COHESIVE AND ADHESIVE PROPERTIES OF SILICATE GROUT ON GROUTED-SAND BEHAVIOR[a]

Discussion by James Warner,[3] Fellow, ASCE

The authors are to be commended for this contribution to the understanding of the strengthening mechanism for chemically grouted sands. The discusser would caution, however, that whereas the material presented is of value for a better understanding of the mechanism involved, design of real world applications based on the indicated strengths could be risky at best. The current study involved only a single reactant system, and a uniform curing regime (72°F and a relative humidity of about 98%) that would certainly never exist on an actual project.

Work reported in 1972 by the discusser (Warner 1972) involved more than 2,500 individual specimens of eight different grout mixtures with five different reactant systems. Therein, large differences in strength behavior were noted between specimens made with the different grout mixtures and reactant systems. It was noteworthy that grouts containing the same proportion of the sodium silicate base material but different reactant systems performed very differently. Of greater importance, however, was the finding that at young ages (less than two years), the strength that a specimen would withstand under sustained loading was appreciably less than that experienced under the relative rapid loading of the unconfined compression test [138 $kN/m^2/s$ (20 psi/s) in accordance with ASTM D 1633]. The strength under sustained load was as little as 20%, and varied within a range of 20 to 80% for the different mixes. Further, it was found that the strength obtained with all of the grout systems was very dependent upon the curing regime.

A continuation of the above work, wherein the specimens were aged for nine to 11 years, was reported by Graf et al. (1982). Therein, the variation of response of the specimens to different loading rates was found to be much less than at earlier ages. Interestingly, microscopic examination of the fracture surfaces disclosed that grout had bonded to the particles only at certain prominent points and not along the entire boundary. Because such examination had not been made in the original work, it is not known if the condition existed in the original specimens or was the result of aging. Although variation of strength of the respective specimens was of the same order of magnitude as that at an age of one or two years, accumulated strain at failure was considerably less. Although the specimens remained in their waxed cardboard molds, they were otherwise allowed to dry during the ensuing years. To evaluate the effect of wetting, a number of samples were immersed in water for 21 days. To evaluate the effect of wetting, a number of samples were immersed in water for 21 days. This resulted in a significant reduction in strength and an increase in strain to failure of about 150%.

Whereas the current work does add to the body of knowledge of the strengthening properties of chemical grout, the designer of actual soil solidification work must pay close attention to the properties of the actual grout mixture to be used. Of particular importance is the sustained load that must be resisted, the age of the grouted mass when the load will be applied, and the level of confinement, if any.

The authors are encouraged to extend their work to include grout mixtures with other reactant systems, as the evidence thus far indicates a vast difference in the adhesive and cohesive properties of different silicate grouts. The variation of adhesive strength with sands of different mineralogy is interesting. The authors postulate that it may be caused by a continued reaction between the grout and the sand. To better understand the influence of sand mineralogy, it would be particularly useful to experiment with not only sands of different mineralogy but silicate grouts with different reagents, with those sands.

The microscopic finding of granular bond only at prominent points along the boundary in the 1982 report presents a question as to cause. If such condition is typical with chemically solidified sands, the shape of the individual sand grains becomes pertinent. Details of the gradation and shape of the limestone sands used by the authors were not given in the paper. It would be particularly useful if the authors could address this point in their closure and in future work.

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Closure by Alaa Ata,[4] Associate Member, ASCE, and Cumaraswamy Vipulanandan,[5] Member, ASCE

The writers are grateful to the discusser for his interest in the paper and for some of his valuable comments and observations. The main objective of this study was to investigate and quantify the contributions of cohesive and adhesive properties of a silicate grout on grouted-sand strength and modulus. Hence the variables were limited to a single silicate grout mixture, two types of sands (Table 1), and one curing condition. Based on this study it was possible to represent the grouted sand unconfined compressive strength and modulus in terms of grout properties and adhesive tensile strength between grout and sand. This is the first study of its kind to relate the cohesive and adhesive properties of grouts to the grouted-sand behavior. A control study, such as the one reported in the paper, also allows other researchers to verify the relationships [(4) and (9)] with various grouts and aggregate systems under different curing conditions. Eq. (9) was also used to predict the silicate grouted sand strength data published in the literature (Vipulanandan and Krizek 1986; Kaga and Yonekura 1991). While Vipulanandan and Krizek (1986) used a similar silicate-Ottawa system, Kaga and Yonekura (1991) used three different types of sands and six silicate grouts with unconfined strengths ranging from 0.005 to 1.0 MPa.

