JOHN F. RAMSAY

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TECHNICAL PUBLICATIONS

Source Control of Water Treatment Waste Solids

Principal Investigators

Donald Dean Adrian
John H. Nebiker

Research Report No. 3
for the
Federal Water Pollution Control Administration
research grant WP-01239-02

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DEWATERING OF BENTHIC DEPOSITS ON SOILS

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of the

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ABSTRACT

Benthic deposits in our nation's rivers, lakes reservoirs, and estuaries are producing very difficult problems of pollution including eutrophication. The only solution for the majority of these problems is the removal of these deposits. However, this does not represent a final solution. Disposal of the voluminous dredgings may result in surface and ground water pollution at the disposal site. Such pollution can be controlled by dewatering the deposits prior to final disposal. Gravity dewatering of these highly organic, compressible materials on prepared soils appears practicable. However, optimum design and operation of the dewatering sites have been hampered by the paucity of loading rate criteria, i.e., weight of solids dewatered per unit time and area.

The authors have made a theoretical analysis of gravity dewatering of dredging slurries and dilute organic suspensions. A three-parameter equation was developed relating dewatering rate to the solids content, specific resistance, and compressibility coefficient of the suspensions. All three parameters are readily measured in the laboratory. The theory was validated by extensive bench scale tests, and it was determined that an accurate, dependable analytical method is now available to determine dewatering rates of sludges on soils. It is felt that a rational basis for design and operation of dredging dewatering sites now exists.

I. INTRODUCTION

Background of Problem

Benthic deposits are defined as the deposits on the bottom of rivers, lakes, estuaries, and impoundments. Benthic deposits are usually differentiated from deposits such as sandbars in that they are normally biologically and chemically active. As such, they may exert a significant influence on the presence or absence of dissolved constituents in the overlying water. The biological and chemical activity of benthic deposits derives from their origin; they may be derived from dead algae which have settled from above, leaves which have been washed or blown into the water; and solids originating from wastewaters, either treated or untreated. In rivers, sludge banks, or benthic deposits, discharges of untreated sewage may attain a thickness of several feet prior to scouring and redeposition by floods. In impoundments and lakes,volume reduction of the deposits is achieved through compaction while biological decomposition to stable end products is underway.

If sufficient oxygen is present in the overlying waters, surface portions of benthic deposits may be aerobic. However, interior portions may consume oxygen at a more rapid rate than it diffuses into the mass, so that anaerobic conditions extend from near the surface to the interior of the deposits. Laboratory analyses at the University of Massachusetts have shown that an oxidized microzone extends less than an inch into the sludge when the overlying water is aerobic.[1] Oxidation-reduction reactions are active near this zone. Hutchinson[2] has pointed out that the presence or absence of an oxidized microzone plays a significant role on the chemical exchange rate between benthic deposits and the overlying waters.

Chemical exchange is enhanced or impeded by the oxygen content of the overlying waters. In lakes and impoundments this is subject to diannual cycles coinciding with the overturning of the lake and impoundment contents. Two constituents, nitrogen and phosphorous, are especially important because of their role as the limiting nutrients which retard algal blooms when they are unavailable in sufficient concentrations.[3] Benthic deposits appear to be a reservoir for both nitrogen and phosphorous once these elements have been added by surface runoff or wastewaters. This reservoir may release these elements to the overlying waters under reducing conditions. If released, algal blooms may be promoted, resulting in rapid eutrophication with accelerated aging of the lake.

Removal of the benthic deposits is the only practical long-term solution which will reverse the aging process. If removed, disposal of the benthic deposits is a problem. Land disposal permits use of the dredged material for fill, although some highly organic deposits may be useful as a soil conditioner. However, prior to such usage volume reduction is necessary, for if removed on a large scale by hydraulic dredging, the resulting slurry will be high in water and low in solids content. This suggests a dewatering, or solids-liquid separation process will be necessary to concentrate the solids.

Dewatering organic sludges has been practical for many years as a wastewater treatment operation. Studies by the City of Chicago show costs of dewatering, drying,and disposal varies from \$12/ton to \$60/ton on a dry solids basis,with the higher cost for mechanical dewatering methods and the lower cost for dewatering on sand beds with subsequent land disposal of the solids.[4] However, design data for sand dewatering beds is limited and empirical formulations or rules of thumb for design are frequently used. A rational design method for dewatering slurries on sand beds is formulated in subsequent sections of this paper.

Classification of Sludges

Sludges are characterized by a high water content and a low solids content prior to concentration. For example, a wet sludge weighing 1000 pounds when removed,may weigh only 30 pounds after it has been dewatered and air dried. This emphasizes the immense importance of the dewatering step in reducing the volume and weight of a sludge to manageable proportions.

Dewatering on beds of granular media is influenced by properties of the sludge and properties of the media. Coarse media may promote rapid dewatering for a short time, but if solids are carried into the bed, clogging may result in attenuated bed life. Also, characteristics of the sludge may limit the dewatering rate if a tight cake is formed on the surface of the dewatering bed.

From the previous discussion it is seen that filter characteristics such as its grain size and permeability are important, as well as sludge characteristics such as its permeability, solids content, and penetrating ability. As benthic deposits may be high in organics and flocculant solids, their sludges are found to be compressible. Compression during dewatering alters the permeability. This further complicates development of dewatering theory. However, as shown subsequently, use of the concept of specific resistance and coefficient of compressibility of a sludge cake permits formulation of a dewatering model applicable to compressible materials, such as benthic deposits.

II. THEORY OF DEWATERING OF COMPRESSIBLE MATERIALS

The elements of filtration theory derive from the concepts of fluid flow through closed conduits. If a Newtonian fluid in laminar flow through a circular pipe is assumed subject to pressure, gravity, and viscous forces, the equation of Hagen-Poiseuille holds. It is generally in the form:

$$\Delta p = \frac{32\mu\bar{v}L}{D^2} \tag{2-1}$$

with Δp = pressure drop along the length of the pipe

μ = dynamic viscosity of the fluid $[F][T][L^{-2}]$

$\bar{v}$ = average velocity of the fluid $[L][T^{-1}]$

L = length $[L]$

D = diameter of pipe $[F][L^{-2}]$

It is here more convenient to express pressure drop in terms of headloss. A dimensionally correct conversion is:

$$\Delta p = \rho\ g\ h_f$$

with ρ = density of the fluid $[M][L^{-3}]$

g = acceleration of gravity $[L][T^{-2}]$

h_f = equivalent height of fluid $[L]$

Thus

$$\rho\ g\ h_f = \frac{32\mu\bar{v}\ L}{D^2} \tag{2-1a}$$

or

$$h_f = \frac{32\mu\bar{v}\ L}{\rho\ g\ D^2} \qquad \text{2-1a}$$

Equation 2-1a may be modified for non-circular cross-sections by use of the concept of hydraulic radius (R_h). The hydraulic radius is defined as the ratio of cross-sectional area A to the wetted perimeter of the conduit P. For circular cross-sections this is:

$$R_h \doteq \frac{A}{p} = \frac{\pi\ D^2/4}{\pi\ D} = \frac{D}{4}$$

R_h has the unit of length. It allows expression of the Hagen-Poiseuille equation as:

$$h_f = \frac{2\mu\bar{v}\ L}{\rho g\ R_h^2} \qquad \text{2-1b}$$

For the more general case of non-circular cross-sections, Equation 2-1b should be modified to:

$$h_f = \frac{K\mu\bar{v}\ L}{\rho g\ R_h^2} \qquad \text{2-1c}$$

with K a dimensionless variable, dependent on the shape of the cross-section.

FLOW THROUGH POROUS MEDIA

Flow through porous media is generally represented as laminar flow through a series of interconnected pores of varying cross-section. An initial assumption of a media composed of identical particles is also made.

If N represents the number of particles in a sample of L length and cross-sectional area A', then the wetted conduit surface P over unit length may be written as:

$$P = \frac{N\ S_p}{L} \qquad 2\text{-}2$$

with S_p = surface area of a single particle $[L^2]$

The cross-sectional flow area A is some fraction of the total cross-sectional area A'. This fraction is the porosity ε:

$$\varepsilon = \frac{V_v}{V_{TOT}} \qquad 2\text{-}3$$

with V_v = volume of voids in the sample $[L^3]$

V_{TOT} = total volume of sample, or the volume of voids and solids $[L^3]$

If the particles are each of volume V_p, then $N \cdot V_p$ represents the volume of solids, allowing one to write:

$$V_{TOT} = V_v + N\ V_p$$

Thus

$$\varepsilon = \frac{V_v}{V_{TOT}}$$

$$= \frac{V_v}{V_v + N\ V_p}$$

or rearranging,

$$V_v = \frac{\varepsilon\ N\ V_p}{1-\varepsilon} \qquad 2\text{-}4$$

Substitution of equation 2-2 into

$$R_h = \frac{A}{P}$$

yields

$$R_h = \frac{A\ L}{N\ S_p} \qquad 2\text{-}5$$

Noting that the product A·L equals the volume of voids in the sample (V_V), combination of Equations 2-4 and 2-5 yields:

$$R_h = \frac{A\ L}{N\ S_p} = \frac{V_v}{N\ S_p} = \frac{\varepsilon\ N\ V_p}{1-\varepsilon} \cdot \frac{1}{N\ S_p}$$

or 2-6

$$R_h = \frac{\varepsilon}{1-\varepsilon} \cdot \frac{V_p}{S_p}$$

$$= \frac{\varepsilon}{1-\varepsilon} \cdot \frac{1}{S_o}$$

with S_o = specific surface or ratio of area of particle to volume $[L^{-1}]$.

The velocity $\overline{V}$ through the pores of the sample is generally not readily determinable. Instead of $\overline{V}$, a velocity called the superficial velocity, $\overline{V}_S$, is used. It may be thought of as the velocity of flow if the particles are considered non-existent. The equation of continuity defines $\overline{V}_S$:

$$\overline{V}_S\ A' = \overline{V}\ A$$

or

$$\overline{V} = \overline{V}_S \frac{A'}{A}$$

Multiplication of the right hand side of this equation by L/L = 1 yields

$$\overline{V} = \overline{V}_S \frac{A' \, L}{A \, L}$$

The product A·L is again the volume of voids in the sample, whereas A'·L is the total volume; thus

$$\overline{V} = \overline{V}_S \frac{V_{TOT}}{V_V}$$

or

$$\overline{V} = \frac{\overline{V}_S}{\varepsilon} \qquad \text{2-7}$$

as V_V / V_{TOT} is the porosity.

Substitution of Equations 2-6 and 2-7 into Equation 2-1c follows:

$$h_f = \frac{K \, \mu \, \overline{V} \, L}{\rho g \, R_h^2}$$

$$= \frac{K \, \mu \, L}{\rho \, g} \cdot \frac{\overline{V}_S}{\varepsilon} \Bigg/ \left[\left(\frac{\varepsilon}{1-\varepsilon} \right) \frac{1}{S_o} \right]^2$$

which simplifies to:

$$h_f = \frac{\mu \, L \, \overline{V}_S \, K}{\rho \, g \, \varepsilon} \left[\frac{(1-\varepsilon) S_o}{\varepsilon} \right]^2 \qquad \text{2-8}$$

This equation is frequently written as:

$$\overline{V}_S = \frac{\rho \, g \, h_f \, \varepsilon}{L \, \mu \, K} \left[\frac{\varepsilon}{(1-\varepsilon) S_o} \right]^2$$

The end terms

$$\frac{\varepsilon}{K} \left[\frac{\varepsilon}{(1-\varepsilon)S_o}\right]^2$$

are recognized as intrinsic properties of the porous medium, and are thus given the substitutive symbol k, the intrinsic permeability. The reciprocal of k is R', the filter resistance. It has units of $[L^{-2}]$. Equation 2-8 may therefore be written as:

$$h_f = \frac{\mu \; L \; \overline{V}_S \; R'}{\rho \; g} \qquad 2\text{-}9$$

with

$$R' = \frac{K}{\varepsilon} \left[\frac{(1-\varepsilon)S_o}{\varepsilon}\right]^2 \qquad 2\text{-}10$$

CARMEN-KOZENY RELATIONSHIP

The variables involved in Equation 2-10

$$R' = \frac{K}{\varepsilon} \left[\frac{(1-\varepsilon)\; S_o}{\varepsilon}\right]^2$$

bear closer scrutiny. It is clear that if the sample consists of identical geometrically describable particles, the value of S_0 may be directly calculable. The values of K and ε, however, depend on the packing or placement of the particles. Empirical corrections are applied, resulting in:

$$R' = \frac{K'}{\varepsilon}\left[\frac{(1-\varepsilon)S'}{\varepsilon}\right]^2 \qquad \text{2-10a}$$

with S' defined as a shape factor. Theoretically, K' is a function of particle shape and orientation (i.e., the shape of cross-section available for flow). Grace[2] reports that for particles of diameters <5μ, K' is a variable, due to compression of the small particles. S', the shape factor is, of course, extremely variable, particularly for flocculent particles.

Rewriting of Equation 2-9 using Equation 2-10a

$$h_f = \frac{\mu\, L\, \overline{V}_S\, R'}{\rho\, g}$$

leads to

$$h_f = \frac{\mu\, L\, \overline{V}_S\, K'}{\rho\, g\, \varepsilon}\left[\frac{(1-\varepsilon)\, S'}{\varepsilon}\right]^2 \qquad \text{2-10b}$$

which is the Carmen-Kozeny relationship.[3] It may be modified for non-uniform materials by reference to a particle size analysis, allowing average values for particle area and volume to be used.

FILTRATION OF COMPRESSIBLE MEDIA

It has been repeatedly proven that Equation 2-10b does not apply to the filtration of very fine suspensions such as organic sludges.[4] This is true for two reasons:

1. The suspended materials are of a gelatinous nature and may flocculate when brought in close proximity to each other.
2. The extremely fine size creates a large headloss through a short distance. The loss is in the form of viscous drag over the particle surfaces resulting in particle deformations and reduction of void sizes.

Hence it is seen that the term R' in Equation 2-9 reflecting the influence of shape, size, and orientation of particle, and the void ratio, is a variable for filtration of fine suspensions. Further analysis requires the assumption of a gradual increase in filter resistance with duration of filtration. A later assumption will be the dependence of filter resistance on the pressure applied across the sample.

Assume a filter material of known and constant resistance which will serve as a supporting media to the filter cake formed on its surface during filtration. The resistance of this filter material may be called R_f $[L^{-2}]$ and the depth, L_f. It follows that Equation 2-9

$$h_f = \frac{\mu L \overline{V}_S R'}{\rho g} \qquad 2\text{-}9$$

may include the effects of the supporting filter by writing

$$h_f = \frac{\mu}{\rho g} (\overline{V}_S L R' + \overline{V}_S L_f R_f) \qquad 2\text{-}11$$

with $\overline{V}_S$ = superficial velocity in supporting filter.

The superficial velocities are best rewritten in terms of the filtrate volume flow rate dV/dt $[L^3][T^{-1}]$. Thus:

$$\overline{V}_S = \frac{1}{A}\ \frac{dV}{dt}$$

and

$$\overline{V}_{S'} = \frac{1}{A}\frac{dV}{dt}$$

Hence Equation 2-11 reads

$$h_f = \frac{\mu}{\rho\ g\ A}\ \frac{dV}{dt}\ (LR' + L_f\ R_f) \qquad \text{2-11a}$$

which reorganizes to

$$\frac{dV}{dt} = \frac{\rho\ g\ A\ h_f}{\mu\ (LR' + L_f\ R_f)} \qquad \text{2-11b}$$

Carmen[3] introduced the concept that the length L of the compressible filter cake was proportional to the volume of filtrate:

$$L = V_c\ \ V/A$$

with V_c = volume of cake deposited per unit volume of filtrate (dimensionless)

This leads to

$$\frac{dV}{dt} = \frac{\rho\ g\ A\ h_f}{\mu\ (\frac{V_c\ VR'}{A}\ + L_f\ R_f)} \qquad \text{2-11c}$$

However, the nature of compressible cakes makes it illogical to measure volume.

More likely is the measurement of weight. The term c, or weight of dry cake solids per unit volume of filtrate is thus introduced. Dimensions of c are $[F][L^{-3}]$. This necessitates a new definition of filter resistance, R, by setting

$$V_c \, R' = c \, R$$

R, entitled the specific resistance, is in units of $[L][F^{-1}]=[T^2][M^{-1}]$.

Equation 2-11c can be reformulated then to

$$\frac{dV}{dt} = \frac{\rho \, g \, A \, h_f}{\mu(\frac{c \, R \, V}{A} + L_f \, R_f)}$$

$$= \frac{\rho \, g \, A^2 \, h_f}{\mu(c \, RV + A \, L_f \, R_f)} \qquad \text{2-11d}$$

This equation is the basis for work done by Coackley and Jones,[5] utilizing the concept of specific resistance to calculate vacuum filter performance in dewatering wastewater sludges.

SPECIFIC RESISTANCE

The specific resistance of a material varies with depth, being a maximum at the supporting filter interface. For shallow depths, however, R varies slightly, and thus an average may be used. The value of R is determined in the laboratory by constant pressure filtration; a constant pressure or headloss, h_f, allows integration of Equation 2-11d

$$\frac{dV}{dt} = \frac{\rho\ g\ A\ h_f}{\mu(\frac{c\ R\ V}{A} + L_f\ R_f)} \qquad \text{2-11d}$$

$$\int_0^t \rho\ g\ A\ h_f dt = \int_0^V \frac{\mu c\ R\ V\ dV}{A} + \int_0^V \mu\ L_f\ R_f\ dV$$

$$t = \frac{\mu\ c\ R\ V^2}{2\ \rho g\ A^2\ h_f} + \frac{\mu\ L_f\ R_f\ V}{\rho\ g\ A\ h_f} \qquad \text{2-12}$$

COEFFICIENT OF COMPRESSIBILITY

It must be emphasized that the value of R is dependent not only upon the sludge characteristics but on the pressure at which the test is run. It is known from experimental analysis that a relation between R and Δ p exists.[2] It is in the form

$$R = a_o + R_c\ (\Delta p)^\sigma \qquad \text{2-13}$$

or using headloss

$$R = a_o + R_c\ (\rho\ g\ h_f)^\sigma \qquad \text{2-13a}$$

with a_o, and R_c and σ constants for a given sample, and independent of pressure. σ is commonly known as the cake compressibility factor, and R_c the cake constant. Several determinations of R for various pressures may be plotted on bi-logarithmic paper, the slope of the line of best fit revealing the cake compressibility factor. Generally a_o is neglected, simplifying Equation 2-13a to

$$R = R_c\ (\rho\ g\ h_f)^\sigma \qquad \text{2-14}$$

Equation 2-14 proves dimensionally incorrect. It may be modified for dimentional correctness by writing

$$R = R_c \left(\frac{\rho \; g \; h_f}{\Delta \; p_a = 1} \right)^{\sigma} \qquad 2\text{-}15$$

which therefore describes R_c as the specific resistance $[T^2][M^{-1}]$ at a pressure of unity $[F][L^{-2}]$. Equation 2-15 does not, however, differ numerically from Equation 2-14.

GRAVITY DEWATERING

Application of the theory and concept of specific resistance to gravity drainage of organic sludges on porous media is most promising. Assume a sludge is applied on a dewatering bed of sand or other porous material. Referring back to Equation 2-11d

$$\frac{dV}{dt} = \frac{\rho \; g \; A^2 \; h_f}{\mu(c \; RV + A \; L_f \; R_f)} \qquad 2\text{-}11d$$

the term dV/dt may be rewritten in terms of the head as the rate of drop of the sludge surface by drainage

$$\frac{dV}{dt} = - A \frac{dH}{dt}$$

with H = the head [L] .

The terms referring to the supporting media may be ignored, as sludge is vastly higher in resistance than permeable soils.[6]

$$-A \frac{dH}{dt} = \frac{\rho \; g \; A^2 \; h_f}{\mu \; c \; R \; V}$$

But h_f is none other than H, assuming of course, that resistance in the filter is minimal. Hence, with cancellation of A

$$\frac{dH}{dt} = \frac{-\rho\ g\ A\ H}{\mu\ \ c\ R\ V} \tag{2-16}$$

This equation may be written for a cumulative drop from height H_o at t=0 upon substitution of $A(H_o-H)$ for V:

$$\frac{dH}{dt} = \frac{-\rho\ g\ A\ H}{\mu\ c\ A\ (H_o-H)\ R} \tag{2-17}$$

Noting that

$$R = R_c\ (\rho g\ h_f)^{\sigma} \tag{2-14}$$

or here

$$R = R_c\ (\rho\ g\ H)^{\sigma}$$

Equation 2-17 can be rewritten as

$$\frac{dH}{dt} = \frac{-\rho\ g\ H}{\mu\ c\ (H_o-H)\ R_c\ (\rho\ g\ H)^{\sigma}}$$

reorganized to

$$dt = -R_c\ \mu\ c\ (H_o-H)(\rho\ g\ H)^{\sigma-1}\ dH \tag{2-18}$$

This equation may be integrated from $H = H_o$ at $t = 0$:

$$\int_o^t dt = -\int_{H_o}^{H} R_c\ \mu\ c\ (H_o-H)(\rho\ g\ H)^{\sigma-1} dH$$

$$= -R_c\ \mu\ c(\rho g)^{\sigma-1}\left(\int_{H_o}^{H} H_o H^{\sigma-1} dH - \int_{H_o}^{H} H^{\sigma}\ dH\right)$$

which yields

$$t = -R_c \mu c (\rho g)^{\sigma-1} \left[\frac{H_o H^{\sigma}}{\sigma} - \frac{H^{\sigma+1}}{\sigma+1} \right]_{H_o}^{H}$$

$$= -R_c \mu c (\rho g)^{\sigma-1} \left(\frac{H_o H^{\sigma} - H_o^{\sigma+1}}{\sigma} + \frac{H_o^{\sigma+1} - H^{\sigma+1}}{\sigma+1} \right) \qquad 2\text{-}19$$

Equation 2-19 allows determination of the drainage rate of a sludge on porous material, provided the initial sludge solids content is evenly distributed. A futher limitation is the assumption of filtration in the absence of consolidation, whereby compaction creates a pressure gradient near the top in the upward direction. Shallow layers or dilute suspensions of sludges do not present this complication.

Benthic sludges are known frequently to settle out rapidly when allowed to dewater. A modification of Equation 2-19 may be developed for the case of a supernatant draining on a settled sludge layer. Another analysis of great potential interest is the drainage of a sludge on a previously applied sludge.

III. EXPERIMENTAL PROCEDURE AND VERIFICATION

Sampling

Benthic deposits were removed by an Ekman clam-shell bottom dredge. Samples were all taken in the vicinity of Amherst from two small ponds and a water supply reservoir. These were in turn characterized as to depth of water and depth of deposit as follows:

Source	Depth of Water Feet	Depth of Deposit Feet	Drainage Area	Water Surface Area
Atkins Reservoir	8	0.1	800 acres	40 acres
Campus Pond	5	3	130 acres	12 acres
Spring Pond	6	3	60 acres	1 acre

Campus Pond was unique in that it is in the midst of a developed area and had been dredged in the past decade. Spring Pond has no surface influent or effluent. Both Spring Pond and Atkins Reservoir lie in protected forests. All samples appeared relatively homogeneous, made up of dense slurries with a few clumps of vegetation which were readily disposed of. All samples were refrigerated previous to testing. A shallow zone of supernatant developed after two or three days.

The samples were classified according to standard methods of determination of water content on a wet-weight basis.[1] Also weight loss at T = 600°C was determined, and expressed as the residue remaining at that temperature in per cent of the residue remaining at 103°C. Results are found in Table 3-1.

Table 3-1: Classification of Deposits

Source	Solids Content (% wet-basis)	Residue @ 600°C (% of residue @ 103°C)	Specific Resistance (sec^2/gm)	Coefficient of Compressibility
Atkins Reservoir	5.19	79.8	7.7×10^8	0.82
Campus Pond	42.75	91.1	1.26×10^8	0.62
Spring Pond	6.88	62.7	11.0×10^8	0.84

Specific Resistance

The method of determination of specific resistance has to date not been standardized. It may be expected that the method chosen will influence the results; however, such influence is thought to be minor.

Basically, the specific resistance is determined by applying a constant vacuum across a sludge cake, and measuring filtrate volume and time at approximately 30 second intervals for a total duration of about seven minutes.[2] Figure 3-1 portrays the necessary equipment.

A 100 ml sample of sludge is poured onto a wetted Whatman #5 filter paper. At time zero the vacuum is applied and readings are taken as previously described. At the end of the test, the cake sample is weighed and dried and a wet-weight water content is determined.

An easy method for data reduction has been demonstrated by Coackley and Jones.[3] Referring back to Equation 2-12, one notes that it may be reformulated in terms of t/V versus V:

$$\frac{t}{V} = \frac{\mu\ c\ R}{2\rho\ g\ A^2\ h_f}\ V + \frac{\mu\ L_f\ R_f}{\rho g\ A\ h_f} \qquad 3\text{-}1$$

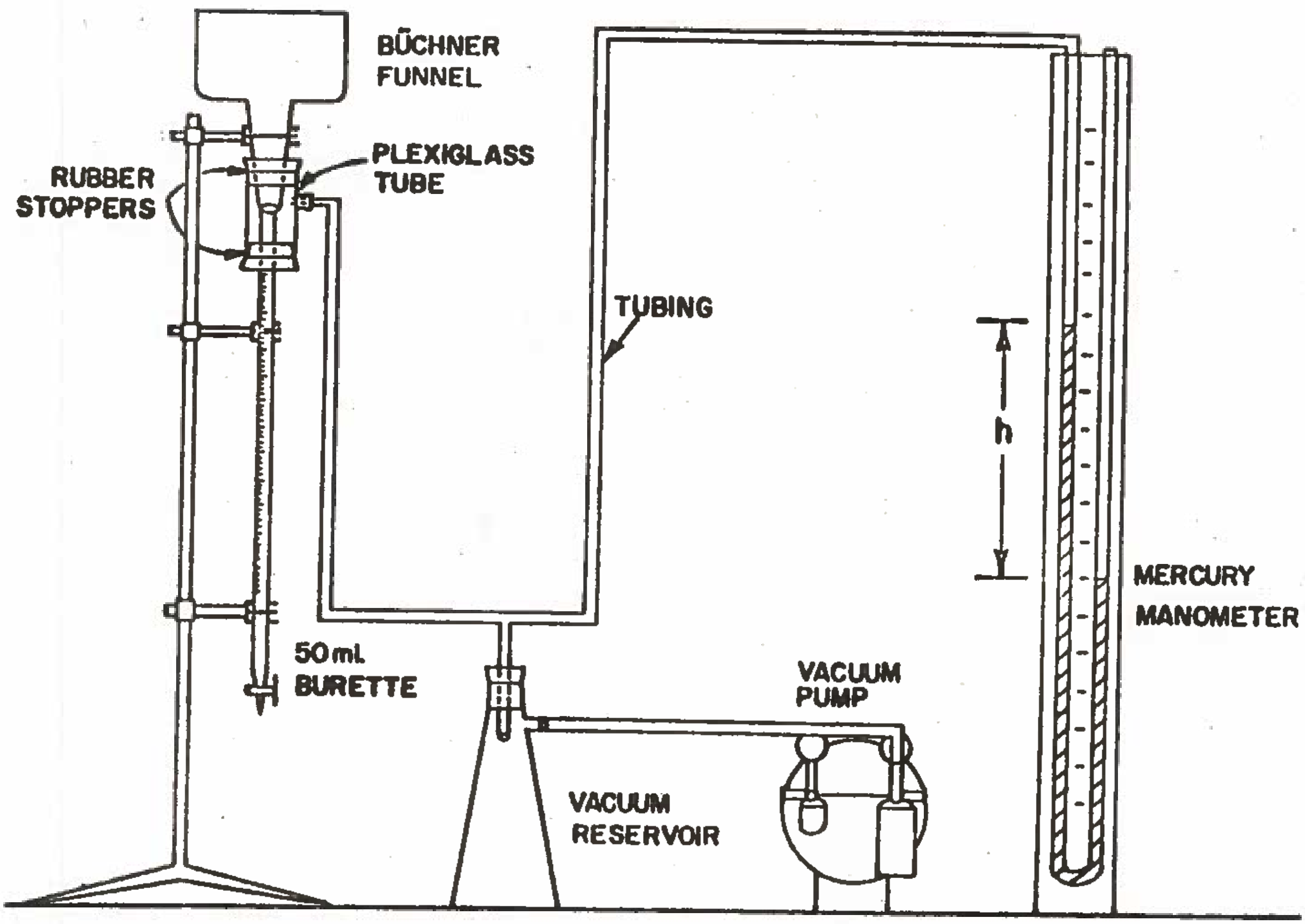

Figure 3-1: Apparatus to determine the specific resistance of compressible sludges.

