4.4 Test Series Two

In the second series of tests, an artificial soil consisting of a mixture of sand, bentonite, and water was used. Bentonite was used to obtain a more compressible soil, having a permeability comparable to deltaic deposits. Laboratory tests indicated that when using Kaolinite, segregation would occur within the soil mixture, and that both the desired soil compressibility and permeability could not be achieved. The method used to mix these materials are detailed in Chapter 3.

4.4.1 Sand Characteristics

A crushed silica sand described in Table 4.4.1 was chosen for this investigation. The silica sand was factory crushed quartz, designated as Silica-24. Fig. 4.4.1 presents a typical grain-size distribution of the silica sand. A grain-size analysis (ASTM D1140-54) was made to determine the relative proportions of the different grain sizes which make up a representative soil mass. The curve gives a coefficient of uniformity, C_u , of 1.67 to 2.75 and a coefficient of concavity, C_c , of 0.66 to 1.85. These coefficients classifies the soil as relatively poorly graded (uniform).

A washed sieve analysis indicated the silica sand contained less than 0.4 percent of material finer than 0.075 mm in diameter (passing # 200 sieve).

The value of relative density of the crushed silica sand, defined as the unit weight of the sand solid divided by the unit weight of distilled water at 4 degrees Celsius, was determined at 2.66 .

Soil density, ρ , was measured between 1250 and 1660 kg/m³ and the void ratio, e , ranged between 0.61 and 1.13 .

4.4.2 Bentonite Characteristics

The bentonite used in the laboratory tests consisted of a type Quick-Gel Bentonite, largely composed of the clay mineral Montmorillonite which is the most active of the identified clay minerals. A particle of montmorillonite consists of a gibbsite (approximately $Al_2O_3 \cdot 3H_2O$) mass sandwiched between two silica sheets for a total thickness of approximately 10 Å (angstrom). This material has a strong affinity for water, and may take on a layer of as much as 200 Å of water for a total of 400 Å between clay particles. It is this affinity for water which accounts in a large part for the high swelling characteristics of the montmorillonite mineral. The usual thickness of water layer is probably 10 to 100 Å (Bowles, 1977).

The relative density of bentonite varied between 2.13 and 2.18.

4.4.3 Characteristics of the Preconsolidated Artificial Soil

The soil mix proportions were determined based on two criteria;

- 1 - the permeability, k , of the artificial soil should range from 10^{-5} to 10^{-6} m/s, representative of deltaic deposits (silty sand),
- 2 - the volumetric strain should be about 10 percent for an applied load increase of about 10 kPa.

Fig. 4.4.2 presents typical values of falling head permeability tests performed on mixed samples consisting of silica sand and varying amounts of bentonite. The results indicate a permeability of $7.5 \cdot 10^{-5}$ m/s to $2.5 \cdot 10^{-9}$ m/s for 1 to 5 percent bentonite respectively. The best results were obtained with 2.5 percent bentonite and 97.5 percent silica sand with a permeability ranging from $1.8 \cdot 10^{-6}$ to $3.3 \cdot 10^{-6}$ m/s.

One dimensional compression tests performed with a Rowe Cell (Taylor, 1977), indicated a volumetric strain, ϵ , of 8.71 to 9.96 percent at an effective pressure of 15 kPa. These tests also indicate a coefficient of consolidation due to pore water flow in the vertical direction, c_v , of $5.2 \cdot 10^{-6}$ to $6.1 \cdot 10^{-6}$ m/s.

Water content of samples taken from each batch of mixed soil

are summarised in Table 4.4.2. These results indicate the average water content of the soil in each barrel varied from 34.5 to 38.8 percent.

4.4.4 Characteristics of Consolidated Artificial Soil

It is very difficult to obtain undisturbed soil samples in a loose deltaic deposit. The only method available to obtain such samples from the test barrels, was to insert a cylinder and excavate around it and supporting the bottom of the sample before removing the cylinder.

Results of tests performed on these samples are presented in Table 4.4.3 and are summarised below :

- the water content obtained from both disturbed and undisturbed samples, varied from 31.1 to 37.1 percent between barrels,
- the void ratio, e , varied between 0.88 and 1.07,
- the degree of saturation, S , varied from 85.7 in the No Drain barrel to 99.5 % in the Drain A barrel,
- the dry density, ρ_d , varied from 1277 to 1409 kg/m^3 and the wet density, ρ , from 1726 to 1858 kg/m^3 .
- the permeability, k , as measured from the falling head method ranged from $0.3 \cdot 10^{-6}$ to $16.7 \cdot 10^{-6}$ m/s.
- the percent of particle passing the # 200 sieve (0.075 mm), measured from a wash sieve analysis (ASTM D2217), varied from 2.34 to 3.22 percent. Subtracting the

percent of fines passing the # 200 sieve present in the sand sample, 0.4 percent (table 4.4.2), the amount of bentonite is estimated to range from 1.9 to 2.8.

CHAPTER - 5 -

EXPERIMENTAL RESULTS

5.1 General

The pore water pressure dissipation and settlement data, monitored with the aid of instruments fabricated at the University of Ottawa are presented below for Test Series One and Two. The unreduced data are presented in tables and figures in Appendix A, and the reduced data, computations and main figures and tables are presented in the end of the body of the report.

5.2 Test Series One

5.2.1 Raw Data

A total of 60 piezometers were used to monitor pore water pressures, 18 in each the Drain M, Drain G, and Drain A barrels and 6 in the No Drain barrel. Results of excess pore water pressures recorded at each instrument are presented in Tables A.1.1 through A1.4 in Appendix A. These data are also plotted with respect to time (days) on Figs. A.1.1 to A1.4 in Appendix A.

To measure settlements, 6 hydraulic gauges were used in each

test barrel for a total of 24 settlement gauges. Results of settlement readings are presented in Tables A2.1 to A2.4 in Appendix A. These results are also plotted on Figs. A2.1 to A2.4 in Appendix A.

The time 0.00 refers to the time load application. The surcharge was placed on all barrels on the 13th day following the mixing and placing of the clay. Pore pressures and settlements were monitored for a maximum of 82 days on the No Drain barrel. In none of the Series One tests, was the end of primary consolidation reached at termination.

5.2.2 Reliability

5.2.2.1 Excess Pore Water Pressure Readings

From the excess pore water pressure plots, Figs. A1.1 to A1.4 in Appendix A, it is possible to distinguish which piezometers were functioning properly from those giving erratic results. Of the 60 piezometers, information from 11 instruments was found faulty and was not utilised. These instruments consisted of P-G2 in the Drain M barrel; P-R1, P-G1, P-G2, P-G3, P-B1, P-B2 and P-B3 in the Drain G barrel and P-R3, P-G1, P-G4 in the Drain A barrel.

The peak excess pore pressure recorded ranged from 1112 to 1177 mm of water column height. Although a peak pressure of 1383 mm at 0.02 days was recorded for Drain A, Table A1.4,

with the electronic transducer, the sudden drop in pressure at 0.03 days indicates an erratically functioning gauge.

A post peak reading of 1165 mm was later recorded for Drain A with a minimum peak excess pore pressure of 1124 mm of water column height recorded for Drain M. The difference between peak excess pore pressures measured both with the fabricated piezometers and the electronic transducer, was a maximum of 25 mm or 2.2 per cent of peak excess pore pressure, recorded for Drain M.

The remoulding and placing of the clay in the barrels resulted in an initial excess pore pressure in the clay body, which had not fully dissipated at the time of the application of the surcharge. As all the drains were initially covered with a plastic sheath, the partial dissipation of this initial pore pressure was equal in all four barrels. The initial excess pore pressure recorded at the bottom of all barrels (P-R3, P-G3 in the No Drain barrel and P-G5 in the Drain M, Drain G, and Drain A barrels), varied from 337 mm to 351 mm.

The theoretical excess pore pressure due to applied surcharge is 10.1 kPa or 1030 mm of water column height. This value is in addition to the initial excess pore pressure existing in the soil matrix at the time of the load application.

Although the difference in initial excess pore pressure is

not significant, 14 mm or 4.1 percent, the difference between the measured peak excess pore pressures (1112 to 1177 mm) and the theoretical excess pore pressures (1367 to 1381 mm) ranges from 204 to 255 mm of water column height or 14.8 to 18.7 percent of the theoretical excess pore pressure.

The difference in peak excess pore pressure could be attributed to partial arching of the sand layer overlying the clay. Arching in the sand could also account for the occasionally observed erratic excess pore pressure data during the initial stage of consolidation.

By superimposing the raw data curves obtained from instruments placed at the same elevation (300 mm level; red and blue piezometers; 600 mm level; green and black piezometers), it is shown that no significant difference in excess pore pressure exists between instruments placed equal distance from the centre of the drain, for the three instruments farthest from the drain. Results from the piezometers closest to the drain (P-R1, P-G1, P-N1 and P-B1), indicate that the distance from the drain has an impact on the pore pressure reading. Based on these observations, the average reading of instruments is considered in the analysis. Where one gauge is defective, only a single reading is considered. These results are presented in Tables A3.1 to A3.4 of Appendix A.

5.2.2.2 Settlement Readings

Of the 24 settlement gauges used in Test Series One, 7 experienced leaking during the test. Three of the defective gauges were in the No Drain barrel, which was the first barrel to be loaded. Of these three gauges, two were found to be defective prior to loading the clay surface, probably caused during the handling of the gauges. The settlement gauges in the other barrels were subsequently coated with silicone gel which prevented further occurrence of defective gauges at the beginning of consolidation and probably increased the life of the gauges.

The seven defective gauges not used for analysis consist of S-1, S-2, S-3 in the No Drain barrel, S-4 in the Drain M barrel, S-1, S-2 in the Drain A barrel. The results presented in Tables A2.1 to A2.4 in Appendix A represent the average of two settlement gauge readings. Where one gauge was defective, the settlement reading recorded on that level is the result of only the one non-defective companion gauge reading. These results are also plotted on Figs. A2.1 to A2.4 in Appendix A.

As indicated from the small scale model test discussed in Chapter 3, the settlement results shown on Figs. A2.1 to A2.4, indicate the gauges were performing exceptionally well, providing consistent results and very smooth settlement curves.

5.3 Test Series Two

5.3.1 Raw Data

As in Test Series One, a total of 60 piezometers and 24 settlement gauges were used. Results of excess pore water pressures recorded at each piezometer are presented in Tables A4.1 to A4.4 in Appendix A together with the artificial soil surface elevation. These data are also plotted with respect to time (days), presented on Figs. A3.1 to A3.4 in Appendix A.

Results of settlement readings are presented in Tables A5.1 to A5.4 in Appendix A. These results are also plotted on Figs. A4.1 to A4.4 in Appendix A.

The surcharge consisting of the concrete cylinders, was placed on all barrels 7 days following the mixing of the soil. Pore pressures and settlements were monitored for a maximum period of 49 days on the No Drain barrel. At termination, all tests had reached the end of primary consolidation.

5.3.2 Reliability

5.3.2.1 Excess Pore Water Pressure Readings

From the excess pore water pressure plots, Figs. A3.1 to A3.4, it is possible to distinguish malfunctioning

piezometers. Of the 60 piezometers used, only one, P-N1 from the Drain G barrel, was deleted as defective from further analysis.

The peak excess pore pressure recorded ranged from 1170 mm of water column height in the Drain M, to 1489 mm in the No Drain barrel. The theoretical excess pore pressure due to the applied surcharge corresponds to 10.1 kPa or 1030 mm of water column height in addition to the initial excess pore pressure existing in the soil matrix at the time of load application. Similar to Test Series One, a plastic sheath was placed over all the drains to insure that the partial dissipation of the initial pore pressures would be equal in all barrels prior to applying the surcharge. This initial excess pore pressure recorded at the bottom of all barrels (P-R3, P-G3 in the No Drain barrel and P-G5 in the Drain M, Drain G and Drain A barrels, varied from 122 mm to 562 mm. Therefore, the difference between the measured (1170 mm and 1489 mm) and the theoretical (1152 mm and 1592 mm) peak excess pore pressures ranges between 18 mm and 103 mm of water column height or 1.6 to 6.5 percent.

The difference between the peak excess pore pressures (1170, 1489) as observed above, compared to the difference in the initial excess pore pressures (1152, 1592), is 319 mm and 440 mm, or 27.3 and 360.7 percent, respectively. The large discrepancy can be accounted for by the following hypotheses, which are discussed in more detail and with

further analysis in Chapter six;

- arching within the artificial soil layer,
- drainage between the walls and the base of the barrel,
- a difference in permeability within the soil layer and between barrels.

The time required to reach a peak excess pore pressure varied from 0.01 days and 0.04 days in the Drain M and No Drain barrels respectively. The time difference of 0.03 days accounts for only 0.25 percent of the time required to reach the end of consolidation in the Drain M barrel.

By superimposing the raw data curves obtained from instruments placed at the same elevation, as performed for Test Series One, it was again observed that no significant difference in excess pore pressures existed between instruments placed equal distance from the centre of the drain, for the three instruments furthest from the drain. Based on these observations, the average reading of both instruments located at the same elevation was considered for further analysis. Where one gauge was defective, only a single reading was considered. These results are presented in Tables A6.1 to A6.4 of Appendix A.

5.3.2.2 Settlement Readings

Of the 24 settlement gauges used in Test Series Two, only one gauge, S-4 in the No Drain barrel, was defective. The results

presented in Tables A5.1 to A5.4 in Appendix A, represent the average of two settlement gauge readings. The settlement reading recorded on the level of the defective gauge, is the result of the remaining properly functioning Gauge S-2. These results are also plotted on Figs. A5.1 to A5.4 in Appendix A.

CHAPTER - 6 -

ANALYSIS AND DISCUSSION OF RESULTS

6.1 Test Series One

The results of consolidation summarized on Fig. 6.1.1, confirm that vertical drains accelerate the consolidation process. It is difficult, however, to determine from these settlement curves, how much of the measured settlements are due to horizontal drainage. The excess pore pressure dissipation results presented in Chapter 5 are difficult to interpret without further analysis.

As mentioned in Chapter 5, the test was terminated before reaching the end of primary consolidation. Therefore, it is not possible to estimate the time and magnitude of settlement at the end of consolidation ($U = 100$) with the conventional semi-log approach presented by Casagrande (1938) nor with the square root of time approach presented by Taylor (1948). However, a recent approach presented by Asaoka (1978), and discussed by Eriksson and Ekstrom (1983) and Magnan (1981), enables an estimation of the time and magnitude of settlement at the end of primary consolidation by extrapolating the initial data in a linear relationship.

6.1.1 Time - Settlement Relationship

In the method of analysis proposed by Asaoka (1978), the trend equation of time series data of settlement from the one-dimensional consolidation equation is first derived. Then, this trend is used to extrapolate to the continued development. Asaoka (1978) proposes two methods, one graphical and one analytical, which are given in detail in Appendix B. For the analysis in this report, the method of interest is the graphical method due to its simplicity.

Figs. B1.1 to B1.4 in Appendix B, present end of consolidation projections using the Asaoka method of analysis as based on the settlement results presented on Figure 6.1.1 at a time interval, t , of five days. From these figures, the end of primary consolidation projections, or final settlement as described by Asaoka, are 286, 232, 230 and 228 mm for the No Drain, Drain M, Drain G and Drain A barrels, respectively.

Upon knowing the final settlement, s_f , the degree of consolidation can be obtained by dividing the recorded settlement at any time, t , by the final settlement. Figure 6.1.2, presents the results of consolidation ratio, U , vs time, t , (days), for a time interval, t , of five days.

From Fig. 6.1.2 it is obvious that the drains have accelerated the consolidation process by as much as 43 percent. Also indicated on Fig. 6.1.2, is that Drain A is

more effective at accelerating consolidation than Drain M and Drain G. Although this may appear to be negligible with respect to total settlement, what is of importance is the difference due to horizontal drainage only.

Before any analysis can be performed on the acceleration of consolidation due to horizontal drainage only, it is important to verify that the same results, or the same trend, can be obtained with regard to pore pressure dissipation.

6.1.2 Time - Pore Pressure Dissipation Relationship

The excess pore pressure dissipation data presented in Chapter 5 can be analysed with the use of vertical and horizontal isochrones indicating the mode and rate of dissipation with both depth and distance from the drain.

Vertical isochrones presented on Fig. 6.1.3, are obtained by plotting the average excess pore pressure monitored at 150, 250 and 350 mm from the edge of the drain and at a known depth as indicated, and at a given time, t , the data for which is presented in Tables A3.1 to A3.4 of Appendix A. The excess pore pressure remaining to be dissipated in the clay is represented by the area under the curve. The area between the curve at a given time, t , and the theoretical curve at time $t=0$, represents the excess pore pressure which has dissipated at time, t . Naturally, the excess pore pressure at the surface of the clay layer will be zero, since the surface

is free draining, and the peak excess pore pressure will be recorded at the bottom where the drainage path is the longest.

Results of this analysis indicate very good agreement in the data recorded at the beginning of consolidation, $t=1$ day. It can also be observed, at time $t=60$ days, that the consolidation trend is similar to that observed in the time-settlement analysis. In agreement with the results of the settlement gauges, the results from the piezometers clearly indicate that the drains accelerate consolidation and also that Drain A is more efficient than Drain M and Drain G.

The effect of the drains on excess pore pressure dissipation can best be observed by plotting the horizontal isochrones. Horizontal isochrones presented on Fig. 6.1.4, are obtained by plotting the average excess pore pressure monitored at levels 2 and 3 (600 mm and 300 mm at $t=0$) and at a known distance from the edge of the drain as indicated, and at given time, t , the data for which is presented in Tables A3.1 to A3.4 of Appendix A. The excess pore pressure remaining to be dissipated in the clay is represented by the area under the curve.

The results of this analysis indicate that there is no significant difference between the excess pore pressure recorded at 150 mm and 350 mm from the edge of the drain. This observation justifies averaging the excess pore pressure

results monitored at 150, 250 and 350 mm from the edge of the drain as in the previous analysis regarding vertical isochrones. Also, the excess pore pressure at the edge of the drain is recorded as zero since the drain is connected to a free draining sand layer at the surface, and the water pressure in the drain is hydrostatic with negligible well resistance.

Again, a good agreement with the data recorded at time $t=1$ day is observed. Also, Drain A is seen to be more efficient at accelerating the rate of consolidation than Drains M and G.

The development of the consolidation ratio, is indicated on Fig. 6.1.5 in terms of excess pore pressure dissipation versus time. The consolidation ratio, U , is obtained by dividing the area under the vertical isochrone curve at time, t , with the area under the theoretical isochrone curve at $t=0$. These results indicate a trend similar to the trend obtained with the consolidation ratio vs time relationship for settlements which is presented on Fig. 6.1.2. Of interest is the similarity of the effects on the rate of consolidation between the three drains at time $t = 70$ days.

Based on these observations, the analysis can be taken one step further to assess the efficiency of the drains due to horizontal pore water flow only.

6.1.3 Efficiency Analysis

In order to properly assess the efficiency of one premanufactured bandshaped drain, as compared with another, it is necessary to separate the effect of horizontal drainage or consolidation due to horizontal pore water flow from the vertical flow. According to Carrillo (1942), the average values of pore water pressure are expressed as

$$\bar{u} = 1 - (1 - \bar{u}_h)(1 - \bar{u}_v) \quad 2.17$$

This equation can be rewritten and expressed in terms of horizontal consolidation ratio, U_h , such as

$$\bar{U}_h = 1 - \frac{1 - \bar{U}}{1 - \bar{U}_v} \quad 6.1$$

From the analysis performed in section 6.1.1, the Time-settlement Relationship, it is seen that the average consolidation ratio, \bar{U} , at time $t = 70$ days was 63.0, 86.2, 87.8 and 90.0 per cent for the No Drain, Drain M, Drain G and Drain A barrels, respectively. Knowing the effect of vertical consolidation to be 63.0 per cent as obtained from the No Drain barrel, simple mathematics, making use of equation 6.1 provides the average horizontal consolidation ratio, \bar{U}_h , for each drain barrel at $t = 70$ days. These results were found to be 62.7, 67.0, and 73.0 per cent for the Drain M, Drain G and Drain A barrels respectively.

The efficiency of the drains at time $t = 70$ days can now be obtained by comparing the horizontal consolidation ratio, where the largest ratio would indicate the greatest efficiency, since at the end of consolidation $U_h = U_v = U = 100$ percent. Assuming Drain A to be 100 percent efficient since it yielded the largest horizontal consolidation ratio ($U_h = 73.0$ at $t = 70$ days), the efficiencies of Drain M and Drain G are found to be 85.9 and 91.8 percent respectively with respect to Drain A. These results are presented in Table 6.1.1 .

6.1.4 Equivalent Drain Diameter

Having the results of a full scale laboratory test, we can now determine which theoretical approach used to determine the effective diameter, d , of a premanufactured bandshaped drain described by equations 2.36 to 2.40 (page 24), is in better agreement with equation 2.41, obtained from laboratory observations. The equivalent drain diameter obtained from test results, d_6 , is derived from equation 2.34 and expressed as follows

$$d_6 = \frac{D}{\exp\left[\frac{-8 T_h}{\ln(1-\bar{U}_h)} + 0.75\right]} \quad 6.2$$

where $T_h = \frac{c_h t}{D^2}$

$D =$ diameter of the zone of influence, 1.0 m

The only parameter which remains to be known to solve the above equation, is the coefficient of consolidation due to horizontal flow, c_h . The coefficient of consolidation due to vertical flow, c_v , was verified in laboratory tests and found to be similar to the horizontal, c_h (refer to Table 4.3.3). This indicates that isotropic soil conditions in the test barrels were created through the mixing process.

With c_v expressed by the following equation;

$$c_v = \frac{-4 H^2 \ln(1-\bar{U}_v)}{\pi^2 t} \quad 6.1.3$$

where H = thickness of the clay layer (m),

t = time (seconds)

we obtain a value of $c_h = c_v = 5.25 * 10^{-8} \text{ m}^2/\text{sec}$ at time $t=70$ days. Inserting this value in equation 6.2 for each drain, equivalent circle drain diameters of 67.6, 47.7 and 35.9 mm are obtained for Drains A, G, and M respectively. These values represent a ratio to Drain A of 53.1 percent for Drain M and 70.6 percent for Drain G respectively, and are presented in Table 6.1.1.

When comparing these equivalent circle drain diameters to a sand drain with a porosity, $n = 0.4$ for the sand, one obtains an equivalent sand drain diameter of 169 mm, 119 mm and 90 mm for the Drain A, Drain G and Drain M respectively.

The results of this analysis, presented in Table 6.1.2,

indicate that equations 2.38 and 2.39 (d_3 and d_4) which are a function of the cross-sectional area of the drains, are inadequate. Equation 2.36 which is a function of the diameter of a circle whose total peripheral area is equivalent to that of the drain, overestimates the equivalent diameter by 6.3 %, 27.3 % and 0.9 % for the Drain M, Drain G and Drain A, respectively. Equation 2.40 which is similar to equation 2.36, but differs by considering only the open or free peripheral area of the drain, underestimates the effective drain diameter by 1.8 %, 41.4 % and 8.1 % for Drain M, Drain G and Drain A respectively. Equation 2.37, which is the mean of the width and the thickness of the drain, underestimates both the Drain M and Drain A by 19.3 %, and 26.1 %, respectively and overestimates Drain G by 7.4 percent.

Test Series Two

6.2.1 Time - Settlement Relationship

Due to the higher permeability of the sand bentonite mixture in Test Series Two, it was possible to record the final settlement in a relatively short period of time, presented on Fig. 6.2.0. This enabled the use of the square root of time approach, as presented by Taylor (1948), to obtain the time and settlement at 90 per cent of consolidation ($U=90$ %).

Upon knowing the final settlement, s_f , the degree of consolidation can be obtained by dividing the recorded

settlement at any time, t , by the final settlement. Fig. 6.2.1 presents the results of the consolidation ratio, U , versus time (days). From these results, the time to reach 90 per cent of consolidation ($U=90\%$) is 1.6, 4.4, 12.7, and 40 days for the Drain M, Drain A, Drain G and No Drain barrels, respectively.

It is noted from Fig. 6.2.1 that there exists a large discrepancy between the settlement obtained for each barrel at the end of consolidation. Since at the end of consolidation, the settlements in each barrel should be the same, these observations are an indication that the soil properties and/or the loading and drainage characteristics for each barrel may be different.

Results from laboratory tests performed on the sand-bentonite mixture obtained from each test barrel at the end of consolidation, indicate that no significant difference existed in the soil properties between all test barrels.

These findings imply that the difference most likely existed in the loading and/or drainage characteristics of each barrel.

6.2.2 Time - Pore Pressure Dissipation Relationship

Vertical isochrones presented on Fig. 6.2.3 are obtained by plotting the average excess pore pressure monitored 150, 250 and 350 mm from the edge of the drain and at a known depth as

indicated, and at a given time, t , for which the data are presented in Tables A6.1 To A6.4 of Appendix A.

Horizontal isochrones presented on Fig. 6.2.4, are obtained by plotting the average excess pore pressure monitored at levels 2 and 3 (600 mm and 300 mm at $t=0$) and at a known distance, t , for which the data are presented in Tables A6.1 to A6.4 of Appendix A.

For both the horizontal and vertical isochrones, the excess pore pressure remaining to be dissipated, is presented by the area below the curve.

The consolidation ratio, U , versus time, t , presented on Fig. 6.2.5, for excess pore pressure dissipation, is obtained by dividing the area below the vertical isochrone curve at time, t , with the area below the theoretical curve at $t=0$. From these results, the time to reach 90 percent of consolidation ($U=90\%$) is 1.1, 4.0, 9.5 and 28.5 days for the Drain M, Drain A, Drain G and No Drain barrels, respectively.

The vertical isochrones presented on Fig. 6.2.3 indicate that significant arching was present in the Drain A and Drain M barrels at about 500 to 600 mm above the bottom. Arching is also present in the Drain G barrel although not as pronounced.

Since the rate of consolidation is a function of time (time for excess pore water dissipation) and not of the applied load, it was believed that the results could best be analyzed by comparing the consolidation ratio versus time curves shown on Figs. 6.2.2 and 6.2.5. The results presented on Fig. 6.2.2 differ slightly from those presented on Fig. 6.2.5 because they were obtained from total settlements measured at the surface as compared with excess pore pressure dissipation measured at four different elevations within the soil layer. Also, results from both figures indicate the Drain M to be more efficient than Drain A which is theoretically impossible. Therefore, it would appear that some other factor may be influencing the rate of consolidation.

The horizontal isochrones presented on Fig. 6.2.4 indicates a drop in excess pore pressure near the outside edge of the barrel for Drain A. This drop in pressure could result from drainage along the inner wall of the test barrel. Drainage along the walls and base of the barrels, combined with arching appear to be the cause for the discrepancy in the consolidation results, the effects of which differs in each barrel.

Since the loading and drainage characteristics of each barrel are no longer the same, the effect of each drain can no longer be isolated. Hence, the results of Test Series Two could not be used to measure the effective drain diameter and compare the efficiency of each drain.

CHAPTER - 7 -

CONCLUSIONS

The results from a full scale laboratory testing programme conducted on premanufactured band shaped drains of three different geometrical design are presented in this report. From these results, the following conclusions are drawn :

1) An accurate experimental investigation into the understanding of the effective drain diameter of band shaped drains, representing in-situ conditions was successfully completed at the University of Ottawa. The manner in which the tests were performed, enabled the elimination and isolation of many factors which have generally obscured the results reported in the technical literature, such as Hansbo (1983), Jansen and Den Hoedt (1983), Eriksson and Ekstrom (1983), Foot and Koutsoftas (1982).

2) Laboratory tests performed on soil samples obtained from the soil mixtures prior to and at the completion of the tests (Chap. 4), indicate that isotropic soil conditions were obtained. These tests also revealed that the coefficients of consolidation due to vertical and horizontal drainage were about the same for all test cylinders. Although the coefficients of consolidation are a function of the effective

stress within the soil matrix which changes with time during the consolidation process, the coefficient of consolidation due to vertical flow used in the analysis was obtained from the No Drain barrel. This approach is considered reasonable for this comparative study since the soil in all test barrels had the same soil properties and were subjected to the same loading conditions.

3) The instruments used to monitor pore pressures and settlements over an extended period of time, consisting of hydraulic pore pressure gauges and settlement gauges fabricated in the laboratory, proved to be more reliable than electronic transducers. The piezometers have a reading precision of 2 mm. The accuracy of the piezometers is 20 mm of water column height and the accuracy of the settlement gauges is 10 mm.

Of the 60 piezometers used in Test Series One, 11 were found to be faulty. This was improved in Test Series Two, where only one piezometer was found to be defective.

Of the 24 settlement gauges used in Test Series One, 7 experienced leaking. Again, this was improved in Test Series Two to only one defective settlement gauge.

4) The results of the analysis performed for Test Series One in Section 6.1.1, indicate that the method of estimating the final settlement proposed by Asaoka yields reasonable

results, the discrepancy between the drain barrels being only 4 mm or 1.8 percent.

Further analysis represented by Fig. 6.1.2 and Figure 6.1.5 confirm that the drains accelerate the consolidation process. Both indicate the same trends, and the results at $t = 70$ days differ by a maximum of only 7.9 percent obtained for the No Drain barrel. Both analyses indicate Drain A to be more effective than Drain M and Drain G.

The results presented on Fig. 6.1.1 also indicate that the settlement gauges were performing very well, providing consistent results. The similar observation is not as evident on Figure 6.1.5 obtained from piezometer observations. This phenomenon may have resulted due to recording excess pore pressures, and referencing these results to the theoretical excess pore pressure, a value which was never reached according to the peak excess pore pressures monitored. When this effect is removed by plotting the isochrones as shown in Fig. 6.1.3 and 6.1.4 a clear indication that the piezometers were performing well and providing consistent results was obtained.

Based on the agreement in results of the above analyses and making use of the theory presented by Carrillo (1942), it is possible to separate the degree of consolidation due to vertical drainage from the effects of horizontal drainage. The results of this analysis indicates that the average

horizontal consolidation ratio, U_h , for Drain M, Drain G and Drain A is 62.7, 67.0 and 73.0 per cent, respectively. Expressed in terms of efficiency with respect to Drain A, we find that the efficiency of Drain M and Drain G are 85.9 and 91.8 per cent of Drain A, respectively.

By assuming the coefficient of horizontal and vertical consolidation, c_h and c_v , to be equal, it is possible to determine the effective drain diameters. This assumption is justified by having isotropic soil conditions at the beginning of the tests and knowing that the degree of consolidation due to vertical flow does not vary significantly from that due to horizontal flow (15 %). Also the coefficients of consolidation $c_v = c_h = 5.25 * 10^{-8} \text{ m}^2/\text{s}$ were found to be well within the range of values obtained from oedometer tests presented in Chapter 4, Table 4.3.3. The results of the effective drain diameter analysis, presented in Table 6.1.2 indicate that the effective drain diameter, d , is a function of both the geometry and the free or open surface of the drain. This argument is based on the fact that both equation 2.36 and 2.40 (page 24) yield similar results for both Drain M and Drain A since the free or open surface of the studded drains are 90 % of the total peripheral surface. The equivalent diameter of the grooved drain with 50 % free surface is overestimated by equation 2.36 (27.3 %) and underestimated by equation 2.40 (41.4 %).

The equivalent circle drain diameters obtained from the

results of Test Series One were 67.6, 47.7, and 35.9 mm for Drains A, G, and M, respectively.

Based on the above equivalent circle diameters and the concept of free surface ratio, the equivalent sand drain diameter with a porosity, $n = 0.4$, for the sand is 169 mm, 119 mm, and 90 mm for Drains A, G, and M, respectively.

5) The results of the analysis performed in Section 6.2.1 for Test Series Two, indicate that the tests did not perform as expected. Having followed the same procedures for all four test barrels, the final settlement, s_f , should have been identical. Fig. 6.2.1 indicates that this was not so.

Laboratory tests performed in Chapter 4 indicate that the soil properties were basically the same for all test barrels.

The vertical isochrones plotted on Fig. 6.2.3 indicates that arching in the sand-bentonite mixture, at about the 500 to 600 mm level, was present in both the Drain A and Drain M barrels and possibly in the Drain G barrel.

The horizontal isochrones plotted on Fig. 6.2.4 indicate that drainage along the walls of the barrels was also present:

6) With the controlled laboratory testing, adequate instrumentation and various methods of analysis performed in this study, it was possible to obtain a better understanding

of the performance of premanufactured drains and how they help to accelerate the rate of consolidation. In addition, the abundance of information made it possible to evaluate external factors affecting the test results, such as with Test Series Two, providing some guidance toward the improvement of future tests.

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Table 3.1. Surface tension of water and capillary rise

Temperature (°C)	Surface tension (N/m)	Specific weight (kN/m ³)	Capillary rise, h (mm)
0	0.0756	9.805	30.84
10	0.0742	9.804	30.27
20	0.0728	9.789	29.75
30	0.0712	9.764	29.17

Table 3.2 Capillarity in teflon (AWG 20) tubes - effects of water in a single tube (dropping datum).

Datum (Rising)	Effect of capillarity (mm)									
	Tube No.									
	1-L	1-R	2-L	2-R	3-L	3-R	4-L	4-R	5-L	5-R
545		-30		-22		-39		-11		-21
776		-33		-24		-36		-24		-27
957		-30		-23		-33		-24		-25
1043	-23	-25	-20	-19	-20	-20	-18	-17	-23	-25
510	-19	-18	-21	-19			-13	-12	-19	-19
830	-27	-27	-26	-26			-25	-25	-27	-27
1096	-25	-26	-22	-22			-22	-23	-26	-27

Note: - = depression
+ = rise

Table 3.3 Capillarity in teflon (AWG 20) tubes - effect of water in a single tube (rising datum).

Datum (Lowering)	Effect of capillarity (mm)									
	Tube No.									
	1-L	1-R	2-L	2-R	3-L	3-R	4-L	4-R	5-L	5-R
777		+23		+3		+16		+4		-4
606		+5		+5		+13		+7		0
362		+16		+7		+16		+14		+2
476	-1	-1	+2	+1	+4	+2	+8	+7	0	+1
1021	-1	-3	+3	+3			+2	+9	-6	-6
822	-1	-1	+5	+6			+2	+2	-3	-4
449	+4	+3	+5	+5			+17	+16	+4	+2

Note: - = depression

+ = rise

Table 3.4. Capillarity in teflon (AWG 20) tubes - effects of water in double tube with fluctuating fluid levels (rising in right tube and dropping in left tube).

Datum elev. (mm)	Effect of capillarity (mm)									
	Tube No.									
	1-L	1-R	2-L	2-R	3-L	3-R	4-L	4-R	5-L	5-R
476	-13 -6.5	0	-31 -14	+3	-11 -4.5	+2	-11 -2	+7	-16 -8	0
747	-25 -16.5	-8	-23 -17.5	-12	-22 -11.5	-1	-22 -11.5	-1	-25 -16.5	-8
920	-25 -16.5	-8	-24 -17.0	-10	-21 -10.5	0	-21 -11.5	+2	-25 -17.5	-10
622	-10 -4.0	+2	-10 +8.0	+26	-7 -2.0	+3	-7 0	+7	-11 -6.0	-1
916	-24 -15.5	-7	-24 -13.0	-2	-20 -10.0	0	-23 -14.0	-5	-26 -19.0	-12

Note: - = depression

+ = rise

Table 3.5 Capillarity in teflon (AWG20) tubes - effect of a single bubble in one tube.

Effect of Capillarity (mm)

Datum (mm)	Tube No.		Average	Comments
	1-L	1-R		
+ 510	- 13	- 16	- 14.5	
+ 830	- 59	- 24	- 41.5	
+ 1096	- 46	- 21	- 33.5	
- 1021	+ 28	+ 3	+ 15.5	Single bubble
- 822	+ 30	+ 3	+ 16.5	in left tube
- 449	+ 18	+ 19	+ 18.5	(1-L)
- 916	+ 25	- 2	+ 11.5	
- 539	+ 4	0	+ 2	
+ 1072	- 18	+ 4	- 7.0	

Note: + = depression / rise in datum

- = rise / lowering of datum

Table 3.6 Capillarity in teflon (AWG20) tubes - effects of alcohol in a single tube.

Datum (mm)	Fluid level in tube (mm)	Capillary rise (mm)
+ 643	630	13
	628	15
	628	15
	630	13
	628	15
+ 966	956	10
	956	10
	956	10
	956	10
	956	10
- 864	853	11
- 860	849	11
- 864	853	11

Note: + = rising of datum
 - = lowering of datum

Table 3.7 : Drain Characteristics

Drain	Characteristics	a (mm)	d (mm)	b' (mm)	a' (mm)
A	Studs on	100	7.2	91	7.2
II	Both Sides	53	7.2	48.2	7.2
G	Grooved	100	3.0	50	3.0

Note: Polyester filter used with all cores

a, b, a', b' - defined on Fig. 3.13 and 3.14

Table 3.8 Mass of concrete cylinders

Cylinder No.	A	B	C	D	E	F	G
Weight (kg)	289.9	286.2	285.3	292.1	288.5	283.0	285.3

Cylinder No.	H	I	J	K	L
Weight (kg)	286.7	285.3	280.8	283.0	285.3

Table 3.9 Test Series One - Surcharge

Cylinder	Mass of rod & base (kg)	1st Cyl. mass (kg)	2nd Cyl. mass (kg)	Total mass (kg)	Sand load (kPa)	Total load (kPa)
No Drain	11.4	280.8	292.1	584.3	2.8	10.06
Drain M	11.4	289.9	283.0	584.3	2.8	10.06
Drain G	11.4	288.5	283.0	582.9	2.8	10.04
Drain A	11.4	285.3	286.2	582.9	2.8	10.04

Table 3.10 Surcharge - Test Series Two

Cylinder	Mass of rod & base (kg)	1st Cyl. mass (kg)	2nd Cyl. mass (kg)	Total mass (kg)	Sand load (kPa)	Total load (kPa)
No Drain	11.4	292.1	280.8	584.3	2.8	10.06
Drain M	11.4	285.3	286.2	582.9	2.8	10.04
Drain G	11.4	289.9	293.0	584.3	2.8	10.06
Drain A	11.4	288.5	283.0	582.9	2.8	10.04

Table 4.3.1 Characteristics of Remoulded Clay

% Clay Size Particles	51 - 57
% Silt Size Particles	39 - 46
% Sand	3 - 4
Classification	Clay and Silt, trace of Sand
Liquid Limit, W_L ,	50.3 - 52.3
Plastic Limit, W_P ,	21.4
Plasticity Index, I_p ,	28.9 - 30.9
Specific Gravity, G_s	2.80
Permeability	min: 1.5×10^{-8} m/sec max: 5.2×10^{-8} m/sec

Rowe Cell Test Results:

Applied Pressure	= 20 kPa
Back Water Pressure	= 5 kPa
Effective Pressure	= 15 kPa
Coefficient of Consolidation c_v	= 11.0×10^{-8} m ² /sec
Coefficient of Compressibility a_v	= -25.5×10^{-6} m ² /N
Initial Void Ratio	$e_o = 3.05$
Initial Water Content	$w = 112.4\%$
Degree of Saturation	$S = 100\%$

Water Content	Min.	Max.	Ave.
No Drain Barrel	107.1	114.4	111.9
Drain M Barrel	111.6	114.9	113.6
Drain G Barrel	111.2	114.4	113.0
Drain A Barrel	113.2	117.2	115.2

Table 4.3.2 Characteristics of Consolidated Clay

Soil , Property	No Drain		Drain M		Drain G		Drain A	
	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
Water Content (W) %	64.12	80.73	68.36	79.81	65.87	76.21	67.85	79.25
Void Ratio e	2.15		2.04		2.10		2.08	
		2.30		2.15		2.30		2.23
Degree of Saturation %	94.0		94.5		94.5		95.4	
		100.0		99.8		99.7		97.3
Dry Density (ρ_d) g/cm ³	0.849		0.889		0.849		0.884	
		0.888		0.920		0.904		0.908
Wet Density (ρ) g/cm ³	1.514		1.533		1.507		1.525	
		1.565		1.586		1.579		1.566

Table 4.3.2-b : Characteristics of Consolidated Clay

Laboratory Test	Sample	Coefficient of Consolidation ($10^{-6} m^2/sec$)	Coefficient of Compressibility (m^2/H)	Preconsolidation Pressure (kPa)	Permeability ($10^{-10} m/sec$)	Water Content w (%)	Void Ratio e	Degree of Saturation S (%)
Cedometer	No Drain	1.11	9.40	14.0	3.14	81.5	2.26	100
Test on	Drain H	13.36	8.76	13.9	37.52	73.5	2.06	99.8
Vertical	Drain G	0.42	8.85	11.8	1.18	74.7	2.10	99.7
Sample	Drain A	2.88	6.6	11.0	5.78	73.7	2.23	95.4
Cedometer	No Drain	1.37	6.12	10.4	2.60	76.2	2.15	99.1
Test on	Drain H	3.25	28.7	12.0	29.1	72.6	2.15	94.5
Horizontal	Drain G	43.31 ?	?	?	?	77.6	2.30	94.5
Sample	Drain A	1.02	10.6	11.5	3.36	73.9	2.16	95.9
Flow Cell	Remoulded	11.03	25.52	-	68.2	112.4	3.05	100
Falling Head	Drain A				69.1	74.6	2.17	96.3

Table 4.4.1 Characteristics of Crushed Silica Sand

Type	Silica-24
% Crushed	100
Coefficient of Uniformity, c_u	1.67 - 2.75
Coefficient of Concavity, c_c	0.66 - 1.85
% Fines (passing # 200 sieve)	0.4
Classification	uniformly graded sand
Specific Gravity, G_s	2.66
Density, ρ_d , (g/cm^3)	Min: 1.250 Max: 1.660
Void Ratio, e ,	Min: 0.61 Max: 1.13

Table 4.4.2 Characteristics of Preconsolidated Artificial Soil

% Silica Sand (Silica-24)	97.5
% Bentonite (Quick-Gel)	2.5
Specific Gravity, G_s ,	2.65
Permeability, k ,	Min: 1.8×10^{-6} m/s Max: 3.3×10^{-6} m/s

Rowe Cell Test Results:

Applied Pressure	= 20 kPa
Back Water Pressure u	= 5 kPa
Effective Pressure	= 15 kPa
Coefficient of Consolidation c_v	= $5.5 - 6.1 \times 10^{-6}$ m ² /s
Coefficient of Compressibility, a_v	= -1.87×10^{-6} m ² /N
Volumetric Strain, (%)	8.71 - 9.96
Initial Void Ratio, e_0 ,	0.893
Initial Water Content, w , (%)	33.0 - 37.1
Degree of Saturation, S ,	100 %

Water Content	Min.	Max.	Ave.
No Drain Barrel	35.43	41.16	38.78
Drain M Barrel	32.36	35.78	34.50
Drain G Barrel	34.59	34.74	36.19
Drain A Barrel	33.21	37.57	35.65

Table 4.4.3 Characteristics of Consolidated Artificial Soil

Soil Property	No Drain		Drain M		Drain G		Drain A	
	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
Water Content w(%)	33.0	37.1	31.1	33.9	31.6	35.2	31.4	36.3
Void Ratio e	0.99	1.02	0.88	0.98	0.96	1.07	0.97	1.01
Degree of Saturation (%)	85.7	97.5	91.8	95.5	86.8	90.1	88.3	99.5
Dry Density (g/cm ³)	1.310	1.333	1.338	1.409	1.277	1.355	1.316	1.347
Wet Density (g/cm ³)	1.743	1.809	1.791	1.858	1.726	1.789	1.763	1.836
% Fines	2.86	3.22	2.65	3.02	2.34	2.77	2.75	2.95
Permeability k (10 ⁻⁶ m/s)	0.3	8.1	4.3	16.7	3.1	12.5	5.2	7.8

Table 6.1.1.1 : Results of Drain Efficiency and Effective Circle Drain and Sand Drain Diameters

Drain Type	\bar{U} (%)	U_v (%)	U_h (%)	Efficiency (%)	c_v & c_h ($10^{-8} \text{m}^2/\text{s}$)	d_6 (mm)	Ratio $d_{6M,G}/d_{6A}$ (%)	d_s (mm)
No Drain	63.0	63.0	-	-	5.25	-	-	-
Drain H	86.2	63.0	62.7	85.9	5.25	35.9	53.1	90
Drain G	87.8	63.0	67.0	91.8	5.25	47.7	70.6	119
Drain A	90.0	63.0	73.0	100.0	5.25	67.6	100.0	169

where : d_6 = equivalent circle drain diameter determined from test results

d_s = equivalent sand drain diameter (porosity, $n = 0.4$)

Table 6.1.2 : Comparison of Theoretical Effective Circle Drain Diameters

Drain Type	d_6 (mm)	d_1 (mm)	d_1/d_6 (%)	d_2 (mm)	d_2/d_6 (%)	d_3 (mm)	d_3/d_6 (%)	d_4 (mm)	d_4/d_6 (%)	d_5 (mm)	d_5/d_6 (%)
Drain M	35.9	38.3	+6.3	30.1	-19.3	22.0	-62.9	14.3	-151.1	35.3	-1.8
Drain G	47.7	65.6	+27.3	51.5	+7.4	19.5	-144.1	17.6	-171.5	33.7	-41.4
Drain A	67.6	68.2	+0.9	53.6	-26.1	30.3	-123.3	18.1	-273.3	62.5	-8.1

where : $d_1 = 2(a+d) \frac{1}{\eta}$

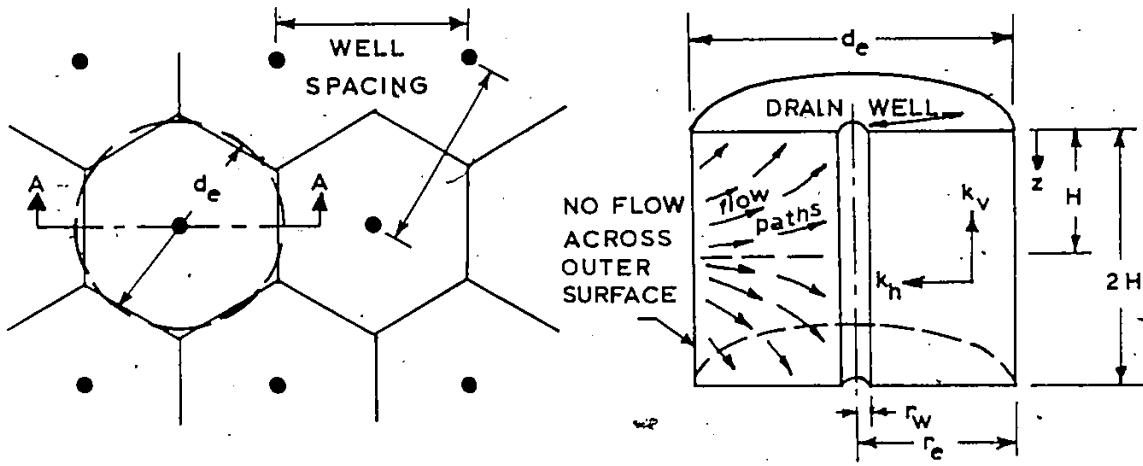
$d_2 = \frac{a+b}{2}$

$d_3 = \frac{4}{\eta} \times axb$

$d_4 = \frac{4}{\eta} (\text{open area of } axb)$

$d_5 = 2(a'+b') \frac{1}{\eta}$

d_6 = determined from Test Series One



(a) Plan of drain well pattern

(b) Section A-A

FIG. 2.1 : Plan of Drain Wells and Concept of Flow Within the Zone of Influence of Each Well

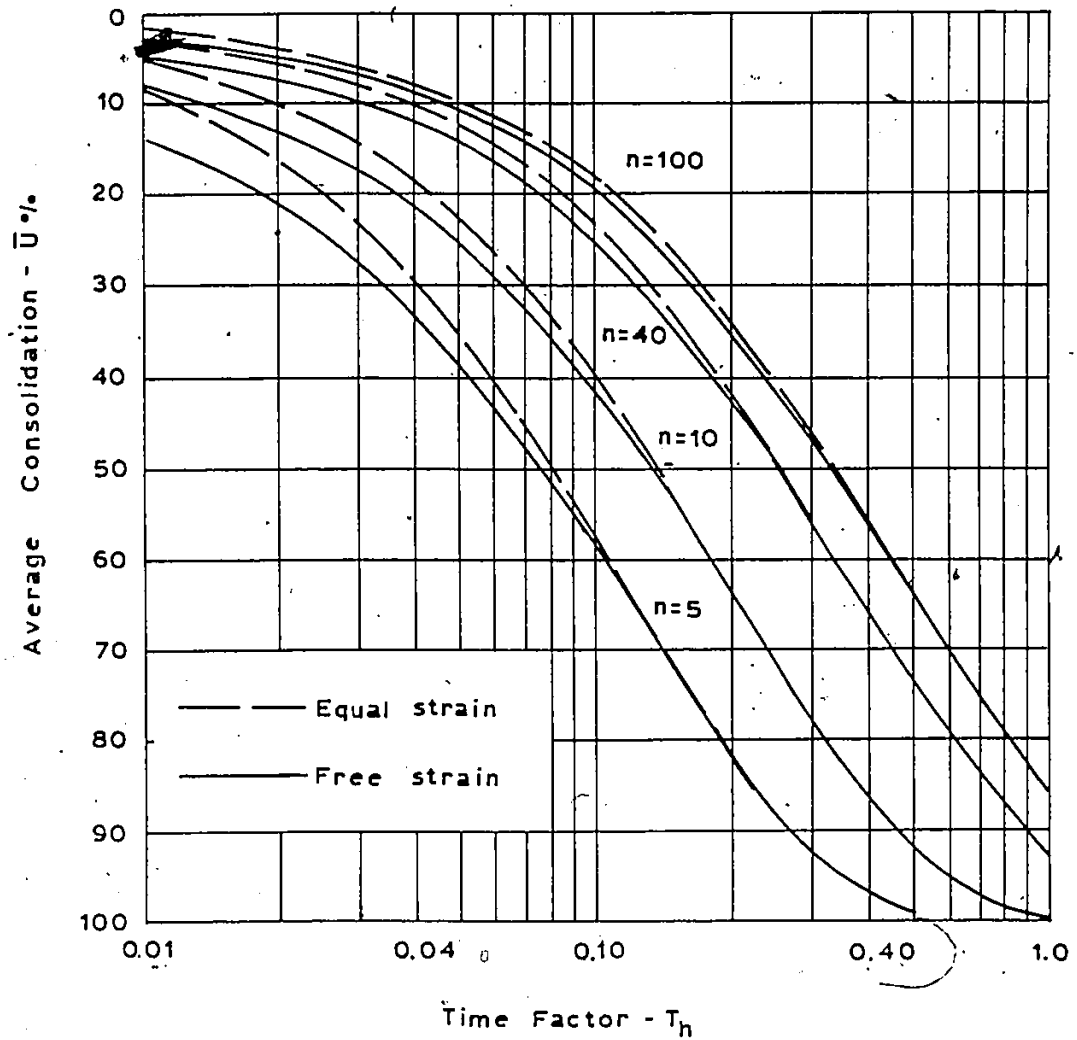


FIG. 2.2 : Average Rates of Consolidation - \bar{U} (%), for Cases of Free Strain and Equal Strain with Varying Drain Spacing - n (after Barron, 1949)

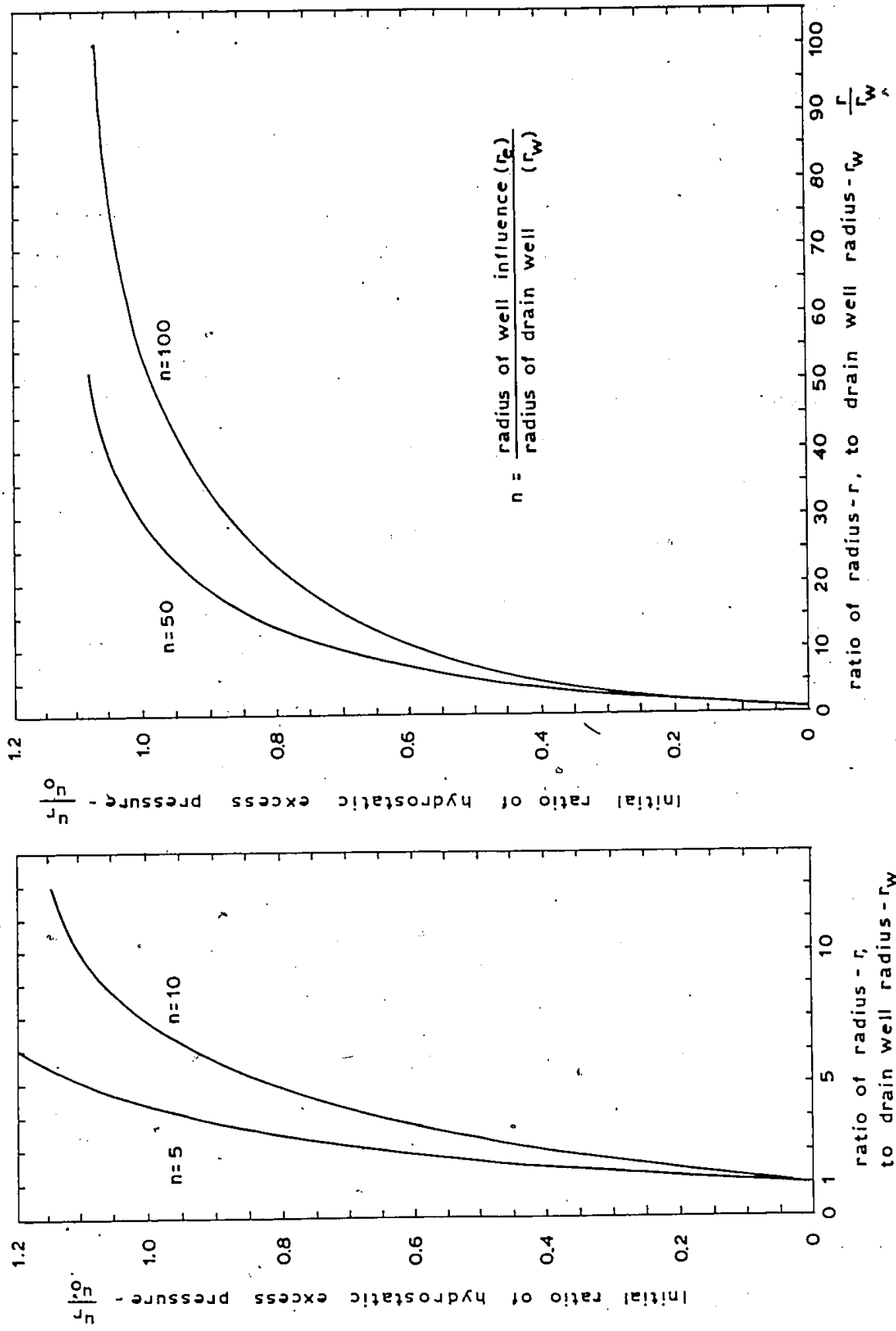


FIG. 2.3 : Initial Values of Excess Pore Water Pressure for "Equal Strain" Case (after Barron, 1948)