It should be noted that the material parameters (n, L, M, N, and P) must be determined for other grouts and sand systems for use in (4) and (9). The writers agree with the discusser's comments that the property relationships developed in this study must be verified for different grouts and sand systems. A property relationship such as (9) can help the designer to

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[a] January 1998, Vol. 124, No. 1, by Alaa Ata and Cumaraswamy Vipulanandan (Paper 12685).

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TABLE 1. Properties of Sands Used in This Study

Sand (1)	Mineralogy (2)	Shape (3)	Coefficient of uniformity, C_u (4)	D_{10} (mm) (5)	D_{50} (mm) (6)
Ottawa 20-30	Siliceous	Rounded	1.1	0.65	0.72
Lime Aggregate A	Limestone	Angular	6.7	0.15	0.70
Lime Aggregate B	Limestone	Angular	1.0	2.00	2.00

select the grout based on the required grouted sand strength. Use of different reagents in the same silicate grout can affect the grout cohesive and adhesive strengths and hence the grouted sand strength as observed by Warner (1972). The writers also agree with the discusser's comment that sustained load capacity of a grouted sand is less than the unconfined compressive strength determined from a monotonically increasing rapid loading. A paper quantifying the rate of loading effects by the writers entitled ''Factors Affecting the Mechanical and Creep Properties of Silicate Grouted-Sand'' has been accepted for publication in the *Journal of Geotechnical and Geoenvironmental Engineering*.

BEAM-COLUMN METHOD FOR TIEBACK WALLS[a]

Discussion by José L. de Justo[3]

The discusser read this paper with interest. We have a similar program and would like to compare both.

In the review of the beam-column approach the name of Rifaat (1935) should be included, as he was the first to assume that the reaction of the soil at a point of a flexible wall depends upon the displacement at that point. A finite-element method and computer solution to the general equation was provided by Matlock and Halliburton (1964, 1965), much earlier than stated by the authors.

There is an error in (6) and (7) in the paper, which should read as follows:

$$K_a = \frac{\cos^2\phi}{\cos\delta\left[1 + \sqrt{\dfrac{\text{sen}(\phi + \delta)\text{sen}\ \phi)}{\cos\delta}}\right]^2} \tag{54}$$

$$K_p = \frac{\cos^2\phi}{\cos\delta\left[1 - \sqrt{\dfrac{\text{sen}(\phi + \delta)\text{sen}\ \phi)}{\cos\delta}}\right]^2} \tag{55}$$

A similar method has been presented by Justo et al. (1994), but using normalized parabolic relationships between the coefficients of earth pressure and the wall displacement, based upon the results of active and passive pressure model tests.

According to the passive pressure tests carried out by James and Bransby (1970) the peak stress at any depth occurs when the displacement of the wall at that depth reaches a value that is a fixed proportion of the wall height, independent of the position on the wall or of the total height of the wall. So, it is correct to assume a constant value of y_p for a given wall, as stated by the author, but this value should be proportional to the wall height when comparing different walls.

Rodriguez (1985), using Rankine theory, has obtained values of y_a and y_p from back-analysis of displacements, stresses, and moments. She has found values of y_p/H_p ranging from 0.003% (grouted sandy gravel) up to 3.7% (medium organic silt). There is no reason to consider only one value of y_p and another of y_a for sand as stated by the authors.