A plot of t/V versus V (Figure 3-2) will yield a straight line, the slope of which is seen to equal the fractional coefficient in Equation 3-1 (see Figure 3-2). The specific resistance will then be

$$R = \frac{2\rho \ g \ A^2 h_f}{\mu \ c} \cdot (\text{slope}) \tag{3-2}$$

The area A of the filter and the density ρ and dynamic viscosity μ of the fluid are readily determinable. The vacuum, h_f, was set and known. There remains the determination of c, the dry weight of cake solids per unit volume of filtrate.

The volume of filtrate that occurs during testing, assuming no solids loss, is

$$V = (W_{TOT_0} - W_{TOT_f})/\rho g \tag{3-3}$$

with V = volume of filtrate $[L^3]$

W_{TOT_0} = weight of unfiltered sludge [F]

W_{TOT_f} = weight of filtered cake [F]

ρg = weight-volume conversion factor $[F][L^{-3}]$

(ρ refers to filtrate)

The weight of dry cake solids per unit volume of filtrate is $c = W_{TS}/V$ which may be written

$$c = \frac{W_{TS} \ \rho g}{W_{TOT_0} - W_{TOT_f}}$$

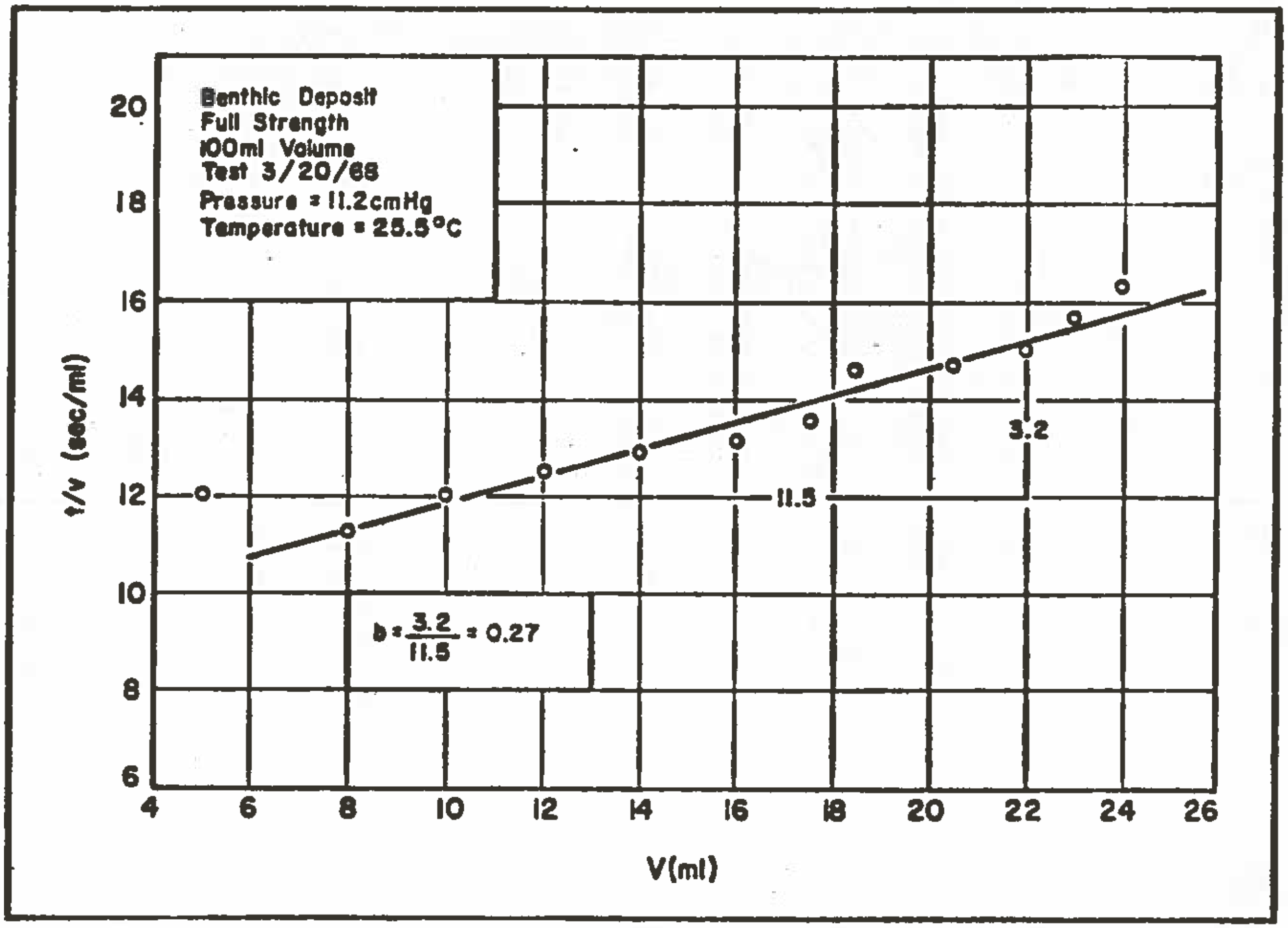

Figure 3-2: Sample plot to determine specific resistance for deposits from Campus Pond.

However, W_{TS} can be expressed in terms of the initial wet-basis water content

$$W_{TS} = W_{wo}\left(\frac{100}{\Theta_o} - 1\right)$$

with W_{wo} = initial water content of sludge [F]

θ_o = initial wet-basis water content [%]

Hence

$$c = \frac{W_{wo}\ \rho\ g\left(\frac{100}{\Theta_o} - 1\right)}{W_{TOT_0} - W_{TOT_f}} \qquad \text{3-3a}$$

Equation 3-3a was used to calculate the value of c, which in turn was placed in Equation 3-2 for eventual calculation of R. Results are shown in Table 3-1. As emphasized in Section II, the value of R is dependent on pressure. The literature generally refers to a pressure of 15 in. Hg (=38.1 cm). This convention is followed here.

Coefficient of Compressibility

The dependence of R on pressure as expressed by Equation 2-14 can be experimentally determined by a series of different R determinations for a number of different constant pressures. As R varies with h_f to the σ power, σ being the coefficient of compressibility, it is evident that a bilogarithmic plot of R versus h_f will yield a slope of value σ. This procedure was used, an example is given in Figure 3-3, and the coefficient for the sludges is to be found in Table 3-1.

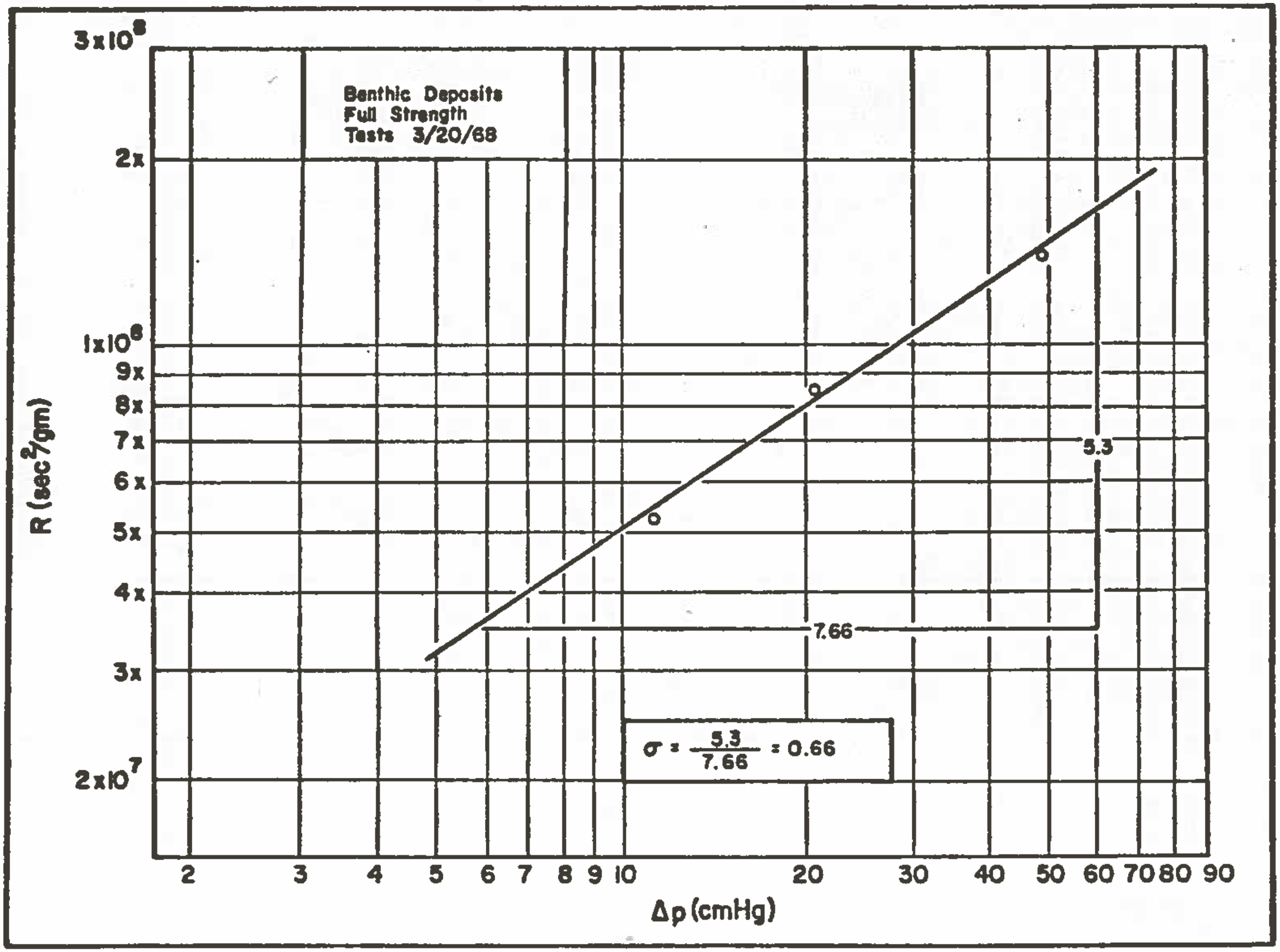

Figure 3-3: Sample plot to determine coefficient of compressibility for deposits from Campus Pond

Gravity Dewatering Apparatus

The gravity dewatering apparatus (Figure 3-4) consisted of 4 inch (10.2 cm) I. D. acrylic tubes, 36 inches (91.5 cm) in length, with flanges at both ends. Blind flanges constructed out of acrylic plate are affixed with nickel-plated nozzles. The bottom nozzle leads to the one liter graduated cylinder, by a combination of plastic and glass tubing.

The filtrate collected in the graduated (2 ml intervals) cylinder displaces air which is forced up a return tube to the top nozzle. This allows an entirely enclosed system to be maintained, preventing any measurement disturbance by evaporation. Above the bottom nozzle lies a 24 B & S gage wire mesh, which in turn supports 5 cm of coarse gravel, followed by 5 cm of A. S. T. M. standard Ottawa sand. Piezometers are attached to the column sides at appropriate intervals.

Dewatering Methodology

After the sand surfaces in the columns are carefully leveled and compacted, the sand is saturated by introduction of water from beneath. The water level is maintained at the sand surface during charging of the columns with homogeneous sludge mixtures to prevent any entrapped air. It is important to note that the effective dewatering head is the distance from the sludge surface to the tip of the discharge tube located in the filtrate collection. When the piezometers are used, they are filled with water and the rubber tubes are pinched until the columns are charged.

A pinch cock (not shown) next to the bottom nozzle is opened at t = 0 and volumetric readings are made at appropriate intervals. The graduated cylinders are removed, emptied, and replaced when needed, and the piezometer levels are recorded at regular intervals if used.

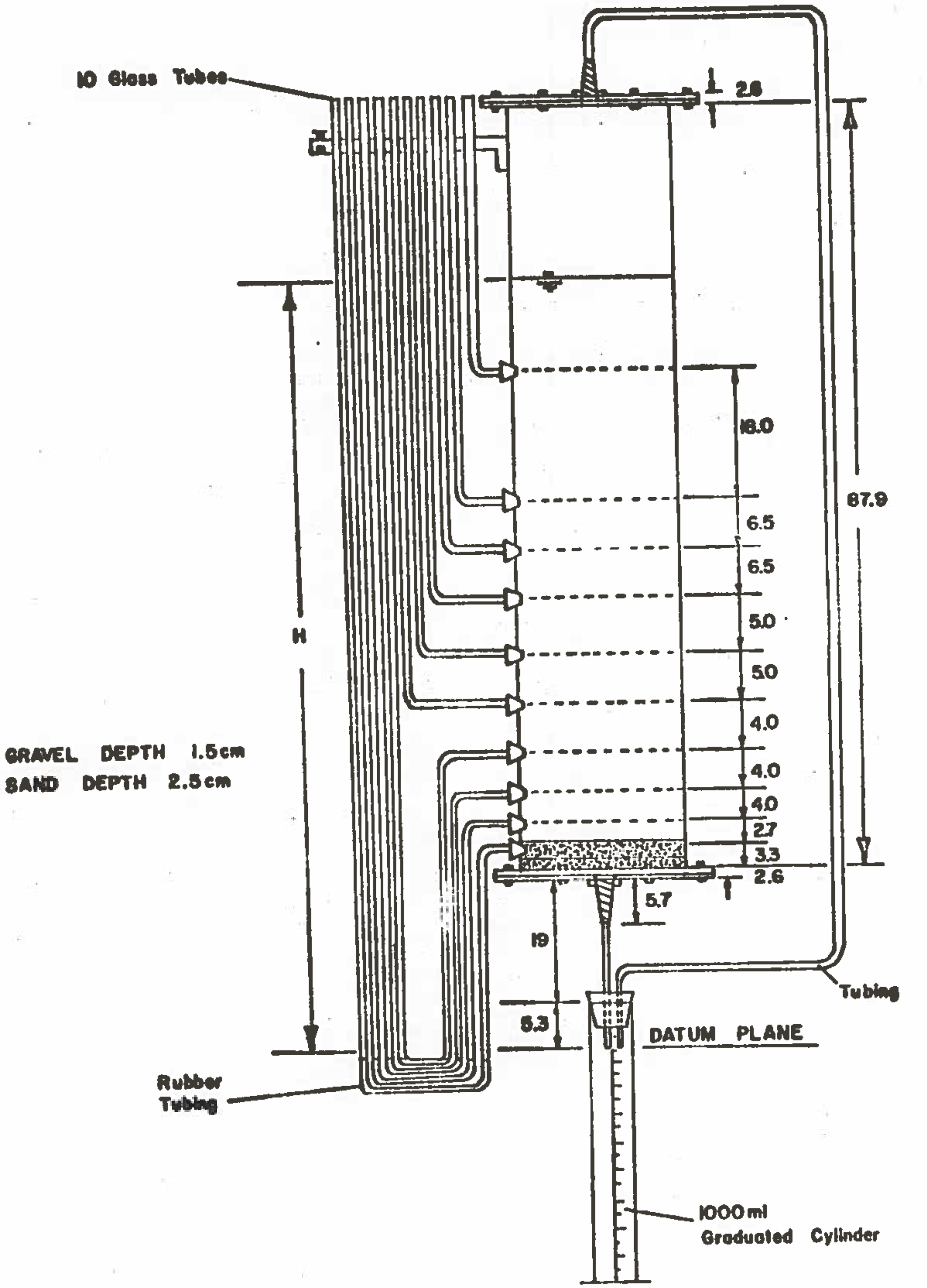

Figure 3-4: Gravity dewatering apparatus with piezometer tubes. The supporting filter media consists of 5 cm of Ottawa standard sand underlain by 5 cm of coarse gravel. Cell dimensions are in cm.

Dewatering effectively ceases upon the attainment of a pseudo-rigid plastic mass, whose surface develops menisci suspending the underlying moisture. A slow process of consolidation results which is not noteworthy for the volume of filtrate produced.

Experimental Results

Experiments conducted by the authors have shown that the headloss occurring across the sludge is predominantly in the first centimeter above the sand-sludge interface. It is clear that the sludge cake filtration theory is thus appropriate to gravity dewatering.

The volumetric readings are reduced to values of H, or total head, by subtraction and conversion of the volumes from the initial head. The values of R_c and σ are determined by Buchner funnel testing. Values for ρ, g and μ are, of course, known. Hence, the only factor remaining to be determined in Equation 2-19 is the value of c, and this can be calculated from Equation 3-3a.

Sample results for a wastewater sludge using Equation 2-19 is shown in Figure 3-5. The initial depth of sludge of 24 cm (note: not the head H) at a wet-basis water content of 96.3% was reduced to a depth of 15 cm.

Three identical samples dewatered almost identically, although one in the latter stages demonstrated a sharp deviation due to a phenomena of solids break-through into the sands. This was most likely due to an oversight in the filling of the sand on top of the gravel, leaving too shallow a layer of sand to support the sludge and the viscous shear of the filtrate.

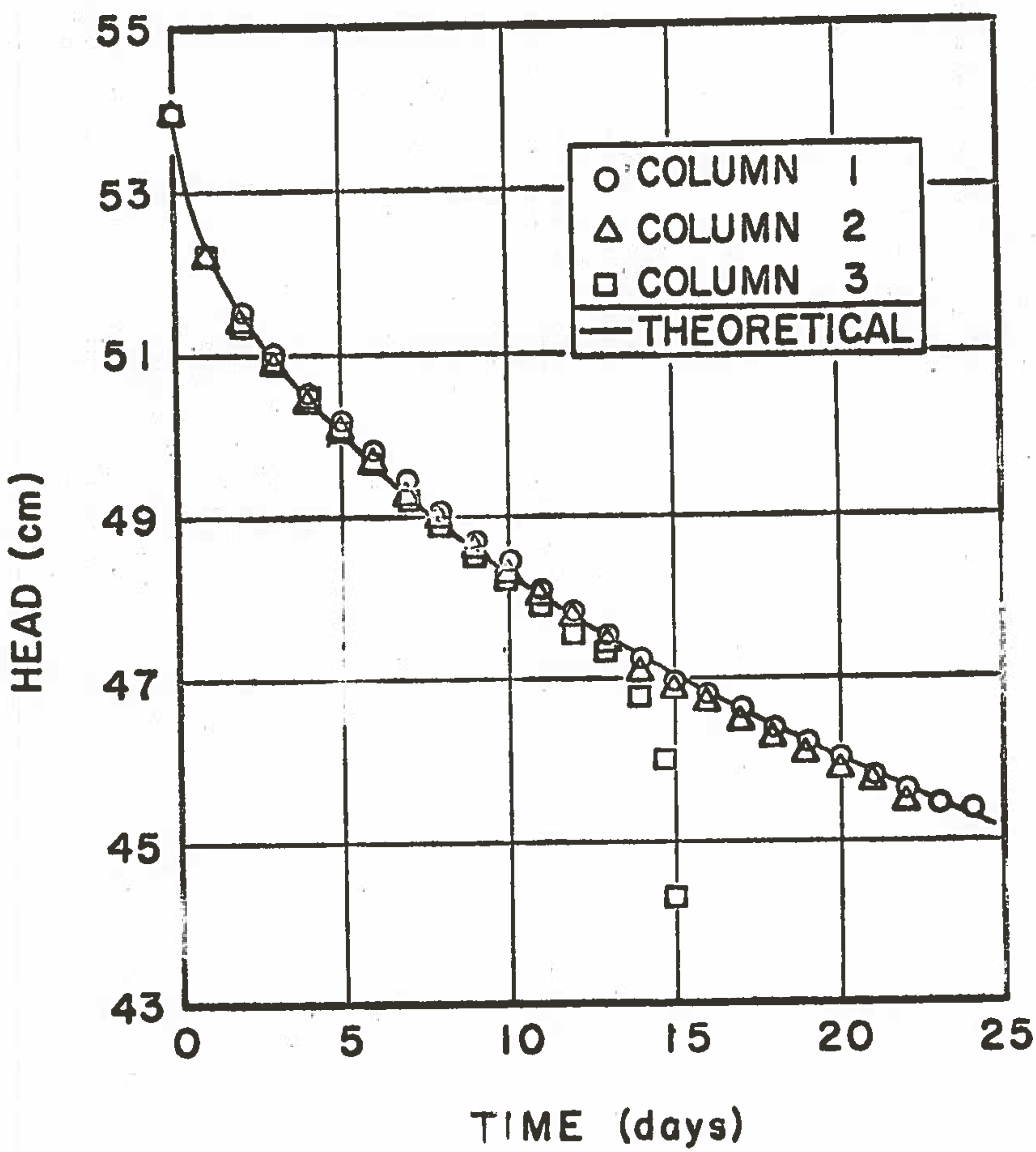

Figure 3-5: Drainage of a wastewater sludge as a function of time. The theoretical curve refers to Equation 2-19.

IV. RESULTS AND CONCLUSIONS

Reversal of the aging process of a eutrophic lake, reservoir or estuary will require removal of the benthic deposits. These deposits serve as a reservoir for the nutrients nitrogen and phosphorous which must be present in solution to promote algal blooms and subsequent eutrophication. Land disposal of the dredged solids will be difficult unless volume and weight reduction can be achieved through a dewatering step, for it is highly likely that as much as 90-95% of the weight of dredged material will be water.

Dewatering on soils such as sand is hampered by the near absence of design data with which to design such a facility. A mathematical model applicable to such design has been developed and verified with laboratory column tests. It is dependent upon three parameters: solids content, specific resistance and coefficient of compressibility, all of which are readily measured through laboratory tests.

The parameters specific resistance and coefficient of compressibility were measured in the laboratory for three benthic deposits for the first time for three Massachusetts lakes. Additional data is needed for a wide spectrum of benthic deposits in order to provide design data for field evaluation of this dewatering process.

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AN INVESTIGATION OF SLUDGE DEWATERING RATES

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of the

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Abstract

An Investigation of Sludge Dewatering Rates

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Wastewater sludge disposal is well recognized as one of the most vexing problems of water pollution control. The problem dimensions promise to increase at a dramatic rate in the future, primarily due to an increase in sludge volume and a concomitant decrease in availability of acceptable sites for sludge or sludge-ash disposal. Volume reduction previous to disposal will continue to receive priority; the use of polyelectrolytes to facilitate reduction will be greatly increased.

Despite the many commercial devices for dewatering available, the most widely used system entails dewatering and drying of digested sludge on drying beds, followed by application to the soil for conditioning purposes or dumping as fill. Marked increases in efficiency have been reported by the use of coagulant polymers, but quantitative evaluation of the results remains impossible until satisfactory parameters for measuring drainage of compressible materials are developed.

The authors made a theoretical analysis of gravity dewatering of wastewater sludge on drying beds, and developed an equation relating depth of sludge and time with the solids content, specific resistance, and coefficient of compressibility. The concept of media factor was introduced to account for various sand characteristics. Extensive bench-scale tests validated the theory. All parameters are readily determinable by simple laboratory tests, thus presenting an accurate, dependable analytical method to predict gravity dewatering. This therefore provides a rational basis for selection and optimization of a sludge dewatering system.

AN INVESTIGATION OF SLUDGE DEWATERING RATES

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Introduction

Wastewater sludge disposal is well recognized as one of the most vexing problems of water pollution control. The problem dimensions promise to increase at a dramatic rate in the future, primarily due to an increase in the degree of treatment and a decrease in availability of acceptable sites for sludge or sludge-ash disposal. Presently some 25 to 65 percent of total capital and operating costs of primary and secondary treatment plants are expended on handling sludge, whose volume is less than 1% of the total plant influent. Some advanced waste treatment processes will produce sludge volumes that approach 10 percent of the total inflow, thus greatly magnifying the sludge disposal problem. Increased emphasis must be placed on volume reduction of the sludge previous to ultimate disposal.

Despite a variety of mechanical methods available for sludge volume reduction, some 72% of treatment plants in the United States utilize sludge drying beds[1]. Irrigation or lagooning methods are also of current great interest, due in part to recent studies undertaken by the Metropolitan Sanitary District of Greater Chicago[2]. These studies suggest that the above mentioned gravity dewatering and drying methods may be feasible and economically advantageous for even large metropolitan areas.

The competitive position of these methods may be considerably enhanced by the use of chemical coagulants. Reports on the acceleration of dewatering resulting from additions of alum date back as far as 1923[3]. Later ferric chloride and lime were used as coagulants. However, such conditioning has not been popular, in part due to the deleterious effect of many of the chemicals to croplands receiving the sludge, or to blinding of the dewatering bed. The use of polyelectrolytes as conditioners promises to quiet these objections.

Optimal gravity dewatering by, for example, the addition of polyelectrolytes, can not be readily determined due to the lack of parameter formulation with accompanying simple laboratory tests. This is in stark contrast to the situation with regard to optimization of vacuum filtration. No doubt the successful application of the vacuum filter in many instances is due to the availability of such readily determinable parameters, such as are found in the specific resistance concept and the role of filter media on dewatering rates. Laboratory determinations of specific resistance as a function of the dosage of conditioner, allows direct computation of the change in vacuum filter yield, selection of the optimal dosage, and accompanying cost justification. Similar parameter formulation and laboratory tests are clearly needed for gravity dewatering.

Previous Research

Despite much research on gravity dewatering dating back over fifty years, there exist few formulations equating dewatering rates to inherent sludge characteristics. Several empirical relationships exist[4] of the form

$$t_f \propto \frac{W_{TS}}{A}$$

with t_f = dewatering time

W_{TS}/A = weight of sludge solids applied per unit area (so-called solids loading).

Several of the formulae present the initial solids content as an additional factor.

Another formula equates dewatering time with the square root of the specific resistance[5]. While thereby the intrinsic permeability characteristics of the sludge are taken into account, a question arises as to the general validity of the formula: it is based on a group of average values on a yearly basis. It is also dimensionally incorrect as are most of the other relationships.

Jeffrey[6] developed an exponential formula from pilot-plant data. This in turn was later criticized by Logsdon and Jeffrey[7]: the coefficients in the equation needed determination for each separate sludge sample; determinations that required three to four weeks. Logsdon and Jeffrey then proposed a Buchner funnel test to estimate gravity dewatering time. The procedure involved a comparison of the times required to remove equal relative amounts of moisture by vacuum filtration and by gravity dewatering. These comparisons are shown in Figure 1, and were expressed as a series of linear relationships. The testing procedure outlined by Logsdon and Jeffrey requires calibration, again necessitating several weeks of testing.

From the previously described work, it is seen that possible parameters for gravity dewatering include solids loading and solids content (and hence depth), and an intrinsic sludge permeability. A priori, the final solids content and characteristics of the supporting media should be included.

Formulation of Specific Resistance and Coefficient of Compressibility

The well-developed and familiar concept of specific resistance holds particular promise as a basis for development of an equation describing gravity dewatering of sludge, in that vacuum filtration differs from gravity dewatering only insofar as a constant vacuum is applied across the filter. Gravity dewatering, of course, as here defined, results from a continually decreasing pressure or head.

Reference to the classic work by Coackley and Jones[8] reveals that specific resistance is derived from basic concepts of fluid flow in porous media:

$$\frac{1}{A}\frac{dV}{dt} = \frac{\rho g\, h_f}{\mu(LR^1 + L_f R_f^1)}$$

The terms used are:

$\frac{1}{A}\frac{dV}{dt}$ = volumetric flow rate of the filtrate per unit area $[L]\,[T^{-1}]$

h_f = head loss in terms of the filtrate $[L]$

ρ = density of the filtrate $[M]\,[L^{-3}]$

g = acceleration constant $[L]\,[T^{-2}]$

μ = dynamic viscosity of the filtrate $[M]\,[L^{-1}]\,[T^{-1}]$

L = thickness of the dewatering cake $[L]$

R^1 = resistance of the cake $[L^{-2}]$

L_f and R_f^1 refer to thickness and resistance of the supporting material, with units as before.

Solids are continuously added to the cake throughout the duration of filtration. A relationship

$$LR^1 = \frac{cRV}{A} \tag{2}$$

has been found valid, allowing Equation 1 to be written

$$\frac{1}{A}\frac{dV}{dt} = \frac{\rho g\ h_f}{\mu(\frac{cRV}{A} + L_f R_f^1)} \tag{3}$$

Here V is the cumulative total volume of filtrate $[L^3]$, c is a proportionality factor $[F]\ [L^{-3}] = [M]\ [L^{-2}]\ [T^{-2}]$. It is closely approximated by[9].

$$c = \rho g\ S_o/100 \tag{4}$$

In turn S_o is the initial solids content [%]. The units of R, specific resistance, in Equation 3 are $[T^2]\ [M^{-1}]$.