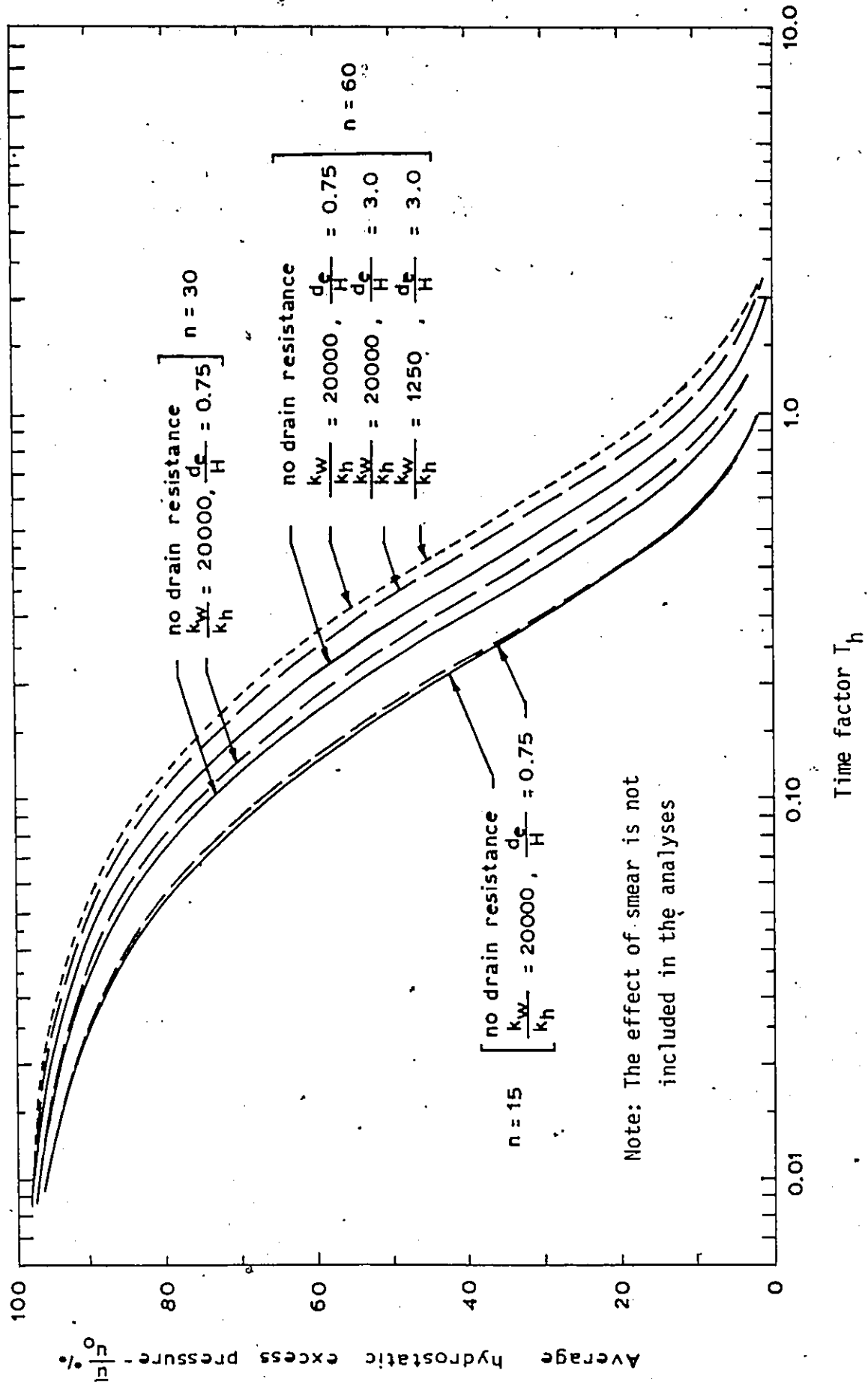


FIG. 2.4 : Effect of Varying Drain Resistance on the Average Consolidation (after Barron 1948)

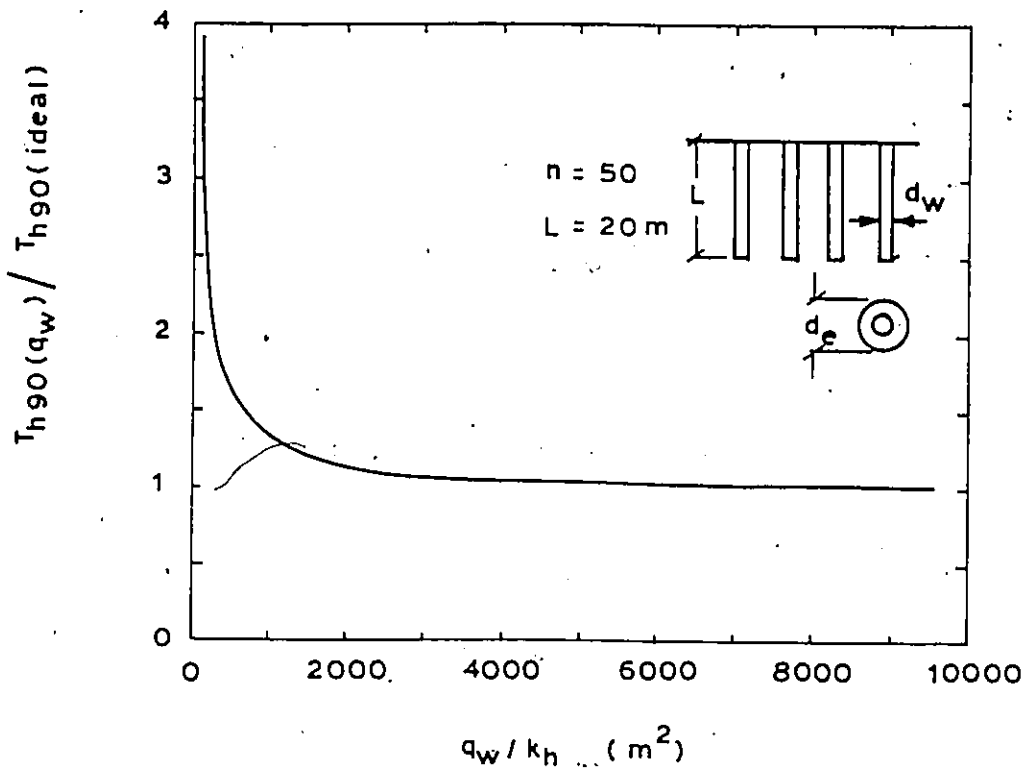


FIG. 2.5 : Influence of Finite Drain Permeability on the Consolidation Rate (after Jamiolkowski et.al, 1983)

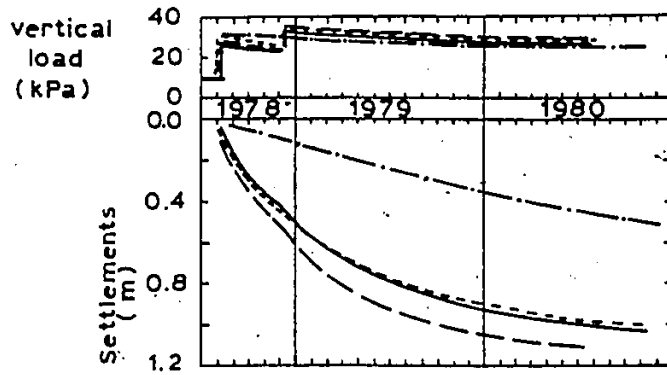


FIG. 2.9 : Measured Time/Settlement curves for the Whole Clay Layer. Drain Spacing 1.1 m (after Eriksson and Ekstrom, 1983)

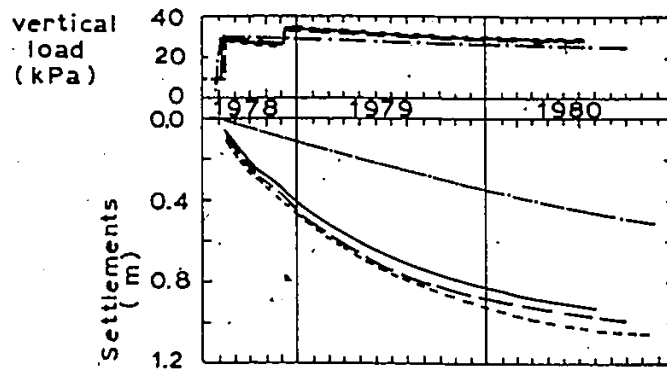


FIG. 2.10 : Measured Time/Settlement Curves for the Whole Clay Layer. Drain Spacing 1.4 m (after Eriksson and Ekstrom, 1983)

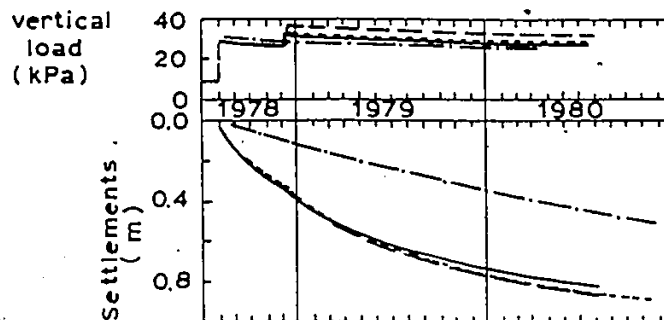


FIG. 2.11 : Measured Time/Settlement Curves for the whole Clay Layer. Drain Spacing 1.6 m (after Eriksson and Ekstrom, 1983)

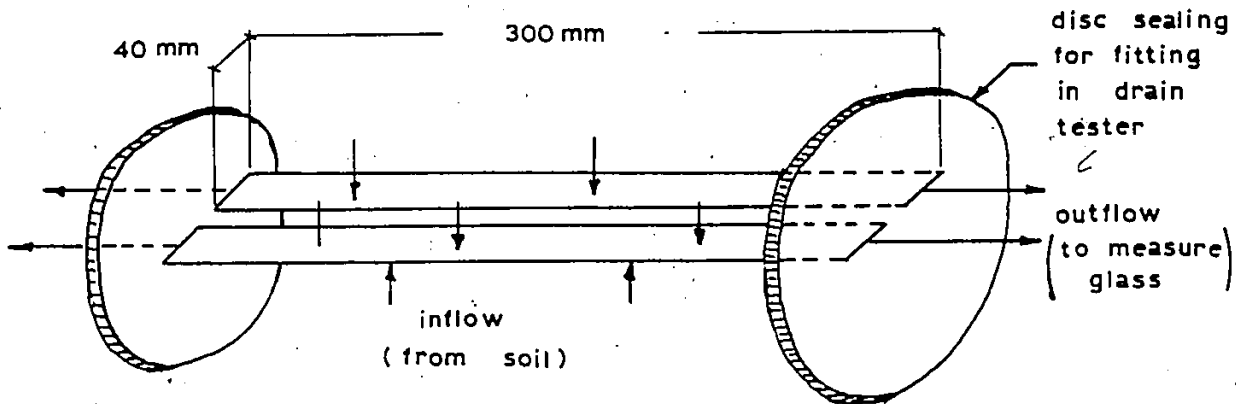


FIG. 2.6 : Mounting of Drain Sample (after Jansen and Den Hoedt, 1983)

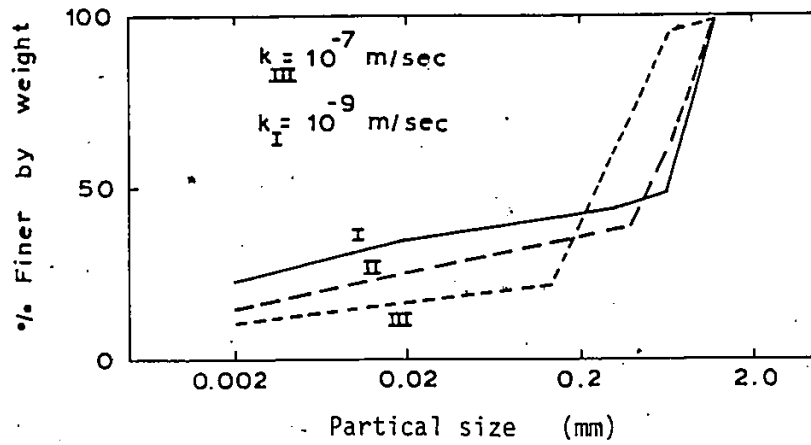


FIG. 2.7 : Grain Size Distribution Curves (after Jansen and Den Hoedt, 1983)

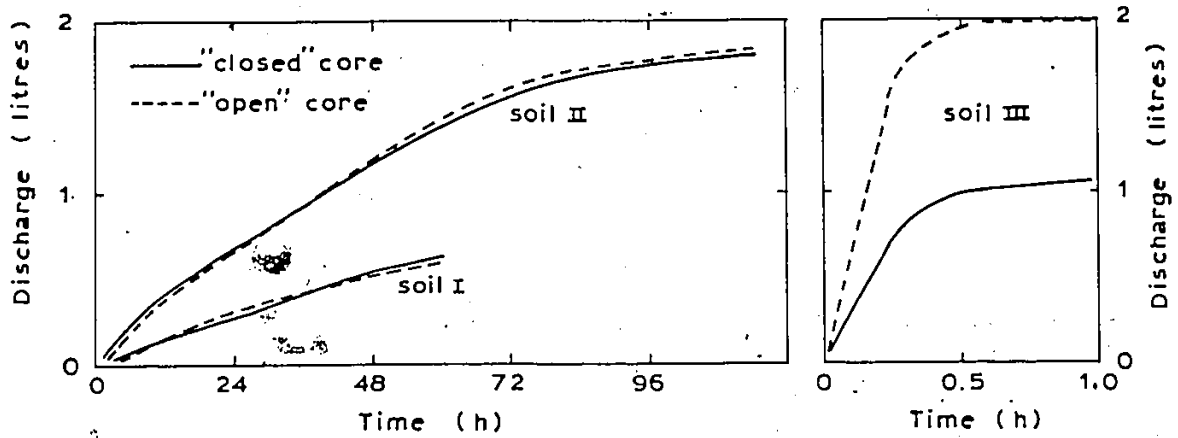
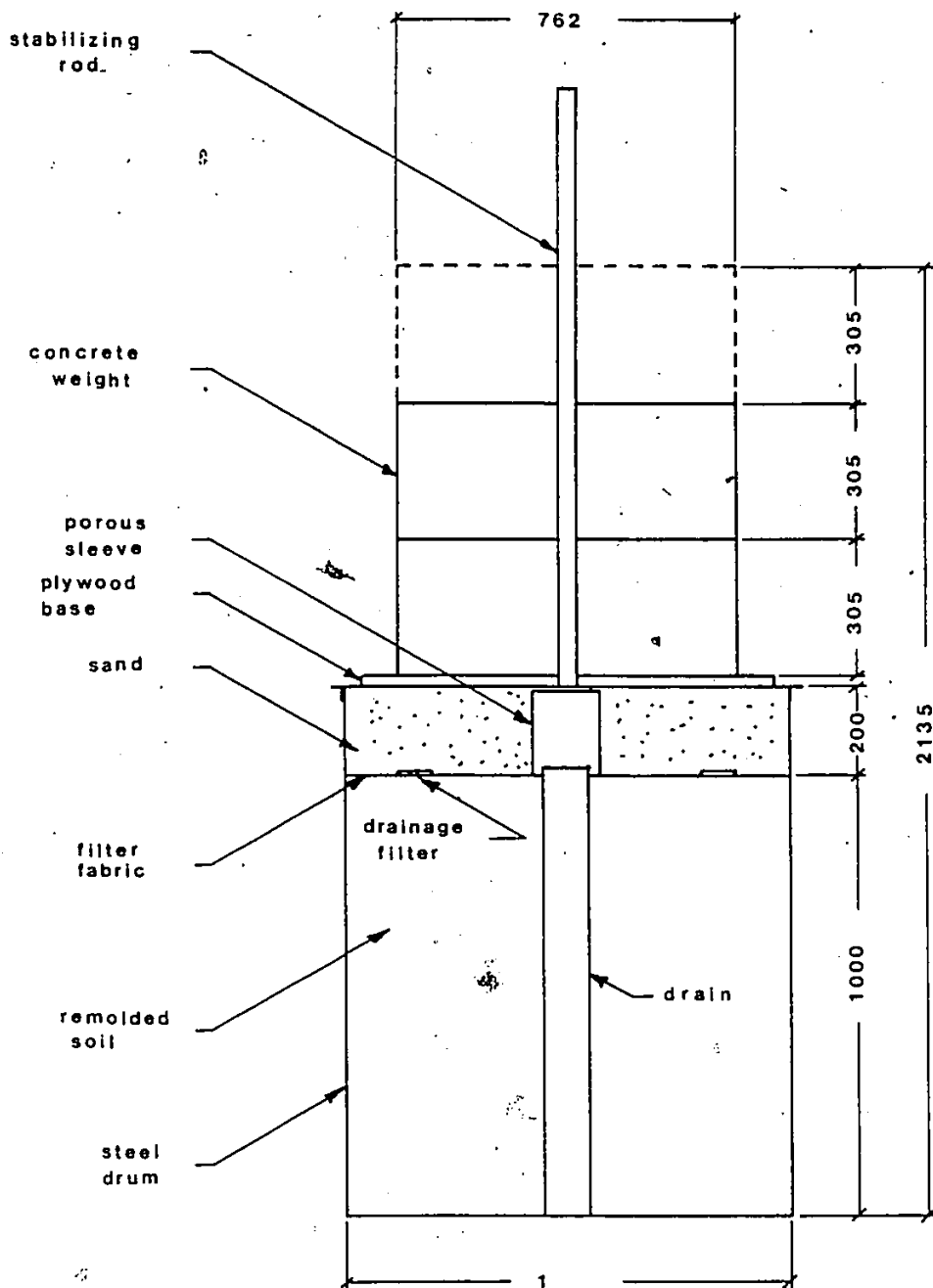


FIG. 2.8 : Results of Drain Test (after Jansen and Den Hoedt, 1983)



Note: all measurements are in mm

FIG. 3.1 : Test Set-Up (Series One and Two)

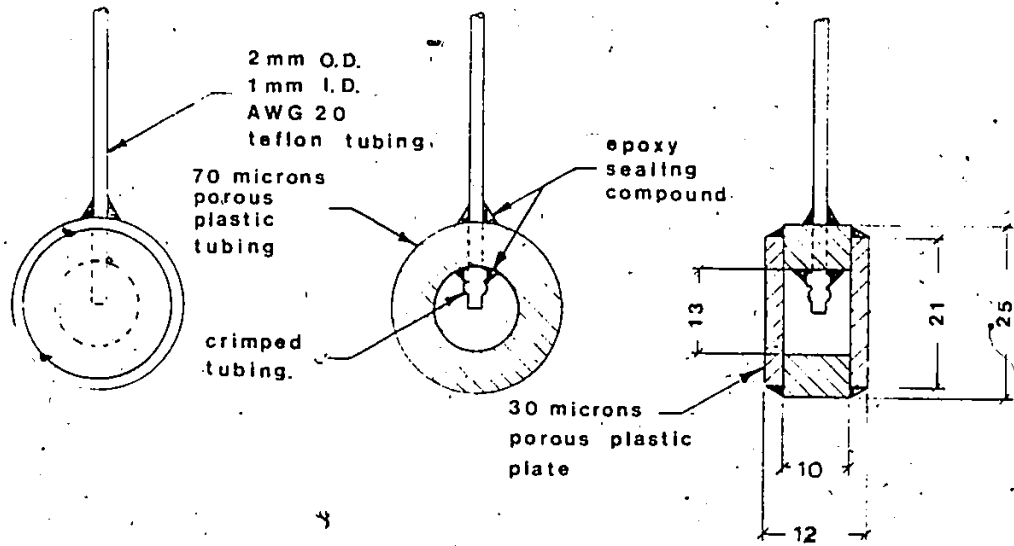


FIG. 3.2 : Piezometer

Note: all measurements are in mm

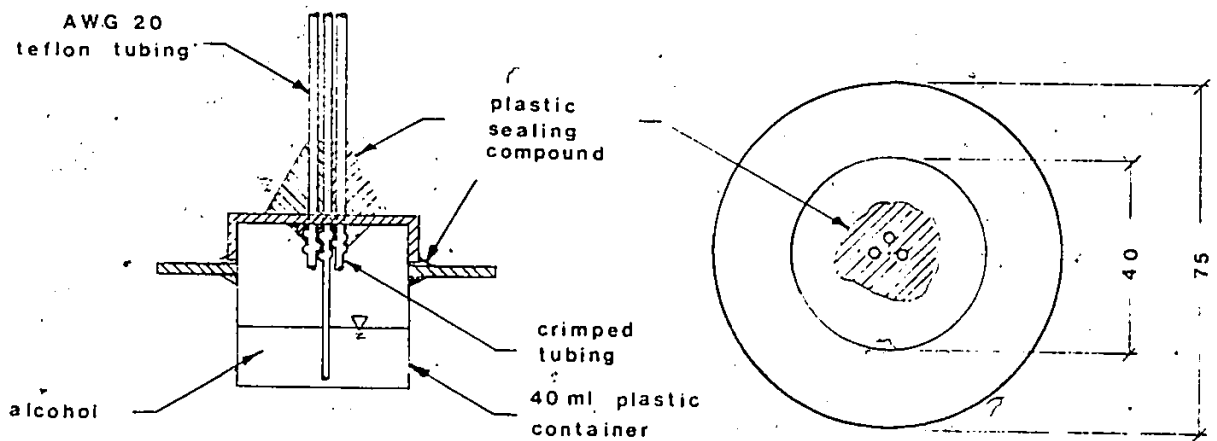


FIG. 3.3 : Settlement Gauge

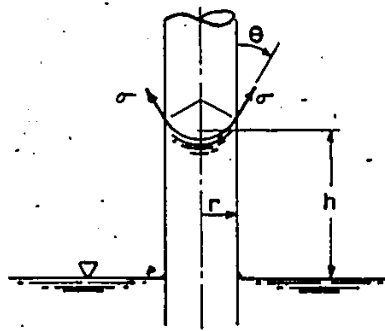


FIG. 3.4 : Capillary Rise in a Tube

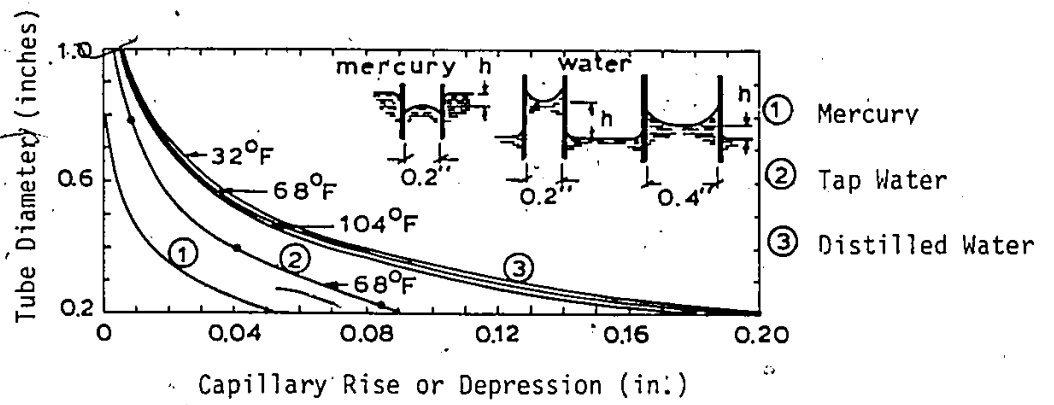


FIG. 3.5 : Capillarity in Clean Circular Glass Tubes (after Daugherty and Franzini, 1977)

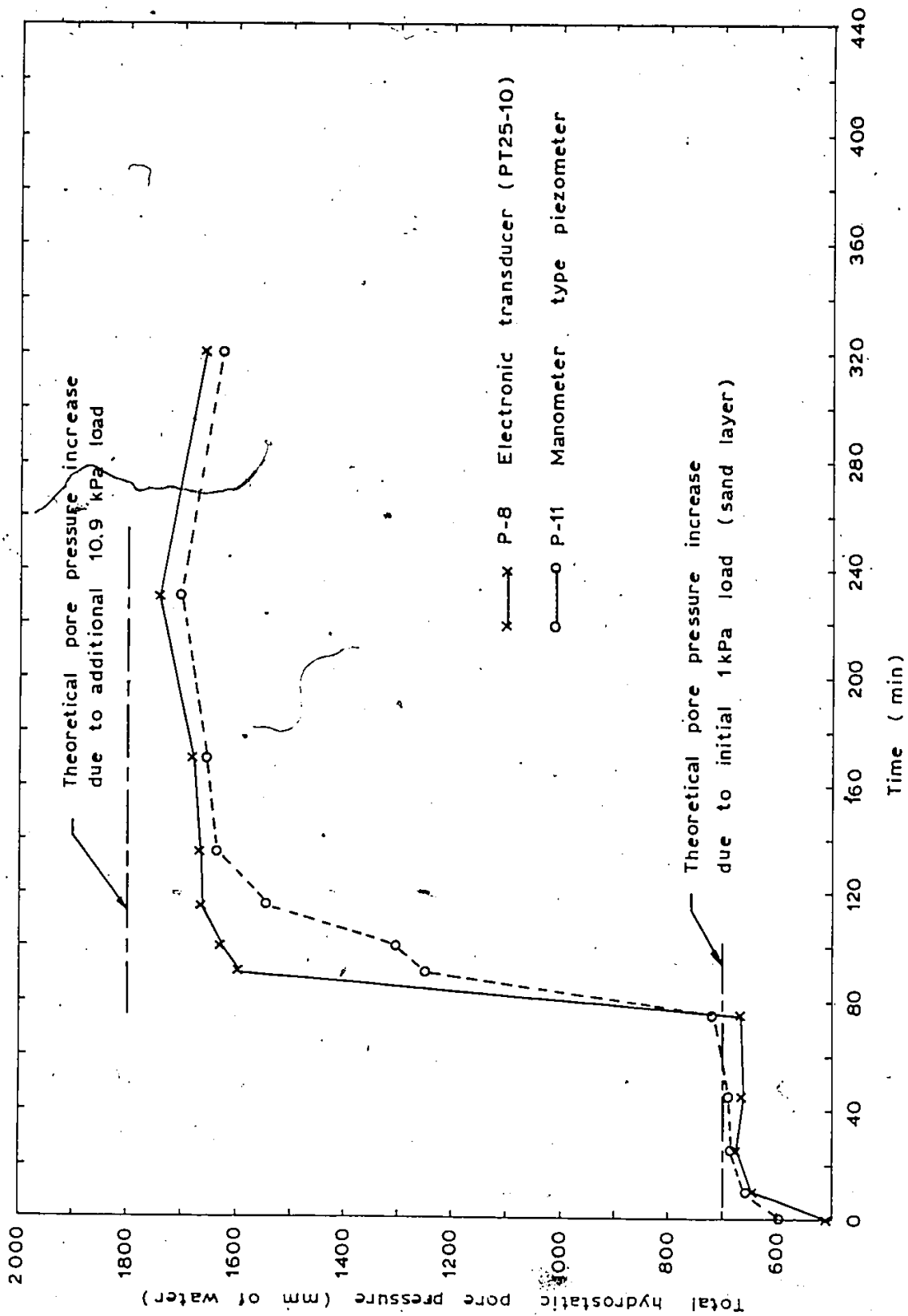


FIG. 3-6: Small Scale Model Test - Piezometer Response

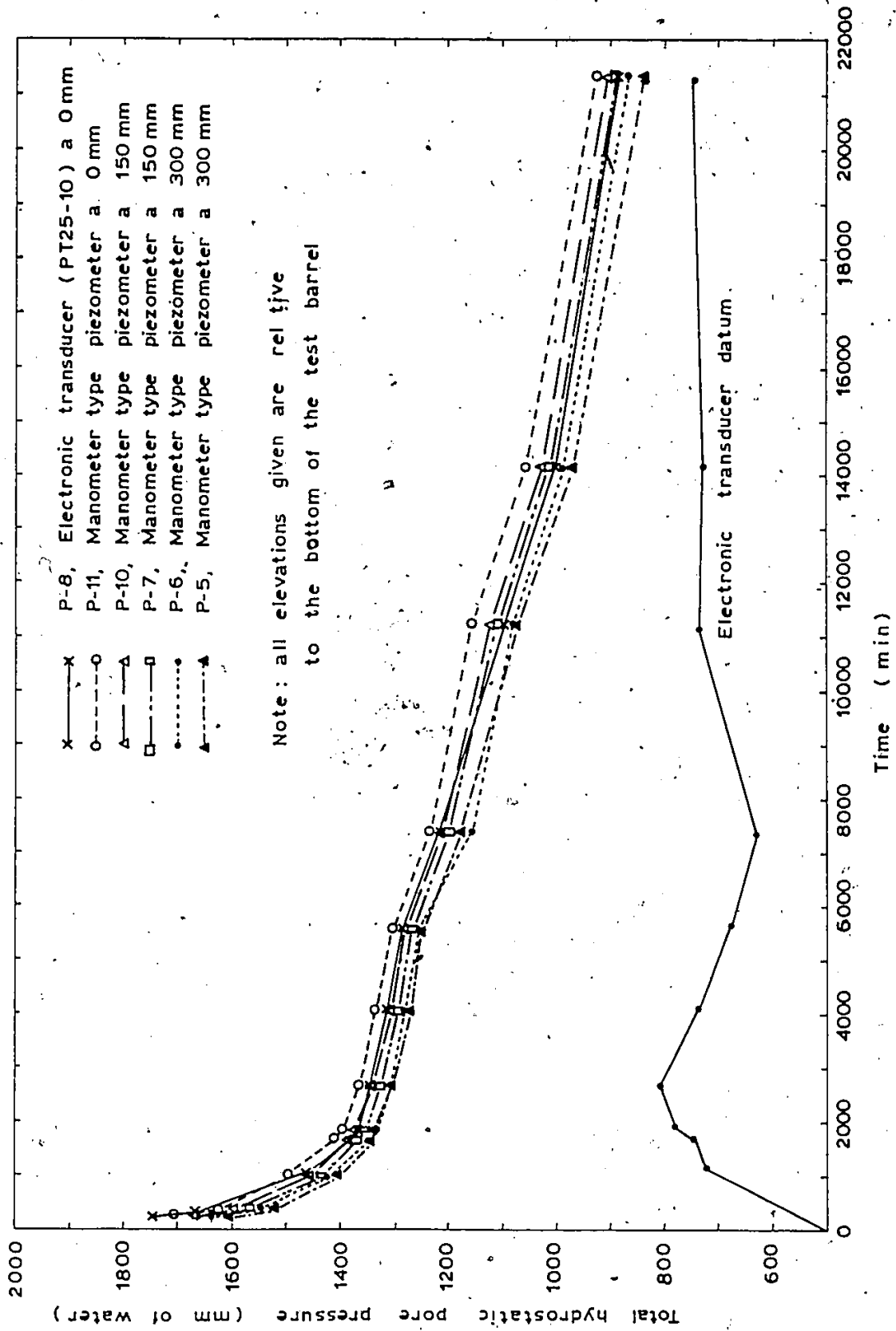


FIG. 3.7 : Small Scale Model Test - Piezometer Response

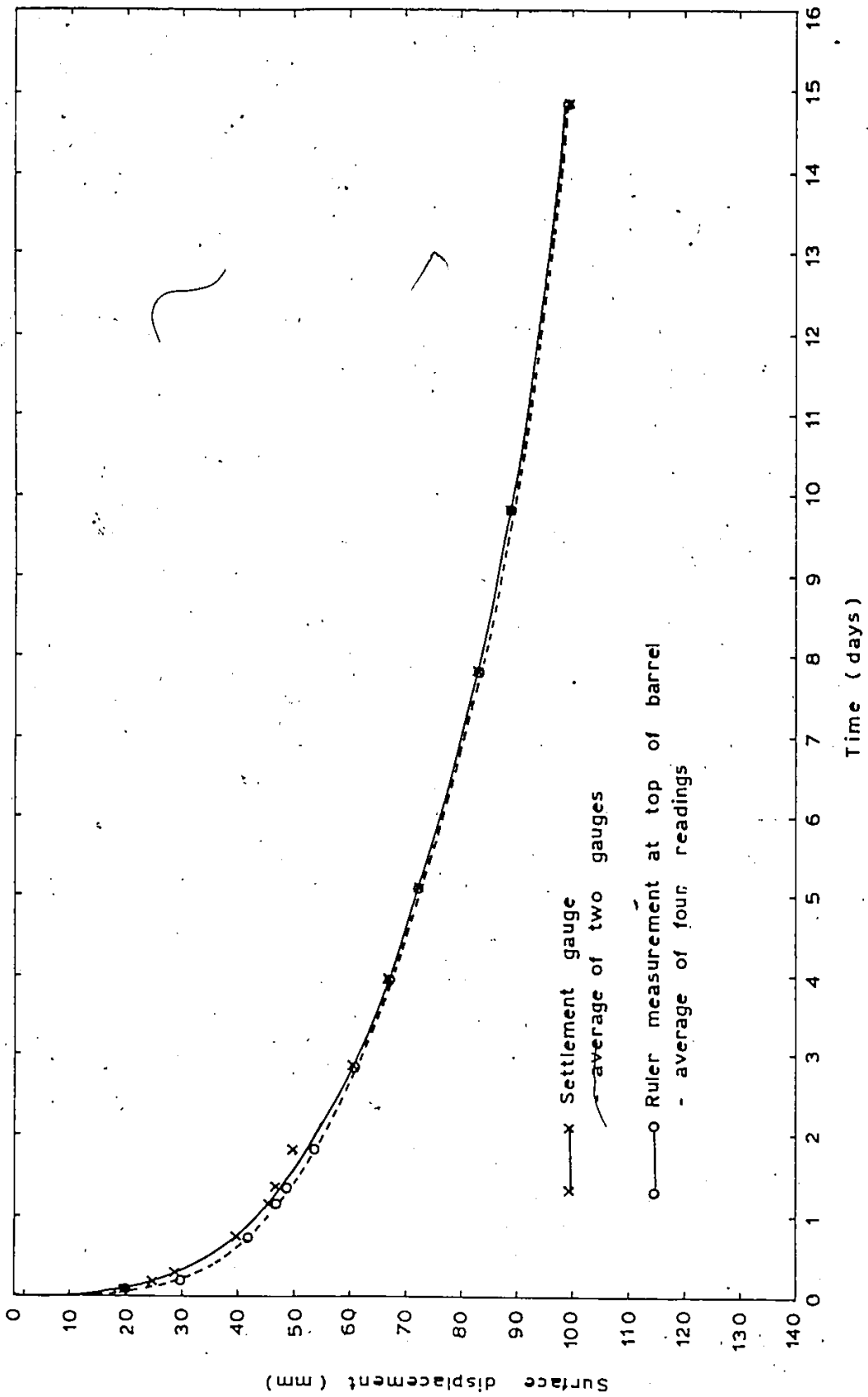


FIG. 3.8 : Small Scale Model Test - Settlement Gauge Performance

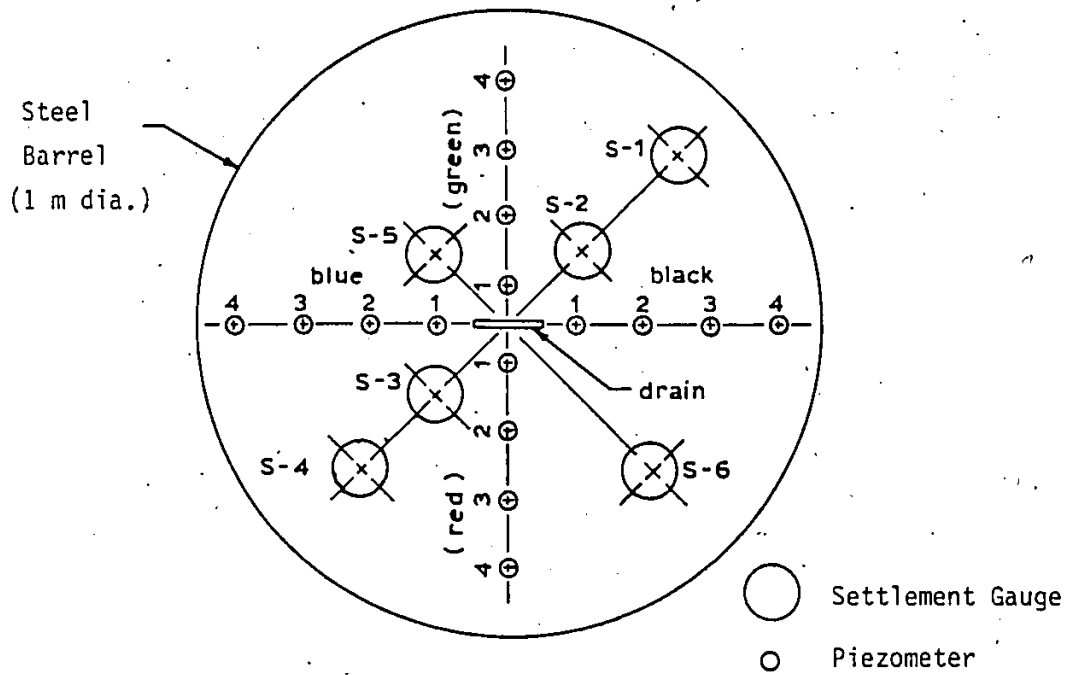
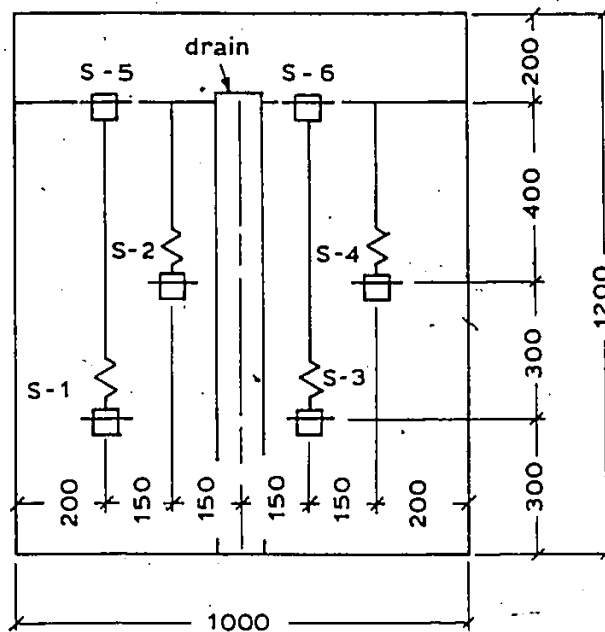


FIG. 3.9 : Plan View - Instrumentation



Note: All measurements are in mm.

FIG. 3.10 : Vertical View - Settlement Gauges

Note: All measurements are in mm.

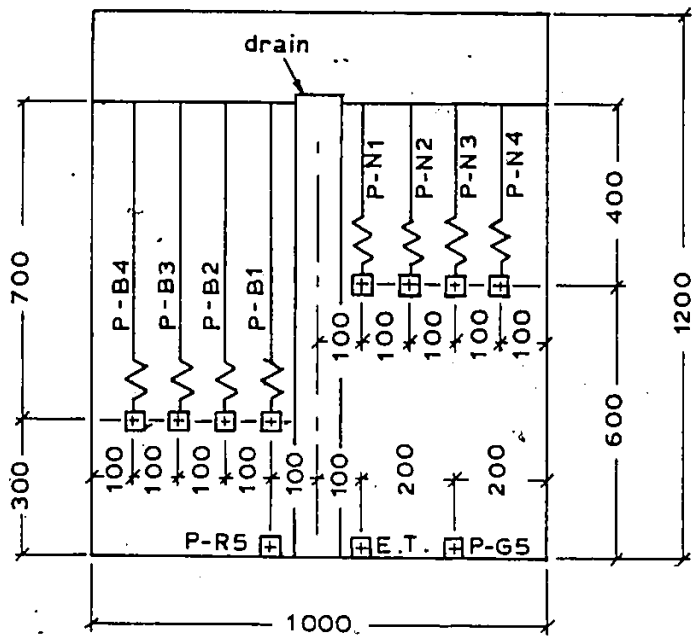


FIG. 3.11 : Vertical View - Piezometers

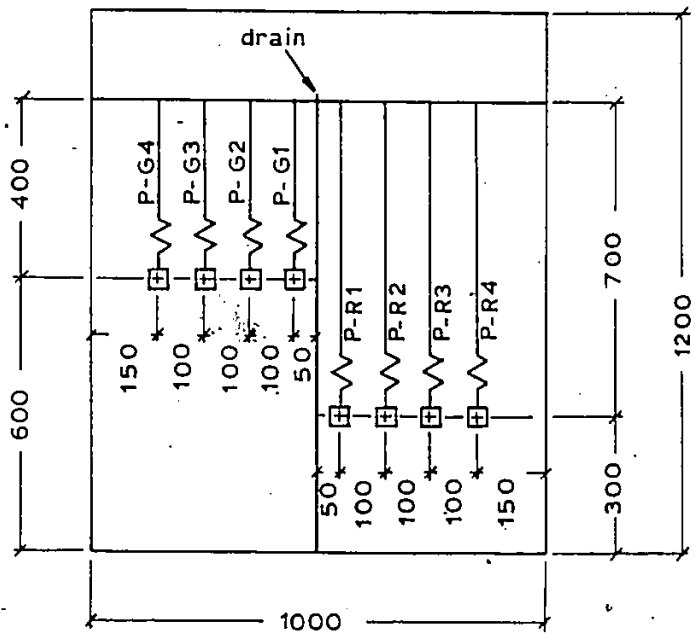


FIG. 3.12 : Vertical View - Piezometers

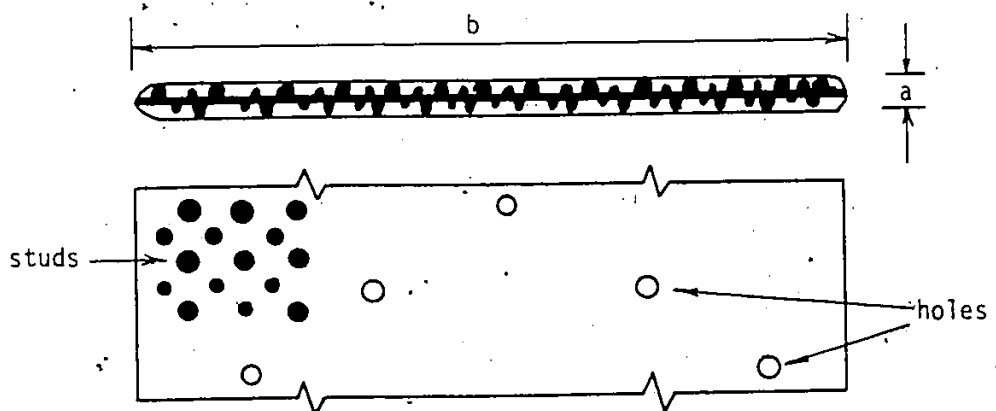


Figure 3.13
 Drain Characteristics - Studded Core

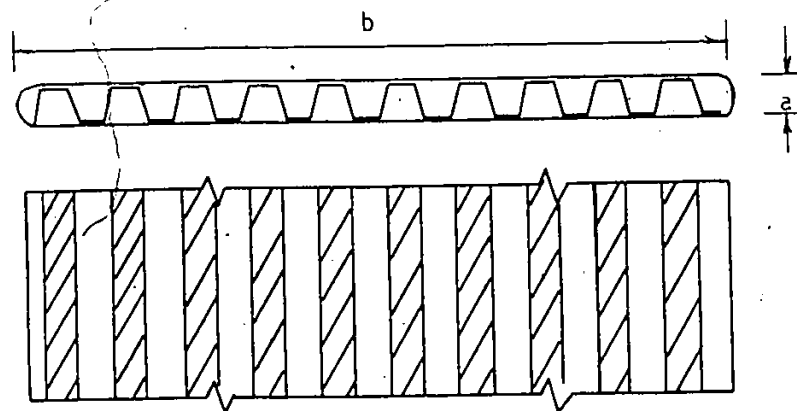


Figure 3.14
 Drain Characteristics - Grooved Core

MECHANICAL ANALYSIS

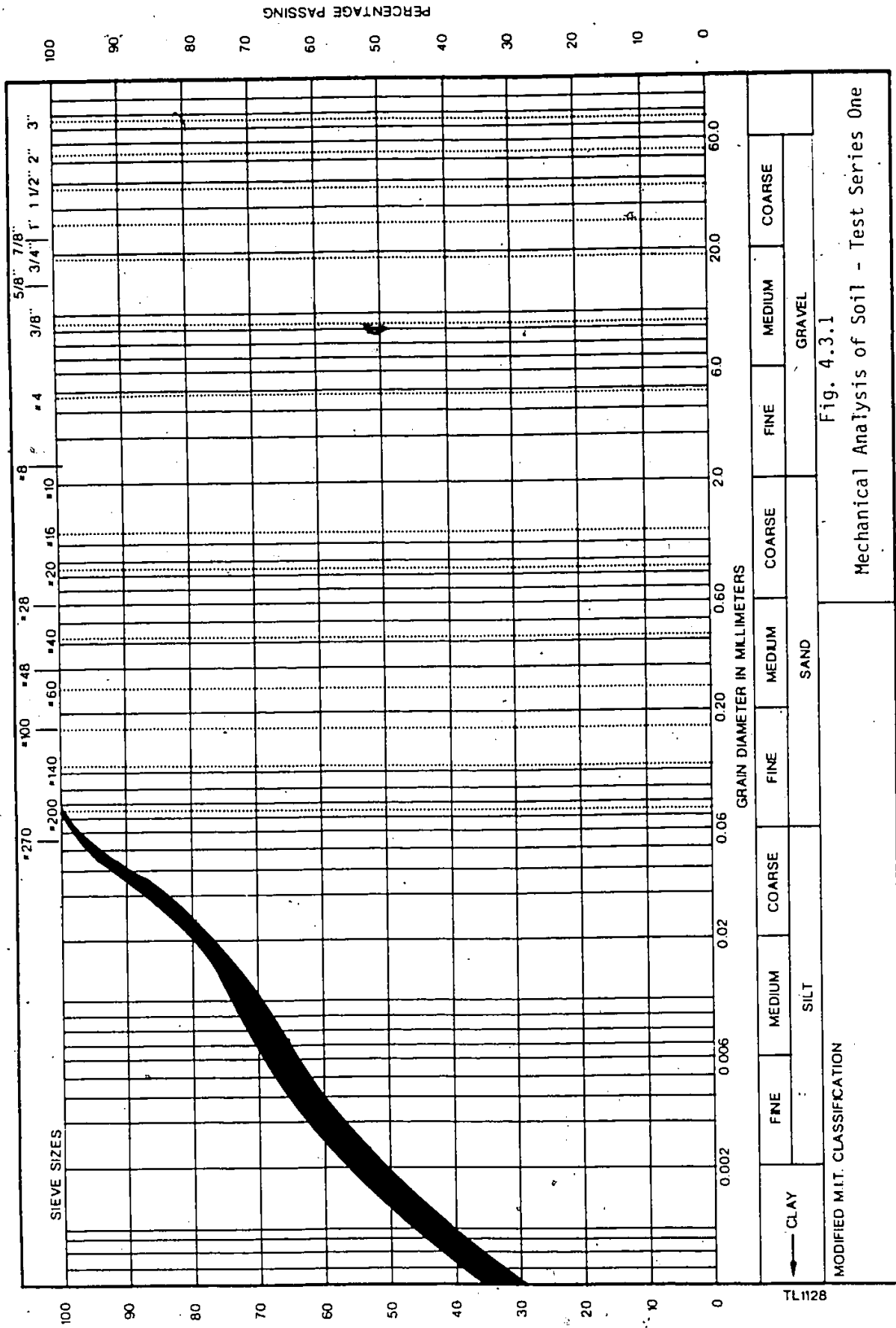
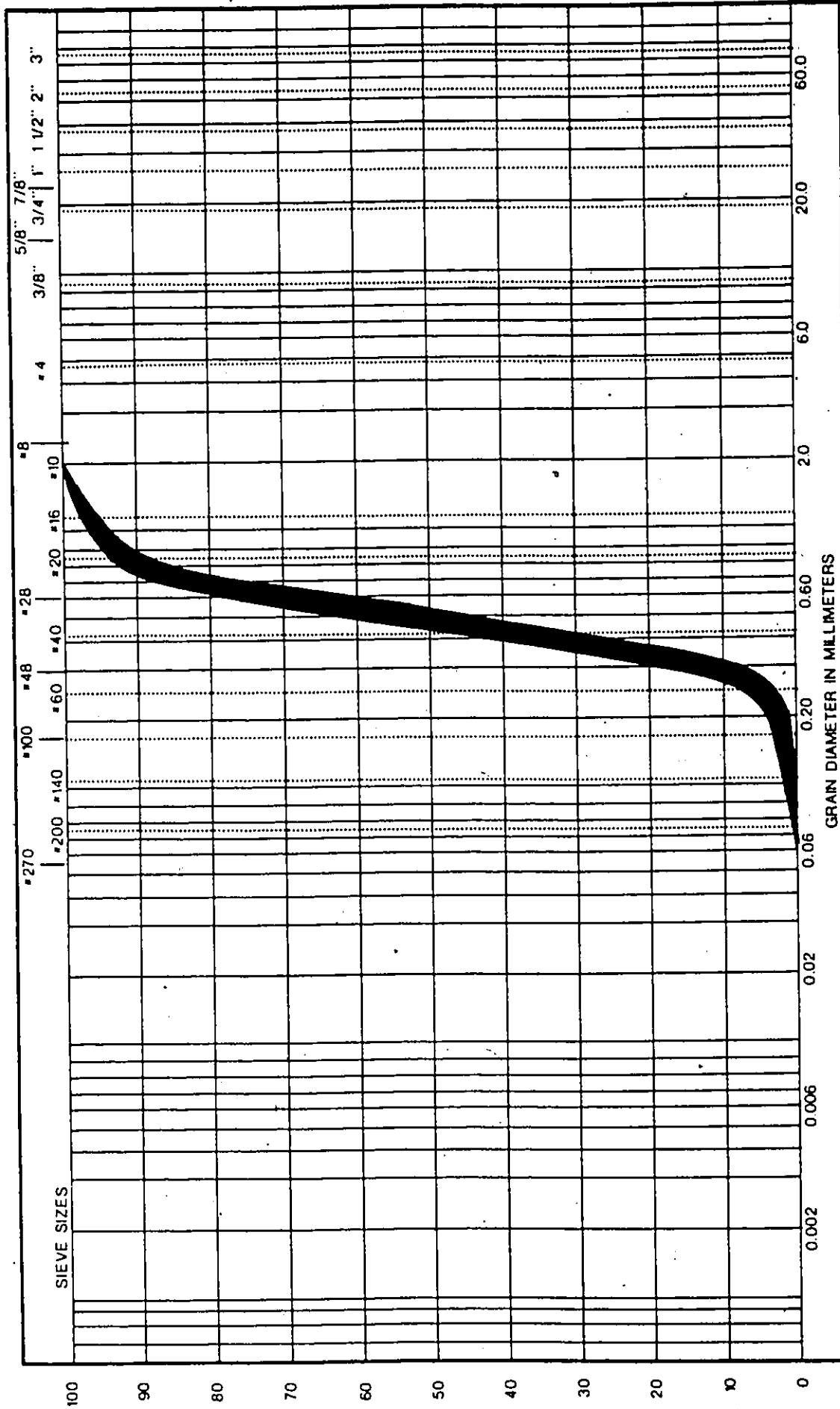


Fig. 4.3.1
Mechanical Analysis of Soil - Test Series One

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MECHANICAL ANALYSIS



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Fig. 4.4.1
Mechanical Analysis of Crushed Silica Sand -
Test Series Two