Rodriguez has found y_p values from back-analysis ranging from 2.3 mm up to 200 mm in sand, and from 15 mm in London clay (c_u = 1,000 kPa) up to 150 mm and more in soft clay. The average value lies around 45 mm for both sand and clay.

The deflections to mobilize active and passive pressure are related by the following relationship (Justo et al. 1994):

$$y_a = 1.225\,\frac{K_0 - K_a}{K_p - K_0}\,y_p \tag{56}$$

For $\phi' = 32°$ and Rankine conditions

$$y_a = 0.07y_p$$

Justo et al. (1994) apply the finite difference method to the moment equation:

$$EI = \frac{d^2y}{dz^2} = M \tag{57}$$

instead of using the fourth-order equation as the authors do. This improves the numerical method.

The consideration of vertical equilibrium by the authors is a novel feature. As the mobilization of friction and adhesion usually occurs at the same time as the mobilization of ϕ and c, we consider, in the modification of our method, that the mobilized friction is proportional to the mobilized horizontal pressure.

The authors employ Coulomb theory for passive pressure. It is well known that Coulomb theory gives passive pressure strongly on the unsafe side. We started specifying Rankine theory that is on the safe side, but at present, we employ the results of Hettiaratchi and Reece (1974) based upon Sokolovsky's theory. We have included in our program adhesion, an inclined soil surface or a previous excavation.

Finally, the discusser believes that the coefficient 0.65 in (30) is not justified.

In any case the discusser wishes to congratulate the authors, because the paper is a good contribution to the advance of earth pressure calculations.

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INFLUENCE OF CATIONS ON COMPRESSIBILITY BEHAVIOR OF A MARINE CLAY[a]

Discussion by K. Prakash[3]

The dependency of the compressibility behavior of fine-grained soils on the physicochemical environment has been studied by many in the past. A study similar to what the authors have presented in the technical note under discussion has been reported by Sridharan et al. (1986) in a more elaborate way. While Sridharan et al. (1986) have conducted their study on homoionized bentonites, the authors have studied the behavior of homoionized marine clay.

One of the observations made by the authors is the presence of an apparent preconsolidation pressure in the remolded clays and its dependency on the cation valency and ionic radius. The discusser is in agreement with the authors in that the remolded clays (with expanding lattice type of clay minerals) exhibit ‘‘preconsolidation-like’’ effect in their $e - \log \sigma'$ curves and that it is a function of cationic valency and ionic radius. However, the method of sample preparation (i.e., initial condition of the specimen) adopted to illustrate such a point by the authors is not correct.

In the opinion of the discusser, to prove the point, it is preferable to remold the soils at or more than their liquid limit water content, if one assumes the state of the soil at its liquid limit (w_L) to correspond to a stress-free reference state. The authors' data indicate that the remolding water contents of the homoionized marine clays vary in the range $0.72w_L$–$0.86w_L$, except for sodium soil, for which $w_t = 0.95w_L$ (assuming an average specific gravity of 2.65, as the values of specific gravities of homoionized clays are not provided in the note). It is a well-known observation that a reduction in the remolding water content below that corresponding to stress-free reference state induces a ‘‘preconsolidation-like’’ effect during the consolidation of soils. Hence, the authors' illustrations and conclusions must be viewed with caution as they indicate the effect of reduced initial water content than the physicochemical effect alone.

The discusser wishes to draw to the attention of the authors an interesting observation that the marine clay tested by them contains swelling chlorite clay mineral (principal clay mineral?) and that the behavior of the soil is very similar to that of montmorillonitic soils. Even though kaolinite is one of the clay minerals present, its effect would have been masked by the swelling chlorite. This observation strengthens the view of Sridharan (1991) that all fine-grained soils can be broadly classified into two groups, namely, montmorillonitic soils and kaolinitic soils, based on their response to the varying physicochemical environment.