Before proceeding further in demonstrating the applicability of Equation 3 to gravity dewatering, note that integration of this equation yields

$$t = \frac{\mu c R V^2}{2\rho g A^2 h_f} + \frac{\mu L_f R_f^1\ V}{\rho g A h_f} \tag{5}$$

By running a Buchner funnel or filter leaf test at a constant h_f, measuring S_o, and observing t and V, a determination of R may be made. This is accomplished by noting the slope of a t/V versus V plot.

It must be emphasized that the value of R is dependent not only upon the sludge characteristics, but also upon the pressure at which the test is run. A dimensionally correct relationship between R and h_f exists[10]:

$$\frac{R}{R^*} = \left(\frac{h_f}{h_f^*}\right)^{\sigma} \tag{6}$$

R^* refers to a value of specific resistance at any arbitrary head loss of h_f^*. The exponent σ, is an intrinsic sludge compressibility characteristic. Dimensionless, it is known as the coefficient of compressibility. Figure 2 illustrates the popular determination of σ, which is the slope of a bilogarithmic plot of R versus h_f.

A Formulation to Describe Gravity Dewatering

Assume a sludge is applied to a dewatering bed of sand or other porous material. Referring back to Equation 3

$$\frac{1}{A}\frac{dV}{dt} = \frac{\rho g\ h_f}{\mu(\frac{cRV}{A} + L_f R_f^1)} \tag{3}$$

the term dV/dt may be rewritten as the rate of drop of the sludge surface by drainage:

$$\frac{1}{A}\frac{dV}{dt} = -\frac{dH}{dt} \tag{7}$$

A minus sign is introduced, for H [L] is measured positive upwards.

The terms in Equation 3 connoting the effect of the supporting media may at this time be ignored. Certainly the resistance to flow in the sludge cake is vastly greater than that introduced by the media. Hence Equation 3 with Equation 7 yields

$$-\frac{dH}{dt} = \frac{\rho g h_f}{\mu cRV/A} \tag{8}$$

but h_f is none other than H, assuming, of course, that resistance in the filter is minimal. Also note that the cumulative drop in head from H_o at $t = 0$, $H_0 - H$, equals V/A. Equation 8 now may read

$$\frac{dH}{dt} = \frac{-\rho gH}{\mu cR(H_o - H)} \tag{9}$$

The specific resistance in Equation 9 will decrease with decreasing head according to Equation 6. Rewriting Equation 6 as

$$R = R^* \left(\frac{H}{h_f^*}\right)^\sigma \qquad (10)$$

allows it to be substituted into Equation 9, yielding

$$\frac{dH}{dt} = \frac{\rho g H}{\mu c R^* (H_o - H)} \left(\frac{h_f^*}{H}\right)^\sigma \qquad (11)$$

Equation 4 stated

$$c = \rho g S_o / 100 \qquad (4)$$

providing a simplification of Equation 11 to

$$\frac{dH}{dt} = - \frac{100H}{\mu S_o R^* (H_o - H)} \left(\frac{h_f^*}{H}\right)^\sigma \qquad (12)$$

This equation may now be reorganized and integrated:

$$t = \int_o^t dt = \frac{\mu S_o R^*}{100(h_f^*)^\sigma} \left[\int_{H_o}^{H} H^\sigma dH - H_o \int_{H_o}^{H} H^{\sigma-1} dH \right]$$

$$= \frac{\mu S_o R^*}{100(h_f^*)^\sigma} \left[\frac{H^{\sigma+1}}{\sigma+1} - \frac{H_o H^\sigma}{\sigma} \right]_{H_o}^{H}$$

$$= \frac{\mu S_o R^*}{100\sigma(\sigma+1)(h_f^*)^\sigma} \left[H_o^{\sigma+1} + \sigma H^{\sigma+1} - (\sigma+1) H_o H^\sigma \right] \qquad (13)$$

Equation 13, which it should be emphasized, is dimensionally correct, equates time of dewatering from an initial head of H_o to a head at time t of H.

It may be rewritten to solve for bed loading intensity and final solids content. For this, the reader in the interest of brevity is referred elsewhere[9, 10, 11]. If the piezometric head in the supporting media is at the sludge-sand interface, depth of sludge may be inserted for head directly into the equation. In such a case the minimum depth calculated should not be unrealistic -- it is controlled by the maximum solids content obtainable.

One would not expect the value of R^*, calculated from Buchner funnel testing, to fully represent a value of specific resistance of a sludge on different supporting media such as a sand. Approximately 40-50% of the gross sand surface is porous; that of a Buchner funnel less so, even though filter paper is used. To illustrate extreme cases of the role of the supporting media, it is clear that impermeable supporting media will allow no dewatering, whereas very coarse supporting media will retain no solids, resulting in a zero resistance to flow.

To account for the interrelationship between the sludge and the supporting media, but mindful of dimensional correctness, a media factor must be introduced into Equation 13. This media factor can be considered as a function of the ratio of a representative sludge floc diameter to an equivalent diameter of a sand grain; the sand grain representing the effect of the supporting media or some equivalent parameter of the Buchner funnel, or filter leaf, or any other dewatering device. The value of the media factor, m, would be larger for finer sands and smaller for coarse sands. As a function of ratio of diameters, m would be dimensionless.

Experimental Apparatus and Procedure

The gravity dewatering apparatus (Figure 3) consist of nine 4 inch (10.2 cm) I. D. acrylic tubes, 36 inches (91.5 cm) in length, with flanges at both ends. Blind flanges constructed from acrylic plate are affixed with nickel-plated nozzles. The bottom nozzle leads to the one-liter graduated cylinder, by a combination of plastic and glass tubing.

The filtrate collected in the graduated (2 ml intervals) cylinder displaces air which is forced up a return tube to the top nozzle. This allows an entirely enclosed system to be maintained, preventing any measurement disturbance by evaporation. Above the bottom nozzle lies a 24 B & S gage wire mesh, which in turn supports 3 cm of coarse gravel, followed by 5 cm of sand. Piezometers are attached to an identical additional column at appropriate intervals to permit observations of pore water pressure.

It is important to note that the effective dewatering head is the distance from the top sludge surface to the tip of the discharge tube located in the graduated cylinder. The distance from the discharge tube to the sand surface approximates 12 inches, the standard design depth for sand in a drying bed[1]. This additional head created is thought to realistically approach the operation of properly constructed drying beds, for the sand becomes rapidly saturated tending to place a vacuum on the bottom of the sludge-sand interface, as is created by the discharge system of the columns. This assumption serves merely to accelerate dewatering, and is not critical for experimental verification.

Three different sands are utilized for the experiments. To provide a basis for later corroboration of the experiments by other researchers, A.S.T.M. Standard Ottawa sand is used. Also to provide information of practical value, sands from drying beds of two nearby treatment plants (Hermitage Hill, and Franklin, Tennessee) are tested. The sand types are coded as Type O, H, and F, and are sieve analyzed. After the sand surfaces in the columns are carefully leveled and compacted, the sand is saturated by introduction of water from beneath. The water level is maintained at the sand surface during charging of the columns with homogenous sludge mixtures to prevent any air entrapment.

Each of the two experiments (4 and 5) are conducted* with mixed digested sludges taken at different times from the Franklin activated sludge treatment plant. To prevent biological activity during the course of the experiments, the sludge is autoclaved at 110°C for 5 minutes, then tested for solids content (per Standard Methods), specific resistance, and coefficient of compressibility. The latter items are measured by the Buchner funnel, using Whatman No. 5 paper, and charged with 100 ml of sludge. Computer programs using least squares analyses are used to evaluate R and σ.

Within one hour of testing for the above indices, all nine columns for each experiment are charged with homogenous mixtures. When the piezometers are used, they are filled with water, and the rubber tubes are pinched until the columns are charged. A pinch cock (not shown in Figure 4) next to the bottom nozzle is then opened at t = 0 and volumetric readings are made at appropriate intervals. The graduated cylinders are removed, emptied, and replaced when needed. Data is stored on computer cards, allowing later use of the computer to evaluate the data per Equation 13, and by converting the filtrate volume to the drop in head.

Experimental Results

The characteristics of the sludges are as follows:

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*Earlier experiments (1 - 3) were designed to test consolidated sludges, which are not germane to the topic of this paper.

Experiment	Solids Content S_o(%)	Specific Resistance @ 15 in. Hg_2= 39.8 cm Hg (sec^2/gm)	Coefficient of Compressibility
4	3.70	$4 \cdot 8 \cdot 10^{10}$	0.63
5	2.78	$2 \cdot 1 \cdot 10^{10}$	0.64

Sieve analyses of the sands are reduced to terms of effective size (D_{10}) and the uniformity coefficient (D_{60}/D_{10}) in Table I.

Table I: Physical Characteristics of Supporting Sand:

Sand Source	Sand Designation	D_{10}(mm)	D_{60}/D_{10}	Media Factor
Franklin Treatment Plant	F	0.16	1.25	0.75
Ottawa Standard Sand	O	0.60	1.23	0.60
Hermitage Hills Treatment Plant	H	0.78	1.41	0.45

Piezometer readings, when taken, clearly show the validity of the assumed resistance to flow by an accumulating sludge cake on the sand interface. Results of the drainage tests are plotted as H versus t in Figures 5a - 5f. Equation 13 is used to create the theoretical curves shown, modified by the media factor determined from curve fitting. (Values for the media factor are listed in Table 1). Substitutions for the various factors are all in cgs units and include μ = 0.00919 gm/cm-sec (experimentally verified as identical to water at the same temperature). Experimental errors include the occasional presence of small amounts of entrapped air in the supporting media, and an irregular sludge-sand interface created by accidental turbulence during charging of the columns with sludge. Probably of greater importance is an initial higher drainage rate than predicted, resulting from initial retarded cake information due to some of the solids being drawn into the supporting media instead of being retained above the sand interface. The effect of the initially higher drainage rate is to lower the head values of an effected column, as seen for the Sand H runs, Figures 5d and 5f. For all 18 runs in experiments 4 and 5, the standard divergence between theoretical and experimental values is less than 5%.

Figure 5d should also be noted for the complete drainage at 6 days that occurs in the Sand H run. At this point the sludge cake has about 30% solids, the liquid interface has reached the sludge cake surface, and the menisci in the sludge cake resist the suction of the underlying water. The dewatering theory does not describe the final consolidation process of the sludge cake.

The values found for the media factor, and listed in Table I, exhibit the correct directional relationship with the corresponding D_{10} size; that is, the decrease in media factor (or drainage time) results from an increase in the sand size. Such an increase in sand size, however, inevitably increases the turbidity and color of the filtrate (see Figure 6).

Significance of Results and Conclusions

The theoretical formula (Equation 13) developed herein for describing gravity dewatering of wastewater sludge is found to be experimentally verified. Initial solids content, depth, specific resistance, coefficient of compressibility, and filtrate viscosity are clearly important parameters affecting the time required for dewatering. Experiments also prove the existance of a dimensionless media factor, m, which relates the sludge dewaterability on sands and other filter media to its dewaterability in the Buchner funnel. Also the following conclusions justified by this work should be noted:

1) Laboratory tests can predict dewatering rates on drying beds in the same manner that vacuum filter performance can be predicted, with aid of specific resistance determinations.
2) The effect of sludge conditioners on gravity dewatering can be predicted by ordinary laboratory specific resistance tests. From these tests the optimum dosage can be calculated.
3) Coarser types of drainage media may offer significant increases in drainage rates although at the cost of a more turbid filtrate.
4) An increase in specific resistance decreases sharply the rate of drainage. Evaluation of the net change in drainage rates produced by modification of the specific resistance and the coefficient of compressibility must be done by the use of the formula developed herein.

Although there exists a great deal of sludge conditioning data in the literature, the data are almost invariably expressed solely in terms of specific resistance, omitting any reference to the here-seen highly important coefficient of compressibility. It would therefore appear that new research is most clearly needed to determine the effect of conditioners, not only on specific resistance, but on the coefficient of compressibility as well. The net effect of conditioners must be evaluated through use of Equation 13.

The design engineer now has available a rational basis on which to predict from laboratory tests, gravity dewatering performance of drying beds and lagoons. The significance of this work should apply to the dewatering of other compressible materials such as water treatment and industrial sludges, liquid manures, and benthic deposits[12].

Acknowledgments

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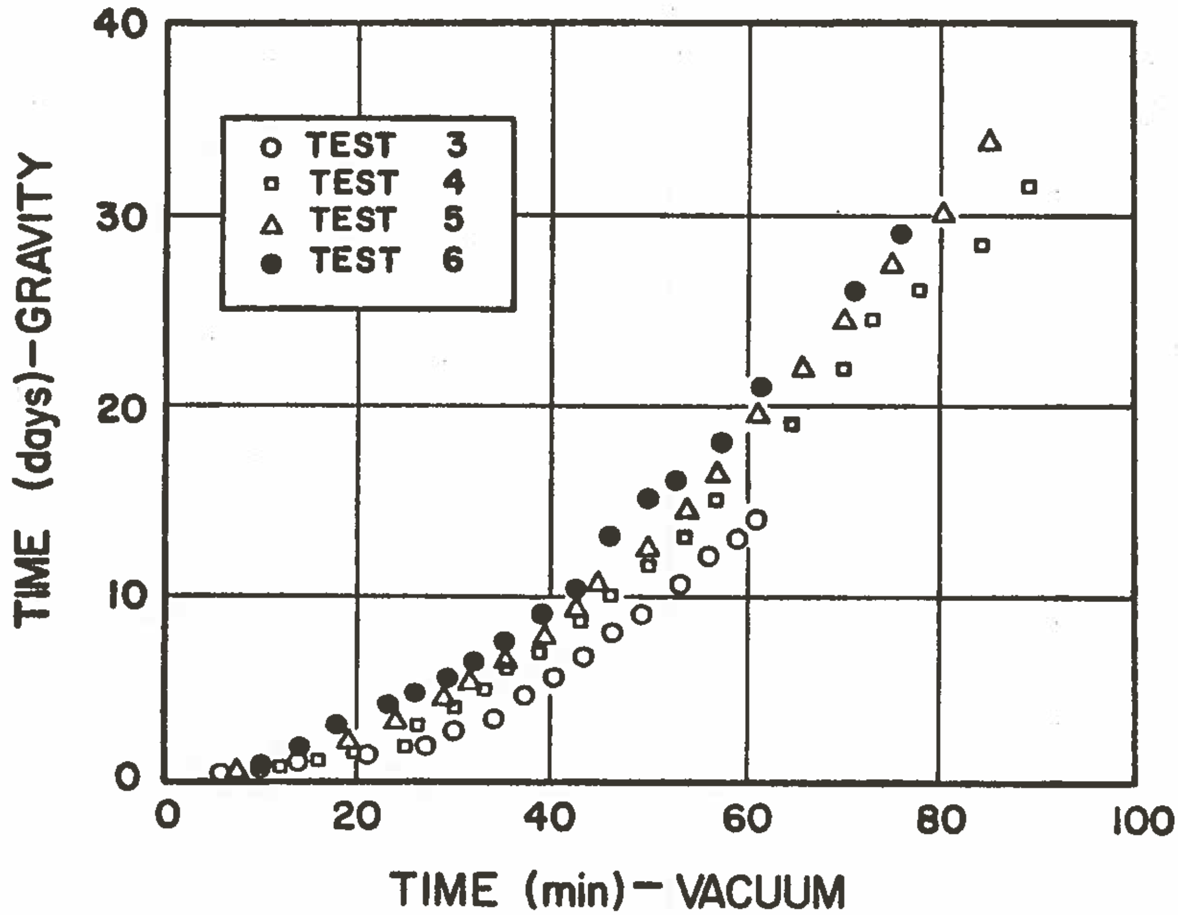

Figure 1: Correlation by Logsdon and Jeffrey[7] of gravity dewatering to vacuum filtration.

Each point represents a value of the relative amount of moisture removed, with the corresponding axial values designating the time required to attain this state by vacuum filtration and by gravity dewatering. The original solids contents varied from 2.08% to 3.13%. Maximum water removal was about 45%, and filling depths on the sand beds were 12 inches.

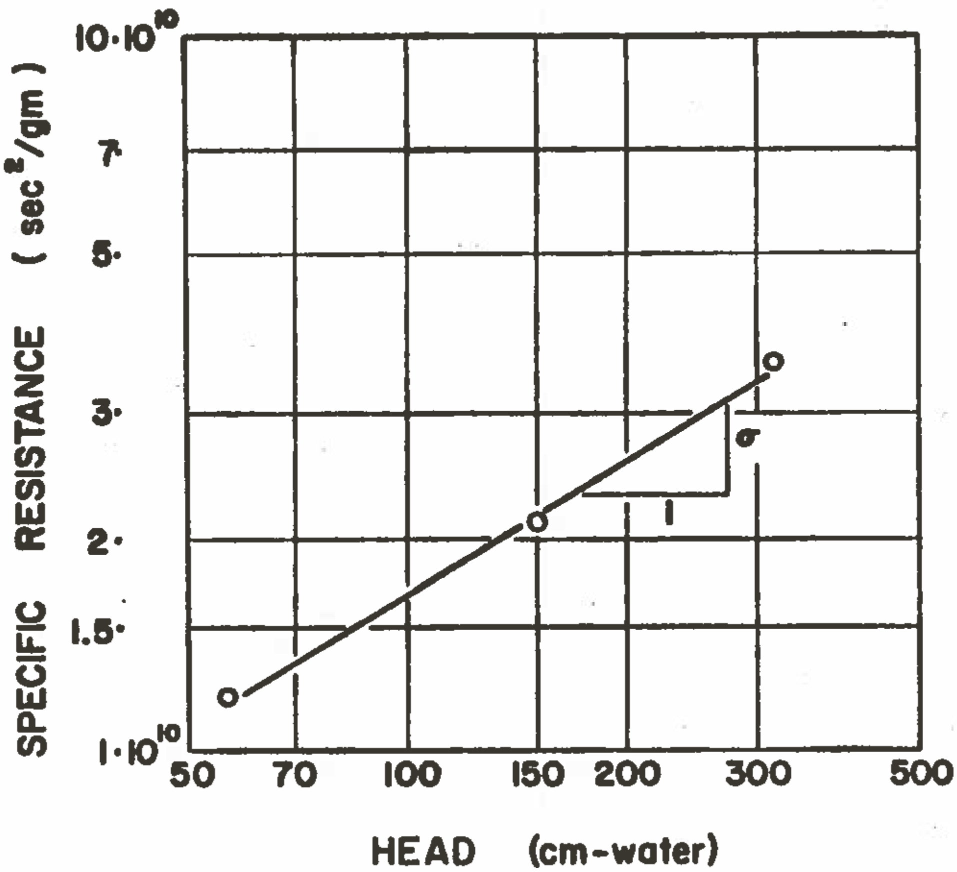

Figure 2: A plot of specific resistance versus the constant head at which sludge was vacuum filtered.

The slope of the line of best fit represents the coefficient of compressibility. Data is from Experiment 4.

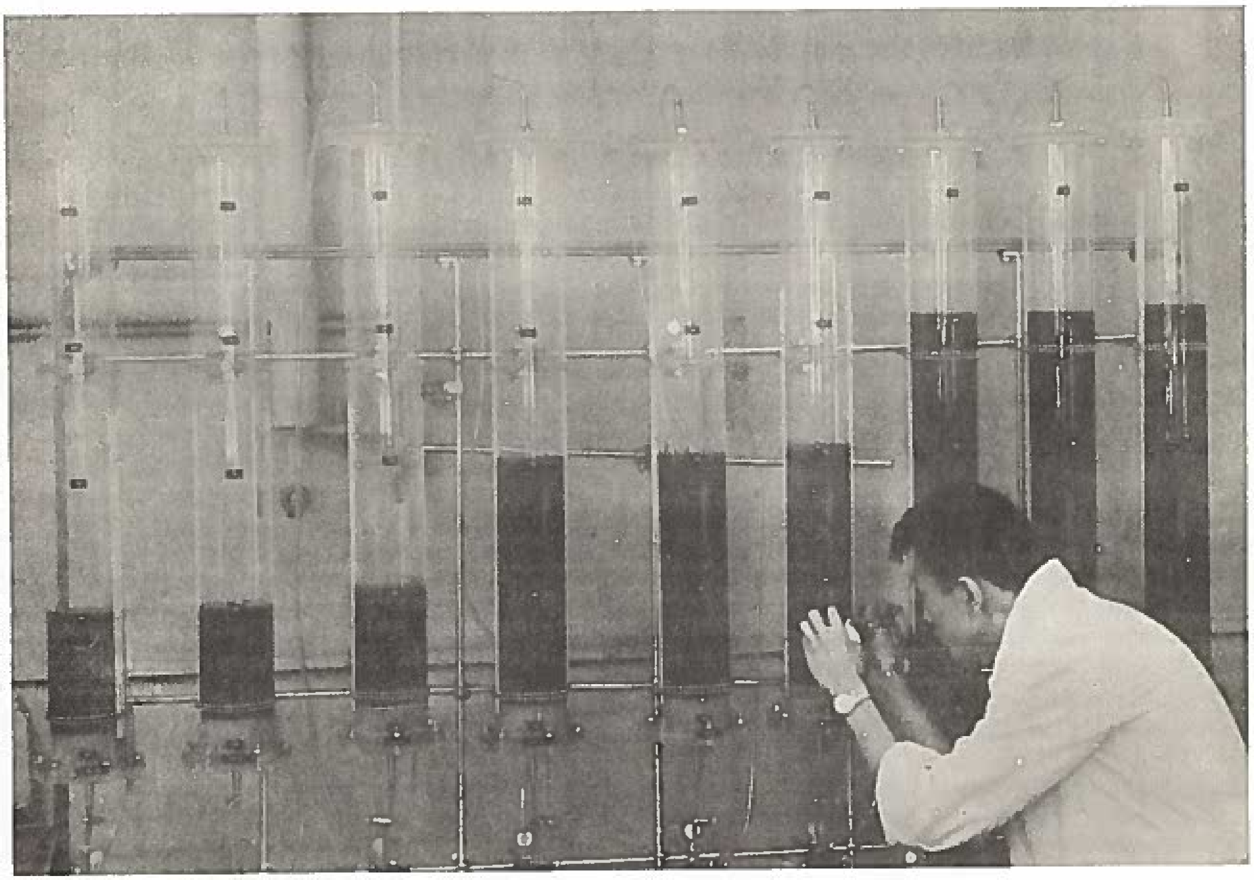

Figure 3: Gravity dewatering columns

The choice of nine columns is dictated by statistical considerations to allow simultaneous experiments with two variables. Here (Experiment 5) the variables include three types of sand and three different filling depths (11, 41, and 81 cm). A sludge solids-supernatant interface is being observed.

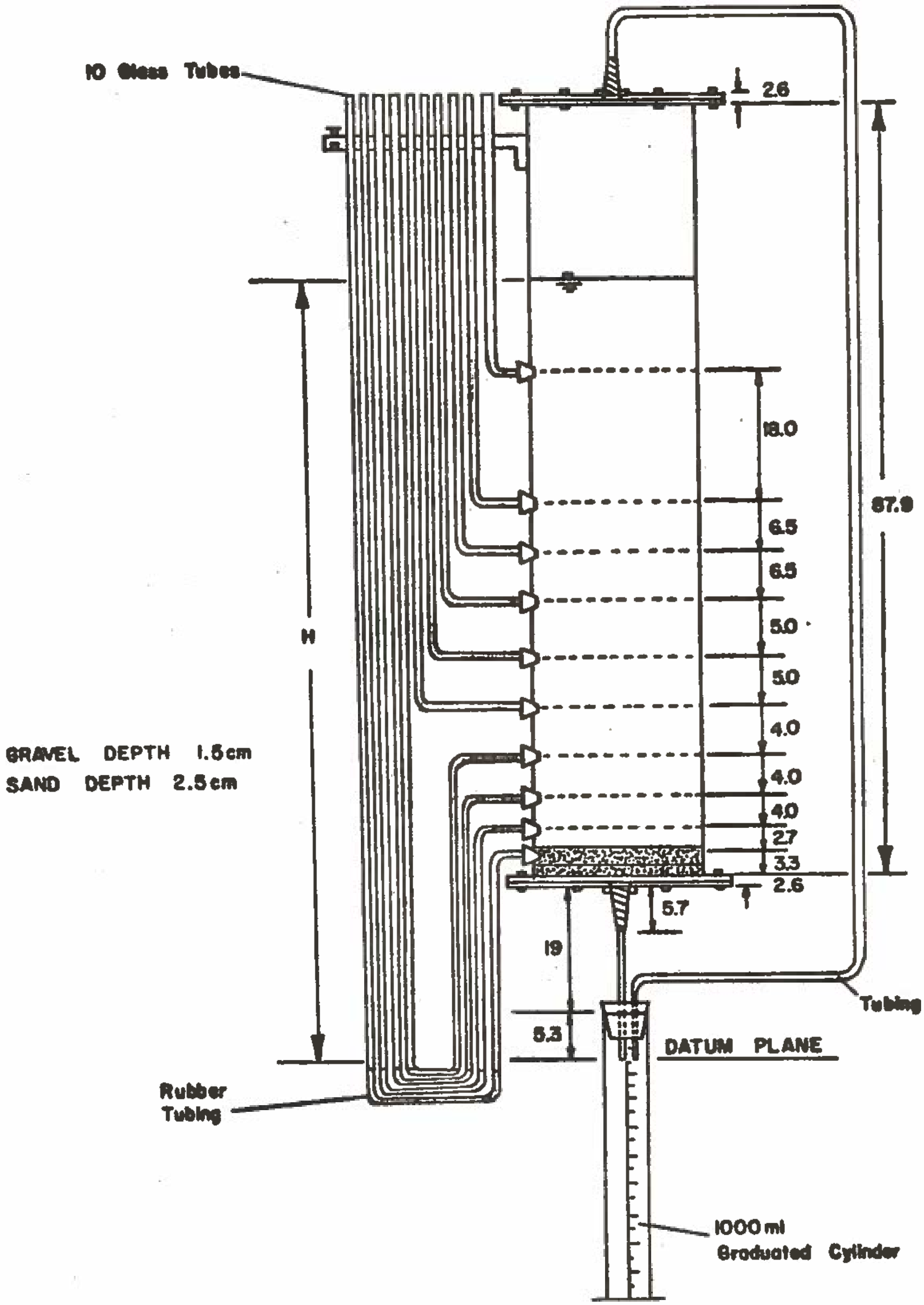

Figure 4: Gravity dewatering apparatus with piezometer tubes.

The supporting filter media consists of 5 cm of sand underlain by approximately 3 cm of coarse gravel. The top sand surface lies 34 cm above the datum plane. All dimensions are in cm.

