TECHNIQUES TO EVALUATE
THE FIELD PERFORMANCE OF VERTICAL DRAINS

by

PATRICE ORLEACH

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Signature of Author: ______________________________
Department of Civil Engineering
September 7, 1983

Certified by: ______________________________
Charles C. Ladd
Thesis Supervisor

Accepted by: ______________________________
Francois M.M. Morel
Chairman, Department Committee

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ABSTRACT

Foundation stabilization of clay deposits by precompression often utilizes vertical drains to accelerate the rate of consolidation. An important design problem concerns selecting the coefficient of consolidation for horizontal drainage, c_h, needed to determine the required drain spacing. This problem occurs due to uncertainties both in the consolidation properties of the clay deposit and in the efficiency of the available types of sand drains and prefabricated band-shaped "wick" drains. Case histories can provide a valuable guidance but only if the techniques used to interprete field settlement and pore pressure data yield reliable results.

The research used theoretical analyses to evaluate three potential factors affecting field rates of consolidation with vertical drains, with the following conclusions: (1) "well resistance" should not usually inhibit consolidation for good quality wick drains and sand drains; (2) the influence of vertical drainage can vary from being insignificant regarding dissipation rates measured via typical piezometer installations to very important regarding rates of surface settlement when the ratio of vertical to horizontal time factor exceeds about 0.1; (3) the effects of the "smear zone" created during installation of displacement sand drains and wick drains can be extremely important, but also difficult to predict due to the uncertainties in the likely thickness and permeability of the remolded soil in this zone.

The thesis reviews the limitations of "conventional" methods of analysis of field performance data based on using chart solutions and presents two new techniques :

(1) use of Asaoka's(1978) procedure for analysing settlement data, which has the great advantage of being able to obtain the field c_h without having to predict the

magnitude of initial settlement or the amount of primary consolidation settlement;

(2) use of log excess pore pressure versus time plots to obtain c_h without having to know the initial excess pore pressure or the location of the piezometer relative to the vertical drains.
These techniques yield an "operational" c_h that should be valid over time spans having constant load, constant c_h and negligible contributions from vertical drainage.

The methods are applied to a precompression project with Alidrains having extensive settlement and piezometer data during virgin compression of a 65 ft thick plastic deltaic clay located in Mobile, Alabama. Fourteen piezometers yielded a mean c_h two times larger than the laboratory c_v, whereas c_h from surface settlements measured at three locations varied from two to six times c_v, depending upon the time span and type of measurement device. Possible reasons for the larger c_h from settlement data are discussed.

Thesis Supervisor: Dr. Charles C. Ladd,
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LIST OF SYMBOLS AND NOTATION

CR	Virgin compression ratio
c_h	Coefficient of consolidation for horizontal drainage
c_v	Coefficient of consolidation for vertical drainage
d_e	Equivalent drain spacing
d_w	Drain diameter
El.	Elevation
F(n)	Spacing factor
$F_s(n)$	Spacing factor which includes smear
H_d	Drainage height
Ideal	Ideal Basic Industries, Inc.
k_h	Horizontal coefficient of permeability
k_v	Vertical coefficient of permeability
k_w	Vertical coefficient of permeability of the drain
ℓ	Characteristic length of the drain
m_v	Coefficient of volume change
n	Spacing ratio
OCR	Overconsolidation ratio
RLSA	Reserve Limestone Storage Area
q_w	Discharge capacity of the drain
r	Coefficient of linear regression, or radius from point to centerline of the drain
s	Smear spacing ratio
T_h	Time factor for radial drainage
T_v	Time factor for vertical drainage
t	Time

U	Degree of consolidation
U_h	Degree of consolidation for radial drainage
U_v	Degree of consolidation for vertical drainage
W_R	Well resistance parameter
u	Pore pressure
u_o	Initial pore pressure
u_n	Pore pressure at time t_n
α_o, α_1	Linear coefficients of the relation: $\ln(u)=\alpha_o-\alpha_1 t$
ρ_o, β_1	Linear coefficients of the relation: $\rho_n=\rho_o+\beta_1\rho_{n-1}$
Δ	Prefix indicating a difference or an increment
ρ	Settlement at time t
ρ_n	Settlement at time t_n
ρ_i	Initial settlement
ρ_{cf}	Final settlement at the end of primary consolidation
ρ_∞	Total settlement at the end of primary consolidation
γ_w	Unit weight of water
ν	Location factor of the tip of the piezometer relative to the drain
ν_c	Value of the location factor when the tip of the piezometer is at the middle of the drain pattern
Σ	Sum
$\bar{\sigma}_{vf}$	Final vertical effective stress
$\bar{\sigma}_{vo}$	Initial vertical effective stress
$\bar{\sigma}_{vu}$	Vertical effective stress at the end of undrained loading
Z	Depth factor

Remark: All other symbols and notations are identified in the text.

CHAPTER 1

INTRODUCTION

1-1. BACKGROUND

An increasing number of engineeering projects are located at sites underlain by weak compressible cohesive soils. Unless these soils are by-passed or replaced by more suitable materials, some method of soil improvement or foundation stabilization is required to insure adequate stability against a shear failure and/or to avoid excessive settlements of the structure. One such method which has become popular since World War II is the precompression technique. As defined by Johnson (1970a): precompression is the action of compressing the soil under an applied stress prior to placing or completing the structure load; preloading is a means for accomplishing precompression, usually by earth fill, water load in tanks, and so forth; surcharging is the action of preloading in excess of the final load.

The precompression technique can be used to eliminate all or a portion of post- construction settlements due to primary consolidation and also to greatly reduce settlements caused by secondary compression. Surcharge loads are frequently employed to decrease the time required for precompression. It is also used to minimize the effects of secondary compression.

If the foundation soils are weak, the design of a precompression scheme must also consider stability. This may require stability berms and the use of a controlled rate of loading to enable a gain in strength of the foundation soils with consolidation during preloading.

Preloading may not be feasible, even with surcharge loadings, if the time required to consolidate the layer is long compared to that available for construction. In such cases, vertical drains can be installed to accelerate the rate of consolidation by reducing the maximum drainage distance within the soil mass.

As shown in Fig. 1-1, vertical drains are installed at the start of the project and are connnected to an upper drainage blanket to handle the flow of water squeezed out of the soil. The structure or the preload is then placed above this layer. Instruments are installed to measure the settlement of the clay layer and the excess pore water pressures within the layer and sometimes to monitor lateral deformations.

The first use of vertical drains was in 1934 for highway fills in California (Johnson, 1970b). The design of drain installations was empirical until 1948 when the first theory on vertical drains was presented by Barron.

Until the mid-1960's, most American drain installations consisted of driving a large diameter closed-end mandrel into the soil and filling it with concrete sand during removal of the casing. This process caused significant disturbance to the surrounding soil, often referred to as "smear". After this period, "low-displacement" and "non-displacement" sand drains appeared and improved the efficiency of drains. The next significant advance occurred in the mid-1970's when low-cost pre- fabricated band-shaped wick drains became widely available. Table 1-1 presents the different types of vertical drains and their major characteristics.

Theory and design practice of foundations with vertical drains have evolved over the years, but some problems still exist due to the complexity of the phenomenon involved. Although analyses range from use of simple charts to sophisticated Finite Difference computer programs there are still theoretical limitations in the design process. Furthermore, the evaluation of the soil properties can be very difficult, especially the determination of the coefficient of consolidation for horizontal drainage. In fact, the relation between the coefficient of compressibility, the coefficient of permeability and the effective stress is required for a "precise" analysis. The effects of the installation and the type of the drain are also very important factors which are still difficult to evaluate in order to predict the performance of vertical drains.

To better define and understand the problems discussed above, well-documented case histories of field performance are required. Nevertheless, it is not easy to evaluate field data in order to draw definite conclusions. This thesis focuses on better methods for interpretation of field settlement and pore pressure data from drain installations.

1-2. THESIS OBJECTIVES AND ORGANIZATION

a) Objectives.

The objective of this thesis is to present new methods for evaluation of field performance of vertical drains. Techniques are detailed for analyzing settlement and excess pore pressure data for field cases having either pure radial drainage or only vertical drainage. The method for analyzing settlement data is based on Asaoka's (1978) procedure. Analysis techniques for combined radial and vertical drainage are also discussed.

These new techniques of analysis are used in a case history for a precompression project with Alidrains for a cement plant located in Mobile, Alabama. This project has already been stüdied in a previous thesis (Noiray, 1982),but using "conventional" methods of analysis and also less complete field data.

b) Organization.

Chapter 2 presents a summary of the theoretical considerations regarding vertical drains. "Ideal" drains with consolidation due only to radial drainage are first considered. The effects of vertical drainage, well resistance and "smear" are then included. Finally, the time varying loading is briefly discussed.

Chapter 3 presents the methods used to analyse the performance of drains. The "conventional" method and its related problems are first summarized. The new methods are then detailed.

Chapter 4 analyses the performance of Alidrains for the Mobile Alabama case history. After a short description of the project and a summary of the results from the previous study, analysis of the settlement and pore pressure data are presented using the methods previously presented.

Chapter 5 gives some practical recommendations on the use of the new methods presented in Chapter 3.

Chapter 6 summarizes the previous chapters and draws some conclusions.

Five appendices follow. Appendix A presents the derivation of a criteria to assess the influence of well resistance. Appendix B presents the theoretical considerations on the methods of analysis of settlement data. Appendix C does the same for the excess pore pressure data. Appendix D includes the field data and their analysis used to study the case history on Alidrains. Appendix E compares the coefficients of consolidation from an oedometer test obtained from the proposed method of analysis of settlement for pure vertical drainage and from the conventional methods.

TABLE 1-1

TYPES OF VERTICAL DRAINS

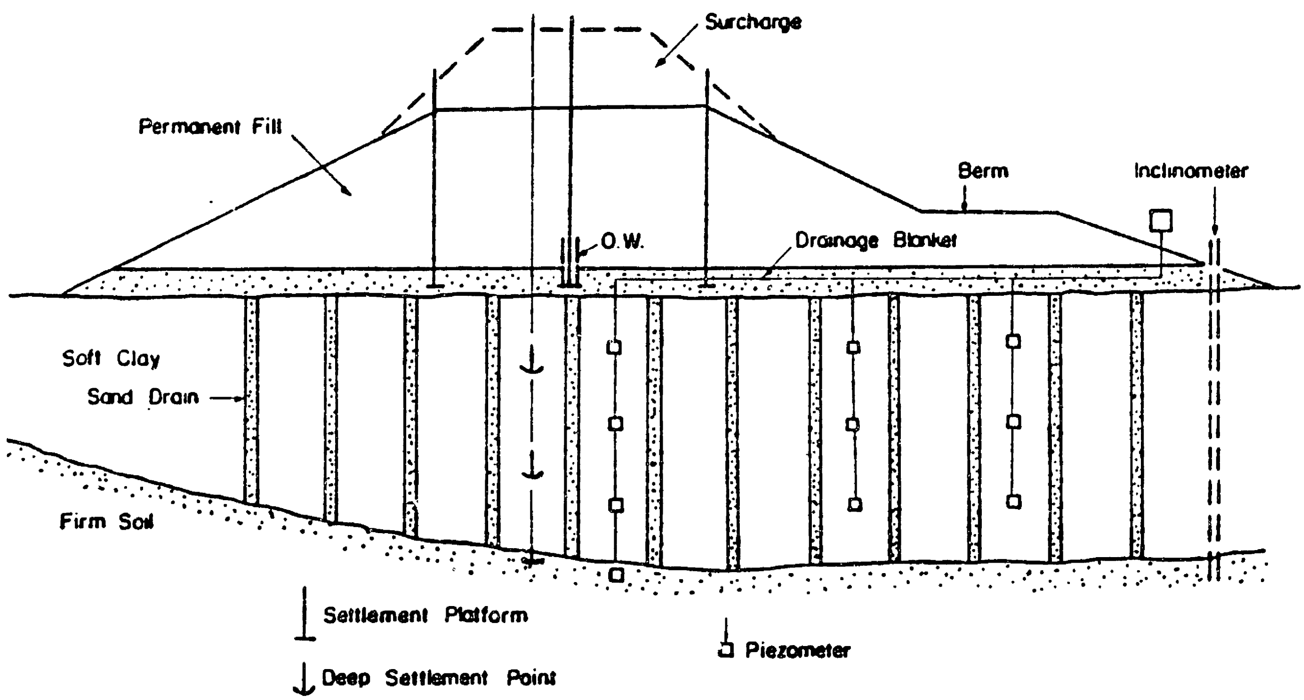
(From Jamiolkowski, Lancellota and Wolski, 1983)

Drain Type	Installation Method	Usual		
		d_w (m)	d_e (m)	L_m (m)
1) Sand drain cast in situ (D)	Impact or vibratory driven closed-end mandrel	0.2 to 0.6	1.5 to 5.0	30
2) Prefab. sand drain (D to LD)	Impact or vibratory driven close-end mandrel	0.06 to 0.15	1.2 to 4	30
3) Sand drain (cast in situ) (LD)	Continuous Flight Hollow Stem Auger	0.3 to 0.5	2.0 to 5	35
4) Sand drain (cast in situ) (ND)	Jetted	0.2 to 0.3	2.0 to 5	30
5) Prefabricated drains: plastic core + paper or non-woven filter dense (D to LD)	Statically or vibratory driven closed-end mandrel	0.05 to 0.1*	1.2 to 3.5	60

d_w = drain diameter; d_e = diameter of the equivalent soil cylinder;
L_m = maximum length; D = displacement; LD = low displacement;
ND = non-displacement

* Due to their band-shape, reference is made to the equivalent diameter (see Chapter 2).

Fig. 1-1: Typical Sand Drain Installation (from Ladd,1975).



CHAPTER 2

THEORETICAL CONSIDERATIONS ON THE CONSOLIDATION PROCESS WITH VERTICAL DRAINS

2-1. INTRODUCTION

The purpose of vertical drains is to accelerate the rate of consolidation of a clay layer by adding radial drainage to the vertical drainage. The drain spacing is usually much less than the vertical drainage distance. This leads to a consolidation process where radial drainage is predominant. Vertical drains can also be especially efficient in cases where the coefficient of consolidation for horizontal drainage, c_h , is significantly greater than the one for vertical drainage, c_v. Consolidation with pure radial drainage is the first topic to be presented in this chapter, followed by the consolidation for combined vertical and horizontal drainage.

Next, the influence of actual drains as opposed to ideal ones will be studied. The fact that drains have a finite coefficient of permeability and area may be such that, during the early stages of the consolidation process, the discharge capacity of the drains may be reached. This would decrease the rate of consolidation of the clay layer compared with ideal drains. This is called the effect of well resistance. Furthermore, the installation of drains often creates a disturbed zone around the drain reducing the coefficient of consolidation in this zone, which is called the smear effect.

Finally, the effect of the time dependent loading will be briefly discussed.

2-2. GENERAL CONSIDERATIONS ON THE THEORY OF DRAINS

Vertical drains are installed following a certain pattern. Barron (1948) concluded that a triangular spacing was the most economical one, whereas Kjellman (in the discussion of Barron's paper) preferred a square pattern (see Fig. 2-1). In both cases, the soil element considered for the model used to analyse the consolidation process is a cylinder with an impermeable outside vertical boundary and a diameter equal to the equivalent diameter, d_e, determined as a function of the pattern (see Fig. 2-1). At the center of the soil element, the cylindrical drain has a diameter d_w. For a band shaped wick drain, d_w is calculated as the diameter of a cylinder of equivalent circumference using:

$$d_w = 2(a+b)/\pi$$

where: a = drain width
b = drain thickness

Top and bottom horizontal boundaries are taken as planes, therefore completing the geometrical aspect of the model used to study vertical drains (see Fig. 2-1).

The theory of the consolidation process is based on Terzaghi's (1925) assumption for one-dimensional consolidation which states:

- --The soil is fully saturated and homogeneous.
- --Darcy's law is valid.
- --The coefficient of permeability k_v, of compressibility m_v, and of consolidation c_v, are constant during primary consolidation.
- --The flow and compression are one-dimensional.
- --The load is applied instantaneously and is constant during consolidation.

The governing equation for one-dimensional consolidation which corresponds to the consolidation of the cylinder without drain (Terzaghi, 1925) is:

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2}$$

where
u = the excess pore pressure
c_v = the coefficient of consolidation for vertical drainage
c_v = $k_v/(m_v \gamma_w)$
γ_w = weight of a unit volume of water

For three-dimensional flow in an isotropic soil (i.e. $c_v = c_h$ for $k_h/k_v = 1$), the governing equation (Terzaghi, 1925), becomes:

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

If the system is anisotropic relative to the coefficient of consolidation, it becomes (Terzaghi, 1925):

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

where $\left.\begin{matrix} c_v \\ c_h \end{matrix}\right\}$ = coefficient of consolidation for $\left\{\begin{matrix} \text{vertical} \\ \text{horizontal} \end{matrix}\right\}$ flow

To finally complete the model, the top boundary needs to be more precisely defined. Barron (1948) defined two cases:

* The Free vertical strain boundary: equal surface stress at the top of the cylinder, but therefore non-uniform strain.

* The Equal vertical strain boundary: equal surface strain, but thus non- uniform stress.

A mixed situation between the two cases should probably be found in actual field conditions, depending on the rigidity of the imposed load and the compressibilities of the soil and the drains. The "Equal strain" boundary is usually used in practice because it leads to an easier solution and is sufficiently accurate considering the uncertainties of the soil properties (Hansbo, 1982).

Barron (1948) developed solutions for both types of boundaries for pure radial drainage and concluded that except for small degrees of consolidation, both conditions yield similar solutions when the spacing ratio $n = d_e/d_w$, is higher or equal to five. Other theoretical solutions were developed by Kjellman (1937) for Equal strain boundary for radial drainage only and by Yoshikuni (1974) for Free strain boundary and including vertical drainage in the isotropic case and the effect of well resistance. The first expression for pure radial drainage was developed by Rendulic (1935) for the Equal strain boundary.

2-3. THEORY FOR PURE RADIAL DRAINAGE

For pure radial drainage, the governing equation becomes:

$$\frac{\partial \bar{u}}{\partial t} = c_h \left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}\right)$$

or

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

For the case of Equal strain boundary, Barron's solution (1948) is:

$$u = \frac{\bar{u}_o}{F(n)} \left[\ln\left(\frac{r}{r_w}\right) - \frac{(r/r_e)^2 - 1/n^2}{2}\right] e^{-\lambda} \qquad \text{(Eq. 2-1)}$$

where:

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

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

or under the simplified form: $F(n) = \ln(n) - 3/4$

u = excess pore pressure at any point and any time

$\bar{u}$ = average excess pore pressure throughout the soil mass at time t
= $\bar{u}_o$ at t=0.

r = distance of the point to the center line

n = r_e/r_w, the spacing ratio

T_h = horizontal time factor;
= $c_h t/(d_e)^2$

The average degree of consolidation over a section of cylinder becomes:

$$\bar{U}_h = U_h = 1-\bar{u}/\bar{u}_o = 1-e^{-\frac{8T_h}{F(n)}} \qquad \text{(Eq. 2-2)}$$

The average degree of consolidation over depth for settlement data is:

$$\bar{U}_h = \frac{\rho - \rho_i}{\rho_\infty - \rho_i} = \frac{\rho-\rho_i}{\rho_{cf}} \qquad \text{(Eq. 2-3)}$$

where:
- ρ = settlement at time t
- ρ_i = initial settlement
- ρ_∞ = total settlement at the end of primary consolidation
- $\rho_{cf} = \rho_\infty - \rho_i$, final settlement due to primary consolidation

2-4. CONSOLIDATION THEORY FOR BOTH RADIAL AND VERTICAL DRAINAGE

In the theory presented up to now, no vertical drainage is considered. Barron (1948) used Carrillo's relation (1942) to account for combined drainage. This states:

$$u = \frac{u_v u_h}{\bar{u}_o}$$

where:
- $\bar{u}_o$ = average initial excess pore pressure throughout the soil mass
- u = excess pore pressure at any point and any time
- u_v =, u_h = excess pore pressure to {vertical, horizontal} flow

As shown by Barron (1948), u_o is not uniform for Equal strain theory (see Eq. 2 -1), but "with passage of time the distribution approaches that for the free strain case", i.e. the initial assumption on the distribution does not affect the solution after the early stage of the consolidation process.

Averaging over a section of the cylinder, leads to:

$(1-\bar{U})=(1-\bar{U}_v)\times(1-\bar{U}_h)$ averaged over depth (Eq. 2-4)

or

$u/\bar{u}_o=(u_v/\bar{u}_o)\times(u_h/\bar{u}_o)$ at one point (Eq. 2-5)

where: v = vertical drainage
h = horizontal drainage

Terzaghi's solutions for U_v and $\bar{U}_v$ are expressed as Fourier series which are:

at (T_v, Z): $$1-U_v = \frac{4}{\pi}\sum_{m=1,3\ldots}^{\infty}\frac{1}{m}\sin\left(m\frac{\pi}{2}Z\right)e^{-\frac{\pi^2}{4}m^2T_v}$$ (Eq. 2-6a)

where: $T_v = c_v t/H_d^2$, time factor for vertical drainage

$Z = z/H_d$, depth parameter.

H_d = drainage height

and

at (T_v): $$\bar{U}_v = 1 - \frac{8}{\pi^2}\sum_{m=1,3\ldots}^{\infty}\frac{1}{m^2}e^{-\frac{\pi^2}{4}m^2T_v}$$ (Eq. 2-7a)

For T_v higher than 0.1, these last two equations can be simplified to:

$$1 - U_v = \frac{4}{\pi}\sin\left(\frac{\pi}{2}Z\right)e^{-\frac{\pi^2}{4}T_v}$$ (Eq. 2-6)

and

$$\bar{U}_v = 1 - \frac{8}{\pi^2}e^{-\frac{\pi^2}{4}T_v}$$ (Eq. 2-7)

Unfortunately, T_v is most of the time lower than this value of 0.1 in the case of primary consolidation with vertical drains:

if $T_v / T_h = 0.01$, $T_v = 0.1$ is equivalent to $T_h = 10.0$

if $T_v / T_h = 0.2$, $T_v = 0.1$ is equivalent to $T_h = 0.5$

Therefore, the simplified form of $(1-U_v)$ and $\bar{U}_v$ cannot be used to express vertical drainage in the case of consolidation with drains. Nevertheless one can use the following approximation of $\bar{U}_v$ which is valid for $T_v \leq 0.2$ (Terzaghi, 1943) to express the vertical drainage averaged over depth:

$$\bar{U}_v = \sqrt{\frac{4T_v}{\pi}} \qquad \text{(Eq. 2-8)}$$

By combining Eq. 2-2 and Eq. 2-6 in Eq. 2-5, one will determine the averaged degree of consolidation for combined vertical and horizontal drainage at any depth and at any time:

$$\text{at } (T_v, Z): \quad 1-U = \frac{4}{\pi} \sum_{m=1,3\ldots}^{\infty} \frac{1}{m} \sin\left(m\frac{\pi}{2} Z\right) e^{-\left(\frac{\pi^2}{4}\frac{T_v}{T_h}m^2 + \frac{8}{F(n)}\right)T_h} \qquad \text{(Eq. 2-9)}$$

This relation was used to calculate the values of 1-U versus depth for different time factors plotted in Figs. 2-2 and 2-3. Figure 2-2 presents a case representative of sand drains with n = 10 for two values of the time factor ratio, $T_v / T_h = 0.01$ and 0.2. Figure 2-3 presents a case representative of wick drains with n = 25 for the same time factor ratios. The vertical lines represent $(1-U_h)$ which are constant over depth for the different time factors, T_h. These two figures show that vertical drainage predominates near the top permeable boundary and that its effect decreases with distance from this boundary, defining a zone of influence which gets larger with time. By comparing these two figures, one can see that the effect of vertical drainage is somewhat more important for n = 25, i.e. wick drain case, than for n = 10, i.e. sand drain case, at a given T_v/T_h. However, wick drains are usually installed at a closer spacing than sand drains, thus leading to smaller values of T_v/T_h. Hence in practice, effects of vertical drainage will usually be less significant for wick drains than for sand drains.

Figures 2-4 (n=10) and 2-5 (n=25) present a similar comparision, but (1-U) is now expressed as a function of T_h at different depths for T_v/T_h equal to 0.01 and 0.2. From these figures it can be seen that for the low ratio of T_v/T_h the effect of vertical drainage can be neglected below a depth, Z, around 0.2 to 0.3. For the high ratio of T_v/T_h, on the other hand, vertical drainage should be considered at most all depths.

Figure 2-6 presents the average degree of consolidation $\bar{U}$ versus the time factor T_h for n equal to 10 and 25 and T_v/T_h equal to 0.01 and 0.2. This figure also shows the large influence of vertical drainage at a high T_v/T_h ratio.

From all these figures, one can conclude that the effect of vertical drainage will have to be included in an analysis of performance of vertical drains with settlement data when the ratio T_v/T_h is higher than about 0.01 - 0.02. Regarding excess pore pressure data, it will depend on the value of the depth factor Z and on the value of the degree of consolidation (see Figs. 2-2 through 2-5). In Chapter 3, this effect on the analysis of performance of drains will be considered because one could overestimate the coefficient of consolidation, c_h, if vertical drainage is neglected.

2-5. EFFECT OF WELL RESISTANCE

The effect of well resistance deals with the fact that the discharge capacity of the drain may be reached during the early part of consolidation. Such an event would retard the process. Barron (1948) developed a solution including this effect for the case of Equal strain boundary, but his relation is time consuming to calculate. Yoshikuni (1974) presented a solution of the Free strain case with effects of vertical drainage and well resistance. This solution is also difficult to use. Hansbo (1981) presented a simple solution which is unfortunately questionable. Nevertheless the correct part of the develcpment of the proof can be used to define a criteria to see if the effect is negligible or not (presented in Appendix A).

From this development, the following differential system is obtained for pure radial drainage and effect of well resistance:

$$\frac{\partial u}{\partial \rho} = \frac{\gamma w}{2k_h} r_w \left(\frac{n^2}{\rho} - \rho\right) \frac{\partial \varepsilon}{\partial t} \qquad \text{(A-1)}$$

$$\left.\frac{\partial^2 u}{\partial Z^2}\right|_{\rho=1} + 2 \frac{k_h}{k_w} \left(\frac{\ell}{r_w}\right)^2 \left.\frac{\partial u}{\partial \rho}\right|_{\rho=1} = 0 \qquad \text{(A-2)}$$

where:
- u = excess pore pressure at (Z, ρ)
- Z = z/ℓ, depth factor with ℓ, either equal to half the length of the drains for fully penetrating drains and bottom drainage, or equal to the length of the drain otherwise.
- ρ = normalized radius: $\rho = r/r_w$
- k_h = coefficient of permeability for horizontal drainage
- k_w = coefficient of permeability of the drain in the vertical direction
- $\frac{\partial \varepsilon}{\partial t}$ = $-m_v \frac{\partial \bar{u}}{\partial t}$

The boundary conditions and the initial conditions of Eq. (A-1), which is the governing equation of the soil behavior, are the same as previously discussed except that Eq. (A-2) is now its boundary condition around the drain. The boundary conditions for Eq. (A-2) are the following:

$$\left| \begin{array}{l} Z = 0 \\ u_{\rho=1} = 0 \end{array} \right. \qquad \left| \begin{array}{l} Z = 1 \\ \left.\frac{\partial u}{\partial Z}\right|_{\rho=1} = 0 \end{array} \right.$$

Let the factor W_R be $W_R = 2(k_h/k_w) \times (\ell/r_w)^2$ (Eq. 2-10) so that Eq. A -2 becomes:

$$\frac{\partial^2 u}{\partial Z^2} + W_R \frac{\partial u}{\partial \rho} = 0$$

In this equation, the term $W_R \frac{\partial u}{\partial \rho}$ represents the effect of well resistance. If W_R is less than 0.1, this term can be neglected as a first order of approximation and the equations (A-1) and (A-2) lead to the solution expressed by Eq. 2-1. Therefore the proposed criteria as to when the effect of well resistance can be neglected becomes:

$$W_R < 0.1\,\lambda \qquad \text{(Eq. 2-11)}$$

The factor W_R which represents the effect of well resistance can also be compared to the factor L of Yoshikuni (1974), which is:

$$L = \frac{32}{\pi^2}\,\frac{k_h}{k_w}\,\left(\frac{H_d}{d_w}\right)^2 = \frac{4}{\pi^2}\,W_R\,\left(\frac{H_d}{\ell}\right)^2 \qquad \text{(A-6)}$$

with H_d/ℓ equal to unity when the drains fully penetrate a clay layer with double drainage.

As the value measured for wick drains is not the coefficient of permeability, but rather the discharge capacity for a hydraulic gradient equal to unity, the following relation can also be used:

$$k_w = 4\,q_w/\pi d_w^2$$

$$W_R = 8(k_h/k_w)\times(\ell/d_w)^2 = 2\pi\,k_h\,\ell^2/q_w \qquad \text{(Eq. 2-12)}$$

For wick drains, the values to be used for the discharge capacity should not be the ones reported in the literature by Hansbo (1981), but the new ones which are ten to one hundred times larger as measured recently by Hansbo and by Professor Jamiolkowski of the Technical University of Torino (personal communication to Professor Ladd).

For Alidrains, the values become:

Lateral Stress (T/M^2)	Discharge Capacity (M^3/Yr)
10	600 - 1000
30	250 - 550

With these values one can see that most of the time, the effect of well resistance can be neglected for high quality wick drains. For example:

$$\left.\begin{aligned} k_h &= 10^{-7} cm/s \\ \ell &= 50 \text{ ft} = 1524 \text{ cm} \\ q_w &= 500 \text{ m}^3/yr = 15.9 \text{ cm}^3/s \end{aligned}\right\} \text{give } W_R = 0.092 < 0.1$$

For conventional sand drains having a diameter of 30 cm, the permeability of the sand must be greater than k_w equal to 0.02 cm/s based on the above conditions to have W_R less than 0.1. This value of k_w is reasonable for a clean concrete sand backfill.

When one cannot neglect the effect of well resistance, one can use Hansbo's (1981) solution which, though being questionable, should approximate the actual behavior. Hansbo's solution states that the term F(n) in Eq. 2-2 becomes:

$$\mu_r \simeq F(n) + \pi z(2\ell - z) k_h/q_w \qquad \text{(Eq. 2-13)}$$

or

$$\mu_r \simeq F(n) + Z(1 - Z/2) W_R \qquad \text{(Eq. 2-14)}$$

where: $Z = z/\ell$

As μ_r varies with depth, the degree of consolidation U_h will now also vary with depth. If a better approximation to account for the effect of well resistance is needed, one can develop a numerical solution using Eq. (A-1) and (A-2). Such a solution can also include the effect of vertical drainage.

To summarize, a new criteria to ascertain that the effect of well resistance can be neglected has been presented. Along with the new values of discharge capacities, one can see that most of the time, the effect is negligible for good quality wick drains and sand drains having a clean sand backfill.

2-6. EFFECT OF SMEAR

The effect of smear is the effect of disturbance due to the drain installation on the surrounding clay. In the zone of smear, the coefficient of permeability, k'_h, is lower than the one for the undisturbed zone, k_h, and therefore radial consolidation will be affected. To include this effect, both Barron (1948) and Hansbo (1981) added an annulus cylinder of smeared clay with an outside diameter d_s around the clay. This leads to a new boundary condition between the undisturbed zone and the smeared one in the analysis and affects the solution by changing the factor F(n) which becomes for both Barron and Hansbo:

$$F_s(n) = \ln(n/s) - 0.75 + (k_h/k'_h)\ \ln(s) \quad \text{(Eq. 2-15)}$$

where: $s = d_s/d_w$
$F_s(n) = F(n)$ including smear

in the case where $(d_s/d_e)^2$ is small compared to unity. The writer prefers to reformulate Eq. 2-15, such that:

$$F_s(n) = \ln(n) - 0.75 + [(k_h/k'_h) - 1]\times\ln(s)$$

or

$$F_s(n) = F(n) + [(k_h/k'_h) - 1]\times\ln(s) \quad \text{(Eq. 2-16)}$$

For non-displacement drains like jetted sand drains, the ratio s should be close to unity. For low displacement drains like wick drains, Hansbo (1981) used the ratio 1.5. From analysis of cone penetration in saturated clay by Baligh and Levadoux (1980), this ratio seems reasonable, corresponding to the limit of the "plastic" zone with a shear strain of 20 to 50%.

The ratio k_h/k'_h, is more difficult to estimate. It can be done by calculating the ratio of the coefficients of consolidation c_h, between an undisturbed and a disturbed sample. As shown in an example in Fig 5-3 of Jamiolkowski et al. (1983), the ratio c_h/c'_h varies from 3 to 5. Table 5.6 of the same report shows a range from 1.5 to 8 for the ratio k_v/k'_r, (k'_r being the remolded coefficient of permeability for three plastic clays compared at the same consolidation stress in the normally consolidated range).

Hansbo (1981) chose a ratio of 3 in the example he gave.

The values 1,3 and 10 for k_h/k'_h , will be used below to illustrate the effect of smear on F(n) with s = 1.1 and 1.5.