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ESTIMATION OF MUNICIPAL SOLID WASTE LANDFILL SETTLEMENT[a]

Discussion by
Seung Rae Lee,[5] Member, ASCE, and Hyun Il Park[6]

The authors reexamined the validity of empirical approaches in which logarithmic and power relationships are used to express the settlement rate, based on published settlement results from three landfill sites. They proposed an improved tool to predict the long-term settlement and showed that a hyperbolic function gives an improved prediction of the long-term settlement.

In refuse landfills, it was pointed out by many researchers that the long-term settlement occurs considerably due to the decomposition of refuses. For example, Sowers (1973) and Gordon et al. (1986) reported that the long-term settlement

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[a] January 1998, Vol. 124, No. 1, by Hoe I. Ling, Dov Leshchinsky, Yoshiyuki Mohri, and Toshinori Kawabata (Paper 15597).

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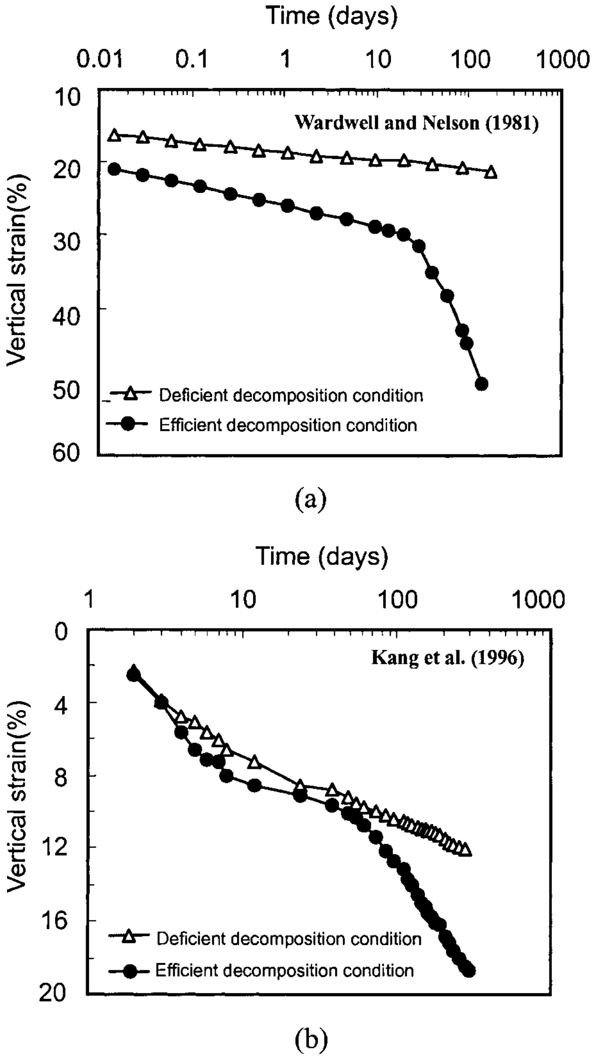


(a)

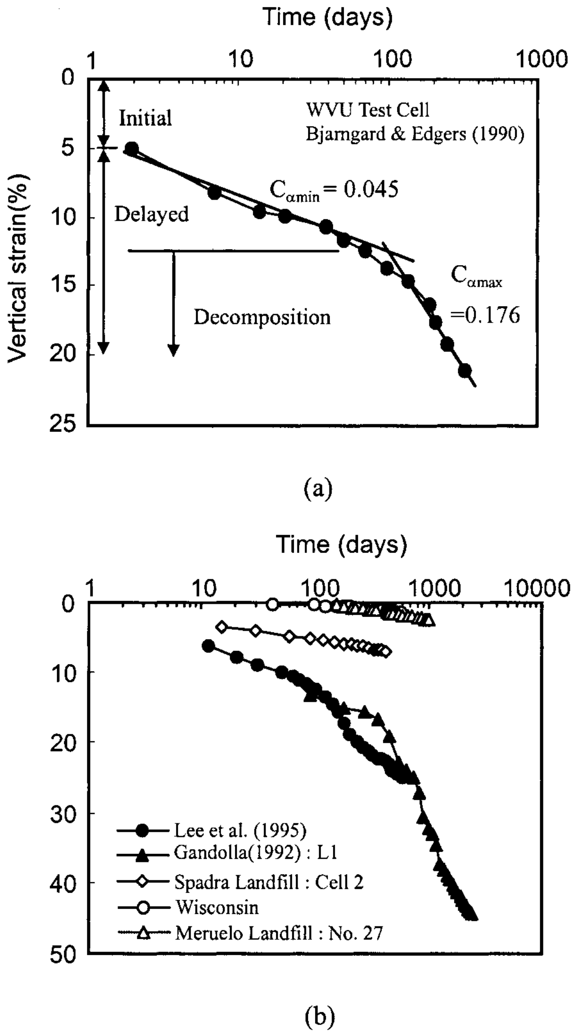
(b)