MODIFIED M.I.T. CLASSIFICATION

CLAY

FINE MEDIUM COARSE
SILT

FINE MEDIUM COARSE
SAND

FINE MEDIUM COARSE
GRAVEL

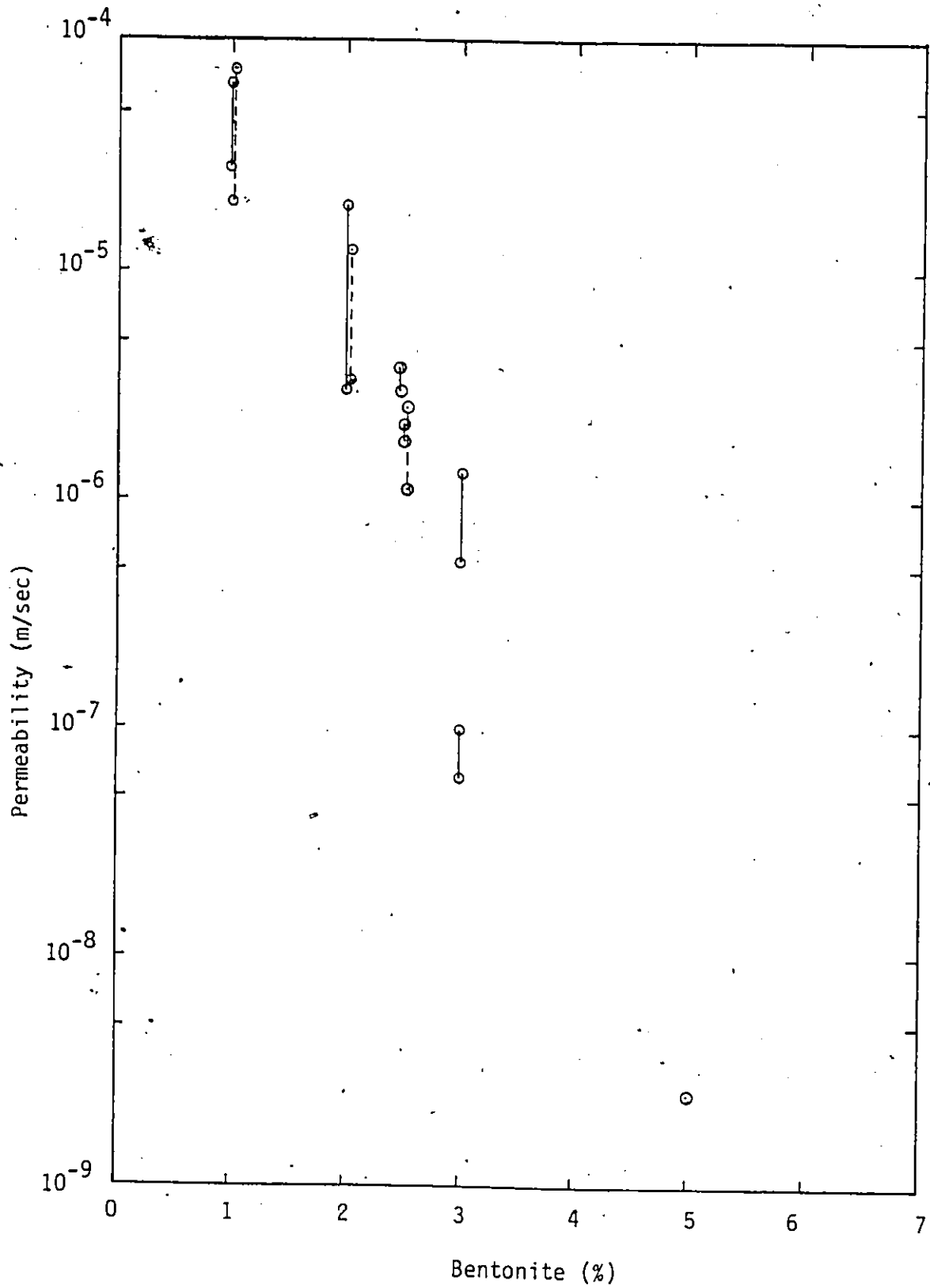
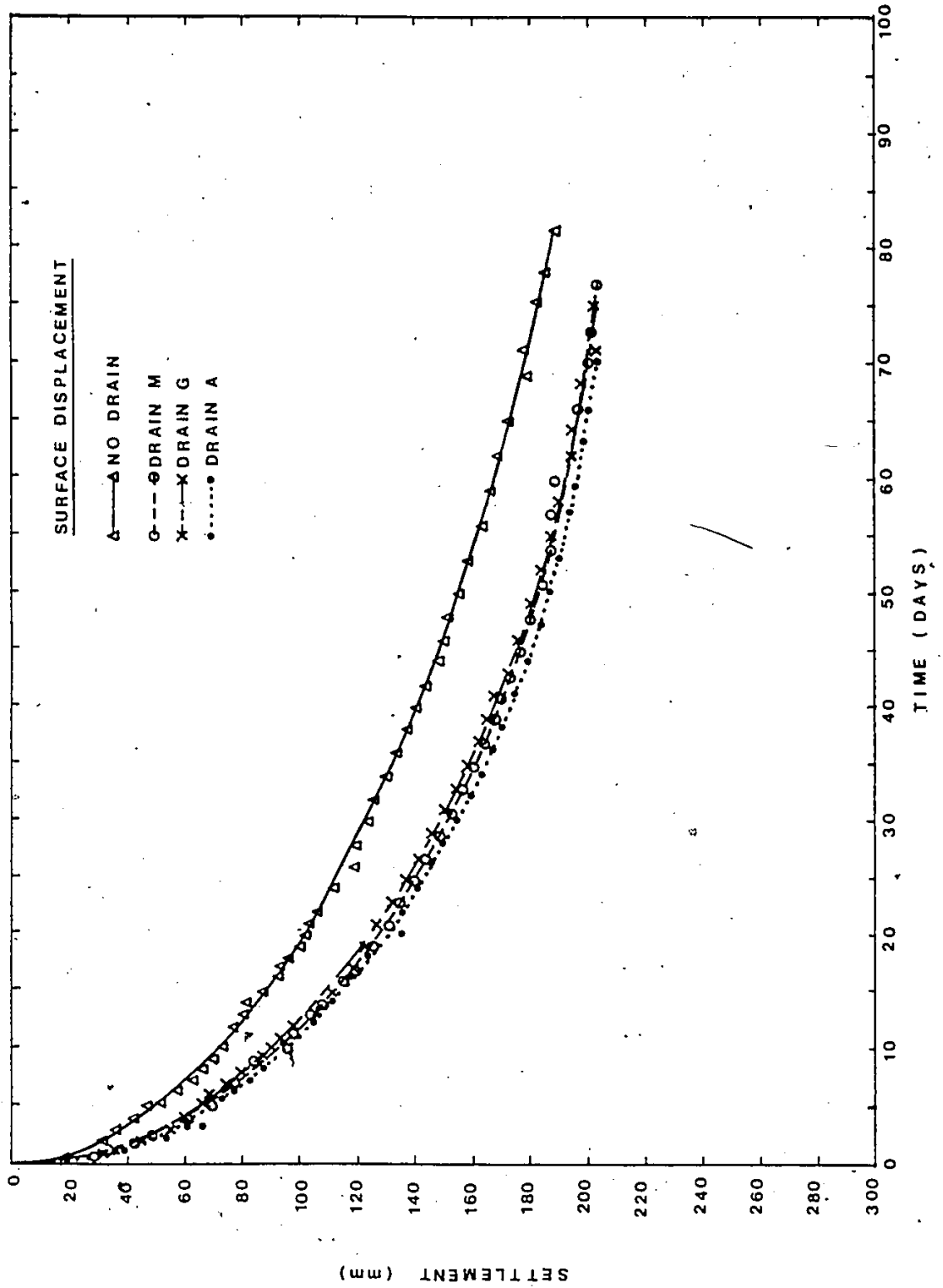


FIG. 4.4.2 : Permeability of Sand-Bentonite Mixture - Determined from Falling Head Permeability Tests

FIG. 6.1.1 : Settlement Curves - Test Series One



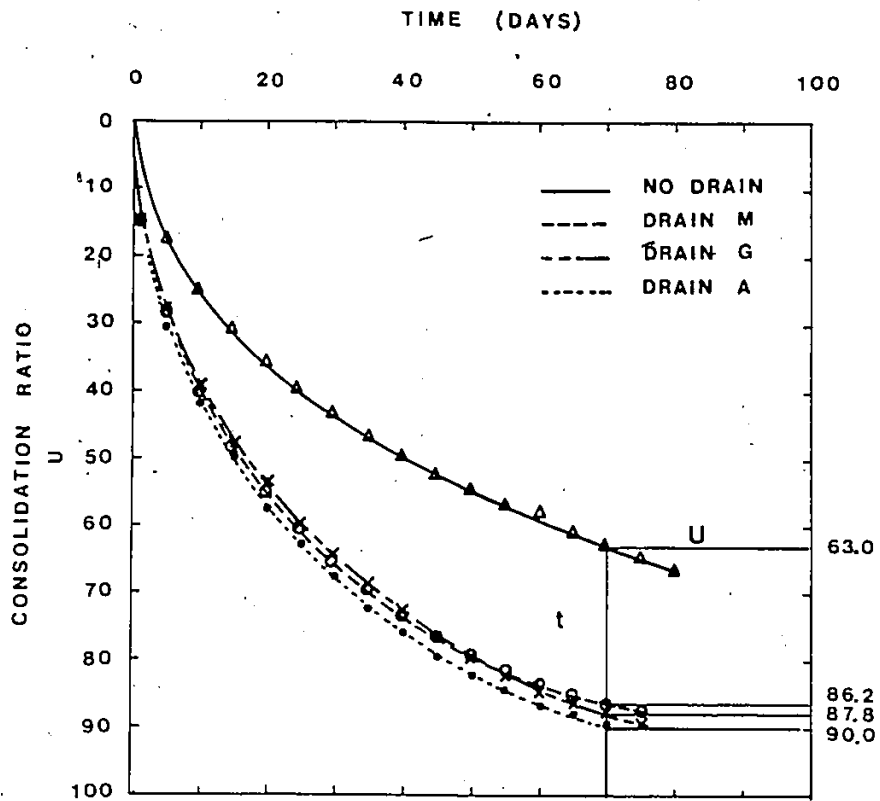


FIG. 5.1.2 : Consolidation Ratio versus Time - Test Series One (Settlements)

FIG. 6.1.3 : Excess Pore Pressure Dissipation - Test Series One
(Vertical Isochrones)

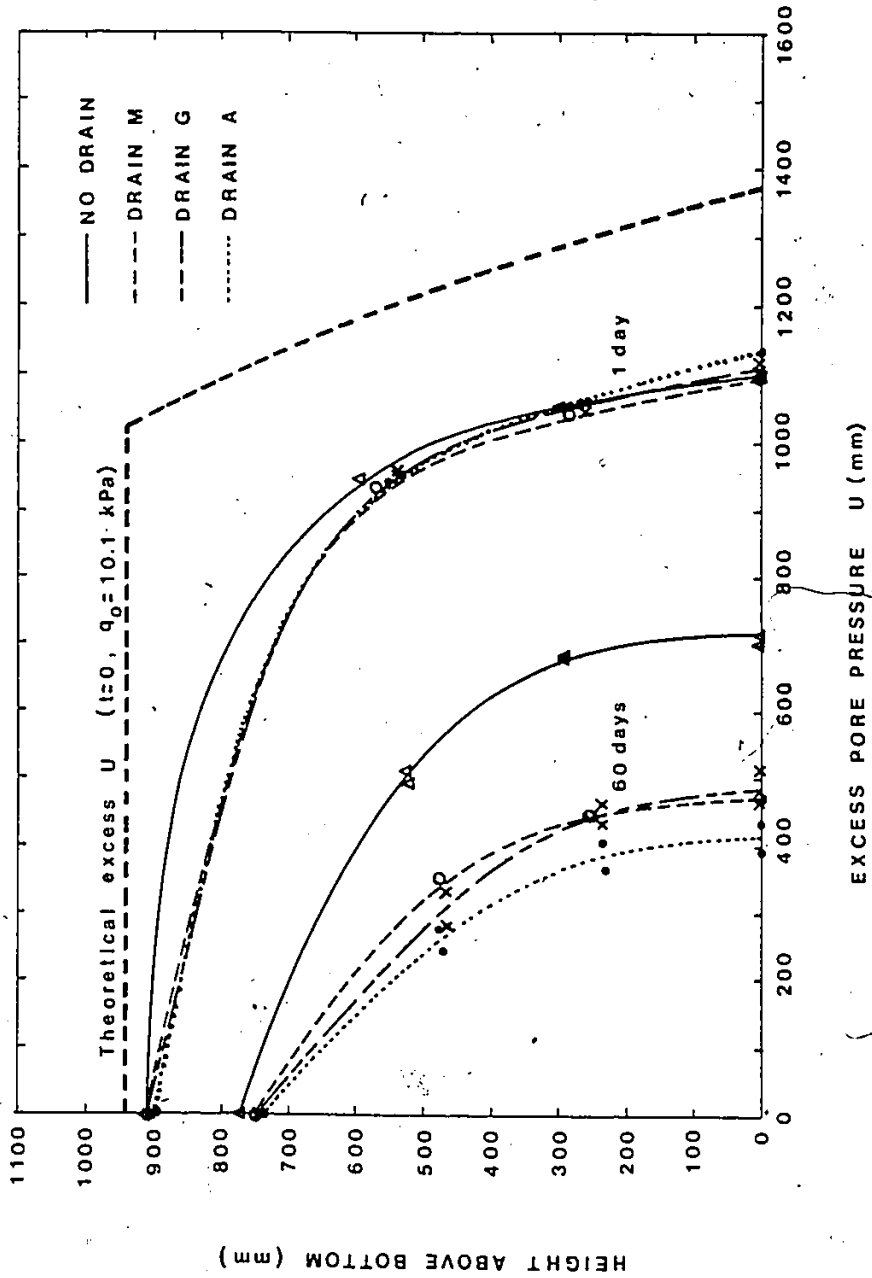
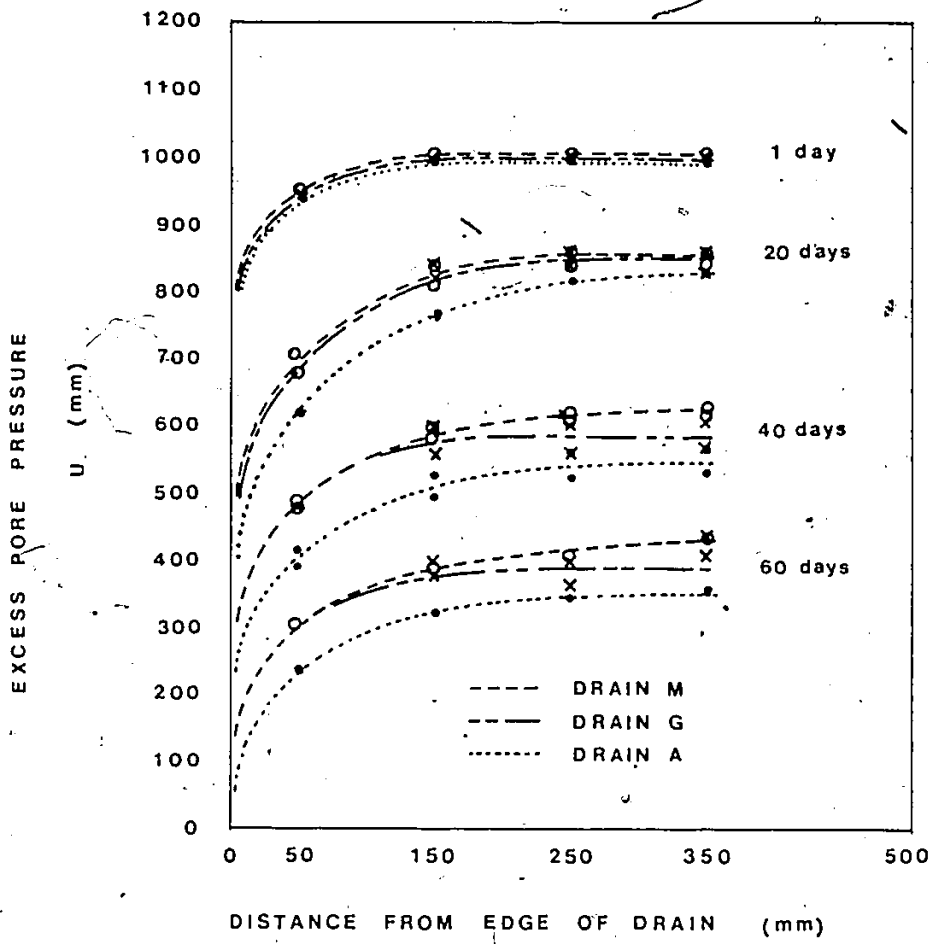


FIG. 6.1.4 : Excess Pore Pressure Dissipation - Test Series One
 (Horizontal Isochrones ; Average - Levels 2 & 3)



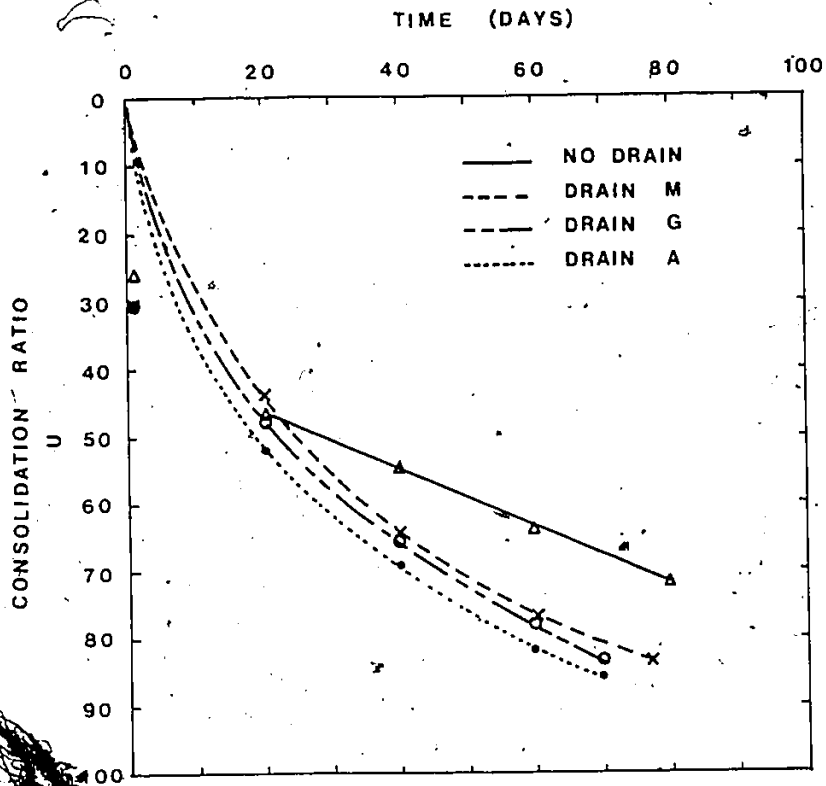
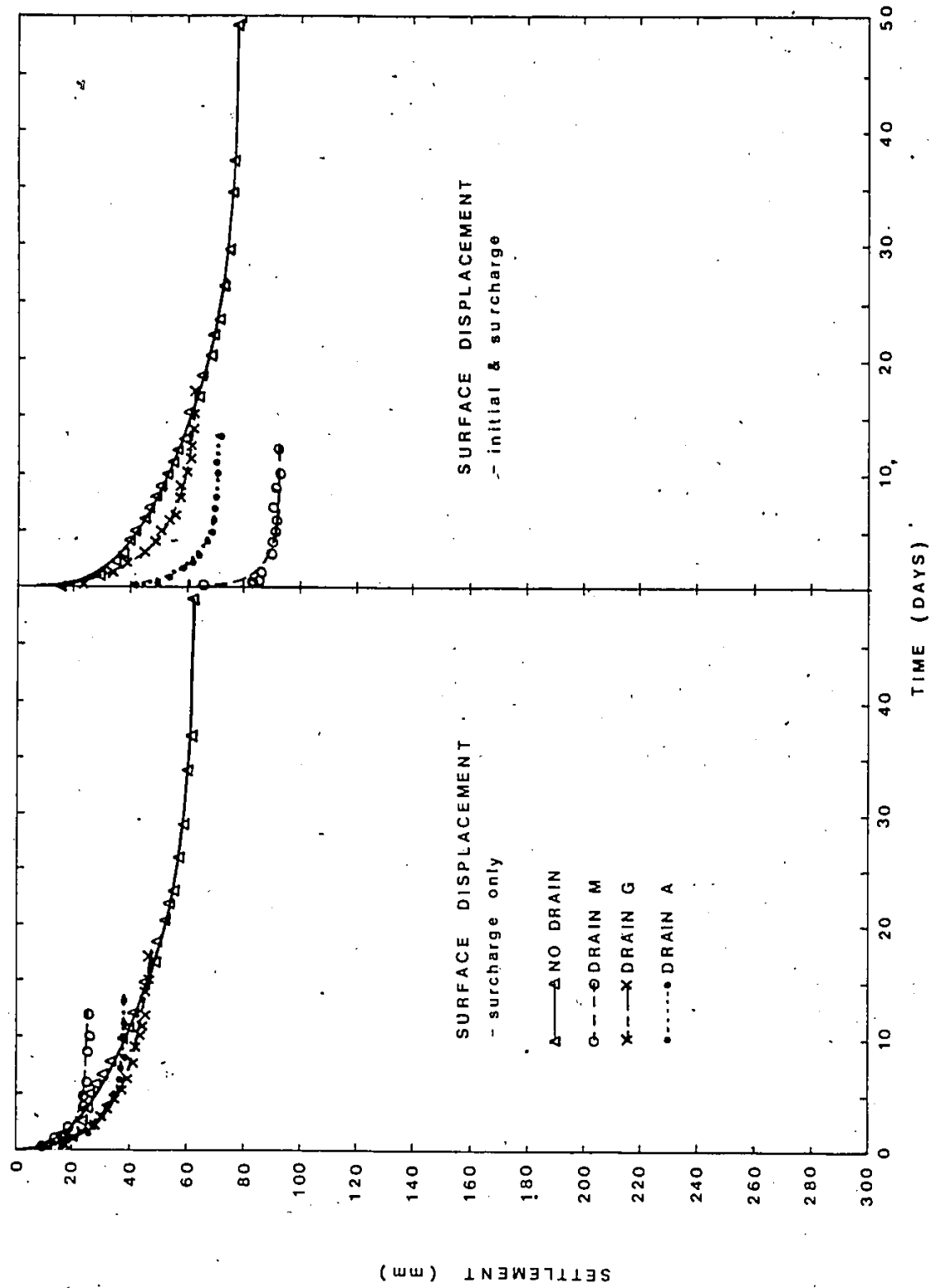


FIG. 6.1.5 : Consolidation Ratio versus Time - Test Series One
(Excess Pore Pressure Dissipation)

FIG. 6.2.1 : Settlement Curves - Test Series Two



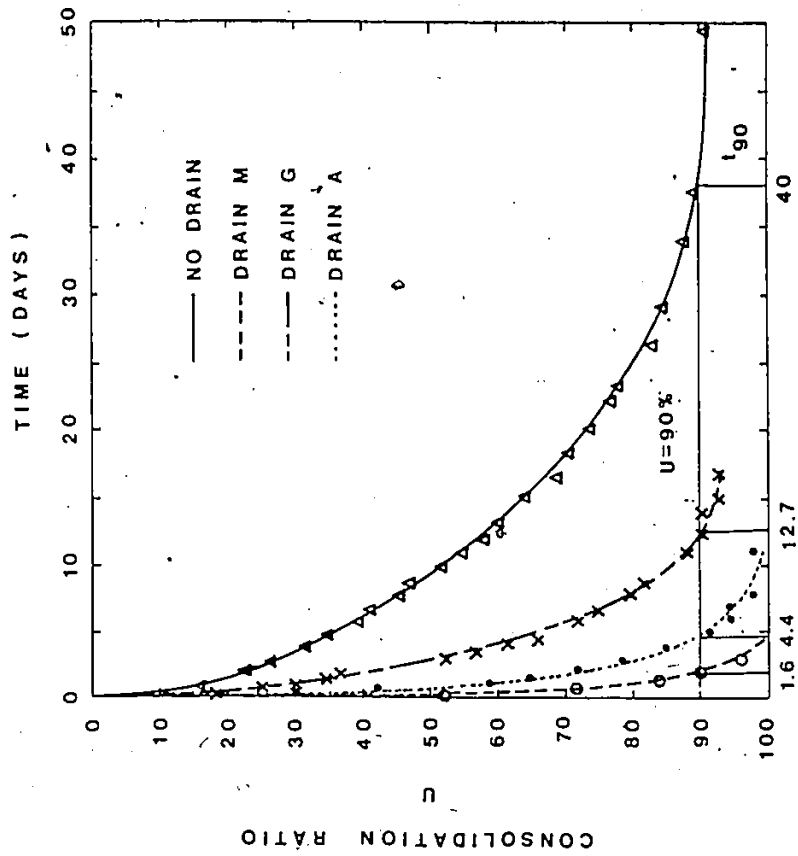


FIG. 6.2.2 : Consolidation Ratio versus Time - Test Series Two (Settlements)

FIG. 6.2.3 : Excess Pore Pressure Dissipation - Test Series Two
(Vertical Isochrones)

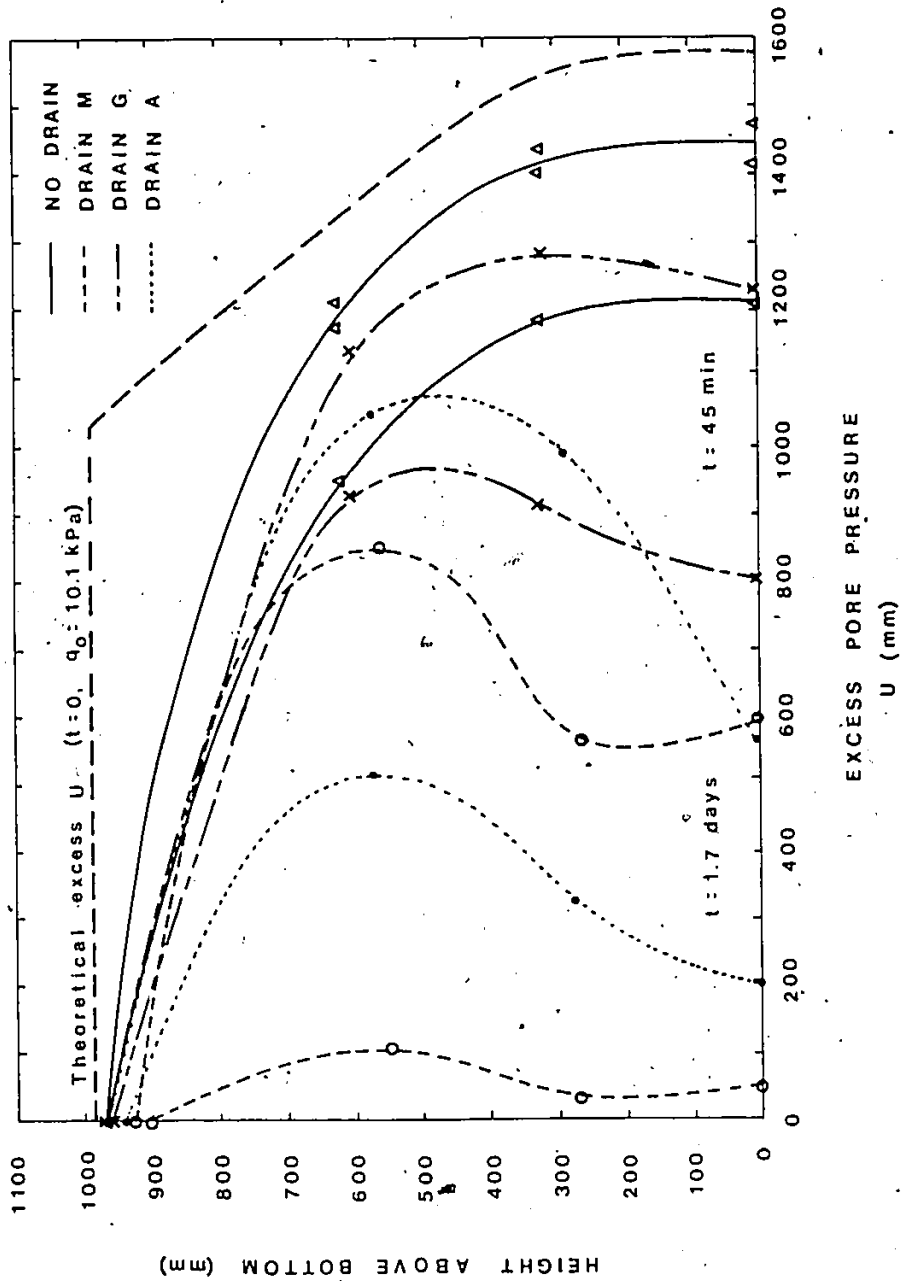
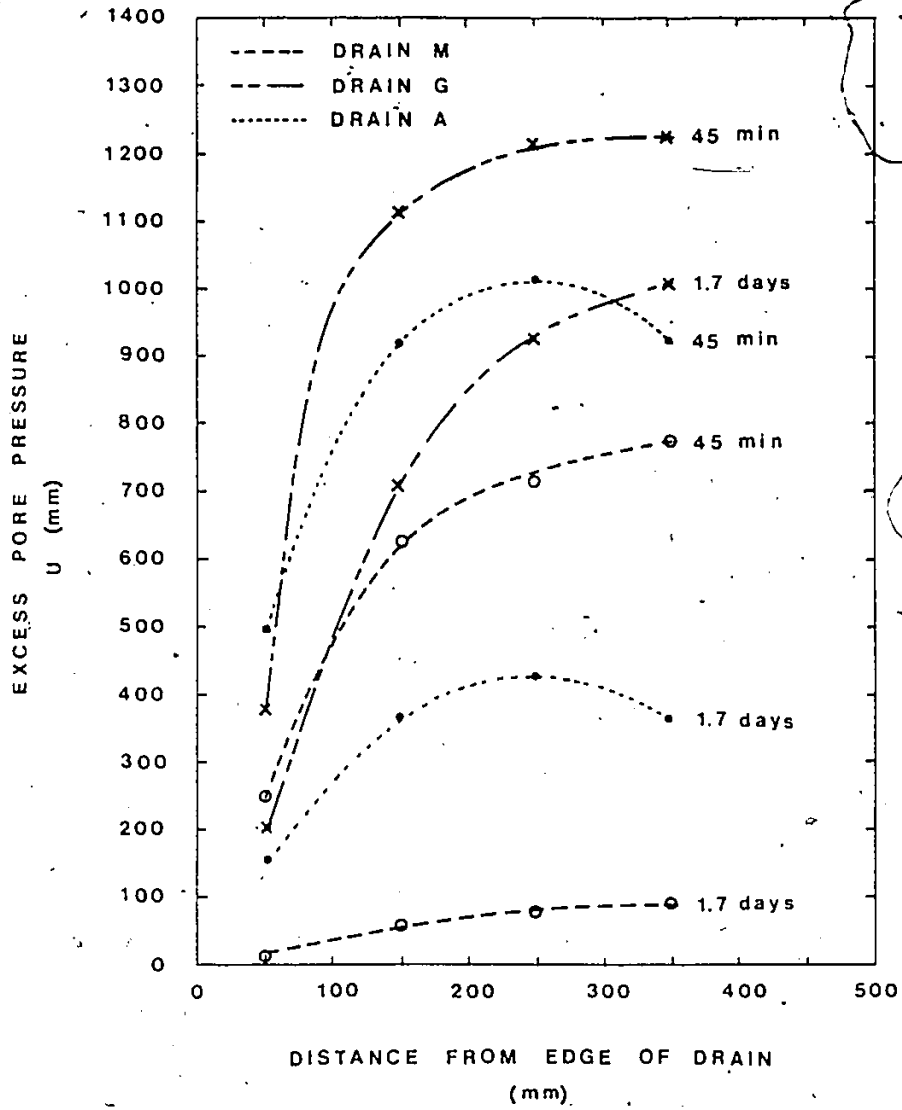


FIG. 6.2.4 : Excess Pore Pressure Dissipation - Test Series Two
 (Horizontal Isochrones ; Average - Levels 2 & 3)



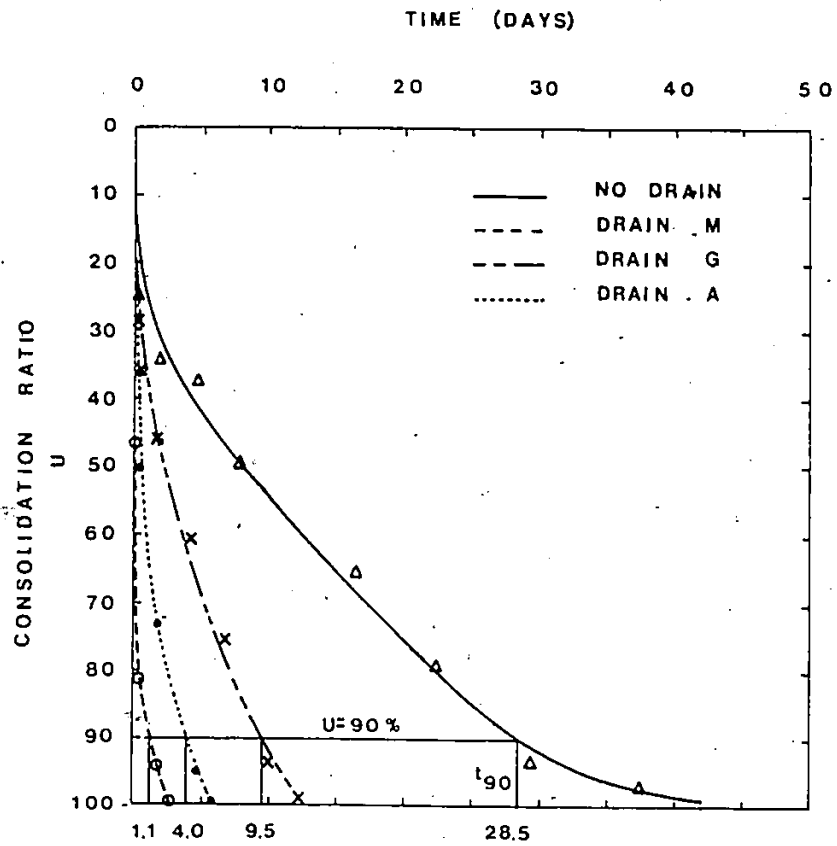


FIG. 6.2.5 : Consolidation Ratio versus Time - Test Series Two
(Excess Pore Pressure Dissipation)

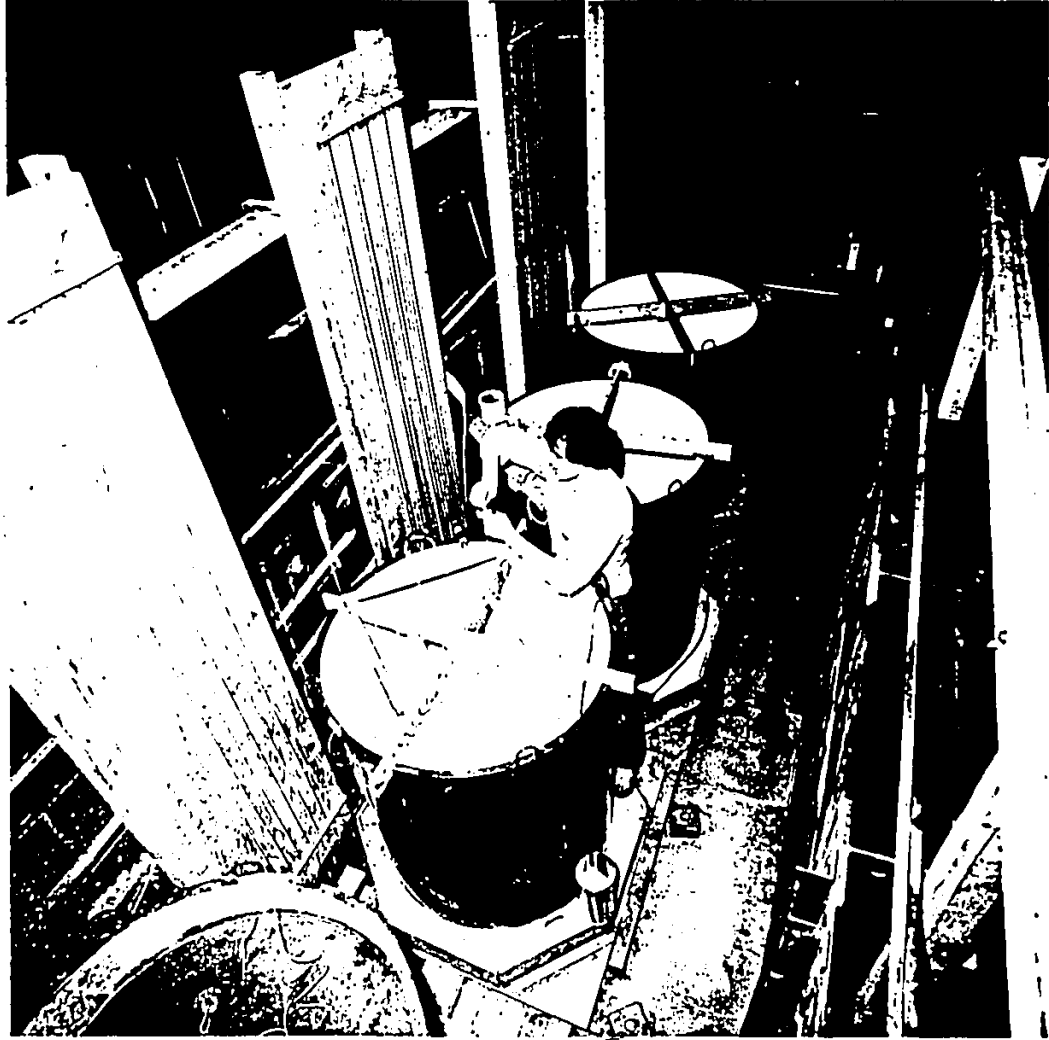


Plate I : Steel Cylinders and Set-Up for Test Series
One and Two



Plate 2 : Piezometers



Plate 3 : Instrumental Set-Up



Plate 4 : Set-Up for Test Series Three with Surcharge

APPENDIX A

TABLE A1.1

NO DRAIN

TIME (DAYS)	SOIL SURFACE ELEVATION LEVEL 1 (MM)	EXCESS PORE PRESSURES U (MM)					
		LEVEL 2		LEVEL 3		LEVEL 4	
		P-G1	P-G2	P-R1	P-R2	P-R3	P-G3
0.00	936	173	168	301	282	343	341
0.02	931	974	1003	1082	1085	1128	1117
0.04	926	1000	1010	1122	1105	1167	1157
0.06	926	987	989	1114	1097	1154	1150
0.08	929	940	950	1059	1041	1113	1099
0.10	926	905	889	1010	993	1068	1060
0.12	926	942	944	1047	1029	1096	1090
0.14	925	896	889	1004	994	1066	1061
0.16	921	929	914	1036	1024	1089	1075
0.20	920	933	932	1043	1025	1090	1084
0.25	918	934	933	1044	1026	1090	1084
0.29	917	934	928	1044	1027	1092	981
0.33	916	932	935	1037	1022	1089	1083
0.70	908	978	977	1087	1071	1133	1125
0.75	908	955	948	1066	1054	1116	1106
0.83	909	948	946	1057	1043	1109	1096
0.91	905	952	947	1058	1048	1109	1099
1.04	904	955	952	1064	1059	1116	1102
1.35	907	951	948	1056	1046	1107	1096
1.72	906	939	934	1044	1031	1099	1086
1.89	905	934	930	1038	1030	1091	1082
2.76	899	907	899	1014	997	1060	1053
3.80	893	886	880	991	975	1037	1029
4.76	889	874	877	976	960	1018	1015
5.20	884	958	958	1065	1045	1105	1099
5.73	880	956	956	1058	1043	1098	1093
6.07	878	952	954	1054	1037	1094	1086
6.77	875	954	957	1058	1040	1095	1090
7.03	873	963	968	1068	1051	1102	1099
7.74	871	946	946	1053	1033	1086	1080
7.99	869	940	938	1041	1023	1079	1071
8.81	866	935	932	1035	1019	1071	1065
9.89	862	928	927	1031	1012	1063	1057
10.78	859	922	925	1029	1010	1058	1053
11.84	859	914	915	1021	1002	1049	1044
12.93	855	919	917	1021	1007	1050	1047
13.85	854	896	896	990	988	1032	1025
14.79	849	899	901	1007	990	1032	1028
16.11	843	877	879	996	979	1017	1013
16.85	842	870	874	991	971	1009	1006
17.80	840	851	853	977	953	993	988
18.76	836	844	843	969	945	982	980

TABLE A1.1

NO DRAIN

TIME (DAYS)	SOIL SURFACE ELEVATION LEVEL 1 (MM)	EXCESS PORE PRESSURES U (MM)					
		LEVEL 2		LEVEL 3		LEVEL 4	
		P-G1	P-G2	P-R1	P-R2	P-R3	P-G3
19.80	834	837	835	960	938	975	971
20.79	833	835	834	959	938	973	970
21.80	830	827	823	954	933	974	969
23.89	823	815	812	949	928	963	960
25.77	817	773	769	914	893	931	927
27.76	817	779	773	925	904	938	936
29.77	813	740	736	899	875	909	906
31.77	810	707	703	885	845	879	876
33.75	806	684	681	870	828	861	861
35.74	802	701	699	876	853	880	880
37.87	799	665	666	860	822	852	851
39.75	795	643	643	844	801	832	842
41.79	792	636	639	841	801	829	834
43.91	788	633	639	842	801	833	836
45.75	786	628	627	840	796	829	833
47.76	785	585	588	792	754	792	790
49.83	781	568	570	777	740	776	778
52.72	777	538	539	750	751	749	749
55.73	773	531	528	741	699	737	738
58.86	770	515	505	713	675	716	716
61.75	767	500	494	699	660	702	703
64.82	764	488	482	684	645	686	687
68.78	757	475	469	667	625	668	666
71.00	758	425	424	637	587	630	630
75.04	754	390	408	611	559	596	601
77.77	751	384	389	576	536	581	586
81.75	747	370	380	536	506	541	542

TABLE A1.2

DRAIN M

TIME (DAYS) EXCESS PORE PRESSURES U (MM)

LEVEL 2

P-G1 P-G2 P-G3 P-G4

TIME (DAYS)	P-G1	P-G2	P-G3	P-G4
0.00	133	187	170	182
0.02	930	863	914	907
0.04	944	889	948	934
0.06	922	901	909	924
0.08	847	904	855	860
0.13	831	905	851	802
0.18	847	905	853	854
0.65	886	914	945	950
0.74	873	921	944	941
0.87	864	925	939	947
0.97	864	927	943	952
1.66	833	933	941	954
1.97	792	937	919	926
2.20	777	938	913	919
2.67	759	936	903	905
2.93	748	940	903	904
3.73	707	936	902	895
4.81	685	930	893	893
5.70	678	924	891	893
6.76	689	913	880	879
7.86	685	907	881	872
8.78	1134	904	878	877
9.72	680	900	878	874
11.03	665	899	862	868
11.77	646	894	852	856
12.72	647	890	853	856
13.68	640	885	840	839
14.73	634	881	825	848
15.72	624	878	812	835
16.72	613	873	804	829
18.81	619	859	796	812
20.69	591	852	765	790
22.68	569	842	755	778
24.72	551	831	728	755
26.73	520	822	696	731
28.72	495	703	656	692
30.70	476	664	631	669
32.81	436	627	597	642
34.72	421	582	572	616
36.75	418	579	544	591
38.80	403	572	524	577
40.71	400	533	506	562

TABLE A1.2

DRAIN M

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 2			
	P-G1	P-G2	P-G3	P-G4
42.70	375	490	468	546
44.80	339	484	459	520
47.68	313	443	427	494
50.69	290	406	402	478
53.82	265	398	378	440
56.72	245	362	348	436
59.78	227	351	326	412
63.75	204	292	289	391
65.97	162	253	233	351
70.00	185	238	240	365
72.73	178	253	248	359
76.72	179	239	204	307

TABLE A1.2

DRAIN M

TIME
(DAYS)EXCESS PORE PRESSURES
U (MM)

LEVEL 2

P-N1

P-N2

P-N3

P-N4

	P-N1	P-N2	P-N3	P-N4
0.00	196	197	180	178
0.02	957	963	944	950
0.04	943	948	933	925
0.06	902	917	907	893
0.08	830	846	836	825
0.13	836	852	842	834
0.18	844	856	854	846
0.65	938	950	946	935
0.74	940	959	952	941
0.87	934	955	946	937
0.97	934	956	950	938
1.66	929	966	958	947
1.97	894	940	935	925
2.20	883	934	927	919
2.67	868	925	919	911
2.93	863	927	921	911
3.73	848	922	917	905
4.81	826	911	901	901
5.70	816	909	901	900
6.76	801	899	893	888
7.86	791	890	886	883
8.78	784	891	884	882
9.72	781	885	881	879
11.03	768	868	867	867
11.77	756	858	859	854
12.72	757	858	861	854
13.68	753	846	849	843
14.73	743	830	840	831
15.72	730	817	837	816
16.72	719	808	823	809
18.81	700	796	811	800
20.69	668	767	792	777
22.68	650	753	795	768
24.72	590	726	772	746
26.73	561	693	743	714
28.72	540	652	706	674
30.70	524	625	680	649
32.81	491	591	644	612
34.72	475	565	618	588
36.75	465	549	601	570
38.80	445	526	577	562
40.71	433	514	560	562

TABLE A1.2

DRAIN M

TIME
(DAYS)

EXCESS PORE PRESSURES
U (MM)

LEVEL 2

P-N1

P-N2

P-N3

P-N4

	P-N1	P-N2	P-N3	P-N4
42.70	397	483	519	482
44.80	381	453	497	493
47.68	325	423	467	445
50.69	323	397	458	416
53.82	315	376	419	411
56.72	282	356	387	407
59.78	270	342	379	395
63.75	235	302	330	313
65.97	167	245	287	255
70.00	170	247	274	278
72.73	191	295	275	271
76.72	151	384	267	278

TABLE A1.2

DRAIN M

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TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 3			
	P-R1	P-R2	P-R3	P-R4
0.00	282	281	278	268
0.02	1023	1025	1019	1004
0.04	1025	1049	1031	1032
0.06	1012	1026	1019	1013
0.08	941	970	960	959
0.13	937	963	954	957
0.18	950	971	971	969
0.65	1006	1056	1054	1052
0.74	990	1048	1045	1043
0.87	982	1044	1045	1044
0.97	984	1050	1051	1051
1.66	958	1049	1052	1052
1.97	926	1020	1025	1029
2.20	913	1017	1025	1023
2.67	891	1008	1017	1017
2.93	884	1009	1019	1017
3.73	859	997	1012	1015
4.81	837	987	1002	1002
5.70	828	981	1002	1002
6.76	814	964	989	989
7.86	805	957	982	985
8.78	790	950	977	979
9.72	788	949	979	980
11.03	783	934	965	965
11.77	783	924	954	952
12.72	779	920	949	951
13.68	763	905	947	939
14.73	755	892	931	924
15.72	750	879	921	912
16.72	741	874	915	908
18.81	733	868	907	903
20.69	709	847	891	883
22.68	709	848	895	881
24.72	696	837	877	864
26.73	663	803	852	841
28.72	636	769	818	806
30.70	610	745	794	782
32.81	576	712	762	748
34.72	558	689	740	726
36.75	549	674	726	710
38.80	529	652	704	690
40.71	516	637	688	676

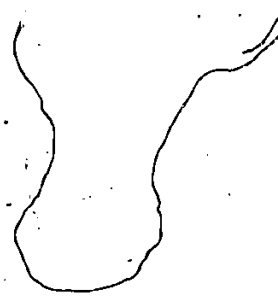


TABLE A1.2

DRAIN M

TIME
(DAYS)

EXCESS PORE PRESSURES
U (MM)

LEVEL 3

P-R1 P-R2 P-R3 P-R4

TIME (DAYS)	P-R1	P-R2	P-R3	P-R4
42.70	482	596	647	633
44.80	466	576	626	613
47.68	438	546	591	578
50.69	417	522	563	551
53.82	403	495	535	522
56.72	361	466	500	489
59.78	346	440	474	461
63.75	304	412	426	416
65.97	262	339	370	358
70.00	270	336	369	354
72.73	273	329	378	351
76.72	233	300	318	263

TABLE A1.2

DRAIN M

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 3			
	P-B1	P-B2	P-B3	P-B4
0.00	281	283	251	263
0.02	1019	1018	998	1003
0.04	1006	1010	993	996
0.06	982	990	975	978
0.08	919	931	918	921
0.13	920	931	918	922
0.18	929	944	933	934
0.65	1023	1040	1029	1024
0.74	1027	1048	1034	1031
0.87	1021	1043	1032	1029
0.97	1020	1046	1036	1032
1.66	963	1061	1043	1038
1.97	974	1035	1020	1019
2.20	962	1029	1014	1014
2.67	944	1020	1006	1006
2.93	939	1019	1009	1009
3.73	918	1013	1004	1002
4.81	893	999	992	989
5.70	883	996	993	995
6.76	866	985	981	982
7.86	856	976	977	977
8.78	855	966	973	975
9.72	845	968	967	971
11.03	830	951	968	958
11.77	824	940	949	946
12.72	821	937	946	945
13.68	805	923	933	933
14.73	789	905	916	914
15.72	787	895	902	903
16.72	778	887	898	897
18.81	769	883	895	895
20.69	749	863	875	878
22.68	748	861	873	871
24.72	733	846	859	851
26.73	701	816	830	824
28.72	669	778	787	785
30.70	644	754	769	764
32.81	611	721	735	733
34.72	593	699	713	713
36.75	582	683	695	696
38.80	557	662	674	676
40.71	545	646	661	661

TABLE A1.2

DRAIN M

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 3			
	P-B1	P-B2	P-B3	P-B4
42.70	509	604	618	621
44.80	492	584	598	603
47.68	467	553	569	573
50.69	444	520	537	545
53.82	424	496	511	507
56.72	389	462	478	489
59.78	379	432	451	467
63.75	333	400	414	438
65.97	283	346	358	376
70.00	298	353	367	376
72.73	292	352	355	371
76.72	249	307	307	324

TABLE A1.2

DRAIN M

TIME (DAYS)	SOIL SURFACE ELEVATION LEVEL 1 (MM)	EXCESS PORE PRESSURES U (MM)		
		P-R5	LEVEL 4	
			P-G5	TRANS.
0.00	968	341	337	252
0.02	954	1039	1063	1093
0.04	952	1027	1060	1067
0.06	952	997	1043	1115
0.08	950	940	991	1069
0.13	949	987	1032	1075
0.18	948	963	1022	1101
0.65 ^p	938	1028	1098	1137
0.74	936	1030	1102	1128
0.87	934	1024	1103	1133
0.97	932	1025	1108	1132
1.66	925	1025	1112	1124
1.97	921	993	1087	
2.20	918	984	1082	
2.67	915	968	1076	
2.93	912	957	1072	
3.73	906	937	1065	
4.81	898	917	1052	
5.70	893	908	1049	
6.76	893	890	1033	
7.86	890	879	1023	
8.78	887	874	1022	
9.72	883	866	1014	
11.03	871	854	1000	
11.77	869	846	987	
12.72	866	843	985	
13.68	863	823	968	
14.73	859	812	953	
15.72	855	796	937	
16.72	852	793	932	
18.81	848	780	923	
20.69	842	761	906	
22.68	836	756	904	
24.72	833	739	885	
26.73	828	711	856	
28.72	824	683	820	
30.70	819	657	797	
32.81	815	623	763	
34.72	811	607	741	
36.75	807	597	727	
38.80	804	579	706	
40.71	800	566	692	

TABLE A1.2

DRAIN M

=====

TIME (DAYS)	SOIL SURFACE ELEVATION LEVEL 1 (MM)	EXCESS PORE PRESSURES U (MM)		
		P-R5	LEVEL 4 P-G5	TRANS.
42.70	798	532	650	
44.80	794	515	631	
47.68	791	485	598	
50.69	788	463	571	
53.82	784	444	542	
56.72	780	406	508	
59.78	780	390	481	
63.75	779	348	435	
65.97	811	302	377	
70.00	771	300	371	
72.73	767	302	373	
76.72	766	286	324	

TABLE A1.3

DRAIN G

TIME EXCESS PORE PRESSURES
(DAYS) U (MM)

LEVEL 2

P-G1 P-G2 P-G3 P-G4

TIME (DAYS)	P-G1	P-G2	P-G3	P-G4
0.00	172	184	178	173
0.02	937	1049	944	946
0.04	823	917	926	923
0.06	785	881	896	895
0.09	707	835	855	848
0.11	693	862	870	864
0.15	705	880	905	897
0.19	677	869	890	889
0.29	657	894	908	905
0.37	646	908	921	916
0.81	598	932	949	940
0.92	587	952	973	960
1.10	552	947	966	961
1.87	506	898	916	920
2.96	297	884	907	898
3.85	318	896	920	926
4.08	1774	900	926	915
4.90	349	859	895	897
6.02	406	850	880	888
6.93	439	857	897	902
7.87	442	832	884	889
9.16	448	805	852	858
9.91	449	825	877	879
10.87	453	811	863	872
11.83	455	798	848	857
12.87	461	783	837	848
13.86	461	764	770	841
14.86	460	742	756	829
16.96	464	705	799	827
18.84	465	684	800	813
20.83	460	647	766	785
22.87	459	612	726	729
24.87	444	564	700	700
26.86	441	524	680	685
28.84	435	498	666	658
30.96	421	464	652	613
32.85	415	441	643	577
34.88	407	415	633	557
37.02	407	399	628	541
38.85	405	392	604	523
40.87	389	372	583	479
42.94	367	346	565	453

TABLE A1.3

DRAIN G

TIME
(DAYS)

EXCESS PORE PRESSURES
U (MM)

LEVEL 2

P-G1 P-G2 P-G3 P-G4

	P-G1	P-G2	P-G3	P-G4
45.83	349	305	545	417
48.83	345	276	535	419
51.95	338	247	531	418
54.86	319	210	533	357
57.91	308	176	534	356
61.89	299	147	531	284
64.15	291	131	524	290
68.14	281	93	505	229
70.86	286	73	500	257
74.85	345	55	477	279