	S = 1.1			S = 1.5		
k_h/k'_h	1	3	10	1	3	10
$F_s(n)-F(n)$	0.0	0.19	0.86	0.0	0.81	3.65

These values have to be compared with the following values of F(n):

$$F(10) = 1.55; \qquad F(25) = 2.47$$

From the above-mentioned values of $F_s(n)$, one should realize that smear can influence the design. As stated by Jamiolkowski et al. (1983), a decrease in the drain spacing will not always be as effective as predicted by the theory for ideal drains since the thickness of the smear zone presumably remains constant. It should also be noticed that an increase in the drain diameter d_w will generate an increase in the overall influence of smear because the ratio $[F_s(n) - F(n)]/F(n)$ increases with n decreasing.

From all this, one should conclude that the effect of smear is important both for the design of drains and for the analysis of performance of drains. One should consider it, even though its effects are difficult to predict due to the uncertainties in the determination of the smear ratio, s, and in the ratio of the permeabilities, k_h/k'_h.

2-7. TIME DEPENDENT LOADING

Time dependent loading includes two types; a ramp loading and a multi-stage loading. The first one occurs since the full load cannot be instantaneously applied. The second takes place when the final height cannot be reached in one step without stability problems, and therefore intermediate stages are used to let the clay strengthen via consolidation before applying more loading.

a) Ramp Loading.

For the ideal drain case, the load is applied instantaneously and therefore the only required parameter is the origin of time. An approximate method proposed by Taylor (1948) for consolidation without drains is that one may consider a linear ramp loading equivalent to an instantaneous loading at a time t_o equal to the middle of the loading period. This method can be used in the case of consolidation with drains and is often used in practice.

If one wants to be more careful about this problem, one can use Olson's method (1977) which is derived from Terzaghi's and Frohlich's method (1936) which was developed for time varying loading on a clay layer without drains. From this method, one will obtain the degree of consolidation during the loading period, assuming that the coefficients of consolidation are constant. Unfortunately, clay layers are often originally over-consolidated, at least in the upper portion, and the final stress will load the clay into the normally consolidated range. Therefore, c_v and

c_h may decrease significantly during the ramp loading in such cases. Olson's method will consequently have to be used with caution. This method is included in the Finite Difference computer program originally described in Olson et al (1974) in which one can input varying c_v and c_h function of the effective stress of consolidation.

Schiffman (1959) proposed a method to account for the time-dependent loading and the varying permeability. This method helps solve the previous problem. Unfortunately, his paper is based on the assumption that the coefficient of permeability is linearly dependent on the excess pore pressure. The writer thinks that this assumption is questionable and that one should be careful when using Shiffman's method.

From these methods, one can see that the problem is still far from being solved. For design purposes, Taylor's method should be useful in most cases. One can also use the Finite Difference Solution that is currently available. For the analysis of drain performance, the methods proposed in the following chapter do not depend on the loading process and therefore the problem of the ramp loading disappears, except if one has to account for the effect of vertical drainage, in which case it will have a slight effect.

b) Two-Stage Loading.

To predict the settlement after the second loading, in the case of a two-stage loading, one usually adds the settlements predicted by each load, that is the one from the first load and the one from the full load minus the first load. One should be aware that this procedure is only exact when the coefficients of consoliation are constant during both stages.

This problem is not important when one wants to analyse the performance of the vertical drains with the methods present in the following chapter because the analysis of the second-stage can be considered independently of the one the first stage. For design purposes, one may also often disregard this problem, considering the uncertainties in the predictions of the different parameters used.

2-8. SUMMARY AND CONCLUSIONS

In this chapter, the theory of consolidation with vertical drains and the effects of vertical drainage, of well resistance, and of smear were presented followed by a brief discussion of the problem of the time dependent loading.

From this, the following major conclusions can be drawn.

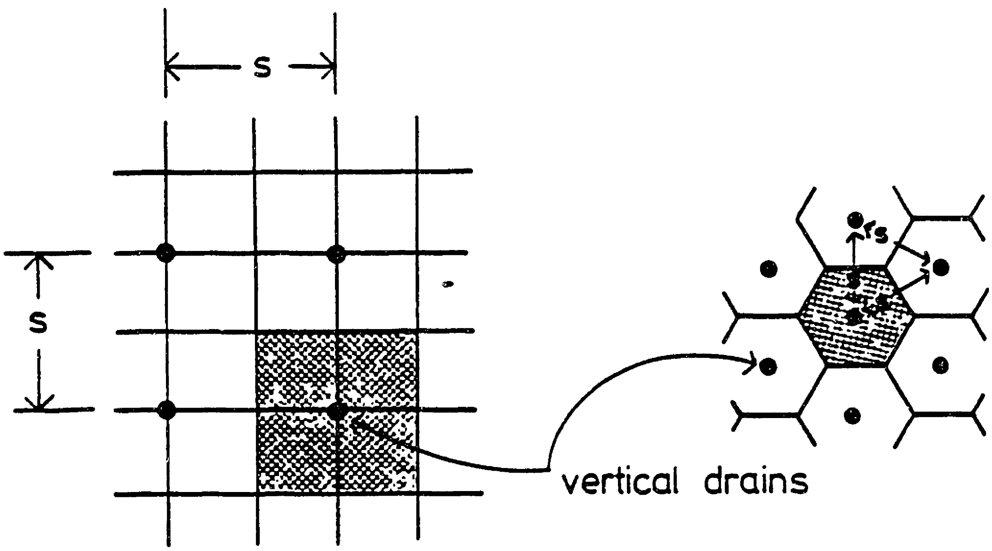
1) The influence of vertical drainage can vary from being insignificant regarding rates of pore pressure dissipation for piezometers located within the central portion of thick clay layers to having a very significant effect on rates of surface settlement when T_v/T_h is greater than almost 0.1. Figures 2-2 through 2-5 can be used to assess the importance of vertical drainage when analysing

piezometer data and Fig. 2-6 when analysing settlement data.

2) In most cases, well resistance should have no effect on the consolidation process with good quality wick drains and sand drains.

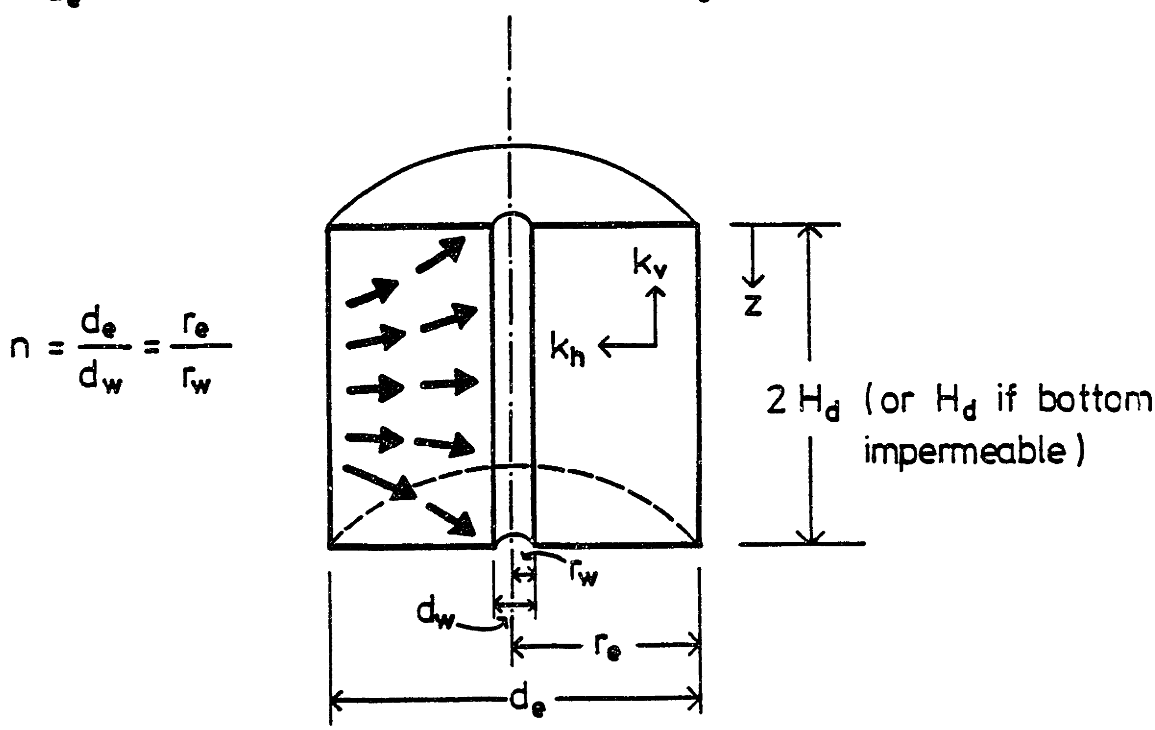
3) Though the effect of smear may be extremely important, its effect is difficult to predict due to the uncertainties in the thickness and properties of the remolded zone which occurs around the perimeter of wick drains and displacement sand drains. The adverse effects increase as the drain spacing decreases.

Fig. 2-1: Drain Patterns - Equivalent Cylinder.



SQUARE PATTERN
$d_e = 1.128\ s$

TRIANGULAR PATTERN
$d_e = 1.050\ s$



SECTION of the EQUIVALENT CYLINDER

Fig. 2-2: Effect of the Vertical Drainage on the Degree of Consolidation versus Depth for Different Time Factors and n=10.

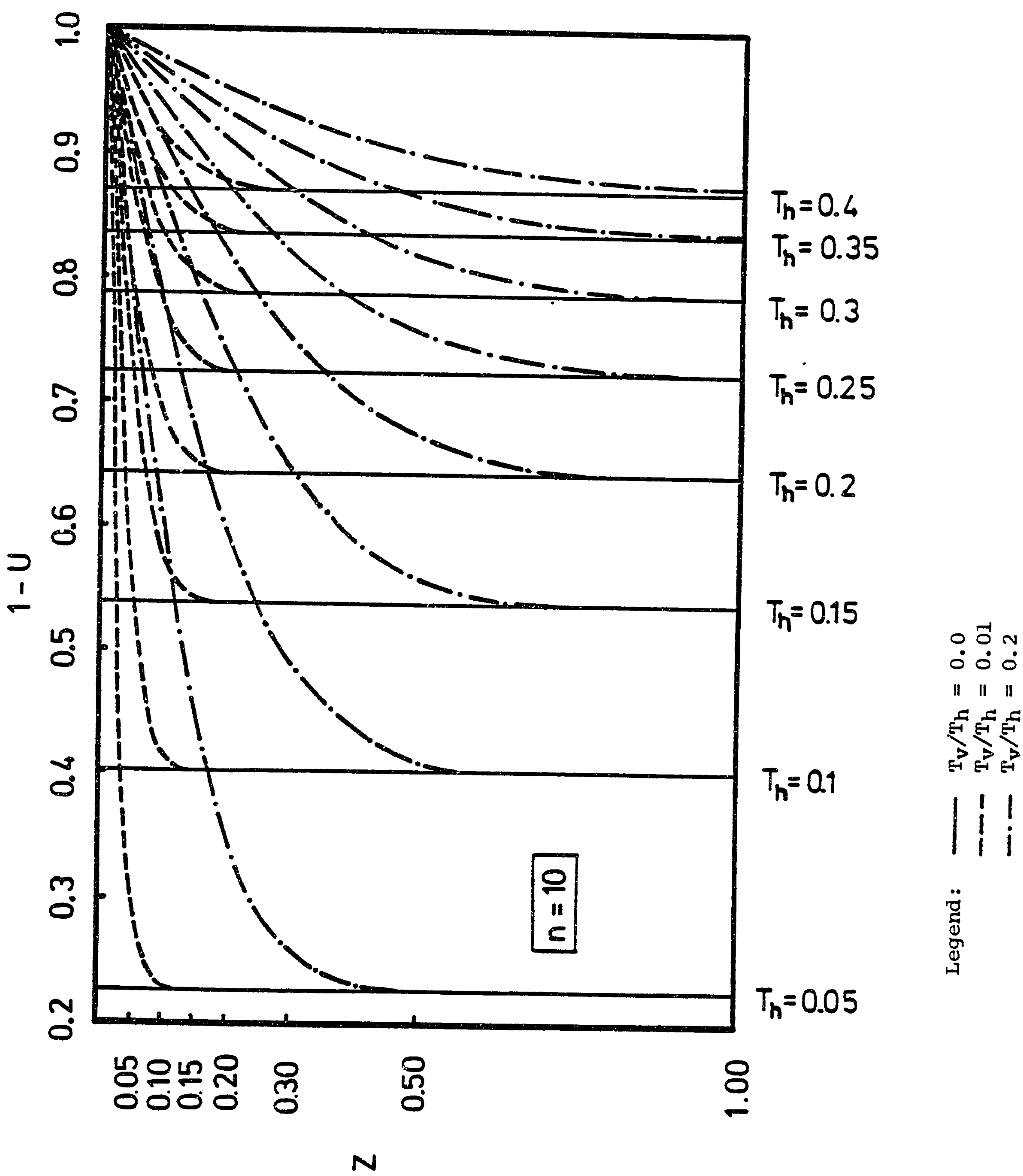


Fig. 2-3: Effect of the Vertical Drainage on the Degree of Consolidation versus Depth for Different Time Factors and n= 25.

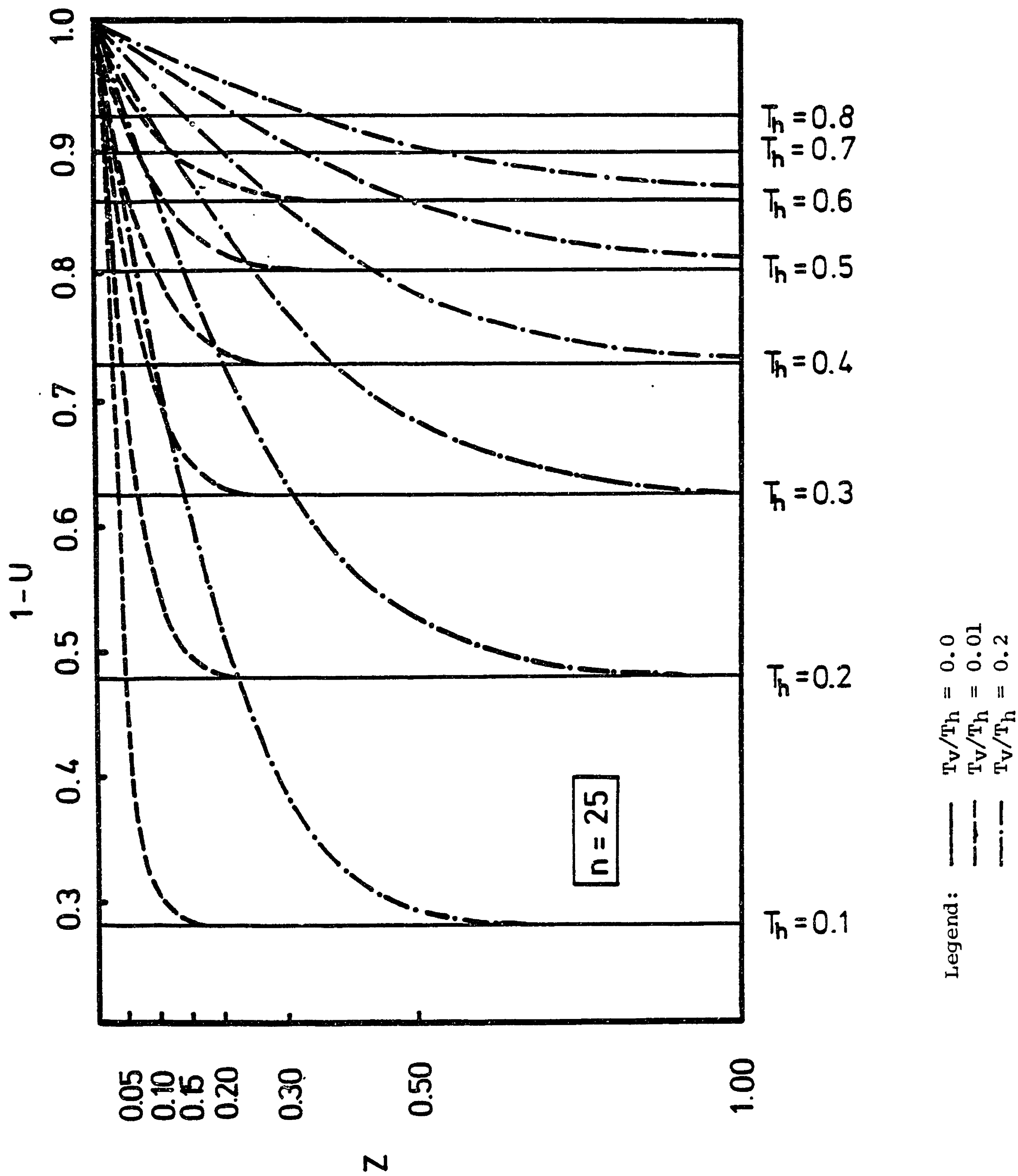


Fig. 2-4: Log (1-U) versus Time for Different Time Factors Ratios T_v/T_h and Depth Ratios Z; Spacing Ratio n= 10.

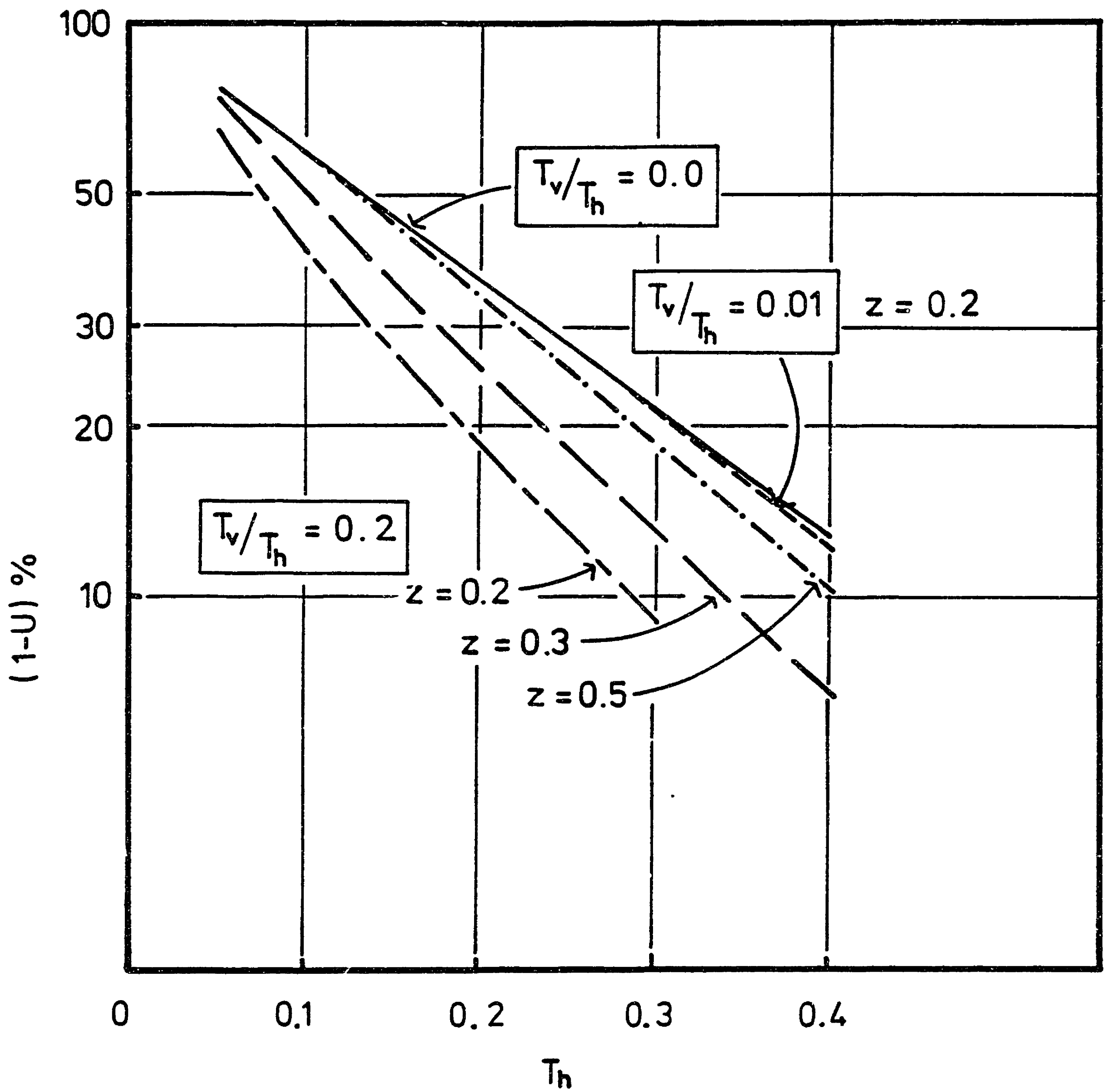


Fig. 2-5: Log (1-U) versus Time for Different Time Factors Ratios T_v/T_h and Depth Ratios Z; Spacing Ratio n=25.

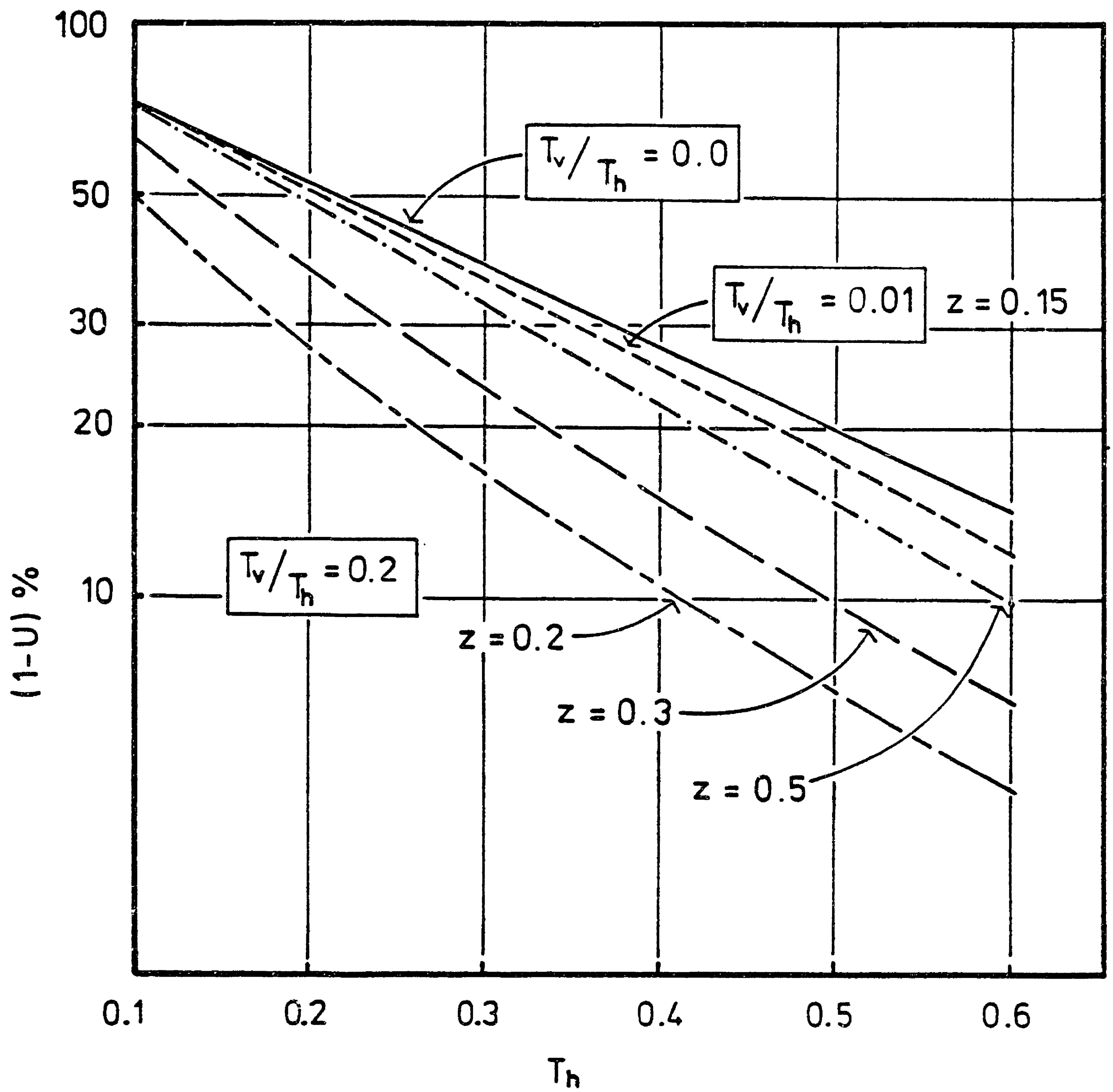
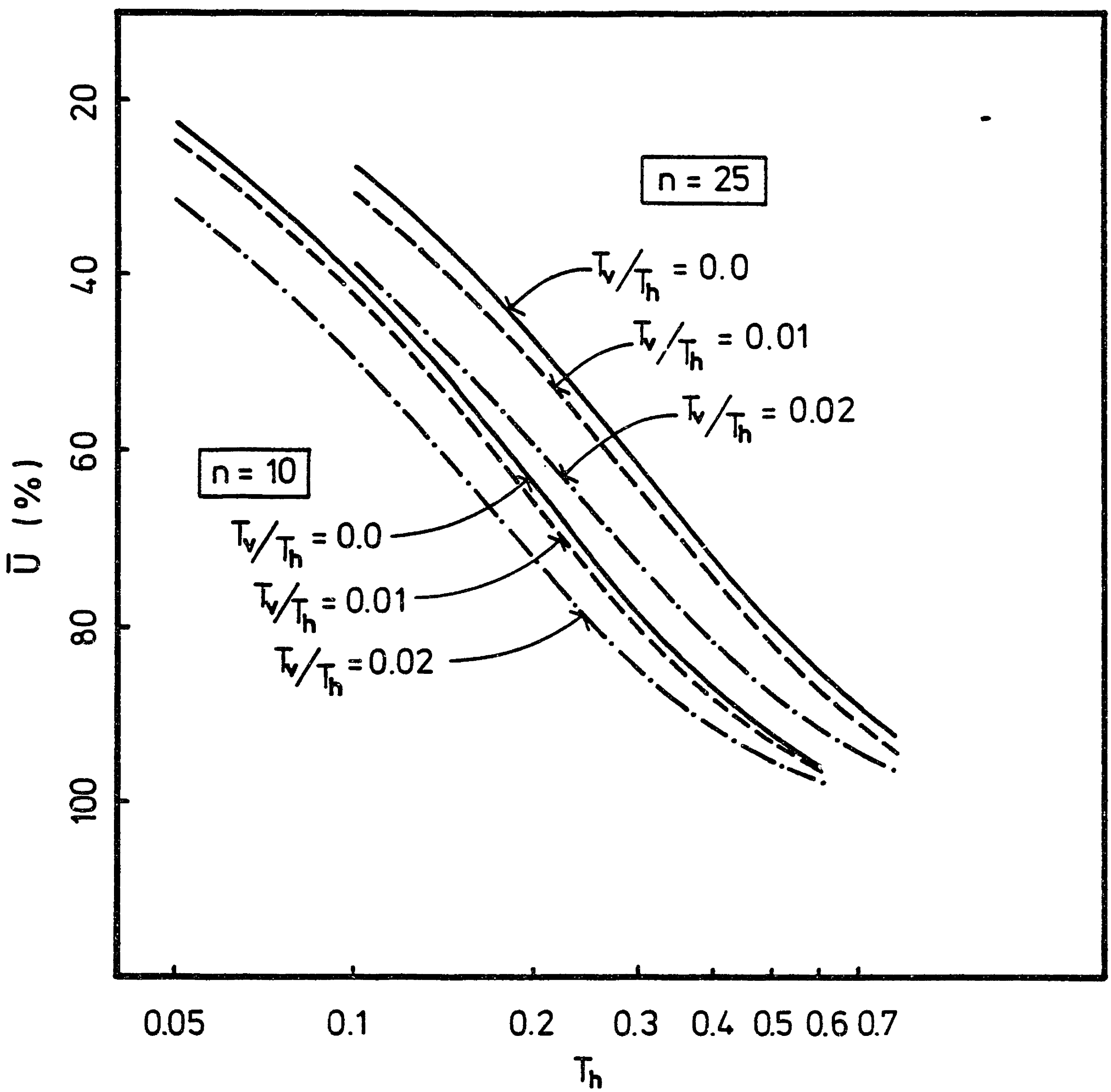


Fig. 2-6: Average Degree of Consolidation versus Log Time for Different Time Factors Ratios T_v/T_h for n= 10 and n= 25.



CHAPTER 3

TECHNIQUES TO ANALYSE THE PERFORMANCE OF VERTICAL DRAINS

3-1. INTRODUCTION

Essentially all projects using vertical drains to accelerate consolidation during precompression have field instrumentations to measure the settlement of the clay layer and to measure excess pore pressures within the layer during the consolidation process. One can also use instruments to monitor lateral displacements. Techniques are required to convert these data into a quantitative assessment of how consolidation is progressing with time for two purposes: First, to compare the actual performance with the predicted one to determine if construction needs to be altered and second, to advance the state of knowledge on soil behavior and on the efficiency of the various types of vertical drains.

A brief review is first made of "conventional" methods of analysis and their problems. Two new methods for pure radial drainage will then be presented; the first one for settlement data via Asaoka's (1978) method and a second one to evaluate excess pore pressure data. The application of these methods to consolidation for vertical drainage only will also be discussed. Lastly, a procedure to analyse combined drainage from settlement data will be considered.

3-2. CONVENTIONAL METHODS OF ANALYSIS AND THEIR PROBLEMS

The analysis, as presented by Ladd (1976), is performed using settlement and/or excess pore pressure data. To start, one needs to predict or measure the initial settlement, ρ_i, the inital excess pore pressure, $\bar{u}_o$, and the final settlement due to primary consolidation, ρ_{cf}. Once an initial time, t_o, has been determined, one also needs the measured values of the settlement at time t, $\rho(t)$ and of the excess pore pressure at time t, u(t).

For settlement data, the average degree of consolidation is first computed for a given soil layer:

$$\bar{U}(t) = (\rho(t) - \rho_i)/\rho_{cf}$$

The time factor T_h is then obtained from Eq. 2-2 or from Johnson's (1970b) chart for radial drainage only. If vertical drainage is also significant, then the time factor T_h is obtained from a plot of $\bar{U}$ versus T_h that one can compute from theory by assuming a value for the ratio c_v/c_h.

From the time factor, one can get the coefficient of consolidation using:

* either total time:

  $c_h = T_h (d_e)^2/t$

* or incremental time (with two values):

  $c_h = [(T_{h_2} - T_{h_1}) (d_e)^2]/(t_2 - t_1)$

With pore pressure data, the same basic procedure is used, except that the degree of consolidation is replaced in the relevant equations or plots by the ratio $u/\bar{u}_o$ (see Fig. 3-1). The problems related to this method can be found either in Ladd (1976) or in Jamiolkowski et al. (1983). In summary, they are:

1) The determination of the settlements due to primary consolidation, which are the measured values of settlement minus the initial settlement, may not be accurate. The initial settlement results from undrained shear deformations. The amount of initial settlement is usually either inferred from field measurements of lateral deformations or estimated via procedures such as presented in D'Appolonia et al. (1971) or sometimes from finite element analyses. In any case, its exact magnitude will generally be uncertain.

2) The measured value of the total settlement at the end of primary consolidation is usually not known when one analyses the data. Therefore, one has to estimate the final settlement due to primary consolidation based on laboratory tests. The accuracy of this prediction may be uncertain.

3) The prediction of the initial excess pore pressure may be inaccurate, leading to an additional uncertainty when using of the total time approach.

4) The location of the tip of the piezometer may not be at the exact center of the drain pattern, especially for piezometers located deep in the layer, thus generating an error in the results.

5) If the clay layer, or part of it, is overconsolidated, the consolidation process will be fast, due to the high value of c_h for the overconsolidated zone of the clay until the maximum past pressure is reached. Once beyond this value, c_h is reduced to its normally consolidated value. Therefore, Jamiolkowski et al. (1983) recommend using the incremental method only in the normally consolidated range for the analysis.

6) Regarding the effect of vertical drainage, it should also be noted that the methods require a trial and error procedure to obtain a ratio c_v/c_h in agreement with the one assumed to construct the graphs $\bar{U}$ vs T_h and $u/\bar{u}_o$ vs T_h.

3-3. PROPOSED METHOD FOR ANALYSIS OF SETTLEMENT DATA

The method applied to only radial drainage will first be presented, then the one for vertical drainage will be considered. Finally, the method applied to combined drainage will be treated. The methods and their problems are discussed. The proofs of these methods are presented in Appendix B.

a) Method applied to radial drainage only.

This method, as presented by Asaoka (1978), is based on Barron's (1948) equation for pure radial drainage (Eq. 2-2) and its relevant assumptions. From the natural time-settlement curve, one can select a series of settlement values (ρ_1, ρ_2,..., ρ_n,...) such that ρ_n is the settlement at time t_n and that $(t_n - t_{n-1})$ is constant and equal to Δt (see Fig. 3-2. Note: The plot log of settlement versus time is not used because it yields a straight line only if the initial settlement is zero). From this series, one can generate Asaoka's construction by plotting the points (ρ_{n-1}, ρ_n) with the settlement values at time t_{n-1} on the x-axis and the ones at time t_n on the y-axis (see Fig. 3-2b). As shown in Appendix B, all the points lie on a straight line, such that:

$$\rho_n = \rho_o + \beta_1 \rho_{n-1} \qquad \text{(Eq. 3-1)}$$

where ρ_o and β_1 are two constants which depend on the selected time interval. (see Fig.3-3a). This straight line intersects the 45°-degree line at the point:

$$\rho_n = \rho_{n-1} = \rho_\infty$$

This is the stable point or convergence point. Physically, this point corresponds to the total settlement at the end of primary consolidation. This point can also be defined by the following relation:

$$\rho_\infty = \rho_o / (1 - \beta_1) \qquad \text{(Eq. 3-2)}$$

The constant β_1 of Eq. 3-1, which represents the slope of the constructed straight line, can be related to the coefficient of consolidation by the following relation:

$$c_h = - [(de)^2 F(n)/8] x [\ln(\beta_1)/ \Delta t] \qquad \text{(Eq. 3-3)}$$

The theory shows that, whatever the time interval chosen, the values of c_h and ρ_∞ are the same. It should also be noted that both values are independent of the time origin.