FIG. 9. Influence of Decomposition on Vertical Strain

of refuse landfills mainly occurs due to creep characteristics of the refuse skeleton and biological decomposition. Coduto and Huitric (1990) stated that settlement due to biological decomposition is probably between 18 and 24% of the refuse thickness. The decomposition effect will be discussed here in predicting the long-term settlement using the hyperbolic function.

Wardwell and Nelson (1981) presented results of a series of experiments designed to determine the effects of organic decomposition on compression strain. They used two sets of cellulose fiber, kaolin clay, and water mixtures. One of each set was seeded with sufficient nutrients to give a more efficient decomposition condition. A representative graph of their results [Fig. 9(a)] shows that the strain-logarithmic time curve for nutrient-deficient mixtures is approximately linear and has relatively flat slope on the whole range of time, while that of the nutrient-efficient sample shows much greater slope after some times.

In order to make the relationship between settlement and decomposition of refuses, Kang (1996) performed two lysimeter tests using municipal solid wastes. One was recycled with leachate to give a more efficient decomposition condition and the other was not. In the logarithmic time and strain curve [Fig. 9(b)], the two lysimeter test results are approximately similar during several days, after which the compression rate of nutrient efficient lysimeter increases considerably. In these experimental results, it is believed that the rapid increase in long-term compression rate as plotted on the logarithm of time basis is related to the decomposition effect.



(a)

(b)

FIG. 10. Settlement Behavior of Refuse Landfills

Bjarngard and Edgers (1990) also stated that the field long-term compression behavior can be separated into two phases, based on an analysis of landfill settlement data collected from 24 case histories. As represented in Fig. 10(a), in the early stage of delayed (or long-term) compression, the settlement is dominated by mechanical interactions such as long-term reorientation and delayed compression of the refuses. However, in the last stage of compression, the logarithmic compression rates are higher because of the added effects of decomposition. In Fig. 10(b), the other published results that show accelerated logarithmic compression rates are represented together with three settlement results that the authors analyzed. The authors' results only show the flat linear characteristics during the logarithmic measurement period.

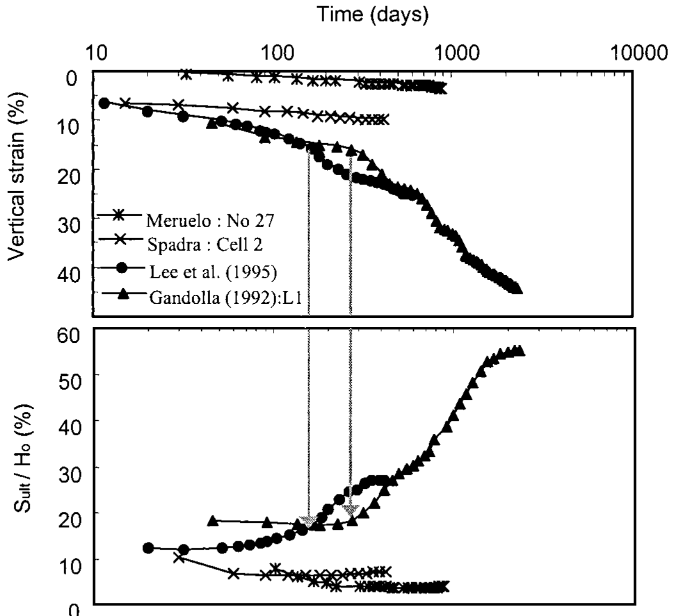
The slope values of these settlement results are compared in Table 2. The slope values of the settlement results of the authors' concern are similar to $C_{\alpha min}$ values of the previous stage before the accelerated compression. Since the landfills are very young, in which the age of refuse is estimated to be approximately less than two years, it is anticipated that the landfills may lately settle more at an accelerated logarithmic compression rate due to decomposition of degradable refuses.