TABLE A1.3

DRAIN G

=====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 2			
	P-N1	P-N2	P-N3	P-N4
0.00	190	197	177	138
0.02	966	965	954	936
0.04	919	917	908	917
0.06	885	886	878	877
0.09	848	843	831	839
0.11	833	845	833	838
0.15	893	889	874	868
0.19	904	896	871	863
0.29	924	918	893	883
0.37	911	918	901	890
0.81	947	951	939	926
0.92	963	972	961	946
1.10	961	964	951	941
1.87	892	914	902	893
2.96	878	907	893	886
3.85	882	930	921	909
4.08	879	909	915	885
4.90	852	898	890	878
6.02	828	882	876	868
6.93	835	898	885	881
7.87	817	879	875	869
9.16	804	845	843	835
9.91	801	867	865	863
10.87	795	859	862	849
11.83	778	843	846	833
12.87	772	833	854	825
13.86	766	824	829	822
14.86	755	811	829	805
16.96	747	812	831	806
18.84	732	798	818	791
20.83	701	768	795	760
22.87	643	711	726	714
24.87	617	691	695	693
26.86	598	667	669	672
28.84	574	652	643	643
30.96	552	599	594	599
32.85	526	579	563	556
34.88	516	561	552	548
37.02	501	556	552	530
38.85	496	536	513	504
40.87	456	497	472	475
42.94	430	465	446	439

TABLE A1.3

DRAIN 'G

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 2			
	P-N1	P-N2	P-N3	P-N4
45.83	394	426	409	405
48.83	393	421	408	393
51.95	382	396	396	381
54.86	330	366	360	360
57.91	302	333	325	336
61.89	268	287	287	287
64.15	240	267	256	254
68.14	210	224	234	226
70.86	212	231	236	229
74.85	200	228	217	220

TABLE A1.3

DRAIN G

=====

TIME
(DAYS)EXCESS PORE PRESSURES
U (MM)

LEVEL 3

P-R1

P-R2

P-R3

P-R4

TIME (DAYS)	P-R1	P-R2	P-R3	P-R4
0.00	252	247	279	271
0.02	1098	980	1102	1077
0.04	964	943	1026	1029
0.06	921	981	986	1003
0.09	877	940	939	950
0.11	888	933	952	957
0.15	934	981	990	995
0.19	889	956	967	979
0.29	881	980	989	997
0.37	871	994	996	1004
0.81	830	1026	1031	1038
0.92	832	1048	1054	1061
1.10	790	1039	1045	1053
1.87	680	989	995	1005
2.96	520	976	989	997
3.85	388	986	1000	1007
4.08	1774	994	1008	1008
4.90	627	965	977	990
6.02	625	949	968	974
6.93	627	958	980	986
7.87	653	944	966	972
9.16	660	913	937	945
9.91	661	931	957	965
10.87	663	922	947	957
11.83	664	908	945	940
12.87	671	898	929	932
13.86	671	892	935	924
14.86	656	893	927	918
16.96	657	888	923	915
18.84	657	883	900	904
20.83	655	877	897	883
22.87	658	872	831	836
24.87	656	794	836	825
26.86	653	782	826	810
28.84	649	774	794	810
30.96	648	730	760	766
32.85	647	702	730	726
34.88	593	685	712	722
37.02	592	672	708	717
38.85	592	657	687	685
40.87	585	613	646	639
42.94	590	585	618	612

TABLE A1.3

DRAIN G

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 3			
	P-R1	P-R2	P-R3	P-R4
45.83	587	546	580	578
48.83	587	541	556	577
51.95	584	529	536	566
54.86	582	497	512	505
57.91	571	465	471	478
61.89	569	458	435	426
64.15	569	371	384	392
68.14	564	327	358	350
70.86	565	329	361	362
74.85	558	308	310	306

TABLE A1.3

DRAIN G

EXCESS PORE PRESSURES
U (MM)

TIME (DAYS)	LEVEL 3			
	P-B1	P-B2	P-B3	P-B4
0.00	233	257	236	262
0.02	985	1050	1046	1035
0.04	986	1007	1008	999
0.06	988	971	971	973
0.09	987	940	946	940
0.11	987	925	930	925
0.15	988	955	967	960
0.19	975	962	970	966
0.29	974	1002	999	998
0.37	978	994	996	997
0.81	983	1031	1034	1033
0.92	986	1053	1053	1059
1.10	987	1043	1048	1051
1.87	989	996	999	1002
2.96	990	986	991	994
3.85	914	1006	1015	1017
4.08	915	974	1008	1001
4.90	897	948	985	979
6.02	893	949	970	972
6.93	895	964	981	984
7.87	891	950	970	970
9.16	897	915	935	936
9.91	897	933	958	957
10.87	899	927	950	948
11.83	899	915	935	934
12.87	903	912	939	931
13.86	904	913	938	923
14.86	903	951	935	907
16.96	901	893	931	908
18.84	893	888	900	906
20.83	810	884	894	879
22.87	702	860	826	857
24.87	719	816	746	849
26.86	692	782	640	836
28.84	687	776	627	825
30.96	678	749	679	817
32.85	670	727	659	769
34.88	662	700	635	754
37.02	653	687	611	739
38.85	642	677	585	704
40.87	538	667	556	662
42.94	532	654	540	637

TABLE A1.3

DRAIN G

TIME
(DAYS)

EXCESS PORE PRESSURES
U (MM)

LEVEL 3

P-B1

P-B2

P-B3

P-B4

	P-B1	P-B2	P-B3	P-B4
45.83	491	614	511	602
48.83	67	598	485	569
51.95	4	579	457	528
54.86	-48	558	437	493
57.91	-84	525	419	458
61.89	881	504	398	426
64.15	868	362	388	411
68.14	1912	304	366	376
70.86	1917	296	264	348
74.85	1917	268	367	289

TABLE A1.3

DRAIN G

TIME (DAYS)	SOIL SURFACE ELEVATION LEVEL 1 (MM)	EXCESS PORE PRESSURES U (MM)		
		P-R5	P-G5	TRANS.
0.00	937	339	341	290
0.02	927	1032	1053	
0.04	926	987	1026	1000
0.06	925	950	1002	1051
0.09	924	941	981	1027
0.11	923	905	952	965
0.15	922	926	978	1032
0.19	921	938	986	910
0.29	921	970	1017	972
0.37	916	961	1058	1043
0.81	906	987	1095	1108
0.92	905	1004	1117	1124
1.10	902	991	1109	1105
1.87	893	925	1063	1094
2.96	883	885	1052	975
3.85	876	890	1064	990
4.08	877			971
4.90	871	854	1035	957
6.02	868	831	1016	
6.93	863	834	1023	
7.87	857	815	1009	
9.16	850	780	972	
9.91	847	795	993	
10.87	843	790	983	
11.83	839	788	965	
12.87	832	786	957	
13.86	830	780	942	
14.86	826	773	933	
16.96	819	772	936	
18.84	814	764	924	
20.83	810	741	906	
22.87	804	691	852	
24.87	800	688	838	
26.86	795	679	823	
28.84	791	657	802	
30.96	786	625	761	
32.85	782	606	730	
34.88	778	599	716	
37.02	774	592	713	
38.85	772	589	701	
40.87	770	563	659	
42.94	765	542	637	

TABLE A1.3

DRAIN G

TIME (DAYS)	SOIL SURFACE ELEVATION LEVEL 1 (MM)	EXCESS PORE PRESSURES U. (MM)		
		P-R5	LEVEL 4 P-G5	TRANS.
45.83	761	499	606	
48.83	757	492	586	
51.95	753	477	569	
54.86	749	448	544	
57.91	747	410	515	
61.89	742	376	469	
64.15	742	344	433	
68.14	739	296	397	
70.86	735	307	393	
74.85	734	251	323	

TABLE A1.4

BRAIN A

=====

TIME EXCESS PORE PRESSURES
(DAYS) U (MM)

LEVEL 2

P-G1 P-G2 P-G3 P-G4

	P-G1	P-G2	P-G3	P-G4
0.00	167	158	169	145
0.01	939	953	952	946
0.02	1073	1093	1089	1064
0.03	858	875	891	857
0.05	838	863	871	844
0.07	818	839	851	821
0.09	861	895	906	876
0.11	833	872	876	851
0.15	829	861	876	846
0.35	846	920	930	907
0.84	784	923	945	947
1.03	783	943	967	948
1.05	769	932	960	939
1.38	740	922	951	926
1.89	714	912	951	924
1.93	718	917	956	926
2.13	723	922	963	936
2.84	727	920	969	938
3.13	674	887	944	918
4.17	642	873	933	915
4.93	635	862	925	910
5.85	640	871	927	908
6.85	608	838	902	885
7.90	610	826	887	874
8.89	605	808	871	859
9.89	597	805	871	864
11.98	593	788	864	865
13.86	574	761	841	843
15.88	559	743	820	819
17.89	528	694	779	779
19.90	503	665	741	741
21.89	493	643	717	717
23.88	485	622	690	697
25.94	463	585	648	643
27.89	420	546	609	603
29.90	366	521	584	583
32.05	383	432	567	562
33.88	392	485	572	543
35.87	401	451	501	506
37.97	407	423	470	484
40.86	411	392	438	447
43.86	413	363	407	418

TABLE A1.4

DRAIN A

=====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 2			
	P-G1	P-G2	P-G3	P-G4
46.98	414	347	385	389
49.90	410	318	354	360
52.93	248	302	332	339
56.91	214	302	290	348
59.15	195	244	268	339
63.17	166	211	237	255
65.89	169	213	243	256
69.89	159	210	194	209

TABLE A1.4

DRAIN A

TIME (DAYS) EXCESS PORE PRESSURES U (MM)

LEVEL 2

P-N1 P-N2 P-N3 P-N4

TIME (DAYS)	P-N1	P-N2	P-N3	P-N4
0.00	179	144	154	163
0.01	961	943	920	851
0.02	1090	1075	1061	929
0.03	872	868	887	859
0.05	854	847	854	836
0.07	832	826	818	817
0.09	888	881	873	868
0.11	859	847	822	833
0.15	863	855	834	854
0.35	918	922	895	905
0.84	920	937	913	933
1.03	922	947	923	948
1.05	908	942	921	945
1.38	884	929	915	935
1.89	864	930	912	936
1.93	869	936	918	941
2.13	870	946	926	944
2.84	864	955	932	957
3.13	822	926	917	938
4.17	788	907	906	929
4.93	773	893	901	928
5.85	770	891	895	927
6.85	734	861	875	907
7.90	723	848	871	898
8.89	709	832	858	886
9.89	702	826	854	881
11.98	688	807	846	876
13.86	664	791	821	857
15.88	646	767	811	840
17.89	606	744	772	807
19.90	574	733	746	772
21.89	557	707	720	745
23.88	542	686	703	726
25.94	495	632	652	670
27.89	459	594	625	629
29.90	444	574	584	609
32.05	433	557	579	588
33.88	416	534	558	563
35.87	389	500	530	524
37.97	361	467	487	490
40.86	336	439	451	448
43.86	307	411	418	428

TABLE A1.4

DRAIN A

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 2			
	P-N1	P-N2	P-N3	P-N4
46.98	306	402	398	396
49.90	277	370	364	363
52.93	269	330	342	341
56.91	232	316	303	308
59.15	205	296	280	286
63.17	186	265	249	257
65.89	173	262	251	253
69.89	151	203	207	211

TABLE A1.4

DRAIN A

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 3			
	P-R1	P-R2	P-R3	P-R4
0.00	249	280	275	263
0.01	1001	1006	1030	1014
0.02	1138	1155	1177	1161
0.03	960	993	1009	989
0.05	951	971	986	964
0.07	940	958	969	952
0.09	995	1013	1022	1001
0.11	915	948	976	954
0.15	962	986	991	978
0.35	968	1016	1024	1005
0.84	918	1036	1042	1031
1.03	913	1056	1067	1057
1.05	890	1049	1059	1050
1.38	852	1037	1047	1039
1.89	815	1028	1040	922
1.93	820	1032	1047	1047
2.13	821	1035	1063	1054
2.84	831	1040	1072	1064
3.13	788	1013	1053	1048
4.17	756	993	1035	1041
4.93	748	978	1027	1038
5.85	746	976	1026	1040
6.85	712	952	1007	1020
7.90	711	936	996	1011
8.89	699	921	982	997
9.89	692	913	976	993
11.98	690	909	975	991
13.86	674	892	957	972
15.88	664	877	943	958
17.89	635	846	911	928
19.90	610	817	880	897
21.89	600	796	860	877
23.88	590	779	845	862
25.94	548	731	793	811
27.89	521	699	759	777
29.90	506	682	742	761
32.05	493	666	724	743
33.88	478	648	704	724
35.87	457	614	669	687
37.97	437	593	639	657
40.86	415	567	604	618
43.86	390	540	564	592

TABLE A1.4

DRAIN A

TIME
(DAYS)

EXCESS PORE PRESSURES
U (MM)

LEVEL 3

P-R1 P-R2 P-R3 P-R4

	P-R1	P-R2	P-R3	P-R4
46.98	444	511	533	559
49.90	443	477	505	529
52.93	326	453	478	496
56.91	295	394	481	452
59.15	269	371	483	427
63.17	242	332	486	387
65.89	234	335	485	380
69.89	249	277	489	323

TABLE A1.4

DRAIN A

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 3			
	P-B1	P-B2	P-B3	P-B4
0.00	269	259	296	246
0.01	1038	1030	1038	1016
0.02	1157	1153	1158	1138
0.03	962	967	980	954
0.05	938	939	950	929
0.07	950	955	975	946
0.09	965	974	989	969
0.11	915	919	934	900
0.15	937	943	955	931
0.35	1008	1021	1037	1008
0.84	1002	1052	1051	1027
1.03	1004	1065	1060	1036
1.05	997	1059	1059	1033
1.38	975	1050	1053	1026
1.89	955	1047	1051	1039
1.93	959	1053	1057	1032
2.13	964	1060	1060	1039
2.84	966	1069	1070	1048
3.13	928	1040	1043	1025
4.17	894	1021	1030	1014
4.93	880	1010	1021	1008
5.85	878	1012	1026	1012
6.85	842	987	1006	993
7.90	831	977	996	986
8.89	816	960	984	974
9.89	810	957	981	974
11.98	799	949	976	972
13.86	784	929	960	956
15.88	771	913	946	944
17.89	737	881	914	913
19.90	711	853	888	889
21.89	694	830	864	866
23.88	680	815	851	855
25.94	632	763	798	801
27.89	599	726	760	764
29.90	584	710	744	747
32.05	570	692	727	730
33.88	555	664	706	709
35.87	528	632	673	676
37.97	500	624	643	646
40.86	474	574	606	609
43.86	444	541	569	575

TABLE A1.4

DRAIN A

=====

TIME
(DAYS)

EXCESS PORE PRESSURES
U (MM)

LEVEL 3

P-B1 P-B2 P-B3 P-B4

	P-B1	P-B2	P-B3	P-B4
46.98	409	515	543	546
49.90	389	477	509	513
52.93	367	455	485	493
56.91	323	404	442	440
59.15	300	378	417	418
63.17	270	347	377	391
65.89	263	334	370	384
69.89	223	336	316	332

TABLE A1.4

DRAIN A

TIME (DAYS)	SOIL SURFACE ELEVATION LEVEL 1 (MM)	EXCESS PORE PRESSURES U (MM)		
		P-R5	LEVEL 4	
			P-G5	TRANS.
0.00	933	350	351	189
0.01	933	993	991	1181
0.02	923	1142	1142	1383
0.03	921	1016	1089	1086
0.05	919	986	1031	1160
0.07	919	996	1034	1138
0.09	915	1030	1061	1131
0.11	914	965	1011	1147
0.15	914	993	1034	
0.35	910	1037	1094	1165
0.84	900	997	1122	1156
1.03	896	993	1130	1133
1.05	893	979	1129	1123
1.38	889	952	1120	1105
1.89	884	921	1115	1069
1.93	880	923	1121	1074
2.13	880	920	1123	1081
2.84	872	920	1131	1079
3.13	867	873	1107	1039
4.77	865	845	1088	1018
4.93	860	832	1078	1003
5.85	856	829	1075	982
6.85	850	793	1050	963
7.90	846	785	1037	940
8.89	842	772	1020	923
9.89	838	763	1011	930
11.98	828	756	1005	912
13.86	822	738	983	782
15.88	816	725	968	909
17.89	810	692	929	832
19.90	798	667	901	802
21.89	797	655	879	780
23.88	793	643	862	749
25.94	788	604	814	706
27.89	784	573	777	687
29.90	779	560	763	672
32.05	774	551	748	665
33.88	770	540	729	653
35.87	767	509	694	629
37.97	763	486	664	600
40.86	758	462	629	565
43.86	754	433	599	682

TABLE A1.4

DRAIN A

TIME (DAYS)	SOIL SURFACE ELEVATION LEVEL 1 (MM)	EXCESS PORE PRESSURES U (MM)		
		P-R5	LEVEL 4 P-G5	TRANS.
46.98	749	411	574	654
49.90	747	407	534	
52.93	743	338	511	
56.91	740	315	463	
59.15	738	287	438	
63.17	734	724	393	
65.89	732	605	386	
69.89	730	509	337	

TABLE A2.1

NO DRAIN

=====

TIME (DAYS)	SETTLEMENT (MM)		STRAIN (%)	
	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)
0.00	0	0	0	0
0.02	6	0	6	0
0.04	11	1	11	2
0.06	10	1	10	2
0.08	8	2	8	3
0.10	10	2	10	3
0.13	10	2	10	3
0.15	11	2	11	3
0.17	16	2	16	3
0.21	16	2	16	3
0.25	18	2	18	3
0.29	19	2	19	3
0.33	20	2	20	3
0.71	29	4	29	7
0.75	29	4	29	7
0.83	27	-3	27	-5
0.92	32	-2	32	-3
1.04	33	-2	33	-3
1.35	30	5	30	8
1.73	31	0	31	0
1.89	32	0	32	0
2.76	37	7	37	12
3.80	43	8	43	13
4.76	48	8	48	13
5.21	52	10	52	17
5.74	56	10	56	17
6.07	59	12	59	20
6.74	62	12	62	20
7.05	64	12	64	20
7.76	66	14	66	23
7.91	67	15	67	25
8.82	71	15	71	25
9.90	74	18	74	30
10.79	78	18	78	30
11.02	78	18	78	30
11.77	78	20	78	33
12.78	81	20	81	33
13.78	82	21	82	35
14.76	88	21	88	35
16.13	93	23	93	38
16.88	94	27	94	45
17.77	97	28	97	47

TABLE A2.1

NO DRAIN

TIME (DAYS)	SETTLEMENT (MM)		STRAIN (%)	
	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)
18.76	101	29	101	48
19.76	103	30	103	50
20.75	104	30	104	50
21.76	107	30	107	50
23.91	113	31	113	52
25.79	120	33	120	55
27.79	120	37	120	62
29.82	124	42	124	70
31.77	126	47	126	78
33.76	130	47	130	78
35.75	134	48	134	80
37.86	138	49	138	82
39.75	141	53	141	88
41.77	144	56	144	93
43.91	149	62	149	103
45.74	151	63	151	105
47.74	152	64	152	107
49.88	156	68	156	113
52.77	159	68	159	113
55.77	164	71	164	118
58.90	167	75	167	125
61.81	169	78	169	130
64.86	173	79	173	132
68.83	180		180	
71.05	179		179	
75.08	183		183	
77.81	186		186	

TABLE A2.2

DRAIN M
=====

TIME (DAYS)	SETTLEMENT (MM)			STRAIN (%)		
	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)
0.00	0	0	0	0	0	0
0.03	14	2	4	14	3	12
0.05	16	2	6	16	3	20
0.07	16	2	5	16	3	15
0.10	18	3	5	18	5	15
0.13	19	3	5	19	5	15
0.18	20	3	5	20	5	15
0.67	30	6	4	30	10	12
0.75	32	10	4	32	17	12
0.88	34	14	5	34	23	15
0.97	36	8	8	36	13	25
1.67	43	12	10	43	20	33
1.98	47	13	11	47	22	35
2.22	50	17	12	50	28	40
2.69	53	15	12	53	25	38
2.94	56	16	13	56	27	43
3.75	62	18	10	62	30	33
4.84	70	21	11	70	35	35
5.72	75	23	12	75	38	38
5.96	75	23	12	75	38	40
6.76	78	25	10	78	42	32
7.69	81	24	0	81	40	0
8.70	85	27	7	85	45	22
9.70	97	32	6	97	53	20
11.07	99	35	12	99	58	38
11.81	102	41	13	102	68	43
12.72	105	37	12	105	62	38
13.70	109	39	13	109	65	43
14.71	113	40	14	113	67	45
15.69	116	42	15	116	70	48
16.70	120	44	16	120	73	52
18.85	126	49	17	126	82	55
20.73	132	53	20	132	88	65
22.73	135	56	22	135	93	72
24.76	140	59	21	140	98	68
26.72	144	63	22	144	105	73
28.70	149	67	22	149	112	73
30.69	153	70	23	153	117	77
32.80	157	72	23	157	120	77
34.69	161	81	25	161	135	82
36.71	164	79	26	164	132	87
38.85	168	82	28	168	137	92

TABLE A2.2

DRAIN M

TIME (DAYS)	SETTLEMENT (MM)			STRAIN (%)		
	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)
40.68	170	84	28	170	140	93
42.68	174	85	28	174	142	93
44.82	177	89	30	177	148	100
47.71	180	92	32	180	153	105
50.72	184	95	32	184	158	105
53.84	188	98	33	188	163	110
56.76	188	99	33	188	165	110
59.81	189	102	34	189	170	113
63.78	191	106	37	157	177	122
65.99	197	107	36	197	178	120
70.02	201	108	40	201	180	132
72.75	202	110	40	202	183	133
76.74	204	113	40	204	188	132

TABLE A2.3

DRAIN G

TIME (DAYS)	SETTLEMENT (MM)			STRAIN (%)		
	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)
0.00	0	0	0	0	0	0
0.03	10	0	1	10	0	3
0.05	11	0	1	11	0	3
0.07	12	0	1	12	0	3
0.09	13	1	2	13	2	7
0.11	14	1	1	14	2	3
0.16	16	1	2	16	2	7
0.19	17	1	2	17	2	7
0.30	16	2	2	16	3	7
0.38	22	3	4	22	5	13
0.82	31	11	8	31	18	27
0.92	33	12	8	33	20	27
1.10	36	13	10	36	22	33
1.91	45	10	13	45	17	43
2.99	55	13	18	55	22	60
3.88	61	13	19	61	22	63
4.12	60	13	15	60	22	50
4.90	66	18	15	66	30	50
5.83	69	17	14	69	28	47
6.84	75	19	13	75	32	43
7.84	80	19	12	80	32	40
9.22	88	22	14	88	37	47
9.97	91	23	15	91	38	50
10.87	94	25	14	94	42	47
11.86	98	27	14	98	45	47
12.85	106	29	14	106	48	47
13.84	108	29	14	108	48	47
14.85	112	32	15	112	53	50
16.99	119	36	17	119	60	57
18.88	123	39	18	123	65	60
20.88	127	43	17	127	72	57
22.91	133	45	19	133	75	63
24.86	138	49	21	138	82	70
26.84	142	53	22	142	88	73
28.83	147	57	23	147	95	77
30.94	151	60	23	151	100	77
32.84	155	62	23	155	103	77
34.85	159	65	25	159	108	83
37.00	163	68	27	163	113	90
38.83	165	69	27	165	115	90
40.83	168	70	28	168	117	93
42.97	173	73	29	173	122	97

TABLE A2.3

DRAIN G

TIME (DAYS)	SETTLEMENT (MM)			STRAIN (%)		
	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)
45.85	176	75	29	176	125	97
48.86	181	78	30	181	130	100
51.99	185	79	41	185	132	137
54.90	188	82	40	188	137	133
57.94	191	83	36	191	138	120
61.92	195	84	35	195	140	117
64.13	196	89	39	196	148	130
68.17	198	87	39	198	145	130
70.89	203	90	40	203	150	133
74.89	203	91	39	203	152	130

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TABLE A2.4

DRAIN A

TIME (DAYS)	SETTLEMENT (MM)			STRAIN (%)		
	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)
0.00	0	0	0	0	0	0
0.02	11	4	0	11	6	0
0.04	13	2	0	13	3	0
0.06	14	2	0	14	3	0
0.08	15	3	1	15	4	3
0.10	18	3	1	18	4	3
0.13	19	4	1	19	6	3
0.15	20	4	5	20	6	17
0.17	24	7	7	24	12	23
0.21	33	12	8	33	19	27
0.25	37	12	8	37	20	27
0.29	40	13	9	40	21	30
0.33	44	15	10	44	24	33
0.71	49	17	9	49	28	30
0.75	54	18	9	54	29	30
0.83	61	19	11	61	31	37
0.92	66	21	12	66	34	40
1.04	68	33	15	68	54	50
1.35	73	37	20	73	61	67
1.73	77	38	17	77	63	57
1.89	77	39	16	77	64	53
2.76	83	40	19	83	66	63
3.80	88	43	20	88	71	67
4.76	91	44	20	91	73	67
5.21	96	46	22	96	76	73
5.74	105	47	26	105	78	87
6.07	111	53	28	111	88	93
6.74	118	55	29	118	92	97
7.05	124	58	31	124	96	103
7.76	136	58	33	136	97	110
7.91	136	57	33	136	95	110
8.82	141	61	36	141	101	120
9.90	146	62	37	146	103	123
10.79	149	64	37	149	106	123
11.02	155	67	40	155	112	133
11.77	160	70	43	160	117	143
12.78	163	71	43	163	118	143
13.78	167	72	44	167	120	147
14.76	171	78	45	171	130	150
16.13	175	80	48	175	133	160
16.88	179	82	49	179	136	163
17.77	184	87	52	184	144	173

TABLE A2.4

DRAIN A

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TIME (DAYS)	SETTLEMENT (MM)			STRAIN (%)		
	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)
18.76	187	86	52	187	143	173
19.76	191	91	54	191	151	180
20.75	194	93	55	194	154	183
21.76	195	93	57	195	154	190
23.91	199	95	58	199	158	193
25.79	201	97	61	201	162	203
27.79	203	98	60	203	163	200

TABLE A3.1

NO DRAIN

TIME (DAYS)	SOIL SURFACE ELEVATION	EXCESS PORE PRESSURES U (MM)			
		LEVEL 1	LEVEL 2	LEVEL 3	LEVEL 4
		(MM)	(G1,G2)	(R1,R2)	(R3,G3)
0.00	936	170	291	342	
0.02	931	988	1083	1122	
0.04	926	1005	1114	1162	
0.06	926	988	1105	1152	
0.08	929	945	1050	1106	
0.10	926	897	1001	1064	
0.12	926	943	1038	1093	
0.14	925	893	999	1063	
0.16	921	921	1030	1082	
0.20	920	932	1034	1087	
0.25	918	933	1035	1087	
0.29	917	931	1035	1036	
0.33	916	934	1030	1086	
0.70	908	977	1079	1129	
0.75	908	951	1060	1111	
0.83	909	947	1050	1103	
0.91	905	950	1053	1104	
1.04	904	953	1061	1109	
1.35	907	949	1051	1102	
1.72	906	937	1037	1092	
1.89	905	932	1034	1086	
2.76	899	903	1005	1056	
3.80	893	883	983	1033	
4.76	889	875	968	1016	
5.20	884	958	1055	1102	
5.73	880	956	1050	1095	
6.07	878	953	1046	1090	
6.77	875	956	1049	1092	
7.03	873	966	1060	1100	
7.74	871	946	1043	1083	
7.99	869	939	1032	1075	
8.81	866	933	1027	1068	
9.89	862	928	1021	1060	
10.78	859	924	1019	1055	
11.84	859	914	1011	1046	
12.93	855	918	1014	1048	
13.85	854	896	989	1028	
14.79	849	900	999	1030	
16.11	843	878	987	1015	
16.85	842	872	981	1007	
17.80	840	852	965	991	
18.76	836	843	957	981	

TABLE A3.1

NO DRAIN

TIME (DAYS)	SOIL SURFACE ELEVATION LEVEL 1 (MM)	EXCESS PORE PRESSURES U (MM)			
		LEVEL 2	LEVEL 3	LEVEL 4	
		(G1,G2)	(R1,R2)	(R3,G3)	
19.80	834	836	949	973	
20.79	833	835	948	972	
21.80	830	825	943	971	
23.89	823	813	938	961	
25.77	817	771	903	929	
27.76	817	776	914	937	
29.77	813	738	887	907	
31.77	810	705	865	877	
33.75	806	682	849	861	
35.74	802	700	864	880	
37.87	799	665	841	851	
39.75	795	643	822	837	
41.79	792	637	821	831	
43.91	788	636	821	834	
45.75	786	628	817	831	
47.76	785	587	773	791	
49.83	781	569	758	777	
52.72	777	538	751	749	
55.73	773	529	720	737	
58.86	770	510	694	716	
61.75	767	497	679	702	
64.82	764	485	664	686	
68.78	757	472	646	667	
71.00	758	425	612	630	
75.04	754	399	585	599	
77.77	751	386	556	583	
81.75	747	375	521	541	

TABLE A3.2

DRAIN M

TIME
(DAYS)EXCESS PORE PRESSURES
U (MM)

LEVEL 2

(G1,N1) (N2) (G3,N3) (G4,N4)

TIME (DAYS)	(G1,N1)	(N2)	(G3,N3)	(G4,N4)
0.00	164	197	175	180
0.02	944	963	929	928
0.04	944	948	941	929
0.06	912	917	908	909
0.08	838	846	845	842
0.13	833	852	846	818
0.18	846	856	853	850
0.65	912	950	946	942
0.74	906	959	948	941
0.87	899	955	943	942
0.97	899	956	946	945
1.66	881	966	950	951
1.97	843	940	927	926
2.20	830	934	920	919
2.67	813	925	911	908
2.93	805	927	912	907
3.73	777	922	909	900
4.81	755	911	897	897
5.70	747	909	896	896
6.76	745	899	887	883
7.86	738	890	883	878
8.78	959	891	881	879
9.72	730	885	879	877
11.03	716	868	865	868
11.77	701	858	855	855
12.72	702	858	857	855
13.68	697	846	844	851
14.73	689	830	833	839
15.72	677	817	824	825
16.72	666	808	813	819
18.81	659	796	803	806
20.69	629	767	778	783
22.68	610	753	775	773
24.72	570	726	750	751
26.73	540	693	720	722
28.72	517	652	681	683
30.70	500	625	656	659
32.81	463	591	620	627
34.72	448	565	595	602
36.75	441	549	573	581
38.80	424	526	550	569
40.71	416	514	533	562

TABLE A3.2

DRAIN M

=====

TIME
(DAYS)EXCESS PORE PRESSURES
U (MM)

LEVEL 2

(G1,N1)

(N2)

(G3,N3)

(G4,N4)

	(G1,N1)	(N2)	(G3,N3)	(G4,N4)
42.70	386	483	493	514
44.80	360	453	478	506
47.68	319	423	447	470
50.69	307	397	430	447
53.82	290	376	398	425
56.72	264	356	367	421
59.78	248	342	352	403
63.75	219	302	309	352
65.97	164	245	260	303
70.00	178	247	257	322
72.73	184	295	261	315
76.72	165	384	236	292

TABLE A3.2

DRAIN M

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 3			
	(R1,B1)	(R2,B2)	(R3,B3)	(R4,B4)
0.00	282	282	265	265
0.02	1021	1021	1008	1003
0.04	1015	1030	1012	1014
0.06	997	1008	997	996
0.08	930	950	939	940
0.13	929	947	936	939
0.18	939	957	952	951
0.65	1014	1048	1041	1038
0.74	1008	1048	1039	1037
0.87	1001	1043	1038	1036
0.97	1002	1048	1043	1041
1.66	960	1055	1047	1045
1.97	950	1027	1022	1024
2.20	937	1023	1019	1019
2.67	917	1014	1012	1012
2.93	911	1014	1014	1013
3.73	888	1005	1008	1008
4.81	865	993	997	996
5.70	856	989	997	998
6.76	840	975	985	986
7.86	830	966	979	981
8.78	823	958	975	977
9.72	816	958	973	976
11.03	806	942	966	962
11.77	803	932	951	949
12.72	800	929	948	948
13.68	784	914	940	936
14.73	772	898	923	919
15.72	768	887	911	907
16.72	759	881	906	902
18.81	751	875	901	899
20.69	729	855	883	880
22.68	729	855	884	876
24.72	714	841	868	857
26.73	682	809	841	832
28.72	652	773	803	795
30.70	627	749	781	773
32.81	593	716	748	741
34.72	575	694	726	719
36.75	566	679	711	703
38.80	543	657	689	683
40.71	531	641	675	669

TABLE A3.2

DRAIN M

=====

TIME
(DAYS)

EXCESS PORE PRESSURES
U (MM)

LEVEL 3

(R1,B1) (R2,B2) (R3,B3) (R4,B4)

TIME (DAYS)	(R1,B1)	(R2,B2)	(R3,B3)	(R4,B4)
42.70	495	600	632	627
44.80	479	580	612	608
47.68	452	549	580	576
50.69	430	521	550	548
53.82	413	495	523	514
56.72	375	464	489	489
59.78	362	436	462	464
63.75	318	406	420	427
65.97	272	342	364	367
70.00	284	344	368	365
72.73	282	340	366	361
76.72	241	303	313	293

TABLE A3.3

DRAIN G

=====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 2			
	(N1)	(N2)	(N3)	(G4,N4)
0.00	190	197	177	155
0.02	966	965	954	941
0.04	919	917	908	920
0.06	885	886	878	886
0.09	848	843	831	844
0.11	833	845	833	851
0.15	893	889	874	883
0.19	904	896	871	876
0.29	924	918	893	894
0.37	911	918	901	903
0.81	947	951	939	933
0.92	963	972	961	953
1.10	961	964	951	951
1.87	892	914	902	906
2.96	878	907	893	892
3.85	882	930	921	918
4.08	879	909	915	900
4.90	852	898	890	887
6.02	828	882	876	878
6.93	835	898	885	891
7.87	817	879	875	879
9.16	804	845	843	846
9.91	801	867	865	871
10.87	795	859	862	860
11.83	778	843	846	845
12.87	772	833	854	836
13.86	766	824	829	832
14.86	755	811	829	817
16.96	747	812	831	816
18.84	732	798	818	802
20.83	701	768	795	772
22.87	643	711	726	722
24.87	617	691	695	696
26.86	598	667	669	678
28.84	574	652	643	650
30.96	552	599	594	606
32.85	526	579	563	566
34.88	516	561	552	552
37.02	501	556	552	535
38.85	496	536	513	513
40.87	456	497	472	477
42.94	430	465	446	446

TABLE A3.3

DRAIN G

=====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 2			
	(N1)	(N2)	(N3)	(G4,N4)
45.83	394	426	409	411
48.83	393	421	408	406
51.95	382	396	396	399
54.86	330	366	360	358
57.91	302	333	325	346
61.89	268	287	287	285
64.15	240	267	256	272
68.14	210	224	234	227

TABLE A3.3

DRAIN G
=====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)	LEVEL 3		
		(R1,B1)	(R2)	(R3) (R4,B4)
0.00	FAULTY	247	279	266
0.02		980	1102	1056
0.04		943	1026	1014
0.06		981	986	988
0.09		940	939	945
0.11		933	952	941
0.15		981	990	978
0.19		956	967	972
0.29		980	989	997
0.37		994	996	1000
0.81		1026	1031	1036
0.92		1048	1054	1060
1.10		1039	1045	1052
1.87		989	995	1003
2.96		976	989	995
3.85		986	1000	1012
4.08		994	1008	1005
4.90		965	977	984
6.02		949	968	973
6.93		958	980	985
7.87		944	966	971
9.16		913	937	940
9.91		931	957	961
10.87		922	947	952
11.83		908	945	937
12.87		898	929	932
13.86		892	935	923
14.86		893	927	912
16.96		888	923	912
18.84		883	900	905
20.83		877	897	881
22.87		872	831	847
24.87		794	836	837
26.86		782	826	823
28.84		774	794	817
30.96		730	760	791
32.85		702	730	747
34.88		685	712	738
37.02		672	708	728
38.85		657	687	694
40.87		613	646	650
42.94		585	618	625

TABLE A3.3

DRAIN G

=====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 3			
	(R1,B1)	(R2)	(R3)	(R4,B4)
45.83	546	580	590	
48.83	541	556	573	
51.95	529	536	547	
54.86	497	512	499	
57.91	465	471	468	
61.89	458	435	426	
64.15	371	384	401	
68.14	327	358	363	

TABLE A3.4

DRAIN A

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 2			
	(N1)	(G2,N2)	(G3 N3)	(N4)
0.00	179	151	161	163
0.01	961	948	936	851
0.02	1090	1084	1075	929
0.03	872	871	889	859
0.05	854	855	862	836
0.07	832	833	834	817
0.09	888	888	890	868
0.11	859	860	849	833
0.15	863	858	855	854
0.35	918	921	912	905
0.84	920	930	929	933
1.03	922	945	945	948
1.05	908	937	940	945
1.38	884	925	933	935
1.89	864	921	931	936
1.93	869	926	937	941
2.13	870	934	945	944
2.84	864	938	950	957
3.13	822	906	931	938
4.17	788	890	919	929
4.93	773	877	913	928
5.85	770	881	911	927
6.85	734	850	889	907
7.90	723	837	879	898
8.89	709	820	865	886
9.89	702	815	862	881
11.98	688	797	855	876
13.86	664	776	831	857
15.88	646	755	815	840
17.89	606	719	775	807
19.90	574	699	744	772
21.89	557	675	718	745
23.88	542	654	696	726
25.94	495	608	650	670
27.89	459	570	617	629
29.90	444	548	584	609
32.05	433	494	573	588
33.88	416	509	565	563
35.87	389	475	515	524
37.97	361	445	478	490
40.86	336	415	445	448
43.86	307	387	413	428

TABLE A3.4

DRAIN A

=====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 2			
	(N1)	(G2, N2)	(G3, N3)	(N4)
46.98	306	374	391	396
49.90	277	344	359	363
52.93	269	316	337	341
56.91	232	309	296	308
59.15	205	270	274	286
63.17	186	238	243	257
65.89	173	238	247	253
69.89	151	207	201	211

TABLE A3.4

DRAIN A

=====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 3			
	(R1,B1)	(R2,B2)	(B3)	(R4,B4)
0.00	259	270	296	255
0.01	1020	1018	1038	1015
0.02	1147	1154	1158	1149
0.03	961	980	980	971
0.05	945	955	950	946
0.07	945	956	975	949
0.09	980	993	989	985
0.11	915	934	934	927
0.15	949	964	955	955
0.35	988	1019	1037	1007
0.84	960	1044	1051	1029
1.03	958	1060	1060	1047
1.05	943	1054	1059	1041
1.38	913	1044	1053	1032
1.89	885	1037	1051	980
1.93	889	1042	1057	1039
2.13	892	1047	1060	1046
2.84	898	1054	1070	1056
3.13	858	1026	1043	1037
4.17	825	1007	1030	1027
4.93	814	994	1021	1023
5.85	812	994	1026	1026
6.85	777	970	1006	1006
7.90	771	957	996	999
8.89	757	940	984	985
9.89	751	935	981	983
11.98	744	929	976	982
13.86	729	911	960	964
15.88	717	895	946	951
17.89	686	863	914	920
19.90	660	835	888	893
21.89	647	813	864	871
23.88	635	797	851	858
25.94	590	747	798	806
27.89	560	712	760	770
29.90	545	696	744	754
32.05	531	679	727	736
33.88	516	656	706	716
35.87	492	623	673	681
37.97	468	608	643	651
40.86	444	570	606	613
43.86	417	540	569	584

TABLE A3.4

DRAIN A

TIME
(DAYS)

EXCESS PORE PRESSURES
U (MM)

LEVEL 3

(R1,B1) (R2,B2) (B3) (R4,B4)

TIME (DAYS)	(R1,B1)	(R2,B2)	(B3)	(R4,B4)
46.98	426	513	543	552
49.90	416	477	509	521
52.93	346	454	485	494
56.91	309	399	442	446
59.15	284	374	417	422
63.17	256	339	377	389
65.89	248	334	370	382
69.89	236	306	316	327

TABLE A4.1 : No Drain

TIME (DAYS)	SOIL SURFACE ELEVATION LEVEL 1 (MM)	EXCESS PORE PRESSURES U (MM)					
		LEVEL 2		LEVEL 3		LEVEL 4	
		P-G1	P-G2	P-R1	P-R2	P-R3	P-G3
0.00	984	291	292	541	515	562	527
0.00	980	1138	1159	1430	1430	1430	1367
0.02	979	1172	1164	1405	1398	1444	1378
0.04	977	1223	1207	1442	1439	1489	1461
0.08	977	1155	1138	1368	1363	1423	1390
0.28	974	1096	1083	1312	1310	1356	1310
0.78	969	1057	1044	1276	1272	1313	1271
1.00	969	1034	1019	1251	1248	1291	1250
1.42	967	992	987	1222	1216	1255	1212
1.78	966	951	959	1199	1193	1231	1187
2.03	965	916	935	1181	1173	1212	1167
2.32	964	898	914	1170	1162	1199	1155
2.82	962	854	885	1157	1146	1187	1141
3.88	960	771	804	1113	1101	1139	1091
4.39	959	742	767	1099	1086	1121	1074
4.83	958	724	743	1087	1075	1110	1062
5.38	957	706	722	1080	1065	1100	1055
5.87	955	696	707	1077	1062	1098	1047
6.34	954	676	691	1061	1050	1085	1036
6.83	953	660	671	1043	1038	1074	1027
7.34	952	649	655	1034	1030	1063	1020
7.85	951	638	640	1037	1025	1059	1009
8.86	949	617	616	1022	1009	1041	990
9.93	947	570	570	984	972	1004	943
11.05	945	544	537	950	939	970	908
11.98	943	521	511	922	910	940	876
13.24	941	492	484	889	870	899	833
15.25	939	432	419	786	782	910	740
16.51	936	405	389	744	740	768	701
18.40	935	374	352	674	672	700	639
20.27	932	320	304	597	593	610	553
22.26	931	243	237	498	472	506	444
23.28	929	219	205	453	441	462	418
26.29	927	134	117	288	258	271	235
29.22	925	73	62	167	135	145	124
34.24	924	14	7	59	58	62	47
37.48	923	9	14	82	80	89	79
49.38	921	-40	-45	-43	-46	-46	-49

TABLE A4.2

DRAIN M

=====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 2			
	P-G1	P-G2	P-G3	P-G4
0.00	133	192	186	169
0.01	178	1123	1089	1022
0.03	122	838	810	793
0.05	105	687	676	666
0.09	94	554	569	567
0.11	92	495	520	533
0.31	67	382	420	440
0.71	46	245	266	290
1.22	36	138	154	172
1.74	36	67	84	87
2.74	22	0	-4	-3
3.82	23	-18	-75	-76
4.93	24	-31	-94	-80
5.87	24	-34	-98	-84
7.13	22	-46	-107	-92
8.72	26	-66	-151	-138
9.98	25	-70	-117	-107
12.03	25	-59	-37	-49
12.05	29	551	-235	735
12.07	29	549	605	663
12.10	28	548	436	473
12.22	30	290	223	251
12.72	31	80	10	9
12.99	32	42	-7	-4
13.75	34	13	-32	-36
14.74	35	10	-36	-37
15.74	36	-27	-50	-74
16.76	41	-22	-63	-58
19.72	45	-78	-122	-117
19.75	46	-84	-134	-157
20.77	51	-83	-113	-110
21.75	-125	-54	-89	-97
21.78	25	952	1250	1186
22.06	26	331	324	408
22.97	25	166	91	104

TABLE A4.2

DRAIN M

=====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 2			
	P-N1	P-N2	P-N3	P-N4
0.00	169	223	210	196
0.01	666	1170	1116	1128
0.03	455	947	913	894
0.05	373	819	810	767
0.09	298	693	712	669
0.11	271	625	671	617
0.31	218	522	560	524
0.71	144	390	403	361
1.22	76	251	253	231
1.74	26	137	146	136
2.74	-18	32	30	26
3.82	-74	-61	-63	-65
4.93	-90	-79	-75	-84
5.87	-89	-78	-81	-83
7.13	-104	-90	-91	-102
8.72	-133	-135	-137	-130
9.98	-128	-118	-98	-108
12.03	-47	-41	-34	-39
12.05	384	798	763	742
12.07	284	682	711	660
12.10	196	492	540	513
12.22	97	277	304	307
12.72	-12	28	31	35
12.99	-19	9	9	14
13.75	-42	-31	-31	-31
14.74	-42	-30	-31	-30
15.74	-52	-88	-89	-77
16.76	-53	-61	-55	-62
19.72	-124	-118	-122	-116
19.75	-126	-164	-163	-165
20.77	-121	-114	-102	-124
21.75	-95	-84	-82	-100
21.78	610	1206	1190	1097
22.06	153	364	463	384
22.97	75	111	137	199

TABLE A4.2

DRAIN M
=====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVE L 3			
	P-R1	P-R2	P-R3	P-R4
0.00	133	133	173	194
0.01	168	703	1045	1164
0.03	122	403	747	930
0.05	107	329	601	790
0.09	95	262	488	633
0.11	93	235	438	571
0.31	65	173	333	448
0.71	24	100	221	307
1.22	4	54	135	196
1.74	-14	42	72	114
2.74	-38	-1	0	17
3.82	-76	-71	-72	-70
4.93	-83	-83	-79	-78
5.87	-84	-84	-73	-80
7.13	-92	-95	-79	-93
8.72	-130	-134	-131	-141
9.98	-110	-118	-111	-113
12.03	-37	-35	-34	-29
12.05	120	391	586	738
12.07	104	305	540	685
12.10	72	212	400	511
12.22	33	116	219	280
12.72	-22	7	32	22
12.99	-26	5	28	5
13.75	-44	-43	-9	-31
14.74	-44	-41	-11	-30
15.74	-85	-57	-75	-91
16.76	-60	-52	-70	-51
19.72	-110	-123	-121	-117
19.75	-126	-125	-128	-145
20.77	-96	-118	-110	-105
21.75	-80	-92	-87	-79
21.78	226	769	1258	1417
22.06	67	170	293	441
22.97	34	76	87	111

TABLE A4.2

DRAIN M

=====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 3			
	P-B1	P-B2	P-B3	P-B4
0.00	124	116	117	114
0.01	430	559	628	822
0.03	265	327	381	479
0.05	212	260	312	374
0.09	167	206	235	277
0.11	153	185	212	246
0.31	110	135	147	170
0.71	53	66	77	92
1.22	23	32	38	47
1.74	-6	-3	2	11
2.74	-38	-37	-35	-31
3.82	-79	-81	-82	-84
4.93	-86	-91	-85	-86
5.87	-85	-91	-87	-88
7.13	-102	-101	-95	-97
8.72	-133	-133	-133	-137
9.98	-129	-123	-105	-105
12.03	-51	-53	-35	-46
12.05	291	374	416	514
12.07	174	220	254	324
12.10	119	144	167	214
12.22	55	64	78	98
12.72	-21	-22	-13	-15
12.99	-27	-27	-19	-21
13.75	-46	-47	-37	-42
14.74	-46	-48	-37	-44
15.74	-88	-90	-80	-91
16.76	-75	-68	-59	-63
19.72	-123	-122	-109	-115
19.75	-141	-148	-149	-152
20.77	-114	-113	-99	-109
21.75	-90	-87	-82	-85
21.78	472	654	773	1139
22.06	99	119	132	176
22.97	36	40	43	51

TABLE A4.2

DRAIN M

=====

TIME (DAYS)	SOIL EXCESS PORE PRESSURES		
	SURFACE ELEVATION LEVEL 1 (MM)	U (MM)	
		LEVEL 4 P-RE	P-G5

0.00	934	97	122
0.01	925	160	853
0.03	924	128	601
0.05	923	119	458
0.09	921	110	357
0.11	920	106	322
0.31	917	69	248
0.71	914	39	158
1.22	912	20	102
1.74	911	2	53
2.74	910	-19	-8
3.82	910	-54	-78
4.93	909	-62	-84
5.87	909	-68	-85
7.13	910	-83	-98
8.72	909	-98	-143
9.98	908	-114	-112
12.03	908	-91	-37
12.05	904	17	410
12.07	904	20	378
12.10	905	20	294
12.22	904	21	174
12.72	904	15	1
12.99	903	11	-9
13.75	904	-2	-38
14.74	904	-10	-36
15.74	905	-24	-93
16.76	903	-32	-56
19.72	903	-80	-123
19.75	903	-82	-144
20.77	902	-88	-111
21.75	902	312	-94
21.78	896	231	1245
22.06	895	66	299
22.97	895	29	87

TABLE A4.3

DRAIN G
=====

TIME (DAYS) EXCESS PORE PRESSURES
 U (MM)

LEVEL 2

P-G1 P-G2 P-G3 P-G4

	P-G1	P-G2	P-G3	P-G4
0.00	320	348	328	316
0.00	798	1007	1100	1046
0.03	480	1120	1143	1110
0.05	439	1134	1143	1116
0.11	378	1098	1101	1077
0.31	321	1015	1062	1036
0.42	312	976	1038	1013
0.78	297	900	1007	990
1.03	261	844	971	940
1.32	246	819	956	921
1.74	241	793	936	898
2.79	222	686	856	791
3.30	209	620	811	734
3.74	200	581	777	696
4.31	191	522	714	635
4.76	187	479	660	594
5.22	171	336	533	530
5.72	145	385	530	488
6.22	122	342	469	439
6.74	98	298	407	378
7.74	48	218	304	299
8.82	20	119	167	167
9.95	-12	64	104	97
10.87	-35	29	56	106
12.13	-55	-10	9	-1
13.72	-86	-71	-60	-73
14.98	-104	-68	-86	-87
16.89	-94	-22	-71	-60
16.93	234	470	542	653
16.95	217	536	663	694
16.99	187	499	630	630
17.06	154	416	535	512
17.21	99	292	374	362
17.73	25	114	154	141
17.98	-2	85	113	95
18.74	-35	32	41	29
19.74	-50	22	8	3
20.73	-78	-50	-54	-68
21.75	-79	-20	-62	-74
24.75	-129	-111	-134	-134
24.77	-134	-14	-40	-64
25.76	-124	-91	-112	-117

TABLE A4.3

DRAIN G

=====

TIME (DAYS)	SOIL EXCESS PORE PRESSURES		
	SURFACE	U (MM)	
	ELEVATION LEVEL 1 (MM)	LEVEL 4 P-R5	P-G5
0.00	984	162	492
0.00	977	717	1343
0.03	976	608	1235
0.05	972	611	1209
0.11	971	608	1100
0.31	971	597	1001
0.42	971	591	965
0.78	967	575	904
1.03	966	563	853
1.32	963	550	828
1.74	962	526	807
2.79	955	486	711
3.30	954	465	670
3.74	952	444	640
4.31	950	421	604
4.76	950	403	575
5.22	947	1700	1704
5.72	947	366	511
6.22	945	349	478
6.74	945	332	437
7.74	943	303	381
8.82	943	265	276
9.95	941	236	175
10.87	939	208	111
12.13	939	168	42
13.72	938	116	-43
14.98	938	80	-79
16.89	938	-151	-68
16.93	935	-124	435
16.95	935	-123	428
16.99	935	-123	404
17.06	935	-126	377
17.21	934	-128	326
17.73	934	-138	161
17.98	934	-143	122
18.74	933	-158	64
19.74	934	-934	55
20.73	934	-934	-16
21.75	934	-934	-27
24.75	932	-932	-107
24.77	932	-932	-114
25.76	931	-931	-122

TABLE A4.3

DRAIN G

=====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 3			
	P-B1	P-B2	P-B3	P-B4
0.00	483	508	531	527
0.00	341	1216	1334	1313
0.03	286	1134	1322	1329
0.05	273	1112	1322	1328
0.11	239	1035	1271	1290
0.31	222	942	1214	1257
0.42	208	911	1179	1235
0.78	200	843	1117	1207
1.03	175	790	1060	1167
1.32	171	760	1024	1143
1.74	168	738	989	1123
2.79	157	649	882	1027
3.30	153	609	833	978
3.74	149	585	799	947
4.31	148	550	749	888
4.76	144	514	708	839
5.22	141	479	654	780
5.72	128	440	601	716
6.22	119	404	549	653
6.74	103	360	493	586
7.74	73	285	385	459
8.82	45	180	251	304
9.95	15	108	157	205
10.87	-5	71	106	135
12.13	-26	22	47	65
13.72	-64	-53	-42	-35
14.98	-74	-62	-57	-48
16.89	-67	-41	-36	-26
16.93	64	288	383	483
16.95	77	362	419	522
16.99	78	369	431	521
17.06	92	321	413	489
17.21	88	261	353	425
17.73	36	133	182	241
17.98	21	105	145	193
18.74	-3	49	75	99
19.74	-934	-934	-934	-934
20.73	-54	-41	-28	-13
21.75	-59	-31	-14	-3
24.75	-122	-122	-119	-125
24.77	-116	-127	-119	-122
25.76	-92	-92	-92	-101

TABLE A4.4

DRAIN A
=====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 2			
	P-G1	P-G2	P-G3	P-G4
0.00	276	323	285	248
0.01	736	1188	1182	1158
0.03	497	1038	1022	1010
0.05	417	932	913	900
0.09	363	838	841	822
0.14	351	793	806	770
0.34	326	711	744	703
0.78	298	629	678	615
1.26	204	499	553	501
1.75	165	428	456	448
2.26	135	361	379	393
2.78	111	298	313	330
3.77	65	188	199	215
4.85	53	76	64	52
5.97	38	23	9	29
6.91	36	-6	-5	-9
8.17	37	-38	-47	-44
9.75	38	-96	-95	-104
11.02	36	-101	-96	-104
12.99	35	-38	-61	-42
13.03	54	390	525	545
13.05	56	393	226	484
13.08	54	349	492	422
13.12	57	298	444	373
13.25	55	201	314	262
13.76	56	53	80	76
14.02	56	38	67	49
14.78	59	1	28	3
15.78	59	4	21	4
16.77	108	-34	-39	-40
17.79	62	-49	-42	-39
20.78	54	-144	-137	-136
20.79	55	-140	-136	-136
21.81	56	-139	-127	-121