The above development was done for the case of a single-stage loading. For the case of a two-stages loading, two straight lines should appear, each one having the properties previously presented for the single line (see

Fig. 3-3b). Both lines should be parallel if the coefficients of consolidation are the same for both stages.

b) Method applied to vertical drainage only.

For consolidation with vertical drainage only the same method can be used, except that the relation between the slope β_1 and the coefficient of consolidation c_v becomes (Magnan and Deroy, 1980):

$$c_v = -[4H_d^2/\pi^2]x[\ln(\beta_1)/\Delta t] \qquad \text{(Eq. 3-4)}$$

This relation is only valid for T_v higher that 0.1. As shown in Appendix B, Asaoka (1978) presented a different formulation for the relation between β_1 and c_v. His approach used an approximate relation between the degree of consolidation $\bar{U}_v$ and the time factor T_v which is not in agreement with Terzaghi's solution, and therefore leads to erroneous values of c_v.

The value of the coefficient of consolidation obtained from Eq. 3-4 should be accepted only after having checked that the assumption of T_v higher than 0.1 is valid for the first points of the settlement series used to define the straight line.

c) Method applied to combined radial and vertical drainage.

The method used in this case is the same as the one for radial drainage only, but the coefficient of consolidation c_h obtained includes the effect of vertical drainage and therefore has to be corrected to obtain the correct c_h.

An approximate relation to correct for this effect is now presented. The theoretical development of the first correction method can be found in Appendix B. This procedure is based on the reasonable assumption that the total settlement at the end of primary consolidation, determined as previously presented, is not influenced by vertical drainage. The method also assumes that the coefficient of consolidation for vertical drainage c_v is known.

Fig. 3-4 presents an example of the effect of vertical drainage on Asaoka's construction. The data used are those from the example calculation in Table 3-1. Curve 1 represents the straight line from vertical drain theory if only radial drainage had occured. Curve 2 is the line which results from Asaoka's construction using Δt=50 days and the settlement data computed with combined radial and vertical drainage. At high degrees of consolidation ($t \geq 150$ days, $\bar{U}_{hv} \geq 0.78$), this becomes more or less linear. Curve 3 is the straight line that one obtains from the linear portion of curve 2. The slope of this straight line can be used with Eq. 3-3 to determine the value $c_{h_{hv}}$, which represents the coefficient of consolidation c_h affected by the vertical drainage. As the slope of line 3 is lower that the one from line 1, the coefficient $c_{h_{hv}}$ is higher than the actual value of c_h. Therefore, one needs a relation to assess the magnitude of the error.

The difference Δc_h between the value of $c_{h_{hv}}$ and the actual c_h can be approximated by the following relation:

$$\Delta c_h = c_{h_{hv}} - c_h = [(de)^2 F(n)/8] \times [\ln(A)/\Delta t] \quad \text{(Eq. 3-5)}$$

where:

* $A = 1 - (\alpha/2)(\Delta t/\sqrt{t_m})(1/1-\alpha\sqrt{t_m})$ (Eq. 3-6)
* $\alpha = \sqrt{4c_v/\pi H_d^2}$ (Eq. 3-7)
* t_m = mean time t_{n-1} of the points used to define the straight line
* Δt = time interval such as:

$$\Delta t < 0.1 \times t_m \quad \text{(Eq. 3-8)}$$

The value of c_v which is required in Eq. 3-7, is obtained from laboratory tests. Table 3-1 shows the validity of Eq. 3-5 for three cases. The values of $c_{h_{hv}}$ were calculated instead of determined from Asaoka's construction to avoid uncertainty in the result due to the plotting procedure. The method of calculation is the following:

1. Determine β_1 for each couple $(1 - \bar{U}_{n-1}, 1-\bar{U}_n)$ corresponding to all the times t_{n-1} chosen using Eq. B-2.

$$\beta_{1h} = (1-\bar{U}_n)/(1-\bar{U}_{n-1}) = (1-\rho_n)/(1-\rho_{n-1})$$

where $\bar{U}$ equals the values of $\bar{U}_{n-1}$ listed in Table 3-1

2. Calculate the average value β_1 of the previous values.

3. Determine $c_{h_{hv}}$ from β_1 using Eq. 3-2.

One can notice that even though the restriction imposed by Eq. 3-8 was not respected in cases 2 and 3, the correction term was correct. This shows that Eq. 3-8 may be too restrictive. The practical aspect of this point will be further discussed in Chapter 5.

d) Discussion of the methods proposed.

The method to analyse the performance of vertical drains from settlement data with only pure radial drainage allows one to determine c_h and ρ_∞ in a simple and easy way. The advantage of this method relative to the conventional one is that one does not need to predict the values of the initial settlement and of the final settlement due to primary consolidation. The problem encountered with the fact that the value of c_h may be higher during the early stages of consolidation still remains. This point will be further discussed in Chapter 5 along with practical recommendations regarding the use of the method.

The procedure to analyse the consolidation process for vertical drainage only, being basically the same as the one above, yields the same advantages. It is simple of use and there is no need for predictions of the initial settlement and of the final settlement at the end of primary consolidation. Nevertheless, one should not forget that this method is not valid in the early stages of the consolidation process due to the restriction on the time factor. The writer used this method to analyse results from an oedometer test. It compared well with Taylor's and Casagrande's methods, but more of such analysis will be required before one can assess the value of c_v from this new method relatively to the ones from the above mentioned conventional methods (see Appendix E).

An analysis of vertical drains in the case of combined radial and vertical drains cannot be made by combining the methods previously described because during most of the consolidation process, the vertical drainage process remains in the stage where Terzaghi's approximate solution (Eq. 2-6) cannot be used. The method proposed is therefore to use the one for pure radial drainage and thereafter to correct for the effect of vertical drainage.

In general, if one wants to check the validity of the analysis, one can back-calculate the original time-settlement curve used by the analysis; this can be called "back-analysis". One can use the following relation to calculate the settlement:

$$\rho(t) = \rho_i + (\rho_\infty - \rho_i)\bar{U}(t) \qquad \text{(Eq. 3-9)}$$

The value of ρ_∞ is known. The degree of consolidation can be calculated with the value of c_h obtained and the other parameters used in the analysis. The only unknown is the initial settlement which can be back-calculated from a point on the original time-settlement curve. This method of assessing the analysis will be further treated in Chapter 5.

3-4. PROPOSED METHODS OF ANALYSIS OF PORE PRESSURE DATA

The method applied to only radial drainage will be first presented. Next, the one for vertical drainage will be considered. The methods will then be discussed. Appendix C contains the pertinent derivations.

a) Method applied to radial drainage only.

From Barron's (1948) Equal strain condition theory for pure radial drainage, the following relation is valid:

$$\ln(u) = \alpha_o - \alpha_1 t \qquad \text{(Eq. 3-10)}$$

where: u = excess pore pressure at time t for any point. The coefficients α_o and α_1 can be obtained by a linear regression between ln(u) and t.

The coefficient of consolidation, c_h, is obtained from α_1 with the following relation:

$$c_h = [(d_e)^2 F(n)/8] \times \alpha_1 \qquad \text{(Eq. 3-11)}$$

where $\alpha_1 = [\ln(u_1/u_2)/(t_2 - t_1)]$

The constant α_o is related to the product of the initial excess pore pressure, $\bar{u}_o$, at the tip of the piezometer, and the location factor, ν, of this point by the relation:

$$\nu \bar{u}_o = e^{\alpha_o} \qquad \text{(Eq. 3-12)}$$

If the tip of the piezometer is located at the center of the drain pattern, the location factor is equal to:

$$\nu_c = [\ln(n) - 0.5]/[\ln(n) - 0.75] \qquad \text{(Eq. 3-13)}$$

Another method of determination of c_h can be derived by an .ogy to Asaoka's method for settlement data. This method is presented in Appendix C. Nevertheless, the previous method will be used because it yields a simple linear relation between excess pore pressure values and time and because it allows one to obtain the value of $\nu \bar{u}_o$.

For the case of a two-stage loading, two straight lines should appear. Each one represents one of the two stages and allows the determination of the coefficient of consolidation.

b) Method applied to vertical drainage only.

Using Terzaghi's approximate solution (Eq. 2-6), one can derive a similar method to the one for pure radial drainage. The only difference is on the meaning of the constants α_o and α_1. The expression of the coefficient of consolidation, c_v, is the following:

$$c_v = (4H_d^2/\pi^2)\,\alpha_1 \qquad \text{(Eq. 3-14)}$$

where: $\alpha_1 = [\ln(u_1/u_2)/(t_2-t_1)]$

This expression is only valid for a time factor T_v higher than 0.1, due to the restriction on Eq. 2-6 . The constant α_o is now related to the initial excess pore, $\bar{u}_o$, at the tip of the piezometer and to the depth factor, Z, at this point, by the relation:

$$\bar{u}_o \sin(\pi Z/2) = (4/\pi)\, e^{\alpha_o} \qquad \text{(Eq. 3-15)}$$

c) Effect of combined radial and vertical drainage.

In most of the actual cases of consolidation with drains, the drainage pattern combines radial and vertical draiange. As already seen in section 2-4, in some cases depending on various parameters such as the depth factor, the ratio T_v/T_h..., one can neglect the effect of vertical drainage for the analysis of excess pore pressure data. In the other cases, as seen in Figs. 2-4 and 2-5, after the early stages of consolidation, the curve log (u) versus time is close to a straight line for piezometers not located too close to a permeable boundary. In such a case, the "conventional" method presented in section 3-2 should be used.

d) Discussion.

The method proposed for radial drainage only is very simple to use. The value of the coefficient of consolidation c_h does not depend either on the value of the initial excess pore pressure, or on the location of the tip of the piezometer relative to the drain pattern. The value of c_h obtained is also independent of the origin of time. On the other hand, one should not rely too much on the value of the initial excess pore pressure computed via Eq. 3-12 because it depends on the determination of the time origin and on the actual value of the location factor. If the value of c_h is higher during the early stage of the consolidation process, the relation between log u and time will not be linear anymore. The method proposed will no longer be valid and thus one should use the conventional method in such a case.

The same comments apply to the method of analysis for vertical drainage except for the added restriction on the time factor.

In the case of combined radial and vertical drainage, it was noted that one often will not be required to correct for the effect of vertical drainage. If one cannot neglect this effect, one should use the "conventional" method which was presented in section 3-2.

A "back-analysis" can be performed using the parameters obtained from the excess pore pressure analysis. The curve of excess pore pressure versus time can be reconstructed using Eq. C-2 and then compared to the original data providing an assessment of tne values c_h and $\nu\bar{u}_o$.

3-5. SUMMARY AND CONCLUSIONS

The conventional methods of analysis of performance of vertical drains were first presented and their problems briefly summarized. Next, new methods of analysis of settlement data for both radial and vertical drainage were developed. Their advantage is that they do not require predictions of the initial settlement and of the final settlement due to primary consolidation. The effect of combined drainage was then discussed and a method of correction for the effect of vertical drainage was presented in order to obtain the actual value of c_h.

New methods of analysis of excess pore pressure data for both pure radial and vertical drainage were also developed. They do not require prediction of the initial excess pore pressure. For radial drainage only, it is seen that the computed coefficient of consolidation c_h should be independent of the location of the tip of the piezometer. In the case of combined drainage, if one cannot neglect the effect of vertical drainage, one should use the conventional method.

TABLE 3-1

Correction of c_h from Settlement Data for Vertical Drainage Effect:

Assumptions:

* $T_v/T_h = 0.2$; $F(n) = 2.0$
* $c_h = 0.10$ ft²/day; $d_e^2 = 50$ ft
* $\rho_\infty = 1.0$ ft; $\rho_i = 0.0$
* $c_v/H_d^2 = (T_v/T_h) \times c_h/(d_e^2) = 0.2 \times (0.10/50) = 0.0004$

Data:

t (days)	T_h	$\bar{U}_h$	$\bar{U}_{hv} = \rho_{hv}$	$1-\bar{U}_{hv} = 1-\rho_{hv}$
0	0	0	0	1.0
50	0.1	0.330	0.437	0.5634
100	0.2	0.551	0.652	0.3479
130	0.26	0.647	0.7375	0.2625
140	0.28	0.674	0.761	0.2392
150	0.30	0.699	0.780	0.2179
160	0.32	0.722	0.801	0.1987
170	0.34	0.743	0.818	0.1811
180	0.36	0.763	0.835	0.1652
190	0.38	0.781	0.849	0.1507
200	0.40	0.798	0.8625	0.1375
250	0.50	0.865	0.913	0.0870
300	0.60	0.909	0.945	0.0553
350	0.70	0.939	0.965	0.035

Correction of c_h for vertical drainage:

Case I : Data used: from t = 130 to t = 200; $\Delta t = 10$; $t_m = 160$

Correction: A = 0.9875, $\Delta c_h = 0.0157$, $c_{hv} = 0.1155$; $c_h = 0.0998$

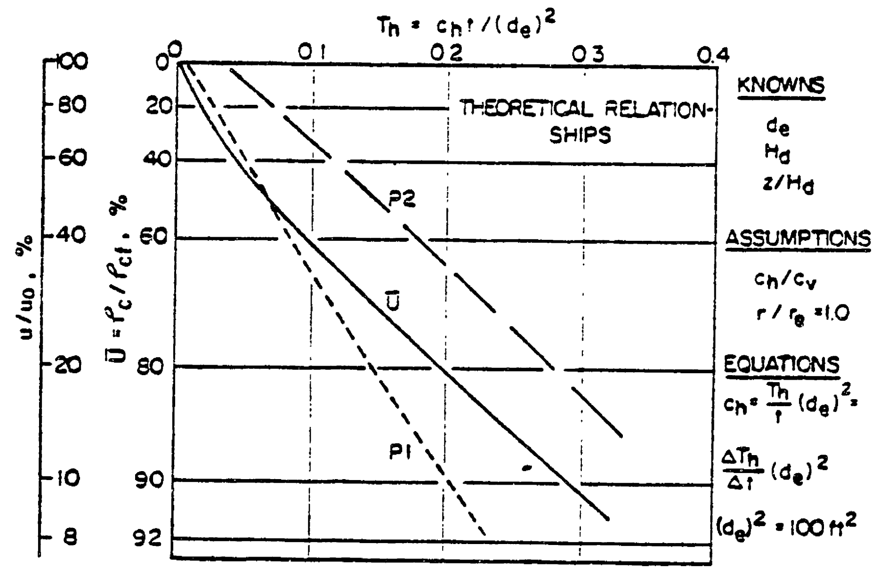
Case II : Data used: from t= 50 to t = 350, $\Delta t = 50$; $t_m = 175$

Correction: A = 0.9392, $\Delta c_h = 0.0157$, $c_{hv} = 0.1157$; $c_h = 0.100$

Case III: Data used: from t = 50 to t= 250; $\Delta t = 50$; $t_m = 125$

Correction: A = 0.9325, $\Delta c_h = 0.0175$, $c_{hv} = 0.1168$; $c_h = 0.0993$

Fig. 3-1: Computation of c_h from Field Settlement and Pore Pressure Data using the "Conventional" Methods. (from Ladd, 1975)

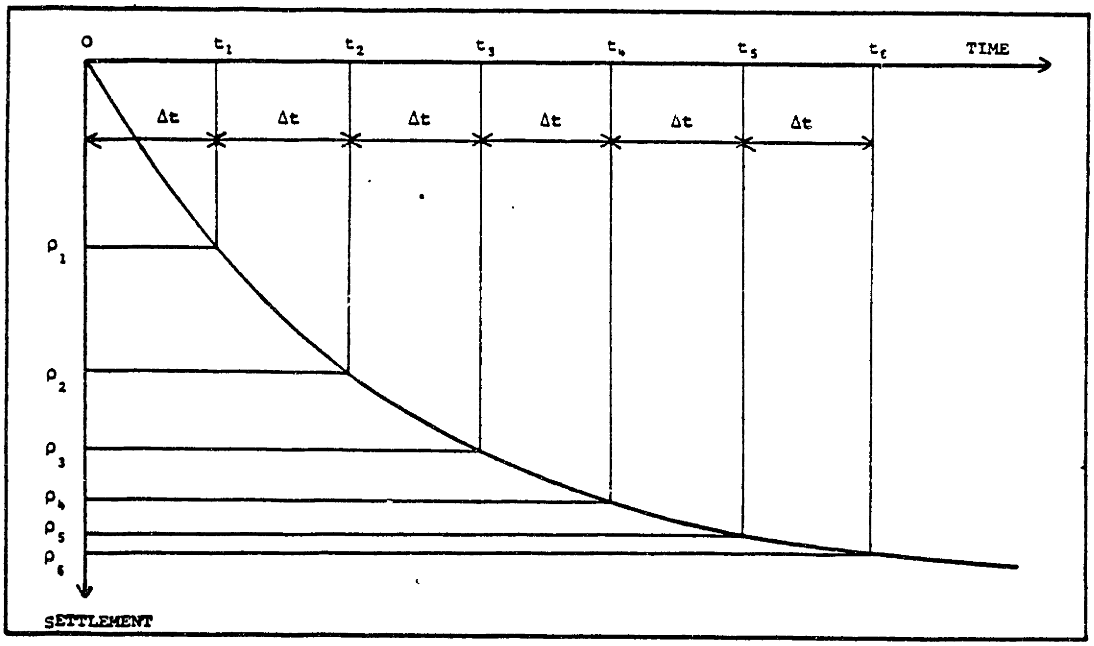


	t Day	Ū %	T_h	Δt	ΔT_h	T_h/t	$\Delta T_h/\Delta t$	c_h, ft²/day: t	c_h, ft²/day: Δt
ρ_c Data	50	39	.049			.00098		.098	
	100	59	.100	50	.051	.00100	.00102	.100	.102
	200	81	.203	100	.103	.00101	.00103	.101	.103

PIEZ.	t Day	u/u_o %	T_h	Δt	ΔT_h	T_h/t	$\Delta T_h/\Delta t$	c_h, ft²/day: t	c_h, ft²/day: Δt
P2	30	96	.035			.00117		.117	
	80	71	.075	50	.040	.00094	.000801	.094	.080
	200	34	.205	120	.130	.00102	.00108	.102	.108

Fig. 3-2: Plot of Settlement Data versus Time and Illustration of Asaoka's(1978) Construction.

a) Settlement vs. Time:



b) Asaoka's construction:

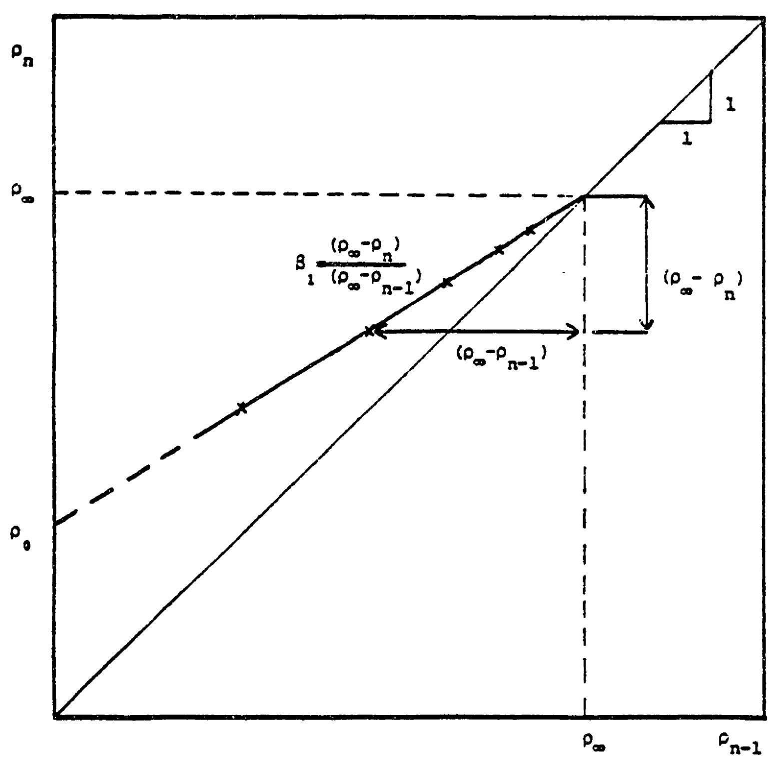
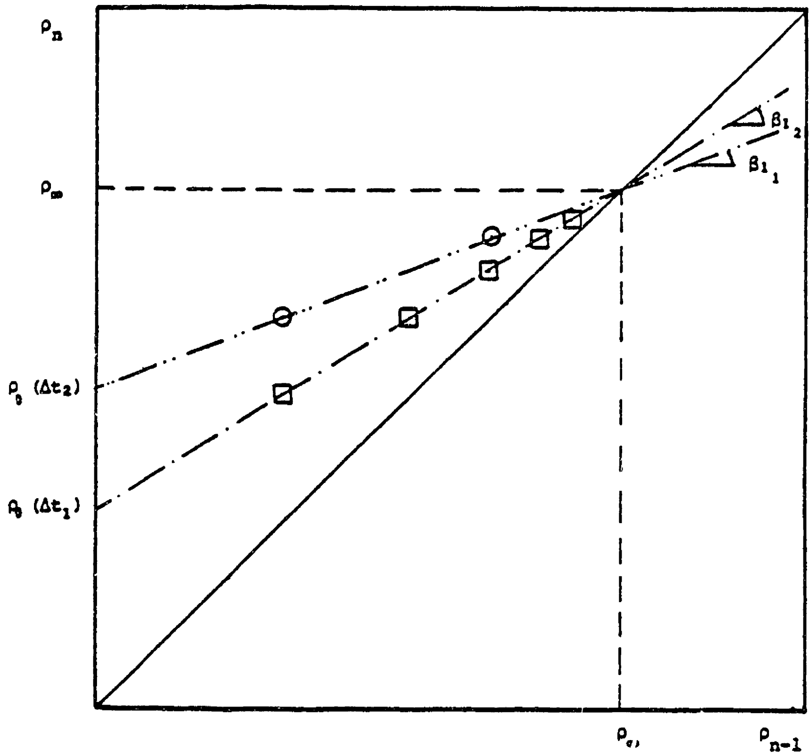


Fig. 3-3: Asaoka' Construction to Obtain c_h from Settlement Data.

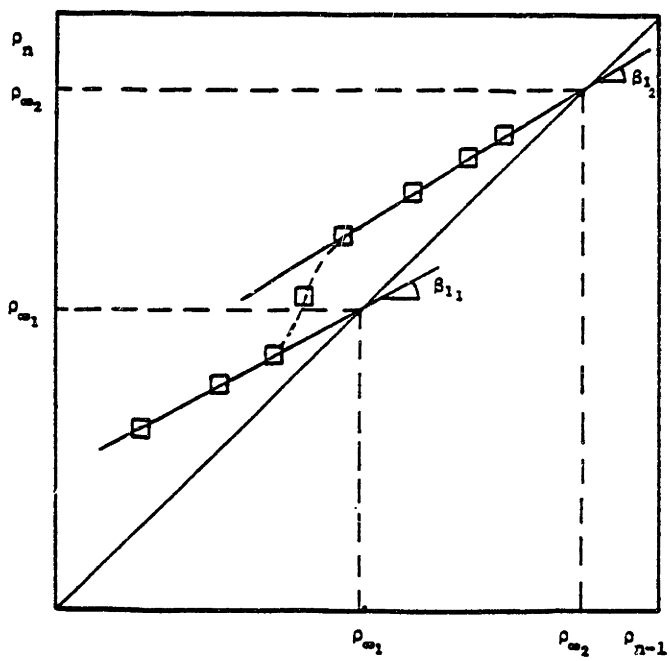
a) One-Stage Loading:



$\Delta t_2 > \Delta t_1$:

$$\frac{\ln(\beta_{1_1})}{\Delta t_1} = \frac{\ln(\beta_{1_2})}{\Delta t_2} = \frac{-8\, c_h}{d_e^2\, F(n)}$$

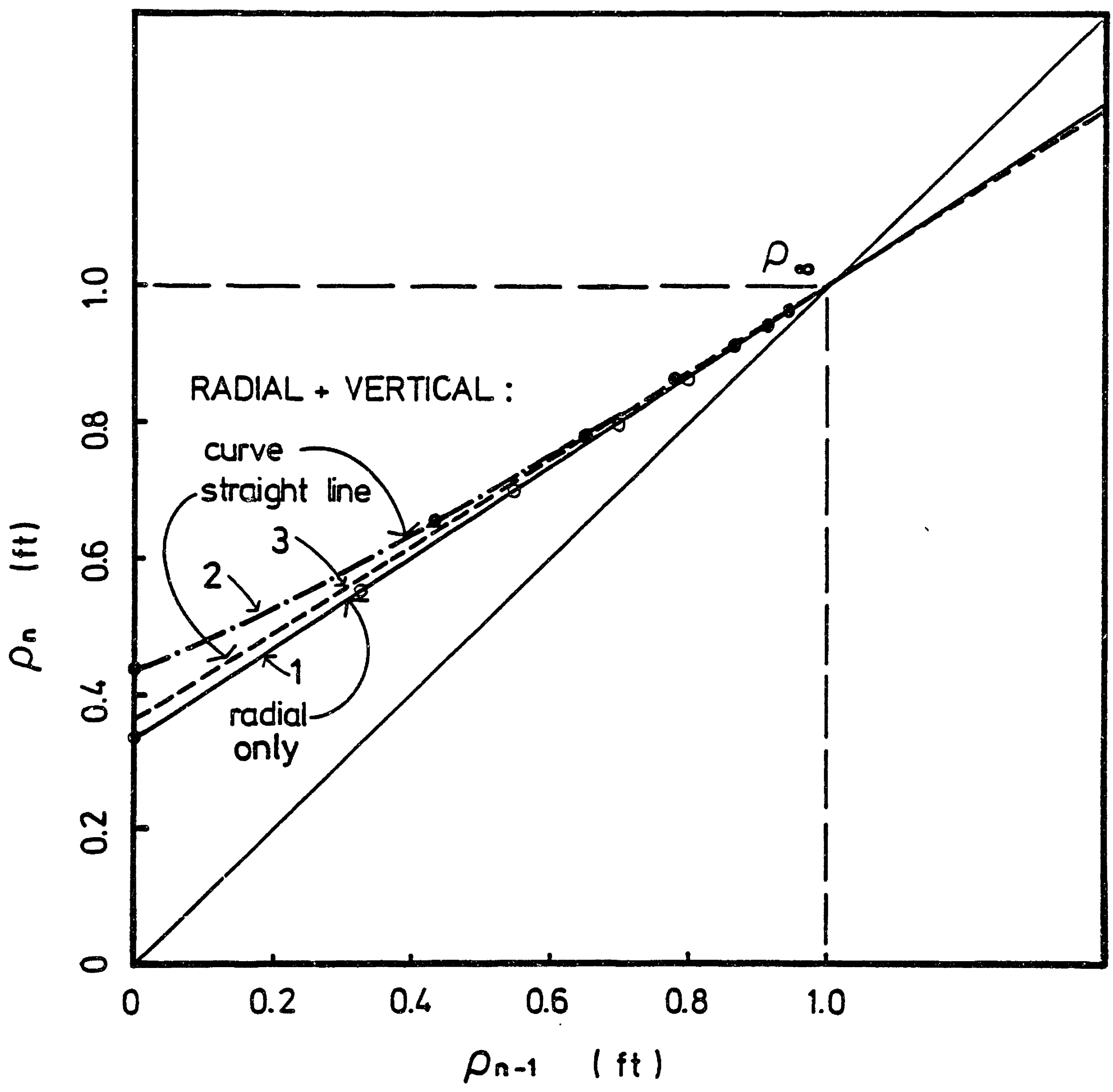
b) Two-Stage Loading:



IF $c_{h_1} = c_{h_2}$,

$\beta_{1_1} = \beta_{1_2}$.

Fig. 3-4: Effect of Vertical Drainage on Asaoka's Construction.



Legend:

Line 1: ——— : Radial drainage only

Line 2: —·— : Combined drainage with curved portion

Line 3: — — — : Straight line portion of combined drainage

Δt = 50 days from t = 350 days.
See table 3-1 for data and assumptions.

CHAPTER 4

CASE HISTORY OF PERFORMANCE OF ALIDRAINS

4-1. BACKGROUND

Ideal Basic Industries, Inc. built a cement manufacturing plant in the Theodore Industrial Park south of Mobile, Alabama. As the site is underlain by thick deposits of a medium to soft, plastic clay, concrete piles were chosen to support the major facilities. The only exception is the Reserve Limestone Storage Area (RLSA) where precompression was used to strengthen the foundation soil, instead of a pile supported mat foundation, in order to save over five million dollars. To accelerate consolidation, Alidrains were installed under the area. Laure Noiray studied the RLSA in her Master's thesis (1982). To analyse the performance of the Alidrains, the present case history uses data from Noiray's thesis in addition to data obtained subsequently. The results of this analysis will be then be discussed and compared to Noiray's results.

Figure 4-1 presents a plan view of the RLSA site prior to loading, while Fig. 4-2 presents a North-South cross-section at the centerline and Fig. 4-3 presents a West-East Middle cross-section. The limestone fill was placed between May and October 1981 up to a maximum height of about 30 ft. As seen in Fig. 4-1, the RLSA was well instrumented, especially at the North, Middle and South sections where piezometers and Sondex extensometers were placed at the centerline. Piezometers were also located at 65 ft East from the centerline at each section and at the Centerline between the North-Middle and Middle-South sections. Horizontal and vertical inclinometers were placed at the three sections to measure surface settlement profiles and the horizontal displacements versus depth.

Figures 4-4 and 4-5 summarize the basic soil conditions. Figure 4-4 shows a simplified soil profile which basically consists of a clay layer (between El.+6ft and El.-58 ft), overlaid by a thin sandy layer, and underlaid by a dense sand layer. This figure also presents the stress history profile. Shown are values of: the initial effective overburden stress, $\bar{\sigma}_{v_o}$: the computed vertical effective stress at the end of an undrained loading, $\bar{\sigma}_{v_u}$; the mean maximum past pressure from linear regression analysis of the laboratory test data; and the final vertical effective stress, $\bar{\sigma}_{v_f}$, for fill heights of 25, 30 and 35 ft. Figure 4-5 presents values of the virgin compression, CR , and of the coefficient of consolidation for vertical drainage, c_v , from laboratory consolidation tests in the normally consolidated range. The clay layer was initially over-consolidated, (OCR=2.3 to 1.5 with decreasing elevation). It will be assumed, as done by Noiray, that the layer became normally consolidated, due to rapid consolidation during reompression, before one month after the end of the loading period.

The Alidrains were installed by the Vibroflotation Company in a 5ft triangular spacing, so that d_e equals 1.05xs or 5.25 ft. Assuming that d_w is equal to 0.22 ft, this gives a spacing ratio n equal to 24. Although the drains were supposed to extend down to the top of the dense sand, leading to a double drainage condition, it may be possible that the mandrel sometimes met "refusal" somewhat above the dense sand layer.

In this chapter, the analysis of pore pressure data will be presented first, followed by an examination of the settlement data. Finally, a comparison of the results will be offered.

4-2. ANALYSIS OF THE EXCESS PORE PRESSURE DATA

The time origin is taken on July 29, 1981, as specified by Noiray, corresponding to the middle of the period of loading which was completed at a time of approximatively 70 to 75 days. Since many piezometers stopped working correctly after about 200 days, the analysis will concentrate on the data between t = 100 and t = 180 days.

a) Analysis of the data.

Figures 4-6 through 4-9 present the plots of the values of the logarithm of excess pore pressure versus time for all the piezometers which were working during the period previously mentioned. All of these data are tabulated in Appendix D. The plots allow one to assess for each piezometer the scatter about and/or the validity of the presumed linear relation between log u and t which is used for the method of analysis to back-calculate field values of c_h, which assumes pure radial drainage. Two piezometers, SP-20 and P-28, show a curved trend and thus are not used in the analysis.

Linear regressions between log u and time were performed on each set of data in order to obtain the coefficients α_o and α_1 of Eq. 3-10[Note: $\alpha_1 = \ln(u_1/u_2)/(t_2-t_1) = 2.3 \log(u_1/u_2)/(t_2-t_1)$]. The lines resulting from such analysis are plotted on the figures. These two values lead to the values of c_h and $\nu\bar{u}_o$ using Eqs. 3-11 and 3-12. Table 4-1 summarizes the results obtained, per location and depth zone.

The mean coefficient of consolidation, c_h, was found to be 0.038 ft²/day, with a standard deviation equal to 0.008. Although the means at the three sections and the means by layers do vary, the differences probably are not very significant given the limited number of values and the large standard deviation in most cases.