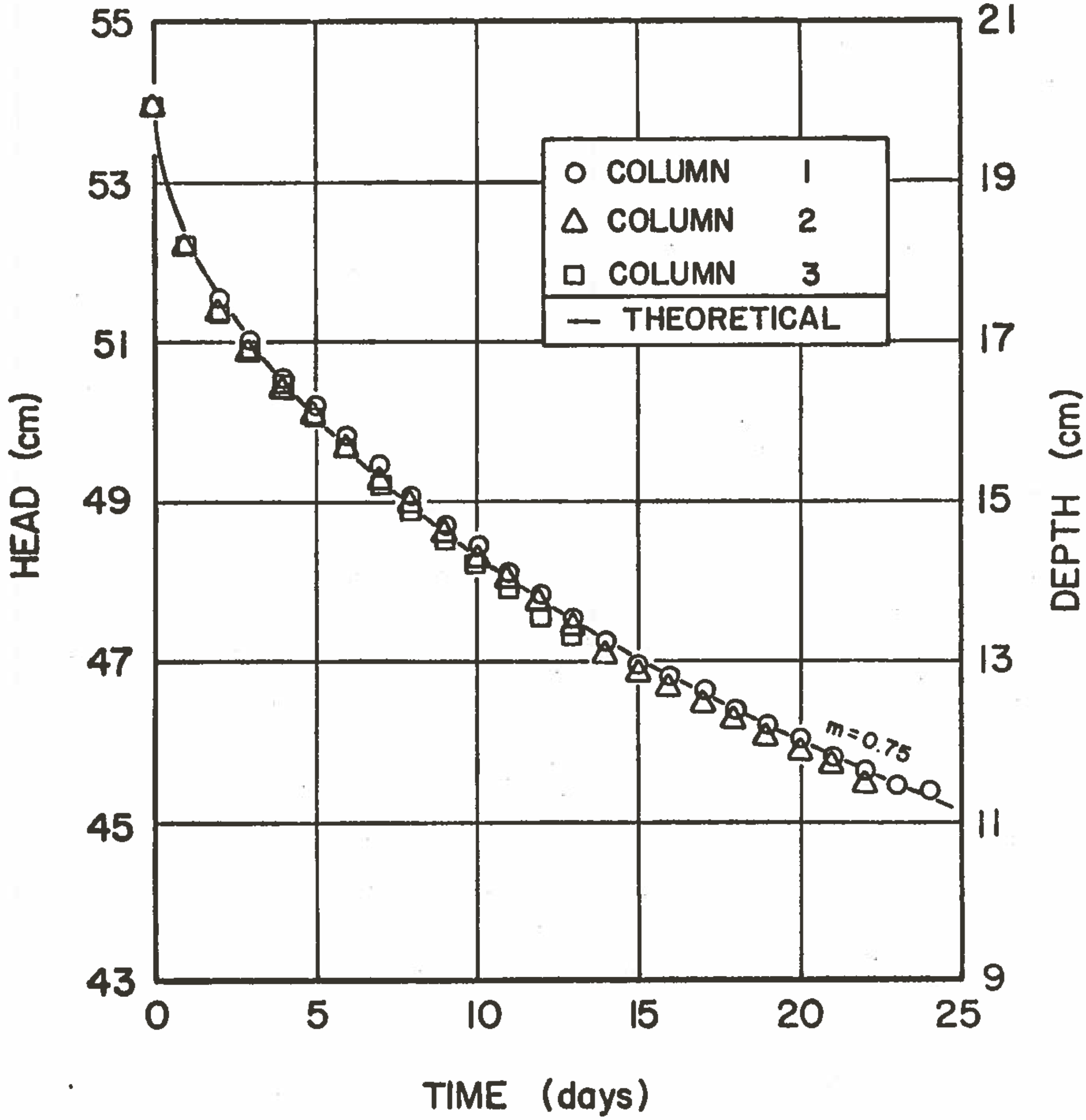

Figure 5a: Experiment 4. Triplicate test of 20 cm of sludge applied on Sand F.

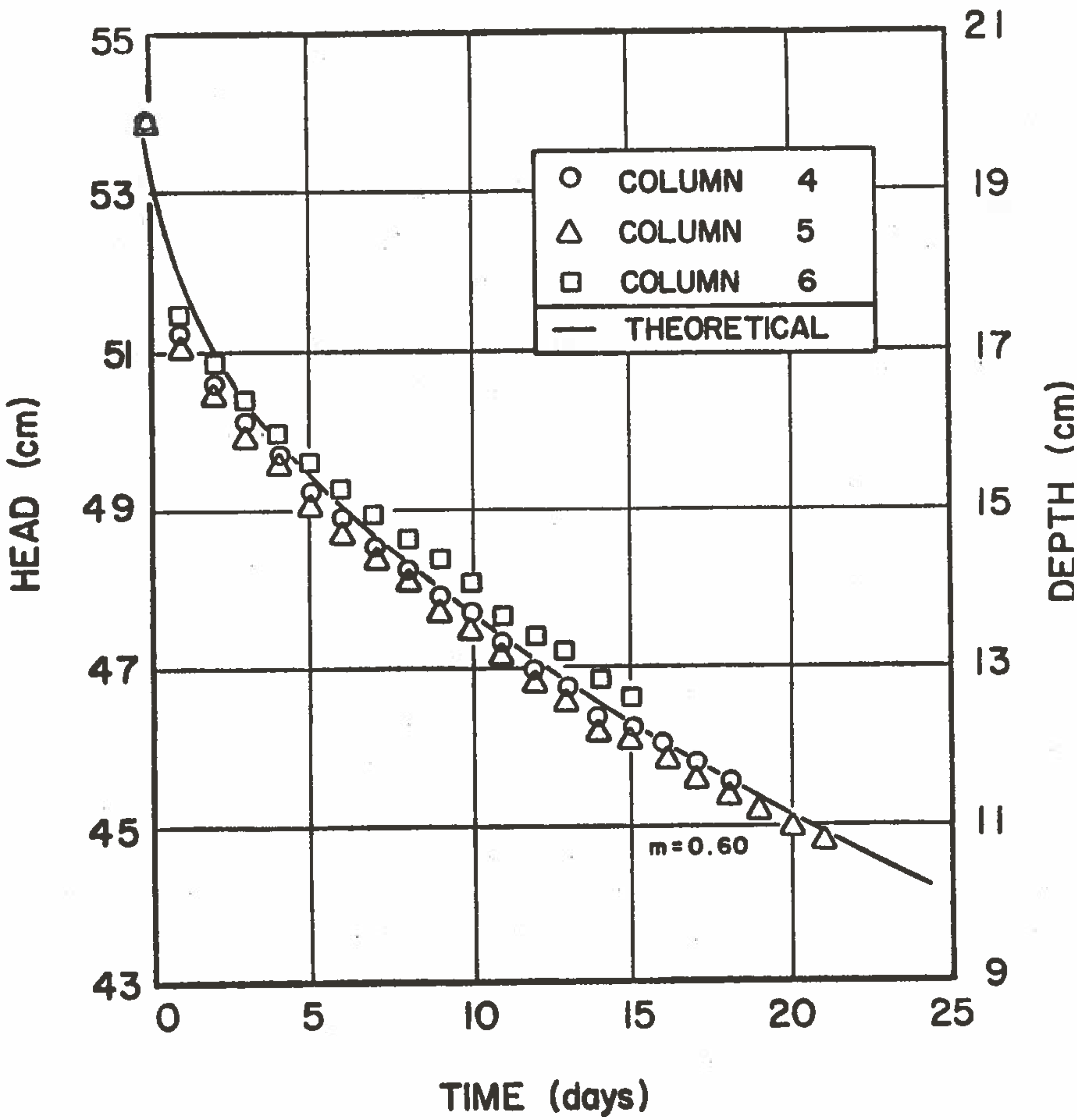

Figure 5B: Experiment 4. Triplicate test of 20 cm of sludge applied on Sand O.

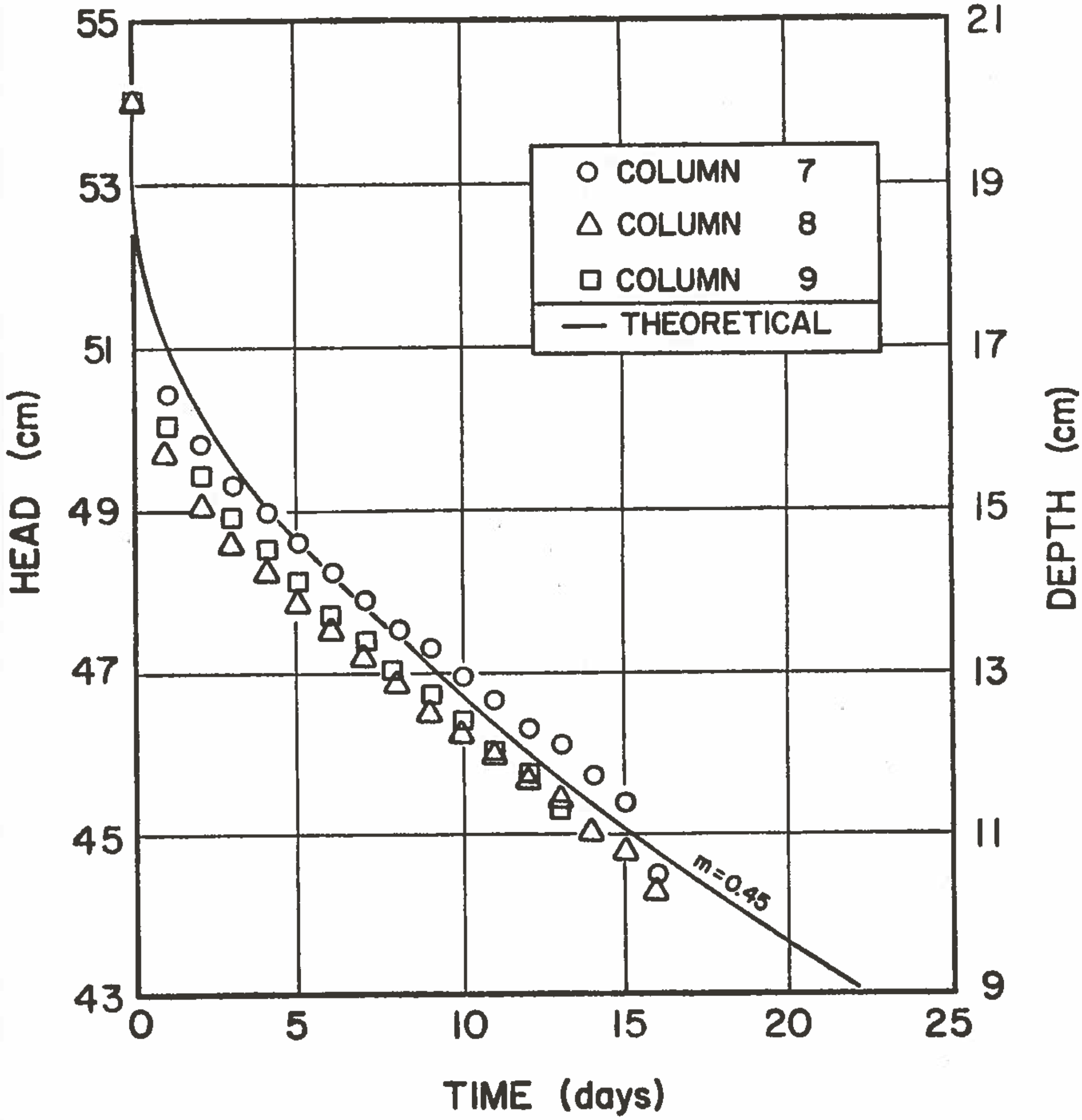

Figure 5c: Experiment 4. Triplicate test of 20 cm of sludge applied on Sand H.

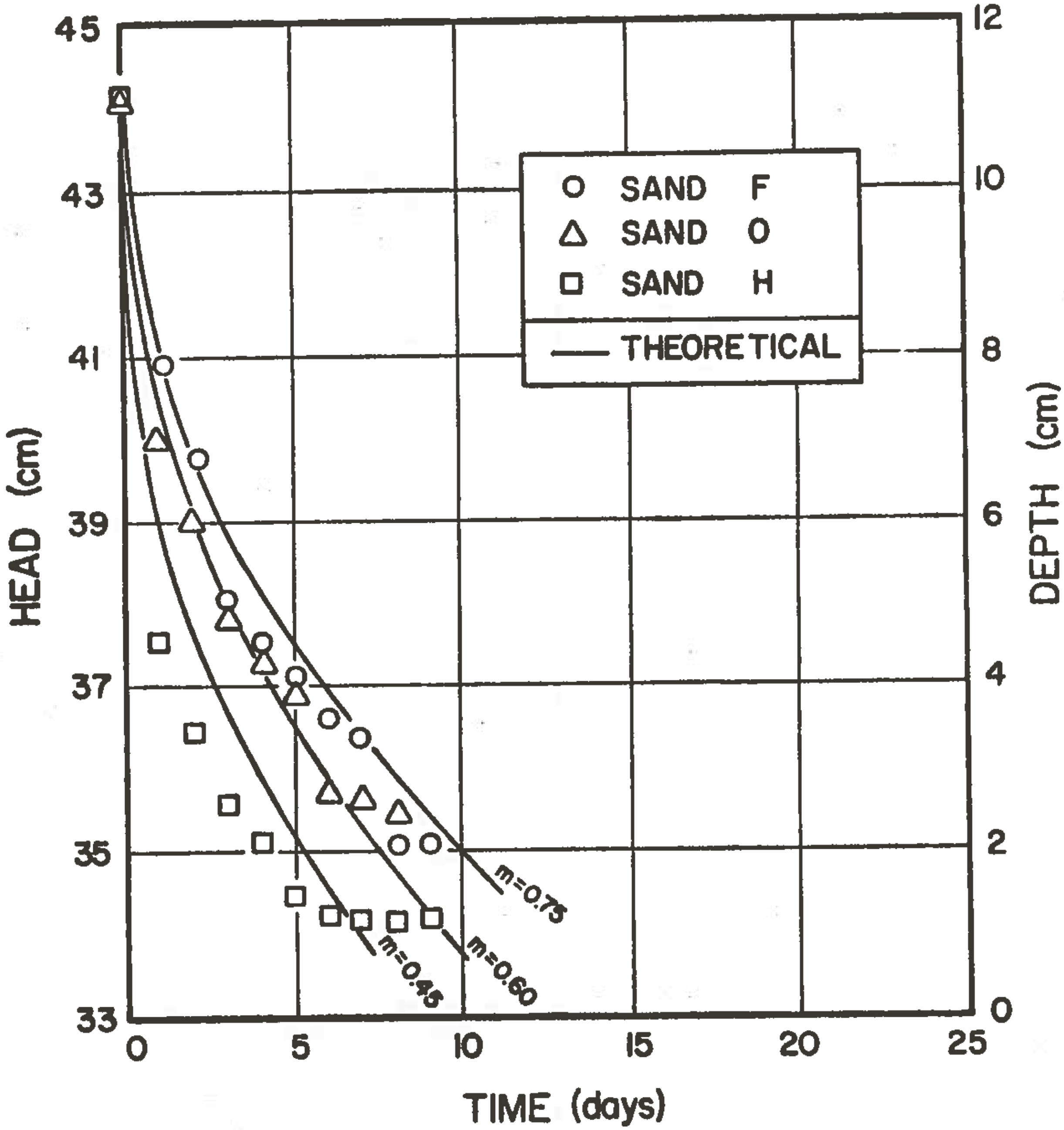

Figure 5d: Experiment 5. 11 cm of sludge applied on three different sands.

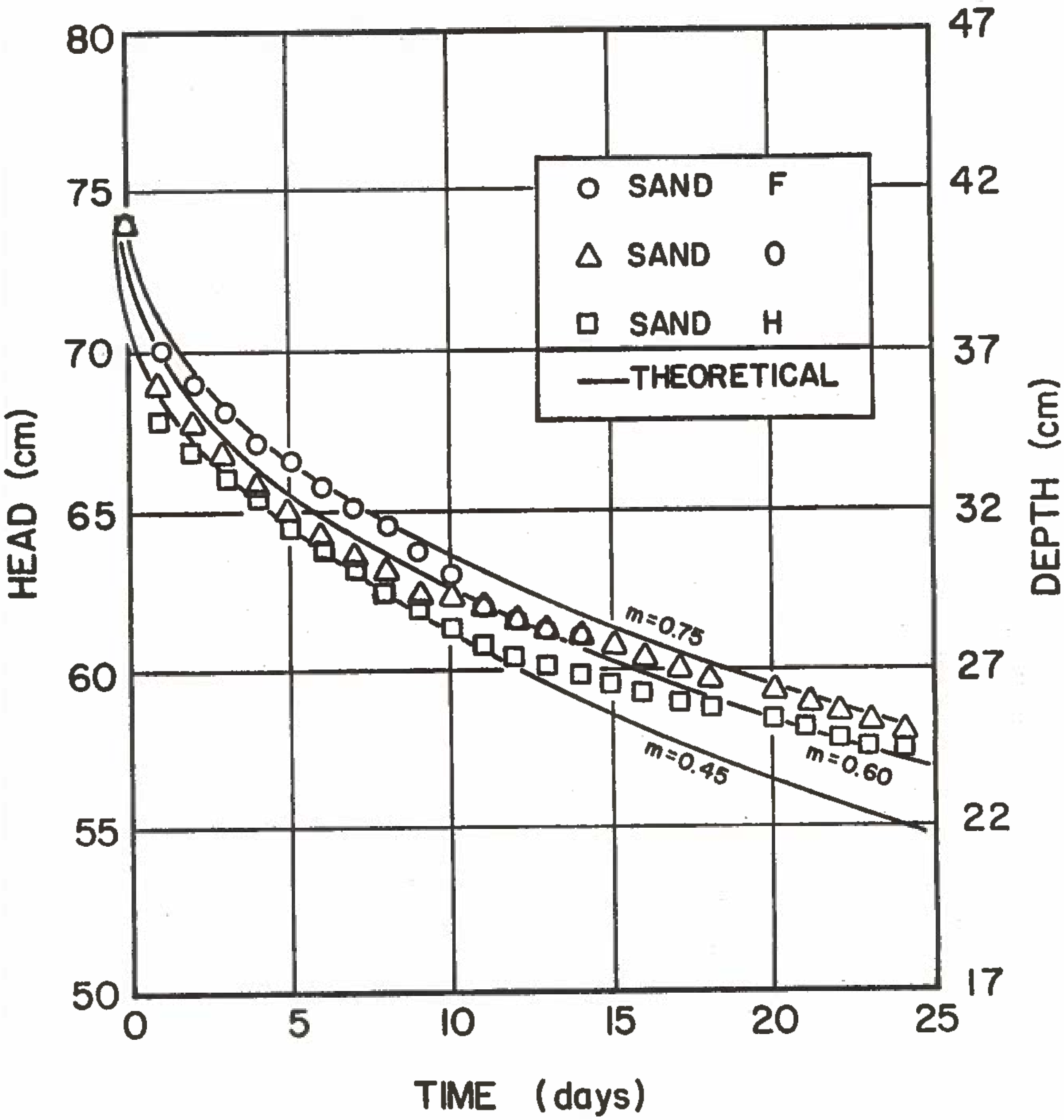

Figure 5e: Experiment 5. 41 cm of sludge applied on three different sands.

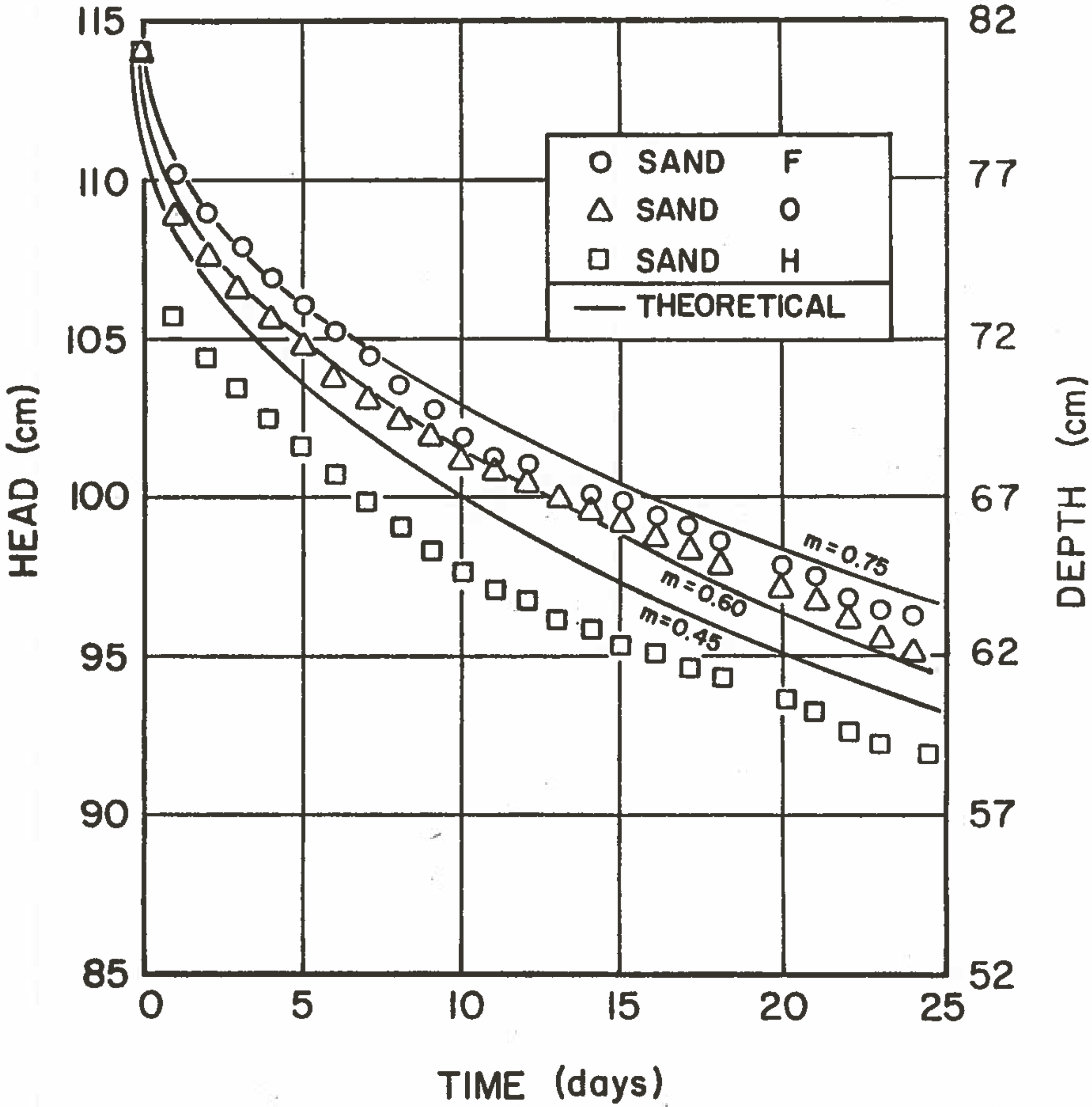

Figure 5f: Experiment 5. 81 cm of sludge applied on three different sands.

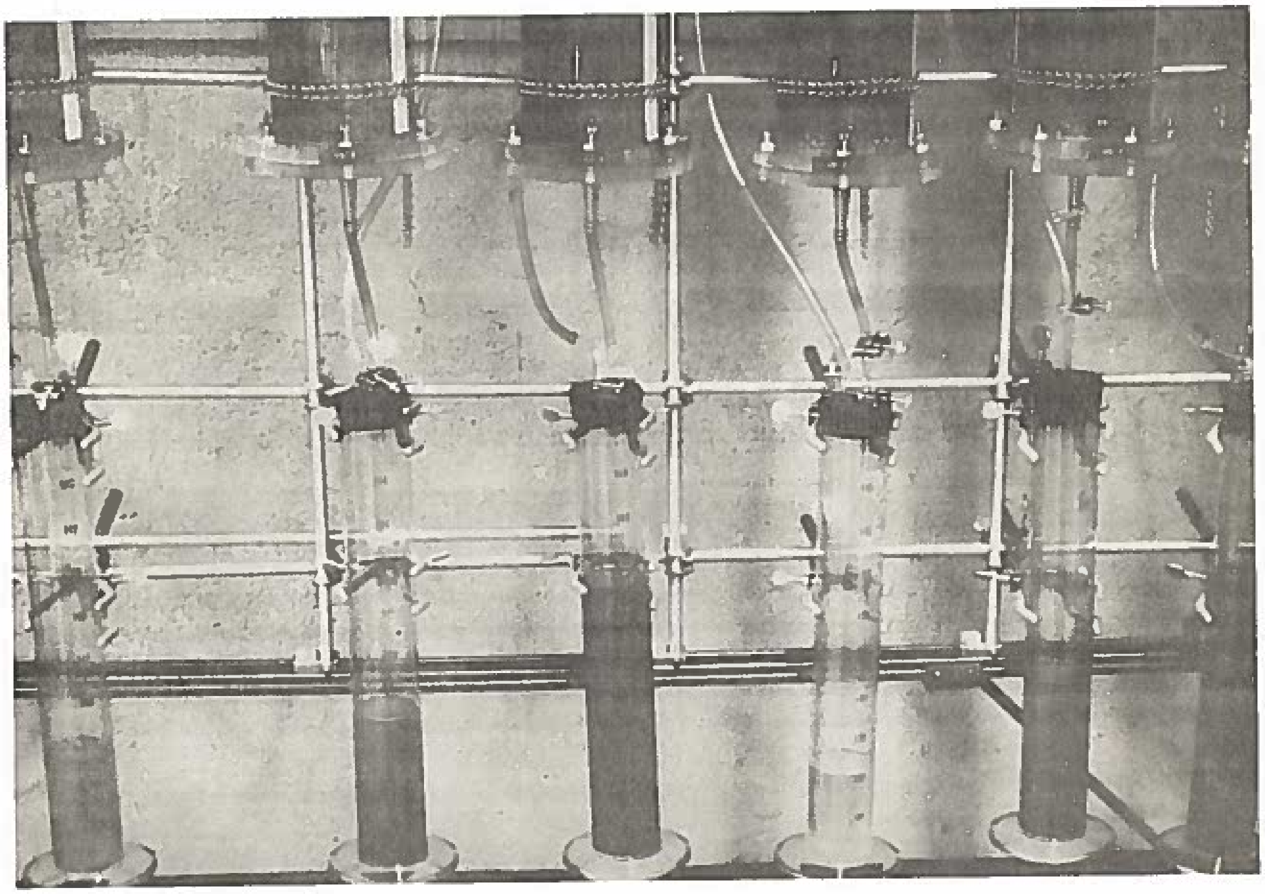

Figure 6: Filtrate collection during the first 3 days showing relationship of filtrate to supporting media (sand).

From left to right, the first and fourth graduated cylinders exhibit a small but clear volume of filtrate; the third and sixth, a larger volyme of filtrate but one of much higher turbidity. The second and fifth cylinders show intermediate results. Respectively, the sands used are types F, O, and H, representing effective sand sizes of 0.16, 0.60, and 0.78 mm.

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EXPERIMENTAL REFINEMENTS IN THE DETERMINATION OF SPECIFIC RESISTANCE AND COEFFICIENT OF COMPRESSIBILITY

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Abstract

Experimental Refinements in the Determination of Specific Resistance and Coefficient of Compressibility

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Equations have been derived and experimentally proven which describe the drainage rate from wastewater sludges on sand beds. Since specific resistance and coefficient of compressibility play an important role in the phenomenon of sludge drainage, studies have been undertaken with the intent of updating the methodology and experimental equipment used in the analyses.

Experimental testing for specific resistance has given rise to a methodology which produces highly replicable results and data reproducibility.

The use of modern computer techniques and equipment for data handling not only facilitates computations; but, virtually eliminates the human physical and judgment errors heretofore inherent in calculations for specific resistance and coefficient of compressibility.

The standard porcelain Buchner funnel, with a perforated filter disk, has been replaced by a transparent glass funnel which dewaters through a fritted glass disk. Since observation of the entire dewatering process is now possible, the effective filtration area is no longer an ellusive factor. Actual field sands may now be used as filter support media during specific resistance testing.

I. INTRODUCTION

Most recently, University of Massachusetts researchers derived and experimentally proved an equation describing the drainage rate from wastewater sludge on sand beds.[1] The equation recognized the following factors:

-- dynamic viscosity of filtrate

-- density of filtrate

-- acceleration of gravity

-- depth of sludge application to the bed

-- solids content of the applied sludge

-- specific resistance

-- coefficient of compressibility.

Only the last four factors are potentially variable. Depth is generally set in wastewater treatment at about 8 inches, to provide approximately monthly periods between cleaning. The solids content is variable, and is maximized to provide optimal dewatering. The specific resistance and coefficient of compressibility are operating variables which are capable of significant change by the addition of coagulants.

The effect of coagulants on gravity dewatering can be ascertained by laboratory tests in conjunction with the equation previously described. Use of this procedure, however, has been limited in the past due to the laborious routine and frequently erratic results using the classic specific resistance and compressibility test and evaluation methods outlined by Coackley and Jones.[2]

Swanwick et al[3] have proposed several refinements to the testing methods. Of primary concern was the determination of the filter area of a Buchner funnel. It was reasoned that the entire filter paper area was not the effective filter area because of the relatively wide spacing of the drain holes in the filter paper support plate of the funnel. Swanwick noted that the filter area appears as a squared term in the evaluation of specific resistance, and hence a faulty evaluation of area could lead to a significant error. He corrected the apparatus by placing a gauze wire mesh between the paper and plate and delineating the filter area using Perspex rings clamped on to the filter paper. However, no difference in results was found using this arrangement over the previous method. Swanwick did notice, however, a difference in specific resistance values using different types of filter paper.

The consulting firm of O'Brien and Gere[4] modified the testing procedure by placing diatomaceous earth onto the filter plate. The reasoning here apparently was that a more representative test for sand bed drainage could be made.

Baskerville and Gale[5] have attempted to reduce the time required to determine specific resistance. A simple automatic capillary suction apparatus is used for which the readings indicate sludge filtrability. These readings can be correlated with specific resistance units. However, accuracy was apparently low, and no method for coefficient of compressibility determination was included.