In order to estimate the decomposition effect on the prediction of long-term settlement of refuse landfill, a hyperbolic function is applied to three sites (A, B, and C) that show the characteristic of the accelerated logarithmic compression rates during measurement period. The change in the estimated ultimate settlement (S_{ult}) is shown in Fig. 11. The values of S_{ult} become stable as t increases, and these characteristics are very similar to those that the authors predicted. But the values of S_{ult} begin to increase at the time when the accelerated com-

TABLE 2. Literature Values Regarding Rate of Long-Term Settlement

Site (1)	Location (2)	$C_{\alpha min(ave)}$ (3)	$C_{\alpha max(ave)}$
A	24 landfill case (Bjarngard and Edgers 1990)	0.019	0.125
B	Laboratory large scale (Gandolla et al. 1992)	0.063	0.34
C	Laboratory large scale (Lee et al. 1995)	0.063	0.149
D	Spadra landfill (the authors)	0.023	
E	Wisconsin (the authors)	0.014	
F	Meruelo landfill (the authors)	0.031	

Note: $C_\alpha = (\Delta H/H_0)/\{\log[(t + \Delta t)/t]\}$.



* Ho : height of refuse landfills

FIG. 11. Variation of Ultimate Settlement S_{ult} with Time

pression rate due to decomposition occurs. And the tendency to change of the parameter ρ_0 value is similar to that of S_{ult}. Fig. 7(c) of the authors' paper states that the ultimate settlement, S_{ult}, becomes approximately between 4 and 7% of the initial height of the refuse landfill as t increases. But if the accelerated logarithmic compression due to decomposition occurs in the three refuse landfills (sites D, E, and F) that the authors analyzed, the ultimate settlement (S_{ult}) will be considerably larger in comparison with the value that the authors predicted from the settlement data measured up to the present.

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Discussion by G. L. Sivakumar Babu[7]

The authors reviewed existing studies on landfill settlements and suggested that the rectangular hyperbola method be used for prediction of settlement of landfill wastes. The discusser agrees with the authors' observations that the methods based on observations are the viable options for estimation of settlements in the case of landfill sites compacted of a heterogeneous mix of various types of materials. Based on observations of landfill settlements, a similar approach has been proposed and validated earlier (Sivakumar Babu 1996; Sivakumar Babu and Fox 1997). The objectives of this discussion are (1) to present the method developed earlier; (2) to bring out conditions under which the methods based on observations are applicable; and (3) to show the use of these models in estimating the capacity of landfill soil covers to withstand differential settlements.

PROPOSED METHOD

The time-compression response of discrete materials such as municipal solid waste (MSW) is generally examined in void ratio-logarithm of elapsed time format as in Casagrande's method during primary consolidation and using secondary compression coefficient in secondary consolidation, and analyzed making use of soil mechanics principles. This is done to linearize time-consolidation behavior over a reasonable time scale for prediction purposes. The slope of the time-compression response (C_s) can be written and expanded in terms of differentials as

$$C_s = de/d \log t = 2.3t\, de/dt \tag{9}$$

Eq. (9) shows that C_s depends on time (t) and change in void ratio with time (de/dt) and suggests that the time-compression at any instance of time is a function of time as well as the rate of compression up to that point. It also suggests that both time and (de/dt) or settlement/time could be considered together to examine the time-settlement behavior of MSW. The term de/dt represents the change in void ratio over time dt and is the same as settlement S until time t. The relationship is in the form of a rectangular hyperbola (of the form $xy = k$) involving t and de/dt and can be linearized using natural logarithmic scales (to cover larger time scales) on both axes as