TABLE A4.4

DRAIN A
 =====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 2			
	P-N1	P-N2	P-N3	P-N4
0.00	276	323	312	231
0.01	611	1221	1206	1109
0.03	440	1076	1077	959
0.05	376	991	999	850
0.09	355	920	933	758
0.14	310	879	896	703
0.34	291	816	842	622
0.78	268	743	786	560
1.26	171	619	677	444
1.75	141	541	609	398
2.26	114	460	536	383
2.78	96	394	459	319
3.77	58	265	313	219
4.85	9	117	153	94
5.97	-15	34	58	56
6.91	-28	24	9	-8
8.17	-55	362	-41	-48
9.75	-83	-71	-102	-87
11.02	-95	-84	-99	-83
12.99	-66	-62	-45	-55
13.03	164	551	591	573
13.05	167	514	574	556
13.08	167	468	518	465
13.12	164	405	457	407
13.25	120	270	309	271
13.76	35	72	83	66
14.02	25	55	65	44
14.78	-4	4	15	12
15.78	-4	5	16	11
16.77	-54	-62	-68	-25
17.79	-58	-51	-45	-28
20.78	-121	-130	-143	-132
20.79	-122	-131	-135	-130
21.81	-118	-113	-124	-124

TABLE A4.4

DRAIN A
 ==#====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 3			
	P-R1	P-R2	P-R3	P-R4
0.00	263	278	307	218
0.01	339	858	1128	1011
0.03	281	642	983	876
0.05	234	1703	849	742
0.09	213	449	742	641
0.14	205	431	669	578
0.34	204	359	562	499
0.78	196	323	494	461
1.26	77	209	375	349
1.75	54	196	313	307
2.26	44	173	260	261
2.78	35	151	216	222
3.77	18	102	134	138
4.85	-11	17	44	39
5.97	-22	-6	-7	-3
6.91	-32	-22	-40	-21
8.17	-56	-46	-78	-56
9.75	-99	-97	-126	-107
11.02	-105	-105	-119	-102
12.99	-54	-44	-49	-39
13.03	83	201	302	399
13.05	85	179	291	370
13.08	77	158	263	332
13.12	71	142	237	298
13.25	55	117	170	194
13.76	9	35	40	40
14.02	1	26	21	29
14.78	-16	12	-16	-5
15.78	-14	-6	-14	-2
16.77	-31	-61	-70	-65
17.79	-37	-47	-41	-44
20.78	-135	-132	-130	-128
20.79	-134	-155	-160	-152
21.81	-133	-122	-121	-121

TABLE A4.4

DRAIN A

=====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 3			
	P-R1	P-R2	P-R3	P-R4
0.00	246	269	266	202
0.01	969	1099	1180	1029
0.03	766	920	995	846
0.05	649	801	882	736
0.09	556	695	775	639
0.14	510	630	709	591
0.34	447	535	620	514
0.78	398	468	534	449
1.26	300	363	404	342
1.75	246	312	344	307
2.26	218	266	275	263
2.78	182	223	247	224
3.77	141	141	152	137
4.85	47	44	44	38
5.97	31	7	5	-2
6.91	19	-20	-19	-26
8.17	-8	-56	-54	-62
9.75	-63	-105	-107	-108
11.02	-82	-108	-100	-114
12.99	-70	-42	-30	-69
13.03	247	331	361	248
13.05	242	286	324	251
13.08	233	244	286	252
13.12	218	214	248	249
13.25	181	147	167	220
13.76	76	32	36	65
14.02	71	24	27	51
14.78	58	-5	-3	19
15.78	41	-2	-1	6
16.77	27	-64	-65	-29
17.79	-19	-44	-35	-46
20.78	-100	-132	-128	-142
20.79	-100	-145	-151	-140
21.81	-112	-125	-118	-149

TABLE A4.4

DRAIN A

=====

TIME SOIL EXCESS PORE PRESSURES
(DAYS) SURFACE U (MM)
ELEVATION
LEVEL 1 LEVEL 4
(MM) P-R5 P-G5

TIME (DAYS)	SOIL SURFACE ELEVATION (MM)	EXCESS PORE PRESSURE U (MM) LEVEL 1 P-R5	EXCESS PORE PRESSURE U (MM) LEVEL 4 P-G5
0.00	968	268	292
0.01	958	296	701
0.03	957	271	570
0.05	956	224	490
0.09	955	215	423
0.14	950	206	396
0.34	951	189	363
0.78	947	188	333
1.26	942	113	246
1.75	940	108	208
2.26	938	103	189
2.78	936	95	164
3.77	934	87	111
4.85	932	75	40
5.97	931	41	5
6.91	931	1	-17
8.17	930	-40	-49
9.75	930	-80	-103
11.02	930	-86	-100
12.99	929	-79	-45
13.03	927	13	175
13.05	926	18	182
13.08	927	23	182
13.12	926	26	173
13.25	926	29	138
13.76	926	28	40
14.02	926	23	29
14.78	925	3	-4
15.78	925	3	1
16.77	925	-11	-60
17.79	925	-36	-43
20.78	925	-113	-132
20.79	924	-113	-134
21.81	924	-110	-122

TABLE A5.1

SETTLEMENT DATA - TEST NO.2

NO DRAIN

TIME (DAYS)	SETTLEMENT (MM)			STRAIN (%)		
	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)
0.00	0	0	0	0	0	0
0.00	4	1	-1	4	2	-2
0.02	5	1	1	5	2	2
0.04	7	2	1	7	3	3
0.08	8	1	2	8	2	5
0.28	11	1	2	11	2	7
0.78	15	1	1	15	2	2
1.00	15	2	-2	15	3	-5
1.42	17	3	-1	17	5	-2
1.78	18	3	-1	18	5	-3
2.03	19	3	-1	19	5	-2
2.32	20	4	1	20	7	2
2.82	22	5	1	22	8	2
3.88	25	5	1	25	8	2
4.39	25	6	1	25	10	3
4.83	27	8	1	27	13	3
5.38	28	10	1	28	17	3
5.87	30	10	2	30	17	5
6.34	31	12	2	31	20	7
6.83	31	12	3	31	20	8
7.34	33	14	4	33	23	12
7.85	34	15	4	34	25	12
8.86	35	17	5	35	29	17
9.93	38	18	5	38	30	15
11.05	40	20	5	40	33	17
11.98	42	22	7	42	37	23
13.24	43	25	8	43	42	27
15.25	46	26	9	46	43	28
16.51	49	28	12	49	47	40
18.40	50	31	13	50	52	42
20.27	52	34	14	52	57	45
21.26	54	35	15	54	59	50
23.28	55	36	16	55	60	52
26.29	58	39	18	58	65	58
29.22	59	39	17	59	65	57
34.24	61	41	19	61	68	62
37.48	62	41	19	62	68	63
49.38	63	44	23	63	73	75

TABLE A5.2

SETTLEMENT DATA - TEST NO.2

DRAIN M

TIME (DAYS)	SETTLEMENT (MM)			STRAIN (%)		
	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)
0.00	0	0	0	0	0	0
0.01	9	4	3	9	6	10
0.03	11	3	2	11	5	5
0.05	11	3	2	11	5	5
0.09	13	4	2	13	6	5
0.11	15	4	2	15	7	5
0.31	17	5	2	17	8	7
1.34	20	6	2	20	10	5
1.22	22	8	3	22	13	8
1.74	23	8	3	23	13	8
2.74	24	9	3	24	15	10
3.82	25	9	2	25	15	7
4.93	25	10	2	25	16	7
5.87	25	10	3	25	17	10
7.13	24	10	3	24	17	10
8.72	26	10	2	26	17	7
9.98	27	12	4	27	20	13
12.03	26	12	4	26	19	13
12.05	30	14	6	30	23	20
12.07	30	14	5	30	23	17
12.10	29	14	5	29	23	17
12.22	31	14	4	31	23	13
12.72	31	14	5	31	23	15
12.99	31	14	4	31	23	13
13.75	31	14	5	31	23	15
14.74	31	13	4	31	22	13
15.74	30	14	4	30	23	13
16.76	32	15	5	32	24	17
19.72	31	15	5	31	24	17
19.75	32	16	6	32	27	20
20.77	32	17	7	32	28	23
21.75	33	16	5	33	26	15
21.78	39	18	5	39	30	15
22.06	40	18	5	40	30	15
22.97	39	19	4	39	31	13

TABLE A5.3

SETTLEMENT DATA - TEST NO.2

DRAIN G

TIME (DAYS)	SETTLEMENT (MM)			STRAIN (%)		
	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)
0.00	0	0	0	0	0	0
0.00	7	3	2	7	4	5
0.03	8	3	1	8	4	3
0.05	12	3	0	12	5	0
0.11	13	4	1	13	6	2
0.31	14	5	-2	14	8	-5
0.42	14	5	-1	14	8	-2
0.78	17	6	-1	17	9	-3
1.03	19	6	0	19	10	0
1.32	21	8	4	21	13	12
1.74	22	9	0	22	14	0
2.79	29	12	1	29	19	3
3.30	31	13	3	31	22	8
3.74	33	15	1	33	24	3
4.31	35	17	2	35	28	5
4.76	35	18	2	35	29	5
5.22	38	19	3	38	32	10
5.72	38	20	3	38	33	10
6.22	39	21	3	39	35	10
6.74	39	22	4	39	36	12
7.74	41	23	5	41	38	15
8.82	42	24	4	42	40	12
9.95	44	25	5	44	42	15
10.87	45	26	6	45	43	20
12.13	46	27	7	46	44	22
13.72	46	27	5	46	44	17
14.98	47	28	7	47	47	22
16.89	47	28	7	47	47	22
16.93	49	30	7	49	50	23
16.95	50	30	7	50	49	22
16.99	50	30	7	50	49	23
17.06	50	30	7	50	49	23
17.21	50	30	8	50	49	25
17.73	51	31	8	51	51	27
17.98	50	31	8	50	51	25
18.74	51	31	8	51	52	25
19.74	51	31	8	51	51	25
20.73	51	31	7	51	51	22
21.75	51	32	7	51	53	23
24.75	52	32	8	52	53	27
24.77	53	34	10	53	56	32
25.76	53	32	9	53	53	27

TABLE A5.4

SETTLEMENT DATA - TEST NO.2 DRAIN A

TIME (DAYS)	SETTLEMENT (MM)			STRAIN (%)		
	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)	LEVEL 1 (SURFACE)	LEVEL 2 (600 MM)	LEVEL 3 (300 MM)
0.00	0	0	0	0	0	0
0.01	10	4	2	10	7	7
0.03	11	4	2	11	7	7
0.05	12	4	3	12	7	10
0.09	18	4	2	13	7	7
0.14	18	5	4	18	8	12
0.34	17	6	3	17	10	10
0.78	21	8	4	21	13	12
1.26	26	10	5	26	16	15
1.75	28	10	4	28	17	13
2.26	30	12	5	30	19	15
2.78	32	13	6	32	21	18
3.77	34	14	5	34	23	15
4.85	36	14	4	36	23	13
5.97	37	15	5	37	24	15
6.91	37	15	5	37	24	15
8.17	38	17	7	38	28	22
9.75	38	17	6	38	28	18
11.02	38	19	8	38	31	25
12.99	39	19	8	39	31	25
13.03	41	19	10	41	32	32
13.05	42	19	8	42	32	27
13.08	41	19	9	41	32	28
13.12	42	19	10	42	32	32
13.25	42	19	9	42	32	28
13.76	42	20	9	42	33	30
14.02	42	20	9	42	33	28
14.78	43	20	10	43	33	32
15.78	43	20	9	43	33	30
16.77	43	20	9	43	33	30
17.79	43	20	10	43	33	32
20.78	43	21	10	43	34	33
20.79	44	21	10	44	35	33
21.81	44	21	10	44	35	33

TABLE A6.1

NO DRAIN

=====

TIME (DAYS)	SOIL SURFACE ELEVATION	EXCESS PORE PRESSURES U (MM)			
		LEVEL 1	LEVEL 2	LEVEL 3	LEVEL 4
		(MM)	(G1,G2)	(R1,R2)	(R3,G3)
0.00	984	291	528	545	
0.00	980	1148	1430	1398	
0.02	979	1168	1401	1411	
0.04	977	1215	1440	1475	
0.08	977	1146	1365	1401	
0.28	974	1090	1311	1333	
0.78	969	1050	1274	1292	
1.00	969	1026	1249	1270	
1.42	967	989	1219	1233	
1.78	966	955	1196	1209	
2.03	965	926	1177	1189	
2.32	964	906	1166	1177	
2.82	962	869	1152	1164	
3.88	960	787	1107	1115	
4.39	959	754	1093	1097	
4.83	958	733	1081	1086	
5.38	957	714	1072	1077	
5.87	955	701	1069	1072	
6.34	954	683	1055	1060	
6.83	953	665	1046	1050	
7.34	952	652	1032	1041	
7.85	951	639	1031	1033	
8.86	949	616	1015	1015	
9.93	947	570	978	973	
11.05	945	540	944	939	
11.98	943	516	916	908	
13.24	941	488	879	866	
15.25	939	425	784	775	
16.51	936	397	742	734	
18.40	935	363	673	670	
20.27	932	312	590	581	
22.26	931	240	485	475	
23.28	929	212	447	440	
26.29	927	126	273	253	
29.22	925	67	151	134	
34.24	924	10	63	54	
37.48	923	11	81	84	
49.38	921	-43	-45	-48	

TABLE A6.2

DRAIN M
=====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 2			
	(G1,N1)	(G2,N2)	(G3,N3)	(G4,N4)
0.00	151	208	198	183
0.01	422	1147	1103	1075
0.03	288	892	861	843
0.05	239	753	743	716
0.09	196	624	640	618
0.11	181	560	595	575
0.31	143	452	490	482
0.71	95	317	334	326
1.22	56	194	203	202
1.74	31	102	115	112
2.74	2	16	13	12
3.82	-26	-39	-69	-70
4.93	-33	-55	-85	-82
5.87	-32	-56	-89	-84
7.13	-41	-68	-99	-97
8.72	-54	-100	-144	-134
9.98	-52	-94	-107	-108
12.03	-11	-50	-36	-44
12.05	207	674	264	739
12.07	157	615	658	662
12.10	112	520	488	493
12.22	63	283	263	279
12.72	10	54	20	22
12.99	7	26	1	5
13.75	-4	-9	-31	-33
14.74	-4	-10	-33	-33
15.74	-8	-55	-69	-75
16.76	-6	-41	-59	-60
19.72	-40	-98	-122	-117
19.75	-40	-124	-148	-161
20.77	-35	-98	-108	-117
21.75	-110	-69	-85	-93
21.78	317	1079	1220	1141
22.06	89	347	393	546
22.97	50	139	114	152

TABLE A6.2

DRAIN M

=====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	(R1,B1)	(R2,B2)	(R3,B3)	(R4,B4)
0.00	128	124	145	154
0.01	299	631	837	993
0.03	193	365	564	704
0.05	160	295	457	582
0.09	131	234	362	455
0.11	123	210	325	408
0.31	88	154	240	309
0.71	39	83	149	200
1.22	14	43	87	122
1.74	-10	20	37	63
2.74	-38	-19	-18	-7
3.82	-77	-76	-77	-77
4.93	-84	-87	-82	-82
5.87	-85	-87	-80	-84
7.13	-97	-98	-87	-95
8.72	-131	-133	-132	-139
9.98	-119	-120	-108	-109
12.03	-44	-44	-34	-37
12.05	206	383	501	626
12.07	139	262	397	505
12.10	96	178	284	362
12.22	44	90	148	189
12.72	-21	-8	9	3
12.99	-27	-11	5	-8
13.75	-45	-45	-23	-36
14.74	-45	-44	-24	-37
15.74	-86	-73	-77	-91
16.76	-67	-60	-64	-57
19.72	-117	-123	-115	-116
19.75	-133	-136	-138	-148
20.77	-105	-115	-104	-107
21.75	-85	-89	-84	-82
21.78	349	711	1015	1278
22.06	83	144	212	308
22.97	35	58	65	81

TABLE A6.3

DRAIN G

=====

TIME (DAYS)	EXCESS PORE PRESSURES U (MM)			
	LEVEL 2			
	(G1)	(G2,N2)	(G3,N3)	(G4,N4)
0.00	320	329	341	335
0.00	798	1035	1123	1096
0.03	480	1107	1144	1127
0.05	439	1109	1143	1130
0.11	378	1047	1103	1091
0.31	321	938	1064	1051
0.42	312	896	1041	1028
0.78	297	819	1006	992
1.03	261	763	968	953
1.32	246	736	953	935
1.74	241	709	933	914
2.79	222	606	834	801
3.30	209	545	777	738
3.74	200	510	738	697
4.31	191	462	673	634
4.76	187	425	621	588
5.22	171	337	533	531
5.72	145	343	505	484
6.22	122	305	450	435
6.74	98	265	393	378
7.74	48	192	294	292
8.82	20	107	164	164
9.95	-12	56	97	94
10.87	-35	24	51	77
12.13	-55	-13	6	1
13.72	-86	-71	-67	-73
14.98	-104	-72	-73	-74
16.89	-94	-40	-42	-37
16.93	234	479	615	670
16.95	217	517	688	703
16.99	187	467	631	631
17.06	154	383	527	516
17.21	99	265	369	363
17.73	25	101	150	144
17.98	-2	73	113	104
18.74	-35	26	45	39
19.74	-50	13	22	20
20.73	-78	-47	-50	-57
21.75	-79	-32	-38	-43
24.75	-129	-111	-132	-133
24.77	-134	-6	-10	-22
25.76	-124	-94	-89	-91

TABLE A6.3

DRAIN 6

=====

TIME
(DAYS)

EXCESS PORE PRESSURES
U (MM)

LEVEL 3

(R1,B1) (R2,B2) (R3,B3) (R4,B4)

TIME (DAYS)	(R1,B1)	(R2,B2)	(R3,B3)	(R4,B4)
0.00	484	493	510	524
0.00	336	1137	1226	1283
0.03	276	1120	1288	1320
0.05	262	1093	1291	1323
0.11	239	1007	1232	1282
0.31	226	908	1150	1243
0.42	216	874	1112	1224
0.78	207	805	1039	1193
1.03	170	753	980	1153
1.32	161	725	944	1128
1.74	155	706	914	1107
2.79	149	628	823	1010
3.30	145	591	769	960
3.74	142	567	737	924
4.31	139	531	689	865
4.76	136	499	647	815
5.22	133	466	598	755
5.72	124	430	548	693
6.22	117	395	499	630
6.74	107	357	448	567
7.74	88	281	346	439
8.82	69	178	218	282
9.95	45	109	133	189
10.87	23	64	84	120
12.13	-6	17	30	53
13.72	-45	-56	-53	-41
14.98	-66	-63	-58	-57
16.89	-68	-44	-22	-31
16.93	79	297	400	474
16.95	85	362	450	516
16.99	86	353	439	505
17.06	91	313	401	472
17.21	87	260	333	408
17.73	55	131	167	219
17.98	44	102	134	181
18.74	10	49	68	95
19.74	-472	-447	-440	-433
20.73	-50	-33	-31	-15
21.75	-63	-32	-13	-11
24.75	-128	-119	-115	-124
24.77	-124	-121	-110	-122
25.76	-98	-99	-89	-105

TABLE A6.4

DRAIN A
=====

TIME
(DAYS)

EXCESS PORE PRESSURES
U (MM)

LEVEL 2

(G1,N1) (G2,N2) (G3,N3) (G4,N4)

TIME (DAYS)	(G1,N1)	(G2,N2)	(G3,N3)	(G4,N4)
0.00	276	323	298	239
0.01	673	1204	1194	1133
0.03	468	1057	1049	984
0.05	396	961	956	875
0.09	359	879	887	790
0.14	330	836	851	737
0.34	308	763	793	662
0.78	283	686	732	587
1.26	187	559	615	472
1.75	153	484	532	423
2.26	124	410	457	388
2.78	104	346	386	324
3.77	61	226	256	217
4.85	31	97	108	93
5.97	12	28	33	42
6.91	4	9	2	-9
8.17	-9	162	-44	-46
9.75	-22	-83	-98	-95
11.02	-29	-92	-97	-93
12.99	-16	-50	-53	-48
13.03	109	470	558	569
13.05	111	453	400	520
13.08	110	409	505	443
13.12	110	352	451	390
13.25	87	235	311	266
13.76	45	62	81	71
14.02	40	46	66	46
14.78	27	3	22	8
15.78	27	5	18	7
16.77	27	-48	-54	-32
17.79	2	-49	-43	-34
20.78	-34	-137	-140	-134
20.79	-34	-135	-135	-133
21.81	-31	-126	-125	-123

TABLE A6.4

DRAIN A

=====

TIME
(DAYS)

EXCESS PORE PRESSURES
U (MM)

LEVEL 3
(R1,B1) (R2,B2) (R3,B3) (R4,B4)

0.00	254	273	286	210
0.01	654	978	1154	1020
0.03	523	781	989	861
0.05	441	1252	866	739
0.09	384	572	759	640
0.14	358	531	689	585
0.34	325	447	591	507
0.78	297	396	514	455
1.26	188	286	390	345
1.75	150	254	329	307
2.26	131	219	267	262
2.78	109	187	231	223
3.77	79	121	143	137
4.85	18	31	44	39
5.97	5	1	-1	-2
6.91	-7	-21	-29	-24
8.17	-32	-51	-66	-59
9.75	-81	-101	-116	-107
11.02	-93	-106	-109	-108
12.99	-62	-43	-40	-54
13.03	165	266	331	323
13.05	163	233	307	310
13.08	155	201	275	292
13.12	144	178	242	273
13.25	118	132	168	207
13.76	42	33	38	52
14.02	36	25	24	40
14.78	21	4	-9	7
15.78	14	-4	-8	2
16.77	-2	-63	-68	-47
17.79	-28	-45	-39	-45
20.78	-117	-132	-129	-135
20.79	-117	-150	-156	-146
21.81	-122	-123	-120	-135

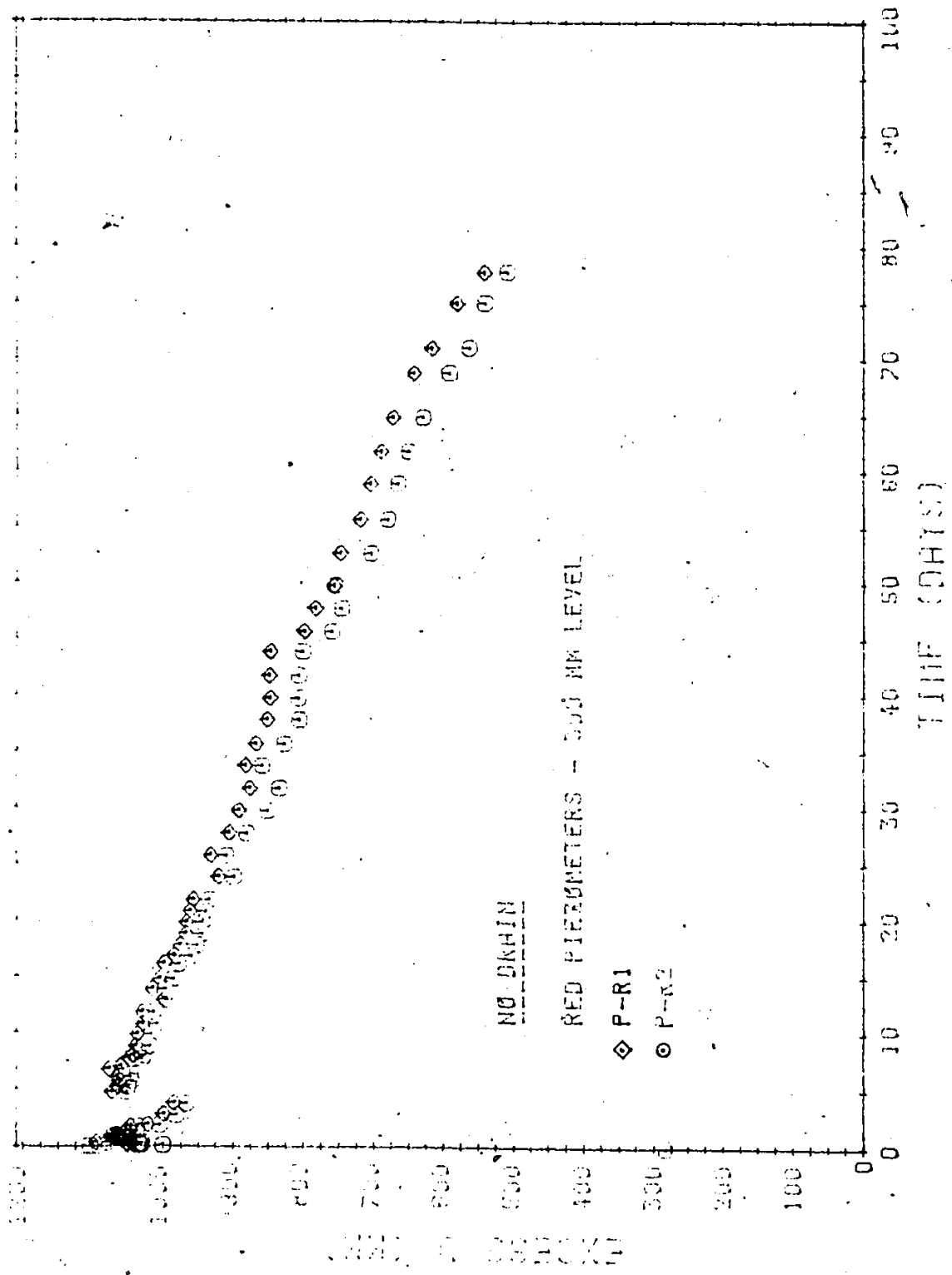


FIG. A1.1 : EXCESS PORE PRESSURE - TEST SERIES ONE

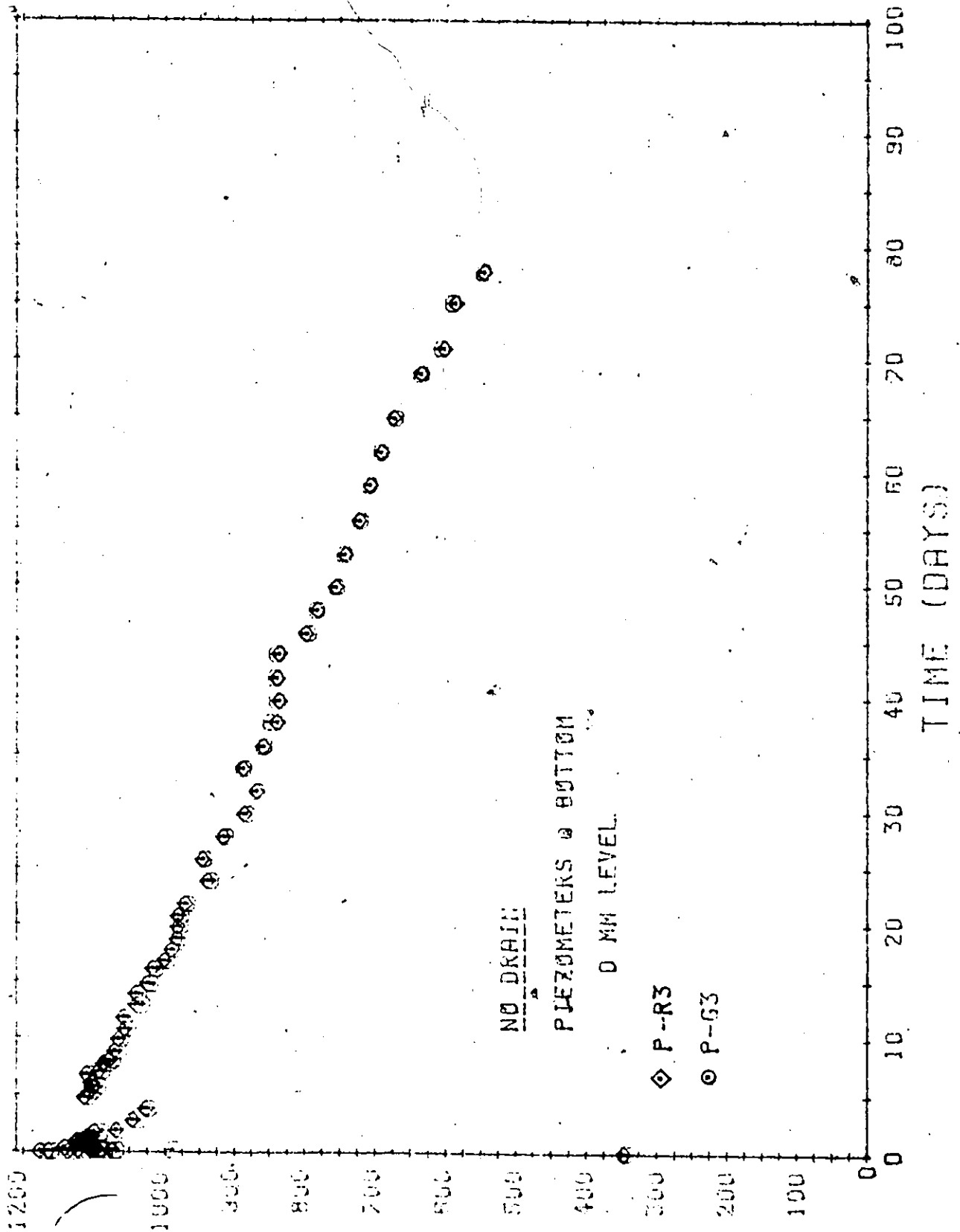


FIG. A1.1 : EXCESS PORE PRESSURE - TEST SERIES ONE

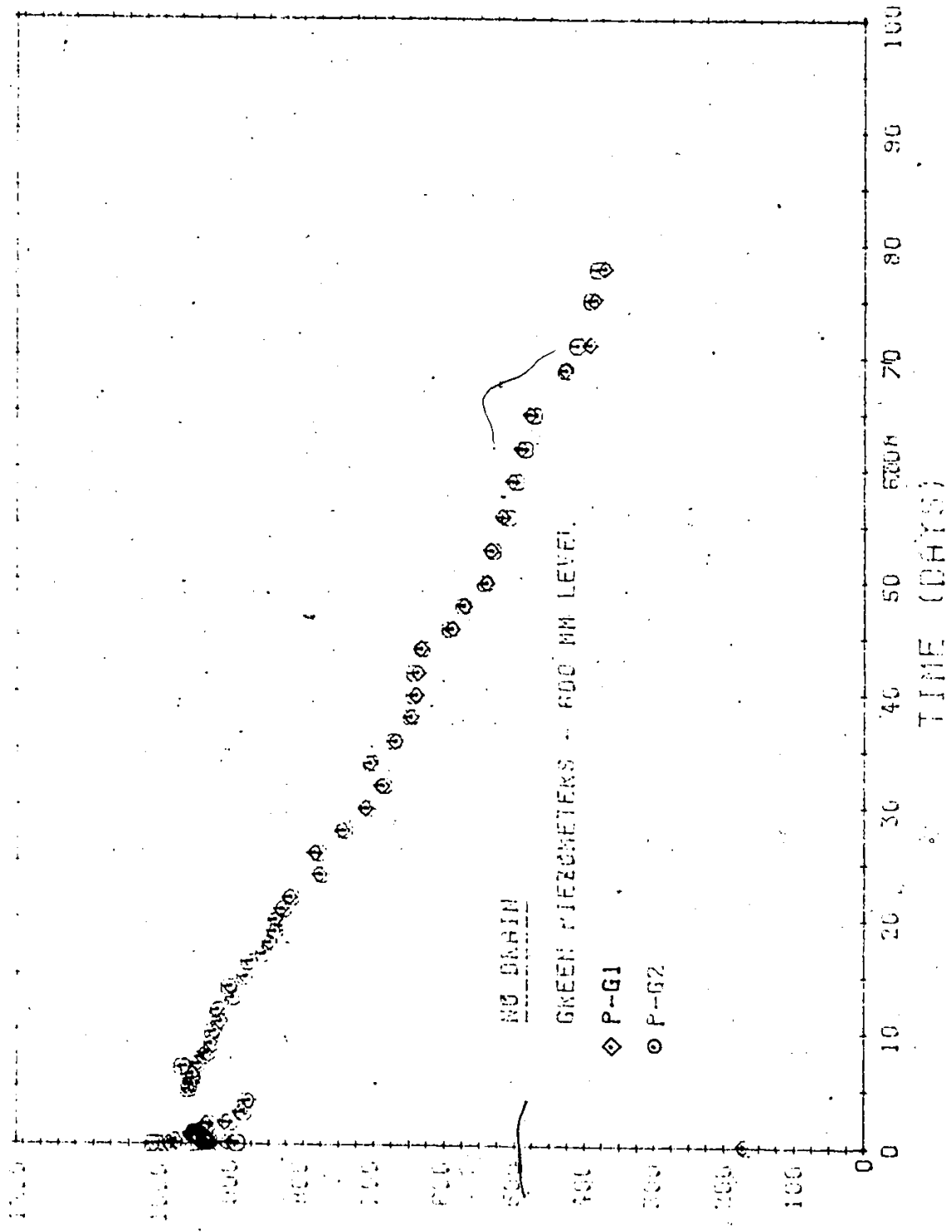


FIG. A1.1 : EXCESS PORE PRESSURE - TEST SERIES ONE

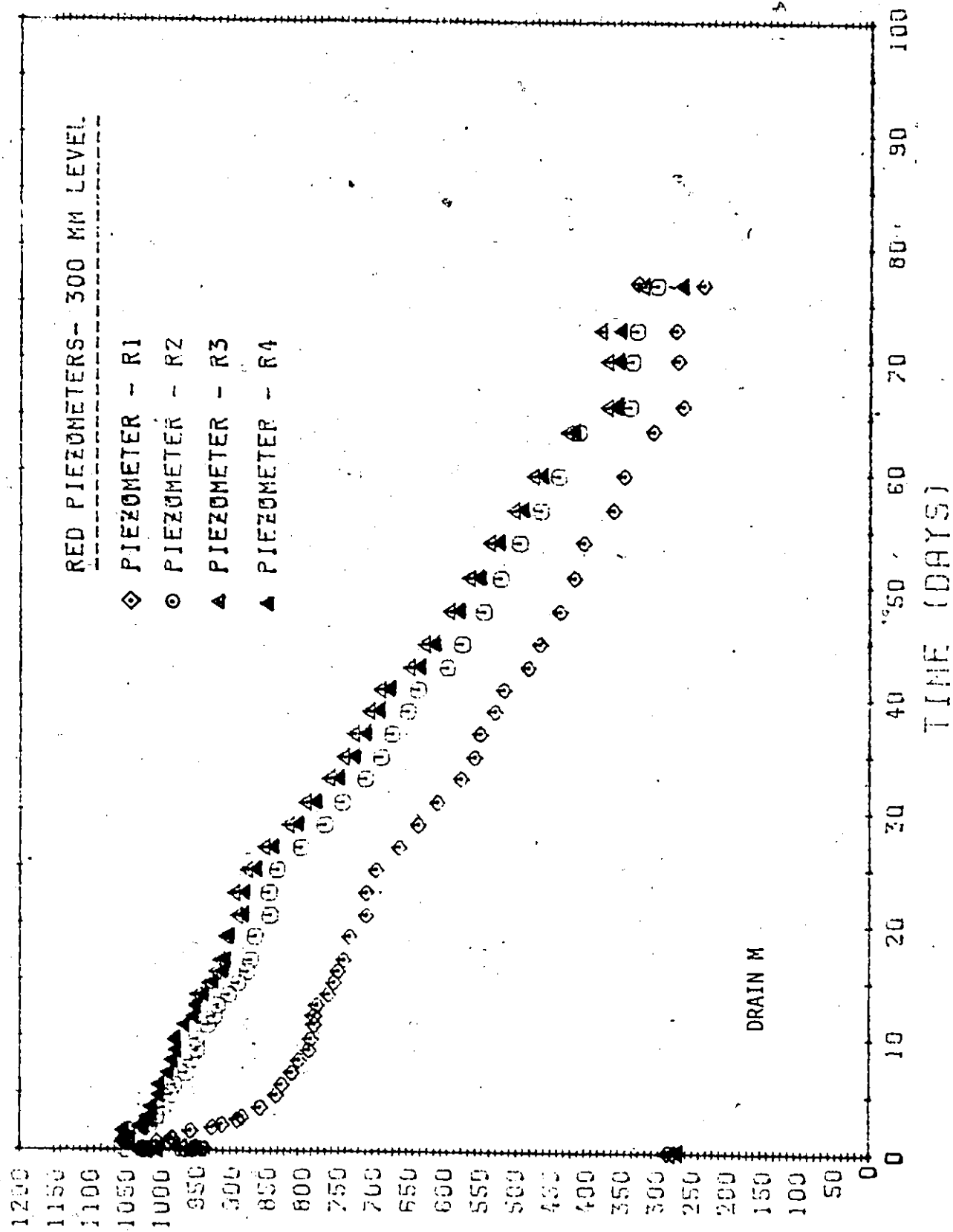


FIG. A1.2 : EXCESS PORE PRESSURE - TEST SERIES ONE

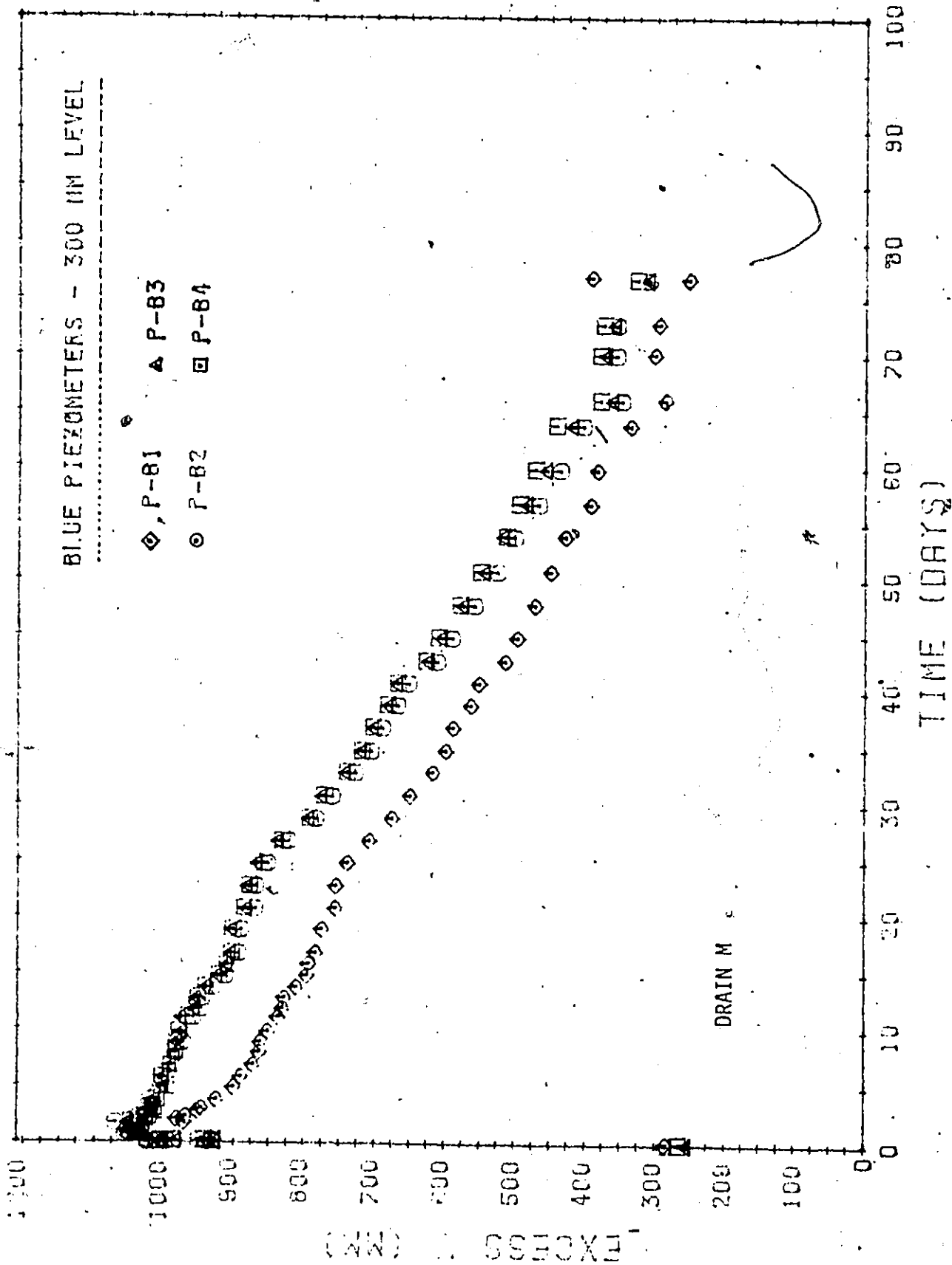


FIG. A1.2 : EXCESS PORE PRESSURE - TEST SERIES ONE

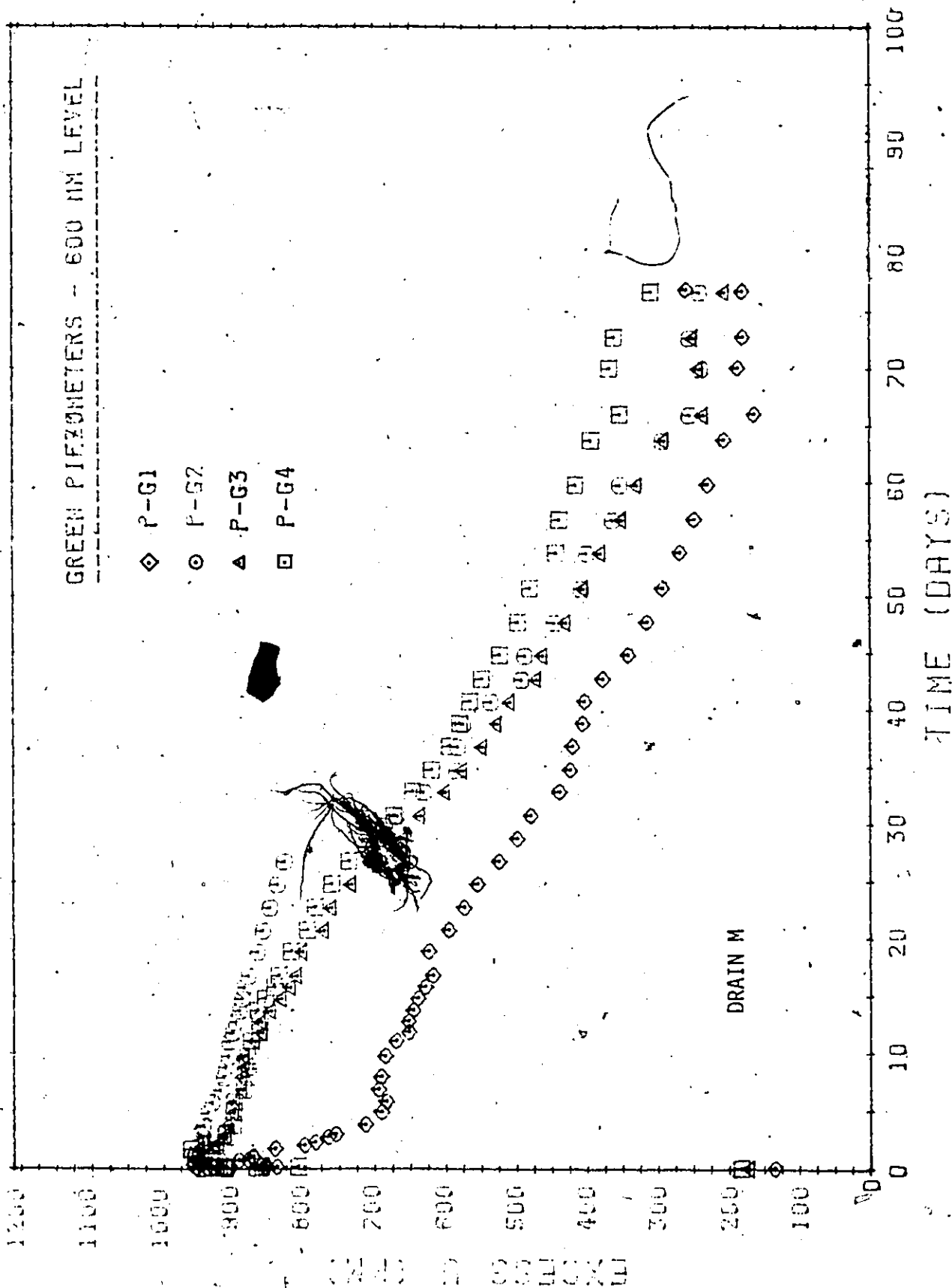


FIG. A1.2 : EXCESS PORE PRESSURE - TEST SERIES ONE.