The approach of O'Brien and Gere[4] appears to have considerable merit for measuring specific resistance and coefficient of compressibility of sludges to be gravity drained. Nebiker, Sanders, and Adrian[1] noted that the classic Buchner funnel results required a correlation factor for use with their gravity dewatering equation. This factor, the so-called media factor, approximated 75%, and depended on the characteristics of the filter in the Buchner funnel test, and on the characteristics of the supporting media used in gravity dewatering. A closer analysis of the media factor appeared justified. A promising procedure would be to test the actual drainage media along with the sludge in the Buchner funnel apparatus. This may then allow elimination of the media factor in the computations required to predict gravity dewatering rates.

II. MATERIALS AND METHODS

The shallow artesian raw water supply for the Town of Amesbury is high in iron content. Treatment includes: aeration followed by manually cleaned slow sand filters, mechanical addition of alum and caustic soda, settling in a four-compartment manually cleaned basin, and chlorination. The sedimentation basins are emptied of accumulated sludge twice annually. This sludge was used throughout the experiments.

The techniques employed during this study are outlined in detail by Adrian, Lutin, and Nebiker[6]; and were perfected on the apparatus shown in Figure 1. Note that the funnel is the standard Buchner type which is classically constructed of porcelain and drains through a perforated filter disk.

The results of preliminary filtrations made through the standard funnel with a dilute slurry of sludge indicated a concentration of solids on the filter paper around each individual perforation in the filter disk. Since, as previously described, this was not a true representation of

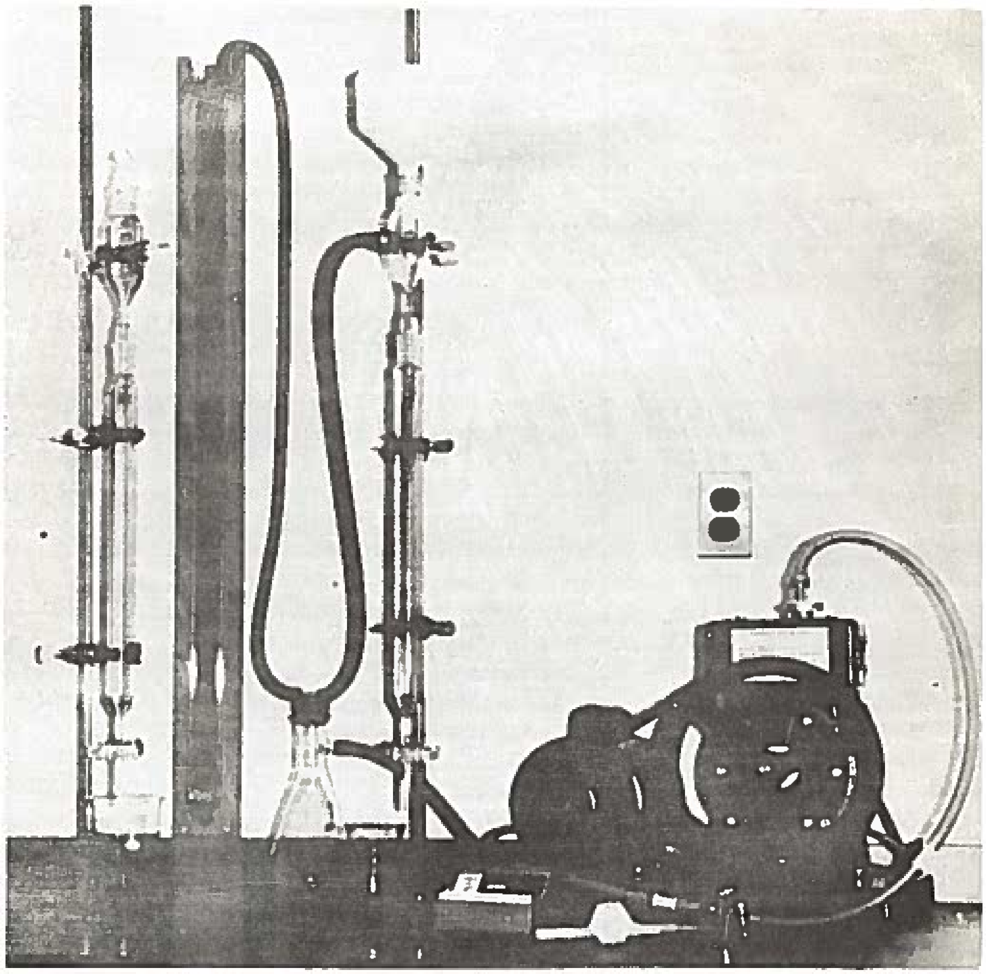


Figure 1. Standard Buchner funnel apparatus set up for specific resistance testing.

gravity dewatering, alternative filtration equipment was sought. A simple solution was found by substituting the funnel shown in Figure 2. Note that in place of the perforated filter disk, a fritted glass disk is used which allows filtration across the entire filter surface, thus eliminating the localized phenomenon observed in the standard funnel. The fritted funnels are available in a wide range of pore sizes and require only the addition of a male standard taper ground glass joint for compatability with the equipment shown in Figure 1. An additional advantage of the fritted funnel shown is the transparency of the filter walls which allows observation of the entire filtration process.

Figures 3, 4, and 5 are included to depict the technique employed in specific resistance testing on bulk filter media. The fritted funnel is filled to a predetermined depth with media as shown in Figure 3. Here, 20 - 30 mesh Ottawa sand is the support. The sand is then washed, repacked, and flooded with sludge as shown in Figure 4. A vacuum is then applied to the system. With a coarse media such as Ottawa sand, there is likely to be some penetration of sludge into the media at higher vacuums. The sand depth is experimentally adjusted to insure against sludge penetration through the filter disk (Figure 5).

Figure 2. Fritted glass funnel. Note the 70-100μ pore disk through which total area filtration is accomplished. (horizontal markings at 2 cm and 5 cm above the disk)

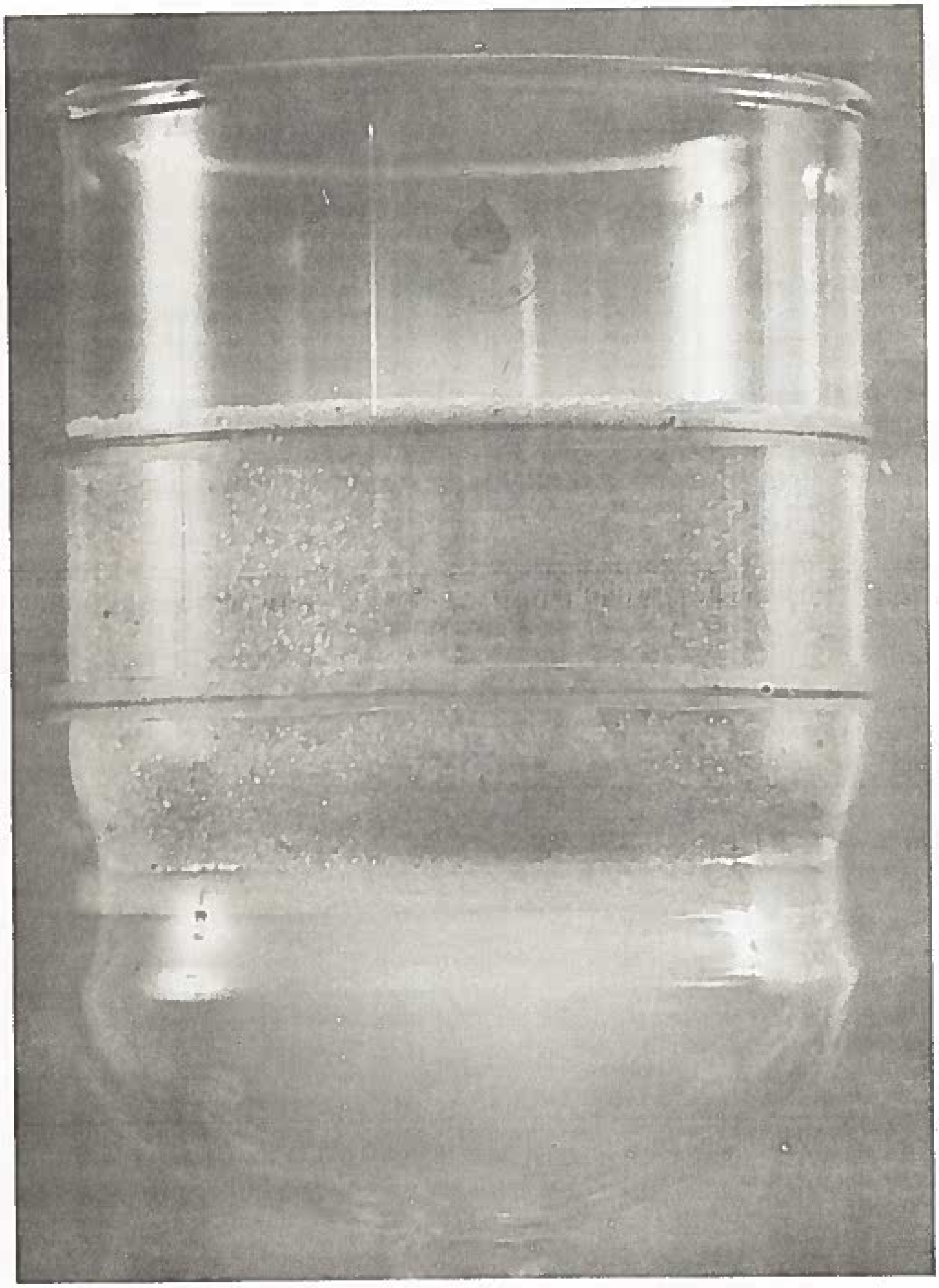


Figure 3. Fritted glass funnel with 20-30 mesh Ottawa sand to a depth of 5 cm above the disk.

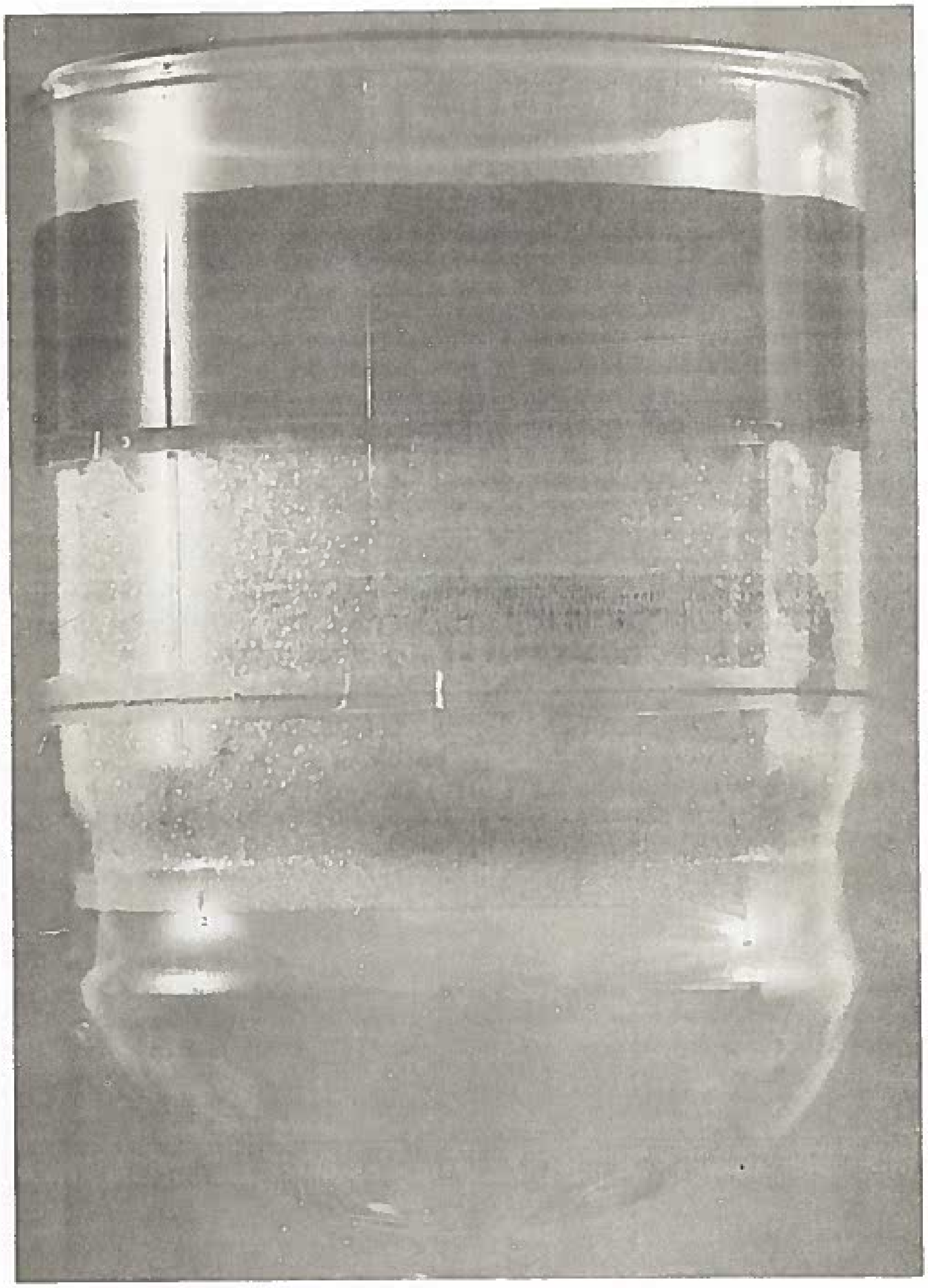


Figure 4. Fritted glass funnel with sludge ready for dewatering placed over the Ottawa sand filter bed.

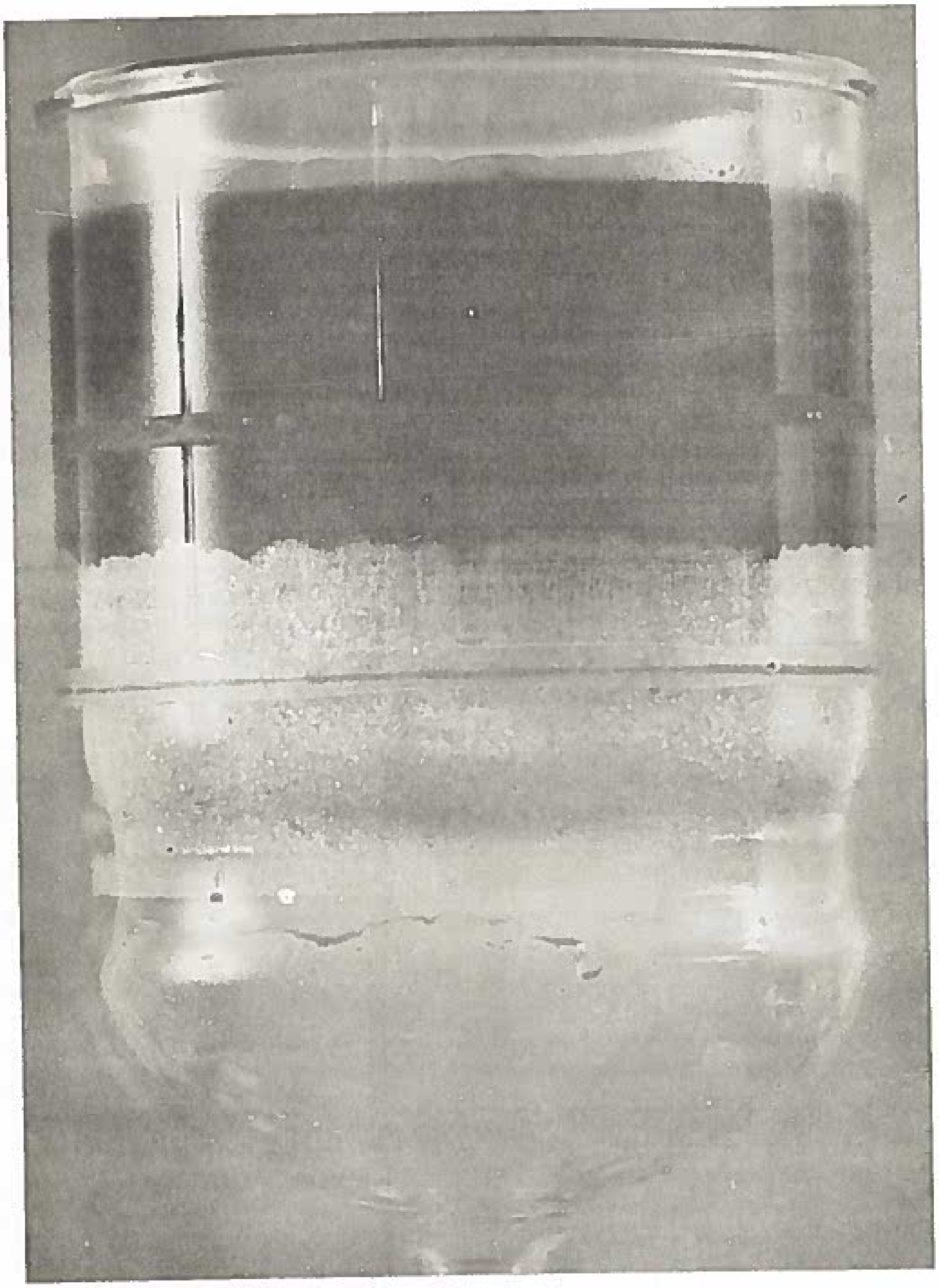


Figure 5. During specific resistance testing, the sludge partially penetrates the sand filter. The sand depth has been set at 5 cm to insure no sludge penetration through the filter disk.

CALCULATIONS

To minimize tedium, the following UMASS Fast Fortran program was written to compute the slope of the t/v-versus v curve,[6] the t/v intercept and the data correlation coefficient:

Least Squares Slope and Regression Analysis Written in UMASS Fast Fortran for Remote Console Operation

```
0010  PROGRAM SLOPE
0020  DIMENSION VOL(61), TOV(61)
0030  K = 0
0032  PRINT 34
0034  FORMAT ( / / / * ENTER DATA * /
0035A * 1 VALUE OF TIME AND VOLUME PER LINE * / )
0040  K = K +1
0050  INPUT, T, VOL(K)
0060  IF ( T .EQ. 0.0 .AND. VOL(K) .EQ. 0.0 ) 90, 70
0070  TOV(K) = T / VOL(K)
0080  GO TO 40
0090  K = K -1
0100  PRINT 0110
0110  FORMAT ( 2X, *NO.*, 7X, *VOLUME*, 8X, *T/VOL*, 6X, *NO.*/)
0114  DO 118 I - 1,K
0116  PRINT 125, I, VOL(I), TOV(I), I
0118  CONTINUE
0125  FORMAT ( 1X, 14, 5X, 2(F8.1,5X), 14 )
```

```
0127  CALL PLOT ( VOL, TOV, 1, K )
0130  PRINT 0140
0140  FORMAT ( // * ENTER SLOPE START AND SLOPE FINISH* )
0150  INPUT, N1, N2
0160  PRINT 170, VOL(N1), TOV(N1), VOL(N2), TOV (N2)
0170  FORMAT ( * REQUESTED RANGE IS AS FOLLOWS * /
0171A * FROM VOL = *, F7.1, 10X, * TOV = *, F7.1 /
0172A *   TO VOL = *, F7.1, 10X, * TOV = *, F7.1 )
0180  PRINT 0190
0190  FORMAT ( * IF THIS IS CORRECT ENTER 1, OTHERWISE 2 * )
0200  INPUT, NGO
0210  GO TO (220, 130), NGO
0220  SUMX = 0.0
0230  SUMY = 0.0
0240  SUMXQ = 0.0
0250  SUMYQ = 0.0
0260  SUMXY = 0.0
0270  DO 330 I = N1, N2
0280  SUMX = SUMX + VOL(I)
0290  SUMY = SUMY + TOV(I)
0300  SUMXQ = SUMXQ + VOL(I) * VOL(I)
0310  SUMYQ = SUMYQ + TOV(I) * TOV(I)
0320  SUMXY = SUMXY + VOL(I) * TOV(I)
0330  CONTINUE
0340  SMXSMY = SUMX * SUMY
0350  ZN - N2 - N1 + 1
```

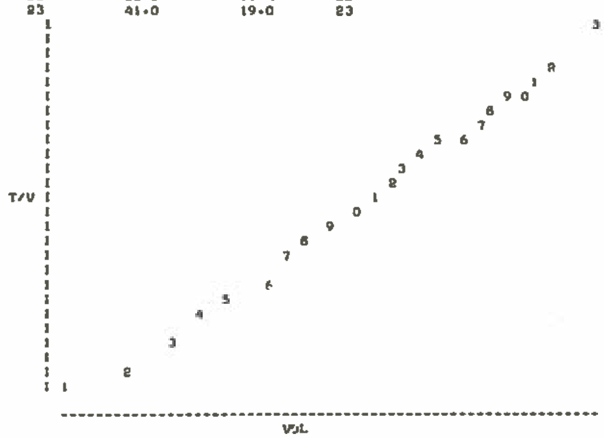
```
0360  XDOT = SUMX / ZN
0370  YDOT = SUMY / ZN
0380  SXX = SUMXQ - ( ( SUMX*SUMX ) / ZN )
0385  SYY = SUMYQ - ( ( SUMY*SUMY ) / ZN )
0390  SXY = SUMXY - ( SMXSMY / ZN )
0400  B = SXY / SXX
0410  A = YDOT - ( B*XDOT )
0415  R = SQRTF( B*B * SXX / SYY )
0420  PRINT 430
0430  FORMAT ( 8X, 1HB, 13X, 1HA, 13X, 1HR)
0440  PRINT 445, B, A, R
0445  FORMAT ( 1X, 3(F10.5, 4X) / / )
0446  PRINT 447
0447  FORMAT ( * TO INPUT NEW RANGE, ENTER 1 * /
0448A * TO INPUT NEW TABLE, ENTER 2 * /
0449A * TO TERMINATE RUN, ENTER 3 * )
0450  INPUT, KGO
0452  GO TO ( 130, 30, 460 ), KGO
0460  STOP
0465  END
2001  SUBROUTINE PLOT ( X, Y, N1, N2 )
2010  DIMENSION X(61), Y(61), NX(61), NP(61)
2020  XMIN = X(N1)
2030  XMAX = X(N1)
2040  YMIN = Y(N1)
2050  YMAX = Y(N1)
```

```
2060  N = ( N2 - N1 ) + 1
2070  DO 2180 I = N1, N2
2080  IF ( X(I) .LT. XMIN ) 2090, 2110
2090  XMIN = X(I)
2100  GO TO 2130
2110  IF ( X(I) .GT. XMAX ) 2120, 2130
2120  XMAX = X(I)
2130  IF ( Y(I) .LT. YMIN ) 2140, 2160
2140  YMIN = Y(I)
2150  GO TO 2180
2160  IF ( Y(I) .GT. YMAX ) 2170, 2180
2170  YMAX = Y(I)
2180  CONTINUE
2190  XDIF = XMAX - XMIN
2200  YDIF = YMAX - YMIN
2220  DO 2250 I = N1, N2
2230  NX(I) = ( ( ( X(I) - XMIN ) / XDIF ) * 60.0 ) + 1.5
2240  NY(I) = ( 25.0-(((Y(I)-YMIN)/YDIF)*25.0)) + 1.5
2250  CONTINUE
2260  DO 2500 J = 1,26
2270  DO 2340 I = 1,61
2275  NP(I) = 1H
2280  DO 2330 K = N1, N2
2290  IF ( NX(K) .EQ. I .AND. NY(K) .EQ. J ) 2300, 2330
2300  CALL TRANS ( K, NP(I) )
2310  GO TO 2330
2330  CONTINUE
2340  CONTINUE
```

```
2350  IF ( J . EQ. 13 ) 2360, 2390
2360  PRINT 2370, NP
2370  FORMAT ( * T/V I *, 61A1 )
2380  GO TO 2500
2390  PRINT 2400, NP
2400  FORMAT ( 5X, 2HI , 61A1 )
2500  CONTINUE
2510  PRINT 2520
2520  FORMAT ( / 7X, 61(1H-) / 32X, *VOL* / )
2530  RETURN
2540  END
3001  SUBROUTINE TRANS ( NX, NP )
3010  DIMENSION LIST(10)
3020  DATA ( LIST = 1H1, 1H2, 1H3, 1H4, 1H5, 1H6, 1H7, 1H8, 1H9, 1H0 )
3030  NX1 = ( NX / 10 ) * 10
3040  NXD = NX - NX1
3050  IF ( NXD .EQ. 0 ) 3060, 3070
3060  NXD = 10
3070  NP = LIST(NXD)
3080  RETURN
3090  END
3100  ENDPROG
```

Figure 6 is a photograph of the actual data output from the remote console Teletype shown in Figure 7. Note the plotting algorithm employed which assigns each set of values a number (instead of an "x" or "dot") and plots that number in correct position on the graph. Another unique function of the program is the great variety of data ranges which may be selected for instantaneous computations. Before plotting begins, the computer scans the data, normalizes it, and expands to include full scale width on each axis. The slope "B" is in units of sec/ml^2 or [T/V^2].

O. O. NO.	VOLUME	T/VOL	NO.
1	5.0	6.0	1
2	9.0	6.7	2
3	12.0	7.5	3
4	14.0	8.6	4
5	16.0	9.4	5
6	19.0	9.5	6
7	20.0	10.5	7
8	21.0	11.4	8
9	23.0	11.7	9
10	25.0	12.0	10
11	26.0	12.7	11
12	27.0	13.3	12
13	28.0	13.9	13
14	29.0	14.5	14
15	30.0	15.0	15
16	32.0	15.0	16
17	33.0	15.5	17
18	34.0	15.9	18
19	35.0	16.3	19
20	36.0	16.7	20
21	37.0	17.0	21
22	38.0	17.4	22
23	41.0	19.0	23




```
ENTER SLOPE START AND SLOPE FINISH

?



4 22
REQUESTED RANGE IS AS FOLLOWS
FROM VOL =     14.0            TOV =      8.6
  TO VOL =     38.0            TOV =     17.4
IF THIS IS CORRECT ENTER 1, OTHERWISE 2

?
1
          B              A             R
     .37841        3.06853        .99307

TO INPUT NEW RANGE, ENTER 1
TO INPUT NEW TABLE, ENTER 2
TO TERMINATE RUN, ENTER 3
```

Figure 6. Data output from remote console Teletype.

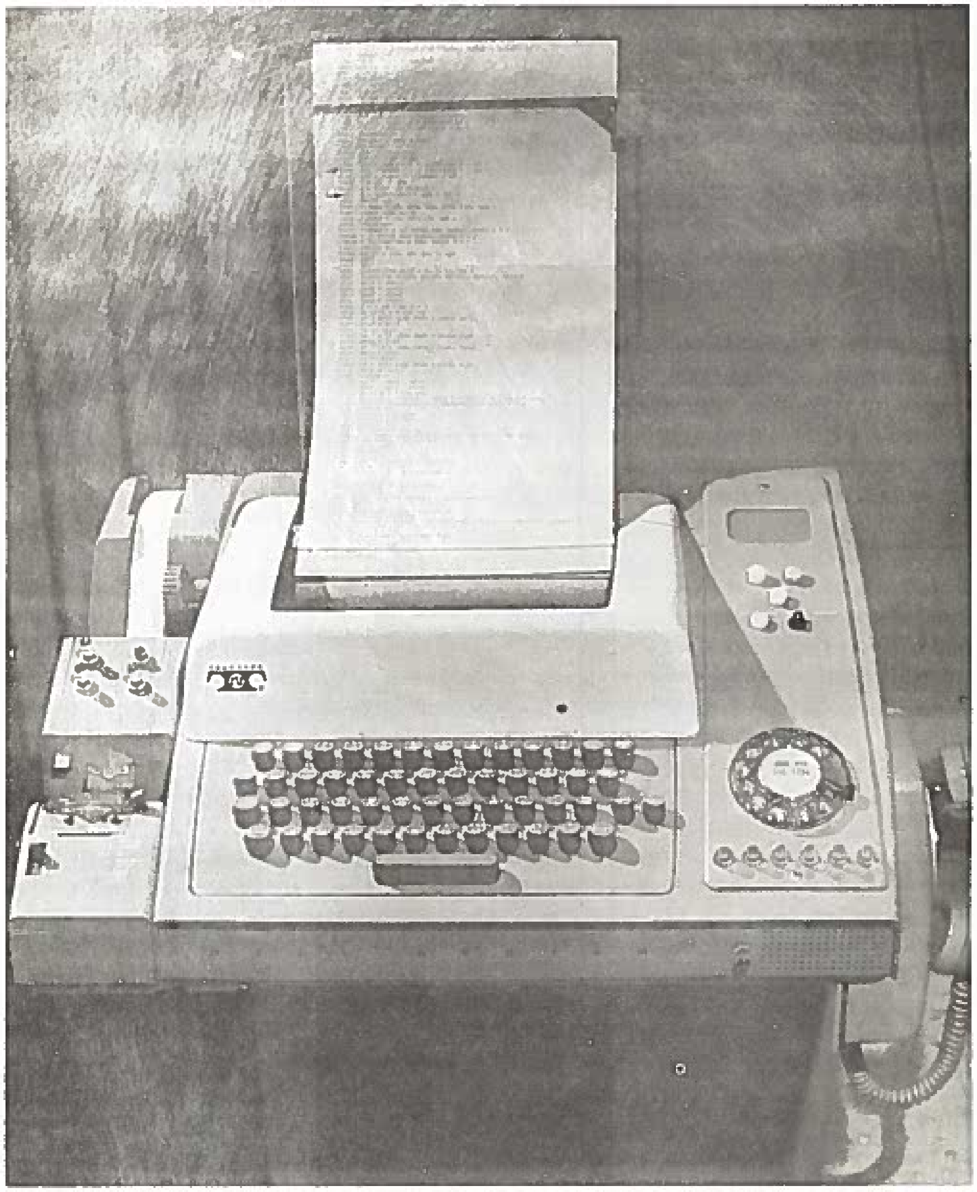


Figure 7. Remote console Teletype located adjacent to the filtration apparatus within the laboratory.