$$\ln(S/t) = -(a + b \ln t) \tag{10}$$

The term S/t represents the magnitude of total percent settlement or settlement up to time t; and a and b are constants. The constant a denotes the initial percent settlement or settlement in a reference time such as month, and b represents subsequent variation of time-deformation response. The proposed relationship is validated with reference to the data of Sowers (1973). The data are examined in terms of (10) and the resulting relationships are shown in Fig. 12. Good degrees of correlation (close to unity) obtained suggest that subsequent settlements can be predicted with reasonable accuracy based on the initial observations. The coefficients a and b of the relationship for each condition of MSW are also given in the figure.

APPLICABILITY OF MODELS

The above relationship or the authors' approach are valid for monotonic loading and could deviate under reloading upon stress release. When the effects of stress history such as those resulting from excavation exist, recompression response could be different till a state on monotonic compression path is reached. When local slips resulting from chemical reactions and readjustments close to settlement point occur, there may be accelerated settlements that are manifested in deviations to the above relationship. This is reflected in the data of landfill settlements from Coduto and Huitric (1990). Fig. 13 shows the results of their study, examined in ln(settlement/time) and ln(time) form. The results show that for monument 113, the relationship is tenable beyond 100 months, and is valid for a subsequent time duration of 12 years. The initial discrepancy is possibly due to compression under surcharge and subsequent rebound. Under further compression, the relationship is valid. For monument 314, the relationship is valid in the initial stage up to 96 months, but is not applicable beyond 96 months. This behavior is likely when deformation response is high in relation to time, and where local slips leading to large deformations could occur at a later stage.

EFFECT OF DIFFERENTIAL SETTLEMENTS ON CAPACITY OF LANDFILL SOIL COVERS

As per regulations, final cover typically is a compacted clay liner or sometimes geocomposite clay liner and must be able to withstand the differential settlements of MSW. The potential for cracking is generally examined by an index of distortion (d/L), where d represents differential settlement over the half-span (L) over a depression of the diameter ($2L$). Murphy and Gilbert (1985) proposed that one set of cracking begins at distortions between 0.05 and 0.1 (10%). The proposed approach for settlement prediction together with distortion criterion is applied to the data of Sowers (1973) to examine the capacity of landfill covers in withstanding differential settlements as follows. Two settlement observation points separated by a distance of 2.5 m in a typical landfill of 5.0 m depth can be represented by landfill conditions indicated in Sowers study that represent different degrees of aerobic and anaerobic decomposition. Settlement-time behavior in two conditions—aerobic condition (e = 5.0) and anaerobic condition (e = 3.7)—are considered for computation of differential settlements over time between two points. The settlement data for

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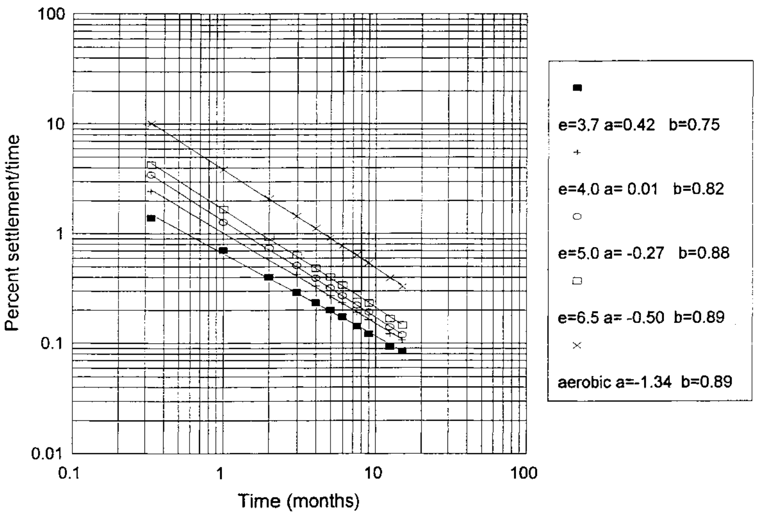