As presented in Appendix D, the linear regressions used to analyse each piezometer yielded coefficients of regression higher than 0.98 , except for three piezometers where the coefficients were between 0.96 and 0.98.

If the coefficient of consolidation, c_v, is chosen as the mean value from Fig. 4-5, that is equal to 0.02 ft²/day, the ratio of the time factors T_v/T_h is equal to 0.013 (i.e.

$(d_e/H_d)^2 x (c_v/c_h) = (5.25/32)^2 x (0.02/0.04) = 0.013)$. All the tips of the piezometer were located at a depth factor Z equal or higher than 0.40. If one refers to Fig. 2-5, one can see that for such values of Z and T_v/T_h, there should be practically no effect of vertical drainage on the calculated values of c_h since T_h ranged from 0.15 to 0.26.

One can also check the effect of well resistance by calculating the paramenter W_R (see section 2-5). In Table 6-1 of Noiray's thesis, the coefficients of permeability from falling head tests run on eight centerline piezometers are all lower than 5 x 10^{-8} cm/s. If the discharge capacity exceeds 500 m^3/yr for a lateral stress of 6.2 KSF, the value of W_R is less than 0.02. Therefore, there should be no effect of well resistance and the values from Table 4-1 represent the "actual" values of c_h, but including smear.

Equation 2-16 will be used to provide a rough estimate of the possible effects of smear. Assuming a smear diameter ratio s equal to 1.5 and a ratio of undisturbed to remolded permeability k_h/k'_h equal to 3, which probably should represent upper limits for this deposit, the factors F(n) and $F_s(n)$ are:

$$F(24) = 2.43 \quad \text{and} \quad F_s(24) = 3.24$$

As the ratio $c_h/F(n)$ from Eq.3-11 is a constant independent of the effect of smear and is equal to 0.016 (i.e. 0.038/2.43) for this study, the actual mean coefficient of consolidation would become: $c_h = 0.051$ ft^2/ day [i.e. $0.016xF_s(24)$].

b) Comparison with Noiray's results.

In the previous analysis of this case history, for the same piezometers during the same period of time, Noiray used the conventional method (without correction for smear) for two different approaches. Her approach number 1 used an initial excess pore pressure, $\bar{u}_o$, corresponding to the full load and the same origin of time as used in this study. Her approach number 2 used a $\bar{u}_o$ corresponding to the applied load minus the amount of in situ precompression and a time origin at around the middle of September 1981 depending on the piezometer. She obtained, using the incremental method she considered more reliable:

Approach	Mean:	Standard Deviation:
No 1	0.04 ft^2/day	0.02
No 2	0.04 ft^2/day	0.01

The mean values, which agree with each other, are essentially the same as obtained by the writer.

c) Back-analysis of the excess pore pressure data.

The piezometer SP-22 at El.-31.1 ft of the South-centerline section will be treated as an example. Figure 4-10 show the plot of the excess pore pressure data versus time on which the back-analysis is made. The excess pore pressure-time curve back-calculated with the parameters obtained from the present analysis matches the measured points very well, showing the validity of the method used.

This type of back-analysis is also performed on Noiray's results. Although approach number 1 gives the same c_h (0.044 versus 0.046), the large $\bar{u}_o$ leads to a significant overprediction of u versus time. Approach number 2, which in essence assumed very rapid consolidation during recompression, gives good agreement with the measured data.

4-3. ANALYSIS OF SETTLEMENT DATA

a) Analysis of the Sondex surface settlement data.

The surface settlement data used for the analysis were measured with Sondex systems located at the centerline of each section: North, Middle and South. The values represent the settlement of the entire clay layer and of the above thin sandy layer, which probably consolidated quickly enough to only affect the values of the "initial settlement".

The analysis was performed using the method for radial drainage only, described in Chapter 3. The settlement values were first plotted versus time and smooth curves were fitted through the points(see Figs. 4-11, 4-13 and 4-15). Asaoka's constructions were then plotted with a time interval of ten days for the period from 110 to 250 days (see Figs. 4-12, 4-14 and 4-16). Linear regressions were used to obtain the coefficients ρ_o and β_1 in Eq. 3-1 in order to determinate the coefficient of consolidation c_h from Eq.3-3 and the total settlement at the end of primary consolidation ρ_∞ from Eq. 3-2. The data used for the linear regression are tabulated in Table D-6.

From the analysis, the following values were found from the surface settlement data:

-- North -℄ : c_h = 0.106 ft^2/day; ρ_∞ = 4.0 ft
-- Middle-℄ : c_h = 0.079 ft^2/day; ρ_∞ = 3.3 ft
-- South -℄ : c_h = 0.088 ft^2/day; ρ_∞ = 2.3 ft

These values of the coefficient of consolidation yield a mean of 0.091 ft^2/day with a standard deviation of 0.014.

All the points plotted on Asaoka's construction were used in the above linear regressions, which yield very high coefficients of regression. Nevertheless, one can see that there are fluctuations about these lines, which are supposed to be straight. If one analyses the first six points of each plot independently , one obtains the following values:

-- North -℄ : c_h = 0.135 ft^2/day; ρ_∞ = 3.9 ft
-- Middle-℄ : c_h = 0.100 ft^2/day; ρ_∞ = 3.1 ft
-- South -℄ : c_h = 0.121 ft^2/day; ρ_∞ = 2.2 ft

These values yield a mean coefficient of consolidation of 0.119 ft^2/day with a standard deviation of 0.014.

The general fluctuations about the lines plotted on Asaoka's construction can yield very large changes in the values of the back-calculated c_h. For example, the settlement curve from the North section was divided into three zones: Zone I, from t=110 to t=160 days; Zone II, from t=160 to t=210 days; Zone III, from t=210 to t=250 days (see Figs. 4-11 and 4-12). The analysis was performed on these zones and yielded the following values:

--Zone I : $c_h = 0.139\ ft^2/day;\ \rho_\infty = 3.8\ ft$
--Zone II : $c_h = 0.061\ ft^2/day;\ \rho_\infty = 4.25\ ft$
--Zone III : $c_h = 0.206\ ft^2/day;\ \rho_\infty = 3.9\ ft$

For each zone, the coefficients of linear regression were higher than 0.99.

The effect of vertical drainage is calculated in Table 4-2 by the method proposed in Chapter 3. Assuming a coefficient of consolidation for vertical draingage c_v equal to 0.02 ft^2/day, the calculated error Δc_h is less than 0.002 ft^2/day. Hence the above values of c_h remain essentially unchanged. For a value of c_v equal to 0.04 ft^2/day, it would be almost the same because the calculated error Δc_h is 0.0025 ft^2/day.

As seen in the previous section, the effect of well resistance can be neglected. Therefore, the last effect which may be considered is the one from smear. If the same assumptions as in the previous section are made (s=1.5 and $k_h/k'_h=3$) and considering that the ratio $c_h/F(n)$, from Eq. 3-3, is also constant and independent of the effect of smear, the actual value of the coefficient of consolidation would be: $c_h = 0.121\ ft^2/day$ [i.e. $= 0.091 x F(24)/F_s(24)$].

The following compares values of the total settlement at the end of primary consolidation predicted by the analysis (based on data from t= 110 to 250 days) with measured total surface settlements obtained from Fig. 4-17 (Note: the times in that figure are about 40 to 50 days less than those plotted in the previous figures):

	Settlement (ft)		
	North*	Middle	South
Predicted by Analysis	4.0 ft	3.3 ft	2.3 ft
Measured (8/82)	4.15 ft	3.4 ft	2.4 ft
Measured (12/82)	4.4 ft	3.75 ft	2.65 ft

(* North values obtained from horizontal inclinometer data)

Since the load increased slightly after August 1982, one may compare the predicted values with those measured at that time. The results agree fairly well if one assumes that the consolidation process was finished at that time, but the piezometer data collected after 8/82 indicate that primary consolidation was not completed. (Note: It is difficult to draw definite conclusions because of significant changes in fill geometry after 8/82 and because no piezometric data were available from May 82 until January 83).

b) Comparison with Noiray's results.

With the conventional method, respectively for the total time approach and the incremental approach, Noiray obtained a mean coefficient of consolidation of 0.14 and 0.11 ft^2/day with standard deviations of 0.03 and 0.01. Since a small error in the origin of time may significantly affect the total time values, she considered the incremental time approach as more reliable. Her analysis of the three sections corresponds to a period of time from about 70 to 170 days based on the same time origin as used in this chapter. The results previously reported for the first six points of each section corresponds to a period from 110 to 170 days. They yielded about the same mean c_h as the one from Noiray's analysis (0.12 ft^2/day versus 0.11 ft^2/day). Therefore, both types of analysis give the same value of c_h when based on the same approximate set of data.

The values of the total final surface settlement used by Noiray in her analaysis are even lower than the one previously presented and correspond to final settlement at the end of primary consolidation which are considered too small. Her values used for the total final settlement for the North, Middle and South section were 3.9 ft, 3.0 ft and 2.2 ft repectively.

c) <u>Back-analysis of the settlement data</u>.

Figures 4-11, 4-13, and 4-15 were used for a back-analysis of the results. The three curves back-calculated from the results of the new method of analysis using Eq.3-9 match the original curves very well, except for times less than 100 days for the Middle section, which suggests that the analysis is valid. It should be noted, however, that the value back-calculated for the pseudo-initial settlement do not agree with the ones predicted by Noiray, which are considered reasonable.

The pseudo-initial settlements were calculated as presented in Chapter 3, that is:

* Choose time: e.g. for North, t = 175 days.

* Obtain settlement value at that time from the smoothed curve. e.g. for North, ρ (175) = 3.607 ft

* Calculate $\bar{U}(t)$ from Eq. 2-2 considering c_h obtained. e.g. for North, $\bar{U}(175)$ = 0.891 for c_h = 0.106.

* Calculate ρ_i from Eq. 3-9:

$$\rho_i = [\rho(t) - \rho_\infty x\bar{U}(t)]/[1-\bar{U}(t)]$$

e.g. for North, ρ_i = 0.47 ft with ρ_∞ = 3.99 ft

The resulting values are compared with those from Table F-2 in Noiray's thesis:

	Settlement (ft)		
	North	Middle	South
From this study*	0.47	-0.11	-0.09
From Noiray*	0.74	0.30	0.25
(Sand Layer)	0.45	0.08	0.17

(* These include the settlement of the sand layer.)

The discrepancies may come from the fact that the values obtained by the method presented above are very sensitive to the time origin. If one inputs the values of "ρ_i" obtained by Noiray in Eq. 3-9, the time origin would have to move 6 days ahead for North, 14.5 days ahead for Middle and 14 days ahead for South. These time changes are reasonable considering the fact that the ramp loading was not linear, especially for the Middle and the South sections where half the load had not been placed by the middle of the loading period (see Noiray's Figs 5-5, 5-6, and 5-7).

d) Analysis of the surface settlement from the horizontal inclinometers.

For each section, the maximum settlement measured from the horizontal inclinometers occurred at the centerline of the section. These data are plotted in Figs. 4-18, 4-20 and 4-22. The points are fitted by smooth curves for the analysis; the accuracy of the curve for the North and the South sections may not be too good due to the limited number of measurements.

The analysis was made for a period of time from around 160 days to about 220 days for the North and South sections and from 180 days to 340 days for the Middle section. These times all end before August 1982, after which the loads probably increased.

The analysis, performed as for the Sondex data, yielded:

- North -℄ : c_h = 0.046 ft²/day; ρ_∞ = 4.5 ft
- Middle-℄ : c_h = 0.042 ft²/day; ρ_∞ = 3.7 ft
- South -℄ : c_h = 0.036 ft²/day; ρ_∞ = 2.9 ft

These values give a mean of 0.041 ft²/day and a standard deviation of 0.005. As before, the calculated effect of vertical drainage is negligible. As can be seen in Fig. 4-21, the data fluctuates about the line resulting from Asaoka's construction for the Middle Section. The time span is too short to make a such judgement about the other two sections. In any case, due to the fluctuations and limited time span, the analysis of these settlement data may not give very reliable results. However, the values of the total settlement at the end of primary consolidation look reasonable when compared to the data in Fig.4-17 if one considers that primary consolidation was not completed in December, 1982 and that the increase of the load since August, 1982 was small.

The back-analysis results in Figs. 4-18, 4-20 and 4-22 show very good agreement between the predicted and measured curves. However the corresponding psuedo-initial settlements are about 2.5 times higher than the ones predicted by Noiray, although they are still lower than the measured settlements at the end of the loading period. Settlement points are also shown on the same figures based on the values of the coefficient of consolidation and total settlement, ρ_∞, obtained from the prior analysis of the Sondex data. This comparison shows that even when c_h is doubled, there is reasonable agreement between the predicted and measured time-settlement points. However, the predicted relationships do have a larger curvature, due to the increased c_h, which is noticeable on these large scale plots.

4-4. SUMMARY AND CONCLUSIONS

This case history of a precompression project with Alidrains for a cement plant located in Mobile, Alabama, illustrates the use of the new techniques of analysis described in Chapter 3. This case history had already been previously studied in a Master's thesis (Noiray, 1982) which allows one to compare the results with those obtained via "conventional" analyses.

The analysis of the excess pore pressure data yields for both methods a mean coefficient of consolidation for horizontal drainage equal to 0.04 ft^2/day.

The analysis of the Sondex surface settlement data yields a mean coefficient of consolidation equal to 0.09 ft^2/day in this study. When the same method is used to analyse data over about the same period of time as used by Noiray, it yields a mean c_n of 0.12 ft^2/day, which is in agreement with Noiray's results. A higher value of c_h for the period from 110 to 170 days would be reasonable if the clay layer just became normally consolidated during that time since one would expect a higher value of c_h during the early portion of virgin compression.

An analysis of surface settlement data from horizontal inclinometers was also performed. It yields a mean coefficient of consolidation of 0.04 ft^2/day, which is in agreement with the results from the analysis of the excess pore pressure values. As the predicted values of ρ_∞ for the analysis are considered more correct, this lower value of c_h may be considered to better reflect the in situ coefficient of consolidation within the normally consolidated range. This would mean that, for unclear reasons, the Sondex system may have overpredicted the actual rate of surface settlements.

From the analysis of this case history, one can also draw the conclusion that one needs settlement data covering a large range of values of the degree of consolidation. Otherwise, one can obtain misleading results if there are significant fluctuations about the line resulting from Asaoka's construction.

TABLE 4-1

c_h From Excess Pore Pressure Data

Note: ℄ = Centerline; E = 65 ft East of ℄

Layer	El.	N-℄	N-E	M-℄	M-E	M-S-℄	S-℄	S-E
I	-5 to -15	0.038 0.038	0.051	0.028			0.030	
II	-15 to -25			0.034		0.038	(0.053)	
III	-25 to -35	0.034 0.043*		0.032	0.042*		0.046	(0.047)
IV	-35 to -45					0.043*	0.045	

	Mean	SD
DATA SET:		
All values	0.038	0.006
BY LAYERS:		
I	0.037	0.009
II	0.036	0.003
III	0.039	0.006
IV	0.044	0.001
BY LOCATION:		
North section	0.041	0.007
Middle section	0.034	0.006
South section	0.040	0.009

N.B: - the sign * means $r < 0.98$
- ( ) means: value not used because the log(u) vs. time shows curved trend.

TABLE 4-2

Correction For Vertical Drainage
for the Surface Sondex Settlement Data

Parameters: d_e = 5.25 ft; n = 24; H_d = 32 ft
Δt = 10 days; t_m = 175 days

*Assuming c_v = 0.02 ft^2/day:

From Eqs. 3-4 and 3-5:

α = 0.00499
A = 0.99798
$\Delta c_h = [(d_e^2) x F(n)/8] x [\ln(A)/\Delta t]$

Δc_h = 0.0017 < 0.002

*Assuming c_v = 0.04 ft^2/day:

α = 0.00705
A = 0.99706

Δc_h = 0.0025

Fig. 4-1: Plan View of the RLSA Site Prior to Loading.
(from Noiray, 1982)

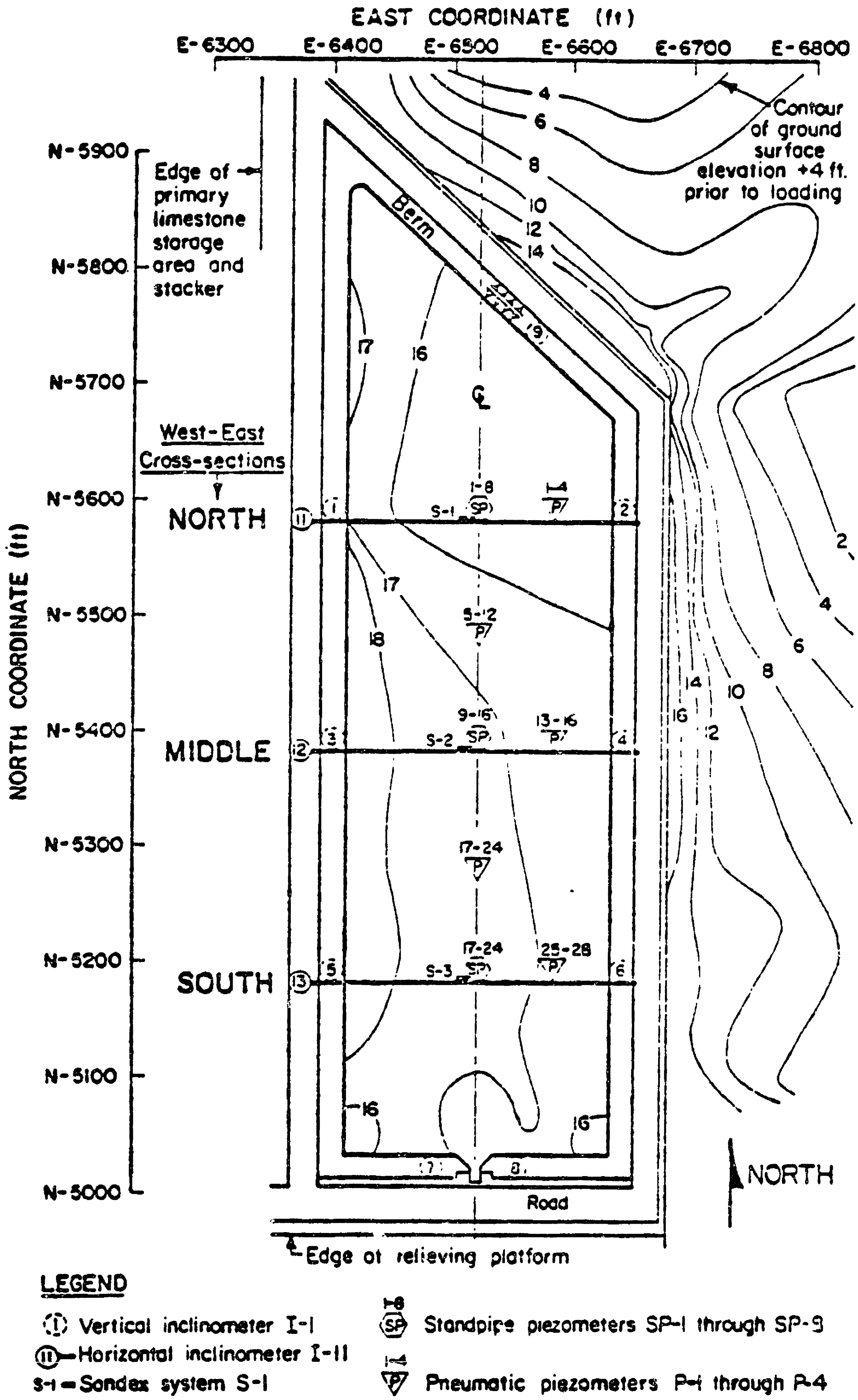


Fig. 4-2: RLSA North-South Cross-Section at the Centerline-
Conditions Prior to Loading.
(from Noiray, 1982)

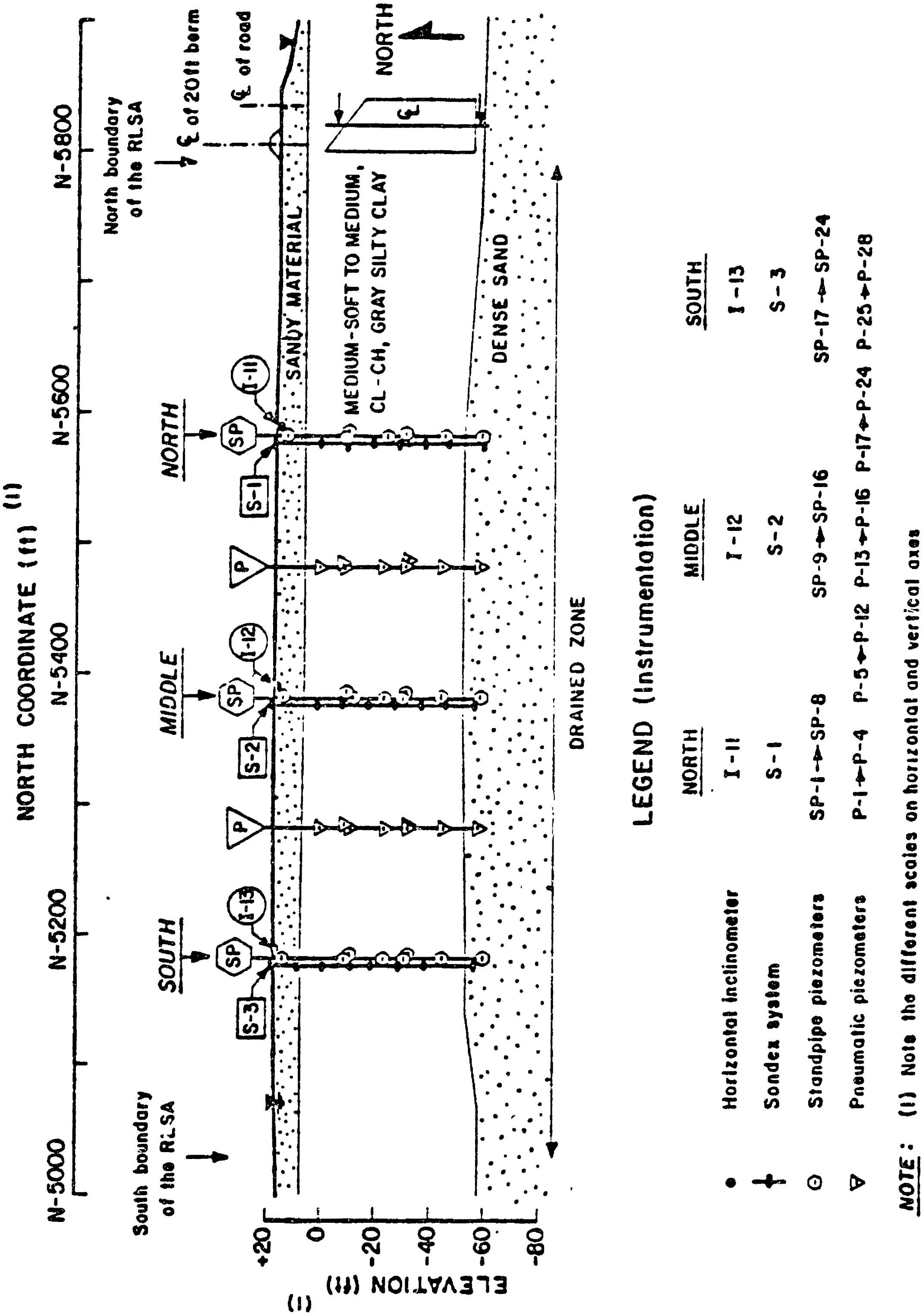


Fig. 4-3: RLSA West-East Cross-Section, Middle Location-
Condition Prior to Loading.
(from Noiray, 1982)

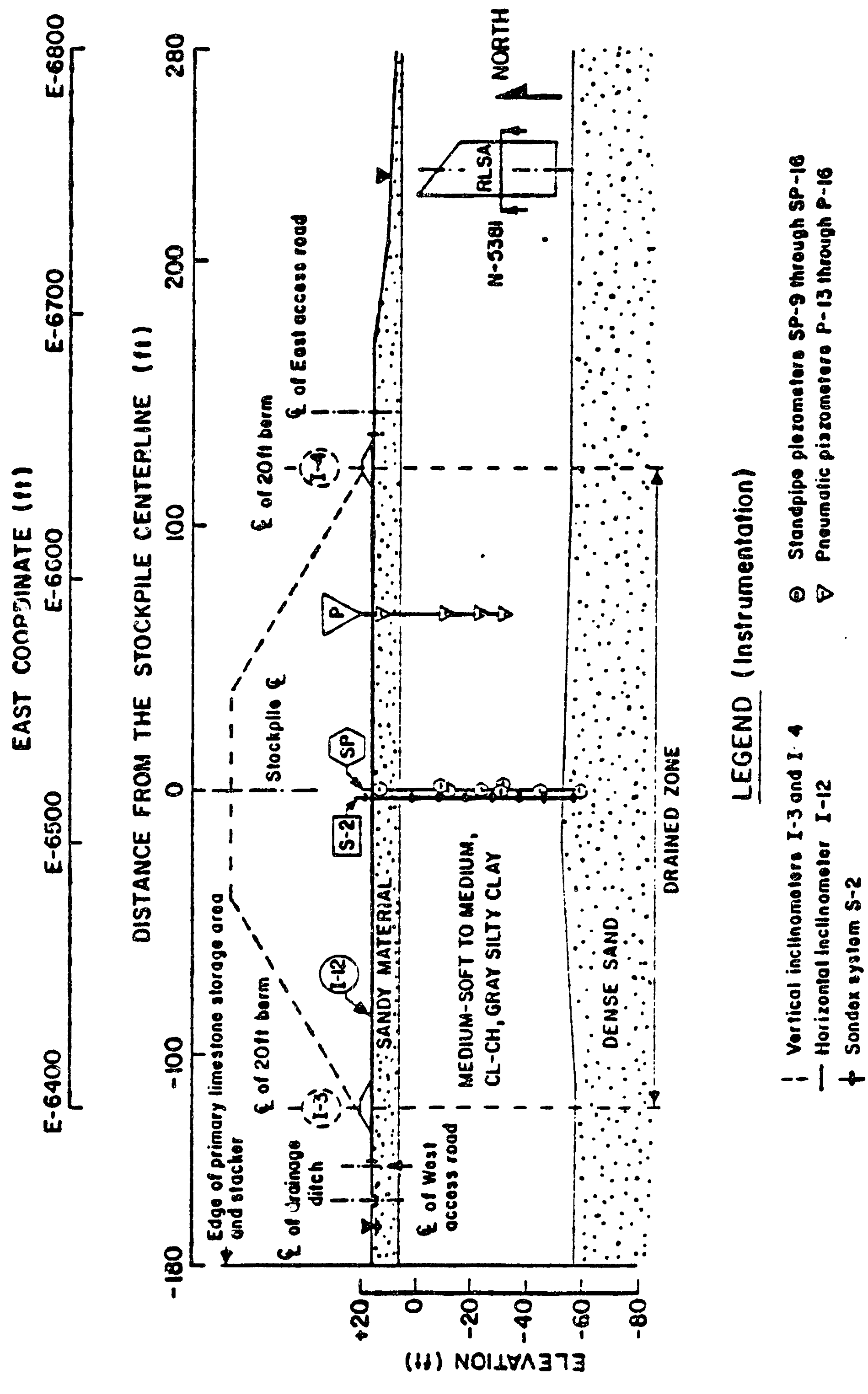
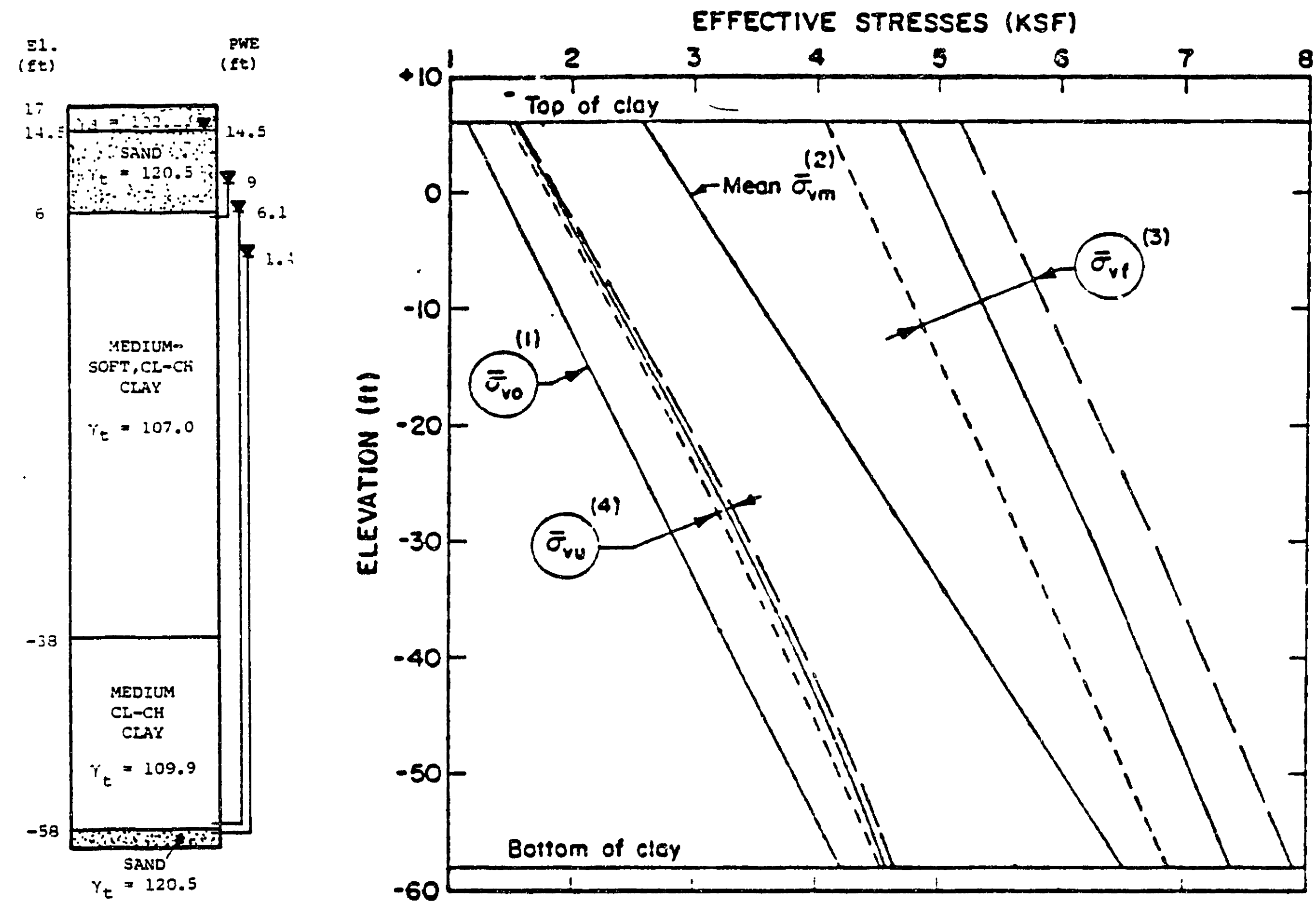


Fig. 4-4: RLSA Soil Profile and Stress History.
(from Noiray, 1982)



All units weight in pcf

Symbol	$\bar{\sigma}_{vu}$ and $\bar{\sigma}_{vf}$ for fill height
- - - - -	H_f = 25 ft
———	H_f = 30 ft
— — —	H_f = 35 ft

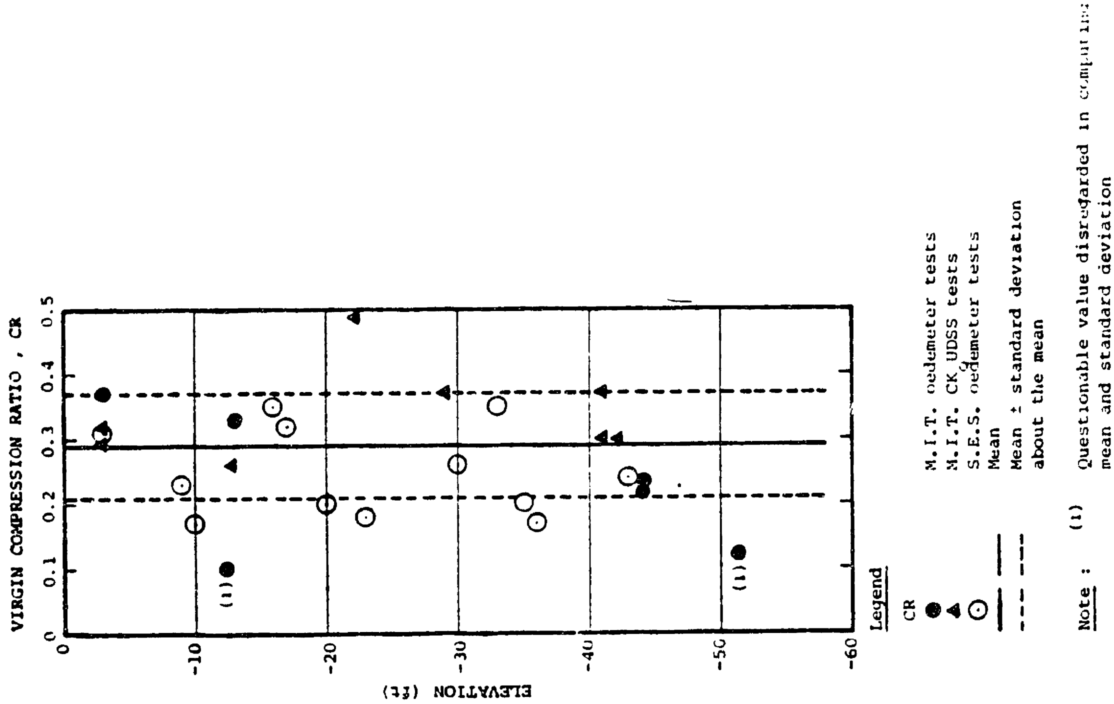
NOTES: (1) $\bar{\sigma}_{vo}$ = initial vertical effective stress; computed at the stockpile centerline, Middle cross-section

(2) $\bar{\sigma}_{vm}$ = maximum past pressure

(3) $\bar{\sigma}_{vf}$ = final vertical effective stress; obtained using FEECON analyses

(4) $\bar{\sigma}_{vu}$ = vertical effective stress at the end of undrained loading; obtained using FEECON analyses

Fig. 4-5: Normally Consolidated Vertical Coefficient of Consolidation, c_v, and Virgin Compression Ratio, CR, versus Depth.