Specific resistance calculations were facilitated with the following UMASS Fast Fortran Program which utilized the formula

$$R = \frac{2bA^2\Delta P}{\mu\left(\dfrac{\rho g}{\left(\dfrac{1.0-S_o}{S_o}\right)-\left(\dfrac{1.0-S_f}{S_f}\right)}\right)}$$

```
5    PROGRAM SR
10   READ 100, RHO
15   READ 101, SO, SF
20   G=980.0
25   C=RHO*G/((1.0-SO)/SO-(1.0-SF)/SF)
30   READ 102, T
35   Z=1.0/(2.1482*((T-8.435)+SQRT(8078.4+(T-8.435)**2))-120.0)
40   READ 103, B, P
45   A=96.77
50   Y=133332.2
55   R=(2.0*A**2*B*P*Y)/(Z*C)
60   PRINT 104, R
100  FORMAT (F10.7)
101  FORMAT (2F9.7)
102  FORMAT (F9.5)
103  FORMAT (2F8.4)
104  FORMAT (E13.6)
105  END
110  ENDPROG
```

The nomenclature is as follows:

RHO*G = ρg = (density of filtrate) (acceleration of gravity) [gm/cm^2·sec^2]

SO = s_o = initial sludge solids concentration (decimal)

ST = s_f = final cake solids concentration (decimal)

C = c = weight of solids per unit volume of filtrate [gm/cm^2·sec^2]

T = test temperature (°C)

Z = μ = dynamic viscosity [dnye·sec/cm^2]

B = b = slope of t/v versus v curve [sec/ml^2]

P = Δp = test pressure [dynes/cm^2]

R = R = specific resistance [sec^2/gm]

A = A = area of the filter surface [cm^2]

Y = conversion of Δp in cm Hg to Δp in dynes/cm^2

"A", the area of the funnel, must be corrected when other than 111 mm. diameter funnels are used. R has dimensionally correct units of (sec^2/gm) or [T^2/M].

The coefficient of compressibility was calculated from the log - log relationship[6] of R and ΔP by means of the following Fortran IV Program:

```
      PROGRAM COEF
      DIMENSION R(100), P(100), PL(100), RL(100), RY(100),
     1          RH(100), RLO(100)
   22 READ 12,K
      DO 45 JI=1,K
      READ 12, N
   12 FORMAT (13)
    1 SUMX=0.0
```

```
      SUMY=0.0
      SUMXQ=0.0
      SUMYQ=0.0
      SUMXY=0.0
      PRINT 41
   41 FORMAT (*ENTER DATA, PRES. AND SP. RESIST.* /)
    2 DO 13 I=1,N
    3 READ 4, P(I), R(I)
    4 FORMAT (F5.1, E13.6)
   13 PRINT 4, P(I), R(I)
      PRINT 4,
      DO 9 I=1,N
      PL(I)=LOGF(P(I))
      RL(I)=LOGF(R(I))
      SUMX=SUMX+PL(I)
      SUMY=SUMY+RL(I)
      SUMXQ=SUMXQ+PL(I)*PL(I)
      SUMYQ=SUMYQ+RL(I)*RL(I)
    9 SUMXY=SUMXY+RL(I)*PL(I)
      SMXSMY=SUMX*SUMY
      ZN=N
      XDOT=SUMX/ZN
      YDOT=SUMY/ZN
      SXX=SUMXQ-((SUMX*SUMX)/ZN)
      SXY=SUMXY-(SMXSMY/ZN)
      B=SXY/SXX
```

```
      A=YDOT-(B*XDOT)
      SYY=SUMYQ-((SUMY*SUMY)/ZN)
      CR=SQRTF(B*B*SXX/SYY)
      SY=SQRTF(SYY/(ZN-1.))
      C=1.-CR**2
      RANG=SY*SQRTF(C)
      PRINT 20,
      PRINT 20, B, A, CR, RANG
20    FORMAT(4F9.5//)
      DO 40 I=1,3
      E=A+B*LOGF(P(I))
      D=E+2.*RANG
      G=E-2.*RANG
      RY(I)=EXPF(E)
      RH(I)=EXPF(D)
      RLO(I)=EXPF(G)
40    PRINT 30,  P(I), RY(I), RH(I), RLO(I)
30    FORMAT (F5.1, 5X, E13.6, 5X, E13.6, 5X, E13.6)
      PRINT 30,
45    CONTINUE
      STOP
      END
```

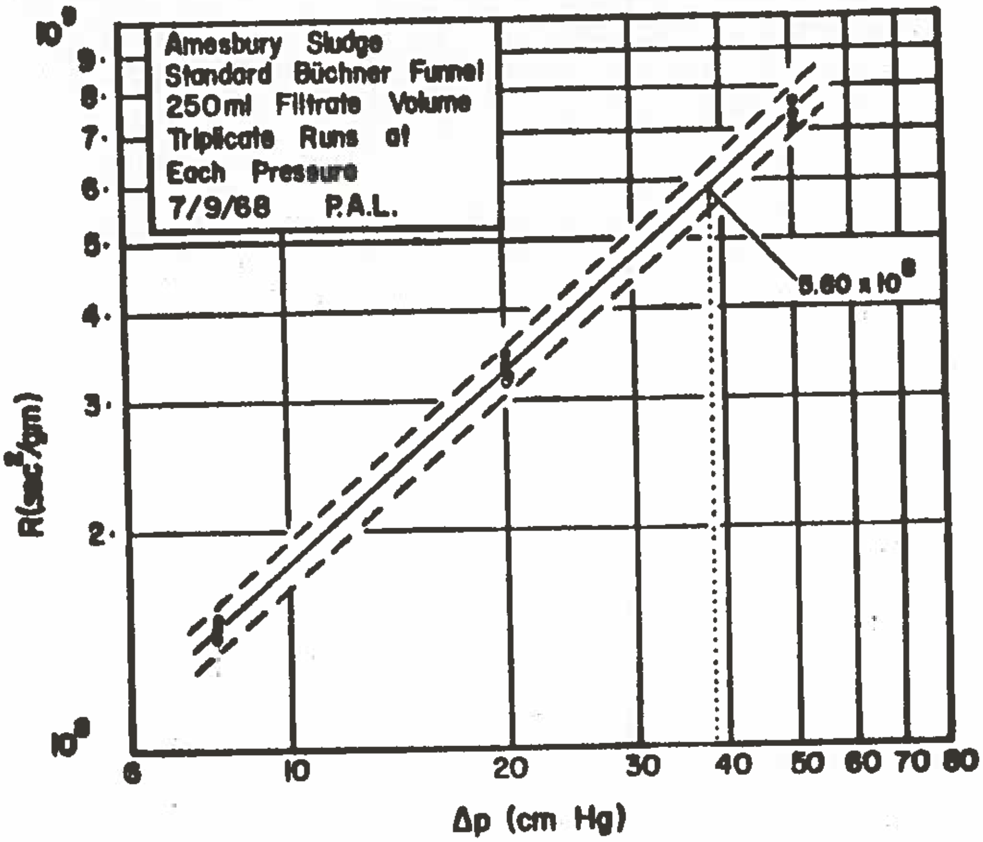
While computing slope, intercept, and correlation, the data output includes three computed specific resistance values which fall on the line of best fit, the relative error of these points, and two specific resistance values at each pressure which represent the 95% confidence limits. The slope "B" is σ, the coefficient of compressibility, which is dimensionless.

III. RESULTS

In order to obtain reliable results for both specific resistance and the coefficient of compressibility, computer handling of data was employed (see Section II) whenever possible. One aspect, however, hardly applicable to computer techniques is human variation in basic specific resistance testing.

Although Swanwick and Davidson[3] noted very high replicability in their experimentation, a series of tests was devised to measure the reproducibility of results on the newly designed equipment and techniques currently being employed. Triplicate specific resistance tests were performed on the same sludge, at each of three different pressures in a constant temperature room. The elapsed time from the beginning of the first test to the end of the last was held to an absolute minimum.

The results as shown in Figure 8 are indeed gratifying because in addition to corroborating the work of Swanwick and Davidson[3], the reliability of past and future testing with later generation equipment and techniques has been substantiated.



σ	Correlation Coefficient	Std. Error of Computed R
0.87401	0.99880	0.03436

Pressure (cm Hg)	Computed R(sec²/gm)	95% Confidence Limits Upper	Lower
50.8	7.44×10^8	7.97×10^8	6.95×10^8
20.0	3.30×10^8	3.53×10^8	3.08×10^8
8.0	1.48×10^8	1.59×10^8	1.38×10^8

Legend

——— Regression Line
- - - - 95% Confidence Limits

Figure 8. Results obtained from triplicate specific resistance tests at each of a range of pressures. (performed with the same sludge sample at constant temperature)

IV. CONCLUSIONS

Results of this study indicate that sludge specific resistance and coefficient of compressibility determinations can be obtained rapidly, reliably and routinely with high levels of reproducibility. Use of the fritted glass funnel is advantageous over the standard Buchner funnel in that the entire cross sectional area is permeable, thus avoiding local concentration of flow. A further advantage is that visual observations of filtrations may be made through the clear glass fritted funnel.

The Teletype console located in the laboratory adjacent to the experimental equipment permits immediate data reduction utilizing the vast resources of the digital computer. Visual display of the experimental points on a Teletype plotted graph assists in detection and elimination of human errors of recording and reading: spurious points on the graph are displayed within a few minutes of completion of an experimental test. Quantification of goodness of fit through a regression coefficient and a regression line avoids human variation inherent in fitting by eye a straight line to experimental points. Ready access to facilities for rapid data reduction and evaluation encourages performance of additional experimental tests, thus further increasing the confidence of the results. The average time required to complete a single specific resistance test is about one hour; thus if three specific resistance readings at three different pressures are used to determine the coefficient of compressibility, about three hours would be required.

Reports in the literature attest to the reproducibility of specific resistance and coefficient of compressibility determinations.

In spite of these published reports, various questions have been voiced about the reproducibility of results. Figure 8 was representative of the results obtained when running multiple tests at each of a range of pressures. Reproducibility was obtainable within close tolerances. Perhaps results obtained by an inexperienced person would show wider variations, but the aforementioned Figure 8 attests to the confidence one can have in specific resistance and coefficient of compressibility data obtained by careful experimentation.

ACKNOWLEDGEMENTS

This work was supported by Federal Water Pollution Control Administration Grant WP-01239-01 and -02. The experiments were conducted by P. A. Lutin in the Environmental Engineering Laboratories of the University of Massachusetts. The authors voice a special note of thanks to Robert Gonter and James Hill of the Research Computer Center for their work on the UMASS Fast Fortran programming, Kuang-Mei Lo, research assistant in Environmental Engineering, Mr. Thomas G. Sanders, a former graduate trainee, and Mrs. Irene Clarke for the manuscript typing.

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EVALUATION OF CHEMICAL CONDITIONING

FOR

GRAVITY DEWATERING OF WASTEWATER SLUDGE

Preprint of a paper presented at the American Chemical Society Symposium on Colloid and Surface Chemistry in Air and Water Pollution, Atlantic City, New Jersey.

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EVALUATION OF CHEMICAL CONDITIONING

FOR

GRAVITY DEWATERING OF WASTEWATER SLUDGE

ABSTRACT

Some 25 to 65 percent of total capital and operating costs for wastewater treatment is expended on sludge processing. A major emphasis to reduce costs has been placed on the concentration of sludge previous to final disposal. Despite the availability of many commercial devices for concentration, the most widely used system entails gravity dewatering of sludge on porous media, such as sand drying beds. Chemical conditioners to improve the rate of gravity dewatering have been used with limited success. Gradual blinding of the drying bed media by lime and ferric chloride, and deleterious effects on the soil conditioning benefits of sludge applied to cropland were noted. Organic polymers as sludge conditioners promise to quiet these objections. Work by the authors demonstrates that optimum coagulant dosage for dewatering can be determined on the basis of Büchner funnel tests of specific resistance using at least two different pressures to calculate the coefficient of compressibility. For accelerated dewatering, the specific resistance should be decreased, the coefficient of compressibility increased. A verified mathematical equation describes the time of sludge dewatering on prototype beds on the basis of Büchner funnel testing, and enables one to properly evaluate conditioner performance.

EVALUATION OF CHEMICAL CONDITIONING FOR GRAVITY DEWATERING OF WASTEWATER SLUDGE

by J. H. Nebiker, D. D. Adrian, and Kuang-Mei Lo[1]

Introduction

Recent data indicates that between 25 and 65 percent of total capital and operating costs for wastewater treatment arise from sludge processing and disposal (Mulbarger, 1967). Current trends suggest that dewatering of raw sludge by vacuum filtration or centrifugation, followed by incineration is the most promising process system of the future. Realistically, however, one must admit that the presently more popular method of dewatering and drying of digested sludge or sand beds followed by land disposal will remain the more widespread process for some time to come. In an age where ever more stringent controls on costs must be emphasized, it is unlikely that sand beds will rapidly be displaced. What is more reasonable to assume is their possible eventual disappearance over many years. During the interim period a great need is evident to utilize sand beds more efficiently and effectively than previously in order to handle the constantly increasing volumes of sludge at minimum cost.

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Importance of Gravity Dewatering

Sand bed operation may consist of a combination of decantation, gravity dewatering, and drying. The relative importance of these processes was illustrated in a series of pilot-plant tests at the British Water Pollution Research Laboratory (Ministry of Technology, 1963). The results are shown in Figure 1.

The relative importance of evaporation in the experiments would be slightly increased if the sludge were allowed to dry beyond the minimum acceptable stage (liftable by handfork), because further water removal would be chiefly by evaporation. However, the amount of water removed by drying a sludge from, say, 25 to a 50 percent solids content represents but 12.5 percent of the water removed in bringing a sludge from 5 percent down to 25 percent solids. The role of evaporation was found, as expected, to increase in the dryer and warmer months; and to increase for sludges of poor drainability, as indicated by specific resistance values. Poor drainability was responsible for increased values of decantation.

Few treatment plants are provided with facilities to decant sludge on the sand beds. Most of a potential decant drains as a subnatant during flotation on the beds, a result of gas bubbles buoying the sludge floc. One should therefore note in Figure 1, that where decantation was absent, an average of over 60% of water removal resulted from drainage. This average indicates the overall significance of drainage on sand bed performance, and the value of sludge conditioners in providing accelerated potential dewatering by gravity.

Sludge Conditioning

Efforts to improve gravity dewatering by conditioning date back over half a century. Alum, ferric salts, and lime all had been used to accelerate dewatering rates of what was then solely primary sludges. Ferric salts were reported to oxidize and clog the sand beds. The most successful conditioner, alum, in addition to providing larger floc, hence larger pores and easier paths of egress for the filtrate, provided additional flotation by production of carbon dioxide (Sperry, 1941).

The effect of conditioning on drainage was noted most vividly by Templeton (1959). See Figure 2. Here aluminum chlorohydrate was added to one sample at a dosage of 10% based on dry solids. The treated sludge dewatered in the first 10 days at twice the rate of the untreated sludge. Furthermore, the treated sludge dewatered from 6.7 percent solids to 14.3 percent, whereas the untreated sludge dewatered to but 7.6 percent solids.

In spite of such impressive results, few treatment plants presently utilize chemical conditioning to aid gravity dewatering. A powerful deterrent to chemical use resides in the belief that the metals in conditioned sludge are deleterious to certain plant life, hence proscribing sludge disposal on farmland (Downing and Swanwick, 1967). The use of organic polymer conditioners promises to quiet such objections; thus, the role of conditioning in gravity dewatering bears closer scrutiny in the future. Evaluation of conditioner performance must be simplified, however, over methods such as outlined by Templeton (1959). In this regard, the applicability of the specific resistance concept to gravity dewatering can prove of great value.

Derivation of Specific Resistance

Before proof is offered of the applicability of specific resistance to determine gravity dewatering rates, a review of the derivation for specific resistance is in order. The permeability equation serves as a basis for derivation of specific resistance:

$$\frac{dV}{dt} = \frac{\rho g H A}{\mu} \sum_{n=1}^{n=N} \frac{1}{L_n R_n} \tag{1}$$

where

$\frac{dV}{dt}$ = volumetric flow rate $[L^3][T^{-1}]$

ρ = mass density of filtrate $[M][L^{-3}]$

g = acceleration of gravity $[L][T^{-2}]$

H = head loss across N filters $[L]$

A = cross-sectional area of flow $[L^2]$

μ = dynamic viscosity of the filtrate $[M][L^{-1}][T^{-1}]$

L_n = depth of filter n $[L]$

R_n = resistance of filter n $[L^{-2}]$

When considering only the sludge cake and the supporting filter material Equation 1 is simplified. Note that

$$\sum_{n=1}^{N} \frac{1}{L_n R_n} = \frac{1}{L_s R_s + L_F R_F} \tag{2}$$

Here subscript s refers to the sludge, F to the filter.

Carmen (1938) recognized that the depth of the sludge cake was proportional to the cumulative volume of filtrate V. Also, it is convenient to redefine the units of the resistance of the sludge:

$$L_sR_s = \frac{cRV}{A} \tag{3}$$

The new R is called specific resistance $[T^2][M^{-1}]$. c is defined as the weight of sludge solids deposited on the sludge cake per unit volume of filtrate. It may be calculated from the initial solids content of the sludge, S_o, and the final solids content, S_f (both in percent, wet basis):

$$c = \frac{\rho g}{\frac{100}{S_o} - \frac{100}{S_f}} \tag{4}$$

Specific resistance is determined most commonly with a Büchner funnel. A constant vacuum is applied to the sludge, and time-volume of filtrate readings are taken. These readings, plus values for initial and final solids contents, allow calculation of R, using an integrated form of Equation 1. Combining Equations 1, 2 and 3 yields

$$\frac{dV}{dt} = \frac{\rho gHA/\mu}{\frac{cRV}{A} + L_FR_F} \tag{5}$$

which when integrated from V = 0 at t = 0, to V = V at t = t, and rearranged results in

$$R = \frac{2A}{cV^2} \left(\frac{\rho gHAt}{\mu} - L_F R_F V\right) \qquad (6)$$

Calculation procedures are well outlined in a number of standard texts (Rich, 1961). ρ and μ are considered to be identical to values for water at the same temperature.

The R value determined is valid for only the constant pressure at which the test was run. R is related with pressure through the empirical relationship

$$R = R_c \left(\frac{H}{H_c}\right)^{\sigma} \qquad (7)$$

Here R is the specific resistance at any head H. R_c is a reference specific resistance at a set constant head H_c. (38.1 cm-Hg = 15 in. Hg is a commonly referenced head used to designate R_c). σ is the coefficient of compressibility, which is dimensionless. It may be noted that two specific resistance determinations at two different pressures allow calculation of σ.

Gravity Dewatering Equation

A relationship describing the gravity dewatering of sludge on sand beds has been determined by Nebiker et al. (1968). Since the head is constantly decreasing, use of Equation 5 must be modified. This is done by using a reference head H_c, which is accomplished by combining Equations 5 and 7:

$$\frac{dV}{dt} = \frac{\rho gHA/\mu}{\frac{cR_c V}{A} \left(\frac{H}{H_c}\right)^{\sigma} + L_F R_F} \qquad (8)$$

dV/dt is equal to the cross-sectional area of flow A times the surface fall velocity dH/dt. V, the cumulative filtrate volume, is the product of area A and cumulative surface drop, or $H_o - H$. Here H_o is the head at $t = 0$. It follows then, that

$$A \frac{dH}{dt} = \frac{\rho g H A}{\mu} \frac{1}{\frac{cR_c}{A} \left(\frac{H}{H_c}\right)^{\sigma} (AH_o - AH) + L_F R_F}$$

An integrated form of this equation describes the time t for sludge to dewater from H_o to H

$$t = \frac{c\mu R_c}{\rho g H_c^{\sigma}} \left[\frac{H^{\sigma+1} - H_o^{\sigma+1}}{\sigma + 1} + \frac{H_o^{\sigma+1} - H_o H^{\sigma}}{\sigma} + \frac{L_F R_F H_c^{\sigma} \ln(H/H_o)}{cR_c}\right] \quad (9)$$

The value of c according to Equation 4 depends on the variables S_o and S_f. S_o is the solids content of the applied sludge, whereas S_f is the solids content of the filter cake. Experiments (Sanders, 1968) have shown that the value of S_f is on the order of 25%. It is thus reasonable, considering S_o to be considerably smaller, that Equation 4 is approximated by

$$c = \frac{\rho g S_o}{100} \quad (10)$$

When the filter resistance is negligible compared to cake resistance, Equation 9, combined with Equation 10, reduces to

$$t = \frac{S_o \mu R_c}{100 H_c^{\sigma}} \left(\frac{H^{\sigma+1} - H_o^{\sigma+1}}{\sigma + 1} + \frac{H_o^{\sigma+1} - H_o H^{\sigma}}{\sigma}\right) \quad (11)$$

Equation 11 has been verified by extensive testing, of which Figure 3 is but one example (Sanders, 1968). Each of the three different sands used as filter media appeared to affect the initial drainage rates; however, in Figure 3 these effects were not differentiated, and a curve general to all was drawn. A dimensionless empirical correction coefficient of 0.75 was used to adjust Equation 11 to fit the data. Mention must also be made that the head values contained in Equation 11 were not synonymous with depth of sludge applied, there existing 33 cm of suspended water beneath the sludge-sand interface, which, as will be noted, may be adjudged typical of sand beds. The head is thus the sum of the depth of sludge and the height of suspended water.

It is hoped that Equation 11 will eventually be further developed to include direct values of the filter parameters, effective size (D_{10}) or the uniformity coefficient (D_{60}/D_{10}). Furthermore, there is a need to relate Equation 11 in terms of specific resistance determined by means other than those used by Sanders. Without these advances, however, it is still possible to effectively reproduce the type of result exemplified by Templeton's work. Instead of using cumbersome drying bed models requiring several weeks of observations, a minimum of two simple specific resistance determinations at differing pressures is all that is required. This suggests that evaluation of optimum conditioner performance for gravity dewatering is now practicable.

Laboratory Analysis of Conditioner Performance

Two wastewater sludges were selected for experimentation to determine conditioner performance. The Amherst, Massachusetts, treatment plant provided a primary digested sludge. Pittsfield, Massachusetts, served as a source of a digested mixed primary and trickling filter sludge. Samples of each sludge were flash mixed with doses of polymer conditioners at 100 rpm for one minute. The effectiveness of each dosage and conditioner was noted qualitatively after one minute of gentle stirring at 30 rpm, with attention being paid to floc structure and depth of supernatant. Eventually two conditioners were selected as sufficiently effective to warrant specific resistance tests.

Samples of each sludge were prepared with conditioner dosages of 0, 100, 200, 300, 400, and 500 mg/l, each sample being readied according to the manufacturer's instructions. Solids contents before filtration were determined per Standard Methods (1965). One hundred ml of sludge was then poured into a 12 cm I.D. Büchner funnel, previously fitted with a wetted Whatman No. 5 filter paper. The vacuums applied were 18, 38, and 60 cm of mercury, each held constant during each separate filter run. Three repetitions were run throughout. The duration of each run was never in excess of 17 minutes, which was sufficient at the higher vacuums to provide a solid cake. These cakes were used to determine the final solids contents. Cakes generally did not develop at the lowest pressures, and because of theoretical justification, the top liquid sludge was poured off and the solids content of the remaining cake measured. Filtrate viscosity and density were assumed to be that of water at the ambient temperatures.

Calculation of Specific Resistance

A rearrangement of Equation 6, together with Equation 4, provided the basis for a UMass Fast Fortran program to determine each individual specific resistance (Lutin et. al. 1968). Equation 7 states that the logarithm of R versus that of the respective H value at which R was determined should plot as a straight line. This would allow a regression analysis to be used to calculate the coefficient of compressibility, the coefficient being the slope. Furthermore, the value on the regression line for the reference head H_c is a weighted mean value of R_c.

Regression analysis provided the values for specific resistance and the coefficient of compressibility in Figures 4 and 5. As can be seen, the specific resistance decreased in both cases with increasing conditioner concentration. Statistical tests proved the variation of specific resistance with concentration was significant (> 90% certainty). The apparent increase in the coefficient of compressibility with dosage, however, could not be statistically proven. This was believed to be caused by the erratic values of specific resistance measured at the lowest heads, possibly resulting from unrepresentative cake samples. For instance, if cake material were to be poured off with the unfiltered sludge at the end of the experiment, the final solids content of the cake would be too high, with inflated values of specific resistance resulting. Nonetheless, all regression lines had a correlation coefficient of better than 0.98.

Calculation of Conditioner Performance

The values for the specific resistances (R_c) and coefficients of compressibility (σ) found in Figures 4 and 5 may be directly inserted into Equation 11:

$$t = \frac{S_o \mu R_c}{100 H_c^{\sigma}} \left(\frac{H^{\sigma+1} - H_o^{\sigma+1}}{\sigma + 1} + \frac{H_o^{\sigma+1} - H_o H^{\sigma}}{\sigma} \right) \qquad (11)$$

S_o, of course, is the initial solids content, which would need to be previously known, as would the filtrate viscosity μ, in order to calculate the specific resistances. H_c is the head at which R_c was determined; however, H_c must be in units referring to the filtrate. Hence H_c in cm-Hg must be multiplied by 13.55.

The heads H_o and H refer to the heads at the beginning (t=o) and at the time in question (t). As previously mentioned, the heads are equal to the depths of sludge above the sand bed, plus the depth of suspended filtrate and other water beneath. Such suspended liquid can occur only in the pores of the fine sand - that is, down to the gravel. Whether or not the sand is moist before sludge is applied, it is clear that filtrate rapidly supplies the necessary pore water to provide suspended water traversing a vertical distance equal to the depth of sand. Design standards refer to 12 inches, or approximately 30 cm, as an average depth of sand (Sewage Treatment Plant Design, 1959).

A typical depth of sludge applied on beds is 8 inches ($\sim$ 20 cm); thus the initial head may be assumed to be 50 cm. A final head could be set at 40 cm (depth = 10 cm), representing an approximate doubling of the solids content. Using these values, and those previously indicated, including the values from Figures 4 and 5, Equation 11 may be solved for t. Results are shown in Figure 6.

It must be emphasized that the curves in Figure 6 represent laboratory data subjected to the assumed field conditions. For example, if other depths of sludge were assumed, other curves would result. Nonetheless, the effect of conditioner dosage for typical treatment plants is exemplified in Figure 6. The results appear to be realistic, particularly when compared to Figure 2.

The addition of 500 mg/l of conditioner to the Amherst sludge shows in Figure 6 an 86% decrease of drainage time (from 58 days down to 8). Referring now to the specific resistance values in Figure 4, a 75% reduction occurred by adding 500 mg/l. Equation 11 relates time directly to specific resistance. It is therefore seen that the other intrinsic parameter of dewaterability, the coefficient of compressibility, plays an important role: an increase in the coefficient of compressibility reduces drainage time[1]. This is corroborated by the Pittsfield sludge. Again, it must be stated that the experiments herein described did not prove that conditioners varied the coefficient of compressibility.