FIG. 12. Analysis of Data in ln(%*S*/*t*) and ln *t* Form

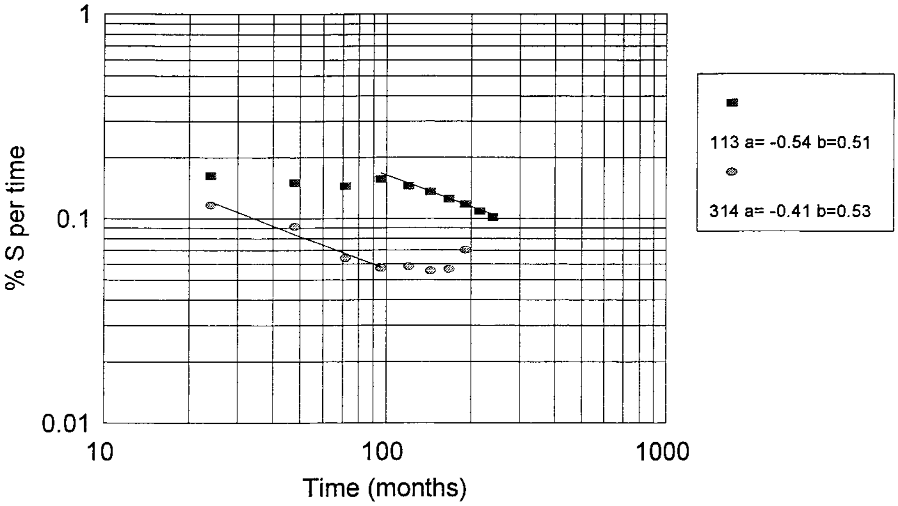

FIG. 13. Relationship between % Settlement/Time (*S*/*t*) and Time (*t*), in ln(*x*) and ln(*y*) Form

15 months for the two conditions can be represented by relationships of the form

$$\ln(S/t) = (1.345 - 0.896 \ln t)$$

and

$$\ln(S/t) = -(0.420 + 0.750 \ln t)$$

where t = total time in months; and S = percent settlement up to time t. The above relationships are used to predict percent settlements for six years and differential settlement between two points computed. From distortion criterion, a limiting distortion index of 0.1 over a distance of 2.5 m corresponds to 25 cm of differential settlement and is equal to 5% settlement of 5.0 m deep landfill. Predicted differential settlement after five years increases to 4.08%, but is still smaller than the limiting value of 5%, and therefore the cover placed on this landfill represented by two observation points can be considered stable. However, stability of cover in relation to differential settlements can be considered to be reducing with time.

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Closure by Hoe I. Ling,[8] Dov Leshchinsky,[9] Yoshiyuki Mohri,[10] and Toshinori Kawabata[11]

Lee and Park indicated that a hyperbolic function was used in their study to estimate the effects of decomposition rate on the ultimate settlement. The writers do agree that the ultimate settlement increases with an increase in the rate of decomposition within a landfill. Note that the writers have focused on the settlement prediction without taking into consideration separately each mechanism that contributes to the total settlement. For practical purposes, the rate of settlement may instead be correlated directly to the properties of the wastes, such as their

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compositions, since the rate of decomposition would be different for different types of waste.

Babu showed how a hyperbolic relationship may be used to predict settlement in a different way. The proposed (10) is similar to the power law that relates settlement rate ρ (=dS/dt) and time t [see (2)]. Although the equation provides a linear relationship in ln-ln (or log-log) plotting, (2) and (10) must be integrated to obtain the settlement at any time instant [(4)]. The discusser stated that proposed equations are valid for monotonic loading. The writers would be cautious in relating proposed mathematical functions to mechanics since the expressions were adopted for the sake of convenience.