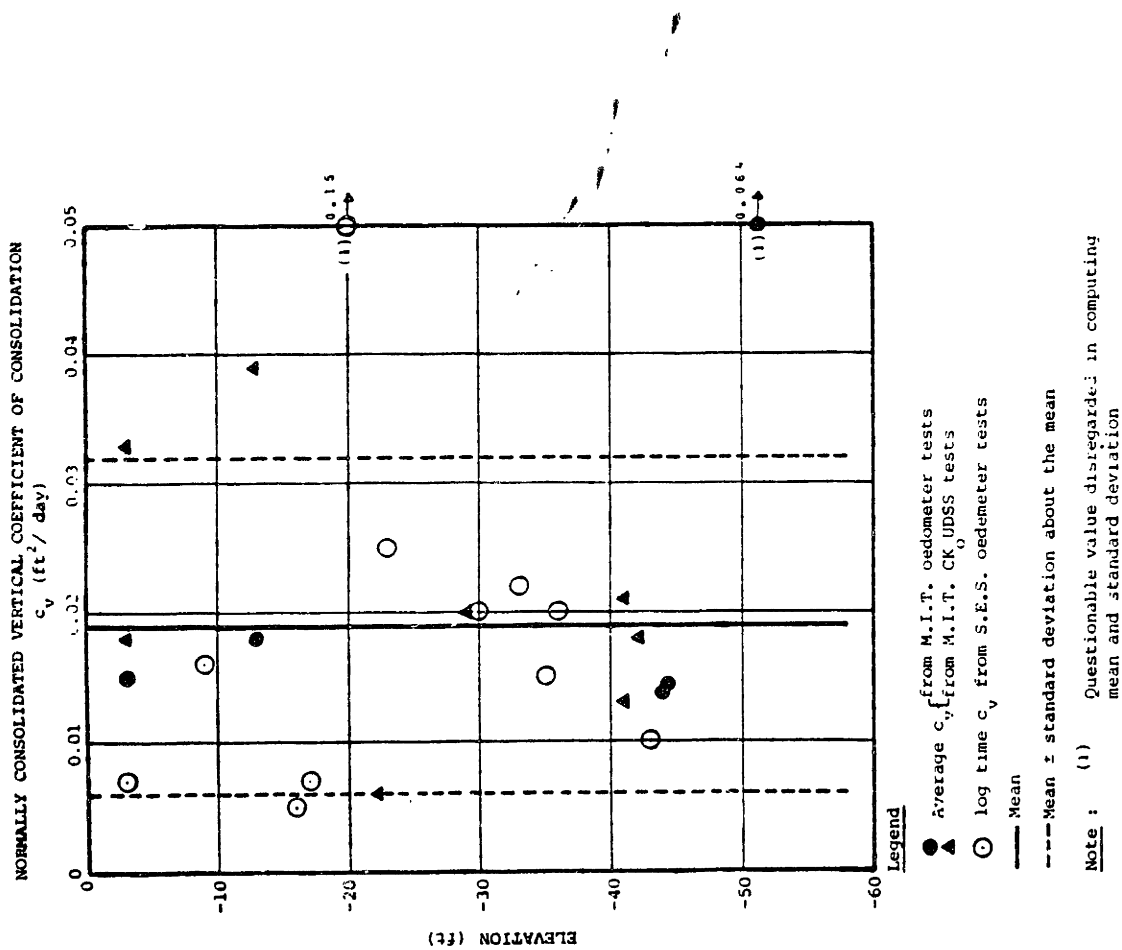
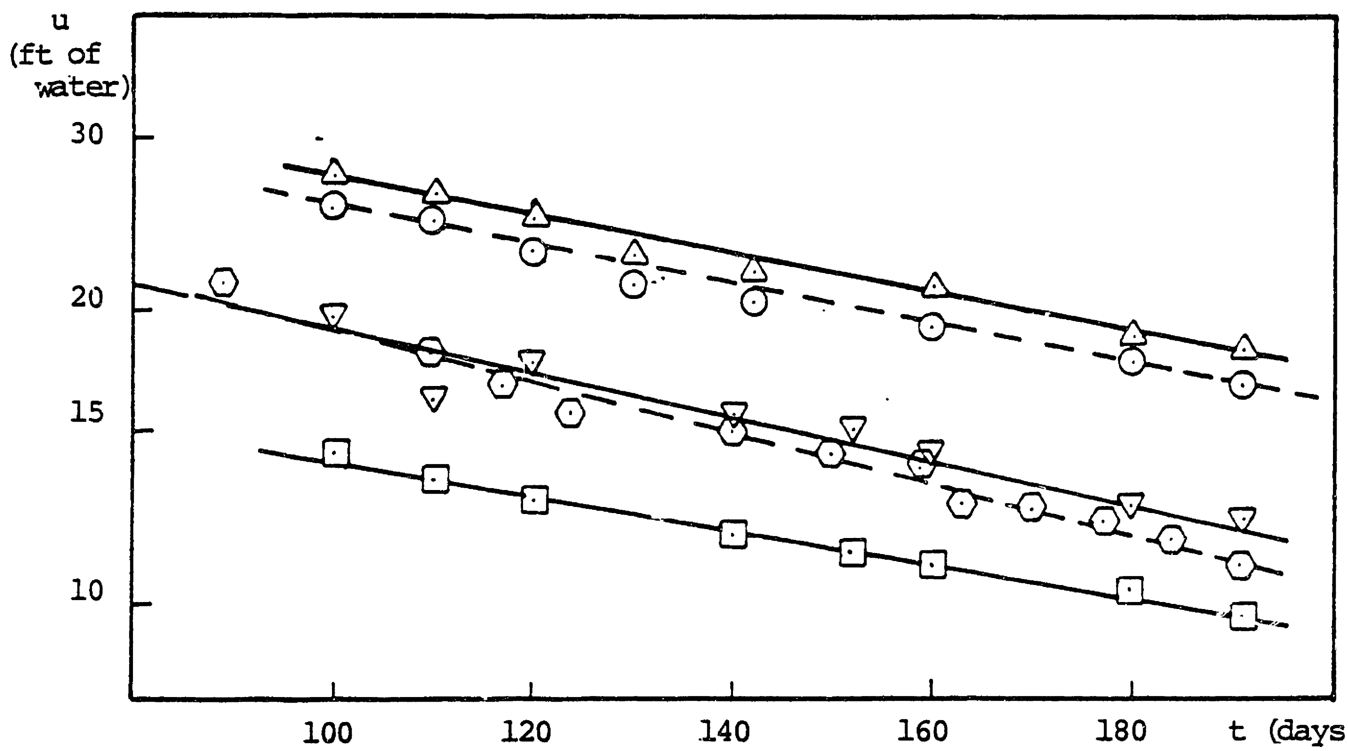


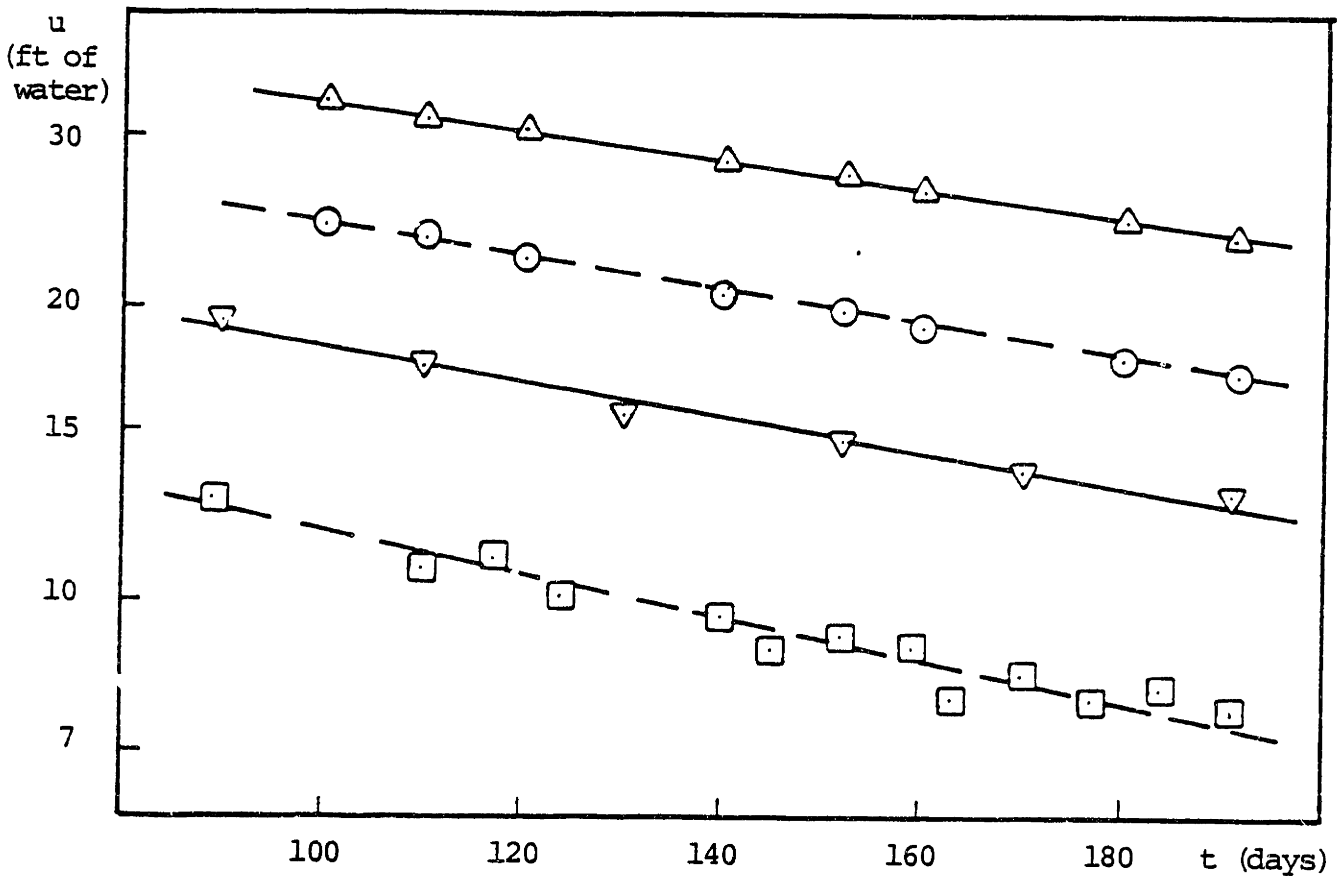
Fig. 4-6: Excess Pore Pressure (log scale) versus Time: North Section.



Piez. No.	El. (ft)	Symbol
SP-2	-9.0	△
SP-3	-9.9	⊙
SP-5	-31.6	⊡
SP-6	-30.7	▽
P-2*	-9.9	⬡

*65 ft East from Centerline

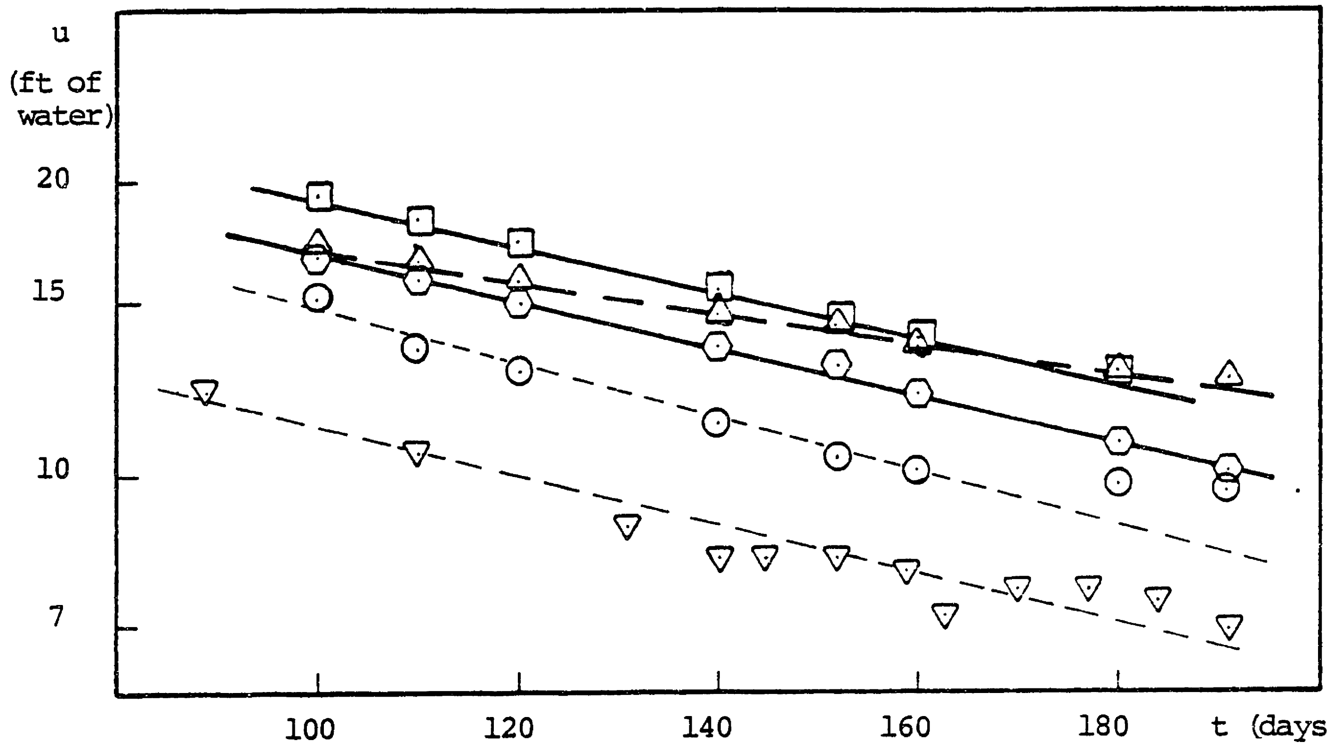
Fig. 4-7: Excess Pore Pressure (log scale) versus Time: Middle Section.



Piez. No.	El. (ft)	Symbol
SP-10	-11.4	△
SP-12	-23.2	▽
SP-14	-30.6	⊙
P-16*	-30.9	⊡

*65 ft East from Centerline

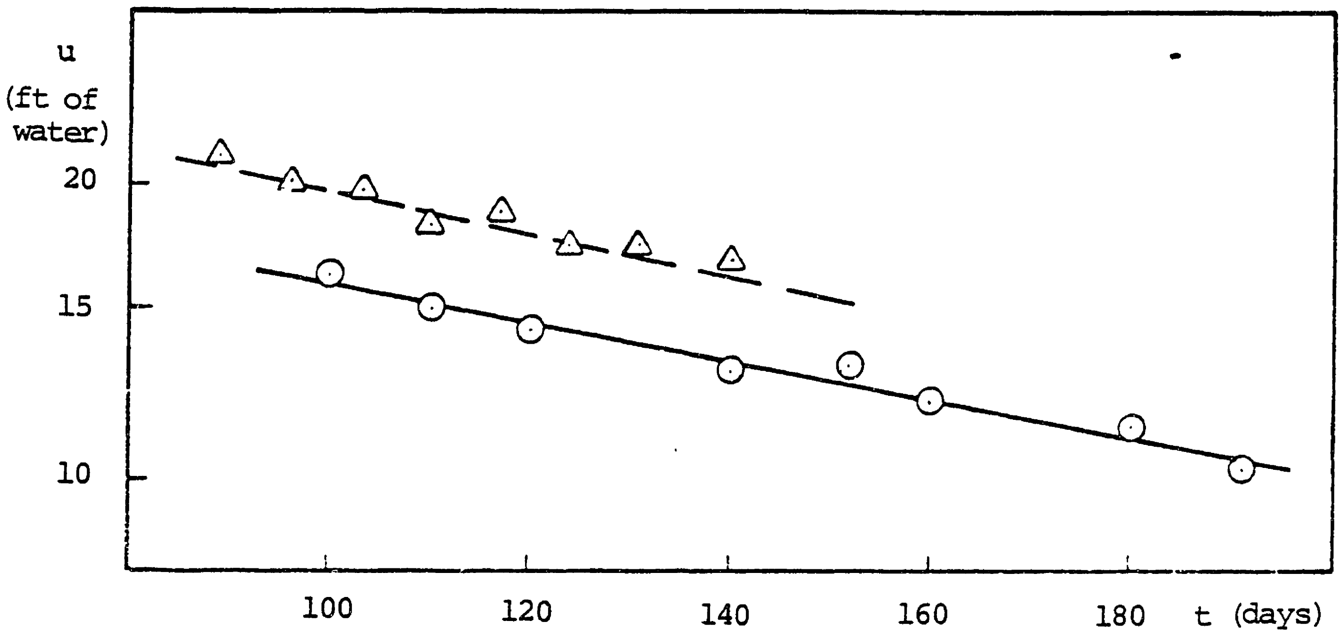
Fig. 4-8: Excess Pore Pressure (log scale) versus Time: South Section.



Piez. No.	El. (ft)	Symbol
SP-18	-9.1	△
(SP-20)	-23.4	⊙
SP-22	-31.1	⬡
SP-23	-45.0	⊡
(P-28)*	-32.1	▽

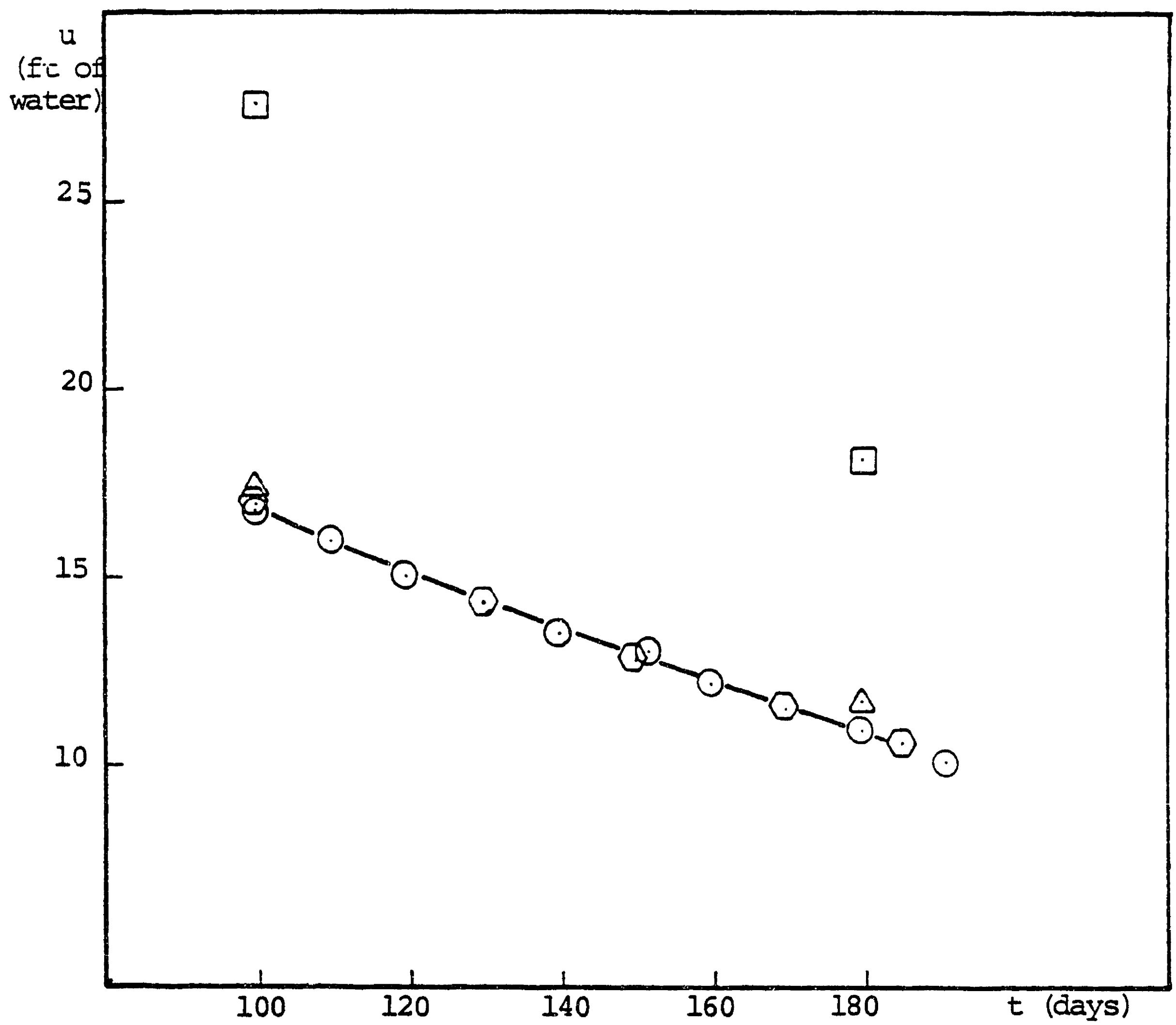
*65 ft East from Centerline
(   ) not used in the analysis

Fig. 4-9: Excess Pore Pressure (log scale) versus Time: Middle-South Section.



Piez. No.	El. (ft)	Symbol
P-20	-23.3	⊙
P-23	-45.4	△

Fig. 4-10: Excess Pore Pressure versus Time: SP-22: Back-Analysis



Legend:

⊙ measured points

⬡ back-calculated points from present analysis:
$\begin{cases} c_h = 0.046\ ft^2/day \\ \nu \bar{u}_o = 29.20\ ft \end{cases}$

back-calculated points using Noiray's data:

⊡ $\begin{cases} c_h = 0.044\ ft^2/day \\ \bar{u}_o = 42.2\ ft \end{cases}$ Approach 1 Incremental method

△ $\begin{cases} c_h = 0.041\ ft^2/day \\ \bar{u}_o = 19.7\ ft \end{cases}$ Approach 2 Incremental Method

Fig. 4-11: Sondex Surface Settlement versus Time:
North Section.

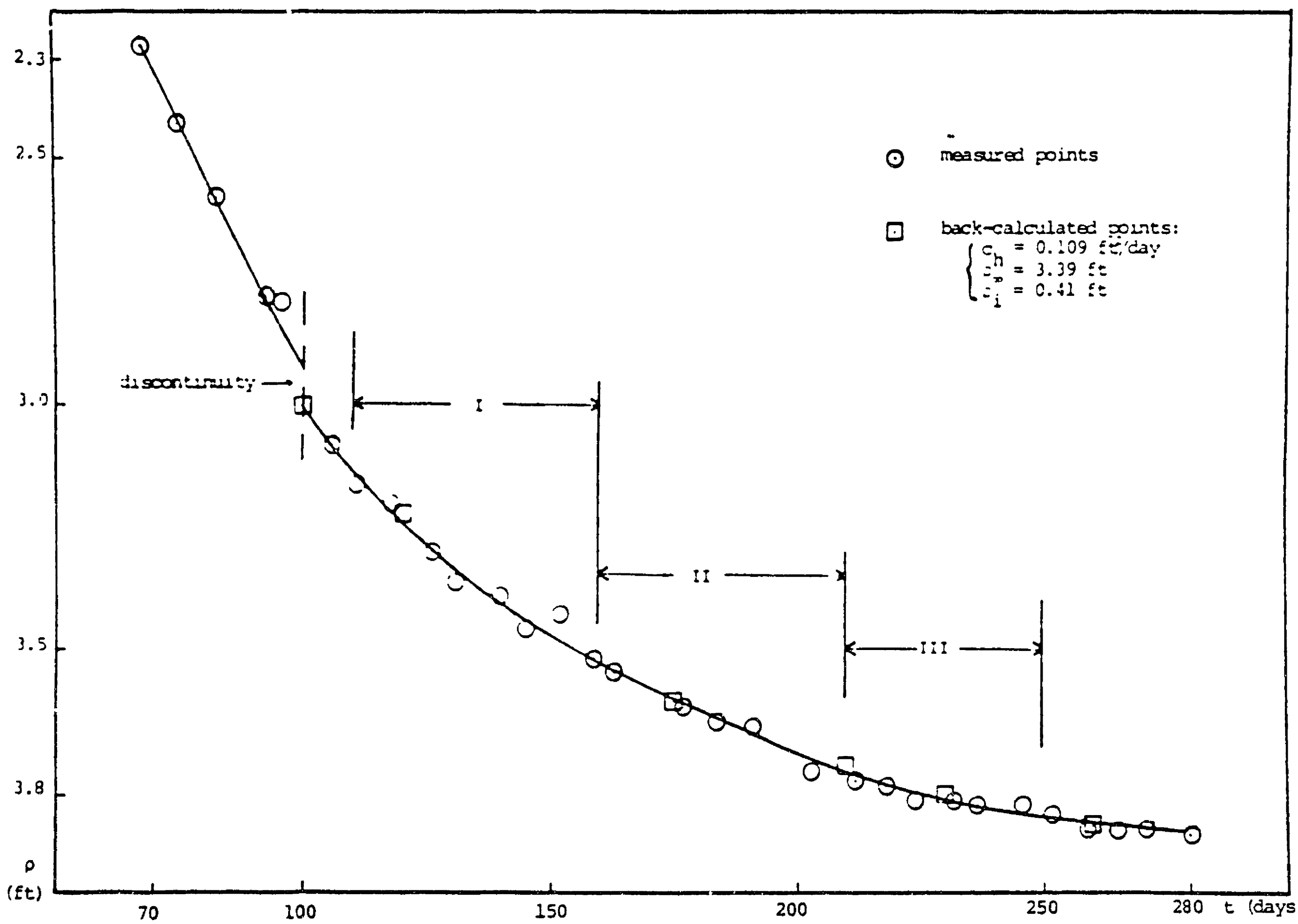
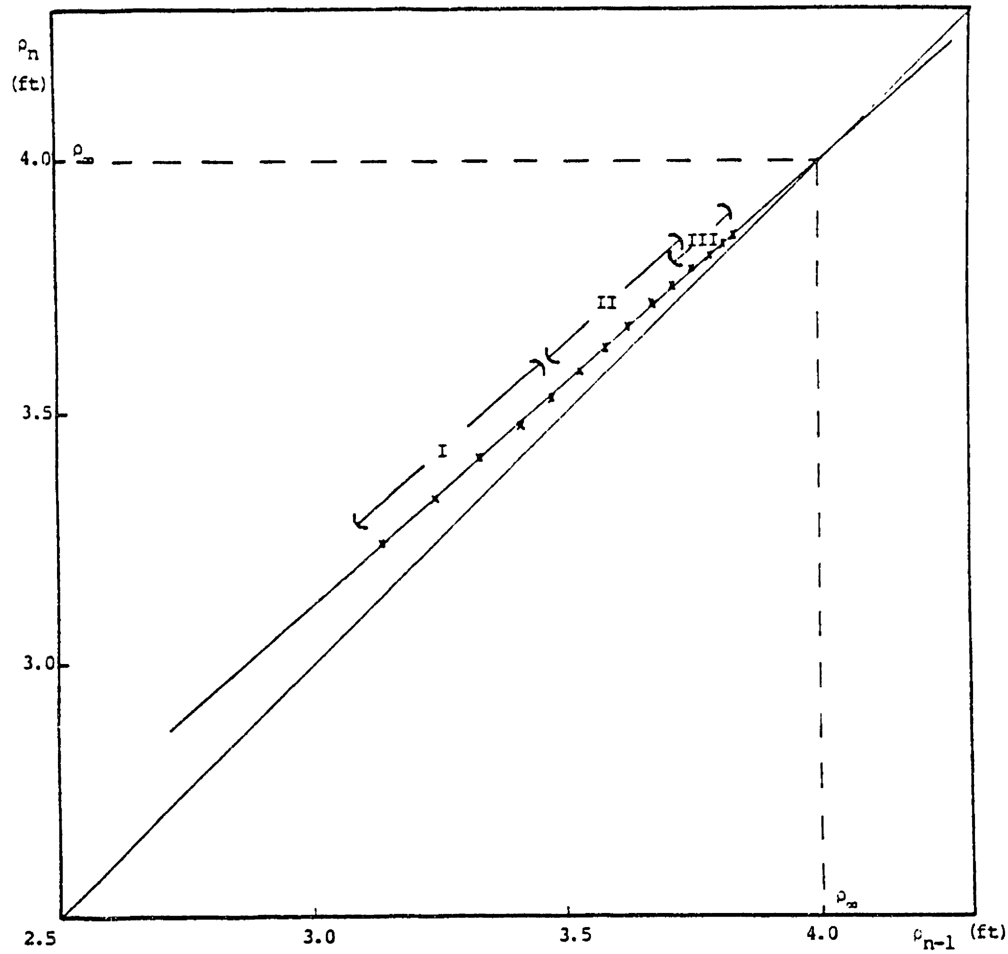


Fig. 4-12: Asaoka's Construction for Sondex Surface Settlement Data: North Section.



--Settlement data: Δt = 10 days; t_{start} = 110 days

--Analysis:

	Parameters	Results
All data	ρ_o = 0.4771 β_1 = 0.8805 r = 0.9997	c_h = 0.106 ft^2/day ρ_∞ = 3.99 ft
First 6 Points	ρ_o = 0.5742 β_1 = 0.851	c_h = 0.135 ft^2/day ρ_∞ = 3.89 ft
Zone I (first 5 points)	ρ_o = 0.8696 β_1 = 0.8474	c_h = 0.139 ft^2/day ρ_∞ = 3.84 ft
Zone II (next 6 points)	ρ_o = 0.2990 β_1 = 0.9297	c_h = 0.061 ft^2/day ρ_∞ = 4.25 ft
Zone III (last 4 points)	ρ_o = 0.8528 β_1 = 0.7817	c_h = 0.206 ft^2/day ρ_∞ = 3.91 ft

Fig. 4-13: Sondex Surface Settlement versus Time: Middle Section.