Conclusions

The importance of the gravity dewatering process to the overall concentration of sludge on sand beds is clearly seen, as is the potential value in conditioning sludge to accelerate the dewatering process. Evaluation of conditioner performance by the use of bench-scale sand beds appears most inefficient. Theoretical considerations involving the specific resistance concept, and substantiated by experimental results, lead to a direct and rapid method to estimate conditioner performance for sand bed dewatering.

The method outlined requires but two specific resistance determinations at two differing pressures for each sludge sample, enabling one to calculate drainage rates for various assumed or real field conditions. These conditions include the depth of sludge application, the depth of the sludge to be removed, and the depth of the supporting sand. Temperature also needs to be assumed to provide values for the dynamic viscosity of the filtrate.

It should be noted that an estimate of the relative effect of conditioners on gravity dewatering may be made directly from the corresponding effect of the conditioner on specific resistance values alone. A more exact estimate will require values for the coefficient of compressibility. To optimize dewatering, the coefficient of compressibility should be increased. Further work is indicated in this direction.

Optimal conditioner preformance, of course, as well as optimum depths, should be defined on an economic basis. Unfortunately, a true basis requires further work into the relationship between drainage, decantation, and evaporation, integrating the drainage formula described here with the role of weather, operation, and design. Until and if such an overall relationship is established, consideration of drainage alone may provide a realistic method of cost optimization.

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1) Theoretically this is true strictly when $H_c \geq H_0$, as is the case here.

ACKNOWLEDGEMENTS

Polymer conditioners used in this study were provided by the Calgon Corporation. Figures 4 and 5 represent the performance of ST260, a cationic polyelectrolyte. Support for this work was provided by FWPCA research grant WP-01239. The aid of Mrs. Nancy Lee in the laboratory testing is acknowledged.

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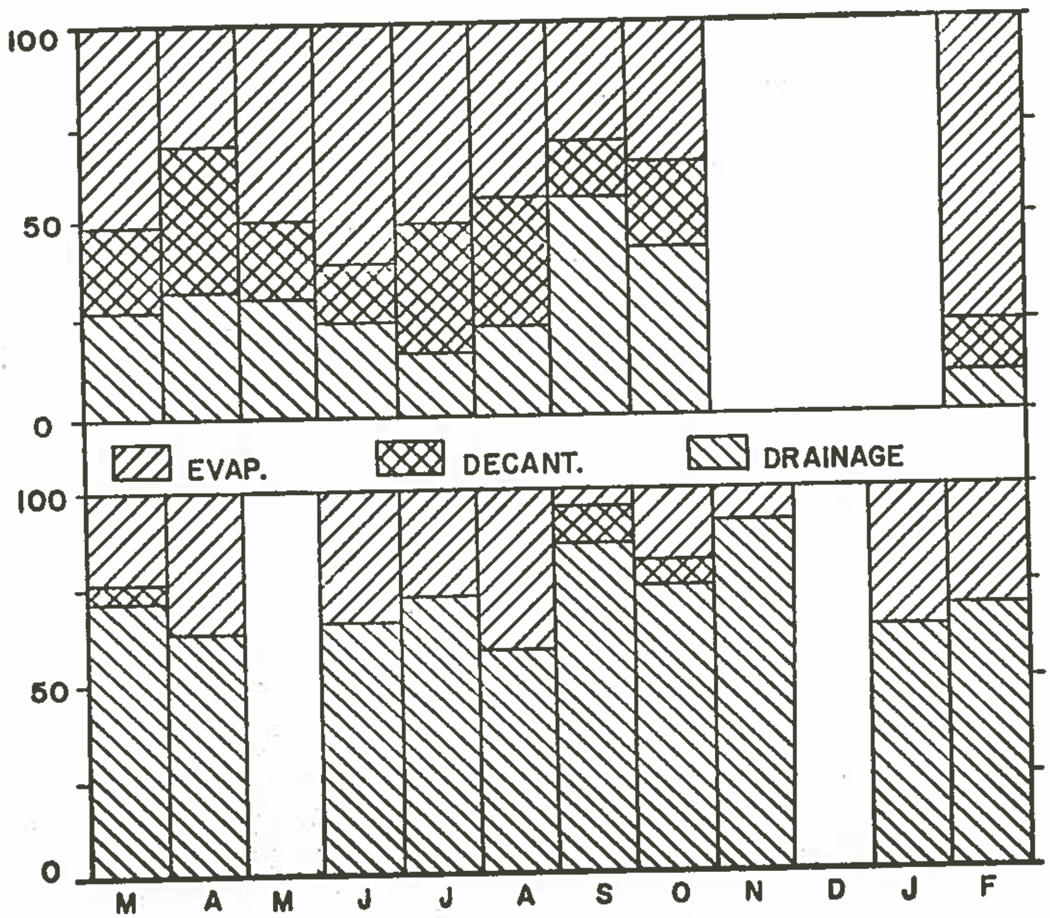

Figure 1: Percentage of water removed by evaporation, decantation, and drainage. 12-in. layers of mixed digested sludges from activated sludge plants were applied in the months shown. The upper half of the figure represents a sludge with a specific resistance of $29.1 \cdot 10^9 \text{sec}^2/\text{gm}$; the lower half, $2.6 \cdot 10^9 \text{sec}^2/\text{gm}$. Both values are at a vacuum of 36.9 cm of mercury.

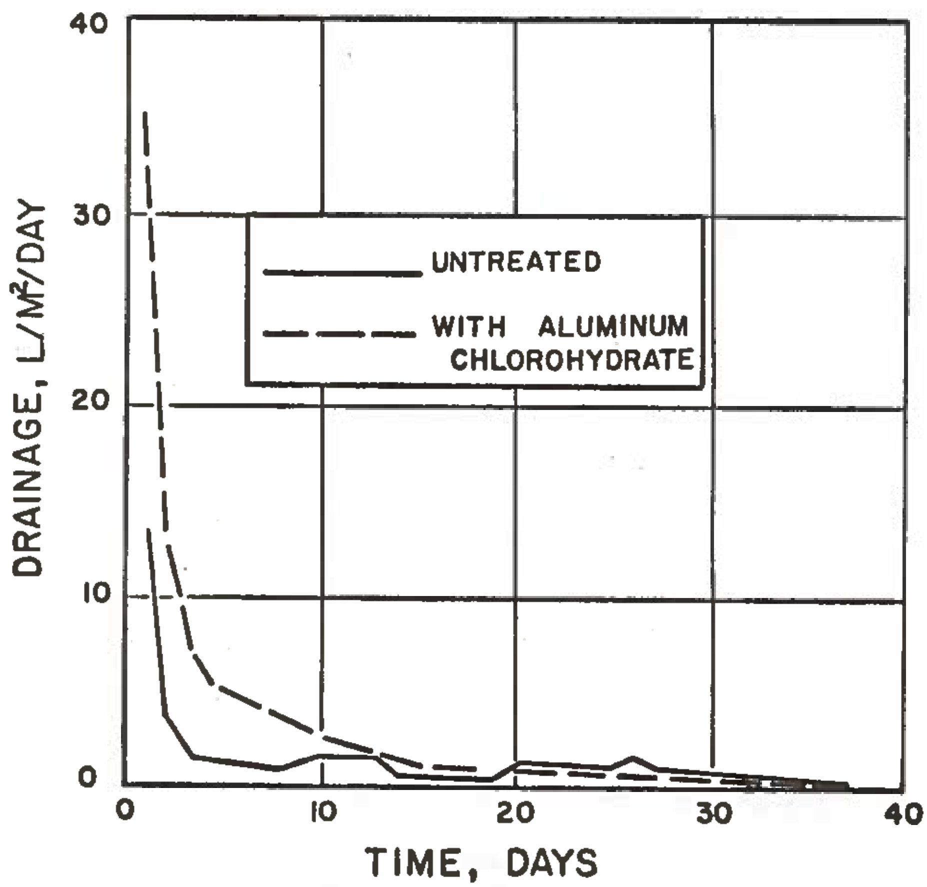

Figure 2: Gravity dewatering rates for a mixed digested primary and waste activated sludge. Initial filling depths were 9 inches.

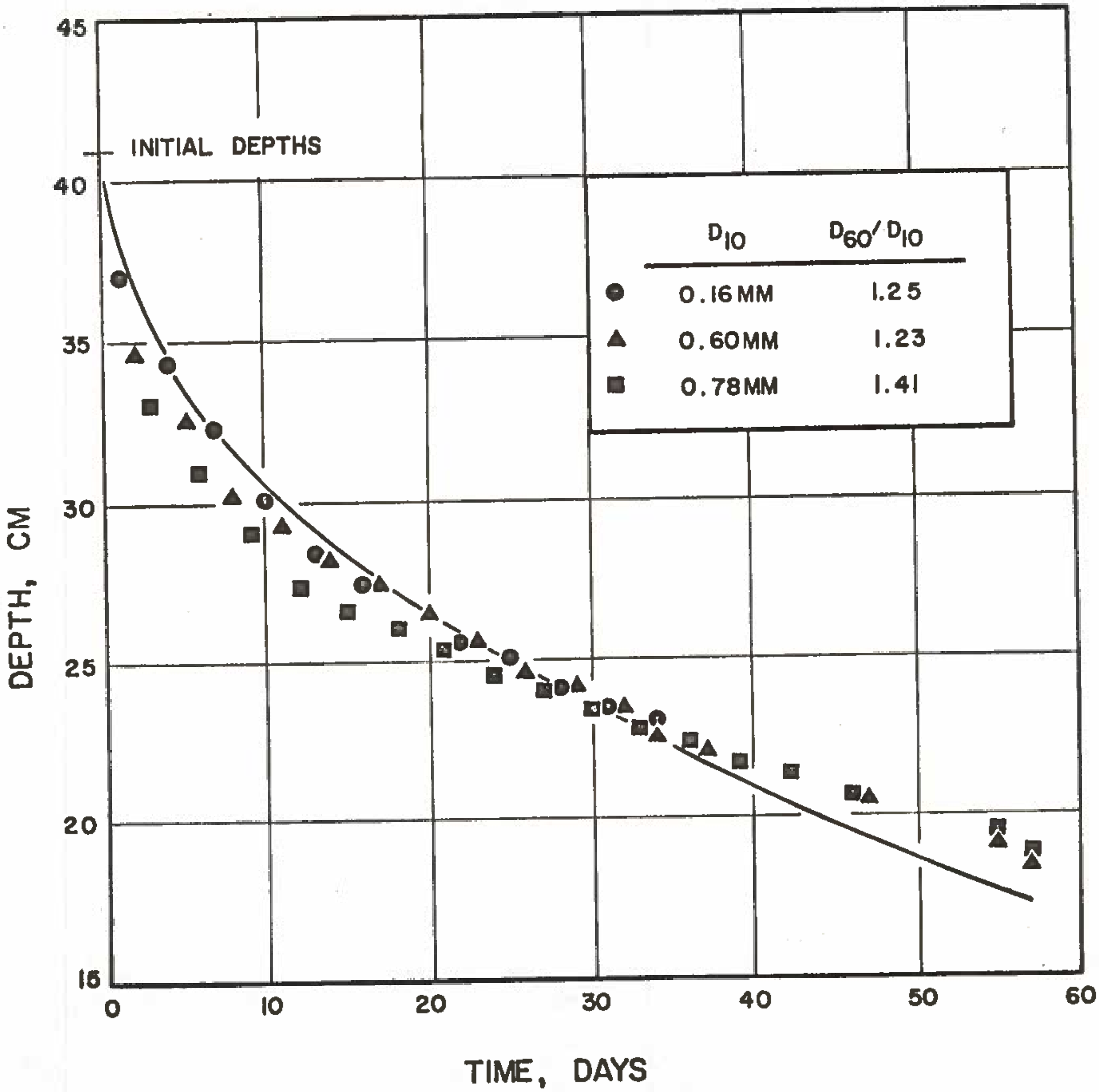

Figure 3: Depth-duration curve for gravity dewatering of identical samples on three different sands with classification sizes as noted. Mixed digested sludge from an activated sludge plant was used. Initial solids content was 2.78%; initial specific resistance, $3.72 \cdot 10^9$ sec^2/gm at H_c = 38.1 cm Hg. The coefficient of compressibility was 0.64 and the suspended water measured 33 cm.

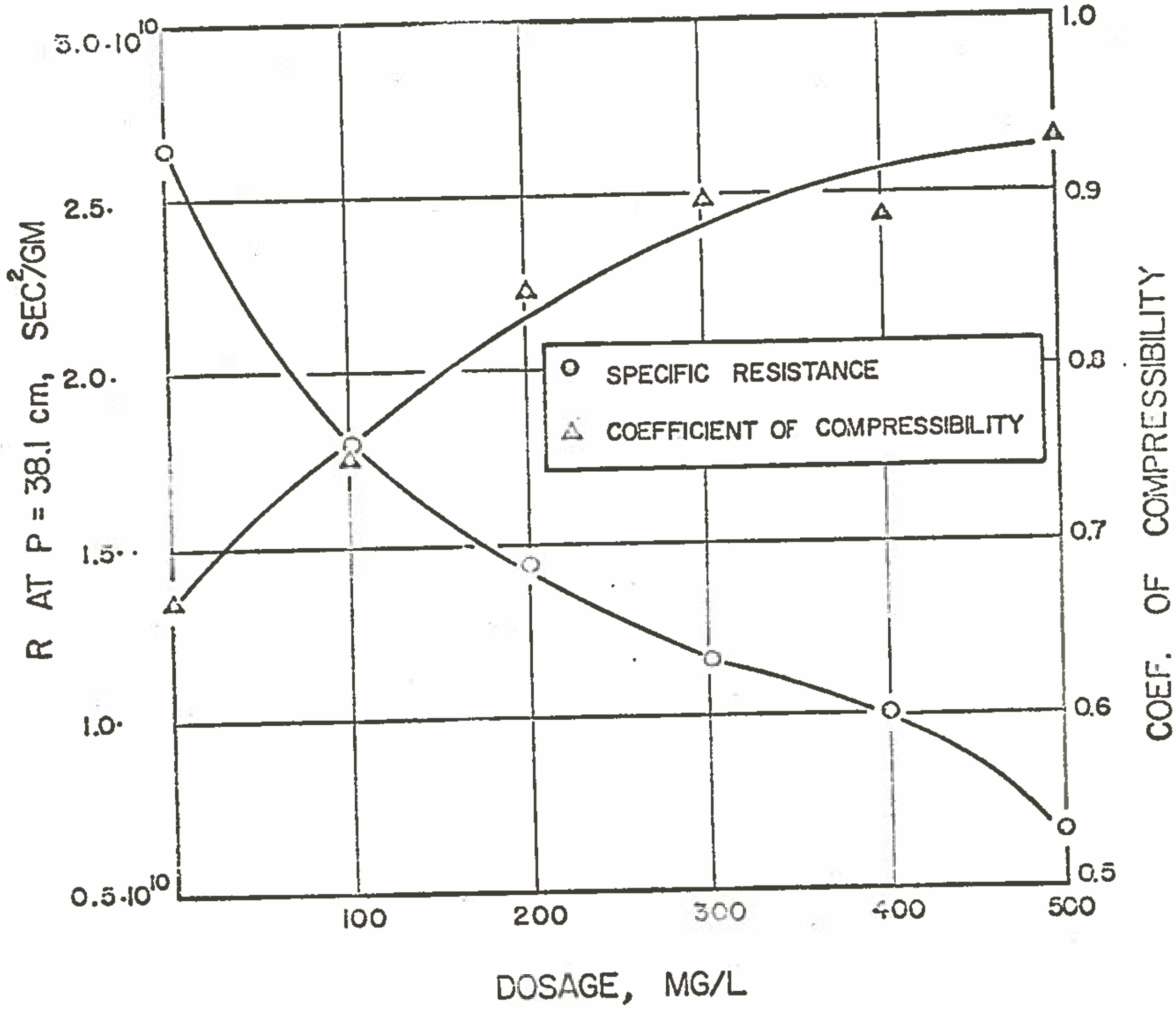

Figure 4: Effect of conditioner dosage on digested sludge from Amherst. The specific resistance values are at a vacuum of 38.1 cm of mercury. Initial solids content = 9.5%.

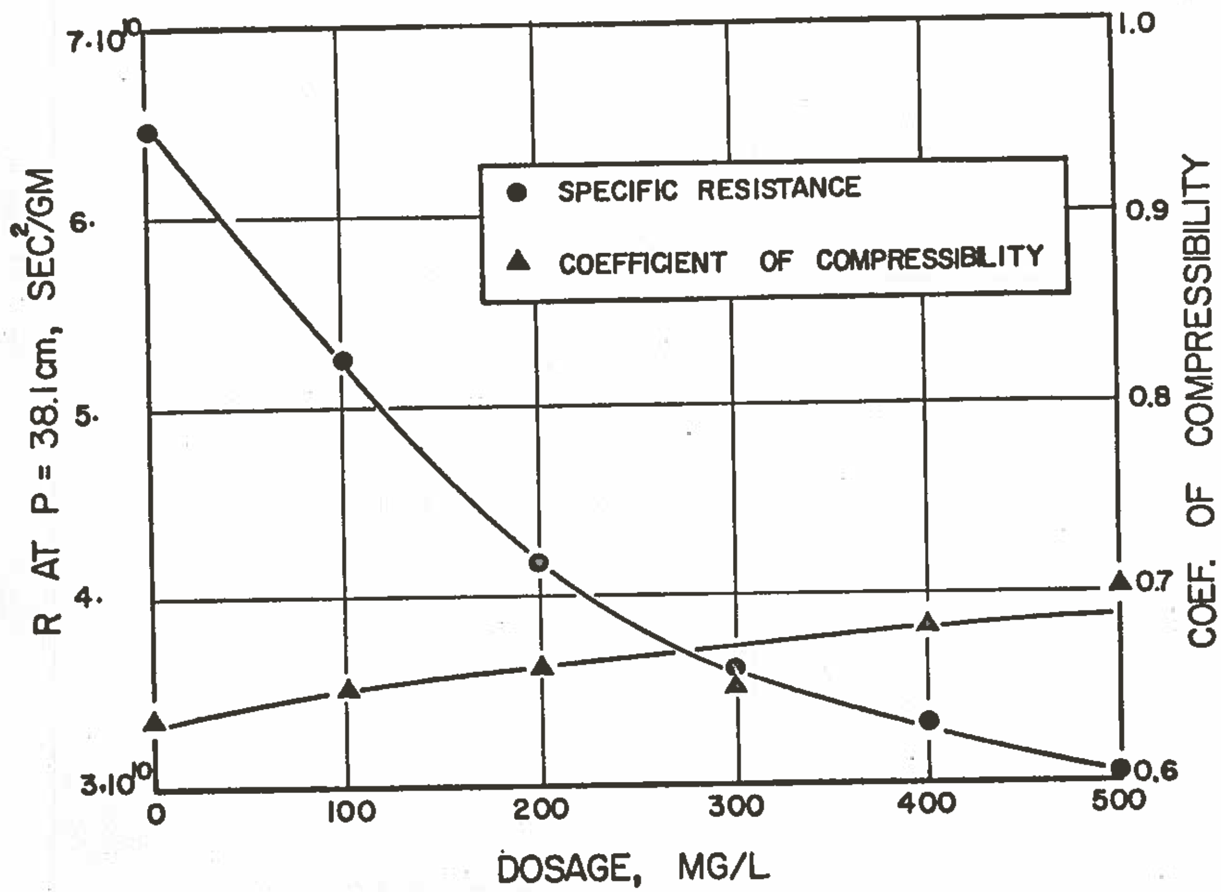

Figure 5: Effect of conditioner dosage on digested sludge from Pittsfield. As in Figure 4, the specific resistance values are for vacuums of 38.1 cm of mercury. Initial solids content was 4.9%.

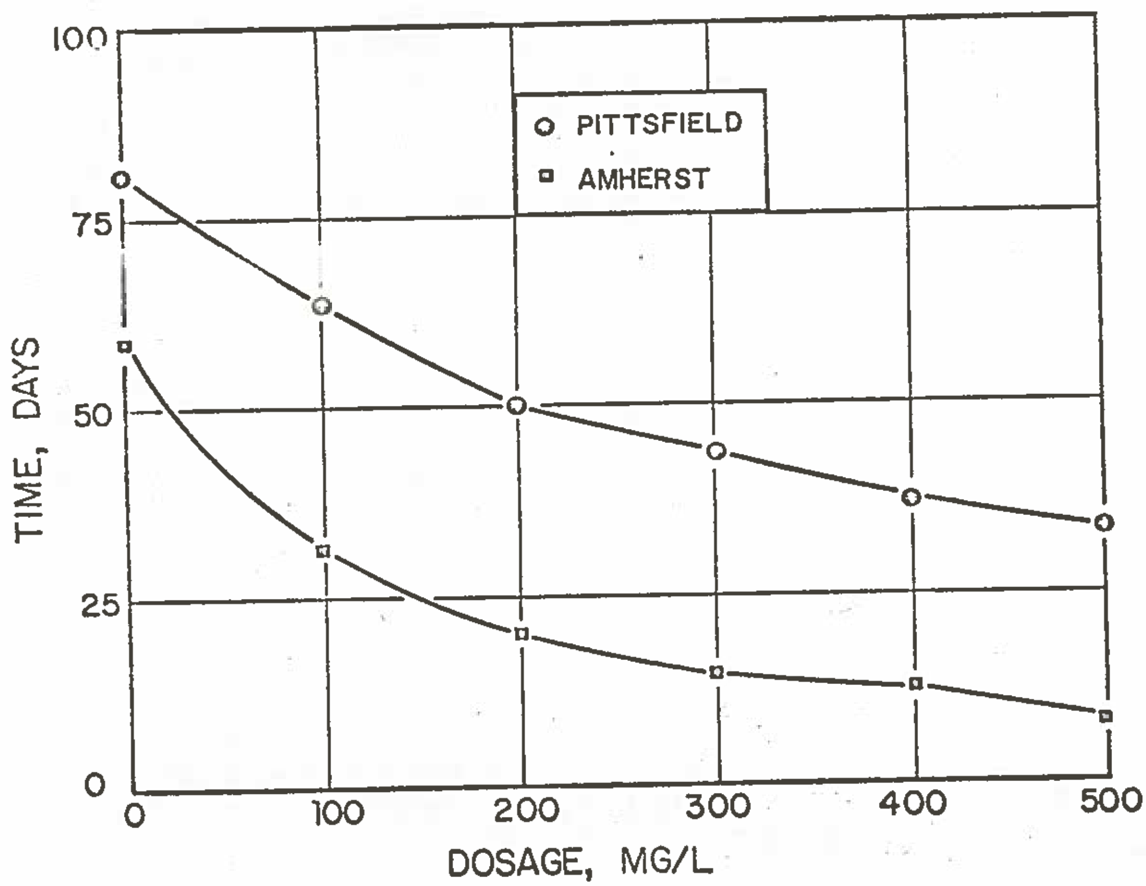

Figure 6: Computed drainage times as a function of conditioner dosage. The assumed initial sludge depth is 20 cm, the final depth 10 cm. Sand depth is 30 cm.

IRRIGATION AND RECLAMATION
USING
SANITARY SLUDGES

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ABSTRACT

Irrigation and Reclamation using Sanitary Sludges, D. D. Adrian and J. H. Nebiker*, University of Massachusetts.

The volume of sludge to be collected from water and wastewater treatment plants is expected to grow rapidly in volume in the near future. Higher levels of treatment required to abate pollution produce correspondingly larger volumes of sludge. For example, the percentage of the total influent flow removed as sludge increases from perhaps two percent for primary treatment, now widely used, to about ten percent for advanced waste treatment and water renovation methods now in the demonstration stage of development. Sludge is high in water content, hence a dewatering operation reduces the volume of sludge to be handled. Dewatering costs vary widely: mechanical dewatering methods yield sludge handling costs up to $80/ton of dry solids, while costs under $20/ton of dry solids may be achieved when disposal is direct to the land. A lack of understanding of sludge dewatering rates on porous materials has delayed widespread acceptance of this method of simultaneous irrigation and land reclamation. The authors have developed and experimentally validated an equation describing dewatering on porous material in terms of sludge solids content, specific resistance, and coefficient of compressibility. These parameters are easily determined in the laboratory.

INTRODUCTION

Sewage sludge is defined as the accumulated semi-liquid suspension of settled solids deposited from wastewaters, raw or treated, in tanks or basins.[1] Water treatment sludge is produced in the treatment of raw water to make it potable, and in some water softening processes. The term "sanitary sludge" is used herein for both sewage sludge and water treatment sludge.

Considerable attention has focused in recent years on the production and disposal of sewage sludge.[2] Less attention has been paid to the problem of water treatment sludge disposal. A recent survey by the American Water Works Association pointed out a widespread apathy toward problems of water treatment sludge disposal. The survey showed that over 90% of the water treatment plants in the United States discharged untreated sludge into the raw water source.[3] Times change, however, and in 1968 the newly formed American Water Works Research Foundation undertook as its first research project the study of waste disposal from water treatment plants.[4] This study undertaken in July 1968 is expected to focus attention on the problem of water treatment sludge disposal.

Cost studies for sewage sludge disposal by the Chicago Metropolitan Sanitary District[2] have indicated the most economical disposal method to be direct application of the liquid sludge on the land, with a side benefit of land reclamation.

The research results reported herein are part of an ongoing study sponsored by the Federal Water Pollution Control Administration at the University of Massachusetts. The purpose of the study is to investigate sanitary sludge dewatering and drying practice. Results of the investigation to date have focussed on dewatering sludge on porous media.[5,6,7,8,9] These results have shown that the solids in sanitary sludges, even though they may only be in the range of 0.5 - 8% by weight, have a profound influence on sludge dewatering. The solids ordinarily are left behind on the soil-sludge interface, and most of the head loss occurs through the sludge cake - not through the supporting media. In the following discussion the relationship between dewatering rate and head loss is discussed to bring out the role of resistance of the sludge cake and resistance of the underlying granular media. Figures 1,2 and 3 point out the development of the sludge cake.

THEORETICAL DEVELOPMENT

The fundamental equation describing flow through a sludge cake and a supporting filter is

$$\frac{dV}{dt} = \frac{\rho g H A/\mu}{\frac{cRV}{A} + L_F R_F} \qquad (1)$$

where the meaning of the terms is

V	=	volume	L^3
t	=	time	T
ρ	=	mass density of liquid	ML^{-3}
H	=	head	L
A	=	cross sectional area	L^2

μ	=	dynamic viscosity of liquid	$ML^{-1}T^{-1}$
c	=	weight of solids deposited per unit volume of filtrate	$ML^{-2}T^{-2}$
R	=	specific resistance of sludge cake	T^2M^{-1}
L_F	=	depth of filter	L
R_F	=	filter resistance	L^{-2}

The derivation of the above equation is available elsewhere.[5,7,9] Its application to gravity dewatering has been experimentally verified through extensive testing; however, the resistance to flow was assumed to occur only across the sludge cake, permitting the term L_FR_F which expresses the resistance of the filter to be neglected.[10] In the present development the assumption of negligible filter resistance will not be introduced.