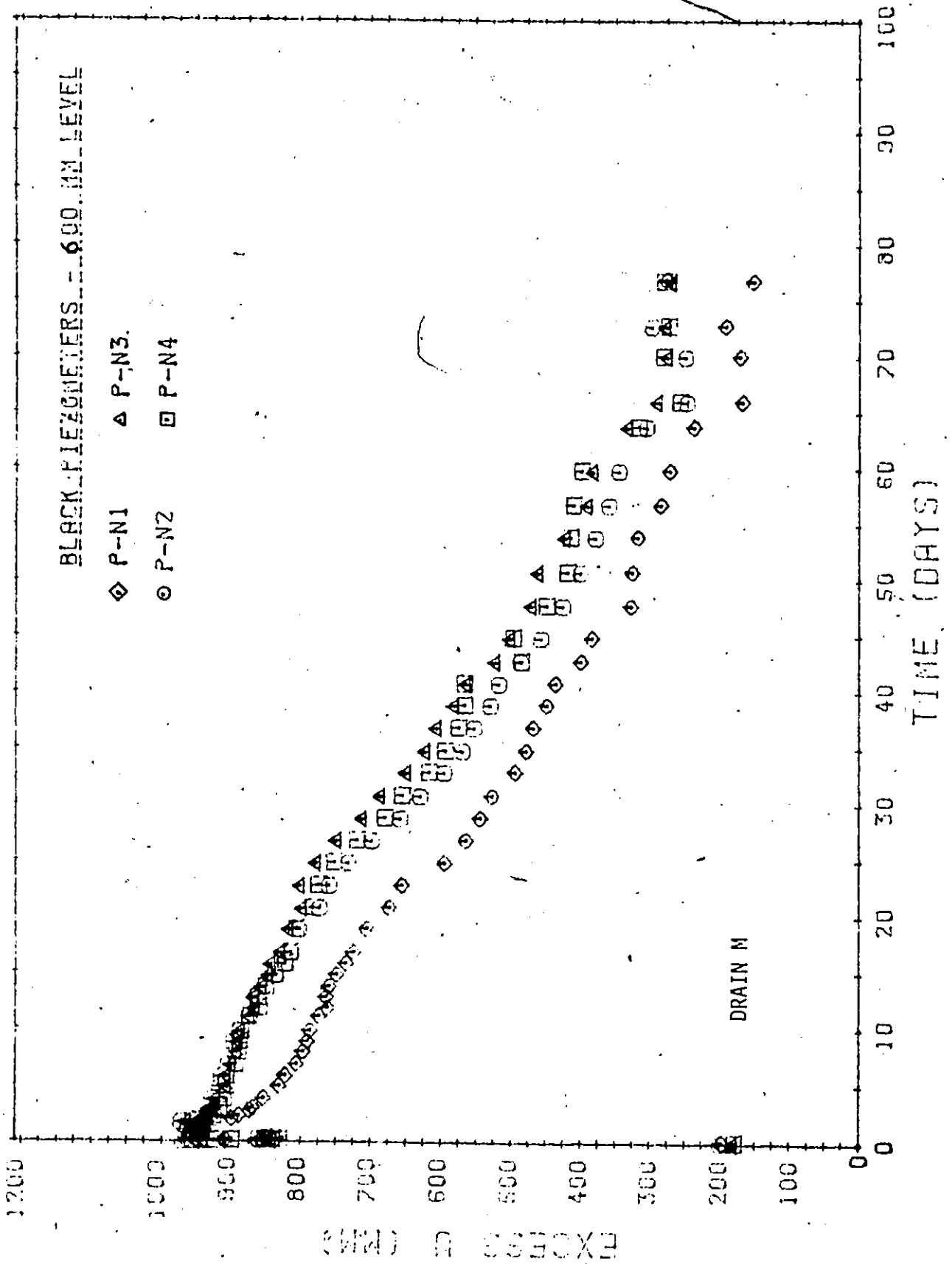


FIG. A1.2 : EXCESS PORE PRESSURE - TEST SERIES ONE

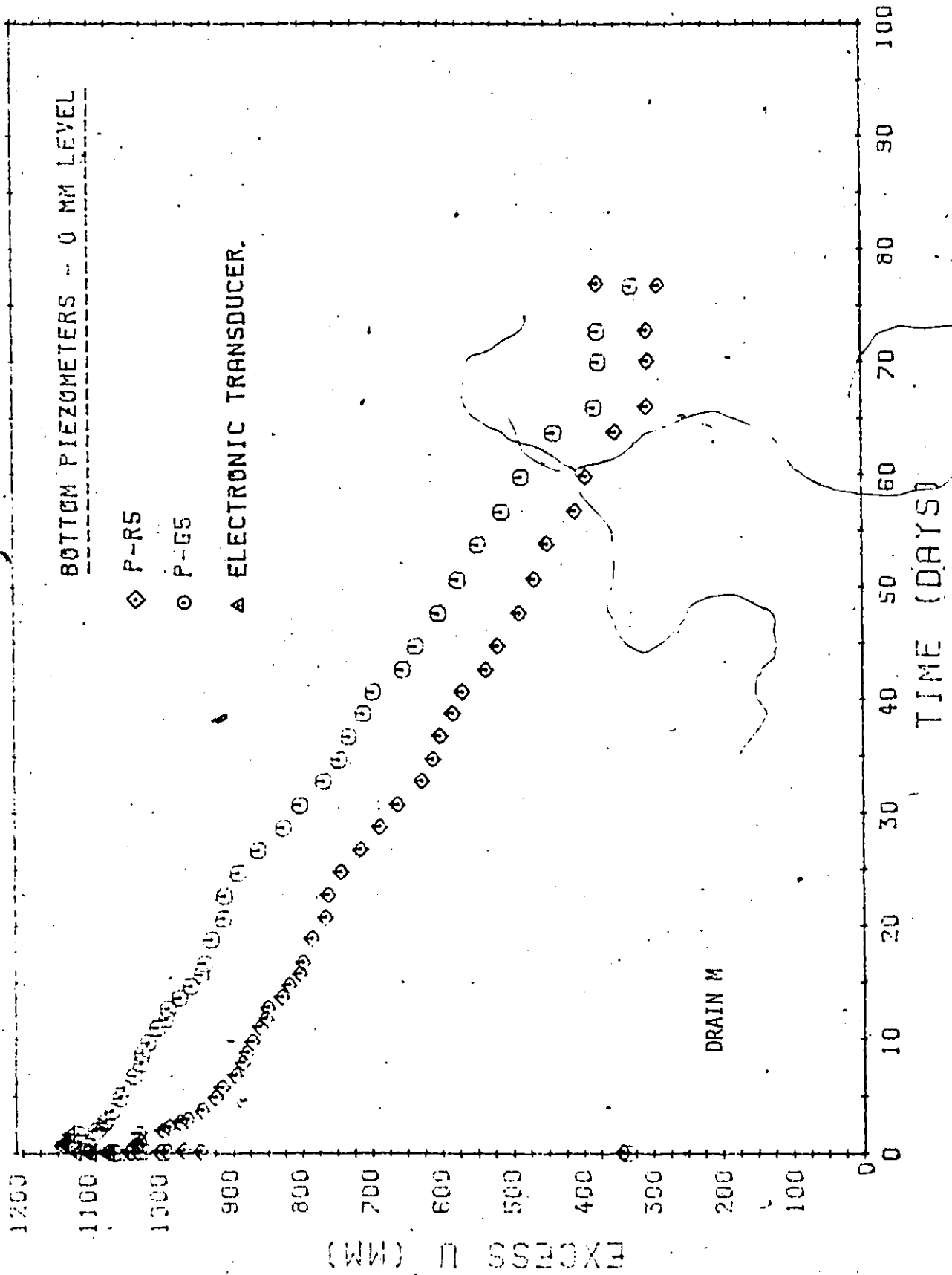


FIG. A1.2 : EXCESS PORE PRESSURE - TEST SERIES ONE

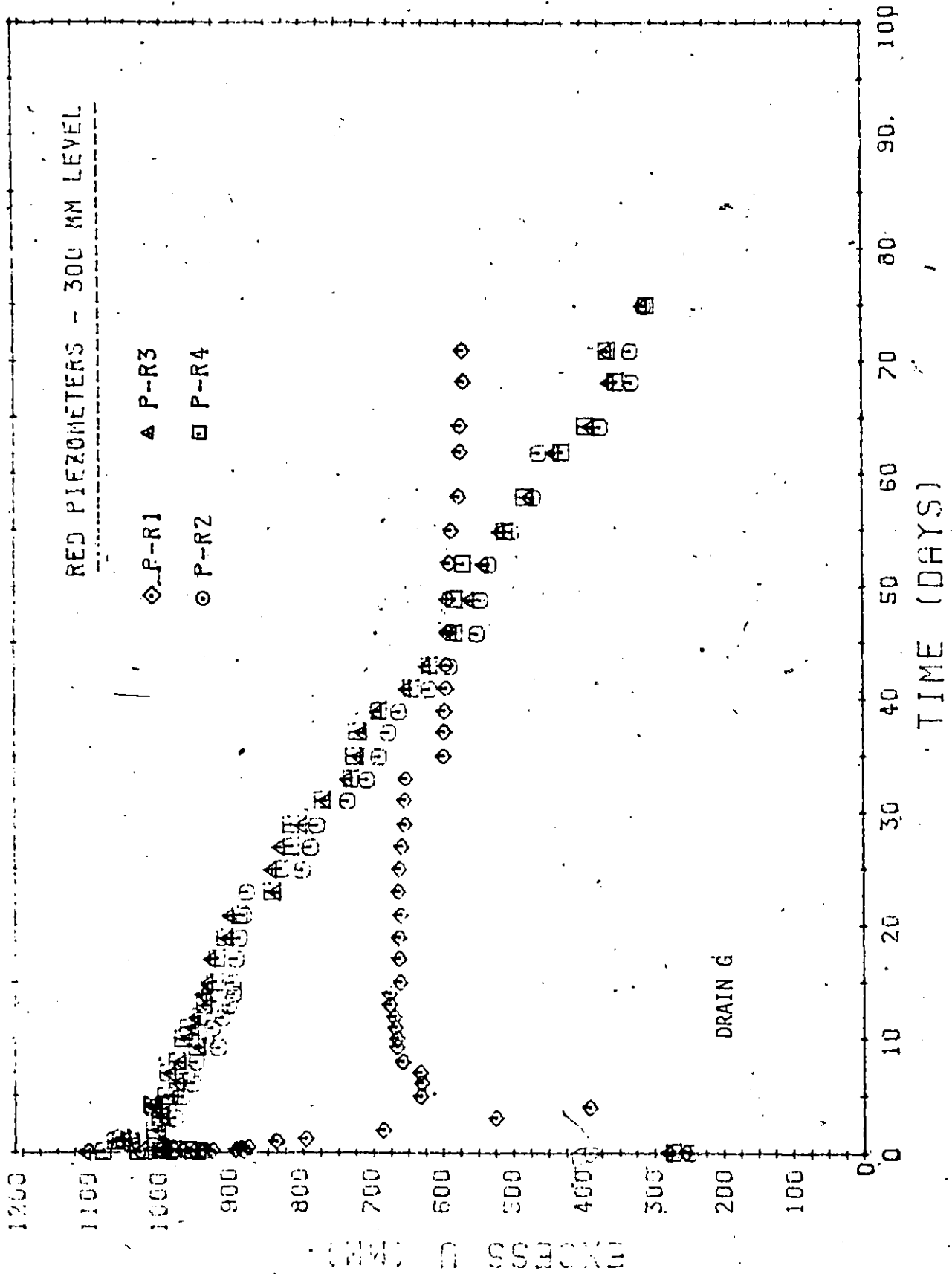


FIG. A1.3 : EXCESS PORE PRESSURE - TEST SERIES ONE

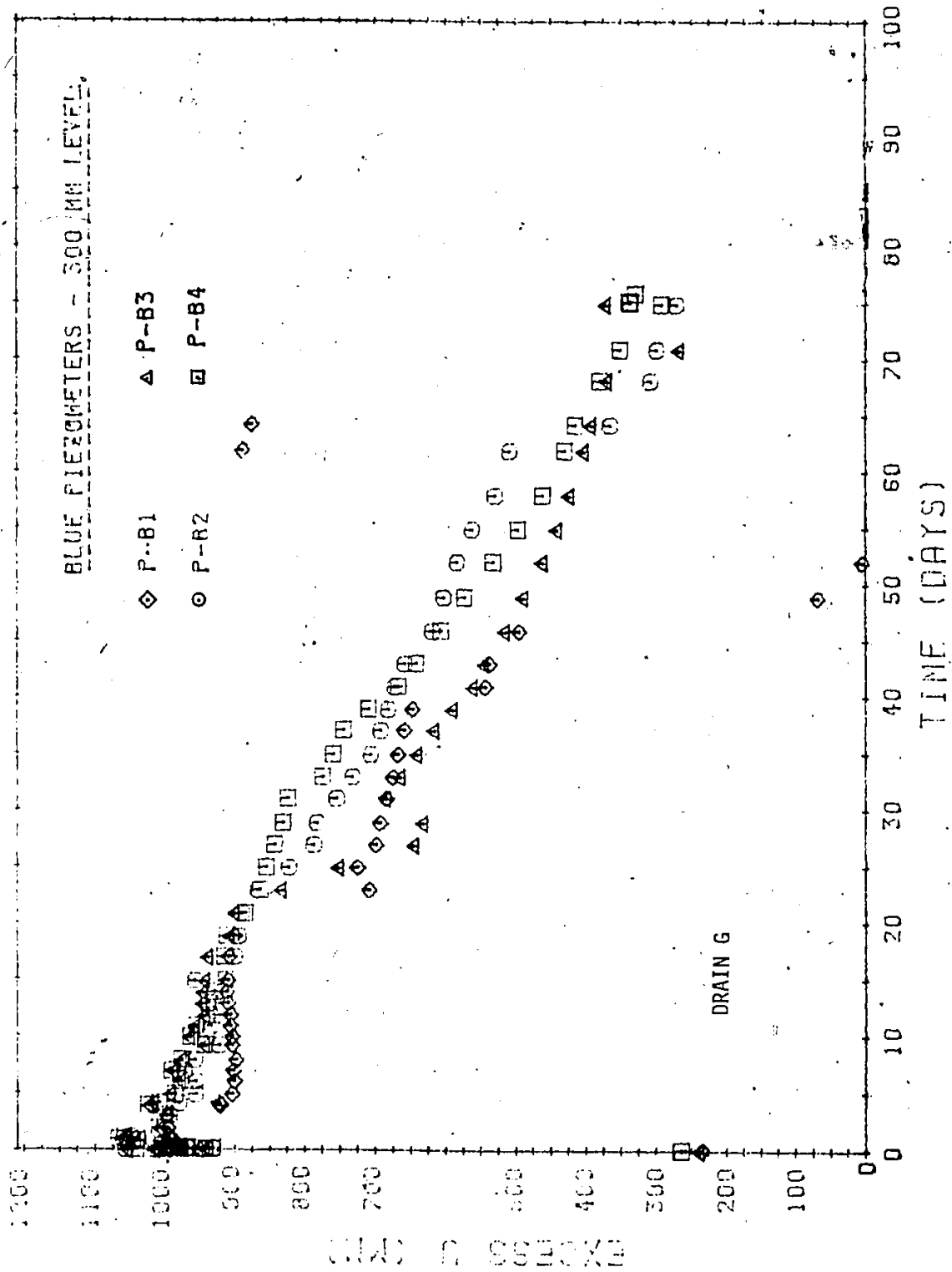


FIG. A1.3 : EXCESS PORE PRESSURE - TEST SERIES ONE

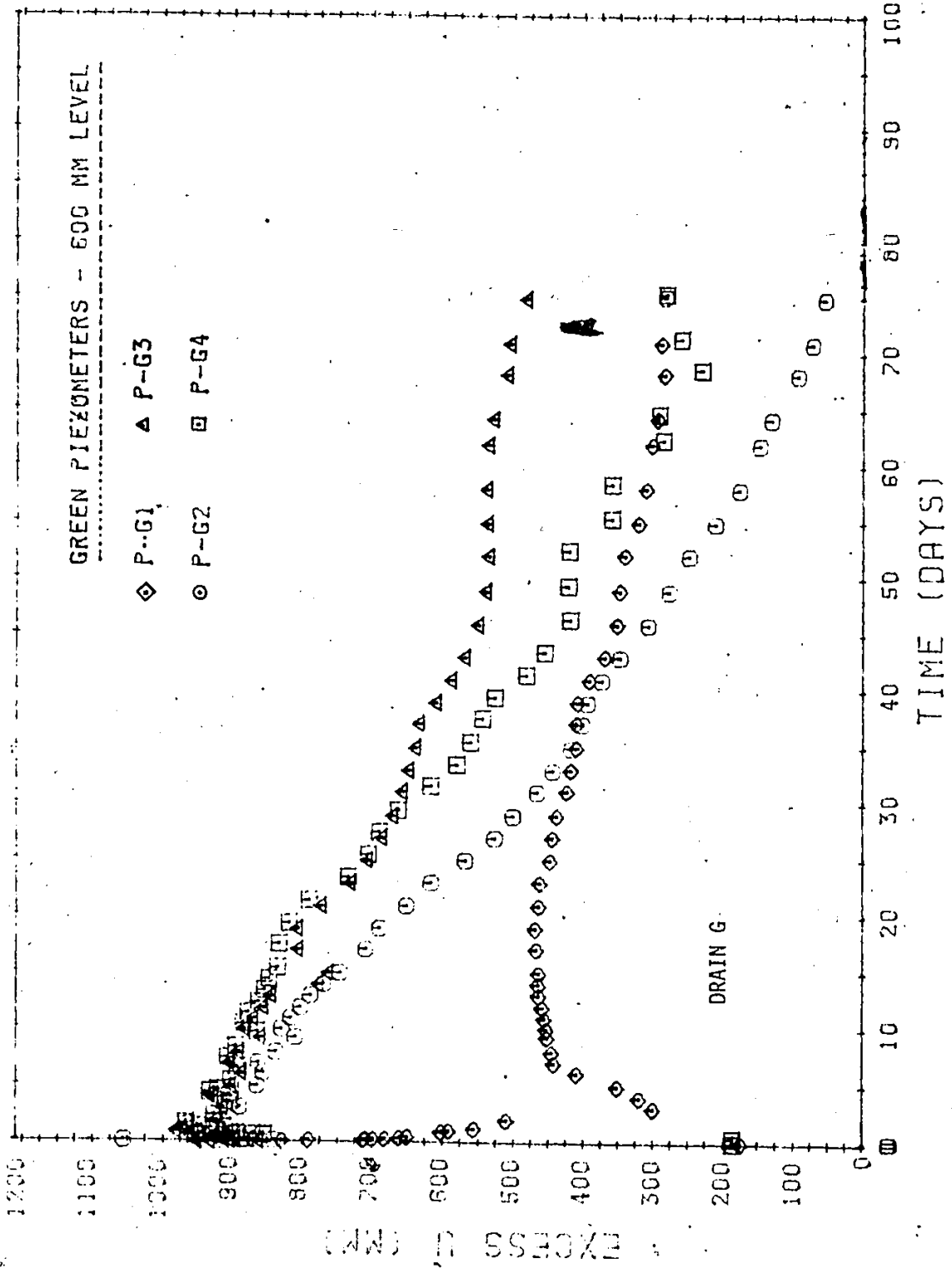


FIG. A1.3 : EXCESS PORE PRESSURE - TEST SERIES ONE

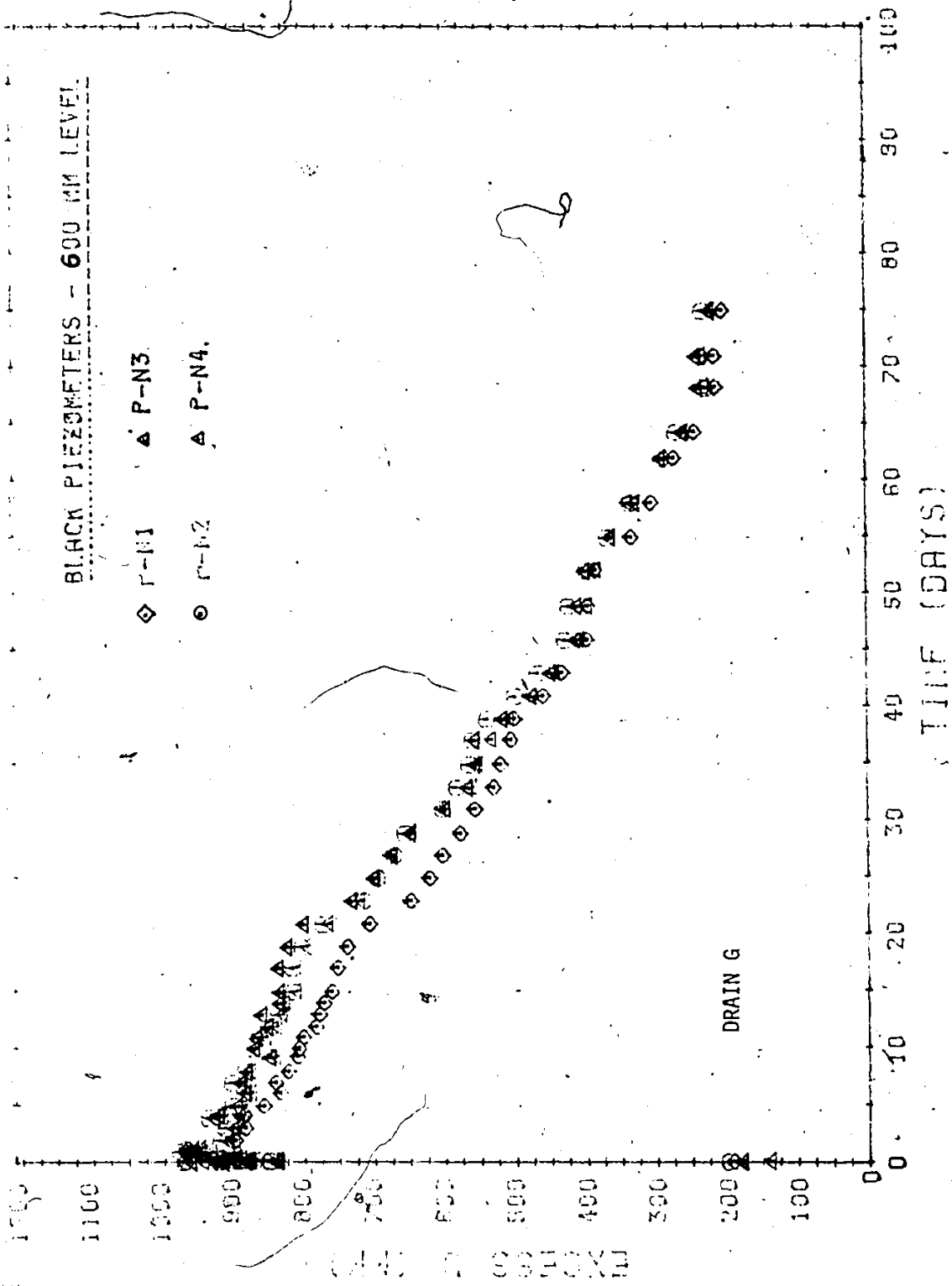


FIG. A1.3 : EXCESS PORE PRESSURE - TEST SERIES ONE

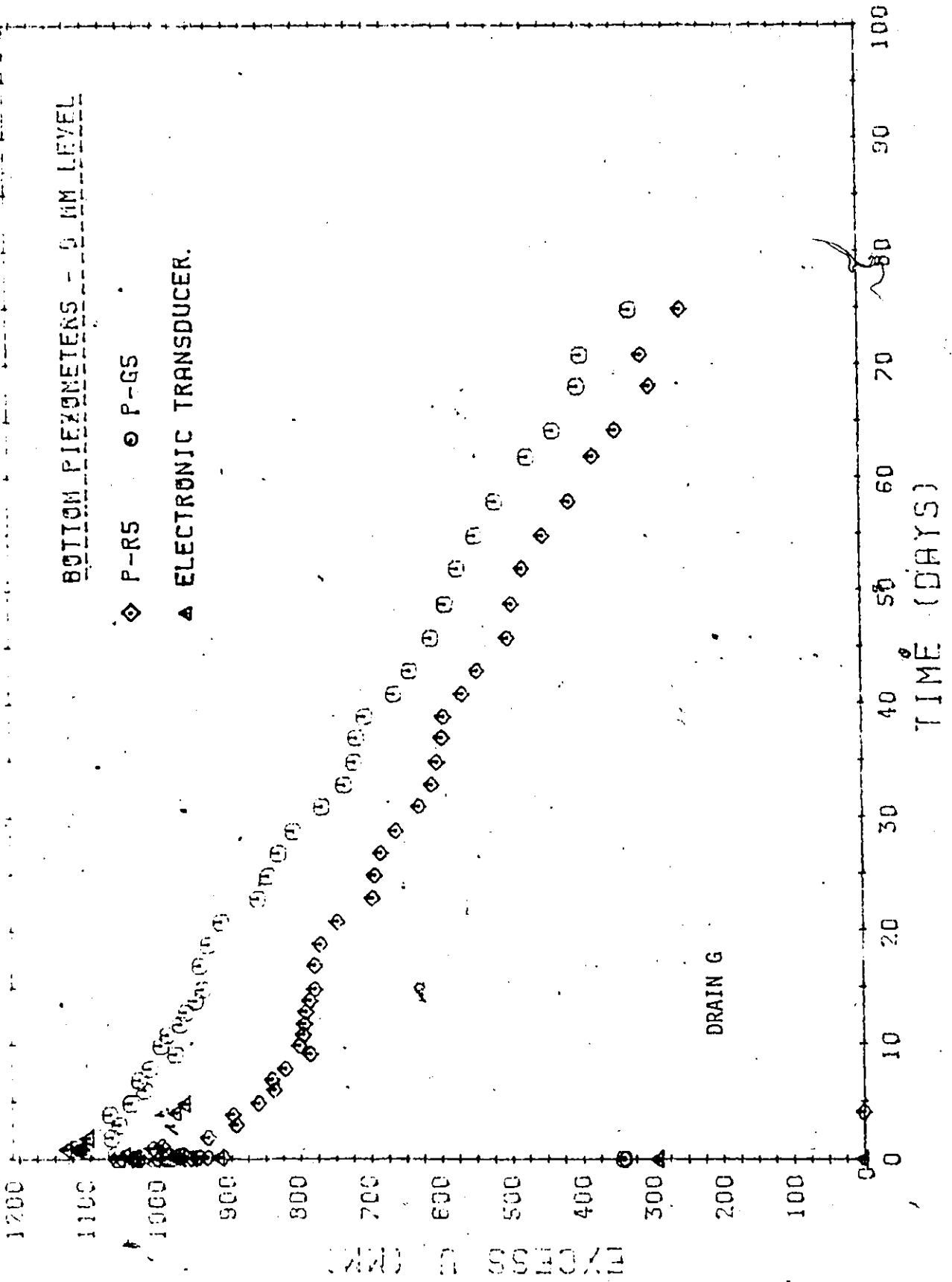


FIG. A1.3 : EXCESS PORE PRESSURE - TEST SERIES ONE

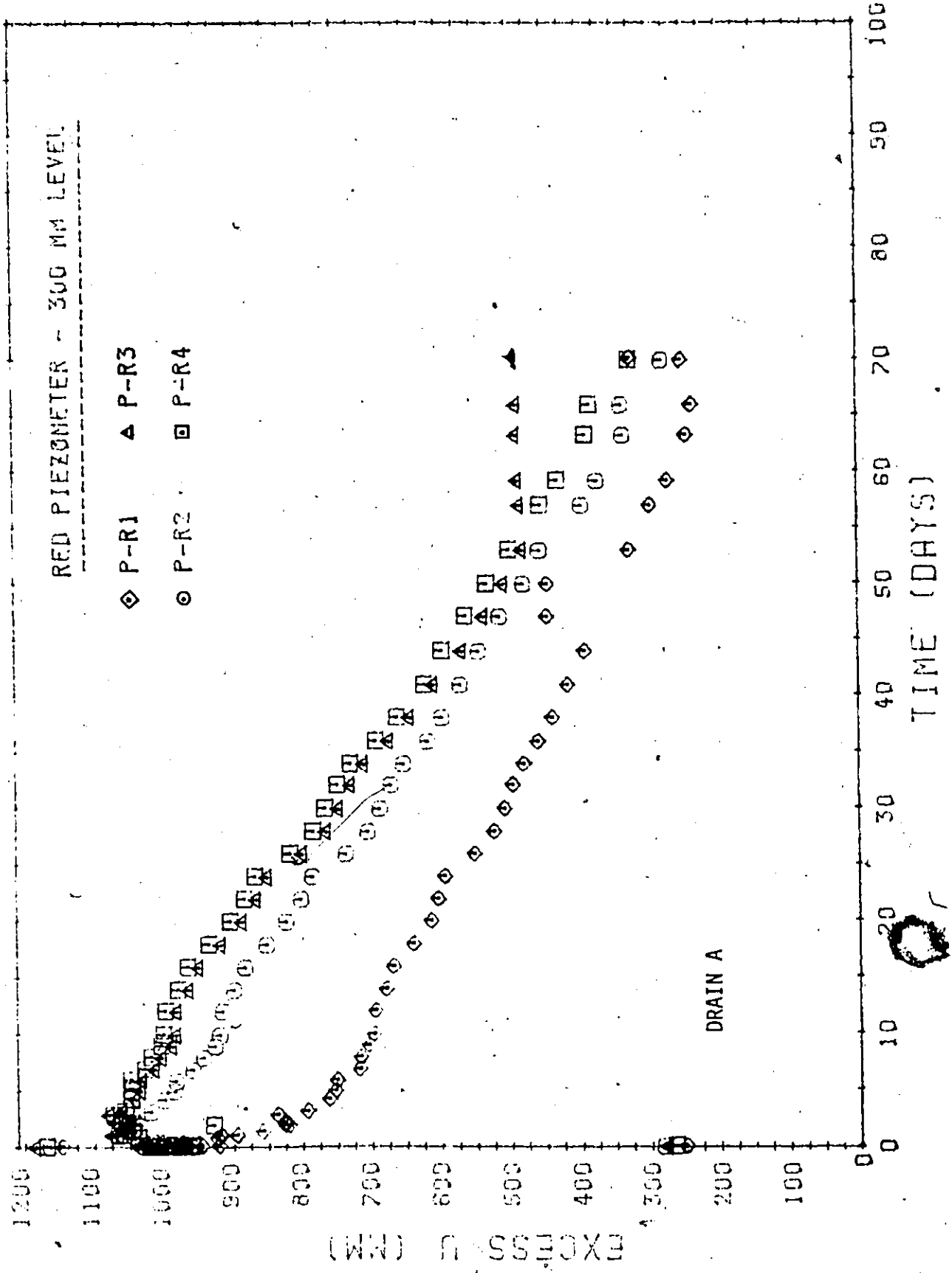


FIG. A1.4 : EXCESS PORE PRESSURE - TEST SERIES ONE

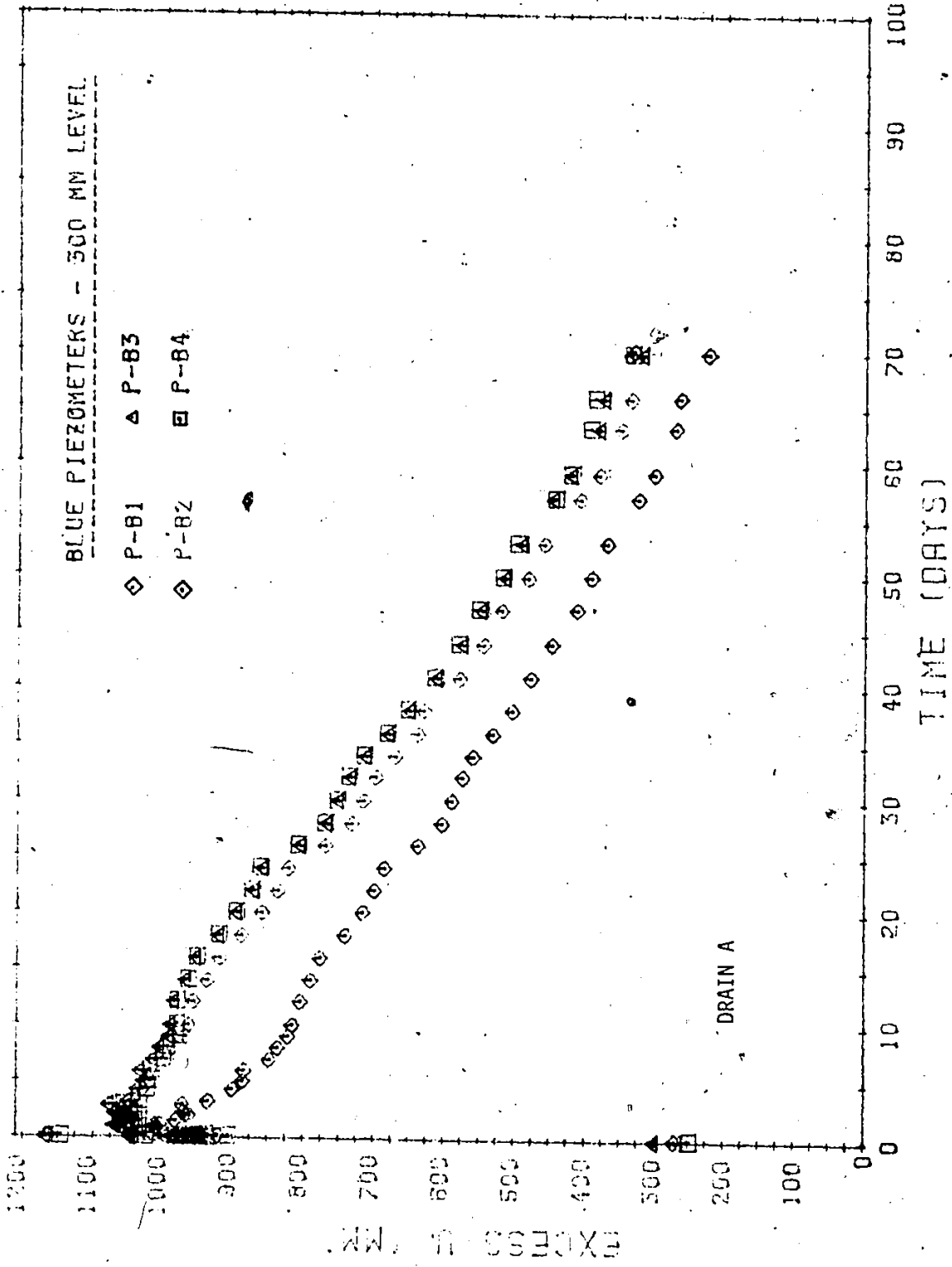


FIG. A1.4 : EXCESS PORE PRESSURE - TEST SERIES ONE

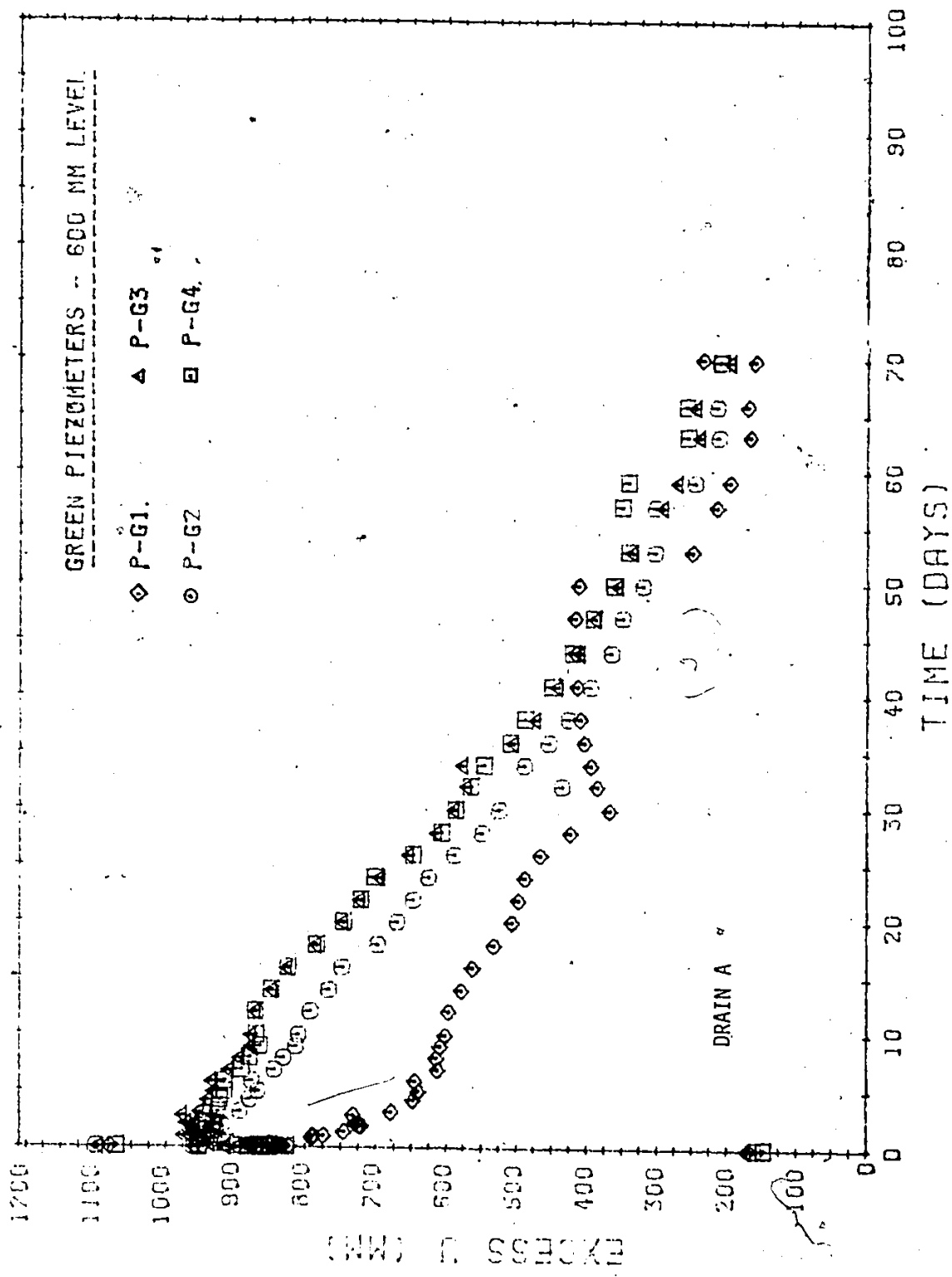


FIG. A1.4 : EXCESS PORE PRESSURE - TEST SERIES ONE

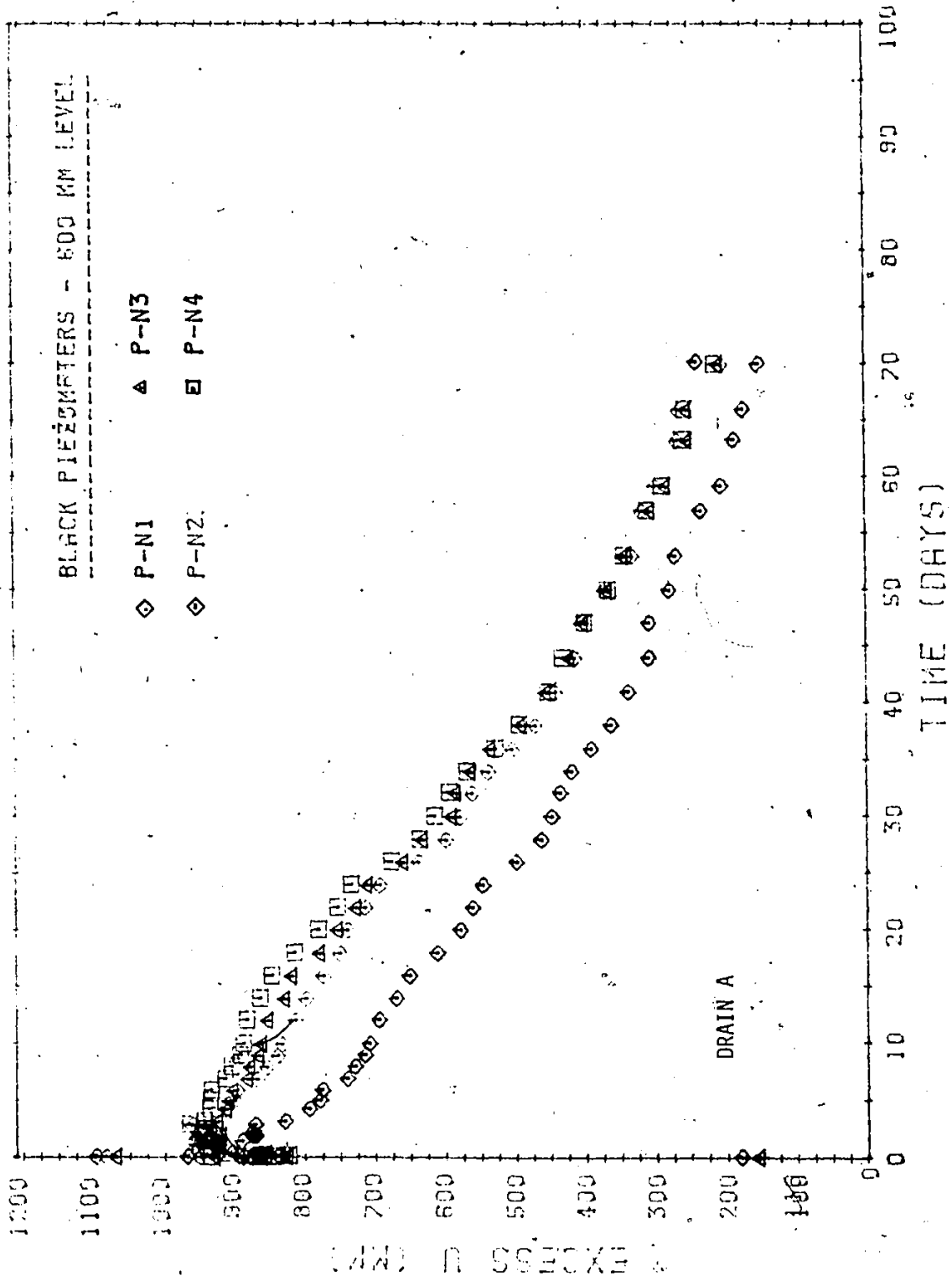


FIG. A1.4 : EXCESS PORE PRESSURE - TEST SERIES ONE

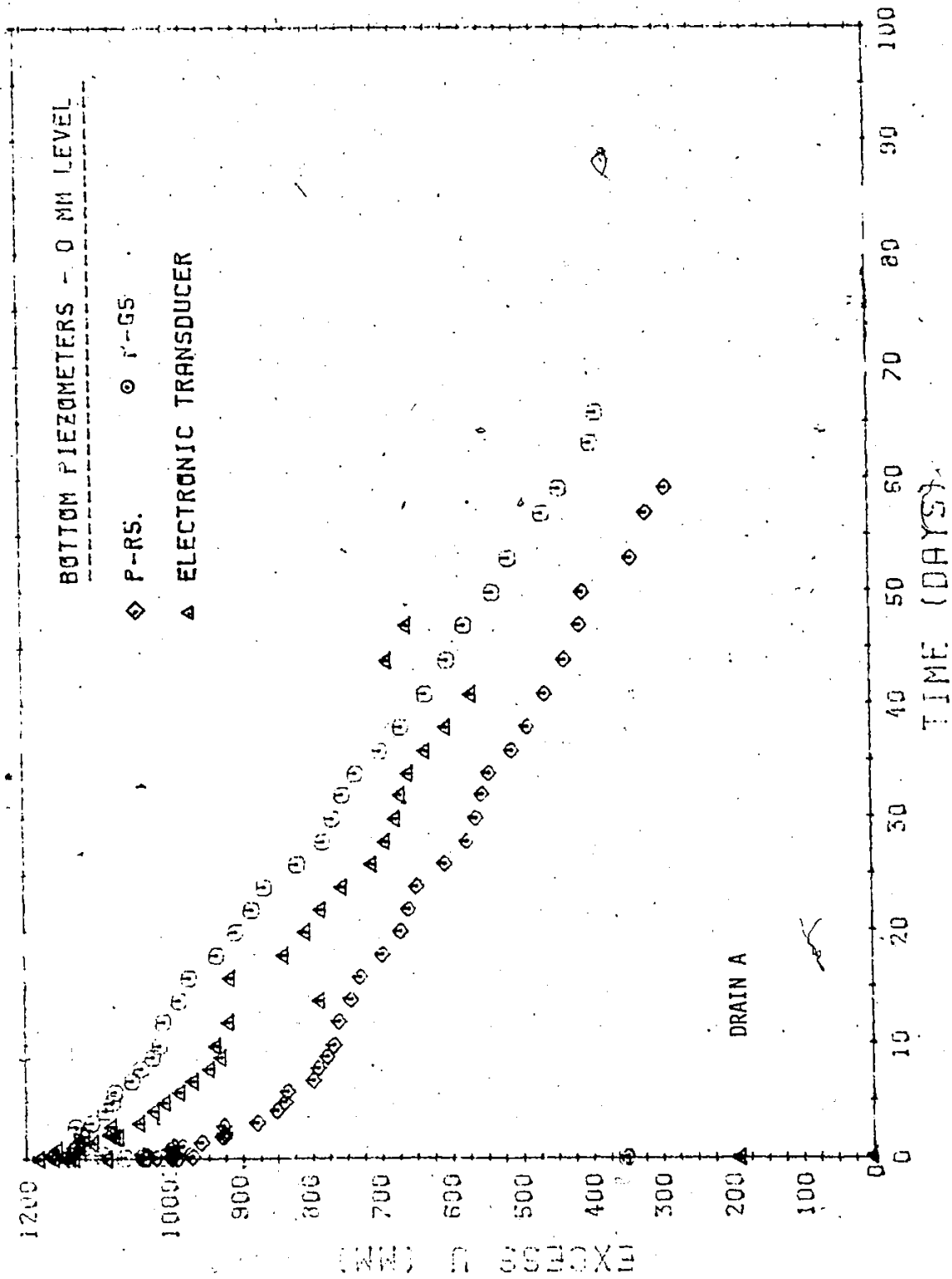


FIG. A1.4 : EXCESS PORE PRESSURE - TEST SERIES ONE

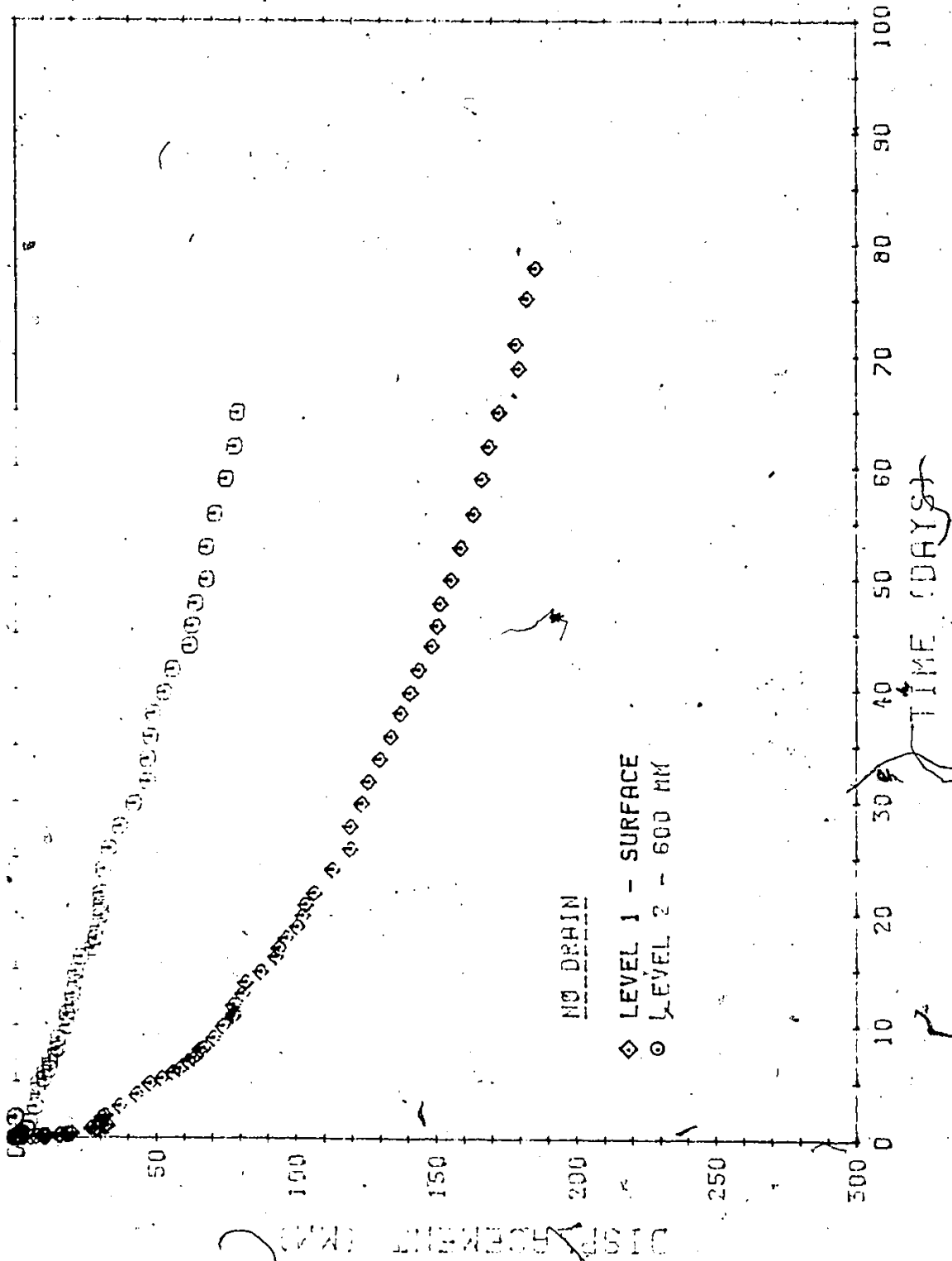


FIG. A2.1 : SETTLEMENT CURVES - TEST SERIES ONE

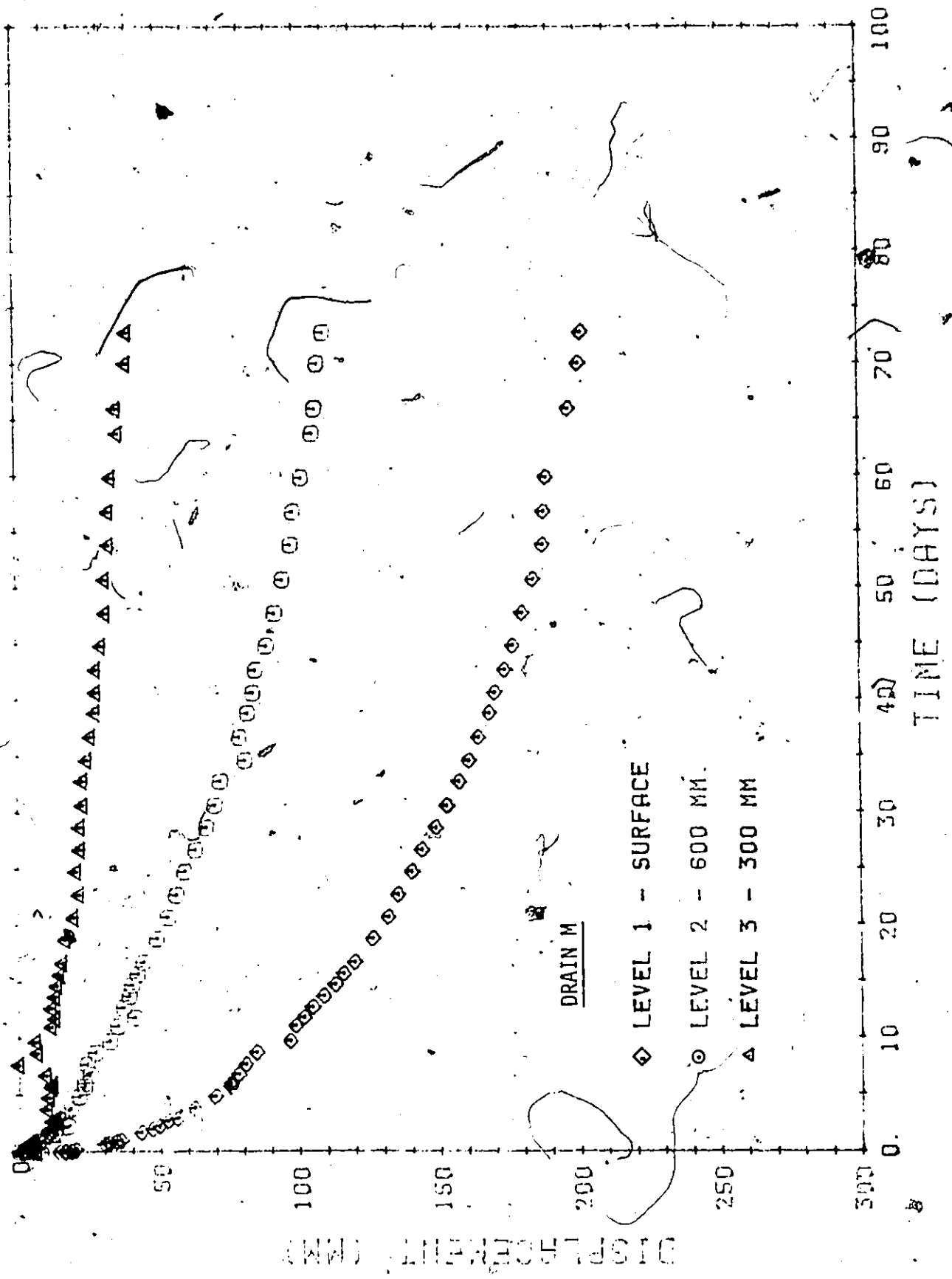


FIG. A2.2 : SETTLEMENT CURVES - TEST SERIES ONE

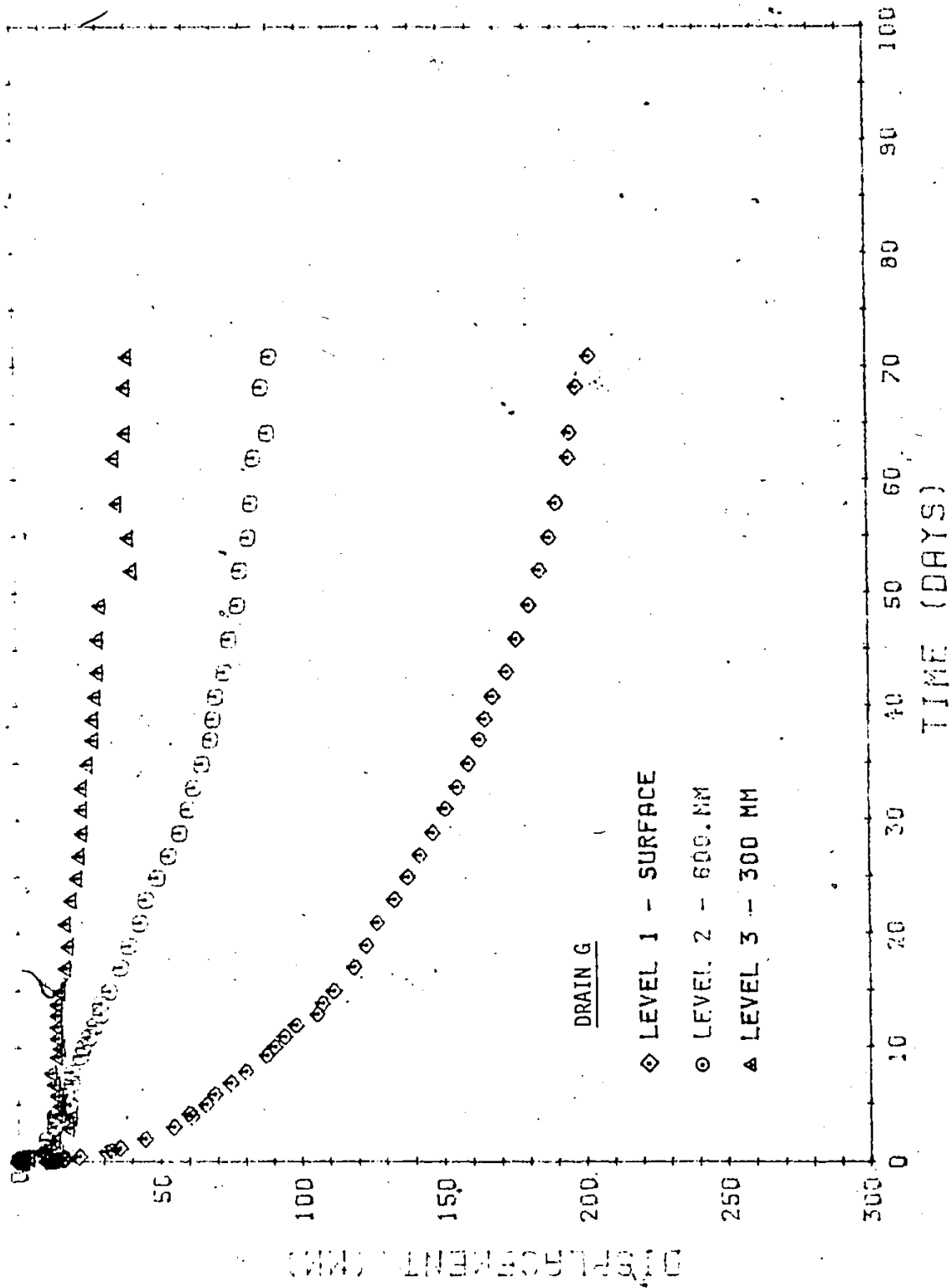


FIG. A2.3 : SETTLEMENT CURVES - TEST SERIES ONE

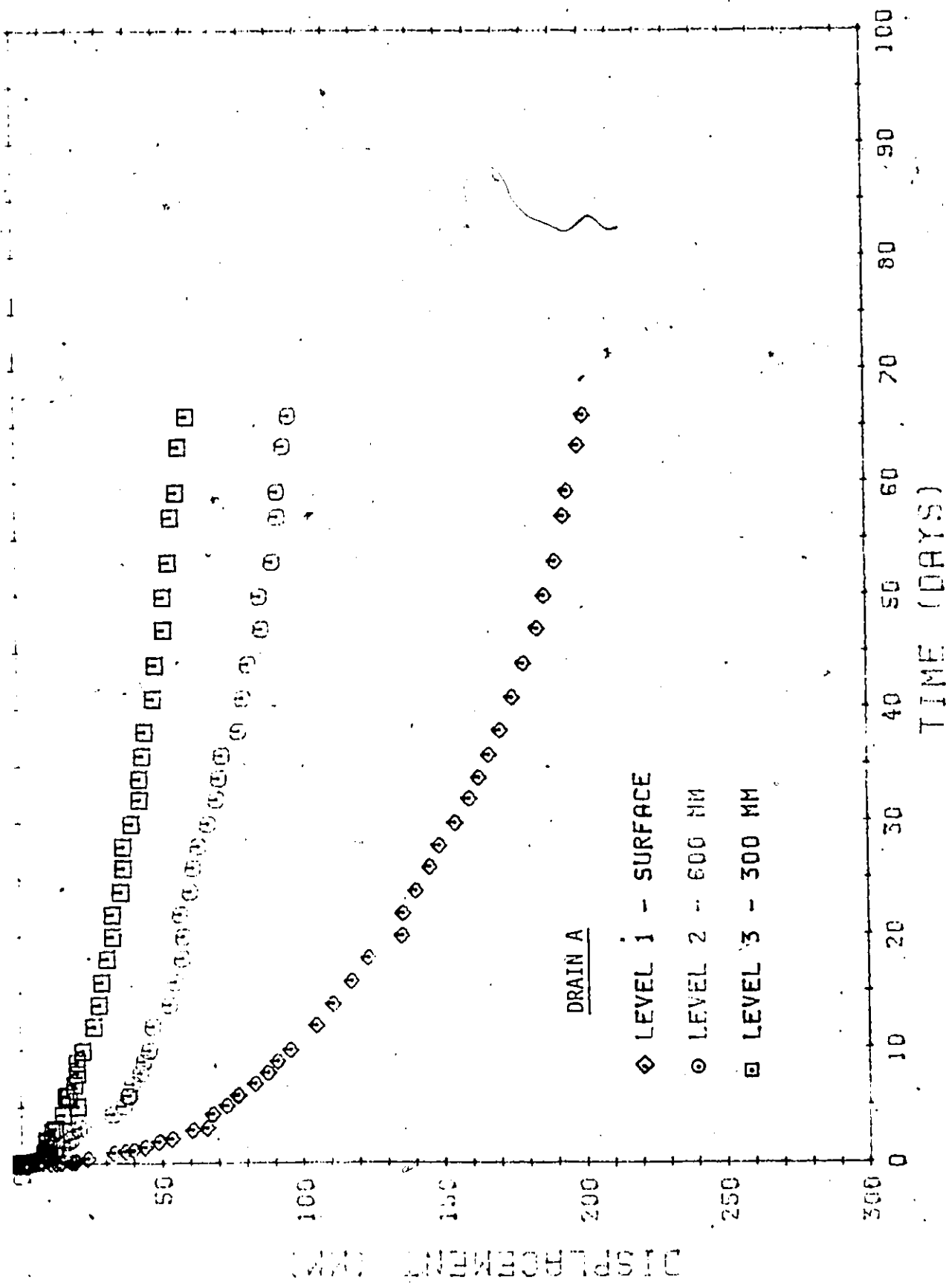


FIG. A2.4 : SETTLEMENT CURVES - TEST SERIES ONE

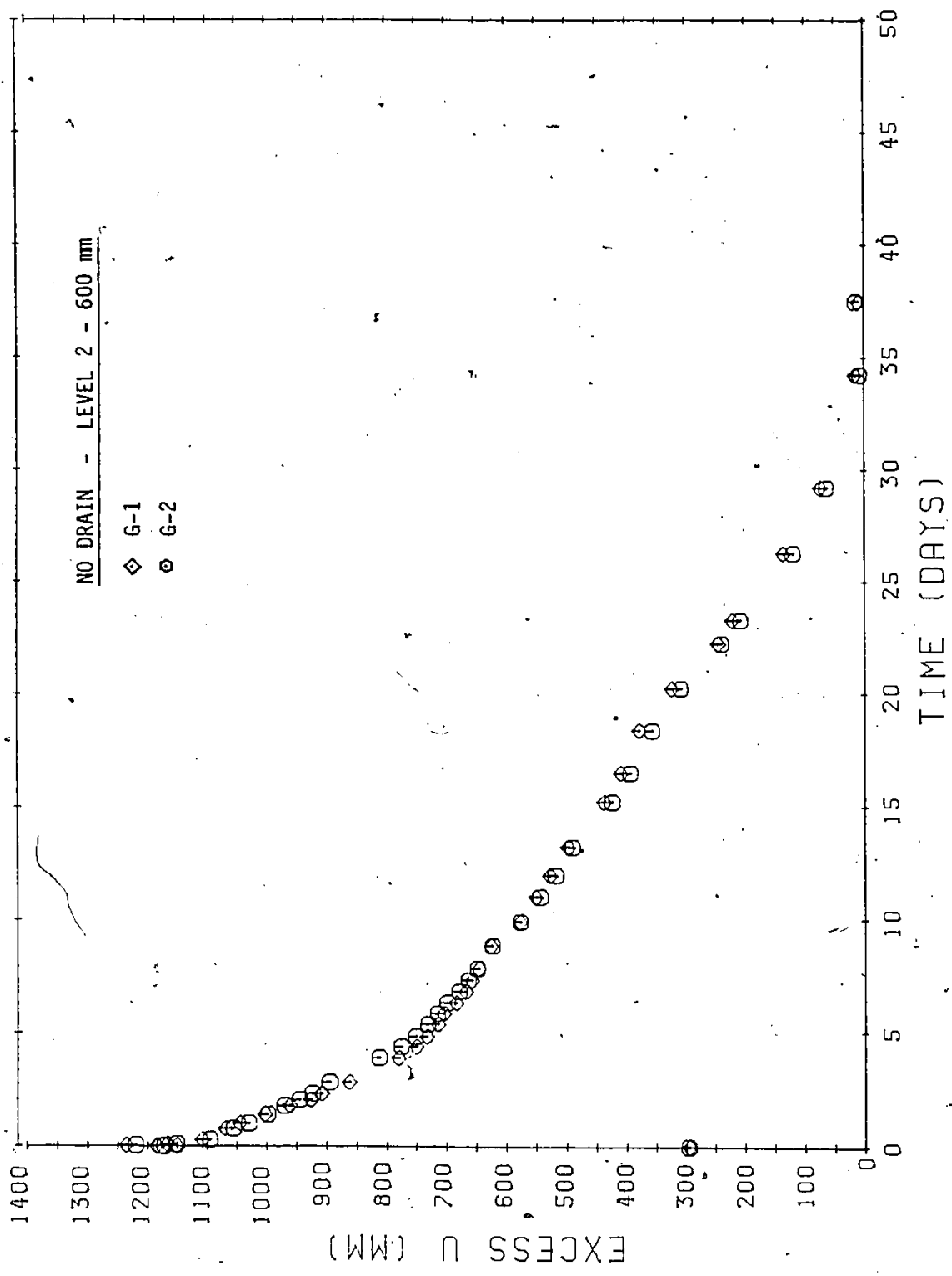