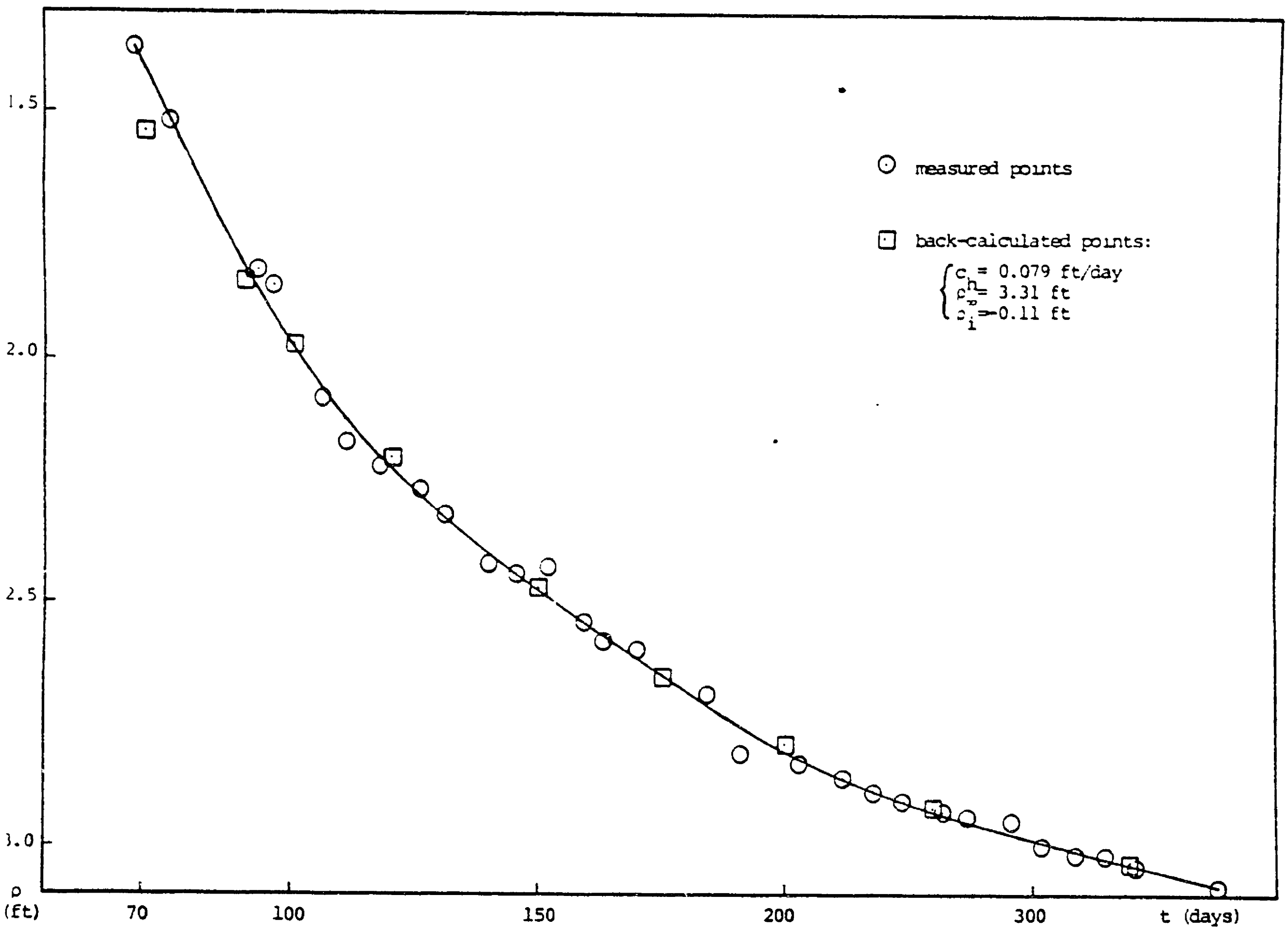
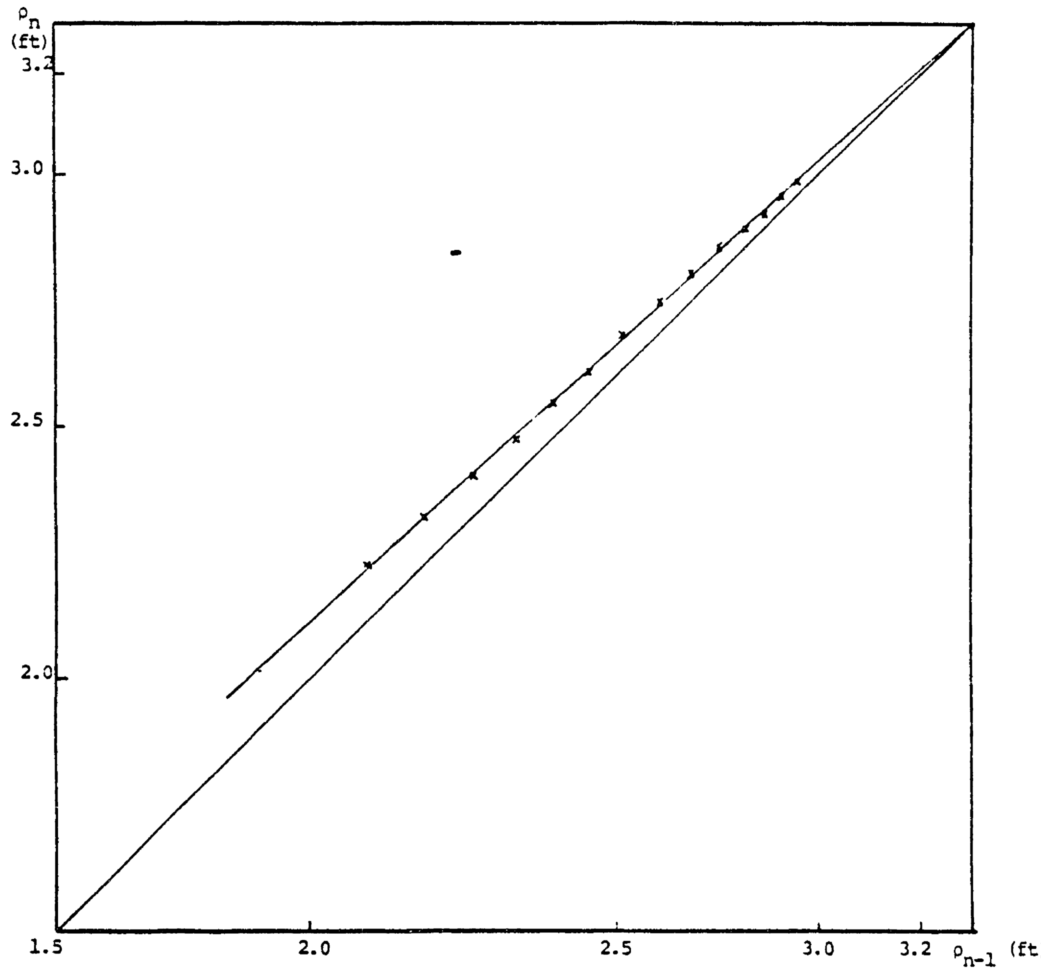


Fig. 4-14: Asaoka's Construction for Sondex Surface Settlement Data: Middle Section.



--**Settlement** data: Δt = 10 days; t_{start} = 110 days

--Analysis:

	Parameters	Results
All Points	ρ_o = 0.2976 β_1 = 0.9101 r = 0.9997	c_h = 0.079 ft^2/day ρ_∞ = 3.31 ft
First 6 Points	ρ_o = 0.3502 β_1 = 0.886	c_h = 0.100 ft^2/day ρ_∞ = 3.07 ft

Fig. 4-15: Sondex Surface Settlement versus Time: South Section.

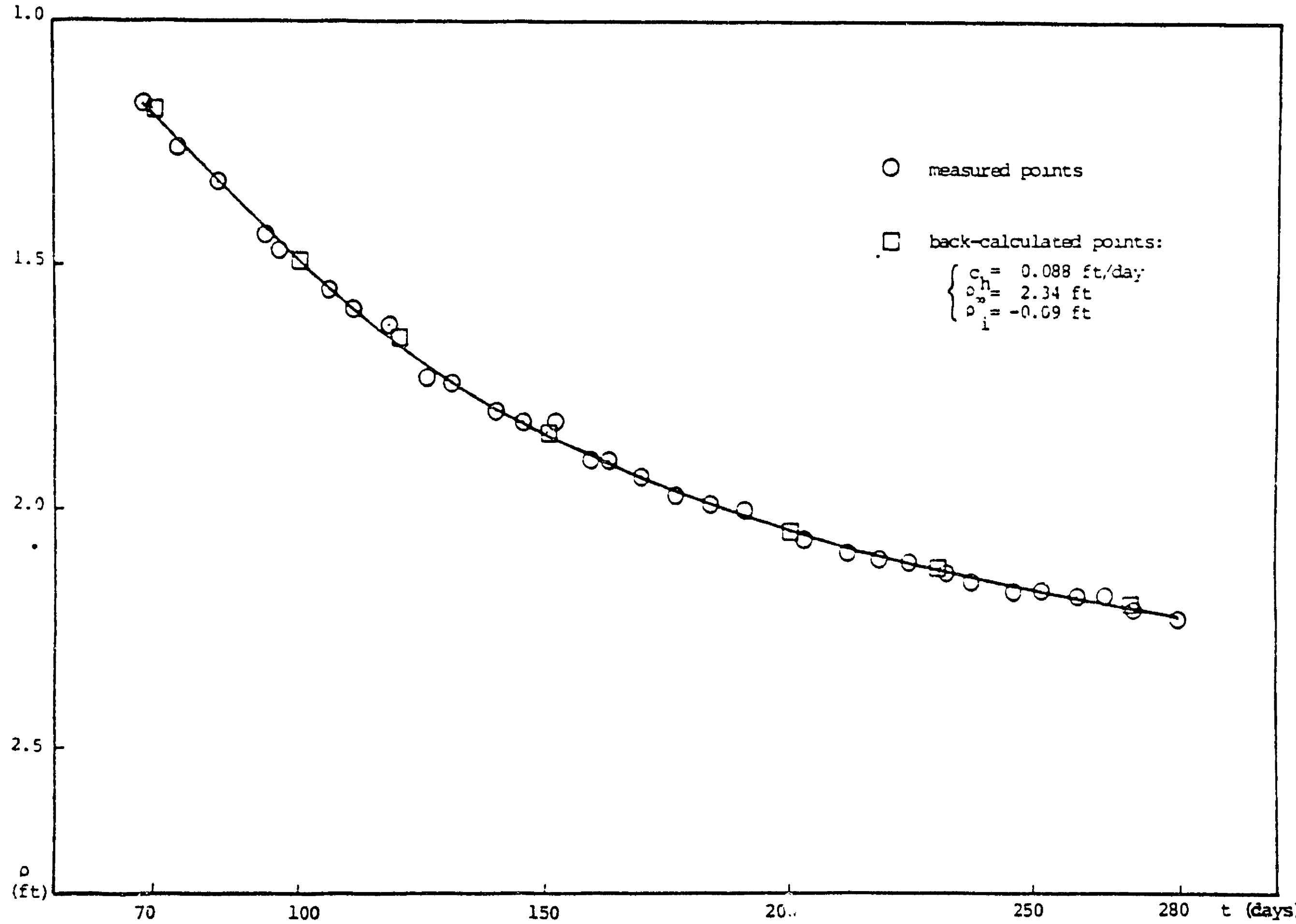
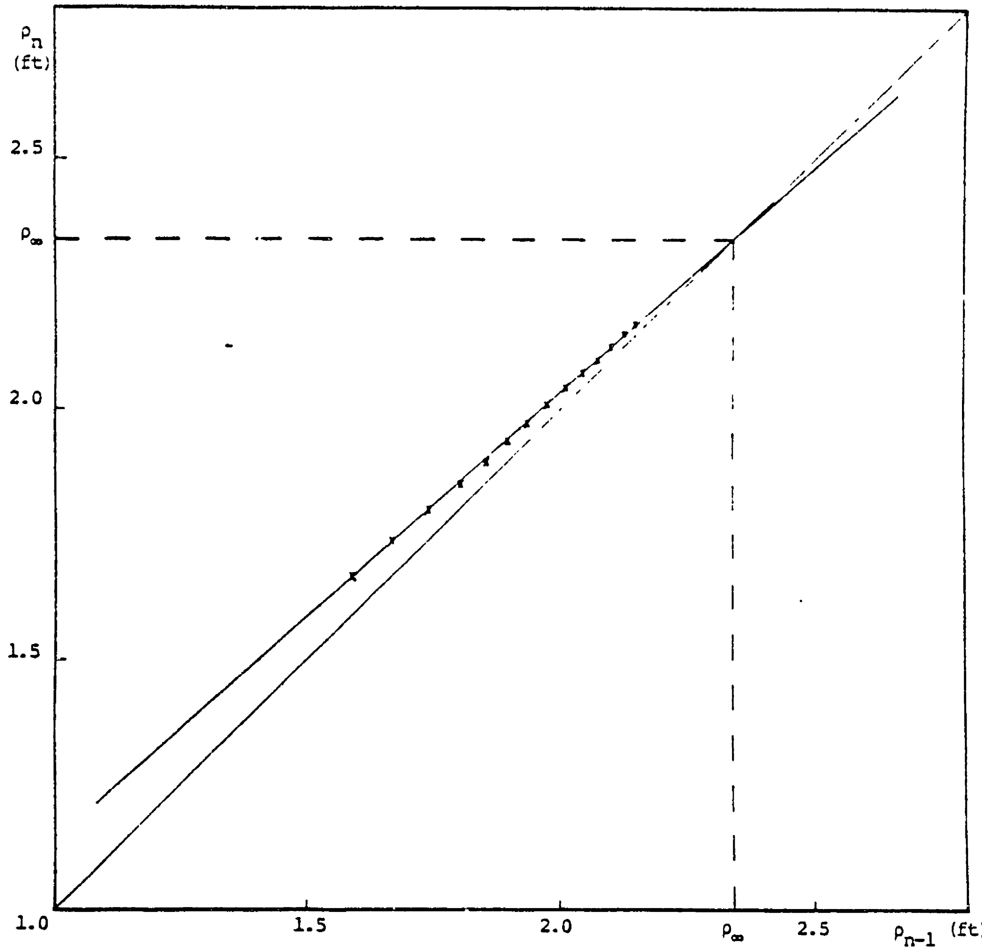


Fig. 4-16: Asaoka's Construction for Sondex Surface Settlement Data: South Section.



--Settlement data: Δt = 10 days; t_{start} = 110 days

--Analysis:

	Parameters	Results
All Points	ρ_o = 0.2345 β_1 = 0.8997 r = 0.9998	c_h = 0.088 ft^2/day ρ_∞ = 2.34 ft
First 6 Points	ρ_o = 0.2946 β_1 = 0.865	c_h = 0.121 ft^2/day ρ_∞ = 2.18 ft

Fig. 4-17: Surface Settlement versus Log Time.
(from Ladd, 1983)

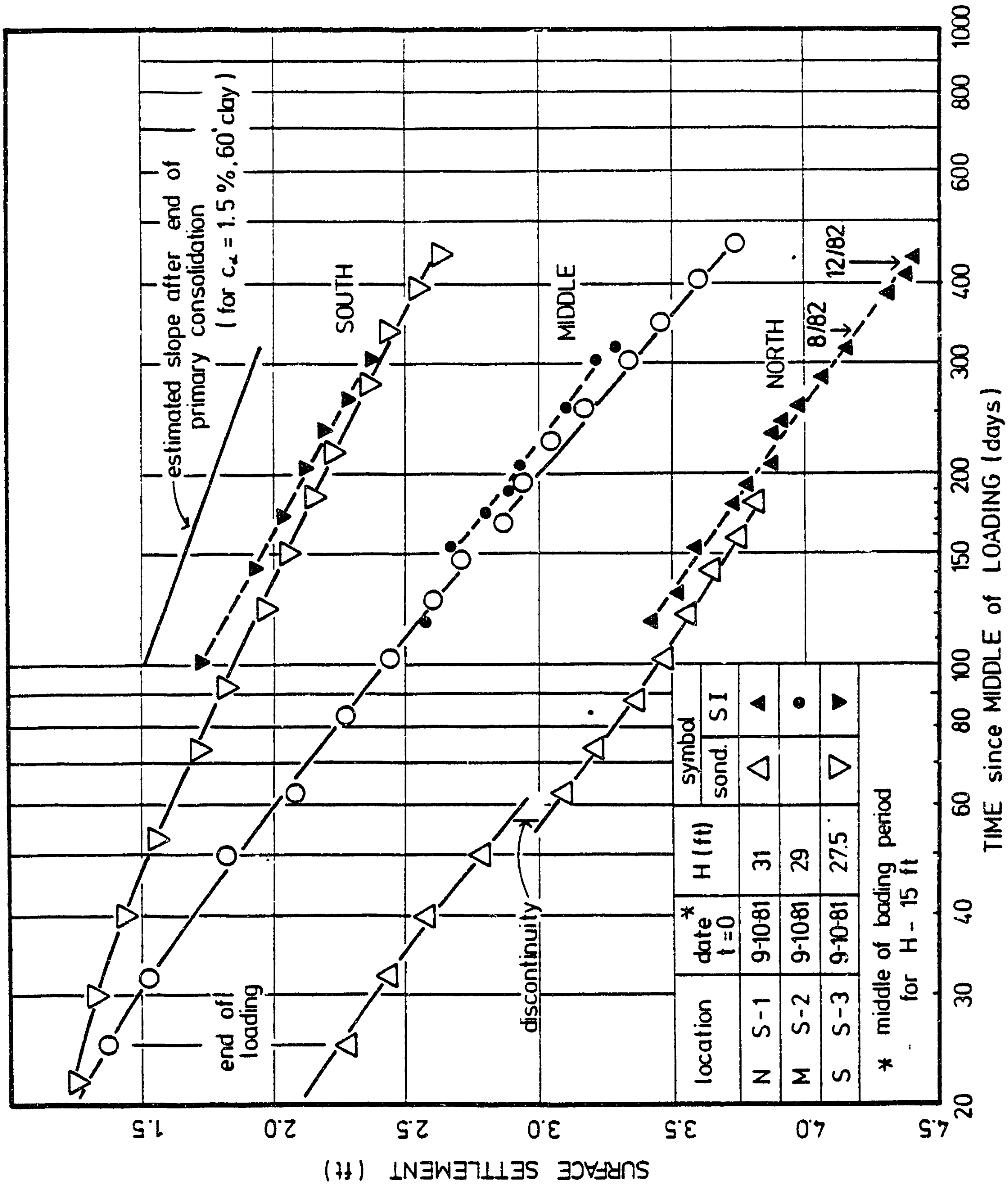
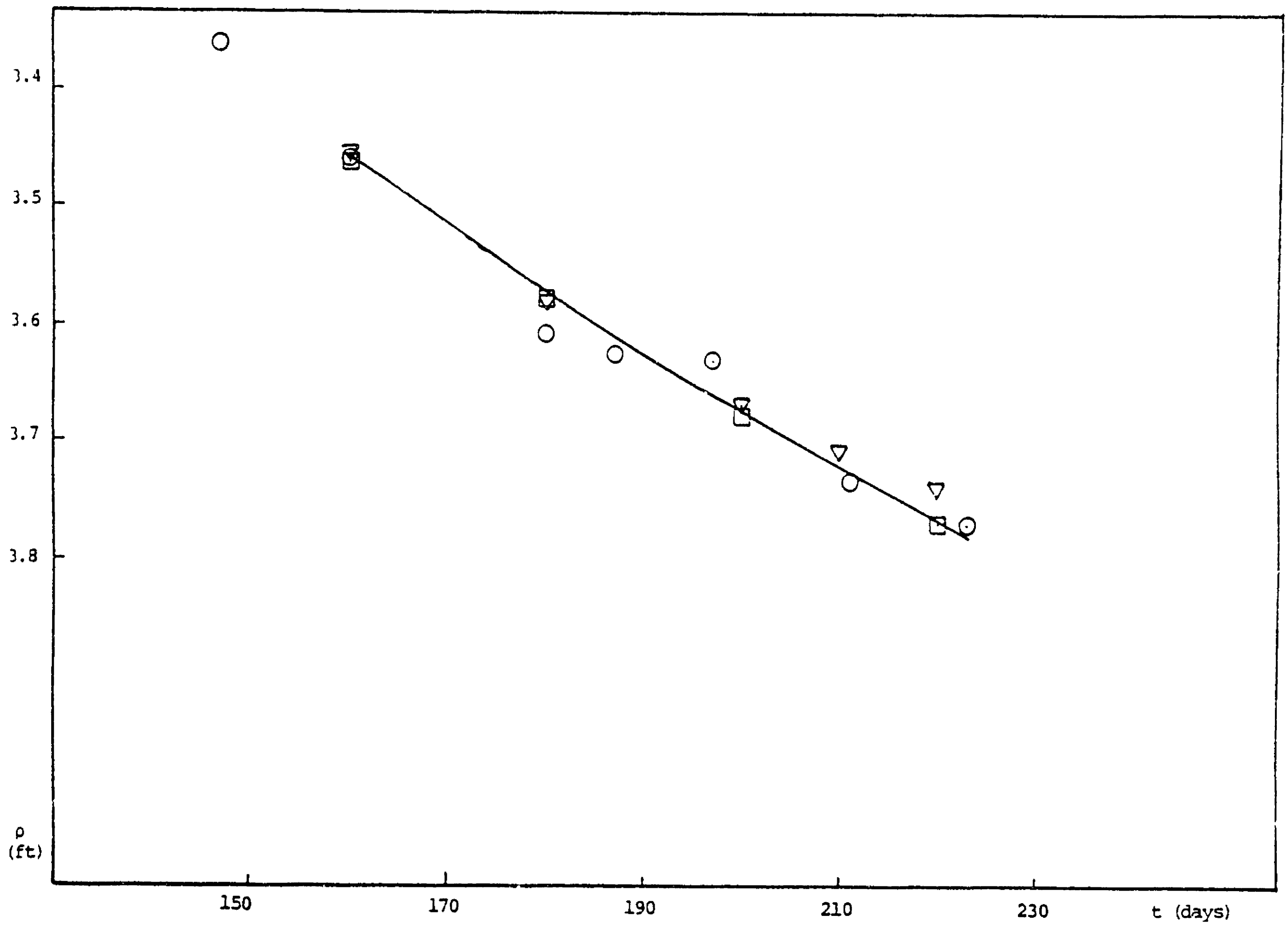


Fig. 4-18: Surface Settlement Data from Horizontal Inclinometer versus Time: North Section.



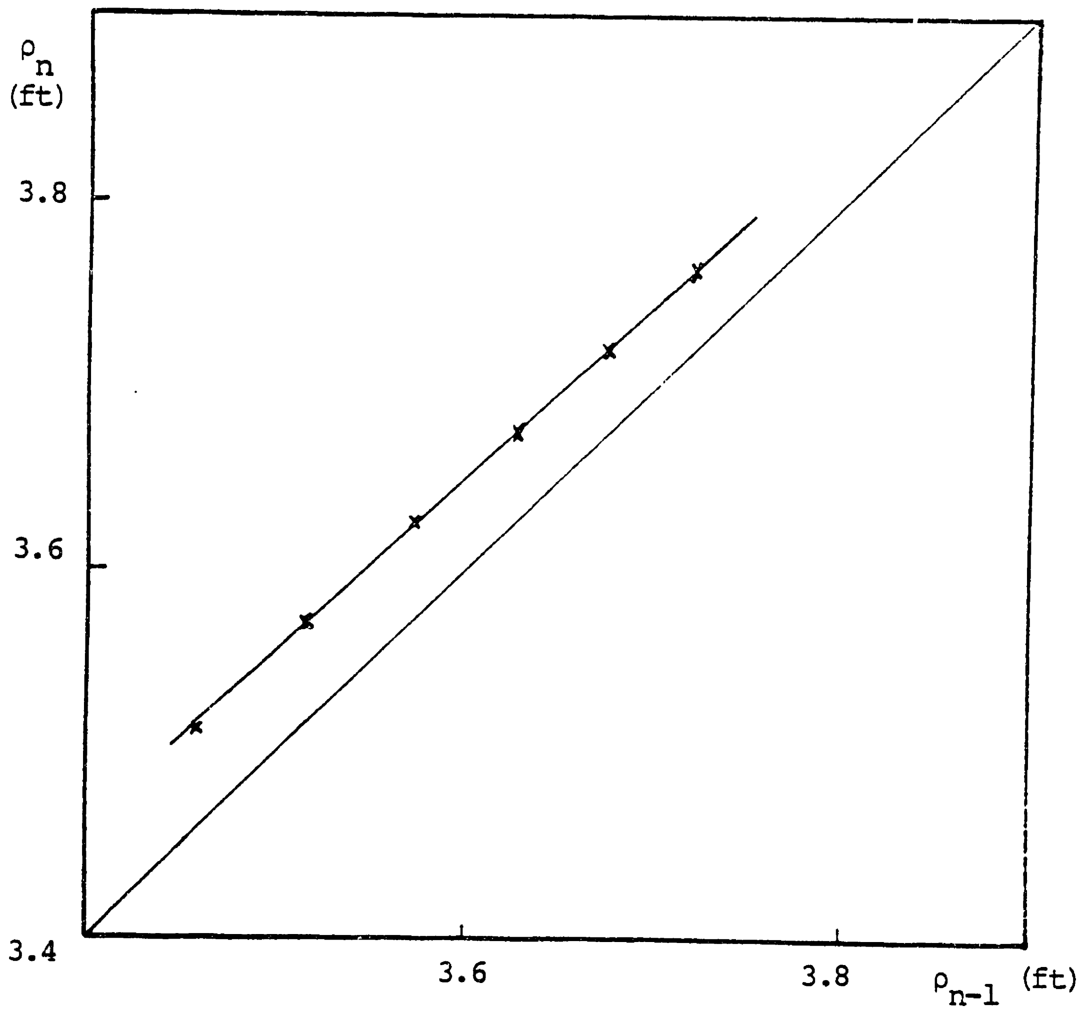
Legend:

⊙ measured points

⊡ back-calculated points from present analysis:
ρ_i = 1.92 ft for t = 190 days

▽ back-calculated points using:
$\begin{cases} c_h = 0.106 \text{ ft}^2\text{/day} \\ \rho_\infty = 3.99 \text{ ft} \end{cases}$ (values from Sondex analysis)
ρ_i = -0.05 ft for t = 190 days

Fig. 4-19: Asaoka's Construction for Surface Settlement Data from Horizontal Inclinometer: North Section.



--Values used for Analysis:

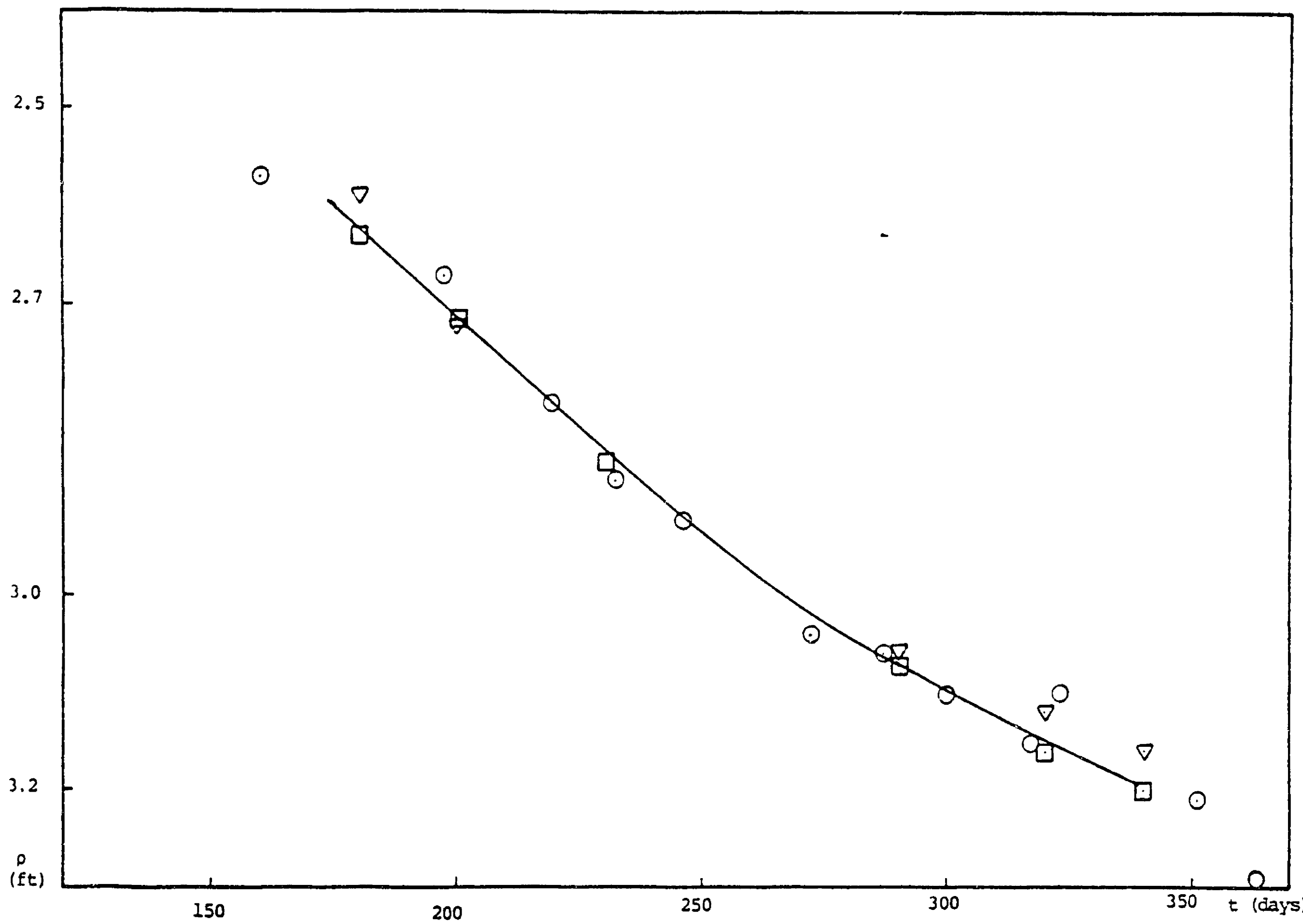
t (days)	ρ (ft)
160	3.46
170	3.51
180	3.57
190	3.63
200	3.68
210	3.72
220	3.77

--Analysis

Parameters	Results
$\rho_o = 0.24$	$c_h = 0.046\ ft^2/day$
$\beta_1 = 0.946$	$\rho_\infty = 4.55$ ft
$r = 0.9998$	

Vertical Drainage Effect: $\Delta c_h = 0.002$

Fig. 4-20: Surface Settlement Data from Horizontal Inclinometer: Middle Section.



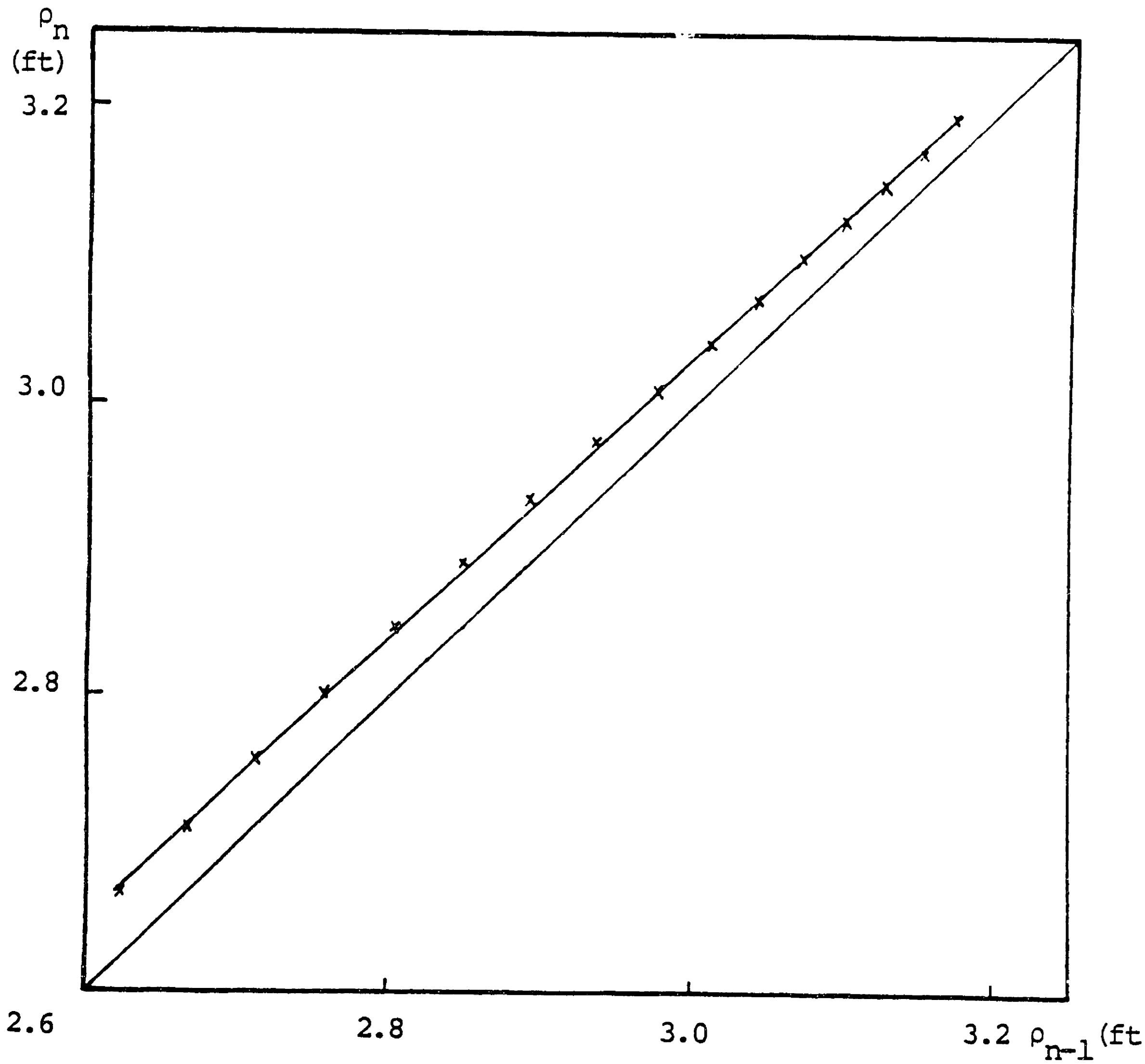
Legend:

⊙ measured points

⊡ back-calculated points from present analysis
ρ_i = 1.11 ft for t = 260 days

▽ back-calculated points using:
$\begin{cases} c_h = 0.079\ ft^2/day \\ \rho_\infty = 3.32\ ft \end{cases}$ (values from Sondex analysis)
ρ_i = -0.58 ft for t = 260 days

Fig. 4-21: Asaoka's Construction for Surface Settlement Data from Horizontal Inclinometer: Middle Section.