Figure 4 indicates that if sludge is ponded on the surface of a soil, then some of the liquid infiltrates; the volume of that infiltrate will be $A(a_0 - a)$. The head will be the distance $a + L_F + h_c$, where L_F is the distance from the sludge-sand interface to the wetting front, and h_c is the capillary head assumed acting at the interface.[11] The continuity equation permits relating a and L_F, as $a_0 - a = nL_F$, where n is the porosity (a more accurate assumption would be that n is the useable porosity, thus excluding entrapped air). The sludge cake formed by the solids left behind as the liquid infiltrates is compressible, causing its resistance to change with the amount of the compression. A widely accepted empirical relation between the specific resistance at one head and the corresponding value at another head is

$$R = R_c \left(\frac{H}{H_c}\right)^{\sigma} \qquad (2)$$

where R_c is the specific resistance at head H_c, and σ is the dimensionless coefficient of compressibility.[12]

Combining the above results into Equation (1) yields

$$-A\frac{da}{dt} = \frac{\rho g A[a_0 + nh_c + (n-1)\,a]/n\mu}{cR_cA\left[\frac{a_0 + nh_c + (n-1)\,a}{nh_c}\right]^{\sigma}\left(\frac{a_0 - a}{A}\right) + R_F\frac{(a_0 - a)}{n}} \qquad (3)$$

which may be rearranged to

$$dt = \frac{\mu(a - a_0)}{\rho\, g}\left\{\frac{cR_c}{H_c^{\sigma}}\left[\frac{a_0 + nh_c + (n-1)a}{n}\right]^{\sigma-1} + \frac{R_F}{[a_0 + nh_c + (n-1)a]}\right\} da \qquad (4)$$

which, with $a - a_0$ at $t = 0$, integrates to

$$t = \frac{\mu}{\rho g(n-1)^2}\left\{\frac{cR_c\, n^{1-\sigma}}{H_c^{\sigma}\,\sigma}\left[\frac{B^{\sigma+1} + \sigma A^{\sigma+1}}{\sigma+1} - BA^{\sigma}\right] + R_F[A - B(1 + \ln A/B]\right\} \qquad (5)$$

with $A = a_0 - a + n(h_c + a)$

$B = n(h_c + a_0)$

Equation (3) permits calculation of the infiltration rate at any depth of ponding a. The infiltration rate is seen to start at a large value at $t = 0$, then decrease with time as the sludge cake increases in thickness. If $R_c = 0$, which could be interpreted as the application of water without solids to the porous medium, the equation for the infiltration rate would correspond to that for idealized infiltration into a soil.[11]

However, rather than approaching zero, R_c for wastewater sludges is far from negligible; it constitutes the predominant resistance to flow in the sludge-soil system. Typical values of R_c for sanitary sludges are given in Table 1.[5]

TABLE 1

FILTRATION PROPERTIES OF SANITARY SLUDGES

Sludge	Solids Content, %	σ	R @ 38.1 cm Hg (sec^2/gm)
Sewage	3.70 - 15.0	0.51 - 0.76	$4.07 \times 10^9 - 72.4 \times 10^9$
Water Treatment	1.00 - 4.65	0.80 - 1.32	$0.098 \times 10^9 - 10.4 \times 10^9$

It is obvious that Equation (3), (4) and (5) could have been developed in terms of L_F instead of a. The infiltration rate would then have been developed in terms of $n \frac{dL_F}{dt}$ instead of $\frac{da}{dt}$. Figures 6 and 8 illustrate the depth of wetting front penetration versus time. The hydraulic conductivity of the sand was taken to be 2.85×10^{-3} cm/sec, from which R_F may be calculated since R_F is the reciprocal of the intrinsic permeability.

The previously discussed model resulting in Equation (3), (4) and (5) is not representative of sludge spreading in agriculture in which the sludge is added at the infiltration rate, without being ponded on the surface. Figure 5 illustrates this case. The depth a may be neglected as negligible relative to L_F. Equation (1) becomes

$$n\ A \frac{dL_F}{dt} = \frac{\rho g(L_F+h_c)\ A/\mu}{\frac{cRnAL_F}{A} + L_F R_F} \qquad (6)$$

while Equation (2) becomes

$$R = R_c\left(\frac{L_F + h_c}{H_c}\right)^{\sigma} \tag{7}$$

which permits Equation (6) to be integrated with $L_F = 0$ when $t = 0$ to give

$$t = \frac{c\mu n^2 R_c}{\rho g H_c^{\sigma}}\left(\frac{h_c(L_F + h_c)^{\sigma} - h_c^{\sigma}}{\sigma + 1} + \frac{(L_F + h_c)^{\sigma+1} - h_c^{\sigma+1}}{\sigma}\right) + \frac{n\mu R_F}{\rho g}\left[L_F - h_c \ln\left(\frac{L_F + h_c}{h_c}\right)\right] \tag{8}$$

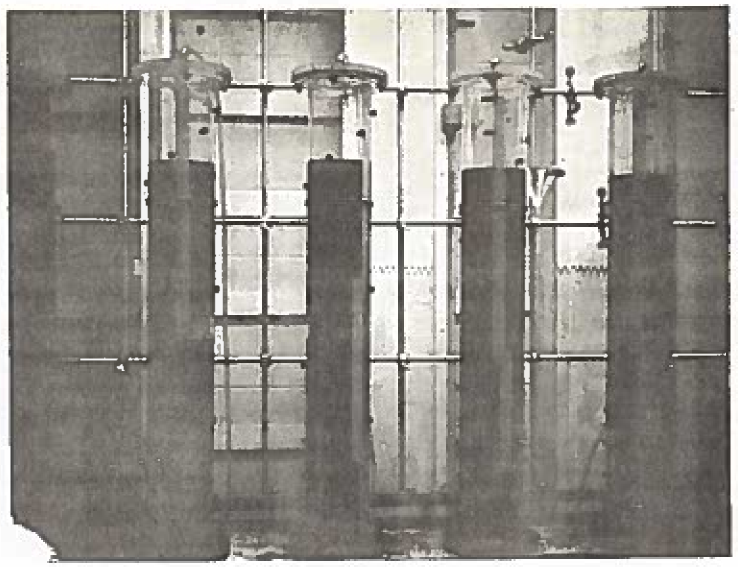
Figure 7 shows a graph of this equation for the experimental columns illustrated in Figure 1. The head is seen to have increased continuously until the wetting front reached the bottom of the column, after which the head declined with time (the equation for the declining head case is presented elsewhere[7]).

DISCUSSION OF RESULTS

Extensive experimental results have justified the applicability of Equations (1) and (2) to describe gravity dewatering of sludge. The experimental determination of R_c and σ is troublesome in that great care must be taken to obtain reproducible results.[8]

More important is the realization that the major resistance to flow occurs in the sludge cake. This suggests that much higher infiltration rates would be obtained when the sludge cake would be broken up by discing the surface. The conclusion is especially applicable to large scale land reclamation and irrigation projects now being planned in which sanitary sludges are to be applied to land.

The results presented in this paper are not applicable after the sludge liquid surface recedes into the sludge cake. Capillary forces within the sludge cake become dominant, and compress the cake as shown in the lower photograph of Figure 1 and in Figures 2 and 3. Also, solids penetration is greater when the sludge is applied to dry columns than it is when the columns are previously saturated. This result is illustrated in Figure 2.



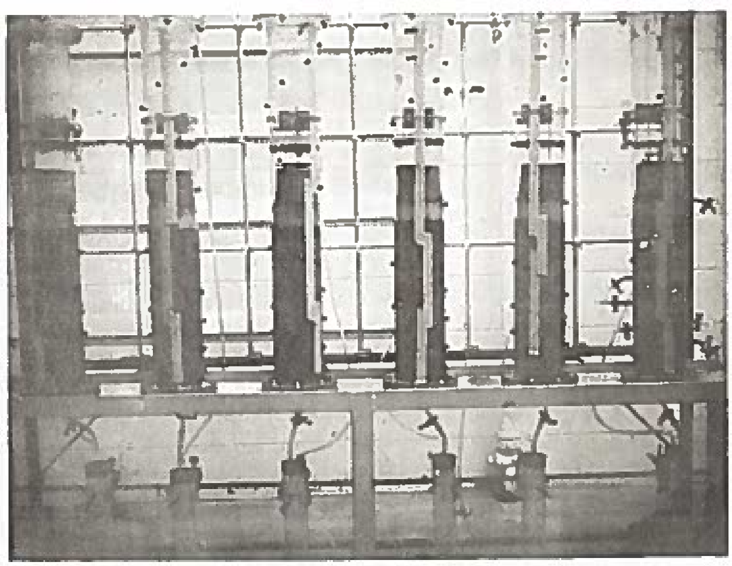


Figure 1 - Column studies of sludge dewatering on sand beds. Filtrate infiltrates into the sand. Top photograph shows some of the columns two hours after sludge was added. The wetting front had penetrated the entire column depth. Bottom photograph shows the columns six days after sludge was added. The dark ring in each column shows the sludge-air interface when the zero liquid pressure horizon receded into the sludge cake. The capillary forces have pulled the sludge cake from the column walls.

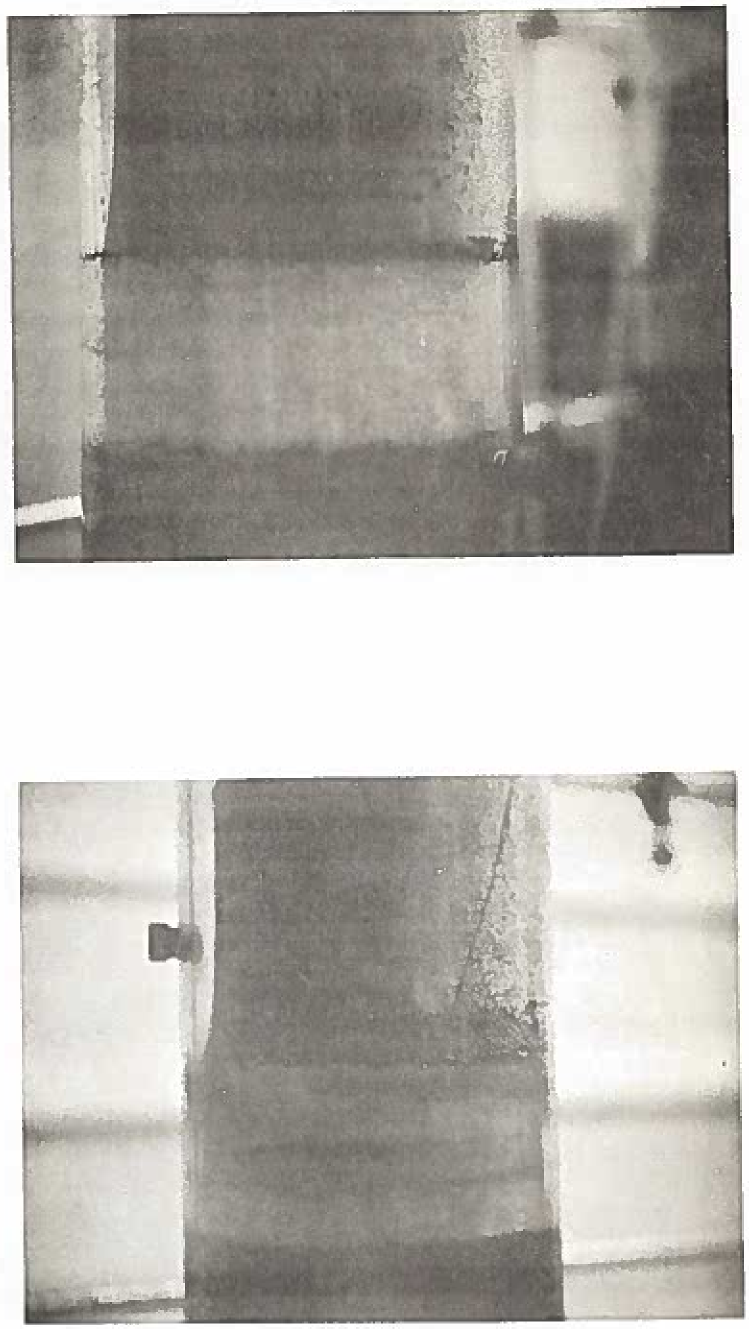


Figure 2 - Penetration of sludge particles into the sand. Top photograph shows minor penetration of the sludge into the pre-saturated sand. Bottom photograph shows more penetration of sludge particles into the initially dry sand. Dark color at the bottom of each picture is a different sand. The white sand was Ottawa sand.

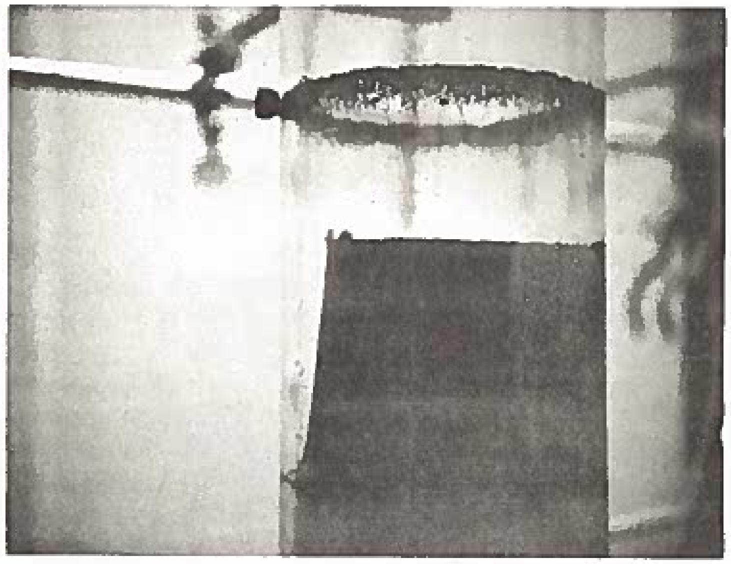


Figure 3 - Shrinkage of the sludge cake. After the free surface passes into the sludge cake capillary forces pull the cake away from the column walls and compress the cake. The ring of debris clinging to the column walls shows the approximate position of the sludge cake - air interface when the free surface receded below this level.

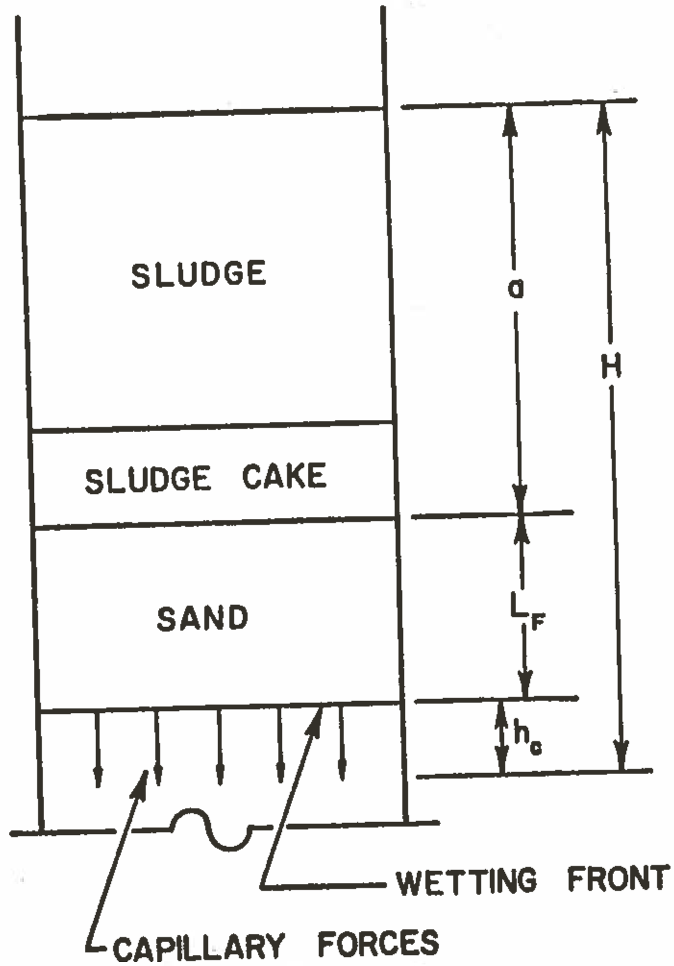


Figure 4 - Definition sketch for sludge ponding.

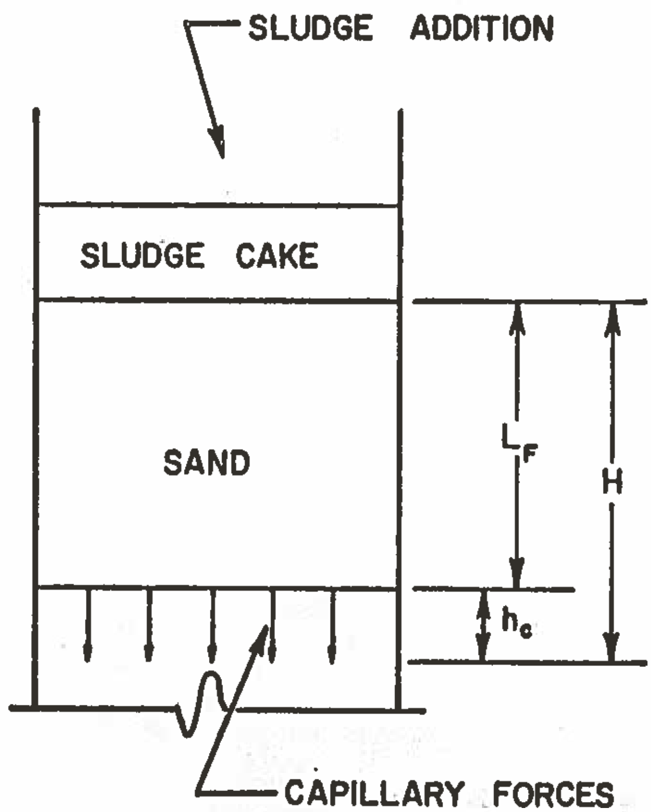


Figure 5 - Definition sketch for sludge addition at the infiltration rate. The sludge cake is built up from the sludge solids which remain behind as the filtrate infiltrates.

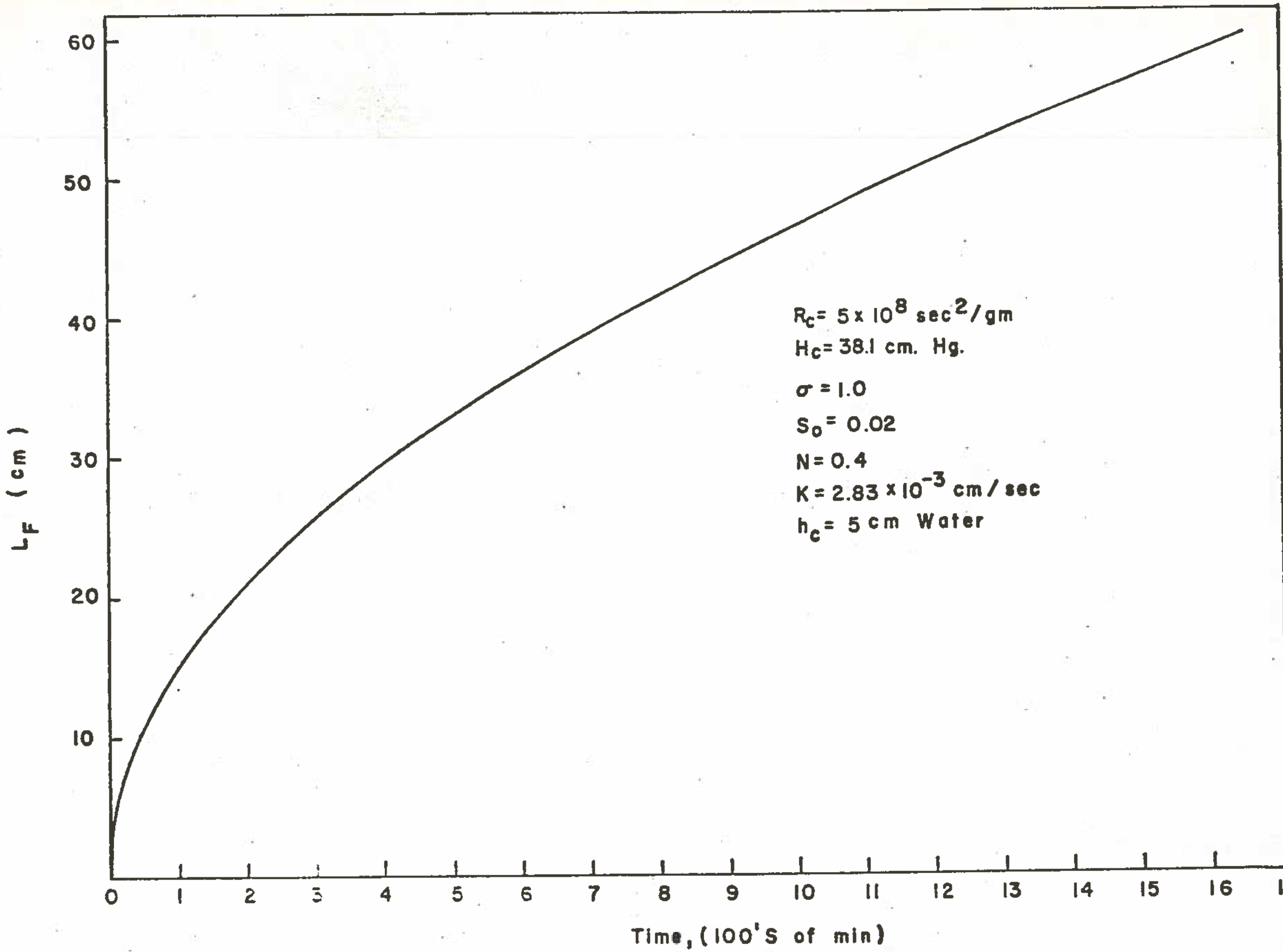


Figure 6 - Wetting frontprogression with surface ponding. The wetting front moves according to the model shown in Figure 4. Note that H_c is a reference head, not the capillary head.

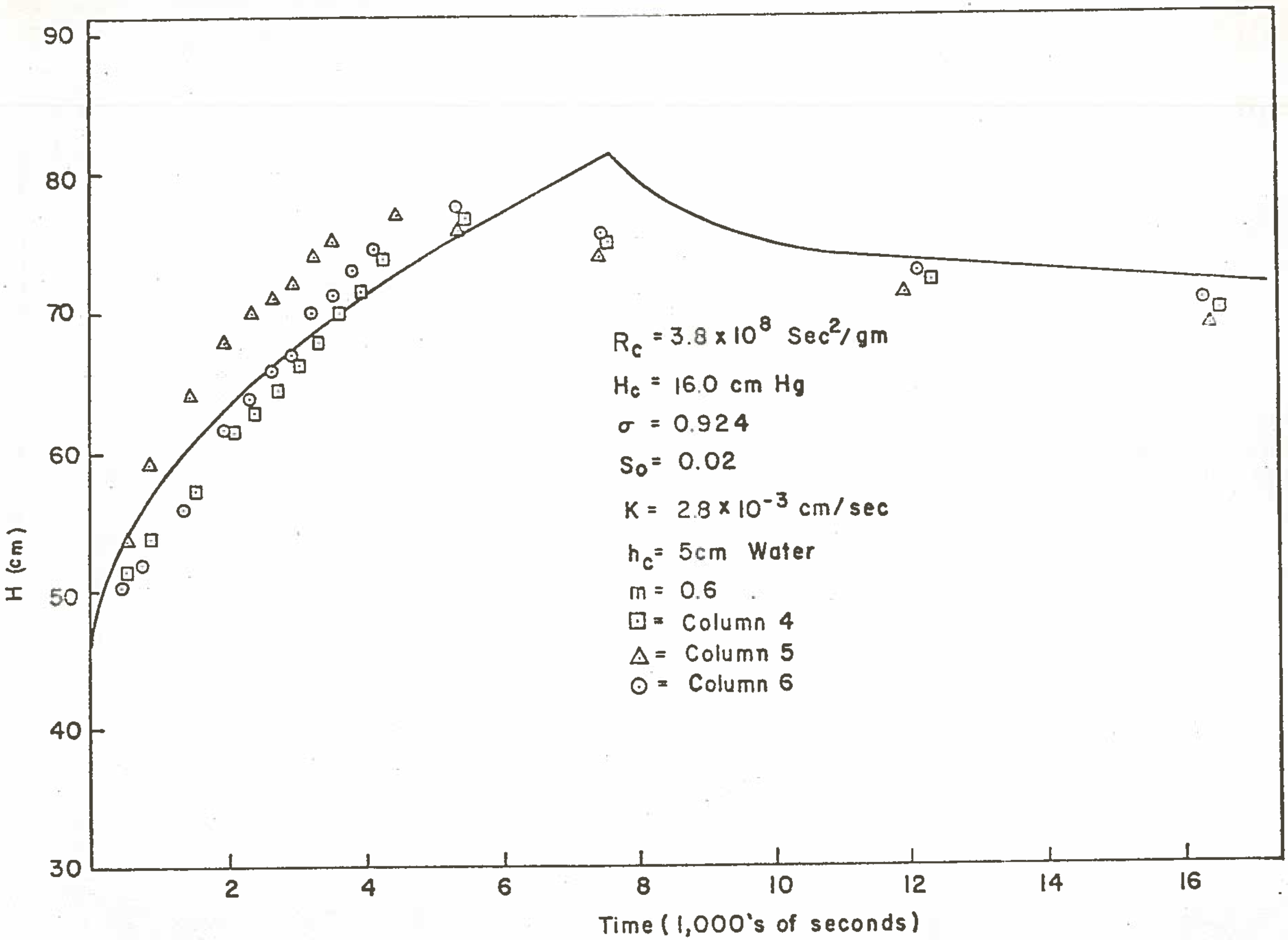


Figure 7 - Variation in head during dewatering and infiltration. The curve represents theoretical conditions for dry columns shown in Figure 1 and described by the model shown in Figure 4. The break in the curve occurs when the wetting front reaches the bottom of the column.

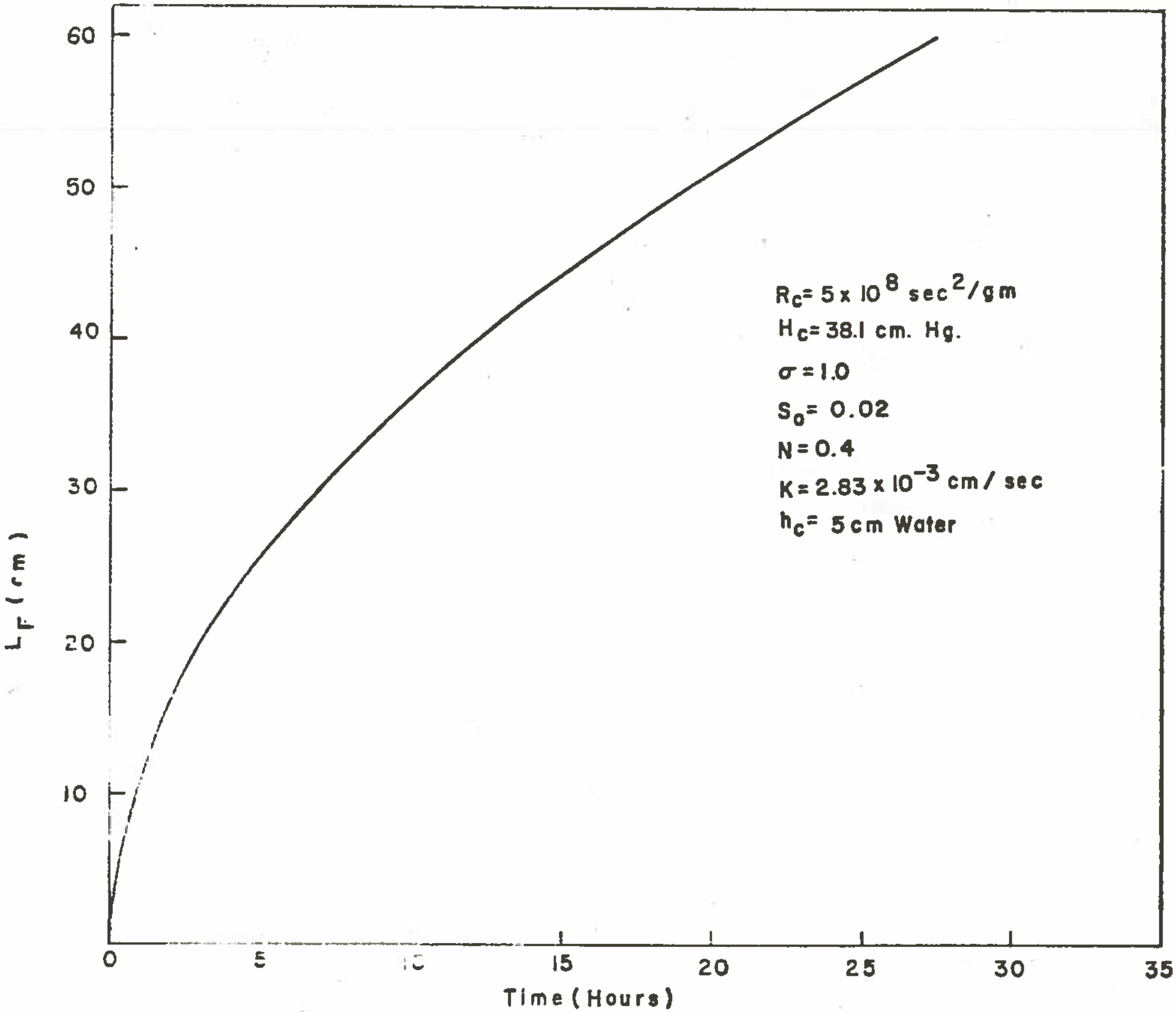


Figure 8 - Depth of infiltration as a function of time. The theoretical curve describes the penetration of the wetting front when there was sludge ponding on the surface.

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