FIG. A3.1 : EXCESS PORE PRESSURE - TEST SERIES TWO

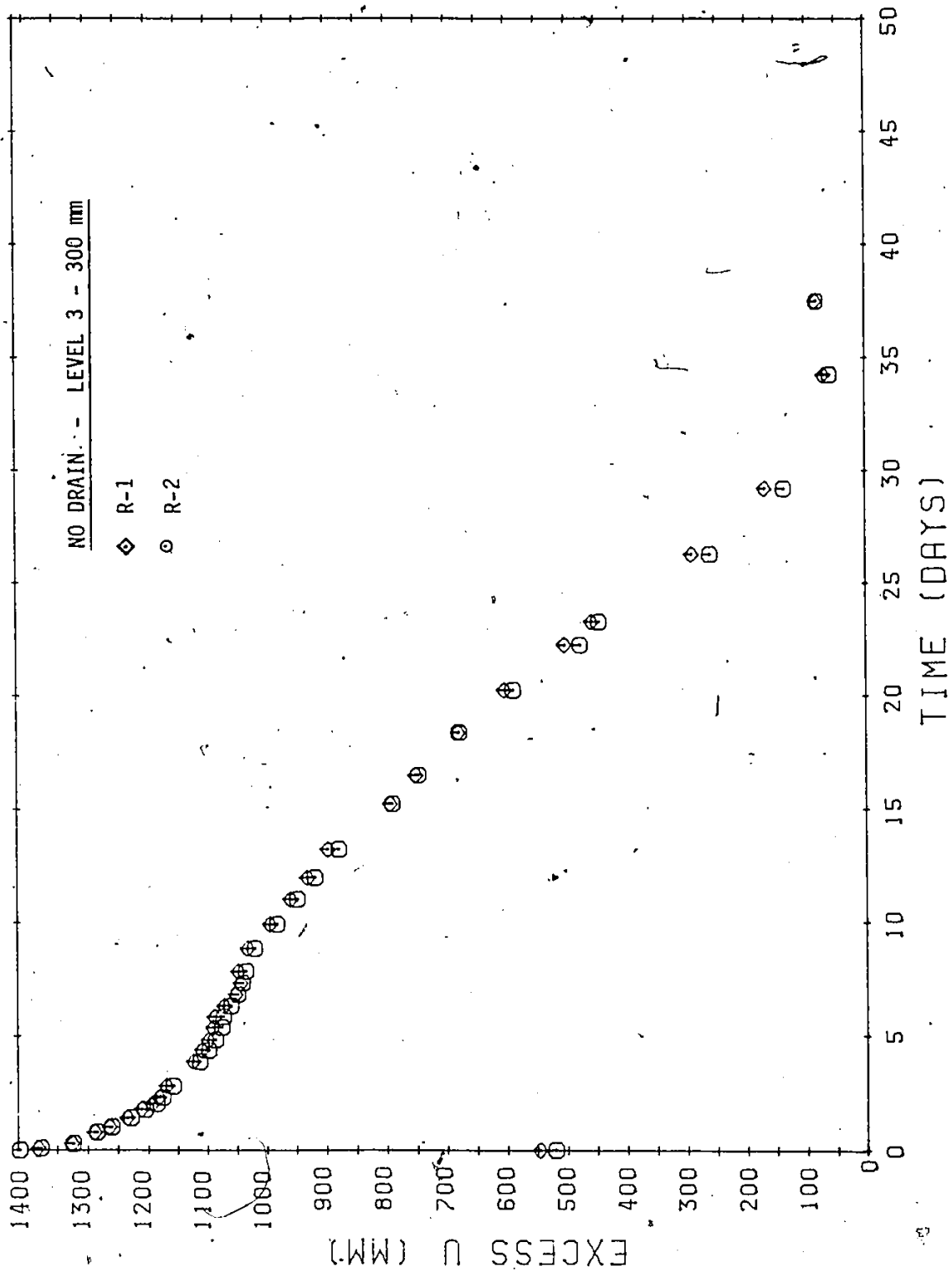


FIG. A3.1 : EXCESS PORE PRESSURE - TEST SERIES TWO

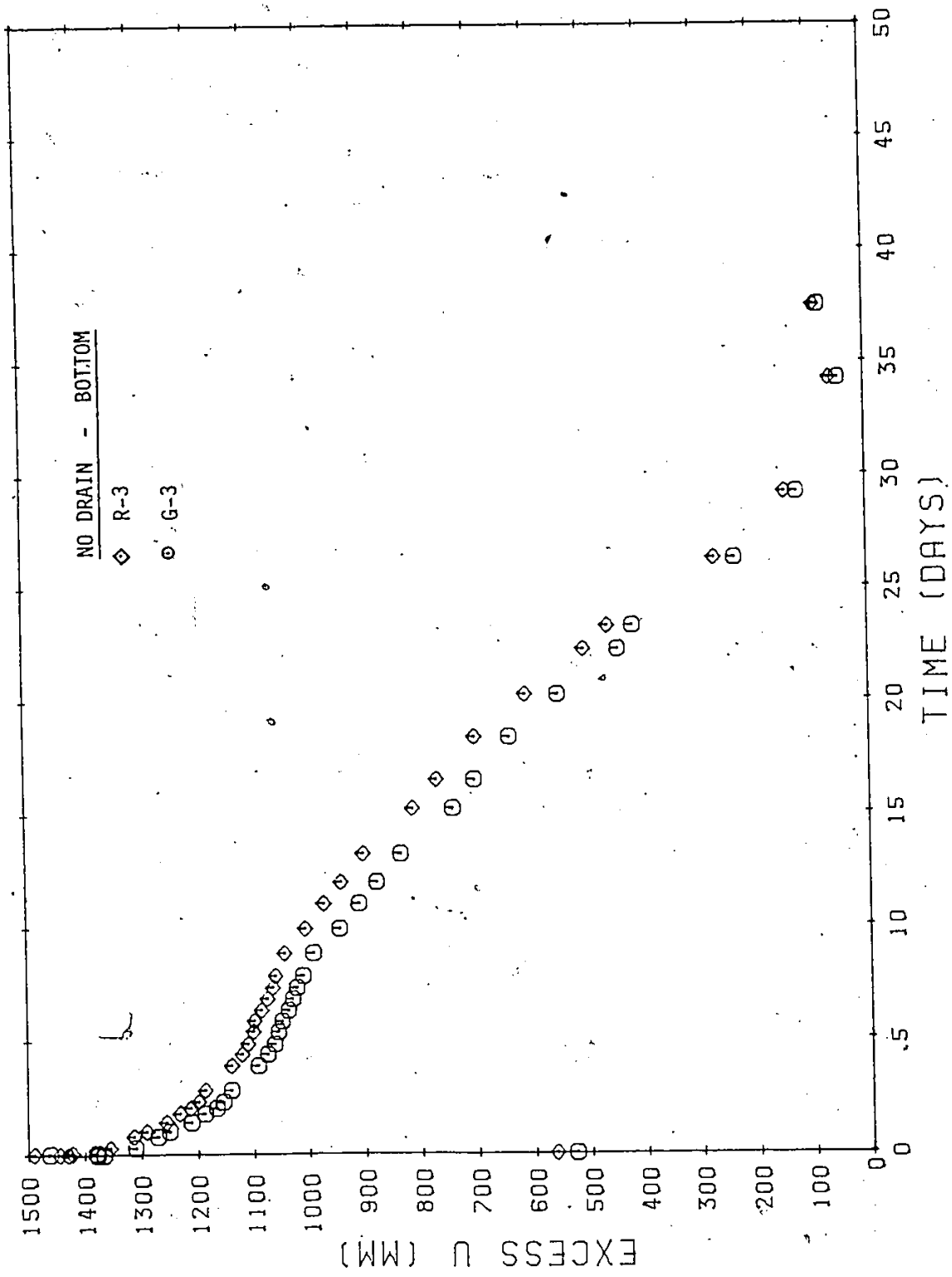


FIG. A3.1 : EXCESS PORE PRESSURE - TEST SERIES TWO

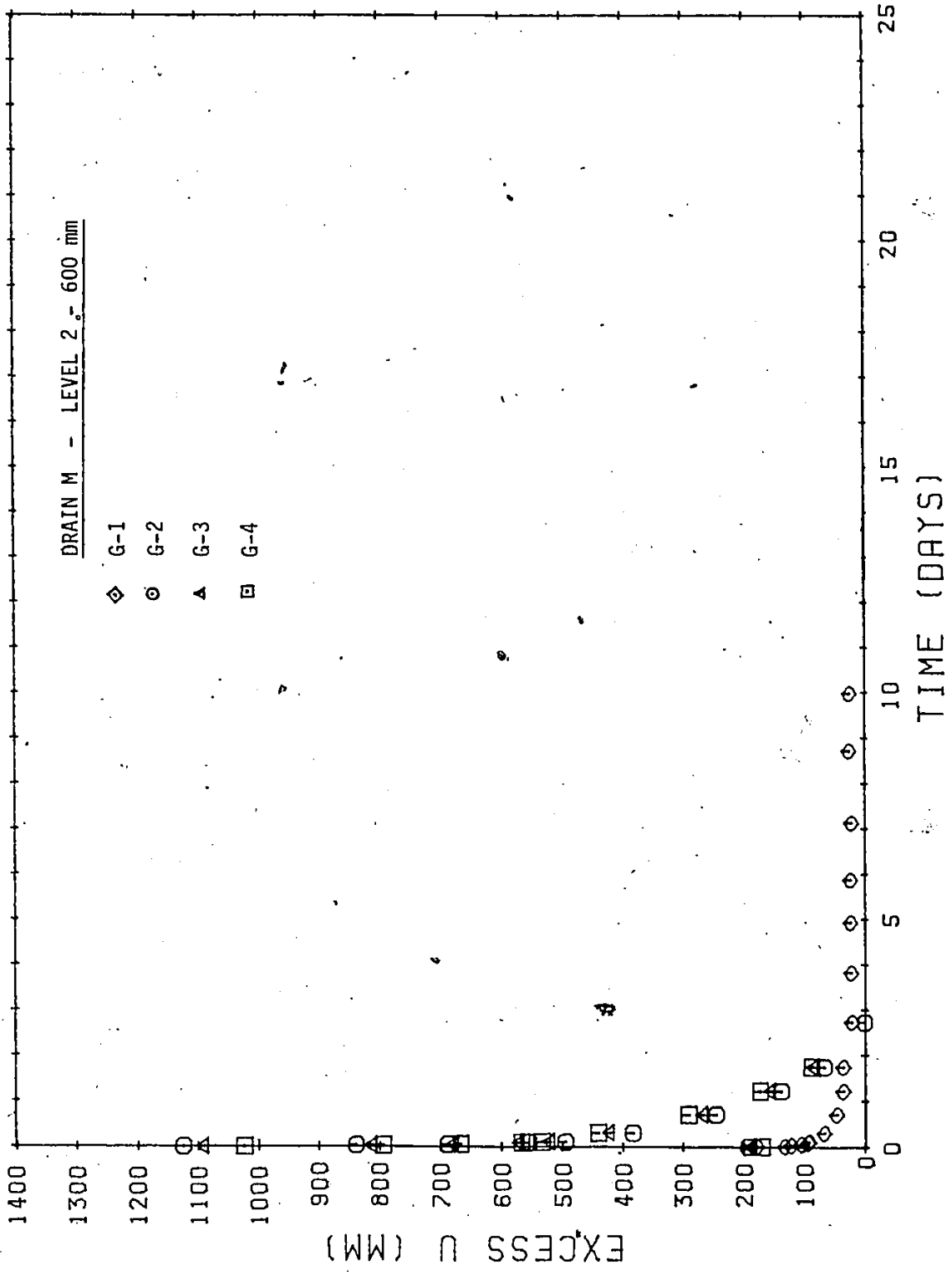


FIG. A3.2 : EXCESS PORE PRESSURE - TEST SERIES TWO

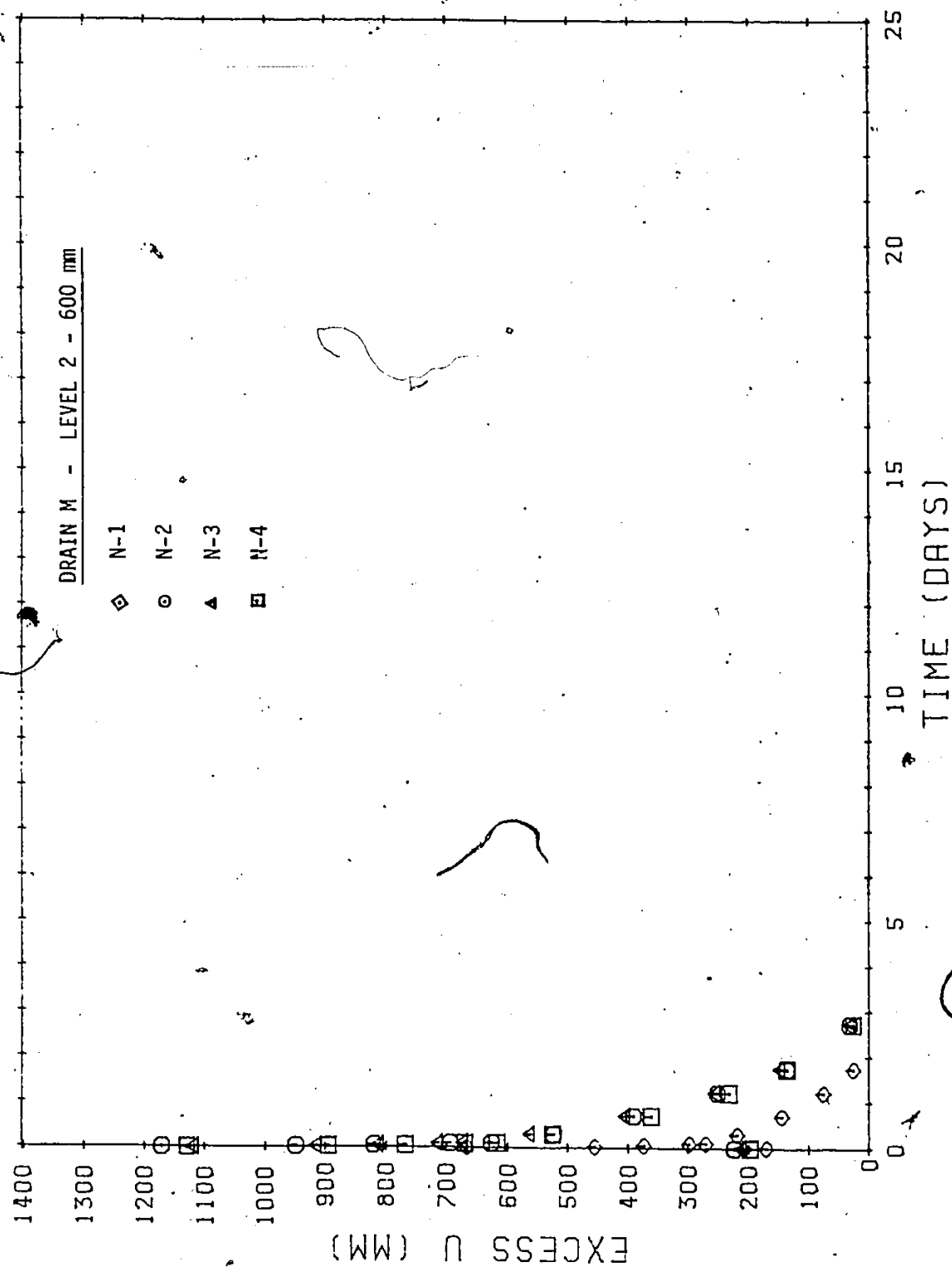


FIG. A3.2 : EXCESS PORE PRESSURE - TEST SERIES TWO

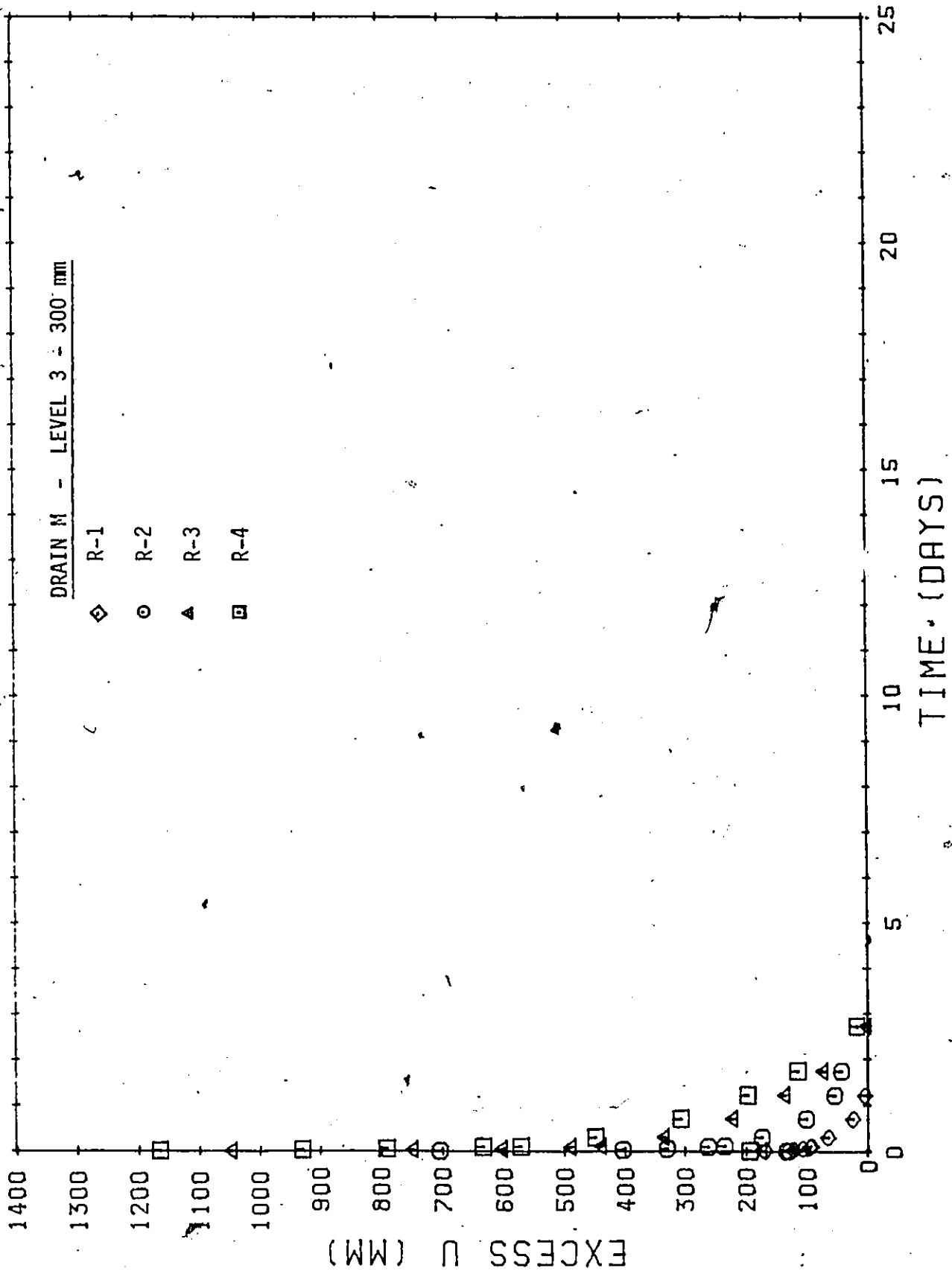


FIG. A3.2 : EXCESS PORE PRESSURE - TEST SERIES TWO

A

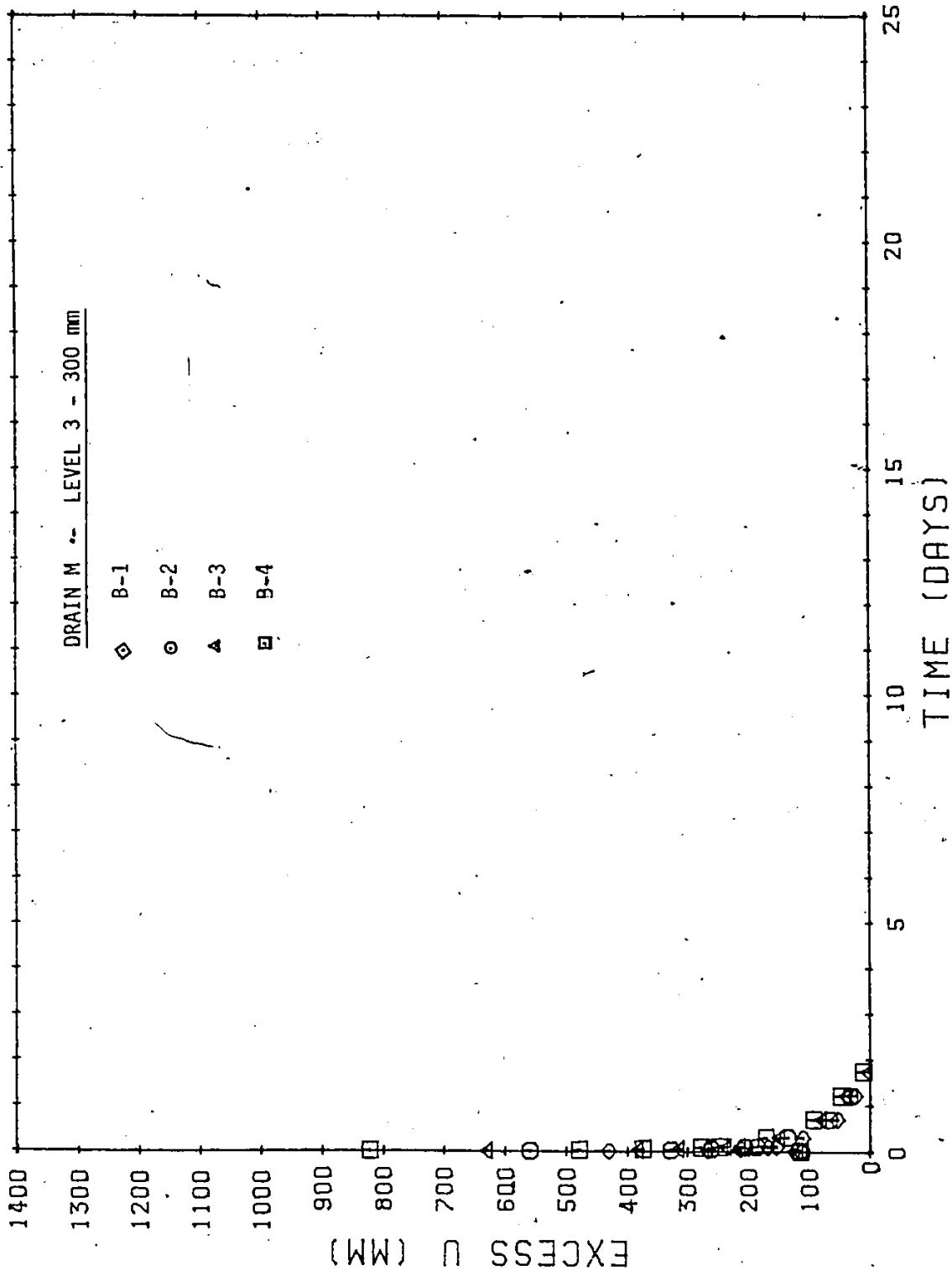


FIG. A3.2 : EXCESS PORE PRESSURE - TEST SERIES TWO

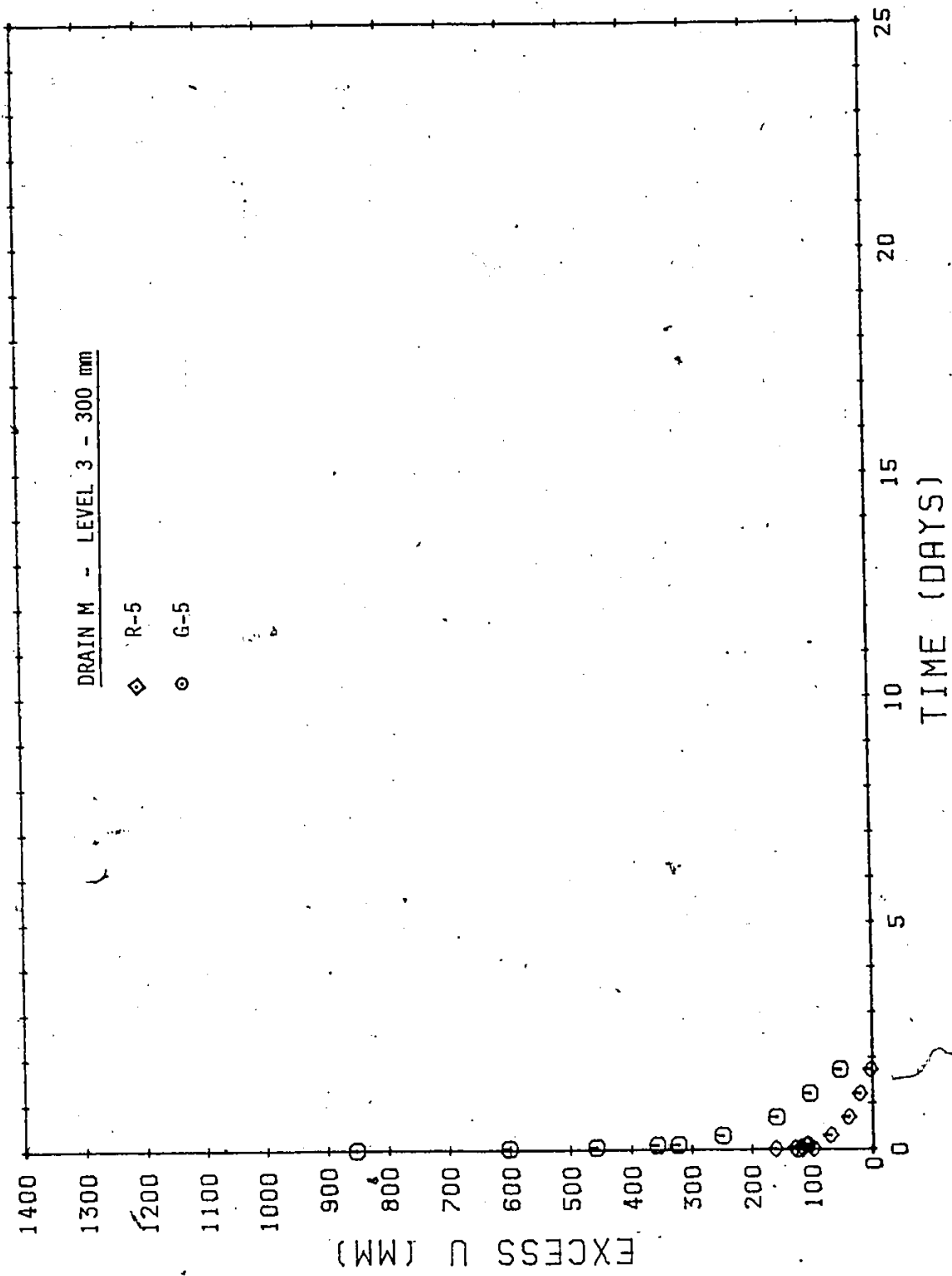


FIG. A3.2. EXCESS PORE PRESSURE - TEST SERIES TWO

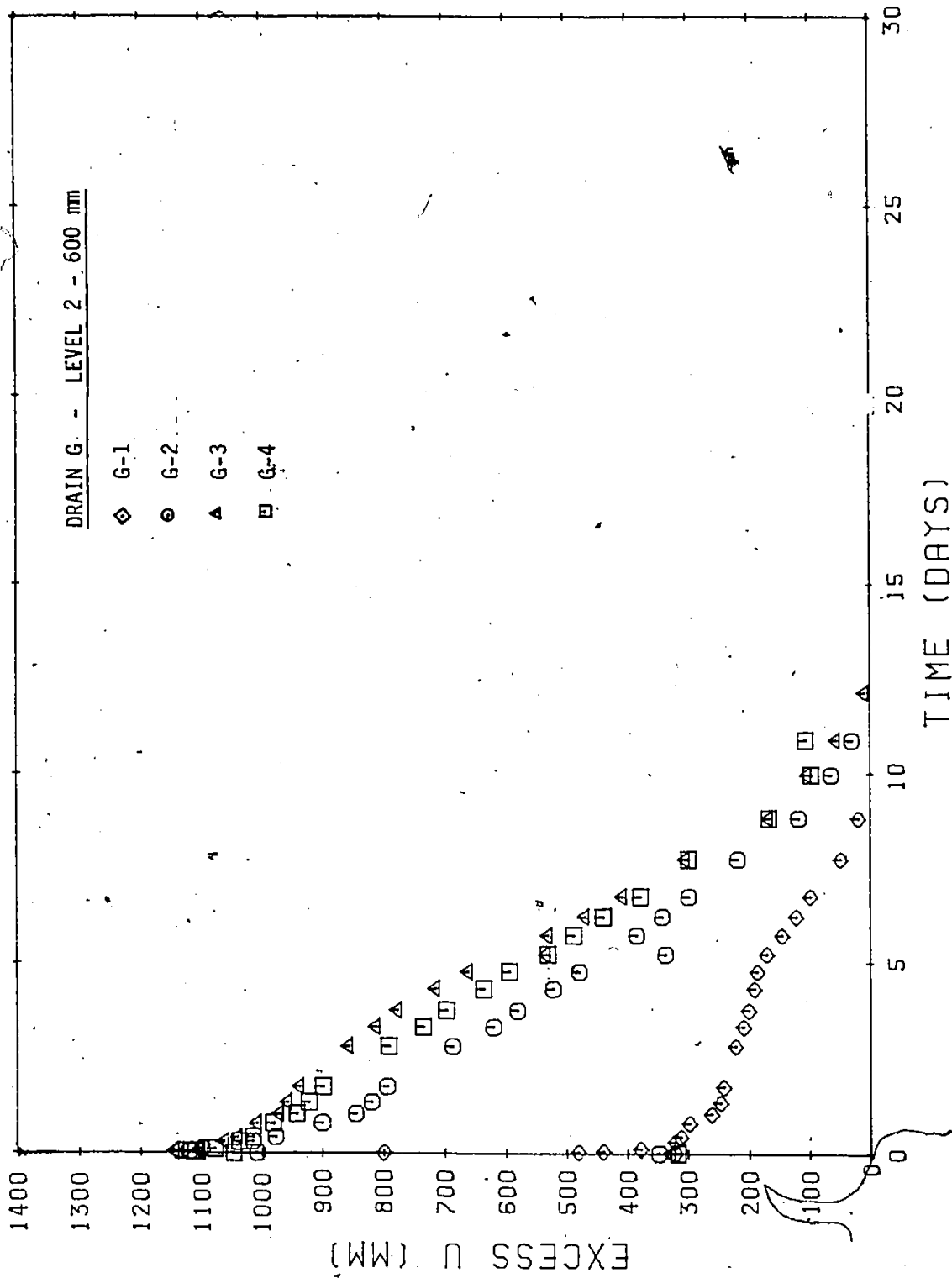


FIG. A3.3 : EXCESS PORE PRESSURE - TEST SERIES TWO

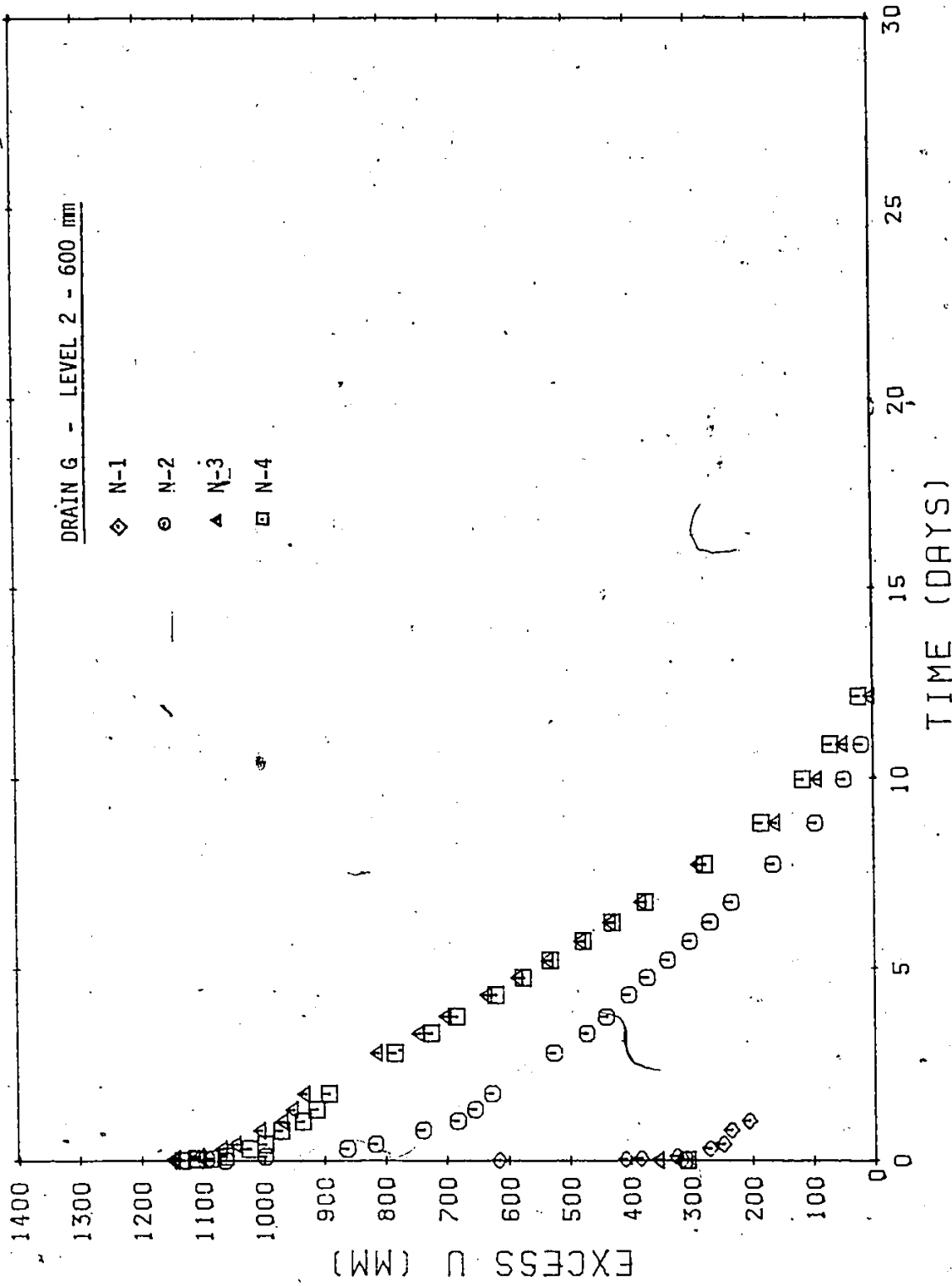


FIG A3.3 : EXCESS PORE PRESSURE - TEST SERIES TWO

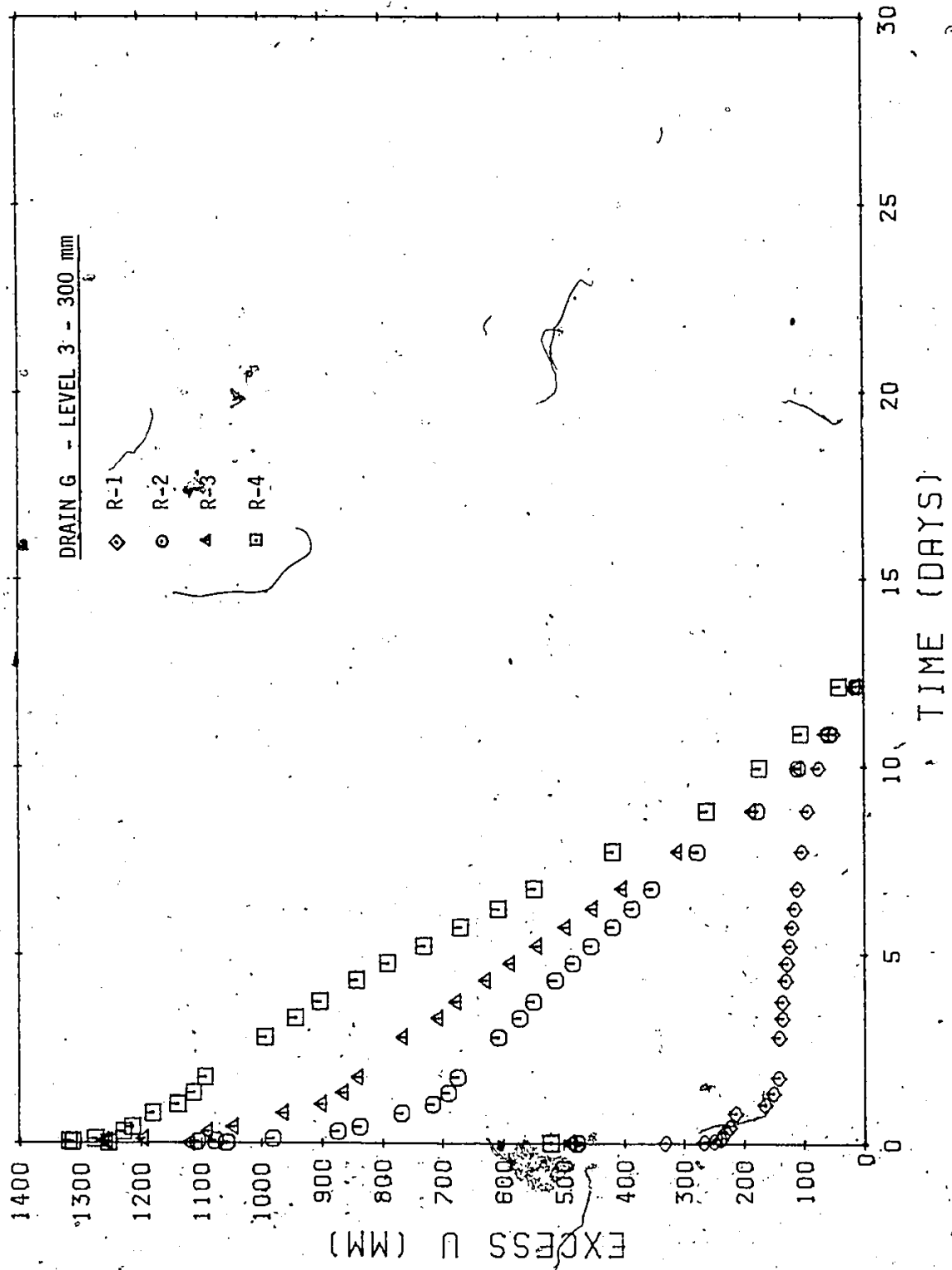


FIG. A3.3 : EXCESS PORE PRESSURE, TEST SERIES TWO

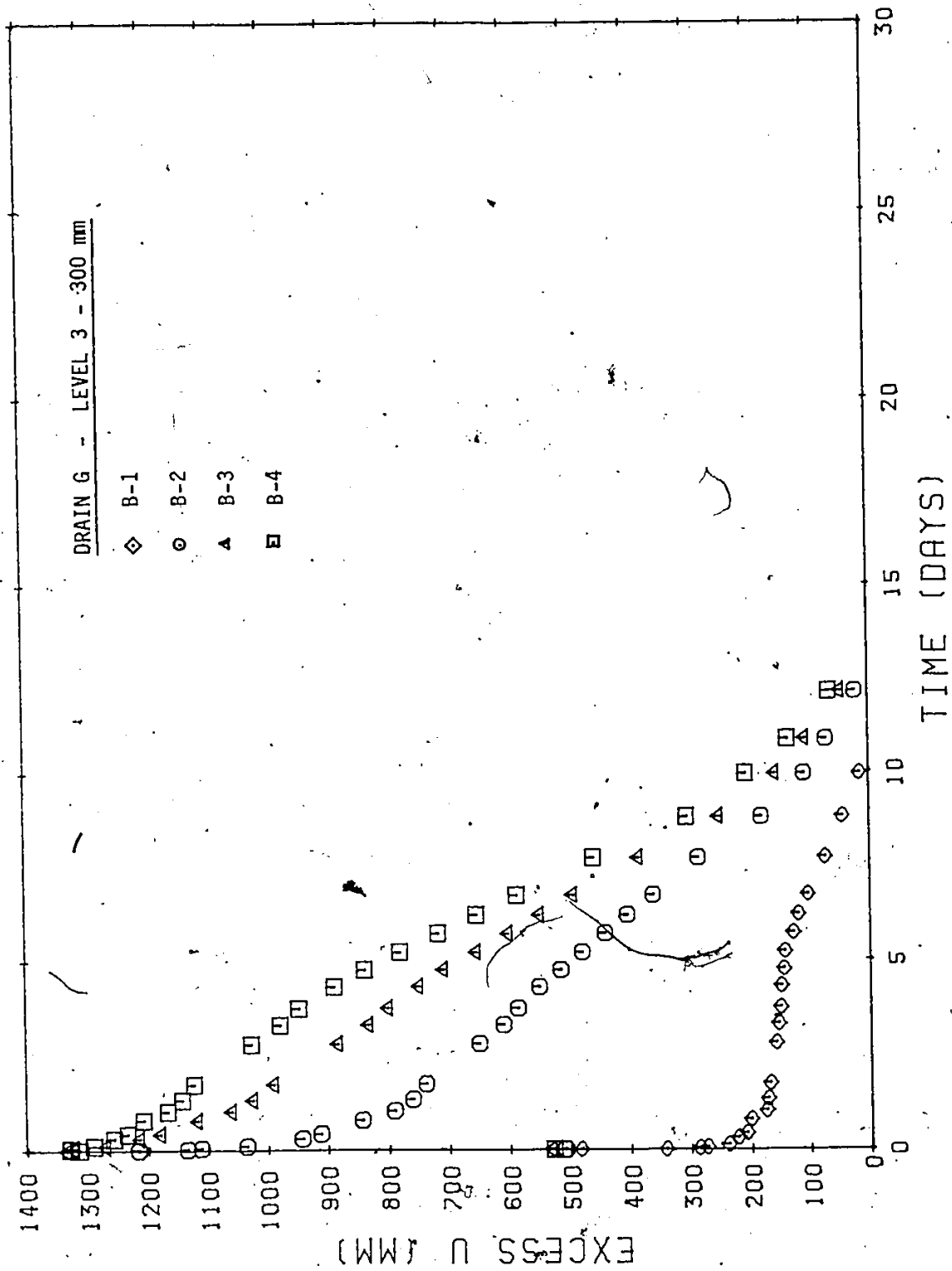


FIG. A3.3 : EXCESS PORE PRESSURE - TEST SERIES TWO

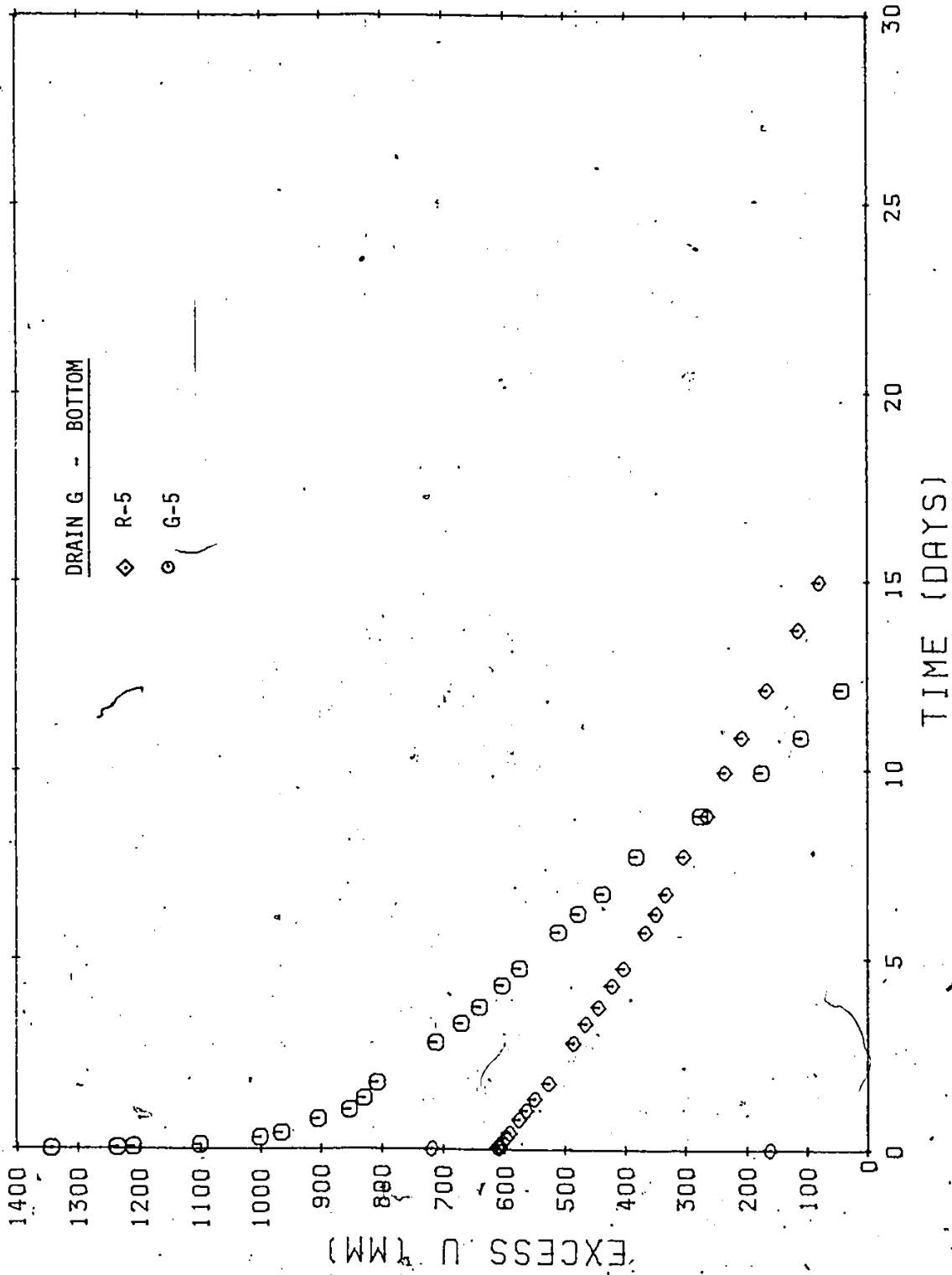


FIG. A3.3 : EXCESS PORE PRESSURE - TEST SERIES TWO

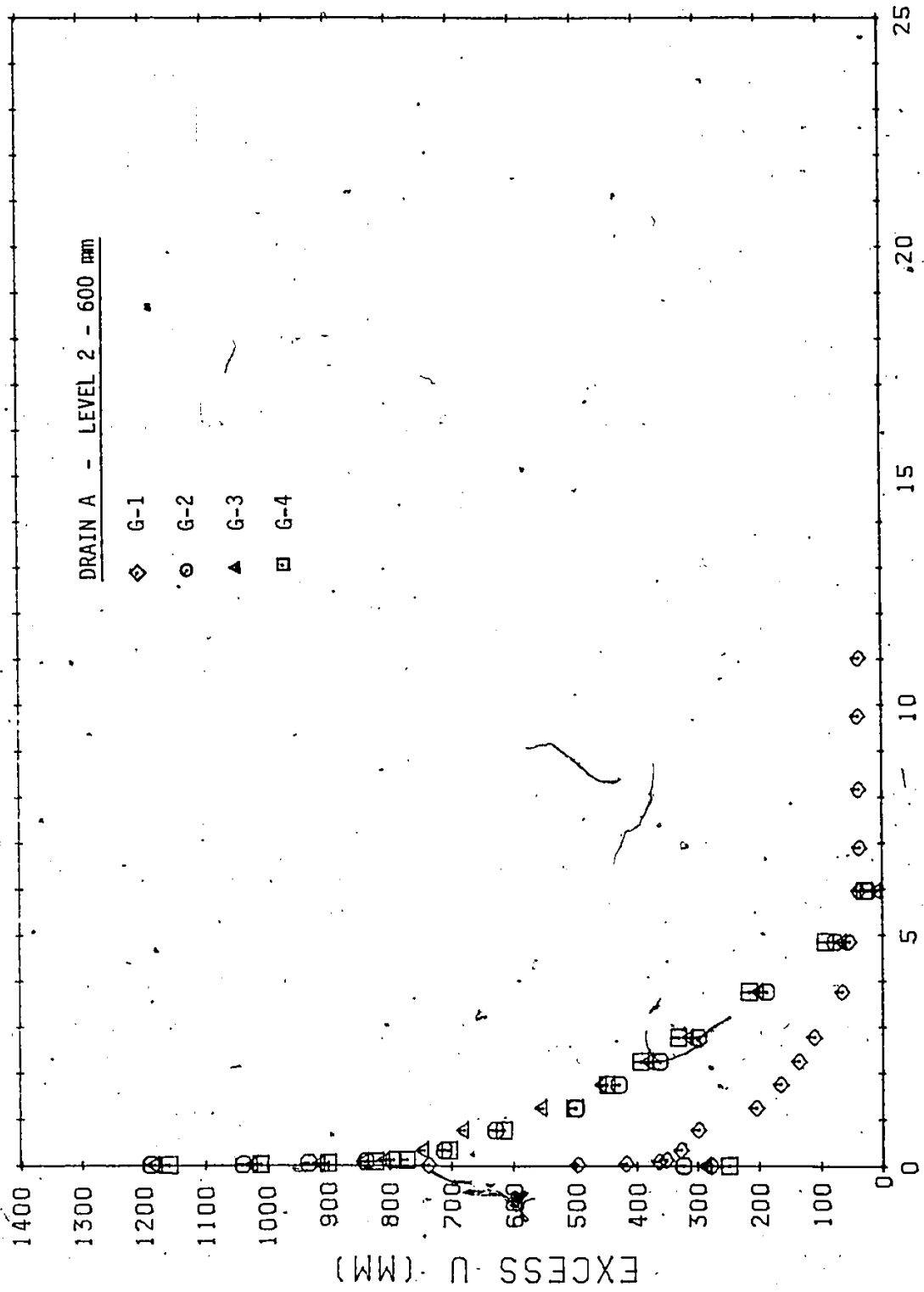


FIG. A3.4 : EXCESS PORE PRESSURE - TEST SERIES TWO

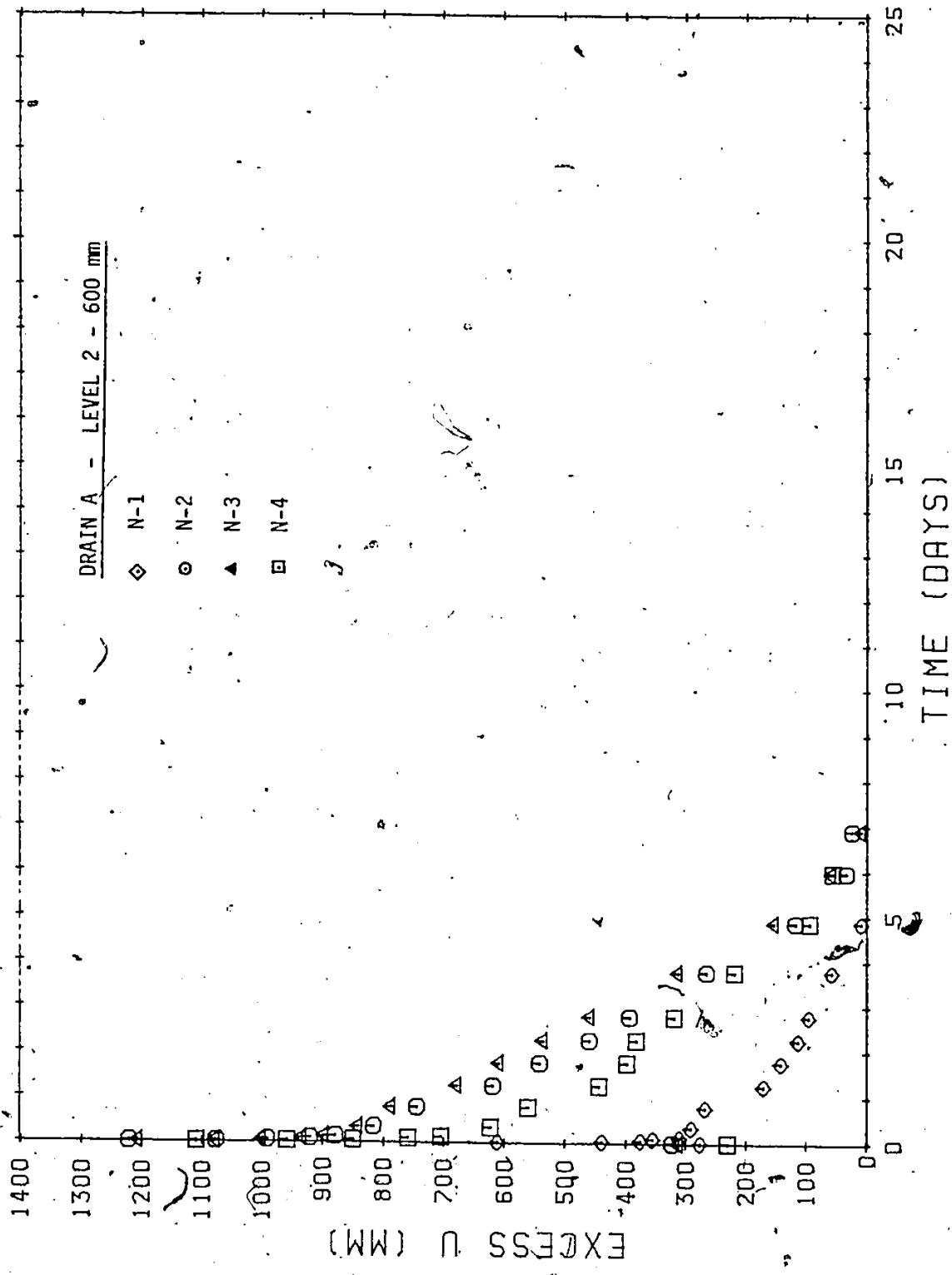


FIG. A3.4 : EXCESS PORE PRESSURE - TEST SERIES TWO

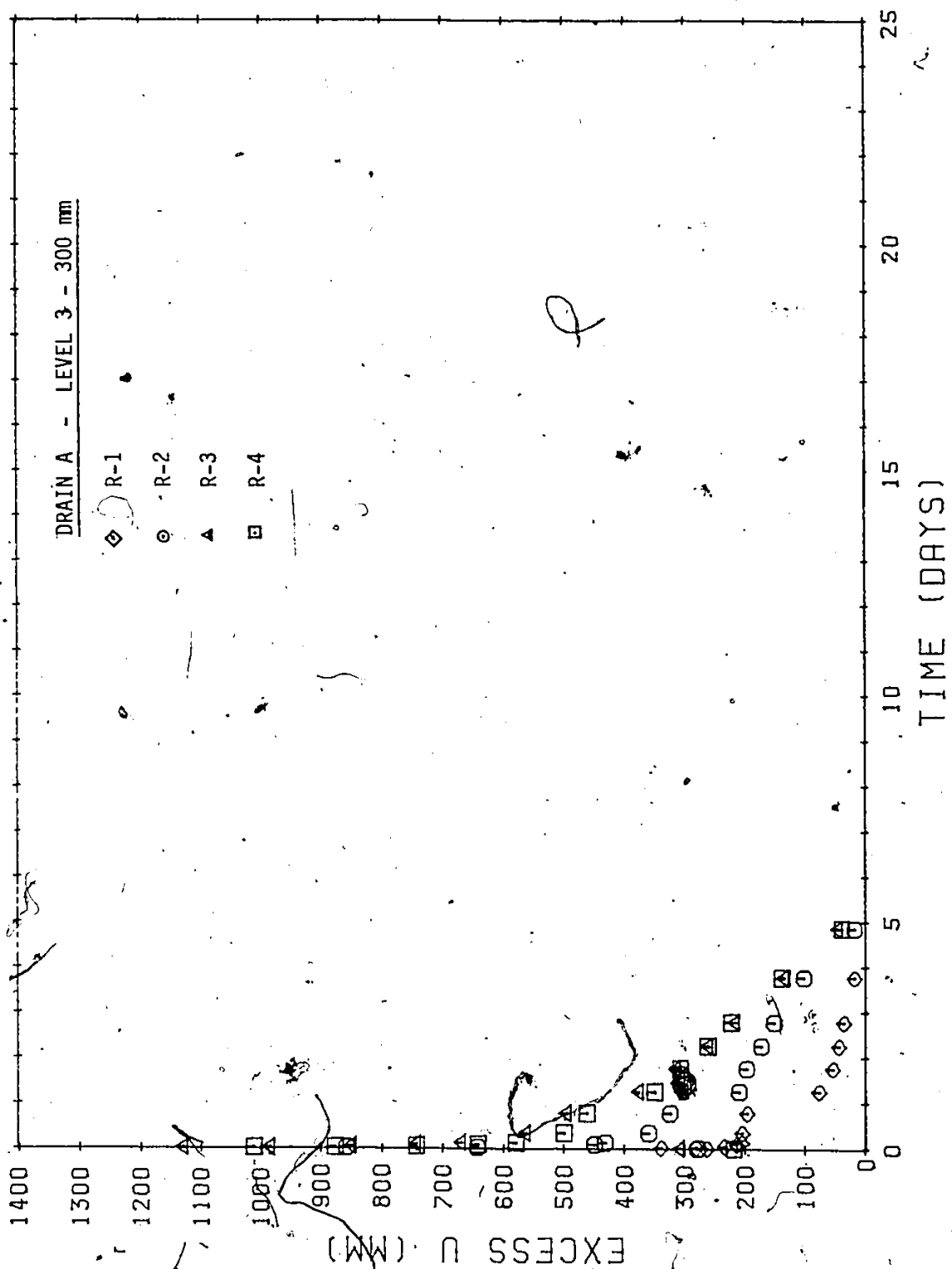


FIG. A3.4: EXCESS PORE PRESSURE - TEST SERIES TWO

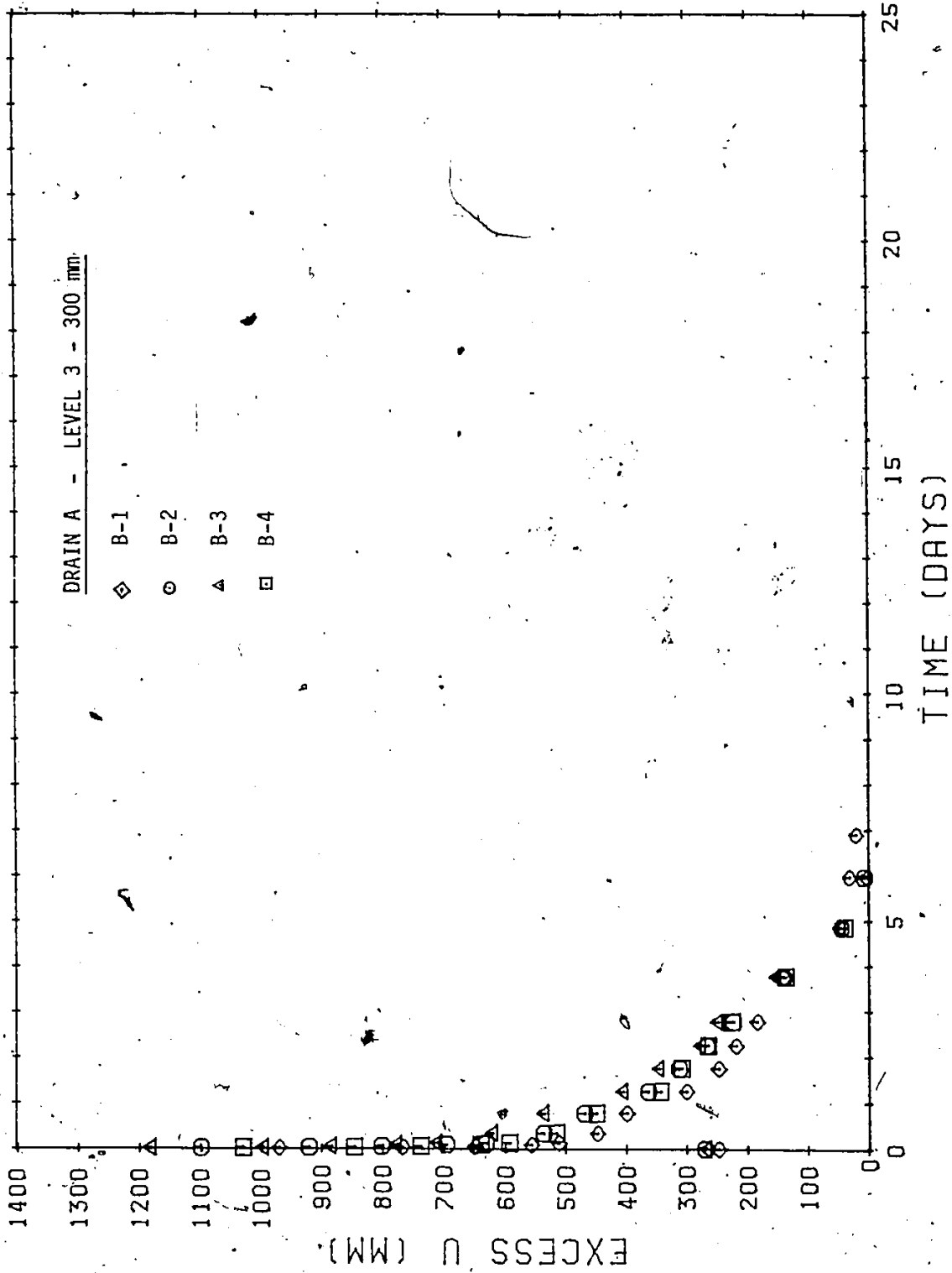


FIG. A3.4 : EXCESS PORE PRESSURE - TEST SERIES TWO