--Values used for analysis:

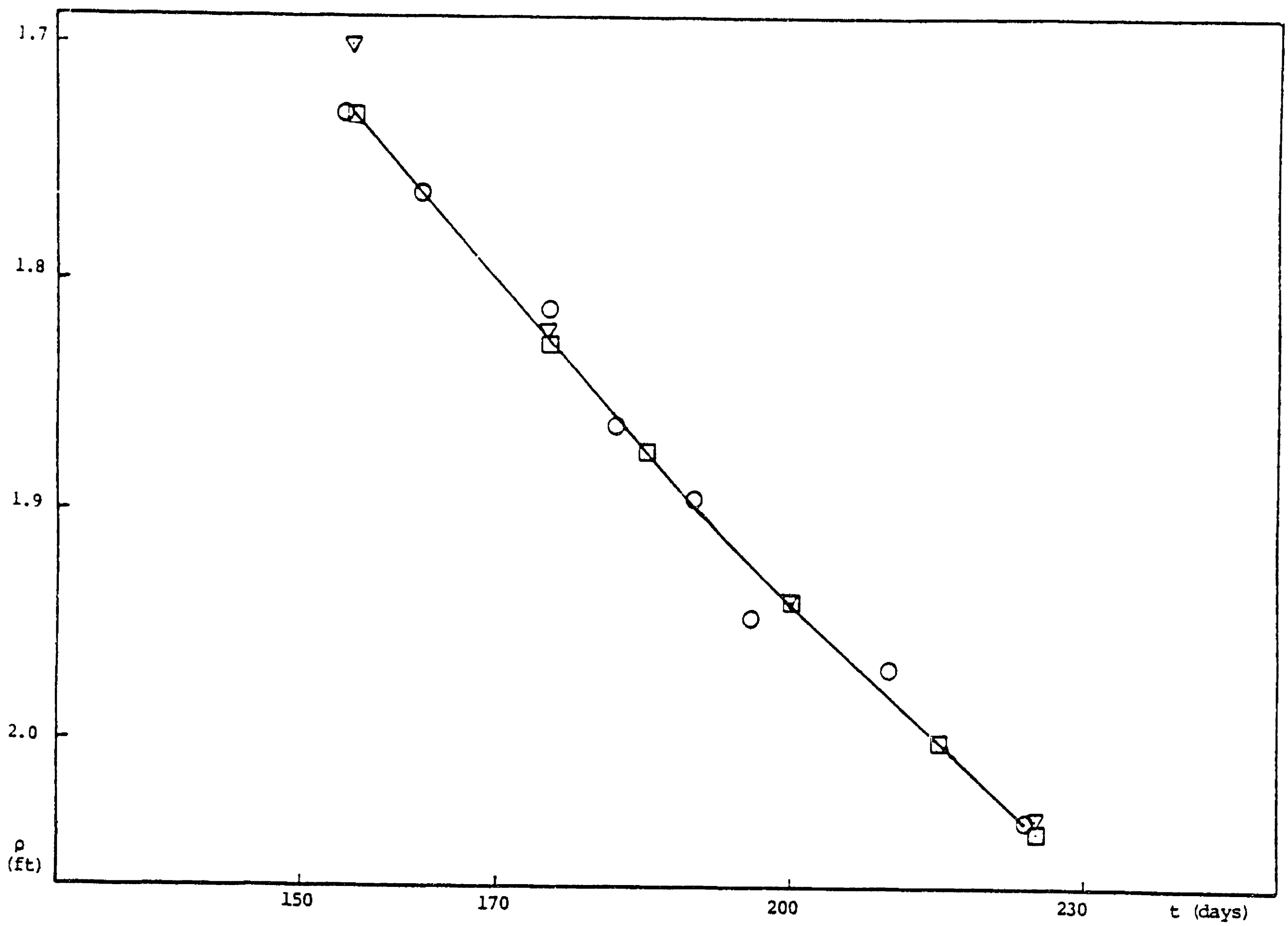
t (days)	ρ (ft)
180	2.622
190	2.667
200	2.712
210	2.757
220	2.802
230	2.848
240	2.892
250	2.935
260	2.975
270	3.01
280	3.041
290	3.071
300	3.093
310	3.125
320	3.149
330	3.171
340	3.194

--Analysis

Parameters	Results
ρ_o = 0.178	c_h = 0.042 ft²/day
β_1 = 0.952	ρ_∞ = 3.67 ft
r = 0.9996	

Vertical drainage effect: Δc_h = 0.002

Fig. 4-22: Surface Settlement Data from Horizontal Inclinometer versus Time: South Section.



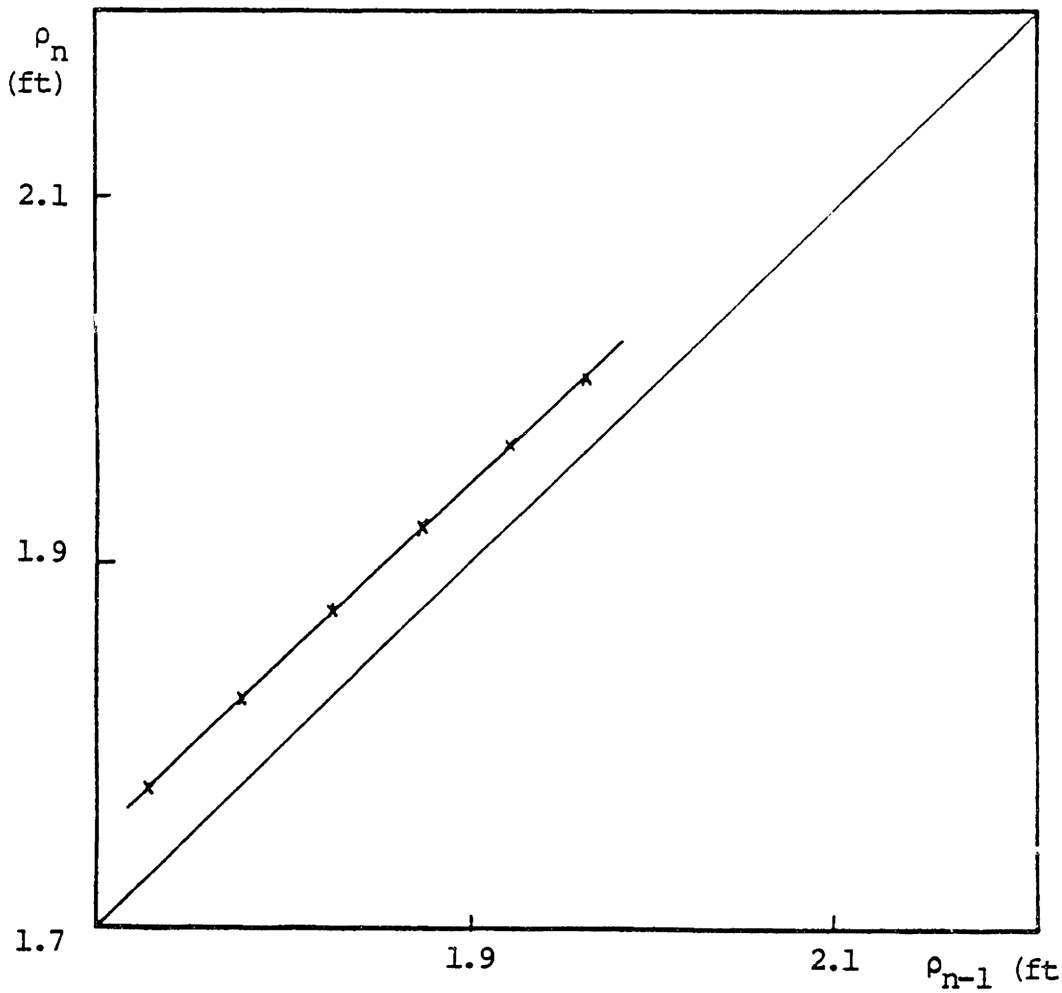
Legend:

⊙ measured points

⊡ back-calculated points from present analysis
$\rho_i = 0.6$ ft for t = 185 days

▽ back-calculated points using:
$\begin{cases} c_h = 0.088 \text{ ft}^2/\text{day} \\ \rho_\infty = 2.34 \text{ ft} \end{cases}$ (values from Sondex analysis)
$\rho_i = -0.93$ ft for t = 185 days

Fig. 4-23: Asaoka's Construction for Surface Settlement Data from Horizontal Inclinometer: South Section.



--Values used for Analysis:

t(days)	ρ (ft)
155	1.73
165	1.78
175	1.83
185	1.87
195	1.92
205	1.96
215	2.00

--Analysis:

Parameters	Results
$\rho_o = 0.12$	$c_h = 0.032$ ft^2/day
$\beta_1 = 0.957$	$\rho_\infty = 2.91$ ft
$r = 0.999$	

Vertical Drainage Effect: $\Delta c_h = 0.002$

CHAPTER 5

PRACTICAL RECOMMENDATIONS ON THE USE OF THE TECHNIQUES OF ANALYSIS

5-1. ANALYSIS OF SETTLEMENT DATA

The method as presented in Chapter 3 is summarized as follows:

1. Plot settlement versus time to a natural scale and fit a smooth curve through the data.

2. Select values of settlement at equal time intervals, Δt, from the smooth curve, denoted as the "settlement series."

3. Use the settlement series to plot Asaoka's construction as shown in Fig. 3-2.

4. Obtain values of the slope β_1 and the intercept ρ_o both by visually fitting a straight line through the points and by linear regression.

5. Calculate the values of the total settlement at the end of primary consolidation, ρ_∞, and the coefficient of consolidation for radial drainage, c_h, as follows:

* ρ_∞ from ρ_o and β_1 via

$$\rho_\infty = \rho_o \ /(1-\beta_1) \qquad \text{Eq. 3-2}$$

* c_h from for pure radial drainage

$$c_h = -[(d_e)^2 F(n)/8] \ [\ln(\beta_1)/\Delta t] \qquad \text{Eq. 3-3}$$

The following discussion explains the above steps.

a) Plot of the natural time-settlement curve.

The first required step of the method is to plot the settlement data versus time in order to obtain the settlement series. One should plot all the measured points and then fit these points with a smooth curve. From this curve, one selects the settlement series.

One should be careful in generating this curve because the analysis is quite sensitive to the accuracy of the plot. A visual procedure can usually be used for this purpose. In some cases it may be helpful to also use a plot of settlement versus log time to aid in deciding how best to fit the data with a smooth curve. A fair amount of settlement data covering a large range of the consolidation process is usually required in order to obtain a reasonably accurate settlement series for Asaoka's construction and hence values of c_h and ρ_∞.

The next step is to select the time interval in order to obtain the settlement series used in Asaoka's construction. However, before dealing with this issue, one needs to be familiar with possible deviation in Asaoka's "straight line".

b) Possible deviation of Asaoka's "straight line".

For field conditions which meet Terzaghi's original assumptions (e.g. constant load, compressibility, coefficient of consolidation, etc.), Asaoka's construction should theoretically generate a straight line for pure radial drainage or for pure vertical drainage when T_v is greater than 0.1. The actual curve may however, deviate from the straight line in the following cases:

1) As seen in Fig. 3-4, settlement data from cases having combined drainage should theoretically generate a line which is not straight (curve no. 2). This line has an initial upward curvature and does not become more or less linear until after the first points (curve no. 3). (Note: This line reflects the effect of a high ratio T_v / T_h equal to 0.2.)

2) If all or part of the clay layer is overconsolidated, the initial coefficient of consolidation should be higher than the value in the normally consolidated range due to the reduced compressibility during recompression. This generates points above the straight line as a higher value of c_h corresponds to a lower value of ρ_1 (see Fig. 5-1).

3) Toward the end of the consolidation process, secondary compression occurs. Thus, ρ_∞, will never be reached and the settlement points plot above the straight line in Asaoka's construction (see Fig. 5-1).

c) <u>Choice of the time interval.</u>

Asaoka (1979) recommends taking an interval of time of about one to three weeks for consolidation with drains and of about two to three months for consolidation without drains. As the writer only used the method for analysing consolidation with drains, what follows only deals with field cases having vertical drains.

The experience gained from the Mobile, Alabama, Alidrain case history leads to the choice of a small time interval. For example, the analysis of the Sondex surface settlement data for the North section (see section 4-3-a) used a time interval of ten days. In that example, the data obtained from Fig. 4-11 when plotted on Asaoka's construction (Fig. 4-12) yield a reasonable straight line both visually and from linear regression ($r=0.9997$). However, when one differentiates three zones on the line of the settlement points by visual observation of fluctuations, one finds values of c_h which differ by a ratio of up to three. These differences in the computed value of c_h would not have appeared so clearly if the time interval had been larger. This fact is shown in Fig. 5-2 which presents Asaoka's construction for the same data for a time interval of 30 days; no valid straight line can be reliably obtained from this graph.

Therefore, Asaoka's recommendation on the choice of the time interval seems appropriate in the case of consolidation with drains, i.e. one should use a relatively small time interval of from one to three weeks in order to properly define the line of Asaoka's construction. In addition, such a time interval should allow one to easily meet the requirement from Eq. 3-8 to correct for vertical drianage when combined drainage is important.

d) Determination of c_h and ρ_∞.

One should first eliminate visually those points which clearly do not lie near the straight line of Asaoka's construction. Next, one should obtain the values of ρ_o and β_1 by both visuallly fitting a straight line through the points and by doing a linear regression on the selected points. Both approaches should yield the same values. However, linear regression should be more accurate, whereas the visual approach insures that one does not blindly apply the regression without first seeing fluctuations about the presumed linear relationship.

e) Back-analysis.

A "back-analysis" can be performed using the values of c_h and ρ_∞ from the previous analysis. The resulting curve of settlement versus time is reconstructed using Eq. 3-9 as described in section 3-3-d. The determination of the initial settlement the only unknown, should be obtained as described in section 4-3-c. The time at which this value is determined should have a degree of consolidation higher than 60% because the accuracy of the calculation is better when the exponential term in the expression of $\bar{U}$ (see Eq. 2-3) has a low value.

However, the value of the initial settlement obtained has a doubtful physical meaning because:

1) This value is dependent on the time origin chosen.

2) The procedure cannot account for changes in the coefficient of consolidation, and hence yields

erroneous values of ρ_i if consolidation involves both recompression and virgin compression.

The curve obtained by the back-analysis should match the original curve in order to declare the analysis valid. However, during the early stages of the consolidation, if c_h has been decreasing, the curves will not match and at the end of the consolidation process, the back-calculated curve may not match the original curve due to secondary compression. Nevertheless, the value of total settlement at the end of primary consolidation, should be reasonable if the analysis is valid. That is, the magnitude of ρ_∞ should be reasonable when compared to the sum of the predicted values of the initial settlement and the final amount of primary consolidation settlement, i.e. $\rho_i + \rho_{cf}$.

f) Limitations.

First one should note that the value which is actually determined by this method is not the coefficient of consolidation, but the ratio $c_h/(d_e)^2F(n)$ for pure radial drainage or c_v/H_d^2 for pure vertical drainage. Therefore, any inaccuracy in the values of H_d, d_e or $F(n)$ will generate an error in the value of the coefficient of consolidation.

The method of analysis is also only valid for a homogeneous clay layer. If the layer actually contains sublayers with different soil parameters or if the deposit has some intermediate drainage layers, the method is not valid. In both of these cases, the method would only be valid if applied to each layer separately.

Furthermore, this method of analysis is only valid if the assumptions of Terzaghi and of Barron (see section 2-2) are valid. A "back-analysis" can be useful in checking the validity of these assumptions.

5-2. ANALYSIS OF EXCESS PORE PRESSURE DATA.

The method as presented in chapter 3 is summarized as follows:

1. Plot the data in terms of log u versus time.

2. If the resulting plot is visually linear, obtain α_o and α_1 by linear regression with the following relation:

$$\ln (u) = \alpha_o - \alpha_1 t$$

where $\alpha_1 = [\ln (u_1/u_2)]/(t_2 - t_1)$

3. Calculate:

$$c_h = [(d_e)^2 F(n)/8] \times \alpha_1$$

or

$$c_v = [4H_d^2/\pi^2) \times \alpha_1 \qquad \text{(only if } T_v > 0.1)$$

a) Determination of the coefficient of consolidation.

The above method is easy to follow and does not include any problems when the plot of log u versus time forms a reasonable straight line. It also allows one to obtain values of c_h versus time independently of the value of the location of the tip of the piezometer and of the value of the initial excess pore pressure. If such is not the case, one can select only those data which appear to have a linear ln(u) vs t relationship and/or use the conventional incremental time method described in section 3-2.

For consolidation without drains, one can also use Eqs. C-6 and C-14 as an incremental procedure. A "back-analysis" can also be performed to check the validity of the determination of the coefficient of consolidation as described in section 3-4-b and used in section 4-2-c. The value used at time t=0 is the term $\nu\bar{u}_o$ obtained from α_o. One cannot back-calculate the value of $\bar{u}_o$ because one does not know the exact value of the location factor. Furthermore, the value of $\nu\bar{u}_o$ is highly dependent on the time origin.

b) Limitations of the method.

First the possible deviations from a linear relation between the logarithm of the excess pore pressure and time are considered. This is the case when:

1. The effect of vertical drainage is important for consolidation with drains (see section 2-4). One should be aware that thin sand lines may be located within the clay layer, generating unexpected vertical drainage at certain locations. For example, Figure 12 of Ladd et al. (1972) showed very high values of c_h from piezometers that were located near an unexpected partial drainage layer located in the middle of a clay deposit.

2. The coefficient of consolidation decreases during the early stages of the consolidation process. The points corresponding to this period will plot above the straight line because they will correspond to a higher c_h and therefore a higher value of α_1 .

3. The equilibirum of the excess pore pressure is not reached just after the end of the loading period.

4. The values of excess pore pressure are incorrect due to errors in the initial preconstruction values and/or changes in the final equilibrium values.

5. The piezometer becomes inoperative.

6. The load is not kept constant.

As noted for settlement data, one should be aware that the value which is actually determined by this method is not the coefficient of consolidation, but the ratio $c_h/(d_e)^2F(n)$ for pure horizontal drainage and c_v/H_d^2 for pure vertical drainage.

This method is also only valid for a homogeneous clay layer, i.e. soil parameters constant over depth with no sand layers within the clay and for clay deposits which meet the assumptions of Terzaghi and of Barron (see section 2-2).

5-3. SUMMARY AND CONCLUSIONS

Some recommendations for the use of Asaoka's method of analysis of settlement data are first given. The main ones are:

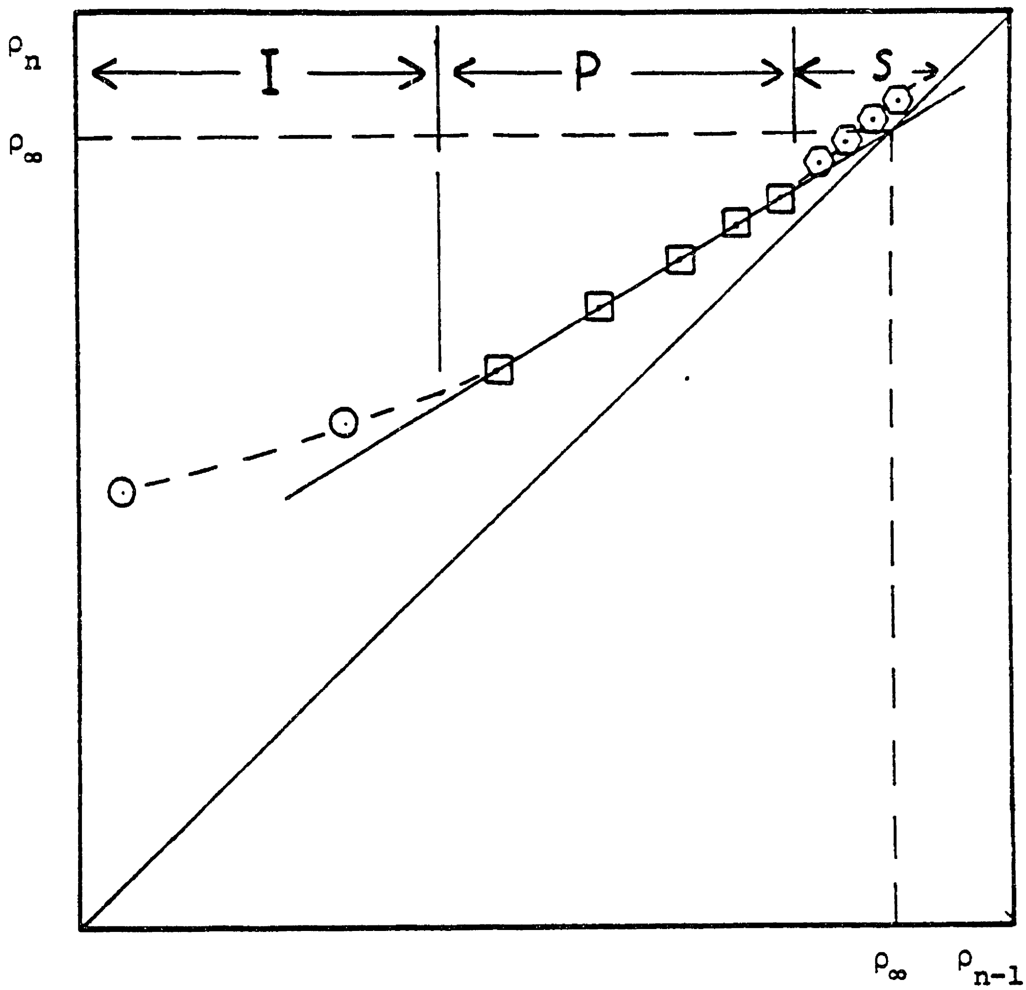
* To plot all the settlement data versus time on a graph and then fit the points with a smooth curve.

* To define the settlement series from this curve with a time interval one to three weeks for consolidation with drains as recommended by Asaoka. For consolidation without drains, Asaoka recommends using two to three months.

Limitations of the method are also listed; the main one is that the assumptions of Terzaghi and of Barron for consolidation should be valid.

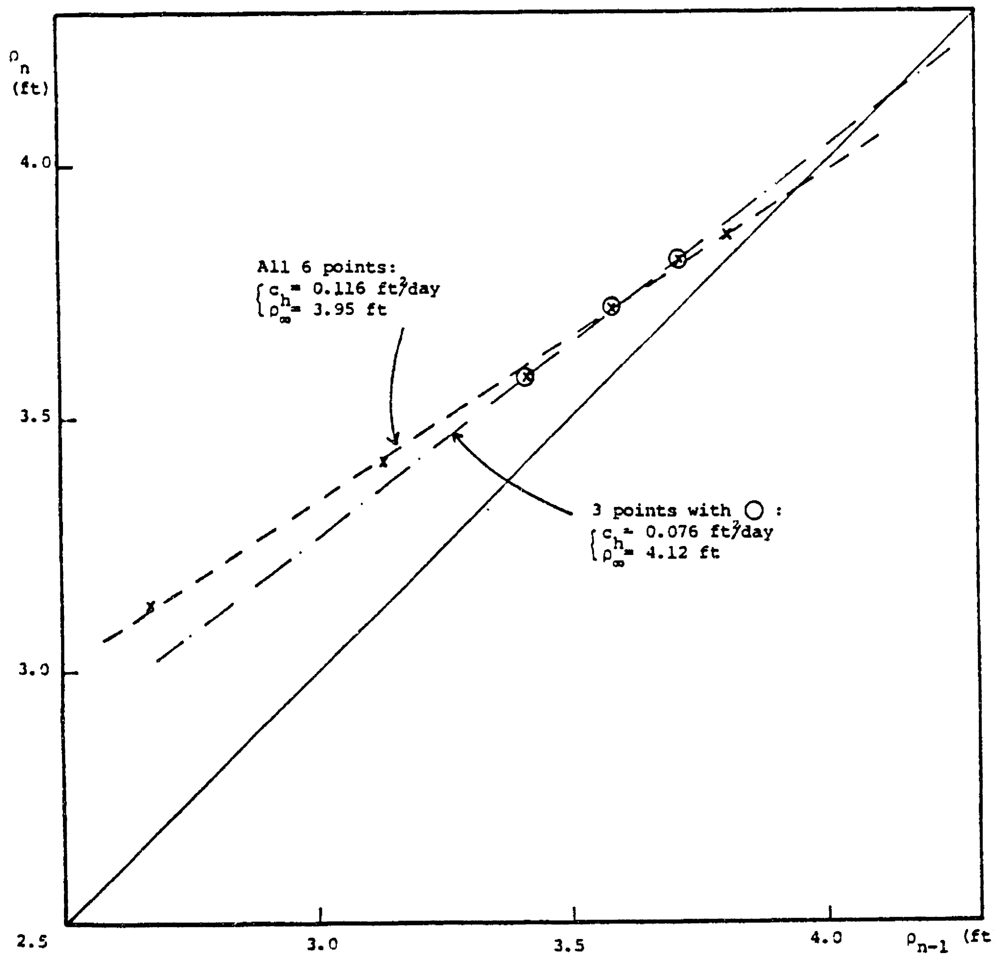
The method of analysis for excess pore pressure data is then discussed. It is recommended to plot the values of log u versus time in order to visually assess the linearity of the relation. If this is the case, one can use the slope of this line to compute c_h . If not, one should use the conventional method or, if the curved trend of the relation between log u and time is not due to vertical drainage, one can use the relation between c_h and ln (u_1/u_2) presented in Appendix C, to obtain c with an incremental procedure. The possible deviations from a linear relation between ln u and time are discussed. Finally, limitations of the method are stated with the main one still being that the assumptions of Terzaghi and of Barron should be valid.

Fig. 5-1: Possible Deviations from Asaoka's Construction.



Zone	Symbol	State
I	⊙	Initial Stage of Consolidation (c_h or c_v decreasing)
P	⊡	Primary consolidation with constant c_h or c_v
S	⬡ (hexagon with dot)	Secondary Compression

Fig. 5-2: Asaoka's Construction for North Sondex Surface Settlement Data from the Case History on Alidrains with a Time Interval Δt= 30 days.



NOTE: Δt = 30 days.
t_{start} = 30 days.

CHAPTER 6

SUMMARY AND CONCLUSIONS

The objective of this thesis was to present new techniques to evaluate field performance of vertical drains. Before dealing with this issue, the theory of consolidation with vertical drains was presented.

a) Review of the theory of consolidation with vertical drains.

First the theory for pure radial drainage was briefly introduced followed by a detailed study of the combined drainage condition. The influence of vertical drainage is found to vary from being insignificant regarding rates of pore pressure dissipation for piezometers located within the central portion of thick clay layers to having a very significant effect on rates of surface settlement when the ratio T_v/T_h is greater than 0.1.

The effect of well resistance was then discussed. A new criteria as to when this effect can be neglected was proposed, namely that W_R should be less than 0.1. W_R is defined as follows:

$$W_R = 2\pi k_h \ell^2/q_w \qquad \text{(Eq. 2-10)}$$

where k_h = coefficient of horizontal permeability for the clay

ℓ = maximum drainage length of the drain

q_w = discharge capacity of the drain for unit hydraulic gradient

Good quality wick drains and sand drains were thus shown to produce no effect of well reisitance on the consolidation process.

The effect of smear was then evaluated with the conclusion that it is a very important factor. Unfortunately it is difficult to include this effect in an analysis due to the uncertainties in the determination of the size and permeability of the soil within the remolded zone which generates this effect.

A brief discussion of the problem of the time dependent loading was also included.

b) Techniques to evaluate field performance of vertical drains.

The conventional methods of analysis were first presented. Next, the new methods to analyse settlement and excess pore pressure data were detailed.

b-1. Analysis of settlement data.

Based on Asaoka's (1978) observational procedure of settlement data, the analysis of these data for both pure radial and vertical drainage is derived from the following linear relation for a constant coefficient of consolidation:

$$\rho_n = \rho_o + \beta_1 \rho_{n-1} \qquad \text{(Eq. 3-1)}$$

where: ρ_n = settlement at time t_n from series where settlement points are spaced at a constant time interval Δt.

ρ_{n-1} = settlement at time $t_n - \Delta t$.

ρ_o = intercept shown in Fig. 3-2

β_1 = slope shown in Fig. 3-2

This relation is valid for pure radial drainage throughout consolidation but for pure vertical drainage only when the time factor T_v is higher than 0.1.

The intersection of the straight line on Asaoka's construction (see Fig. 3-2) with the 45^0-degree line determines the convergence point of the series, ρ_∞ , which represents the total settlement at the end of primary consolidation. It also can be computed from the relationship:

$$\rho_\infty = \rho_o / (1-\beta_1) \qquad \text{(Eq. 3-2)}$$

The coefficients of consolidation are determined using Eqs. 3-3 and 3-4 for pure radial drainage and pure vertical drainage respectively:

$$c_h = -[(d_e)^2 F(n)/8] [\ln(\beta_1)/ \Delta t] \qquad \text{(Eq. 3-3)}$$

$$c_v = -[4H_d^2/ \pi^2] [\ln(\beta_1)/ \Delta t] \qquad \text{(Eq. 3-4)}$$

where d_e = equivalent diameter of the drain

H_d = drainage height

$F(n) = \ln(n) - 0.75$

n = spacing ratio = d_e/d_w

This last relation is only valid for a time factor T_v higher than 0.1. The main advantage of these two formulations is that they do not require predictions of the initial settlement or of the final settlement at the end of primary consolidation.

Combined drainage for cases with vertical drains is treated as a deviation from the case of pure radial drainage. A method of correction for the effect of vertical drainage was presented in order to obtain the actual value of the coefficient of consolidation for horizontal drainage. It requires making an estimate of c_v for the soil.

Practical recommendations for the use of the new method of analysis are given in Chapter 5. The main ones are:

* To plot all the settlement data versus time on a graph before fitting these points with a smooth curve.

* To define the settlement series from this

curve with a time interval between one to three weeks for consolidation with drains, as recommended by Asaoka. For consolidation without drains, Asaoka recommends using two to three months.

b-2. Analysis of excess pore pressure data.

The analysis of these data for both pure radial and pure vertical drainage is derived from the linear relation expressed in Eq. 3-10, assuming that the coefficient of consolidation is constant:

$$\ln (u) = \alpha_o - \alpha_1 t \qquad \text{(Eq. 3-10)}$$

The α_o term reflects the piezometer location and initial excess pore pressure and α_1 equals the slope of the change in ln u with time, i.e. $\alpha_1 = \ln(u_1/u_2)/(t_2 - t_1)$. Both of the constants can be obtained by linear regression, but only after visually checking that Fig. 3-10 reasonably represents the field data. The relation is only valid for $T_v > 0.1$ in the case of pure vertical drainage.

The coefficients of consolidation are obtained from the constant α_1 using the following relations:

$$c_h = [(d_e)^2 F(n)/8]\alpha_1 \qquad \text{(Eq. 3-11)}$$

or

$$c_v = [4H_d^2/\pi^2]\alpha_1 \qquad \text{(Eq. 3-14)}$$

The first advantage of this method is that it does not require a prediction of the initial excess pore pressure. The second one is that, for pure radial drainage, the computed coefficient of consolidation should be independent of the location of the tip of the piezometer between vertical drains, and for pure vertical drainage, it is independent of the depth of the tip of the piezometer (but again, T_v must be greater than 0.1).

As seen in the review of the theory of consolidation with vertical drains, the combined drainage conditon is often such that the values of excess pore pressure for piezometers deep in the layer should not be affected by vertical drainage. When this is true, the method for pure radial drainage can be used in order to evaluate the coefficient of consolidation; otherwise, use of the conventional method is recommended.

Other conditions which would produce a curved line on the plot of log u and time are discussed in section 5-2. In these cases, the method of analysis is not valid and it is recommended to use either the conventional incremental time method or only those portions of the ln u versus time data which appear to be reasonably linear. This approach can probably handle changes in c_h or c_v with time, but cannot resolve problems caused by erroneous values of excess pore pressure or the presence of undetected intermediate drainage layers.

b-3. Back-analysis.

To better evaluate the results of the analysis, a "back-analysis" is recommended. That is to back-calculate the original curve of measured values versus time. The analysis is considered reasonable if the back-calculated curve matches the measured one. This technique also helps to identify cases where the basic assumptions of Terzaghi (or Barron) are not met in the field.

c) The case history involving Alidrains.

These new methods were used in a case history for a precompression project with Alidrains for a cement plant located in Mobile, Alabama. (See Figs. 4-1 through 4-4 for location plan, typical cross-section, etc.). This project had already been studied in a previous thesis (Noiray, 1982) thus allowing comparison of the results with those obtained using "conventional" methods.

The results from both analyses are summarized in Table 6-1. Both studies are in agreement with the analyses of the excess pore pressure and settlement data when the analyses are performed during similar periods of time. Additional settlement data allowed the author to analyse settlement data for a longer period of time; a lower value of c_h was found. Such behavior could be reasonable if the clay layer became just normally consolidated at the beginning of the period studied since one would expect a higher value of c_h

during the early portion of virgin compression.

The analysis of surface settlement data from horizontal inclinometers yields a mean value of c_h which is in agreement with the results from the analysis of the excess pore pressure data. Though the amount of data used was insufficient for two section to be sure of the determination of c_h, the value of c_h obtained may better reflect the actual in situ coefficient of consolidation because, for this analysis, the predicted values of the total settlement at the end of primary consolidation seem more correct. This would mean that, for unclear reasons, the Sondex system may have overpredicted the actual rate of surface settlements.

d) Conclusion.

The new techniques of analysis which have been presented are certainly interesting from a theoretical standpoint and should also provide improved techniques for evaluating field performance data compared to the "conventional" methods. However, these techniques have been established assuming a uniform soil profile and constant compressibility-consolidation properties during the time span of interest. One should always be aware of these limitations which are the same as for Terzaghi's theory for one-dimensional consolidation.

Finally it would be instructive to further study the limitation of these new techniques by applying them to computer generated time-settlement pore pressure data using a soil model with correcting accounts for variations in compressiblity, permeablilty and the coefficient of consolidation with time.