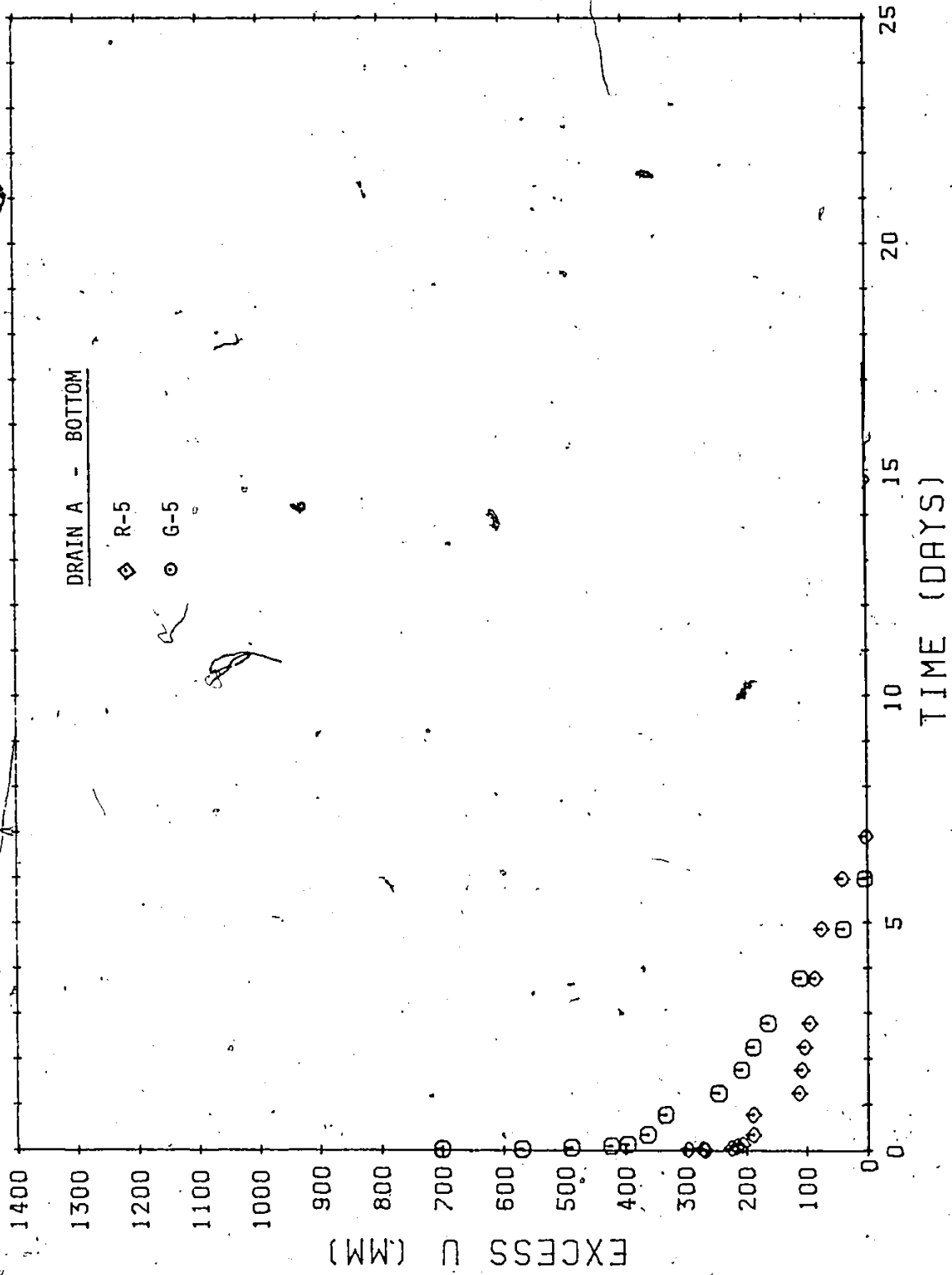


FIG. A3.4 : EXCESS PORE PRESSURE - TEST SERIES TWO

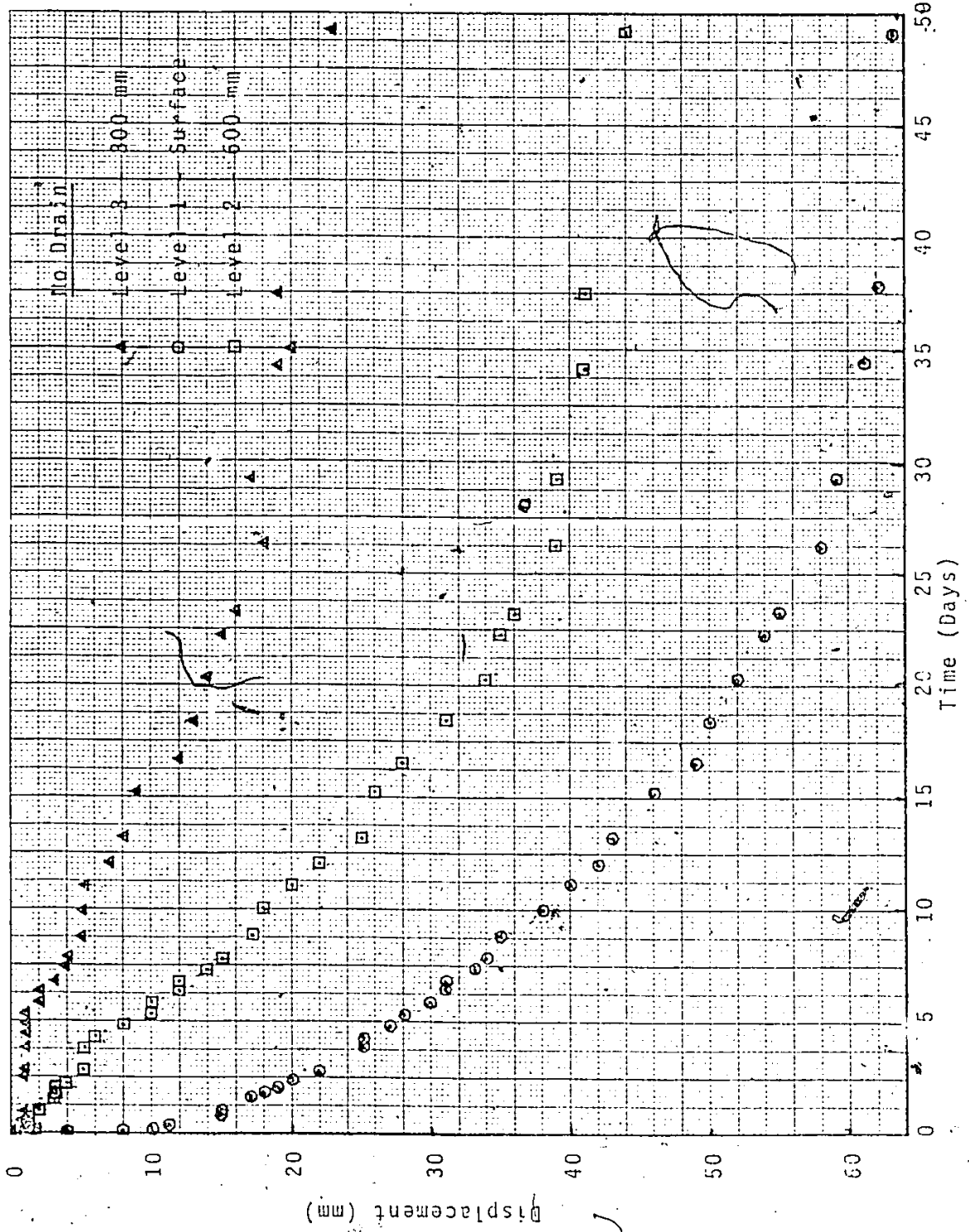


Fig) A4.1 : Settlement Curves - Test Series Two

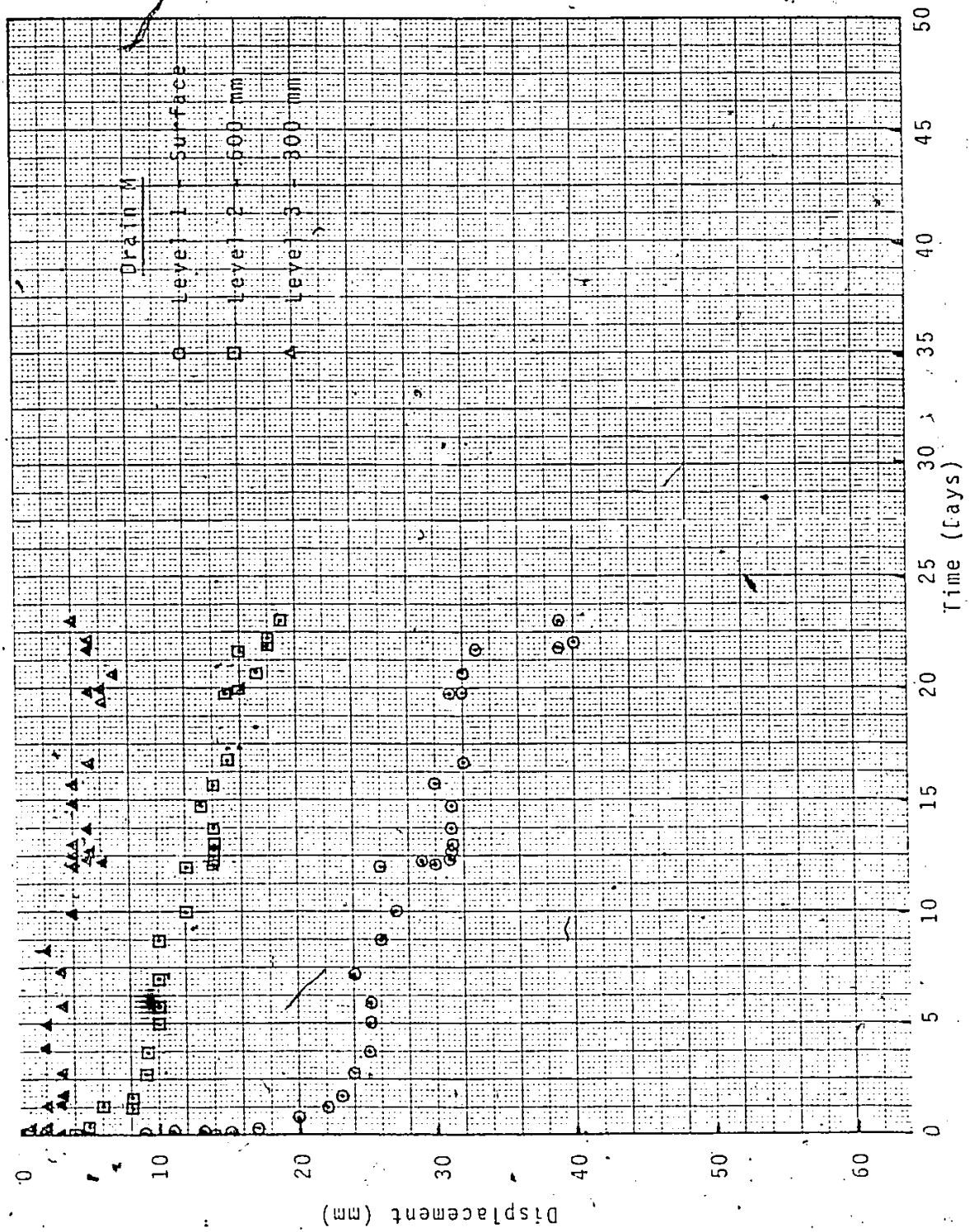


Fig. A4.2 : Settlement Curves - Test Series Two

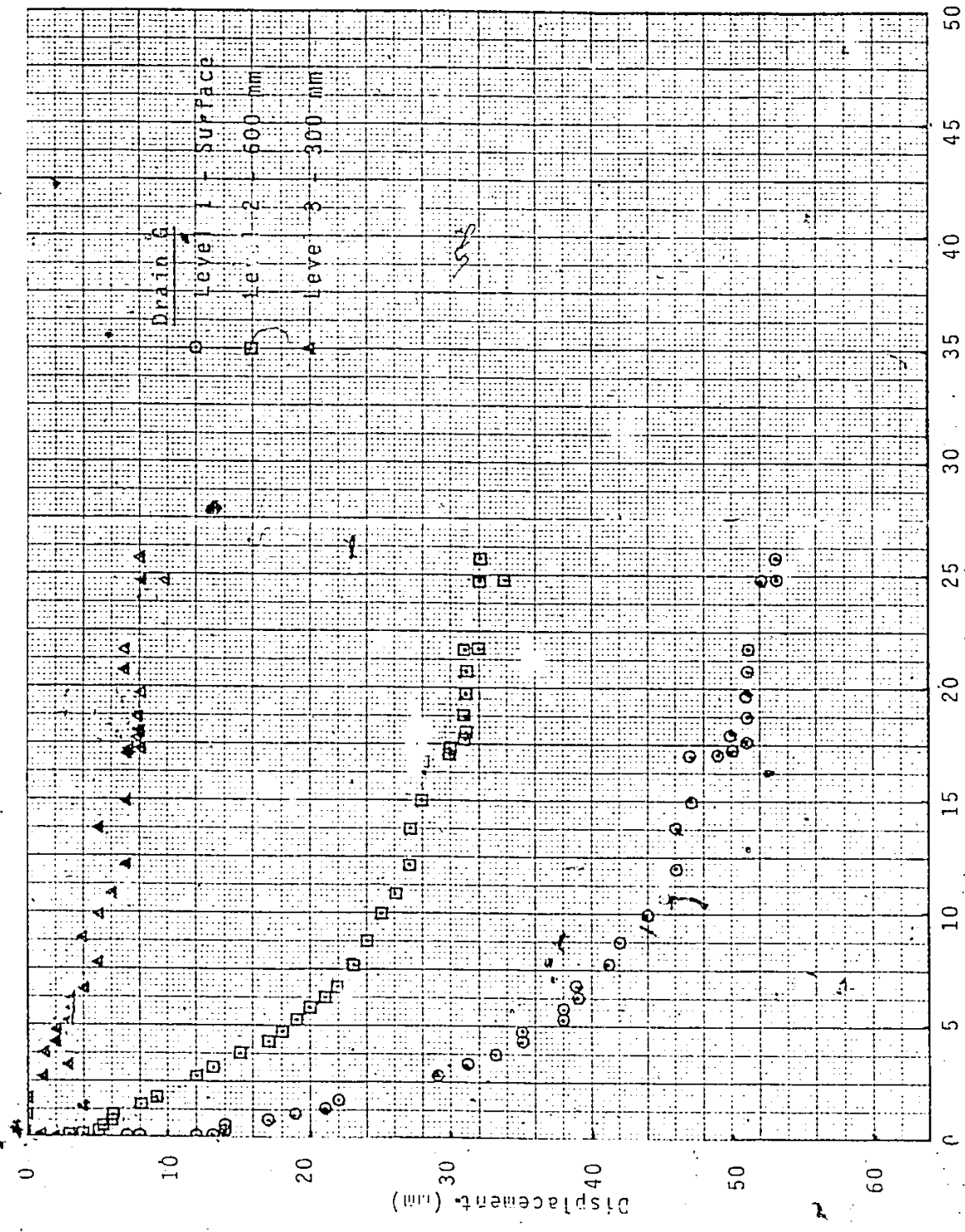


Fig. A4.3 : Settlement Curves - Test Series Two

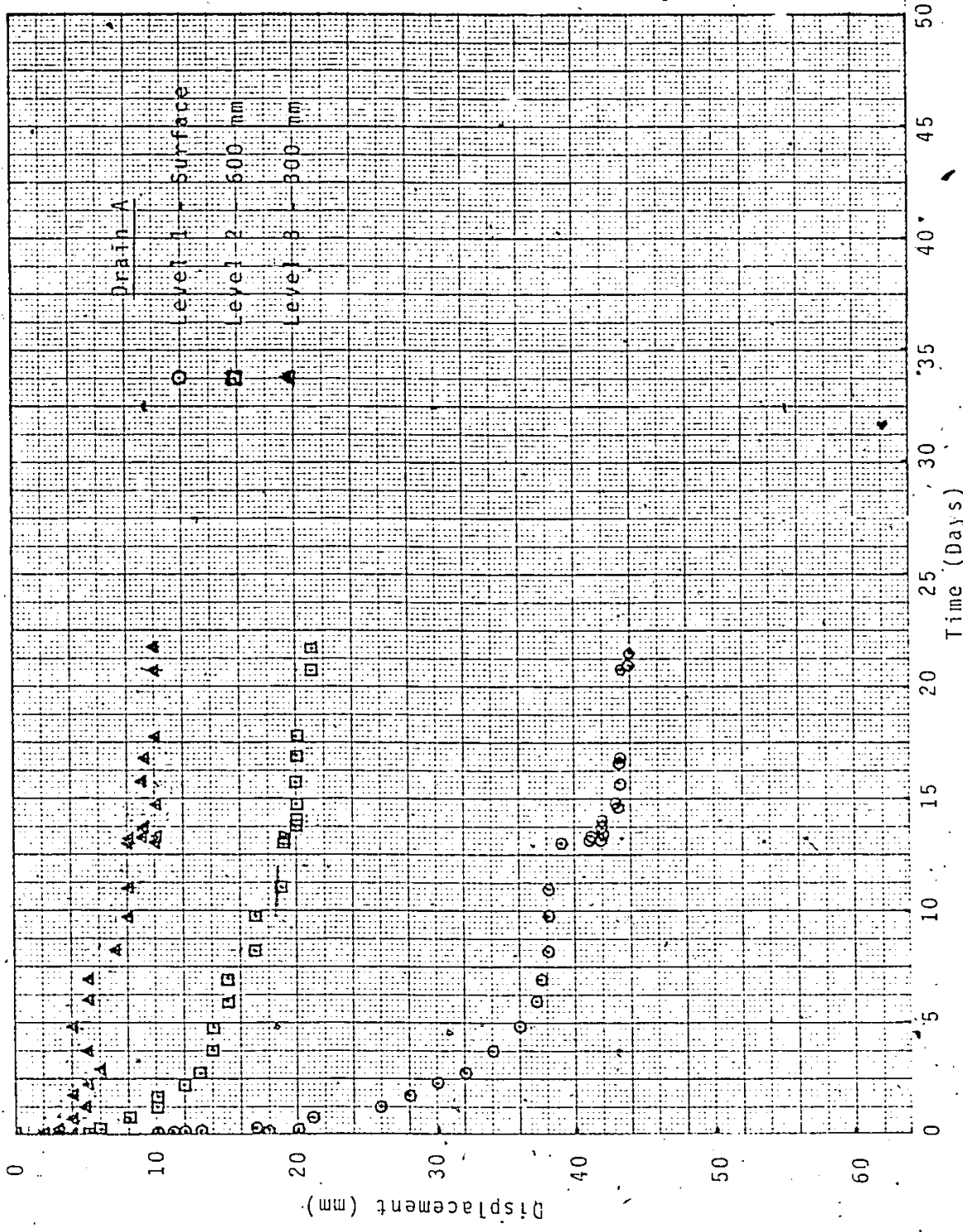


Fig. A4.4 : Settlement Curves - Test Series Two

APPENDIX B

Flk B11: Consolidation Projections - Asoka's Method

METRIC

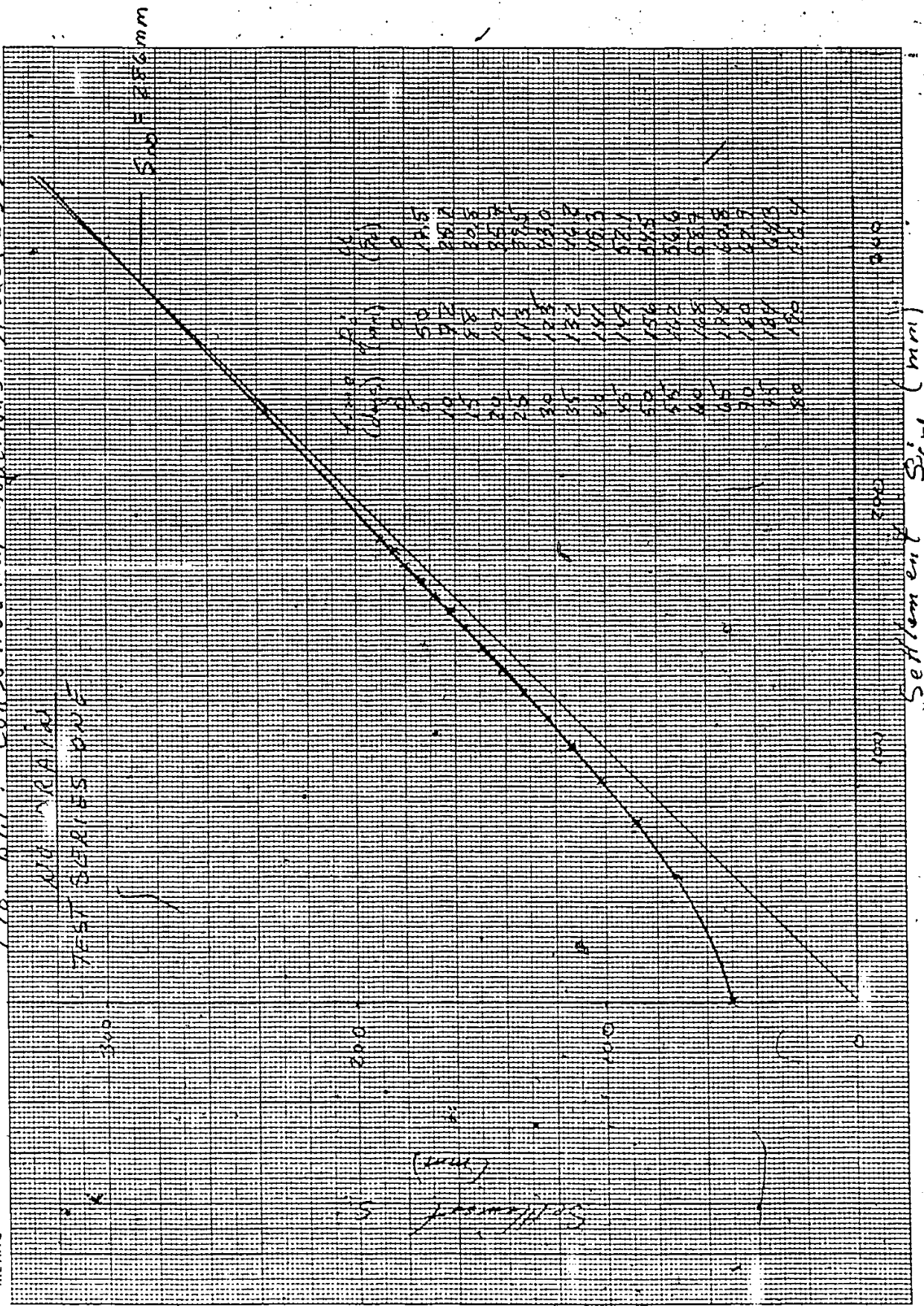
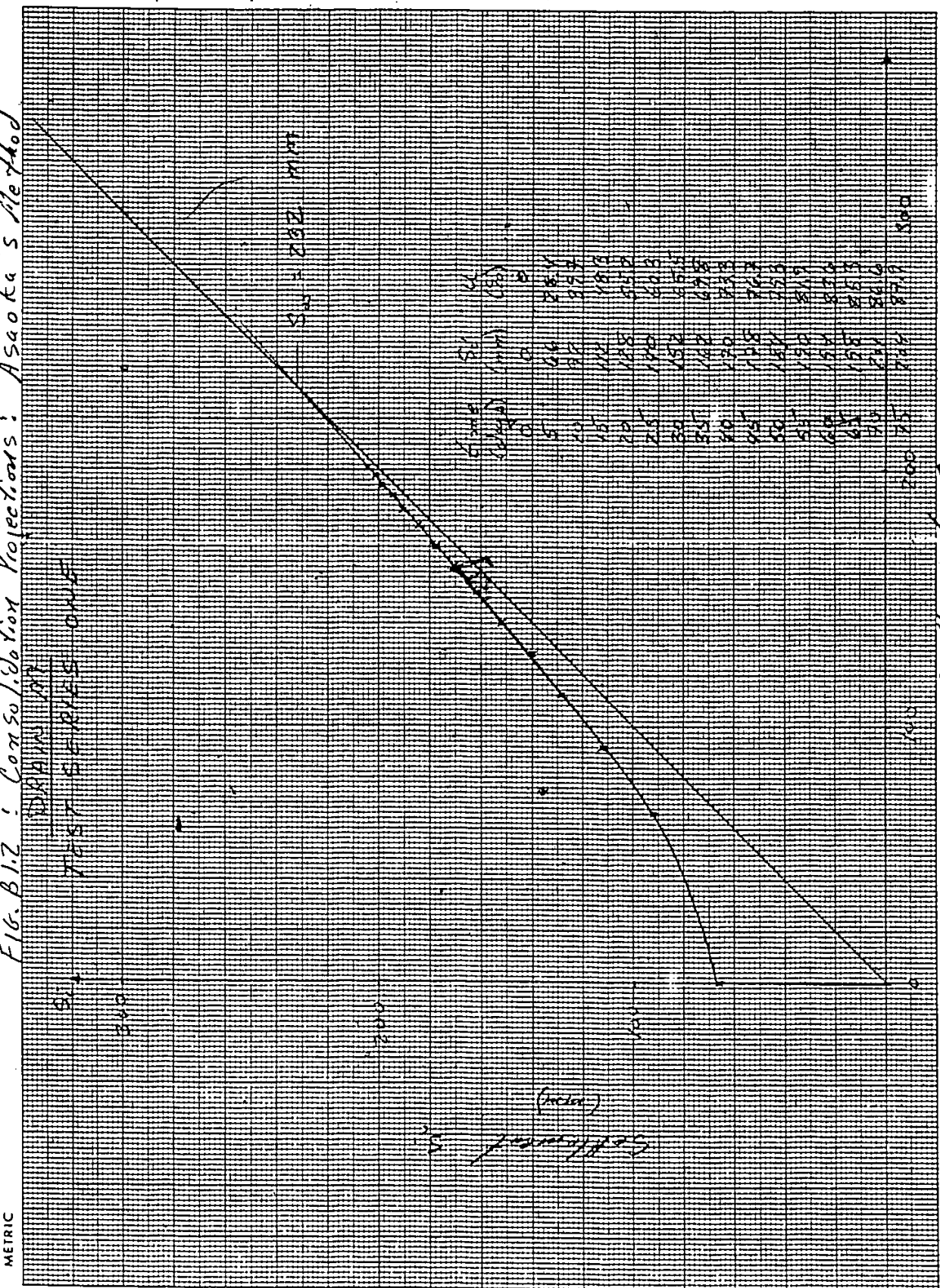


Fig. B1.7 : Consolidation Projections: Asoka's Method



Settlement Sc-1

FIG. B.1.3 : Consolidation Projection Projected - Asaoka's Method

METRIC

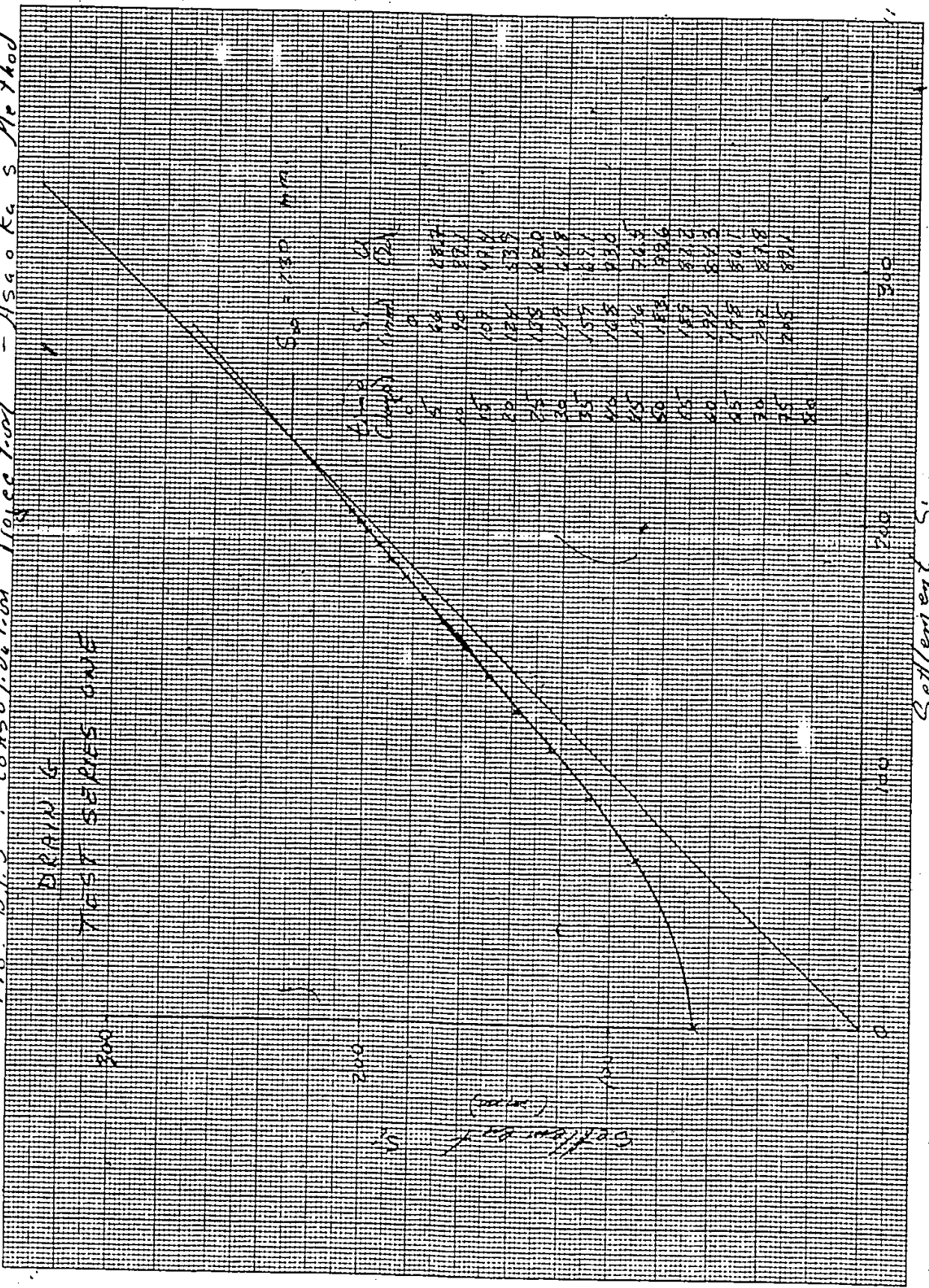


FIG. B.1.1. Consolidation Projection - Asoka's Method

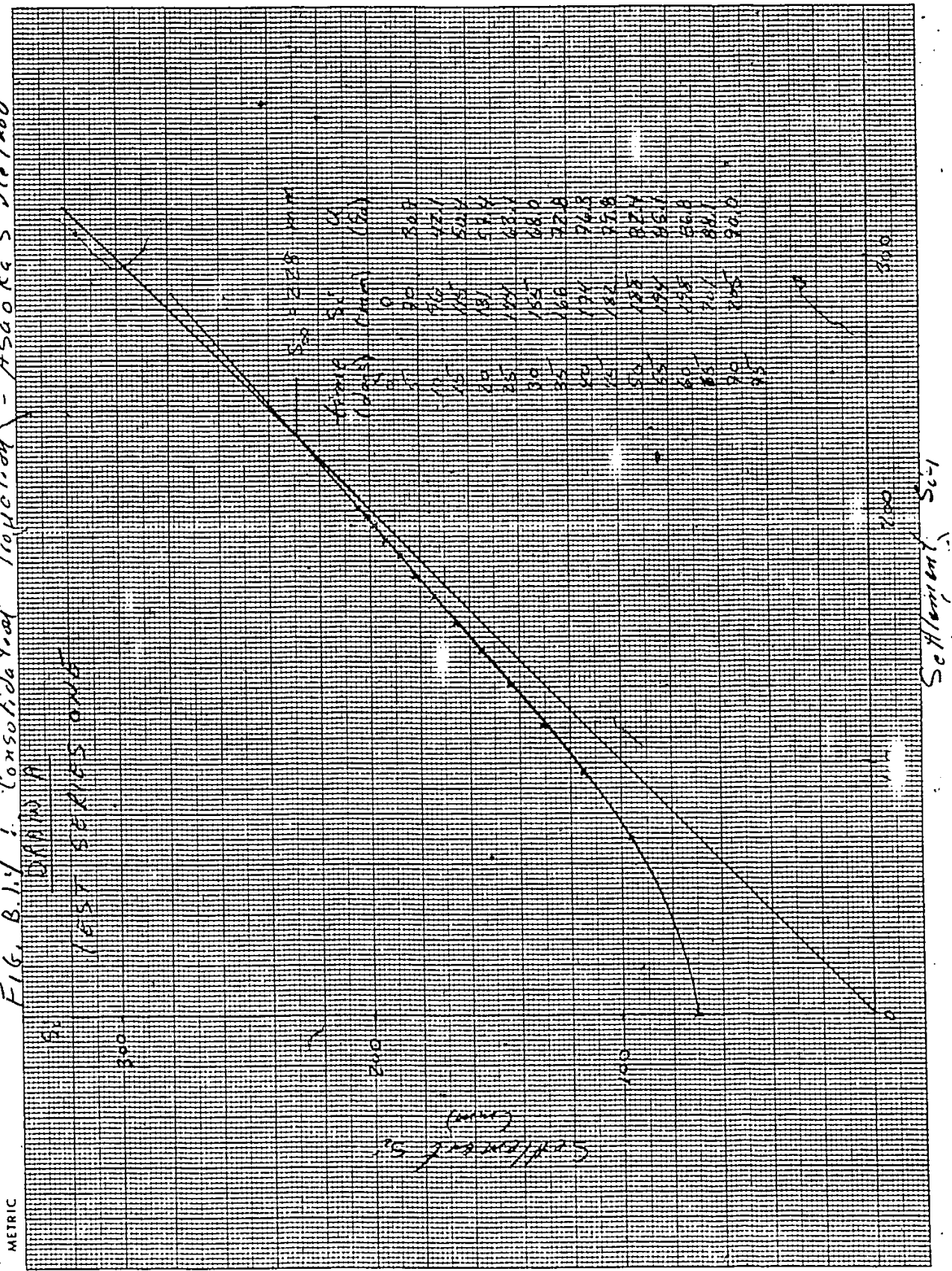
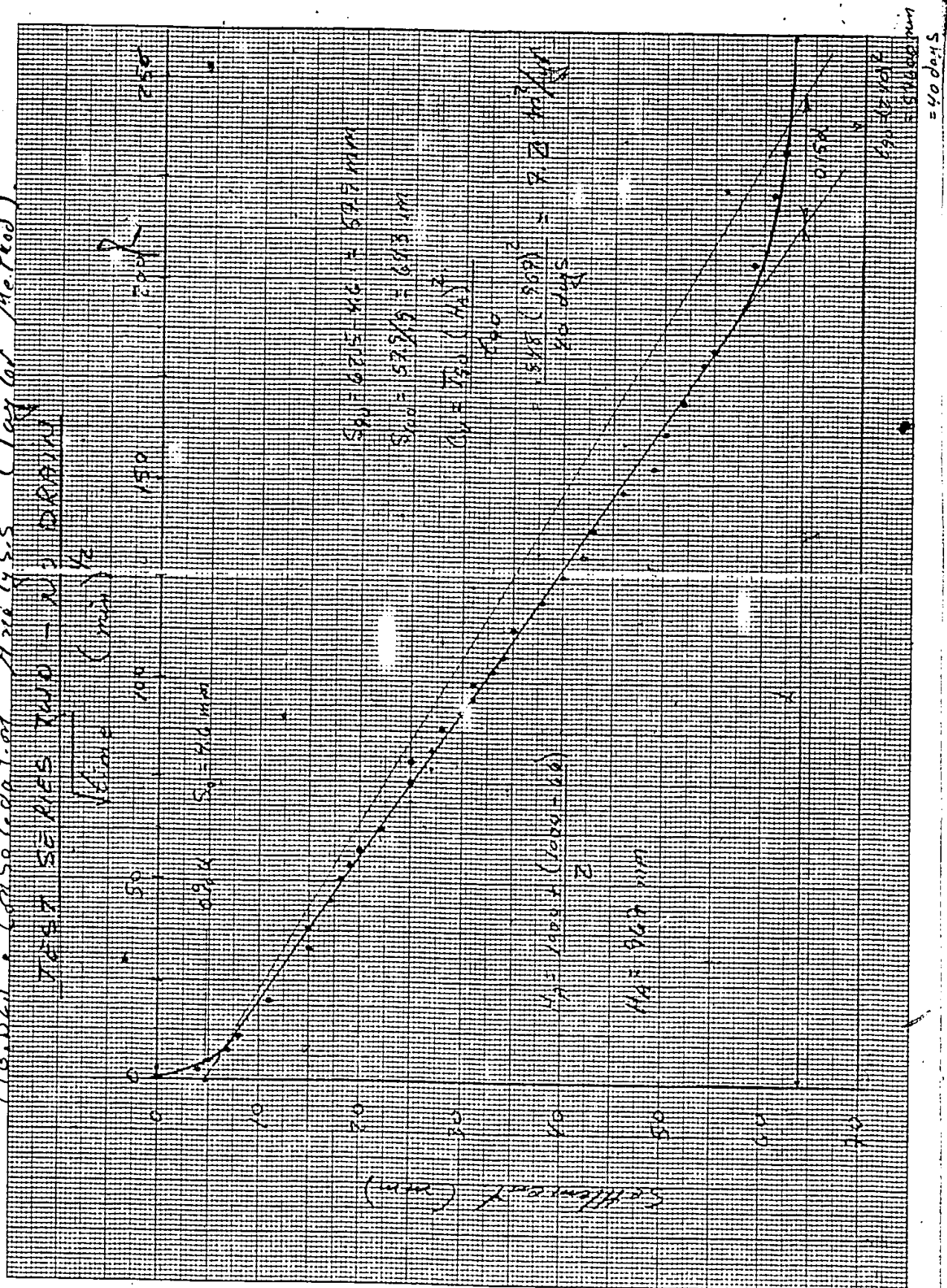


Fig. B21 - Consolidation Analysis (Taylor Method)



METRIC

FIG. 822: CONVERSION TABLE FROM 1500 TO 2000 (SPIN SPEED TEST SERIES TWO) - CONTINUED

TEST BEARING TWO - CONTINUED

3010000

0 50 100 150 200

0.004 51.85500 SPS = 230.85 = 145 RPM
S₁₀₀₀ = 1457.9 = 16.1 RPM

St. Helmut (mm)



0.004 51.85500 SPS = 230.85 = 145 RPM
S₁₀₀₀ = 1457.9 = 16.1 RPM

METRIC

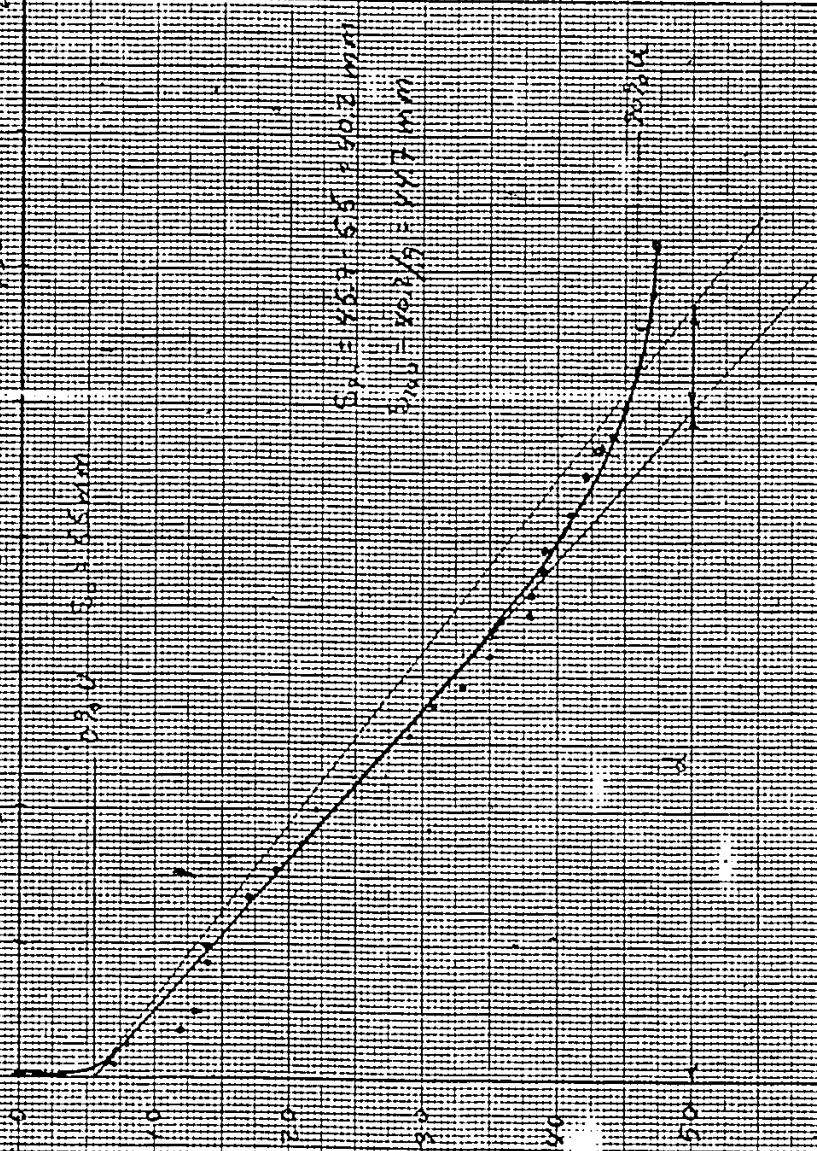
FIG. 823 - COMBINATION ANALYSIS (TAYLOR METHOD)

TEST SERIES TWO - DRAW 5
JUNE 1948

50 100 150 200

0.200 0.300 0.400

Settlements (mm)



S.P. = 467.55 = 40.2 mm

S.P. = 402.4 = 44.7 mm

S.M. = 18.225 mm
E.I. = 7.0 mm

METRIC