TABLE 6-1

SUMMARY OF COEFFICIENTS OF CONSOLIDATION DATA FROM ANALYSIS OF SETTLEMENT AND PIEZOMETER DATA FOR CASE HISTORY WITH ALIDRAINS (for Normally Consolidated range)

TYPE OF DATA	NUMBER/ LOCATION	TIME* INTERVAL (day)	c_h or c_v		REMARKS
			MEAN	SD	
Laboratory Consolidation Tests	26 at N, M and S	----	0.019	0.01	By Noiray
Piezometers	14 at N,	100 - 180	0.04	0.01	By Noiray
	M and S + offsets (see Table 4-1)	100 - 180	0.038	0.006	
Sondex Surface Settlement	3 at N,	75 - 170	0.13	0.03	By Noiray
	M and S	110 - 170	0.119	0.014	
		110 - 250	0.091	0.014	
Inclinometer Surface Settlement	N and S	160 - 220	0.041	0.007	
	M	180 - 340	0.042		

* End of loading at t = 75 days and load was probably increased after t = 370 days.

All values of c_h or c_v are in ft^2/day.

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NOTE: ASCE = American Society of Civil Engineers
JSMFD = Journal of the Soil Mechanics and Foundations Division
JGED = Journal of the Geotechnical Engineering Division

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APPENDIX A

DERIVATION OF THE RELATION TO ASSESS THE INFLUENCE OF WELL RESISTANCE

The following is based on a paper by Hansbo (1981) on "Consolidation of Fine-Grained Soils by Prefabricated Drains", which presents Kjellman's solution for the ideal drain and introduces Yoshikuni's (1974) boundary condition to account for the effect of well resistance.

The model used to represent the process is shown in Fig. 2-1, that is a circular soil cylinder with impervious boundaries except for the central circular drain. The following assumptions are used:

* Axial load on the cylinder.
* Soil is saturated with water.
* No horizontal strains.
* Darcy's law is valid: $v_r = k_h i_r$, at a distance r, from the centerline,

  where: i_r = the hydraulic gradient
  k_h = the horizontal coefficient of permeability
  v_r = the velocity of the flow in the radial direction

  This equation can be rewritten as:

$$v_r = \frac{kh}{\gamma w} \frac{\partial u}{\partial r} \qquad (1)$$

  where: u = the excess pore pressure at r
  γ_w = the volume weight of water

The outflow from the hollow cylinder of inside radius, r, and outside radius r_e, has to be equal to the change of volume of the hollow cylinder, which gives:

$$2\pi r v_r = \pi (r_e^2 - r^2) \frac{\partial \varepsilon}{\partial t} \quad (2)$$

where: $\frac{\partial \varepsilon}{\partial t}$ = the rate of strain in vertical direction.

(1) and (2) give Kjellman's governing equation in the soil:

$$\frac{\partial u}{\partial r} = \frac{\gamma_w}{2k_h} \left(\frac{r_e^2}{r} - r \right) \frac{\partial \varepsilon}{\partial t} \quad (3)$$

If k_w is the vertical coefficient of permeability of the drain and if one considers a horizontal cross-sectional slice of dz of the cylindrical drain, the horizontal rate of inflow of water into the slice is:

$$dQ_1 = \frac{r_w k_w}{\gamma_w} \left(\frac{\partial u}{\partial r} \right)_{r=r_w} dz$$

The total change in the rate of flow from the entrance face to the exit face of the slice is:

$$dQ_2 = \frac{\pi r_w^2 k_w}{\gamma_w} \left(\frac{\partial^2 u}{\partial z^2} \right)_{r=r_w} dz$$

if u is assumed constant in a cross-section of the drain. It is required to have:

$$|dQ_1| = |dQ_2|$$

which leads to Yoshikuni's boundary condition at the drain interface:

$$\left(\frac{\partial u}{\partial r} \right)_{r=r_w} + \frac{r_w}{2} \frac{k_w}{k_h} \left(\frac{\partial^2 u}{\partial z^2} \right)_{r=r_w} = 0 \quad (4)$$

Hansbo combines (3) and (4) and obtains:

$$\left.\frac{\partial^2 u}{\partial z^2}\right|_{r_w} = -\frac{\gamma_w}{k_w}(n^2-1)\left.\frac{\partial \varepsilon}{\partial t}\right|_{r_w} \qquad \text{(in Hansbo's paper, Eq.11)}$$

Then, Hansbo integrates twice u over depth considering the rate of strain constant with depth. But if one cannot neglect well resistance, the rate of strain will vary with depth. Therefore, the integration is incorrect. In fact, (3) and (4) represent the governing differential system:

$$\frac{\partial u}{\partial r} = \frac{\gamma_w}{2k_h}\left(\frac{r_e^2}{r} - r\right)\frac{\partial \varepsilon}{\partial t} . \tag{3}$$

$$\left.\frac{\partial u}{\partial r}\right|_{r_w} + \frac{r_w}{2}\frac{k_w}{k_h}\left(\frac{\partial^2 u}{\partial z^2}\right)_{r=r_w} = 0 \tag{4}$$

If the parameters are normalized with $\rho = r/r_w$ and $Z = z/\ell$, the system becomes:

$$\frac{\partial u}{\partial \rho} = \frac{\gamma_w}{2k_h} r_w^2 \left(\frac{n^2}{\rho} - \rho\right)\frac{\partial \varepsilon}{\partial t} \tag{A-1}$$

$$\left.\frac{\partial^2 u}{\partial z^2}\right|_{\rho=1} + 2\frac{k_h}{k_w}\left(\frac{\ell}{r_w}\right)^2 \left.\frac{\partial u}{\partial \rho}\right|_{\rho=1} = 0 \tag{A-2}$$

where ℓ = the "characteristic" length of the drain equal to either half the length of the drain for fully penetrating drains and bottom drainage,or the length of the drain otherwise.

Equation A-1 is the soil governing equation. Equation A-2 is the boundary condition, with the second term of the left side of the equation representing the effect of well resistance. Therefore, if W_R is defined as:

$$W_R = 2\frac{k_h}{k_w}\left(\frac{\ell}{r_w}\right)^2 = 8\frac{k_h}{k_w}\left(\frac{\ell}{d_w}\right)^2 \tag{A-3}$$,

the effect of well resistance is said to be negligible as a first order of approximation if:

$$W_R < 0.1 \qquad \text{(A-4)}$$

As the vertical coefficient of permeability in the drain is related to q_w, the vertical discharge capacity of the drain at a hydraulic gradient in the drain $i_w = 1$ in the vertical direction, by the relation:

$$q_w = \pi r_w^2 \, k_w \, ,$$

W_R, can also be expressed as follows:

$$W_R = \frac{2\pi k_h \ell^2}{q_w} \qquad \text{(A-5)}$$

Yoshikuni (1974) has the following factor, L, to represent the effect of well resistance:

$$L = \frac{32}{\pi^2} \frac{k_h}{k_w} \left(\frac{H_d}{d_w} \right)^2 = \frac{4}{\pi^2} W_R \left(\frac{H_d}{\ell} \right)^2 \qquad \text{(A-6)}$$

$4/\pi^2$ is a constant introduced in the calculation, but should not be considered as the term corresponding to the effect of vertical drainage for c_v and c_h equal. H_d and ℓ are equal when the drains fully penetrate the clay layer.

If W_R is only slightly higher than 0.1, the effect of well resistance is very small and therefore one can exclude it in the analysis of the drains. If W_R is significantly higher than 0.1, one may want to consider the influence of well resistance by using Hansbo's solution which, though being questionable, will give an approximate representation of the phenomenon. Following Hansbo's development, the term F(n) in Eq. 2-2 becomes:

$$\mu_r \cong F(n) + \pi z \; (2\ell - z) \; k_h/q_w \qquad \text{(A-7)}$$

where: z = depth in the clay layer

One can re-write Eq. A-7 into:

$$\mu_r \cong F(n) + Z \; (1-Z/2) W_R \qquad \text{(A-8)}$$

where: $Z = z/\ell$, depth factor relative to the "characteristric" length of the drain.

It should be noted that W_R is a function of Z because the discharge capacity is a function of the lateral stress around the drain at a depth Z.

Note that this term μ_r instead of F(n) in the expression of the degree of consolidation, U_h, will no longer be constant with depth. One should also remember that this expression was established only for pure radial drainage. If a better estimate to account for the effect of well resistance is required, one can develop a numerical solution with Eqs. A-1 and A-2. The boundary condition for Eq. A-2 is:

$$\left|\begin{array}{l} Z = 0 \\ u\big|_{\rho=1} = 0 \end{array}\right. \qquad \text{and} \qquad \left|\begin{array}{l} Z = 1 \\ \frac{\partial u}{\partial Z}\Big|_{\rho=1} = 0 \end{array}\right.$$

APPENDIX B

THEORETICAL CONSIDERATIONS UPON THE ANALYSIS OF SETTLEMENT DATA

This method is based on Asaoka's (1978) approach to determine the total settlement at the end of primary consolidation, and the coefficient of consolidation. This method can be used to analyse the performance of the drains and to predict the time and magnitude of the settlement at the end of primary during consolidation. The method of analysis for pure radial drainage will be presented first, followed by the case for consolidation for vertical drainage only and finally the case for combined drainage.

B-1. RADIAL DRAINAGE ONLY

From the time settlement curve, one can select a series of settlement values [$\rho_1, \rho_2, \ldots \rho_n, \ldots$] such that ρ_n is the settlement at time t_n, and that $(t_n - t_{n-1})$ is constant and equal to Δt. From this series, one then plots the settlement values ρ_{n-1} at time t_{n-1} on the x-axis and the settlement values ρ_n at time t_n on the y-axis (see Fig. 3-2). To prove that these points plot on a straight line and that this line intersects the 45°-degree line at ρ_∞, the total settlement at the end of primary consolidation, it is sufficient to prove that any straight line between one of the points (ρ_{n-1}, ρ_n) and the point $(\rho_\infty, \rho_\infty)$ has the same slope β_1.

This means that:

$$\beta_1 = \frac{\rho_\infty - \rho_n}{\rho_\infty - \rho_{n-1}} \qquad \text{(Eq. B-1)}$$

or

$$\beta_1 = \frac{\rho_{cf} - (\rho_n - \rho_i)}{\rho_{cf} - (\rho_{n-1} - \rho_i)}$$

or

$$\beta_1 = \frac{1 - [(\rho_n - \rho_i)/\rho_{cf}]}{1 - [(\rho_{n-1} - \rho_i)/\rho_{cf}]}$$

where: ρ_i = initial settlement

ρ_{cf} = primary consolidation settlement

Since $\bar{U}_{n-1}$ and $\bar{U}_n$ are the average degrees of consolidation from settlement at times t_{n-1} and t_n, then:

$$\beta_1 = \frac{1 - \bar{U}_n}{1 - \bar{U}_{n-1}} \qquad \text{(Eq. B-2)}$$

From Barron's (1948) theory for the Equal strain case, assuming pure radial drainage:

$$1 - \bar{U} = e^{-\frac{8c_h}{(d_e)^2 F(n)} t}$$

Therefore:

$$\beta_1 = e^{-\frac{8c_h}{(d_e)^2 F(n)} (t_n - t_{n-1})}$$

or

$$\beta_1 = e^{-\frac{8c_h}{(d_e)^2 F(n)} \Delta t}$$

β_1 is a constant independent of which point (ρ_{n-1}, ρ_n) is selected. This proves that all the points (ρ_{n-1}, ρ_n) lay on a straight line of slope β_1 and that the point $(\rho_\infty, \rho_\infty)$ is on this straight line. This last point is also the convergence point of the series and therefore represents the total settlement at the end of primary consolidation.

The value of the coefficient of consolidation can therefore be computed as follows:

$$c_h = -\frac{(d_e)^2 F(n)}{8} \frac{\ln \beta_1}{\Delta t} \quad \text{(Eq. B-3)}$$

The following relation also applies:

$$\rho_n = \rho_o + \beta_1 \rho_{n-1} \quad \text{(Eq. B-4)}$$

$$\rho_\infty = \frac{\rho_o}{1-\beta_1} \quad \text{(Eq. B-5)}$$

Equations B-3, B-4 and B-5 are therefore proven to be exact, following Terzaghi's assumptions relative to one-dimensional consolidation and assuming pure radial drainage for consolidation with vertical drains.

B-2. VERTICAL DRAINAGE ONLY

Asaoka (1978) also presented a method for analysis of settlement data for the consolidation of a clay layer by vertical drainage. Asaoka used Terzaghi's assumptions and Mikasa's (1963) equation relative to one-dimensional consolidation:

$$\partial \varepsilon / \partial t = c_v \frac{\partial^2 \varepsilon}{\partial t^2}$$

Asakoa's development yielded an expression for the degree of consolidation which is:

$$\overline{U}_A = 1 - e^{-6T_v}$$

This expression for the degree of consolidation is compared in Table B-1 to the values found using the exact solution of Terzaghi (Eq. 2-7) and the approximate solution (Eq. 2-7a). One can see that Asaoka's relation does not agree with Terzaghi's solution.

Therefore, the writer used the approximate expression for the degree of consolidation (Eq. 2-7) to describe the consolidation process. This equation, which is valid for a time factor T_v higher than 0.1, states:

$$\bar{U} = 1 - (8/\pi^2)e^{-\frac{\pi^2}{4}T_v}$$

With this equation, the same procedure as the one for consolidation for pure radial drainage can be used and the expression of the coefficient of consolidation, c_v, becomes, as presented by Magnan and Deroy (1980):

$$c_v = -(4H_d^2/\pi^2)(\ln\beta/\Delta t) \qquad \text{(Eq. B-6)}$$

This formulation is only valid for T_v higher than 0.1 due to the limitation of (Eq. 2-7) which is only an approximation of the exact solution.

B-3. COMBINED VERTICAL AND RADIAL DRAINAGE

Although a method to analyse the consolidation process for both types of drainage has been presented, one cannot combine them to analyse the consolidation process for combined radial and vertical drainage. The problem lies in the fact that the degree of consolidation due to vertical drainage in most cases involving use of vertical drains will be small. Hence T_v is likely to be less than 0.1 and therefore Eq. 2-7 becomes invalid. Consequently, as shown in Fig. 3-4, the method to analyse settlement data for pure radial drainage is first used to determine a coefficient of consolidation, c_{hv}, which includes both effects. If one assumes a coefficient of consolidation for vertical drainage, c_v, the value of c_{hv} is then corrected to obtain the actual c_h.

This correction procedure is based on the assumption that vertical drainage does not affect the value of the total settlement at the end of primary consolidation, ρ_∞. Defining β_1 as in section B-1 i.e. the slope of a straight line between a point (ρ_{n-1}, ρ_n) and the point $(\rho_\infty, \rho_\infty)$, then $\beta_{1_{hv}}$ is the value of β_1 for the settlement values ρ_{n-1} and ρ_n due to combined drainage and β_{1h} is the value of β_1 which would correspond to the values of ρ_{n-1} and ρ_n if only pure radial drainage occurs. From Eq. B-2 one can derive the following expressions using Eq. 2-8:

$$\beta_{1_{hv}} = (1-\overline{U}_n)/(1-\overline{U}_{n-1}) = [(1-\overline{U}_{vn})(1-\overline{U}_{hn})]/[(1-\overline{U}_{v(n-1)})(1-\overline{U}_{h(n-1)})]$$

$$\beta_{1_{hv}} = \left[(1-\sqrt{\frac{4c_v}{\pi H_d^2}t_n})/(1-\sqrt{\frac{4c_v}{\pi H_d^2}t_{n-1}})\right]\beta_{1_h} \qquad \text{(Eq. B-7)}$$

Defining $\alpha = \sqrt{\frac{4c_v}{\pi H_d^2}}$,

$$\beta_{1_{hv}} = \frac{(1-\alpha\sqrt{t_{n-1}+\Delta t})}{(1-\alpha\sqrt{t_{n-1}})}\ \beta_{1_h}$$

or

$$\beta_{1_{hv}} = \frac{[1-\alpha\sqrt{t_{n-1}}(1+\Delta t/t_{n-1})^{\frac{1}{2}}]}{(1-\alpha\sqrt{t_{n-1}})}\ \beta_{1_h}$$

If $\Delta t << t_{n-1}$ (i.e. $t_{n-1} > 10 x \Delta t$), $(1 + \Delta t/t_{n-1})^{\frac{1}{2}} \simeq (1 + \Delta t/2t_{n-1})$

So, $$\beta_{1_{hv}} = \frac{(1-\alpha\sqrt{t_{n-1}} - \frac{\alpha}{2}\frac{\Delta t}{\sqrt{t_{n-1}}})}{(1-\alpha\sqrt{t_{n-1}})}\ \beta_{1_h}$$

or

$$\beta_{1_{hv}} = [1- \frac{\alpha}{2}\ (\frac{\Delta t}{\sqrt{t_{n-1}}} \times \frac{1}{1-\alpha\sqrt{t_{n-1}}})]\ \beta_{1_h}$$

This leads to:

$$\beta_{1_h} = \beta_{1_{hv}}/A \qquad \text{(Eq. B-8)}$$

where: $A = 1 - \frac{\alpha}{2}\frac{\Delta t}{\sqrt{t_{n-1}}}\ \frac{1}{1-\alpha\sqrt{t_{n-1}}}$ (Eq. B-9)

These two expressions are valid approximations when the time interval Δt is less than $(0.1 \times t_{n-1})$.

Equations B-8 and B-9 can be applied to all the points which are used to define the straight line in order to obtain the value of $c_{h_{hv}}$. To correct for the effect of vertical drainage, one can use the following relations derived from Eqs. B-8 and B-9:

$$\Delta c_h = c_{h_{hv}} - c_{h_h}$$

$$\Delta c_h = [(d_e^2)F(n)/8 \times \ln(A)/\Delta t] \qquad \text{(Eq. B-10)}$$

where: $$A = 1 - \frac{\alpha}{2} \frac{\Delta t}{\sqrt{t_n}} \frac{1}{(1-\alpha\sqrt{t_m})} \qquad \text{(Eq. B-11)}$$

t_m = mean time t_{n-1} of the points used to define the straight line

$t_m > 10 \times \Delta t$

Using the mean time t_m is less restrictive than a criteria bases on values of t_{n-1} at early times. Moreover, as illustrated in Table 3-1, the proposed correction appears to work reasonably well even when t_m is less than 10 times Δt. This point will be further discussed in Chapter 5.

TABLE B-1

COMPARISON BETWEEN ASAOKA'S AND TERZAGHI'S DEGREE OF CONSOLIDATION FOR CONSOLIDATION WITH PURE VERTICAL DRAINAGE

Terzaghi (approximate formula): $\bar{U}_v = 1 - (8/\pi^2)\, e^{-\frac{\pi^2}{4} T_v}$ (for $T_v > 0.1$)

Asaoka: $\bar{U}_A = 1 - e^{-6T_v}$

T_v	0.0	0.05	0.1	0.2	0.3	0.4	0.5
$\bar{U}_v$	0.189	0.284	0.367	0.505	0.613	0.698	0.764
$\bar{U}_A$	0.000	0.259	0.451	0.699	0.835	0.909	0.950
Actual	0.00	0.25	0.356	0.504	0.613	0.697	0.764

APPENDIX C

THEORETICAL CONSIDERATIONS

UPON THE ANALYSIS OF PORE PRESSURE DATA

C-1. RADIAL DRAINAGE ONLY

a) Determination of c_h.

From Barron's (1948) Equal Strain condition theory for pure radial drainage, the excess pore pressure distribution around the drain is expressed by the following relationship:

$$\frac{u}{\bar{u}_o} = \frac{1}{r_e^2} \frac{1}{F(n)} [r_e^2 \ln(\frac{r}{r_w}) - \frac{r^2 - r_w^2}{2}] e^{-\frac{8c_h}{(d_e)^2 F(n)} t} \qquad \text{(Eq. C-1)}$$

One can define the term in front of the exponential as the location factor, ν. This term is independent of time and only depends on the location of the tip of the piezometer and the spacing ratio. Therefore, Eq. C-1 becomes:

$$u = \bar{u}_o \nu \, e^{-\frac{8c_h}{(d_e)^2 F(n)} t} \qquad \text{(Eq. C-2)}$$

or

$$\ln(u) = \ln (\bar{u}_o \nu) - \frac{8c_h}{(d_e)^2 F(n)} t \qquad \text{(Eq. C-3)}$$

This shows that there is a linear relation between ln(u) and time. Eq. C-3 can be re-written as:

$$\ln (u) = \alpha_o - \alpha_1 t \qquad \text{(Eq. C-4)}$$

where α_0 and α_1 are two constants which can be determined by linear regression between ln(u) and t.

The coefficient of consolidation, c_h, can be determined as follows:

$$c_h = [(d_e)^2 \times F(n)/8] \times \alpha_1 \qquad \text{(Eq. C-5)}$$

where $\alpha_1 = [\ln(u_1/u_2)/(t_2 - t_1)]$

This evaluation of the coefficient of consolidation, c_h, is independent of the initial excess pore pressure and also of the location of the tip of the piezometer.

b) <u>Other determination of c_h.</u>

If, as for settlement data, an excess pore pressure series $[u_1, u_2, \ldots u_n, \ldots]$ is defined, with the values of excess pore pressure taken at a constant time interval Δt, one can derive the following relation valid for any n:

$$u_n = \beta u_{n-1} \qquad \text{(Eq. C-6)}$$

where: β = constant

Eqs. C-6 shows that if one plots the points (u_{n-1}, u_n) as is done for settlement data in Asaoka's construction, one will generate a straight line which has a slope, β, and which intersects the origin of the axis. This last point is the convergence point of the series defined above because the excess pore pressure is zero at the end of primary consolidation. The slope of this line is related to the coefficient of consolidation by the relation:

$$c_h = [(d_e)^2 xF(n)/8] x [\ln(\beta)/\Delta t] \qquad \text{(Eq. C-7)}$$

c) <u>Value of the Location Factor, ν_c, at the center of the drain pattern</u>.

If the tip of the piezometer is located at the center of the drain pattern, the pore pressure is measured at the radius r_e of the centerline of the drain. Therefore, the location factor becomes:

$$\nu_c = \nu = [\ln(n) + 1/(2n^2) - 1/2] \;/F(n)$$

Since F(n) can be approximated by the following relation:

$$F(n) = \ln(n) - 3/4,$$

an approximate value of the location factor, ν_c, can be found using the following relation:

$$\nu_c = [\ln(n) - 1/2]/[\ln(n) - 3/4] \qquad \text{(Eq. C-8)}$$

The following relation also applies for the centerline pore pressure:

$$u(t)/\bar{u}_0 = \nu_c[1 - \bar{U}_h(t)] \qquad \text{(Eq. C-9)}$$

d) Value of the initial excess pore pressure.

From Eqs. C-3 and C-4, one has the following relation:

$$\nu\, \overline{u}_0 = e^{\alpha_0} \qquad \text{(Eq. C-10)}$$

One can theoretically calculate the average initial excess pore pressure at any depth using the above relation, but, unfortunately, in practice, the value obtained is questionable because it is dependent on the determination of the origin of time and on the true location of the tip of the piezometer.

C-2. VERTICAL DRAINAGE ONLY

The expression for the excess pore pressure for consolidation for vertical drainage only is, for T_v higher that 0.1:

$$u/\overline{u}_0 = (4/\pi)\ \sin\ (\pi Z/2)\ e^{-\frac{\pi^2}{4}\frac{c_v}{H_d^2}t} \qquad \text{(Eq. 2-6a)}$$

As in the case of pure radial drainage, one can get the following relation:

$$\ln(u) = \ln[(4/\pi)\ \sin\ (\pi Z/2)\ \overline{u}_0] - (\pi^2/4)\ c_v t/H_d^2 \qquad \text{(Eq. C-11)}$$

Eq. C-12 can also be expressed in the same form as Eq. C-4:

$$\ln u = \alpha_0 - \alpha_1 t$$

The coefficients α_0 and α_1 can be obtained by a linear regression between ln(u) and t. Therefore, the coefficient of consolidation, c_v, can be obtained from the following relation:

$$c_v = (4\ H_d^2/\pi^2) \times \alpha_1 \qquad \text{(Eq. C-12)}$$

where $\alpha_1 = [\ln(u_1/u_2)/(t_2 - t_1)]$

This relation is independent of the depth of the piezometer but is only valid for T_v, higher that 0.1. The determination of α_0 also gives the following relation:

$$\bar{u}_0 \sin(\pi Z/2) = (\pi/4)\ e^{\alpha_0} \qquad \text{(Eq. C-13)}$$

As in the case of pure radial drainage, knowing the depth factor Z, one could then calculate the initial excess pore pressure. However, in practice this value depends on the accuracy of the origin of time.

The second method of determination of c_h for pure radial drainage is also valid for the determination of c_v for vertical drainage only. Eq. C-7 becomes:

$$c_v = [4\ H_d^2/\pi^2] \times [\ln(\beta)]/\Delta t] \qquad \text{(Eq. C-14)}$$

APPENDIX D

DATA FOR THE CASE HISTORY ON ALIDRAINS

Tables D-1 through D-5 present data relative to excess pore pressures. The values shown are the values of the piezometric water elevation at time t minus the initial piezometric elevation selected by Professor Ladd (see Noiray, p.133). The coefficient of consolidation c_h and the product $\nu\bar{u}_o$, equal to the location factor times the initial excess pore pressure, are calculated for each piezometer.

Table D-6 contains the values of the Sondex surface settlement obtained from the curves fitted through the measured data plotted in Figs. 4-11, 4-13 and 4-15.

TABLE D-1

EXCESS PORE PRESSURE DATA AND c_h VALUES (North-Centerline)

SP-2 (EL:-9.0ft)		SP-3 (EL:-9.9ft)		SP-5 (EL:-31.6ft)		SP-6 (EL:-30.7ft)	
t (days)	u (ft. of water)	t (days)	u (ft. of water)	t (days)	u (ft. of water)	t (days)	u (ft. of water)
100	27.5	100	25.5	100	14.3	100	19.7
110	26.26	110	24.89	110	13.46	110	16.27
120	24.9	120	23.0	120	12.8	120	17.7
140	22.8	140	21.1	140	11.8	140	15.6
152	21.95	152	20.4	152	11.41	152	15.02
160	21.1	160	19.3	160	11.0	160	14.4
180	18.8	180	17.8	180	10.4	180	12.6
191	18.25	191	16.89	191	9.71	191	12.26
c_h = 0.038		c_h = 0.038		c_h = 0.034		c_h = 0.043	
$\nu\bar{u}_0$ =43.35		$\nu\bar{u}_0$ =40.52		$\nu\bar{u}_0$ =21.04		$\nu\bar{u}_0$ =31.85	
r = 0.9982		r = 0.9973		r = 0.9953		r = 0.9602	

Notes: t since 7/29/81
r = coefficient of linear regression.
c_h values in ft^2/day.
$\nu\bar{u}_0$ in ft of water.

TABLE D-2

EXCESS PORE PRESSURE DATA AND c_h VALUES (Middle-Centerline)

SP-10 (EL:-11.4ft)		SP-12 (EL:-23.3ft)		SP-14 (EL:-30.6ft)	
t (days)	u (ft. of water)	t (days)	u (ft. of water)	t (days)	u (ft. of water)
100	32.3	90	19.36	100	24.07
110	31.14	110	17.46	110	23.67
120	30.3	130	15.6	120	22.5
140	28.2	152	14.69	140	20.8
152	27.42	170	13.6	152	19.94
160	26.3	191	12.96	160	19.2
180	24.8			180	17.8
191	23.72			191	17.17
c_h = 0.028		c_h = 0.034		c_h = 0.032	
$\nu\bar{u}_0$ = 45.22		$\nu\bar{u}_0$ = 27.37		$\nu\bar{u}_0$ = 35.77	
r = 0.9989		r = 0.9896		r = 0.9987	

Notes: t since 7/29/81
r = coefficient of linear regression.
c_h values in ft^2/day.
$\nu\bar{u}_0$ values in ft of water.

TABLE D-3

EXCESS PORE PRESSURE DATA AND c_h VALUES (South-Centerline)

SP-18 (EL:-9.1ft)		SP-20* (EL:-23.4ft)		SP-22 (EL:-31.1ft)		SP-23 (EL:-45.0ft)	
t (days)	u (ft. of water)	t (days)	u (ft. of water)	t (days)	u (ft. of water)	t (days)	u (ft. of water)
100	17.3	100	15.1	100	16.7	100	19.3
110	16.5	110	13.5	110	15.91	110	18.31
120	15.9	120	12.8	120	15.0	120	17.3
140	14.6	140	11.3	140	13.5	140	15.5
152	14.32	152	10.44	152	12.99	152	14.4
160	13.7	160	10.1	160	12.1	160	13.8
180	12.8	180	9.1	180	10.9	180	12.8
191	12.61	191	8.19	191	10.04		
c_h = 0.030 $\nu\bar{u}_0$ =24.43 r = 0.9953		c_h = 0.053 $\nu\bar{u}_0$ =27.65 r = 0.9960		c_h = 0.046 $\nu\bar{u}_0$ =29.20 r = 0.9976		c_h = 0.045 $\nu\bar{u}_0$ =32.80 r = 0.9975	

Notes: t since 7/29/81
r = coefficient of linear regression.
* SP-20 is not used in the analysis (see section 4-2-a)
c_h values in ft^2/day.
$\nu\bar{u}_0$ values in ft of water.

TABLE D-4

EXCESS PORE PRESSURE DATA AND c_h VALUE

(Centerline, Middlepoint between Middle and South Section)

P-20 (EL:-23.3ft)		P-23 (EL:-45.4ft)	
t (days)	u (ft. of water)	t (days)	u (ft. of water)
100	16.0	89	21.17
110	14.84	96	20.02
120	14.1	103	19.56
140	12.9	110	18.17
152	13.0	117	18.63
160	12.0	124	17.25
180	11.3	131	17.25
191	10.24	140	16.56
c_h = 0.038 $\nu\bar{u}_0$ =24.85 r = 0.9859		c_h = 0.043 $\nu\bar{u}_0$ =32.96 r = 0.9681	

Notes: t since 7/29/81
r = coefficient of linear regression.
c_h values in ft^2/day.
$\nu\bar{u}_0$ values in ft of water.

TABLE D-5

EXCESS PORE PRESSURE DATA AND c_h VALUE

(65 ft. East Centerline for the North, Middle and South Sections)

P-2 (EL:-9.9ft)		P-16 (EL:-30.9ft)		P-28* (EL:-32.1ft)	
t (days)	u (ft. of water)	t (days)	u (ft. of water)	t (days)	u (ft. of water)
82	20.81	82	14.76	82	12.36
89	21.27	89	12.68	89	12.13
110	18.04	110	10.83	110	10.51
117	16.66	117	11.07	131	8.9
124	15.74	124	10.14	140	8.21
131	15.74	131	10.14	145	8.21
140	15.04	140	9.68	152	8.21
145	14.35	145	8.99	159	7.98
159	13.89	152	9.22	163	7.19
163	12.83	159	8.99	170	7.65
170	12.60	163	7.95	177	7.65
177	12.14	170	8.41	184	7.41
184	11.67	177	7.95	191	6.95
191	10.98	184	8.18		
		191	7.72		
c_h = 0.051		c_h = 0.042		c_h = 0.047	
$\nu\bar{u}_0$ =35.20		$\nu\bar{u}_0$ =19.50		$\nu\bar{u}_0$ =19.31	
r = 0.9908		r = 0.9710		r = 0.9545	

Notes: t since 7/29/81
r = coefficient of linear regression.
* P-28 is not used in the analysis (see section 4-2-a)
c_h values in ft^2/day.
$\nu\bar{u}_0$ values in ft of water.

TABLE D-6

VALUES OF SONDEX SURFACE SETTLEMENT
FROM FITTED CURVES

t (days)	NORTH	MIDDLE	SOUTH
	ρ (ft)	ρ (ft)	ρ (ft)
110	3.133	2.111	1.589
120	3.241	2.225	1.669
130	3.331	2.321	1.738
140	3.414	2.402	1.80
150	3.476	2.478	1.85
160	3.532	2.548	1.894
170	3.582	2.612	1.934
180	3.629	2.686	1.972
190	3.672	2.75	2.01
200	3.718	2.807	2.043
210	3.752	2.857	2.072
220	3.787	2.896	2.10
230	3.811	2.929	2.128
240	3.832	2.96	2.15
250	3.849	2.988	2.17

NOTE: t since 7/29/81

APPENDIX E

COMPARISON OF THE DETERMINATION OF c_v FOR A CONSOLIDATION TEST USING THE METHODS OF TAYLOR, CASAGRANDE AND ASAOKA

The comparison between the different methods of determination of c_v uses the data from MIT Oedometer test No-1 from the case history on Alidrains presented in Chapter 4. The sample comes from the South section at El. -3 ft. The maximum past pressure is 3.7 KSF. The soil is a Medium Soft, CL-CH CLAY.

The data are presented in Table E-1. Taylor's method is the $\sqrt{t}$ method. Casagrande's method is the log t method. Asaoka's construction is also used to obtain the value of c_v. The results of the first two methods can be found in Noiray's thesis (page 200).

TABLE E-1

COMPARISON OF THE DETERMINATION OF c_v BY THE METHODS OF TAYLOR, CASAGRANDE AND ASAOKA

Stress	c_v (ft^2/day)		
(KSF)	$\sqrt{t}$	log t	Asaoka
0.28	----	-----	0.329
0.77	0.429	0.224	0.239
1.54	0.369	0.147	0.061
3.08	0.062	0.028	0.016
6.15	0.018	0.013	0.011
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5.12*	0.025	0.026	0.022
1.28*	0.0097	0.0065	0.0097
----	-----	-----	-----
1.28	0.035	0.027	0.019
5.12	0.031	0.028	0.023
12.3	0.012	0.0091	0.0071

* Unload Increments.